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Guide for Mechanistic-Empirical Design

OF NEW AND REHABILITATED PAVEMENT STRUCTURES

FINAL REPORT

PART 2. DESIGN INPUTS

CHAPTER 2. MATERIAL CHARACTERIZATION

NCHRP

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DISCLAIMER

CHAPTER 2
MATERIAL CHARACTERIZATION

2.2.1 INTRODUCTION

This chapter describes the material characterization required for the mechanistic-empirical (M-E) design approach described in this Guide. The interaction of the materials inputs with other components of the mechanistic-empirical design procedure outlined in the Guide is shown in figure 2.2.1. The interaction between the materials, climatic, traffic, structural response, and performance prediction components is apparent from the figure. To provide a common basis for understanding the material requirements, the following M-E–based subcategories have been developed:

- Material properties required for computing pavement responses.
- Additional materials inputs to the distress/transfer functions.
- Additional materials inputs required for climatic modeling.

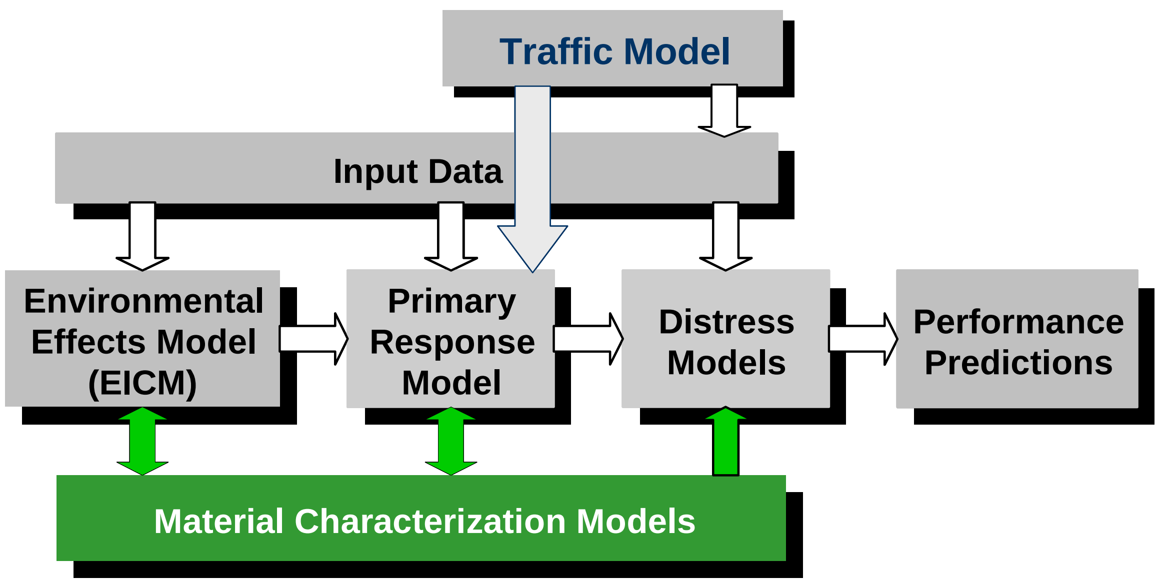


Figure 2.2.1. Interaction between materials module with other components of the M-E framework in the Design Guide.

In the first category are material properties required to predict the states of stress, strain, and displacement within the pavement structure when subjected to an external wheel load. In the M-E approach selected for the Design Guide, these properties include elastic modulus (E) and Poisson's ratio (μ) of the material. These properties are mandatory inputs for each pavement layer in the system.

In the second category are all the materials-related inputs that enter the distress or smoothness models directly. As stated in PART 1, Chapter 1, the key distress types or performance measures considered in flexible pavement design include load-related fatigue fracture (top-down and bottom-up), permanent deformation, and transverse fracture. The primary performance measures considered in rigid pavement design include load-related fatigue fracture of PCC (top-down and bottom-up) in JPCP, faulting of transverse joints in JPCP, and punchouts in CRCP. For each of these distresses, the critical structural response under a given wheel and climatic loading condition is affected by pavement material properties such as modulus and Poisson's ratio. In addition, parameters such as strength, expansion-contraction characteristics, friction between slab and base, erodibility of underlying layers, layer drainage characteristics, plasticity and gradation, and other material attributes directly influence how a material contributes to a given distress mechanism. These additional materials inputs are specific to the pavement type and distress model under consideration.

Finally, in the third category are materials-related inputs that enter the climatic module to help determine the temperature and moisture profiles through the pavement cross-section. These include engineering index properties (e.g., plasticity index), gradation parameters (porosity, effective grain sizes, etc), and thermal properties (absorptivity, heat capacity, coefficient of thermal expansion, and so on).

Table 2.2.1 is a tabular summary of the materials inputs required for M-E design arranged by the major material categories considered in the Design Guide including recycled asphalt and PCC layers used in rehabilitation design and construction. The material categories are explained later in this chapter.

2.2.1.1 Material Factors Considered

Time-Dependent Properties

Because some materials' properties undergo time-dependent changes, it is quite important to be able to consider these changes in any M-E analysis. Generally speaking, pavement material properties are constantly changing over time due to chemical and physical forces, influence of climate, as well as the onset of fracture or deformation. Consequently, the magnitude of the material properties either undergoes an enhancement or deterioration. These should be accounted for in design and are discussed below.

Property Enhancement

Long-term, time-dependent property enhancements occur due to one of the following conditions: chemical and physical hardening of asphalt binders due to short- and long-term aging of the asphalt binder, curing caused by the evaporation of moisture within asphalt emulsion systems, and pozzolanic reactions of cementitious materials. Normally, time-dependent property enhancements are viewed as increasing the relative modulus and strength of the material. Strength gain in PCC is an example of such a beneficial effect. Physical hardening of an asphalt layer is also beneficial in a flexible pavement system to the extent that it lowers the states of stress in the underlying layers and decreases the likelihood of permanent deformation occurring in the pavement system. However, in asphalt mixtures, the general increase in rigidity of the material layer also increases the possibility that fracture will occur due to load and environmental effects.

Table 2.2.1. Major material input considerations by material group.

Materials Category	**Materials Inputs Required**		
	Materials inputs required for critical response computations	**Additional materials inputs required for distress/transfer functions**	**Additional materials inputs required for climatic modeling**
Hot-Mix Asphalt Materials (this covers surface, binder, base, and subbase courses)	• Time-temperature dependent dynamic modulus (E*) of HMA mixture. • Poisson's ratio.	• Tensile strength, creep compliance, coefficient of thermal expansion.	• Surface shortwave absorptivity (only required for surface course), thermal conductivity, and heat capacity of HMA. • Asphalt binder viscosity (stiffness) characterization to account for aging.
PCC Materials (this covers surface layer only)	• Static modulus of elasticity (E) adjusted with time. • Poisson's ratio. • Unit weight • Coefficient of thermal expansion.	• Modulus of rupture, split tensile strength, compressive strength, cement type, cement content, water-to-cement (w/c) ratio, ultimate shrinkage, amount of reversible shrinkage.	• Surface shortwave absorptivity, thermal conductivity, and heat capacity of PCC.
Chemically Stabilized Materials (this covers lean concrete, cement treated, soil cement, lime-cement-flyash, lime-flyash, and lime stabilized layers)	• Elastic modulus (E) for high quality lean concrete, cement treated material, soil cement, lime-cement-flyash, and lime-cement-flyash. Resilient modulus (M_r) for lime stabilized soil. • Poisson's ratio. • Unit weight.	• Minimum resilient modulus (used in flexible design), Modulus of rupture (used in flexible design), base erodibility (for rigid design).	• Thermal conductivity and heat capacity of PCC.
Unbound Base/ Subbase and Subgrade Materials	• Seasonally adjusted resilient modulus (M_r). • Poisson's ratio. • Unit weight. • Coefficient of lateral pressure.	• Gradation parameters and base erodibility (for rigid design).	• Plasticity index, gradation parameters, effective grain sizes, specific gravity, saturated hydraulic conductivity, optimum moisture contents, parameters to define the soil water characteristic curve.
Recycled Concrete Materials—Fractured PCC Slabs	• Resilient modulus (M_r). • Poisson's ratio.	• Base erodibility (for rigid design).	• Thermal conductivity and heat capacity.
Recycled hot asphalt mix (central plant processed)	Treated same as hot-mix asphalt surface course.		
Recycled cold asphalt mix (central plant or on-grade)	Treated same as hot-mix asphalt base course.		
Cold recycled asphalt pavement (used as aggregate)	Treated same as granular materials with no moisture sensitivity.		
Bedrock	• Elastic modulus (E). • Poisson's ratio. • Unit weight.	None.	None.

Property Deterioration

Materials that are subjected to load-related fatigue distress may experience a severe degradation of properties with time and load repetitions. Micro-cracks may develop, leading to a reduced stiffness or modulus. This reduction in modulus will, in turn, lead to an increase in stress states within the pavement and a greater possibility of permanent deformation. The Design Guide considers the impact of load-related damage on property degradation.

<u>Time-Temperature Effects</u>

All asphaltic materials are highly sensitive to temperature and the rate of loading. Because asphalt is a viscoelastic-plastic material, the modulus of an asphalt mix may approach that of an unbound granular material at high temperatures and long loading rates (i.e., slow speed of passing vehicles). In contrast, at cold temperatures and very short load rates, the material will tend to behave in a pure elastic mode and have modulus values that approach that of PCC material. In the Design Guide, the methodology for asphaltic mixtures will take into account the range of temperatures expected in the design period. In addition, the use of an asphalt master curve, based on time–temperature superposition principles, will allow the engineer to account for the approximate speed of the vehicle under consideration.

Asphalt is not the only material that is influenced by vehicular speed. For example, rigid pavements show a 30 percent reduction in corner deflection and PCC edge strain as vehicle speeds increase from 3 to 60 mph. The dynamic modulus of PCC under typical Interstate vehicle speeds is approximately 20 percent higher than the static modulus historically used in rigid pavement analysis. Soft, wet clay subgrades that are viscoelastic-plastic may also exhibit a time-dependent modulus response; however, the ability to adjust subgrade moduli for a particular design speed is not intrinsically a part of the Guide. If a particular design situation requires that this factor be considered, direct modulus testing can be altered to characterize the material behavior under the specified loading rate.

The rate of load effect upon material response is a function not only of the vehicular speed, but also of the location of the material within the pavement structure. In general, as one proceeds deeper into the pavement, the length of the stress pulse acting on a given material will increase, suggesting that the time of the load pulse will also increase. The Guide does not automatically consider this aspect of material behavior, except in the analysis of asphaltic mixtures. Asphalt materials are characterized by a master curve incorporating time and temperature effects directly into the solution methodology.

<u>Non-Linear Behavior</u>

A material is considered non-linear if the value of the elastic modulus depends on the state of stress in the material. While many materials start to exhibit this behavior at very high stress states, the only materials that are considered to be non-linear in this Guide are unbound base/subbase and subgrade materials. Because the Guide inputs have been developed in a hierarchical structure, the most advanced level of M-E analysis can incorporate this factor into the design analysis methodology. If a non-linear analysis is desired due to use of non-linear moduli for the unbound materials present in the system, a finite element model must be used to compute the flexible pavement response (states of stress, strain, and displacement). If the engineer will not be using the non-linear properties of unbound materials as the design input for flexible pavement systems, the pavement response model will automatically use a linear layered elastic analysis.

2.2.1.2 Material Categories

Many combinations of material types and quality are used in flexible and rigid pavement systems. Through the years, many organizations and agencies have developed their own major material categorical classifications that suit their specific views and uses of these materials. Because no convenient functional grouping of materials best categorizes typical properties required for use in M-E analysis and evaluation of pavement systems, a material category grouping has been developed for use in the Design Guide.

The major categorical system developed for the Design Guide is presented in table 2.2.2. Six major material groups have been developed: asphalt materials, PCC materials, chemically stabilized materials, non-stabilized granular materials, subgrade soils, and bedrock.

One of the more complicated groups is "Asphalt Materials," because the response and behavior of these materials are heavily influenced by temperature, time rate of load, method of mixture, the mixing process, and the degree of damage of the material (new versus rehabilitated pavement systems). In reality, this category may include material subgroups for which a great deal of historical information is available concerning typical modulus, Poisson's ratio, strength, fracture and permanent deformation properties, as well as materials that will only allow for general, educated guesses about their properties. An example of this group would be in-place cold recycled materials where millings, new virgin materials, and emulsified asphalts may be blended during the paving or construction process.

PCC materials are grouped into two major subgroups: intact and fractured slabs. A great deal of information is available concerning the material properties of intact slab materials. The fractured slab subgroup is generally applicable only to PCC rehabilitation involving the intentional reduction of the effective slab length to minimize the influence of reflective cracking in subsequent overlays. The degree to which slab fracturing takes place is of paramount importance to properly assigning effective modulus values to the fractured slab if linear elastic layer procedures are used. It should also be recognized that as the degree of fracture is increased (PCC slab becomes "rubblized"), the effective moduli of the PCC layer may approach that of a high-quality crushed stone base. One of the most reliable ways of determining the insitu moduli of these systems is through backcalculation with FWD data.

The category of chemically stabilized materials covers a broad range of cementitious or pozzolanically (chemical) reactive materials. They range from materials that only slightly modify the plasticity characteristics of the original material to materials having major gains in modulus, strength, and other key engineering properties that may approach the behavior of PCC material. Lime, flyash, and portland cement are the major types of cementing material in this category.

Table 2.2.2. Major material categories.

Asphalt Materials

- Hot Mix Asphalt (HMA)—Dense Graded
  - Central Plant Produced
  - In-Place Recycled
- Stone Matrix Asphalt (SMA)
- Hot Mix Asphalt—Open Graded Asphalt
- Hot Mix Asphalt—Sand Asphalt Mixtures
- Cold Mix Asphalt
  - Central Plant Processed
  - In-Place Recycled

PCC Materials

- Intact Slabs
- Fractured Slabs
  - Crack/Seat
  - Break/Seat
  - Rubblized

Chemically Stabilized Materials

- Cement Stabilized Aggregate
- Soil Cement
- Lime Cement Fly Ash
- Lime Fly Ash
- Lime Stabilized Soils
- Open graded Cement Stabilized Aggregate

Non-Stabilized Granular Base/Subbase

- Granular Base/Subbase
- Sandy Subbase
- Cold Recycled Asphalt (used as aggregate)
  - RAP (includes millings)
  - Pulverized In-Place
- Cold Recycled Asphalt Pavement (HMA plus aggregate base/subbase)

Subgrade Soils

- Gravelly Soils (A-1;A-2)
- Sandy Soils
  - Loose Sands (A-3)
  - Dense Sands (A-3)
  - Silty Sands (A-2-4;A-2-5)
  - Clayey Sands (A-2-6; A-2-7)
- Silty Soils (A-4;A-5)
- Clayey Soils
  - Low Plasticity Clays (A-6)
    - Dry-Hard
    - Moist Stiff
    - Wet/Sat-Soft
  - High Plasticity Clays (A-7)
    - Dry-Hard
    - Moist Stiff
    - Wet/Sat-Soft

Bedrock

- Solid, Massive and Continuous
- Highly Fractured, Weathered

The major material characteristics associated with unbound materials are related to the fact that moduli of these materials may be highly influenced by the stress state (non-linear) and in-situ moisture content. As a general rule, coarse-grained materials have higher moduli as the state of confining stress is increased. In contrast, clayey materials tend to have a reduction in modulus as the deviatoric or octahedral stress component is increased. Thus, while both categories of unbound materials are stress dependent (non-linear), each behaves in an opposite direction as stress states are increased.

Additionally, permanent deformation (repetitive shear displacements) may be of paramount concern in weaker material layers or layers that are not well protected. While this category of materials may exhibit strong non-linear behavior, the current state of the art in backcalculation methodologies could not accurately estimate the in-situ non-linear coefficients that can be evaluated and obtained through laboratory testing programs. Therefore, extreme care needs to be exercised during rehabilitation NDT surveys to ensure that the range of loads used are comparable to those used in the design M-E analysis.

The bedrock category is also worth mentioning because its presence near the pavement structure may require the designer to properly account for the high layer modulus to obtain accurate predictions of the pavement stress, strain and displacement. While the precise measure of the modulus is seldom, if ever, warranted, any bedrock layer must be incorporated into the analysis if it lies close to the pavement structure, particularly if backcalculation will be used as part of a rehabilitation analysis.

2.2.1.3 Hierarchical Input Approach Concepts

Philosophy

The general approach for selecting or determining design inputs for materials in the Design Guide is a hierarchical (level) system. In its simplest and most practical form, the hierarchical approach is based on the philosophy that the level of engineering effort exerted in the pavement design process should be consistent with the relative importance, size, and cost of the design project. Level 1 is the most current implementable procedure available, normally involving comprehensive laboratory or field tests. In contrast, Level 3 requires the designer to estimate the most appropriate design input value of the material property based on experience with little or no testing. Inputs at Level 2 are estimated through correlations with other material properties that are measured in the laboratory or field.

The advantages of this hierarchical approach include the following:

- Provides the engineer with greater flexibility in selecting an engineering approach consistent with the size, cost, and overall importance of the project.
- Allows each agency to develop an initial design methodology consistent with its internal technical capabilities.
- Provides a very convenient method to increase an agency's technological skills gradually over time.
- Ensures the development of the most accurate and cost-efficient design, consistent with agency financial and technical resources.

Factors Influencing Input Level Selection

The selection of the hierarchical level while configuring inputs for a particular material type will be highly influenced by whether the design is for a new or rehabilitated pavement system. Existing pavement structures provide an in-situ "laboratory" of the exact state of damage and behavior of each material. For example, the moduli of existing pavement layer materials can be more precisely estimated from backcalculation of FWD data or through the use of dynamic cone penetration testing. Thus, any hierarchical system for materials should logically discriminate between new and rehabilitated structures.

2.2.2 INPUT CHARACTERIZATION FOR THE ASPHALT MATERIALS GROUP

The discussion under this category applies to asphalt treated materials that fall under the following general definitions (see table 2.2.2):

- Hot Mix Asphalt—Dense Graded
  - Central Plant Produced.
  - In-Place Recycled.
- Hot Mix Asphalt—Open Graded Asphalt.
- Hot Mix Asphalt—Sand Asphalt Mixtures.
- Recycled Asphalt Concrete Pavement
  - Recycled hot-mix (central plant processed)
  - Recycled cold-mix (central plant processed or processed on-grade).

These materials could be used in the construction of surface, base, and subbase courses in a pavement system. The hierarchical approach to derive the required design inputs of these materials for both new and rehabilitation design is discussed below.

2.2.2.1 Layer Modulus for New or Reconstruction Design

The primary stiffness property of interest for asphalt materials is the time-temperature dependent dynamic modulus (E^*). Table 2.2.3 provides a brief summary of the procedures at various input hierarchical levels to derive E^* for new or reconstruction design.

<u>Overview of Dynamic Modulus Estimation</u>

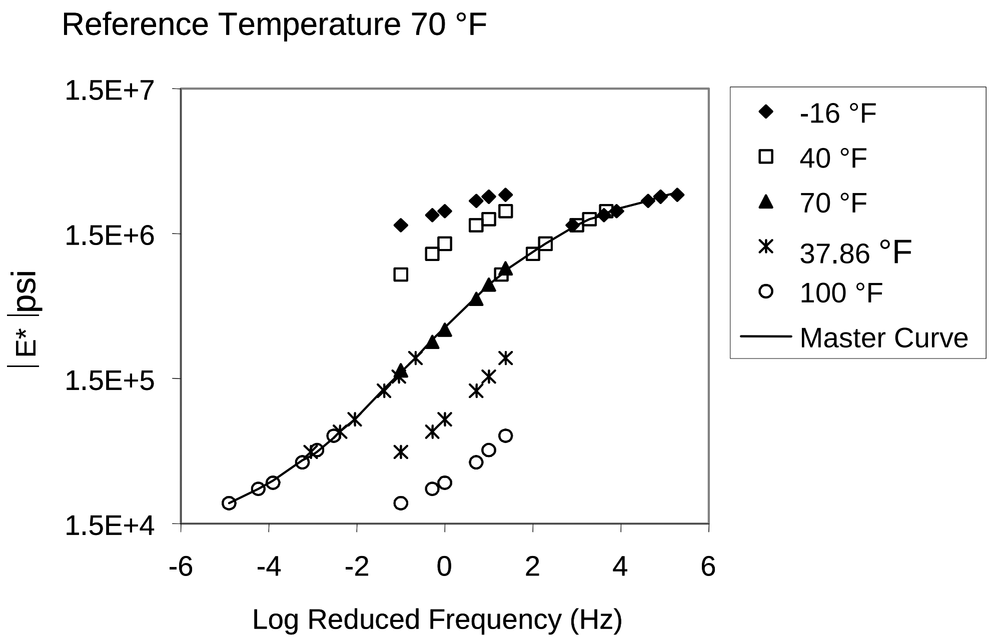
The modulus properties of asphalt concrete are known to be a function of temperature, rate of loading, age, and mixture characteristics such as binder stiffness, aggregate gradation, binder content, and air voids. To account for temperature and rate of loading effects, the modulus of the asphalt concrete at all analysis levels will be determined from a master curve constructed at a reference temperature of 70 °F.

