GEORGIA DOT RESEARCH PROJECT 14-12

FINAL REPORT

Effects of Reclaimed Asphalt Pavement (RAP) Contents and Sources on Dynamic Modulus (|E*|) and Fatigue Performance of Asphalt Mixtures in Georgia

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16. Abstract:

This report presents the effect of RAP contents and sources on the dynamic modulus and the performance of Georgia asphalt concrete mixtures. Asphalt concrete mixtures were prepared based on two Job Mix Formulas from North and South with 12.5mm nominal maximum aggregate sizes and three asphalt binders (PG 64-22, PG67-22, and PG 76-22). Dynamic modulus tests and controlled crosshead cyclic tension fatigue tests were performed for the prepared specimens and predicted performance by LVECD and Pavement ME programs to investigate the effect of RAP contents and sources on mixture characteristics. Analyses reveals that Superpave mixtures with higher PG binder and increased RAP content (up to 30% RAP) result in higher dynamic modulus as the mixtures become stiffer. S-VECD and LVECD analyses shows that the addition of RAP up to 25 percent using GDOT's COAC method significantly improved the mixtures' fatigue resistance, especially in the mixtures with binder grades of PG 64-22 and PG 67-22 since the RAP mixtures are likely to contain more binder than the virgin mixes in accordance with the COAC method. Further, Pavement ME analyses were conducted for 25% RAP mixtures with PG64-22 and PG 67-22. Based on Pavement ME analyses, it was concluded that the half-grade change of binder (PG64-22 and PG67-22) would not induce significantly different performance albeit rut depth prediction indicates some variation.

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Effects of Reclaimed Asphalt Pavement (RAP) Contents and Sources on Dynamic Modulus (|E*|) and Fatigue Performance of Asphalt Mixtures in Georgia

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EXECUTIVE SUMMARY

Use of 25-30% Reclaimed Asphalt Pavement (RAP) in asphalt mixture is common practice in Georgia. With more RAP content in HMA, an associated concern emerges because adding RAP to a virgin mixture generally increases the stiffness and brittleness (cracking) of mixture. Therefore, it is important to obtain the RAP mixture properties that will be used in constructing pavement in advance and take it into consideration during the design process of asphalt concrete pavement. Thus, pavement engineers at GDOT need to obtain the RAP mixture properties that are necessary for MEPDG pavement design from assumed RAP contents and sources. To meet GDOT's needs, this study aims to investigate the effects of various RAP contents and sources on E* and cracking performance of asphalt mixture.

Asphalt concrete mixtures were prepared based on two Job Mix Formulas from North and South with 12.5mm nominal maximum aggregate sizes and three asphalt binders (PG 64-22, PG67-22, and PG 76-22). Dynamic modulus tests and controlled crosshead cyclic tension fatigue tests were performed for the asphalt mixtures. Based on the test results, performance of mixtures was predicted using LVECD and AASHTOWare Pavement ME programs to investigate the effect of RAP contents and sources on mixture characteristics.

Analyses revealed that Superpave mixtures with higher PG binder and increased RAP content (up to 25% RAP) result in higher dynamic modulus as the mixtures become stiffer. S-VECD and LVECD analyses shows that the addition of RAP up to 25 percent using GDOT's COAC method (specified in GDOT Specification Section 828 and SOP 2) significantly improved the mixtures' fatigue resistance, especially in the mixtures with binder grades of PG 64-22 and PG 67-22 since the RAP mixtures are likely to contain more binder than the virgin mixes in accordance with the COAC method. No significant difference of fatigue resistance was observed between PG 64-22 and PG 67-22. However, the use of PG 76-22 binder seems to improve the fatigue cracking resistance compared to the other binder grades. Also, it is observed that two different RAP sources and mixing plants are likely to produce similar mixtures in GA.

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1. INTRODUCTION

1.1 Problem Statement

Recycled asphalt pavement (RAP) is considered a viable alternative to virgin materials because it reduces the need for both virgin aggregate and the amount of new asphalt binder, which is the most expensive component in asphalt concrete (Mogawer et al., 2012). RAP has become an increasingly attractive material to state highway agencies because asphalt binder costs have increased over the last few years. In each year, approximately 100 million tons of asphalt concrete are reclaimed and about 80% of them is reused as RAP. In Georgia, there are approximately 150 approved RAP stockpiles by Georgia Department of Transportation (GDOT), owned by 26 different contractors across the state. Because of economic savings and environmental benefits, the GDOT's specifications allow up to 40% RAP for drum plants and 25% RAP for batch plants. Use of 25 - 30% RAP in GDOT approved mixtures is most common. While some evidence points to mixes with 25% RAP having similar |E*| as virgin mixtures, the recent NCHRP Report 752 shows that RAP content and source significantly affect |E*| and pavement performance. Since |E*| is a direct input for flexible pavement design and performance evaluation, it is important to evaluate the effects of RAP content and source on |E*| for reliable new and overlay pavement designs.

This concern originates from the fact that the addition of RAP to a virgin mixture increases the stiffness of the mixture because the aged binder that is used in RAP mixtures makes RAP mixtures stiffer and more brittle than non-RAP mixtures resulting in premature cracking that may occur in flexible pavements containing high percentage of RAP due to traffic loads and thermal stresses. With more RAP content in HMA, some associated concerns also emerged. Since RAP is highly variable in its size distribution, composition, and properties, uniformity has been one of the concerns about incorporation of more RAP in HMA. Unfortunately, pavements are often designed a year or more before contractors ever see the plans and prepare their bids. The RAP that will be used in constructing a pavement has not even been generated or has been approved on a "continuous" basis where the AC content and gradation must meet certain parameters. Therefore, there is no way a pavement designer can know what the RAP mixture properties will be as he/she designs the pavement. This situation warrants the development of a RAP material database that can be used by pavement designers in GDOT to quickly obtain the RAP mixture properties that are necessary in the pavement design from assumed RAP content and source.

RAP has both advantages and disadvantages that must be considered for its appropriate usage as a paving material (Norouzi et al., 2014). Based on the aforementioned concerns and information, the main purpose of this study is to evaluate the effect of RAP content and binder grade on the fatigue performance of asphalt mixtures via laboratory characterization tests and to predict pavement performance trends using the Layered Visco-Elastic pavement analysis for Critical Distresses (LVECD) program. To this end, dynamic modulus and fatigue tests were conducted in order to evaluate and compare the performance of mixtures with different binder grades and RAP contents. Dynamic modulus tests were conducted to provide linear viscoelastic properties such as stiffness along with temperature, frequency, and time-temperature shift factors. Direct tension cyclic fatigue tests were conducted to evaluate the performance of the mixtures following the Simplified Visco-Elastic Continuum Damage (S-VECD) model protocol. The results of these two tests provided inputs for the LVECD program to evaluate pavement performance by simulating actual field conditions such as climate, loading, pavement structures, and so forth.

1.2 Objectives

The objectives of this study are: (1) to investigate the effects of various percentages of RAP and its sources on E* and cracking performance of asphalt mixture, and (2) to develop a RAP material database for pavement designs. This study will allow the GDOT to use RAP mixtures more effectively with more confidence and knowledge, potentially leading to longer lasting and reduced associated maintenance costs. In addition, a |E*| material database will be available to the GDOT for the Mechanistic-Empirical (ME) pavement designs.

2. LITERATURE REVIEWS

2.1 Effect of RAP Content on Asphalt Mixture Performance

There have been several research projects on high RAP mix design with balance performance. The purpose of those studies was to evaluate the impact of RAP on optimum asphalt content, moisture resistance, and cracking resistance while proposing a solution to the concern related to the RAP variability. Zhou et al. (2011) conducted a research on high RAP content mixes. Testing procedures such as aggregate gradation for the material variability, Hamburg Wheel Tracking Test (HWTT) for the rutting/moisture resistance and Overlay Test (OT) for the cracking resistance were performed. The research highlights that an increase in RAP content enhances the moisture resistance and causes a significant increase of OAC (optimum asphalt content) from a RAP content of 20%. However, a reverse reaction was observed with cracking resistance from 30% RAP content or a combination RAP/RAS. In their study, a RAP mix design methodology with balancing rutting/moisture resistance and cracking resistance was developed to overcome the challenges presented by high RAP mixes (>25%) such as virgin and RAP binder blending, bulk specific gravity of RAP aggregates, RAP handling, and mixing and compaction temperatures. The methodology proposed the use of balanced mix design approach to obtain the OAC through target air voids, and moisture and cracking resistance through OT and HWTT analysis. For the RAP handling, Zhou et al. (2011) adopted a two-step procedure with a warm up of the RAP materials overnight, then a preheating of the RAP at the mixing target temperature for 2 hr. As for the mixing and compaction temperature, they advised the use of the mixing and compaction temperatures corresponding to virgin binder for RAP mixes design to get a better cracking resistance due to higher OAC for RAP mixes and preservation of virgin asphalt binder (no over age issue).

In 2010, the National Center for Asphalt Technology (NCAT) edited a methodology to design high RAP content mixes (NCAT, 2010). Based on currents RAP mix design guidelines analysis, the NCAT emits recommendations to improve the cracking resistance of the Superpave through a more accurate and faster mix process. The methodology presented was detailed as follows:

- A procedure of Sampling and testing the RAP finalized by the determination of the RAP's aggregate bulk specific gravity (Gmb) through maximum theoretical specific gravity (Gmm) method.
- The RAP handling process which includes a drying of the RAP during overnight aeration with fans then preheats in oven for a 1 hour at 230°F and batching the RAP by developing trials blends after screening down to the 4.75 mm sieve. The batching RAP is then heated to the mixing temperature for 2 to 4 hours.
- The high RAP content mix design calibration where the optimum binder content was determined were estimated before performing the appropriate performances test.
- Conduct appropriate performance tests depending of the input conditions or the studied parameters.

The definition of the continuum damage level at which the material is considered as failed is an important parameter to describe the asphalt mix property. However, the definition of the failure criterion of the fatigue cracking under traffic loading was very controversial if not limiting.

Zhang et al. (2013) found a way to predict a target indicator consistently and with accuracy via an energy-based theory. Their study was based on four (4) 9.5 mm Superpave surface mixtures from different sources, which properties are shown in Table 1.

Table 1. Summary of Mixture Properties

Mix Type	Binder	AC (%)	NMAS (mm)	RAP (%)	Target air void (%)
S9.5C	PG70-22	5.20	9.5	0	5.5
VTe00LC	PG64-28	6.50	9.5	0	6.0
VTe30LC	PG64-28	6.61	9.5	30	6.0
VTe40LC	PG64-28	6.55	9.5	40	6.0

Their analysis underlines a unique relationship between the stable rate of the pseudo strain energy release (G^R) and the fatigue life, N_f. The interaction highlighted implied that, whatever the loading condition, determining the rate of damage accumulation induces the automatically knowledge of the corresponding fatigue life for a given mixture. The statistical analysis of the data showed an overall linear regression relationship between (G^R) and the fatigue life, and between the measured Stable rate G^R (via experiment) and the predicted Stable rate G^R. The stable energy release rate, G^R is obtained through the total released pseudo strain energy which relies on the variation in the pseudo stiffness. Zhang et al. (2013) proved that the percentage of RAP content does not influence the above proved relationship between G^R and N_f. In addition, they showed that a calibration tests at only one temperature is sufficient to obtain its parameters because of the unicity of the relationship. The application of this energy-based failure criterion is a key for a significant improve of the existing viscoelastic continuum damage model dedicated to the prediction of the asphalt mixture fatigue life. In addition, it's less expensive testing due to the unicity of the founded relationship is appealing.

The Federal Highway Administration (FHWA) observed that more than 90% of the roads in the USA are part of asphalt mixtures with RAP content (FHWA 2011). FHWA and the Transportation Research Board (TRB) edited publications detailing the actual practices and testing of the Recycled Asphalt pavement in asphalt mixtures (FHWA 2011, TRB 2014). The FHWA report (2011) was a summary of the current practices of RAP in asphalt mixture in US and describes a step by step procedure, detailing the chronology of the tasks, from the RAP source, its storage and handling, collecting and processing to its mix to the asphalt through the HMA process and thus along with the tests description and their standard. The TRB report (TRB 2014) is the summary of a workshop providing a forum to exchange finding of research on RAP and RAS pavement performance. In accordance with the TRB report, the last seven-years have seen a sensible evolution of the use of RAP content in asphalt mixture in United States. That evolution was observed through the qualitative and quantitative increase of the annual used tonnage. To substantiate their assumption, they explained how the last surveys showed an increase of the average RAP content of asphalt mixture from 12% to 17% beside the increase of the annual tonnage and the progressive introduction of the Recycled Asphalt Shingles (RAS).

For the mix design method and the testing method, the NCHRP Project 9-46 (TRB 2013) defines the current guidelines for using RAP in Superpave mixes. All currently used methods in

US were briefly described and then the most common were highlighted. Further, the standards of the different experiments needed to meet the requirement of the mix design were explained. For example, for the RAP parameters, the standards are AASHTO T 308 & T164, respectively, for the ignition method and the solvent extraction method for the RAP binder removal. As for AASHTO T85 and T84, it indicated to be the standard for the specific gravities of the coarse and fine materials recovered from the RAP aggregates. Moreover, the Gmm test which is needed to determine the bulk specific gravity and the VMA of the mixture follows the testing standard of AASHTO T 209. The virgin binder grade selection was proposed to follow the RAP Binder Ratio (RAPBR) method as a simple way of estimating the total binder needed in the mix based on the RAP content; however, the latest recommendations was to adapt to environmental requirements and determine adapted temperatures of the chosen binder.

As for the testing, the parameters requested for the performance testing was defined. The different methods used and the standard followed as well as the concerns and common points of the methods were presented. A summary of the testing conclusions is shown in Table 2.

Table 2. Asphalt Mixture Performance Test Results

<table>
<tr><th colspan="4">Testing Methods and Standard</th><th rowspan="2">Conclusion</th></tr>
<tr><th colspan="2">Testing Parameters</th><th>Method 1</th><th>Method 2</th></tr>
<tr><td rowspan="5">Tested Parameters</td><td>Moisture Damage Susceptibility</td><td>Tensile Strength Ratio (TSR) AASHTO T283</td><td>Hamburg Wheel Tracking Test AASHTO T324</td><td>The conditioned and unconditioned tensile strengths of the high RAP content mixes exceeded those of the virgin mixes from the same materials source.</td></tr>
<tr><td>Rutting Susceptibility</td><td>Loaded wheel test AASHTO TP 63</td><td>Hamburg Wheel Tracking Test AASHTO T325</td><td>The mixtures dynamic modulus are less than the high RAP mixtures in the same testing conditions.</td></tr>
<tr><td>Dynamic Modulus</td><td colspan="2">AASHTO TP 62</td><td>The virgin mixtures dynamic modulus are less than the high RAP mixtures in the same testing conditions.</td></tr>
<tr><td>Fatigue Cracking</td><td colspan="2">Indirect Tension Fracture Energy Test</td><td>Fracture energy decreased as the RAP content increased.</td></tr>
<tr><td>Low-temperature cracking</td><td colspan="2">Semi-Circle Bend (SCB) test</td><td>RAP content increased. Fracture toughness of the mixtures increased but fracture energy decreased. Critical thermal cracking temperature dominated by the virgin binder critical low-temperature grade.</td></tr>
</table>

To conclude, the authors indexed the NCHRP Report 752 finding as a big step toward an advancement of RAP usage and recommended to identify practical laboratory Tests which results could be realistic in field performance. Further, the Transportation Research Board discussed about the Recent Advances in Field Evaluation of RAP in US and the current practices in others countries abroad.

Tomlinson (2012) studied the effects of RAP content and asphalt binder on the dynamic modulus and fatigue life of asphalt concrete because of the concern of potential increase of fatigue cracking in pavement due to the addition of RAP to asphalt concrete. To achieved the purpose, the study focused on two different RAP content (20% & 40%) at three different asphalt content (plant mix, plant-mix + 0.5%, and plant mix + 1.0%). The Input parameters of the study were: Air void (AV), RAP, Binder, Temperature and Frequency. The Dynamic Modulus testing was conducted at six frequencies (25, 10, 5, 1, 0.5 and 0.1 Hz) for four (4) different temperatures (40, 70, 100 and 130°F). As for the fatigue beam testing, it was performed at a controlled-strain condition, with a strain level of 400$\mu\varepsilon$ at a frequency of 10 Hz in ambient air of 20°C (68°F) and with a target air void content of 7.0 ± 0.5 AV for tested samples. The testing failure criteria was 50% reduction in the specimen's initial stiffness and the initial stiffness of the beam was considered at the 50th cycle to allow for preconditioning of the beam. Based on the ANOVA analysis, the Dynamic Modulus and Fatigue cracking models were influenced by all of the input parameters identified, except the air void which influence was statistically insignificant. Through the data analyses, it was found that both the percentage of RAP and asphalt content had a significant effect on the dynamic modulus and fatigue life of asphalt concrete. Moreover, additional valuable information came up which will be listed below:

a. Fewer numbers of gyrations are needed to reach 7.0% AV in the asphalt concrete with higher RAP content (40% RAP).

b. The increase of binder content induces a small difference in AV between the original sample and the cut and cored.

c. Based on statistical analyses, dynamic modulus increases with the increase of RAP content or frequency.

d. The increase of the binder content improves the fatigue resistance for the mixture with 20% RAP.

3. MATERIALS AND SAMPLE FABRICATION

The materials for this study were obtained from the HMA production plants of two GDOT Highway Contractors, Plant A and Plant B. The selection of Plant A and Plant B was primarily for the locations, one in the South and one in the North, and for representing different aggregate sources.

3.1 Gradations

Figure 1 illustrates blended gradation of each mixture according to Nominal Maximum Aggregate Size (NMAS) and sources. Mixture gradation between different sources (i.e., Plant A and Plant B) seems similar. After sieving individual stock piles, aggregates were batched to reproduce the gradations shown in Figure 1 which is same as mixtures produced at mixing plant.

3.2 Asphalt Binder Type

Georgia has historically used Performance Grade (PG) 67-22 asphalt binder for highways having low to intermediate traffic loadings, but in the last few years this grade has become less available. Recently, the GDOT has allowed the substitution of the more readily available asphalt binder of PG 64-22 in state asphalt mixtures. Since both PG 64-22 and PG 67-22 mixtures are now allowed to contractors, it is necessary to investigate the influence of a new binder mixture on asphalt pavement.

The effect of grade difference of asphalt binder on mixture performance can be evaluated through systematic laboratory testing. High PG is likely related to high temperature characteristics of asphalt mixtures such as rutting, whereas low PG is associated with low temperature behavior of mixtures such as fatigue cracking and thermal cracking (Asphalt Institute, 2003). In allowed binders, high PG is different but low PG is kept the same. In this regard, it can be expected that high temperature behavior would change but low temperature behavior would not change a lot. However, binder PG is determined by a certain range of criteria, so binders' behavior may differ within the specified criteria in spite of the same binder PG.

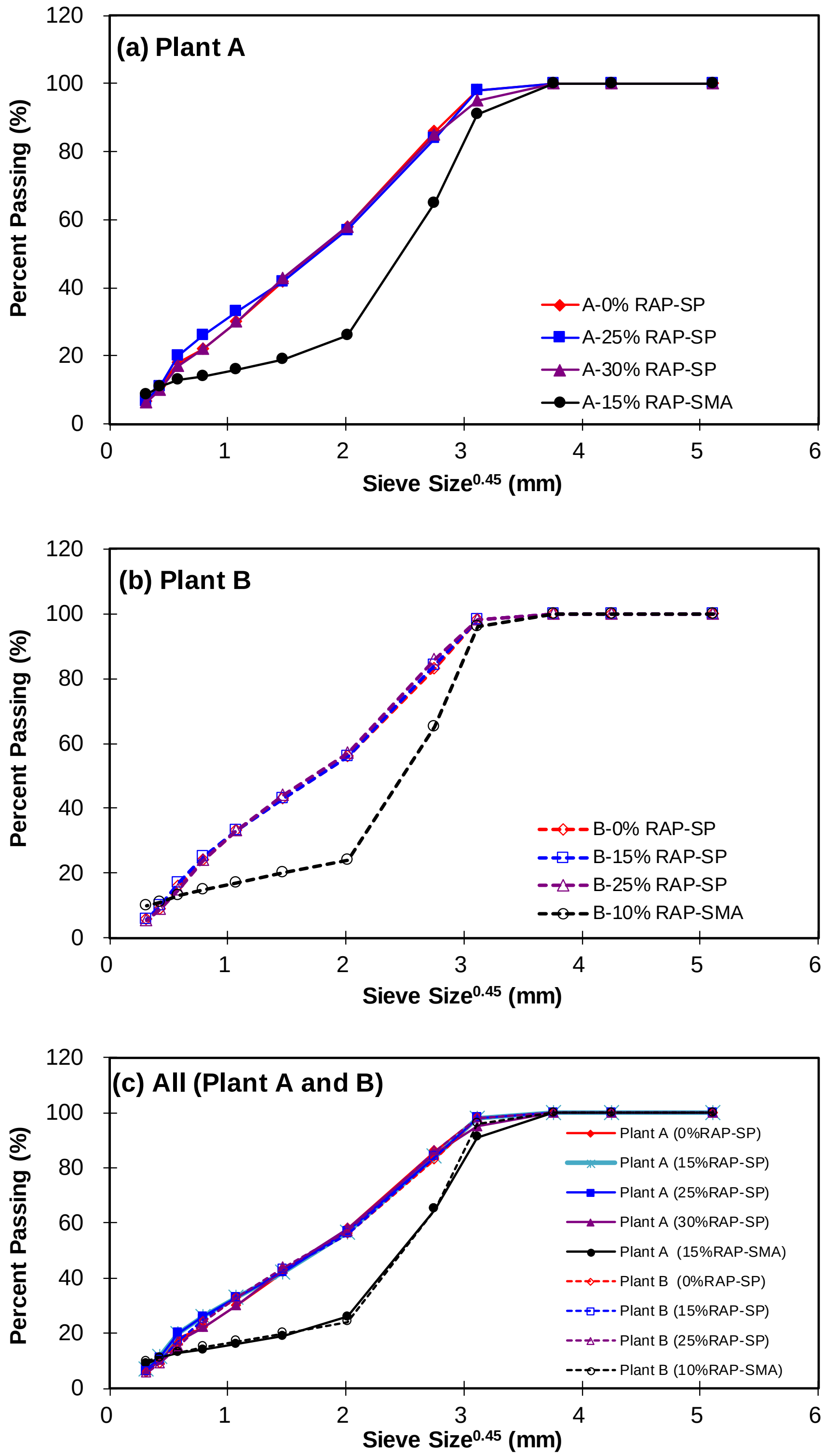


Figure 1. Gradation of mixtures in: (a) Plant A, (b) Plant B, (c) Plant A and Plant B.

For this study, three different binder types (e.g., PG 64-22, PG 67-22 and PG 76-22) were used to fabricated 12.5 mm Superpave mixtures while one binder type (PG 76-22) was used to fabricate 12.5 mm SMA mixtures. The effect of grade difference of asphalt binder on mixture characteristics was then evaluated through comparisons of dynamic moduli, fatigue test results, and performance analyses predicted by L-VECD program.

3.3 Corrected Optimum Asphalt Content (COAC)

Pavements are often designed a year or more before contractors ever see the plans and prepare their bids. Often, the RAP that will be used in a pavement has not even been generated. Therefore, there is no way a pavement designer can know what the RAP percentage or properties will be as he/she designs the pavement. While RAP stockpiles can be less variable than virgin aggregate stockpiles, increasing RAP percentage does not increase mixture variability (Kandhal et al., 1997).

Many studies have indicated a large amount of co-mingling of virgin and RAP asphalts (McDaniel et al., 2002; Mogawer et al., 2012) when RAP is introduced to HMA. However, Huang et al. (2005) found only 11% of RAP AC being transferred to virgin aggregate during the mixing process. GDOT also performed an in-house study of numerous RAP stockpiles around Georgia. In GDOT's study, virgin aggregate was heated above normal mixing temperatures and combined it with unheated RAP in a pugmill. The goal was to determine how much mass of AC would transfer to the virgin aggregate in a more real-world scenario. It was noticed that only occasional scuffing of the virgin material without any appreciable mass transfer or coating. GDOT then oven-heated samples of stockpiled RAP to observe the consistency and coating of the RAP aggregate. The AC was then removed from the samples in the ignition oven. Virgin AC was added back to the RAP aggregate in 0.25% increments until the original consistency and coating was achieved. The difference between the initial and recoated RAP AC percentages was calculated as the effective AC ratio in order to determine how much of the RAP AC was contributing to the effective AC content and AC film thickness. GDOT eventually settled on an average ratio of 75%, meaning that 75% of the AC in RAP was contributing to the effective AC in the mix. Minimum film thickness was set to 7 microns. This investigation led GDOT to develop the Corrected Optimum Asphalt Content (COAC) for asphalt mix designs. The COAC reflects the Original Optimum Asphalt Content (OAC) plus the addition of virgin AC in the amount of 25% of the RAP AC, and is typically used in production. More detailed information of COAC can be found in Georgia Standard Specifications Section 828 and SOP2 (GDOT 2013). The COAC calculation formula in SOP2 is presented in Appendix A.

Table 3 contains OAC and COAC of the mixtures. In Table 3, the COAC values were bolded when the GDOT approved JMFs are available. When the COAC was not available, same COAC along with same %RAP was used to study the effect of binder type on asphalt mixture performance. These asphalt contents have been determined by contractors working with GDOT and thus the COACs are also applied to this study in order to evaluate the effect of binder change on real mixtures used in GA. As shown in Table 3, it should be noticed that COAC increases as % RAP increases in accordance with COAC calculations.

