CHAPTER 6
PRINCIPLES OF DESIGN FOR FLEXIBLE PAVEMENT

6.1 Introduction

Design of flexible pavement structures involves the consideration of numerous factors, the most important are truck volume, weight and distribution of axle loads, HMA, underlying material properties, and the supporting capacity of the subgrade soils. **Typical reconstruction projects should have a design life of 20 years for reconstructions and 10 years for rehabilitations** unless mitigating circumstances exist.

Methods are presented in this section for the design of the flexible pavement structure with respect to thickness of the subbase, base, surface courses, and the quality and strength of the materials in place. Interaction between pavement materials and climate is evaluated as part of the M-E Design process.

6.2 M-E Design Methodology for Flexible Pavement

M-E Design uses an iterative process. The key steps in the design process include the following:

1. **Select a Trial Design Strategy**

2. **Select Appropriate Performance Indicator Criteria for the Project:** Establish criteria for acceptable pavement performance (i.e. distress/IRI) at the end of the design period. Performance criteria were established to reflect different magnitudes of key pavement distresses which trigger major rehabilitation or reconstruction. CDOT criteria for acceptable performance is based on highway functional class and location.

3. **Select Appropriate Reliability Level for the Project:** The reliability is in essence a factor of safety that accounts for inherent variations in construction, materials, traffic, climate, and other design inputs. The level of reliability selected should be based on the criticality of the design and selected for each individual performance indicator. CDOT criteria for a desired reliability is based on highway functional class and location.

4. **Assemble All Inputs for the Pavement Trial Design Under Consideration:** Define subgrade support, asphalt concrete and other paving material properties, traffic loads, climate, pavement type and design, and construction features. The inputs required to run the M-E Design program may be obtained using one of three hierarchical levels and need not be consistent for all inputs in a given design. The hierarchical level for a given input is selected based on the importance of the project, input, and resources at the disposal of the user.

5. **Run the M-E Design Software:** The software calculates changes in layer properties, damage, key distresses, and IRI over the design life. The key steps include:

a) Processing input to obtain monthly values of traffic, seasonal variations of material, and climatic inputs needed in design evaluations for the entire design period.
b) Computing structural responses (stresses and strains) using multilayer elastic theory or finite element based pavement response models for each axle type and load and each damage-calculation increment throughout the design period.
c) Calculating accumulated distress at the end of each analysis period for the entire design period.
d) Predicting key distresses (rutting, bottom-up/top-down fatigue cracking, and thermal cracking) at the end of each analysis period throughout the design life using calibrated mechanistic-empirical performance models.
e) Predicting IRI as a function of initial IRI, distresses accumulating over time, and site factors at the end of each analysis increment.

6. **Evaluate Adequacy of the Trial Design:** The trial design is considered "adequate" if none of the predicted distresses/IRI exceed the performance indicator criteria at the design reliability level chosen for the project. If any criteria has been exceeded, one must determine how the deficiency can be remedied by altering material types, properties, layer thicknesses, or other design features.

7. **Revise the Trial Design, as Needed:** If the trial design is deemed "inadequate", one must revise the inputs and re-run the program until all performance criteria have been met. Once met, the trial design becomes a feasible design alternative.

Design alternatives that satisfy all performance criteria are considered feasible from a structural and functional viewpoint and may be considered for other evaluations, such as life cycle cost analysis. Consultation of the mix design(s) with the RME shall occur. A detailed description of the design process is presented in the interim edition of the *AASHTO Mechanistic-Empirical Pavement Design Guide Manual of Practice,* AASHTO 2008.

6.3 Select a Trial Design Strategy

6.3.1 Flexible Pavement Design Types

Figure 6.1 Asphalt Concrete Pavement Layer Systems illustrates well known CDOT combinations of asphalt concrete structural pavement layers. Designers can select from among several flexible pavement options as shown below:

- **Conventional Flexible Pavements:** Flexible pavements consisting of a relatively thin asphalt concrete layer placed over an unbound aggregate base layer and subgrade.

- **Deep-Strength AC Pavements:** Flexible pavements consisting of a relatively thick asphalt concrete layer placed over an unbound aggregate base layer and subgrade.

- **Full-Depth AC Pavements:** Asphalt concrete layers placed directly over the subgrade.

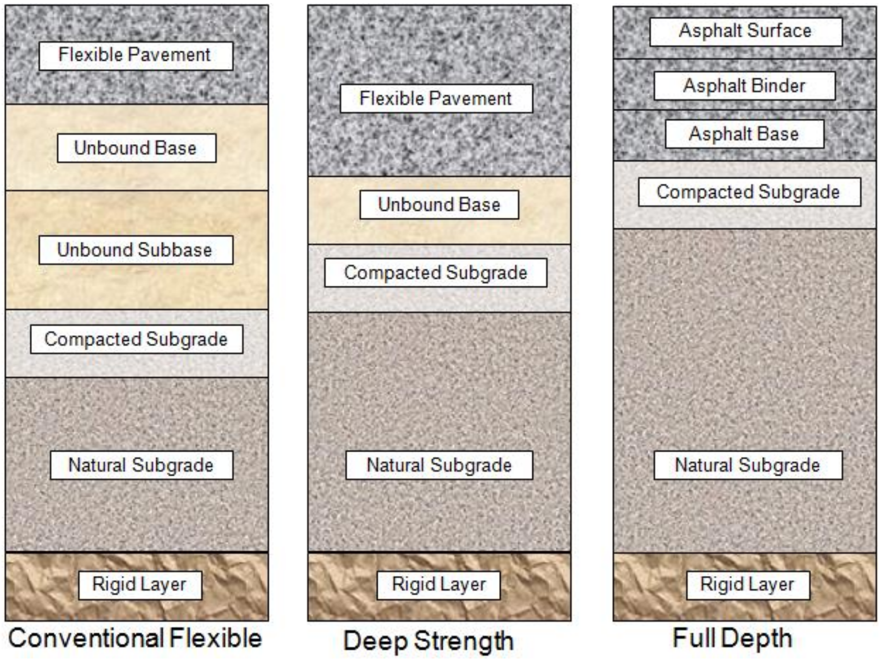


Figure 6.1 Asphalt Concrete Pavement Layer Systems

The asphalt concrete layer in **Figure 6.1 Asphalt Concrete Pavement Layer Systems** may be comprised of several layers of asphalt concrete courses to include a surface course, intermediate or binder course, and a base course (see **Figure 6.2 Structural Layers**). The surface, binder, and base courses are typically different in composition and are placed in separate construction operations (3).

- **Surface Course:** The surface course normally contains the highest quality materials. It provides characteristics such as friction, smoothness, noise control, rut and shoving resistance, and drainage. It also serves to prevent the entrance of excessive quantities of surface water into the underlying HMA courses, bases, and subgrade.

- **Intermediate/Binder Course**: The intermediate course, sometimes called binder course, consists of one or more lifts of structural HMA placed below the surface course. Its purpose is to distribute traffic loads so stresses transmitted to the pavement foundation will not result in permanent deformation to the course. It also facilitates the construction of the surface course.

- **Base Course:** The base course consists of one or more HMA lifts located at the bottom of the structural HMA course. Its major function is to provide the principal support of the pavement structure. The base course should contain durable aggregates that will not be damaged by moisture or frost action.

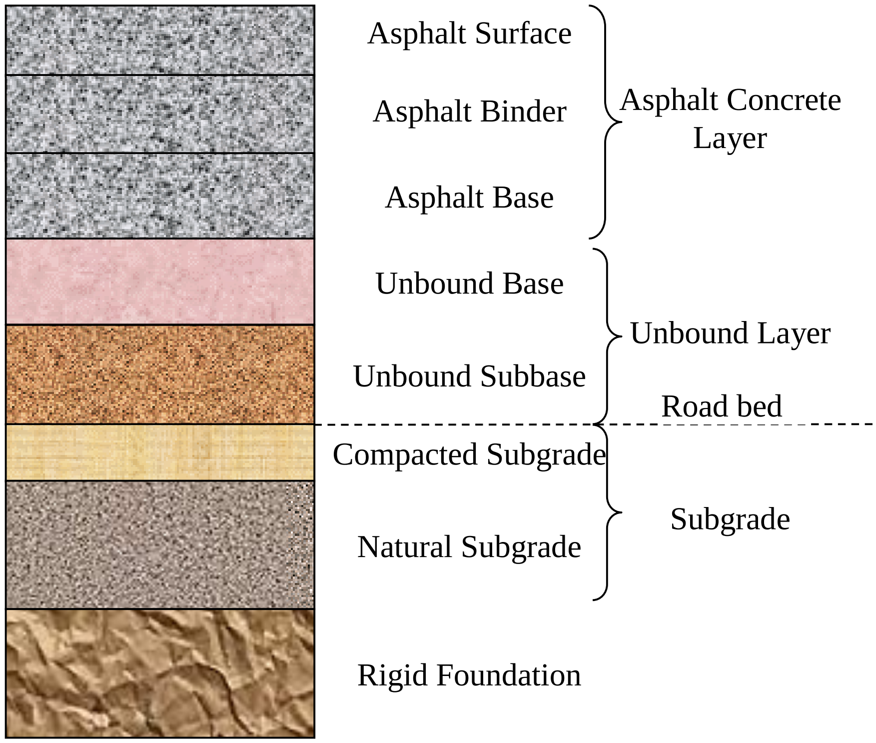


Figure 6.2 Structural Layers

6.3.2 Concept of Perpetual Pavements

A perpetual pavement is defined as an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement (6). Full depth and deep-strength asphalt pavement structures have been constructed since the 1960s. Full-depth pavements are constructed directly on subgrade soils and deep-strength sections are placed on relatively thin (4 to 6 inches) granular base courses. A 20-year traffic design period is to be used for the traffic loading. One of the chief advantages of these pavements is that the overall section of the pavement is thinner than those employing thick granular base courses. Such pavements have the added advantage of significantly reducing the potential for fatigue cracking by minimizing the tensile strains at the bottom of the asphalt layer (7) (see **Figure 6.1 Asphalt Concrete Pavement Layer Systems)**. An asphalt perpetual pavement structure is designed with a durable, rut and wear resistant top layer with a rut resistant intermediate layer and a fatigue resistant base layer (see **Figure 6.3 Perpetual Pavement Design Concept**)

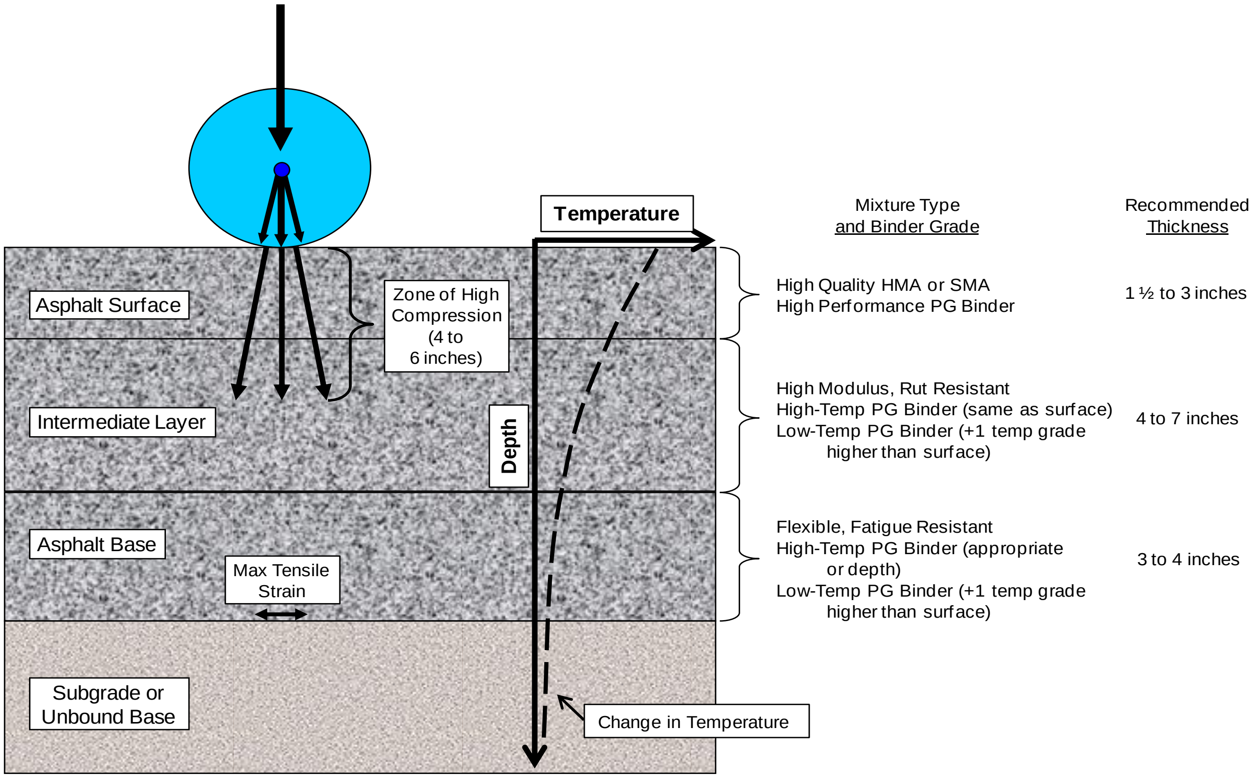


Figure 6.3 Perpetual Pavement Design Concept

This concept may be used in conventional, deep strength, or full depth asphalt structural layering. In mechanistic design, the principles of physics are used to determine a pavement's reaction to loading. Knowing the critical points in the pavement structure, one can design against certain types of failure or distress by choosing the right materials and layer thicknesses (7). Therefore, the uppermost structural layer resists rutting, weathering, thermal cracking, and wear. SMAs or dense-graded SuperPave mixtures provide these qualities. The intermediate layer provides rutting resistance through stone-on-stone contact and durability is imparted by the proper selection of materials. Resistance to bottom-up fatigue cracking is provided by the lowest asphalt layer having a higher binder content or by the total thickness of pavement reducing the tensile strains in this layer to an insignificant level (6).

6.3.3 Establish Trial Design Structure

The designer must establish a trial design structure (combination of material types and thicknesses). This is done by first selecting the pavement type of interest (see **Figure 6.5 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)**). M-E Design automatically provides the top layers of the selected pavement type. The designer may add or remove pavement structural layers and/or modify the layer material type and thickness as appropriate. **Figure 6.4 M-E Design Software Screenshot of Flexible Pavement Trial Design Structure** shows an example of flexible pavement trial design with pavement layer configuration on the left and layer properties of the AC surface course on the right.

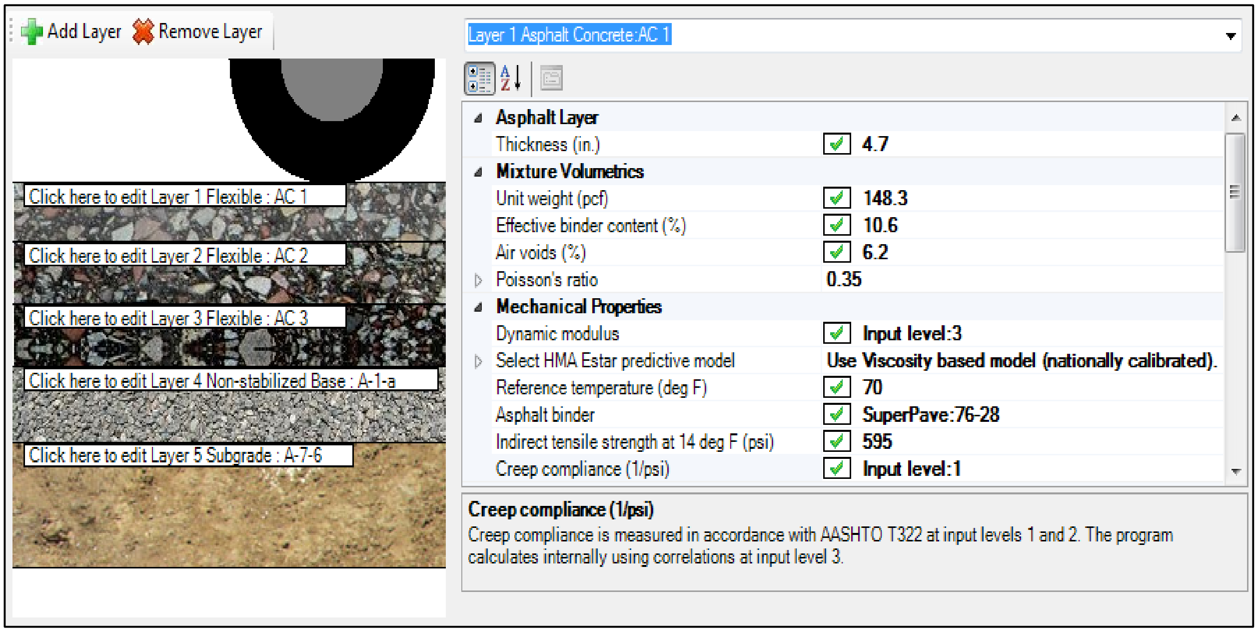


Figure 6.4 M-E Design Software Screenshot of Flexible Pavement Trial Design Structure

6.4 Select the Appropriate Performance Indicator Criteria for the Project

Table 2.4 Recommended Threshold Values of Performance Criteria for New Construction or Reconstruction Projects presents recommended performance criteria for flexible pavement design. The designer should enter the appropriate performance criteria based on functional class. An appropriate initial smoothness (IRI) is also required, **For new flexible pavements, the recommended initial IRI is 50 inches/mile**.