Master Curve and Shift Factors

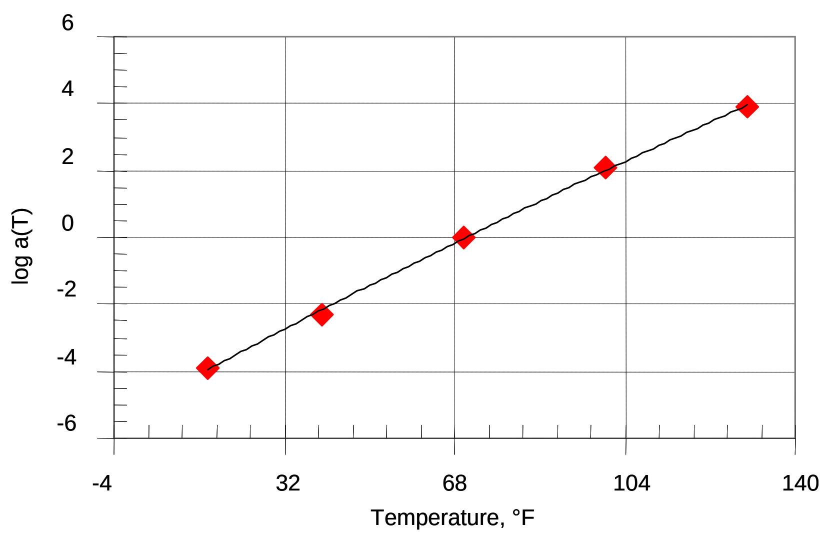
Master curves are constructed using the principle of time-temperature superposition. First, a standard reference temperature is selected (in this case, 70 °F), and then data at various temperatures are shifted with respect to time until the curves merge into a single smooth function. The master curve of modulus as a function of time formed in this manner describes the time dependency of the material. The amount of shifting at each temperature required to form the master curve describes the temperature dependency of the material. Thus, both the master curve and the shift factors are needed for a complete description of the rate and temperature effects. Figure 2.2.2 presents an example of a master curve constructed in this manner and the resulting shift factors.

Table 2.2.3. Asphalt dynamic modulus (E^*) estimation at various hierarchical input levels for new or reconstruction design.

Material Group Category	**Input Level**	**Description**
Asphalt Materials	1	• Conduct E^* (dynamic modulus) laboratory test (NCHRP 1-28A) at loading frequencies and temperatures of interest for the given mixture. • Conduct binder complex shear modulus (G*) and phase angle (δ) testing on the proposed asphalt binder (AASHTO T315) at ω = 1.59 Hz (10 rad/s) over a range of temperatures. • From binder test data estimate Ai-VTSi for mix-compaction temperature. • Develop master curve for the asphalt mixture that accurately defines the time-temperature dependency including aging.
	2	• No E* laboratory test required. • Use E* predictive equation. • Conduct G*-δ on the proposed asphalt binder (AASHTO T315) at ω = 1.59 Hz (10 rad/s) over a range of temperatures. The binder viscosity or stiffness can also be estimated using conventional asphalt test data such as Ring and Ball Softening Point, absolute and kinematic viscosities, or using the Brookfield viscometer. • Develop Ai-VTSi for mix-compaction temperature. • Develop master curve for asphalt mixture that accurately defines the time-temperature dependency including aging.
	3	• No E* laboratory testing required. • Use E* predictive equation. • Use typical Ai-VTS- values provided in the Design Guide software based on PG, viscosity, or penetration grade of the binder. • Develop master curve for asphalt mixture that accurately defines the time-temperature dependency including aging.



a. Master Curve.



b. Shift Factors.

Figure 2.2.2. Schematic of master curve and shift factors.

The dynamic modulus master curve can be represented by the sigmoidal function described by equation 2.2.1:

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta + \gamma(\log t_r)}} \tag{2.2.1}$$

where

E*	=	dynamic modulus.
t_r	=	time of loading at the reference temperature.
δ, α	=	fitting parameters; for a given set of data, δ represents the minimum value of E* and δ+α represents the maximum value of E* .
β, γ	=	parameters describing the shape of the sigmoidal function.

The fitting parameters δ and α depend on aggregate gradation, binder content, and air void content. The fitting parameters β and γ depend on the characteristics of the asphalt binder and the magnitude of δ and α. The sigmoidal function describes the time dependency of the modulus at the reference temperature. The shift factors describe the temperature dependency of the modulus. Equation 2.2.2 provides the general form of the shift factors.

$$t_r = \frac{t}{a(T)} \tag{2.2.2a}$$

$$\log(t_r) = \log(t) - \log[a(T)] \tag{2.2.2b}$$

where

t_r	=	time of loading at the reference temperature.
t	=	time of loading at a given temperature of interest.
a(T)	=	Shift factor as a function of temperature.
T	=	temperature of interest.

Thus, using equation 2.2.2, the time of loading at the reference temperature can be calculated for any time of loading at any temperature. Then the appropriate modulus can be calculated from equation 2.2.1 using the time of loading at the reference temperature.

Master curves and the corresponding shift factors can be developed experimentally by shifting laboratory frequency sweep data from either dynamic modulus tests, NCHRP 1-28A, or shear tests, AASHTO T320, "Determining Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Test (SST)." This method is used for the Level 1 analysis. It requires nonlinear optimization using equations 2.2.1 and 2.2.2, and actual laboratory test data. For the Level 2 and Level 3 analyses, the master curves will be developed directly from the dynamic modulus predictive equation shown in equation 2.2.3 (see Appendix CC). This equation has the ability to predict the dynamic modulus of asphalt mixtures over a range of temperatures, rates of loading, and aging conditions from information that is readily available from material specifications or volumetric design of the mixture.

$$\begin{aligned}\log E^* &= 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a \\ &- 0.802208\left(\frac{V_{beff}}{V_{beff} + V_a}\right) + \frac{3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017(\rho_{38})^2 + 0.005470\rho_{34}}{1 + e^{(-0.603313 - 0.313351\log(f) - 0.393532\log(\eta))}}\end{aligned} \tag{2.2.3}$$

where:

E^*	=	dynamic modulus, psi.
η	=	bitumen viscosity, 10^6 Poise.

f	=	loading frequency, Hz.
V_a	=	air void content, %.
V_{beff}	=	effective bitumen content, % by volume.
ρ_{34}	=	cumulative % retained on the ¾ in sieve.
ρ_{38}	=	cumulative % retained on the 3/8 in sieve.
ρ_4	=	cumulative % retained on the No. 4 sieve.
ρ_{200}	=	% passing the No. 200 sieve.

The statistical summary for equation 2.2.3 is as follows:

R^2	=	0.96
Se/Sy	=	0.24
Number of Data Points	=	2750
Temperature Range	=	0 to 130 ^{o}F
Loading Rates	=	0.1 to 25 Hz
Number of mixtures	=	205 Total, 171 with unmodified asphalt binders, 34 with modified binders
Number of Binders used	=	23 Total, 9 Unmodified, 14 Modified
Number of Aggregate Types	=	39
Compaction methods	=	Kneading and gyratory
Specimen sizes	=	Cylindrical 4 in x 8 in or 2.75 in x 5.5 in

Equation 2.2.3 can also assume the form of a sigmoidal function shown in equation 2.2.1. Equation 2.2.4 presents the dynamic modulus equation shown in equation 2.2.3 in the form of a mixture specific master curve.

$$\log(E^*) = \delta + \frac{\alpha}{1+e^{\beta+\gamma \log t_r}} \tag{2.2.4}$$

where

E*	=	Dynamic modulus (psi)
t_r	=	Time of loading at the reference temperature
δ	=	Minimum value of E*
$\delta+\alpha$	=	Maximum value of E*
β, γ	=	Parameters describing the shape of the sigmoidal function

From equation 2.2.3 and 2.2.4, the following can be stated:

$$\delta = 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a - 0.802208\left[\frac{Vb_{eff}}{Vb_{eff} + V_a}\right]$$

$$\alpha = 3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017\rho_{38}{}^2 + 0.005470\rho_{34}$$

$$\beta = -.603313 - .393532\log(\eta_{T_r})$$

$$\log(t_r) = \log(t) - c\left(\log(\eta) - \log(\eta_{T_r})\right)$$

$$\gamma = 0.313351$$

$$c = 1.255882$$

Binder Viscosity

The viscosity of the asphalt binder at the temperature of interest is a critical input parameter for the dynamic modulus equation and the determination of shift factors as described above. For unaged conditions, the viscosity of the asphalt binder at the temperature of interest is determined from the ASTM viscosity temperature relationship (*1*) defined by equation 2.2.5:

$$\log\log\eta = A + VTS\log T_R \tag{2.2.5}$$

where

η	=	viscosity, cP.
T_R	=	temperature, Rankine.
A	=	regression intercept.
VTS	=	regression slope of viscosity temperature susceptibility.

At hierarchical input level 1, the A and VTS parameters in equation 2.2.5 can be estimated from the dynamic shear rheometer test data conducted in accordance with AASHTO T315, "Determining the Rheological of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)." Alternately, at all input levels, the A and VTS parameters can be obtained from a series of conventional tests, including viscosity, softening point, and penetrations. The procedure to estimate the A and VTS parameters will be explained later in this chapter.

Effect of Asphalt Aging

The effect of aging is incorporated into the determination of dynamic modulus using the Global Aging System (*2*). This system provides models that describe the change in viscosity that occurs during mixing and compaction, as well as long-term in-situ aging. The Global Aging System includes four models:

- Original to mix/lay-down model.
- Surface aging model.
- Air void adjustment.
- Viscosity-depth model.

The original to mix/lay-down model accounts for the short-term aging that occurs during mixing and compaction. The surface aging model then predicts the viscosity of the binder at the surface of the pavement after any period of time using the viscosity at mix/lay-down. If warranted, the surface viscosity from the surface aging model can be adjusted for different air void contents using the air void adjustment model. Finally, the viscosity as a function of depth is determined using the viscosity from the surface aging model or the air void adjusted model, along with the viscosity-depth model. The output of the Global Aging System is a prediction of the binder viscosity at any time and any depth in the pavement system. The Global Aging System is an integral part of the Design Guide software.

Equation 2.2.6 presents the Global Aging System model for short-term aging. The code value is related to the hardening ratio, defined as the ratio of the log-log mix/lay-down viscosity (RTFO) to the log-log original viscosity. Table 2.2.4 summarizes recommended code values.

$$\begin{aligned} \log\log(\eta_{t=0}) &= a_0 + a_1 \log\log(\eta_{orig}) \\ a_0 &= 0.054405 + 0.004082 \times code \\ a_1 &= 0.972035 + 0.010886 \times code \end{aligned} \tag{2.2.6}$$

where

$\eta_{t=0}$ = mix/lay-down viscosity, cP.
η_{orig} = original viscosity, cP.
code = hardening ratio (0 for average).

Table 2.2.4. Recommended code values.

Mix/Lay-Down Hardening Resistance	Expected Hardening Ratio Values	Code Value
Excellent to Good	HR ≤ 1.030	-1
Average	1.030 < HR ≤ 1.075	0
Fair	1.075 < HR ≤ 1.100	1
Poor	HR > 1.100	2

Equation 2.2.7 presents the in-service viscosity aging model for surface conditions. The model is a hyperbolic function and includes the effect of environment on the long-term aging. The environmental considerations enter through the use of the mean annual air temperature in the parameter A.

$$\log\log(\eta_{aged}) = \frac{\log\log(\eta_{t=0}) + At}{1 + Bt} \tag{2.2.7}$$

where

A = $-0.004166+1.41213(C)+(C)\log(Maat)+(D)\log\log\eta_{t=0})$
B = $0.197725+0.068384\log(C)$
C = $10^{(274.4946-193.831\log(T_R)+33.9366\log(T_R)^2}$
D = $-14.5521+10.47662\log(T_R)-1.88161\log(T_R)^2$
η_{aged} = aged viscosity, cP.
$\eta_{t=0}$ = viscosity at mix/lay-down, cP.
Maat = mean annual air temperature, °F.
T_R = temperature in Rankine.
t = time in months.

The air void adjustment factor adjusts the viscosity from the surface aging model for air void effects. Equation 2.2.8 presents the equation to adjust the aged viscosity for air void. The air voids adjustment factor, F_v, is a function of the air voids at the time of interest as shown in equation 2.2.9. The air voids at the time of interest can in turn be estimated from the initial air voids using equation 2.2.10.

$$\log\log(\eta_{aged})' = F_v \log\log(\eta_{aged}) \qquad (2.2.8)$$

$$F_v = \frac{1 + 1.0367\times10^{-4}(VA)(t)}{1 + 6.1798\times10^{-4}(t)} \qquad (2.2.9)$$

$$VA = \frac{VA_{orig} + 0.011(t) - 2}{1 + 4.24\times10^{-4}(t)(Maat) + 1.169\times10^{-3}\left(\dfrac{t}{\eta_{orig,77}}\right)} + 2 \qquad (2.2.10)$$

where

VA_{orig} = initial air voids.
t = time in months.
Maat = mean annual air temperature, °F.
$\eta_{orig,77}$ = original binder viscosity at 77 °F, MPoise.

Finally, the depth model describes the aged viscosity as a function of depth based on the aged viscosity from the surface aging model and viscosity at mix/lay-down. Equation 2.2.11 presents the viscosity-depth relationship.

$$\eta_{t,z} = \frac{\eta_t(4+E) - E(\eta_{t=0})(1-4z)}{4(1+Ez)} \qquad (2.2.11)$$

where

$\eta_{t,z}$ = Aged viscosity at time t, and depth z, MPoise
η_t = Aged surface viscosity, MPoise
z = Depth, in
E = $23.83e^{(-0.0308\ Maat)}$
Maat = Mean annual air temperature, °F

As discussed in the Master Curve and Shift Factors section, the aged viscosity can then be used in equation 2.2.4 (in the $\log[t_r]$ term) to arrive at combined shift factors that account for both temperature and aging effects. Using these shift factors and the master curve for the original mixture, the dynamic modulus for any depth, age, temperature, and rate of loading can be determined.

Summary

In summary, dynamic modulus master curves in the form of a sigmoidal function can be developed either by shifting laboratory test data or by using the dynamic modulus equation. In both cases, the resulting shift factors can be expressed as a function of binder viscosity. Expressing the shift factors as a function of binder viscosity allows the consideration of binder aging using the Global Aging System. The Global Aging System provides a series of models for adjusting the viscosity of the original binder for short-term aging that occurs during mixing and lay-down operations and for long-term aging during service.

Implementation at Various Hierarchical Levels

This Guide presents a unified approach to the characterization of asphalt concrete stiffness properties. The same basic approach and models is used at all input levels; the primary difference is the amount of laboratory test data required. Figure 2.2.3 presents a general flow diagram for determining dynamic modulus and Poisson's ratio (to be discussed later) for any rate of loading, temperature, and age. The sections below describe the data requirements at each input levels in detail.

Input Level 1 —Required Test Data

At Level 1 actual laboratory test data are required to develop the master curve and shift factors. The laboratory data requirements for the Level 1 analysis are summarized in table 2.2.5. The temperature and frequencies given in table 2.2.5 are recommended values. However, up to 8 temperatures and 6 frequencies can be used. Test data on both the mixture and the asphalt binder are needed for this analysis. To account for short-term aging that occurs during mixing and compaction, the mixture testing should be performed after short-term oven aging in accordance with AASHTO R30, "Standard Practice for Mixture Conditioning of Hot-Mix Asphalt (HMA)" and the binder testing should be performed after Rolling Thin Film Oven Test aging (AASHTO T240) in accordance with AASHTO T315.

The mixture data consist of dynamic modulus frequency sweep tests on replicate specimens for five temperatures and four rates of loading. The test should be conducted in accordance with NCHRP 1-28A using the temperatures and loading rates specified in table 2.2.5. Replicate specimens can be used in lieu of the triplicate specimens specified in NCHRP 1-28A. The specimens should have a diameter of 100 mm and a height-to-diameter ratio of 1.5. They should be tested using lubricated ends, and axial deformations should be measured over the middle 100 mm of the specimen at three locations around the circumference. Specimens meeting these size requirements can be cored from the middle of gyratory compacted specimens.

Additionally, binder complex modulus and phase angle data are needed over a range of temperatures for a loading rate of 1.59 Hz (10 rad/sec). Although it is desirable to measure the binder properties over the same temperature range, dynamic shear rheometer measurements using the normal parallel plate geometry are subject to significant compliance errors below about 40 °F. Alternatively, the binder may be characterized using the series of conventional binder tests summarized in table 2.2.6.

Penetration data can be converted to viscosity using equation 2.2.12:

$$\log \eta = 10.5012 - 2.2601 \log(Pen) + .00389 \log(Pen)^2 \qquad (2.2.12)$$

where

η	=	viscosity, in Poise
Pen	=	penetration for 100 g, 5 sec loading, mm/10

LEVEL I Test Data

$$\log(E^*) = \delta + \frac{\alpha}{1+e^{\beta+\gamma\left[\log(t)-c\left(\log(\eta)-\log(\eta_{T_r})\right)\right]}}$$

$$\log(t_r) = \log(t) - c\left(\log(\eta) - \log(\eta_{T_r})\right)$$

Determine: α,δ,β,γ, and c by nonlinear optimization

LEVEL II and III Dynamic Modulus Equation

$$\log(E^*) = d + \frac{a}{1+e^{b+g_1\log(f)+g_2(\log\eta)}}$$

$$\log(t_r) = \log(t) - \frac{g_2}{g_1}\left(\log(\eta) - \log(\eta_{T_r})\right)$$

a,b,d,g_1, and g_2 known from dynamic modulus equation

Step 1

Develop Master Curve and Shift Factors at 70 F for Original Condition

Step 2

Compute Aged Viscosity From Global Aging Model

- Mix/Laydown
- Long Term
- Depth

Step 3

Calculate Shift Factors Using Appropriate Viscosity

Level I:

$$\log(t_r) = \log(t) - c\left(\log(\eta) - \log(\eta_{T_r})\right)$$

c determined experimentally in Step 1.

Level II and III:

$$\log(t_r) = \log(t) - \frac{g_2}{g_1}\left(\log(\eta) - \log(\eta_{T_r})\right)$$

Step 4

Calculate Dynamic Modulus

Level I:

$$\log(E^*) = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log t_r)}}$$

Level II and III:

$$\log(E^*) = d + \frac{a}{1+e^{b+g_1\log(f)+g_2(\log\eta)}}$$

Step 5

Calculate Poisson's ratio

$$\mu = 0.15 + \frac{0.35}{1+e^{(-12.452+2.291\log E^*)}}$$

Figure 2.2.3. Flowchart for asphalt concrete material characterization.

Table 2.2.5. Summary of required characterization tests at input Level 1.

Temperature, °F	**Mixture E* and δ[1]**				**Binder G* and δ[2] 1.59 Hz**
	0.1 Hz	**1 Hz**	**10 Hz**	**25 Hz**	
10	X	X	X	X	
25					
40	X	X	X	X	X
55					X
70	X	X	X	X	X
85					X
100	X	X	X	X	X
115					X
130	X	X	X	X	X

[1] Testing to be performed in accordance with NCHRP 1-28A.
[2] Testing to be performed in accordance with AASHTO T315.

Table 2.2.6. Conventional binder test data.

Number	**Test**	**Temp, °C**	**Conversion to Viscosity, Poise**
1	Penetration	15	See equation 2.2.12
2	Penetration	25	See equation 2.2.12
3	Brookfield Viscosity	60	None
4	Brookfield Viscosity	80	None
5	Brookfield Viscosity	100	None
6	Brookfield Viscosity	121.1	None
7	Brookfield Viscosity	135	None
8	Brookfield Viscosity	176	None
9	Softening Point	Measured	13,000 Poise
10	Absolute Viscosity	60	None
11	Kinematic Viscosity	135	Value x 0.948

Input Level 1—Development of Master Curve from Test Data

The master curve at Level 1 is developed using numerical optimization to shift the laboratory mixture test data into a smooth master curve. Prior to shifting the mixture data, the relationship between binder viscosity and temperature must be established. This is done by first converting the binder stiffness data at each temperature to viscosity using equation 2.2.13 (see Appendix CC*)*. The parameters of the ASTM VTS equation are then found by linear regression of equation

2.2.14 after log-log transformation of the viscosity data and log transformation of the temperature data.

$$\eta = \frac{G^*}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628} \tag{2.2.13}$$

$$\log\log\eta = A + VTS\log T_R \tag{2.2.14}$$

where

G*	=	binder complex shear modulus, Pa.
δ	=	binder phase angle, °.
η	=	viscosity, cP.
T_R	=	temperature in Rankine at which the viscosity was estimated.
A, VTS	=	regression parameters.

If the alternative binder characterization using conventional testing is used, the data are first converted to common viscosity units as specified in table 2.2.6, and the regression of equation 2.2.14 is performed.

The resulting regression parameters from equation 2.2.14 can then be used to calculate the viscosity for any temperature. Care must be taken when using this equation at low temperatures. It is well known that asphalt binders approach a maximum viscosity of $2.7\text{x}10^{10}$ Poise at low temperatures and high rates of loading. Thus, the viscosity at low temperatures is equal to the lesser of that calculated using equation 2.2.14 or $2.7\text{x}10^{10}$ Poise. After the viscosity–temperature relationship is established, the laboratory test data can be shifted into a smooth master curve using equation 2.2.15:

$$\log(E^*) = \delta + \frac{\alpha}{1 + e^{\beta+\gamma\left[\log(t)-c\left(\log(\eta)-\log(\eta_{T_r})\right)\right]}} \tag{2.2.15}$$

where

E*	=	Dynamic modulus, psi
t	=	Time of loading, sec
η	=	Viscosity at temperature of interest, CPoise
η_{Tr}	=	Viscosity at reference temperature, CPoise
$\alpha,\beta,\delta,\gamma,c$	=	Mixture specific fitting parameters.

Table 2.2.7 presents a summary of the data required for the numerical optimization. It includes 40 measurements of dynamic modulus on 2 replicate specimens, and 5 viscosity values calculated from equation 2.2.14. The numerical optimization of equation 2.2.15 is performed very efficiently using a gradient search algorithm. The optimization will return five model parameters: α, β, γ, δ, and c, which define the dynamic modulus' time-temperature dependency over the design life.

Table 2.2.7. Data spreadsheet for numerical optimization for Level 1 E* estimation.

