ENHANCEMENTS TO THE MECHANISTIC-EMPIRICAL PAVEMENT DESIGN GUIDE:

A MANUAL OF PRACTICE, JULY 2008 INTERIM EDITION

ADDENDUM NUMBER: **FY2015.3**

ADDENDUM TITLE: **LAYER DEPENDENT PLASTIC DEFORMATION COEFFICIENTS**

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CHAPTER 5 – PERFORMANCE INDICATOR PREDICTION METHODOLOGIES

5.3 Distress Prediction Equations for Flexible Pavements and HMA Overlays

5.3.2 Rut Depth

The model for calculating total permanent deformation uses the plastic vertical strain under specific pavement conditions for the total number of trucks within that condition. The rate of accumulation of plastic deformation is measured in the laboratory using repeated load permanent or plastic deformation triaxial tests for HMA mixtures.

One set of model coefficients were originally assumed for all HMA layers within the same pavement structure (equation 5-1a in the MEPDG Manual of Practice). The hypothesis was that the different in dynamic modulus between the different layers would properly account for any difference in plastic deformations. Version 2.2 of the software allows the use of layer dependent model coefficients measured in the laboratory.

CHAPTER 10 – DETERMINATION OF MATERIAL PROPERTIES FOR NEW PAVVING MATERIALS

10.2 HMA Mixtures; Including SMA, Asphalt-Treated or Stabilized Base Layers, Asphalt Permeable Treated Base Mixes

Measuring the plastic deformation coefficients of equation 5.1a was excluded from Table 10-2, because of the hypothesis noted under section 5.3.2 above. The following is incorporated into Table 10-2 for measuring the laboratory plastic deformation coefficients and adjusting those values to match field performance.

Table 10-2. Asphalt Materials and the Test Protocols for Measuring the Material Property Inputs for New and Existing HMA Layers

Design Type	Measured Property	Source of Data		Recommended Test Protocol and/or Data Source
		Test	Estimate	
New HMA 9new pavement and overlay mixtures), as built properties prior to opening to truck traffic	Plastic Deformation Coefficients (k_1, k_2, k_3)	X		NCHRP project 9-30A Procedure (see Appendix A attached)

APPENDIX A

STANDARD METHOD OF TEST FOR:

DETERMINATION AND EVALUATION OF PLASTIC STRAIN COEFFICIENTS OF DENSE GRADED ASPHALT CONCRETE MIXTURES USING THE AMPT

1 SCOPE

1.1 This method covers the use of repeated load triaxial tests with the Asphalt Mixture Performance Tester (AMPT) for evaluating the rutting resistance of dense-graded asphalt concrete mixtures in accordance with the Mechanistic-Empirical Pavement Design Guide (MEPDG) rut depth computational methodology (see Notes 1 and 2).

NOTE 1: This Standard Method of Test is based on Appendix A of NCHRP Report #719. The data analysis and interpretation is structured towards integrating mixture and structural design in accordance with the MEPDG Manual of Practice and procedure (AASHTO, 2008). The method focuses on the use of the AMPT device for measuring the plastic strain coefficients of dense-graded asphalt concrete mixtures for use in the MEPDG software (Von Quintus, et al., 2012).

NOTE 2: National Highway Institute (NHI) Course #131118 provides an introduction and details on the equipment, use, test procedure, and other specifics for measuring the dynamic modulus and flow number of asphalt concrete mixtures with the AMPT (NHI, 2012).

1.2 The rutting resistance of dense-graded asphalt concrete mixtures is defined by the use of plastic strain coefficients of the rut depth transfer function included in the MEPDG software referred to as the Kaloush vertical strain rut depth transfer function. The plastic strain coefficients are determined from an analysis of the accumulated plastic strain from the repeated load triaxial test (see Notes 1, 3, and 4).

1.3 The values stated in English units are to be regarded as the standard.

1.4 This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.

2 APPLICABLE DOCUMENTS

2.1 AASHTO Test Standards:

R 30	Mixture Conditioning of Hot Mix Asphalt (HMA)
T 269	Percent Air Voids in Compacted Dense and Open Bituminous Paving Mixtures
PP 60	Preparation of Cylindrical Performance Test Specimens Using the Superpave Gyratory Compactor (SGC)

NOTE 3: The rut depth transfer functions included in the MEPDG version 9-30A software are listed below (see NCHRP Report #719: Von Quintus, et al., 2012):

1. Kaloush vertical strain transfer function; the transfer function included in the original MEPDG software developed under NCHRP Project 1-37A.
2. WesTrack shear strain and shear stress transfer function; added to version 9-30A of the software under NCHRP Project 9-30A.
3. Modified Leahy vertical strain and deviator stress transfer function; added to version 9-30A of the software under NCHRP Project 9-30A. This transfer function is a modified form of the original Leahy or Asphalt Institute transfer function. The modified Leahy transfer function excludes the temperature and asphalt property factors included in the original Leahy (Asphalt Institute) transfer function, because dynamic modulus of the mixture is used to compute the HMA mixture response for predicting rut depths.

This test method provides inputs to the Kaloush vertical strain rut depth transfer function.

NOTE 4: A more detailed discussion on the use of different test procedures and transfer functions (in comparing the predicted rut depth and standard error terms) is included in Chapter 3 of the NCHRP Project 9-30A research report – NCHRP Report #719.

3 SIGNIFICANCE AND USE

3.1 The method is used in support of the design and evaluation of dense-graded asphalt concrete mixtures using mixture specific plastic strain coefficients instead of the default coefficients derived from the global calibration of the Kaloush vertical strain rut depth transfer function (see Note 5). The plastic strain coefficients are given and defined in Section 8 of this test method.

NOTE 5: The MEPDG software developed under NCHRP Project 1-37A uses the same set of default values for the plastic strain coefficients for all layers and assumes the dynamic modulus accounts for any difference in rutting between the different layers and mixtures. Use of only dynamic modulus, however, did not improve on the accuracy and precision of the transfer functions, as explained in NCHRP Report #719. Use of the repeated load triaxial tests and resulting asphalt concrete plastic strain coefficients improved the goodness-of-fit and overall accuracy of the rut depth transfer function and prediction methodology included in the MEPDG software.

3.2 Repeated load triaxial tests are used to determine values for the plastic strain coefficients to the Kaloush vertical strain rut depth transfer function included in the MEPDG software. The laboratory-derived plastic strain coefficients are adjusted to represent field conditions (see Note 6).

NOTE 6: The shift factors for adjusting the laboratory derived coefficients were developed using multiple test sections included in the FHWA-LTPP program, full-scale test track test sections, accelerated loading facility test sections, and other roadway segments.

3.3 Repeated load testing is suggested for use in calculating the level of rutting of HMA mixtures for rehabilitation and new pavement construction during design and/or to create a materials library on a mixture specific basis (see Notes 3 and 5).

3.4 The following bullets summarize the conditions most appropriate for using this test method and rut depth transfer function identified in the Scope.

- Repeated load triaxial tests using the AMPT in support of the Kaloush rut depth transfer function can be used during the mixture design stage and mixture production when the component materials or bulk mixture can be sampled for preparing test specimens in accordance with AASHTO TP 79.
- The height-to-diameter requirement required for the test specimens eliminates the use of cores for HMA lifts less than 4 inches in thickness. The test procedure can be used for lift thicknesses that are greater than 6 inches. For lift thicknesses of 4 to 6 inches, test specimens can be cored laterally along a larger sample extracted from the HMA mat assuming that aggregate alignment has little to no impact on the plastic deformation parameters. That assumption or hypothesis is believed to be false (Von Quintus, et al., 1991). In addition, coring the test specimens laterally from a larger diameter sample is not included within this test method.
- The height-to-diameter requirement in AASHTO TP 79 restricts the use of this procedure to determine the MEPDG inputs for forensic investigations or follow-up studies when there are disputes between the owner and contractor over any warranty work and actual materials are unavailable to reconstitute the HMA mixture, since most HMA lifts or layers are less than 4 inches in thickness.
- The use of the gyratory compactor is known to result in variable air voids across the radial axis as well as along the vertical axis of the compacted specimen. AASHTO TP 79 requires that the gyratory compacted specimens be cored to test the center part of the specimen. Coring the center portion of the gyratory specimen reduces the air void gradients within the test specimen and is a preferred surface for mounting the LVDTs on the test specimen.

4 TERMINOLOGY

AMPT	Asphalt Mixture Performance Tester
Equivalent Temperature	The temperature that will result in the same level of rutting at the end of the design period with the rutting calculated using varying temperature defined for that climate and structure using the MEPDG software and computational methodology.
I_S-Value	The intercept from the steady state region of the laboratory relationship between the log of the plastic strain and log of the number of loading cycles that has a constant slope.
MEPDG	Mechanistic-Empirical Pavement Design Guide

m-Value	The region of the laboratory relationship between the log of the plastic strain and log of the number of loading cycles that has a constant slope. This is also defined or referred to as the secondary and steady state region.
Plastic Strain	The strain in an asphalt concrete layer or test specimen from an applied load that is unrecoverable or permanent.

5 TEST TEMPERATURE OPTION AND NUMBER OF TEST SPECIMENS

5.1 Two test temperature options are available for use, which are applicable to all rut depth transfer functions: Option A – the multiple test temperature option; and Option B – the equivalent test temperature option. The number of test specimens is dependent on whether the multiple temperature or equivalent temperature option is selected.

5.2 Option A ─ Multiple Test Temperatures

5.2.1 The multiple temperature option uses three test temperatures, defined as: (1) 50 percent reliability PG high temperature minus 5°C, (2) 20°C, and (3) the middle temperature between the first two.

5.2.2 For the multiple test temperature option, 2 test specimens at each temperature are required for a total of six test specimens. The multiple test temperature option should be used for pavement structural designs, during the final mixture design stage, and for detailed forensic investigations.

5.3 Option B ─ Equivalent Test Temperatures

5.3.1 The equivalent temperature option uses one test temperature that is defined as the equivalent annual or representative temperature that will result in the same level of rutting at the end of the design period with the rutting predicted using daily temperatures defined for that climate and structure. Determination of the equivalent test temperature is provided in Section 8.2.2.

5.3.2 For the equivalent test temperature option, 3 test specimens are required. The equivalent test temperature option should be used for mixture design verification and acceptance of HMA mixtures during construction.

6 TEST SPECIMEN PREPARATION

6.1 This section provides guidance on preparing the mixture samples for measuring the plastic strain coefficients. Three types of samples can be used: reconstituted samples, bulk mixture sampled during mixture production, or cores of a sufficient height and diameter. Cores can be tested, but they must satisfy the strict height to diameter ratio requirement specified in AASHTO TP 79. If cores are being tested in accordance with this test method, refer to Section 6.4.5.

6.2 The required number of test specimens shall be prepared depending on whether the multiple test temperature or equivalent test temperature option is being used (refer to Section 5 of this method). All test specimens should be prepared from reconstituted samples of the aggregate and asphalt that are blended and compacted to the volumetric conditions after construction, or bulk mixture sampled during production.

6.3 Short Term Aging of the Asphalt Concrete Mixture

6.3.1 The test specimen preparation process for reconstituting the materials includes the short-term aging procedure to simulate asphalt concrete production. Mixture for all specimens should be short-term oven aged for 4 hours at 135°C in accordance with AASHTO R 30, *Mixture Conditioning of Hot Mix Asphalt (HMA),* prior to compaction.

6.3.2 The short term aging procedure should not be used on test specimens prepared from bulk mixture sampled during construction or on cores for obvious reasons (they already include short term aging).