Figure 6.5 M-E Design Software Screenshot Showing General Information (left) Performance Criteria and Reliability (right) shows performance criteria for a sample flexible pavement trial design. The coefficients of performance prediction models considered in the design of a new flexible pavement are shown in **Figure 6.6 Performance Prediction Model Coefficients for Flexible Pavement Designs (Marshall Mix)** through **Figure 6.8 Performance Prediction Model Coefficients for Flexible Pavement Designs (PMA Mix)**. The value of AC rutting coefficient (BR1) is based on the type of HMA

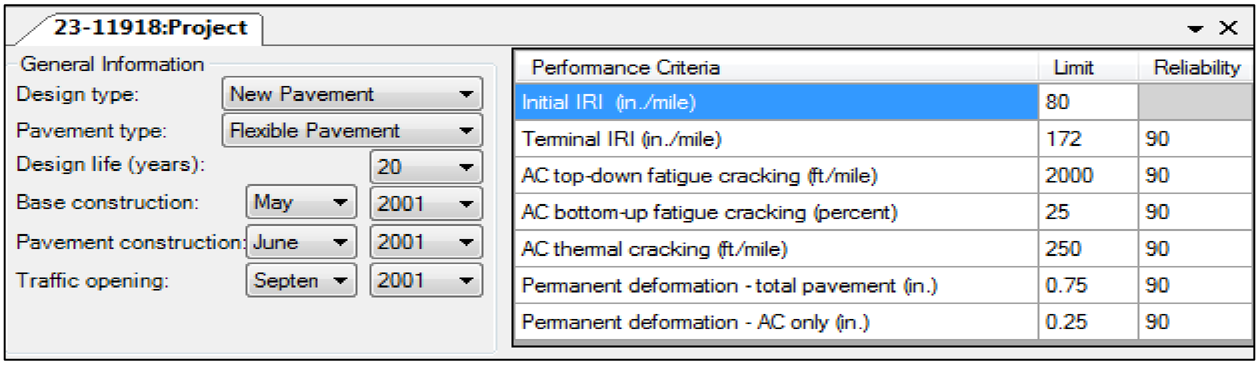


Figure 6.5 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)

New Flexible Pavement-Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.021
AC Cracking C2 Bottom	2.35
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	130.3674
AC Fatigue BF2	1
AC Fatigue BF3	1.217799
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.3791
AC Rutting BR1	7.6742
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.1414*Pow(RUT,0.25)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	50
IRI Flexible C2	0.55
IRI Flexible C3	0.0111
IRI Flexible C4	0.02
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.22
Granular Subgrade Rutting Standard Deviation	0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.37
Fine Subgrade Rutting Standard Deviation	0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	6.3
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	6.3
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 6.6 Performance Prediction Model Coefficients for Flexible Pavement Designs (Marshall Mix)

New Flexible Pavement-Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.021
AC Cracking C2 Bottom	2.35
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	130.3674
AC Fatigue BF2	1
AC Fatigue BF3	1.217799
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.3791
AC Rutting BR1	6.7
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.1414*Pow(RUT,0.25)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	50
IRI Flexible C2	0.55
IRI Flexible C3	0.0111
IRI Flexible C4	0.02
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.22
Granular Subgrade Rutting Standard Deviation	0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.37
Fine Subgrade Rutting Standard Deviation	0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	6.3
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	6.3
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 6.7 Performance Prediction Model Coefficients for Flexible Pavement Designs (Superpave Mix)

New Flexible Pavement-Calibration Settings

AC Cracking	
AC Cracking C1 Top	7
AC Cracking C2 Top	3.5
AC Cracking C3 Top	0
AC Cracking C4 Top	1000
AC Cracking Top Standard Deviation	200 + 2300/(1+exp(1.072-2.1654*LOG10(TOP+0.0001)))
AC Cracking C1 Bottom	0.021
AC Cracking C2 Bottom	2.35
AC Cracking C3 Bottom	6000
AC Cracking Bottom Standard Deviation	1+15/(1+exp(-3.1472-4.1349*LOG10(BOTTOM+0.0001)))
AC Fatigue	
AC Fatigue K1	0.007566
AC Fatigue K2	3.9492
AC Fatigue k3	1.281
AC Fatigue BF1	130.3674
AC Fatigue BF2	1
AC Fatigue BF3	1.217799
AC Rutting	
AC Rutting K1	-3.35412
AC Rutting K2	1.5606
AC Rutting K3	0.3791
AC Rutting BR1	4.3
AC Rutting BR2	1
AC Rutting BR3	1
AC Rutting Standard Deviation	0.1414*Pow(RUT,0.25)+0.001
CSM Cracking	
CSM Fatigue	
IRI	
IRI Flexible C1	50
IRI Flexible C2	0.55
IRI Flexible C3	0.0111
IRI Flexible C4	0.02
IRI Flexible Over PCCC1	40.8
IRI Flexible Over PCCC2	0.575
IRI Flexible Over PCCC3	0.0014
IRI Flexible Over PCCC4	0.00825
Subgrade Rutting	
Granular Subgrade Rutting K1	2.03
Granular Subgrade Rutting BS1	0.22
Granular Subgrade Rutting Standard Deviation	0.0104*Pow(BASERUT,0.67)+0.001
Fine Subgrade Rutting K1	1.35
Fine Subgrade Rutting BS1	0.37
Fine Subgrade Rutting Standard Deviation	0.0663*Pow(SUBRUT,0.5)+0.001
Thermal Fracture	
AC thermal cracking Level 1K	6.3
AC thermal cracking Level 1 Standard Deviation	0.1468 * THERMAL + 65.027
AC thermal cracking Level 2K	0.5
AC thermal cracking Level 2 Standard Deviation	0.2841 *THERMAL + 55.462
AC thermal cracking Level 3K	6.3
AC thermal cracking Level 3 Standard Deviation	0.3972 * THERMAL + 20.422
Identifiers	

Figure 6.8 Performance Prediction Model Coefficients for Flexible Pavement Designs (PMA Mix)

6.5 Select the Appropriate Reliability Level for the Project

Recommended reliability levels for flexible pavement designs are located in **Table 2.3 Reliability (Risk)**. The designer should select an appropriate reliability level based on highway functional class and location. **Figure 6.5 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)** shows design reliability values for a sample flexible pavement trial design.

6.6 Assemble M-E Design Software Inputs

6.6.1 General Information

6.6.1.1 Design Period

The design period for new flexible pavement construction and reconstruction is at least 20 years. For special designs, the designer may use a different design period as appropriate.

6.6.1.2 Construction Dates and Timeline

The following inputs are required to specify the construction dates and timeline (see **Figure 6.5 M-E Design Software Screenshot Showing General Information (left), Performance Criteria and Reliability (right)**):

- Base/subbase construction month and year
- Pavement construction month and year
- Traffic open month and year

The designer may select the most likely month and year for construction completion of the key activities listed above. Selection is based on the designer's experience, agency practices, or estimated from the planned construction schedule. For large projects that extend into different paving seasons, it is suggested each paving season be evaluated separately and the designer judge the acceptability of the trial design based on the more conservative situation. The M-E Design software does not consider staged construction events, nor does it consider the impact of construction traffic on damage computations.

Note: The pavement performance predictions begin from the month the pavement is open to traffic. The changes to pavement material properties due to time and environmental conditions are calculated beginning from the month and year the material was placed.

6.6.1.3 Identifiers

Identifiers are helpful in documenting the project location and recordkeeping. M-E Design allows designers to enter site or project identification information such as the location of the project (route signage, jurisdiction, etc.), identification numbers, beginning and ending milepost, direction of traffic, and date.

6.6.2 Traffic

Several inputs are required for characterizing traffic for the M-E Design software and are described in detail in **Section 3.1 Traffic**.

6.6.3 Climate

The climate input requirements for the M-E Design software are described in detail in **Section 3.2 Climate**.

6.6.4 Pavement Layer Characterization

As shown in **Figure 6.2 Structural Layers**, a typical flexible pavement design comprises of the following pavement layers: asphalt concrete, unbound aggregate base layers, and subgrade. The inputs required by M-E Design for characterizing these layers are described in the following sections.

6.6.4.1 Asphalt Concrete Characterization

Asphalt concrete types used in Colorado include:

- **Hot Mix Asphalt (HMA)**: Composed of aggregates with an asphalt binder and certain anti-stripping additives.
- **Stone Matrix Asphalt (SMA)**: Gap-graded HMA that maximizes rutting resistance and durability with a stable stone-on-stone skeleton held together by a rich mixture of AC, filler, and stabilizing agents.

The designers should apply the following guidelines when defining an asphalt concrete layer:

- As much as possible and as appropriate, the asphalt concrete layers must be combined into three layers: surface, intermediate and base. Asphalt layers with similar HMA mixtures may be combined into a single layer.
- When multiple layers are combined, the properties of the combined layer should be the weighted average of the individual layers.
- The M-E Design software does not consider very thin layers (thickness less than 1.5 inches).
- Weakly stabilized asphalt materials (i.e. sand-asphalt) should not be considered an asphalt concrete layer.
- M-E Design models layer by layer rutting. **Table 6.1 Layered Rut Distribution** shows the percentages used for calculating the final rutting in Colorado.

Table 6.1 Layered Rut Distribution

Layer	Colorado Percent Distribution	Global Percent Distribution
Hot Mix Asphalt	60	80
Aggregate Base Course	10	5
Subgrade	30	15

Designers are required to input volumetric properties such as air voids, effective asphalt content by volume, aggregate gradation, mix density, and asphalt binder grade (see **Figure 6.9 Asphalt Concrete Layer and Material Properties in M-E Design**). The designers are also required to input the engineering properties such as the dynamic modulus, creep compliance, indirect tensile strength of HMA materials, and the viscosity versus temperature properties of rolling thin film oven (RTFO) aged asphalt binders. These inputs can be obtained following the input hierarchy levels depending on the criticality of the project. The volumetric properties entered into the program need to be representative of the in-place asphalt concrete mixture. The project-specific in-place mix properties will not be available at the design stage. The designer should use typical values available from previous construction records or target values from the project specifications.

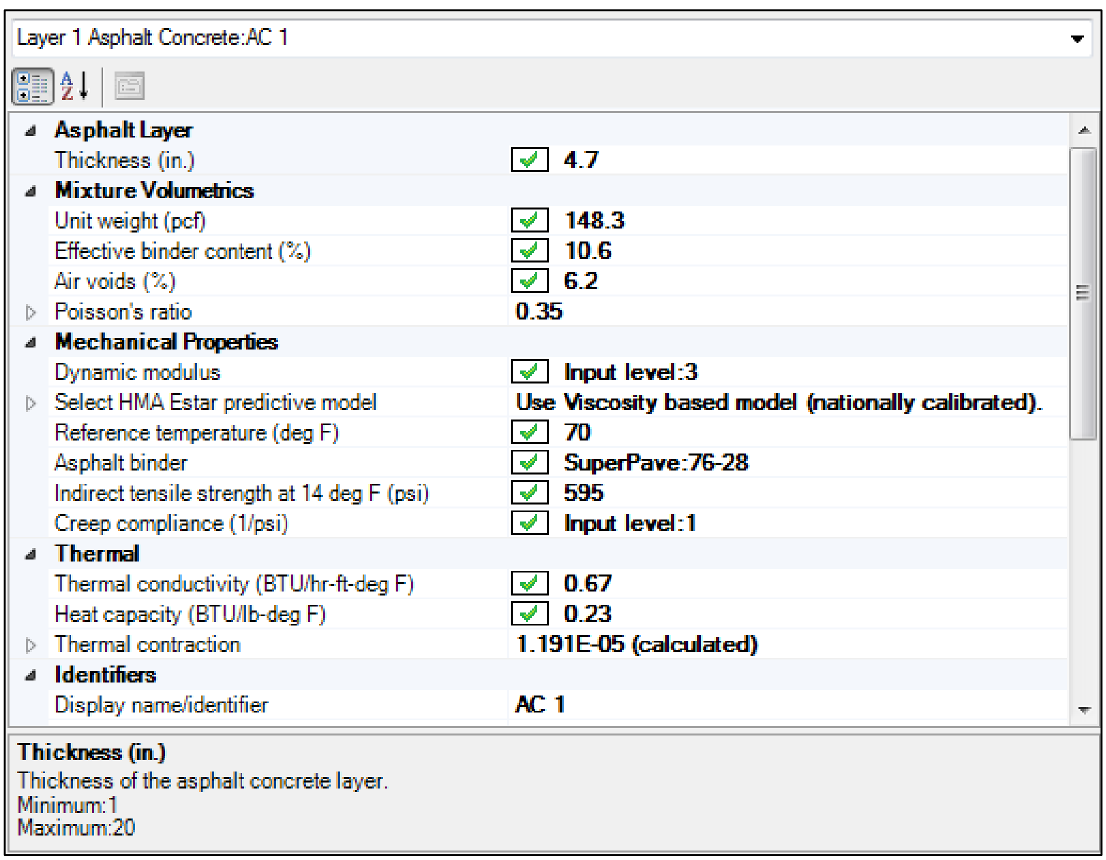


Figure 6.9 Asphalt Concrete Layer and Material Properties in M-E Design

Table 6.2 Input Properties and Recommendations for HMA Material Characterization presents the HMA input requirements of the M-E Design Method and recommendations for obtaining inputs at each hierarchical input level. The designer may use Level 1 inputs of typical CDOT HMA mixtures for Level 2 and 3 inputs. See **APPENDIX F** and **Table 2.6 Selection of Input Hierarchical Level** for selection of an appropriate hierarchical level for HMA characterization. For new construction (i.e. new HMA) the designer **should always click "True" for the Possion's Ratio** (currently the default value is "False").

Table 6.2 Input Properties and Recommendations for HMA Material Characterization

Input Property	Level 1	Level 2	Level 3
Dynamic Modulus (E*)	Mix specific E* and/or AASHTO TP62 test results	Gradation (**APPENDIX E**)	
Asphalt Binder Properties	Binder properties from laboratory testing of HMA using AASHTO T315		Binder grade (**APPENDIX E**)
Tensile Strength [1] at 14 °F Creep Compliance	AASHTO T322 test results	Use tensile strength and creep compliance (**APPENDIX E**)	
Poisson's Ratio	M-E Design software option *(Is Poisson's ratio calculated?)*		Use 0.35
Air Voids	Use air voids (**APPENDIX E**)		
Volumetric Asphalt Content	Use volumetric asphalt content (**APPENDIX E**)		
Total Unit Weight	Use total unit weight (**APPENDIX E**)		
Surface Shortwave Absorptivity	Use 0.85		
Coefficient of Thermal Contraction of the Mix	1.3E-05 in./in./°F (mix CTE) and 5.0 E-06 in./in./°F (aggregate CTE)		
Thermal Conductivity	0.67 Btu/(ft)(hr)(°F)		
Heat Capacity	0.23 BTU/lb.- ˚F		
Reference Temperature	70 ˚F		

Note: [1] The designer should use Level 1 Inputs. The Level 3 Inputs for tensile strength are much smaller which will cause more thermal cracking and greater creep compliance.

6.6.4.2 Unbound Layers and Subgrade Characterization

Refer to **Section 5.3.1 Unbound Layer Characterization in M-E Design** for unbound aggregate base layer characterization. Refer to **Section 4.4 Subgrade Characterization for M-E Design** for subgrade characterization.

6.7 Run M-E Design Software

Designers should examine all inputs for accuracy and reasonableness prior to running the M-E Design software. Next, one should run the software to obtain outputs required to determine if the trial design is adequate. After a trial run has been successfully completed, M-E Design will generate a report in form of a PDF and/or Microsoft Excel file, refer to **Figure 6.10 Sample Flexible Pavement Trial Design PDF Output Report**. The output report has input information, reliability of design, material properties, and predicted performance. It also includes the month to month estimates of material properties over the entire design period in either tabular or graphical form. For a flexible pavement trial design, the report provides the following:

- Monthly estimates of HMA dynamic modulus for each sublayer
- Monthly estimates of resilient modulus of unbound layers and subgrade
- Monthly estimates of AADTT
- Monthly estimates of climate parameters
- Cumulative trucks (FHWA Class 4 through 13) over the design period
- Cumulative ESALs over the design period (an intermediate file in the project folder)

After the trial run is complete, the designer should re-examine all inputs and outputs for accuracy and reasonableness before accepting a trial design as complete.

23-11918

File Name: C:\Users\bbhattacharya.RESTRICT\Desktop\COLORADO MEPDG\FINAL REPORT\23-11918.dgpx

Design Inputs

Design Life:	20 years	Base construction:	May, 2001	Climate Data Sources (Lat/Lon)	39.909, -105.117
Design Type:	Flexible Pavement	Pavement construction:	June, 2001		39.702, -104.752
		Traffic opening:	September, 2001		39.833, -104.658
					39.57, -104.849
					39.783, -104.55
					40.452, -105.001
					39.217, -104.633
					40.436, -104.618

Design Structure

Layer type	Material Type	Thickness (in.):
Flexible	AC 1	4.7
Flexible	AC 2	5.0
Flexible	AC 3	5.0
NonStabilized	A-1-a	6.0
Subgrade	A-7-6	Semi-infinite

Volumetric at Construction:	
Effective binder content (%)	10.6
Air voids (%)	6.2

Traffic

Age (year)	Heavy Trucks (cumulative)
2001 (initial)	1,936
2011 (10 years)	8,349,420
2021 (20 years)	20,287,400

Design Outputs

Distress Prediction Summary

Distress Type	Distress @ Specified Reliability		Reliability (%)		Criterion Satisfied?
	Target	Predicted	Target	Achieved	
Terminal IRI (in./mile)	172.00	170.82	90.00	90.59	Pass
Permanent deformation - total pavement (in.)	0.75	0.50	90.00	100.00	Pass
AC bottom-up fatigue cracking (percent)	25.00	1.53	90.00	100.00	Pass
AC thermal cracking (ft/mile)	250.00	84.34	90.00	99.99	Pass
AC top-down fatigue cracking (ft/mile)	2000.00	256.64	90.00	100.00	Pass
Permanent deformation - AC only (in.)	0.25	0.28	90.00	80.53	Fail

Figure 6.10 Sample Flexible Pavement Trial Design PDF Output Report

6.8 Evaluate the Adequacy of the Trial Design

The output report of a flexible pavement trial design includes the monthly accumulation of the following key distress types at their mean values and chosen reliability for the entire design period:

- **Alligator Fatigue Cracking:** Traditional wheel path cracking that initiates at the bottom of the HMA layer and propagates to the surface under repeated load applications. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures. Fatigue cracking is highly dependent on the effective asphalt content by volume and air voids.

