RESEARCH & DEVELOPMENT

Effect pf the Use of Higher Percentages of RAP in NCDOT Hot Mix Asphalt

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EFFECT OF THE USE OF HIGHER PERCENTAGES OF RAP

IN NCDOT HOT MIX ASPHALT

by

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16. Abstract

The recycling of asphalt pavements has become a very routine procedure throughout the country. Research has shown that the Recycled Asphalt Pavement (RAP) recovered from construction sites still contains usable materials, both in the recycled aggregates and recycled binder. However, since the RAP binder has been aged during its service life, the use of RAP in new pavement construction may cause the stiffness of the blended binder to increase. Due to this increased stiffness, it is sometimes necessary for a "grade shift" in the virgin binder in order to result in the specifications for the desired performance grade. As this complicates the procedure most contractors limit the use of RAP to 20% in order to avoid the need for a grade shift. This has resulted in large amounts of RAP going unused. The use of higher percentages of RAP in construction would provide initial cost savings. However, a life cycle cost analysis is needed in order to determine whether use of higher percentages of RAP provides an economical advantage for the life cycle, not just initially. In order to predict life cycle costs, the fatigue life and rut resistance of mixtures containing various amounts of RAP will be compared to a 100% virgin material mixture. The fatigue life and rut resistance of the mixtures were calculated from laboratory testing using Frequency Sweep Testing and Repeated Simple Shear Tests using Superpave Simple Shear Tester (SST). The SHRP A-003A surrogate models and the Asphalt Institute models were both used in order to predict pavement performance using the results from the Frequency Sweep Testing and the Repeated Simple Shear Test. Based on these results, the life cycle economic analysis were completed and the optimum percentages of RAP were determined for use in the construction of new pavements.

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EXECUTIVE SUMMARY

With the increasing cost of asphalt binder and the growing concern over the availability of quality aggregates, the use of higher percentages of reclaimed asphalt pavements (RAP) has become of much interest across the country. Research has shown that the RAP recovered from construction sites still contains usable materials, both in the recycled aggregates and recycled binder. However, since the RAP binder has been aged during its service life, the use of RAP in new pavement construction may cause the stiffness of the blended binder to increase. Due to this increased stiffness, it is sometimes necessary for a “grade shift” in the virgin binder in order to result in the specifications for the desired performance grade. The use of higher percentages of RAP in construction would provide initial cost savings. However, a life cycle cost analysis was needed in order to determine whether use of higher percentages of RAP provided an economical advantage for the life cycle, not just initially. In order to predict life cycle costs, the fatigue life and rut resistance of mixtures containing various amounts (15%, 30% and 40%) of RAP were compared to a 100% virgin material mixture. The fatigue life and rut resistance of the mixtures were calculated from laboratory testing using Frequency Sweep Testing using Superpave Simple Shear Tester (SST) and Repeated Simple Shear Tests using SST. The SHRP A-003A surrogate models and the Asphalt Institute models were both used in order to predict pavement performance using the results from the Frequency Sweep Testing and the Repeated Simple Shear Test. Based on these results, the life cycle economic analysis was completed and the optimum percentage of RAP was determined for use in the construction of new pavements for NCDOT.

The development of warm mix asphalt (WMA) technology over the recent past has sparked interest in many researchers, engineers and government officials. The use of WMA has incentives including fuel savings, lower emissions, longer hauling distances and longer construction seasons since the use of WMA allows for lower production temperatures. Research has also shown that the reduced production temperatures may reduce the amount of oxidative hardening which may help reduce thermal cracking and block cracking. Due to this benefit, along with the anticipated benefit of better compactability using WMA, it is thought that WMA can be used in mixes containing higher percentages of RAP since less oxidative hardening of the asphalt binder may occur for the already more stiff RAP binder. Mixtures with higher

percentages of RAP often have issues with thermal cracking and block cracking due to the stiff binder from the RAP blending with the virgin binder. In order to determine how the use of WMA additives effect the binder rheology of blends containing higher percentages of RAP, Dynamic Shear Rheometer (DSR) and Bending Beam Rheometer (BBR) testing were performed on virgin, blended and RAP binders containing Sasobit®, a WMA additive. These results can be used to determine the allowable amount of RAP that can be used with WMA.

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1.1 Introduction and Problem Statement

The recycling of asphalt pavements has become a very routine procedure throughout the country. Research has shown that the Recycled Asphalt Pavement (RAP) recovered from construction sites still contains usable materials, both in the recycled aggregates and recycled binder. The use of RAP in construction of new asphalt pavements has become more prevalent over the years. Specification limits make it cost prohibitive for Contractors to use higher RAP contents in their mixes. This practice has led to vast quantities of RAP going unused and stockpiled. The North Carolina Department of Transportation (NCDOT) has a long, successful history using RAP in Hot Mix Asphalt (HMA) that dates back to the 1970s. Therefore, its history is known when used in limited amounts. Research is needed to show that RAP materials can be used successfully in higher percentages.

With the road building boom of the early 2000s, stockpiles of RAP continue to grow. RAP is a product that the NCDOT initially owns, but then relinquishes to the Contractor once the pavement is removed from a project. However, due to requirements governing adjustments in PG Binder grade based on total percentage of RAP used in a mix, most Contractors choose to limit the RAP content of their mixes to 20%. Therefore, the NCDOT is unable to realize the full cost savings that such a recycled product should provide. Research is needed to determine if the NCDOT could specify higher percentages of RAP in mixes and show if that RAP would give the NCDOT an equal or improved life cycle return on its money.

When considering the use of RAP in Superpave mixtures, several design and performance uncertainties arise, some of which include:

- With respect to Superpave binder tests and specifications, how much stiffer is the binder in RAP compared to that of virgin asphalt binders?
- Does the addition of RAP affect the design of Superpave mixtures?
- Should the mixing and compaction procedure be modified to account for the addition of RAP?
- Is there a maximum threshold percentage on the addition of RAP to a mixture with respect to design and performance? How much is too much? How much is practical?
- Is the use of RAP in asphalt mixtures economically attractive?

1.2 Research Objectives and Tasks

The proposed research project will address the above questions. In order to investigate the effects of RAP in the design and performance of new asphalt concrete mixtures, the specific research objectives will be to:

- Evaluate the performance of mix designs using higher RAP percentages
- Determine which layers of the pavement structure could contain higher percentage of RAP without any significant reduction in performance life
- Perform a life cycle cost analysis showing the cost savings that could be realized if higher percentages of RAP are allowed.

1.3 Research Methodology and Tasks

1.3.1 Task 1 Sample Procurement and Characterization

This research task included the selection and procurement of two RAP samples. The properties of the two RAP samples were determined for use in the design of mixtures containing RAP.

1.3.1.1 Task 1.1 RAP Stockpile Site Selection and Sample Procurement

Two RAP stockpile sites were selected for use in this research subtask. The two sources of RAP were obtained from the CC Mangum Westgate Asphalt Plant and the Blythe Pineville Asphalt plant. The RAP stockpiles selected were based on results from Dynamic Shear Rheometer (DSR) testing on recovered binder from the respective RAP sources, gradation and asphalt content. It was desired that the two sources of RAP have differing stiffness, gradation and asphalt content in hopes to be able to apply the results of this research as widely as possible.

1.3.1.2 Task 1.2 RAP Binder Content by Ignition

As part of the RAP stockpile selection, the binder content of the RAP sources was desired. This research subtask used AASHTO T 308 “Test Method for Determining the Asphalt Content of Hot Mix Asphalt (HMA) by Ignition Method” in order to determine the binder content of each RAP sample which was later used in Task 4 during the design of mixtures containing RAP. This method also yielded aggregate from the RAP that was used to determine the aggregate gradation.

1.3.1.3 Task 1.3 RAP Aggregate Gradation

In addition to the RAP binder content, the gradations for each RAP source needed to be determined for use in mixture designs of Task 4. A sieve analysis using aggregates remaining after the ignition test was performed according to AASHTO T27-88 “Sieve Analysis of Fine and Coarse Aggregates” and AASHTO T11-90 “Material Finer Than 75μm (No. 200) Sieve in Mineral Aggregates by Washing”.

1.3.1.4 Task 1.4 RAP Aggregate Specific Gravity

In this research subtask, the RAP aggregate specific gravity was determined. This was needed in the design of mixtures containing RAP in order to calculate the specimen volumetric values (percent air voids, percent voids in mineral aggregate and percent voids filled with asphalt). AASHTO T84-88 “Specific Gravity and Absorption of Fine Aggregate” and AASHTO T85-88 “Specific Gravity and Absorption of Coarse Aggregate” were used to determine the respective specific gravities.

1.3.1.5 Task 1.5 RAP Binder Rheology

In this research subtask, the binder was reclaimed from the RAP samples using AASHTO T 319 "Test Method for the Quantitative Extraction and Recovery of Asphalt Binder from Hot Mix Asphalt". The RAP binder was dissolved and washed from the aggregate using toluene, a strong solvent. The binder was then reclaimed by heating the mixture and distilling the solvent.

AASHTO T 5-93 "Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer", AASHTO T 313-09 "Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer" and AASHTO R-29 "Standard Practice for Grading or Verifying the Performance Grade of an Asphalt Binder" were used to determine the rheological properties and Superpave performance grade of the reclaimed RAP binders. This allowed the binders to be characterized completely at high-, intermediate-, and low-temperature values.

1.3.2 Task 2 Procurement and Characterization of Virgin Materials

This research task selected and procured the virgin materials needed to complete the mixture design in research Task 4. Characterization of the virgin materials was also completed in order to determine the required properties for the mixture design. It was desired that the virgin materials selected were comprised of materials that are an actual representation of materials used in mixture design, production and construction.

1.3.2.1 Task 2.1 Virgin Material Selection and Procurement

In order to develop applicable results for the NCDOT, the virgin materials selected for use were an actual representation of materials used in asphalt concrete pavements in North Carolina. Since manufactured aggregates are less variable than natural aggregates in both specific gravity and particle size distribution, manufactured aggregates were used for this research. The types of manufactured aggregate most commonly used in North Carolina are granites and limestone. Availability of aggregate in close proximity to North Carolina State University was another element that had a deciding factor in the selection of virgin aggregate type as it is more convenient and less costly. For this reason, granite aggregate from the Knightdale Quarry was selected and procured.

Again, in order to produce the most applicable results for NCDOT, the virgin binder selected was an actual representation of binders used for asphalt concrete pavements in North Carolina. The two asphalt binder grades most commonly used in North Carolina are PG 64-22 and PG70-22.

Since the binder recovered from the RAP has an increased stiffness from aging, a softer virgin binder grade may be necessary in order for the blended RAP-virgin binder to have the desired grade for the project requirements. For this reason, this research included virgin binder grades of PG 52-28, PG 58-28 and PG 64-22.

1.3.2.2 Task 2.2 Virgin Aggregate Specific Gravity

The purpose of this subtask was to determine the specific gravity of the virgin aggregate. AASHTO T84-88 “Specific Gravity and Absorption of Fine Aggregate” and AASHTO T85-88 “Specific Gravity and Absorption of Coarse Aggregate” were used in order to determine this. This is needed for Task 4 during the design of mixtures in order to calculate the specimen volumetric values (percent air voids, percent voids in mineral aggregate and percent voids filled with asphalt).

1.3.2.3 Task 2.3 Virgin Asphalt Binder Grade Verification

In this research subtask, AASHTO TP5-93 “Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer” and AASHTO T 313-09 “ Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer” were used to verify the performance grade of the virgin binders selected. These values were used in the development of the asphalt binder blending curves discussed in Task 3 below.

1.3.3 Task 3 Development of Asphalt Binder Blending Curves

Since the binder from RAP materials has been aged during the life of the pavement, the stiffness of the blended virgin-RAP binder increases. The resulting increase in the blended binder may require a “grade shift” of the virgin binder selected in order to result in the required stiffness of the desired binder grade. The relationship of the resulting blended binder stiffness and the percentage of RAP contained in the mixture for each virgin binder grade needed to be determined. Blending curves showing this relationship were developed in order to aid in the selection of the virgin binder grade or percentage of RAP needed to produce the final binder grade required.

1.3.3.1 Task 3.1 Rheological Testing of Binder Blends

In this research subtask, the rheological properties of the asphalt binder blends containing various percentages of RAP binder blended with virgin binder was determined using AASHTO T 5-93 “Test Method for Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer” and AASHTO T 313-09 “Standard Test Method for Determining the Flexural Creep Stiffness of Asphalt Binder Using the Bending Beam Rheometer”.

1.3.3.2 Task 3.2 Development of Binder Blending Charts

The information gathered in the previous subtask was used in this subtask in order to develop binder blending charts for determining the amount of RAP allowable in mixture designs for the desired binder grade.

1.3.4 Task 4 Superpave Mixture Design

The mixtures produced in the task were used in research Task 5 and 6 for mixture performance and analysis.

1.3.4.1 Task 4.1 Design of Mixtures Containing 100% Virgin Material

In this subtask, the designs of mixtures containing 100% virgin materials were developed. The mixture designs were governed by AASHTO R 35-04 “Superpave Volumetric Design for Hot Mix Asphalt (HMA)”.

1.3.4.2 Task 4.2 Design of Mixtures Containing Various Percentages of RAP

In this subtask, specimens containing various percentages of RAP were fabricated for use in analysis during Tasks 5 and 6. Since the mixtures containing RAP were compared to the control mixture containing 100% virgin materials, the material proportions used in the RAP mixtures should replicate the material proportions used in the virgin mixture as closely as possible. To accomplish this, a gapped gradation of the virgin aggregate was used in order to account for the gradation of the RAP aggregate in order to produce a combined gradation similar to that used in the 100% virgin material mixture. The amount of virgin asphalt binder used for the virgin material mixtures was modified in order to account for the binder contributed by the RAP material in order to yield a similar optimum asphalt content.

1.3.5 Task 5 Mixture Characterization

The mixtures developed in Task 4 were characterized in research Task 5 using Superpave simple shear tests. These characterizations were used for comparisons between the various mixtures which were then used as part of the performance predictions for the mixtures. The simple shear tests are described below.

1.3.5.1 Task 5.1 Frequency Sweep Testing Using Superpave Simple Shear Tester (SST)

In this subtask, the Frequency Sweep Test at Constant Height (FSTCH) will be performed according to Procedure E of AASHTO TP7-94 “Standard Test Method for Determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalts (HMA) Using the Simple Shear Test (SST) Device” in order to provide the rheological properties of the various mixtures. The FSTCH applies a sinusoidal shear strain of 0.01% at frequencies of 10, 5, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. In accordance with AASHTO TP7, the FSTCH were performed at $20^{o}C$.

Resulting from the FSTCH, the complex modulus (G*) is known. The complex modulus measures the mixture stiffness for the range of frequencies. These values were used to compare the similarities between mixtures containing 100% virgin materials and mixtures containing various percentages of RAP (15%, 30% and 40%). As a part of the performance analysis, the complex modulus was used in the Strategic Highway research Program's (SHRP) A-003A surrogate performance prediction model for fatigue life in Task 6.1.

1.3.5.2 Task 5.2 Repeated Simple Shear Tests Using Superpave Simple Shear Tester (SST)

Device

The Repeated Simple Shear Test at Constant Height (RSSTCH) is used to measure the accumulations of permanent strain in the test specimen for the test period. The shear load is applied in a haversine pulse for a duration of 0.1 second followed by a 0.6 second unload period. This results in a total loading cycle of 0.7 second. The RSSTCH is conducted for 5000 cycles or 5 percent of the permanent shear strain for the specimen is reached. The RSSTCH measures the accumulated permanent shear strain for each specimen. As the accumulated permanent shear strain correlates to rut resistance, the values obtained from mixtures containing 100% virgin materials were compared with values obtained from mixtures containing various amounts of RAP material.

1.3.6 Task 6 Mixture Performance Analysis

In this research task, prediction models were utilized in order to estimate the performance of the mixtures for comparison between mixtures containing 100% virgin materials to mixtures containing various amounts of RAP. Analysis was also conducted in order to determine which layers of pavement could contain higher percentages of RAP without a significant reduction in performance life. This analysis provided the information needed to conduct a life cycle cost analysis for all mixtures. From the cost analysis, the cost savings resulting from use of higher percentages of RAP was determined.

1.3.6.1 Task 6.1 Pavement Performance Prediction

This subtask predicted the performance of pavements against fatigue cracking and rutting using the results from the FSTCH and RSSTCH. The surrogate models from the SHRP A-003A as well as distress models developed by the Asphalt Institute (AI) were used to perform fatigue and rutting analysis. The models used for analysis are described below.

1.3.6.1.1 Task 6.1.1 Fatigue Model Analysis

The fatigue cracking model from SHRP A-003A considers horizontal tensile strain at the bottom of the pavement layer during loading, the initial flexural loss stiffness of the mixture (S_o”) and the voids filled with asphalt (VFA). The shear stiffness measured during the FSTCH at 10Hz and 20^{o}C was used to determine the initial flexural loss stiffness using the following relationships:

$$S_o = 8.56 * (G_o)^{0.913}$$
$$S_o^{"} = 81.125 * (G_o^{"})^{0.725}$$

where,
S_o = initial flexural stiffness at 50th loading cycle (psi)
G_o = shear stiffness at 10Hz (psi)
S_o” = initial flexural loss stiffness at 50th loading cycle (psi)
G_o’’ = shear loss stiffness at 10Hz (psi)

This model requires the principle tensile strain at the bottom of the asphalt concrete layer. This was estimated using EVERSTRESS pavement analysis and design software. For this software, a pavement section must be assumed. The strain values were assessed at three locations: directly beneath the tire, at the edge of the tire and at the center of the dual tire configuration in order to consider the full loading configuration. The fatigue life of the pavement was estimated by the SHRP A-003A model as follows:

$$N_{supply} = 2.738 * 10^5 e^{0.077VFA} \varepsilon_0^{-3.624} S_o^{"-2.72}$$

where,

N_{supply} = estimated fatigue life of the pavement section in 18 kip axles (ESALs)
VFA = voids filled with asphalt for the mixture
ε_o = critical strain at the bottom of the asphalt layer

The Asphalt Institute model for determining fatigue life was also considered. Like the surrogate SHRP A-003A fatigue model, the tensile strain at the bottom of the asphalt concrete layer is needed. The AI model for estimating fatigue life follows:

$$N_f = 0.00796\varepsilon_t^{-3.291} E_1^{-0.854}$$

where,
Nf = number of load applications to fatigue failure (20% cracked area)
ε_t = tensile strain at the bottom of the asphalt layer
E_1 = elastic modulus of asphalt layer (psi)

If the two models give similar fatigue life predictions, the economic analysis will use the average predicted axle loading value. However, if the models differ significantly, the more conservative, or lowest cycles to failure will be used for the economic analysis.

1.3.6.1.2 Task 6.1.2 Rutting Model Analysis

According to SHRP A-003A, the rut depth is estimated as a function of the maximum permanent shear strain from the RSSTCH test. This relationship is as follows:

*Rut Depth (in) = 11 * Maximum permanent shear strain*

This relationship should hold true for all tire pressures but is expected to decrease with a decrease from the original pavement thickness of 15 inches [7]. The following equation converts the number of RSSTCH test loading cycles to 18-kip equivalent single axle loads (ESALs):

$$\log(\text{cycles}) = -4.36 + 1.24 \log(\text{ESALs})$$

where,
cycles = number of cycles obtained from the RSSTCH test,
ESALs = equivalent 18-kip single axle loads

Again, the Asphalt Institute model for rutting was used to estimate the rutting resistance for the pavement systems produced by the various mixtures. However, unlike the SHRP model, the AI model not only considers the mixture properties but also the pavement geometry and pavement system as a whole. The AI model considers the vertical compressive strain at the top of the subgrade layer as part of the model to determine the number of loading repetitions (N_d) required to cause a rut depth of 0.5 inch. The model is as follows:

$$N_d = 1.365 * 10^{-9} \varepsilon_c^{-4.477}$$

where,

ε_c = compression strain at top of the subgrade

Similar to the fatigue cracking analysis, EVERSTRESS pavement analysis software was used to estimate the vertical compressive strain at the top of the subgrade layer.

1.3.6.2 Task 6.2 Pavement Layer Analysis

In this research subtask, an analysis to determine the layers of pavement that can contain a higher percentage of RAP without any significant reduction in performance was performed. Mixtures with varying nominal aggregate size were used in order to simulate different layers.

1.3.6.3 Task 6.3 Economic Analysis

In this research subtask, an economic analysis was performed as a basis for comparison between mixtures containing various amounts of RAP to mixtures containing 100% virgin materials. This economic analysis of the life cycle cost of pavement sections aided in the decision making concerning the use of RAP in pavements.

An economic analysis for the life cycle cost of the pavements was conducted in order to take into account more than the initial difference in cost between pavements containing RAP material versus pavements containing 100% virgin materials. Although using RAP material for pavement construction will decrease the initial cost of construction, pavements containing RAP may require more maintenance or have a shorter service life than pavements containing 100% virgin materials reducing the initial construction savings or even resulting in more cost than pavements containing 100% virgin materials. For this reason, the life cycle analysis was utilized.

In order to conduct a life cycle cost analysis, it was recommended that either the present worth method or annual cost method be implemented. Both of these methods took into account initial costs of the pavement construction and all future year costs as well as salvage values. The present worth method converts all costs or returns to the present value for analysis. The annual cost method converts all costs and returns to a uniform annual cost for analysis. Both of these methods should result in similar conclusions and the pavement with the lowest life cycle cost should be selected. The following relationships are used for the present worth method and the annual cost method, respectively.

The present worth of a future sum can be found by:

$$PW = \frac{F}{(1+i)^n}$$

where,
PW = present worth of a sum of money that takes place *N* years from the base year.
F = future sum of an improvement at the end of year *N*, and
i = discount rate.

The present worth for several equal costs can be found by:

$$PW = \frac{A[(1+i)^N - 1]}{[i(1+i)^N]}$$

where,
PW = present worth of an annual uniform expenditure,
A = the uniform annual cost for N years,
N = the number of years in which the annual cost is experienced, and
i = discount rate.

The present worth of a gradient annual cost, or an annual cost that is expected to increase at a constant rate over time, can be calculated by:

$$PW = A_o + \left[G * \left(\frac{1}{i}\right) * \left\{\frac{(1+i)-1}{i(1+i)^N}\right\} - \left\{\frac{N}{(1+i)^N}\right\} \right]$$

where,

PW = present worth of a gradient annual cost,
A_o = value of the first expenditure at the end of the first year,
G = amount of the uniform increase per year,
N = number of years the gradient expenditure is encountered, and
i = discount rate.

The annual cost of a present worth can be calculated by:

$$A = PW * \left[\frac{i(1+i)}{(1+i)^N - 1} \right]$$

where,
A = annual uniform cost,
PW = present worth of a capital investment,
N = number of years in the analysis period, and,
i = discount rate.

The annual cost of a future investment can be calculated by:

$$A = F \left[\frac{1}{(1+i)^N - 1} \right]$$

where,
A = annual uniform cost,
F = future value of a discrete expenditure,
N = number of years from the baseline year the expenditures will take place, and,
i = discount rate.

If an annual cost is expected in increase at a constant rate over a period, the uniform annual cost can by calculated by:

$$A = A_o + G * \left(\frac{1}{i} - \frac{N}{(1+i)^N - 1} \right)$$

where,
A = uniform annual cost,
Ao = value of the expenditure at the end of the first year,
G = annual increase in the expenditure,
N = number of years in the analysis period, and,
i = discount rate.

1.4 Organization of Report

The goal of this report is to answer the research objectives. This report is organized into seven chapters. Chapter 2 gives a brief overview of the topic through a literature review. Chapter 3 discusses the material characterizations used in the research. Chapter 4 discusses the blending charts developed for aiding in the selection of RAP percentage to use. Chapters 5 and 6 discuss the mixture characterization and performance. The report is brought to a close with the summary of results and conclusions in Chapter 7.

Chapter 2
Literature Review

This chapter will give an overview of the past and current approaches towards using Recycled Asphalt Pavement (RAP) in new pavements.

As RAP is so widely available and can provide cost savings, there has been a significant amount of research in the past on the effect of using RAP in pavement design and pavement performance. Many different approaches have been developed to determine the effect of RAP on the PG grade of the blended binder. Some of these approaches include the development of blending charts from rheological testing on the binder blends using the Dynamic Shear Rheometer (DSR) and Bending Beam Rheometer (BBR) according to AASHTO TP5-93 and AASHTO TP1-93, respectively, as well as conducting Superpave testing on the blended binder.

The studies conducted by Kandhal and Foo as well as Soleymani, Bahia and Bergan developed blending charts based on Superpave criteria in order to determine the effects of RAP on Superpave binder grades.

Kandhal and Foo conducted research to develop blending charts based on Superpave criteria based on the amount of virgin binder needed to be blended with the RAP in order to produce the desired PG binder grade [3]. Although the PG binder grade is dependent on high, intermediate and low temperature criteria, this study only focused on the high and intermediate temperatures.

The high temperature value of the blended mix was determined using two blending charts, one measuring the temperatures that satisfied the criteria of $G^*/\sin\delta$ □equal to 1.0 kPa with varying percentage of RAP used in the blend and the other measuring the temperature that satisfied the criteria of $G^*/\sin\delta$ □equal to 2.2 kPa with varying percentage of RAP used in the blend on unaged and Rolling Thin Film Oven (RTFO) aged residue, respectively [3]. The more conservative or lower of these two temperatures was determined to be the high temperature value of the blended mix. The intermediate temperature value of the blended mix was determined using a blending chart that measured the temperatures that satisfied the criteria of G*sin δ equal to 5.0 MPa on RTFO and Pressure Aging Vessel (PAV) aged residue [3].

The study used a virgin binder with a Superpave grade of PG 58-22 and varying percentages of RAP with the goal of determining the amount of RAP that was needed in order to produce a final blended mixture with a Superpave grade of PG 64-22. Table 2.1 below shows the results from the blending charts developed as a result of the research.

Table 2.1. Temperature Values of Superpave Criteria based on Varying Percentages of RAP

Recycled Mix	Temperature Sweep Blending Charts	Temperature Value (^oC)	High Temperature Value (^oC)	Intermediate Temperature Value (^oC)
20% RAP	G*/sinδ = 1 kPa	64.1	64.1	19.2
	G*/sinδ = 2.2 kPa	64.8		
	G*sinδ = 5 MPa	19.2		
30% RAP	G*/sinδ = 1 kPa	66.8	66.8	19.9
	G*/sinδ = 2.2 kPa	67.3		
	G*sinδ = 5 MPa	19.9		
40% RAP	G*/sinδ = 1 kPa	70.3	70.3	21.8
	G*/sinδ = 2.2 kPa	70.4		
	G*sinδ = 5 MPa	21.8		

Source: (3)

Kandhal and Foo's analysis of the data claimed that since there was a large margin between the observed intermediate temperatures and the required intermediate temperature of 25^{o}C, "that much more RAP (more than 40% RAP) can be added to the recycled mix and still meet the intermediate temperature performance requirement. This indicates that the intermediate temperature requirement as specified by Superpave is too liberal" [3].

The study conducted by Soleymani, Bahia and Bergan. also produced blending charts based on Superpave criteria in order to determine the amount of RAP that could be included while still meeting Superpave binder grade specifications. However, this study looked at all three temperature values needed to determine the Superpave binder grade.

The Dynamic Shear Rheometer (DSR) test was used to calculate the change in shear modulus (G^*) and phase angle (δ) for the varying blended mixes in order to analyze the high and intermediate temperature criteria. The Bending Beam Rheometer (BBR) test was used to calculate the change in stiffness (S) and creep rate (m-value) for the varying blended mixes in order to analyze the low temperature criteria.

Analysis showed linear relationships for the proportion of RAP in the blended mix and all criteria: G^*, δ, S and m-value [8]. The validity of this claim was confirmed by statistical analysis. The blending chart is created by plotting the temperature that the binder satisfies the PG criteria. By conducting Superpave test on both the virgin binder and the binder recovered from the RAP to be used in the blend, two points are produced. Since a linear relationship is valid, the relationship between the temperature that the binder satisfies the PG criteria and percentage of RAP in the blend can be determined from the two points. Figure 2.1 below shows an example of the blending chart produced by a sample in this study for all three temperature values.

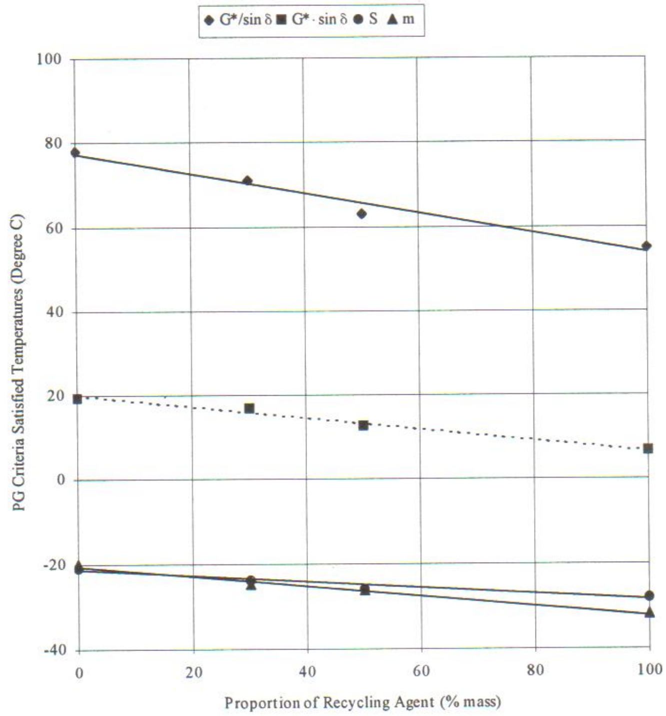

FIGURE 8 PG grading blending chart for binders blended with 300–400 asphalt binder.

Figure 2.1. Blending Chart for 300-400 Asphalt Binder

Source: (8)

Kennedy, Tam and Solaimanian conducted a research study looking into the use of Superpave testing in order to determine the effect of RAP in the blended mix. Criteria for all temperature values were considered.

Samples were prepared with compositions of varying percentages of RAP binder mixed with virgin binder. The blended binders were subjected both to short term aging through RTFO to simulate aging during construction and long term aging though PAV to simulate field aging.

In order to determine the PG grade of the blended mix, the DSR and BBR Superpave tests were used for the high and intermediate temperature values and the low temperature value, respectively, after being properly aged for each method. The DSR results for high temperature values indicated that the stiffness, measured by G*/sinδ, increased with increasing levels of RAP in the blend [4]. Results also indicate that at higher percentages of RAP, the stiffness increase becomes larger with increasing RAP percentage, whereas at smaller percentages of RAP, the increase of the percentage of RAP has a minimal effect on the stiffness [4]. Again, at the intermediate and low temperature values, higher percentages of RAP had the greatest effect on the stiffness of the blend [4]. However, the logarithmic creep rate experiences the opposite relationship, with lower percentages of RAP having a greater influence change than higher percentages of RAP [4]. Figure 2.2 shows the effect of the percentage of RAP in a blend at high temperatures on the stiffness. Similar figures were developed for all blends of different binder types and different temperature values. Table 2.2 below contains the resulting PG grade for the blended mixes in this study at varying percentages of RAP. Kennedy et al. state that “Superpave tests are recommended to be performed on the blend at four different blend percentages to determine the behavior of the blend. From this, the range of allowable RAP content is determined” [4].

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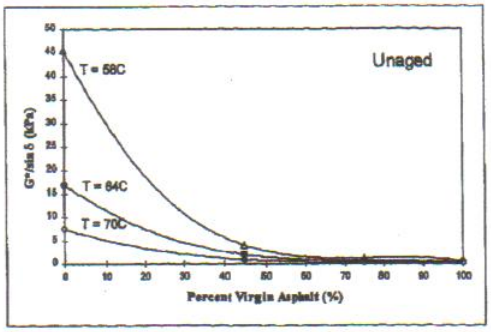

Figure 1. Plot of G*/sinδ versus Percent Virgin Asphalt for Unaged AF Blend

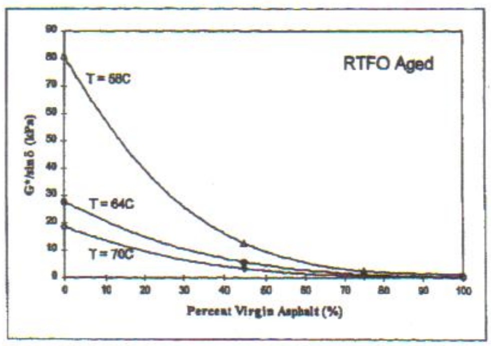

Figure 2. Plot of G*/sinδ versus Percent Virgin Asphalt for RTFO Aged AF Blend

Figure 2.2. G*/sinδ Versus Percent Virgin Asphalt for Unaged AF and RTFO Aged AF

Source: (4)

Table 2.2. PG Grade and Blend Combinations

Percent Virgin Asphalt (%)	Virgin: AAM-1 RAP: AAF-1	Virgin: AAA-2 RAP: AAF-1	Virgin: AAD-1 RAP: AAG-1
0	PG 76-10	PG 76-10	PG 76-16
25	PG 70-10	-	-
45	PG 70-16	PG 64-22	PG 70-16
65	PG 70-16	-	-
75	PG 70-16	PG 58-34	PG 64-22
85	PG 64-16	-	-
100	PG 64-16	PG 46-34	PG 58-28

Source: (4)

Analysis concludes that the binder performance grade is not affected by the addition of up to 15% RAP, whereas the binder performance grade is increased one high temperature grade by the addition of 25% RAP [4]. This same addition only affects one blend low temperature grade, with an increase of one grade.

Kennedy et al. concluded that conducting Superpave testing on blended mixes is a legitimate procedure for determining the percentage of RAP that can be used in order to produce a blend that meets the specified binder criteria [4].

As the above research shows, various methods have been used to determine the effect of the addition of RAP on the performance grade of the blended binder. However, this research will focus on the effect of higher percentages of RAP used in mixtures.

The research conducted by Stephens, Mahoney and Dippold was twofold. First they studied the effect of heating the RAP before mixing on the amount of blending between the virgin binder and RAP binder. Second, they studied the effective PG binder grade of a blended mix using the indirect tension test.

In order to determine the effect of heating the RAP before mixing on the amount of blending between the virgin binder and RAP binder, samples were fabricated with RAP that had been heated prior to mixing for varying amounts of time. The specimens were then tested using both unconfined compression and indirect tension tests in order to determine the change in strength of the specimens which would correlate the amount of blending that occurred between the virgin binder and the RAP binder. Analysis showed that the strength did increase with the addition of RAP that had not been heated prior to mixing suggesting that blending between the virgin binder and RAP binder had occurred. Further increases in strength with heating shows that "more complete binder blending occurs in HMA mix containing RAP if the RAP during the mixing reaches a temperature that softens the RAP binder allowing intimate blending" [9].

