278

National Cooperative Highway Research Program

# NCHRP Synthesis 278

# Measuring In Situ Mechanical Properties of Pavement Subgrade Soils

A Synthesis of Highway Practice



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# Measuring In Situ Mechanical Properties of Pavement Subgrade Soils

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## PREFACE

A vast storehouse of information exists on nearly every subject of concern to highway administrators and engineers. Much of this information has resulted from both research and the successful application of solutions to the problems faced by practitioners in their daily work. Because previously there has been no systematic means for compiling such useful information and making it available to the entire community, the American Association of State Highway and Transportation Officials has, through the mechanism of the National Cooperative Highway Research Program, authorized the Transportation Research Board to undertake a continuing project to search out and synthesize useful knowledge from all available sources and to prepare documented reports on current practices in the subject areas of concern.

This synthesis series reports on various practices, making specific recommendations where appropriate but without the detailed directions usually found in handbooks or design manuals. Nonetheless, these documents can serve similar purposes, for each is a compendium of the best knowledge available on those measures found to be the most successful in resolving specific problems. The extent to which these reports are useful will be tempered by the user's knowledge and experience in the particular problem area.

## FOREWORD

*By Staff*
*Transportation Research Board*

This synthesis report will be of interest to pavement and geotechnical design and research engineers, geologists and engineering geologists, and similar laboratory personnel. It describes the current practice for measuring the in situ mechanical properties of pavement subgrade soils. The tests conducted to estimate the mechanical properties of soil strength and stiffness are the primary topics, and these are discussed in the context of design procedures, factors affecting mechanical properties, and the variability of measurements. Information for the synthesis was collected by surveying U.S., Canadian, and selected European transportation agencies and by conducting a literature search.

Administrators, engineers, and researchers are continually faced with highway problems on which much information exists, either in the form of reports or in terms of undocumented experience and practice. Unfortunately, this information often is scattered and unevaluated and, as a consequence, in seeking solutions, full information on what has been learned about a problem frequently is not assembled. Costly research findings may go unused, valuable experience may be overlooked, and full consideration may not be given to available practices for solving or alleviating the problem. In an effort to correct this situation, a continuing NCHRP project, carried out by the Transportation Research Board as the research agency, has the objective of reporting on common highway problems and synthesizing available information. The synthesis reports from this endeavor constitute an NCHRP publication series in which various forms of relevant information are assembled into single, concise documents pertaining to specific highway problems or sets of closely related problems.

This report of the Transportation Research Board provides information on existing and emerging technologies for static and dynamic, and destructive and nondestructive testing for measuring in situ mechanical properties of pavement subgrade soils. Correlations between in situ and laboratory tests are presented. The effects of existing layers on the measurement of subgrade properties, and the subjects of soil spatial and seasonal variability are also discussed. Most importantly, the use of measured soil properties in pavement design and evaluation are explained. New applications or improvements to existing in situ test methods to support the use of mechanistic/stochastic-based pavement design procedures are also explained.

To develop this synthesis in a comprehensive manner and to ensure inclusion of significant knowledge, the Board analyzed available information assembled from numerous sources, including a large number of state highway and transportation departments. A topic panel of experts in the subject area was established to guide the research in organizing and evaluating the collected data, and to review the final synthesis report.

This synthesis is an immediately useful document that records the practices that were acceptable within the limitations of the knowledge available at the time of its preparation. As the processes of advancement continue, new knowledge can be expected to be added to that now at hand.

## CONTENTS

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# MEASURING IN SITU MECHANICAL PROPERTIES OF PAVEMENT SUBGRADE SOILS

## SUMMARY

This synthesis presents a review of the practice for characterizing the mechanical properties of pavement subgrades in situ. The practices of highway agencies in the United States, Canada, Austria, Belgium, Finland, Iceland, Norway, and Switzerland with respect to in situ subgrade testing are summarized and discussed. The primary topics are the tests conducted to estimate soil mechanical properties of strength and stiffness and these are discussed in the context of design procedures, factors affecting mechanical properties, and the variability of measurements.

A questionnaire was prepared and distributed to the participating agencies in order to provide an overall picture of practice. This questionnaire was divided into four major parts: (1) Flexible Pavement Design, (2) Rigid Pavement Design, (3) Laboratory Testing of Soils, and (4) In Situ Testing of Soils. The survey was devised to document the type of pavement design practices in use, parameters within the design procedures used to describe the subgrade, the type of laboratory testing used, and, most importantly, the type and frequency of in situ testing performed on soils. The results of this survey were incorporated into general discussions of subgrade testing and interpretation that were based on a review of the literature.

The survey showed that most state agencies in the United States use the 1993 American Association of State Highway and Transportation Officials (AASHTO) pavement design guide for both flexible and rigid pavements. However, it is significant that a number of agencies employ mechanistic-empirical analysis as a secondary method of flexible pavement structural design. For both the 1993 AASHTO and mechanistic-empirical flexible pavement design procedures the parameter used to describe the subgrade is resilient modulus. The other primary flexible pavement design procedures involve test results that are indicative of the shear strength, namely the *R*-value or California Bearing Ratio (CBR). Subgrades for rigid pavements are almost universally characterized by the modulus of subgrade reaction (*k*-value), sometimes known as a liquid foundation. The AASHTO rigid pavement design procedure allows for the estimation of the *k*-value by means of a nomograph, which uses subgrade modulus, base modulus, and base thickness. The strength or stiffness of the subgrade is one of the three primary inputs to the pavement design process; the other two being traffic and the environment. Adequately and realistically characterizing this material is critical to the success of the pavement system.

It is clear that the state of stress, moisture content, state of moisture (frozen or thawed), and density all have profound effects on the properties of soils. The state of stress defines the magnitude of loading and the degree of confinement for the soil. There are numerous models that describe the relationship between the state of stress and stiffness of the soil, but it is generally acknowledged that plastic soils soften with increasing stress and that granular soils stiffen. Moisture content may cause the material to soften or weaken with increasing amounts of water, depending on the particular soil type. The state of moisture, however,

can have a counter effect because frozen soils will support more load than those containing thawed water. Increasing density contributes to soil strength or stiffness, but its effect must be considered along with moisture content. Thus, in testing it is important to have a clear understanding of the in situ conditions.

Deflection testing is the most popular means of evaluating pavement structures for the agencies surveyed. Although the majority of the agencies employ falling weight deflectometers (FWD) for this purpose, high-speed deflectometers are now being used. Most of the states, provinces, and European countries reviewed use deflection parameters such as the maximum deflection or subgrade modulus calculation. Some states use an equation to relate FWD maximum deflection to the Benkelman beam deflection. A surprising number of agencies use backcalculation techniques to estimate layer moduli in the interpretation of deflection measurements. Other types of in situ tests in common use include small-load techniques and intrusive approaches. The small-load methods provide measures of soil stiffness and include the Finnish Loadman, the spectral analysis of surface waves (SASW), and the Soil Stiffness Gauge. Of these, only the SASW is currently used by any of the states on a routine basis. Intrusive techniques include the dynamic cone penetrometer (DCP), the borehole pressuremeter, the standard penetration test (SPT), cone penetration testing, miniature cone penetrometer, dilatometer testing, vane shear testing, and field CBR testing. The most popular of these methods are the DCP and the SPT. Both of these tests are relatively simple and provide a good indication of the soil shear strength.

Subgrade spatial and seasonal variabilities are important considerations in pavement design. Sampling frequencies for the purpose of design are based on the type of soil, traffic volume, and the difficulty of testing and analysis. Statistical techniques exist to assist engineers in determining the appropriate frequency of testing, and these should be employed. Furthermore, it would be wise for agencies to investigate subgrade spatial variability within their jurisdictions. Seasonal variability of subgrade properties is a regional issue that may be addressed by a combination of reviewing historical records and establishing a testing program to provide the needed data. Data for determining the patterns of seasonal variability may be obtained from facilities such as the Minnesota Road Research Project and studies such as the Long-Term Pavement Performance program.

In situ characterization of subgrade soils is critical to the realistic design of pavement structures. A number of means exist to provide such information and these should be exploited to provide timely and accurate input to the process of pavement design and the monitoring of pavement performance.

CHAPTER ONE

# INTRODUCTION

## BACKGROUND

The Finnish philosophy in pavement design presumes that any treatment to the subgrade should last from 60 to 100 years, the base and subbase should last from 30 to 50 years, and the surface should have a life of from 15 to 20 years. This "bottom-up" approach to pavement design reflects the relative cost of rehabilitation associated with each of these layers and the importance of engineering in characterizing the soil and selecting materials for the lower pavement layers. Consequently, a great deal of attention is given in Finland to laboratory and field testing of subgrade soils. Techniques for measuring the soil characteristics and conditions, predictions of soil behavior, and methods for subgrade improvements are given top priority in research, design, and construction *(1)*.

Soil strength or stiffness is one of the primary inputs to any pavement design procedure, in addition to traffic and the environment. In the past, this input for flexible pavements was routinely an *R*-value or California Bearing Ratio (CBR) value, taken from a laboratory test of material in a saturated condition. For concrete pavements, the practice was to use a modulus of subgrade reaction (*k*-value) based primarily on either past experience or from correlation to other test methods. The *k*-value was generally adjusted to reflect the quality and thickness of base materials overlying the subgrade. In general, the expense associated with material sampling, sample preparation, and laboratory testing precluded extensive amounts of soils characterization for the purpose of pavement design. Although it may have been widely acknowledged that the geological characteristics of pavement sites changed more frequently than the soil sampling frequency, it was believed that laboratory testing in a weakened state would ensure a conservative value for design purposes.

Advances in technology and the recent application of geophysical techniques to subgrade characterization have given pavement engineers greater ability to measure, analyze, and account for subsurface moisture and temperature conditions than at any previous time; however, there does not seem to be a widespread application or acceptance for much of the new technology. Instrumentation is available for measuring subsurface environmental conditions such as temperature, moisture content, and state of moisture. Techniques such as ground-penetrating radar and resistivity tomography provide indications of moisture conditions without the intrusiveness of more traditional methods, such as time-domain reflectometry or potentiometers. These methods are very useful in understanding the temporal changes in subsurface moisture conditions, which dictate the strength or stiffness of subgrade materials.

The application of in situ strength or stiffness measurements is somewhat more widely accepted than measurements of in situ moisture content and state. Measurements of deflections date back to the plate bearing test (*2*) and the Benkleman beam procedure (*3*). Currently, falling weight deflectometers (FWD) are commonly used to characterize the stiffness of pavements, thanks largely to the Strategic Highway Research Program (SHRP) in the late 1980s (*4*). Other in situ strength or stiffness measurement techniques for pavements, such as the dynamic cone penetrometer (DCP) or spectral analysis of surface waves (SASW), are not as well known, but they are gaining in popularity. In the future, it is expected that rolling wheel deflectometers (RWD) will be used in place of FWDs; their chief advantages being that the measurement is done at higher speed and is continuous.

## DEFINITIONS

There is room for a great deal of ambiguity regarding the terminology associated with the materials characterization used for pavement design. To provide some consistency, at least within this document, the following terms are defined:

- *Dynamic Modulus*—The maximum axial stress applied to a material in sinusoidal loading, divided by the maximum axial strain occurring during that loading.
- *Elastic Modulus*—The applied axial stress divided by the resulting axial strain, within the linear range of stress-strain behavior of a material.
- *Modulus of Subgrade Reaction*—The applied stress imposed by a loaded plate of a specified dimension acting on a soil mass divided by the displacement of the plate within the linear portion of the stress-deformation curve.
- *Resilient Modulus*—The stress generated by an impulse load divided by the resulting recoverable strain after loading.
- *Shear Strength*—A combination of a material's interparticle friction and its cohesion in resisting deformation from an applied stress. This is the largest stress that the material can sustain.
- *Stiffness*—A qualitative term meaning a general resistance to deformation. It is often used interchangeably

with elastic modulus, modulus of subgrade reaction, and resilient modulus. It largely determines the strains and displacements of the subgrade as it is loaded and unloaded.

## OBJECTIVES

The objectives of this synthesis are to identify the following:

- Existing and emerging methods for measuring in situ subgrade properties to include static and dynamic as well as destructive and nondestructive techniques;
- Relationships between mechanical properties measured in situ and those measured in the laboratory;
- Effects of existing layers on the measurement of subgrade properties;
- The use of measured soil properties in the design or evaluation of pavements; and
- New applications or improvements to existing in situ test methods to support the use of mechanistic/stochastic-based pavement design procedures.

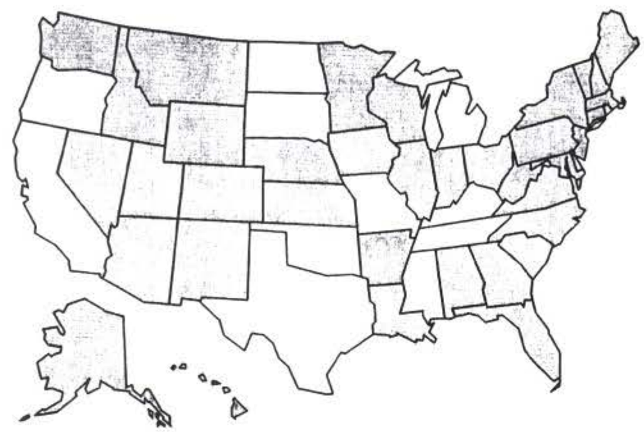
## SCOPE

This synthesis was completed by first developing a work plan, a synthesis outline, and an agency questionnaire. Literature relevant to the synthesis was acquired, the results of the questionnaire were compiled, and the synthesis was prepared.

## ORGANIZATION OF SURVEY

A questionnaire regarding the practice of subgrade characterization for pavement design was mailed to state and provincial agencies within the United States and Canada. Thirty-five states and six Canadian provinces returned the questionnaire, and responses from six European countries (Austria, Belgium, Finland, Iceland, Norway, and Switzerland) were obtained as opportunities presented themselves. Maps showing the location of responding organizations (shaded) are presented in Figure 1. There is good representation of all the geographic areas in the United States and Canada. Responses came from European countries located in the north and central part of the continent.

*a)* States responding to questionnaire.



*b)* Canadian provinces responding to questionnaire.

*c)* European countries responding to questionnaire.

FIGURE 1 Respondents to questionnaire.

The questionnaire was used to summarize information on an agency-by-agency basis, and the responses provide a useful picture of overall practice. The questionnaire was divided into four major parts: (1) Flexible Pavement Design, (2) Rigid Pavement Design, (3) Laboratory Testing of Soils, and (4) In Situ Testing of Soils. At the end of the survey, an opportunity was given to the respondents to add comments. The survey was intended to document the type of pavement design procedures in use, the parameters within the design procedures used to describe the subgrade, the type of laboratory tests used to characterize the subgrade, and, most importantly, the type and frequency of in situ testing performed on soils. An example of the questionnaire is presented in Appendix A.

## SURVEY RESULTS

Responses to general questions in the survey are presented in Appendix B (Tables B-1 through B-5). Responses to more specific questions dealing with detailed information such as correlations and test methods will be covered in the appropriate chapters.

Table B-1 presents the results of the survey relative to the design practices for flexible pavements. Respondents were asked what their primary design method was, what type of input was used for soil strength or stiffness, and what type of secondary design method, if any, was used. The same questions were asked with respect to rigid pavements, and the results are given in Table B-2. Four U.S., two Canadian, and two European respondents indicated little or no rigid pavements within their jurisdictions. Table B-3 shows the subgrade laboratory test methods used by the agencies for flexible and rigid pavement designs, as well as the frequency of sampling and testing associated with these methods. It should be noted that most agencies used the same test methods for both pavement types. Methods for deflection testing of pavements are given in Table B-4, along with whether the deflection test results are used in pavement design and the frequency of testing. Table B-5 shows the other types of in situ tests used by the agencies and the reasons for using them.

## ORGANIZATION OF SYNTHESIS

Chapter 2 of this report discusses soil properties relative to a number of commonly used pavement design procedures. A general discussion of factors influencing the behavior of soils is presented in chapter 3. In chapter 4, methods for in situ estimation of subgrade mechanical properties are discussed in detail. Seasonal and spatial variabilities of subgrade properties are the topics of chapter 5. Finally, the conclusions and recommendations from this work are given in chapter 6.

CHAPTER TWO

# USE OF SUBGRADE MECHANICAL PROPERTIES IN PAVEMENT DESIGN

## INTRODUCTION

The objective of pavement design is to provide a structural and economical combination of materials to carry traffic in a given climate over the existing soil conditions for a specified time interval. Traffic magnitude and the variability of climate and soils define the structural requirements of rigid and flexible pavements, and a failure to adequately characterize any of them will result in substandard pavement performance or in an inefficient expenditure of funds. Traffic estimates are normally based on the volume and load characteristics of present traffic combined with projections of traffic growth and changes in land use. This information is usually provided by a traffic or planning division within an agency. Considerations of climatic conditions are most often incorporated in design by accounting for their effects on material properties, either on a seasonal basis or in the so-called "worst condition." The subgrade may be characterized by laboratory or field testing or a combination of both. It is essential that the methods selected for the characterization accurately reflect the subgrade's role in the pavement structure, and that the frequency of sampling adequately depicts the spatial variability in the soil mass. As noted by Yoder and Witczak (*2*), "All pavements derive their ultimate support from the underlying subgrade; therefore, a knowledge of basic soil mechanics is essential."

In this chapter, the role of subgrade properties with respect to pavement design will be examined in detail. The assumptions and practices inherent in obtaining a design value of subgrade strength or stiffness will discussed.

## FLEXIBLE PAVEMENTS

This discussion of flexible pavement design procedures encompasses both empirical procedures such as the American Association of State Highway and Transportation Officials (AASHTO) and granular equivalency methods, and more mechanistic-based procedures such as that recommended by the Asphalt Institute, and state design procedures such as that used in Illinois.

As shown in Figure 2, most of the states responding to the questionnaire have adopted the 1993 AASHTO guide (*5*) as their primary flexible pavement design. The state of Pennsylvania recently changed to the newer AASHTO design method. The second most commonly used method is the 1986 AASHTO procedure (*6*), followed by granular equivalency methods used by Idaho and Minnesota. Illinois uses a mechanistic design procedure and Alaska a method based on the amount of fine material in the subgrade. Many states indicating primary use of the older AASHTO procedure use the 1993 AASHTO guide for a secondary method. It is interesting to note that seven states use a mechanistic-empirical approach for a secondary design method and two use the Asphalt Institute procedure.

Of the six Canadian provinces, two use the 1993 AASHTO procedure, three use a granular equivalency

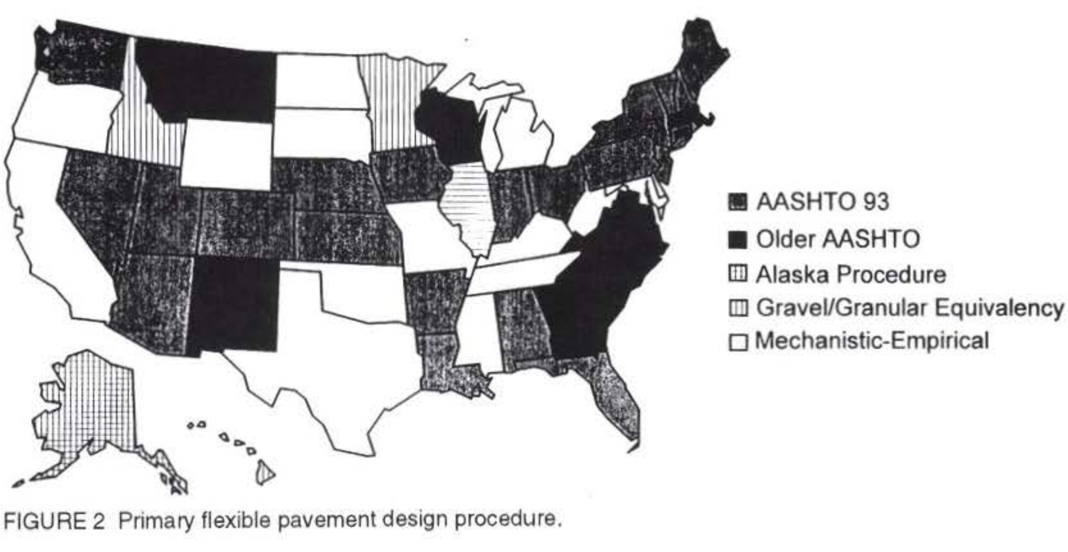



FIGURE 2 Primary flexible pavement design procedure.

method, and one did not specify a design approach. In addition, Alberta indicated the use of stage construction in their approach to the design of flexible pavements. Norway and Iceland specified the use of a Norwegian design standard, whereas Austria relies primarily on a design catalog. Finland uses a mechanistic-empirical design procedure, Belgium has adopted analytical and empirical methods, and Switzerland uses an adaptation of the 1993 AASHTO guide.

### AASHTO

The original AASHTO guide for the design of pavement structures was prepared in 1961 and revised in 1972 (*7*). In this approach, subgrade stiffness or strength was accounted for by the assignment of a soil support value (SSV), and the effects of differing climates were considered by means of a regional factor (*R*). The SSV has a scale ranging from 1 to 10, with a value of 3 representing the natural soil at the Road Test and a value of 10 representing the performance of sections containing thick granular bases of crushed rock (*2*). Because of the arbitrary scale of the SSV, correlations were developed to results of standard laboratory tests (*8–10*). The regional factor was devised so that it would have an inverse effect on the number of allowable loads (*2*). Thus, areas subject to milder temperatures and drier conditions would have a smaller "*R*" than areas subject to wet conditions and spring thaw.

In 1986, the AASHTO guide was substantially revised to include replacement of the SSV with roadbed soil modulus ($M_R$) and accounting for seasonal changes in soil strength by obtaining a weighted value of $M_R$ according to the damage done to the roadway during periods when soils soften (*7,8*). The roadbed soil modulus may be estimated directly from laboratory resilient modulus testing, indirectly through correlation with another standard laboratory test (e.g., CBR and *R*-value) (*11,12*), or by backcalculation of the modulus from deflection testing results. The AASHTO guide emphasizes the need to use an average value of $M_R$ for each season, representative of the stiffness of the compacted soil in the design process in order to avoid being too conservative. This is because conservatism is inherent in the incorporation of reliability in the design process.

Seasonal changes in the roadbed modulus are considered using a table such as that shown in Figure 3. Here the modulus of the material is entered for each season considered in the design process. A value of relative damage for each modulus value is found by using the scale to the right of the table. As the modulus increases on this scale, the relative damage factor decreases. The relative damage factors for all seasons are summed and the average damage factor is found, after which the corresponding modulus is read from the scale at the right. This value is the effective roadbed soil modulus used in the design nomograph to determine the structural number. The $M_R$ has a profound effect on the design structural number; therefore, it is important to use an appropriate value.

For pavement rehabilitation, the AASHTO guide suggests an evaluation of the pavement structure using nondestructive deflection testing. The subgrade modulus may be estimated directly using an equation in which the deflection at a point located some distance from the center of the load is directly a function of the subgrade modulus (see chapter 4). Although this gives the proper modulus of the soil, it does not correlate well to the resilient modulus measured at the AASHO Road Test (*13*). Darter et al. (*13*), have suggested that a correction factor is needed to match the modulus calculated from deflection to the modulus used in the development of the AASHTO design procedure. The concern is that the uncorrected modulus value would be too great for a valid pavement design.

### Gravel or Granular Equivalency

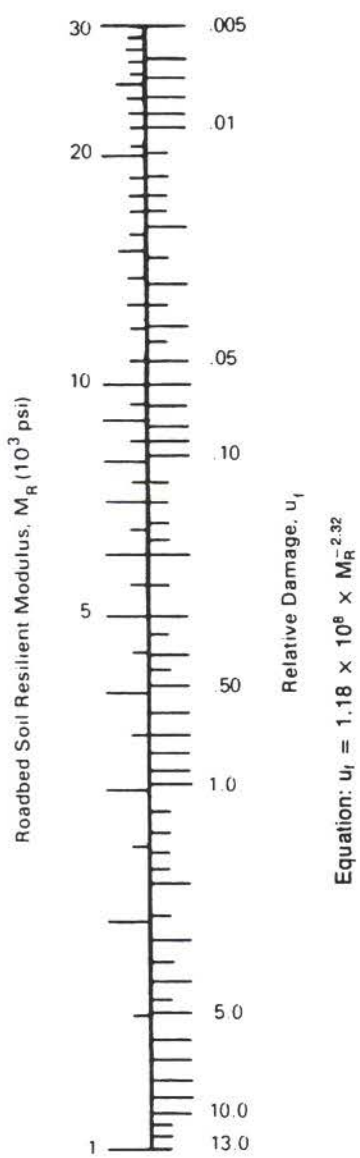
The gravel or granular equivalency (GE) approach is based upon the results of stabilometer *R*-value testing. This methodology has been used in states such as California (*14*), Washington (*15*), and Minnesota (*16*). The traffic, in terms of Traffic Index (TI) or 18,000-lb equivalent single axle loads (ESAL), and the subgrade *R*-value determine the required GE of the section. An example of a design chart used by the Minnesota Department of Transportation (DOT) is shown in Figure 4. Materials are assigned a value of GE to the thickness of granular material considered to equal the thickness of the material under consideration. For instance, in Minnesota (*16*) a high-quality asphalt concrete mixture is considered to have a GE of 2.25 (Table 1).

The greatest disadvantage to a GE flexible pavement design procedure is that it relies on a laboratory determined or an assumed *R*-value for the soil. Although laboratory testing is preferable to the use of engineering judgment for an assumed value, it still limits the user to relatively few test results on laboratory compacted specimens that may not be representative of field conditions. The effects of variability and soil structure will be discussed in more detail later.

### Corps of Engineers

The Corps of Engineers' method of flexible pavement design (*17*) is based on traffic levels, frost effects, and the

| Month | Roadbed Soil Modulus, $M_R$ (psi) | Relative Damage, $u_f$ |
|---|---|---|
| Jan. | 20,000 | 0.01 |
| Feb. | 20,000 | 0.01 |
| Mar. | 2,500 | 1.51 |
| Apr. | 4,000 | 0.51 |
| May | 4,000 | 0.51 |
| June | 7,000 | 0.13 |
| July | 7,000 | 0.13 |
| Aug. | 7,000 | 0.13 |
| Sept. | 7,000 | 0.13 |
| Oct. | 7,000 | 0.13 |
| Nov. | 4,000 | 0.51 |
| Dec. | 20,000 | 0.01 |
| Summation: $\Sigma u_f$ = | | 3.72 |




Average: $\bar{u}_f = \frac{\Sigma u_f}{n} = \frac{3.72}{12} = 0.31$

Effective Roadbed Soil Resilient Modulus, $M_R$ (psi) = 5,000 (corresponds to $\bar{u}_f$)

FIGURE 3 AASHTO method for determining effective roadbed soil modulus *(5)*.

CBR of subgrade and base materials. The traffic is considered in terms of a Design Index (DI), which is related to a range of daily ESAL expected in the design lane. The subgrade CBR and the DI are used to determine a total pavement thickness. The total pavement thickness is increased according to subgrade frost susceptibility and traffic level to provide full or partial frost protection in areas subject to winter freezing.

Another important consideration in the Corps of Engineers' procedure is the level of subgrade compaction achieved during construction. The required depth of compaction, shown in Table 2, is defined in terms of soil type (cohesive or cohesionless) and traffic level. The compacted effort is specified in terms of the modified Proctor test (ASTM D 1557 or AASHTO T 180).

The CBR test has the same disadvantages as the *R*-value test in that there are relatively few samples taken for laboratory evaluation and these are remolded before testing. The design CBR value is based on the soil strength in a saturated condition to account for moisture equilibrium over the life of the pavement; therefore, seasonal effects are considered only in the worst case.

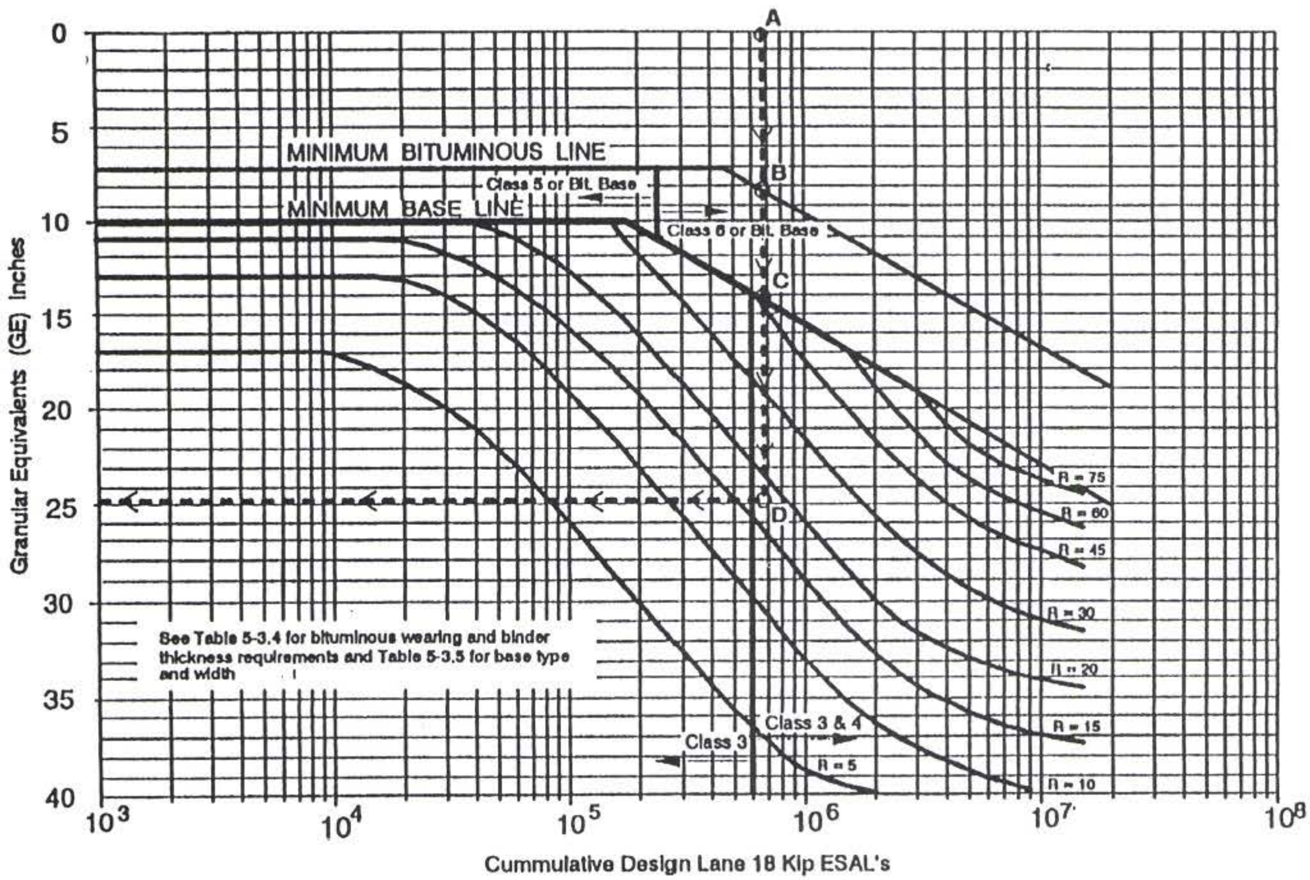



FIGURE 4 Minnesota GE design chart *(16)*.

