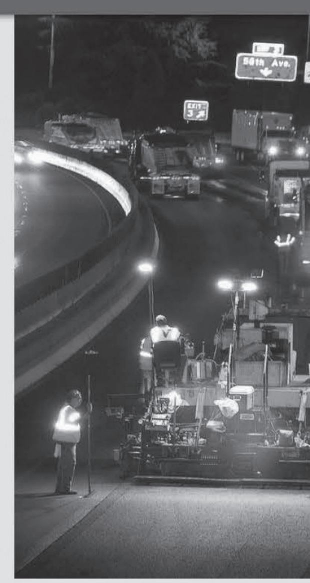



Mechanistic-Empirical Pavement Design Guide

~ *A Manual of Practice* ~



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PREFACE

This document describes a pavement design methodology that is based on engineering mechanics and has been validated with extensive road test performance data. This methodology is termed mechanistic-empirical (M-E) pavement design, and it represents a major change from the pavement design methods in practice today.

Interested agencies have already begun implementation activities in terms of staff training, collection of input data (materials library, traffic library, etc.), acquiring of test equipment, and setting up field sections for local calibration. This manual presents the information necessary for pavement design engineers to begin to use the MEPDG design and analysis method.

This manual refers to AASHTOWare Pavement Me Design™, M-E Pavement design software which is commercially available through AASHTOWare, AASHTO's software development program (see http://www.aashtoware.org/Pavement/Pages/default.aspx). AASHTOWare Pavement ME Design has been revised from the software described in the previous edition of this manual based upon evaluations performed by state Departments of Transportation and others in the community of practice.

The following table summarizes the key differences noted between the format and calibration factors used in the MEPDG version 1.1 software and the AASHTOWare Pavement ME Design software.

Table i-1. Summary of Key Differences in Software Format and Calibration Factors

Format and Calibration Factors	MEPDG Version 1.1	AASHTOWare Pavement ME Design
Output Format	Excel-based	PDF- and Excel-based
Climatic Data in Output Summary	Not included	Included
Axle Configuration Data in Output Summary	Not included	Included
Special Axle Load Configuration	Included	Not included
Reflection Cracking	Not included	Included
Coefficient of Thermal Expansion (CTE)	CTE for Basalt of 4.6	CTE for Basalt of 5.2
PCC Zero Stress Temperature	PCC Zero Stress Temperature (Range 60° to 120°F)	PCC Set Temperature (Range 70° to 212° F)
Heat Capacity of Asphalt Pavement	Default value of 0.23 BTU/lb-°F	Default value of 0.28 BTU/lb-°F
Thermal Conductivity of Asphalt Pavement	Default value of 0.67 BTU/(ft)(hr)(F)	Default value of 1.25 BTU/(ft)(hr)(F)
Surface Shortwave Absorptivity	Default value of 0.95	Default value of 0.85
Global Calibration Coefficient for Unbound Materials and Soils in Flexible Pavement Subgrade Rutting Model	k_{S1} granular of 1.63	k_{S1} granular of 2.03
Global Field Calibration Coefficients in the Fatigue Cracking Prediction Model in Flexible Pavement	k_{f2} of -3.9492	k_{f2} of 3.9492
	k_{f3} of -1.281	k_{f3} of 1.281
Global Field Calibration Coefficients in the Thermal Cracking Model for HMA	k_t (Level 1) of 5.0	k_t (Level 1) of 1.5
	k_t (Level 2) of 1.5	k_t (Level 2) of 0.5
	k_t (Level 3) of 3.0	k_t (Level 3) of 1.5
Global Field Calibration Coefficients in the Rut Depth Prediction Model	k_{2r} of 0.4791	k_2 of 1.5606
	k_{3r} of 1.5606	k_3 of 0.4791
Calibration Coefficients in the Rigid Pavement Faulting Prediction Model	C_1 of 1.29	C_1 of 1.0184
	C_2 of 1.1	C_2 of 0.91656
	C_3 of 0.001725	C_3 of 0.0021848
	C_4 of 0.0008	C_4 of 0.0008837
	C_7 of 1.2	C_7 of 1.83312
Calibration Coefficient in the Rigid Pavement Punchout Prediction Model	*A_{PO} of 195.789*	C_3 of 216.8421
	α_{PO} of 19.8947	C_4 of 33.15789
	β_{PO} of -0.526316	C_5 of -0.58947

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CHAPTER 1

Introduction

The overall objective of AASHTOWare Pavement ME Design is to provide the highway community with a state-of-the-practice tool for the design and analysis of new and rehabilitated pavement structures, based on mechanistic-empirical (M-E) principles. This means that the design and analysis procedure calculates pavement responses (stresses, strains, and deflections) and uses those responses to compute incremental damage over time. The procedure empirically relates the cumulative damage to observed pavement distresses. This M-E based procedure is shown in flowchart form in Figure 1-1.

AASHTOWare Pavement ME Design represents a major change in the way pavement design is performed. AASHTOWare Pavement ME Design predicts multiple performance indicators (refer to Figure 1-1) and it provides a direct tie between materials, structural design, construction, climate, traffic, and pavement management systems. Figures 1-2 and 1-3 are examples of the interrelationship between these activities for hot mix asphalt (HMA) and Portland cement concrete (PCC) materials.

1.1 PURPOSE OF MANUAL

This manual of practice presents information to guide pavement design engineers in making decisions and using AASHTOWare Pavement ME Design for new pavement and rehabilitation design. The manual does not provide guidance on developing regional or local calibration factors for predicting pavement distress and smoothness. A separate document, *Guide for the Local Calibration of the Mechanistic-Empirical Design*, provides guidance for determining the local calibration factors for both HMA and PCC pavement types (2).

1.2 OVERVIEW OF THE MEPDG DESIGN PROCEDURE

AASHTOWare Pavement ME Design is a production-ready design tool to support the day-to-day operations of public and private pavement engineers. When analyzing a pavement design project using AASHTOWare Pavement ME Design, whether new construction, an overlay, or restoration, an iterative process that follows three basic steps is utilized:

1. Create a trial design for the project.
2. Run AASHTOWare Pavement ME Design to predict the key distresses and smoothness for the trial design.
3. Review the predicted performance of the trial design against performance criteria and modify trial design as needed in order to produce a feasible design that satisfies the performance criteria.

Pavement responses (stresses, strains, and deflections) are combined with other pavement, traffic, climate, and materials parameters to predict the progression of key pavement distresses and smoothness loss over time. These outputs are the basis for checking the adequacy of a trial design.

AASHTOWare Pavement ME Design software also includes an automated process to iterate to an optimized thickness.

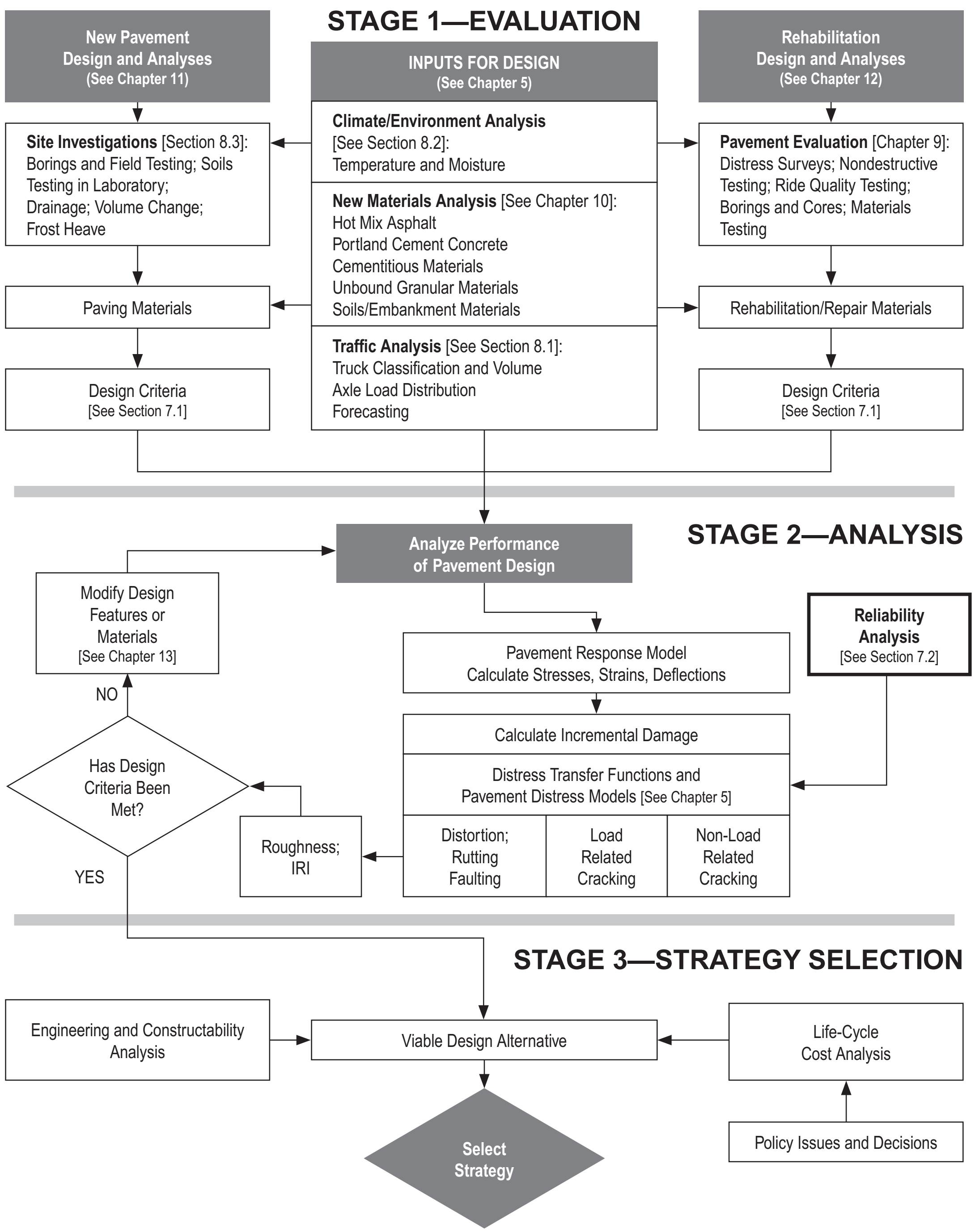

Figure 1-1. Conceptual Flow Chart of the Three-Stage Design/Analysis Process for AASHTOWare Pavement ME Design

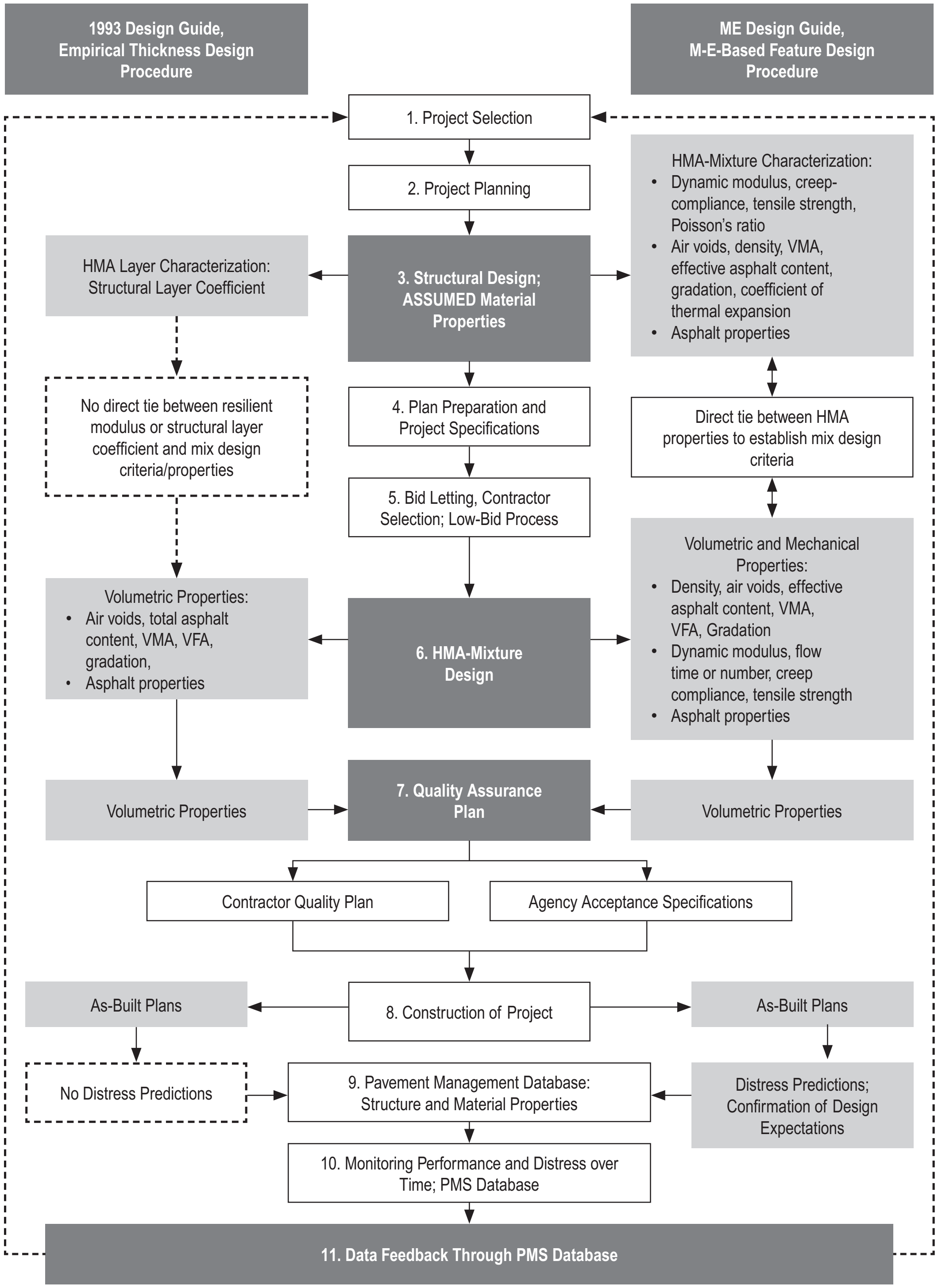

Figure 1-2. Typical Differences Between Empirical Design Procedures and an Integrated M-E Design System, in Terms of HMA-Mixture Characterization

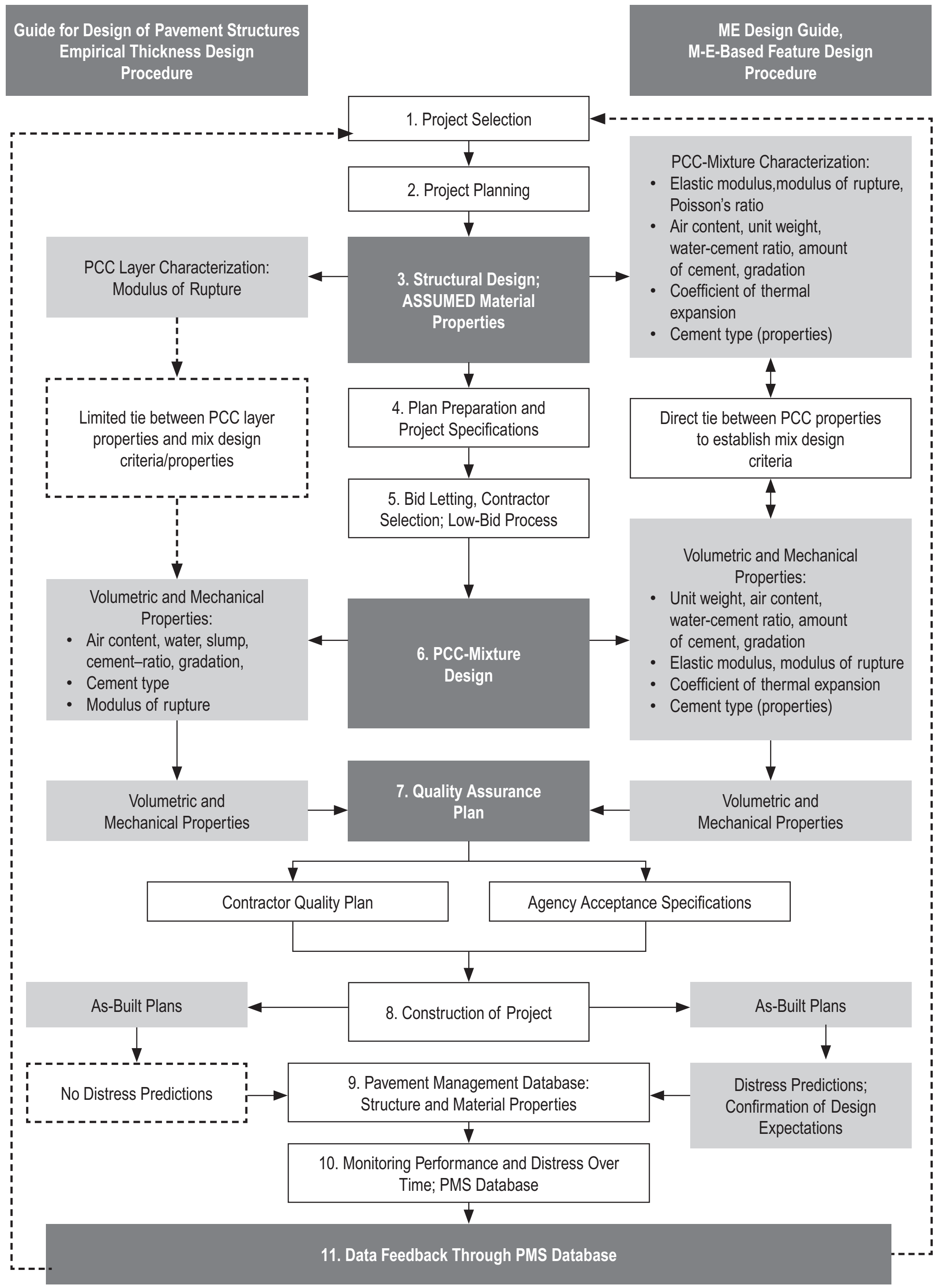

Figure 1-3. Typical Differences Between Empirical Design Procedures and an Integrated M-E Design System, in Terms of PCC-Mixture Characterization

The M-E approach makes it possible to optimize the design and to more fully ensure that specific distress types will be limited to values less than the failure criteria within the design life of the pavement structure. The basic steps included in the MEPDG design process are listed below and presented in flow chart form in Figures 1-4 and 1-5. The steps shown in Figures 1-4 and 1-5 are referenced to the appropriate sections within this manual of practice.

1. Select a trial design strategy. The pavement designer may use an agency-specific design procedure to determine the trial design cross section.

2. Select the appropriate performance indicator criteria (threshold value) and design reliability level for the project. Design or performance indicator criteria should include magnitudes of key pavement distresses and smoothness that trigger major rehabilitation or reconstruction. These criteria could be a part of an agency's policies for deciding when to rehabilitate or reconstruct. AASHTOWare Pavement ME Design allows the user to select the performance indicator criteria to be analyzed. The user can uncheck the box next to the criteria that needs no evaluation. (See Section 4.1 for definitions.)

3. Obtain all inputs for the pavement trial design under consideration. This step may be a time-consuming effort, but it is what separates the MEPDG from other design procedures. The MEPDG allows the designer to determine the inputs using a hierarchical structure in which the effort required to quantify a given input is selected based on the importance of the project, importance of the input, and the resources at the disposal of the user. The inputs required to run the software may be obtained using one of three levels of effort and need not be consistent for all of the inputs in a given design. The hierarchical input levels are defined in Sections 4 and 5. The inputs are grouped under six broad topics—general project information, design criteria, traffic, climate, structure layering, and material properties (including the design features).

4. Run AASHTOWare Pavement ME Design software and examine the inputs and outputs for engineering reasonableness. The software calculates changes in layer properties, damage, key distresses, and the International Roughness Index (IRI) over the design life. The sub-steps for step 4 include:

a) Examine the input summary to ensure the inputs are correct and what the designer intended. This step may be completed after each run, until the designer becomes more familiar with the program and its inputs.
b) Examine the outputs that comprise the intermediate process—specific parameters, such as climate values, monthly transverse load transfer efficiency values for rigid pavement analysis, monthly layer modulus values for flexible and rigid pavement analysis to determine their reasonableness, and calculated performance indicators (pavement distresses and IRI). This step may be completed after each run, until the designer becomes more familiar with the program. Review of important intermediate processes and steps is presented in Section 13.
c) Assess whether the trial design has met each of the performance indicator criteria at the design reliability level chosen for the project. As noted above, IRI is an output parameter predicted over time and a measure of surface smoothness. IRI is calculated from other distress predictions (refer to Figure 1-1), site factors, and initial IRI.

d) If any of the criteria have not been met, determine how this deficiency can be remedied by altering the materials used, the layering of materials, layer thickness, or other design features.

5. Revise the trial design, as needed. If the trial design has input errors, material output anomalies, or has exceeded the failure criteria at the given level of reliability, revise the inputs/trial design and rerun the program. An automated process to iterate to an optimized thickness is done by AASHTOWare Pavement ME Design to produce a feasible design.

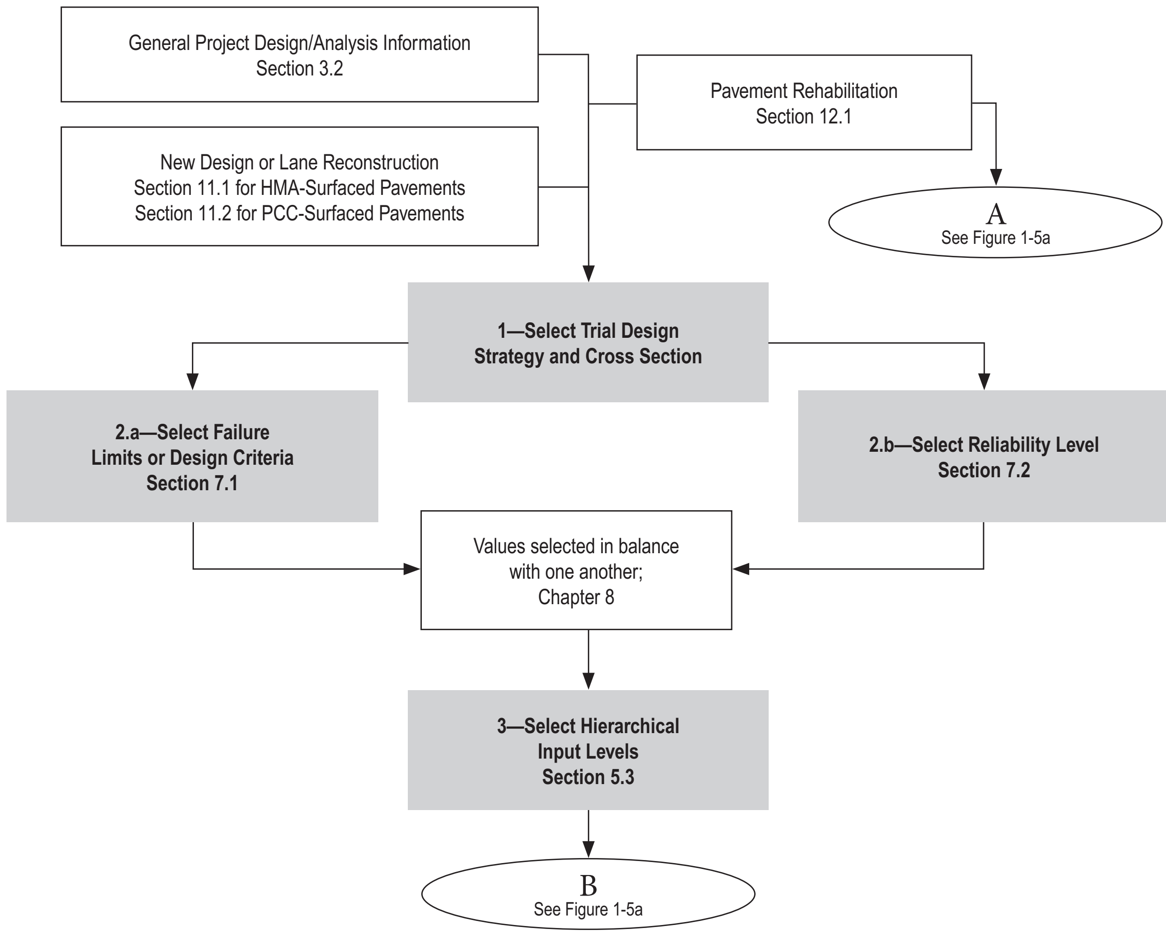

Figure 1-4. Flow Chart of the Steps That Are More Policy Decision Related and Are Needed to Complete an Analysis of a Trial Design Strategy

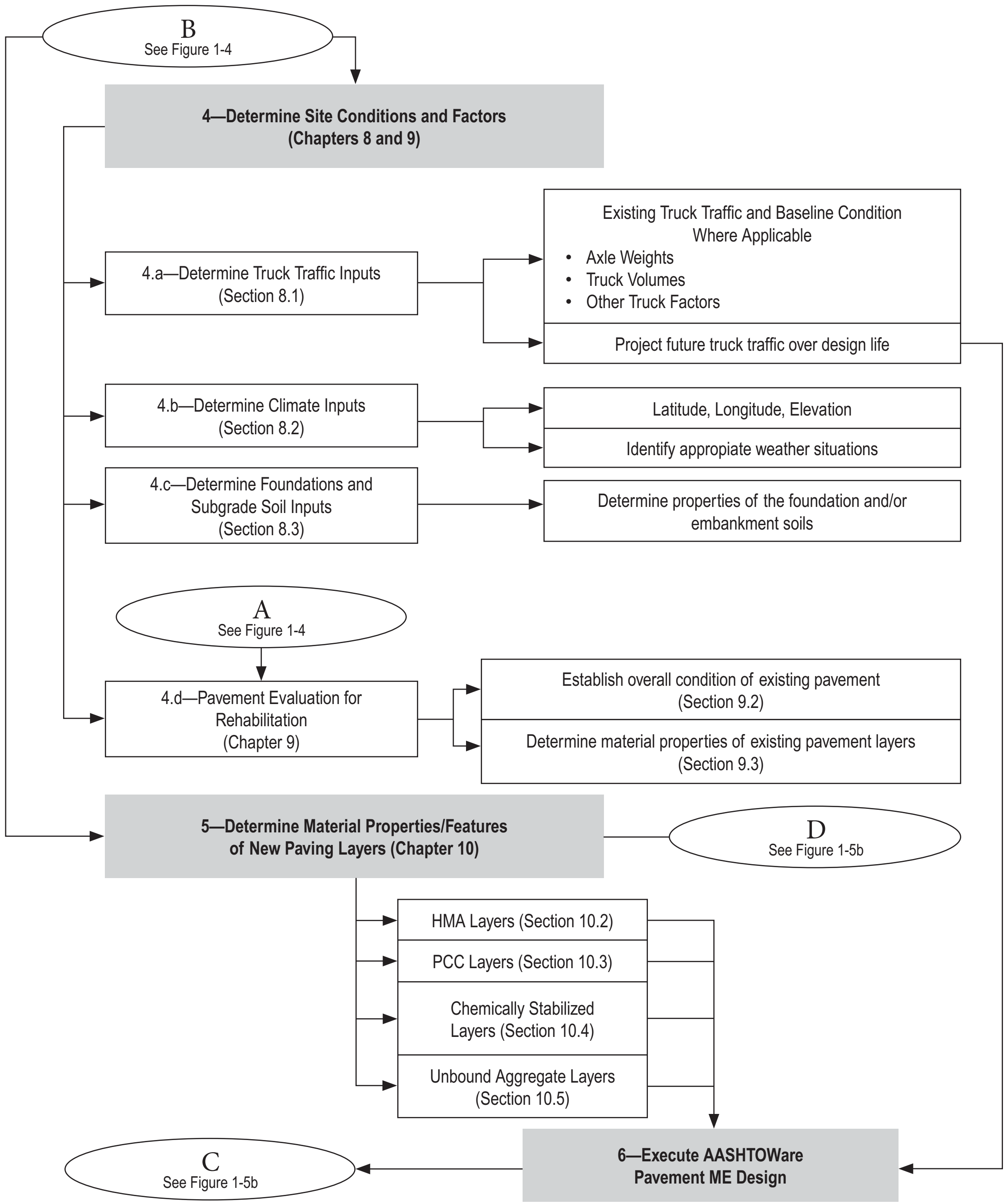

Figure 1-5a. Flow Chart of the Steps Needed to Complete an Analysis of a Trial Design Strategy

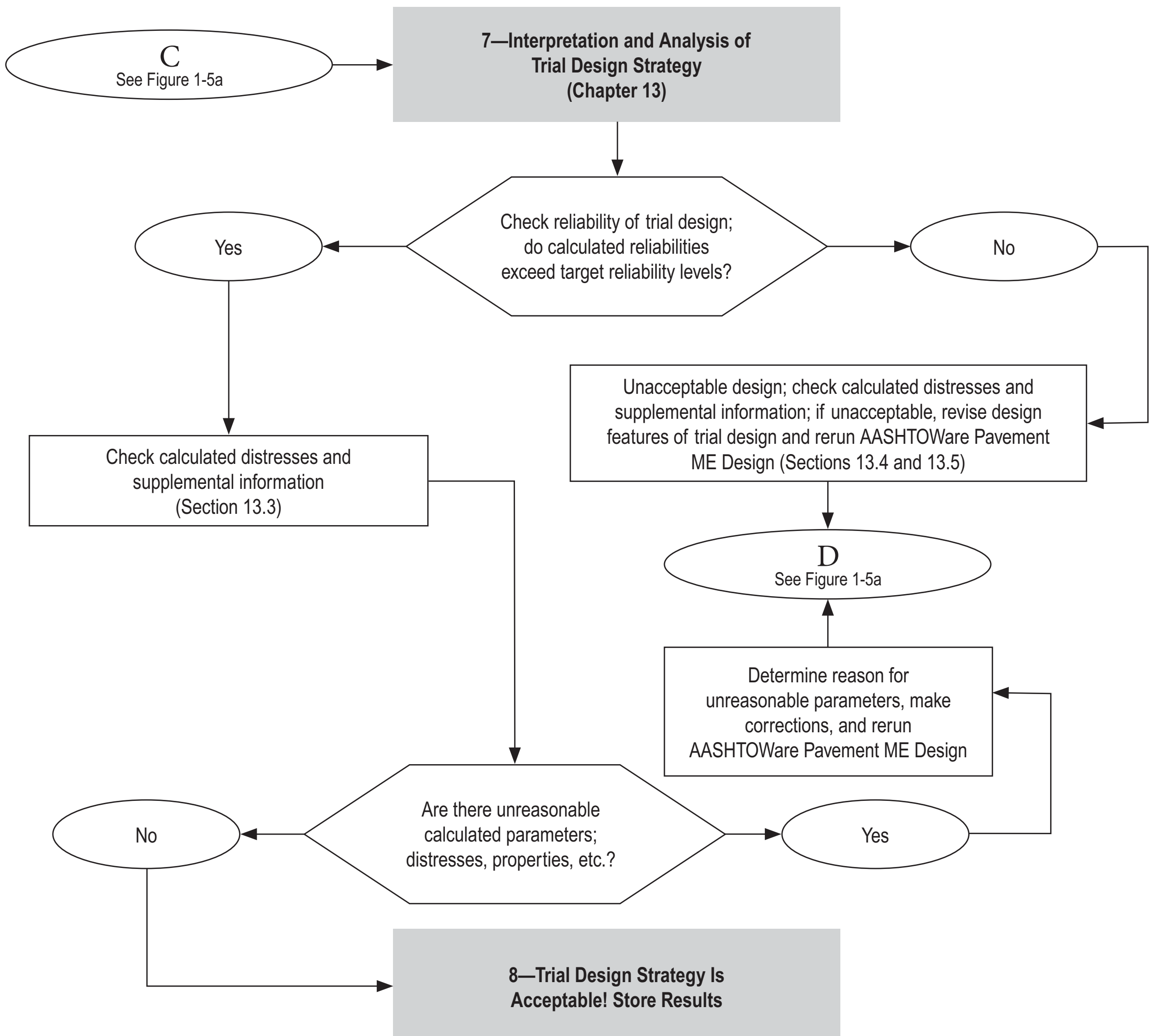

Figure 1-5b. Flow Chart of the Steps Needed to Complete an Analysis of a Trial Design Strategy

CHAPTER 2

Referenced Documents and Standards

This section includes a listing of the laboratory and field test protocols for different paving materials, recommended practices, material specifications, and the referenced documents needed for using AASHTOWare Pavement ME Design.

2.1 TEST PROTOCOLS AND STANDARDS

From the test protocols listed in this section, the designer needs to execute only those for the hierarchical input levels selected. Refer to Chapter 4 for a definition of hierarchical input levels. The listing of test procedures is organized into two sections: Laboratory Materials Characterization and In-Place Materials/Pavement Layer Characterization.

2.1.1 Laboratory Materials Characterization

Unbound Materials and Soils

AASHTO T 88	Particle Size Analysis of Soils
AASHTO T 89	Determining the Liquid Limits of Soils
AASHTO T 90	Determining the Plastic Limit and Plasticity Index of Soils
AASHTO T 99	Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop
AASHTO T 100	Specific Gravity of Soils
AASHTO T 180	Moisture-Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and an 457-mm (18-in.) Drop
AASHTO T 190	Resistance R-Value and Expansion Pressure of Compacted Soils
AASHTO T 193	The California Bearing Ratio
AASHTO T 206	Penetration Test and Split-Barrel Sampling of Soils
AASHTO T 207	Thin-Walled Tube Sampling of Soils
AASHTO T 215	Permeability of Granular Soils (Constant Head)
AASHTO T 258	Determining Expansive Soils
AASHTO T 265	Laboratory Determination of Moisture Content of Soils
AASHTO T 307	Determining the Resilient Modulus of Soils and Aggregate Materials
ASTM D2487	Classification of Soils for Engineering Purposes

Treated and Stabilized Materials/Soils

AASHTO T 220	Determination of the Strength of Soil-Lime Mixtures
ASTM C593	Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization
ASTM D1633	Compressive Strength of Molded Soil-Cement Cylinders
ASTM D1635	Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading

Asphalt Binder

AASHTO T 49	Penetration of Bituminous Materials
AASHTO T 53	Softening Point of Bitumen (Ring-and-Ball Apparatus)
AASHTO T 201	Kinematic Viscosity of Asphalts (Bitumens)
AASHTO T 202	Viscosity of Asphalts by Vacuum Capillary Viscometer
AASHTO T 228	Specific Gravity of Semi-Solid Bituminous Materials
AASHTO T 315	Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)
AASHTO T 316	Viscosity Determination of Asphalt Binder Using Rotational Viscometer
AASHTO T 319	Quantitative Extraction and Recovery of Asphalt Binder from Asphalt Mixtures

Hot Mix Asphalt and Asphalt Treated/Stabilized Mixtures

AASHTO T 27	Sieve Analysis of Fine and Coarse Aggregates
AASHTO T 84	Specific Gravity and Absorption of Fine Aggregate
AASHTO T 85	Specific Gravity and Absorption of Coarse Aggregate
AASHTO T 164	Quantitative Extraction of Asphalt Binder from Hot Mix Asphalt (HMA)
AASHTO T 166	Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens
AASHTO T 209	Theoretical Maximum Specific Gravity (G_{mm}) and Density of Hot Mix Asphalt Paving Mixtures
AASHTO T 269	Percent Air Voids in Compacted Dense and Open Asphalt Mixtures
AASHTO T 308	Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method
AASHTO T 312	Preparing and Determining the Density of Asphalt Mixture Specimens by Means of the Superpave Gyratory Compactor
AASHTO T 322	Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device
AASHTO T 342	Determining Dynamic Modulus of Hot Mix Asphalt (HMA)

Portland Cement Concrete and Cement Treated/Stabilized Base Mixtures

AASHTO T 22	Compressive Strength of Cylindrical Concrete Specimens
AASHTO T 97	Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
AASHTO T 121M /T 121	Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
AASHTO T 152	Air Content of Freshly Mixed Concrete by the Pressure Method

AASHTO T 196	Air Content of Freshly Mixed Concrete by the Volumetric Method
AASHTO T 198	Splitting Tensile Strength of Cylindrical Concrete Specimens
AASHTO T 336	Coefficient of Thermal Expansion of Hydraulic Cement Concrete
ASTM C469	Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression

Thermal Properties of Paving Materials

ASTM D2766	Specific Heat of Liquids and Solids
ASTM E1952	Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry

2.1.2 In-Place Materials/Pavement Layer Characterization

AASHTO T 256	Pavement Deflection Measurements
ASTM D5858	Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory
ASTM D6951	Standard Test for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications

2.2 MATERIAL SPECIFICATIONS

AASHTO M 320	Performance-Graded Asphalt Binder
AASHTO M 323	Superpave Volumetric Mix Design

2.3 STANDARD PRACTICES AND TERMINOLOGY

AASHTO M 145	Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes
AASHTO R 13	Conducting Geotechnical Subsurface Investigations
AASHTO R 37	Application of Ground Penetrating Radar (GPR) to Highways
AASHTO R 43	Quantifying Roughness of Pavements
AASHTO R 50	Geosynthetic Reinforcement of the Aggregate Base Course of Flexible Pavement Structures
AASHTO R 59	Recovery of Asphalt from Solution by Abson Method
ASTM E1778	Standard Terminology Relating to Pavement Distress
AASHTO LCG-1	*Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design*

2.4 REFERENCED DOCUMENTS

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CHAPTER 3

Significance and Use of the MEPDG

The MEPDG represents a major change in the way pavement design is performed. Mechanistic refers to the application of the principles of engineering mechanics, which leads to a rational design process that has three basic elements: (1) the theory used to predict critical pavement responses (strains, stresses, deflections, etc.), as a function of traffic and climatic loading (the mechanistic part); (2) materials characterization procedures that support and are consistent with the selected theory; and (3) defined relationships between the critical pavement response parameter and field-observed distress (the empirical part).

The MEPDG provides a uniform and comprehensive set of procedures for the analysis and design of new and rehabilitated flexible and rigid pavements. AASHTOWare Pavement ME Design employs common design parameters for traffic, materials, subgrade, climate, and reliability for all pavement types, and is used to develop alternative designs using a variety of materials and construction procedures. Recommendations are provided for the structure (layer materials and thickness) of new (including lane reconstruction) and rehabilitated pavements, including procedures to select pavement layer thickness, rehabilitation treatments, subsurface drainage, foundation improvement strategies, and other design features.

The output from the AASHTOWare Pavement ME Design is predicted distresses and IRI (smoothness) at the selected reliability level. The thickness optimization tool allows the AASHTOWare Pavement ME Design to be used not only for analysis, but also for design by evaluating a combination of layer types, layer thickness, and design features for a given set of site conditions and failure criteria at a specified level of reliability.

3.1 PERFORMANCE INDICATORS PREDICTED BY AASHTOWARE PAVEMENT ME DESIGN

The MEPDG includes transfer functions and regression equations that are used to predict various performance indicators considered important in many pavement management programs. The following lists the specific performance indicators calculated by AASHTOWare Pavement ME Design, which were calibrated using data extracted from the Long-Term Pavement Performance (LTPP) database. The specific prediction models for all pavement types are presented in Section 5.

- HMA-Surfaced Pavements and HMA Overlays
  - Total Rut Depth and HMA, unbound aggregate base, and subgrade rutting
  - Non-Load-Related Transverse Cracking
  - Load-Related Alligator Cracking, Bottom Initiated Cracks
  - Load-Related Longitudinal Cracking, Surface Initiated Cracks
  - Reflection Cracking in HMA Overlays of Cracks and Joints in Existing Flexible, Semi-Rigid, Composite, and Rigid Pavements
  - Smoothness (IRI)

- Portland Cement Concrete-Surfaced Pavements and PCC Overlays
  - Jointed Plain Concrete Pavement (JPCP)—Mean Joint Faulting
  - JPCP—Joint Load Transfer Efficiency (LTE)
  - JPCP—Load-Related Transverse Slab Cracking (includes both bottom and surface initiated cracks)
  - JPCP—Joint Spalling (embedded into the IRI prediction model)
  - Continuously Reinforced Concrete Pavement (CRCP)—Crack Spacing and Crack Width
  - CRCP—LTE
  - CRCP—Punchouts
  - JPCP and CRCP—Smoothness (IRI)

3.2 MEPDG GENERAL DESIGN APPROACH

The design approach provided in AASHTOWare Pavement ME Design consists of three major stages and multiple steps, as shown in Figures 1-1, 1-4, and 1-5. Stage 1 consists of the determination of input values for the trial design. During this stage, strategies are identified for consideration in the design stage.

A key step of this process is the foundation analysis. For new pavements, the foundation analysis or site investigation consists of resilient modulus determination, and an evaluation of the shrink-swell potential of high-plasticity soils, frost heave-thaw weakening potential of frost susceptible soils, and drainage concerns (refer to Section 8.3).

The foundation analysis or pavement evaluation for rehabilitation design projects includes recommendations for a pavement structure condition evaluation to identify the types of distresses exhibited and the underlying causes for those distresses (refer to Chapter 9). The procedure focuses on quantifying the strength of the existing pavement layers and foundation using nondestructive deflection basin tests and backcalculation procedures. Deflection basin tests are used to estimate the damaged modulus condition of the existing structural layers. However, the procedure also includes recommendations for and use of pavement condition survey, drainage survey, and ground penetrating radar (GPR) data to quantify the in-place condition (damaged modulus values) of the pavement layers.

The materials, traffic, and climate characterization procedures are also included in Stage 1 of the design approach. Materials characterization is an important part of this design procedure, and modulus is the key layer property needed for all layers in the pavement structure. . Unbound paving layers and foundation are characterized by resilient modulus whereas HMA layers and PCC layers are characterized by dynamic modulus and elastic modulus respectively. Depending on the availability of modulus data, the user has the option through different input levels to either enter resilient modulus values obtained from testing or use other material property inputs that are converted to resilient modulus values within the software. A more detailed listing of the required material properties for all pavement types is presented in Chapters 9 and 10.

Traffic characterization consists of estimating the axle-load distributions applied to the pavement structure (refer to Section 8.1). The MEPDG does not use equivalent single-axle loads (ESAL) and does not require the development of load equivalency factors.

Another major improvement to pavement design that is embedded in the AASHTOWare Pavement ME Design is the consideration of climatic effects on pavement materials, responses, and distress in an integrated manner (refer to Section 8.2). These effects are estimated using the Enhanced Integrated Climatic Model (EICM), which is a tool used to model temperature and moisture within each pavement layer and the foundation. This climatic model considers hourly ambient climatic data in the form of temperatures, precipitation, wind speed, cloud cover, and relative humidity from weather stations across the United States for estimating pavement layer temperatures and moisture conditions. The pavement layer temperature and moisture predictions from the EICM are calculated hourly and used in a variety of applications to estimate the material properties for the foundation and pavement layers throughout the design life.

Stage 2 of the design process (refer to Figure 1-1) is the structural analysis and predictions of selected performance indicators and smoothness. The analysis approach is an iterative one that begins with the selection of an initial trial design. Initial trial designs are created by the designer, obtained from an existing design procedure, or from a general catalog. The trial section is analyzed incrementally over time using the pavement response and distress models. The outputs of the analysis include material properties, accumulated damage (defined in Section 4), the amount of distress, and smoothness over time, among other significant process-specific predictions. If the trial design does not meet or exceed the design criteria at the specified level of reliability, modifications are made and the analysis is re-run until a satisfactory result is obtained.

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. Stage 3 is not covered in this manual.

3.3 NEW FLEXIBLE PAVEMENT AND HMA OVERLAY DESIGN STRATEGIES APPLICABLE FOR USE WITH AASHTOWARE PAVEMENT ME DESIGN

AASHTOWare Pavement ME Design can be used to analyze the expected performance of new and reconstructed HMA-surfaced pavements, as well as HMA overlays. The HMA-surfaced pavement types include the following, which are illustrated in Figures 3-1 and 3-2.

- **Conventional Flexible Pavements**—Flexible pavements that consist of relatively thin HMA surfaces (less than 6 in. thick) and unbound aggregate base layers (crushed stone or gravel, and soil-aggregate mixtures). Many of the pavements used in the global calibration process had multiple aggregate base layers. Conventional flexible pavements may also have a stabilized or treated subgrade layer.
- **Deep Strength Flexible Pavements**—Flexible pavements that consist of a relatively thick HMA surface and a dense-graded HMA or asphalt stabilized base mixture placed over an aggregate base layer. Deep strength flexible pavements may also have a stabilized or treated subgrade layer. Many of the flexible pavements used in the global calibration process had asphalt stabilized base layers and would be defined deep strength flexible pavements.
- **Full-Depth HMA Pavements**—HMA layers placed on a stabilized subgrade layer or placed directly on the prepared embankment or foundation soil. Full-depth flexible pavements were also included in the global calibration process, but there were fewer test sections than for conventional and deep strength flexible pavements.
- **Semi-Rigid Pavements**—HMA placed over cementitious stabilized materials. Cementitious materials may include lime, lime-fly ash, and Portland cement stabilizers. This type of pavement is also referred to as composite pavements in the MEPDG. Semi-rigid pavements were not included in the global calibration process, and are not recommended for analysis using AASHTOWare Pavement ME Design until this type of pavement has been calibrated.

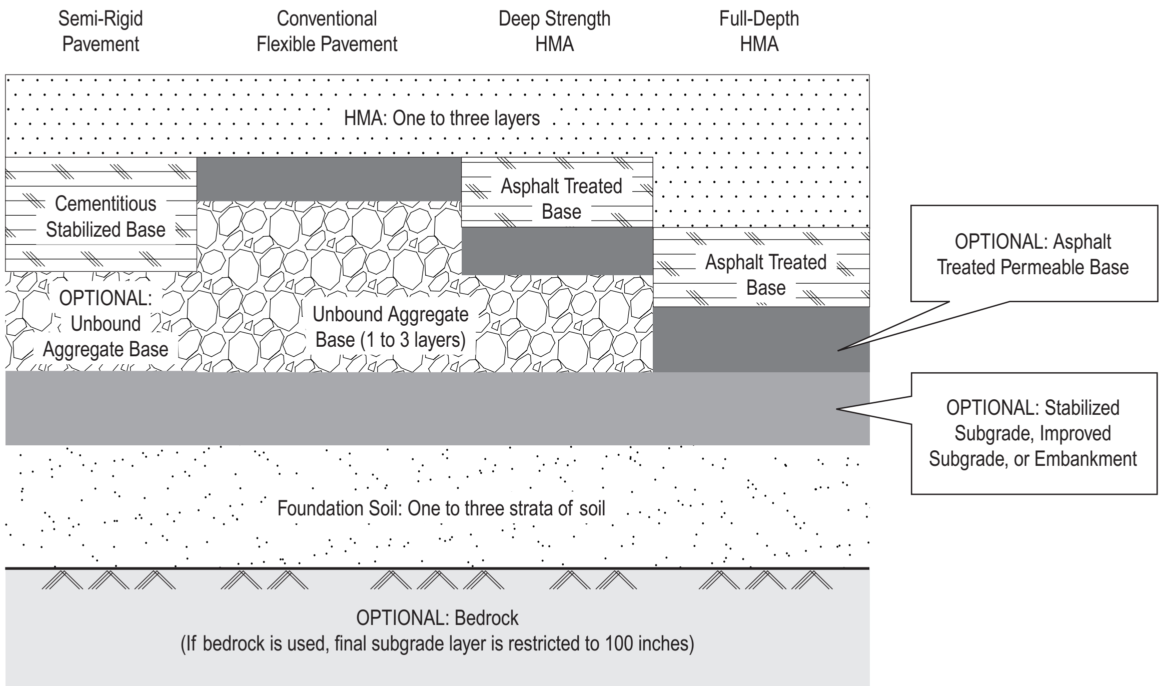
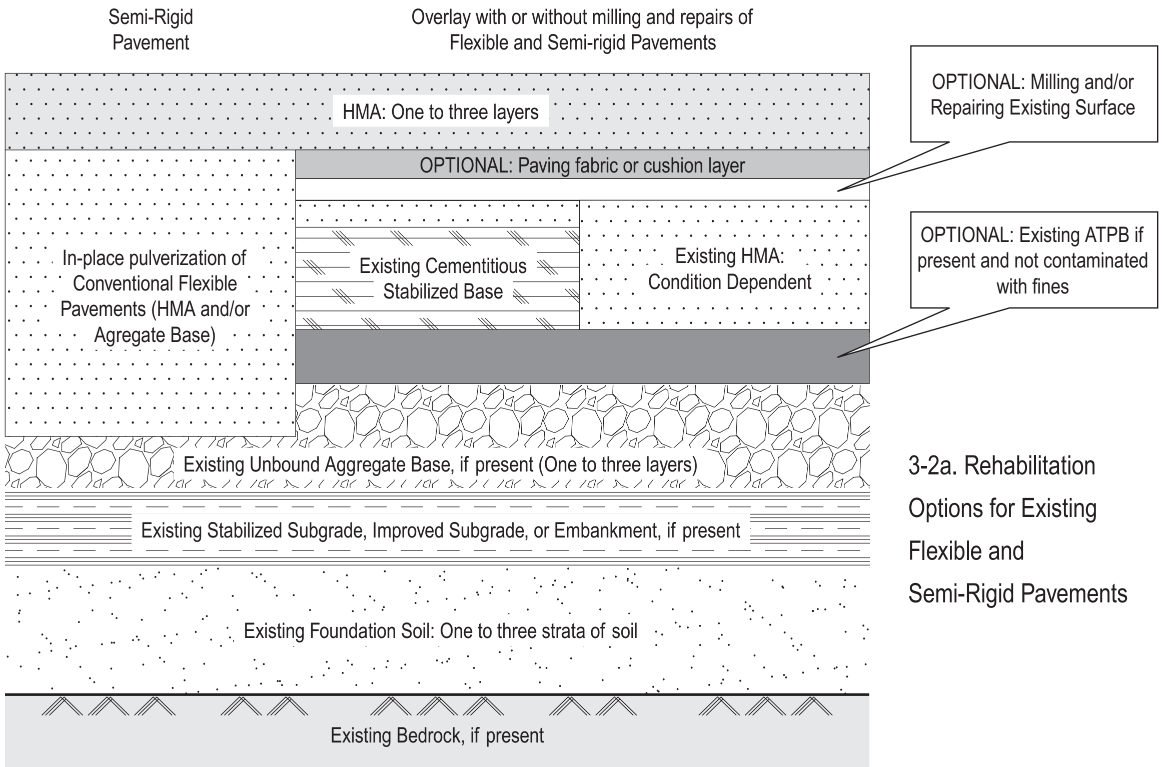
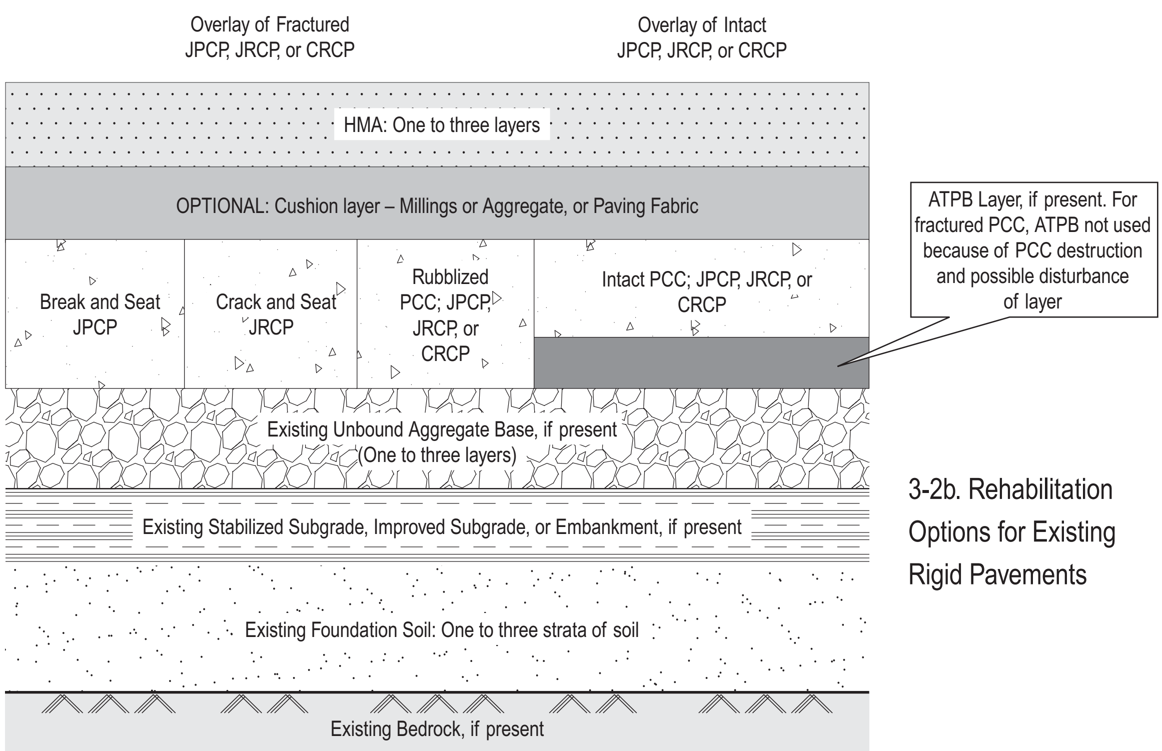


Figure 3-1. New (Including Lane Reconstruction) Flexible Pavement Design Strategies That Can Be Simulated with AASHTOWare Pavement ME Design (Refer to Section 11.1); Layer Thickness Not to Scale



3-2a. Rehabilitation Options for Existing Flexible and Semi-Rigid Pavements



3-2b. Rehabilitation Options for Existing Rigid Pavements

Figure 3-2. HMA Overlay Design Strategies of Flexible, Semi-Rigid, and Rigid Pavements That Can Be Simulated with the AASHTOWare Pavement ME Design (Refer to Section 12.2); Layer Thickness Not to Scale

- **Full-Depth Reclamation (In-Place Pulverization of Conventional Flexible Pavements)**—Cold in-place recycling of the HMA and existing aggregate base layers, and hot in-place recycling of HMA. Cold in-place recycling as a rehabilitation strategy is considered reconstruction under the MEPDG design/analysis process and would be defined as a new flexible pavement. Hot in-place recycling as a rehabilitation strategy is considered mill and fill with an HMA overlay of the existing flexible pavement. The thickness of the hot in-place recycled material is considered part of the HMA overlay, as well as the thickness of the milled material. Full-depth reclamation, however, was not included in the global calibration of AASHTOWare Pavement ME Design.
- **HMA Overlays** of all types of flexible and intact rigid pavements, with or without pavement repairs and surface milling. Pavement repairs and milling of the existing surface layer is considered by AASHTOWare Pavement ME Design. The expected milling depth is an input value, and pavement repairs are considered by entering the condition of the pavement prior to overlay placement. AASHTOWare Pavement ME Design is also be used to design HMA overlays of fractured PCC slabs (break and seat [applicable to JPCP]; and rubblization [applicable to all PCC pavements]). HMA overlays of fractured PCC slabs, however, were not included in the global calibration process.

3.4 NEW RIGID PAVEMENT, PCC OVERLAY, AND RESTORATION OF RIGID PAVEMENT DESIGN STRATEGIES APPLICABLE FOR USE WITH AASHTOWARE PAVEMENT ME DESIGN

AASHTOWare Pavement ME Design can be used to analyze the expected performance of new and reconstructed PCC-surfaced pavements, as well as PCC overlays and concrete pavement restoration (CPR). The PCC-surfaced pavement types include the following, which are illustrated in Figures 3-3 and 3-4.

- **JPCP**—The minimum thickness of JPCP modeled in the software is 6 in. In this type of PCC pavement, the minimum joint spacing is 10 ft. The transverse joints are spaced relatively close (e.g., ranging from 10 to 20 ft) in order to minimize transverse cracking from temperature gradient and drying gradient shrinkage stresses. This pavement contains no distributed steel to control random cracking and may or may not contain transverse joint load transfer devices (e.g., dowels). JPCP may have tied or untied longitudinal joints. However, most of the test sections included in the global calibration process had tied longitudinal joints. The effect of tied or untied longitudinal joints would need to be defined and considered through the local calibration process. The base (layer directly beneath the PCC slab) and subbase layers may consist of a wide variety of unbound aggregates, asphalt stabilized granular, cement stabilized, lean concrete, crushed concrete, lime stabilized, recycled asphalt pavement (RAP), and other materials. The base layer may be dense graded or permeable drainage layers.
- **CRCP**—The minimum thickness of CRCP modeled in the software is 7 in. In this type of PCC pavement, longitudinal reinforcement at or above mid-depth designed to hold shrinkage cracks tightly closed. Transverse joints exist only for construction purposes and to separate on-grade

structures. Transverse reinforcement may or may not exist. Longitudinal joints exist similar to other types of concrete pavements. The base (layer directly beneath the PCC slab) and subbase layers may consist of a wide variety of unbound aggregates, asphalt stabilized granular, cement stabilized, lean concrete, crushed concrete, lime stabilized, RAP, and other materials. The base layer may be dense graded or permeable drainage layers.

- **JPCP Overlays**—JPCP placed over existing rigid pavements, composite pavements, and flexible pavements. Composite pavements consist of HMA placed over PCC, lean concrete, or a cement stabilized base (including roller compacted concrete). Composite pavements are the same as semi-rigid pavements (defined in Section 3.3), as used in AASHTOWare Pavement ME Design.
- **CRCP Overlays**—CRCP placed over existing rigid pavements, composite pavements, and flexible pavements.
- **Restoration of JPCP**—Work performed on an existing JPCP that includes diamond grinding of the surface. Other work may include dowel bar retrofit, joint reseal, edge drains, slab replacement, full-depth repair, spall repair, and shoulder replacement.