Table 3. Corrected Optimum Asphalt Content

Mixture	%RAP	Binder	OAC(%)	RAP AC(%)	COAC(%)
Superpave	0%	PG64-22	5.30	N/A	5.30
		PG67-22	**5.30**	**N/A**	**5.30**
		PG76-22	5.30	N/A	5.30
	15%	PG64-22	5.10	5.05	5.30
		PG67-22	5.10	5.05	5.30
		PG76-22	5.10	5.05	5.30
	25%	PG64-22	4.96	5.05	5.52
		PG67-22	**4.96**	**5.05**	**5.52**
		PG76-22	**4.92**	**5.05**	**5.52**
	30%	**PG64-22**	**5.03**	**5.09**	**5.41**
		PG67-22	5.03	5.09	5.41
		PG76-22	**5.03**	**5.09**	**5.41**
SMA	0%	**PG76-22**	**6.20**	**N/A**	**6.20**
	15%	**PG76-22**	**6.45**	**5.28**	**6.65**
Superpave	0%	PG64-22	5.10	N/A	5.10
		PG67-22	**5.10**	**N/A**	**5.10**
		PG76-22	**5.27**	**N/A**	**5.27**
	15%	PG64-22	4.84	4.46	5.20
		PG67-22	**4.84**	**4.46**	**5.20**
		PG76-22	**5.30**	**4.46**	**5.47**
	25%	**PG64-22**	**5.00**	**4.80**	**5.30**
		PG67-22	5.00	4.80	5.30
		PG76-22	**5.07**	**4.98**	**5.38**
SMA	10%	**PG76-22**	**6.35**	**4.46**	**6.46**

Note: Bold when JMF was approved by GDOT

3.4 Specimen Fabrication

A 12.5 mm NMAS surface mixture was chosen for this study. GDOT's recommendation is use of 25-30% RAP for 12.5 mm surface mixture, but 0% and 15% RAP was included for this study to investigate the effects of RAP and AC contents on mixture performance. In addition, a 12.5 mm Stone Matrix Asphalt (SMA) mix with PG 76-22 binder were included in our testing plan for |E*| material input library. The JMF approved by GDOT were used for the fabrication of the non-RAP and RAP mixtures.

Mix design variables are: (1) two different RAP sources (Plant A and Plant B) for 12.5mm surface mix and SMA mix, (2) four different RAP contents (0%, 15%, 25%, 30%), (3) three different binder types (PG 64-22, PG 67-22, PG 76-22) for Superpave mix and one binder type (PG 76-22) for SMA mix. HMA specimens were fabricated with three replicates and three replicates for one mixture is believed to reduce testing variability and specimen variability.

The cylindrical specimens were fabricated in accordance with AASHTO PP 60-09 (AASHTO, 2009). Mixed materials were compacted by Superpave gyratory compactor by the dimension of 150 mm in a diameter and 170 mm height. The compacted specimen is cored and cut, so the specimen for dynamic modulus has diameter of 100 mm and height of 150 mm to get evenly-distributed air void throughout the specimen.

4. LABORATORY TEST

4.1 Dynamic Modulus

The GDOT is in the process of local calibration for AASHTOWare Pavement ME Design (hereafter referred to as Pavement ME) (Kim et al., 2013). As part of the procedure, the GDOT, along with many other states, has begun to create material characteristic libraries of typical asphalt mix properties, such as dynamic modulus (E*), to use in the Pavement ME designs. Dynamic modulus (|E*|) represents linear viscoelastic properties of asphalt concrete. Asphalt concrete is a viscoelastic material. |E*| illustrates basic features of asphalt concrete. Researchers have tried to relate |E*| to performance of asphalt pavements in order to develop a simple performance model. One good example is *Mechanistic-Empirical Pavement Design Guide* (Pavement ME) closed form solutions suggested under NCHRP 9-22 (Fugro 2011). |E*| is used as a main variable to predict rut depth and fatigue cracking in accordance with environmental change and volumetric properties of a mixture. With Pavement ME, performances (i.e., cracking and rut depth) are evaluated through measured |E*|, which is Level 1 input.

To determine the fundamental material properties of mixtures through dynamic modulus testing it is important to understand the principle of time-temperature superposition (t-TS). Generally speaking, the behavior of a material at high temperatures is the same as that under long loading times or slow loading rates/frequencies, and the material behavior at low temperatures is the same as that under short loading times or fast loading rates/frequencies. Simply stated, the same modulus value of a material can be obtained both at low test temperatures and long times (slow frequencies) or at high test temperatures but short times (fast frequencies). Materials that exhibit this type of behavior are called thermorheologically simple. The t-TS of a material can be checked by performing dynamic modulus tests at various temperatures and frequencies. Since the loading frequency (or rate) and temperature are interchangeable, these two variables can be converted into a one variable, so-called a *reduced frequency*. Then, the effect of temperature is converted into a reduced frequency and one representative single curve that can describe dynamic modulus at any frequency and temperature. The mastercurve can be described by a sigmoidal function expressed in Equation (1).

$$\log|E^*| = a + \frac{b}{1 + \frac{1}{e^{d + g\log(fr)}}} \tag{1}$$

where *a*, *b*, *d*, and *g* are optimized constants, and f_r is the reduced frequency. The constant, *a*, describes lower asymptotic value which is related to high temperature behavior (or very low frequency) and *a+b* determines upper asymptotic value of |E*| that is related to glassy behavior of a mixture. The constant, *d* and *g*, determines shape of S-shape curve.

Time-temperature superposition principle explains the relationship between time (rate) and temperature. The relationship is expressed by time-temperature (t-T) shift function. The t-T shift factor is defined as logarithmic distance between a mastercurve and its original position in frequency (or loading rate) domain. The shifted frequency by applying t-T shift factor is calculated by Equation (2).

$$f_r = a_T f \tag{2}$$

where a_T is the time-temperature shift factor, and *f* is the loading frequency in Hz. The shift function for this study is represented by a quadratic function:

$$\log(a_T) = a_1 T^2 + a_2 T + a_3, \tag{3}$$

where a_1, a_2, and a_3 are constants, and *T* is the temperature. The result is a curve that provides the relationship between shift factors and temperature with a reference temperature. Figure 2 shows an example how the measured |E*| can be shifted to develop mastercurve.

As mentioned earlier, asphalt material is known as temperature and time dependent material, which is a viscoelastic material. Knowing viscoelastic characteristics of asphalt concrete plays a major role in evaluating asphalt concrete and its performance as well. In this regard, Pavement ME considers dynamic modulus as a key input to evaluate performance of asphalt pavement. NCHRP Report 547 (Witczak, 2005) has developed performance predictive equations based on dynamic modulus (|E*|) in conjunction with climate and volumetric properties.

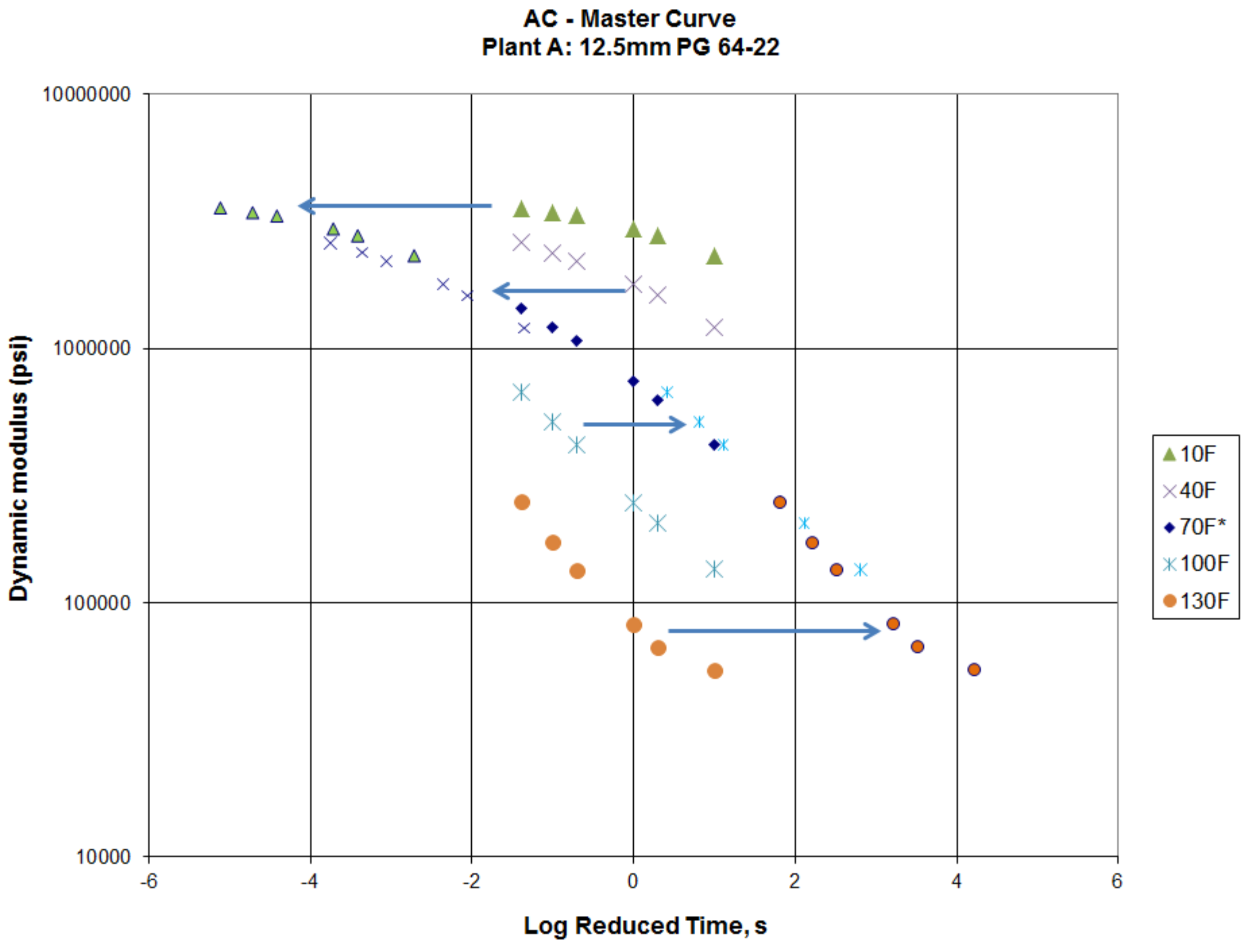


Figure 2. Mastercurve Generation

4.2 Controlled Crosshead Cyclic Tension Testing Using Simplified Viscoelastic Continuum Damage Model

Rigorous laboratory tests such as direct tension cyclic fatigue test or beam fatigue test for fatigue cracking and flow number test or triaxial stress sweep (TSS) test for rutting provides more accurate performance evaluation than dynamic modulus only (Choi et al, 2013). Therefore, fatigue cracking performance of the RAP mixtures has been evaluated using the simplified viscoelastic continuum damage (S-VECD) protocol developed under the Federal Highway Administration (FHWA) project, DTFH61-08-H-00005 Hot-Mix Asphalt Performance-Related Specifications based on Viscoelastoplastic Continuum Damage Models (AASHTO, 2014). The S-VECD protocol includes dynamic modulus testing based on TP 79/PP 61 and direct tension cyclic testing, and can be implemented using the Asphalt Mixture Performance Tester (AMPT). A draft AASHTO specification has been developed for the S-VECD direct tension cyclic test, which has been approved by the Asphalt Mixtures and Construction Expert Task Group and has been submitted to the AASHTO Subcommittee on Materials for their approval. The major strength of the S-VECD test and model is that a complete cracking characterization (including the strain level, temperature, and loading frequency) that is necessary to develop a traditional relationship between the tensile strain and the number of cycles to failure can be accomplished in a mere two days.

4.3 Viscoelastic Continuum Damage Model

Because a true controlled strain test using cylindrical specimens is difficult to run and can damage equipment if improperly performed, the controlled crosshead (CX) cyclic test was used for fatigue performance characterization. In this test, the machine actuator’s displacement is programmed to reach a constant peak level at each loading cycle. All the CX tests in this study were conducted at a constant frequency of 10 Hz and at 13ºC. Due to machine compliance issues, the actual on-specimen strain was less than the programmed level.

The viscoelastic continuum damage (VECD) model is a rigorous mechanical model that can characterize the properties of fatigue cracking using the elastic-viscoelastic correspondence principle, continuum damage mechanics, and the t-TS principle for any given strain or stress loading history (Chehab et al, 2002; Underwood et al., 2006; Shapery, 1987). A major concern associated with mechanistic viscoelastic material analysis is controlling the time concept for constitutive equations that show a relationship between strain and stress. The power of the elastic-viscoelastic correspondence principle is that it allows viscoelastic material to use linear constitutive equations, such as a generalized Hooke's law, by removing the time effects. Examples of constitutive equations for isotropic elastic and viscoelastic bodies can be expressed by Equations (3) and (4), respectively. Equation (4) is transformed in the Laplace domain, and the Laplace inversion of Equation (4) can be expressed as Equation (5). Finally, the form of viscoelastic constitutive Equation (6) is the same as that of elastic constitutive Equation (3) when using a reference modulus (E_R) and the pseudo strain (ε^R) concept. The pseudo strain can be derived using a convolution integral, as shown in Equation (7). The convolution integral allows the viscoelastic material response to be derived from any input loading history via the relaxation modulus, *E(t)*.

$$E\varepsilon_{11} = \sigma_{11} - \nu(\sigma_{22} + \sigma_{33}) \tag{3}$$

$$\tilde{E}\bar{\varepsilon}_{11} = \bar{\sigma}_{11} - \tilde{\nu}(\bar{\sigma}_{22} + \bar{\sigma}_{33}) \tag{4}$$

$$\int_0^t E(t-\tau)\frac{\partial \varepsilon_{11}}{\partial \tau} d\tau = \sigma_{11} - \nu(\sigma_{22} + \sigma_{33}) \tag{5}$$

$$E_R[\frac{1}{E_R}\int_0^t E(t-\tau)\frac{\partial \varepsilon_{11}}{\partial \tau} d\tau] = \sigma_{11} - \nu(\sigma_{22} + \sigma_{33}) \tag{6}$$

$$E_R\varepsilon^R = \sigma_{11} - \nu(\sigma_{22} + \sigma_{33}) \quad , \quad \varepsilon^R = \int_0^t E(t-\tau)\frac{\partial \varepsilon_{11}}{\partial \tau} d\tau \tag{7}$$

where

ε^R	=	pseudo strain,
ε	=	strain,
σ	=	stress,
ν	=	Poisson's ratio,
$E(t-\tau)$	=	linear viscoelastic relaxation modulus (unit response function), and
τ	=	integration term.

Continuum damage mechanics considers a damaged body with some stiffness as an undamaged body with reduced stiffness. The damage parameter (D) represents the damaged condition of a material in terms of effective stress or modulus value. This parameter helps to create a fundamental constitutive model for damaged material properties, as expressed by Equation (8).

$$\varepsilon = \frac{\sigma / (1-D)}{E} = \frac{\sigma'}{E} = \frac{\sigma}{E(1-D)} = \frac{\sigma}{E'} \quad (D = 0\text{: no damage, } D = 1\text{: full damage}) \tag{8}$$

Viscoelastic continuum damage theory adopts pseudo stiffness (C) as a material integrity parameter, which is a function of an internal state variable that represents damage (S) to define a constitutive equation of damaged material, as shown in Equation (9).

$$\sigma = E'\varepsilon = f(S)\varepsilon = C(S) \times E_R \times \varepsilon \tag{9}$$

Pseudo stiffness (C), which is an indicator of a material's integrity, and damage (S) are defined based on Schapery's work potential theory *(7)*. Damage (S) is represented by internal state variables that are related to the strain history of the material *(8, 9)*. Pseudo strain energy density, the viscoelastic constitutive equation, and damage evolution law concepts inherent of the work potential theory can be used to quantify pseudo stiffness and damage, as expressed in Equations (10) through (12), respectively. Intuitively speaking, the pseudo stiffness (C) and material integrity create a ratio of the stress of a damaged specimen to the stress of an undamaged specimen (C = 1: no damage, C = 0: full damage), which represents the damage parameter (D) in viscoelastic continuum damage.

Alpha (α) is related to the damage evolution rate in work potential theory and is calculated from the maximum slope of the relaxation modulus and time in log-log scale. The alpha has a relationship with m, the maximum slope between log creep compliance and log time, depending on the test control mode. For monotonic or CX tests, $\alpha = 1/m + 1$. Depending on the asphalt mixture, a unique damage characteristic curve can represent the relationship between pseudo stiffness (C) and damage (S), regardless of loading condition and temperature. The damage characteristic curves can be fitted by an exponential or power form, such as Equation (15). Finally, the number of cycles to failure (N_f) can be predicted regardless of any loading frequency (f_{red}) or history (ε^R) until damage accumulates to failure. Damage at failure is determined by the drop point of the phase angle during cyclic direct tension testing, as suggested by Reese *(10)*. The derivation of the number of cycles to failure for any input is described in Equations (13) to (19).

$$W^R = \frac{1}{2} C(S)(\varepsilon^R)^2 \tag{10}$$

$$\sigma = C(S)\varepsilon^R, C(S) = \frac{\sigma}{\varepsilon^R} \tag{11}$$

$$\dot{S} = \left(-\frac{\partial W^R}{\partial S} \right)^\alpha \tag{12}$$

$$\frac{dS}{dt} = \left(-\frac{1}{2}\frac{dC}{dS}(\varepsilon^R)^2 \right)^\alpha = \left(-\frac{1}{2}\frac{dC}{dt}\frac{dt}{dS}(\varepsilon^R)^2 \right)^\alpha = \left(-\frac{1}{2}\frac{dC}{dt}(\varepsilon^R)^2 \right)^{\frac{\alpha}{1+\alpha}} \tag{13}$$

$$S \cong \sum_{i=1}^{n} \left(-\frac{(\varepsilon^R)^2}{2}(C_i - C_{i-1}) \right)^{\frac{\alpha}{1+\alpha}} (t_i - t_{i-1})^{\frac{1}{1+\alpha}} \tag{14}$$

$$C(S) = e^{aS^b},\ C(S) = 1 - c_{11}S^{C_{12}},\ \frac{dC}{dS} = -c_{11}c_{12}S^{c_{12}-1} \tag{15}$$

$$\frac{dS}{dN} = \frac{dS}{dt}\frac{dt}{dN} = \frac{dS}{dt}\frac{1}{f_{red}} = \left(-\frac{1}{2}\left(\varepsilon^R\right)^2 \frac{\partial C}{\partial S} \right)^\alpha \left(\frac{1}{f_{red}} \right) \tag{16}$$

$$\left(S^{(C_{12}-1)} \right)^{-\alpha} dS = \left(\frac{1}{2}\left(\varepsilon^R\right)^2 C_{11}C_{12} \right)^\alpha \left(\frac{1}{f_{red}} \right) dN \tag{17}$$

$$\int_{S_{ini}}^{S_{failure}} \left(S^{(C_{12}-1)}\right)^{-\alpha} dS = \int_{1}^{N_{failure}} \left(\frac{1}{2}\left(\varepsilon^{R}\right)^{2} C_{11}C_{12}\right)^{\alpha} \left(\frac{1}{f_{red}}\right) dN \tag{18}$$

$$N_{failure} = \frac{\left(f_{red}\right)\left(2^{\alpha}\right) S_{failure}^{\alpha-\alpha C_{12}+1}}{\left(\alpha-\alpha C_{12}+1\right)\left(C_{11}C_{12}\right)^{\alpha}\left(\varepsilon^{R}\right)^{2\alpha}} \tag{19}$$

The S-VECD model is a simplified version of the VECD model and is designed to reduce computational time while maintaining the accuracy of the VECD model. The S-VECD model employs a piecewise approach to separate the first load cycle analysis from the other load cycles. Approximately 15 percent to 25 percent reduction in material integrity occurs during the first load cycle, which indicates the necessity of full analysis using all the data points. However, the S-VECD model assumes that the steady state starts from the second cycle to simplify the cyclic fatigue test. That is, after the first load cycle, each peak value for the subsequent load cycles is used to calculate the pseudo stiffness (C) and damage (S) using a load shape function extracted from the analysis of the first load cycle. By multiplying the peak load amplitude by the load shape function (k_1), an entire load cycle history can be created for accurate analysis. For the pseudo strain and pseudo stiffness calculations in the S-VECD model, a loading form (β) is used to extract the tension-only stress from the combined compression and tension stress. Also, the dynamic modulus ratio (DMR) is adopted to check the specimen variability between the dynamic modulus tests used to determine the linear viscoelastic properties and the direct cyclic tension tests used to determine the S-VECD properties. The damage characteristic curves fit well when using normalized pseudo stiffness values calculated by applying the DMR concept within ±10 percent specimen variability. Therefore, specimens with ±10 percent difference in their DMRs can be utilized for S-VECD model characterization (Underwood et al, 2010; Kim et al, 2008; Zhang et al, 2008).

4.4 Failure Criterion

The VECD theory is based on continuum damage mechanics that assume that material damage is the accumulation of dispersed microcracks. When significant damage has accumulated, the microcracks become concentrated and form a macrocrack. At this point, the VECD model loses its predictive capability. The behavior of the macrocrack then can be described using fracture mechanics. Knowing the level of damage that a mixture can tolerate before these microcracks are even observed is critical for pavement life predictions. In short, the damage evolution curve, or *C* versus *S* curve, describes the increase in damage as the number of cycles increases. However, the curve by itself cannot provide information about when the failure or macrocrack starts. Therefore, a criterion for failure is needed.

The first challenge with developing a failure criterion is to define failure in a strain-controlled test where there is no catastrophic failure because the specimen is pushed and pulled a specified distance. The most common method used to define failure is to reduce the initial stiffness by 50 percent, as seen in Figure 3 (a). This failure criterion works well for neat binder mixtures, but severely under-predicts the fatigue life of modified binder mixtures. Reese (1997) suggests that the change in the phase angle is an effective way to determine failure, as described in Figure 3 (b). The mixture shown in Figure 3 uses a modified binder mixture. The number of cycles to failure increases three-fold using the phase angle drop approach. The theoretical strength of the phase angle drop approach is that the phase angle increases as more microcracks are created, because opening the microcracks takes energy and delays the strain response. Once the phase angle starts decreasing rapidly, a different mechanical mechanism must be active to represent such behavior.

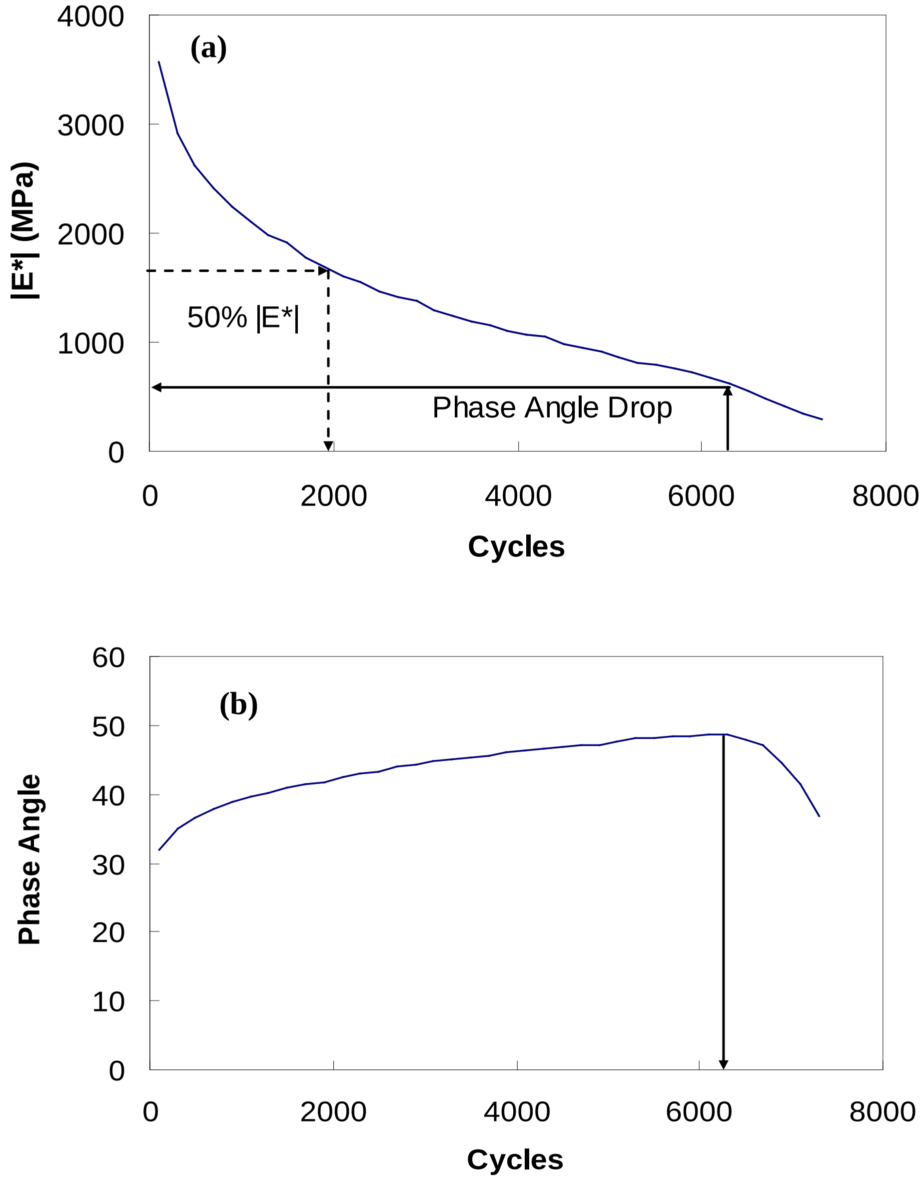


Figure 3. Determination of failure in repeated load cyclic test: (a) comparison between 50% modulus reduction and phase angle drop and (b) phase angle change.

The S-VECD model can represent the damage evolution well; however, it cannot clearly specify when the failure occurs. In order to represent failure using the S-VECD model, Zhang et al. (2012) proposed a way to bridge the gap between VECD and failure by characterizing the failure criterion in relationship to the pseudo strain energy release rate. However, Zhang et al.

developed this approach based on CX mode-of-loading test data only. Later, Sabouri and Kim (2014) applied Zhang et al.'s failure criterion to different modes of loading and found it to be mode-of-loading dependent. Sabouri and Kim (2014) thus proposed an advanced failure criterion using the so-called G^R method, also based on the pseudo strain energy release rate, which remedies the problems associated with Zhang et al.'s failure criterion. The idea behind the G^R method is that during each cycle of a fatigue test, some of the energy is stored and some is released by damaging the specimen and/or generating microcracks. The energy or work stored during a given cycle decreases as the damage level increases and the *C*-value decreases. This failure criterion relies on the pseudo strain energy concept and focuses on the dissipated energy that is related to stiffness reduction. During cyclic tests, the stiffness and phase angle both change with the cycles, and thus, a specific amount of energy is dissipated at each cycle. A characteristic relationship between the rate of change of the averaged released pseudo strain energy during fatigue testing and the final fatigue life, and that is independent of mode of loading, strain amplitude, and temperature, was found to exist in both the RAP and non-RAP mixtures.

