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North Dakota Implementation of Mechanistic-Empirical Pavement Design Guide (MEPDG)

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Understanding Mechanistic-Empirical Pavement Design Guide (MEPDG) for North Dakota Implementation

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ABSTRACT

North Dakota currently designs roads based on the AASHTO Design Guide procedure, which is based on the empirical findings of the AASHTO Road Test of the late 1950s. However, limitations of the current empirical approach have prompted AASHTO to move toward the new mechanistically based pavement design procedure described in the Mechanistic Empirical Pavement Design Guide (MEPDG), which was released to the public for review in 2004 under NCHRP Project 1-37A. MEPDG combines mechanistic and empirical methodology and provides more realistic characterization of in-service pavements. Its mechanistic approach is both more thorough and more computationally complex than the existing AASHTO design method, and as a result the method can require an extensive number of detailed material, foundational, traffic, and environmental inputs. This and other factors can present a challenge to agencies wishing to implement the new method.

Because AASHTO has adopted the MEPDG and highway agencies across the nation are moving towards its implementation, it is critical that North Dakota becomes familiar with the MEPDG documentation and software and identify input data requirements for design. This report summarizes the findings of MEPDG implementation in North Dakota, identifies input data needs and research steps of the MEPDG implementation in the state and also prepares North Dakota for successful implementation of the MEPDG statewide.

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1. INTRODUCTION

1.1 Background

The 1993 version of the American Association of State Highway and Transportation Officials (AASHTO) Guide for the Design of Pavement Structures is currently the primary document used to design new and rehabilitated highway pavements in the United States. According to a 2007 FHWA survey (FHWA 2007), 63% of state DOT's use the 1993 design procedure. AASHTO 1993 design equations were based on the empirical findings of the AASHO Road Test, conducted from 1958 to 1960 in Ottawa, Illinois. The Road Test was conducted in a single geographical location on a limited number of flexible and rigid pavement sections and subjected to a limited number and type of traffic loads. In that sense, the empirical relationships derived from the Road Test are truly representative only of the conditions present at the Road Test in Illinois. Moreover, the models developed and modified from the Road Test only relate key pavement properties and traffic to performance and do not consider the climatic effects.

In order to overcome the limitations of the existing design method, the AASHTO Joint Task Force on Pavements, in conjunction with the National Cooperative Highway Research Program (NCHRP) in NCHRP Report 1-37A, developed the Mechanistic-Empirical Pavement Design Guide (MEPDG). The new MEPDG procedure involves three major steps:

1. Use the known mechanistic properties of materials to compute the internal material responses in deflections, stresses, and strains in a trial design when subjected to predicted future traffic and climatic factors.
2. Convert predicted material response to accumulated pavement damages in terms of cracking, rutting, and smoothness. Repeat steps 1 and 2 until pavement performance passes the design criteria.
3. Continue testing trial designs; select the best one based on life cycle cost analysis and other considerations.

The Mechanistic-Empirical Pavement Design Guide (MEPDG) is being widely implemented and used by many highway agencies across the nation. As of 2007 (FHWA 2007), 80% of states have an MEPDG implementation plan and many of them consider the MEPDG as the goal for future pavement design and analysis. Moreover, AASHTO is expected to adopt the MEPDG in the future. However, a number of challenges in successful MEPDG implementation have been identified. Understanding these challenges is important for agencies considering implementing the Guide.

First, MEPDG models require an extensive amount of input, including detailed traffic spectra, pavement material properties, and environmental conditions. The procedures, training, and personnel required to collect much of these data is neither easy or inexpensive; moreover, preparation of the collected data to meet the MEPDG input requirement can also be challenging.

Further, the new MEPDG models were nationally calibrated and validated using Long-Term Pavement Performance (LTPP) data from the Federal Highway Administration (FHWA). However, nationally calibrated models can have limited accuracy in local applications because of significant differences between national and local traffic and environmental and material properties. As such, it is critical that MEPDG models be calibrated and validated for local conditions before they are implemented on an agency-wide level.

1.2 Research Objectives

The goal of this research is to prepare North Dakota to successfully implement MEPDG. Objectives include:

- To compare the principle, major inputs, and distress models of the two design procedures (1993 Design and MEPDG)
- To identify the input data needs for local calibration of MEPDG flexible pavement distress models in North Dakota
- To perform a review of sensitivity analysis literature, identifying key input parameters that are most significant to MEPDG pavement performance predictions
- To document procedures of the new AASHTOWare Pavement ME-Design software

1.3 Report Organization

This section is the introduction of the organization of the report. Section 2 summarizes the flexible pavement design procedures of AASHTO 1993 and MEPDG. Section 3 introduces the need for local calibration, including identification and summarization of the requirements for collecting input data. Section 4 addresses sensitivity analysis and identifies MEPDG input variables which have the most significant impact on pavement performance predictions. Section 5 introduces and documents the procedures and steps necessary to design and analyze flexible pavements in the current version of AASHTOWare ME-Design software, build 1.3.29. Finally, Section 6 summarizes the conclusions and recommendations from this study.

2. PAVEMENT DESIGN PROCEDURES

North Dakota Department of Transportation (NDDOT) currently designs its highway pavements in accordance with the AASHTO Guide for Design of Pavement Structures. The AASHTO guide has long been widely accepted as the standard in pavement design, with versions released in 1961, 1972, 1986, and most recently in 1993. The guide is fundamentally based upon empirical models built from field performance data collected at the AASHO Road Test in Ottawa, Illinois, from 1958 to 1960. The design method not only calculates required thicknesses of pavement layers but also evaluates pavement performance in terms of surface distresses due to traffic loading over pavement design life.

Empirical pavement design methods are typically based on prediction equations or curves derived from field or laboratory data. The relationships built from field data are typically verified against performance expectations or engineering judgment. In general, the overall serviceability of a pavement section is often modeled using a relationship between traffic, typically measured by a single index, such as Equivalent Single Axle Load (ESAL), and a composite performance index which represents combined distresses.

A mechanistic design approach, in contrast, is based on theories of mechanics of structural behavior and uses, for example, layered elastic analysis to calculate internal elastic strains induced by traffic loads and environmental conditions.

Mechanistic-empirical design is a hybrid procedure. It involves calculating elastic strains in pavement in response to traffic load and environmental conditions based on mechanics theory. These structural responses are then linked to stress predictions from empirical models. National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004) developed the most recent M-E based model to predict distresses by traffic load and environmental conditions. The NCHRP 1-37A project also incorporated national calibration models and detailed vehicle load spectra. The NCHRP also released a companion computational software, DARWin-ME, which has since been rebranded AASHTOWare Pavement ME Design. The latest software build as of this writing is 1.3.29.

The majority of state transportation agencies currently utilize AASHTO 1993 empirical design, although many have implemented or have begun to implement the new MEPDG. A 2007 survey by FHWA found that 63% of state DOTs currently use the AASHTO 1993 design guide and 80% of state DOTs have plans for MEPDG implementation. The current AASHTO 1993 guide and the MEPDG design procedure for flexible pavement will be described in the following sections.

2.1 The AASHTO 1993 Design Guide

The AASHTO 1993 design approach produces a required pavement structure from an empirical design equation with traffic, material, and climatic inputs. The output, layer thicknesses, are deterministically calculated using the design equation.

The first version of the AASHTO Design Guide was released in 1961 as the “AASHO Interim Guide for the Design of Rigid and Flexible Pavements.” All versions of the AASHTO design guide were based on the results of the AASHO Road Test. The AASHO Road Test and evolution of the AASHTO Guide will be introduced in this section, followed by the 1993 design equations and input requirements.

2.1.1 AASHO Road Test and Various Versions of the Design Guide

The AASHO Road Test studied structural performance of pavements with known thickness under moving loads of known magnitude and frequencies (HRB 1961). It was designed to investigate, through a series of experiments, the relationship between repeated traffic loading and highway pavement deterioration.

The road test was carried out from October 1958 to November 1960 in Ottawa, Illinois. The seven-mile test road was constructed from August 1956 to September 1958 and the road test consisted of six two-lane loops along Interstate 80 and eventually became part of the highway. The subgrade consisted of fine-grained silty clay (AASHTO soil classification A-6 or A-7). The base course material was a crushed dolomitic limestone, and hot-mix asphalt (HMA) mixes included crushed limestone coarse aggregate, natural siliceous coarse sand, mineral filler (limestone dust), and penetration grade asphalt cement.

Loop 1 was not subject to traffic and was used only to test environmental effects. Loops 2 through 6 were subject to five different traffic load conditions, which included interaction of both vehicle type and weight. Each loop consisted of segments of four-lane divided highway (two lanes in each direction) connected at both ends by a turnaround. The climate was a typical for the region with average temperature 76°F in summer and 27°F in winter. Average annual precipitation was 34 inches (HRB 1961).

Roughness, pavement deflections, strains, and the Present Serviceability Index (PSI) were collected from the test sections and then used to develop a pavement design procedure. Overall pavement serviceability was quantified using a single measure, PSI, which is a composite performance measure designed to represent the combined effect of cracking, patching, rutting, and other distresses on road user experience. The principal component of pavement performance and governing factor of PSI is roughness (Li, Q., Xiao, D., Wang, K., Hall, K., and Qiu, Y., 2011).

The first design procedure based on road test results was issued in 1961. The design equation was empirically developed for the test road's specific subgrade, pavement materials, and climate conditions. AASHO began to accommodate various regional conditions into the original empirical relationships in an updated 1972 release. The major changes of the 1972 version (AASHTO 1972) include an empirical soil support scale for various local subgrade soils and a new regional factor for adjusting the structural number for the local environment. The 1986 version (AASHTO 1986) revised the 1972 guide by adding more features to the design procedure. The major four additions include subgrade and unbound materials effects by resilient modulus, pavement drainage coefficients in the structural number equation, environmental effects in total serviceability loss and subgrade resilient modulus, and the concept of reliability factor. Few changes to the design guide occurred between the 1986 and 1993 versions except the use of non-destructive testing for evaluating existing pavement and back-calculation of layer moduli to determine layer coefficients.

The flexible pavement design equation used in the 1993 AASHTO Design Guide is shown below (AASHTO, 1993):

$$\log(W_{18}) = Z_R S_0 + 9.36\log(SN+1) - 0.2 + \frac{\frac{\log(\Delta PSI)}{4.2-1.5}}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32\log(M_R) - 8.07 \qquad (1)$$

where:

W_{18} = Predicted accumulated 18 kip equivalent single axle load for the design period
Z_R = Reliability factor (standard normal deviate)
S_0 = Combined standard error of the traffic prediction and performance prediction
ΔPSI = Difference between initial design serviceability index and the terminal design serviceability index
M_R = Subgrade resilient modulus (psi)

SN = Structural number:

$$SN = \alpha_1 D_1 + \alpha_2 D_2 m_2 + \alpha_3 D_3 m_3 \quad (2)$$

α_i = i^{th} layer coefficient

D_i = i^{th} layer thickness, in

m_i = i^{th} layer drainage coefficient

Structural Number (SN) is calculated by entering the required traffic, reliability, serviceability, and subgrade inputs. The SN equation can then be used to determine layer thicknesses. The SN equation allows different combinations of thicknesses to be used, so the final selection of pavement structure must be constrained, for example, by cost or policy considerations. The following steps describe a top-to-bottom design procedure for a three-layer pavement:

1) Calculate SN_1 needed to protect base layer, using E_2 as M_R in Equation (1), and compute the thickness of layer 1 as: $D_1 \geq \frac{SN_1}{\alpha_1}$
2) Calculate SN_2 needed to protect subgrade layer, using subgrade effective resilient modulus as M_R in Equation (1), and compute the thickness of layer 2 as: $D_2 \geq \frac{SN_2 - \alpha_1 D_1}{\alpha_2 m_2}$

2.1.2 AASHTO 1993 Design Guide Inputs

The inputs required for the AASHTO guide are separated into four categories: (1) initial input, (2) traffic input, (3) material input, and (4) environmental input.

Initial input

Performance criterion ΔPSI, defined as the difference between initial (i.e., post-construction) serviceability index and terminal (i.e., end of design life) serviceability, is required. Therefore, both the initial and terminal serviceability indices are required to determine the acceptable change in serviceability throughout design life. The average initial serviceability value at the AASHO Road Test was 4.2. The 1993 AASHTO Guide recommends a terminal PSI of 2.5 for major highways and 2.0 for low volume highways. With these specifications, ΔPSI can range from 1.7 to 2.2. While the 1993 Design Guide recommends these values, most state DOTs use their own specifications that are suitable to their unique conditions. Washington State DOT, for example, has used an initial serviceability index of 4.5 and terminal serviceability index of 3 (Li, Uhlmeyer, Mahoney, and Muench 2011).

Reliability, defined by AASHTO as the probability that the designed pavement will perform adequately over the design period, is another initial input. It consists of two variables. First, the combined standard error of the traffic prediction and performance prediction, S_0, defines the acceptable variability of traffic and performance inputs. The 1993 Design Guide recommends S_0 of 0.4-0.5 for flexible pavements. The design reliability level, R, must also be selected. The reliability factor (Z_R) is the area under a normal distribution curve for p $\leq$ R. The Design Guide demonstrates a detailed approach to identify an optimal level of reliability for a particular project based on total overall cost. The final initial input is design life, or analysis period.

Traffic Input

The AASHTO Guide uses a single parameter, Equivalent Single Axle Load (ESAL), to represent all traffic loading. ESALs are defined as equivalent moving applications of 18-kip single axles that cause an amount of serviceability loss (i.e., pavement damage) equal to the damage caused by the actual mixed axle load and axle configuration.

The number of ESALs can be calculated with Equation (3):

$$ESAL = AADT \times T_f \times T \times G \times D \times L \times 365 \times Y \tag{3}$$

where:

$AADT$	=	Annual Average Daily Traffic
T	=	Percentage of trucks
G	=	Traffic growth factor
D	=	Trucks in design direction (%)
L	=	Truck in design lane (%)
Y	=	Design period
T_f	=	Truck factor,

$$T_f = \sum_i (p_i \times LEF_i) \times A \tag{4}$$

p_i	=	Percentage of repetitions for i^{th} load group
LEF_i	=	Load Equivalency Factor (LEF) for the i^{th} load group
A	=	Average number of axles per truck

To compute Load Equivalency Factor (LEF) for each load group, agencies need to consider: (1) axle load, (2) axle configuration, (3) structural number, and (4) terminal serviceability. The 1993 AASHTO Design Guide, in Appendix D, provides LEFs for various combinations of axle load, axle configuration, structural number and terminal serviceability. The LEF equations are computationally cumbersome but can be roughly approximated by using a generalized fourth power formula, i.e., relating the axle load to an equivalent single axle load and raising the quantity to a power of four. For example, for a given SN = 3.0 and terminal serviceability index 2.5, the LEF for a 24,000-pound single axle can be approximated by:

$$(24{,}000 \text{ lb} \div 18{,}000 \text{ lb})^4 = 3.2$$

This is relatively close to the LEF of 3.1 calculated by the AASHTO equation.

Material Input

The basis for materials characterization is elastic resilient modulus (M_R). The resilient modulus is a measure of the elastic energy able to be absorbed by a given material when load is applied, and is known to have certain nonlinear characteristics. The 1993 AASTHO Guide recognized that agencies might not have equipment for performing the resilient modulus test to determineM_R, and provided correlation equations to estimate M_R from standard CBR, R-value, layer coefficient and other soil index test results. It is strongly recommended, however, that user agencies measure M_R directly or at least develop their own correlations based on regional conditions.

Environmental Input

Environmental effects are accounted for in two ways: (1) seasonally-adjusted subgrade resilient modulus and (2) drainage coefficient m_i.

The seasonally-adjusted subgrade resilient modulus, or effective resilient modulus, is an equivalent modulus which will cause the same damage to the pavement as if separate seasonal moduli were used. The average relative damage for all seasons is used to calculate the effective subgrade resilient modulus. The drainage coefficient measures the materials' permeability and the amount of time that the material is expected to be at or near saturation condition. The values are hard to obtain in reality because of the "near-saturation" condition requirement. The 1993 Guide provides recommendations for drainage coefficient values as a function of the quality of drainage and the percent of time during the year the pavement structure would normally be exposed to moisture levels approaching saturation for untreated base and subbase layers.

2.1.3 Limitations of AASHTO Design Guide

While the various AASHTO Design Guides have proven an important tool and served the industry well for several decades, its empirical approach limits its effectiveness as a modern pavement design method (ARA 2004). Specifically,

1. Modern traffic loads are much different than they were at 1950s,
2. Only one climate, one subgrade type, one hot-mix asphalt, and one PCC mixture were studied in the 1950s Road Test,
3. Rehabilitated pavements were not studied,
4. Drainage considerations were not tested, and
5. Test roads were monitored for only the first two years after opening to traffic.

Several studies have claimed that traffic is a controversial parameter in the 1993 AASHTO Guide. The fact that the guide relies on a single value (ESAL) to represent the overall traffic spectrum is questionable (Schwartz and Carvalho 2007). Zhang et al. (2000) have found that the ESAL, used to quantify damage equivalency in terms of serviceability or even deflections in the 1993 AASHTO Guide, is not enough to represent the complex failure modes of flexible pavements. Today it is widely accepted that load equivalency factor is not a sufficient technique for incorporating mixed traffic into design equations. Just after the development of AASHTO's 1986 Design Guide, NCHRP Project 1-26 initiated the push to develop mechanistic-empirical pavement design procedures (Li et al. 2011).

To address some of the limitations of its original design guide and meet the need for mechanistic-empirical design procedures, AASHTO in 2004 published NCHRP Report 1-37A, also called the Mechanistic-Empirical Pavement Design Guide (MEPDG) (NCHRP 2004). This new design procedure incorporates mechanistic principles, including calculations of pavement stress, strain, and deformation responses using site-specific climatic, material, and traffic characteristics. It replaces the 1993 guide's subjective-based performance index, PSI, with objective distress models for various modes of pavement failure and allows calibration of the distress models in order to allow the design method to represent each region's unique conditions.

2.2 The Mechanistic-Empirical Pavement Design Guide (MEPDG)

MEPDG is a state-of-the-art pavement design and analysis tool based on mechanistic-empirical principles. It differs significantly from the earlier AASHTO design procedures, which were based on empirical performance equations developed using the 1950's AASHO Road Test data.