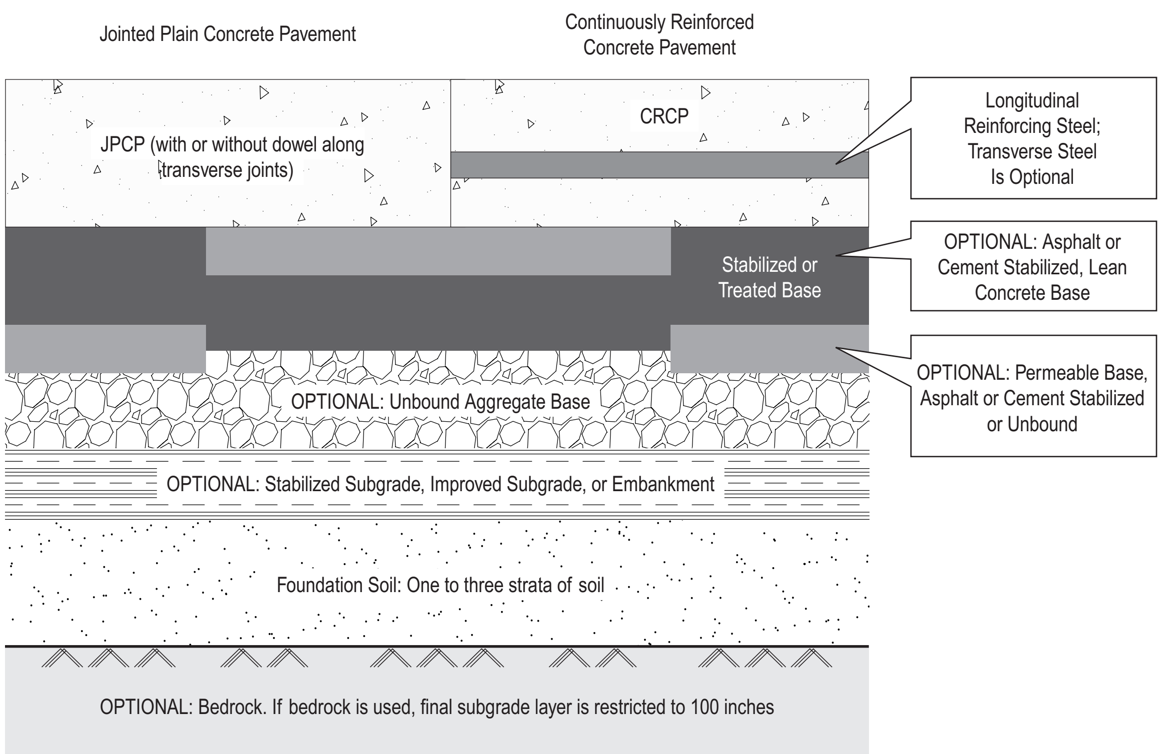


Figure 3-3. New (Including Lane Reconstruction) Rigid Pavement Design Strategies That Can Be Simulated with the AASHTOWare Pavement ME Design (Refer to Section 11.2); Layer Thickness Not Be Scale

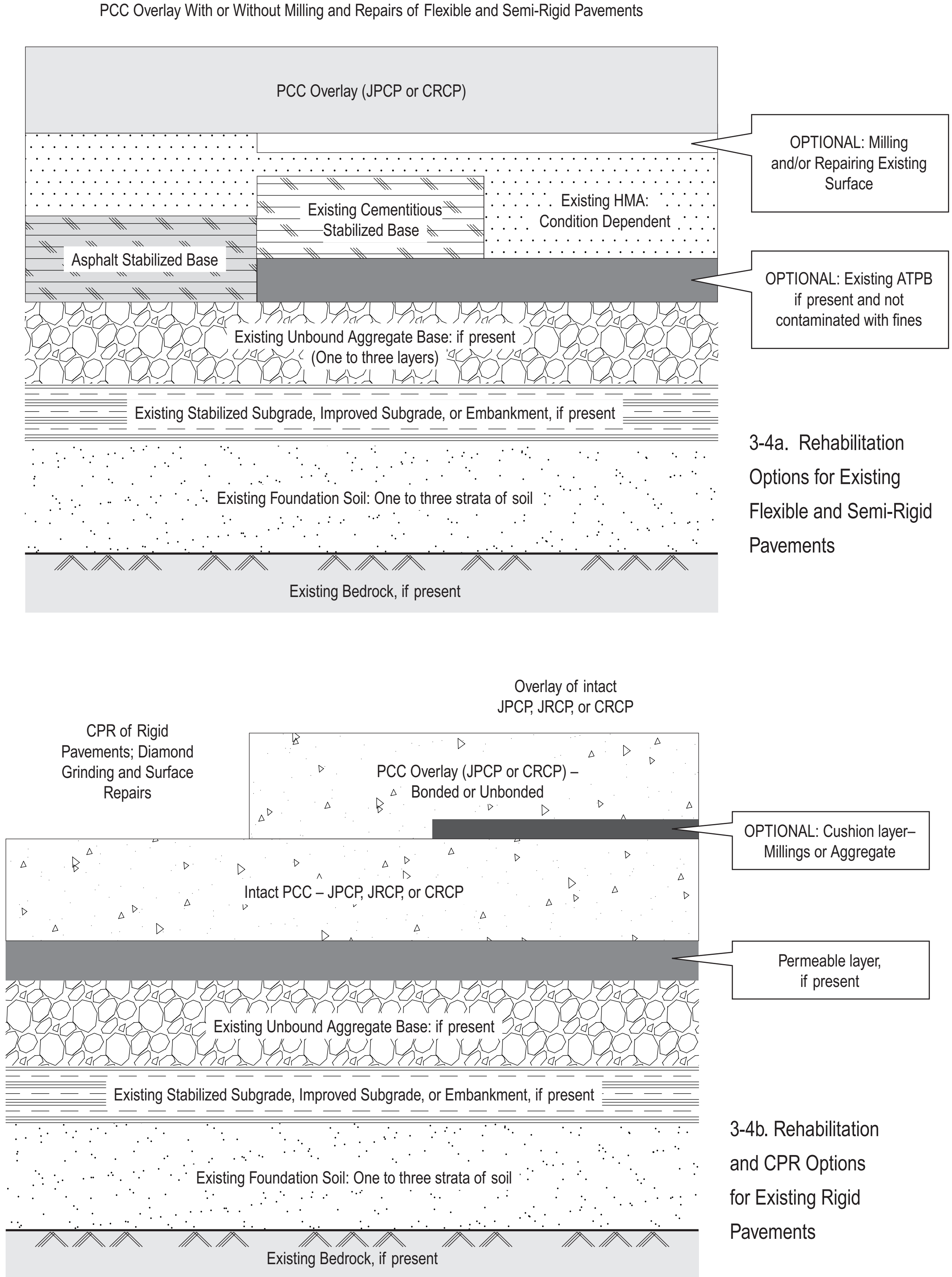


Figure 3-4. PCC Overlay Design Strategies of Flexible, Semi-Rigid, and Rigid Pavements That Can Be Simulated with AASHTOWare Pavement ME Design (Refer to Section 12.3); Layer Thickness Not to Scale

3.5 DESIGN FEATURES AND FACTORS NOT INCLUDED WITHIN THE MEPDG PROCESS

The intent of this section is to identify the features and distress prediction models that have not been calibrated for lack of adequate data, theoretical basis for modeling, and other possible reasons. The user should take this into account when using such prediction models. If such models are considered important for a given agency, effort could be expended during local calibration to ensure that they are valid for the conditions under which they are intended to be used. A standard practice is available that agencies may use in completing a local calibration effort (*2*). Some items not explicitly considered in the MEPDG are listed below.

- **Friction or Skid Resistance and Noise**—AASHTOWare Pavement ME Design does not predict the loss of surface characteristics related to skid resistance and noise attenuation. The designer needs to consider historical data and experience in evaluating the surface layer's capability to retain minimum skid resistance and noise attenuation values through the materials' specifications external to AASHTOWare Pavement ME Design.
- **Single and Super-Single Tires**—AASHTOWare Pavement ME Design assumes that all axles within the truck traffic mix have dual tires; however, the software does have the capability to simulate a super-single tire loading.
- **Durability and Mixture Disintegration**—AASHTOWare Pavement ME Design does not have the capability to predict mixture durability and surface disintegration distresses, such as raveling and stripping of HMA mixtures and spalling and alkali silica reactivity (ASR) or D-cracking of PCC layers. Mixture durability issues may be addressed during the mixture design process or by the material specifications for a project, external to AASHTOWare Pavement ME Design. The spalling of PCC joints, however, is modeled empirically as a function of water/cement ratio, air content, strength, and other parameters.
- **Volume Change in Problem Soils**—AASHTOWare Pavement ME Design does not have the capability to predict the volume change potential from frost susceptible soils (frost heave potential) or expansive-highly plastic clay soils (shrink-swell potential; AASHTO T 258). When problem soils are encountered along the project, appropriate subgrade improvement and strengthening techniques could be used to minimize the detrimental impact of these problem soils on pavement performance.
- **Asphalt Treated Permeable Base (ATPB)**—Flexible pavement sections with an ATPB were omitted from the global calibration process of flexible pavements, but were included in many rigid pavement sections used for global calibration. These ATPB layers below the PCC surface were treated as asphalt-treated materials with high air void contents.

If these layers are included in the trial design just below the lowest HMA dense-graded layer of an HMA-surfaced pavement, AASHTOWare Pavement ME Design calculates the tensile strain at the bottom of the ATPB for use in predicting alligator cracking. The high air void content of this drainage layer significantly reduces the fatigue life of the flexible pavement. This reduction was found to be inappropriate for some of the LTPP SPS-1 test sections (*2*).

As an option for its use, the ATPB layer may be treated as a high-quality aggregate base layer when analyzing the trial design. The resilient modulus considered appropriate for this simulation is 65 ksi, but could be verified through expanded local calibration efforts that include flexible pavements with an ATPB layer.

- **Geogrids and Other Reinforcing Materials**—These materials cannot be simulated in AASHTOWare Pavement ME Design at this time. In addition, none of the test sections included in the global calibration process had geogrids or other reinforcing materials included in the pavement structure.
- **Semi-Rigid Pavements**—Semi-rigid pavements consist of HMA mixtures placed over cement treated base (CTB), lean concrete base (LCB), or cement-aggregate mixtures (CAM), with or without aggregate subbase layers. AASHTOWare Pavement ME Design can analyze this pavement type, but the fatigue cracking incremental damage and transfer function for semi-rigid pavements was not calibrated. Thus, the global calibration factors are set to 1.0 in the program and there is no standard error reported for this pavement design strategy. A designer analyzing a semi-rigid pavement with this software should be aware that local calibration is necessary to generate reasonable output accuracy.
- **Pavement Preservation Programs**—Pavement preservation programs and strategies are policy decisions which are not considered directly in the distress predictions. Pavement preservation treatments applied to the surface of HMA layers early in their life may have an impact on the performance of flexible pavements and HMA overlays. The pavement designer needs to consider the impact of these programs in establishing the local calibration coefficients or develop agency specific values—primarily for load and non-load related cracking. This pavement preservation issue is discussed in more detail in the *Guide for the Local Calibration of the Mechanistic-Empiracl Pavement Design* (2), for determining the regional or agency specific calibration factors. Preservation is considered in JPCP design only in the ability to design a restoration project.
- **Staged Construction**—AASHTOWare Pavement ME Design does not have the capability to evaluate staged construction events that are offset by extended periods of time. When staged construction is planned for a project, the designer may enter a traffic open month and year that the final pavement layer has been placed. Section 6.2 provides more discussion on staged construction events.
- **Ultra-Thin PCC overlays**—Ultra-thin PCC overlays cannot be designed with AASHTOWare Pavement ME Design. The minimum thickness of JPCP overlay is 6 in. and the minimum thickness of CRCP is 7 in. Joint spacing is also limited to 10 ft and above.
- **JRCP**—These pavements were not directly considered in AASHTOWare Pavement ME Design development and cannot be designed using this procedure.
- **Early-Age PCC Opening to Traffic**—Twenty-eight days is the minimum time for opening of PCC pavements, as provided in AASHTOWare Pavement ME Design.
- **Interface Friction of HMA Overlay and Existing PCC Pavement**—AASHTOWare Pavement ME Design excluded the capability to vary the interface friction between the HMA overlay and ex-

isting PCC pavement. Interface friction, however, is considered between all HMA layers of flexible pavements and HMA overlays of flexible pavements, and between the JPCP and base layer. Section 9.2.7 provides more discussion on the use of interface friction between bound layers. Full bond was assumed in all cases, with the exception of CTB bases, for the global calibration effort (*18*).

CHAPTER 4

Terminology and Definition of Terms

This section provides the definitions of selected terms as used within AASHTOWare Pavement ME Design.

4.1 GENERAL TERMS

- **Calibration Factors**—Two calibration factors are used in AASHTOWare Pavement ME Design—global and local calibration factors. These calibration factors are adjustments applied to the coefficients and/or exponents of the transfer function to eliminate bias between the predicted and measured pavement distress. The combination of calibration factors (coefficients and exponents for the different distress prediction equations) is also be used to minimize the standard error of the prediction equation. The standard error of the estimate (s_e) measures the amount of dispersion of the data points around the line of equality between the observed and predicted values. See Chapter 5 for further discussion on this issue.
- **Construction Month and Traffic Open Month**—Construction completion and traffic opening dates (month and year) are site construction features. The construction months in AASHTOWare Pavement ME Design represent the month and year that the unbound layers have been compacted and finished (base/subgrade construction month), and the month and year that the HMA or PCC has been placed to cover the unbound layers (pavement construction month). The traffic open month represents the month and year that the roadway is opened to the public. These dates are keyed to the monthly traffic loadings, monthly climatic inputs that affect all monthly layer and subgrade modulus values, and material-aging models. The construction and traffic opening month both begin on the first day of the month. AASHTOWare Pavement ME Design excludes any damage caused by construction traffic. See Section 6.2 for further discussion on these input parameters.
- **Design Criteria or Threshold Values**—These values are used to determine the life of the pavement structure and rehabilitation strategy, and are inputs to AASHTOWare Pavement ME Design software. These values represent the amount of distress or roughness that would trigger some type of major rehabilitation activity, and are typically policy decisions. See Section 7.1 for further discussion on this input parameter.
- **Design Life**—The design life of a new, reconstructed, or rehabilitated pavement is the time from initial construction until the pavement has structurally deteriorated to the point when significant

rehabilitation or reconstruction is needed. The design life of a particular trial design is defined by the initial pavement construction until a specified critical pavement condition has been reached. The software can handle design lives from one year (e.g., detour) to 99 years. Refer to discussion under Section 6.1 regarding design lives exceeding 30 years.

- **Endurance Limit**—The endurance limit is defined as the tensile strain or stress below which no load-related fatigue damage occurs. AASHTOWare Pavement ME Design does consider the endurance limit as a material property for HMA layers, which is input by the designer. The endurance limit is assumed to be independent of temperature or mixture modulus—a single value is used for all HMA mixtures within a single run of the software. The endurance limit, however, was excluded from the global calibration effort (*18*) and, thus, should not be used without re-calibration of the fatigue cracking model.
- **Incremental Damage**—Incremental damage (ΔDI) is a ratio defined by the actual number of wheel load applications (n) for a specified axle load and type within an interval of time divided by the allowable number of wheel load applications (N) defined for the same axle load and type for the conditions that exist within the same specific period of time. The incremental damage indices are summed to determine the cumulative damage index over time.
- **Long-Life Pavements**—Flexible or rigid pavements that have been designed for a 50+ year service life. In other words, the design life of the pavement equals or exceeds 50 years. Long-life pavements are also referred to as perpetual pavements. Refer to discussion under Section 6.1 regarding long-life pavements.
- **Reliability of Trial Design**—The probability that the predicted performance indicator of the trial design will not exceed the design criteria within the design-analysis period. The design reliability (R) is the probability that the pavement will not exceed specific failure criteria over the design traffic. See Section 7.2 for further discussion on this input parameter.
- **Standard Error of the Estimate (s_e)**—The standard deviation of the residual errors (predicted minus measured values) for the pavement sections included in the global calibration data set.
- **Structural Response Model**—The structural response model is a mechanistic model based on fundamental engineering principles and used to calculate critical pavement responses (deflections, stresses, and strains). The Jacob Uzan Layered Elastic Analysis (JULEA) program is the structural response model used for flexible pavements, while for rigid pavements, the ISLAB2000 program is used. A stress dependent finite element program is also available for flexible pavement analyses using input Level 1 for unbound materials, but was not included in the global calibration effort. The use of the finite element program for flexible pavements is intended for research purposes only.
- **Transfer Function**—The transfer function is the empirical part of the distress prediction model that relates the critical pavement response parameter, either directly or through the damage concept, to pavement distress.

4.2 HIERARCHICAL INPUT LEVELS

The hierarchical input level included in AASHTOWare Pavement ME Design is an input scheme that is used to categorize the designer's knowledge of the input parameter. Three levels are available for determining the input values for most of the material and traffic parameters. Chapter 5 provides more detailed discussion on the purpose, use, and selection of the hierarchical input level for pavement design. The following list defines each hierarchical input level that may be used by the designer:

- **Input Level 1**—Input parameter is measured directly; it is site- or project-specific. This level represents the greatest knowledge about the input parameter for a specific project but has the highest testing and data collection costs to determine the input value. Level 1 should be used for pavement designs having unusual site features, materials, or traffic conditions that are outside the inference-space used to develop the correlations and defaults included for input Levels 2 and 3.
- **Input Level 2**—Input parameter is estimated from correlations or regression equations. In other words, the input value is calculated from other site-specific data or parameters that are less costly to measure. Input Level 2 may also represent measured regional values that are not project-specific.
- **Input Level 3**—Input parameter is based on "best-estimated" or default values. Level 3 inputs are based on global or regional default values—the median value from a group of data with similar characteristics. This input level has the least knowledge about the input parameter for the specific project but has the lowest testing and data collection costs.

4.3 TRUCK TRAFFIC TERMS

- **Axle-Load Spectra**—The axle-load spectra is a histogram or distribution of axle loads for a specific axle type (single, tandem, tridem, quad). In other words, the number of axle applications within a specific axle-load range.
- **Hourly Distribution Factors**—The percentage of trucks using a facility for each hour of the day. This input is only necessary for rigid pavement design. The sum of the hourly distribution factors must total 100 percent.
- **Monthly Distribution Factors**—This value defines the distribution of truck volumes on a monthly basis in a typical year. The sum of all monthly distribution factors for a specific truck class must total 12, as used in AASHTOWare Pavement ME Design.
- **Normalized Axle-Load Spectra**—The normalized axle-load spectra is a normalized histogram of axle loads for a specific axle type. To determine the normalized load spectra, the number of axle applications weighed within a specific load range for an axle type is divided by the total number of axles weighed for that axle type. The cumulative sum of all incremental values in the distribution for a specific axle type equal 100 percent.
- **Normalized Truck Classification Distribution**—The normalized truck volume distribution is a normalized distribution of the different truck classes within the traffic stream. To determine the normalized truck class volume distribution, the number of trucks counted within a specific classification is divided by the total number of trucks counted. The cumulative sum of all incremental values for all of the truck classifications equals 100 percent.

- **Truck Classification Distribution**—The distribution of the number of truck applications for each truck classification for all trucks counted. Trucks are defined as vehicle classes 4 through 13 using the FHWA classifications (*10*).
- **Truck Traffic Classification (TTC) Group**—An index type number that defines a group of roadways with similar normalized axle-load spectra and normalized truck volume distribution. Stated differently, the truck traffic classification (TTC) group is a value used to define the axle-load spectra and truck volume distribution from count data. In summary, it provides default values for the normalized axle-load spectra and normalized truck classification volume distributions.

The default normalized axle-load spectra for each axle type and normalized truck classification volume distribution for the 17 different TTC groups included in AASHTOWare Pavement ME Design were determined from analyzing the traffic data collected on over 180 LTPP test sections.

4.4 SMOOTHNESS

Functional adequacy is quantified by pavement smoothness for both flexible and rigid pavements. Rough roads lead not only to user discomfort but also to higher vehicle operating costs. The parameter used to define pavement smoothness in AASHTOWare Pavement ME Design is IRI. IRI is derived from the simulation of a "quarter-car" traveling along the longitudinal profile of the road and is calculated from the mean of the longitudinal profiles in each wheel path.

In AASHTOWare Pavement ME Design, IRI is predicted empirically as a function of pavement distresses (defined in Sections 4.5 and 4.6), site factors that represent the foundation's shrink/swell and frost heave capabilities, and an estimate of the IRI at the time of construction (the initial IRI). The pavement distress types that enter the IRI prediction are a function of the pavement or rehabilitation type under consideration (see Chapter 5 for details of the prediction equations). The unit of smoothness calculated by AASHTOWare Pavement ME Design is inches per mile.

4.5 DISTRESSES OR PERFORMANCE INDICATORS TERMS—HMA-SURFACED PAVEMENTS

- **Alligator Cracking (Bottom Up Cracking)**—A form of fatigue or wheel load related cracking and is defined as a series of interconnected cracks (characteristically with a "alligator" pattern) that initiate at the bottom of the HMA layers. Alligator cracks initially show up as multiple short, longitudinal or transverse cracks in the wheel path that become interconnected laterally with continued truck loadings. Alligator cracking is calculated as a percent of total lane area in AASHTOWare Pavement ME Design.
- **Longitudinal Cracking (Top Down Cracking)**—A form of fatigue or wheel load related cracking that occurs within the wheel path and is defined as cracks predominantly parallel to the pavement centerline. Longitudinal cracks initiate at the surface of the HMA pavement and initially show up as short longitudinal cracks that become connected longitudinally with continued truck loadings. Raveling or crack deterioration may occur along the edges of these cracks but they do not form an alligator cracking pattern. The unit of longitudinal cracking calculated by AASHTOWare Pavement ME Design is total feet per mile, including both wheel paths.

Thermal Transverse Cracking—Non-wheel load-related cracking that is predominately perpendicular to the pavement centerline and caused by low temperatures or thermal cycling. The unit of transverse cracking calculated by AASHTOWare Pavement ME Design in feet per 12-ft-wide lane.

- **Reflection Transverse Cracking—**Non-wheel load cracking induced by reflection from transverse joint or crack in underlying pavement. Cracking units are in percent lane area (crack width = 1 ft) (area cracked = linear ft of crack × 1 ft width).
- **Rutting or Rut Depth**—A surface depression in the wheel path resulting from plastic or permanent deformation in each pavement layer. The rut depth is representative of the maximum vertical difference in elevation between the transverse profile of the HMA surface and a wire-line across the lane width. The unit of rutting calculated by AASHTOWare Pavement ME Design is inches (millimeters), and represents the maximum mean rut depth between both wheel paths. AASHTOWare Pavement ME Design also computes the rut depths within the HMA, unbound aggregate layers, and foundation.

4.6 DISTRESS OR PERFORMANCE INDICATORS TERMS—PCC-SURFACED PAVEMENTS

- **Mean Transverse Joint Faulting (JPCP)**—Transverse joint faulting is the differential elevation across the joint measured approximately 1–3 ft from the slab edge (longitudinal joint for a conventional lane width), or from the rightmost lane paint stripe for a widened slab. Since joint faulting varies significantly from joint to joint, the mean faulting of all transverse joints in a pavement section is the parameter predicted by AASHTOWare Pavement ME Design. The unit of faulting calculated by AASHTOWare Pavement ME Design is inches.

  Faulting is an important deterioration mechanism of JPCP because of its impact on ride quality. Transverse joint faulting is the result of a combination of repeated applications of moving heavy axle loads, poor load transfer across the joint, free moisture beneath the PCC slab, erosion of the supporting base/subbase, subgrade, or shoulder base material, and upward curling of the slab.
- **Bottom-Up Transverse Cracking (JPCP)**—When the truck axles are near the longitudinal edge of the slab, midway between the transverse joints, a critical tensile bending stress occurs at the bottom of the slab under the wheel load. This stress increases greatly when there is a high-positive temperature gradient through the slab (the top of the slab is warmer than the bottom of the slab). Repeated loadings of heavy axles under those conditions result in fatigue damage along the bottom edge of the slab, which eventually result in a transverse crack that propagates to the surface of the pavement. Bottom-up transverse cracking is calculated by AASHTOWare Pavement ME Design as a percent of the total number of slabs. The output parameter (percent of slabs with transverse cracks) combines the percentage of slabs with bottom-up and top-down transverse cracks.
- **Top-Down Transverse Cracking (JPCP)**—Repeated loading by heavy truck tractors with certain axle spacing when the pavement is exposed to high negative temperature gradients (the top of the slab cooler than the bottom of the slab) result in fatigue damage at the top of the slab, which eventually results in a transverse or diagonal crack that is initiated on the surface of the pavement. The

critical wheel loading condition for top-down cracking involves a combination of axles that loads the opposite ends of a slab simultaneously. In the presence of a high-negative temperature gradient, such load combinations cause a high-tensile stress at the top of the slab near the critical pavement edge. This type of loading is most often produced by the combination of steering and drive axles of truck tractors and other vehicles. Multiple trailers with relatively short trailer-to-trailer axle spacing are other common sources of critical loadings for top-down cracking. Top-down transverse cracking is calculated by AASHTOWare Pavement ME Design as a percent of the total number of slabs. The output parameter (percent of slabs with transverse cracks) combines the percentage of slabs with top-down transverse cracks and the percentage of slabs with bottom-up transverse cracks.

- **CRCP Punchouts**—When truck axles pass along near the longitudinal edge of the slab between two closely spaced transverse cracks, a high-tensile stress occurs transversely across the pavement at the top of the slab within a distance of 48 in. from the edge. This stress increases greatly when there is loss of load transfer across the transverse cracks or loss of support along the edge of the slab. Repeated loading of heavy axles results in fatigue damage at the top of the slab, which results first in micro-cracks that initiate at the transverse crack and propagate longitudinally across the slab to the other transverse crack resulting in a punchout. The punchouts in CRCP are predicted considering the loss of crack LTE and erosion along the edge of the slab over the design life, and the effects of permanent and transitory moisture and temperature gradients. The transverse crack width is the most critical factor affecting LTE and, therefore, punchout development. Only medium- and high-severity punchouts, as defined by LTPP (9), are included in AASHTOWare Pavement ME Design model global calibration. The unit of punchouts calculated by AASHTOWare Pavement ME Design is the number of medium- and high-severity punchouts per lane mile (number per kilometer).

CHAPTER 5

Performance Indicator Prediction Methodologies

The design and analysis of a trial design is based upon the accumulation of damage as a function of climate and traffic loadings over time. The MEPDG methodology is based upon an incremental damage approach. Distress or damage is estimated and accumulated for each analysis interval. An analysis interval of one month is defined as the basic unit for estimating incremental damage. The analysis interval reduces to semi-monthly during freeze and thaw periods because of the possible rapid change in the resilient modulus of the unbound layers under these conditions.

This section of the manual introduces the mathematical relationships used to predict each of the performance indicators (distresses and smoothness). The section is divided into four parts: (1) an overview of selecting input levels for flexible and rigid pavement designs, (2) a brief overview of the calibration factors, (3) an overview of the distress prediction equations for flexible pavements and HMA overlays, and (4) an overview of the distress prediction equations for rigid pavements and PCC overlays. The standard error for each prediction equation and transfer function is included in the discussion.

5.1 SELECTING THE INPUT LEVELS

The hierarchical input levels, defined in Section 4.2, allow state agencies and users with minimal experience in M-E based procedures and standard test equipment to start using the method at the basic level with little initial investment. The pavement designer has flexibility in obtaining the inputs for a design project based on the importance of the project and the available resources.

For any given design project, inputs can be a combination of various levels. For example, a rigid pavement design may have a Level 1 input for concrete modulus of rupture, a Level 2 input for traffic load spectra, and a Level 3 input for subgrade resilient modulus. This approach is possible because the computational algorithm for damage and distress is exactly the same, regardless of the input levels. The agency determines which input level to adopt with the understanding that each input level for each parameter will have an associated standard error.

Table 5-1 provides a general listing of the typical input levels used for the re-calibration effort to assist the user in judging the applicability of the standard error terms to the trial design. The best quality inputs available for pavement sections were used to calibrate AASHTOWare Pavement ME Design to determine the standard error of each prediction model presented in this chapter.

Chapters 8 through 10 provide guidance on determining the input level for each input group. If a user decides to routinely use all Level 3 inputs, the standard errors will probably be higher than those included in AASHTOWare Pavement ME Design. Also, if an agency selects input levels that deviate from the levels used in the re-calibration effort, the agency should definitely consider completing a local calibration to determine the appropriate standard errors for each distress prediction model. In the interim, designers may use the standard errors determined from the global calibration process.

Table 5-1. Typical Input Levels Used in AASHTOWare Pavement ME Design Models

Input Group		Input Parameter	Recalibration Input Level Used
Truck Traffic		Axle Load Distributions (single, tandem, tridem)	Level 1
		Truck Volume Distribution	Level 1
		Lane and Directional Truck Distributions	Level 1
		Tire Pressure	Level 3
		Axle Configuration, Tire Spacing	Level 3
		Truck wander	Level 3
Climate		Temperature, Wind Speed, Cloud Cover, Precipitation, Relative Humidity	Level 1 Weather Stations
Material Properties	Unbound Layers and Subgrade	Resilient Modulus—All Unbound Layers	Level 1; Backcalculation
		Classification and Volumetric Properties	Level 1
		Moisture-Density Relationships	Level 1
		Soil-Water Characteristic Relationships	Level 3
		Saturated Hydraulic Conductivity	Level 3
	HMA	HMA Dynamic Modulus	Level 3
		HMA Creep Compliance and Indirect Tensile Strength	Levels 1, 2, and 3
		Volumetric Properties	Level 1
		HMA Coefficient of Thermal Expansion	Level 3
	PCC	PCC Elastic Modulus	Level 1
		PCC Flexural Strength	Level 1
		PCC Indirect Tensile Strength (CRCP Only)	Level 2
		PCC Coefficient of Thermal Expansion	Level 1
All Materials		Unit Weight	Level 1
		Poisson's Ratio	Levels 1 and 3
		Other Thermal Properties; conductivity, heat capacity, surface absorptivity	Level 3
Existing Pavement		Condition of Existing Layers	Levels 1 and 2

5.2 CALIBRATION FACTORS INCLUDED IN AASHTOWARE PAVEMENT ME DESIGN

The distress prediction models in AASHTOWare Pavement ME Design have been calibrated using data from a large set of actual roadway sections distributed throughout the United States. The primary source of data was the LTPP database supplemented by data obtained from the MnRoad experiment and other state and Federal agency research projects. The data included in the data set represent a wide variety of inputs, such as foundation soil types, traffic, climate, pavement types, design features within a pavement type, and time history of pavement performance.

A summary of the number of observations used to calibrate each distress model is presented in the sections that follow for each performance indicator.

Despite extensive efforts to collect data to perform global calibration, not all pavement types or design aspects of a given pavement type could be included due to the limitations inherent within the databases used to construct the calibration data set. To minimize the impact of the lack of data AASHTOWare Pavement ME Design has a unique feature that allows the designer to "adjust" the global calibration factors or use agency specific regression constants for individual distress damage functions based on user generated local and regional data sets.

The *Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide, 1st Edition*, provides specific guidance on determining agency specific calibration adjustment factors with AASHTOWare Pavement ME Design (2). The steps required for determining the local or agency specific calibration factors are not included in this manual of practice. The standard error equation defined from the global calibration process may also be changed; however, care must be exercised in doing so.

5.3 DISTRESS PREDICTION EQUATIONS FOR FLEXIBLE PAVEMENTS AND HMA OVERLAYS

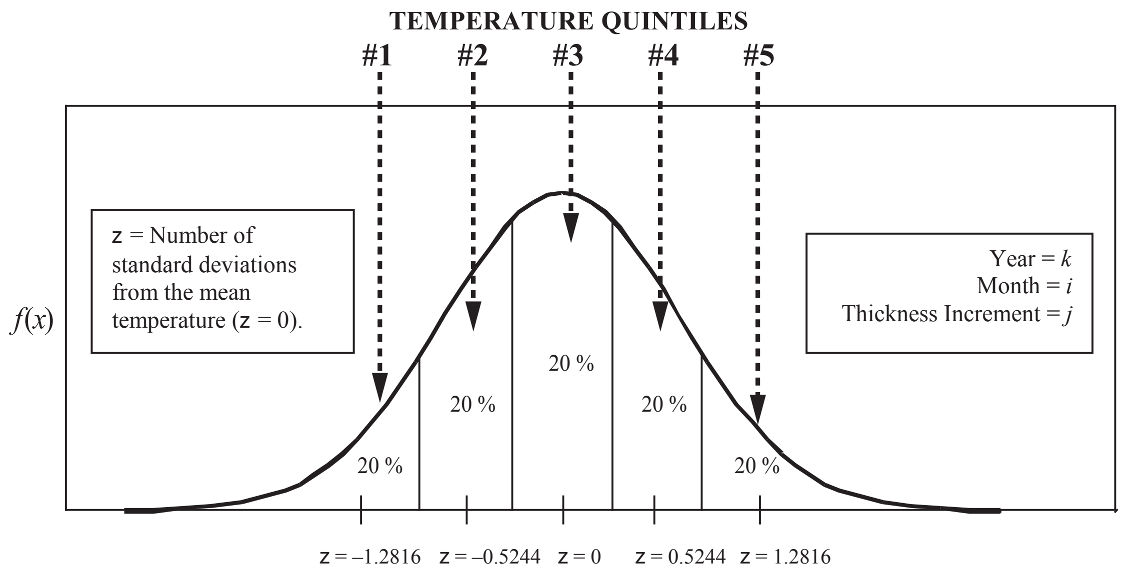
The following summarizes the methodology and mathematical models used to predict each performance indicator. (See Figure 5-1.)

5.3.1 Overview of Computational Methodology for Predicting Distress

AASHTOWare Pavement ME Design software subdivides the structural layers and foundation of the trial design into sublayers. The thickness of the sublayers is dependent on the material type, actual layer thickness, and depth within the pavement structure. The number of layers considered permissible for the different design strategies is given and discussed in more detail in Chapters 11 and 12.

Critical pavement responses are calculated in each sublayer using the elastic layer theory program identified as JULEA, which is embedded in AASHTOWare Pavement ME Design software. AASHTOWare Pavement ME Design makes extensive use of the EICM that is embedded in the software for adjusting the pavement layer modulus values with temperature and moisture. The EICM calculates the temperature and moisture conditions throughout the pavement structure on an hourly basis (*15*).

The frequency distribution of HMA temperatures using the EICM is assumed to be normally distributed. The temperatures in each HMA sublayer are combined into five quintiles. Each quintile represents 20 percent of the frequency distribution for each month of the analysis period for the load-related distresses. This is accomplished by computing pavement temperatures corresponding to accumulated frequencies of 10, 30, 50, 70, and 90 percent within a given month. The average temperature within each quintile of a sublayer for each month is used to determine the dynamic modulus of that sublayer. The truck traffic is assumed to be equal within each of the five temperature quintiles. Thus, the flexible pavement procedure does not tie the hourly truck volumes directly to the hourly temperatures.



Pavement temperatures within each thickness increment of the HMA layers are calculated for each month via the EICM. The pavement temperatures are then combined into five equal groups, as shown above, assuming a normal distribution. The mean pavement temperature within each group for each month for each HMA thickness increment is determined for calculating the dynamic modulus as a function of time and depth in the pavement.

Figure 5-1. Graphical Illustration of the Five Temperature Quintiles Used in the AASHTOWare Pavement ME Design to Determine HMA-Mixture Properties for Load-Related Distresses

The dynamic modulus is used to compute the horizontal and vertical strains at critical depths on a grid to determine the maximum permanent deformation within each layer and location of the maximum fatigue damage in the asphalt concrete layers. For transverse cracks (non-load-related cracks), the EICM calculates the HMA temperatures on an hourly basis and AASHTOWare Pavement ME Design uses those hourly temperatures to estimate the HMA properties (creep compliance and indirect tensile strength) to calculate the tensile stress throughout the HMA surface layer.

The EICM also calculates the temperatures within each unbound sublayer and determines the months when any sublayer is frozen. The resilient modulus of the frozen sublayers is then increased during the frozen period and decreased during the thaw weakening period. The EICM also calculates the average moisture content in the unbound layers for each month of the analysis period. The average monthly moisture content relative to the optimum moisture content is used to adjust the resilient modulus of each unbound sublayer for each month throughout the analysis period.

The critical pavement responses are used to calculate the fatigue damage, thermal cracking damage, and permanent deformation. The remainder of this subsection provides the mathematical relationships used to predict each performance indicator.

5.3.2 Rut Depth

Surface distortion in the form of rutting is caused by the plastic or permanent vertical deformation in the HMA, unbound layers, and foundation soil. The approach used in the MEPDG is based upon calculating incremental distortion or rutting within each sublayer. In other words, rutting is estimated for each sub-season at the mid-depth of each sub-layer within the pavement structure. The plastic deformation for a given season is the sum of the plastic vertical deformations within each layer.

The model for calculating total permanent deformation uses the plastic vertical strain under specific pavement conditions for the total number of trucks within that condition. Conditions vary from one month to another, so it is necessary to use a special approach called the "strain hardening" approach to incorporate those plastic vertical strains within each month in a cumulative deformation subsystem.

The rate or accumulation of plastic deformation is measured in the laboratory using repeated load permanent deformation triaxial tests for both HMA mixtures and unbound materials. The laboratory-derived relationship is then adjusted to match the rut depth measured on the roadway. For all HMA mixtures, the MEPDG field calibrated form of the laboratory derived relationship from repeated load permanent deformation tests is shown in Eq. 5-1a.

$$\Delta_{p(HMA)} = \varepsilon_{p(HMA)} h_{HMA} = \beta_{1r} k_z \varepsilon_{r(HMA)} 10^{k_{1r}} n^{k_{2r}\beta_{2r}} T^{k_{3r}\beta_{3r}} \tag{5-1a}$$

where:

$\Delta_{p(HMA)}$	=	Accumulated permanent or plastic vertical deformation in the HMA layer/sublayer, in.,
$\varepsilon_{p(HMA)}$	=	Accumulated permanent or plastic axial strain in the HMA layer/sublayer, in./in.,
$\varepsilon_{r(HMA)}$	=	Resilient or elastic strain calculated by the structural response model at the mid-depth of each HMA sublayer, in./in.,
$h_{(HMA)}$	=	Thickness of the HMA layer/sublayer, in.,
n	=	Number of axle-load repetitions.,
T	=	Mix or pavement temperature, °F,
k_z	=	Depth confinement factor,
$k_{1r,2r,3r}$	=	Global field calibration parameters (k_{1r} = −3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606), and
β_{ir}, β_{2r}, β_{3r},	=	Local or mixture field calibration constants; for the global calibration, these constants were all set to 1.0.

$$k_z = (C_1 + C_2 D)0.328196^D \tag{5-1b}$$

$$C_1 = -0.1039(H_{HMA})^2 + 2.4868H_{HMA} - 17.342 \tag{5-1c}$$

$$C_2 = 0.0172(H_{HMA})^2 - 1.7331H_{HMA} + 27.428 \tag{5-1d}$$

where:

D	=	Depth below the surface, in., and
H_{HMA}	=	Total HMA thickness, in.

Eq. 5-2a shows the field-calibrated mathematical equation used to calculate plastic vertical deformation within all unbound pavement sublayers and the foundation or embankment soil.

$$\Delta_{p(soil)} = \beta_{s1} k_{s1} \varepsilon_v h_{soil} \left(\frac{\varepsilon_o}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}} \tag{5-2a}$$

where:

$\Delta_{p(Soil)}$	=	Permanent or plastic deformation for the layer/sublayer, in.,
n	=	Number of axle-load applications,
ε_o	=	Intercept determined from laboratory repeated load permanent deformation tests, in./in.,
ε_r	=	Resilient strain imposed in laboratory test to obtain material properties ε_o, β, and ρ, in./in.,
ε_v	=	Average vertical resilient or elastic strain in the layer/sublayer and calculated by the structural response model, in./in.,
h_{Soil}	=	Thickness of the unbound layer/sublayer, in.,
k_{s1}	=	Global calibration coefficients; k_{s1}=2.03 for granular materials and 1.35 for fine-grained materials, and
ε_{s1}	=	Local calibration constant for the rutting in the unbound layers; the local calibration constant was set to 1.0 for the global calibration effort.

$$Log\beta = -0.61119 - 0.017638(W_c) \tag{5-2b}$$

$$\rho = 10^9 \left(\frac{C_o}{\left(1-(10^9)^{\beta}\right)}\right)^{\frac{1}{\beta}} \tag{5-2c}$$

$$C_o = Ln\left(\frac{a_1 M_r^{b_1}}{a_9 M_r^{b_9}}\right) \tag{5-2d}$$

W_c	=	Water content, %,
M_r	=	Resilient modulus of the unbound layer or sublayer, psi,
$a_{1,9}$	=	Regression constants; a_1= 0.15 and a_9= 20.0, and
$b_{1,9}$	=	Regression constants; b_1= 0.0 and b_9= 0.0.

Figure 5-2 shows a comparison between the measured and predicted total rut depths, including the statistics from the global calibration process. The standard error (s_e) for the total rut depth is the sum of the standard error for the HMA and unbound layer rut depths and is a function of the average predicted rut depth. Eqs. 5-3a through 5-3c show the standard error (standard deviation of the residual errors) for the individual layers—HMA and unbound layers for coarse and fine-grained materials and soils.

$$s_{e(HMA)} = 0.24(\Delta_{HMA})^{0.8026} + 0.001 \tag{5-3a}$$

$$s_{e(Gran)} = 0.1235(\Delta_{Gran})^{0.5012} + 0.001 \tag{5-3b}$$

$$s_{e(Fine)} = 0.1477(\Delta_{Fine})^{0.6711} + 0.001 \tag{5-3c}$$

where:

Δ_{HMA} = Plastic deformation in the HMA layers, in.,

Δ_{Gran} = Plastic deformation in the aggregate and coarse-grained layers, in., and

Δ_{Fine} = Plastic deformation in the fine-grained layers and soils, in.

These equations for the standard errors of the predicted rut depths within each layer were not based on actual measurements of rutting within each layer, because trenches were unavailable for all LTPP test sections used in the global calibration process. The so-called "measured" rut depths within each layer were only estimated by proportioning the total rut depth measured to the different layers using a systematic procedure.

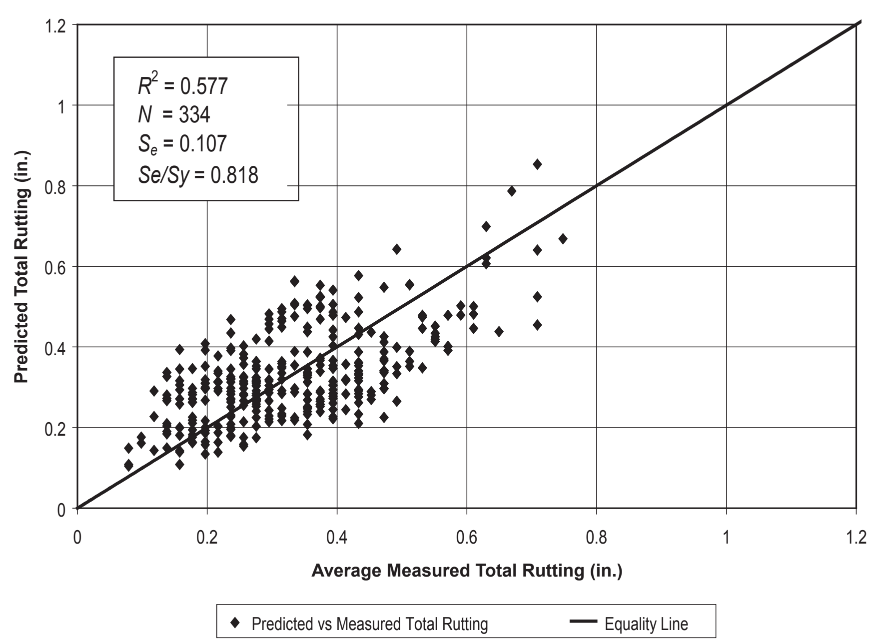


Figure 5-2. Comparison of Measured and Predicted Total Rutting Resulting from Global Calibration Process

5.3.3 Load-Related Cracking

Two types of load-related cracks are predicted by AASHTOWare Pavement ME Design, alligator cracking and longitudinal cracking. The MEPDG assumes that alligator or area cracks initiate at the bottom of the HMA layers and propagate to the surface with continued truck traffic, while longitudinal cracks are assumed to initiate at the surface. The allowable number of axle-load applications needed for the incremental damage index approach to predict both types of load related cracks (alligator and longitudinal) is shown in Eq. 5-4a.

$$N_{f-HMA} = k_{f1}(C)(C_H)\beta_{f1}(\varepsilon_t)^{k_{f2}\beta_{f2}}(E_{HMA})^{k_{f3}\beta_{f3}} \tag{5-4a}$$

where:

$N_{f\text{-}HMA}$ = Allowable number of axle-load applications for a flexible pavement and HMA overlays,
ε_t = Tensile strain at critical locations and calculated by the structural response model, in./in.,
E_{HMA} = Dynamic modulus of the HMA measured in compression, psi,
k_{f1}, k_{f2}, k_{f3} = Global field calibration coefficients (k_{f1} = 0.007566, k_{f2} = +3.9492, and k_{f3} = +1.281), and
β_{f1}, β_{f2}, β_{f3} = Local or mixture specific field calibration constants; for the global calibration effort, these constants were set to 1.0.

$$C = 10^M \tag{5-4b}$$

$$M = 4.84\left(\frac{V_{be}}{V_a + V_{be}} - 0.69\right) \tag{5-4c}$$

where:

V_{be} = Effective asphalt content by volume, %,
V_a = Percent air voids in the HMA mixture, and
C_H = Thickness correction term, dependent on type of cracking.

For bottom-up or alligator cracking:

$$C_H = \frac{1}{0.000398 + \dfrac{0.003602}{1 + e^{(11.02 - 3.49 H_{HMA})}}} \tag{5-4d}$$

For top-down or longitudinal cracking:

$$C_H = \frac{1}{0.01 + \dfrac{12.00}{1 + e^{(15.676 - 2.8186 H_{HMA})}}} \tag{5-4e}$$

where:

H_{HMA} = Total HMA thickness, in.

AASHTOWare Pavement ME Design calculates the incremental damage indices on a grid pattern throughout the HMA layers at critical depths. The incremental damage index (ΔDI) is calculated by dividing the actual number of axle loads by the allowable number of axle loads (defined by Eq. 5-4a, and referred to as Miner's hypothesis) within a specific

{	if $H_{HMA} \leq 2.5$ in.	$1 / (0.005169 H_{HMA}^{2.913059})$
	if $2.5 \text{ in} < H_{HMA} < 14.5$ in.	$1/(-0.046908\, H_{HMA}^{3} + 0.729644\, H_{HMA}2 - 0.635578\, H_{HMA} - 1.555892)$
	if $H_{HMA} \geq 14.5$ in.	$1/(0.235)$

time increment and axle-load interval for each axle type. The cumulative damage index (DI) for each critical location is determined by summing the incremental damage indices over time, as shown in Eq. 5-5.

$$DI = \sum (\Delta DI)_{j,m,l,p,T} = \sum \left(\frac{n}{N_{f-HMA}} \right)_{j,m,l,p,T} \tag{5-5}$$

where:

n = Actual number of axle-load applications within a specific time period,
j = Axle-load interval,
m = Axle-load type (single, tandem, tridem, or quad),
l = Truck type using the truck classification groups included in AASHTOWare Pavement ME Design,
p = Month, and
T = Median temperature for the five temperature intervals or quintiles used to subdivide each month, °F.

As noted under Section 4.1, General Terms, an endurance limit for HMA mixtures can be input into AASHTOWare Pavement ME Design, but this concept was excluded from the global calibration process. If the endurance limit concept is selected for use when running AASHTOWare Pavement ME Design all tensile strains that are less than the endurance limit input are excluded from calculating the incremental damage index for bottom-up or alligator cracking. The endurance limit concept is not applied in calculating the incremental damage for top-down or longitudinal cracking.

The area of alligator cracking and length of longitudinal cracking are calculated from the total damage over time (Eq. 5-5) using different transfer functions. Eq. 5-6a is the relationship used to predict the amount of alligator cracking on an area basis, FC_{Bottom}.

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1+e^{\left(C_1C_1^* + C_2C_2^* Log\left(DI_{Bottom}*100\right)\right)}}\right) \tag{5-6a}$$

where:

FC_{Bottom} = Area of alligator cracking that initiates at the bottom of the HMA layers, % of total lane area,

DI_{Bottom} = Cumulative damage index at the bottom of the HMA layers, and

$C_{1,2,4}$ = Transfer function regression constants; C_4= 6,000; C_1=1.00; and C_2=1.00.

$$C_1^* = -2C_2^* \tag{5-6b}$$

$$C_2^* = -2.40874 - 39.748\left(1 + H_{HMA}\right)^{-2.856} \tag{5-6c}$$

Figure 5-3 shows the comparison of the cumulative fatigue damage and measured alligator cracking, including the statistics from the global calibration process. The standard error, s_e, (standard deviation of the residual errors) for the alligator cracking prediction equation is shown in Eq. 5-7, and is a function of the average predicted area of alligator cracks.

$$s_{e(Alligator)} = 1.13 + \frac{13}{1+e^{7.57 - 15.5Log\left(FC_{Bottom}+0.0001\right)}} \tag{5-7}$$

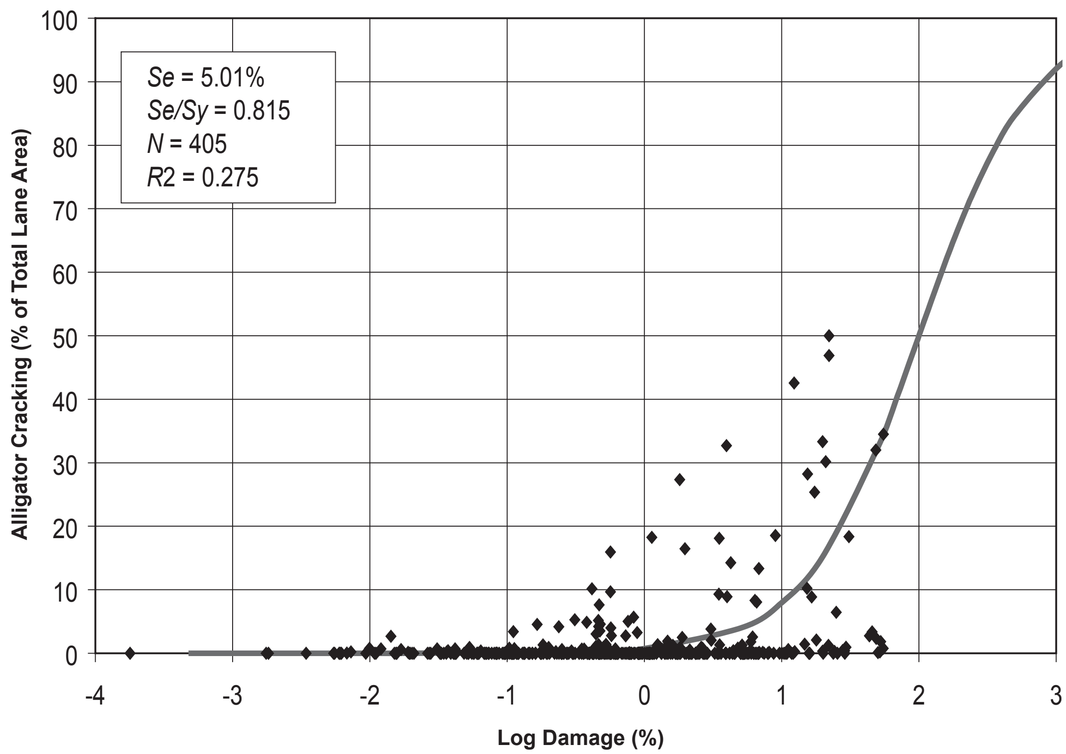


Figure 5-3. Comparison of Cumulative Fatigue Damage and Measured Alligator Cracking Resulting from Global Calibration Process

Eq. 5-8 is the relationship used to predict the length of longitudinal fatigue cracks, FC_{Top}.

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{\left(C_1 - C_2 Log\left(DI_{Top}\right)\right)}}\right) \tag{5-8}$$

where:

FC_{Top}	=	Length of longitudinal cracks that initiate at the top of the HMA layer, ft/mi,
DI_{Top}	=	Cumulative damage index near the top of the HMA surface, and
$C_{1,2,4}$	=	Transfer function regression constants; C_1= 7.00; C_2= 3.5; and C_4= 1,000.

Figure 5-4 shows a comparison between the measured and predicted lengths of longitudinal cracking (top-down cracking) and statistics resulting from the global calibration process. The standard error, s_e, (standard deviation of the residual errors) for the longitudinal cracking prediction equation is shown in Eq. 5-9, and is a function of the average predicted length of the longitudinal cracks.

$$s_{e(Long)} = 200 + \frac{2300}{1+e^{1.072-2.1654Log\left(FC_{Top}+0.0001\right)}} \tag{5-9}$$

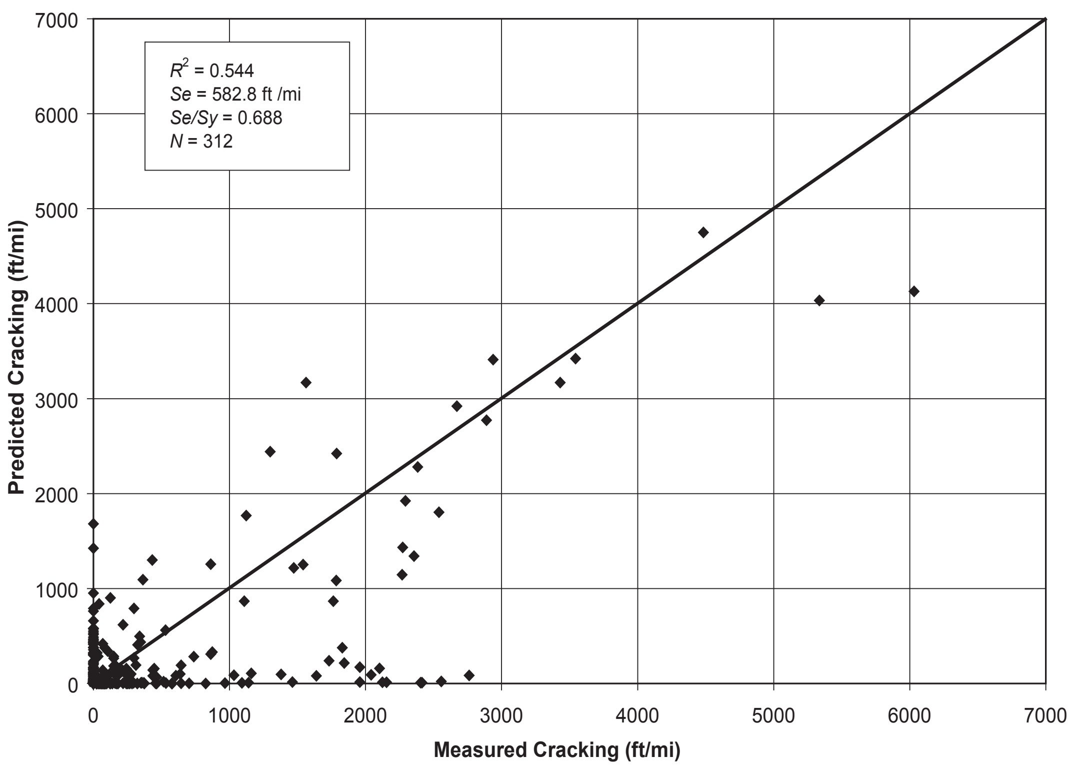


Figure 5-4. Comparison of Measured and Predicted Lengths of Longitudinal Cracking (Top-Down Cracking) Resulting from Global Calibration Process

For fatigue cracks in CTB layers, the allowable number of load applications, $N_{f\text{-}CTB}$, is determined in accordance with Eq. 5-10a and the amount or area of fatigue cracking is calculated in accordance with Eq. 5-10b. These damage and distress transfer functions were never calibrated under any of the NCHRP projects. The prediction equations are provided in this manual for completeness, but they are not recommended for use until the transfer function (Eq. 5-10b) has been calibrated.

$$N_{f-CTB} = 10^{\left[\frac{k_{c1}\beta_{c1}\left(\frac{\sigma_t}{M_R}\right)}{k_{c2}\beta_{c2}}\right]} \tag{5-10a}$$

$$FC_{CTB} = C_1 + \frac{C_2}{1+e^{\left(C_3 - C_4 Log\left(DI_{CTB}\right)\right)}} \tag{5-10b}$$

where:

$N_{f\text{-}CTB}$	=	Allowable number of axle-load applications for a semi-rigid pavement,
σ_t	=	Tensile stress at the bottom of the CTB layer, psi,
M_R	=	28-day modulus of rupture for the CTB layer, psi.
DI_{CTB}	=	Cumulative damage index of the CTB or cementitious layer and determined in accordance with Eq. 5-5,
$k_{c1,c2}$	=	Global calibration coefficients—Undefined because prediction equation was never calibrated; these values are set to 1.0 in the software. From other studies, k_{c1}=0.972 and k_{c2} = 0.0825,
$\beta_{c1,c2}$	=	Local calibration constants; these values are set to 1.0 in the software,
FC_{CTB}	=	Area of fatigue cracking, sq ft, and
$C_{1,2,3,4}$	=	Transfer function regression constants; C_1=1.0, C_2=1.0, C_3=0, and C_4=1,000. To date, this transfer function has not been calibrated and these values will change when it is calibrated.

The computational analysis of incremental fatigue cracking for a semi-rigid pavement uses the damaged modulus approach. In summary, the elastic modulus of the CTB layer decreases as the damage index, DI_{CTB}, increases. Eq. 5-10c is used to calculate the damaged elastic modulus within each season or time period for calculating critical pavement responses in the CTB and other pavement layers.

$$E_{CTB}^{D(t)} = E_{CTB}^{Min} + \left(\frac{E_{CTB}^{Max} - E_{CTB}^{Min}}{1+e^{\left(-4+14\left(DI_{CTB}\right)\right)}}\right) \tag{5-10c}$$

where:

$E_{CTB}^{D(t)}$	=	Equivalent damaged elastic modulus at time t for the CTB layer, psi,
E_{CTB}^{Min}	=	Equivalent elastic modulus for total destruction of the CTB layer, psi, and
E_{CTB}^{Max}	=	28-day elastic modulus of the intact CTB layer, no damage, psi.

5.3.4 Non-Load Related Cracking—Transverse Cracking

The thermal cracking model (*17*) is presented below.

$$\Delta C = A\left(\Delta K\right)^n \tag{5-11a}$$

where:

ΔC = Change in the crack depth due to a cooling cycle,

ΔK = Change in the stress intensity factor due to a cooling cycle, and

A, n = Fracture parameters for the HMA mixture.

Experimental results indicate that reasonable estimates of A and n can be obtained from the indirect tensile creep-compliance and strength of the HMA in accordance with Eqs. 5-11b and 5-11c.

$$A = k_t \beta_t 10^{[4.389 - 2.52 Log(E_{HMA} \sigma_m n)]} \quad (5\text{-}11b)$$

where:

$$\eta = 0.8\left[1 + \frac{1}{m}\right] \quad (5\text{-}11c)$$

k_t = Coefficient determined through global calibration for each input level (Level 1 = 1.5; Level 2 = 0.5; and Level 3 = 1.5),

E_{HMA} = HMA indirect tensile modulus, psi,

σ_m = Mixture tensile strength, psi,

m = The m-value derived from the indirect tensile creep compliance curve measured in the laboratory, and

β_t = Local or mixture calibration factor.

The stress intensity factor, K, has been incorporated in AASHTOWare Pavement ME Design through the use of a simplified equation developed from theoretical finite element studies (Eq. 5-11d).

$$K = \sigma_{tip}\left[0.45 + 1.99 (C_o)^{0.56}\right] \quad (5\text{-}11d)$$

where:

σ_{tip} = Far-field stress from pavement response model at depth of crack tip, psi, and

C_o = Current crack length, ft.

