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Guide for Mechanistic-Empirical Design
OF NEW AND REHABILITATED PAVEMENT STRUCTURES

FINAL REPORT

PART 1. INTRODUCTION
CHAPTER 1. BACKGROUND, SCOPE AND OVERVIEW

NCHRP

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PART 1—INTRODUCTION

CHAPTER 1
BACKGROUND, SCOPE, AND OVERVIEW

1.1.1 BACKGROUND

1.1.1.1 Objective of the Design Guide

The overall objective of the Guide for the Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures (referred to hereinafter as the Design Guide) is to provide the highway community with a state-of-the-practice tool for the design of new and rehabilitated pavement structures, based on mechanistic-empirical principles. This objective was accomplished through developing the following:

- The Design Guide itself, which is based on comprehensive pavement design procedures that use existing mechanistic-empirical technologies.
- User-oriented computational software and documentation based on the Design Guide procedure.

The Design Guide represents a major change in the way pavement design is performed. The designer first considers site conditions (traffic, climate, subgrade, existing pavement condition for rehabilitation) and construction conditions in proposing a trial design for a new pavement or rehabilitation. The trial design is then evaluated for adequacy through the prediction of key distresses and smoothness. If the design does not meet desired performance criteria, it is revised and the evaluation process repeated as necessary. Thus, the designer is fully involved in the design process and has the flexibility to consider different design features and materials for the prevailing site conditions. This approach makes it possible to optimize the design and to more fully insure that specific distress types will not develop.

The mechanistic-empirical (M-E) format of the Design Guide provides a framework for future continuous improvement to keep up with changes in trucking, materials, construction, design concepts, computers, and so on. In addition, guidelines for implementation and staff training have been prepared to facilitate use of the new design procedure, as well as strategies to maximize acceptance by the transportation community.

1.1.1.2 Economic Justification for a Revised and Improved Design Guide

The nation's highways reached an estimated 2.7 trillion vehicle-miles in 2000. This is four times the 1960 level. This amounts to 7.4 billion vehicle-miles of travel every day. Truck travel (single-unit and combinations) has increased 231 percent since 1970. Combination truck travel has increased 285 percent over 1970 levels and now accounts for 4.9 percent of total annual vehicle-miles of travel versus 3.2% in 1970. *(1)* The 4 million miles of U.S. roadways (with 2 million miles of paved roads) have been constructed, rehabilitated, and maintained over the previous century, and they represent a huge national investment that has provided a safe and

comfortable means of transportation for both private and commercial vehicles. Highways have contributed significantly to the economic growth of the nation.
Pavement structures wear down and deteriorate under heavy axle loadings and exposure to the elements (very hot and very cold temperatures, freezing and thawing, precipitation). Therefore, they must be maintained and improved on a regular basis. This requires a very significant commitment of resources on the part of the nation's highway agencies (State, Federal, and local). Figure 1.1.1 illustrates the magnitude and increasing level of highway expenditures by function from 1980 to 2000.

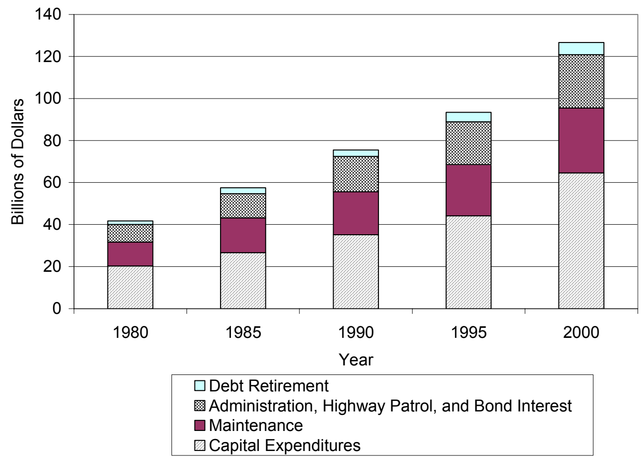

Figure 1.1.1 The magnitude and increasing level of highway expenditures by function from 1980 to 2000. *(1)*

Total highway expenditure by all units of government in 2000 was $126.7 billion, a 203 percent increase compared to 1980 (average annual increase of 10 percent). Note from figure 1.1.1 that the annual level of expenditure is clearly accelerating over time. *(1)* The 2000 total disbursement by State highway agencies was $89.8 billion, of which 53.1 percent went to capital outlays which includes 10.5 percent for new highway construction and 42.6 percent for improvements on existing highways (*1*). Approximately one-half of the capital outlay goes to pavement related work. The sheer magnitude of annual expenditures on highway pavements justifies the application of the best available design procedures to optimize the use of highway funds. Any improvements in design of new or rehabilitated pavement structures will have significant and sizeable implications in reducing the cost of maintaining these highway pavements.

As of the publication of this Design Guide, the *AASHTO Guide for Design of Pavement Structures* is the primary document used to design new and rehabilitated highway pavements. The Federal Highway Administration's 1995-1997 National Pavement Design Review found that some 80 percent of States use the 1972, 1986, or 1993 AASHTO Guides *(2, 3 ,4)*. All those versions are empirically based on performance equations developed using 1950's AASHO Road Test data. The 1986 and 1993 AASHTO Guides contain some refinements in materials input parameters, design reliability, and empirical procedures for rehabilitation design *(3, 4)*.

Since the AASHO Road Test, the AASHTO Joint Task Force on Pavements (JTFP) has been responsible for the development and implementation of pavement design technologies. This charge has led to many significant initiatives, including the development of every revision of the AASHTO Guide. More recently, and in recognition of the limitations of the AASHTO Guide, the JTFP initiated an effort to develop an improved Design Guide. As part of this effort, a workshop was convened on March 24-26, 1996, in Irvine, California, to develop a framework for improving the Guide. The workshop attendees—pavement experts from public and private agencies, industry, and academia—addressed the areas of traffic loading, foundations, materials characterization, pavement performance, and environment to help determine the technologies best suited for the new Design Guide. At the conclusion of that workshop, a major long-term goal identified by the JTFP was the development of a design guide based as fully as possible on mechanistic principles. This Design Guide is the end result of that goal.

1.1.1.3 Need for the Design Guide

The various editions of the *AASHTO Guide for Design of Pavement Structures* have served well for several decades; nevertheless, many serious limitations exist for their continued use as the nation's primary pavement design procedures:

- Traffic loading deficiencies: Heavy truck traffic design volume levels have increased tremendously (about 10 to 20 times) since the design of the pavements used in the Interstate system in the 1960's. The original Interstate pavements were designed for 5 to 15 million trucks, whereas today these same pavements must be designed for 50 to 200 million trucks and more over an even longer design life (e.g. 30-40 years versus 20 years). The existing AASHTO Guide cannot be used reliably to design for this level of traffic. Road Test pavements sustained slightly over 1 million axle load applications—less than the traffic carried by many modern pavements within the first few years of their use—and the equations forming the basis of the earlier procedures were based on regression analyses of the AASHO Road Test data. Thus, application of the procedure to modern traffic streams means the designer often must extrapolate the design methodology far beyond the data and experience providing the basis for the procedure. Clearly, the result is that designers have been working "in the dark" on highly trafficked projects. Such projects may well have been either "under-designed" or "over-designed," with the result of significant economic loss.
- Rehabilitation deficiencies: Pavement rehabilitation design procedures were not considered at the AASHO Road Test. Procedures in the 1993 Guide are completely empirical and very limited, especially in consideration of heavy traffic. Major economic losses will continue unless improved capabilities for rehabilitation design are provided to meet today's highway traffic needs, as most projects today include rehabilitation design. Improved rehabilitation designs will lead to longer-lasting and more cost-effective rehabilitated pavements.
- Climatic effects deficiencies: Because the AASHO Road Test was conducted at one specific geographic location, it is impossible to address the effects of different climatic conditions on pavement performance. For example, at the Road Test a significant amount of distress occurred in the pavements during the spring thaw, a condition that

does not exist in a large portion of the country. Direct consideration of climatic effects at a project site will lead to improved pavement performance and reliability.

- Subgrade deficiencies: One type of subgrade was used for all test sections at the Road Test, but many types exist nationally that result in different performance of highway pavements. More adequate characterization and consideration of subgrade support will make it possible to improve design of the pavement structure resulting in improved performance and reliability.
- Surfacing materials deficiencies: Only one hot mix asphalt (HMA) mixture and one portland cement concrete (PCC) mixture were used at the Road Test. Today, there exist many different hot mix asphalt concrete (HMAC) and PCC mixtures (e.g., Superpave, stone-mastic asphalt, high-strength PCC) whose effects cannot be fully considered.
- Base course deficiencies: Only two unbound dense granular base/subbase materials were included in the main flexible and rigid pavement sections of the AASHO Road Test (limited testing of stabilized bases was included for flexible pavements). These exhibited significant loss of modulus due to frost and erosion. Today, various stabilized types of higher quality are used routinely, especially for heavier traffic loadings.
- Truck characterization deficiencies: Vehicle suspension, axle configurations, and tire types and pressures were representative of the types used in the late 1950's. Many of these are outmoded (tire pressures of 80 psi versus 120 psi today), resulting in deficient pavement designs to carry these loadings.
- Construction and drainage deficiencies: Pavement designs, materials, and construction were representative of those used at the time of the Road Test. No subdrainage was included in the Road Test sections, but positive subdrainage has become common in today's highways.
- Design life deficiencies: Because of the short duration of the Road Test, the long-term effects of climate and aging of materials were not addressed. The AASHO Road Test was conducted over 2 years, while the design lives for many of today's pavements are 20 to 50 years. Direct consideration of the cyclic effect on materials response and aging will lead to improved design life reliability.
- Performance deficiencies: Earlier AASHTO procedures relate the thickness of the pavement surface layers (asphalt layers or concrete slab) to serviceability. However, research and observations have shown that many pavements need rehabilitation for reasons that are not related directly to pavement thickness (e.g., rutting, thermal cracking, faulting). These failure modes are not considered directly in previous versions of the AASHTO Guide, which may be leading to more premature failures.
- Reliability deficiencies: The 1986 AASHTO Guide included a procedure for considering design reliability that has never been fully validated. This procedure resulted in a large multiplier of design traffic loadings to achieve a desired reliability level (e.g., a pavement designed for 50 million equivalent single axle loads [ESALs] was actually designed for 228 million). The multiplier increased greatly with design level of reliability and may result in excessive layer thicknesses for heavier trafficked pavements that may not be warranted but are commonly being designed today.