6.4 Specimen Compaction

6.4.1 Specimens for triaxial testing are fabricated in accordance with AASHTO PP 60, *Preparation of Cylindrical Performance Test Specimens Using the Superpave Gyratory Compactor*. This is the specimen fabrication standard developed for making specimens for the AMPT.

6.4.2 The target air void level for laboratory compacted test specimens using the Superpave Gyratory Compactor shall be the average air void level expected after rolling (the expected mean air void level from construction). This value can be determined from historical records of recent construction projects.

6.4.3 The air void tolerance for all test specimens compacted in the laboratory using the Superpave Gyratory shall be in accordance with AASHTO TP 79.

6.4.4 All triaxial test specimens shall be prepared in accordance with AASHTO TP 79; *Compaction of Triaxial Test Specimens for Dynamic Modulus and Repeated Load Permanent Deformation Tests*. The test specimen size used in the repeated load triaxial test is 100mm by 150mm. As noted above, materials should be sampled and reconstituted during the mixture design stage or bulk HMA sampled during production.

6.4.5 If cores of sufficient height are recovered for testing, the air voids of the test specimen should be measured in accordance with AASHTO T 269 after the test specimen has been prepared and sized in accordance with AASHTO TP 79.

6.5 Grouping of Test Specimens

6.5.1 The air void content for each test specimen should be measured and reported, as noted in sections 6.4.4 or 6.4.5.

6.5.1 When the multiple test temperature option is selected, sort the test specimens into three subsets of two specimens each (or the number of test specimens selected for each test temperature) so that the average air voids of the different subsets are as equal as possible. This requirement applies to laboratory compacted reconstituted mixture and bulk mixture sampled during construction, and cores of sufficient height.

6.5.2 Grouping of the test specimen is not required when the equivalent temperature option is selected because all test specimens are tested at the same test temperature.

7 REPEATED LOAD TRIAXIAL TESTING

7.1 The dynamic modulus and repeated load triaxial test shall be measured in accordance with AASHTO TP 79, *Determining the Dynamic Modulus and Flow Number for HMA Using the AMPT*, with the following exceptions.

7.1.1 Conditioning Cycles; 100 conditioning cycles shall be applied at the beginning of the repeated load triaxial test. The repeated axial load applied to the test specimen for the conditioning cycles is 10 psi with the use of a confining pressure of 10 psi.

7.1.2 Testing Condition; The repeated axial load and confining pressure for measuring the repeated load plastic strain is listed below.

- Repeated axial load applied to the test specimen is 70 psi.
- Confining pressure is 10 psi.

7.2 The plastic and total strains should be measured and stored in the data acquisition system in accordance with AASHTO TP 79. Figure 1 shows a graphical example of the test results – cumulative plastic strain versus number of repeated axial loads or repetitions.

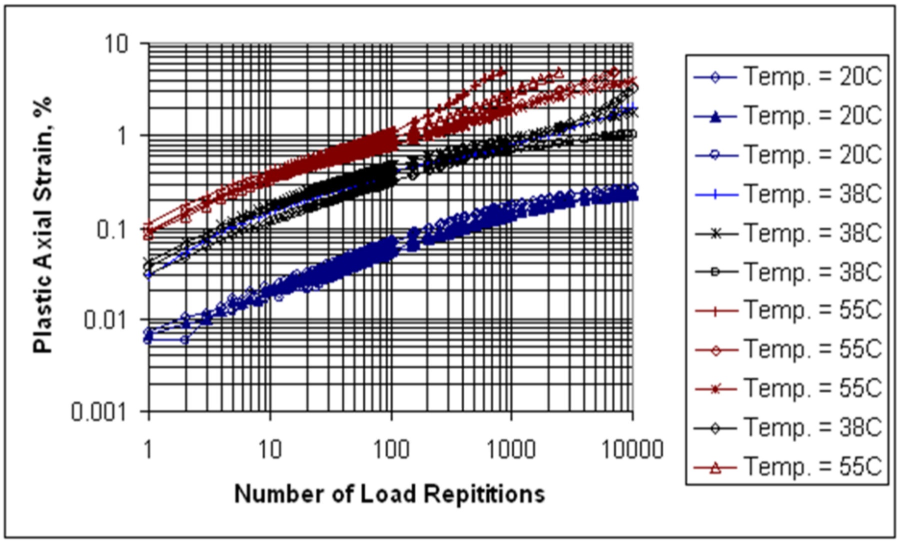

Figure 1. Test Results from a Repeated Load Triaxial (Confined) Plastic Strain Test

8 DETERMINATION OF PLASTIC STRAIN COEFFICIENTS

8.1 This part of the standard explains the determination of the plastic strain coefficients for the Kaloush vertical strain rut depth transfer function.

8.1.1 Kaloush-Witzcak Vertical Resilient Strain Transfer Function; The following equation is the plastic strain relationship included in the MEPDG software to predict rut depth in the HMA layer increments.

$$\varepsilon_p = \varepsilon_r K_Z \beta_{r1} 10^{k_{r1}} (T)^{k_{r2}\beta_{r2}} (N)^{k_{r3}\beta_{r3}} \tag{1}$$

Where:

ε_p = Incremental plastic strain at the mid-depth of a thickness increment.
ε_r = Resilient strain calculated at the mid-depth of a thickness increment.
T = Temperature at the mid-depth of a thickness increment.
N = Number of axle load applications of a specific axle type and load interval within a specific time interval.
β_{r1}, β_{r2}, β_{r3}, = Local calibration coefficients, all equal to 1.0 for the global calibration effort and within NCHRP Project 9-30A.
k_{r1} = Plastic strain factor or coefficient (for the global calibration effort under NCHRP Project 1-40D, the coefficient equals -3.35412).
k_{r2} = Plastic strain factor related to the effect of temperature on the intercept (for the global calibration effort under NCHRP Project 1-40D the temperature exponent equals 1.5606).
k_{r3} = Plastic strain factor related to the effect of wheel load (for the global calibration effort under NCHRP Project 1-40D, the loading cycles exponent equals 0.4791).
K_Z = Depth function and equal to:

$$K_Z = (C_1 + C_2 D)(0.328196)^D \tag{2}$$

$$C_1 = -0.1039 H_{HMA}^2 + 2.4868 H_{HMA} - 17.342 \tag{3}$$

$$C_2 = 0.0172 H_{HMA}^2 - 1.7331 H_{HMA} + 27.428 \tag{4}$$

D = Depth to the mid-depth of the thickness increment, inches.
H_{HMA} = Total thickness of the asphalt concrete layer, inches.

8.1.2 The plastic strain factors (k_{r1}, k_{r2}, *and* k_{r3}) are determined from repeated load plastic strain tests conducted in the laboratory and adjusted to field conditions. The k_{r3} factor is the slope within the steady state or secondary region, while the k_{r1} is the intercept of the log-log relationship between the number of load applications and cumulative plastic strain. The k_{r2} factor is the effect of temperature on the intercept. The k_{r1} and k_{r3} coefficients are graphically illustrated in Figure 2.

8.2 HMA Mixture Evaluation Procedure

The steps for determining the plastic strain coefficients from a repeated load triaxial test to evaluate the rutting resistance of asphalt concrete mixtures are included in this section. Two options are available, each of which is discussed separately.

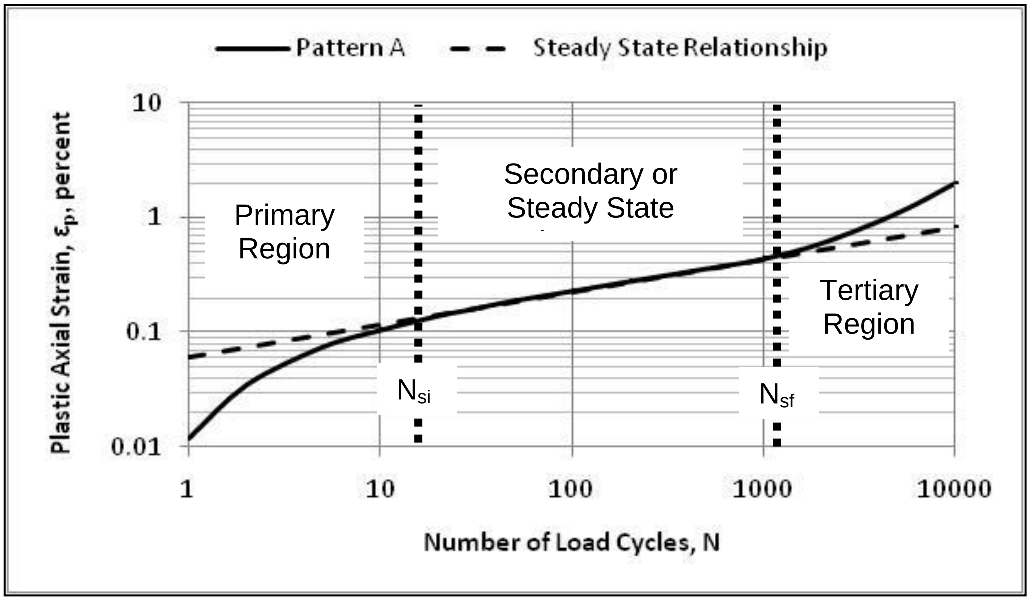

Figure 2. Accumulation of Plastic Axial Strain Measured in the Laboratory – Defined as Pattern A [Extracted from NCHRP Report #719]

8.2.1 Option A—Multiple Temperature Option

8.2.1.1 The multiple temperature option uses three test temperatures, defined as: (1) 50 percent reliability PG high temperature minus 5°C, (2) 20°C, and (3) middle range temperature between the first two.

1. Determine the laboratory-derived slope or m-value of the steady state or secondary region for each test specimen. The steady state region is the area where the slope or m-value becomes constant between the number of loading cycles and accumulated plastic strain (refer to Figures 2 and 3).

    a. The average steady state slope for laboratory repeated load tests should be determined based on a moving decade of loading cycles. [For example; 1 to 10, 2 to 20, 3 to 30, …; 10 to 100, 20 to 200, 30 to 300, …; 100 to 1000, 200 to 2000, 300 to 3000, …; etc.] Once the slope becomes constant, defined as the steady state region of the laboratory test, that value represents the exponent for the load cycle term (N) of the rut depth transfer functions or m-value (Figure 3 is an illustration of this response – Pattern B). Appendix A includes some examples of test results from repeated load triaxial tests. Figure A.1 is an example of Pattern B.

    b. Two other opposite possibilities exist over the entire number of loading cycles for the laboratory repeated load tests; (1) the slope continues to decrease over the entire number of loading cycles, and (2) the slopes starts to increase at an increasing rate. Figures 2 (Pattern A) and 4 (Pattern C) are illustrations of these responses. Figure A.2 is an example of Pattern C, while Figure A.3 includes an example of Pattern A (see test specimen AL-3-3). Determining the slope for these conditions is discussed in the following paragraphs (see Note 7).