The degree of cracking is predicted by AASHTOWare Pavement ME Design using an assumed relationship between the probability distribution of the log of the crack depth to HMA-layer thickness ratio and the percent of cracking. Eq. 5-11e shows the expression used to determine the extent of thermal cracking.

$$TC = \beta_{t1} N\left[\frac{1}{\sigma_d} Log\left(\frac{C_d}{H_{HMA}}\right)\right] \quad (5\text{-}11e)$$

where:

TC = Observed amount of thermal cracking, ft/mi,

β_{t1} = Regression coefficient determined through global calibration (400),

$N[z]$ = Standard normal distribution evaluated at $[z]$,

σ_d = Standard deviation of the log of the depth of cracks in the pavement (0.769), in.,

C_d = Crack depth, in., and

H_{HMA} = Thickness of HMA layers, in.

Figure 5-5 includes a comparison between the measured and predicted cracking and the statistics from the global calibration process using each input level. The standard error for the transverse cracking prediction equations for the three input levels is shown in Eqs. 5-12a through 5-12c.

$$s_e(Level\ 1) = -0.1468(TC + 65.027) \tag{5-12a}$$

$$s_e(Level\ 2) = -0.2841(TC + 55.462) \tag{5-12b}$$

$$s_e(Level\ 3) = 0.3972(TC + 20.422) \tag{5-12c}$$

5.3.5 Reflection Cracking in HMA Overlays

AASHTOWare Pavement ME Design predicts reflection cracks in HMA overlays or HMA surfaces of semi-rigid pavements using an empirical equation. The empirical equation is used for estimating the amount of fatigue and thermal cracks from a non-surface layer that has reflected to the surface after a certain period of time. This empirical equation predicts the percentage of area of cracks that propagate through the HMA as a function of time using a sigmoid function, shown in Eq. 5-13a. However, this empirical equation was not recalibrated globally.

$$RC = \frac{100}{1 + e^{a(c)+bt(d)}} \tag{5-13a}$$

where:

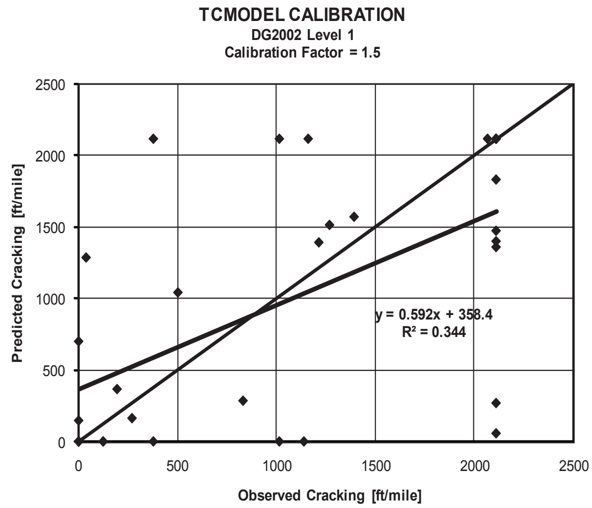
RC	=	Percent of cracks reflected. [Note: The percent area of reflection cracking is output with the width of cracks being 1 ft.],
t	=	Time, yr,
a, b	=	Regression fitting parameters defined through calibration process, and
c,d	=	User-defined cracking progression parameters.

The empirical equation also is used to estimate the reflection of fatigue and thermal cracks from a stabilized layer or existing flexible pavement, as well as from joints and cracks in a rigid pavement. The regression fitting parameters of Eq. 5-13a (*a* and *b*) are a function of the effective HMA overlay thickness (H_{eff}), the type of existing pavement, and for PCC pavements, load transfer at joints and cracks, as shown in Eqs. 5-13b and 5-13c. The effective HMA overlay thickness is provided in Table 5-2. The user-defined cracking progression parameters can be used by the user to accelerate or delay the amount of reflection cracks, which also are included in Table 5-2. Non-unity cracking progression parameters (*c* and *d*) can be used after local calibration

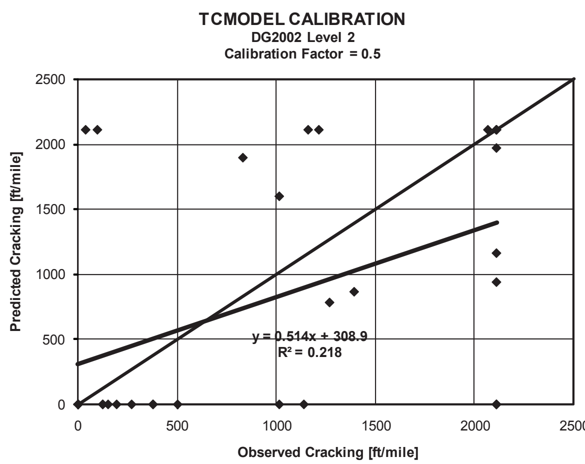
$$a = 3.5 + 0.75(H_{eff}) \tag{5-13b}$$

$$b = -0.688684 - 3.37302(H_{eff})^{-0.915469} \tag{5-13c}$$

5-5a Input Level 1 Using the Global Calibration Factor of 1.5



5-5b Input Level 2 Using the Global Calibration Factor of 0.5



5-5c Input Level 3 Using the Global Calibration Factor of 5.0

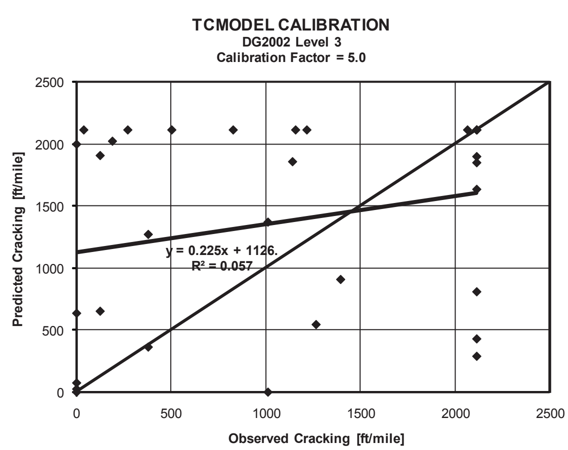


Figure 5-5. Comparison of Measured and Predicted Transverse Cracking Resulting from Global Calibration Process

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After HMA overlay placement, the underlying bound layers (all HMA, asphalt-bound layers, chemically stabilized layers, and PCC layers) undergo load-related damage with continued truck loadings. The continual fatigue damage accumulation of these layers is considered in the MEPDG HMA overlay analysis procedure. For any given month, *m*, the total fatigue damage is estimated by Eq. 5-14a.

$$DI_m = \sum_{i=1}^{m} \Delta DI_i \tag{5-14a}$$

where:

DI_m = Damage index for month *m*, and

ΔDI_i = Increment of damage index in month *i*.

Table 5-2. Reflection Cracking Model Regression Fitting Parameters

Pavement Type	Fitting and User-Defined Parameters; Eq. 5-13a			
	a and b	c	d	
	H_{eff} of Equations 5-13d and 5-13c		Delay Cracking by 2 yr	Accelerate Cracking by 2 yr
Flexible	$H_{eff} = H_{HMA}$	—	—	—
Rigid-Good Load Transfer	$H_{eff} = H_{HMA} - 1$	—	—	—
Rigid-Poor Load Transfer	$H_{eff} = H_{HMA} - 3$	—	—	—
Effective Overlay Thickness, H_{eff}, in.	—	—	—	—
<4	—	1.0	0.6	3.0
4 to 6	—	1.0	0.7	1.7
>6	—	1.0	0.8	1.4

Note: Minimum recommended H_{HMA} is 2 in. for existing flexible pavements, 3 in. for existing rigid pavements with good load transfer, and 4 in. for existing rigid pavements with poor load transfer.

The area of fatigue damage for the underlying layer at month *m* (CA_m) is given by Eq. 5-14b.

$$CA_m = \frac{100}{1 + e^{6-(6DI_m)}} \tag{5-14b}$$

For each month *i*, there will be an increment of damage ΔDI_i which will cause an increment of cracking area CA_i to the stabilized layer. To estimate the amount of cracking reflected from the stabilized layer to the surface of the pavement for month *m*, the reflective cracking prediction equation is applied incrementally, in accordance with Eq. 5-14c.

$$TRA_m = \sum_{i=1}^{m} RC_t\left(\Delta CA_i\right) \tag{5-14c}$$

where:

TRA_m = Total reflected cracking area for month *m*,

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RC_t = Percent cracking reflected for age *t* (in years), refer to Eq. 5-13a, and
ΔCA_i = Increment of fatigue cracking for month *i*.

5.3.6 Smoothness

The design premise included in AASHTOWare Pavement ME Design for predicting smoothness degradation is that the occurrence of surface distress will result in increased roughness (increasing IRI value), or in other words, a reduction in smoothness. Eqs. 5-15a through 5-15c were developed from data collected within the LTPP program and are embedded in AASHTOWare Pavement ME Design to predict the IRI over time for HMA-surfaced pavements.

Equation for New HMA Pavements and HMA Overlays of Flexible Pavements:

$$IRI = IRI_o + C_1(RD) + C_2(FC_{Total}) + C_3(TC) + C_4(SF) \tag{5-15a}$$

where:

IRI_0 = Initial IRI after construction, in./mi,
SF = Site factor, refer to Eq. 5-15b,
FC_{Total} = Area of fatigue cracking (combined alligator, longitudinal, and reflection cracking in the wheel path), percent of total lane area. All load related cracks are combined on an area basis—length of cracks is multiplied by 1 ft to convert length into an area basis,
TC = Length of transverse cracking (including the reflection of transverse cracks in existing HMA pavements), ft/mi, and
RD = Average rut depth, in.
$C_{1,2,3,4}$ = Calibration factors; $C_1 = 40.0$, $C_2 = 0.400$, $C_3 = 0.008$, $C_4 = 0.015$

The site factor (SF) is calculated in accordance with the following equation.

$$SF = Age^{1.5}\{\ln[(Precip+1)(FI+1)p_{02}]\} + \{\ln[(Precip+1)(PI+1)p_{200}]\} \tag{5-15b}$$

where:

Age = Pavement age, yr,
PI = Percent plasticity index of the soil,
FI = Average annual freezing index, °F days, and
$Precip$ = Average annual precipitation or rainfall, in.
p_{02} = Percent passing the 0.02 mm sieve
p_{200} = Percent passing the 0.075 mm sieve

Equation for HMA Overlays of Rigid Pavements:

$$IRI = IRI_o + PCC\ C_1(RD) + PCC\ C_2(FC_{Total}) + PCC\ C_3(TC) + PCC\ C_4(SF) \tag{5-15c}$$

where:

$PCC\ C_{1,2,3,4}$ = Calibration factors; $PCC\ C_1 = 40.8$, $PCC\ C_2 = 0.575$, $PCC\ C_3 = 0.0014$, $PCC\ C_4 = 0.00825$

Figures 5-6 and 5-7 compare the measured and predicted IRI values and include the statistics resulting from the global calibration process for flexible pavements and HMA overlays of flexible pavements and HMA overlays of PCC pavements, respectively. The standard error of the estimate for new flexible pavements and HMA overlays of flexible and semi-rigid pavements is 18.9 in./mi and for HMA overlays of intact PCC pavements it is 9.6 in./mi. The MEPDG assumes that the standard error for HMA overlays of fractured PCC pavements is the same as for HMA overlays of intact PCC pavements.

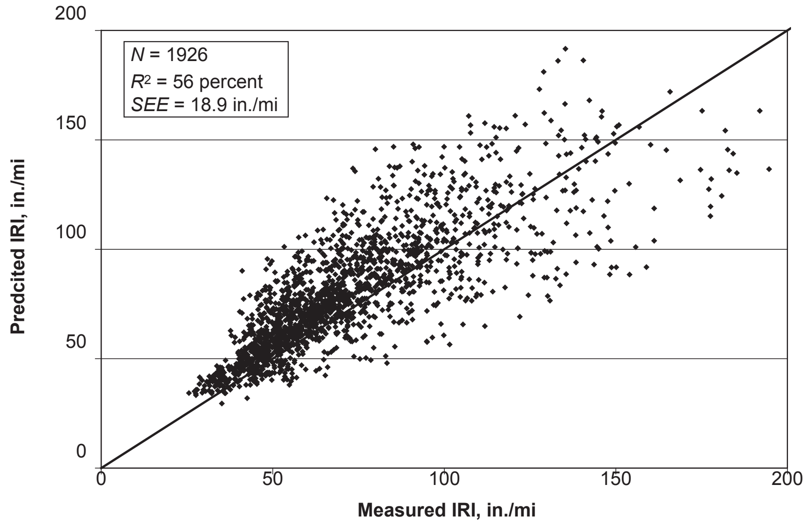

Figure 5-6. Comparison of Measured and Predicted IRI Values Resulting from Global Calibration Process of Flexible Pavements and HMA Overlays of Flexible Pavements

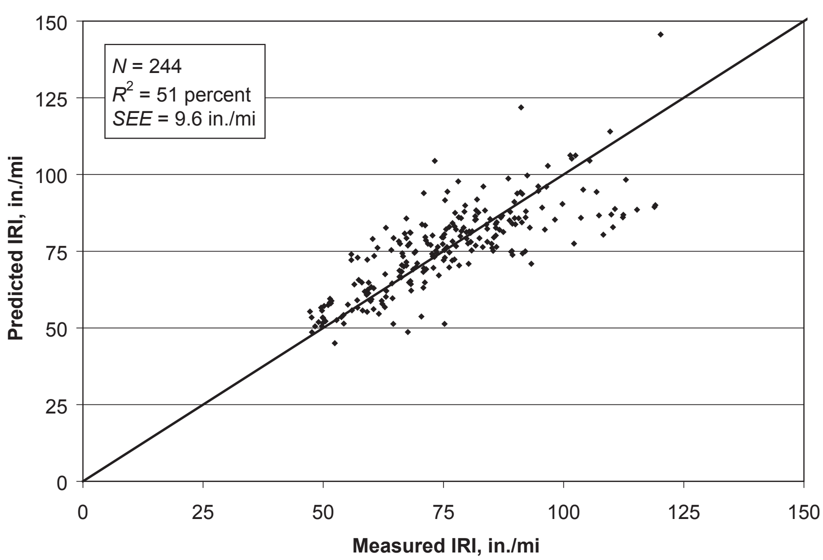

Figure 5-7. Comparison of Measured and Predicted IRI Values Resulting from Global Calibration Process of HMA Overlays of PCC Pavements

5.4 DISTRESS PREDICTION EQUATIONS FOR RIGID PAVEMENTS AND PCC OVERLAYS

The following summarizes the methodology and mathematical models used to predict each performance indicator.

5.4.1 Transverse Slab Cracking (Bottom-Up and Top-Down)—JPCP

As stated earlier for JPCP transverse cracking, both bottom-up and top-down modes of cracking are considered. Under typical service conditions, the potential for either mode of cracking is present in all slabs. Any given slab may crack either from bottom-up or top-down, but not both. Therefore, the predicted bottom-up and top-down cracking are not particularly meaningful by themselves, and combined cracking is reported excluding the possibility of both modes of cracking occurring on the same slab.

The percentage of slabs with transverse cracks (including all severities) in a given traffic lane is used as the measure of transverse cracking and is predicted using the following global equation for both bottom-up and top-down cracking:

$$CRK = \frac{100}{1 + C_4 \left(DI_F\right)^{C_5}} \tag{5-16}$$

where:

CRK	=	Predicted amount of bottom-up or top-down cracking (fraction), and
DI_F	=	Fatigue damage calculated using the procedure described in this section.
$C_{4,5}$	=	Calibration coefficients; $C_4 = 1.0$, $C_5 = -1.98$

The general expression for fatigue damage accumulations considering all critical factors for JPCP transverse cracking is as follows and referred to as Miner's hypothesis:

$$DI_F = \sum \frac{n_{i,j,k,l,m,n,o}}{N_{i,j,k,l,m,n,o}} \tag{5-17a}$$

where:

DI_F	=	Total fatigue damage (top-down or bottom-up),
$n_{i,j,k,\ldots}$	=	Applied number of load applications at condition *i, j, k, l, m, n*,
$N_{i,j,k,\ldots}$	=	Allowable number of load applications at condition *i, j, k, l, m, n*,
i	=	Age (accounts for change in PCC modulus of rupture and elasticity, slab/base contact friction, deterioration of shoulder LTE),
j	=	Month (accounts for change in base elastic modulus and effective dynamic modulus of subgrade reaction),
k	=	Axle type (single, tandem, and tridem for bottom-up cracking; short, medium, and long wheelbase for top-down cracking),
l	=	Load level (incremental load for each axle type), and
m	=	Equivalent temperature difference between top and bottom PCC surfaces.

n = Traffic offset path, and
o = Hourly truck traffic fraction.

The applied number of load applications ($n_{i,j,k,l,m,n}$) is the actual number of axle type k of load level l that passed through traffic path n under each condition (age, season, and temperature difference). The allowable number of load applications is the number of load cycles at which fatigue failure is expected (corresponding to 50 percent slab cracking) and is a function of the applied stress and PCC strength. The allowable number of load applications is determined using the following PCC fatigue equation:

$$\log\left(N_{i,j,k,l,m,n}\right) = C_1 \cdot \left(\frac{MR_i}{\sigma_{i,j,k,l,m,n}}\right)^{C_2} \tag{5-17b}$$

where:

$N_{i,j,k,\ldots}$ = Allowable number of load applications at condition *i, j, k, l, m, n,*
M_{Ri} = PCC modulus of rupture at age *i*, psi,
$\sigma_{i,j,k,\ldots}$ = Applied stress at condition *i, j, k, l, m, n,*
C_1 = Calibration constant, 2.0, and
C_2 = Calibration constant, 1.22.

The fatigue damage calculation is a process of summing damage from each damage increment. Once top-down and bottom-up damage are estimated, the corresponding cracking is computed using Eq. 5-16 and the total combined cracking determined using Eq. 5-18.

$$TCRACK = \left(CRK_{Bottom-up} + CRK_{Top-down} - CRK_{Bottom-up} \cdot CRK_{Top-down}\right) \cdot 100\% \tag{5-18}$$

where:

$TCRACK$ = Total transverse cracking (percent, all severities),
$CRK_{Bottop-up}$ = Predicted amount of bottom-up transverse cracking (fraction), and
$CRK_{Top-down}$ = Predicted amount of top-down transverse cracking (fraction).

It is important to note that Eq. 5-18 assumes that a slab cracks from either bottom-up or top-down, but not both. A plot of measured versus predicted transverse cracking and the statistics resulting from the global calibration process is shown in Figures 5-8 through 5-10.

Calculation of critical responses using neural nets (for speed) requires that the slab and base course are combined into an equivalent section based on equivalent stresses (load and temperature/moisture gradients), and contact friction between slab and base. This is done monthly as these parameters change over time.

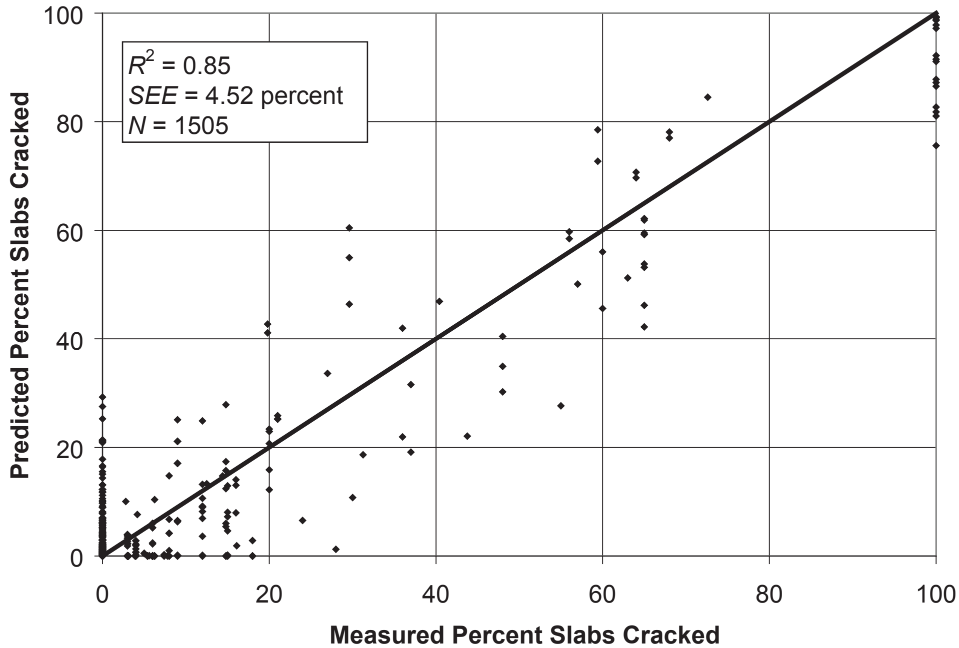

Figure 5-8. Comparison of Measured and Predicted Percentage JPCP Slabs Cracked Resulting from Global Calibration Process

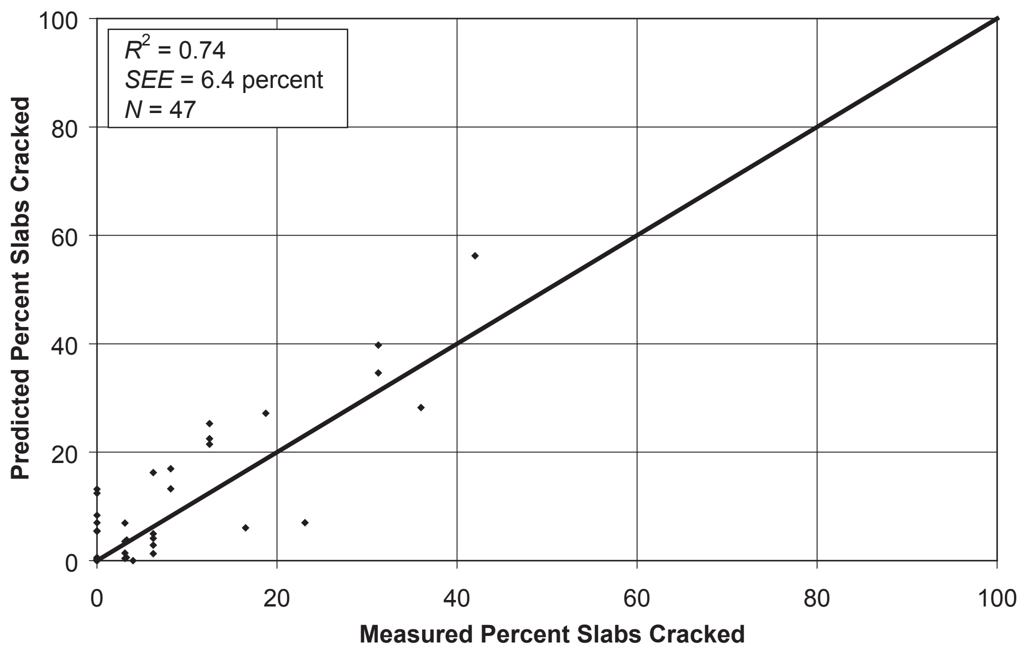

Figure 5-9. Comparison of Measured and Predicted Transverse Cracking of Unbounded JPCP Overlays Resulting from Global Calibration Process

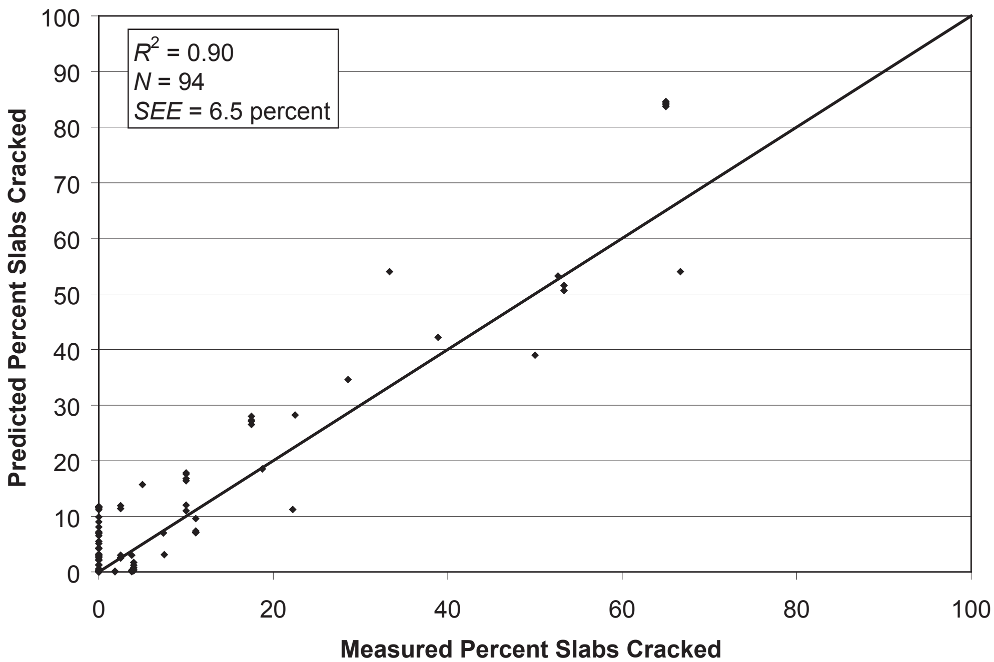

Figure 5-10. Comparison of Measured and Predicted Transverse Cracking for Restored JPCP Resulting from Global Calibration Process

The standard error (or standard deviation of the residual error) for the percentage of slabs cracked prediction global equation is shown in Eq. 5-19.

$$s_{e(CR)} = s_{e(CR)} = (5.3116*CRACK)^{0.3903} + 2.99 \tag{5-19}$$

where:

$CRACK$ = Predicted transverse cracking based on mean inputs (corresponding to 50 percent reliability), percentage of slabs, and

$s_{e(CR)}$ = Standard error of the estimate of transverse cracking at the predicted level of mean cracking.

5.4.2 Mean Transverse Joint Faulting—JPCP

The mean transverse joint faulting is predicted month by month using an incremental approach. A faulting increment is determined each month and the current faulting level affects the magnitude of increment. The faulting at each month is determined as a sum of faulting increments from all previous months in the pavement life from the traffic opening date using the following equations:

$$Fault_m = \sum_{i=1}^{m} \Delta Fault_i \tag{5-20a}$$

$$\Delta Fault_i = C_{34} * (FAULTMAX_{i-1} - Fault_{i-1})^2 * DE_i \tag{5-20b}$$

$$FAULTMAX_i = FAULTMAX_0 + C_7 * \sum_{j=1}^{m} DE_j * Log(1 + C_5 * 5.0^{EROD})^{C_6} \tag{5-20c}$$

$$FAULTMAX_0 = C_{12} * \delta_{curling} * \left[Log(1 + C_5 * 5.0^{EROD}) * Log(\frac{P_{200} * WetDays}{p_s}) \right]^{C_6} \quad (5\text{-}20d)$$

where:

$Fault_m$	=	Mean joint faulting at the end of month *m*, in.,
$\Delta Fault_i$	=	Incremental change (monthly) in mean transverse joint faulting during month *i*, in.,
$FAULTMAX_i$	=	Maximum mean transverse joint faulting for month *i*, in.,
$FAULTMAX_0$	=	Initial maximum mean transverse joint faulting, in.,
$EROD$	=	Base/subbase erodibility factor,
DE_i	=	Differential density of energy of subgrade deformation accumulated during month *i* (see Eq. 5-23),
$\delta_{curling}$	=	Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping,
P_S	=	Overburden on subgrade, lb,
P_{200}	=	Percent subgrade material passing #200 sieve,
$WetDays$	=	Average annual number of wet days (greater than 0.1 in. rainfall), and
$C_{1,2,3,4,5,6,7,12,34}$	=	Global calibration constants (C_1 = 1.0184; C_2 = 0.91656; C_3 = 0.0021848; C_4 = 0.0008837; C_5 = 250; C_6 = 0.4; C_7 = 1.83312; and C_{12} and C_{34} are defined by Eqs. 5-20e and 5-20f). Constants used for restored rigid pavements are: C_1 = 0.6; C_2 = 1.2; C_3 = 0.002125; C_4 = 0.000884; C_5 = 400; C_6 = 0.4; C_7 = 1.83312)

$$C_{12} = C_1 + C_2 * FR^{0.25} \quad (5\text{-}20e)$$

$$C_{34} = C_3 + C_4 * FR^{0.25} \quad (5\text{-}20f)$$

FR	=	Base freezing index defined as percentage of time the top base temperature is below freezing (32°F) temperature.

For faulting analysis, each passing of an axle may cause only one occurrence of critical loading, (i.e., when DE has the maximum value). Since the maximum faulting development occurs during nighttime when the slab is curled upward and joints are opened and the load transfer efficiencies are lower, only axle-load repetitions applied from 8:00 p.m. to 8:00 a.m. are considered in the faulting analysis.

For faulting analysis, the equivalent linear temperature difference for nighttime is determined for each calendar month as the mean difference between top and bottom PCC surfaces occurring from 8:00 p.m. to 8:00 a.m. For each month of the year, the equivalent temperature gradient for the month is then determined as follows:

$$\Delta T_m = \Delta T_{t,m} - \Delta T_{b,m} + \Delta T_{sh,m} + \Delta T_{PCW} \quad (5\text{-}21)$$

where:

ΔT_m = Effective temperature differential for month *m*,

$\Delta T_{t,m}$ = Mean PCC top-surface nighttime temperature (from 8:00 p.m. to 8:00 a.m.) for month *m*,

$\Delta T_{b,m}$ = Mean PCC bottom-surface nighttime temperature (from 8 p.m. to 8 a.m.) for month *m*,

$\Delta T_{sh,m}$ = Equivalent temperature differential due to reversible shrinkage for month *m* for old concrete (i.e., shrinkage is fully developed), and

ΔT_{PCW} = Equivalent temperature differential due to permanent curl/warp.

The temperature in the top PCC layer is computed at 11 evenly spaced points through the thickness of the PCC layer for every hour using the available climatic data. These temperature distributions are converted into the equivalent difference of temperatures between the top and bottom PCC surfaces.

Using the effective temperature differential for each calendar month and corresponding effective *k*-value and base modulus for the month, the corner deflections due to slab curling and shrinkage warping is determined for each month. The corner deflections are determined using a finite element-based neural network rapid response solution methodology implemented in AASHTOWare Pavement ME Design software. The initial maximum faulting is determined using the calculated corner deflections and Eq. 5-20d.

Using Eq. 5-20c, the maximum faulting is adjusted for the past traffic damage using past cumulative differential energy, (i.e., differential energy accumulated form axle-load applications for all month prior to the current month). For each increment, for each axle type and axle-load, deflections at the loaded and unloaded corner of the slab are calculated using the neural networks.

The magnitudes of corner deflections of loaded and unloaded slabs are highly affected by the joint LTE. To evaluate initial transverse joint LTE, the LTE from aggregate interlock, dowels (if present), and base/subgrade are determined. Table 5-3 lists the LTE_{base} values that are included in AASHTOWare Pavement ME Design software. After the contributions of the aggregate interlock, dowels, and base/subgrade are determined, the total initial joint load transfer efficiency is determined as follows:

$$LTE_{joint} = 100\left(1-(1-LTE_{dowel}/100)(1-LTE_{agg}/100)(1-LTE_{base}/100)\right) \tag{5-22}$$

where:

LTE_{joint} = Total transverse joint LTE, %,

LTE_{dowel} = Joint LTE if dowels are the only mechanism of load transfer, %,

LTE_{base} = Joint LTE if the base is the only mechanism of load transfer, %, and

LTE_{agg} = Joint LTE if aggregate interlock is the only mechanism of load transfer, %.

The LTE is determined and output for each calendar month and can be observed over time to see if it maintains a high level. If the mean nighttime PCC temperature at the mid-depth is below freezing (32°F) then joint LTE for that month is increased. That is done by assigning base LTE for that month equal to 90 percent. The aggregate interlock and dowel component of LTE are adjusted every month.

Table 5-3. Assumed Effective Base LTE for Different Base Types

Base Type	LTE_{Base}
Aggregate Base	20%
ATB or CTB	30%
Lean Concrete Base	40%

Using Eq. 5-20c, the maximum faulting is adjusted for the past traffic damage using past cumulative differential energy, (i.e., differential energy accumulated from axle-load applications for all months prior to the current month). For each increment, for each axle type and axle load, deflections at the loaded and unloaded corner of the slab are calculated using the neural networks. Using these deflections, the differential energy of subgrade deformation, *DE*, shear stress at the slab corner, τ, and (for doweled joints) maximum dowel bearing stress, σ_b are calculated:

$$DE = \frac{k}{2}\left(\delta_{loaded}^{2} - \delta_{unloaded}^{2}\right) \tag{5-23a}$$

$$\tau = \frac{AGG * (\delta_{loaded} - \delta_{unloaded})}{h_{PCC}} \tag{5-23b}$$

$$\sigma_b = \frac{\zeta_d * (\delta_{loaded} - \delta_{unloaded})}{d * dsp} \tag{5-23c}$$

where:

DE	=	Differential energy, lb/in.,
δ_{loaded}	=	Loaded corner deflection, in.,
$\delta_{unloaded}$	=	Unloaded corner deflection, in.,
AGG	=	Aggregate interlock stiffness factor,
k	=	Coefficient of subgrade reaction, psi/in.,
h_{PCC}	=	PCC slab thickness, in.,
ζ_d	=	Dowel stiffness factor = $J_d*k*l*dsp$,
d	=	Dowel diameter, in.,
dsp	=	Dowel spacing, in.,
J_d	=	Non-dimensional dowel stiffness at the time of load application, and
l	=	Radius of relative stiffness, in.

The loss of shear capacity (Δ_s) due to repeated wheel load applications is characterized in terms of the width of the transverse joint based on a function derived from the analysis of load transfer test data developed by the Portland Cement Association (PCA). The following loss of shear occurs during the time increment (month):

$$\Delta s = \begin{cases} 0 & if; \quad w < 0.001\, h_{PCC} \\ \sum_j \dfrac{0.005}{1.0 + (jw \ / h_{PCC})^{-5.7}} \left(\dfrac{n_j}{10^6}\right)\left(\dfrac{\tau_j}{\tau_{ref}}\right) & if; \quad jw < 3.8\, h_{PCC} \\ \sum_j \dfrac{0.068}{1.0 + 6.0 * (jw \ / h_{PCC} - 3)^{-1.98}} \left(\dfrac{n_j}{10^6}\right)\left(\dfrac{\tau_j}{\tau_{ref}}\right) & if; \quad jw > 3.8 h_{PCC} \end{cases} \tag{5-24a}$$

where:

n_j	=	Number of applied load applications for the current increment by load group *j*,
w	=	Joint opening, mils (0.001 in.), and
τ_j	=	Shear stress on the transverse crack from the response model for the load group *j*, psi.

$$\tau_j = \frac{AGG * (\delta_{loaded} - \delta_{unloaded})}{h_{PCC}} \tag{5-24b}$$

τ_{ref}	=	Reference shear stress derived from the PCA test results, psi,
τ_{ref}	=	$111.1 * exp\{-exp[0.9988 * exp(-0.1089 \log J_{AGG})]\}$, and (5-24c)
J_{AGG}	=	Joint stiffness on the transverse crack computed for the time increment.

The dowel damage, DAM_{dow} is determined as follows:

$$DAM_{dow} = C_8 \sum_j \left(\frac{J_d * (\delta_{loaded} - \delta_{unloaded}) * DowelSpace}{d\, f_c^{'}} \right) \tag{5-24d}$$

where:

DAM_{dow}	=	Damage at dowel-concrete interface,
C_8	=	Coefficient equal to 400,
n_j	=	Number of load applications for the current increment by load group *j*,
J_d	=	Non-dimensional dowel stiffness at the time of load application,
δ_L	=	Deflection at the corner of the loaded slab induced by the axle, in.,
δ_U	=	Deflection at the corner of the unloaded slab induced by the axle, in.,
dsp	=	Space between adjacent dowels in the wheel path, in.,
f'_c	=	PCC compressive strength, psi, and
d	=	Dowel diameter, in.

Using Eq. 5-20b, the faulting increment developed using the current month is determined. The magnitude of the increment depends on the level of maximum faulting, level of faulting at the beginning of the month, and total differential energy, *DE*, accumulated for a month from all axle loads passed from 8:00 p.m. to 8:00 a.m. Using Eq. 5-20a, the faulting at the end of the current month is determined. These steps are repeated for the number of months in the pavement design life.

More than one-third of the sections used to calibrate this prediction model were non-doweled. The dowel diameter in the remaining sections varied from 1 to 1.625 in. A plot of measured versus predicted mean transverse joint faulting based on the global calibration exercise is shown in Figures 5-11 through 5-13. The standard error for the transverse joint faulting global prediction equation is shown in Eq. 5-25.

$$s_{e(F)} = [0.0097 * Fault(t)]^{0.5178} + 0.014 \tag{5-25}$$

where:

$Fault\ (t)$ = Predicted mean transverse joint faulting at any given time *t*, in.

5.4.3 CRCP Punchouts

The following globally calibrated model predicts CRCP punchouts as a function of accumulated fatigue damage due to top-down stresses in the transverse direction:

$$PO = \frac{A_{PO}}{1 + \alpha_{PO} \cdot DI_{PO}^{\beta_{PO}}} \tag{5-26}$$

where:

PO = Total predicted number of medium and high-severity punchouts/mi,

DI_{PO} = Accumulated fatigue damage (due to slab bending in the transverse direction) at the end of y^{th} yr, and

$A_{PO}, \alpha_{PO}, \beta_{PO}$ = Calibration constants (216.8421, 33.15789, –0.58947, respectively).

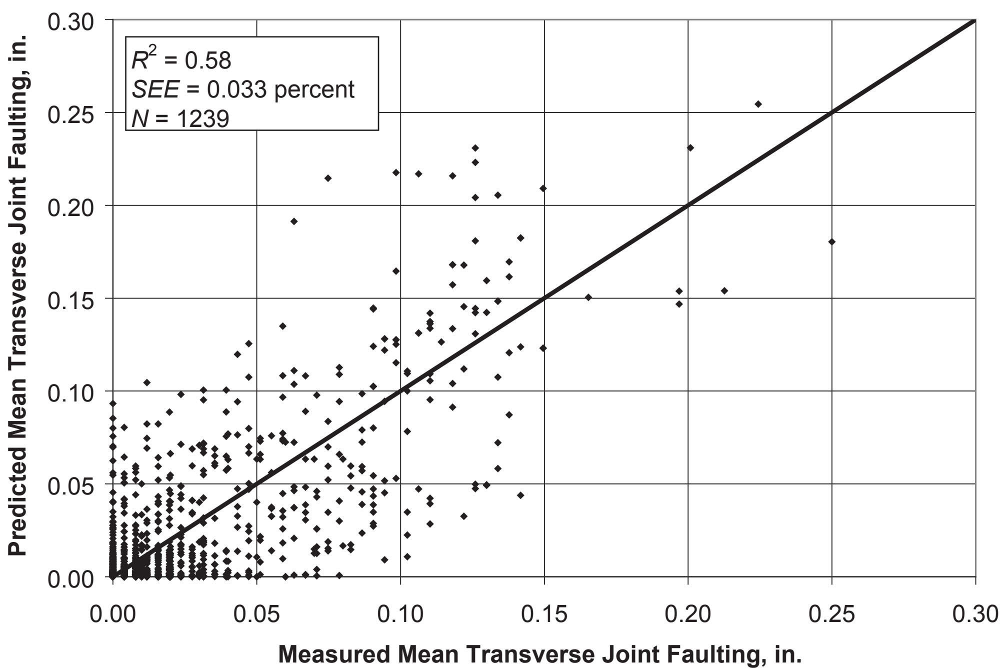

Figure 5-11. Comparison of Measured and Predicted Transverse Joint Faulting for New JPCP Resulting from Global Calibration Process

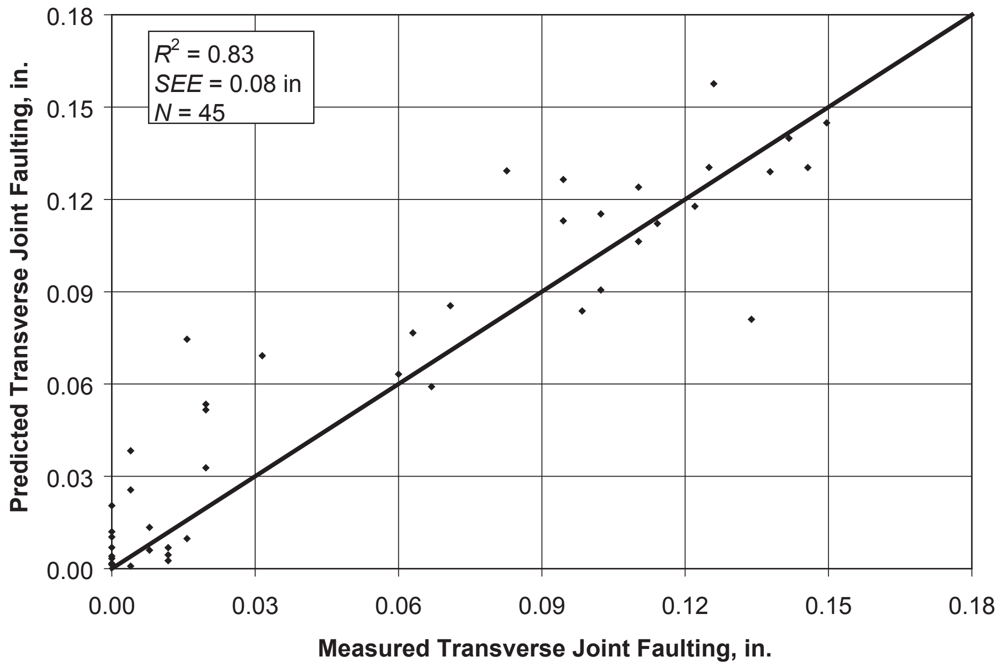

Figure 5-12. Comparison of Measured and Predicted Transverse Joint Faulting for Unbound JPCP Overlays Resulting from Global Calibration Process

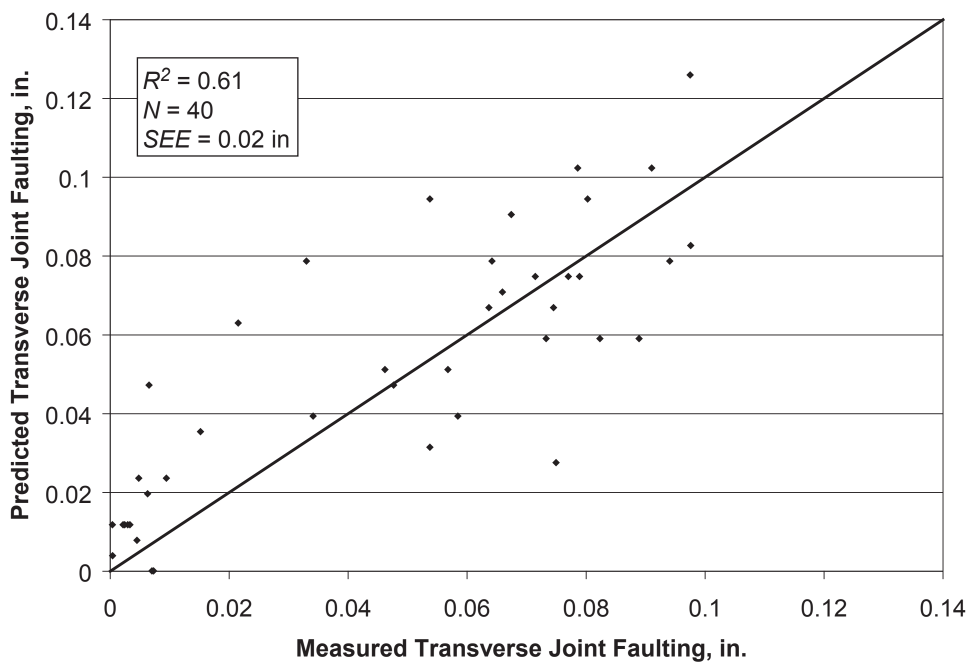

Figure 5-13. Comparison of Measured and Predicted Transverse Joint Faulting for Restored (Diamond Grinding) JPCP Resulting from Global Calibration Process

Section 11.2.3, CRCP Design, identifies the more important factors that affect the number of punchouts and crack spacing, which determine the overall performance of CRCP. The mean crack spacing for the selected trial design and time of construction is calculated in accordance with Eq. 5-27.

$$\overline{L} = \frac{\left[f_t - B_{curl}\sigma_0 \left(1 - \frac{2D_{steel}}{H_{PCC}} \right) \right]}{\frac{f}{2} + \frac{U_m P_{steel}}{c_1 d_b}} \tag{5-27}$$

where:

$\overline{L}$	=	Mean transverse crack spacing, in.,
f_t	=	Concrete indirect tensile strength, psi,
f	=	Base friction coefficient,
U_m	=	Peak bond stress, psi,
P_{steel}	=	Percent longitudinal steel,
d_b	=	Reinforcing steel bar diameter, in.,
c_1	=	First bond stress coefficient,
σ_{env}	=	Tensile stress in the PCC due to environmental curling, psi,
H_{PCC}	=	Slab thickness, in.,
D_{steel}	=	Depth to steel layer, in.,
B_{curl}	=	Bradbury's curling/warping stress coefficient, and
σ_0	=	Westergaard's nominal stress factor based on PCC modulus, Poisson's ratio, and unrestrained curling and warping strain.

The damage accumulated at the critical point on top of the slab is calculated for each time increment of the design life. Damage is calculated in the following manner:

- For the given time increment calculate crack width at the level of steel as a function of drying shrinkage, thermal contraction, and the restraint from reinforcing steel and base friction:

$$cw = Max\left[L \cdot \left(\varepsilon_{shr} + \alpha_{PCC}\Delta T_{\varsigma} - \frac{c_2 f_{\sigma}}{E_{PCC}} \right) \cdot 1000 \cdot C_C \cdot 0.001 \right] \tag{5-28}$$

where:

cw	=	Average crack width at the depth of the steel, mils,
L	=	Mean crack spacing based on design crack distribution, in.,
ε_{shr}	=	Unrestrained concrete drying shrinkage at steel depth, $\times 10^{-6}$,
α_{PCC}	=	PCC coefficient of thermal expansion, /°F,
ΔT_{ζ}	=	Drop in PCC temperature from the concrete "zero-stress" temperature at the depth of the steel for construction month, °F,
c_2	=	Second bond stress coefficient,
σ_{Long}	=	Maximum longitudinal tensile stress in PCC at steel level, psi,
E_{PCC}	=	PCC elastic modulus, psi, and
C_C	=	Local calibration constant (C_C = 1 for the global calibration).

- For the given time increment calculate shear capacity, crack stiffness, and LTE across transverse cracks. LTE is determined as:

$$LTE_{TOT} = 100 * \left(1 - \left(1 - \frac{1}{1 + \log^{-1}\left[(0.214 - 0.183\frac{a}{l} - \log(J_c) - r_d) / 1.18 \right]} \right) \left(1 - \frac{LTE_{Base}}{100} \right) \right) \tag{5-29}$$

where:

LTE_{TOT}	=	Total crack LTE due to aggregate interlock, steel reinforcement, and base support, %,
l	=	Radius of relative stiffness computed for time increment i, in.,
a	=	Radius for a loaded area, in.,
r_d	=	Residual dowel-action factor to account for residual load transfer provided by the steel reinforcement = $2.5P_{steel} - 1.25$,
LTE_{Base}	=	Base layer contribution to the LTE across transverse crack, percent. Typical values were given in Table 5-3,
J_c	=	Joint stiffness on the transverse crack for current time increment, and
P_{steel}	=	Percent steel reinforcement.

- The loss of support for the given time increment is calculated using the base erosion model in AASHTOWare Pavement ME Design. This loss of support is a function of base type, quality of base material, precipitation, and age.
- For each load level in each gear configuration or axle-load spectra, the tensile stress on top of slab is used to calculate the number of allowable load repetitions, $N_{i,j}$, due to this load level in this time increment as:

$$\log\left(N_{i,j}\right) = C_1 * \left(\frac{M_{R_i}}{\sigma_{i,j}} \right)^{C_2} - 1 \tag{5-30}$$

where:

M_{Ri}	=	PCC modulus of rupture at age i, psi, and
$\sigma_{i,j}$	=	Applied stress at time increment i due to load magnitude j, psi.
$C_{1,2}$	=	Calibration constants; $C_1 = 2.0$, $C_2 = 1.22$.

- The loss in shear capacity and loss in load transfer is calculated at end of time increment in order to estimate these parameters for the next time increment. The crack LTE is output monthly for evaluation. A minimum of 90–95 percent is considered good LTE over the design period.

The critical stress at the top of the slab which is transverse and located near a transverse crack was found to be 40 to 60 in. from the edge (48 in. was used, since this was often the critical location). A crack spacing of 2 ft was used as the critical width after observations that a very high percentage of punchouts were 2 ft or less. This stress is calculated using the neural net models, which are a function of slab thickness, traffic offset from edge, PCC properties, base course properties and thickness, subgrade stiffness, equivalent temperature gradient, and other factors. Fatigue damage, *FD*, due to all wheel loads in all time increments is accumulated according to Miner's damage hypothesis by summing the damage over design life in accordance with Eq. 5-17a. Once damage is estimated using Eq. 5-17a, the corresponding punchouts is computed using the globally calibrated Eq. 5-26.

A plot of measured versus predicted CRCP punchouts and statistics from the global calibration is shown in Figure 5-14. The standard error for the CRCP punchouts prediction model is shown in Eq. 5-31.

$$s_{e(PO)} = 2 + 2.2593 * PO^{0.4882} \tag{5-31}$$

where:

PO = Predicted mean medium- and high-severity punchouts, no./mi

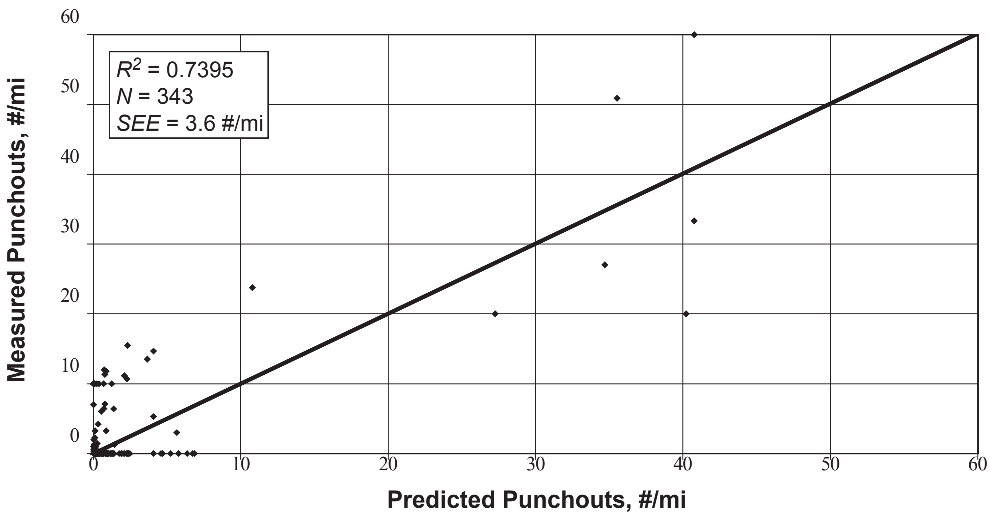

Figure 5-14. Comparison of Measured and Predicted Punchouts for New CRCP Resulting from Global Calibration Process

5.4.4 Smoothness—JPCP

In AASHTOWare Pavement ME Design, smoothness is predicted as a function of the initial as-constructed profile of the pavement and any change in the longitudinal profile over time and traffic due to distresses and foundation movements. The IRI model was calibrated and validated using LTPP field data to assure that it would produce valid results under a variety of climatic and field conditions. The following is the final calibrated model:

$$IRI = IRI_I + C1*CRK + C2*SPALL + C3*TFAULT + C4*SF \tag{5-32a}$$

where:

IRI = Predicted IRI, in./mi,
IRI_I = Initial smoothness measured as IRI, in./mi,
CRK = Percent slabs with transverse cracks (all severities),
$SPALL$ = Percentage of joints with spalling (medium and high severities),
$TFAULT$ = Total joint faulting cumulated per mi, in., and
$C1$ = 0. 8203.

$C2$ = 0.4417
$C3$ = 1.4929
$C4$ = 25.24
SF = Site factor

$$SF = AGE\,(1+0.5556*FI)\,(1+P_{200})*10^{-6} \tag{5-32b}$$

where:
AGE = Pavement age, yr,
FI = Freezing index, °F-days, and
P_{200} = Percent subgrade material passing No. 200 sieve.

The transverse cracking and faulting are obtained using the models described earlier. The transverse joint spalling is determined in accordance with Eq. 5-33a, which was calibrated using LTPP and other data.

$$\mathrm{SPALL} = \left[\frac{\mathrm{AGE}}{\mathrm{AGE}+0.01}\right]\left[\frac{100}{1+1.005^{(-12*\mathrm{AGE}+\mathrm{SCF})}}\right] \tag{5-33a}$$

where:
$SPALL$ = Percentage joints spalled (medium- and high-severities),
AGE = Pavement age since construction, yr, and
SCF = Scaling factor based on site-, design-, and climate-related.

$$\begin{aligned} SCF = -1400 + 350 \cdot AC_{PCC} \cdot (0.5 + PREFORM) + 43.4\, f'c^{\,0.4} \\ - 0.2\,(FT_{cycle} \cdot AGE) + 43\, H_{PCC} - 536\, WC_{PCC} \end{aligned} \tag{5-33b}$$

AC_{PCC} = PCC air content, %,
AGE = Time since construction, yr,
$PREFORM$= 1 if preformed sealant is present; 0 if not,
$f'c$ = PCC compressive strength, psi,
FT_{cycles} = Average annual number of freeze-thaw cycles,
H_{PCC} = PCC slab thickness, in., and
WC_{PCC} = PCC water/cement ratio.

Model statistics for Eq. 5-33b are listed below:
R^2 = 78%
SEE = 6.8%
N = 179

A plot of measured versus predicted IRI values (smoothness) for new JPCP and the statistics from the global calibration is shown in Figure 5-15. The standard error for the initial JPCP IRI is 5.4 (in./mi). The equation for the standard error of predicted mean JPCP is shown in Eq. 5-34.

$$s_e JCPC\_IRI\_model = 29.03\ln(\text{IRI}) - 103.8 \tag{5-34}$$

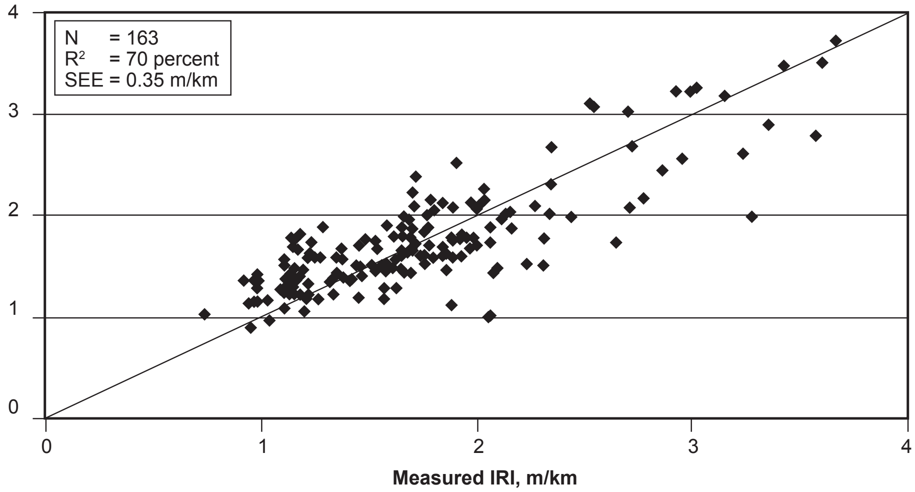

Figure 5-15. Comparison of Measured and Predicted IRI Values for New JPCP Resulting from Global Calibration Process

5.4.5 Smoothness—CRCP

Smoothness change in CRCP is the result of a combination of the initial as-constructed profile of the pavement and any change in the longitudinal profile over time and traffic due to the development of distresses and foundation movements. Key distresses affecting the IRI for CRCP include punchouts. The global IRI model for CRCP is given as follows:

$$IRI = IRI_I + C_1 \bullet PO + C_2 \bullet SF \tag{5-35a}$$

where:

IRI_I	=	Initial IRI, in./mi,
PO	=	Number of medium- and high-severity punchouts/mi,
C_1	=	3.15,
C_2	=	28.35, and
SF	=	Site factor.

$$\text{SF}=AGE \bullet (1 + 0.556\, FI) \bullet (1 + P_{200})*10^{-6} \tag{5-35b}$$

where:

AGE	=	Pavement age, yr,
FI	=	Freezing index, °F days, and
P_{200}	=	Percent subgrade material passing No. 200 sieve.

A plot of measured versus predicted IRI values for new CRCP and the statistics from the global calibration process is shown in Figure 5-16. The standard error for the initial CRCP IRI is 5.4 (in./mi). The equation for the standard error of predicted mean CRCP is shown in Eq. 5-36.

$$s_e CRCP\_IRI\_model = 7.08\ln(IRI) - 11 \tag{5-36}$$

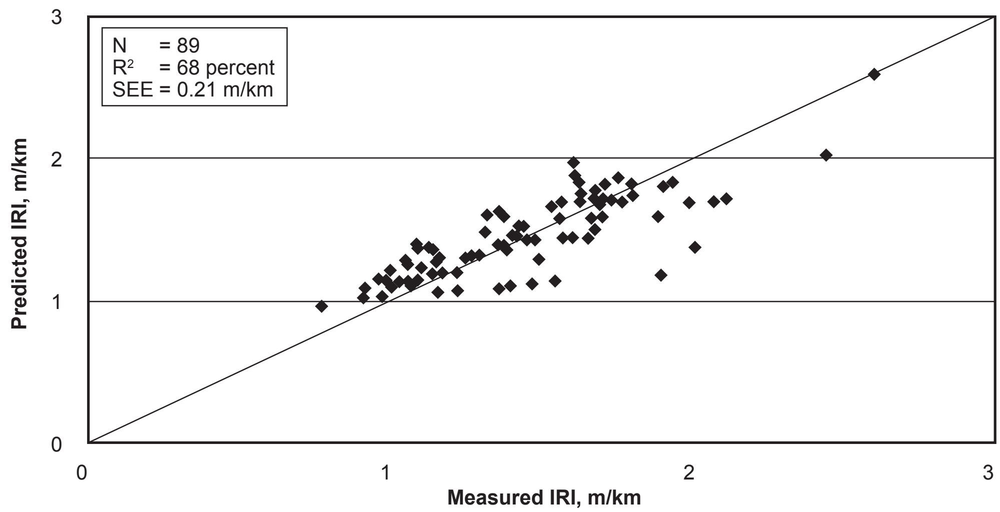

Figure 5-16. Comparison of Measured and Predicted IRI Values for New CRCP Resulting from Global Calibration Process

CHAPTER 6

General Project Information

6.1 DESIGN/ANALYSIS LIFE

As noted under the definition of terms (Section 4.1), the design life of a new or reconstructed pavement is the time from initial construction until the pavement has structurally deteriorated to a specified pavement condition—the time when significant rehabilitation or reconstruction is needed. The design life of an overlay or Concrete Pavement Restoration (CPR) is the time from when the overlay is placed, or CPR performed until significant rehabilitation or reconstruction is needed. AASHTOWare Pavement ME Design can handle design lives from 1 year (e.g., temporary travel way or a detour) to over 50 years. The use of 50+ years design life is considered a long-life pavement.

