# 4. STRUCTURAL DESIGN

Since the late 1950s, pavement design procedures have evolved from empirical-based methods (those based on observations of how pavements perform) to mechanistic-empirical based methods (those that incorporate basic pavement responses and behavior). This movement was made in large part due to improved characterization of traffic and materials, improved understanding of climate effects on materials, improved understanding of pavement performance, as well as improved computational capabilities.

In 2008, the American Association of State Highway and Transportation Officials (AASHTO) released the *Mechanistic-Empirical Pavement Design Guide – A Manual of Practice* (*MEPDG*), which documents a pavement design methodology based on engineering mechanics that has been nationally calibrated using in-service pavement performance data (AASHTO 2008). In 2011, AASHTO released the first version of DARWin-ME, rebranded to [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), which is a production ready pavement design software tool that expands and improves the features of the prototype computational software developed as part of NCHRP 1-37A Project, *Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures.*

The following describes a brief history of the AASHTO pavement design procedures, the various aspects of a mechanistic-empirical based pavement design process, and a summary of the [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) software program. In relation to [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), only a summary is provided in this document and the reader is referred to the *MEPDG* ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)), the software user manual, or other relevant documentation.

## AASHTO Pavement Design Procedures

The following provides a brief history of previous and current AASHTO pavement design procedures.

### AASHO Road Test (1958 – 1960)

The AASHO Road Test was the largest road experiment of its time and was initiated to evaluate the performance of highway pavements under moving loads with known magnitude and frequency (HRB 1961). To more fully understand subsequent revisions to the AASHO pavement design procedures, a brief summary of the AASHO Road Test is provided below.

* Location – Ottawa, Illinois (later became part of Interstate 80).
* Climatic details at the time of the test (see Table 4-1).
* Time of construction – August 1956 to September 1958.
* Traffic application – October 1958 to November 1960.
* Test facilities – six 2-lane loops.
* Traffic (fixed load magnitudes on each test loop, see Table 4-2).
* Performance measurements – roughness and visual distress, deflection, strains, and Present Serviceability Index (PSI).
* Asphalt pavement (see Table 4-3).
* Concrete pavement (see Table 4-4).

Table 4-1. AASHO Road Test climatic conditions.

|  |  |
| --- | --- |
| **Condition** | **Measure** |
| Average mean temperature – January | 27 °F |
| Average mean temperature – July | 76 °F |
| Average annual rainfall | 34 in. |
| Average depth of frost | 28 in. |

Table 4-2. AASHO Road Test traffic loads (HRB 1961).

|  |  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- | --- |
| **Loop** | **Lane** | **TotalNumberof Axles** | **FrontAxle(lbs)** | **LoadAxleType** | **LoadAxle(lbs)** | **GrossWeight(lbs)** |
| 2 | 1 | 2-axle | 2,000 | Single | 2,000 | 4,000 |
| 2 | 2-axle | 2,000 | Single | 6,000 | 8,000 |
| 3 | 1 | 3-axle | 4,000 | Single | 12,000 | 28,000 |
| 2 | 5-axle | 6,000 | Tandem | 24,000 | 54,000 |
| 4 | 1 | 3-axle | 6,000 | Single | 18,000 | 42,000 |
| 2 | 5-axle | 9,000 | Tandem | 32,000 | 73,000 |
| 5 | 1 | 3-axle | 6,000 | Single | 22,400 | 50,800 |
| 2 | 5-axle | 9,000 | Tandem | 40,000 | 89,000 |
| 6 | 1 | 3-axle | 9,000 | Single | 30,000 | 69,000 |
| 2 | 5-axle | 12,000 | Tandem | 48,000 | 108,000 |

1 Loop 1 not subjected to vehicle loading and used to test environmental effects only.

Table 4-3. AASHO Road Test asphalt pavement details (HRB 1961).

|  |  |  |
| --- | --- | --- |
| **Material** | **Description** | **Other Details** |
| Aggregate | Crushed limestone (3/4 in. nominal maximum aggregate size), natural siliceous coarse sand, limestone dust | Marshall Method, 50 blows/face, asphalt layer thickness ranged from 1.0 to 6.0 in. (all loops) |
| Asphalt Binder | 85 – 100 penetration grade | 4.4 – 5.4 percent by weight of total mix, average 7.7 percent in-place air voids (all loops) |
| Base Course | Crushed dolomitic limestone, top aggregate size of 1-1/2 in., 10 percent passing the No. 200 | Frost susceptible, thickness ranged from 0 to 9.0 in. (all loops) |
| Subbase | Sand-gravel, top aggregate size of 1-1/2 in., 6.5 percent passing the No. 200 sieve | Non-frost susceptible, thickness ranged from 0 to 16.0 in. (all loops) |
| Subgrade | A-6 | --- |

Table 4-4. AASHO Road Test concrete pavement details (HRB 1961).

|  |  |  |
| --- | --- | --- |
| **Material** | **Description** | **Other Details** |
| Concrete | Type I cement, cement content of 564 lb/yd3, water-cement ratio of 0.47, 1-1/2 to 2-1/2 in. maximum aggregate size, mean air content of 3.7 to 4.2 percent, minimum compressive strength of 3500 psi, minimum modulus of rupture of 550 psi, slump of 1.5 to 2.5 in., coefficient of thermal expansion of 4.6 to 5.1 x 10-6/°F | Thickness ranged from 2.5 to 12.5 in., all loops contained transverse dowel bars ranging from 3/8 to 1-5/8 in. diameter, and 12 and 18 in. in length |
| Base Course (same as asphalt pavement) | Crushed dolomitic limestone, top aggregate size of 1-1/2 in., 10 percent passing the No. 200 | Frost susceptible, thickness ranged from 0 to 9.0 in. (all loops) |
| Subgrade (same as asphalt pavement) | A-6 | --- |

One of the significant results from the AASHO Road Test was the development of equivalent single axle loads (ESALs) ([TRB 2007](http://onlinepubs.trb.org/onlinepubs/circulars/ec118.pdf)). ESAL is a numerical factor that expresses the relationship of a given axle load (typically 18,000 lbs) to another axle load in terms of the relative damage to the pavement structure. It is important to note, that ESAL factors are not only dependent on the axle type and weight but also on pavement type (i.e., ESAL factors are different for asphalt and concrete pavements). ESALs may be calculated using Equation 4-1 (for asphalt pavements) and Equation 4-2 (for rigid pavements).



(4-1)

where:

 W = axle applications

 Lx = axle load being evaluated (kips)

 L18 = 18 (standard axle load in kips)

 L2 = code for axle configuration

 1 = single axle
2 = tandem axle
3 = triple axle
x = axle load equivalency factor being evaluated
s = code for standard axle = 1 (single axle)

 G =

 pt = "terminal" serviceability



 β =

 SN = structural number



 (4-2)

where:

 W = axle applications

 Lx = axle load being evaluated (kips)

 L18 = 18 (standard axle load in kips)

 L2 = code for axle configuration

 1 = single axle
2 = tandem axle
3 = triple axle
x = axle load equivalency factor being evaluated
s = code for standard axle = 1 (single axle)



 G =

 pt = "terminal" [serviceability index](http://training.ce.washington.edu/wsdot/Modules/09_pavement_evaluation/09-6_body.htm#psi)



 b =

 D = Slab thickness, in.

### AASHO Interim Guide for the Design of Flexible Pavement Structures (1961)

Following the completion of the AASHO Road Test the *AASHO Interim Guide for the Design of Flexible Pavement Structures* was released.

### AASHO Interim Guide for the Design of Rigid Pavement Structures (1962)

Empirical models for rigid pavements were developed to predict the number of axle load applications as a function of slab thickness, axle type and weight, and terminal serviceability for both jointed plain concrete pavement (JPCP) and jointed reinforced concrete pavement (JRCP). Soon after development, the rigid pavement model was modified to include concrete strength, subgrade k value, and concrete elastic modulus (Langsner, Huff, and Little 1962). An important note with the rigid pavement model is that the presumed loss of serviceability was due entirely to slab cracking and did not include any contribution of faulting. The doweled pavements at the AASHO Road Test experienced loss of support, however, they did not fault (Langsner, Huff, and Little 1962). In addition, for undoweled jointed pavements, jointed pavements with stabilized bases, jointed pavements with joint spacings not used at the Road Test, continuously reinforced concrete pavements (CRCP) or concrete pavements in any other environment than those at the AASHO Road Test, prediction of axle load applications was done so by extrapolating the original AASHO model.

### AASHO Interim Guide for the Design of Pavements (1972)

An interim guide was released in 1972, but the basic design methods and procedures remained unchanged from those originally published in 1961 and 1962. Revisions for the 1972 Interim Guide included:

* Rearranging and simplifying text.
* Additional explanatory text was added, related to:
* Examples for determining traffic loadometer data.
* New design examples to cover a wider range of designs.
* Flexible pavement overlay design procedures.
* For rigid pavements, model modification to include the J factor.

### AASHTO Rigid Pavement Design Revisions (1981)

Revisions to the rigid pavement design included incorporation of a safety factor in the form of a reduction in the concrete modulus of rupture.

### AASHTO Guide for the Design of Pavements (1986)

The 1986 revision included major revision to both the concrete and flexible designs, though the basic models remained the same. Final revisions were published as two volumes: Volume 1 presented the basic design guide information, and Volume 2 included appendices for additional explanations. Specifically, revisions included the following additions:

* Reliability – incorporation of the normal deviate (ZR) and the overall standard deviation (So).
* Resilient modulus for soil support – recommend use of AASHTO T 274.
* Drainage – guidance in the design of subsurface drainage systems and modifying the design equations to take advantage of improvements in performance related to good drainage. Addition of the drainage adjustment factor (Cd).
* Improved environment considerations – adjust designs as a function of environment (e.g., frost heave, swelling soils, and thaw-weakening), with major emphasis on thaw-weakening and the effect that seasonal variations have on performance.
* Extension of load equivalency values – include heavier loads, more axles, and terminal serviceability levels up to 3.0.
* Improved traffic data – method for calculating equivalent single axle load (ESAL) and specific problems related to obtaining reliable estimates of traffic loading.
* New procedure for determining the design k value for rigid pavements.
* New J factor values for rigid pavements.
* Resilient modulus for flexible pavement layer coefficients – the resilient modulus test is recommended for assigning layer coefficients to both stabilized and unstabilized material.
* Rehabilitation – major additions regarding rehabilitation with or without overlays.
* Tied concrete shoulders or widened lanes – procedures added for designing rigid pavements with tied shoulders or widened outside lanes.
* Life cycle cost considerations.
* Pavement management information.
* Design of pavements for low-volume roads.
* State of the knowledge on mechanistic-empirical design concepts (Part IV).
* Incorporation of change in present serviceability index (*ΔPSI = p0 – pt*).

### AASHTO Guide for the Design of Pavements (1993)

Modifications incorporated into 1993 Guide included major revisions of the overlay design procedures, such as:

* Revisions to the rehabilitation chapter and Appendices L through M.
* Section III, Chapter 5, *Rehabilitation Methods with Overlays* was revised and expanded to include:
* Important consideration in overlay design.
* Pavement evaluation for overlay design.
* Expanded specifications for the following overlay options:
* Asphalt concrete overlay of asphalt concrete pavement.
* Asphalt concrete overlay of fractured concrete pavement.
* Asphalt concrete overlay of JPCP, JRCP, and CRCP.
* Asphalt concrete overlay of Asphalt concrete over JPCP, Asphalt concrete over JRCP, and Asphalt concrete over CRCP.
* Bonded concrete overlay of JPCP, JRCP, and CRCP.
* Unbonded JPCP, JRCP, or CRCP overlay of JPCP, JRCP, CRCP, or Asphalt concrete over concrete.
* JPCP, JRCP, and CRCP overlay of Asphalt concrete pavement.

The design equations for asphalt pavements include:

 (4-3)

where:

 W18 = predicted number of 18-kip equivalent single axle load applications

 ZR = standard normal deviate

 So = combined standard error of the traffic prediction and performance prediction

 ΔPSI = difference between the initial design serviceability index, po, and the design terminal serviceability index, pt

 MR = resilient modulus (psi)

 ai = ith layer coefficient

 Di = ith layer thickness (in.)

 mi = ith layer drainage coefficient

The design equations for concrete pavements include:

 (4-4)

where:

 W18 = predicted number of 18-kip equivalent single axle load applications

 ZR = standard normal deviate

 So = combined standard error of the traffic prediction and performance prediction

 D = thickness of pavement slab (in.)

 ΔPSI = difference between the initial design serviceability index, po, and the design terminal serviceability index, pt

 S’c = concrete modulus of rupture for (psi)

 J = load transfer coefficient

 Cd = drainage coefficient

 Ec = concrete modulus of elasticity (psi)

 k = modulus of subgrade reaction (pci)

### AASHTO Guide for Design of Pavement Structures, Supplement (1998)

In 1998, revisions were made to the AASHTO design model for concrete pavements based on work performed under NCHRP Project 1-30 (Darter, Hall, and Kuo 1995) and field-validated using data from the Long-Term Pavement Performance (LTPP) GPS-3, GPS-4, and GPS-5 sites ([Hall et al. 1997](http://www.tfhrc.gov/pavement/ltpp/reports/96198/96198.pdf)). Specifically, the 1998 revision developed a new design model incorporating an improved process for characterizing the design k value, and also directly considered the effects of base modulus, base thickness, slab/base friction, joint spacing, edge support, temperature and moisture gradients, and traffic loading on critical slab stresses. In addition, faulting models for undoweled and doweled joints were included to “check” the adequacy of the joint load transfer design ([Hall et al. 1997](http://www.tfhrc.gov/pavement/ltpp/reports/96198/96198.pdf)).