Stephens et al. claim that the unconfined compression and indirect tension tests could both "produce a method for determining the effective PG grade of the binder in a HMA mix that contains RAP" [9] due to the results of the tests conducted on the specimens to determine the amount of preheating needed. For this research, however, the indirect tension test was used.

Virgin binder with grades of PG 58-34 and PG 64-28 were both used. Specimens were fabricated using all virgin binder as well as a blend with 15% RAP binder and virgin binder of both grades. The average tensile loads for the two specimens with all virgin binder were then plotted against the PG grade. A linear relationship between these two points was assumed. Then the average tensile load for the two specimens with RAP and virgin binder blend were plotted. The PG grade for the blended specimens can be calculated by moving horizontally from the point to intersect the line connecting the all virgin binder indirect tensile average strengths and dropping down to the x-axis in order to read the PG grade [9]. Figure 2.3 below shows the graph for the indirect tensile tests conducted at 28°C. Testing was also done at other temperatures to simulate low temperature criteria as well as temperatures closer to Superpave service temperatures. Specimens with other percentages of RAP were also tested.

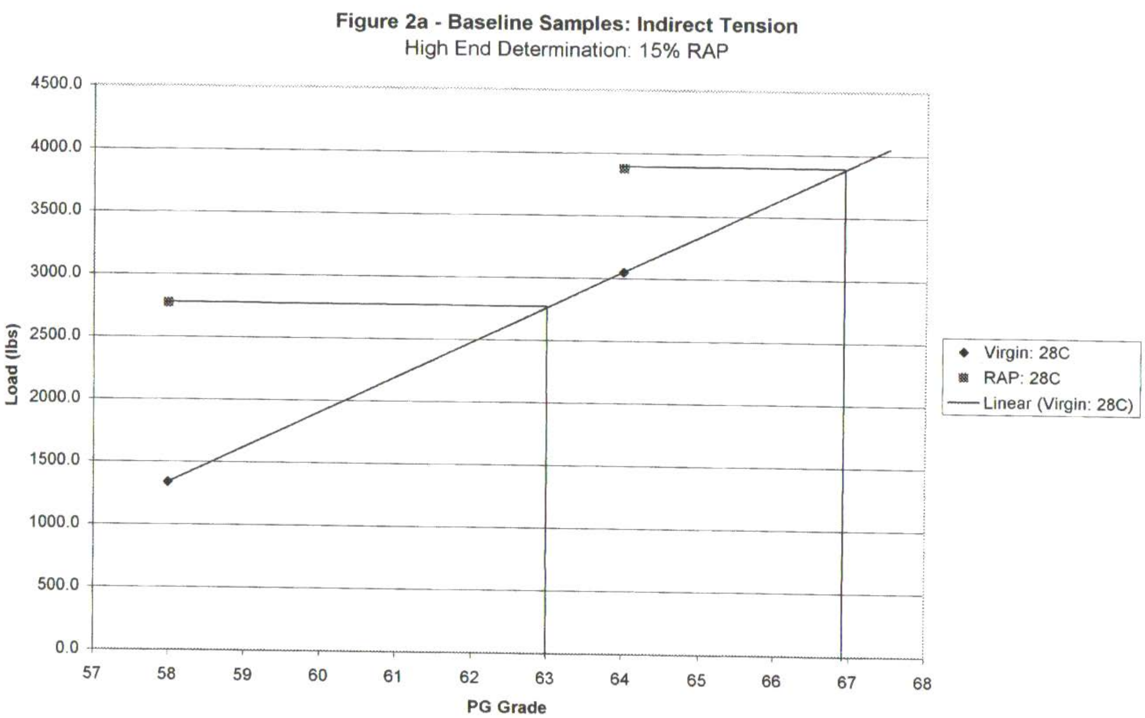

Figure 2.3. Indirect Tension vs. PG Grade at 28°C

Source: (9)

The analysis of the results show that an increase in RAP increases both the high and low temperature PG values. Stephens et al. state: “This change is anticipated because of the additional RAP binder tends to increase the hardness of the combined binders. This indicates that the test is sensitive enough to detect changes in the effective PG grade” [9].

It is a widely accepted fact that the use of Recycled Asphalt Pavement in the construction of new asphalt pavements can be beneficial, as it provides both cost savings and a reduction of environmental effects. However, the development of Superpave did not consider the use of RAP when determining the criteria for mix and pavement design. Therefore, research has been conducted in order to determine a procedure to measure the effects of the addition of RAP to Superpave mixtures. This chapter provided an overview of different research studies that examined this effect. However, most of these studies focused on the addition of lower percentages of RAP and also did not research the effect of performance of using RAP in construction. Therefore, this research will focus on using higher percentages of RAP and the effects on pavement performance.

The following chapter will describe the characteristics of the materials used. It will also describe the mix design procedure and results used for this research.

Chapter 3

Material Characteristics and Mix Design

This chapter will discuss the characteristics of the materials used as well as the mix design procedure for the selected mixtures.

3.1 Virgin Materials

A single source of aggregate was used for this research. Granite aggregate from the Knightdale Quarry was procured since it reflected an aggregate commonly used in North Carolina mix designs and due to its close proximity to North Carolina State University.

Once the material was procured, the aggregates properties were measured. Table 3.1 below contains these properties.

Table 3.1 Aggregate Specific Gravity

	Bulk S.G.	Apparent S.G.
Coarse	2.610	2.652
Fine	2.614	2.656

Three virgin binder grades were used in this research, PG 52-28, PG 58-22 and PG 64-22. Each of the virgin binders were artificially aged in the Rolling Thin Film Oven (RTFO) in order to simulate the aging due to the mixture process and construction. The residue from the RTFO was then aged further in the Pressure Aging Vessel (PAV) in order to simulate long term aging.

Dynamic Shear Rheometer (DSR) testing was completed on the virgin binders. The DSR testing produced rheological properties for the binders including complex modulus (G*) and the phase angle (delta, δ). Each binder was tested at several temperatures in order to form various temperature gradients to be used for the blending charts discussed in the next chapter.

Bending Beam Rheometer (BBR) testing was also completed on the virgin binders. The BBR testing produced rheological properties for the binders including creep stiffness (S) and slope (m).

Table 3.2, Table 3.3, Table 3.4 and Table 3.5 below contain the rheological data obtained from the DSR and BBR tests used to verify the virgin binder grades.

Table 3.2 Binder Rheological Properties-DSR Original Binder

Virgin Binder Grade	Average G*/sinδ (Standard Deviation) At Test Temperature				
	52°C	**58 °C**	**64 °C**	**70 °C**	**76 °C**
PG 52-28	1.58 (0.02)	0.76 (0.10)	0.41 (0.03)	0.21 (0.01)	0.12 (0.01)
PG 58-22		1.62 (0.12)	0.78 (0.05)	0.38 (0.04)	0.21 (0.02)
PG 64-22			1.71 (0.14)	0.85 (0.07)	0.83 (0.02)

Table 3.3 Binder Rheological Properties-DSR RTFO Binder

Virgin Binder Grade	Average G*/sinδ (Standard Deviation) At Test Temperature				
	52°C	**58 °C**	**64 °C**	**70 °C**	**76 °C**
PG 52-28	3.54 (0.18)	1.57 (0.09)	0.75 (0.03)	0.38 (0.02)	0.20 (0.01)
PG 58-22		5.05 (0.27)	2.43 (0.07)	1.23 (0.04)	0.64 (0.05)
PG 64-22			5.25 (0.14)	2.51 (0.04)	1.23 (0.03)

Table 3.4 Binder Rheological Properties-DSR PAV Binder

Virgin Binder Grade	Average $G^*(\sin\delta)$ (Standard Deviation) At Test Temperature					
	16 °C	19 °C	22 °C	25 °C	28 °C	31 °C
PG 52-28	3284 (504)	2110 (329)	1336 (212)	827 (134)	516 (88)	322 (58)
PG 58-22	5472 (128)	3639 (76)	2393 (40)	1551 (15)	1019 (3)	675 (3)
PG 64-22	12115 (486)	8755 (356)	5884 (242)	3954 (173)	2659 (126)	1776 (95)

Table 3.5 Virgin Binder Rheological Properties-BBR PAV Binder

Virgin Binder Grade	Creep Stiffness (Mpa)	Slope (m)
PG 52-28	194	0.375
PG 58-22	80.7	0.405
PG 64-22	170.5	0.349

3.2 RAP Sources

Two RAP sources were sought out to be used in this research. It was desired to choose two sources that had various stiffness, gradation and asphalt content in hopes to be able to apply the results of this research as widely as possible. In order to achieve this, eight different sources of RAP were selected from across the state. RAP samples from each of the eight sources were procured. The binder from the RAP was extracted by NCDOT personnel for rheological testing. Ignition testing was also performed on each of the samples in order to perform sieve analysis and to determine the asphalt content.

Table 3.6 contains the source of each RAP. Table 3.7 below contains the gradation for each of the RAP sources. Table 3.8 below contains the average asphalt content for each of the RAP sources.

Table 3.6 RAP Source Key

RAP	Source
RAP #1	Barnhill, Fayetteville RAP #1
RAP #2	Barnhill, Fayetteville RAP #2
RAP #3	Maymead Marion
RAP #4	Blythe Pineville
RAP #5	ST Wooten Wilmington
RAP #6	CC Mangum Knightdale
RAP #7	CC Mangum Westgate
RAP #8	ReaWest Raleigh

Table 3.7 Average RAP Aggregate Gradations

Sieve Size (mm)	RAP #1	RAP #2	RAP #3	RAP #4	RAP #5	RAP #6	RAP #7	RAP #8
37.5	100	100	100	100	100	100	100	100
25.0	100	100	100	100	100	100	100	100
19.0	100	100	100	100	100	100	100	100
12.5	99	99	98	98	98	100	98	99
9.50	97	96	91	93	95	97	95	96
4.75	81	85	70	74	84	83	79	77
2.36	65	73	48	59	70	67	63	60
1.18	54	62	34	50	60	52	50	48
0.600	40	47	25	40	49	38	37	35
0.300	25	27	18	27	35	24	26	24
0.150	15	14	13	17	18	14	16	15
0.075	8.5	8.6	9.4	10.2	11.4	8.8	10.3	9.4

Table 3.8 Average RAP Asphalt Content

RAP Source	Asphalt Content (%)
RAP #1	5.6
RAP #2	5.1
RAP #3	4.1
RAP #4	4.8
RAP #5	5.5
RAP #6	4.7
RAP #7	5.2
RAP #8	5.2

The binders extracted from the RAP were artificially aged in the RTFO in order to simulate the aging due to the mixture process and construction. The residue from the RTFO was then aged further in the PAV in order to simulate long term aging.

DSR testing was completed on the binder extracted from each RAP source. The DSR testing produced rheological properties for the binders including complex modulus (G*) and the phase angle (delta, δ). Tables 3.9-3.11 contain the rheological data for the original, RTFO and PAV aged binders, respectively.

Table 3.9 Average G*/sinδ for Original RAP Binders

Temperature (^oC)	Average G*/sinδ for Original RAP Binders (kPa)							
	Barnhill 1	Barnhill 2	CCM-Knightdale	CCM-Westgate	Maymead Marion	Pineville	Rea	ST Wooten
64		45.669		36.9785		29.7765	22.0775	32.9245
70		20.371		17.7695		13.3955	10.2735	14.803
76	5.377	9.18	6.38	8.585	3.4665	6.1475	4.891	6.767
82	2.5615	4.1755	3.303	4.08	1.651	2.88	2.3735	3.1135
88	1.28	2.01	1.702	2.052	0.8125	1.414	1.192	1.4775

Table 3.10 Average G*/sinδ for RTFO Aged RAP Binders

Temperature (^oC)	Average G*/sinδ for Original RAP Binders (kPa)							
	Barnhill 1	Barnhill 2	CCM-Knightdale	CCM-Westgate	Maymead Marion	Pineville	Rea	ST Wooten
64				112.753				
70				52.858				
76	46.7245	13.0045	24.1365	24.8505	15.7495	20.6775	25.7955	42.3425
82	22.4805	6.1205	11.9325	11.5555	7.0235	9.624	12.3005	19.0525
88	10.846	2.958	5.842	5.5995	3.1535	4.5435	5.9585	8.6075

Table 3.11 Average G*sinδ for PAV Aged RAP Binders

Temperature (^oC)	Average G*/sinδ for Original RAP Binders (kPa)							
	Barnhill 1	Barnhill 2	CCM-Knightdale	CCM-Westgate	Maymead Marion	Pineville	Rea	ST Wooten
28							7134.3	
31	12518.3	7460.65	8758.7	6213.6	10651.4		5472.6	13491.9
34	9757.95	5415.35	6565.35	5505.6	7906.65	7039.6	4547.4	10266.55
37	7360.1	3845.2	4834.9	4453	5669.1	5133.7		7620.05
40	5428.95		3767.4		5193.6	3749.7		7879.8
43	4419.5				3643.8			5626.5

In order for the results of the research to be applicable to as much of the state as possible, it is desirable to choose two RAP sources that differ in characteristics. The two RAP sources selected are Blythe Pineville and CC Mangum Westgate. The Blythe Pineville RAP consists of a coarse gradation and a low asphalt content. The DSR testing shows that the binder is less stiff than the others. The CC Mangum Westgate RAP consists of a coarse gradation and a higher asphalt content. The DSR testing shows that the binder is more stiff than most others. The two RAP sources also represent various regions of the state, with Blythe Pineville coming from the southwestern region and CC Mangum Westgate coming from the central region. The gradations resulting from blending the RAP sources at various percentages with the virgin gradation meets specifications for both 9.5 mm mixtures as well as 19.0 mm mixtures.

Tables 3.12-3.14 show the full comparison of DSR testing for the two RAP sources. Due to the increase in stiffness of the reclaimed RAP binder, the specifications for DSR testing on both unaged binder and RTFO aged binder are met for all temperatures. However, for the DSR testing on the PAV aged binder, the reclaimed RAP binder does not pass the specification at any temperature. Blending the RAP binder with virgin binders is needed in order for the specification to be met.

Table 3.12 RAP Binder Rheological Properties-Original Binder

RAP Source	Average G*/sinδ (Standard Deviation) At Test Temperature				
	52°C	**58 °C**	**64 °C**	**70 °C**	**76 °C**
Westgate	154.6 (5.2)	70.3 (2.1)	33.5 (3.3)	15.8 (1.8)	7.5 (1.0)
Pineville	228.8 (2.8)	97.6 (1.4)	36.2 (6.8)	16.0 (2.9)	7.2 (1.2)

Table 3.13 RAP Binder Rheological Properties-RTFO Binder

RAP Source	Average G*/sinδ (Standard Deviation) At Test Temperature				
	52°C	**58 °C**	**64 °C**	**70 °C**	**76 °C**
Westgate	630.1 (40.7)	283.8 (16.9)	112.8 (N/A)	52.9 (N/A)	24.9 (0.03)
Pineville	563.2 (34.9)	251.2 (15.0)	110.8 (6.0)	49.8 (2.4)	21.4 (1.1)

Table 3.14 RAP Binder Rheological Properties-PAV Binder

RAP Source	Average G*(sinδ) (Standard Deviation) At Test Temperature					
	16 °C	**19 °C**	**22 °C**	**25 °C**	**28 °C**	**31 °C**
Westgate	30708.6 (1089.0)	25441.2 (888.1)	20494.4 (679.2)	15982.0 (609.0)	12192.1 (467.2)	9084.7 (345.7)
Pineville	26916.1 (3758.8)	22378.6 (3283.3)	18242.3 (2889.0)	14358.6 (2355.3)	11092.3 (1838.0)	8366.3 (1369.3)

3.3 Mix Design Procedure

Since PG 64-22 is the standard asphalt binder grade used in North Carolina, mixtures using PG 64-22 were desired for this project. One surface course and one intermediate course were desired in order to analyze the different effects of higher percentages of RAP in the different courses. S-9.5B and I-19.0C were the two NCDOT mixture types selected for use in this project.

Virgin mix designs, mixes containing 0% RAP, were conducted for both the 9.5 mm and 19.0 mm mixtures. Initially, three trial aggregate structures were chosen that passed the control points for the respective nominal maximum size aggregate mixtures. Table 3.15 contains the three trial aggregate structures for the 9.5 mm mixtures. The three trial aggregate structures were mixed at a constant binder content. Based on the volumetric calculations for the trial aggregate structures, a design aggregate structure was selected based on the specified NCDOT volumetric properties. Table 3.16 contains the volumetric properties for each of the trial aggregate structures. As can be seen in the table, only the first trial aggregate structure satisfies all the specifications. Therefore, this will become the design aggregate structure. Table 3.17 below contains the design aggregate structure for the 9.5 mm mixture. Figure 3.1 below shows the 9.5 mm design aggregate gradation.

Table 3.15 9.5 mm Trial Aggregate Structures

Sieve Size, mm	% Passing		
	Trial Blend #1	Trial Blend #2	Trial Blend #3
12.5	100	100	100
9.50	97	93	95
4.75	86	58	70
2.36	65	41	40
1.18	50	27	29
0.600	30	18	22
0.300	22	13	17
0.150	10	8	8
0.075	6	4	5

Table 3.16 Trial Aggregate Structure Volumetric Properties

	Trial Blend #1	Trial Blend #2	Trial Blend #3	NCDOT Spec
Trial Asphalt Content % of Total Mixture	5.3	5.3	5.3	--
% G_{mm} at N_{des}	94.9	96.9	96.8	--
% Air Voids at N_{des}	5.06	3.06	3.19	--
Corrected Air Voids at N_{des}	4.0	4.0	4.0	**4.0**
Estimated Asphalt Content (P_b) for 4% Air Voids	5.6	4.8	4.9	--
Estimated VMA at P_b	15.5	**13.8**	**14.0**	**Min. 15.0 %**
Estimated VFA at P_b	74.2	71.1	71.3	**65-80**
% G_{mm} at N_{ini}	87.9	86.3	86.9	**Max. 90.5**
Estimated Effective Asphalt Content	5.15	4.35	4.40	--
Dust Proportion	1.165	0.920	1.136	**0.6-1.4**

Table 3.17 9.5 mm Design Aggregate Structure

	% Passing	
Sieve Size, mm	**Mix Gradation**	**Control Points**
12.5	100	100
9.50	97	90-100
4.75	86	> 90
2.36	65	32-67
1.18	50	
0.600	30	
0.300	22	
0.150	10	
0.075	6	2-10

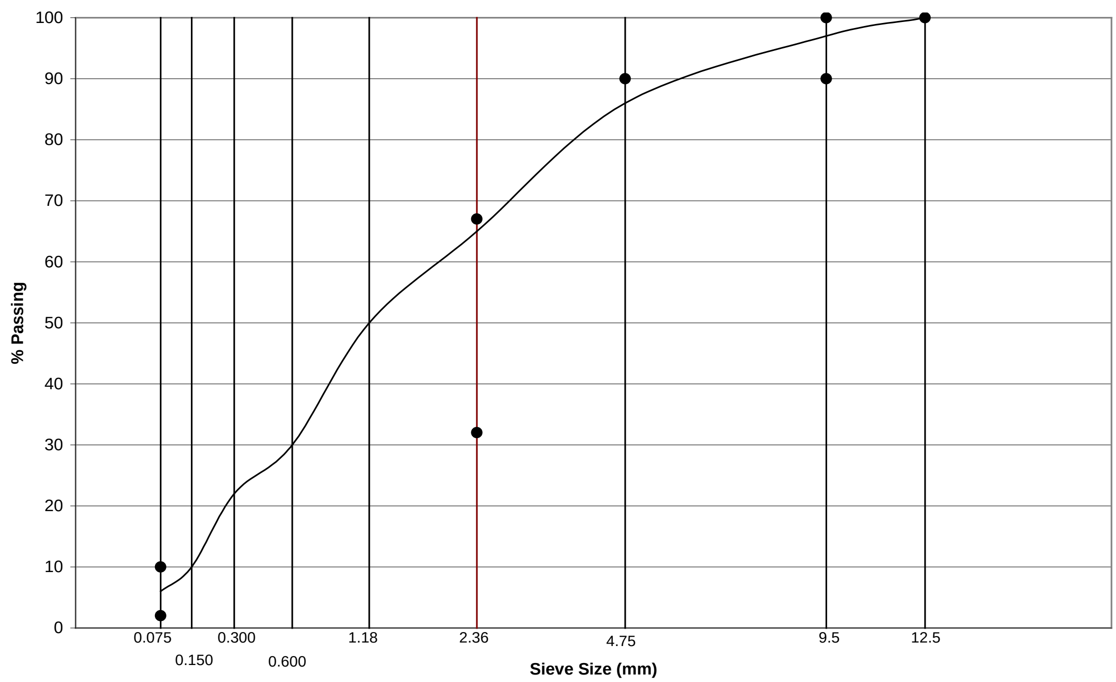

Figure 3.1 Gradation

Table 3.18 contains the three trial aggregate structures for the 19.0 mm mixtures. The three trial aggregate structures were mixed at a constant binder content. Based on the volumetric calculations for the trial aggregate structures, a design aggregate structure was selected based on the specified NCDOT volumetric properties. Table 3.19 contains the volumetric properties for each of the trial aggregate structures. As can be seen in the table, only the third trial aggregate structure satisfies all the specifications. Therefore, this will become the design aggregate structure. Table 3.20 below contains the design aggregate structure for the 19.0 mm mixture. Figure 3.2 below shows the 19.0 mm design aggregate gradation.

Table 3.18 19.0 mm Trial Aggregate Structures

Sieve Size, mm	% Passing		
	Trial Blend #1	Trial Blend #2	Trial Blend #3
25.0	100	100	100
19.0	95	94	93
12.5	76	73	75
9.5	61	60	65
4.75	44	37	40
2.36	32	25	27
1.18	21	14	16
0.6	15	11	10
0.3	12	8	7
0.15	10	5	5
0.075	5	4	4

Table 3.19 Trial Aggregate Structure Volumetric Properties

	Trial Blend #1	Trial Blend #2	Trial Blend #3	NCDOT Spec
Trial Asphalt Content % of Total Mixture	4.4	4.4	4.4	--
% G_{mm} at N_{des}	94.9	95.1	92.6	--
% Air Voids at N_{des}	5.1	4.96	7.36	--
Corrected Air Voids at N_{des}	4	4	4	**4.0**
Estimated Asphalt Content (P_b) for 4% Air Voids	5.1	5.0	6.1	--
Estimated VMA at P_b	14.0	14.6	15.5	**Min. 13.0 %**
Estimated VFA at P_b	**63.7**	66.1	74.1	**65-75**
% G_{mm} at N_{ini}	84.7	84.7	82.9	**Max. 90.0**
Estimated Effective Asphalt Content	3.82	4.32	5.6	--
Dust Proportion	**1.832**	**1.619**	0.710	**0.6-1.4**

Table 3.20 19.0 mm Design Aggregate Structure

Sieve Size, mm	% Passing	
	Mix Gradation	Control Points
25.0	100	100
19.0	93	90-100
12.5	75	> 90
9.5	65	
4.75	40	
2.36	27	23-49
1.18	16	
0.6	10	
0.3	6	
0.15	5	
0.075	4	3-8

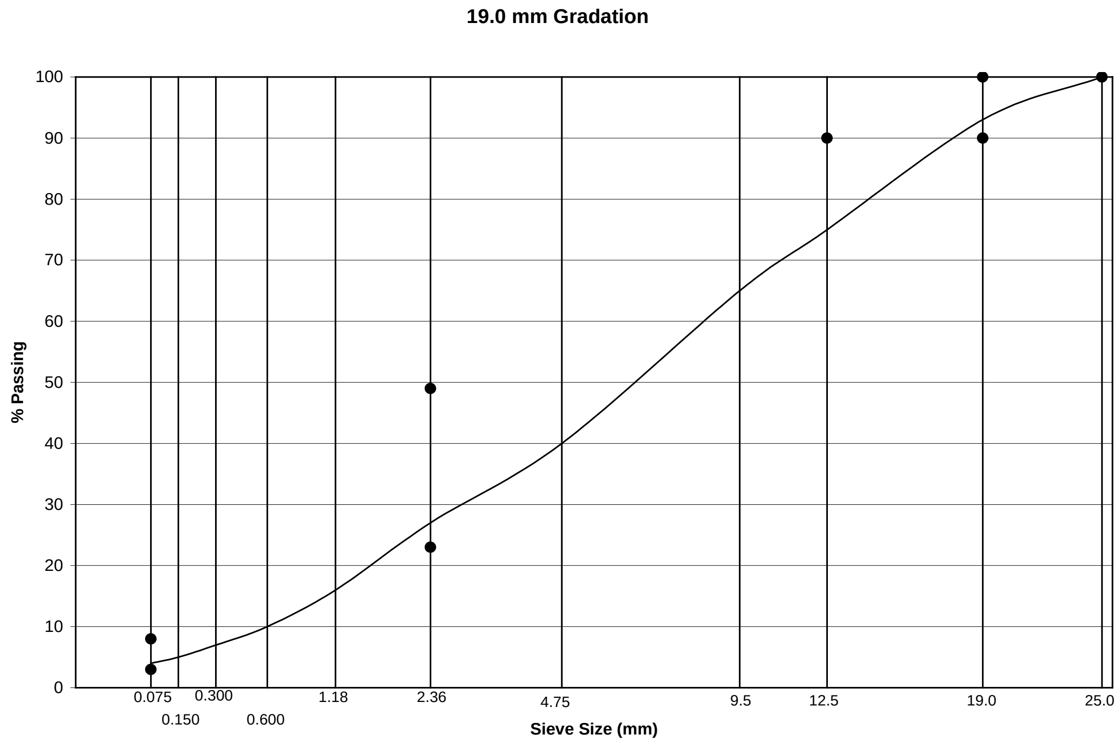

Figure 3.2 Gradation

Once the design aggregate structure was selected, mixtures with different asphalt contents were tested in order to determine the optimum asphalt content based on 4% air voids at N_{des}. The optimum asphalt contents for the 9.5 mm mixtures with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder were 5.56%, 5.37% and 5.765%, respectively. The optimum asphalt contents for the 19.0 mm mixtures with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder were

5.24%, 5.37% and 5.04%, respectively. Once the optimum asphalt content was determined, volumetric properties were checked at the corresponding asphalt content to determine if the specifications were met. Table 3.21, Table 3.22 and Table 3.23 below compare the volumetric properties at the optimum asphalt content to the specifications for the 9.5 mm mixtures with PG 64-22 binder, PG 58-22 binder and PG 52-28, respectively. Table 3.24, Table 3.25 and Table 3.26 below compare the volumetric properties at the optimum asphalt content to the specifications for the 19.0 mm mixtures with PG 64-22 binder, PG 58-22 binder and PG 52-28, respectively.

Table 3.21 Volumetric Properties for 9.5 mm Mixture with PG 64-22 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	15.53	> 15%
%VFA	73.8	65-80%
%Gmm @ Nini	87.9	$\leq$ 90.5%

Table 3.22 Volumetric Properties for 9.5 mm Mixture with PG 58-22 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	15.61	> 15%
%VFA	74.1	65-80%
%Gmm @ Nini	88.2	$\leq$ 90.5%

Table 3.23 Volumetric Properties for 9.5 mm Mixture with PG 52-28 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	15.48	> 15%
%VFA	74.1	65-80%
%Gmm @ Nini	88.1	$\leq$ 90.5%

Table 3.24 Volumetric Properties for 19.0 mm Mixture with PG 64-22 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	14.3	> 13%
%VFA	72.2	65-75%
%Gmm @ Nini	85.5	$\leq$ 90%

Table 3.25 Volumetric Properties for 19.0 mm Mixture with PG 58-22 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	14.52	> 13%
%VFA	72.9	65-75%
%Gmm @ Nini	87.8	$\leq$ 90%

Table 3.26 Volumetric Properties for 19.0 mm Mixture with PG 52-28 Binder

	Mixture	**Specification**
%Air	4%	4%
%VMA	14.26	> 13%
%VFA	72.27	65-75%
%Gmm @ Nini	86.3	$\leq$ 90%

These mix designs were also used for the fabrication of specimens that contained RAP. For the results of the mixture characterization discussed in Chapter 5 of both the virgin and RAP specimens to be comparable, the RAP specimens were fabricated using the same mix design as the virgin specimens. In order to account for the aggregates contained in the RAP, the gradation used for the virgin mix designs was adjusted so that the combination of the aggregates contributed by the RAP and the virgin aggregates would follow the original virgin gradation as closely as possible. The amount of virgin asphalt binder was also adjusted to account for the binder contributed by the RAP.

This chapter has described the characteristics of the materials used in this research. It also gave an overview of the mix design for the virgin mixtures. The next chapter will discuss the blending charts developed from the rheological testing conducted for this chapter.

Chapter 4

Asphalt Binder Blending Charts

In this chapter, the binder rheology of the blended RAP binders and the formation of blending charts will be discussed. The blending charts will allow contractors to determine the amount of RAP that is acceptable to use in mixtures.

4.1 Dynamic Shear Rheometer Testing

Dynamic Shear Rheometer (DSR) testing was completed on the blended binders. The DSR testing produced rheological properties for the binders including complex modulus (G*) and the phase angle (delta, δ). Each binder was tested at several temperatures in order to form various temperature gradients for the blending charts. The blended binders consisted of a mix of 70% virgin binder and 30% binder extracted from the RAP sources.

Table 4.1 below contain the average G*/sinδ values for the original blended binders. The standard deviation is denoted in parentheses. Figure 4.1 depicts the results of Table 4.1.

Table 4.1. G*/sinδ Values for Original Blended Binders

Virgin Binder Grade	RAP Source	Average G*/sinδ (Standard Deviation) At Test Temperature				
		52°C	58 °C	64 °C	70 °C	76 °C
PG 52-28	**Westgate**	8.52 (0.82)	3.69 (0.30)	1.59 (0.14)	0.76 (0.07)	0.42 (0.02)
	Pineville	6.20 (0.27)	2.30 (0.45)	1.06 (0.18)	0.60 (0.03)	0.30 (0.02)
PG 58-22	**Westgate**		6.16 (0.50)	2.73 (0.16)	1.34 (0.04)	0.68 (0.02)
	Pineville		5.47 (0.27)	2.41 (0.21)	1.19 (0.09)	0.61 (0.04)
PG 64-22	**Westgate**			4.87 (0.32)	2.35 (0.15)	1.14 (0.07)
	Pineville			3.74 (0.04)	1.83 (0.04)	0.91 (0.01)

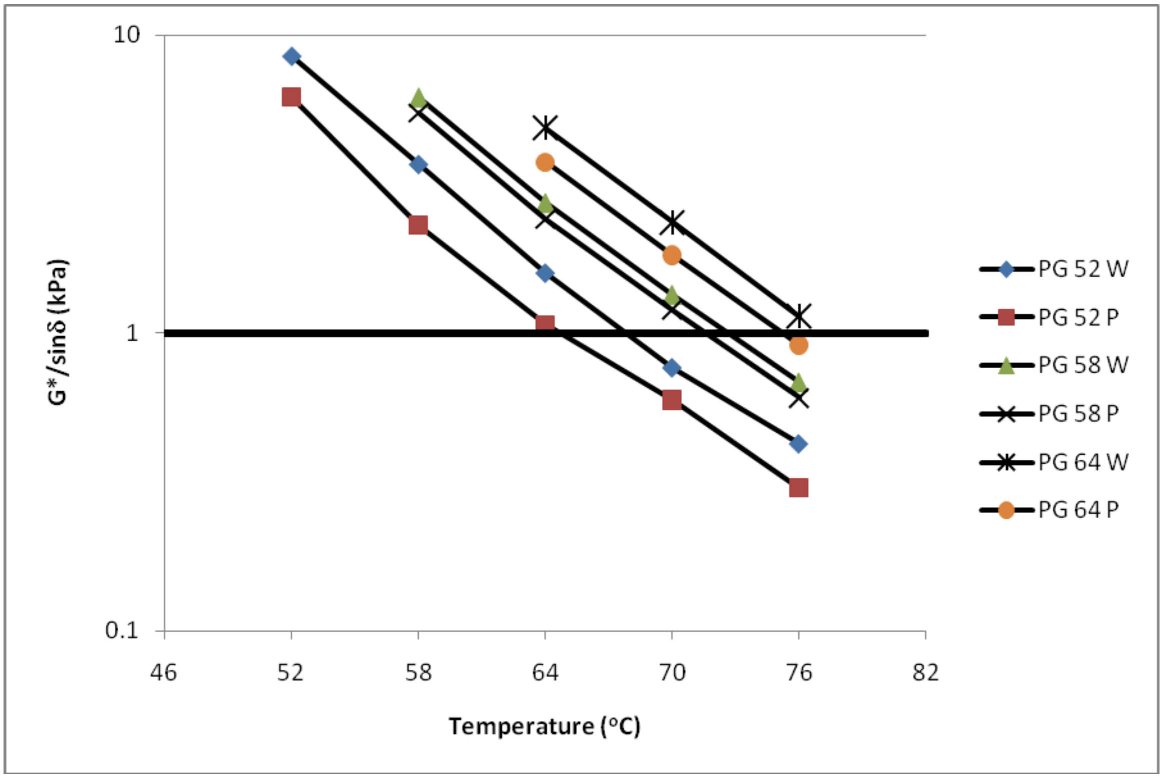

Figure 4.1 DSR Test Results for Blended Original Aged Binders Containing 30% RAP

The results of Figure 4.1 depict how the addition of 30% RAP increases the stiffness of the binder grade. Due to this increase, the highest temperature grading for the binder has increased from the original virgin binder. To determine the highest temperature grade, determine the highest temperature that G*/sinδ ≥ 1.0 kPa. PG 52 binders become PG 64, PG 58 binders become PG 70 and PG 64 binders become PG76. This shows that the addition of 30% RAP increases the original binder by two grades.

Table 4.2 below contain the average G*/sinδ values for the RTFO aged blended binders. The standard deviation is denoted in parentheses. Figure 4.2 depicts the results of Table 4.2.

