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Research

Investigation Of Recycled Asphalt Pavement (RAP) Mixtures

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16. Abstract (Limit: 200 words)

This report presents the results of a study on recycled asphalt pavement (RAP) mixtures. The report includes a literature review on RAP and a summary of past research.

The research focuses on the use of resilient modulus and complex modulus testing to compare mixtures compacted with only virgin materials to those compacted with varying amounts of RAP.

The additional of RAP makes the mixture stiffer, as evidenced by an increase in resilient modulus and complex modulus measurements, according to the research. The addition of RAP also decreases the mixture phase angle, which corresponds to an increase in the elastic properties and a decrease in the viscous mixture properties.

The report includes recommendations regarding the proper temperatures and loading frequencies for use in the complex modulus test. It also gives the percentage of RAP and the respective asphalt binder grade necessary to yield the stiffness similar to a virgin mixture.

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INVESTIGATION OF RECYCLED ASPHALT PAVEMENT (RAP) MIXTURES

Final Report

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EXECUTIVE SUMMARY

This report presents research findings from the Investigation of Recycled Asphalt Pavement (RAP) Mixtures project. The objectives of this report were to:

1. Describe the RAP material used in the project.
2. Describe the test methods used and the results obtained from the testing.
3. Provide conclusions and recommendations relating to the amount of RAP and the asphalt binder grade which should be used to achieve properties similar to a virgin asphalt concrete mixture.

The RAP material was characterized in a number of ways. Asphalt content was determined by the ignition oven method. RAP aggregate gradation was determined before and after the asphalt binder was removed by solvent extraction. A dynamic shear rheometer (DSR) was used to determine the RAP binder Superpave performance grade after solvent extraction The properties of the RAP aggregate were also evaluated by determining the specific gravity, absorption, and fine aggregate angularity (FAA).

Samples were compacted in the laboratory using a Superpave gyratory compactor. The samples contained from 0 to 40 percent RAP from either MnDOT District 6 or District 8, and either PG 58-28, PG 52-34 or PG 46-40 virgin asphalt binder. RAP material was blended with virgin aggregate such that all samples tested had approximately the same gradation.

Samples were tested for resilient modulus, complex modulus, and moisture sensitivity. The resilient modulus test provides a measure of the elastic properties of the mixture, and allows for comparison with testing performed by other researchers. Complex modulus tests make it possible to determine both the elastic and viscous properties of the mixtures. Moisture sensitivity tests were conducted to determine how durable or susceptible to moisture related problems the mixtures were.

Software developed at the University of Minnesota was used to perform the complex modulus test using the indirect tensile test (IDT) setup. The data obtained form the test was used as inputs for viscoelastic equations derived for the IDT test geometry. Some problems were experienced with the complex modulus test equipment, and as a result the table below contains recommendations relating to testing temperature and frequency.

Recommended Complex Modulus Test Parameters

Test Temperature, °C	Minimum Loading Frequency, Hz	Maximum Loading Frequency, Hz
-18	0.03	5.0
1	0.03	5.0
25	0.03	5.0
32	0.1	5.0

Based upon resilient modulus and complex modulus test results, the RAP contents and respective asphalt binders shown below will result in a stiffness similar to a virgin mixture:

Recommended RAP Contents and Asphalt Binders

Original Asphalt Grade	Asphalt Grade with RAP	RAP Content with District 6 RAP	RAP Content with District 8 RAP
PG 58-28	PG 52-34	20 %	10 %
PG 58-28	PG 46-40	50 %	35 %
PG 52-34	PG 46-40	25 %	15 %

However, it is important to note that additional testing is recommended to verify these mixtures will have adequate performance in the field. The low temperature cracking potential of these mixtures should be evaluated prior to use.

CHAPTER 1
- INTRODUCTION -

BACKGROUND

Economic and environmental considerations have prompted the recycling of steel, aluminum, plastic, and many other materials. One of these recyclable materials is hot mix asphalt (HMA). According to Taylor (1), the concept of HMA recycling was documented as far back as 1915, though it did not gain popularity until the oil embargo of the mid-1970's. Demand for HMA recycling was driven by increased costs for asphalt, coupled with the scarcity of quality aggregates near the point of utilization. These economic incentives still exist, and environmental incentives to recycle are also prominent.

