

**FLEXIBLE PAVEMENT DESIGN –  
STATE OF THE PRACTICE**

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## **1. INTRODUCTION**

After decades of development, mechanistic-empirical (M-E) pavement design is finally transitioning from research into use by practicing engineers. For years, the paving industry has needed a structural design system that can rationally, and realistically, account for new and innovative materials, advanced construction methods, use of recycled materials, various climatic conditions and changes in trucking technology. The American Association of State Highway and Transportation Officials (AASHTO) Pavement Design Guides, from 1962 through 1993 (1), simply do not have the capability to easily update and adapt to the significant improvements made in pavement engineering, design, materials and construction. Based primarily on the original American Association of State Highway Officials (AASHO) Road Test (2), the empirical design equations were strictly limited to the conditions during the Road Test from 1958-1960. Therefore, most pavement designs conducted today using the 1993 AASHTO Design Guide are extrapolations, which can often lead to significant overdesign. While overdesign may be conservative, optimized pavement structures for better use of resources should be an industry goal.

M-E design, which predicts specific modes of distress resulting from modeling pavement responses to loadings under various conditions, alleviates many of the shortcomings of the existing empirical design systems. New technologies and changes in conditions can be modeled in an M-E framework to provide optimized pavement design solutions. A subset of M-E design is the perpetual pavement concept. Using the same tools and techniques within M-E design, but accounting for limiting strain levels inherent to all materials, flexible pavements can be designed such that deep structural distresses (i.e., bottom-up fatigue cracking and substructure rutting) do not develop. Perpetual pavement design can also provide an approach for arriving at a maximum pavement thickness under a given set of conditions. The limiting thickness concept is critical to building high-performance and long-lasting, yet economical, pavement structures.

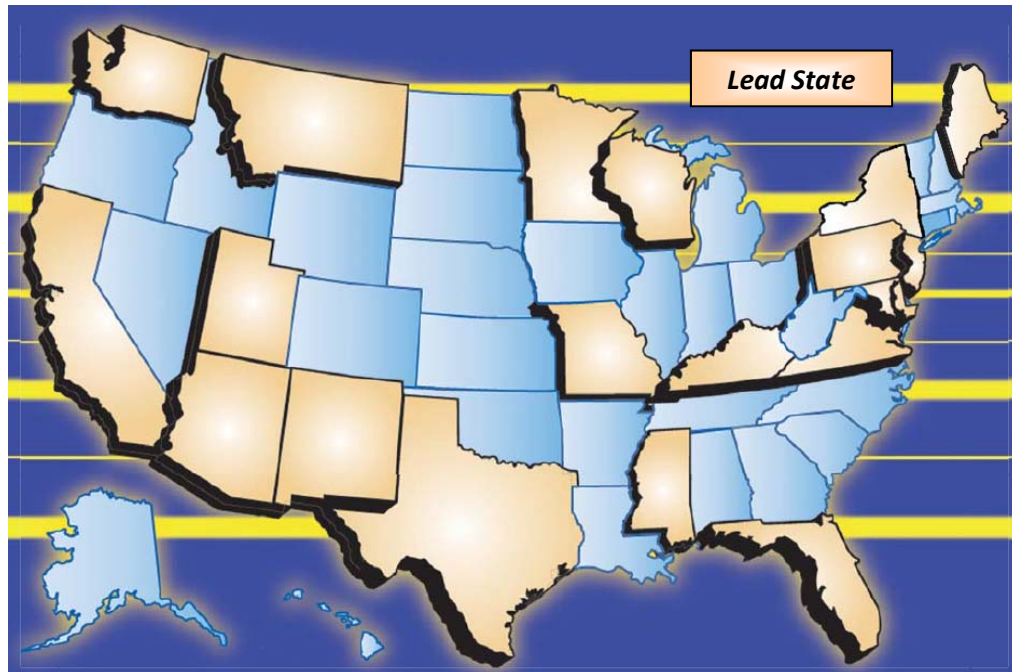
While many view M-E and perpetual pavement design as dramatic improvements over existing methodologies, they have a host of challenges that must be addressed for effective use in practice. Specifically, M-E design requires validation and local calibration. The empirical part of M-E design typically requires that local calibration take place before implementation to ensure accurate prediction of pavement distresses over time. Failure to locally calibrate can lead to unrealistic predictions of pavement performance. The challenge within perpetual pavement design is to quantify the limiting strain levels for specific materials to achieve optimized pavement cross-sections and determine a maximum thickness level.

Recognizing the need for an improved pavement design system, the National Cooperative Highway Research Program (NCHRP) sponsored Project 1-26 entitled "Calibrated Mechanistic Structural Analysis Procedure for Pavements." The final reports for the project (3-6) were published in the early 1990's and established mechanistic-empirical design as a viable option for pavement engineers, though 1-26 did not formally establish the new AASHTO pavement design procedure. The next project, 1-37A, entitled "Development of

the 2002 Guide for the Design of New and Rehabilitated Pavement Structures: Phase II” began in 1998. The project ran through 2004 and resulted in the Mechanistic Empirical Pavement Design Guide (MEPDG). “MEPDG” was used to refer to both the software and the accompanying documentation. The software was freely available, though considered a beta-test version. Further improvements to the MEPDG were made as part of NCHRP 1-40D, “Technical Assistance to NCHRP and NCHRP Project 1-40A: Versions 0.9 and 1.0 of the M-E Pavement Design Software.” This project ran from 2005 through 2009 and the MEPDG software continued beta testing by agencies, universities and practitioners. Also during this time, the MEPDG Manual of Practice was voted on by the AASHTO Joint Technical Committee on Pavements and was approved with 81% of the vote in 2007. Subsequent votes by the AASHTO Subcommittees on Design and Materials, respectively, also were in favor of the new procedure. In 2008, the MEPDG was transitioned to the AASHTOWare series of programs and was renamed DARWin-ME as the program developers continued to improve the program’s capabilities. In 2013, the software became commercially available under the name AASHTOWare™ Pavement ME Design (hereafter called ME Design software). The software and accompanying documentation (7) represent a tremendous leap forward from the 1993 Design Guide (1) and DARWin software.

As AASHTO is no longer supporting the DARWin software and 1993 Design Guide, many agencies are grappling with how best to make the transition from their existing methodology into M-E or perpetual pavement design. This challenge is magnified by the fact that there is a wide range of agency-specific design methods currently in use across the U.S. Therefore, there is no single set of steps for every agency to use for implementation. Recognizing the need for implementation guidance, the FHWA formed a Design Guide Implementation Team (DGIT). Central to DGIT is a group of lead states (Figure 1.1) with the following attributes (8):

- One of the first states to pursue implementation of the design guide and obtain upper management support.
- A champion for implementation.
- Knows the political, funding, and internal hurdles that need to be addressed.
- Able to compare pavement design/analysis technologies to determine which is most advantageous for a given project.
- Able to focus on advanced technologies and/or refinements.
- Advertises both successes and failures.
- Shares funding success stories.
- Develops short and long-term plans for implementation.
- Becomes an expert in the implementation process.



**Figure 1.1 M-E Design Implementation Lead States (9).**

While the states shown in Figure 1.1 are visibly leading efforts toward implementation, with much documentation on their activities, many other states are working toward the same goal without serving as a lead state. Indiana, for example, was an early implementer of the design software when it was called the MEPDG and currently uses it as the primary design tool as specified in Chapter 52 of the Indiana Design Manual (10). Other states may have longer-term plans for implementation while some may opt to continue with the empirical AASHTO design system with appropriate updates made to account for new materials and conditions. In any case, there are lessons to learn and document from every agency with respect to the current state of the practice and plans for agency implementation in the future.

## **2. PAVEMENT DESIGN METHODS**

Prior to the 1950's, pavement thickness design in the U.S. was based primarily upon experience. Relatively low traffic levels did not warrant highly engineered structures at that time as most freight was carried by rail or barge. With the rise of the U.S. interstate system in the mid-1950's, there was a recognized need for better-engineered pavement structures and a uniform set of design standards. The AASHTO Road Test (2) met this need and provided the basis for the empirical AASHTO Design Guides that would be used from the 1960's until today. The 1980's saw interest grow in developing M-E methods to address the shortcomings of purely empirical methodologies, but it wasn't until National Cooperative Highway Research Program (NCHRP) Project 1-37A in the 1990's that AASHTO began to seriously consider, and work toward, adopting an M-E approach to pavement thickness design. The turn of the century brought beta versions of the M-E design software and corresponding documentation called the Mechanistic-Empirical Pavement Design Guide (MEPDG) that were tested and evaluated by researchers and practitioners. As of 2013,



AASHTO fully adopted the MEPDG documentation and commercially released the AASHTOWare™ Pavement ME Design software. In the early 2000's, the National Asphalt Pavement Association (NAPA), in conjunction with the Asphalt Pavement Alliance (APA), began endorsing use of long-life, or perpetual pavement concepts, as a viable flexible pavement design alternative. The three design approaches (empirical, mechanistic-empirical and perpetual) have unique attributes that will be discussed in the following subsections.

## 2.1 Empirical Pavement Design

Empirical thickness design is based primarily upon observations from road tests. The most famous test was the AASHO Road Test conducted from 1958-1960 in Ottawa, Illinois (2). This road test formed the basis of the AASHTO Pavement Design Guides published from 1962 through 1993 (1). The design inputs, procedure and limitations are discussed below.

### 2.1.1 AASHTO Empirical Design Inputs

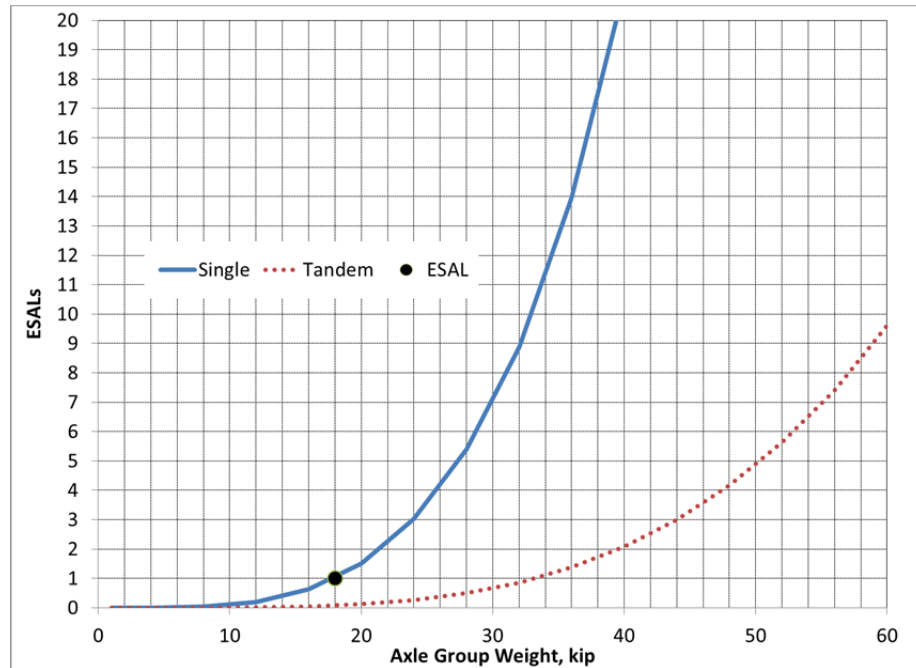
Observations from the AASHO Road Test established correlations between the following main factors for flexible pavements:

- Soil condition as quantified by the resilient modulus ( $M_r$ )
- Traffic as quantified by equivalent single axle loads (ESALs)
- Change in pavement condition as quantified by a change in pavement serviceability index ( $\Delta PSI$ )
- Pavement structure as quantified by a structural number (SN)

The soil resilient modulus describes the inherent ability of the soil to carry load and can be measured in the laboratory through triaxial resilient modulus testing or in the field through falling weight deflectometer (FWD) testing. Generally, lower  $M_r$  values will require more pavement thickness to carry the given traffic. The soil modulus during the AASHO road test was approximately 3,000 psi. Care should be taken when using the AASHTO empirical method to ensure  $M_r$  values obtained through modern means are adjusted to reflect test conditions. For example, AASHTO recommends dividing the soil modulus obtained through FWD testing by three before using in the empirical design equation (1). It is also important to emphasize that there was only one soil type used during the AASHO Road Test. Though there were seasonal fluctuations in the soil modulus from which empirical correlations between soil modulus and pavement condition were developed, the analysis was strictly limited to that soil type.

The AASHO Road Test featured various test loops that were constructed of similar pavement thicknesses but trafficked with different axle types and load levels (2). The researchers noted an approximate fourth-power relationship between the amount of pavement damage and the load level applied to the pavement section. This relationship was the central idea in the equivalent single axle load (ESAL), which was selected to be an 18,000-lb single axle with dual tires. AASHTO developed empirical equations to relate the number of applications of all other axle types (single, tandem and tridem) and load magnitudes to that of the ESAL. Figure 2.1 illustrates ESAL values for single and tandem

axles over a range of axle weights. The single and tandem curves clearly show the fourth-order nature of ESALs versus axle weight. The benefit of spreading the load over more axles is evident in Figure 2.1 by the dramatic reduction in ESALs for the tandem axle group at any given axle weight, relative to the single axles. Finally, the ESAL standard is shown in the plot at 18 kip with an ESAL value of one. Within the AASHTO empirical design system, total traffic must be decomposed into vehicle types with known axle weight distributions. The axle weight distributions are then used with the ESAL equations to determine ESALs per vehicle from which a total design ESAL over the pavement life is computed.