Table 4.2 G*/sinδ Values for RTFO Aged Blended Binders

Virgin Binder Grade	RAP Source	Average G*/sinδ (Standard Deviation) At Test Temperature				
		52°C	58 °C	64 °C	70 °C	76 °C
PG 52-28	Westgate	21.74 (1.90)	9.20 (0.66)	4.14 (0.23)	1.90 (0.13)	0.92 (0.07)
	Pineville	6.31 (0.46)	2.65 (0.13)	1.21 (0.07)	0.55 (0.05)	0.30 (0.03)
PG 58-22	Westgate		22.35 (1.88)	10.77 (0.62)	5.12 (0.26)	2.52 (0.09)
	Pineville		15.95 (0.60)	7.35 (0.29)	3.50 (0.12)	1.72 (0.06)
PG 64-22	Westgate			17.11 (1.44)	7.81 (0.82)	3.63 (0.47)
	Pineville			17.26 (0.95)	7.99 (0.59)	3.73 (0.36)

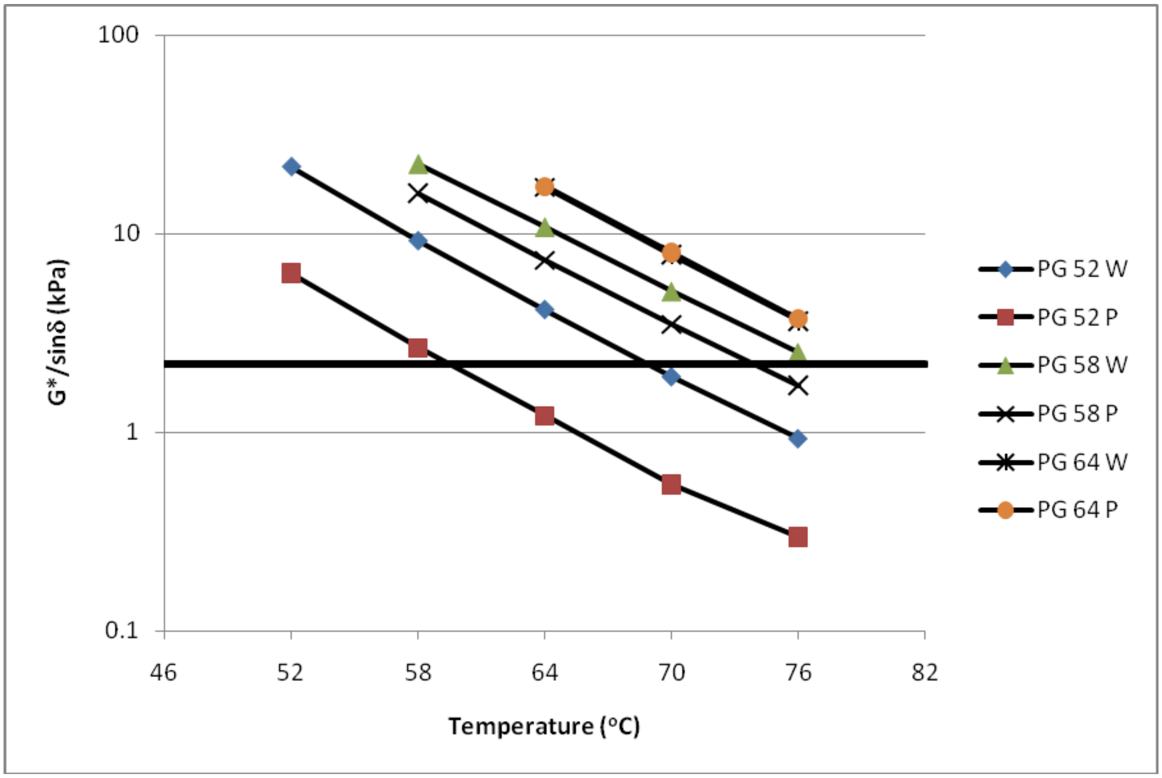

Figure 4.2 DSR Test Results for Blended RTFO Aged Binders Containing 30% RAP

The results of Figure 4.2 depict how the addition of 30% RAP increases the stiffness of the binder grade. Due to this increase, the highest temperature grading for the binder has increased from the original virgin binder. To determine the highest temperature grade, determine the highest temperature that G*/sinδ ≥ 2.2 kPa. The PG 52 Pineville blended binder becomes PG 58, the PG 52 Westgate blended binder becomes PG 64, the PG 58 Pineville blended binder becomes PG 70, the PG 58 Westgate blended binder becomes PG 76 and the PG 64 binders become PG 76. This shows that the addition of 30% RAP increases the original binder by at least two grades, with the exception of the PG 52 Pineville binder blend.

For both the original and RTFO aged binders, as the temperature increases the value of G*/sinδ decrease while holding the binder constant. Similarly, the value of G*/sinδ increases as the virgin binder grade increases from PG 52-28 to PG 64-22 while holding the temperature constant. This is expected because the PG 52-28 is a softer binder than both PG 58-22 and PG 64-22.

Table 4.3 below contains the average G*(sinδ) values for the PAV aged blended binders. The standard deviation is denoted in parentheses. Figure 4.3 depicts the results of Table 4.3.

Table 4.3 Average G*(sinδ) for PAV Aged Binders.

Virgin Binder Grade	**RAP Source**	**Average G*(sinδ) (Standard Deviation) At Test Temperature**					
		16 °C	**19 °C**	**22 °C**	**25 °C**	**28 °C**	**31 °C**
PG 52-28	**Westgate**	9753 (1261)	7022 (917)	4923 (639)	3378 (440)	2309 (302)	1437 (830)
	Pineville	10196 (580)	7233 (384)	5039 (272)	3411 (185)	2304 (124)	1533 (84)
PG 58-22	**Westgate**	8223 (230)	5913 (N/A)	3998 (N/A)	2761 (1594)	1906 (1102)	1285 (38)
	Pineville	8962 (781)	6245 (528)	4309 (364)	2904 (245)	1964 (164)	1319 (110)
PG 64-22	**Westgate**	11438 (442)	8795 (345)	6345 (269)	4541 (195)	3259 (148)	2307 (114)
	Pineville	11062 (416)	8490 (322)	6209 (320)	4300 (95)	3073 (75)	2167 (69)

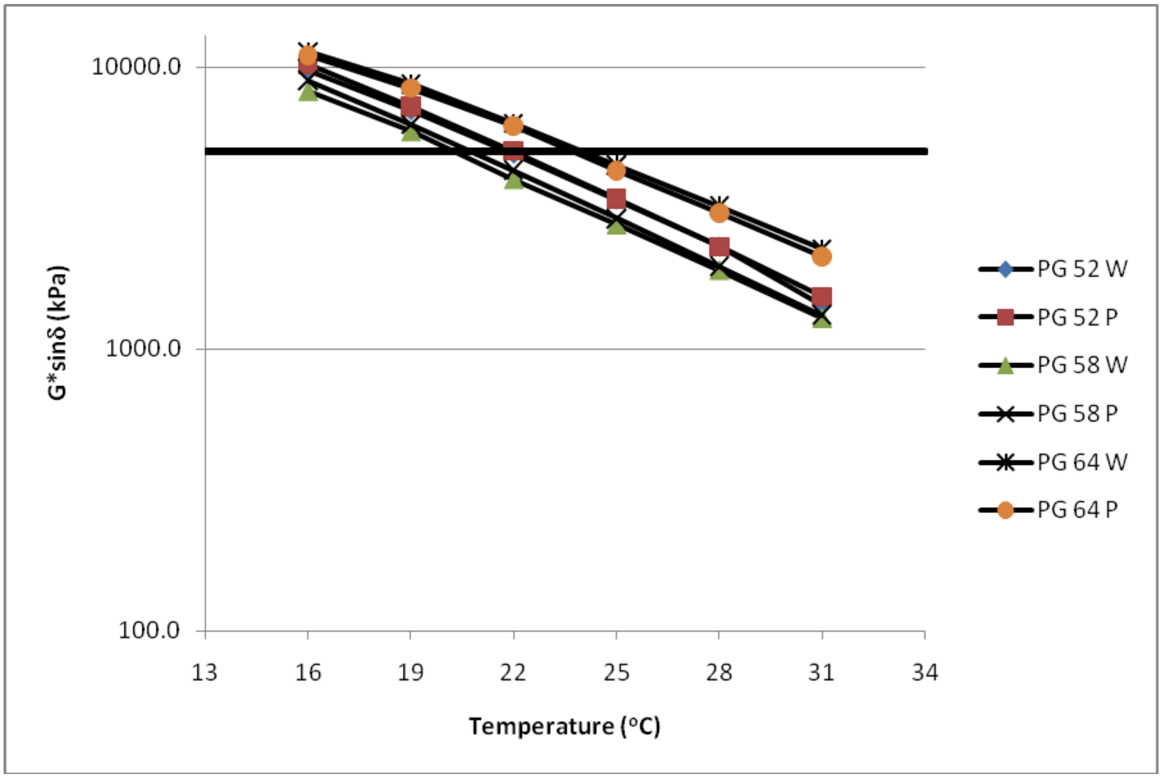

Figure 4.3 DSR Test Results for Blended PAV Aged Binders Containing 30% RAP

The results of Figure 4.3 depict how the addition of 30% RAP increases the stiffness of the binder grade. The performance grade of the binder can be determined by selecting the temperature that satisfies the G*sinδ ≤ 5000 kPa requirement. The PG 52 Westgate blend and PG 58 binders satisfy the requirement at 22°C while the PG 52 Pineville blend and PG 64 binders satisfy the requirement at 25°C. The PG 58 and PG 64 binders passing at these temperatures translate to a low binder grade of -22, the same as the virgin binder. However, the PG 52 Pineville blend and PG 52 Westgate blend pass at temperatures that translate to a lower binder grade of -10 and -16, respectively.

The above data, along with the rheological data from the virgin binders and the RAP binders discussed in the previous chapter were compiled together to form blending charts. Once the data was plotted, exponential trendlines were added for each temperature combination. The exponential equations will be used to determine the amount of RAP that can be used at certain temperatures and meet the specifications for the given test. Figures 4.4-4.12 below contain the blending charts for each combination of virgin binder. The figures can be used to determine the percentage of RAP necessary to meet the requirement of G*/sinδ ≥ 1.0 kPa for original binders, G*/sinδ ≥ 2.2 kPa for RTFO aged binders and G*sinδ ≤ 5000 kPa for PAV aged binders.

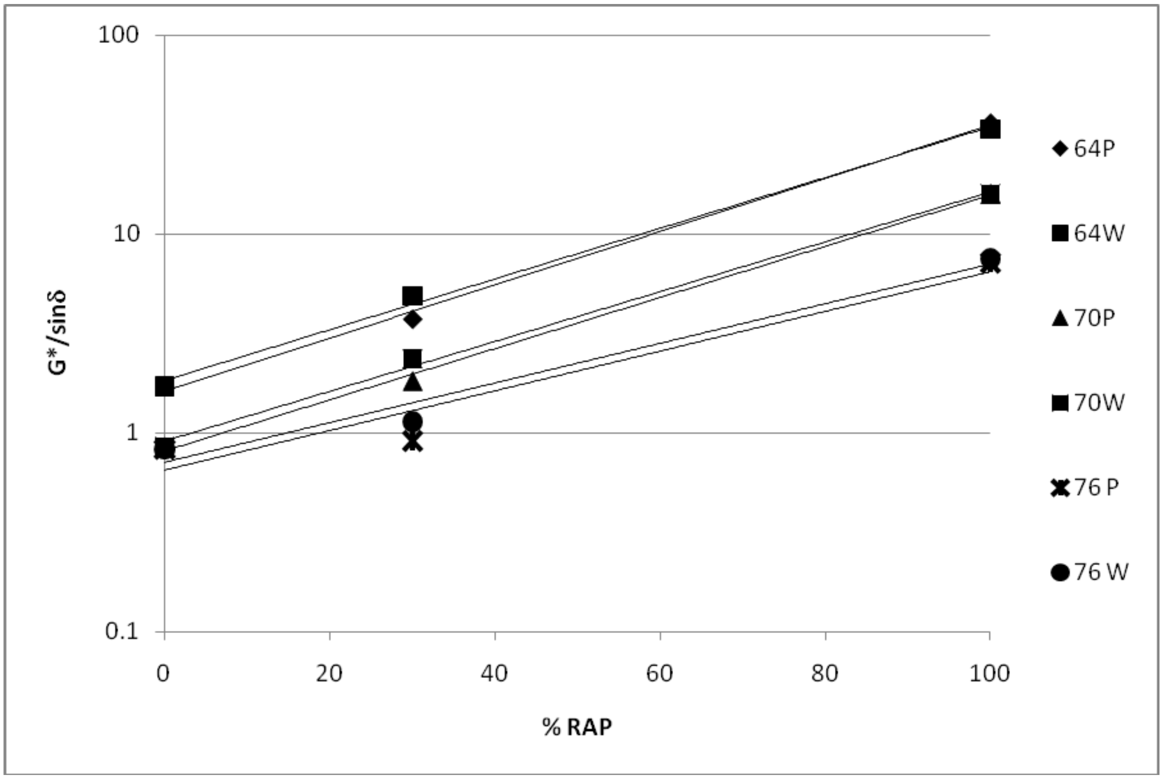

Figure 4.4 DSR Original PG 64-22 Blending Chart

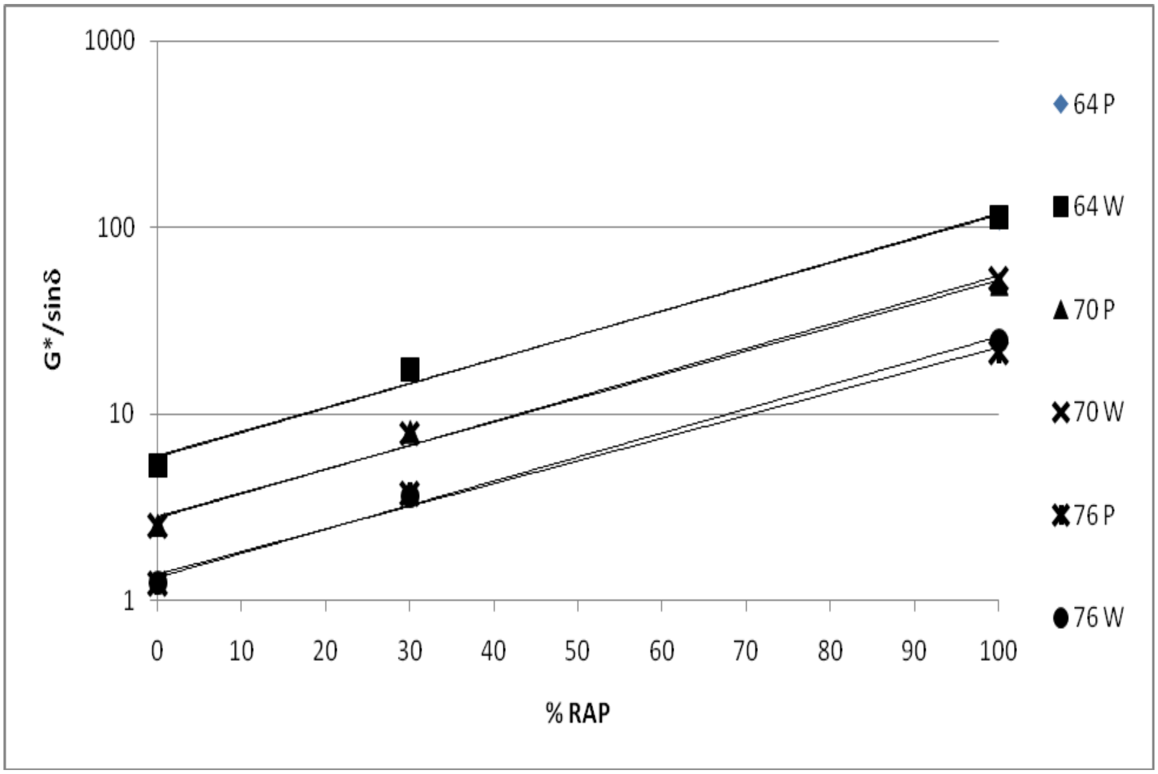

Figure 4.5 DSR RTFO PG 64-22 Blending Chart

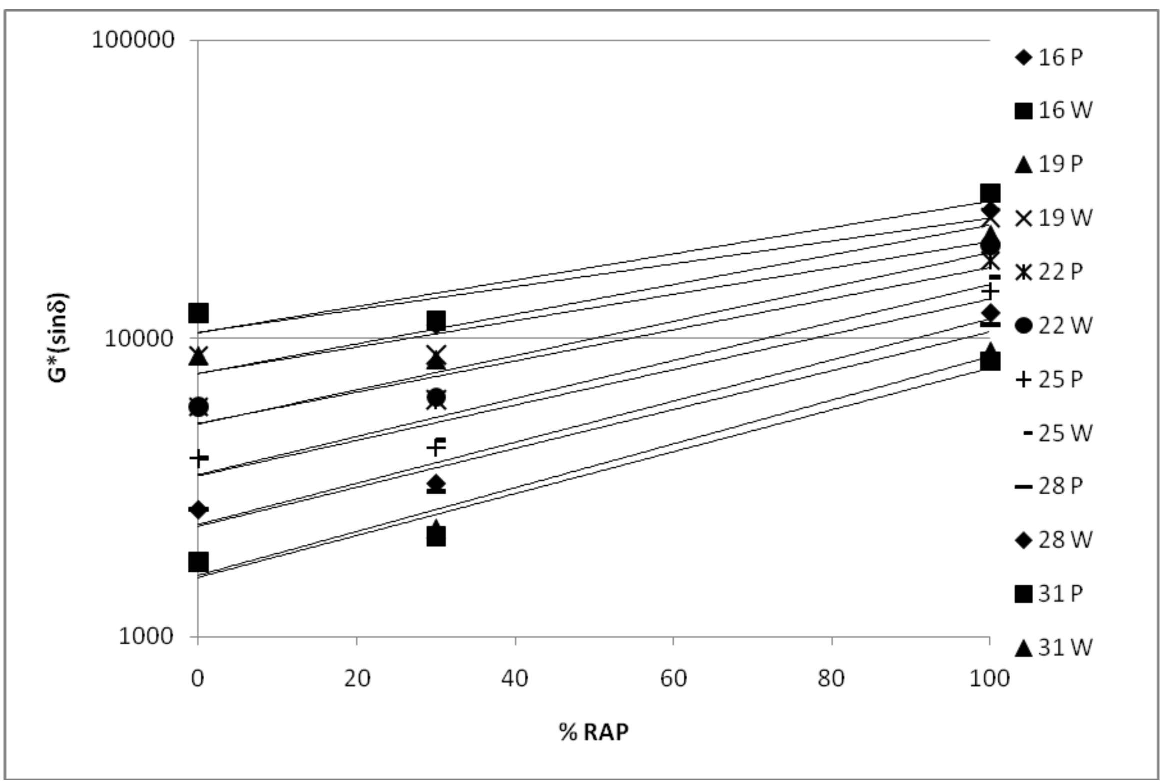

Figure 4.6 DSR PAV PG 64-22 Blending Chart

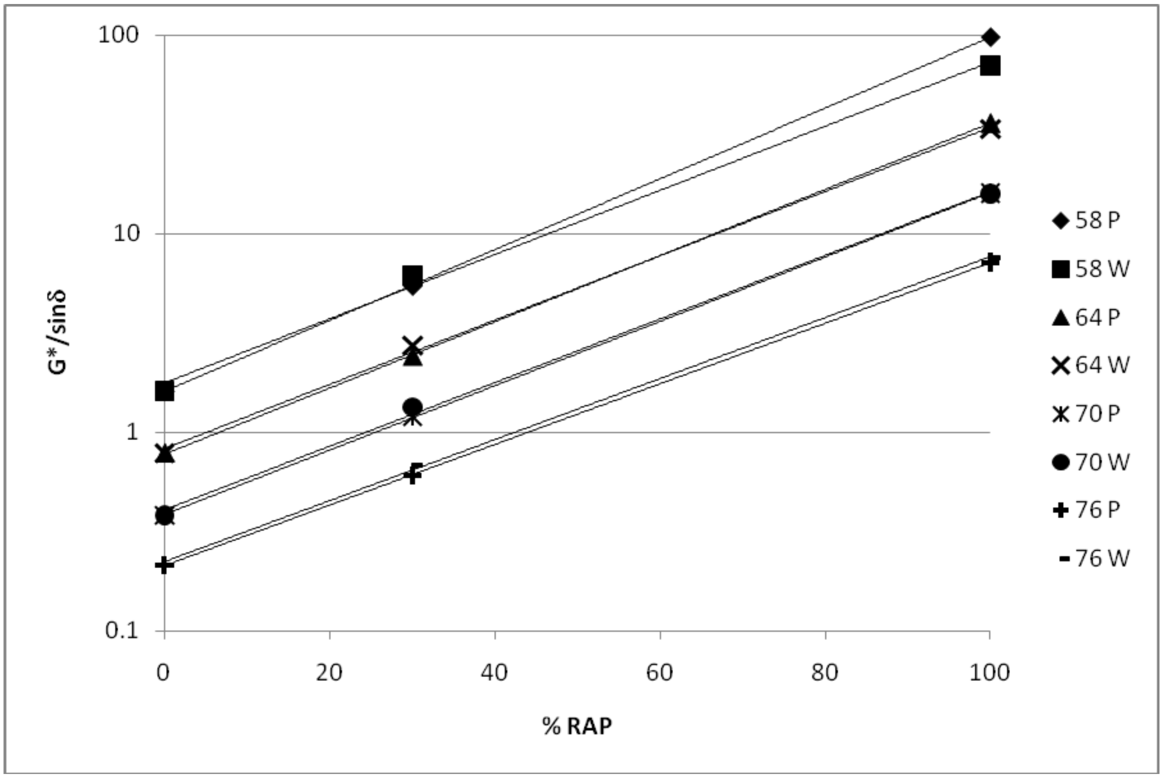

Figure 4.7 DSR Original PG 58-22 Blending Chart

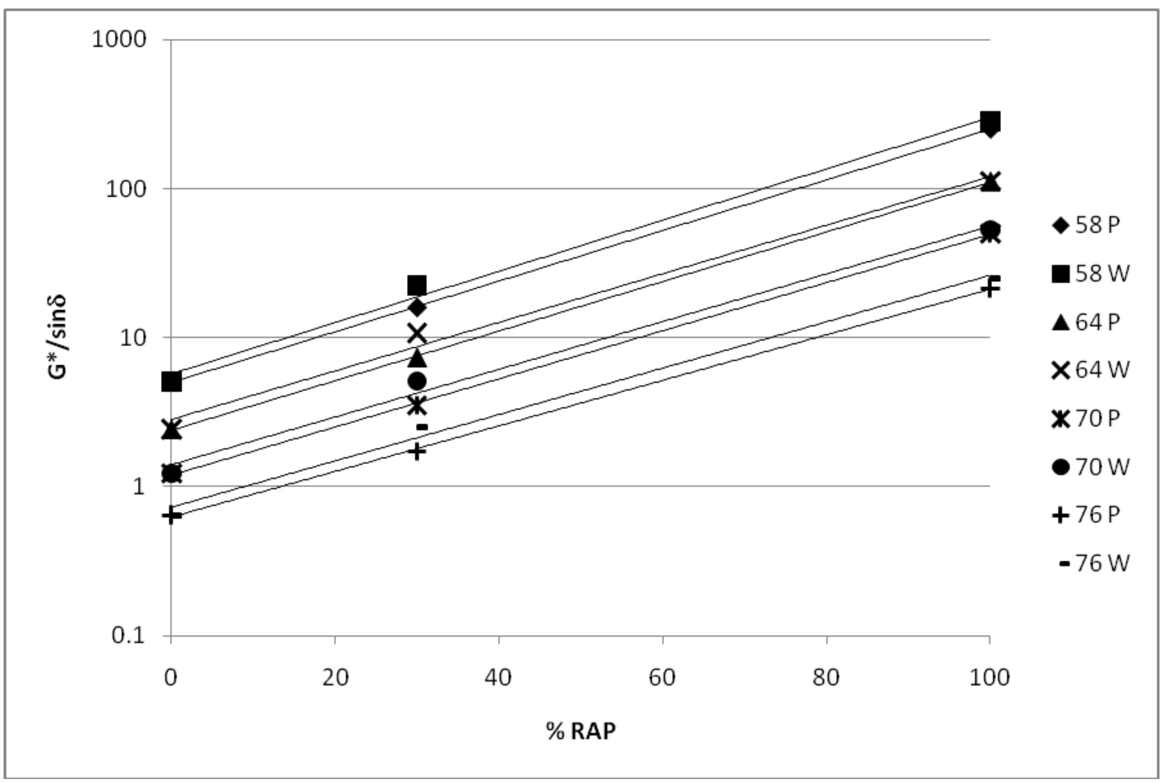

Figure 4.8 DSR RTFO PG 58-22 Blending Chart

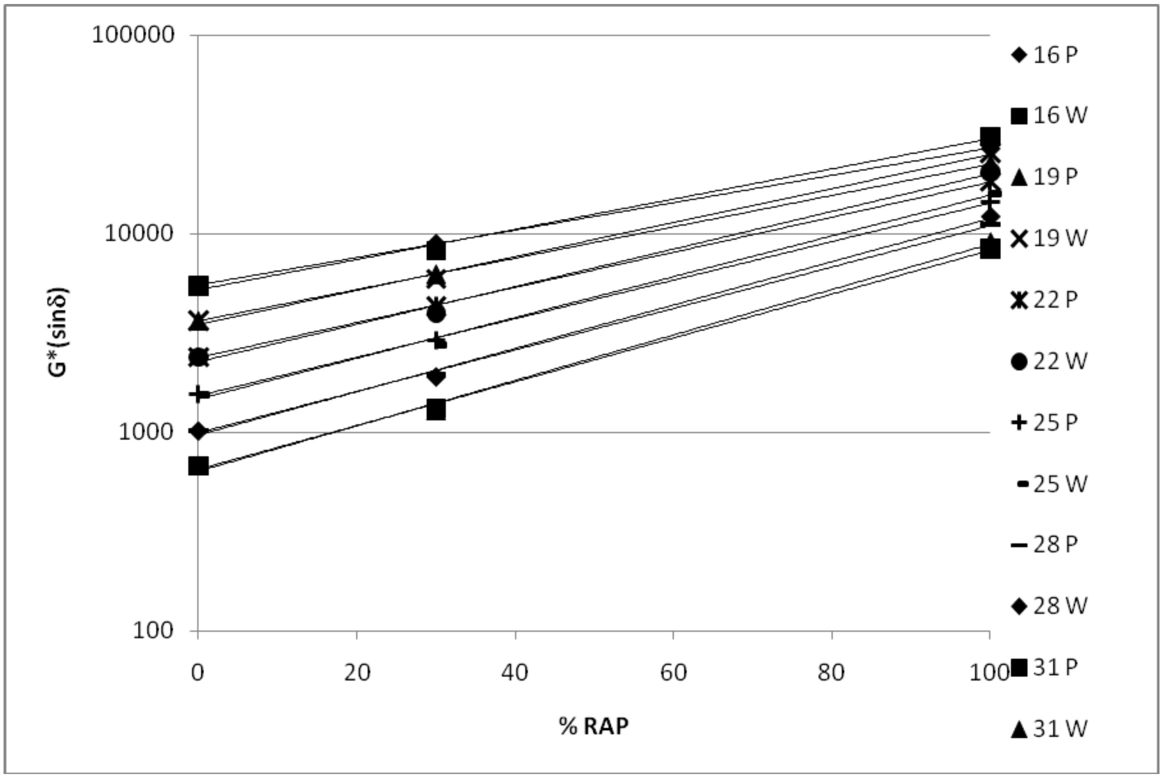

Figure 4.9 DSR PAV PG 58-22 Blending Chart

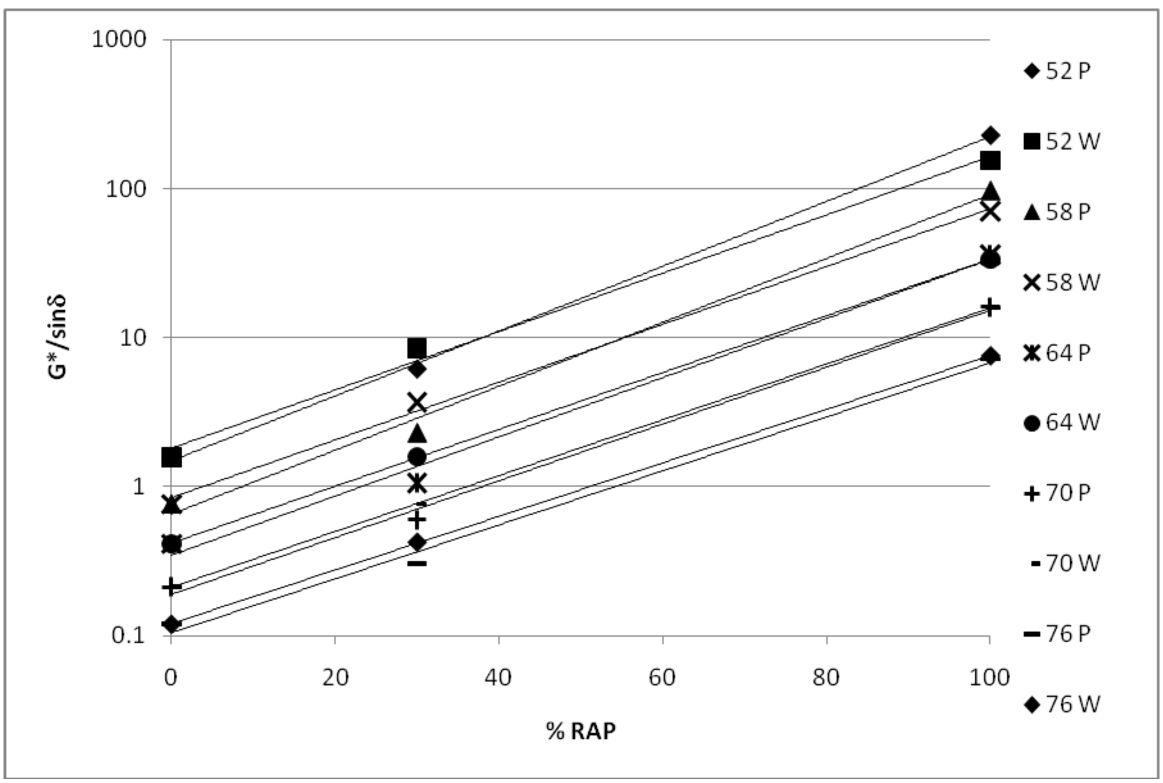

Figure 4.10 DSR Original PG 52-28 Blending Chart

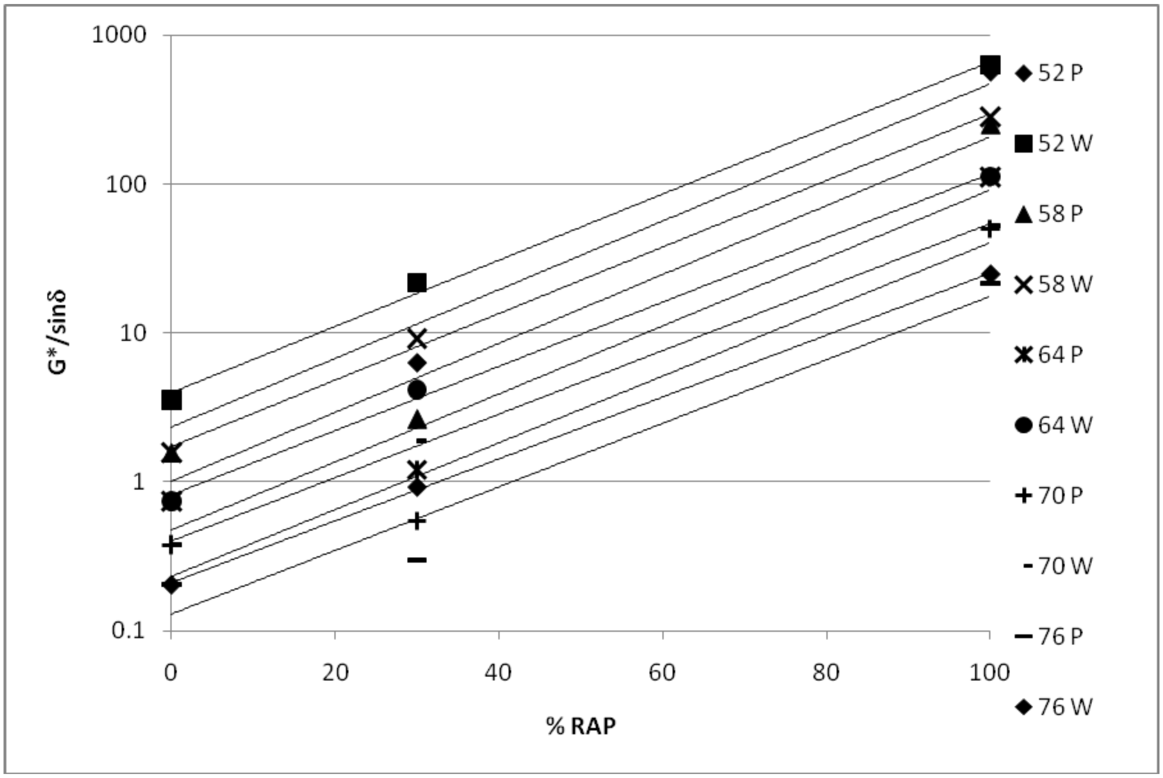

Figure 4.11 DSR RTFO PG 52-28 Blending Chart

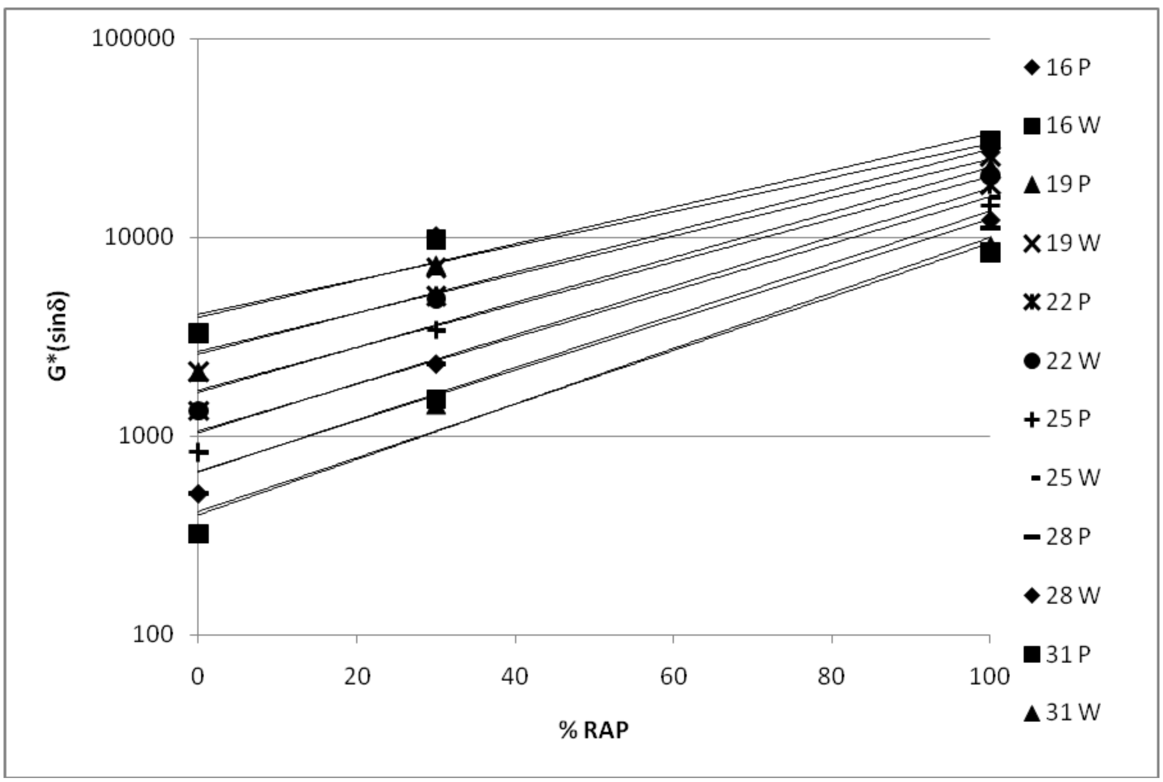

Figure 4.12 DSR PAV PG 52-28 Blending Chart

Table 4.4 and Table 4.5 below contain the minimum percentage of RAP needed to meet the specification of $G^*/\sin\delta = 1.0$ and $G^*/\sin\delta = 2.2$, respectively, based on the exponential equations from the blending charts at the different temperatures. For the most part, the binder from the Westgate RAP exhibits a higher $G^*/\sin\delta$. Therefore, blends containing binder from the Westgate RAP generally need less RAP in order to meet the specification than mixtures containing the softer Pineville RAP. For example, if a mixture using PG 52-28 virgin binder is blended with Pineville RAP, in order to meet the specification for the high temperature grade of PG 64 and unaged binder, there needs to be a minimum of 23.20 % RAP in the blend to meet the specification of $G^*/\sin\delta = 1.0$. Similarly, in order to meet the specification of $G^*/\sin\delta = 2.2$ for RTFO aged binder, there needs to be a minimum of 29.14% RAP for a high temperature grade of PG 64. Therefore, the minimum for the high temperature grade for this blend would be 29.14% RAP.