Based on the consistency between the experimental measurements and the model predictions for the energy release rate, a linear relationship exists between the G^R value and N_f, which is the number of cycles for specimen failure when the G^R value is plotted against the number of cycles to failure (N_f) in log-log space. Because the G^R value characterizes the overall rate of damage accumulation during fatigue testing, it is reasonable to hypothesize that a correlation must exist between the G^R and the fatigue life (N_f), because the faster the damage accumulates, the quicker the material should fail. Thus, the proposed failure criterion combines the advantages of the VECD model and this characteristic relationship, which both originate from fundamental mixture properties, and is able to predict the fatigue life of asphalt concrete mixtures across different modes of loading, temperatures, and strain amplitudes within typical sample-to-sample variability observed in fatigue testing. This proposed failure criterion was applied in this project to describe the fatigue performance of asphalt materials using LVECD program simulations.

5. LABORATORY TEST RESULTS AND DISCUSSION

5.1 Dynamic Modulus Test Results

Dynamic modulus tests are performed according to AASHTO T 342-11 (2011) with three replicates and |E*| measured at four temperatures (40, 70, 100, 130°F) with six frequencies (25, 10, 5, 1, 0.5, 0.1 Hz) in accordance with AASHTO T 342. Testing device for the test is *Interlaken UniSystem.* Specimens are tested from low to high temperature and with the frequency from high to low at each testing temperature. Environmental chamber maintains target testing temperature constant for about four hours for a specimen to reach the target temperature. Two LVDTs are glued on a specimen at 180° directions and the length between LVDT gauge points is 70 mm.

|E*| is calculated by Equation (4), where $|\sigma_0|$ and $|\varepsilon_0|$ are amplitudes of stress and strain, respectively, obtained by fitting cyclic data in steady state. In order to get data at steady state, last ten cycles at each frequency are used in the analysis.

$$|E^*| = \frac{|\sigma_0|}{|\varepsilon_0|} \tag{4}$$

Figures 4 and 5 show |E*| test results of all tested specimens. Relatively small variability among replicates likely makes average |E*| be a mixture representative property in terms of linear viscoelasticity. In this regard, average |E*| is utilized for this study for the sake of clear comparison instead of including all specimens. The average |E*| test results are summarized in Tables 4 through 11. As expected, |E*| increases as the loading frequency increases and decreases as the temperature increases. From the measured three replicates, average values of |E*| were calculated at each loading frequency and temperatures to construct master curve.

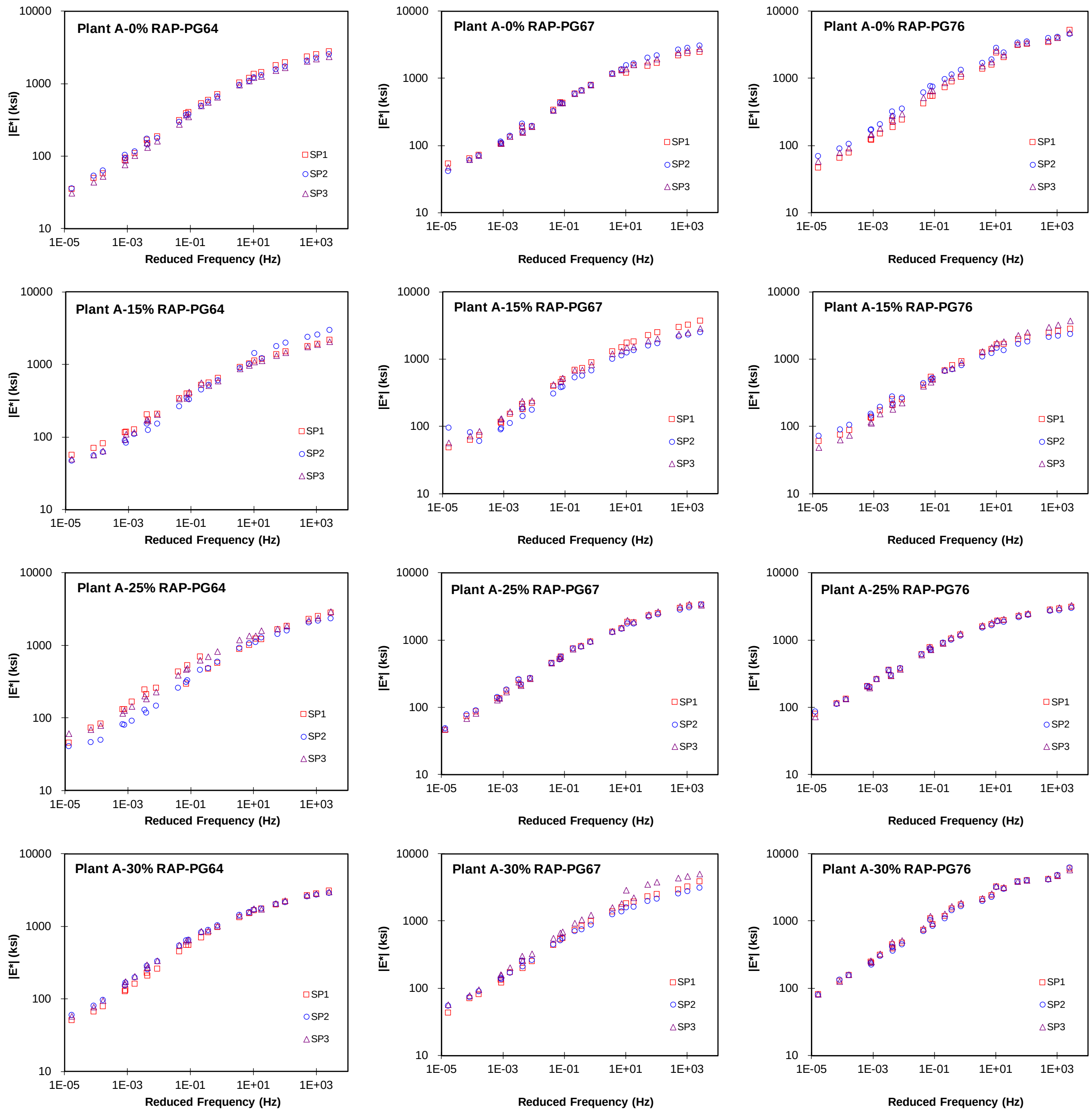


Figure 4. |E*| of individual specimens and the average |E*| of a mixture for Plant A

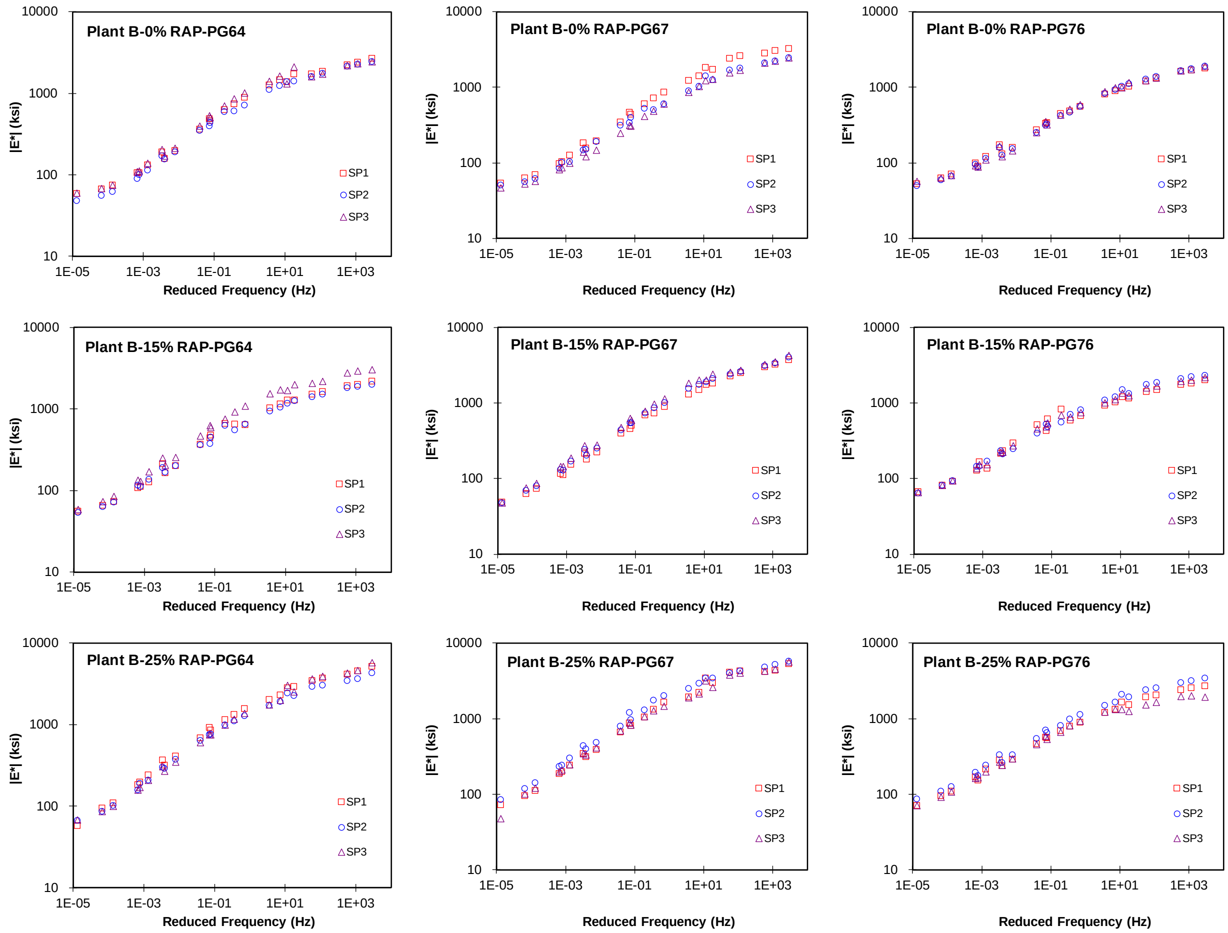


Figure 5. |E*| of individual specimens and the average |E*| of a mixture for Plant B

Table 4. Average E* Results (Plant A, 12.5 mm Superpave, 0% RAP)

Temp. (F)	PLANT A								
	12.5 mm NMAS - RAP 0%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.30	0.1	1,263,184	5.30	0.1	1,355,298	5.30	0.1	2,153,616
40	5.30	0.5	1,627,783	5.30	0.5	1,710,511	5.30	0.5	2,631,230
40	5.30	1.0	1,779,326	5.30	1.0	1,873,168	5.30	1.0	2,749,797
40	5.30	5.0	2,152,927	5.30	5.0	2,326,637	5.30	5.0	3,039,696
40	5.30	10.0	2,318,562	5.30	10.0	2,484,918	5.30	10.0	3,322,337
40	5.30	25.0	2,547,765	5.30	25.0	2,671,673	5.30	25.0	3,882,565
70	5.30	0.1	374,712	5.30	0.1	373,059	5.30	0.1	594,433
70	5.30	0.5	567,681	5.30	0.5	562,006	5.30	0.5	893,687
70	5.30	1.0	671,387	5.30	1.0	681,975	5.30	1.0	1,034,040
70	5.30	5.0	979,973	5.30	5.0	1,018,492	5.30	5.0	1,342,579
70	5.30	10.0	1,115,398	5.30	10.0	1,157,344	5.30	10.0	1,515,968
70	5.30	25.0	1,327,183	5.30	25.0	1,373,635	5.30	25.0	1,872,828
100	5.30	0.1	91,956	5.30	0.1	99,449	5.30	0.1	136,736
100	5.30	0.5	142,393	5.30	0.5	147,251	5.30	0.5	213,194
100	5.30	1.0	175,047	5.30	1.0	178,765	5.30	1.0	267,251
100	5.30	5.0	294,676	5.30	5.0	307,844	5.30	5.0	459,030
100	5.30	10.0	374,164	5.30	10.0	389,784	5.30	10.0	570,886
100	5.30	25.0	506,620	5.30	25.0	532,022	5.30	25.0	738,893
130	5.30	0.1	33,974	5.30	0.1	45,892	5.30	0.1	53,653
130	5.30	0.5	49,003	5.30	0.5	58,737	5.30	0.5	75,073
130	5.30	1.0	57,772	5.30	1.0	67,153	5.30	1.0	88,302
130	5.30	5.0	88,805	5.30	5.0	101,643	5.30	5.0	138,223
130	5.30	10.0	108,771	5.30	10.0	123,858	5.30	10.0	171,501
130	5.30	25.0	163,807	5.30	25.0	189,466	5.30	25.0	256,949

Table 5. Average E* Results (Plant A, 12.5 mm Superpave, 15% RAP)

Temp. (F)	PLANT A								
	12.5 mm NMAS - RAP 15%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.30	0.1	1,205,531	5.30	0.1	1,488,158	5.30	0.1	1,606,136
40	5.30	0.5	1,498,243	5.30	0.5	1,891,907	5.30	1.1	1,971,289
40	5.30	1.0	1,638,892	5.30	1.0	2,065,742	5.30	2.1	2,132,465
40	5.30	5.0	1,963,472	5.30	5.0	2,490,693	5.30	3.1	2,513,972
40	5.30	10.0	2,102,483	5.30	10.0	2,685,061	5.30	4.1	2,682,392
40	5.30	25.0	2,380,088	5.30	25.0	2,995,115	5.30	5.1	2,944,959
70	5.30	0.1	359,991	5.30	0.1	434,778	5.30	6.1	496,874
70	5.30	0.5	528,860	5.30	0.5	662,787	5.30	7.1	743,393
70	5.30	1.0	615,335	5.30	1.0	795,548	5.30	8.1	873,736
70	5.30	5.0	886,066	5.30	5.0	1,164,366	5.30	9.1	1,211,347
70	5.30	10.0	998,498	5.30	10.0	1,319,697	5.30	10.1	1,375,140
70	5.30	25.0	1,170,047	5.30	25.0	1,555,397	5.30	11.1	1,611,751
100	5.30	0.1	103,260	5.30	0.1	111,836	5.30	12.1	129,504
100	5.30	0.5	154,995	5.30	0.5	172,043	5.30	13.1	198,995
100	5.30	1.0	187,561	5.30	1.0	212,766	5.30	14.1	245,611
100	5.30	5.0	312,612	5.30	5.0	372,584	5.30	15.1	414,836
100	5.30	10.0	378,836	5.30	10.0	468,342	5.30	16.1	506,728
100	5.30	25.0	510,679	5.30	25.0	632,533	5.30	17.1	673,719
130	5.30	0.1	51,145	5.30	0.1	66,827	5.30	18.1	59,950
130	5.30	0.5	60,917	5.30	0.5	71,654	5.30	19.1	75,890
130	5.30	1.0	69,150	5.30	1.0	72,512	5.30	20.1	88,533
130	5.30	5.0	98,517	5.30	5.0	111,179	5.30	21.1	134,870
130	5.30	10.0	116,693	5.30	10.0	143,088	5.30	22.1	171,974
130	5.30	25.0	175,061	5.30	25.0	210,636	5.30	23.1	246,087

Table 6. Average E* Results (Plant A, 12.5 mm Superpave, 25% RAP)

Temp. (F)	PLANT A								
	12.5 mm NMAS - RAP 25%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.52	0.1	1,222,849	5.52	0.1	1,704,182	5.52	0.1	1,694,202
40	5.52	0.5	1,579,768	5.52	0.5	2,141,133	5.52	0.5	1,974,151
40	5.52	1.0	1,744,462	5.52	1.0	2,342,242	5.52	1.0	2,091,742
40	5.52	5.0	2,176,674	5.52	5.0	2,794,338	5.52	5.0	2,414,072
40	5.52	10.0	2,332,231	5.52	10.0	3,015,681	5.52	10.0	2,511,930
40	5.52	25.0	2,670,635	5.52	25.0	3,211,761	5.52	25.0	2,663,873
70	5.52	0.1	354,594	5.52	0.1	616,951	5.52	0.1	664,417
70	5.52	0.5	546,766	5.52	0.5	938,473	5.52	0.5	898,469
70	5.52	1.0	655,243	5.52	1.0	1,105,601	5.52	1.0	1,017,824
70	5.52	5.0	987,327	5.52	5.0	1,545,822	5.52	5.0	1,354,219
70	5.52	10.0	1,124,083	5.52	10.0	1,762,749	5.52	10.0	1,474,263
70	5.52	25.0	1,350,967	5.52	25.0	2,123,242	5.52	25.0	1,664,139
100	5.52	0.1	111,194	5.52	0.1	128,058	5.52	0.1	175,727
100	5.52	0.5	169,559	5.52	0.5	204,307	5.52	0.5	262,523
100	5.52	1.0	210,150	5.52	1.0	255,737	5.52	1.0	324,830
100	5.52	5.0	356,848	5.52	5.0	431,804	5.52	5.0	531,312
100	5.52	10.0	443,228	5.52	10.0	535,432	5.52	10.0	626,297
100	5.52	25.0	585,542	5.52	25.0	710,122	5.52	25.0	770,947
130	5.52	0.1	48,374	5.52	0.1	75,976	5.52	0.1	70,745
130	5.52	0.5	62,486	5.52	0.5	73,623	5.52	0.5	103,608
130	5.52	1.0	70,124	5.52	1.0	84,893	5.52	1.0	120,455
130	5.52	5.0	108,284	5.52	5.0	133,202	5.52	5.0	188,162
130	5.52	10.0	132,702	5.52	10.0	171,043	5.52	10.0	240,278
130	5.52	25.0	188,585	5.52	25.0	242,963	5.52	25.0	320,086

Table 7. Average E* Results (Plant A, 12.5 mm Superpave, 30% RAP)

Temp. (F)	PLANT A 12.5 mm NMAS - RAP 30%								
	PG64-22 Average			PG67-22 Average			PG76-22 Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.03	0.1	1,701,918	5.03	0.1	2,102,148	5.03	0.1	2,749,022
40	5.03	0.5	2,032,630	5.03	0.5	2,611,355	5.03	0.5	3,353,269
40	5.03	1.0	2,192,737	5.03	1.0	2,810,666	5.03	1.0	3,518,172
40	5.03	5.0	2,637,902	5.03	5.0	3,302,186	5.03	5.0	3,792,356
40	5.03	10.0	2,783,364	5.03	10.0	3,552,917	5.03	10.0	4,253,993
40	5.03	25.0	2,993,048	5.03	25.0	3,997,529	5.03	25.0	5,249,372
70	5.03	0.1	610,488	5.03	0.1	574,735	5.03	0.1	883,187
70	5.03	0.5	866,429	5.03	0.5	880,382	5.03	0.5	1,270,439
70	5.03	1.0	1,006,535	5.03	1.0	1,030,555	5.03	1.0	1,479,430
70	5.03	5.0	1,395,400	5.03	5.0	1,412,131	5.03	5.0	1,817,855
70	5.03	10.0	1,550,423	5.03	10.0	1,603,089	5.03	10.0	2,110,128
70	5.03	25.0	1,743,847	5.03	25.0	1,940,462	5.03	25.0	2,662,345
100	5.03	0.1	157,368	5.03	0.1	137,876	5.03	0.1	194,292
100	5.03	0.5	244,538	5.03	0.5	223,575	5.03	0.5	318,625
100	5.03	1.0	308,296	5.03	1.0	282,673	5.03	1.0	393,825
100	5.03	5.0	515,248	5.03	5.0	482,462	5.03	5.0	626,626
100	5.03	10.0	620,350	5.03	10.0	598,702	5.03	10.0	762,480
100	5.03	25.0	795,819	5.03	25.0	785,256	5.03	25.0	1,020,078
130	5.03	0.1	56,316	5.03	0.1	52,000	5.03	0.1	69,991
130	5.03	0.5	74,643	5.03	0.5	74,914	5.03	0.5	107,517
130	5.03	1.0	89,676	5.03	1.0	89,281	5.03	1.0	95,997
130	5.03	5.0	146,517	5.03	5.0	145,093	5.03	5.0	204,084
130	5.03	10.0	187,316	5.03	10.0	182,334	5.03	10.0	260,664
130	5.03	25.0	269,080	5.03	25.0	269,687	5.03	25.0	368,490

Table 8. Average E* Results (Plant B, 12.5 mm Superpave, 0% RAP)

Temp. (F)	PLANT B								
	12.5 mm NMAS - RAP 0%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.1	0.1	1,346,579	5.10	0.1	1,488,887	5.27	0.1	996,422
40	5.1	0.5	1,637,676	5.10	1.1	1,878,955	5.27	0.5	1,234,284
40	5.1	1.0	1,778,319	5.10	2.1	2,024,962	5.27	1.0	1,347,477
40	5.1	5.0	2,180,520	5.10	3.1	2,340,952	5.27	5.0	1,642,525
40	5.1	10.0	2,304,090	5.10	4.1	2,496,577	5.27	10.0	1,711,924
40	5.1	25.0	2,512,126	5.10	5.1	2,717,171	5.27	25.0	1,867,031
70	5.1	0.1	470,246	5.10	6.1	370,685	5.27	0.1	331,489
70	5.1	0.5	730,375	5.10	7.1	568,957	5.27	0.5	484,831
70	5.1	1.0	867,320	5.10	8.1	685,244	5.27	1.0	565,891
70	5.1	5.0	1,258,990	5.10	9.1	992,461	5.27	5.0	839,603
70	5.1	10.0	1,436,100	5.10	10.1	1,149,502	5.27	10.0	941,794
70	5.1	25.0	1,733,799	5.10	11.1	1,416,031	5.27	25.0	1,095,860
100	5.1	0.1	105,979	5.10	12.1	96,634	5.27	0.1	88,610
100	5.1	0.5	161,309	5.10	13.1	142,233	5.27	0.5	125,783
100	5.1	1.0	200,057	5.10	14.1	178,233	5.27	1.0	153,113
100	5.1	5.0	367,436	5.10	15.1	302,953	5.27	5.0	258,763
100	5.1	10.0	467,773	5.10	16.1	381,149	5.27	10.0	328,166
100	5.1	25.0	640,464	5.10	17.1	512,769	5.27	25.0	432,035
130	5.1	0.1	55,311	5.10	18.1	50,441	5.27	0.1	53,140
130	5.1	0.5	63,543	5.10	19.1	57,349	5.27	0.5	61,872
130	5.1	1.0	70,553	5.10	20.1	62,982	5.27	1.0	68,323
130	5.1	5.0	101,142	5.10	21.1	87,782	5.27	5.0	95,543
130	5.1	10.0	128,509	5.10	22.1	109,177	5.27	10.0	114,773
130	5.1	25.0	187,419	5.10	23.1	155,830	5.27	25.0	165,431

Table 9. Average E* Results (Plant B, 12.5 mm Superpave, 15% RAP)

Temp. (F)	PLANT B								
	12.5 mm NMAS - RAP 15%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
	5.20	0.1	1,365,472	5.20	0.1	1,624,469	5.47	0.1	1,241,176
	5.20	0.5	1,645,312	5.20	0.5	2,045,399	5.47	1.1	1,468,677
	5.20	1.0	1,774,256	5.20	1.0	2,215,934	5.47	2.1	1,572,930
	5.20	5.0	2,149,558	5.20	5.0	2,701,865	5.47	3.1	1,813,094
	5.20	10.0	2,257,154	5.20	10.0	2,862,189	5.47	4.1	1,910,198
	5.20	25.0	2,392,638	5.20	25.0	3,299,281	5.47	5.1	2,055,353
	5.20	0.1	478,570	5.20	0.1	518,570	5.47	6.1	412,303
	5.20	0.5	704,366	5.20	0.5	788,353	5.47	7.1	567,937
	5.20	1.0	786,632	5.20	1.0	935,082	5.47	8.1	659,754
	5.20	5.0	1,162,242	5.20	5.0	1,404,047	5.47	9.1	920,120
	5.20	10.0	1,298,947	5.20	10.0	1,572,016	5.47	10.1	1,017,987
	5.20	25.0	1,500,942	5.20	25.0	1,829,045	5.47	11.1	1,146,273
	5.20	0.1	117,386	5.20	0.1	125,553	5.47	12.1	131,373
	5.20	0.5	177,391	5.20	0.5	192,458	5.47	13.1	187,459
	5.20	1.0	218,499	5.20	1.0	238,100	5.47	14.1	230,998
	5.20	5.0	399,418	5.20	5.0	413,058	5.47	15.1	388,260
	5.20	10.0	502,292	5.20	10.0	509,225	5.47	16.1	468,531
	5.20	25.0	674,680	5.20	25.0	687,047	5.47	17.1	602,634
	5.20	0.1	55,579	5.20	0.1	49,688	5.47	18.1	60,888
	5.20	0.5	67,067	5.20	0.5	68,017	5.47	19.1	74,675
	5.20	1.0	76,490	5.20	1.0	78,235	5.47	20.1	84,388
	5.20	5.0	118,402	5.20	5.0	124,137	5.47	21.1	123,025
	5.20	10.0	144,363	5.20	10.0	159,418	5.47	22.1	137,208
	5.20	25.0	215,246	5.20	25.0	230,483	5.47	23.1	199,108

Table 10. Average E* Results (Plant B, 12.5 mm Superpave, 25% RAP)

Temp. (F)	PLANT B								
	12.5 mm NMAS - RAP 25%								
	PG64-22			PG67-22			PG76-22		
	Average			Average			Average		
	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)	COAC(%)	Hz	E* (psi)
40	5.30	0.1	2,742,577	5.30	0.1	3,296,857	5.30	0.1	1,665,143
40	5.30	0.5	3,328,037	5.30	0.5	3,924,465	5.30	0.5	1,945,354
40	5.30	1.0	3,516,878	5.30	1.0	4,144,315	5.30	1.0	2,062,214
40	5.30	5.0	3,931,127	5.30	5.0	4,370,807	5.30	5.0	2,419,460
40	5.30	10.0	4,243,317	5.30	10.0	4,635,884	5.30	10.0	2,536,637
40	5.30	25.0	5,030,056	5.30	25.0	5,496,970	5.30	25.0	2,651,221
70	5.30	0.1	801,223	5.30	0.1	975,227	5.30	0.1	614,187
70	5.30	0.5	1,188,976	5.30	0.5	1,426,031	5.30	0.5	853,954
70	5.30	1.0	1,392,746	5.30	1.0	1,687,479	5.30	1.0	972,394
70	5.30	5.0	1,814,495	5.30	5.0	2,085,417	5.30	5.0	1,292,045
70	5.30	10.0	2,061,582	5.30	10.0	2,403,137	5.30	10.0	1,419,839
70	5.30	25.0	2,528,729	5.30	25.0	2,965,975	5.30	25.0	1,552,806
100	5.30	0.1	183,676	5.30	0.1	214,208	5.30	0.1	164,610
100	5.30	0.5	290,957	5.30	0.5	341,515	5.30	0.5	243,714
100	5.30	1.0	372,924	5.30	1.0	427,160	5.30	1.0	300,898
100	5.30	5.0	635,936	5.30	5.0	706,647	5.30	5.0	482,680
100	5.30	10.0	781,451	5.30	10.0	854,143	5.30	10.0	571,084
100	5.30	25.0	1,033,468	5.30	25.0	1,131,150	5.30	25.0	714,673
130	5.30	0.1	63,419	5.30	0.1	76,052	5.30	0.1	75,374
130	5.30	0.5	87,944	5.30	0.5	103,240	5.30	0.5	97,636
130	5.30	1.0	102,585	5.30	1.0	124,158	5.30	1.0	113,689
130	5.30	5.0	164,264	5.30	5.0	202,228	5.30	5.0	173,310
130	5.30	10.0	216,194	5.30	10.0	260,898	5.30	10.0	216,179
130	5.30	25.0	321,562	5.30	25.0	369,837	5.30	25.0	289,989

Table 11. Average E* Results (Plant A and B, 12.5 mm SMA)

SMA_ PG 76 - 22						
Temp. (F)	PLANT A				PLANT B	
	RAP 0%		RAP 15%		RAP 10%	
	Log Reduced Freq.	E* (psi)	Log Reduced Freq.	E* (psi)	Log Reduced Freq.	E* (psi)
40	0.990	883,210	1.032	1,019,841	1.198	840,139
40	1.689	1,172,849	1.731	1,310,311	1.897	1,108,865
40	1.990	1,295,287	2.032	1,429,152	2.198	1,218,985
40	2.689	1,571,828	2.731	1,701,619	2.897	1,518,188
40	2.990	1,714,780	3.032	1,826,750	3.198	1,624,292
40	3.388	1,993,709	3.430	2,029,011	3.596	1,797,783
70	-1.134	232,580	-1.137	256,335	-1.148	246,019
70	-0.435	356,471	-0.438	399,347	-0.449	375,817
70	-0.134	424,932	-0.137	476,708	-0.148	459,036
70	0.565	641,060	0.562	731,566	0.551	710,636
70	0.866	743,348	0.863	847,348	0.852	819,864
70	1.264	920,225	1.261	1,018,364	1.250	1,007,140
100	-3.030	64,789	-3.074	71,090	-3.243	78,331
100	-2.331	90,135	-2.375	99,981	-2.544	105,884
100	-2.030	106,409	-2.074	120,308	-2.243	125,824
100	-1.331	179,143	-1.375	206,190	-1.544	205,141
100	-1.030	225,530	-1.074	259,307	-1.243	258,209
100	-0.632	321,145	-0.676	362,647	-0.845	355,909
130	-4.734	42,494	-4.813	42,769	-5.124	49,656
130	-4.035	47,654	-4.114	49,770	-4.425	55,936
130	-3.734	50,654	-3.813	53,830	-4.124	57,043
130	-3.035	64,607	-3.114	69,508	-3.425	68,069
130	-2.734	78,585	-2.813	82,258	-3.124	88,166
130	-2.336	109,445	-2.415	118,490	-2.726	123,640

5.2 Qualitative Observations

Dynamic moduli of PG64-22, PG67-22 and PG76-22 mixtures are compared in Figure 6 through Figure 8. In general, |E*| increases when %RAP increases. Mixtures with higher RAP content revealed the higher dynamic modulus because asphalt mixture becomes stiffer as RAP content increases. As seen, it is well agreed that the addition of 25 percent RAP to the virgin mixes significantly increased the mixture stiffness at low, intermediate, and high temperatures.

