



AASHTO GUIDE FOR

Design of Pavement Structures

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AASHTO® Guide for Design of Pavement Structures 1993



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SPECIAL NOTICE

The *Guide for Design of Pavement Structures*, when it was published in 1986, was published as two volumes. Volume 1 was written as a basic design guide and provided all of the information required to understand and apply the "Guide" to pavement design. Volume 2 was a series of appendices prepared to provide documentation or further explanations for information contained in Volume 1. Volume 2 is not required for design.

This 1993 edition of the "Guide" contains only one Volume. This Volume replaces the 1986 "Guide" Volume 1 and serves the same purpose. The major changes included in the 1993 "Guide" are changes to the overlay design procedure and the accompanying appendices L, M, and N. There are other minor changes and some of an editorial nature throughout the new Volume 1.

Volume 2 of the 1986 "Guide" is still applicable to most sections of Volume 1 of the 1993 "Guide" and is available through AASHTO, 444 N. Capitol Street, N.W., Suite 249, Washington, D.C. 20001; 202-624-5800. Request book code "GDPS3-V2." A copy of the Table of Contents from Volume 2 of the 1986 "Guide" follows.

VOLUME 2 APPENDICES

AA.	Guidelines for the Design of Highway Internal Drainage Systems
BB.	Position Paper on Pavement Management
CC.	Remaining Life Considerations in Overlay Design
DD.	Development of Coefficients for Treatment of Drainage
EE.	Development of Reliability
FF.	Relationship Between Resilient Modulus and Soil Support
GG.	Relationships Between Resilient Modulus and Layer Coefficients
HH.	Development of Effective Roadbed Soil Moduli
II.	Survey of Current Levels of Reliability
JJ.	Development of Design Nomographs
KK.	Determination of J-Factor for Undowelled Pavements
LL.	Development of Models for Effects of Subbase and Loss of Support
MM.	Extension of Equivalency Factor Tables
NN.	Recommendations for the Selection of an AASHTO Overlay Method Using NDT Within the AASHTO Performance Model Framework
OO.	Pavement Recycling Fundamentals
PP.	Development of NDT Structural Capacity Relationships



PREFACE

When construction, maintenance, and rehabilitation costs are considered, the single most costly element of our nation's highway system is the pavement structure. In an effort to reduce this cost, the state highway and transportation departments and the Federal Government have sponsored a continuous program of research on pavements. One output of that research effort was the *Interim Guide for the Design of Pavement Structures* published in 1972 and revised in 1981. It was based largely upon the findings at the AASHO Road Test.

Because this is such an important topic, the Joint Task Force on Pavements—composed of members from the Subcommittee on Design, one member each from the Materials, Construction, and Maintenance Subcommittees, and one from the Planning Committee of AASHTO—was assigned the task of rewriting the Interim Guide incorporating new developments and specifically addressing pavement rehabilitation.

Because many states were found to be using at least portions of the Interim Guide and because no other generally accepted procedures could be identified, it was decided that this Guide would retain the basic algorithms developed from the AASHO Road Test as used in the Interim Guide. Because the Road Test was very limited in scope, i.e. a few materials, one subgrade, non-mixed traffic, one environment, etc., the original Interim Guide contained many additional models to expand the framework so designers could consider other conditions. The new Guide has been further expanded with the following 14 major new considerations:

- (1) Reliability
- (2) Resilient Modulus for Soil Support
- (3) Resilient Modulus for Flexible Pavement Layer Coefficients
- (4) Drainage
- (5) Improved Environment Considerations
- (6) Tied Concrete Shoulders or Widened Lanes
- (7) Subbase Erosion for Rigid Pavements
- (8) Life Cycle Cost Considerations
- (9) Rehabilitation
- (10) Pavement Management

- (11) Extension of Load Equivalency Values
- (12) Improved Traffic Data
- (13) Design of Pavements for Low Volume Roads
- (14) State of the Knowledge on Mechanistic-Empirical Design Concepts

The Task Force recognizes that a considerable body of information exists to design pavements utilizing so-called mechanistic models. It further believes that significant improvements in pavement design will occur as these mechanistic models are calibrated to in-service performance, and are incorporated in everyday design usage. Part IV of this document summarizes the mechanistic/empirical status.

In order to provide state-of-the-art approaches without lengthy research, values and concepts are shown that have limited support in research or experience. Each user should consider this to be a reference document and carefully evaluate his or her need of each concept and what initial values to use. To most effectively use the Guide it is suggested that the user adopt a process similar to the following:

- (1) Conduct a sensitivity study to determine which inputs have a significant effect on pavement design answers for its range of conditions.
- (2) For those inputs that are insignificant or inappropriate, no additional effort is required.
- (3) For those that are significant and the state has sufficient data or methods to estimate design values with adequate accuracy, no additional effort is required.
- (4) Finally, for those sensitive inputs for which the state has no data or methodology to develop the inputs, research will be necessary. Because of the complexity of pavement design and the large expansion of this Guide, it is anticipated that some additional research will be cost-effective for each and every user agency in order to optimally utilize the Guide.

One significant event, the pavement performance research effort being undertaken in the Strategic High-

way Research Program (SHRP), should aid greatly in improving this document.

The Task Force believes that pavement design is gradually, but steadily transitioning from an art to a science. However, when one considers the nebulous nature of such difficult, but important inputs to design considerations such as traffic forecasting, weather forecasting, construction control, maintenance practices, etc.; successful pavement design will always depend largely upon the good judgment of the designer.

Finally, the national trend toward developing and implementing pavement management systems, PMS, appears to the Task Force to be extremely important in developing the good judgment needed by pavement designers as well as providing many other elements needed for good design, i.e. information to support adequate funding and fund allocation.

The AASHTO Joint Task Force on Pavements

EXECUTIVE SUMMARY

One of the major objectives of the AASHO Road Test was to provide information that could be used to develop pavement design criteria and pavement design procedures. Accordingly, following completion of the Road Test, the AASHO Design Committee (currently the AASHTO Design Committee), through its Subcommittee on Pavement Design Practices, developed and circulated in 1961 the "AASHO Interim Guide for the Design of Rigid and Flexible Pavements." The Guide was based on the results of the AASHO Road Test supplemented by existing design procedures and, in the case of rigid pavements, available theory.

After the Guide had been used for several years, the AASHTO Design Committee prepared and AASHTO published the "AASHTO Interim Guide for Design of Pavement Structures—1972." Revisions were made in 1981 to Chapter III of the Guide relative to design criteria for Portland Cement Concrete pavements. Evaluation of the Guide by the AASHTO Design Committee in 1983 led to the conclusion that some revisions and additions were required. Representations from government, industry, consultants, and academia led to the conclusion that the Guide should be strengthened to incorporate information developed since 1972 and that a new section on rehabilitation should be added. It is also pertinent to note that, based on responses to a questionnaire sent to the States, there was an indication that the Guide was serving its main objectives and no serious problems were indicated. In other words, the States were generally satisfied with the Guide but acknowledged that some improvements could be made.

Based on the overall evaluation of input from user agencies and the status of research, it was determined by the AASHTO Joint Task Force on Pavements that the revisions to the Guide would retain the AASHO Road Test performance prediction equations, as modified for use in the 1972 Guide, as the basic model to be used for pavement design. This determination also established the present serviceability index (PSI) as the performance variable upon which design would be based.

The major changes which have been included in the revised Guide include the following considerations:

1. *Reliability.* The procedure for design of both rigid and flexible pavements provides a common method for incorporating a reliability factor into the design based on a shift in the design traffic.
2. *Soil support value.* AASHTO test method T 274 (resilient modulus of roadbed soils) is recommended as the definitive test for characterizing soil support. The soil property is recommended for use with both flexible and rigid pavement design.
3. *Layer coefficients (flexible pavements).* The resilient modulus test has been recommended as the procedure to be used in assigning layer coefficients to both stabilized and unstabilized material.
[NOTE: Guidelines for relating resilient modulus to soil support value and layer coefficients are provided in the Guide; however, user agencies are encouraged to obtain equipment and to train personnel in order to measure the resilient modulus directly.]
4. *Drainage.* Provision has been made in the Guide to provide guidance in the design of subsurface drainage systems and for modifying the design equations to take advantage of improvements in performance to good drainage.
5. *Environment.* Improvements in the Guide have been made in order to adjust designs as a function of environment, e.g., frost heave, swelling soils, and thaw-weakening. Major emphasis is given to thaw-weakening and the effect that seasonal variations have on performance.
6. *Tied shoulders and widened lanes (rigid pavements).* A procedure is provided for the design of rigid pavements with tied shoulders or widened outside lanes.
7. *Subbase erosion.* A method for adjusting the design equations to represent possible soil erosion under rigid pavements is provided.
8. *Life-cycle costs.* Information has been added relative to economic analysis and economic comparisons of alternate designs based on life-

- cycle costs. Present worth and/or equivalent uniform annual cost evaluations during a specified analysis period are recommended for making economic analyses.
9. *Rehabilitation.* A major addition to the Guide is the inclusion of a section on rehabilitation. Information is provided for rehabilitation with or without overlays.
 10. *Pavement management.* Background information is provided regarding pavement management and the role of the Guide in the overall scheme of pavement management.
 11. *Load equivalency values.* Load equivalency values have been extended to include heavier loads, more axles, and terminal serviceability levels of up to 3.0.
 12. *Traffic.* Extensive information concerning methods for calculating equivalent single axle loads and specific problems related to obtaining reliable estimates of traffic loading are provided.
 13. *Low-volume roads.* A special category for design of pavements subjected to a relative small number of heavy loads is provided in the design section.
 14. *Mechanistic-Empirical design procedure.* The state of the knowledge concerning mechanistic-empirical design concepts is provided in the Guide. While these procedures have not, as yet, been incorporated into the Guides, extensive information is provided as to how such methods could be used in the future when enough documentation can be provided.

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PART I PAVEMENT DESIGN AND MANAGEMENT PRINCIPLES

CHAPTER 1

INTRODUCTION AND BACKGROUND

1.1 SCOPE OF THE GUIDE

This *Guide for the Design of Pavement Structures* provides a comprehensive set of procedures which can be used for the design and rehabilitation of pavements; both rigid (portland cement concrete surface) and flexible (asphalt concrete surface) and aggregate surfaced for low-volume roads. The Guide has been developed to provide recommendations regarding the determination of the pavement structure as shown in Figure 1.1. These recommendations will include the determination of total thickness of the pavement structure as well as the thickness of the individual structural components. The procedures for design provide for the determination of alternate structures using a variety of materials and construction procedures.

A glossary of terms, as used in this Guide, is provided in Appendix A. It is recognized that some of the terms used herein may differ from those used in your local practice; however, it is necessary to establish standard terminology in order to facilitate preparation of the Guide for nationwide use. Insofar as is possible, AASHTO definitions have been used herein.

It should be remembered that the total set of considerations required to assure reliable performance of a pavement structure will include many factors other than the determination of layer thicknesses of the structural components. For example, material requirements, construction requirements, and quality control will significantly influence the ability of the pavement structure to perform according to design expectations. In other words, "pavement design" involves more than choosing thicknesses. Information concerning material and construction requirements will be briefly described in this Guide; however, a good pavement designer must be familiar with relevant publications of AASHTO and ASTM, as well as the local agencies, i.e., state agencies or counties, for whom the design is being prepared. It is extremely important that the designer prepare special provisions to the standard specifications when circumstances indicate that non-standard conditions exist for a specific project. Examples of such a condition could involve a roadbed soil which is known to be expansive or nonstandard mate-

rials which are to be stabilized for use in the pavement structure or prepared roadbed.

Part I of this Guide has been prepared as general background material to assist the user in the proper interpretation of the design procedures and to provide an understanding of the concepts used in the development of the Guide. Detailed information related directly to a number of design considerations, e.g., reliability, drainage, life-cycle costs, traffic, and pavement type selection, will be found in the Appendices. References used in the preparation of the Guide can be found following each of the four major Parts.

Part I, Chapter 3 of the Guide provides information concerning economic evaluation of alternate pavement design strategies. It should not be concluded that the selection of a pavement design should be based on economics alone. There are a number of considerations involved in the final design selection. Appendix B of the Guide on pavement type selection provides an extensive list of guidelines which should be used in comparing alternate design strategies.

Part II of this Guide provides a detailed method for the design of new pavements or for reconstruction of existing pavements on the existing alignment with new or recycled materials.

Part III of this Guide provides alternative methods for pavement rehabilitation with or without the addition of an overlay. The methodology used in this part of the Guide represents the state of the knowledge regarding the deterioration of a pavement structure before and after an overlay has been applied. It is recognized that there are alternate methods for the determination of overlay requirements; a number of these methods are cited in Appendix C. The method included in Part III is somewhat more basic in concept than other existing methods and has the capability for broader application to different types of overlays, e.g., flexible on rigid, flexible on flexible, rigid on rigid, and rigid on flexible type pavements. The method is also compatible with the performance and design concepts used in Part II. In this way, consideration of such factors as drainage, reliability, and traffic is the same for both new and rehabilitated (overlaid) pavement structures.