Table 4.4 Minimum Percentage of RAP Binder to Satisfy G*/sinδ= 1.0 for Unaged Binder

Virgin Binder Grade	RAP Source	Temperature °C	$G^*/\sin\delta = Ae^{B(\%RAP)}$		Minimum % RAP
			A	B	
PG 52-28	Pineville	52	-	-	0
		58	0.6536	0.0494	8.61
		64	0.3463	0.0457	23.20
		70	0.1891	0.0439	37.94
		76	0.1044	0.0417	54.19
	Westgate	52	-	-	0
		58	0.8418	0.0447	3.85
		64	0.4178	0.0439	19.88
		70	0.2111	0.0431	36.09
		76	0.1205	0.0414	51.11
PG 58-22	Pineville	58	-	-	0
		64	0.7745	0.0348	7.34
		70	0.3854	0.0373	25.56
		76	0.2128	0.0351	44.09
	Westgate	58	-	-	0
		64	0.8261	0.0373	5.12
		70	0.4065	0.0369	24.39
		76	0.2229	0.0354	42.40
PG 64-22	Pineville	64	-	-	0
		70	0.9064	0.0298	3.30
		76	0.7099	0.023	14.90
	Westgate	64	-	-	0
		70	0.8094	0.0296	7.14
		76	0.6479	0.023	18.87

Table 4.5 Minimum Percentage of RAP Binder to Satisfy $G^*/\sin\delta = 2.2$ for RTFO Binder

Virgin Binder Grade	RAP Source	Temperature °C	$G^*/\sin\delta = Ae^{B(\%RAP)}$		Minimum % RAP
			A	B	
PG 52-28	Pineville	52	-	-	0
		58	1.0098	0.0533	14.61
		64	0.475	0.0526	29.14
		70	0.232	0.0516	43.59
		76	0.1296	0.0491	57.67
	Westgate	52	-	-	0
		58	1.7238	0.0514	4.75
		64	0.8175	0.0497	19.92
		70	0.3998	0.0491	34.73
		76	0.2101	0.0479	49.03
PG 58-22	Pineville	58	-	-	0
		64	-	-	0
		70	1.1944	0.0372	16.42
		76	0.6251	0.0352	35.75
	Westgate	58	-	-	0
		64	-	-	0
		70	1.4024	0.0369	12.20
		76	0.7266	0.0358	30.95
PG 64-22	Pineville	64	-	-	0
		70	-	-	0
		76	1.3762	0.0279	16.81
	Westgate	64	-	-	0
		70	-	-	0
		76	1.3332	0.0296	16.92

Table 4.6 below contains the maximum percentage of RAP allowed in order to meet the specification $G^*\sin\delta = 5000$ kPa, based on the exponential equations from the blending charts. For the most part, the binder from the Westgate RAP exhibits a higher $G^*\sin\delta$. Therefore, blends containing the binder from the Westgate RAP generally are allowed less RAP in order to meet the specification than mixtures containing the softer Pineville RAP. For example, if a mixture using PG 52-28 virgin binder is blended with Pineville RAP, in order to meet the specification for the low temperature grade of PG 64-22 binder, there can be a maximum 28.28% RAP to meet the specification of $G^*\sin\delta = 5000$ kPa. For PG 52 binders, the temperature of 19°C corresponds to a low grade of -22.

Table 4.6 Maximum Percentage of RAP Binder to Satisfy G*sinδ= 5000 kPa for PAV Binder

Virgin Binder Grade	RAP Source	Temperature °C	$G^*/\sin\delta = Ae^{B(\%RAP)}$ A	B	Maximum % RAP
PG 52-28	Pineville	16	4102.0	0.0198	10.0
		19	2661.0	0.0223	28.28
		22	1699.9	0.0248	43.50
		25	1059.8	0.0271	57.25
		28	665.5	0.0292	69.06
		31	416.88	0.0311	--
	Westgate	16	3952.2	0.0213	11.04
		19	2581.7	0.0237	27.89
		22	1656.7	0.0261	42.32
		25	1040.3	0.0283	55.47
		28	657.91	0.0302	67.16
		31	400.62	0.0322	78.39
PG 58-22	Pineville	16	-	-	0
		19	3631.4	0.0182	17.57
		22	2370.4	0.0204	36.59
		25	1523.4	0.0224	53.06
		28	992.37	0.024	67.38
		31	649.79	0.0254	80.34
	Westgate	16	-	-	0
		19	3484.7	0.0197	18.33
		22	2258.0	0.0218	36.47
		25	1468.5	0.0236	51.92
		28	967.07	0.0251	65.46
		31	635.31	0.0263	78.44
PG 64-22	Pineville	16	-	-	0
		19	-	-	0
		22	-	-	0
		25	3457.5	0.0137	26.93
		28	2344.8	0.015	50.48
		31	1578.8	0.0162	71.16
	Westgate	16	-	-	0
		19	-	-	0
		22	-	-	0
		25	3491.7	0.0147	24.43
		28	2376.5	0.0159	46.78
		31	1605.6	0.0169	67.22

4.2 Bending Beam Rheometer Testing

Bending Beam Rheometer (BBR) testing was completed on the blended binders. The BBR testing produced rheological properties for the binders including creep stiffness (s) and the m-value (slope). Each binder was tested at -12^{o}C. The blended binders consisted of a mix of 70% virgin binder and 30% binder extracted from the RAP sources.

Table 4.7 below contains the average creep stiffness and m-values for the virgin binders and binders recovered from the RAP sources. Table 4.8 below contains the average creep stiffness and m-values for the blended binders. The addition of 30% RAP to the PG 52-28 virgin binder resulted in an increase in creep stiffness of 95.9% and 90.3% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 58-22 virgin binder resulted in an increase in creep stiffness of 53.7% and 59.9% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 64-22 virgin binder actually resulted in a decrease in creep stiffness of 7.3% and 0.3% for Westgate and Pineville blends, respectively. This result is not expected. As the virgin binder grade in the blend increases from PG 52-28 to PG 64-22, the increase in creep stiffness becomes less. This is expected since as the virgin binder creep stiffness increases with increasing binder grade, the higher creep stiffness from the reclaimed RAP binder has less of an effect on the binder blend.

The addition of 30% RAP to the PG 52-28 virgin binder resulted in a decrease in m-value of 21.4% and 20.3% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 58-22 virgin binder resulted in a decrease in m-value of 11.5% and 10.0% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 64-22 virgin binder actually resulted in a decrease in m-value of 10.5% and 9.0% for Westgate and Pineville blends, respectively. This reduction in m-value is expected since the binder reclaimed from the RAP is stiffer thus likely to need more time to recover, which is what the m-value signifies.

Table 4.7 Creep Stiffness and m-values of Virgin and RAP Binders

Binder	Creep Stiffness (Mpa)	m-value
PG 52-28 Virgin	72.5	0.453
PG 58-22 Virgin	80.7	0.4045
PG 64-22 Virgin	170.5	0.3485
Westgate RAP	263	0.253
Pineville RAP	322	0.234

Table 4.8 Creep Stiffness and m-values of Blended Binders

	RAP Source	Creep Stiffness (Mpa)	m-value
PG 52-28	**Westgate**	142	0.356
	Pineville	138	0.361
PG 58-22	**Westgate**	124	0.358
	Pineville	129	0.364
PG 64-22	**Westgate**	158	0.312
	Pineville	170	0.317

The values from Table 4.7 and Table 4.8 were combined in order to produce blending charts for the creep stiffness and m-values in order to determine the maximum allowable amount of RAP in a mixture to pass the BBR specifications to control for thermal cracking. Figures 4.13-4.18 show the blending charts for the creep stiffness and m-values at the various performance grades.

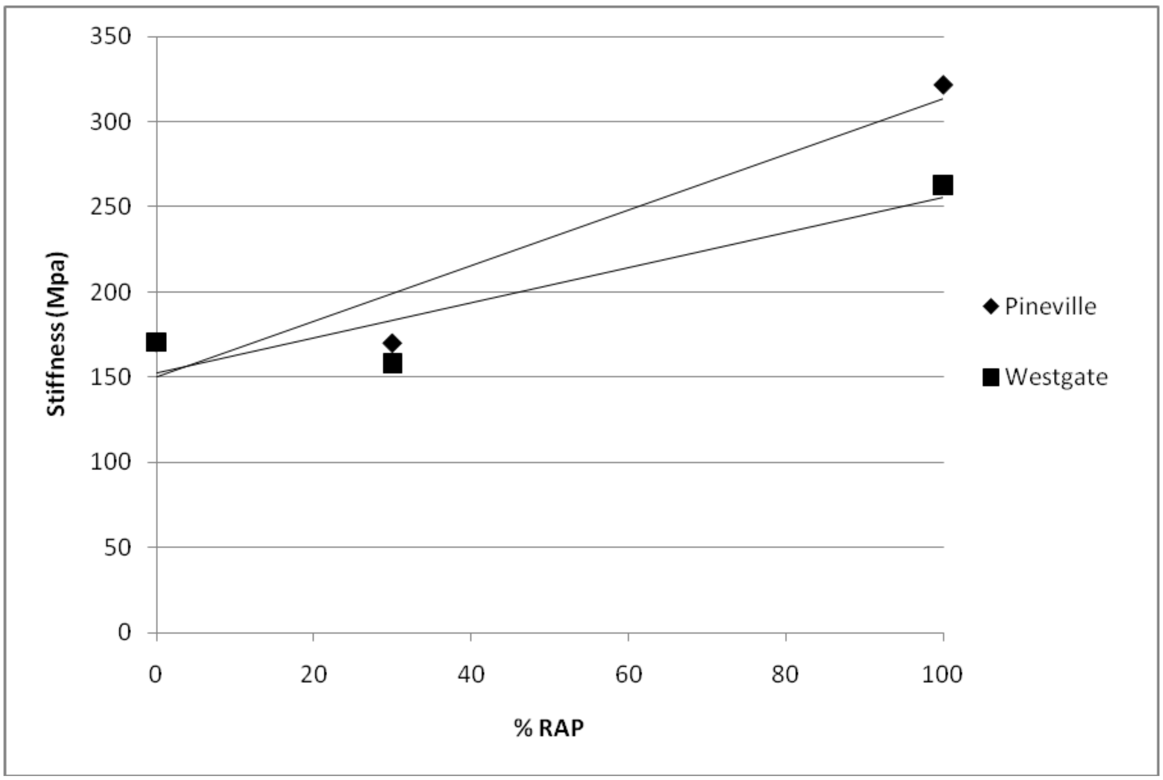

Figure 4.13 BBR PG 64-22 Creep Stiffness Blending Chart

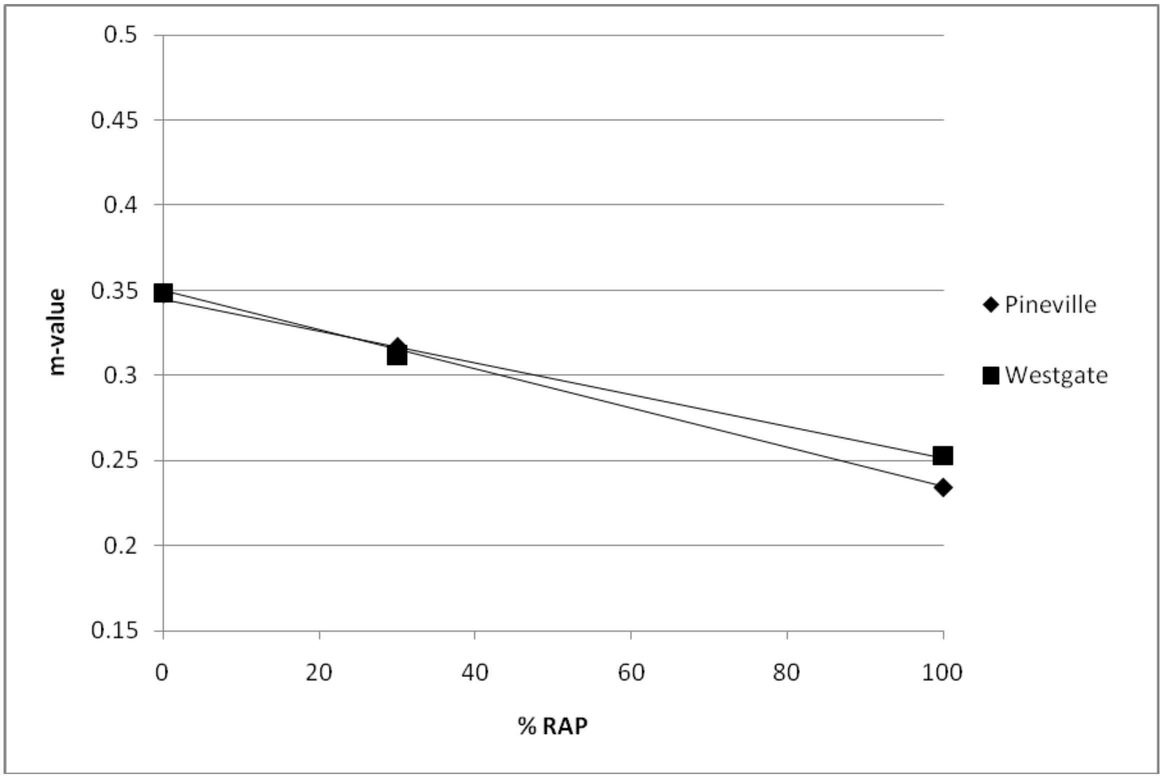

Figure 4.14 BBR PG 64-22 m-value Blending Chart

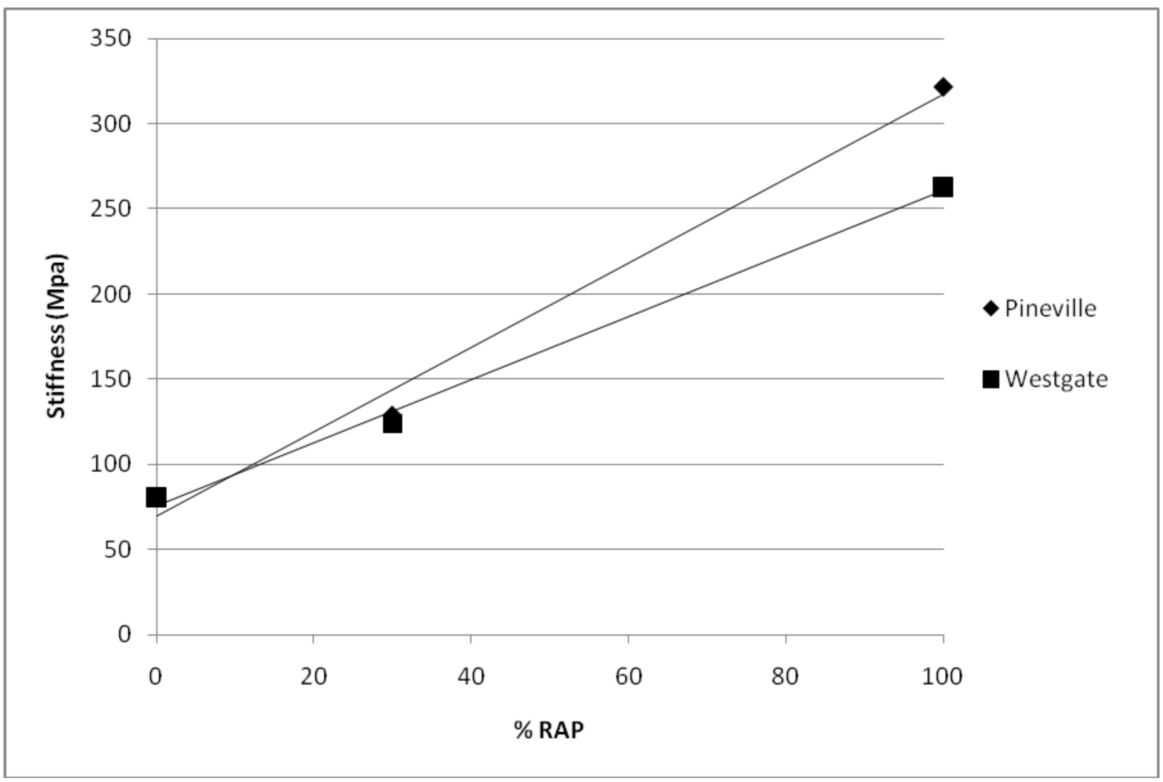

Figure 4.15 BBR PG 58-22 Creep Stiffness Blending Chart

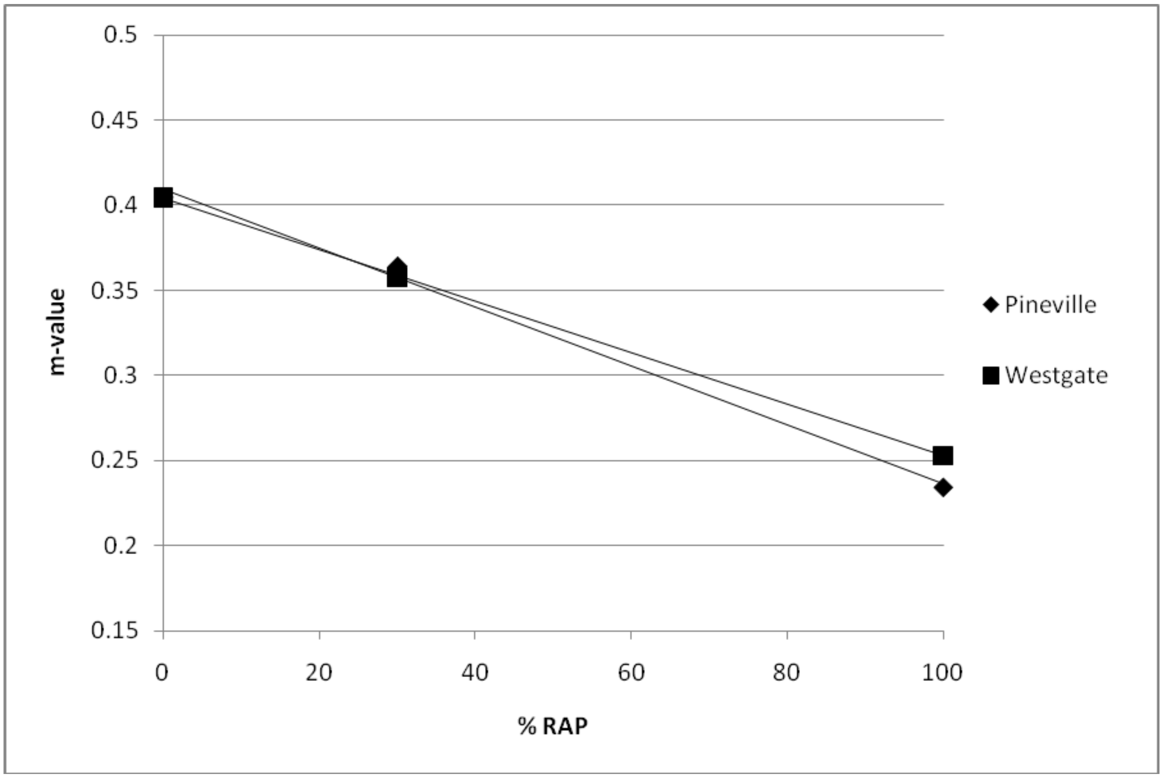

Figure 4.16 BBR PG 58-22 m-value Blending Chart

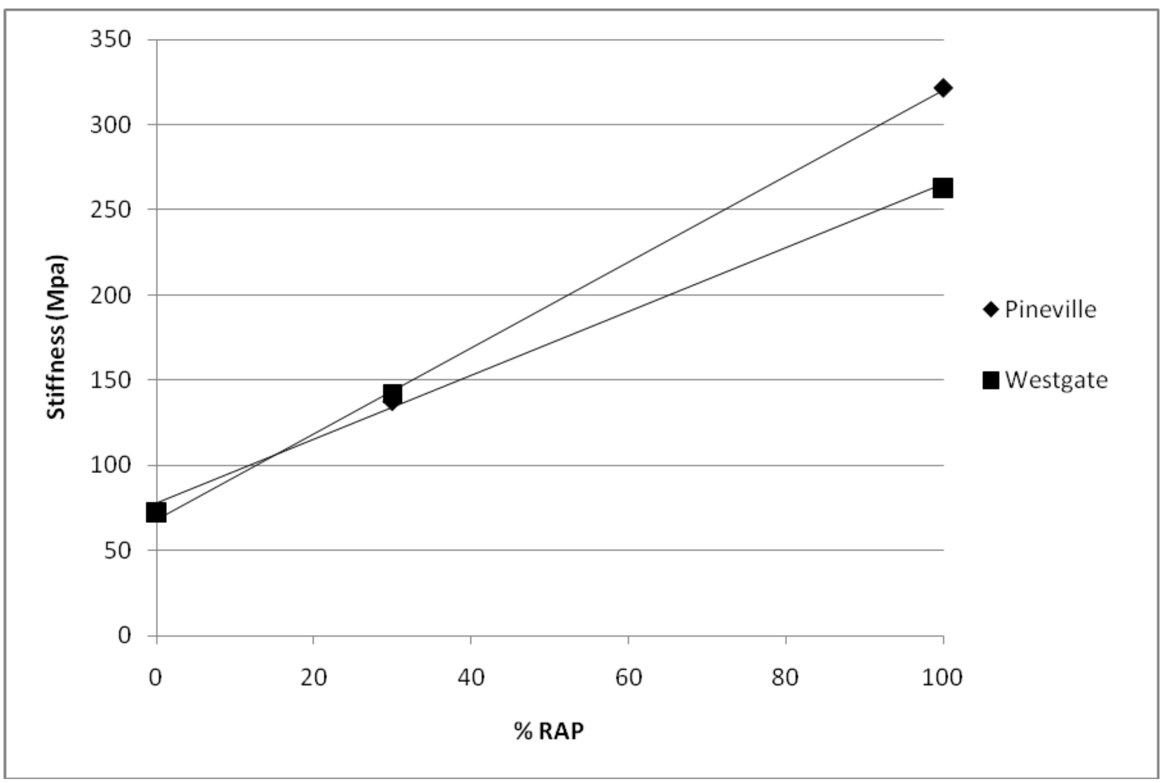

Figure 4.17 BBR PG 52-28 Creep Stiffness Blending Chart

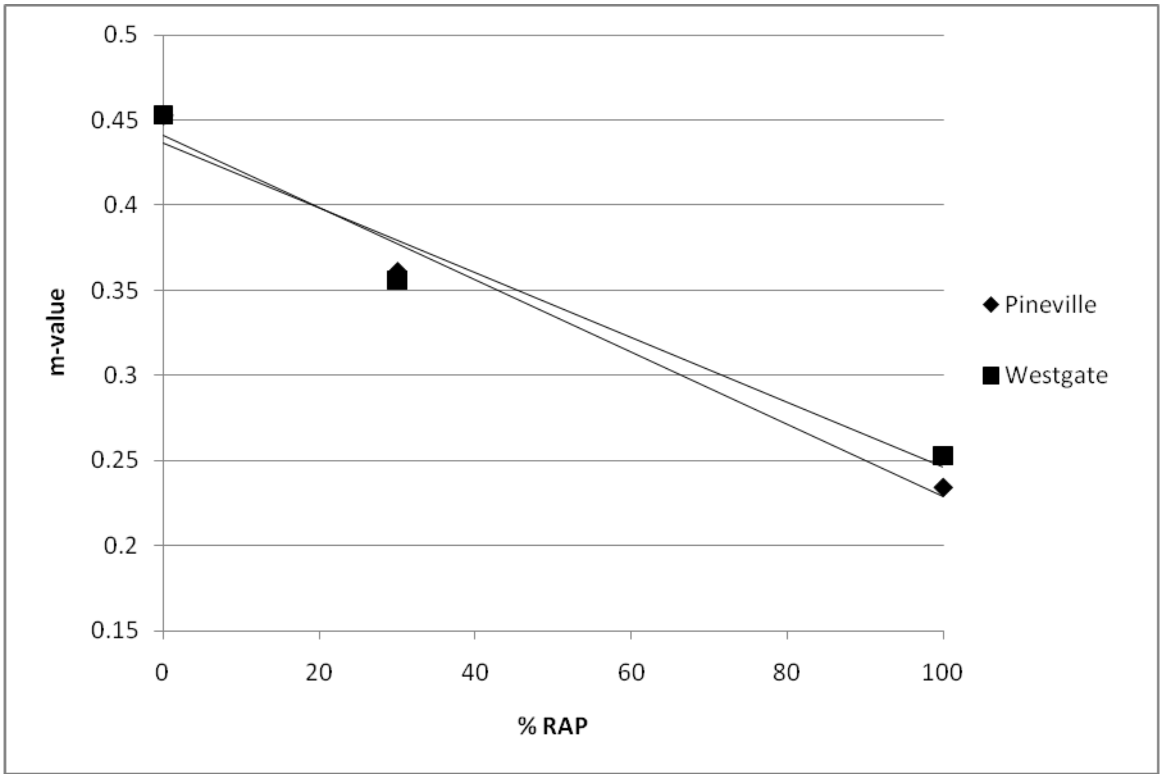

Figure 4.18 BBR PG 52-28 m-value Blending Chart

Table 4.9 and Table 4.10 below contains the maximum percentage of RAP allowed in order to meet the specification S=300 Mpa and m-value= 0.300, respectively, based on the linear equations from the blending charts. Table 4.9 shows that mixtures are able to satisfy the stiffness specification with 91%-100% RAP. However, the controlling variable for thermal cracking from the BBR test is the m-value. Mixtures with PG 64-22 binder are able to contain 41%-50% RAP. Mixtures with PG 58-22 binder are able to contain 64%-69% RAP. Mixtures with PG 52-28 binder are able to contain 67%-71% RAP.

Table 4.9 Maximum Percentage of RAP Binder to Satisfy S = 300 Mpa for PAV Binder

Virgin Binder Grade	RAP Source	S = m*x+b		Maximum % RAP
		m	b	
PG 52-28	Pineville	2.5187	68.358	91
	Westgate	1.8737	77.972	100
PG 58-22	Pineville	2.4740	70.027	92
	Westgate	1.8518	75.654	100
PG 64-22	Pineville	1.6313	150.14	91
	Westgate	1.0269	152.67	100

Table 4.10 Maximum Percentage of RAP Binder to Satisfy m-value= 0.300 for PAV Binder

Virgin Binder Grade	RAP Source	m-value = m*x+b		Maximum % RAP
		m	b	
PG 52-28	Pineville	-0.0021	0.4413	67
	Westgate	-0.0019	0.4366	71
PG 58-22	Pineville	-0.0017	0.4092	64
	Westgate	-0.0015	0.4040	69
PG 64-22	Pineville	-0.0012	0.3498	41
	Westgate	-0.0009	0.3450	50

Combining the results from Tables 4.4-4.6 and Table 4.9-4.10, the minimum and maximum amount of RAP is determined in order to still pass all specifications for a PG 64-22 binder. Table 4.11 below contains these values. Using virgin PG 64-22 binder, the maximum amount of RAP is 24%-27%. Using PG 58-22 binder, the maximum amount of RAP is 37%. Using PG 52-28 binder, the maximum amount of RAP is 28%. However, for Pineville RAP, the minimum amount of RAP needed is 30%. As the properties of RAP are variable depending on previous use, mixture etc., it is not recommended to use PG 52-28 binder in RAP mixtures in order to meet PG 64-22 specifications.

Table 4.11 Minimum and Maximum Percentage of RAP Binder to Satisfy all PG 64-22 Specifications

Virgin Binder Grade	RAP Source	Minimum Original DSR	Minimum RTFO DSR	Maximum PAV DSR	Maximum BBR S	Maximum BBR m-value
PG 52-28	Pineville	24	30	28	91	67
	Westgate	20	20	28	100	71
PG 58-22	Pineville	8	-	37	92	64
	Westgate	6	-	37	100	69
PG 64-22	Pineville	-	-	27	91	41
	Westgate	-	-	24	100	50

This chapter has discussed the rheology of the blended RAP binders and the formation of blending charts. The blending charts will allow contractors to determine the amount of RAP that is acceptable to use in mixtures. The values formulated in this chapter will be combined with the values formulated in Chapter 5 regarding the pavement performance in order to determine the optimum amount of RAP to use in mixtures.

Chapter 5

Mixture Characterization

The mixtures developed and fabricated in Chapter 3 will be characterized in this chapter using Superpave simple shear tests. These characterizations will be used for comparisons between the various mixtures. The results will be used in the next chapter as part of the performance predictions for the mixtures.

The Superpave Simple Shear Tester (SST) was used to conduct both Frequency Sweep Test at Constant Height (FSTCH) and Repeated Simple Shear Test at Constant Height (RSSTCH).

The FSTCH applies a sinusoidal shear strain of 0.01% at frequencies of 10, 5, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. In accordance with AASHTO TP7, the FSTCH were performed at 20^{o}C. Resulting from the FSTCH, the complex modulus (G*) is known. The complex modulus measures the mixtures stiffness for the range of frequencies. These values were used to compare the similarities between mixtures containing 100% virgin materials and mixtures containing various percentages of RAP. As a part of the performance analysis, the complex modulus was used in the SHRP A-003A surrogate performance prediction model for fatigue life in Chapter 6.

The RSSTCH is used to measure the accumulations of permanent strain in the test specimen for the test period. The shear load is applied in a haversine pulse for a duration of 0.1 second followed by a 0.6 second unload period. This results in a total loading cycle of 0.7 second. The RSSTCH is conducted for 5000 cycles or 5 percent of the permanent shear strain for the specimen is reached. The RSSTCH measures the accumulated permanent shear strain for each specimen. The RSSTCH were performed at 59.3 ^{o}C as this is the average seven day high temperature with 98% reliability for Raleigh, North Carolina. As the accumulated permanent shear strain correlates to rut resistance, the values obtained from mixtures containing 100% virgin materials were compared with values obtained from mixtures containing various amounts of RAP material.

The SST machine is depicted in Figure 5.1. Figure 5.2 below shows the specimen setup in the molds with the Linear Variable Differential Transformer (LVDT).

Figure 5.1 Superpave Simple Shear Tester (SST)

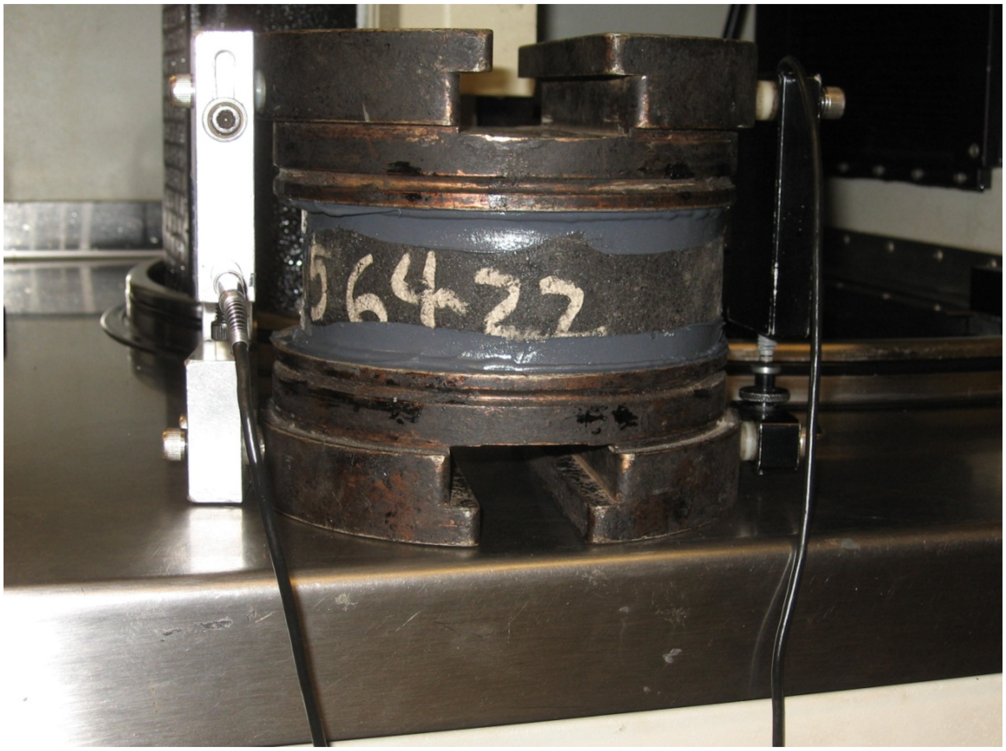

Figure 5.2 Simple Shear (FSTCH and RSTCH) Test Specimen

Using the SST, testing for FSTCH and RSSTCH was completed for the virgin 9.5mm and 19.0 mm specimens with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder. Figures 5.3-5.6 below depict comparisons among mixtures containing the three various binder grades for both 9.5 mm mixtures and 19.0 mm mixtures for FSTCH and RSSTCH, respectively.

The results from the FSTCH indicate trends of increasing complex modulus with increasing frequency. Also, as the binder grade increases, from PG 52-28 to PG 64-22, the complex modulus increases. This would be expected as the complex modulus of PG 64-22 binder is higher than the complex modulus of PG 52-28 binder when temperature is constant.