MEPDG uses project specific traffic, climate, and materials data for mechanistically calculating pavement responses (stresses, strains, and deflections) and then applies those responses to empirical performance models to compute incremental damage (i.e., loss in rideability) over a specified pavement service life. Calibrated distress prediction models are used.

The MEPDG design process is iterative in nature. The first iteration requires an assumption of a trial pavement structure to produce initial performance predictions. If the output of distress predictions does not meet a user-specified acceptable level, the assumed pavement structure is modified and the MEPDG performance predictions are repeated until the structure satisfies user-specified performance criteria.

Detailed design procedures and inputs requirements will be summarized in this section.

2.2.1 Design Approach

The MEPDG design approach provides uniform guidance for designing flexible, rigid, and composite pavements. It is believed to be a more robust design system that considers more realistic characterization of modern in-service pavements (Li et al. 2011), including modern pavement materials, modern trucking and tire technologies, and environmental effects. As mentioned before, the MEPDG method calculates structural response (stresses, strains, and deflections) mechanistically based on material properties, environmental conditions, and traffic loading characteristics. These responses are subsequently treated as inputs in empirical models to estimate distress quantities over pavement design life.

The design approach consists of three major stages: (1) collect/develop input values, (2) analysis, and (3) evaluation. In stage 1, potential trial designs and all required inputs are identified and prepared. In stage 2, structural and performance analysis beginning with a selected trial design are conducted. Accumulated damage (i.e., amount of distress) and smoothness over time are output, and a structural design is obtained through an iterative process, which will be explained below in greater detail. In the final stage, evaluation of the structurally viable alternative is conducted. This can include life-cycle cost analysis and policy considerations.
The MEPDG uses an iterative process to predict pavement performances for a pavement structure and follows the following steps:

1) Select predefined trial pavement structure (specific site subgrade support, material properties).
2) Define design criteria for acceptable pavement performance at the end of the design period.
3) Prepare traffic inputs.
4) Prepare climate condition inputs.
5) Prepare material properties inputs.
6) Compute structure responses (stresses, strains, and deflections) for each axle type and load.
7) Calculate predicted distresses (e.g., rutting, fatigue cracking) using the calibrated empirical models and computed structure responses from step 7 as inputs.
8) Evaluate the predicted performance of the trial design from step 7 against defined design criteria from step 2. If the trial design does not meet the criteria, redefine the design and repeat the steps until the design meets the criteria.

2.2.2 Inputs

The MEPDG inputs include traffic characterization, material properties, and climatic data on three hierarchical input levels that are defined by the quality of data (ARA 2004). Generally, Level 1 provides the highest level of accuracy and Level 3 the lowest. The three level input structure allows designers the flexibility to scale their data collection effort according to available resources and project criticality. The hierarchical input levels are described below (NCHRP 2004):

Level 1 provides the highest level of accuracy. Level 1 inputs typically need to be measured directly (e.g., site specific and laboratory test data)

Level 2 provide an intermediate level of accuracy. Level 2 input parameters are estimated from empirical correlations with other parameters that are less costly to measure

Level 3 provide the lowest level of accuracy. Level 3 input parameters are based on default values which are the median or average value from a group of data with similar characteristics.

The hierarchical approach is employed with regard to traffic, material, and environmental inputs.

Traffic

MEPDG requires traffic data that do not incorporate the ESAL concept used in AASHTO 1993. MEPDG requires the full axle-load spectrum traffic inputs for estimating the magnitude, configuration, and frequency of traffic loading to accurately determine the axle loads that will exert damage on the pavement over the design life (Li, et al. 2011). Table 2.1 summarizes the traffic input parameters required for MEPDG.

Table 2.1 Traffic Input Parameters

Traffic Input Parameters	
Number of lanes in design direction	Vehicle class distribution factor
Design lane width	Axle load distribution
Annual average daily traffic (AADT)	Mean wheel location
Percent trucks in design direction	Traffic wander standard deviation
Percent trucks in design lane	Number of axle types per truck class
Operational speed	Axle configuration
Truck traffic growth factor	Wheelbase
Monthly distribution factor	Tire dimension and pressures
Hourly distribution factor	

Material

MEPDG requires a great deal of material property input for the climate, pavement response, and distress models. Li et al. (2011) summarized the general material input needs of each model: first, that the pavement response model characterizes layer behavior using material inputs such as moduli and Poisson's ratio. The pavement distress model, meanwhile, characterizes pavement structure strength (including shear strength and distress effects) using relevant material properties. Finally, the climate model requires material inputs often associated with special properties such as optimum moisture content. Table 2.2 summarizes required material properties for MEPDG flexible pavement analysis.

Table 2.2 Material Input Parameters

Material Input Parameters	
Asphalt Material	Dynamic modulus, new/reconstructed
	Dynamic modulus, rehabilitated
	Poisson's ratio
	Creep compliance
	Coefficient of thermal contraction
	Surface shortwave absorptivity
	Thermal conductivity
	Heat capacity
Chemically Stabilized Material	Elastic Modulus
	Flexural strength for design
	Poisson's Ratio
	Thermal conductivity
	Heat capacity
Unbound Material	Resilient modulus
	Poisson's ratio
	Plasticity index
	Gradation
	Maximum dry unit weight/ optimum moisture content
	Specific gravity of solids
	Saturated hydraulic conductivity
	Degree of saturation
	Coefficient of lateral pressure

Environmental

MEPDG uses the Enhanced Integrated Climatic Model (EICM) to predict environmental conditions. Detailed environmental data is required; however, most of the data can be obtained from weather stations, and the MEPDG software includes a library of climatic information from weather stations throughout the United States. Some additional information also needs to be collected by the user. Table 2.3 summarizes the environmental inputs for MEPDG.

Table 2.3 Environmental Input Parameters

Environmental Input Parameters	
Weather station data	Hourly air temperature
	Hourly precipitation
	Hourly humidity
	Hourly percentage sunshine (radiation and cloud cover)
	Hourly wind speed
Additional data	Water table depth
	Surface shortwave absorptivity
	Infiltration
	Drainage path length
	Pavement cross slope

2.2.3 Pavement Modeling

The MEPDG includes two major types of forecast models: (1) structural models, which represent the mechanistic part of the design procedure, and (2) performance models, which represent the empirical part. Structural models involve the application of engineering mechanics to calculate pavement structural responses under loads. They estimate stresses (σ), strains (δ), and deformation (ε) due to physical causes such as loads, environmental factors, and material properties, in order to model the propagation of pavement damage incrementally (due to repeated loads) over continuous time periods. Empirical performance models define the relationships between the calculated stresses, strains, and deflections and pavement failure based on performance observations from the field. Ride quality as quantified by the International Roughness Index (IRI) is the dominant characteristic of functional performance in MEPDG. The design procedure assumes that ride quality is dependent on the various modeled pavement distresses. Flexible pavement analysis in MEPDG uses three main distress prediction models: permanent deformation, fatigue cracking, and transverse cracking. They are listed here with their associated equations. A complete list and discussion of models can be referred to in the *MEPDG Manual of Practice* (AASHTO 2008).

Permanent Deformation (Rutting) Model

Permanent deformation, manifested as rutting in the wheel path that develops over time with each load repetition, is one major type of load-related distress. Overall rutting for a flexible pavement structure in MEPDG is expressed as the sum of permanent deformation for each individual layer:

$$\mathrm{RD} = \sum_{i=1}^{\#\ of\ sublayers} \varepsilon_p^i h^i \tag{5}$$

where:

RD = Total rut depth (total permanent deformation) (in)
ε_p^i = Accumulated plastic strain in sublayer *i*
h^i = Thickness of sublayer *i*

The cumulative rutting in asphalt mixture layers is given by Equation (6):

$$\varepsilon_p / \varepsilon_r = k_z \times 10^{-k_{1r}} N^{k_2 \beta_2} T^{k_3 \beta_3} \tag{6}$$

where:

ε_p = Accumulated plastic strain at *N* load repetitions (in/in)
ε_r = Resilient or elastic strain of the asphalt material
k_z = Depth factor to correct for the confining pressure at different depths,

$$k_z = (C_1 + C_2 * depth) * 0.328196^{depth} \tag{7}$$

$$C_1 = -0.1039(h_{ac})^2 + 2.4868 h_{ac} - 17.342 \tag{8}$$

$$C_2 = 0.0172(h_{ac})^2 - 1.7331 h_{ac} + 27.428 \tag{9}$$

h_{ac} = Total AC thickness (in)
N = Number of axle-load repetitions
T = Mix or pavement temperature (°F)
$k_{1r,2r,3r}$ = Global field calibration parameters (k_{1r} = -3.35412, k_{2r} = 0.4791, k_{3r} = 1.5606)
$\beta_{1r,2r,3r}$ = Local field calibration constants. For global calibration use 1

The cumulative rutting in unbound base and subgrade layers is given by Equation (10):

$$\delta_a(N) = \beta_{s1} k_{s1} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{N}\right)^{\beta}} \varepsilon_v h \tag{10}$$

where:

δ_a = Permanent deformation for the unbound layer (in)
N = Number of axle-load applications
ε_r, β, and ρ = Material properties
ε_r = Resilient strain imposed in laboratory test to obtain the above listed material properties, ε_r, β, and ρ (in/in)
ε_v = Average vertical resilient strain in the layer/sublayer, calculated by the structural response model (in/in)
h = Thickness of the unbound layer/sublayer (in)
β_{s1} = Local calibration factor for rutting in the unbound layer. For global calibration, it is 1.
k_{s1} = Global calibration coefficients. For granular materials use 1.673 and for fine-grained use 1.35.

Total rutting in the pavement structure can be expressed as the sum of the layer deformations (asphalt concrete, granular base, and subgrade), as shown below:

$$RD_{Total} = RD_{AC} + RD_{GB} + RD_{SG} \qquad (11)$$

Fatigue Cracking Model

Fatigue cracking in MEPDG includes damage propagating from the bottom of the pavement layer (i.e. bottom-up cracking) and from the top (i.e. top-down cracking). The number of load repetitions to fatigue cracking is calculated by Equation (12):

$$N_f = k_{f1} C C_h \beta_{f1} (\varepsilon_t)^{k_{f2}\beta_{f2}} (E)^{k_{f3}\beta_{f3}} \qquad (12)$$

where:

N_f = Number of axle-load applications to fatigue cracking
ε_t = Tensile strain at the critical location
E = Dynamic modulus of asphalt concrete measured in compression (psi)
$k_{f1,f2,f3}$ = Global field calibration parameters (0.007566, -3.9492, -1.281 respectively)
$\beta_{f1,f2,f3}$ = Local field calibration parameters. For global calibration, they are 1.
C = Laboratory to field adjustment factor, C=10^M. Where:

$$M=4.84\left(\frac{V_b}{V_b+V_a} - 0.69\right) \qquad (13)$$

V_b = Effective asphalt contents by volume, %
V_a = Percent air voids in the AC mixture, %
C_H = Thickness correction term, dependent on type of cracking:

$$C_{bottom-up,H} = \frac{1}{0.000398 + \frac{0.003602}{1+e^{(11.02-3.49H)}}} \qquad (14)$$

$$C_{top-down,H} = \frac{1}{0.01 + \frac{12.00}{1+e^{(15.676-2.8186H)}}} \qquad (15)$$

H = Total HMA thickness, inches

The allowable number of strain repetitions to failure criteria, N_f, is used in the cumulative damage index equation, which sums the incremental damage over time:

$$DI = \sum \left(\frac{n}{N_f}\right)_{j,m,l,p,T} \qquad (16)$$

where:

DI	=	Damage Index
N	=	Actual number of axle load applications within a given time period
Nf	=	Allowable number of strain repetitions to failure criteria
j	=	Axle load interval
m	=	Axle load type (single, tandem, tridem, quad, or special configuration)
l	=	Truck type (using the classification groups included in MEPDG)
p	=	Month
T	=	Median temperature for the five temperature intervals used to subdivide each month

Cumulative damage index is in turn used to predict the amount of alligator cracking area:

$$FC_{Bottom} = \left(\frac{1}{60}\right)\left(\frac{C_4}{1+e^{\left(C_1C_1^* + C_2C_2^*Log(DI_{Bottom}*100)\right)}}\right) \tag{17}$$

where:

FC_{Bottom}	=	Fatigue cracking at the bottom of the HMA layer
C_1, C_2, C_4	=	Regression coefficients; C_1 = 1.00, C_2 = 1.00, C_4 = 6,000
C_1^*	=	$-2C_2^*$
C_2^*	=	$-2.40874 - 39.748(1 + h_{HMA})^{-2.856}$
DI_{Bottom}	=	DI at the bottom of the HMA layers, as calculated above

Longitudinal cracking distresses are assumed to propagate from the top of the asphalt layer (i.e. top-down) and are modeled using the following equation:

$$FC_{Top} = 10.56\left(\frac{C_4}{1+e^{\left(C_1 - C_2Log(DI_{Top})\right)}}\right) \tag{18}$$

where:

FC_{Top}	=	Fatigue cracking at the top of the HMA layer
C_1, C_2, C_4	=	Regression coefficients; C_1 = 7.00, C_2 = 3.50, C_4 = 1,000
DI_{Bottom}	=	DI at the bottom of the HMA layers, as calculated above

Thermal Cracking Model

MEPDG models non-load-related transverse (thermal) cracking using the equation below:

$$TC = \beta_{t1}N\left[\frac{1}{\sigma_d}Log\left(\frac{C_d}{h_{HMA}}\right)\right] \tag{19}$$

where:

TC	=	Observed amount of thermal cracking (ft/mi)
β_{t1}	=	Regression coefficient determined through field validation
$N(z)$	=	Standard normal deviation evaluated at (z).
σ_d	=	Standard deviation of the log of the depth of cracks in the pavement
C_d	=	Crack depth (in)
h_{HMA}	=	Thickness of surface layer (in)

The crack propagation model expresses the increase in crack depth induced by a given thermal cooling cycle:

$$\Delta C = A\Delta K^n \tag{20}$$

where:

ΔC	=	Change in crack depth due to a cooling cycle
ΔK	=	Change in the stress intensity factor due to a cooling cycle
A,n	=	Fracture parameters for the asphalt mixture

Fracture parameters *A* and *N* can be estimated using indirect tensile creep compliance and HMA strength:

$$A = 10^{k_t \beta_t (4.389 - 2.52 Log(E \sigma_m n))} \quad (21)$$

where:

A	=	Fracture parameter
n	=	$0.8\left[1 + \frac{1}{m}\right]$
k_t	=	Coefficient determined through field calibration for each input level (Level 1 = 5.0, Level 2 = 1.5, Level 3 = 3.0)
β_t	=	Local or mixture calibration factor
E	=	Asphalt modulus
σ_m	=	Mixture tensile strength (psi)

Stress intensity factor *K* is calculated as:

$$K = \sigma_{tip}(0.45 + 1.99(C_o)^{0.56}) \quad (22)$$

where:

σ_{tip}	=	Far-field stress from response model at depth of crack tip (psi)
C_o	=	Current crack length (ft)

Smoothness (IRI) Model

Past literature (Ayres and Witczak 1998, Carey and Irick 1990, NCHRP 1990) has shown smoothness to be able to be modeled accurately using pavement distresses such as those included in the MEPDG distress models. MEPDG characterizes smoothness in terms of IRI using the equation described below:

$$IRI = IRI_O + 0.015SF + 0.4FC_{total} + 0.008TC + 40RD \quad (23)$$

where:

IRI_O	=	Initial IRI after construction (in/mi)
SF	=	Site Factor,

$$SF = AGE\,(0.02003(PI+1) + 0.007947(PRECIP+1) + 0.000636(FI+1)) \quad (24)$$

AGE	=	Pavement age (yr)
PI	=	Percent plasticity index of the soil
FI	=	Average annual freezing index (°F days)
PRECIP	=	Average annual precipitation (in)
FC_{total}	=	Area of fatigue cracking (percent total lane area)
TC	=	Length of transverse cracking (ft/mi)
RD	=	Average rut depth (in)

The main benefit of MEPDG is that it is based on pavement fatigue and deformation characteristics of all layers, rather than solely on pavement surface condition (Quintus, and Moulthrop, 2007). As part of the ongoing push to implement such advanced pavement design procedures, several states have undertaken research activities on calibration to ensure the models' validity and accuracy.

3. LOCAL CALIBRATION OF MEPDG

As described in the previous section, MEPDG uses mechanistic processes to model pavement structural responses and subsequently applies empirically derived transfer functions to these response predictions in order to forecast pavement distresses. These forecasted distresses, measured throughout pavement design life, are the key output of MEPDG analysis. The performance models include designated calibration coefficients, which must be adjusted to reflect the impact of local material, traffic, and climatic conditions in order to predict rutting, cracking, and faulting distress mechanisms with confidence (Kim et al. 2010).

The critical nature of accurate distress predictions as key MEPDG outputs makes calibration an essential step of any agency's MEPDG implementation process. The default MEPDG transfer functions were initially calibrated in NCHRP 1-37A (2003) and later recalibrated in NCHRP 1-40D (2009). These national calibration efforts were based on the design information and observed distresses included in the nationwide Long-Term Pavement Performance Program (LTPP) data set. These projects will henceforth be collectively referred to as national calibration.

Although the nationally calibrated model represents a valuable resource, enormous variance often exists between local and national averages in terms of climatic conditions, material properties, construction techniques, geography, traffic patterns, maintenance activities, and various pavement design variables. These differences make it necessary for each state to undertake its own calibration of MEPDG distress models using local or regional design inputs and distress data in order to reflect the unique pavement needs for that state.

The goal of local calibration is to reduce the standard error and bias associated with the design predictions used for local conditions. Calibration is conducted by comparing MEPDG-predicted pavement performance with actual performance data. Transfer function calibration factors are adjusted to minimize the difference between observed and predicted distresses. Numerous states have conducted their own research projects to customize the MEPDG models to specific regional conditions and pavement design standards. These local calibration studies include work in North Carolina (Kim et al. 2007), Montana (Von Quintus and Moulthrop 2007), Texas (Banerjee et al. 2008), Utah (Darter, Titus-Glover, and Von 2009), Arizona (Souliman 2009), Minnesota (Velasquez et al. 2009), Arkansas (Hall, Xiao and Wang 2011), Washington (Li et al. 2011), Missouri (Schroer 2012), New Mexico (Tarefder et al. 2012), Iowa (Ceylan et al. 2013), Oregon (Williams and Shaidur 2013), Tennessee (Zhou et al. 2013), Michigan, Ohio and Wisconsin (Kang and Adam 2007).