TABLE 1

MINNESOTA GE FACTORS *(16)*

| Material | GE Factor |
|---|---|
| Asphalt concrete | 2.00–2.25 |
| Bituminous-treated base | 1.25–1.50 |
| Aggregate base | 0.75–1.00 |
| Granular borrow | 0.50 |

TABLE 2

DEPTH OF COMPACTION REQUIREMENTS FOR CORPS OF ENGINEERS *(17)*

| | | Design Index of Pavement | | | | | |
|---|---|---|---|---|---|---|---|
| Material Type | T-180 Compaction | DI-I | DI-2 | DI-3 | DI-4 | DI-5 | DI-6 |
| Cohesive | 100 | 127 | 152 | 178 | 203 | 229 | 254 |
| | 95 | 279 | 305 | 330 | 356 | 381 | 406 |
| Cohesionless | 100 | 203 | 229 | 279 | 305 | 356 | 381 |
| | 95 | 381 | 432 | 508 | 559 | 635 | 711 |

**Asphalt Institute**

The Asphalt Institute *(11,18)* presents a mechanistic-based design procedure for streets and highways. The initial inputs for the method include the ESAL for the design period, the design resilient modulus of the subgrade, the mean monthly air temperature, and the combination of layered materials to be used. Selection of the design resilient modulus is crucial, because it defines a portion of the conservatism in the cross-section determination. For up to 10,000 ESAL, the design resilient modulus should represent the value above which 60 percent of the soil test

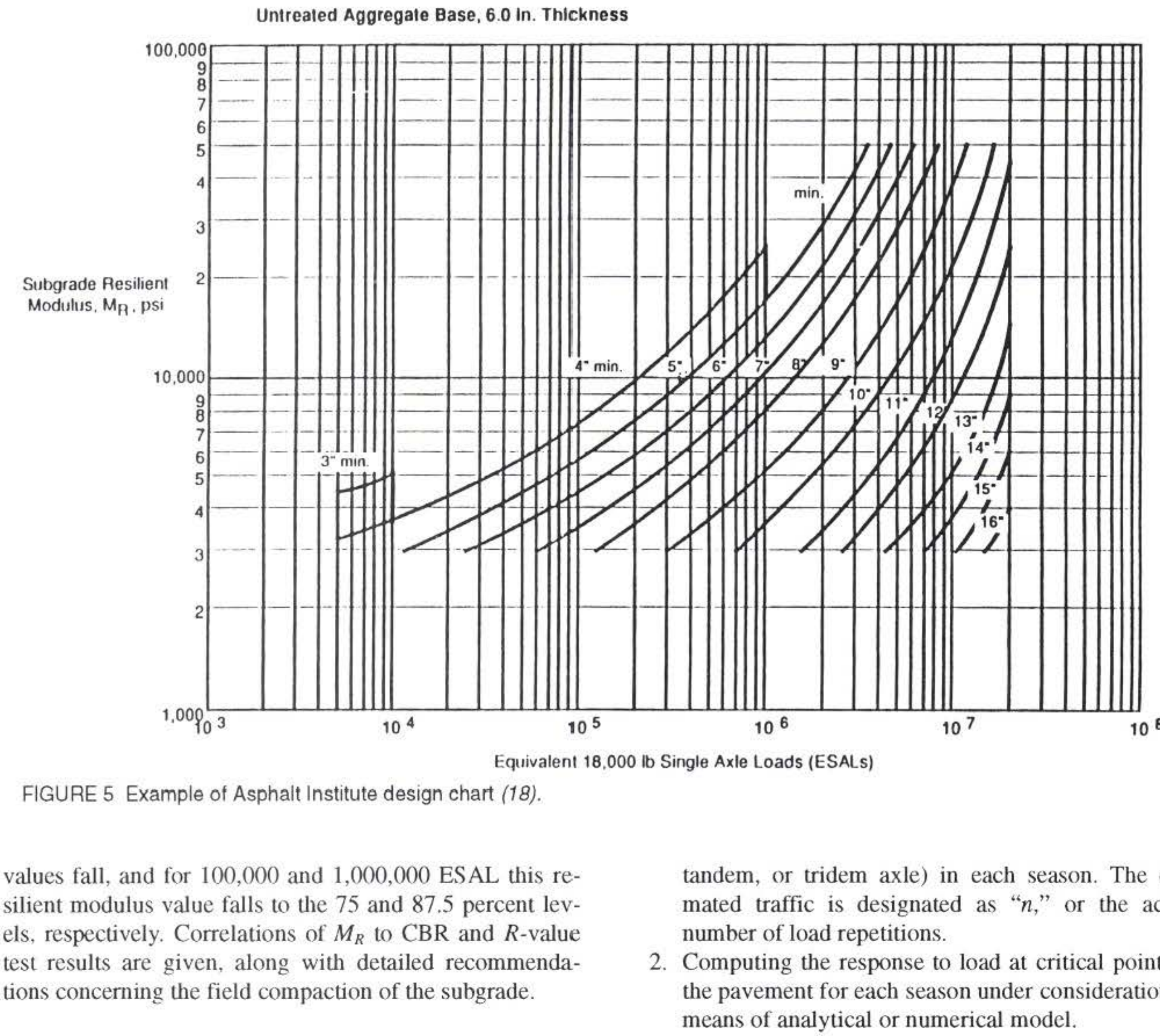



FIGURE 5 Example of Asphalt Institute design chart *(18)*.

values fall, and for 100,000 and 1,000,000 ESAL this resilient modulus value falls to the 75 and 87.5 percent levels, respectively. Correlations of $M_R$ to CBR and $R$-value test results are given, along with detailed recommendations concerning the field compaction of the subgrade.

The design subgrade resilient modulus is used with the ESAL to determine the thickness of asphalt-bound materials to be used above the subgrade or unbound granular base. This design thickness comes from nomographs, such as that shown in Figure 5. The types of structures for which these charts are given include full-depth asphalt concrete, hot-mix asphalt over emulsified asphalt base, and hot-mix asphalt over granular base.

#### Mechanistic-Empirical

Designing a pavement using a mechanistic-empirical approach is an iterative process that can incorporate the several steps shown in Figure 6. These include:

1. Analyzing the traffic for the design period using either ESAL or the load spectrum expected for each season of the year. If a spectrum approach is used, then the distribution of wheel or axle weights must be considered for each loading condition (e.g., single, tandem, or tridem axle) in each season. The estimated traffic is designated as "$n$," or the actual number of load repetitions.
2. Computing the response to load at critical points in the pavement for each season under consideration by means of analytical or numerical model.
3. Calculating the number of load cycles to failure ($N_f$) for each season, according to failure criteria or transfer functions calibrated for local conditions (climate and materials).
4. Calculating the damage ratio ($n/N_f$) for each season.
5. Summing the damage ratio for all seasons ($D$).
6. Adjusting the layer thicknesses if $D$ is not approximately 1.0.
7. Determining the final cross section.

This is the general approach common to the primary design procedure used in Illinois *(19)* and the secondary methods used in Washington *(20)*, Idaho *(21)*, and Minnesota *(22)*. Austria *(23)* used this type of procedure in developing its design catalog of pavement structures.

Establishing the duration of each season to be considered is a critical part of this design process. The seasons should reflect expected changes in material properties, such as the elastic modulus defined in chapter 1, as related to environmental conditions throughout the year. For

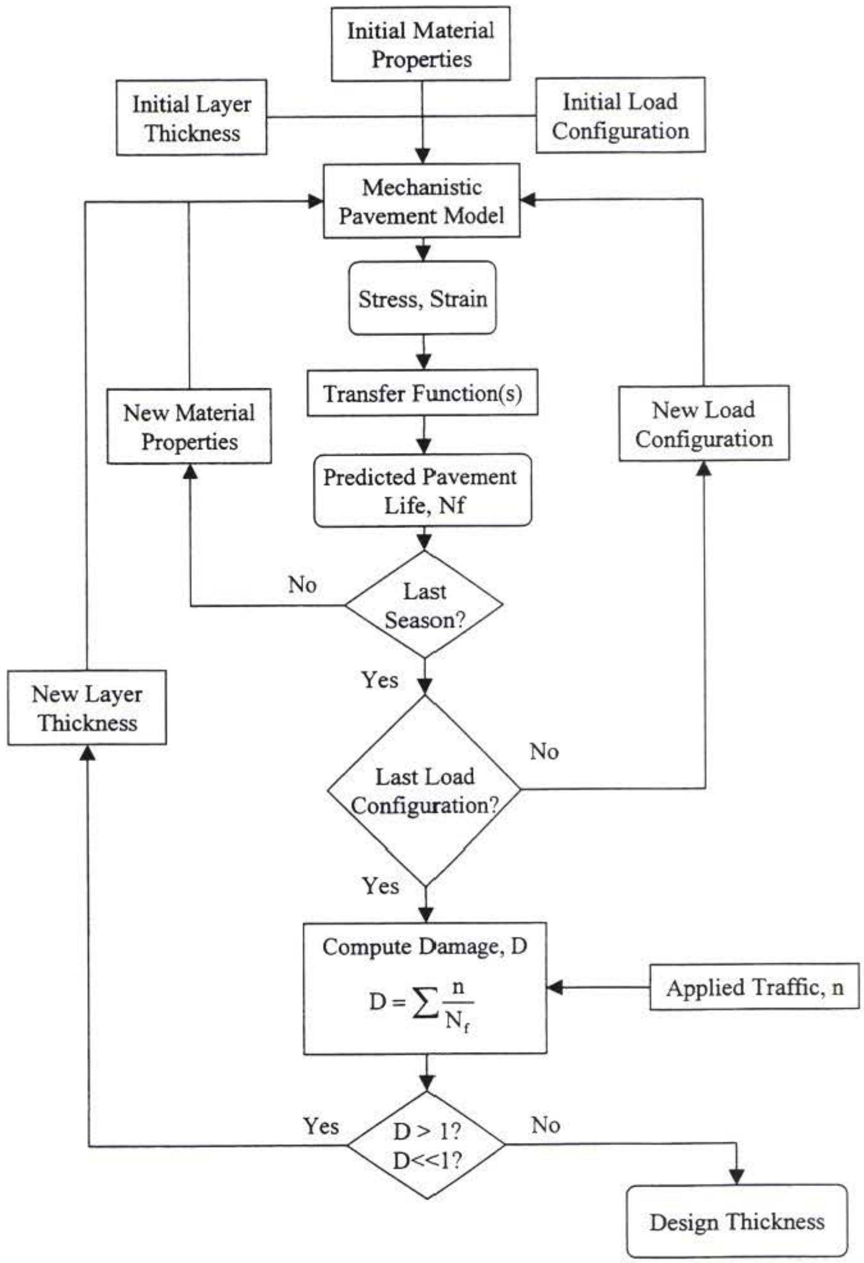



FIGURE 6 Minnesota mechanistic-empirical design flow chart *(22)*.

instance, in a northern climate there may be a 3-month period during which the water in unbound base and subgrade materials may be frozen and the asphalt concrete surface has a high modulus. This is followed by a period of time during which spring thaw occurs and the base and subgrade materials weaken considerably while the asphalt modulus only softens to an intermediate value. During the summer, the asphalt modulus will be at its lowest value, the base modulus may recover somewhat, and the subgrade might remain soft. In the fall, as temperatures decrease, the asphalt modulus may increase to an intermediate value due to cooler temperatures, and the base and subgrade moduli may increase to intermediate values due to drier conditions. These seasonal modulus values, along with the definition of the traffic loads, dictate the critical pavement responses used as input to the transfer functions that determine the allowable number of loads to failure for each seasonal condition.

Seasonal changes in the subgrade modulus will be discussed in greater detail in chapter 5, along with considerations of spatial variability. However, the value of the subgrade modulus selected for design is critical to the effort of obtaining a practical cross section. If the modulus selected is too soft, the computed pavement response will indicate the need for an overly conservative thickness. If the soil modulus is too stiff to be representative, the resulting pavement will be underdesigned, resulting in premature

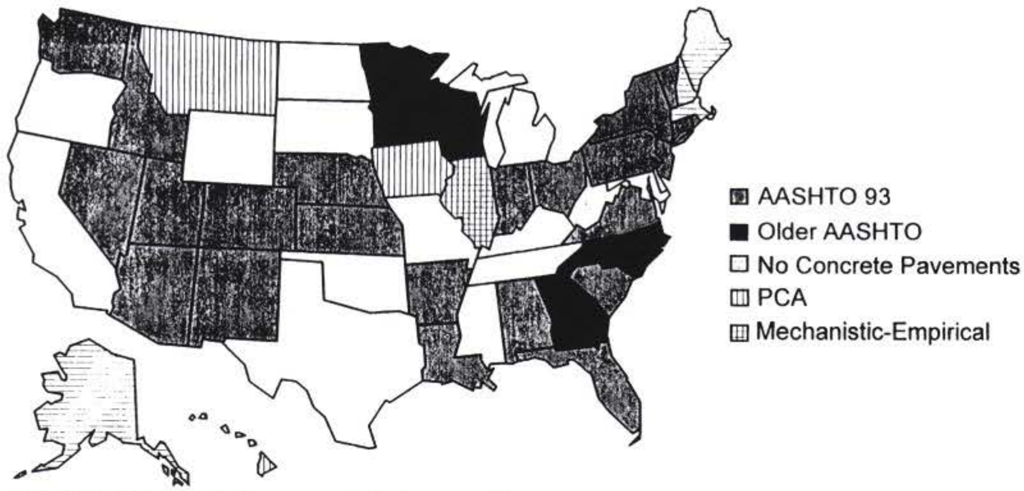



FIGURE 7 Primary rigid pavement design procedure.

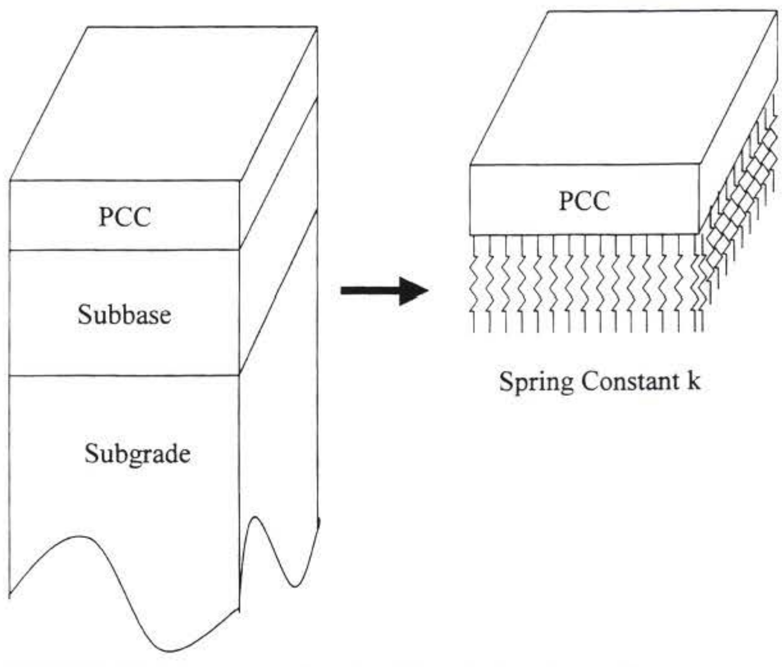



FIGURE 8 Model representation of modulus of subgrade reaction.

failure. The way to avoid this is to select layer moduli that are indicative of the average in a given season, and to apply a reliability analysis to assess the desired level of risk.

#### RIGID PAVEMENTS

As with flexible pavements, the design procedures examined for rigid pavements include empirical methods such as AASHTO and mechanistic-based procedures such as that proposed by the Portland Cement Association (PCA). The vast majority of responding state highway agencies (SHA) indicated the use of the 1993 AASHTO method as the primary procedure for rigid pavement design (Figure 7). The 1972 AASHTO design guide is used in Georgia, Minnesota, North Carolina, and Wisconsin, whereas the PCA method is used in Hawaii, Iowa, and Montana. A mechanistic-empirical approach is used in Illinois for jointed, reinforced concrete pavements and a state-developed process is used for continuously reinforced concrete pavement. Alaska, Massachusetts, Maine, and New Hampshire indicated that rigid pavements are not built by these SHAs.

Since Westergaard (*24,25*) first suggested modeling concrete pavements as slabs on liquid foundations, the modulus of subgrade reaction or *k*-value has been used to represent the overall stiffness of the underlying materials. In this model, the subgrade and base materials are viewed to act together as a spring supporting the concrete slab as shown in Figure 8. Thus, the *k*-value is considered a

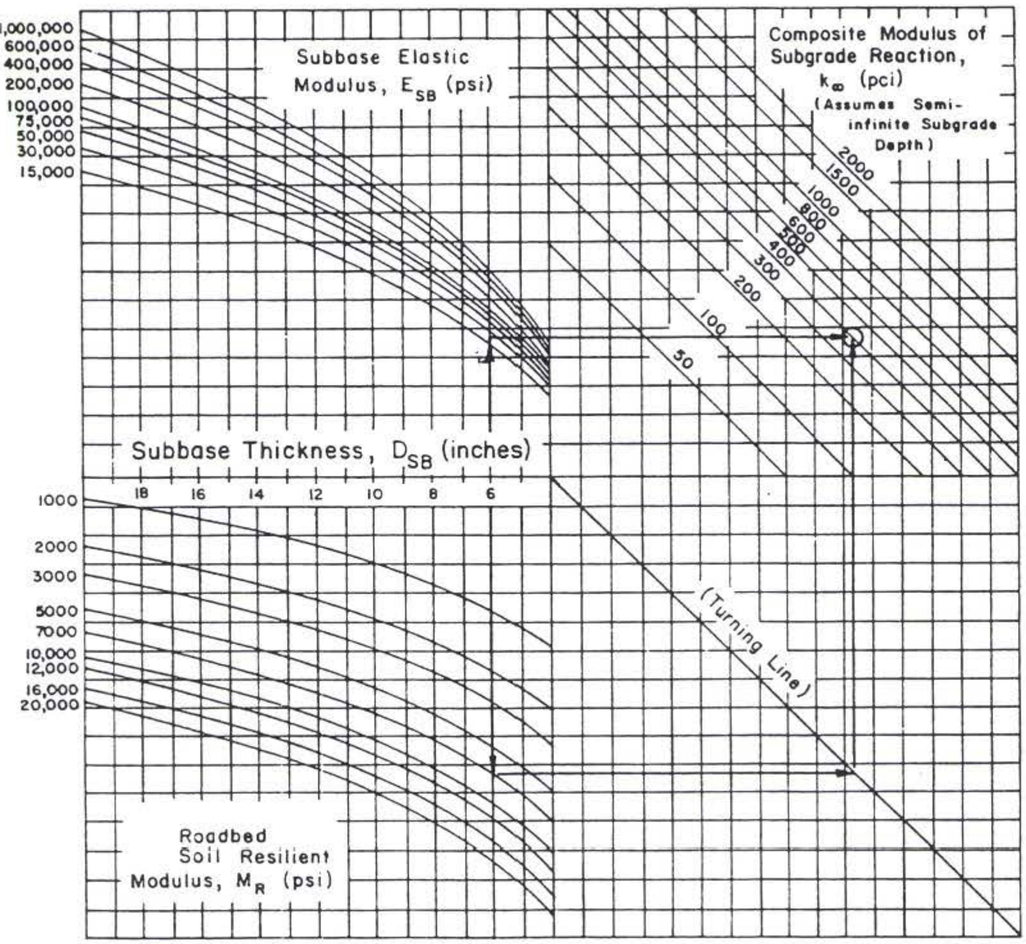



FIGURE 9 AASHTO chart for estimating modulus of subgrade reaction *(5)*.

spring constant and is expressed in terms of the unit pressure exerted on the top of the material immediately below the slab to cause a unit deflection. Although the assumption inherent in this model, that the material is linearly elastic, is generally not true for most highway materials, the range of stresses below rigid pavements is usually narrow, which reduces the effects of nonlinearity. Furthermore, the in situ plate bearing test to determine the $k$-value is time consuming, difficult, and normally cannot be performed in a timely manner for the purpose of pavement design. Thus, assumed values for certain classes of materials and correlations to other test methods have been developed. Fortunately, the performance of a rigid pavement is not very dependent upon the modulus of subgrade reaction.

## AASHTO

The 1972 AASHTO pavement design guide (*7*) presents nomographs to solve for the required portland cement concrete (PCC) slab thickness. Although empirically derived from the performance of the concrete sections at the AASHO Road Test, these nomographs were based, in part, on the Westergaard solution for corner stresses in the slab (*2*). The $k$-value is used along with the traffic, expressed in ESAL, and the working stress in the concrete to arrive at a solution. The $k$-value's effect is rather minimal compared with the other two parameters.

The latest AASHTO guide (*5*) presents many additional factors to be considered in the design of rigid pavements. Load transfer as a function of shoulder type and dowel placement, the drainage characteristics of underlying materials, the concrete modulus of elasticity, the concrete modulus of rupture, the level of desired reliability, and the overall standard deviation of pavement performance are included in the new design equation.

The 1993 guide (*5*) presents an elaborate method for determining the modulus of subgrade reaction. One begins by using the seasonal modulus of the roadbed soil and the modulus of the subbase and its thickness to determine a seasonal composite $k$-value for the underlying materials using the nomograph shown in Figure 9. If a rigid layer exists within 3 m of the surface, the seasonal $k$-value must be adjusted to account for it. Then, a seasonal damage

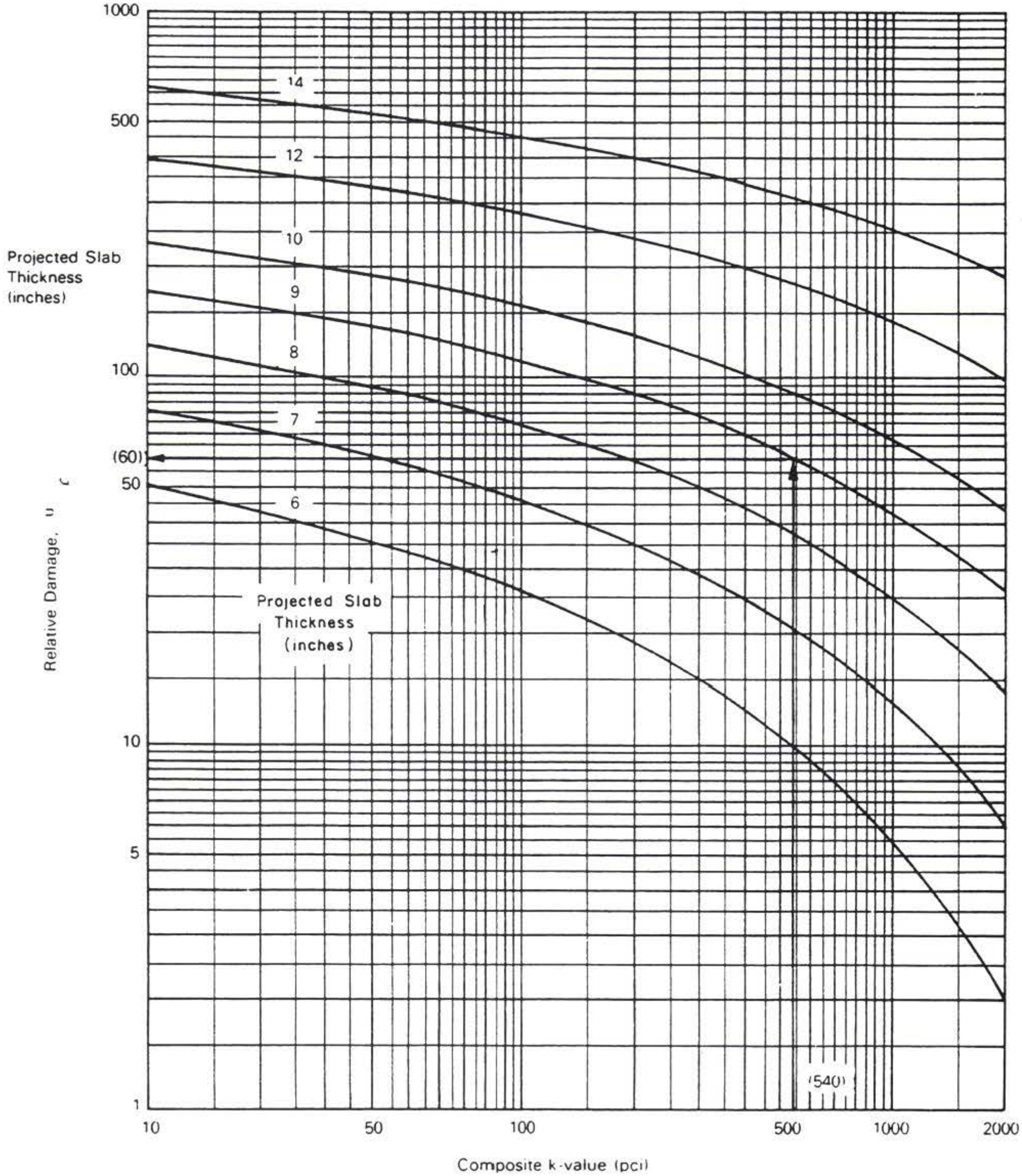



FIGURE 10 AASHTO chart for estimating relative damage to concrete pavements *(5)*.

factor is determined by using Figure 10 for which the input parameters are the composite *k*-value and the estimated concrete slab thickness. An average damage factor is determined for all the seasons of the year, and an effective modulus of subgrade reaction is found using Figure 11. Finally, an adjustment to the effective modulus is made to account for the loss of support or erodability of the underlying material. This loss of support factor ranges from a value of 0 to 3, and it is greater for unbound materials than bound materials.

The 1993 AASHTO guide (*5*) also allows for the estimation of in situ *k*-values for pavement rehabilitation by means of deflection testing. For this, a parameter called AREA (*26*) is calculated from the deflection basin and is used with the maximum deflection measurement to arrive

Trial Subbase: Type Granular

Thickness (inches) 6

Loss of Support, LS 1.0

Depth to Rigid Foundation (feet) 5

Projected Slab Thickness (inches) 9

| (1) Month | (2) Roadbed Modulus, $M_R$ (psi) | (3) Subbase Modulus, $E_{SB}$ (psi) | (4) Composite k-Value (pci) (Fig. 3.3) | (5) k-Value (pci) on Rigid Foundation (Fig. 3.4) | (6) Relative Damage, $u_r$ (Fig. 3.5) |
|---|---|---|---|---|---|
| Jan. | 20,000 | 50,000 | 1,100 | 1,350 | 0.35 |
| Feb. | 20,000 | 50,000 | 1,100 | 1,350 | 0.35 |
| Mar. | 2,500 | 15,000 | 160 | 230 | 0.86 |
| Apr. | 4,000 | 15,000 | 230 | 300 | 0.78 |
| May | 4,000 | 15,000 | 230 | 300 | 0.78 |
| June | 7,000 | 20,000 | 410 | 540 | 0.60 |
| July | 7,000 | 20,000 | 410 | 540 | 0.60 |
| Aug. | 7,000 | 20,000 | 410 | 540 | 0.60 |
| Sept. | 7,000 | 20,000 | 410 | 540 | 0.60 |
| Oct. | 7,000 | 20,000 | 410 | 540 | 0.60 |
| Nov. | 4,000 | 15,000 | 230 | 300 | 0.78 |
| Dec. | 20,000 | 50,000 | 1,100 | 1,350 | 0.35 |
| | | | | Summation: $\Sigma u_r$ = | 7.25 |

Average: $\bar{u}_r = \frac{\Sigma u_r}{n} = \frac{7.25}{12} = 0.60$

Effective Modulus of Subgrade Reaction, k (pci) = 540

Corrected for Loss of Support: k (pci) = 170

FIGURE 11 AASHTO method for determining effective modulus of subgrade reaction *(5)*.

at an effective dynamic *k*-value. An effective static *k*-value is calculated by dividing the dynamic value by two.

#### Portland Cement Association

The PCA's method of rigid pavement design includes consideration of load transfer at transverse joints, the type of shoulder, the type of base, and the configurations of loads (*27*). It is based on finite-element analyses that were used to develop tables and nomographs; therefore, it may be considered a mechanistic-based approach. Stresses due to edge loading were found to be critical for fatigue, and the type of shoulder (concrete versus nonconcrete) was shown to be important. Deflection of the slab corner is considered critical for the erosion of the underlying material. The presence of a load transfer mechanism and the type of shoulder are important for designing against erosion. Unlike the AASHTO rigid pavement design procedure, the PCA method uses a load spectrum approach to determining damage. Conservatism in the process is obtained in the form of a multiplier used for increasing the magnitude of axle loads.