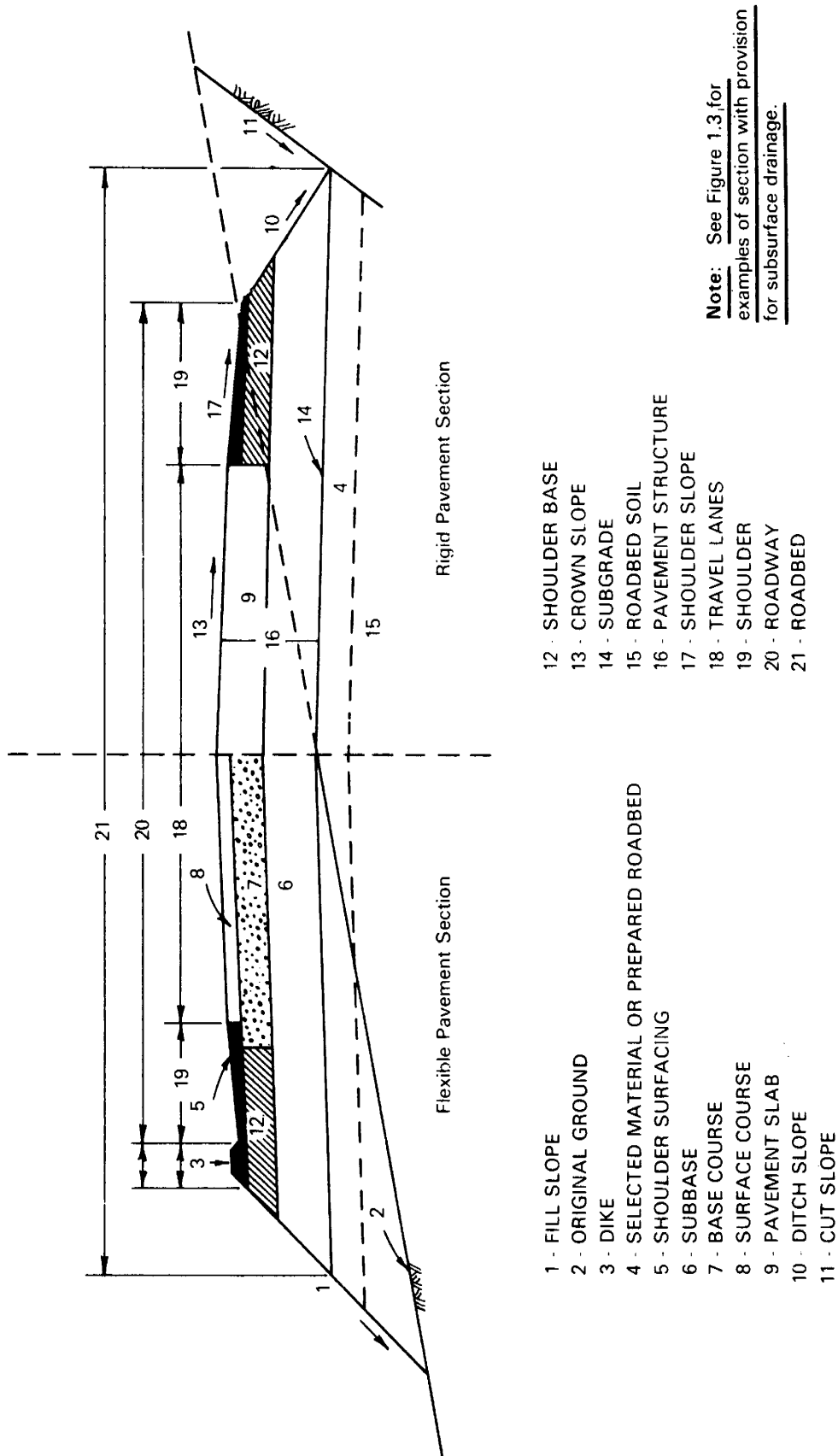


Figure 1.1. Typical Section for Rigid or Flexible Pavement Structure

State of the art procedures for rehabilitation of pavement structures without overlay, including drainage and the use of recycled material, are emphasized in Part III. These techniques represent an alternative to overlays which can reduce long-term costs and satisfy design constraints associated with specific design situations.

As an adjunct to pavement rehabilitation it is important to first determine what is wrong with the existing pavement structure. Details of the method for interpretation of the information are contained in Part III. A procedure for measuring or evaluating the condition of a pavement is given in Appendix K and Reference 1. It is beyond the scope of this Guide to discuss further the merits of different methods and equipment which can be used to evaluate the condition of a pavement. However, it is considered essential that a detailed condition survey be made before a set of plans and specifications are developed for a specific project. If at all possible, the designer should participate in the condition survey. In this way, it will be possible to determine if special treatments or methods may be appropriate for site conditions, specifically, if conditions warrant consideration of detailed investigations pertinent to the need for added drainage features.

Part IV of this Guide provides a framework for future developments for the design of pavement structures using mechanistic design procedures. The benefits associated with the development of these methods are discussed; a summary of existing procedures and a framework for development are the major concerns of that portion of the Guide.

1.2 DESIGN CONSIDERATIONS

The method of design provided in this Guide includes consideration of the following items:

- (1) pavement performance,
- (2) traffic,
- (3) roadbed soil,
- (4) materials of construction,
- (5) environment,
- (6) drainage,
- (7) reliability,
- (8) life-cycle costs, and
- (9) shoulder design.

Each of these factors is discussed in Part I. Parts II, III, and IV carry these concepts and procedures forward and incorporate each into a pavement structure design methodology.

It is worth noting again that while the Guide describes and provides a specific method which can be used for the determination of alternate design or rehabilitation recommendations for the pavement structure, there are a number of considerations which are left to the user for final determination, e.g., drainage coefficients, environmental factors, and terminal serviceability.

The Guide by its very nature cannot possibly include all of the site specific conditions that occur in each region of the United States. It is therefore necessary for the user to adapt local experience to the use of the Guide. For example, local materials and environment can vary over an extremely wide range within a state and between states.

The Guide attempts to provide procedures for evaluating materials and environment; however, in the case where the Guide is at variance with proven and documented local experience, the proven experience should prevail. *The designer will need to concentrate on some aspects of design which are not always covered in detail in the Guide.* For example, material requirements and construction specifications are not detailed in this Guide and yet they are an important consideration in the overall design of a pavement structure. The specifics of joint design and joint spacing will need careful consideration. The effect of seasonal variations on material properties and careful evaluation of traffic for the designed project are details which the designer should investigate thoroughly.

The basic design equations used for flexible and rigid pavements in this Guide are as follows:

Flexible Pavements

$$\begin{aligned} \log_{10}(W_{18}) = & Z_R \times S_o + 9.36 \times \log_{10}(SN + 1) \\ & - 0.20 + \frac{\log_{10} \left[\frac{\Delta PSI}{4.2 - 1.5} \right]}{0.40 + \frac{1094}{(SN + 1)^{5.19}}} \\ & + 2.32 \times \log_{10}(M_R) - 8.07 \quad (1.2.1) \end{aligned}$$

where

- W_{18} = predicted number of 18-kip equivalent single axle load applications,
 Z_R = standard normal deviate,
 S_o = combined standard error of the traffic prediction and performance prediction,

ΔPSI = difference between the initial design serviceability index, p_o , and the design terminal serviceability index, p_t , and
 M_R = resilient modulus (psi).

SN is equal to the structural number indicative of the total pavement thickness required:

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

where

a_i = i^{th} layer coefficient,
 D_i = i^{th} layer thickness (inches), and
 m_i = i^{th} layer drainage coefficient.

Rigid Pavements

$$\begin{aligned} \log_{10}(W_{18}) = & Z_R \times S_o + 7.35 \times \log_{10}(D + 1) \\ & - 0.06 + \frac{\log_{10} \left[\frac{\Delta\text{PSI}}{4.5 - 1.5} \right]}{1 + \frac{1.624 \times 10^7}{(D + 1)^{8.46}}} \\ & + (4.22 - 0.32 \times p_t) \\ & \times \log_{10} \left[\frac{S'_c \times C_d \times (D^{0.75} - 1.132)}{215.63 \times J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}} \right]} \right] \end{aligned} \quad (1.2.2)$$

where

W_{18} = predicted number of 18-kip equivalent single axle load applications,
 Z_R = standard normal deviate,
 S_o = combined standard error of the traffic prediction and performance prediction,
 D = thickness (inches) of pavement slab,
 ΔPSI = difference between the initial design serviceability index, p_o , and the design terminal serviceability index, p_t ,
 S'_c = modulus of rupture (psi) for portland cement concrete used on a specific project,
 J = load transfer coefficient used to adjust for load transfer characteristics of a specific design,
 C_d = drainage coefficient,

E_c = modulus of elasticity (psi) for portland cement concrete, and
 k = modulus of subgrade reaction (pci).

The design nomographs presented in Part II solve these equations for the structural number (SN) for flexible pavements and thickness of the pavement slab for rigid pavements.

The structural number is an abstract number expressing the structural strength of a pavement required for given combinations of soil support (M_R), total traffic expressed in equivalent 18-kip single axle loads, terminal serviceability, and environment. The required SN must be converted to actual thickness of surfacing, base and subbase, by means of appropriate layer coefficients representing the relative strength of the construction materials. Average values of layer coefficients for materials used in the AASHO Road Test are as follows:

Asphaltic concrete surface course—	.44
Crushed stone base course	— .14
Sandy gravel subbase	— .11

The layer coefficients given in Part II are based on extensive analyses summarized in NCHRP Report 128, "Evaluation of AASHTO Guide for Design of Pavement Structures," (1972). In effect, the layer coefficients are based on the elastic moduli M_R and have been determined based on stress and strain calculations in a multilayered pavement system. Using these concepts, the layer coefficient may be adjusted, increased, or decreased in order to maintain a constant value of stress or strain required to provide comparable performance.

Part II details how each of the design considerations are to be treated in selecting the SN value and how to decompose SN into layers according to material properties and function, i.e., surface, base, subbase, and so forth. The pavement slab thickness, in inches, is provided directly from the design nomographs.

It is important to recognize that equations (1.2.1) and (1.2.2) were derived from empirical information obtained at the AASHO Road Test. As such, these equations represent a best fit to observations at the Road Test. The solution represents the mean value of traffic which can be carried given specific inputs. In other words, there would be a 50-percent chance that the actual traffic to terminal serviceability could be more or less than predicted. In order to decrease the risk of premature deterioration below acceptable levels of serviceability, a reliability factor is included

in the design process. An explanation of the reliability factor is given in Chapter 4 of Part I. In order to properly apply the reliability factor, the inputs to the design equation should be the mean value without adjustment. This will be discussed further in Chapter 4 of Part I and in sections of Part II. *The designer must remember to use mean values for such factors as soil support, traffic, layer coefficients, drainage coefficients, etc.* Increased reliability will be obtained by adjustments which are based on uncertainty in each of the design variables as well as traffic.

Each of the terms used in the design equations is discussed as necessary in Parts I and II of this Guide. It is pertinent to note that a few changes have been made in the design equations when compared with the 1972 Interim Guide (2). The soil support value has been replaced with M_R (flexible) and a drainage coefficient has been added to the rigid equation. For the flexible equation, the structural number (SN) has been modified by the addition of drainage coefficients and the regional factor (R) has been deleted. Lastly, both the rigid and flexible equations have been modified to consider both total serviceability loss ($p_o - p_t$), and terminal serviceability.

There are two important factors to consider concerning these equations: (1) the equations are predictors of the amount of traffic that can be sustained before deteriorating to some selected terminal level of serviceability and (2) the basic prediction equations were developed empirically from field observations at the AASHO Road Test with modifications considered necessary to improve the Guide based on research completed during the past 20 years.

There are a number of alternate procedures which can be used for the design of pavement structures. In fact, all 50 states have adopted their own design procedures, many of which are based on past AASHTO Guide methods. A list of other suitable pavement design procedures is presented in Appendix C.

1.3 PAVEMENT PERFORMANCE

Current concepts of pavement performance include some consideration of functional performance, structural performance, and safety. This Guide is primarily concerned with functional and structural performance. Information pertinent to safety can be found in appropriate publications of NCHRP, FHWA, and AASHTO. One important aspect of safety is the frictional resistance provided at the pavement/tire interface. AASHTO has issued a publication, *Guidelines*

for Skid Resistant Pavement Design, which can be referred to for information on this subject.

The structural performance of a pavement relates to its physical condition; i.e., occurrence of cracking, faulting, raveling, or other conditions which would adversely affect the load-carrying capability of the pavement structure or would require maintenance.

The functional performance of a pavement concerns how well the pavement serves the user. In this context, riding comfort or ride quality is the dominant characteristic. In order to quantify riding comfort, the "serviceability-performance" concept was developed by the AASHO Road Test staff in 1957 (3, 4). Since the serviceability-performance concept is used as the measure of performance for the design equations in this Guide, an explanation of the concept herein seems worthwhile.