### NCHRP 1-37A, Development of the 2002 Guide for the Design of New and Rehabilitated Pavement Structures (1998 – 2004)

In 1996, the AASHTO Joint Technical Committee on Pavements recommended the development of a mechanistic-empirical based pavement design procedure under NCHRP Project 1-37A, *Guide for the Design of New and Rehabilitated Pavement Structures.* The outcome of NCHRP Project 1-37A was a research document and rudimentary software for designing pavements using mechanistic-empirical procedures ([ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)). The resulting procedure, *MEPDG* ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)), incorporates additional features such as climate impacts, material aging properties, and axle load spectra for predicting pavement distress (e.g., fatigue cracking, rutting, thermal cracking, joint faulting, slab cracking, punchouts) over the designated performance period.

### AASHTO Mechanistic-Empirical Pavement Design Guide—A Manual of Practice (2008)

In 2008, AASHTO approved the adoption of an interim edition of the *Mechanistic-Empricial Pavement Design Guide, A Manual of Practice* (*MEPDG*) ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)).

### [AASHTOWare](http://www.aashtoware.org/Pages/default.aspx)® [Pavement ME Design™](http://www.aashtoware.org/Pages/DARWin-ME.aspx) Software (2011)

As part of NCHRP 1-37A, rudimentary software was developed to accompany the *MEPDG* ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)). [AASHTOWare](http://www.aashtoware.org/Pages/default.aspx) has updated the rudimentary software developed as part of NCHRP 1-37A to incorporate a number of functional and operational modifications, such as (not an inclusive list):

* Increase processing speed, especially for the flexible pavement design/analysis.
* Inclusion of SI units.
* Improved batch mode functionality.
* Import of backcalculation results.
* Ability to create individual weather stations for use in the Enhanced Integrated Climatic Model (EICM).
* Import traffic data from third-party software.

The updated [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) software is available for licensing through [AASHTOWare](http://www.aashtoware.org/Pages/default.aspx).

## Mechanistic-Empirical Pavement Design

Although concepts for a mechanistic-empirical pavement design procedure were described as early as the 1950s (referred to as “rational” design procedures), interest in mechanistic-based procedures became renewed in the 1980s. Since that time, significant advancements in computer (hardware and software) capabilities have occurred, pavement technology, analysis, and testing has evolved, and a number of state departments of transportation have either initiated or implemented mechanistic-empirical pavement design procedures (e.g., California, Kentucky, Minnesota, Washington State).

Although the previous versions of the AASHTO pavement design procedures have served the pavement community well, a number of limitations exist that minimizes the effectiveness and applicability of the AASHO Road Test results. For example:

* Limited design traffic levels. The AASHO Road Test pavements received slightly more than 1 million axle load applications (or approximately 6.2 million ESALs). Therefore, for higher traffic loadings, the recommended layer thicknesses are based on extrapolations and may result in conservative designs.
* Single climatic conditions. Since the AASHO Road Test was subject only to the climatic conditions in the area of Ottawa, IL it is difficult to quantify the effects of different climatic forces on pavement performance. Though the more recent AASHTO pavement design procedures include factors associated with climatic conditions (e.g., seasonal variation of subgrade, effective modulus of subgrade reaction), the model is unable to effectively characterize other climatic conditions than those experienced at the AASHO Road Test.
* Single subgrade, base, and surfacing materials. Significant advancements in material technology have been made since the AASHO Road Test, and today a much broader availability of materials are being incorporated into pavement structures. For example, materials such as warm mix asphalt, stone-mastic asphalt, open-graded friction courses, and high-strength concrete or additions to mixes such as RAP and RAS were not included in the AASHO Road Test experiment and therefore the corresponding performance impacts cannot be accurately modeled using previous AASHTO pavement design procedures.
* Pavement condition. At the time of the Road Test modeling of individual distress types posed several challenges (e.g., computationally). Today more accurate and reliable models can be developed for pavement specific distress (e.g., rutting, cracking, faulting).

Considering the above limitations and advancements in computer and pavement technology, the movement to a mechanistic-empirical based pavement design procedure is fitting and appropriate. Many mechanistic empirical procedures require the designer to assume the pavement design and the procedure checks that design for adequacy. The newest procedure(s) use an initial design, but iteratively analyze various thicknesses to achieve the performance targets. A mechanistic-empirical pavement design process attempts to correlate critical pavement responses to pavement performance. The “mechanistic” portion uses analytical calculations for transforming pavement loading (e.g., truck and climate) and inherent material properties (e.g., elastic modulus) to critical pavement responses (e.g., stress, strain or deflection), and then the “empirical” portion relates those pavement response to the development of observed distresses (e.g., rutting, cracking, faulting, International Roughness Index [IRI]).

### Advantages of Mechanistic-Empirical Pavement Design

A number of advantages can be realized through the use of a mechanistic-empirical based pavement design procedure, including:

* Analyze a broader range of vehicle types, axle loadings, and tire pressures.
* Incorporate material parameters that better relate to pavement performance.
* Enable characterization of local materials.
* Model the effects of climate and aging of materials.
* Improve the characterization of existing pavement layer parameters.
* Improve the reliability of pavement performance prediction.

### Mechanistic-Empirical Pavement Design Process

In order to perform a mechanistic-empirical based pavement design, there are a number of fundamental aspects that need consideration. These include, estimation of traffic loads, characterization of materials, climate, and performance prediction. For rehabilitation design, characterization of the existing pavement structure may also need to be considered. Each of these fundamental aspects is described below.

#### Estimation of Traffic Loads

Characterization of traffic loadings for the majority of mechanistic-empirical pavement design procedures has been through the use of an equivalent single-axle load (ESAL). Though developed as part of the AASHO Road Test, the ESAL concept has been an integral feature of many pavement design procedures and a useful way of expressing the effects of a mix of truck axle types and weights into a single value. However, with the implementation of [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), traffic loading is characterized through the use of axle load spectra (further discussed below).

#### Characterization of Materials

Pavement materials are typically characterized according to layer moduli (e.g., dynamic modulus, resilient modulus, modulus of elasticity) and Poisson’s ratio. Layer moduli for new pavement materials can be characterized through laboratory testing, based on historical knowledge, or using engineering-based assumptions. Values for Poisson’s ratio are typically assumed because the resulting thickness designs are largely insensitive to Poisson’s ratio (within the typical range). In addition, nationally accepted testing procedures for determining Poisson’s ratio are not available, excluding ASTM C469, *Standard Test Method for Static Modulus of Elasticity and Poisson’s Ratio of Concrete in Compression*.

#### Climate

Many materials are temperature and/or moisture sensitive, meaning some of their material properties are influenced by air temperatures (e.g., modulus of the asphalt layer increases with colder temperatures and decreases in warmer temperatures) or moisture content (e.g., resilient modulus of fine grained soil decreases with increased moisture content). In a mechanistic-empirical pavement design procedure, climatic effects are typically included through adjustments in layer moduli.

#### Performance Prediction

One of the key benefits of a mechanistic-empirical design procedure is the ability to predict pavement performance at any point in time as a function of key material parameters and traffic loadings. The [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) software, using regression equations, predicts key pavement distress types (e.g., cracking, faulting, punchouts) and smoothness over the entire pavement design life. Distress prediction is based on the mean value (i.e., 50 percent reliability) of all inputs. Performance prediction equations included in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) are further described in the [Design of New and Reconstructed Asphalt Pavements](#Flexible) and the [Design of New and Reconstructed Concrete Pavements](#Rigid).

#### Characterization of the Existing Pavement Structure

In order to assist in the determination of treatment timing, treatment type, and, when applicable, overlay thickness requirements, the accurate characterization of the existing pavement structure becomes pivotal in the selection of viable treatments. Typically, characterization of the existing structure is determined through:

* Pavement condition surveys measure the severity and extent of pavement distress, such as cracking, rutting, roughness, spalling, raveling, delaminations, and so on. A number of state highway agencies have developed their own pavement condition survey manuals. In addition, the [LTPP Distress Identification Manual](http://www.tfhrc.gov/pavement/ltpp/reports/03031/03031.pdf) provides definitions of distress type, severity, and extent ([Miller and Bellinger 2003](http://www.tfhrc.gov/pavement/ltpp/reports/03031/03031.pdf)).
* Pavement coring to quantifying layer condition (e.g., stripping, delamination), layer thicknesses, and to obtain surface and subsurface samples for additional testing (e.g., dynamic modulus of the asphalt layer, concrete strength, resilient modulus).
* Soil borings for classifying subgrade material type(s) and thickness, determine depth to bedrock, stiff layer or water table, as well as to provide samples for laboratory testing (e.g., moisture content, resilient modulus).
* Pavement deflection testing to compute deflection basin parameters, determine load transfer efficiency of concrete pavements, determine seasonal variations (e.g., load restrictions due to spring-thaw), and estimate pavement layer moduli.
* Various specialized testing procedures, such as ground penetrating radar (GPR) for determining pavement layer thicknesses and identifying rebar location, seismic testing (seismic analysis of surface waves [SASW], impact echo [IE], and impulse response [IR]) for determining layer moduli and thickness, and dynamic cone penetrometer for determining the strength of unbound materials.

## [AASHTOWare](http://www.aashtoware.org/Pages/DARWin-ME.aspx) Pavement ME Design™

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is the next generation of [AASHTOWare](http://www.aashtoware.org/Pages/default.aspx)® pavement design software, which builds upon the *MEPDG* ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)) and expands and improves the features of the prototype computational software developed as part of NCHRP 1-37A. [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is a production-ready software tool to support the day-to-day operations of public and private pavement engineers.

For many pavement engineers, the new *MEPDG* ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)) is a paradigm shift away from a nomograph-based design to one based on engineering principles and mechanics. Instead of entering basic site and project information into an equation and getting an empirically based pavement design output, the engineer can use detailed traffic, materials, and environmental information to assess the short- and long-term performance of a pavement design using nationally and/or locally calibrated models.

The [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) software is a comprehensive pavement design and analysis tool, capable of providing insights on pavement performance through the entire pavement structure life cycle, from design through rehabilitation. This state-of-the-practice approach represents the current advancements in pavement design and is expected to contribute to the design of smoother, longer-lasting, and more cost-effective pavements.

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) contains three primary modules: Project Definition, Project Inputs, and Project Results. These modules present a logical progression of required information for the designer. The Project Definition module provides for project-specific information and analysis parameters to help distinguish various “what-if” scenarios. The Projects Inputs module provides the designer the option to scrutinize the traffic, climate, and materials to be considered for design. The Projects Results module provides a summary of project input parameters and organizes the performance results by type and time period.

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) also provides tools to generate optimized pavement design based on given requirements and provides extensive reports to evaluate and fine-tune the design. The final design is saved in database format so it can subsequently be used for various distress and performance analyses and for other management purposes.

As previously mentioned, [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) reflects a change in the methods and procedures engineers use to design pavement structures. It takes advantage of the advances in material mechanics, axle-load spectra, and climate data for predicting pavement performance. While this software does not address all of the challenges to pavement design, it is a quantum leap forward from previous pavement design procedures and facilitates future development in pavement modeling and analysis.

## AASHTOWare [Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) Analysis/Design Approach

The analysis/design approach used in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) incorporates a broad range of input values and analysis techniques for the determination of pavement performance. Several key analysis procedures that are unique to [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) includes characterization of foundation materials (stiffness, and where applicable, volume change, frost heave, thaw weakening, and drainage concerns) and the use of the Enhanced Integrated Climatic Model (EICM) for modeling temperature and moisture conditions within each layer. Specific details of [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) are further described below.

### Hierarchical Levels

The use of hierarchical levels for all input values is unique to [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx). The primary reasoning for the hierarchical levels is to provide the pavement designer with flexibility in obtaining needed design inputs according to budget, time, and project importance. In this manner, more detailed information based on measurement of inputs (such as dynamic modulus of the asphalt layer or characterization of traffic loadings using weigh-in-motion) would be obtained for critical roadway segments (e.g., interstate), while inputs for roadways of less significance (e.g., minor collectors) would be based on default values or engineering expertise. It should also be noted that the use of level 1 inputs will not necessarily result in more reliable pavement designs since the current pavement performance prediction models where calibrated using primarily level 2 and level 3 inputs.

#### Selection of Hierarchical Level

In general, each of the input categories (i.e., traffic and materials) incorporates three levels of input, ranging from Level 1 (representing the highest level of accuracy ) to Level 3 (representing the lowest level of accuracy). Each input level is further described as follows:

* Level 1 – Inputs are based on measured values obtained from material testing (e.g., modulus, compressive strength), deflection testing (FWD) and backcalculation, and site specific traffic information. This level requires the most resources, time, and level of effort to collect.
* Level 2 – Inputs are based on user-selected values that could be determined through limited testing or based on correlations.
* Level 3 – Inputs are based on typical regional averages or expert opinion.

Ideally, designing the more critical roadways using all Level 1 inputs would provide the most accurate estimation of pavement performance. However, since this requires the greatest level of effort, it may also not be possible to collect all inputs at this level due to budget constraints, time constraints for data collection and analysis, and the uncertainty of the material source. One of the beneficial characteristics of [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is the ability to mix input levels based on criticality of the roadway section, resources, and level of effort. An important characteristic is that regardless of the input used, the determination of resulting pavement distress is based on the same computational algorithm ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

#### Impact of Hierarchical Level on Performance Prediction

As previously described, Level 1 inputs provide the highest level of accuracy, while Level 3 provides the lowest. To date, the impact of the input of level accuracy has not been fully quantified. However, the objective of [NCHRP 1-47, *Sensitivity Evaluation of MEPDG Performance Prediction*](http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2487) is to determine the sensitivity of performance prediction to the variability of input values.

### Characterization of Traffic Loading

As with any pavement design procedure, characterization of the estimated future traffic loadings is critical to the thickness design results. Though the estimation and use of ESALs has served the pavement community well, a more precise method for characterizing traffic loading involves the use of axle load spectra. An axle load spectrum characterizes traffic loading according to the axle configuration and weights of each vehicle class (for FHWA class 4 vehicles and above). [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) utilizes axle load spectra as an input to the analysis process. The axle load spectra represent the hourly, daily, monthly, and seasonal distributions of the traffic with respect to axle type/load of various vehicle classes. It is important to note that unlike previous AASHTO pavement design guides,  [[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx)](http://www.aashtoware.org/Pages/DARWin-ME.aspx) uses the same methodology for characterizing traffic loadings regardless of the pavement type (e.g., concrete, asphalt, composite) or design type (new or rehabilitated).