The *k*-value used in the PCA method may be obtained by means of a plate bearing test on top of the subbase or by

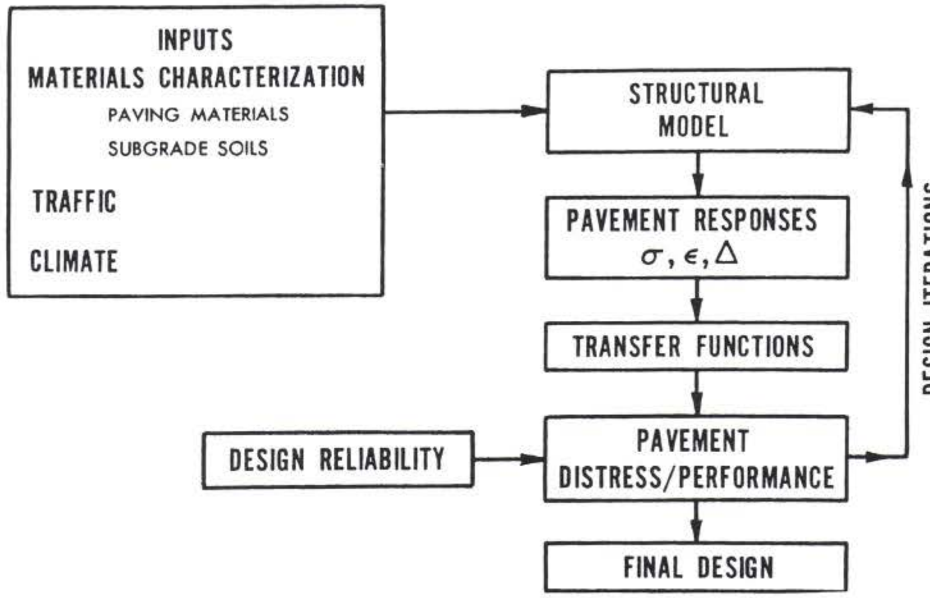



FIGURE 12 Mechanistic-empirical design concept from NCHRP Project 1-26 *(19)*.

a correlation to CBR, *R*-value, or soil classification. An effective *k*-value on top of the subbase may be obtained from tables by considering the subgrade *k*-value and the type and thickness of the proposed subbase. The range of effective *k*-values is from 18 MPa/m (65 pci) for a thin, untreated subbase over a relatively soft subgrade to 226 MPa/m (830 pci) for a thick, cement-treated subbase over a relatively stiff subgrade. The PCA (*28*) emphasizes the importance of uniformity in the support conditions to obtain good pavement performance.

#### Mechanistic-Empirical

The final report for NCHRP Project 1-26 (*19*) presents a procedure to mechanistically design concrete pavements. The concept for this process is shown in Figure 12, and it very similar to the mechanistic-empirical process for flexible pavements. A user-friendly computer program called ILLI-CONC is available for rigid pavement design. Transfer functions have been developed to consider failure due to fatigue cracking and pumping. The basic structural model consists of Westergaard equations (*24,25*) and the results are modified to account for slab size, load configuration, load transfer, shoulder type, and curling. The adjustment factors were the result of modeling with the finite-element program ILLI-SLAB. The pavement responses of interest are: (1) maximum edge stress on the outside of the slab, (2) combined load and curling stress at the transverse joint, (3) maximum corner deflection, and (4) maximum bearing stress at the concrete/dowel interface.

A complete discussion on the justification for considering the support under a concrete slab as a dense liquid is given in NCHRP Project 1-26 *(19)*. Essentially, the *k*-value allows the model to represent the behavior at joints and cracks much more easily than using the resilient modulus. Other advantages include designers' familiarity with the *k*-value and the probability that different $M_R$ values would have to be used when analyzing flexible pavements over the same soil due to the effects of nonlinearity. These researchers concluded that the proper *k*-value for design should be that for the natural soil, and not the "effective" *k*-value on top of the subbase. Their rationale is that the effect of stabilized subbases can be considered as part of the pavement system during the analysis and that unstabilized subbase materials do not significantly affect the support conditions.

### SUMMARY

Subgrade strength and stiffness are primary parameters in both flexible and rigid pavement design. It is important for the designer to understand the factors influencing the behavior of the soil relative to the pavement structure. Seasonal changes in moisture content and the state of moisture have a profound effect on the properties of the subgrade. Spatial variability in the soil or rock mass must be accounted for in design by selecting a strength or stiffness value representative of the roadway section under consideration. This implies that a sufficient amount of characterization must take place to accurately quantify the variability of the soil. If index measures such as CBR or *R*-value are used, the designer must be aware of the assumptions and limitations of the test method. Furthermore, if an analytical method such as layered elastic theory or a numerical method such as a finite-element method is to be used in calculating pavement response for a mechanistic-based procedure, it is important that the designer account for an appropriate modulus according to the stress range to which the soil will be subjected in service.

New pavements usually need to be designed before the subgrade is prepared for construction. Thus, testing of in situ conditions at this point in the process may be inappropriate in many ways. However, it is important that in situ testing of the soils take place during the construction to verify assumptions made in the design process and to take action to correct any problems created by insufficiently stiff or nonuniform conditions. It is very important that in situ soils and material testing be performed on any pavement being considered for rehabilitation. This will provide the designer with critical information concerning the foundation support for the pavement. Testing on rehabilitation projects may also be used to establish a database for the design of new pavements.

CHAPTER THREE

# FACTORS AFFECTING STIFFNESS AND STRENGTH

In pavement engineering, knowledge of the strength and stiffness of subgrades is important. The strength of the subgrade denotes the largest stress that the material can sustain and it governs the bearing capacity of the subgrade. Subgrade soil stiffness on the other hand largely determines the strains and displacements of the subgrade, as it is loaded and unloaded. It is important to understand which factors influence the strength and stiffness of pavement subgrades for the following reasons. First, knowledge of the strength of subgrade materials is necessary to evaluate bearing capacity of the subgrade for future road construction. Second, knowledge of the stiffness of the subgrade materials is necessary for evaluating both the initial, time-dependent, and long-term movements of pavement embankments under static and dynamic loads. However, both strength and stiffness are influenced by a number of factors, such as the state of stress, moisture content, state of moisture (temperature), and relative density. These factors will be discussed later in the chapter. First, a brief overview of the basic definitions of soil strength and stiffness are provided.

## SUBGRADE SOIL STRENGTH

Materials that have strength can sustain shear stresses and the strength is the maximum shear stress that can be sustained. The shear strength of soil can be regarded as the resistance to deformation by continuous shear displacement of soil particles along surfaces of rupture (i.e., slip lines). From a practical point of view, knowledge of the shear strength of a pavement subgrade is necessary to determine the bearing capacity of a highway embankment. The shear strength of subgrade soil can be attributed to three basic components: first, the frictional resistance between solid particles; second, the cohesion and adhesion between soil particles; and third, the interlocking of solid particles to resist deformation. In simple terms, the strength of subgrade soils may be expressed in terms of the Mohr-Coulomb failure criterion, given by

$$s = c + \sigma_n \tan \phi \qquad (1)$$

where

$s$ = shear strength at failure,
$c$ = cohesion intercept,
$\sigma_n$ = normal stress, and
$\phi$ = friction angle.

This illustrates the intuitive notion that the shear strength of soils is stress-dependent, because it is dependent upon the normal stress.

## SUBGRADE DEFORMATION PROPERTIES

The deformation properties of elastic materials can be described by the modulus of elasticity ($E$) and Poisson's ratio ($\nu$). Although these parameters are defined strictly based on the theory of elasticity, they are used commonly for inelastic materials such as subgrade soils because the design is intended to ensure that only low strains, which do not produce permanent deformation, occur in lower pavement layers.

For soils, the resilient modulus is typically obtained from cyclic triaxial compression testing, and this is used as a proxy value for the modulus of elasticity. The triaxial compression testing of soil can be described by the following steps. First, a sample is prepared in a manner that replicates field conditions; however, undisturbed samples, if available, are best. Second, the specimen is enclosed within a thin rubber membrane, placed in a pressure chamber, and confining pressure ($\sigma_3$) is applied. Third, the sample undergoes repeated pulses of an increasing axial stress until failure. The deviator stress ($\sigma_1$ - $\sigma_3$) is equal to the total vertical stress applied by the testing apparatus ($\sigma_1$) minus the confining stress ($\sigma_3$).

Figure 13 (*29*) shows a typical stress-strain curve for soil. The Young's modulus is the gradient of the stress-strain curve. If the curve is linear the gradient is easy to determine, but if it is curved the modulus at a point such as A may be taken as a tangent or a secant, as shown in Figure 13 and given by

$$\text{tangent modulus } (E_t) = d(\sigma_1 - \sigma_3)/d\varepsilon_a \qquad (2)$$

$$\text{secant modulus } (E_s) = \Delta(\sigma_1 - \sigma_3)/\Delta\varepsilon_a \qquad (3)$$

in which ($\sigma_1$ - $\sigma_3$ ) = deviator stress, and $\varepsilon_a$ = axial strain. The tangent modulus at zero deviatoric stress is termed the initial tangent modulus and denoted as $E_i$. Similarly, Poisson's ratio ($\nu$) is defined as the ratio of the radial strain ($\varepsilon_r$) to the axial strain ($\varepsilon_a$)

$$\nu = - d\varepsilon_r / d\varepsilon_a \qquad (4)$$

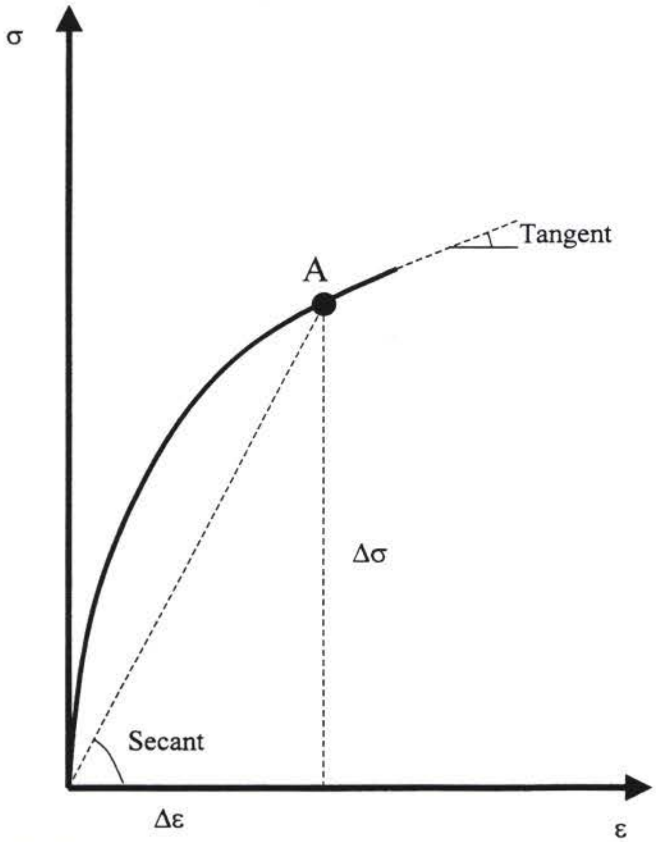



FIGURE 13 Definitions of tangent and secant moduli *(29)*.

The values of Poisson's ratio, and especially the value of the modulus, will be dependent on the confining stress and the degree of overconsolidation that exists in the soil. Therefore, the subgrade soil moduli and Poisson's ratio can be regarded as being both nonlinear and stress-dependent.

Besides the secant and tangent modulus, there is the resilient modulus ($M_R$), which is the most commonly used modulus in pavement design, as shown in Table B-1. The resilient modulus is used in place of the modulus of elasticity, because it is obtained under dynamic rather than static conditions. The resilient modulus is based on stress and strain measurements from rapidly applied loads—more like those that pavement materials experience from wheel loads.

In the laboratory, the resilient modulus of soil is obtained from cylic triaxial testing of the soil. When the deviator stress is applied, the sample deforms, changing in length. This change in length is directly proportional to the modulus.

To calculate the resilient modulus, use the following equation

$$\text{resilient modulus } (M_R) = d(\sigma_1 - \sigma_3)/d\varepsilon_{rec} \quad (5)$$

where $d\varepsilon_{rec}$ = recoverable axial strain.

In the following discussion the term modulus will be taken to mean the resilient modulus, because of its widespread use in pavement engineering.

## STATE OF STRESS

In pavement design, knowledge of the estimated in situ state of stress is valuable. First, in the case of subgrade characterization before construction, these stresses represent the original conditions on which the construction of the highway embankment imposes stress increments. Second, in the case of constructed pavements, the in situ state of stress represents the operating stress of the pavement system. Third, nearly all pavement engineering properties are a function of the stresses within the various layers of the pavement, either directly or indirectly, as discussed previously for the modulus and Poisson's ratio. Therefore, in situ stresses are needed to evaluate the stress dependency of pavement materials under operating conditions. Figure 14 (*30*) shows typical drained triaxial compression test results on soil at different confining stresses $\sigma_c > \sigma_b > \sigma_a$. It is evident that both the modulus and Poisson's ratio vary with confining stress. However, both Poisson's ratio and the modulus may increase, decrease, or remain relatively constant depending on the particular soil involved.

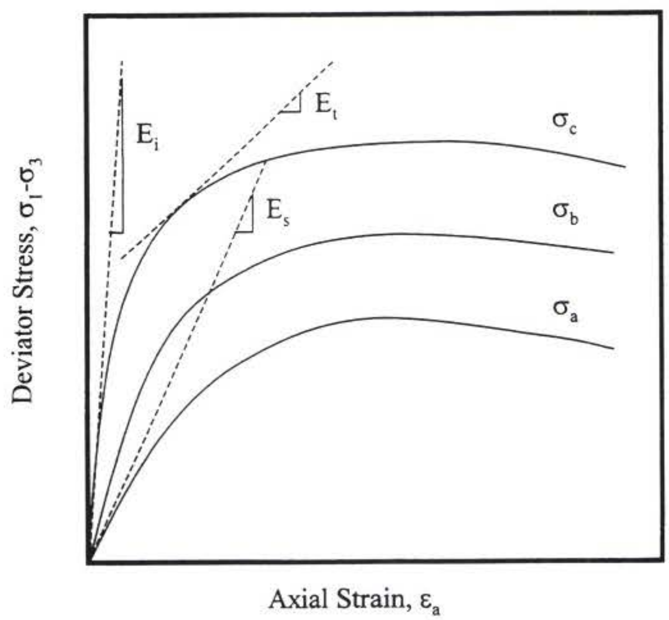



FIGURE 14 Typical drained triaxial compression results *(30)*.

In the following discussion, the effect of the state of stress on the strength and modulus of pavement subgrade materials will be discussed in both granular and fine-grained soils. Numerous models available to describe the stress dependency of the subgrade modulus will be discussed with a particular emphasis on the results relative to in situ testing.

## POISSON'S RATIO

Poisson's ratio will vary according to the particular soil involved. For isotropic elastic materials the range of Poisson's ratio is from 0 to 0.5. For dilatant soils that are inelastic Poisson's ratio may exceed 0.5, because at that point the material is no longer elastic or it is stressed to the point of cracking. Unfortunately, the literature does not provide much information for the characterization and correlation studies of Poisson's ratio. For undrained loading of saturated cohesive soil no volume change occurs, and in this case the undrained Poisson's ratio is equal to 0.5. For drained loading, Poisson's ratio varies with soil type as shown in Table 3.

TABLE 3

TYPICAL VALUES OF POISSON'S RATIO *(2)*

| Soil | Range |
|---|---|
| Untreated granular material | 0.3–0.4 |
| Cement-treated granular material | 0.1–0.2 |
| Cement-treated fine-grained material | 0.15–0.35 |
| Lime-stabilized material | 0.1–0.25 |
| Lime-flyash mixture | 0.1–0.15 |
| Loose sand or silty sand | 0.2–0.4 |
| Dense sand | 0.3–0.45 |
| Fine-grained soils | 0.3–0.5 |
| Saturated soft clays | 0.4–0.5 |

## SHEAR STRENGTH OF SUBGRADE MATERIALS

The shear strength of soils can be described within the framework of drained (effective stress) or undrained (total stress) analysis. In drained analysis, the Mohr-Coulomb failure criterion in Eq. (1) is expressed as

$$s = \sigma'_n \tan \phi' \tag{6}$$

where

$\sigma'_n$ = effective normal stress, and
$\phi'$ = effective stress friction angle.

No cohesion intercept is shown in Eq. (6), because it occurs only in special cases such as with cemented soils and heavily overconsolidated clays *(30)*. The friction angle of soils varies with many factors. For a given soil at a constant normal effective stress and moisture content the friction angle varies with density and strain (Figure 15).

Granular soils are best described by Eq. (6) in its current form, whereas the shear strength of fine-grained soils is often described within the framework of total stress analysis ($\phi = 0$) where

$$s = s_u \tag{7}$$

where $s_u$ = the undrained strength of the soil.

The strength of soil can be measured or inferred from a number of different in situ soil tests. Some of these tests, such as the vane shear test, provide a direct measure of the undrained strength of soils, whereas measured parameters obtained from other in situ tests, such as the standard penetration test (SPT), the cone penetration test (CPT), and the pressuremeter test (PMT) provide empirical links

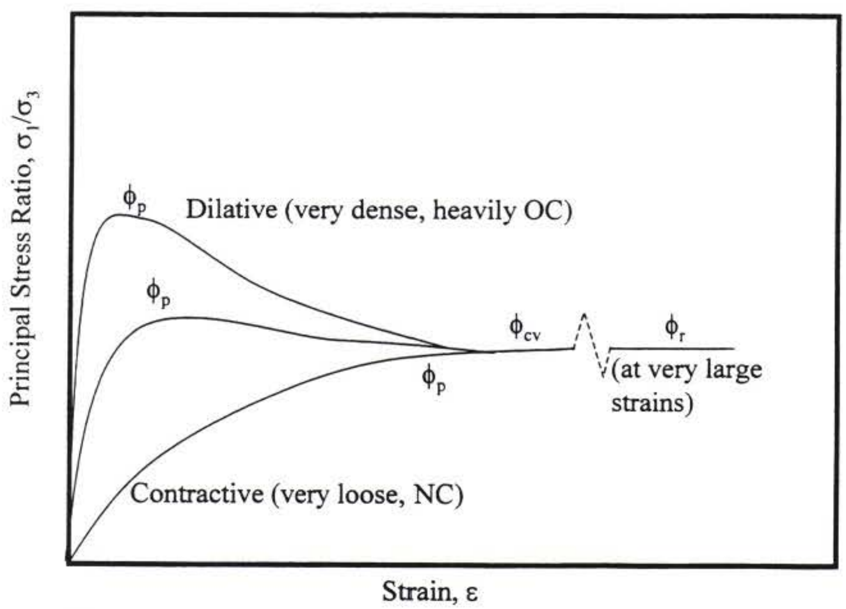



FIGURE 15 Variations in friction angle *(30)*.

to undrained shear strength obtained from, for example, triaxial compression tests. Finally, other in situ tests, such as the DCP and the field CBR, may only provide an indirect measure of the undrained shear strength of soils, with only a very limited number of existing correlations with undrained shear strength obtained from traditional laboratory undrained triaxial compression tests.

### RESILIENT MODULUS

Changes in stress can have a large impact on resilient modulus. "Typical" relationships are shown in Figure 16. As shown in Figure 16(*a*), the resilient modulus for granular materials is assumed to be a function of bulk stress (*31,32*)

$$M_R = k_1\theta^{k_2} = k_1(\sigma_1 + \sigma_2 + \sigma_3)^{k_2} = k_1 I_1^{k_2} = k_1(3p)^{k_2} \quad (8)$$

where $\theta$ includes static and dynamic stresses, and $k_1$ and $k_2$ are regression constants ($k_2$ is positive so that the modulus is always increasing with increasing bulk stress). For cohesive materials the nonlinear relation is often written as (*31,32*)

$$M_R = k_1\sigma_d = k_1(\sigma_1 - \sigma_3)^{k_2} \quad (9)$$

where $k_2$ is negative, so that the modulus is now decreasing with increasing deviator stress [Figure 16(*b*)].

It should be noted that the $k$-$\theta$ model presented in Eq. (8) for granular materials can give inaccurate results, because it neglects the important effect of shear stress on the resilient modulus (*33*). To account for the effects of shear stress on the resilient modulus other constitutive relationships have been used to represent laboratory test results of all granular and cohesive subgrade soils. One of these relationships is (*34*)

$$M_R = k_1(\sigma_d)^{k_2}(1 + \sigma_3)^{k_3} \quad (10)$$

and the other is (*33*)

$$M_R = k_1 p_a [\theta / p_a]^{k_2} [\sigma / p_a]^{k_3} \quad (11)$$

where

$\sigma_3$ = confining pressure;
$p_a$ = atmospheric pressure; and
$k_1$, $k_2$, and $k_3$ = regression constants.

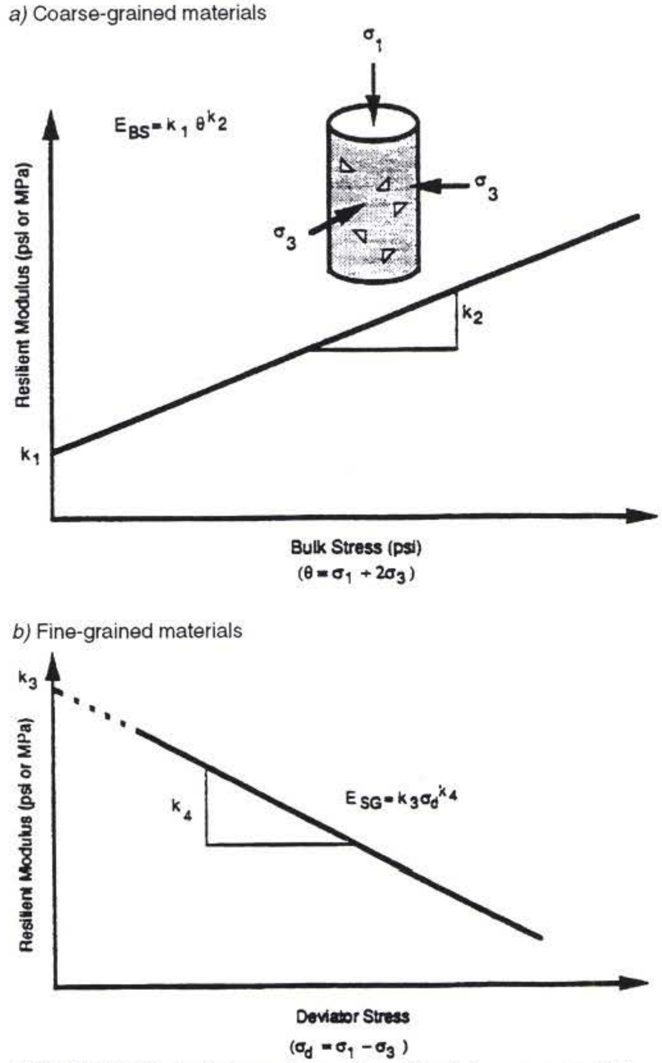



FIGURE 16 Typical stress-dependency for *(a)* coarse- and *(b)* fine-grained materials *(31)*.

Because of its simplicity, the constants used in the model presented in Eq. (11) can be determined readily from resilient modulus tests such as the AASHTO T 294-94.

### SUBGRADE REACTION

In contrast with elastic theories that use elastic or resilient modulus values, an alternative method for analyzing load-displacement relationships beneath portland cement concrete pavements is the concept of subgrade reaction. In subgrade reaction models there is a basic parameter that is analogous to a spring constant as discussed in chapter 2 (Figure 8). This parameter is defined as the modulus of subgrade reaction ($k$).

The modulus of subgrade reaction is primarily dependent on certain soil characteristics such as density, moisture, and soil texture. As with the resilient modulus, $k$ varies with stress level. However, $k$ also varies with the width of the loaded area (*35*). Finally, other factors, such as the state of moisture and whether or not bedrock lies within 3 m (10 ft) of subgrade surface, also influence the $k$-value (*5*). To account for the moisture effects, the following adjustment in the modulus of subgrade reaction is typically made to reflect the most unfavorable subgrade condition expected

$$k = (d/d_s)k_{\text{field}} \tag{12}$$

where

$d$ = deformation at field moisture content,
$d_s$ = deformation at saturated moisture content,
$k_{\text{field}}$ = $k$ obtained from the field, and
$k$ = modified modulus of subgrade reaction.

## EFFECTS OF MOISTURE AND DENSITY

The shear strength of soil is dependent on the environmental conditions, such as the moisture content and the state of moisture (i.e., frozen or unfrozen), as discussed by Lambe and Whitman (*36*). In general, the strength of soil decreases with increasing moisture content and increases as the soil goes from an unfrozen to a frozen state. Similarly, both the resilient modulus and the modulus of subgrade reaction of soils will typically increase with decreasing water content, as well as when soil moisture changes from an unfrozen to a frozen state.

Density also plays an important role in defining the strength of soils. For the same dry density the CBR of a soil will decrease with moisture content (*2*), and for a given level of compactive effort the density will increase with moisture content to a peak and then decrease. In the latter case, the soil structure takes the load up to the peak density, but beyond this point the load is increasingly transferred to the pore water. Because water acts as a lubricant, the soil strength is decreased. Explained simply, as the relative density increases, there is more soil to carry the load and less pore space for water. Thus, for soils at the appropriate moisture content, as the compactive effort increases, the density increases, and consequently the strength and stiffness increase.

Lary et al. (*37*) performed laboratory resilient modulus tests on soil samples for the U.S. Forest Service. Figure 17 shows the relationships they obtained for various levels of dry density and moisture contents for base and subgrade materials from a Forest Service road in the Willamette National Forest. For a number of base and subgrade materials they proposed the stress-dependency models shown in Table 4.

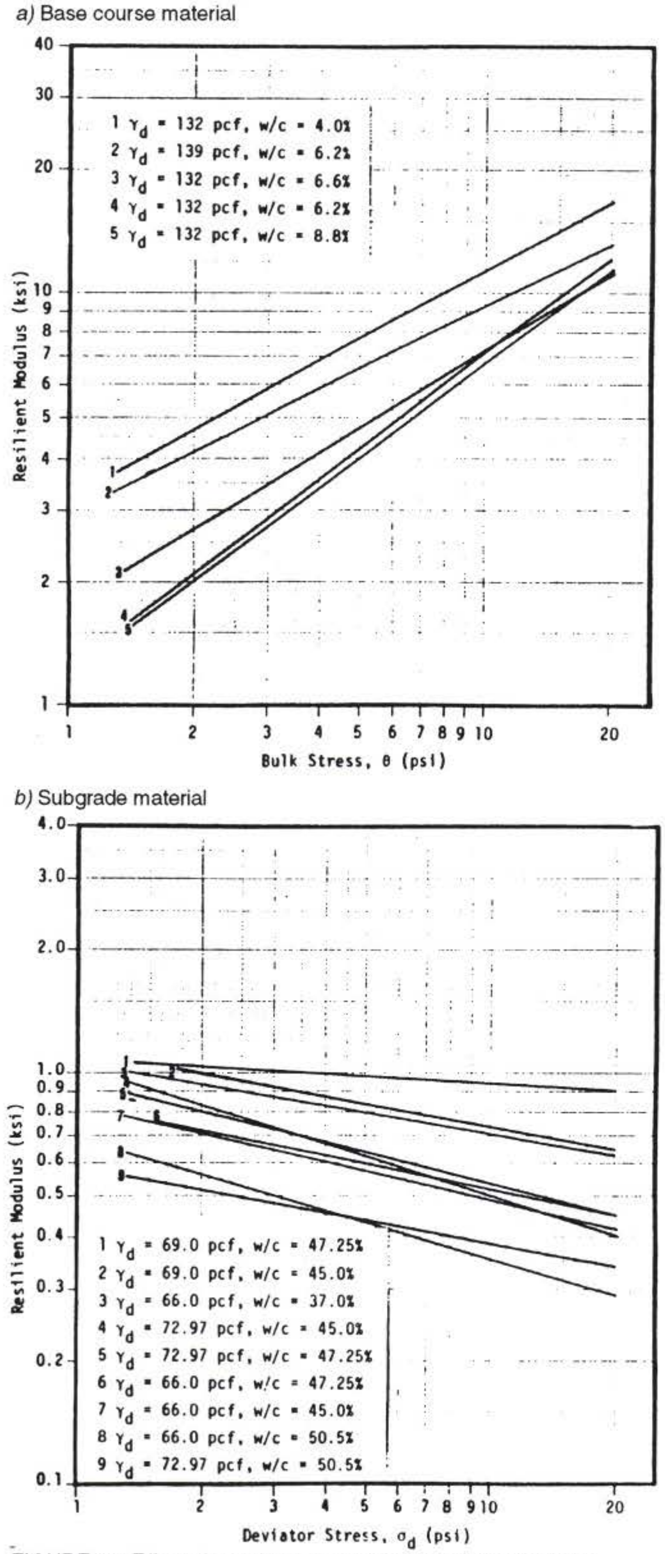



FIGURE 17 Effect of density and moisture content on resilient modulus of unbound materials (*37*).

As noted in the questionnaire response from the Virginia DOT, moisture and soil conditions at the time of pavement design are not those of the in-service conditions. Generally speaking, it is their experience that soil suction occurring during the life of the of the pavement

TABLE 4

SUBGRADE AND SOIL STRESS DEPENDENCY MODELS ACCOUNTING FOR DENSITY AND MOISTURE CONTENT *(37)*

| National Forest Site | Layer | Equation | $r^2$ |
|---|---|---|---|
| Olympic | Subgrade | $\log M_R = 0.749 + 0.673 (\log \theta) - 0.0286 (mc) - 0.0008 (\gamma d)$ | 0.84 |
| Olympic | Base | $\log M_R = -0.102 + 0.796 (\log \theta) - 0.0124 (mc) + 0.0053 (\gamma d)$ | 0.84 |
| Wenatchee | Subgrade | $\log M_R = -0.266 + 0.551 (\log \theta) - 0.0554 (mc) + 0.0097 (\gamma d)$ | 0.89 |
| Wenatchee | Base | $\log M_R = -0.636 + 0.581 (\log \theta) - 0.0254 (mc) + 0.0102 (\gamma d)$ | 0.88 |
| Deschutes | Subgrade | $\log M_R = -0.850 + 0.671 (\log \theta) - 0.0122 (mc) + 0.0182 (\gamma d)$ | 0.90 |
| Deschutes | Base | $\log M_R = 0.473 + 0.584 (\log \theta) - 0.0324 (mc) + 0.0022 (\gamma d)$ | 0.93 |
| Willamette | Subgrade | $\log M_R = 1.61 - 0.231 (\log \sigma d) - 0.0346 (mc) + 0.0130 (\gamma d)$ | 0.42 |
| Willamette | Base | $\log M_R = -0.0143 + 0.645 (\log \theta) - 0.0304(mc) + 0.0035 (\gamma d)$ | 0.93 |

leads to higher effective stresses and, therefore, a stiffer subgrade.

Because soil suction is a function of the degree of saturation, it is important to consider the degree of saturation during construction, compared with the eventual in-service saturation. Lee et al. *(38)* found that cohesive soils under pavements could achieve saturation levels on the order of 90 to 100 percent after a number of years, independent of the level of compaction during construction.