The serviceability-performance concept is based on five fundamental assumptions, summarized as follows (5):

- (1) Highways are for the comfort and convenience of the traveling public (User).
- (2) Comfort, or riding quality, is a matter of subjective response or the opinion of the User.
- (3) Serviceability can be expressed by the mean of the ratings given by all highway Users and is termed the serviceability rating.
- (4) There are physical characteristics of a pavement which can be measured objectively and which can be related to subjective evaluations. This procedure produces an objective serviceability index.
- (5) Performance can be represented by the serviceability history of a pavement.

The serviceability of a pavement is expressed in terms of the present serviceability index (PSI). The PSI is obtained from measurements of roughness and distress, e.g., cracking, patching and rut depth (flexible), at a particular time during the service life of the pavement. Roughness is the dominant factor in estimating the PSI of a pavement. Thus, a reliable method for measuring roughness is important in monitoring the performance history of pavements.

The specific equations developed at the Road Test to calculate the present serviceability index have been modified by most users of the AASHTO Guide. These changes reflect local experience and are assumed to represent results from the Road Test; i.e., the PSI values continue to represent ride quality as evaluated at the Road Test. Because of the relatively small contribution to PSI made by physical distress, and the difficulty in obtaining the information, many agencies

rely only on roughness to estimate ride quality. It is acknowledged that physical distress is likely to influence a decision to initiate maintenance or rehabilitation. For purposes of this Guide, it is assumed that the amount of distress associated with the terminal PSI is acceptable.

Because roughness is such an important consideration for the design of pavements, the change in roughness will control the life cycle of pavements. In this regard, the quality of construction will influence performance and the life cycle of the designed pavement. The initial pavement smoothness is an important design consideration. For example, the life cycle of a pavement initially constructed with a smoothness or PSI of 4.5 will have a significantly longer life cycle than one constructed to a PSI of 4.0. Thus, quality control in the construction of a pavement can have a beneficial impact on performance (life cycle).

The scale for PSI ranges from 0 through 5, with a value of 5 representing the highest index of serviceability. For design it is necessary to select both an initial and terminal serviceability index.

The initial serviceability index (p_i) is an estimate by the user of what the PSI will be immediately after construction. Values of p_i established for AASHTO Road Test conditions were 4.2 for flexible pavements and 4.5 for rigid pavements. Because of the variation of construction methods and standards, it is recommended that more reliable levels be established by each agency based on its own conditions.

The terminal serviceability index (p_t) is the lowest acceptable level before resurfacing or reconstruction becomes necessary for the particular class of highway. An index of 2.5 or 3.0 is often suggested for use in the design of major highways, and 2.0 for highways with a lower classification. For relatively minor highways, where economic considerations dictate that initial expenditures be kept low, at p_t of 1.5 may be used. Expenditures may also be minimized by reducing the performance period. Such a low value of p_t should only be used in special cases on selected classes of highways.

The major factors influencing the loss of serviceability of a pavement are traffic, age, and environment. Each of these factors has been considered in formulating the design requirements included in this Guide. However, it should be recognized that the separate or the interacting effects of these components are not clearly defined at the present time, especially with regard to age. It is known that the properties of materials used for pavement construction change with time. These changes may be advantageous to performance;

however, in most cases, age (time) is a net negative factor and works to reduce serviceability.

An effort has been made in the Guide to account for the effects of environment on pavement performance in situations where swelling clay or frost heave are encountered. Thus, the total change in PSI at any time can be obtained by summing the damaging effects of traffic, swelling clay, and/or frost heave, as shown in Equation 1.3.1 and illustrated in Figure 1.2.

$$\Delta \text{PSI} = \Delta \text{PSI}_{\text{Traffic}} + \Delta \text{PSI}_{\text{Swell/Frost Heave}} \quad (1.3.1)$$

where

ΔPSI = total loss of serviceability,

$\Delta \text{PSI}_{\text{Traffic}}$ = serviceability loss due to traffic (ESAL's), and

$\Delta \text{PSI}_{\text{Swell/Frost Heave}}$ = serviceability loss due to swelling and/or frost heave of roadbed soil.

It can be noted in Figure 1.2 that the effect of swelling soils or frost heave is to reduce the predicted service life of the pavement. The Guide does not recommend increasing pavement structural thickness to offset the serviceability loss due to swelling soils; but it is feasible, however, to control frost heave by increasing the thickness of non-frost-susceptible material.

In many swelling situations, it may be possible to reduce to acceptable limits the effect of swelling soil by stabilization of the expansive soil or by replacement of these soils with nonexpansive material. When experience indicates this is a viable procedure, it is not necessary to estimate the effect of swelling soil on the life cycle.

The predicted effect of frost heave is based on a limited amount of information available in the literature. If agency design procedures include provisions to mitigate the detrimental effects of frost, the serviceability loss due to frost heave should be ignored, i.e., assumed to be zero. The most accepted procedure to minimize the effect of frost heave is to replace the frost-susceptible material with non-frost-susceptible material to a depth of one-half or more of the frost depth.

A further discussion of the influence of environment will be found in Section 1.7 of this chapter.

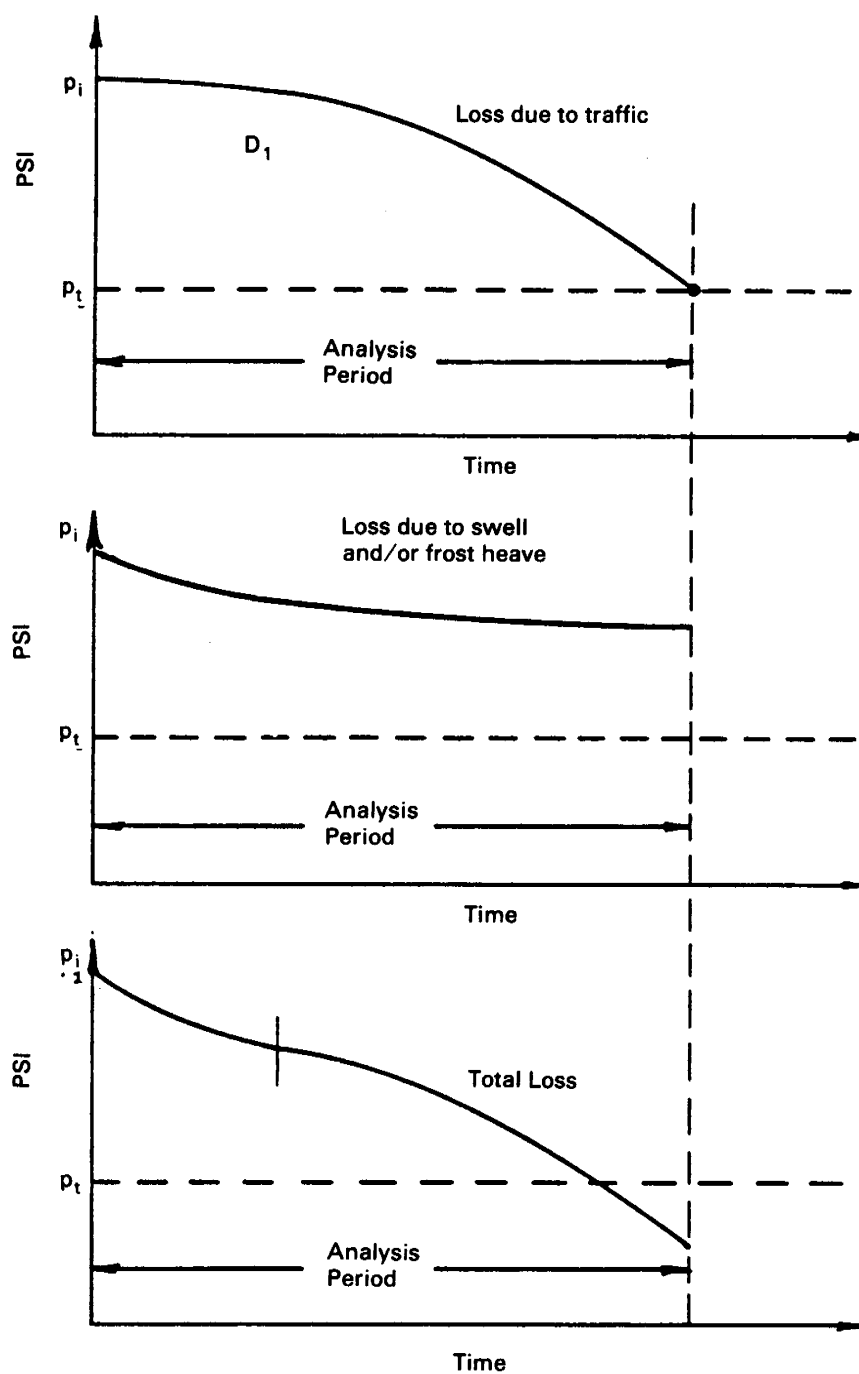


Figure 1.2. Pavement Performance Trends

1.4 TRAFFIC

Traffic information required by the design equations used in this Guide includes axle loads, axle configuration, and number of applications.

The results of the AASHO Road Test have shown that the damaging effect of the passage of an axle of any mass (commonly called load) can be represented by a number of 18-kip equivalent single axle loads or ESAL's. For example, one application of a 12-kip single axle was found to cause damage equal to approximately 0.23 applications of an 18-kip single axle load, and four applications of a 12-kip single axle were required to cause the same damage (or reduction in serviceability) as one application of an 18-kip single axle. This concept has been applied to the design equations and nomographs in Part II. The determination of design ESAL's is a very important consideration for the design of pavement structures using this Guide, as it is in previous versions of the Guide.

1.4.1 Evaluation of Traffic

The procedure used in this Guide to convert a mixed traffic stream of different axle loads and axle configurations into a design traffic number is to convert each expected axle load into an equivalent number of 18-kip single axle loads and to sum these over the design period. The procedure for converting mixed traffic to ESAL's is discussed in Appendix D.

There are four key considerations which influence the accuracy of traffic estimates and which can significantly influence the life cycle of a pavement: (1) the correctness of the load equivalency values used to estimate the relative damage induced by axle loads of different mass and configurations, (2) the accuracy of traffic volume and weight information used to represent the actual loading projections, (3) the prediction of ESAL's over the design period, and (4) the interaction of age and traffic as it affects changes in PSI.

The available load equivalency factors are considered the best available at the present time, representing information derived from the AASHO Road Test. The empirical observations on the Road Test covered a range of axle loads from 2 to 30 kips on single axles and 24 to 48 kips on tandem axles. No tridem axles were included in the Road Test experiment; load equivalency values for tridem axles are included in Appendix D, but they are the result of research carried out since completion of the Road Test. Load equivalency values for single and tandem axles which exceed

the loads given above are also extrapolations of the basic data from the Road Test.

It should be noted that load equivalency factors are, to a minor degree, functions of pavement type (rigid or flexible), thickness, and terminal serviceability (p_t) used for design. For designing composite pavements (rigid base with flexible wearing surface), the use of load equivalency values for rigid pavements is recommended.

State DOT's accumulate traffic information in the format of the Federal Highway Administration W-4 truck weight tables, which are tabulations of the number of axles observed within a series of load groups with each load group covering a 2-kip interval. Traffic information relative to truck type, i.e., axle configuration, is provided in W-2 tabulations (distribution of vehicles counted and weighed). As illustrated in Appendix D, these tabulations can be used to estimate the number of equivalent single axle loads associated with mixed traffic at the particular reporting loadometer station. From this information it is possible to obtain average load equivalency factors for all trucks or for trucks by configuration, i.e., the averages for singles, tandems, or tridems.

Most states have taken the information from the W-4 tables and converted it into relatively simple multipliers (truck equivalency factors) which represent each truck type in the traffic stream. These multipliers can be used to convert mixed streams of traffic to ESAL's. It must be realized that such conversions represent estimates when applied to highways other than those from which the data were obtained. Weigh station information represents only a sample of the total traffic stream with weighings at a limited number of locations and for limited periods of time. Such information must be carefully interpreted when applied to specific projects. Results from different weigh stations in one state have been reported to produce truck factors which vary by a factor of 6. Thus, one source of error in ESAL predictions is the use of estimated truck equivalency factors for various classes of highways based on a relatively small sample. Increased sampling of this type of information is necessary in order to reduce the error of the estimate due to insufficient information on a specific project. Users of this Guide are urged to gather the best possible traffic data for each design project.

Since pavements, new or rehabilitated, are usually designed for periods ranging from 10 years to 20 years or more, it is necessary to predict the ESAL's for this period of time, i.e., the performance period. The performance period, often referred to as the design period, is defined as the period of time that an initial

(or rehabilitated structure) will last before reaching its terminal serviceability. Any performance period may be used with the Guide since design is based on the total number of equivalent single axle loads; however, experience may indicate a practical upper limit based on considerations other than traffic. The ESAL's for the performance period represent the cumulative number from the time the roadway is opened to traffic to the time when the serviceability is reduced to a terminal value (e.g., p_t equal 2.5 or 2.0). If the traffic is underestimated, the actual time to p_t will probably be less than the predicted performance period, thereby resulting in increased maintenance and rehabilitation costs.