It is estimated that tens of millions of tons of recycled asphalt pavement (RAP) have been used since the mid-1970's, with a substantial cost savings over virgin HMA mixes and similar performance characteristics. According to an Sullivan (2), 33 percent of all asphalt concrete pavement is recycled into HMA.

Decker (3) lists the five recycling methods as defined by the Asphalt Recycling and Reclaiming Association: cold planing, hot recycling, hot in-place recycling, cold in-place recycling, and full depth reclamation. Hot recycling, the focus of this study, combines RAP with virgin asphalt and/or aggregate to produce HMA. Either a batch or drum type hot mix plant may be used to produce the recycled mix, and the mix is placed and compacted in the same way as virgin HMA. A study by Johnson and Han (4) showed that hot mix recycling was the most common method of asphalt concrete recycling in Minnesota.

The development of the Superpave mixture design procedure brought a new asphalt binder selection and mixture design methodology to the asphalt paving industry (see Asphalt Institute (5)). Superpave was developed for virgin asphalt-aggregate mixtures with no consideration for recycled mixtures. However, economic and environmental concerns dictate the use of recycled materials in paving. Thus, in order to use the Superpave system, it is necessary to develop a modified methodology, which allows the incorporation of RAP in the mixture.

RELATIONSHIP TO PREVIOUS WORK

Development of the Superpave mixture design method began in 1987, but it did not include a framework for incorporation of RAP. According to Bukowski (6), in March of 1997 the Superpave Mixtures Expert Task Group drafted guidelines for the design of Superpave mixtures containing RAP. The guidelines suggest that Superpave mixtures containing RAP should generally follow the same mix design requirements as conventional mixtures with a modification for binder grade selection. This research used the Superpave method as the foundation for the mix design procedure, and modified it to include recycled asphalt concrete.

Research from this work was compared to past work. The results obtained for complex modulus, resilient modulus, and moisture sensitivity tests are compared to results obtained by past researchers for similar tests.

OBJECTIVE

There were two main objectives to this research. The first involved sampling RAP stockpiles from around the state to characterize typical Minnesota RAP gradation and binder properties. The second objective was to develop a mix design methodology, using the Superpave approach, to proportion the materials in mixtures containing RAP. This methodology was arrived at by performing moisture sensitivity, resilient modulus, and complex modulus testing. The indirect tension test was used to compare virgin and recycled mixture properties, with the complex modulus test forming the test program core. Software developed by Zhang (7) at the University of Minnesota was used to perform the test and determine the elastic and viscous properties of the mixtures.

Complex modulus testing allowed the mixture analysis to be conducted on a rheological basis. Both elastic and viscous properties of virgin and recycled mixtures were examined. A recycled mixture was considered acceptable when its properties were similar to those of a mixture composed entirely of virgin material.

SCOPE

A rheological approach employing previously developed diametral test methods was used for the evaluation of the recycled mixtures. While the resilient modulus test for asphalt concrete is well established, the complex modulus test is not. Previous complex modulus work by Zhang (7) was extended to include testing at a range of frequencies and temperatures, which enabled determination the mixture temperature susceptibility. In addition, Zhang's work was based upon the geometry of a 100 mm diameter sample. This research uses a 150-mm diameter sample, as required by Superpave. Therefore, it was necessary to validate that the software, test setup, so that the equations developed could be applied to a 150-mm sample.

A testing matrix was constructed for mixtures containing 0, 15, 30, and 40 percent RAP, and various grades of virgin asphalt. The matrix was designed to determine the amount of RAP and the grade of virgin asphalt binder which may be added to the mixture in order to yield properties considered acceptable for a mixture composed entirely of virgin materials.

ORGANIZATION OF REPORT

This report has five sections: Introduction, Literature Review, Research Methodology, Results and Discussion, and Conclusions and Recommendations. The literature review provides a background of RAP characteristics, the Superpave mix design method, and methods for testing mixtures composed of RAP and virgin materials. Research methodology discusses laboratory procedures including mixture design, sample preparation, and the test methods used to evaluate the virgin and recycled mixtures. The results of all laboratory data are presented and discussed. Conclusions and recommendations based upon the results of the test data are provided. Literature sources used for background information are cited in the bibliography, and several appendices present details of RAP properties, mixture design, testing procedures, and raw data.