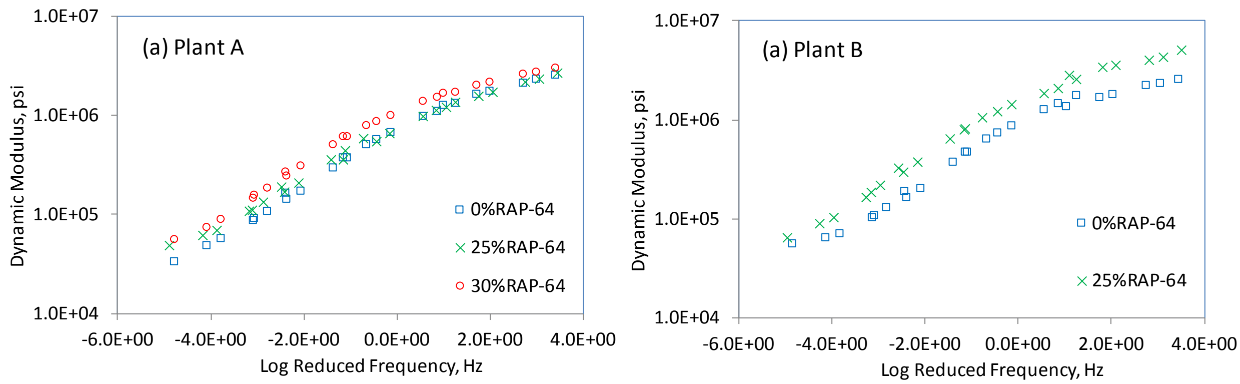


Figure 6. Dynamic modulus (|E*|) results of mixtures with binder PG 64-22:
(a) Plant A, (b) Plant B

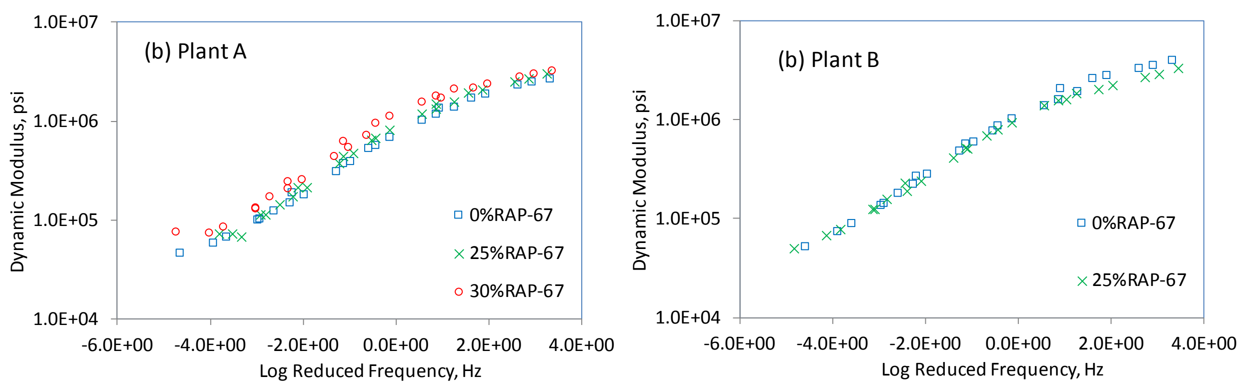


Figure 7. Dynamic modulus (|E*|) results of mixtures with binder PG 67-22:
(a) Plant A, (b) Plant B

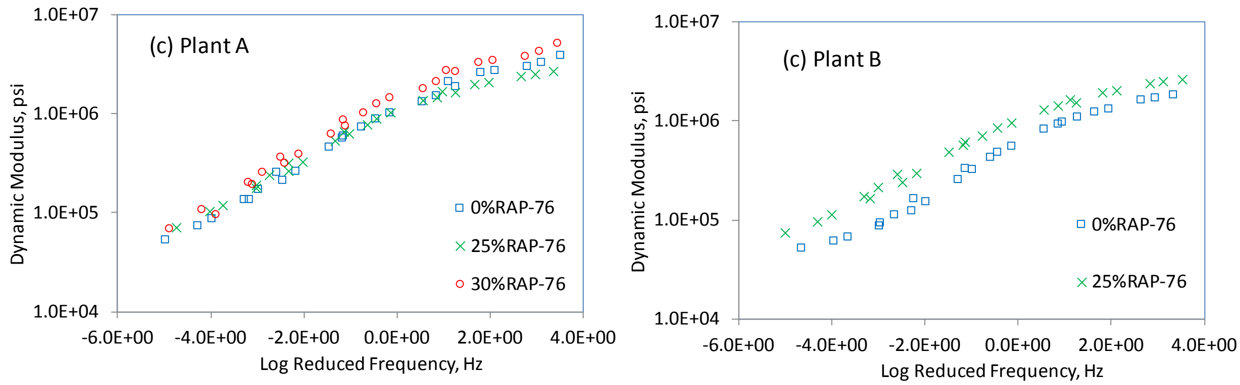


Figure 8. Dynamic modulus (|E*|) results of mixtures with binder PG 76-22:
(a) Plant A, (b) Plant B

Figure 9 shows mastercurves of dynamic moduli of Plant A and Plant B mixtures with various RAP contents. A Plant A - PG76 mixture with 30%RAP was the stiffest mixture and all 30% RAP mixtures fell into the stiffest mixture group. Dynamic moduli of PG64-22, PG67-22 and PG76-22 mixtures with 0% RAP fell into softest mixture group. As shown in Figure 9, the highest |E*| among Plant A mixtures was consistently observed in mixtures with PG76-22, which indicate that higher PG grade of binder makes asphalt mixtures stiffer. Even half grade difference in binder PG (i.e., PG 67-22 and PG 64-22) can be captured by |E*|.

As discussed in Chapter 3.3, the JMF for Plant B 30% RAP mixtures were not available. Thus, the maximum RAP content used in this study was 25%. As shown in Figure 9, 25% RAP with PG64 (25%RAP-64) was found to be the stiffest mixture, whereas virgin mixtures with PG67 (0%RAP-67) and PG76 (0%RAP-76) were among the softest mixtures.

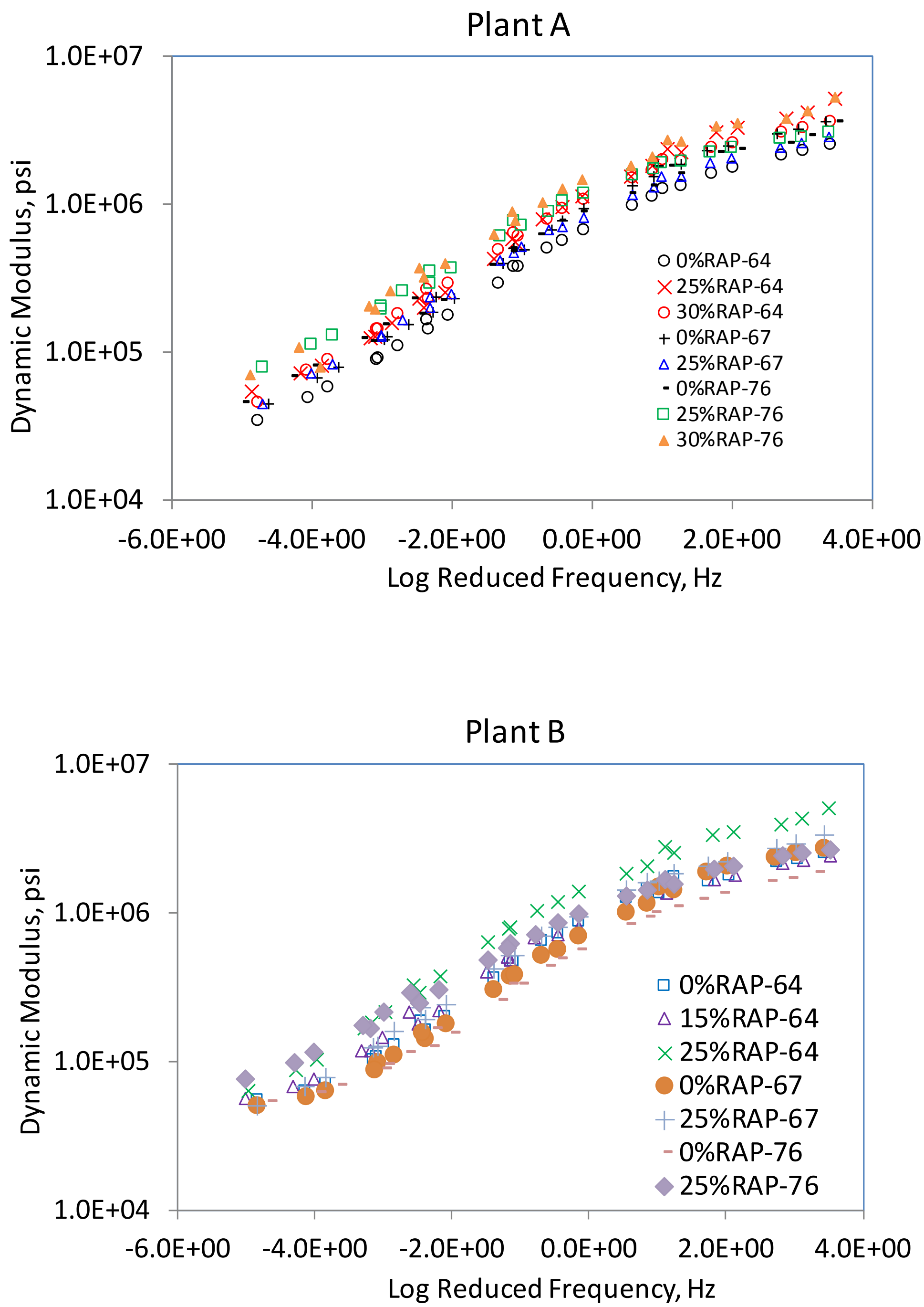


Figure 9. |E*| comparison between PG64-22, PG67-22, and PG 76-22 according to %RAP in log-log plot.

5.3 Statistical Analysis – Effects of RAP Content and Binder Type on Dynamic Modulus

Statistical analyses were performed for both Superpave and SMA. The test data was compiled to generate boxplots with respect to two design variables: RAP content and binder type. Then, mixed design ANOVA was conducted to reflect the between and within subject errors. Considering each specimen as a "subject", the "between" factors are RAP content and binder type; the "within" factors are temperature and frequency as each specimen is subject to the dynamic modulus test for a range of temperatures and frequencies. Following the mix design ANOVA, the paired t test was performed for the significant variables identified in ANOVA to evaluate the direction of impact of those variables on dynamic modulus. The analysis results are presented separately for Superpave and SMA.

For Superpave mixtures, four variables have been considered in the analysis, including RAP content, binder type, and frequency and temperature in accordance with the test protocol. The levels of those variables are depicted in Table 12 below.

Table 12. Design Variables and Coding (Superpave)

Variable	Value	Code	Plant
RAP Content	0%	R1	A, B
	15%	R2	A, B
	25%	R3	A, B
	30%	R4	A
Binder	PG64-22	B1	A, B
	PG67-22	B2	A, B
	PG76-22	B3	A
Frequency	0.1 Hz	F1	A, B
	0.5 Hz	F2	A, B
	1.0 Hz	F3	A, B
	5.0 Hz	F4	A, B
	10 Hz	F5	A, B
	25 Hz	F6	A, B
Temperature	40 ^{o}F	T1	A, B
	70 ^{o}F	T2	A, B
	100 ^{o}F	T3	A, B
	130 ^{o}F	T4	A, B

Note that the last column in Table 12 indicates the level of variables applicable to each plant. For example, 30% RAP content and PG76-22 binder are only applicable to Plant A. Because of this design, the analyses were conducted separately for Plant A and Plant B. It should be noted that three specimens were prepared for all combinations of RAP content levels and binder types in Table 12. This results in 36 specimens (4*3*3) for Plant A and 18 specimens (3*2*3) for Plant B.

5.3.1 Analysis Results for Plant A

Since the focus of this section is on the effect of RAP content on dynamic modulus, boxplots were first generated by RAP content. The boxplots are shown in Figures 10 through 12, respectively for each binder type.

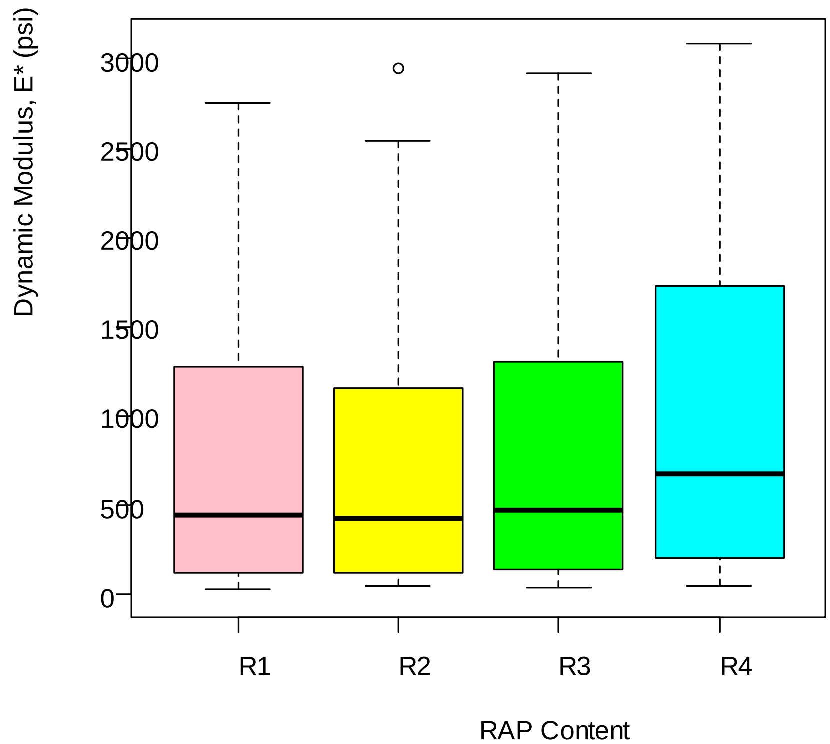


Figure 10: Dynamic Modulus by RAP content (Superpave, Binder: PG64-22, Plant A)

As shown in Figure 10, for binder PG64-22, there were not much difference among 0% RAP (R1), 15% RAP (R2), and 25% RAP (R3). As RAP content increases to 30% (R4), the mean and variance of dynamic modulus increase considerably. As shown in Figure 11, a similar trend can be observed for the PG67-22 Plant A mixture. It suggests that higher RAP contents (25% and 30%) result in higher dynamic modulus with increased variance.

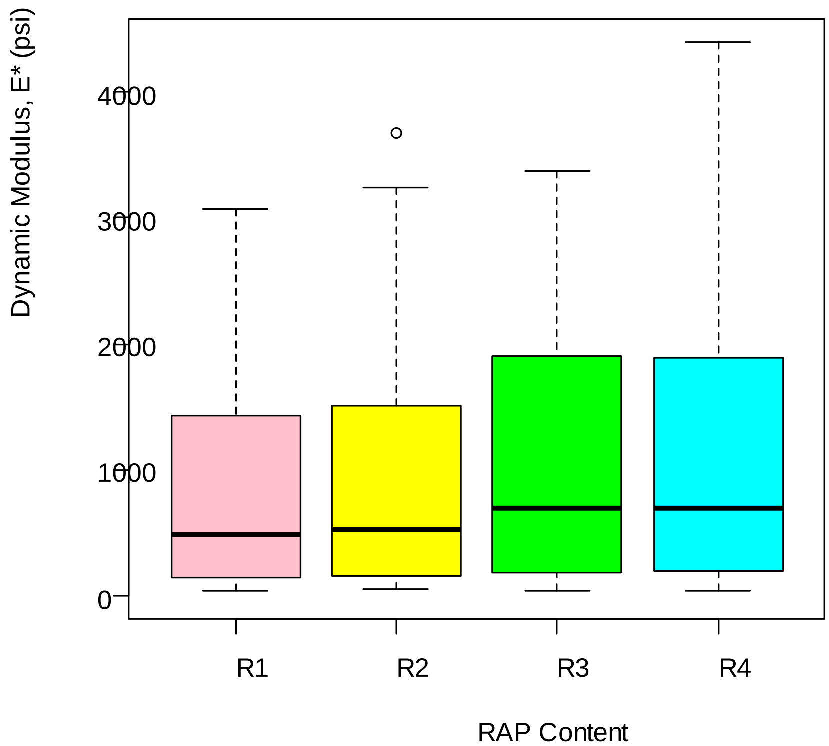


Figure 11: Dynamic Modulus by RAP content (Superpave, Binder: PG67-22, Plant A)

For binder PG76-22, higher RAP contents (25% and 30%) results in higher dynamic modulus as shown in Figure 12. The variance of dynamic modulus increase dramatically when RAP content increases to 30%.

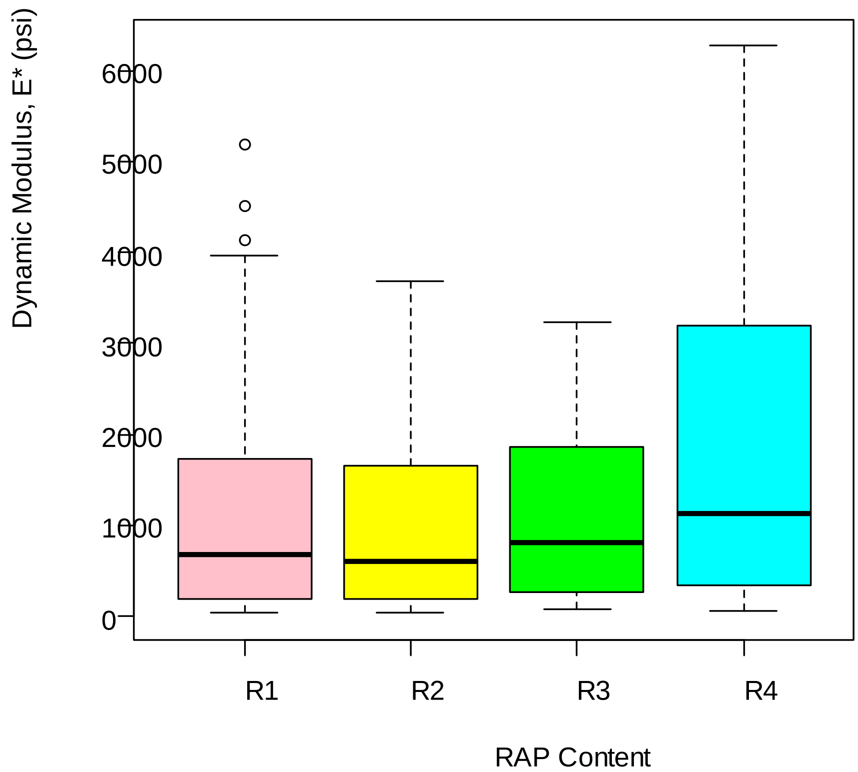


Figure 12: Dynamic Modulus by RAP content (Superpave, Binder: PG76-22, Plant A)

In summary, it was observed that:

- Higher PG binder type results in higher dynamic modulus, implying that the stiffer the binder, the higher the dynamic modulus.
- The dynamic modulus generally increases as RAP content increases for Plant A mixture.

As described previously, a mixed design ANOVA was applied to capture the between and within subject errors. The ANOVA results are presented in Table 13.

Table 13. Analysis of Variance of Dynamic Modulus (Superpave, Plant A)

Error: Specimen

Effect	DF	SS	MS	F	p value	
RAP	3	29770465	9923488	16.337	5.35E-06	***
Binder	2	26187773	13093887	21.557	4.37E-06	***
RAP:Binder	6	9909723	1651620	2.719	0.0369	*
Residuals	24	14577785	607408			

Error: Specimen:Temp

Effect	DF	SS	MS	F	p value	
Temp	2	734571308	3.67E+08	762.142	<2e-16	***
Temp:RAP	6	24068646	4011441	8.324	3.36E-06	***
Temp:Binder	4	19497295	4874324	10.115	5.09E-06	***
Temp:RAP:Binder	12	11795026	982919	2.04	0.0408	*
Residuals	48	23131779	481912			

Error: Specimen:Freq

Effect	DF	SS	MS	F	p value	
Freq	1	59473547	59473547	1495.302	<2e-16	***
Freq:RAP	3	1032956	344319	8.657	0.000453	***
Freq:Binder	2	582040	291020	7.317	0.003303	**
Freq:RAP:Binder	6	382165	63694	1.601	0.190027	
Residuals	24	954567	39774			

Error: Specimen:Temp:Freq

Effect	DF	SS	MS	F	p value	
Temp:Freq	2	18447964	9223982	440.673	<2e-16	***
Temp:Freq:RAP	6	173895	28982	1.385	0.2401	
Temp:Freq:Binder	4	191467	47867	2.287	0.0736	.
Temp:Freq:RAP:Binder	12	360921	30077	1.437	0.1825	
Residuals	48	1004715	20932			

Error: Within

	DF	SS	MS
Residuals	648	36748072	56710

Notes

Significance level: ***0, ** 0.001, *0.01, · 0.05

DF = degrees of freedom

SS = sum of squares

MS = mean sum of squares

F = F statistic

The level of significance for factorial effects is indicated by the p value. The generally accepted significance level thresholds are 0.05, 0.01, and 0.001, noted by dot, single asterisk, and double asterisks, respectively. If the p value for a corresponding effect is less than any of the established significance level thresholds, it will be considered significant at that level. As seen in Table 13, both RAP content and binder type (between-subject factors), and their interaction have a significant effect on the dynamic modulus, as indicated by the very small p values (5.35E-06 and 4.37E-06). Temperature and frequency (within-subject factors) and their interaction are also significant, as expected, so do their interactions with RAP content and binder type.

ANOVA indicates whether particular factors and/or their interactions have significant effects on dynamic modulus. However, it does not provide insight in the direction of influence. To identify the direction of significant effects, paired t test was employed. The paired t test was conducted for each two consecutive levels of RAP content with respect to each binder type. The results are presented in Table 14. Similarly, the paired t test was conducted for each two consecutive levels of binder with respect to each level of RAP content. The results are presented in Table 15.

Table 14. Effects of RAP contents on Dynamic Modulus (Superpave, Plant A)

Binder type	Difference in RAP content	Mean	Standard deviation	t statistic	p-value	
PG 64-22	R15 - R0	-51.914	183.066	-2.406	0.00936	*
	R25 - R15	69.406	240.120	2.453	0.00832	*
	R30 - R25	239.467	199.284	10.196	7.5E-16	***
PG 67-22	R15 - R0	71.113	297.810	2.026	0.02325	·
	R25 - R15	173.087	335.866	4.373	2.1E-05	***
	R30 - R25	116.361	439.640	2.246	0.01391	·
PG 76-22	R15 - R0	-212.180	741.584	-2.428	0.00886	*
	R25 - R15	151.981	180.050	7.162	3.0E-10	***
	R30 - R25	690.570	815.046	7.189	2.6E-10	***

Significance level: ***0, ** 0.001, *0.01, · 0.05

As shown in Table 14, for binder PG64-22 and PG76-22, RAP content has a significant effect (at least 1% level since all p values are less than 0.01) on dynamic modulus. Specifically, 15% RAP results in significantly lower dynamic modulus as compared to 0% RAP. It should be noted that all the JMFs was approved by GDOT except one for Plant A 15% RAP mix. Further, the binder content was not designed based on GDOT Section 828, SOP 2 procedure. For this reason, comparison of 15% RAP mix performance with other mixture performance will not be appropriate. Mixtures with 25% RAP result in significantly higher dynamic modulus as compared to 15% RAP. Mixtures with 30% RAP result in significantly higher dynamic modulus as compared to the mixtures with 25% RAP.