**Figure 2.1 ESALs versus Axle Weight.**

During the AASHTO Road Test, a panel of raters made routine inspections of each section. Figure 2.2 shows the rating form and zero to five scale the raters used to quantify current pavement condition. This rating scale was the only performance parameter used in the thickness design procedure. The researchers compiled the average ratings and plotted them against the amount of applied traffic in each section to develop performance history curves as shown schematically in Figure 2.3. The AASHTO design procedure relies upon characterizing the change in serviceability ( $\Delta PSI$ ) from the start ( $p_o$ ) to the end ( $p_t$ ) of the design life as a function of applied ESALs. Typical  $\Delta PSI$  design values range from 2 to 3 as a function of roadway classification (1). For example, a high volume interstate would be designed with a smaller  $\Delta PSI$  compared to a low volume county road.

**Acceptable ?**

Yes

No

Undecided

5  
4 Very Good  
3 Good  
2 Fair  
1 Poor  
0 Very Poor

Section Identification \_\_\_\_\_ **Rating**

Rater \_\_\_\_\_ Date \_\_\_\_\_ Time \_\_\_\_\_ Vehicle \_\_\_\_\_

Figure 2.2 AASHTO Road Test Present Serviceability Rating Form (2).

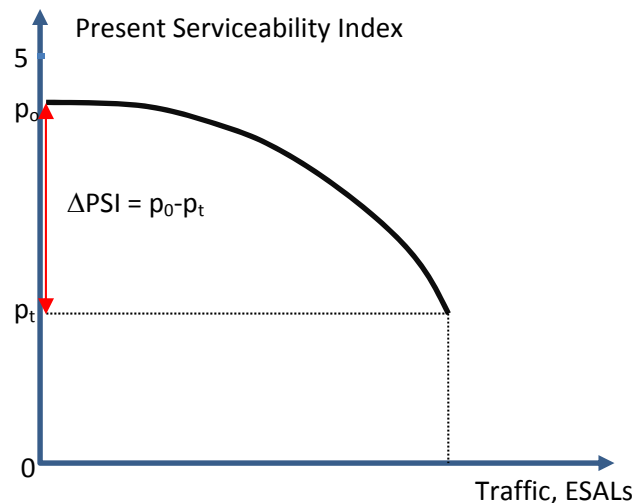
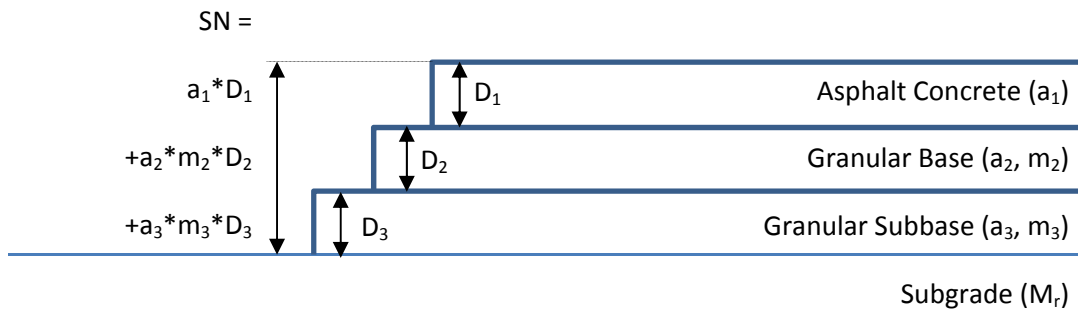


Figure 2.3 Pavement Performance History.

Since flexible pavements are typically comprised of diverse layers with varying engineering properties, it was necessary for AASHTO to introduce the pavement structural number (SN) concept. SN represents the cumulative pavement structure above the subgrade expressed as a product of individual layer thicknesses ( $D_i$ ), their respective structural coefficients ( $a_i$ ) and drainage coefficients ( $m_i$ ) as illustrated in Figure 2.4. The layer thicknesses are obtained from the AASHTO design process as described below. The structural coefficients are empirical values meant to relate the relative load-carrying capacity of different materials. For example, many state agencies use 0.44 for asphalt and 0.14 for granular base as originally recommended by AASHTO (2). These particular structural coefficients mean that one inch of asphalt is roughly equivalent to 3.1 inches ( $0.44 \div 0.14$ ) of aggregate base. The drainage coefficients are meant to empirically adjust the design according to site-specific

rainfall expectations and quality of drainage provided by the material itself (1). Drainage coefficients range from 0.4 to 1.4 with the original AASHTO Road Test condition represented as 1.0.



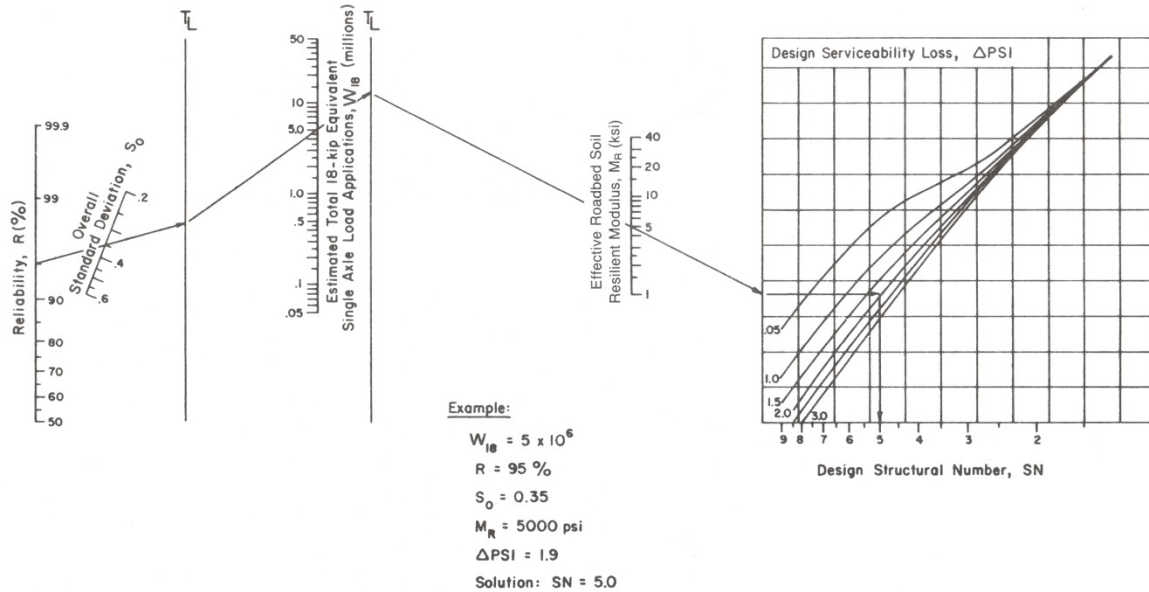
**Figure 2.4 Structural Number Concept.**

### 2.1.2 AASHTO Empirical Design Procedure

As described above, the AASHTO Road Test (2) established a correlation between soil condition, traffic, change in pavement condition and pavement structure. This relationship is shown in Equation 1 (1). The  $M_r$ ,  $\Delta PSI$  and SN terms are as defined above. ESALs are represented by the  $W_{18}$  term. The  $Z_R$  and  $S_0$  terms are reliability and variability factors not originally part of the AASHTO design procedure but added later to describe the ability of the pavement to function under the design conditions, essentially acting as factors of safety. The other quantities in the equation are regression coefficients that provided the best match between the independent variables (SN,  $\Delta PSI$ ,  $M_r$ ) and the performance of the pavement section as quantified by ESALs.

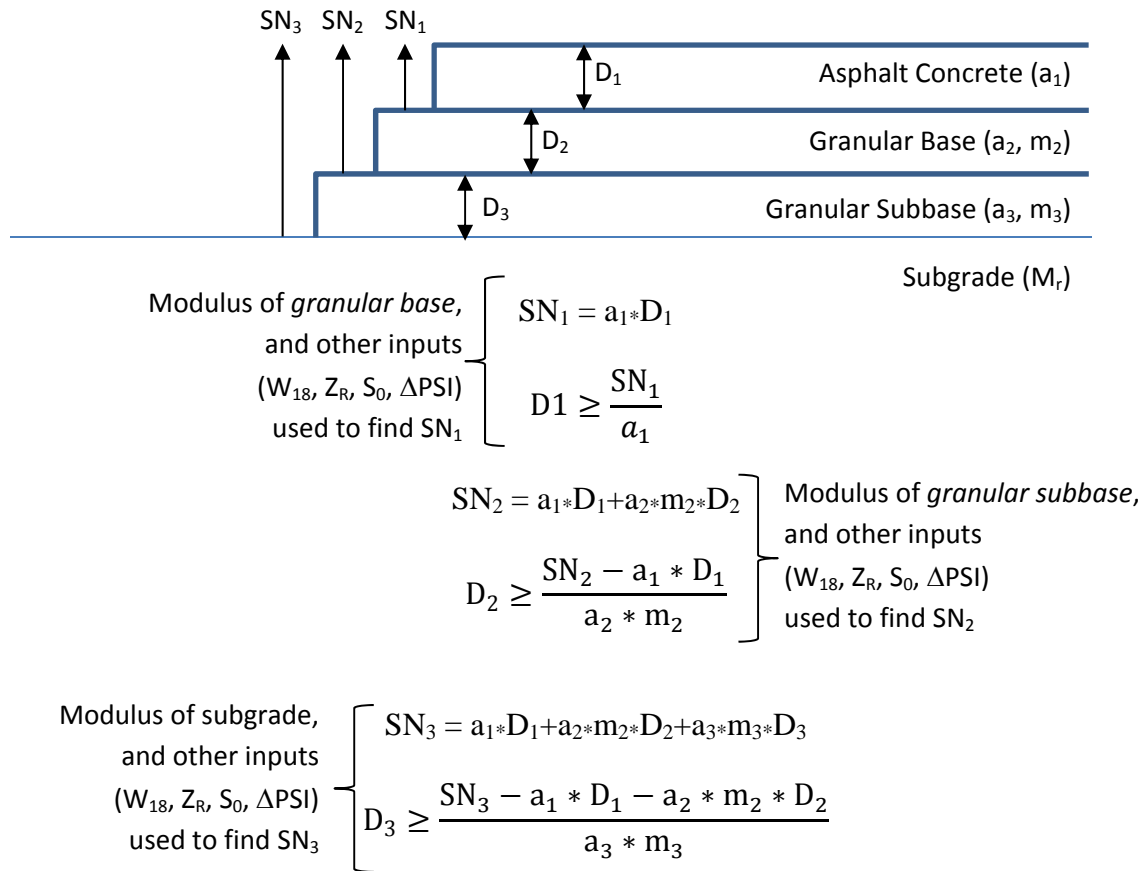
$$\log W_{18} = Z_R S_0 + 9.36 \log(SN + 1) - 0.20 + \frac{\log \left[ \frac{\Delta PSI}{4.2 - 1.5} \right]}{0.4 + \frac{1094}{(SN + 1)^{5.19}}} + 2.32 \log M_R - 8.07 \quad (1)$$

While the purpose of Equation 1 is to determine the required structural number of a proposed pavement section, it is written to compute ESALs ( $W_{18}$ ), and solving algebraically for SN is a daunting task. To alleviate this problem, AASHTO published a design nomograph (Figure 2.5) that solves for SN given the other inputs. Notice that  $W_{18}$  (ESALs) is treated as another input with the nomograph solving toward SN. Alternatively, solver subroutines in spreadsheet programs or customized software can also solve the equation for SN.



**Figure 2.5 AASHTO Flexible Pavement Design Nomograph (1).**

The AASHTO design equation (Equation 1 or Figure 2.5) is to be used successively for each layer in a multilayer pavement structure to determine the required pavement thicknesses. As described by AASHTO (1), this operation is performed in a top-down fashion as depicted in Figure 2.6. The design begins by finding the required structural number above the granular base ( $\text{SN}_1$ ) using the granular base modulus and other input parameters in the design equation or nomograph. By definition, this structural number is the product of the structural coefficient and thickness of layer one, and is used to solve for the thickness of the first layer. This procedure is followed for each of the subsequent layers, as shown in Figure 2.6, to arrive at a unique set of pavement layer thicknesses.



**Figure 2.6 Pavement Design with Empirical AASHTO Design Equation.**

### 2.1.3 AASHTO Empirical Design Limitations

Though the empirical AASHTO design procedure has been used since the 1960's, there are many factors that limit its continued use and provide motivation for developing and implementing more modern methods. Most notably among these factors is the very nature of the method itself: *empirical*. This term means that the design equations described above are strictly limited to the conditions of the original road test. This includes all the coefficients in Equation 1, the structural coefficients ( $a_i$ ), drainage coefficients ( $m_i$ ), ESAL equations and so forth. Any deviation from these conditions results in an unknown extrapolation.