- **Transverse Cracking:** Thermal cracks typically appear as transverse cracks on the pavement surface due to low temperatures, hardening of the asphalt, and/or daily temperature cycles. Excessive transverse cracking may adversely affect ride quality.

The designer should examine the results to evaluate if the performance criteria for each of the above-mentioned indicators are met at the desired reliability. **If alligator fatigue cracking or transverse cracking criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.**

The output report also includes the monthly accumulation of the following secondary distress types and smoothness indicators at their mean values and chosen reliability for the entire design period:

- **Permanent Deformation**: The report includes HMA rutting and total permanent deformation (includes rutting on unbound layers and subgrade). Excessive rutting may cause safety concerns.

- **Surface-Initiated Fatigue Cracking or Longitudinal Cracking**: These load-related cracks appear at the HMA surface and propagate downwards. Beyond a critical threshold, the rate of cracking accelerates and may require significant repairs and lane closures.

- **IRI**: The roughness index represents the profile of the pavement in the wheel paths. Higher IRI indicates unacceptable ride quality.

The designer should examine the results to evaluate if the performance criteria for permanent deformation, surface-initiated fatigue cracking or longitudinal cracking, and IRI meet the minimum of 12 years at the desired reliability. If any of the criteria have not been met, the trial design is deemed unacceptable and revised accordingly to produce a satisfactory design.

Another important output is the reliability level of each performance indicator at the end of the design period. If the reliability value predicted for the given performance indicator is greater than the target/desired value, the trial design passes for that indicator. If the reverse is true, then the trial design fails to provide the desired confidence and the performance indicator will not reach the critical value during the pavement's design life. In such an event, the designer needs to alter the trial design to correct the problem.

The strategies for modifying a trial design are discussed in **Section 6.9 Modifying Trial Designs**. The designer can use a range of thicknesses to optimize the thickness of the trial design to make it more acceptable. In addition, the software allows the designer to perform a sensitivity analysis on the key inputs. The results of the sensitivity analysis can be used to further optimize the trial design if modifying AC thickness alone does not produce a feasible design alternative. A detail description of thickness optimization procedure and sensitivity analysis is provided in the *Software HELP Manual*.

6.9 Modifying Trial Designs

An unsuccessful trial design may require revisions to ensure all performance criteria are satisfied. The trial design is modified by systematically revising the design inputs. In addition to layer thickness, many other design factors influence performance predictions. The design acceptance is distress-specific; in other words, the designer needs to first identify the performance indicator that failed to meet the performance target and modify one or more design inputs that has a significant impact on the given performance indicator. The impact of design inputs on performance indicators is typically obtained by performing a sensitivity analysis. Strategies used to produce a satisfactory design by modifying design inputs can be broadly categorized into to following:

- Pavement layer considerations
- Increasing layer thickness
- Modifying layer type and layer arrangement
- Foundation improvements (i.e. stabilize the upper subgrade soils)
- Pavement material improvements:
    - Use of higher quality materials (i.e. use of polymer modified asphalt, crushed stones)
    - Material design modifications (i.e. increase asphalt content, reduce amount of fines, modify gradations etc.)
    - Construction quality (i.e. reduce HMA air voids, increase compaction density, decrease as-constructed pavement smoothness)

Once again, when modifying the design inputs, the designer needs to be aware of the sensitivity of these inputs to various distress types. Changing a single input to reduce one distress may result in an increase in another distress. For example, the designer may consider using a harder asphalt to reduce HMA rutting, but that will likely increase the predicted transverse cracking. **Table 6.3 Modifying Flexible Pavement Trial Designs** presents a summary of inputs that may be modified to optimize trial designs and produce a feasible design alternative.

Table 6.3 Modifying Flexible Pavement Trial Designs

Distress/IRI	Design Inputs that Impact
AC Rutting	• Use a polymer modified asphalt for the HMA surface layer • Increase the dynamic modulus of the HMA mixture(s) • Reduce the asphalt content in the HMA mixture(s) • Increase the amount of crushed aggregate • Increase the amount of manufactured fines in the HMA mixture

Transverse Cracking	• Decrease the stiffness of the AC surface mix
	  ▪ Use a softer asphalt
	  ▪ Increase asphalt binder
	  ▪ Increase indirect tensile strength
	  ▪ Reduce creep compliance
	• Increase AC layer thickness

Alligator Cracking	• Increase HMA layer thickness
	• Increase HMA dynamic modulus for HMA layers thicker than 5 inches and decrease HMA dynamic modulus for HMA layers thinner than 3 inches
	• Revise the mixture design of the HMA base layer
	  ▪ Increase asphalt binder content
	  ▪ Achieve higher density and lower air voids during compaction
	  ▪ Use harder asphalt/polymer modified asphalt but ensure good compaction is achieved
	  ▪ Increase percent manufactured fines, and/or percent crushed aggregates
	• Reduce stiffness gradients between upper and lower layers
	  ▪ Using a higher quality/stiffer HMA layer on top of poor quality/low resilient modulus granular base or foundation tends to increase fatigue cracking
	• Increase the thickness or stiffness of a high quality unbound base layer and/or use a stabilized layer

Unbound Base Rutting	• Increase the resilient modulus of the aggregate base • Increase the density of the aggregate base • Stabilize the upper foundation layer for weak, frost susceptible, or swelling soils • Place a layer of select embankment material with adequate compaction • Increase the HMA or granular layer thickness • Address drainage related issues to protect from the detrimental effects of moisture

Subgrade Rutting	• Increase the layer stiffness and layer thickness of any layers above the subgrade layers:
	  ▪ Increase HMA and/or unbound layer thickness or stiffness
	  ▪ Include a stabilized drainable base

Distress/IRI	Design Inputs that Impact
	• Improve the engineering properties of the subgrade material: ▪ Increase the stiffness (modulus) of the subgrade layer(s) itself through the use of lime stabilized subgrade ▪ Effective use of subsurface drainage systems, geotextile fabrics, and impenetrable moisture barrier wraps to protect from the detrimental effects of moisture ▪ Increase the grade elevation to increase the distance between the subgrade surface and ground water table
IRI	• Reduce initial IRI (achieving smoother as-constructed pavement surface through more stringent smoothness criteria) • Improve roadbed foundation (replace frost susceptible or expansive subgrade with non-frost susceptible or stabilized subgrade materials) • Place subsurface drainage system to remove ground water

Figure 6.11 Sensitivity of HMA Alligator Cracking to Truck Volume through **Figure 6.32 Sensitivity of HMA IRI to Base Thickness**. **Figure 6.30 Sensitivity of HMA IRA to AC Thickness** presents sensitivity plots of a sample flexible pavement trial design showing the effects of key inputs, such as traffic volume, asphalt binder content, asphalt binder grade, air voids, base type, base thickness, and climate on key distresses/IRI. **Note:** The plots do not exhaustively cover the effects of all key factors on flexible pavement performance; other significant factors are not shown herein.