This section will outline the suggested steps for local calibration as outlined by NCHRP (2009) and will discuss the considerations necessary for proper use of pavement management system (PMS) data in calibration.

3.1 Steps for Local Calibration

A national guideline for local calibration was developed by NCHRP under Project 1-40B and published by AASHTO as the Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide, henceforth referred to as the local calibration guide. This project suggests an 11-step procedure for local calibration, which has subsequently been applied in state calibration projects (Williams and Shaidur 2013). The steps are outlined in the sections to follow. It can be helpful to visualize the calibration steps as a flowchart, as illustrated in Figures 3.1 and 3.2 (NCHRP 2010).

3.1.1 Step 1: Select Hierarchical Input Level for Each Input Parameter

MEPDG's hierarchical approach to design inputs allows the procedure to be adapted to the level of data that are available based on an agency's data collection policies, equipment, personnel, and test facilities. As previously discussed, hierarchical input levels typically involve a tradeoff between more accurate analysis results at Level 1 and ease of data collection at Level 3. The input levels selected for the calibration process should be consistent with the levels that the agency will ultimately use for pavement design (NCHRP 2010).

3.1.2 Step 2: Develop Local Experimental Plan and Sampling Template

Calibration should account for all possible local conditions, materials, and design types by including a robust and statistically sound sampling plan. NCHRP recommends a fractional factorial sampling matrix with the primary tier including distress-dependent parameters such as pavement type, surface layer thickness, and subgrade type. The secondary tier can include pavement type dependent parameters such as temperature, moisture, and traffic.

In order to overcome condition-dependent bias and/or standard error, effort should be made to fill as many cells of the sampling matrix as possible using available data. Some agencies have undertaken calibration efforts with limited and/or unbalanced sampling plans (NCHRP 2010).

3.1.3 Step 3: Estimate Sample Size for Each Distress Prediction Model

In this step, the user determines the number of sample segments required to eliminate bias and standard error for each individual distress model.

Sample size must be adequate to validate both bias and precision in the performance prediction models. The required number of model runs (roadway segments) can be expressed in the following equation.

$$\frac{s_e}{s_y} \geq \left[\frac{x_\alpha^2}{n-1}\right]^{0.5} \tag{25}$$

where:

s_e / s_y	=	Relative error standard deviation
x_α^2	=	Chi-square statistic for level of significance α
n	=	Number of model evaluations, i.e., sample size

A level of significance – 75%, 90%, or 95% – is selected for each distress model in order to determine the relative error deviation. NCHRP recommends a practical value of 90%. NCHRP also offers suggestions, based on LTPP data, for the minimum number of roadway segments that should be used for each performance model:

- Total rutting: 20 segments
- Load-related cracking (e.g., alligator cracking): 30 segments
- Non-load-related cracking (e.g., thermal cracking): 26 segments
- Reflection cracking (for HMA overlays): 26 segments

3.1.4 Step 4: Select Roadway Segments

Sample segments should be selected based on available data in order to satisfy the requirements of the defined sampling matrix while minimizing field data collection costs. NCHRP recommends using roadway segments from either of two categories for calibration and validation of distress models:

Pavement Management System (PMS) segments and long-term research test sites, e.g., LTPP segments. Most segments used for national calibration in NCHRP Project 1-40D were LTPP test segments, however, many states will rely on PMS data to build a calibration sample set, which includes the full range of local conditions, materials, and structures.

Selection of roadway segments should consider not only the requirements of the defined factorial matrix but also a number of other factors outlined in the *Guide for Local Calibration*:

- Segments should be structurally simple, i.e., include the fewest number of structural layers, to minimize the amount of data collection required for material characterization.
- Segments with and without overlays should be selected in order to represent both new/reconstructed and rehabilitated pavements. Segments with condition data before and after overlay can serve both purposes.
- If non-conventional pavement layers or mixes are used in the state, they should be represented in the sample segments in order to properly calibrate MEPDG distress models, as the LTPP segments used for national calibration typically included only conventional HMA and PCC mixtures.

Selected roadway segments should include a minimum of three distress surveys over at least a 10-year period in order to adequately capture time-dependent material responses and the propagation of distress mechanisms. Roadway segments should each include a similar number of distress observations over 10-year period.

3.1.5 Step 5: Extract and Evaluate Roadway Segment Data

In this step, both distress and project data are collected, extracted, compiled and evaluated to determine its adequacy and completeness for use in MEPDG calibration and validation.

The first part of this step involves extracting and converting condition data to the format used by MEPDG. This includes a review of the data to ensure that distress data are collected and recorded in a manner consistent with MEPDG distress predictions (which in turn follow the format of the LTPP database from which MEPDG was calibrated). If data are not consistent with MEPDG format, distress data can be collected or existing distress data can be used and adjusted.

Once distress data are extracted, maximum recorded distresses must be compared against agency design criteria to ensure that measured distresses are meeting or exceeding the maximum allowable levels set by the agency before rehabilitation or reconstruction is undertaken. If maximum measured distresses are consistently below allowable limits, it may be difficult to minimize error and bias of the performance models later in the calibration process.

It is important to check for outliers, unusual trends, or other potential errors in distress data. This verification can consist of a visual inspection or a more detailed statistical evaluation. Zero values should be inspected to determine whether they represent missing data, maintenance, or rehabilitation/reconstruction. Other unusual values should be inspected and removed if they cannot be explained.

Once distress data have been reduced as described above, project data for the remaining road segments must be collected. This information can be obtained from a variety of sources, including but not limited to construction records, quality acceptance (QA) test records, and PMS data. Missing project data must be identified to prepare for the field investigations in the next step.

For PMS segments, NCHRP recommends the application of nondestructive testing, including ground penetrating radar (GPR) and falling weight deflectometer (FWD) to verify in-service pavement layer thickness and layer moduli. This is to reduce input error, a component of the total error term (the minimization of which is one of the goals of calibration).

3.1.6 Step 6: Conduct Field and Forensic Investigations

In this step, the user must first develop a data collection plan to satisfy the data needs identified in the previous step. In keeping with the recommendation in Step 1 to calibrate for the hierarchical input levels that will be used for pavement design, agencies should follow their own data collection protocols for pavement evaluation on rehabilitation projects.

The second process in this step requires a decision by the calibrating agency whether to accept the assumptions built into the MEPDG distress models. These include the share of total rutting attributable to each layer as well as the location of surface layer crack initiation. Because of the limited distress data contained in the LTPP database that was used for MEPDG development and calibration, certain key assumptions had to be made regarding distress observations.

First, MEPDG calculates percentage share (rather than directly predicting absolute quantity) of total rutting that each pavement layer can be expected to contribute to total rutting at the surface layer. These percentages are then multiplied by the forecasted total rutting to obtain layer-by-layer rutting predictions. Second, MEPDG assumes that alligator cracking is the result of bottom-up cracking alone, and that longitudinal cracking occurs from cracks propagating from the top of pavement. At the time of MEPDG development and national calibration, the mechanism for top-down cracking was not well understood and so its implementation in the Design Guide is somewhat limited compared with the more traditional bottom-up cracking in that the same distress model and calibration coefficients were used for both distress types.

If these assumptions are acceptable to the calibrating agency, distress outputs for the calibration effort should be limited to total rutting and total load-related (bottom-up plus top-down) cracking. In this case, no forensic investigation is required.

If the agency decides to reject or verify the distress assumptions, forensic testing will need to be conducted. For rutting, trenches or test pits should be used to measure rutting in each pavement layer. For location of crack initiation, cores should be extracted from areas of pavement with load-related cracking and crack properties (location of initiation, direction of propagation, width at initiation point, and depth from initiation point) should be reported.

Before proceeding to the next step in the calibration process, the user must at this point verify the number of roadway segments remaining with all required data for calibration-validation and add additional segments if too many have been removed through the data reduction process.

3.1.7 Step 7: Assess Local Bias from Global Calibration Values

At this point, MEPDG simulation is conducted for each roadway segment using national calibration factors to generate distress outputs. These performance predictions are compared to field distress observations to determine bias and standard error for each performance model.

The null hypothesis, that there is no significant difference between measured and predicted performance for a given confidence level, is tested for the entire sample set. Significance can be determined using a paired t-test. The null hypothesis is described in the equation below, where $y_{Measured}$ represents measured values and $x_{Predicted}$ represents predicted values for each performance model:

$$H_0 : \sum_{i=1}^{n} (y_{Measured} - x_{Predicted})_i = 0 \tag{26}$$

NCHRP recommends plotting a comparison of predicted and measured values for each distress model to get a clearer picture of their relation to the line of equality.

Two other model parameters, slope and intercept, should be used to evaluate model bias. This is to prevent a situation in which the residual error shows no significance, i.e., the paired t-test fails to reject the null hypothesis, but bias remains in the performance model(s). The following fitted linear regression model can be used

$$\hat{y}_i = b_0 + m(x_i) \tag{27}$$

where:

$\hat{y}_i$	=	Mean measured value
b_0	=	Intercept
m	=	Slope
x_i	=	Predicted value

The following hypothesis tests are then applied to the slope and intercept of the regression model:

$$H_0 : b_0 = 0$$
$$H_0 : m = 1$$

If any of the three null hypotheses proposed in this step are rejected (i.e., there is significant difference between measured and predicted performance), the performance model(s) in question should be recalibrated (see Step 8). If, for all three hypothesis tests, the user cannot find evidence to reject the null hypothesis for the selected level of significance, then standard error for the local data set should be compared against the global standard errors that are provided with the MEPDG.

3.1.8 Step 8: Eliminate Local Bias of Distress and IRI Prediction Models

This step focuses on eliminating any significant local bias which resulted from using national calibration factors in the previous step.

MEPDG includes two sets of calibration parameters for most distress model transfer functions: agency-specific parameters and local calibration parameters. Results from NCHRP's national calibration effort are entered by default to the agency-specific parameters. The default value for local calibration factors is 1. The calibrating agency, must then decide which set of parameters to adjust in order to eliminate bias in the performance models.

NCHRP Progect 1-40B (2009) includes a list of coefficients that should be considered for eliminating bias and reducing standard error for each distress type. See Table 3.1.

Table 3.1 Calibration Parameters by Distress Type

Distress		**Eliminate Bias**	**Reduce Standard Error**
Total Rutting	Unbound Materials and HMA Layers	k_1, β_{s1}, or β_{r1}	k_2, k_3, and β_{r2}, β_{r3}
Load-Related Cracking	Alligator Cracking	C_2 or k_1	k_2, k_3, and C_1
	Longitudinal Cracking	C_2 or k_1	k_2, k_3, and C_1
	Semi-Rigid Pavements	C_2 or β_{c1}	C_1, C_2, C_4
Non-Load-Related Cracking	Transverse Cracking	β_{t3}	β_{t3}
IRI		C_4	C_1, C_2, C_3

The NCHRP Report goes on to describe three possible types of bias and the ways in which they can be addressed, listed below:

1. **Reasonable precision, poor accuracy**: The residual errors are, for the most part, always positive or negative with a low standard error of the estimate in comparison to the trigger value, and the slope of the residual errors versus predicted values is relatively constant and close to zero. The precision of the prediction model is reasonable but the accuracy is poor (large bias). In this case, the local calibration coefficient is used to reduce the bias. This condition generally requires the least level of effort and the fewest number of runs or iterations of the MEPDG to reduce the bias.
2. **Reasonable accuracy, poor precision**: The bias is low and relatively constant with time or number of loading cycles, but the residual errors have a wide dispersion varying from positive to negative values. The accuracy of the prediction model is reasonable, but the precision is poor. In this case, the coefficient of the prediction equation is used to reduce the bias but the value of the local calibration coefficient is probably dependent on some site feature, material property, and/or design feature included in the sampling template. This condition generally requires more runs and a higher level of effort to reduce the bias.
3. **Poor accuracy and precision**: The residual errors versus the predicted values exhibit a significant and variable slope that appears to be dependent on the predicted value. The precision of the prediction model is poor and the accuracy is time- or number-of-loading-cycles-dependent – there is poor correlation between the predicted and measured values. This condition is the most difficult to evaluate because the exponent of the number of loading cycles needs to be considered. This condition also requires the highest level of effort and many more runs to reduce the bias.

3.1.9 Step 9: Assess the Standard Error of the Estimate

After reducing or eliminating the bias for each of the distress models, the standard error for the local calibration is compared and evaluated against the standard error from the global calibration. Standard error of the estimate for global calibration are included in the *MEPDG Manual of Practice* (AASHTO 2008) and in the MEPDG software.

The standard error of the estimate for each transfer function is to be evaluated, with the null hypothesis that there is no significant difference between the standard error for each calibration effort. If there exists sufficient evidence to reject the null hypothesis, it can be concluded at the selected confidence level that there is a significant difference between the global and local calibration standard error terms. In that case, if the local calibration has a higher standard error, the agency should recalibrate local calibration coefficients; see Step 10.

If the null hypothesis is rejected but the local calibration has a standard error lower than the global calibration, or if there is not enough evidence to reject the null hypothesis at the selected level of confidence, the current calibration coefficients can be used for design. The agency should proceed to Step 11.

3.1.10 Step 10: Reduce the Standard Error of the Estimate

Before recalibration of local distress coefficients, the components of the MEPDG standard error term must be quantified to determine the extent to which a reduction in the lack-of-fit error component will improve model performance. Lack-of-fit is the only error which can be reduced through local calibration. If this component accounts for a small portion of the total error, the calibrating agency must decide whether the cost of additional calibration will be worth the improved precision of performance prediction models.

Standard error should be computed for each block of the sampling template (or each combination of climatic/material/traffic/policy characteristics applicable to the sample set) to verify whether the error term is dependent on any of the sample parameters. If the standard error shows correlation to any sample parameters, local calibration coefficients should be adjusted for each type of correlated parameters.

At this point, the local calibration coefficients can be adjusted to reduce the standard error, referring back to the tables in section 3.1.8 to determine which parameters to adjust for each transfer function. The calibration coefficients that yield the lowest standard error of the estimate for each distress model should be used for design.

3.1.11 Step 11: Interpretation of Results, Deciding on Adequacy of Calibration Parameters

Finally, the standard error for each performance model is evaluated for each structure or rehabilitation type and at several different reliability levels. The agency has three options at this point:

1. Forecasted design life is determined to be “reasonable,” i.e., resulting designs are not overly conservative, based on distress observations. Reasonableness can be defined by comparing reliability (i.e., probability that the segment will not fail in the predicted design lift) against probability of failure curves developed from historic pavement data. If this conclusion is selected, the agency concludes the calibration process by entering the new local calibration coefficients and standard error of estimate into the MEPDG software.
2. The design life forecast is too short, resulting in overly conservative designs for the reliability levels used by the agency. In this case, the agency should return to Step 10, focusing on reducing standard error of the estimate for the controlling performance model.
3. The design life forecast is too short because the measurement error and pure error components of total calibration error are too high, resulting in overly conservative designs for the reliability levels used by the agency. If this is the case, little can be done to improve the standard error of the estimate. The agency may consider relaxing failure criteria and concluding the calibration process by entering the new local calibration coefficients and standard error estimate into the MEPDG software.