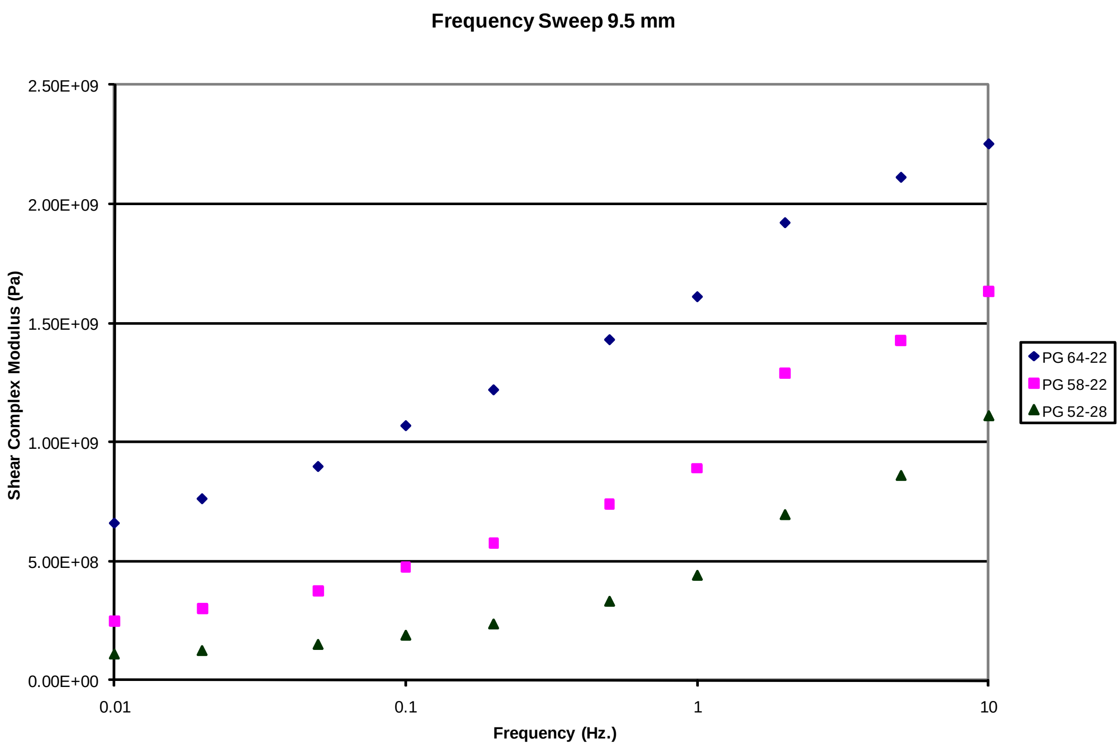

Figure 5.3 9.5 mm Virgin Frequency Sweep Comparison

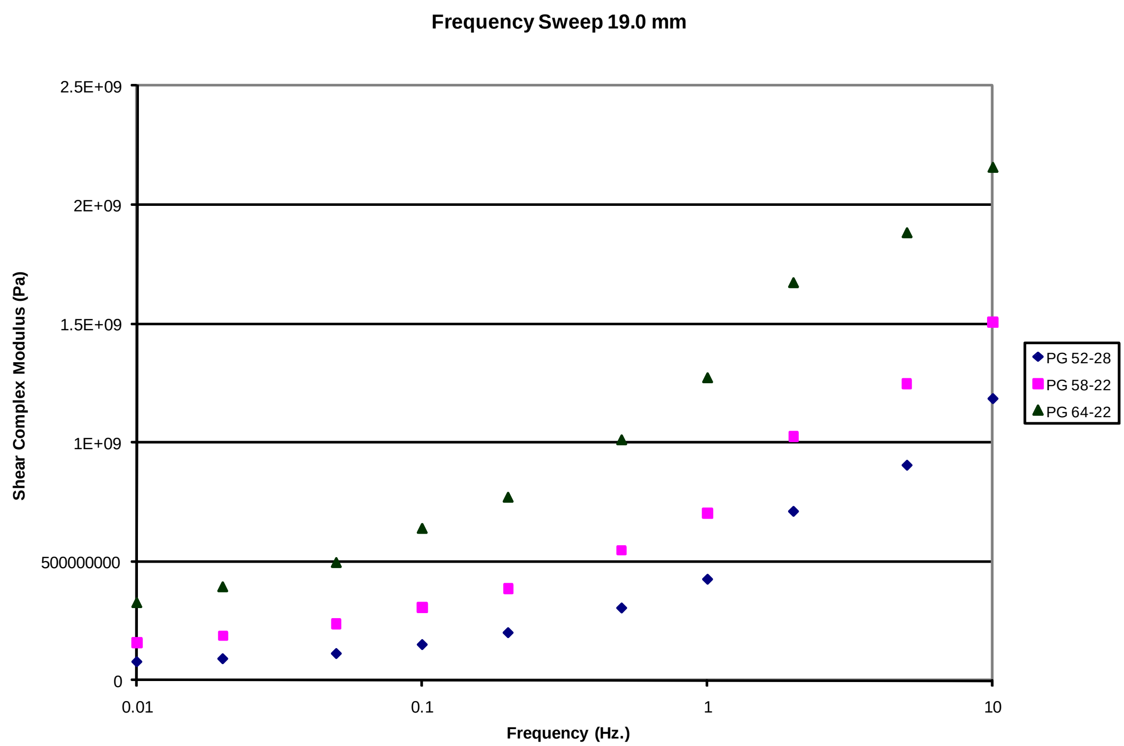

Figure 5.4 19.0 mm Virgin Frequency Sweep Comparison

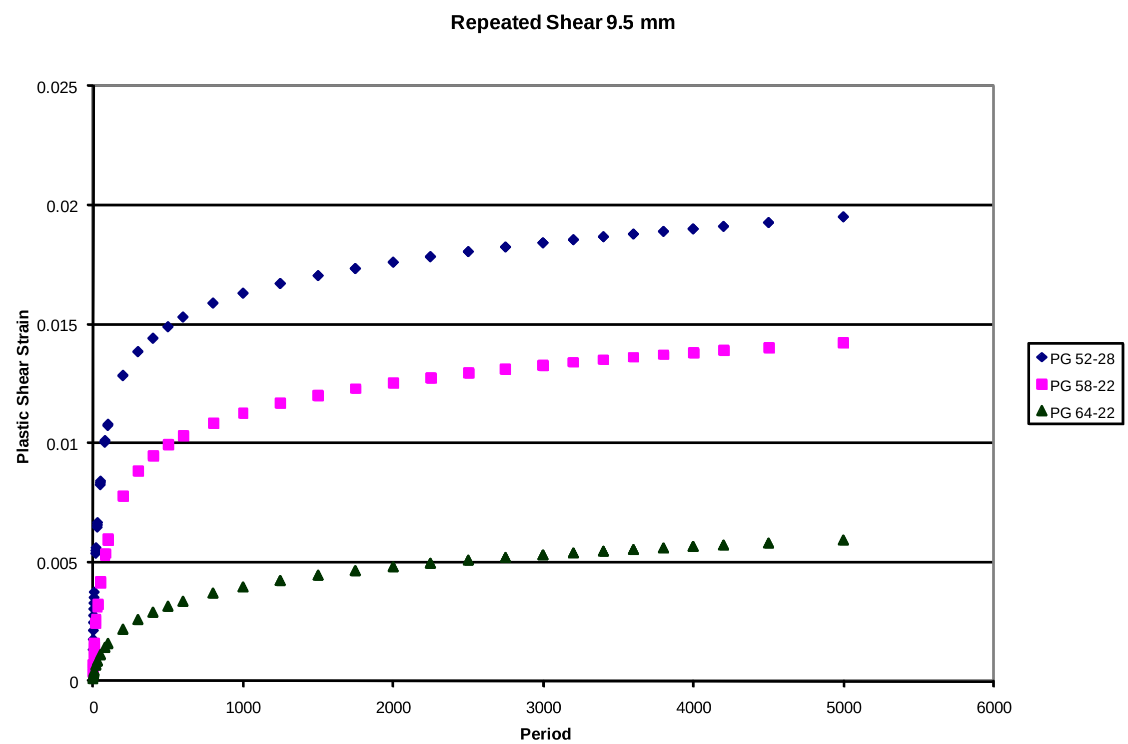

Figure 5.5 9.5 mm Virgin Repeated Shear Comparison

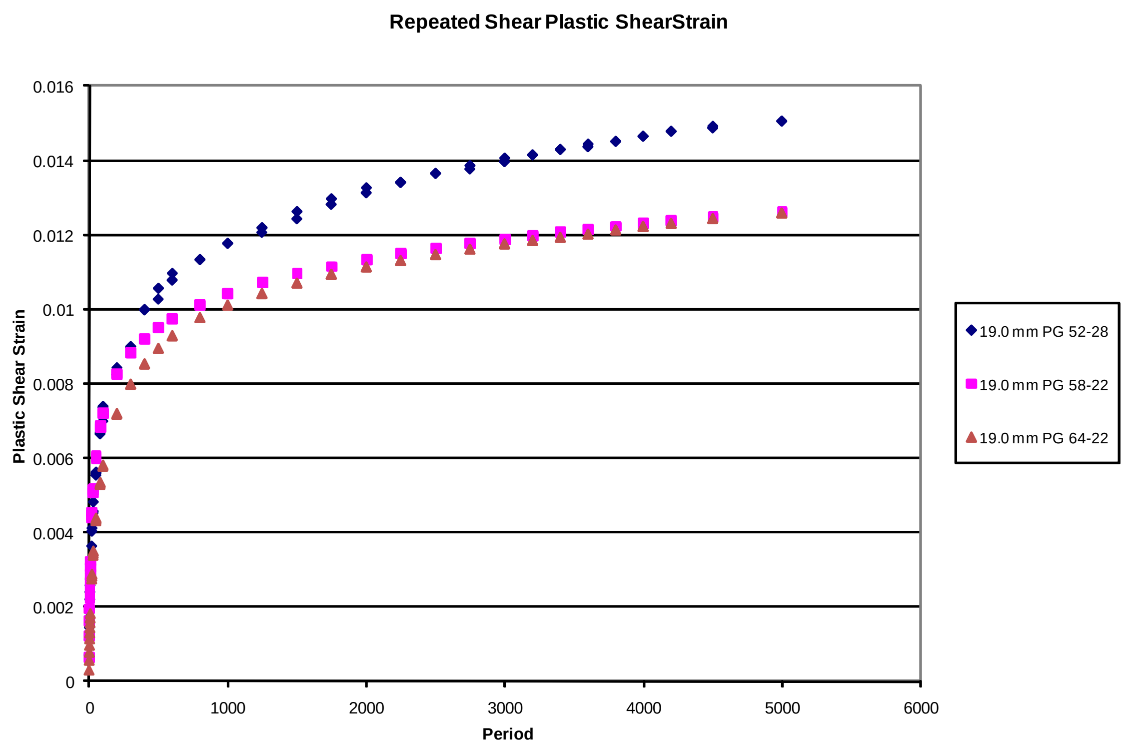

Figure 5.6 19.0 mm Virgin Repeated Shear Comparison

After 5000 cycles of repeated shear, the mixtures containing PG 64-22 binder exhibited the least amount of strain for both virgin 9.5mm mixtures and 19.0 mm mixtures. Mixtures with PG 58-22 binder exhibited less strain than the mixtures with PG 52-28 binder. This can be expected as mixtures with stiffer binders exhibited less strain.

5.1 Frequency Sweep Analysis

Using the SST, testing for FSTCH was completed for the 9.5mm specimens at the various RAP percentages (15%, 30%, and 40%) with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder and both RAP sources (Westgate and Pineville). Figures 5.7-5.12 below depict comparisons among mixtures containing the three various binder grades, various percentages of RAP and both RAP sources for 9.5 mm mixtures for FSTCH.

The results from the FSTCH indicate trends of increasing complex modulus with increasing frequency. Also, as the binder grade increases, from PG 52-28 to PG 64-22, the complex modulus increases. This trend holds true at all percentages of RAP. This would be expected as the complex modulus of PG 64-22 binder is larger than the complex modulus of PG 52-28 binder when temperature is constant.

Table 5.1 Comparison of Complex Modulus for 9.5 mm Westgate Mixtures

		Complex Modulus (Pa)	
Binder Grade	**RAP Percentage**	**0.01 Hz**	**10 Hz**
PG 64-22	15%	4.56E+08	2.15E+09
	30%	5.26E+08	2.21E+09
	40%	7.45E+08	2.60E+09
PG 58-22	15%	2.30E+08	1.48E+09
	30%	2.97E+08	1.66E+09
	40%	3.39E+08	1.89E+09
PG 52-28	15%	1.20E+08	1.29E+09
	30%	1.30E+08	1.29E+09
	40%	1.55E+08	1.48E+09

Table 5.2 Comparison of Complex Modulus for 9.5 mm Pineville Mixtures

		Complex Modulus (Pa)	
Binder Grade	**RAP Percentage**	**0.01 Hz**	**10 Hz**
PG 64-22	15%	3.71E+08	2.07E+09
	30%	4.06E+08	2.13E+09
	40%	4.47E+09	2.17E+09
PG 58-22	15%	2.01E+08	1.45E+09
	30%	2.70E+08	1.56E+09
	40%	3.26E+08	1.56E+09
PG 52-28	15%	9.76E+07	1.14E+09
	30%	1.91E+08	1.41E+09
	40%	2.11E+08	1.53E+09

The complex modulus of mixtures with higher percentages of RAP increases while holding the binder grade constant. Again, this can be expected since the complex modulus of the RAP binder is greater due to aging caused during its service life.

Tables 5.1 and 5.2 above contain the complex modulus at 0.01 Hertz and 10 Hertz for each binder grade and the three percentages of RAP for Westgate RAP and Pineville RAP, respectively. The stiffness at 10 Hertz will be used in later analysis when computing the fatigue life.

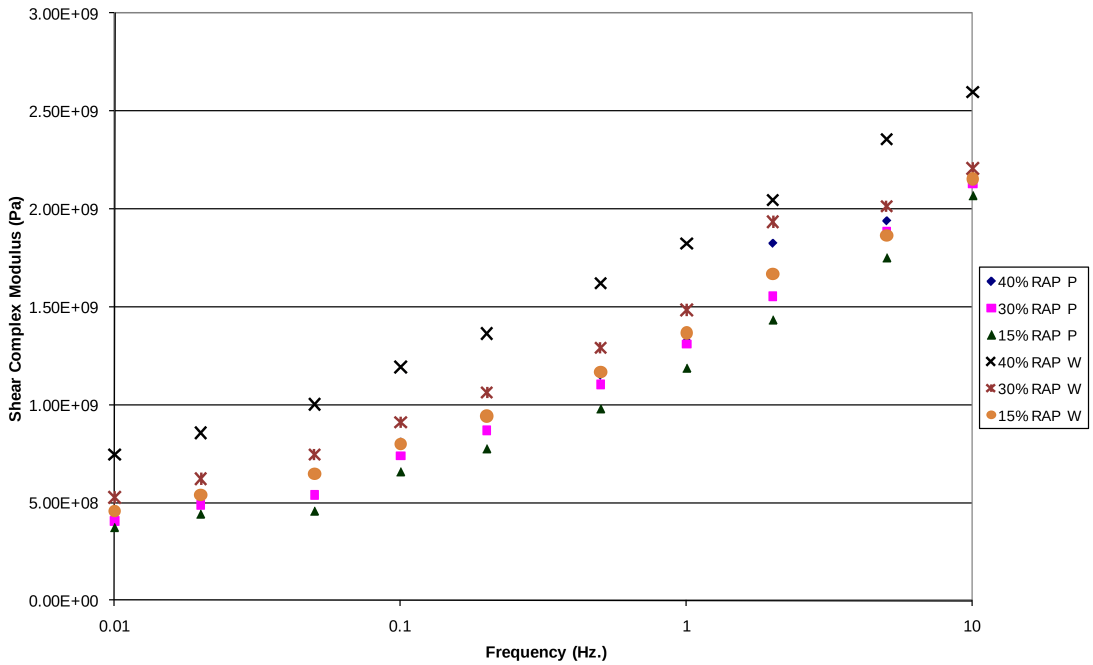

Figure 5.7 9.5 mm Frequency Sweep Comparison for PG 64-22

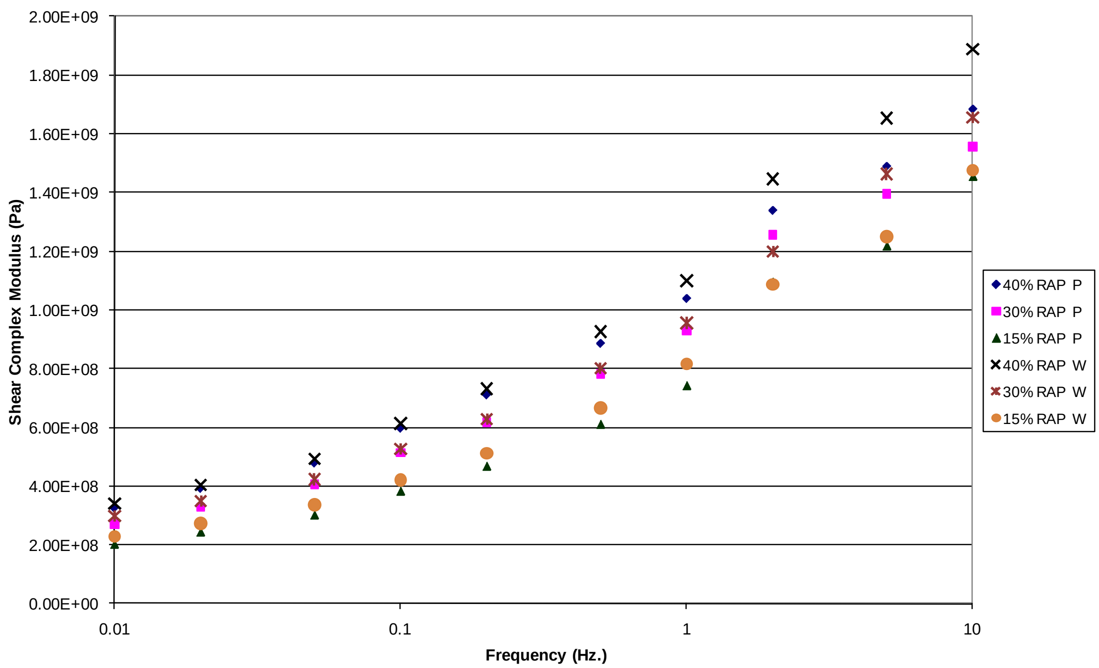

Figure 5.8 9.5 mm Frequency Sweep Comparison for PG 58-22

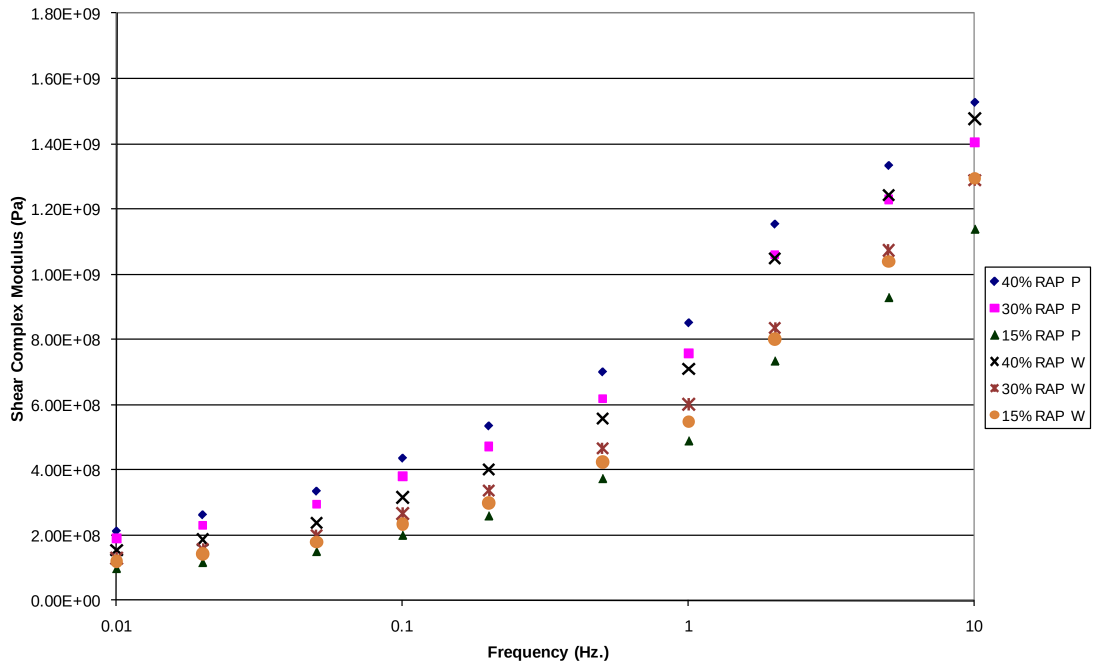

Figure 5.9 9.5 mm Frequency Sweep Comparison for PG 52-28

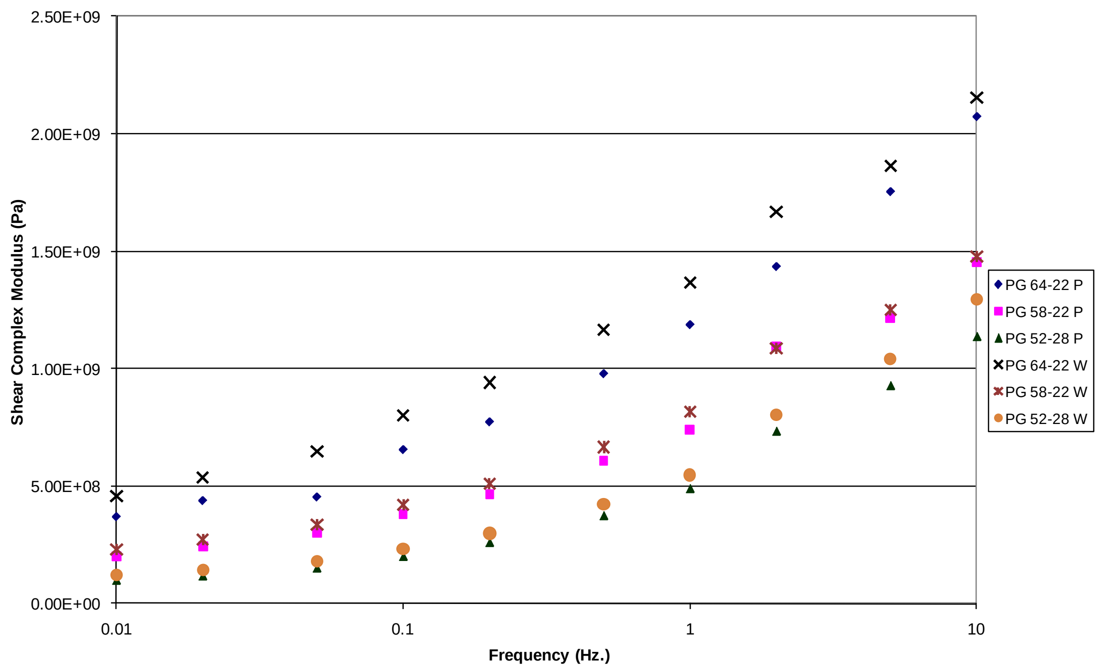

Figure 5.10 9.5 mm Frequency Sweep Comparison with 15% RAP

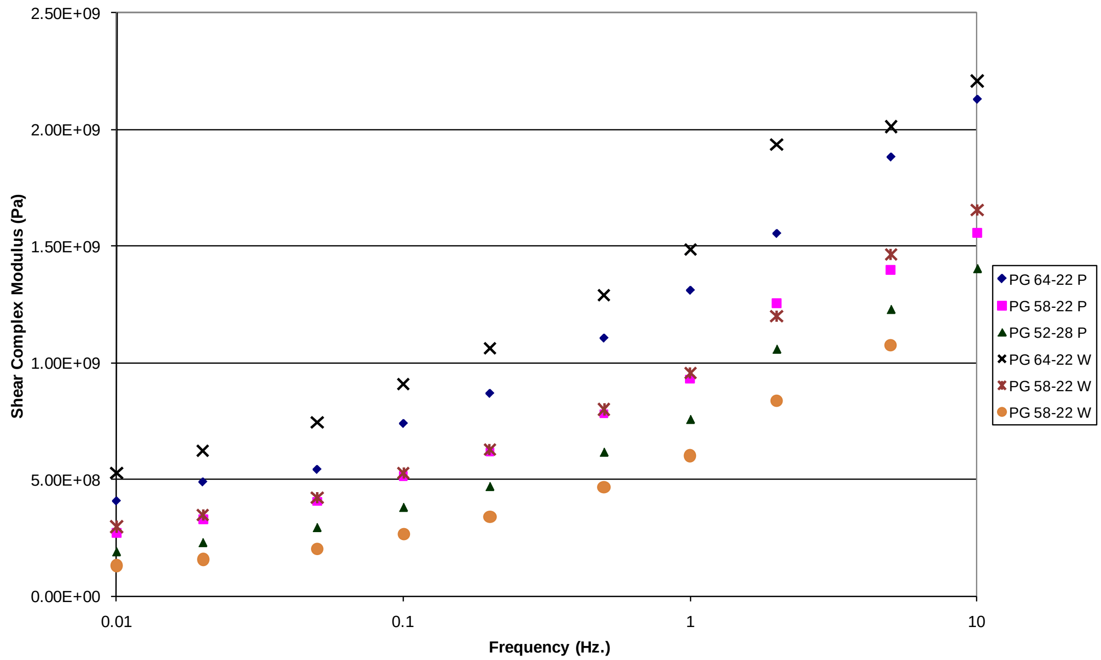

Figure 5.11 9.5 mm Frequency Sweep Comparison with 30% RAP

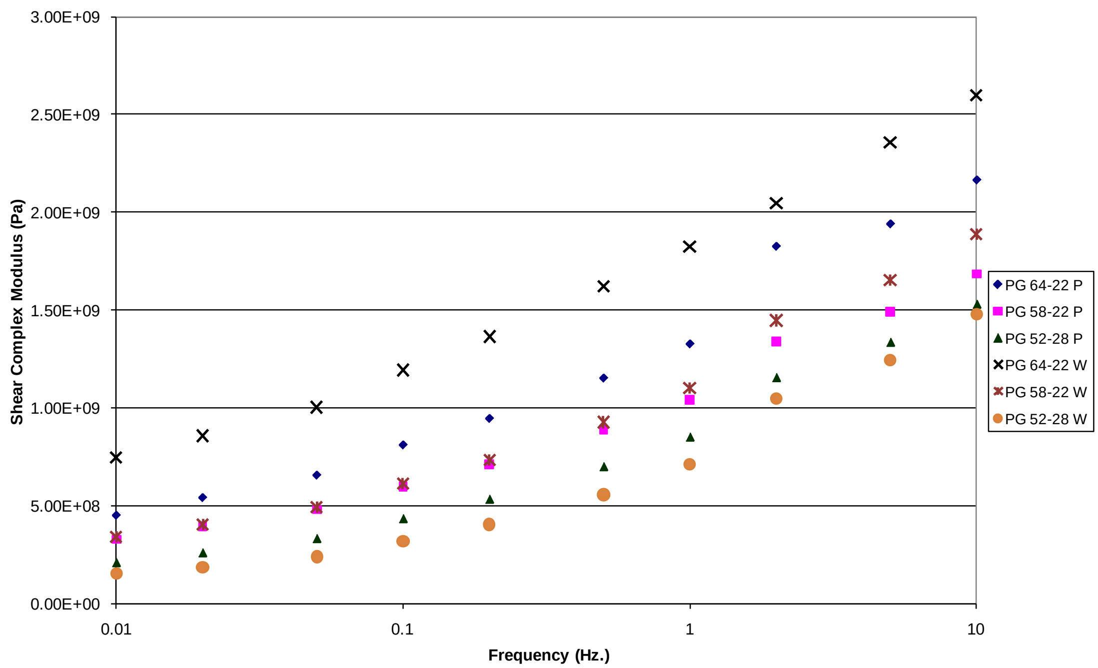

Figure 5.12 9.5 mm Frequency Sweep Comparison with 40% RAP

Using the SST, testing for FSTCH was completed for the 19.0 mm specimens at the various RAP percentages (15%, 30%, and 40%) with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder and both RAP sources (Westgate and Pineville). Figures 5.13-5.18 below depict comparisons among mixtures containing the three various binder grades, various percentages of RAP and both RAP sources for 19.0 mm mixtures for FSTCH.

Table 5.3 Comparison of Complex Modulus for 19.0 mm Westgate Mixtures

		Complex Modulus (Pa)	
Binder Grade	**RAP Percentage**	**0.01 Hz**	**10 Hz**
PG 64-22	15%	2.81E+08	2.24E+09
	30%	3.37E+08	2.87E+09
	40%	3.70E+08	4.17E+09
PG 58-22	15%	1.41E+08	1.64E+09
	30%	2.04E+08	1.80E+09
	40%	2.50E+08	2.42E+09
PG 52-28	15%	1.01E+08	1.35E+09
	30%	1.01E+08	1.47E+09
	40%	1.38E+08	1.81E+09

Table 5.4 Comparison of Complex Modulus for 19.0 mm Pineville Mixtures

		Complex Modulus (Pa)	
Binder Grade	**RAP Percentage**	**0.01 Hz**	**10 Hz**
PG 64-22	15%	2.71E+08	2.05E+09
	30%	3.01E+08	2.69E+09
	40%	3.72E+08	4.02E+09
PG 58-22	15%	1.24E+08	1.57E+09
	30%	1.87E+08	1.77E+09
	40%	2.49E+08	2.18E+09
PG 52-28	15%	9.21E+07	1.34E+09
	30%	1.18E+08	1.56E+09
	40%	1.58E+08	1.74E+09

The complex modulus of mixtures with higher percentages of RAP increases while holding the binder grade constant. Again, this can be expected since the complex modulus of the RAP binder is greater due to aging caused during its service life.

Tables 5.3 and 5.4 above contain the complex modulus at 0.01 Hertz and 10 Hertz for each binder grade and the three percentages of RAP. The stiffness at 10 Hertz will be used in later analysis when computing the fatigue life.