The primary measure of pavement performance in the earlier procedures is present serviceability, and the dominant factor affecting serviceability is longitudinal profile. Yet, in many cases, pavement managers find that distress factors other than ride—such as cracking, rutting, and joint faulting—control the cost-effective timing of pavement rehabilitation. To improve the reliability of design and to meet the needs of asset management, the management criteria and the pavement design procedure must relate to the same measured performance factors (*5*). To help alleviate these problems, the Design Guide uses key distress types for flexible and rigid pavements and the International Roughness Index (IRI) as the measure of pavement smoothness or ride quality.

During the development of the 1986 AASHTO Guide, it was recognized that future design procedures would be based on mechanistic-empirical principles. However, for such an approach to be practical would require pavement designers to have ready access to computers capable of handling the increased computational effort to perform the necessary calculations. The amount of computing power available on today's personal computers makes a mechanistic approach to pavement design possible.

1.1.1.4 Philosophy of the Design Guide Development

The design philosophy embraced by this Guide includes the following major tenets:

- The Guide generally applies validated, state-of-the-practice technologies.
- The Guide provides designers with the versatility to consider a wide variety of design and material options.
- The Guide provides an equitable design basis from the standpoint of pavement type selection.
- The Guide addresses both new and rehabilitation design issues.
- The Guide and associated software are user-friendly.
- The Guide provides for three hierarchical levels of design inputs that allow the designer to match the level of effort to the importance of the project. The input levels also allow for using improved procedures that may be developed in the future.

1.1.1.5 Benefits of a Mechanistic-Empirical Procedure

<u>Impact of Reduced Early Failures</u>

One of the major concerns of the previous AASHTO design procedure was the inability to incorporate significant materials properties into the design procedure. In the previous AASHTO flexible design procedure, the only material property incorporated was the loosely defined layer coefficient "a." Nothing could be more nebulous than this parameter. The rigid design procedure only considered the strength and modulus of the concrete and other properties such as the highly significant concrete coefficient of thermal expansion were not considered at all. This lack of materials property consideration can lead to early failures. These limitations resulted from the fact that variation in material quality was not a primary experimental variable included in the AASHO Road Test.

In addition, various design features cannot be directly considered. The flexible pavement procedure cannot determine the required thickness of asphalt bound material to limit fatigue cracking. This value must be chosen by the designer which can lead to early failures. The rigid design procedure did not consider joint spacing for joint load transfer, which can lead to early failures. The Design Guide directly considers all of these factors and more. For example, the bending strain at the bottom of the asphalt bound layer to control fatigue cracking is directly considered along with appropriate structural responses for other pavement distress modes. Another observed problem with existing pavements is the large variation in actual performance life as compared to design life. This variation is to be expected because the AASHTO design procedure is based on only 2 years of performance data, so long-term climatic effects could not be considered. This Guide includes technology that directly considers aging of materials, month by month, over the design period.

The mechanistic-empirical design procedure included in the Design Guide provides the tools for evaluating the effect of variations in materials on pavement performance. The Design Guide provides a rational relationship between construction and materials specification and the design of the pavement structure. Because the mechanistic procedures are able to better account for climate, aging, present day materials, and present day vehicle loadings, variation in performance in relation to design life should be reduced. This capability will reduce life cycle costs significantly over an entire highway network.

Impact of Increased Pavement Longevity

A FHWA study on the value of the Long-Term Pavement Performance program highlights the benefits of adopting the Design Guide in terms of increased pavement life (*6*). The study considered the stream of costs for a given pavement design over many years into the future (initial construction, maintenance, rehabilitation, user delay costs due to lane closures for rehabilitation). These costs were found to vary widely depending on the design of the pavement, its materials, and its construction. The improved technology of pavement design and rehabilitation incorporated into the Design Guide is expected to increase pavement longevity, resulting in economic benefits to highway agencies (lower facility construction and rehabilitation costs) and highway users (reduced delay time and costs due to longer time periods between lane closures required for rehabilitation).

The findings conservatively estimated a reduction in life cycle costs to State highway agencies of at least 5 percent and perhaps twice as much if full implementation occurs. The analysis also considered highway user delay costs due to lane closures for maintenance and rehabilitation. The results showed that there would be very significant economic benefits to the traveling public due to reduced maintenance and rehabilitation activities that require lane closures.

Additional Practical Benefits

More benefits of mechanistic design procedures that are difficult to quantify were recognized in the 1986 Guide *(3)*:

1. Estimates of the consequences of new loading conditions can be evaluated. For example, the damaging effects of increased loads, high tire pressures, multiple axles, and other factors can be modeled using mechanistic procedures.

2. Better utilization of available materials can be considered. For example, the use of stabilized materials in both rigid and flexible pavements can be simulated to predict future performance.
3. Improved procedures to evaluate premature distress can be developed, and it is possible to analyze why some pavements exceed their design expectations. In effect, better diagnostic techniques can be utilized.
4. Aging can be included in estimates of performance (e.g., asphalt hardens with time, which, in turn, affects low temperature thermal cracking).
5. Seasonal effects such as thaw-weakening can be included in estimates of performance.
6. Consequences of base erosion under rigid pavements can be evaluated.
7. Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

All of these aspects are directly addressed in this Design Guide.

<u>Benefit of Potential Future Continuous Improvements</u>

An additional important benefit of the mechanistic-based approach included herein is that, unlike empirical procedures, the concepts are generally applicable and modular such that a full range of future enhancements can be developed and implemented (e.g., improved rutting model, improved damage accumulation algorithm, improved laboratory testing procedures, improved backcalculation). These improvements can be incorporated continuously over time. Therefore, the procedure will not become outmoded with changes in construction materials, traffic patterns, vehicle types, or tire types and configurations.

1.1.2 PRINCIPLES OF A MECHANISTIC PROCEDURE

From an engineering point of view, there is much to be desired about a mechanistic approach to pavement design. "Mechanistic" refers to the application of the principles of engineering mechanics, which leads to a rational design process. Yoder and Witczak (*7*) pointed out that, for any pavement design procedure to be completely rational, three elements must be considered fully: the theory used to predict the assumed failure or distress parameter, the evaluation of the materials properties applicable to the selected theory, and the determination of the relationship between the magnitude of the parameter in question to the performance level desired. While this Design Guide considers all three elements, only an overview of the first is discussed at this time. The others are discussed in later parts of the Guide.

Figure 1.1.2 illustrates the general concept of a multi-layered elastic system as defined by Yoder and Witczak (*7)*. Generally, the analytical solution to the state of stress or strain within a pavement using the multi-layered elastic theory makes several assumptions. Some of these are (see figure 1.1.2):

- The material properties of each layer are homogeneous; that is, the properties at point A_i are the same as at point B_i.
- Each layer has a finite thickness except for the lower layer, and all are infinite in the lateral directions.

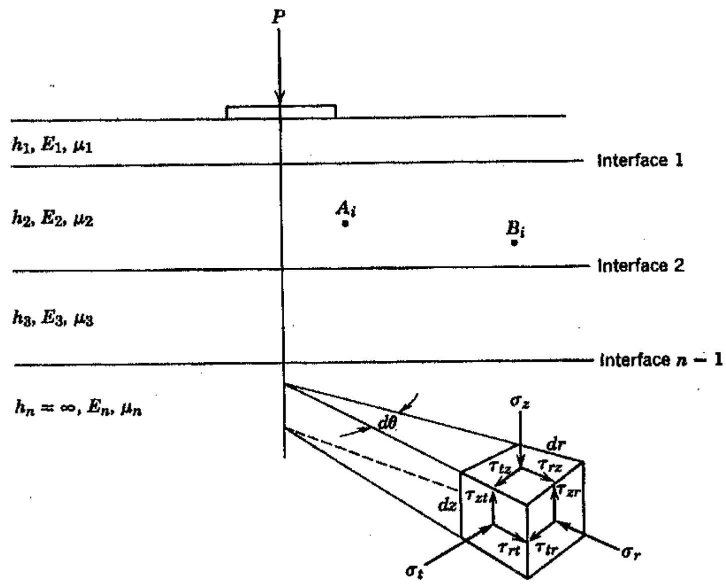

Figure 1.1.2. Generalized multi-layered elastic system (After Yoder and Witczak [7]).

- Each layer is isotropic; that is, the property at a specific point such as A_i is the same in every direction or orientation.
- Full friction is developed between layers at each interface.
- Surface shearing forces are not present.
- The stress solutions are characterized by two material properties for each layer. These properties are Poisson's ratio, μ, and the elastic modulus E.