Temperature, °F	**Rate, Hz**	**Measured E***	**Binder Viscosity[1]**
10	0.1	Specimen 1	η_{10}
10	1	Specimen 1	η_{10}
10	10	Specimen 1	η_{10}
10	25	Specimen 1	η_{10}
10	0.1	Specimen 2	η_{10}
10	1	Specimen 2	η_{10}
10	10	Specimen 2	η_{10}
10	25	Specimen 2	η_{10}
40	0.1	Specimen 1	η_{40}
40	1	Specimen 1	η_{40}
40	10	Specimen 1	η_{40}
40	25	Specimen 1	η_{40}
40	0.1	Specimen 2	η_{40}
40	1	Specimen 2	η_{40}
40	10	Specimen 2	η_{40}
40	25	Specimen 2	η_{40}
70[2]	0.1	Specimen 1	η_{70}
70[2]	1	Specimen 1	η_{70}
70[2]	10	Specimen 1	η_{70}
70[2]	25	Specimen 1	η_{70}
70[2]	0.1	Specimen 2	η_{70}
70[2]	1	Specimen 2	η_{70}
70[2]	10	Specimen 2	η_{70}
70[2]	25	Specimen 2	η_{70}
100	0.1	Specimen 1	η_{100}
100	1	Specimen 1	η_{100}
100	10	Specimen 1	η_{100}
100	25	Specimen 1	η_{100}
100	0.1	Specimen 2	η_{100}
100	1	Specimen 2	η_{100}
100	10	Specimen 2	η_{100}
100	25	Specimen 2	η_{100}
130	0.1	Specimen 1	η_{130}
130	1	Specimen 1	η_{130}
130	10	Specimen 1	η_{130}
130	25	Specimen 1	η_{130}
130	0.1	Specimen 2	η_{130}
130	1	Specimen 2	η_{130}
130	10	Specimen 2	η_{130}
130	25	Specimen 2	η_{130}

[1] From equation 2.2.14, limited to 2.7×10^{10} Poise
[2] Reference temperature

Input Level 1—Shift Factors

Equation 2.2.16 presents the shift factors for the master curve. The parameter c is one of the values returned by the numerical optimization. The shift factors depend only on the binder viscosity for the age and temperature of interest, and the Rolling Thin Film Oven (RTFO) aged viscosity at the reference temperature. The viscosity at the age and temperature of interest is obtained from the global aging model.

$$\log(t_r) = \log(t) - c\left(\log(\eta) - \log(\eta_{T_r})\right) \qquad (2.2.16)$$

where

t_r = Reduced time, sec
t = Loading time, sec
η = Viscosity at the age and temperature of interest, CPoise
η_{Tr} = Viscosity at reference temperature and RTFO aging, CPoise

Input Level 2 —Required Test Data

At Level 2, the dynamic modulus equation is combined with specific laboratory test data from the binder grade being considered for the use in the pavement, to derive the E* values over the design life. The input data required by the dynamic modulus equation are summarized in table 2.2.8. Table 2.2.9 summarizes the binder test data needed for dynamic modulus estimation at input Level 2. This is the same binder test data needed for the Level 1 analysis. It consists of measurements of binder complex shear modulus (G*) and phase angle over a range of temperatures at 1.59 Hz (10 rad/sec). These measurements should be made after RTFOT aging to simulate short-term plant aging conditions. Alternatively, the binder may be characterized using the series of conventional binder tests previously summarized in table 2.2.6. Representative values for this equation should be available from past mixture design reports.

Table 2.2.8. Dynamic modulus equation input data at Level 2.

Parameter	Description	Units
η	Binder Viscosity	10^6 Poise
F	Loading Rate	Hz
V_a	Air Void Content	Percent by total mixture volume
Vb_{eff}	Effective Asphalt Content	Percent by total mixture volume
ρ_{34}	Cumulative Percent Retained On ¾ in Sieve	Cumulative percent by weight of aggregate
ρ_{38}	Cumulative Percent Retained On 3/8 in Sieve	Cumulative percent by weight of aggregate
ρ_4	Cumulative Percent Retained On #4 Sieve	Cumulative percent by weight of aggregate
ρ_{200}	Percent Passing #200 Sieve	Percent by weight of aggregate

Table 2.2.9. Summary of required Level 2 binder test data.

Temperature, °F	Binder G* and δ[1]
40	X
55	X
70	X
85	X
100	X
115	X
130	X

[1] Testing performed in accordance with AASHTO T315.

Input Level 2—Development of Master Curve from Test Data

The master curve for the Level 2 analysis is developed from the dynamic modulus equation (equation 2.2.4) using actual binder test data. The parameter δ in the equation is a function of aggregate gradation, the effective binder content, and the air void content. The parameter α is only a function of the aggregate gradation. Thus, the minimum and maximum modulus values are independent of the binder stiffness. They are functions of aggregate gradation, effective binder content, and air void content and are determined from volumetric mixture data. This is quite rational since binders tend toward similar stiffnesses at very low temperatures. At very high temperatures, internal friction dominates and the influence of the binder stiffness is small. The rate at which the modulus changes from the maximum to the minimum depends on the characteristics of the binder.

To account for binder effects at input Level 2 requires the establishment of a relationship between binder viscosity and temperature. If AASHTO T315 test data are available, this is done by first converting the binder stiffness data at each temperature to viscosity using equation 2.2.13. The parameters of the ASTM VTS equation are then found by linear regression of equation 2.2.14 after log-log transformation of the viscosity data and log transformation of the temperature data.

If the alternative binder characterization using conventional testing is used (as shown in table 2.2.6), the data are first converted to common viscosity units as specified in table 2.2.6, and the regression of equation 2.2.14 is performed. The resulting regression parameters from equation 2.2.14 can then be used to calculate the viscosity for any temperature.

Care must be taken when using equation 2.2.14 at low temperatures. It is well known that, at low temperatures and high rates of loading, asphalt binders approach a constant maximum viscosity of 2.7×10^{10} Poise. Thus, in application, the viscosity at low temperatures is equal to the lesser of that calculated using equation 2.2.14 or 2.7×10^{10} Poise.

After the viscosity–temperature relationship is established, the master curve can be developed using equation 2.2.4. Representative mixture data should be used to establish δ and α.

Level 2—Shift Factors

Equation 2.2.17 presents the equation to derive the shift factors for the master curve. The shift factors depend only on the binder viscosity for the age and temperature of interest and the RTFOT aged viscosity at the reference temperature. The viscosity at the age and temperature of interest are obtained from the global aging model.

$$\log(t_r) = \log(t) - 1.25588\left(\log(\eta) - \log(\eta_{T_r})\right) \qquad (2.2.17)$$

where

- t_r = Reduced time, sec.
- t = Loading time, sec.
- η = Viscosity at the age and temperature of interest, CPoise.
- η_{Tr} = Viscosity at reference temperature and RTFO aging, CPoise.

Input Level 3—Required Test Data

At input Level 3, equation 2.2.4 is used to estimate the dynamic modulus just as was outlined for Level 2. However, no laboratory test data are required. Table 2.2.8 presents the data required by the dynamic modulus equation. With the exception of the binder viscosity and loading rate, the mixture data required for the dynamic modulus equation can be obtained directly from representative data for similar mixtures. The binder viscosity information as a function of time can be estimated from typical temperature–viscosity relationships that have been established for a variety of asphalt grades derived from different grading systems.

Default A and VTS Parameters

To estimate the viscosity of the asphalt binder at any given temperature and age (a required input to the dynamic modulus equation), the A and VTS parameters for temperature-viscosity relationship (equation 2.2.14) must be known *a priori.* At Level 3, since no testing is required, these values can be estimated if any of the following binder-related information is known:

- Performance Grade (PG) of the asphalt binder based on AASHTO M320.
- Viscosity grade of asphalt binder based on AASHTO M 226.
- Penetration grade of asphalt binder based on AASHTO M 20.

The recommended default A and VTS values based on each of these criteria are presented in tables 2.2.10 through 2.2.12.

Level 3—Development of Master Curve from Test Data

The procedure to develop the time-temperature dependent asphalt dynamic modulus master curve at input Level 3 is similar to that described earlier for input Level 2. The only exception is the usage of default A and VTS parameters in place of those derived from laboratory testing.

Table 2.2.10. Recommended RTFO A and VTS parameters based on asphalt PG grade (27).

High Temp Grade	Low Temperature Grade													
	-10		-16		-22		-28		-34		-40		-46	
	VTS	A	VTS	A	VTS	A	VTS	A	VTS	A	VTS	A	VTS	A
46									-3.901	11.504	-3.393	10.101	-2.905	8.755
52	-4.570	13.386	-4.541	13.305	-4.342	12.755	-4.012	11.840	-3.602	10.707	-3.164	9.496	-2.736	8.310
58	-4.172	12.316	-4.147	12.248	-3.981	11.787	-3.701	11.010	-3.350	10.035	-2.968	8.976		
64	-3.842	11.432	-3.822	11.375	-3.680	10.980	-3.440	10.312	-3.134	9.461	-2.798	8.524		
70	-3.566	10.690	-3.548	10.641	-3.426	10.299	-3.217	9.715	-2.948	8.965	-2.648	8.129		
76	-3.331	10.059	-3.315	10.015	-3.208	9.715	-3.024	9.200	-2.785	8.532				
82	-3.128	9.514	-3.114	9.475	-3.019	9.209	-2.856	8.750	-2.642	8.151				

Table 2.2.11. Recommended RTFOT A and VTS parameters based on asphalt viscosity grade (see Appendix CC) (28).

Grade	***A***	***VTS***
AC-2.5	11.5167	-3.8900
AC-5	11.2614	-3.7914
AC-10	11.0134	-3.6954
AC-20	10.7709	-3.6017
AC-30	10.6316	-3.5480
AC-40	10.5338	-3.5104

Table 2.2.12. Recommended RTFOT A and VTS parameters based on asphalt penetration grade (see Appendix CC) (28).

Grade	***A***	***VTS***
40-50	10.5254	-3.5047
60-70	10.6508	-3.5537
85-100	11.8232	-3.6210
120-150	11.0897	-3.7252
200-300	11.8107	-4.0068

Level 3—Shift Factors

The procedure to derive the shift factors at input Level 3 is similar to that shown for input Level 2. The shift factors depend only on the binder viscosity for the temperature of interest and the RTFOT aged viscosity at the reference temperature. The viscosity at the temperature of interest is obtained from the global aging model presented in equation 2.2.17.

2.2.2.2 Layer Modulus for Rehabilitation Design

The determination of the asphalt layer dynamic modulus for rehabilitation design follows the same general concepts as for new or reconstruction design, with the exceptions noted in the discussion below. Table 2.2.13 summarizes the approach utilized to determine the layer dynamic modulus for rehabilitation design.

<u>Input Level 1 – Approach to Determine Field Damaged Dynamic Modulus Master Curve</u>

1. Conduct nondestructive testing in the outer wheelpath using the Falling Weight Deflectometer (FWD) over the project to be rehabilitated and compute the mean backcalculated asphalt bound modulus, Ei, for the project. Be sure to include cracked as well as uncracked areas. The corresponding asphalt pavement temperature at the time of testing should also be recorded. Perform coring to establish layer thickness along the project. Layer thickness can also be determined using ground penetrating radar (GPR). See PART 2, Chapter 5. Backcalculate the E_i asphalt bound modulus by combining layers with similar properties at each FWD test point along the project (with known pavement temperature).

2. Perform field coring and establish mix volumetric parameters (air voids, asphalt volume, gradation), and asphalt viscosity parameters to define Ai-VTSi values required for computing dynamic modulus using the equation 2.2.4. Establish binder viscosity-temperature properties (A and VTS parameters) as was done for Level 1 inputs for new or reconstruction design.

3. Develop undamaged dynamic modulus master curve from the data collected in step 2 using the equation 2.2.4.

4. Estimate damage, dj, expressed as follows:

$$d_j = \frac{Ei}{E^*} \tag{2.2.18}$$

   where,

   Ei = Backcalculated modulus at a given reference temperature recorded in the field.

   E* = Predicted modulus at the same temperature as above from equation 2.2.4.

5. Determine α' as shown below:

$$\alpha' = (1\text{-}dj)\ \alpha \tag{2.2.19}$$

Determine field damaged master curve using α' instead α in equation 2.2.4.

Table 2.2.13. Asphalt dynamic modulus (E^*) estimation at various hierarchical input levels for rehabilitation design.

Material Group Category	Type Design	Input Level	Description
Asphalt Materials (existing layers)	Rehab	1	• Use NDT-FWD backcalculation approach. Measure deflections, backcalculate (combined) asphalt bound layer modulus at points along project. • Establish backcalculated Ei at temperature-time conditions for which the FWD data was collected along project. • Obtain field cores to establish mix volumetric parameters (air voids, asphalt volume, gradation, and asphalt viscosity parameters to determine undamaged Master curve). • Develop, by predictive equation, undamaged Master curve with aging for site conditions. • Estimate damage, dj, by: $dj = Ei(NDT)/E^*(Pred)$ • In sigmoidal function (equation 2.2.4), δ is minimum value and α is specified range from minimum. • Define new range parameter α' to be: $\alpha' = (1-dj)\ \alpha$ • Develop field damaged master curve using α' rather than α
		2	• Use field cores to establish mix volumetric parameters (air voids, asphalt volume, gradation, and asphalt viscosity parameters to define Ai-VTSi values). • Develop by predictive equation, undamaged master curve with aging for site conditions from mix input properties determined from analysis of field cores. • Conduct indirect Mr laboratory tests, using revised protocol developed at University of Maryland for NCHRP 1-28A from field cores. • Use 2 to 3 temperatures below 70 ^{o}F. • Estimate damage, dj, at similar temperature and time rate of load conditions: $dj = Mri/E^*(Pred)$ • In sigmoidal function (eq. 2.2.1), δ is minimum value and α is specified range from minimum. Define new range parameter α' to be: $\alpha' = (1-dj)\ \alpha$ • Develop field damaged master curve using α' rather than α
		3	• Use typical estimates of mix modulus prediction equation (mix volumetric, gradation and binder type) to develop undamaged master curve with aging for site layer. • Using results of distress/condition survey, obtain estimate for pavement rating (excellent, good, fair, poor, very poor) • Use a typical tabular correlation relating pavement rating to pavement layer damage value, dj. • In sigmoidal function, δ is minimum value and α is specified range from minimum. Define new range parameter α' to be: $\alpha' = (1-dj)\ \alpha$ • Develop field damaged master curve using α' rather than α

Input Level 2 – Approach to Determine Field Damaged Dynamic Modulus Master Curve

1. Perform field coring and establish mix volumetric parameters (air voids, asphalt volume, gradation) and asphalt viscosity parameters to define Ai-VTSi values.

2. Develop undamaged dynamic modulus master curve from the data collected in step 2 using equation 2.2.4.

3. Conduct indirect resilient modulus, M_{ri}, laboratory tests using revised protocol developed at University of Maryland for NCHRP 1-28A from field cores. Use two to three temperatures below 70 ^{o}F.
4. Estimate damage, d_j, at similar temperature and time rate of load conditions using the expression below:

$$d_j = \frac{M_{ri}}{E^*} \qquad (2.2.20)$$

where,

M_{ri} = Laboratory estimated resilient modulus at a given reference temperature.
E^* = Predicted modulus at the same temperature as above from equation 2.2.4.

5. Determine α’ as shown below:

$$\alpha' = (1\text{-}dj)\ \alpha \qquad (2.2.21)$$

6. Determine field damaged master curve using α’ instead α in equation 2.2.4.

Input Level 3 – Approach to Determine Field Damaged Dynamic Modulus Master Curve

1. Use typical estimates of mix parameters (mix volumetric, gradation and binder type) to develop undamaged master curve with aging for the in situ pavement layer using equation 2.2.4.

2. Using results of the distress/condition survey, obtain estimates of the pavement rating – excellent, good, fair, poor, very poor (see PART 3, Chapter 6 for details).

3. Use pavement rating to estimate(asphalt bound) pavement layer damage value, d_j. Discussions on the estimation of the damage factor from visual condition data are available in PART 3, Chapter 6.

4. Determine α' as shown below:

$$\alpha' = (1\text{-}dj)\ \alpha \qquad (2.2.22)$$

5. Develop field damaged master curve using α’ rather than α in equation 2.2.4.

2.2.2.3 Poisson's Ratio for Bituminous Materials

Poisson's ratio for bituminous road materials normally ranges between 0.15 and 0.50 and is a function of temperature. Ideally, at input Level 1 Poisson's ratio would be estimated from laboratory testing; however, a Level 1 approach to determine Poisson's ratio is not warranted until such time when pavement response models that utilize non-linear moduli to model dilation effects on the pavement response can be implemented in the design procedure on a routine basis. Therefore, the use of correlations or typical assumed values for analysis can be considered satisfactory (see Appendix CC).

Hot-Mix Asphalt—Dense Graded

This category includes virgin (conventional) and recycled HMA mixtures, as well as "asphalt-treated" mix categories.

Level 2: User-Defined Input

Under Level 2, there are three sublevels to estimate the Poisson's ratio. These are explained below.

- Level 2A: Use equation 2.2.23 along with user entered values for parameters a and b estimated for specific mixtures.

$$\mu_{ac} = 0.15 + \frac{0.35}{1 + e^{(a + b E_{ac})}} \qquad (2.2.23)$$

  where,
  μ_{ac} = Poisson's ratio of asphalt mixture at a specific temperature.
  E_{ac} = Modulus of asphalt mixture at a specific temperature, psi.

  The a and b parameters can be developed from regression analysis of laboratory estimated mixture modulus values and Poisson's ratios (see Appendix CC).

- Level 2B: Use equation 2.2.24 (with typical a and b values) to estimate Poisson's ratio.

$$\mu_{ac} = 0.15 + \frac{0.35}{1 + e^{(-1.63 + 3.84x10^{-6} E_{ac})}} \qquad (2.2.24)$$

- Level 2C: Select from typical range of possible Poisson's ratios (see table 2.2.14).

Level 3: Typical Poisson's Ratios

Tables 2.2.15 presents the typical Poisson's ratios that can be used at input Level 3.

Open-Graded Asphalt Treated Materials

This category includes asphalt treated permeable base (ATPB) materials.

Table 2.2.14. Typical Poisson's ratio ranges at input Level 2C for dense-graded HMA.

Temperature °F	**Level 2C** μ_{range}
< 0 °F	< 0.15
0 – 40 °F	0.15 – 0.20
40 – 70 °F	0.20 – 0.30
70 – 100 °F	0.30 – 0.40
100 – 130 °F	0.40 – 0.48
> 130 °F	0.45 – 0.48

Table 2.2.15. Typical Poisson's ratios at input Level 3 for dense-graded HMA.

Temperature °F	**Level 3** $\mu_{typical}$
< 0 °F	0.15
0 – 40 °F	0.20
40 – 70 °F	0.25
70 – 100 °F	0.35
100 – 130 °F	0.45
> 130 °F	0.48

Level 2: User-Defined Input

At this input level, the Poisson's ratios for ATPB materials can be selected from the possible ranges of Poisson's ratio values presented in table 2.2.16.

Table 2.2.16. Typical Poisson's ratio ranges at input Level 2 for ATPB.

Temperature °F	**Level 2** μ_{range}
< 40 °F	0.30 – 0.40
40 – 100 °F	0.35 – 0.40
> 100 °F	0.40 – 0.48

Level 3: Typical Poisson's Ratios

Tables 2.2.17 presents the typical Poisson's ratios that can be used at input Level 3 for ATPB materials.

Table 2.2.17. Typical Poisson's ratios at input Level 3 for open-graded ATPB.

Temperature °F	Level 3$\mu_{typical}$
< 40 °F	0.35
40 – 100 °F	0.40
> 100 °F	0.45

<u>Cold-Mix Asphalt (CMA) Materials</u>

This category includes both conventional cold-mix asphalt and recycled cold-mix materials.

Level 2: User-Defined Input

At this input level, the Poisson's ratios for CMA materials can be selected from the possible ranges of Poisson's ratio values presented in table 2.2.18.

Table 2.2.18. Typical Poisson's ratio ranges at input Level 2 for CMA.

Temperature °F	Level 2 μ_{range}
< 40 °F	0.20 - 0.35
40 – 100 °F	0.30 – 0.45
> 100 °F	0.40 – 0.48

Level 3: Typical Poisson's Ratios

Tables 2.2.19 presents the typical Poisson's ratios that can be used at input Level 3 for ATPB materials.

Table 2.2.19. Typical Poisson's ratios at input Level 3 for open-graded ATPB.

Temperature °F	Level 3 $\mu_{typical}$
< 40 °F	0.30
40 – 100 °F	0.35
> 100 °F	0.45

2.2.2.4 Other HMA Material Properties

Additional HMA material properties are required for use in predicting HMA thermal cracking. Note that HMA thermal cracking is material related and hence its development and progression are determined by the HMA properties along with climatic condition. The HMA properties that are used to predict thermal cracking are:

- Tensile strength.
- Creep compliance.
- Coefficient of thermal contraction.
- Surface shortwave absorptivity.
- Thermal conductivity and heat capacity.

Levels of inputs for these HMA properties are described in the following sections.

<u>Tensile Strength</u>

Level 1—Required Test Data

At level 1 actual laboratory test data for HMA tensile strength at 14 ^{o}F is required. Testing should be done in accordance with AASHTO T322, "Determining the Creep Compliance and Strength of Hot Mix Asphalt (HMA) Using the Indirect Tensile Test Device."

Level 2—Test Data

At level 1 actual laboratory test data for HMA tensile strength at 14 ^{o}F is required. Testing should be done in accordance with AASHTO T322.

Level 3—Correlations with Other HMA Properties

The tensile strength for level 3 is based on regression equation developed under NCHRP1-37A as explained in PART 3 Chapter 3 of this guide. The tensile strength equation is given by equation 2.2.25.

$$\begin{aligned} TS(psi) = {} & 7416.712 - 114.016 * Va - 0.304 * Va^2 - 122.592 * VFA + 0.704 * VFA^2 \\ & + 405.71 * Log10(Pen77) - 2039.296 * log10(A) \end{aligned} \qquad (2.2.25)$$

where:

TS = indirect tensile strength at 14 ^{o}F
V_a = as construction HMA air voids, %
VFA = as construction voids filled with asphalt, %
Pen77 = binder penetration at 77 ºF, mm/10
A = viscosity-temperature susceptibility intercept.

Creep Compliance

Level 1—Required Test Data

At level 1 actual laboratory test data for HMA creep compliance is required. The specific data requirements are presented in table 2.2.20. Testing should be done in accordance with AASHTO T322.