The maximum performance period to be used in designing for a particular pavement type, i.e., flexible, rigid, or composite, should reflect agency experience.

The performance period and corresponding design traffic should reflect real-life experience. The performance period should not be confused with pavement life. The pavement life may be extended by periodic rehabilitation of the surface or pavement structure.

The equivalent loads derived from many traffic prediction procedures represent the totals for all lanes for both directions of travel. This traffic must be distributed by direction and by lanes for design purposes. Directional distribution is usually made by assigning 50 percent of the traffic to each direction, unless available measured traffic data warrant some other distribution. In regard to lane distribution, 100 percent of the traffic in one direction is often assigned to each of the lanes in that direction for purposes of structural design if measured distributions are not available. Some states have developed lane distribution factors for facilities with more than one lane in a given direction. These factors vary from 60 to 100 percent of the one-directional traffic, depending on the total number of lanes in the facility. Part II and Appendix D provide more details pertinent to this lane distribution factor.

Traffic information is often provided to the designer by a Planning or Traffic group. The designers should work closely with traffic personnel to be sure the proper information is provided and that the consequences of poor estimates of present and future traffic are understood by all personnel involved.

Predictions of future traffic are often based on past traffic history. Several factors can influence such predictions.

For purposes of pavement structure design, it is necessary to estimate the cumulative number of 18-kip equivalent single axle loads (ESAL's) for the design

(performance) period. The number of ESAL's may or may not be proportional to the average daily traffic. Truck traffic is the essential information required to calculate ESAL's; it is therefore very important to correctly estimate future truck traffic for the facility during the design period.

Traffic may remain constant or increase according to a straight line or at an accelerating (exponential) rate. In most cases, highways classified as principal arterial or interstate will have exponential growth (comparable to compound interest on investments). Traffic on some minor arterial or collector-type highways may increase along a straight line, while traffic on some residential streets may not change because the use remains constant. Thus, the designer must make provision for growth in traffic from the time of the last traffic count or weighing through the performance period selected for the project under consideration. Appendix D provides appropriate information for estimating future traffic growth based on an assumed exponential compounded growth rate. If zero or negative growth in traffic is anticipated, a zero or negative growth factor can be used. In most cases, appropriate growth factors can be selected from the table in Appendix D. For major arterials and interstate highways, the growth rate should be applied by truck class rather than to the total traffic since growth in truck traffic may differ from the total traffic stream.

The percent trucks for the design period is often assumed to be constant; yet on some sections of the interstate system, the truck traffic in rural areas has been reported to increase from an estimated 6 percent to 25 to 30 percent over a 10- to 20-year period.

The load equivalency factor increases approximately as a function of the ratio of any given axle load to the standard 18-kip single axle load raised to the fourth power. For example, the load equivalency of a 12-kip single axle is given as 0.19 (Appendix D), while the load equivalency for 20-kip single axle is 1.51. Thus, the 20-kip load is 8 times as damaging as the 12-kip load, i.e., $(20/12)^4$. This relationship will vary depending on the structural number and terminal serviceability; however, it is generally indicative of load effects. Thus, it is especially important to obtain reliable truck weight information for each truck class and especially for the multi-axle trucks since these vehicles will constitute a high percentage of the total ESAL's on most projects.

Calculation of future ESAL's is often based on truck factors by truck class. For example, based on truck weight information for five-axle tractor and trailer units, it is possible to develop an average multiplier for each five-axle truck. Thus, if the designer

can estimate the number of five-axle trucks over the design period, it is possible to calculate the cumulative ESAL's due to this particular truck class. A similar procedure is described in Appendix D for most of the truck classes on the highways at the present time.

In regard to the use of truck factors, it will be important to use truck weight information representative of the truck traffic on the designed facility. Some truck weight data indicate that truck weights can vary by a factor of six or more between weigh stations. Thus, it is very important to obtain information as nearly site specific as possible when estimating ESAL's per truck for each truck classification.

Procedures described in Appendix D may be applied to stage-construction design, i.e., where the initial design (performance) period is varied in order to consider alternative designs for economic comparisons.

It should be clear from this discussion that the estimate or prediction of future traffic (ESAL's) is not a trivial problem. Poor estimates of traffic can produce pavement performance significantly different than that expected and cause a major increase in the cost of the specific project. This increased cost, when applied to all sections being designed by an agency, will adversely affect the overall programming of highway projects and reduce the work which can be done.

Future deregulation or relaxation of truck loads could also result in changes in the load distributions by truck class, possibly resulting in an increased percentage of five-axle (or more) vehicles being used. Also, inflation pressures used in truck tires are increasing as tire manufacturers improve their technology and the truck industry evaluates the potential advantage of using higher inflation tires. It is not known exactly what the net effect of higher tire inflation is; however, pavement engineers and designers need to keep apprised of possible changes which can influence pavement performance.

In summary, reliable information concerning cumulative ESAL's is important for the determination of pavement structure requirements for both new construction and for rehabilitation. Continuous monitoring of traffic on selected routes to compare predicted and actual traffic loadings is an important and vital set of information needed to produce reliable designs.

The reliability factor included in the Guide (Part I, Chapter 4 and Volume 2, Appendix EE) has been developed to provide consideration of uncertainties in both traffic predictions and performance predictions. Investigations by several states and industry have provided some information concerning the uncertainties in traffic predictions, i.e., comparison of predicted

ESAL's and actual ESAL's. The standard deviation of the relationship between predicted and actual traffic has been reported (27) to be on the order of 0.2. In effect, the actual traffic may be 1.6 (one standard deviation) to 4.0 times (three standard deviations) as much as predicted. It should be clear that improvements in traffic loading information and predictions will contribute significantly to the precision which can be achieved in thickness design.

Detailed information and procedures for calculating ESAL's are given in Appendix D. Designs in Part II take into consideration the uncertainty in traffic estimates. The designer must use the best estimate for traffic without any adjustment based on his or her interpretation of the accuracy of such information. Provision has been made in the treatment of reliability in Part II to accommodate the overall effect of variances in the cumulative axle load predictions and other design- and performance-related factors.

1.4.2 Limitations

It is pertinent to note that the load equivalency factors used in this Guide are based on observations at the AASHO Road Test in Ottawa, Illinois. In this regard some limitations should be recognized, such as (1) limited pavement types, (2) loads and load applications, (3) age, and (4) environment.

The pavement types at the AASHO Road Test, from which load equivalency values were derived, included conventional flexible construction, i.e., surface, base and subbase, and rigid pavements with and without reinforcement but always with load transfer devices (dowels). The same load equivalency factors are being applied in this Guide to (1) flexible pavements with stabilized base and subbase, (2) rigid pavements without dowels in the transverse joints, and (3) continuously reinforced concrete pavements. Modifications to the load equivalency values can only come through controlled experiments. The values used in this Guide are considered the best available at the present time.

The experimental design at the AASHO Road Test included a wide range of loads as previously discussed (Section 1.4.1); however, the applied loads were limited to a maximum of 1,114,000 axle applications for those sections which survived the full trafficking period. Thus, the maximum number of 18-kip equivalent single axle loads (ESAL's) applied to any test section was approximately one million. However, by applying the concept of equivalent loads to test sections subjected to only 30-kip single axle loads, for example, it

is possible to extend the findings to 8×10^6 ESAL's. Use of any design ESAL's above 8×10^6 requires extrapolation beyond the equations developed from the Road Test results. Such extrapolations have, however, provided reasonable results, based on application of the Guide since 1972.

The AASHO Road Test, from which the basic design equations were derived, was completed after 2 years of traffic testing. The prediction models represented by equations (1.2.1) and (1.2.2) do not include a term for age, i.e., an interactive term for age and traffic. For the present state of knowledge there is very little information available to quantify the effect of aging on performance as expressed in terms of PSI or axle load applications. There is a need for more information regarding the combined effect of traffic and aging on performance. If a user agency has such information it may be possible to modify the performance model accordingly. However, this Guide makes no direct evaluation of aging effects. Evaluation of aging factors along with traffic (ESAL's) should be a high priority for long-term monitoring of pavement performance.

Only one set of materials and one roadbed soil were included in the AASHO Road Test for each pavement type. A small experiment also included performance observations of stabilized base materials under asphaltic surfaces. Use of alternate construction materials represents an extrapolation of the basic data. However, as previously indicated, such extrapolations are based on investigations using analytical techniques and are considered reasonable pending results from field investigations.

The weather at the Road Test in Ottawa, Illinois, is representative of a large portion of the United States, subject to freezing temperatures during the winter and medium to high rainfall throughout the year. An effort has been made in Part II of this Guide to provide a procedure for estimating the effects of seasonal conditions and modifying these for site specific locations. More information on environment is provided in a later section of Part I as well as in Part II of the Guide.

A number of new concepts have been included in these Guides, e.g., reliability, drainage coefficients, use of resilient modulus to estimate layer coefficients, remaining life estimates for overlays, and NDT methods to estimate in situ resilient modulus. These concepts have limited documentation based on actual field observations; however, they are based on an extensive evaluation of the present state of the knowledge. To the extent possible, explanations are provided in the Guide in either this volume or Volume 2. It is hoped that these concepts will find sufficient usage in order

to evaluate and eventually modify and improve the design procedures and effectiveness of using the Guide.

1.4.3 Special Cases

This Guide is based on performance equations from the AASHO Road Test which may not apply directly to some urban streets, county roads, parkways, or parking lots. For city streets, the major traffic loads will be generated by service vehicles, buses, and delivery trucks. Load equivalency values for such vehicles are not generally well-estimated by truck load equivalency factors from truck weighing stations. If the Guide is used for design of urban streets, an effort should be made to obtain information on actual axle loads and frequencies typical of vehicles operating on those streets. If this is done, the Guide can be used at a selected level of reliability.

For parkways, i.e., highways which limit the use of heavy trucks, it may be necessary to adjust the design based on a combination of traffic factors, environmental factors, and experience. Use of load equivalency factors as given in Appendix D may result in an under-designed pavement and premature deterioration.

1.5 ROADBED SOIL

The definitive material property used to characterize roadbed soil for pavement design in this Guide is the resilient modulus (M_R). The procedure for determination of M_R is given in AASHTO Test Method T 274.

The resilient modulus is a measure of the elastic property of soil recognizing certain nonlinear characteristics. The resilient modulus can be used directly for the design of flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid or composite pavements. Direct measurement of subgrade reaction can be made if such procedures are considered preferable to the design agency.

The resilient modulus was selected to replace the soil support value used in previous editions of the Design Guide for the following reasons:

- (1) It indicates a basic material property which can be used in mechanistic analysis of multi-layered systems for predicting roughness, cracking, rutting, faulting, etc.

- (2) Methods for the determination of M_R are described in AASHTO Test Method T 274.
- (3) It has been recognized internationally as a method for characterizing materials for use in pavement design and evaluation.
- (4) Techniques are available for estimating the M_R properties of various materials in-place from nondestructive tests.

It is recognized that many agencies do not have equipment for performing the resilient modulus test. Therefore, suitable factors are reported which can be used to estimate M_R from standard CBR, R-value, and soil index test results or values. The development of these factors is based on state of the knowledge correlations. It is strongly recommended that user agencies acquire the necessary equipment to measure M_R . In any case, a well-planned experiment design is essential in order to obtain reliable correlations. A range of soil types, saturation, and densities should be included in the testing program to identify the main effects. Guidelines for converting CBR and R-value to M_R are discussed in this chapter. These correlations are used in Part II of this Guide pending the establishment of agency values.

Heukelom and Klomp (6) have reported correlations between the Corps of Engineers CBR value, using dynamic compaction, and the in situ modulus of soil. The correlation is given by the following relationship:

$$M_R(\text{psi}) = 1,500 \times \text{CBR} \quad (1.5.1)$$

The data from which this correlation was developed ranged from 750 to 3,000 times CBR. This relationship has been used extensively by design agencies and researchers and is considered reasonable for fine-grained soil with a soaked CBR of 10 or less. Methods for testing are given in Appendix F. The CBR should correspond to the expected field density.

Similar relationships have also been developed by the Asphalt Institute (7) which relate R-value to M_R as follows:

$$M_R(\text{psi}) = A + B \times (\text{R-value}) \quad (1.5.2)$$

where

$$\begin{aligned} A &= 772 \text{ to } 1,155 \text{ and} \\ B &= 369 \text{ to } 555. \end{aligned}$$

For the purposes of this Guide, the following correlation may be used for fine-grained soils (R-value less than or equal to 20) until designers develop their own capabilities:

$$M_R = 1,000 + 555 \times (\text{R-value}) \quad (1.5.3)$$

This discussion summarizes estimates for converting CBR and R-values to a resilient modulus for roadbed soil. Similar information is provided for granular materials in Section 1.6, Materials of Construction.

Placement of roadbed soil is an important consideration in regard to the performance of pavements. In order to improve the general reliability of the design, it is necessary to consider compaction requirements. For average conditions it is not necessary to specify special provisions for compaction. However, there are some situations for which the designer should request modifications in the specifications.