Yoder and Witczak continue to show that, at a given point within any layer, static equilibrium requires that nine stresses exist. Figure 1.1.2 illustrates these stresses in a polar coordinate system where the stresses act on vertical (z), radial (r), and tangential (t) planes. These stresses are comprised of three normal stresses (σ_z, σ_r, σ_t) acting perpendicular to the element faces and six shearing stresses (τ_{rt}, τ_{tr}, τ_{rz}, τ_{zr}, τ_{tz}, τ_{zt}) acting parallel to the faces. Static equilibrium conditions also show that the shear stresses acting on intersecting faces are equal. Thus, $\tau_{rz} = \tau_{zr}$, $\tau_{rt} = \tau_{tr}$, and $\tau_{tz} = \tau_{zt}$. At each point in the system, there exists a certain orientation of the element such that the shear stresses acting on each face are zero. The normal stresses under this condition are defined as principal stresses and are denoted by σ_z (major stress), σ_r (intermediate), and σ_t (minor). The sum of the principal stresses at a point is defined as the bulk stress, θ.

Considering this triaxial stress state of any element, the strains (for the vertical, radial, and tangential directions, respectively) may be determined from the following equations:

$$\varepsilon_z = (1/E)[\sigma_z - \mu(\sigma_r + \sigma_t)] \quad (1.1.1)$$

$$\varepsilon_r = (1/E)[\sigma_r - \mu(\sigma_t + \sigma_z)] \quad (1.1.2)$$

$$\varepsilon_t = (1/E)[\sigma_t - \mu(\sigma_r + \sigma_z)] \quad (1.1.3)$$

Building on these general equations, the type of theory used is generally distinguished by three properties of the material behavior response (*7*). They are the relationship between stress and strain (linear or nonlinear), the time dependency of strain under a constant stress level (viscous or non-viscous), and the degree to which the material can rebound or recover strain after stress removal (plastic or elastic). All three properties are addressed in later parts of this Guide, as appropriate to the various paving materials. Some of the "empirical" part of mechanistic-empirical design relates to the characterization of materials or to traffic, environment, or other inputs to the design process. Other empirical parts of this Guide relate to field performance data used to correlate to accumulated damage. This "transfer" function, as it is sometimes called, relates the theoretical computation of "damage" (which is, in turn, a function of pavement deflection, strain, or stress responses) at some critical location with measured distress, completing the full mechanistic-empirical loop of the pavement design.

Generally, flexible and rigid pavements respond to loads in such different ways that there are fundamental differences in the analysis theories applied. Basically, for rigid pavement slabs non-linearity of the stress-strain relationship is not an issue, but discontinuities such as cracks and joints are of major importance. Essentially, the opposite is true with flexible pavements—non-linearity of the stress-strain relationship is a major issue while discontinuities are secondary or non-existent. Rehabilitated pavements, especially those with a combination of rigid and flexible layers, are an entirely different class and are handled separately in PART 2 and PART 3 of this Guide.

While the mechanistic approach to pavement design and analysis is much more rational than the empirical, it also is much more technically and computationally demanding. As mentioned earlier, mechanistic-empirical procedures were not practical until the advent of high-speed computers because of the computational demands associated with the differential equations and finite element matrix solutions employed by the various analysis models. Such models are incorporated in this Guide. As will be noted in later descriptions, the choice of a pavement analysis model and how it is applied often will relate to the computational requirements and how much time is required to accomplish those computations.

1.1.3 SCOPE AND CONTENTS OF THE GUIDE

This Guide provides a uniform and comprehensive set of procedures for the design of new and rehabilitated flexible and rigid pavements. The Guide employs common design parameters for traffic, subgrade, environment, and reliability for all pavement types. Recommendations are provided for the structure (layer materials and thickness, as in figure 1.1.3) of new and rehabilitated pavements, including procedures to select pavement layer thickness, rehabilitation treatments, subsurface drainage, foundation improvement strategies, and other design features. The procedures can be used to develop alternate designs using a variety of materials and construction procedures.

Materials from the 1986 and 1993 Guides relating to reliability and to rehabilitation design have been updated to incorporate mechanistic approaches and broadened to include rehabilitation considerations not included earlier. A suggested life cycle cost analysis procedure also has been prepared and presented in appendix C.

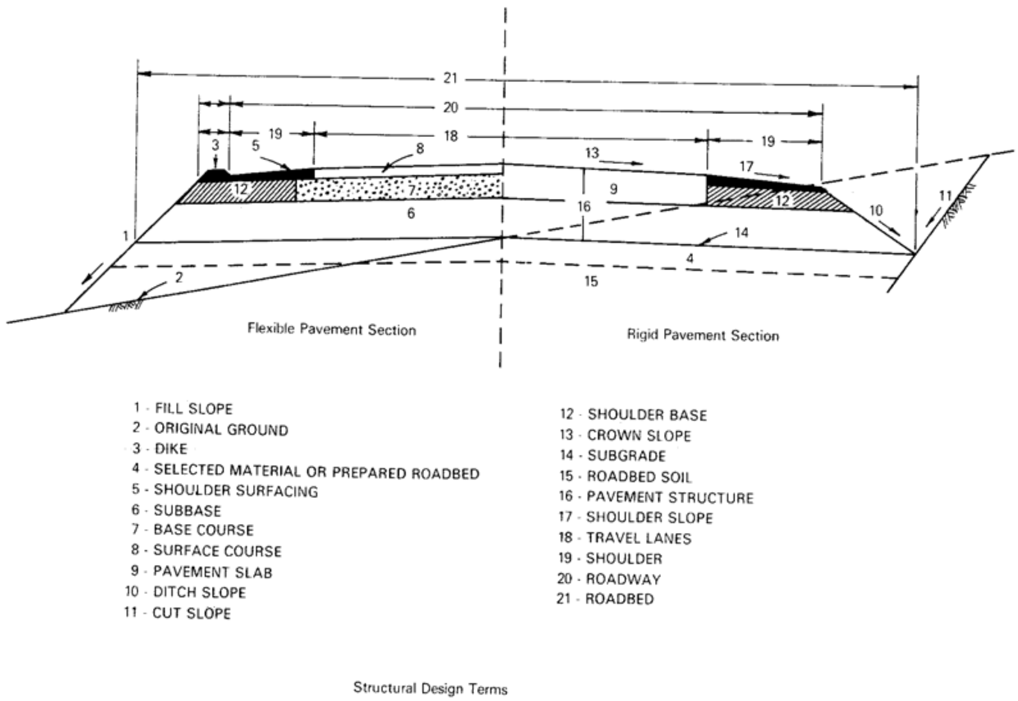

Figure 1.1.3. Typical sections for rigid and flexible pavement structures.

A glossary of terms used in this Guide is provided in appendix A. Some of the terms used herein may differ from those used in local practice; however, in the interest of seeking some form of a national terminology, AASHTO and other accepted definitions are used as much as possible.

The designer should keep in mind that "pavement design" involves far more than choosing layer thicknesses. For example, materials requirements, construction requirements, joint design, and quality assurance will influence the ability of the pavement structure to perform according to design expectations. Information concerning material and construction requirements is described in this Guide; however, to supplement the Design Guide materials a designer must be familiar with relevant publications of AASHTO and ASTM International (ASTM), as well as the local agencies for which a given design is being prepared. It is extremely important that the designer prepare special provisions to the standard specifications when circumstances indicate that nonstandard conditions exist for a specific project. Examples of such a condition could involve a roadbed soil that is known to be expansive or nonstandard materials that are to be stabilized for use in the pavement structure or prepared roadbed.

PART 2 of this Guide includes definitions and procedures on how to obtain the necessary inputs for the design of new or reconstruction projects, as well as for rehabilitation projects. For rehabilitation design, PART 2, Chapter 5 discusses procedures for identifying feasible project-level rehabilitation strategies and procedures for identifying preferred rehabilitation strategies for flexible, composite, and rigid pavements.

PART 3 of this Guide includes detailed background material to assist the user in understanding the concepts used in the development of the Guide and in the proper application of the design procedures. All aspects of pavement design including subsurface drainage design, shoulder design, as well as the design of new, reconstruction, and rehabilitation flexible and rigid pavement projects are discussed in this part of the Guide.

PART 4 discusses how the mechanistic-empirical process can be applied in designing pavements subjected to low traffic volumes.

Appendices A through D contain detailed information related to a number of additional design considerations. For example, appendix B provides an extensive list of guidelines for comparing alternate design strategies. Similarly, appendix C includes information concerning economic evaluation of alternate pavement design strategies. However, the selection of a pavement design should not be based on economics alone. There are a number of considerations involved in the final design selection. Appendix D contains a software User's Guide and detailed examples of each major design and rehabilitation alternative.

Extensive background information and documentation of many aspects of the design procedure are provided in additional appendixes (AA, BB, CC, etc.). References used in the preparation of the Guide can be found following each of the chapters.

1.1.4 DESIGN APPROACH

1.1.4.1 General Approach

This Design Guide represents a major change in the way design is performed. The design approach provided in this Guide consists of three major stages, summarized in figure 1.1.4.

Stage 1 consists of the development of input values for the analysis. During this stage, potential strategies are identified for consideration in the analysis stage. A key step of this process is foundation analysis. For new pavements, the foundation analysis consists of stiffness determination and, where appropriate, an evaluation of volume change, frost heave, thaw weakening, and drainage concerns (see PART 2, Chapter 1). As part of the foundation analysis, subgrade improvements such as strengthening and drainage are considered.