Table 2.2.20. Summary of required characterization tests for HMA creep compliance at level 1.

Time of Loading	**Temperature, °F**		
	-4	**14**	**32**
1	X	X	X
2	X	X	X
5	X	X	X
10	X	X	X
20	X	X	X
50	X	X	X
100	X	X	X

Level 2—Test Data

At level 2 actual laboratory test data for HMA creep compliance is required only for the intermediate temperature of 14 °F. Testing should be done in accordance with AASHTO T322.

Level 3— Correlations with Other HMA Properties

The creep compliance for level 3 is based on regression equation developed under NCHRP1-37A as explained in PART 3 Chapter 3 of this guide. The creep compliance equation is given by equation 2.2.26.

$$D(t) = D_1 * t^m \tag{2.2.26a}$$

$$\begin{aligned}\log(D_1) = &-8.524 + 0.01306 * \text{Temp} + 0.7957 * \log10(\text{Va}) + 2.0103 * \log10(\text{VFA}) \\ &-1.923 * \log10(\text{A})\end{aligned} \tag{2.2.26b}$$

$$\begin{aligned}m = &1.1628 - 0.00185 * \text{Temp} - 0.04596 * \text{Va} - 0.01126 * \text{VFA} + 0.00247 * \text{Pen77} \\ &+ 0.001683 * \text{Temp} * \text{Pen77}^{0.4605}\end{aligned} \tag{2.2.26c}$$

where:

Temp = temperature at which creep compliance is measured, °F.
Va = as construction air voids, %
VFA = as construction voids filled with asphalt, %
Pen77 = binder penetration at 77 °F, mm/10

Coefficient of Thermal Contraction

There are no AASHTO or ASTM standard tests for determining the coefficient of thermal contraction (CTC) of HMA materials. The Design Guide software computes CTC internally using the HMA volumetric properties such as VMA and the thermal contraction coefficient for the aggregates. The model used to estimate CTC for asphalt concrete mixtures is shown as equation 2.2.27. Note that this model is suitable for the lower temperature ranges and is a modified version of the relationship proposed by Jones et al. (*3*).

$$L_{MIX} = \frac{VMA * B_{ac} + V_{AGG} * B_{AGG}}{3 * V_{TOTAL}} \qquad (2.2.27)$$

where

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

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

- L_{MIX} = 2.2 to $3.4*10^{-5}$ /°F (linear).
- B_{ac} = 3.5 to $4.3*10^{-4}$ /°C (cubic).
- B_{AGG} = 21 to $37*10^{-6}$ /°C (cubic) (see table 2.2.39 for B_{AGG} of specific aggregate types).

Surface Shortwave Absorptivity

The surface short wave absorptivity of a given layer depends on its composition, color, and texture. This quantity directly correlates with the amount of available solar energy that is absorbed by the pavement surface. Generally speaking, lighter and more reflective surfaces tend to have lower short wave absorptivity and vice versa. The following are the recommended ways to the estimate this parameter at each of the hierarchical input levels:

- Level 1 – At this level it is recommended that this parameter be estimated through laboratory testing. There are no current AASHTO certified standards for estimating shortwave absorptivity of paving materials.
- Level 2 – Correlations are not available. Use default values from Level 3.
- Level 3 – At Level 3, default values can be assumed for various pavement materials as follows:

- Weathered asphalt (gray) 0.80 – 0.90
- Fresh asphalt (black) 0.90 – 0.98

Thermal Conductivity and Heat Capacity

Thermal conductivity, *K*, is the quantity of heat that flows normally across a surface of unit area per unit of time and per unit of temperature gradient. The moisture content has an influence upon the thermal conductivity of asphalt concrete. If the moisture content is small, the differences between the unfrozen, freezing and frozen thermal conductivity are small. Only when the moisture content is high (e.g., greater than 10%) does the thermal conductivity vary substantially. The EICM does not vary the thermal conductivity with varying moisture content of the asphalt layers as it does with the unbound layers. The heat or thermal capacity is the actual amount of heat energy *Q* necessary to change the temperature of a unit mass by one degree. Table 2.2.21 outlines the recommended approaches to characterizing *K* and *Q* at the various hierarchical input levels.

Table 2.2.21. Characterization of asphalt concrete materials inputs required for EICM calculations.

Material Property	Input Level	Description
Thermal Conductivity, K	1	A direct measurement is recommended at this level (ASTM E 1952, "Standard Test Method for Thermal Conductivity and Thermal Diffusivity by Modulated Temperature Differential Scanning Calorimetry").
	2	Correlations are not available. Use default values from Level 3.
	3	User selects design values based upon agency historical data or from typical values shown below: • Typical values for asphalt concrete range from 0.44 to 0.81 Btu/(ft)(hr)(^{o}F).
Heat Capacity, Q	1	A direct measurement is recommended at this level (ASTM D 2766, "Specific Heat of Liquids and Solids").
	2	Correlations are not available. Use default values from Level 3..
	3	User selects design values based upon agency historical data or from typical values shown below: • Typical values for asphalt concrete range from 0.22 to 0.40 Btu/(lb)(^{o}F).

2.2.3 INPUT CHARACTERIZATION FOR THE PCC MATERIALS

2.2.3.1 Modulus of Elasticity of PCC Materials

Overview

The ratio of stress to strain in the elastic range of a stress-strain curve for a given concrete mixture defines its modulus of elasticity (*4*). The PCC modulus of elasticity is a complex parameter that is influenced significantly by mix design parameters and mode of testing. The mixture parameters that most strongly influence elastic modulus include:

- Ratio of water to cementitious materials (w/(cm)).
- Relative proportions of paste and aggregate.

- Aggregate type.

The w/(cm) ratio is important in determining the porosity of the paste. In general, as w/(cm) increases, porosity increases and the PCC elastic modulus decreases. The degree of hydration of the cement paste also affects paste porosity; increasing hydration (e.g., longer curing, higher temperatures) and age result in decreased porosity and increased elastic modulus.

Aggregate characteristics are important in determining the elastic modulus of PCC because of their relatively high elastic modulus (compared to that of cement paste) and their control of the volumetric stability of the PCC. Higher aggregate contents and the use of high-modulus aggregates (e.g., basalt, granite, dense limestone) generally are associated with higher PCC elastic modulus values. The strength, angularity, and surface texture of the aggregate also affect the mode and rate of crack development and propagation in PCC. This, in turn, affects PCC elastic modulus. A high-strength matrix and low-strength aggregate combination will tend to fail at the aggregate/matrix interface (especially when the aggregate is smooth and round) or by fracturing the aggregate. Conversely, a low-strength matrix and high-strength aggregate will fail either at the bond interface or by development of matrix cracks.

Test parameters also affect the indicated or measured elasticity of the PCC. These test factors include specimen size, method of testing or computation (e.g., static vs. dynamic, chord modulus vs. secant modulus), rate of load application, degree of PCC saturation when tested, and PCC temperature when tested. For instance, the elastic modulus of PCC increases as the degree of saturation increases and the rate of loading increases. These effects also imply that PCC response varies significantly under service conditions.

In mechanistic pavement response analysis, the PCC elastic modulus (E_c) has a strong effect on pavement deflection and the stresses throughout the pavement structure and must be properly accounted for. The characterization of the PCC elastic modulus varies as a function of the design type. The discussion on the PCC materials modulus falls under the following three broad groupings:

- Characterization of PCC modulus for new or reconstruction JPCP and CRCP projects and PCC overlays.
- Characterization of existing PCC pavement layer modulus being considered for rehabilitation with overlays or for restoration (applicable only for JPCP).
- Characterization of fractured PCC layer modulus (crack and seat, break and seat, or rubblization).

<u>PCC Modulus Characterization for New/Reconstruction JPCP and CRCP and PCC Overlays</u>

Table 2.2.22 summarizes the procedures to estimate the PCC elastic modulus at the various input levels for freshly laid concrete materials. For these design scenarios, the modulus gain over time is considered directly, to allow more accurate accumulation of incremental damage over time.

Table 2.2.22. PCC elastic modulus estimation for new, reconstruction, and overlay design.

<table>
<tr><th>Material Group</th><th>Type of Design</th><th>Input Level</th><th>Description</th></tr>
<tr><td rowspan="3">PCC (Slabs)</td><td rowspan="3">New</td><td>1</td><td>• PCC modulus of elasticity, E_c, will be determined directly by laboratory testing. This is a chord modulus obtained from ASTM C 469 at various ages (7, 14, 28, 90-days).
• Estimate the 20-year to 28-day (long-term) elastic modulus ratio.
• Develop modulus gain curve using the test data and long-term modulus ratio to predict Ec at any time over the design life.</td></tr>
<tr><td>2</td><td>• PCC modulus of elasticity, E_c, will be determined indirectly from compressive strength testing at various ages (7, 14, 28, and 90 days). The recommended test to determine f'_c is AASHTO T22. The E_c can also be entered directly if desired.
• Estimate the 20-year to 28-day compressive strength ratio.
• Convert f'_c to E_c using the following relationship:
$E_c = 33\rho^{3/2} (f'_c)^{1/2}$ psi
• Develop modulus gain curve using the test data and long-term modulus ratio to predict E_c at any time over the design life.</td></tr>
<tr><td>3</td><td>• PCC modulus of elasticity, E_c, will be determined indirectly from 28-day estimates of flexural strength (MR) or f'_c. MR can be determined from testing (AASHTO T97) or from historical records. Likewise, f'_c can be estimated from testing (AASHTO T22) or from historical records. The E_c can also be entered directly.
• If 28-day MR is estimated, its value at any given time, t, is determined using:
$MR(t) = (1 + \log_{10}(t/0.0767) - 0.01566*\log_{10}(t/0.0767)^2)* MR_{28\text{-day}}$
• Estimate Ec(t) by first estimating $f'_c(t)$ from MR(t) and then converting $f'_c(t)$ to Ec(t) using the following relationships:
$f'_c = (MR/9.5)^2$ psi
$Ec = 33\rho^{3/2} (f'_c)^{1/2}$ psi
• If 28-day f'_c is estimated, first convert it to an MR value using equation above and then project MR(t) as noted above and from it Ec(t) over time.</td></tr>
</table>

The designer is required to provide an estimate of how modulus changes over time, as shown in figure 2.2.4. At input Level 1, the modulus-time relationships are established based on the results of testing conducted on the actual project materials; at Level 3, the relationships for typical materials are used.

Estimating PCC Elastic Modulus at Input Level 1

At input Level 1, it is recommended that the elastic modulus of the proposed PCC mixture be estimated from laboratory testing. PCC elastic modulus values for the proposed mixture are required at 7, 14, 28, and 90 days. In addition, the estimated ratio of 20-year to 28-day E_c is also a required input (a maximum value of 1.20 is recommended for this parameter). Based on these inputs, a modulus gain curve can be developed for the incremental damage analysis. The required input data at Level 1 for this parameter are summarized in table 2.2.23.

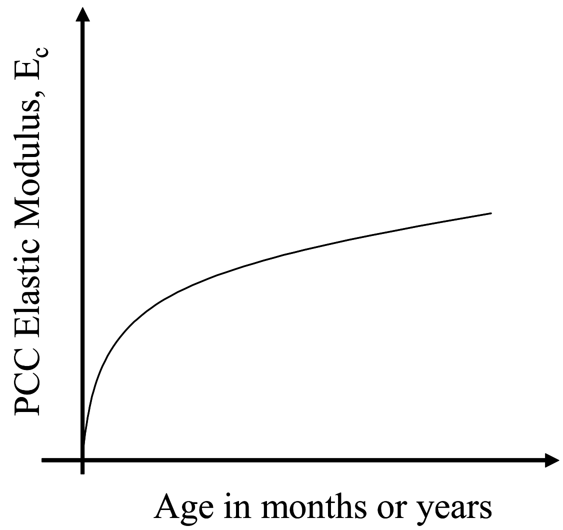


Figure 2.2.4. Modulus data required for M-E design.

Table 2.2.23. Required input data at Level 1.

Input Parameter	Required Test Data				Ratio of 20-yr/28-day Modulus	Recommended Test Procedure
	7-day	14-day	28-day	90-day		
E_c	✓	✓	✓	✓	✓	ASTM C 469

The recommended test procedure for obtaining E_c is ASTM C 469, "Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." The test provides a stress-strain ratio value (chord modulus) and a ratio of the lateral to longitudinal strain (Poisson's ratio) for hardened concrete at all ages and for all curing conditions. The E_c values obtained from this test are usually less than the moduli obtained from rapid load applications (dynamic FWD backcalculation or seismic testing conditions). The ratio is approximately 0.8. The step-by-step approach to construct the modulus gain curve at Level 1 is described below:

Step 1 – User Input: Enter E_c results from testing and the estimated ratio of 20 year to 28-day modulus (a maximum value of 1.2 is recommended based on Wood (*5*)). The 20-year to 28-day modulus ratio can be determined for agency specific mixes if historical data are available.

Step 2 – Develop Modulus Gain Curve: The modulus gain curve over time for design purposes will be developed based on the laboratory test data using the following regression model form:

$$MODRATIO = \alpha_1 + \alpha_2 \log_{10}(AGE) + \alpha_3 \left[\log_{10}(AGE)\right]^2 \qquad (2.2.28)$$

where

MODRATIO = ratio of E_c at a given age to E_c at 28 days.
AGE = specimen age in years.
$\alpha_1, \alpha_2, \alpha_3$ = regression constants.

The end product of the regression analysis is the determination of the regression constants α_1, α_2, and, α_3. Once established, equation 2.2.28 can be used in design to predict the PCC elastic modulus at any given time.

Note that E_c values obtained by test methods other than those recommended should be converted to the equivalent values using appropriate correlations that are based on laboratory testing.

Estimating PCC Elastic Modulus at Input Level 2

For input Level 2, E_c can be estimated from compressive strength (f'_c) testing through the use of standard correlations as explained in the step-by-step approach below.

Step 1 – User Input: Input compressive strength results at 7, 14, 28, and 90 days and the estimated ratio of 20-year to 28-day compressive strength (see table 2.2.24). Testing should be performed in accordance with AASHTO T22, "Compressive Strength of Cylindrical Concrete Specimens."

Table 2.2.24. Required input data at Level 2.

Input Parameter	**Required Test Data**				**Ratio of 20-yr/28-day Strength**	**Recommended Test Procedure**
	7-day	**14-day**	**28-day**	**90-day**		
Compressive Strength	✓	✓	✓	✓	✓	AASHTO T22

A maximum value of 1.35 for the ratio of 20-year to 28-day compressive strength is recommended. In environments of low relative humidity, compressive strength may not increase appreciably after 28 days. In such cases a maximum ratio of 1.20 is recommended. However, if agency specific historical strength data are available, the 20-year to 28-day strength ratios can be determined using that information.

Step 2 – Convert Compressive Strength Data into E_c: Static elastic modulus can be estimated from the compressive strength of the PCC using the American Concrete Institute (ACI) equation (equation 2.2.29). Note that this relationship results in a secant modulus rather than a chord modulus, as determined by ASTM C 469. The difference in the chord and secant elastic moduli is minimal for the levels of strain typically encountered in pavement design, as shown in figure 2.2.5. These relationships are valid only for normal concrete mixes and may be inappropriate for high-performance or other specialty concrete.

$$E_c = 33\rho^{3/2} (f'_c)^{1/2} \qquad (2.2.29)$$

where,

E_c = PCC elastic modulus, psi.
ρ = unit weight of concrete, lb/ft^3.
f'_c = compressive strength of PCC, psi.

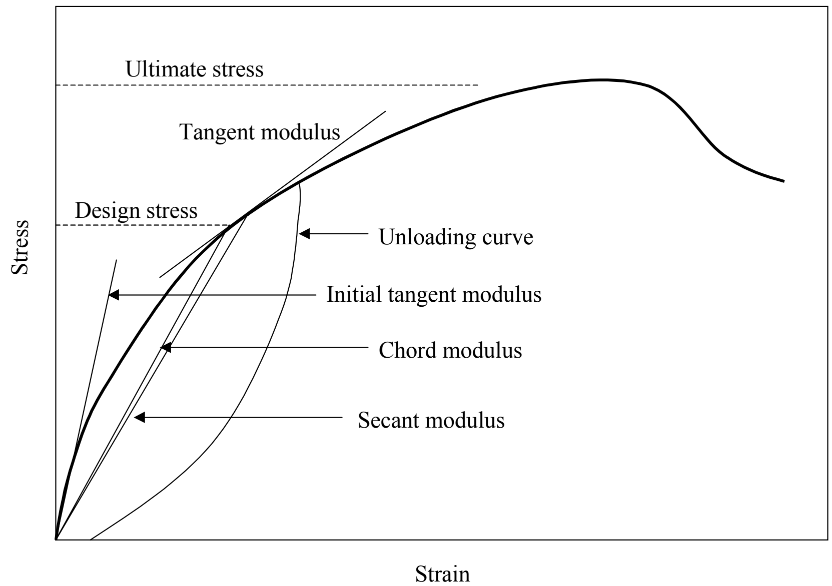


Figure 2.2.5. Typical stress strain diagram for concrete showing the different elastic moduli (*6*).

Step 3 – Develop Modulus Gain Curve: Once the time-series E_c values and the long-term modulus ratio are determined, the procedure for developing the modulus gain curve for design purposes is the same as that outlined for Level 1.

Estimating PCC Elastic Modulus at Input Level 3

At input Level 3, the modulus gain over the design life can be estimated from a single point (28-day) estimate of the concrete strength (either flexural strength, often termed modulus of rupture [MR], or f'_c) using strength gain equations developed from data available in literature. Additionally, if the 28-day E_c is known for the project mixtures, it can also be input to better define the strength-modulus correlation.

The step-by-step approach to estimate the PCC strength and modulus gain over time at level 3 is provided below:

Step 1 – User Input: Input 28-day compressive or flexural strength results (see table 2.2.25). Data could be obtained from historical records. Optionally, the 28-day E_c could also be input, if known, along with either of the strength estimates.

Step 2a – Develop Strength and Modulus Gain Curves Using User Input 28-day MR Values: The following equation can be used to develop strength ratios based on single-point MR estimate. This equation was developed from data available in literature (*5*,*7*).

$$F\_STRRATIO\_3 = 1.0 + 0.12*\log_{10}(AGE/0.0767) - 0.01566*[\log_{10}(AGE/0.0767)]^2 \qquad (2.2.30)$$

where

F_STRRATIO_3	=	ratio of MR at a given age to MR at 28 days at input level 3.
AGE	=	specimen age in years.

Table 2.2.25. Required input data at Level 3.

Input Parameter	28-day Value	Recommended Test Procedure
Flexural Strength	✓	AASHTO T97 or from records
Compressive strength	✓	AASHTO T22 or from records
Elastic modulus	Optional – to be entered with either the flexural or compressive strength inputs	ASTM C 469 or from records

MR for a given age is obtained by multiplying the predicted strength ratio for that age by the 28-day MR (user input).

If the 28-day E_c is not input by the user, E_c at any point in time can then obtained from the MR estimate by first converting the predicted MR to f'_c using the equation 2.2.31 (*8*) and then subsequently converting f'_c to E_c using equation 2.2.29.

$$MR = 9.5*(f'_c)^{0.5} \text{ (MR and } f'_c \text{ in psi)} \qquad (2.2.31)$$

If the 28-day E_c is input by the user, the ratio of the 28-day MR and E_c is computed as a first step. This ratio is then used to estimate E_c at any given time from a known MR value at that point in time estimate from equations 2.2.30.

Step 2b – Develop Modulus Gain Curve Using User Input 28-day Compressive Strength:

In this approach, the user input 28-day f'_c is first converted to a 28-day MR value using equation 2.2.31. Subsequently, the strength and modulus gain over time is estimated as outlined in Step 2a above.

PCC Elastic Modulus Characterization of Existing Intact PCC Pavement Layers

During rehabilitation design with overlays or when existing JPCP pavements are being considered for restoration, the modulus of the existing PCC slabs must be characterized for design purposes. The approach to characterize existing PCC slabs is described below. The primary differences between characterizing new concrete layers and existing layers are that, for the latter:

- For existing PCC slabs to be overlaid by unbonded PCC, the estimated existing intact slab modulus values need to be adjusted for the damage caused to the pavement by the traffic and environmental loads.
- For restoration, the modulus gain over time is not considered because it does not increase significantly in old PCC.

The approaches to estimate the modulus of existing PCC layers at the three hierarchical input levels is presented in table 2.2.26 and explained in detail below.

Table 2.2.26. Recommended condition factor values used to adjust moduli of intact slabs.

Qualitative Description of Pavement Condition[1]	**Recommended Condition Factor, C**
Good	0.42 to 0.75
Moderate	0.22 to 0.42
Severe	0.042 to 0.22

[1] Table 2.5.15 in PART 2, Chapter 5 presents guidelines to assess pavement condition.

Estimating PCC Elastic Modulus at Input Level 1

Step 1: Core the PCC layer at select locations along the project and determine the project mean undamaged PCC elastic modulus from core testing using ASTM C 469. Alternatively, perform nondestructive evaluation at mid-slab location on the existing JPCP or CRCP using the FWD. Estimate the project mean PCC modulus, E_{TEST}, using the Best Fit method (*9*) (which is part of the Design Guide software). Multiply this backcalculated value by 0.8 to arrive at a static elastic modulus value for the uncracked PCC.

Step 2: Determine the overall condition of the existing pavement using the guidelines presented in PART 2, Chapter 5. Choose a pavement condition factor, C_{BD}, based on the existing pavement condition. Possible C_{BD} factor values are presented in table 2.2.26.

Step 3: Adjust the E_{TEST} determined in step 1 using equation 2.2.32 to determine the design modulus input of the existing PCC slab.

$$E_{BASE/DESIGN} = C_{BD} * E_{TEST} \tag{2.2.32}$$

where E_{TEST} is the static elastic modulus obtained from coring and laboratory testing or backcalculation of uncracked intact slab concrete and C_{BD} is a factor based on the overall PCC condition (see table 2.2.26).