For binder PG67-22, all RAP contents have significant effects (at least 5% level since all p values are less than 0.05) on dynamic modulus. A consistent trend was observed. The higher RAP content results in significantly higher dynamic modulus as compared to the lower RAP content.

Table 15. Effect of Binder Types on Dynamic Modulus (Superpave, Plant A)

RAP content	Binder difference	Mean	Standard deviation	t statistic	p-value	
RAP 0%	B2 - B1	69.820	161.851	3.660	0.00024	**
	B3 - B2	321.413	586.372	4.651	7.4E-06	***
RAP 15%	B2 - B1	192.847	357.773	4.574	9.9E-06	***
	B3 - B2	38.121	239.424	1.351	0.09049	
RAP 25%	B2 - B1	296.527	371.749	6.768	1.6E-09	***
	B3 - B2	17.015	241.252	0.598	0.27572	
RAP 30%	B2 - B1	173.422	471.341	3.122	0.00130	*
	B3 - B2	591.223	651.656	7.698	3.0E-11	***

Significance level: ***0, ** 0.001, *0.01, · 0.05

As shown in Table 15, for 0% and 30% RAP mixtures, the binder type has a significant effect (at least 1% level) on dynamic modulus. Higher PG grade binder results in higher dynamic modulus. For 15% and 25% RAP mixture, PG67-22 results in significantly higher dynamic modulus as compared to PG64-22. PG76-22 generally results in higher dynamic modulus as compared to PG67-22, but the effects are not significant at the 5% level. From the observation, it is concluded that:

- Higher RAP content generally results in higher dynamic modulus.
- Higher binder type generally results in higher dynamic modulus.

5.3.2 Analysis Results for Plant B

Plant B data was subject to same statistical analyses. The boxplots were generated first, followed by mixed design ANOVA and paired t test. As shown in Figure 13, with PG64-22 binder, 25% RAP (R3) result in much higher mean and variance of dynamic modulus as compared to 0% and 15% RAP (R1 and R2), which are similar to Plant A. As shown in Figure 14, with PG67-22 binder, a higher RAP content results in a higher mean and variance of dynamic modulus.

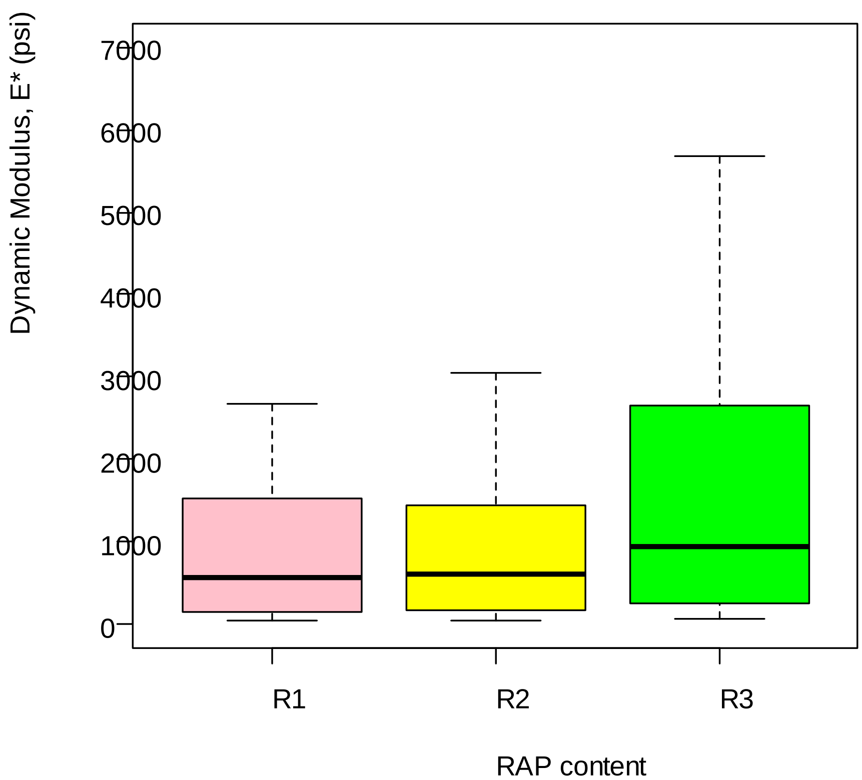


Figure 13: Dynamic Modulus by RAP content (Superpave, Binder: PG64-22, Plant B)

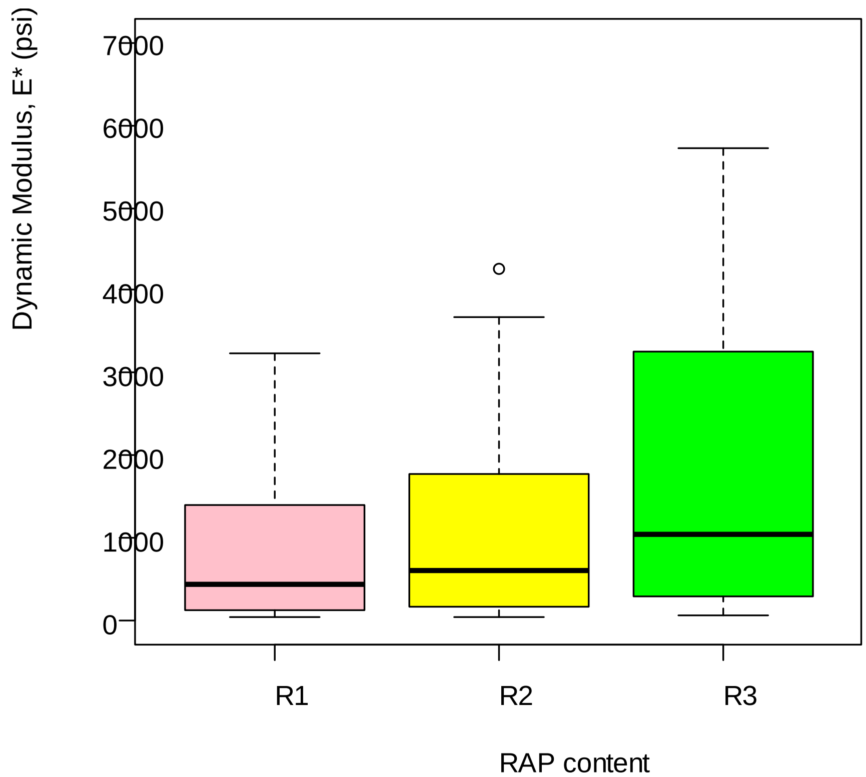


Figure 14: Dynamic Modulus by RAP content (Superpave, Binder: PG67-22, Plant B)

The results of mixed design ANOVA for Plant B is presented in Table 16. It can be seen that RAP content has a significant effect on dynamic modulus. However, the effect of binder type is not significant. Temperature and frequency are significant and so do their interactions with RAP content. Paired t test was conducted for Plant B data as well and the effects of RAP contents and binder types are summarized in Table 17 and Table 18, respectively.

Table 16. Analysis of Variance on Dynamic Modulus (Superpave, Plant B)

Error: Specimen

Effect	DF	SS	MS	F	p value	
RAP	2	54602363	27301182	37.979	6.45E-06	***
Binder	1	1778978	1778978	2.475	0.142	
RAP:Binder	2	1258461	629231	0.875	0.442	
Residuals	12	8626155	718846			

Error: Specimen:Temp

Effect	DF	SS	MS	F	p value	
Temp	2	4.53E+08	2.26E+08	550.323	<2e-16	***
Temp:RAP	4	50021710	12505427	30.417	4.41E-09	***
Temp:Binder	2	3026426	1513213	3.681	0.0403	*
Temp:RAP:Binder	4	735004	183751	0.447	0.7735	
Residuals	24	9867271	411136			

Error: Specimen:Freq

Effect	DF	SS	MS	F	p value	
Freq	1	34309264	34309264	475.664	5.06E-11	***
Freq:RAP	2	1488878	744439	10.321	0.00247	**
Freq:Binder	1	14733	14733	0.204	0.65937	
Freq:RAP:Binder	2	175613	87806	1.217	0.3301	
Residuals	12	865550	72129			

Error: Specimen:Temp:Freq

Effect	DF	SS	MS	F	p value	
Temp:Freq	2	10282515	5141257	173.034	5.54E-15	***
Temp:Freq:RAP	4	211179	52795	1.777	0.166	
Temp:Freq:Binder	2	19284	9642	0.325	0.726	
Temp:Freq:RAP:Binder	4	251582	62895	2.117	0.11	
Residuals	24	713097	29712			

Error: Within

	DF	SS	MS
Residuals	324	22054215	68069

Notes

Significance level: ***0, ** 0.001, *0.01, · 0.05

DF = degrees of freedom

SS = sum of squares

MS = mean sum of squares

F = F statistic

Table 17. Effect of RAP Contents on Dynamic Modulus (Superpave, Plant B)

Binder type	Difference in RAP content	Mean	Standard deviation	t statistic	p-value	
PG 64-22	R15 - R0	-21.781	219.240	-0.843	0.20104	
	R25 - R15	689.634	783.246	7.471	8.0E-11	***
PG 67-22	R15 - R0	183.490	400.035	3.892	0.00011	**
	R25 - R15	731.339	864.872	7.175	2.8E-10	***

Significance level: ***0, ** 0.001, *0.01, · 0.05

As shown in Table 17, the significance of the RAP effect is indicated by the p value. Similar to the ANOVA tables, the smaller the p value is, the more significant the effect will be. For the lower binder grade (PG64-22), a large p value of 0.20104 indicates that 15% RAP does not have a significant impact per the established significance levels on dynamic modulus as compared to 0% RAP. When RAP content goes up to 25%, the effect becomes significant as seen by the small p value of 8.0E-11. For the higher binder grade (PG67-22), both 15% and 25% RAP significantly affect dynamic modulus as compared to 0% RAP because of the small p values (0.00011 and 2.8E-10).

Table 18. Effect of Binder Types on Dynamic Modulus (Superpave, Plant B)

RAP content	Binder difference	Mean	Standard deviation	t statistic	p-value	
RAP 0%	B2 - B1	-22.405	246.362	-0.772	0.22143	
RAP 15%	B2 - B1	182.865	331.731	4.677	6.7E-06	***
RAP 25%	B2 - B1	224.570	403.788	4.719	5.8E-06	***

Significance level: ***0, ** 0.001, *0.01, · 0.05

Note: only two binder types (PG64-22 and PG67-22) were tested for Plant B. As shown in Table 18, no significant effect on dynamic modulus was found between the two binders for 0% RAP content, as seen by the large p value of 0.22143. However, PG67-22 results in significantly higher dynamic modulus as compared to PG64-22 when RAP content increases up to 15% and 25%, as indicated by the small p values of 6.7E-06 and 5.8E-06, respectively.

Based on the analyses for Plant B mixtures, it was observed that:

- In general, higher RAP content results in higher dynamic modulus. This is especially the case when higher RAP content is used in combination with a higher binder type.

SMA design was also studied with different RAP contents. For SMA, only PG76-22 binder was used. For RAP content, 10% RAP was used for Plant B and 15% RAP was used for Plant A. Those design variables are summarized in Table 19. Same analyses were carried out for SMA data set. The boxplots were generated first and are shown in Figure 15. As seen, there was not much difference in dynamic modulus for the three studied mix designs.

Table 19. Design Variables and Coding (SMA)

Variable	Value	Code	Plant
RAP Content	0%	R0	A
	10%	R10	B
	15%	R15	A
Binder	PG76-22		A, B
Frequency	0.1 Hz	F1	A, B
	0.5 Hz	F2	A, B
	1.0 Hz	F3	A, B
	5.0 Hz	F4	A, B
	10 Hz	F5	A, B
	25 Hz	F6	A, B
Temperature	40 ^{o}F	T1	A, B
	70 ^{o}F	T2	A, B
	100 ^{o}F	T3	A, B
	130 ^{o}F	T4	A, B

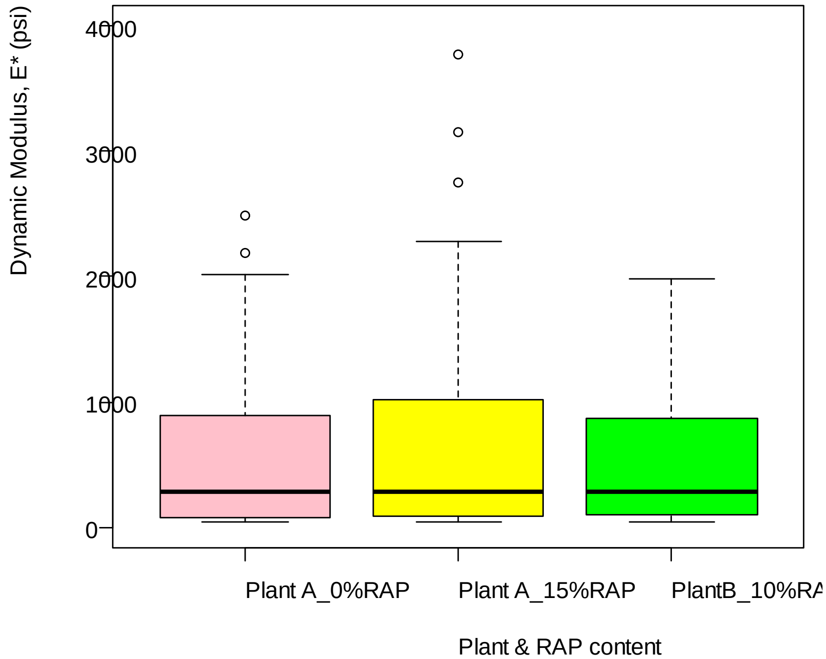


Figure 15: Dynamic Modulus by RAP content (SMA, Binder: PG76-22, Plant A, B)

For Plant A, the two RAP contents (0% and 15%) were analyzed using the same mixed design ANOVA and the results are presented in Table 20. As indicated by the large p values for RAP (0.344), 15% RAP does not have a signficant effect on dynanmic modulus. As expected, temperature, frequency and their interaction are significant as indicated by the small p values in Table 20.

Table 20. Mixed Design Analysis of Variance (SMA, Plant A)

Error: Specimen

Effect	DF	SS	MS	F	p value	
RAP	1	492141	492141	1.148	0.344	
Residuals	4	1714232	428558			

Error: Specimen:Temp

Effect	DF	SS	MS	F	p value	
Temp	2	57650066	28825033	48.921	3.26E-05	***
Temp:RAP	2	1347667	673834	1.144	0.366	
Residuals	8	4713696	589212			

Error: Specimen:Freq

Effect	DF	SS	MS	F	p value	
Freq	1	4186531	4186531	151.553	0.00025	***
Freq:RAP	1	21943	21943	0.794	0.42316	
Residuals	4	110497	27624			

Error: Specimen:Temp:Freq

Effect	DF	SS	MS	F	p value	
Temp:Freq	2	2569814	1284907	19.811	0.000796	***
Temp:Freq:RAP	2	61466	30733	0.474	0.639019	
Residuals	8	518863	64858			

Error: Within

	DF	SS	MS
Residuals	108	2565549	23755

Notes

Significance level: ***0, ** 0.001, *0.01, · 0.05

DF = degrees of freedom

SS = sum of squares

MS = mean sum of squares

F = F statistic

5.4 Fatigue Performance Test Results

Controlled crosshead (CX) tension cyclic fatigue tests were performed in order to evaluate the fatigue performance of different mixtures. For a reasonable comparison, only Superpave mixtures fabricated by GDOT-approved JMF were subjected to CX tension cyclic fatigue tests. Depending on the material availability and test results, two to four replicates were tested.

5.4.1 Damage Characterization

Damage characteristic curves can show the material properties of stiffness and brittleness or ductility. For further detailed investigation, damage characteristic curves were plotted in one graph for each binder performance grade, as presented in Figure 16. As illustrated, the non-RAP mixes with different binder grades show a rapid decrease in material integrity with an increase in damage compared to the mixtures with 25 percent and 30 percent RAP content. This outcome is due to the mixture becoming more brittle once the RAP is incorporated into the mix. It can be interpreted that the damage in mixes with higher RAP contents grows slightly faster than in other mixes at the beginning of loading.

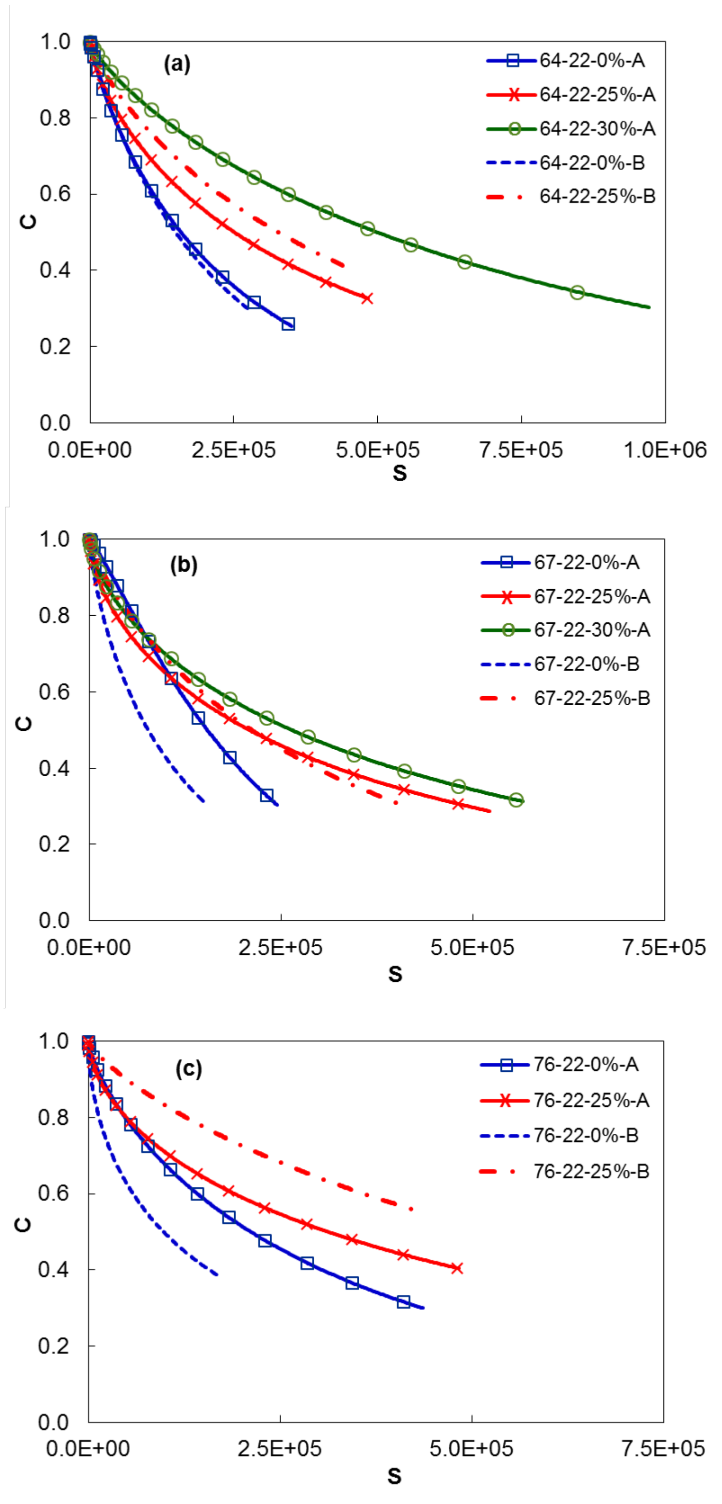


Figure 16. Damage characteristic curves: (a) PG 64-22, (b) PG 67-22, and (c) PG 76-22.

5.4.2 Fatigue Performance Assessment at Material Level Using Failure Criterion

The limitation of the damage characteristic curve, i.e., the *C* versus *S* curve, is that it cannot explain when a specimen fails. Thus, in order to evaluate the failure of asphalt materials more comprehensively, a failure criterion is required. As explained previously, the concept of the pseudo strain energy release rate provides a failure criterion that is independent of strain rate, temperature, and mode of loading. This failure criterion thus is considered a mixture property.

Figure 17 describes this failure criterion for the RAP 0%, RAP 25%, and RAP 30% mixtures for all the binder grades and plant mixture types. Fatigue cracking potentials of different mixes can be evaluated from Figure 17 by observing the locations of log of G^R versus log of N_f lines. A log G^R versus log N_f line that is positioned above has a better cracking resistance than the one positioned lower. Based on this observation, it can be seen from Figure 17 that the virgin mixes with the different binder grades show poorer performance than the mixtures containing RAP. On the other hand, the addition of RAP up to 25 percent significantly improved the mixtures' fatigue resistance, especially in the mixtures with binder grades of PG 64-22 and PG 67-22, as the corresponding failure criterion lines for the RAP mixtures lie above those of the virgin mix for each binder grade. Based on the GDOT's COAC concept, the RAP mixtures are likely to contain more binder than the virgin mixes, so the inclusion of RAP into a virgin mix increases the overall mixture binder content. However, the G^R also is related to the stiffness of the material and the pavement structure. Therefore, in order to evaluate fatigue life accurately, simulations should be performed on pavement structures. However, due to the lack of specimens, only two replicates were tested for some of the mixtures. With only two points, it was difficult to reach a definitive conclusion on the validity of the failure criterion. Therefore, more comprehensive COAC study including field validation is highly recommended. The COAC study with additional testing plan at different strain amplitudes will provide more reliable failure envelope.

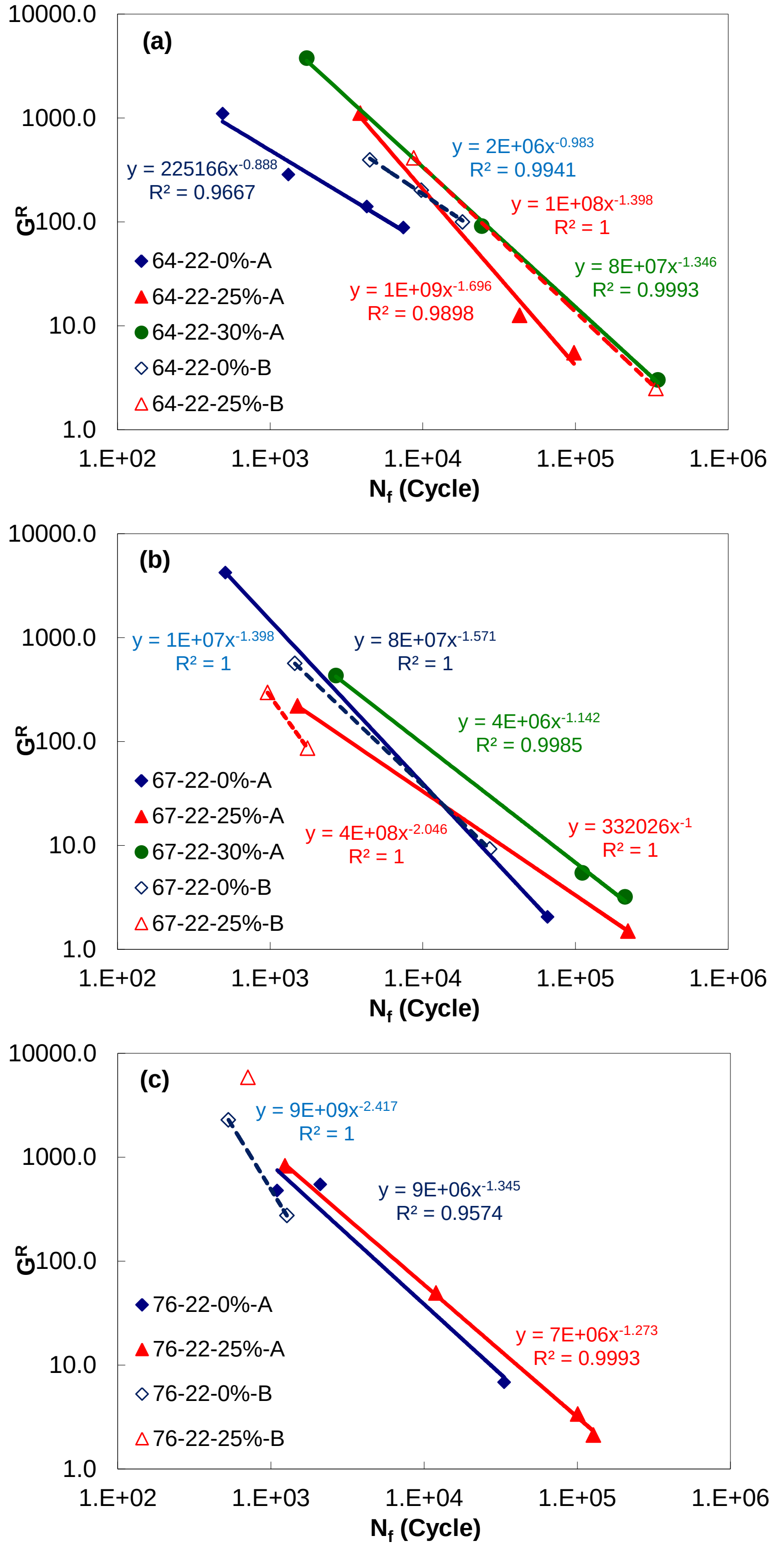


Figure 17. Failure criterion of mixtures: (a) PG 64-22, (b) PG 67-22, and (c) PG 76-22.

6. PERFORMANCE EVALUATION USING LVECD PROGRAM SIMULATIONS

6.1 Overview of LVECD Program

As expressed in the name of the LVECD program, i.e., 'Layered ViscoElastic pavement analysis for Critical Distresses' (LVECD), this software adopts viscoelastic analysis and the VECD model to obtain stress-strain responses and pavement performance. This program can simulate layered structures, including the asphalt layer, base layer, and subgrade. Thus, users can simulate an actual pavement structure. The LVECD program also evaluates thermal stress, fatigue, and rutting performance, which are the critical distresses of asphalt pavements, using various loading conditions. This study employed LVECD program simulations to evaluate pavement performance using laboratory test results.