The limitations of the AASHTO Road Test are numerous. The experiment had one soil type, one climate, one type of asphalt mix (pre-Marshall mix design), limited pavement cross-sections (maximum asphalt thickness was 6 inches), limited load applications and tires inflated to 70 psi (2). Any deviation from these factors in modern design means extrapolation, which can lead to under or over-design. Most designs conducted today are extrapolations beyond the original experimental conditions. Consider, for example, the thickness design curves published in 1962 as part of the AASHTO Road Test report shown in Figure 2.7. The shaded gray area above 1.1 million axle loads is entirely extrapolated. Also,

the dashed portions of the curves are extrapolations. Though one could compute ESAL equivalencies for each curve in Figure 2.7, the 18 kip single axle curve represents ESALs and is entirely extrapolated above 1.1 million ESALs. As evidenced by Figure 2.7, there was very little that was not an extrapolation, even in 1962. These same arguments apply to the other factors listed above and provide great motivation for developing more modern design systems that can adapt to changing conditions.

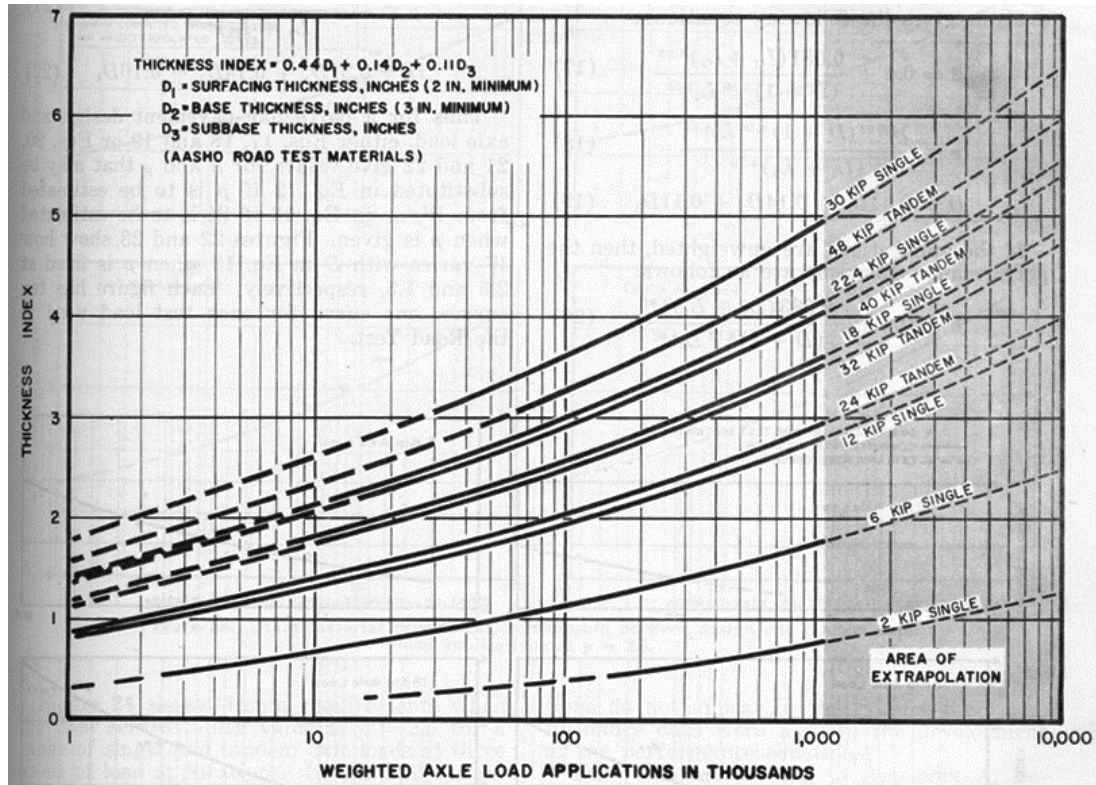


Figure 2.7 Flexible Pavement Design Curves (2).

## 2.2 Mechanistic-Empirical Pavement Design

Researchers and practitioners have long recognized the limitations of empirical design, but M-E design did not become a viable option for pavement design on a routine basis until personal computers were commonly available in the 1990s. The main advantage of M-E design is its ability to adapt to changing conditions. When new materials, loads or other conditions are introduced, they may simply be simulated to determine the effect on pavement thickness with more certainty than through empirical design.

Figure 2.8 illustrates the M-E design process, which can be considered as four major components that will be discussed below:

- Mechanistic modeling
- Empirical performance prediction
- Damage accumulation
- Design assessment

### 2.2.1 Mechanistic Modeling

Unlike the empirical approach described above, M-E design *begins* with a trial pavement section that will be evaluated under a given set of conditions. Figure 2.8 shows that the layer thicknesses ( $D_1$ ,  $D_2$ ) have been selected for the two layer pavement. The layer properties are characterized by their modulus ( $E_1$ ,  $E_2$ ,  $E_3$ ) and Poisson ratios ( $\mu_1$ ,  $\mu_2$ ,  $\mu_3$ ). These properties enable pavement responses throughout the structure to be calculated using a layered elastic computer model and are primary inputs for most M-E methods. While there is some correspondence between these mechanistic material properties and the AASHTO empirical structural coefficients discussed previously, modulus and Poisson ratio are fundamental properties that characterize the mechanistic response of the material to an applied load. The structural coefficients are simply empirical regression constants that have no true engineering meaning.

A single tire load with weight ( $P$ ) and contact pressure ( $q$ ) has been applied to the pavement surface in Figure 2.8. M-E design allows the analyst to simulate many types of load configurations and load magnitudes rather than computing empirical ESALs. The range of axle types and load levels for a given design are known as the load spectra. Each load simply must be defined in terms of its location relative to other tires, load magnitude and inflation pressure. Characterizing loads in this way is advantageous in that the design is not limited to a pre-defined set of empirical equations that relate non-standard loadings to that of a standard. It also allows for unique loading conditions that may be found on aircraft or specialized transport vehicles. The ESAL equations are strictly limited to single, tandem and tridem axles over a prescribed range of weights.

Critical locations have been identified in the structure at the bottom of the asphalt layer (A) and top of subgrade (B) in Figure 2.8. Historically, tensile strain at A has been empirically linked to bottom up fatigue cracking, while the compressive strain at B is often used to predict total pavement rutting. It is important to note, however, that other critical locations may be used for M-E design. For example, the ME Design software uses vertical strain predictions throughout the pavement depth to predict rutting in each particular layer. Predicting specific distress modes is a major advantage of M-E design. Rather than estimating a change in pavement serviceability ( $\Delta$ PSI) with no information regarding the type of distress, M-E design allows designers to identify particular distresses and design accordingly. For example, if surface rutting is predicted, the solution may be to change the binder grade or other mix-specific properties rather than just increasing thickness.

Another important consideration is that the pavement depicted in Figure 2.8 represents one particular condition (i.e., pavement layer properties and loading). In the field, however, pavements are under a constant state of change as the temperatures and moisture conditions fluctuate on a daily and seasonal basis in addition to the wide variety of applied loads. For this discussion, a particular set of material properties and applied load will be called condition  $i$ , and it is critical that the design consider the complete range of expected conditions as shown by the iterative loop in Figure 2.8.



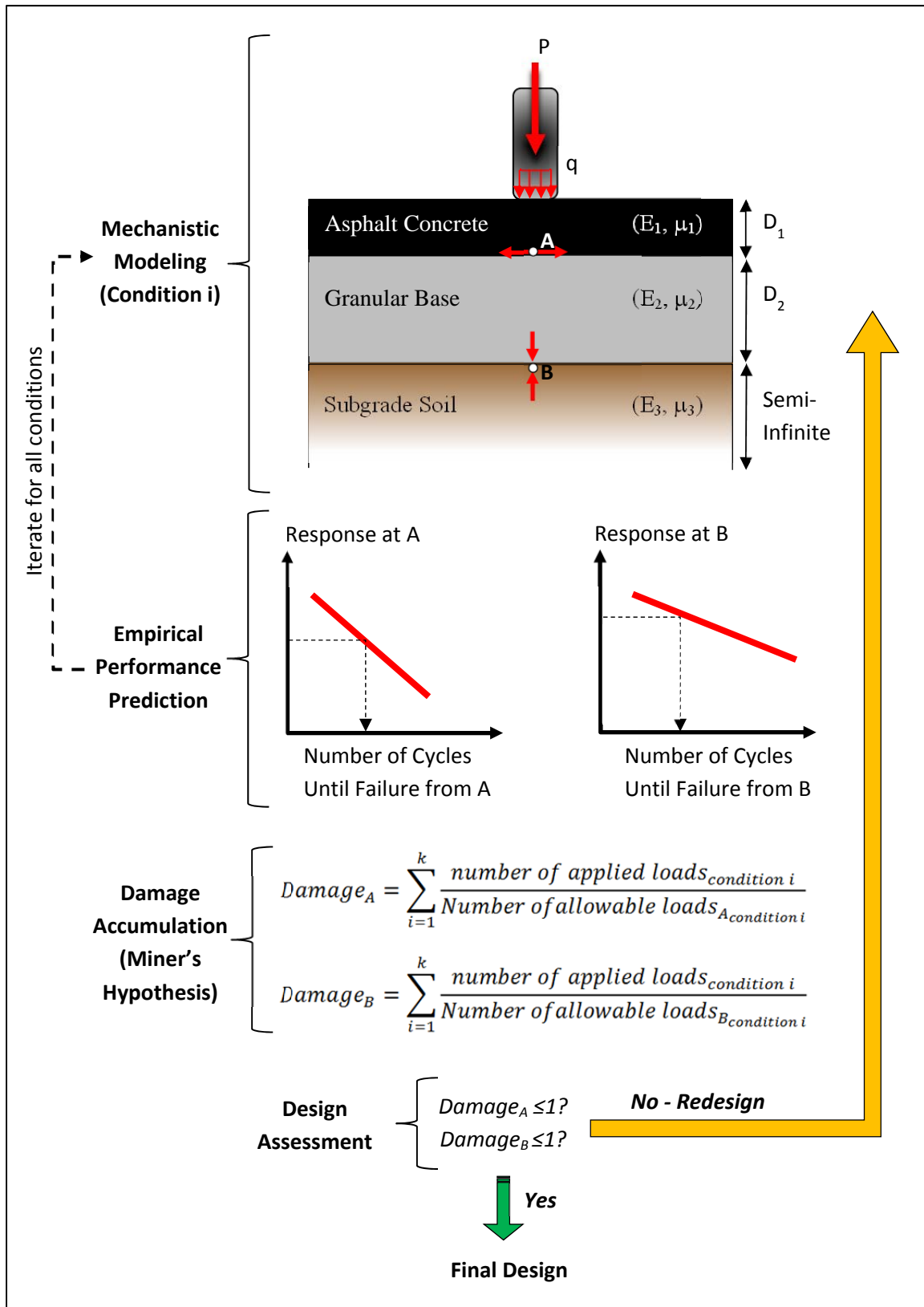


Figure 2.8 M-E Design Framework.

### 2.2.2 Empirical Performance Prediction

For each pavement condition, a mechanistic simulation is conducted to determine the pavement response at critical locations in the structure. The pavement responses are then used with empirical transfer functions to predict the number of allowable load repetitions until failure for each mode of distress. Figure 2.8 shows two different transfer functions for A and B, respectively. Transfer functions are the well-recognized weak link of M-E design because they require calibration to field performance. AASHTO strongly recommends local calibration of the transfer functions in the ME Design software prior to implementation and has published a calibration manual of practice (11). The calibration process requires an agency to compare historical pavement performance from existing pavements to performance predicted by the ME Design software and adjust the transfer function coefficients to minimize the error between measured and predicted performance.

### 2.2.3 Damage Accumulation

The transfer functions produce a predicted number of *allowable* load repetitions until a specific mode of failure occurs under a particular condition. This value is used with the *expected* number of load repetitions in that condition to determine an incremental damage ratio (D), as shown in Figure 2.8. Total damage for each distress is simply the sum of the respective damage increments. This approach is widely used in M-E pavement design and is known as Miner's Hypothesis (12).

### 2.2.4 Design Assessment

Assessing the design in terms of each distress mode is the final stage of the M-E analysis. Since damage is the ratio of applied to allowable loads, a perfectly optimized design is achieved when the ratio reaches 1.0. Damage exceeding 1.0 represents failure, while values much less than 1.0 may indicate overly conservative designs. In either of these cases, the designer should try a new set of thicknesses or change materials to achieve an optimal balance between materials and overall thickness. Therefore, unlike the AASHTO empirical method that results in a unique set of layer thicknesses given a set of inputs, M-E methods predict pavement performance based on a set of layer thicknesses. Therefore, there may be many acceptable cross-sections, depending on what the designer chooses as performance targets. Finally, a designer must consider the economic aspects of any structural design and will typically develop a number of viable options and choose the most cost-effective option.