Estimating PCC Elastic Modulus at Input Level 2

Step 1: Core the existing PCC layer at select locations along the project and determine the project mean uncracked PCC compressive strength from testing using AASHTO T22.

Step 2: Estimate the project mean uncracked PCC elastic modulus from the compressive strength determined in step 1 using equation 2.2.29.

Step 3: Determine the overall condition of the existing pavement using the guidelines presented in PART 2, Chapter 5. Choose a pavement conditions factor, C_{BD}, based on the existing pavement condition. Possible C_{BD} factor values are presented in table 2.2.26.

Step 4: Adjust the E_{TEST} determined in step 2 using equation 2.2.32 to determine the design modulus input of the existing PCC slab.

Estimating PCC Elastic Modulus at Input Level 3

Step 1: Determine the overall condition of the existing pavement using the guidelines presented in PART 2, Chapter 5.

Step 2: Based on the pavement condition, select typical modulus values from the range of values given in table 2.2.27.

Table 2.2.27. Recommended condition factor values used to adjust moduli of intact slabs.

Qualitative Description of Pavement Condition[1]	Typical Modulus Ranges, psi
Adequate	3 to 4 x 10^6
Marginal	1 to 3 x 10^6
Inadequate	0.3 to 1 x 10^6

[1] Table 2.5.15 in PART 2, Chapter 5 presents guidelines to assess pavement condition.

PCC Elastic Modulus Characterization of Fractured PCC Pavement Layers

The three common methods of fracturing PCC slabs include crack and seat, break and seat, and rubblization. In terms of materials characterization, cracked or broken and seated PCC layers can be considered in a separate category from rubblized layers. Level 1 and 2 input characterization is not applicable to fractured PCC layers. At Level 3, typical modulus values can be adopted for design. These values are presented in table 2.2.28.

Table 2.2.28. Recommended modulus values for fractured PCC layers.

Fractured PCC Layer Type	Typical Modulus Ranges, psi
Crack and Seat or Break and Seat	300,000 to 1,000,000
Rubblized	50,000 to 150,000

2.2.3.2 Poisson's Ratio of PCC Materials

Poisson's ratio is a required input to the structural response computation models, although its effect on computed pavement responses is not great. As a result, this parameter is rarely measured and is often assumed with minimal regard for the specific PCC mix design.

At input Level 1, Poisson's ratio may be determined simultaneously with the determination of the elastic modulus, in accordance with ASTM C 469. Input Level 2, is not applicable since there are no correlations or relationships that may be used to estimate Poisson's ratio from the constituent material characteristics or from other tests. And, finally, at input Level 3, typical values shown in table 2.2.29 can be used. Poisson's ratio for normal concrete typically ranges between 0.11 and 0.21, and values between 0.15 and 0.18 are typically assumed for PCC design.

Table 2.2.29. Typical Poisson’s ratio values for PCC materials.

PCC Materials	Level 3 μ_{range}	Level 3 $\mu_{typical}$
PCC Slabs (newly constructed or existing)	0.15 – 0.25	0.20
Fractured Slab		
Crack/Seat	0.15 – 0.25	0.20
Break/Seat	0.15 – 0.25	0.20
Rubbilized	0.25 – 0.40	0.30

2.2.3.3 Flexural Strength of PCC Materials

The flexural strength (MR) can be defined as the maximum tensile stress at rupture at the bottom of a simply supported concrete beam during a flexural test with third point loading. Like all measures of PCC strength, MR is strongly influenced by mix design parameters. These include:

- Mix constituents
  - Cement type.
  - Cement content.
  - Presence and type of chemical or mineral admixtures.
  - Ratio of water to cementitious material (i.e., cement and pozzolans, such as fly ash and silica fume).
  - Aggregate properties (including aggregate type, maximum particle size, gradation, particle shape and surface texture).
- Curing.
- Age.
- Test condition, method, and equipment.

The MR value has a significant effect on the fatigue cracking potential of the PCC slab for any given magnitude of repeated flexural stress. Therefore, this parameter needs to be characterized with care in rigid pavement design. The characterization of the PCC materials varies as a function of the design type. The discussion on the PCC flexural strength falls under the following two broad groupings:

- Flexural strength characterization for PCC materials used in new or reconstruction JPCP and CRCP projects and PCC overlays.
- Flexural strength characterization of existing PCC pavement layers being considered for rehabilitation with overlays or for restoration (applicable only for JPCP).

PCC Flexural Strength Characterization for New or Reconstruction JPCP and CRCP Projects and PCC Overlays

Table 2.2.30 summarizes the procedures to estimate PCC MR at the various input levels for freshly laid concrete materials. For these design scenarios, the strength-gain over time is considered to allow for more accurate accumulation of incremental damage over time. The designer is required to provide an estimate of strength gain over time, as shown in figure 2.2.6. At the highest level of input (Level 1), the strength-time relationships are established based on the results of laboratory testing conducted on the actual project materials; at Level 3, the relationships for typical materials are used.

Estimating PCC Modulus of Rupture at Input Level 1

At input Level 1, it is recommended that the flexural strength of the proposed PCC mixture be estimated from laboratory testing. PCC MR values for the proposed mixture are required at 7, 14, 28, and 90 days. In addition, the estimated ratio of 20-year to 28-day MR is also a required input (a maximum value of 1.20 is recommended for this parameter). Based on these inputs, a strength gain curve can be developed for the incremental damage analysis. The required input data at Level 1 for this parameter is summarized in table 2.2.31.

The test procedure for obtaining MR for this Guide is AASHTO T97, "Standard Test Method for Flexural Strength of Concrete (Using Third Point Loading)." This test method is used to determine the flexural strength of specimens prepared and cured in accordance with AASHTO T23, T 24, or T 126.

The step-by-step approach to construct the strength gain curve for MR at Level 1 is described below:

Step 1 – User Input: Enter MR results from testing and the estimated ratio of 20 year to 28-day strength.

Step 2 – Develop Strength Gain Curve: The strength gain curve for design purposes will be developed based on the laboratory test data using the following regression model form:

$$F\_STRRATIO = \alpha_1 + \alpha_2 \log_{10}(AGE) + \alpha_3 \left[\log_{10}(AGE)\right]^2 \qquad (2.2.33)$$

where,

F_STRRATIO	=	ratio of MR at a given age to MR at 28 days at input level 1.
AGE	=	specimen age in years.
α_1, α_2, α_3	=	regression constants.

The end product of the regression analysis is the determination of the regression constants α_1, α_2, and, α_3. Once established, equation 2.2.33 can be used in design to predict the PCC flexural strength at any given time.

Table 2.2.30. PCC MR estimation for new or reconstruction design and PCC overlay design.

Material Group Category	Type Design	Input Level	Description
PCC (Slabs)	New	1	• PCC MR will be determined directly by laboratory testing using the AASHTO T97 protocol at various ages (7, 14, 28, 90-days). • Estimate the 20-year to 28-day (long-term) MR ratio. • Develop strength gain curve using the test data and long-term strength ratio to predict MR at any time over the design life.
		2	• PCC MR will be determined indirectly from compressive strength testing at various ages (7, 14, 28, and 90 days). The recommended test to determine f'_c is AASHTO T22. • Estimate the 20-year to 28-day compressive strength ratio. • Develop compressive strength gain curve using the test data and long-term strength ratio to predict f'_c at any time over the design life. • Estimate MR from f'_c at any given time using the following relationship: $MR = 9.5 * (f'_c)^{1/2}$ psi
		3	• PCC flexural strength gain over time will be determined from 28-day estimates of MR or f'_c. • If MR is estimated, use the equation below to determine the strength ratios over the pavement design life. The actual strength values can be determined by multiplying the strength ratio with the 28-day MR estimate. $F\_STRRATIO = 1.0 + 0.12\log_{10}(AGE/0.0767) - 0.01566[\log_{10}(AGE/0.0767)]^2$ • If f'_c is estimated, convert f'_c to MR using equation 2.2.31 and then use the equation above to estimate flexural strength at any given pavement age of interest.

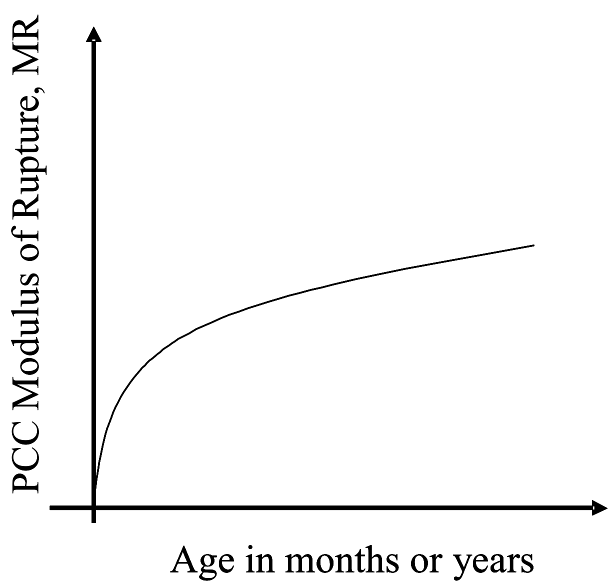


Figure 2.2.6. Modulus data required for M-E design.

Table 2.2.31. Required input data at Level 1 for MR.

Input Parameter	Required Test Data				Ratio of 20-yr/28-day Strength	Recommended Test Procedure
	7-day	14-day	28-day	90-day		
MR	✓	✓	✓	✓	✓	AASHTO T97

Note that MR values obtained by test methods other than those recommended should be converted to the equivalent values using appropriate correlations and models that are based on laboratory testing.

Estimating PCC Modulus of Rupture at Input Level 2

At input Level 2, MR can be estimated from compressive strength (f'_c) testing through the use of standard correlations as explained in the step-by-step approach below.

Step 1 – User Input: Input compressive strength results at 7, 14, 28, and 90 days and the estimated ratio of 20-year to 28-day compressive strength. Testing should be performed in accordance with AASHTO T22 (see table 2.2.32).

Table 2.2.32. Required input data at Level 2 for MR.

Input Parameter	Required Test Data				Ratio of 20-yr/28-day Strength	Recommended Test Procedure
	7-day	14-day	28-day	90-day		
Compressive Strength	✓	✓	✓	✓	✓	AASHTO T22

A maximum value of 1.35 is recommended for the ratio of 20-year to 28-day compressive strength. In environments of low relative humidity, compressive strength may not increase appreciably after 28 days. In such cases a maximum ratio of 1.20 is recommended.

Step 2 – Develop Strength Gain Curves: Once the time-series data for f'_c and the long-term strength ratio are established, the regression curve for developing compressive strength gain over time can be established.

Step 3 – Convert Compressive Strength Data into MR: Flexural strength can be estimated from compressive strength at any point in time using equation 2.2.31.

Estimating PCC Modulus of Rupture at Input Level 3

At input Level 3, the MR gain over time can be estimated using typical strength gain models found in the literature. A single point estimate (28-day) estimate of either MR or f'_c is required. The step-by-step approach to estimate the flexural strength gain over time is provided below:

Step 1 – User Input: Input 28-day flexural or compressive strength results. Data could be obtained from historical records or from testing (optional) (see table 2.2.33).

Table 2.2.33. Required input data at Level 3 for MR.

Input Parameter	28-day Strength	Recommended Test Procedure
Flexural Strength	✓	AASHTO T97 or from records
Compressive strength	✓	AASHTO T22 or from records

Step 2a – Develop Flexural Strength Gain Curve Using User Input 28-day MR Value: Equation 2.2.30 can be used to estimate flexural strength gain as a function of time based on single-point MR estimate.

Step 2b – Develop Flexural Strength Gain Curve Using User Input 28-day Compressive Strength:

- Estimate PCC flexural strength from the user input 28-day compressive strength using equation 2.2.31.
- Use equation 2.2.30 to estimate flexural strength gain as a function of time based on the computed single-point MR estimate.

<u>Existing PCC Flexural Strength Characterization for Restoration and Concrete Overlays Projects</u>

Flexural strength of existing pavements is an input for JPCP restoration design and bonded concrete overlay design. The following are the steps to obtain this input at the three hierarchical input levels.

Estimating MR at Input Level 1 for JPCP Restoration and Bonded Concrete Overlay Design

At input Level 1, an adequate number of prismatic beams can be cut from the existing concrete pavement and the project mean MR can be estimated in accordance with the AASHTO T97 test protocol. The size of the beams saw cut from the existing pavement should comply with the dimensions specified in the test protocol. The measured MR is directly input for level 1.

Estimating MR at Input Level 2 for JPCP Restoration and Bonded Concrete Overlay Design

At input Level 2, an adequate number of cylindrical cores can be removed from the existing concrete pavement and the project mean compressive strength can be estimated in accordance with the AASHTO T22 test protocol. The size of cores removed from the existing pavement should comply with the dimensions specified in the test protocol. The project mean MR can be estimated from the compressive strength using equation 2.2.31. The measured core compressive strength is directly input for level 2.

Estimating MR at Input Level 2 for JPCP Restoration and Bonded Concrete Overlay Design

At input Level 3, an estimate of the 28-day MR or compressive strength is required based on past historical records or local experience. If a compressive strength value is input, MR is computed from it using equation 2.2.31 and the appropriate long term value of MR projected to the age of the existing pavement.

2.2.3.4 Indirect Tensile Strength of PCC Materials

This parameter is a required input in CRCP design. The indirect tensile strength value has a significant effect on the accumulation of damage and the development of punchouts in CRCP. All the factors affecting the measurement of the flexural strength have a great bearing on the indirect tensile strength also. The characterization of this input parameter at the three hierarchical levels for new, reconstruction, and rehabilitation design is discussed below.

Just as with the discussion on flexural strength, the discussion on the PCC indirect tensile strength is presented under the following two broad groupings:

- Indirect tensile strength characterization for PCC materials used in new or reconstruction CRCP projects and CRCP overlays.
- Indirect tensile strength characterization of existing PCC pavement layers being considered for rehabilitation with overlays.

<u>PCC Indirect Tensile Strength Characterization for New or Reconstruction CRCP Projects and CRCP Overlays</u>

Estimating PCC Indirect Tensile Strength at Input Level 1

At input Level 1, it is recommended that the indirect (or split) tensile strength of the proposed PCC mixture (f_t) be estimated from laboratory testing. At Level 1, f_t values for the proposed mixture are required at 7, 14, 28, and 90 days. In addition, the estimated ratio of 20-year to 28-day f_t is also a required input (a maximum value of 1.20 is recommended; however, if agency specific historical strength data are available, the 20-year to 28-day tensile strength ratio can be determined using that information). Based on these inputs, a strength gain curve can be developed for the incremental damage analysis. The required input data at Level 1 for this parameter are summarized in table 2.2.34.

Table 2.2.34. Required input data at Level 1 for f_t.

Input Parameter	Required Test Data				Ratio of 20-yr/28-day Strength	Recommended Test Procedure
	7-day	14-day	28-day	90-day		
f_t	✓	✓	✓	✓	✓	AASHTO T198

The test procedure for obtaining f_t for this Guide is AASHTO T198, "Splitting Tensile Strength of Cylindrical Concrete Specimens."

The step-by-step approach to construct the strength gain curve for f_t at Level 1 is described below:

Step 1 – User Input: Enter f_t results from testing and the estimated ratio of 20 year to 28-day strength.

Step 2 – Develop Strength Gain Curve: The strength gain curve for design purposes will be developed based on the laboratory test data using the regression model form shown in equation 2.2.33.

$$T\_STRRATIO = \alpha_1 + \alpha_2 \log_{10}(AGE) + \alpha_3 [\log_{10}(AGE)]^2 \qquad (2.2.34)$$

where

T_STRRATIO	=	ratio of f_t at a given age to f_t at 28 days at input level 1.
AGE	=	specimen age in years.
$\alpha_1, \alpha_2, \alpha_3$	=	regression constants.

The end product of the regression analysis is the determination of the regression constants α_1, α_2, and, α_3. Once established, equation 2.2.34 can be used in design to predict the PCC tensile strength at any given time.

Note that f_t values obtained by test methods other than those recommended should be converted to the equivalent values using appropriate correlations and models that are based on laboratory testing.

Estimating PCC Indirect Tensile Strength at Input Level 2

At input Level 2, f_t can be estimated from compressive strength (f'_c) testing through the use of standard correlations as explained in the step-by-step approach below.

Step 1 – User Input: Input compressive strength results at 7, 14, 28, and 90 days and the estimated ratio of 20-year to 28-day compressive strength (see table 2.2.35). Testing should be performed in accordance with AASHTO T22.

Table 2.2.35. Required input data at Level 1 for f_t.

Input Parameter	**Required Test Data**				**Ratio of 20-yr/28-day Strength**	**Recommended Test Procedure**
	7-day	**14-day**	**28-day**	**90-day**		
Compressive Strength	✓	✓	✓	✓	✓	AASHTO T22

A maximum value of 1.35 is recommended for the ratio of 20-year to 28-day compressive strength. In environments of low relative humidity, compressive strength may not increase appreciably after 28 days. In such cases a maximum ratio of 1.20 is recommended.

Step 2 – Develop Strength Gain Curves: Once the time-series data for f'_c and the long-term strength ratio are established, the regression curve for developing compressive strength gain over time can be established.

Step 3 – Convert Compressive Strength Data into f_t: Indirect tensile strength can be estimated from compressive strength at any given time by first converting the f'_c values to MR values using equation 2.2.31. The MR values are then multiplied by a factor of 0.67 to obtain the equivalent f_t values (0.6 to 0.7 is the typical range of ratios used in literature to correlate flexural strength to split tensile strength).

Estimating PCC Indirect Tensile Strength at Input Level 3

At input Level 3, the f_t gain over time can be estimated from a single point estimate (28-day) estimate of either MR or f'_c. The step-by-step approach to estimate the indirect tensile strength gain over time is provided below:

Step 1 – User Input: Input 28-day flexural or compressive strength results (see table 2.2.36). Data could be obtained from historical records or from testing (optional).

Table 2.2.36. Required input data at Level 1 for f_t.

Input Parameter	28-day Strength	Recommended Test Procedure
Flexural Strength	✓	AASHTO T97 or from records
Compressive strength	✓	AASHTO T22 or from records

Step 2a – Develop Indirect Tensile Strength Gain Curve Using User Input 28-day MR Value: Equation 2.2.30 can be used to develop flexural strength as a function of time based on single-point MR estimate. The indirect tensile strength can be estimated from the MR strength gain curve by multiplying the MR value at any given time by 0.67 as indicated in equation 2.2.35 below:

$$T\_STRRATIO\_3 = 0.67*(1.0 + 0.12*\log_{10}(AGE/0.0767) - 0.01566*[\log_{10}(AGE/0.0767)]^2) \quad (2.2.35)$$

where

T_STRRATIO_3	=	level 3 ratio of f_t at a given age to f_t at 28 days.
AGE	=	specimen age in years.

Step 2b – Develop Indirect Tensile Strength Gain Curve Using User Input 28-day Compressive Strength:

- Determine PCC flexural strength from the user input 28-day compressive strength using equation 2.2.31.
- Follow the procedure outlined in step 2a to obtain the indirect tensile strength at any given time.

Existing PCC Indirect Tensile Strength Characterization for PCC Overlay Projects

Indirect tensile strength of existing PCC materials sections is an input for bonded PCC overlays of existing CRCP sections. The following are the steps to obtain this input at the three hierarchical input levels.

Estimating Indirect Tensile Strength at Input Level 1 Bonded PCC Overlays of CRCP

At input Level 1, an adequate number of cores can be taken from the existing concrete pavement and the project mean f_t can be estimated in accordance with the AASHTO T198 test protocol. The size of the cores to be removed from the existing pavement should comply with the dimensions specified in the test protocol.

Estimating Indirect Tensile Strength at Input Level 2 Bonded PCC Overlays of CRCP

At input Level 2, an adequate number of cylindrical cores can be removed from the existing concrete pavement and the project mean compressive strength can be estimated in accordance with the AASHTO T22 test protocol. The size of cores removed from the existing pavement should comply with the dimensions specified in the test protocol. As a first step, the project mean MR can be estimated from the compressive strength using equation 2.2.31. Subsequently, the project mean f_t can be estimated by multiplying the estimated MR value by a factor of 0.67.

Estimating Indirect Tensile Strength at Input Level 3 Bonded PCC Overlays of CRCP

At input Level 3, either a 28-day estimate of the MR or compressive strength is required based on past historical records or local experience. These values are then converted to an equivalent f_t as explained previously.

2.2.3.5 Compressive Strength of PCC Materials

The compressive strength of PCC materials is only required to estimate the elastic modulus, flexural strength, and indirect tensile strength at hierarchical input Levels 2 and level 3. The procedures to estimate this quantity were covered in the previous sections of this chapter.

2.2.3.6 Unit Weight of PCC Materials

Table 2.2.37 presents the recommended approach to determine the unit weight of PCC materials used as surface layers in JPCP and CRCP at the various hierarchical levels.

Table 2.2.37. Unit weight estimation of PCC materials.

Material Group Category	Type Design	Input Level	Description
PCC	New	1	• Estimate value from testing performed in accordance with AASHTO T121, "Mass per Cubic Meter (Cubic Foot), Yield, and Air Content (Gravimetric) of Concrete"
		2	• Not applicable.
		3	• User selects design values based upon agency historical data or from typical values shown below: o Typical range for normal weight concrete: 140 to 160 lb/ft^3
PCC	Rehab	1	• Estimate in situ unit weight in accordance with AASHTO T271, "Density of Plastic and Hardened Concrete in-Place by Nuclear Method"
		2	• Not applicable.
		3	• User selects design values based upon agency historical data or from typical values shown below: o Typical range for normal weight concrete: 140 to 160 lb/ft3

2.2.3.7 PCC Coefficient of Thermal Expansion

The coefficient of thermal expansion (α_{PCC}) is defined as the change in unit length per degree of temperature change. When the α_{PCC} is known, the unrestrained change in length produced by a given change in temperature can be calculated as:

$$\Delta L = \alpha_{PCC} \, \Delta T \, L \qquad (2.2.36)$$

where

- ΔL = change in unit length of PCC due to a temperature change of ΔT.
- α_{PCC} = coefficient of linear expansion of PCC, strain per °F.
- ΔT = temperature change (T_2 - T_1), °F.
- L = length of specimen (i.e., joint spacing)

Measurements of the α_{PCC} of a wide range of PCC mixes have shown that it ranges generally between 3 and $8*10^{-6}$/°F. This is a very wide range for an important parameter in M-E design for all types of concrete pavements because it affects both critical slab stresses and also joint and crack openings.