At this time, durability issues and material disintegration resulting from surface distresses are not predicted with AASHTOWare Pavement ME Design. It should be noted that material disintegration distresses will limit the expected pavement service life to less than the predicted number. Additionally, few pavements included in the global calibration of the software's models had exceeded 30 years of performance data. Thus, the designer should recognize the importance of adequate material and construction specifications (especially for the surface layer) for design periods exceeding 30 years. In addition, it is key for a designer to recognize which distress predictions relate to structural issues and can be addressed by preventative maintenance techniques to extend the life of the pavement.

6.2 CONSTRUCTION AND TRAFFIC OPENING DATES

Pavement construction dates establish the starting point for analysis in the individual distress prediction models. These dates were defined in Section 4.1 and are keyed to the monthly traffic loadings and monthly climatic inputs which affect all monthly layer and subgrade modulus values, (including aging of HMA and PCC).

The designer selects the most likely month and year, based on the probable project schedule, for construction completion of the unbound (subgrade) layer, placement of the base layers, and opening the roadway to traffic. For large projects that extend into multiple sequential paving seasons, each paving season could be evaluated separately. For example, there maybe portions of a project that are opened to traffic in the spring, through summer, and then autumn. It is suggested that each be evaluated separately to better judge the acceptability of the trial design based on the more conservative one.

AASHTOWare Pavement ME Design also has the functionality to simulate an unbound aggregate base layer being left exposed for an extended period of time prior to placing the first HMA layer in flexible pavement design. When and if this condition is permitted, the user may evaluate its effect on short- and long-term pavement performance predictions.

For concrete pavements, the traffic opening affects the curing time (28 days as been established as the minimum for this design procedure), which in turn affects concrete strength and modulus.

AASHTOWare Pavement ME Design does not consider construction traffic in the computation of the incremental damage. Construction traffic is assumed to be nil relative to the design life of the pavement structure. However, construction loading, if excessive, may cause damage to pavements in their early life. This assumption is believed to be reasonable for new pavement and rehabilitation projects.

CHAPTER 7

Selecting Design Criteria and Reliability Level

Design performance criteria and design reliability greatly affect deterioration of an adequately performing pavement. Chapter 5 summarized all of the performance indicators that are predicted with AASHTOWare Pavement ME Design for both HMA- and PCC-surfaced pavements. Guidance is provided within this section for selecting the design criteria and reliability for a particular project. Each user may consider these recommendations and modify them according to their experience, agency policies, and local needs.

The design criteria and design reliability levels could be selected in balance with each other. A low level of distress should not be selected in conjunction with a high level of reliability because this may make it impossible or costly to obtain an adequate design. These levels could become policy values that are usually fixed for routine designs.

7.1 RECOMMENDED DESIGN-PERFORMANCE CRITERIA

Performance criteria are used to ensure that a pavement design will perform satisfactorily over its design life. The designer select performance threshold distress values to judge the adequacy of a trial design. These performance threshold values should reflect agency policies regarding the pavement condition at which some type of major rehabilitation activity or reconstruction is required. These limits include consideration of materials properties (e.g., thermal cracking in flexible pavements or cracking in rigid pavements), structural distress (e.g., fatigue cracking in flexible pavements or faulting in rigid pavements), and functional distress such as IRI.

It is recommended that the distress and IRI design criteria be selected by visualizing the pavement condition over time and its impact on safety. Other considerations include maintenance needs (e.g., amount of lane closures), effort required to rehabilitate the pavement nearing its terminal condition, and the realization that the criteria are set at a given level of design reliability (e.g., 90 percent).

Performance threshold values may also be determined from an analysis of the agency's pavement management database. This could be done either by conducting survivability analyses (e.g., conditions when major rehabilitation activities are undertaken) or by considering users and safety reasons (e.g., a rut depth threshold that reduces the probability of hydroplaning). The consequences of a project exceeding a particular performance criterion could likely require maintenance or rehabilitation activities earlier than programmed. Table 7-1 provides values for considerations by highway agencies, realizing that these levels may vary between agencies based on their specific conditions.

Table 7-1. AASHTOWare Pavement ME Design—Design Criteria or Threshold Values Recommended for Use in Judging the Acceptability of a Trial Design

Pavement Type	Performance Criteria	Threshold Value at End of Design Life
HMA pavement and overlays	Alligator cracking (HMA bottom up cracking)	Interstate:10% lane area Primary: 20% lane area Secondary: 35% lane area
	Rut depth (permanent deformation in wheel paths)	Interstate: 0.40 in. Primary: 0.50 in. Others (<45 mph): 0.65 in.
	Transverse cracking length (thermal cracks)	Interstate: 500 ft./mi Primary: 700 ft./mi Secondary: 700 ft./mi
	IRI (smoothness)	Interstate: 160 in./mi Primary: 200 in./mi Secondary: 200 in./mi
JPCP new, CPR, and overlays	Mean joint faulting	Interstate: 0.15 in. Primary: 0.20 in. Secondary: 0.25 in.
	Percent transverse slab cracking	Interstate: 10% Primary: 15% Secondary: 20%
	IRI (smoothness)	Interstate: 160 in./mi Primary: 200 in./mi Secondary: 200 in./mi

7.2 RELIABILITY

Reliability has been incorporated in AASHTOWare Pavement ME Design in a consistent and uniform fashion for all pavement types. A designer may specify the desired level of reliability for each distress type and smoothness. The level of design reliability could be based on the general consequence of reaching the terminal condition earlier than the design life. Design reliability levels selected may vary by distress type and IRI or may remain constant for each. Reliability could be selected based on the type of distress and the standard error of the distress prediction model. In all cases engineering judgment and experience should be used when selecting a particular reliability value. Since reliability can significantly impact the pavement predictions, it is advisable that all stakeholders are consulted before selecting a value(s). Design reliability (R) is defined as the probability (P) that the predicted distress will be less than the critical level over the design period.

$$R = P\ [Distress\ over\ Design\ Period < Critical\ Distress\ Level] \tag{7-1a}$$

A distinction is made for smoothness which is a cumulative function of other individual distresses. Design reliability is defined as follows for smoothness (IRI):

$$R = P\ [IRI\ over\ Design\ Period < Critical\ IRI\ Level] \tag{7-1b}$$

An example follows that attempts to help describe the reliability definition. If 10 projects were designed and constructed using AASHTOWare Pavement ME Design and each had a design reliability for fatigue cracking of 90 percent, one of those projects, on average, would show more than the threshold or terminal value of fatigue cracking at the end of the design period. For example, a design reliability of 90 percent (mean faulting) represents the probability (9 out of 10 projects) that the mean faulting for the project will not exceed the faulting criteria. The reliability of a particular design analyzed by AASHTOWare Pavement ME Design is dependent on the standard errors of the transfer functions.

The designer inputs critical or threshold values for each predicted distress type and IRI. The software accompanying the MEPDG procedure predicts the mean distress types and smoothness over the design life of the pavement, as illustrated by the solid line in Figure 7-1 for IRI. This prediction is based on average values for all inputs. The distresses and smoothness predicted therefore represent mean values that may be thought of as being at a 50 percent reliability estimate at the end of the analysis period (i.e., there is a 50 percent chance that the predicted distress or IRI will be greater than or less than the mean prediction).

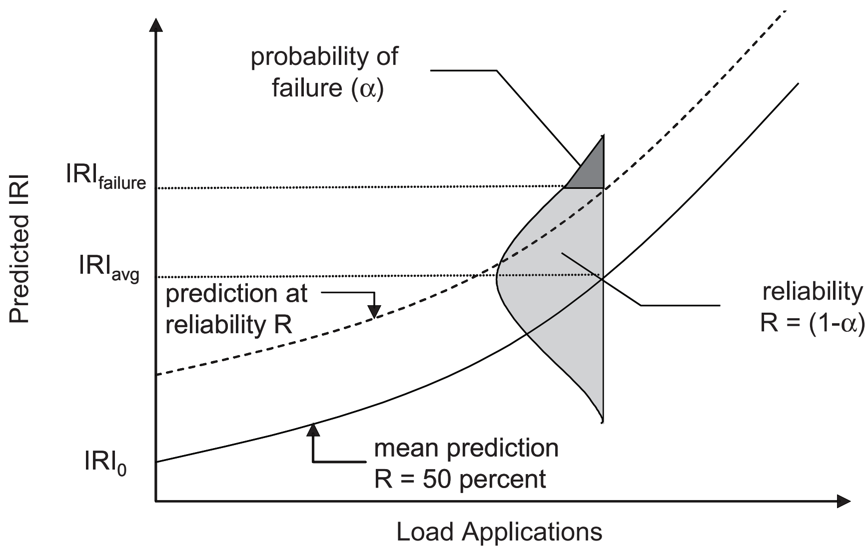

Figure 7-1. Design Reliability Concept for Smoothness (IRI)

In practice, the designer will likely require a reliability higher than 50 percent that the design will meet the performance criteria over the design life. Agencies can select different design reliability for each distress type based on the effect of early failure of each distress. In order to design a feasible pavement structure, higher reliability is usually assigned to distresses which are more critical and more difficult to repair.

The dashed curve in Figure 7-1 shows the prediction at a level of reliability, *R* (e.g., 90 percent). For the design to be at least 90 percent reliable the dashed curve at reliability *R* should not cross the IRI at the criteria throughout the design analysis period.

AASHTOWare Pavement ME Design software calculates the reliability of the trial section relative to the design criteria or threshold values selected by the user. The reliability of the trial design is dependent on the model prediction error (standard error) of the distress prediction equations, provided in Section 5. In summary, the mean distress or IRI value (50 percent reliability) is increased by the number of standard errors that apply to the reliability level selected. For example, a 75 percent reliability uses a factor of 0.674 times the standard error, a 90 percent reliability uses a factor of 1.282, and a 95 percent reliability uses a value of 1.645.

The calculated distresses and IRI are assumed to be approximately normally distributed over ranges of the distress and IRI that are of interest in design. As noted above, the standard deviation for each distress type was determined from the model prediction error from calibration results used for each key distress. Each model was calibrated from LTPP and other field performance data. For example, the error of prediction of, say, rutting was obtained as the difference of predicted and measured rutting results for all sections of the pavement sections included in the calibration efforts. This difference, or residual error, contains all available information on the ways in which the prediction model fails to properly explain the observed rutting. The standard deviation of IRI was determined using a closed form variance model estimation approach.

The calculated reliability values are output to a table of the reliability of the trial design at the end of the design period showing the mean prediction, the prediction at *R* percent, and the estimated reliability of the design for each distress and IRI.

The design reliability could be selected in balance with the performance criteria desired. For example, the selection of a high-design reliability value (e.g., 99 percent) and a low-performance criterion (3 percent alligator cracking) might make it not feasible to build. The selection of a high reliability (e.g., >96 percent) is not recommended at the present time, because this may increase construction costs too much. Table 7-2 provides values that are believed to be in balance with the performance criteria included in Table 7-1 and are suggested for use in design. It is recommended that each agency evaluate these values and adjust them to meet their needs. Each distress should be evaluated independently for the reliability level set in the design analysis. Reliability values recommended for use in previous *AASHTO Guide for Design of Pavement Structures* versions should not be used with AASHTOWare Pavement ME Design.

Table 7-2. Suggested Minimum Levels of Reliability for Different Functional Classifications of the Roadway

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

CHAPTER 8

Determining Site Conditions and Factors

This section identifies and presents the site factors needed for each trial design—truck traffic, climate, foundation, and condition of existing pavement (for rehabilitation design) inputs.

8.1 TRUCK TRAFFIC

Truck traffic is a key data element for the structural design/analysis of pavement structures. AASHTOWare Pavement ME Design uses the full axle-load spectrum data for each axle type for both new pavement and rehabilitation design procedures. Traffic volume, lane distribution, volume adjustment factors (i.e., class distribution, traffic growth factors, etc.) and weight data are used as inputs along with some miscellaneous data such as tire pressure. More details regarding the traffic related inputs are included in Section 8.1.1.

The axle-load spectra are obtained from processing weighing-in-motion (WIM) data. Tables 8-1 and 8-2 provide recommendations for the minimum sample size to estimate the normalized axle-load distributions and truck-volume distribution. In addition, the *FHWA Traffic Monitoring Guide* (*10*) and NCHRP Report 538 provide guidance on collecting and analyzing truck weight data (*6*).

The axle-weight and truck-volume data require detailed and extensive processing to determine the numerous truck traffic related inputs to AASHTOWare Pavement ME Design. AASHTOWare Pavement ME Design provides several default traffic-related values to be used when the designer has no access to that data.

Default values were determined from an analysis of nearly 200 WIM sites included in the LTPP program, and significantly simplify use of AASHTOWare Pavement ME Design related to truck traffic. These default values are included in the AASHTOWare Pavement ME Design software, and were determined from WIM data collected on predominantly Interstate highways and primary arterials.

The following sections provide guidance for estimating the truck traffic inputs used for evaluating the adequacy of a design strategy. For rehabilitation and realignment projects, the designer could request any WIM data collected within the project limits. If WIM data are unavailable, the designer could request the installation of portable WIM devices to measure truck traffic characteristics over the short-term, as a minimum. If the installation of WIM devices is not possible, the following is suggested for determining the truck traffic inputs.

Table 8-1. Minimum Sample Size (Number of Days per Year) to Estimate the Normalized Axle-Load Distribution—WIM Data

Standard Error (±%)	Level of Confidence or Significance, %				
	80	90	95	97.5	99
20	1	1	1	1	1
10	1	1	2	2	3
5	2	3	5	7	10
2	8	19	30	43	61
1	32	74	122	172	242

Table 8-2. Minimum Sample Size (Number of Days per Season) to Estimate the Normalized Truck Traffic Distribution—Automated Vehicle Classifier (AVC) Data

Standard Error (±%)	Level of Confidence or Significance, %				
	80	90	95	97.5	99
20	1	1	1	2	2
10	1	2	3	5	6
5	3	8	12	17	24
2	20	45	74	105	148
1	78	180	295	Note 1	Note 1

Note:
1. Continuous sampling is required for these conditions.
2. If the difference between weekday and weekend truck volumes is required, the number of days per season should be measured on both the weekdays and weekends.
3. A season in this table is based on changing truck patterns to define the normalized truck volume distribution at the specified level of confidence and standard error. This season is not the same as used in AASHTOWare Pavement ME Design software for calculating pavement responses and incremental damage values.

- For rehabilitation or realignment projects, the truck traffic data may be estimated using WIM and AVC sites that are located on nearby segments of the highway, assuming that there are no features or major intersections that could change the truck traffic stream. The inputs determined from this type data are considered Level 1.
- If there are no WIM sites located along the same segment of highway or for new roadway construction projects, WIM and AVC data from other similar roadways located within the same region may be used. The designer may contact the agency's traffic and planning departments to identify the WIM and AVC sites that may be used to estimate the truck traffic inputs for the project location. The inputs determined from this type data are considered Level 2.
- If no WIM sites are available from similar roadways, the defaults included in AASHTOWare Pavement ME Design software may be used (Level 3 inputs).

The remainder of Section 8.1 is divided into three parts; determining roadway specific inputs, determining the truck traffic inputs that may be extracted from WIM data, and estimating the inputs not recorded in the WIM data.

8.1.1 Roadway-Specific Inputs

The following input parameters are considered site-specific and need to be obtained from the traffic or planning department.

- **Initial Two-Way Average Annual Daily Truck Traffic (AADTT)**—AADTT has a significant effect on the predicted pavement performance indicators and represents a weighted average between weekday and weekend truck traffic. AADTT may be obtained from WIM data, automated vehicle counters, or manual traffic counts. The value entered into AASHTOWare Pavement ME Design software is the AADTT after the roadway is opened to traffic or the rehabilitation has been completed. In addition, the user should ensure that the value entered represents both directions and all lanes. If one-way truck traffic is entered, the percent trucks in the design direction should be set to 100 percent.
- **Percent Trucks in Design Lane**—The percent of truck in the design lane typically is determined by estimating the percentage of truck traffic in the design lane relative to all truck traffic in one direction. However, the definition used in AASHTOWare Pavement ME Design is slightly different; it is defined by the primary truck class for the roadway. The primary truck class represents the truck class with the majority of applications using the roadway. In other words, the percentage of trucks in the design lane is estimated for each truck class, and the predominant truck class is used to estimate this value. The percent trucks in the design lane may be estimated from AVC data or manual vehicle count data.
- **Percent Trucks in Design Direction**—This value represents the percent of trucks in the design direction relative to all trucks using the roadway in both directions. This value may be estimated from AVC data or manual vehicle count data.
- **Operational Speed**—Truck speed has a definite effect on the predicted E^* of HMA and, thus, distresses. Lower speeds result in higher incremental damage values calculated by AASHTOWare Pavement ME Design (more fatigue cracking and deeper ruts or faulting). The posted speed limit was used in all calibration efforts. As such, it is suggested that the posted truck speed limit be used to evaluate trial designs, unless the pavement is located in a special low-speed area such as a steep upgrade and bus stop.
- **Growth of Truck Traffic**—The growth of truck traffic is difficult to estimate accurately because there are many site and social-economic factors that are difficult, if not impossible, to predict over 20+ years. The traffic and/or planning departments within an agency may be consulted to estimate the increase in truck traffic over time. AASHTOWare Pavement ME Design has the capability to use different growth rates for different truck classes, but assumes that the growth rate is independent over time; in other words the rate of increase remains the same throughout the analysis period. Truck class dependent growth rates have a significant effect of predicted pavement performance and may be determined with as much information as possible about the commodities being transported within and through

the project location. There is also a practical limitation on the amount of traffic volume built into AASHTOWare Pavement ME Design that prevents a designer from overpopulating the lane capacity.

8.1.2 Inputs Extracted from WIM Data

The truck traffic input parameters needed for running AASHTOWare Pavement ME Design software that are recorded in WIM data are listed and defined in this section. As noted above, the NCHRP Project 1-39 software may be used to provide the truck traffic inputs recorded in the WIM data. If the NCHRP Project 1-39 or other software is unavailable, the input traffic files may be created separately that represent each individual window of input data (e.g., axles per truck, monthly adjustment factor, single axle-load distribution). The following also provides guidance on determining the inputs for these values.

- **Axle-Load Distributions** (single, tandem, tridem, and quads)—The axle-load distribution represents a massive amount of data and the data processing should be completed external to AASHTOWare Pavement ME Design software. There are multiple software tools or packages available for processing the axle-load distribution data. These software tools have varying capabilities and functionality, and users may want to evaluate the options so as to select the tool most suitable to their agency needs.
- **Normalized Truck-Volume Distribution**—The average normalized truck-volume distribution is needed when limited WIM data are available to determine the total axle-load distribution for a project. The normalized truck-volume distribution represents the percentage of each truck class within the truck traffic distribution. This normalized distribution is determined from an analysis of AVC data and represent data collected over multiple years. The default normalized truck volume distributions determined from the LTPP sites is included in Table 8-3, as a function of different TTC groups. The TTC index value is used to select an appropriate truck volume distribution for a specific roadway and can be determined from traffic counts and highway functional classifications. Table 8-4 defines the TTC groups included in AASHTOWare Pavement ME Design software for determining the normalized truck volume distribution and normalized axle weight distributions.
- **Axle-Load Configurations** (axle spacing and wheelbase)—The spacing of the axles is recorded in the WIM database. These values have been found to be relatively constant for the standard truck classes. The values used in all calibration efforts are listed below and suggested for use, unless the predominant truck class has a different axle configuration.
  - Tandem axle spacing; 51.6 in.
  - Tridem axle spacing; 49.2 in.
  - Quad axle spacing; 49.2 in.

Table 8-3. TTC Group Description and Corresponding Truck Class Distribution Default Values Included in AASHTOWare Pavement ME Design Software

TTC Group and Description		Truck Class Distribution (%)									
		4	5	6	7	8	9	10	11	12	13
1	Major single-trailer truck route (type I)	1.3	8.5	2.8	0.3	7.6	74.0	1.2	3.4	0.6	0.3
2	Major single-trailer truck route (type II)	2.4	14.1	4.5	0.7	7.9	66.3	1.4	2.2	0.3	0.2
3	Major single-trailer truck route (type I)	0.9	11.6	3.6	0.2	6.7	62.0	4.8	2.6	1.4	6.2
4	Major single-trailer truck route (type III)	2.4	22.7	5.7	1.4	8.1	55.5	1.7	2.2	0.2	0.4
5	Major single and multi-trailer truck route (type II)	0.9	14.2	3.5	0.6	6.9	54.0	5.0	2.7	1.2	11.0
6	Intermediate light and single trailer truck route (type I)	2.8	31.0	7.3	0.8	9.3	44.8	2.3	1.0	0.4	0.3
7	Major mixed truck route (type I)	1.0	23.8	4.2	0.5	10.2	42.2	5.8	2.6	1.3	8.4
8	Major multi-trailer truck route (type I)	1.7	19.3	4.6	0.9	6.7	44.8	6.0	2.6	1.6	11.8
9	Intermediate light and single-trailer truck route (type II)	3.3	34.0	11.7	1.6	9.9	36.2	1.0	1.8	0.2	0.3
10	Major mixed truck route (type II)	0.8	30.8	6.9	0.1	7.8	37.5	3.7	1.2	4.5	6.7
11	Major multi-trailer truck route (type II)	1.8	24.6	7.6	0.5	5.0	31.3	9.8	0.8	3.3	15.3
12	Intermediate light and single-trailer truck route (type III)	3.9	40.8	11.7	1.5	12.2	25.0	2.7	0.6	0.3	1.3
13	Major mixed truck route (type III)	0.8	33.6	6.2	0.1	7.9	26.0	10.5	1.4	3.2	10.3
14	Major light truck route (type I)	2.9	56.9	10.4	3.7	9.2	15.3	0.6	0.3	0.4	0.3
15	Major light truck route (type II)	1.8	56.5	8.5	1.8	6.2	14.1	5.4	0.0	0.0	5.7
16	Major light and multi-trailer truck route	1.3	48.4	10.8	1.9	6.7	13.4	4.3	0.5	0.1	12.6
17	Major bus route	36.2	14.6	13.4	0.5	14.6	17.8	0.5	0.8	0.1	1.5

Monthly Distribution Factors—The monthly distribution factors are used to distribute the truck traffic within each class throughout the year. Monthly distribution factors of 1.0 were used for all truck classes during all calibration efforts. The reason for using values of 1.0 is that most of the calibration sites were located along the interstate system or along primary arterials, and no significant seasonal changes in the truck traffic operations were found. For more local routes, seasonal changes in truck traffic operations could be expected. These monthly distribution factors may be determined from WIM, AVC, or manual truck traffic counts.

Hourly Distribution Factors—The hourly distribution factors are used to distribute the total truck traffic throughout a typical day. The hourly distribution factors may be estimated from WIM, AVC, or manual truck traffic counts. Average default values were determined from an analysis of the LTPP WIM data. Hourly distribution factors are only required for the analysis of rigid pavements, which keys hourly truck volume to temperature gradients through the PCC slab. The flexible pavement analysis bases all computations related to temperature on a monthly basis. Refer to the discussion in Section 5.3.1 for flexible pavements.

Table 8-4. Definitions and Descriptions for the TTC Groups

Buses in Traffic Stream	Commodities Being Transported by Type of Truck: Multi-Trailer	Commodities Being Transported by Type of Truck: Single-Trailer and Single Unit Trucks	TTC Group No.
Low to None (<2%)	Relatively high amount of multi-trailer trucks (>10%)	Predominantly single-trailer trucks	5
		High percentage of single-trailer trucks, but some single-unit trucks	8
		Mixed truck traffic with a higher percentage of single-trailer trucks	11
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	13
		Predominantly single-unit trucks	16
	Moderate amount of Multi-Trailer Trucks (2 to 10%)	Predominantly single-unit trucks	3
		Mixed truck traffic with a higher percentage of single-trailer trucks	7
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	10
		Predominantly single-unit trucks	15
Low to Moderate (>2%)	Low to None (<2%)	Predominantly single-unit trucks	1
		Predominantly single-trailer trucks, but with a low percentage of single-unit trucks	2
		Predominantly single-trailer trucks with a low to moderate amount of single-unit trucks	4
		Mixed truck traffic with a higher percentage of single-trailer trucks	6
		Mixed truck traffic with about equal percentages of single-unit and single-trailer trucks	9
		Mixed truck traffic with a higher percentage of single-unit trucks	12
		Predominantly single-unit trucks	14
Major Bus Route (>25%)	Low to None (<2%)	Mixed truck traffic with about equal single-unit and single-trailer trucks	17

8.1.3 Truck Traffic Inputs Not Included in the WIM Data

The truck traffic input parameters needed for running AASHTOWare Pavement ME Design software that are not recorded in the WIM data are listed and defined in this section. The following lists those input parameters and provides guidance on determining the inputs for these values.

- **Dual Tire Spacing**—AASHTOWare Pavement ME Design software assumes that all standard truck axles included in the WIM data contain dual tires. The dual tire spacing represents the majority of trucks using the roadway and taken from trucking industry standards. The default value of 12 in. was selected based on the spacing of the tires used by most trucks. It is recommended that this default value be used unless the predominant type of truck has special loading conditions. The use of super-single tires or single tires may be simulated in AASHTOWare Pavement ME Design software by increasing the dual tire spacing to a value where the influence from one of the dual tires becomes insignificant to the other. This distance between the dual tires for this to occur is 60 in. for most cases.

- **Tire Pressure**—AASHTOWare Pavement ME Design software assumes a constant tire pressure for all loading conditions that represents operating condition (hot inflation tire pressure). A median value of 120 psi was used in all calibration efforts. It is recommended that this value be used, unless hot inflation pressures are known from previous studies or a special loading condition is simulated.
- **Lateral Wander of Axle Loads**—AASHTOWare Pavement ME Design software assumes a constant wander for all trucks. A value of 10 in. was used for all calibration efforts, independent of the lane width. In some urban areas, narrower lane widths have been built because of right-of-way width restrictions. For narrow lane widths (less than 10 ft) it is recommended that a lower lateral wander value be used; a value of 8 in. is suggested unless the user has measured this value. Similarly, for wide lanes (greater than 12 ft) it is recommended that a higher lateral wander value be used; a value of 12 in. is suggested unless the user has measured this value.
- **Truck Wheel Base—**The wheel base of truck tractors is the distance from the front steering axle to the next axle. Due to truck tractor length variation, this has been divided into short (12 ft), medium (15 ft), and long (18 ft) categories. National averages of the percentages of trucks that fit into these categories are 17 percent short, 22 percent medium, and 61 percent long.

8.2 CLIMATE

Detailed climatic data are required for predicting pavement distress with AASHTOWare Pavement ME Design and include hourly temperature, precipitation, wind speed, relative humidity, and cloud cover. These data are used to predict the temperature and moisture content in each of the pavement layers, as well as provide some of the inputs to the site factor parameter for the smoothness prediction models.

All of the climate data needed by AASHTOWare Pavement ME Design are available from weather stations, generally located at airfields around the United States. AASHTOWare Pavement ME Design has an extensive number of weather stations embedded in its software for ease of use and implementation. The user simply needs to know the longitude and latitude of the project and the software will automatically select six weather stations closest to that location. The longitude, latitude, elevation, and number of months of available data are viewed by the user in selecting the weather stations to be used by the software to create a virtual weather station at the project location for the distress predictions.

Multiple weather stations could be selected to provide the climatic data needed by AASHTOWare Pavement ME Design. The weather stations selected by the user are used to calculate a virtual weather station for the project location. Multiple weather stations are recommended because of the possibility of missing data and errors in the database for an individual station. Missing data and errors from a single weather station could cause AASHTOWare Pavement ME Design software to hang-up or crash in the climatic module, if used alone. The weather stations selected to create the virtual weather station for the project site should have similar elevations, if possible, although temperatures are adjusted for elevation differences.

It is recommended that highway agencies that span a wide range of climatic conditions divide into similar climatic zones (approximately the same ambient temperature and moisture) and identify representative weather stations for each of these zones. It is important to understand that proximity is not the best indicator of similar weather conditions. In order to attain a more accurate analysis, it is recommended to create a weather station by importing a new climatic file created with locally collected climatic data.

The depth to the water table is another climate input parameter, and is discussed in the next section.

8.3 FOUNDATION AND SUBGRADE SOILS

8.3.1 Subsurface Investigations for Pavement Design

The horizontal and vertical variations in subsurface soil types, moisture contents, densities, water table depth, and location of rock strata need to be considered during the pavement design process. Swelling soils, frost susceptible soils, and water flow need to be identified and considered in pavement design, because of their detrimental effect on pavement performance. When problem soils are found along a project, they need to be dealt with external to AASHTOWare Pavement ME Design because the program does not predict volume change potential (AASHTO R 13). Chapter 11 provides some guidance on selecting different options to minimize the effects of volume change on pavement performance.

The subsurface investigation (number of borings drilled) needs to define the depth, thickness, and location of the major soil and rock strata that may reduce the pavement's service life and determine the need for foundation improvements and strengthening. The steps involved in a subsurface investigation are summarized below.

Prepare a boring layout and sampling plan to determine the vertical and horizontal profile of the subsurface soils. Soil Conservation Service Series maps may be used in planning the subsurface investigation, and in estimating the location of and number of borings. These maps show the different types of subsurface soils in an area on a county-wide basis and may be obtained from libraries or the geotechnical department in most state highway agencies.

Conduct a topographic and subsurface investigation, and take sufficient samples (undisturbed and bulk samples) for laboratory testing. Thin-walled tube samples need to be taken in accordance with AASHTO T 207 whenever possible to recover undisturbed samples for density determination and resilient modulus testing. Recovering soils with thin-walled tubes, however, is not always possible. For soils where undisturbed samples cannot be recovered during the site investigation, auger or split-barrel sampling methods need to be used (AASHTO T 206). The designer may input seasonal water table depths, if sufficient data has been collected at the site. It is recommended that one depth be used unless field measurements or historical data dictate seasonal values.

Field logs need to be prepared and used in setting up the laboratory testing plan. AASHTO R 13 or an equivalent procedure may be used as a guide in preparing the field logs.

Perform field tests to measure the in-place properties of the subsurface soil strata. Different tests may be used to estimate the in-place stiffness, such as the California Bearing Ratio (CBR, AASHTO T 193). However, use of the dynamic cone penetrometer (DCP) also provides an estimate of the in-place modulus of the existing soil strata. DCP tests need to be performed in accordance with ASTM D 6951 or an equivalent procedure. The field tests and their use will be discussed under the next section.

- Prepare soil borings summarizing the results from the investigation. The borings may note the depth and thickness of the different soil layers, depth to a rigid layer or rock strata, the depth to a water table or wet soil layers, and usual conditions that will affect pavement construction and performance. The depth to the water table is an important input because AASHTOWare Pavement ME Design has the capability, through the use of the EICM, to estimate changes in the resilient modulus of the aggregate layers and foundation soils over time. For most pavement designs, water table depths greater than 20 ft below the planned surface elevation will have a minimal effect of the pavement distress predictions.
- A laboratory test program needs to be planned based on results from the subsurface investigation.

8.3.2 Laboratory and Field Tests of Soils for Pavement Design

A program of laboratory and field tests could be used to determine the properties of the foundation. The properties of the soil that are needed for design are discussed in Chapter 10, while the type of treatment used to improve the foundation is provided in Chapter 11. The test program may be grouped into measuring three basic properties; classification tests, volumetric tests, and strength or stiffness tests. Each is summarized below.

- Classification tests are used to determine the volume change potential, frost susceptibility, and drainage potential of the foundation soils. Table 8-5 provides a summary of the soil characteristics. Classification tests include sieve analysis or gradation and Atterberg limits, and need to be performed on each major soil strata encountered during the subsurface investigation. Classification tests may be performed in accordance with ASTM D 2487 or an equivalent procedure to classify the soil strata. AASHTO M 145 is a standard practice that may be used to classify all soils and soil-aggregate mixtures for highway construction. Results from the classification tests and Table 8-5 may be used to determine the types of improvements to the foundation to reduce the effect of problem soils, if present.
- Volumetric tests (dry density and moisture content) need to be performed on undisturbed samples recovered from soil strata that will not be removed or reworked. If undisturbed samples cannot be obtained, moisture contents need to be measured on disturbed samples recovered during the drilling operation in accordance with AASHTO T 265.
- The modulus of the in-place foundation soils (not to be removed or reworked during construction) is an important input, especially for new flexible pavement designs. The resilient modulus of the in-place subgrade soils may be estimated from the DCP, physical properties of the soil strata, or measured in the laboratory using AASHTO T 307 or the procedure recommended in NCHRP Project 1-28A (31). Section 10.5 provides guidance on determining the design resilient modulus.

8.4 EXISTING PAVEMENTS

The condition of the existing surface is estimated from the distress measurements (condition surveys), from coring and materials testing, and from backcalculated elastic modulus. Chapter 9 provides guidance for determining the condition of the existing pavement layers for use in rehabilitation design.

Table 8-5. Summary of Soil Characteristics as a Pavement Material

Major Divisions	Name	Strength When Not Subject to Frost Action	Potential Frost Action	Compressibility and Expansion	Drainage Characteristics
Gravel and Gravelly Soils	Well-graded gravels or gravel–sand mixes, little to no fines; GW	Excellent	None to very slight	Almost none	Excellent
	Poorly graded gravels or gravel–sand mixes little or no fines; GP	Good to excellent	None to very slight	Almost none	Excellent
	Silty gravels, gravel–sand silt mixes; GM	Good to excellent	Slight to medium	Very slight	Fair to poor
	Very Silty gravels, gravel–sand silt mixes; GM	Good	Slight to medium	Slight	Poor to practically impervious
	Clayey gravels, gravel–sand–clay mixes; GC	Good	Slight to medium	Slight	Poor to practically impervious
Sand and Sandy Soils	Well-graded sands or gravelly sands, little to no fines; SW	Good	None to very slight	Almost none	Excellent
	Poorly graded sands or gravelly sands. Little or no fines; SP	Fair to good	None to very slight	Almost none	Excellent
	Silty sands, sand–silt mixes; SP	Fair to good	Slight to high	Very slight	Fair to poor
	Silty sands, sand–silt mixes; SM	Fair	Slight to high	Slight to medium	Poor to practically impervious
	Clayey sands, sand–clay mixes; SC	Poor to fair	Slight to high	Slight to medium	Poor to practically impervious
Silts and Clays with the Liquid Limit Less Than 50	Inorganic silts and very fine sand, rock flour, silty or clayey fine sand or clayey silts with slight plasticity	Poor to fair	Medium to very high	Slight to medium	Fair to poor
	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays. silty clays, lean clays	Poor to Fair	Medium to high	Slight to medium	Practically Impervious
	Organic silts and organic silt–clays or low plasticity	Poor	Medium to high	Medium to high	Poor
Silts and Clays with Liquid Limit Greater Than 50	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts	Poor	Medium to very high	High	Fair to Poor
	Inorganic clays of high plasticity, fat clays	Poor to fair	Medium to very high	High	Practically Impervious
	Organic clays of medium to high plasticity, organic silts	Poor to very poor	Medium	High	Practically Impervious
Highly Organic Soils	Peat and other highly organic soils	Not Suitable	Slight	Very high	Fair to poor

Note: The information presented in this table is adopted after publications of the U.S. Army Corps of Engineers, Federal Aviation Administration, and the Federal Highway Administration.

CHAPTER 9

Pavement Evaluation for Rehabilitation Design

Rehabilitation design requires an evaluation of the existing pavement to provide key information. This section provides detailed and specific guidance for conducting a pavement evaluation program and taking the results from that program to establish inputs to the AASHTOWare Pavement ME Design software.

It is important to note that the MEPDG inputs of existing pavement layers for overlay design are similar to those required for new or reconstructed pavements except that the values may differ due to deterioration of the existing layers and materials. Determining the extent of damage and material properties of the in-place layers is the most critical challenge in pavement evaluation. The following section presents general information on assessing practical conditions for pavement rehabilitation design. Specific details on inputs necessary for conducting pavement rehabilitation design will be noted throughout the chapter. In addition, the test protocols for measuring the material properties are listed in Chapter 10.

9.1 OVERALL CONDITION ASSESSMENT AND PROBLEM DEFINITION CATEGORIES

The first step in the pavement rehabilitation design process involves assessing the overall condition of the existing pavement and fully defining the existing pavement problems. It is recommended that agencies collect and evaluate sufficient information about the existing pavement to minimize the chances of under- or over-designing the rehabilitated structure. Some high-speed, non-destructive testing data, such as pavement surface profile testing and GPR are viable tools that may assist in making decisions related to timing of the improvement and whether more detailed data collection efforts are needed. Overall pavement condition of existing pavement is organized in AASHTOWare Pavement ME Design in the following eight major categories:

1. Structural adequacy (load related).
2. Functional adequacy (as rated by the roadway user).
3. Subsurface drainage adequacy.
4. Material durability.
5. Shoulder structural profile and condition.

6. Extent of maintenance activities performed in the past.
7. Variation of pavement condition or performance within a project.
8. Miscellaneous constraints (e.g., bridge and lateral clearance and traffic control restrictions).

Some of the categories are interrelated, e.g., structural and material durability categories are tied to features that define the pavement responses to traffic loads. This data is used in AASHTOWare Pavement ME Design for rehabilitation alternatives. The functional category relates to the surface and subsurface properties that define the smoothness of the roadway, and 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 important in terms of rehabilitation type selection and in affecting project construction cost.

Variation within a project refers to areas where there is a significant difference 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 reflective cracking in composite pavements, are important to the overall condition of such pavements but only need to be evaluated where relevant.

Table 9-1 contains a comprehensive checklist of factors designed to identify the problems that need to be addressed during rehabilitation design. The following provides some guidance on the amount of work or extensiveness of the pavement evaluation plan for determining the input values related to the condition of the existing pavement layers.

- If the pavement has significant and extensive levels of distress that exceed the user's failure criteria or threshold values, extensive field and laboratory testing to characterize the pavement surface layers becomes less important. The condition of the existing pavement may be determined from results of the visual distress surveys. (For example, if an assessment of the pavement's condition reveals that it has over 50 percent high-severity, load-related cracks, an attempt to accurately estimate the modulus and volumetric properties of the existing HMA layer would not be cost effective for selecting and designing rehabilitation strategies). A better strategy in this case would be to assess the conditions of the underlying unbound layers.
- If the pavement has marginal levels of distress, the results from the visual distress survey may be used to determine the location and frequency of field tests and core extraction. In this case, both methods of assessment are equally effective.
- If the pavement has exhibited no structural distress, field (deflection basin and DCP tests) and laboratory testing are recommended to more accurately determine the remaining structural capacity of the existing pavement layers.
- If the pavement has marginal levels of distress, the results from the visual distress survey may be used to determine the location and frequency of the field tests and cores. In this case, both assessments become equally important.

The remainder of this section provides a summary of those pavement evaluation activities to determine the existing pavement condition for rehabilitation design with AASHTOWare Pavement ME Design.

Table 9-1. Checklist of Factors for Overall Pavement Condition Assessment and Problem Definition

Facet	Factors	Description
Structural Adequacy	Existing Distress	1. Little or no load/fatigue-related distress 2. Moderate load/fatigue-related distress (possible deficiency in load-carrying capacity) 3. Major load/fatigue-related distress (obvious deficiency in current load-carrying capacity) 4. Load-carrying capacity deficiency: (yes or no)
	Nondestructive testing (FWD deflection testing)	1. High deflections or weak layers: (yes or no) 2. Are backcalculated layer moduli reasonable? 3. Are joint load transfer efficiencies reasonable?
	Nondestructive testing (GPR, Pspa testing, and SASW)	1. Determine layer thickness 2. Are voids located beneath PCC pavements?
	Nondestructive testing (profile testing)	Determine joint/crack faulting
	Destructive testing	1. Adequate core strength and condition? 2. Adequate layer thicknesses?
	Previous maintenance performed	Minor | Normal | Major
	Has lack of maintenance contributed to structural deterioration?	Yes____ No____ Describe ________________________
Functional Adequacy	Smoothness:	Measurement Very Good | Good | Fair | Poor | Very Poor
	Cause of smoothness deficiency:	Foundation movement Localized distress or deterioration Other
	Noise	Measurement Satisfactory | Questionable | Unsatisfactory
	Friction resistance	Measurement Satisfactory | Questionable | Unsatisfactory
Subsurface Drainage	Climate (moisture and temperature region)	Moisture throughout the year: Seasonal moisture or high water table Very little moisture Deep frost penetration Freeze-thaw cycles No frost problems
	Presence of moisture-accelerated distress	Yes | Possible | No
	Subsurface drainage facilities	Satisfactory | Marginal | Unsatisfactory
	Surface drainage facilities	Satisfactory | Marginal | Unsatisfactory
	Has lack of maintenance contributed to deterioration of drainage facilities?	Yes | No Describe: ________________________

Table 9-1 continued on the next page.

Table 9-1. Checklist of Factors for Overall Pavement Condition Assessment and Problem Definition —*continued*

Facet	Factors	Description		
Materials Durability	Presence of durability-related distress (surface layer)	1. Little to not durability-related distress. 2. Moderate durability-related distress 3. Major durability-related distress		
	Base erosion or stripping	1. Little or no base erosion or stripping 2. Moderate base erosion or stripping 3. Major base erosion or stripping		
	Nondestructive testing (GPR testing)	Determine areas with material deterioration/moisture damage (stripping)		
Shoulder Adequacy	Surface condition	1. Little or not load-associated/joint distress 2. Moderate load-associated/joint distress 3. Major load-associated/joint distress 4. Structural load-carrying capacity deficiency: (yes or no)		
	Localized deteriorated areas	Yes	No	Location:
Condition-Performance Variability	Does the project section include significant deterioration of the following: • Bridge approaches • Intersections • Lane-to-lane • Cuts and fills	Yes	No	
	Is there a systematic variation in pavement condition along project (localized variation)?	Yes	No	
	Systematic lane to lane variation in pavement condition	Yes	No	
Miscellaneous	PCC joint damage: • Is there adequate load transfer (transverse joints)? • Is there adequate load transfer (centerline joint)? • Is there excessive centerline joint width? • Is there adequate load transfer (lane-shoulder)? • Is there joint seal damage? • Is there excessive joint spalling (transverse)? • Is there excessive joint spalling (longitudinal)? • Has there been any blowups?	Yes	No	
Constraints	Are detours available for rehabilitation construction?	Yes	No	
	Should construction be accomplished under traffic	Yes	No	
	Can construction be done during off-peak hours	Yes	No	
	Bridge clearance problems?	Yes	No	
	Lateral obstruction problems	Yes	No	
	Utility problems/issues	Yes	No	
	Other constraint problems	Yes	No	

9.2 DATA COLLECTION TO DEFINE CONDITION ASSESSMENT

This section summarizes the steps and activities to complete a detailed assessment on the condition of the existing pavement for selecting a proper rehabilitation strategy, as shown in Figure 9-1. All steps to complete a detailed assessment of the pavement and individual layers are not always needed. Table 9-2 lists the input levels associated with setting up and conducting a pavement evaluation plan in support of AASHTOWare Pavement ME Design.

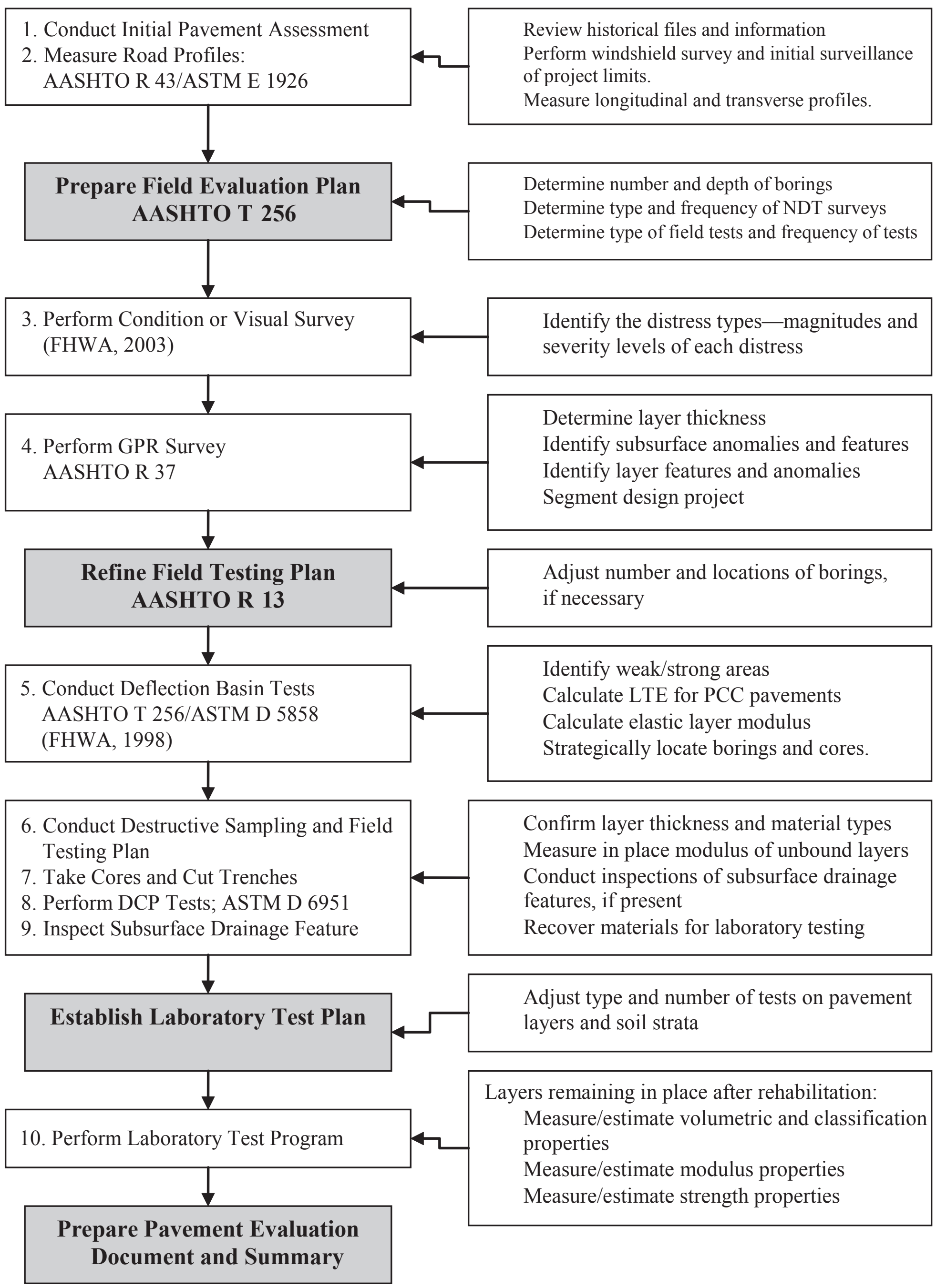

Figure 9-1. Steps and Activities for Assessing Condition of Existing Pavements for Rehabilitation Design (Refer to Table 9-2)

Table 9-2. Hierarchical Input Levels for a Pavement Evaluation Program to Determine Inputs for Existing Pavement Layers for Rehabilitation Design Using AASHTOWare Pavement ME Design

Assessment Activity	Input Level for Pavement Rehabilitation Design			Purpose of Activity
	1	2	3	
1. Initial Assessment: Review files and historical information, conduct windshield survey.	Yes	Yes	Yes	Estimate the overall structural adequacy and materials durability of existing pavement, segment project into similar condition of: Existing layers Shoulders, if present Drainage features (surface and subsurface) Identify potential rehabilitation strategies
2. Surface Feature Surveys: Measure profile, noise, and friction of existing surface.	Yes, Only Profile	Yes, Only Profile	No	Determine functional adequacy of surface; Profile, friction and noise surveys are only needed to determine if rehabilitation is needed, because the surface will usually be replaced or modified. Profile surveys are used to select a proper rehabilitation strategy—milling depth or diamond grinding, leveling course thickness, or none needed; estimate the initial IRI value after HMA overlay; and CPR appropriateness.
3. Detailed Condition Survey: Determine type, amount, and severity of existing distresses	Yes	Yes	No	Estimate structural adequacy or remaining life and materials durability of existing pavement layers and to select a rehabilitation strategy. Distortion; faulting of PCC and rutting in HMA Cracking; non-load related cracks versus fatigue cracks Material disintegration distresses (raveling, D-cracking, etc.) Define/segment areas with different distresses
4. GPR Survey: Estimate layer thickness, locate subsurface anomalies and features	Yes	No	No	Determine structural adequacy, subsurface features and anomalies, and materials durability of existing pavement layers: Estimate layer thickness Identify potential subsurface anomalies Locate voids beneath pavement surface Locate HMA layers with stripping SASW
5. Deflection Basin Tests: Measure load-response of pavement structure and foundation	Yes	Yes	No	Determine structural adequacy and in-place modulus of existing pavement layers and foundation. Calculate LTE of cracks and joints in PCC pavements Calculate layer modulus of all lifts Locate borings and cores for destructive tests Level 2—Uniform spacing of deflection basin tests in areas with different distresses. Level 1— Clustered spacing of deflection basin tests in areas with different distresses along entire project.

Table 9-2 continued on next page.

Table 9.2. Hierarchical Input Levels for a Pavement Evaluation Program to Determine Inputs for Existing Pavement Layers for Rehabilitation Design Using the AASHTOWare Pavement ME Design —*continued*

Assessment Activity	Input Level for Pavement Rehabilitation Design			Purpose of Activity
	1	2	3	
6. Destructive Sampling: Cores extracted and soil borings taken to recover materials for visual observation and lab testing	Yes	Yes	Yes	Determine structural adequacy and materials durability. Visual classification of materials and soils Confirm layer thickness and material types Identify/confirm subsurface anomalies—HMA stripping, voids, etc. Determine depth to rigid layer or bedrock Determine water table depth Identify seams with lateral water flow Level 3—Limited borings in the areas identified from the initial pavement assessment activity. Levels 1 and 2— Boring and cores drilled in each segment identified from the condition survey, deflection basin tests, SASW and GPR survey.
7. Field Inspections: Cores and trenches in distressed areas	Yes	No	No	Structural adequacy and rehabilitation strategy selection: Determine the rutting in each paving layer from the excavated trenches. Determine where cracking initiated and the direction of crack propagation.
8. Field Tests: DCP tests of unbound layers	Yes	No	No	Determine structural adequacy—estimate the in-place modulus from DCP tests performed on the unbound layer through the core locations.
9. Field Inspections: Subsurface drainage features	Yes	No	No	Subsurface drainage adequacy—Inspecting drainage features with mini-cameras to check condition of and ensure positive drainage of edge drains.
10. Laboratory Tests: Unbound materials and soils, HMA mixtures, and PCC mixtures	Yes	Yes	No	Layers which will remain in place after rehabilitation: Classification tests (gradation and Atterberg limits tests) Unit weight and moisture content tests Coefficient of thermal expansion—PCC Strength tests—PCC and HMA layers Modulus tests—PCC layers only Level 3—All inputs based on defaults and visual classification of materials and soils; no laboratory tests are performed on layers that will remain in place. Level 2—Modulus estimated from DCP and deflection basin tests for unbound layers and volumertric properties for bound layers. Layer 1—Laboratory tests listed above.

9.2.1 Initial Pavement Assessment

Regardless of the input level adopted for the pavement evaluation, the condition assessment needs to begin with an assembly of historic data. This information is obtained from a windshield pavement condition field survey of the entire project followed by a detailed survey of selected areas of the project. The following activities should be performed to assist in preparing the field evaluation plan.

- Review historical records for the roadway segment planned for rehabilitation. The information needed includes the original pavement construction month and year (a required input to AASHTOWare Pavement ME Design), and any preventive maintenance, pavement preservation, or repair activities that have been applied to the roadway segment. The preventive maintenance, pavement preservation, and repair activities are only needed to assist the designer in establishing the condition of the existing pavement and help explain performance anomalies.

- Review construction files and results from previous borings and laboratory results, if available. The Soil Conservation Service Series maps may also be used to ensure that the different subsurface soils along the project are sampled and tested, if needed. These maps were identified and discussed in Chapter 8 on characterizing the foundation soils for new alignments.
- Review previous distress and profile surveys and pavement management records to establish performance trends and deterioration rates, if available.
- Review previous deflection surveys, if available.
- Perform a cursory pavement condition survey or complete a windshield inspection of the roadway's surface, cross-sectional and drainage features, and other related items. This initial survey consists of photo logs, low-aerial photographs, and automated distress surveys.
- Group segments of the roadway together that have similar layer thickness, surface distresses, subsurface features, and foundation soils.

As part of the initial condition assessment or the more detailed condition survey (see Section 9.2.3), longitudinal and transverse profiles may be measured and used to decide on the types of pre-overlay treatments that might be needed.

9.2.2 Prepare Field Evaluation Plan

It is recommended that an engineer prepares an evaluation plan that outlines all activities needed for investigating and determining the causes of the pavement defects. The plan should include damage observed during the initial surveillance and for selecting and designing an appropriate repair strategy for those defects. The field evaluation plan could consist of a detailed pavement condition survey, nondestructive testing, destructive sampling and testing, and traffic control, as a minimum. Table 9-3 may be used as an example in setting up the field evaluation plan.

9.2.3 Conduct Condition or Visual Survey

A key factor to determine the condition or strength of the existing pavement layers is the result from a detailed pavement condition index survey. Pavement visual surveys are performed to identify the types, locations, and severities of distress. The survey should be performed on the pavement, shoulders and on any drainage feature along the project site. Automated distress surveys are adequate for rehabilitation design purposes.

Table 9-4 provides a summary of the visual survey data needed for determining the inputs to AASHTOWare Pavement ME Design software related to the condition of the existing pavement. For AASHTOWare Pavement ME Design, distress identification for flexible, rigid, and composite pavements is based on the *Distress Identification Manual* for the LTPP program (9). The approach in the LTPP manual was used to identify and measure the distresses for all pavement segments that were included in the global calibration process of the AASHTOWare Pavement ME Design software.

Table 9-3. Field Data Collection and Evaluation Plan

Step	Title	Description
1	Historic data collection	This step involves collecting of information such as location of the project, year constructed, year and type of major maintenance, pavement design features, materials and soils properties, traffic climate, conditions, and any available performance data.
2	First field survey	This step involves conducting a windshield and detailed distress survey of sampled areas within the project to assess the pavement condition. Data required includes distress information, drainage conditions, subjective smoothness, traffic control options, and safety considerations.
3	First data evaluation and the determination of additional data requirements	This step requires determining critical levels of distress/ smoothness and the causes of distress and smoothness loss using information collected during the first field survey. This list will aid in assessing preliminarily existing pavement condition and potential problems. Additional data needs will also be addressed during this step.
4	Second field survey	This step involves conducting detailed measuring and testing such as coring and sampling, profile (smoothness) measurement, skid-resistance measurement, deflection testing, drainage tests, and measuring vertical clearances.
5	Laboratory testing of samples	This step involves conducting tests such as materials strength, resilient modulus permeability, moisture content, composition, density, and gradations, using samples obtained form the second field survey.
6	Second data evaluation	This step involves the determination of existing pavement condition and an overall problem definition. Condition will be assessed and the overall problem defined by assessing the structural, functional, and subsurface drainage adequacy of the existing pavement. Condition assessment and overall problem definition also involve determining material durability, shoulder condition, variability in pavement condition along project, and potential constraints. Additional data requirements for designing rehabilitation alternatives will also be determined during this step.
7	Final field and office data compilation	Preparation of a final evaluation report.

Table 9-4. Guidelines for Obtaining Non-Materials Input Data for Pavement Rehabilitation

Existing Pavement Layer	Design Input	Measurements and Tests Required for Design Inputs
Flexible pavement	Alligator cracks (bottom-up) cracking plus previous repair of this distress	Level 1 and 2: Conduct visual survey along design lane of project and measure area of all severities of alligator fatigue cracking plus any previous repair of this cracking. Compute percent area affected (cracked and repair).
	Rutting of each layer in the existing pavement	Level 1: Measure from transverse trench data across the traffic lane. Level 2 and 3: Proportion the total surface rutting to each layer of the pavement and the subgrade. Utilize cores from the wheel path and non-wheel path to help estimate layer rutting.
	Pavement Rating	Level 3: Pavement Rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).
JPCP concrete slab	Cracked (transverse) slabs in design lane plus previous slab replacements	Conduct visual survey along design lane of project and identify slabs with transverse cracking (all severity levels) and slab replacements of transverse cracks. Compute percent slabs affected (cracked and replacements of cracked slabs).
	Joint load transfer (for reflection cracking prediction with HMA overlay)	Use as-built plans to determine if dowels are present and if so, note their diameter and spacing. Alternatively, conduct FWD testing of joints to determine the load transfer efficiency (LTE). Joint should be rated as having Good LTE when dowels are present. Joint should be rated as having Poor LTE when dowels are not present. When using FWD, a measured LTE of >60% and the temperature is <80°F, the joint should be rated as Good; otherwise, the joint shojuld be rated as having Poor LTE.
	Thickness of slab	Obtain representative cores and measure for thickness. Input mean thickness.
	Joint spacing and skew	Measure joint spacing and skew in the field. If random spacing, measure spacing pattern. If uniform spacing, enter mean spacing. If joints are skewed, add 2-ft to input joint spacing. Cracking is computed for the longest joint spacing but faulting and IRI for mean spacing.
	Shoulder type	Identify shoulder type (next to design lane), and if PCC determine whether or not it is tied to the traffic lane.
	Pavement Rating (Level 3)	Level 3: Pavement rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).
CRCP concrete slab	Punchouts (and repairs of punchouts)	Conduct visual survey along design lane of project and identify number of punchouts at Medium and High levels of severity and full-depth repairs of punchouts. Compute number of punchouts and repairs of punchouts per mile.
	Longitudinal reinforcement	Use as-built plans to determine bar size and spacing and depth from surface. Compute percent reinforcement of concrete area.
	Thickness of slab	Obtain representative cores (or other method) and measure thickness. Input mean thickness.
	Transverse cracking spacing	Conduct a visual survey along design lane of project and determine mean crack spacing. Include all severity levels of transverse cracks.
	Pavement Rating (Level 3)	Level 3: Pavement rating described as: Poor, Fair, Good, Very Good, and Excellent from the windshield survey of the initial assessment (no specific definitions are available).