The foundation analysis for rehabilitation projects also includes a subgrade analysis (see PART 2, Chapter 5). However, the most important part of the foundation analysis for rehabilitation projects is the investigation of distress types occurring in the existing pavements and the underlying causes of those distresses. The overall strength/stiffness of the existing pavement is evaluated using deflection testing and backcalculation procedures.

Also during the first stage, pavement materials characterization (see PART 2, Chapter 2) and traffic input (see PART 2, Chapter 4) data are developed. The Enhanced Integrated Climate Model (EICM), a powerful climatic effects modeling tool, is used to model temperature and moisture within each pavement layer and the subgrade. The version of the EICM incorporated in the Guide is based on improvements to an earlier version of the Integrated Climatic Model (*8*).

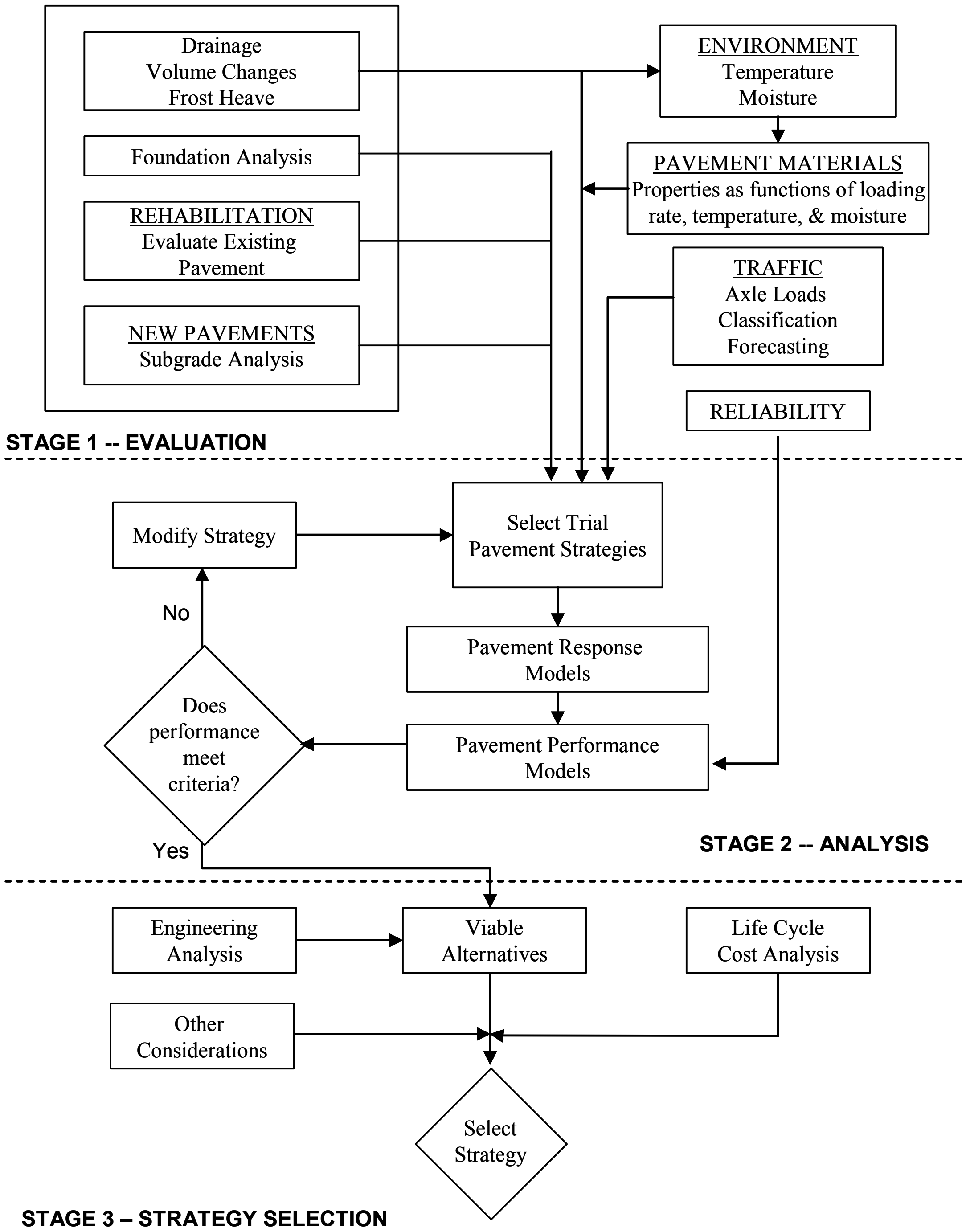

Figure 1.1.4. Conceptual schematic of the three-stage design process.

The climatic model considers hourly climatic data from weather stations across the country (temperature, precipitation, solar radiation, cloud cover, and wind speed). The pavement layer temperature and moisture predictions from the EICM are calculated hourly over the design period and used in various ways to estimate material properties for the foundation and pavement layers throughout the design life (see PART 2, Chapter 3). The frost depth is determined, and the proper moduli are estimated above and below this depth.

A detailed systematic approach for the use of subdrainage in new or reconstructed pavements as well as rehabilitated pavements is included in this Guide. Both an engineering and cost-effectiveness evaluation are included to determine feasibility. The Design Guide describes how to perform subdrainage design (including hydraulic design using the FHWA software DRIP included with the Design Guide software) which ultimately leads to the preparation of cross-sections with adequate drainage features. The structural section is then evaluated for structural distresses using the mechanistic-empirical approach (PART 3, Chapter 1 and Appendices SS and TT). The practical considerations of subsurface drainage design are also discussed to help augment the Guide's design approach by addressing factors that cannot be directly input into the design equations.

Stage 2 of the design process is the structural/performance analysis. The analysis approach is an iterative one that begins with the selection of an initial trial design. Initial trial designs can be created by the designer, obtained from an existing design procedure, or from a general catalog. (*9*). The trial design requires initial estimates of layer thickness, geometric features, initial smoothness, required repairs to the existing pavements, pavement materials characteristics, and many other inputs. The trial section is analyzed incrementally over time using the pavement response and distress models, and the outputs of the analysis are accumulated damage the expected amount of distress and smoothness over time. If the trial design does not meet the performance criteria, modifications are made and the analysis re-run until a satisfactory result is obtained. If subdrainage features (e.g., permeable layer) are included, viable drainage alternatives should be selected and properly designed from a hydraulic and structural standpoint. Guidelines on materials selection, construction, and maintenance of the drainage systems are provided. Subdrainage is not a substitute for poor design; adequate structure must be present in order for these systems to function effectively.

Stage 3 of the process includes those activities required to evaluate the structurally viable alternatives. These activities include an engineering analysis and life cycle cost analysis of the alternatives.

In summary, the design process for new and rehabilitated pavement structures includes consideration of the following:

- Foundation/subgrade.
- Existing pavement condition.
- Paving materials.
- Construction factors.
- Environmental factors (temperature and moisture).
- Traffic loadings.

- Subdrainage.
- Shoulder design.
- Rehabilitation treatments and strategies.
- New pavement and rehabilitation options.
- Pavement performance (key distresses and smoothness).
- Design reliability.
- Life cycle costs.

Each of these factors is discussed briefly in this chapter, and later parts of the Guide incorporate them into the pavement design methodology.

It is worth noting that, although the Guide describes and provides a specific method that can be used for the determination of alternate design or rehabilitation recommendations for the pavement structure, there are a number of considerations left to the user for final determination. Some of these are drainage provisions, pavement surface friction and texture, special environmental considerations, and local practices that affect the design.

The Guide 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, even within a State.

The Guide attempts to provide procedures for evaluating materials and environment; however, if the Guide is at variance with proven and documented local experience, the proven experience should prevail. For example, material requirements and construction specifications are not detailed in this Guide, yet they are an important consideration in the overall design of a pavement structure. The effect of seasonal variations on material properties and evaluation of future traffic for the designed project should be investigated thoroughly.

1.1.4.2 Hierarchical Design Inputs

The hierarchical approach to design inputs is a feature of the Design Guide not found in existing versions of the AASHTO Guide for the design of pavement structures. This approach provides the designer with a lot of flexibility in obtaining the design inputs for a design project based on the criticality of the project and the available resources. The hierarchical approach is employed with regard to traffic, materials, and environmental inputs. In general, three levels of inputs are provided.

Level 1 inputs provide for the highest level of accuracy and, thus, would have the lowest level of uncertainty or error. Level 1 inputs would typically be used for designing heavily trafficked pavements or wherever there are dire safety or economic consequences of early failure. Level 1 material input require laboratory or field testing, such as the dynamic modulus testing of hot-mix asphalt concrete, site-specific axle load spectra data collections, or nondestructive deflection testing. Obtaining Level 1 inputs requires more resources and time than other levels.

Level 2 inputs provide an intermediate level of accuracy and would be closest to the typical procedures used with earlier editions of the AASHTO Guide. This level could be used when resources or testing equipment are not available for tests required for Level 1. Level 2 inputs typically would be user-selected, possibly from an agency database, could be derived from a limited testing program, or could be estimated through correlations. Examples would be estimating asphalt concrete dynamic modulus from binder, aggregate, and mix properties, estimating Portland cement concrete elastic moduli from compressive strength tests, or using site-specific traffic volume and traffic classification data in conjunction with agency-specific axle load spectra.

Level 3 inputs provide the lowest level of accuracy. This level might be used for design where there are minimal consequences of early failure (e.g., lower volume roads). Inputs typically would be user-selected values or typical averages for the region. Examples include default unbound materials resilient modulus values or default Portland cement concrete coefficient of thermal expansion for a given mix classes and aggregates used by an agency.