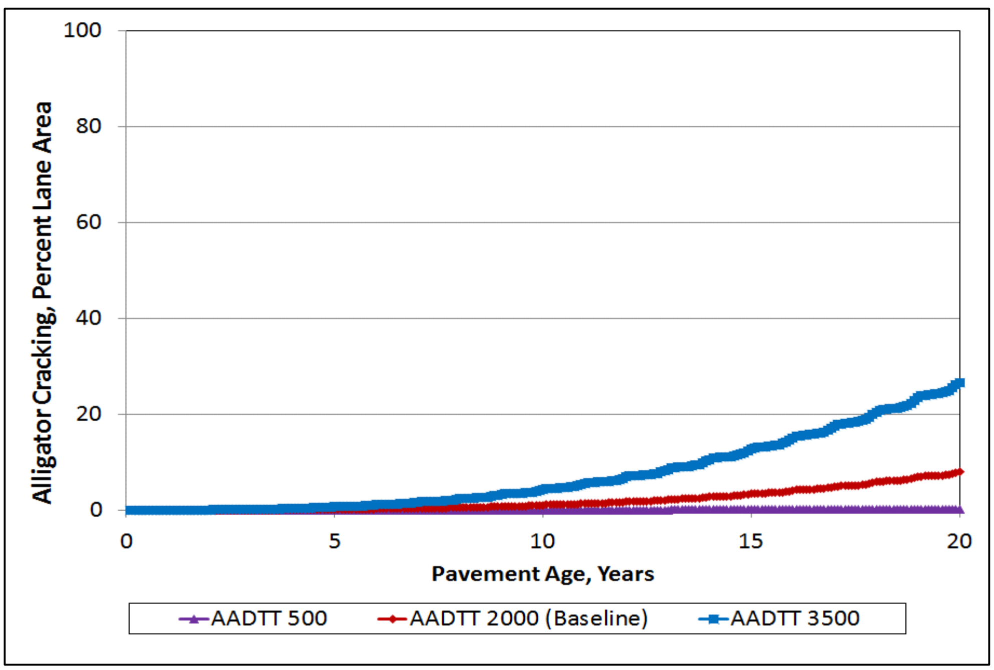


Figure 6.11 Sensitivity of HMA Alligator Cracking to Truck Volume

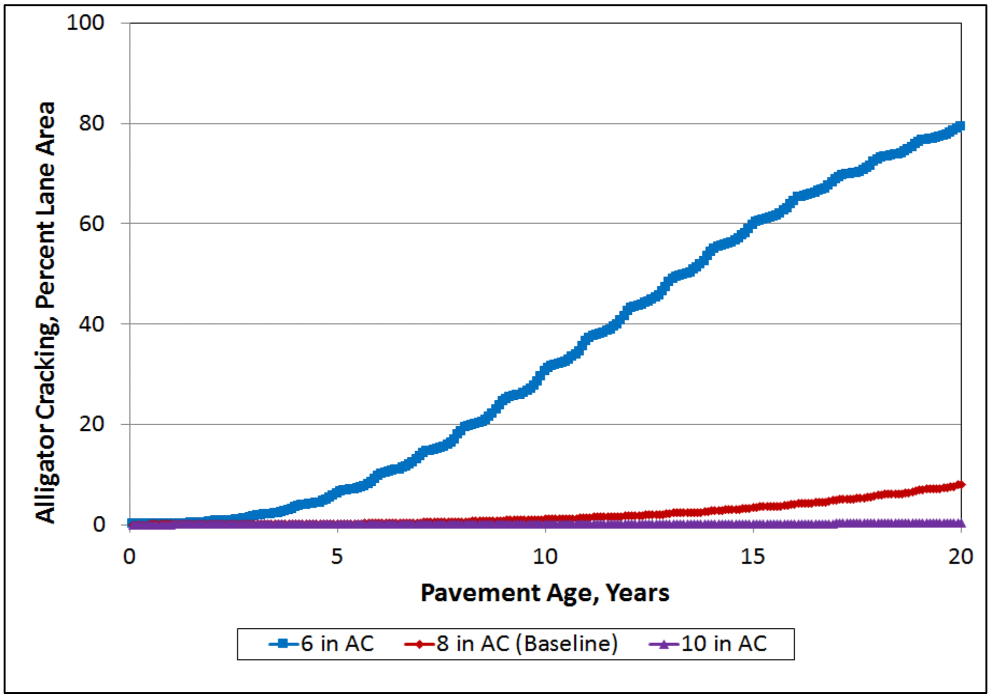


Figure 6.12 Sensitivity of HMA Alligator Cracking to AC Thickness

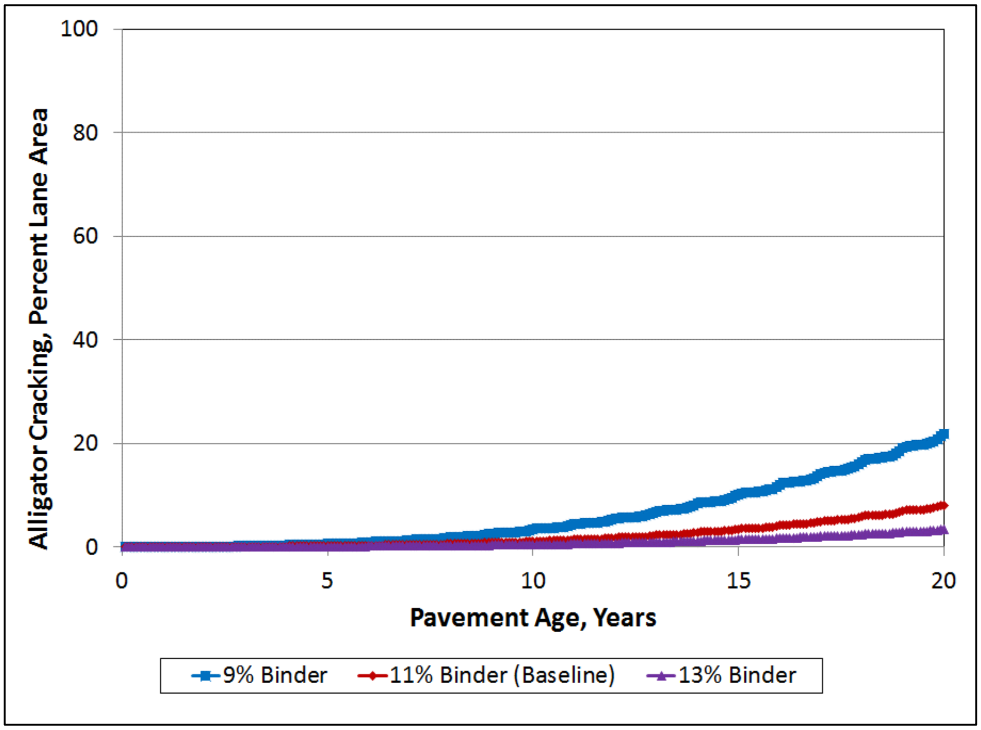


Figure 6.13 Sensitivity of HMA Alligator Cracking to Asphalt Binder Content

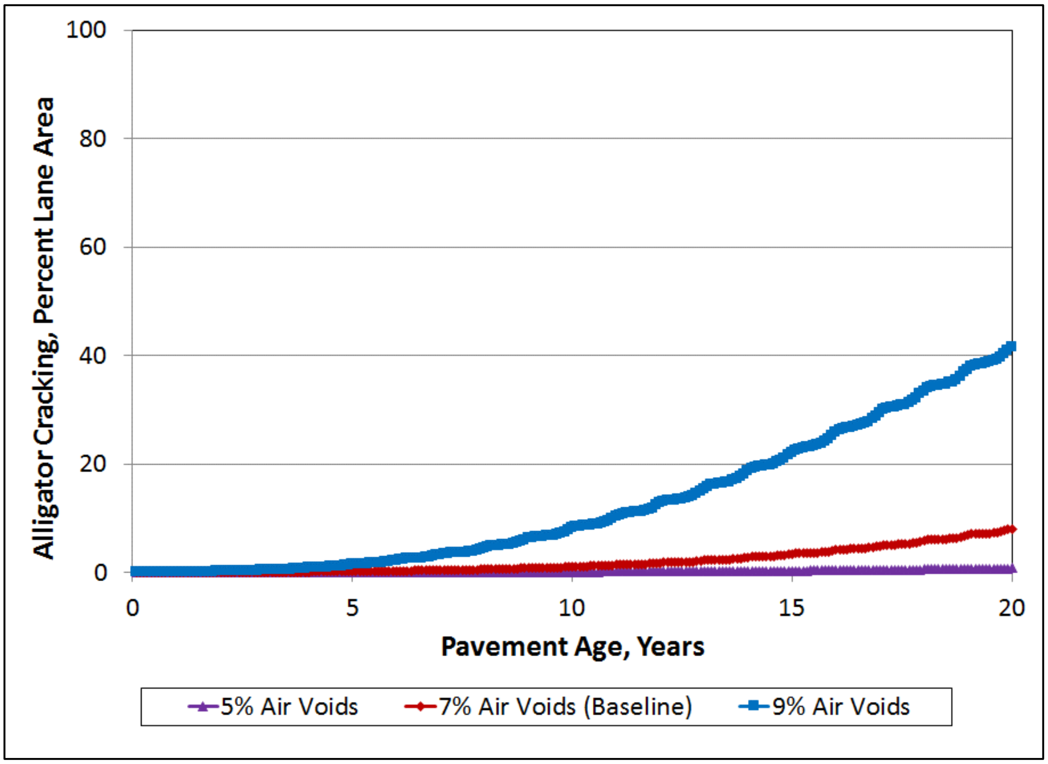


Figure 6.14 Sensitivity of HMA Alligator Cracking to Air Voids

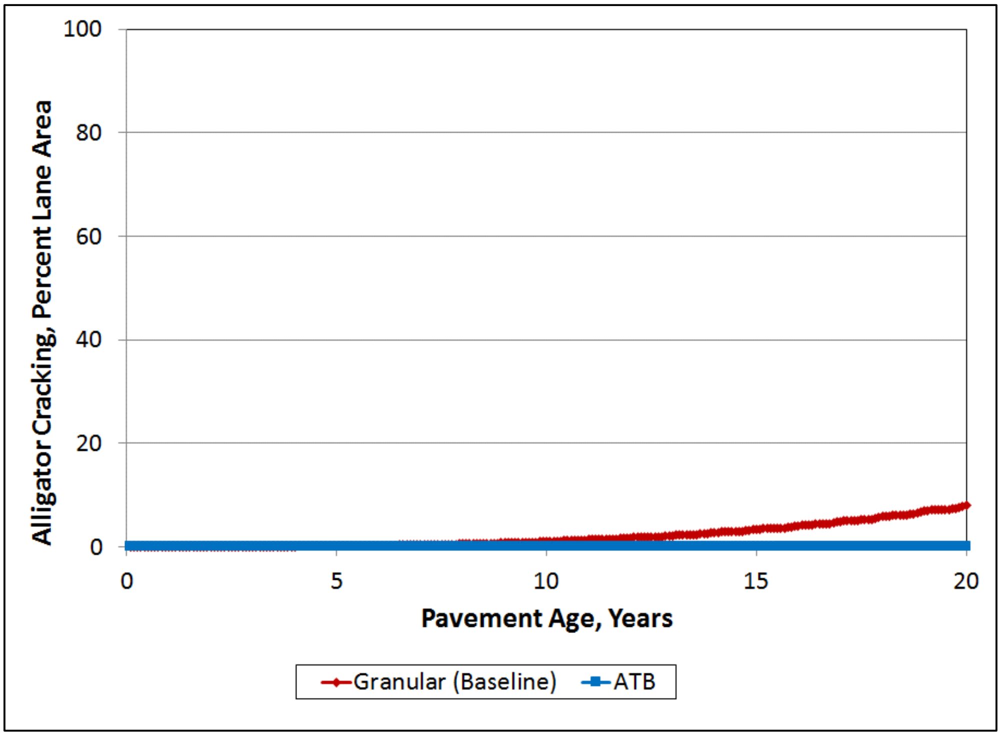


Figure 6.15 Sensitivity to HMA Alligator Cracking to Base Type

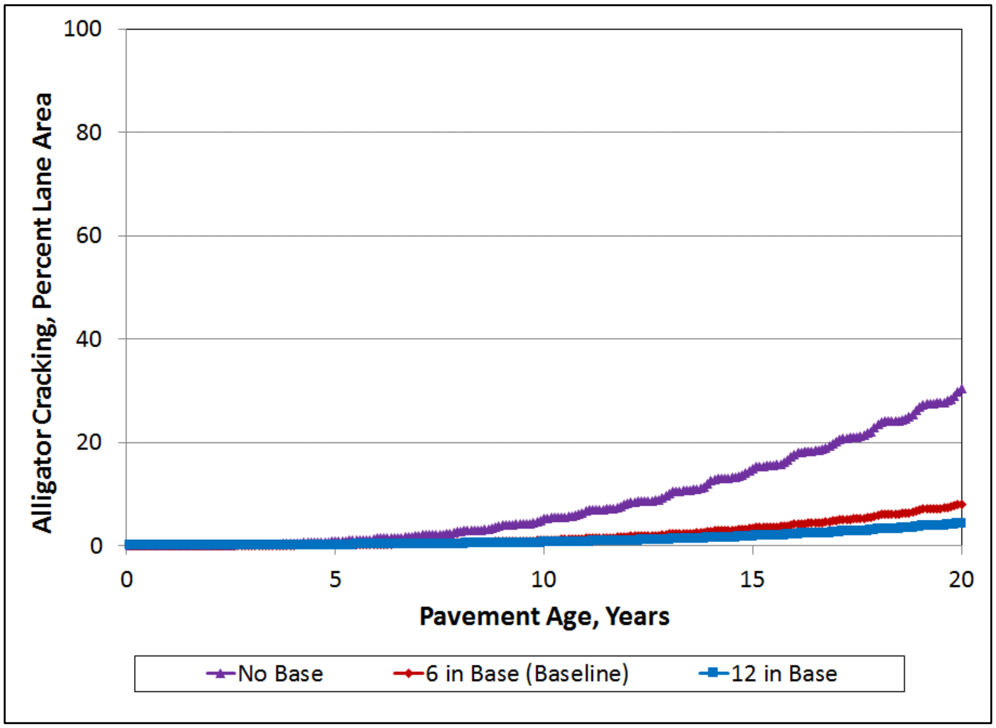


Figure 6.16 Sensitivity of HMA Alligator Cracking to Base Thickness

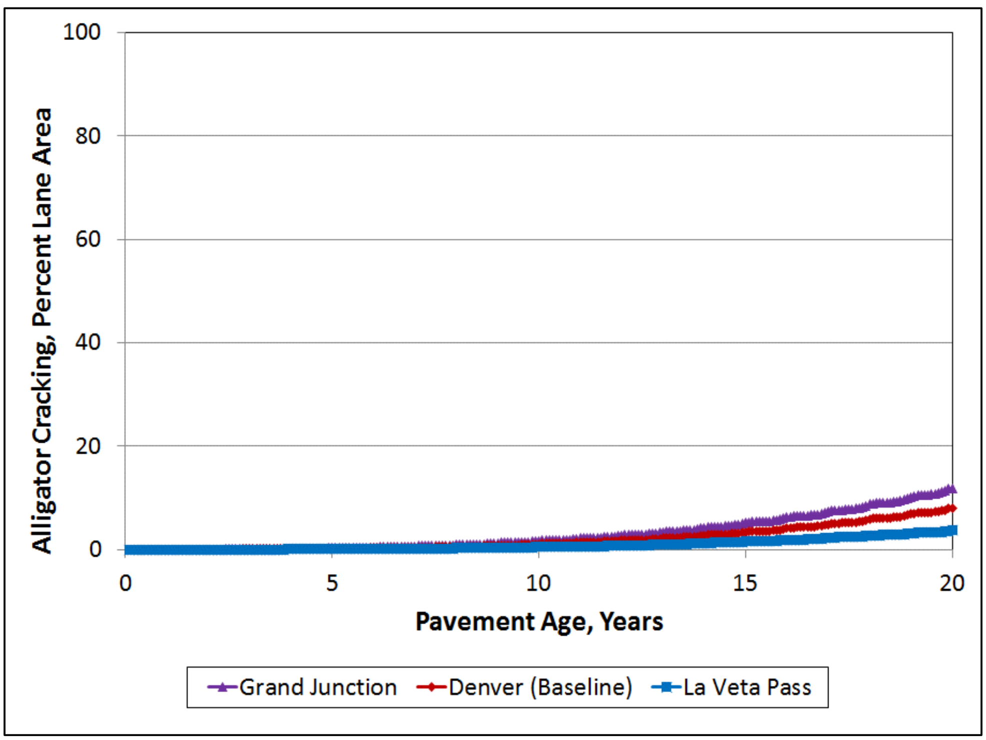


Figure 6.17 Sensitivity of HMA Alligator Cracking to Climate

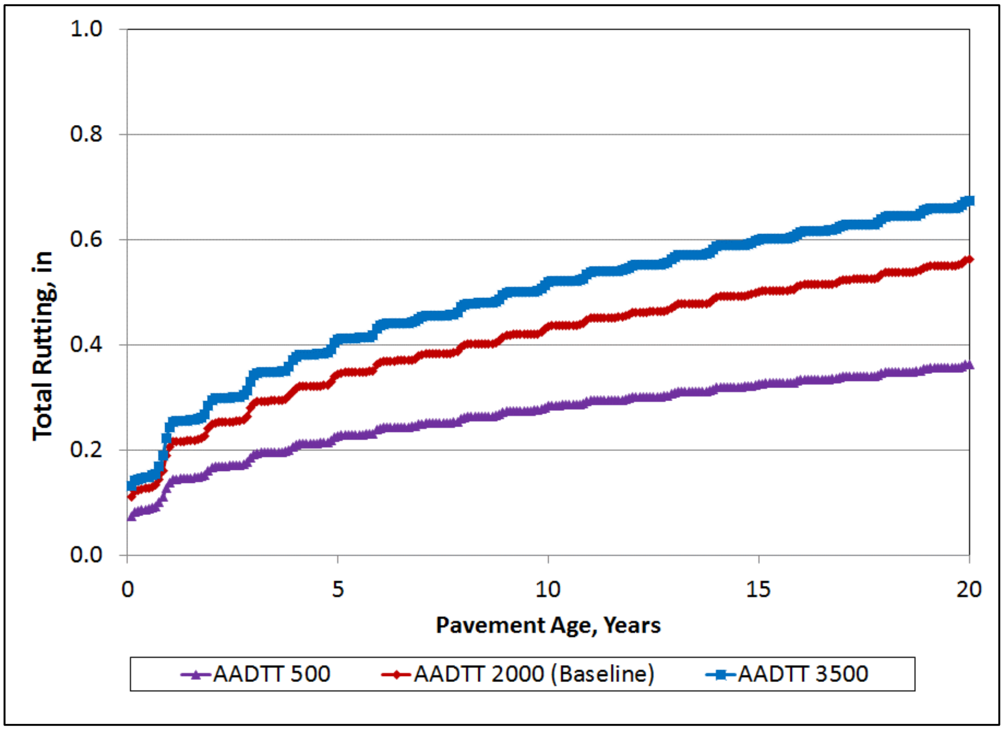


Figure 6.18 Sensitivity of Total Rutting to Truck Volume

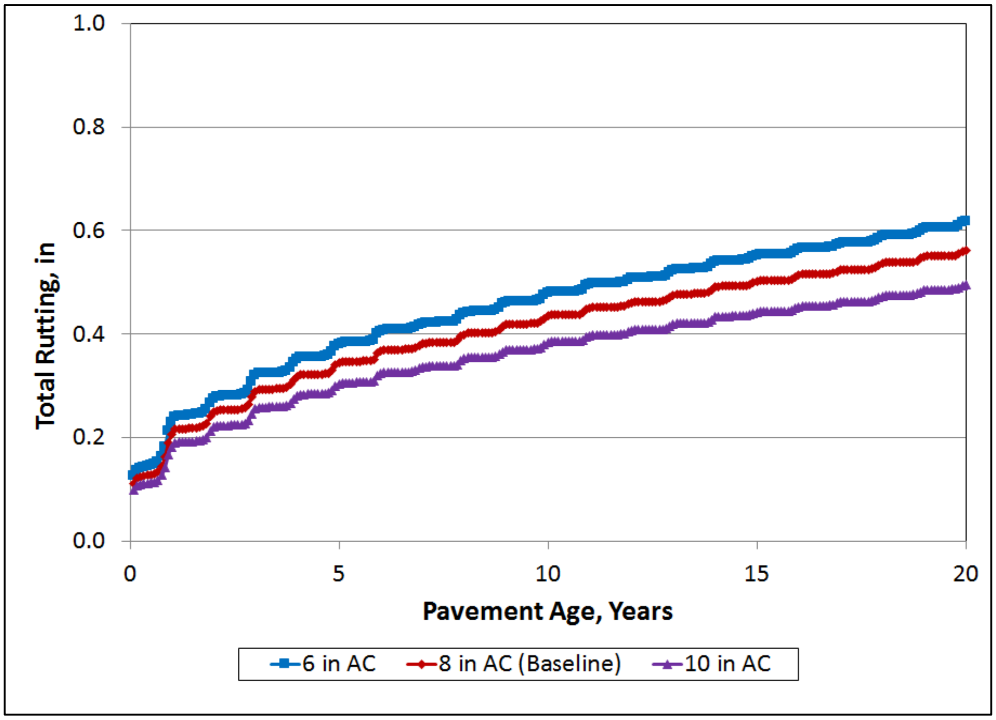


Figure 6.19 Sensitivity of Total Rutting to AC Thickness

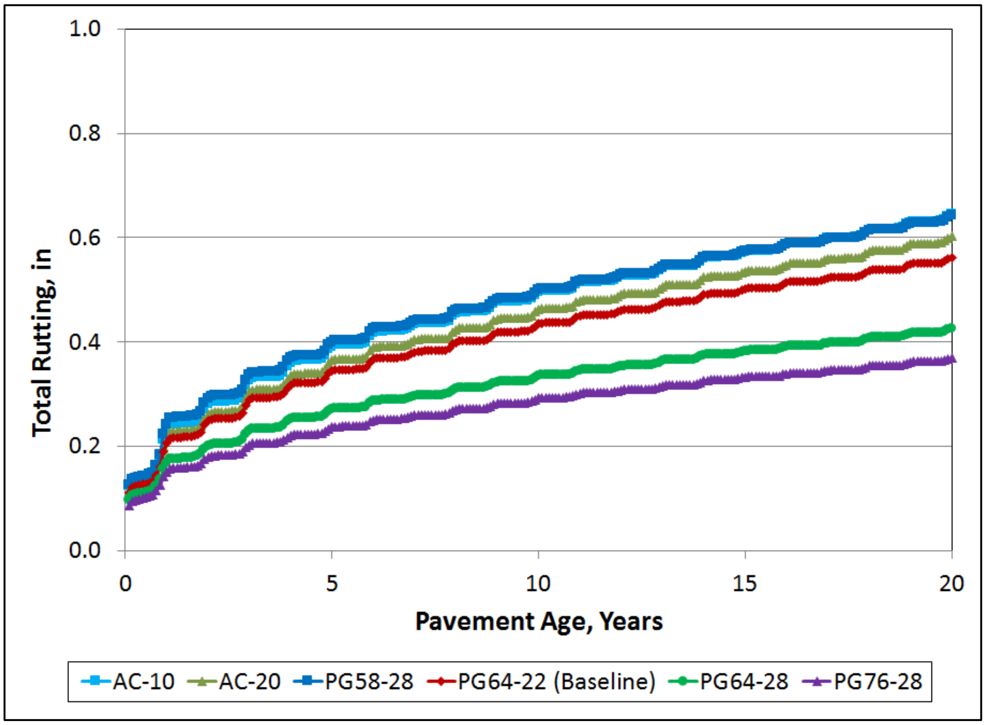


Figure 6.20 Sensitivity of Total Rutting to Asphalt Binder Grade

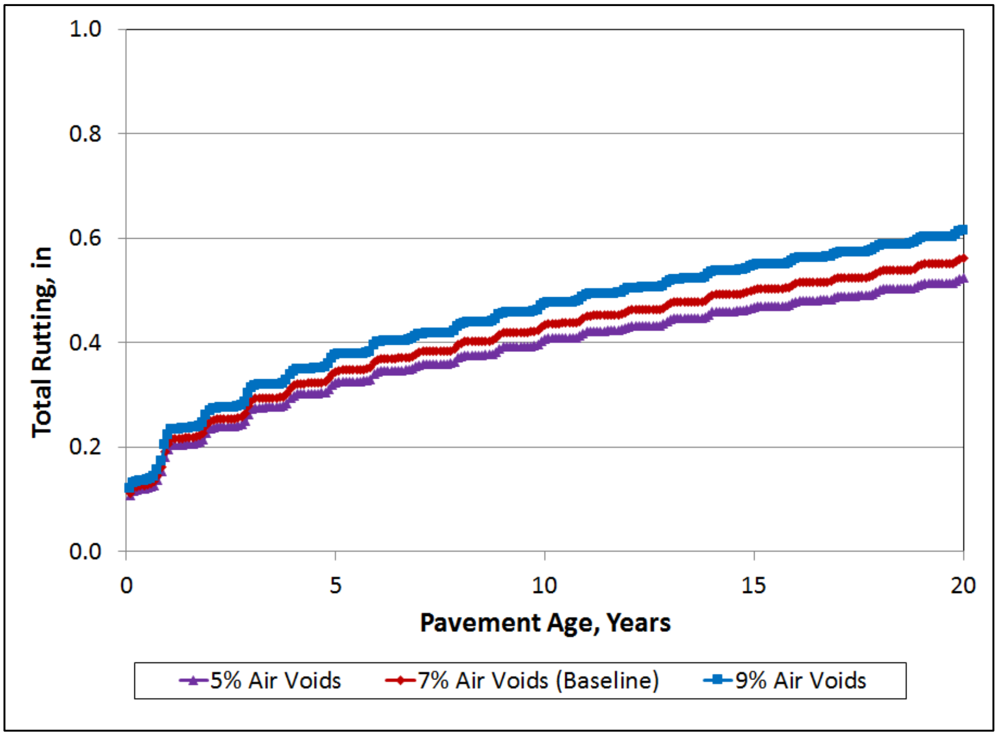


Figure 6.21 Sensitivity of Total Rutting to Air Voids

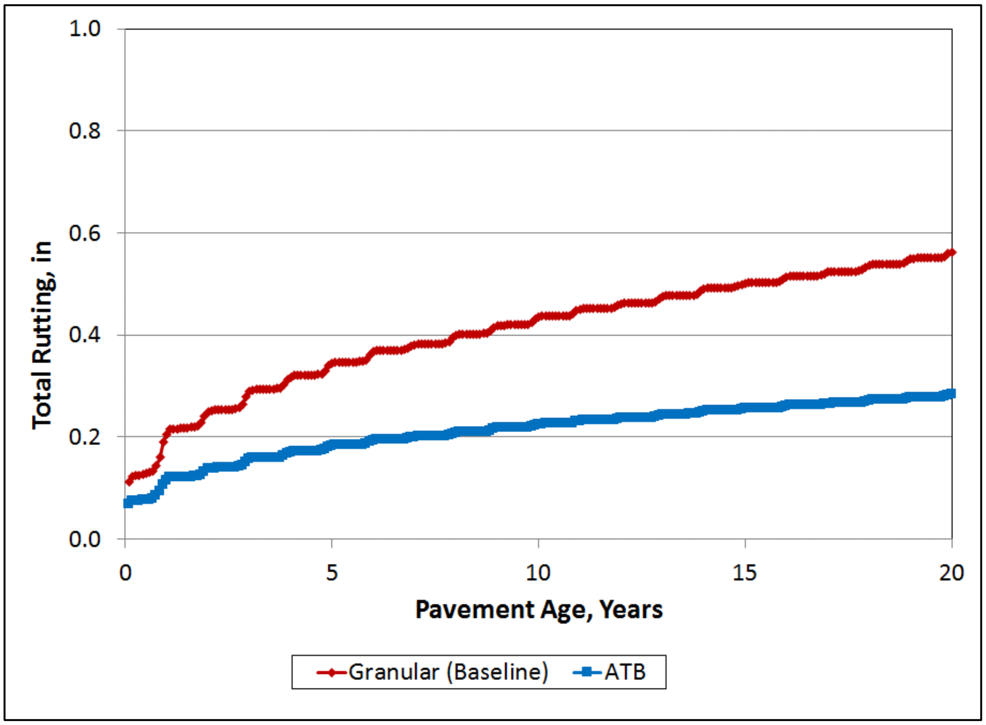


Figure 6.22 Sensitivity of Total Rutting to Base Type

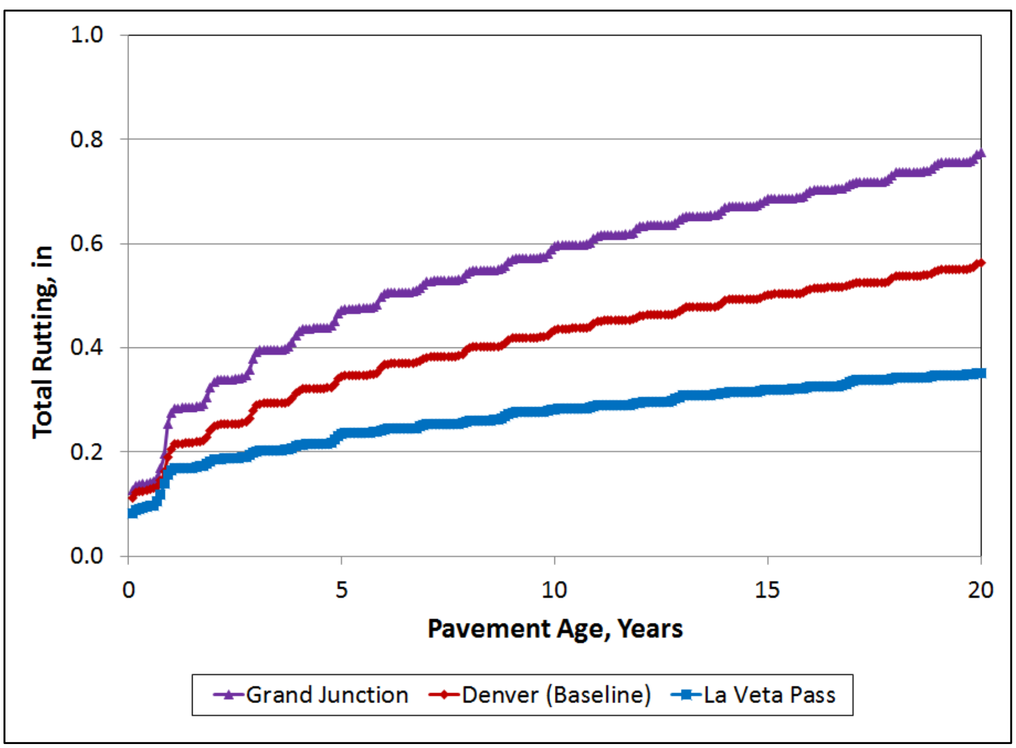


Figure 6.23 Sensitivity of Total Rutting to Climate

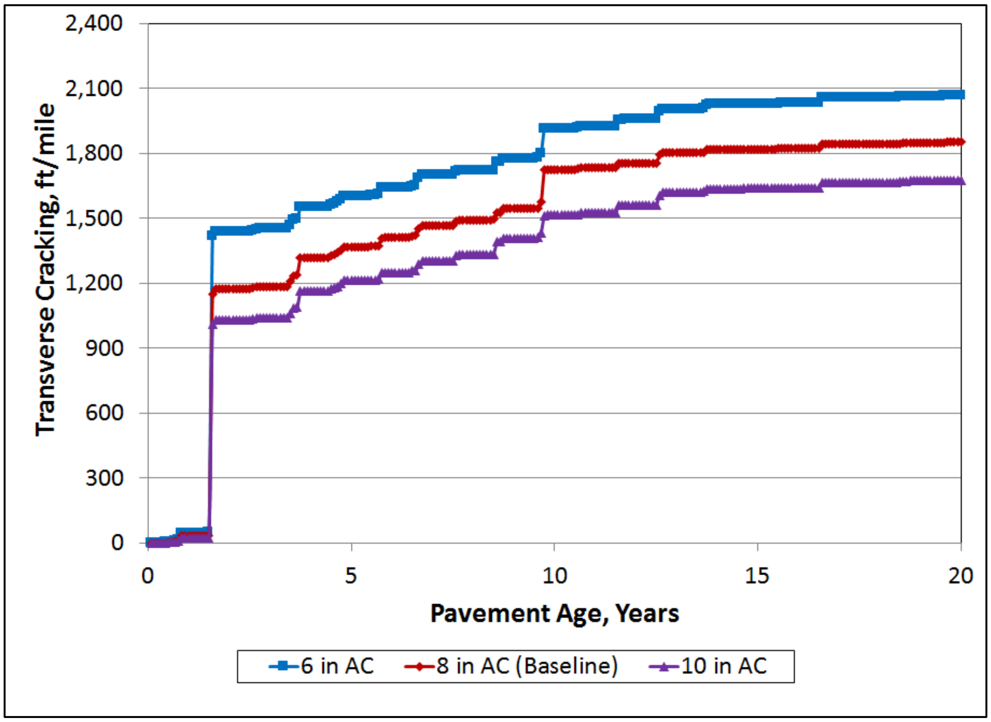


Figure 6.24 Sensitivity of HMA Transverse Cracking to Thickness

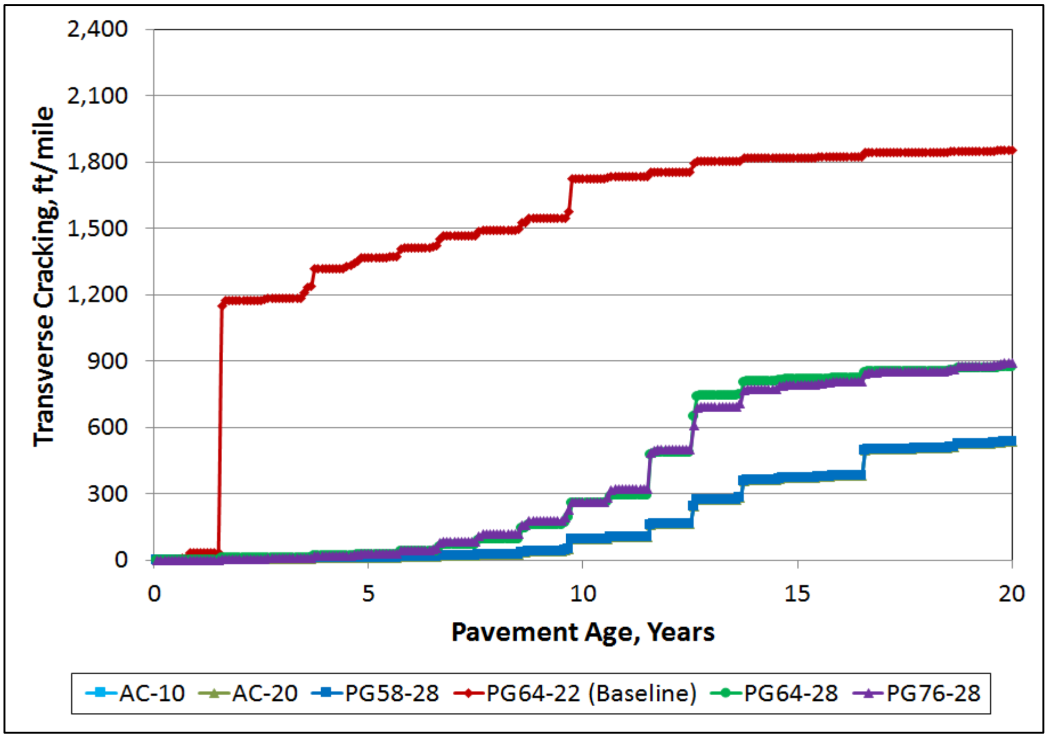


Figure 6.25 Sensitivity of HMA Transverse Cracking to Asphalt Binder Grade

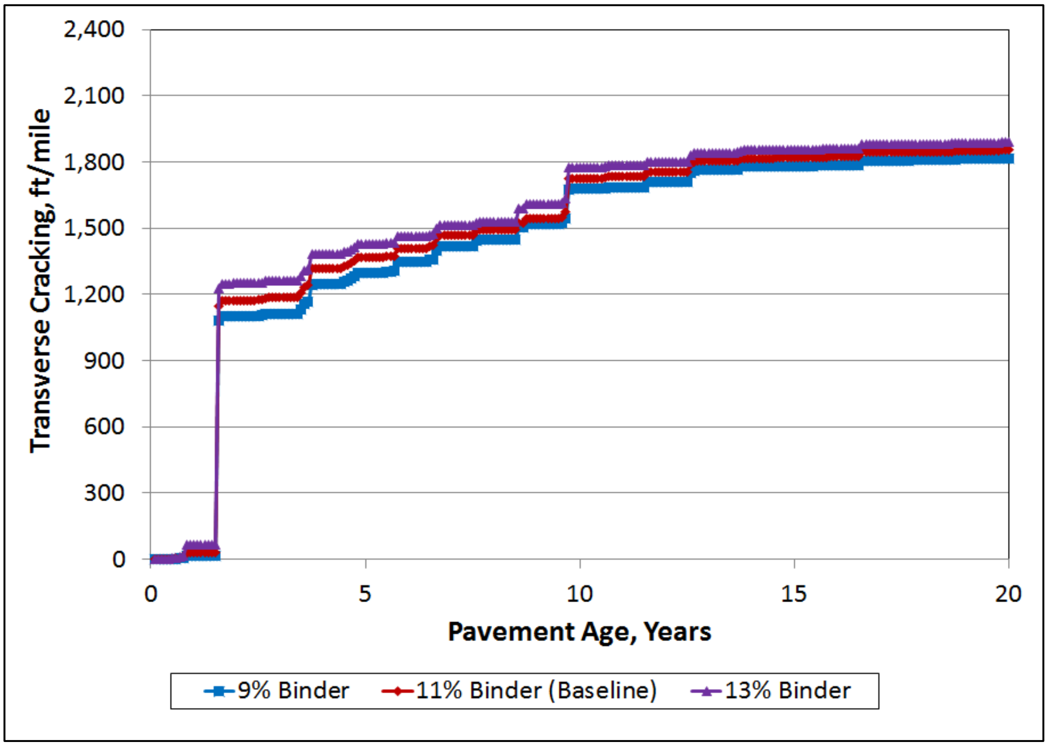


Figure 6.26 Sensitivity of HMA Transverse Cracking to Asphalt Binder Content

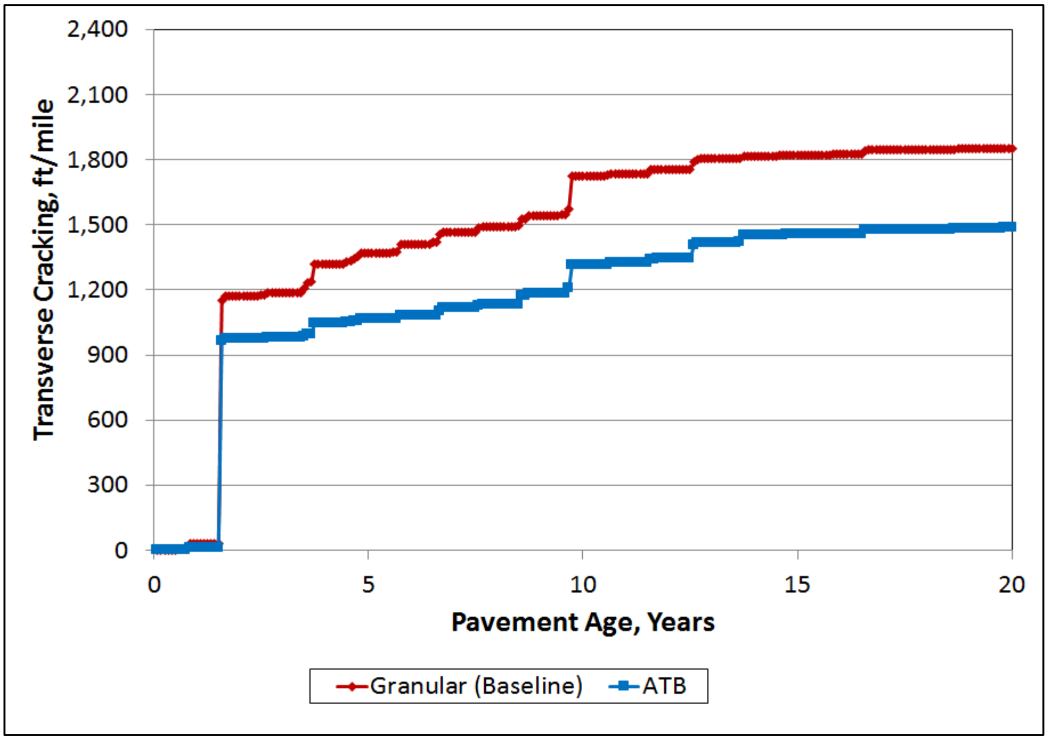


Figure 6.27 Sensitivity of HMA Transverse Cracking to Base Type

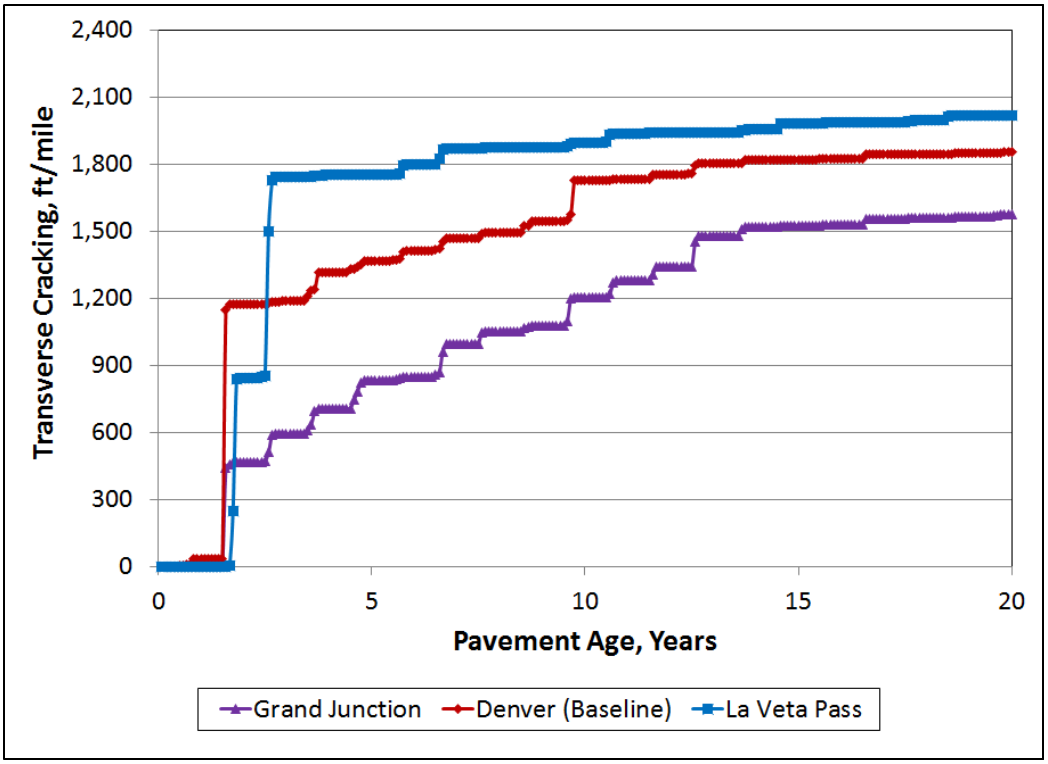


Figure 6.28 Sensitivity of HMA Transverse Cracking to Climate

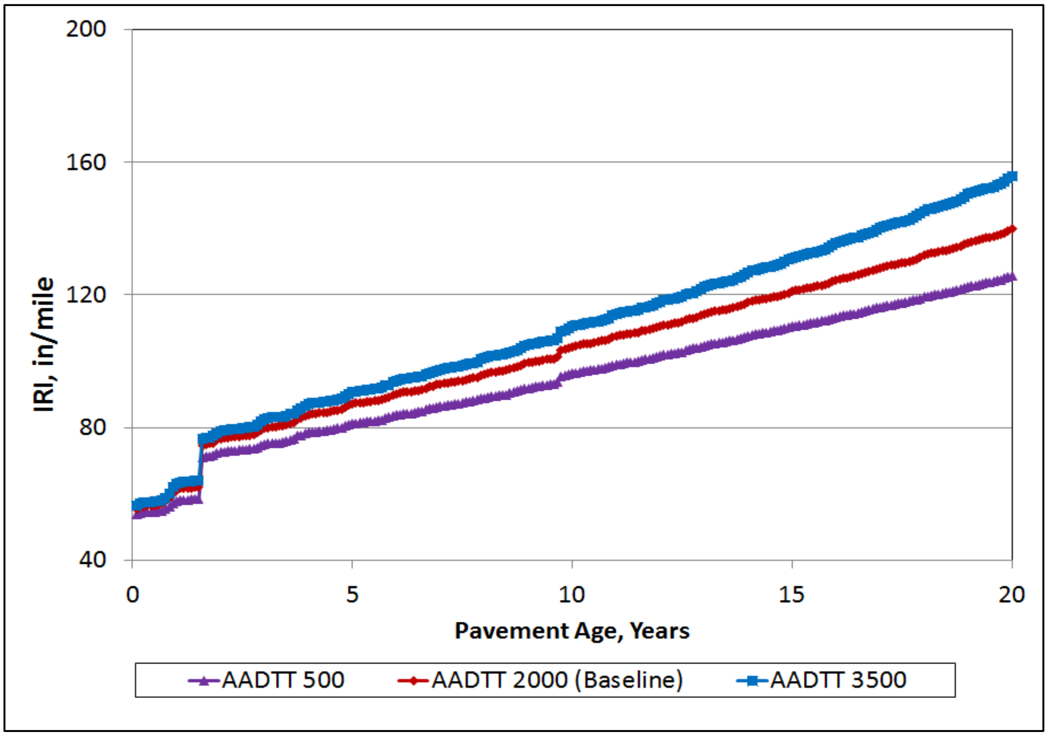


Figure 6.29 Sensitivity of HMA IRI to Truck Volume

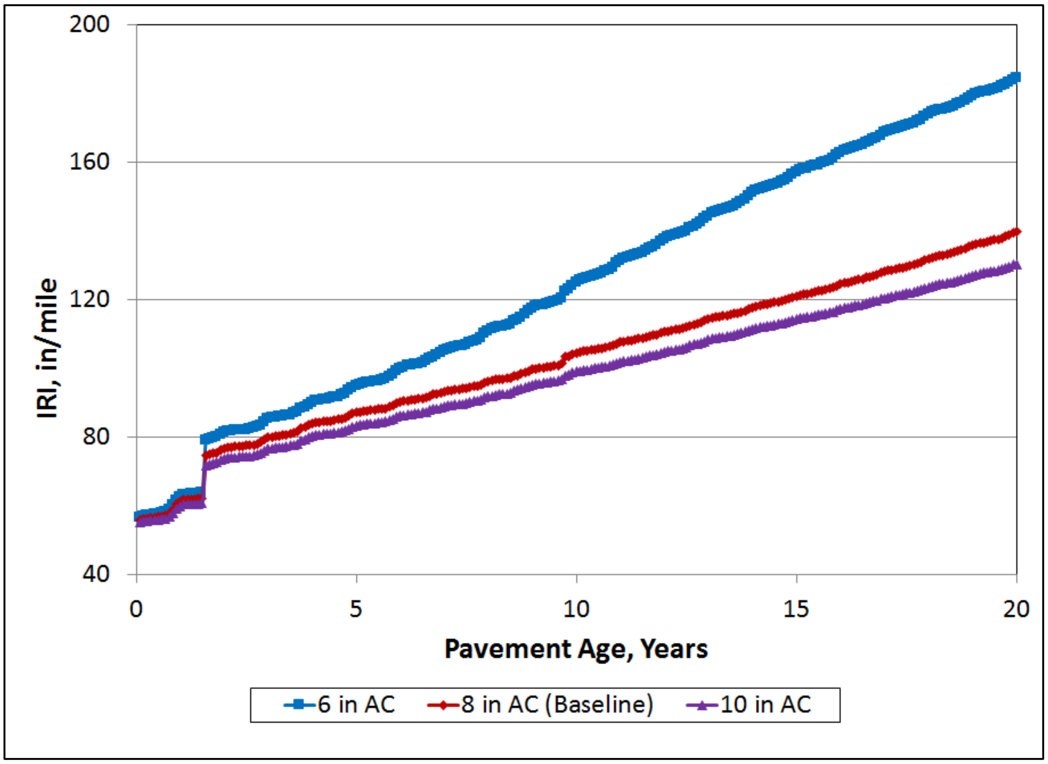


Figure 6.30 Sensitivity of HMA IRI to AC Thickness

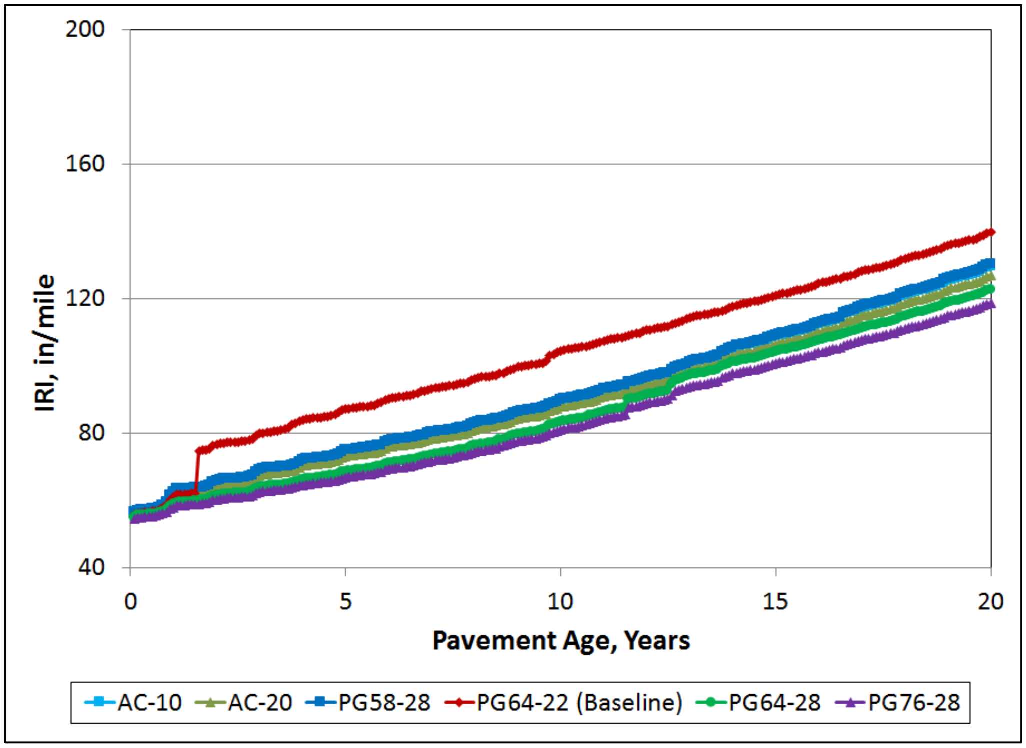


Figure 6.31 Sensitivity of HMA IRI to Asphalt Binder Grade

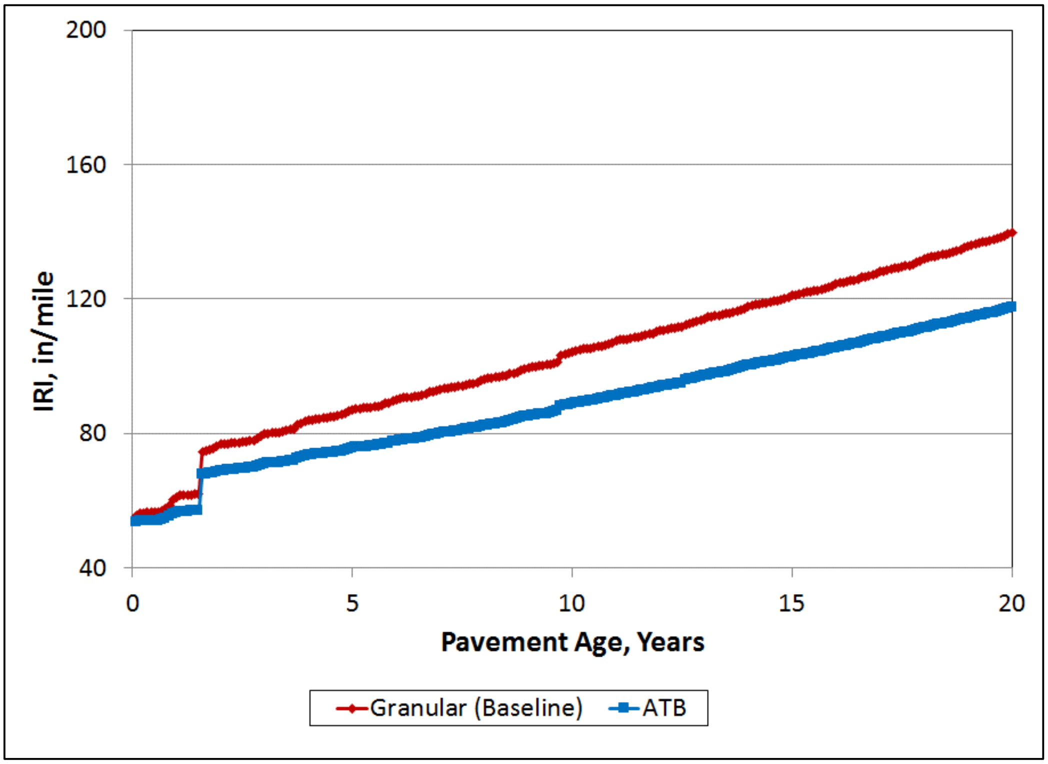


Figure 6.32 Sensitivity of HMA IRI to Base Thickness

6.10 HMA Thickness with ABC

As a minimum, the designer should include 4 inches of ABC for any thickness of HMA when the design truck traffic is less than 1,000 trucks per day. Six inches of ABC should be used for any thickness of HMA when the design truck traffic is greater than 1,000 trucks per day.

6.11 Required Minimum Thickness of Pavement Layer

Compaction of a hot mix asphalt pavement during its construction is the single most important factor affecting the ultimate performance of the pavement. Achieving adequate compaction increases pavement performance by decreasing rutting, reducing damage due to moisture and oxidation, and increasing the stability of the mix. Factors affecting the cooling rate of the mat include the layer thickness, the temperature of the mix when placed, ambient temperature, temperature of the base, and wind conditions. Layer thickness is the single most important variable in the cooling rate of an asphalt mat, especially for thin layers. This is especially true in cool weather because thin layers of an asphalt mat have less capacity to retain heat than thicker lifts of pavement. The thicker layers of an asphalt mat help to maintain the temperature at a workable level, thus increasing the time available for compaction. Because of the increased difficulty in achieving density and the importance of achieving compaction, a minimum layer thickness for construction of HMA pavement is two inches. A designer of special mixes, such as stone matrix asphalt or thin lift HMA should look at minimum thickness requirements of the particular product. The minimum thickness of these special mixes is likely to be a dimension other than two inches.

6.12 Asphalt Materials Selection

6.12.1 Aggregate Gradation

Definitions of Aggregate Size:

- **Nominal Maximum Aggregate Size (NMAS):** The size of aggregate of the smallest sieve opening through which the entire amount of aggregate is permitted to pass.