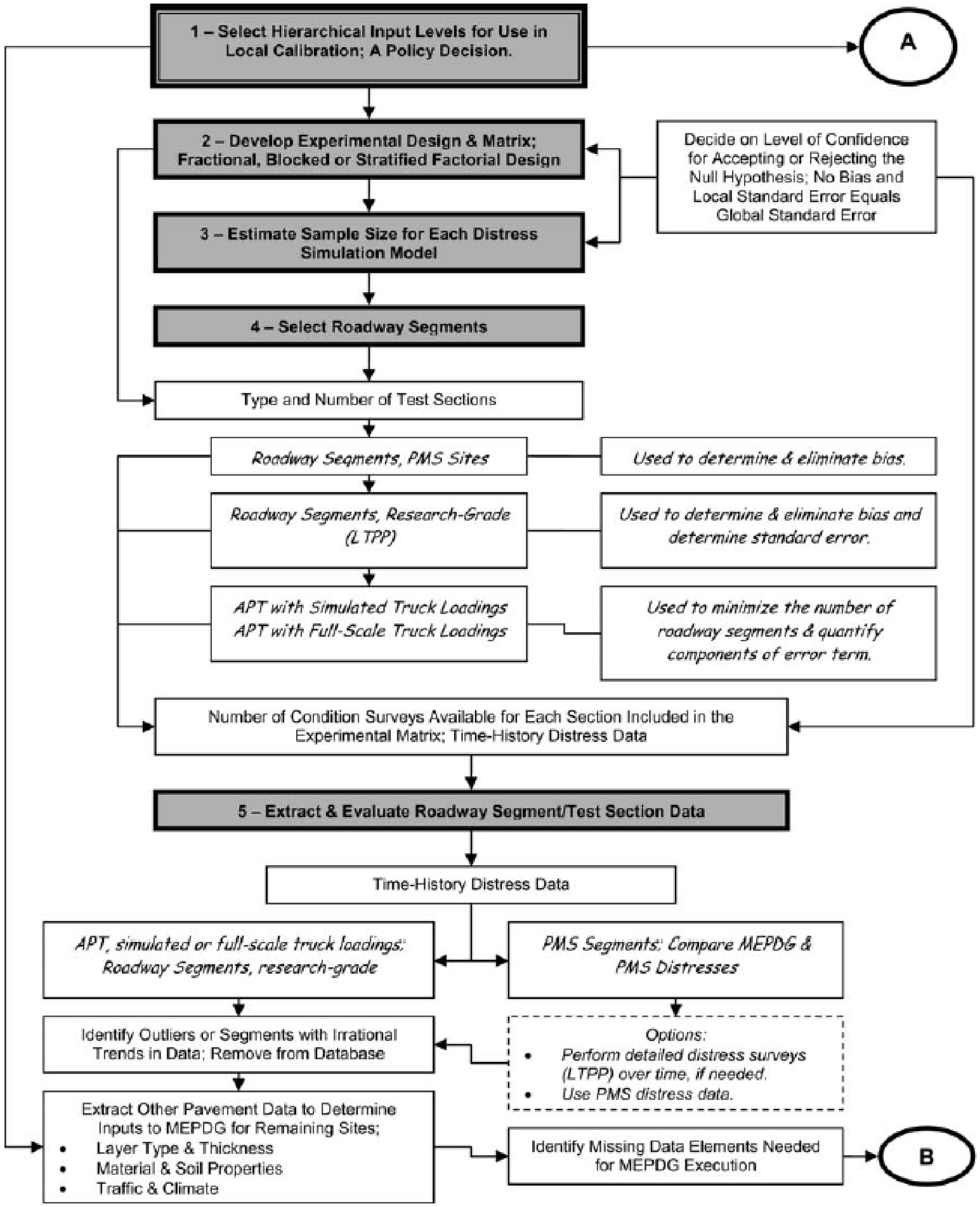


Figure 3.1 Local Calibration Process, Part 1 (NCHRP 2009)

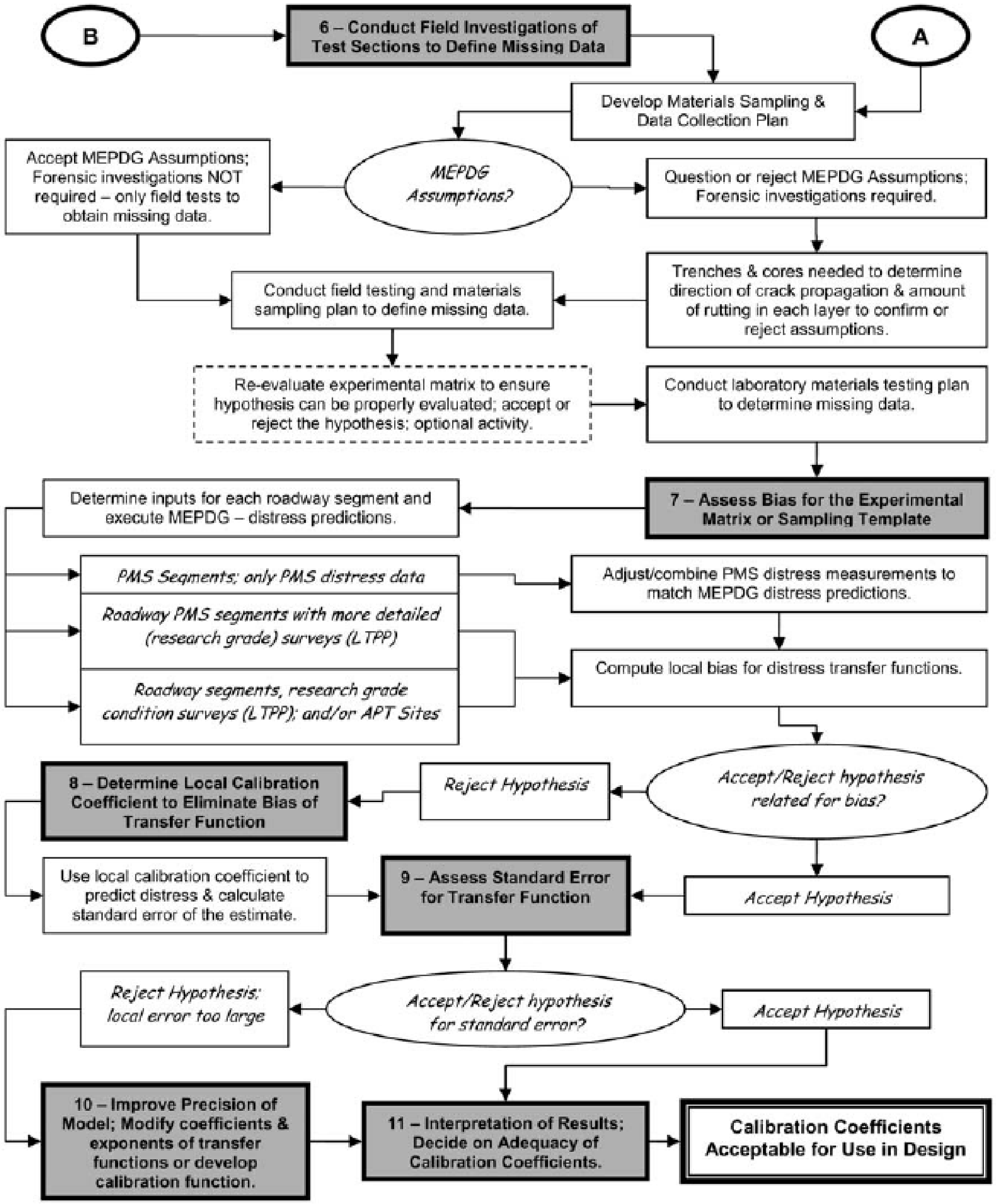


Figure 3.2 Local Calibration Process, Part 2 (NCHRP 2009)

3.2 Challenges of Using Pavement Management System (PMS) Data for MEPDG

MEPDG was calibrated using data from the LTPP program, but for most states, PMS data represent the most complete source of historical pavement data (Pierce 2011). PMS databases, however, typically do not include all the data necessary for MEPDG calibration. Further, the data provided are often formatted in a way that isn't directly compatible with MEPDG model input.

Various agencies that have completed the MEPDG calibration/validation process have identified the following challenges which should be considered when considering using PMS data in an MEPDG calibration effort.

3.2.1 Project Data

A PMS can be a good source for basic project information, including design properties and basic project identification information and historical IRI. Limitations may exist, however, regarding the extent and nature of historical distress data. MEPDG includes the following distress outputs and corresponding units for flexible pavement design:

- Top-down fatigue cracking (ft/mile)
- Bottom-up fatigue cracking (percent lane area)
- Thermal cracking (ft/mile)
- Permanent deformation (i.e., rutting) – total pavement (in.)
- Permanent deformation – asphalt layer (in.)

Many states do not collect these particular distress observations or measure them in different units (Hudson et al. 2008, NCHRP 2009, Pierce et al. 2011, Ceylan et al. 2013, Williams and Shaidur 2013). North Dakota, for example, quantifies distress extent and severity based on a 100-point pavement condition rating deduct scale. Distresses are assigned a score that corresponds to a range of distress quantities and a given severity (low, medium, or high). When this is the case, a state may choose to undertake a multi-year implementation plan in which MEPDG-compliant distress data are collected. This of course involves significant time and cost and may not be a viable option when resources are scarce. Alternatively, distress values may be adjusted or modified to meet MEPDG calibration requirements. Maintenance history may be incomplete or entirely excluded from PMS data, but may be stored instead in a separate maintenance management system or construction history database.

3.2.2 Traffic Data

MEPDG requires a variety of detailed traffic data, which are described in the preceding sections of this document. The majority of these input parameters fall outside the extents of a typical pavement management system. States should turn to weigh in motion data, traffic counts, and regional assumptions for missing data.

3.2.3 Pavement Structure

Detailed layer thicknesses are often unavailable or inaccurate. In these cases, coring or nondestructive testing effort is recommended to collect complete and accurate structural details.

3.2.4 Materials Data

While state PMSs may include limited material characterization, Level 1 and 2 MEPDG material inputs require test data which is more detailed than that of a traditional PMS (FHWA 2011). Agencies should review construction records, field coring history, as-built plans, and material QC data to achieve the highest possible hierarchical input level for these data. A data collection plan, including coring, nondestructive testing, and field testing, can be designed to fill the remaining holes in required inputs.

4. SENSITIVITY ANALYSIS

Numerous studies have investigated the MEPDG distress models' sensitivity to pavement design inputs since the release of the Guide's software (Version 0.7) in 2004. This literature review will focus on sensitivity analyses for flexible pavement design, with a focus on recent sensitivity literature, including NCHRP 1-47 (Schwartz 2011).

4.1 Early Calibration Research

Numerous changes to the flexible and rigid pavement analysis models in NCHRP Project 1-40D (2006) were incorporated into MEPDG Version 1.0 in 2007. The NCHRP update and recalibration project introduced changes to the climatic model, the flexible and rigid pavement design procedures, and several design inputs. A Version 1.1 release in 2009 updated the rehabilitation analysis process and distress prediction models. The latest software release at the time of this research, Pavement ME Design Version 1.3.29, uses MEPDG Version 1.1.

The numerous changes to MEPDG performance models from 2004-2011, particularly the major 1.0 update, resulted in a need for updated sensitivity analyses using the most recent design software. For flexible pavements, these include studies by Buch et al. (2008), Khazanovich et al. (2008), Aguiar-Moya et al. (2009), Ahn et al. (2009), Schwartz (2009), Thyagarajan et al. (2010), Velaszuez et al. (2009), Ayyala et al. (2010), Kim et al. (2010), Hall et al. (2010), Schwartz and Li (2010), Yin et al. (2010), Schwartz et al. (2011), and Buch et al. (2013). These studies have investigated a wide range of design inputs and their significance to the MEPDG distress models. Their findings have been disparate, but some trends do emerge when reviewing the breadth of literature.

Surface layer properties in particular have been a focus of several studies. Some have shown sensitivity of distress models to asphalt mix volumetric properties (Buch et al. 2008, Khazanovich et al. 2008, Thyagarajan et al. 2010), which represent Level 2/3 inputs to the Witczak dynamic modulus predictive equation. Witczak's model, however, has been shown to have limited accuracy in predicting asphalt dynamic modulus master curve (Thyagarajan et al. 2010, Kim et al. 2010, Schwartz et al. 2011), resulting in significant performance differences between analyses using Level 1 vs. Level 2/3 (i.e., predictive) dynamic modulus inputs. For this reason, some have recommended using Level 1 master curve test data when possible.

Others (Yin et al. 2010, Kim et al. 2010) have studied the differences in performance predictions related to hierarchical input level for asphalt property creep compliance. In these studies, the thermal cracking model has demonstrated sensitivity to creep compliance input level, suggesting potential limitations of the MEPDG creep compliance predictive model used for Level 2/3 data.

Pavement structure is, of course, another important factor in accurate performance prediction. Buch et al. (2008) and Aguiar-Moya et al. (2009) investigated variations in surface and base layer thicknesses and reported significant effects on MEPDGs predicted performance for rutting and cracking models. These studies emphasized the importance of accurate thickness data, such as that obtained by ground-penetrating radar (GPR) in flexible pavement rehabilitation design.

Since the initial publication of MEPDG in 2004, the guide's handling of material stiffness properties has been a topic of repeated discussion. Studies which have investigated subgrade resilient modulus have reported mixed results (Schwartz and Li 2010, Kim et al. 2010). Some have reported some impact of subgrade stiffness on performance predictions while other research suggests MEPDG distress has little or no sensitivity to this input. Kim et al. also demonstrated no significant impact of subgrade modulus input level on cracking, rutting, or IRI. This seems counterintuitive considering the generally stress-dependent nature of subgrade stiffness and the stress-independent nature of Level 2/3 MEPDG inputs, but the findings are echoed in several studies (Hoerner et al. 2007, McCracken et al. 2008, Kim et al. 2005). Traffic inputs volume and axle load distribution were shown by Ahn et al. (2009) and Schwartz and Li (2010) to have significant impact on all distress models. MEPDG's fatigue cracking model was demonstrated by Ahn et al. to be particularly sensitive to volume (AADTT). Further, detailed axle load distribution data of the kind obtainable from weigh-in-motion (WIM) stations were shown to improve performance prediction accuracy over the use of MEPDG default axle load distributions.

4.2 NCHRP Report 1-47

NCHRP Report 1-47 (Schwartz et al. 2011) was unique among sensitivity studies up to that point. This report reviewed recent MEPDG sensitivity research and identified several key problems common to existing literature. These include the subjectivity of engineering judgment in selecting design inputs to be tested, the limited range of values typically tested in a limited one-at-a-time (OAT) analysis, the failure to investigate potential correlations and/or interactions between input parameters, and the use of outdated MEPDG software.

To avoid the limitations identified above, Schwartz et al. undertook a detailed two-part sensitivity analysis involving an initial OAT analysis followed by a thorough global sensitivity analysis (GSA). An initial triage effort identified 36 key input parameters and ranges to be used in the OAT analysis. This first analysis followed a methodology similar to previous sensitivity research by modifying one variable at a time along a limited range of realistic values and measuring the resulting sensitivity of MEPDG distress models.

The subsequent GSA allowed inputs to be varied simultaneously across their entire problem domain (rather than adjusting one-at-a-time on a set of pre-selected values), making it possible to look for interactions between design inputs. This type of analysis is much more computationally intensive than a traditional OAT analysis and required over 41,000 MEPDG runs for the five pavement types, five climatic regions, and three traffic levels tested.

Sensitivity results in NCHRP 1-47 were reported in terms of a Normalized Sensitivity Index (NSI), which relates a given percentage change in a design input to a corresponding percentage change in predicted distress relative to its defined design limit. At NSI = 1, in other words, a 1% change in design input would result in a 1% change in predicted distress. OAT results were sorted into four categorized based on NSI output:

- Hypersensitive (NSI > 5)
- Very Sensitive (1 < NSI < 5)
- Sensitive (0.1 < NSI < 1)
- Insensitive (NSI < 0.1)

For new flexible pavements, the hypersensitive category included asphalt dynamic modulus (E*) master curve parameters Alpha and Delta (i.e., the basis for the Witczak predictive model) and asphalt layer thickness. This matches the findings of several earlier projects described above.

Global sensitivity results closely matched the findings of the OAT analysis. Design inputs were categorized in this analysis based on mean NSI plus/minus two standard deviations. Category cutoffs were the same as the OAT analysis. Results showed almost complete agreement between the two analyses. Table 4.1 shows flexible pavement design inputs, which fell into the top two sensitivity categories. Sensitivity dropped sharply at each category boundary – from Hypersensitive to Very Sensitive and again from Very Sensitive to Sensitive. As a result, the design inputs that do not fall into the two categories listed in Table 4.1 are predicted to have little practical significance in MEPDG's flexible pavement models.

Table 4.1 NCHRP 1-47 Global Sensitivity Analysis (GSA) Results

Hypersensitive	HMA E* Alpha Parameter
	HMA E* Delta Parameter
	HMA Thickness
Very Sensitive	HMA Creep Compliance m Exponent
	Base Resilient Modulus
	Surface Shortwave Absorptivity
	HMA Air Voids
	HMA Poisson's Ratio
	Truck Volume (AADTT)
	HMA Effective Binder Volume
	Subgrade Resilient Modulus
	Base Thickness
	Subgrade Percent Passing No. 200 Sieve
	HMA Tensile Strength at 14°F
	Operational Speed
	HMA Creep Compliance D Parameter

4.3 Buch et al. 2013

A more recent study published by Michigan DOT (Buch et al. 2013), as part of its MEPDG implementation effort, followed a methodology similar to NCHRP 1-47. The study included a two-part analysis, including an initial OAT and GSA, but focused solely on inputs relevant for rehabilitation design. Based on MDOT practice, the study investigated four rehabilitation types and several material, condition, and structural design inputs. MEPDG version 1.1 was used.

The detailed OAT analysis used inputs identified through previous literature and a preliminary sensitivity analysis as significant for new (i.e., overlay) and existing pavements in rehabilitation design. The analysis varied these inputs over the entire range of potential values for the state of Michigan. Results from this analysis were subsequently used as inputs for the final GSA. GSA results were reported in units of NSI, similar to the NCHRP 1-47 study.

Results showed significant sensitivity of cracking models to percent air voids in new asphalt. Existing asphalt thickness, effective binder content, unbound layer moduli, and existing pavement condition all demonstrated significance on longitudinal cracking predictions. Rutting and IRI showed little sensitivity to the selected inputs. The study ranks design inputs according to NSI as shown in Table 4.2. Results confirm that layer thicknesses and asphalt properties are critical design inputs and should be a primary focus of MEPDG data collection efforts.

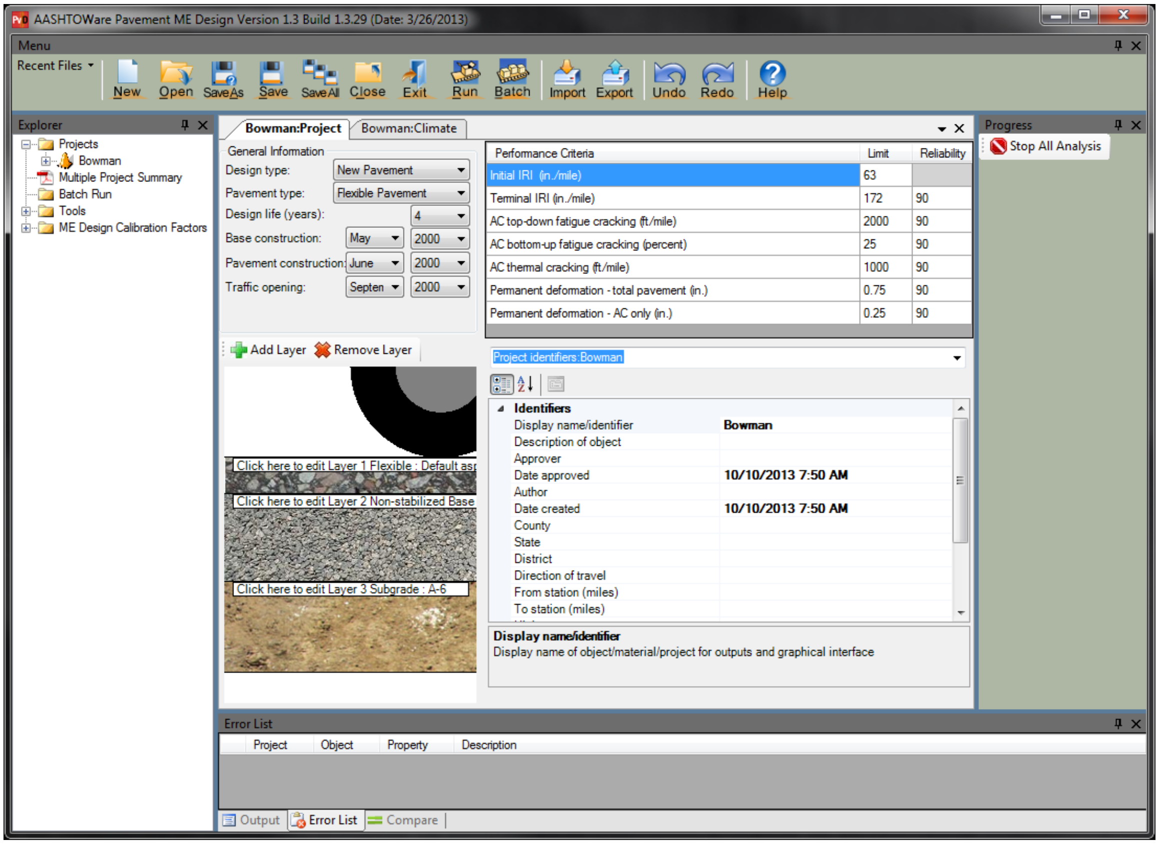
Table 4.2 Buch et al. GSA Results

Input variable	Rank	NSI
Overlay air voids	1	6
Existing thickness	2	5
Overlay thickness	3	4
Existing pavement condition rating	4	4
Overlay effective binder	5	2
Subgrade modulus	6	2
Subbase modulus	7	1

Existing literature has identified a number of key design inputs for MEPDG which can help agencies form a targeted data collection plan as part of an MEPDG implementation. The most comprehensive analysis project, NCHRP 1-47, generally supported the findings of previous research that asphalt dynamic modulus is highly significant and have a strong influence on distress predictions. Slightly less significant are surface and base layer thicknesses and base and subgrade moduli. Sensitivity analysis have also reported that sensitivity tends to drop sharply beyond a few highly sensitive variables. This supports the conclusion that states should follow a data collection plan that allows cost-effective MEPDG analysis by focusing their limited funding on the most sensitive design inputs.