- The magnitude of calculated curling stress (caused by differences in temperature through the slab thickness) is very sensitive to α_{PCC}. Under certain exposure conditions, curling stresses can comprise 50 percent or more of the critical stress experienced by a loaded pavement slab and thus greatly affects slab cracking and CRCP punchouts. Thus, the thermal coefficient plays an important role in optimizing JPCP joint design, CRCP reinforcement, and in accurately calculating pavement stresses and joint and crack LTE over the design life which is critical to faulting.

- It is an important factor in designing joint sealant reservoirs and in selecting sealant materials.
- The α_{PCC} is also critical in affecting crack spacing and more importantly width in CRCP over the entire design life. The crack width directly affects crack LTE which is the key factor in punchout development.

The α_{PCC} is strongly influenced by the aggregate type but is also affected by the hardened paste content and other PCC mix factors. Since the α_{PCC} can be estimated from the weighted average of the PCC mix components, the aggregates have the most pronounced effect because they typically comprise 70 to 80 percent of the PCC volume. However, the coefficient is also a function of the volume of cement paste, moisture content, porosity, and degree of hydration (age) of the paste.

The fact that this characteristic of concrete has not been used in previous pavement design procedures, has a wide range of possible values, and is somewhat unfamiliar to pavement designers, makes it all the more important to obtain proper values for design. Three levels of estimation have been provided for α_{PCC}.

α_{PCC} Estimation at Input Level 1

The Level 1 method for determining α_{PCC} involves direct measurement of the change in length of laboratory specimens subjected to changes in temperatures, using AASHTO TP 60, "Standard Test Method for the Coefficient of Thermal Expansion of Hydraulic Cement Concrete." This is the procedure used to measure all of the α_{PCC} for the LTPP program and all of the sections used for calibration of this Guide.

α_{PCC} Estimation at Input Level 2

The Level 2 method for estimating α_{PCC} uses a linear, weighted average of the constituent coefficient of thermal expansion (i.e., aggregate and paste) values based on the relative volumes of the constituents (see equation 2.2.37). Table 2.2.38 provides typical coefficient of thermal expansion for various common PCC components and mixes compiled from several sources.

$$\alpha_{PCC} = \alpha_{agg} * V_{agg} + \alpha_{paste} * V_{paste} \tag{2.2.37}$$

where,

α_{agg} = Coefficient of thermal expansion of aggregate.
V_{agg} = Volumetric proportion of the aggregate in the PCC mix.
α_{paste} = Coefficient of thermal expansion of cement paste.
V_{paste} = Volumetric proportion of the paste in the PCC mix.

Table 2.2.38. Typical α ranges for common components and concrete (compiled from various sources of information including www.engr.psu.edu/ce/concrete_clinic)

Material Type	Coefficient of Thermal Expansion, 10^{-6}/°F	Concrete Coefficient of Thermal Expansion (made from this material), 10^{-6}/°F
Aggregates		
Marbles	2.2-3.9	2.3
Limestones	2.0-3.6	3.4-5.1
Granites & Gneisses	3.2-5.3	3.8-5.3
Syenites, Diorites, Andesite, Basalt, Gabbros, Diabase	3.0-4.5	4.4-5.3
Dolomites	3.9-5.5	5.1-6.4*
Blast Furnace Slag		5.1-5.9
Sandstones	5.6-6.7	5.6-6.5
Quartz Sands & Gravels	5.5-7.1	6.0-8.7
Quartzite, Cherts	6.1-7.0	6.6-7.1
Cement Paste (saturated)		
w/c = 0.4 to 0.6	10-11	--
Concrete Cores		
Cores from LTPP pavement sections, many of which were used in calibration	N/A	$4.0*10^{-6}$ – $5.5*10^{-6}$ – $7.2*10^{-6}$ (Min – Mean – Max)

α_{PCC} Estimation at Input Level 3

The Level 3 method for estimating the coefficient of linear thermal expansion is based on overall historical averages. The greatest potential for error is associated with this option because PCC materials vary considerably. If nothing is known about the source or type of aggregates used in the PCC, an overall mean of many PCC mixes could be utilized. For example, the mean of the hundreds of LTPP concrete pavement sections α_{PCC} tested with the AASHTO TP 60 and used in the calibration process was $5.5*10^{-6}$/°F. The range was from $4*10^{-6}$/°F to $7.2*10^{-6}$/°F.

It is highly recommended that an agency test their typical PCC mixes containing a range of aggregate types and cement contents to obtain typical values for their materials so that the estimation of this important parameter will be reasonable for local conditions.

2.2.3.8 PCC Shrinkage

Drying shrinkage of hardened concrete is an important factor affecting the performance of PCC pavements. Drying shrinkage affects crack development in CRCP, as well as long-term performance of load transfer across the cracks. For JPCP, the principal effect of drying shrinkage is slab warping caused by differential shrinkage due to the through-thickness variation in moisture conditions leading to increased cracking susceptibility. For JPCP faulting performance, both slab warping and the magnitude of shrinkage strains are important for joint opening.

Note that PCC shrinkage is considered in the Design Guide in two ways: permanent and transitory as explained in PART 2, Chapter 4. The permanent part is considered as part of the Permanent Curl/Warp Effective Temperature Gradient input which affects slab curling/warping and also used to compute joint and crack opening over time. The transitory component is used to compute the monthly changes in responses at the top of the slab for varying relative humidity.

The magnitude of drying shrinkage depends on numerous factors, including water per unit volume, aggregate type and content, cement type, ambient relative humidity and temperature, curing, and PCC thickness.

Drying shrinkage develops over time when PCC is subjected to drying. Upon rewetting, PCC expands to reverse a portion of drying shrinkage, but some of the shrinkage that occurs on first drying is irreversible. The main factor that affects the reversible portion of drying shrinkage is ambient relative humidity. Figure 2.2.7 illustrates the typical behavior of PCC upon drying and rewetting.

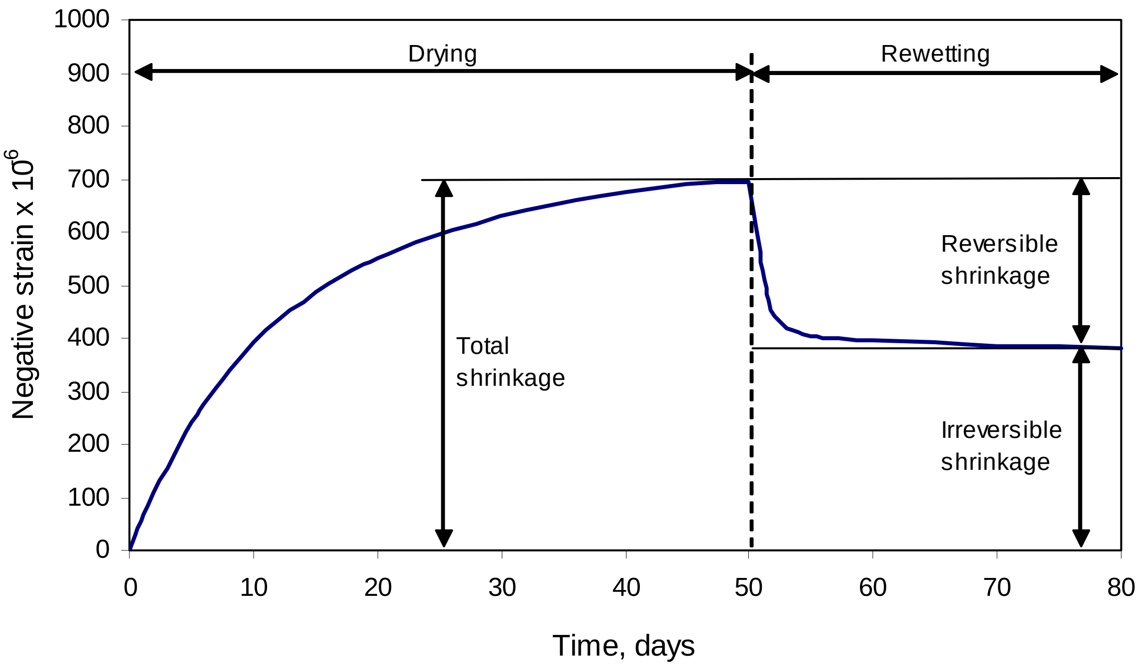


Figure 2.2.7. Typical behavior of PCC upon drying and rewetting (*6*).

Drying shrinkage-related inputs in this Guide include the following:

- Ultimate shrinkage strain, microstrain units.
- Time required to develop 50 percent of the ultimate shrinkage strain, days.
- Anticipated amount of reversible shrinkage, percent.
- Mean monthly ambient relative humidity for the project site (provided by the EICM model from weather station data).

The following discussion highlights how these inputs are estimated at the various hierarchical input levels.

Ultimate Shrinkage Strain

The ultimate shrinkage strain is the shrinkage strain that PCC will develop upon prolonged exposure to drying conditions, which by definition is at 40 percent relative humidity. Since ambient relative humidity is a significant factor affecting PCC drying shrinkage, the moisture condition must be specified for ultimate shrinkage. In this guide, 40 percent relative humidity is specified as the standard condition.

Estimation of Ultimate Shrinkage Strain at Input Level 1

Ideally, at input level 1, the ultimate shrinkage value of a particular concrete mixture should be established in the laboratory. However, this is not a practical approach since it could take several years to realize the ultimate shrinkage strain (i.e., to attain a value that is time stable). Field studies have shown that it could take at least 5 years to reach a stable maximum drying shrinkage value (*10*). Further, there are no established methods currently available to project or extrapolate short-term shrinkage measurements to ultimate shrinkage values. Therefore, laboratory testing currently does not provide a feasible means to establish level 1 ultimate shrinkage strain. The Design Guide software does allow the direct input of ultimate drying shrinkage so when a procedure becomes available the user could input the value.

However, agencies are encouraged to use the AASHTO T160 protocol to measure short-term shrinkage strains at 40 percent relative humidity in the laboratory (up to 180 days or beyond) to develop confidence in the ultimate shrinkage strains estimated using levels 2 and 3 approaches.

Estimation of Ultimate Shrinkage Strain at Input Level 2

At input Level 2, ultimate shrinkage can be estimated from a standard correlation based on PCC mix parameters (cement type, cement content, and water-cement ratio), 28-day PCC compressive strength, and curing conditions. The correlation to estimate ultimate shrinkage is given in equation 2.2.38 (*11,12*).

$$\varepsilon_{su} = C_1 \cdot C_2 \cdot \left\{ 26 w^{2.1} \left(f_c^{'} \right)^{-0.28} + 270 \right\} \qquad (2.2.38)$$

where,

ε_{su} = ultimate shrinkage strain, x 10^{-6}

C_1 = cement type factor:
- 1.0 for type I cement
- 0.85 for type II cement
- 1.1 for type III cement

C_2 = type of curing factor:
- 0.75 if steam cured
- 1.0 if cured in water or 100% relative humidity

1.2 if sealed during curing (curing compound)

w = water content, lb/ft^3 for the PCC mix under consideration.

f'_c = 28-day PCC compressive strength, psi (determined from AASHTO T22).

The Design Guide software calculates the ultimate shrinkage from Equation 2.2.36 when the calculation box is not checked on the PCC materials screen.

Estimation of Ultimate Shrinkage Strain at Input Level 3

At input Level 3, equation 2.2.36 can be utilized with the only difference being that agency typical values can be used for w and f'_c from historical records instead of mixture specific values as required in level 2.

<u>Time Required to Develop 50 Percent of Ultimate Shrinkage</u>

At all input levels, unless more reliable information is available, a value of 35 days, as recommended by the ACI Committee 209 (*13*), is recommended to be used for the time required to develop 50 percent of ultimate shrinkage. This value was used in calibrating the pavement performance models.

Note that if the AASHTO T160 test is used to estimate shrinkage in the laboratory, the time required to develop 50 percent of ultimate shrinkage refers to the number of days to reach half the ultimate shrinkage after the specimen has been removed from a fully soaked condition.

<u>Anticipated Amount of Reversible Shrinkage</u>

At all input levels, unless more reliable information is available, a value to 50 percent is recommended. This value was used in calibrating the pavement performance models.

2.2.3.9 PCC Thermal Conductivity, Heat Capacity, and Surface Absorptivity

The recommended values for PCC thermal conductivity, heat capacity, and surface absorptivity at the various hierarchical input levels are shown in Table 2.2.39. The approach outlined in table 2.2.39 can be used in new, reconstruction, and rehabilitation design.

2.2.4 INPUT CHARACTERIZATION FOR THE CHEMICALLY STABILIZED MATERIALS GROUP

The chemically stabilized materials group consists of lean concrete, cement stabilized, open graded cement stabilized, soil cement, lime-cement-flyash, and lime treated materials. For all of these materials required inputs for this design procedure are:

Table 2.2.39. Estimation of PCC thermal conductivity, heat capacity, and surface absorptivity at various hierarchical input levels.

Input Level	Required Properties	Options for Input Estimation
1	Thermal conductivity	Estimate using laboratory testing in accordance with ASTM E 1952.
	Heat capacity	Estimate using laboratory testing in accordance with ASTM D 2766.
	Surface short wave absorptivity	Laboratory estimation is recommended[1].
2	Thermal conductivity	Same as level 1
	Heat capacity	
	Surface short wave absorptivity	
3	Thermal conductivity	Reasonable values range from 1.0 to 1.5 Btu/(ft)(hr)(^{o}F). A typical value of 1.25 Btu/(ft)(fr)(^{o}F) can be used for design.
	Heat capacity	Reasonable values range from 0.2 to 0.28 Btu/(lb)(^{o}F). A typical value of 0.28 Btu/(lb)(^{o}F) can be used for design.
	Surface short wave absorptivity	However, default property values are available for user convenience: Fresh snow cover 0.05 – 0.25 Old snow cover 0.30 – 0.60 PCC pavement 0.70 – 0.90 A typical value of 0.85 can be used for PCC pavements.

[1] Currently, there are no available AASHTO or ASTM procedures to estimate these quantities for concrete materials. Other protocols may be used as appropriate.

- Strength and modulus properties.
    - Initial 28-day elastic modulus (E) or resilient modulus (M_r) (the type of modulus is a function of material type under consideration).
    - Minimum elastic modulus or resilient modulus after damage from traffic (required for flexible pavement design only).
    - Initial 28-day flexural strength (required for flexible pavement design only).
    - Poisson's ratio.
- Thermal properties.
    - Thermal conductivity.
    - Heat capacity.

This section covers chemically stabilized materials that are engineered to achieve design properties. Lightly stabilized materials for construction expediency are not included. They could be considered as unbound materials for design purposes. Also, note that stabilizers such as lime are frequently used to temporarily modify materials to facilitate construction or to simply lower the plasticity index of a granular material. For these cases, a small amount of lime is used (generally less than 4 percent). These lime-modified materials should be treated as an unbound granular material in design, because there is an insufficient amount of lime added to permanently increase the strength of the material. If there is inadequate data to estimate the modulus of these materials, then the modulus associated with the unbound material should be used for design. Guidance on selecting these properties is provided in the following sections.

Generally speaking, lean concrete and cement treated mixtures are considered to be higher quality materials and are assumed to have higher modulus properties than materials such as soil cement or lime stabilized soil (*14*,*15*). If these relatively weaker chemically stabilized layer are located deeper in the pavement structure (under other structural layers such as a base or subbase course), they can be considered as constant modulus materials that are moisture-insensitive, i.e., treat them like materials with a representative design modulus. In other words, fatigue consumption of these layers is not an issue when they are deeper in the structure and therefore the effort required to characterize these materials for design purposes can be lowered. However, on the other hand, if these layers are higher up in the structure, such as in low-volume roads, the inputs required will be similar to those required for other chemically stabilized layers (e.g., lean concrete or cement-treated bases). In this latter case fatigue fracture in these layers is modeled during the design phase. The amount of stabilizer added to the material for use in structural layers should be determined in accordance with standard stabilized mix design procedures.

The chemically stabilized materials are generally required to have some minimum compressive strength requirement depending on the type of pavement under consideration and the relative importance of the layer in the pavement structure as shown in table 2.2.40. Apart from ensuring that the minimum strength requirements are satisfied, the mix design process for chemically stabilized materials should also ensure that durability requirements (e.g., freeze-thaw durability) are also satisfied. This is absolutely critical for the long-term durability of these materials.

Table 2.2.40. Minimum Compressive Strengths for Cement, Lime, and Combine Lime, Cement, Flyash Stabilized Materials (*16*).

Stabilized Layer	**Minimum Unconfined Compressive Strength, psi[1,2]**	
	Rigid Pavement	**Flexible Pavement**
Base Course	500	750
Subbase, Select Material, or Subgrade	200	250

1. Compressive strength determined at 7-days for cement stabilization and 28-days for lime and lime-cement-flyash stabilization.
2. These values shown in the table should be modified as needed by the local highway agency for specific site conditions.

<u>Elastic Modulus or Resilient Modulus for Design</u>

The required modulus (elastic modulus [E] for lean concrete, cement stabilized, open graded cement stabilized materials, soil cement, lime-cement-flyash, and resilient modulus [M_r] for lime stabilized soils) for design is the 28-day value. The stress state (deviatoric stress and confining pressure) at which the M_r should be estimated can be determined from structural analysis of the trial design (after properly accounting for overburden pressure). While it is noted that these materials could continue to gain strength with time or could possibly degrade over time (the exact changes being a function of the temperature and moisture conditions, freeze-thaw cycles, properties and quantities of the stabilizer, the properties of the material being stabilized, other site conditions, and the efficiency of the material production process), the 28-day values are conservatively used in design.

Leve1 1—Laboratory Testing

At input level 1, E or M_r of the chemically stabilized materials are determined as summarized in table 2.2.41. Note that M_r test should be conducted on the chemically stabilized materials containing the target stabilizer content and molded and conditioned at optimum moisture and maximum density. Curing must also be as specified by the test protocol and must reflect field conditions.

Table 2.2.41. Test methods for determining E or M_r (level 1).

<table>
<tr><th>Chemically Stabilized Material</th><th>Recommended Test Procedure[1]</th></tr>
<tr><td>Lean concrete (E)</td><td rowspan="2">ASTM C 469, “Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression”</td></tr>
<tr><td>Cement treated aggregate (E)</td></tr>
<tr><td>Open graded cement stabilized (E)</td><td rowspan="3">Modulus of elasticity is a required input for these materials. However, modulus testing at level 1 is not possible due to lack of standard test protocols.</td></tr>
<tr><td>Lime-cement-flyash (E)</td></tr>
<tr><td>Soil cement (E)</td></tr>
<tr><td>Lime stabilized soils (M_r)</td><td>Mixture Design and Testing Protocol (MDTP) in conjunction with the AASHTO T307 test protocol[2]</td></tr>
</table>

[1] An equivalent test can be used to estimate the resilient modulus based on local experience.
[2] MDTP is described in the report by Little (*17*).

Note that laboratory testing can be performed on both stabilized mixtures molded in the laboratory or field samples obtained through destructive testing (e.g., coring).

Also, for in-service pavements the modulus at the current damage level can be obtained from non-destructive evaluation of the pavement being rehabilitated using FWD. The layer moduli can be determined from the FWD data using standard backcalculation programs. It is recommended that FWD testing be conducted over the entire project length to more accurately estimate the project mean inputs required for design. Since layer thicknesses are important inputs to the backcalculation process, it is recommended that select coring be performed to verify layer thicknesses. Alternatively, other nondestructive testing techniques such as the GPR can also be used to determine layer thicknesses (see PART 2, Chapter 5). Further, it is recommended that limited testing be performed on cored lime stabilized soil specimens to verify or confirm the backcalculated values.

Note that backcalculation of modulus values for layers less than 6 inches thick located below other paving layers can be problematic. When the lime stabilized layer is less than 6 inches, samples may be need to be recovered for laboratory testing.

Level 2—Correlations with Other Material Properties

At input level 2, E or M_r of the chemically stabilized materials can be estimated using the models presented in table 2.2.42.

Table 2.2.42. Models/Relationships used for determining Level 2 E or M_r.

Chemically Stabilized Material	Recommended Relationships*
Lean concrete[1]	$E = 57000\sqrt{f_c^{'}}$ (*18*) where, E is the modulus of elasticity, psi; f'_c = compressive strength, psi tested in accordance with AASHTO T22
Cement treated aggregate[1]	
Open graded cement stabilized	No correlations are available.
Soil cement[2]	E = 1200 * q_U (*18*) where, E is the modulus of elasticity, psi; q_u = unconfined compressive strength, psi tested in accordance with ASTM D 1633, “Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders”
Lime-cement-flyash[2]	E = 500 + q_U (*19*) where, E is the modulus of elasticity, psi; q_u = unconfined compressive strength, psi tested in accordance with ASTM C 593, “Standard Specification for Fly Ash and Other Pozzolans for Use with Lime”
Lime stabilized soils[2]	$M_r = 0.124 q_U + 9.98$ (*17*) where, M_r = resilient modulus, ksi, q_u = unconfined compressive strength, psi.tested in accordance with ASTM D 5102, “Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures”

[1] Compressive strength f_c can be determined using AASHTO T22.
[2] Unconfined compressive strength q_u can be determined using the MDTP.

At input level 2, the modulus estimates of the lime stabilized layer can be derived by correlating the compressive strength of the retrieved cores to the relationships in table 2.2.42. Alternatively, the DCP can be used to obtain estimates of stiffness. The DCP provides a log of resistance to penetration under an impact load that has been effectively correlated to in situ modulus (see PART 2, Chapter 5).

Level 3—Typical Values

At input level 3, E or M_r is estimated from experience or historical records. A summary of typical M_r value for chemically stabilized materials is presented in table 2.2.43.