## 2.3 MEPDG and AASHTOWare Pavement ME Design

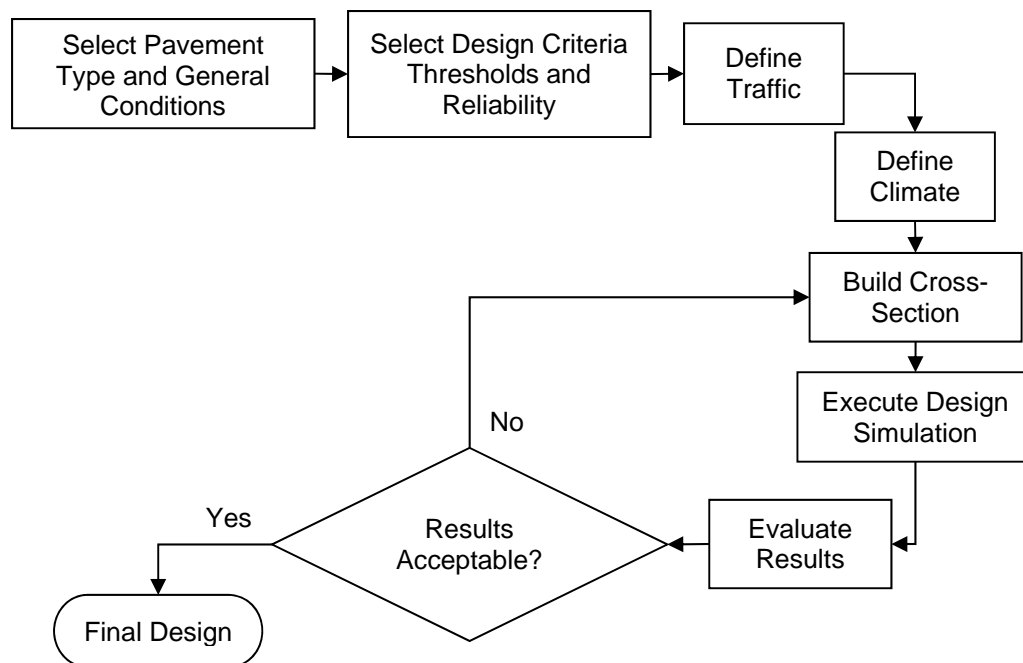
Many approaches that utilize the M-E concepts described in Section 2.2 have been developed and used for flexible pavement design. A good example is the Asphalt Institute MS-1 procedure (13) that utilized mechanistic modeling with transfer functions calibrated to the AASHTO Road Test. However, the newly-developed MEPDG (7) and ME Design software represent the current AASHTO standard for pavement design in the U.S. The following subsections describe the main features of this approach.

### 2.3.1 MEPDG Overview

The MEPDG and accompanying ME Design software were developed as part of NCHRP 1-37A (10). This extensive research program was tasked with developing an M-E process using existing technology to facilitate routine pavement design. The MEPDG and software were meant to be modular so that new features could be added, exchanged or improved as they became available. For example, though the software currently contains a top-down cracking model, it is merely a placeholder until NCHRP Project 1-52 (A Mechanistic-Empirical Model for Top-Down Cracking of Asphalt Pavement Layers) is completed, which will update the existing model with the best-available research. The project's expected completion date is March 2016.

Another important feature of the MEPDG approach was the establishment of three levels of design inputs (14). Level 3 represents the lowest accuracy and relies on default values built into the design software. Level 2 provides an intermediate level of accuracy to be used when resources or testing equipment are not available for tests required for Level 1, but some other limited testing, or correlations from other tests, has been conducted. Level 1 represents the highest accuracy and requires laboratory or field testing. This level is recommended for heavily trafficked pavements or designs requiring high reliability.

Though the ME Design software is incredibly complex, it operates under the general principles shown in Figure 2.8. Figure 2.9 shows more specifically how the ME Design software operates. Each block is described briefly in the following subsections.



**Figure 2.9 M-E Design Software Flowchart.**

### 2.3.2 *Select Pavement Type and General Conditions*

Pavement type selection is the first step of analysis. Designers select whether the project is new construction or rehabilitation and the type of pavement (flexible or rigid) to be analyzed. These decisions dictate to the program which models will be used to compute pavement responses and make performance predictions. The designer also establishes the construction timeline, which enables the program to estimate climate conditions during construction and after the project is open to traffic.

### 2.3.3 *Select Design Criteria Thresholds and Reliability*

Once the design and pavement type have been selected, designers must consider the various distresses that will be predicted and decide upon failure thresholds and accompanying levels of reliability for each distress. Currently, the ME Design software computes the following:

- International Roughness Index (IRI)
- Top-down cracking
- Bottom-up fatigue cracking
- Thermal cracking
- Total pavement rutting
- Rutting in the asphalt concrete layer
- Reflective cracking

The software has default threshold values and design reliabilities for each distress that agencies should carefully evaluate prior to implementation. Agencies should also carefully consider which distresses to use as a basis for design. As mentioned above, the top-down model is currently under further development and should be ignored until an improved model is developed in NCHRP 1-52. In any case, it is likely that agencies will need to decide upon their distresses, terminal levels and level of reliability as a policy matter. It is unlikely that designers will adjust these values on a design-by-design basis.

### 2.3.4 *Define Traffic*

Characterizing traffic within the ME Design software is highly detailed and requires these general categories of information:

- Traffic volume (annual, monthly and hourly), traffic growth and operational speed
- Traffic composition
  - Vehicle class distribution
- Axle configurations
  - Axle types, axle spacing, tire spacing and tire inflation pressure
- Axle weight distributions
  - May be adjusted on a monthly basis
- Lateral wander of vehicles

These factors are collectively known as the load spectra for a given design and enable the software to carefully account for all of the expected loading conditions on the pavement.

While the inputs are highly detailed, agencies will likely develop state-approved values (Level 2) to populate each of the input categories or simply rely on the defaults (Level 3). In some cases, such as high-volume/high-cost projects, site-specific (Level 1) traffic data will be warranted. Regardless of project scale, site-specific traffic volume information (average annual daily truck traffic (AADTT)) should be developed for each design. This task is already current practice within most agency design procedures.

### 2.3.5 *Define Climate*

The ME Design software makes use of the enhanced integrated climatic model (EICM) to forecast future pavement temperatures and moisture contents as a function of historical weather records. These predictions are critical to the models used in the software to make adjustments to the asphalt concrete modulus as a function of temperature and the unbound materials as a function of moisture content. These factors are also used in some performance models to predict levels of distress over time. While the EICM is incredibly complex from a computational standpoint, the user is required to provide relatively little information. The software comes preloaded with historical weather data files for locations across the U.S. The designer simply selects a nearby station, or selects several to let the software interpolate to a specific location. The designer also must enter the project elevation and depth to the water table. No other information is required for the software to generate a climate file for the project. It is expected that many agencies will use the preloaded climate data when using the ME Design software, though others may choose to develop a more comprehensive library of climate data for their state.

### 2.3.6 *Build Cross-Section*

The designer must carefully consider the types of layers to use in the design. As depicted in Figure 2.8, the designer must select the layer thicknesses before running the simulation, then assess the outcome from the simulation and decide how to proceed. In this way, the ME Design software is really an iterative pavement analysis tool rather than a true design program.

Within the ME Design program, the designer adds layers and selects the thickness and material type for each layer. The program requires particular types of information depending upon the type of material selected. Additionally, for each material type, the designer can enter data at various levels. For example, stiffness of asphalt concrete can be input at three levels. To estimate the dynamic modulus of the mixture based on the predictive model in the software, the binder performance grade and mixture volumetric properties are required at Level 3. At Level 2, some measured properties such as the dynamic shear modulus of the binder or conventional binder test data are required. At Level 1, the laboratory-measured dynamic modulus of the mixture is required. Unbound materials can also be specified at three levels. They can be specified based on the typical resilient moduli corresponding to their type or classification (Level 3), estimated based on some testing data and correlations within the software (Level 2) or defined by entering resilient moduli from laboratory testing (Level 1).

In any case, since the program uses layered elastic analysis, a set of modulus and Poisson ratio values are generated. At Level 1, the user enters these measured values directly. At Level 2, the user enters some other test data and the software establishes the modulus and Poisson ratio values based on correlations. At Level 3, the user gives the program some basic information that is used with additional models to estimate a set of modulus values and Poisson ratios. Different projects will require different levels of precision. Agencies will need to establish policies and, potentially, material libraries to facilitate design on a day-to-day basis. This approach is very similar to many states' practice of established structural coefficients and soil moduli to use within the AASHTO empirical procedure.

### 2.3.7 *Execute Design Simulation*

After the pavement type, design criteria, traffic, climate and cross-section have been established, a simulation may be executed. The ME Design software executes a series of M-E computations, following the general flow of Figure 2.8, to predict damage accumulation over time for each type of distress. As the program runs, climate predictions are made to estimate layer properties during each increment of time. Loads are simulated for each increment, and pavement responses are computed at critical locations in the structure. These responses are used with a series of transfer functions to predict the number of allowable loads until failure during each increment of time. The corresponding applied volume of loadings during the same time period is used to compute incremental damage. The incremental damage is plotted against time to illustrate the expected performance against the predefined critical distress levels. Figure 2.10 shows an example of the simulation output where IRI, total rutting, bottom-up cracking and thermal cracking are plotted versus time. Each plot shows the failure level defined by the designer as a horizontal red line with the predicted performance shown at the 50<sup>th</sup> percentile reliability and at the user-specified level of reliability that accounts for variation in expected performance.

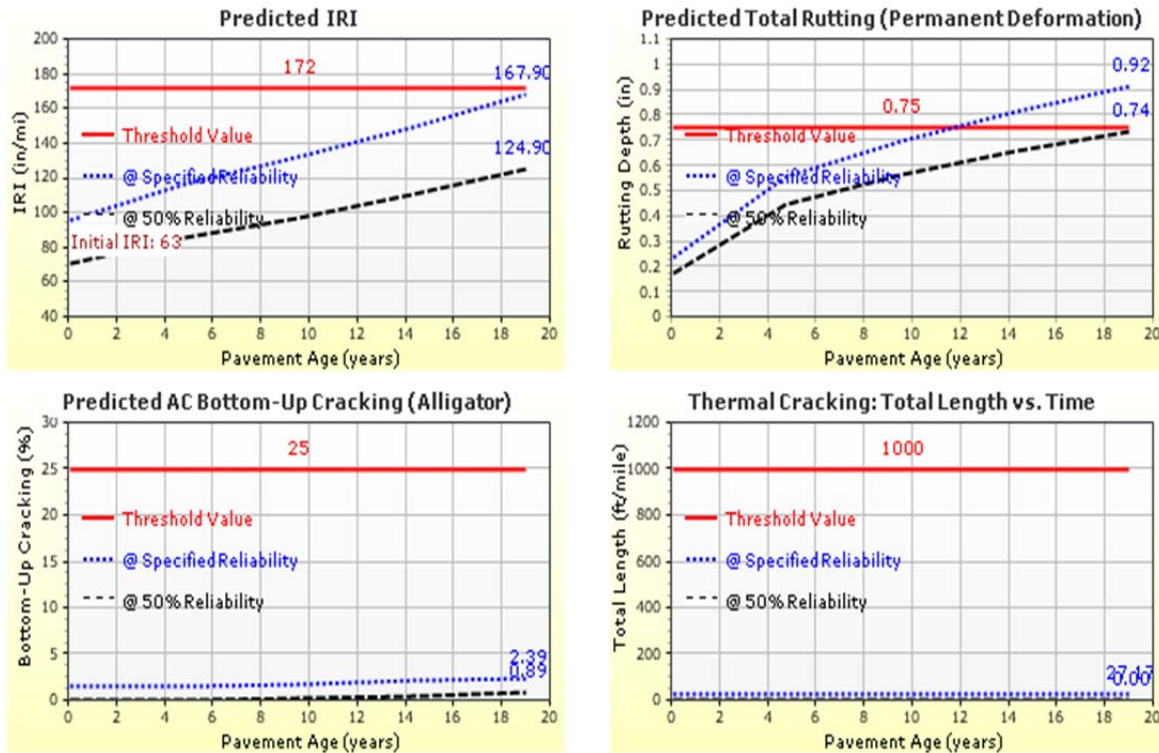


Figure 2.10 Example M-E Design Software Output.

### 2.3.8 Evaluate Results

The example output in Figure 2.10 shows that the hypothetical design has been nearly optimized with respect to IRI, oversized in terms of bottom-up cracking and thermal cracking, but under-designed with respect to rutting. At this point, the designer begins making decisions on how to improve the design. The pavement appears to have sufficient thickness to control bottom-up cracking, but rutting is a problem. Though thickness could be added to potentially control rutting, using thickness would result in even further over-designing for fatigue cracking. Another option may be to try different types of materials (change the binder grade, mix design or unbound materials in the base/subbase layers) to control rutting. In any case, the designer has the freedom to choose many different options to improve the design that may or may not include changes to thickness. In an iterative manner, changes are made and the simulations are conducted until the designer is satisfied with the pavement cross-section and predicted performance.