For a given design project, inputs may be obtained using a mix of levels, such as concrete modulus of rupture from Level 1, traffic load spectra from Level 2, and subgrade resilient modulus from Level 3. In addition, it is important to realize that no matter what input design levels are used, the computational algorithm for damage is exactly the same. The same models and procedures are used to predict distress and smoothness no matter what levels are used to obtain the design inputs.

1.1.5 PAVEMENT PERFORMANCE

The concept of pavement performance includes 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 by NCHRP, FHWA, and AASHTO. One important aspect of safety is the frictional resistance provided at the pavement/tire interface. On this subject, AASHTO has issued a publication titled, *Guidelines for Skid Resistant Pavement Design* (*10*) in 1976. Renewed guidance on this topic will be published under the title *Guide for Pavement Friction*.

The structural performance of a pavement relates to its physical condition (fatigue cracking and rutting for flexible pavements, and joint faulting, and slab cracking for rigid jointed pavements, or other conditions that would adversely affect the load-carrying capability of the pavement structure or would require maintenance). Several of these key distress types can be predicted directly using mechanistic concepts and are directly considered in the design process.

The functional performance of a pavement concerns how well the pavement serves the highway user. Note that adequate geometry for the design speed is assumed. Riding comfort or ride quality is the dominant characteristic of functional performance. To quantify riding comfort, the "serviceability-performance" concept was developed by the AASHO Road Test staff in 1957 (*11*). All previous versions of the Guide used empirical design equations built around the serviceability-performance concept. In that approach, the serviceability of a pavement is expressed in terms of the mean Present Serviceability Rating (PSR) of a panel of highway users.

The PSR was correlated with measurements of pavement condition and called Pavement Serviceability Index (PSI). Pavement performance was represented by the serviceability history of a given pavement. The PSI is obtained from measurements of roughness and distress (e.g., cracking, patching, and rut depth for flexible pavements) at a given time during the service life of the pavement. Longitudinal profile is the dominant factor in estimating the PSI of a pavement and is, therefore, the principal component of performance.

In this Guide, the chosen functional performance indicator is pavement smoothness as indicated by the International Roughness Index (IRI). The IRI was adapted as a standard measure of smoothness for the following reasons:

- The IRI is a computed statistic of road profile and can be easily produced from elevation data along the wheel path.
- The correlations between IRI and other smoothness measures are consistently high at various speeds.
- The IRI has been shown to correlate well with user panel serviceability ratings. (*12*)

The practical approach employed is to begin with the initial as-constructed IRI (which is dependent on construction smoothness specifications) and then predict changes in IRI over time as a function of pavement distress, site conditions, and maintenance. The details of this approach are described in PART 3, Chapters 3 and 4, as well as in related appendixes.

The smoothness-performance concept is based on the following assumptions:

- Highways are for the comfort and convenience of the traveling public, and this is only possible through the provision of smooth pavements.
- Comfort and ride quality, though subjective, can be related accurately to the measured profile of the pavement (the correlation is the same regardless of type of surfacing).
- Measured smoothness is the dominant factor in predicting serviceability and serviceability-based performance as defined in the AASHO Road Test (i.e., performance is represented by the serviceability history of the pavement).
- Certain pavement distresses can be measured objectively and related to smoothness.
- The term *smoothness* is used herein (instead of *roughness*) because it denotes a positive aspect of the pavement.

The 1993 AASHTO design procedure selects design features of a given pavement to satisfy performance criteria based on initial and terminal serviceability. The initial serviceability, p_i, measured in terms of PSI, is an estimate of the PSI immediately after construction. The terminal serviceability, p_t, also measured in terms of PSI, is the lowest acceptable level of performance before maintenance or rehabilitation.

Based on these serviceability values and typical initial smoothness being obtained with today's construction procedures and smoothness specifications, the following typical initial smoothness (IRI_i) and terminal smoothness (IRI_t) ranges are provided for all types of new or rehabilitated pavements as default values in the software. Of course, the designer can enter different values as desired. Agencies having an effective long-term smoothness specification can achieve IRI values towards the lower end of this range.

- Typical initial as-constructed IRI_i = 50 to 100 in/mi.
- Typical terminal IRI_t = 150 to 200 in/mi.

The major factors influencing loss of smoothness of a pavement are distresses such as cracking, rutting, faulting, and punchouts, which are in turn influenced by design, materials, subgrade, traffic, age, and environment. Smoothness is accounted for by considering the initial smoothness at construction and subsequent changes over the pavement life by summing the damaging effect of distress, as shown in equation 1.1.4 and illustrated in figure 1.1.5.

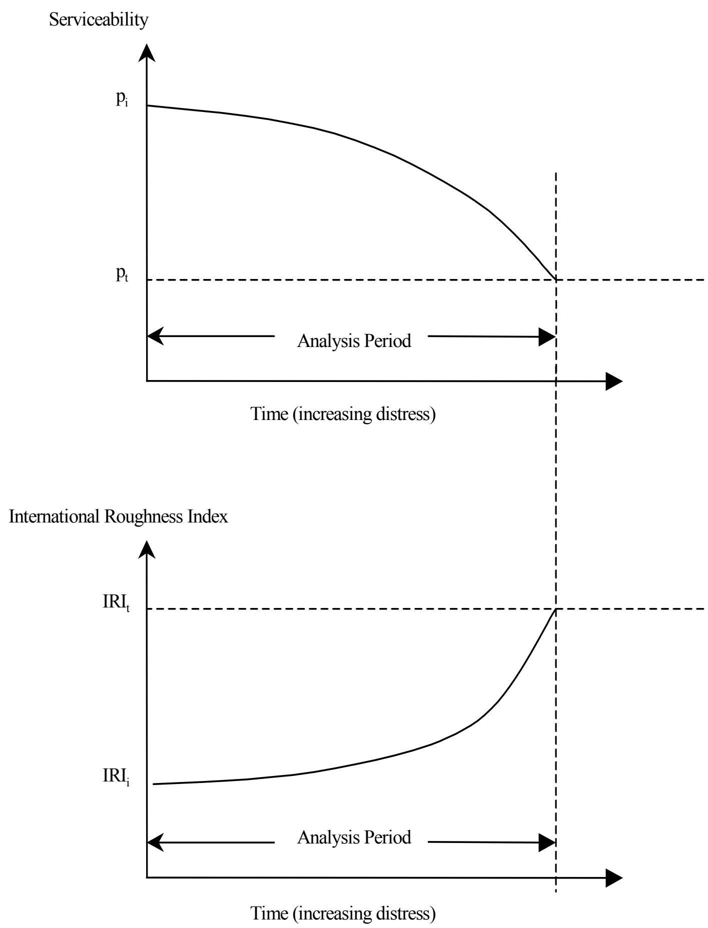

Figure 1.1.5. Pavement performance trends illustrated using serviceability (used in previous versions of the AASHTO Guide) and IRI values (used in this Guide).

The smoothness model form is based on an additive combination of initial smoothness, change in smoothness due to the increase in individual distress, change in smoothness due to site conditions, and maintenance activities. Figure 1.1.5 shows pavement performance based on the existing serviceability, PSI, and IRI. The pavement design process in this Guide provides design features for existing site conditions that ensure that the terminal smoothness values is with acceptable limits for a given design.

$$S(t) = S_0 + (a_1 S_{D(t)1} + a_2 S_{D(t)2} + \ldots\ldots\ldots\ldots + a_n S_{e(t)n}) + b_j S_j + c_j M_j \qquad (1.1.4)$$

where

$S(t)$	=	pavement smoothness at a specific time, t (IRI, in/mi).
S_0	=	initial smoothness immediately after construction (IRI, in/mi).
$S_{D(t)(i = 1 \text{ to } n)}$	=	change of smoothness due to i^{th} distress at a given time t in the analysis period.
$a_{(i = 1\ldots n)}$, b_j, c_j	=	regression constants.
S_j	=	change in smoothness due to site factors (subgrade and age)
M_j	=	change in smoothness due to maintenance activities.

1.1.6 TRAFFIC CHARACTERIZATION

The Guide considers truck traffic loadings in terms of axle load spectra, as described in detail in PART 2, Chapter 4. The full axle load spectra for single, tandem, tridem, and quad axles are considered. The software uses the number of heavy trucks as an overall indicator of the magnitude of truck traffic loadings (FHWA class 4 and above).

The hierarchical levels of traffic data are:

- Level 1, the recommended approach for high volume roads, requires the gathering and analysis of site-specific traffic data, including vehicle count by class and by direction and lane. Axle load spectra distributions are developed for each vehicle class from axle weight data collected at or near the project site. Traffic volumes by vehicle class are forecast for the design analysis period, and the load spectra developed for each class are used to estimate axle loads. Default or user input tire contact pressures, tire spacings, and axle spacings may be used.
- Level 2 is similar to Level 1, requiring site-specific volume and classification data. However, State or regional axle load spectra distributions for each vehicle class may be used to estimate loadings over the design analysis period.
- Level 3 will provide default load spectrum data for a specific functional class of highway. The designer applies these default values to available or estimated vehicle volume data.

1.1.7 PAVEMENT MATERIAL CHARACTERIZATION

1.1.7.1 General Considerations

Material characterization guidelines are provided so the designer can develop appropriate material property inputs for use in the analysis portion of the design process. The material parameters needed for the design process may be classified in one of three major groups:

- Pavement response model material inputs.
- Material-related pavement distress criteria.
- Other material properties.