CHAPTER FOUR

# METHODS FOR MEASURING IN SITU SUBGRADE MECHANICAL PROPERTIES

During the past 40 years, several test devices have been developed for the nondestructive structural evaluation of pavements systems. Some of the older methods include heavy vibrators, truck wheel loads (e.g., Benkelman beam and Lacroix deflectograph), static plate-loading tests, the FWD, and steady-state devices. Newer methods include the RWD, SASW, the Finnish Loadman, and the Humboldt Stiffness Gauge. These methods/devices may be divided into those providing modulus values at either large or small loads. The large-load methods include the FWD, steady-state dynamic devices, the Benkelman beam, the plate bearing test, and the RWD. The small-load methods include the Finnish Loadman, the SASW, and the Humboldt Stiffness Gauge.

## DEFLECTION METHODS

The idea of using deflection measurements to evaluate the structural integrity of pavements dates back to at least 1938 when the California Division of Highways used electrical gauges implanted in roadways to measure displacements induced by truck loads *(39)*. The plate load bearing test was used by the Department of Transport of Canada starting in the mid-1940s *(40)*. The Benkelman beam was introduced in the late 1940s and is still in use in some local jurisdictions. Other quasi-static deflection measuring devices were developed based on the same principles, but were more easily transported. Dynamic load deflection devices were first introduced in the early 1960s. The earlier types of devices used an oscillating force superimposed on a static load to produce surface deflections. The FWD were the next generation of dynamic equipment and used a single impulse load. Devices operating by the measurement of pavement displacement under a rolling load are currently being developed.

The evolution of deflection testing equipment has been such that researchers like Smith and Lytton *(41)* have expressed concern over the lack of standardization. They stated that loads, loading plate geometry, sensor spacing, and sensor sensitivity are among the items needing definition. One step toward providing standardization for FWD measurements is the calibration process proposed during the SHRP *(4)*. Van Gurp et al. *(42)* have taken the approach that harmonization may provide a means of obtaining uniform evaluations of pavement structures.

Deflection tests are the most common method of ascertaining the stiffness of pavement layer materials, especially the subgrade. A variety of means are employed to impart a load to a pavement layer or pavement surface, and the load may range from static to dynamic in its application. One or more deflections may be measured using techniques ranging from dial gauges to velocity transducers to lasers. Analysis of the deflection measurements may range from using empirical relationships between deflection and pavement life to the backcalculation of layer moduli.

### Deflection Testing Equipment

#### *Impulse (Falling Weight Deflectometer)*

Devices in this category are defined by Smith and Lytton *(41)* as equipment that "delivers a transient force impulse to the pavement surface." Numerous descriptions of different FWDs can be found in the literature *(43,44–46)*. These are all trailer mounted devices, which may be towed by passenger vehicles or light trucks. As the name implies, the FWD imparts its test load by means of a specific weight falling a given distance and striking a buffered plate resting on the pavement surface. The peak force *(F)* generated by the FWD is

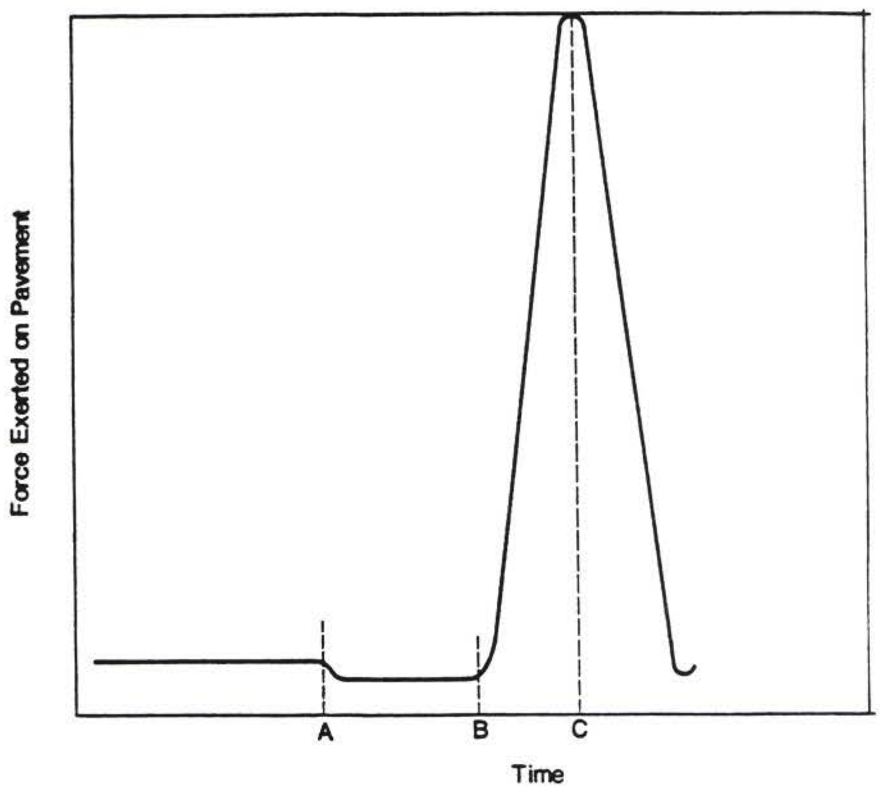
$$F = \sqrt{2Mghk} \qquad (13)$$

where

$M$ = falling mass,
$g$ = gravitational acceleration,
$h$ = drop height, and
$k$ = spring constant.

The force may be changed by varying any of the parameters in the test, but the most common method is to change the drop height.

Figure 18 shows that for this type of equipment the static load is very small compared with the dynamic load. The dynamic load produced by a FWD for highway work typically ranges between 4 and 107 kN (1,000 and 24,000 lb), although a heavier version called a heavy weight deflectometer may be used on concrete pavements with loads of up to 245 kN (55,000 lb). The impulse load duration of




A–Time at which weights are released
B–Time at which weight package make first contact load plate
C–Peak load reached

FIGURE 18 Force-time relationship for FWD *(41)*.

from 25 to 30 msec approximates the same load duration of a vehicle traveling at 65 to 80 kph (40 to 50 mph). Deflections with this equipment are measured at the center of the load and at one to eight locations away from the loading plate as shown in Figure 19. A standard method of testing using an FWD may be found in ASTM test designation D 4694 *(47)*.

As shown in Table B-4, the majority of U.S., Canadian, and European respondents listed the FWD as the primary device for deflection testing. In the United States, this may be attributed to the use of the FWD for the monitoring of Long-Term Pavement Performance General Pavement Studies test sections in the former SHRP. States were encouraged to test the road sections to monitor changes in the structural characteristics of these pavements.

*Steady-State Dynamic Devices*

Steady-state dynamic devices are defined *(41)* as those producing "a sinusoidal vibration in the pavement with a dynamic force generator." In this type of equipment, a dynamic force is superimposed on a static load (Figure 20). For the device to remain in contact with the road surface the static load must be at least twice that of the dynamic

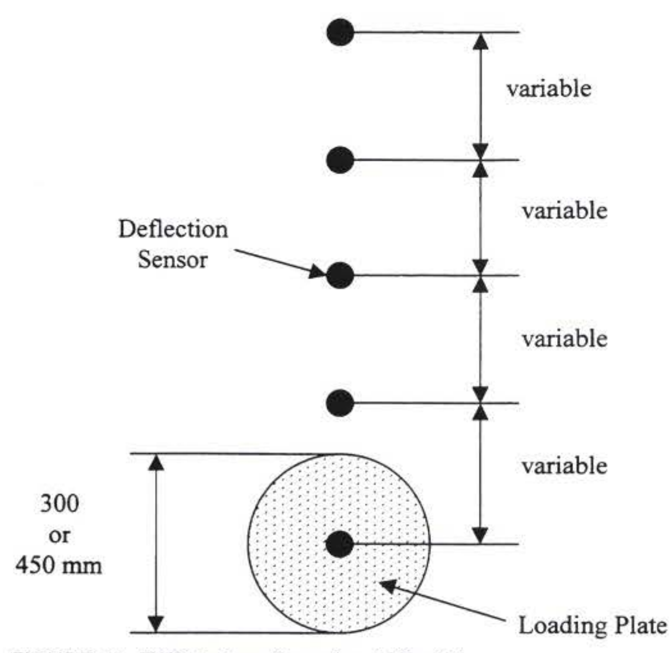



FIGURE 19 FWD test configuration *(after 41)*.

force. The use of a large static load can produce a stiffened response in the pavement structure in the presence of nonlinear materials.

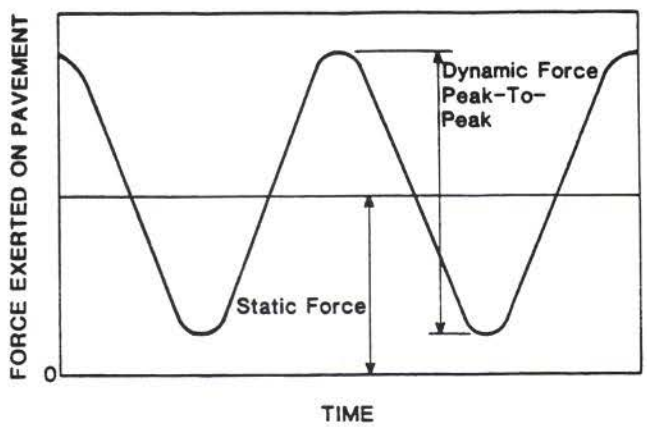



FIGURE 20 Force-time relationship for steady-state dynamic NDT devices *(41)*.

A light, trailer-mounted device *(41)* was one of the first commercially available deflection machines. The load is transmitted to the pavement by steel wheels, 100 mm (4 in.) wide by 400 mm (16 in.) in diameter, spaced 516 mm (20 in.) apart. Counter-rotating, eccentric flywheels spinning at 480 rpm (8 Hz) provide a 4448-kN (1,000-lb) peak-to-peak dynamic force. The static weight of the device is approximately 9000 kN (2,000 lb). Five geophones spaced at 305-mm (12-in.) intervals from the center of the load are used to measure surface deflections. Load and frequency may not be adjusted in this device.

A heavier and more versatile device is also available *(41)*. It may either be installed in a vehicle or towed by a trailer with a passenger vehicle or light truck. The dynamic force is produced by means of a hydraulic actuator oscillating a lead-filled, steel mass. Peak-to-peak loads may be varied from 11 kN (2,400 lb) to 26 kN (5,800 lb), depending on the model. The deflections are measured by four geophones spaced at 305-mm (12-in.) intervals starting from the center of the load. The frequency may be varied from 5 to 70 Hz for all models. The ability to vary the frequency allows for the analysis of pavement behavior with respect to testing frequency so that undamped oscillations do not result in exaggerated deflections.

The lighter device is listed along with the FWD as a primary testing device for Florida, Indiana, and Ohio, as well as in Norway. The heavier device is the primary choice for Iowa, is used in addition to the FWD in Montana and Pennsylvania, and is not used at all in the Canadian and European agencies surveyed.

*Quasi-Static Devices*

Devices in this category are those that produce pavement deflections due to a slow, rolling load. Normally, only the maximum deflection is measured with this type of loading and, thus, it is possible only to gain insight regarding the overall structural condition of the pavement.

The Benkelman beam *(41,48)* has a probe resting on the pavement surface at one end of a 3.7-m (12-ft) beam. A pivot point is located 2.4 m (8 ft) from the tip of the probe and the remaining 1.3 m (4 ft) is used to activate a dial gauge for measuring displacement. The beam assembly is shown in Figure 21. A loaded truck axle with dual tires is the mechanism used for creating the pavement deflection. Normally, the probe is centered between the rear dual tires and the truck is driven slowly away from the probe tip. A technician reads the point at which the dial gauge moves no further and records this as the rebound deflection.

The La Croix Deflectograph was developed in France and has been used in many countries *(49,50)*. It is an automated device in which a loaded truck serves to transport

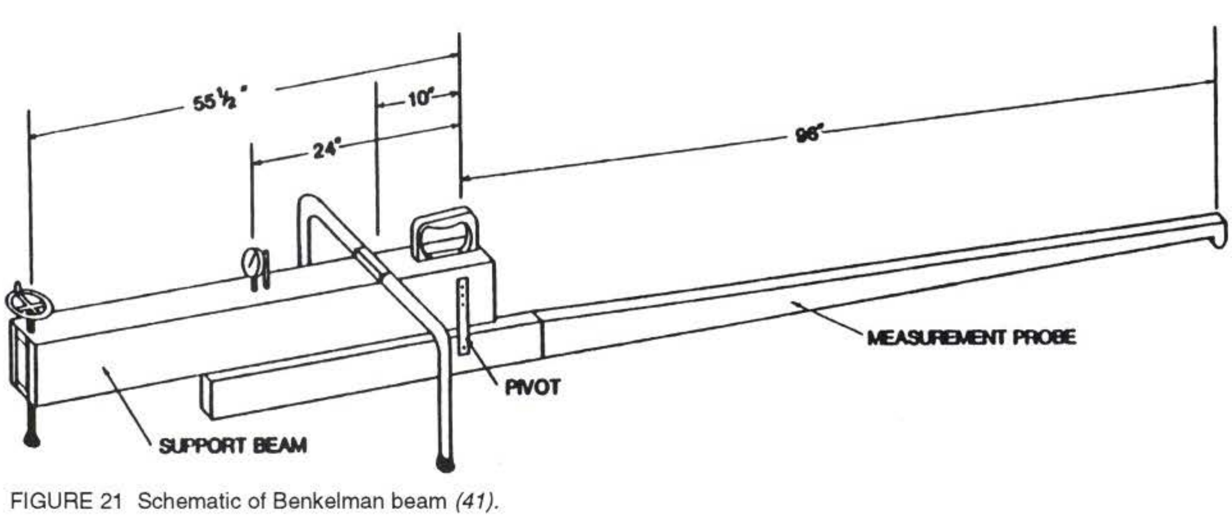



FIGURE 21 Schematic of Benkelman beam *(41)*.

the deflection-measuring device as well as provide the test load. The principle of this machine is the same as that for the Benkelman beam. The deflection measurement beam rests on the road surface between the front and rear axles of the truck. When the rear axle reaches the deflection measurement point, the maximum deflection is recorded and the measurement assembly is automatically moved forward to the next position. The contact area of the deflectograph tires is greater than that normally used for the Benkelman beam, and the axle load is approximately 18 kN (4,000 lb) less. In using the deflectograph, the pavement response to loading is measured, as opposed to the Benkelman beam with which the pavement response to unloading is measured. The response to unloading is similar to the condition in laboratory measurement of the resilient modulus, where the recoverable strain is used to calculate the modulus of the material. This distinction can be very important if the system response to loading is nonlinear, i.e., the response follows a different path upon loading and unloading.

California developed a traveling deflectometer between 1955 and 1960. It used a 67-kN (15,000-lb) axle load applied by a trailer (*3,39,49*). Deflections were measured by means of two Benkelman beam probes mounted on a frame below the trailer. The principle of operation for the California device was very similar to the La Croix device, but the equipment was significantly different.

Although the technology associated with quasi-static measurements dates back to the late 1940s, there are still a number of agencies that continue to use these devices, presumably to build on the extensive experience gained with them in pavement evaluation. The province of Manitoba and Austria and Switzerland all indicated the use of the Benkelman beam, as shown in Table B-4. Austria and Switzerland also use the La Croix Deflectograph.

*Static (Plate Bearing Test)*

The plate bearing test may be used to directly measure the modulus of subgrade reaction (*k*-value). The procedure is given in detail in AASHTO test method T 222 (*51*) and ASTM test method D 1196 (*52*), "Nonrepetitive Static Plate Load for Soils and Flexible Pavement Components for Use in Evaluation and Design of Airport and Highway Pavements." The standard test calls for the plate to be 762 mm (30 in.) in diameter by 25.4 mm (1 in.) thick. A hydraulic jack equipped with a pressure gauge is used to transfer the reaction force from a piece of heavy equipment to the plate. The deflection at the edge of the bearing plate is measured with a dial gauge as the load is incrementally increased. The dial gauge is mounted to a reference point outside the limit of influence for the applied pressure.

As shown in Table B-5, none of the U.S. or Canadian agencies surveyed indicated the use of the plate bearing test for in situ evaluation of soils. Austria and Norway did list it as a means of determining layer stiffnesses. This method is time consuming and labor intensive and therefore it is not possible to perform many tests for a given section of roadway.

*High-Speed Deflectometers*

High-speed deflectometers are defined here as instruments capable of measuring deflections while continuously moving. Other methods discussed to this point have required that the equipment remain stationary while deflection measurements are made. The advantages of high-speed measurements in terms of safety, productivity, and volume of information are very evident. However, there is an increase in the complexity of the measurements as well as in the interpretation of the results.

One such device currently in use in France and Belgium is the French Curviameter (*53,54*). It is mounted on a two-axle truck with a dual-wheel rear axle. Three geophones are spaced at 5-m intervals on a chain track that passes under the rear dual tires. The movement of the chain is synchronized with the speed of the truck so that the geophones are not damaged. Beginning at a distance of 4 m from the center of the dual tires, the deflection is measured 100 times until the geophone passes under the tires. The monitoring speed is 18 km/hr, and up to 3,000 deflection basins can be measured every hour. The sensitivity of the geophones is 0.02 mm. The French Curviameter has been used in Belgium to evaluate the structural integrity of pavements, delineate sections according to pavement and soil conditions, isolate problem soil areas, provide feedback for pavement management studies, and provide data for pavement rehabilitation.

RWD are being developed in the United States (*55,56*) and other countries (*57,58*). The U.S. device described by Bay and co-workers (55) is mounted on a three-axle military vehicle. As the vehicle moves along the pavement at 3 to 6 km/hr, a large vibrator produces a sinusoidal vertical dynamic loading pattern at 20 to 40 Hz superimposed on a static load. This load is transmitted to the pavement through an isolated dual wheel in the center of the vehicle. An accelerometer is mounted on this wheel and records the deflections due to the sinusoidal load. According to Bay et al. (56), the rolling dynamic deflectometer can be used to show variations in pavement stiffness, locate areas of discontinuity, and identify areas to be further characterized.

The Swedish version of the RWD, called a road deflection tester (RDT), has been under development for a number of years (*57,58*). This device is a two-axle vehicle with a rear axle equipped with super-single tires. Deflections are measured with 40 laser sensors, and these operate at either 16 or 32 kHz with a vertical resolution of 0.032 to 0.064 mm. A reference measurement of the unloaded road surface is taken 6 m away from the load. The RDT is capable of measuring deflections at speeds of up to 90 km/hr, although 72 km/hr is recommended for a smooth road and a range of 10 to 15 km/hr is recommended for rougher roads.

#### Interpretation of Deflection Measurements

The following parameters and properties have been most frequently used by the survey respondents to interpret the results of deflection tests:

- Maximum deflection (15 states, 3 Canadian provinces, and all 6 European countries);
- Subgrade modulus (20 states, 1 Canadian province, and 2 European countries);
- "AREA" (10 states);
- Surface curvature index (2 states and 2 European countries);
- Modulus of subgrade reaction (7 states);
- Backcalculated layer moduli (22 states, 3 Canadian provinces, and 3 European countries).

*Deflection Parameters*

The center-of-load deflection represents the total deflection of the pavement and thus is indicative of the total stiffness of the pavement section including the subgrade. This was the first deflection parameter used with the Benkelman beam. It is used as an input to overlay design procedures such as those of the Asphalt Institute (*59*) and California (*3*). Often the maximum deflection measured by dynamic load devices such as the FWD is correlated to deflection measurements from the Benkelman beam. For example, in Washington State (*31*), the relationship was found to be

$$BB = 1.33269 + 0.93748(FWD) \tag{14}$$

where

$BB$ = Benkelman beam deflection, 0.025 mm ($10^{-3}$ in.); and
$FWD$ = FWD maximum deflection under a 40-kN (9,000-lb) load, 0.025 mm ($10^{-3}$ in.).

A considerable amount of scatter has been noted for this equation; however, the correlation is seen as important for building on a database that spans a number of decades.

The subgrade modulus may be calculated directly by using deflection measurements outside of the influence of the pavement structure. Figure 22 illustrates that surface deflections outside the distance $a_e$ are directly attributable to the subgrade stiffness. For deflections occurring outside this distance, the subgrade modulus may be calculated according to the formula

$$M_R = \frac{P(1-\mu^2)}{(\pi)(D_r)(r)} \tag{15}$$

where

$M_R$ = subgrade resilient modulus,
$P$ = applied load, and
$D_r$ = pavement surface deflection at a distance $r$ from the center of the load

This is the procedure recommended by the 1993 AASHTO pavement design guide (*5*) for estimating in situ subgrade modulus and it is the procedure embedded in the program DARWIN, which is used by a number of states. This relatively simple equation is useful for: (1) studying the variability of subgrade stiffness, (2) delineating different pavement sections, and (3) obtaining a design value for subgrade modulus. For the last item, the 1993 AASHTO guide (*5*) recommends the application of a correction factor to obtain a modulus value representative of those used to develop the AASHTO design equation. Illinois uses this type of an approach for estimating subgrade modulus from deflection measurements at 915 mm (36 in.) from the center of the loading plate, and Virginia uses the deflection at 1220 mm (48 in.). The equations used by Illinois and Virginia are selected depending on the type or stiffness of pavement structure. Florida uses a correlation between the deflection at 1220 mm (48 in.) and the plate bearing modulus

$$\log P_b = 4.0419 + [0.5523 (\log D_4)] \tag{16}$$

where

$P_b$ = plate bearing modulus, lb/in$^2$/in.; and
$D_4$ = deflection at 1220 mm (48 in.), in.

The AREA parameter was introduced by researchers at the University of Illinois (*44*) as a means of quantifying the relative stiffness of a pavement section. It is computed by

$$\text{"AREA"} = 6(1 + \frac{2D_1}{D_0} + \frac{2D_2}{D_0} + \frac{D_3}{D_0}) \tag{17}$$

where

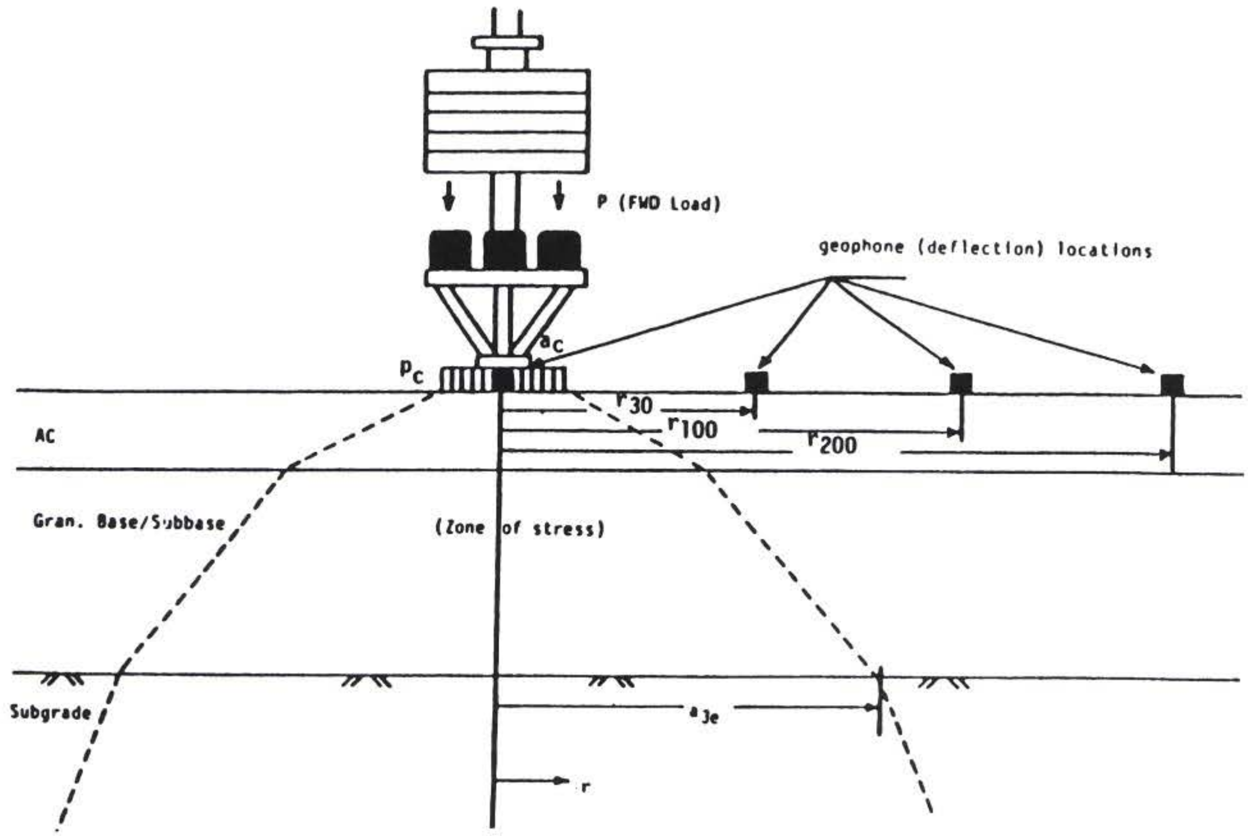



FIGURE 22 Determination of pavement deflections due only to subgrade modulus *(5)*.

$D_1$ = deflection at 305 mm (12 in.), in.;
$D_2$ = deflection at 610 mm (24 in.), in.; and
$D_3$ = deflection at 915 mm (36 in.), in.

The AREA value is the normalized area of a slice taken through any deflection basin between the center of the loaded area and 915 mm (36 in.). This area is said to be normalized because it is divided by the maximum deflection, $D_0$. The maximum value of the AREA parameter is 915 mm (36 in.) and it occurs when all four deflection values are equal. This would result from testing an extremely rigid section of pavement. The minimum AREA is 280 mm (11.02 in.), which would result from deflection measurements on a one-layer system of homogeneous material. This would imply that the pavement structure is of the same stiffness as the underlying soil. The Corps of Engineers Cold Regions Research and Engineering Laboratory has found this parameter useful in monitoring pavement conditions during spring thaw.

The state of Washington uses a combination of the maximum deflection, subgrade modulus, and AREA for project evaluation prior to rehabilitation *(31)*. By examining these parameters it is possible to distinguish sections in need of structural upgrading from those that are structurally competent.

The surface curvature index is typically the difference between the maximum deflection at the center of the load and the deflection at 305 mm (12 in.) or 508 mm (20 in.) away from the load *(60)*. It is used as an indication of the relative stiffness of the bound layers or upper regions of the pavement section.

The modulus of subgrade reaction may be estimated from deflection testing using the procedure outlined in the 1993 AASHTO pavement design guide *(5)*.

*Backcalculation of Moduli*

Many of the respondents to the questionnaire indicated the use of backcalculation in analyzing deflection data from the FWD and Road Rater. There are three general techniques into which these methods may be grouped.

1. There are traditional backcalculation techniques that match measured deflections against those calculated from theory. Some of the more commonly listed were EVERCALC *(61)*, MODCOMP *(62)*, and WESDEF *(63)*.
2. A pattern search technique is employed in MODULUS *(64)* to obtain a match between measured and calculated deflections.

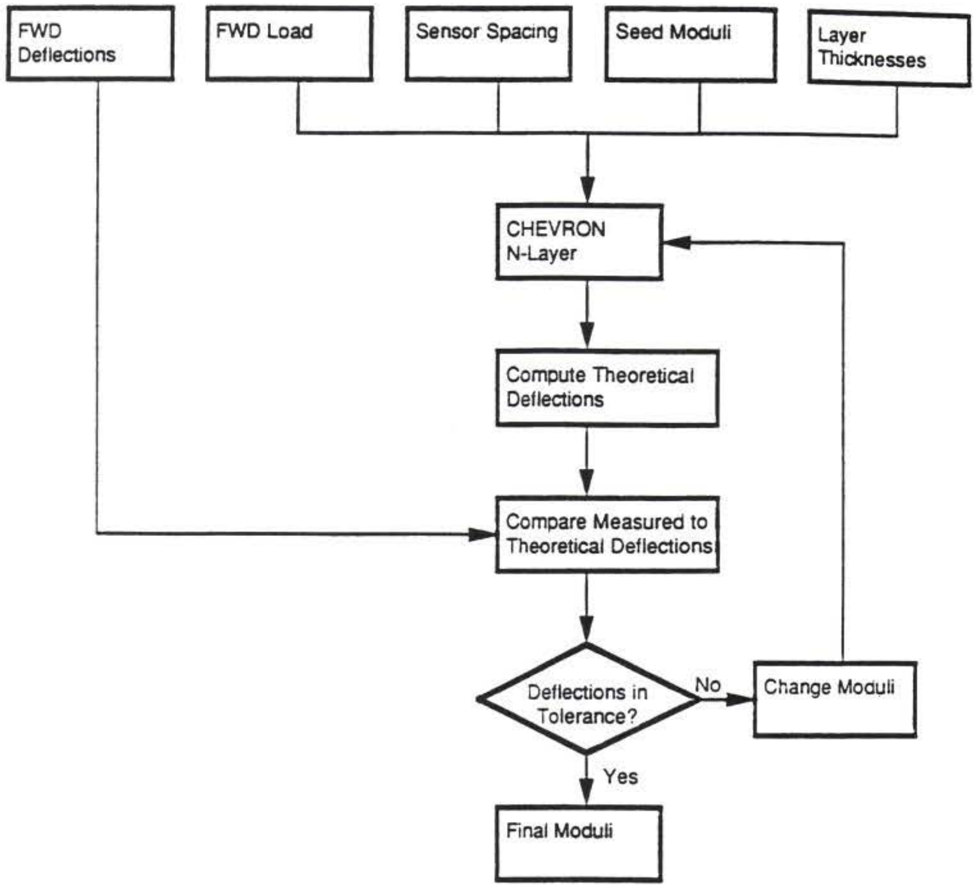



FIGURE 23 Simplified flow chart for EVERCALC *(61)*.

3. BOUSDEF *(65)* and ELMOD *(66)* are examples of techniques based on an equivalent layer method.

The traditional backcalculation techniques use the deflection test conditions (i.e., load, plate geometry, layer thicknesses) and estimated layer moduli to generate a theoretical deflection basin. The theoretical deflections are compared with the measured deflections, and the error is computed. If the error is not within a specified tolerance, the process is repeated with new layer modulus values until the two deflection basins are considered to be sufficiently close or until the modulus value for any given layer reaches a given limit. This procedure is shown in Figure 23. The computer program EVERCALC *(61)* was developed by the University of Washington for the Washington State DOT. MODCOMP *(62)* was created at Cornell University and WESDEF *(63)* was developed by the U.S. Army Corps of Engineers Waterways Experiment Station.