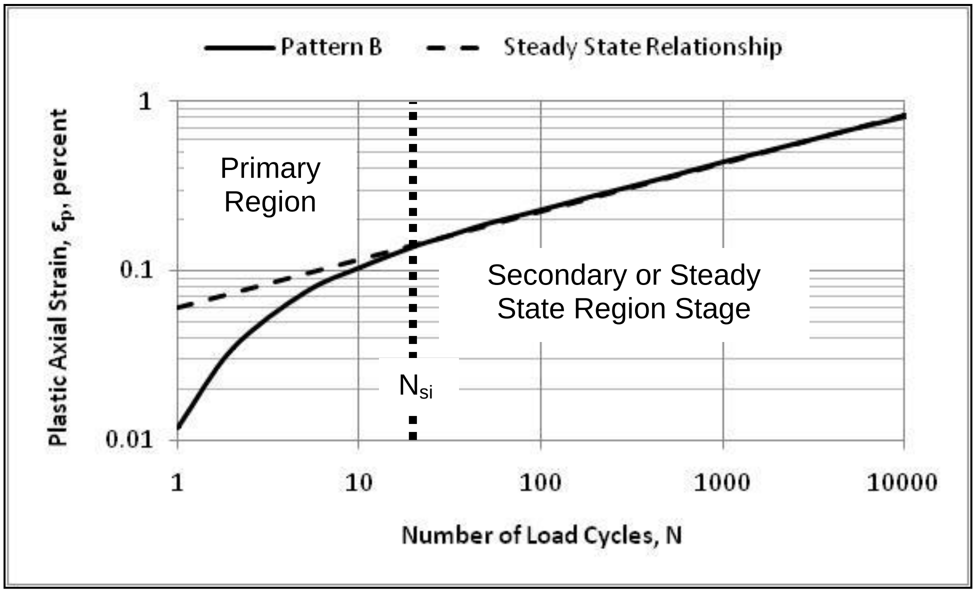

Figure 3. Accumulation of Plastic Axial Strain Defined as Pattern B [Extracted from NCHRP Report #719]

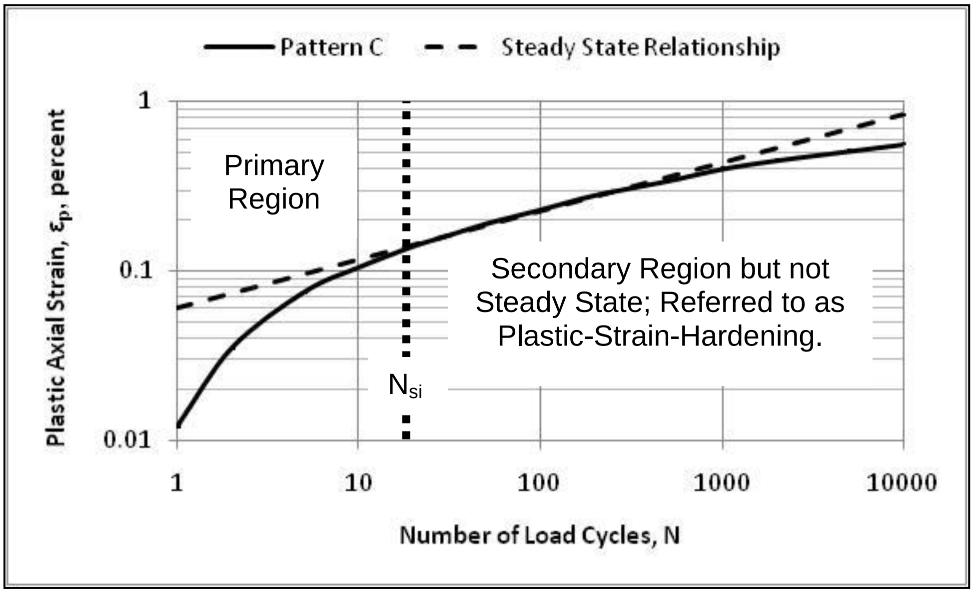

Figure 4. Accumulation of Plastic Axial Strain Defined as Pattern C [Extracted from NCHRP Report #719]

NOTE 7: The primary stage or region (see Figures 2 through 4) should be excluded from determining the slope of the steady state region. NCHRP Report #719 discusses this difference in greater detail and provides examples for determining the slope in the steady state or secondary region.

i. When a test specimen starts to exhibit continual decrease in plastic strains with continued loading cycles, the average slope should be determined for the region where the change in slope is relatively constant. Figure A.2 in Appendix A is an example of test results that are characteristic of loading Pattern C (see Figure 4). The m-value for each test specimen is determined between 2,000 and 10,000 axial loading cycles.

   ii. When a test specimen exhibits accelerated plastic deformation (the m-value or slope continually increases at an increasing rate; typically referred to as tertiary flow), that part of the test should be excluded from determining the m-value in the steady state region of the test. Test Specimen #AL-3-3 in Figure A.3 is an example of test results that are characteristic of loading Pattern A (see Figure 2). The m-value for test specimen AL-3-3 is determined between 1,500 and 4,500 loading cycles.

2. Determine the laboratory-derived m-value or slope from all test specimens. The methodology included in the MEPDG assumes the steady state slope is independent of test temperature. Any change in temperature is accounted for in the temperature exponent of the transfer function and/or through the effects of temperature on dynamic modulus (see Note 8).
   a. If the slope does not consistently change with test temperature, average all slopes.
   b. If the slope consistently changes with test temperature (increasing or decreasing with test temperature), determine the representative slope at the equivalent temperature. Equivalent temperature is defined under Option B (Section 8.2.2).

NOTE 8: NCHRP Report #719 concluded the average slopes between all test temperatures were approximately equal. There will be mixtures for which the m-value consistently increases or decreases with test temperature. Test specimens with significantly different slopes should be investigated to determine the reason for different slopes. An additional test specimen can be prepared and tested when the variability is high and there is no obvious reason for the difference in the steady state slopes.

3. Determine the laboratory-derived intercept or I_s-value (intercept from the steady state region) for each test specimen and test temperature using equation 5 based on the m-value or slope within the steady state region determined from Step #2.

$$I_s = \log(\varepsilon_p) - mLog(N) \qquad (5)$$

4. Using only the test specimens used to determine the m-value or secondary slope, determine average m-value for all test temperatures.

5. Determine the average I_s-value at each test temperature for the asphalt concrete mixture. The laboratory-derived intercept or I_s-value will be test temperature dependent.

6. Determine the temperature dependency of the intercept through the exponent of the temperature term using equation 6; the exponent to the temperature term should be determined from the laboratory test results of all test specimens and temperatures by fitting the m-value and I_s-value to the data.

$$k_{r2} = \frac{Log(\varepsilon_p) - I_s - mLog(N)}{Log(T)} \qquad (6)$$

7. Determine the field matched coefficients for the asphalt concrete mixture by the following.
    a. Figure 5 is used to adjust the laboratory-derived average m-value or secondary slope to the “field matched” slope or k_{r3} value.
    b. Figure 6 is used to adjust the laboratory-derived average I_s–value or intercept from the steady state region to the “field matched” intercept or k_{r1} value.
    c. The temperature exponent for the Kaloush transfer function, k_{r2}, is not adjusted from the laboratory measured values. In other words, the laboratory derived temperature exponent is assumed to be equal to the field adjusted temperature exponent.

8. The field matched intercept from Figure 6 is multiplied by the thickness adjustment factors provided in Table 1 (see Note 9).

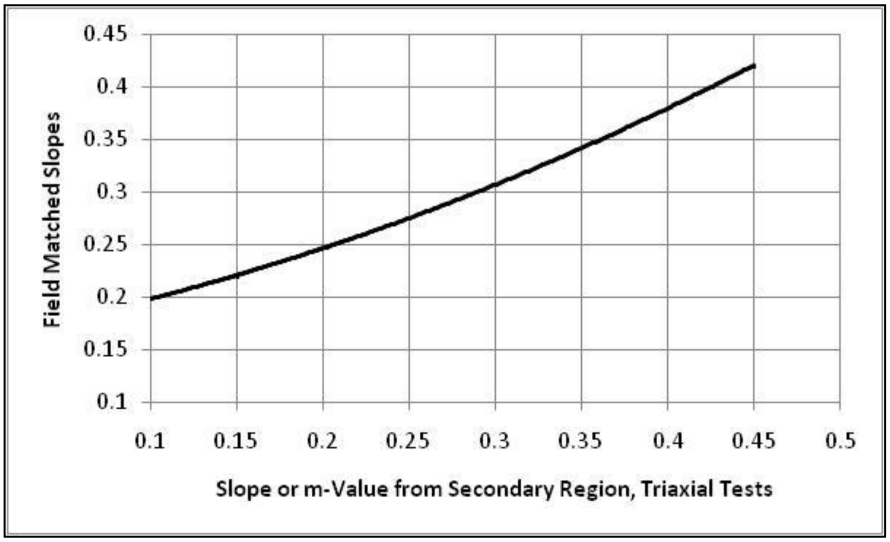

Figure 5. Determining the Field Matched Slopes from Laboratory-Derived m-Values from Repeated Load Triaxial Tests; Kaloush Transfer Function [Extracted from NCHRP Report #719. The dispersion in the data for this relationship is included in the NCHRP report.]

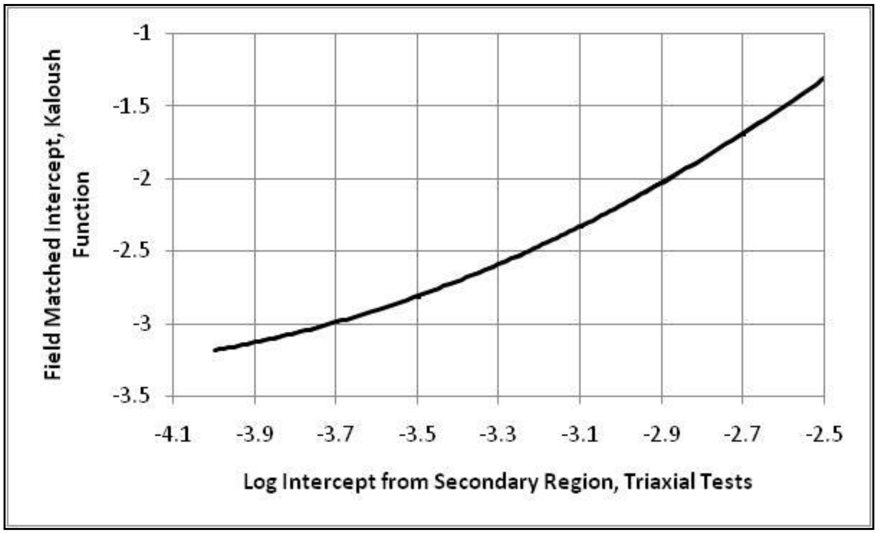

Figure 6. Determining the Field Matched Intercept from Laboratory Derived I_S-Values from Repeated Load Triaxial Tests; Kaloush Transfer Function [Extracted from NCHRP Report #719. The dispersion in the data for this relationship is included in the NCHRP report.]

Table 1. Thickness Adjustment or Shift Factors for Determining the Field Matched Intercept Value of the Transfer Functions

HMA Mixture Application	HMA Layer Thickness, in.	Adjustment Factor for Kaloush Transfer Function
HMA Overlays of PCC or Semi-Rigid Pavements	< 3.0	0.83
	3 to 4	0.90
	> 4.0	1.0
HMA Overlays of Flexible Pavements	< 4.0	1.4
	4 to 6	1.2
	> 6.0	1.0
New Construction, Unbound Aggregate Base or Full-Depth	< 4.0	1.05
	4.0 to 6.0	1.02
	6.0 to 8.0	1.0
	> 8.0	1.0

8.2.2 Option B—Equivalent Test Temperature Option

8.2.2.1 The equivalent test temperature option uses one test temperature defined as the equivalent annual temperature, which will result in the same level of rutting at the end of the design period with the rutting predicted using temperatures defined for that climate and structure. The following lists the steps and determination of the values for the plastic strain coefficients based on the equivalent temperature concept.

1. Determine the equivalent annual or representative temperature for the climate and structure of the project. Two methods can be used to determine the equivalent or representative temperature for a specific climate and area, as listed below.

a. Determine the temperature for the site in accordance with LTPPBind-2.1. The temperature from LTPPBind-2.1 is entered in Figure 7 and used to estimate the equivalent test temperature (see Note 10).

b. Use the MEPDG software to estimate the equivalent test temperature that will result in the same level of rutting for multiple roadway segments using the actual climate data within a specific site or region. This method is considered the more accurate one because it uses the MEPDG rut depth computational methodology directly in determining that temperature and can be completed during the local calibration process (see Note 10). The following paragraphs briefly discuss using the MEPDG software to estimate the equivalent test temperature.