## 2.4 Perpetual Pavement Design

Perpetual pavement design can be considered a subset of M-E design with two important distinctions. First, the goal of perpetual design is to achieve long life with no deep structural distresses. These distresses include bottom-up fatigue cracking and rutting below the asphalt concrete layers. Therefore, while the AASHTO empirical and MEPDG design for a terminal level of serviceability or distress, perpetual pavement design seeks to avoid a terminal structural condition. Second, perpetual design recognizes that all materials have inherent endurance limits below which no damage will occur. Perpetual pavement design

works toward determining layer thicknesses that will keep the materials below their respective endurance limits to prevent structural damage.

Figure 2.11 illustrates the perpetual pavement design approach. Note that this approach is nearly identical to that presented in Figure 2.8 with two exceptions. First, an endurance limit has been added to the empirical performance prediction plots. Strain levels that exceed these endurance limits may cause damage to the pavement as shown by the higher strains in each plot intersecting the sloped portion of the line, resulting in a finite number of cycles until failure. Strain levels that fall below the endurance limit, as shown in Figure 2.11, result in an infinite number of cycles until failure in the particular mode of distress. The second modification is that the design assessment does *not* rely upon a damage value equal to one but, rather, checks that the damage is much less than one. In the case of PerRoad, described below, the damage target is 0.1. This step ensures that the pavement will not approach a terminal level of structural distress.

The other components in Figure 2.11 are handled in the same fashion whether conducting a perpetual pavement design or conventional M-E design. Loadings must be characterized and simulated on the pavement cross-section represented by its layer thicknesses and material properties. Critical pavement responses are computed according to a mechanistic model and then checked against respective endurance limits and transfer functions from which damage may be computed. Damage is computed according to Miner's Hypothesis, and the designer assesses the outcome and iterates as necessary.



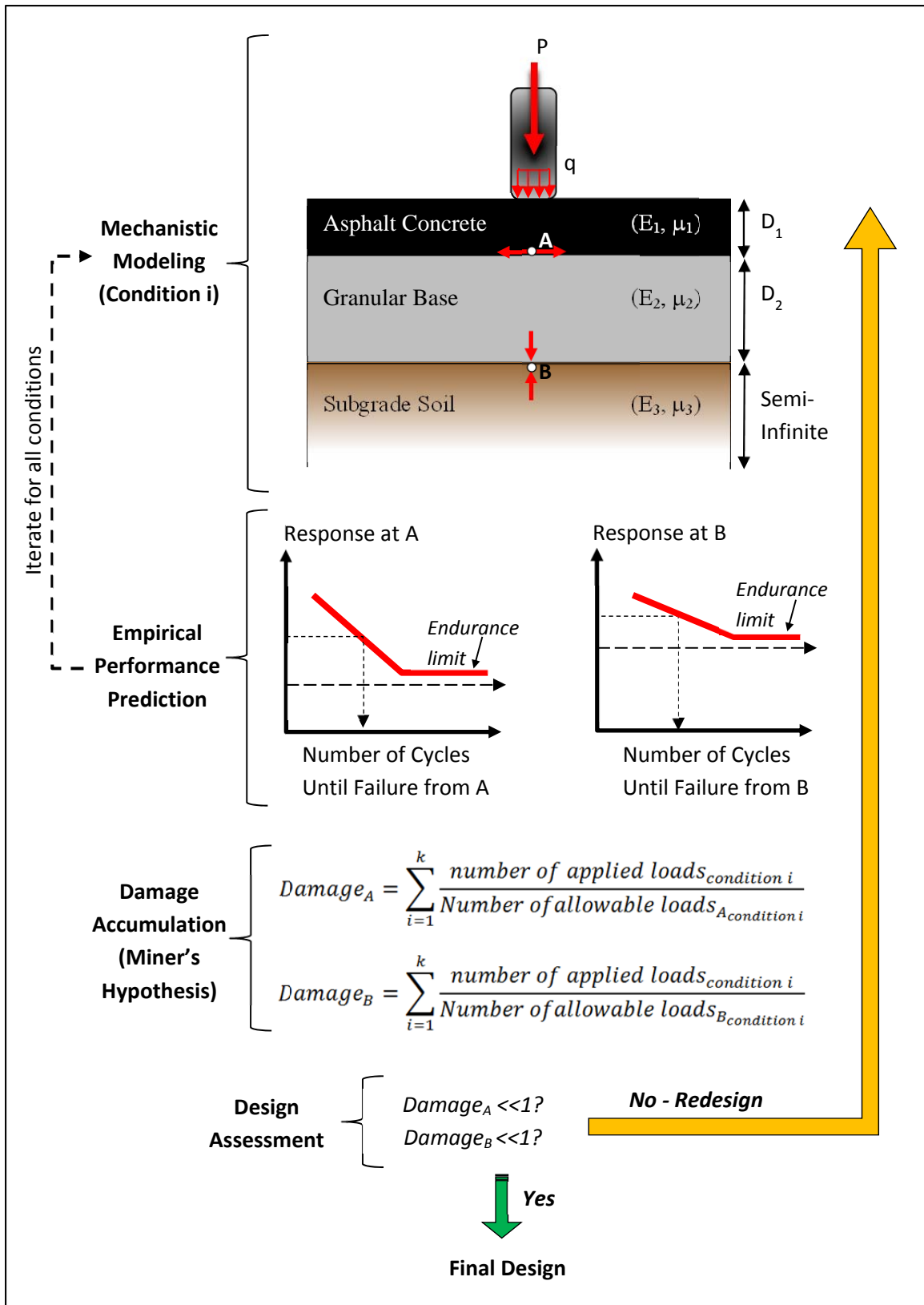


Figure 2.11 Perpetual Pavement Design Framework.

As discussed previously, tensile strain at the bottom of the asphalt concrete is critical in the prediction and prevention of bottom-up fatigue cracking. The fatigue endurance limit (FEL) is used to help control this distress by keeping strain levels in the pavement below this limit. An endurance limit of 70 microstrain ( $\mu\epsilon$ ) was first reported for asphalt pavements by Monismith and McLean (15). In a more recent study, Prowell et al. (16) provided laboratory testing data supporting the existence of a higher endurance limit that varied from 75 to 200  $\mu\epsilon$ . Endurance limits were also determined based on the analysis of long-life asphalt pavements. Nishizawa et al. (17) reported an endurance limit of 200  $\mu\epsilon$  for in-service pavements in Japan. For a long-life pavement in Kansas, strain levels at the bottom of the asphalt layer were between 96 and 158  $\mu\epsilon$  as calculated from back-calculated stiffness data (18). Yang et al. (19) reported a successful perpetual pavement design in China using an endurance limit of 125  $\mu\epsilon$  instead of a more conservative limit of 70  $\mu\epsilon$ .

Structural rutting occurs in either the aggregate base or subgrade layer under the imposed traffic. To control structural rutting in a perpetual pavement design, the vertical strain or stress at the top of the subgrade has been used as the limiting design parameter. Harvey et al. (20) proposed a limiting vertical strain of 200  $\mu\epsilon$ . They suggested that computed vertical strains at the top of the subgrade should be below this value to prevent structural rutting. Conversely, instead of limiting the vertical strain, Bejarano and Thompson (21) proposed controlling the vertical stress by evaluating the ratio of the vertical stress at the top of subgrade to the unconfined compressive strength of the soil, referred to as the subgrade stress ratio. They recommended using a subgrade stress ratio of 0.42 for design purposes.

There are several specific approaches to perpetual pavement design that warrant further discussion. These include the PerRoad program, the Illinois procedure and the Texas procedure. Each procedure is briefly discussed in the following subsections. Beyond these, very few states have established perpetual pavement design procedures. Perpetual pavements are mentioned in Chapter 14, Section 10 of the Wisconsin DOT Facilities Development Manual (22). Their approach uses mechanistic analysis to control strain at the bottom of the HMA pavement and is conducted by the WisDOT central office. The West Virginia DOT also has a perpetual pavement provision that utilizes the PerRoad software to facilitate design described in Design Directive 646 (23).

#### 2.4.1 *PerRoad and PerRoadXPress*

The PerRoad computer program was developed for the Asphalt Pavement Alliance to facilitate perpetual pavement design using the principles discussed above. The program allows the designer to incorporate up to five pavement layers and simulate seasonal effects and natural construction variability of each material. The program requires the designer to specify critical locations in the pavement and the type and magnitude of response (i.e., stress, strain or deflection) to use as endurance limits. PerRoad uses load spectra to characterize the types of axles and load magnitudes that will be simulated. Taking into account the seasonal effects, construction variability and load spectra, the program uses a technique called Monte Carlo Simulation to model the real-world nature of the pavement

and predict distributions of pavement response. These responses are used to predict the likelihood that the endurance limits will be exceeded, and if exceeded, how much time and traffic will pass before damage reaches 10% of the predicted failure. This level, much less than using 1.0 as the failure point, helps ensure deep structural distresses do not occur.

Another program, also created for the Asphalt Pavement Alliance, is a simpler version of PerRoad called PerRoadXPress. It uses the same design principles as PerRoad but has streamlined the inputs for designs that do not warrant full and detailed characterization of all the environmental, material property and load characteristics. It is primarily intended for low volume road design. Figure 2.12 shows the entire program. Relatively basic inputs are needed such as traffic volume, percent trucks and percent growth in addition to soil type, base thickness and asphalt modulus. These inputs are used to estimate the thickness of asphalt concrete needed to achieve a 35-year perpetual pavement.

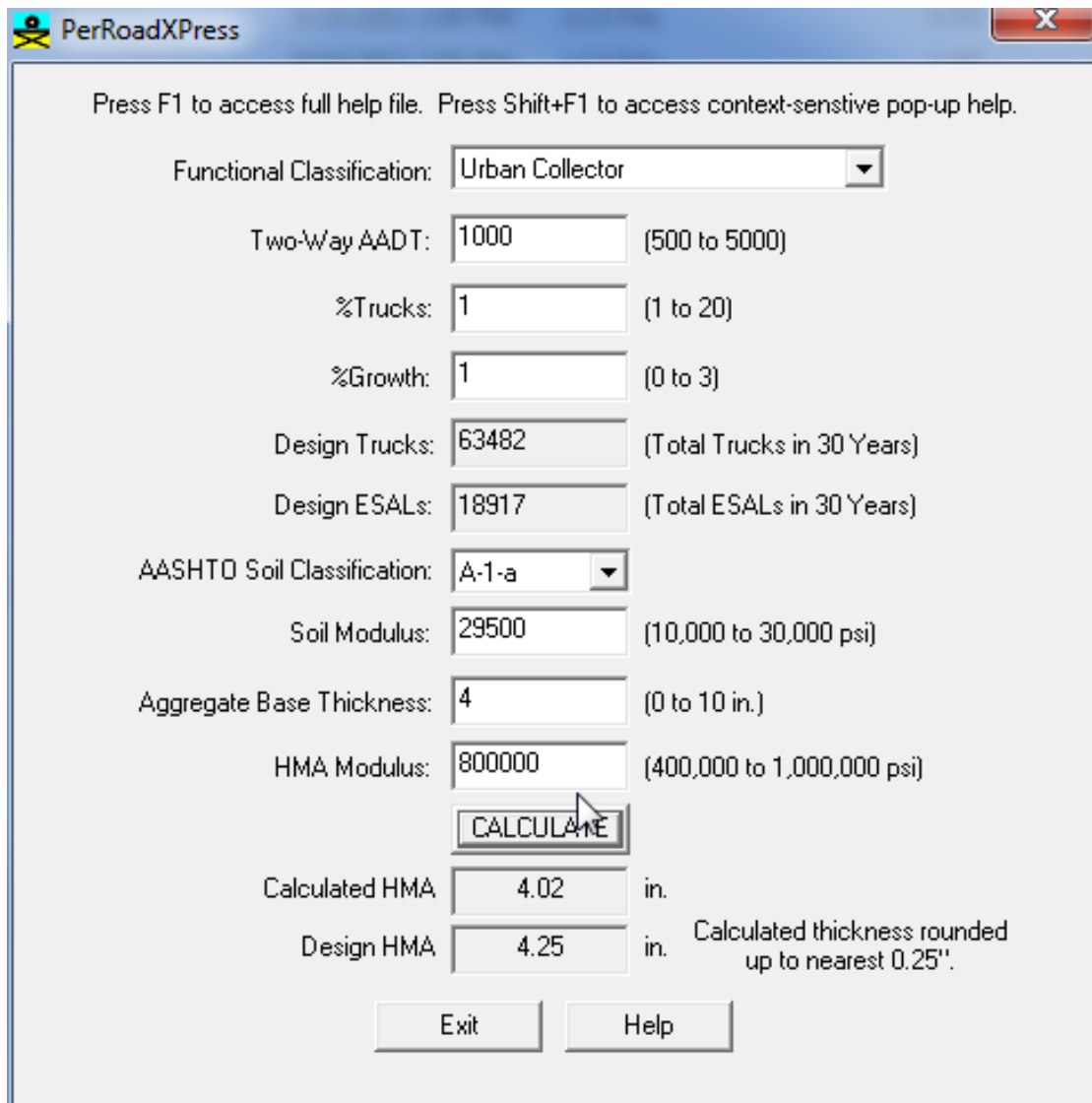
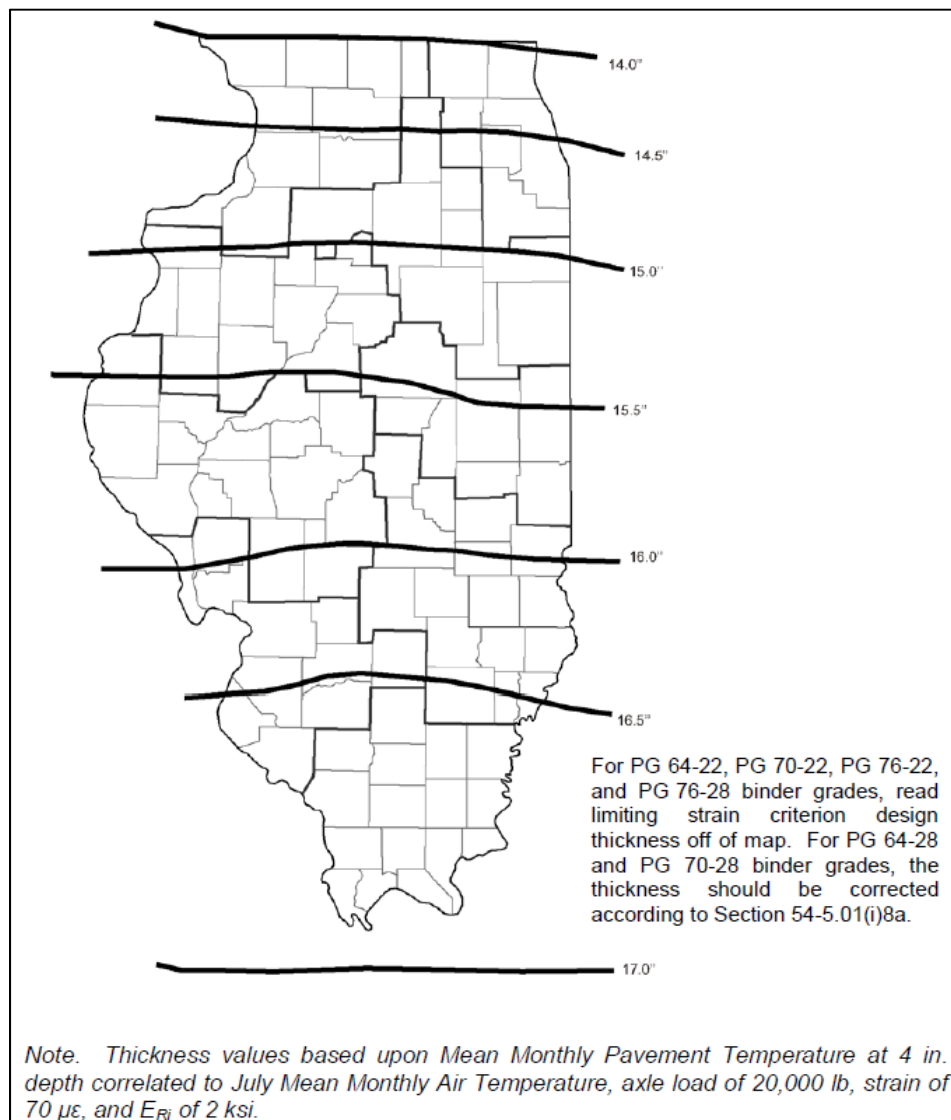