Traffic information related to volume and loading may be collected via weigh-in-motion (WIM), automated vehicle classification (AVC), and vehicle counts.

#### WIM

WIM uses measurement systems (e.g., bending plate, quartz-piezo sensors) installed in the pavement surface to measure site-specific traffic load and volume data, truck traffic (e.g., number of axles, axle spacing) and tire factors (e.g., tire spacing), truck traffic distribution and volume variables (e.g., average annual daily truck traffic, percent trucks).

#### AVC

AVC is a measurement system used to determine the site specific normalized truck class distribution (FHWA Class 4 through 13). AVC data can also be used to determine annual average daily traffic (AADT).

#### Vehicle Count

Vehicle counts can be collected to provide the total number of vehicles categorized by the type of vehicle (i.e., FHWA Class 1 through 13), and results in a broad categorizing of the number of vehicles in each classification group. One drawback is that it provides only number of trucks in each classification and does not provide axle specific details, such as axle spacing, axle load, and tire pressure).

Additional details of field acquisition of traffic data can be found in the FHWA *Traffic Monitoring Guide* ([FHWA 2001](http://www.fhwa.dot.gov/ohim/tmguide/tmg0.htm)), NCHRP Report 509, *Equipment for Collecting Traffic Load Data* ([Hallenbeck and Weinblatt 2004](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_509.pdf)), and ASTM E1318, *Standard Specification for Highway Weigh-In-Motion (WIM) Systems with User Requirements and Test Methods*.

### Truck Traffic Characterization Parameters

In order to characterize axle load spectra and to improve prediction of pavement distress, considerable traffic data are required in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) as compared to previous AASHTO pavement design procedures. Specifically, detailed truck traffic characterization is needed in the calculation of both asphalt and concrete pavement responses. The traffic data parameters, by input level, are summarized in Table 4-5.

Table 4-5. Traffic data requirements by input level ([[Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Initial two-way AADTT | 1 | Site specific WIM, AVC or traffic forecasting models |
| 2 | Regional WIM, AVC, vehicle counts or traffic forecasting models |
| 3 | National WIM, AVC, vehicle counts or traffic forecasting models |
| Trucks in thedesign direction | 1 | Site specific WIM, AVC or vehicle counts |
| 2 | Regional WIM, AVC or vehicle counts |
| 3 | National WIM, AVC or local vehicle counts/experience |
| Trucks in thedesign lane | 1 | Site specific WIM, AVC or vehicle counts |
| 2 | Regional WIM, AVC or vehicle counts |
| 3 | National WIM, AVC or local vehicle counts/experience |
| Operational speed | 1 | Direct measurement of site specific segment or calculate based on Highway Capacity Manual |
| 2 |
| 3 |
| Hourlydistribution | 1 | Site specific WIM, AVC or vehicle counts |
| 2 | Regional WIM, AVC or vehicle counts |
| 3 | National WIM, AVC or local vehicle counts/experience |
| Monthlyadjustment | 1 | Site specific WIM or AVC |
| 2 | Regional WIM or AVC |
| 3 | National WIM or AVC |
| Vehicle class distribution | 1 | Site specific WIM, AVC or vehicle counts |
| 2 | Regional WIM, AVC or vehicle counts |
| 3 | National WIM, AVC or local vehicle counts/experience |
| Traffic growth rate | 1 | Continuous or short duration AADTT counts |
| 2 |
| 3 |
| Axle load distribution factors | 1 | Site specific WIM or AVC |
| 2 | Regional WIM or AVC |
| 3 | National WIM or AVC |

Table 4-5. Traffic data requirements by input level ([continued](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Mean wheel location | 1 | Direct measurement of site specific segment |
| 2 | Regional/statewide average |
| 3 | National average or local experience |
| Traffic wander standard deviation | 1 | Direct measurement of site specific segment |
| 2 | Regional/statewide average |
| 3 | National average or local experience |
| Design lane width | 1 | Direct measurement of site specific segment |
| 2 |
| 3 |
| Number of axles per truck | 1 | Site specific WIM, AVC or vehicle counts |
| 2 | Regional WIM, AVC or vehicle counts |
| 3 | National WIM, AVC or local vehicle counts/experience |
| Axle, tire spacing, and tire pressure | 1 | Measure directly, obtain information from manufacturers, national average or local experience |
| 2 |
| 3 |

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) truck traffic inputs are further described below ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter4_Traffic.pdf), [NCHRP 2009](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/software.htm)).

#### Axle Configuration

The axle configurations are used for both characterizing user-defined gear loads (i.e., special truck/vehicle configurations) and for determining pavement responses that are sensitive to wheel and axle locations (e.g., analysis of JPCP). Axle characteristics to be considered include (see also Section 5.3.1 of the [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx)*™ Help System*):

* Average axle width (ft) – average distance between the outside wheels of an axle; the default value is 8.5 ft.
* Axle spacing (in.) – distance between consecutive axles; default values are 51.6 in. for tandem axles, 49.2 in. for tridem axles, and 49.2 in. for quad axles.
* Dual tire spacing (in.) – distance between the centers of a dual tire configuration; the default value is 12 in.
* Tire pressure (psi) – hot inflation pressure of the tire; the default value is 120 psi.

#### Axle Load Distribution Factors

Axle load distribution factors are the percent of the total axle applications for each load interval within an axle type (single, tandem, tridem, and quad) and vehicle class (FHWA Class 4 through 13). Within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), axle load distribution factors remain constant over the design period. The sum of axle load distribution percentages must equal 100 percent. Axle load distribution factors can only be determined for WIM data; however, default values for single, tandem, tridem, and quad axles, determined using the LTPP traffic database, are provided within the [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) software.

#### Design Lane Width

Design lane width (ft) is defined as the distance between the lane markings; a default value of 12 ft is used.

#### Initial Two-Way AADTT

The initial two-way average annual daily truck traffic is defined as the number of FHWA Class 4 to 13 vehicles in both directions during the first year of the design period.

#### Lateral Traffic Wander

Lateral traffic wander is the standard deviation of the transverse wander of the truck tire location, across the lane width (see Figure 4-1); a default value of 10 in. is used.



Figure 4-1. Lateral traffic wander.

#### Mean Wheel Location

The mean wheel location is the distance from the outer edge of the wheel load to the right pavement marking (see Figure 4-2); the default value is 18 in.



Figure 4-2. Mean wheel location.

#### Number of Axles per Truck

Within the software, truck traffic is quantified into one of four axle groups, single, tandem, tridem, and quad (for every month within the analysis period). In order to convert truck traffic volumes to axle type default values are used for each of the FHWA class of trucks (see Table 5.3.4 of the AASHTOWare[*Pavement ME Design™*](http://www.aashtoware.org/Pavement/Pages/default.aspx) *Help System*).

#### Number of Lanes in the Design Direction

The number of lanes in the design direction is defined as the number of lanes that carry truck traffic in the design direction. The number of lanes in the design direction is used to estimate the traffic distribution in each lane.

#### Vehicle Operational Speed

The vehicle operation speed is the expected speed of truck traffic in the design direction; a default value of 60 mph is used.

#### Percent of Trucks in Design Lane

The percent of trucks in the design lane is the percent of trucks in the design direction that are expected to travel in the design lane. This percentage is used to calculate the total number of trucks in the design lane.

#### Traffic Volume Adjustment Factors

Truck traffic is characterized according to each of the following factors:

* Monthly adjustment – this factor is used to adjust AADTT to monthly truck traffic; the default values are set to 1.0 for all vehicle class for all months (note that the sum must add up to 12 and are assumed to be constant over the design period).
* Vehicle class distribution – this is the distribution of vehicle class obtained from WIM, AVC, vehicle counts, or default values. Note that the sum of the vehicle class distribution factors must equal 100. Additionally, based on analysis of the LTPP Traffic database, truck traffic classification (TTC) groups have been identified to classify the traffic stream.
* Hourly distribution – this is the percent of average annual daily truck traffic (AADTT) per hour. Note that the sum of the hourly distribution factors must equal 100.
* Traffic growth rate – this is the rate of traffic growth relative to the base year of the design period. Three options are available in relation to growth rate: no growth rate, linear growth rate, and compound growth rate. In addition, the growth rate can be applied to the total volume or varied according to specific vehicle-classes.

#### Truck Traffic volume

Truck traffic volume is defined as the number of vehicles (FHWA Class 4 to 13) in the base year of the design period.

#### Wheelbase

The wheelbase is defined as the distance between the steer axle and first load axle and is needed to define only FHWA Class 8 through 13 vehicles. Wheelbase information is used in the determination of top-down cracking in JPCP. The following factors are used to define the vehicle wheelbase:

* Average axle spacing (ft) – average longitudinal distance between consecutive axles according to short (12 ft), medium (15 ft), and long (18 ft) axle spacings.
* Percentage of trucks (%) – percent of trucks with short, medium, and long axle spacing.

### Materials Characterization

Incorporating a mechanistic-empirical design procedure allows for the consideration of material properties and their impact on pavement responses (e.g., stress, strain, and deflection) in relation to environmental effects and truck traffic loading. In the mechanistic-empirical approach, materials are characterized according to elastic modulus (E) and Poisson’s ratio (μ). Other specific material parameters—such as strength, erodibility, plasticity, and gradation—are used where the material attributes influence the distress mechanism ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). In addition, material properties that affect the response to climatic conditions (e.g., moisture, temperature) can also be incorporated into the mechanistic-empirical model. Material properties that are influenced by climatic effects include, but not limited to, plasticity index, porosity, heat capacity, and coefficient of thermal expansion ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). The following provide details associated with asphalt, concrete, and stabilized mixtures, and unbound materials (base and subgrade).

#### Asphalt Mixtures

Asphalt mixtures currently analyzed in the mechanistic-empirical design procedure include dense-graded, open-graded, sand-asphalt, and recycled asphalt pavements ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). The design procedure is limited to these asphalt types since these were the only asphalt pavement types included in the LTPP database with sufficient data. Although the incorporation of other mix types is possible, additional testing, evaluation, and analysis are needed to adequately characterize material properties and to develop corresponding performance prediction equations.

Asphalt mixture characteristics required for input are shown in Table 4-6. Where different, Table 4-6 includes needed characteristics for both new and existing asphalt layers.

Table 4-6. Estimating new asphalt layer parameters ([[Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Dynamic Modulus, E\*(new asphalt) | 1 | AASHTO T342, *Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA)* |
| 2 | E\* predictive equation and AASHTO T 315, *Standard Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)* |
| 3 | E\* predictive equation using default values for Ai-VTS |
| Poisson'sratio, μ | 1 | Ideally estimated from laboratory testing, but not recommended until models that account for dilation effects are available |
| 2 | Regression equations and user entered values for parameters a and b |
| 3 | Agency historical data or default values |
| Tensilestrength, TS | 1 | AASHTO T 322, *Standard Method of Test for Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device* |
| 2 | AASHTO T 322 |
| 3 | Regression equation (input values obtained from testing or historical data) |

Table 4-6. Estimating new asphalt layer parameters (continued).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Creep compliance | 1 | AASHTO T 322 |
| 2 | AASHTO T 322 |
| 3 | Regression equation (input values obtained from testing or historical data) |
| Coefficient of thermal contraction, Lmix | 1 | No AASHTO or ASTM test procedures currently available, computed internally using asphalt volumetric properties |
| 2 |
| 3 |
| Surface shortwave absorptivity (EICM input) | 1 | No AASHTO tests available for paving materials, use default values |
| 2 |
| 3 |
| Thermal conductivity, K | 1 | ASTM E1952, *Standard Test Method for Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry* |
| 2 | No correlation currently available |
| 3 | Agency historical data or default values |
| Heatcapacity, Q | 1 | ASTM D2766, *Standard Test Method for Specific Heat of Liquids and Solids* |
| 2 | No correlation currently available |
| 3 | Agency historical data or default values |
| Effective binder content, Vbeff | 1 | AASHTO R 35, *Standard Practice for Superpave Volumetric Design for Hot Mix Asphalt (HMA)* |
| 2 | AASHTO R 35 |
| 3 | Agency historical data or default values |
| Complex shear modulus, G\* and phase angle, δ | 1 | AASHTO T 315 or conventional binder tests |
| 2 | AASHTO T 315 or conventional binder tests |
| 3 | Select Superpave binder, conventional viscosity or penetration grade |
| Air voids, Va(new asphalt) | 1 | AASHTO T 269, *Standard Method of Test for Percent Air Voids in Compacted Dense and Open Asphalt Mixtures* |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Air voids, Va(existing asphalt) | 1 | AASHTO T 209, *Standard Method of Test Theoretical Maximum Specific Gravity and Density of Hot Mix Asphalt (HMA)* |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Total unit weight | 1 | AASHTO T 166, *Standard Method of Test for Bulk Specific Gravity of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens* and AASHTO T 209 |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |

##### Dynamic Modulus of Existing Asphalt Layer

For rehabilitation, the procedure used in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) to determine the HMA dynamic modulus is similar to level 1; however, for the HMA layers to remain in place [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) also accounts for incurred damage during the life of the layer. Details for determining the master curve for existing HMA are as follows ([NCHRP 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)):

###### Level 1

1. Conduct FWD tests and determine the average backcalculated HMA modulus, Edam, over the project length.
2. Determine air void content, asphalt content, gradation, and asphalt viscosity parameters from cores. Determine binder viscosity-temperature properties following the same procedures as used for level 1.
3. Use the modified Witczak equation (see Equation 4-5) to develop an undamaged dynamic modulus master curve using the pavement temperature and testing frequency used during FWD testing.