Table 2.2.43. Summary of typical resilient modulus values for chemically stabilized materials.

Chemically Stabilized Material	E or M_r Range, psi	E or M_r Typical, psi
Lean concrete	1,500,000 to 2,500,000	2,000,000
Cement stabilized aggregate	700,000 to 1,500,000	1,000,000
Open graded cement stabilized aggregate	—	750,000
Soil cement	50,000 to 1,000,000	500,000
Lime-cement-flyash	500,000 to 2,000,000	1,500,000
Lime stabilized soils*	30,000 to 60,000	45,000

*For reactive soils with 25 percent passing No. 200 sieve and PI of at least 10.

Minimum Elastic Modulus for Design (required for HMA Pavements Only)

The HMA pavements repeated applications of traffic loading can result in the deterioration of the semi-rigid materials chemically stabilized materials. The extent of deterioration or damage is highly correlated to the magnitude of the applied traffic loads and the frequency of loading. 4 2.2.44 presents typical resilient modulus values for deteriorated chemically stabilized materials (after the material has been subjected cycles of traffic loading). The resilient modulus values presented can be modified to reflect local conditions. This is applicable to new design, reconstruction, and rehabilitation design.

Table 2.2.44. Summary of typical resilient modulus values for deteriorated chemically stabilized materials.

Chemically Stabilized Material	Deteriorated M_r Typical, psi
Lean concrete	300,000
Cement stabilized aggregate	100,000
Open graded cement stabilized	50,000
Soil cement	25,000
Lime-cement-flyash	40,000
Lime stabilized soils	15,000

Flexural Strength for Design (required for HMA Pavement Only)

The fatigue life of chemically stabilized materials is linked to the critical flexural stress induced within the stabilized layer. The required MR input for design purposes is the 28-day value. This is applicable to new design, reconstruction, and rehabilitation design.

Leve1 1—Laboratory Testing

At input level 1, flexural strength (denoted as MR) should be estimated from laboratory testing of beam specimens of chemically stabilized materials. Table 2.2.45 presents some recommend tests to determine flexural strengths of various stabilized materials.

Table 2.2.45. Test methods for determining flexural strength MR (level 1).

Chemically Stabilized Material	Recommended Test Procedure[1, 2]
Lean concrete (E)	AASHTO T97
Cement treated aggregate (E)	
Open graded cement stabilized (E)	Not available.
Lime-cement-flyash (E)	AASHTO T97
Soil cement (E)	ASTM D 1635
Lime stabilized soils (M_r)	No current AASHTO or ASTM tests available. Therefore, level 1 testing is not recommended.

[1] An equivalent test can be used to estimate the resilient modulus based on local experience.
[2] MDTP is described in the report by Little (*17*).

Level 2—Correlations with Other Material Properties

At input level 2, MR can be estimated from unconfined compressive strength (q_u) testing of the cured chemically stabilized material samples. Recommended test protocols and the relationship between q_u and MR for chemically stabilized materials are summarized in table 2.2.46.

Table 2.2.46. Relationship between unconfined compressive strength and flexural strength for chemically stabilized materials.

Chemically Stabilized Material	Test Protocol	Typical MR, psi
Lean concrete	AASHTO T22	MR can be conservatively estimated as being 20 percent of the q_u (*15*)
Cement treated aggregate		
Open graded cement stabilized aggregate	Not available	—
Soil cement	ASTM D 1633	MR can be conservatively estimated as being 20 percent of the q_u (*15*)
Lime-cement-flyash	ASTM C 593	
Lime stabilized soils	ASTM D 5102	

Level 3—Typical Values

At input level 3, MR is estimated from experience or historical records based on material description. Typical values are presented in table 2.2.47

Table 2.2.47 Typical flexural strength (MR) values for chemically stabilized materials.

Chemically Stabilized Material	Typical MR, psi
Lean concrete	450
Cement stabilized aggregate	200
Open graded cement stabilized	200
Soil cement	100
Lime-cement-flyash	150
Lime stabilized soils	25

<u>Poisson's Ratio for Design</u>

Another important input required for structural analysis. Although this parameter can be determined from laboratory testing the cost and time required may not be justified. Typical values may be used for new, reconstruction, and rehabilitation design with overlays. Recommended ranges of values are noted in table 2.2.48 (*15*):

Table 2.2.48. Recommended ranges of Poisson's ratios for chemically stabilized materials.

Material	Poisson's Ratio
Cement Stabilized Aggregate (including Lean Concrete)	0.1 to 0.2
Soil cement	0.15 to 0.35
Lime-Fly Ash Materials	0.1 to 0.15
Lime Stabilized Soil	0.15 to 0.2

Thermal Conductivity and Heat Capacity for Design

Thermal conductivity, K, and heat capacity, Q, are the material properties are those that control the heat flow through the pavement system and thereby influence the temperature and moisture regimes within it. Thermal conductivity and heat capacity are key inputs to EICM and are used for estimating temperature and moisture profiles in the pavement structure and subgrade over the design life of a pavement. Table 2.2.49 outlines the recommended approaches to characterizing K and Q at the various hierarchical input levels for chemically stabilized materials. This is applicable to new design, reconstruction, and rehabilitation design.

Table 2.2.49. Recommended approach for thermal conductivity and heat capacity estimation of chemically stabilized materials for EICM calculations.

Material Property	Input Level	Description
Thermal Conductivity, K	1	A direct measurement is recommended at this level (ASTM E 1952).
	2	Not applicable.
	3	User selects design values based upon agency historical data or from typical values shown below: • Typical values for lime stabilized layers range 1.0 to 1.5 Btu/(ft)(hr)(°F). A typical value of 1.25 Btu/(ft)(hr)(°F) can be used for design.
Heat Capacity, Q	1	A direct measurement is recommended at this level (ASTM D 2766).
	2	Not applicable.
	3	User selects design values based upon agency historical data or from typical values shown below: • Reasonable values range from 0.2 to 0.4 Btu/(lb)(°F). A typical value of 0.28 Btu/(lb)(°F) can be used for design.

2.2.5 INPUT CHARACTERIZATION FOR THE UNBOUND GRANULAR MATERIALS AND SUBGRADE MATERIALS GROUP

Unbound granular and subgrade materials are described in this Design Guide using standard AASHTO and unified soil classification (USC) definitions. The AASHTO soil classification system classifies soils based on particle-size distribution and Atterberg limits. This classification system is described in the test standard AASHTO M 145 "The Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes." AASHTO soil classification is based on the portion of unbound granular and subgrade materials that is smaller than 3-in diameter.

The AASHTO classification system identifies two material types:

- Granular materials (i.e., materials having 35 percent or less, by weight, particles smaller than 0.0029 in in diameter).
- Silt-clay materials (i.e., materials having more than 35 percent, by weight, particles smaller than 0.0029 in in diameter).

These two divisions are further subdivided into 7 main group classifications (i.e., A-1 though A-7). The group and subgroup classifications are based on estimated or measured grain-size distribution and on liquid limit and plasticity index values.

The USC system is described in the test standard ASTM D2487, "Standard Method for Classification of Soils for Engineering Purposes." The USC system identifies three major soil divisions:

- Coarse-grained soils (i.e., materials having less than 50 percent, by weight, particles smaller than 0.0029-in in diameter).
- Fine-grained soils (i.e., materials having 50 percent or more, by weight, particles smaller than 0.0029-in in diameter).
- Highly organic soils (materials that demonstrate certain organic characteristics).

These divisions are further subdivided into a total of 15 basic soil groups. The major soil divisions and basic soil groups are determined on the basis of estimated or measured values for grain-size distribution and Atterberg limits. ASTM D 2487 shows the criteria chart used for classifying soil in the USC system and the 15 basic soil groups of the system. For this design procedure unbound granular materials are defined using the AASHTO classification system and are the materials the fall within the specifications for soil groups A-1 to A-3. Subgrade materials are defined using both the AASHTO and USC and cover the entire range of soil classifications available under both systems.

The material parameters required for unbound granular materials, subgrade, and bedrock may be classified in one of three major groups:

- Pavement response model material inputs.
- EICM material inputs.
- Other material properties.

Pavement response model materials input required are resilient modulus, M_r, and Poisson's ratio, μ (elastic modulus for bedrock) parameters used for quantifying the stress dependent stiffness of unbound granular materials, subgrade materials, and bedrock materials under moving loads. Resilient modulus is defined as the ratio of the repeated deviator axial stress to the recoverable axial strain. They are used to characterize layer behavior when subjected to stresses. Unbound materials display stress-dependent properties (i.e., granular materials generally are "stress hardening" and show an increase in modulus with an increase in stress while fine-grained soils generally are "stress softening" and display a modulus decrease with increased stress).

Material parameters associated with EICM are those parameters that are required and used by the EICM models to predict the temperature and moisture conditions within a pavement system. Key inputs include gradation, Atterberg limits, and hydraulic conductivity.

The "other" category of materials properties constitutes those associated with special properties required for the design solution. An example of this category is the coefficient of lateral pressure.

2.2.5.1 Pavement Response Model Unbound Material Inputs

The pavement response model material inputs required are resilient modulus and Poisson's ratio. They are described in the following sections.

Resilient Modulus

Resilient modulus is a required input to the structural response computation models. It has a significant effect on computed pavement responses and the dynamic modulus of subgrade reaction, k-value, computed internally by the Design Guide software. Resilient modulus can be measured directly from the laboratory or obtained through the use of correlations with other material strength properties such as CBR. The different levels of inputs for resilient modulus are presented in the following sections for new, reconstruction, and rehabilitation design.

Level 1—Laboratory Testing

Level 1 resilient modulus values for unbound granular materials, subgrade, and bedrock are determined from cyclic triaxial tests on prepared representative samples. The recommended standard test methods for modulus testing are:

- NCHRP 1-28A, "Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design."
- AASHTO T307, "Determining the Resilient Modulus of Soil and Aggregate Materials."

These test methods describe the laboratory preparation, testing, and computation of test results. For unbound granular materials and subgrade the stress conditions used in the test must represent the range of stress states likely to be developed beneath flexible or rigid pavements subjected to moving wheel loads. Stress states used for modulus testing are based upon the depth at which the material will be located within the pavement system (i.e., the stress states for specimens to be used as base or subbase or subgrade may differ considerably).

For M-E design, resilient modulus is estimated using a generalized constitutive model. The nonlinear elastic coefficients and exponents of the constitutive model is determined by using linear or nonlinear regression analyses to fit the model to laboratory generated M_r test data. The generalized model (NCHRP 1-28A) used in design procedure is as follows:

$$M_r = k_1 p_a \left(\frac{\theta}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3} \tag{2.2.39}$$

where

M_r = resilient modulus, psi
θ = bulk stress = $\sigma_1 + \sigma_2 + \sigma_3$
σ_1 = major principal stress.
σ_2 = intermediate principal stress = σ_3 for M_r test on cylindrical specimen.
σ_3 = minor principal stress/confining pressure
τ_{oct} = octahedral shear stress

$$= \frac{1}{3}\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_1 - \sigma_3)^2 + (\sigma_2 - \sigma_3)^2}$$

P_a = normalizing stress (atmospheric pressure)

k_1, k_2, k_3 = regression constants (obtained by fitting resilient modulus test data to equation)

The constitutive model coefficients determined for each test specimen should be such that the multiple correlation coefficient, r^2, exceeds 0.90. Constitutive model coefficients from similar soils and test specimen conditions can be combined to obtain a "pooled" k_1, k_2, and k_3. If the r^2 for a particular test specimen is less than 0.90, the test results and equipment should be checked for possible errors and/or test specimen disturbance. If no errors or disturbances are found, the use of a different constitutive relationship should be considered.

Coefficient k_1 is proportional to Young's modulus. Thus, the values for k_1 should be positive since M_r can never be negative. Increasing the bulk stress, θ, should produce a stiffening or hardening of the material, which results in a higher M_r. Therefore, the exponent k_2, of the bulk stress term for the above constitutive equation should also be positive. Coefficient k_3 is the exponent of the octahedral shear stress term. The values for k_3 should be negative since increasing the shear stress will produce a softening of the material (i.e., a lower M_r).

Note that the input data required is not the actual M_r test data but rather the coefficients k_1, k_2, and k_3. Coefficient k_1, k_2, and k_3 must therefore be determined outside the Design Guide software.

The level 1 procedure described is applicable to new design, reconstruction, and rehabilitation design. For reconstruction and rehabilitation material samples can be obtained through destructive testing (i.e., coring). Furthermore, for rehabilitation and reconstruction of the existing pavement layer, M_r at level 1 could be obtained by performing nondestructive testing using a falling weight deflectometer (FWD). Descriptions of testing and backcalculation of layer moduli are presented in PART 2, Chapter 5.

Level 2—Correlations with Other Material Properties

General correlations that describe the relationship between soil index and strength properties and resilient modulus can be used in estimating M_r. The relationships could be direct or indirect. For the indirect relationships the material property is first related to CBR and then CBR is related to M_r. Models used in this Design Guide for estimating M_r are presented in table 2.2.50. For level 2 the Design Guide software allows users the following two options:

- Input a representative value of M_r and use EICM to adjust it for the effect of seasonal climate (i.e., the effect of freezing, thawing, and so on).
- Input M_r for each month (season) of the year (total of 12 months).

Table 2.2.50. Models relating material index and strength properties to M_r.

Strength/Index Property	Model	Comments	Test Standard
CBR	$M_r = 2555(CBR)^{0.64}$ (TRL) Mr, psi	CBR = California Bearing Ratio, percent	AASHTO T193, "The California Bearing Ratio"
R-value	$M_r = 1155 + 555R$ (*20*) Mr, psi	R = R-value	AASHTO T190, "Resistance R-Value and Expansion Pressure of Compacted Soils"
AASHTO layer coefficient	$M_r = 30000\left(\frac{a_i}{0.14}\right)$ (*20*) Mr, psi	a_i = AASHTO layer coefficient	AASHTO Guide for the Design of Pavement Structures
PI and gradation*	$CBR = \frac{75}{1 + 0.728(wPI)}$ (see Appendix CC)	wPI = P200*PI P200= percent passing No. 200 sieve size PI = plasticity index, percent	AASHTO T27. "Sieve Analysis of Coarse and Fine Aggregates" AASHTO T90, "Determining the Plastic Limit and Plasticity Index of Soils"
DCP*	$CBR = \frac{292}{DCP^{1.12}}$	CBR = California Bearing Ratio, percent DCP =DCP index, mm/blow	ASTM D 6951, "Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications"

*Estimates of CBR are used to estimate M_r.

Note that primary use of EICM in this design procedure is to estimate the temperature and moisture profiles within the pavement system throughout the pavements design life. The estimated temperature and moisture profiles within the unbound granular and subgrade layers can also be used modify the representative M_r to account for the effects of climate on M_r properties. Users also have the option of taking advantage of EICM or testing representative samples under the climatic conditions anticipated for each of the 12 months of the year and directly inputting these M_r values. The level 2 procedure described is applicable to new design, reconstruction, and rehabilitation design. Note for reconstruction and rehabilitation material samples can be obtained through destructive testing (i.e., coring and bulk samples).

Level 3—Typical Values (Based on Calibration)

For input Level 3, typical the M_r values presented in table 2.2.51 are recommended (see Appendix CC for more detailed discussion). Note that for level 3 only a typical representative M_r value is required at optimum moisture content. EICM is used to modify the representative M_r for the seasonal effect of climate. Users have the option of specifying that the representative M_r value be used without modification for climate by EICM. The Mr values used in calibration were those recommended in table 2.2.51 adjusted for the effect of bedrock and other conditions the influence the pavement foundation strength.

Significant caution is advised on using the M_r values presented in table 2.2.51 as these are very approximate. Levels 1 and 2 testing are strongly preferred, especially FWD and backcalculation.

The reason for caution is that if for example an A-1-a subgrade is truly semi-infinite (i.e., thickness of 20 ft or more) then the use of a 40,000 psi M_r may be justified for infinite depth.

Table 2.2.51. Typical resilient modulus values for unbound granular and subgrade materials (modulus at optimum moisture content) (Appendix CC).

Material Classification	M_r Range	Typical M_r
A-1-a	38,500 – 42,000	40,000
A-1-b	35,500 – 40,000	38,000
A-2-4	28,000 – 37,500	32,000
A-2-5	24,000 – 33,000	28,000
A-2-6	21,500 – 31,000	26,000
A-2-7	21,500 – 28,000	24,000
A-3	24,500 – 35,500	29,000
A-4	21,500 – 29,000	24,000
A-5	17,000 – 25,500	20,000
A-6	13,500 – 24,000	17,000
A-7-5	8,000 – 17,500	12,000
A-7-6	5,000 – 13,500	8,000
CH	5,000 – 13,500	8,000
MH	8,000 – 17,500	11,500
CL	13,500 – 24,000	17,000
ML	17,000 – 25,500	20,000
SW	28,000 – 37,500	32,000
SP	24,000 – 33,000	28,000
SW-SC	21,500 – 31,000	25,500
SW-SM	24,000 – 33,000	28,000
SP-SC	21,500 – 31,000	25,500
SP-SM	24,000 – 33,000	28,000
SC	21,500 – 28,000	24,000
SM	28,000 – 37,500	32,000
GW	39,500 – 42,000	41,000
GP	35,500 – 40,000	38,000
GW-GC	28,000 – 40,000	34,500
GW-GM	35,500 – 40,500	38,500
GP-GC	28,000 – 39,000	34,000
GP-GM	31,000 – 40,000	36,000
GC	24,000 – 37,500	31,000
GM	33,000 – 42,000	38,500

However, if the A-1-a subgrade is only a few feet thick (e.g., < 5ft) and thus acts as an embankment over a weaker subgrade material then the composite M_r of the two materials would be less than 40,000 psi. The reverse is true for a weak A-7-6 soil overlying an A-1-a or bedrock (composite M_r > 8,000 psi). Designers should select the M_r value that represents the entire pavement foundation. This requires an extensive knowledge of the sublayers on which the pavement is to be constructed.

Note that for new, reconstruction, and rehabilitation design material type may be obtained by reviewing historical boring record and material reports or county soil reports. The presence of bedrock is important and should always be investigated.

Calibration results for rigid pavements showed that when granular materials A-1-a through A-2-7 were encountered, M_r values that matched FWD backcalculated results were often 60 to 80 percent of the typical laboratory tested values presented in table 2.2.51. For fine grained soils A-5 through A-7-6, M_r values that matched FWD backcalculated results were often 1.05 to 1.2 times the typical laboratory tested values presented in table 2.2.51. The level 3 procedure described is applicable to new design, reconstruction, and rehabilitation design.

<u>Poisson's Ratio</u>

Poisson's ratio is a required input to the structural response computation models, although its effect on computed pavement responses is not very significant. As a result, this parameter is rarely measured and is often assumed.

Level 1—Laboratory Testing

Direct measurement of Poisson's ratio is normally not justified because it has low sensitivity on structural responses. Poisson's ratio of unbound granular bases may also be determined from cyclic triaxial tests on prepared samples using test data obtained from routine resilient modulus procedures as outlined in the section on resilient modulus testing (level 1). This level 1 procedure is applicable to new design, reconstruction, and rehabilitation design.

Level 2—Correlations with Other Material Properties

There are appropriate models and correlations that can be used to estimate Poisson's ratio. However, they are not recommended in this design procedure. Designers can, however, adopt models and correlations based on local knowledge and experience. This level 2 procedure is applicable to new design, reconstruction, and rehabilitation design.

Level 3—Typical Values

For input Level 3, typical values shown in table 2.2.52 can be used (see Appendix CC). Poisson's ratio for unbound granular materials and subgrades typically ranges between 0.2 and 0.45. This level 3 procedure is applicable to new design, reconstruction, and rehabilitation design.

Table 2.2.52. Typical Poisson's ratio values for unbound granular and subgrade materials.

Material Description	μ_{Range}	$\mu_{Typical}$
Clay (saturated)	0.4—0.5	0.45
Clay (unsaturated)	0.1—0.3	0.2
Sandy clay	0.2—0.3	0.25
Silt	0.3—0.35	0.325
Dense sand	0.2—0.4	0.3
Coarse-grained sand	0.15	0.15
Fine-grained sand	0.25	0.25
Bedrock	0.1—0.4	0.25

2.2.5.2 EICM Inputs Unbound Materials

Material properties required as inputs for EICM include Atterberg limits, gradation, and saturated hydraulic conductivity. They are mostly required for unbound granular base and subgrade materials and are described in the following sections.

<u>Plasticity Index</u>

Plasticity Index, PI, of a soil is the numerical, difference between the liquid limit and the plastic limit of the soil and indicates the magnitude of the range of the moisture contents over which the soil is in a plastic condition.

Plastic limit, PL, is the moisture content, expressed as a percentage of the mass of the oven-dried soil, at the boundary between the plastic and semi-solid states while liquid limit, LL, is defined as the water content of a soil at the arbitrarily determined boundary between the liquid and plastic states, expressed as a percentage of the oven-dried mass of the soil. PI is defined as follows:

$$PI = LL - PL \qquad (2.2.40)$$

The AASHTO test standards used for determining PI, LL, and PL are AASHTO T90, "Determining the Plastic Limit and Plasticity Index of Soils" and AASHTO T89, "Determining the Liquid Limit of Soils." Note that there are no distinct hierarchical levels of input for this parameter as only test values (level 1) are recommended. A full discussion of how EICM uses this input variable is presented in PART 2, Chapter 3. This level 1 procedure is applicable to new design, reconstruction, and rehabilitation design.

<u>Sieve Analysis</u>

The sieve analysis is performed to determine the particle size distribution of unbound granular and subgrade materials. The particle size distribution can be checked against specification requirements to determine compliance, and can also be plotted graphically to determine the nature of the grain size distribution (i.e. dense-graded vs. gap-graded).

The particle size analysis information required are the percentage of materials passing the No. 4 sieve (P_4), the percentage of material passing the No. 200 sieve (P_{200}), and the diameter of the sieve in mm at which 60 percent of the soil material passes, D60. The AASHTO test standard used for particle size analysis is AASHTO T27. Note that there are no levels of input for this parameter as only test values are recommended. A full discussion of how EICM uses this input variable is presented in PART 2, Chapter 3. This level 1 procedure is applicable to new design, reconstruction, and rehabilitation design.