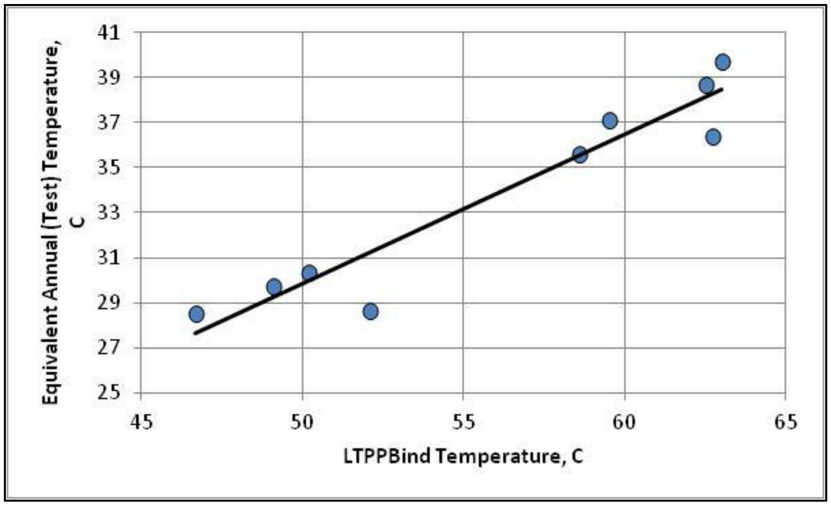

Figure 7. Graphical Relationship between the LTPPBind2.1 Pavement Temperature and the Equivalent Test Temperature

i. Determine an initial estimate for the laboratory adjusted transfer function coefficients. The initial values can be extracted from historical data, if available, or the input level 2 values determined for the mixture being evaluated. In other words, values for the plastic strain coefficients are estimated using the relationships provided in Appendix B.
ii. Execute the MEPDG to predict the rut depth over a range of constant temperatures for the trial pavement structure and design truck traffic. Constant temperatures of 10, 20, 40, and 50°C will be sufficient for most climates. These constant temperature files are available within the MEPDG Version 9-30A software. Plot maximum predicted rut depth at the end of the design period as a function of temperature (See Figure 8).

iii. Execute the MEPDG software to predict the rut depth using the actual climatic files and the same default values, trial structure, and design truck traffic used in the above step.

iv. Use the maximum rut depth predicted over the design period to determine the equivalent annual or representative temperature that results in that same value. In order words, enter Figure 8 for the example with the predicted rut depth for the actual climatic values to find the single temperature value over the entire design period and design truck traffic. This temperature is defined as the equivalent test temperature for the specific structure, climate, and truck traffic.

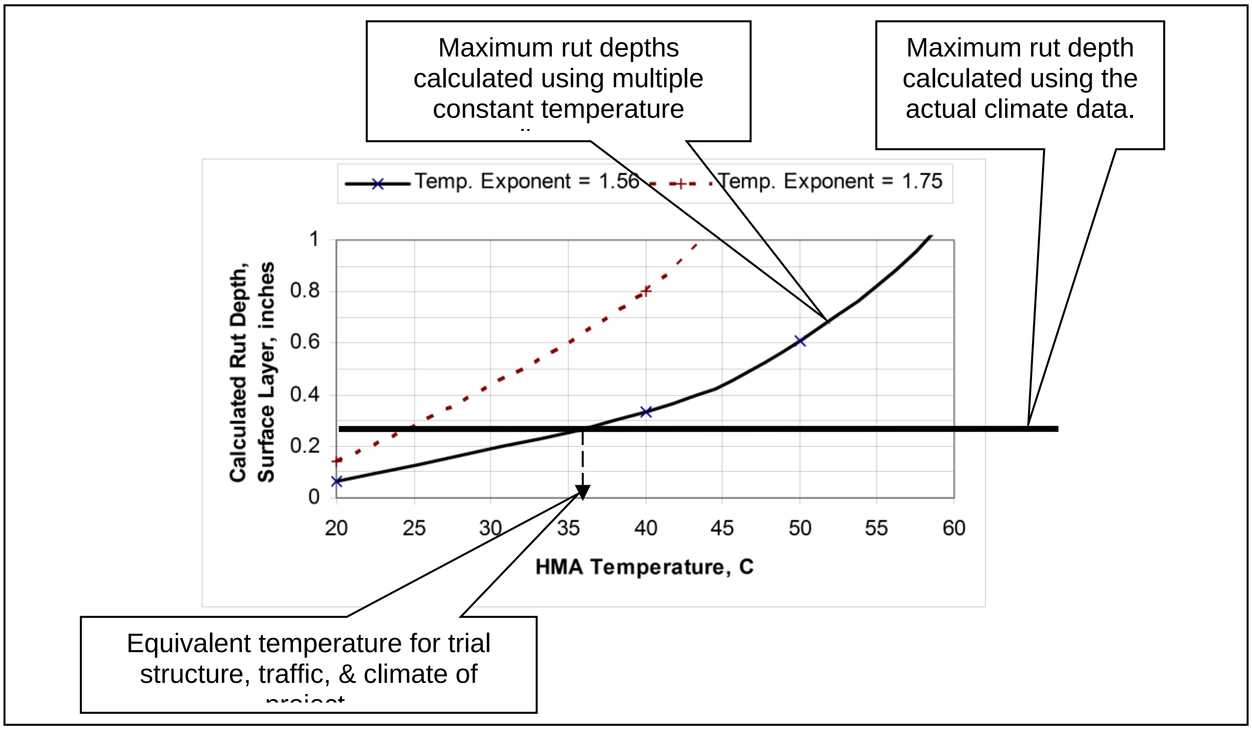

Figure 8. Example of Rut Depths Predicted with MEPDG (Kaloush Transfer Function) Using a Constant Temperature Environment to Estimate the Equivalent Test Temperature

2. Three test specimens are prepared and tested at the defined equivalent test temperature. The test specimens are compacted at the average in place air voids determined by the specifications, as noted in Section 6.4.2. This value can be determined from historical records of recent construction projects.

3. Determine the laboratory-derived slope or m-value within the steady state or secondary region for each test specimen in accordance with Step #1 under Section 8.2.1.

4. Determine the laboratory-derived intercept or I_S-value from the steady state or secondary region for each test specimen in accordance with Step #3 of Section 8.2.1.

5. Determine the average m-value and I_S-value for all test specimens at the equivalent test temperature.

6. Determine the field matched coefficients for the asphalt concrete mixture by the following.
   a. Figure 4 is used to adjust the laboratory-derived steady state slope or average m-value to the "field matched" slope.
   b. Figure 5 is used to adjust the laboratory-derived intercept or average I_s-value from the steady state region to the "field matched" intercept.
   c. The temperature exponent for the Kaloush transfer function is not adjusted from the laboratory measured values (a constant value of 1.5606 is used for the equivalent temperature concept.

7. The field matched parameters for the intercept are multiplied by the appropriate thickness adjustment factor provided in Table 1 (see Note 9).

8. There will be cases where the equivalent annual or representative temperature for a site or layer may change because of a change in pavement structural design or other reasons. For these cases, the plastic strain coefficients determined from the repeated load triaxial tests at the equivalent test temperature can be adjusted to a higher or lower equivalent test temperature. The adjustment of these coefficients is included as Appendix C.

9 INTERPRETATION OF PLASTIC DEFORMATION COEFFICIENTS

9.1 This section of the report uses the plastic strain coefficients for the field adjusted slope and intercept to estimate the rutting resistance of the asphalt concrete mixture. Two methods can be used to determine if the asphalt concrete mixture will exhibit rutting less than the threshold value selected by the user: (1) the MEPDG software and (2) a graphical analysis.

9.2 MEPDG Software: The MEPDG software is used to calculate the total rut depth over the design period for the field matched or adjusted plastic strain coefficients. The field matched parameters for the slope and intercept are entered into the MEPDG software for predicting the rut depth for that mixture and evaluated as:

a. If the total predicted rut depth is less than the threshold value for rutting, the mixture is considered adequate.
b. If the predicted rutting is greater than the threshold value, however, a revised mixture design maybe needed (see Note 11).

NOTE 11: The procedure assumes the asphalt concrete mixture is not susceptible to moisture damage, adequate bond exists between all individual lifts, and compaction of the asphalt concrete mixture excludes the occurrence of checking, segregation, and other construction defects.

9.3 Relationship between Plastic Strain Coefficients; k_{r1} and k_{r3}: The relationship used to determine the rutting resistance was determined by using the MEPDG software for different levels of truck traffic using a 90 percent reliability level.

9.3.1 Figure 9 applies to new pavement structures designed to carry less than 3 million trucks, Figure 10 applies to new pavement structures designed to carry less than 10 million truck, and Figure 11 applies to new pavement structures designed to carry less than 30 million trucks over the design life of the pavement.

9.3.2 Figure 12 applies to asphalt concrete overlays designed to carry less than 3 million trucks, Figure 13 applies to asphalt concrete overlays designed to carry less than 10 million trucks, and Figure 14 applies to asphalt concrete overlays designed to carry less than 30 million trucks.

9.3.3 For new pavements and asphalt concrete overlays along roadways with truck traffic exceeding 30 million trucks, it is suggested that the software be used to ensure the asphalt concrete mixtures will have sufficient rutting resistance to not exceed the specific design criteria.

9.3.4 Plot the secondary field adjusted slope (k_{r3}) and field adjusted intercept (k_{r1}) on the appropriate figure or graph for the conditions where the asphalt concrete mixture will be placed. If the point plots below the specific line with the noted rut depth, there is a 90 percent reliability that the asphalt concrete mixture will be resistance to that level of rutting. A typical rut depth criteria used by many agencies is 0.50 inches. The lines for 0.25 and 0.75 inches of rutting were added to determine the level of rutting for a specific asphalt concrete mixture for judging the mixture's rutting resistance at a 90 percent reliability level.