The LVECD program utilizes a time-scale separation scheme to increase its computing efficiency for stress-strain calculations (i.e., response analysis). The LVECD program combines the concepts of Fourier transform and finite element discretization to provide simulation times that are orders of magnitude smaller than those found in conventional three-dimensional finite element models (Eslaminia, 2012). This method captures the effects of viscoelasticity and the moving nature of traffic loads with high efficiency. In addition, the time-scale separation scheme was developed under the assumption of gradual changes in damage (over a few weeks) and pavement temperature (over a few hours). This scheme allows millions of cycles to be reduced to hundreds of cycles by integrating the analysis with an extrapolation technique. The entire procedure, from entering inputs to viewing outputs, is operated through a user-friendly graphic interface, as shown in Figure 18, which is similar to that found in the Mechanistic-Empirical Pavement Design Guide (Pavement ME) software. Other inputs for LVECD program analysis are likewise similar to those of the Pavement ME. Structural information, such as layer thickness and VECD properties, which can be obtained through the S-VECD model protocol, can be entered easily (see Figure 18). The LVECD program uses pavement temperatures obtained from Enhanced Integrated Climate Model (EICM) software. The EICM program provides hourly temperatures of asphalt pavements in terms of pavement depth. Moreover, the traffic data window allows users to enter various types of vehicle loading. For example, standard loading that includes equivalent single-axle load (ESAL) input, single wheel data, and user-defined vehicle configurations can be simulated. Analysis results (i.e., damage evolution, stress, and strain) can be evaluated in terms of spatial distribution and time history distribution.

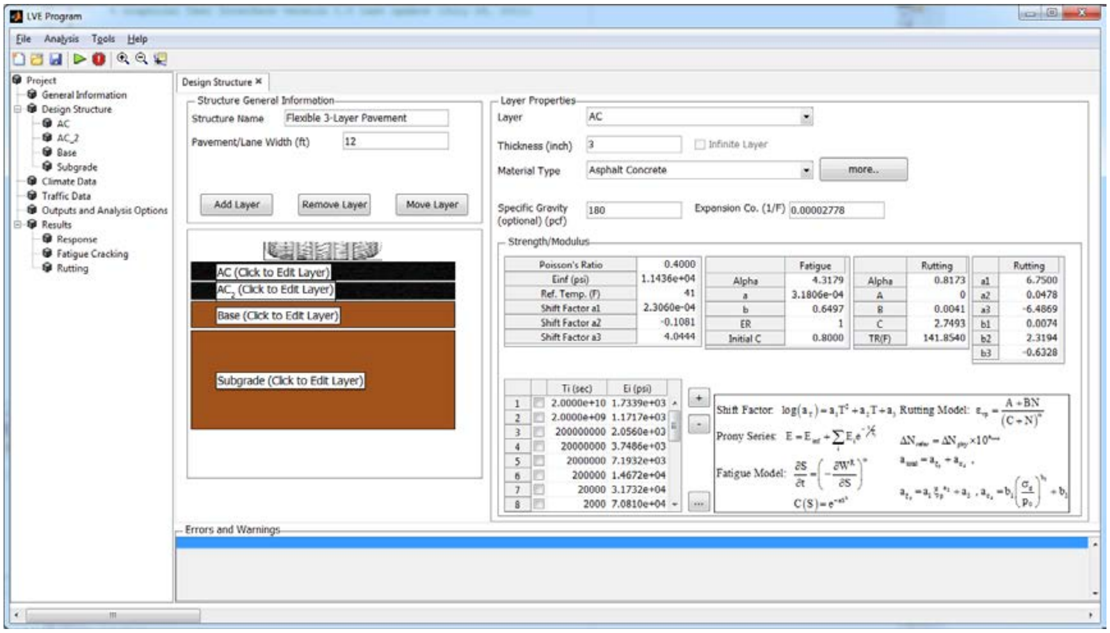


Figure 18. Screen shot of LVECD program input window.

6.2 LVECD Program Simulation Conditions

In order to evaluate the performance of asphalt concrete, dynamic modulus and fatigue tests were conducted in this study and the mixture properties were entered into the LVECD program. The properties of the base layer and subgrade that typically are used in pavements also were entered. As mentioned, the LVECD program can simulate the effects of climate change. For this study, the climate of Atlanta, GA was converted into pavement temperature via the EICM software; then, the temperature of the asphalt pavement was applied to the LVECD program simulations.

As shown in Figure 19, a pavement structure was applied to help determine the performance of the different asphalt pavement systems. The asphalt mixture properties entered into the LVECD program simulations already had been obtained from the material testing. All the conditions were kept the same except for the asphalt properties. The LVECD program can simulate

20 years of asphalt concrete pavement performance using climate and traffic data. The average annual daily truck traffic (AADTT) is 2,000, so the total ESAL is 14.4 million for a 20-year simulation. Also, the stress-strain distribution and damage evolution including a damage index were compared.

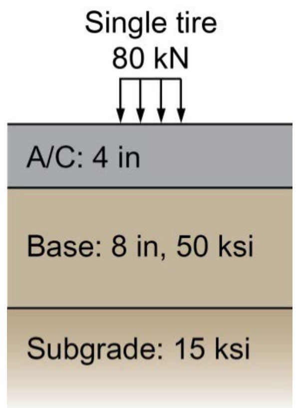


Figure 19. Pavement structure used in LVECD program simulations.

6.3 Performance Predictions Using LVECD Program

6.3.1 Stress-Strain

Asphalt concrete is a viscoelastic and viscoplastic material; thus, temperature will change the properties of asphalt concrete. Transverse strain, especially tensile strain, is related more to bottom up cracking and vertical stress is related directly to the rut depth evolution.

6.3.2 Fatigue Performance Evaluation

The LVECD program calculates damage growth (i.e., the reduction of the secant modulus) and the damage factor that is defined as Equation (20). N is the accumulated number of cycles and N_f is the number of cycles to failure. The *damage factor* indicates how many cycles are left to failure. If the damage factor is equal to ‘0’, the element does not have any damage. A damage factor of ‘1’ indicates failure of the element. At the time of analysis, the LVECD software calibration was in process. Therefore, this software has been used to compare the general behavior of different mixtures under cyclic loading.

$$Damage\ Factor = \frac{N}{N_f} \tag{20}$$

Figure 20 presents the amount of fatigue cracking for different pavement sections with multiple binder grades and plant mixture types. The mixture with 0% RAP shows poor performance compared to the other mixes for all conditions. With the inclusion of RAP, the mixture's fatigue resistance improves, probably due to the increase in binder content in accordance with COAC method. However, a change in damage would not necessarily clearly present any specific trend about which plant mixture has better fatigue resistance. Furthermore, as seen in Figure 20, no significant difference can be observed between the PG 64-22 and PG 67-22 sections. However, the use of PG 76-22 binder seems to improve the fatigue cracking resistance compared to the other binder grades.

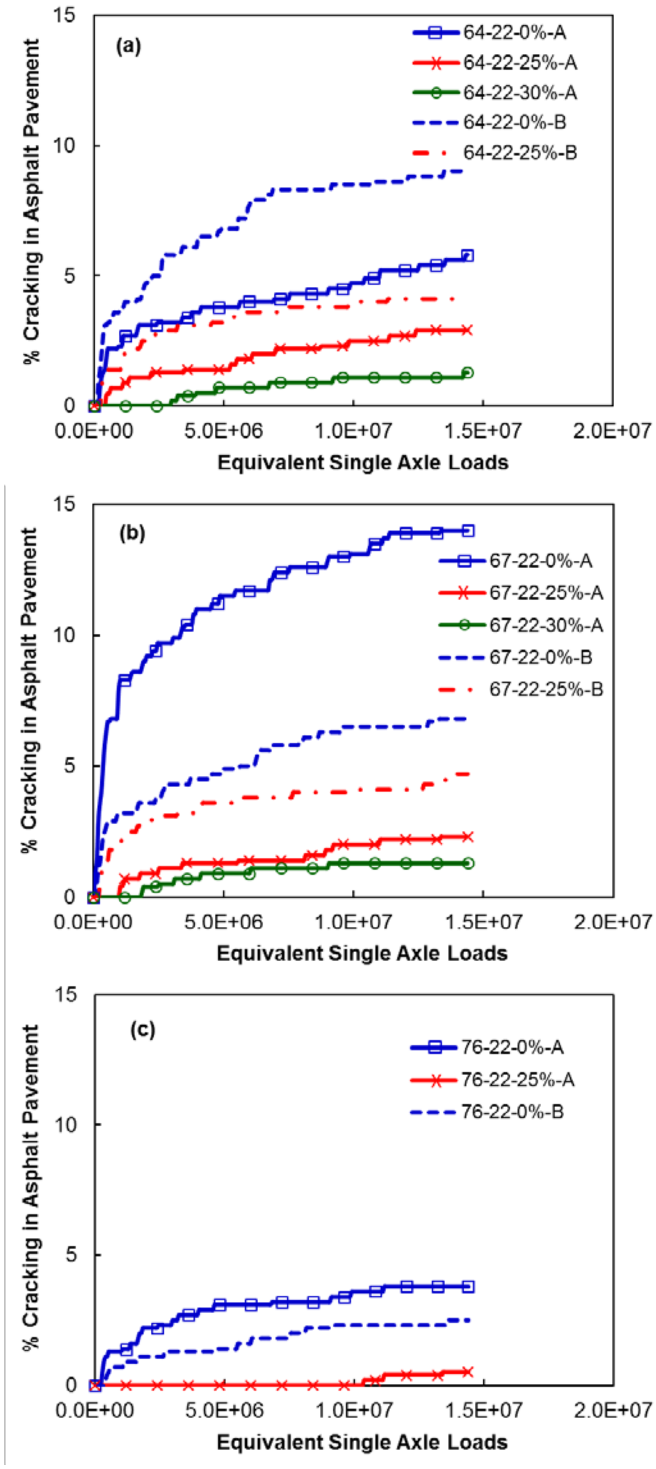


Figure 20. Amount of fatigue cracking for different sections: (a) PG 64-22, (b) PG 67-22, and (c) PG 76-22.

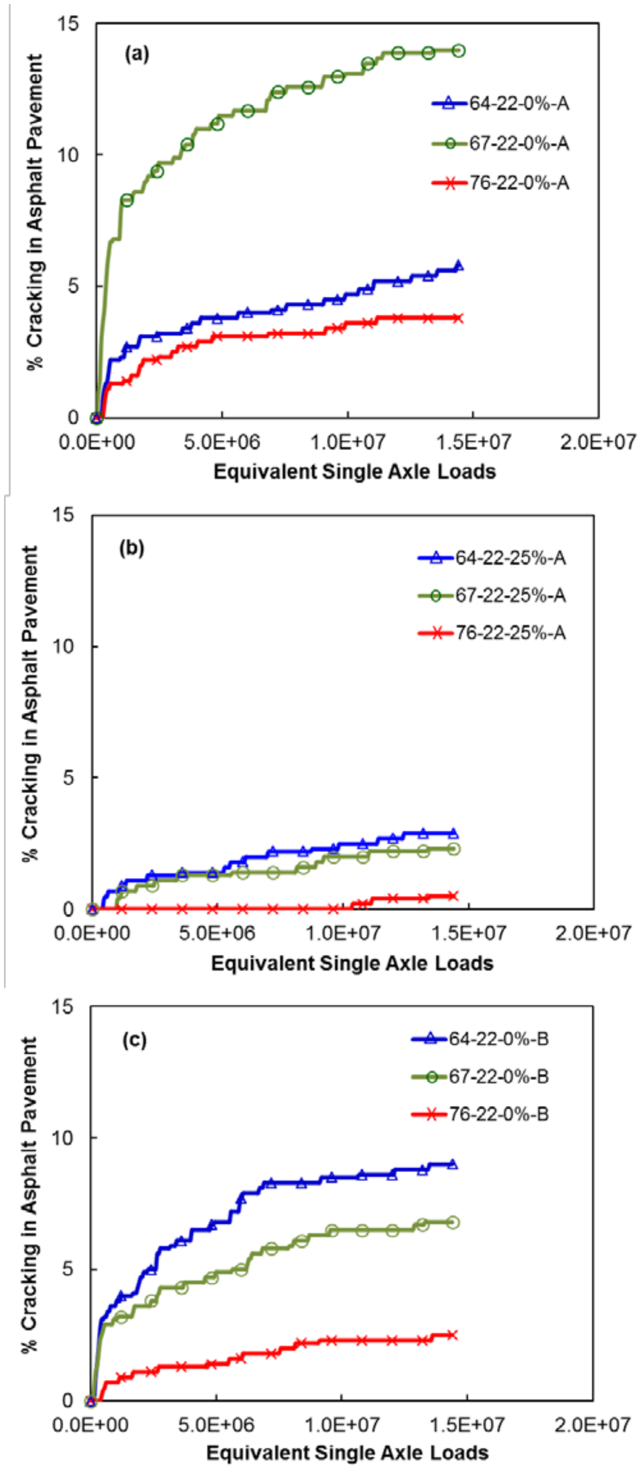


Figure 21. Amount of fatigue cracking for different binder grades: (a) 0% RAP Plant A, (b) 25% RAP Plant A, and (c) 0% RAP Plant B.

7. PERFORMANCE PREDICTION

In addition to LVECD analyses, simple performance prediction using Pavement ME was conducted.

7.1 Pavement ME Simulation Conditions

Pavement ME (TRB 2004) has been used to design and evaluate pavement. For this study, Pavement ME is used. To evaluate the effect of PG 64-22 and PG 67-22 on asphalt mixture performance, other simulation conditions are maintained constant. Only dynamic modulus properties are changed. For the accurate and rigorous comparison, both fatigue and rutting tests should be conducted and used to calibrate fatigue (Equation (21) and rutting (Equation (22)) models in Pavement ME.

NCHRP Project 9-22 developed closed-form solutions for both fatigue and rutting performance of Pavement ME based on |E*| values, which implies that |E*| is one of the key factors that governs performance in Pavement ME. Therefore, the nation-default values of k_1, k_2, and k_3 for both fatigue and rutting are used.

$$N_f = k_1\left(\frac{1}{\varepsilon_t}\right)^{k_2}\left(\frac{1}{E^*}\right)^{k_3} \tag{21}$$

$$\frac{\varepsilon_p}{\varepsilon_r} = 10^{k_1} T^{k_2} N^{k_3} \tag{22}$$

where,

$\varepsilon_r = \frac{\sigma}{|E^*(T)|_{@fr}}$

k_1, k_2, k_3 = calibration coefficients for both fatigue and rutting models,

[Note: coefficients k_1, k_2, k_3 in Equations (21) and (22) are not the same.]

ε_t = tensile strain at the bottom of asphalt pavement,

E^* = modulus,

ε_p = permanent strain,

ε_r = resilient strain,

$|E^*(T)|_{@fr}$ = the dynamic modulus at 10 Hz and the sublayer temperature for the given time increment of the analysis,

σ = the stress induced by a vehicle on the pavement structure,

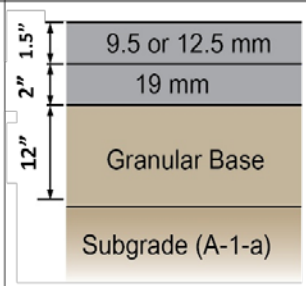
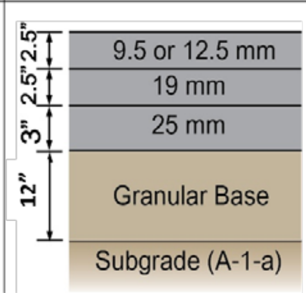
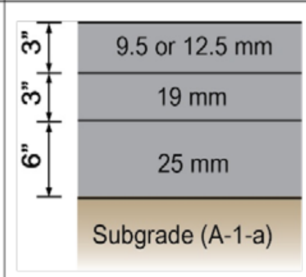
T = temperature, and

N = the number of repeated cycles.

All other inputs for Pavement ME simulation were default values in Pavement ME. Three-layer asphalt pavement structure was taken into consideration because asphalt pavement is usually constructed with multiple layers. The asphalt structures for the Pavement ME simulations are shown in Table 20. Three different structures (thin, intermediate, and thick) of asphalt pavement with equivalent traffic levels were investigated in order to evaluate the overall performance of the mixtures.

For simulations, three structures from one source (one plant) and same binder type were entered. For example, "Plant A PG64 C1" stands for the mixtures used in the simulation is Plant A mixtures that has PG64-22 binder and 12.5 mm surface mix. The climate around Atlanta, Georgia was selected for the Pavement ME simulations. The annual average daily truck traffic (AADTT) and designed ESALs (Equivalent Single Axle Loads) for 20-year simulation can be found in Table 21.

Table 21. Pavement ME Simulation Conditions

Thickness	Thin	Intermediate	Thick
Cross-Section			
Traffic (AADTT)	466	1553	3106
Design ESALs	3 million	10 million	20 million
Remarks	*Case 1*: surface layer is 9.5 mm mixture *Case 2*: surface layer is 12.5 mm mixture* Design service life (simulation period): 20 years		

7.2 Asphalt Performance

Pavement ME shows no transverse cracking (or thermal cracking). It would be because Atlanta, GA is considered as warm to hot region and thus, thermal cracking would not be a significant issue for asphalt pavement. However, longitudinal cracking and alligator cracking are estimated for all cases.

7.2.1 Low Temperature Performance: Cracking

Alligator cracking and longitudinal cracking are compared with six different cases by changing thickness of pavements (Thin, Intermediate, and Thick) and surface mixtures (case 1 and case 2). Each structure contains different binders and sources to compare their effects on the predicted performance. Figure 22 shows the evolution of alligator and longitudinal cracking. In the figure, solid lines present PG64-22 mixtures and dotted lines represent PG67-22 mixtures for Plants A and B. Alligator cracking for both intermediate and thick pavement structures is not shown in Figure 22 because they have about 0 % cracking.

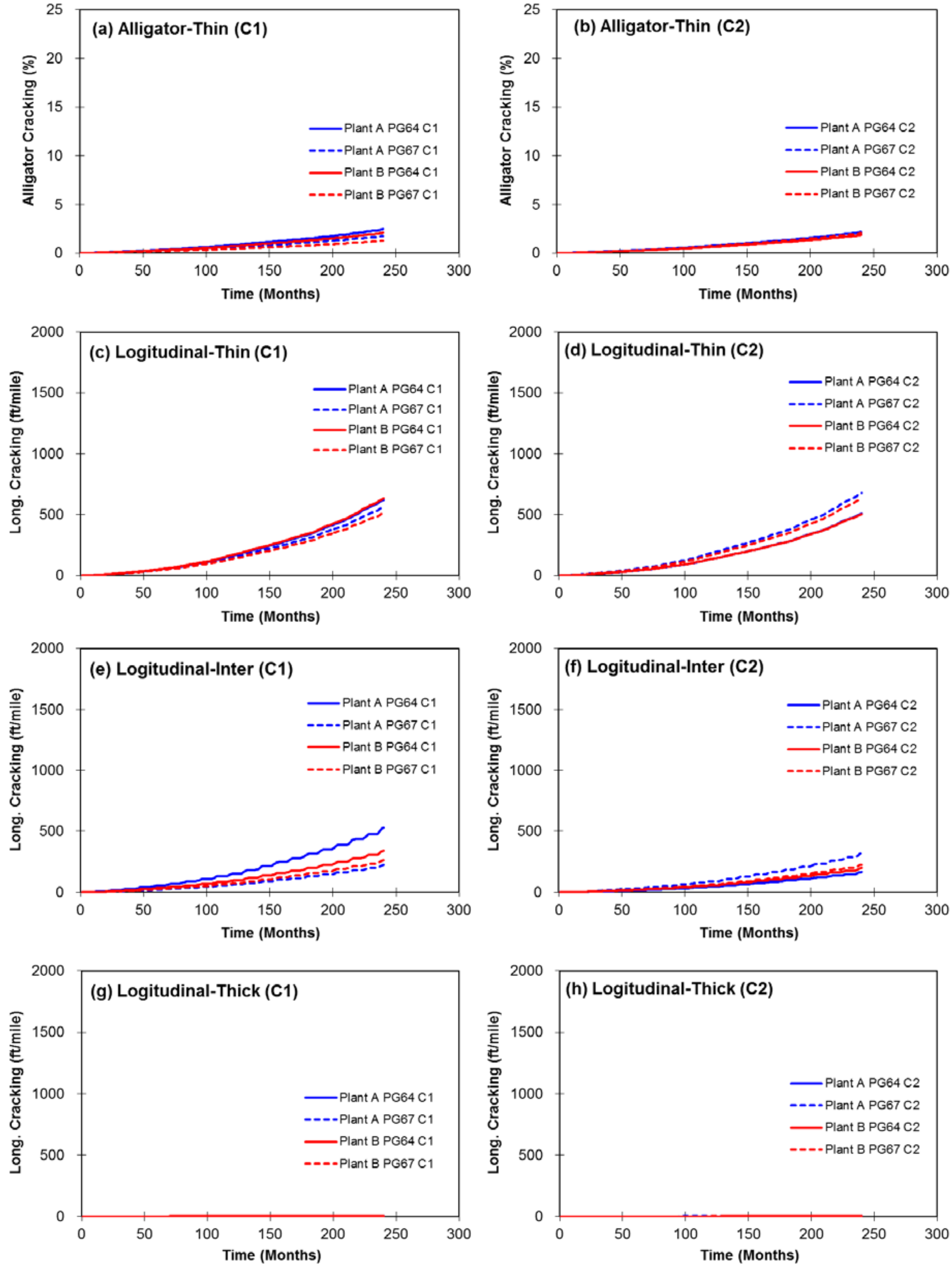


Figure 22. Cracking Development According to Pavement Structures

Both alligator and longitudinal cracking were within the design limits. The upper limit of vertical axis (y-axis) represents design limit values of each performance. As the pavement thickness increases (refer to Figure 22 (c) to (h)), the cracking decreases; thus, the alligator and longitudinal cracking might not be a major distress for thick pavement structures based on the simulation results. There seems to be no performance difference between Plant A and Plant B mixtures, because the trend of cracking evolution and the cracking amount are quite similar.

PG67-22 mixtures perform better in Case 1 (9.5 mm mixture at surface), but PG64-22 mixtures perform better in Case 2 (12.5 mm mixture at surface) in spite of small difference. In addition, considering design performance limits, the limit is 25% for alligator (or bottom-up cracking) and 2000 ft/mile for longitudinal cracking (or surface down cracking), the amounts of cracks estimated by Pavement ME are quite small that one can conclude that half-grade difference may not cause any typical performance difference in terms of cracking. In addition to this observation, two binders' low PG grade were both -22, so it can be expected that cracking behavior would be similar for these two mixtures because cracking is known as low temperature characteristics.

7.2.2 High Temperature Performance: Rutting

Binder with high PG is related to high temperature performance, i.e., rutting. That is, higher PG implies stronger rutting resistance, and thus higher PG is used at hot region to prevent rutting in asphalt pavement. Therefore, it is expected that PG67-22 mixtures would produce less rut depth than PG64-22 mixtures.

Figure 22 shows rut depth of asphalt concrete layer only. 20-years simulation shows that rut depths of all cases are also lower than design limit which is 0.25 inch and presented as dotted horizontal line in Figure 22. As expected, PG67-22 produces lower rut depth except for the Plant A in Case 2 (12.5 mm mixture at surface layer). It seems that stiffer dynamic modulus tends to result in lower rut depth in the predictions. Moreover, Pavement ME applies depth correction function to match predicted rut depth with field measurement. Top layer plays a key role in rut depth prediction of Pavement ME.

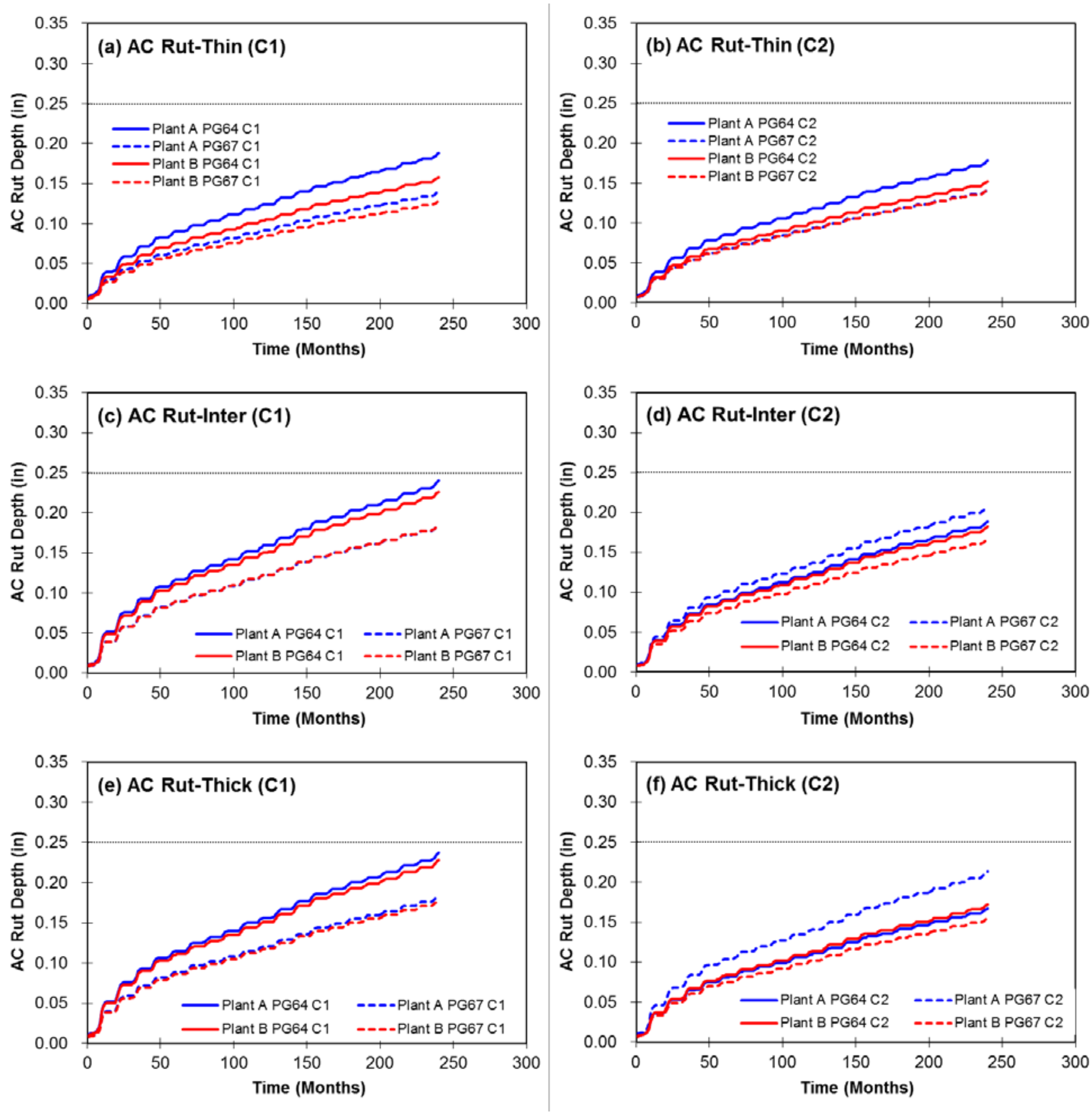


Figure 23. Rutting Prediction of Asphalt Concrete Layer Only

In this sense, in order to evaluate the effect of lowering the binder grade one half-grade of binder on rutting performance, the rutting model in Pavement ME should be calibrated with laboratory tests such as flow number type tests at 20, 40, and 54°C, which was out of the scope of this paper. The PG67-22 mixtures, in general, appear to have greater rutting resistance than PG64-22 mixtures, as expected. The average rut depth difference between Plant A and Plant B is about 10.5% and between PG67 and PG64 mixtures is about 21.3% at the end of 20-year by simulation. In the case of Case 2 with intermediate pavement thickness, differences between the PG64 and PG67 mixtures are 9.2% and 9.6% for Plant A and Plant B, respectively.