 (4-5)

where:

 E\* = dynamic modulus, lbf/in2

 δ, α = fitting parameters that are dependent on aggregate gradation, binder content, and air void content; δ represents the minimum value of E\* and δ+α represents the maximum value of E\*

 β and γ = regression parameters that are dependent on the characteristics of the asphalt binder and the magnitude of δ, α

 tr = time of loading at reference temperature, sec

1. Estimate the fatigue damage by solving for dac in Equation 4-6.

 (4-6)

where:

 E\*dam = damaged modulus, lbf/in2 from step 1

 δ = fitting parameters that are dependent on aggregate gradation, binder content, and air void content; represents the minimum value of E\*

 E\* = undamaged modulus, lbf/in2 from step 3

 dac = fatigue damage in the HMA layer

1. Calculate α’= (1 – dac) α; where α is a function of mix gradation parameters.
2. Determine the field-damaged dynamic modulus master curve using α’ instead of α in Equation 4-7.

 (4-7)

where:

 tr = reduced time for NDT loading, sec

 t = NDT loading time, sec

 η = binder viscosity at the NDT test temperature

 ηTr = binder viscosity at 70°F

 (4-8)

where:

 η = binder viscosity

 A = viscosity temperature susceptibility intercept

 VTS = viscosity temperature susceptibility slope

 Tr = temperature, °Rankine

###### Level 2

FWD testing is not required for level 2; however, for level 2 field cores are used to determine the undamaged modulus and an estimate of fatigue damage is based on the pavement condition of the existing asphalt layer.

###### Level 3

Under level 3, FWD testing and coring is not required. [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) determines the undamaged master curve using user inputs for HMA mix volumetric and binder properties.

#### Concrete Mixtures

Concrete mixtures currently analyzed in the mechanistic-empirical design procedure include intact slabs and fractured slabs from crack and seat, break and seat and rubblization processes ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). As with asphalt pavements, incorporation of other mix types is possible; however, testing, evaluation, and analysis are needed to adequately characterize material properties and develop corresponding performance prediction equations. Concrete mixture input requirements are shown in Table 4-7.

Table 4-7. Estimating concrete layer parameters ([[Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Compressive strength, f’c | 1 | AASHTO T 22, *Standard Method of Test for Compressive Strength of Cylindrical Concrete Specimens* |
| 2 | AASHTO T 22 |
| 3 | AASHTO T 22, historical data, or default values |
| Flexural strength, (new concrete) | 1 | AASHTO T 97, *Standard Method of Test for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading* |
| 2 | Correlation based on f’c (AASHTO T 22) |
| 3 | Estimate using AASHTO T 22 or AASHTO T 97 or historical records |
| Flexural strength, (existing concrete)1 | 1 | AASHTO T 97, using prismatic beams obtained from existing pavement |
| 2 | Correlation based on f’c (AASHTO T 22) from field cores |
| 3 | Estimate using AASHTO T 22 or AASHTO T 97 or historical records |

1 JPCP restoration and bonded concrete overlay designs only.

Table 4-7. Estimating concrete layer parameters (continued).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Elasticmodulus, Ec | 1 | ASTM C469, *Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression* |
| 2 | Correlation based on f’c (AASHTO T 22) |
| 3 | Estimate using AASHTO T 22 or AASHTO T 97 or historical records |
| Poisson'sratio, μ | 1 | ASTM C469 |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Indirect tensile strength, ft(new concrete)2 | 1 | AASHTO T 198, *Standard Method of Test for Splitting Tensile Strength of Cylindrical Concrete Specimens* |
| 2 | Correlation based on f’c (AASHTO T 22) |
| 3 | Estimate using AASHTO T 22 or AASHTO T 97 or historical records |
| Indirect tensile strength, ft(existing concrete)2 | 1 | AASHTO T 198, using cores obtained from existing pavement |
| 2 | Estimate f’c (AASHTO T 22) from field cores, multiply by 0.67 |
| 3 | Estimate using AASHTO T 22 or AASHTO T 97 or historical records |
| Unit weight, ρ (new concrete) | 1 | AASHTO T 121, *Standard Method of Test for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete* |
| 2 | Not applicable |
| 3 | Agency historical data or default values |
| Unit weight, ρ (existing concrete) | 1 | AASHTO T 271, *Standard Method of Test for Density of Plastic and Hardened Portland Cement Concrete In-Place by Nuclear Methods* |
| 2 | Not applicable |
| 3 | Agency historical data or default values |
| Coefficient of thermal expansion (CTE) | 1 | AASHTO T336, *Standard Method of Test for. Coefficient of Thermal Expansion of Hydraulic Cement Concrete* |
| 2 | Correlation based on aggregate and paste CTE values |
| 3 | Agency historical data or default values |
| Thermal conductivity, K | 1 | ASTM E1952 |
| 2 | ASTM E1952 |
| 3 | Agency historical data or default values |
| Heatcapacity, Q | 1 | ASTM D2766 |
| 2 | ASTM D2766 |
| 3 | Agency historical data or default values |
| Ultimate shrinkage (at 40 percent relative humidity) | 1 | Current laboratory testing does not allow for feasible determination |
| 2 | Correlation based on concrete mix parameters |
| 3 | Correlation based on concrete mix parameters using historical records |
| Surface shortwave absorptivity (EICM input) | 1 | No AASHTO tests for paving materials |
| 2 | No AASHTO tests for paving materials |
| 3 | Default values |

2 CRCP only.

##### Elastic Modulus of Existing Concrete Layer

In rehabilitation design (i.e., unbonded concrete overlays and restoration), the existing slabs to remain in place are characterized according to their elastic modulus; however, the elastic modulus for existing slabs to be overlaid with unbonded concrete overlays are adjusted to account for damage caused over the life of the pavement ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). For slab restoration, since the elastic modulus does not increase significantly in aged concrete it is not considered in the design procedure ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)). The methodologies used to predict the elastic modulus for existing (intact) slabs is provided below for each input level ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)):

###### Level 1

1. Determine the elastic modulus (ASTM C469) from core or estimate from FWD testing (ETEST). The FWD testing should be conducted at mid-slab locations away from joints and cracks. The elastic modulus is estimated using the Best Fit method ([Khazanovich, Tayabji, and Darter 2001](http://www.tfhrc.gov/pavement/ltpp/pdf/00086.pdf)), and the results are then multiplied by 0.8 to obtain the static elastic modulus for uncracked concrete.
2. Assess the overall condition of the existing concrete layer considering various distress levels (see Table 4-8).
3. Based on the overall pavement condition, select a pavement condition factor, CBD (see Table 4-9).

Table 4-8. Recommended overall condition assessment of the existing
concrete layer ([[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| **Distress Type** | **FunctionalClass** | **Good** | **Moderate** | **Severe** |
| IRI (in/mi) | Interstate | < 100 | 100 – 175 | > 175 |
| Primary | < 110 | 110 – 200 | > 200 |
| Secondary | < 125 | 125 – 250 | > 250 |
| JPCP medium and high severity cracked slabs (%) | Interstate | < 5 | 5 – 10 | > 10 |
| Primary | < 8 | 8 – 15 | > 15 |
| Secondary | < 10 | 10 – 20 | > 20 |
| JRCP medium and high severity cracked slabs (no./ln-mi) | Interstate | < 15 | 15 – 40 | > 40 |
| Primary | < 20 | 20 – 50 | > 50 |
| Secondary | < 25 | 25 – 60 | > 60 |
| JPCP mean faulting (in.) | Interstate | < 0.10 | 0.10 – 0.150 | > 0.15 |
| Primary | < 0.125 | 0.125 – 0.20 | > 0.20 |
| Secondary | < 0.15 | 0.15 – 0.30 | > 0.30 |
| JRCP mean faulting (in.) | Interstate | < 0.15 | 0.15 – 0.30 | > 0.30 |
| Primary | < 0.175 | 0.175 – 0.35 | > 0.35 |
| Secondary | < 0.20 | 0.20 – 0.40 | > 0.40 |
| CRCP medium to high severity punchouts (no./ln-mi) | Interstate | < 5 | 5 – 10 | > 10 |
| Primary | < 8 | 8 – 15 | > 15 |
| Secondary | < 10 | 10 – 20 | > 20 |

Table 4-9. Recommended concrete pavement condition factor ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)).

|  |  |
| --- | --- |
| **PavementCondition1** | **CBD** |
| Good | 0.42 – 0.75 |
| Moderate | 0.22 – 0.42 |
| Severe | 0.042 – 0.22 |

 1 see Table 2.5.15, [[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm).

1. Determine the concrete design modulus by adjusting ETEST to account for the existing pavement condition using Equation 4-9.

 (4-9)

where:

 EBASE/DESIGN = design modulus, psi

 CBD = overall pavement condition factor

 ETEST = static elastic modulus of existing concrete, psi

###### Level 2

1. Determine the uncracked concrete compressive strength from cores using AASHTO T 22.
2. Estimate the mean uncracked concrete elastic modulus using Equation 4-10.

 (4-10)

where:

 ETEST = mean uncracked elastic modulus, psi

 ρ = concrete unit weight, lb/ft3

 f’c = concrete compressive strength, psi

Select CBD factor (Table 4-9) based on the overall condition of the existing concrete layer and multiply by ETEST to determine the design modulus of the existing concrete.

###### Level 3

Based on the overall condition, select a typical modulus value (see Table 4-10).

Table 4-10. Recommended modulus range based on concrete pavement
condition factor (modified from [[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

|  |  |
| --- | --- |
| **PavementCondition** | **Typical Modulus, psi** |
| Good | 3 – 4 x 106 |
| Moderate | 1 – 3 x 106 |
| Severe | 0.3 – 1 x 106 |

 1 see Table 2.5.15 [[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm).

For fractured slabs (crack and seat, break and seat, and rubblized), level 1 and level 2 inputs are not applicable. Recommended modulus values for level 3 inputs are shown in Table 4-11.

Table 4-11. Recommended modulus range for fractured concrete slabs ([[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

|  |  |
| --- | --- |
| **Fracture Type** | **Typical Modulus, psi** |
| Crack or break and seat | 300,000 – 1,000,000 |
| Rubblized | 50,000 – 150,000 |

#### Stabilized Mixtures

Stabilized mixtures currently analyzed in the mechanistic-empirical design procedure include lean concrete, cement stabilized, open-graded cement stabilized, soil cement, lime-cement flyash, and lime-treated materials ([[[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter2_Materials.pdf)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)). As previously noted, incorporation of other mix types is possible, but extensive testing, evaluation, and analysis is needed to adequately characterize material properties and develop corresponding performance prediction equations. Stabilized mixture input requirements are summarized in Table 4-12.

Table 4-12. Estimating stabilized layer parameters ([[Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Elastic modulus, E(lean concrete & cement treated aggregate) | 1 | ASTM C469 or backcalculated from FWD testing |
| 2 | Correlation based on f’c (AASHTO T 22) |
| 3 | Agency historical data or default values |
| Elastic modulus, E(open-grade cement stabilized) | 1 | No AASHTO or ASTM test procedure available at this time |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Elastic modulus, E(lime-cement, flyash) | 1 | No AASHTO or ASTM test procedure available at this time |
| 2 | Correlation based on unconfined compressive strength (ASTM C593, *Standard Specification for Fly Ash and Other Pozzolans for Use with Lime for Soil Stabilization*) |
| 3 | Agency historical data or default values |

Table 4-12. Estimating stabilized layer parameters (continued).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Elastic modulus, E(soil cement) | 1 | No AASHTO or ASTM test procedure available at this time |
| 2 | Correlation based on unconfined compressive strength (ASTM D1633, *Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders*) |
| 3 | Agency historical data or default values |
| Resilient modulus, Mr(lime-stabilized soils) | 1 | Mixture Design and Testing Protocol ([Little 2000](http://www.lime.org/documents/publications/free_downloads/soils-aggregates-vol-3.pdf)) and AASHTO T 307, *Standard Method of Test for Determining the Resilient Modulus of Soils and Aggregate Materials* or backcalculated from FWD testing |
| 2 | Correlation based on unconfined compressive strength (ASTM D5102, *Standard Test Methods for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures*) |
| 3 | Agency historical data or default values |
| Elastic modulus, E(asphalt pavement) | 1 | Agency historical data or default values for deteriorated chemically stabilized materials |
| 2 |
| 3 |
| Flexural strength, (lean concrete & cement treated aggregate)(asphalt pavement) | 1 | AASHTO T 97 |
| 2 | AASHTO T 22 |
| 3 | Agency historical data or default values |
| Flexural strength, (open-graded cement stabilized)(asphalt pavement) | 1 | No current AASHTO or ASTM tests available |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Flexural strength, (lime-cement, flyash)(asphalt pavement) | 1 | AASHTO T 97 |
| 2 | ASTM C593 |
| 3 | Agency historical data or default values |
| Flexural strength, (soil cement)(asphalt pavement) | 1 | ASTM D1635, *Standard Test Method for Flexural Strength of Soil-Cement Using Simple Beam with Third-Point Loading* |
| 2 | ASTM D1633 |
| 3 | Agency historical data or default values |
| Flexural strength, (lime-stabilized soils)(asphalt pavement) | 1 | No current AASHTO or ASTM tests available |
| 2 | ASTM D5102 |
| 3 | Agency historical data or default values |
| Poisson’s ratio | 1 | Agency historical data or default values |
| 2 |
| 3 |
| Thermal conductivity, K | 1 | ASTM E1952 |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |
| Heat capacity, Q | 1 | ASTM D2766 |
| 2 | No correlation currently exists |
| 3 | Agency historical data or default values |

#### Unbound and Subgrade Materials

Unbound and subgrade materials in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) are characterized according to AASHTO and/or the unified soil classification systems. Unbound and subgrade material input requirements are shown in Table 4-13.