Some agencies, however, may have to use condition survey data recorded in their pavement management database for establishing the condition of the existing pavements. It is important that consistency be used to identify and measure pavement distresses. Without re-calibrating AASHTOWare Pavement

ME Design to local policies and practices, an agency or designer could use the LTPP *Distress Identification Manual* for determining the surface condition of the existing pavement. The *Standard Practice for Determining the Local Calibration Parameters* (2) addresses the use of condition surveys that have different measures of the distresses and smoothness values included in the LTPP *Distress Identification Manual* and predicted by AASHTOWare Pavement ME Design.

9.2.4 Ground Penetrating Radar Survey

GPR is a well-established, high-speed nondestructive technology used to estimate the thickness of different pavement and soil strata layers, and is frequently used to survey prior to use of destructive sampling. It is possible that GPR may be valuable in reducing the number of cores and borings required for a project, for example by segmenting the project based on similar subsurface features or anomalies identified with this technology prior to drilling the borings. Specifically, dielectric and thickness contours may be prepared along the project to locate areas with different structural features and material conditions. GPR data may be collected at highway speeds so that there is no interference with existing traffic flow.

Consider the use of other types of nondestructive testing such as spectral analysis surface waves (SASW) and seismic analysis, as these tools evolve in practice.

9.2.5 Refine Field Testing Plan

Results from the condition and GPR surveys could be used to strategically designate areas along the project for clustered deflection testing, DCP testing, and sampling the pavement layers and foundation soils. A well-planned field testing plan may minimize the amount of time that the roadway is closed for field activities requiring lane closures. Deflection basin tests, limited DCP tests, and drilling cores and borings can be used to identify pavement sections with different surface distress and dielectric readings to ensure that all areas with different physical features and characteristics are fully investigated.

9.2.6 Conduct Deflection Basin Tests

Agencies are encouraged to measure deflection basins along a project to help select adequate rehabilitation strategies and to provide input for backcalculating layer moduli. The backcalculated layer moduli are helpful in establishing the in-place structural condition of the pavement layers. Table 9-5 lists some of the specific uses of the deflection basin data for eventual inputs to the AASHTOWare Pavement ME Design software.

Table 9-5. Use of Deflection Basin Test Results for Selecting Rehabilitation Strategies and in Estimating Inputs for Rehabilitation Design with AASHTOWare Pavement ME Design

Existing Pavement Layer	Design Input	Measurements and Tests Required for Design Inputs
All types of existing pavements	Deflection or deflection based indexes along the project	Used to select rehabilitation strategies and selection of design sections along project.
HMA	Dynamic modulus, E_{HMA}	Backcalculation of HMA-layer modulus.
PCC	Elastic modulus, E_{PCC}	Backcalculation of PCC-layer modulus.
	Joint (LTE)	Input for determining need for retro fit dowels, and reflection cracking (poor, good)
	Loss of support under corner	Input for determining rehabilitation strategy and repair (subsealing, crack and seat, etc.)
Stabilized base, subbase	Elastic modulus, E_{CTB}	Input for stabilized base or subbase (cement, asphalt, lime, fly ash, etc.).
Unbound materials (base, subbase, subgrade)	Resilient modulus, M_r	Backcalculation of unbound layer and subgrade modulus.

The most widely used deflection testing device is the falling weight deflectometer (FWD). However, the use of seismic testing devices is increasing in popularity and does provide an estimate of the in-place modulus of the pavement layers. Data from both of these types of NDT technologies need to be calibrated to laboratory conditions in providing inputs to the MEPDG procedure. The adjustment to laboratory conditions is discussed in a latter part of this section and in Chapter 10.

Deflection basin tests can be measured with different drop heights to evaluate the load-response characteristics of the pavement structure. Four drop heights are typically used to categorize the pavement structure into three distinct load-response categories; elastic, deflection softening, and deflection hardening. (*20*)

The spacing of the deflection tests will vary along a project. A closer spacing of testing points is suggested in pavements with history of fatigue cracking. In addition, deflection basin tests may be effective in cut and fill areas and in transition areas between cut and fill. Transition areas are where water can accumulate and weaken the underlying soils.

The analysis of deflection basin data measured at different temperatures (morning and evening) may assist in determining the in-place properties of the HMA and assist in evaluating the support conditions of PCC pavements.

For JPCP, deflections are measured at the mid-slab (intact condition), along the transverse joints, and along the edge of the slabs to evaluate the load transfer efficiency and check for voids beneath the PCC layer.

9.2.7 Destructive Sampling and Testing—Recover Cores and Boring for the Existing Pavement

Destructive tests require the physical removal or damage of the pavement layer to observe the condition of the material. Table 9-6 provides a summary of the types of destructive testing and their purposes, the procedures used, and the inputs needed for AASHTOWare Pavement ME Design for rehabilitation design.

Cores and Borings

Cores and borings may be taken from pavement sections where different pavement response characteristics and surface conditions exist. Cores may be used to confirm the layer thicknesses, material types, examine the pavement materials for material durability problems, and collect samples for laboratory tests.

For pavements with excessive rutting (greater than 0.75 in.), trenches may be necessary to determine if the rutting has occurred in the HMA or subsurface layers, in order to select a proper repair strategy. However, trenches are time-consuming and expensive. The engineer should make an assessment of the necessity of taking trenches. Otherwise, cores can be extracted from the wheel path and from the center of the lane in order to determine rutting in each layer and select the appropriate rehabilitation strategy.

Table 9-6. Summary of Destructive Tests, Procedures, and Inputs for the AASHTOWare Pavement ME Design

Destructive Tests	Procedures	Input for AASHTOWare Pavement ME Design
Coring to recover samples for visual inspection and observations and lab testing	Coring and auguring equipment for HMA, PCC, stabilized materials and unbound materials; DCP for unbound layers	Thickness of all layers. HMA durability condition. HMA layer to layer bonding. HMA lab testing for asphalt content, air voids, density, gradation. PCC coefficient of thermal expansion. PCC modulus of elasticity. PCC compressive or IDT strength. Stabilized base compressive strength to estimate the elastic modulus, *E*. PCC to stabilized base bonding. Obtain bulk samples of unbound materials and subgrade for gradation and classification tests. Resilient modulus for the unbound layers.
Test pit	Saw cut rectangular pit to depth of stabilized materials, obtain samples of all materials	Test unbound materials in laboratory for Atterberg limits, gradation, water content. Observe condition of materials in each layer and layer interface bonding. Beam of PCC for flexural strength testing.
Trenching of HMA pavements (see Note)	Two saw cuts far enough apart to remove material with available equipment transversely across traffic lane	Measure permanent deformation at surface and at each interface to determine amount within each layer. Observe condition of HMA, base, and subbase materials and interfaces to see if HMA layers should be partially or completely removed for rehabilitation purposes.
Milling HMA overlay in composite pavement	Mill HMA down to PCC surface at joints	Observe HMA/PCC interface to determine if bond exists and if any stripping of HMA exists. Determine if HMA overlay should be completely removed for rehabilitation purposes. Observe durability of PCC at joint to determine need for repair or replacement.
Removal of PCC at joint	Full depth saw cut on both sides of joint and lift out joint	Examine condition of dowels, durability of PCC, deterioration of base to determine need for joint replacement.

Note: Trenches are expensive and time-consuming. Trenches should only be used in areas where the designer believes that extensive rutting has occurred in the subsurface layers.

In-Place Strength of Individual Unbound Layers

The Dynamic Cone Penetrometer (DCP) may be used in pavement evaluations to measure the strength of unbound layers and materials. It may also be used for estimating soil layer thickness by identifying sudden changes in strength within the pavement structure and foundation. AASHTOWare Pavement ME Design software allows the user to input the DCP test results directly or indirectly depending on the model of choice for converting the raw penetration data into layer moduli. The options include; directly entering the average penetration rate, converting the average penetration rate into a CBR value using locally calibrated models to calculate a CBR value; and then entering that CBR value, or converting the average penetration rate into a resilient modulus using locally calibrated models and then entering that resilient modulus (refer to Tables 9-7 and 9-8).

Table 9-7. Models/Relationships Used for Determining Level 2 *E* or M_r

Chemically Stabilized Material	Recommended Relationships*
Lean concrete[1] Cement treated aggregate[1]	$E = 57000\sqrt{(f'_c)}$ (18) Where, E is the modulus of elasticity, psi; f'_c = compressive strength, psi tested in accordance with AASHTO T 22
Open graded cement stabilized	No correlations are available
Soil cement[2]	$E = 1200 * q_u$ (18) where E, is the modulus of elasticity, psi; q_u = unconfined compressive strength, psi tested in accordance with ASTM D1633, "Standard Test Method for Compressive Strength of Molded Soil–Cement Cylinders"
Lime–cement–flyash[2]	$E = 500 + q_U$ (19) Where, E is the modulus of elasticity, psi; q_u = unconfined compressive strength, psi tested in accordance with ASTM C593 "Standard Specification for Fly Ash and Other Pozzolans for Use with Lime"
Lime stabilized soils[2]	$M_r = 0.124q_U + 9.98$ (17) where, M_r = resilient modulus, ksi, q_u = unconfined compressive strength, psi tested in accordance with ASTM D5102, "Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures"

[1] Compressive strength f_c can be determined using AASHTO T 22.

[2] Unconfined compressive strength q_u can be determined using the MDTP.

Table 9-8. Models Relating Material Index and Strength Properties to M_r

Strength/Index Property	Model	Comments	Test Standard
CBR	$M_r = 2555(\text{CBR})^{0.64}$ (TRL) M_r, psi	CBR = California Bearing Ratio, percent	AASHTO T 193, "The California Bearing Ratio"
R-value	$M_r = 1155 + 555R$ (20) M_r, psi	R = R-value	AASHTO T 190, "Resistance R-Value and Expansion Pressure of Compacted Soils"
AASHTO layer coefficient	$M_r = 3000\left(\frac{a_i}{0.14}\right)$ (20) M_r, psi	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures
PI and gradation*	$\text{CBR} = \left(\frac{75}{1+0.728(\text{wPI})}\right)$	wPI = P200**PI P200 = percent passing No. 200 sieve size PI = plasticity index, percent	AASHTO T 27 "Sieve Analysis of Coarse and Fine-Aggregates" AASHTO T 90, "Determining the Plastic Limit and Plasticity Index of Soils"
DCP*	$\text{CBR} = \left(\frac{292}{\text{DCP}^{1.12}}\right)$	CBR = California Bearing Ratio, percent DCP = DCP index, mm/blow	ASTM D6951, "Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications"

* Estimates of CBR are used to estimate M_r.

Interface Friction Between Bound Layers

Layer interface friction is an input parameter to AASHTOWare Pavement ME Design but is difficult to define and measure. Cores and visual surveys may be used to determine if debonding exists along the project. Slippage cracks and two adjacent layers separating during the coring process may be a result of low interface friction between two HMA layers. If these conditions are found to exist along a project, the designer could consider assuming no bond or a low interface friction during the rehabilitation design using AASHTOWare Pavement ME Design, if those layers are to remain in place and not be milled or removed. All of the global calibration efforts for flexible pavements, however, were completed assuming full friction between all layers—an interface friction value of 1.0 in AASHTOWare Pavement ME Design. This value could be used unless debonding is found. Interface friction values less than 1.0 will increase rutting and cracking of the HMA layers. The decrease in rutting and cracking of HMA is minimal until the condition of full bond, a value of 1, is used. Thus, friction can be defined for just two conditions without significantly affecting the accuracy of the answer; fully bonded (a value of 1.0) or no bond (a value of 0). It should be noted that incomplete bonding is a condition that should be limited and that the use of milling down to a stable layer is recommended in practice.

JPCP allows the user to define the PCC-base contact friction with a simple true/false statement. A statement of false designates no contact friction. A statement of true designates no slippage between layers and requires the user to input "Months until friction loss". Calibration results for new/reconstructed JPCP showed that full contact friction existed over the life of the pavements for all base types, with the exception for CTB or lean concrete where extraordinary efforts were made to debond the layers. For this situation, the months of full contact friction was reduced to a range of 0 to 100 years, with a default value equal to the design life, to match the cracking exhibited. For new and reconstructed PCC designs, full friction should be assumed, unless debonding techniques are specified and confirmed through historical pavement construction records and defaults to 20 years, based on design life.

For rehabilitation of JPCP (CPR and overlays), full contact friction is input over the rehabilitation design life, when cores through the base course show that interface bond exists. Otherwise, the two layers are considered as having zero friction over the design life.

Edge Drains

If the existing pavement has subsurface drains that may remain in place, the outlets need to be found and inspected. Mini-camera may also be used to ensure that the edge drains and lateral lines are free-flowing and not restricting the removal of water from the pavement structure.

9.2.8 Laboratory Tests for Materials Characterization of Existing Pavements

Table 9-6 provided a listing of the materials properties that need to be measured for determining the inputs to AASHTOWare Pavement ME Design relative to the condition of the existing pavement layers. Chapter 10 includes details on the testing of different pavement layers that is required in support of AASHTOWare Pavement ME Design.

It is recommended that a sufficient laboratory test program is established to estimate the material properties of each layer, required as inputs to AASHTOWare Pavement ME Design. The following section lists the type of samples needed for measuring the properties of the in-place layers (refer to Table 9-5).

HMA Mixtures and Layers

- **Volumetric Properties** (air voids, asphalt content, gradation)—Air voids (bulk specific and maximum theoretical specific gravities) of existing layers are obtained from as-built project records and used as input Levels 1 and 2 (Table 9-2). The average effective asphalt content by volume and gradation measured during construction are used for the rehabilitation design. Whenever this volumetric data is unavailable from construction records, selected cores recovered from the project are used to measure these properties. Samples recovered from 6-in.-diameter cores should be used to ensure a sufficient amount of material for gradation tests. The ignition oven is used to measure the asphalt content (in accordance with AASHTO T 308 or an equivalent procedure) and then the gradation is estimated based on the aggregate remaining (in accordance with AASHTO T 27). The HMA density and VMA may be calculated from the HMA bulk specific gravity (AASHTO T 166), maximum theoretical specific gravity (AASHTO T 209), aggregate specific gravity, and asphalt content (refer to Section 10.2).

- **Dynamic Modulus**—Use backcalculated modulus from deflection basin or seismic tests to estimate the amount of damage of the in-place HMA layers. Laboratory dynamic modulus tests are not needed for measuring the in-place modulus because the test needs to be performed on intact, but age-hardened specimens. The resulting modulus values will likely be higher than those for new HMA mixtures, suggesting no damage to the in-place mixture, which may not be the case. Thus, it is recommended that the modulus be determined from the deflection basin tests.
- **Creep Compliance**—Not needed for the existing HMA layers.
- **Indirect Tensile Strength**—The relationship between the IDT modulus and tensile strain at failure may be used to estimate the amount of damage of the in-place HMA layer (*26*). If an HMA layer is believed to have exhibited stripping or some moisture damage, indirect tensile tests could be used to measure the strength, tensile strain at failure, and dynamic modulus of moisture-conditioned and unconditioned specimens of the in-place mixtures to confirm the amount of moisture damage that might be present. If moisture damage is found, this finding could be used in establishing the modulus input values and condition to AASHTOWare Pavement ME Design, if that layer is left in place. If stripping is found near the surface, that layer could be considered for removal in the rehabilitation design.
- **Asphalt Classification**—Extract asphalt from selected cores to determine the performance-grade (PG) of the recovered asphalt (AASHTO M 320). The asphalt classification and volumetric test results are used to determine the undamaged condition of the HMA layer and compare that value to the average backcalculated value in cracked areas to estimate the amount of damage. Extracting the asphalt from existing HMA layers of flexible pavements is expensive, time-consuming, and becoming problematic because of environmental restrictions. For the projects where asphalt is not extracted, historical information and data is used to estimate the PG of the age-hardened asphalt for the lower HMA layers that will remain in place after rehabilitation.

PCC Mixtures and Layers

- **Elastic Modulus of PCC**—Use either the backcalculated modulus values (multiplied by 0.8) to estimate the static modulus, or test for the static modulus of elasticity using a limited number of samples recovered from the coring process. Otherwise, estimate using inputs for flexural strength. The adjustment factor of 0.8 is used to reduce the dynamic modulus value calculated from deflection basin tests to a static modulus value measured in the laboratory.
- **Indirect Tensile Strength** (for CRCP only)—The indirect tensile strength is measured on samples recovered during the coring process and is used to estimate the flexural strength of the in-place PCC layer. If cores are unavailable, the compressive strength may be used to estimate the in-place flexural strength.
- **Flexural Strength**—Not needed for the existing PCC layer; the indirect tensile strength or compressive strength may be used to estimate the flexural strength.

Unbound Layers

- **Resilient Modulus**—The backcalculated modulus values adjusted to laboratory conditions is the preferred and suggested technique for rehabilitation design because the resulting layer modulus value is an equivalent value of the materials that vary horizontally and vertically. The resilient modulus also may be calculated from DCP penetration rates or measured in the laboratory on test specimens prepared and compacted to the in-place moisture content and dry density found during the subsurface investigation. These techniques are not suggested because they do not capture the variability of materials in the vertical and horizontal direction without increasing the test program. The laboratory resilient modulus test represents a discrete specimen in the horizontal and vertical direction, while the DCP test captures the variability vertically, but not horizontally with one test. More importantly, unbound layers and foundations that contain large boulders or aggregates are difficult to test in the laboratory and in-place with the DCP.

- **Volumetric Properties**—Measure the moisture content and dry density of undisturbed samples recovered during the subsurface investigation. The in-place volumetric properties are used for estimating the in-place resilient modulus value of the unbound layers from the regression equations developed from the LTPP data, if deflection basin data and DCP test results for estimating in-place modulus values are unavailable (*27*).
- **Classification Properties**—Measure the gradation and Atterberg limits from bulk sample recovered from the subsurface investigation.

9.3 ANALYSIS OF PAVEMENT EVALUATION DATA FOR REHABILITATION DESIGN CONSIDERATIONS

The pavement structural evaluation for determining the condition of the existing pavement layers is based on an analysis of the visual distress surveys, deflection basin and other field tests, and laboratory tests. It is recommended that the highest input level available be used for rehabilitation design of high volume roadways.

9.3.1 Visual Distress Survey to Define Structural Adequacy

Surface distresses provide a valuable insight into a pavement's current structural condition. Tables 9-7 and 9-8 provide a recommended assessment of rigid and flexible pavements, respectively. These two tables relate the condition of the pavement surface as to whether the pavement is structurally adequate, marginal or inadequate. All of the distresses included in Tables 9-9 and 9-10 are not predicted with AASHTOWare Pavement ME Design. Adequate implies that the surface condition or individual distresses would not trigger any major rehabilitation activity and the existing pavement has some remaining life; marginal implies that the existing pavement has exhibited distress levels that do require maintenance or some type of minor repairs; and inadequate implies that the pavement has distresses that require immediate major rehabilitation and has no remaining life. The values included in these two tables depend on the importance of the distress to an individual agency.

9.3.2 Backcalculation of Layer Modulus Values

Deflection basin data are considered one of the more important factors to assess the structural condition of the pavement. One of the more common methods for analysis of deflection data is to backcalculate the elastic properties for each layer in the pavement structure and foundation. Backcalculated elastic layer modulus values were used during the global calibration process of AASHTOWare Pavement ME Design.

Backcalculation programs provide the elastic layer modulus typically used for pavement evaluation and rehabilitation design. *Standard Guide for Calculating in Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory* is a procedure for analyzing deflection basin test results to determine layer elastic moduli (i.e., Young's modulus). The Long Term Pavement Performance (LTPP) database contains backcalculated moduli results from thousands of deflection basin tests that can be used for reference.

The absolute error or Root Mean Squared (RMS) error is the value that is used to judge the reasonableness of the backcalculated modulus values. The absolute error term is the absolute difference between the measured and computed deflection basins expressed as a percent error or difference per sensor; the RMS error term represents the goodness-of-fit between the measured and computed deflection basins. The RMS and absolute error terms needs to be as small as possible. An RMS error value in excess of 3 percent generally implies that the layer modulus values calculated from the deflection basins are inaccurate or questionable. RMS error values less than 3 percent should be used in selecting the layer modulus values for determining the minimum overlay thickness.

Table 9-9. Distress Types and Severity Levels Recommended for Assessing Rigid Pavement Structural Adequacy

Load-Related Distress	Highway Classification	Current Distress Level Regarded as: Inadequate	Marginal	Adequate
JPCP Deteriorated Cracked Slabs (medium and high-severity transverse and longitudinal cracks and corner breaks), % slabs	Interstate, Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10
JRCP Deteriorated Cracked Slabs (medium and high-severity transverse cracks and corner breaks), #/lane-mi	Interstate, Freeway	>40	15 to 40	<15
	Primary	>50	20 to 50	<20
	Secondary	>60	25 to 60	<25
JPCP Mean Transverse Joint/Crack Faulting, in.	Interstate, Freeway	>0.15	0.1 to 0.15	<0.1
	Primary	>0.20	0.12 to 0.20	<0.125
	Secondary	>0.30	0.15 to 0.30	<0.15
JRCP Mean Transverse Joint/Crack Faulting, in.	Interstate, Freeway	>0.30	0.15 to 0.30	<0.1
	Primary	>0.35	0.18 to 0.35	<0.125
	Secondary	>0.40	0.20 to 0.40	<0.15
CRCP Punchouts (medium and high severity), #/lane-mi.	Interstate, Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10

Note: The above distresses can be used to access the condition of the existing rigid pavement, all of which are not predicted by AASHTOWare Pavement ME Design.

The absolute error (percent error per sensor) and RMS error (goodness-of-fit) vary from station-to-station and depend on the pavement's physical features that have an effect on the deflection basin measured with the FWD. For example, thickness variations, material density variations, surface distortion, and cracks, which may or may not be visible at the surface and may cause small irregularities within the measured deflection basin, which are not consistent with the assumptions of elastic layer theory. Thus, the calculated layer modulus represents an "effective" Young's modulus that adjusts for stress-sensitivity and discontinuities or anomalies (variations in layer thickness, localized segregation, cracks, slippage between adjacent layers, and the combinations of similar materials into a single layer). Distress types and the input levels recommended for assessing the structural adequacy of a current flexible pavement are presented in Table 9-8.

Layer thickness is a critical parameter for backcalculating layer modulus values. The use of borings and cores to measure layer thickness becomes expensive, considering traffic control requirements and the time needed for the drilling operation. GPR is another test method that may be used to determine the variation in layer thickness along a project.

Table 9-10. Distress Types and Levels Recommended for Assessing Current Flexible Pavement Structural Adequacy

Distress Type	Highway Classification	Current Distress Level Regarded as: Inadequate (Poor)	Marginal (Fair)	Adequate (Good)
Fatigue Cracking, percent of total lane area	Interstate, Freeway	>20	5 to 20	<5
	Primary	>45	10 to 45	<10
	Secondary	>45	10 to 45	<10
Longitudinal Cracking in Wheel Path, ft/mi	Interstate, Freeway	>1060	265 to 1060	<265
	Primary	>2650	530 to 2650	<530
	Secondary	>2650	530 to 2650	<530
Reflection Cracking, percent of total lane area.	Interstate, Freeway	>20	5 to 20	<5
	Primary	>45	10 to 45	>10
	Secondary	>45	10 to 45	<10
Transverse Cracking Length, ft/mi	Interstate, Freeway	>800	500 to 800	<500
	Primary	>1000	800 to 1000	<800
	Secondary	>1000	800 to 1000	<800
Rutting, mean depth, maximum between both wheel paths, in.	Interstate, Freeway	>0.45	0.25 to 0.45	<0.25
	Primary	>0.6	0.35 to 0.60	<0.35
	Secondary	>0.8	0.40 to 0.80	<0.4
Shoving, percent of wheel path area	Interstate, Freeway	>10	1 to 10	None
	Primary	>20	10 to 20	<10
	Secondary	>50	20 to 45	<20

Note: The above distresses can be used to access the condition of the existing flexible pavement, all of which are not predicted by AASHTOWare Pavement ME Design.

Flexible Pavements

The elastic modulus of each structural layer typically is calculated using programs based on elastic layer theory that use an iterative technique to match the calculated deflection basin to the measured one. Backcalculation programs that use this iterative technique do not result in a unique solution or set of layer moduli; therefore, an experienced pavement designer is needed to interpret useful and accurate data from Falling Weight Deflectometer tests.

Rigid Pavements

Rigid pavements generally are analyzed as slab on grade with or without a base or subbase. In the past decade, much progress has been made in the development of reliable methods for backcalculation of concrete slab, base layer, and subgrade moduli from deflection measurements. Several methods for backcalculating the PCC slab, base, and subgrade moduli or moduli of subgrade reaction (*k*-value) are available. Each method has its strengths and its limitations. The following are algorithms specifically developed for rigid pavement; based on slab on elastic solid or slab on dense liquid models:

- *AREA* method-based procedures.
- Best Fit-based procedures.

Both backcalculation procedures/algorithms are based on plate theory and are used to backcalculate layer material properties—elastic modulus, Poisson's ratio, and modulus of subgrade reaction. The Best Fit method solves for a combination of the radius of relative stiffness, ℓ, and the coefficient of subgrade reaction, k, that produce the best possible agreement between the predicted and measured deflections at each sensor. The *AREA* method, which was described in the *Guide to Design of Pavement Structures* (*1*), estimates the radius of relative stiffness as a function of the *AREA* of the deflection basin. This estimation, along with the subsequent calculation of subgrade k and slab modulus of elasticity, E, is made using simple closed form equations. Both methods are based on Westergaard's solution (*30*) for the interior loading of a plate consisting of a linear elastic, homogeneous, and isotropic material resting on a dense liquid foundation.

To account for the effect of a stabilized base, a ratio of the moduli of elasticity of PCC and base layers should be assumed according to the LTPP guidelines (*13*).

9.3.3 Loss of Support Detection

Detection of loss of support under joints and cracks in rigid pavements is one of the important uses of the GPR and FWD. The FWD deflection data is analyzed in several ways to estimate the approximate area where loss of support has occurred under a concrete pavement. If extensive loss of support is found along a project this may require subsealing or slab fracturing to establish a uniform layer for an overlay. GPR may also be used to locate areas with this type of anomaly, but it does not provide a quantitative measure of the loss of support.

9.3.4 Joint Load Transfer Efficiency

Deflection testing may also be used to evaluate the LTE of joints and cracks in rigid pavements. This information is used in selecting rehabilitation strategies, needed repair (e.g., retro-fit dowels), and in assessing the reflection cracking potential if the jointed concrete pavement is overlaid with an HMA overlay.

9.3.5 Variability Along a Project

Variation along a project creates a much more difficult task to obtain the appropriate inputs for a project. This variability may be quantified based on the field data sets; visual survey, GPR, and deflection basin data. The visual surveys are used to define if there are significant differences in the surface distresses over the length of the project. The deflection basins and GPR readings may also be used to estimate the variability along a project and determine if the load-response or layer thicknesses of the pavement structure are significantly different along the project.

The variation can be handled for cases where large differences occur along the existing project by dividing the project into multiple design sections. The decision as to subdividing the project into two or more design sections could be based on whether or not the recommended rehabilitation work should actually change. For example, one portion of a project may exhibit extensive fatigue cracking, while another portion has only rutting. The overlay design could logically be different for each section, or the possibility of removal and replacement of the existing damaged material may be the deciding factor to subdivide the project.

CHAPTER 10

Determination of Material Properties for New Paving Materials

The MEPDG procedure requires that all material properties entered into the program for new layers represent the values that exist right after construction. The in-place properties for new paving layers will be unavailable to the designer because the project has yet to be built. Thus, most of the material property inputs need to be estimated for most runs (inputs Levels 2 or 3). This section provides guidance for estimating the critical properties of the paving layers for new pavement and rehabilitation design strategies.

10.1 MATERIAL INPUTS AND THE HIERARCHICAL INPUT CONCEPT

The general approach for determining design inputs for materials in AASHTOWare Pavement ME Design is a hierarchical (level) system (as defined in Chapters 4 and 5). In its simplest and most practical form, the hierarchical approach is based on the philosophy that the level of engineering effort exerted in the pavement design process for characterizing the paving materials and foundation should be consistent with the relative importance, size, and cost of the design project.

Input Level 1 involves comprehensive laboratory tests. In contrast, Level 3 requires the designer to estimate the most appropriate design input value of the material property based on experience with little or no testing. The major material types for which default values (input Level 3) are available in AASHTOWare Pavement ME Design are presented in Table 10-1. Level 2 inputs are estimated through correlations with other material properties that are commonly measured in the laboratory or field. Regardless of input level selected, the program runs the same analysis. As noted above, most of the analysis runs will be completed using input Levels 2 and 3, because the paving layers have yet to be placed at the time that the structural analysis is completed.

10.2 HMA MIXTURES; INCLUDING SMA, ASPHALT-TREATED OR STABILIZED BASE LAYERS, ASPHALT PERMEABLE TREATED BASE MIXES

Fundamental properties are required for all HMA-mixture types or layers to execute AASHTOWare Pavement ME Design. Table 10-2 lists the HMA material properties that are required for the HMA material types listed in Table 10-1, as well as identify the recommended test protocols and other sources for estimating these properties.

The input properties for all HMA-material types may be grouped into volumetric and engineering properties. The volumetric properties include air voids, effective asphalt content by volume, aggregate gradation, mix density, and asphalt grade. The volumetric properties entered into the program need to be representative of the mixture after compaction, before the pavement is opened to truck traffic. The project-specific values will be unavailable to the designer because the new pavement layers have yet to be produced and placed. However, these parameters could be available from previous construction records.

The engineering or mechanistic properties for HMA materials include the dynamic modulus, creep compliance, and indirect tensile strength. It is recommended that input Levels 2 or 3 be used to estimate these properties, unless the agency or user has a library of laboratory test results for different HMA mixtures. The use of library test data is considered input Level 2.

Table 10-1. Major Material Types for AASHTOWare Pavement ME Design

Asphalt Materials	**Non-Stabilized Granular Base/Subbase**
Stone Matrix Asphalt (SMA) Hot Mix Asphalt (HMA) – Dense Graded – Open Graded Asphalt – Asphalt Stabilized Base Mixes – Sand Asphalt Mixtures Cold Mix Asphalt – Central Plant Processed – Cold In-Place Recycling **PCC Materials** Intact Slabs—PCC – High-Strength Mixes – Lean Concrete Mixes Fractured Slabs – Crack/Seat – Break/Seat – Rubblized **Chemically Stabilized Materials** Cement Stabilized Aggregate Soil Cement Lime Cement Fly Ash Lime Fly Ash Lime Stabilized Soils Open-Graded Cement Stabilized Aggregate	Granular Base/Subbase Sandy Subbase Cold Recycled Asphalt (used as aggregate) – RAP (includes millings) – Pulverized In-Place Cold Recycled Asphalt Pavement; (HMA plus aggregate base/subbase) **Subgrade Soils** Gravelly Soils (A-1; A-2) Sandy Soils – Loose Sands (A-3) – Dense Sands (A-3) – Silty Sands (A-2-4; A-2-5) – Clayey Sands (A-2-6; A-2-7) Silty Soils (A-4; A-5) Clayey Soils, Low-Plasticity Clays (A-6) – Dry-Hard – Moist Stiff – Wet/Sat-Soft Clayey Soils, High-Plasticity Clays (A-7) – Dry-Hard – Moist Stiff – Wet/Sat-Soft **Bedrock** Solid, Massive, and Continuous Highly Fractured, and Weathered

Table 10-2. Asphalt Materials and the Test Protocols for Measuring the Material Property Inputs for New and Existing HMA Layers

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New HMA (new pavement and overlay mixtures), as built properties prior to opening to truck traffic	Dynamic modulus	X		AASHTO T 342
	Tensile strength	X		AASHTO T 322
	Creep Compliance	X		AASHTO T 322
	Poisson's ratio		X	National test protocol unavailable. Select AASHTOWare Pavement ME Design default relationship.
	Surface shortwave absorptivity		X	
	Thermal conductivity	X		ASTM E1952
	Heat capacity	X		ASTM D2766
	Coefficient of thermal contraction		X	National test protocol unavailable. Use AASHTOWare Pavement ME Design default values.
	Effective asphalt content by volume	X		AASHTO T 308
	Air voids	X		AASHTO T 166
	Aggregate specific gravity	X		AASHTO T 84 and T 85
	Gradation	X		AASHTO T 27
	Unit Weight	X		AASHTO T 166
	Voids filled with asphalt (VFA)	X		AASHTO T 209
Existing HMA mixtures, in-place properties at time of pavement evaluation	FWD backcalculated layer modulus	X		AASHTO T 256 and ASTM D5858
	Poisson's ratio		X	National test protocol unavailable. Use AASHTOWare Pavement ME Design default values.
	Unit Weight	X		AASHTO T 166 (cores)
	Asphalt content	X		AASHTO T 164 (cores)
	Gradation	X		AASHTO T 27 (cores or blocks)
	Air voids	X		AASHTO T 209 (cores)
	Asphalt recovery	X		AASHTO T 164/R 59/T 319 (cores)
Asphalt (new, overlay, and existing mixtures)	Asphalt Performance Grade (PG), OR	X		AASHTO T 315
	Asphalt binder complex shear modulus (G*) and phase angle (δ), OR	X		AASHTO T 49
	Penetration, OR	X		AASHTO T 53
	Ring and Ball Softening Point Absolute Viscosity Kinematic Viscosity Specific Gravity, OR	X		AASHTO T 202 AASHTO T 201 AASHTO T 228
	Brookfield Viscosity	X		AASHTO T 316

Note: The global calibration factors included in AASHTOWare Pavement ME Design software for HMA pavements were determined using the NCHRP 1-37A viscosity based predictive model for dynamic modulus (E_{HMA}*).

For specialty mixtures, such as warm mix asphalt (WMA) or mixtures produced with reclaimed asphalt pavement (RAP) and reclaimed asphalt shingles (RAS), it is recommended that the dynamic complex modulus be measured in the laboratory and used as a Level 1 HMA input to the AASHTOWare Pavement ME Design software.

If a library of HMA-test data has been established, the user could select the test results from previous HMA mixtures most similar to the one being used or use an average of the results from other similar mixtures. The following summarizes the recommended input parameters and values for the HMA mixtures.

Aggregate gradation—For new HMA mixtures, use values that are near the mid-range of the project specifications or use average values from previous construction records for a particular type of mix. For existing HMA layers, use the average value recovered from as built construction records, or if construction records are unavailable, measure the gradation from the aggregates recovered from cores or blocks of the HMA (refer to Chapter 9).

Air voids, effective asphalt content by volume, density, voids in mineral aggregate (VMA), voids filled with asphalt (VFA)—For new HMA mixtures, use values that are near the mid-range of the project specification or use average values from previous construction records for a particular type of HMA mixture. More detail is provided in the latter part of this section for determining the volumetric properties for new HMA mixtures. For existing HMA layers, measure the air voids from cores recovered from the project. The other volumetric properties may be calculated from the in-place air voids and volumetric properties recovered from as built construction records (refer to Chapter 9). If construction records are unavailable, measure the effective asphalt content, VMA, and VFA from the cores or blocks taken from the project.

Poisson's ratio—For new HMA mixtures, use the temperature calculated values within the MEPDG. In other words, check the box to use the predictive model to calculate Poisson's ratio from the pavement temperatures. For existing, age-hardened HMA mixtures, use the default values recommended in AASHTOWare Pavement ME Design (refer to Table 10-3).

Dynamic modulus, creep compliance, indirect tensile strength—For new HMA mixtures, input Levels 2 or 3 could be used, unless the agency has a library of test results. Material properties needed for input Levels 2 and 3 include gradation, asphalt PG classification, and test results from the dynamic shear rheometer (DSR; AASHTO T 315). AASHTOWare Pavement ME Design software provides the user with two options for estimating the dynamic modulus; one listed as NCHRP 1-37A viscosity based model and the other listed as NCHRP 1-40D (dynamic shear modulus of the asphalt) based model. The global calibration factors for all HMA predictive equations (refer to Section 5.3) were determined using the NCHRP 1-37A viscosity based model. The option selected depends on the historical data available to the designer. For existing HMA layers, use input Levels 2 or 3 and the backcalculated values from the FWD deflection basins for estimating the dynamic modulus. The creep compliance and indirect tensile strength are not needed for the existing HMA layers.

- **Surface shortwave absorptivity**—Use default value set in AASHTOWare Pavement ME Design, 0.85.
- **Coefficient of thermal contraction of the mix**—Use default values set in AASHTOWare Pavement ME Design for different mixtures and aggregates.
- **Reference temperature**—70°F should be used.
- **Thermal conductivity of asphalt**—Use default value set in program, 1.25 BTU/fr-ft-°F.
- **Heat capacity of asphalt**—Use default value set in program, 0.28 BTU/lb-°F.

Although input Level 1 is the preferred category of inputs for pavement design, many agencies have yet to acquire the testing capabilities to characterize HMA mixtures. Thus, input Levels 2 and 3 are summarized in Table 10-3. For most analyses, it is permissible for designers to use a combination of Level 1, 2, and 3 material inputs that are based on their unique needs and testing capabilities. The following provides more detailed discussion on determining the volumetric properties that may be used to estimate these input parameters for new HMA mixtures.

- **Air Voids (AASHTO T 269), V_a**—The air voids at construction need to represent the average in-place air voids expected after the HMA has been compacted with the rollers, but prior to opening the roadway to truck traffic. This value will be unavailable during structural design because it has yet to be produced. It is recommended that this value be obtained from previous construction records for similar mixtures or the designer could enter the target value from the project specifications.
- **Bulk Specific Gravity of the Combined Aggregate Blend (AASHTO T 84 and T 85), G_{sb}**—This value is dependent on the type of aggregates used in the HMA and gradation. Most agencies will have an expected range of this value from previous mixture designs for the type of aggregates used, their source, and combined gradation (type of mixture dependent) specified for the project.
- **Maximum Specific Gravity of Mixture (AASHTO T 209), G_{mm}**—This value is dependent on the type of aggregate, gradation, and asphalt content used in the HMA. Most agencies will have an expected range of this value from previous mixture designs using the aggregate source and gradation (type of mixture) specified for the project. The maximum specific gravity can be calculated from the component properties, if no historical information exists for the HMA mixture specified for the project.
- **Voids in Mineral Aggregate, VMA**—VMA is an input to the AASHTOWare Pavement ME Design or thermal cracking predictions and determination of other volumetric properties. The mixture VMA needs to represent the condition of the mixture after it has been compacted with the rollers, but prior to opening the roadway to truck traffic. This value will be unavailable during structural design because it has yet to be produced and placed. It is recommended that the value be calculated from other volumetric properties that may be obtained from construction records for similar type mixtures, aggregate sources, and gradations.
- **Effective Asphalt Content by Volume, V_{be}**—The effective asphalt content by volume needs to represent the in-place asphalt content; after the mix has been placed by the paver. This value will be unavailable during structural design because it has yet to be produced. It is recommended that the value be calculated from the other volumetric properties, as shown below.

Table 10-3. Recommended Input Parameters and Values; Limited or No Testing Capabilities for HMA (Input Levels 2 or 3)

Measured Property	Input Levels 2 or 3
Dynamic modulus, E_{HMA} (new HMA)	• No dynamic modulus, E_{HMA}, laboratory testing required. • Use MEPDG E_{HMA} predictive equation. Inputs are gradation, bitumen viscosity, loading frequency, air void content, and effective bitumen content by volume. Input variables may be obtained through testing of extracted cores or from agency historical records. • Use typical *Ai-VTS* values based on asphalt binder grade (PG, or viscosity, or penetration grades).
Dynamic modulus, E_{HMA} (existing HMA layer)	• No dynamic modulus, E_{HMA}, laboratory testing required. • Use MEPDG E_{HMA} predictive equation. Inputs are gradation, bitumen viscosity, loading frequency, air void content, and effective bitumen content by volume. Input variables may be obtained through testing of extracted cores or from agency historical records. • Use typical *Ai-VTS* values based on asphalt binder grade (PG, or viscosity, or penetration grades). • Determine existing pavement condition rating (excellent, good, fair, poor, very poor).
Tensile strength , TS (new HMA surface; not required for existing HMA layers)	Use MEPDG regression equation: $TS(\text{psi}) = 7416.712 - 114.016 * Va - 122.592 * VFA + 0.704 * VFA^2 + 405.71 * \text{Log10}(Pen77) - 2039.296 * \log10(A)$ where: TS = Indirect tensile strength at 14 °F, psi. Va = HMA air voids, as-constructed, percent VFA = Voids filled with asphalt, as-constructed, percent. $Pen77$ = Asphalt penetration at 77 °F, mm/10. A = Asphalt viscosity-temperature susceptibility intercept. Input variables may be obtained through testing of lab prepared mix samples, extracted cores (for existing pavements), or from agency historical records.
	Use MEPDG regression equation: $D(t) = D_1 * t^m$ $\log(D_1) = -8.524 + 0.01306 * T + 0.7957 * \log10(Va) + 2.0103 * \log10(VFA) - 1.923 * \log10(A)$ $m = 1.1628 - 0.00185 * T - 0.04596 * Va - 0.01126 * VFA + 0.00247 * Pen77 + 0.001683 * T * Pen77^{0.4605}$ where: t = Time, months. T = Temperature at which creep compliance is measured, °F. Va = HMA air voids, as-constructed, %. VFA = Voids filled with asphalt, as-constructed, %. $Pen77$ = Asphalt penetration at 77 °F, mm/10. Input variables may be obtained through testing of lab prepared mix samples, extracted cores (for existing pavements), or from agency historical records.
Air voids	Use as-constructed mix type specific values available from previous construction records.
Volumetric asphalt content	Use as-constructed mix type specific values available from previous construction records.
Total unit weight	Use as-constructed mix type specific values available from previous construction records.

Note: AASHTOWare Pavement ME Design computes input Levels 2 and 3 dynamic modulus, tensile strength, creep compliance, etc. internally once; all the required input variables required by the various equation are provided. (Table 10-3 continued on next page.)

Table 10-3. Recommended Input Parameters and Values; Limited or No Testing Capabilities for HMA (Input Levels 2 and 3)—*continued*

<table>
<tr><th>Measured Property</th><th>Recommended Level 3 Input</th></tr>
<tr><td>Poisson's ratio</td><td>Use predictive equation based on temperature included in the MEPDG for new HMA mixtures and the typical values listed below for the existing HMA layers:
<table>
<tr><th>Reference Temperature °F</th><th>Dense-Graded HMA (Level 3) $\mu_{typical}$</th><th>Open-Graded HMA (Level 3) $\mu_{typical}$</th></tr>
<tr><td><0 °F</td><td>0.15</td><td>0.35</td></tr>
<tr><td>0 – 40 °F</td><td>0.20</td><td>0.35</td></tr>
<tr><td>41 – 70 °F</td><td>0.25</td><td>0.40</td></tr>
<tr><td>71 – 100 °F</td><td>0.35</td><td>0.40</td></tr>
<tr><td>101 – 130 °F</td><td>0.45</td><td>0.45</td></tr>
<tr><td>>130 °F</td><td>0.48</td><td>0.45</td></tr>
</table></td></tr>
<tr><td>Surface shortwave absorptivity</td><td>Use AASHTOWare Pavement ME Design default of 0.85.</td></tr>
<tr><td>Thermal conductivity</td><td>Typical values for asphalt concrete range from 0.244 to 2.0 BTU/(ft)(hr) (°F). Use default value set in program—1.25 BTU(ft)(hr)(°F).</td></tr>
<tr><td>Heat capacity</td><td>Typical values for asphalt concrete range from 1 to 0.50 BTU/(ft)(hr) (°F). Use default value set in program—1.28 BTU(ft)(hr).-°F</td></tr>
<tr><td>Coefficient of thermal contraction</td><td>Use MEPDG predictive equation shown below:

$$L_{MIX} = \frac{VMA * B_{ac} + V_{AGG} * B_{AGG}}{3 * V_{TOTAL}}$$

where:
L_{MIX} = Linear coefficient of thermal contraction of the asphalt concrete mixture (1/° C).
B_{ac} = Volumetric coefficient of thermal contraction of the asphalt cement in the solid state (1/° C).
B_{AGG} = Volumetric coefficient of thermal contraction of the aggregate (1/° C)
VMA = Percent volume of voids in the mineral aggregate (equals percent volume of air voids plus percent volume of asphalt cement minus percent volume of absorbed asphalt cement).
V_{AGG} = Percent volume of aggregate in the mixture.
V_{TOTAL} = 100 percent

Typical values for linear coefficient of thermal contraction, volumetric coefficient of thermal contraction of the asphalt cement in the solid state, and volumetric coefficient of thermal contraction of aggregates measured in various research studies are as follows:

- L_{MIX} = 2.2 to $3.4*10^{-5}$ /° C (linear).
- B_{ac} = 3.5 to $4.3*10^{-4}$ /° C (cubic).
- B_{AGG} = 21 to $37*10^{-6}$ /° C (cubic).
</td></tr>
</table>

Note: That AASHTOWare Pavement ME Design computes input Level 2 and 3 coefficient of thermal extraction, etc. internally; once all the required equation input variables are available.

10.3 PCC MIXTURES, LEAN CONCRETE, AND CEMENT-TREATED BASE LAYERS

Table 10-4 summarizes all the Level 1 inputs required for the PCC-material types listed in Table 10-1. Also presented in Table 10-4 are recommended sources of input data (that is recommended test protocols and other sources of estimates).

Although input Level 1 is preferred for pavement design, most agencies are not equipped with the testing facilities required to characterize the paving materials. Thus, for the more likely situation where agencies have only limited or no testing capability for characterizing PCC materials, Levels 2 and 3 inputs are recommended as presented in Table 10-5. It must be noted that for most situations designers used a combination of Levels 1, 2, and 3 material inputs based on their unique needs and testing capabilities.

10.4 CHEMICALLY STABILIZED MATERIALS, INCLUDING LEAN CONCRETE AND CEMENT-TREATED BASE LAYERS

The compressive strength or modulus of rupture, elastic modulus, and density are required inputs to AASHTOWare Pavement ME Design for any cemenititious or pozzolonic stabilized material. However, the fatigue cracking prediction equation for semi-rigid pavements was not calibrated within the NCHRP Projects 1-37A and 1-40D. As such, these layers should not be used until the prediction model is calibrated.

Agency specific calibration factors could be determined based on the quality of the CAM material. The recommended values to be used in the interim are discussed within the *Standard Practice for Local Calibration* (2).

Table 10-6 summarizes all the Level 1 inputs required for the chemically stabilized material types listed in Table 10-1. Also presented in Table 10-6 are recommended sources of input data (that is recommended test protocols and other sources of estimates). Although Level 1 is the preferred input category for pavement design, most agencies are not equipped with the testing facilities required to characterize the paving materials. Thus, for the more likely situation where agencies have only limited or no testing capability for characterizing chemically stabilized materials, Levels 2 and 3 inputs are recommended as presented in Table 10-7. For most situations, designers use a combination of Levels 1, 2, and 3 material inputs based on their unique needs and testing capabilities.

Table 10-4. PCC Material Input Level 1 Parameters and Test Protocols for New and Existing PCC

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New PCC and PCC overlays and existing PCC when subject to a bonded PCC overlay	Elastic modulus	X		ASTM C469
	Poisson's ratio	X		ASTM C469
	Flexural strength	X		AASHTO T 97
	Indirect tensile strength (CRCP only)	X		AASHTO T 198
	Unit weight	X		AASHTO T 121
	Air Content	X		AASHTO T 152 or T 196
	Coefficient of thermal expansion	X		AASHTO T 336
	Surface shortwave absorptivity		X	Use AASHTOWare Pavement ME Design defaults
	Thermal conductivity	X		ASTM E1952 (or use AASHTOWare Pavement ME Design defaults)
	Heat capacity	X		ASTM D2766 (or use AASHTOWare Pavement ME Design defaults)
	PCC zero-stress temperature		X	National test protocol not available. Estimate using agency historical data or select AASHTOWare Pavement ME Design defaults
	Cement type		X	Select based on actual or expected cement source
	Cementitious material content		X	Select based on actual or expected concrete mix design
	Water to cement ratio		X	Select based on actual or expected concrete mix design
	Aggregate type		X	Select based on actual or expected aggregate source
	Curing method		X	Select based on agency recommendations and practices
	Ultimate shrinkage		X	Testing not practical. Estimate using prediction equation in AASHTOWare Pavement ME Design
	Reversible shrinkage		X	Estimate using agency historical data or select AASHTOWare Pavement ME Design defaults
	Time to develop 50 percent of ultimate shrinkage		X	Estimate using agency historical data or select AASHTOWare Pavement ME Design defaults
Existing intact and fractured PCC	Elastic modulus		X	ASTM C469 (extracted cores) AASHTO T 256 (non-destructive deflection testing)
	Poisson's ratio		X	ASTM C469 (extracted cores)
	Flexural strength		X	AASHTO T 97 (extracted cores)
	Unit weight		X	AASHTO T 121 (extracted cores)
	Surface shortwave absorptivity		X	National test protocol not available. Use AASHTOWare Pavement ME Design defaults
	Thermal conductivity	X		ASTM E1952 (extracted cores)
	Heat capacity	X		ASTM D2766 (extracted cores)

Table 10-5. Recommended Input Parameters and Values; Limited or No Test Capabilities for PCC Materials (Input Levels 2 or 3)

<table>
<tr><th>Measured Property</th><th>Recommended Input Levels 2 and 3</th></tr>
<tr><td>New PCC Elastic modulus and flexural strength</td><td>

28-day flexural strength AND 28-day PCC elastic modulus, OR
28-day compressive strength AND 28-day PCC elastic modulus, OR
28-day flexural strength ONLY, OR
28-day compressive strength ONLY

</td></tr>
<tr><td>Existing intact PCC elastic modulus</td><td>
Based on the pavement condition, select typical modulus values from the range of values given below:
<table>
<tr><th>Qualitative Description of Pavement Condition</th><th>Typical Modulus Ranges, psi</th></tr>
<tr><td>Adequate</td><td>3 to 4 × 10^6</td></tr>
<tr><td>Marginal</td><td>1 to 3 × 10^6</td></tr>
<tr><td>Inadequate</td><td>0.3 to 1 × 10^6</td></tr>
</table>
</td></tr>
<tr><td>Existing fractured PCC elastic modulus</td><td>
The three common methods of fracturing PCC slabs include crack and seat, break and seat, and rubblization. In terms of materials characterization, cracked or broken and seated PCC layers is considered in a separate category from rubblized layers. At Level 3, typical modulus values may be adopted for design (see below):
<table>
<tr><th>Fractured PCC Layer Type</th><th>Typical Modulus Ranges, psi</th></tr>
<tr><td>Crack and Seat or Break and Seat</td><td>150,000 to 1,000,000</td></tr>
<tr><td>Rubblized</td><td>50,000 to 150,000</td></tr>
</table>
</td></tr>
<tr><td>Poisson’s ratio</td><td>
Poisson's ratio for new PCC typically ranges between 0.10 and 0.21, and a value of 0.20 0.15 and 0.18 are typically assumed for PCC design. See below for typical Poisson’s ratio values for PCC materials.
<table>
<tr><th>PCC Materials</th><th>Level 3 μtypical</th></tr>
<tr><td>PCC Slabs (newly constructed or existing)</td><td>0.20</td></tr>
<tr><td>Fractured Slab
Crack/Seat
Break/Seat
Rubblized</td><td>
0.20
0.20
0.30</td></tr>
</table>
</td></tr>
<tr><td>Unit weight</td><td>Select agency historical data or from typical range for normal weight concrete: 140 to 160 lb/ft3</td></tr>
</table>

Note: Project specific testing is not required at Level 3. Historical agencies test values assembled from past construction with tests conducted using the list protocols are all that is required.

Table 10-5 continued on next page.

Table 10-5. Recommended Input Parameters and Values; Limited or No Test Capabilities for PCC Materials (Input Levels 2 and 3)—*continued*

<table>
<tr><th>Measured Property</th><th>Recommended Level 3 Input</th></tr>
<tr><td>Coefficient of thermal expansion</td><td>Select agency historical values or typical values based on PCC coarse aggregate type.

<table>
<tr><th>Aggregates Type</th><th>Coefficient of Thermal Expansion (10^{-6}/° F)</th></tr>
<tr><td>Andesite</td><td>5.3</td></tr>
<tr><td>Basalt</td><td>5.2</td></tr>
<tr><td>Diabase</td><td>4.6</td></tr>
<tr><td>Gabbro</td><td>5.3</td></tr>
<tr><td>Granite</td><td>5.8</td></tr>
<tr><td>Schist</td><td>5.6</td></tr>
<tr><td>Chert</td><td>6.6</td></tr>
<tr><td>Dolomite</td><td>5.8</td></tr>
<tr><td>Limestone</td><td>5.4</td></tr>
<tr><td>Quartzite</td><td>6.2</td></tr>
<tr><td>Sandstone</td><td>6.1</td></tr>
<tr><td>Expanded shale</td><td>5.7</td></tr>
</table>
Where coarse aggregate type is unknown, use AASHTOWare Pavement ME Design default value of 5.5×10^{-6}/° F</td></tr>
<tr><td>Surface shortwave absorptivity</td><td>Use level 3 AASHTOWare Pavement ME Design default of 0.85</td></tr>
<tr><td>Thermal conductivity</td><td>Typical values for asphalt concrete range from 0.2 to 2.0 to BTU/(ft)(hr)(°F). Use default value set in program—1.25 BTU(ft)(hr)(°F).</td></tr>
<tr><td>Heat capacity</td><td>Typical values for asphalt concrete range from 0.1 to 0.50 to BTU/(ft)(hr)(°F). Use default value set in program—0.28 BTU/lb.-°F</td></tr>
</table>

Table 10-5 continued on next page.

Table 10-5. Recommended Input Parameters and Values; Limited or No Test Capabilities for PCC Materials (Input Levels 2 and 3)—*continued*

<table>
<tr><th>Measured Property</th><th>Recommended Level 3 Input</th></tr>
<tr><td>PCC set temperature</td><td>
Zero stress temperature, T_z, can be input directly or can be estimated from monthly ambient temperature and cement content using the equation shown below:
$$T_z = (C_C{*}0.59328{*}H{*}0.5{*}1000{*}1.8/(1.1{*}2400) + MMT)$$
where,
T_z = PCC set temperature (allowable range: 70 to 212 ˚F).
C_C = Cementitious content, lb/yd^3.
H = –0.0787 + 0.007*MMT – 0.00003*MMT^2
MMT = Mean monthly temperature for month of construction, ˚F.
An illustration of the zero stress temperatures for different mean monthly temperatures and different cement contents in the PCC mix design is presented below:
<table>
<tr><th rowspan="2">Mean Monthly Temperature</th><th rowspan="2">H</th><th colspan="4">Cement Content, lbs/cy</th></tr>
<tr><th>400</th><th>500</th><th>600</th><th>700</th></tr>
<tr><td>40</td><td>0.1533</td><td>52*</td><td>56</td><td>59</td><td>62</td></tr>
<tr><td>50</td><td>0.1963</td><td>66</td><td>70</td><td>74</td><td>78</td></tr>
<tr><td>60</td><td>0.2333</td><td>79</td><td>84</td><td>88</td><td>93</td></tr>
<tr><td>70</td><td>0.2643</td><td>91</td><td>97</td><td>102</td><td>107</td></tr>
<tr><td>80</td><td>0.2893</td><td>103</td><td>109</td><td>115</td><td>121</td></tr>
<tr><td>90</td><td>0.3083</td><td>115</td><td>121</td><td>127</td><td>134</td></tr>
<tr><td>100</td><td>0.3213</td><td>126</td><td>132</td><td>139</td><td>145</td></tr>
</table>
Note: Mean PCC temperature in degrees F.
</td></tr>
</table>

Measured Property	Recommended Level 3 Input
Cement type	Estimate based on agency practices.
Cementitious material content	Estimate based on agency practices.
Water to cement ratio	Estimate based on agency practices.
Aggregate type	Estimate based on agency practices.
Curing method	Estimate based on agency practices.
Ultimate shrinkage	Estimate using prediction equation in AASHTOWare Pavement ME Design.
Reversible shrinkage	Use AASHTOWare Pavement ME Design default of 50 percent unless more accurate information is available.
Time to develop 50 percent of ultimate shrinkage	Use AASHTOWare Pavement ME Design default of 35 days unless more accurate information is available.

Note: Project specific testing is not required at Level 3. Historical agencies test values assembled from past construction with tests conducted using the list protocols are all that is required.

Table 10-6. Chemically Stabilized Materials Input Requirements and Test Protocols for New and Existing Chemically Stabilized Materials

Design Type	Material Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
			Test	Est.	
New	Lean concrete and cement-treated aggregate	Elastic modulus	X		ASTM C 469
		Flexural strength (Required only when used in HMA pavement design)	X		AASHTO T 97
	Lime–cement–fly ash	Resilient modulus		X	No test protocols available. Estimate using Levels 2 and 3.
	Soil cement	Resilient modulus	X		Mixture Design and Testing Protocol (MDTP) in conjunction with AASHTO T 307.
	Lime stabilized soil	Resilient modulus		X	No test protocols available. Estimate using Levels 2 and 3.
	All	Unit weight		X	No testing required. Estimate using Levels 2 and 3.
		Poisson's ratio		X	No testing required. Estimate using Levels 2 and 3.
		Thermal conductivity	X		No testing required. Estimate using Levels 2 and 3.
		Heat capacity	X		No testing required. Estimate using Levels 2 and 3.
		Surface short wave absorptivity		X	No test protocols available. Estimate using Levels 2 and 3.
Existing	Lean concrete and cement-treated aggregate	FWD backcalculated modulus	X		AASHTO T 256
	Lime–cement–fly ash	FWD backcalculated modulus	X		AASHTO T 256
	Soil cement	FWD backcalculated modulus	X		AASHTO T 256
	Lime stabilized soil	FWD backcalculated modulus	X		AASHTO T 256
	All	Unit weight		X	No testing required. Estimate using Levels 2 and 3.
		Poisson's ratio		X	No testing required. Estimate using Levels 2 and 3.
		Thermal conductivity	X		ASTM E1952 (cores)
		Heat capacity	X		ASTM D2766 (cores)
		Surface short wave absorptivity		X	No test protocols available. Estimate using Levels 2 and 3.