CHAPTER 2
- LITERATURE REVIEW –

RAP CHARACTERISTICS

Asphalt Binder

Binder Quantity

When performing a mix design to incorporate RAP, it is desirable to know the asphalt characteristics and content and the aggregate gradation of the RAP. Before the aggregate gradation can be determined, the binder and the aggregate must be separated. There are a number of methods that have been developed to separate the aggregate from the binder and/or determine the binder content. These include: solvent extraction, nuclear asphalt content gauge (NAC), pycnometer method, automatic recordation, and the ignition method. Zhang (8) states that of the above methods, only solvent extraction and the ignition oven permit determination of binder content and aggregate gradation, both of which are required when a RAP mixture is being designed.

Solvent Extraction

Traditionally, the solvent extraction methods (centrifuge, reflux, and vacuum) were performed using methylene chloride, trichloroethylene or 1,1,1-trichloroethane, which dissolves and separates the asphalt from the mineral aggregate. In a study conducted for NCHRP, Peterson et al. (9) recommend the centrifuge method because of the possibility that the reflux method causes increased aging of the extracted binder and the lack of widespread use of the vacuum method. Peterson et al. (9) cite two existing American Society for Testing and Materials (ASTM) methods of centrifuge extraction: ASTM D1856, *Recovery of Asphalt from Solution by Abson* Method and ASTM D5404, *Recovery of Asphalt Using the Rotavapor Apparatus*. The latter method was modified by SHRP to reduce aging of the binder, resulting in the AASHTO TP2 method. Peterson et al. (9) state that the Asphalt Institute has further modified the TP2 system to improve accuracy and streamline the procedure. Warren and Rugg (10) describe a procedure for calculating the asphalt content from the masses of the original sample, extracted aggregate, moisture in the sample, mineral filler in the extract, and the mass of filler collected on the filter.

The solvents used in solvent extraction have been shown by Brown and Murphy (11) to be both carcinogenic and ozone-depleting. Various biodegradable solvents have been developed to overcome this problem. Colgrave and Tredrea (12) reference a 1996 study by the Texas Department of Transportation that showed the biodegradable solvent BioAct performed with accuracy very similar to trichloroethylene. However, say Brown and Murphy (11), the use of a biodegradable solvent makes the test more complicated and time consuming, and, as Zhang (8) states, the disposal of biodegradable solvents is no cheaper than the disposal of trichloroethane. Anderson et al. (9) recommend using an n-Propyl Bromide based solvent to minimize the impact on the environment.

An additional drawback of the solvent extraction method is the high standard deviation of the test. Brown and Murphy (11) report a standard deviation in asphalt content of 0.21 percent. Another source of error in this test lies in the absorptive characteristics of aggregate. Warren and Rugg (10) report that about 0.3 percent of the asphalt cement by mass will be not be extracted by solvent extraction and will be retained in the aggregate pore structure. Despite its drawbacks, one benefit of the solvent method is the ability for residual asphalt to be retained for future testing (viscosity, penetration testing, etc.) and it was the best method for determining asphalt content until the advent of the ignition oven.

Ignition Oven

The development of the ignition method for determining AC content dates back to 1969, when Antrim and Busching (13) placed HMA samples into an oven at 843 °C (1550 °F) with an excess of oxygen. The authors reported that at this temperature burning of the aggregate could result in aggregate mass loss. In fact, although granite gneiss lost a negligible amount of mass, it was shown that limestone can lose up to 30 percent of its mass after one hour of heating.

Yu (14) continued this work in 1992 by conducting asphalt content studies with a muffle furnace. Yu used a temperature of 593 °C (1100 °F), which reduced the effect of aggregate type and increased the accuracy of the test. Eventually it was determined a temperature of 538 °C (1000 °F) minimized both aggregate mass loss and the time requirement for each test. The temperature reduction yielded accuracy comparable to a centrifuge extraction.

In the ignition method, the mass of the asphalt concrete sample is determined before ignition and the sample is placed in the oven at 538 °C (1000 °F) until all of the asphalt is burned off, which, according to Brown and Mager (15), generally takes between 30 and 40 minutes for a 1,200-gram sample. The latest generation ignition ovens (such as the NCAT oven) are equipped with internal scales, so the test may be stopped when the sample mass no longer decreases. The mass of the sample after ignition is determined and the difference of these two masses is divided by the mass of the sample before ignition to obtain the asphalt content as a percent.