Table 4-13. Characterizing unbound layer and subgrade inputs ([[Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Poisson'sratio, μ | 1 | Low sensitivity to structural response does not warrant laboratory testing |
| 2 | No recommended correlation procedures |
| 3 | Agency historical data or default values |
| Resilient Modulus, Mr | 1 | AASHTO T 307 or backcalculated from FWD testing |
| 2 | Correlations to Mr based on CBR, R-value, or AASHTO layer coefficients, or correlations of PI and gradation, and DCP to CBR |
| 3 | Agency historical data or default values (which are very approximate) |
| CBR | 1 | AASHTO T 193, *Standard Method of Test for The California Bearing Ratio* |
| 2 | No recommended correlation procedures |
| 3 | Agency historical data or default values |
| R-value | 1 | AASHTO T 190*, Standard Method of Test for Resistance R-Value and Expansion Pressure of Compacted Soils* |
| 2 | No recommended correlation procedures |
| 3 | Agency historical data or default values |
| DCP | 1 | ASTM D6951, *Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications* |
| 2 | No recommended correlation procedures |
| 3 | Agency historical data or default values |
| Coefficient of lateral pressure, k0 | 1 | No AASHTO or ASTM test procedure currently available |
| 2 | Computed internally using Poisson’s ratio and effective angle of internal friction |
| 3 |
| Gradation(EICM input) | 1 | AASHTO T 27 |
| 2 |
| 3 |
| Plasticity index (EICM input) | 1 | AASHTO T 90, *Standard Method of Test for Determining the Plastic Limit and Plasticity Index of Soils* |
| 2 |
| 3 |
| Liquid limit(EICM input) | 1 | AASHTO T 89, *Standard Method of Test for Determining the Liquid Limit of Soils* |
| 2 |
| 3 |
| Max. Dry Unit Weight, γdmax(EICM input) | 1 | AASHTO T 99 (or T 180), *Standard Method of Test for Moisture-Density Relations of Soils Using a 2.5-kg (5.5-lb) Rammer and a 305-mm (12-in.) Drop* |
| 2 | Computed internally using Plasticity Index and gradation |
| 3 | Not applicable |
| Moisture content, *w*opt (EICM input) | 1 | AASHTO T 99 (or T 180) |
| 2 | Computed internally using Plasticity Index and gradation |
| 3 | Not applicable |

Table 4-13. Characterizing unbound layer and subgrade inputs (continued).

|  |  |  |
| --- | --- | --- |
| **Variable** | **Level** | **How to acquire and/or measure** |
| Specific gravity of solids, Gs(EICM input) | 1 | AASHTO T 100, *Standard Method of Test for Specific Gravity of Soils* |
| 2 | Computed internally using Plasticity Index and gradation |
| 3 | Not applicable |
| Saturated Hydraulic Conductivity, ksat (EICM input) | 1 | AASHTO T 215*, Standard Method of Test for Permeability of Granular Soils (Constant Head)* |
| 2 | Computed internally using Plasticity Index and gradation |
| 3 | Not applicable |
| Degree of Saturation, Sopt (EICM input) | 1 | Computed internally using Plasticity Index and gradation |
| 2 |
| 3 |

### Environmental Effects

Many pavement materials (e.g., asphalt, unbound base, subgrade soil) are affected by climatic conditions (e.g., depth to water table, freeze-thaw cycles, precipitation, temperature), which makes the accurate characterization of these effects essential in predicting pavement performance. For example ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)):

* The modulus of asphalt mixtures is dependent on temperature, with lower values experienced under warmer temperatures and higher values experienced under colder temperatures. Excessive moisture can lead to stripping in the asphalt mixture.
* Cementitious materials experience very little modulus changes with normal temperature ranges, but can exhibit significantly higher stresses under temperature and moisture gradients. The presence of moisture can also result in degradation of the cementitious layer.
* Freeze-thaw effects, seasonal modulus variability, and the development of ice lenses in fine-grained soils affect material behavior and resulting pavement performance.
* Weaker stiffness with saturated and oversaturated unbound materials influence the way that materials respond to traffic loadings.

Environmental effects related to temperature and moisture profiles are included in the pavement design process through the use of the Enhanced Integrated Climatic Model ([ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)). In addition, as part of NCHRP 9-23A, [*Implementing a National Catalog of Subgrade Soil-Water Characteristic Curve (SWCC) Default Inputs for Use with the MEPDG*](http://www.trb.org/Main/Blurbs/163721.aspx),” a national database was developed that contains subgrade material soil properties needed for use with [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) ([Zapata 2010](http://www.trb.org/Main/Blurbs/163721.aspx)). The national database focuses on key parameters for implementation of level 1 environmental analysis, but also includes soil properties need for all hierarchical levels ([Zapata 2010](http://www.trb.org/Main/Blurbs/163721.aspx)). A summary of input data contained (not a complete list) in the database includes ([Zapata 2010](http://www.trb.org/Main/Blurbs/163721.aspx)):

* AASHTO soil classification.
* AASHTO group index (e.g., clayey-granular, silty-clay).
* Gradation (percent passing No. 4, No. 10, No. 40, and No. 200).
* Estimated clay content.
* Liquid limit.
* Plasticity index.
* Soil-water characteristic curve fitting parameters (a, n, m, hr).
* CBR.
* Mr.
* Elevation.
* Depth to bedrock.
* Depth to water table.

#### Enhanced Integrated Climatic Model

The original version of the Enhanced Integrated Climatic Model (EICM) was developed for the Federal Highway Administration by the Texas Transportation Institute (Lytton et al. 1990). That original version received additional modifications and updates in 1997 (Larson and Dempsey 1997). Additional improvements to the moisture prediction capabilities occurred as part of NCHRP 1-37A and ultimately became the EICM version that was incorporated into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx). The EICM is described as ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)):

“…a one-dimensional coupled heat and moisture flow program that simulates changes in the behaviour and characteristics of pavement and subgrade materials in conjunction with climatic conditions over several years of operation.”

The EICM is used to compute and predict pavement temperature, resilient modulus adjustment factors, pore water pressure, water content, frost and thaw depths, frost heave, and drainage performance over the entire pavement and subgrade profile ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)).

#### EICM Inputs

As previously noted, required inputs for the EICM are incorporated into the applicable material characterization portions of the software. Climatic weather station data (hourly air temperature, hourly precipitation, hourly wind speed, hourly percentage sunshine, and hourly relative humidity) are also required for the analysis and are currently available for download at the NCHRP website ([NCHRP 2009](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/climatic_state.htm)). The EICM computations are performed internally within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) and the outputs sent to the applicable software modules (material characterization, structural response, and performance prediction), so no user interaction is required.

Specific inputs required by the EICM include the following ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)):

* Month and year of the base/subgrade completion – used to determine when the moisture model for asphalt pavements should be initiated.
* Month and year of existing pavement construction – used to estimate the aging of the asphalt layer or the strength and modulus of the concrete layer at the time of rehabilitation.
* Month and year of new pavement construction – used to estimate the stiffness and strength parameters of asphalt pavements, estimate the zero-stress temperature of the concrete layer at the time of construction, and estimate the relative humidity for CRCP in the determination of initial crack spacing and crack width.
* Month and year pavement is opened to traffic – defines initial climate conditions.
* Design type – determines the type of climate inputs required for input.
* Latitude, longitude, and elevation – used to establish the location of the project in relation to the interpolation of surrounding weather station data. [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) will show the six closest weather stations to the input location.
* Depth to groundwater table (ft) – preferably determined from borings for level 1 and estimated for level 3. The depth to groundwater table significantly impacts the prediction accuracy of layer modulus values; therefore, this value should be determined as accurately as possible.

#### EICM Outputs

Internal outputs of the EICM include ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)):

* Asphalt bound layers
	+ Surface and mid-depth temperatures of sublayers every month or every two weeks (reporting rate is increased during freeze-thaw period). Surface and mid-depth temperatures are used in the prediction models for fatigue cracking and permanent deformation (rutting).
	+ Temperatures at the surface and at 1-in. increments. These temperatures are used in the prediction of thermal cracking.
* Concrete layers
	+ Temperature profile every hour. This is used in the prediction of cracking and faulting in JPCP and punchouts in CRCP.
	+ Number of freeze-thaw cycles and freezing index. This is used in predicting pavement performance in JPCP.
	+ Relative humidity every month. This is used for modeling moisture gradients in JPCP and CRCP.
* Unbound layers
	+ Resilient modulus adjustment factors as a function of position within pavement structure and time. These factors are used to account for seasonal variation in the resilient modulus of unbound and subgrade layers.
	+ Volumetric moisture content. This is used in the calculation of the rutting in the unbound layers.

Details related to the internal computations and interaction of the EICM can be found in Part 2, Chapter 3 of the NCHRP 1-37A Project Final Report ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf)).

### Design Methodology

The design methodology used within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is based on determining the predicted pavement distress (e.g., rutting, cracking, faulting, IRI) for a given traffic level, pavement section (thickness and material types), axle load spectra, and so on. The predicted distress is then compared to the critical distress level and reliability specified by the designer. The general design methodology is illustrated in Figure 4-3.



Figure 4-3. General design methodology (redrawn from [AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)) Used by permission.

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) currently has two methods for evaluating and determining layer thickness:

* **Analysis Tool Approach**. In the analysis tool approach, the designer enters an estimated thickness for each pavement layer (and all other applicable inputs related to climate, material, traffic, reliability, and so on) and the software predicts the applicable pavement distress. If the predicted distress exceeds the critical distress criteria, the designer modifies the design by changing the layer thickness, material type, material properties, or other design features (e.g., mix design, joint spacing), and reruns [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) until the predicted distress is equal to or less than the critical distress criteria.
* **User Input Approach**. In the user input approach, which is a new feature included with the release of [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), the user enters all necessary inputs, the software fixes all input values and then internally iterates layer thickness until the predicted distress is equal to or less than the critical distress criteria.

The methodology used for predicting distress is exactly the same for the two procedures, but the analysis tool approach provides the designer the flexibility to change more than just the layer thickness to achieve the appropriate performance criteria. It must be noted that in the user input approach, layer thickness alone may not be sufficient to meet traffic and environmental loadings. For example, the analysis may result in an unreasonably thick asphalt or concrete layer that is not economically feasible or may have issues with constructability.

### Incremental Damage Approach

In the incremental damage approach, the actual number of wheel load applications (n) is divided by the allowable number of wheel load applications (N) for a specified axle load and type, the idea being that as the wheel load applications approaches the allowable wheel load applications, damage (in terms of cracking) will occur. The incremental damage indices for all axle loads and types are summed for each month (semi-monthly during freeze and thaw periods) over the analysis period ([[AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/part_12_cover_ack_toc.pdf)). The incremental damage approach allows for consideration of changes in material parameters (e.g., aging effects, layer moduli, degree of saturation), changes in temperature and moisture, changes in joint openings, and so on over the analysis period, in essence reflecting field performance.

### Design Reliability

The reliability methodology incorporated into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) accounts for the variability in design inputs (e.g., traffic, climate, materials, performance). Unlike previous AASHTO pavement design procedures that defined reliability in terms of traffic loading, [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) requires the pavement designer to assign a reliability level for each pavement distress type (including IRI).

Reliability is defined as the probability that the pavement distress will be less than the critical level of distress over the design period (see Equation 4-11).

 Reliability = Probability (Predicted Distress < Critical Distress) (4-11)

This is further illustrated in Figure 4-4, where the mean distress is predicted over the intended pavement design life using the average values (i.e., 50 percent reliability) for all inputs ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)). As shown in Figure 4-4 all pavement distresses, excluding IRI, are set to zero at zero load applications. The initial IRI will result in an offset (upward adjustment) of the 50 percent reliability prediction curve shown in Figure 4-4.



Figure 4-4. Reliability concept (redrawn from [AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)) Used by permission.

Reliability is dependent on the standard error of each distress (including IRI) prediction model as determined from the calibration results. The error in prediction is assumed to be normally distributed for all pavement distresses. The mean value of the distress is increased by the number of standard errors associated with the reliability level.

The desired reliability level for each distress type is further described by Equation 4-12.

 (4-12)

where:

 *DistressR* = distress level at specified reliability level

 *DistressMean* = distress prediction using mean inputs and 50 percent reliability

 *STD* = standard deviation of distress prediction using mean inputs

 *ZR* = standardized normal deviate (mean 0 and standard deviation 1) corresponding to the specified reliability level

Recommended reliability levels are shown in Table 4-14. Due to the potential for increased pavement thickness (and resulting higher construction costs), reliability levels greater than 96 percent are not recommended at this time ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)).

Table 4-14. Recommended reliability levels by functional class ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)) Used by permission.

|  |  |
| --- | --- |
| **Functional Class** | **Reliability Level** |
| **Urban** | **Rural** |
| Interstate | 95 | 95 |
| Principal Arterial | 90 | 85 |
| Collector | 80 | 75 |
| Local | 75 | 70 |

### Local Calibration

The performance prediction models contained within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) have been developed and calibrated on a national basis using data obtained from the LTPP database and from various test roads (e.g., MnROAD, FHWA ALF). It is important to note that the characterization of measured distress is based on the distress definitions contained in the [LTPP Distress Identification Manual](http://www.tfhrc.gov/pavement/ltpp/reports/03031/03031.pdf). Therefore, the calibration process may also need to include an assessment of the impact of possible differences in distress definitions between the [LTPP Distress Identification Manual](http://www.tfhrc.gov/pavement/ltpp/reports/03031/03031.pdf) and agency procedures.

Each highway agency should conduct a validation analysis to ensure that the performance prediction models reflect field performance. Bias can be removed and model prediction improved through local calibration. The general calibration process is shown in Figure 4-5 (AASHTO 2010, [Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).



Figure 4-5. General calibration flowchart ([Pierce et al. 2010](http://www.fhwa.dot.gov/pavement/management/pubs/hif11026/index.cfm)).