- (1) The basic criteria for compaction of roadbed soils should include an appropriate density requirement. Inspection procedures must be adequate to assure that the specified density is attained during construction. If, for any reason, the basic compaction requirements cannot be met, the designer should adjust the design M_R value accordingly.
- (2) Soils that are excessively expansive or resilient should receive special consideration. One solution is to cover these soils with a sufficient depth of selected material to modify the detrimental effects of expansion or resilience. Expansive soils may often be improved by compaction at water contents of 1 or 2 percent above the optimum. In some cases it may be more economical to treat expansive or resilient soils by stabilizing with a suitable admixture, such as lime or cement, or to encase a substantial thickness in a waterproof membrane to stabilize the water content. Information concerning expansive soil is covered in Reference 8. Methods for evaluating the potential consequences of expansive roadbed soils are provided in Appendix G.
- (3) In areas subject to frost, frost-susceptible soils may be removed and replaced with selected, nonsusceptible material. Where such soils are too extensive for economical removal, they may be covered with a sufficient depth of suitable material to modify the detrimental effects of freezing and thawing. Methods for evaluat-

ing the consequences of frost heave are provided in Appendix G and have been reviewed previously in this chapter. Methods for compensating for seasonal thaw-weakening are provided in Part II.

- (4) Problems with highly organic soils are related to their extremely compressible nature and are accentuated when deposits are nonuniform in properties or depth. Local deposits, or those of relatively shallow depth, are often most economically excavated and replaced with suitable select material. Problems associated with deeper and more extensive deposits have been alleviated by placing surcharge embankments for preconsolidation, sometimes with special provisions for rapid removal of water to hasten consolidation.
- (5) Special provisions for unusually variable soil types and conditions may include: scarifying and recompact; treatment of an upper layer of roadbed soils with a suitable admixture; using appreciable depths of more suitable roadbed soils (select or borrow); over-excavation of cut sections and placing a uniform layer of selected material in both cut-and-fill areas; or adjustment in the thickness of subbase at transitions from one soil type to another.
- (6) Although the design procedure is based on the assumption that provisions will be made for surface and subsurface drainage, some situations may require that special attention be given to design and construction of drainage systems. Drainage is particularly important where heavy flows of water are encountered (i.e., springs or seeps), where detrimental frost conditions are present, or where soils are particularly susceptible to expansion or loss of strength with increase in water content. Special subsurface drainage may include provision of additional layers of permeable material beneath the pavement for interception and collection of water, and pipe drains for collection and transmission of water. Special surface drainage may require such facilities as dikes, paved ditches, and catch-basins.
- (7) Certain roadbed soils pose difficult problems in construction. These are primarily the cohesionless soils, which are readily displaced under equipment used to construct the pavement, and wet clay soils, which cannot be compacted at high water contents because of displacement under rolling equipment and which require long periods of time to dry to a suitable water

content. Measures used to alleviate such construction problems include: (1) blending with granular materials, (2) adding suitable admixtures to sands to provide cohesion, (3) adding suitable admixtures to clays to hasten drying or increase shear strength, and (4) covering with a layer of more suitable selected material to act as a working platform for construction of the pavement.

Resilient Modulus (M_R) values for pavement structure design should normally be based on the properties of the compact layer of the roadbed soil. It may, in some cases, be necessary to include consideration of the uncompacted foundation if these in situ materials are especially weak. It is important to note that the design of the pavement structure by this Guide is based on the *average* M_R value. Although reliability considers the variation of many factors associated with design, it is treated by adjusting the design traffic. (See Chapter 4.) The design traffic is the expected value of 18-kip ESAL's during the design period. The designer *must not* select a design M_R value based on some minimum or conservative criteria as this will introduce increased conservatism in design beyond that provided by the reliability factor.

1.6 MATERIALS OF CONSTRUCTION

Materials used for construction of the pavement structure can be divided into two general classes; (1) those for flexible pavements and (2) those for rigid pavements. Materials used for composite pavements include those for roadbed preparation, for a subbase, and for a portland cement concrete slab with an asphalt concrete wearing surface. An asphalt concrete overlay on a rigid pavement is considered a composite pavement.

In order to complete the design requirements for flexible pavements, it may be necessary to convert CBR or R-value information to resilient modulus, M_R . In the absence of agency correlations, the following correlations are provided for unbound granular materials (base and subbase):

θ (psi)	M_R (psi)
100	$740 \times \text{CBR or } 1,000 + 780 \times R$
30	$440 \times \text{CBR or } 1,000 + 450 \times R$
20	$340 \times \text{CBR or } 1,000 + 350 \times R$
10	$250 \times \text{CBR or } 1,000 + 250 \times R$

where θ = sum of the principal stresses, $\sigma_1 + \sigma_2 + \sigma_3$; referring to AASHTO T 274, this corresponds to $\sigma_d + 3\sigma_3$ when $\sigma_d = \sigma_1 - \sigma_3$.

The strength of the granular base or subbase is related to the stress state which will occur under operating conditions. The sum of the principal stresses, θ , is a measure of the stress state, which is a function of pavement thickness, load, and the resilient modulus of each layer. As an agency becomes increasingly familiar with these parameters, it will be possible to determine the stress state from a layered system analysis following procedures given in Part IV of the Guide. However, if such information is not available, estimates of resilient modulus values provided in Part II of this Guide may be used.

1.6.1 Flexible Pavements

As shown in Figure 1.1, flexible pavements generally consist of a prepared roadbed underlying layers of subbase, base, and surface courses. In some cases the subbase and/or base will be stabilized to maximize the use of local materials. The engineering literature contains a good deal of information relative to soil and aggregate stabilization (9, 10).

References 9 and 10 provide a state of the knowledge description of procedures for selecting the stabilizing agents appropriate to various soil types and construction methods. Pavement design examples in Reference 9 refer to the 1972 Interim Guide; however, the examples can still be used to illustrate design concepts appropriate for use with stabilized materials.

Prepared Roadbed. The prepared roadbed is a layer of compacted roadbed soil or select borrow material which has been compacted to a specified density.

Subbase Course. The subbase course is the portion of the flexible pavement structure between the roadbed soil and the base course. It usually consists of a compacted layer of granular material, either treated or untreated, or of a layer of soil treated with a suitable admixture. In addition to its position in the pavement, it is usually distinguished from the base course material by less stringent specification requirements for strength, plasticity, and gradation. The subbase material should be of significantly better quality than the roadbed soil. For reasons of economy, the subbase is often omitted if roadbed soils are of high quality.

When roadbed soils are of relatively poor quality and the design procedure indicates that a substantial thickness of pavement is required, several alternate designs should be prepared for structural sections with and without subbase. The selection of an alternate may then be made on the basis of availability and relative costs of materials suitable for base and subbase. Because lower quality materials may be used in the lower layers of a flexible pavement structure, the use of a subbase course is often the most economical solution for construction of pavements over poor roadbed soils.

Although no specific quality requirements for subbase material are presented in this Guide, the *AASHTO Construction Manual for Highway Construction* can be used as a guide. Many different materials have been used successfully for subbase. Local experience can be used as the basis for selection. For use in this design procedure, subbase material, if present, requires the use of a layer coefficient (a_3), in order to convert its actual thickness to a structural number (SN). Special consideration must be given to determining the minimum thickness of base and surfacing required over a given subbase material. Procedures that may be used for this purpose are given in Part II. Procedures for assigning appropriate layer coefficients based on expected M_R are given in Part II.

Untreated aggregate subbase should be compacted to 95 percent of maximum laboratory density, or higher, based on AASHTO Test T 180, Method D, or the equivalent. In addition to the major function as a structural portion of the pavement, subbase courses may have additional secondary functions, such as:

- (1) Preventing the intrusion of fine-grained roadbed soils into base courses—relatively dense-graded materials must be specified if the subbase is intended to serve this purpose.
- (2) Minimize the damaging effects of frost action—materials not susceptible to detrimental frost action must be specified if the subbase is intended for this purpose.
- (3) Preventing the accumulation of free water within or below the pavement structure—a relatively free-draining material may be specified for the subbase if this is the intention. Provisions must also be made for collecting and removing the accumulated water from the subbase if this layer is to be included as part of the drainage system. If the subbase is to be designed as a drainage layer, it will be necessary to limit the fraction passing the No. 8 sieve to a very small percent.

- (4) Providing a working platform for construction equipment—important when roadbed soil cannot provide the necessary support.

Base Course. The base course is the portion of the pavement structure immediately beneath the surface course. It is constructed on the subbase course, or, if no subbase is used, directly on the roadbed soil. Its major function in the pavement is structural support. It usually consists of aggregates such as crushed stone, crushed slag, crushed gravel and sand, or combinations of these materials. It may be used untreated or treated with suitable stabilizing admixtures, such as portland cement, asphalt, lime, cement-flyash and lime-flyash, i.e., pozzolonic stabilized bases. Specifications for base course materials are generally considerably more stringent than for subbase materials in requirements for strength, plasticity, and gradation. Guidelines for stabilization can be found in References 9 and 10.

When utilizing pozzolonic stabilized bases under a relatively thin asphaltic wearing surface, it can usually be expected that uncontrolled transverse reflection cracks will occur in the surface in a relatively short period of time, e.g., 1 to 3 years. Sawed and sealed joints (through the asphalt concrete into the base) may be utilized to minimize the adverse effects on appearance and to provide for better future sealing operations. Joint spacing may vary from 20 to 40 feet depending on local experience with past uncontrolled crack-spacing problems.

Although no specific quality requirements for base courses are presented in this Guide, the specifications included in AASHTO's *Manual for Highway Construction* or in ASTM Specification D 2940, "Graded Aggregate Material for Bases or Subbase for Highways and Airports," are often used. Materials varying in gradation and quality from these specifications have been used in certain areas and have provided satisfactory performance. Additional requirements for quality of base materials, based on test procedures used by the constructing agency, may also be included in materials or construction specifications.

Untreated aggregate base should be compacted to at least 95 percent of maximum laboratory density based on AASHTO Test T 180, Method D, or the equivalent. A wide variety of materials unsuitable for use as untreated base course have given satisfactory performance when improved by addition of a stabilizing admixture, such as portland cement, asphalt, or lime. Consideration should be given to the use of such treated materials for base courses whenever they are economically feasible, particularly when suitable un-

treated materials are in short supply. Economic advantages may result not only from the use of low-cost aggregates but also from possible reduction in the total thickness of the pavement structure that may result from the use of treated materials. Careful study is required in the selection of the type and amount of admixture to be used for optimum performance and economy.

For use in this design procedure, base material must be represented by a layer coefficient (a_2) in order that its actual thickness may be converted to a structural number. Procedures for the determination of layer coefficients based on M_R are given in Part II.

Drainage Layer. A number of agencies are now considering or constructing pavements with a drainage course, or layer, as shown in Figure 1.3 (11). Figure 1.3 illustrates one configuration; alternate designs are shown in Appendix AA of Volume 2 and in References 12 and 13.

The cross section shown in Figure 1.3 is illustrative only. The location of the longitudinal drain with respect to the traveled way can vary depending on designer preference and local experience. Also, this figure does not show the collector systems and outlet requirements for a total drainage design. Reference should be made to Appendix AA of Volume 2 and References 11, 12, 13, 22, and 23 for additional information regarding the design of drainage systems.

The designer should give some consideration to the preferred construction sequence when specifying a drainage system, e.g., excavation and installation after the travel lane paving has been completed. Local practice should be followed; however, the designer should be aware that special provisions to the specifications may be necessary. Additional information concerning the design of the drainage layer is provided in Section 1.8 of Part I and in Appendix AA of Volume 2.

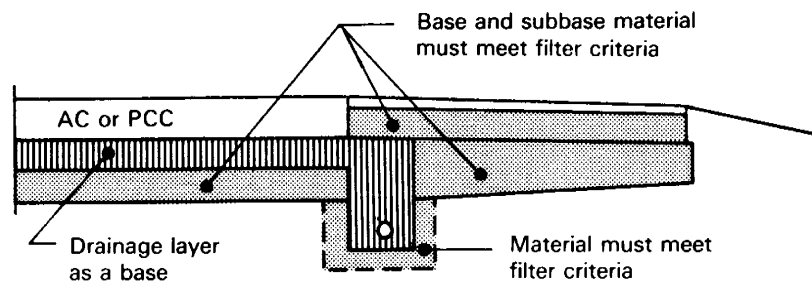
Tables 1.1, 1.2, and 1.3 provide some background information for estimating the permeability of various types of material.

Table 1.1 provides general relationships between coarse-graded unstabilized materials and their coefficients of permeability (11).