According to Zhang (8), the major source of error in the ignition method is mass loss of the aggregate, which is attributed to the combustion of the mineral aggregate, as well as the loss of fine particles caused by convection during the ignition process. This problem is often addressed by determining a correction factor, which is dependent on the type of aggregate in the HMA sample. Brown and Mager (15) describe a procedure for determining the correction factor by placing an aggregate-only sample into the ignition oven and measuring the mass loss. The mass loss of an aggregate-only sample is approximately the same as in an HMA sample with that aggragate.

In 1996, the National Center for Asphalt Technology (NCAT) published the results of a round-robin study conducted by Brown and Mager (15) that determined the accuracy and the precision of the ignition method. Samples using four types of aggregate and one type of binder were sent to 12 different laboratories across the country, and the results were compared. The overall difference of the measured and actual AC content for 192 samples was –0.02 percent. Not only did the study confirm the ignition method was accurate and precise, it also showed the ignition method to be more precise than the solvent extraction method (in-laboratory standard deviations of 0.04 percent versus 0.21 percent). The round-robin study also showed that while mass loss of aggregate was occurring, the ignition method caused only a negligible change in the gradation of the sample. Additional work supporting the NCAT results has been performed by the Texas Department of Transportation and by the Australian Roads Research Board.

Though the NCAT study showed the ignition method was accurate for HMA made from a single aggregate, it did not address RAP or HMA made from multiple sources of aggregate. Zhang (8) describes a study conducted by the Minnesota Department of Transportation (Mn/DOT) in 1996 addressing:

1. Accuracy of the ignition method for RAP mixtures
2. Estimation of a combined correction factor
3. Equivalency between mixture calibration and aggregate only calibration
4. Effect of ignition on aggregate gradation
5. Time required to remove moisture from the sample

This study developed an estimated combined aggregate correction factor, which was shown to be within 0.05 percent of the predicted combined weight loss factor. Using the combined correction factor, the accuracy of the ignition test for virgin and RAP mixtures was shown to be about 0.11 percent at a 90 percent confidence level. The study also supported previous work stating the ignition method did not significantly affect the aggregate gradation. It was recommended that the 2,000 gram samples be placed in a 110 °C (230 °F) oven for at least 40 minutes prior to the ignition test in order to remove most of the moisture from the sample.

Binder Aging

Bell (16) states that the aging, or age hardening, of asphalt binder causes an increase in binder viscosity, and subsequent stiffening of the asphalt mixture. Vallerga (17) determined that this can result in an excessively hard and brittle behavior, and a mixture with increased susceptibility to disintegration and cracking failure. In addition, according to Barth (18), *in situ* aging can decrease mixture wear resistance and increase moisture susceptibility, yielding a less durable mixture.

Asphalt cements exhibit two stages of aging: short term and long term. According to Bell (16), short term aging is mainly due to volatilization, and is a product of heating during mixing or construction, while long term aging occurs in-situ and is predominantly caused by oxidation. Consideration of the effects aging has on asphalt binder is important when RAP is involved since the binder has already been age-hardened prior to construction.

The physical effects of aging described above are caused by chemical changes within the binder. Petersen (19) lists the three main factors of asphalt cement aging as follows:

1. Loss of oily components by volatility or absorption
2. Changes in composition by reaction with atmospheric oxygen
3. Molecular structuring that produces thixotropic effects (steric hardening)

Though Traxler (20) lists 15 effects that may reduce asphalt binding properties, Bell (16) reports that the majority of research on asphalt aging is limited to the above factors.

Physical changes in asphalt binder are brought about by changes in its chemical composition. Petersen (19) lists three methods of determining asphalt composition by fractionation: partitioning with partial solvents as described by Schweyer and Traxler (21); the chemical precipitation method from Rostler and White (22); and the selective adsorption-desorption method described by Corbett (23). Of the three methods, selective adsorption-desorption is reported to have been used the most in research. It involves separating the binder into its two major component groups, asphaltenes and maltenes, in a non-polar paraffininc solvent (e.g. n-heptane). The asphaltene phase is insoluble in the solvent, while the maltene phase solubilizes. The maltene phase can be further separated by adsorption on a chromatographic column into saturates, napthene aromatics, and polar aromatics.