Figure 2.12 PerRoadXPress.

### 2.4.2 Illinois Perpetual Pavement Design

The Illinois procedure, described in Chapter 54 of the Illinois DOT Bureau of Design and Environment Manual (24), takes a simpler, yet effective, approach to perpetual pavement design. Developed by Thompson and Carpenter (25), this procedure uses the highest mean monthly pavement temperature to determine the corresponding asphalt modulus at this design temperature. A design load is selected (typically 18 or 20 kip) and applied to the pavement at the critical temperature and chosen level of subgrade support. The cross-section is designed to maintain the maximum tensile strain below  $70 \mu\epsilon$ . Figure 2.13 shows a geographically-based design chart developed from this process for the state of Illinois (24) where thicknesses range from 14 to 17 inches, increasing for more southerly counties where strain levels will be higher due to increased temperatures. It should be noted that this procedure is used to determine the maximum pavement thickness as a check of the conventional mechanistic-empirical approach in Illinois.



**Figure 2.13 Illinois Perpetual Pavement Thickness Design Chart (24).**

### 2.4.3 Texas Perpetual Pavement Design

According to Walubita and Scullion (26), the state of Texas has been designing and constructing perpetual pavements for routes with traffic exceeding 30 million ESALs since 2001. In the initial design proposals, the typical cross-section consisted of approximately 22 inches of asphalt concrete over at least 8 inches of treated (lime or cement) base material on a well-compacted subgrade soil (26). In 2005, an investigation was conducted to evaluate and possibly revise this typical design. This investigation formed the basis of the current Texas perpetual pavement design framework (26).

To facilitate perpetual pavement design in Texas, the analysis software FPS (flexible pavement system) Version 21W is used (26). The software, much like other programs, uses layered elastic analysis to compute pavement responses at the bottom of the asphalt concrete and at the top of the subgrade. A table of recommended design values for various material properties at 77°F was developed and shown in Table 2.1. This information emphasizes that regardless of the design system it is important to provide designers with some recommendations for material property values prior to executing a design.

**Table 2.1 Recommended Perpetual Pavement Material Property Design Values (26)**

<b>Layer/Material TxDOT</b>	<b>2004 Spec Item</b>	<b>Proposed Design Modulus Value (ksi)</b>	<b>Recommended Design Modulus Range (ksi)</b>	<b>Poisson's Ratio</b>
PFC (optional)	Item 342	350	300 – 450	0.30
SMA	Item 346	600	500 – 850	0.35
RRL – ¾" Superpave	Item 344	800	600 – 1200	0.35
RRL – Type B	Item 341	800	700 – 1300	0.35
RBL– Type C or ½" Superpave	Item 341	500	400 – 650	0.35
Base/foundation	Items 247, 260, 263, 275, & 276	Min 35	35 – 150	0.30 – 0.35
Subgrade		Should be back-calculated from existing or adjacent structure	-	0.40 – 0.45

Walubita and Scullion (26) report using the heaviest expected load as the design load, rather than load spectra, as discussed above within the ME Design software and PerRoad. Though not discussed in their report, the design load is presumably determined from an evaluation of available traffic data.

Given the material properties and loading condition mentioned above, a proposed pavement cross-section is evaluated in the FPS 21W software. The design is checked

against horizontal tensile strain at the bottom of the asphalt concrete ( $<70 \mu\epsilon$ ) and vertical compressive strain ( $<200 \mu\epsilon$ ) at the top of the subgrade to prevent fatigue cracking and rutting, respectively. Layer thicknesses are then adjusted until an optimum pavement structure is achieved.

The Texas design approach was further refined to provide proposed perpetual pavement cross-sections for three traffic levels depicted in Table 2.2 (26). At less than 30 million ESALs, the recommendation is for 12 inches of total asphalt concrete on at least 6 inches of aggregate base. For designs between 30 and 50 million ESALs, the design increases the AC thickness by 3 inches. At the highest traffic level ( $> 50$  million ESALs), the design calls for 15 inches of AC over at least 8 inches of aggregate base. All of the designs specify a rich bottom layer to mitigate bottom-up fatigue cracking and some kind of treatment (e.g., lime or cement) to the aggregate base layer to provide a stable foundation and prevent structural rutting. It is important to note the significant reduction in AC thickness (7 inches) of these newer designs compared to those initially developed in Texas.

**Table 2.2 Texas Proposed Perpetual Pavement Designs (26)**

<b>Layer#</b>	<b>Thickness (inches)</b>	<b>Mix Type</b>	<b>Designation</b>	<b>2004 TxDOT Spec Item</b>	<b>Material</b>
<b>(a) Traffic ESALs ≤ 30 million</b>					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	2	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 6	Type B	Main structural load- carrying rut-resistant layer	Item 341	PG 64-22 or better
4	2	Type C or ½-inch Superpave	Rich bottom fatigue- resistant layer (durability & impermeability)	Item 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 18 inches (12 inches HMA and 6 inches base)					
<b>(b) 30 million &lt; Traffic ESALs ≤ 50 million</b>					
1	2	SMA	Surfacing	Item 346	PG 70-28 or better
2	3	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 8	Type B	Main structural load- carrying rut-resistant layer	Item 341	PG 64-22 or better
4	2	Type C or ½-inch Superpave	Rich bottom fatigue- resistant layer (durability & impermeability)	Item 341	PG 64-22
5	≥ 6	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 21 inches (15 inches HMA and 6 inches base)					
<b>(c) Traffic ESALs &gt; 50 million</b>					
1	2-3	SMA	Surfacing	Item 346	PG 70-28 or better
2	≥ 3	¾-inch Superpave	Load transitional layer	Item 344	PG 70-22 or better
3	≥ 8	Type B	Main structural load- carrying rut-resistant layer	Item 341	PG 70-22 or better
4	2-4	Type C or ½-inch Superpave	Rich bottom fatigue- resistant layer (durability & impermeability)	Item 341	PG 64-22
5	≥ 8	Base	Lime or cement treatment	Items 260, 263, 275, & 276	
Subgrade (in-situ soil material)					
Minimum PP structure thickness = 23 inches (15 inches HMA and 8 inches base)					
*On top of the SMA, a PFC (TxDOT 2004 spec item 342) can be added as an "optional" surface promoting drainage, splash/spray reduction, noise-reduction, and skid-resistance. Preferably, the PFC layer thickness should be 1.5 inches.					

### 3. PAVEMENT DESIGN – STATE-OF-THE-PRACTICE

A variety of pavement design methods are currently in use throughout the U.S. A recent survey conducted by Pierce and McGovern (27) as part of NCHRP Project 20-05, Topic 44-06 grouped the methodologies into the following general categories:

- Empirical: includes AASHTO design guides through the 1993 edition and state-specific empirical methods. The state-specific empirical methods consist of granular equivalency or gravel factor design approaches. These strategies are very similar to the empirical AASHTO design approach; however, they do not use the structural coefficient terminology. Instead, they use a gravel factor or granular equivalency value to express the relative contribution of different materials to the overall strength of the pavement cross-section.
- MEPDG: AASHTO-endorsed M-E method
- Other ME: mechanistic-empirical methods other than the MEPDG

Figure 3.1 shows the geographic distribution of pavement design methodologies while Figure 3.2 provides a summary of design method usage. Currently, 56% of states are using some form of empirically-based design. 22% of states use both their pre-existing empirical method and the MEPDG, while 14% are using an empirical method along with some other form of M-E design. Three states (Illinois, Minnesota and Alaska) currently use a state-specific M-E method with one state (Indiana) using the MEPDG exclusively for pavement design.