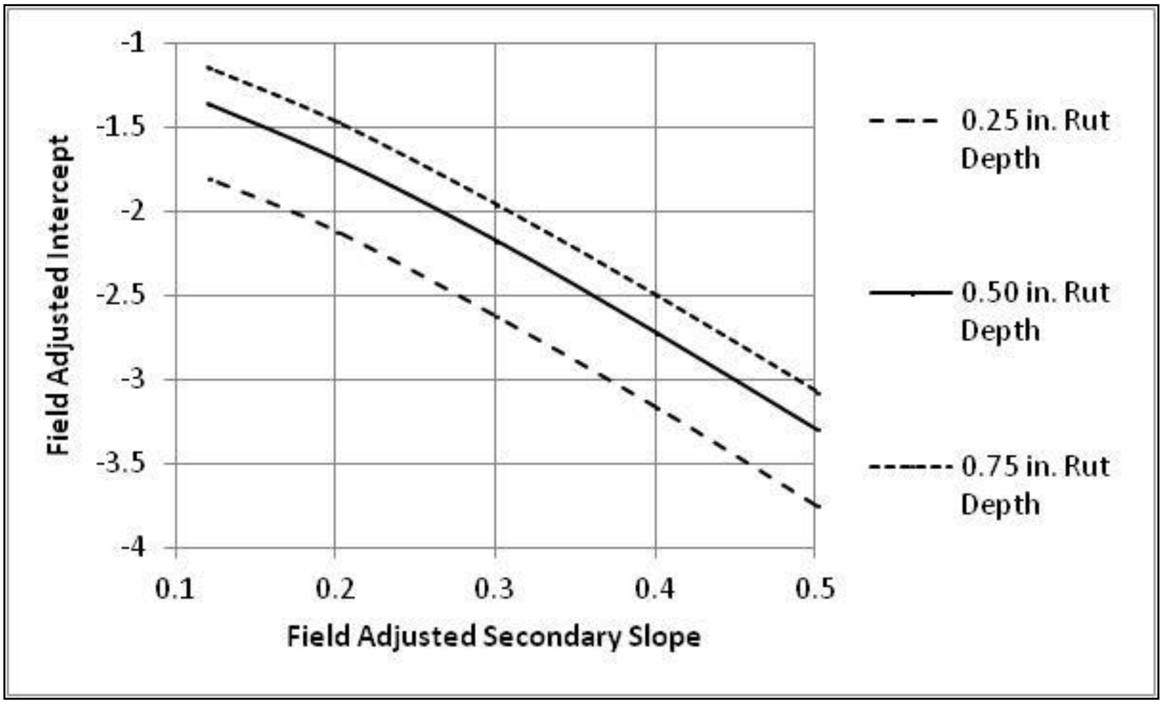

Figure 9. New Construction, Asphalt Concrete Plastic Strain Coefficients for Lower Levels of Truck Traffic – less than 3 million Trucks over 20 years

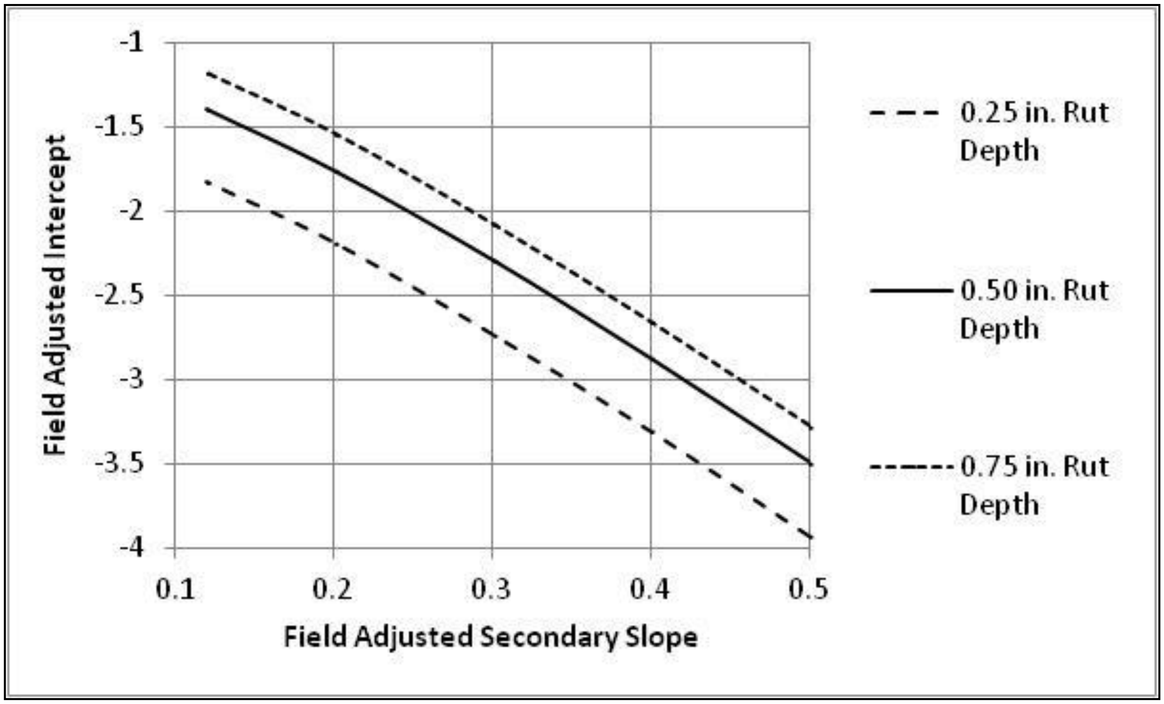

Figure 10. New Construction, Asphalt Concrete Plastic Strain Coefficients for Moderate Levels of Truck Traffic – less than 10 million Trucks over 20 years

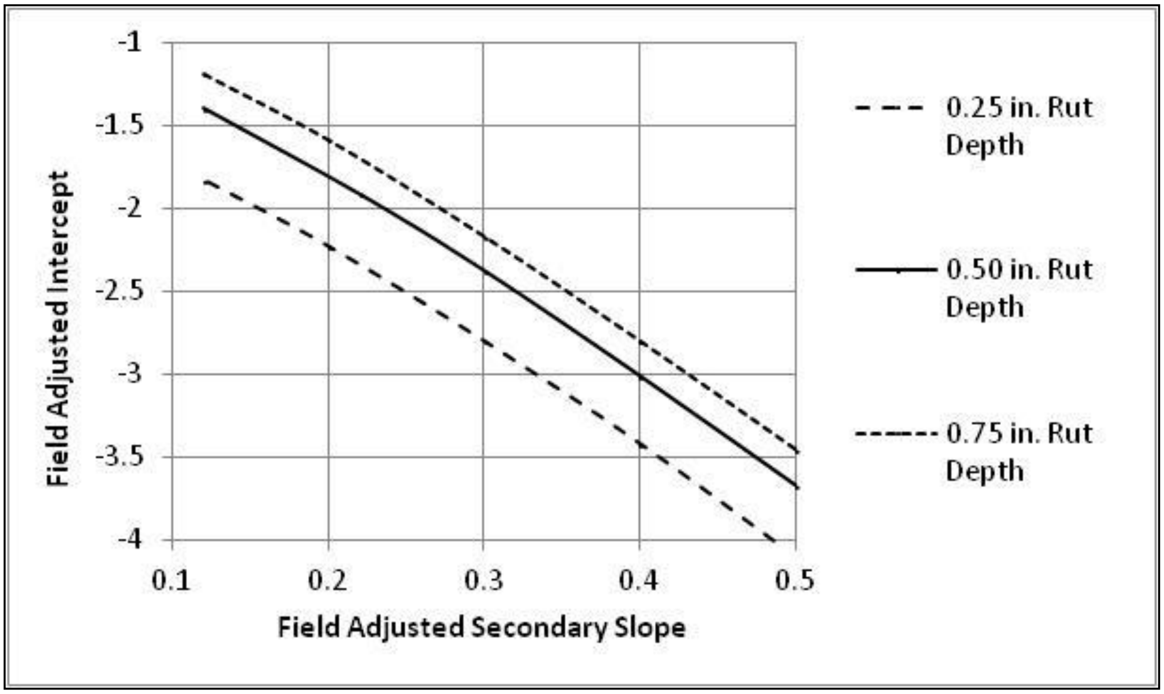

Figure 11. New Construction, Asphalt Concrete Plastic Strain Coefficients for High Levels of Truck Traffic – less than 30 million Trucks over 20 years

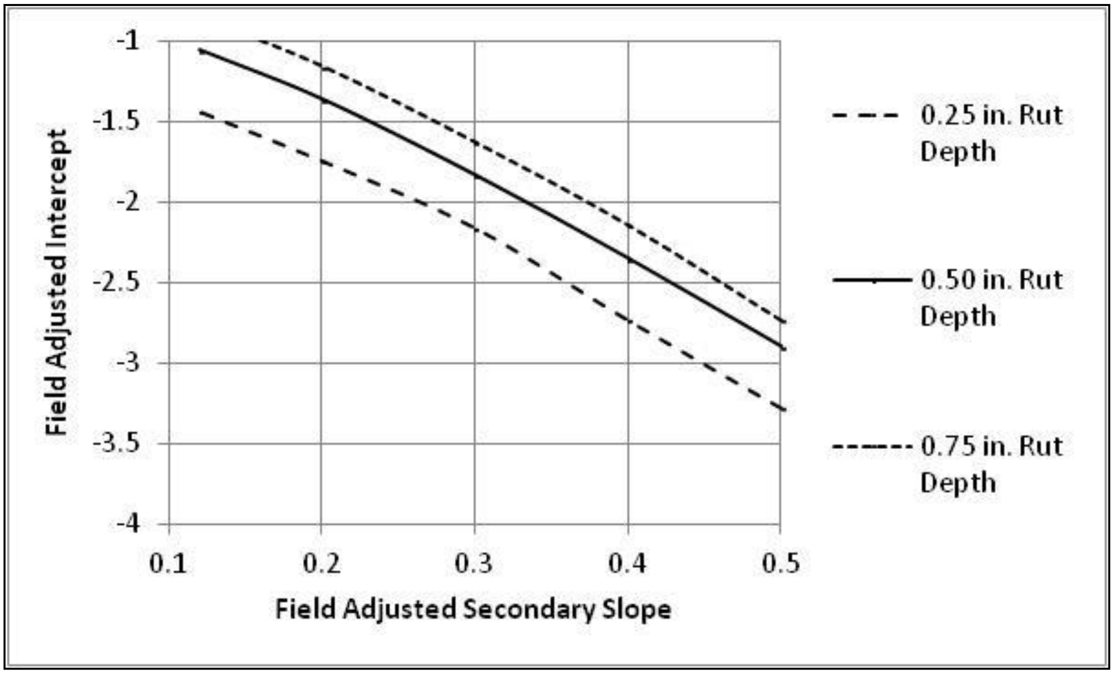

Figure 12. Asphalt Concrete Overlay Plastic Strain Coefficients for Lower Levels of Truck Traffic – less than 3 million Trucks over 20 years

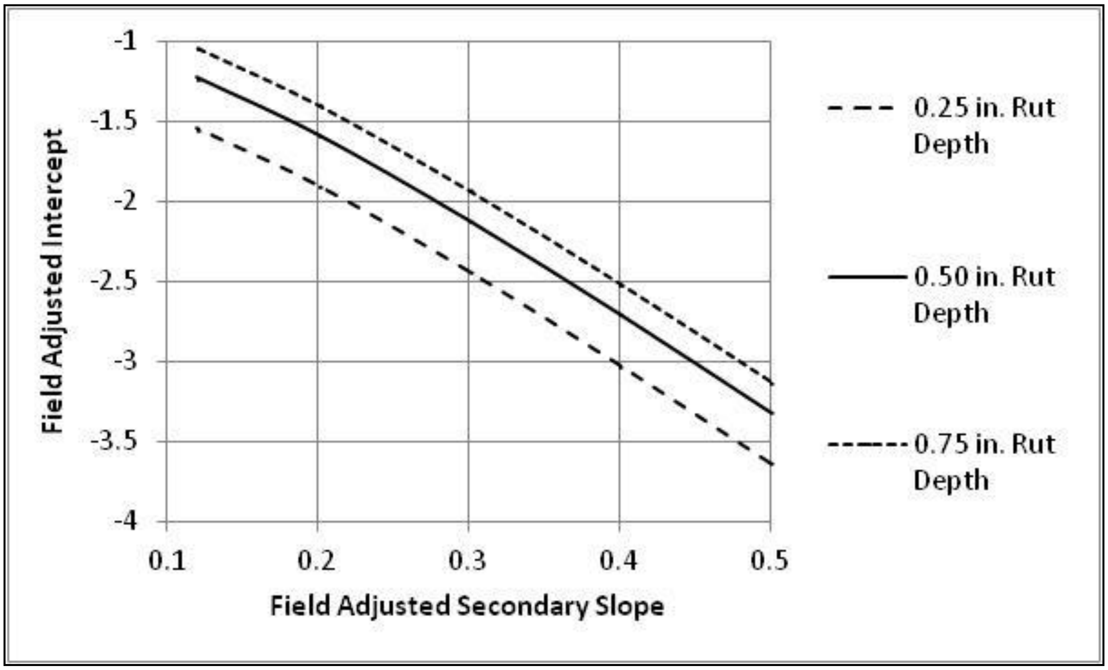

Figure 13. Asphalt Concrete Overlay Plastic Strain Coefficients for Moderate Levels of Truck Traffic – less than 10 million Trucks over 20 years

10 REPORT

10.1 The following parameters shall be reported from the plastic strains measured during the repeated load triaxial test.