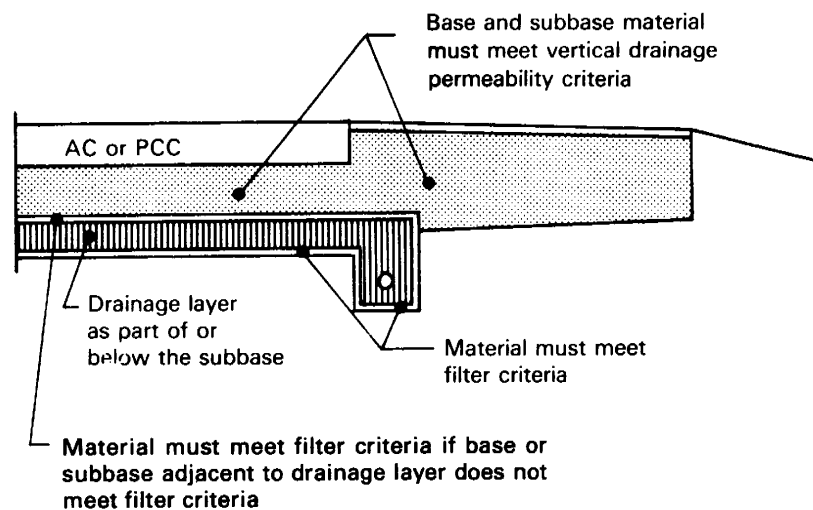
Table 1.2 provides guidelines for the gradation of asphalt-treated permeable material (11). At least one state agency has reported the same gradation for porous concrete used as a drainage layer.

Table 1.3 summarizes information relative to the permeability of graded aggregates as a function of the percent passing the No. 200 mesh sieve. Additional information concerning materials to be used for the drainage course is provided in Reference 12.

A. Base is used as the drainage layer.



B. Drainage layer is part of or below the subbase.



Note: Filter fabrics may be used in lieu of filter material, soil, or aggregate, depending on economic considerations.

Figure 1.3. Example of Drainage Layer in Pavement Structure (11)

Table 1.1. Permeability of Graded Aggregates (11)

Percent Passing	Sample Number					
	1	2	3	4	5	6
3/4-inch sieve	100	100	100	100	100	100
1/2-inch sieve	85	84	83	81.5	79.5	75
3/8-inch sieve	77.5	76	74	72.5	69.5	63
No. 4 sieve	58.5	56	52.5	49	43.5	32
No. 8 sieve	42.5	39	34	29.5	22	5.8
No. 10 sieve	39	35	30	25	17	0
No. 20 sieve	26.5	22	15.5	9.8	0	0
No. 40 sieve	18.5	13.3	6.3	0	0	0
No. 60 sieve	13.0	7.5	0	0	0	0
No. 140 sieve	6.0	0	0	0	0	0
No. 200 sieve	0	0	0	0	0	0
Dry density (pcf)	121	117	115	111	104	101
Coefficient of permeability (ft. per day)	10	110	320	1,000	2,600	3,000

NOTE: Subsurface drainage systems should be capable of removing.

The approximate coefficient of permeability of the asphalt-treated permeable material is 3,000 feet or more per day when treated with 2-percent asphalt and 8,000 feet per day with no asphalt.

Table 1.2. Gradation for Asphalt Treated Permeable Layer (11)

Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	30-50
No. 4	0-5
No. 8	0-2

Table 1.3. Effect of Percentage Passing 200 Mesh Sieve on Coefficient of Permeability of Dense Graded Aggregate, Feet Per Day (11)

Types of Fines	Percent Passing No. 200 Sieve			
	0	5	10	15
Silica or limestone	10	0.07	0.08	0.03
Silt	10	0.08	0.001	0.0002
Clay	10	0.01	0.0005	0.00009

Specifications, for both design and construction, of drainage courses are under development; hence, material requirements should be referenced to the latest guide specifications of AASHTO, ASTM, or the appropriate state agency responsible for developing statewide criteria and requirements. Information in Tables 1.1, 1.2, and 1.3 provides some guidelines for estimating permeability.

The N.J. Department of Transportation has developed specifications for bituminous stabilized and non-stabilized open-graded mixes for drainage layers. The gradation requirements used by the NJDOT are:

Sieve Size	Percent Passing
1.5 in.	100
1.0 in.	95-100
0.5 in.	60-80
No. 4	40-55
No. 8	5-25
No. 16	0-8
No. 50	0-5

This material can be made with a 50/50 blend of No. 57 and No. 9 stone of a crushed stone. The target permeability suggested by NJDOT is 1,000-3,000 ft. per day. Laboratory testing for permeability is recommended prior to approval of the porous layer material.

A "cookbook" approach to the internal drainage problem is given by G.S. Kozloo in Transportation Record 993.

The measurement of subsurface drainage is generally based on the time required for 50-percent of the unbound water to be removed from the layer to be drained. The Casagrande flow equation for estimating the 50-percent drainage time is expressed as:

$$t_{50} = (\eta_e \times L^2) / [2 \times K \times (H + L \times \tan \alpha)] \quad (1.6.1)$$

where

- t_{50} = time for 50 percent of unbound water to drain (days),
- η_e = effective porosity (80 percent of absolute porosity),
- L = length of flow path (feet),
- K = permeability constant (ft./day), and
- $\tan \alpha$ = slope of the base layer.

Filter Material. A detailed description of filter layers is contained in Appendix AA, Volume 2. *Ridgeway (11)* provides the following general comments:

The drainage layer and the collector system must be prevented from clogging if the system is to remain functioning for a long period of time. This is accomplished by means of a filter between the drain and the adjacent material. The filter material, which is made from select aggregates or fabrics, must meet three general requirements: (1) it must prevent finer material, usually the subgrade, from piping or migrating into the drainage layer and clogging it; (2) it must be permeable enough to carry water without any resistance; and (3) it must be strong enough to carry the loads applied and, for aggregate, to distribute live loads to the subgrade.

Surface Course. The surface course of a flexible structure consists of a mixture of mineral aggregates and bituminous materials placed as the upper course and usually constructed on a base course. In addition to its major function as a structural portion of the pavement, it must also be designed to resist the abrasive forces of traffic, to reduce the amount of surface

water penetrating the pavement, to provide a skid-resistance surface, and to provide a smooth and uniform riding surface.

The success of a surface course depends to a degree on obtaining a mixture with the optimum gradation of aggregate and percent of bituminous binder to be durable and to resist fracture and raveling without becoming unstable under expected traffic and climatic conditions. The use of a laboratory design procedure is essential to ensure that a mixture will be satisfactory.

Although dense-graded aggregates with a maximum size of about 1 inch are most commonly specified for surface courses for highways, a wide variety of other gradations, from sands to coarse, open-graded mixtures, have been used and have provided satisfactory performance for specific conditions. Surface courses are usually prepared by hot plant mixing with an asphalt cement, but satisfactory performance has also been obtained by cold plant mixing, or even mixing, in-place, with liquid asphalts or asphalt emulsions. Hot plant mixes, e.g., asphalt concrete, are recommended for use on all moderate to heavily trafficked highways.

Construction specifications usually require that a bituminous material be applied on untreated aggregate base courses as a prime coat, and on treated base courses and between layers of the surface course to serve as a tack coat.

No specific quality requirements for surface courses are presented in this Guide. It is recognized that each agency will prepare specifications that are based on performance, local construction practices, and the most economical use of local materials. ASTM Specification D 3515 provides some guidelines for designing asphalt concrete paving mixes.

It is particularly important that surface courses be properly compacted during construction. Improperly compacted surface courses are more likely to exhibit a variety of types of distress that tend to reduce the life and overall level of performance of the pavement. Types of distress that are often related to insufficient compaction during construction include rutting resulting from further densification under traffic, structural failure resulting from excess infiltration of surface water through the surface course, and cracking or raveling of the surface course resulting from embrittlement of the bituminous binder by exposure to air and water in the mixture. Specific criteria for compaction must be established by each highway agency based on local experience. Theoretical maximum densities of 92 percent or more are sometimes specified for dense-graded mixes.

1.6.2 Rigid Pavements

As shown in Figure 1.1, rigid pavements generally consist of a prepared roadbed underlying a layer of subbase and a pavement slab. The subbase may be stabilized or unstabilized. In cases of low volume road design where truck traffic is low, a subbase layer may not be necessary between the prepared roadbed and the pavement slab.

A drainage layer can be included in rigid pavements in much the same manner described for flexible pavements as shown in Figure 1.3. Alternate drainage designs are shown in Appendix AA, Volume 2.

Subbase. The subbase of a rigid pavement structure consists of one or more compacted layers of granular or stabilized material placed between the subgrade and the rigid slab for the following purposes:

- (1) to provide uniform, stable, and permanent support,
- (2) to increase the modulus of subgrade reaction (k),
- (3) to minimize the damaging effects of frost action,
- (4) to prevent pumping of fine-grained soils at joints, cracks, and edges of the rigid slab, and
- (5) to provide a working platform for construction equipment.

If the roadbed soils are of a quality equal to that of a subbase, or in cases where design traffic is less than 1,000,000 18-kip ESAL's, an additional subbase layer may not be needed.

A number of different types of subbases have been used successfully. These include graded granular materials and materials stabilized with suitable admixtures. Local experience may also provide useful criteria for the selection of subbase type. The prevention of water accumulations on or in roadbed soils or subbases is essential if satisfactory performance of the pavement structure is to be attained. It is recommended that the subbase layer be carried 1 to 3 feet beyond the paved roadway width or to the inslope if required for drainage.

Problems with the erosion of subbase material under the pavement slab at joints and at the pavement edge have led some designers to use a lean concrete or porous layers for subbase. While the use of a porous layer is encouraged it should be noted that design criteria for such materials are still in the development stage and the designer should review the literature or

contact agency personnel familiar with current requirements.

Pavement Slab. The basic materials in the pavement slab are portland cement concrete, reinforcing steel, load transfer devices, and joint sealing materials. Quality control on the project to ensure that the materials conform to AASHTO or the agency specifications will minimize distress resulting from distortion or disintegration.

Portland Cement Concrete. The mix design and material specifications for the concrete should be in accordance with, or equivalent to, the requirements of the AASHTO *Guide Specifications for Highway Construction* and the *Standard Specifications for Transportation Materials*. Under the given conditions of a specific project, the minimum cement factor should be determined on the basis of laboratory tests and prior experience of strength and durability.

Air-entrained concrete should be used whenever it is necessary to provide resistance to surface deterioration from freezing and thawing or from salt or to improve the workability of the mix.

Reinforcing Steel. The reinforcing steel used in the slab should have surface deformations adequate to bond and develop the working stresses in the steel. For smooth wire mesh, this bond is developed through the welded cross wires. For deformed wire fabric, the bond is developed by deformations on the wire and at the welded intersections.

Joint Sealing Materials. Three basic types of sealants are presently used for sealing joints:

- (1) *Liquid sealants.* These include a wide variety of materials including: asphalt, hot-poured rubber, elastomeric compounds, silicone, and polymers. The materials are placed in the joint in a liquid form and allowed to set. When using liquid sealants, care should be taken to provide the proper shape factor for the movement expected.
- (2) *Preformed elastomeric seals.* These are extruded neoprene seals having internal webs that exert an outward force against the joint face. The size and installation width depend on the amount of movement expected at the joint.
- (3) *Cork expansion joint filler.* There are two types of cork fillers: (a) standard expansion joint filler, and (b) self-expanding (SE) type.

Longitudinal Joints. Longitudinal joints are needed to form cracks at the desired location so that they may be adequately sealed. They may be keyed, butted, or tied joints, or combinations thereof. Longitudinal joints should be sawed or formed to a minimum depth of one-fourth of the slab thickness. Timing of the sawcutting is critical to the crack formation at the desired location. The maximum recommended longitudinal joint spacing is 16 feet.

Load-Transfer Devices. Mechanical load-transfer devices for *transverse joints* should possess the following attributes:

- (1) They should be simple in design, be practical to install, and permit complete encasement by the concrete.
- (2) They should properly distribute the load stresses without overstressing the concrete at its contact with the device.
- (3) They should offer little restraint to longitudinal movement of the joint at any time.
- (4) They should be mechanically stable under the wheel load weights and frequencies that will prevail in practice.
- (5) They should be resistant to corrosion when used in those geographic locations where corrosive elements are a problem. (Various types of coatings are often used to minimize corrosion.)

A commonly used load-transfer device is the plain, round steel dowel conforming to AASHTO Designation M 31-Grade 60 or higher. Specific design requirements for these relative to diameter, length, and spacing are provided in Part II. Although round dowels are the most commonly used, other mechanical devices that have proven satisfactory in field installations may also be used.

Consideration may also be given to omitting load transfer devices from transverse weakened plane joints in plain jointed concrete pavement when supported on a treated permeable base.

Tie Bars. Tie bars, either deformed steel bars or connectors, are designed to hold the faces of abutting slabs in firm contact. Tie bars are designed to withstand the maximum tensile forces required to overcome subgrade drag. They are not designed to act as load-transfer devices.

Deformed bars should be fabricated from billet or axle steel of Grade 40 conforming to AASHTO M 31 or M 53. Specific recommendations on bar sizes,

lengths, and spacings for different pavement conditions are presented in Part II.

Other approved connectors may also be used. The tensile strength of such connectors should be equal to that of the deformed bar that would be required. The spacing of these connectors should conform to the same requirements given for deformed tie bars in Part II.