Table 10-7. Recommended Input Levels 2 and 3 Parameters and Values for Chemically Stabilized Material Properties

<table>
<tr><th>Required Input</th><th>Recommended Input Level</th></tr>
<tr><td>Elastic/resilient modulus</td><td>

- Use unconfined compressive strength (f_c' or q_u) in psi of lab samples or extracted cores converted into elastic/resilient modulus by the following:

<table>
<tr><th>Material</th><th>Relationship for Modulus</th><th>Test Method</th></tr>
<tr><td>Lean concrete and cement treated aggregate</td><td>$E = 57000\left(f_c'\right)^{0.5}$</td><td>AASHTO T 22</td></tr>
<tr><td>Open graded cement stabilized aggregate</td><td>Use input Level 3</td><td>None</td></tr>
<tr><td>Lime-cement-fly ash</td><td>$E = 500 + q_u$</td><td>ASTM C593</td></tr>
<tr><td>Soil cement</td><td>$E = 1200(q_u)$</td><td>ASTM D1633</td></tr>
<tr><td>Lime stabilized soil</td><td>$M_r = 0.124(q_u) + 9.98$</td><td>ASTM D5102</td></tr>
</table>

OR

- Select typical E and M_r values in psi as follows:

<table>
<tr><td>Lean concrete, E</td><td>2,000,000</td></tr>
<tr><td>Cement stabilized aggregate, E</td><td>1,000,000</td></tr>
<tr><td>Open graded cement stabilized aggregate, E</td><td>750,000</td></tr>
<tr><td>Soil cement, E</td><td>500,000</td></tr>
<tr><td>Lime–cement–flyash, E</td><td>1,500,000</td></tr>
<tr><td>Lime stabilized soils, M_r</td><td>45,000</td></tr>
</table>
</td></tr>
<tr><td>Flexural strength (required only for flexible pavements)</td><td>

- Use 20% of the compressive strength of lab samples or extracted cores as an estimate of the flexural strength for all chemically stabilized materials.

OR

- Select typical M_R values in psi as follows:

<table>
<tr><td>Chemically stabilized material placed under flexible pavement (base)</td><td>750</td></tr>
<tr><td>Chemically stabilized material used as subbase, select material, or subgrade under flexible pavement</td><td>250</td></tr>
</table>
</td></tr>
<tr><td>Poisson's ratio</td><td>Select typical Poisson's ratio values are as follows:

<table>
<tr><td>Lean concrete and cement stabilized aggregate</td><td>0.1 to 0.2</td></tr>
<tr><td>Soil cement</td><td>0.15 to 0.35</td></tr>
<tr><td>Lime-Fly Ash Materials</td><td>0.1 to 0.15</td></tr>
<tr><td>Lime Stabilized Soil</td><td>0.15 to 0.2</td></tr>
</table>
</td></tr>
<tr><td>Unit weight</td><td>Use default AASHTOWare Pavement ME Design values of 150 pcf</td></tr>
<tr><td>Thermal conductivity</td><td>Use default AASHTOWare Pavement ME Design values of 1.25 BTU/h-ft-°F</td></tr>
<tr><td>Heat capacity</td><td>Use default AASHTOWare Pavement ME Design values of 0.28 BTU/lb-°F</td></tr>
</table>

10.5 UNBOUND AGGREGATE BASE MATERIALS AND ENGINEERED EMBANKMENTS

Similar to HMA and PCC, physical and engineering properties are required for the unbound pavement layers and foundation. The physical properties include dry density, moisture content, and classification properties, while the engineering property includes the resilient modulus. Designers must be aware that the resilient modulus values have to be determined at the optimum moisture content and maximum dry density, thus ensuring the unbound layers are representative of conditions when the pavement is opened to truck traffic.

For new alignments or new designs, the default resilient modulus values included in AASHTOWare Pavement ME Design (input Level 3) may be used, the modulus may be estimated from other properties of the material (input Level 2), or measured in the laboratory (input Level 1). For rehabilitation or reconstruction designs, the resilient modulus of each unbound layer and embankment may be backcalculated from deflection basin data or estimated from DCP or CBR tests. If the resilient modulus values are determined by backcalculating elastic layer modulus values from deflection basin tests, those values need to be adjusted to laboratory conditions (*28, 29*). Table 10-8 lists the values recommended in those design pamphlets. If the resilient modulus values are estimated from the DCP or other tests, those values may be used as inputs to the MEPDG, but should be checked based on local material correlations and adjusted to laboratory conditions, if necessary. The DCP test should be performed in accordance with ASTM D 6951 or an equivalent procedure.

Table 10-8. *C*-Values to Convert the Calculated Layer Modulus Values to an Equivalent Resilient Modulus Measured in the Laboratory

Layer Type	Location	*C*-Value or M_r / E_{FWD} Ratio
Aggregate Base/Subbase	Between a Stabilized and HMA Layer	1.43
	Below a PCC Layer	1.32
	Below an HMA Layer	0.62
Subgrade-Embankment	Below a Stabilized Subgrade/Embankment	0.75
	Below an HMA or PCC Layer	0.52
	Below an Unbound Aggregate Base	0.35

Table 10-9 summarizes the input Level 1 parameters required for the unbound aggregate base, subbase, embankment, and subgrade soil material types listed in Table 10-1. The recommended test protocols are also listed in Table 10-9. Although input Level 1 is preferred for pavement design, most agencies are not equipped with the testing facilities required to characterize the paving materials. Thus, for the more likely situation where agencies have only limited or no testing capability for characterizing unbound aggregate base, subbase, embankment, and subgrade soil materials, input Levels 2 and 3 are recommended, which are provided in Table 10-10. For most analyses, designers will use a combination of Levels 1, 2, and 3 material inputs based on their unique needs and testing capabilities, which is permissible.

The following summarizes the recommended input parameters and values for the unbound layers and foundation:

- **Gradation—**For new materials, the mid-range of the material specifications or the average gradation from previous construction records for similar materials is recommended for use as the input values. For existing pavement layers, use the average gradation from as built construction records. If those records are unavailable, use average results from laboratory tests performed on materials recovered during the field investigation. The gradation of the unbound aggregate or embankment soil could be measured in accordance with AASHTO T 88. If sufficient material was not recovered during the field investigation, the default values included in AASHTO Pavement ME Design for the material classification could be used.

Table 10-9. Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Material Requirements and Test Protocols for New and Existing Materials

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New (lab samples) and existing (extracted materials)	Two Options: Regression coefficients k_1, k_2, k_3 for the generalized constitutive model that defines resilient modulus as a function of stress state and regressed from laboratory resilient modulus tests. Determine the average design resilient modulus for the expected in-place stress state from laboratory resilient modulus tests.	X		AASHTO T 307 or NCHRP 1-28A The generalized model used in MEPDG design procedure is as follows: $M_r = k_1 p_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}+1\right)^{k_3}$ where M_r = resilient modulus, psi θ = bulk stress $= \sigma_1 + \sigma_2 + \sigma_3$ σ_1 = major principal stress. σ_2 = intermediate principal stress σ_3 = minor principal stress confining pressure τ_{oct} = octahedral shear stress $= \frac{1}{3}\sqrt{(\sigma_1-\sigma_2)^2+(\sigma_1-\sigma_3)^2+(\sigma_2-\sigma_3)^2}$ P_a = normalizing stress k_1, k_2, k_3 = regression constants
	Poisson's ratio		X	No national test standard, use default values included in AASHTOWare Pavement ME Design.
	Maximum dry density	X		AASHTO T 180
	Optimum moisture content	X		AASHTO T 180
	Specific gravity	X		AASHTO T 100
	Saturated hydraulic conductivity	X		AASHTO T 215
	Soil water characteristic curve parameters	X		Pressure plate (AASHTO T 99) OR Filter paper (AASHTO T 180) OR Tempe cell (AASHTO T 100)
Existing material to be left in place	FWD backcalculated modulus	X		AASHTO T 256 and ASTM D5858
	Poisson's ratio		X	No national test standard, use default values included in AASHTOWare Pavement ME Design.

Table 10-10. Recommended Levels 2 and 3 Input Parameters and Values for Unbound Aggregate Base, Subbase, Embankment, and Subgrade Soil Material Properties

<table>
<tr><th>Required Input</th><th colspan="4">Recommended Input Level</th></tr>
<tr><td rowspan="17">Resilient modulus</td><td colspan="4">Use Level 3 inputs based on the unbound aggregate base, subbase, embankment, and subgrade soil material AASHTO Soil Classification. AASHTO Soil Class is determined using material gradation, plasticity index, and liquid limit.</td></tr>
<tr><td rowspan="2">AASHTO Soil Classification</td><td colspan="3">Recommended Resilient Modulus at Optimum Moisture (AASHTO T 180), psi</td></tr>
<tr><td>Base/Subbase for Flexible and Rigid Pavements</td><td>Embankment and Subgrade for Flexible Pavements</td><td>Embankment and Subgrade for Rigid Pavements</td></tr>
<tr><td>A-1-a</td><td>40,000</td><td>29,500</td><td>18,000</td></tr>
<tr><td>A-1-b</td><td>38,000</td><td>26,500</td><td>18,000</td></tr>
<tr><td>A-2-4</td><td>32,000</td><td>24,500</td><td>16,500</td></tr>
<tr><td>A-2-5</td><td>28,000</td><td>21,500</td><td>16,000</td></tr>
<tr><td>A-2-6</td><td>26,000</td><td>21,000</td><td>16,000</td></tr>
<tr><td>A-2-7</td><td>24,000</td><td>20,500</td><td>16,000</td></tr>
<tr><td>A-3</td><td>29,000</td><td>16,500</td><td>16,000</td></tr>
<tr><td>A-4</td><td>24,000</td><td>16,500</td><td>15,000</td></tr>
<tr><td>A-5</td><td>20,000</td><td>15,500</td><td>8,000</td></tr>
<tr><td>A-6</td><td>17,000</td><td>14,500</td><td>14,000</td></tr>
<tr><td>A-7-5</td><td>12,000</td><td>13,000</td><td>10,000</td></tr>
<tr><td>A-7-6</td><td>8,000</td><td>11,500</td><td>13,000</td></tr>
<tr><td colspan="4">Note:</td></tr>
<tr><td colspan="4">1. The resilient modulus is converted to a k-value internally within the software for evaluating rigid pavements.
2. The resilient modulus values at the time of construction for the same AASHTO soil classification are different under flexible and rigid pavements because the stress-state under these pavements is different. Soils are stress dependent and the resilient modulus will change with changing stress-state (refer to Table 10-9). The default values included in the NCHPRP beta-test software were estimated as the median value from the test sections included in the LTPP database and used engineering judgment.These default values can be used assuming the soils are at the maximum dry density and optimum water content as defined from AASHTO T 180.
3. Only A-1-a and A-1-b soils are used as base courses.</td></tr>
<tr><td>Maximum dry density</td><td colspan="4">Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Optimum moisture content</td><td colspan="4">Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Specific gravity</td><td colspan="4">Estimate using the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Saturated hydraulic conductivity</td><td colspan="4">Select based on the following inputs: gradation, plasticity index, and liquid limit.</td></tr>
<tr><td>Soil water characteristic curve parameters</td><td colspan="4">Select based on aggregate/subgrade material class.</td></tr>
</table>

- **Atterberg Limits**—For new materials, the mid-range allowed by the material specifications or the average liquid limit and plasticity index from previous construction records for similar materials is recommended for use as the input values. For existing pavement layers, use the average results from the Atterberg limits test for similar materials that were placed using the same material specifications. The liquid limit could be measured in accordance with AASHTO T 89, and the plastic limit and plasticity index determined in accordance with AASHTO T 90. If sufficient material was not recovered during the field investigation, the default values included in AASHTOWare Pavement ME Design for the material classification could be used.
- **Dry Density**—For new materials, the maximum dry density defined by the material specifications using the compaction effort specified for the project, or the average dry density measured on previous construction projects for similar material is recommended for use as the input value. For existing pavement layers that will remain in-place for the rehabilitation, use the average dry density from as-built construction records or the average value measured during the field investigation. AASHTOWare Pavement ME Design default values for dry density represent the median maximum dry unit weight for specific material classifications. These default values need not be used for existing pavement layers that remain in-place for rehabilitation without confirming those values during the field investigation.
- **Moisture Content**—For new materials, the optimum moisture content using the compaction effort specified for the project, or the average moisture content measured on previous construction projects for a similar material is recommended for use as the input value. For existing pavement layers that will remain in-place for the rehabilitation, use the average moisture content measured during the field investigation. AASHTOWare Pavement ME Design default values for moisture content represent the median optimum moisture content for specific material classifications. These default values need not be used for existing layers remaining in-place without confirming those values during the field investigation.
- **Poisson's Ratio**—Use the default values provided in AASHTOWare Pavement ME Design, unless the designer has test data for using different values.
- **Resilient Modulus**—For new materials, use input Levels 2 or 3, unless the agency has a library of test results. Material properties needed for input Levels 2 and 3 include gradation, classification, Atterberg limits, moisture content, and dry density. The resilient modulus for the unbound layers and foundation may also be estimated from the CBR test (AASHTO T 193) or the *R*-Value test (AASHTO T 190).

If resilient modulus tests are available in a library of materials information and data, the designer could use the average value for the in-place material. The resilient modulus may be estimated based on equivalent stress states (*28, 29*). If input Level 3 is used to estimate the resilient modulus from classification tests, these modulus values represent the optimum moisture content and dry density (refer to Table 10-10). Those default values will need to be adjusted if the in-place layer deviates from the optimum moisture content and maximum dry unit weight, as defined by AASHTO T 180 at the time of construction. Adjustments for lower or higher moisture contents and dry densities can be made using the regression equations derived from the LTPP resilient modulus test results (*27*).

For existing unbound layers, use backcalculated modulus values from the FWD deflection basins for estimating the resilient modulus. As noted above, the backcalculated elastic modulus values need to be adjusted to laboratory conditions as input to AASHTOWare Pavement ME Design. However, results from DCP tests on the in-place materials may be used when FWD deflection basin tests have not been performed or were found to be highly variable with large errors to the measured deflection basins.

- **Saturated Hydraulic Conductivity**—For new and existing unbound layers, AASHTO T 215 may be used to measure this input parameter. However, all calibration work completed for version 1.0 of the software was completed using the default values included in the AASHTOWare Pavement ME Design software. Use of these default values is recommended.
- **Soil Water Characteristics Curve Parameters**—For new and existing unbound layers, there are AASHTO test standards that may be used to measure these input parameters for predicting the change in moisture content of the unbound layers over time. However, all calibration work completed for version 1.0 was completed using the default values included in the AASHTOWare Pavement ME Designsoftware. Use of these default values is recommended.

CHAPTER 11

Pavement Design Strategies

The MEPDG design process requires the selection of a trial design with all inputs defined. As noted earlier, the initial trial design may be determined using the *Guide for Design of Pavement Structures* (AASHTO, 1993), other M-E-based design procedures, a design catalog, or the user simply identifying the design features and layer thicknesses. This section provides guidance to the designer in developing the initial pavement design strategy for the site conditions and describes new or reconstructed pavement design strategies for flexible and rigid pavements. The designer is referred back to Chapter 3 to ensure that the design strategy selected and prepared for analysis is consistent with those calibrated globally or locally in accordance with AASHTOWare Pavement ME Design software.

11.1 NEW FLEXIBLE PAVEMENT DESIGN STRATEGIES—DEVELOPING THE INITIAL TRIAL DESIGN

The MEPDG flexible pavement design procedure allows a wide variety of HMA mixtures, aggregate base layers, and foundation improvements. Specific types of flexible pavement systems that may be analyzed include conventional flexible sections, deep strength sections, full-depth sections, and semi-rigid sections (refer to Figure 3-1 under Section 3.3). The definition for each of these pavement systems was included in Chapter 3.

In setting up an initial new design strategy for flexible pavements, the designer should simulate the pavement structure and foundation as detailed as possible, and then combine layers, as needed. It is recommended that the designer start with the fewest layers as possible to decrease the amount of inputs and time needed to estimate those inputs. Although more than 10 layers may be included in the trial design, the designer needs to limit the number of layer to no more than 7 to begin the design iteration process—3 HMA layers, an unbound aggregate base, a stabilized subgrade or improved embankment, the subgrade layer, and a rigid layer, if present.

The designer could identify the types of layers and materials to be included in the trial design, and then decide on the inputs for the project site. The following sections provide some simple rules to start developing the design strategy.

11.1.1 Should the Subgrade Soil be Strengthened/Improved?

The designer needs to evaluate the boring logs and test results prepared from the subsurface or field investigation and determine the subsurface soil strata—the different types of soils, their stiffness, and their thickness (refer to Section 8.3). If different soil strata are located with significantly different resilient modulus values along the project, those layers could be included as different soil layers. For example, a wet silty–sandy clay strata with a resilient modulus less than 8,000 psi overlying an over-consolidated, dense clay strata with a resilient modulus exceeding 25,000 psi.

An important step of the new flexible pavement design strategy is to begin with a good foundation for the pavement layers. Proper treatment of problem soil conditions and the preparation of the foundation layer are important to ensure good performance of flexible pavements. It is fundamental to have a strong foundation that provides proper support of the flexible pavement. A strong foundation will result in thinner paving layers. AASHTOWare Pavement ME Design does not directly predict the increase in roughness or IRI caused by expansive, frost susceptible, and collapsible soils. If these types of problem soils are encountered, treatments to minimize their long-term effects on flexible pavements need to be included in the design strategy.

The designer needs to review the results from the subsurface investigation (refer to Chapter 8) and provide a foundation layer with a resilient modulus of at least 10,000 psi for supporting any unbound aggregate layer. If the subgrade has a resilient modulus less than 10,000 psi, the designer could consider improving or strengthening the subgrade soils. Different options that may be used depending on the conditions encountered include using select embankment materials, stabilizing the subgrade soil, removing and replacing weak soils, and/or adding subsurface drainage layers. Figure 11-1 is a flowchart of some options that may be considered, depending on the thickness and condition of the problem soils encountered along the project.

More importantly, AASHTOWare Pavement ME Design does not predict or consider the lateral flow of subsurface water. If subsurface lateral flow is expected based on the experience of the designer in the area or from observations made during the subsurface investigation, subsurface drainage systems need to be considered to prevent water from saturating the pavement layers and foundation. Saturation of the paving materials and foundation will significantly decrease the resilient modulus of the unbound materials and soils. AASHTOWare Pavement ME Design only predicts the effects of water moving upward into the pavement layers from ground water tables located close to the surface.

In addition, filter fabrics, geotextiles, and geogrids (for example, AASHTO M 288) cannot be directly simulated in the pavement structure. Agencies that routinely use these materials in their standard design sections or strategies need to determine their benefit or effect through the local calibration process for each performance indicator (distresses and smoothness) (*12*, *14*).

11.1.2 Is a Rigid Layer or Water Table Present?

A rigid or apparent rigid layer is defined as the lower soil stratum that has a high resilient or elastic modulus (greater than 100,000 psi). A rigid layer may consist of bedrock, severely weathered bedrock, hard-pan, sandstone, shale, or even over-consolidated clays.

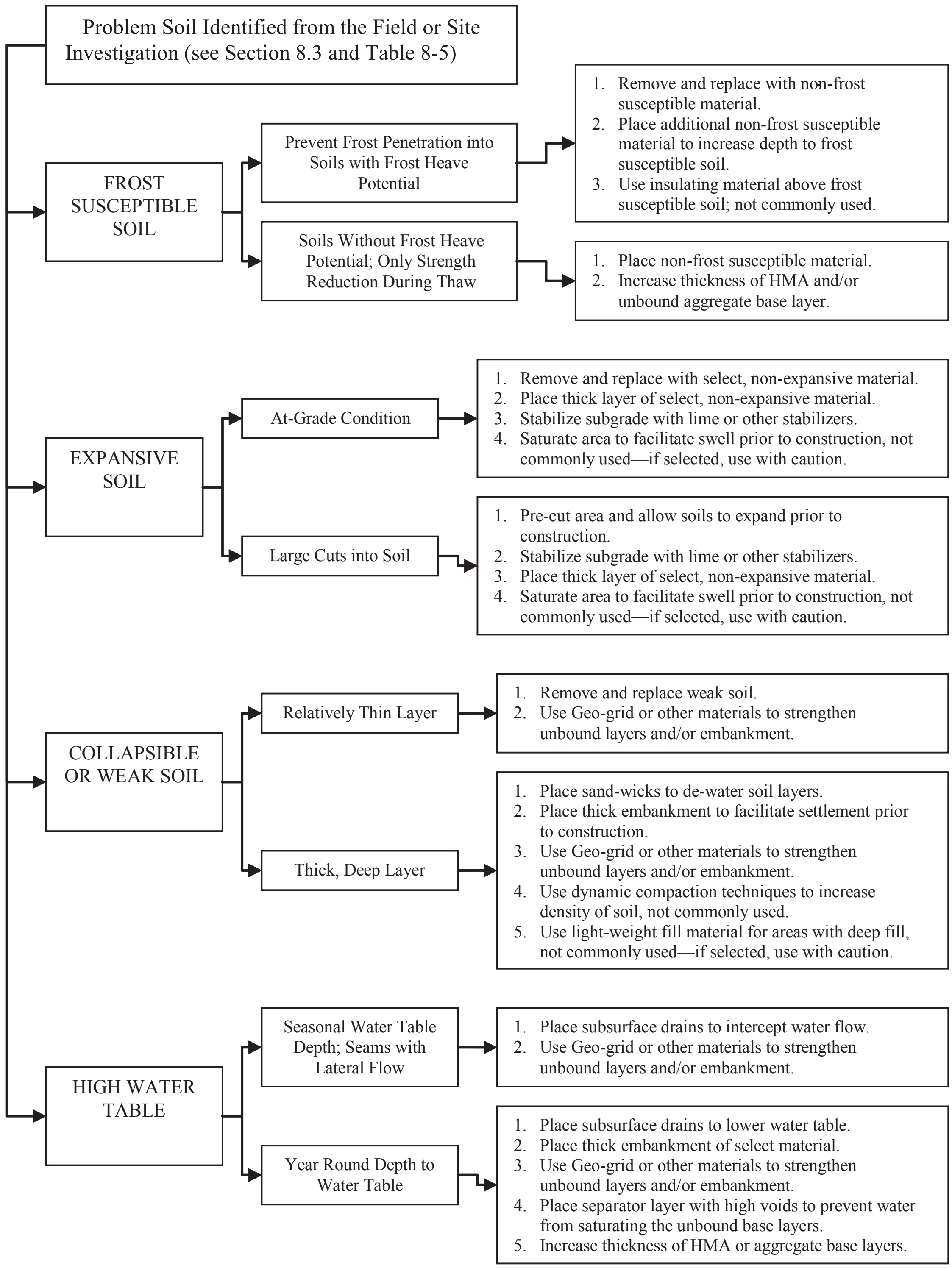

Figure 11-1. Flow Chart for Selecting Some Options to Minimize the Effect of Problem Soils on Pavement Performance

If a rigid layer is known to exist along the project boundaries, that layer could be included in the analysis. When a rigid layer is simulated, however, AASHTOWare Pavement ME Design limits the thickness of the last subgrade layer to no more than 100 in. The designer may need to use multiple subgrade layers when the depth to bedrock exceeds 100 in. In some areas, multiple-thin strata of rock or hard-pan layers will be encountered near the surface. The designer could enter an equivalent elastic modulus for this condition and assume that it is bedrock.

Another important point when a rigid layer or rock outcropping is known to exist is the possibility of subsurface water flow above the rigid layer. The designer could have considered this in setting up the subsurface investigation plan for sites with rock outcroppings and rigid layers near the surface. The designer could evaluate the results from the subsurface investigation to determine whether a subsurface drainage system is needed to quickly remove and/or intercept subsurface water flow. This design feature does not relate to the surface infiltration of rainfall water.

When a water table is located near the surface (within 5 ft), a subsurface drainage system is recommended as part of the design strategy (22). The depth to a water table that is entered into AASHTOWare Pavement ME Design software is the depth below the final pavement surface. The designer has the option to enter an annual depth to the water table or seasonal water table depths. The average annual depth could be used, unless the designer has historical data to determine the seasonal fluctuations of the water table depth. If a subsurface drainage system is used to lower that water table, that lower depth could be entered into the program, not the depth measured during the subsurface investigation.

11.1.3 Compacted Embankment or Improved Subgrade Layer Present?

The designer could divide the subgrade into two layers, especially when bedrock or other hard soils are not encountered. Most new alignment projects or new construction projects require that the surface of the subgrade be scarified and compacted after all vegetation has been removed and the elevation has been rough cut. The designer could consider simulating the compacted subgrade as a separate layer, as long as that layer is compacted to a specified density and moisture content that are based on laboratory prepared moisture-density relationships. When used in the trial design, this layer needs to be a minimum of 8 in. thick.

The default values included in AASHTOWare Pavement ME Design for resilient modulus of unbound materials and soils (refer to Section 10.5) represent the material placed at optimum moisture content and compacted to its maximum dry unit weight (as defined by AASHTO T 180). If an embankment, improved subgrade, or other material is placed and compacted to a different moisture content and dry unit weight, the default values for resilient modulus need not be used. The design resilient modulus could be determined from an agency's historical database, repeated load resilient modulus tests (performed on test specimens compacted to the agency's specifications), other strength tests (CBR and R-Value), or estimated from regression equations (for example, those developed from the LTPP resilient modulus database [27]).

11.1.4 Should a Drainage Layer be Included in the Design Strategy?

The use of a drainage system to remove surface water infiltration is dependent on the user's standard design practice. It is recommended to avoid water accumulation within the pavement structure. Water may significantly weaken aggregate base layers and the subgrade soil, and result in stripping of HMA layers. AASHTOWare Pavement ME Design assumes that all water-related problems will be addressed via the materials and construction specifications, and/or inclusion of subsurface drainage features in the design strategy. NHI Course 131026 provides guidelines and recommendations for the design and construction of subsurface drainage features (*22*).

The value and benefit of a drainage layer (either an asphalt treated permeable base or permeable aggregate base layer) beneath the dense-graded HMA layers is debatable. If an asphalt treated permeable base drainage layer is used directly below the last dense-graded HMA layer, the ATPB needs to be treated as a high quality, crushed stone base layer (refer to Sections 3.5 and 5.3.3). The equivalent annual modulus for an ATPB (high-quality aggregate base) that has been used is 65,000 to 75,000 psi. The minimum thickness of an ATPB layer should be 3 in.

These edge drains need to be inspected after placement and must be maintained over time to ensure positive drainage. The inspection at construction and over time is no different than required for new pavement construction. Mini-cameras may be used to facilitate the inspection and maintenance needs of edge drains. If an agency or owner does not have some type of periodic inspection and maintenance program for these drainage layers and edge drains, the designer could consider other design options, and accordingly reduce the strength of the foundation and unbound layers.

11.1.5 Use of a Stabilized Subgrade for Structural Design or a Construction Platform?

Lime and/or lime-fly ash stabilized bases should be considered a separate layer if they are sufficiently stabilized to provide structural support and these properties can be measured through coring or laboratory sample preparation. Lime is primarily used for controlling swelling and frost heave. If these layers are engineered to provide structural support and have a sufficient amount of stabilizer mixed in with the soil, they need to be treated as a structural layer. Under this case, they could be treated as a material that is insensitive to moisture and the resilient modulus or stiffness of these layers can be held constant over time. The National Lime Association manual may be used for designing and placing a lime stabilized layer to provide structural support (*16*). If other stabilizers such as Portland cement and lime-fly ash combinations are used, other manuals could be followed for designing and placing stabilized subgrade layers (*24*).

On the other hand, when a stabilized subgrade is used as a construction platform for compacting other paving layers, only a small amount of lime or lime-fly ash is added and mixed with the soil. For this case, these layers could be treated as unbound soils. In addition, if these materials are not "engineered" to provide long-term strength and durability, they could also be considered as an unbound material and possibly combined with the upper granular layer.

11.1.6 Should an Aggregate Base/Subbase Layer Be Placed?

Unbound aggregate or granular base layers are commonly used in flexible pavement construction. In most cases, the number of unbound granular layers need not exceed two, especially when one of those layers is thick (more than 18 in.). Sand and other soil-aggregate layers could be simulated separately from crushed stone or crushed aggregate base materials, because the resilient modulus of these materials will be significantly different.

When aggregate or granular base/subbase layers are used, the resilient modulus of these layers is dependent on the resilient modulus of the supporting layers. As a rule of thumb, the ratio of resilient modulus of the granular layer to the resilient modulus of the supporting layers should be kept to a maximum of 3 to avoid decompaction of the supporting layer. This rule of thumb may apply to all unbound layers. Figure 11-2 may be used to estimate the maximum resilient modulus of an unbound layer that depends on its thickness and the resilient modulus of the supporting layers (5).

11.1.7 HMA Layers—What Type and How Many?

AASHTOWare Pavement ME Design is limited to three layers of asphalt. As for the unbound materials, similar HMA mixtures could be combined into one layer. Thin layers (less than 1.5 in. in thickness) could be combined with other layers. The minimum lift or layer thickness used for construction may be four times the nominal maximum aggregate size of the HMA mixture.

More importantly, thin wearing courses of a plant seal mix, porous friction course, open-graded friction course and other similar mixtures could be combined with the next layer beneath the wearing surface. The low temperature cracking and load related top-down (longitudinal) cracking models use the properties of the wearing surface in predicting the length of transverse and longitudinal cracks throughout the HMA layers.

Similarly, the alligator cracking model takes the properties of the lowest HMA layer and predicts the percent of total lane area with alligator cracking. As a result, the designer needs to carefully consider the properties being entered into AASHTOWare Pavement ME Design software for the lowest HMA layer and HMA wearing surface.

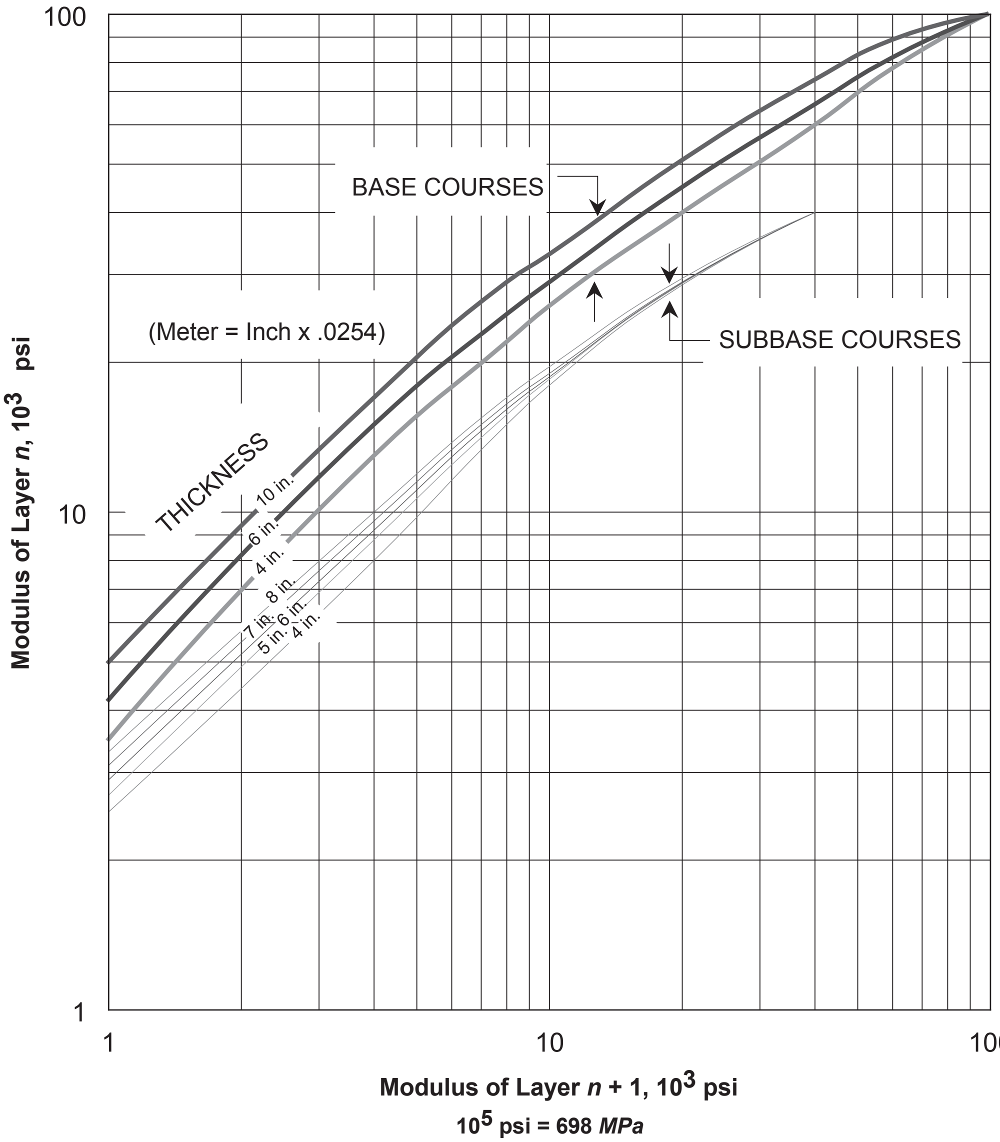

Figure 11-2. Limiting Modulus Criteria of Unbound Aggregate Base and Subbase Layers

When multiple layers are combined for the trial design, the volumetric properties (air voids, effective asphalt content, gradation, unit weight, and VFA) entered into AASHTOWare Pavement ME Design software need to represent weighted average values based on the layer thickness of the layers that are combined. A wearing surface greater than 1.5 in. in thickness that has different PG asphalt than the underlying HMA layer needs to be considered as a separate layer. Similarly, a dense-graded HMA base layer (the lowest HMA layer) that is more than 3 in. thick could be considered as a separate layer. It is recommended that all other layers be combined into an intermediate layer.

If an APTB layer with high air voids (typically greater than 15 percent voids) is included as an HMA layer, the high air voids will significantly increase the amount of fatigue cracking of the pavement structure. As a remedy, the designer may consider modeling the APTB as a high-quality aggregate layer.

11.1.8 What Initial IRI Value Should Be Used?

An initial IRI value is required for each pavement strategy or trial design considered. The initial IRI value could be taken from previous years' construction acceptance records, if available. Not all agencies, however, use IRI in accepting the pavement related to smoothness criteria. The following provides some recommendations for those agencies or users that do not use IRI as a basis for accepting the final surface.

Table 11-1. General IRI Recommendation

Pavement Design Strategy	Initial IRI, in./mi	
	IRI Included as an Acceptance Test	IRI Excluded from Acceptance Test
Conventional Flexible Pavements	65	80
Deep -Strength Flexible Pavements	60	70
Full -Depth HMA Pavements	60	70
Semi -Rigid Pavements	65	80

Note: The values listed above are higher than for those agencies that typically use IRI for acceptance, because the contractors would have little incentives to ensure a smooth ride surface, as measured by IRI.

11.2 NEW RIGID PAVEMENT DESIGN STRATEGIES— DEVELOPING THE INITIAL TRIAL DESIGN

11.2.1 Structure—Trial Layer Type, Thickness, and Design Features

New or reconstructed rigid pavement types include JPCP and CRCP, as the surfacing layer.

- JPCP is defined in Section 3.4. This pavement type is the most widely constructed rigid pavement in the United States and in the world. It is used for all pavement applications including low-volume roads, urban streets, and heavily trafficked highways. A major national calibration was conducted that included hundreds of sections throughout the United States. Reasonable distress and IRI models were developed and calibrated. Local agency validation of the distress models and local consideration of design inputs is desirable during implementation.

- CRCP is defined in Section 3.4. This pavement type is used extensively by several states and other countries. It is used primarily for heavily trafficked highways but has been used for lower volume roads as well. A major national calibration was conducted that included over a hundred sections throughout the United States. Reasonable distress and IRI models were developed and calibrated. Local agency validation of the distress models and local consideration of design inputs is desirable during implementation.

The concrete slab is usually placed over one or more sublayers but may be placed directly on a prepared subgrade for low-volume roads. It is critical that the CRCP bases be stable over time. Sublayers include a wide variety of materials and layering and may also include permeable drainage layers. Note that the base course is defined as the layer directly beneath the PCC slab and subbase layers are below the base layer.

- **Dense Graded Base Course—**Asphalt stabilized, cement stabilized, lean concrete, and unbound granular can be considered. Many varieties of layer characteristics may be considered but the designer must enter appropriate structural, thermal, and hydraulic parameters for these layers. See Chapter 5 for recommended inputs.
- **Permeable (Drainage Layer) Base Course—**Asphalt stabilized, cement stabilized, and unbound granular permeable layers may be considered.
  - A permeable asphalt stabilized base may be modeled in two ways:
    - Select asphalt base and asphalt permeable base. This choice requires entering a high air void content (e.g., specifying 15–20 percent air typically results in reasonable E_{HMA} dynamic seasonal value).
    - Select stabilized base and cement stabilized material. This choice requires entering an appropriate modulus for a permeable asphalt stabilized base that does not change over temperature or time.
  - A permeable cement stabilized base may be modeled by selecting stabilized base and cement stabilized. This choice requires entering an appropriate modulus that does not change over time.
  - A permeable asphalt base may be modeled by selecting a high-quality aggregate layer. This choice requires entering appropriate inputs for gradation and other parameters.
  - A permeable unbound aggregate base may be modeled by selecting unbound base and permeable aggregate material. This choice requires entering appropriate inputs for gradation and other parameters.
  - Sandwich section—if an unbound permeable aggregate layer is placed between the PCC slab and an impermeable layer (e.g., dense HMA or lean concrete) no drainage analysis will occur in the permeable layer. The user needs to select unbound base and permeable aggregate material and input an appropriate constant modulus which will not change over time or with moisture content.
- **Subbase Layers**—Asphalt stabilized, compacted RAP, cement stabilized, lime stabilized, lime fly ash, lime cement fly ash, soil cement, and unbound granular materials. Many varieties of layer characteristics may be considered but the designer needs to enter appropriate structural, thermal, and hydraulic parameters for these layers.
- **Embankment and Natural Soil**—Materials are classified according to the AASHTO and unified procedure and require appropriate structural, thermal, and hydraulic parameters. See Chapter 5 for recommended inputs.
- **Bedrock**—Bedrock may consist of massive and continuous bedrock and highly fractured and weathered bedrock. Recommended modulus values are provided in Chapter 5 for both of these types of conditions.

11.2.2 JPCP Design

There are several key design inputs for JPCP for which recommendations are provided in this section.

- **Contact Friction** (Between JPCP and Base Course)—The time over which full contact friction exists between the PCC slab and the underlying layer (usually the base course) is an input. This factor is usually significant in affecting cracking of the JPCP in that a monolithic slab/base structure is obtained when full friction exists at the interface. While the actual friction may often vary between zero and full or no slippage, the global calibration results for hundreds of JPCP test sections indicated that full contact friction existed over the life of the pavements for all base types. Accurate amounts of cracking was predicted when full friction with the base was assumed, except for CTB or lean concrete bases when extraordinary efforts were made to debond the slab from the base. For this condition, the months of full contact friction was found to be much less; zero to 15 years to match the observed cracking. A rapid increase in transverse cracking occurred within the life for some of the JPCP sections, which could be explained by a zero friction interface with the base course.

Thus, it is recommended that the designer set the "months to full contact friction" between the JPCP and the base course equal to the design life of the pavement for unbound aggregate, asphalt stabilized, and cementitious stabilized base courses. The only exception to this recommendation is when extraordinary efforts are made to debond a cementitious base course from the JPCP.

- **Tied Concrete Shoulder**—The long-term LTE must be input. The lane shoulder LTE is defined as the ratio of deflection of the unloaded side to the loaded side of the joint multiplied by 100. The greater the LTE the greater the reduction in deflections and stresses in the concrete slab. Recommended long-term lane/shoulder LTE, in descending order of benefit, are as follows:
  - Monolithically placed and tied with deformed bars traffic lane and shoulder: 50 to 70 percent. During calibration, a number of test sections were modeled with 70 percent LTE to help explain low levels of cracking and faulting.
  - Separately placed and tied with deformed bars traffic lane and shoulder: 30 to 50 percent. During calibration, a typical value of 40 percent was used unless knowledge concerning placement was know.
  - Untied concrete shoulders or other shoulder types were modeled with zero LTE during calibration.
- **Joint LTE**—JPCP may be designed with or without dowel bars at the transverse joints. The key inputs are dowel diameter and spacing. The key performance output is joint faulting which is subjected to a limiting criteria selected by the designer. Sensitivity analysis of the program shows that the use of dowels of sufficient size may virtually eliminate joint faulting as a problem.
  - Dowel trial diameter of 1/8 the slab thickness (e.g., a 12-in. slab would have a 1.5-in. dowel diameter). Diameter may vary from about 1 (minimum) to 1.75 in.
  - Dowel trial spacing of 12 in. is recommended, but the spacing may vary from 10 to 14 in.
- **Joint Spacing**—This factor has a very significant effect on JPCP cracking, joint faulting, and IRI. The shorter the spacing, the less faulting and cracking occur. However, this leads to increased construction costs so a balance is recommended. The natural crack spacing is dependent on the slab friction with the base layer. Crack widths must remain below 20 mils to allow effective load transfer from aggregate interlock. Projects with bedrock near the surface may result in very stiff foundations which may require a shortening of the joints spacing to avoid cracking.

- **Joint Random Spacing**—If a JPCP has random spacing, each spacing could be run separately to estimate the amount of transverse cracking. The longest spacing will be the most critical. Project percent slabs cracked is then averaged from the results for the different joint spacing used.
- **Joint Skew**—Joint skewing is not recommended when dowels are used. However, if used, to account for the increase in effective joint spacing when joints are skewed, an extra 2 ft is added to the joint spacing. This will increase joint faulting and transverse cracking.
- **Base Erodability**—The potential for base or subbase erosion (layer directly beneath the PCC layer) has a significant impact on the initiation and propagation of pavement distress. The design input is the erodibility class, which is classified based on long-term erodability behavior of different base types as follows:
  - Class 1—Extremely erosion resistant materials.
  - Class 2—Very erosion resistant materials.
  - Class 3—Erosion resistant materials.
  - Class 4—Fairly erodible materials.
  - Class 5—Very erodible materials.
- **Set Temperature and Ultimate Shrinkage** (described under CRCP Design)—These factors affect JPCP in terms of joint opening which affects joint LTE and joint faulting in the same way that crack width and loss of LTE is affected in CRCP. Joint LTE over the design life is an output that could be examined and not allowed to be lower than about 90 percent.
- **Permanent Curl/Warp Effective Temperature Difference**—This input includes built-in temperature gradient at time of set plus effective gradient of moisture warping (dry on top and wet on bottom) plus any effect of long-term creep of the slab and settlement into the base. A value of –10°F was established as optimum to minimize cracking during the national calibration. This optimum temperature difference could be utilized unless local calibration shows different. Night-time construction and wet curing would reduce this factor in the same manner that extreme temperature changes and solar radiation during morning placement would increase this factor. It is recommended that this input be confirmed through local calibration since it significantly impacts the pavement service life.

11.2.3 CRCP Design

The performance of CRCP is highly dependent upon several factors. Recommendations for specific CRCP inputs are as follows:

- **Tied Concrete Shoulder**—The long-term load transfer across the lane/shoulder joint is modeled so that the impact of a tied shoulder may be considered in design. The user selects the type of shoulder under consideration under design features in AASHTOWare Pavement ME Design software and the program assigns the appropriate LTE:
  - Monolithically placed lane and shoulder and tied with deformed reinforcing bars.
  - Separately placed lane and shoulder and tied with deformed reinforcing bars.
  - Untied concrete shoulders or other shoulder types.

- **Bar Diameter**—Varies from #4 (0.500-in. diameter) to #9 (1.00-in.), typically. Heavier trafficked highways currently utilize #6 or #7 size deformed reinforcing bars. These are typically coated with epoxy in areas that use large amounts of deicing salts.
- **Trial Percentage of Longitudinal Reinforcement**—This parameter may vary from 0.50 to 1.00 percent. Climatic conditions affect the required amount with higher amounts in cold climates. As the amount of longitudinal reinforcement increases, crack spacing and width decrease. Crack LTE over time stays at higher and higher values which minimizes punchout development.
- **Reinforcement Depth**—Depth of reinforcing steel has a significant effect on holding the crack width tight at the top of the slab. A minimum depth of 3.5-in. and a maximum depth at the slab mid-depth is recommended. Placement of the steel above mid-depth will hold the cracks tighter which will reduce punchouts.
- **Crack Spacing**—Crack spacing is either input by the user if experience warrants, or may be calculated directly by a prediction model given in Chapter 5. The recommended range of spacing is 3 to 6 ft.
- **Base/Slab Friction Coefficient**—This friction coefficient varies by base type. Typical average values were established through matching crack spacing. Recommended values and ranges are as follows:

Table 11-2. Range and Median Slab/Base Friction Coefficients by Base Type

Subbase/Base type	Friction Coefficient (Low–Mean–High)
Fine-grained soil	0.5–1.1–2
Sand*	0.5–0.8–1
Granular	0.5–2.7–5.8
Lime-stabilized clay*	3–4.1–5.3
ATB	2.0–8.5–18.7
CTB	2.9–9.6–20.9
Soil cement	6.0–7.9–23
LCB	6.0–10.7–21.5
LCB not cured*	>36 (higher than LCB cured)

* Base type did not exist or not considered in calibration sections.

- **Set Temperature**—Set temperature is defined as the average concrete set temperature when the slab becomes a solid. It is either entered by the user or estimated from the following inputs: average of hourly ambient temperatures for month of construction and the cementitious materials content (used to calculate the zero stress temperature and ultimate shrinkage only). The set temperature is very significant for CRCP performance. The lower this temperature the tighter the transverse cracks will be over time and the lower the occurrence of punchouts. Thus, the month of construction affects greatly the zero stress temperature of the concrete.
- **Permanent Curl and Warp**—Permanent curl/warp effective temperature difference (same recommendations as JPCP).
- **Ultimate Shrinkage**—Ultimate shrinkage at 40 percent relative humidity (%) (R. H.) is either input by the user or estimated from models provided in Chapter 5. It depends on curing type (curing

compound or water cure, cement type (I, II, III), water content (through w/c ratio), and 28-day compressive strength. To minimize ultimate shrinkage, use Type II cement, cure with water, reduce water content, and increase concrete strength in general and within reasonable limits on each of these factors.

- **Crack Width**—Crack width is estimated over the entire design life and is a very critical factor. It initially depends on the temperature of construction. The user either selects the expected month of construction which then is used to estimate the zero-stress temperature of the concrete. The ultimate shrinkage of the concrete also controls crack width over time. Thus, anything that will reduce shrinkage will be desirable for CRCP.
- **Crack LTE**—The crack LTE is initially 100 percent during the first 20 years or so but then could deteriorate over time and loadings to an unacceptable level. As LTE decreases the chance of punchouts increases as critical bending stress at the top of the CRCP increases. Crack LTE depends greatly on crack width over time but also on the number of heavy axles crossing the crack and causing vertical sheer and potential damage. Thus, keeping LTE above 90 or 95 percent is an important criterion because this will virtually ensure that minimal or no punchouts will occur.
- **Erosion and Loss of Support Along Slab Edge**—This parameter depends on several inputs, particularly base type and quality.
  - HMA base: volumetric asphalt content.
  - CTB/LCB: modulus of elasticity, E_c.
  - Unbound granular base: fines content (minus #200 sieve).
  - Annual precipitation.
  - Type and quality of subbase/subgrade (strength, fines).

Erosion is calculated for 10 years but uniformly accumulated year by year with a practical maximum amount.

11.2.4 Initial Surface Smoothness

The initial IRI of JPCP and CRCP falls within a range of 50 to 100 in./mi with a typical value of 63 in./mi. This value could be adjusted to that typically obtained by the local highway agency for these pavements.

11.2.5 Narrow or Widened Slabs

This input is commonly called "Lane Width," but it is actually slab width. The paint strip marking the lane edge is always striped at the conventional width of 12 ft. Design alternatives include the use of a conventional slab width of 12 ft or to widen the slab by 0.5 to 3 ft. It is also possible to analyze a narrower slab such as 10 or 11 ft. The width controls the closeness of the edge of the tires traversing the JPCP and CRCP. The farther away from the edge, the lower the fatigue damage along the edge which results in transverse cracking.

- JPCP slab width is assumed to be 12 ft unless the box is checked and a different slab width is entered. This value may range from greater than 12 to 15 ft with the assumption that the paint stripe is painted at the 12-ft width. Wider slabs can be used for outside truck lanes to move the wheel path away from the edges and corner of the slab to reduce the stresses in these areas. As little as a one foot widening has a significant effect. However, the potential for longitudinal cracking is increased with wider slabs, especially in thinner slabs (<10 inches), although AASHTOWare Pavement ME Design does not predict longitudinal cracking. When a widened slab is used, fatigue damage is also calculated at the inside longitudinal joint edge (the joint between lanes) where LTE is set at 70 percent. If a narrower lane width is of interest, this can be approximately handled by using a 12-ft-wide slab but reducing the mean offset distance from slab edge to outside of tire (e.g., instead of 18-in. typical, it would be reduced by 12-in. to 6-in. for a 11-ft-wide slab). The structural advantage to the widened slab design does have a limit in AASHTOWare Pavement ME Design. When the outside wheel path reaches a certain distance from the outside slab edge, the inside wheel path proximity to the inside slab edge becomes the more critical source of predicted fatigue damage.

- CRCP slab width is assumed to be 12 ft, and there is no formal way to increase its width. An approximate way is to increase the offset distance from the lane edge to the truck tire by the amount of slab widening. Thus, if a lane is widened by 12 in., the mean tire offset would be 18 + 12 = 30 in. A narrow lane would be handled the same as JPCP.

CHAPTER 12

Rehabilitation Design Strategies

12.1 GENERAL OVERVIEW OF REHABILITATION DESIGN USING AASHTOWARE PAVEMENT ME DESIGN

A feasible rehabilitation strategy is one that addresses the cause of the pavement distress and deterioration and is effective in both repairing it and preventing or minimizing its reoccurrence. AASHTOWare Pavement ME Design has the capability to evaluate a wide range of rehabilitation designs for flexible, rigid, and composite pavements. MEPDG rehabilitation design process is an iterative, hands-on approach by the designer—starting with a trial rehabilitation strategy. Similar to developing the initial trial design for new pavements, the trial rehabilitation design may be initially determined using a rehabilitation design catalog, or an agency specific design procedure. AASHTOWare Pavement ME Design software may then be used to analyze the trial design to ensure that it will meet the user's performance expectations.

A considerable amount of analysis and engineering judgment is required when determining specific treatments required to design a feasible rehabilitation strategy for a given pavement condition (23). AASHTOWare Pavement ME Design considers four major strategies, as listed below, which may be applied singly or in combination to obtain an effective rehabilitation plan based on the pavement condition that was defined under Chapter 8.

- Reconstruction without lane additions—this strategy is considered under new pavement design strategies.
- Reconstruction with lane additions—this strategy is considered under new pavement design strategies.
- Structural overlay, which may include removal and replacement of selected pavement layers.
- Non-structural overlay.
- Restoration without overlays.

The MEPDG provides detailed guidance on the use and design of rehabilitation strategies, depending on the type and condition of the existing pavement, and provides specific details on the use of material specific overlays for existing flexible and rigid pavements. This section provides an overview of strategies for the rehabilitation of existing flexible, rigid, and composite pavements. Figure 12-1 shows the steps that are suggested for use in determining a preferred rehabilitation strategy.

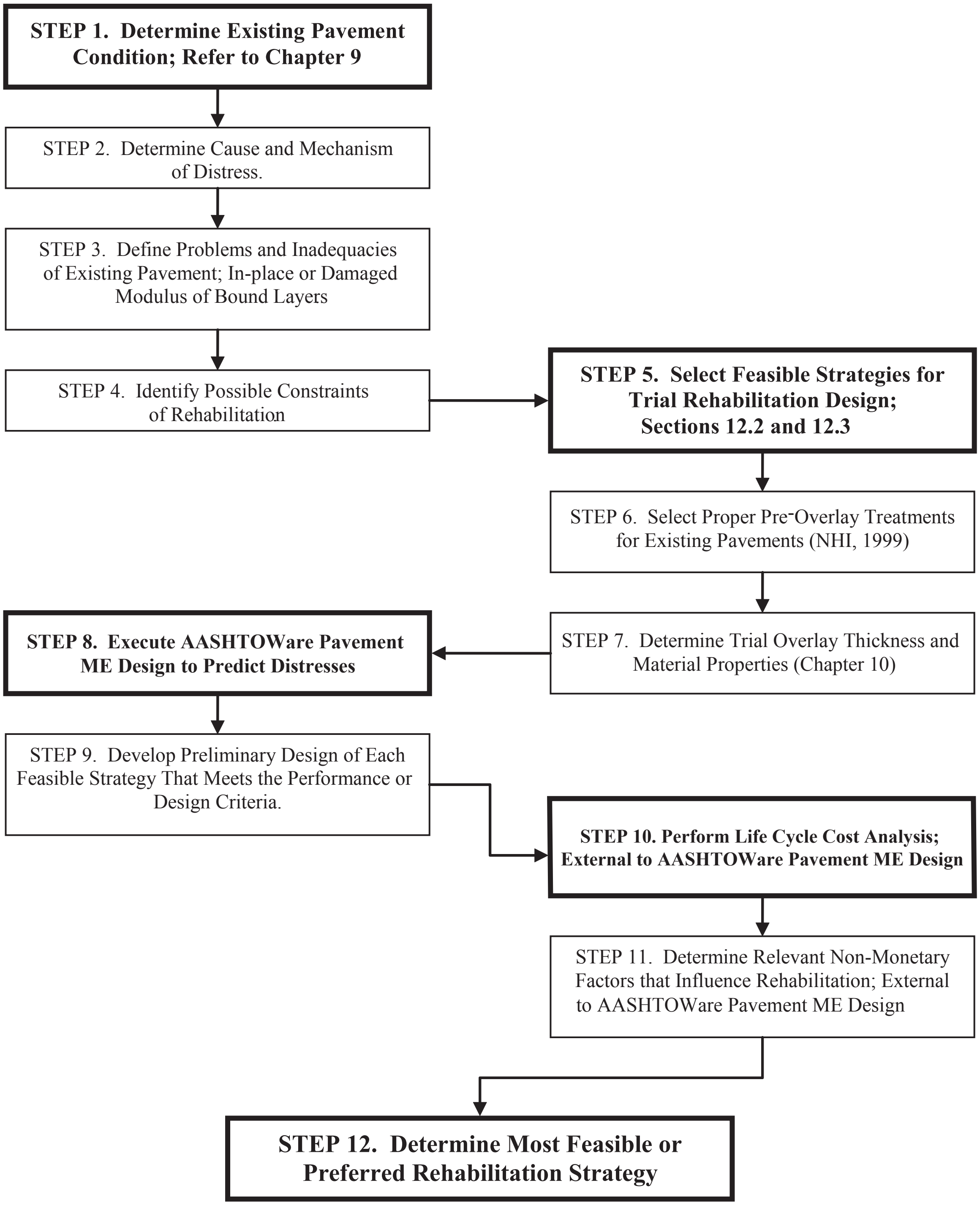

Figure 12-1. Steps for Determining a Preferred Rehabilitation Strategy

12.2 REHABILITATION DESIGN WITH HMA OVERLAYS

12.2.1 Overview

The MEPDG includes specific details for selecting and designing HMA overlays to improve the surface condition or to increase the structural capacity of the following pavements (refer to Figure 3-2 under Section 3.3).

- HMA overlays of existing HMA-surfaced pavements; both flexible and semi-rigid.
- HMA overlays of existing PCC pavements that has received fractured slab treatments; crack and seat, break and seat, and rubblization.
- HMA overlays of existing intact PCC pavements (JPCP and CRCP), including composite pavements or second overlays of original PCC pavements.

Figure 12-2 presents a generalized flow chart for pavement rehabilitation with HMA overlays of HMA-surfaced flexible, semi-rigid, or composite pavements, fractured PCC pavements and intact PCC pavements.

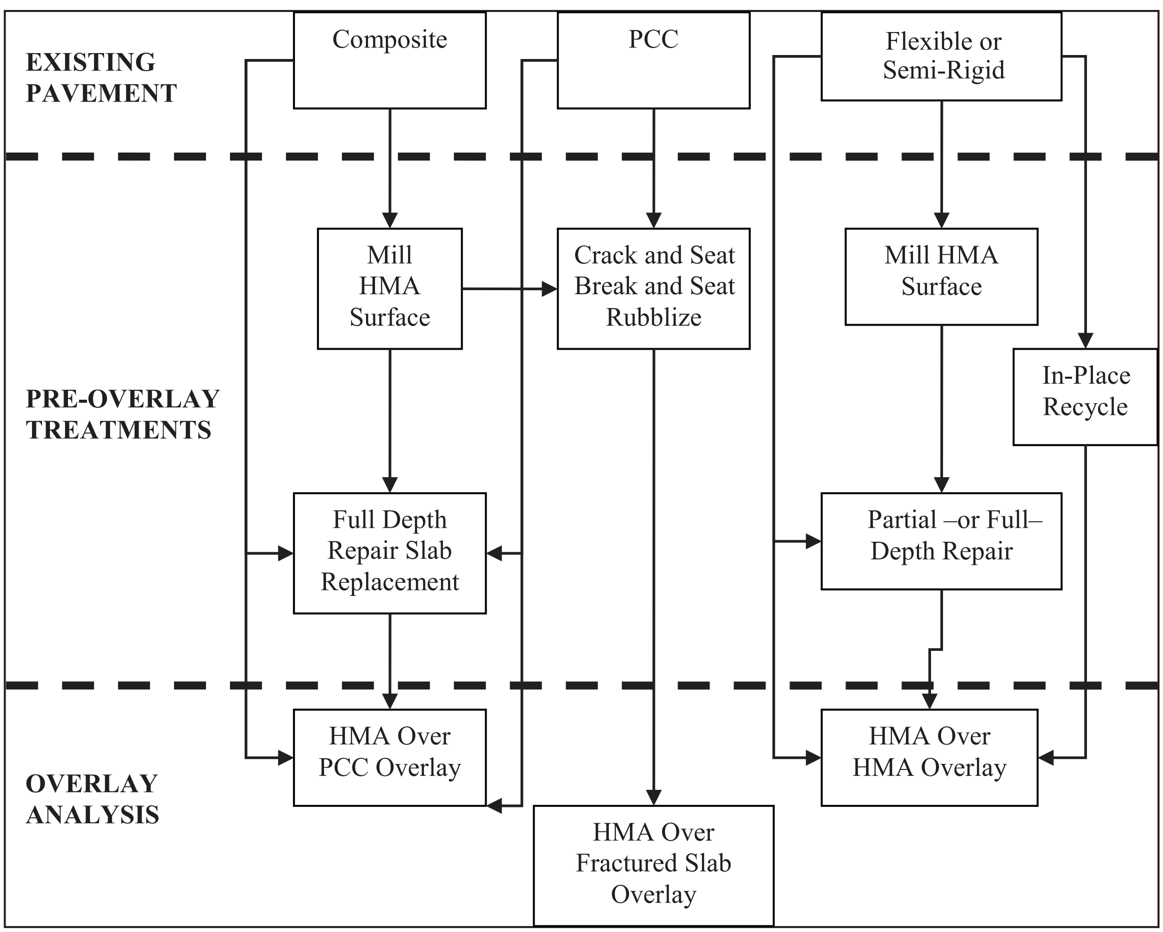

Figure 12-2. Flow Chart of Rehabilitation Design Options Using HMA Overlays

12.2.2 HMA Overlay Analyses and Trial Rehabilitation Design

AASHTOWare Pavement ME Design has the capability of analyzing existing PCC pavements may be either an HMA over PCC analysis or an HMA over fractured slab analysis depending on whether or not crack and seat, break and seat, or rubblization techniques are applied to the existing PCC pavement. Existing composite pavements may result in either an HMA over PCC analysis or an HMA over fractured slab analysis depending on whether or not the existing HMA surface is removed and the underlying PCC pavement is fractured.