- Number of conditioning cycles used; Section 7.1.1 requires 100 cycles to be used.
- Average repeated axial stress.
- Average confining pressure.
- Test Temperature.
- Air Voids for each test specimen.
- For each test temperature of the multiple temperature option or the equivalent test temperature option:
    - The m-value or slope of the steady state region.
    - The I_s-value or intercept from the steady state region.
- For the asphalt concrete mixtures tested using the equivalent test temperature option:
    - The average m-value and field adjusted slope or k_{r3} value.
    - The average I_s-value and field adjusted intercept or k_{r1} value. (A value of 1.5606 is assumed for the temperature exponent or coefficient, k_{r2}.)
- For the asphalt concrete mixture tested using the multiple temperature option:
    - The average m-value and field adjusted slope or k_{r3} value.
    - The k_{r2} coefficient or exponent of the temperature term.
    - The average Is value at the equivalent test temperature and field adjusted intercept or k_{r3} value.

11 PRECISION AND BIAS

11.1 The precision and bias of this test standard and interpretation of the test results have yet to be determined.

12 KEY WORDS

MEPDG, Mechanistic-Empirical Pavement Design Guide, Plastic Strain Coefficients, Kaloush Rut Depth Transfer Function

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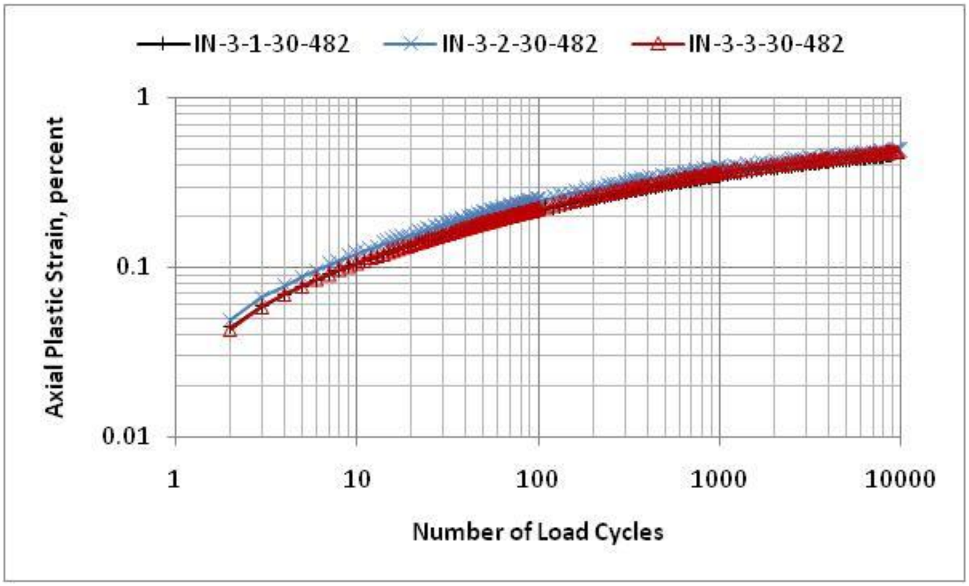
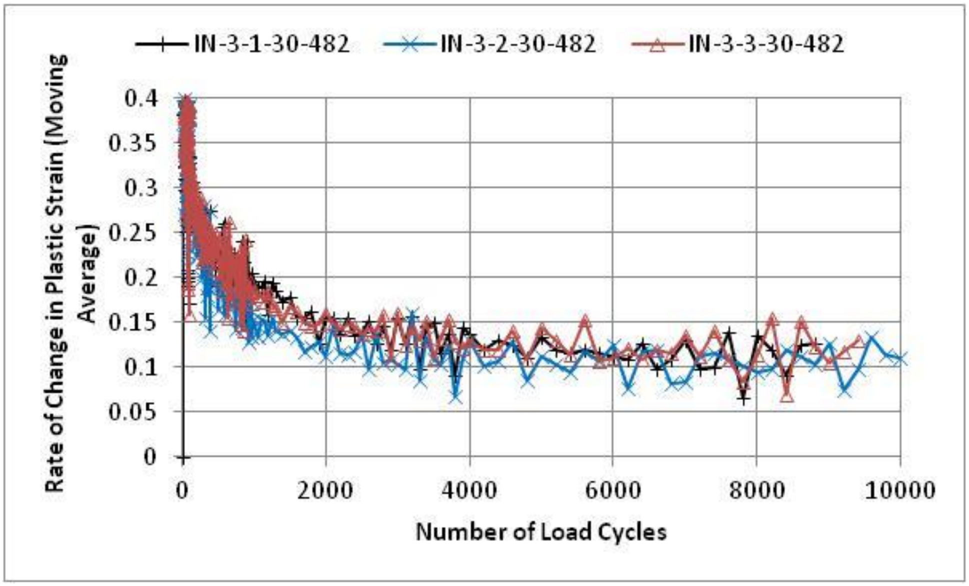
Appendix A
Repeated Load Triaxial Test Examples

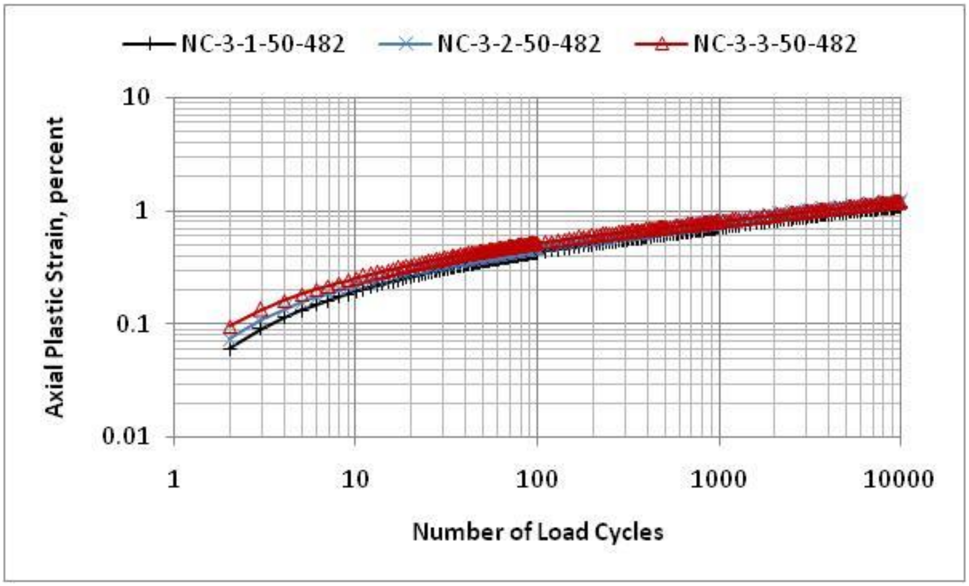


(a.) Test Results from the Repeated Load Plastic Strain Test.

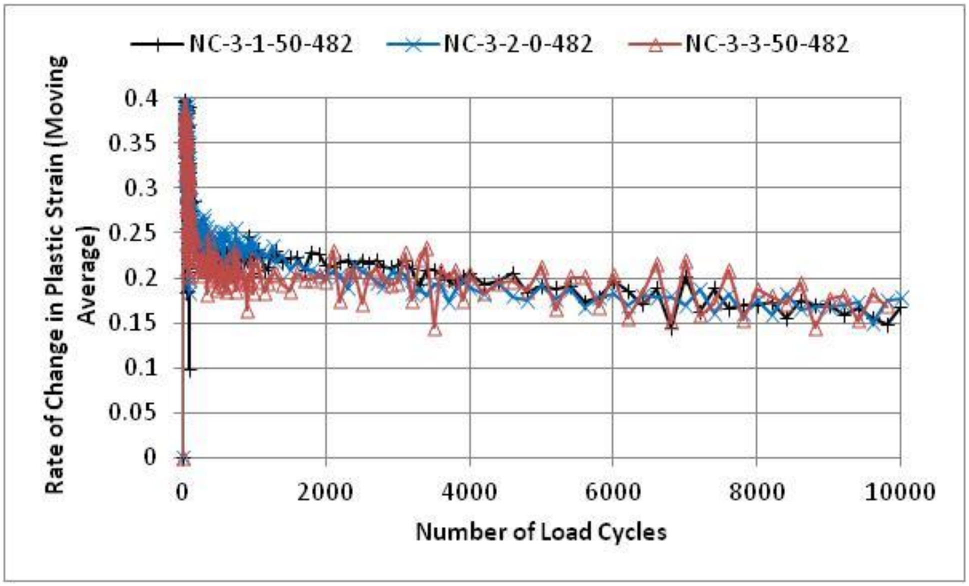


(b.) Average Slope or Rate of Change in the Plastic Axial Strain with Number of Load Cycles.

Figure A.1. Indiana Mixture

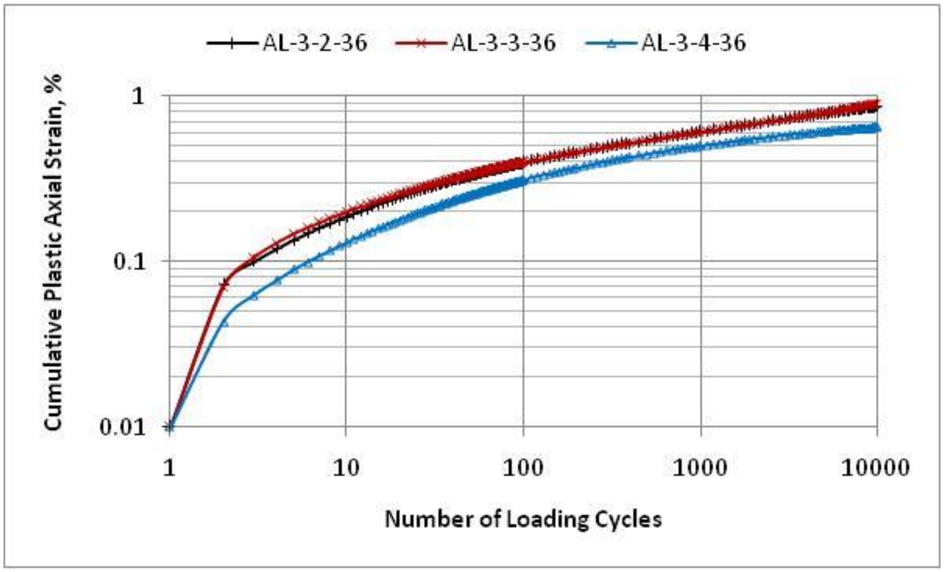


(a.) Test Results from the Repeated Load Plastic Strain Test.