Consideration should be given to the use of corrosion-resistant materials or coatings for both tie bars and dowels where salts are to be applied to the surface of the pavement.

1.6.3 Shoulders

Shoulders have often in the past been constructed of a flexible base with an asphalt surfacing or of a stabilized base with an asphalt surfacing. The combination of a dissimilarity between the outside lane and shoulder and the encroachment of heavy wheel loads onto the shoulder have sometimes resulted in joint problems between the travel lanes and the shoulder. Research has shown that strengthening of the shoulder and adding special sealants have helped to alleviate this problem. The use of tied concrete shoulders or 3-foot monolithic widening of the outside PCC lane has also proven beneficial (1.5-foot monolithic widening is acceptable if a rumble strip is provided as a deterrent to edge encroachment). Thickening the outside edge of the travel lane or using a monolithic curb (where appropriate) also strengthens the pavement edge and reduces the shoulder-joint problem. Provision for slab design which incorporates tied shoulders and widened outside lanes is provided in Part II of this Guide.

Additional information pertinent to shoulder design is given in Section 1.9.

1.7 ENVIRONMENT

Two main environmental factors are considered with regard to pavement performance and pavement structure design in this Guide; specifically, these are temperature and rainfall.

Temperature will affect (1) the creep properties of asphalt concrete, (2) thermal-induced stresses in asphalt concrete, (3) contraction and expansion of portland cement concrete, and (4) freezing and thawing of the roadbed soil. Temperature and moisture differential between the top and bottom of concrete slabs in jointed concrete pavements creates an upward curling

and warping of the slab ends which can result in pumping and structural deterioration of undrained sections.

Rainfall, if allowed to penetrate the pavement structure or roadbed soil, will influence the properties of those materials. This section of the Guide covers problems associated with temperature. Section 1.8 covers drainage requirements as related to rainfall.

Freezing and thawing of roadbed soil has traditionally been a major concern of pavement designers. The major effect is with regard to the thaw-weakening which can occur during the spring thaw period. Figure 1.4 illustrates the seasonal effects which can occur in many regions of the United States. A second effect of freezing is the occurrence of frost heaving, causing a reduction in the serviceability of the pavement.

Procedures for calculating the damage during various seasons of the year as a function of thaw-weakening and frost heaving are given in Part II. It is beyond the scope of the Guide to describe in detail the mechanism related to frost susceptibility, thaw-weakening, and frost heaving. The user is referred to Reference 14 for more information on this subject. A few of the more pertinent considerations from Reference 14 which relate to pavement structure design in frost areas are reproduced in this section of the Guide.

Frost heaving of soil within or beneath a pavement is caused by the accumulation of ice within the larger soil voids and, usually, a subsequent expansion to form continuous ice lenses, layers, veins, or other ice masses. The growth of such distinct bodies of ice is termed ice segregation. A lens grows in thickness in the direction of heat transfer until the water supply is depleted, as by formation of a new lens at a lower level, or until freezing conditions at the freezing interface will no longer support further crystallization. Investigations (12, 13, 16) have shown that ice segregation occurs only in soils containing fine particles. Such soils are said to be frost susceptible; clean sands and gravels are nonfrost-susceptible soils. The degree of frost susceptibility is principally a function of the percentage of fine particles and, to a lesser degree, of particle shape, distribution of grain sizes, and mineral composition.

The following three conditions of soil, temperature, and water must be present simultaneously in order for ice segregation to occur in the subsurface materials:

- (1) *Soil.* The soil must be frost susceptible.
- (2) *Temperature.* Freezing temperatures must penetrate the soil. In general, the thickness of a particular layer or lens of ice is inversely pro-

portional to the rate of penetration of freezing temperature into the soil.

- (3) *Water.* A source of water must be available from the underlying groundwater table, infiltration or gravitational flow, an aquifer, or the water held within the voids of fine-grained soil.

Periods of thawing are among the most critical phases in the annual cycle of environmental changes affecting pavements in seasonal frost areas. Such thawing cycles are in many cases very disruptive, depending on the rapidity of the thaw and the drainage capabilities of the pavement system. During thaw periods considerable melting of snow may occur, with melt water filling the ditches and infiltrating into the pavement from the shoulders and through surface cracks in the pavement itself. During thawing periods, the bearing capacity of the roadbed soil may be severely reduced, and frost heaving frequently is more severe after midwinter thaw periods. In areas of deep frost penetration, the period of complete thawing of thicker pavement structures in the spring is usually the most damaging type of thaw period because it affects the roadbed as well as subbase and base layers. The severity of the adverse effect on the supporting capacity of a given roadbed is largely dependent on the temperature distribution in the ground during the thawing period.

Thawing can proceed from the top downward, from the bottom upward, or both. The manner of thawing depends on the pavement surface temperature. During a sudden spring thaw, melting will proceed almost entirely from the surface downward. This type of thawing leads to extremely adverse drainage conditions. The still-frozen soil beneath the thawed layer traps the water released by the melting ice lenses so that lateral and surface drainage are the only means of egress. In granular soils, lateral drainage may be restricted by still-frozen shoulders resulting from the insulating effect of snow and/or different thermal conductivity and surface reflectivity characteristics. If air temperatures in the spring remain cool and frosty at night, upward conduction of heat stored in the ground from the previous summer and of heat from the interior of the earth will produce thawing, principally from the bottom upward. Such thawing permits soil moisture from melted ice lenses to drain downward while the material above it remains frozen.

The climatic factors of air temperature, solar radiation received at the surface, wind, and precipitation are major parameters that effect the severity of frost effects in a given geographical area. The first three

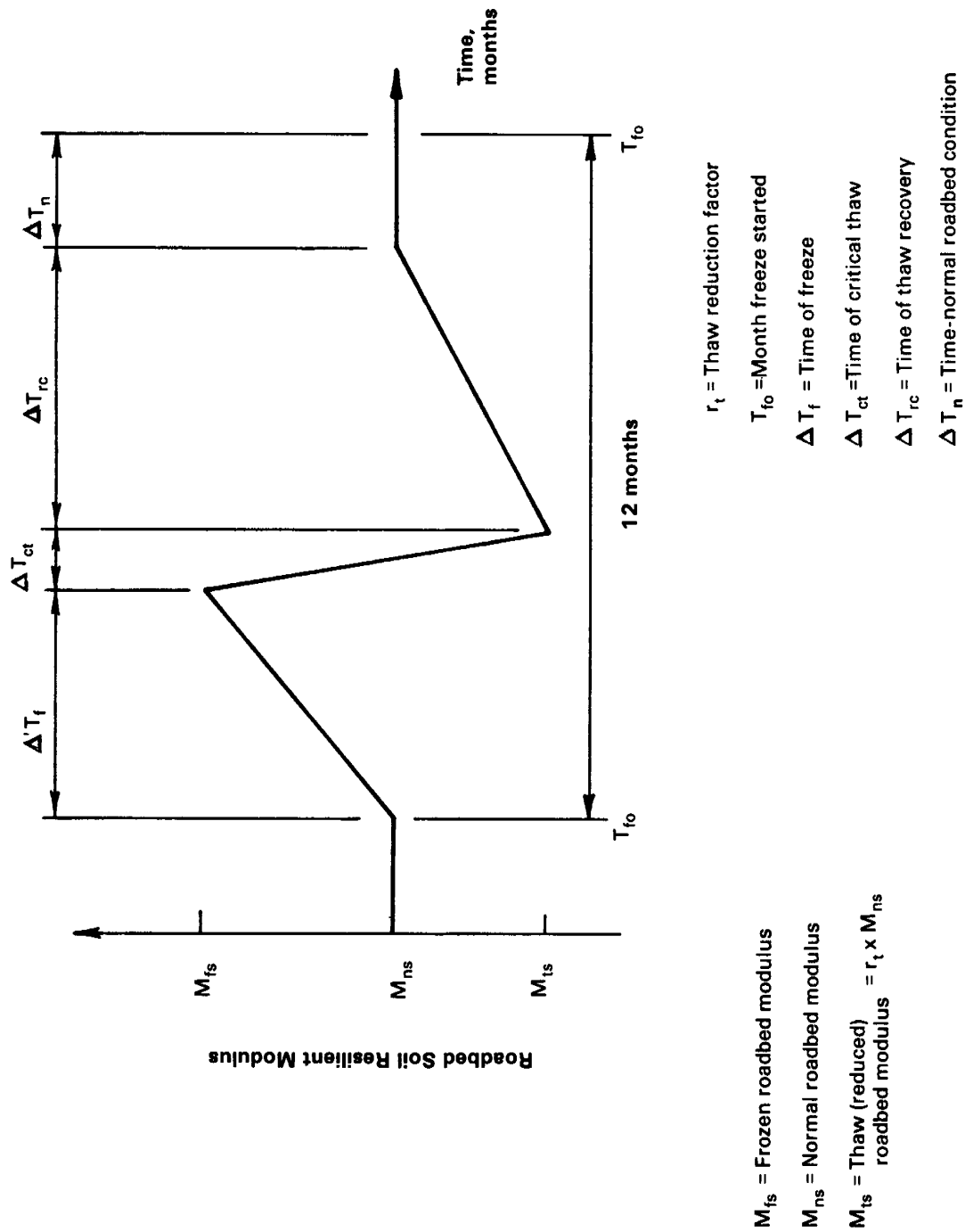


Figure 1.4. Representation of Roadbed Modulus Variations throughout Year

mainly affect the temperature regime in the pavement structure, including the important parameters of depth of frost penetration, number of freeze-thaw cycles, and duration of the freezing and thawing periods. Precipitation affects mainly the moisture regime but causes changes in the thermal properties of the soil and interacts with the other climatic variables determining ground temperatures as well.

Investigators who have endeavored to calculate the depth of frost penetration have found it convenient to make use of a freezing index (15), which expresses the cumulative effect of intensity and duration of sub-freezing air temperatures. The freezing index is expressed in degree days and represents the difference between the highest and lowest points on a curve of cumulative degree days versus time for one freezing season. The degree days for any one day equals the difference between the average daily air temperature and 32°F. Degree days are plus when the average daily temperature is below 32°F (freezing degree days) and minus when above 32°F (thawing degree days). Thus, an average daily temperature of 31°F is equal to one degree day, 33°F is equal to minus one degree day, and 22°F is equal to 10 degree days.

The freezing index for a given year and site location can be calculated from average daily air temperature records, which should be obtained from a station situated close to the construction site. This is necessary because differences in elevation and topography, and nearness to centers of population or bodies of water (rivers, lakes, seacoast) and other sources of heat, are likely to cause considerable variations in the value of the freezing index over short distances. Such variations may be of sufficient magnitude to affect a pavement design based on depth of frost penetration, particularly in areas where the freezing index used in the calculation is more than about 100 degree days. Table 1.4 provides an indication of the depth of frost based on the penetration of the 32°F (0°C) isotherm below the surface of 12 inches of portland cement

concrete. Variations due to pavement type, soil type, duration of low temperature, and water content may affect the actual frost penetration; however, it is clear that frost penetration can extend well into the roadbed soils during sustained periods of freezing temperatures.

Most studies have shown that a soil is frost susceptible only if it contains fine particles. Soils free of material passing the 200 mesh sieve generally do not develop significant ice segregation or frost heave.

A reliable method for recognizing a frost susceptible material for site specific conditions has not, as yet, been identified. Some guidelines are available in the literature and are described by Johnson, et al. (14). The U.S. Corps of Engineers have reported that most inorganic soils containing 3 percent (by weight) or more of grains finer than 0.02 mm in diameter are considered frost susceptible for pavement design purposes (16).

In summary, frost action due to freezing temperatures in soil, can cause both heaving and thaw-weakening. However, thaw-weakening is not necessarily directly proportional to heaving since field experience shows that thaw-weakened but well-drained sandy or gravelly materials recover bearing strength quite rapidly, whereas clayey soils may show little heave but recover their stability very slowly (14). The design procedure in Part II of this Guide provides for both frost heave and thaw-weakening.

The period of thaw-weakening can be estimated from deflection measurements, as shown in Figure 1.5. These data were obtained at the AASHTO Road Test and indicate that the thaw-weakening period can range from a few weeks to a few months, with varying degrees of reduction in structural capacity. Further guidelines relative to thaw-weakening periods are given in Part II of this Guide; however, user agencies are encouraged to develop these relationships based on site specific measurements within their areas and to compare such experience with other agencies nationally.

Laboratory tests and field evaluations indicate that the retained modulus during the thaw-weakening period may be 20 to 50 percent of the normal modulus obtained during the summer and fall periods.