The MODULUS program *(64)* was developed at Texas A&M University for the Texas Department of Highways and Public Transportation. It uses a layered elastic computer program called WES5 to generate a database of deflection basins for a range of layer moduli. A pattern search method and interpolation are employed to minimize the error between the measured and calculated deflection basins. MODULUS will perform the detection of nonlinear subgrade behavior and select the optimum number of sensors to use in backcalculation. It also contains a default database for common Texas pavement sections.

BOUSDEF *(65)*, developed at Oregon State University, and ELMOD *(66)*, a product of Dynatest, Inc. (Ojai, Calif.), are based on the principle of equivalent thickness and Boussinesq theory. In this approach, the pavement cross section is converted into one layer of the same stiffness as the subgrade in order to arrive at a thickness giving a comparable deflection. This concept is illustrated in Figure 24. This approach simplifies the calculation of theoretical deflections in the basin, making the process very quick. The greatest limitation to this approach is that the layer moduli should decrease with depth, preferably by a factor of two or more between consecutive layers. Also, the computed thickness of the equivalent layer should be greater than the radius of the loaded area.

Although backcalculation is widely used, techniques for obtaining reasonable modulus values for various layers still require the application of judgment. In most instances, it is advantageous not to backcalculate the moduli of more than three or four layers. Sometimes this entails fixing the modulus of one or more layers. Especially

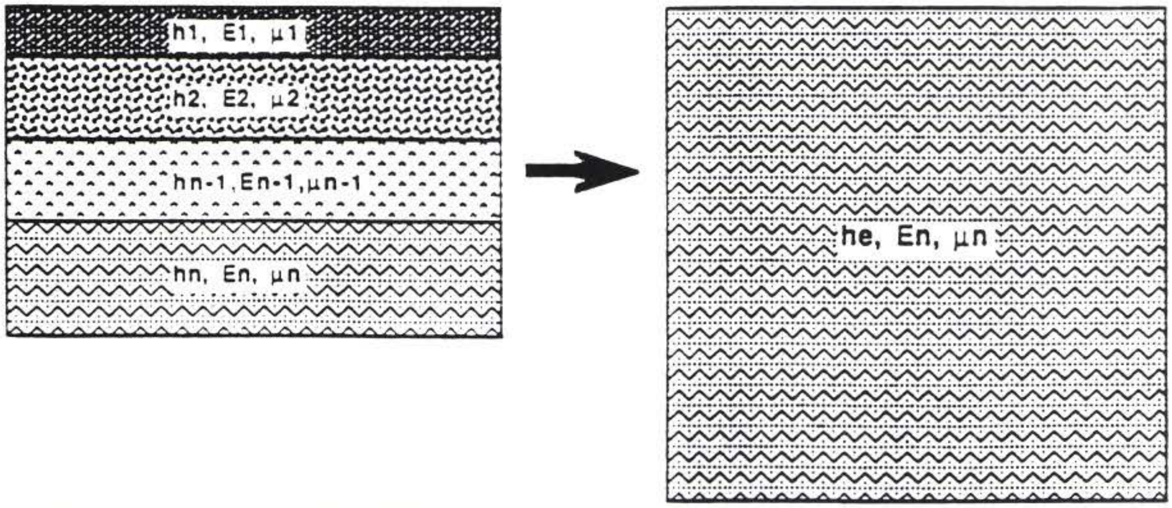



FIGURE 24 Concept of equivalent thickness of a pavement structure *(65)*.

problematic is the task of moduli backcalculation during the spring thaw period. Obtaining reasonable answers in this condition requires detailed knowledge of the depth of thaw at the time of deflection testing. Another condition leading to problems in backcalculation is the presence of a water table or rigid layer at some unknown depth. In this instance a procedure developed by Rohde *(67)* can be used to estimate the depth to a stiff layer to aid in the backcalculation process. Finally, it is also difficult to backcalculate the modulus of a relatively thin layer that does not contribute significantly to the overall stiffness of the system.

## SMALL-LOAD METHODS

In this category of tests, a very light impulse or sinusoidal load is imparted to the pavement or soil, and the response in terms of wave velocities is measured and analyzed. The analysis may be as simple as examining the variability of deflections or as complicated as determining the shear or elastic modulus of one or more layers. In the following, the Clegg Impact Soil Tester, the Finnish Loadman, the SASW, and the Humboldt Stiffness Gauge will be reviewed. The review will emphasize the methodology of testing, as well as the information obtained from the testing.

### Clegg Impact Soil Tester

This device is comprised of a 4.5-kg drop hammer suspended at a height of 450 mm inside a plastic tube. The tube is brought in contact with the soil surface, and the weight is released. The deceleration of the hammer is measured as contact is made and maximum deceleration is converted directly into the so-called Clegg Impact Value. This value may then be related to other parameters such as the CBR, soil modulus, or relative density.

### Finnish Loadman

The Loadman is a relatively new portable device for the measurement of deflections. The principle behind the Loadman is to measure with an accelerometer the deflection caused by the load of a falling weight. Figure 25 shows a diagram of the Loadman. The reported results are the maximum deflection, the modulus, the length of loading impulse, and the percentage of the rebound deflection compared with the maximum deflection, as well as the ratio of the modulus of the second measurement compared with the modulus of the first measurement.

The current methods of interpretation of Loadman testing results are based on the assumption that the modulus needed to cause the maximum measured deflection under a circular load can be found by relating the maximum deflection caused by a uniformly applied pressure to the elastic stresses as calculated by Boussinesq's theory in the underlying pavement and soil. For simplicity, Poisson's ratio is typically assumed to be $\nu = 0.5$, denoting incompressible material. Thus, the resulting modulus ($E$) can be found as *(68)*

$$E = 1.5\ (pa/\Delta) \qquad (18)$$

where

$p$ = uniformly applied pressure,
$a$ = radius of the plate, and
$\Delta$ = measured maximum deflection under the circular plate.

This approach assumes that the stresses in the soil are relatively insensitive to the modulus values and that the soil is homogeneous and isotropic throughout.

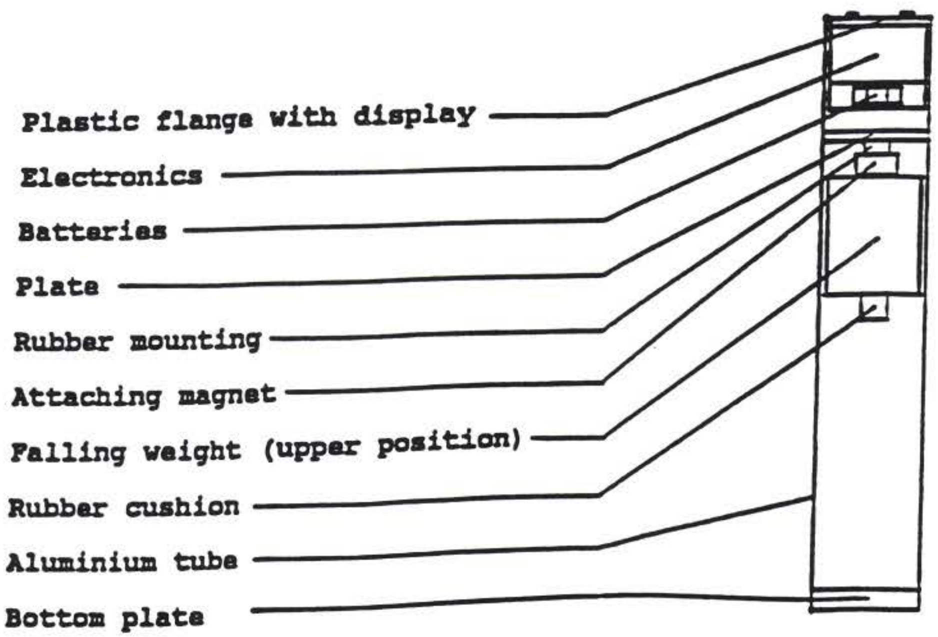



FIGURE 25 Schematic diagram of Loadman *(68)*.

The Loadman's structure consists of an aluminum tube (132 mm diameter) containing a free moving 10 kg steel weight. A circular plate of 132 mm diameter is attached to the base of the tube (the loading surface) with controls, electronics, accelerometer, and weight support magnet positioned at the top.

The Loadman is operated by placing its base on the surface of the soil or pavement layer and then activating the electronics, which drop the weight and record all the relevant data. After each test the Loadman has to be reset by upending it, so that the weight can be returned to its original position.

Gros *(68)* performed a detailed field study to compare results obtained from the Loadman, the FWD, and the plate-loading test. The findings indicate that for the subgrade soils, the results obtained from the Loadman correlated reasonably well with those obtained from the FWD, as was the case between the Loadman and the plate-loading test, with an average correlation coefficient of 0.77. Furthermore, the results showed that the Loadman modulus values were always lower than those of the FWD or the plate-loading test. In summary, Gros *(68)* concluded that the modulus values obtained by the Loadman are almost equal to the plate-loading test values and equal, except for a shift, to FWD values. However, the Loadman is still a relatively new test with a limited database of experience. As more experience is gathered from various different sites from around the world it will become easier to evaluate the Loadman. Improvements in the theory behind the test may also lead to better predictions. The introduction of layered materials, rather than the homogeneous half-space represented by Boussinesq's theory and the allowance for Poisson's ratios different from 0.5, may significantly improve the use of the Loadman.

#### Spectral Analysis of Surface Waves

In an infinite, homogeneous, isotropic, linear elastic soil two types of body waves may occur, namely, compression and shear waves. The velocities of these two types of waves depend on the elastic properties, density, and moisture content of the soil. From these wave velocities, the modulus may be obtained from *(69)*

$$E = (1-\nu^2)/(1-\nu)\ c_l\ c_s\ \rho \tag{19}$$

or

$$E = 2(1+\nu)\ c^2_s \tag{20}$$

where

$c_l$ = compressional wave velocity,
$c_s$ = shear wave velocity, and
$\rho$ = soil density.

Near the soil surface the conditions are more complicated because of reflection away from the surface interfering with the impinging body waves. This inference of waves leads to another type of wave termed the Rayleigh wave or simply the surface wave. The Rayleigh wave travels along the surface at a wave velocity, $c_R$, which is less than both the compressional and shear wave velocity of the soil.

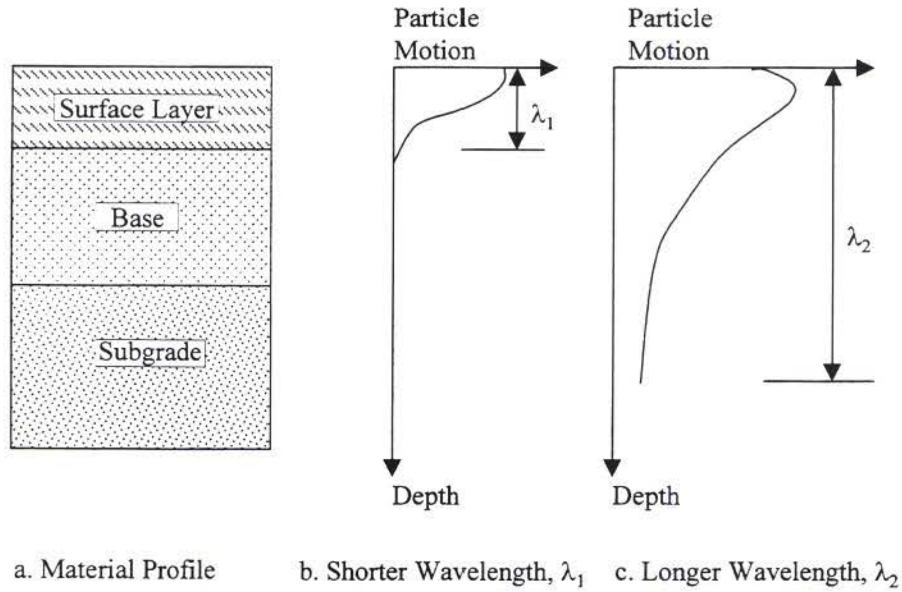



FIGURE 26 Schematic of vertical particle motion for different surface wavelengths *(after 74).*

In the late 1950s and 1960s, a number of methods were developed to determine the layer thicknesses and elastic parameters of pavement structures, including the subgrade. One method was developed by Jones *(70,71)*, and this was followed by another method developed by the Shell Oil Company in Amsterdam *(72)*. They found that the velocity measured on the surface corresponded to the characteristics of the material at a depth roughly equal to one-half of the wavelength. Later experiments by Szendrei and Fremme *(73)* showed that this depth was probably closer to one-third of the wavelength. As the wavelength increases, the particle motion in the soil extends to greater depths in the profile (Figure 26). The velocities of surface waves are representative of the material properties over depths where there is significant particle motion.

These observations lead to the development of the SASW technique, which may be used to determine layer moduli and layer thicknesses through an inverse analysis. The SASW method is a nondestructive seismic method that uses the dispersive property of Rayleigh waves to determine the shear modulus profile at soil sites *(74)*. The objective in SASW testing is to make field measurements of surface wave dispersion (i.e., measurements of surface wave velocity at various wavelengths) at soil sites and then to determine the shear wave velocities of the layers in the profile. These velocities can, in turn, be used to calculate modulus values as given by Eqs. (19 and 20). Figure 27 shows a typical configuration of equipment used in SASW field testing. It should be noted that the surface waves in the SASW method are generated by either using a hammer (or different weight hammers) or a piezoelectric generator. In both cases the strain level is very low compared with the strains induced by heavy traffic loading *(69)*. In this context, the modulus values calculated from the SASW are typically for a range of axial strains less than 0.001 percent.

A frequently expressed concern with the SASW is the lack of ability to scale up the SASW backcalculated low-strain moduli to the strain levels associated with actual loading conditions. Rix and Stokoe *(74)* studied the nonlinear behavior of a silty clay subgrade and suggested the following relationship for determining the field modulus ($E_{\varepsilon,\text{field}}$) at any strain amplitude ($\varepsilon$) desired by the engineer

$$E_{\varepsilon,\text{field}} = E_{\text{seismic}}\,(E_\varepsilon/E_{\max})_{\text{lab}} \tag{21}$$

where

$E_{\text{seismic}}$ = small-strain modulus determined in situ with the SASW,
$(E_\varepsilon/E_{\max})_{\text{lab}}$ = normalized modulus determined by cyclic laboratory test at a strain amplitude of $\varepsilon$ (determined from torsional resonant column testing), and
$E_{\varepsilon,\text{field}}$ = modulus in the field at a strain amplitude of $\varepsilon$.

This general approach follows that used in earthquake engineering to evaluate the nonlinear soil response during an earthquake *(75)*.

The SASW is a particularly useful tool for identifying material layer boundaries or thicknesses. However, the time for testing and interpretation of data is still somewhat long for routine applications in pavement systems.

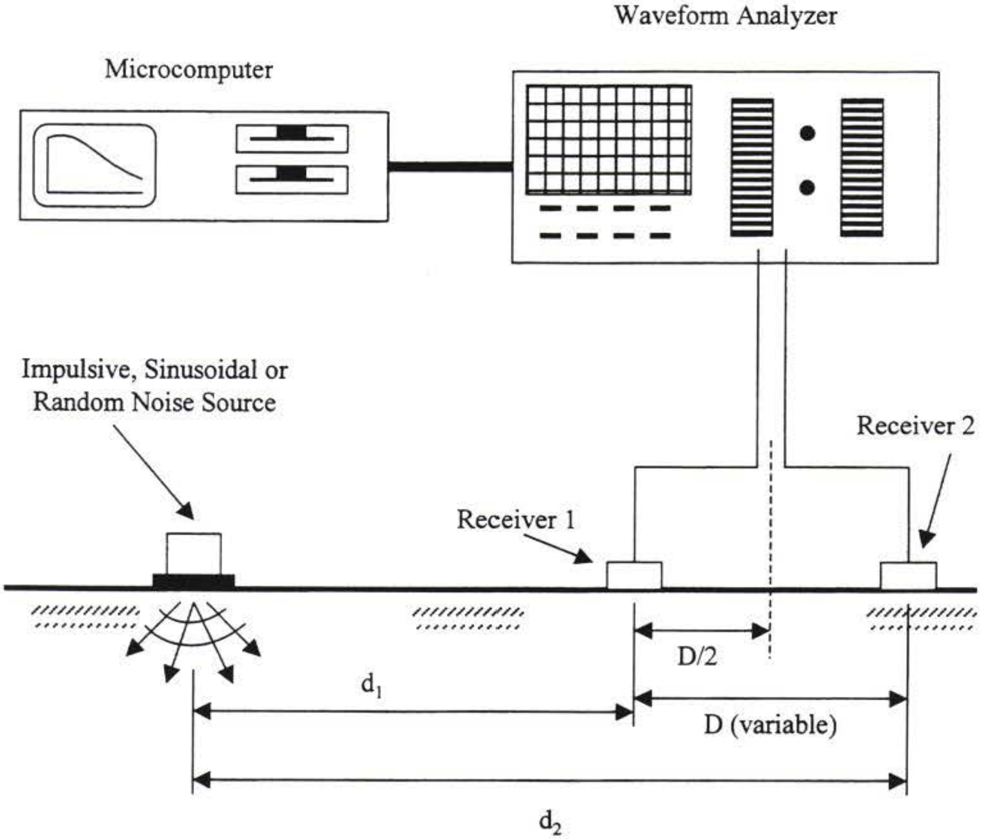


FIGURE 27 SASW equipment configuration *(after 73).*

$$G = \frac{K(1-\upsilon)}{4a} \qquad G = \frac{C_1\sigma(p)}{0.3+0.7e^2} \qquad \rho_D = \frac{\rho_0}{1+e}$$

$\upsilon$ = Poisson's ratio
a = foot radius
e = void ratio
$\sigma$ = overburden stress
p = typically 0.5 to 0.25
$\rho_0$ = density with no voids
$\rho_D$ = actual density
G = shear modulus
$C_1$ = function of moisture and soil type

FIGURE 28 Relationship between modulus and density *(after 76).*

Although not widely used by the survey respondents, the SASW technique was listed by Florida and New Jersey as a research tool, as indicated in Table B-5.

#### Soil Stiffness Gauge

The Soil Stiffness Gauge (SSG) (*76*) is a recently developed nondestructive testing device that measures the in-place stiffness of compacted soils at the rate of about one test per minute. The SSG weighs about 11.4 kg, is 28 cm in diameter, 25.4 cm tall, and rests on the soil surface via a ring-shaped foot. Resting on the surface, the SSG produces a vibrating force that is measured by sensors that record the force and displacement-time history of the foot. The theoretical basis for the SSG is that the stiffness of soil can be expressed as the ratio of applied force to measured displacement, i.e., $K = P/\Delta$. As shown with the formulas in Figure 28, the SSG can be viewed as the dynamic equivalent to the plate-loading test, with the exception that the induced strains in the soil are smaller. In both cases, a force $P$ is applied to the soil by means of a plate or ring. The soil deflects an amount, $d$, which is proportional to the foot geometry, the modulus, and Poisson's ratio of the soil. In plate-loading tests, large forces are necessary to produce adequate deflection to measure. On the other hand, the SSG uses technology borrowed from the defense industry to measure very small deflections, allowing much smaller loads, but also restricting the range of loads and induced strains to the lower end of the spectrum. Rather than measuring the deflection resulting from the SSG weight directly, the SSG is vibrated, producing small changes in $P$ that produce small deflections. To filter out the deflections resulting from equipment operating nearby, the SSG is used over a range of frequencies.

Currently, the SSG is still in a developmental stage and is being tested and evaluated by the Federal Highway Administration and the Minnesota, New York, and Texas DOTs. Preliminary results (*76*) indicate that the SSG may be a promising alternative to the nuclear density gauge for evaluating the compaction of constructed embankments. The only potential disadvantage with the SSG is the low-strain level at which the soil is tested. This test would be significantly enhanced if a correlation was established between the measured low-strain moduli and those associated with actual loading conditions at the higher strain levels.

### INTRUSIVE METHODS

#### Dynamic Cone Penetrometer

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. The DCP was originally formulated and developed in South Africa (*77,78*) for estimating the in situ soil and pavement subgrade strengths. The DCP consists of a cone attached to a rod that is driven into soil by the means of a drop hammer that slides along the penetrometer shaft. The mass of the hammer is typically 8 kg, and the drop height is 575 mm. Early versions of the DCP had a cone angle of 30 degrees with a diameter of 20 mm. More recent versions of the DCP include a cone angle of 60 degrees and the option to use a 4.6-kg hammer for weaker soils (*79*). The ASTM is currently working to standardize this test.

Because the relationship between soil strength and the DCP cone penetration resistance does not rest on a theoretical framework, the link between the strength and cone resistance is obtained through empirical correlations. The most common correlations for estimating subgrade soil strength are often in the form of equations for CBR as a function of the DCP Penetration Index (DPI), defined as penetration in millimeters per blow. A number of researchers have developed empirical relationships relating soil strength parameters to DPI (*78,80,81*). One of the most widely used correlations in the United States is that developed by Webster et al. (*79*)

$$CBR = 292/(DPI)^{1.12} \tag{22}$$

This empirical relationship was developed for the manual DCP, and Webster et al. (*82*) recently updated the equation for heavy and lean clays (Figure 29). A variation of this relationship, used in Norway is

$$\log CBR = 2.57 - 1.25 \log DPI \tag{23}$$

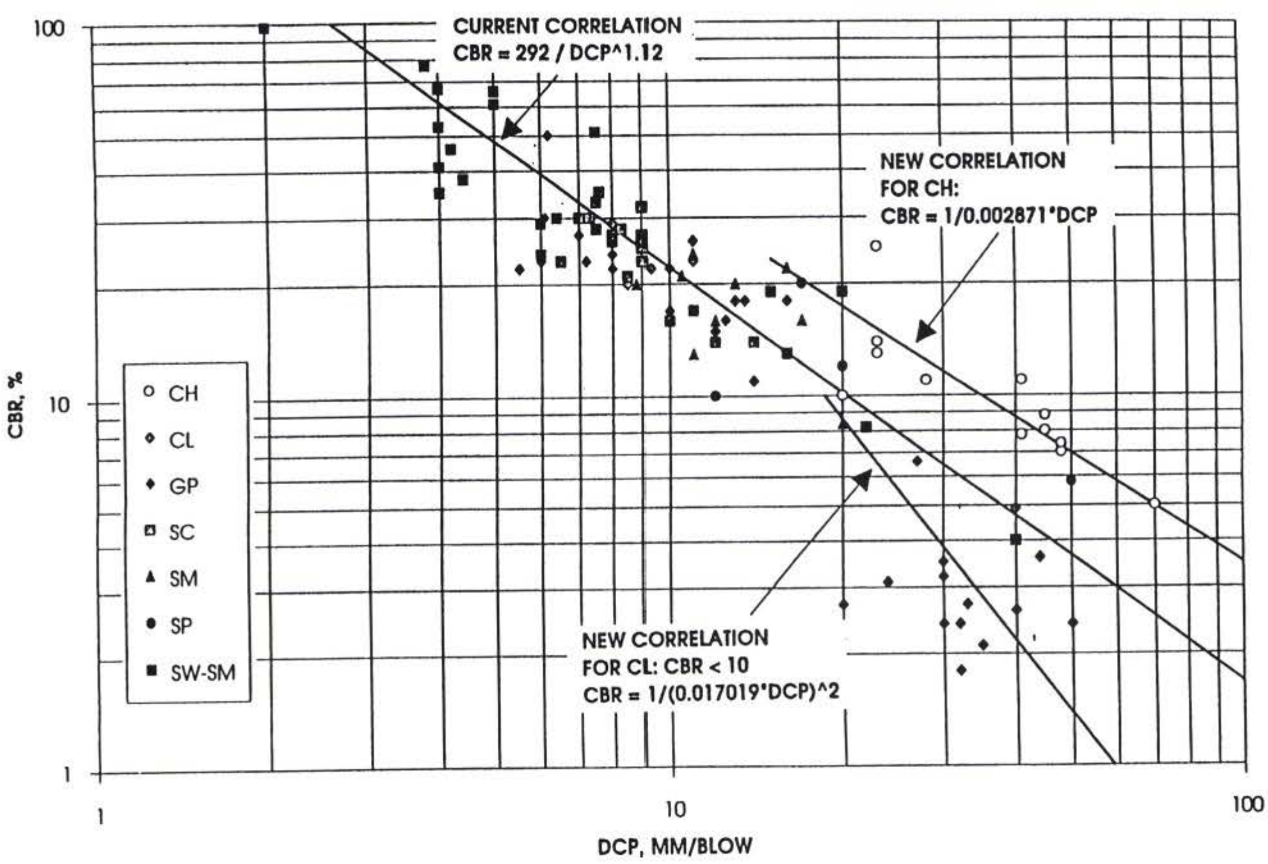



FIGURE 29 Correlation between DPI and CBR *(82)*.

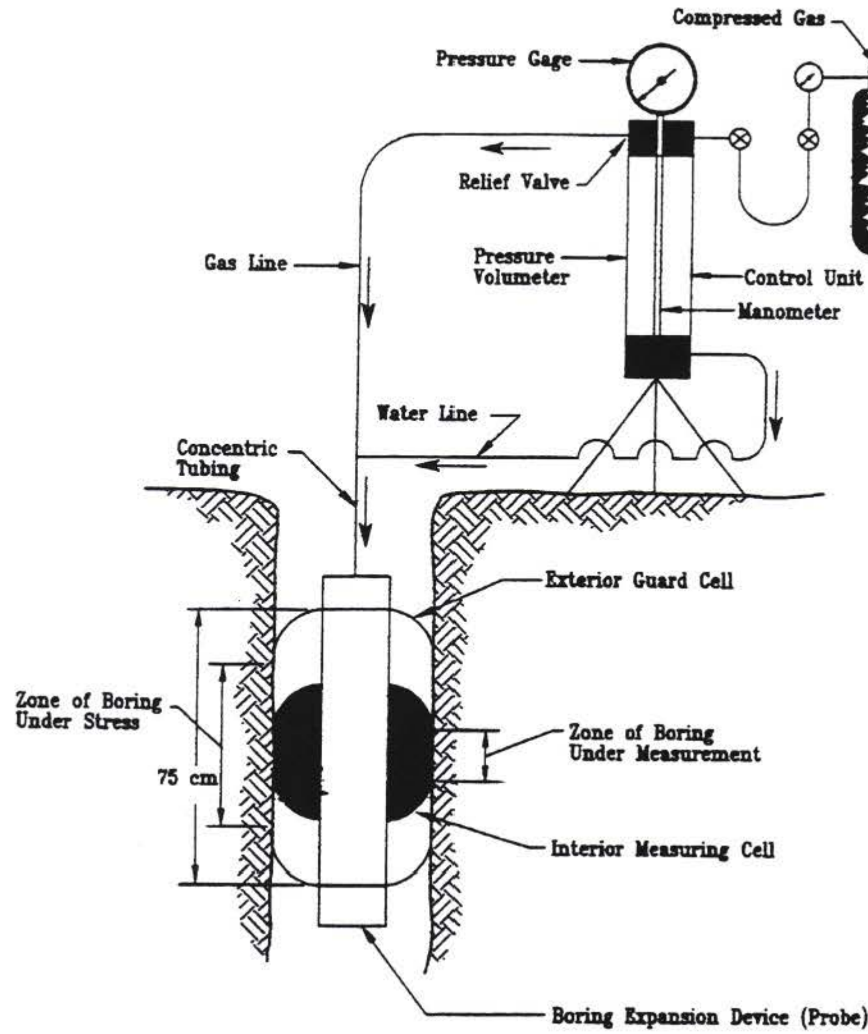



FIGURE 30 Components of Menard pressuremeter *(85)*.

Recently, however, some DCPs have been developed (*81*), in which the hammer is picked up and released automatically. The results indicate that the CBR values obtained from the Israeli automated DCP are about 15 percent greater than CBR values computed using DPI from the manual DCP.

As shown in Table B-5, nine state agencies (Florida, Illinois, Kansas, Minnesota, North Carolina, New Jersey, Ohio, Pennsylvania, and Utah) responding to the survey indicated use of the DCP. The Canadian province of New Brunswick and Belgium and Norway also use the DCP. For the most part, these agencies stated that they use the DCP to check soil stiffness and compaction uniformity. A major drawback with the DCP is the lack of well-defined boundary conditions during testing. This limits the development of a relationship between the DPI and soil strength and stiffness to empirical relationships that are dependent on local soil conditions. However, the relative simplicity and expedience of testing continue to make the DCP a very attractive way of characterizing subgrade soils for many agencies worldwide.

#### Borehole Pressuremeter

The borehole PMT was developed in France in 1956. Although none of the surveyed agencies listed this as a standard subgrade test, it has been used widely in France and much of the rest of the world (*83–85*).

Pressuremeter testing is conducted in a carefully prepared borehole (*86*), which is about 10 percent oversized. A pressuremeter probe is then inserted into the hole and expanded into the soil. Figure 30 shows the components of the pressuremeter during a test. ASTM D 4719 presents the procedures for a Menard-type PMT. Additional guidelines to be followed during testing are presented by the manufacturer of the instrument (*87*). Briaud (*88*) also discusses the PMT in detail. The parameters obtained from the PMT include undrained shear strength ($s_u$), coefficient of *lateral* earth pressure at rest ($K_o$), and tangent ($E_t$) and secant ($E_s$) soil moduli.

The theoretical basis for the PMT is the radial expansion of cavity in an infinite linear elastic medium (*89*). Details of the use of cavity expansion theory for the interpretation of PMT results can be found in Baguelin et al. (*90*) and Mair and Wood (*91*). Based on cavity expansion theory (*90*), the undrained shear strength ($s_u$) can be evaluated from

$$s_u = (p_L - p_o)/N_p \tag{24}$$

where

$p_L$ = PMT limit pressure,
$p_o$ = PMT total horizontal pressure,
$N_p$ = $1 + \ln(E_{PMT}/3s_u)$, and
$E_{PMT}$ = PMT modulus.

Values of $N_p$ may range from 2 to 20 (*91*), but typical values range from 5 to 12, with an average of 8.5 (*92*). Because the pressuremeter provides a direct measurement of the horizontal modulus of soils, the $E_{PMT}$ is often assumed to be roughly equivalent to the Young's modulus when performing an elastic analysis in which the layers are considered as beams on an elastic foundation. The ratio of the pressuremeter modulus $E_{PMT}$ to the limit pressure $p_L$ also tends to be a constant that is characteristic of any given soil type. Typical values are shown in Table 5.