1. **Select hierarchical input level**. The selection of the hierarchical input level is an agency by agency decision, based largely on the characteristics of the project and the availability of data. The selected hierarchical input level can be the same for all inputs, but preferably is specified individually for each input parameter. The latter is preferable since it allows agencies the flexibility to determine the level of effort needed in the data collection process. For example, a given agency may already have Level 1 traffic data, but only Level 2 material property data. In that example, it is more beneficial to match the selected hierarchical level based on the availability of data and not on a standard level for all inputs.
2. **Develop experimental plan and sampling template**. The intent of this step of the calibration process is to ensure the selection of pavement section samples are representative of the agency’s standard specifications, construction and design practices, and materials. In this manner, an agency selects pavement sections that are based on current design or construction practices (e.g., asphalt mixtures designed using Superpave rather than Hveem or Marshall Mix designs). In addition, to improve the statistical significance of the calibration process, selected pavement sections should also encompass performance data that extend over the entire pavement design life. For example, if a pavement design is evaluated over a 20-year period, the selected pavement sections used in the calibration process should include 20 years of pavement performance data.
3. **Estimate sample size**. To have the results of the calibration process to be statistically meaningful, the needed number of pavement sections, by distress type, must be determined (see Table 4-15). The same test sections should be used for all distress types since the distress transfer functions are uncoupled (occurrence and magnitude is independent of other distress types) within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx). The intent is to minimize both the bias (which distorts the prediction of actual observations) and precision (repeatability of estimates).

Table 4-15. Recommended minimum sample size.

|  |  |
| --- | --- |
| **Distress** | **Number of roadwaysegments** |
| Total rutting or faulting | 20 |
| Load related cracking | 30 |
| Non-load related cracking | 26 |
| Reflection cracking (asphalt pavements only) | 26 |
| IRI | > 501 |

1 Since IRI predictions are dependent on other distresses, a larger number of roadway sections may be required.

1. **Select roadway segments**. This step includes the selection of roadway segments based on the availability of existing data. To minimize costs, agencies should select representative pavement sections that require minimal field sampling and testing. Agencies should also select replicate pavement sections to be used during the validation process. Selected roadway segments should include:
* Only a few structural layers and material types.
* Segments with and without overlays to allow for calibration of both new and rehabilitated pavement performance prediction models.
* Non-conventional mixes or layers (e.g., warm mix, stone matrix asphalt, open-graded friction courses, and high strength concrete mixtures).
* Selected roadway segments should have at minimum of three pavement condition surveys over a 10-year period.
1. **Evaluate project and distress data**. This step validates that all selected roadway segments have the needed data, that all data are in the proper format (e.g., distress data is in accordance with the [LTPP Distress Identification Manual](http://www.tfhrc.gov/pavement/ltpp/reports/03031/03031.pdf)), and that performance data are available over the pavement design life. Furthermore, the data are checked for anomalies/outliers and for their hierarchical level.
2. **Conduct field testing and forensic investigation**. As needed, field sampling and testing may be required to obtain any missing data elements. For example, the asphalt layer performance prediction models include a rutting model that predicts the rut depth within the asphalt layer, the unbound layer, and the total pavement section; however, only total rut depth was recorded on the LTPP pavement sections. Similarly, load-related cracking models for asphalt pavements and transverse slab cracking for concrete pavements include a top-down and bottom-up component, yet this information was not included in the LTPP data collection process. Therefore, ideally, to improve the calibration process, both trenching of asphalt pavements (to confirm rut depth in bound and unbound layers) and coring of asphalt and concrete pavements (to confirm cracking initiation location) is recommended to better define these factors.
3. **Assess local bias**. In this step the predicted pavement performance is compared to the field performance and the bias and the standard error are determined (using the null hypothesis).
4. **Eliminate local bias**. If the null hypothesis is rejected and a significant bias exists, then steps should be taken to eliminate the bias by adjusting the calibration coefficients.
5. **Assess standard error of the estimate**. In this step, the standard error from the local calibration process is compared to the standard error from the global data set contained within [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx).
6. **Reduce standard error of the estimate**. If the standard error is determined to be too large, revisions to either the local calibration coefficients or the statistical model may be needed.
7. **Interpretation of the results**. Finally, in this step the reasonableness of the predicted pavement distress, at a given reliability level, can be determined by comparing the predicted pavement distress to actual pavement distress contained within the pavement management system.

## Design of New and Reconstructed Asphalt Pavements

Based on the availability of data contained within the LTPP database, the following new and reconstructed asphalt pavement types can be analyzed and/or designed using [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)):

* Conventional, thin dense-graded asphalt layer over granular base/subbase.
* Deep-strength, thick dense-graded asphalt layer over granular base.
* Full-depth, dense-graded asphalt layer and base course placed directly over subgrade.
* Semi-rigid, dense-graded asphalt layer over a chemically stabilized layer

Asphalt pavements included in the performance prediction models primarily included dense-graded asphalt. Although the analysis process can include other mix types, such as stone matrix asphalt (SMA), polymer modified mixes, or mixtures including recycled asphalt pavement (RAP), they should be characterized using level 1 inputs. In addition, unless the performance of other mix types has been calibrated to local conditions, the resulting prediction performance should be scrutinized.

### Asphalt Pavement Design Methodology

Two asphalt pavement design methodologies have been incorporated into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx), nonlinear finite element theory and multilayer elastic theory ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter3_Flexible%20Design.pdf)).

* **Nonlinear finite element theory**. The DSC2D finite element code is used for the determination of pavement stresses, strains, and deflections in nonlinear unbound materials. However, use of the finite element process (i.e., level 1) is not recommended at this time due to lack of calibration and validation of the structural response models using this analysis approach.
* **Multilayered elastic theory**. The JULEA multilayered elastic theory program is used for linear elastic materials in the determination of stresses, strains, and deflections. Critical response locations are shown in Figure 4-6 and include:
1. Tensile horizontal strain at the top of the asphalt layer, used to determine fatigue cracking in the asphalt layer.
2. Compressive vertical stress/strain at mid-depth of each asphalt layer, used to determine rutting in the asphalt layer.
3. Tensile horizontal strain at a depth of 0.5 in. from the asphalt layer surface and at the bottom of each bound or stabilized layer, used to determine fatigue cracking in the bound layers.
4. Compressive vertical stress/strain at mid-depth of each unbound base/subbase layer, used to determine rutting of the unbound layers. Rutting in chemically stabilized base/subbase layers, bedrock, and concrete fractured slab materials is assumed to be zero.
5. Compressive vertical stress/strain at the top of the subgrade and 6.0 in. below the top of the subgrade, used to determine subgrade rutting.



Figure 4-6. Critical asphalt pavement response locations.

### Asphalt Pavement Performance Models

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is capable of predicting the following asphalt pavement distresses ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)):

* Rut depth, in. (see AASHTO 2008, paragraph 5.2.2).
* Load related cracking: alligator cracking, percent cracked per section length and longitudinal cracking, ft/mile (see AASHTO 2008, paragraph 5.2.3).
* Transverse cracking, ft/mile (see AASHTO 2008, paragraph 5.2.4).
* Reflection cracking, percent area (see AASHTO 2008, paragraph 5.2.5)
* IRI, in/mile (see AASHTO 2008, paragraph 5.2.6).

At this time models for predicting reflective cracking have not been included in [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx). Reflective cracking models have been developed as part of NCHRP 1-42, [*Models for Predicting Reflection Cracking of Hot-Mix Asphalt Overlays*](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_669.pdf) (Lytton et al. 2010); however, acceptance of these models by AASHTO has yet to be conducted.

### Factors Affecting Distress Prediction in Asphalt Pavements

The following includes a list of design features, by distress type, that may be modified to improve the performance of the recommended pavement section ([ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

#### Fatigue Cracking in Asphalt Layers

* Increase the asphalt layer thickness.
* Increasing the Vbeff and lowering the Va will result in a reduction of fatigue cracking (must be balanced with the potential increase in rutting of the asphalt layer).
* For alligator cracking, increase the asphalt binder stiffness for the asphalt layer thicknesses greater than 3 to 5 in. For thinner asphalt pavements, reducing the asphalt layer stiffness will tend to result in lower levels of fatigue cracking.
* For longitudinal cracking, reduce the asphalt binder stiffness. This is in direct conflict with the methodology for reducing alligator cracking and the designer must determine the balance between these two forms of fatigue cracking. Work conducted under NCHRP 1-42a, *Models for Predicting Top-Down Cracking of Hot-Mix Asphalt Layers*, identified two models (viscoelastic continuum damage and asphalt mixture fracture mechanics) for addressing top-down cracking in mechanistic-empirical design. However, additional work is needed to evaluate damage zone effects on performance prediction, to verify material property sub-models, and to validate and calibrate the models over a broader range of pavements and environmental conditions ([Roque et al. 2010](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w162.pdf)).
* To minimize the amount of alligator cracking, increase the stiffness of the underlying layers. This can be accomplished by increasing the quality of the base/subbase layers (i.e., layer stiffness and/or thickness of a higher quality base/subbase layers) and/or increasing the subgrade stiffness.
* Reduce the presence of moisture in the base/subbase layers and subgrade (e.g., incorporate subsurface drainage, lower the ground water table, and/or raise the road grade).

#### Fatigue Cracking in Chemically Stabilized Layers

* Increase the thickness of the chemically stabilized layer.
* Place the chemically stabilized layer deeper in the pavement system.
* One of the primary issues with use of a chemically stabilized layer is the potential for reflective cracking in the overlaying asphalt layers. Increasing the asphalt layer thickness and/or including an effective crack relief system may reduce the potential of reflective cracking. NCHRP 1-42, [*Models for Predicting Reflection Cracking of Hot-Mix Asphalt Overlays*](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_669.pdf), has been completed (Lytton et al. 2010); however, the results of this study have yet to be incorporated into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx).

#### Thermal Cracking

* Reduce the asphalt mixture stiffness by using a lower asphalt binder grade.
* Increase the asphalt layer thickness.

#### Rutting

* In the asphalt layers
	+ Increase the asphalt mixture dynamic modulus. The asphalt mixture dynamic modulus can be increased by using a stiffer binder grade or using less asphalt binder; however, adherence to field compaction requirements must be met.
	+ Increasing the asphalt layer thickness may result in only a slight reduction in predicted rut depth. Increasing the asphalt layer thickness will result in a reduction in the amount of rutting in the unbound layers and subgrade.
	+ Since the majority of asphalt pavement rutting occurs in the upper 5 in., increasing the thickness of a poorer quality material will not minimize the amount of asphalt pavement rutting. The use of a higher quality asphalt layer in the upper lifts should be considered.
* In the base (and subbase) layers
	+ Improve the quality (i.e., increase the CBR, R-value, Mr) of the unbound layer.
	+ Use a chemically stabilized layer.
	+ In general, increasing the thickness of a poor quality material will result in an increase in predicted rut depth.
	+ Increasing the thickness of a good quality base/subbase layer will reduce subgrade rutting.
* In the subgrade layers
	+ Increase the overall stiffness of the entire pavement structure (asphalt mixture, base/subbase layers).
	+ Increase the thickness of the asphalt and base/subbase layers.
	+ To decrease the effects of in-situ moisture, consider subgrade treatments (e.g., lime, cement), subsurface drainage systems, geotextiles, and possibly increasing road grade elevation.

## Design of New and Reconstructed Concrete Pavements

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is able to evaluate and/or design both new and reconstructed JPCP and CRCP ([AASHTO 2008a](https://bookstore.transportation.org/item_details.aspx?ID=1249)). For the design of new and reconstructed JPCP, a number of design features can be evaluated including joint spacing (including random), joint sealant type (no sealant, liquid sealant, silicone sealant, or preformed), dowel bars, dowel bar diameter and spacing, tied concrete shoulders, and widened slabs. For CRCP, design features include shoulder type (asphalt, tied concrete, or gravel), percent steel, bar diameter, and steel depth.

### Concrete Pavement Design Methodology

Finite element analysis is a proven tool for the analysis of concrete pavement responses (stress and deflections) subjected to traffic and environmental loadings ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter4_Rigid%20Design.pdf)). In order to reduce the computational time needed to conduct a finite element analysis a series of neural networks using ISLAB2000 was developed and incorporated into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) for the determination of concrete stresses and deflections.

Critical response locations for JPCP are shown in Figure 4-7 and include ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter4_Rigid%20Design.pdf)):

1. Tensile stress at the top of the slab, high negative temperature gradient (top of slab cooler than bottom of slab), and heavy trucks with certain axle spacing (e.g., multi-trailer with relatively short trailer-to-trailer axle spacing) to determine top-down transverse cracking.
2. Tensile bending stress at the bottom of the slab with a single axle load at mid-slab location to determine bottom-up transverse cracking.
3. Differential deflection across the transverse joint to determine joint faulting.



Figure 4-7. Critical JPCP response locations.

The critical response location for CRCP is shown in Figure 4-8 and includes the tensile stress at the top of the slab when a truck passes near the horizontal edge between two closely spaced transverse cracks, used for determining punchout potential.



Figure 4-8. Critical CRCP response locations.

### JPCP Performance Models

For JPCP pavements, [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is capable of predicting the following distresses ([AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)):

* Transverse cracking, percent of slabs (see AASHTO 2008, paragraph 5.3.1).
* Mean transverse joint faulting, in. (see AASHTO 2008, paragraph 5.3.2).
* IRI, in/mile (see AASHTO 2008, paragraph 5.3.4).

### CRCP Performance Models

For CRCP pavements, [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is capable of predicting the following distresses:

* Number of punchouts, no./mile (see AASHTO 2008, paragraph 5.3.3).
* IRI, in/mile (see AASHTO 2008, paragraph 5.3.5).

### Factors Affecting Distress Prediction in Concrete Pavements

The following includes a list of design features, by distress type, that may be modified to improved performance of the recommended pavement section ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter4_Rigid%20Design.pdf)).