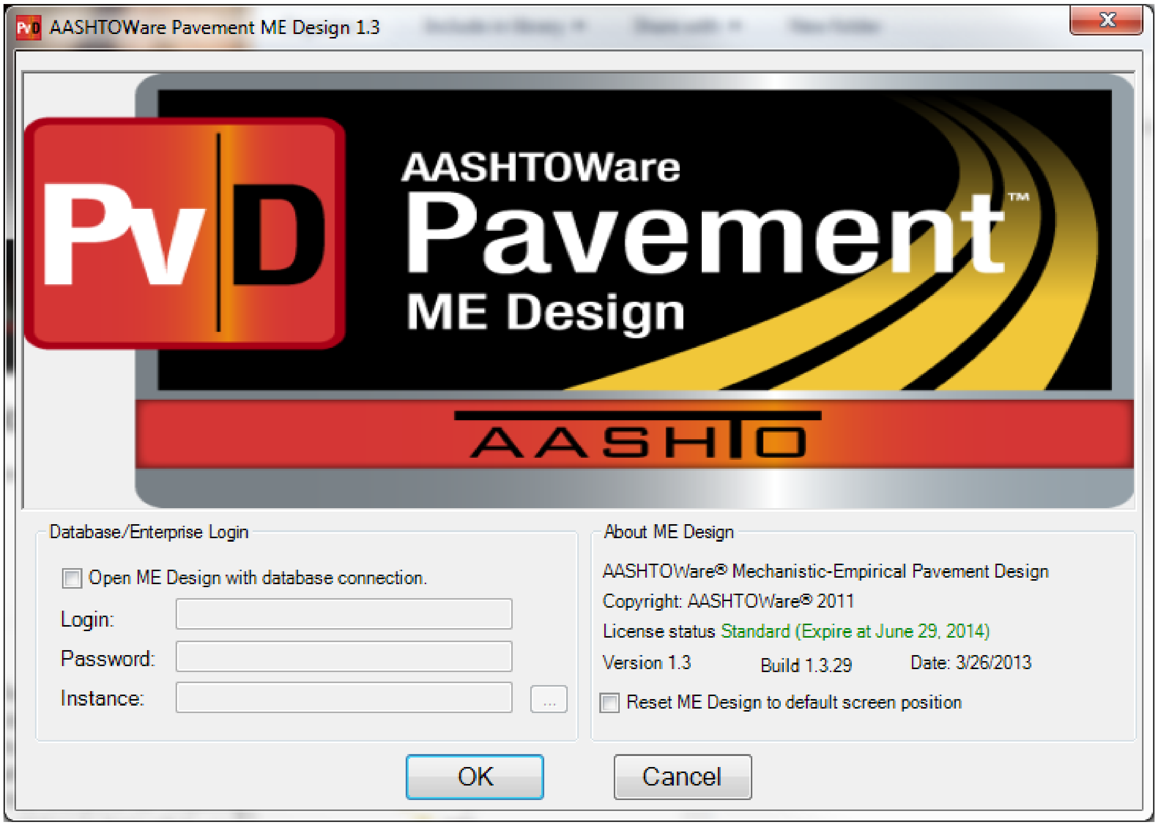
5. AASHTOWARE PAVEMENT ME DESIGN SOFTWARE

MEPDG design software, ME Design (formerly DARWin-ME), has undergone numerous significant updates since its initial release in 2004, not only with respect to the analysis engine but also the user interface. The following section is based on the most recent (as of this writing) release of AASHTOWare Pavement ME Design software, build 1.3.29, released March 2013. This assumes an existing software installation and active license. The documentation included with the software installation is, of course, a very thorough source of information, however, its current iteration is over 1,500 pages. This section is instead intended to help the new user get acquainted with the ME Design user interface and the general workflow of a new flexible pavement project. It assumes local calibration has been completed and will not discuss the adjustment of calibration parameters.

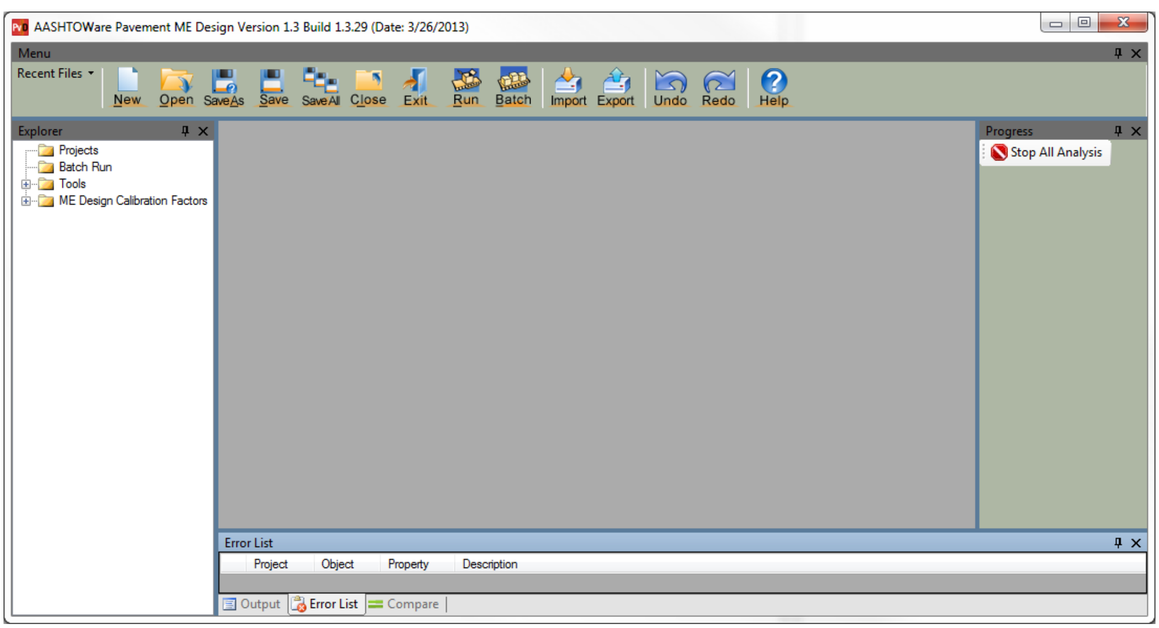


5.1 Open Project

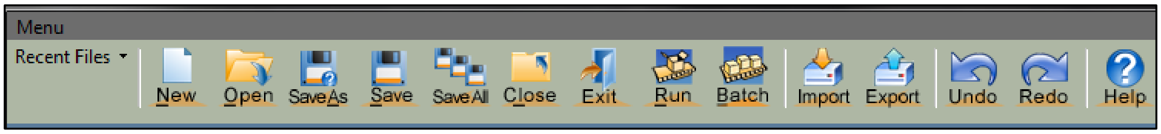
Upon launching ME Design, the following splash screen appears. This screen contains version and license information as well as database login options that can be used to interface with enterprise systems.



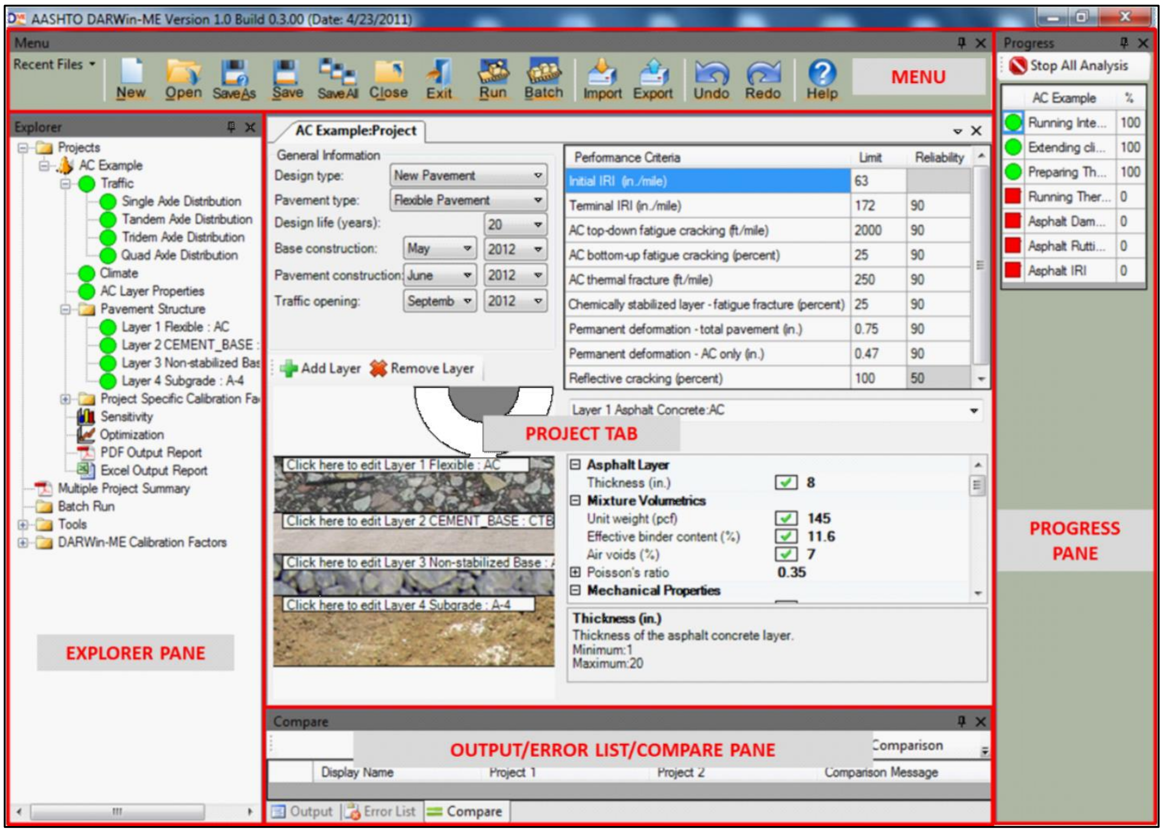
Selecting **OK** from the splash screen opens the ME Design main window.



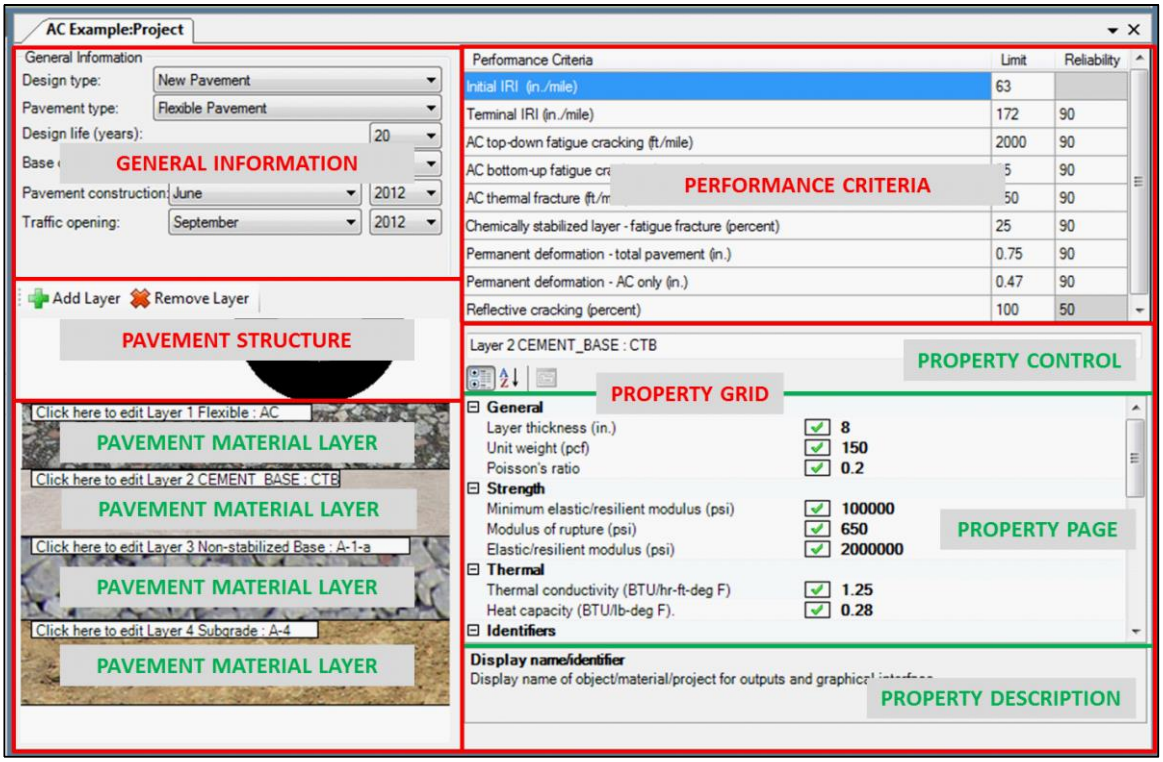
Selecting **New** from the Menu pane at the top of the window will open a new Project tab with blank inputs.



The following figure outlines the five distinct sections of the ME Design Main Window which will appear upon opening or starting a project. Each section will be discussed in the sections to follow.

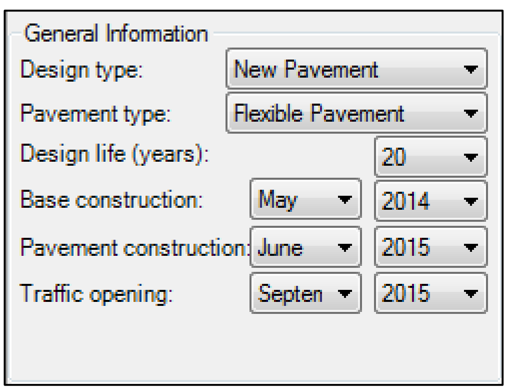


The Project tab is where most project-specific design inputs will be entered. It can be further broken down into four general areas. Those areas are outlined in the following figure.



The next step in this new flexible pavement design example is to define basic project information in the General Information pane.

5.2 General Information



The General Information pane contains the general inputs design type (i.e., new, overlay, or reconstruction), pavement type (flexible, rigid, or composite), design life, and relevant construction/opening dates.