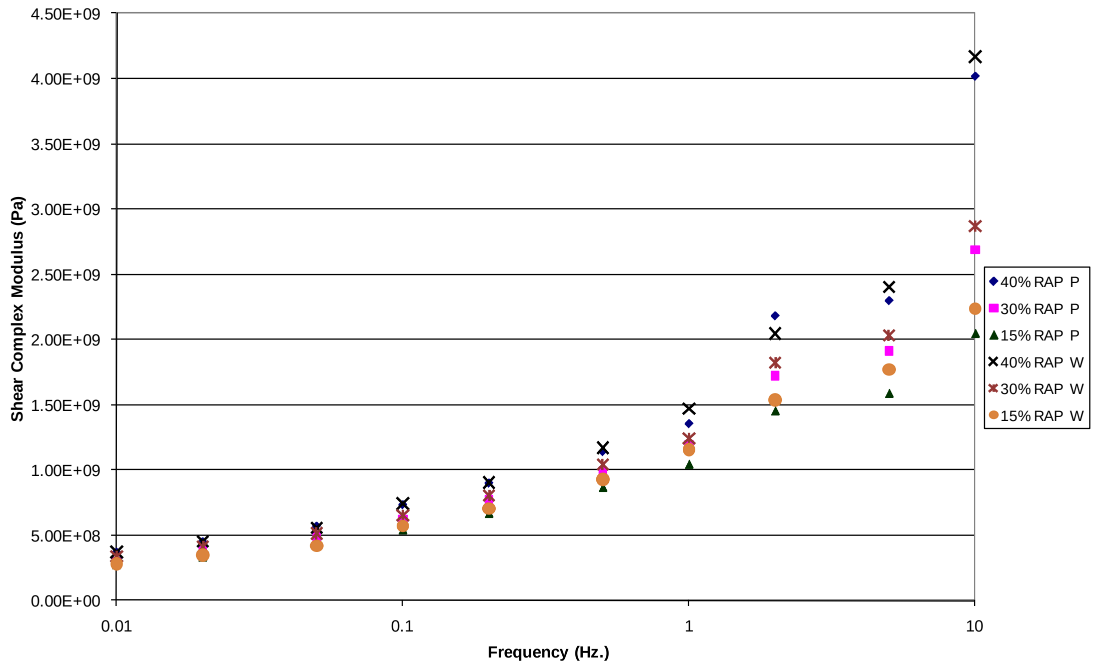

Figure 5.13 19.0 mm Frequency Sweep Comparison for PG 64-22

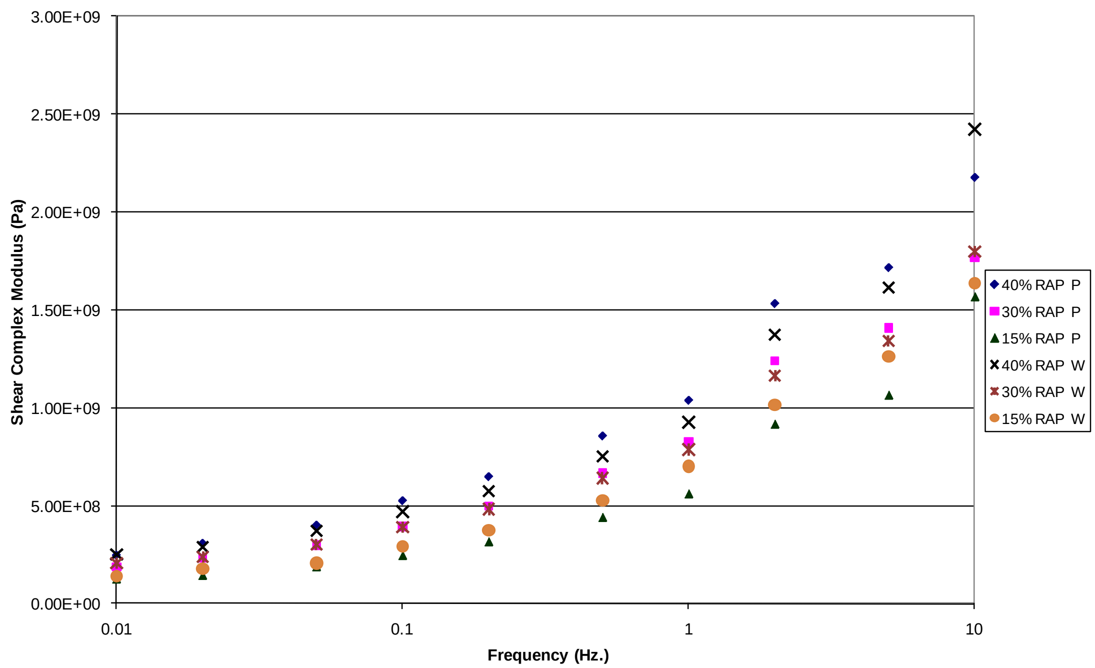

Figure 5.14 19.0 mm Frequency Sweep Comparison for PG 58-22

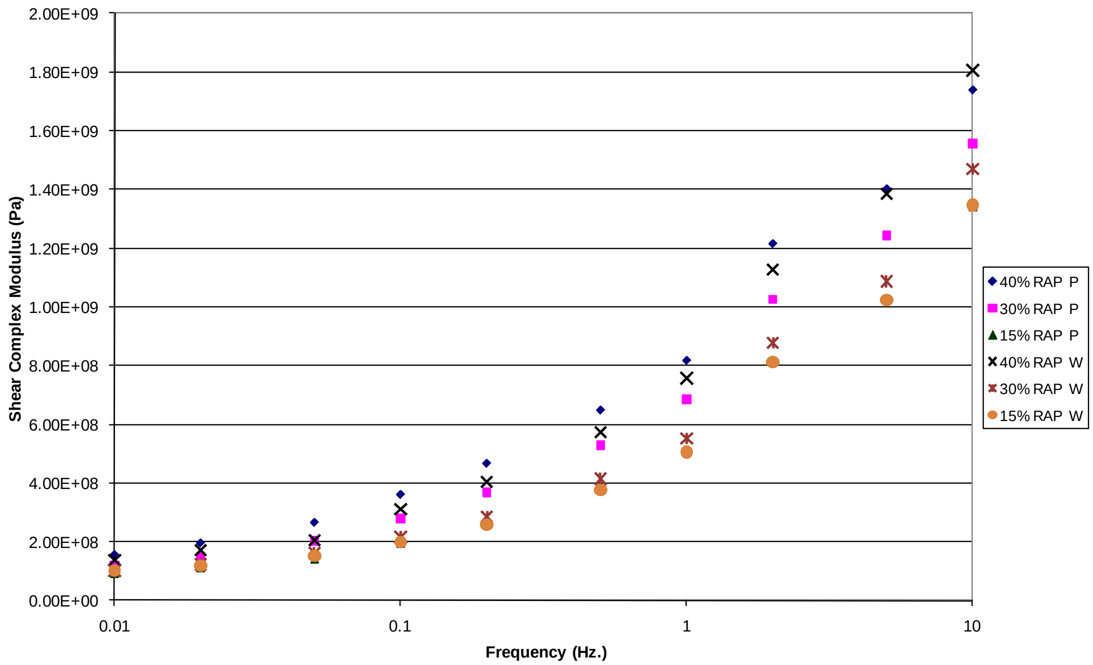

Figure 5.15 19.0 mm Frequency Sweep Comparison for PG 52-28

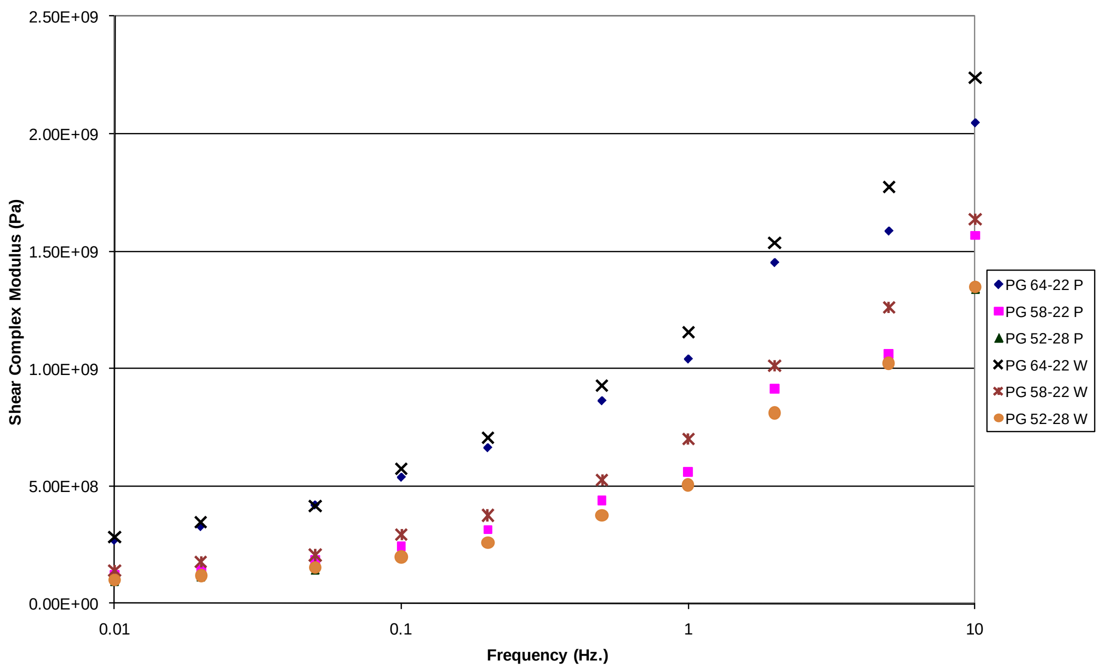

Figure 5.16 19.0 mm Frequency Sweep Comparison with 15% RAP

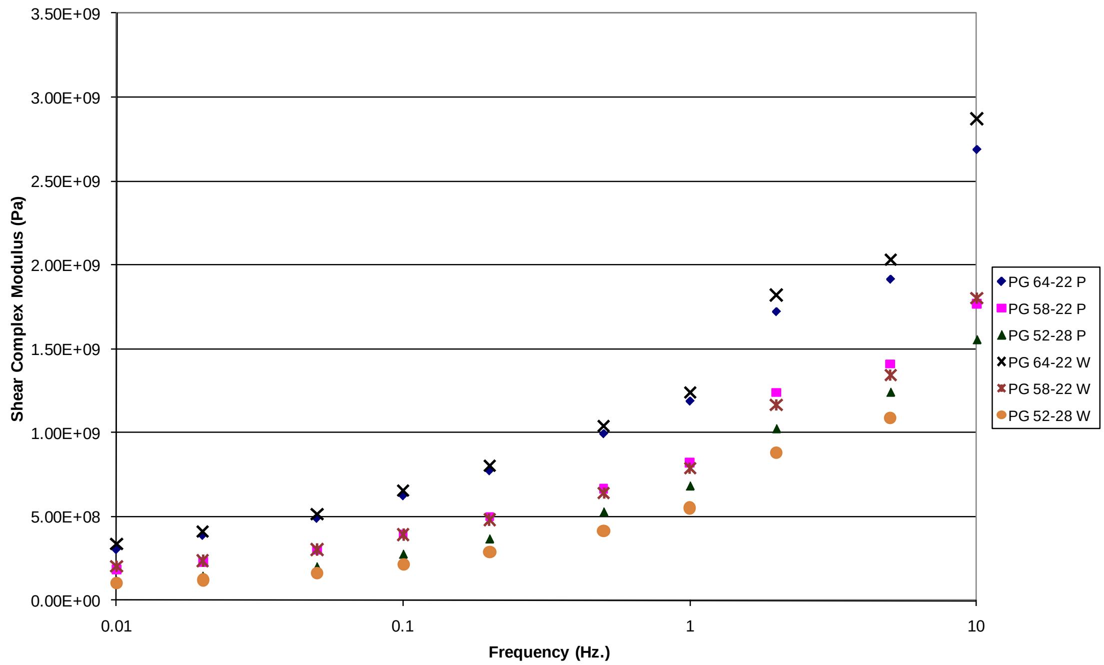

Figure 5.17 19.0 mm Frequency Sweep Comparison with 30% RAP

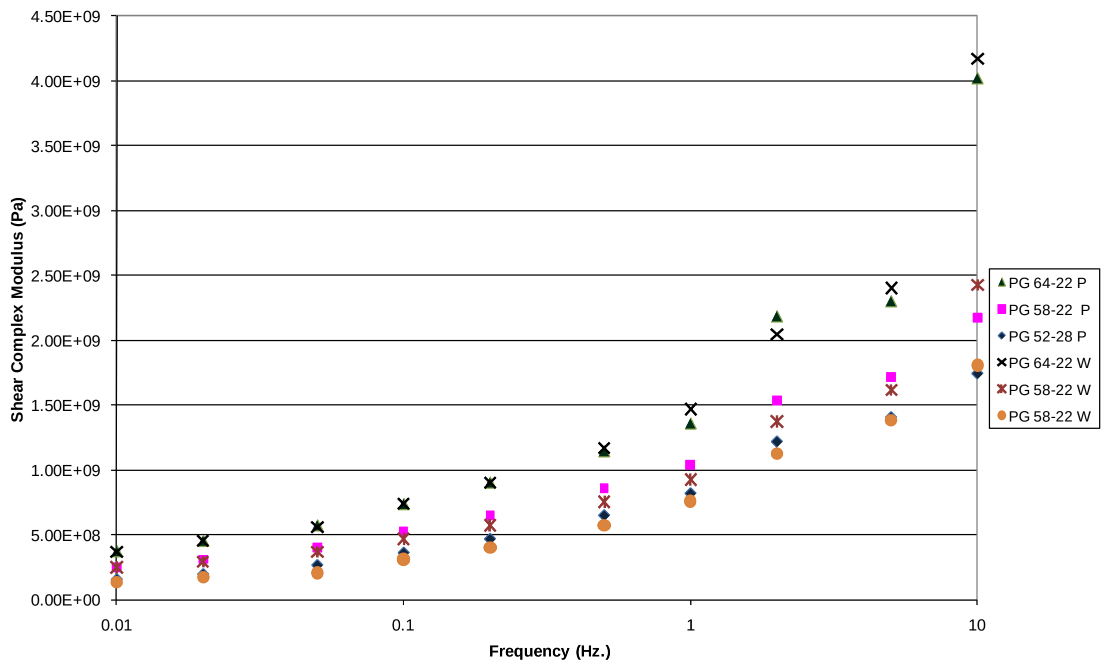

Figure 5.18 19.0 mm Frequency Sweep Comparison with 40% RAP

The results outlined above will be used in the next chapter for the pavement performance analysis in order to predict fatigue life.

5.2 Repeated Shear Analysis

Using the SST, testing for RSSTCH was completed for the 9.5mm specimens at the various RAP percentages (15%, 30%, and 40%) with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder and both RAP sources (Westgate and Pineville). Figures 5.19-5.24 below depict comparisons among mixtures containing the three various binder grades, various percentages of RAP and both RAP sources for 9.5 mm mixtures for RSSTCH.

After 5000 cycles of repeated shear, the mixtures containing PG 64-22 binder exhibited the least amount of strain at all percentages of RAP. Mixtures with PG 58-22 binder exhibited lesser strain than the mixtures with PG 52-28 binder. This can be expected as mixtures with stiffer binders exhibited less strain.

Comparing mixtures with various percentages of RAP with constant binder grade shows general trends that the strain decreases with increasing RAP. Again, this can be expected as the binder recovered from the RAP was stiffer than the virgin binders due to its aging during its service life. Therefore, mixtures with higher percentages of RAP are going to generally exhibit less strain as a result.

Table 5.5 below compares the values of the plastic shear strain accumulated by the 9.5 mm virgin mixtures to the 9.5 mm mixtures containing various percentages of Westgate RAP.

Table 5.5 Comparison of Plastic Strain Values After RSSTCH for 9.5 mm Mixtures Westgate

Binder Grade	RAP Percentage	Plastic Strain (%) @ 5,000 Cycles	Percent Difference from Virgin Mixtures
PG 64-22	0	1.07	--
PG 64-22	15%	0.99	-7
	30%	0.92	-14
	40%	0.59	-45
PG 58-22	15%	1.37	28
	30%	1.29	21
	40%	1.01	-6
PG 52-28	15%	2.14	100
	30%	1.52	42
	40%	1.42	33

Table 5.6 below compares the values of the plastic shear strain accumulated by the 9.5 mm virgin mixtures to the 9.5 mm mixtures containing various percentages of Pineville RAP.

Table 5.6 Comparison of Plastic Strain Values After RSSTCH for 9.5 mm Mixtures Pineville

Binder Grade	RAP Percentage	Plastic Strain (%) @ 5,000 Cycles	Percent Difference from Virgin Mixtures
PG 64-22	0	1.07	--
PG 64-22	15%	0.83	-22
	30%	0.75	-30
	40%	0.39	-64
PG 58-22	15%	1.64	53
	30%	1.20	12
	40%	0.81	-24
PG 52-28	15%	2.42	126
	30%	1.66	55
	40%	1.56	46

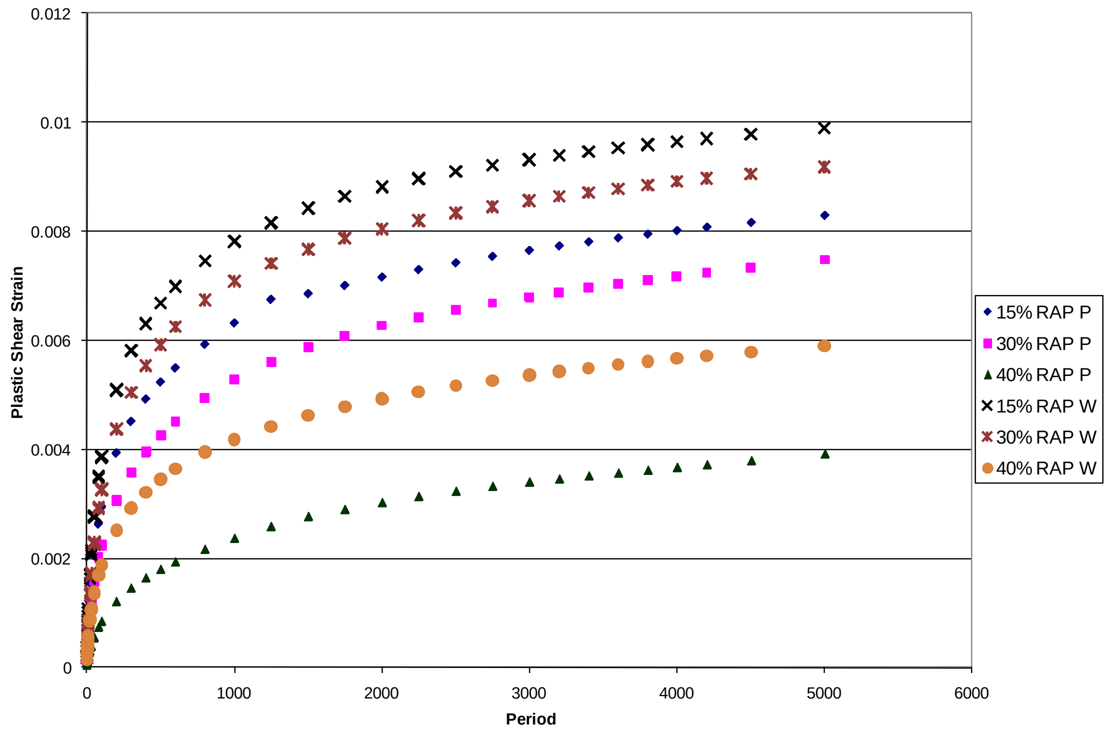

Figure 5.19 9.5 mm Repeated Shear Comparison for PG 64-22

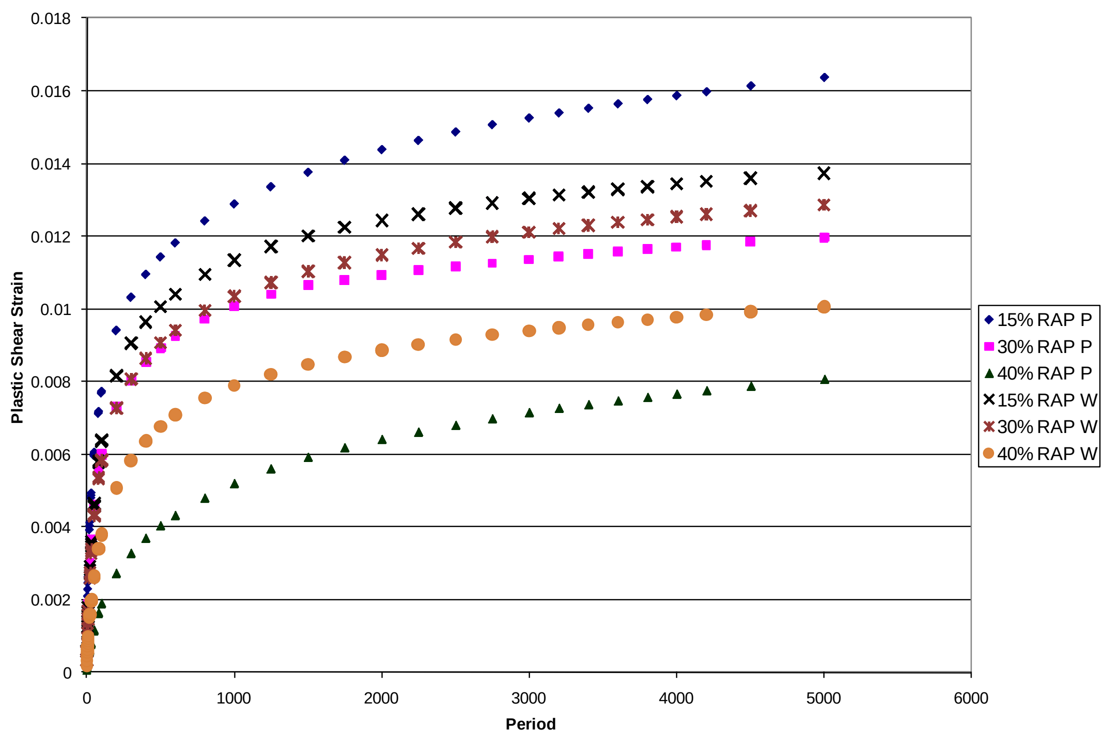

Figure 5.20 9.50 mm Repeated Shear Comparison for PG 58-22

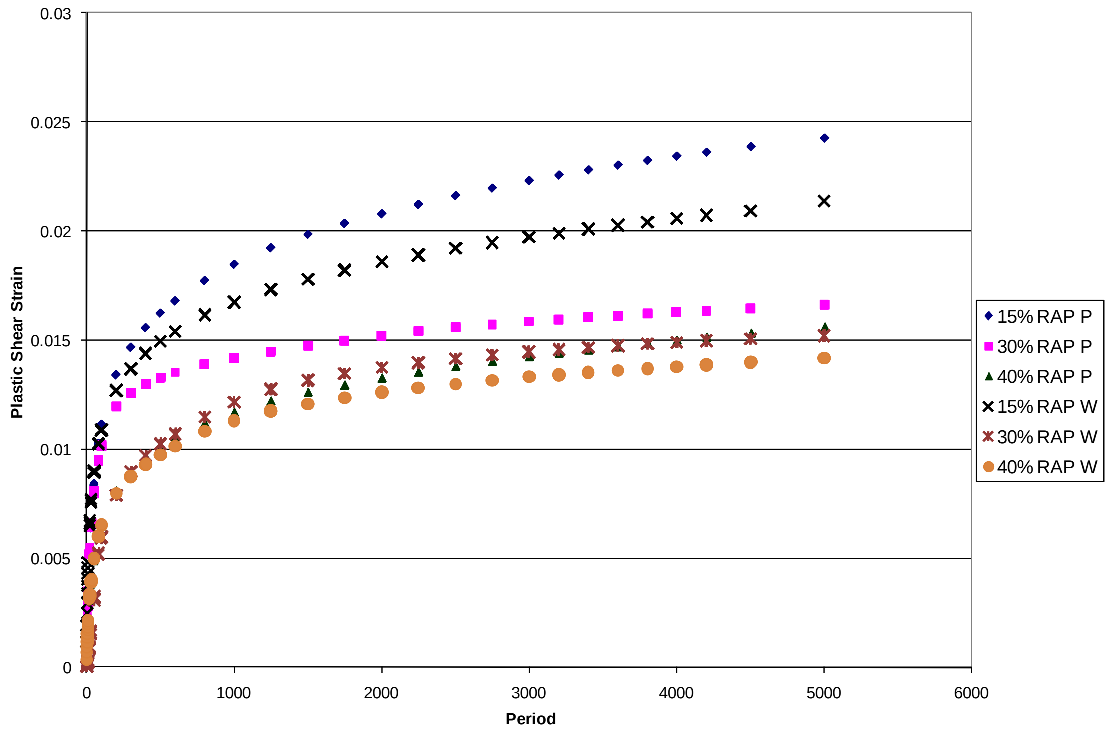

Figure 5.21 9.5 mm Repeated Shear Comparison for PG 52-28

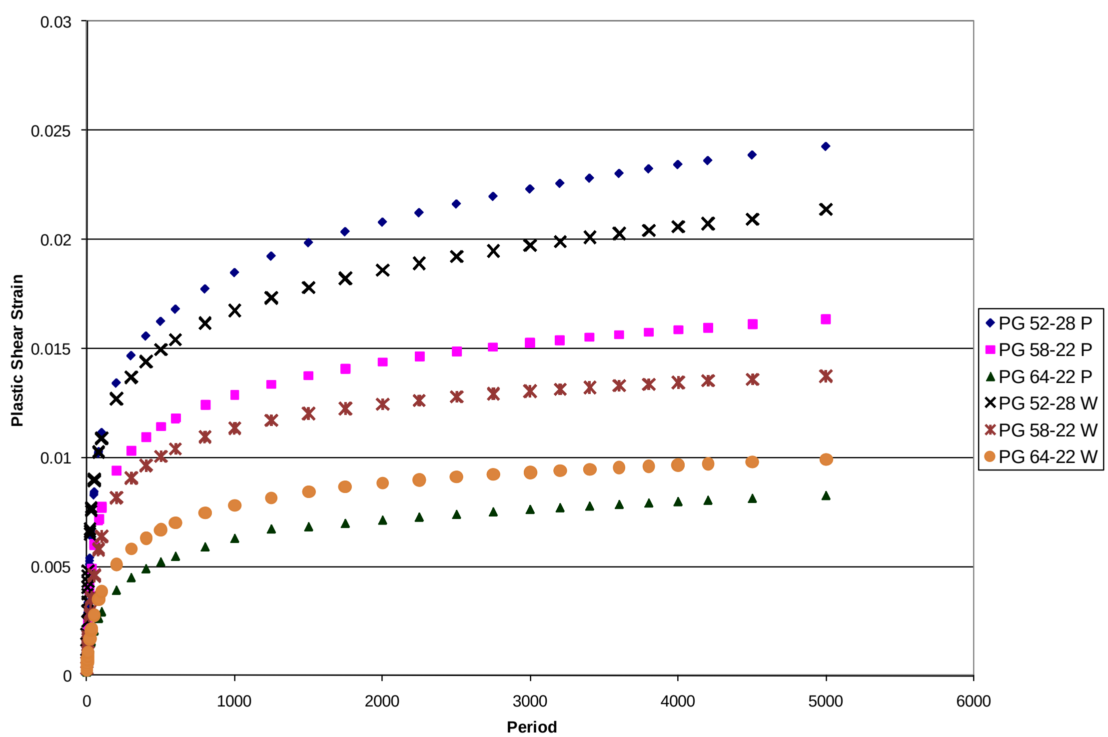

Figure 5.22 9.5 mm Repeated Shear Comparison with 15% RAP

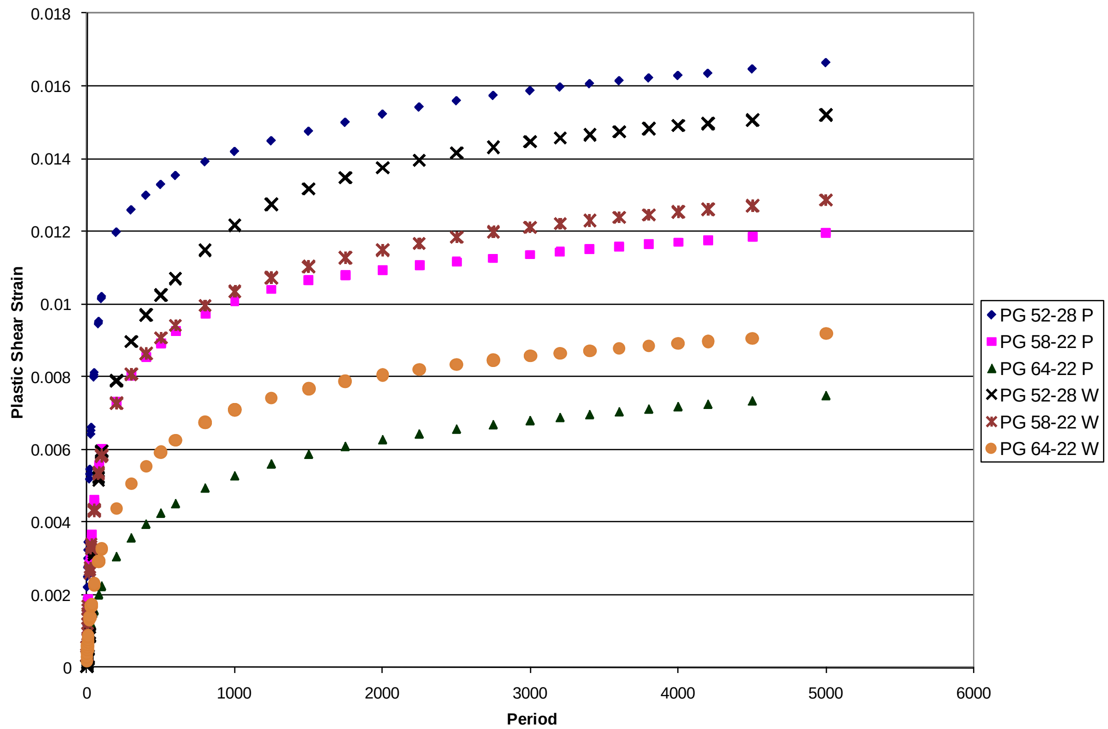

Figure 5.23 9.5 mm Repeated Shear Comparison with 30% RAP

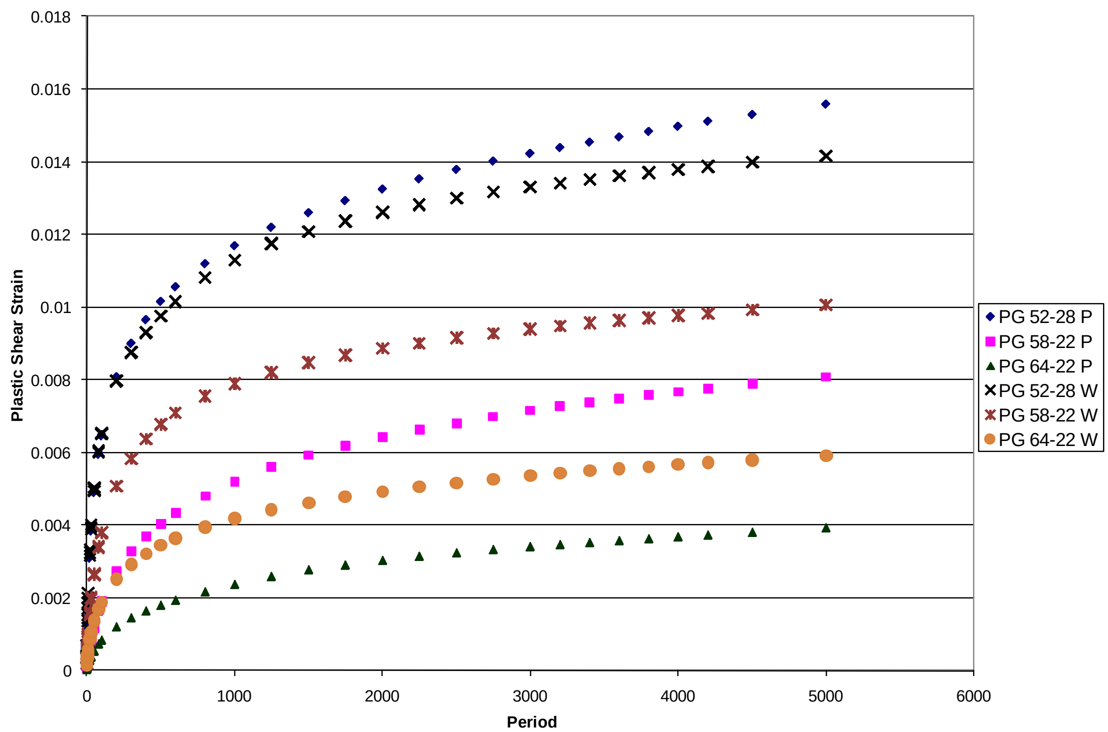

Figure 5.24 9.5 mm Repeated Shear Comparison with 40% RAP

Using the SST, testing for RSSTCH was completed for the 19.0 mm specimens at the various RAP percentages (15%, 30%, and 40%) with PG 64-22 binder, PG 58-22 binder and PG 52-28 binder and both RAP sources (Westgate and Pineville). Figures 5.25-5.30 below depict comparisons among mixtures containing the three various binder grades, various percentages of RAP and both RAP sources for 19.0 mm mixtures for RSSTCH.

Tables 5.7 and 5.8 below compare the values of the plastic shear strain accumulated by the 19.0 mm virgin mixtures to the 19.0 mm mixtures containing various percentages of Westgate and Pineville RAP, respectively. The amount of plastic shear strain reduces for all PG 64-22 mixtures containing RAP varying from 27%-35% reduction and 10%-38% reduction for Westgate and Pineville mixtures, respectively. The PG 58-22 Westgate mixtures containing 15% RAP result in the plastic shear strain increase of 8%. This increase is caused by the reduction in binder stiffness of the PG 58-22 binder from the original PG 64-22 virgin binder. However, this reduction in binder stiffness is overcome when 30% and 40% Westgate RAP is included in the mix and the plastic shear strain reduces 15% and 19%, respectively, from the virgin mixture. With a reduction of one binder grade from the original virgin binder to a PG 58-22 binder, the plastic shear strain increases 37%, 27%, and 19% for mixtures containing 15%, 30%, and 40% Pineville RAP, respectively. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 23%, 14% and 13% for mixtures containing 15%, 30% and 40% Westgate RAP, respectively. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 56%, 39% and 34% for mixtures containing 15%, 30% and 40% RAP, respectively. For all binder grades, as the percentage of RAP increases, the plastic shear strain reduces. This is because the RAP binder has a higher complex modulus than the virgin binder and results in a stiffer blend.