Although higher PG (PG 67) mixtures were more resistant to rutting than PG 64 mixtures, the change in rut depth performance prediction due to different binder PG grades and sources is insignificant based on Pavement ME analysis. The average difference is around 21% and the change of the typical asphalt pavement structure remains less than 10%. It should be noted that this comparison was conducted by Pavement ME simulation with default rutting coefficients, not with mixture specific coefficients.

7.2.3. Rideability

International roughness index, IRI, is used to represent road roughness for evaluating and managing road system. IRI is calculated based on the longitudinal profiles of wheel paths and is a function of pavement distresses, including fatigue and thermal cracking. IRI starts from initial IRI and combines with site factors such as subgrade and climate factors to compute IRI development. Therefore, the IRI is affected by performance predictions (i.e., longitudinal and alligator cracking) shown in previous sections. Therefore, IRI could be an index that explains the overall performance of asphalt pavement. Figure 24 illustrates IRI over time. IRIs from all different simulation conditions converge. This observation supports that only half-grade difference would not make a difference in the overall performance of asphalt pavements.

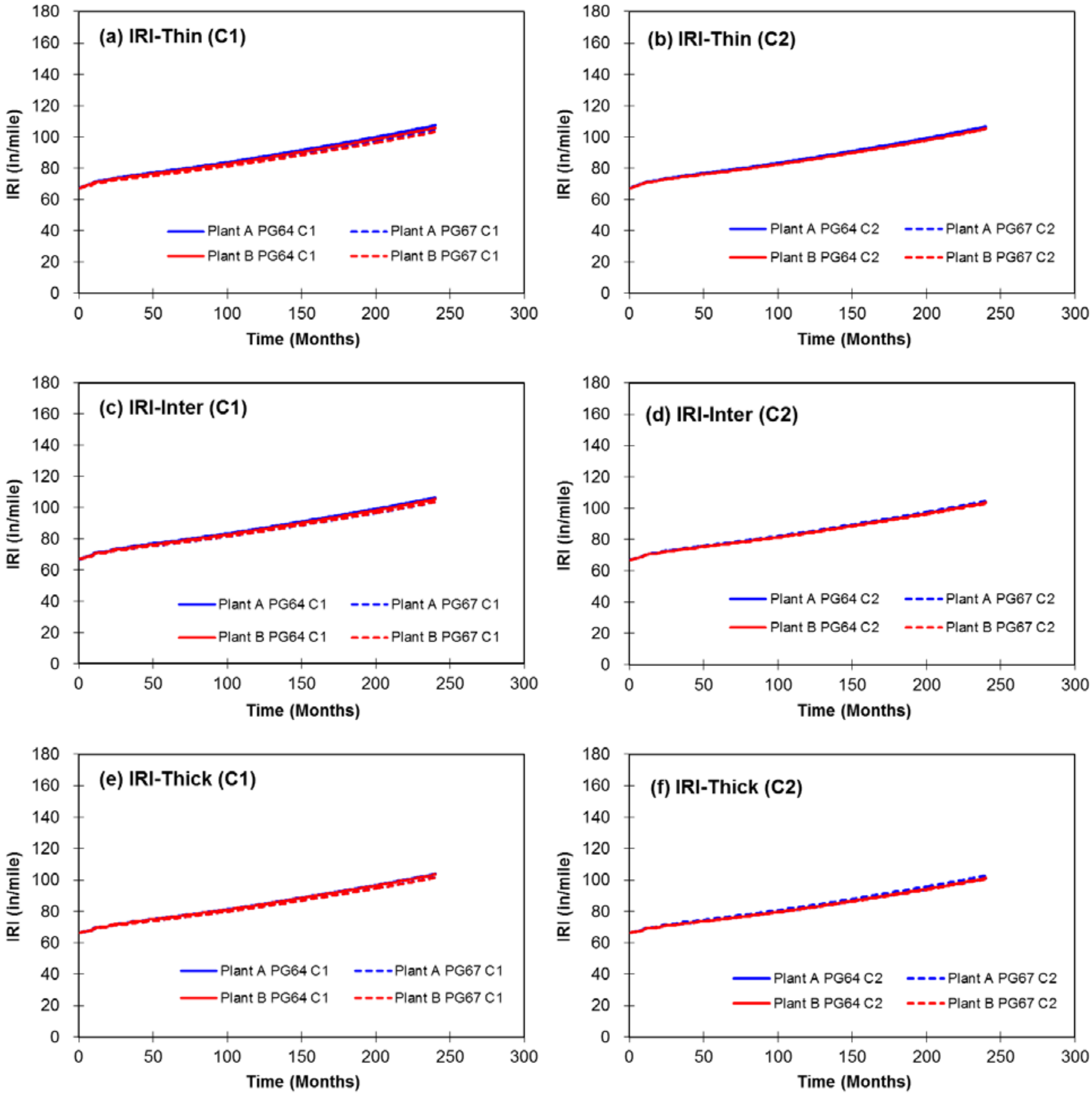


Figure 24. IRI prediction with initial IRI of 63

8. CONCLUSIONS

In the study, effects of RAP content, RAP source, and half-grade difference of high binder PG on mixture characteristics and performance were investigated. Categorical mixtures used in GA were fabricated and tested for dynamic modulus (|E*|) and direct cyclic tension fatigue tests.

- Overall, |E*| increases as %RAP increases. 12.5mm Superpave mixtures with higher PG binder and increased RAP content (up to 30% RAP) result in higher dynamic modulus as the mixtures become stiffer. All 30% RAP mixtures fell into the stiffest mixture group while dynamic moduli of 0% RAP mixtures fell into softest mixture group.
- GDOT has adopted the COAC method for asphalt mix design. For the usual Superpave asphalt concrete mixtures (25% RAP content) with PG 64-22 and PG 67-22, the increase of the binder percentage in accordance with COAC method induced an improvement of the fatigue life.
- Controlled crosshead tension cyclic fatigue tests were performed to investigate the fatigue performance of mixtures with different RAP content and binder type. The virgin mixes with the different binder grades show poorer performance than the mixtures containing RAP. On the other hand, the addition of RAP up to 25 percent significantly improved the mixtures' fatigue resistance, especially in the mixtures with binder grades of PG 64-22 and PG 67-22. It seems that this improvement is attributed to the GDOT's COAC method. According to the COAC method, the RAP mixtures are likely to contain more binder than the virgin mixes, so the inclusion of RAP into a virgin mix increases the overall mixture binder content.
- The LVECD program simulations were conducted to evaluate the critical distress of asphalt mixtures such as thermal stress, fatigue, and rutting performance in actual field performance via laboratory testing.
- A change in damage would not necessarily clearly present any specific trend about which plant mixture has better fatigue resistance.

- In Georgia, PG 64-22 and PG 67-22 mixtures are allowed to contractors. To investigate the effect of a half grade difference on pavement performance, Pavement ME analyses were conducted for 25% RAP mixtures with PG 64-22 and PG 67-22. The amounts of cracks estimated by Pavement ME are quite small that one can conclude that half-grade difference may not cause any typical performance difference in terms of cracking. It could be explained that two binders' low PG grade were both -22, so cracking behavior would be similar for these two mixtures because cracking is known as low temperature characteristics.
- Two different RAP sources and mixing plants are likely to produce similar mixtures in GA. Statistical analysis with |E*| cannot find any different linear viscoelastic characteristics due to different sources and mixing plants.
- Higher PG (PG 67) mixtures were more resistant to rutting than PG 64 mixtures. Difference of about 8 to 14% of total rut depth was observed between PG 64 and PG 67 mixtures. However, it is noted that the rut depth prediction is based on nationwide default values for rutting. Therefore, it is unclear if the half-grade change of binder would induce different mixture performance or not.

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APPENDIX A: GEORGIA STANDARD SPECIFICATIONS SECTION 828 AND SOP2

Section 828—Hot Mix Asphaltic Concrete Mixtures

828.1 General Description

This specification includes the requirements for hot mix asphaltic concrete mixtures, including:

- Open-graded surface mixtures
- Stone Matrix Asphalt mixtures
- Superpave asphaltic concrete mixtures
- Fine-graded mixtures

828.1.01 Definitions

Nominal Maximum Sieve Size: One standard sieve size larger than the first sieve to retain more than ten percent.

828.1.02 Related References

A. Standard Specifications

Section 800–Coarse Aggregate

Section 802–Aggregates for Asphaltic Concrete

Section 820–Asphalt Cement

Section 831–Admixtures

B. Referenced Documents

AASHTO TP 4

AASHTO PP 2

AASHTO TP 8-94

AASHTO T 112

AASHTO T 209

AASHTO T 305

Standard Operating Procedure (SOP) 2 SP–Control of Superpave Bituminous Mixture Designs

GDT 4

GDT 56

GDT 66

GDT 115

GDT 125

QPL 26

QPL 41

828.2 Materials

A. Requirements

All mixtures are designated based on the Nominal Maximum Sieve Size. Determine the amount finer than No. 200 (75 µm) by washing (See GDT 4) or by the correlation procedure described in GDT 125.

Use hot mix asphaltic concrete mixtures that meet the following requirements:

1. Ensure the materials used to prepare the mixtures are approved by the Engineer before incorporating into the Work.
2. Use aggregate groups and blends that meet the following pay item designations, as indicated in the Proposal and Plans:

Pay Item Designation	Allowable Aggregate Groups
Group I or II	100% of Group I, Group II, or Blend I.
Group II only	Only 100% Group II.
Blend I	Either 100% Group II material or a blend of Group I and Group II. Do not use Group I material for more than 60% by weight of the total aggregates, nor more than 50% by weight of the coarse aggregate portion.

3. Use Group I, Group II, or a blend of both aggregate groups, for patching or leveling. Mixes are listed in Subsection 828.2.03 and Subsection 828.2.04.
4. Design mixes using the Superpave System for Volumetric Design (AASHTO TP 4 and AASHTO PP 2) unless stated otherwise. Designs shall be performed by qualified and approved laboratories and technicians as specified in SOP-2 SP - Control of Superpave Bituminous Mixture Designs.
5. Ensure individual test results meet Mixture Control Tolerances
6. Include hydrated lime in all paving courses except where noted. For a list of hydrated lime sources, see QPL 41.
   a. Add lime to virgin aggregate mixtures at a minimum rate of 1 percent of the total dry aggregate weight.
   b. Add lime to recycled mixtures at a minimum rate of 1 percent of the virgin aggregate portion, plus a minimum of 0.5 percent of the aggregate in the reclaimed asphalt pavement (RAP) portion.
   c. Add more lime and an approved heat-stable, anti-stripping additive that meets the requirements of Subsection 831.2.04, “Heat Stable Anti-Stripping Additive,” if necessary, to meet requirements for mixture properties. However, the Department will not pay for the additional required materials. For a list of Heat Stable Anti-Stripping Additive sources, please see QPL 26.
   d. On PR, LARP, airport, bridge replacement, and parking lot projects designated at Mix Design Level A, asphalt cement may include an approved, heat-stable, anti-stripping additive that meets the requirements of Subsection 831.2.04, “Heat Stable Anti-Stripping Additive” instead of hydrated lime, unless specified in the Pay Item.
      1) Add at a minimum rate of 0.5 percent of the AC portion.
      2) Ensure the additive treated mix meets the minimum tensile splitting ratio:

Tensile Splitting Ratio	Type of Asphaltic Concrete
0.4	4.75 mm mix
0.6	All other mixes

7. Use performance grade PG 67-22 asphalt cement in all mixtures except as follows:
   a. For RAP mixtures, the Engineer will determine the performance grade to be used.
   b. On PR, LARP, airport, bridge replacement, and parking lot projects, PG 64-22 may be substituted for PG 67-22.
   c. Use only performance grade PG 76-22 for all mixtures that specify polymer-modified asphalt in the pay item designation.
8. Use of local sand is restricted as follows:
   a. No more than 20 percent, based on total aggregate weight, may be used in mixtures for shoulder construction and on projects designed at Mix Design Level A.
   b. For mixtures placed on the mainline traveled way of projects designed at Mix Design Level B, C, or D (except interstate projects), local sand may be used only in the 25 mm Superpave and shall not exceed 20 percent based on total aggregate weight.
   c. Do not use local sand in any mixture placed on the traveled way of Interstate mainline or ramps. No more than 20 percent local sand, based on total aggregate weight, may be used in mixtures for shoulder construction.
   d. Do not use local sand that contains more than 7 percent clay.
   e. Do not use local sand that contains any clay lumps as determined by AASHTO T 112.

B. Fabrication

General Provisions 101 through 150.

C. Acceptance

Ensure the mix design has been reviewed and approved by the Department prior to beginning production.

1. Rutting Susceptibility Testing
   a. Fabricate three beams or six cylindrical specimens from each asphalt mix for the test using GDT 115.
   b. Design mixtures which meet the following criteria for rutting where tested using GDT 115:
   c. Mix Design Level A – 0.3 in (7 mm) maximum
      - Mix Design Level B – 0.25 in (6 mm) maximum
      - Mix Design Level C & D – 0.2 in. (5 mm) maximum

      Mixtures designed prior to July 1, 2001 which do not exceed 0.2 in (5 mm) rutting when tested at 120 ºF (49 ºC) using GDT 115 may be acceptable.

   Tests will not be required for mixtures designed exclusively for trench widening nor for the 4.75 mm mix, nor for open-graded surface mixtures.
2. Fatigue Testing

   The Department may perform the test according to AASHTO TP 8-94 or other Department approved procedure.

D. Materials Warranty

General Provisions 101 through 150.

828.2.01 Open-Graded Surface Mixture

A. Requirements

1. Use the information in the following table for job mix formulas and design limits:

Mixture Control Tolerance	Asphaltic Concrete	9.5 mm OGFC	12.5 mm OGFC	12.5 mm PEM
	Grading Requirements	**Percent Passing**		
±0.0	3/4 in (19 mm) sieve		100	100
±6.1	1/2 in (12.5 mm) sieve	100*	85-100	80-100
±5.6	3/8 in (9.5 mm) sieve	85-100	55-75	35-60
±5.7	No. 4 (4.75 mm) sieve	20-40	15-25	10-25
±4.6	No. 8 (2.36 mm) sieve	5-10	5-10	5-10
±2.0	No. 200 (75 µm) sieve	2-4	2-4	1-4
	Design Requirements			
±0.4	Range for % AC	6.0-7.25	5.75-7.25	5.5-7.0
	Class of stone (Section 800)	“A” only	“A” only	“A” only
	Coating retention (GDT-56)	95	95	95
	Drain-down, AASHTO T 305 (%)	<0.3	<0.3	<0.3

* Mixture control tolerance not applicable to this sieve for this mix.

2. Use only PG 76-22 (specified in Section 820) in the 12.5 mm OGFC and 12.5 mm PEM mixtures.
3. Use a stabilizing fiber, which meets the requirements of Section 819 in 12.5 mm OGFC and 12.5 mm PEM mixtures. The dosage rate will be as recommended by the Engineer and shall be sufficient to prevent excessive drain-down.

B. Fabrication

General Provisions 101 through 150.

C. Acceptance

General Provisions 101 through 150.

D. Materials Warranty

General Provisions 101 through 150.

828.2.02 Stone Matrix Asphalt Mixtures

A. Requirements

Use the information in the following table for the job mix formula and design limits.

Mixture Control Tolerance	Asphaltic Concrete	9.5 mm SMA	12.5 mm SMA	19 mm SMA
	Grading Requirements	**Percent Passing**		
±0.0	1- in (25 mm) sieve			100
±7.0	3/4 in (19 mm) sieve		100*	90-100
±6.1	1/2 in (12.5 mm) sieve	100*	85-100	44-70
±5.6	3/8 in (9.5 mm) sieve	70-100	50-75	25-60
±5.7	No. 4 (4.75 mm) sieve	28-50	20-28	20-28
±4.6	No. 8 (2.36) mm sieve	15-30	16-24	15-22
±3.8	No. 50 (300 μm) sieve	10-17	10-20	10-20
±2.0	No. 200 (75 μm) sieve	8-13	8-12	8-12
	Design Requirements			
±0.4	Range for % AC	6.0-7.5	5.8-7.5	5.5-7.5
	Design optimum air voids (%)	3.5 ±0.5	3.5 ±0.5	3.5 ±0.5
	% aggregate voids filled with AC (VFA)	70-90	70-90	70-90
	Tensile splitting ratio after freeze-thaw cycle GDT-66	80%	80%	80%
	Drain-down AASHTO T 305 (%)	<0.3	<0.3	<0.3

* Mixture control tolerance not applicable to this sieve for this mix.

1. Compact SMA mixtures at 50 gyrations with the Superpave Gyratory compactor or 50 blows with the Marshall compactor.
2. A Tensile splitting ratio of no less than 70% may be acceptable so long as all individual test values exceed 100 psi (690 kPa).
3. Stone Matrix Asphalt mixtures shall contain asphalt cement, mineral filler, and fiber stabilizing additives which meet the following requirements:
   a. Use asphalt cement that meets requirements of PG 76-22 of Section 820.
   b. Use mineral filler that meets requirements of Section 883 and has been approved by the Engineer. Local sand shall not be used in lieu of mineral filler.
   c. Treat these mixes with a fiber-stabilizing additive, which meets the requirements of Section 819. The dosage rate will be as recommended by the Engineer and shall be sufficient to prevent excessive drain-down.

B. Fabrication

General Provisions 101 through 150.

C. Acceptance

See Subsection 828.2.C.

D. Materials Warranty

General Provisions 101 through 150.

828.2.03 Superpave Asphaltic Concrete Mixtures

A. Requirements

Use the information in the following table for job mix formula and design limits:

Mixture Control Tolerance	Asphaltic Concrete	9.5 mm Superpave Level A	9.5 mm Superpave Level B,C,D	12.5 mm Superpave	19 mm Superpave	25 mm Superpave
	Grading Requirements	**Percent Passing**				
	1-1/2 in (37.5 mm) sieve					100
± 8.0	1- in (25.0 mm) sieve				100*	90-100
±8.0	3/4 in (19.0 mm) sieve			100*	90-100	55-89
±6.0**	1/2 in (12.5 mm) sieve	100*	100*	90-100	60-89	50-70
±5.6	3/8 in (9.5 mm) sieve	90-100	90-100	70-85	55-75	
±5.6	No. 4 (4.75 mm) sieve	65-85	55-75			
±4.6	No. 8 (2.36 mm) sieve	53-58	42-47	34-39	29-34	25-30
±2.0	No. 200 (75 µm) sieve	4.0-7.0	4.0-7.0	3.5-7.0	3.5-6.0	3.0-6.0

* Mixture control tolerance not applicable to this sieve for this mix.

**Mixture control tolerance shall be ± 8.0% for this sieve for 19 mm Superpave.

Superpave mixtures shall also meet the following requirements:

1. The Mixture Control Tolerance for asphalt cement shall be ± 0.4%.
2. Volumetric Criteria

Design Parameter	Design Criteria
a. Percent of Maximum Specific Gravity (%G_{mm}) at the design number of gyrations, (N_d) (See Note 1)	96%
b. % G_{mm} at the initial number of gyrations, (N_i)	Level A <91.5% Level B <90.5% Level C & D <89%
c. Percent voids in mineral aggregate (VMA) at N_d	See Table 828.2.03.A.3
d. Percent voids filled with asphalt (VFA) at N_d	See Table 828.2.03.A.4
e. Fines to effective asphalt binder ratio (F/P_{be})	
1) Asphaltic concrete 9.5 mm Superpave (Level A)	0.6-1.2
2) All Superpave mixtures excluded in Item 1	0.8-1.6
f. Tensile strength (GDT 66)	
1) Ratio (See Note 2)	80% min.
2) Stress	60 psi (414 kPa) min.
g. Retention of Coating (GDT 56)	95% min.

Note 1: Maximum specific gravity (G_{mm}) determined in accordance with AASHTO T 209.

Note 2: A tensile splitting ratio of no less than 70% may be acceptable so long as all individual test values exceed 100 psi (690 kPa).

3. VMA Criteria

Nominal Maximum Sieve Size	Minimum % VMA*
1 in (25 mm)	12
3/4 in (19 mm)	13
1/2 in (12.5 mm)	14
3/8 in (9.5)	15

* VMA is to be determined based on effective specific gravity of the aggregate (G_{se}).

4. VFA Criteria

MIX DESIGN LEVEL	RANGE % VFA	
	Minimum	Maximum
A	67	80
B	65	78
C	65	76
D	65	75

5. Superpave Gyratory Compaction Criteria

MIX DESIGN LEVEL	NUMBER OF GYRATIONS	
	N_i	N_d
A	6	50
B	7	75
C	8	100
D	9	125

Use mix Design Level A for all Superpave mixes used as shoulder surface mixture, trench widening, temporary detour, or sub-base mixture under Portland cement concrete pavement unless specified otherwise in the plans.

B. Fabrication

General Provisions 101 through 150.

C. Acceptance

See Subsection 828.2.C.

D. Materials Warranty

General Provisions 101 through 150.

828.2.04 Fine Graded Mixtures

A. Requirements

Use the following table for the job mix formula and design limits:

ASPHALTIC CONCRETE - 4.75 mm Mix		
MIXTURE CONTROL	GRADING	

TOLERANCE	REQUIREMENTS	% Passing
±0.0	1/2 in (12.5 mm) sieve	100*
±5.6	3/8 in (9.5 mm) sieve	90-100
±5.7	No. 4 (4.75 mm) sieve	75-95
±4.6	No. 8 (2.36 mm) sieve	60-65
±3.8	No. 50 (300 µm) sieve	20-50
±2.0	No. 200 (75 µm) sieve	4-12
	DESIGN REQUIREMENTS	
±0.4	Range for % AC	6.00-7.50
	Design optimum air voids (%)	4-7
	% Aggregate voids filled with AC	50-80
	Tensile splitting ratio after freeze-thaw cycle (GDT 66)	80% minimum

* Mixture control tolerance not applicable to this sieve for this mix.

Design this mixture at Superpave Mix Design Level A.

B. Fabrication

General Provisions 101 through 150.

C. Acceptance

General Provisions 101 through 150.

D. Materials Warranty

General Provisions 101 through 150.

Georgia Department of Transportation
Office of Materials and Testing

Standard Operating Procedure (SOP) 2
Control of Superpave Bituminous Mixture Designs

I. General

Monitoring the quality of Bituminous Mixtures used on Georgia Department of Transportation work is a responsibility of the Bituminous Construction Branch of the Office of Materials and Testing. This branch is under the direction of the State Bituminous Construction Engineer. The Bituminous Construction Branch comprises the Asphalt Design Unit, the Bituminous Control Unit, and the Bituminous Technical Services Unit.

The Asphalt Design Unit performs, verifies, and recommends approval of designs for Superpave mixtures, Open-Graded Friction Course (OGFC), Porous European Mix (PEM) mixtures, Stone Matrix Asphalt (SMA), slurry seals, sand-bituminous bases, micro-surfacing, and other asphalt mixtures as assigned.

The Asphalt Design Engineer oversees design activities statewide, including designs and verifications performed by the Office of Materials and Testing and Branch Laboratories. The Asphalt Design Engineer reviews and recommends approval of designs made in commercial laboratories which have been certified in accordance with SOP 36. Designs submitted by certified laboratories shall be prepared, verified and approved in accordance with this Standard Operating Procedure. The Asphalt Design Engineer forwards acceptable designs to the State Bituminous Construction Engineer with recommendation for approval or approval for provisional use, as appropriate. Once approved, a design shall be published and transmitted to the certified laboratory which performed the design. Designs found to be incorrect or deficient shall be referred back to the designer within two weeks of receipt . Designers may resubmit their designs for approval when appropriate changes or corrections have been made. The State Bituminous Construction Engineer may make field adjustments of the Job Mix formula and may require field verification of mix designs, as discussed below.

II. Approval Process

A. Governing Documents

Commercial laboratories wishing to perform mix designs for use in GDOT projects shall comply with SOP 36, *Certification of Laboratory and Personnel for the Design of Asphaltic Concrete Mixtures*.

All mix designs shall meet current contract specifications and shall be prepared in accordance with applicable standard methods, described below. Mix designs from commercial laboratories shall be approved only for work covered under state funded contracts, and designs for mix types and levels not specified for state work are not eligible for approval.

Aggregates used in Asphaltic Concrete mixes must meet the requirements of Sections 800 and 802 of the Specifications. Asphalt Cement used in the mixture shall meet the requirements of Section 820 for Superpave Asphalt Binder. All designs for publication must meet the requirements of Section 828, “Hot Mix Asphaltic Concrete Mixtures”. All ingredients of asphalt mixtures shall be from sources approved by the Department. Approved aggregate sources, except proprietary RAP stockpiles and sand pits, are listed in Qualified Products Lists 1 and 2. Other approved sources are listed in their respective Qualified Products Lists.