The example described herein will involve a new flexible pavement project. From the General Information pane, select:

- **Design type**: New Pavement
- **Pavement type**: Flexible Pavement

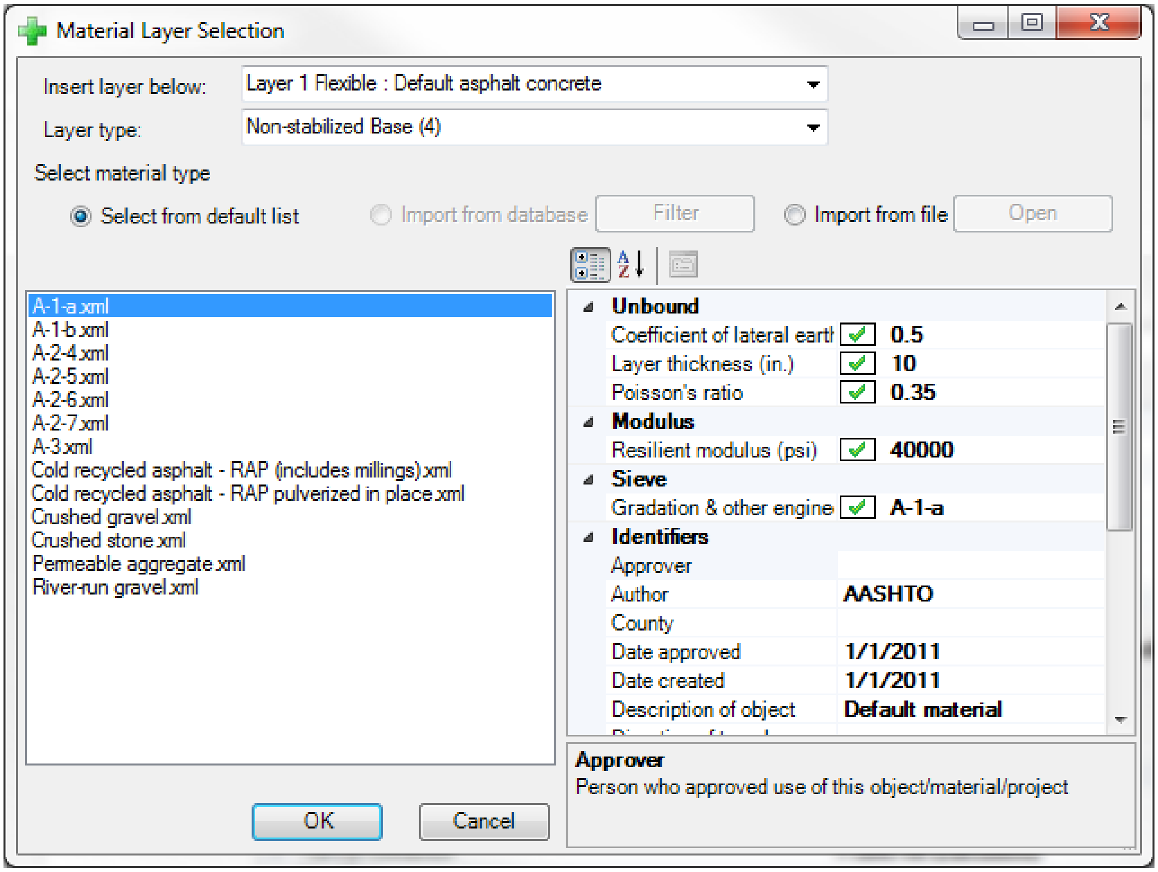
5.3 Performance Criteria

Performance Criteria	Limit	Reliability
Initial IRI (in./mile)	63	
Terminal IRI (in./mile)	172	90
AC top-down fatigue cracking (ft/mile)	2000	90
AC bottom-up fatigue cracking (percent)	25	90
AC thermal cracking (ft/mile)	1000	90
Permanent deformation - total pavement (in.)	0.75	90
Permanent deformation - AC only (in.)	0.25	90

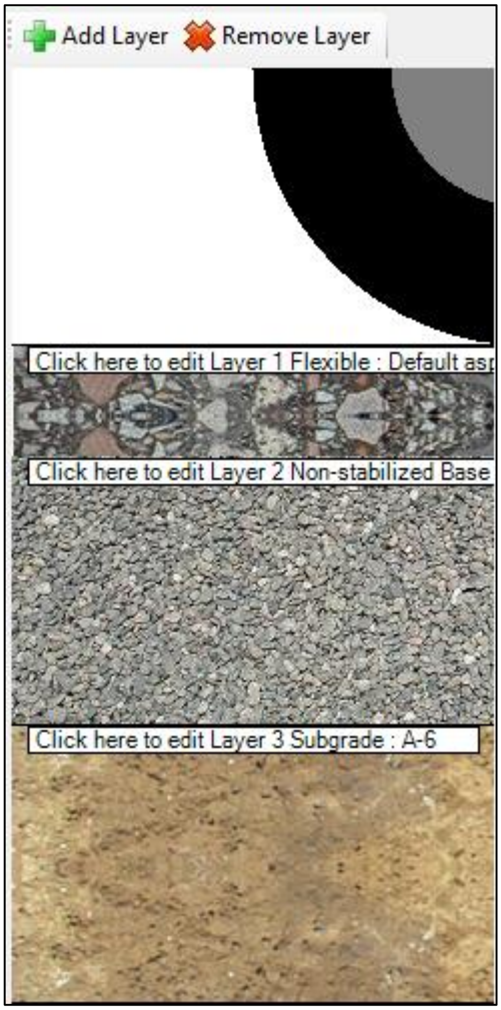
The Performance Criteria pane allows the definition of agency-specific allowable limits for distresses and smoothness (i.e., IRI), and the associated reliability levels for each criterion. These represent the basis for the acceptance or rejection of a pavement trial design. The distress types included in the performance criteria represent the MEPDG-modeled distresses described earlier in this document.

5.4 Pavement Structure and Material

Upon starting a new flexible pavement project, the Pavement Structure pane will include only one pavement layer. Use the **Add Layer** button to open the Material Layer Selection dialog (shown below). This dialog allows the user to define new layer(s), including position (relative to other layers), layer type, and material type. ME Design includes a number of pre-loaded material type templates, which can be customized from this dialog or inserted as is and customized later. Customized material types from other project files can also be imported at this point.

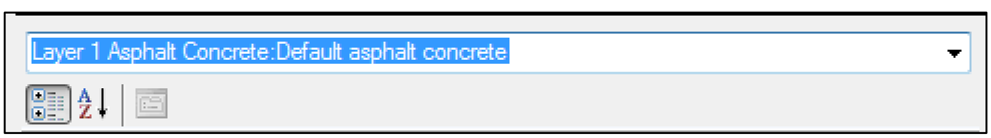


After selecting a layer type, position, and material type, select **OK** to create the layer. This will add the layer to the diagram in the Pavement Structure pane. Continue the process to build the desired pavement structure. See below.

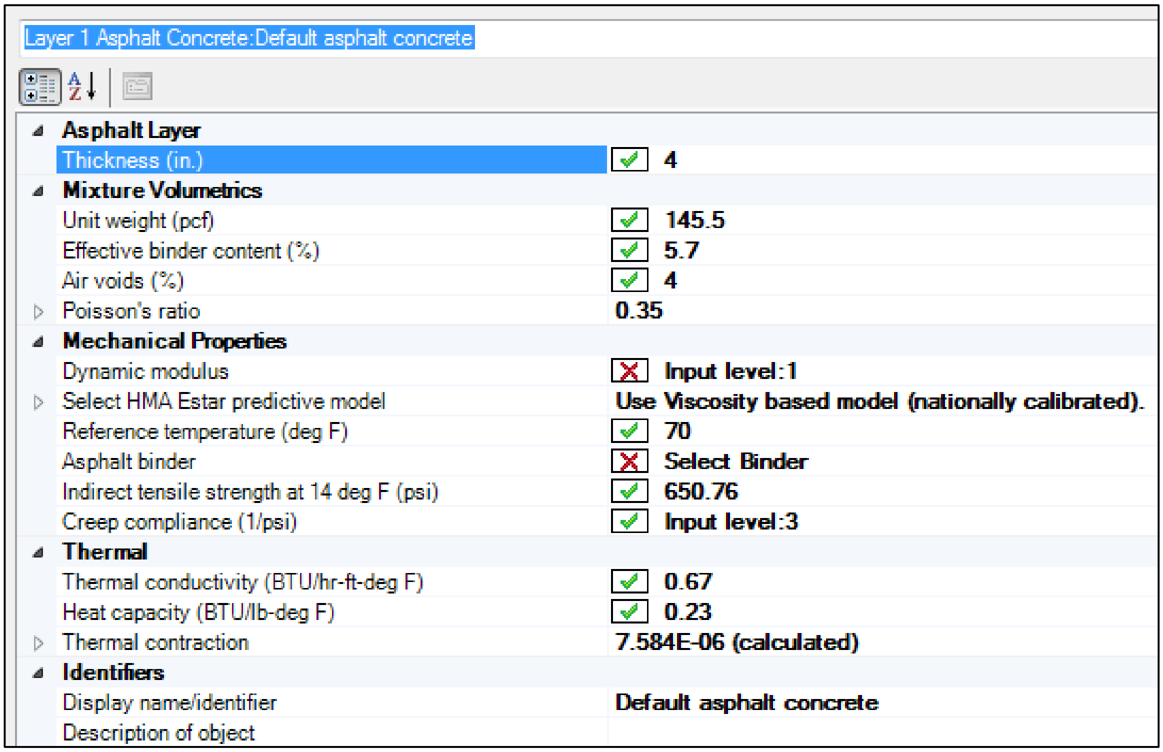


5.4.1 Property Grid

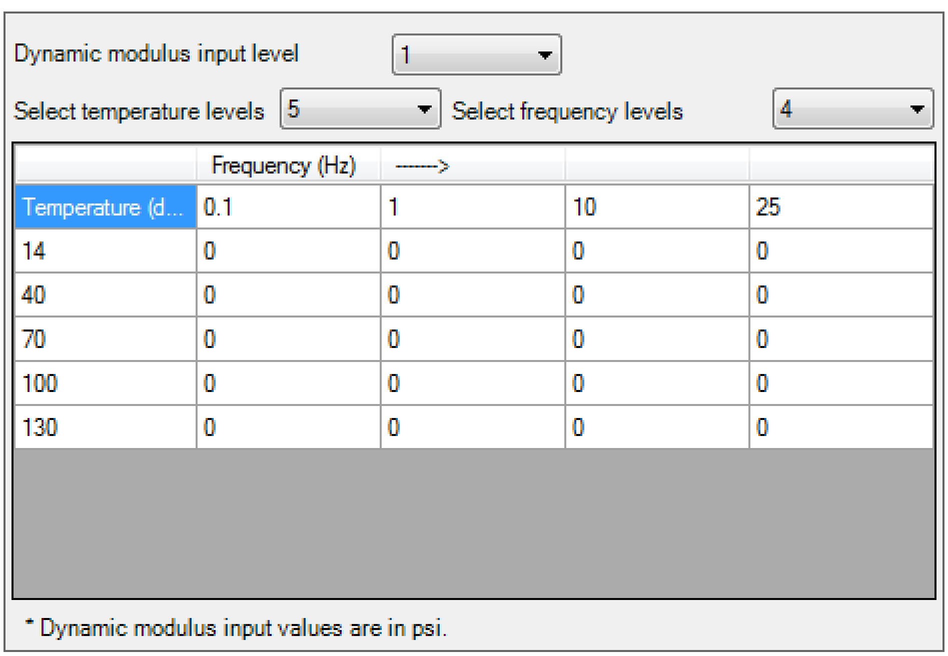
Once the desired structure has been established, it is necessary to define layer-specific material inputs. Open the asphalt concrete layer in the Property Grid by either left-clicking the layer in the Pavement Structure pane or by selecting it from the Property Grid dropdown menu:



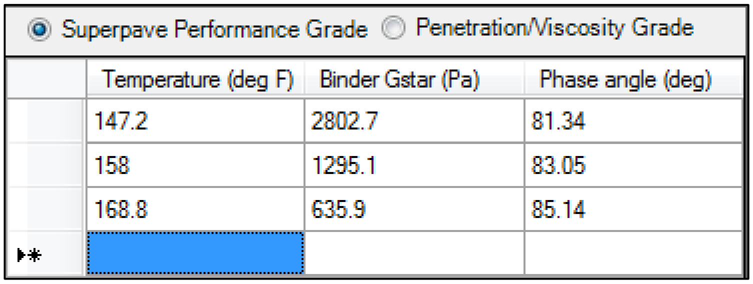
The Property Grid dropdown menu allows the user to select several input parameter sets, including layer properties, project-specific calibration factors, and project identifiers. This example will focus on the entry of layer properties which would be required for a new flexible pavement design.



Asphalt layer input parameters are shown in the figure above. Note that the dynamic modulus parameter, which has been demonstrated to have significant impact on performance predictions (see Section 4 in this document), requires different inputs at its different hierarchical input levels. At Level 1, MEPDG dynamic modulus requires actual laboratory data at several temperatures and frequencies in order to construct a master curve for the asphalt concrete. Levels 2 and 3 require only gradation test results for the asphalt mixture.

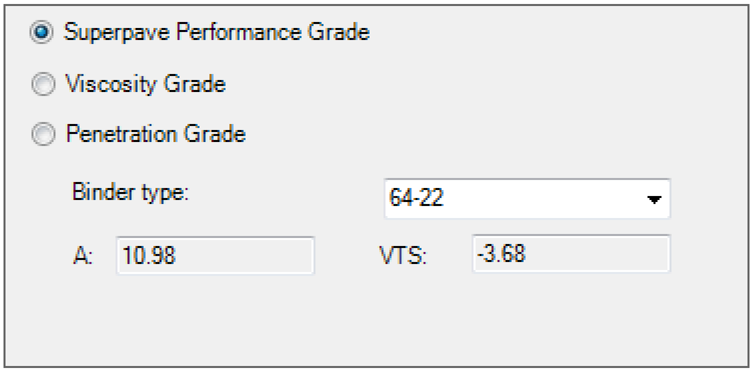


Binder inputs are dependent on the input Level selection for asphalt dynamic modulus. At Levels 1 and 2, ME Design requires binder properties in the form of laboratory test data for either binder grading system, Superpave or Penetration/Viscosity Grade.



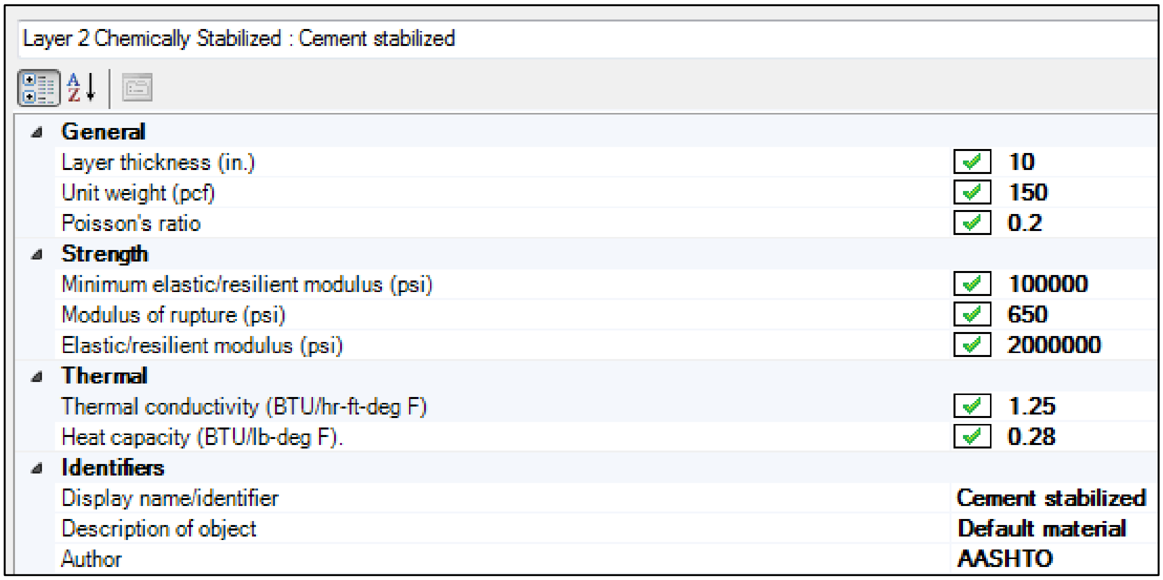
Temperature (deg F)	Binder Gstar (Pa)	Phase angle (deg)
147.2	2802.7	81.34
158	1295.1	83.05
168.8	635.9	85.14

At Level 3 dynamic modulus, the user need only select (1) binder grading system and (2) binder grade.



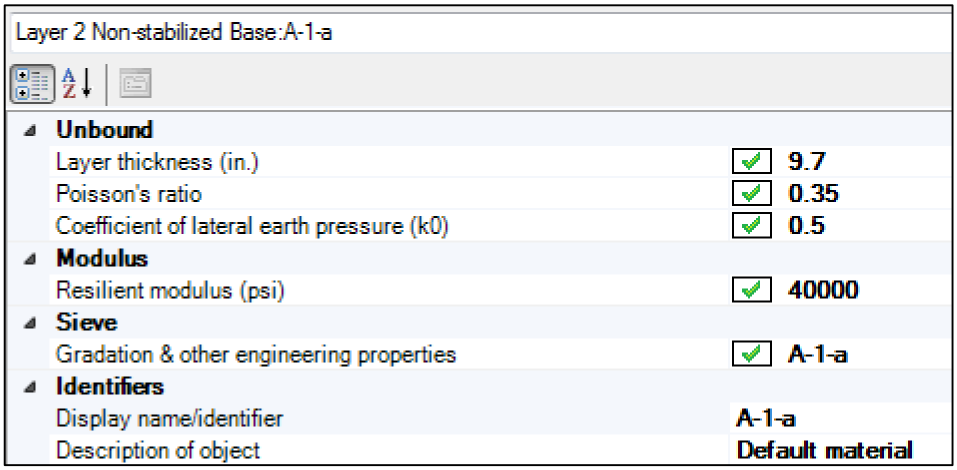
Creep compliance inputs are also level-dependent. At Level 1, laboratory test results are required at temperatures -4, 14, and 32°F. At Level 2, laboratory test results are only required at 14°F. At the lowest level, ME Design internally calculates creep compliance at various temperatures and loading times based on relationships with other inputs.

After entering surface layer data, continue through the Property Grid dropdown menu, defining the necessary input data for each layer. See below for chemically stabilized, unbound base, and subgrade layer properties.