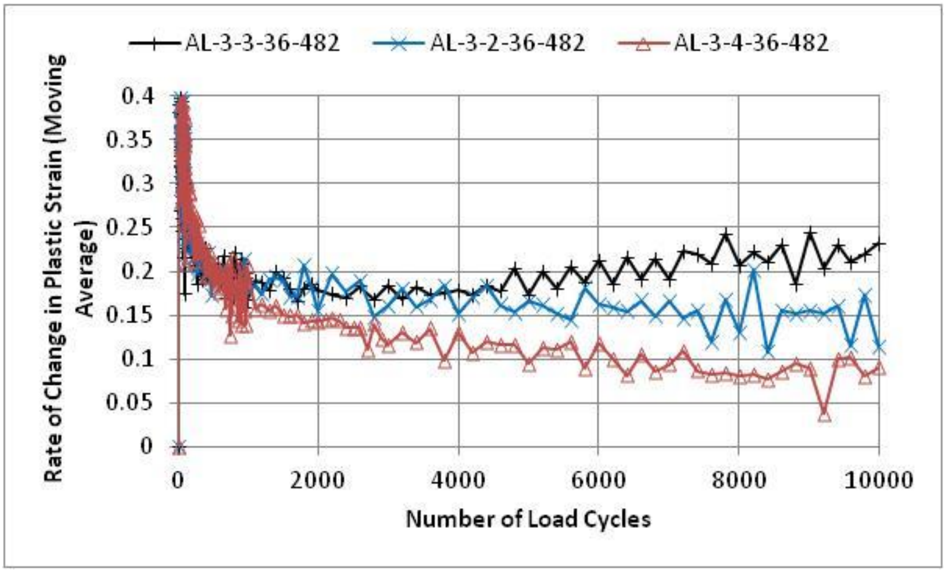


(b.) Average Slope or Rate of Change in the Plastic Axial Strain with Number of Load Cycles.

Figure A.2. North Carolina Mixture

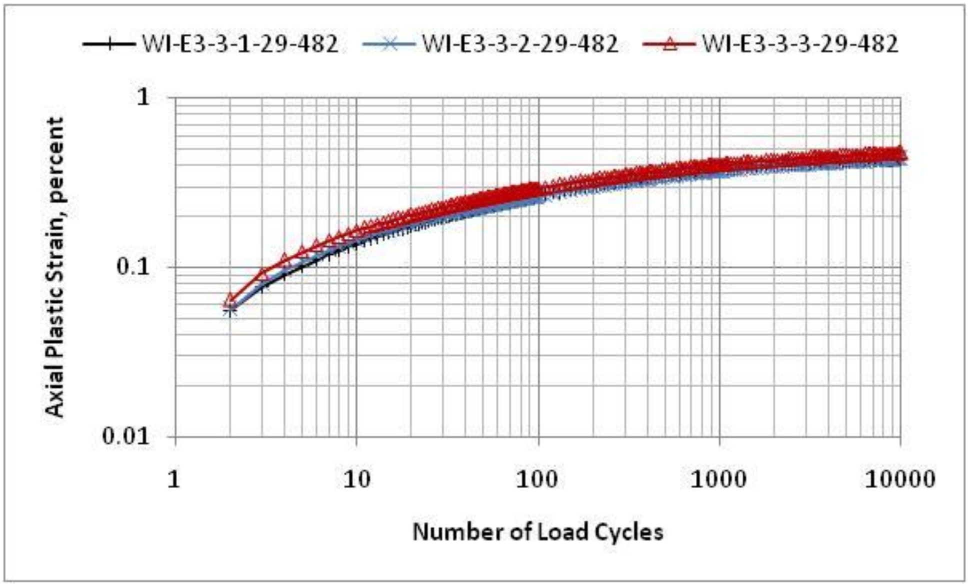


(a.) Test Results from the Repeated Load Plastic Strain Test.

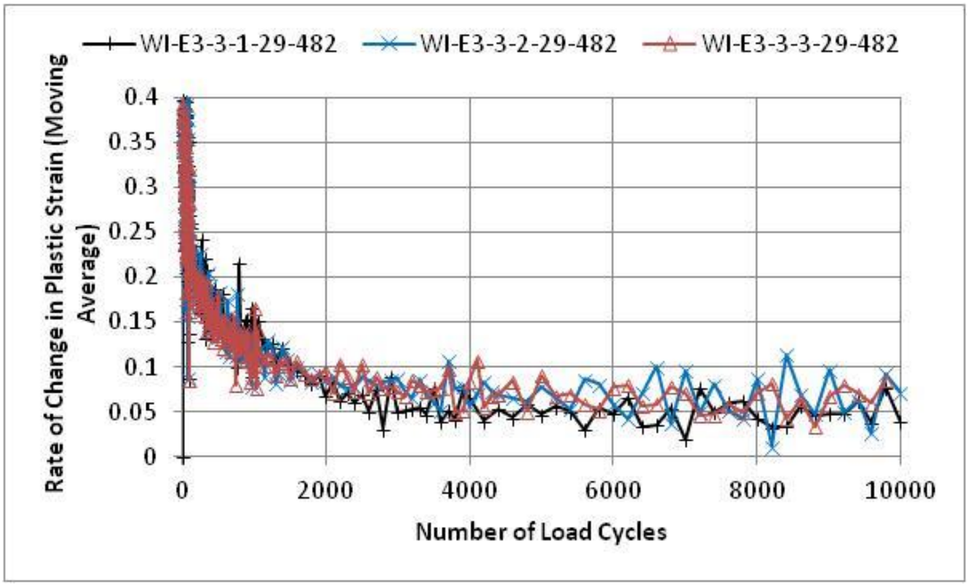


(b.) Average Slope or Rate of Change in the Plastic Axial Strain with Number of Load Cycles.

Figure A.3. Alabama Mixture



(a.) Test Results from the Repeated Load Plastic Strain Test.



(b.) Average Slope or Rate of Change in the Plastic Axial Strain with Number of Load Cycles.

Figure A.4. Wisconsin E3 Mixture; Monroe County

Appendix B
Estimating the Plastic Strain Coefficients from Volumetric Properties

Appendix B provides a procedure to estimate the plastic strain coefficients from the volumetric properties of the asphalt concrete mixture. Section B.4 defines those properties that are used to estimate the laboratory-derived intercept and m-value from repeated load plastic strain tests. This procedure follows the method included in NCHRP Report #719.

B.1 Intercept of Transfer Function

B.1.1 The field adjusted, laboratory-derived intercept is required to estimate the transfer function intercept for each transfer function. The recommended relationships to estimate the laboratory-derived intercept from the secondary region of triaxial tests is provided below.

$$I_{Triaxial} = 10^{-3.6}\left(\frac{V_a}{V_{Design}}\right)^{0.52}\left(Log(VFA)\right)\left(F_{Index}\right)\left(C_{Index}\right) \qquad \text{(B.1)}$$

Where:

V_a = In place air voids of the HMA layer, percent.
V_{Design} = Design air void level for selecting the target asphalt content, percent.
VFA = Voids Filled with Asphalt, percent.
C_{Index} = An index number related to the fine aggregate angularity (FAA) of the combined aggregate blend; refer to Table B.1 for the recommended values.
C_{Index} = An index number related to the coarse aggregate angularity (CAA) of the combined aggregate blend; refer to Table B.1 for the recommended values.

B.1.2 The values from equation B.1 are entered in Figures 5 and 6 in the test method to estimate the field matched intercept for the Kaloush transfer function. The field matched values are multiplied by the thickness adjustment factors provided in Table 1 within the test method for determining the inputs to the MEPDG software.

Table B.1. Aggregate Properties for Determining the Mixture Adjustment Factors

<table>
<tr><th rowspan="2">Fine Aggregate</th><th rowspan="2">Gradation</th><th colspan="5">Fine Aggregate Angularity; AASHTO T 304</th></tr>
<tr><th colspan="2"><45</th><th colspan="3">>45</th></tr>
<tr><td rowspan="2">FAA Index Value</td><td>External to Restricted Zone</td><td colspan="2">1.0</td><td colspan="3">0.9</td></tr>
<tr><td>Through Restricted Zone</td><td colspan="2">1.05</td><td colspan="3">1.0</td></tr>
<tr><th rowspan="2">Coarse Aggregate</th><th rowspan="2">Gradation</th><th colspan="5">Percentage Coarse Aggregate with Two Crushed Faces; AASHTO TP 61</th></tr>
<tr><th>0</th><th>25</th><th>50</th><th>75</th><th>100</th></tr>
<tr><td rowspan="2">CAA Index Value</td><td>Well Graded</td><td>1.1</td><td>1.05</td><td>1.0</td><td>1.0</td><td>0.9</td></tr>
<tr><td>Gap Graded</td><td>1.2</td><td>1.1</td><td>1.05</td><td>1.0</td><td>0.9</td></tr>
</table>

B.2 m-Value of Transfer Functions

B.2.1 The relationship for estimating the m-value for dense-graded designed aggregate blends is provided in equation B.2.

$$m-Value_{Neat}=0.265\left(\frac{P_b}{P_{b(Opt)}}\right)^{0.75} \qquad \text{(B.2)}$$

Where:

P_b = Asphalt content by weight at construction (the in place value), percent.

$P_{b(Opt)}$ = Saturation or optimum asphalt content by weight, percent. This parameter defines the asphalt content at which the VMA starts to increase or the density of the mixture starts to decrease. An example and demonstration in determining this variable is provided in Section B.4 in this appendix.

B.2.2 For the use of modified asphalts, the m-value for neat asphalt mixtures is adjusted by equation B.3:

$$m-Value_{Modified}=m_b\left(m-Value_{Neat}\right) \qquad \text{(B.3)}$$

Where:

m_b = An adjustment that accounts for the use of modified mixtures for the same aggregate blend of neat asphalt mixtures and defined below.

For m-values less than or equal to 0.2: $m_b = 1.0$.

For m-Values greater than 0.2: $m_b = 0.072+\left(m-Value\right)0.64$ (B.4)

B.3 Temperature Term Exponent of Kaloush Transfer Function

B.3.1 The Kaloush transfer function is the only one of the three recommended for use that includes temperature as a dependent variable. The temperature exponent should be set to 1.5606.

B.4 HMA Properties Used in Determining Level 2 Inputs

B.4.1 Design Air Void Content to Select Target Asphalt Content, $V_{a(design)}$

B.4.1.1 This parameter is determined from mixture design charts (air voids as a function of asphalt content), and is the air void content at the target asphalt content (or the value expected during production of the mixture). Figure B.1 shows an example in determining this value or parameter for a specific mixture. The reality of this parameter is dependent on how close the laboratory compactive effort simulates the field compaction that occurs under the rollers and truck traffic.

B.4.1.2 In most cases, the HMA mixture design will be unavailable when the structural design is completed. In this case, it is recommended that an agency's policy on design air void content be used – this will be 4 percent in most cases. However, some agencies now use 3 and 5 percent for some of their mixtures to select the asphalt content for production.

B.4.2 Saturation Asphalt Content by Weight, $P_{b(sat)}$

B.4.2.1 This parameter is determined from the mixture design charts (mixture density as a function of asphalt content), and is the asphalt content where the density begins to significantly decrease or where the VMA begins to significantly increase. This value is determined in the laboratory and is not a well defined parameter. Figure B.2.a shows an example of a sensitive HMA mixture in determining this value, while Figure B.2.b shows an example for a non-sensitive mixture.