Maximum Dry Unit Weight and Optimum Moisture Content (gravimetric or by weight)

The Design Guide software allows for both a direct input of the maximum dry unit weight (MDD) and the optimum moisture content (OMC) (levels 1 and 3) of unbound granular materials and subgrade. MDD and OMC can also be computed internally by the software (level 2) using the PI and gradation information. For level 1, MDD and OMC are estimated for a representative test sample of the unbound granular material or subgrade through laboratory testing. The relevant AASHTO test standard is AASHTO T99, "Moisture-Density Relations of Soils Using a 5.5-lb. Rammer and a 12-in. Drop." For level 2, MDD and OMC are estimated using correlations or model (see PART 2, Chapter 3 for a description on the models). For level 3, typical MDD and OMC values are assumed based on local experience. MDD typically ranges from 100 to 140 pcf while OMC ranges from 4 to 15 percent. This level 1 procedure is applicable to new design, reconstruction, and rehabilitation design.

Specific Gravity of Solids

The Design Guide software allows for both a direct input of the specific gravity of solids (G_s) (levels 1 and 3) of unbound granular materials and subgrade and for it to be computed internally by the software (level 2) using the PI and gradation information. See PART 2, Chapter 3 for guidance on levels of input for this parameter.

Saturated Hydraulic Conductivity

The Design Guide software allows for both a direct input of the saturated hydraulic conductivity (k) (levels 1 and 3) of unbound granular materials and subgrade and for it to be computed internally by the software (level 2) using the PI and gradation information. See PART 2, Chapter 3 for guidance on levels of input for this parameter.

Degree of Saturation, percent

Degree of saturation, S, is the proportion of the void space in an unbound granular or subgrade material occupied by water. The Design Guide software computes degree of saturation internally using unbound material and subgrade parameters. See PART 2, Chapter 3 for guidance on levels of input for this parameter.

2.2.5.3 Other Unbound Material Properties

Coefficient of Lateral Pressure

The coefficient of lateral pressure, k_o, is the term used to express the ratio of the lateral earth pressure to the vertical earth pressure. For unbound granular, subgrade, and bedrock materials the in-situ typical k_o ranges from 0.4 to 0.6. The coefficient of lateral pressure can be estimated using the following models:

Cohesionless Materials

$$k_o = \frac{\mu}{1-\mu} \tag{2.2.41}$$

Cohesive Materials

$$k_o = 1 - \sin\phi \tag{2.2.42}$$

where

μ = Poisson's ratio
ϕ = the effective angle of internal friction

Typical ranges of μ and ϕ are presented in tables 2.2.52 and 2.2.53, respectively. This procedure is applicable to new design, reconstruction, and rehabilitation design.

Table 2.2.53. Typical effective angle of internal friction for unbound granular, subgrade, and bedrock materials.

Material Description	Angle of Internal Friction, ϕ	Coefficient of Lateral Pressure, k_o
Clean sound bedrock	35	0.495
Clean gravel, gravel-sand mixtures, and coarse sand	29 to 31	0.548 to 0.575
Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.575 to 0.645
Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.645 to 0.717
Fine sandy silt, non plastic silt	17 to 19	0.717 to 0.746
Very stiff and hard residual clay	22 to 26	0.617 to 0.673
Medium stiff and stiff clay and silty clay	19 to 19	0.717

2.2.6 INPUT CHARACTERIZATION FOR BEDROCK MATERIALS

2.2.6.1 Modulus of Elasticity of Bedrock Materials

As mentioned earlier, shallow bedrock layers, if found under an alignment, could have a significant impact on the pavement's mechanistic responses and therefore need to be fully accounted for in design. This is especially true if backcalculation of layer moduli is adopted in rehabilitation design to characterize pavement materials. While the precise measure of the stiffness is seldom, if ever, warranted, any bedrock layer must be incorporated into the analysis.

Table 2.2.54 summarizes the recommended range of modulus values for two forms of bedrock materials for use in new, reconstruction, or rehabilitation design of flexible and rigid pavements. These values apply across all hierarchical input levels.

Table 2.2.54. Bedrock layer elastic modulus estimation for new, reconstruction, and rehabilitation design.

Material Group Category	Type Design	Input Level	Description*
Bedrock	New	1-2	• None of these levels are considered applicable for bedrock conditions
		3	• User selects typical design values: <u>Solid, Massive Bedrock</u> Typical ranges: 750-2000 ksi Default Value: 1000 ksi <u>Highly Fractured, Weathered</u> Typical ranges: 250-1000 ksi Default Value: 500 ksi
	Rehab	1-2	• None of these levels are considered applicable for bedrock conditions
		3	• User selects typical design values: <u>Solid, Massive Bedrock</u> Typical ranges: 750-2000 ksi Default Value: 1000 ksi <u>Highly Fractured, Weathered</u> Typical ranges: 250-1000 ksi Default Value: 500 ksi

2.2.6.2 Poisson's Ratio of Bedrock Materials

Poisson's ratio is a required input to the structural response computation models, although its effect on computed pavement responses is not great. As a result, this parameter is rarely measured and is often assumed, particularly with bedrock materials. Table 2.2.55 presents typical Poisson's ratio values for bedrock materials at hierarchical input Levels 2 and 3. Input Level 1 is not applicable for this material group.

Table 2.2.55. Poisson's Ratio for Bedrock.

Bedrock Materials	Level 2 μ_{range}	Level 3 $\mu_{typical}$
Solid, Massive, Continuous	0.10 – 0.25	0.15
Highly Fractured, Weathered	0.25 – 0.40	0.30

2.2.7 OTHER MATERIALS CONSIDERATIONS

2.2.7.1 Consideration of Erodibility in Design (JPCP and CRCP Only)

Preventing significant erosion of the base and subbase materials is very important for the control of moisture-related distresses such as pumping and faulting in JPCP and punchouts in CRCP. This section provides guidance for assessing the erodibility potential of various materials used in new JPCP and CRCP design and in PCC overlays of existing flexible or rigid pavements. For the purposes of erodibility classification, the base or subbase course is defined as the layer residing directly beneath the PCC slab, while the subbase is described as any manufactured layer between the base and compacted subgrade. Any of these layers can erode under repeated heavy traffic loadings and cause various types of deterioration in rigid pavements.

While several studies have investigated the erosion of the base and/or subbase beneath rigid pavements, no mechanistic procedures have been developed to the level of acceptance at this time that predict how a particular material will respond in the field and what loss of support will occur under specific design, climate, and traffic conditions. Rather than ignore erosion completely, the effect of erosion is considered empirically in this Guide so that designers will be made aware of its importance and adjust for it in design. The design procedure provides the framework for which erosion can be considered on a more mechanistic basis in the future (such as iterative month by month damage accumulation, and inclusion of Level 1 laboratory erosion test).

Traffic Level Effect on Erosion of Base/Subbase Courses

Traffic level is a very critical factor in the consideration of base/subbase course erosion. During the design of the Interstate highway system in the 1960's and 1970's, the design ESALs were generally 5 to 15 million. Today, the design of a reconstruction project on those same highways would be 10 to 20 times greater, which means that the base/subbase course beneath PCC slabs built today will receive 10 to 20 times more load repetitions than in the past over their design life. Thus, added durability must be considered for highways subjected to these heavy traffic.

Prevention of Erosion Beneath a Stabilized Base Course

While the base course is the layer most often affected by erosion, any layer directly beneath a treated base can experience serious erosion. There are many examples of the erosion of fine-grained soils beneath a stabilized base course causing loss of support and joint faulting. Thus, some agencies place a dense graded granular subbase layer between the base and compacted subgrade to reduce this problem. Other agencies stabilize the top layer of a fine-grained soil with lime; however, this approach must produce a sufficiently hard material with adequate compressive strength and uniformity along the project. Geotextiles are also used as separation layers to hold the subgrade materials in place. Another alternative that has been used successfully is to place a layer of recycled crushed PCC beneath the dense treated base course.

Base/Subbase Erodibility Class Assessment for Design

This section provides guidelines for assessing the erodibility potential for base and subbase materials at the various hierarchical input levels.

Level 1 Material Classification

Level 1 classification is based on the material type and test results from an appropriate laboratory test that realistically simulates erosion action beneath a PCC slab. Although tests exist that approximately simulate erosion, these have not been developed fully to the stage of nationwide usage. Thus, Level 1 cannot be implemented at this time. Future research work is needed to further develop erosion test standards. The tests currently being used to assess the erodibility of paving materials include:

- Rotational shear device for cohesive or stabilized materials (*21*).
- Jetting test (*21*).
- Linear and rotational brush tests (*22*).
- South African erosion test (*23*).

Level 2 Material Classification

At input Level 2, base/subbase erodibility class is assigned to various materials based on both material classifications, compositions, and related tests. The descriptions for Level 2 erodibility classes are presented in table 2.2.56. Five classes of erodibility resistance are shown in the table. These recommendations were based on erodibility classifications developed by the Permanent International Association of Road Congresses (PIARC) (*23*,*24*). The PIARC recommendations were modified to include permeable base/subbase materials and correlative testing (e.g., compressive strength, stripping potential, etc.) was added. In table 2.2.56, the erodibility resistance ratio is in the order of about five between each class (i.e., class 1 materials are five times more erosion resistant than class 2 and so on). Note that at the present time, there is no correlation of level 2 erodibility classification with the Level 1 testing.

Level 3 Material Classification

Level 3 guidelines are based solely on material type description. This method of estimation of erosion class would be the least accurate methodology. It is sometimes necessary, however, as no additional information will be known about a material. Table 2.2.59 shows these descriptions, which can be modified by design agencies based upon their experience and performance of base materials.

Table 2.2.56. Level 2 recommendations for assessing erosion potential of base material (adapted after *24,25*).

Erodibility Class	Material Description and Testing
1	(a) Lean concrete with approximately 8 percent cement; or with long-term compressive strength > 2,500 psi (>2,000 psi at 28-days) and a granular subbase layer or a stabilized soil layer, or a geotextile fabric is placed between the treated base and subgrade, otherwise class 2. (b) Hot mixed asphalt concrete with 6 percent asphalt cement that passes appropriate stripping tests (figure 2.2.8) and aggregate tests and a granular subbase layer or a stabilized soil layer (otherwise class 2). (c) Permeable drainage layer (asphalt treated aggregate (see figure 2.2.8 and table 2.2.57 for guidance) or cement treated aggregate (see table 2.2.58 for guidance) and with an appropriate granular or geotextile separation layer placed between the treated permeable base and subgrade.
2	(a) Cement treated granular material with 5 percent cement manufactured in plant, or long-term compressive strength 2,000 to 2,500 psi (1,500 to 2,000 psi at 28-days) and a granular subbase layer or a stabilized soil layer, or a geotextile fabric is placed between the treated base and subgrade; otherwise class 3. (b) Asphalt treated granular material with 4 percent asphalt cement that passes appropriate stripping test and a granular subbase layer or a treated soil layer or a geotextile fabric is placed between the treated base and subgrade; otherwise class 3.
3	(a) Cement-treated granular material with 3.5 percent cement manufactured in plant, or with long-term compressive strength 1,000 to 2,000 psi (750 psi to 1,500 at 28-days). (b) Asphalt treated granular material with 3 percent asphalt cement that passes appropriate stripping test.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated soils (PCC slab placed on prepared/compacted subgrade)

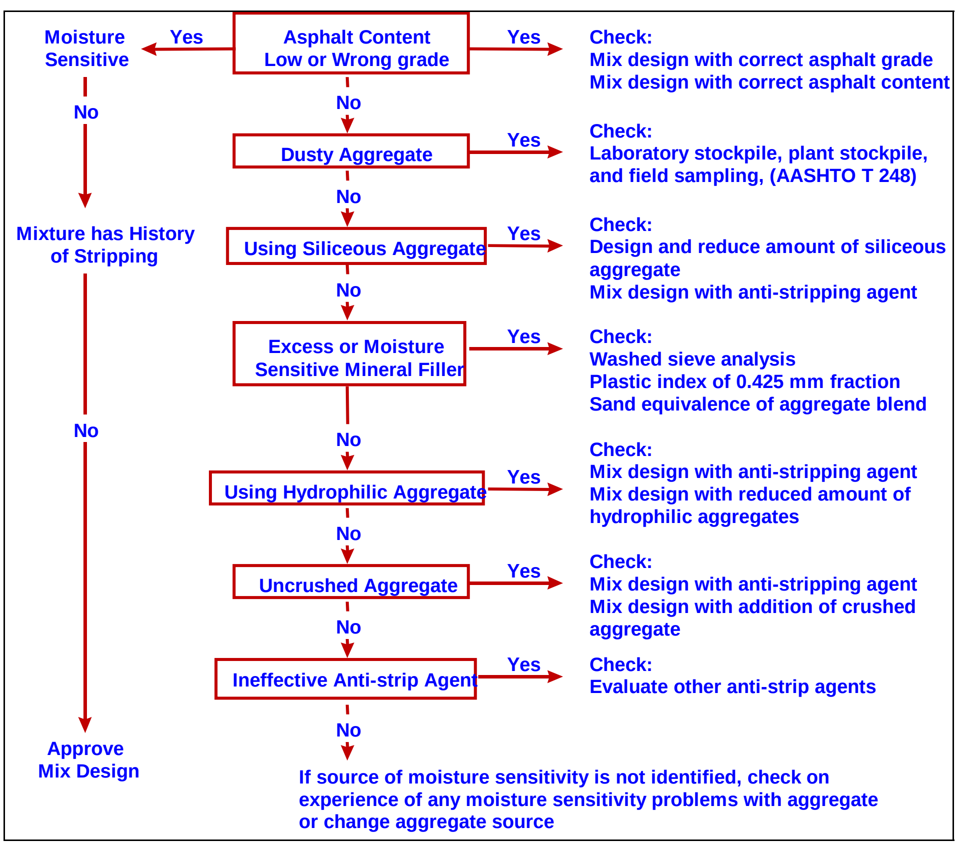


Figure 2.2.8. Guide to evaluating HMA mixture stripping potential.

Table 2.2.57. Recommended asphalt stabilizer properties for asphalt-treated permeable base/subbase materials.

Specification	Requirement	Test Method
Asphalt binder content	Asphalt binder content must ensure that aggregates are well coated. Minimum recommended binder content is between 2.5 to 3 percent by weight. Final binder content should be determined according to mix gradation and film thickness around the coarse aggregates.	AASHT0 T 195-67(1998), *Determining Degree of Particle Coating of Bituminous-Aggregate Mixtures*
Asphalt binder grade	A stiff asphalt grade (typically 1 grade stiffer than the surface course is recommended)	Penetration, viscosity, or Superpave binder testing can be performance to determine the binder grade.
Anti-stripping	Anti-stripping test should be performed on all asphalt treated materials.	AASHTO T283, "Resistance of Compacted Bituminous Mixture to Moisture Induced Damage" AASHTO T165, "Effect of Water on Cohesion of Compacted Bituminous Mixtures"
Anti-stripping agents	Aggregates exhibiting hydrophilic characteristics can be counteracted with 0.5 to 1 percent lime.	NCHRP Report 274 (*26*).
Permeability	Minimum mix permeability: 1000 ft/day.	AASHTO T3637, "Permeability of Bituminous Mixtures"

Table 2.2.58. Recommended Portland cement stabilizer properties for cement treated permeable base/subbase materials.

Specification	Requirement
Cement content*	Portland cement content selected must ensure that aggregates are well coated. An application rate of 220 to 285 lb/yd^3 is recommended.
w/c ratio	Recommended w/c ratio to ensure strength and workability: 0.3 to 0.5.
Workability	Mix slump should range between 1 to 2 in.
Cleanness	Use only clean aggregates
Permeability	Minimum mix permeability: 1,000 ft/day.

*Cement must conform to the specification of AASHTO M 85, "Portland Cement."

Table 2.2.59. Level 3 recommendations for assessing erosion potential of base material based on material description only.

Erodibility Class	Material Description and Testing
1	(1) Lean concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 2. (2) Hot mixed asphalt concrete with previous outstanding past performance and a granular subbase layer or a stabilized soil layer is placed between the treated base and subgrade, otherwise class 2. (3) Permeable drainage layer (asphalt or cement treated aggregate) and a granular or a geotextile separation layer between the treated permeable base and subgrade. Unbonded PCC Overlays: HMA separation layer (either dense or permeable graded) is specified.
2	(1)Cement treated granular material with good past performance and a granular subbase layer or a stabilized soil or a geotextile fabric layer is placed between the treated base and subgrade, otherwise class 3. (2) Asphalt treated granular material with good past performance and a granular subbase layer or a stabilized soil layer or a geotextile soil layer is placed between the treated base and subgrade, otherwise class 3.
3	(1) Cement-treated granular material that has exhibited some erosion and pumping in the past. (2) Asphalt treated granular material that has exhibited some erosion and pumping in the past. Unbonded PCC Overlays: Surface treatment or sand asphalt is used.
4	Unbound crushed granular material having dense gradation and high quality aggregates.
5	Untreated subgrade soils (compacted).

Matching Base Type to Design and Traffic

This section provides some general recommendations on relating the base erosion class type to pavement design and traffic level. The key design variable that affects erosion is joint load transfer. A design can include no dowel bars, or bars of adequate diameter and spacing. This decision has a very profound effect on erosion beneath the joint and the faulting and loss of support that develops.

Performance data from many JPCP sections across North America show that when no dowel bars are used, the base type and erosion resistance has a huge effect on joint faulting. When adequate dowel bars are used, joint faulting is very much reduced and the base has a reduced effect on joint faulting. For example, a Class 3 base may work well with joints that are doweled, but it may erode significantly when no dowels are used due to the much greater deflections under axle loads.

The level of heavy truck loadings is obviously another very important factor. One base material may work very well at a lower traffic level, while it may erode badly at a higher level of load applications. This effect is interactive with the use of dowel bars.

Table 2.2.60 has been prepared to provide approximate guidance on the selection of the appropriate base for a given JPCP or CRCP project. Table 2.2.57 shows that as truck-traffic

Table 2.2.60. Recommendations for base type to prevent significant erosion.

Design Lane Initial ADTT*	JPCP		CRCP
	Nondoweled	Doweled	
>2,500	n/a—nondoweled design not recommended	Class 1	Class 1**
1,500 – 2,500	n/a—nondoweled design not recommended	Class 1	Class 1**
800 – 1,500	n/a—nondoweled design not recommended	Class 2, 3, or 4	Class 1**
200 – 800	Class B or C	Class 3 or 4	Class 2
< 200	Class, D or E	Class 4 or 5	Class 3

*Lane ADTT: Initial Year Design Average Daily Truck Traffic in design lane, one-direction.
**Permeable base course not recommended for CRCP.

volume gets higher, a more adequate joint load transfer design and erosion resistant base class are needed. For example, a project is being designed that has an initial ADTT in the design lane of 1,600 trucks and the JPCP will have doweled joints. An acceptable base type for this level of traffic is class 1. This should be the base type utilized on the first trial design.

Estimating the Erosion Potential of Base Materials in JPCP Design

Tables 2.2.56 or 2.2.59 can be used to estimate the erosion potential of the base for JPCP design. The base erosion class selected will have a significant effect on the joint faulting and cracking as it directly enters the performance prediction equations for joint faulting and slab cracking.

Estimating the Erosion Potential of Base Materials Beneath CRCP

Tables 2.2.56 or 2.2.59 can also be used to estimate the erosion potential of the base for CRCP design. This factor has a major effect on the loss of support along the edge of the slab. Loss of support along the edge is one of the primary factors causing punchouts in CRCP. No model, procedure, or even field data were available for developing relationships between the base erosion class, type of subbase or subgrade beneath the base, precipitation, and area of eroded base/subbase material. Expected erosion width values developed from expert opinion were used to develop an empirical model relating erosion width (e) to materials and climate factors (equation 2.2.43). This model was incorporated into the punchout prediction model. Erosion width, e, is illustrated in figure 2.2.9.

$$e = -7.4 + 0.342P_{200} + 1.557BEROD + 0.234PRECIP \qquad (2.2.43)$$

where

- e = maximum width of eroded base/subbase measured inward from the slab edge, in (if $e < 0$, set $e = 0$).
- P_{200} = percent subgrade soil (layer beneath treated base course) passing the No. 200 sieve.
- BEROD = base material erosion class (1, 2, 3, or 4).
- PRECIP = mean annual precipitation, in.

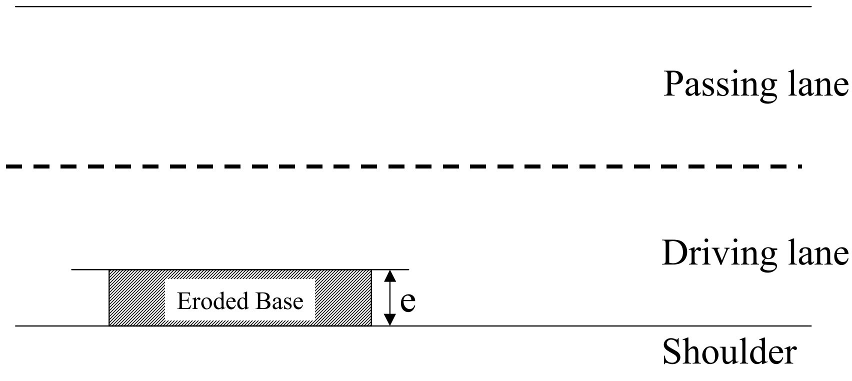


Figure 2.2.9. CRCP with eroded base; definition of "e".

Equation 2.2.42 is used to estimate e for all levels of base erodibility class input as follows:

1. Determine maximum value of e using equation 2.2.37 over the design life.
2. The initial value of e at age = 0 years is 0.
3. Assume e increases linearly from its initial value of 0 to e_{max} in the first 20 years of a pavement life.
4. There is no change in e (i.e., e = e_{max}) after 20 years and therefore the loss of support along the pavement edge remains constant at e_{max}.

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