TABLE 5
TYPICAL PRESSUREMETER VALUES (*92*)

| Type of Soil | Limit Pressure (kPa) | $E_m/p_1$ |
|---|---|---|
| Soft clay | 50–300 | 10 |
| Firm clay | 300–800 | 10 |
| Stiff clay | 600–2,500 | 15 |
| Loose silty clay | 100–500 | 5 |
| Silt | 200–1,500 | 8 |
| Sand and gravel | 1,200–5,000 | 7 |
| Till | 1,000–5,000 | 8 |
| Old fill | 400–1,000 | 12 |
| Recent fill | 50–300 | 12 |

The PMT is best suited for the testing of specific soil properties, but not as a logging tool. Therefore, the soil must be characterized in advance for optimal use of the PMT results. Among the most attractive features of the pressuremeter is its ability to provide reasonable estimates of the in situ horizontal stress. The PMT is also capable of yielding data on soil modulus and shear strength on a fairly routine basis once the subgrade soil deposits of interest for further testing have been identified. Test accuracy is also subject to drilling procedures, insertion techniques, proper instrument calibration, and the theory used for interpretation. This rather sophisticated nature of the PMT requires the presence of trained personnel during testing and test interpretation.

#### Standard Penetration Test

The SPT was developed in 1927 (*83*) and is one of the oldest soil testing methods. Currently, the SPT is the most common in situ geotechnical test in the world (*93*), because of the abundance of experience over the years and its relative simplicity and cost effectiveness.

The detailed procedure for the SPT is described in ASTM D 1586, and a complete analysis of the statics and dynamics of the SPT is given by Schmertmann (*94,95*). Although the SPT can be performed in a wide variety of soils, the most consistent results are found in sandy soils where large gravel particles are absent.

The basis for the SPT is that the blow count per foot (*N*-value), as a standard split spoon sampler is driven into the ground, is correlated with the relative density, the unit weight, the angle of internal friction, the undrained shear strength, and the elastic modulus of soils (*92*). Because of all the potential sources of error associated with the SPT, many SPT correlations have a large scatter, leading to the recommendation that the SPT not be used alone for design purposes. A comprehensive review of SPT correlations is beyond the scope of this study, but can be found in Kulhawy and Mayne (*92*). However, some of the more common correlations with the angle of internal friction, undrained shear strength, and modulus will be reviewed briefly in the following.

For coarse-grained soils, the relationship between the SPT *N*-value and the angle of internal friction determined in triaxial compression tests can be written as (*92*)

$$\phi \approx \tan^{-1}[N/(12.2 + 20.3\,\sigma'_{vo}/p_a)]^{0.34} \tag{25}$$

in which $\sigma'_{vo}$ = effective vertical overburden stress, and $p_a$ = atmospheric pressure. These results should not be used at very shallow depths of less than 1 to 2 m. The inclusion of atmospheric pressure is intended to make the stress term in this equation dimensionless and thus independent of the units of measure. Peck et al. (*96*) also present the relationship between the friction angle $\phi$ and the SPT *N*-value in the chart shown in Figure 31.

Similarly, Wroth et al. (*97*) reviewed a number of relationships for fine-grained soils between the small strain shear modulus, $G_{max}$, and the SPT *N*-value. Despite considerable scatter in the data, they suggested the following relationship

$$G_{max}/p_a = 120\,N^{0.77} \tag{26}$$

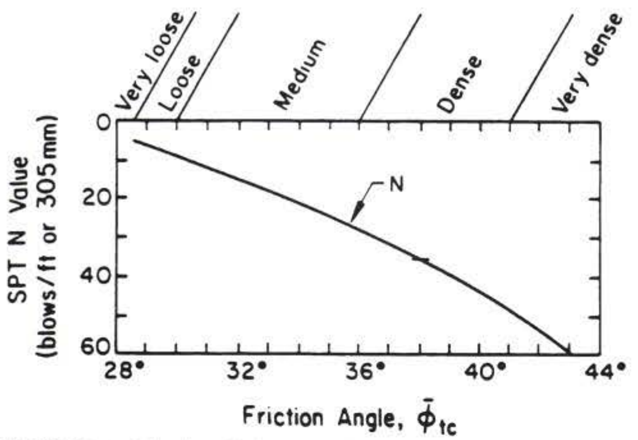



FIGURE 31 Relationship between SPT $N$-value and friction angle *(96)*.

with limits of the data being $60\ N^{0.71} < G_{max}/p_a < 300\ N^{0.8}$. Consequently, the static shear modulus at higher strains would then be some 5 to 10 percent of the computed $G_{max}$ value *(92)*.

For fine-grained soils, the relationship between the undrained shear strength and the SPT $N$-value has been characterized by Terzaghi and Peck *(98)* and is shown in Table 6. These relationships were developed primarily using unconfined compression tests. Kulhawy and Mayne *(92)* approximated the relationship in Table 6 as follows

$$s_u / p_a \approx 0.06N \qquad (27)$$

TABLE 6

APPROXIMATE $s_u$ - $N$ RELATIONSHIP *(92)*

| $N$ Value (blows/m) | Consistency | Approximate $s_u/p_a$ |
|---|---|---|
| 0–6 | Very soft | < 1/8 |
| 6–12 | Soft | 1/8–1/4 |
| 12–24 | Medium | 1/4–1/2 |
| 24–45 | Stiff | 1/2–1 |
| 45–90 | Very stiff | 1–2 |
| > 90 | Hard | > 2 |

To date, as pointed out by Kulhawy and Mayne *(92)*, all attempts to correlate a modulus with the SPT $N$-value for fine-grained soils show considerable scatter. Therefore, they suggest the following relationships, as first order estimators

$$E/p_a = 5\ N_{60} \quad \text{(sands with fines)} \qquad (28)$$

$$E/p_a = 10\ N_{60} \quad \text{(clean normally consolidated sands)} \qquad (29)$$

$$E/p_a = 15\ N_{60} \quad \text{(clean overconsolidated sands)} \qquad (30)$$

in which $N_{60}$ is the $N$-value corrected for field procedures to an average energy ratio of 60 percent *(92)*.

Given the convenience and age of this test, it is not surprising that a number of the responding agencies use it. Connecticut, Illinois, Indiana, Minnesota, Pennsylvania, Utah, Vermont, and Wisconsin all list the SPT as a standard in situ test. The province of British Columbia and Austria also use it. Determination of pavement layer stiffness and soil stiffness are the primary reasons for performing the STP. Although New York did not list this method as being used to characterize soils, it did indicate a relationship between soil classification, STP results, and resilient modulus.

The primary advantages of the SPT are that it is has a huge experience database, it is widely available, and that it is relatively quick and simple to perform. It also provides both a soil test result, along with a sample of the soil. The principle disadvantage of the SPT is that it has many sources of error, such as the method of winding the hammer rope around the cathead on the drill rig. The SPT blow counts should not be relied on in soils containing coarse boulders, cobbles, or coarse gravel, because the sampler can become obstructed and give artificially high blow counts. The SPT should also not be relied on in soft and sensitive clays, because it tends to yield results inconsistent with actual field conditions.

#### Cone Penetration Testing

The CPT is a versatile sounding procedure that can be used to classify the materials in a soil profile and to obtain estimates of soil properties. In the CPT, a conical penetrometer tip is pushed slowly into the ground and monitored. Earlier versions of the CPT are known as mechanical friction cone penetrometers. However, more recent versions of the CPT have electrical transducers to measure both the tip ($q_c$) and side resistances ($f_s$) as the cone is advanced. More recently, piezocone penetrometers (CPTU) have been developed that measure the pore water pressures during penetration, as well as the cone tip and side resistances. Also, several new cone devices have been developed to measure additional parameters, including the seismic cone to measure P and S waves, resistivity cones for measuring the electrical resistivity of the soil, and cone penetrometers for environmental cones with water sampling capabilities.

The procedure for the CPT is described in ASTM D3441. To perform the test, an electric cone penetrometer tip is attached to a string of hollow steel rods and pushed vertically into the ground at a rate of about 20 mm/sec. As the cone is being pushed, the tip and side resistances, as well as the excess pore pressures ($\Delta u$) from the CPTU, are recorded continuously. Figure 32 *(99)* shows typical designs of electric cone penetrometers.

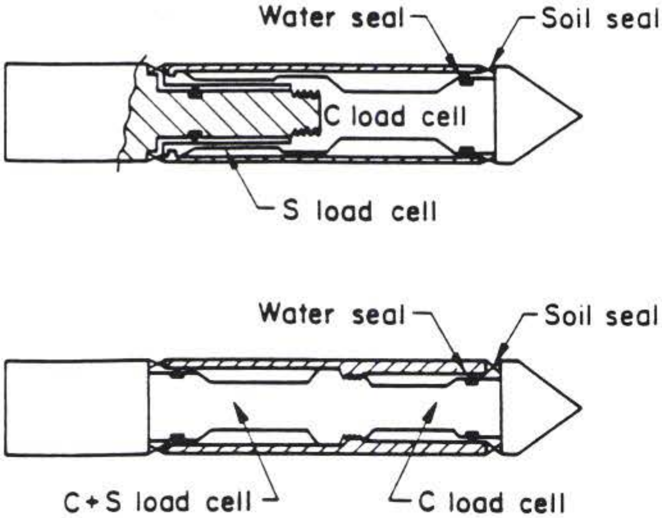



FIGURE 32 Schematic of typical electric cone penetrometer *(99)*.

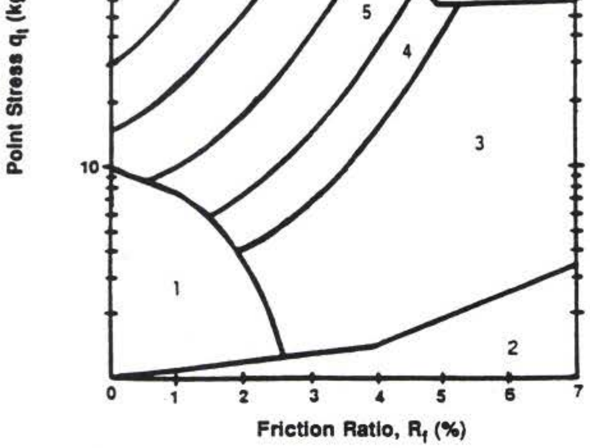
The accumulation of information from the CPT has resulted in the development of soils charts (*100*) (Figure 33). The CPT performs well in most soils; the exceptions being gravelly soils or soils containing cobbles, boulders, or cemented seams. Generally, in soils suitable for CPT testing, the test data has much less scatter than SPT data. Cone tip resistance ($q_c$), sleeve friction ($f_s$), and excess pore pressure ($\Delta u$) have been used in various combinations in empirical relationships with undrained shear strength ($s_u$), internal friction angle of sands ($\phi$), the elastic modulus, the overconsolidation ratio, and soil classification (*83, 101–103*). In particular, Figure 34 from Pamukcu and Fang (*83*) provides a rapid means of obtaining preliminary estimates of the various relationships often used in the characterization of pavement subgrades, including both the modulus of subgrade reaction ($k$) and the CBR.

The CPT has a number of advantages. It is a relatively fast and inexpensive way of testing soils. The electric cone provides an almost continuous record downhole that lends itself well to identifying problem soils, such as peat or soft clay layers. The test can be performed in a wide variety of soils, although dense soils and gravel cannot be penetrated easily. One major disadvantage of the CPT is that no sample is obtained during testing. Another big disadvantage is that the test may not be available everywhere. Many drilling contractors still do not have the test equipment readily available. A third disadvantage is that the cone may drift from vertical at greater depths, allowing for the possibility of false or misleading soil stratification data. However, many new cone penetrometers have inclinometers in their tips to monitor any deviations from vertical.

Only Illinois listed the static cone penetrometer as a standard test, and Illinois and New Jersey both indicated the use of the Dutch cone penetrometer.

#### Miniature Cone Penetrometer

Kurup and Tumay (104) described the development of a miniature cone penetrometer test (MCPT). In contrast



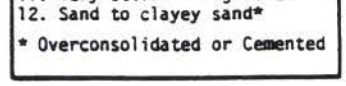




FIGURE 33 Soil classification by CPT *(95)*.

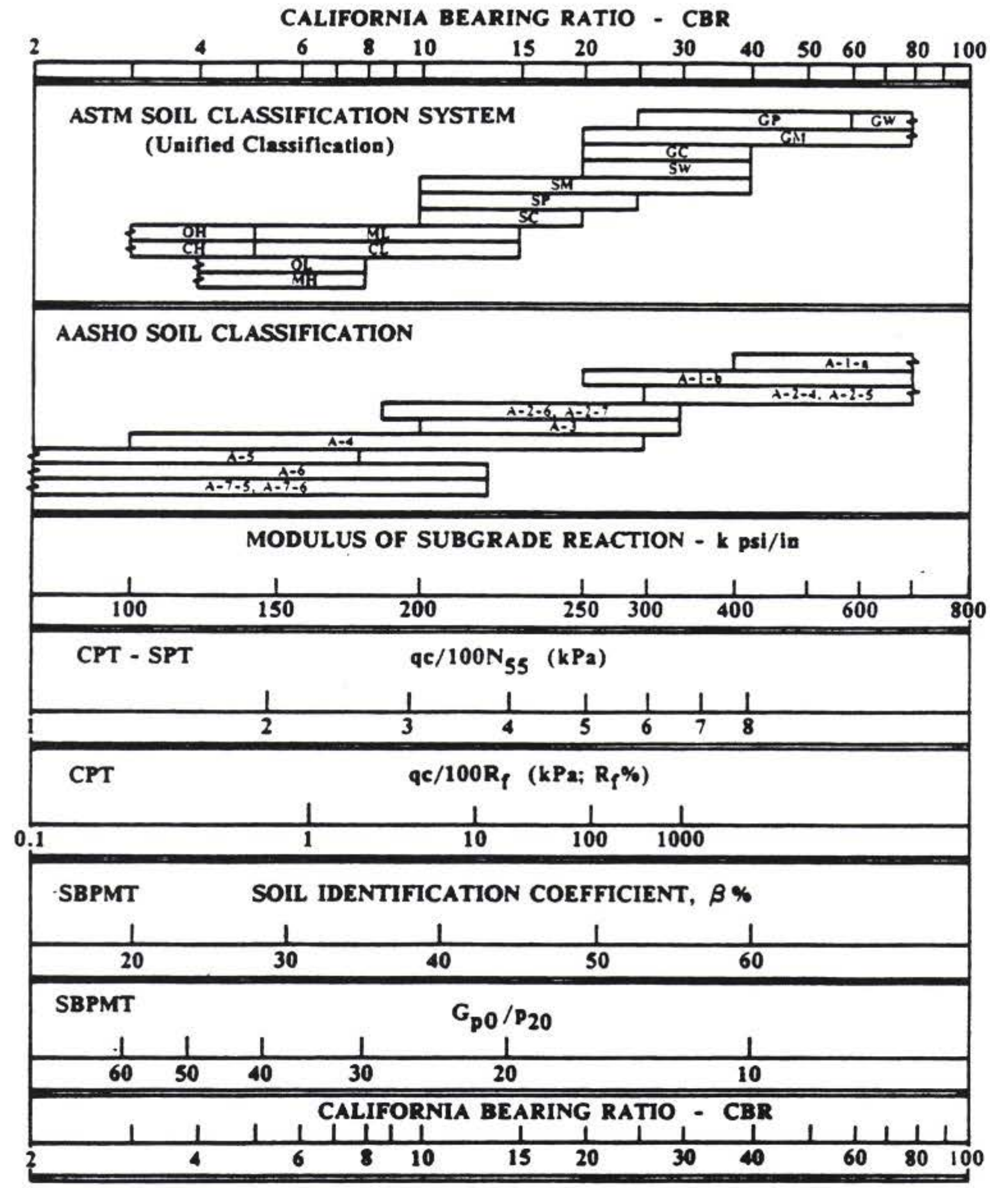



FIGURE 34 Soil strength and stiffness correlations *(83)*.

with the standard cone penetrometer with a projected cone area of 10 $cm^2$, the MCPT has a cone area of 2 $cm^2$. A flexible, coiled rod powered by a hydraulic system provides a means to continuously insert the cone tip in the soil mass at a constant rate. Output from this test includes tip resistance, sleeve friction, and friction ratio. The tip resistance was found to be 10 percent higher for the MCPT than the standard CPT, and the sleeve resistance of the CPT was greater than that of the MCPT. The friction ratio of the CPT was 23 percent greater. The MCPT has a variety of uses, including the characterization of subgrade soils, embankments, and earthen structures. Increased sensitivity through reduced tip resistance and sleeve friction is the main advantage for the use of this device.

In a study conducted in Louisiana, Tumay and Kurup (*105*) correlated the MCPT using a 1.27 $cm^2$ cone area to a friction cone penetrometer with a cone area of 15 $cm^2$. They observed that the joints in the standard device were points of weakness where water could infiltrate the device and damage the electronics. Also, the standard device penetration was not continuous, causing pore pressure dissipation during periods of stress release.

#### Dilatometer Testing

The flat-plate dilatometer test (DMT) was developed in Italy, in 1980 (*92*). Because of its rather recent introduction as a site investigation tool, the experience database behind the DMT is still somewhat limited. This may explain why it is not listed as a standard test method by any of the agencies in the survey. However, its relative ease of operation, durability, and reliability suggest that its use will increase over time *(92)*. Currently, no ASTM standard exists on the use of the DMT, although Schmertmann *(100)* proposed an ASTM procedure for performing the dilatometer. Figure 35 shows the equipment required to perform the DMT test. The dilatometer itself is a flat blade, 14 mm thick, 95 mm wide, and 220 mm long. A flexible stainless steel membrane, 60 mm in diameter, is located on the center of the blade. A combination gas and electrical line extends from the surface control box through a series of push rods and into the dilatometer blade. The test is performed by pushing the blade to the desired depth at a rate of penetration of 20 mm/sec. Once the blade has reached the desired depth, three readings are taken, namely the A, B, and C readings *(92)*. These readings are then used to obtain empirical relationships with the coefficient of lateral earth pressure at rest ($K_o$), and the undrained shear strength ($s_u$), as well as a direct measure of the modulus for cohesionless soils. As discussed by Kulhawy and Mayne *(92)*, many of these relationships are still somewhat preliminary, because of the limited database of DMT test data currently available. However, one relationship that is worth noting is the relationship between the dilatometer modulus ($E_D$) and the Young's modulus ($E$), which is given as follows

$$E_D = E/(1 - \nu^2) \tag{31}$$

where $\nu$ = Poisson's ratio.

The DMT is a simple and rapid way of testing soil. The test is rugged and can be used in a wide variety of soils. A big advantage is that the DMT seems to provide reasonable estimates of the horizontal stress and overconsolidation ratio of soils. The test is relatively inexpensive, allowing numerous data points to be obtained quickly. The greatest disadvantages with the DMT is that it currently has a somewhat limited experience database and most contractors do not have ready access to the DMT. Finally, the DMT has limited use in very dense or cemented soils, as well as in soils containing boulders, cobbles, or gravel.

#### Vane Shear Testing

The vane shear test (VST) is used to determine the peak and remolded undrained shear strength of soft to medium stiff clays. The procedure for the VST is described in ASTM D2573. Some important issues regarding the interpretation of the test are given elsewhere *(106,107)*. In this test, a shear vane is pushed into the soil and rotated from the surface at a standard rate of 0.1 degree per second. The peak torque that develops is related to the peak shear strength on a cylindrical failure surface by a constant, which is a function of the shape and dimensions of the vane. Once the peak shear strength has been determined, the soil is remolded by rotating the vane quickly about 10 times, and the torque is then measured again to determine the remolded shear strength. The ratio of peak to remolded shear strength is a measure of the material's sensitivity to disturbance.

The value of the undrained shear strength ($s_u$) determined from the VST should not be used directly in analysis, because it may need to be corrected for the soil anisotropy and the strain rate during testing *(108)*.

The VST provides a fairly rapid and economical means of testing homogeneous soil deposits. The experience database behind the VST is large, with numerous well-published

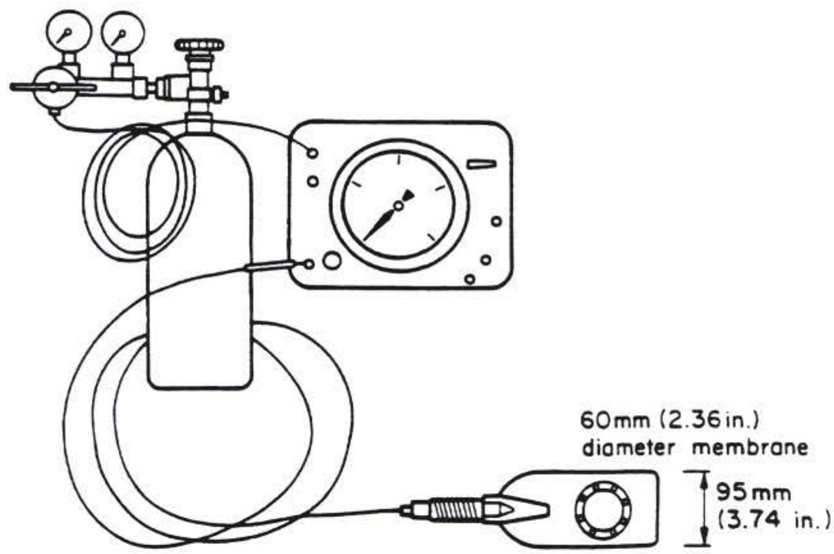



FIGURE 35 Dilatometer test equipment *(100)*.

published correlations to soil properties available. Some of the limitations of the VST are that the test is more easily applied to soft and medium stiff clays, and it is mainly useful for determining the undrained shear strength of soils. Finally, the theoretical nature of the failure mechanism is not well understood, leaving considerable uncertainty in correlations between field and laboratory measurements of the same soil.

### Field California Bearing Ratio Testing

The field CBR is similar to the laboratory CBR in that it consists of the same 1935-mm$^2$ (3-in$^2$) piston and that surcharge weights are again used that simulate the confining pressure of the pavement. In performing the field CBR, it is important to make sure that the deflection dial is fastened securely well outside the loaded area. Just as with the laboratory CBR, the field CBR determines the CBR of soils tested in place by comparing the penetration load of the soil to that of a standard material. The procedure for the field CBR is described in ASTM D4429.

As noted by Yoder and Witczak (*2*), correlations between field and laboratory CBR test values on granular materials are erratic because of the different boundary conditions between the field and the laboratory. However, for fine-grained materials at similar moisture and density conditions, the two tests will give similar results. It should also be pointed out that the field test is made at field moisture content, whereas the laboratory test is made in a soaked condition.

The field CBR test is used by two of the responding European countries, Austria and Switzerland, as an indication of soil stiffness. In North America the test is used by Connecticut, North Carolina, Pennsylvania, Utah, Vermont, and the Canadian province of New Brunswick.

## CORRELATIONS BETWEEN METHODS

Siekmeier and colleagues (*109*) conducted a study to investigate the correlation of results between the DCP, Loadman, SSG, and FWD. Their research showed a strong relationship between test results from equipment designed to estimate the modulus. The correlation was not as strong between the DCP and the results from the small-strain devices, SSG and Loadman. They recommended that the modulus results be accompanied by qualifiers specifying whether the loading is dynamic or static, the stress level, boundary conditions, relative density, and moisture. This research led to a new specification for requirements of a maximum penetration index value from DCP testing for dense-graded aggregate base materials in Minnesota.

A commonly accepted relationship between the CBR and subgrade modulus is (*2*)

$$E_{sg} = 1500(\mathrm{CBR}) \tag{32}$$

where $E_{sg}$ = subgrade modulus, psi.

The SI version of this equation, where $E_{sg}$ is in MPa, is

$$E_{sg} = 10.3(\mathrm{CBR}) \tag{33}$$

This is the relationship used in Indiana and the province of Alberta. Some states, such as Ohio, have modified this equation such that the constant is 1200, instead of 1500. Virginia uses a range of 750 to 3000 for the coefficient, with a maximum CBR value of 10. Ohio also uses a relationship to estimate the CBR from the mean plus two standard deviations of the deflection measured at 1525 mm. Alabama uses the following relationship to define the subgrade modulus from CBR

$$E_{sg} = 10^{(0.851\log\mathrm{CBR}+2.971)} \tag{34}$$

This equation was based on test results that Alabama obtained on AASHO Road Test soils, in which the SSV was related to the CBR and to the resilient modulus.

CHAPTER FIVE

# VARIABILITY IN SUBGRADE MECHANICAL PROPERTIES

There is considerable momentum to progress toward mechanistic-empirical design and introduce the concept of reliability to pavement design. To properly accommodate reliability, the variability of design input parameters must be quantified, which includes consideration of seasonal and spatial changes in material properties. Thus, it is important to understand the magnitude of spatial variability and seasonal variability in the soil and find appropriate terms to express them.

## SPATIAL VARIABILITY

In any given geologic formation there is a limited amount of control over spatial variability. For instance, in a dry desert lakebed, the soil may be very uniform, but very soft. On the other hand, cut and fill operations in mountain passes may result in roadbeds that transition from stiff to soft in a very short distance (Figure 36) (*110*). In this study, Test Site 22 was located on a mountainside, whereas Test Site 24 was in a relatively uniform desert valley. Given that a log scale is used to depict the modulus, the variability in the modulus for the mountain road is considerable.

Spatial variability is addressed by a number of agencies in terms of the frequency of sampling for laboratory tests and the frequency of testing for deflection measurements. It is interesting to note (see Table B-3) that those agencies performing simple laboratory tests such as soil classification were more likely to indicate a high rate of sampling, say on the order of 12 per kilometer (e.g., Iowa, New Hampshire, and British Columbia), whereas those performing tests such as the CBR or *R*-value were more likely to use a considerably lower rate of sampling and testing (on the order of 2 per kilometer). Many of the respondents differentiate sampling rates according to soil type and traffic level. Lower sampling rates were noted for coarse-grained materials than for fine-grained materials. In some

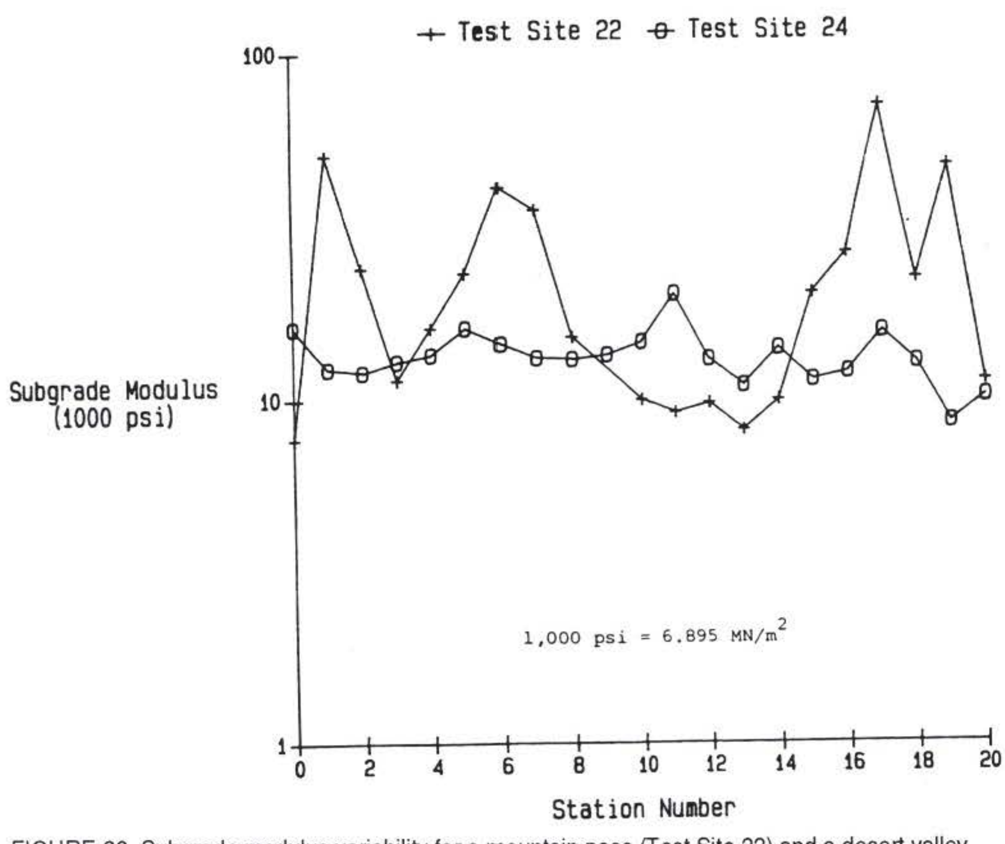



FIGURE 36 Subgrade modulus variability for a mountain pass (Test Site 22) and a desert valley (Test Site 24) *(110)*.

TABLE 7

VARIABILITIES OF A VARIETY OF IN SITU TESTS *(after 112)*

| Test | COV Due to Equipment (%) | COV Due to Procedure (%) | COV Due to Random Error (%) | COV Total (%) | COV Range (%) |
|---|---|---|---|---|---|
| Standard penetration | 5–75 | 5–75 | 12–15 | 14–100 | 15–45 |
| Mechanical cone penetration | 5 | 10–15 | 10–15 | 15–22 | 15–25 |
| Electrical cone penetration | 3 | 5 | 5–10 | 7–12 | 5–15 |
| Vane shear | 5 | 8 | 10 | 14 | 10–20 |
| Dilatometer | 5 | 5 | 8 | 11 | 5–15 |
| Pressuremeter | 5 | 12 | 10 | 16 | 10–20 |
| Self-boring pressuremeter | 8 | 15 | 8 | 19 | 15–25 |

Note: COV, coefficient of variance.

instances, no soil sampling is done in the case of coarse-grained subgrades. Generally speaking, high-volume facilities were reported to have a greater sampling rate than medium- to low-volume roads. Again, some agencies indicated no subgrade sampling and laboratory testing for low-volume roads.

Deflection testing is generally performed at a much higher frequency than material sampling and laboratory testing, as shown in Table B-4, presumably because of the ease and value of in situ deflection testing. Many respondents indicated testing rates of about 6 to 12 measurements per kilometer. Fewer agencies indicated discrimination in testing rates according to soil type or traffic levels. As with laboratory testing, higher volume roadways and fine-grained soils seemed to dictate higher rates of deflection testing.