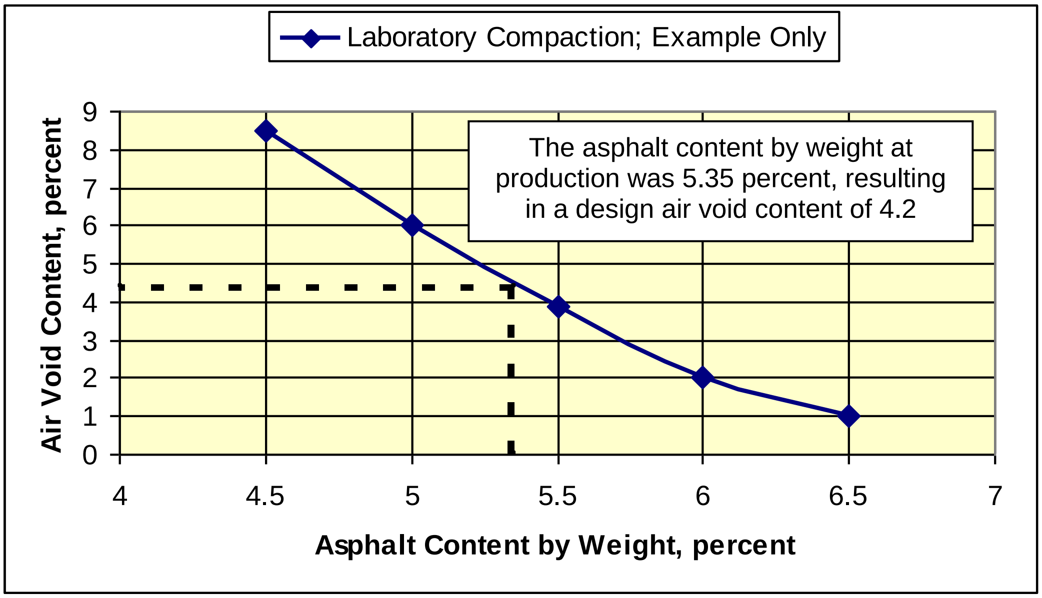


Figure B.1. Graphical Example Determining the Design Air Void Content from the Laboratory Mixture Design Chart

B.4.2.2 In most cases, the saturation asphalt content will be unknown when the structural design is completed. In this case, it is recommended that previous mixture design records be reviewed to select a reasonable ratio between the target asphalt content and saturation asphalt content by weight. This value generally varies from 0.90 to 1.0 for HMA mixtures that are resistant to rutting. The asphalt content where the density begins to significantly decrease and VMA begins to significantly increase should never be selected for mixtures to be placed on higher volume roadways.

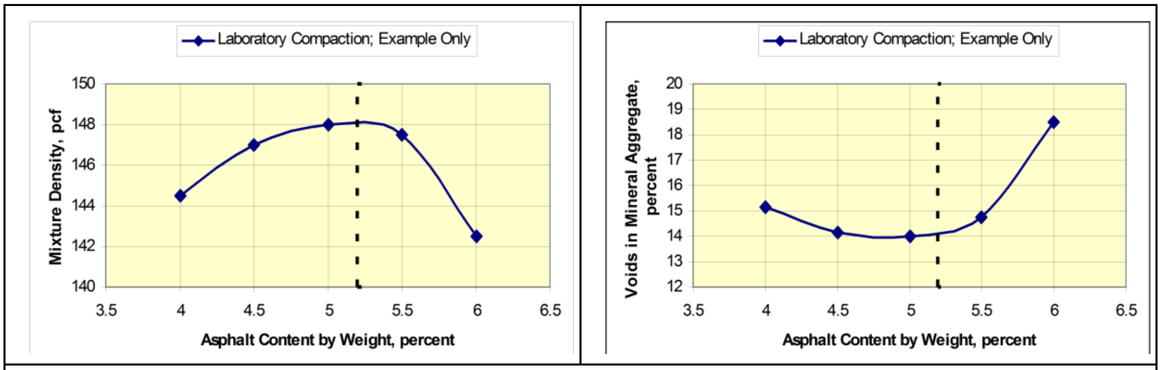
B.4.2.3 This parameter is dependent on the laboratory compaction device and compaction effort used in the laboratory. The assumption is that the laboratory compaction device and effort accurately simulates the compaction from the rollers and truck traffic over time.

B.4.3 Fine Aggregate Angularity Index, F_{Index}

B.4.3.1 An index number or value related to the fine aggregate angularity (FAA) of the combined fine aggregate of a mixture (refer to Table B.1). The FAA index is entered into the MEPDG software. Most agencies measure the FAA value during mixture design and for aggregate source approval (AASHTO T 304).

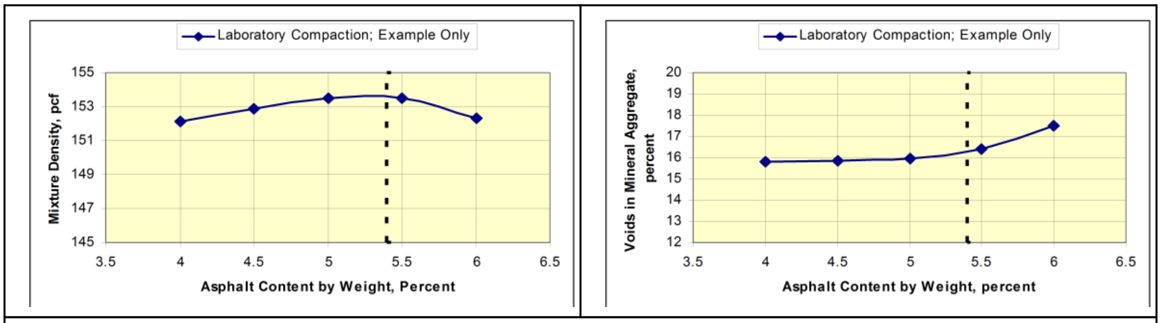
B.4.4 Coarse Aggregate Angularity Index, C_{Index}

B.4.4.1 An index number or value related to the coarse aggregate angularity (CAA) of the combined coarse aggregate of a mixture. Few agencies measure the CAA value in the laboratory (AASHTO T 326), but most do have required limits for the minimum amount of coarse aggregate with two crushed faces for varying truck volumes (AASHTO TP 61). For high volume roadways, most agencies require 100 percent crushed coarse aggregate. The C_{Index} value, as used in the MEPDG, is related to the amount of crushed coarse aggregate (refer to Table B.1). The CAA index is entered into the MEPDG software.



The saturation asphalt content is defined as 5.2 percent by weight for this example, because the mixture density begins to significantly decrease and the VMA starts to increase at about that value.

Figure B.2.a. Example for a Sensitive Mixture



The saturation asphalt content is defined as 5.4 percent by weight for this example, because the VMA starts to increase at about that value. The HMA density also starts to decrease at asphalt content slightly above that value.

Figure B.2.b. Example for a Non-Sensitive Mixture

Figure B.2. Graphical Example Determining the Saturation Asphalt Content from the Laboratory Mixture Design Chart

B.5 Estimation of Repeated Load Plastic Deformation Parameters─Input Level 2

B.5.1 The following provides a few notes that should be remembered regarding use of the mixture adjustment factors in predicting rutting of HMA mixtures.

1. These mixture adjustment factors were not optimized in terms of the minimizing the residual errors of the predicted rut depths.
2. The ratio of the actual asphalt content to saturation asphalt content by weight should be less than 1.1 and greater than 0.90. It is possible that this ratio can be greater than 1.1 and less than 0.90 for some mixture designs, because of the differences in compaction devices and compaction effort used in the laboratory. It is recommended that the range of this value be limited to the values listed above because too few data outside that range were included in the initial development of this factor.

3. The design air void content based on the actual asphalt content should be within the range of less than or equal to 5.0 and greater than or equal to 3.0 percent.
4. The C_{Index} values for 0 percent coarse aggregate with two crushed faces was estimated, because all mixtures included in the original evaluation to estimate the C_{Index} value were greater than 50 percent.

Appendix C
Procedure for Adjusting the Plastic Strain Coefficients for Different Equivalent Test Temperatures

Appendix C provides a procedure for making changes to the plastic strain coefficients for slightly different equivalent temperatures than were used in repeated load triaxial test. Specifically, this appendix under Option B (Equivalent Test Temperature option; Section 8.2.2) permits the slope and intercept to be corrected based on minor temperature differences between the equivalent and actual test temperatures. The corrections should be limited to temperature differences of no more than 10 °F. The following is a step by step basis for making those corrections.

C.1 The temperature influence on the intercept of the secondary region, everything else being equal, is defined by equation C.1:

$$Log(I_s) = Log(d) + nLog(T) \qquad \text{(C.1)}$$

Where:

I_s = Intercept of the secondary region; see Figures 4 and 5.
T = Test temperature, °F.
d, n = Constants (Figure C.1 shows the relationship between d and n for a range of mixtures, test procedures, and types of test specimens.

I_s is determined from repeated load triaxial plastic strain tests at the test temperature using the equivalent test temperature Option B, while T is the test temperature.

C.2 The second step is to determine the "*d*" value, which is calculated by equation B.2.

$$Log(d) = Log(I_s) - nLog(T) \qquad \text{(C.2)}$$

I_s is measured from the repeated load plastic strain test and T is the test temperature for that test. The value of "*n*" is assumed to be the average value from similar mixtures tested at three test temperatures or Option A in the test method. Figure C.1 shows the relationship between "*n*" and "*d*" for a wide range of mixtures, test procedures, and type of test specimen. For this evaluation, a constant value of "*n*" was assumed to be 2.0, which represents the mid-range of values shown in Figure C.1 for reconstituted, laboratory compacted test specimens of the repeated load triaxial tests.

C.3 The third step is to calculate the I_s value for the specific equivalent annual temperature using equation C.1, but with the calculated "*d*" parameter from step b and the actual equivalent annual temperature for the specific location, design period, and HMA thickness.

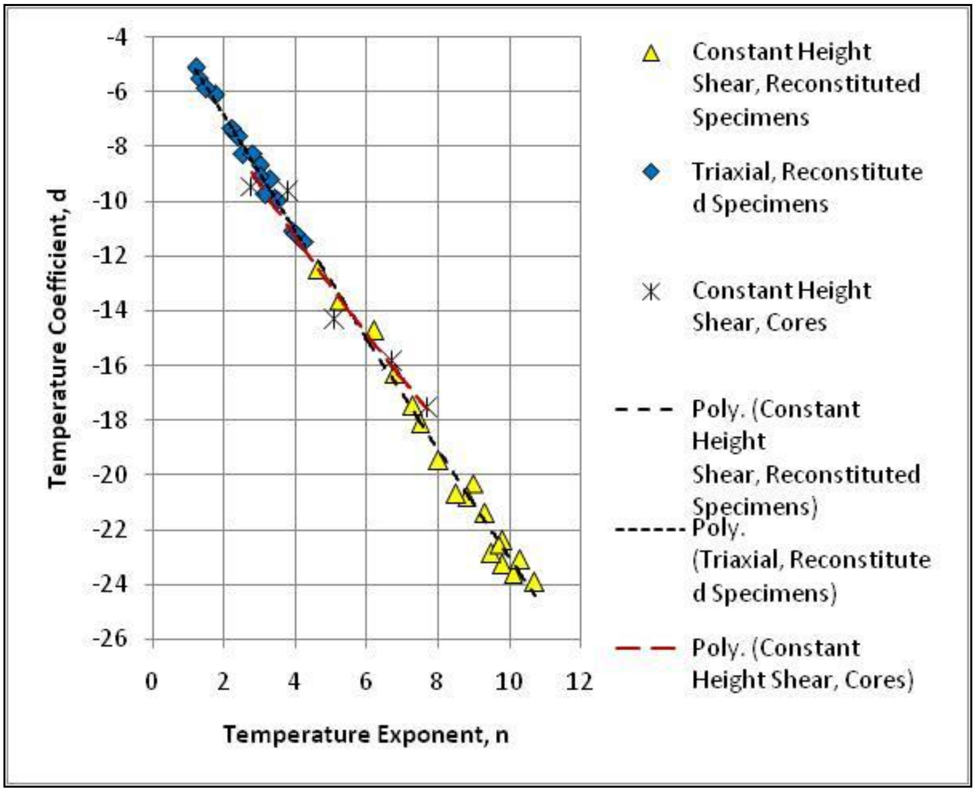


Figure C.1. Relationship of the Temperature Exponent (n-value) and Coefficient (d-value) for Different Repeated Load Tests; extracted from NCHRP Report #719