The pavement response model material inputs relate to the moduli and Poisson's ratio used to characterize layer behavior within the specific model. Bound materials generally display a linear or nearly linear stress-strain relationship. Unbound materials display stress-dependent properties. Granular materials generally are “stress hardening” and show an increase in modulus with an increase in stress. Fine-grained soils generally are “stress softening” and display a modulus decrease with increased stress. Modulus-stress state relations have been developed for granular materials and for fine-grained soils. The details of moduli characterization are discussed in PART 2, Chapter 2. In practice, assumed Poisson's ratio values are acceptable for routine mechanistic-empirical pavement design based on isotropic elastic structural analysis models. This is true because the parameter has well-defined limits for specific materials types and because the stress, strain, and displacement outputs of the response model are not particularly sensitive to the parameter.

Material parameters associated with pavement distress criteria normally are linked to some measure of material strength (shear strength, compressive strength, modulus of rupture) or to some manifestation of the actual distress effect (repeated load permanent deformation, fatigue failure of PCC materials).

The “other” category of materials properties constitutes those associated with special properties required for the design solution. Examples of this category are the thermal expansion and contraction coefficients of both PCC and asphalt mixtures.

1.1.7.2 Classes of Materials

For the purposes of this guide, all pavement materials have been classified in one of the following categories:

- Dense-graded, hot-mix asphalt concrete (HMAC).
- Open-graded, asphalt-treated permeable base (ATPB) materials.
- Cold mix asphalt (CMA).
- Portland cement concrete (PCC).
- Cement treated base (CTB) and lean concrete base (LCB) materials.
- Open-graded, cement-treated permeable (CTPB) materials.
- Non-stabilized aggregate base (AB) materials also referred to as granular aggregate base (GAB) or coarse aggregate (CA) materials.
- Lime modified or lime-stabilized layers.
- Subgrade soils.
- Bedrock.

These categories are more fully defined in PART 2, Chapter 2, where testing procedures are described and typical values for Poisson's ratio are given.

1.1.7.3 Levels of Materials Characterization

Material characterization inputs are defined at three levels designated as Levels 1, 2, and 3. Level 1 data are derived from laboratory or field testing. Level 2 data are obtained using correlations with available tests. Level 3 data typically are default values. The details of hierarchical input characterization are given in PART 2, Chapter 2.

1.1.8 STRUCTURAL MODELING OF THE PAVEMENT

1.1.8.1 Structural Response Models

Proper structural modeling of new and rehabilitated flexible and rigid pavement structures is the heart of a mechanistic-based design procedure. Structural response models are used to compute critical stresses, strains, and displacements in flexible and rigid pavement systems due to both traffic loads and climatic factors (temperature and moisture). These responses are then utilized in damage models to accumulate damage, month by month, over the design period. The accumulated damage at any time is related to specific distresses such as fatigue cracking or rutting, which is then predicted using a field calibrated cracking model (this is the main empirical part of a mechanistic-empirical design procedure).

The structural models selected for use in this Guide for flexible pavements include the multi-layer elastic program JULEA for linear elastic analysis (LEA). If the user opts to use the Level 1 hierarchical inputs to characterize the non-linear moduli response of any unbound layer materials (bases, subbases and/or subgrades), then the 2-D finite element program DSC2D is utilized (*13*) to conduct finite element analysis (FEA). However, this approach has not been calibrated and is only recommended for research purposes at this time. The neural network (NN) approach to provide more rapid solutions was not used to solve for the LEA responses due to the complexity of the problem.

The structural model for rigid pavement analysis is a 2-D finite element program, ISLAB2000, developed under a pooled fund study (*14*). Because thousands of computations of responses are needed for any design, this FEA-based structural model was used as a basis for developing rapid solution neural networks (NN). The neural networks were trained with the thousands of results from ISLAB2000. These NN provide accurate and virtually instantaneous solutions for critical responses and were developed so that the large numbers of computations needed could be accomplished rapidly.

These structural response models require several inputs, including the following for each month over the entire design period:

- Traffic loading.
- Pavement cross-section.
- Poisson's ratio each layer.
- Elastic modulus each layer.
- Thickness each layer.

- Layer to layer friction (for PCC to base).
- Coefficient of thermal contraction and expansion for HMA and PCC, respectively.
- Temperature in HMA materials and temperature gradient and moisture gradient in PCC slab.

Given these inputs, the structural models produce stresses, strains, and displacements at critical locations in the pavement and subgrade layers.

1.1.8.2 Incremental Damage Accumulation

This design procedure is the first to include the capability to accumulate damage on a monthly basis (or semi-monthly, depending on frost conditions) over the entire design period. This approach attempts to simulate how pavement damage occurs in nature, incrementally, load by load, over continuous time periods. By accumulating damage semi-monthly or monthly, the design procedure becomes very versatile and comprehensive. The major advantages of the incremental damage accumulation approach are as follows:

- The design procedure accumulates damage similar to how it occurs in the field, incrementally.
- The increments (typically monthly) are selected to match climatic (temperature and moisture) changes that cause changes in layer materials, changes in joint openings, changes in traffic loadings, and material aging and property changes over time.
- The effect of traffic loadings during daytime and nighttime (due to differences in temperature gradients) can be considered.

This approach allows the use of elastic moduli within a given time period, such as a month, that are representative of that time increment. For example, in the heat of summer, the dynamic modulus of HMA (E*) is much lower than in the cold of winter. The resilient modulus of an unbound base course and of the fine-grained subgrade can vary with degree of saturation. This procedure also allows for the aging of paving materials. For example, asphalt materials age with time, increasing their stiffness. This is modeled so that the E* of the asphalt mix is constantly increasing over time. The same is true for PCC slabs. The PCC ages month by month and year by year over the design period. This change in elastic modulus (E_c) and flexural strength of the PCC is utilized in the design procedure. Thermal gradients are different in summer than in winter months. It is believed that the added capabilities of incremental damage accumulation far outweigh its main disadvantage of computation time.

1.1.8.3 Analysis of Trial Design

The approach is iterative and begins with the selection of initial trial designs. A trial design is selected based on past agency experience or on general design catalogs (*9*). Each design strategy analyzed includes all details, such as initial estimates of layer thickness, required repairs to the existing pavement, and pavement materials characteristics. The trial sections are analyzed by accumulating incremental damage over time using the pavement structural response and performance models. The outputs of the analysis (the expected amounts of damage over time) are then used to estimate distress over time and traffic through calibrated distress models.

Modifications are made to the trial strategies and further iterations performed until a satisfactory design that meets the performance criteria and design reliability is obtained.

1.1.9 EVALUATION OF EXISTING PAVEMENTS FOR REHABILITATION

The Guide includes procedures and guidance for performing project-level evaluation of pavement structures for identifying rehabilitation alternatives (see PART 3, Chapter 5) and in rehabilitation design (see PART 3, Chapters 6 and 7). It also provides guidance for determining those inputs that are considered essential for the different types of rehabilitation design.

Some of the input data discussed include:

- Traffic lane pavement condition (e.g., distress, smoothness, surface friction, and deflections).
- Condition of pavement-shoulder interface.
- Pavement design features (e.g., layer thicknesses, structural characteristics, and construction requirements).
- Material and soil properties.
- Traffic volumes and loadings.
- Climatic conditions.
- Drainage conditions.
- Geometric factors (e.g., bridge clearance).
- Safety aspects (e.g., rate and location of accidents).
- Miscellaneous factors (e.g., utilities and clearances).

The project-level evaluation presented in this chapter covers three common pavement structures—flexible, rigid, and composite (HMA/PCC). Also presented in this chapter are the procedures used for pavement evaluation and assessing existing pavement condition. Overall pavement condition and problem definition is determined by evaluating the following major aspects of the existing pavement:

- Structural adequacy (load related).
- Functional adequacy (user related).
- Subsurface drainage adequacy.
- Material durability.
- Shoulder condition.
- Variation of pavement condition or performance within a project.
- Miscellaneous constraints (e.g., bridge and lateral clearance and traffic control restrictions).

The structural category relates to those properties and features that define the response of the pavement to traffic loads. The data will be used in mechanistic-empirical design of rehabilitation alternatives. The functional category relates to the surface and subsurface characteristics and properties that define the smoothness of the roadway or to those surface characteristics that define the frictional resistance or other safety characteristics of the pavement's surface.

Subsurface drainage and material durability may affect both structural and functional condition. Shoulder condition is very important in terms of rehabilitation type selection and in affecting project cost. Variation within a project refers to areas where there is a significant variability in pavement condition. Such variation may occur along the length of the project, between lanes (truck lane versus other lanes), among cut and fill portions of the roadway, and at bridge approaches, interchanges, or intersections.
Miscellaneous factors, such as joint condition for jointed concrete pavements and reflection cracking for composite pavements, are important to the overall condition of such pavements but must be evaluated only where relevant.

Lastly, project-level pavement evaluation includes the identification of constraints that may be encountered during rehabilitation (such as the availability of adequate bridge clearance for placing overlays and traffic control restrictions) are documented. This is very important for selecting feasible rehabilitation alternatives and for life cycle cost analysis.

1.1.10 IDENTIFICATION OF FEASIBLE REHABILITATION STRATEGIES

The Guide provides an overview of strategies for the rehabilitation of existing flexible, rigid, and composite pavements. A feasible rehabilitation strategy is one that addresses the cause of pavement distress and deterioration and is effective in both repairing it and preventing or minimizing its reoccurrence. A feasible rehabilitation strategy must meet critical constraints such as traffic control. Repair treatments are actions taken to restore the pavement's integrity (i.e., to repair the problem definitively), such as filling a pothole. Prevention treatments are actions taken to stop or delay the deterioration process, such as a structural overlay to reduce critical deflections.