#### Variability of Intrusive Measurements

Most in situ tests are somewhat limited in application because of the difficulties involved with determining boundary conditions around the tests and the unknown magnitude of the soil disturbance caused by advancing the in situ test device into the soil. To overcome some of these difficulties in practice, the design engineer interprets in situ measurements of subgrade soils to obtain the soil properties. This interpretation process is very subjective and relies heavily on the experience and judgment of the design engineer. The soil properties obtained from in situ tests are influenced by the natural (geologic) variability of the soil, the variability from the in situ test measurement, and the uncertainty in the empirical correlation between the in situ test and the desired soil property.

Kulhawy et al. *(111)* identified three sources of uncertainty in obtaining a soil property for design from a measured in situ test parameter, such as the cone penetration resistance. The first source is the natural or inherent variability of the subgrade soil resulting from its method of deposition and subsequent geologic history. The second source is the measurement error that is the difference between the measured in situ parameter and its actual field value. This difference can result from equipment, procedural, operator, and random test effects *(112)*. The third source is the sampling error that results from the limited availability of information about site-specific subgrade soil conditions. This source can be decreased with additional testing.

Orchant et al. *(112)* estimated the variability of a number of in situ tests (Table 7). These estimates suggest that the electric cone penetration test (ECPT) and the DMT have a smaller total variability than the other in situ tests. It is also striking that the most commonly used test, namely the STP, can have coefficients of variation of up to 100 percent.

Kulhawy et al. *(111)* also evaluated the measurement error associated with the CPTU. Table 8 presents the results from their study. It is of interest to note that the coefficient of variation of the measurement error was found to be on the average about 6 percent, which is close to the lower bound (5 percent) for the values reported by Orchant et al. *(112)* for the ECPT.

#### Variability of Deflection Measurements

Houston and Perera *(113)* conducted FWD tests at 20 different pavement sites. These deflection measurements were made at an interval of 3 m in each 28-m section. They concluded that most of the variability in the deflections within a section were due to variability in the subgrade soils and that the variations occurred over distances of less than 28 m for the sites investigated. It was also their conclusion that most of the variability occurred in the basement soils, not the engineered (compacted) soils.

Deflection testing for the Minnesota Road Research Project (Mn/ROAD) *(114,115)* was conducted after the preparation of the subgrade and after the placement and

TABLE 8

ERROR ASSOCIATED WITH PIEZOCONE PENETRATION TEST *(after 111)*

| Site | Soil Description | COV Measurement Error (%)[1] |
|---|---|---|
| San Francisco Bay | Dark silty clay | 4.6 |
| Rio De Janeiro | Gray plastic clay | 9.9 |
| Yorktown | Very sandy clay | 5.8 |
| McDonald Farm | Sand and gray silt | 7.8 |
| Ottawa STP | Gray marine clay | 3.6 |
| Arnprior | Sensitive gray clay | 5.8 |
| Brent Cross | Fissured gray–blue clay | 7.6 |
| Glava | Coarse gray marine clay | 5.1 |
| Anacostia | Dark organic silty clay | 10.8 |
| Beaumont | Light gray clay | 3.1 |
| Average | | 6.1 |

[1]COV, coefficient of variance.

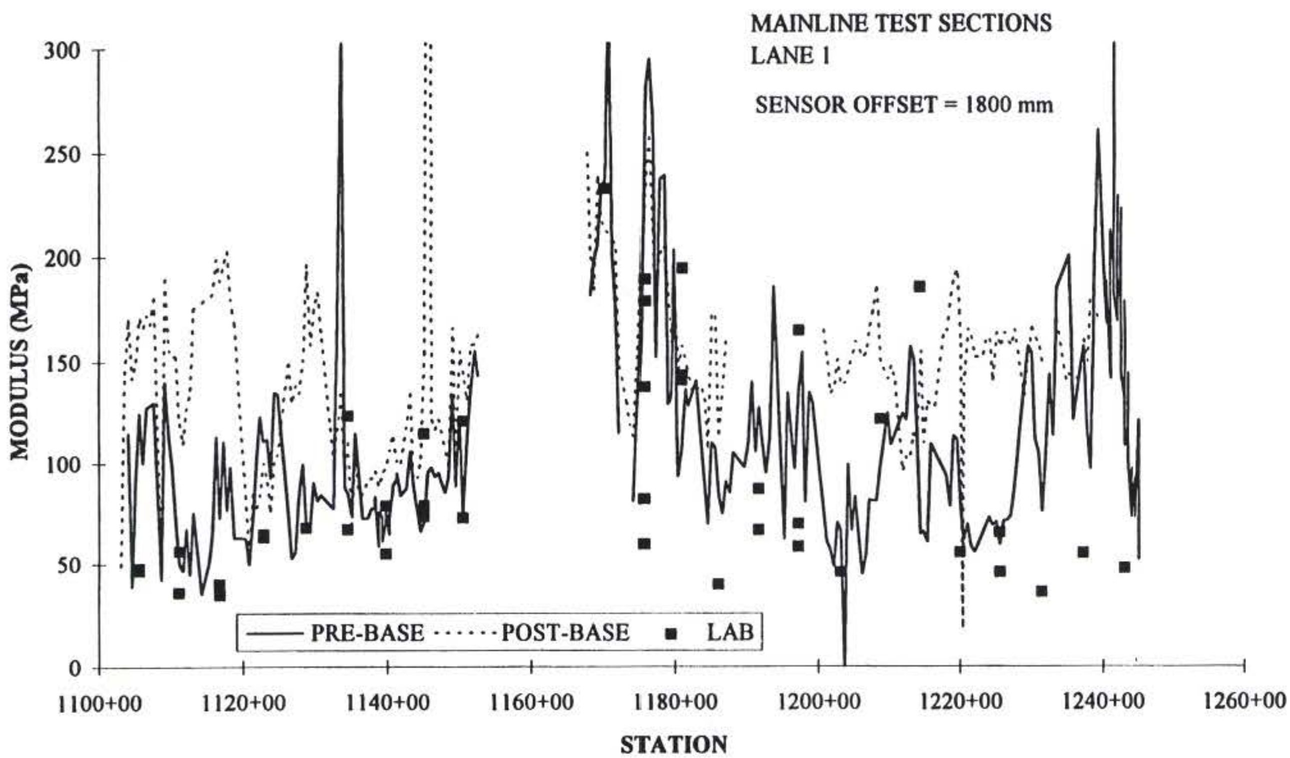



FIGURE 37 Comparison of pre- and post-base backcalculated subgrade moduli at Mn/ROAD *(114)*.

compaction of base materials. It was shown that a relatively large variation in subgrade modulus could occur in a relatively short distance in this formation of engineered fill consisting of glacial till. The subgrade surface condition, as it affected the loading plate contact area, had a significant effect on measured deflections, particularly near the plate. It was also noted that the subgrade modulus showed an increase in stiffness after placement of the base, most probably due to a reduction in stress on the fine-grained soil. As shown in Figure 37, the increase in subgrade modulus due to the presence of the base results in a reduction in variability.

#### Measures of Spatial Variability

Kulhawy et al. *(111)* evaluated the inherent geologic variability for a number of well-documented clay sites. The method used to estimate the inherent geologic variability of soil relies on the assumption that each deposit has what is termed a "correlation distance" ($\delta$). The correlation distance is defined as the distance within which the soil property in question (i.e., undrained shear strength) is correlated, but outside of which no correlation exists. Typically, in highly variable soil, the correlation distance is low, whereas in a relatively homogeneous soil profile, the

TABLE 9

PIEZOCONE PENETRATION TEST VERTICAL CORRELATION DISTANCES FOR VARIOUS SITES *(after 111)*

| Site | Soil Description | Vertical Correlation Distance (m) |
|---|---|---|
| San Francisco Bay | Dark silty clay | 0.49 |
| Rio De Janeiro | Gray plastic clay | 0.48 |
| Yorktown | Very sandy clay | 0.40 |
| McDonald Farm | Sand and gray silt | 0.30 |
| Ottawa STP | Gray marine clay | 0.29 |
| Arnprior | Sensitive gray clay | 0.29 |
| Brent Cross | Fissured gray–blue clay | 0.26 |
| Glava | Coarse gray marine clay | 0.23 |
| Anacostia | Dark organic silty clay | 0.21 |
| Beaumont | Light gray clay | 0.19 |
| Average | | 0.31 |

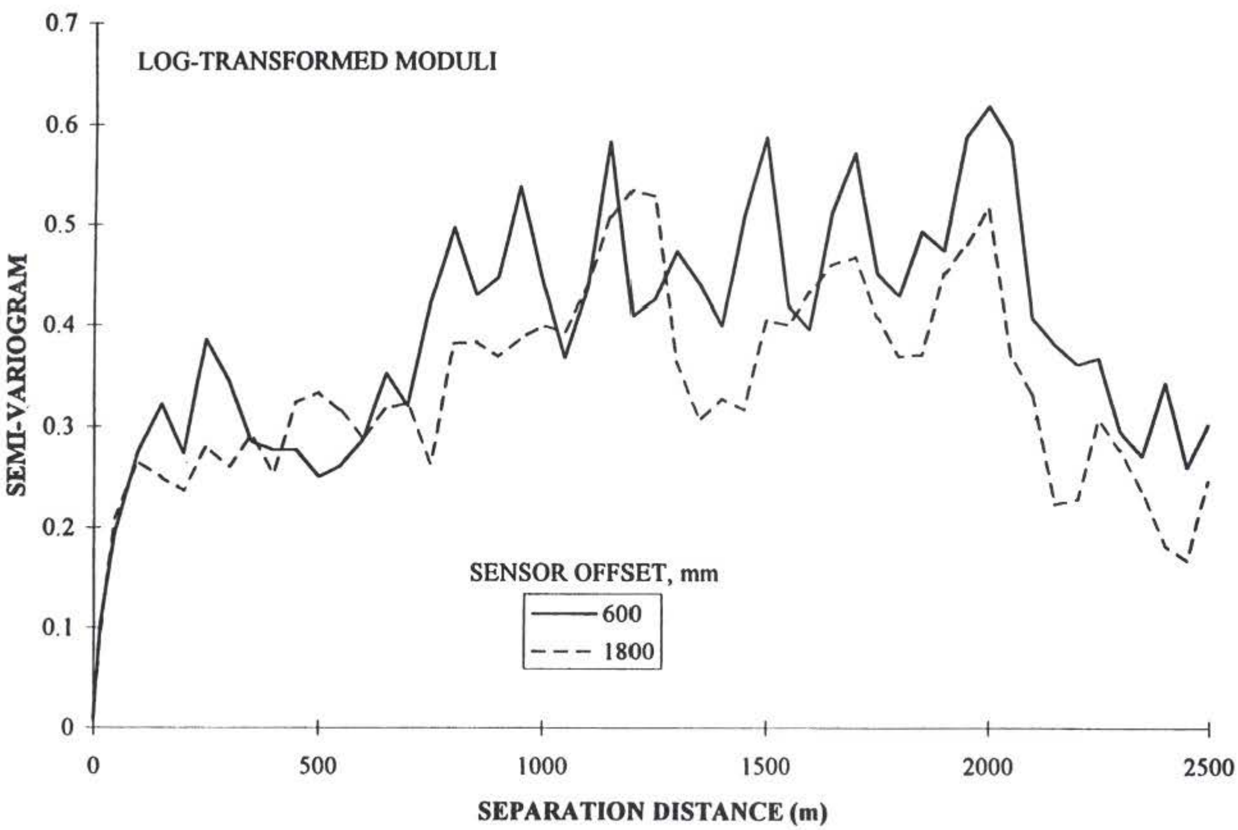



FIGURE 38 Variogram of Mn/ROAD backcalculated subgrade modulus *(114)*.

correlation distance is high. Table 9 presents a summary of the vertical correlation distances for the sites studied. The average correlation distance was found to be 0.31 m (≈1 ft).

Barnes (*116*) used the correlation length in discussing some practical considerations based on the desired design objectives. He introduces this concept as a means to establish the required frequency of sampling. Although it is not a simple calculation, the concept is illustrated in Figure 38 in the form of a variogram. The idea is that the smaller the variation, the longer the correlation length, and the greater the variability, the shorter the correlation length. In Figure 38, the variability of the subgrade modulus is fairly constant after about 100 m. In the end, the correlation length along with the project size may be used to determine the frequency of sampling for a given level of desired precision, as shown in Figure 39. The correlation length may be determined using the computer program *Subgrade Geostatistics*, available from the Minnesota DOT (*116*).

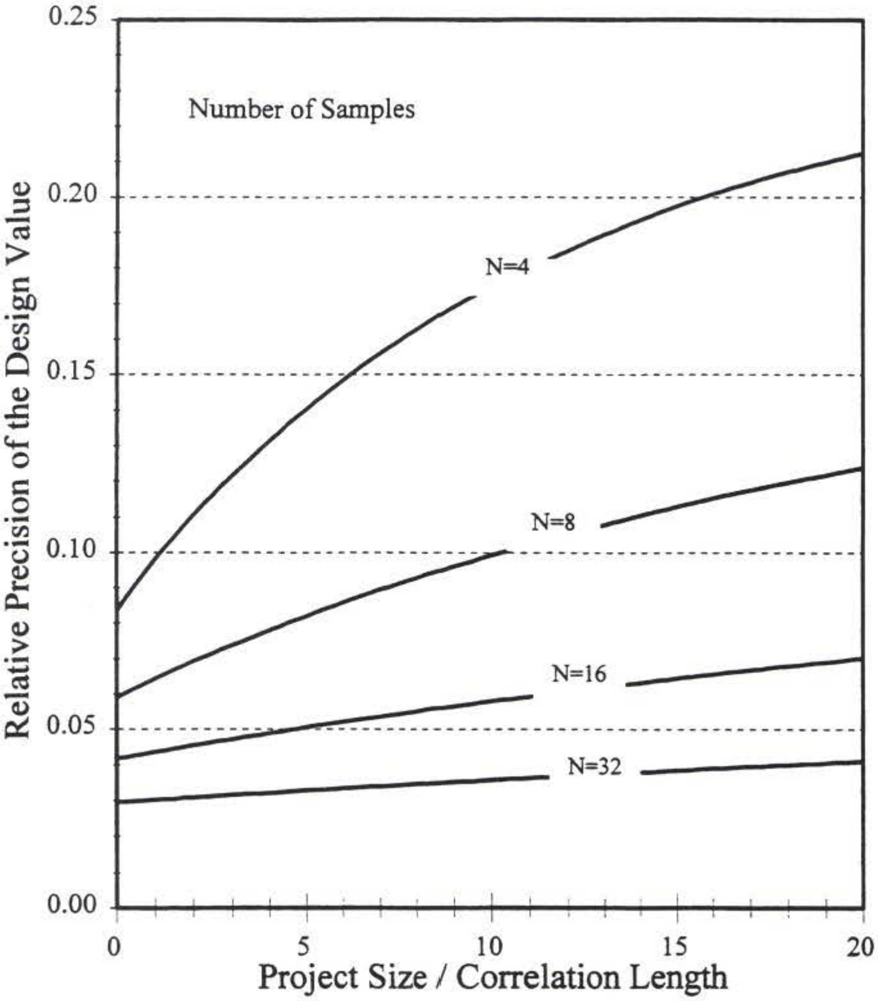



FIGURE 39 Use of correlation length to determine the appropriate sampling frequency *(116)*.

TABLE 10

HORIZONTAL CORRELATION DISTANCES FOR VARIOUS SOIL PARAMETERS *(after 111)*

| Site | Soil Description | Parameter | Horizontal Correlation Distance (m) |
|---|---|---|---|
| McDonald Farm | Sand and silt | Cone tip resistance | 37 |
| Isselmeer Delta | Sand and silt | Cone tip resistance | 20 |
| Isselmeer Delta | Sand and silt | Porosity | 20–30 |
| North Sea Site 1 | Clay | Cone tip resistance | 66 |
| North Sea Site 1 | Clay | Cone tip resistance | 23 |
| Chicago | Clay | Water content | 170 |
| Unknown | Sand | Coefficient of compressibility | 55 |
| New Liskegard | Varved clay | Undrained shear strength | 46 |

Kulhawy et al. *(111)* summarized the results from some of the very few studies that have reported horizontal correlation distances for soil. Table 10 lists the reported values of the horizontal correlation distances from various sites around the world. The horizontal correlation distances for the cone tip resistance range from 20 m for Isselmeer Delta sand *(117)* to 66 m for North Sea clay *(118)*. The maximum reported horizontal correlation distance of 170 m was for the in situ water content of Chicago clay *(119)*.

#### Subgrade Atlas

In an application of statistical principles to spatial variability in subgrade properties, Barnes developed a computerized

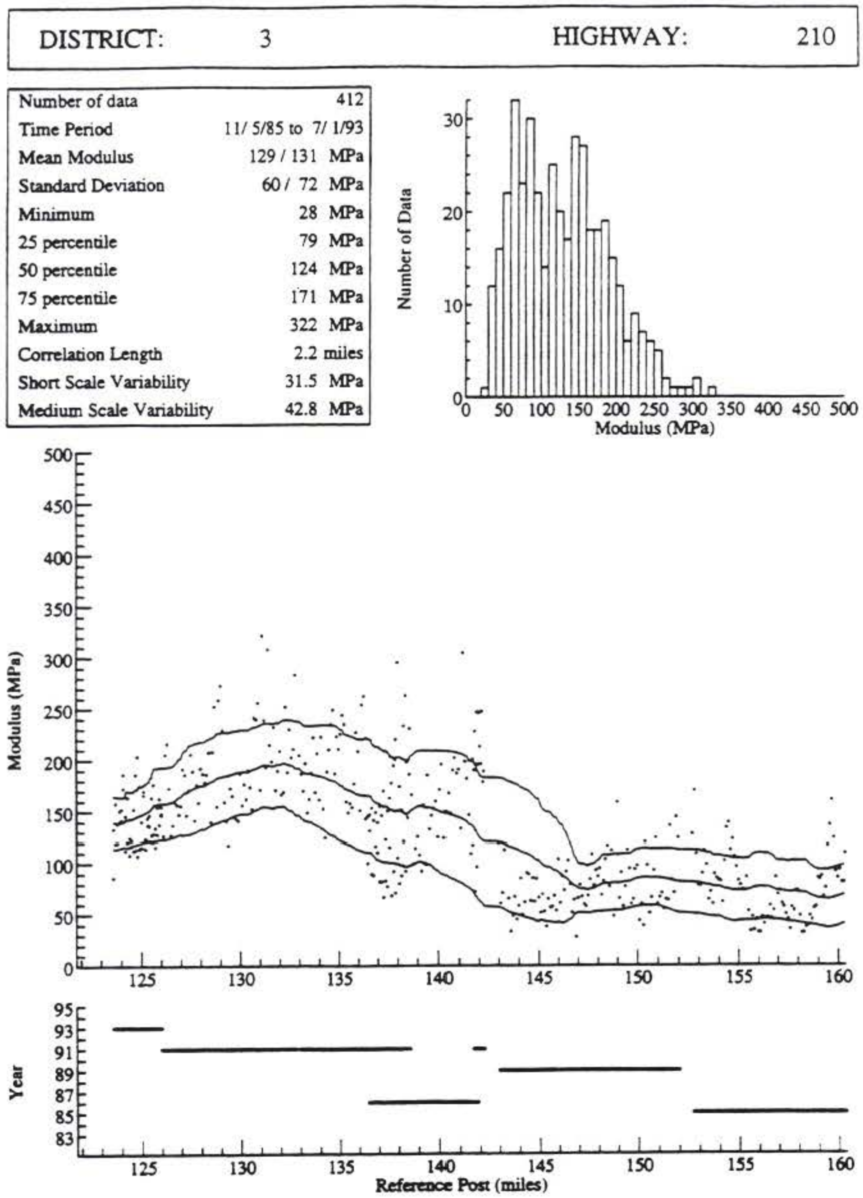



FIGURE 40 Example of output from *Subgrade Atlas—Online (120).*

subgrade modulus atlas for Minnesota (*120*). FWD deflection test results collected on state highways over a 10-year period were converted to subgrade modulus using the technique described in chapter 4. Over 120,000 test values were collected and analyzed. For each road in each Minnesota DOT district, the data may be plotted over any given length. The running mean value is plotted along with the standard deviation along the length of the section. Furthermore, a histogram is plotted showing a distribution of the data for that segment. An example of the output is shown in Figure 40.

This program is useful because it runs in a Windows-based environment and the analysis of the data can be performed on scales ranging from the network level to the project level. Designers may choose to discriminate sections according to changes in the running mean or they may choose to select a design value that reflects the relative variability in the soil mass. Areas deserving additional field investigation may be selected on the basis of the plots.

Many state agencies have large databases containing deflection measurements collected over a number of years. Similar efforts to analyze and present the information could provide engineers with a powerful tool for investigation, analysis, and design.

## SEASONAL VARIABILITY

Seasonal variability takes on importance in pavement design as practice evolves from considering the worst case

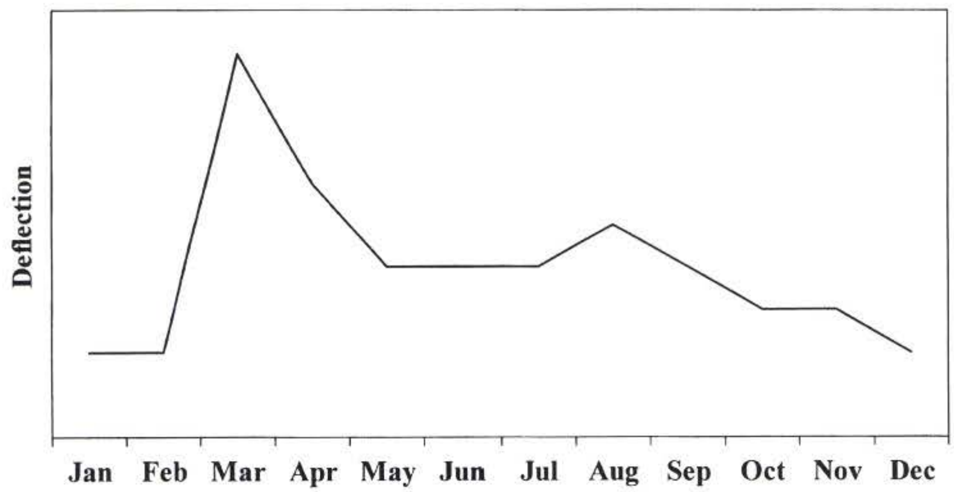



FIGURE 41 Change in pavement deflection with time for frost areas.

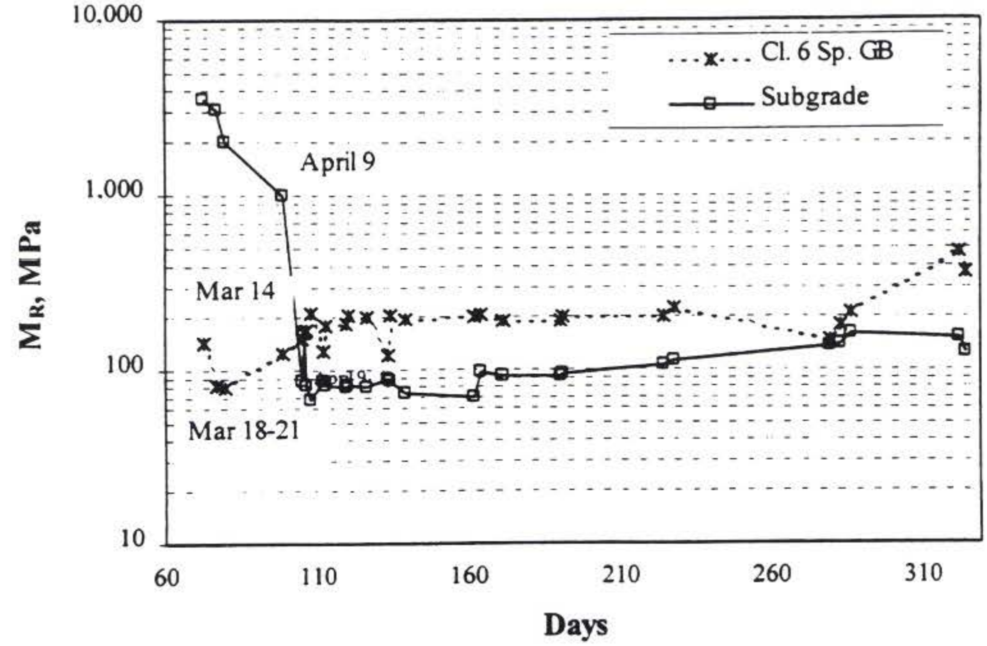



FIGURE 42 Seasonal changes in base and subgrade moduli at Mn/ROAD *(121)*.

(saturated) to a recognition that damage needs to realistically reflect the soil conditions at various times. This latter approach is suggested in the 1993 AASHTO guide (*5*) and it is common to most mechanistic-empirical design procedures. Testing a soil in a so-called worst case is an acknowledgment that most of the damage to a pavement due to soil weakening will occur in the wettest, unfrozen conditions. This type of an approach is typical of design procedures involving the CBR value of soils, because the laboratory CBR test is performed on soil in a saturated condition.

The typical view of seasonal weakening in a frozen climate is shown in Figure 41. Here it can be seen that the deflection is relatively low in the winter and increases dramatically during the spring thaw. The roadway strength recovers somewhat during the summer and the deflection decreases again. The material stiffens in the winter with the onset of freezing temperatures. In the past, it was traditional to attribute the spring weakening to the subgrade; in other words, the deflection would increase as the subgrade softens. However, research studies on test sites in Washington State (*20*) and at the Mn/ROAD (*121*) have shown that the critical period during the spring is largely due to unfrozen water being trapped in dense granular base materials. During this time, ice in the subgrade does not allow the melt water to drain. The result is that the modulus of the base softens to a point where the modulus of the subgrade is greater, as shown in Figure 42.

However, not every highway agency is concerned about the effects of freezing and thawing, and some agencies have two different sets of climates within the same jurisdiction. Poehl and Scrivner (*122*) suggested that the spatial variability in deflection measurements in Texas is usually greater than the seasonal variability, and that the seasonal variability is usually related to cycles of rainfall. In the state of Washington (*20*), it was found that seasonal adjustments on the west side of the Cascade Mountains were best made considering rainfall, and that on the east side of the mountains it was best to consider the effects of freeze-thaw cycles.

### Techniques for Seasonal Adjustments

The great majority of U.S. state highway agencies and some Canadian and European agencies indicated that they use the 1993 version of the AASHTO design procedure (Table B-1). The guide places a heavy emphasis on the seasonal variations in resilient modulus (*5*). Almost all the compensation for seasonal variability in the AASHTO guide is handled through the selection of the design modulus for the subgrade soil.

#### AASHTO

The AASHTO guide allows for the use of two different procedures for determining the seasonal variation of the subgrade modulus. One of these relies on obtaining a laboratory relationship between the modulus and the moisture content in the soil. The modulus is then varied for each of the different seasons by the expected change in moisture content of the soil. However, the problem with this method is the prediction of the moisture contents of the field subgrade soil by season. An alternative procedure is to backcalculate the resilient modulus from FWD tests for different seasons by testing the pavement at different times over the year. Because it is currently difficult to predict the changes in the moisture content in subgrade soils, the backcalculation of layer moduli remains a reasonable alternative for measuring seasonal variation of pavement subgrades.

If the seasonal modulus values are determined through the use of backcalculated FWD results, then these subgrade moduli must be multiplied by an adjustment factor ($C$), defined as

$$C = \text{(Laboratory Modulus)}/\text{(Backcalculated Modulus)} \tag{35}$$

This factor adjusts the backcalculated modulus to an equivalent laboratory value because the AASHTO design procedure is based on laboratory moduli. The correction or adjustments to the backcalculated equivalent modulus for roadbed or subgrade soils are dependent on the materials above the subgrade. Table 11 (*34*) lists some typical $C$-values.

TABLE 11

AASHTO MODULUS CORRECTION VALUES FROM LONG-TERM PAVEMENT PERFORMANCE SECTIONS (*34*)

| Layer Type and Location | $C$-Value |
|---|---|
| Granular base/subbase under PCC | 1.32 |
| Granular base/subbase under AC | 0.62 |
| Granular base/subbase between stabilized layer and AC | 1.43 |
| Subgrade soils under stabilized subgrade | 1.32 |
| Subgrade under full-depth AC or PCC | 0.52 |
| Subgrade under granular base subbase | 0.35 |

Note: PCC, portland cement concrete; AC, asphalt concrete.

#### Mechanistic-Empirical Methods

Similarly, a number of states use mechanistic-empirical procedures as secondary design methods and these require the adjustment of seasonal variations in modulus values as a part of the design. Mahoney et al. (*20*) developed a mechanistic-empirical design procedure for use in overlay design based on the backcalculation of material properties and fatigue and rutting failures. In their approach, the environmental effects of temperature and precipitation were incorporated into the design method. The data on the seasonal variations were based on the backcalculated moduli from 3 years of FWD testing at various locations in the state of Washington, and climatic data were obtained from published climate information. The ratios of the moduli for the different seasons were determined and are presented in Table 12. In this study, the subgrade was assumed to be homogeneous and semi-infinite in depth.

TABLE 12

UNBOUND MATERIAL SEASONAL ADJUSTMENT FACTORS FOR WASHINGTON STATE (*20*)

| | Base | | Subgrade | |
|---|---|---|---|---|
| Region | Wet/Thaw | Dry/Other | Wet/Thaw | Dry/Other |
| East | 0.65 | 1.00 | 0.95 | 1.00 |
| West | 0.80 | 1.00 | 0.90 | 1.00 |

Similarly, Hein and Jung (*123*) studied the seasonal variations in pavement strength in Ontario, Canada. In their study, they calculated various pavement layer indicators, such as normalized dynamic deflection, subgrade modulus, subgrade deflection, and vertical compressive strain using the MTO Probe pavement layer analysis program. As a result, they were able to identify a series of environmental factors that affect pavement performance and response. Finally, the variations in these factors

TABLE 13

UNBOUND MATERIAL SEASONAL ADJUSTMENT FACTORS FOR IDAHO *(21)*

| | | Seasonal Adjustment Factors | | | |
|---|---|---|---|---|---|
| Climate Type | Material Type | Winter | Spring | Summer | Fall |
| Significant frost penetration | Subgrade | 11.2 | 0.43 | 1.00 | 1.00 |
| | Base | 1.00 | 0.65 | 1.00 | 1.00 |
| Little frost penetration | Subgrade | 0.27–0.81 | 0.63–0.90 | 1.00 | 1.00 |
| | Base | 0.65 | 0.85 | 1.00 | 1.00 |

throughout spring thaw were compared with measurements taken 1 year earlier.