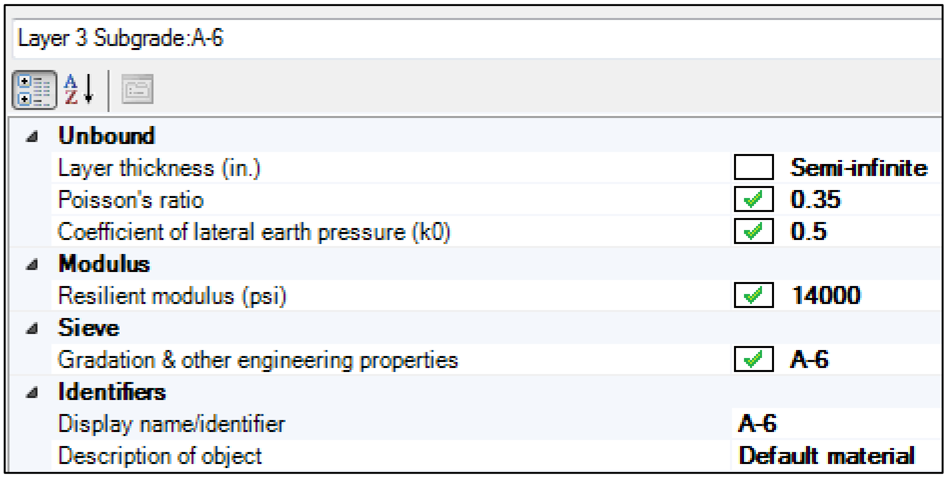
Layer 2 Chemically Stabilized : Cement stabilized

General	
Layer thickness (in.)	10
Unit weight (pcf)	150
Poisson's ratio	0.2
Strength	
Minimum elastic/resilient modulus (psi)	100000
Modulus of rupture (psi)	650
Elastic/resilient modulus (psi)	2000000
Thermal	
Thermal conductivity (BTU/hr-ft-deg F)	1.25
Heat capacity (BTU/lb-deg F).	0.28
Identifiers	
Display name/identifier	Cement stabilized
Description of object	Default material
Author	AASHTO



Layer 2 Non-stabilized Base:A-1-a

Unbound	
Layer thickness (in.)	9.7
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	40000
Sieve	
Gradation & other engineering properties	A-1-a
Identifiers	
Display name/identifier	A-1-a
Description of object	Default material

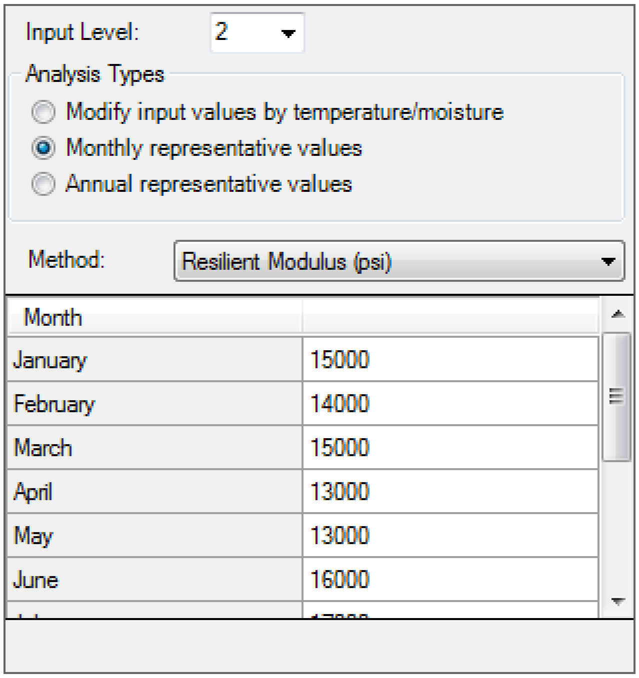


Layer 3 Subgrade:A-6

Unbound	
Layer thickness (in.)	Semi-infinite
Poisson's ratio	0.35
Coefficient of lateral earth pressure (k0)	0.5
Modulus	
Resilient modulus (psi)	14000
Sieve	
Gradation & other engineering properties	A-6
Identifiers	
Display name/identifier	A-6
Description of object	Default material

ME Design does not provide Level 1 resilient modulus input for non-stabilized material. For unbound base and subgrade layers, resilient modulus must be defined based on Level 2 or 3 criteria:

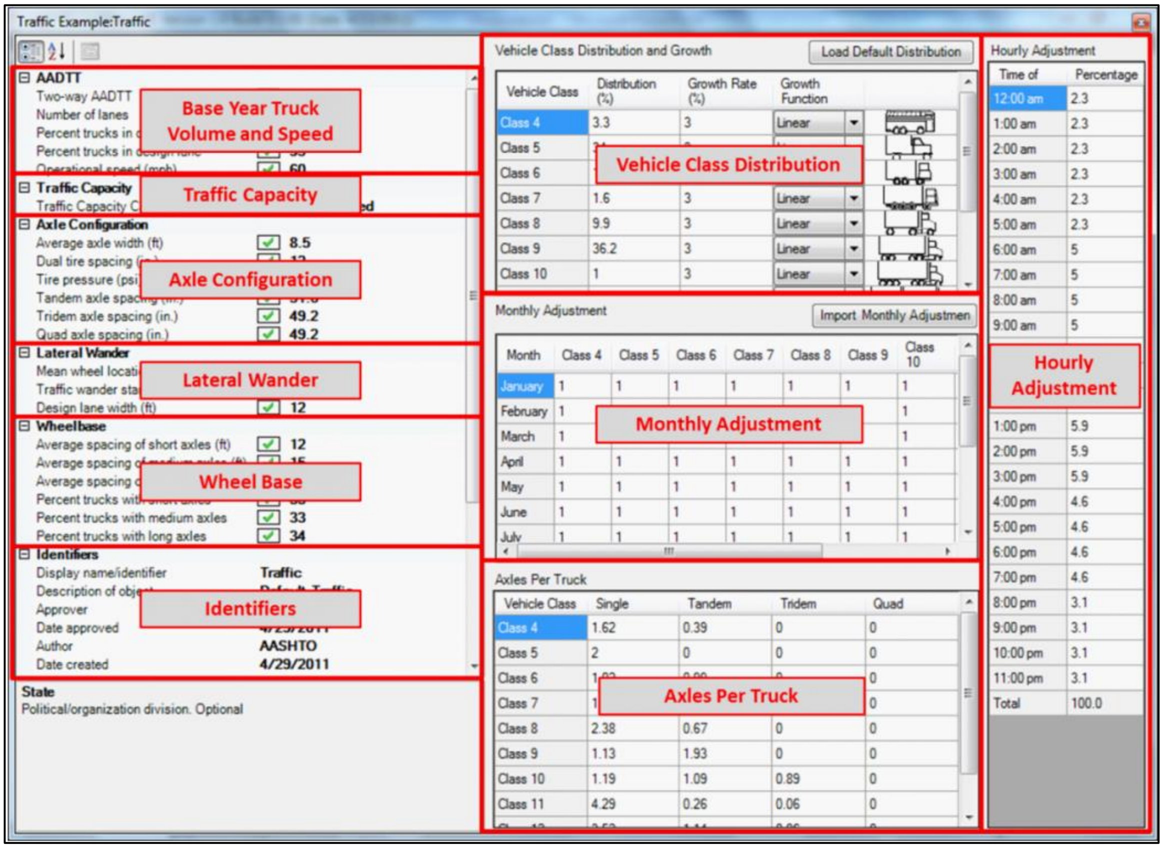
- At Level 2, resilient modulus can be defined directly or using correlations with other soil properties. Level 2 input also supports the use of monthly average resilient modulus values. See figure below.
- At Level 3, default resilient modulus of the material can be overwritten manually.



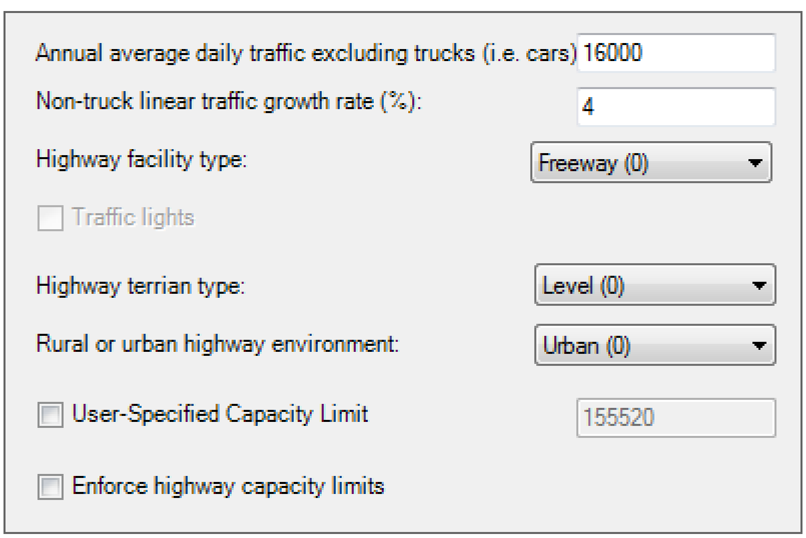
5.5 Traffic

Traffic data in ME Design are accessed either by left-clicking the tire icon in the Pavement Structure Definition area or via the Explorer tab > Project tree > Traffic node. This opens the Traffic tab, which includes many of the traffic inputs described earlier in this document. These inputs can be divided into the following general sections, as shown in the following figure.

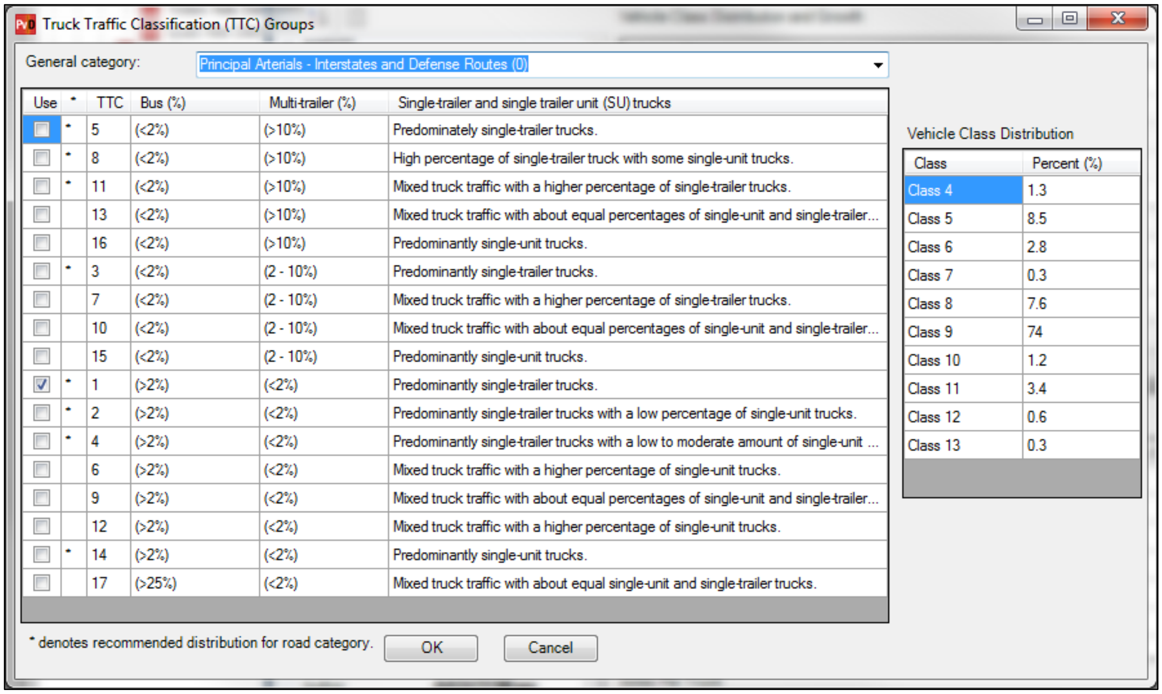
- Base Year Truck Volume and Speed
- Traffic Capacity
- Axle Configuration
- Lateral Wander
- Wheel Base
- Vehicle Class Distribution
- Axles Per Truck
- Monthly Adjustment
- Hourly Adjustment
- Identifiers



Traffic capacity is an optional setting which allows a cap on forecasted traffic volume based on ME Design's internal Highway Capacity Manual (HCM) 2000 capacity calculations. This prevents unrealistically high volumes from negatively impacting predicted pavement performance. The following figure shows the capacity input dialog, which contains the necessary inputs for HCM calculations.



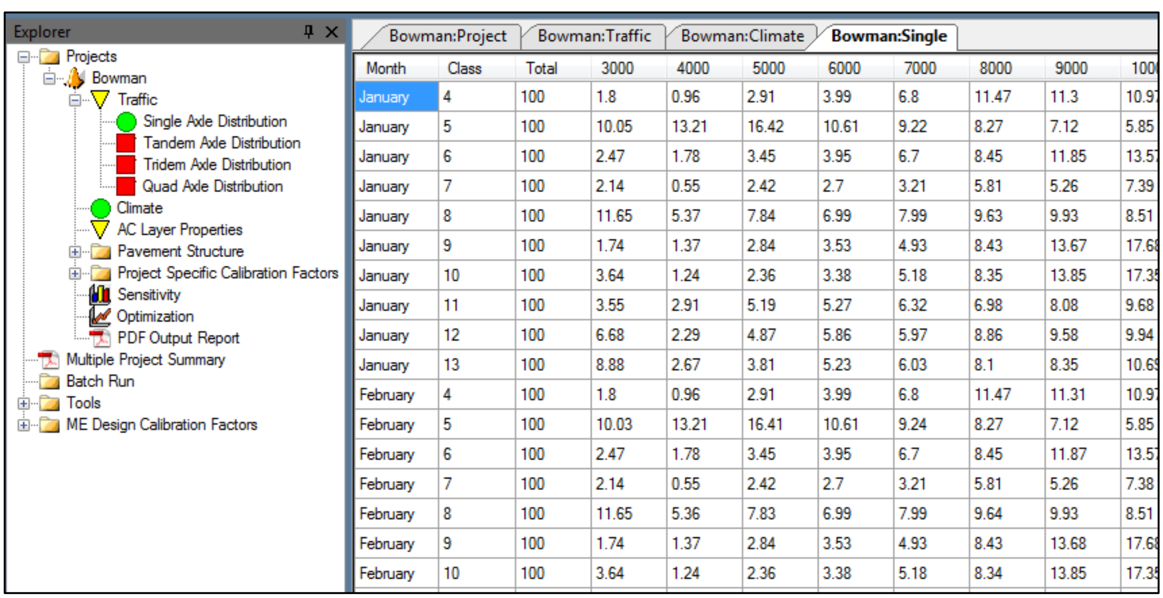
Vehicle class distribution can be defined with varying accuracy according to hierarchical input level. At Level 3, predefined class distributions (or Truck Traffic Classification Groups) can be loaded from ME Design which represent average values for a variety of selectable traffic stream types. See below. At higher input levels, class distribution must be entered manually based on regional (Level 2) or local (Level 1) values.



Use	*	TTC	Bus (%)	Multi-trailer (%)	Single-trailer and single trailer unit (SU) trucks
☐	*	5	(<2%)	(>10%)	Predominately single-trailer trucks.
☐	*	8	(<2%)	(>10%)	High percentage of single-trailer truck with some single-unit trucks.
☐	*	11	(<2%)	(>10%)	Mixed truck traffic with a higher percentage of single-trailer trucks.
☐		13	(<2%)	(>10%)	Mixed truck traffic with about equal percentages of single-unit and single-trailer...
☐		16	(<2%)	(>10%)	Predominantly single-unit trucks.
☐	*	3	(<2%)	(2 - 10%)	Predominantly single-trailer trucks.
☐		7	(<2%)	(2 - 10%)	Mixed truck traffic with a higher percentage of single-trailer trucks.
☐		10	(<2%)	(2 - 10%)	Mixed truck traffic with about equal percentages of single-unit and single-trailer...
☐		15	(<2%)	(2 - 10%)	Predominantly single-unit trucks.
☑	*	1	(>2%)	(<2%)	Predominantly single-trailer trucks.
☐	*	2	(>2%)	(<2%)	Predominantly single-trailer trucks with a low percentage of single-unit trucks.
☐	*	4	(>2%)	(<2%)	Predominantly single-trailer trucks with a low to moderate amount of single-unit ...
☐		6	(>2%)	(<2%)	Mixed truck traffic with a higher percentage of single-unit trucks.
☐		9	(>2%)	(<2%)	Mixed truck traffic with about equal percentages of single-unit and single-trailer...
☐		12	(>2%)	(<2%)	Mixed truck traffic with a higher percentage of single-unit trucks.
☐	*	14	(>2%)	(<2%)	Predominantly single-unit trucks.
☐		17	(>25%)	(<2%)	Mixed truck traffic with about equal single-unit and single-trailer trucks.

Class	Percent (%)
Class 4	1.3
Class 5	8.5
Class 6	2.8
Class 7	0.3
Class 8	7.6
Class 9	74
Class 10	1.2
Class 11	3.4
Class 12	0.6
Class 13	0.3

Axle load distributions are defined via a separate tab for each axle type: single, tandem, tridem and quad. From the Explorer pane, open the Project tree > Traffic node and select the desired axle type to open the associated Axle Load Distribution tab. These tabs contain axle load distribution factors, which represent the total axle load applications within each load interval by vehicle class. Each axle type uses a separate sheet. A single axle load distribution tab is shown below.