It should be noted that the resilient modulus for roadbed soils may also vary by season even when no thaw-weakening period is experienced. For example, during the heavy rainy periods it might be expected that some seasonal variation in bearing capacity will occur. There may be other situations in which no seasonal variations occur and a constant modulus can be used for the roadbed soil. Note that the modulus is

Table 1.4. Frost Penetration under Portland Cement Concrete Pavement (11)

Air-Freezing Index (degree days)	Frost Penetration (feet)
200	1.8
400	3.0
600	4.0
800	5.0
1,000	6.0

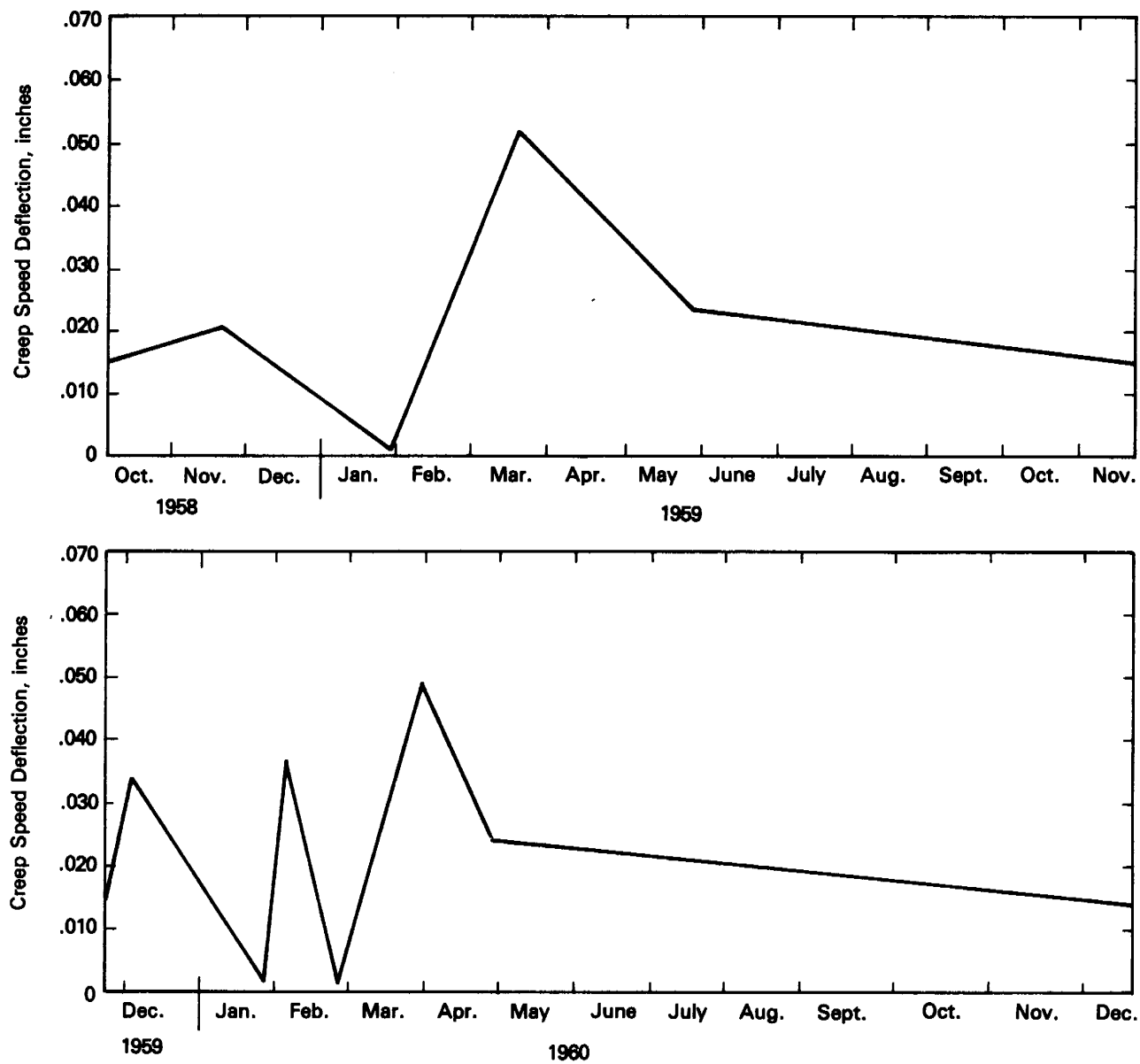


Figure 1.5. Seasonal Deflection on Nontraffic Loop, 6-kip Single Axle Load (17)

related to CBR, R-value, or plate bearing value and, hence, experience with these types of strength tests can be used to infer the seasonal effects on the modulus.

While information is generally lacking with regard to the effects of thaw-weakening or water saturation on untreated aggregate base and subbase, some research does suggest that a reduced modulus can occur during certain critical seasons. Reference 18, based on results of spring deflection measurements at the AASHO Road Test, indicates that the retained modulus ranged from 80 to 85 percent of the normal modulus obtained during the summer and fall. If these layers had been well drained no reduction in modulus would be anticipated.

In order to compensate for thaw-weakening effects on pavement performance, provision has been made in Part II to calculate an effective annual roadbed soil resilient modulus. The effective modulus used for a full 12 months will produce the same change in PSI as would be obtained by calculating the change with the respective seasonal moduli.

The design for frost areas included in this Guide depend to a large extent on the performance of rigid and flexible pavements at the AASHO Road Test. It is recognized that experience in some northern tier states and Alaska may indicate that alternate procedures can be used. For example, some state agencies require a 12- to 24-inch granular layer over frost susceptible roadbed soils. Other agencies require full or partial replacement of frost susceptible materials (16). Such requirements could increase the total thickness of the pavement structure when compared with requirements of this Guide. Careful review of the cost and benefit (performance) of such design policies should be considered; however, if field data indicate that life-cycle costs can be reduced by following such a procedure there should be no problem in justifying this type of design and construction.

In addition to the seasonal effect on the subgrade and granular materials, temperature will also influence the characteristics of the asphalt concrete. Performance will be affected in three ways: (1) low temperature cracking, (2) fatigue cracking, and (3) rutting. It is not clear from research studies just how much these factors will influence PSI (19). However, low temperature cracking and fatigue cracking will increase maintenance costs, and rutting is a safety consideration related to potential hydroplaning as well as a maintenance problem.

Reference 20 summarizes information concerning premature cracking in asphalt concrete due to low temperature induced stresses and fatigue due to traf-

fic. The recommendations from this study indicate that the softer grades of asphalt, i.e., AC-5 or equivalent, should be used in cold climates (when the mean annual air temperature is less than 45°F); and harder grades, i.e., AC-20 or equivalent, in hot climates (when the mean annual air temperature is greater than 75°F). The specific selection of asphalt grade will be a function of local experience; however, it is recommended that consideration be given to the above guidelines.

For thick, full-depth asphalt concrete, there are indications from research that fatigue cracking can be significantly affected by temperature (21). In general, these findings suggest that the harder grades of asphalt will provide improved performance in terms of fatigue cracking. Thus, an AC-40 would be appropriate in warm climates for thick (7 inches or more) pavements.

It should be noted that the selection of the grade of asphalt, per se, will not solve all of the problems of premature cracking. The designer must also give careful attention to *all* of the factors which can influence performance, e.g., structural design, drainage, construction, thaw-weakening, etc.

1.8 DRAINAGE

Drainage of water from pavements has always been an important consideration in road design; however, current methods of design have often resulted in base courses that do not drain well. This excess water combined with increased traffic volumes and loads often leads to early pavement distress in the pavement structure.

Water enters the pavement structure in many ways, such as through cracks, joints, or pavement infiltration, or as groundwater from an interrupted aquifer, high water table, or localized spring. Effects of this water (when trapped within the pavement structure) on pavements include:

- (1) reduced strength of unbounded granular materials,
- (2) reduced strength of roadbed soils,
- (3) pumping of concrete pavements with subsequent faulting, cracking, and general shoulder deterioration, and
- (4) pumping of fines in aggregate base under flexible pavements with resulting loss of support.

Less frequently noticed problems due to entrapped water include (but are not limited to):

- (1) stripping of asphaltic concrete,

- (2) differential heaving over swelling soils, and
- (3) frost heave.

Prior editions of the *AASHTO Guide for Design of Pavement Structures* have not treated the effects of drainage on pavement performance. In this Guide, drainage effects are directly considered in terms of the effect of moisture on roadbed soil and base strength (for flexible pavements) and the effect of moisture on subgrade strength and on base erodability (for concrete pavements). Though consideration for stripping of asphalt concrete is not directly considered, the effects of swelling soils and frost heave are.

1.8.1 General Design Considerations

Methods for treating water in pavements have generally consisted of:

- (1) preventing water from entering the pavement,
- (2) providing drainage to remove excess water quickly, and
- (3) building the pavement strong enough to resist the combined effect of load and water.

When all possible sources of water are considered, protection of the pavement structural section from water entry requires interception of groundwater as well as sealing of the pavement surface. Considerable attention has generally been given to intercepting groundwater, whereas less attention has been given to sealing the surface to exclude infiltration from rain and snow melt. As a result, a considerable amount of water often enters the pavement substructure, resulting in a need for some type of drainage.

To obtain adequate pavement drainage, the designer should consider providing three types of drainage systems: (1) surface drainage, (2) groundwater drainage, and (3) structural drainage. Such systems, however, are only effective for "free water." Water held by capillary forces in soils and in fine aggregates cannot be drained. The effects of this "bound" moisture must be considered in the design of pavement structures through its effect on the pavement material properties. Most existing pavements do not include drainage systems capable of quickly removing free water.

Most existing design methods have relied on the practice of building pavements strong enough to resist the combined effects of load and water. However, they do not always account for the potential destructive effects of water within the pavement structure. As a result, increased emphasis is needed to exclude water

from the pavement and provide for rapid drainage. While both approaches are extremely difficult, this Guide will emphasize only the latter treatment. However, maintenance policies should recognize the benefits and necessity of maintaining the joint sealant and thus preventing water from leaking into the subbase layer.

1.8.2 Design of Pavement Subsurface Drainage

Two general types of pavement subsurface design criteria have been proposed for use in pavements (11). These include:

- (1) criterion for the time of drainage of the base or subbase beginning with the flooded condition and continuing to an established acceptable level, and
- (2) an inflow-outflow criterion, by which drainage occurs at a rate greater than or equal to the inflow rate, thus avoiding saturation.

Removal of the free water can be accomplished by draining the free water vertically into the subgrade, or laterally through a drainage layer into a system of pipe collectors. Generally, the actual process will be a combination of the two.

1.8.3 Incorporation of Drainage Into Guide

Drainage effects on pavement performance have been considered in this Guide. Drainage is treated by considering the effect of water on the properties of the pavement layers and the consequences to the structural capacity of the pavement. Additional work is needed to document the actual effect of drainage on pavement life.

For new design (Part II), the effect of drainage is considered by modifying the structural layer coefficient (for flexible pavements) and the load transfer coefficient (for rigid pavements) as a function of

- (1) the quality of drainage (e.g., the time required for the pavement to drain), and
- (2) the percent of time the pavement structure is exposed to moisture levels approaching saturation.

For rehabilitation of existing pavements, additional questions need to be asked. These include (22):

- (1) Is the original drainage design adequate for the existing road?

- (2) What changes are necessary to ensure that drainage inadequacies, which may contribute to structural distress, are corrected?
- (3) If the original drainage system design was adequate, have environmental or structural changes taken place since it was built that require reconstruction of the system?
- (4) Does the present or projected land use in areas adjacent to the road indicate that surface drainage flow patterns have changed or are likely to change, thus rendering existing drainage facilities inadequate?

Details of the design of subsurface drainage systems are important and, therefore, Appendix AA of Volume 2 has been provided to assist the engineer in this effort.

1.9 SHOULDER DESIGN

As defined by AASHTO, a highway shoulder is the "portion of roadways contiguous with the traveled way for accommodation of stopped vehicles for emergency use, and for lateral support of base and subbase courses." The shoulder is also considered by some agencies as a temporary detour to be used during rehabilitation of the usual traveled way.

No specific design criteria are provided in this Guide for the determination of the pavement structure for shoulders. An AASHTO position paper on shoulder design is included herein as Appendix E.

A number of agencies have developed specific design criteria for shoulders. Where such criteria are available within specific governmental jurisdictions it is recommended that such criteria be followed pending the development of more specific recommendations by AASHTO.

If design criteria for shoulders are based on pavement structure requirements similar to those used for the traveled way, the design and rehabilitation procedures included in Parts II and III of this Guide are considered applicable.

The use of tied shoulders or a widened width of paving in the lane adjacent to the shoulder has proven to be beneficial to overall performance of rigid pavements. Provision has been made in both Parts II and III to recognize the benefits to be derived from this type of design.

It is recognized that paved shoulders adjacent to flexible pavements will provide lateral support for the base and surface courses. No provision is made in this Guide to modify the design of flexible pavements as a function of shoulder design. Local practice, experience, and cost analysis should, in all cases, be considered as prime factors in shoulder design. The benefits of a paved shoulder will be enhanced if the traffic is concentrated in the traffic lanes. The use of a contrasting shoulder color or texture (seal coats) will help achieve this objective. Truck encroachment onto the shoulder is a major cause of shoulder distress; hence, any treatment which will minimize operations on the shoulder will benefit the performance of pavements in the traveled way and on the shoulder.