    - **Note:** For Item 403 - HMA and SMA, the Nominal Maximum Size is defined as one sieve size larger than the first sieve to retain more than ten percent of the aggregate.

- **Maximum Aggregate Size** is defined as one size larger than nominal maximum size. The flexible pavement usually consists of ¾ inch nominal maximum aggregate size (NMAS) in the lower layers, with a hot mix asphalt (HMA) Grading S. The top surface layer, should be either stone matrix asphalt (SMA) or a Grading SX. SMA mixes are often used in areas expected to experience extreme traffic loading. When low to high traffic loads are expected, a ½ inch NMAS, Grading SX should be used.

CDOT uses the No. 30 sieve as one of the job-mix formula tolerance sieves. **Table 6.4 Master Range Table for Stone Mix Asphalt**. is based (with some exceptions) on NCHRP No. 4 and $^3/_8$ inch and AASHTO ½ inch and ¾ inch SMA gradations ranges, where the No. 30 sieve range is included in the ½ inch and ¾ inch gradations.

SMA Gradation Nomenclature Example:
The ¾ inch (19.0 mm) gradation is named the ¾ inch Nominal Maximum Aggregate Size gradation because the first sieve that retains more than 10 percent is the ½ inch sieve, and the next sieve larger is the ¾ inch sieve, refer to **Table 6.4 Master Range Table for Stone Mix Asphalt**.

A CDOT study (1) found less thermal segregation in the top lift when Grading SX mixes were used. HMA Grading SX can also be used where layers are very thin or where the pavement must taper into an existing pavement. A study from Auburn University (2) found little difference in the stability or rutting of ¾ inch and ½ inch NMAS mixes. CDOT cost data for 2005 showed a slight increase in the cost per ton of Grading SX mixes as compared to Grading S mixes with the same bid quantities.

HMA with a 1-inch NMAS, Grading SG, should not be used in the surface layer. Although Grading SG mixes have been used in specialized situations, they are not currently used or accepted on a regular basis for pavement mixes. CDOT has found that the production and placement of Grading SG mixes are prone to segregation and the use should be discouraged.

Table 6.4 Master Range Table for Stone Matrix Asphalt

Sieve Size	Percent by Weight Passing Square Mesh Sieves			
	#4 (4.75 mm) Nominal Maximum	3/8" (9.5 mm) Nominal Maximum	1/2" (12.5 mm) Nominal Maximum	3/4" (19.0 mm) Nominal Maximum
1 " (25 mm)	-	-	-	100
¾" (19.0 mm)	-	-	100	90-100
½" (12.5 mm)	100	100	90-100	50-88
3/8" (9.5 mm)	100	90-100	50-80	25-60
#4 (4.75 mm)	90-100	26-60	20-35	20-28
#8 (2.36 mm)	28-65	20-28	16-24	16-24
#16 (1.18mm)	22-36	-	-	-
#30 (600 µm)	18-28	12-18	12-18	12-18
#50 (300 µm)	15-22	10-15	-	-
#100 (150 µm)	-	-	-	-
#200 (75 µm)	12-15	8-12	8-11	8-11

For structural overlays, the minimum allowed layer thickness will be 2 inches. For functional overlays used in preventive maintenance or other treatments, thinner lifts are allowed.

Table 6.5 HMA Grading Size and Location Application and **Table 6.6 HMA Grading Size and Layer Thickness** gives guidance for mix selection and recommended layer thicknesses for various layers and nominal maximum aggregate sizes.

Table 6.5 HMA Grading Size and Location Application

CDOT HMA Grade	Nominal Maximum Aggregate Size (NMAS)	Application
SF	No. 4 sieve	Leveling course, rut filling, scratch course, etc.
ST	3/8 inch	Thin lifts and patching
SX	½ inch	Top layer (preferred)
S	¾ inch	Top layer, layers below the surface, patching
SG	1 inch	Layers below the surface, deep patching

Table 6.6 HMA Grading Size and Layer Thickness

CDOT HMA Grade	Nominal Maximum Aggregate Size (NMAS)	Structural/Overlay Layer Thickness (inches)	
		Minimum	Maximum
SX	½ inch	2.00	3.00
S	¾ inch	2.25	3.50
SG	1 inch	3.00	4.00
		Functional Overlay Layer Thickness (Inches)	
SF	No. 4 sieve	0.75[1]	1.50
ST	3/8 inch	1.125	2.50

Note: [1] Layers of SF mixes may go below 1 inch as needed to taper thin lift to site conditioning (i.e. rut filling).

6.12.2 Selection of SuperPave™ Gyratory Design

To choose the appropriate number of revolutions of a SuperPave™ gyratory asphalt mix design on a particular project, determining the design 18k ESALs and the high temperature environment for the project is necessary. The following steps should be followed to determine the proper SuperPave™ gyratory design revolutions for a given project:

Step 1. Determine 18k ESALs: In order to obtain the correct SuperPave™ gyratory compaction effort (revolutions), the 18k ESALs must be a 20-year cumulative 18k ESAL of the design lane in one direction. The compaction effort simulates the construction compaction roller to obtain the correct voids properties to resist the intended traffic in the design lane. The department's traffic analysis unit of the Division of Transportation Development (DTD) automatically provides an ESAL calculator. One must use a 20-year design, appropriate number of lanes, and a specified flexible pavement. Even a 10-year asphalt overlay must use a 20-year cumulative 18k ESAL number for the design lane.

Step 2. Reliability for the 7-Day Average Maximum Air Temperature: The next decision is to determine the type of project being designed. For new construction or reconstruction, asphalt cement with 98 percent reliability for low and high temperature properties is recommended. For overlays, asphalt cement with 98 percent reliability for high temperature properties (rutting resistance) and 50 percent reliability for low temperature properties (cracking resistance) is recommended. Asphalt cements with lower than 98 percent reliability against rut resistance should not be specified. In the SuperPave™ system, anything between 50 percent and 98 percent reliability is considered 50 percent reliability for the purpose of binder selection. The low temperatures are specified at a lower reliability for overlays because of reflection cracking.

Step 3. Determine Weather Data for the Project: Obtain the highest 7-day average maximum air temperature, based on weather data in the project area from the computer program LTPPBind 3.1 (beta). Refer to **Section 6.12.3 Binder Selection** for a further explanation of LTPPBind 3.1 (beta). From the appropriate high temperature, find the environmental category for the project from **Table 6.7 Environmental Categories**. The Environmental Categories are from CDOT Pavement Management Program's Environmental Zones. The Environmental Zones (Categories) are one of four pavement groupings used to group pavements into families that have similar characteristics.

Table 6.7 Environmental Categories

Highest 7-Day Average Air Temperature	High Temperature Category
> 97°F (> 36°C)	Hot (southeast and west)
> 88° to 97°F (> 31° to 36°C)	Moderate (Denver, plains and west)
81° to 88°F (27° to 31°C)	Cool (mountains)
< 81°F (< 27°C)	Very Cool (high mountains)

Step 4. Selection of the Number of Design Gyrations (N_{DES}): Select the N_{DES} from **Table 6.8 Recommended SuperPave™ Gyratory Design Revolution (NDES)**. For example, Table 6.7 shows that for 5,000,000 18k ESALs and a high temperature category of "Cool", the design revolutions should be 75.

Table 6.8 Recommended SuperPave™ Gyratory Design Revolution (N_{DES})

CDOT Pavement Management System Traffic Classification (20 Year Design ESAL)	20 Year Total 18k ESAL in the Design Lane	High Temperature Category			
		Very Cool	Cool	Moderate	Hot
Low	< 100,000	50	50	50	50
	100,000 to < 300,000	50	75	75	75
Medium	300,000 to < 1,000,000	75	75	75	75
	1,000,000 to < 3,000,000	75	75	75	100
High	3,000,000 to < 10,000,000	75	75	100	100
Very High	10,000,000 to < 30,000,000	---	---	100	---
Very Very High	≥ 30,000,000	---	---	125	---

Note: Based on *Standard Practice for SuperPave™ Volumetric Design for Hot-Mix Asphalt (HMA)*, AASHTO Designation R 35-04.

6.12.3 Binder Selection

Performance graded (PG) binders have two numbers in their designation, such as PG 58-34. Both numbers describe the pavement temperatures in degrees Celsius at which the pavement must perform. The first number (58 in the example) is the high temperature standard grade for the pavement, and the second number (minus 34 in the example) is the low temperature standard grade. PG 64-28 (rubberized) or PG 76-28 (polymerized) or bituminous mixtures should only be placed directly on an existing pavement or milled surface that does not show signs of stripping or severe raveling. Cores should be taken to determine if stripping is present. Because of a limited number of tanks, Colorado local suppliers only have the capacity to supply a limited number of asphalt cement grades. **Table 6.9 Available Asphalt Cement Grades in Colorado** shows available grades that maybe used and/or available on CDOT projects.