Mix designs must be submitted using the GDOT approved mix design software. Completed design studies shall be submitted to the Asphalt Design Engineer by letter request, including the technician's certification required under SOP 36. The letter request should also identify any entity, other than the firm which produced the design, which is authorized to use it. Other required information is as follows:

1. Types and sources of aggregate ingredients
2. Asphalt binder grade and source
3. Gyratory compaction sheets
4. Results of ignition calibration tests, including worksheet and print-out
5. Test results required for the Superpave mix design study
6. RAP stockpile number, if RAP is included
7. Results of permeability test plus sample, as required

Test results for the mix design study shall be entered into the GDOT Mix Design Software and submitted as an Asphaltic Concrete Mix Design Report. Mix designs shall be approved which are correct and complete and which conform to the design criteria set forth in Section 828 of the Specifications.

Approved asphalt mix designs shall be identified by a mix identification number which will identify the designer, aggregate sources, mix type, and design level.

B. Verification of Designs

Mix designs shall be verified by the Office of Materials and Testing at a minimum frequency of ten percent of the designs submitted by each certified laboratory, or at the discretion of the State Bituminous Construction Engineer. These verifications shall be performed by a GDOT laboratory designated by the Asphalt Design Engineer. A verification will consist of replicating all or part of the design test procedures, as the Asphalt Design Engineer may require. Samples shall be tested at the asphalt and air void contents required for certain design tests or at optimum asphalt content, as appropriate. Sufficient quantities of stockpile samples shall be retained for at least two weeks after submittal of a design, or until approval of design is granted, whichever comes first. Results of the verification must match the design results within the tolerances below. In addition, when design volumetrics are verified by gyrating a full set of new samples, the resulting VMA and VFA must also fall within the tolerances specified in Section 828.

Test	Verification Tolerance
G_{mb} - AASHTO T-166	±0.03
G_{se} - AASHTO T-209 and T-308	±0.03
% VTM - AASHTO T-312	4% ± 1.0%
% G_{mm} @ N_{ini} - AASHTO T-312	± 1.0%
% G_{mm} @ N_{des} - AASHTO T-312	± 1.0%
VMA - AASHTO R 35	- 0.5% to +0.8%
VFA - AASHTO R 35	± 5%
Dust/AC Ratio - AASHTO T-312	± 0.2
Gradation:	
Upper Control Sieve - % Passing	± 3.5 %
No. 8 (2.36 mm) Sieve – % Passing	± 2.5 %
No 200 (75 μm) Sieve –% Passing	± 1.6 %
LWT - GDT-115	±2.0 mm, but not to exceed design limit
Retained Tensile Strength - GDT- 66	(average of three) ± 10 % must also meet design minima for strength and % retained
Calibration Factor for ignition tests	± 0.12 %

Where G_{mb} is the bulk specific gravity of the mix, G_{se} is the effective specific gravity of the aggregate, and N_{ini} and N_{des} are the numbers of initial gyrations and design gyrations, respectively. VTM and VMA are the percent air voids and percent voids in the mineral aggregate, respectively, and VFA is percent voids filled with asphalt. LWT refers to the loaded wheel test result using the Asphalt Pavement Analyzer (APA).

In applying the tolerances above for percent of G_{mm} at N_{ini} and percent of G_{mm} at N_{des} , the G_{mm} shall be re-calculated using the G_{se} determined in the verification.

If the verification result does not match the design values within the above tolerances, an investigation shall be initiated by the State Bituminous Construction Engineer. The investigation may include a review of design procedures and equipment calibrations as well as the results of a field verification. If the cause for the discrepancy cannot be resolved, approval of the design may be withdrawn.

C. Field Verification

All mix designs shall be subject to one or more field verifications during production at the discretion of the State Bituminous Construction Engineer. Verification shall consist of replicating certain mix design tests on samples of the mixture delivered to a state project, normally when the design is first used and subsequently in some cases, at the discretion of the State Bituminous Construction Engineer. Field verification tests shall normally include AASHTO T-209, AASHTO T-166, and AASHTO T-312 to verify design volumetrics and may include , GDT 115, GDT-66, and other tests as the State Bituminous Construction Engineer may require. A field verification shall be acceptable when results fall within the tolerances in the table below. Designs which fail field verification shall be invalid unless an approved revision is made to correct the deficiency, or unless it is shown that the production sample was deficient and that the deficiency has been corrected.

Test	Field Verification Tolerance
G_{mb} - AASHTO T-166	± 0.03
G_{se} - AASHTO T-209 (and GDT-125)	±0.03
GDT-66	not to exceed specified design limits
Design Volumetrics - AASHTO R 35:	
VMA	not to exceed specified design limits
VTM (air voids) @ optimum AC	not to exceed specified design limits

D. Continuity and Cancellation of Mix Designs

An approved and field verified mix design may be used from project to project as long as the design meets current specifications, provided that satisfactory performance of the mixture is obtained, that the properties of the mixture remain consistent with the design values, and that no significant change occurs in the properties or approval status of the ingredients. The State Bituminous Construction Engineer may withdraw approval of a mix design on the basis of unsatisfactory or erratic test results, poor performance of the mixture in place, or evidence that the properties of the mixture differ substantially from the properties predicted in the design. In the case of RAP mixtures, approval will be withdrawn if the RAP stockpile is depleted or if the average gradation of the RAP, based on five random samples, varies to the extent that the combined gradation of the design is altered by more than one-half the mixture control tolerance.

E. Ownership, Use, and Disclosure of Mix Designs

Mix designs shall be made available only to the designer and to users authorized by the designer. Mix designs are considered to be proprietary information. They are not subject to public disclosure under the Georgia Open Records Act by virtue of O.C.G.A. 50-18-72(b)(1), which protects the confidentiality of trade secrets obtained from a business entity that are confidential and required to be submitted to a government agency.

III. Design Process

The object of an Asphaltic Concrete Design is to produce a combination of the proposed ingredients that will perform satisfactorily throughout the design life of the pavement. Such a mixture must contain sufficient asphalt cement to provide a thick film and limited air voids so the mix can resist stripping and weathering due to intrusion of water and air. The mix must also be stable enough to resist permanent deformation, flushing, excessive densification, and loss of friction properties. The volumetric design process is complicated by the facts that asphalt is thermoplastic and that specific elevated temperatures must be maintained in the design work. Superpave Mixtures are to be designed in accordance with AASHTO R 35 except as altered by Georgia Department of Transportation's specifications including but not limited to SOPs, GDTs and GSPs. ,Many design details are difficult to remember; therefore a ready reference entitled "Asphalt Hot Mix Design Reference Guide" can be found in Appendix A.

A. Sampling and Grading

Sampling of aggregates proposed for use in bituminous mix designs may be initiated by the Contractor, commercial laboratory, or materials supplier. The requesting party should submit the samples to the design laboratory. Materials sampled for design work must be representative of quarry production intended for use on the project. The average ingredient characteristics should be represented in the design. The designer shall resolve any discrepancies in the ingredient properties before beginning any design work.

Each aggregate sample submitted for design is initially dried, and sieve analysis is performed to determine its gradation. Grading of coarse aggregate samples is done using the appropriate sieves for the specific mix type involved. These sieve sizes can be found in Section 828 of the Specifications. In addition, appropriate "breaker" sieves must be used to prevent overloading the sieves. Each ingredient shall be batched individually. Bulk batching of aggregates is prohibited.

Aggregate used for batching Superpave specimens is not separated below the No. 8 (2.36 mm) sieve, with the exception that a washed gradation is performed on minus 2.36 mm portion by washing over the No 200 (75 μm) sieve.

If the coarse or fine aggregate is excessively dusty, soft, easily broken, or shows other signs of potential problems, the Asphalt Design Engineer should be consulted for investigation of the source, stockpiles, and operations. The Revised decision in such matters will rest with the State Materials and Research Engineer.

Once the appropriate blend, meeting requirements established in Section 828 and Appendix B, has been established, batches of Superpave design specimens to determine optimum asphalt content shall be prepared to produce a compacted Superpave specimen 115.0 ± 5.0 mm high and 150 mm in diameter for density testing. The height of test samples should be 95.0 ± 5.0 mm for tensile splitting specimens and 75.0 ± 1.0 mm for loaded wheel test specimens. Designers should ensure that all samples, including those for gradation and specific gravities, will meet the minimum sample size requirements for their respective tests.

B. Preparing Superpave Specimens

1. Asphalt Cement

   Samples shall be heated to the appropriate temperature for the asphalt binder being used. Temperatures for preparing Superpave specimens are based on the viscosity of the asphalt cement involved. These values are very important; they can be found in the Asphalt Mixture Control Temperature Chart which is available from the Asphalt Design Engineer.

2. Short term Aging

   The short term aging procedure applies to laboratory-prepared loose mix only. The laboratory aging process is necessary to simulate mixture aging during typical plant production and placement. All samples for testing shall be aged by placing the mixture in a pan and spreading it to an even thickness of approximately 55 ± 5 lbs/yd^2 (30 ± 2 kg/m^2) immediately after sample mixing. Place the mixture and pan in a forced draft oven for 2 hours at compaction temperature.

C. Superpave Gyratory Compactor

A gyratory compactor meeting the requirements of AASHTO T-312 shall be used to compact density specimens for testing. The gyratory compactor may also be used for preparing samples for performance testing as detailed in Section 828. The gyratory compactor shall be calibrated and the operation of the data acquisition device shall be checked based on the interval established in AASHTO R18. The compaction pressure should be checked and set to the proper value; 600 ± 18 kPa, and the rate of revolution should be set at 30 gyrations per minute. The internal angle is to be set at 1.16 ±

0.02 degrees. It is recommended that the calibration be done for the internal angle using the Dynamic Angle Validator (DAV) if different brands or models of the gyratory compactor are being used.

Samples shall be gyrated to the number specified for the N_{des} level required in Section 828.

D. Testing Superpave Specimens

All testing shall be in accordance with the appropriate AASHTO or GDT procedure, as follows:

Test	Test Method
Volumetric Properties	**AASHTO T-312,** “Determining the Density of Hot Mix Asphalt (HMA) Specimens by Means of Superpave Gyratory Compactor” AASHTO R 35, “Superpave Volumetric Design for Hot Mix Asphalt (HMA)
Bulk Density	**AASHTO T-166,** “Bulk Specific Gravity of Compacted Bituminous Mixtures Using Saturated Surface-Dry Specimens”
Short Term Aging	**AASHTO R-30,** “Mixture Conditioning of Hot Mix Asphalt (HMA)” Note: The procedure is modified for GDOT mix designs to require only two hours aging.
Maximum Density and Effective gravity	**AASHTO T-209** “Maximum Specific Gravity of Bituminous Paving Mixtures”
Aggregate Gravities	**AASHTO T-84** “Specific Gravity and Absorption of Fine Aggregate” and **AASHTO T-85,** “Specific Gravity and absorption of Coarse Aggregate” (The designer may obtain coarse aggregate gravities from GDOT or perform this test.)
Moisture Susceptibility	**GDT-66** “Method of Test for Evaluating the Moisture Susceptibility of Bituminous Mixtures by Diametral Tensile Splitting”
Rutting Susceptibility	**GDT-115** “Determining Rutting Susceptibility of Asphalt Paving Mixtures Using the Asphalt Pavement Analyzer (APA)”
Permeability	**GDT-1** Measurement of Water Permeability of Compacted Paving Mixtures

Use the design calculations as outlined in AASHTO R 35 and T-312. However, replace G_{sb} with G_{se} when calculating VMA. When designing a Superpave mix containing RAP materials, the effective specific gravity (G_{se}) of the RAP shall be used in place of the bulk specific gravity (G_{sb}) in determining the combined aggregate bulk specific gravity for the blend. A method of calculating batch weights for RAP mixes is presented in Appendix C. Additionally, when designing Superpave mixtures containing RAP and/or RAS; a Corrected Optimum AC Content (COAC) is to be calculated and used as detailed in Appendix D.

E. Moisture Susceptibility

Moisture susceptibility will be determined by the tensile splitting method according to GDT 66. For these tests, the specimens will be fabricated at optimum asphalt cement content. All mixtures containing RAP and/or RAS shall be fabricated at the corrected optimum asphalt cement content **(COAC)**. The compactive effort for the specimens is to be reduced such that the air voids fall in a range required in Section 828. Specimens prepared for this test will include hydrated lime, or anti-stripping additive, or both, as specified for the ingredients proposed. For gyratory specimens that fail moisture susceptibility, Marshall specimens (4 inch) may be substituted.

F. Rutting Susceptibility Testing

Results of tests with the Asphalt Pavement Analyzer shall be provided for all Superpave mixtures. The rutting susceptibility test will be conducted according to GDT-115. For these tests, the specimens will be fabricated at optimum asphalt cement content. All mixtures containing RAP and/or RAS shall be fabricated at the corrected optimum asphalt cement content **(COAC)**. Three beam specimens or six gyratory specimens should be tested for each mix design. If the average rut depth for the three specimens exceeds specified limits, the asphaltic concrete mixture shall not be used in the work. The compactive effort for the specimens is reduced such that the air voids fall in a range required in Section 828. Test temperature for this test shall be 149 ºF (64 ºC), except for 19 mm and 25 mm Superpave mixes, for which it shall be 120 ºF (49 ºC).

G. Fatigue Testing

The Office of Materials and Testing may conduct a fatigue test on any Superpave asphalt mixture design or Superpave asphalt mixture used in construction to determine acceptability of the materials. The test shall be performed according to test procedure AASHTO T 321, or other procedure approved by the Office of Materials and Testing. All mixtures containing RAP and/or RAS; shall be fabricated at the corrected optimum asphalt cement content **(COAC)**.

H. Calibration Factor for Ignition Test

The designer shall, as part of the design process, perform calibration tests for use when testing the mixture in the ignition furnace, according to GDT 125. All results, including the worksheet and the print-out from the ignition furnace, shall be submitted with the design study and request for approval. All mixtures containing RAP and/or RAS shall be fabricated at the corrected optimum asphalt cement content (COAC).

Verification. The approved calibration factor shall remain in use unless, in the judgment of the State Bituminous Construction Engineer, the accuracy of the testing technique, calibration, or apparatus is found to be invalid or unreliable.

The contractor shall provide samples of the mix ingredients to the Department for verification of the CF on request. On receiving evidence that invalid or unreliable test results have been obtained, the State Bituminous Construction Engineer may suspend use of the ignition test on the mixture being produced until a correct calibration is obtained and until all other discrepancies involving calibration, apparatus and technique have been resolved. Where an incorrect CF has been applied in acceptance testing, results shall be corrected by applying a valid CF.

When a Job Mix Formula is submitted for approval prior to beginning production, the calibration factor of the mixture shall be included in the submittal. (This shall apply in all cases, regardless of the test method to be used for quality control testing.)

IV. Changes in Established Design Procedures, Criteria, or Mix Requirements

Changes in established procedures, criteria, and mix requirements are the prerogative of the State Materials and Research Engineer. Specifications, procedures, and other changes may apply to all bituminous mixtures, or only to a particular mixture. Any certified laboratory designing mixes for use in GDOT work will be placed on a list to receive information on revisions pertaining to bituminous mix design specifications and procedures.

V. Revisions of Approved Designs

Generally, when a particular ingredient of a mix design becomes unavailable, the contractor must provide a different design in order to continue work on a project. While the contractor is always responsible for the supply of materials, it is recognized that certain aggregate sizes may become unavailable due to unforeseeable causes. Often this interrupts paving work in progress, causing inconvenience to the public. In some instances, it may be possible to substitute one coarse aggregate ingredient for a similar material from a different source without affecting the quality of the mixture. In these cases only, the laboratory which designed the mix may submit a design revision for consideration. Design revisions will be subject to the following conditions:

A. Actual Shortage Required

The revision must be necessitated by an actual shortage, sufficient to delay work in progress, of a coarse aggregate ingredient of an approved design.

B. Similar Substitute Ingredient

The substitute ingredient must be similar to the replaced ingredient in mineralogy, particle size and shape, specific gravity, and abrasion resistance.

C. Revised Design Support Requirements

The proposed revised design shall be supported by volumetric tests on a minimum of two pairs of specimens, at asphalt content checkpoints above and below the optimum asphalt content of the original design. The State Bituminous Construction Engineer may require verification of previous tests for susceptibility to rutting, fatigue, and moisture when these properties of the design are marginal.

____
State Materials Engineer

____
Director of Construction

Appendix A

Hot Mix Asphalt Design Reference Guide

(Note: Preparation and Testing requires the use of metric units only)

Sequence No.	Description
1	Dry incoming aggregate as described in AASHTO: T 27-93.
2	Grade aggregates as described in AASHTO: T 27-93. Use Gilson shaker and shake at least 10 minutes.
3	Calculate gradation of each aggregate type. Carry calculations to the nearest 0.1%. Compare to source average values and consider plant breakdown.
4	Calculate blend, keeping within control limits. Use AASHTO R 35 as a reference.
5	Batch aggregates as described in AASHTO 312 and AASHTO R 35. The design specimens must be 115 ± 5 mm high (95 mm for moisture susceptibility and 75 mm for LWT). Thoroughly mix the minus 2.36 mm aggregate during batching. Sample weights for AASHTO T-209 (maximum theoretical specific gravity) and gradation must be 2000 grams, except samples for 25 mm mixtures, which shall weigh at least 2500 g.
6	Heat the pans of aggregate to temperature specified on Mixing and Compaction Temperature Control Chart for the source of asphalt cement being used.
7	Heat the asphalt cement to temperature specified on Mixing and Compaction Temperature Control Chart for the source of asphalt cement being used. Heat only a half day's run. Never overheat or reheat AC.
8	Add and mix RAP material, if required, with the hot aggregates. Mix only until the RAP material is blended with the aggregate.
9	Add and mix hydrated lime. Add 1.0% by weight of the aggregate for virgin mixes or as calculated in Appendix C for RAP mixes. Add hydrated lime to the heated aggregate and mix until the aggregate is coated with lime.
10	Mix the heated AC and aggregate in a preheated bowl. The temperature at the time of mixing is very important. Care should be exercised to thoroughly coat the aggregate with AC.
11	When sample has been thoroughly mixed, place the mixture in a pan and spread it uniformly to approximately55 ± 5 lbs/yd^2 (30 ± 2 kg/m^2). Place the mixture and pan in a forced draft oven for 2 hours at the upper limit of the compaction temperature range. All samples for testing (with the exception of moisture susceptibility samples) shall be aged.
12	At least 30 minutes before compaction of the first specimen, place the compaction molds and base plates in an oven at compaction temperature.
13	At the end of the aging process, remove a mold and base plate from the oven. Assemble base plate and mold. Place a paper disk on top of the base plate. Place the aged mixture in the mold (do not spade). Be extremely careful to keep segregation to a minimum when transferring the sample to the heated mold. Place a paper disk on top of the sample.
14	Compact specimen using the Superpave Gyratory Compactor in accordance with AASHTO 312.
15	Remove the mold containing the compacted specimen from the compactor and extrude the specimen from the mold. A short cooling period is allowable to facilitate specimen removal to minimize sample damage. Remove the paper disks from the top and bottom of the specimen. Place the specimen on a flat, well supported surface where it will not be disturbed during cooling. A fan can be used to accelerate cooling, if necessary. Repeat this procedure for each specimen.
16	Determine G_{mb} in accordance with AASHTO T-166. Use balance accurate to 1.0 g. Be sure the water is clean and at correct temperature. Beware of specimens that release excessive bubbles when submerged. Such samples may prove misleading density values. Be sure the basket and suspension wire do not contact anything.

Appendix B

Ensure that Superpave Asphalt Concrete Mixtures Designs meet the following mix design limits:

Sieve Size	Design Gradation Limits, Percent Passing				
	9.5 mm Superpave Type I	9.5 mm Superpave Type II	12.5 mm Superpave	19 mm Superpave	25 mm Superpave
1½ in (37.5 mm)					100*
1- in (25.0 mm)			100*	100*	90-100
3/4 in (19.0 mm)	100*	100*	98-100****	90-100	55-89** (85 – 89)$_1$
1/2 in (12.5 mm)	98-100****	98-100****	90-100	60-89*** (85 – 89)$_1$	50-70
3/8 in (9.5 mm)	90-100	90-100	70-89 (85 – 89)$_1$	55-75	
No. 4 (4.75 mm) s	65-85	55-75			
No. 8 (2.36 mm)	48-55	42-47	38-46 (42 – 45)$_1$	32-36 (33 – 35)$_1$	30-36 (33 – 35)$_1$
No. 200 (75 µm)	5.0-7.0 (5.5 – 6.5)$_1$	5.0-7.0 (5.5 – 6.5)$_1$	4.5-7.0 (5.0 – 6.0)$_1$	4.0-6.0 (4.5 – 5.2)$_1$	3.5-6.0 (4.5 – 5.2)$_1$
Range for % AC (Note 4)	5.4-7.25	5.25-7.00	5.00-6.25	4.25-5.50	4.00-5.25

Note 1 details the desired Mix Design combined gradation for each referenced sieve

Appendix C

Method of Calculating Batch Weights for Mix Designs With Recycled Asphalt

PURPOSE: To calculate the weights of reclaimed asphalt pavement (RAP), virgin aggregate, and liquid asphalt cement (AC) for preparing volumetric samples of asphalt mixtures.

Example calculations are for an aggregate batch weight of 4800 g. Assume mix will contain 30% RAP and RAP contains 6.3% AC by extraction. For this example, assume one point of the design will use 5.5% total AC.

1. Total weight of mix = $\dfrac{\text{Agg. Wt.}}{100 - \%\text{ AC}}$

   Example: $\dfrac{4800\text{g}}{100\% - 5.5\%} = 5079\text{g}$

2. Grams of RAP to batch = (Total Wt of mix)(% RAP)

   Example: (5079)(30%) = 1524 grams RAP

3. (2)(% AC in RAP) = Grams of old AC from RAP

   Example: (1524 grams)(6.4%) = 97.5 grams old AC

4. (1) – Agg. Wt. – (3) = Grams of new AC to add

   Example: 5079 – 4800 –97.5 = 181.5 grams of new AC to add

5. (2) – (3) = Grams of aggregate in RAP

   Example: 1524 – 97.5 = 1426.5 grams

6. % Aggregate contributed by RAP = $\dfrac{(5)}{\text{Agg. Batch}}$

   Example: $\dfrac{1426.5}{4800} = 29.7\%$ total aggregate from RAP

7. % lime in mix = [100% - (6)][1.0 %] + [(6)][0.5%]

   Example: (1.0%)(100% – 29.7%) + (0.5%)(29.7%) = 0.9 % Lime

   NOTE: This step assumes 50% of RAP will have fractured faces which need to be treated with hydrated lime.

8. % Aggregate available for other sizes = 100 – (6) – (7)

   Example: 100 – 29.7 – 0.9 = 69.4% available for virgin aggregate

9. Calculate Blend

   Example: For this example, assume the following blend will be used:

   29.7% - RAP aggregate

   20.0% - 89 stone

   25.0% - 810 screenings

   24.4% - 777 (manufactured sand)

   0.9% - hydrated lime

   100% - Total aggregate

10. Calculate Batch Weights

    Batch wt. of virgin agg. = agg. batch wt. times % of blend

RAP	= (2)	= 1524 grams
#89	= 4800 X 20%	= 960
#810	= 4800 X 25%	= 1200
#777	= 4800 X 24.4%	= 1171
Lime	= 4800 X 0.9%	= 43
New AC (for 5.5%)	= (4)	= 181.5
Total Wt.		= 5,079.5 grams (Differs from (1) above due to round-off error.)

NOTE: As the total weight for each point of the design changes (Step 1), the grams of RAP to batch up in Step 2 will also change slightly, as will the available aggregate in Step 8. Therefore, use the AC content nearest the anticipated optimum (usually the third point of the design) as the value to use in Step 1 and on which the blend percentages and batch weights are to be calculated.

Steps 1 through 4 should be repeated for each point in the design to determine the amount of new AC.

NOTE: Use the extracted gradation (or gradation after burning in the ignition oven) of the RAP to calculate the mix blends; use the gradation of the RAP "as is" (from the Gilson shaker) to determine individual sizes for the batch weight. (See pages 1 and 3 of the design software.)

Appendix D

Method of Calculating Credited Asphalt Cement Content for Corrected Optimum AC Content for Asphaltic Concrete Mixtures Incorporating Reclaimed Asphalt Pavement (RAP) or Post-Consumer Recycled Asphalt Shingles (RAS)

Purpose: To calculate the Credited AC Content (**CAC**) and Not Credited AC Content (**NCAC**) to be used to determine the Corrected Optimum AC Content **COAC** of Asphaltic Concrete Mixtures incorporating RAP and/or Recycled Asphalt Shingles (RAS) for all mixtures. The **CAC** and **NCAC** shall be used to determine the amount of additional new AC required to be added to an Asphaltic Concrete Mix Design's Original Optimum AC Content (**OOAC** as determined in AASHTO R 35-09 Section 10.5 at VTM = 4.0% air voids. **OOAC must meet the requirements of Section 828.2.03.A.** The **CAC** and **NCAC** shall be calculated using an applied factor as follows: **CAC** shall be calculated using a factor of 0.75 while the **NCAC** is equivalent to 0.25 where 1.0 – 0.75 equals 0.25.

The **COAC,** as determined using this procedure, shall be used in fabricating samples for all performance tests established in Section 828.2.B.2. Additionally, the **COAC** is to be listed on the Mix Design Summary Sheet **(as a note)** and used for JMF purposes.

Example calculations detailed are for a 12.5 mm Superpave Mix Type. Assume mix will contain 25% RAP and RAP contains 5.75% AC (RAP Stockpile Specific) determined using GDT-83 or GDT-125. For this example, assume the **OOAC**, as determined in AASHTO R 35-09 Section 10.5 is 5.10% total AC.

12.5 mm Superpave Mix with 5.10% **OOAC** (AASHTO R 35-09 Section 10.5 @ VTM 4% Air Voids). RAP = 25 % with 5.75% AC in RAP

1. Using Standard Mix Design Procedure RAP contributes 5.75 % x 0.25 = 1.44 % AC to the blended total AC of mix
2. Using factor to calculate **CAC** = 1.44 % x 0.75 = 1.08% AC
3. Using factor to calculate **NCAC** = 1.44 % - 1.08 % = 0.36 % AC
4. Add the 0.36 % **NCAC** to 5.10 % **OOAC =** 5.46 %
5. The **COAC** = 5.46 %
6. 5.46 % **COAC** shall be used for specimen fabrication for all performance test required in Section 828.2.B.2
7. **COAC** of 5.46 % will be listed as Corrected Optimum on Mix Design Summary Sheet as a note at the bottom.

Note: All Required Performance Test as specified in Section 828.2.B.2 shall be conducted at the Corrected Optimum AC Content (COAC). Mix Design Summary Sheet will list the COAC as the Corrected Optimum AC Content.