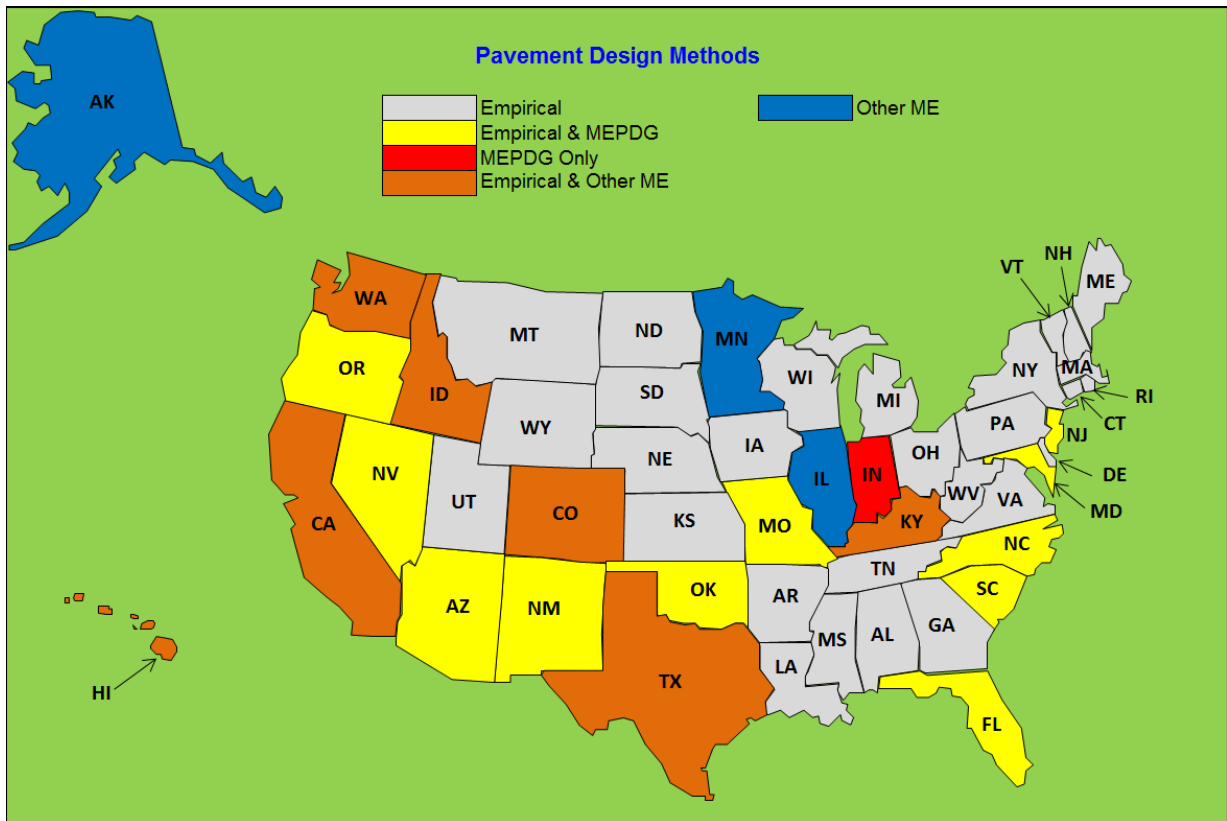
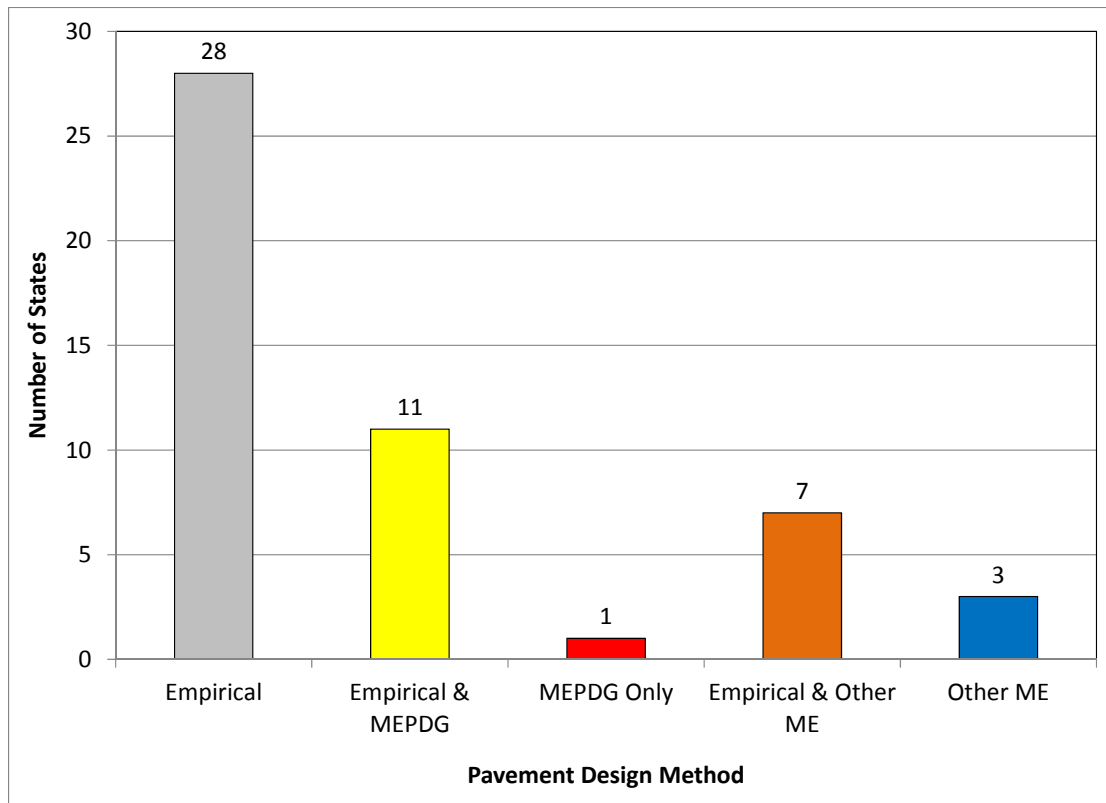


Figure 3.1 Pavement Design Methodologies by State (data from 27).





**Figure 3.2 Pavement Design Summary (data from 27).**

### 3.1 Empirical Design Approaches

As noted in Figures 3.1 and 3.2, empirical design is still the most popular approach in the U.S. with states using the following:

- 1972 AASHTO Design Guide (10%)
- 1986 AASHTO Design Guide (2%)
- 1993 AASHTO Design Guide (68%)
- Other Empirical - Granular Equivalency or Gravel Factor Design (10%)

Figure 3.3 shows the geographic distribution of empirical design methods across the U.S. with a two-thirds majority using the 1993 AASHTO Design Guide.

States using empirical methods have established structural coefficients for their asphalt materials that have a direct impact on the thickness of the pavement structure. Figure 3.4 shows the current distribution of asphalt structural coefficients in the U.S. (28). Currently, 45% of states use 0.44 for at least one paving layer, though some states specify coefficients according to the lift or mix design using a number of design gyrations ( $N_{des}$ ). Many states (28%) use less than the originally recommended AASHTO value of 0.44. Two states, Alabama and Washington, recently revised their structural coefficients to 0.54 and 0.50, respectively (29, 30). These increases reflect modern advances in the materials and construction practices and are more consistent with actual performance of flexible pavements in these states. The significant savings in optimizing asphalt pavement thickness through updating the structural coefficient should be noted. A change from 0.33 to 0.44 would result in 25%

thinner sections. An increase from 0.44 to 0.54, as implemented in Alabama, reduces the pavement thickness by 18.5%. These changes help agencies stretch their limited paving budgets to cover more miles through optimized pavement cross-sections, especially on high-volume pavements. Any increase in structural coefficients considered by a state agency should be carefully evaluated and supported by actual pavement performance data. Caution should also be exercised when considering these changes on lower volume routes where a minimum thickness may be considered.

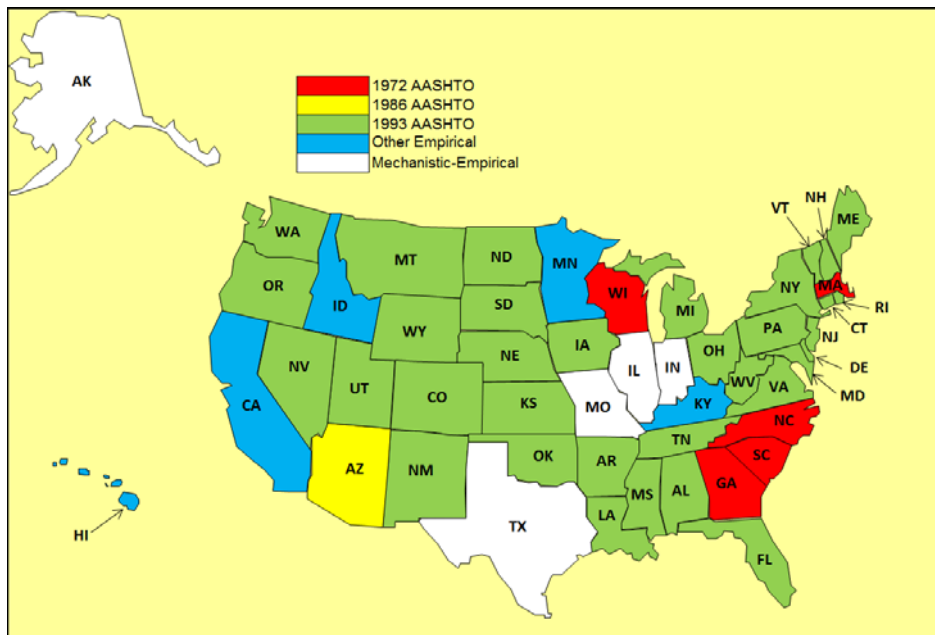


Figure 3.3 Empirical Design Methods.

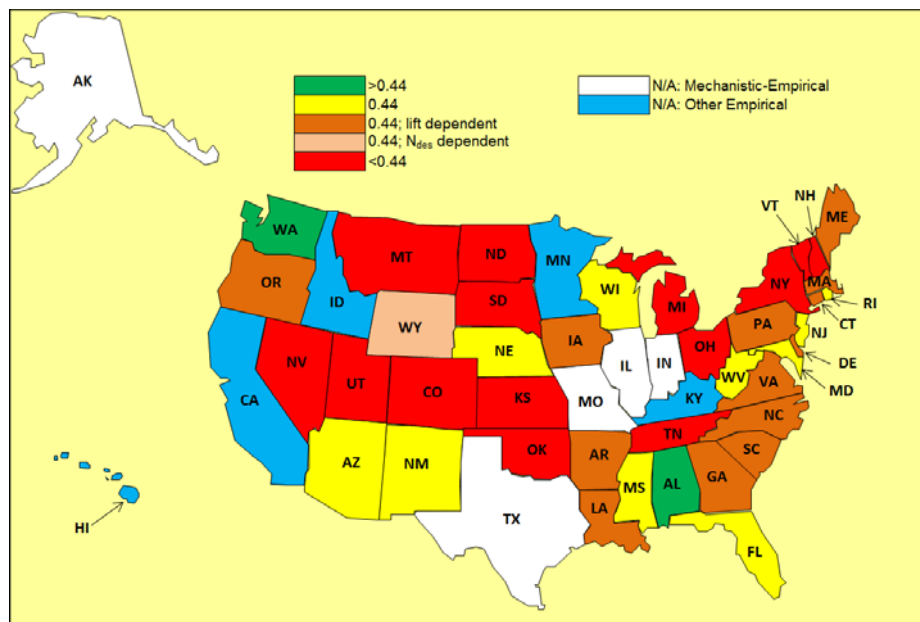


Figure 3.4 Asphalt Structural Coefficients (data from 28).

### 3.2 Mechanistic-Empirical Pavement Design

Figure 3.5 summarizes efforts towards implementing the MEPDG and ME Design software as reported by Pierce and McGovern (27). Three states currently have implemented the method while 28% are working toward implementation in the next two years. Half of the states expect to implement within the next two to five years with 4% projecting at least five years until implementation. There are 12% of states that do not currently plan to implement the new design approach (27). It should be emphasized that these implementation plans are estimates from a single survey of state agencies and certainly may change as the implementation process progresses.

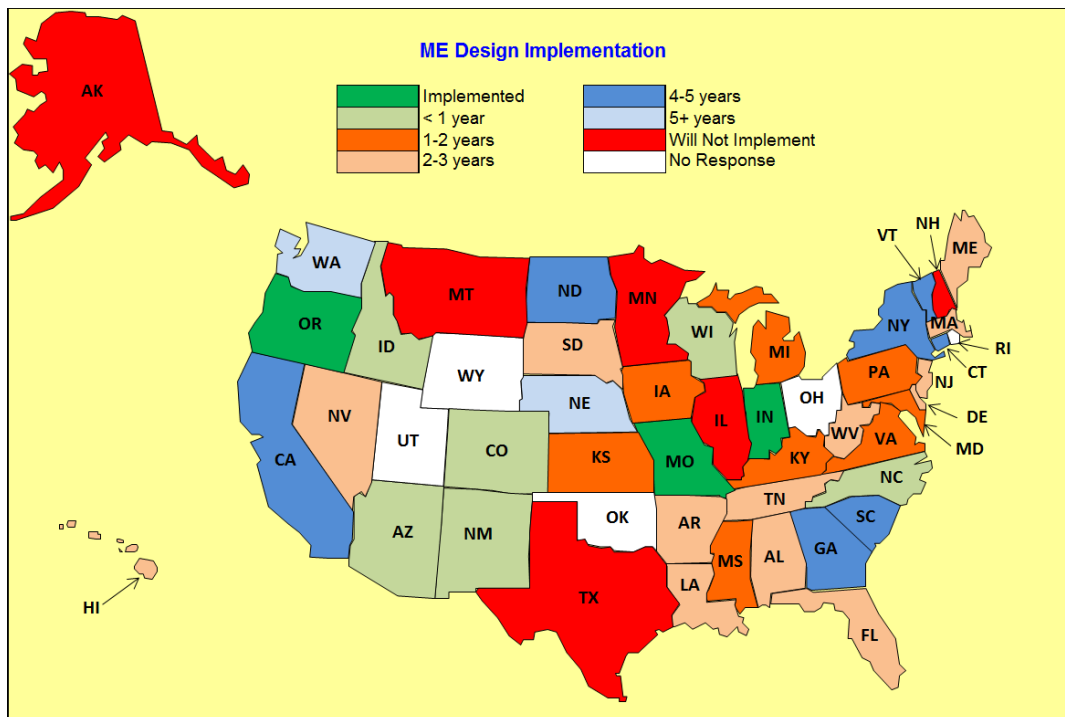


Figure 3.5 MEPDG and ME Design Software Implementation (*data from 27*).