Table 5.7 Comparison of Plastic Strain Values After RSSTCH for 19.0 mm Mixtures Westgate

Binder Grade	RAP Percentage	Plastic Strain (%) @ 5,000 Cycles	Percent Difference from Virgin Mixtures
PG 64-22	0	1.26	--
PG 64-22	15%	0.92	-27
	30%	0.87	-31
	40%	0.82	-35
PG 58-22	15%	1.36	8
	30%	1.07	-15
	40%	1.02	-19
PG 52-28	15%	1.55	23
	30%	1.44	14
	40%	1.43	13

Table 5.8 Comparison of Plastic Strain Values After RSSTCH for 19.0 mm Mixtures Pineville

Binder Grade	RAP Percentage	Plastic Strain (%) @ 5,000 Cycles	Percent Difference from Virgin Mixtures
PG 64-22	0	1.26	--
PG 64-22	15%	1.14	-10
	30%	1.02	-19
	40%	0.78	-38
PG 58-22	15%	1.73	37
	30%	1.60	27
	40%	1.50	19
PG 52-28	15%	1.97	56
	30%	1.75	39
	40%	1.69	34

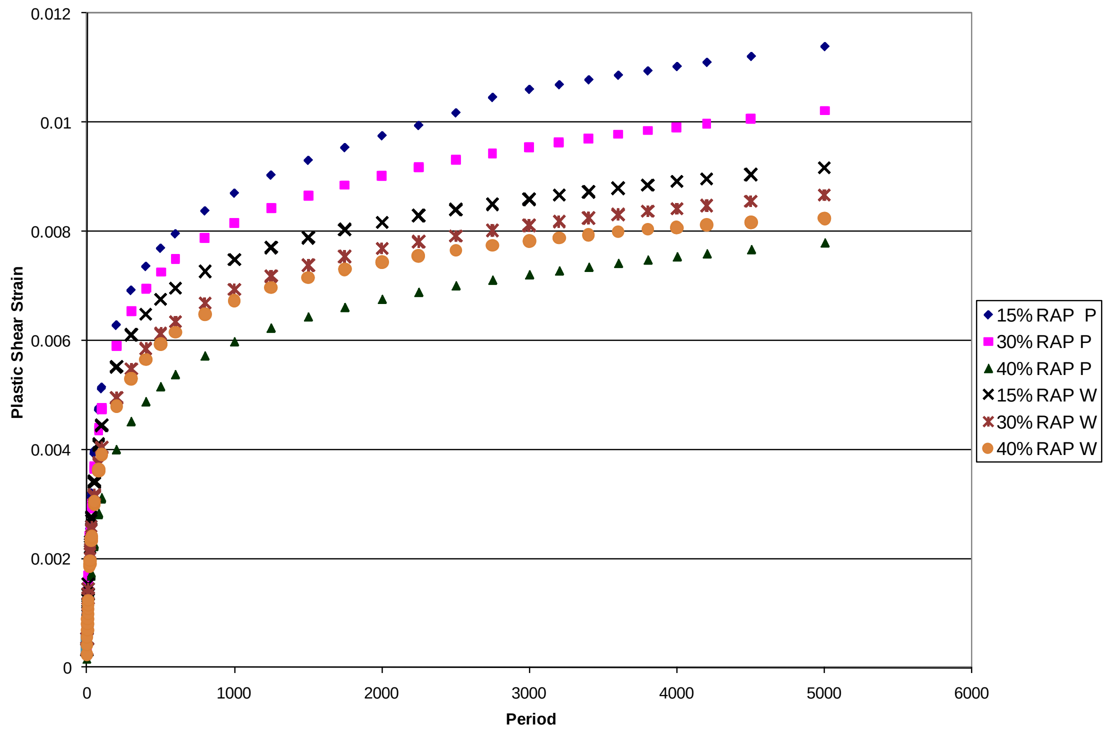

Figure 5.25 19.0 mm Repeated Shear Comparison for PG 64-22

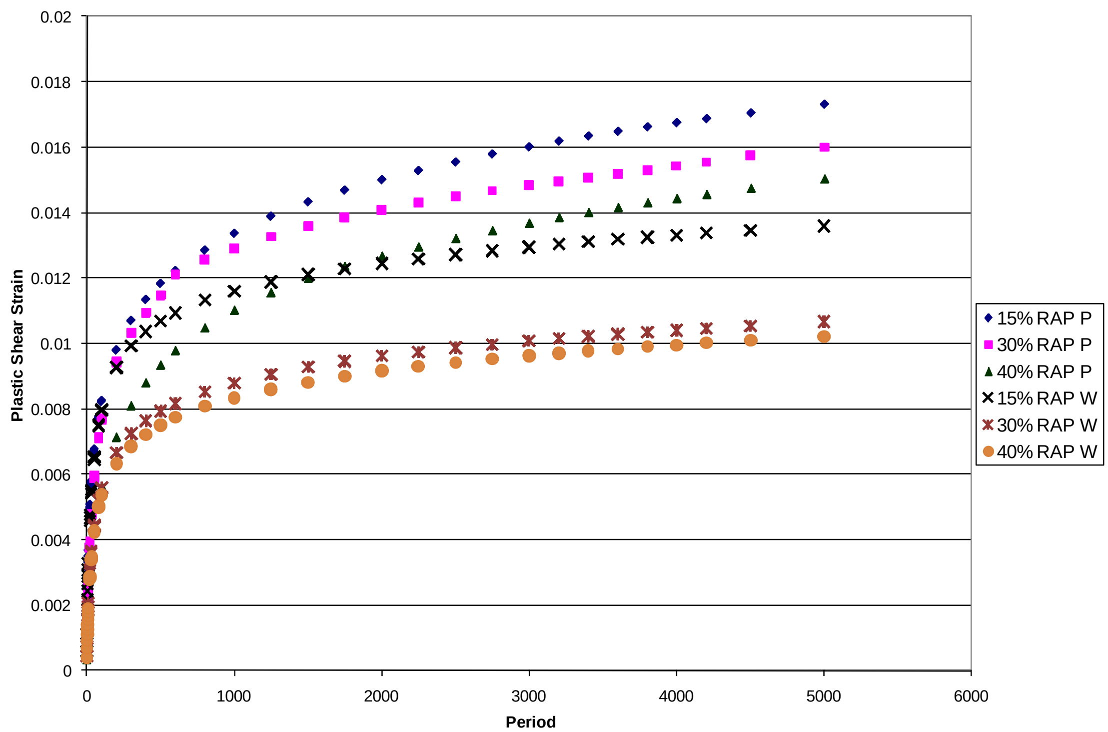

Figure 5.26 19.0 mm Repeated Shear Comparison for PG 58-22

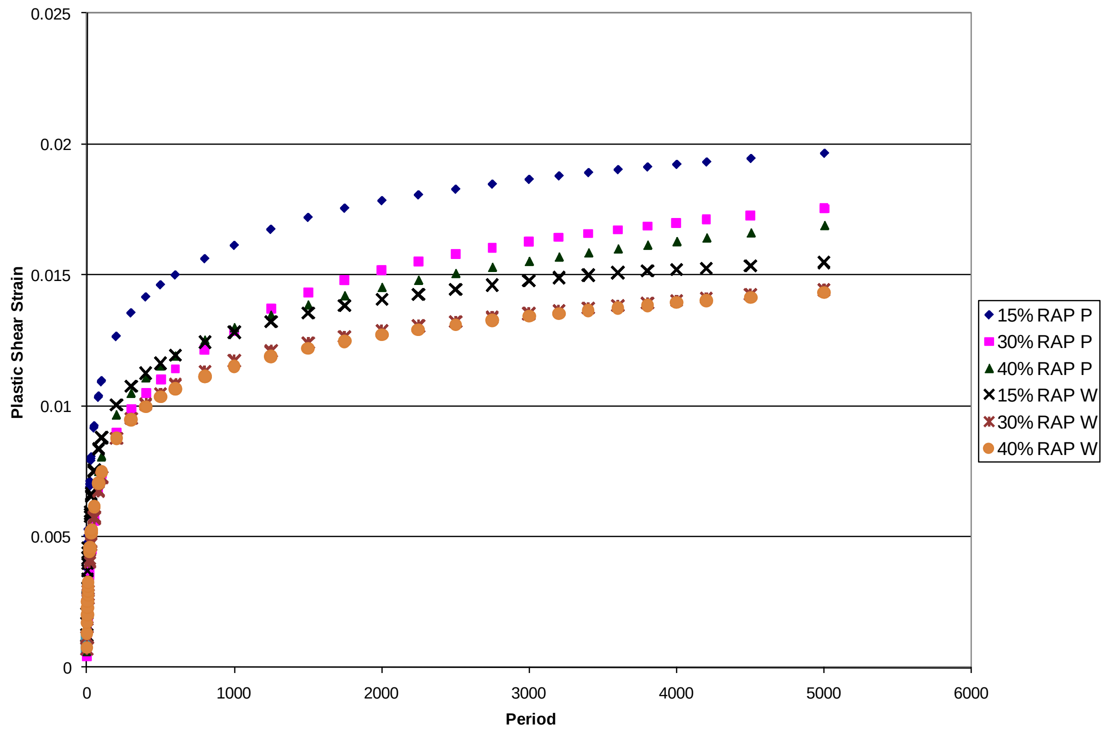

Figure 5.27 19.0 mm Repeated Shear Comparison for PG 52-28

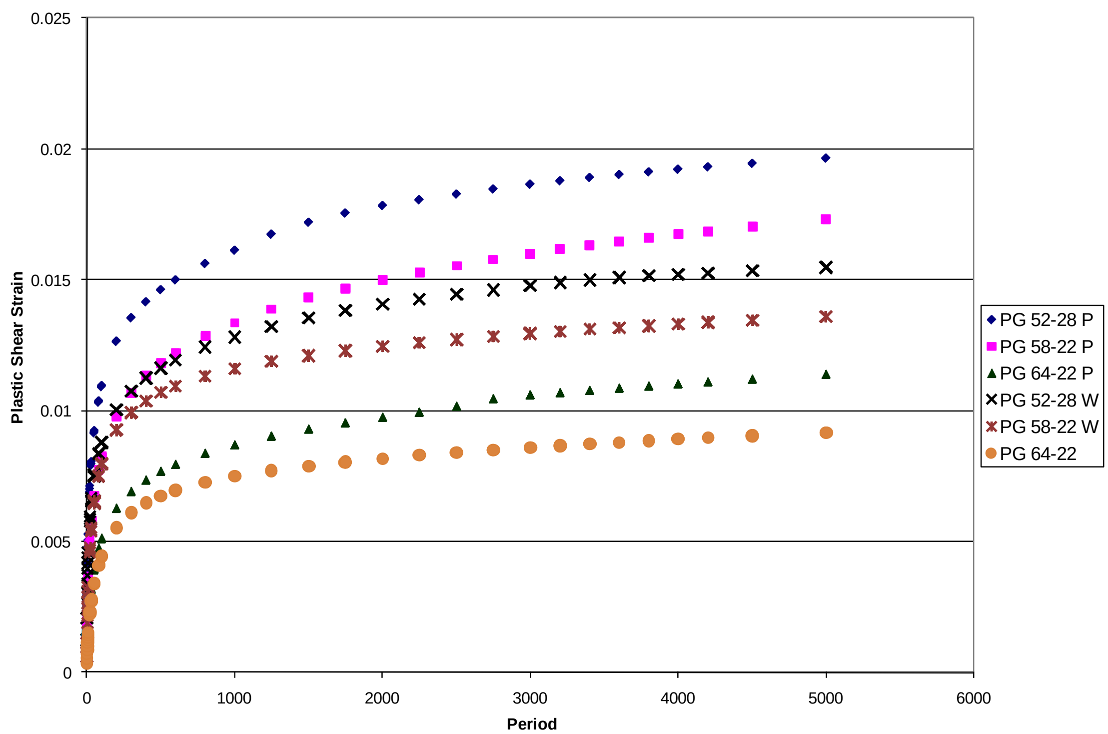

Figure 5.28 19.0 mm Repeated Shear Comparison with 15% RAP

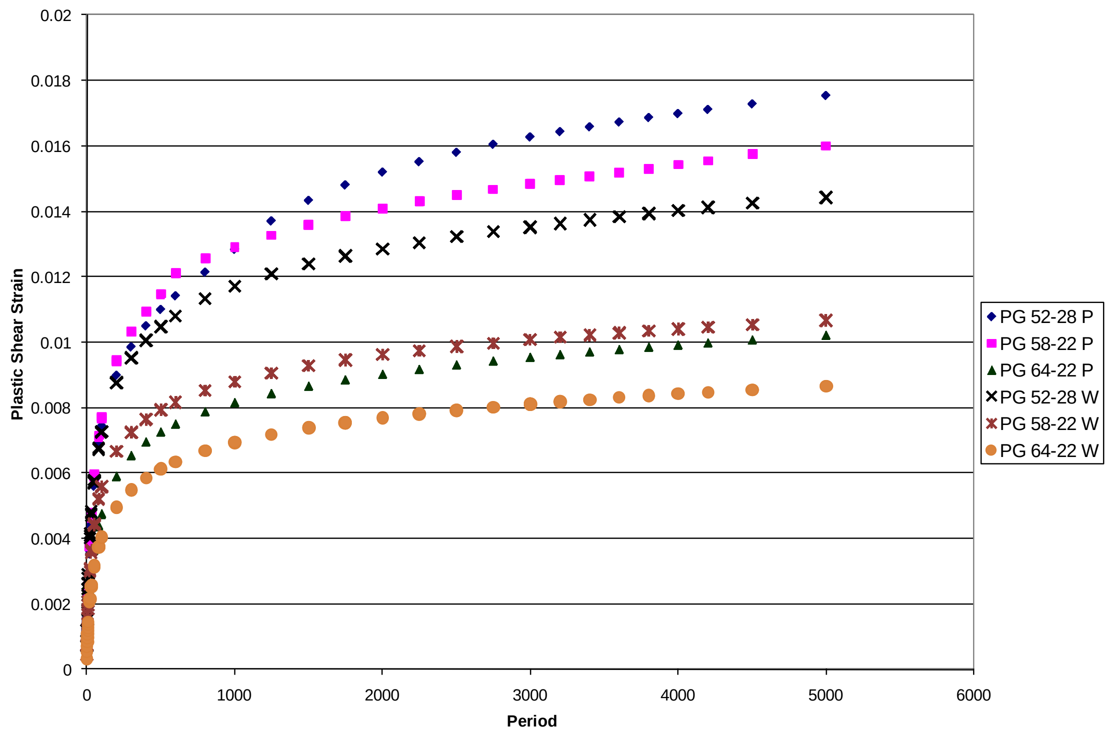

Figure 5.29 19.0 mm Repeated Shear Comparison with 30% RAP

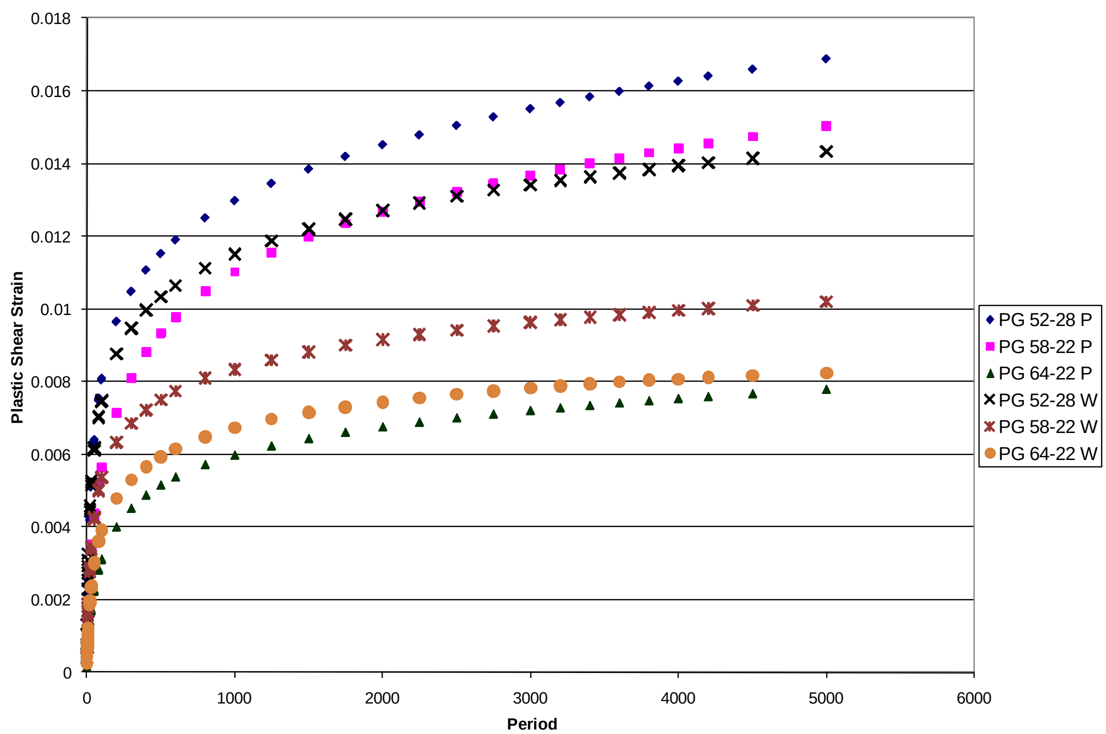

Figure 5.30 19.0 mm Repeated Shear Comparison with 40% RAP

The results outlined above will be used in the next chapter for the pavement performance analysis in order to predict rut depth and rut life.

Chapter 6

Performance Analysis

This chapter predicts the performance of pavements against fatigue cracking and rutting using the results from the Frequency Sweep Test at Constant Height (FSTCH) and Repeated Simple Shear Test at Constant Height (RSSTCH). The surrogate models from the Strategic Highway Research Program's (SHRP) A-003A as well as distress models developed by the Asphalt Institute (AI) were used to perform fatigue and rutting analysis. The models used for analysis are described below. The results from these analyses will then be used to perform an economic analysis.

6.1 Fatigue Model Analysis

The fatigue cracking model from SHRP A-003A considers horizontal tensile strain at the bottom of the pavement layer during loading, the initial flexural loss stiffness of the mixture (S_o") and the voids filled with asphalt (VFA). The shear stiffness measured during the FSTCH at 10Hz and 20^{o}C was used to determine the initial flexural loss stiffness using the following relationships:

$$S_o = 8.56*(G_o)^{0.913}$$
$$S_o^{"} = 81.125*(G_o^{"})^{0.725}$$

where,
S_o = initial flexural stiffness at 50th loading cycle (psi)
G_o = shear stiffness at 10Hz (psi)
S_o" = initial flexural loss stiffness at 50th loading cycle (psi)
G_o'' = shear loss stiffness at 10Hz (psi)

This model requires the principle tensile strain at the bottom of the asphalt concrete layer. This was estimated using EVERSTRESS pavement analysis and design software. For this software, a pavement section must be assumed. Figure 6.1 below depicts the pavement profile used for the analysis. The strain values were assessed at three locations: directly beneath the tire, at the edge of the tire and at the center of the dual tire configuration in order to consider the full loading configuration. The fatigue life of the pavement was estimated by the SHRP A-003A model as follows:

$$N_{supply} = 2.738 * 10^5 e^{0.077VFA} \varepsilon_0^{-3.624} S_o^{''-2.72}$$

where,

N_{supply} = estimated fatigue life of the pavement section in 18 kip axles (ESALs)

VFA = voids filled with asphalt for the mixture

ε_o = critical strain at the bottom of the asphalt layer

Figure 6.1 below depicts the simulated pavement cross-section geometry used for estimating the strains for the pavement performance. It contains a four inch thick asphalt concrete surface layer, eight inch thick asphalt concrete base layer, eight inch thick aggregate subbase layer and the semi-infinite subgrade. The moduli values for the asphalt concrete layers are estimated by the complex modulus at 10 Hz from the Frequency Sweep Test at Constant Height (FSTCH). The Poisson's ratio (μ) for all layers and the elastic modulus (E) for the subbase and subgrade layers were assumed. The assumed values are typical for pavement system analysis. The loading configuration consists or dual tires twelve inches on center with 100 psi tire pressure and single axel 18 kip load configuration.

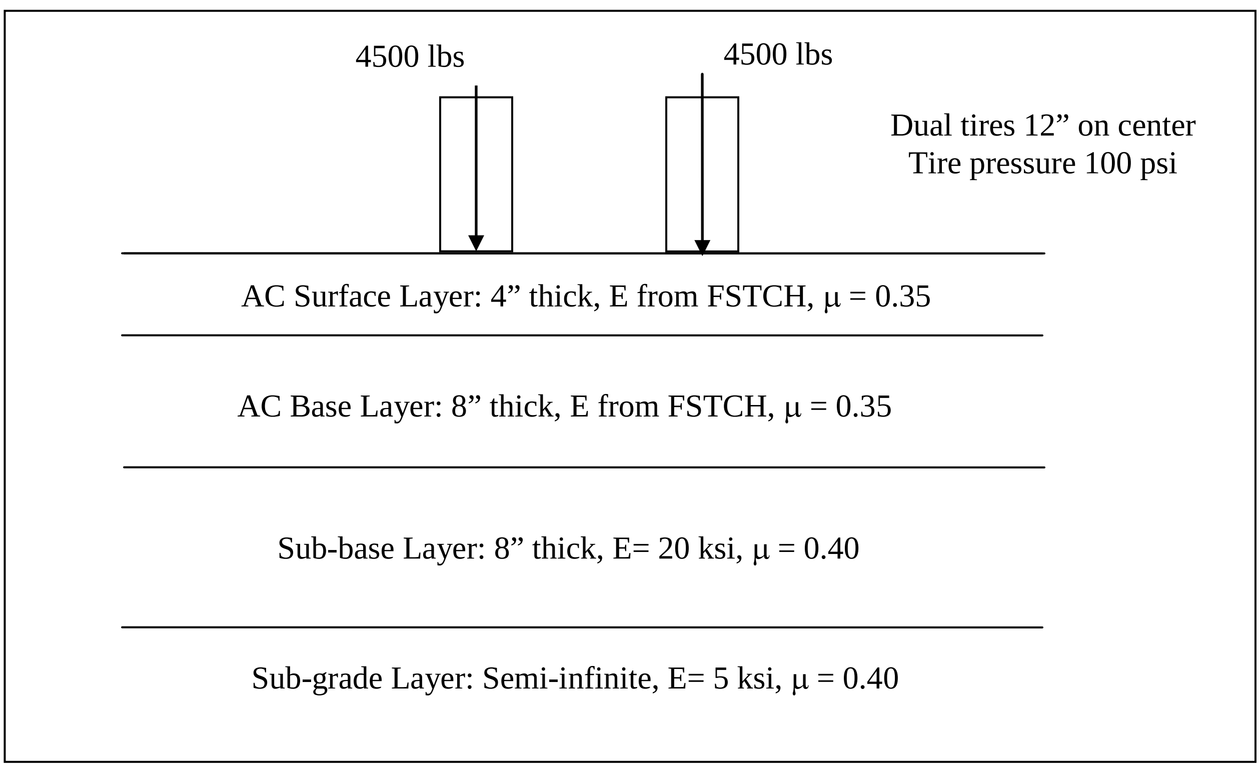

Figure 6.1 Simulated Pavement Cross-Section Geometry

The Asphalt Institute model for determining fatigue life was also considered. Like the surrogate SHRP A-003A fatigue model, the tensile strain at the bottom of the asphalt concrete layer is needed. The AI model for estimating fatigue life follows:

$$N_f = 0.00796 \varepsilon_t^{-3.291} E_1^{-0.854}$$

where,

N_f = number of load applications to fatigue failure (20% cracked area)

ε_t = tensile strain at the bottom of the asphalt layer

E_1 = elastic modulus of asphalt layer (psi)

If the two models give similar fatigue life predictions, the economic analysis will use the average predicted axle loading value. However, if the models differ significantly, the more conservative, or lowest cycles to failure will be used for the economic analysis.

6.1.1 Fatigue Analysis Results From SHRP A-003A

Table 6.1 below contains the initial flexural stiffness (S_o') of the surface and base layer and initial flexural loss stiffness (S_o") of the base layer. The initial flexural stiffness of each layer was used to estimate the elastic modulus of the layer during the simulation of the critical tensile strain at the bottom of the asphalt base layer using EVERSTRESS and the pavement geometry in Figure 6.1. The initial flexural loss stiffness is a variable for the SHRP fatigue model. Each pavement system has two initial flexural loss stiffness values with the surface asphalt layer and the base asphalt layer each having one. However, for the SHRP fatigue model, only one is needed. The base asphalt layer initial flexural loss stiffness was used in the SHRP fatigue model since this value would represent the material where the fatigue crack would initiate and propagate through due to the tensile strain at the bottom of the asphalt layer.

Table 6.1 Flexural Stiffness Properties for Simulated Pavement Systems

	Pavement System	Surface Mixture	Base Mixture	Surface S_o' (psi)	Base S_o' (psi)	Base S_o" (psi)
	1	9.5 B 64 0	19.0 C 64 0	1,009,002	894,102	504,822
Westgate	2	9.5 B 64 15	19.0 C 64 15	889,419	920,847	575,434
	3	9.5 B 64 30	19.0 C 64 30	909,824	1,156,171	595,801
	4	9.5 B 64 40	19.0 C 64 40	1,055,207	1,625,296	732,789
	5	9.5 B 58 15	19.0 C 58 15	630,276	692,410	478,815
	6	9.5 B 58 30	19.0 C 58 30	700,064	755,161	457,421
	7	9.5 B 58 40	19.0 C 58 40	788,289	990,707	514,411
	8	9.5 B 52 15	19.0 C 52 15	558,429	579,416	402,566
	9	9.5 B 52 30	19.0 C 52 30	556,129	627,678	432,256
	10	9.5 B 52 40	19.0 C 52 40	630,276	757,395	437,910
Pineville	11	9.5 B 64 15	19.0 C 64 15	858,555	848,854	453,268
	12	9.5 B 64 30	19.0 C 64 30	880,375	1,088,549	571,161
	13	9.5 B 64 40	19.0 C 64 40	893,818	1,572,658	796,572
	14	9.5 B 58 15	19.0 C 58 15	621,177	665,257	478,815
	15	9.5 B 58 30	19.0 C 58 30	661,379	742,383	457,421
	16	9.5 B 58 40	19.0 C 58 40	710,987	898,214	514,411
	17	9.5 B 52 15	19.0 C 52 15	496,328	576,797	397,249
	18	9.5 B 52 30	19.0 C 52 30	602,288	661,379	400,329
	19	9.5 B 52 40	19.0 C 52 40	650,250	732,146	423,626

Table 6.2 contains the values (Base S_o", VFA, critical strain) needed to determine the resulting fatigue life for each mixture. As expected, the general trend is that as the percentage of RAP increases in the mixtures, the fatigue life increases, all else being constant. This is expected as the increase in RAP increases the stiffness of the pavement, which results in lower critical strain values. All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the fatigue life is reduced. This is also expected as the stiffness of the virgin binder reduces with decreasing binder grade. The results show that by maintaining the original binder grade of PG 64-22, the fatigue life increases 12.3% and 23.1% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the fatigue life increases 0.9% and 10.5% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. The increase is larger for mixtures containing samples of Westgate RAP since the stiffness of the binder recovered from the Westgate RAP was stiffer than the binder recovered from the Pineville RAP. By reducing the

binder grade one grade to PG 58-22, the fatigue life increases for mixtures containing Westgate RAP 3.0% and 7.9% for mixtures containing 30% and 40% RAP, respectively. However, the fatigue life decreases for mixtures containing Pineville RAP 23.5% and 11.4% for mixtures containing 30% and 40% RAP, respectively. This reduction is due to the combination of the reduced binder grade and the less stiff binder from the Pineville RAP. By reducing the binder grade another grade to PG 52-28, the fatigue life decreases for mixtures containing Westgate RAP 19.6% and 6.6% for mixtures containing 30% and 40% RAP, respectively, and mixtures containing Pineville RAP 22.9% and 5.6% for mixtures containing 30% and 40% RAP, respectively. This reduction is due to the combination of the reduced binder grade and the less stiff binder from the Pineville RAP.

Table 6.2 Fatigue life of Simulated Pavements by SHRP Model

	Pavement System	**Surface Mixture**	**Intermediate Mixture**	**Base S_o" (psi)**	**Base VFA (%)**	**Critical Stain (10^{-5})**	**N_{supply} ESALs (10^6)**
	1	9.5 B 64 0	19.0 C 64 0	504,822	72.2	4.55	118.8
Westgate	2	9.5 B 64 15	19.0 C 64 15	575,434	76.7	4.55	117.4
	3	9.5 B 64 30	19.0 C 64 30	595,801	73.1	3.97	133.4
	4	9.5 B 64 40	19.0 C 64 40	732,789	70.9	3.16	146.1
	5	9.5 B 58 15	19.0 C 58 15	478,815	75.3	5.63	112.3
	6	9.5 B 58 30	19.0 C 58 30	457,421	74,1	5.29	119.8
	7	9.5 B 58 40	19.0 C 58 40	514,411	74.7	4.44	131.2
	8	9.5 B 52 15	19.0 C 52 15	402,566	75.2	6.33	83.8
	9	9.5 B 52 30	19.0 C 52 30	432,256	76.8	6.06	91.6
	10	9.5 B 52 40	19.0 C 52 40	437,910	74.1	5.36	112.1
Pineville	11	9.5 B 64 15	19.0 C 64 15	453,268	71.1	4.80	120.6
	12	9.5 B 64 30	19.0 C 64 30	571,161	72.4	4.13	122.3
	13	9.5 B 64 40	19.0 C 64 40	796,572	74.4	3.32	128.2
	14	9.5 B 58 15	19.0 C 58 15	478,815	75.1	5.78	88.2
	15	9.5 B 58 30	19.0 C 58 30	457,421	72.3	5.38	95.5
	16	9.5 B 58 40	19.0 C 58 40	514,411	73.1	4.77	110.9
	17	9.5 B 52 15	19.0 C 52 15	397,249	71.0	6.44	58.9
	18	9.5 B 52 30	19.0 C 52 30	400,329	72.1	5.82	90.9
	19	9.5 B 52 40	19.0 C 52 40	423,626	72.8	5.44	105.2

6.1.2 Fatigue Analysis Results from Asphalt Institute Model

Table 6.3 below contains the parameters needed to estimate the fatigue life using the Asphalt Institute model. The base initial flexural stiffness was used to estimate the elastic modulus of the asphalt layer. This value was used since the base layer material will govern the crack initiation and propagation. The critical strain is the same as that calculated and used in the SHRP model above.

Table 6.3 Fatigue life of Simulated Pavements by Asphalt Institute

Pavement System	Surface Mix	Base Mix	Base E1=So' (psi)	Critical Strain (10^{-5})	N_f ESALs (10^6)
1	Virgin	Virgin	894102	4.55E-05	12.8
2	64W15	64W15	920847	4.55E-05	12.5
3	64W30	64W30	1156171	3.97E-05	16.2
4	64W40	64W40	1625296	3.16E-05	25.5
5	64P15	64P15	848854	4.80E-05	11.2
6	64P30	64P30	1088549	4.13E-05	14.9
7	64P40	64P40	1572658	3.32E-05	22.4
8	58P15	58P15	665257	5.78E-05	7.5
9	58P30	58P30	742383	5.38E-05	8.7
10	58P40	58P40	898214	4.77E-05	10.9
11	58W15	58W15	692410	5.63E-05	7.9
12	58W30	58W30	755161	5.29E-05	9.0
13	58W40	58W40	990707	4.44E-05	12.7
14	52P15	52P15	576797	6.44E-05	5.9
15	52P30	52P30	661379	5.82E-05	7.4
16	52P40	52P40	732146	5.44E-05	8.5
17	52W15	52W15	579416	6.33E-05	6.3
18	52W30	52W30	627678	6.06E-05	6.8
19	52W40	52W40	757395	5.36E-05	8.6

As is expected, the general trend is that as the percentage RAP increases in the mixtures, the fatigue life increases, all else being constant. This is expected as the increase in RAP increases the stiffness of the pavement, which results in lower critical strain values. All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the fatigue life is reduced. This is also expected as the stiffness of the virgin binder reduces with decreasing binder grade.

The results show that maintaining the original binder grade of PG 64-22, the fatigue life increases 26.2% with 30% RAP and 99.2% with 40% RAP for mixtures containing Westgate RAP and the fatigue life increases 16.2% and 75.1% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. Similarly to the SHRP model, the increase is larger for mixtures containing samples of Westgate RAP since the stiffness of the binder recovered from the Westgate RAP was stiffer than the binder recovered from the Pineville RAP. By reducing the binder grade one grade to PG 58-22, the fatigue life reduces 14.9% with 30% RAP and 0.7% with 40% RAP for mixtures containing Westgate RAP and the fatigue life reduces 32.5% and 29.4% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. This reduction is due to the reduced stiffness of the reduced binder grade. By reducing the binder grade another grade to PG 52-28, the fatigue life reduces 47.2% with 30% RAP and 32.6% with 40% RAP for mixtures containing Westgate RAP and the fatigue life reduces 42.4% and 33.9% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. This reduction is due to the reduced stiffness of the reduced binder grade.

Initially, the vast difference between the estimates for fatigue life between the SHRP model and the Asphalt Institute model may cause suspicion. However, these differences can be attributed to two main reasons. First, the two models define failure differently. The SHRP model defines failure as 50% reduction in flexural stiffness of the pavement while the Asphalt Institute defines failure as cracking cover 20% of the pavement surface area. Secondly, the two models were developed based on different types of testing with the SHRP model using controlled-strain testing and the Asphalt Institute using controlled-stress testing. In order to remain conservative in our estimates of service life and cost savings, the estimates from the Asphalt Institute will be used in further analysis. Therefore, any conclusions could potentially result in more cost savings since this is the most conservative approach.

6.2 Rutting Model Analysis

According to SHRP A-003A, the rut depth is estimated as a function of the maximum permanent shear strain from the RSSTCH test. This relationship is as follows:

*Rut Depth (in) = 11 * Maximum permanent shear strain*

This relationship should hold true for all tire pressures but is expected to decrease with a decrease from the original pavement thickness of 15 inches [7]. The following equation converts the number of RSSTCH test loading cycles to 18-kip equivalent single axle loads (ESALs):

log (cycles) = -4.36 + 1.24 log (ESALs)

where,
cycles = number of cycles obtained from the RSSTCH test,
ESALs = equivalent 18-kip single axle loads

Again, the Asphalt Institute model for rutting was used to estimate the rutting resistance for the pavement systems produced by the various mixtures. However, unlike the SHRP model, the AI model not only considers the mixture properties but also the pavement geometry and pavement system as a whole. The AI model considers the vertical compressive strain at the top of the subgrade layer as part of the model to determine the number of loading repetitions (N_d) required to cause a rut depth of 0.5 inch. The model is as follows:

$$N_d = 1.365 * 10^{-9} \varepsilon_c^{-4.477}$$

where,
ε_c = compression strain at top of the subgrade

Similar to the fatigue cracking analysis, EVERSTRESS pavement analysis software was used to estimate the vertical compressive strain at the top of the subgrade layer.

6.2.1 Rutting Analysis Results from SHRP A-003A

Table 6.4 below contains the surface mixture strain measured by the Repeated Simple Shear Test at Constant Height which is used in the SHRP model to predict the rut depth. The surface mixture average shear strain for the mixture was used for the prediction as it is assumed that the small rut depths estimated by this model are a result of the surface mixture densification in the wheel paths. As is expected, the general trend is that as the percentage of RAP increases in the mixtures, the rut depth decreases, all else being constant. This is expected as the increase in RAP increases the stiffness of the pavement, which results in lower shear strain. All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the rut depth increases. This is also expected as the stiffness of the virgin binder reduces with decreasing binder grade and therefore results in larger shear strains. The results show that maintaining the original binder grade of PG 64-22, the rut depth decreases 14.3% and 44.9% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut depth decreases 30.18% and 63.40% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. By reducing the binder grade one grade to PG 58-22, the rut depth increases 20.0 % and 11.72% with 30% Westgate RAP and Pineville RAP, respectively. The rut depth decreases 6.18% and 24.64% for mixtures containing 40% Westgate RAP and Pineville RAP, respectively. The increase in rut depth for mixtures containing 30% RAP is due to the softer PG 58-22 binder resulting in larger shear strains. By reducing the binder grade another grade to PG 52-28, the rut depth increases for mixtures containing Westgate RAP 41.9% and 32.2% for mixtures containing 30% and 40% RAP, respectively, and mixtures containing Pineville RAP 55.2% and 45.4% for mixtures containing 30% and 40% RAP, respectively. This increase in rut depth for mixtures is due to the softer PG 52-28 binder resulting in larger shear strains.

Table 6.4 Rut Depth Estimates of Simulated Pavements by SHRP Model

	Pavement System	Surface Mixture	Base Mixture	Surface Mixture Ave. Shear Strain from RSSTCH	Rut Depth (in)
	1	9.5 B 64 0	19.0 C 64 0	0.01072	0.118
Westgate	2	9.5 B 64 15	19.0 C 64 15	0.00989	0.109
	3	9.5 B 64 30	19.0 C 64 30	0.00918	0.101
	4	9.5 B 64 40	19.0 C 64 40	0.00590	0.065
	5	9.5 B 58 15	19.0 C 58 15	0.01373	0.151
	6	9.5 B 58 30	19.0 C 58 30	0.01286	0.141
	7	9.5 B 58 40	19.0 C 58 40	0.01005	0.111
	8	9.5 B 52 15	19.0 C 52 15	0.02137	0.235
	9	9.5 B 52 30	19.0 C 52 30	0.01520	0.167
	10	9.5 B 52 40	19.0 C 52 40	0.01416	0.156
Pineville	11	9.5 B 64 15	19.0 C 64 15	0.00829	0.091
	12	9.5 B 64 30	19.0 C 64 30	0.00748	0.082
	13	9.5 B 64 40	19.0 C 64 40	0.00392	0.043
	14	9.5 B 58 15	19.0 C 58 15	0.01636	0.180
	15	9.5 B 58 30	19.0 C 58 30	0.01197	0.132
	16	9.5 B 58 40	19.0 C 58 40	0.00808	0.089
	17	9.5 B 52 15	19.0 C 52 15	0.02425	0.267
	18	9.5 B 52 30	19.0 C 52 30	0.01663	0.183
	19	9.5 B 52 40	19.0 C 52 40	0.01558	0.171

6.2.2 Rutting Analysis Results from Asphalt Institute Model

Table 6.5 below contains the compressive strain estimated at the top of the subgrade layer using EVERSTRESS similar to the tensile strain calculated for the fatigue life calculations and the loading cycles to failure which is stated as a rut depth of 0.5 inch. As is expected, the general trend is that as the percentage of RAP increases in the mixtures, the rut life increases, all else being constant. This is expected as the increase in RAP increases the stiffness of the pavement, which results in lower shear strain. All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the rut life increases. This is also expected as the stiffness of the virgin binder reduces with decreasing binder grade and therefore results in larger compressive strains in the subgrade. The results show that maintaining the original binder grade of PG 64-22, the rut life increases 41.5% and 210.7% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life increases 22.4% and 141.1% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. By reducing the binder grade one grade to PG 58-22, the rut life decreases 50.1 % and 8.6% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 54.6% and 31.4% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. This reduction in rut life is due to the decrease in stiffness of the binder resulting in an increase in compressive strain in the subgrade. By reducing the binder grade another grade to PG 52-28, the rut life decreases 71.6% and 55.3% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 66.2% and 56.3% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP. This reduction in rut life is due to the decrease in stiffness of the binder resulting in an increase in compressive strain in the subgrade.