Table 6.9 Available Asphalt Cement Grades in Colorado

Polymer Modified	Unmodified
PG 76-28 PG 70-28 PG 64-28 PG 58-34	PG 64-22 PG 58-28
Note: The Region Materials Engineer may select a different gyratory design revolution for the lower HMA lifts.	

LTPPBind 3.1 (beta) is a working version, dated September 15, 2005. Beta only means it is going through the 508-compliance process for the visually disabled users as required by the Federal Government. The computer program may be obtained from the following web address:
http://www.fhwa.dot.gov/pavement/ltpp/ltppbind.cfm

The program allows the user to select the asphalt binder grade for the appropriate project site conditions. In the *Preferences* under the *File* menu, use 12.5mm (½ inch) for the CDOT target rut depth default value. The computer program has a help menu to assist the user and supporting technical information regarding the computation of design temperatures required for the selection of the asphalt binder grade as provided in the *Climatic Data* and *Algorithms* sections. The algorithms are broken down under four subsections. Each algorithm equation is shown and briefly explained for high temperature, low temperature, PG with depth, and PG grade bumping.

- **High Temperature:** The high temperature is based on a rutting damage model. The LTPP high temperature model was not used in this version since it provided very similar results to the SHRP Model at 98 percent reliability. Initially, the user must select a preference for a target rut depth, but they have the option to change the target rut depth. The default is 12.5 mm (½ inches).

- **Low Temperature:** The low temperature is based on LTPP climatic data using air temperature, latitude, and depth to surface.

- **PG with Depth:** LTPP pavement temperature algorithms were used to adjust the PG for a depth into the pavement. The LTPP algorithms are empirical models developed from seasonal monitoring data.

- **PG Grade Bumping:** PG grade bumping was based on the rutting damage concept for high temperature adjustments. Adjustments were developed as the difference between PG for standard traffic conditions (ESAL of 3 million and high speed) and site conditions. 187 sites throughout the U.S. for five target rut depths were analyzed. The PG adjustments were then averaged by various ESAL ranges, traffic speeds, and Base PG.

The following steps should be followed to determine the proper SuperPave™ asphalt cement grade for a given project:

Step 1. Determine Proper Reliability to Satisfy Pavement Temperature Property Requirements: The first step is to determine what type of project is being designed.

- For new construction or reconstruction, asphalt cement with 98 percent reliability for both low and high pavement temperature properties is recommended.

- For overlays, asphalt cement with 98 percent reliability for high pavement temperature properties (rutting resistance) and 50 percent reliability for low pavement temperature properties (cracking resistance) are recommended.

- Asphalt cements with lower than 98 percent reliability against rut resistance should not be specified.

- In the SuperPave™ system, anything between 50 and 98 percent reliability is considered 50 percent reliability for the purpose of binder selection.

- The low pavement temperatures are specified at a lower reliability for overlays because of reflection cracking.

- Refer to **Figure 6.33 PG Binder Grades** for a graphical representation of reliability.

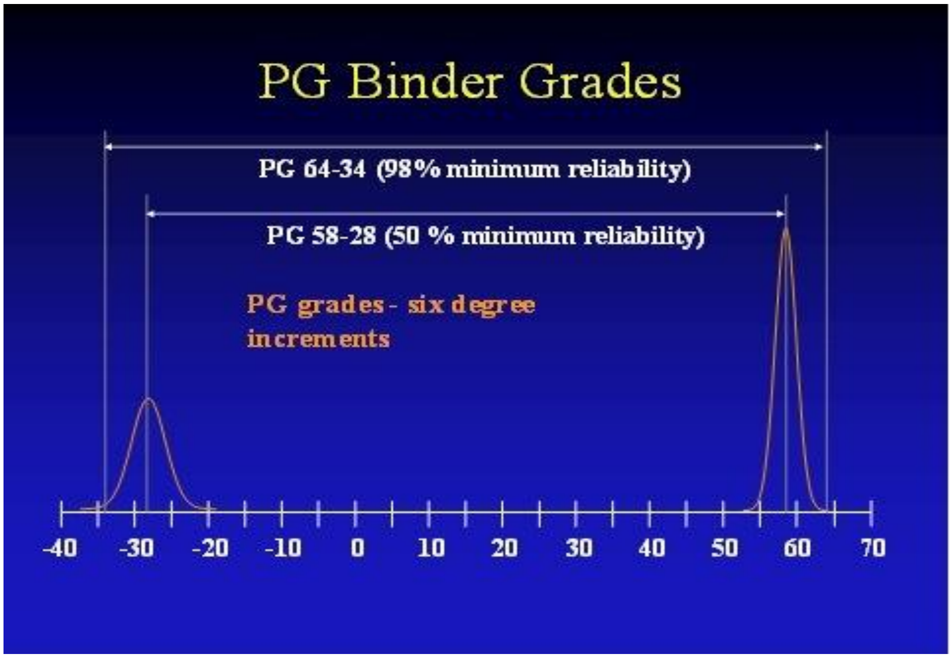


Figure 6.33 PG Binder Grades

Step 2. Determine Weather Data for the Project: Obtain the SuperPave™ recommended asphalt cement grade, based on weather data and traffic in the project area. Recommendations on 98 percent reliability high and low pavement temperature weather stations are found in **Figure 6.34 Colorado 98 Percent Reliability LTPP High Pavement Temperature Weather Station Models** and **Figure 6.35 Colorado 98 Percent Reliability LTPP Low Pavement Temperature Weather Station Models**, neither of which accounts for grade bumping. The program also calculates the reliability of various asphalt cements for a given location. This source will yield the 98 and 50 percent reliability asphalt cement for a project area with a free flowing traffic condition, which is described in Step 3. For example, when the recommendations call for a PG 58-22 for a given project, due to the available binder grades in Colorado, a PG 64-22 would be specified. This selection provides for rut resistance while preserving the same level of resistance to cracking. Because of the danger of rutting, in no case should the recommended high temperature requirements be lowered based on availability. Each RME has a copy of this program.

Step 3. Select Location of Roadway: Place the cross hair on the location of area of interest in the weather data program LTPP Bind. The program selects five weather stations surrounding the area of interest. The designer has the option to use any number of weather stations representative of the climate at the area of interest.

Step 4. Adjust HMA Performance Grade Binder to Meet Layer Depth, Traffic Flow and Loading Requirements: SuperPave™ high temperature reliability factors are based on historical weather data and algorithms to predict pavement temperature. At a depth layer of 1 inch or more below the surface, high temperature recommendations are changed because of the depth and temperatures at that depth.

For pavements with multiple layers a lesser grade may be specified for lower layers based on the amount of material needed and other economical design decisions. In many cases, the requirements for lower layers might be obtained with an unmodified or more economical grade of asphalt cement. It is recommended that at least 10,000 tons of mix in the lower layer is needed before a separate asphalt cement is specified for the lower layer.

Adjustments can be made to the base high temperature binder through the '*PG Binder Selection'* screen. Adjustments to reliability, depth of layer, traffic loading, and traffic speed (fast and slow) will be required. These adjustments are called grade bumping. Additional grade bumping may be performed for stop and go traffic characteristics such as intersections. This extra grade bump may be applied, but is suggested the designer have prior regional experience on doing such.

6.12.3.1 Example

Example: A new roadway project will be constructed near Sugarloaf Reservoir. It will have two lanes per direction and a traffic characteristic of slow moving because it is a winding mountain road. Find the appropriate binder grade. N_{DES} for the surface layer is obtained in the same manner as the previous example and has a design revolution of 75.

Step 1. Determine 18k ESAL: Design Lane ESALs = 4,504,504 from DTD web site (20 year 18k ESAL in the design lane).

Step 2. Use LTPP Software Database: Use LTPPBind software database to obtain the data from the nearest weather station, Sugarloaf Reservoir. Appropriate weather stations can be determined from information on state, county, coordinates, location, and/or station ID. **Figure 6.36 LTPP Interface Form for Weather Station Selection (Version 3.1)** is where the cross hair is placed for the new roadway project. **Figure 6.37 LTPP Weather Station Output Data (Version 3.1)** shows the data at the weather station Sugarloaf Reservoir.

Step 3. Select the Desired Weather Stations: The LTPPBind software gives the option to select the weather stations that provide the best weather data at the project location (see the upper table in **Figure 6.38 LTPP PG Binder Selection at 98 Percent Reliability**). Check the first three weather stations. Uncheck the two weather stations furthest from the project, these stations are too far from the site and not representative of site conditions.

Step 4. Select the Temperature Adjustments: Because this is a principal arterial and a new construction project, 98 percent reliability is chosen with a layer depth of zero (0) for the surface layer (see **Figure 6.38 LTPP PG Binder Selection at 98 Percent Reliability)**.

Step 5. Select the Traffic Adjustments for High Temperature: Select the appropriate traffic loading and traffic speed. The design lane ESALs are 4,504,504 and the traffic speed is slow. Grade bumping is automatic and is demonstrated by toggling in appropriate cells.

The following data summarized in **Table 6.10 SuperPave^{TM} Weather Data Summary** are obtained from Steps 1 through 5.

Step 6. Select Final Binder: Table 6.8 Available Asphalt Cement Grades in Colorado lists the binder grades available in Colorado. A PG 58-28 (unmodified) is available, but it does not meet the low temperature requirement. The lowest temperature binders available in Colorado can meet is -34° C. This is available in PG 58-34 (polymer modified). Therefore, at 98 percent reliability use PG 58-34.

Step 7. Find the Temperature that Falls into the Environmental Category: Use **Table 6.10 Environmental Categories (restated)** to obtain the highest 7-day average air temperature, 24.3°C. **Table 6.11 Environmental Categories (restated)** shows the temperature falls into the category 'Very Cool' (high mountains).

Step 8 Select the Gyratory Design Revolution (N_{DES}): Table 6.11 Recommended SuperPave^{TM} Gyratory Design Revolution (N_{DES}) shows at 4,504,504 18k ESAL and a high temperature category of "Very Cool" the design revolutions should be 75.

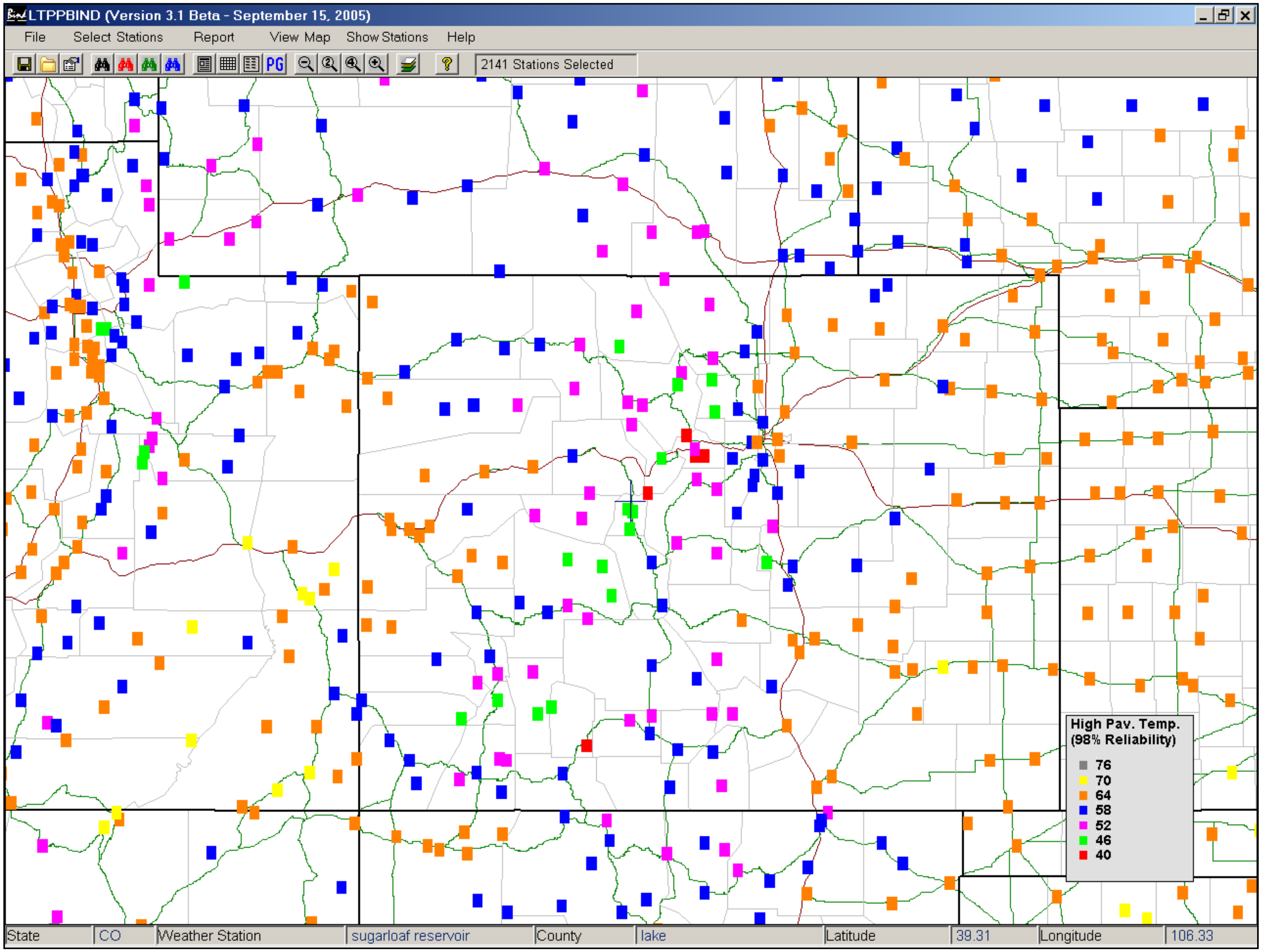


Figure 6.34 Colorado 98 Percent Reliability LTPP High Pavement Temperature Weather Station Models

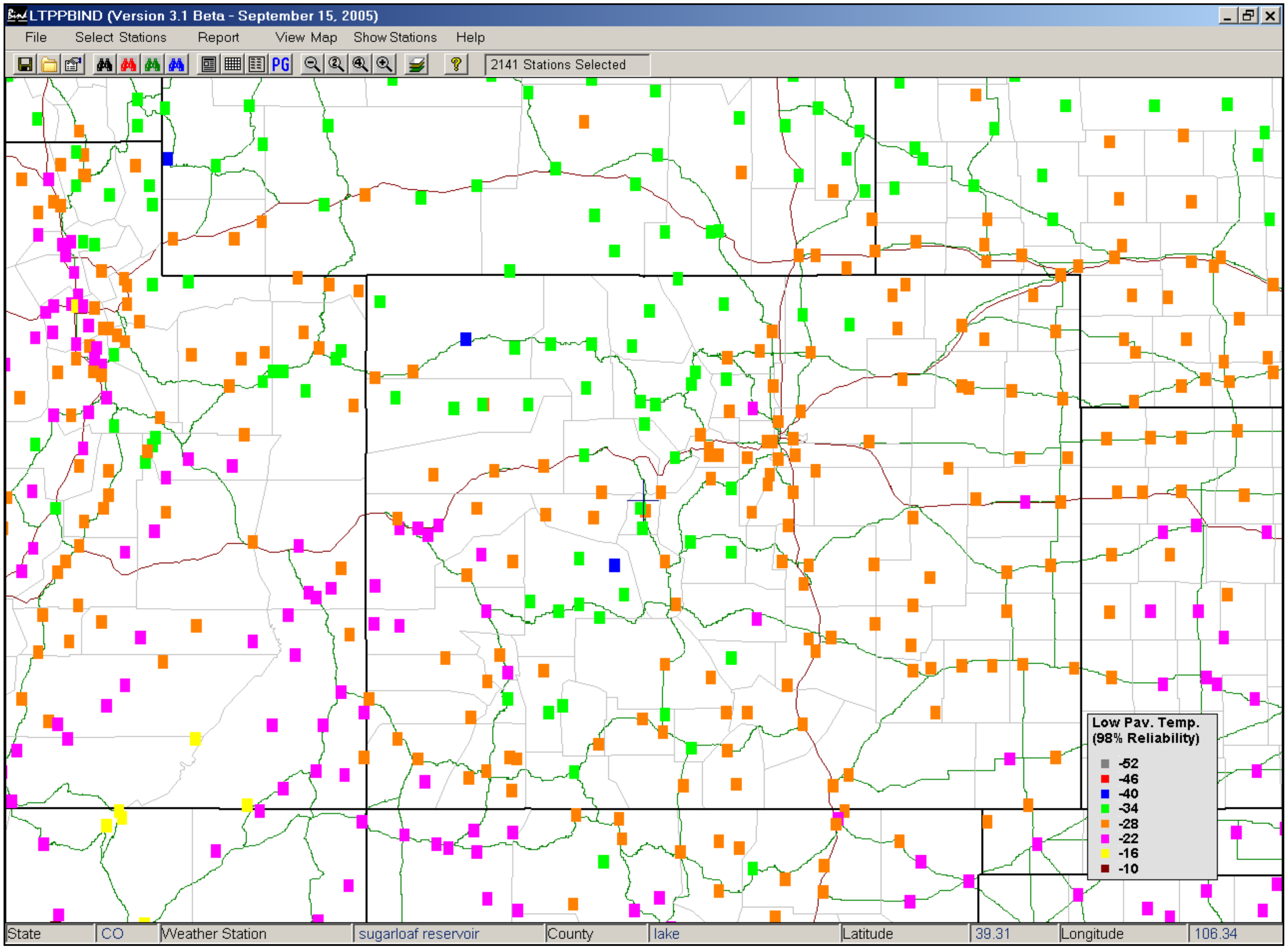


Figure 6.35 Colorado 98 Percent Reliability LTPP Low Pavement Temperature Weather Station Models