Implementation activities may be grouped into materials characterization, climate characterization, load spectra characterization, transfer function calibration and training. Many state agencies have worked or are currently working in these areas with the ultimate goal of successfully implementing the new design procedure. Pierce and McGovern (27), based on their survey, indicated that a majority of states need assistance in local calibration of the MEPDG performance prediction models and software training. They also received suggestions to develop a dedicated MEPDG website for sharing technical information, training in interpreting MEPDG results, training in obtaining inputs, training in ME principles and training on modifying pavement sections to meet design criteria. Pierce and McGovern (27) also conducted a literature review that identified the following common elements in agency implementation planning:

- Identification of pavement types to include in the implementation process
- Determining needed data

- Defining materials and traffic input libraries
- Establishing threshold limits and reliability levels for each performance prediction model
- Verifying the predicted pavement performance
- Updating agency documents to include pavement analysis using the MEPDG
- Providing training to agency staff on ME principles, MEPDG and the ME Design software

State-specific implementation activities were further investigated by Pierce and McGovern (27) and included the following:

- Alabama – currently working on traffic study with plans to develop a materials library and conduct local calibration
- Arizona – full implementation on major roadways expected in 2014
- Georgia – currently conducting local calibration
- Idaho – conducted software training through consultant; developed ME user guide and implementation roadmap; DOT staff comparing results between current procedure and MEPDG
- Iowa – locally calibrated performance prediction models
- Louisiana – plans to begin local calibration process and have conducted comparisons between current method and MEPDG
- Michigan – plans to transition to MEPDG in 2014
- Mississippi – conducting local calibration of performance models
- Oklahoma – currently working on local calibration of asphalt pavement performance prediction models
- South Carolina – conducting side-by-side comparisons and materials characterization; future plans for local calibration
- Wisconsin – completed studies related to asphalt mixtures, concrete properties and resilient modulus of subgrade soils; currently developing a user manual and conducting local calibration

It is interesting to note that six U.S. agencies have no plans to implement the MEPDG and software. Reasons for not implementing were cited as follows (27):

- Currently using state-specific M-E procedure
- Current design practice is acceptable
- Software cost
- Waiting for more agencies to implement
- Disagreement with the MEPDG modeling approach

As shown in Figure 3.5, three agencies (Indiana, Oregon and Missouri) have fully adopted the MEPDG and ME Design software. Of these, only Indiana is using the new approach exclusively while the others use both empirical and M-E procedures. The following subsections, based on Pierce and McGovern's report (27), document these states' experiences.

### 3.2.1 Indiana Implementation

The Indiana DOT (INDOT) began evaluating the MEPDG in 2002 at the direction of the INDOT Pavement Steering Committee. The design approach was fully implemented seven years later in 2009. According to Pierce and McGovern (27), the implementation plan included the following:

- Review current state-of-knowledge in pavement engineering and management
- Review and document hierarchical design input parameters for each level of design accuracy (document sensitivity of design inputs to distress and smoothness prediction). Review and document relevant data contained in the Indiana DOT and LTPP databases
- Review the readiness of laboratory and field equipment needed for quantifying higher-level MEPDG inputs. Acquire needed equipment and develop a testing program.
- Develop and execute a plan to establish:
  - Local calibration and validation of distress prediction models
  - Regions and segments for traffic input module
  - Populate software with additional climatic data
- Establish “mini LTPP” program to more accurately calibrate the MEPDG performance prediction models
- Develop correlations and equations for soil resilient modulus, load spectra regions and segments based on existing WIM (weigh-in-motion) and AVC (automated vehicle classification) data and a process to aid designers in easily migrating traffic data into the software
- Provide technology and knowledge transfer and MEPDG training to other divisions, districts, local agencies, contractors and consultants
- Revise INDOT Design Manual Chapter 52, Pavement and Underdrain Design Elements

To evaluate material property needs, INDOT conducted a sensitivity analysis with the MEPDG on a typical Indiana cross-section using input values similar to their previous design method (1). They identified the need for an asphalt materials library that characterized the dynamic modulus, creep compliance and indirect tensile strength of mixtures commonly specified by INDOT (27). Furthermore, the INDOT LTPP sites were evaluated using the MEPDG, and the calibration coefficients of the asphalt transfer functions were adjusted to provide a better match between measured and predicted performance (27). Additional calibration was conducted using “mini LTPP” sites in Indiana and the INDOT accelerated pavement testing facility (27). Table 3.1 lists the recommendations made by INDOT regarding their transfer functions.

**Table 3.1 INDOT Transfer Functions (data from 27)**

Pavement Type	Performance Indicator	Transfer Function Calibration	
		Local	National
All	IRI	✓	
Asphalt	Longitudinal cracking <sup>1</sup>		
	Alligator cracking	✓	
	Transverse cracking <sup>1</sup>		
	Rutting – asphalt layers		✓
	Rutting – total <sup>1</sup>		
	Reflective cracking <sup>1</sup>		
JPCP	Transverse cracking		✓
	Joint faulting		✓

<sup>1</sup> Distress criteria is not used to determine the recommended pavement structure.

It should be noted that when asked about local calibration, INDOT responded with the following (27), “Local calibration is a plus, but should not be a requirement for implementing the MEPDG. The Indiana DOT utilized eighteen LTPP test sections and agency research sections, but this certainly was not enough to fully calibrate the performance prediction models. However, obtaining data to meet all aspects of the calibration process may take years. Indiana DOT took the approach that it was better to conduct verification and validation of selected pavement sections with good pavement history, rather than attempt to obtain data on all agency-applicable pavement types.”

INDOT reported that one of the most difficult aspects of their implementation effort was handling traffic data (27). Therefore, INDOT developed a software tool that visualizes WIM and AVC site locations for easy access to MEPDG traffic input data. The tool also allows for direct export of the traffic data into the MEPDG and established a Level 2 traffic database for designers to easily select traffic inputs on the basis of average annual daily truck traffic (27).

According to Nantung (31), INDOT is saving approximately \$10 million per year when comparing designs from the 1993 AASHTO Design Guide to the new design approach. INDOT also reports improved reliability of the design recommendations, improved characterization of local and new materials, improved characterization of the existing pavement layers, better traffic characterization, improved confidence in distress prediction and improved knowledge of DOT staff in pavement design and performance (27).

### 3.2.2 Missouri Implementation

Missouri DOT (MODOT) began their implementation activities in 2005 and decided that the potential benefits of using the new approach would outweigh the risks associated with using a procedure that had not yet been fully evaluated, calibrated or validated (27). In 2008, MODOT became the first state highway agency to implement the MEPDG for the design of thick asphalt pavements. MODOT currently uses the MEPDG for the design of

new asphalt and jointed plain concrete pavement. MODOT worked with a consultant to evaluate their historical traffic data, collected by portable WIM sites, with respect to the MEPDG. They determined that the MEPDG default truck traffic defaults (Level 3) were suitable for routine designs (27).

Asphalt material testing was conducted by MODOT. They determined that the MEPDG dynamic modulus predictive model adequately reflects MODOT conventional dense-graded mixtures (27). This allows MODOT to design with Level 3 asphalt material inputs. They also determined that the MEPDG prediction equation tended to under predict the dynamic modulus at high frequencies (27). This under prediction has the potential to yield overly conservative designs.

The calibration conducted by MODOT included 36 agency-specified sites and 41 LTPP sites (27). Table 3.2 shows the distribution of sites according to pavement type while Table 3.3 shows the MODOT recommendations for whether to use the MEPDG default or locally calibrated transfer functions based on their investigation. Though MODOT has recommendations regarding transfer function calibration for a variety of flexible pavement distresses, only alligator cracking and rutting in the AC layers is currently used for design purposes due to concerns regarding data validity (27).

**Table 3.2 MODOT Field Testing Sites (data from 27)**

Pavement Type	Agency	LTPP	Total
New JPCP	25	7	32
New asphalt	6	14	20
Asphalt overlay of existing asphalt	0	11	11
Asphalt overlay of existing JPCP	0	5	5
Asphalt overlay of rubblized JPCP	0	5	4
Unbonded concrete overlay	5	0	5
Total	36	41	77

**Table 3.3 MODOT Transfer Functions (data from 27)**

Pavement Type	Performance Indicator	Transfer Function Calibration	
		Local	National
All	IRI	✓	
Asphalt	Longitudinal cracking		✓
	Alligator cracking		✓
	Transverse cracking	✓	
	Rutting – asphalt layers	✓	
	Rutting – total	✓	
	Reflective cracking		✓
JPCP	Transverse cracking		✓
	Joint faulting		✓

Pierce and McGovern (27) reported the following benefits to MODOT for having implemented the new design approach:

- Cost savings due to more economical designs
- Improved characterization of local materials
- Improved characterization of existing pavement layers and traffic
- Improved confidence in distress prediction

### 3.2.3 Oregon Implementation

The Oregon DOT (ODOT) began evaluating the MEPDG in 2006 and implemented it for new construction or reconstruction for high volume routes in 2009 (27). The MEPDG is currently used for both flexible and rigid pavement design and ODOT is looking for opportunities to expand the use of mechanistic design and design concepts (32).

ODOT has adopted a Level 3 approach to material characterization. A study by Lundy et al. (33) determined the dynamic modulus for ODOT standard asphalt mixtures and confirmed that the MEPDG regression equation to predict dynamic modulus was in good agreement with their data (27). Furthermore, ODOT uses default (Level 3) data for the majority of their inputs.

ODOT maintains twenty-two WIM sites across the state from which four were used to generate virtual truck classifications that represent low, moderate and high traffic volumes (27). These virtual classifications are available for import into the ME Design software.

A local calibration using 108 pavement surveys from 36 test sections was conducted by ODOT (27, 34). The calibration was divided into regional locations (coastal, valley and eastern) and traffic levels (low, moderate and high). The resulting transfer function recommendations are listed in Table 3.4.



**Table 3.4 ODOT Transfer Functions (data from 27)**

Pavement Type	Performance Indicator	Transfer Function Calibration	
		Local	National
All	IRI		✓
Asphalt	Longitudinal cracking	✓	
	Alligator cracking	✓	
	Transverse cracking	✓	
	Rutting – asphalt layers	✓	
	Rutting – total	✓	
	Reflective cracking		✓
JPCP	Transverse cracking		✓
	Joint faulting		✓
Continuously Reinforced Concrete Pavement	Punchouts		✓

ODOT reported a number of challenges while implementing the MEPDG. These included availability of materials characterization data, funding restrictions, limited time available and justification of benefits for implementing more advanced procedures (27). Despite these challenges, ODOT reported increased confidence in the performance prediction models though the benefits have yet to be quantified.

#### 4. SUMMARY

Pavement thickness design is rapidly transitioning from empirical methods developed in the 1960's to a more modern and adaptable mechanistic-empirical framework. The limitations of the AASHTO empirical design system and frequent extrapolations leading to overly conservative designs are motivating many agencies to consider the new AASHTO-approved MEPDG and ME Design software. Design and construction of perpetual pavements, using mechanistic principles in combination with endurance limits, have also become a viable option as agencies seek long-life, high-performing pavement structures.

As of 2013, 56% of state agencies use some form of empirically-based pavement thickness design. Of these, the majority (68%) use the 1993 AASHTO Design Guide with 10% using the 1972 edition and 2% using the 1986 version. A small group of states (10%) use their own granular equivalency or gravel factor empirical design procedures. Within the states using one of the AASHTO empirical methods, 45% use 0.44 as the asphalt structural coefficient. This value was originally recommended by AASHTO in 1962 with the publication of the AASHTO Road Test reports (2). A large portion (28%) of states use a value less than 0.44 with only two (Alabama and Washington) using a value greater than 0.44. Given the many advances made in asphalt technology and pavement construction since 1962, it is reasonable that the structural coefficient has increased over time and states should

consider evaluating their design value if continued use of the AASHTO empirical method is expected.

Some states (22%) are transitioning from empirical to mechanistic-empirical, where both methodologies are used concurrently, and one state (Indiana) uses the MEPDG approach exclusively. Two other states (Oregon and Missouri) have fully implemented the MEPDG but use it in combination with an empirical approach. Each of the fully-implemented states have documented their implementation process and it is of note that they each use a combination of local and national (default) input data and transfer function calibration factors in their design procedures.

While three states have implemented the MEPDG and ME Design software, many are working toward that goal. It is well recognized as a time consuming process, as materials, traffic, climate and performance must be locally characterized and calibrated for each agency. There are 28% of state agencies working toward implementation in the next two years. Half of the states expect to implement within the next two to five years with 4% projecting at least five years until implementation. There are 12% of states that do not currently plan to implement the new design approach (27).

Very few states have an established perpetual pavement design procedure. Illinois' procedure involves evaluating the pavement structure under the warmest conditions and designing for the heaviest expected load to find pavement thickness. The approach in Texas is similar, using the flexible pavement system software to evaluate the pavement section under critical conditions to find the perpetual pavement thickness. Both states have published recommended design thicknesses for a range of conditions. Illinois' maximum thickness, under the warmest condition, is 17 inches of full asphalt concrete, while the Texas approach results in 15 inches of asphalt concrete over at least 8 inches of aggregate base for the heaviest traffic condition. The PerRoad and PerRoadXPress procedures are also available for perpetual pavement design, though they are not currently an officially accepted procedure in any state. These programs rely upon modeling the pavement variability and loading variation to determine the appropriate perpetual pavement cross-section given a set of input conditions.

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