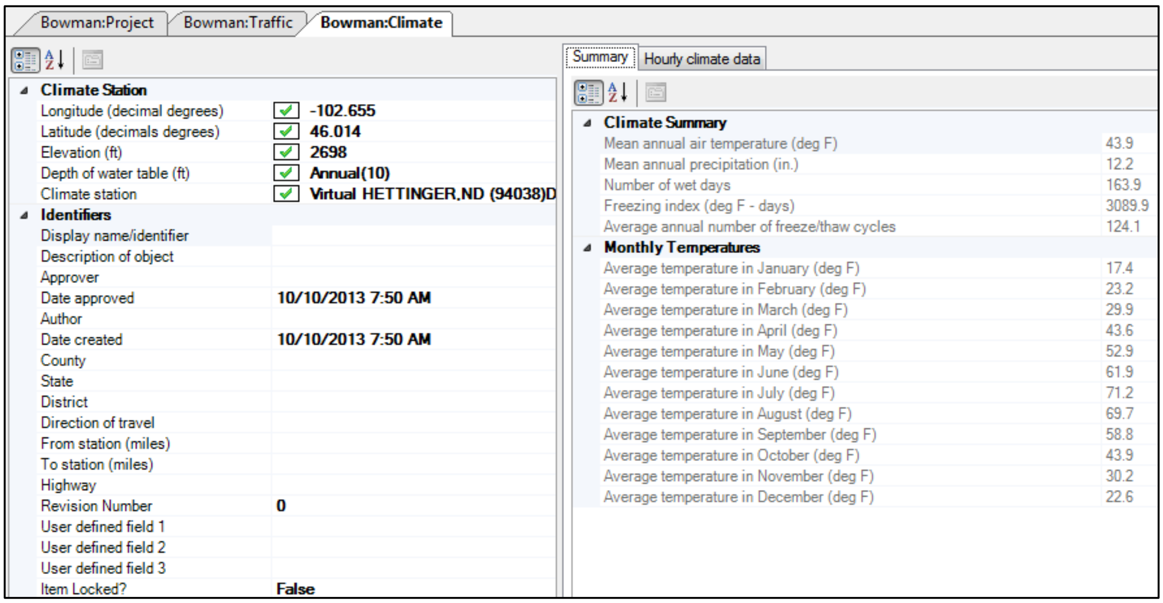
Month	Class	Total	3000	4000	5000	6000	7000	8000	9000	100
January	4	100	1.8	0.96	2.91	3.99	6.8	11.47	11.3	10.9
January	5	100	10.05	13.21	16.42	10.61	9.22	8.27	7.12	5.85
January	6	100	2.47	1.78	3.45	3.95	6.7	8.45	11.85	13.5
January	7	100	2.14	0.55	2.42	2.7	3.21	5.81	5.26	7.39
January	8	100	11.65	5.37	7.84	6.99	7.99	9.63	9.93	8.51
January	9	100	1.74	1.37	2.84	3.53	4.93	8.43	13.67	17.6
January	10	100	3.64	1.24	2.36	3.38	5.18	8.35	13.85	17.3
January	11	100	3.55	2.91	5.19	5.27	6.32	6.98	8.08	9.68
January	12	100	6.68	2.29	4.87	5.86	5.97	8.86	9.58	9.94
January	13	100	8.88	2.67	3.81	5.23	6.03	8.1	8.35	10.6
February	4	100	1.8	0.96	2.91	3.99	6.8	11.47	11.31	10.9
February	5	100	10.03	13.21	16.41	10.61	9.24	8.27	7.12	5.85
February	6	100	2.47	1.78	3.45	3.95	6.7	8.45	11.87	13.5
February	7	100	2.14	0.55	2.42	2.7	3.21	5.81	5.26	7.38
February	8	100	11.65	5.36	7.83	6.99	7.99	9.64	9.93	8.51
February	9	100	1.74	1.37	2.84	3.53	4.93	8.43	13.68	17.6
February	10	100	3.64	1.24	2.36	3.38	5.18	8.34	13.85	17.3

5.6 Climate

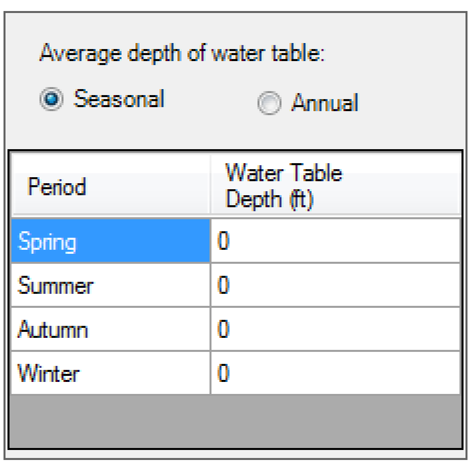
Climatic input parameters are accessible in two ways, similar to the Traffic tab:

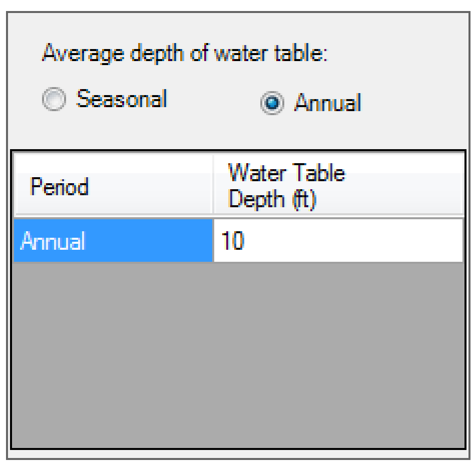
1. From the Explorer tab, expand Project tree and double-left-click the Climate node, or
2. Left-click the white area (left of the tire) in the Pavement Structure Definition diagram.

The Climate tab contains all necessary climate inputs as outlined earlier in this document. These are displayed below.



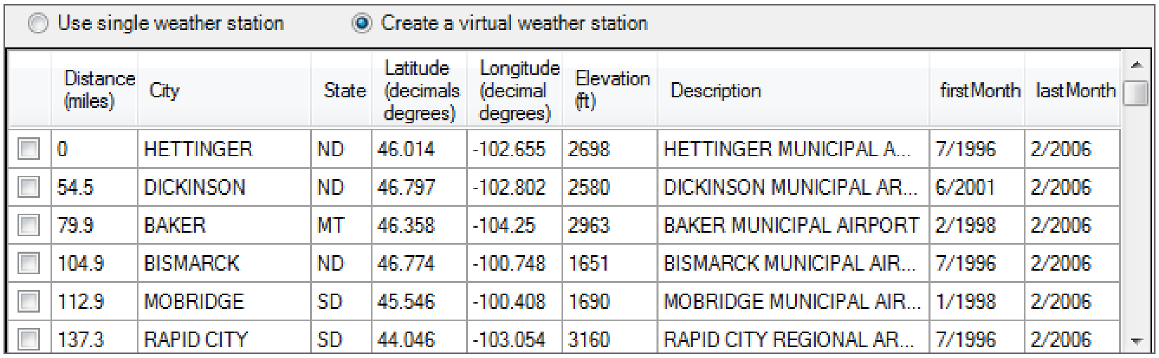
Average depth of water table can be defined on an annual or seasonal basis. This value is measured from the top of subgrade surface to the ground water level. The view in the input entry window for this parameter changes depending on the radio button selection. See below:





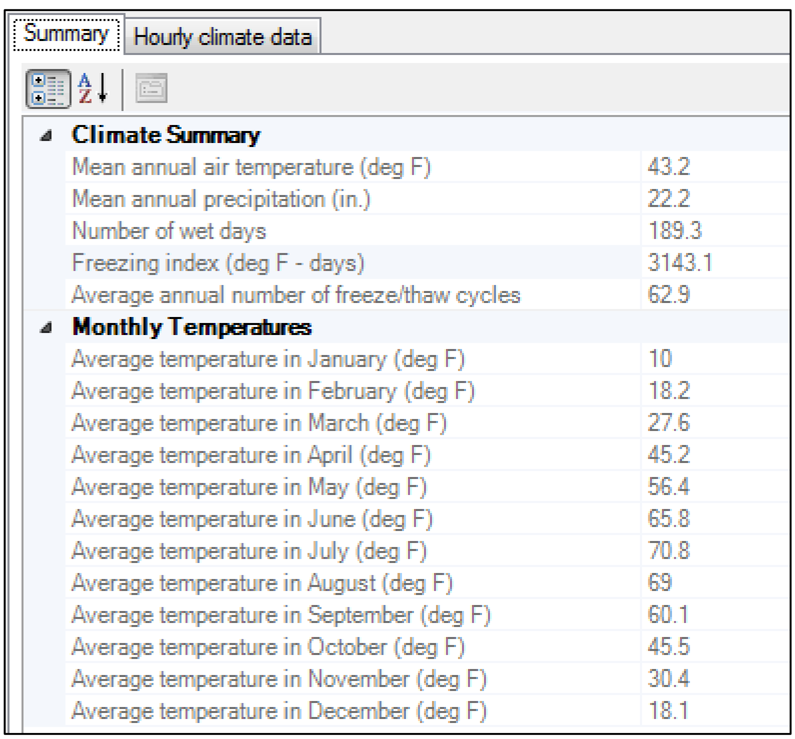
The Climate Station dialog allows two options for weather station selection. First, the user can choose a single station from the list included in the station.dat file (stored by default in the C:\...\AASHTOWare\ME Design\Defaults directory). Choosing a single station will also update latitude and longitude fields accordingly.

Alternatively, a virtual weather station can be built by selecting multiple weather stations from the list. This allows ME Design to build a composite set of weather conditions based on data from all the selected sites. For each station, the Climate Station dialog displays distance from current project location (i.e., based on latitude/longitude), city, state, latitude and longitude, elevation, a brief description, and first and last months of available data for that station. Climate data can be downloaded from the AASHTOWare website (http://www.darwinme.org/MEDesign/ClimaticData.html) and loaded to the software by placing the files in the C:\...\AASHTOWare\ME Design\HCD\ directory.



	Distance (miles)	City	State	Latitude (decimals degrees)	Longitude (decimal degrees)	Elevation (ft)	Description	firstMonth	lastMonth
☐	0	HETTINGER	ND	46.014	-102.655	2698	HETTINGER MUNICIPAL A...	7/1996	2/2006
☐	54.5	DICKINSON	ND	46.797	-102.802	2580	DICKINSON MUNICIPAL AR...	6/2001	2/2006
☐	79.9	BAKER	MT	46.358	-104.25	2963	BAKER MUNICIPAL AIRPORT	2/1998	2/2006
☐	104.9	BISMARCK	ND	46.774	-100.748	1651	BISMARCK MUNICIPAL AIR...	7/1996	2/2006
☐	112.9	MOBRIDGE	SD	45.546	-100.408	1690	MOBRIDGE MUNICIPAL AIR...	1/1998	2/2006
☐	137.3	RAPID CITY	SD	44.046	-103.054	3160	RAPID CITY REGIONAL AR...	7/1996	2/2006

Once a weather station or virtual weather station is defined, the Summary tab displays a summary of climatic input data, including annual and monthly average air temperature, annual precipitation, number of wet days, freezing index, and average number of freeze/thaw cycles.

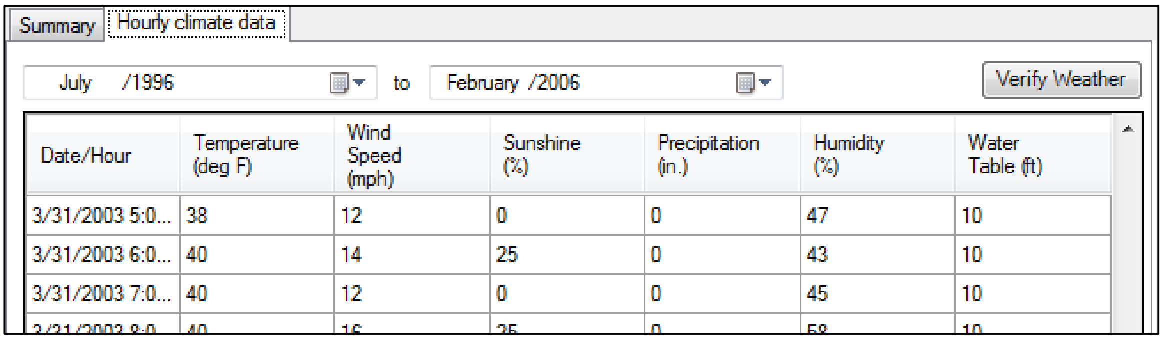


Climate Summary	
Mean annual air temperature (deg F)	43.2
Mean annual precipitation (in.)	22.2
Number of wet days	189.3
Freezing index (deg F - days)	3143.1
Average annual number of freeze/thaw cycles	62.9
Monthly Temperatures	
Average temperature in January (deg F)	10
Average temperature in February (deg F)	18.2
Average temperature in March (deg F)	27.6
Average temperature in April (deg F)	45.2
Average temperature in May (deg F)	56.4
Average temperature in June (deg F)	65.8
Average temperature in July (deg F)	70.8
Average temperature in August (deg F)	69
Average temperature in September (deg F)	60.1
Average temperature in October (deg F)	45.5
Average temperature in November (deg F)	30.4
Average temperature in December (deg F)	18.1

The Hourly Climate Data tab contains hourly data for the selected weather station. It is important to error-check this data before attempting to run an analysis. Select the Verify Weather button to generate a list of potential data errors in the Error List pane. These errors can include:

- Missing or blank data
- Data which are outside an acceptable range
- The difference between the values of two consecutive hourly records is outside an acceptable range

Any listed errors must be resolved by changing the offending values before ME Design can run analysis.



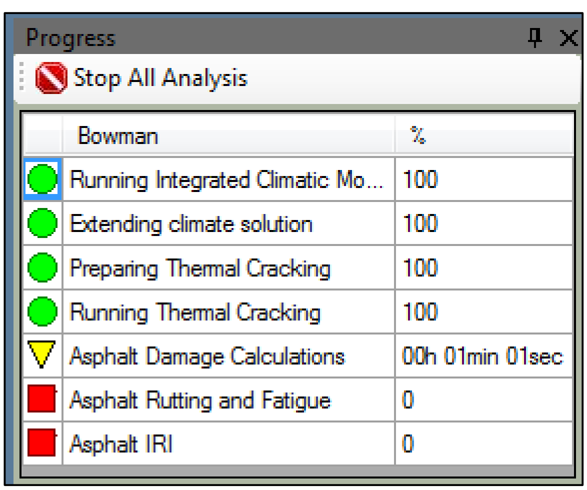
5.7 Analysis

Once inputs are completed, it is time to run analysis. To begin a model run, from the Menu toolbar select **Run.** If a batch run is desired, select **Batch Run.**

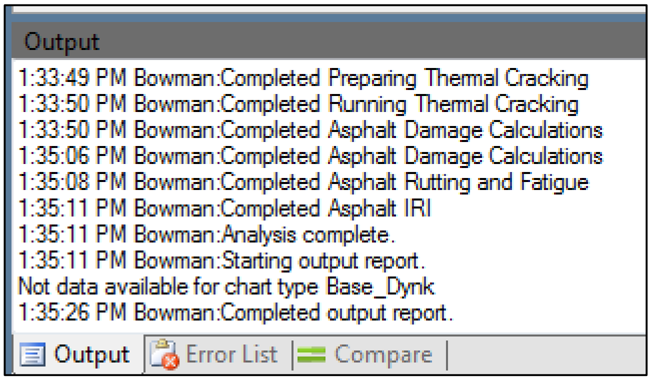
During a model run, the Progress pane will display the status of any currently running analyses. Each step in the model run process will be appear next to a color-coded status icon:

- Green circle: The analysis is complete
- Yellow triangle: The analysis is in progress
- Red square: The analysis has not yet been run

To terminate analysis at any point, select **Stop All Analysis** from the Progress pane



Analysis steps are logged, with timestamps, in the Output pane.



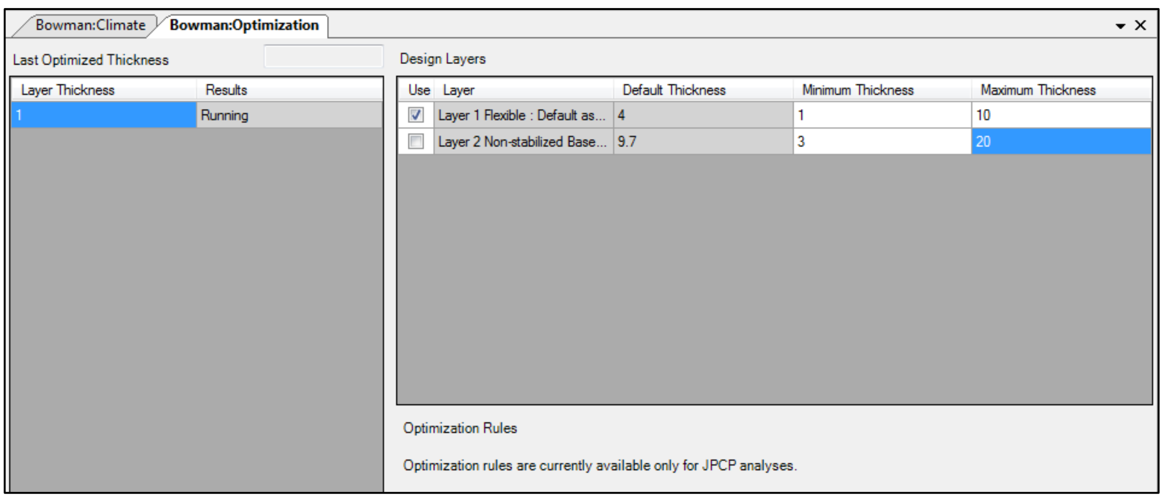
5.8 Reporting

After completing an analysis run or batch run, a PDF report containing an input summary and output reports will appear. This report is saved to the project directory (i.e., where the project *.dgpx file is saved) and can also be accessed via the **PDF Output Report** item in the project tree of the Explorer pane.

The report is also available in Excel format if the corresponding option has been selected. To generate Excel reports, go to the Explorer Pane > Tools > Options to open the Options tab. In the tab, set Generate Excel Reports to True. After running analysis, select the item Excel Output Report in the project tree of the Explorer Pane.

5.9 Optimization

ME Design includes an Optimization feature that allows the user to optimize the thickness of any one layer at a time, in 0.5 inch increments, between user-defined minimum and maximum thicknesses. To access the Optimization tab, go to the Explorer pane, expand the Project tree, and double-left-click the Optimization node.



To perform optimization, select a layer in the Use column of the Design Layers field and define minimum and maximum thicknesses. Then select the Optimize Thickness button at the bottom of the tab to run the optimization process. ME Design displays the results of the optimization process in real time. When the process has completed, the lowest satisfactory thickness will be displayed in the Last Optimized Thickness field.

6. RECOMMENDATIONS FOR FUTURE RESEARCH

This document introduces the Mechanistic-Empirical Pavement Design Guide, including its conceptual framework and methodology. It compares the design guide with the other major pavement design guide, AASHTO's Guide for Design of Pavement Structures, 1993 edition. It also introduces considerations relevant for agencies considering implementation, including local calibration and sensitivity analysis.

Future research will continue to focus on factors relevant to a potential North Dakota MEPDG implementation. First, MEPDG transfer functions must be locally calibrated using North Dakota historical distress data. Local calibration is an important step toward implementation, which was discussed above in detail. A calibrated set of MEPDG distress functions must be achieved before any further investigations of region-specific MEPDG behavior can occur.

Local calibration requires agencies to initially select hierarchical input level for each parameter based on the understanding that higher level inputs generally contribute to more accurate pavement performance predictions. However, relatively little literature exists to quantify the expected impact of input level on model accuracy. This information could be useful to an agency interested in the most cost-effective implementation and data collection plan.

The move toward MEPDG implementation is predicated on the understanding that MEPDG yields more cost-effective pavement structures than the 1993 AASHTO method does. Many agencies, however, have been interested in testing this assertion under local material, traffic, climate, and policy conditions. A comparison between the 1993 AASHTO empirical model and a locally calibrated set of MEPDG performance models could either confirm or cast doubt on the usefulness of the new design method on North Dakota roads. Cost-effective pavement design is, of course, of particular interest to the oil-impacted region of western North Dakota.

While MEPDG is a more theoretically sound pavement design procedure than the AASHTO method, its implementation must ultimately offer long-term financial benefits to each adopting agency in terms of better pavement designs. It is not enough, particularly in the current climate of shrinking financial resources, for a pavement design procedure to be solely based on sound engineering principles. The method must be demonstrated to be cost-effective in implementation and in practice. The research needs identified above will help satisfy these needs for the state of North Dakota.

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