#### JPCP Transverse Cracking

* Increase slab thickness (bottom-up and top-down cracking).
* Reduce joint spacing (bottom-up and top-down cracking).
* Use widened slab (bottom-up and top-down cracking).
* Incorporate concrete mix with lower CTE (bottom-up and top-down cracking).
* Use tied concrete shoulder (bottom-up and top-down cracking).
* Increase concrete mix strength (bottom-up and top-down cracking).
* Use a stabilized base (bottom-up cracking).
* Reduce built-in curl after placement (top-down cracking).

#### JPCP Faulting

* Use dowel bars at transverse joints.
* Increase dowel bar diameter.
* Use widened slab.
* Provide a less erodible base.
* Use shorter joint spacing.
* Improve quality of coarse aggregate.
* Provide tied concrete shoulder.
* Improve subdrainage (especially when non-doweled pavements are used).

#### CRCP Punchouts

* Increase the longitudinal steel content.
* Place reinforcement above mid-depth.
* Incorporate concrete mix with lower CTE.
* Provide tied concrete shoulder.
* Use a stabilized base.
* Reduce built-in curl after placement.
* Increase concrete mix strength.

## Design of New and Reconstructed Composite Pavements

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) currently does not include a procedure for designing new and reconstructed composite pavements. [SHRP II R21, *Composite Pavement Systems*](http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2173) (estimated completion September 2011) will develop and validate performance prediction models and design procedures for this pavement type. Incorporation of developed models into [AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) will be contingent on acceptance by AASHTO.

## Design of Rehabilitated Pavements

[AASHTOWare Pavement ME Design™](http://www.aashtoware.org/Pavement/Pages/default.aspx) is able to evaluate and/or design the following pavement rehabilitation (and restoration) treatments ([[AASHTO 2008](https://bookstore.transportation.org/item_details.aspx?ID=1249)](https://bookstore.transportation.org/item_details.aspx?ID=1249)):

* Asphalt pavements
	+ Asphalt pavement overlays of existing asphalt and concrete pavements (intact or fractured), with or without pavement repair and surface milling.
* Concrete pavements
	+ JPCP overlays of existing JPCP/CRCP pavements (bonded and unbonded) and asphalt pavements.
	+ CRCP overlays of existing JPCP/CRCP pavements (bonded and unbonded) and asphalt pavements.
	+ JPCP restoration treatments including slab replacement, dowel bar retrofit, and joint resealing.

### Pavement Rehabilitation Design Methodology

The analysis of rehabilitation strategies follows the same procedure as outlined for asphalt and concrete pavements. However, unique to pavement rehabilitation design are the evaluation of the existing pavement structure, material characterization, and the selection of viable rehabilitation strategies.

#### Evaluation of Existing Pavement Structure

The evaluation of the existing pavement structure includes quantifying structural, functional, and drainage adequacy, material durability, shoulder condition, previously conducted preservation treatments, condition variation along the project length, and miscellaneous constraints (e.g., bridge clearance, right of way). The pavement evaluation features, by input level, are shown in Table 4-16.

#### Material Characterization of Existing Pavement Structure

A summary of material characterization inputs of existing asphalt pavement, concrete, chemically stabilized, and unbound layers are shown in Table 4-17, Table 4-18, Table 4-19, and Table 4-20, respectively.

Table 4-16. Pavement evaluation by hierarchical level (modified from [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter5_Evaluation.pdf)).

|  |  |  |
| --- | --- | --- |
| **Features** | **Factor** | **Data Level** |
| **1** | **2** | **3** |
| Structural adequacy | Load-related distress | 50 to 100 percent visual survey of entire project | 10 to 50 percent visual survey of entire project | Windshield survey of entire project |
| Deflection testing | Test every 500 ft or less over the entire project | Test every 500 ft or greater over the entire project | Historic data or perform limited testing at select locations  |
| GPR testing |
| Profile testing (IRI) |
| Coring, DCP | Perform at 2000 ft or less | Perform at 2000 ft or greater | Use historic data or perform limited testing at select locations |
| Maintenance data | Historic data and visual survey | Historic data | Historic data |
| Functional evaluation | Profile testing (IRI) | Test along entire project | Test along select sample units within project | Use historic data |
| Friction testing (FN) | Test along entire project | Test along select sample units within project | Use historic data |
| Surfacedrainage | Climate data | See [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm), Part 2, Chapter 3](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part2_Chapter3_Environmen.pdf) |
| Moisture-related distress | 100 percent survey of entire project | 100 percent survey of sample area along project | Windshield survey of entire project |
| Signs of moisture-accelerated damage | 100 percent survey of entire project | 100 percent survey of sample area along project | Windshield survey of entire project |
| Condition of subsurface drainage facilities | 100 percent survey of entire project | 100 percent survey of sample area along project | Windshield survey of entire project |
| Condition of surface drainage facilities | 100 percent survey of entire project | 100 percent survey of sample area along project | Windshield survey of entire project |
| Materials durability | Durability-related surface distress | 100 percent visual survey of entire project | 100 percent visual survey of sample area along project | Windshield survey of entire project |
| Base condition(erosion, stripping)or contamination | Test every 50 ft along project | Test every 500 ft along project | Historic data or limited testing at select locations |
| Shoulder | Surface condition(distress and joint) | 100 percent visual survey of entire project | 100 percent visual survey of sample area along project | Windshield survey of entire project |

Table 4-17. Summary of inputs for asphalt pavement rehabilitation (modified from [ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm), AASHTO 2008) Used by permission.

|  |  |  |  |
| --- | --- | --- | --- |
| **Variable** | **Level 1** | **Level 2** | **Level 3** |
| **Layer Thickness** | Determined through coring or other accepted methods. |
| **Asphalt Pavement Rehabilitation** | * Estimate damage through materials testing and FWD testing.
* Trenching to determine rutting in each layer.
 | * Enter the amount of fatigue cracking in the existing pavement.
* Estimate rut depth for each layer.
 | Enter subjective “Pavement Rating” (*Excellent*, *Good*, *Fair*, *Poor*, and *Very Poor)*. |
| **Dynamic Modulus, E\*** | 1. Backcalculate average modulus (Ef) from FWD testing.
2. Extract cores to obtain volumetric parameters.
3. Develop undamaged-dynamic modulus (E\*) using Witczak equation.
4. Estimate damage, dj as a function of Ef and E\*.
5. Compute α’ as function of dj and α.
6. Determine field-damaged master curve using α’.
 | Use predictive equation (inputs include gradation, viscosity or dynamic shear modulus or phase angle, loading frequency, air voids, and effective bitumen content). Inputs obtained through testing of cores or agency historical records. Damage, dj, determined using detailed pavement condition survey. | Undamaged asphalt mixture master curve determined using typical mix parameters; damage is estimated from pavement condition categories.

|  |  |
| --- | --- |
| **Rating** | **Damage** |
| Excellent | 0.00 – 0.20 |
| Good | 0.20 – 0.40 |
| Fair | 0.40 – 0.80 |
| Poor | 0.80 – 1.20 |
| Very poor | > 1.20 |

 |
| **Binder Properties** | Superpave binder or conventional binder tests from field cores. | Select Superpave binder grading, conventional viscosity grade, or conventional penetration grade. |
| **Reference Temperature** | 70° F |
| **Volumetric Properties** | Determine volumetric properties from cores. | Use construction records. |
| **Poisson’s Ratio, μ** | Typical values. |
| **Surface shortwave absorptivity** | No national test standard, use default value of 0.85. |
| **Thermal conductivity** | Use default value of 0.67 BTU/ft-hr-ºF. |
| **Heatcapacity, Q** | Use default value of 0.23 BTU/lb-ºF. |
| **Coefficient of thermal contraction** | No national test standard, value computed internally. |

Table 4-18. Summary of inputs for concrete rehabilitation (modified from [ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm), AASHTO 2008) Used by permission.

|  |  |  |  |
| --- | --- | --- | --- |
| **Variable** | **Level 1** | **Level 2** | **Level 3** |
| **Layer Thickness** | Determined through coring or other accepted methods. |
| **Elastic Modulus, E(intact)** | Testing of field cores(ASTM C469) or backcalculation. | Typical values based on pavement condition.* Adequate: 3 to 4 x 106 psi
* Marginal: 1 to 3 x 106 psi
* Inadequate: 0.3 to 1 x 106 psi
 |
| **Elastic Modulus, E(fractured)** | Testing of field cores(ASTM C469) or backcalculation. | Typical values.* Crack/seat and break/seat: 150,000 – 1,000,000 psi
* Rubblized: 50,000 – 150,000 psi
 |
| **FlexureStrength,Ec** | Testing of field cores(AASHTO T 97). | Compressive strength (f’c) at 7, 14, 28, and 90 days(ASTM C39). | 28-day MR or f’c of specific mix or agency default value. |
| **Unitweight, ρ** | Testing of field cores(AASHTO T 121). | Typical values. |
| **Poisson’sratio, μ** | Testing of field cores(ASTM C469). | Typical values. |
| **CTE** | Testing of field cores(AASHTO TP 60). | Agency historical, typical values based on coarse aggregate type, or use default value of 5.5 x 10-6/°F. |
| **Thermal Conductivity** | Testing of field cores(ASTM E1952). | Use default value of 1.25 BTU/ft-hr-°F. |
| **HeatCapacity, Q** | Testing of field cores(ASTM D2766). | Use default value of 0.28 BTU/lb-°F. |
| **Cement Type** | Based on actualcement source. | Based on agency practice. |
| **Cementitious Content** | Based on mix design. | Based on agency practice. |
| **Water/Cement Ratio** | Based on mix design. | Based on agency practice. |
| **Aggregate Type** | Based on actualaggregate source | Based on agency practice. |
| **Curing Method** | Based on agency recommended practice. |
| **Concrete Zero-Stress Temperature** | No national test standard, value computed internally based on cement content and the mean monthly ambient temperature during construction |
| **Ultimate Shrinkage** | Test procedure not practical. Computed internally using prediction equation. |
| **Reversible Shrinkage** | Unless more reliable information is available, 50 percent is recommended. |
| **Surface Shortwave Absorptivity** | No national test standard, use default value of 0.85. |
| **Time to develop 50% ultimate shrinkage** | Unless more reliable information is available, 35 days is recommended. |

Table 4-19. Summary of rehabilitation inputs for chemically stabilized layers (modified from [ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm), AASHTO 2008) Used by permission.

|  |  |  |  |
| --- | --- | --- | --- |
| **Variable** | **Level 1** | **Level 2** | **Level 3** |
| **Layer Thickness** | Determined through coring or other accepted methods. |
| **Elastic Modulus, E** | Backcalculation. | Testing of field cores or typical values.

|  |  |  |
| --- | --- | --- |
| **Material** | **TestMethod** | **Typicalvalue (psi)** |
| Lean concrete/cement treated | AASHTO T 22 | 2,000,000 |
| Cement stabilized aggregate | --- | 1,000,000 |
| Lime-cement, flyash | ASTM C593 | 1,500,000 |
| Soil cement | ASTM D1633 | 500,000 |
| Lime-stabilized soil | ASTM D5102 | 45,000 |

 |
| **Flexural Strength** | Required only for asphalt pavement designs. Use 20 percent of field core compressive strength or typical values (base = 750 psi or subbase/subgrade = 250 psi) |
| **Unitweight, ρ** | Use default value of 150 pcf. |
| **Poisson’s Ratio, μ** | Typical values. |
| **Thermal Conductivity** | Testing of field cores(ASTM E1952). | Use default value of 1.25 BTU/h-ft-°F. |
| **Heat Capacity, Q** | Testing of field cores(ASTM D2766) | Use default value of 0.28 BTU/lb-°F. |

## Surface and Subsurface Drainage Design

One of the key elements for obtaining long-term pavement performance is providing adequate and effective drainage of surface and subsurface moisture. Excessive moisture in the asphalt layers may result in such distress as stripping and potholes, and may also contribute to fatigue cracking and rutting. In concrete pavements, excessive moisture can contribute to the development of pumping, faulting, and loss of support. This section briefly describes various methods for reducing moisture in the pavement section although it should be noted that one method by itself may not be sufficient to prevent or reduce moisture-related damage.

Table 4-20. Summary of rehabilitation inputs for soil and unbound layers (modified from [ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm), AASHTO 2008) Used by permission.

|  |  |  |  |
| --- | --- | --- | --- |
| **Variable** | **Level 1** | **Level 2** | **Level 3** |
| **Unbound Material** | Classified using standard AASHTO or unified soil classification (USC) definitions. |
| **Layer Thickness** | Layer thickness determined through coring or other accepted methods. |
| **Poisson’s Ratio, ** | No national test standard, use typical values. |
| **Maximum Dry Density, γd** | Testing of field samples (AASHTO T 180) | Estimated based on gradation, plasticity index, and liquid limit. |
| **Optimum Moisture Content** | Testing of field samples(AASHTO T 180) | Estimated based on gradation, plasticityindex, and liquid limit. |
| **Specific Gravity** | Testing of field samples(AASHTO T 100) | Estimated based on gradation, plasticityindex, and liquid limit. |
| **Saturated Hydraulic Conductivity** | Testing of field samples(AASHTO T 215) | Estimated based on gradation, plasticityindex, and liquid limit |
| **Soil Water Characteristic Curve Parameters** | Testing of field samples(AASHTO T 99 or AASHTO T 180, or AASHTO T 100) | Estimated based on aggregate/subgradematerial classification. |
| **Coefficient of Lateral Pressure,** ko | Use default value = 0.5. |
| **Resilient Modulus1** | Backcalculate. Apply correction factor (to laboratory conditions) of 0.40 for subgrade soils and 0.67 for granular bases and subbases. | Based on material classification. |
| **Gradation Information** | Determined from sieve analysis in accordance with AASHTO T 27. |
| **Plasticity index (PI) and Liquid Limit (LL)** | Determined in accordance with AASHTO T 90 and AASHTO T 89. |

1 Converted to k-value internally for concrete pavements. Due to different stress-states, the resilient modulus for the same soil classification is different beneath asphalt and concrete pavements.