Table 6.5 Loading Cycles to Rutting Failure for Simulated Pavements Using Asphalt Institute Model

	Pavement System	Surface Mixture	Base Mixture	ε_c at sub-base layer (x10^{-5})	N_f ESALs (x10^9)
	1	9.5 B 64 0	19.0 C 64 0	1.63E-04	0.12
Westgate	2	9.5 B 64 15	19.0 C 64 15	1.66E-04	0.11
	3	9.5 B 64 30	19.0 C 64 30	1.51E-04	0.18
	4	9.5 B 64 40	19.0 C 64 40	1.26E-04	0.39
	5	9.5 B 58 15	19.0 C 58 15	2.02E-04	0.05
	6	9.5 B 58 30	19.0 C 58 30	1.90E-04	0.06
	7	9.5 B 58 40	19.0 C 58 40	1.66E-04	0.11
	8	9.5 B 52 15	19.0 C 52 15	2.22E-04	0.03
	9	9.5 B 52 30	19.0 C 52 30	2.16E-04	0.04
	10	9.5 B 52 40	19.0 C 52 40	1.95E-04	0.06
Pineville	11	9.5 B 64 15	19.0 C 64 15	1.73E-04	0.10
	12	9.5 B 64 30	19.0 C 64 30	1.56E-04	0.15
	13	9.5 B 64 40	19.0 C 64 40	1.34E-04	0.30
	14	9.5 B 58 15	19.0 C 58 15	2.05E-04	0.04
	15	9.5 B 58 30	19.0 C 58 30	1.94E-04	0.06
	16	9.5 B 58 40	19.0 C 58 40	1.77E-04	0.09
	17	9.5 B 52 15	19.0 C 52 15	2.28E-04	0.03
	18	9.5 B 52 30	19.0 C 52 30	2.07E-04	0.04
	19	9.5 B 52 40	19.0 C 52 40	1.96E-04	0.05

6.3 Economic Analysis

An economic analysis for the life cycle cost of the pavements was conducted in order to take into account more than the initial difference in cost between pavements containing RAP material versus pavements containing 100% virgin materials. Although using RAP material for pavement construction will decrease the initial cost of construction, pavements containing RAP may require more maintenance or have a shorter service life than pavements containing 100% virgin materials reducing the initial construction savings or even resulting in more cost than pavements containing 100% virgin materials. For this reason, the life cycle analysis was utilized.

The binder rheological properties stated in Chapter Four were combined with the pavement performance above. In Chapter Four, Table 4.11 showed that when using virgin PG 64-22 binder, the maximum amount of RAP is 24%-27%, when using PG 58-22 binder, the maximum amount of RAP is 52-53%, and when using PG 52-28 binder, the maximum amount of RAP is 55-57%. Based on these results, the following economic analysis will compare virgin mixtures, PG 64-22 mixtures containing 30% RAP and PG 58-22 mixtures and PG 52-28 mixtures containing 40% RAP. (Note that although the binder analysis states that more than 40% RAP can be used with PG 58-22 and PG 52-28 binder, this data was not available. Also, further performance testing regarding thermal cracking needs to be performed with mixtures containing that amount of RAP before it can be included in the economic analysis.)

6.3.1 Economic Analysis Period

The life cycle cost for the pavement systems will be compared for a 30 year analysis period as per the NCDOT Pavement Management Unit. This 30 year analysis period should not be confused with the typical 20 year pavement design life. In order to complete the 30 year analysis period, each pavement system will need to undergo an initial rehabilitation and possibly a second or third rehabilitation.

6.3.2 Estimated Pavement Service Life

The pavement performance analysis above for fatigue life and rutting shows that fatigue cracking is the limiting distress for the pavement service life as the rut life is able to handle many more loading repetitions. Also as stated above, the AI model fatigue life analysis is more conservative and appropriate for this analysis. As a result, the estimated initial service life of the pavement systems are based on the AI model fatigue life calculations. The surface mixture contains a 9.5 B mixture and is designed for 0.3-3 millions ESALs while the base layer contains a 19.0 C mixture and is designed for 3-30 million ESALS. It is assumed that the pavement systems were designed for a rural secondary highway having an annual traffic level of 1,000,000 ESALs. It is also assumed that the growth rate is minimal and, therefore, not considered. Using these assumptions and the predicted number of loadings until fatigue failure based on the AI model, the estimated initial service life was calculated. Table 6.6 below contains the estimated initial service life of the pavement systems. The service life varies from 8.5 to 16.2 years depending on the materials of the mixture.

Table 6.6 Estimated Initial Service Life for Simulated Pavements Systems

Pavement System	Surface Mixture	Base Mixture	Estimated Initial Service Life (years)
1	9.5 B 64 0	19.0 C 64 0	12.8
3	9.5 B 64 30 W	19.0 C 64 30 W	16.2
7	9.5 B 58 40 W	19.0 C 58 40 W	12.7
10	9.5 B 52 40 W	19.0 C 52 40 W	8.6
12	9.5 B 64 30 P	19.0 C 64 30 P	14.9
16	9.5 B 58 40 P	19.0 C 58 40 P	10.9
19	9.5 B 52 40 P	19.0 C 52 40 P	8.5

6.3.3 Material Costs

Table 6.7 below contains the material costs for pavement construction. The costs per ton for the asphalt concrete mixtures were obtained from the NCDOT 2008 bid averages. The RAP screening and processing cost is an average cost from asphalt plants throughout the state.

Table 6.7 Material Costs for Pavement Construction

Material Type and/or Description	Cost (or Savings) Per Ton
Asphalt Concrete Surface Course Mixture 9.5 B	41.21
Asphalt Concrete Surface Course Mixture 19.0 C	42.51
RAP Screening and/or Processing	12.25

Table 6.8 below contains the estimated cost per ton for each mixture as well as the percentage of RAP by weight of the total mix. The previous percentages used in the mix design (i.e., 15%, 30%, 40%) were the percent of RAP binder in the mix, not the total amount of RAP in the mixture. The percent of RAP of the total mix was calculated using the percent of RAP binder in the mixture, the percent binder from each RAP source and the percent binder for the mixture. Comparing the cost per ton of the mixtures containing RAP to the cost per ton of virgin mixtures, both the surface and base course show roughly a 26-28%% and 35-37% reduction in cost per ton for 30% and 40% RAP, respectively.

Table 6.8 Material Costs for RAP Mixtures

Mixture Designation	%RAP by Weight of Total Mix	Cost per Ton
9.5 64 30 W	40	$29.63
9.5 64 30 P	40	$29.63
9.5 58 40 W	53	$25.86
9.5 58 40 P	53	$25.86
9.5 52 40 W	53	$25.86
9.5 52 40 P	53	$25.86
19.0 64 30 W	37	$31.31
19.0 64 30 P	37	$31.31
19.0 58 40 W	50	$27.38
19.0 58 40 P	50	$27.38
19.0 52 40 W	50	$27.38
19.0 52 40 P	50	$27.38

6.3.4 Initial Costs of Pavement Systems

Now that the cost per ton of each mixture has been calculated, these figures need to be translated to pavement systems. For this, several assumptions are applied to all pavement systems. The pavement cross-section contains a two-lane roadway with total width of 28 feet, which consists of two twelve foot travel lanes and two, two foot shoulders. The total volume of material needed was estimated using this road configuration and the pavement's cross section geometry shown in Figure 6.1. The compacted density for the asphalt pavement was assumed to be 150 lb/ft^3. Since it is desired to determine the difference in cost between the different mixtures, the preparation of the subbase and subgrade layers are assumed to be equal for all pavement systems, and therefore, were not included in this analysis. Table 6.9 below contains the estimated material cost per mile for each pavement system. By comparing the material cost per mile of the virgin mixtures to the mixtures containing various percentages of RAP shows that the inclusion of 30% RAP reduces the cost about 27% and the inclusion of 40% RAP reduces the cost about 36%.

Table 6.9 Initial Construction Cost for Pavement Systems

Pavement System	Surface Mixture	Base Mixture	Material Cost per mile
1	9.5 B 64 0	19.0 C 64 0	$466,546
3	9.5 B 64 30 W	19.0 C 64 30 W	$340,956
7	9.5 B 58 40 W	19.0 C 58 40 W	$297,972
10	9.5 B 52 40 W	19.0 C 52 40 W	$297,972
12	9.5 B 64 30 P	19.0 C 64 30 P	$340,956
16	9.5 B 58 40 P	19.0 C 58 40 P	$297,972
19	9.5 B 52 40 P	19.0 C 52 40 P	$297,972

6.3.5 Pavement System Rehabilitation

Once the pavement systems reached the end of their service life, a rehabilitation needed to be performed on the pavement system in order to restore serviceability of the pavement system. For this analysis, the rehabilitation performed on each pavement system was a two inch overlay. It was assumed that the overlay contained the same material as the initial pavement system. In order to determine the service life of the rehabilitated pavement, EVERSTRESS was used to determine the critical strain at the bottom of the asphalt layer in order to determine the new fatigue life of the pavement system. In order to simulate the damage the existing pavements have experienced in the initial service life before the rehabilitation, the moduli for the asphalt surface course and asphalt base course were reduced 50% and 30%, respectively. The Asphalt Institute model was again used to determine the service life of the rehabilitated pavement system.

Table 6.10 below contains the estimated service life for pavement systems after rehabilitation and the cost of the material for the rehabilitation. Pavement systems 1, 7, 10, 16 and 19 still do not meet the total service life of 30 years needed for the analysis after the initial rehabilitation, so a second rehabilitation will be performed on those pavement systems. Pavement Systems 7 and 19 need a third rehabilitation. It is assumed that the first overlay rehabilitation will be milled up and the second rehabilitation will have the same service life as the initial rehabilitation. The cost for the milling was not considered as these costs were assumed to be minimal compared to the cost of the rehabilitation.

Table 6.10 Estimated Service Life for Pavement Systems After Rehabilitation

Pavement System	Surface Mixture	Base Mixture	Initial Service Life (years)	Rehab. Service Life (years)	Total Service life (years)	Rehab. Material Cost (per mile)
1	9.5 B 64 0	19.0 C 64 0	12.8	14.7	27.5	$76,156
3	9.5 B 64 30 W	19.0 C 64 30 W	16.2	17.9	34.1	$54,756
7	9.5 B 58 40 W	19.0 C 58 40 W	10.9	12.1	23.0	$47,789
10	9.5 B 52 40 W	19.0 C 52 40 W	8.6	9.5	18.1	$47,789
12	9.5 B 64 30 P	19.0 C 64 30 P	14.9	16.5	31.4	$54,756
16	9.5 B 58 40 P	19.0 C 58 40 P	12.7	14.1	26.8	$47,789
19	9.5 B 52 40 P	19.0 C 52 40 P	8.5	10.2	18.7	$47,789

Figure 6.2 below contains the pavement system service life and rehabilitation activity. As stated above, it shows that pavement systems 1, 7 and 16 require a second rehabilitation and pavement systems 7 and 19 required a third rehabilitation in order to meet the analysis period of 30 years.

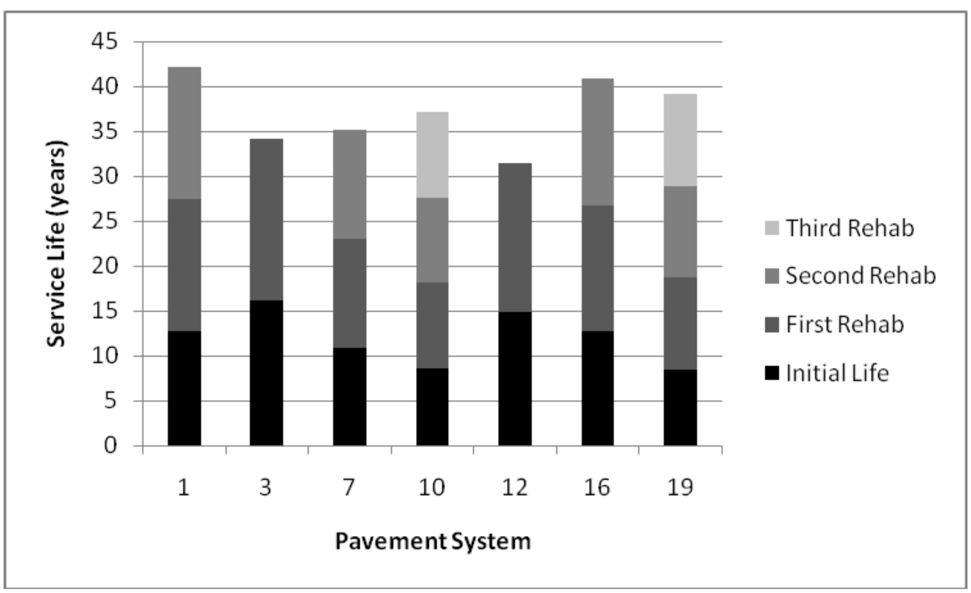

Figure 6.2 Pavement System Service Life and Rehabilitation Activity

6.3.6 Salvage Value of Pavement Systems

Each of the pavement systems had a remaining service life after the initial, second or third rehabilitation beyond the analysis period of 30 years. The remaining service life of the pavement systems after the rehabilitation was converted to a salvage value using the equation [2]:

$$SV = \left(1 - \frac{Y}{Y_e}\right) C$$

Where,

SV = salvage value

Y = the difference of the analysis period and the number of years left before next rehabilitation

Y_e = the service life of the rehabilitated system, and

C = the rehabilitation or initial construction cost of the system.

Table 6.11 below contains the rehabilitation service life, the service life of the rehabilitation used and the calculated salvage value per mile for the pavement systems.

Table 6.11 Salvage Value of Rehabilitated Pavement Systems

Pavement System	Surface Mixture	Base Mixture	Rehab. Service Life (years)	Service Life Used (years)	Salvage Value Per Mile
1	9.5 B 64 0	19.0 C 64 0	14.7	2.5	$63,204
3	9.5 B 64 30 W	19.0 C 64 30 W	17.9	13.8	$12,542
7	9.5 B 58 40 W	19.0 C 58 40 W	12.1	7.0	$20,142
10	9.5 B 52 40 W	19.0 C 52 40 W	9.5	2.4	$35,716
12	9.5 B 64 30 P	19.0 C 64 30 P	16.5	15.1	$4,646
16	9.5 B 58 40 P	19.0 C 58 40 P	14.1	3.2	$36,943
19	9.5 B 52 40 P	19.0 C 52 40 P	10.2	1.1	$42,635

6.3.7 Present Value (Costs) of Pavement Systems

In order to conduct a life cycle cost analysis, it was recommended that either the present worth method or annual cost method be implemented. Both of these methods took into account initial costs of the pavement construction and all future year costs as well as salvage values. The present worth method converts all costs or returns to the present value for analysis. The annual cost method converts all costs and returns to a uniform annual cost for analysis. Both of these methods should result in similar conclusions and the pavement with the lowest life cycle cost should be selected. The following relationship is used for the present worth method.

The present worth of a future sum can be found by:

$$PW = \frac{F}{(1+i)^n}$$

where,

PW = present worth of a sum of money that takes place N years from the base year.

F = future sum of an improvement at the end of year N, and

i = discount rate.

According to the NCDOT Pavement Management life cycle strategies, a discount rate of 4% is used [13]. Table 6.12 below contains the total present cost of the pavement systems and the percent difference from the virgin mixture. Mixtures containing PG 64-22 binder and 30% RAP have a present worth 18.2-18.9% less than the virgin mixture. Mixtures containing PG 58-22 binder and 40% RAP have a present worth 34.1-36.0% less than the virgin mixture. Mixtures containing PG 52-28 binder and 40% RAP have a present worth 30.5-31.1% less than the virgin mixture.

Table 6.12 Total Present Cost of Pavement Systems

Pavement System	Surface Mixture	Base Mixture	Total Present Cost Per Mile	% Difference from Virgin Mixture
1	9.5 B 64 0	19.0 C 64 0	$519,056	--
3	9.5 B 64 30 W	19.0 C 64 30 W	$420,851	18.9
7	9.5 B 58 40 W	19.0 C 58 40 W	$342,316	34.1
10	9.5 B 52 40 W	19.0 C 52 40 W	$360,753	30.5
12	9.5 B 64 30 P	19.0 C 64 30 P	$424,803	18.2
16	9.5 B 58 40 P	19.0 C 58 40 P	$332,327	36.0
19	9.5 B 52 40 P	19.0 C 52 40 P	$357,403	31.1

6.3.8 Annual Value (Costs) of Pavement Systems

The annual cost of a present worth can be calculated by:

$$A = PW * \left[\frac{i(1+i)}{(1+i)^N - 1} \right]$$

where,

A = annual uniform cost,

PW = present worth of a capital investment,

N = number of years in the analysis period, and,

i = discount rate.

Again, the discount rate of 4% was applied to calculate the annual cost of the pavement systems. Table 6.13 below contains the Annual cost of the pavement systems. The annual cost per year for the duration of the 30 year analysis period ranges from $6,162 for the mixture containing PG 58-22 binder and 40% RAP to $9,625 for the virgin mixture.

Table 6.13 Annual Cost of Pavement Systems

Pavement System	Surface Mixture	Base Mixture	Total Present Cost Per Mile
1	9.5 B 64 0	19.0 C 64 0	$9,625
3	9.5 B 64 30 W	19.0 C 64 30 W	$7,804
7	9.5 B 58 40 W	19.0 C 58 40 W	$6,348
10	9.5 B 52 40 W	19.0 C 52 40 W	$6,690
12	9.5 B 64 30 P	19.0 C 64 30 P	$7,877
16	9.5 B 58 40 P	19.0 C 58 40 P	$6,162
19	9.5 B 52 40 P	19.0 C 52 40 P	$6,627

6.4 Modeling and Economic Analysis Summary and Conclusions

This chapter predicted the performance of pavements against fatigue cracking and rutting using the results from the FSTCH and RSSTCH. The surrogate models from the SHRP A-003A as well as distress models developed by the AI were used to perform fatigue and rutting analysis. The results from this analysis were then used to perform an economic analysis.

An economic analysis for the life cycle cost of the pavements was conducted in order to take into account more than the initial difference in cost between pavements containing RAP material versus pavements containing 100% virgin materials. Based on the binder rheological properties stated in Chapter Four and the pavement performance of this chapter, the economic analysis compared virgin mixtures, PG 64-22 mixtures containing 30% RAP and PG 58-22 and PG 52-28 mixtures containing 40% RAP.

The pavement performance analysis for fatigue life and rutting shows that fatigue cracking is the limiting distress for the pavement service life as the rut life is able to handle many more loading repetitions. Also as stated previously, the AI model fatigue life analysis was more conservative and appropriate for this analysis. As a result, the estimated initial service life of the pavement systems were based on the AI model fatigue life calculations. The service life varies from 8.5 years to 16.2 years depending on the materials of the mixture.

In order to determine the initial construction cost and rehabilitation costs, the costs per ton for the asphalt concrete mixtures were obtained from the NCDOT 2008 bid averages. The RAP screening and processing cost is an average cost from asphalt plants throughout the state. These values were used in a present cost comparison and annual cost comparison of the pavement systems. Based on these analyses, the following results have been concluded:

6.4.1Fatigue Life Conclusions

- The general trend is that as the percentage RAP increases in the mixtures, the fatigue life increases, all else being constant.
- All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the fatigue life is reduced.
- Maintaining the original binder grade of PG 64-22, the fatigue life increases 26.2% with 30% Westgate RAP and 99.2% with 40% Westgate RAP and the fatigue life increases 16.2% and 75.1% with 30% and 40% Pineville RAP, respectively.
- By reducing the binder grade one grade to PG 58-22, the fatigue life reduces 14.9% with 30% Westgate RAP and 0.7% with 40% Westgate RAP and the fatigue life reduces

32.5% and 29.4% with 30% and 40% Pineville RAP, respectively. This reduction is due to the reduced stiffness of the reduced binder grade.

- By reducing the binder grade one grade to PG 52-28, the fatigue life reduces 47.2% with 30% Westgate RAP and 32.6% with 40% Westgate RAP and the fatigue life reduces 42.4% and 33.9% with 30% and 40% Pineville RAP, respectively.

6.4.2 Rut Life Conclusions

- According to the SHRP model, maintaining the original binder grade of PG 64-22, the rut depth decreases 14.3% and 44.9% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut depth decreases 30.18% and 63.40% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the SHRP model, by reducing the binder grade one grade to PG 58-22, the rut depth increases 20.0 % and 11.72% with 30% Westgate RAP and Pineville RAP, respectively. The rut depth decreases 6.18% and 24.64% for mixtures containing 40% Westgate RAP and Pineville RAP, respectively.
- According to the SHRP model, by reducing the binder grade two grades to PG 52-28, the rut depth increases for mixtures containing Westgate RAP 41.9% and 32.2% for mixtures containing 30% and 40% Rap, respectively, and mixtures containing Pineville RAP 55.2% and 45.4% for mixtures containing 30% and 40% RAP, respectively.
- According to the AI model, the results show that maintaining the original binder grade of PG 64-22, the rut life increases 41.5% and 210.7% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life increases 22.4% and 141.1% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the AI model, by reducing the binder grade one grade to PG 58-22, the rut life decreases 50.1 % and 8.6% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 54.6% and 31.4% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the AI model, by reducing the binder grade two grades to PG 52-28, the rut life decreases 71.6% and 55.3% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 66.2% and 56.3% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.

6.4.3 Economic Analysis Conclusions

- Mixtures containing PG 64-22 binder and 30% RAP have a present worth 18.2-18.9% less than the virgin mixture. Mixtures containing PG 58-22 binder and 40% RAP have a present worth 34.1-36.0% less than the virgin mixture. Mixtures containing PG 52-28 binder and 40% RAP have a present worth 30.5-31.1% less than the virgin mixture.
- The use of mixtures containing one binder grade lower (PG 58-22) than the original virgin binder grade (PG 64-22) and 40% RAP provides the most economical pavement.

Chapter 7

Summary of Results and Conclusions

The recycling of asphalt pavements has become a very routine procedure throughout the country. Research has shown that the Recycled Asphalt Pavement (RAP) recovered from construction sites still contains usable materials, both in the recycled aggregates and recycled binder. The use of RAP in construction of new asphalt pavements has become more prevalent over the years. Specification limits make it cost prohibitive for Contractors to use higher RAP contents in their mixes. This practice has led to vast quantities of RAP going unused and stockpiled. NCDOT has a long, successful history using RAP in HMA that dates back to the 1970s. Therefore, RAP's history is known when used in limited amounts. Research was needed to show that RAP materials can be used successfully in higher percentages.

In order to investigate the effects of RAP in the design and performance of new asphalt concrete mixtures, the specific research objectives were to:

- Evaluate the performance of mix designs using higher RAP percentages
- Determine which layers of the pavement structure could contain higher percentage of RAP without any significant reduction in performance life
- Perform a life cycle cost analysis showing the cost savings that could be realized if higher percentages of RAP are allowed.

7.1 Binder Rheology Conclusions:

- The addition of 30% RAP increases the stiffness of the binder grade. To determine the highest temperature grade for unaged binders, determine the highest temperature that $G^*/\sin\delta \geq 1.0$ kPa. PG 52 binders become PG 64, PG 58 binders become PG 70 and PG 64 binders become PG 76. This shows that the addition of 30% RAP increases the original binder by two grades.

- To determine the highest temperature grade for RTFO aged binders, determine the highest temperature that $G^*/\sin\delta \geq 2.2$ kPa. The PG 52 Pineville blended binder becomes PG 58, the PG 52 Westgate blended binder becomes PG 64, the PG 58 Pineville blended binder becomes PG 70, the PG 58 Westgate blended binder becomes PG 76 and the PG 64 binders become PG76. This shows that the addition of 30% RAP increases the original binder by at least two grades, with the exception of the PG 52 Pineville binder blend.
- The performance grade of the binder can be determined by selecting the temperature that satisfies the $G^*\sin\delta \leq 5000$ kPa requirement for PAV aged binders. The PG 52 Westgate blend and PG 58 binders satisfy the requirement at 22^{o}C while the PG 52 Pineville blend and PG 64 binders satisfy the requirement at 25^{o}C. The PG 58 and PG 64 binders passing at these temperatures translate to a low binder grade of -22, the same as the virgin binder. However, the PG 52 Pineville blend and PG 52 Westgate blend pass at temperatures that translate to a lower binder grade of -10 and -16, respectively.
- The addition of 30% RAP to the PG 52-28 virgin binder resulted in an increase in creep stiffness of 95.9% and 90.3% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 58-22 virgin binder resulted in an increase in creep stiffness of 53.7% and 59.9% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 64-22 virgin binder actually resulted in a decrease in creep stiffness of 7.3% and 0.3% for Westgate and Pineville blends, respectively. This result is not expected.
- As the virgin binder grade in the blend increases from PG 52-28 to PG 64-22, the increase in creep stiffness becomes less. This is expected since as the virgin binder creep stiffness increases with increasing binder grade, the higher creep stiffness from the reclaimed RAP binder has less of an effect on the binder blend.
- The addition of 30% RAP to the PG 52-28 virgin binder resulted in a decrease in m-value of 21.4% and 20.3% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 58-22 virgin binder resulted in a decrease in m-value of 11.5% and 10.0% for Westgate and Pineville blends, respectively. The addition of 30% RAP to the PG 64-22 virgin binder actually resulted in a decrease in m-value of 10.5% and 9.0% for Westgate and Pineville blends, respectively. This reduction in m-value is expected since

the binder reclaimed from the RAP is stiffer, and thus, likely to need more time to recover, which is what the m-value signifies.

- Mixtures are able to satisfy the creep stiffness specification with 91%-100% RAP. However, the controlling variable for thermal cracking from the BBR test is the m-value. Mixtures with PG 64-22 binder are able to contain 41%-50% RAP. Mixtures with PG 58-22 binder are able to contain 64%-69% RAP. Mixtures with PG 52-28 binder are able to contain 67%-71% RAP.
- Using virgin PG 64-22 binder, the maximum amount of RAP is 24%-27%. Using PG 58-22 binder, the maximum amount of RAP is 37%. Using PG 52-28 binder, the maximum amount of RAP is 28%. However, for Pineville RAP, the minimum amount of RAP needed is 30%. As the properties of RAP are variable depending on previous use, mixture etc., it is not recommended to use PG 52-28 binder in RAP mixtures in order to meet PG 64-22 specifications.

7.2 Mixture Characterization Conclusions

- The results from the FSTCH indicate trends of increasing complex modulus with increasing frequency. Also, as the binder grade increases, from PG 52-28 to PG 64-22, the complex modulus increases. This would be expected as the complex modulus of PG 64-22 binder is larger than the complex modulus of PG 52-28 binder when temperature is constant.
- The complex modulus of mixtures with higher percentages of RAP increases while holding the binder grade constant. Again, this can be expected since the complex modulus of the RAP binder is greater due to its aging caused during its service life.
- After 5000 cycles of repeated shear, the mixtures containing PG 64-22 binder exhibited the least amount of strain for both virgin 9.5 mm mixtures and 19.0 mm mixtures. Mixtures with PG 58-22 binder exhibited less strain than the mixtures with PG 52-28 binder. This can be expected as mixtures with stiffer binders exhibited less strain.
- Comparing mixtures with various percentages of RAP with constant binder grade shows general trends that the strain decreases with increasing RAP. Again, this can be expected as the binder recovered from the RAP was stiffer than the virgin binders due to aging

during its service life. Therefore, mixtures with higher percentages of RAP are going to generally exhibit less strain as a result.

- For 9.5 mm mixtures containing Westgate RAP, the amount of plastic shear strain reduces for all PG 64-22 mixtures containing RAP varying from 7% to 45% reduction. The PG 58-22 mixtures containing 15% and 30% RAP result in the plastic shear strain increases of 28% and 21%, respectively. This increase is caused by the reduction in binder stiffness of the PG 58-22 binder from the original PG 64-22 virgin binder. However, this reduction in virgin binder stiffness is overcome when 40% RAP is included in the mix and the plastic shear strain reduces 6% from the virgin mixture. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 100%, 42% and 33% for mixtures containing 15%, 30% and 40% RAP, respectively.
- For all binder grades, as the percentage of RAP increases, the plastic shear strain reduces. This is because the RAP binder has a higher complex modulus than the virgin binder and results in a stiffer blend.
- For 9.5 mm mixtures containing Pineville RAP, the amount of plastic shear strain reduces for all PG 64-22 mixtures containing RAP varying from 22% to 64% reduction. The PG 58-22 mixtures containing 15% and 30% RAP result in the plastic shear strain increases 53% and 12%, respectively. This increase is caused by the reduction in binder stiffness of the PG 58-22 binder from the original PG 64-22 virgin binder. However, this reduction in virgin binder stiffness is overcome when 40% RAP is included in the mix and the plastic shear strain reduces 24% from the virgin mixture. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 126%, 55% and 46% for mixtures containing 15%, 30% and 40% RAP, respectively.
- For 19.0 mm mixtures containing Wesgate RAP, the amount of plastic shear strain reduces for all PG 64-22 mixtures containing RAP varying from 27% to 35% reduction. The PG 58-22 mixtures containing 15% RAP result in the plastic shear strain increase of 8%. This increase is caused by the reduction in binder stiffness of the PG 58-22 binder from the original PG 64-22 virgin binder. However, this reduction in binder stiffness is overcome when 30% and 40% RAP is included in the mix and the plastic shear strain reduces 15%

and 19%, respectively, from the virgin mixture. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 23%, 14% and 13% for mixtures containing 15%, 30% and 40% RAP, respectively.

- For 19.0 mm mixtures containing Pineville RAP, the amount of plastic shear strain reduces for all PG 64-22 mixtures containing RAP varying from 10% to 38% reduction. With a reduction of one binder grade from the original virgin binder to a PG 58-22 binder, the plastic shear strain increases 37%, 27%, and 19% for mixtures containing 15%, 30%, and 40% RAP, respectively. With the reduction of two binder grades from the original virgin binder to a PG 52-28 binder, the plastic shear strain increases 56%, 39% and 34% for mixtures containing 15%, 30% and 40% RAP, respectively.

7.3 Pavement Performance Conclusions

- The pavement performance analysis for fatigue life and rutting showed that fatigue cracking was the limiting distress for the pavement service life as the rut life is able to handle many more loading repetitions. The Asphalt Institute fatigue life analysis was more conservative and appropriate for the analysis. As a result, the estimated initial service life of the pavement systems were based on the Asphalt Institute fatigue life calculations. The service life varies from 10.9 years to 16.2 years depending on the materials of the mixture.
- The general trend is that as the percentage RAP increases in the mixtures, the fatigue life increases, all else being constant.
- All else being held constant, as the binder grade is reduced from PG 64-22 to PG 52-28, the fatigue life is reduced.
- Maintaining the original binder grade of PG 64-22, the fatigue life increases 26.2% with 30% RAP and 99.2% with 40% RAP for mixtures containing Westgate RAP and the fatigue life increases 16.2% and 75.1% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- By reducing the binder grade one grade to PG 58-22, the fatigue life reduces 14.9% with 30% RAP and 0.7% with 40% RAP for mixtures containing Westgate RAP and the fatigue life reduces 32.5% and 29.4% with 30% and 40% RAP, respectively, for mixtures

containing Pineville RAP. This reduction is due to the reduced stiffness of the reduced binder grade.

- By reducing the binder grade two grades to PG 52-28, the fatigue life reduces 47.2% with 30% RAP and 32.6% with 40% RAP for mixtures containing Westgate RAP and the fatigue life reduces 42.4% and 33.9% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the SHRP model, maintaining the original binder grade of PG 64-22, the rut depth decreases 14.3% and 44.9% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut depth decreases 30.18% and 63.40% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the SHRP model, by reducing the binder grade one grade to PG 58-22, the rut depth increases 20.0 % and 11.72% with 30% Westgate RAP and Pineville RAP, respectively. The rut depth decreases 6.18% and 24.64% for mixtures containing 40% Westgate RAP and Pineville RAP, respectively.
- According to the SHRP model, by reducing the binder grade two grades to PG 52-28, the rut depth increases for mixtures containing Westgate RAP 41.9% and 32.2% for mixtures containing 30% and 40% Rap, respectively, and mixtures containing Pineville RAP 55.2% and 45.4% for mixtures containing 30% and 40% RAP, respectively.
- According to the Asphalt Institute model, the results show that maintaining the original binder grade of PG 64-22, the rut life increases 41.5% and 210.7% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life increases 22.4% and 141.1% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the Asphalt Institute model, by reducing the binder grade one grade to PG 58-22, the rut life decreases 50.1 % and 8.6% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 54.6% and 31.4% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.
- According to the Asphalt Institute model, by reducing the binder grade two grades to PG 52-28, the rut life decreases 71.6% and 55.3% with 30% and 40% RAP, respectively, for mixtures containing Westgate RAP and the rut life decreases 66.2% and 56.3% with 30% and 40% RAP, respectively, for mixtures containing Pineville RAP.

7.4 Economic Analysis Conclusions

- Comparing the cost per ton of the mixtures containing RAP to the cost per ton of virgin mixtures, both the surface and base course show roughly a 5.5%, 11% and 14.5% reduction in cost per ton for 15%, 30% and 40% RAP, respectively.
- By comparing the material cost per mile of the virgin mixtures to the mixtures containing various percentages of RAP shows that the inclusion of 15% RAP decreases the cost about 13% while 30% RAP reduces the cost about 26% and the inclusion of 40% RAP reduces the cost about 34%.
- Mixtures containing PG 64-22 binder and 30% RAP have a present worth 16.9%-17.7% less than the virgin mixture. Mixtures containing PG 58-22 binder and 40% RAP have a present worth 32.4%-34.4% less than the virgin mixture.
- The use of mixtures containing one binder grade lower (PG 58-22) than the original virgin binder grade (PG 64-22) and 40% RAP provides the most economical pavement.

7.5 General Conclusions

- The general trend is that as the percentage RAP increases in the mixtures, the fatigue life and rut life increase, all else being constant.
- Mixtures containing PG 64-22 binder and 30% RAP have a present worth 16.9%-17.7% less than the virgin mixture. Mixtures containing PG 58-22 binder and 40% RAP have a present worth 32.4%-34.4% less than the virgin mixture.
- The use of mixtures containing one binder grade lower (PG 58-22) than the original virgin binder grade (PG 64-22) and 40% RAP provides the most economical pavement.

7.6 Recommendations For Future Research

The following future research is recommended:

- Investigation of how the use of Warm Mix Asphalt (WMA) technology affects the amount of RAP allowable in the mixture. Determine how the decreased mixing temperatures affect the blending of the RAP binder with the virgin binder.
- Investigation of modifiers to add to mixtures to increase the amount of allowable RAP. The limiting factor to the amount of RAP allowable in mixtures is the specification for $G^*\sin\delta$

$\leq$ 5000 kPa. Determine if there is an additive to the mixture that can reduce the $G^*\sin\delta$ value of the blends containing RAP so that more RAP can be used.

7.7 Implementation and Technology

No training will be needed to use the products of the research and the results could be implemented readily into practice.

Chapter 8
References

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