In the AASHTOWare Pavement ME Design software, the HMA over PCC analysis considers continued damage of the PCC slab under the HMA overlay using the rigid pavement performance models presented in Chapter 5 and Section 12.2.8. The three overlay analyses in the software also provide the capability to address reflection cracking of joints and cracks in PCC pavements and thermal and load associated cracking in HMA surfaced pavements. However, it needs to be noted that the reflection cracking models incorporated in AASHTOWare Pavement ME Design were based strictly on empirical observations and were not a result of rigorous M-E analyses. Finally, the predicted distresses are linked to estimates of IRI to form a functional performance criterion that may be considered along with the specific distresses in the design-analysis process.

The maximum number of overlay layers that may be specified is four. This includes up to three HMA layers, and one unbound or chemically stabilized layer. The total number of layers of the existing pavement and the overlay is limited to 14. For the initial design, however, it is suggested that the total number of layers be limited to no more than eight to reduce the number of required inputs and run time.

12.2.3 Determine Condition of Existing Pavement

A critical element for determining the HMA overlay design features and thickness is the characterization of the existing pavement, including determination of the damaged modulus of the existing bound layers. General recommendations for evaluating the existing pavement for rehabilitation were included in Chapter 9. As for new pavement designs, all properties of the existing and new pavement layers need to be representative of the conditions expected right after rehabilitation—when the roadway is opened to traffic.

Table 9-8 in Chapter 9 provided general recommendations for assessing the current condition of flexible, semi-rigid, composite, and HMA overlaid pavements, while Table 9-2 provided the pavement evaluation activities for the different input levels. For input Level 3, a generalized rating for the existing pavement is an input to AASHTOWare Pavement ME Design. The designer has five options to select from: Excellent, Good, Fair, Poor, and Very Poor. Table 12-1 provides a definition of the surface condition and summarizes the rehabilitation options suggested for each of these general ratings. For input Level 1, cores and trenches are used to determine the amount of rutting within each paving layer and whether any cracks that have occurred initiated at the surface or bottom of the HMA layers. For input Level 2, cores are used to estimate the amount of rutting within each layer and determine where any load related cracks initiated.

Table 12-1. Definitions of Surface Condition for Input Level 3 Pavement Condition Ratings and Suggested Rehabilitation Options

Overall Condition (Table 9-1)	General Pavement Condition Rating; Input Level 3		Rehabilitation Options to Consider (With or Without Pre-Overlay Treatments; Section 12.2.4)
Adequate (Has Remaining Life)	Excellent	No cracking, minor rutting, and/or minor mixture-related distresses (e.g., raveling); little to no surface distortions or roughness.	• Surface repairs without overlays (not analyzed with AASHTOWare Pavement ME Design). • Pavement preservation strategy (not analyzed with AASHTOWare Pavement ME Design). • Non-structural overlay. • Overlay designed for future truck traffic levels.
	Good	Limited load –and/or non-load– related cracking, minor to moderate rutting, and/or moderate mixture-related distresses; some surface distortions and roughness.	• Pavement preservation strategy (not analyzed with the AASHTOWare Pavement ME Design). • Overlays designed for future truck traffic levels, with or without milling and surface repairs.
Marginal (May or May Not Have Remaining Life)	Fair	Moderate load and/or non-load related cracking, moderate rutting, moderate amounts of mixture-related distresses, and/or some roughness (IRI > 120 in./mi).	Pre-Overlay Treatments Recommended. • Structural overlay, with or without milling and surface repairs. • Remove and replace surface layer prior to overlay. • In-place recycling prior to overlay.
Inadequate (No Remaining Life)	Poor	Extensive non-load-related cracking, moderate load-related cracking, high rutting, extensive-mixture-related distresses, and/or elevated levels of roughness (IRI > 170 in./mi).	Pre-Overlay treatment recommended if not reconstructed. • Structural overlay, with milling or leveling course and surface repairs. • Remove and replace existing layers prior to overlay. • In-place recycling prior to overlay. • Reconstruction.
	Very Poor	Extensive load-related cracking and/or very rough surfaces (IRI > 220 in./mi).	Pre-Overlay treatment recommended if not reconstructed. • Structural overlay with milling and surface repairs. • Remove and replace existing layers prior to overlay. • In-place recycling prior to overlay. • Reconstruction.

12.2.4 Decide on Pre-Overlay Treatment

Various pre-overlay treatments and repairs need to be considered to address deterioration of the existing pavement, improve surface smoothness, and provide uniform support conditions for the HMA overlay. For existing flexible or semi-rigid pavements, the pre-overlay treatments may include; do nothing, placement of a leveling course, a combination of milling, full or partial depth repairs, or in-place recycling. For existing rigid pavements, the pre-overlay repair may include; do nothing, diamond grinding, full or partial depth slab repair of JPCP and JRCP and punchouts of CRCP, and/or mud-jacking the slabs to fill any voids and re-level the slabs. Crack sealing is not a recommended pre-overlay treatment prior to overlay placement because the HMA overlay when placed at elevated temperatures may cause the sealant material to expand creating a bump in the overlay and significantly reducing the smoothness of the final surface. However, that may not be true if Warm Mix Asphalt overlay is placed instead of HMA.

Determining how much of the distress or damage could be repaired before the HMA overlay is placed requires a careful mix of experience and engineering judgment. Table 12-2 lists some of the candidate repair or pre-overlay treatments for all types of pavements, while Table 12-3 lists the major rehabilitation treatments of existing HMA and HMA over PCC pavements. Deciding on the pre-overlay treatment to be used could be based more on experience and historical data, rather than on the distresses and IRI predicted with AASHTOWare Pavement ME Design.

If the distress in the existing pavement is likely to affect overlay performance within a few years, it could be repaired prior to overlay placement. Premature distress in the overlay is often the result of deterioration in the existing pavement that was not properly repaired before overlay placement (*3, 4*).

For HMA-surfaced pavements, cold milling, and in-place recycling has become common pre-overlay treatments. Cold milling equipment can easily remove as much as 3 to 4 in. of HMA in a single pass. Removal of a portion of the existing cracked and hardened HMA surface by cold milling frequently improves the performance of an HMA overlay—because it provides good interface friction and removes surface defects. Cold milling also increases the smoothness of the existing pavement by removing rutting and other surface distortions. The depth of milling is an input to AASHTOWare Pavement ME Design and is intended to indicate the extent of damage removed from the existing surface prior to the placement of a new overlay.

In-place recycling may be considered an option to reconstruction for those cases where an HMA overlay is not feasible due to the extent of repair that needs to be required to provide uniform support conditions. Recent equipment advances provide the capability to recycle pavements in place to a depth of 8 to 12 in. If the in-place recycling process includes all of the existing HMA layers (defined as pulverization), this option could be treated as a new flexible pavement design strategy. The pulverized layer may be treated as a granular layer if not stabilized or a stabilized layer if asphalt emulsion or some other type of stabilizer is added prior to compaction.

Agencies have used a wide range of materials and techniques as part of a rehabilitation design strategy to delay the occurrence of reflection cracks in HMA overlays of existing pavements. These materials include paving fabrics, stress-absorbing interlayer (SAMI), chip seals, crack relief layer or mixture, cushion course, and hot in-place recycling. Paving fabrics, thin layers, pavement preservation techniques, preventive maintenance activities, and other non-structural layers are not analyzed mechanistically in AASHTOWare Pavement ME Design.

Table 12-2. Candidate Repair and Preventive Treatments for Flexible, Rigid, and Composite Pavements

Pavement Type	Distress	Preventive Treatments	Repair Treatments
Flexible and Composite	Alligator Cracking	Surface/fog seal	Full-depth repair
		Surface patch	
	Longitudinal Cracking	Crack sealing	Partial-depth repair
	Reflective Cracking	Rout and seal cracks	Full-depth repair
		Saw and seal cuts above joints in PCC layer	
	Block Cracking	Seal cracks	Chip Seal
		Chip seal	
	Depression	None	Leveling course
			Mill surface
	Rutting	None	Leveling course
			Mill surface
	Raveling	Rejuvenating seal	Chip seal/surface seal
	Potholes	Crack sealing	Full-depth or partial-depth repairs
		Surface patches	
Rigid	JPCP Pumping	Reseal joints	Subseal or mud-jack PCC slabs (effectiveness depends on materials and procedures)
		Restore joint load transfer	
		Subsurface drainage	
		Edge support (tied PCC should edge beam)	
	JPCP Joint Faulting	Subseal joints	Grind surface; Structural overlay
		Reseal joints	
		Restore load transfer	
		Subsurface drainage	
		Edge support (tied PCC should edge beam)	
	JPCP Slab Cracking	Subseal (loss of support)	Full-depth repair
		Restore load transfer	Partial-depth repair
		Structural overlay	
	JPCP Joint or Crack Spalling	Reseal joints	Full-depth repair
			Partial-depth repair
	Punchouts (CRCP)	Polymer or epoxy grouting	Full-depth repair
		Subseal (loss of support)	
	PCC Disintegration	None	Full-depth repair
			Thick overlay

Table 12-3. Summary of Major Rehabilitation Strategies and Treatments Prior to Overlay Placement for Existing HMA and HMA/PCC Pavements

Pavement Condition	Distress Types	Candidate Treatments for Developing Rehabilitation Design Strategy											
		Full-Depth HMA Repair	Partial-Depth HMA Repair	Cold Milling	Hot or Cold In-Place Recycling	Cracking Sealing	Chip Seal	HMA Overlay	HMA Overlay of Fractured PCC Slab	Bonded PCC Overlay	Unbounded PCC Overlay	Subsurface Drainage Improvement	Reconstruction (HMA or PCC)
Structural	Alligator Cracking	✓			✓		✓	✓	✓	✓	✓		✓
	Longitudinal Cracking (low severity)		✓	✓	✓	✓		✓		✓	✓		✓
	Thermal Cracking	✓		✓	✓	✓		✓		✓	✓		✓
	Reflection Cracking	✓	✓	✓				✓	✓	✓	✓		✓
	Rutting—Subsurface			✓	✓			✓		✓	✓		✓
	Shoving—Subsurface	✓						✓					✓
Functional	Excessive Patching							✓			✓		✓
	Smoothness			✓				✓					
Drainage, Moisture Damage	Raveling		✓	✓				✓					
	Stripping	✓	✓					✓		✓	✓	✓	✓
	Flushing/Bleeding		✓				✓	✓					
Durability	Raveling		✓	✓	✓		✓	✓					
	Flushing/Bleeding		✓	✓	✓		✓	✓					
	Shoving—HMA		✓	✓	✓			✓					
	Rutting—HMA			✓	✓			✓					
	Block Cracking			✓	✓	✓	✓	✓					
Shoulders	Same as traveled lanes	Same treatments as recommended for the traveled lanes.											

The fitting and user-defined cracking progression parameters in the MEPDG empirical reflection crack prediction equation are provided only for the HMA overlay with paving fabrics (refer to Table 5-2 in Section 5.3.5). The fitting parameters were estimated from limited test sections with a narrow range of existing pavement conditions and in localized areas. Additional performance data are needed to determine the values for both the fitting and user-defined cracking progression parameters for a more diverse range of conditions and materials.

In the interim, designers may use the default fitting parameters for predicting the amount of reflection cracks over time, but they should not consider the predicted amount of reflection cracks in making design decisions. Design strategies to delay the amount of reflection cracks could be based on local and historical experience, until a reliable M-E-based prediction methodology is added to the MEPDG or the empirical regression equation has been calibrated for a more diverse set of existing pavement conditions for the different materials noted above.

12.2.5 Determination of Damaged Modulus of Bound Layers and Reduced Interface Friction

Deterioration in the existing pavement includes visible distress, as well as damage not visible at the surface. Damage not visible at the surface must be detected by a combination of NDT and pavement investigations (cores and borings).

In the overlay analysis, the modulus of certain bound layers of the existing pavement is characterized by a damaged modulus that represents the condition at the time of overlay placement. The modulus of chemically stabilized materials and HMA is reduced due to traffic induced damage during the overlay period. The modulus reduction is not applied to JPCP and CRCP because these type pavements are modeled exactly as they exist. Cracks in these slabs are considered as reflective transverse cracks through the HMA overlay. Damage of HMA is simulated in AASHTOWare Pavement ME Design as a modulus reduction of that layer.

Results from the pavement investigation need to identify any potential areas or layers with reduced or no interface friction. Reduced interface friction is usually characterized by slippage cracks and potholes. If this condition is found, the layers where the slippage cracks have occurred could be considered for removal or the interface friction input parameter in the overlay design should be reduced to zero between those adjacent layers.

12.2.6 HMA Overlay Options of Existing Pavements

Table 12-3 listed different repair strategies for existing HMA and HMA over PCC pavements with different surface conditions that have some type of structural-material deficiency.

HMA Overlay of Existing Flexible and Semi-Rigid Pavements

An HMA overlay is generally a feasible rehabilitation alternative for an existing flexible or semi-rigid pavement, except when the conditions of the existing pavement dictate substantial removal and replacement or in-place recycling of the existing pavement layers. Conditions where an HMA overlay is not considered feasible for existing flexible or semi-rigid pavements are listed below:

1. The amount of high-severity alligator cracking is so great that complete removal and replacement of the existing pavement surface layer is dictated.
2. Excessive structural rutting indicates that the existing materials lack sufficient stability to prevent rutting from reoccurring.
3. Existing stabilized base show signs of serious deterioration and requires a large amount of repair to provide a uniform support for the HMA overlay.
4. Existing granular base must be removed and replaced due to infiltration and contamination of clay fines or soils, or saturation of the granular base with water due to inadequate drainage.
5. Stripping in existing HMA layers dictate that those layers need to be removed and replaced.

In the MEPDG, the design procedure for HMA overlays of existing HMA surfaced pavements considers distresses developing in the overlay as well as the continuation of damage in the existing pavement structure. The overlay generally reduces the rate at which distresses develop in the existing pavement. The design procedure provides for the reflection of these distresses through the overlay layers when they become critical. The condition of the existing pavement also has a major effect on the development of damage in the new overlay layers.

HMA Overlay of Intact PCC Slabs

An HMA overlay is generally a feasible option for existing PCC and composite pavements provided reflection cracking is addressed during the overlay design. Conditions under which an HMA overlay is not considered feasible include:

- The amount of deteriorated slab cracking and joint spalling is so great that complete removal and replacement of the existing PCC pavement is dictated.
- Significant deterioration of the PCC slab has occurred due to severe durability problems.

The design procedure presented in the MEPDG considers distresses developing in the overlay as well as the continuation of damage in the PCC. For existing JPCP, the joints, existing cracks, and any new cracks that develop during the overlay period are reflected through the HMA overlay using empirical reflection cracking models that can be adjusted to local conditions. A primary design consideration for HMA overlays of existing CRCP is to full-depth repair all working cracks and existing punchouts and then provide sufficient HMA overlay to increase the structural section to keep the cracks sufficiently tight and exhibit little loss of crack LTE over the design period. A sufficient HMA overlay is also needed to reduce the critical top of slab tensile stress and fatigue damage that leads to punchouts.

HMA Overlay of Fractured PCC Slabs

The design of an HMA overlay of fractured PCC slabs is very similar to the design of a new flexible pavement structure. The primary design consideration is the estimation of an appropriate elastic modulus for the fractured slab layer. One method to estimate the elastic modulus of the fractured PCC pavement condition is to backcalculate the modulus from deflection basins measured on previous projects (refer to Chapter 9). The three methods referred to as fractured PCC slabs are defined below:

- **Rubblization**—Fracturing the slab into pieces less than 12 in. reducing the slab to a high-strength granular base, and used on all types of PCC pavements with extensive deterioration (severe mid-slab cracks, faulting, spalling at cracks and joints, D-cracking, etc.).
- **Crack and Seat**—Fracturing the JPCP slabs into pieces typically one to three feet in size.
- **Break and Seat**—Fracturing the JRCP slabs to rupture the reinforcing steel across each crack or break its bond with the concrete.

12.2.7 HMA Overlays of Existing HMA Pavements, Including Semi-Rigid Pavements

HMA overlays of flexible and semi-rigid pavements may be used to restore surface profile or provide structural strength to the existing pavement. The trial overlay and pre-overlay treatments need to be selected considering the condition of the existing pavement and foundation, and future traffic levels. The

HMA overlay may consist of up to four layers, including three asphalt layers and one layer of an unbound aggregate (sandwich section) or chemically stabilized layer.

The same distresses used for new flexible pavement designs are also used for rehabilitation designs of flexible and semi-rigid pavements (refer to Section 5.3). For overlaid pavements, the distress analysis includes considerations of distresses (cracking and rutting) originating in the HMA overlay and the continuation of damage and rutting in the existing pavement layers. The total predicted distresses from the existing pavement layers and HMA overlay are used to predict the IRI values over time (refer to Section 5.4).

Longitudinal and thermal cracking distresses in the HMA overlay are predicted at the same locations as for new pavement designs. Fatigue damage is evaluated at the bottom of the HMA layer of the overlay using the alligator fatigue cracking model. Reflection cracking is predicted by applying the empirical reflection cracking model to the cracking at the surface of the existing pavement.

The continuation of damage in the existing pavement depends on the composition of the existing pavement after accounting for the effect of pre-overlay treatments, such as milling or in-place recycling. For existing flexible and semi-rigid pavements where the HMA layers remain in place, fatigue damage will continue to develop in those layers in the existing structure using the damaged layer concept. All pavement responses used to predict continued fatigue damage in the existing HMA layers remaining in place are computed using the damaged modulus as determined from the pavement evaluation data using the methods discussed in Chapter 9. The pavement responses used to predict the fatigue damage of the HMA overlay use the undamaged modulus of that layer.

Plastic deformations in all HMA and unbound layers are included in predicting rutting for the rehabilitated pavement. As discussed in Chapter 5, rutting in the existing pavement layers will continue to accumulate but at a lower rate than for new materials due to the strain-hardening effect of past truck traffic and time.

12.2.8 HMA Overlays of Existing Intact PCC Pavements Including Composite Pavements (one or more HMA overlays of existing JPCP and CRCP)

HMA overlays may be used to remedy functional or structural deficiencies of all types of existing PCC pavements. It is important for the designer to consider several aspects, including the type of deterioration present, before determining the appropriate rehabilitation strategy to adopt.

Analysis Parameters Unique to HMA Overlay of JPCP and CRCP

Number of HMA Layers for Overlay

The HMA overlay may consist of a maximum of three layers. All mixture parameters normally required for HMA need to be specified for each of the layers.

Reflection Cracking of JPCP Through HMA Overlay
The transverse joints and cracks of the underlying JPCP will reflect through the HMA overlay depending on several factors. The empirical reflection cracking models included in AASHTO Pavement ME Design may be calibrated to local conditions prior to use of the software (refer to Section 5.4). They have not been nationally calibrated and thus local calibration is even more important. Both the time in years to 50 percent of reflected joints and the rate of cracking may be adjusted depending on the HMA overlay thickness and local climatic conditions.

It is recommended that reflection cracking be considered outside of AASHTOWare Pavement ME Design by considering options such as fabrics and grids or saw and sealing of the HMA overlay above joints. AASHTOWare Pavement ME Design only considers reflection cracking treatments of fabrics through empirical relationships (refer to Section 5.4) and grids, saw, and sealing options are not included as inputs to the AASHTOWare Pavement ME Design software.

For CRCP, there is no reflection cracking of transverse joints. The design procedures assumes that all medium- and high-severity punchouts will be repaired with full-depth reinforced concrete repairs.

Impact of HMA Overlay on Fatigue Damage
The HMA overlay has a very significant effect on thermal gradients in the PCC slab. Even a thin HMA overlay greatly reduces the thermal gradients in the PCC slab, thereby reducing the amount of fatigue damage at both the top and bottom of the slab. This typically shows that even thin HMA overlays have a sufficient effect as to reduce future fatigue damage in the PCC slab. The extent of reflection cracking, however, is greatly affected by HMA thickness and this often becomes the most critical performance criteria for overlay design.

Estimate of Past Damage
For JPCP and CRCP subjected to an HMA overlay, an estimate of past fatigue damage accumulated since opening to traffic is required. This estimate of past damage is used (along with future damage) to predict future slab cracking and punchouts. For JPCP, the past damage is estimated from the total of the percent of slabs containing transverse cracking (all severities) plus the percentage of slabs that were replaced on the project. Required inputs for determining past fatigue damage are as follows:

1. Before pre-overlay repair, percent slabs with transverse cracks plus percent previously repaired/replaced slabs. This represents the total percent slabs that have cracked transversely prior to any restoration work.
2. After pre-overlay repair, total percent repaired/replaced slabs (note, the difference between [2] and [1] is the percent of slabs that are still cracked just prior to HMA overlay).

Repairs and replacement refers to full-depth repair and slab replacement of slabs with transverse cracks. The percentage of previously repaired and replaced slabs is added to the existing percent of transverse cracked slabs to establish past fatigue damage caused since opening to traffic. This is done using the MEPDG national calibrated curve for fatigue damage versus slab cracking. Future slab cracking is then computed over the design period as fatigue damage increases month by month.

Example: A survey of the existing pavement shows six percent slabs with transverse cracks and four percent slabs that have been replaced. It is assumed that all replaced slabs had transverse cracks. During pre-overlay repair, five percent of the transversely cracked slabs were replaced leaving one percent still cracked. Inputs to AASHTOWare Pavement ME Design are as follows:

- Six percent slabs with transverse cracks plus four percent previously replaced slabs equals ten percent.
- After pre-overlay repair, total percent replaced slabs equals nine percent. Note that the percent of slabs still cracked, prior to overlay, is therefore 10–9 = 1 percent.

For CRCP, the same approach is used. The number of existing punchouts per mile (medium- and high-severity only) is added to the number of repairs of punchouts per mile. This total punchouts per mile is a required input to establish past fatigue damage caused by repeated axle loads since opening to traffic. This is done using the MEPDG global calibrated curve for fatigue damage versus punchouts. An estimate of future punchouts is then computed over the design period as fatigue damage increases month by month.

Dynamic Modulus of Subgrade Reaction (Dynamic k*-value)*

The subgrade in rigid pavements is modeled as an elastic spring foundation. The stiffness of the elastic springs is also called the modulus of subgrade reaction. The subgrade reaction may be determined in the following ways:

1. Provide resilient modulus inputs of the existing unbound sublayers including the subgrade soil similar to new design. AASHTOWare Pavement ME Design software will backcalculate an effective single dynamic modulus of subgrade reaction (*k*-value) for each month of the design analysis period for these layers. The effective *k*-value, therefore, essentially represents the compressibility of underlying layers (i.e., unbound base, subbase, and subgrade layers) upon which the upper bound layers and existing HMA or PCC layer is constructed. These monthly values will be used in design of the rehabilitation alternative.
2. Measure the top of slab deflections with an FWD and conduct a backcalculation process to establish the mean *k*-value during a given month. Enter this mean value and the month of testing into AASHTOWare Pavement ME Design. This entered *k*-value will remain for that month throughout the analysis period, but the k-value for other months will vary according to moisture movement and frost depth in the pavement.

Modulus of Elasticity of Existing JPCP or CRCP Slab

The modulus of elasticity of the existing slab is that existing at the point of time of rehabilitation. This value will be higher than the 28-day modulus of course. It is estimated using procedures given in Table 12-4. This modulus is the intact slab value. It is not a reduced value due to slab cracking as is done for unbonded PCC overlays. This layer is the primary load carrying layer of the overlaid composite pavement structure. The amount of cracking in the existing slab is accounted for in two ways:

1. Percent of slabs cracked are determined and used to compute past damage which will affect the future cracking of the existing slab.
2. Percent of slabs cracked are considered to reflect through the HMA overlay in a predicted rate thereby affecting the performance through limiting criteria (percent area of traffic lane) and through impacting the IRI.

Table 12-4. Data Required for Characterizing Existing PCC Slab Static Elastic Modulus for HMA-Overlay Design

Input Data	Hierarchical Level		
	1	2	3
Existing PCC slab design static elastic modulus	The existing PCC slab static elastic modulus $E_{BASE/DESIGN}$ for the existing age of the concrete is obtained from (1) coring the intact slab and laboratory testing for elastic modulus or (2) by back calculation (using FWD deflection data from intact slab and layer thicknesses) and multiplying by 0.8 to convert from dynamic to static modulus.	$E_{BASE/DESIGN}$ obtained from coring and testing for compressive strength. The compressive strength value is converted into elastic modulus. The design elastic modulus is obtained as described for Level 1.	$E_{BASE/DESIGN}$ estimated from historical agency 28-day values which are extrapolated to the date of construction.

Trial Rehabilitation with HMA Overlays of JPCP and CRCP

A range HMA overlay thickness may be run and the performance projected by AASHTOWare Pavement ME Design. The ability of the overlay to satisfy the performance criteria is then determined. Some general guidelines on criteria are given in Table 12-5. Note that for some overlay/PCC slab design situations, the structural analysis will show that only a thin HMA overlay is needed (structural adequacy is acceptable). The addition of a relatively thin HMA overlay changes the thermal gradients so much that fatigue damage becomes minimal. In this case, the designer may choose a minimum overlay thickness that can meet all other criteria including (1) the smoothness specification, (2) can be placed and compacted properly, and (3) has adequate thickness to remain in place over the design life. Most highway agencies specify minimum thicknesses of HMA overlays for just this purpose.

Design Modifications to Reduce Distress for HMA Overlays

Trial designs with excessive amounts of predicted distress/smoothness need to be modified to reduce predicted distress/smoothness to tolerable values (within the desired reliability level). Some of the most effective ways of accomplishing this are listed in Table 12-6. It should be noted that reflection crack control treatments, such as saw and seal, are not a direct input in the AASHTOWare Pavement ME Design software.

Table 12-5. Recommendations for Performance Criteria for HMA Overlays of JPCP and CRCP

Distress Type	Recommended Modifications to Design
Rutting in HMA	Criteria for rutting should be selected similar to new or reconstructed pavement design. This rutting is only in the HMA overlay.
Transverse cracking in JPCP existing slab	The placement of an HMA overlay will significantly reduce the amount of future fatigue transverse cracking in the JPCP slab and this is not normally a problem. A typical limit of 10 percent (all severities) appears to be reasonable in that exceeding this value indicates that the overlaid JPCP is experiencing significant load-fatigue damage and a structural improvement is needed.
Punchouts in CRCP existing slab	The placement of an HMA overlay will significantly reduce the amount of future punchout development in CRCP, and this is not normally a problem. Exceeding the typical limit of 5 to 10 per mile (medium– and high–severity) appears to be reasonable in that exceeding this value indicates that the overlaid CRCP is experiencing significant load-fatigue damage and a structural improvement is needed.
Reflection cracking from existing JPCP or CRCP slab	The extent of reflection cracking is dependent on any special reflection cracking treatments that the designer may have specified. Thus, if the designer feels that this treatment will reduce or eliminate reflection cracking from the existing slab then this criterion may be ignored. The MEPDG predicted reflection cracking is from transverse joints and transverse cracks in JPCP but it is converted into a percent area of traffic lane. A maximum recommended value of 1.0% area is recommended for reflection cracking of all severities (Note: This represents 100 transverse cracks per mile or one crack every 53 ft which creates significant roughness).
Smoothness	The limiting IRI should be set similar to that of new or reconstructed pavements. The only exception to this would be when the existing pavement exhibits a large amount of settlements or heaves that would make it difficult to level out. If this is the case, a level up layer should be placed first and then the designed overlay placed uniformly on top.

12.2.9 HMA Overlay of Fractured PCC Pavements

The objective of rubblizing PCC slabs is to eliminate reflection cracking in an HMA overlay by destroying the integrity of the existing slab. This objective is achieved by fracturing the PCC slab in place into fragments of nominal 3- to 8-in.-size or less, while retaining good interlock between the fractured particles. The rubblized layer acts as an interlocked unbound layer, reducing the existing PCC to a material comparable to a high-quality aggregate base course.

The rubblization process is applicable to JPCP, JRCP, and CRCP. Reinforcing steel in JRCP and CRCP must become debonded from the concrete to be successful and meet the performance expectations. The purpose of this section is to provide guidance on the use of rubblization of PCC pavements to maximize the performance of this rehabilitation option.

Table 12-6. Recommendations for Modifying Trial Design to Reduce Distress/Smoothness for HMA Overlays of JPCP and CRCP

Distress Type	Recommended Modifications to Design
Rutting in HMA	Modify mixture properties. See recommendations in Section 13.2.
Transverse cracking in JPCP existing slab	Repair more of the existing slabs that were cracked prior to overlay placement. Increase HMA-overlay thickness.
Crack width CRCP	It is desirable to have crack width <0.020 in over the design period. However, there is not much the designer can do to control this parameter.
Crack LTE CRCP	It is desirable to have crack LTE greater than 95% over the design period. This will prevent any reflection cracking or punchouts from occurring. The only design feature that will affect this parameter is overlay thickness.
Punchouts in CRCP existing slab	Repair all of the existing punchouts prior to overlay placement. Increase HMA overlay thickness.
Reflection cracking from existing JPCP or CRCP slab	Apply an effective reflection crack control treatment such as saw and seal the HMA overlay over transverse joints. Increase HMA overlay thickness.
Smoothness	Build smoother pavements initially through more stringent specifications. Reduce predicted slab cracking and punchouts.

It should be noted that a designer may or may not get the desired effects listed in the table above, depending on the structural profile and location of the pavement. The relationships reported in the table may not be effective for every type of pavement and depends on project-specific conditions.

Project Selection Criteria for Rubblization

Rubblization is an effective reconstruction technique in many situations, but inadequate project scoping may lead to constructability and performance problems. Proper project scoping should follow the following steps, which are illustrated in flow chart form in Figures 12-3 through 12-6.

1. Identify roadway site features and conditions that may have a detrimental effect on constructability and performance of rubblized PCC pavements (Figure 12-3). In general, rubblizing PCC pavements may be considered a viable option when there is no rigid layer within 3 ft, no water table within 5 ft, and no old utility lines within 5 ft of the PCC layer. When these conditions exist, other rehabilitation strategies maybe more appropriate. Rubblization may still be considered for use even under these conditions, but may require more detailed investigations as to the uniformity of the rubblized PCC slabs.

2. Determine the condition and distresses of the existing PCC pavement (Figures 12-4 and 12-5). Rubblization is considered a viable option when the PCC pavement has no remaining life (i.e., when there is extensive structural distress along the project). If horizontal cracks or delamination between different PCC layers has occurred along the project site, however, other rehabilitation options maybe more cost-effective and should be considered.

3. Determine the foundation support conditions and strength (Figure 12-6). A foundation investigation may be performed using the FWD and DCP tests. The FWD deflection basin and DCP data are used to determine the elastic modulus of the foundation layers. The frequency of these tests needs to be determined to identify any weak areas along the project. The project engineer may identify areas where the support modulus for the PCC slabs is less than 5,000 psi (34 MPa), based on laboratory measured resilient modulus. A backcalculated modulus value from deflection basin data of 10,000 psi beneath a PCC pavement corresponds to a laboratory measured resilient modulus value of approximately 5,000 psi. Foundation modulus values, backcalculated from deflection basins, less then 10,000 psi may have a detrimental effect on the rubblization process. Rubblization of PCC slabs that are resting directly on a fine grained soil subgrade have experienced significant problems in the vibrating head settling into the fractured slab and into the subgrade.

Design Features for Rubblization PCC Pavements

Installation of Edge Drains

Rubblizing the PCC slabs results in a layer with significant permeability. Any water infiltrating the rubblized layer should be quickly removed through the use of edge drains, especially for pavements supported by fine-grained soils with low permeability. Edge drains are not required in areas with coarse-grained soils that have high permeability.

Edge drains may be used in all rubblized projects to drain any saturated foundation layer. These drains may be placed continuously or intermittently along the project. Their use and location could be based on engineering judgment to remove water from the pavement structure. When used, edge drains need to be installed prior to the rubblization process to ensure that there is sufficient time to allow the subbase and subgrade to drain and dry out (usually two weeks before rubblization starts).

Leveling Courses

A leveling course is needed to restore the grade and make profile corrections to the surface of the rubblized PCC layer. Leveling course material may consist of crushed aggregate, milled or recycled asphalt pavement (RAP), or a fine-graded HMA mixture that is workable. A 2- to 4-in. leveling course should be included in the design to fill in depressions or low spots along the rubblized surface. This leveling course also acts as a cushion layer for the HMA overlay. If a workable, fine-graded HMA mixture (a HMA mixture with higher asphalt content) is used, the designer could ensure that there is sufficient cover so that rutting does not become a problem within that workable layer.

In many cases, the use of crushed aggregate base materials as the leveling course cannot be used because of clearance or height restrictions at bridges and other overhead structures. HMA leveling courses with specific fracture resistant properties are more beneficial to long term pavement performance. These mixtures could be compacted to in-place air voids less than seven percent. In either case, leveling courses could be accounted for in the structural design, but not for the sole purpose of reducing the HMA overlay thickness. When HMA leveling courses are used, sufficient HMA overlay thickness needs to be placed to ensure that the heavier trucks will not cause rutting or any lateral distortions in the leveling course.

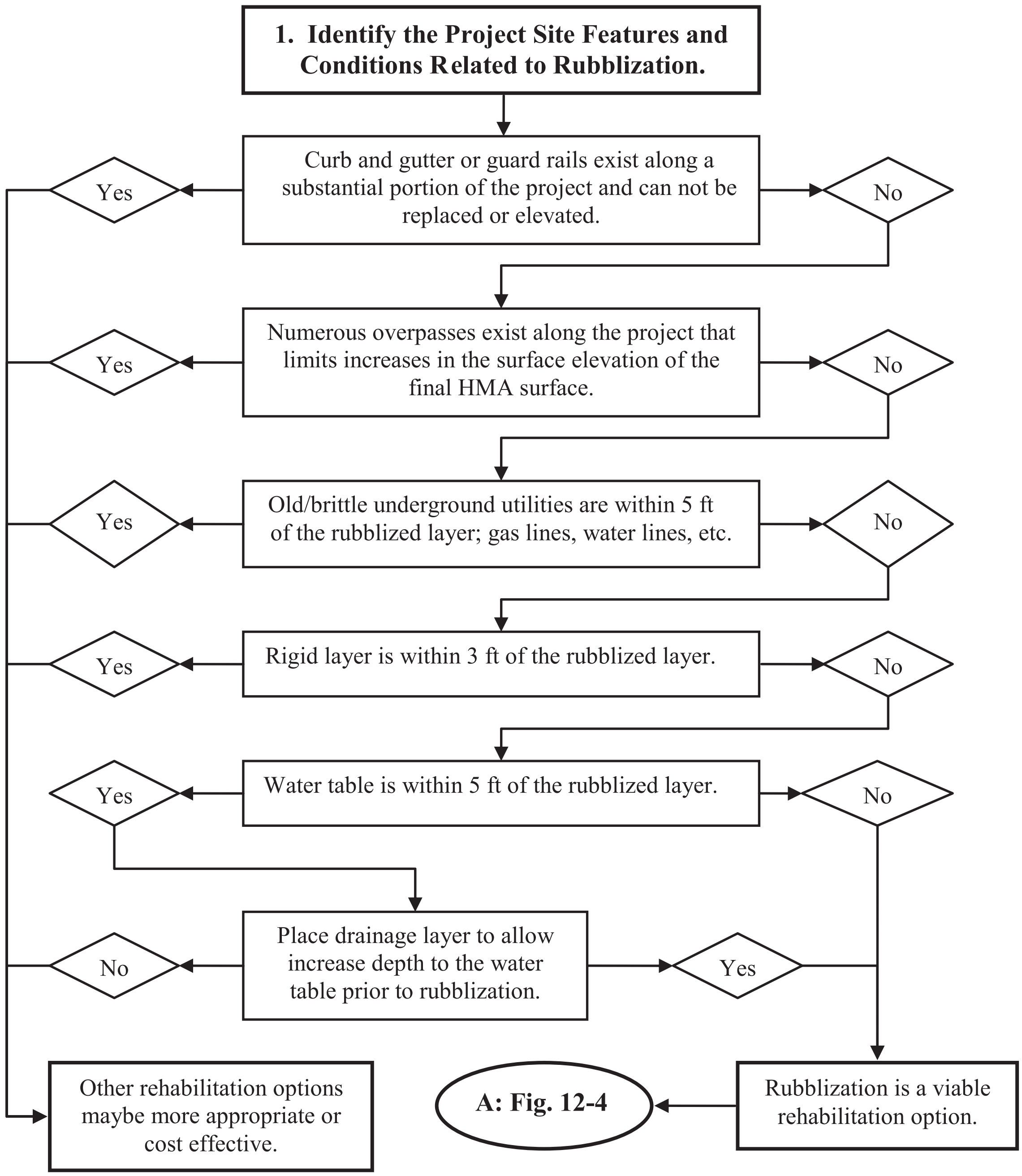

Figure 12-3. Site Features Conducive to the Selection of the Rubblization Process for Rehabilitating PCC Pavements

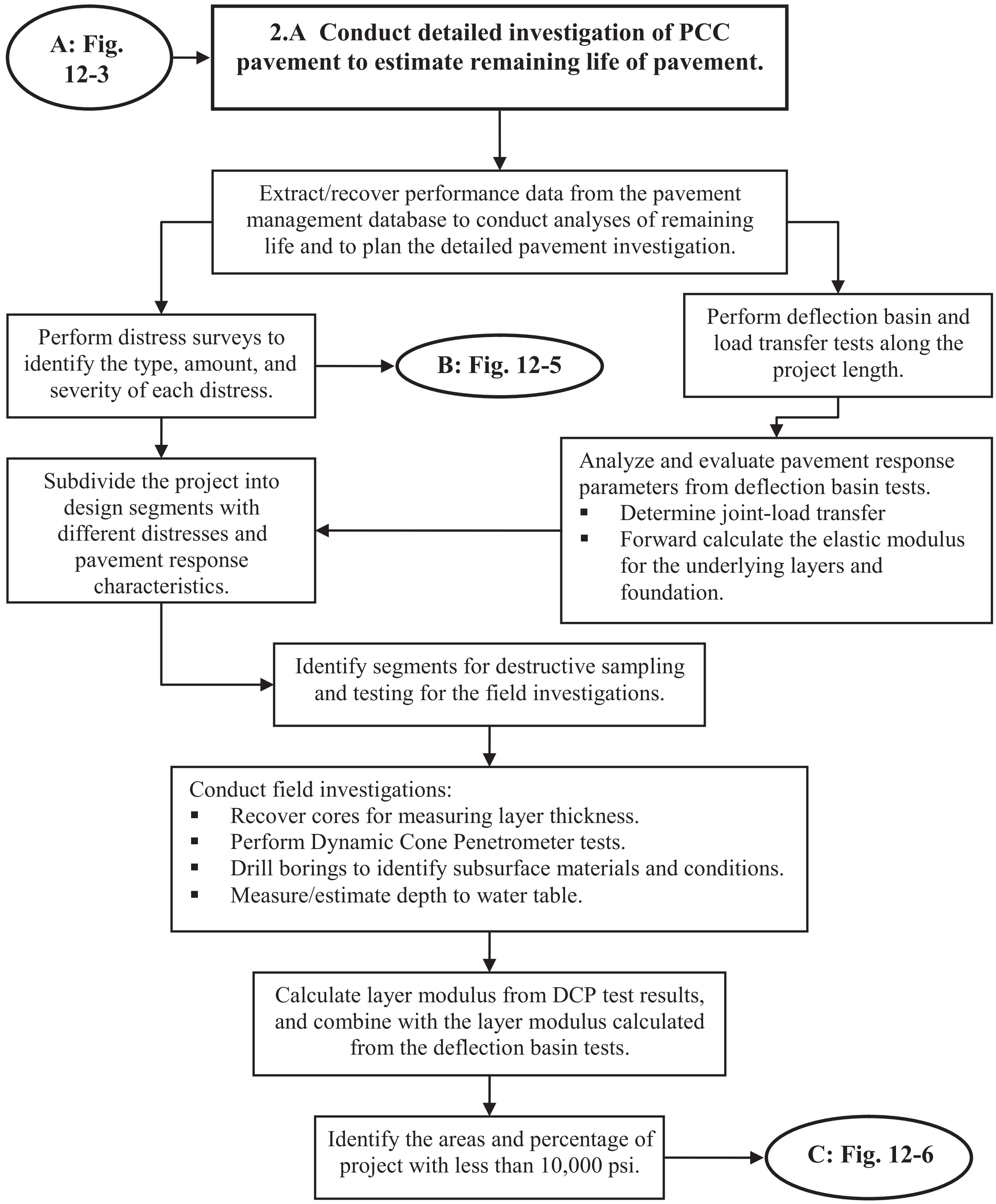

Figure 12-4. Recommendations for a Detailed Investigation of the PCC Pavement to Estimate Remaining Life and Identifying Site Features and Conditions Conducive to the Rubblization Process

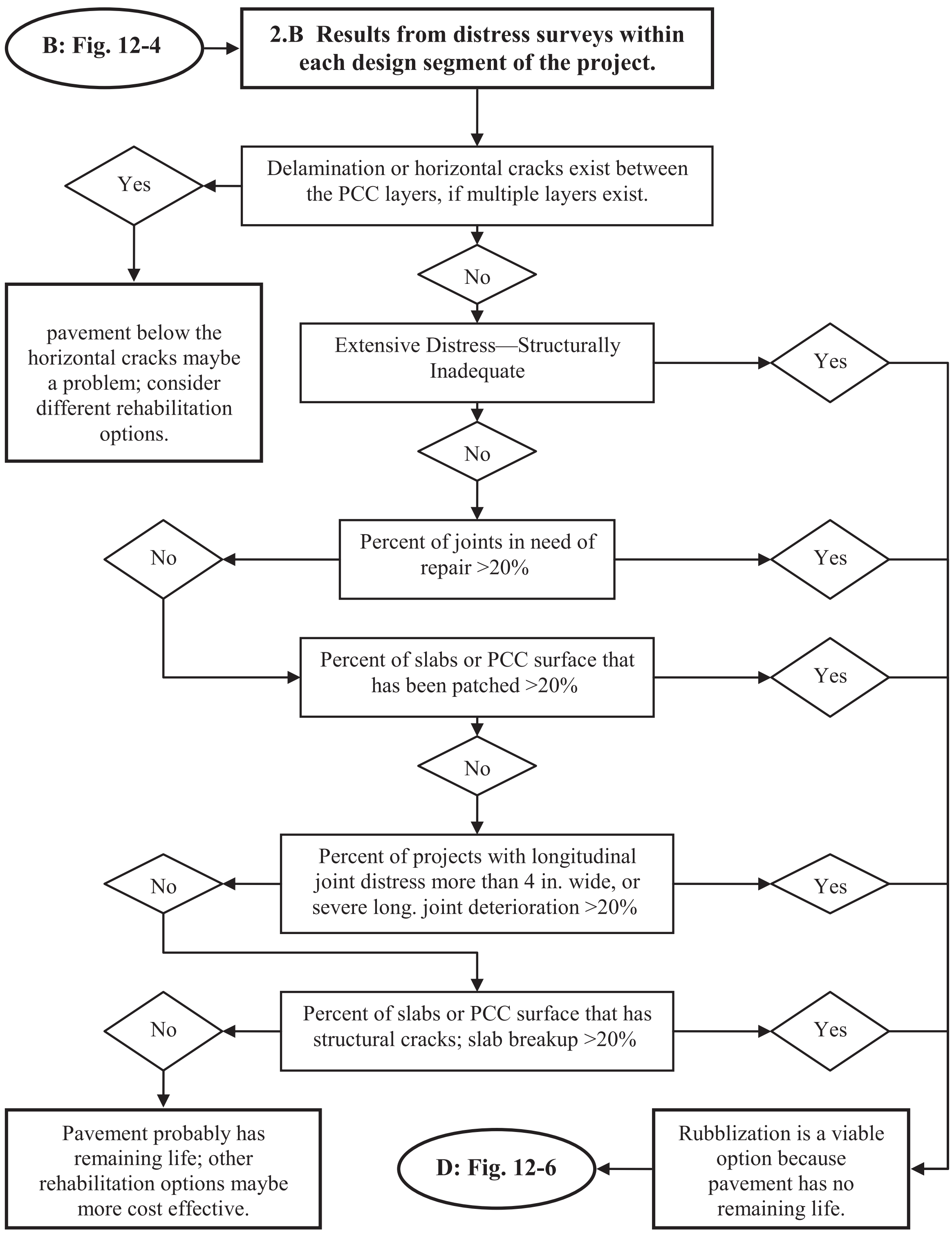

Figure 12-5. Evaluate Surface Condition and Distress Severities on Selection of Rubblization Option

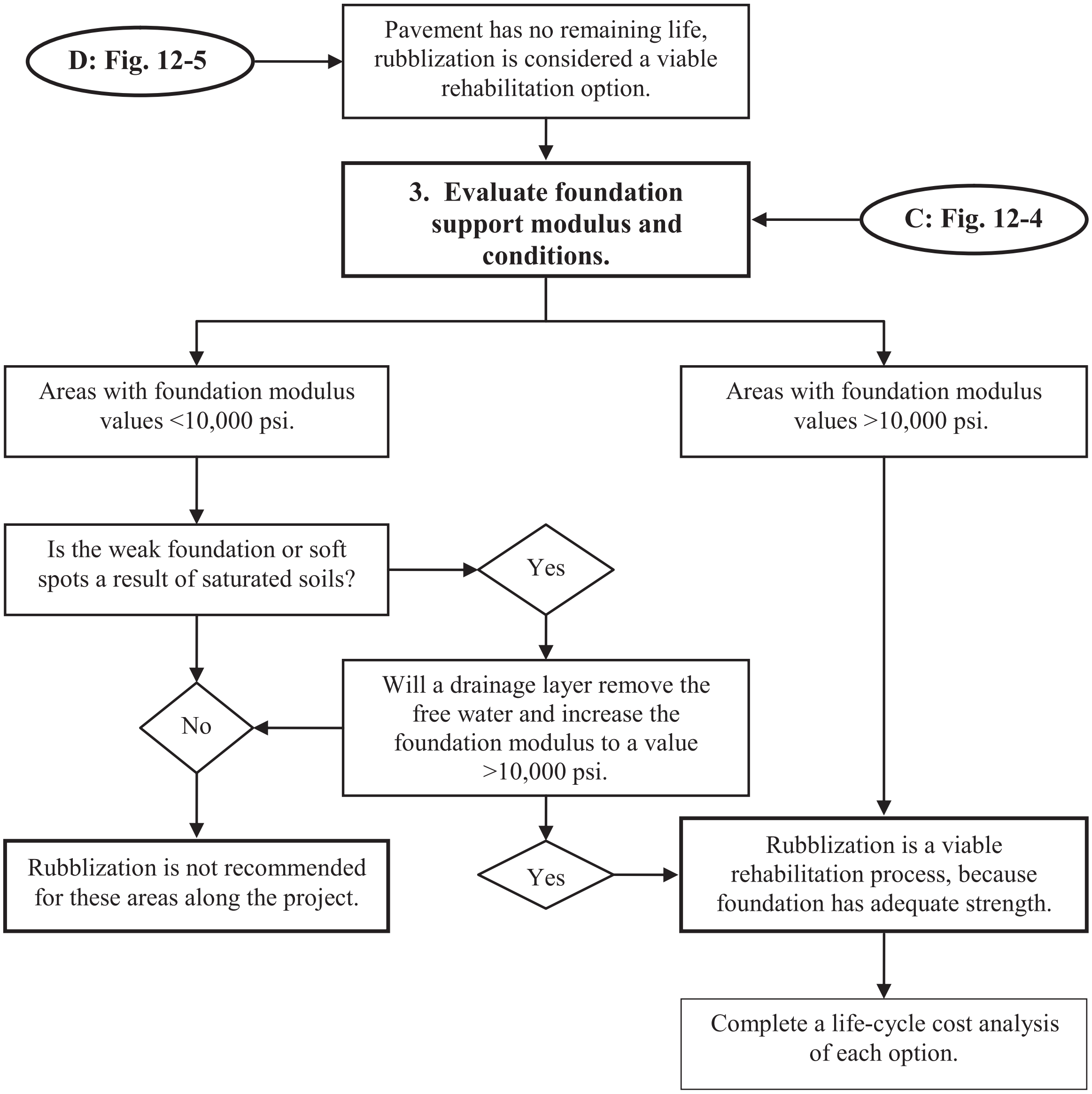

Figure 12-6. Foundation Support Conditions Related to the Selection of the Rubblization Process

Each design situation and material needs to be evaluated to determine the rehabilitation option that will provide the better long-term performance, while meeting the project requirements. An HMA leveling course could be considered for use on projects where the rubblized pavement must carry traffic temporarily until additional HMA lifts are placed. The thickness of the leveling course and its properties need to be determined to carry the expected traffic during construction.

Minimum HMA Overlay Thickness Above Rubblized PCC Slabs

The minimum HMA overlay thickness placed over rubblized PCC layers from a constructability standpoint is 4 in. This minimum thickness excludes any HMA leveling course mixture that is placed to correct surface profiles.

The performance of a pavement structure is dependent upon the interaction between pavement response and strength of the different layers. Wheel loads induce stresses and strains in each layer, which may result in deformation and cracking of the HMA layer. The rehabilitation design procedure has to determine the HMA overlay thickness that satisfies both constructability and structural requirements of the rubblized pavement. M-E based design procedures are being used by many agencies, but primarily for forensic studies and post-construction evaluation of the pavement structure. The HMA overlay fatigue considerations control the overlay thickness requirements for rubblized pavement using the M-E-based procedures.

Table 10-5 in Chapter 10 provided a range of equivalent elastic modulus values that may be used. The equivalent modulus of the rubblized layer is dependent on the agency's specifications for that layer. An elastic modulus value of 65,000 psi (450 MPa) for the rubblized layer is recommended for use in HMA overlay design. This value is less than the value recommended in the NAPA Information Series 117, but is based on backcalculation of layer modulus from deflection basin data and performance analyses of rubblized pavements built in around the United States.

For thick JPCP exceeding 10 in. and JRCP, a large modulus gradient between the surface and bottom of the rubblized layer typically exists because the fractured particle size varies from top to bottom. The designer can subdivide the rubblized layer into an upper and lower portion of the JPCP or above and below the reinforcement of JRCP or just use an average value throughout the fractured slab. Without deflection basin data, it is suggested that an average or equivalent value of 65,000 psi be used for the rubblized layer.

12.3 REHABILITATION DESIGN WITH PCC OVERLAYS

This section describes the M-E design procedures for rehabilitation of existing flexible, rigid, and composite pavements with PCC. Lane additions and widening of narrow lanes are also considered. Many aspects of rehabilitation design are similar to new design; thus, the designer should become familiar with the design of new and reconstructed PCC pavements described in Chapter 11.

12.3.1 Overview

PCC overlays and restoration may be used to remedy functional or structural deficiencies of all types of existing pavements. It is important for the designer to consider several aspects, including the type of deterioration present, before determining the appropriate rehabilitation strategy to adopt. Several different rehabilitation strategies using PCC may be applied to existing pavements to extend their useful service life. These are summarized in Table 12-7.

The design of rehabilitated pavements requires an iterative, hands-on approach by the designer. The designer needs to select a proposed trial rehabilitation design and then analyze the design in detail to determine whether it meets the applicable performance criteria (i.e., joint faulting and slab cracking for JPCP, punchouts for CRCP, and smoothness for both JPCP and CRCP) established by the designer. If a particular trial rehabilitation design does not meet the performance criteria, the design is modified and reanalyzed until it meets the criteria. The designs that meet the applicable performance criteria are then considered feasible from a structural and functional viewpoint and may be further considered for other evaluations, such as life-cycle cost analysis (LCCA).

Table 12-7. PCC Rehabilitation Options—Strategies to Correct Surface and Structural Deficiencies of All Types of Existing Pavement

Type of PCC Overlay	Existing Pavement	Rehab of Existing Pavement	Separation Layer and Surface Preparation
Unbonded JPCP Overlay	JPCP and CRCP	Repair by slab replacement or full-depth repair (FDR)	Place HMA layer for level up and separation. Do not diminish bonding between PCC overlay and HMA.
	Fractured JPCP and CRCP	Fracture and seat existing pavement if concerns over rocking slabs exists.	Place HMA layer for level up and separation. Do not diminish bonding between PCC overlay and HMA.
	Composite pavement (HMA/PCC)	Mill off portion or all of existing HMA for level up (all if stripping exists), FDR existing PCC pavement, or fracture and seat existing pavement.	Place HMA layer for level up and separation. Do not diminish bonding between PCC overlay and HMA.
Unbonded CRCP Overlay	JPCP and CRCP	Repair by FDR, or fracture and seat existing pavement if concerns over poor transverse joint load transfer or rocking slabs exists.	Place HMA layer for level up and separation. Increase thickness if poor joint and crack LTE. Maximize bonding between CRCP overlay and HMA layers.
	Fractured JPCP and CRCP	Fracture existing pavement if concerns over rocking slabs or reflection cracking exists (poor existing joint LTE).	Place HMA layer for level up and separation. Maximize bonding between CRCP overlay and HMA layers.
Bonded PCC Overlay	JPCP and CRCP in fair or better condition only.	FDR deteriorated joints and cracks	Preparation of existing surface to maximize bond with PCC overlay
JPCP and CRCP Overlays	Existing flexible pavement	Mill portion of existing HMA material for level up and removal of deterioration. Patch as needed.	Place HMA layer for level up and separation. Maximize bonding between PCC overlay and HMA layers.

It should be noted that a designer may or may not get the desired effects listed in the table above, depending on the structural profile and location of the pavement. The relationships reported in the table may not be effective for every type of pavement and depends on project-specific conditions.

The design procedures described in this chapter can utilize recycled materials. The use of recycled materials in rehabilitation is acceptable so far as the material properties may be characterized by the parameters used in design and the recycled material meets durability requirements. PCC rehabilitation design process requires nine steps listed below.

- Steps 1–4—Evaluation of the existing pavement (see Chapter 11).
    1. Determine existing pavement condition.
    2. Determine causes and mechanism of distress.
    3. Define problems and inadequacies of existing pavement.
    4. Identify possible constraints.
- Step 5—Rehabilitation strategy selection (see Section 3.4).
- Step 6—Rehabilitation design (see Chapter 12).
- Step 7—Perform life-cycle cost analysis (as desired).
- Step 8—Determine non-monetary factors that influence rehabilitation (as desired).
- Step 9—Determine preferred rehabilitation strategy (as desired).

Figure 12-7 presents the design process for major PCC rehabilitation strategies included in AASHTOWare Pavement ME Design.

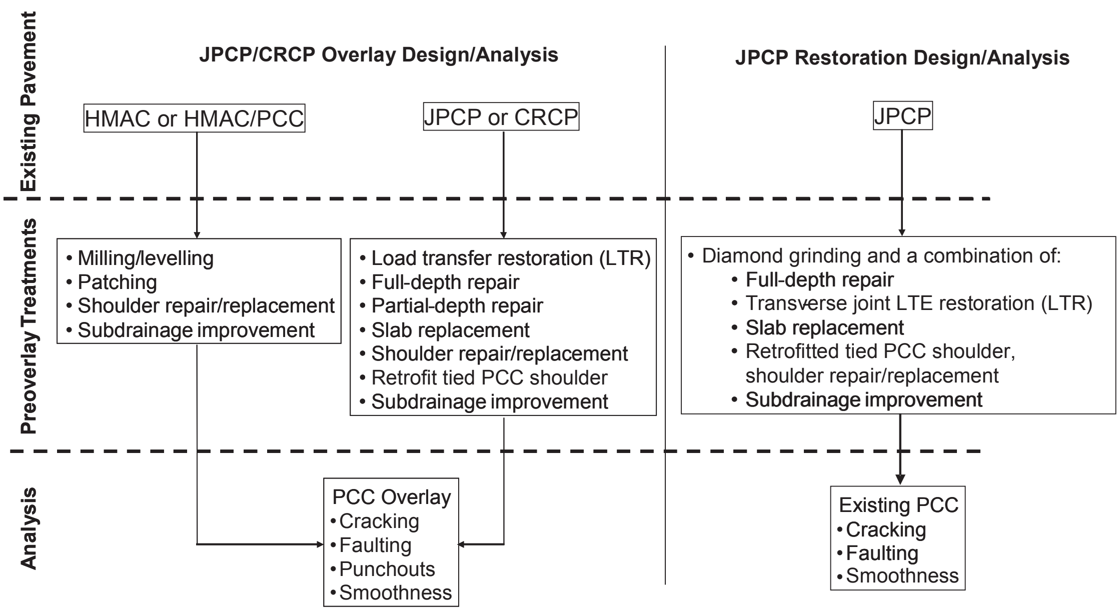

Figure 12-7. Overall Design Process for Major PCC Rehabilitation Strategies of All Pavement Types

12.3.2 Analysis Parameters Unique to Rehabilitation

Initial Smoothness

Recommendations for initial smoothness (IRI) are similar to new construction for JPCP and CRCP overlays. They depend greatly on the project smoothness specifications. The estimate of initial smoothness for restored JPCP depends on the diamond grinding specifications (for this design procedure restoration needs to always include diamond grinding). The initial IRI may, however, need to be adjusted upward for a given project if a significant amount of settlements or heaves exist, as this problem cannot be easily rectified through diamond grinding alone. Local leveling, such as slab jacking or thin localized overlays, may be needed.

JPCP Overlay Design Features

Guidelines on unique joint design and interlayer friction features of JPCP overlays are provided in Table 12-8. It should be noted that joint mismatching is not an input provided within the software and is intended to be a guideline for construction purposes. The only input for joint spacing in AASHTOWare Pavement ME Design is in the JPCP layer or, in this case, the overlay layer.

Characterization of Existing PCC Slab

The elastic modulus of the existing slabs including existing cracking that will not be repaired is a critical input for the design of an unbonded overlay. The mean modulus depends mainly upon the amount of cracking in the existing slab. Tables 12-9 and 12-10 provide general recommendations on how to estimate this input.

Dynamic Modulus of Subgrade Reaction (Dynamic k*-value)*

The subgrade modulus may be characterized in the following ways for PCC rehabilitation:

1. Provide modulus inputs of the existing unbound sublayers including the subgrade soil similar to new design. AASHTOWare Pavement ME Design software will backcalculate an effective single dynamic modulus of subgrade reaction (*k*-value) for each month of the design analysis period for these layers. The effective *k*-value, therefore, essentially represents the compressibility of underlying layers (i.e., unbound base, subbase, and subgrade layers) upon which the upper bound layers and existing HMA or PCC layer is constructed. These monthly values will be used in design of the rehabilitation alternative.
2. Measure the top of slab deflections with an FWD and conduct a backcalculation process to establish the mean *k*-value during a given month. Enter this mean value and the month of testing into AASHTOWare Pavement ME Design. This entered *k*-value will remain for that month throughout the analysis period, but the *k*-value for other months will vary according to moisture movement and frost depth in the pavement.