Rehabilitation strategy is defined as a combination of repair and preventive treatments (ranging from simple repair treatments such as crack sealing to complex treatments such as the placement of overlays) performed over a defined period to restore the ability of an existing pavement to carry expected future traffic with adequate functional performance.

This chapter covers major rehabilitation treatments and strategies typically used in the rehabilitation of flexible, rigid, and composite pavements. It also presents general guidelines for identifying feasible rehabilitation strategies and then for selecting the preferred rehabilitation strategy for use on a specific project.

The following is a list of the common major rehabilitation options that may be applied singly or in combination to obtain an effective rehabilitation strategy:

- Reconstruction without lane additions.
- Reconstruction with lane additions.
- Structural overlay (may include removal and replacement of selected pavement layers).
- Non-structural overlay.
- Restoration without overlays (PCC pavements).

1.1.11 DESIGN OF REHABILITATION PROJECTS

The Guide presents procedures for utilizing the results for the evaluation of the existing flexible, rigid, and composite pavements and the design of HMA and PCC rehabilitation for these pavements. The structural design of these rehabilitations is very similar to that of mechanistic-empirical design of new or reconstructed pavement, with the major exception that the existing pavement condition is considered fully. A trial rehabilitation design must first be proposed and it is then evaluated using the Design Guide software which predicts the key distress types and smoothness. Its adequacy is checked and revisions made if necessary to meet the performance criteria.

1.1.12 DESIGN RELIABILITY

Practically everything associated with the design of new and rehabilitated pavements is variable or uncertain in nature. Perhaps the most obviously uncertain of all is estimating truck axle loadings many years into the future. Materials and construction also introduce a significant measure of variability. Furthermore, pavements exhibit significant variation in condition along their length.

Even though mechanistic concepts provide a more accurate and realistic methodology for pavement design, a practical method to consider the uncertainties and variations in design is needed so that a new or rehabilitated pavement can be designed for a desired level of reliability.

Reliability has been incorporated in the Guide in a consistent and uniform fashion for all pavement types. An analytical solution that allows the designer to design for a desired level of reliability for each distress and smoothness is available. Design reliability is defined as the probability that each of the key distress types and smoothness will be less than a selected critical level over the design period.

$$R = P\,[\text{ Distress over Design Period} < \text{Critical Distress Level }] \tag{1.1.5}$$

Design reliability is defined as follows for smoothness (IRI) as follows:

$$R = P\,[\text{ IRI over Design Period} < \text{Critical IRI Level }] \tag{1.1.6}$$

For example, the reliability for fatigue cracking is defined as follows:

$$R = P\,[\text{ Fatigue Cracking over Design Period} < 20 \text{ percent lane area }] \tag{1.1.7}$$

Note that this definition varies from the previous versions of the AASHTO Design Guide in that it considers each key predicted distress and the IRI directly in the definition. Previous Guides define reliability in terms of the number of predicted equivalent single axle loads to terminal serviceability (N) being less than the number of equivalent single axle loads actually applied (n) to the pavement.

$$R = P\,[\, N < n \,] \tag{1.1.8}$$

This approach produced results that indicated that thicker pavements always increased design reliability. However, this is not always be true for the key performance measures adopted in this Guide. In the approach taken in the Guide, several design features other than thickness (e.g., HMAC mixture design, dowels for jointed plain concrete pavements, and subgrade improvement for all pavement types) can be considered to improve the reliability estimate of the design.

The designer begins the design process by configuring a trial design. The software accompanying the Design Guide procedure then provides a prediction of key distress types and smoothness over the design life of the pavement. This prediction is based on mean or average values for all inputs, as illustrated in figure 1.1.6. The distresses and smoothness predicted therefore represent mean values that can be thought of as being at a 50 percent reliability estimate (i.e., there is a 50 percent chance that the predicted distress or IRI will be greater than or less than the mean prediction).

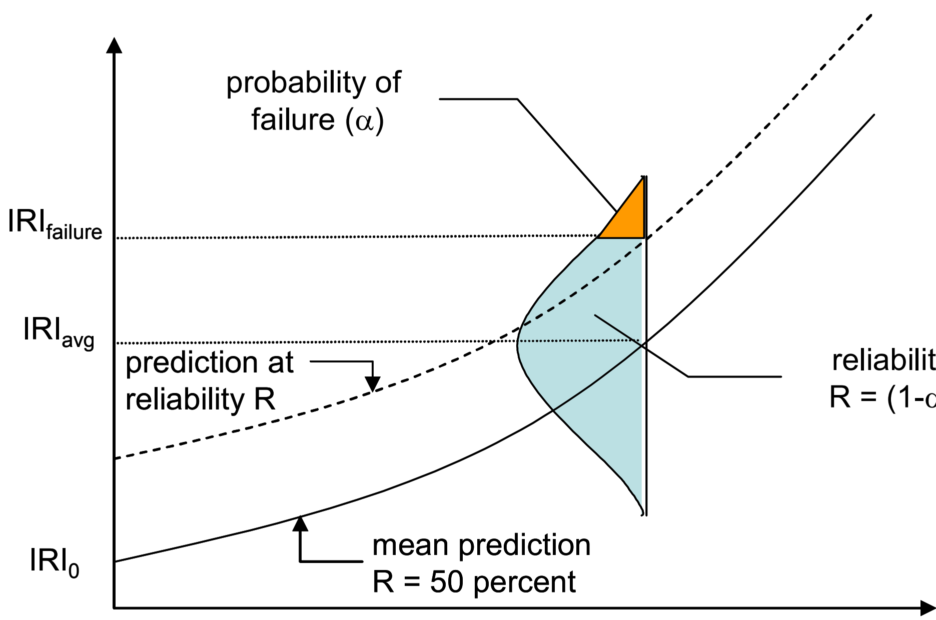

Figure 1.1.6 Design reliability concept for smoothness (IRI).

For nearly all projects, the designer will require a higher probability that the design will meet the performance criteria over the design life. In fact, the more important the project in terms of consequences of failure, the higher the desired design reliability. The consequence of failure of an urban freeway is far more than the failure of a farm-to-market roadway. Often, agencies have used the level of traffic volume or truck traffic as the parameter for selecting design reliability.

Due to the error associated in predicting pavement distresses and smoothness using transfer functions, the actual distress or IRI could be higher or lower than the mean expected value. The distribution of the error term for a given distress or IRI about the mean expected prediction is a function of the many sources of variation and uncertainty, including:

- Errors in estimating traffic loadings.
- Fluctuations in climate over many years.
- Variations in layer thicknesses, materials properties, and subgrade characteristics along the project.
- Differences between as-designed and as-built materials and other layer properties.
- Errors in the measurement of the distress and IRI quantities.
- Prediction model limitations and errors.

The probability distributions of the key performance measures about their mean values are important in establishing design reliability for each measure. Figure 1.1.6 illustrates a probability distribution for IRI, and figure 1.1.7 illustrates a probability distribution for cracking. From these figures, the probability, R, that each distress or IRI is greater than its associated user-defined failure criteria is computed over the design life and plotted (reliability predictions at an arbitrary level R above the mean predictions are shown as dashed lines in the figures).

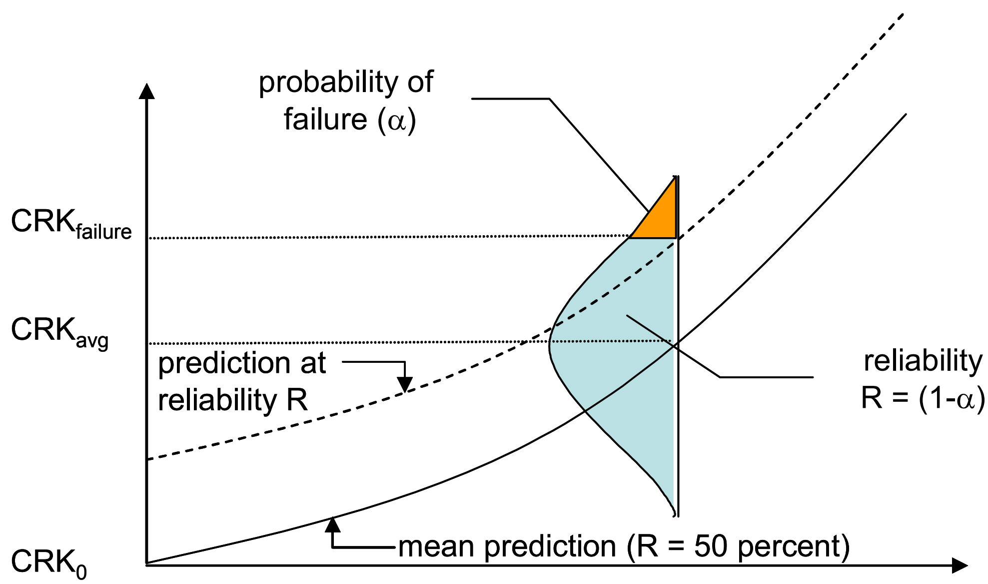

Figure 1.1.7. Design reliability concept for a given distress.

The shape and width of the probability distribution of the random variate (each distress and IRI in question) are important and must be known *a priori* in this approach. Distress and IRI are approximately normally distributed over ranges of the distress and IRI that are of interest in design. The standard deviation for each distress type was determined from the model prediction error (standard error of the estimate) from calibration results used for each key distress. Each model was calibrated from LTPP and other field performance data. The error of prediciton of, say, rutting was obtained as the difference of predicted rutting and measured rutting results for all sections in the database. This difference, or residual error, contains all available information on the ways in which the prediction model fails to properly explain the observed rutting. As stated, the distribution of the distress about the mean prediction was assumed to be normal for each month within the range of interest for design. The standard deviation of IRI was determined using a closed form variance model estimation approach. This approach accounts for all sources previously listed, but not for predicting future traffic growth.