Bayomy et al. *(21)* also developed a mechanistic-based flexible overlay design procedure for Idaho that accounts for the seasonal variation of pavement subgrade materials. In their study, Idaho was divided into six pavement climate zones. In the zones that experience significant subgrade frost penetration, the average year was divided into four periods, summer, freezing transition, winter, and spring thaw recovery. Based on their findings, they created seasonal adjustment factors for the subgrade soils to adjust for the changes in the resilient modulus during these periods. These factors are $R_f$, $R_t$, and $R_w$ for the frozen, thaw, and wet periods, respectively. Typical values are given in Table 13. They were inserted into the following equation to obtain the appropriate resilient modulus ($M_f$, $M_t$, or $M_w$)

$$M_i = M_{\text{summer}} \times R_I \tag{36}$$

Finally, the freeze-thaw period resilient modulus was reduced for granular soils to various degrees.

The most recent of these studies *(121)* used data obtained from the Mn/ROAD, located on Interstate 94 in central Minnesota.

Figure 42 shows the seasonal variation in the resilient modulus ($M_R$) for a granular base and fine-grained subgrade soil. For completeness of presentation, it should be noted that the Class 6 Special consists of crushed granite with from 0 to 5 percent passing the 0.075-mm sieve. The subgrade soil is a naturally deposited silty clay with an $R$-value of 12. Similarly, this study also indicated that the modulus of soil decreases with increasing volumetric water content.

CHAPTER SIX

# CONCLUSIONS

The purpose of this synthesis was to report on the state-of-the-practice with respect to in situ testing of pavement subgrade soils. Existing and emerging technologies for static and dynamic, destructive, and nondestructive testing were discussed. Correlations between in situ and laboratory tests were presented. Effects of existing layers on the measurement of subgrade properties and the subjects of soil spatial and seasonal variability were considered. Most importantly, the use of measured soil properties in pavement design and evaluation was explained. New applications or improvements to existing in situ test methods to support the use of mechanistic/stochastic-based pavement design procedures were also explained.

Based on the results of the survey and a review of the literature the following findings were noted:

- The most popular primary flexible pavement design procedure among the state agencies surveyed in the United States is the 1993 AASHTO pavement design guide. An older version of the AASHTO procedure (1986) is the second most popular method, followed by the granular equivalency method.
- A number of states use a mechanistic-empirical design procedure as a secondary, flexible pavement design method or as a check on their primary approach.
- Most states use the 1993 AASHTO guide for the design of their rigid pavements. The 1972 version of the AASHTO guide is used by a number of states as is the PCA method. Illinois uses a mechanistic-empirical approach.
- When characterizing the mechanical properties of soils it is important to understand the factors that affect those properties. Specifically, it is necessary to account for the state of stress, moisture content, state of moisture, and density.
- The vast majority of U.S., Canadian, and European respondents use FWDs for deflection testing. High-speed deflectometers are being developed and are beginning to be used. The Belgian Road Research Centre currently uses the French Curviameter on a routine basis.
- Many states, provinces, and European countries use the maximum deflection and subgrade modulus calculated from deflection measurements. Backcalculation of layer moduli is also extensively used by a large number of agencies.
- Of the three small-load methods of in situ testing presented, only the SASW is routinely used by state agencies. The Loadman and the Humboldt Stiffness Gauge are currently under evaluation.
- Among the agencies surveyed, the DCP and SPT are popular methods for checking soil strength and compaction uniformity. Some agencies also listed the field CBR test as a means of in situ testing.
- Subgrade property spatial variability is an important consideration in the design of pavement structures. Agency decisions on sampling frequency are determined by factors such as the type of soil, volume of traffic, the difficulty of testing, and analysis and cost. Sampling techniques exist to assist designers in determining the appropriate frequency of testing.
- In procedures such as the 1993 AASHTO method or mechanistic-empirical design approaches require knowledge of changes in material properties with seasons of the year. In frost regions, it is most important to account for the frozen, spring thaw, and summer/fall conditions. In nonfrost areas, the differentiation is mostly done on the basis of wet versus dry seasons.

The following conclusions and suggestions are based on the information gathered for this synthesis:

- As agencies change from purely empirical to mechanistic-empirical design procedures, in situ test and analysis methods must be developed to provide the parameters required for design and for verification during construction by using new approaches.
- An effort should be made to compare and synthesize backcalculation techniques. Greater uniformity in the approach to analyzing deflection data will be increasingly important as design procedures change.
- Methods for analyzing data from high-speed deflection testing need to be developed to provide information pertaining to pavement structural performance and design.
- New methods of in situ testing should be vigorously explored to understand their usefulness in project evaluation. Nonintrusive small strain techniques and high-speed deflection devices are among those that should be researched.
- Rational tools should be developed to assist engineers with decisions concerning the frequency and testing of subgrade materials.

- States should develop resources on in situ subgrade properties, such as Minnesota's *Subgrade Atlas—Online,* to assist engineers. This computerized document allows an a priori evaluation of mean subgrade modulus, modulus variability, and spatial variability.
- Resources in the form of appropriate information or the means to gather this information, such as instrumentation, should be made available to states to better quantify the seasonal changes in material properties, especially as these changes pertain to pavement design.

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# APPENDIX A

## Survey Questionnaire

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Project 20-5, Topic 29-08

Measuring In-Situ Mechanical Properties of Pavement Subgrade Soils

# QUESTIONNAIRE

*Agency:* ______________________________

*Respondent:* ______________________________

*Title:* ______________________________

*Phone:* __________ *Fax:* __________ *E-mail:* __________

***Flexible Pavement Design***

1. What is the *primary* method of flexible pavement design used by your agency?

   a. AASHTO 93
   b. AASHTO 62
   c. Asphalt Institute 81
   d. Granular/Gravel Equivalency
   e. Corps of Engineers (CBR)
   f. Mechanistic-Empirical
   g. Other (please specify) __________

2. What input parameter is used to describe soil strength or stiffness in your agency's primary flexible pavement design procedure?

   a. Resilient Modulus
   b. CBR
   c. R-Value
   d. Soil Support Value
   e. Other (please specify) __________

3. What *secondary* methods of flexible pavement design are used by your agency?

   a. AASHTO 93
   b. AASHTO 62
   c. Asphalt Institute 81
   d. Granular/Gravel Equivalency
   e. Corps of Engineers (CBR)
   f. Mechanistic-Empirical
   g. Other (please specify) __________

***Rigid Pavement Design***

4. What is the *primary* method of rigid pavement design used by your agency?

   a. AASHTO 93
   b. AASHTO 72
   c. Portland Cement Association
   d. Corps of Engineers
   e. Other (please specify)________________________________

5. What input parameter is used to describe soil strength or stiffness in your agency's primary rigid pavement design procedure?

   a. Resilient Modulus
   b. Modulus of Subgrade Reaction
   c. Other (please specify) ____________________

6. What *secondary* methods of rigid pavement design are used by your state?

   a. AASHTO 93
   b. AASHTO 72
   c. Portland Cement Association
   d. Corps of Engineers
   e. Other (please specify)________________________________

***Laboratory Testing of Soils***

7. What is the *primary* laboratory test your agency uses to characterize the strength or stiffness of subgrade soils for flexible pavement design?

   a. CBR
   b. R-Value
   c. Resilient Modulus
   d. Triaxial Compression
   e. Soil Classification
   f. Other (please specify) ____________________

8. Please list all applicable AASHTO, ASTM, SHRP or other methods used in performing the laboratory tests in Question 7.

   AASHTO T-____________ SHRP P-______________

   ASTM _______________ Other_________________

9. Is another laboratory test other than the one noted in Question 7 used for rigid pavements?

Yes No

If yes, please specify the test and the test method used.

10. What frequency of testing (number of tests/mile) is used for a typical road project for the laboratory test designated in Question 4 for the soil and traffic condtions below?

| Type of Road | Plastic Soils | Non-Plastic Soils |
|---|---|---|
| Interstate or Freeway | | |
| Principal Arterial | | |
| Collector | | |
| Local | | |

***In-Situ Testing of Soils***

11. What types of deflection measuring equipment are used by your agency?

a. Falling Weight Deflectometer
b. Road Rater
c. Dynaflect
d. Benkelman Beam

12. What parameters or other data manipulations are used from deflection testing?

a. Maximum Deflection
b. Surface Curvature Index
c. Base Curvature Index
d. "Area"
e. Subgrade Modulus
f. Modulus of Subgrade Reaction
g. Backcalculation of All Layer Moduli

13. What is the typical frequency of deflection testing (tests/mile) for each of the following situations?

| Type of Road | Cut Section | Fill Section |
|---|---|---|
| Interstate or Freeway | | |
| Principal Arterial | | |
| Collector | | |
| Local | | |

14. What are the applicable AASHTO, ASTM, SHRP or other methods used in the deflection testing?

AASHTO T-___________ SHRP P-_____________

ASTM _______________ Other_________________

15. Does your agency use one or more correlations to transform deflection parameters to subgrade input for pavement design? (For instance, converting subgrade modulus to CBR.)

Yes No

If yes, please provide the correlations used. (For instance: CBR = $E_{sg}$/1500)

16. If you indicated the calculation of subgrade modulus from deflections in Question 12, please give the equation used.

17. If you indicated the calculation of the modulus of subgrade reaction from deflections in Question 12, please give the equation used.

18. If you indicated the use of backcalculation in Question 12, please indicate which program and version is used.

a. ELMOD______
b. EVERCALC______
c. MODCOMP______
d. MODULUS______
e. BOUSDEF______
f. WESDEF______
g. Other (please specify) ___________________

19. Does your agency use other methods of in-situ testing for subgrade soils?

a. Field CBR
b. Standard Penetration Test
c. Dutch Cone Penetrometer
d. Dynamic Cone Penetrometer
e. Spectral Analysis of Surface Waves (SASW)
f. Other (please specify)____________________

20. The reason(s) for using in-situ test methods other than deflection are:

a. Measuring Layer Stiffness
b. Compaction Uniformity
c. Overall Soil Stiffness
d. Other (please specify) __________________

21. Does your agency use one or more correlations to transform the results of these tests to subgrade input for pavement design?

    Yes No

    If yes, please provide the correlations used.

22. We realize it is difficult to summarize your agency's complete practice of subgrade strength or stiffness characterization in a general questionnaire. If you have additional information or insights you would like to share, please use the space below.

**Thank you for your time and contribution to this study!**

# APPENDIX B
## Survey Results

Table B-1. Summary of Flexible Pavement Design Practices.

| Agency | Primary Design Method[1] | Soil Stiffness/Strength Input[2] | Secondary Design Methods[3] |
|---|---|---|---|
| **United States** | | | |
| AK | AK Excess Fines | % passing 0.075mm (-No. 200) | M-E |
| AL | AASHTO 93 | CBR converted to $M_R$ | None |
| AR | AASHTO 93 | R-value | Roadhog |
| AZ | AASHTO 93 | R-value | None |
| CO | AASHTO 93 | $M_R$ | None |
| CT | AASHTO 93 | $M_R$ | None |
| FL | AASHTO 93 | $M_R$ | Minimum Catalog |
| GA | AASHTO 62 | SSV | AASHTO 93, M-E |
| HI | GE | R-value | AASHTO 93 |
| IA | AASHTO 93 | $M_R$ | None |
| ID | GE | R-value | M-E |
| IL | M-E, IDOT | Subgrade Support Rating, CBR (mod) | AI 81 for LV Overlays |
| IN | AASHTO 93 | CBR converted to $M_R$ | None |
| KS | AASHTO 93 | $M_R$ | M-E |
| LA | AASHTO 93 | $M_R$, SSV | None |
| MA | AASHTO 62 | CBR, SSV | AASHTO 93 |
| ME | AASHTO 93 | $M_R$, SSV | None |
| MN | GE, Mn/DOT Deep Strength | R-value | AASHTO 93, AI 81, M-E |
| MT | AASHTO 62 | R-value, SSV | AASHTO 93 |
| NC | AASHTO 72 | CBR, SSV | NC Deflection Method |
| NE | AASHTO 93 | $M_R$ | None |
| NH | AASHTO 93 | SSV | None |
| NJ | AASHTO 93 | $M_R$ | M-E |
| NM | AASHTO 62 (81 Revision) | R-value | None |
| NV | AASHTO 93 | R-value | M-E |
| NY | AASHTO 93, Catalog for Overlays | $M_R$ | AASHTO 93 |
| OH | AASHTO 93 | $M_R$ | None |
| PA | AASHTO 93, AASHTO 72 | $M_R$, CBR | None |

Table B-1. Summary of Flexible Pavement Design Practices (cont.).

| Agency | Primary Design Method | Soil Stiffness/Strength Input | Secondary Design Methods |
|---|---|---|---|
| **United States** | | | |
| RI | AASHTO 93 | $M_R$ | None |
| SC | AASHTO 62 | SSV | AASHTO 93 |
| UT | AASHTO 93 | CBR | None |
| VA | AASHTO 62 (VA modified) | $M_R$, SSV | AASHTO 93 |
| VT | AASHTO 93 | $M_R$ | M-E |
| WA | AASHTO 93, M-E for Overlays | $M_R$ | WSDOT Method |
| WI | AASHTO 62 | SSV, Design Group Index (WI method) | None |
| **Canada** | | | |
| AB | AASHTO 93 | $M_R$ | Engineering Judgement |
| BC | AASHTO 93, GE, Engrg. Judgement | $M_R$, Unified Soil Class. | GE, Engrg. Judgement |
| MB | GE | Group Index | AASHTO 93 |
| NB | GE, CGRA | CBR | None |
| NF | Empirical | CBR | AI 81 |
| ON | GE, M-E (Ontario) | $M_R$, Soil Class. (GBE) | AASHTO 93 |
| **Europe** | | | |
| Austria | Austrian Design Catalog | Modulus of Deformation | M-E |
| Belgium | British TRL Method (Belgian mod) | CBR | French Catalog |
| Finland | M-E | $M_R$ | None |
| Iceland | Norwegian Index System (GE system) | Soil Classification | None |
| Norway | Norwegian Index System (GE system) | CBR | Component Analysis |
| Switzerland | AASHTO 93 (Swiss mod) | CBR or Plate Modulus (ME) | None |

1,3. GE - Granular Equivalency, M-E – Mechanistic-Empirical, CGRA – Canadian Good Roads Assn., TRL – Transport Research Laboratory
2. $M_R$ – Resilient Modulus, SSV – Soil Support Value

Table B-2. Summary of Rigid Pavement Design Practices.

| Agency | Primary Design Method[1] | Soil Stiffness/Strength Input[2] | Secondary Design Methods[3] |
|---|---|---|---|
| **United States** | | | |
| AK | N/A | N/A | N/A |
| AL | AASHTO 93 | CBR converted to $M_R$ | None |
| AR | AASHTO 93 | k-value | None |
| AZ | AASHTO 93 | k-value | None |
| CO | AASHTO 93 | k-value | AASHTO Soil Class. |
| CT | AASHTO 93 | k-value | PCA |
| FL | AASHTO 93 | k-value | None |
| GA | AASHTO 72 | k-value | AASHTO 93 |
| HI | PCA | k-value | None |
| IA | PCA | k-value | AASHTO 93 |
| ID | AASHTO 93 | $M_R$, k-value | Corps of Engineers |
| IL | M-E for JRCP, IDOT for CRCP | Subgrade Support Rating, CBR (mod) | AASHTO 93 for Overlays |
| IN | AASHTO 93 | k-value | None |
| KS | AASHTO 93 | k-value | None |
| LA | AASHTO 93 | k-value | None |
| MA | N/A | N/A | N/A |
| ME | N/A | N/A | N/A |
| MN | AASHTO 72 | k-value | AASHTO 93 |
| MT | PCA | k-value | AASHTO 93 |
| NC | AASHTO 72 | k-value | None |
| NE | AASHTO 93 | $M_R$ | None |
| NH | N/A | N/A | N/A |
| NJ | AASHTO 93 | k-value | PCA |
| NM | AASHTO 93 | k-value | None |
| NV | AASHTO 93 | k-value | None |
| NY | AASHTO 93, Catalog for Overlays | k-value | M-E for Overlays |
| OH | AASHTO 93 | k-value | None |
| PA | AASHTO 93, AASHTO 72 | $M_R$, CBR | None |

Table B-2. Summary of Rigid Pavement Design Practices (cont.).

| Agency | Primary Design Method | Soil Stiffness/Strength Input | Secondary Design Methods |
|---|---|---|---|
| **United States** | | | |
| RI | AASHTO 93 | $M_R$ | None |
| SC | AASHTO 93 | k-value | AASHTO 72 |
| UT | AASHTO 93 | k-value | None |
| VA | AASHTO 93 | k-value | None |
| VT | AASHTO 93 | $M_R$, k-value | None |
| WA | AASTHO 93 | k-value | None |
| WI | AASHTO 72 | k-value | AASHTO 93 |
| **Canada** | | | |
| AB | N/A | N/A | N/A |
| BC | AASHTO 93 | $M_R$ | PCA |
| MB | AASHTO 93 | k-value | None |
| NB | PCA | k-value | None |
| NF | N/A | N/A | N/A |
| ON | PCA (Canadian) | k-value | AASHTO 93 |
| **Europe** | | | |
| Austria | Austrian Design Catalog | Modulus of Deformation | M-E |
| Belgium | Belgian Analytical and Empirical Methods | k-value | N/A |
| Finland | N/A | N/A | N/A |
| Iceland | N/A | N/A | N/A |
| Norway | Norwegian (Combination PCA/CE) | k-value | None |
| Switzerland | AASHTO 93 (Swiss mod) | CBR or Plate Modulus (ME) | None |

1, 3. PCA – Portland Cement Association, M-E – Mechanistic Empirical, CE – Corps of Engineers, N/A – Not Applicable
2. $M_R$ – Resilient Modulus

Table B-3. Laboratory Test Methods for Subgrade.

| Agency | Lab Test for Flexible Pavements[1] | Lab Test for Rigid Pavements[2] | Sampling Frequency[3] |
|---|---|---|---|
| **United States** | | | |
| AK | Soil Class. | N/A | NSF |
| AL | CBR | N/A | 1/km (2/mi) |
| AR | R-value, Soil Class. | N/A | 1/km (2/mi) (HV) 1/km (1/mi) (MV, LV) |
| AZ | R-value, Soil Class. | Soil Class. | 2/km (3/mi) |
| CO | R-value | N/A | 1/300 m (1/1000) ft in fills, 1/150 m (1/500 ft) in cuts |
| CT | Soil Class. | N/A | NSF |
| FL | Limerock Bearing Ratio | Gradation/Permeability | 3/lift/km (4/lift/mile) |
| GA | CBR | N/A | 1/km (1/1.5 mi) |
| HI | R-value | N/A | 1/km (1/mi) (FG), 0 (CG) |
| IA | Soil Class. | N/A | 16-19/km (25-30/mi) (HV), 13-19/km (20-30/mi) (MV), 0 (LV) |
| ID | R-Value | N/A | 1-2/km (1-3/mi) (HV), 1/km (1-2/mi) (MV, LV) |
| IL | Soil Class. | N/A | 3/km (4/mi) |
| IN | CBR, Soil Class. | N/A | 1/350 m (1/1200 ft) (FG), 1/450 m (1/1500 ft) (CG) |
| KS | MR | N/A | 1/km (1/mi) |
| LA | Soil Class. | N/A | 4/km (6/mi) |
| MA | Soil Class. | N/A | 3/km (4/mi) (FG), 1/km (1/mi) (CG) |
| ME | CBR, Soil Class. | N/A | NSF |
| MN | R-value | N/A | 2/km (3/mi) (FG), 0-1/km (0-1/2mi) (CG) |
| MT | R-value | Soil Class. | 1/km (2/mi) |
| NC | CBR | N/A | 1-2/km (2-3/mi) (HV), 1/km (1/mi) (MV), 0 (LV) |
| NE | MR, Triaxial | N/A | Depends on number of cuts/fills |
| NH | Soil Class. | N/A | 50/km (75/mile) (HV, MV), 30/km (50/mile) (LV) |
| NJ | MR | N/A | NSF |
| NM | R-value | N/A | 4/km (6/mi) |
| NV | R-value | N/A | 3/km (5/mi) |
| NY | Soil Class. | N/A | 4/km (7/mi) (HV, FG), 4/km (6/mi) (HV, CG), 3/km (5/mi) (MV), 2/km (3/mi) (LV) |
| OH | Soil Class. | N/A | 1/425 m (1/1400 ft) |
| PA | MR | N/A | 3/km (4/mi) (HV), 1/km (1/mi) (MV), 0 (LV) |

Table B-3. Laboratory Test Methods for Subgrade (cont.).

| Agency | Lab Test for Flexible Pavements | Lab Test for Rigid Pavements | Sampling Frequency |
|---|---|---|---|
| **United States** | | | |
| RI | Soil Class. | N/A | 1/km (2/mi) |
| SC | CBR | N/A | 6/km (10/mi) |
| UT | CBR | N/A | |
| VA | CBR | N/A | NSF |
| VT | Soil Class. | N/A | 6/km (10/mi) (HV, MV), 0 (LV) |
| WA | MR | N/A | NSF |
| WI | Soil Class. | N/A | 1-2/km (1-3/mi) (FG), 1/km (1/mi) (CG) |
| **Canada** | | | |
| AB | Soil Class. | N/A | NSF |
| BC | Soil Class. | N/A | 33/km (52/mi) |
| MB | Soil Class. | N/A | 5/km (8/mi) (HV,MV), 0 (LV) |
| NB | Soil Class. | N/A | |
| NF | Soil Class. | N/A | |
| ON | Soil Class. | N/A | 1/50m for fill, 1/25m for cut |
| **Europe** | | | |
| Austria | Soil Class. | N/A | |
| Belgium | CBR | N/A | 1/200m (FG, HV), 1/500m (FG,MV), 0 (CG, HV, MV), 0 (LV) |
| Finland | MR, Soil Class. | N/A | 5-20/km |
| Iceland | Soil Class. | N/A | |
| Norway | Soil Class. | N/A | Varies |
| Switzerland | CBR | N/A | |

1. $M_R$ – Resilient Modulus
2. N/A – Not Applicable
3. NSF – No Set Frequency, HV – High Volume, MV – Medium Volume, LV – Low Volume, FG – Fine-Grained, CG – Coarse-Grained

Table B-4. Pavement Deflection Test Methods.

| Agency | Use In-Situ Testing for Design Input | | Deflection Equipment[1] | Parameters from Deflection Testing[2] | Testing Frequency[3] |
|---|---|---|---|---|---|
| | Original | Overlay | | | |
| **United States** | | | | | |
| AK | No | Yes | FWD | Backcalc. E | 6/km (10/mi) |
| AL | No | Yes | FWD | $D_0$, $M_R$ | 3/km (5/mi) (HV, MV) |
| AR | No | No | FWD | $M_R$, Backcalc. E | 50-75/project |
| AZ | No | Yes | FWD | $D_0$, SCI | 3/km (5/mi) |
| CO | No | No | FWD | $D_0$, "Area", k, Backcalc. E | 6/km (10/mi) |
| CT | Yes | Yes | None | | |
| FL | No | Yes | FWD, Dynaflect | $D_0$, $M_R$ | 18/km (28/mi) |
| GA | No | No | FWD | Backcalc. E | |
| HI | No | No | None | | |
| IA | No | No | RR | k | 15 tests (< 2 km (3 mi)), 30 tests (> 2 km (3mi)) |
| ID | No | Yes | FWD | $D_0$, "Area", Backcalc. E | 6+/km (10+/mi) (HV), 6/km (10/mi) (MV), 3-6/km (5-10/mi) (LV) |
| IL | No | No | FWD | $D_0$, "Area", k, Backcalc. E | |
| IN | No | Yes | FWD, Dynaflect | Backcalc. E | 1/30 m (1/100 ft) |
| KS | Yes | Yes | FWD | $D_0$, $M_R$, k, Backcalc E | 6/km (10/mi) |
| LA | No | No | FWD | $D_0$, $M_R$, Backcalc E, SN | Project Specific |
| MA | No | Yes | FWD | $D_0$, $M_R$ | 2-3/km (3-5/mi) (HV, MV) |
| ME | Yes | Yes | FWD | $D_0$, $M_R$ | 6/km (10/mi) |
| MN | Yes | Yes | FWD | $D_0$, $M_R$, "Area", Backcalc. E | 6/km (10/mi) |
| MT | No | No | FWD, RR | "Area", Backcalc. E | 10/km (16/mi) |
| NC | No | Yes | FWD | $D_0$, $M_R$, "Area", k-value, Backcalc. E (occasional) | 6/km (10/mi) |
| NE | No | Yes | FWD | $D_0$, $M_R$, SCI | 6+/km (10+/mi) (HV, FG), 3-6/km (5-10/mi) (HV, CG), 6/km (10/mi) (MV, LV, FG), 3-6/km (5-10/mi) (MV, LV, CG) |

Table B-4. Pavement Deflection Test Methods (cont.).

| Agency | Use In-Situ Testing for Design Input | | Deflection Equipment[1] | Parameters from Deflection Testing[2] | Testing Frequency[3] |
|---|---|---|---|---|---|
| | Original | Overlay | | | |
| **United States** | | | | | |
| NH | No | No | None | | |
| NJ | No | Yes | FWD | $M_R$, Backcalc. E | 10/km (16/mi) (HV) |
| NM | No | No | FWD | | 13/km (21/mi) |
| NV | No | Yes | FWD | $D_0$, $M_R$, Backcalc. E | 6/km (10/mi) |
| NY | Yes | Yes | FWD | $D_0$, $M_R$, "Area", k, Backcalc. E | 16/km (25/mi) |
| OH | No | No | FWD(research), Dynaflect(design) | "Area", $M_R$ (limited), Backcalc. E (research) | 1/60m (1/200 ft) (AC) 1/5 slabs (PCC) |
| PA | Yes | Yes | FWD, RR | $M_R$, k, Backcalc. E | 6/km (10/mi) (HV, FG), 3/km (4/mi) (HV, CG), 3/km (4/mi) (MV) |
| RI | No | No | None | | |
| SC | No | Yes | FWD | $M_R$, Composite Pavement Modulus | 6/km (10/mi), min of 15/project |
| UT | No | Yes | FWD | $D_0$, $M_R$, Backcalc E | 6/km (10/mi) |
| VA | No | No | FWD | $D_0$, $M_R$, "Area", Backcalc. E | |
| VT | Yes | Yes | FWD | Backcalc. E | 3-6/km (4-10/mi) |
| WA | No | Yes | FWD | $D_0$, $M_R$, "Area", Backcalc. E | 13/km (20/mi) |
| WI | No | No | FWD | $M_R$, Backcalc. E | 6/km (10/mi) |
| **Canada** | | | | | |
| AB | No | No | FWD | Backcalc. E | 10/km |
| BC | No | Yes | FWD | $M_R$, Backcalc. E | 33/km (52/mi) |
| MB | No | Yes | FWD and Benkelman Beam | $D_0$, Backcalc. E | 10/km (16/mi) |
| NB | No | Yes | Dynaflect | $D_0$ | 1/200m |
| NF | No | Yes | Dynaflect | $D_0$ | 10/km (16/mi) |
| ON | No | No | FWD (only special circumstances) | | |

Table B-4. Pavement Deflection Test Methods (cont.).

| Agency | Use In-Situ Testing for Design Input | | Deflection Equipment[1] | Parameters from Deflection Testing[2] | Testing Frequency[3] |
|---|---|---|---|---|---|
| | Original | Overlay | | | |
| **Europe** | | | | | |
| Austria | Yes | Yes | FWD, Benkelman Beam, LaCroix Deflectometer | $D_0$, $M_R$, Backcalc. E | 20-40/km |
| Belgium | No | Yes | French Curvameter | $D_0$, $M_R$, Radius of Curvature | 1/5m |
| Finland | No | Yes | FWD | $D_0$, SCI, Backcalc. E | 20/km |
| Iceland | No | No | FWD | $D_0$, $D_{20}$, $D_{45}$ | |
| Norway | Yes | Yes | FWD, Dynaflect | $D_0$, SCI, Backcalc. E | 20/km |
| Switzerland | Yes | Yes | FWD, Benkelman Beam, LaCroix Deflectometer | $D_0$ | LaCroix @ 3.2m, BB @ 25m |

1. FWD – Falling Weight Deflectometer, RR – Road Rater
2. $D_0$ – Deflection @ Load Center, $D_{20}$ – Deflection @ 20 cm, $D_{45}$ – Deflection @ 45 cm, $M_R$ – Resilient Modulus, SCI – Surface Curvature Index
4. HV – High Volume, MV – Medium Volume, LV – Low Volume, FG – Fine-Grained, CG – Coarse-Grained

Table B-5. In-Situ Subgrade Tests Other Than Deflection.

| Agency | Other In-Situ Tests[1] | Reasons for Other Tests |
|---|---|---|
| **United States** | | |
| CT | Field CBR, SPT | Soil Class., Relative Density |
| FL | DCP, SASW in research | |
| IL | SPT, Dutch, DCP, Static Cone Pen, Field Unconf. Comp. Strength | |
| IN | SPT | Layer Stiffness, Material Comparisons |
| KS | DCP | Soil Stiffness |
| MN | SPT, DCP | Layer Stiffness, Soil Stiffness |
| NC | Field CBR, DCP | Layer Stiffness, Depth/Strength of Stone Base |
| NE | Nuclear Density, Moist. Content | Compaction Uniformity |
| NJ | Dutch Cone Pen., DCP, SASW | Research Uniformity |
| OH | DCP | Research |
| PA | Field CBR, SPT, DCP | Soil Stiffness |
| UT | Field CBR, SPT, DCP | Layer Stiffness, Compaction Uniformity, Soil Stiffness |
| VA | Nuclear Density | Compaction Uniformity, Density |
| VT | Field CBR, SPT | Layer Stiffness |
| WI | SPT | SSV or $M_R$ |
| **Canada** | | |
| BC | SPT | Layer Stiffness |
| NB | Field CBR, DCP | |
| ON | | |
| **Europe** | | |
| Austria | Field CBR, SPT, Plate Bearing Test | Layer Stiffness, Compaction Uniformity |
| Belgium | DCP | Layer Stiffness, Compaction Uniformity, Overall Soil Stiffness |
| Norway | DCP, Plate Bearing Test | |
| Switzerland | Field CBR | Overall Soil Stiffness |

1. DCP – Dynamic Cone Penetrometer, SPT – Standard Penetration Test

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