12.3.3 Estimate of Past Damage (for JPCP Subjected to CPR)

For JPCP subjected to CPR, an estimate of past fatigue damage is required. An estimate of past damage is used with estimates of future damage to predict future cracking. Required inputs for determining past fatigue damage are as follows:

1. Before restoration, percent slabs with transverse cracks plus percent previously repaired/replaced slabs. This represents the total percent slabs that have cracked transversely prior to any restoration work.
2. After restoration, total percent repaired/replaced slabs (note, the difference between [2] and [1] is the percent of slabs that are still cracked after restoration).

Table 12-8. Summary of Key Aspects of Joint Design and Interlayer Friction for JPCP Overlays

Rehabilitation Strategy	Key Issues	Description
Unbonded JPCP overlay over existing concrete pavement (with separation layer)	Joint spacing	Joint spacing of the overlay is a direct input to M-E design and has a significant effect on transverse cracking. Unbonded JPCP overlays are subject to greater curling stresses because of the stiff support from the existing pavement and this effect can be determined through sensitivity analysis. For thinner overlays a shorter joint spacing than conventional JPCP may be desirable (e.g., a 6-in. overlay could utilize a 12-ft joint spacing).
	Joint mismatching	The transverse joints in unbonded concrete overlays are usually mismatched with those in the underlying pavement. A minimum offset distance of 3 ft between the joints in the overlay and the underlying joints or cracks is usually recommended which provides improved load transfer in the overlay.
	Load transfer	Adequate joint load transfer can be provided by both the underlying pavement through mismatching the joints and by dowels for heavy truck traffic. Dowels may be needed to provide additional long-term, high-load transfer for pavements where significantly heavy traffic loads are expected. The need for dowels to meet the joint faulting criteria can be determined using the program. To decrease the susceptibility of the dowels to corrosion (in regions where the use of deicing salts are common), epoxy coated, stainless steel coated or metallic sleeved dowels are recommended.
	Friction JPCP and HMA Layer	The calibration of unbonded overlays utilized the "zero-friction contact" be used between the JPCP slab and the HMA separation layer.
Bonded PCC overlay over existing JPCP	Joint spacing	The joint system in the existing pavement dictates jointing system in a bonded overlay. The joint type and location in the existing pavement should be closely matched in the overlay.
	Joint width and depth	Critical Recommendation: The width of the joint must be wider than that in the existing pavement and must be sawed completely through the bonded overlay plus 0.5 in. The overlay joint sawing must be completed as soon as the concrete can be sawed to prevent debonding and erratic reflective cracking. Failure to follow the above recommendation will lead to debonding of the overlay.
	Load transfer	Load transfer devices are normally not used in bonded overlay joints.
JPCP overlay over existing flexible pavement	—	The design of joints for conventional concrete overlays of existing flexible pavements is similar to that for new JPCP.

Table 12-9. Data Required for Characterizing the Existing PCC Slab

<table>
<tr><th rowspan="2">Input Data</th><th colspan="3">Hierarchical Input Level</th></tr>
<tr><th>1</th><th>2</th><th>3</th></tr>
<tr><td>Existing PCC-slab design elastic modulus (applicable in situations where the existing intact PCC slab is considered the base)</td><td>The test static elastic modulus E_{TEST} is obtained from (1) coring the intact slab and laboratory testing for elastic modulus or (2) by backcalculation (using FWD deflection data from intact slab and layer thicknesses) and multiplying by 0.8 to convert from dynamic to static modulus. The design existing PCC-slab static elastic modulus is adjusted for unrepaired cracking:
$E_{BASE/DESIGN} = C_{BD} * E_{TEST}$
where E_{TEST} is the static elastic modulus defined above. The C_{BD} is a reduction factor based on the overall PCC condition as follows:
C_{BD} = 0.42 to 0.75 for existing pavement in overall “good” structural condition.
C_{BD} = 0.22 to 0.42 for existing pavement in “moderate” condition.
C_{BD} = 0.042 to 0.22 for existing pavement in “severe” condition
Pavement condition is defined in Table 9-1. A maximum $E_{BASE/DESIGN}$ of 3 million psi is recommended due to existing joints even if few cracks exist.</td><td>$E_{BASE/DESIGN}$ obtained from coring and testing for compressive strength. The compressive strength value is converted into elastic modulus.
The design elastic modulus is obtained as described for Level 1.</td><td>$E_{BASE/DESIGN}$ estimated from historical agency data and local experience for the existing project under design.</td></tr>
<tr><td>Rubblized PCC</td><td>N/A</td><td>N/A</td><td>$E_{BASE/DESIGN}$ typically ranges from 50,000 to 150,000 psi.</td></tr>
</table>

Table 12-10. Description of Existing Pavement Condition

<table>
<tr><th rowspan="2">Existing Pavement Type</th><th colspan="4">Structural Condition</th></tr>
<tr><th>Good</th><th>Moderate</th><th>Severe</th><th>Rubblized</th></tr>
<tr><td>JPCP (percent slabs cracked)</td><td><10</td><td>10–50</td><td>>50 or crack and seat</td><td>Use Rubblized Elastic Modulus</td></tr>
<tr><td>CRCP (Number of punchouts)</td><td><3</td><td>3–10</td><td>>10</td><td>Use Rubblized Elastic Modulus</td></tr>
<tr><td>Flexible pavement (overall estimate of surface cracking)</td><td colspan="4">Excellent: <5% area cracked (estimated)
Good: 5–15% area cracked (estimated)
Fair: 15–35% area cracked (estimated)
Poor: 35–50% area cracked (estimated)
Very Poor: >50% area cracked (estimated)</td></tr>
</table>

Note that the types of transverse cracking referred to are only those due to fatigue damage. Also, repairs and replacement refers to full-depth repair and slab replacement of slabs with transverse cracks only. The percentage of previously repaired and replaced slabs is used to account for past slab repairs/replacements when predicting future cracking. Using the fatigue damage/cracking relationships developed and calibrated nationally for AASHTOWare Pavement ME Design (Please refer to the example shown below).

Example: A survey of the existing pavement shows six percent slabs with transverse cracks and four percent slabs that have been replaced. It is assumed that all replaced slabs had transverse cracks. During pre-restoration repair, five percent of the transversely cracked slabs were replaced leaving one percent still cracked. Inputs to AASHTOWare Pavement ME Design are as follows:

- Six percent slabs with transverse cracks + four percent previously replaced slabs = 10 percent.
- After pre-overlay repair, total percent replaced slabs = nine percent. Note that the percent of slabs still cracked, prior to restoration, is therefore 10 – 9 = 1 percent.

The estimated total fatigue damage is used internally in the design software to estimate the proportion of total fatigue damage due to bottom-up and top-down cracking as follows:

1. Determine future fatigue damage estimates (total damage from percent slabs cracked, top-down damage, and bottom-up damage).
2. Compute the percentage of total fatigue damage due to top-down and bottom-up damage mechanism (e.g., 45 percent top-down and 55 percent bottom-up fatigue damage).
3. Use the computed percentage to divide past total fatigue damage (shown in Table 10-2) into the amounts due to top-down and bottom-up mechanism.

The effect of existing PCC pavement past damage on bonded PCC over existing JPCP/CRCP is negligible and therefore not considered in design. For unbonded JPCP or CRCP overlays over existing rigid pavement, PCC damage in the existing slab is considered through a reduction in its elastic modulus as previously outlined, while for JPCP or CRCP overlays over existing flexible pavement HMA damage is considered as outlined in Section 12.2.

12.3.4 JPCP Rehabilitation Design

Brief descriptions of the following JPCP rehabilitation design options are provided.

- **CPR**—For AASHTOWare Pavement ME Design, CPR is defined as diamond grinding and any combination of the following repair treatments (1) joint-load transfer restoration, (2) retrofit edge drains, (3) full-depth patching, (4) slab replacement, and (5) shoulder replacement. Properly designed and constructed CPR needs to reduce pavement deterioration and prolong pavement life. However, CPR performance also depends on the combination of CPR treatments applied. Each distress could be repaired with an appropriate CPR treatment and one or more preventive treatments applied to provide a cost-effective rehabilitation strategy.

- **Unbonded JPCP Overlay of Existing Rigid Pavement**—Unbonded JPCP overlay (equal to or greater than 6-in.-thick) placed on an existing rigid pavement, composite pavement, or fractured PCC pavement (with an appropriate separation layer). Unbonded overlays (over intact PCC slab) do not require much pre-overlay repair because of a separator layer placed between the overlay and existing pavement. The separator layer is usually a thin HMA layer 1- to 2-in. thick. The purpose of the separator layer is to separate the movements in the existing and overlay concrete layers and to prevent distresses in the existing pavement from reflecting through the overlay. Full-contact friction between the JPCP and the HMA separator layer needs to be assumed over the design life, which was used in the global calibration effort to match PCC slab cracking in the field.
- **Bonded PCC Overlay of Existing JPCP**—Bonded PCC overlays (with thickness 3-5 in.) over existing JPCP involve the placement of a thin concrete layer on top of the prepared existing JPCP to form a permanent monolithic JPCP section. Achieving a long-term bond is essential for good performance. Thus, the existing JPCP slab needs to be in sound condition to help ensure good bonding and little reflection cracking. The monolithic section increases load carrying capacity and provides a new surface for improved rideability and friction resistance.
- **JPCP Overlay of Existing Flexible Pavement**—Conventional JPCP overlays (thickness ≥6 in.) of existing flexible pavements can be handled in AASHTOWare Pavement ME Design. When subjected to axle loads, the JPCP overlaid flexible pavement behaves similar to a new JPCP with an HMA base course and other underlying layers. For this design, the contact friction between the JPCP and the existing surface of the HMA could be full friction throughout the design life. Efforts during construction such as milling the top surface will enhance the contact friction between the JPCP and HMA surface.

Design Considerations

- **Performance Criteria**—Performance indicators used for JPCP rehabilitation design are (1) transverse joint faulting, (2) transverse cracking, and (3) smoothness or IRI. These are used by AASHTOWare Pavement ME Design to evaluate the adequacy of trial designs.
- **Design Reliability**—Handled same as for new design (see Chapter 7).
- **Factors That Affect Distress—**A detailed description of the factors that affect the performance indicators noted above for JPCP rehabilitation design are presented in Table 12-11. By selecting the appropriate values of these factors, designers may reduce specific distress and improve overall pavement performance in a cost-effective manner.

Table 12-11. Summary of Factors That Influence Rehabilitated JPCP Distress

Parameter	Distress Type		Comment
	Transverse Joint Faulting	Transverse Cracking*	
Presence of dowels and dowel diameter	✓		Restored JPCP could be retrofitted with dowels while dowels could be specified for unbonded JPCP overlays and JPCP overlays over existing flexible pavements.
Overlay PCC thickness.	✓	✓	Overlay slab thickness can be modified.
Overlay PCC flexural strength		✓	The flexural strength of JPCP overlays can be increased to reduce cracking. Increasing strength generally results in increased elastic modulus which leads to an increase in pavement stresses and partially offsets benefits of increased strength.
Joint spacing	✓	✓	Joint spacing can be modified for unbonded JPCP overlays and JPCP overlays of existing HMA pavements.
Use of HMA separation layer	✓		HMA separation layer (base) erodibility significantly influences faulting. A non-erodible HMA layer should be specified that will not strip.
Contact friction between JPCP and flexible pavement surface		✓	Full contract friction for unbonded JPCP overlays of existing PCC pavements when separated with an HMA layer should be input. The full contract friction for JPCP overlays of existing flexible pavements should be full over the entire design life.
Placement of vehicle loads near unsupported pavement edges.	✓		Use of 12–to 24in. widened slabs or tied PCC shoulders provides significantly improved edge support from lateral truck wander.
Poor slab edge support (e.g., lack of widened lanes or tied PCC shoulders).	✓	✓	Existing JPCP can be retrofitted with tied PCC shoulder to improve edge support while JPCP overlays can be constructed with tied PCC shoulders or widened slabs.
Subsurface drainage	✓		Including an open-graded separator layer for unbonded JPCP or retrofitting restored JPCP and bonded JPCP overlays will reduce the potential for joint faulting.
Permanent curl/warp	✓	✓	Permanent curl/warp of the overlay slab can be controlled by adopting sound mix design and construction curing practices.
Subgrade stiffness (*k*-value)		✓	For rehabilitation, the designer mostly has no control over these parameters. Design features can be selected however to mitigate the negative effects of such parameters if they pose a problem.
Stabilized base thickness		✓	
Shrinkage of slab surface		✓	JPCP overlay mix design should minimize shrinkage.
CTE (α_{PCC})	✓	✓	Aggregate materials should be selected to reduce CTE so as to reduce stresses induced in the PCC due to temperature differences and thermal gradients

* For both bottom-up and top-down cracking.

Trial Rehabilitation with JPCP Designs

Design Process Summary

A generic overview of rehabilitation design is provided in Section 12.1. As with new pavement design, the first step in rehabilitation design is to select a trial design with defined layers, material types and properties, and relevant design features based on the future level of traffic anticipated. This is followed by the selection of the design performance criteria (used for evaluating the adequacy of the trial design) and the desired level of reliability. Next, AASHTOWare Pavement ME Design software is used to process the input data. Data processing includes estimating climate-related aspects such as pavement temperature profile for each analysis period using the EICM and computing long-term PCC flexural strength, as discussed in Section 5.4.

Next, the processed data is used to perform a design analysis by computing pavement structural responses (stress, deflections) required for each distress type incrementally. Computed structural responses are used in transfer functions to estimate distress and smoothness.

The trial rehabilitation design is then evaluated for adequacy using prescribed performance criteria at the given reliability level. Trial designs deemed inadequate are modified and reevaluated until a suitable design is achieved. Design modifications could range from making simple changes to JPCP overlay thickness, varying joint spacing, varying PCC strength, or adopting a new rehabilitation strategy altogether.

The design process for rehabilitation design with JPCP overlays or CPR of existing JPCP is very similar to new or reconstructed JPCP design. Some exceptions are noted in the sections below.

Performance Prediction Models

The globally calibrated performance models for new pavements apply for rehabilitation design as well with one exception—the JPCP CPR faulting prediction model has slightly different coefficients than the corresponding one new or reconstructed JPCP.

Materials Inputs

In terms of materials inputs, the key difference between new and rehabilitation design is that the latter deals with characterizing in situ materials properties along with those for the overlay. A description of the material inputs for existing pavement layers and how to estimate them is presented in Chapter 9.

Selection of Design Features

The choice of design features is restricted to those variables being introduced as part of the rehabilitation. For most rehabilitated JPCP design situations, the pavement design features is a combination of the existing design features and new features introduced as part of rehabilitation. Selecting the appropriate design features for the rehabilitated JPCP is key to achieving a successful design. Guidance on how to select the right design features is presented in Table 12-12.

Design Modifications to Reduce Distress for JPCP Rehabilitation

Trial designs with excessive amounts of predicted distress/smoothness need to be modified to reduce predicted distress/smoothness to tolerable values (within the desired reliability level). Some of the most effective ways of accomplishing this are listed in Table 12-13.

Table 12-12. Guidance on How to Select the Appropriate Design Features for Rehabilitated JPCP Design

Type of JPCP Rehabilitation	Specific Rehabilitation Treatments	Recommendation on Selecting Design Feature
Concrete Pavement Restoration (CPR)	Diamond grinding	Select initial smoothness (IRI) based on agency grinding specifications and values typically achieved on CPR projects. If significant settlements/heaves exist the initial IRI should be set higher than new/reconstruction design.
	Load transfer restoration (LTR)	Select load transfer mechanism based on the type of retrofit load transfer mechanism installed (e.g., 1.5-in. dowels). For situations were LTR was not applied, the existing JPCP LTE must be assessed. Existing doweled JPCP with very poor LTE may be considered undoweled.
	Shoulder repair, retrofit, replacement	A new edge support condition reflective of the repairs, retrofit, or replacement applied. For example if an existing asphalt shoulder is replaced with tied PCC shoulders, the rehabilitated design must reflect this change in edge support. Also, where no shoulder repair is carried out, the condition of the current shoulder must be considered in characterizing edge support conditions.
	Retrofit edge drains	The rehabilitated JPCP design should reflect improved drainage conditions by upgrading the base erodobility.
	Full-depth repairs, slab replacement	The effect on full-depth repairs and/or slab replacement on existing damage and future cracking estimates must be fully accounted for.
Unbonded JPCP Overlay	Separation layer	An HMA separator layer prevents reflection of underlying joints and cracks, provides a highly erosion resistant material, and provides sufficient contact friction so that joints will form in the JPCP overlay. The JPCP overlay behaves structurally as if it is built on a strong non-erodible "base" course consisting of the HMA separation layer and the existing slab. The program combines structurally the JPCP overlay and the HMA separator layer into an equivalent slab. Full contact friction interface shou ld be input over the entire design life. The HMA material must be specified to be extremely resistant to stripping.
	Exiting PCC condition	The existing PCC overall condition must be considered in selecting the appropriate layer elastic modulus. This is done by adjusting backcalculated or lab tested estimates of elastic modulus with a damage factors determined based on existing JPCP visual condition.
	JPCP overlay	Selection of design features for the JPCP overlay (including shoulder type and slab width) is similar to that outlined for new design in Chapter 10 of this manual.
Bonded JPCP Overlay	PCC overlay	Design features must reflect the condition of the existing pavement as very few pre-overlay repairs are typically done for this rehabilitation.
JPCP Overlay Over Existing Flexible Pavement	JPCP overlay	Selection of design features for the JPCP overlay (including shoulder type and slab width) is similar to that outlined for new or reconstructed design in Chapter 10. Condition of existing flexi ble pavement is rated as Excellent, Good, Fair, Poor, or Very Poor as defined in Table 12-10. These ratings will result in adjustments to the dynamic modulus E_{HMA} of the existing HMA layer that now becomes the base course. Full friction should be input over the full design life of the concrete overlay.

Table 12-13. Recommendations for Modifying Trial Design to Reduce Distress/Smoothness for JPCP Rehabilitation Design

Distress Type	Recommended Modifications to Design
Faulting	**Include dowels or increase diameter of dowels.** This is applicable to both restored JPCP and non-doweled JPCP overlays. The use of properly sized dowels is generally the most reliable and cost-effective way to control joint faulting. A slight increase of diameter of the dowels (i.e. , 0.25 in) will significantly reduce the mean steel-to-PCC bearing stress and, thus, the joint faulting. **Improve subsurface drainage.** This is applicable to both restored JPCP and JPCP overlays. Subsurface drainage improvement for rehabilitated pavements basically consists of providing retrofit edge-drains and other related facilities. For unbonded JPCP over existing rigid pavements a permeable separator layer (usually asphalt or chemically stabilized) can be used to improve drainage. Studies have shown that subsurface drainage improvement with retrofit edge-drains can reduce faulting, especially for non-doweled JPCP. This is considered in design by reducing the amount of precipitation infiltrating into the pavement structure. **Widen the traffic lane slab by 1 to 2 ft.** This is applicable to JPCP overlays. Widening the slab effectively moves the wheel load away from the slab corner, greatly reducing the deflection of the slab and the potential for erosion and pumping. Studies have shown that slab widening can reduce faulting by about 50 percent. **Decrease joint spacing.** This is applicable to JPCP overlays over existing flexible pavements and unbonded JPCP overlays. Shorter joint spacing generally result in smaller joint openings, making aggregate interlock more effective and increasing joint LTE. **Erodibility of separator layer.** This is mostly applicable only to unbonded JPCP overlays. It may be applicable to the leveling course placed during the construction of JPCP overlays of existing flexible pavements. Specifying a non-erodible HMA material as the separator reduces the potential for base/underlying layer erosion and, thus, faulting.
Transverse Cracking	**Increase slab thickness.** This is only applicable to JPCP overlays. Thickening the overlay slab is an effective way to decrease critical bending stresses from both truck axle loads and from temperature differences in the slab. Field studies have shown that thickening the slab can reduce transverse cracking significantly. At some thickness, however, a point of diminishing returns is reached and fatigue cracking does not increase significantly. **Decrease joint spacing.** This is only applicable to JPCP overlays. A shorter joint spacing results in lower curling stresses in the slab. This effect is very significant, even over the normal range of joint spacing for JPCP, and should be considered a critical design feature. **Increase PCC strength (and concurrent change in PCC elastic modulus and CTE).** This is applicable only to JPCP overlays. By increasing the PCC strength, the modulus of elasticity also increases, thereby reducing its effect. The increase in modulus of elasticity will actually increase the critical bending stresses in the slab. There is probably an optimum PCC flexural strength for a given project that provides the most protection against fatigue damage. **Widen the traffic lane slab by 2 ft.** This is applicable to rehabilitation with overlays. Widening the slab effectively moves the wheel load away from the longitudinal free edge of the slab, thus, greatly reducing the critical bending stress and the potential for transverse cracking **Add a tied PCC shoulder (monolithically placed with the traffic lane**). This is applicable to rehabilitation with or without overlays. The use of monolithically placed tied-PCC shoulder that has the properly sized tie-bars is generally an effective way to reduce edge bending stress and reduce transverse cracking. A PCC shoulder that is placed after the traffic lane does not generally produce high LTE and significantly reduces bending stresses over the design period.
Smoothness	**Build smoother pavements initially and minimizing distress**. The smoothness prediction model shows that smoothness loss occurs mostly from the development of distresses such as cracking, faulting, and spalling. Minimizing or eliminating such distresses by modifying trial design properties that influence the distresses would result in a smoother pavement. Hence, all of the modifications discussed in previous sections (for cracking and faulting) are applicable to improving smoothness.

12.3.5 CRCP Rehabilitation Design

A brief description of the CRCP rehabilitation designs options is described in this section.

- **Unbonded CRCP Overlay of Existing Rigid Pavement**—Unbonded CRCP (thickness ≥7 in.) placed on existing intact concrete pavement (JPCP, JRCP, or CRCP), existing composite pavement, or fractured PCC pavement. Unbonded overlays must have a separator layer similar to that described for unbonded JPCP overlays (see Section 12.3.3).
- **Bonded PCC Overlay of Existing CRCP**—Bonded PCC overlays over existing CRCP involve the placement of a thin concrete layer atop the prepared existing CRCP to form a permanent monolithic CRC section.
- **CRCP Overlay of Existing Flexible Pavement**—Conventional CRCP overlays (thickness >7 in.) can be applied to existing flexible pavements. When subjected to axle loads, the CRCP overlaid flexible pavement behaves similar to a new CRCP with an asphalt base course.

Design Considerations

Performance Criteria—Performance indicators used for CRCP rehabilitation design are (1) crack width, (2) crack load transfer efficiency (LTE), (3) punchouts, and (4) smoothness.

Design Reliability—Handled same as for new design (see Chapter 7).

Factors That Affect Distress—A detailed description of the factors that affect the performance indicators noted above to CRCP rehabilitation design are presented in Table 12-14. By selecting the appropriate values of these factors, designers may reduce specific distress and improve overall pavement performance.

Trial Rehabilitation with CRCP Designs

The rehabilitation design process described under Section 12.3.3 for JPCP rehabilitation design is valid for CRCP as well. The performance prediction models for new CRCP are also valid for CRCP overlays. Further, as with JPCP rehabilitation, selecting the appropriate design features for the rehabilitated CRCP is key to achieving a successful design. For most rehabilitated CRCP design situations, the pavement design features is a combination of the existing design features and new features introduced as part of rehabilitation. Guidance on how to select the appropriate design features is presented in Table 12-15.

Design Modifications to Reduce Distress for CRCP Overlays

Crack width, longitudinal reinforcement percentage, slab thickness, and support conditions are the primary factors affecting CRCP performance and punchout development and hence modifying the factors that influence them is the most effective manner of reducing punchouts and smoothness loss. Crack spacing cannot be modified for bonded PCC over existing CRCP.

Table 12-14. Summary of Factors that Influence Rehabilitated CRCP Distress and Smoothness

Parameter	Comment
Transverse Crack Width and Spacing	Transverse crack width is very critical to CRCP performance. It plays a dominant role in controlling the degree of load transfer capacity provided at the transverse cracks. It is strongly influenced by the reinforcement content, PCC shrinkage, construction PCC set temperature, and PCC CTE. Smaller crack widths increase the capacity of the crack for transferring repeated shear stresses (caused by heavy axle loads) between adjacent slab segments over the long term. Wider cracks exhibit lower and lower LTE over time and traffic, which results in increased load-related critical tensile stresses at the top of the slab, followed by increased fatigue damage and punchouts. A maximum crack width of 0.020-in. over the design life is recommended.
Transverse Crack LTE	The load transfer of transverse cracks is a critical factor in controlling the development of punchout related longitudinal cracking. Maintaining load transfer of 95 percent or greater (through aggregate interlock over the CRC overlay design life) will limit the development of punchout distress. This is accomplished by limiting crack width over the entire year, especially the cold months.
Lane to Shoulder Longitudinal Joint Load Transfer	The load transfer of the lane to shoulder joint affects the magnitude of the tensile bending stress at the top of the slab (between the wheel loads in a transverse direction)—the critical pavement response parameter that controls the development of longitudinal cracking between adjacent transverse cracks and, consequently, the development of punchout. The use of design features that could provide and maintain adequate edge support throughout the pavement rehabilitation design life is therefore key to adequate performance.
Overlay CRC Thickness	This is an important design feature from the standpoint of slab stiffness that has a very significant influence on performance. Note that for bonded PCC over existing CRCP the equivalent stiffness of the overlay and existing PCC layer is used in analysis. In general, as the slab thickness of a CRC overlay increases, the capacity to resist critical bending stress increases, as does the slab's capability to transfer load across the transverse cracks. Consequently, the rate of development of punchouts decreases and smoothness loss is also reduced.
Amount of Longitudinal Reinforcement and Depth of Reinforcement	Longitudinal steel reinforcement is an important design parameter because it is used to control the opening of the transverse cracks for unbonded CRCP overlays and CRCP overlays over existing flexible pavement. Also, the depth at which longitudinal reinforcement is placed below the surface also greatly affects crack width. It is recommended that longitudinal steel reinforcement be placed above mid-depth in the slab. For bonded PCC over existing CRCP, the amount of reinforcement entered into the models is the same as that of the existing CRCP because cracks are already formed and no reinforcement is placed in the overlay PCC. Depth of the steel reinforcement is equal to the depth to the reinforcement in the existing CRCP (ignore the overlay PCC thickness because cracks are already formed through the slabs).
Slab Width	Slab width has typically been synonymous with lane width (usually 12 ft). Widened lanes typically are 14 ft. Field and analytical studies have shown that the wider slab keeps truck axles away from the free edge, greatly reducing tensile bending stresses (in the transverse direction) at the top slab surface and deflections at the lane-shoulder joint. This has a significant effect on reducing the occurrence of edge punchouts. This design procedure does not directly address CRCP with widened slabs but can be approximately modeled by shifting the mean lateral load position by the width of slab widening.

Table 12-15. Guidance on How to Select the Appropriate Design Features for Rehabilitated CRCP Design

Type of CRCP Rehabilitation	Specific Rehabilitation Treatments	Recommendation on Selecting Design Feature
Unbonded CRCP Overlay	Interlayer placement	An adequate asphalt separator layer is very important for a CRCP overlay to ensure that no working joints or cracks in the existing pavement will reflect upward through the CRCP. This normally requires 1 in. of HMA but if joints with poor LTE exist then a thicker HMA layer may be necessary. The HMA separator layer should have normal contact friction with the CRCP overlay and the existing PCC layer to improve the structural capacity of the pavement. Erodibility of the separation layer is calculated based upon properties of the HMA separation layer which utilizes percent asphalt by volume. If this separation layer is permeable with a typically very low asphalt content, the designer must adjust the percent asphalt to a value of 11 percent.
	Exiting PCC condition	The existing PCC overall condition must be considered in selecting the appropriate layer elastic modulus. This is done by adjusting backcalculated or lab tested estimates of elastic modulus with a damage factors determined based on existing CRCP visual condition.
	CRCP overlay	Selection of design features for the CRCP overlay (including shoulder type and slab width) is similar to that outlined for new/reconstruction design in Chapter 10.
Bonded PCC Overlay on CRCP	PCC bonded overlay	The existing CRCP surface must be prepared and a new PCC overlay bonded on top. The only joint that needs sawing is the longitudinal lane to lane joint which should be sawed completely through plus ½-in. This bonded PCC design is unusual but has performed well in a number of projects in Texas and elsewhere. Design input features must reflect the condition of the existing CRCP.
CRCP Overlay Over Existing Flexible Pavement	CRCP overlay	Selection of design features for the CRCP overlay (including shoulder type and slab width) is similar to that outlined for new or reconstructed design in Chapter 10. Condition of existing flexible pavement is rated as Excellent, Good, Fair, Poor, or Very Poor as described in Table 12-10. These ratings will result in adjustments to the dynamic modulus E_{HMA} of the existing HMA layer that now becomes the base course. The lower the rating the larger the downward adjustment of E^* of the existing HMA layer.

Increase overlay slab thickness. An increase in CRCP slab thickness will reduce punchouts based on (1) a decrease in critical tensile fatigue stresses at the top of the slab, (2) an increase in crack shear capability and a greater tolerance to maintain a high-load transfer capability at the same crack width that also allows for reduced tensile stress at top of the slab.

Increase percent longitudinal reinforcement in overlay. Even though an increase in steel content will reduce crack spacing, it has been shown to greatly reduce punchouts overall due to narrower cracks widths.

Reduce the PCC Set Temperature (when PCC sets) through improved curing procedure (water curing). The higher the PCC zero-stress temperature the wider the crack openings at lower temperature.

Reduce the depth of reinforcement in overlay. This is applicable only to unbonded CRCP overlay and CRCP over existing flexible pavement. Placement of steel closer to the pavement surface reduces punchouts through keeping cracks tighter. (However, do not place closer than 3.5 in. from the surface to avoid construction problems and limit infiltration of chlorides.)

Increase PCC tensile strength. Increasing of CRCP tensile strength decreases the fatigue damage and hence punchouts. It must be noted however that there is a corresponding increase in PCC elastic modulus which increases the magnitude of stresses generated within the PCC reducing the benefit of increase tensile strength somewhat.

Reduce coefficient of thermal expansion of overlay PCC. Use of a lower thermal coefficient of expansion concrete will reduce crack width opening for the same crack spacing.

Increase HMA separator layer thickness. The thicker the separator layer the less sensitive the overlay is to the deterioration in the existing pavement. For badly deteriorated existing pavements thick (thickness ≥3 in.) HMA separator layers are recommend for CRCP overlays.

Reduction in PCC shrinkage. Reducing the cement content and improved curing are two ways to reduce ultimate shrinkage.

12.3.6 Additional Considerations for Rehabilitation with PCC

There are several important considerations that need to be addressed as part of rehabilitation design to ensure adequate performance of the rehabilitation design throughout its design life. These issues include:

Shoulder reconstruction.
Subdrainage improvement.
CPR/pre-overlay repairs.
Separator layer design (for unbonded JPCP/CRCP over existing rigid pavements).
Joint design (for JPCP overlays).
Reflection crack control (for bonded PCC over existing JCPC/CRCP).
Bonding (for bonded PCC overlays over existing JPCP/CRCP).
Guidelines for addition of traffic lane.
Guidelines for widening of narrow traffic lanes.

CHAPTER 13

Interpretation and Analysis of the Trial Design

AASHTOWare Pavement ME Design software predicts the performance of the trial design in terms of key distress types and smoothness at a specified reliability (refer to Chapter 5). The designer initially decides on a "trial design" for consideration, as discussed in Chapters 11 and 12. This trial design may be obtained from the current G*uide for the Design of Pavement Structures* (*1*), the result of another design program, a design catalog, or a DOT design procedure.

AASHTOWare Pavement ME Design software analyzes that trial design over the selected design period. The program outputs the following information: inputs, reliability of design, materials and other properties, and predicted performance. Each of these outputs needs to be examined by the designer to achieve a satisfactory design as described in this section. An unacceptable design is revised and re-run to establish its performance until all criteria are met. This "trial and error" process allows the pavement designer to simulate building the pavement prior to letting a contract for construction to ensure that the performance expectations will be met as economically as possible.

The purpose of this section is to provide some guidance on what design features could be revised for the trial design to be accepted.

13.1 SUMMARY OF INPUTS FOR THE TRIAL DESIGN

A unique feature of AASHTOWare Pavement ME Design software is that all of the actual program inputs are included in the output file. AASHTOWare Pavement ME Design includes both climatic data and axle configuration in the output, which is a change from the original NCHRP beta-version software. In addition, AASHTOWare Pavement ME Design generates a PDF file output as well as Excel-downloadable files. The designer can review all of these inputs to ensure that the data entered are complete. Given the large number of inputs, this check is essential.

13.2 RELIABILITY OF TRIAL DESIGN

Another important output is an assessment of the design reliability, which may be seen under the Grand Summary tab. The Target and Prediction Distress @ Specified Reliability are the first left-hand columns listed, followed by the Target and Achieved Reliability. If the Achieved Reliability is greater than the Target Reliability then the pavement passes. If the reverse is true, then the pavement fails. If any key distress fails, the designer needs to alter the trial design to correct the problem.

Examples are shown below for a flexible and rigid pavement (Tables 13-1 and 13-2, respectively).

- For the flexible pavement example (Table 13-1), the asphalt concrete (AC) surface down cracking met the reliability criterion (99.92 > 90 percent), but terminal IRI did not (52.51 < 90 percent). This trial design is not acceptable at the 90 percent reliability level and needs to be revised.
- For the JPCP example (Table 13-2), the mean joint faulting met the reliability criterion (98.09 > 95 percent), but terminal IRI did not (93.98 < 95 percent). This trial design is not acceptable at the 95 percent reliability level and needs to be revised.

Table 13-1. Reliability Summary for Flexible Pavement Trial Design Example

Distress Prediction Summary					
Distress Type	Stress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mi)	172	169.3	90	52.51	Fail
Permanent deformation—total pavement (in.)	0.75	5	90	99.92	Pass
AC bottom-up fatigue cracking (percent)	25.00	0.1	90	99.999	Pass
AC thermal fracture (ft/mi)	250	1	90	94.16	Pass
AC top-down fatigue cracking (ft./mile)	2000	0.71	90	1.66	Fail
Permanent deformation—AC only (in.)	0.25	169.3	90	59.13	Fail

Table 13-2. Reliability Summary for JPCP Trial Design Example

Distress Prediction Summary					
Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mi)	**172**	**112.5**	**95**	**93.83**	**Fail**
Mean joint faulting (in.)	**0.12**	**21.2**	**95**	**32.9**	**Fail**
JPCP transverse cracking (percent slabs)	**15**	**0.051**	**95**	**98.09**	**Pass**

13.3 SUPPLEMENTAL INFORMATION (LAYER MODULUS, TRUCK APPLICATIONS, AND OTHER FACTORS)

Another unique feature of AASHTOWare Pavement ME Design software is that the materials properties and other factors are output on a month-by-month basis over the design period. The designer needs to examine the output materials properties and other factors to assess their reasonableness. For flexible pavements, the output provides the HMA dynamic modulus (E_{HMA}) and the resilient modulus (M_r) for unbound layers for each month over the design period. Moisture content and frost condition greatly affects the unbound materials M_r.

AASHTOWare Pavement ME Design provides a graphical output of selected modulus values for the HMA layers. The dynamic modulus for the first quintile of temperatures (the lower temperatures) for each sublayer is plotted over the design life of the pavement. All HMA dynamic modulus values for each temperature quintile and sublayer are included in a tabular format. In addition, the resilient modulus for the unbound layers and foundation are also included in that tabular format for each month over the design life of the pavement.

The designer should examine the monthly output materials properties, number of trucks (Class 4 and higher), and other factors to assess their reasonableness. These are all output at the end of the month.

- Flexible pavements key outputs that need to be observed and evaluated include the following.
  - HMA Dynamic Modulus (E_{HMA}) of each layer. The software divides each HMA input layer into sublayers and each need to be examined for reasonableness. Materials properties as well as temperature and load speed typically have significant effects on E_{HMA}.
  - Unbound material resilient modulus (M_r) for unbound layers for each month over the design period can be examined. The software divides each unbound material input layer (such as a granular base course) into sublayers and each need to be examined for reasonableness. Moisture content and frost condition greatly affects the unbound materials M_r.
  - The number of cumulative Heavy Trucks (Class 4 and above) are output shown for the design traffic lane. The total cumulative Heavy Trucks may be examined at the last month of the analysis period. This parameter is a good general indicator of how heavy the truck traffic (volume) is for the design (e.g., 1 million trucks, 20 million trucks, or 100 million trucks is the terminology recommended for design purposes). Note that these may be converted into flexible pavement 18-kip ESALs by multiplying them by an average truck factor, or the actual number of ESALs may be determined by examining an intermediate file by this name that has this information.
- Rigid pavements key outputs that need to be observed and evaluated include the following.
  - Flexural strength/modulus of rupture of PCC—represents the bending strength of the PCC over all months of the design period.
  - Modulus of elasticity of PCC—represents the traditional elastic modulus of the PCC over all months of the design period.
  - Unbound material resilient modulus (M_r) for unbound layers for each month over the design period may be examined. See above for flexible pavements.

– Subgrade *k*-value—this value is backcalculated for each monthly condition of slab *E*, base and subbase modulus [E_{HMA} for HMA, *E* for cement treated, and unbound material resilient modulus (M_r)], and subgrade M_r.
– The number of cumulative "Heavy Trucks" (Class 4 and above) are output shown for the design traffic lane. The total cumulative "Heavy Trucks" may be examined at the last month of the analysis period. This parameter is a good general indicator of how heavy the truck traffic (volume) is for the design (e.g., 1 million trucks, 20 million trucks, or 100 million trucks is the terminology recommended for design purposes). Note that these may be converted into rigid pavement 18-kip ESALs by multiplying them by an average truck factor, or the actual number of ESALs may be determined by examining an intermediate file by this name that has this information.

13.4 PREDICTED PERFORMANCE VALUES

The software outputs month-by-month the key distress types and smoothness over the entire design period. A designer can examine the outputs as a test for engineering reasonableness by checking that they meet the specified performance criteria.

- Flexible pavements.
  – **Longitudinal fatigue cracking**—Surface-initiated fatigue cracking occurs in or at the edges of the wheel paths, propagating in the direction of travel. A critical value is reached when longitudinal cracking accelerates and begins to require significant repairs and lane closures.
  – **Alligator fatigue cracking**—Traditional bottom-up fatigue cracking in the wheel paths. A critical value is reached when alligator cracking accelerates and begins to require significant repairs and lane closures.
  – **Transverse cracking**—Excessively low temperatures may result in fracture cracks that span across traffic lanes, perpendicular to the direction of travel. A critical value is reached when transverse cracking results in significant roughness.
  – **Rutting or permanent deformation**—HMA rutting is only in the asphalt bound layers and total rutting combines all of the pavement layers and the subgrade. A critical value is reached when rutting becomes sufficient enough to cause safety concerns such as hydroplaning in inclement weather.
  – **IRI**—This index represents the profile of the pavement in the wheel paths. A critical value is reached as judged by highway users as unacceptable ride quality. IRI is a function of longitudinal cracking, transverse cracking, alligator cracking, and total rutting along with climate and subgrade factors.
  – **Reflection cracking**—Reflection cracking occurs only when an HMA overlay is placed over an existing flexible pavement that has alligator fatigue cracking in the wheel paths, or over a jointed rigid pavement where transverse joints and cracks exist and occur. A critical value is reached when reflection alligator cracking results in significant maintenance requirements or when reflection transverse cracking results in significant maintenance requirements or roughness.
- Rigid pavements (JPCP).
  – **Joint faulting**—The mean joint faulting at the outer slab edge of the heaviest trafficked lane is an indicator of erosion of sublayers and the effectiveness of joint LTE. A critical value is reached

when joint faulting results in excess roughness which is unacceptable to drivers and also difficult to remove through retexturing.

  - **Percent slabs cracked**—The mean predicted transverse cracks (in the heaviest trafficked lane) that form as a result of fatigue damage at both the top and bottom of the slab. The location (either top or bottom) of the most damage can be determined from output tables and graphs. Significantly higher fatigue damage at the top of the slab means it will initiate cracking from the top down. A critical value is reached when cracking accelerates and begins to require significant repairs and lane closures.
  - **IRI**—This index represents the profile of the pavement in the wheel paths. A critical value is reached as judged by highway users as unacceptable ride quality. IRI is a function of joint faulting and slab cracking along with climate and subgrade factors.

- Rigid pavements (CRCP).
  - **Crack spacing**—Transverse shrinkage cracks occur due to the restraint caused by the steel and drying shrinkage and cooling of the PCC slab. It is output on the crack width graph. A value of 3 to 6 ft is desirable.
  - **Crack width**—A very critical parameter that varies with temperature of the PCC at set, crack spacing, shrinkage of the PCC over time, reinforcement content, and base friction. A critical value of less than 0.020 in. is required to maintain crack LTE at high levels.
  - **Crack LTE**—Crack deterioration or loss of load transfer ability must be carefully controlled. Crack LTE should remain above 90 to 95 percent throughout the design life. When crack LTE is reduced the potential for punchouts to develop increases greatly.
  - **Punchouts**—Caused by fatigue damage at the top of the slab between two closely spaced transverse cracks that result in a short longitudinal crack. The rectangular piece of PCC formed by the two narrow transverse cracks and the longitudinal crack about 48 in. from the slab edge comprises the area termed as a punchout. The punchout may further disintegrate over time and with repetitive heavy loads. A critical value is reached when the punchouts accelerate and begin to require significant repairs and lane closures.
  - **IRI**—This index represents the profile of the pavement in the wheel paths. A critical value is reached as judged by highway users as unacceptable ride quality. IRI is a function of punchouts and climate and subgrade factors.

13.5 JUDGING THE ACCEPTABILITY OF THE TRIAL DESIGN

While layer thickness is important, many other design factors also affect distress and IRI or smoothness. The designer needs to examine the performance prediction and determine which design feature to modify to improve performance (e.g., layer thickness, materials properties, layering combinations, geometric features, and other inputs). This section provides guidance on revising the trial design when the performance criteria have not been met.

The guidance given is distress-specific. The designer needs to be aware, however, that changing a design feature to reduce one distress might result in an increase in another distress. As an example, for excessive transverse cracking of a HMA pavement for which Level 3 inputs were used, the user may consider designing with a softer asphalt to address the occurrence of transverse cracking. However, affecting this change may also increase the rutting predicted in the HMA layer itself.

More importantly, some of the input parameters are interrelated; changing one parameter might result in an unwanted change to another one. For example, decreasing asphalt content to make the HMA mixture more resistant to rutting will likely increase the in-place air voids resulting in more fatigue cracking. It is important that designers monitor the resultant impacts of changing individual layer properties. It should be noted that some of these modifications are construction dependent and may be difficult to anticipate prior to building the pavement or placing the HMA overlay.

Table 13-3. Guidance for Modifying HMA Trial Designs to Satisfy Performance Criteria

Distress and IRI	Design Feature Revisions to Minimize or Eliminate Distress
Alligator Cracking (Bottom Initiated)	Increase thickness of HMA layers. For thicker HMA layers (>5-in.) increase dynamic modulus. For thinner HMA layers (<3-in.) reduce dynamic modulus. Revise mixture design of HMA-base layer (increase percent crushed aggregate, use manufactured fines, increase asphalt content, use a harder asphalt but ensure that the same percent compaction level is achieved along the roadway, use a polymer modified asphalt, etc.) Increase density, reduce air void of HMA-base layer. Increase resilient modulus of aggregate base (increase density, reduce plasticity, reduce amount of fines, etc.)
Thermal Transverse Cracking	Use softer asphalt in the surface layer Reduce the creep compliance of the HMA-surface mixture Increase the indirect tensile strength of the HMA-surface mixture Increase the asphalt content of the surface mixture
Rutting in HMA	Increase the dynamic modulus of the HMA layers Use a polymer modified asphalt in the layers near the surface. Increase the amount of crushed aggregate Increase the amount of manufactured fines in the HMA mixtures Reduce the asphalt content in the HMA layers
Rutting in Unbound Layers and Subgrade	Increase the resilient modulus of the aggregate base; increase the density of the aggregate base Stabilize the upper foundation layer for weak, frost susceptible, or swelling soils; use thicker granular layers. Place a layer of select embankment material with adequate compaction Increase the HMA thickness
IRI HMA	Require more stringent smoothness criteria and greater incentives (building the pavement smoother at the beginning). Improve the foundation; use thicker layers of non-frost susceptible materials Stabilize any expansive soils Place subsurface drainage system to remove ground water.
Longitudinal Fatigue Cracking (Surface Initiated)	Note: Refer to Chapter 3; it is recommended that the surface initiated crack prediction equation not be used as a design criterion until the critical pavement response parameter and prediction methodology has been verified. The cumulative damage and longitudinal cracking transfer function (Eqs. 5-5 and 5-8) should be used with caution in making design decisions regarding the adequacy of a trial design, in terms of longitudinal cracking (top-down cracking). Reduce the dynamic modulus of the HMA-surface course. Increase HMA thickness. Use softer asphalt in the surface layer. Use a polymer modified asphalt in the surface layer; the AASHTOWare Pavement ME Design does not adequately address the benefit of PMA mixtures.
Reflection Cracking	Note: It is recommended that the amount of reflection cracks not be used as a design criterion until the prediction equation has been calibrated. Increase HMA overlay thickness. Increase the modulus of the HMA overlay.

Table 13-4. Guidance on Modifying JCPC Trial Designs to Satisfy Performance Criteria

Distress and IRI	Modifications to Minimize or Eliminate
Joint Crack Width	• Build JPCP to set at lower temperature (cool PCC, place cooler temperatures). • Reduce drying shrinkage of PCC (increase aggregate size, decrease w/c ratio, decrease cement content). • Decrease joint spacing. • Reduce PCC coefficient of thermal expansion.
Joint LTE	• Use mechanical load transfer devices (dowels). • Increase diameter of dowels. • Reduce joint crack width (see joint crack width recommendations). • Increase aggregate size.
Joint Faulting	• Use mechanical load transfer devices (dowels). • Increase slab thickness. • Reduce joint width over analysis period. • Increase erosion resistance of base (specific recommendations for each type of base). • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient. • PCC tied shoulder. • Widened slab (by 1 to 2 ft).
Slab Cracking	• Increase slab thickness. • Increase PCC strength. • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient. • PCC tied shoulder (separate placement or monolithic placement better). • Widened slab (1 to 2 ft). • Use PCC with lower coefficient of thermal expansion.
IRI JPCP	Require more stringent smoothness criteria and greater incentives.

Table 13-5. Guidance on Modifying CRCP Trial Designs to Satisfy Performance Criteria

Distress and IRI	Modifications to Minimize or Eliminate
Crack Width	• Build CRCP to set at lower temperature (cool PCC, place cooler temperatures). • Reduce drying shrinkage of PCC (increase aggregate size, decrease w/c ratio, decrease cement content). • Increase percent longitudinal reinforcement. • Reduce depth of reinforcement (minimum depth 3.5 in.). • Reduce PCC coefficient of thermal expansion.
Crack LTE	• Reduce crack width (see crack width recommendations). • Increase aggregate size. • Reduce depth of reinforcement.
Punchouts	• Increase slab thickness. • Increase percent longitudinal reinforcement. • Reduce crack width over analysis period. • Increase PCC strength. • Increase erosion resistance of base (specific recommendations for each type of base). • Minimize permanent curl/warp through curing procedures that eliminate built-in temperature gradient. • PCC tied shoulder or widened slab.
IRI CRCP	Require more stringent smoothness criteria and greater incentives.

Once again, it should be noted that a designer may or may not get the desired effects listed in the tables above, depending on the structural profile and location of the pavement. The relationships reported in the table may not be effective for every type of pavement and depends on project-specific conditions.

Abbreviations and Terms

ABBREVIATIONS

AADT	Average Annual Daily Traffic
AADTT	Average Annual Daily Truck Traffic
AASHTO	American Association of State and Highway Transportation Officials
AC	Asphalt Concrete
ADT	Average Daily Traffic
ASTM	American Society of Testing and Materials
ATPB	Asphalt Treated Permeable Base
AVC	Automated Vehicle Classification
CAM	Cement-Aggregate Mixture
CBR	California Bearing Ratio
CPR	Concrete Pavement Restoration
CRCP	Continuously Reinforced Concrete Pavement
CTB	Cement Treated Base
CTE	Coefficient of Thermal Expansion
DCP	Dynamic Cone Penetrometer
DE	Differential Energy
DI	Damage Index
DSR	Dynamic Shear Rheometer
EICM	Enhanced Integrated Climatic Model
ESAL	Equivalent Single Axle-Load
FD	Fatigue Damage
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
HMA	Hot Mix Asphalt
IDT	Indirect Tensile
IRI	International Roughness Index
JPCP	Jointed Plain Concrete Pavement
JRCP	Jointed Reinforced Concrete Pavement
LCB	Lean Concrete Base
LCCA	Life-Cycle Cost Analysis

LTE	Load Transfer Efficiency
LTPP	Long-Term Pavement Performance
M-E	Mechanistic-Empirical
MEPDG	Mechanistic-Empirical Pavement Design Guide
NAPA	National Asphalt Pavement Association
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NDT	Non-Destructive Deflection Testing
NHI	National Highway Institute
P	Probability
PCA	Portland Cement Association
PCC	Portland Cement Concrete
PMS	Pavement Management Systems
QA	Quality Assurance
QC	Quality Control
R	Reliability
RAP	Recycled Asphalt Pavement
RC	Reflection Cracking
RMS	Root Mean Squared
SAMI	Stress-Absorbing Membrane Interlayer
SEE	Standard Error of the Estimate
SHRP	Strategic Highway Research Program
SMA	Stone Matrix Asphalt
SWCC	Soil Water Characteristic Curve
TC	Thermal Cracking
TTC	Truck Traffic Classification
VFA	Voids Filled with Asphalt
VMA	Voids In Mineral Aggregate
WIM	Weighing-In-Motion
WMA	Warm Mix Asphalt

TERMS

a	Radius of a loaded area
A_{PO}	Calibration coefficient for the CRCP punchout model
AC_{PCC}	PCC air content
Age	Pavement age
AGG	Aggregate interlock stiffness factor for the JPCP faulting model
B_{curl}	Bradbury's curling/warping stress coefficient
c_1	First bond stress coefficient
c_2	Second bond stress coefficient
cw	Average crack width at the depth of the steel for the CRCP model
C	Global calibration constants for the transfer functions; numbered subscripts refer to the specific parameter or constant
C_0	Current transverse crack length for the thermal cracking model
C_d	Crack depth of a transverse crack in the thermal cracking model
C_H	Thickness correction term for fatigue cracking in HMA mixtures
CA_m	Total cracking area in month m, used in the reflection cracking model
CRACK	Predicted transverse cracking based on mean inputs
CRK	Predicted amount of bottom-up or top-down cracking in the JPCP cracking model; subscripts refer to where crack initiates
d	Dowel diameter
d_b	Reinforcing steel bar diameter
D	Depth below the pavement surface
D_{steel}	Depth to steel layer
DAM_{dow}	Damage at dowel-concrete surface
dsp	Dowel spacing
DE	Differential density of energy of subgrade deformation accumulated in a particular month
DI	Damage index; subscripts define whether it is bottom-up or top-down cracking and specific layer accumulating damage
E	Elastic modulus of bound paving material; subscripts refer to specific layer or material
$E_{CTB}^{D_{(t)}}$	Equivalent damaged elastic modulus at time *t* for the *CTB* layer
E_{CTB}^{Min}	Equivalent elastic modulus for total destruction of the *CTB* layer
E_{CTB}^{Max}	28-day elastic modulus of the intact *CTB* layer
E_{HMA}, E^*	Dynamic modulus of hot mix asphalt mixtures
EROD	Base/subbase erodibility factor for PCC pavements
f	Base friction coefficient
f'_c	PCC compressive strength
f_t	PCC indirect tensile strength
$FAULT_m$	Mean joint faulting at end of month m
$Fault_t$	Predicted mean transverse joint faulting at any give time *t*
FAULTMAX	Maximum mean transverse joint faulting for a month
$FAULTMAX_0$	Initial maximum mean transverse joint faulting
FC	Area of fatigue cracking in HMA mixtures; subscripts define whether it is bottom-up, top-down cracking or total

FC_{CTB}	Area of fatigue cracking of the CTB layer
FD	Fatigue damage
FI	Freezing Index
FR	Base freezing index defined as percentage of time the top base temperature is below freezing temperature for the JPCP faulting model
FT_{cycles}	Average annual number of freeze-thaw cycles
h	Thickness of the incremental or sublayer; subscripts refer to specific material or layer
H	Total thickness of the pavement layer; subscript refers to the individual layer
H_{eff}	Effective HMA overlay thickness for the reflection cracking regression model
IRI_o and IRI_I	Initial IRI, after construction
jw	Joint opening
J_{AGG}	Joint stiffness on the transverse crack computed for the time increment
J_c	Joint stiffness on the transverse crack for current time increment
J_d	Non-dimensional dowel stiffness at the time of load application
k	Coefficient of subgrade reaction
$k_{1r,2r,3r}$	Global field calibration coefficients for the rut depth prediction model
$k_{c1,c2}$	Global field calibration coefficients for fatigue cracking model of CTB
$k_{f1,f2,f3}$	Global field calibration coefficients for the fatigue cracking prediction model
k_{s1}	Global calibration coefficient for unbound materials and soils
k_t	Global calibration coefficients for the thermal cracking model for HMA
k_z	Depth confinement factor
K	Stress intensity factor
l	Radius of relative stiffness for a dowel
L	Mean transverse crack spacing, calculated and mean crack spacing based on design crack distribution
LTE_{agg}	Joint-load transfer efficiency if the aggregate interlock is the only mechanism for load transfer
LTE_{base}	Joint-load transfer efficiency if the base is the only mechanism for load transfer
LTE_{dowel}	Joint-load transfer efficiency if dowels are the only mechanism for load transfer
LTE_{joint}	Total transverse joint-load transfer efficiency
LTE_{TOT}	Total crack-load transfer efficiency due to aggregate interlock, steel reinforcement, and base support
m	Slope derived from the indirect tensile creep compliance curve measured in the laboratory, or month within the analysis period
M_r	Resilient modulus
M_R	Modulus of rupture of PCC and chemically stabilized materials
n	Number of axle load applications
N	Allowable number of axle load applications (subscripts refer to the distress type and layer), or number of data points used in a regression
P	Probability
P_s	Overburden on the subgrade or foundation
P_{steel}	Percent longitudinal steel
P_{200}	Percent material passing the #200 sieve
PI	Plasticity Index
PO	Total number of medium- and high-severity punchouts per mile
Precip	Average annual precipitation or rainfall
PREFORM	1 if preformed sealant is present; 0 if not

r_d	Residual dowel-action factor to account for residual load transfer provided by the steel reinforcement
R	Reliability
RC	Percent of cracks reflected
RD	Rut depth
SCF	Scaling factor based on site-, design-, and climate-related for the regression equation to predict spalling within the IRI equation for PCC
s_e, SEE	Standard error of the estimate; standard deviation of residual error
SF	Site factor for the IRI regression models
SPALL	Percentage of joints with spalling of a medium and high severity
t	Time
T	Temperature
TC	Length of thermal or transverse cracking
TCRACK	Total transverse cracking combining all types of cracks in the PCC cracking model for JPCP
TFAULT	Total joint faulting cumulated per mile
TRA_m	Total reflected cracking area for month m
U_m	Peak bond stress
V_a	Percent air voids in the HMA mixture
V_{be}	Effective asphalt content by volume
Var	Variance of a value; subscripts are the predicted distress value for that variance
VMA	Voids in Mineral Aggregate
VFA	Voids Filled with Asphalt
w	Joint opening in JPCP
W_c	Water content of the unbound layer and soil
WC_{PCC}	PCC water/cement ratio
WetDays	Average annual number of wet days, greater than 0.1-in. rainfall
α_{PCC}	PCC coefficient of thermal expansion
α_{PO}	Calibration constant for the CRCP punchout prediction model
$\beta_{1r,2r,3r}$	Local or mixture calibration constants for the rut depth prediction model of HMA
$\beta_{c1,c2}$	Local calibration constants for the fatigue cracking model of CTB
$\beta_{f1,f2,f3}$	Local or mixture calibration constants for the fatigue cracking model of HMA
β_{PO}	Calibration constant for the CRCP punchout prediction model, –0.52316
β_{s1}	Local calibration constant for the rut depth model of the unbound layers
β_t	Local calibration constant for the thermal cracking model of HMA
β_{t1}	Regression coefficient determined through global calibration thermal cracking model of HMA
Δ, Δp	Accumulated permanent or plastic deformation in the pavement layers and foundation; subscripts refer to the individual layers
ΔC	Change in the crack depth due to a cooling cycle
ΔCA	Increment of fatigue cracking area
ΔDI	Incremental damage index; subscripts define whether it is bottom-up or top-down cracking and specific layer accumulating damage
$\Delta Fault$	Incremental change in mean transverse joint faulting for a specific month
ΔK	Change in the stress intensity factor due to a cooling cycle
Δs	Incremental loss of shear capacity of the load transfer at the joint due to repeated wheel load application

ΔT_m	Effective temperature differential for month *m*
$\Delta T_{t,m}$	Mean PCC top-surface nighttime temperature; from 8:00 p.m. to 8:00 a.m. for month *m*
$\Delta T_{b,m}$	Mean PCC bottom-surface nighttime temperature; from 8:00 p.m. to 8:00 a.m. for month *m*
$\Delta T_{sh,m}$	Equivalent temperature differential due to reversible shrinkage for month *m* for old concrete (shrinkage fully developed)
ΔT_{PCW}	Equivalent temperature differential due to permanent curl/warp
$\Delta T\xi$	Drop in PCC temperature from the concrete "zero-stress" temperature at the depth of the steel for construction month
$\delta_{curling}$	Maximum mean monthly slab corner upward deflection PCC due to temperature curling and moisture warping
δ_L	Deflection at the corner of the loaded slab
δ_U	Deflection at the corner of the unloaded slab
ε_o	Intercept determined from laboratory repeated load permanent deformation tests
ε_p	Accumulated permanent or plastic strain in the pavement layers; subscripts refer to the individual layers
ε_r	Resilient or elastic strain; subscripts refer to the individual layers
ε_{shr}	Unrestrained concrete drying shrinkage at steel depth
ε_t	Tensile strain in the HMA layer at critical locations
ε_v	Vertical resilient or elastic strain in the layer/sublayer
ξ_d	Dowel stiffness factor
σ, σ_t	Tensile stress at the bottom of the bound paving layer; subscript refers to the specific layer or condition (month, load, axle type, etc.)
σ_0	Westergaard's nominal stress factor based on PCC modulus
σ_d	Standard deviation of the log of the depth of cracks in the pavement for thermal cracking model
σ_{env}	Tensile stress in the PCC due to environmental curling
σ_{Long}	Maximum longitudinal tensile stress in PCC at steel level
σ_m	HMA-mixture tensile strength
σ_{tip}	Far-field stress from pavement response model at depth of crack tip
τ_j	Shear stress on the transverse crack
τ_{ref}	Reference shear stress derived from the PCA test results

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