### Methods for Reducing Surface Moisture Infiltration

Surface moisture typically is deposited on the roadway surface in the form of rainwater and/or snow melt. The issue with surface moisture includes both the build-up of water on the pavement surface as well as the potential for the surface moisture to enter the pavement section. Methods for minimizing the impact of surface moisture include ([Anderson et al. 1998](http://books.nap.edu/openbook.php?record_id=6357&page=R1)a, [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)):

* Provide an impermeable surface.
* Provide adequate cross-slope and longitudinal slope to promote surface runoff. A cross-slope of 2 percent is typically used. Geometric design considerations must be in accordance with AASHTO’s *A Policy on Geometric Design of Highways and Streets* ([AASHTO 2004](https://bookstore.transportation.org/item_details.aspx?ID=110)).
* Use of an open-graded wearing surface to mitigate surface water ponding and spraying.
* Seal cracks and joints.
* Ensure drainage systems (i.e., curb, gutter, and catch basin or ditch) is of sufficient size to accommodate the estimated runoff.

### Use of Moisture-Insensitive Materials

The use of moisture-insensitive (or nonerodible) materials may also provide a reduction or delay in the development of moisture-related damage. The following provides a brief summary of moisture-insensitive materials ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)):

* Lean concrete and cement-treated base, which are effective in minimizing pumping and faulting of concrete pavements. These base types should be designed using high-quality crushed aggregate, higher cement contents, and higher compressive strengths to improve their resistance to moisture damage. For JPCP, if bonding is allowed, notches should be cut into the lean concrete or cement-treated base to match the jointing pattern. If bonding is not allowed, an asphalt seal coat or wax-based curing compound should be applied prior to concrete placement. In addition, placement of an aggregate subbase should be considered for JPCP in areas of high traffic, wet climates, and fine subgrade soils to minimize pumping and loss of fines.
* Asphalt-treated base, which are effective beneath both asphalt and concrete pavements. Design considerations should include the use of high-quality crushed aggregate and sufficient binder content to minimize the stripping potential. The mix design for asphalt-treated bases should be similar to that used in dense-graded asphalt mixtures.
* Granular base, which are effective beneath both asphalt and concrete pavements. Design considerations include use of high-quality aggregate and low fines content.

### Subsurface Drainage Systems

The primary purpose of subsurface drainage systems is to ([Asphalt Institute 2007](http://www.asphaltinstitute.org/store_category_browse.asp?ic_id=11)):

* Collect and drain water from the pavement surface.
* Remove subsurface water from unbound layers and keep the subgrade moisture content as low as possible.
* Minimize or prevent embankment erosion.
* Intercept water from surrounding areas.
* Lower the groundwater table.

The need for subsurface drainage will be dependent on the conditions of the specific roadway section, climatic conditions, and surround terrain (e.g., ability to drain). In general, not all pavement sections require a subsurface drainage system. Conditions when a subsurface drainage system should be considered include locations with wet climates and poorly draining soils. However, incorporation of other design features (e.g., use of stiffer base, incorporation of dowel bars) may be a more cost-effective solution ([Hall and Crovetti 2007](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_583.pdf)). A number of subsurface systems are briefly described and include the use of permeable bases, longitudinal edge drains, and daylighting.

#### Permeable Bases

Permeable base are open-graded drainage layers that typically drain water at a rate of 1,000 ft/day or greater. They can consist of asphalt-treated, cement-treated, or untreated materials (see Table 4-21). Permeable bases provide a drain path that moves laterally across the roadway into either an enclosed drainage system or ditches (both of which require routine maintenance to ensure base does not become blocked or clogged). Due to potential stability issues during placement, a maximum recommended thickness of a permeable base is 4 in. ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)). In addition, a separator layer (either unbound aggregate layer or geosynthetic material) should also be placed beneath the permeable base to minimize the infiltration of fine material from underlying layers. Based on the findings of NCHRP 1-34D, *Effects of Subsurface Drainage on Performance of Asphalt and Concrete Pavements: Further Analysis of LTPP SPS-1 and SPS-2 Field Sections*, the addition of a permeable base does not necessarily result in improved pavement performance. Instead, when considering pavement performance in relation to deflection, rutting, faulting, cracking, and roughness, improved pavement performance is more influenced by base stiffness rather than drainability ([Hall and Crovetti 2007](http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_583.pdf)). Cement content must be controlled in cement treated bases to avoid shrinkage cracks which could reflect through the asphalt pavement.

Table 4-21. Material characteristics of permeable bases ([FHWA 2002](http://isddc.dot.gov/OLPFiles/FHWA/013293.pdf)).

|  |  |
| --- | --- |
| **Material Type** | **Material Specifications** |
| Unstabilized Aggregate | Hard, durable material; at least 2 fractured faces; preferably consisting of 98 percent crushed stone; LA Abrasion not to exceed 45 percent (AASHTO T 96); soundness loss not to exceed 12 or 18 percent sodium sulfate or magnesium sulfate tests, respectively (AASHTO T 104); material passing No. 40 sieve shall be non-plastic (AASHTO T 90) |
| Asphalt-Treated | Hard, durable material; at least 2 fractured faces; preferably consisting of 98 percent crushed stone; LA Abrasion not to exceed 45 percent (AASHTO T 96); soundness loss not to exceed 12 or 18 percent sodium sulfate or magnesium sulfate tests, respectively (AASHTO T 104); asphalt binders that minimize draindown and permit thorough coating of aggregate; and binder content should be 3±1/2 percent by weight of dry aggregate. |
| Cement-Treated | Hard, durable material; at least 2 fractured faces; preferably consisting of 98 percent crushed stone; LA Abrasion not to exceed 45 percent (AASHTO T 96); soundness loss not to exceed 12 or 18 percent sodium sulfate or magnesium sulfate tests, respectively (AASHTO T 104); cement types consisting of Type I, Type I-P or Type II (AASHTO M 85); minimum cement content of 235 lbs/yd3; water-cement ratio that provides the minimum amount of water while providing workability, uniform material, with well coated aggregates. |

#### Subsurface Drainage Options

The following provides a number of different subsurface drainage options. This is by no means are an exhaustive list, but provides a summary of the more common subsurface drainage practices used by state highway agencies. Details related to the design of subsurface drainage systems can be found in the FHWA *Drainable Pavement Systems Participant Notebook* ([FHWA 1992](http://isddc.dot.gov/OLPFiles/FHWA/013226.pdf)) and [Part 2, Chapter 3](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf) and [Appendix SS](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/2appendices_SS.pdf) of the NCHRP 1-37A Project Final Report ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)).

##### Permeable Base with Edgedrains

This system consists of a permeable base (treated or untreated), separator layer, edgedrains, outlets, headwalls, and ditches or storm drains. Moisture is moved from the permeable base into longitudinal edgedrains and deposited into the roadway ditch using outlet pipes. Longitudinal edgedrains consist of a trench, typically wrapped with a geotextile to minimize the amount of fine material entering the edgedrain, free draining aggregate material, and smooth, slotted pipes (see Figure 4-9). Smooth non-slotted outlet pipes are placed at specified locations to transversely remove water collected by the edgedrain into the roadside ditch or storm drain. Where outlet pipes deposit water into roadside ditches, headwalls are constructed to minimize damage to the end of the outlet pipe (see Figure 4-10).



Figure 4-9. Permeable base with edgedrain (redrawn from [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)).



Figure 4-10. Headwall ([FHWA 1992](http://isddc.dot.gov/OLPFiles/FHWA/013226.pdf)).

##### Nonerodible Base with Edgedrains

This subsurface drainage system is similar to the permeable base and edgedrain system except for the use of a nonerodible base (e.g., asphalt base or lean-concrete base) beneath the pavement section (see Figure 4-11). With this subsurface drainage system, moisture is collected from the lane/shoulder joint and from surface cracks into the edgedrain.



Figure 4-11. Nonerodible base with edgedrain (redrawn from [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)).

##### Nonerodible Base with Porous Concrete Edgedrains

This subsurface drainage system incorporates the use of porous concrete (see Figure 4-12) rather than permeable backfill material in the edgedrain trench. This system allows for increased support under the shoulder section, minimizing shoulder settlement problems ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)).



Figure 4-12. Nonerodible base, edgedrain, and porous concrete shoulder
(redrawn from [[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)).

##### Daylighted Permeable Base

A daylighted pavement section is one where the permeable base layer is carried out to the longitudinal ditches, allowing the direct outflow of trapped water (see Figure 4-13). In general, daylighted permeable bases are used on roadways with flat grades (1 percent or less) and shallow ditches where it would be otherwise difficult to get an edgedrain system to flow into a ditch ([Hall and Tayabji 2009](http://www.fhwa.dot.gov/pavement/concrete/pubs/hif09009/hif09009.pdf)). Daylighted permeable bases are not recommended where the depth of freeze exceeds the combined depth of the pavement section. Daylighted permeable bases should be constructed on a 3 percent slope and at least 6 in. above the 10-year-storm flow line ([Hall and Tayabji 2009](http://www.fhwa.dot.gov/pavement/concrete/pubs/hif09009/hif09009.pdf)). Maintenance on daylighted permeable bases should include weeding, debris removal, flushing with low-pressure water, and grading of ditch to allow for free flow of water into the ditch ([Hall and Tayabji 2009](http://www.fhwa.dot.gov/pavement/concrete/pubs/hif09009/hif09009.pdf)).



Figure 4-13. Daylighted permeable base ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)).

##### Daylighted Pavement Section

In a daylighted pavement section base (and/or subbase) layers may be constructed with either nonerodible or dense-graded aggregate materials (see Figure 4-14). Base (and/or subbase) layers are exposed to allow water to flow into roadside ditches. Though this section is technically not a subsurface drainage system, it is successfully used by many state highway agencies



Figure 4-14. Daylighted dense-graded aggregate base.

### Pavement Design Features to Minimize Moisture Damage

In addition to reducing the infiltration of surface moisture, using moisture-insensitive materials, and subsurface drainage systems, there are several pavement design features that could be considered for minimizing or eliminating the impact of moisture damage. These include ([[ARA 2004](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/guide.htm)](http://onlinepubs.trb.org/onlinepubs/archive/mepdg/Part3_Chapter1_Subdrainage.pdf)):

* Concrete pavements
	+ Use of dowel bars at transverse joints to reduce joint faulting.
	+ Use of a widened slab (typically 14 ft) in the rightmost lane to reduce slab deflection and edge/corner stresses and minimize damage related to faulting and slab cracking.
	+ Use of tied concrete shoulders to reduce edge deflections and the potential for pumping.
* Conventional and deep-strength asphalt pavements
	+ Eliminate the lane/shoulder joint by placing asphalt pavement full –width (i.e., place shoulder and adjacent lane in one pass).
	+ Use of a granular layer between the subgrade and base course to minimize erosion and frost susceptibility.

### Freeze-Thaw Considerations

In northern or cold climates, consideration must be given to the combined impacts of freezing temperature, frost susceptible soils, and the presence of moisture. If these three conditions exist, potential damage of the pavement section may occur due to frost heave and thaw weakening of the unbound materials and subgrade soils. Frost heave is defined as the accumulation of moisture that results in the development of ice lenses (see Figure 4-15) during the freezing period. Thaw weakening is the reduction in strength due to thawing of the ice lenses during the thawing period.

In order for frost heaves to occur, three conditions must exist: subfreezing temperatures, presence of moistures, and frost susceptible soils (generally considered to be unbound or subgrade materials containing more than 10 percent passing the No. 200 sieve) (see Table 4-22).

Table 4-22. Frost susceptibility by soil classification (WSDOT 1999).

|  |  |  |
| --- | --- | --- |
| **Frost Classification** | **USCS** | **AASHTO** |
| No frost sensitivity | GW, GP, SW, SP | A-1, A-2, A-3 |
| Little frost sensitivity | GM, GC | A1-b |
| Medium frost sensitivity | SM, SC, CH, OH | A-2, A-4, A-6, A-7 |
| Heavy reaction to frost | ML, MH, CL, OL | A-4, A-5, A-7-5 |

Removal of any one of these conditions will eliminate or at least minimize the frost heave potential. There are two heaving possibilities (WSDOT 1999):

* Uniform heave – the overall uniform heaving of subgrade materials generally does not adversely affect pavement performance.
* Differential heave – this generally occurs at locations where subgrade changes from a non-frost susceptible to a frost-susceptible soil, abrupt transitions from cut to fill where the ground water table is close to the surface, where excavation exposes a water-bearing layer, and in the vicinity of drains, culverts, and so on where different backfill material or compactive effort occurred.



Figure 4-15. Formation of ice lenses in a pavement structure (WSDOT 1999).

Methods for mitigating the potential damaging effects of frost susceptible soils include:

* Increase pavement total depth (surfacing and/or base/subbase depth) to a minimum percentage of the total frost depth. Typically the total pavement depth should be at least half the depth of the total expected freeze depth.
* Remove and replace frost susceptible soils with non-frost susceptible materials.
* Use of a capillary break to disrupt the flow of moisture. The states of Idaho and Washington have found the placement of a rock cap layer to be effective in mitigating frost action (Uhlmeyer et al 2003). In addition, Idaho Transportation Department has developed [rock cap specifications](http://itd.idaho.gov/manuals/Online_Manuals/Current_Manuals/Materials/Sec%20550.pdf) for use as subsurface pavement drainage.
* Incorporate lower subgrade support values (e.g., resilient modulus) in design of the pavement structure. This will increase the total pavement thickness, thereby minimizing the loss of support during the thawing period.
* Restrict pavement loading during thawing conditions, typically a load reduction of 40 to 50 percent is acceptable.
* Ensure ditches are of sufficient depth (in general should be deep enough to drain the base/subbase layers) and are maintained to allow removal of moisture during thawing conditions. If an underdrain system is used, ensure that it is adequately designed and maintained (i.e., free of debris, outlets are not plugged or blocked).

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