The standard deviation of the distribution of distress is determined as a function of the predicted cracking. Figure 1.1.8 shows an illustration of data from one distress type (slab cracking) that shows that the standard deviation of cracking prediction (from calibration data) is related to the mean cracking. Thus, for any mean cracking prediction, an estimate of the standard deviation of cracking can be obtained from this result.

Given the mean and standard deviation of a normal distribution, the reliability of the design can be calculated using the following steps:

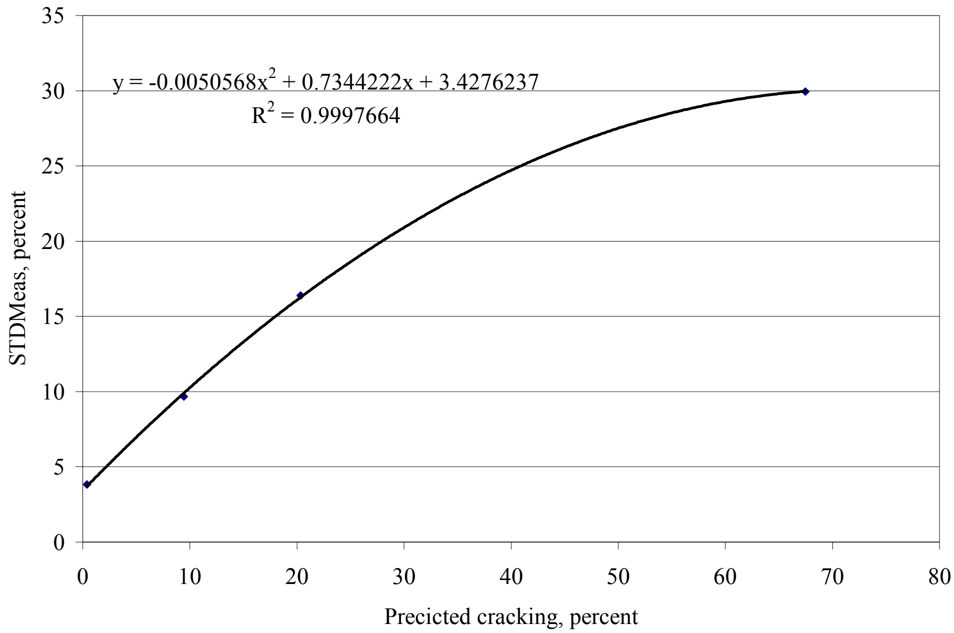

Figure 1.1.8. Standard deviation of measured cracking vs predicted cracking obtained from calibration data results.

1. Using the Design Guide cracking model, predict the cracking level over the design period using mean inputs to the model. This corresponds approximately to a "mean" slab cracking due to symmetry of residuals.

2. Estimate cracking at the desired relability level using the following relationship:

$$CRACK\_P = CRACK\_mean + STDmeas * Zp \qquad (1.1.9)$$

where,

CRACK_P = cracking level corresponing to the reliability level p.
CRACK_mean = cracking predicted using the deterministic model with mean inputs (corresponing to 50 percent reliabality).
STDmeas = standard deviation of cracking corresponding to cracking predicted using the deterministic model with mean inputs
Zp = standardized normal deviate (mean 0 and standard deviation 1) corresponding to reliability level p.

Desired levels of reliability for each distress can be based on the functional class of the roadway being designed. Some broad guidelines are provided in table 1.1.1; however, each agency must establish these values for design.

Figure 1.1.9 shows predicted cracking for different relaibility levels for a specific jointed plain concrete pavement section in the Long Term Pavement Performance (LTPP) program. An increase in reliability level leads to a reasonable increase in predicted cracking.

Table 1.1.1. Illustrative levels of reliability for new and rehabilitation design (each agency should evaluate and select levels appropriate for their conditions).

Functional Classification	**Recommended Level of Reliability**	
	Urban	**Rural**
Interstate/Freeways	85 – 97	80 – 95
Principal Arterials	80 – 95	75 – 90
Collectors	75 – 85	70 – 80
Local	50 – 75	50 – 75

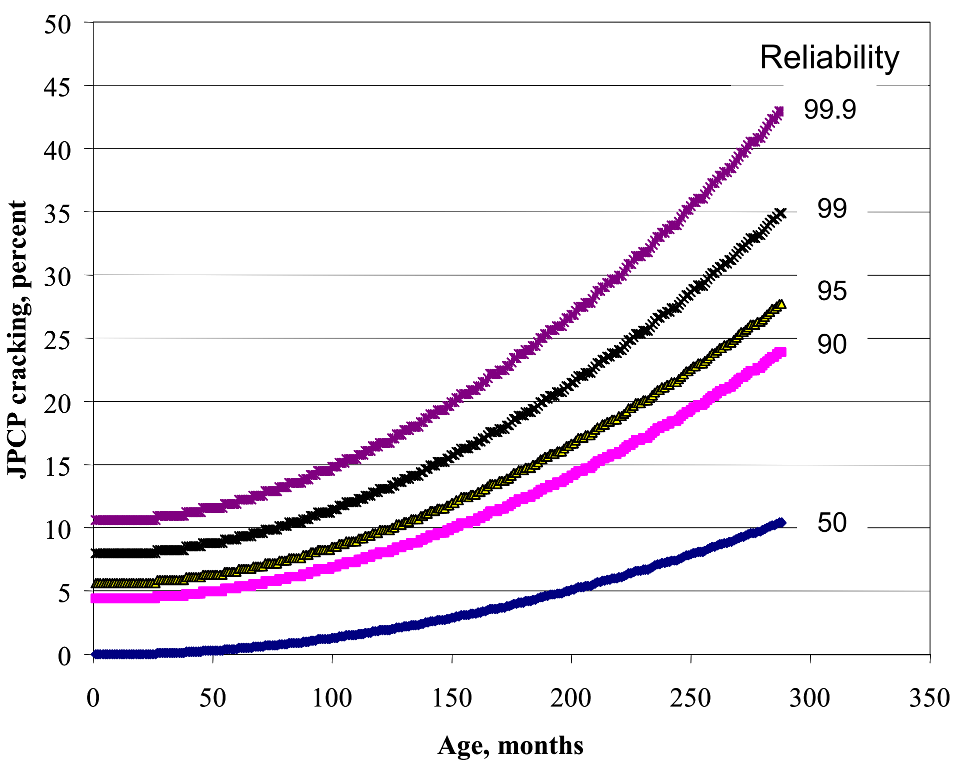

Figure 1.1.9. Cracking estimation at different levels of reliability.

If a pavement designer wants a 90 percent reliability for cracking, then the predicted 90 percent curve must not exceed some preselected critical value of cracking. This level is selected by the designer prior to conducting the pavement design. The levels of reliability used for design used for this Design Guide are not the same as those used in the previous version of the AASHTO Design Guide.

For example, in figure 1.1.9 it can be seen that the mean predicted cracking (at a 50 percent reliability level) is 10 percent at the end of 300 months. On the other hand, the predicted cracking at a 90 percent reliability level is 24 percent at the end of 300 months. Comparing these predictions against a user-defined design criterion that the trial design should not have more than 20 percent area cracked at the end of 300 months at a 90 percent reliability level, the design shown in figure 1.1.9 is not adequate. Thus, the design must be altered iteratively so that the mean cracking is low enough to ensure that the design meets the performance criteria.

Selection of an appropriate level of reliability for design is a challenging task. Some specific guidance is provided as follows:

- The design reliability must be selected for each distress type and for IRI. The levels for each distress type and for IRI do necessarily need to be equal.
- There is an inherent connection between the levels selected for each distress type and IRI and the level of design reliability. For example, a designer could select either a 97 percent reliability level for cracking with 20 percent area as the critical level, or a lower reliability, say 80 percent, and 10 percent area as the critical level.
- The more important the project in terms of consequences of failure (such as closures of traffic lanes over time causing massive congestion), the higher the design reliability. The consequence of failure of an urban freeway is far more than the failure of a farm to market pavement. Often, agencies have used the level of traffic volume or truck traffic as the parameter for selecting design reliability.
- The use of smoothness (IRI) as the "overall" design reliability would be similar to the previous use of serviceability index. Use of the IRI as the "main" definition of design reliability by itself is not recommended because failure of the pavement depends more on any key distress maintaining a reasonable value. It is recommended to use all distresses and IRI and to ensure that they all meet the design reliability requirements.
- Selection of design reliability must be done in conjunction with the selection of the critical level of distress or IRI. Joint selection of high design reliability and low distress criteria might make it impossible to obtain an acceptable trial design.

1.1.13 IMPLEMENTATION OF THE GUIDE WITHIN AN AGENCY

The Guide identifies and discusses the various issues an agency will need to address in order to implement the pavement design procedure described within. Included are discussions and guidelines related to the following in each of the main design and rehabilitation chapters for flexible and rigid pavements:

- Design input data needed, how the agency will collect the inputs, and establishing a database for inputs.
- Performance and reliability design criteria.
- Existing and new testing equipment required.
- Computer hardware and software requirements.
- Local calibration and validation of distress models:
    - Establishing a database of projects.
    - Input guidelines for local conditions, materials, and traffic.
    - Adjusting distress and IRI models to fit performance in State.
- Training requirements for staff doing pavement design.

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