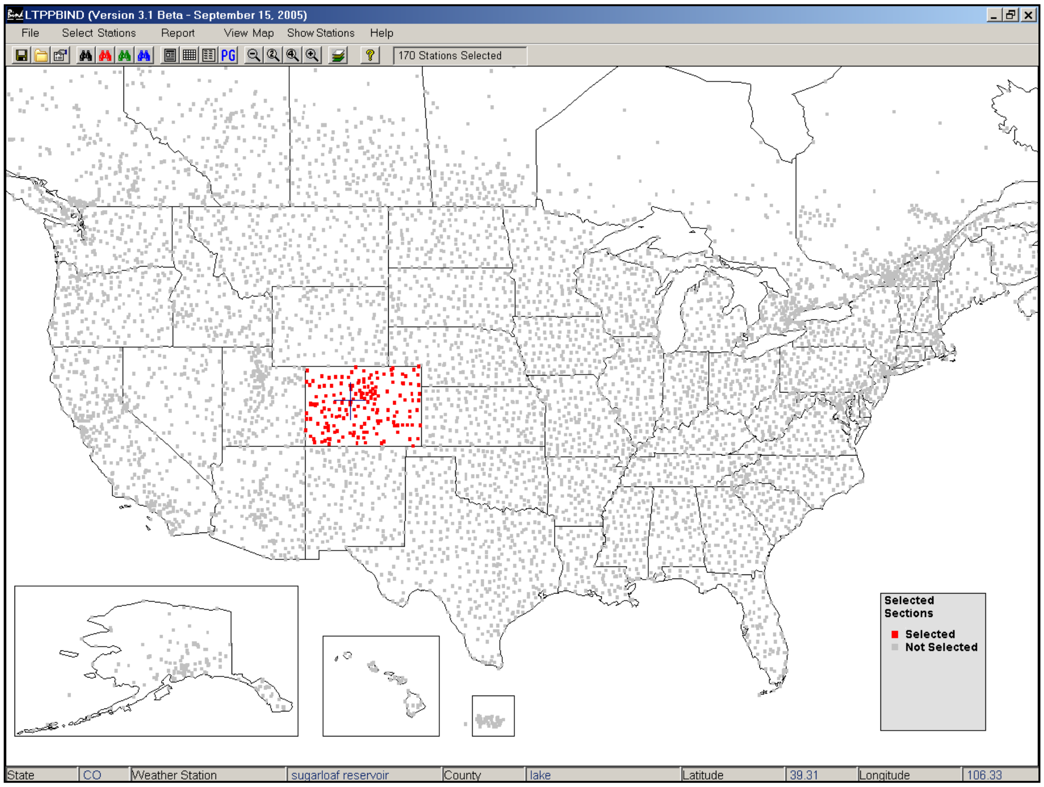
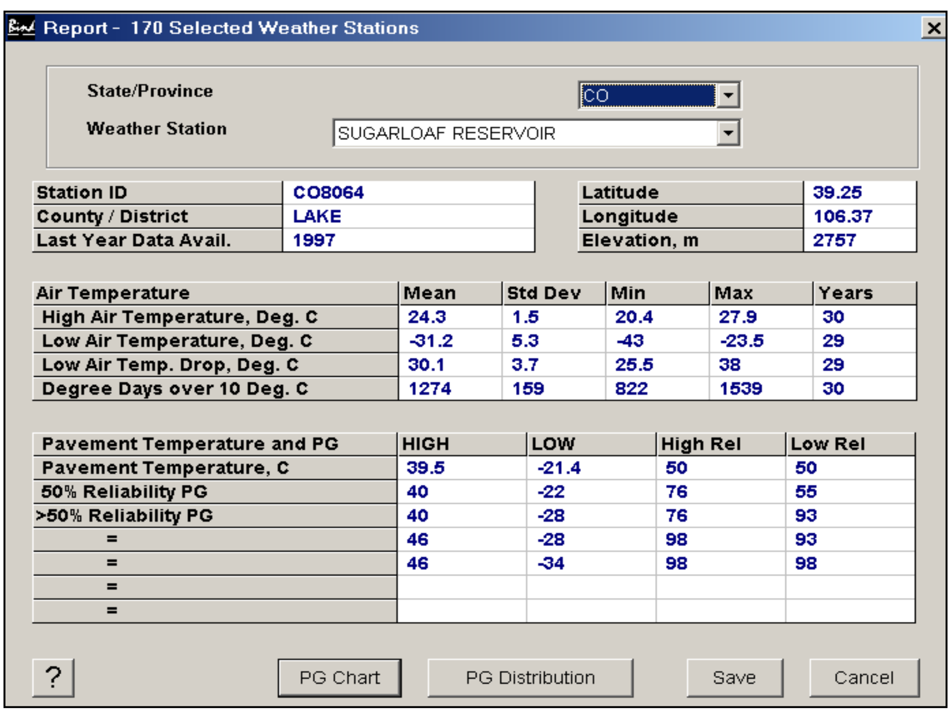


Figure 6.36 LTPP Interface Form for Weather Station Selection (Version 3.1)


Report - 170 Selected Weather Stations

State/Province: CO

Weather Station: SUGARLOAF RESERVOIR

Station ID	CO8064		Latitude	39.25
County / District	LAKE		Longitude	106.37
Last Year Data Avail.	1997		Elevation, m	2757

Air Temperature	Mean	Std Dev	Min	Max	Years
High Air Temperature, Deg. C	24.3	1.5	20.4	27.9	30
Low Air Temperature, Deg. C	-31.2	5.3	-43	-23.5	29
Low Air Temp. Drop, Deg. C	30.1	3.7	25.5	38	29
Degree Days over 10 Deg. C	1274	159	822	1539	30

Pavement Temperature and PG	HIGH	LOW	High Rel	Low Rel
Pavement Temperature, C	39.5	-21.4	50	50
50% Reliability PG	40	-22	76	55
>50% Reliability PG	40	-28	76	93
=	46	-28	98	93
=	46	-34	98	98
=				
=				

? | PG Chart | PG Distribution | Save | Cancel

Figure 6.37 LTPP Weather Station Output Data (Version 3.1)

Table 6.10 SuperPave™ Weather Data Summary

98 Percent Reliability		
Depth of Layer	0 mm	
Traffic Loading and Speed Adjustment	10.3°C (slow)	
PG Binder Grade	52	-34

Table 6.11 Environmental Categories (restated)

Highest 7-Day Average Air Temperature	High Temperature Category
> 97°F (> 36°C)	Hot (southeast and west)
> 88° to 97°F (> 31° to 36°C)	Moderate (Denver, plains and west)
81° to 88°F (27° to 31°C)	Cool (mountains)
< 81°F (< 27°C)	Very Cool (high mountains)

Table 6.12 Recommended SuperPave™ Gyratory Design Revolution (N_{DES}) (restated)

CDOT Pavement Management System Traffic Classification (20 Year Design ESAL)	20 Year Total 18k ESAL in the Design Lane	High Temperature Category			
		Very Cool	Cool	Moderate	Hot
Low	< 100,000	50	50	50	50
	100,000 to < 300,000	50	75	75	75
Medium	300,000 to < 1,000,000	75	75	75	75
	1,000,000 to < 3,000,000	75	75	75	100
High	3,000,000 to < 10,000,000	75	75	100	100
Very High	10,000,000 to < 30,000,000	---	---	100	---
Very Very High	≥ 30,000,000	---	---	125	---

Note: Based on *Standard Practice for SuperPave™ Volumetric Design for Hot-Mix Asphalt (HMA)*, AASHTO Designation R 35-04.

6.12.4 Asphalt Binder Characterization for M-E Design

For flexible pavement design using M-E Design, the viscosity of the asphalt binder is a critical input parameter to incorporate the viscoelastic response (i.e. time-temperature dependency) of asphalt concrete mixtures. The asphalt binder viscosity is used in the calculations of dynamic modulus values of asphalt mixtures for aged and unaged conditions. The key input parameters that define the viscosity temperature relationship are the slope (A) and intercept (VTS) resulting from a regression of the asphalt binder viscosity values measured or estimated at different temperatures.

Laboratory testing of asphalt binders is required to develop viscosity temperature relationships at the Level 1 input hierarchy. For performance grade binders, the asphalt binder viscosity values can be estimated from the dynamic shear rheometer test data conducted in accordance with AASHTO T 315, Determining the Rheological of Asphalt Binder Using a Dynamic Shear Rheometer (DSR). Alternatively, for conventional grade binders (i.e. penetration grade or viscosity grade), the asphalt binder viscosity values can be obtained from a series of conventional tests, including absolute and kinematic viscosities, specific gravity, softening point, and penetrations. At the hierarchical input Level 3, the default values of A-VTS parameters included in M-E Design are based on the asphalt binder grade selection.

For flexible pavement rehabilitation designs, the age-hardened binder properties can be determined using asphalt binder extracted from field cores of asphalt pavement layers that will remain in place after rehabilitation. For projects where asphalt is not extracted, historical information and data may be used. **Table 6.13 Recommended Sources of Inputs for Asphalt Binder Characterization** presents recommended sources for asphalt binder characterization at different hierarchical input levels. Refer to the *AASHTO Intrim MEPDG Manual of Practice* and MEPDG Documentation for more information.

Table 6.13 Recommended Sources of Inputs for Asphalt Binder Characterization

<table>
<tr><th rowspan="2">Materials Category</th><th rowspan="2">Measured Property</th><th rowspan="2">Recommended Test Protocol</th><th colspan="3">Hierarchical Input Level</th></tr>
<tr><th>3</th><th>2</th><th>1</th></tr>
<tr><td rowspan="9">Asphalt Binder</td><td>Asphalt binder complex shear modulus (G*) and phase angle (δ); at 3 test temperatures, or</td><td>AASHTO T 315</td><td rowspan="5"></td><td rowspan="5">✓</td><td rowspan="5">✓</td></tr>
<tr><td>Conventional binder test data: Penetration, or</td><td>AASHTO T 49</td></tr>
<tr><td>Ring and ball softening point
Absolute viscosity
Kinematic viscosity
Specific gravity, or</td><td>AASHTO T 53
AASHTO T 202
AASHTO T 201
AASHTO T 228</td></tr>
<tr><td>Brookfield viscosity</td><td>AASHTO T 316</td></tr>
<tr><td>Asphalt binder grade: PG grade, or</td><td>AASHTO M 320</td><td rowspan="3">✓</td><td rowspan="3"></td><td rowspan="3"></td></tr>
<tr><td>Viscosity grade, or</td><td>AASHTO M 226</td></tr>
<tr><td>Penetration grade</td><td>AASHTO M 20</td></tr>
<tr><td>Rolling thin film oven aging</td><td>AASHTO T 315</td><td></td><td>✓</td><td>✓</td></tr>
</table>

6.13 Asphalt Mix Design Criteria

6.13.1 Fractured Face Criteria

CDOT's aggregate fractured face criteria requires the aggregate retained on the No. 4 sieve must have at least two mechanically induced fractured faces (2) (see **Table 6.14 Fracture Face Criteria**).

Table 6.14 Fractured Face Criteria

Percent Fractured Faces of 20 Year 18k ESAL in Design Lane	SF	ST	SX	S	SG	SMA
Non-Interstate Highways or Pavements with < 10,000,000 Total 18K ESALs	60%	60%	60%	60%	90%	90%
Interstate Highways or Pavements with > 10,000,000 Total 18K ESALs	70%	70%	70%	70%	90%	90%

6.13.2 Air Void Criteria

A design air void range of 3.5 to 4.5 percent with a target of 4.0 percent will be used on all SX, S, SG, and ST mixes. A design air void range of 4.0 to 5.0 percent with a target of 4.5 percent will be used on all SF Mixes. Refer to **Table 6.15 Minimum VMA Requirements** for design air voids and minimum VMA requirements and criteria for voids at N_{DES}. The air void criteria will be applied to the approved design mix. The nominal maximum size is defined as one size larger than the first sieve to retain more that 10 percent. The designer should interpolate specified VMA values for design air voids between those listed in the table. All mix designs shall be run with a gyratory compactor angle of 1.25 degrees. CDOT Form #43 will establish construction targets for asphalt cement and all mix properties at air voids up to 1.0 percent below the mix design optimum. The designer should extrapolate VMA values for production (CDOT Form 43) air voids beyond those listed in **Table 6.15 Minimum VMA Requirements.**

Table 6.15 Minimum VMA Requirements

Nominal Maximum Size[1] mm (in)	Design Air Voids [2, 3]			
	3.5%	4.0%	4.5%	5.0%
37.5 (1 1/2")	11.6	11.7	11.8	N/A
25.0 (1")	12.6	12.7	12.8	N/A
19.0 (3/4")	13.6	13.7	13.8	N/A
12.5 (1/2")	14.6	14.7	14.8	N/A
9.5 (3/8")	15.6	15.7	15.8	16.9

Note:
[1] The nominal maximum size defined as one size larger than the first sieve to retain more than 10%.
[2] Interpolate specified VMA values for design air voids between those listed.
[3] Extrapolate specified VMA values for production air voids between those listed.

6.13.3 Criteria for Stability

Criteria for stability and voids filled with asphalt (VFA) are shown in **Table 6.16 Criteria for Stability and Voids Filled with Asphalt (VFA)**.

Table 6.16 Criteria for Stability and Voids Filled with Asphalt (VFA)

SuperPave™ Gyratory Revolutions (N_{DES})	Hveem Minimum Stability*	VFA (%)
125	30	65-75
100	30	65-75
75	28	65-80
50	**	70-80

Note: 1-inch mix (CDOT Grade SG) has no stability requirements.
* Hveem Stability criteria for mix design approval and for field verification.
** Hveem Stability is not a criterion for mixes with a N_{DES} of 50.

6.13.4 Moisture Damage Criteria

Moisture damage criteria are shown in **Table 6.17 Moisture Damage Criteria.**

Table 6.17 Moisture Damage Criteria

Characteristic	Value
Minimum dry split tensile strength, (psi)	30
Minimum tensile strength ratio, CP-L 5109, (%)	80
Minimum tensile strength ratio, CP-L 5109, SMA, (%)	70

6.14 Effective Binder Content (By Volume)

Effective binder content (P_{be}) is the amount of binder not absorbed by the aggregate, i.e. the amount of binder that effectively forms a bonding film on the aggregate surfaces. Effective binder content is what the service performance is based on and is calculated based on the aggregate bulk specific gravity (G_{sb}) and the aggregate effective specific gravity (G_{se}). The higher the aggregate absorption, the greater the difference between G_{se} and G_{sb}. The effective binder content by volume is the effective binder content (P_{be}) times the ratio of the bulk specific gravity of the mix (G_{mm}) and the specific gravity of the binder (G_b). The formula is:

$$\mathbf{P_{be}\ (by\ volume) = P_{be} * (G_{mm} / G_b)}$$

Where

P_{be} = effective asphalt content, percent by total weight of mixture
G_{mm} = bulk specific gravity of the mix
G_b = specific gravity of asphalt (usually 1.010)

P_{be} is determined as follows:

$$\mathbf{P_{be} = P_b - (P_{ba}/100) * P_s}$$

Where

P_b = asphalt, percent by total weight of mixture
P_{ba} = absorbed asphalt, percent by total weight of aggregate
P_s = aggregate, percent by total weight of mixture

P_{ba} is determined as follows:

$$\mathbf{P_{ba} = 100\ ((G_{se} - G_{sb}) / (G_{sb} * G_{se})) * G_b}$$

Where

P_{ba} = absorbed asphalt, percent by total weight of aggregate
G_{se} = effective specific gravity of aggregate
G_{sb} = bulk specific gravity of aggregate

6.15 Rumble Strips

When Rumble Strips are installed, they shall be of the style and location as shown on CDOT's *Standard Plans, M & S Standards*, July 2012 Plan Sheet No. M-614-1, Rumble Strips.

References

1. Gilbert, K., *Thermal Segregation*, 2005, CDOT Report No. CDOT-DTD-R-2005-16. http://www.coloradodot.info/programs/research/pdfs/2005/thermalsegregation.pdf..

2. Brown, E.R. and Bassett, C.E. *Effects of Maximum Aggregate Size on Rutting Potential and Other Properties of Asphalt-Aggregate Mixtures 1990*. Transportation Research Record 1259, TRB, National Research Council, Washington, D.C., 1990, pp. 107-119.

3. *HMA Pavement Mix Type Selection Guide*, FHWA Information Series 128, Federal Highway Administration, 400 7th Street S.W., Washington, DC 20590 and National Asphalt Pavement Association (NAPA), 5100 Forbes Blvd, Lanham, MD 20706, January 2001.

4. *Guide for Mechanistic-Empirical Design, Final Report*, NCHRP Project 1-37A, National Cooperative Highway Research Program, Transportation Research Board, National Research Council, Submitted by ARA, INC., ERES Consultants Division, Champaign, IL, March 2004.

5. Locander, Robert, *Analysis of Using Reclaimed Asphalt Pavement (RAP) as a Base Material*, Final Report, CDOT Report No. CDOT-2009-5, Colorado Department of Transportation, February 2009.

6. *Perpetual Pavements - A Synthesis.* Asphalt Pavement Alliance, 5100 Forbes Boulevard, Lanham, MD 20706 : Asphalt Pavement Alliance, 1/2002. Synthesis.

7. *Perpetual Bituminous Pavements.* Transportation Research Board (TRB). 2101 Constitution Avenue, NW, Washington, DC 20418 : Transportation Research Board, National Research Center, December 2001. Research Circular. TRB Committee on General Issues in Asphalt Technology (A2D05). Number 503, ISSN 0097-8515.

8. *AASHTO Mechanistic-Empirical Pavement Design Guide, A Manual of Practice, Interim Edition, July 2008*, American Association of State Highway and Transportation Officials, Washington, DC, 2008.