A good deal of research has been performed on the relationship of asphaltenes and maltenes and how they relate to aging. Corbett (23) found that as asphalt ages, some of the maltene phase is transformed into the asphaltene phase, thereby increasing the asphaltene content and decreasing the maltenes. The result is that there are fewer maltenes available to disperse the asphaltenes. The asphaltene phase will flocculate without the presence of enough maltenes for dispersion, leading to increased viscosity and decreased ductility. According to Rostler and White (22), the viscosity of asphalt and its colloidal nature are primarily due to the presence of asphaltenes. With regard to aging and asphalt performance, Petersen (19) writes:

> If during aging the concentration of polar functional groups (i.e. asphaltenes) becomes sufficiently high to cause molecular immobilization through increased intermolecular interaction forces, that is, the asphalt molecules or micelles are not sufficiently mobile to flow past one another under the stress applied, fracturing or cracking of the asphalt will result.

Though authors such as Corbett, Rostler, and Petersen have performed research on asphalt fractions, most physical testing is performed on the binder as a whole, and is used to estimate binder quality.

Binder Quality Tests

According to Bullin et al. (24), aging of asphalt binder can have a large impact on the quality of the binder, as well as the mix. Aging increases the viscosity, stiffness, and viscosity temperature susceptibility. In addition, penetration and ductility are decreased. These factors combine to result in a harder, more brittle asphalt cement.

The quality of an asphalt binder is often determined by its durability. Petersen (19) describes durability as:

1. Possessing the physical properties necessary to produce the desired initial product performance properties
2. Having resistance to change in physical properties during long-term in-service environmental aging.

A number of laboratory procedures have been developed to examine binder durability and simulate the field aging of an asphalt binder. Superpave currently uses two of these methods, the Rolling Thin Film Oven Test (RTFO) (ASTM D2872) and the Pressure Aging Vessel (PAV). The RTFO simulates short term aging due to heating. For this test, a predetermined amount of asphalt is put into a sample bottle and placed on the rack in the rolling thin film oven. The rack rotates at a specified rate, which maintains a thin film of fresh asphalt. With each rotation of the rack, a jet of heated air enters a hole in the sample bottle, purging the volatilized vapors from the bottle. Roberts et al. (25) report that in the past, some states have used the viscosity of the aged residue to grade paving asphalt cements.

The pressure aging vessel (PAV), developed in the Strategic Highway Research Program (SHRP), simulates the aging of asphalt binder that occurs in a five to ten year old in-service pavement. Since asphalt in an in-service pavement has also been exposed to short term aging, the asphalt residue from the RTFO is placed in the PAV and exposed to high temperature and pressure air for 20 hours. Following 20 hours of pressure aging, the aged asphalt is placed in a 163 °C (325 °F) oven for 30 minutes to remove entrapped air. Samples are then stored for testing by methods such as penetration, viscosity, dynamic shear rheometer, bending beam rheometer, or direct tension test.

Aging is sometimes quantified in terms of percent retained penetration, which is defined by Roberts et al. (25) as the penetration of aged asphalt divided by the penetration of the original asphalt. Penetration of asphalt binder has been correlated with cracking performance of HMA in the field. Hubbard and Gollomb (26) reported the following:

1. When the penetration of asphalt cement (measured at 25 °C) falls below 20, serious pavement cracking may occur.
2. Some cracking may occur when penetration is between 20 and 30.
3. High resistance to cracking may occur when a mixture is well designed and properly constructed and the penetration of the binder is well above 30.

Some states grade paving asphalts on the basis of the viscosity of aged residue from the RTFOT. According to Kandhal et al. (27), the Georgia Department of Transportation specifies that when blended with virgin asphalt cement and aged in the thin film oven test, the RAP binder should have a viscosity between 6,000 and 16,000 poises.

Ductility, the ability to stretch without breaking, is considered by some researchers to be an important property of asphalt binder. Roberts et al. (25) have shown that asphalt binders with acceptable penetrations and lower ductility will be poorer performers (with regard to cracking, raveling, etc.) than asphalt binders with similar penetrations and higher ductility.

In addition to the RTFO and PAV, Superpave also uses the dynamic shear rheometer (DSR) to characterize asphalt cement. The DSR is used to determine the complex shear modulus (G*) and the phase angle (d) of asphalt cement at high and intermediate service temperatures. The asphalt is repeatedly sheared in the DSR and the resistance of the binder to deformation is expressed in terms of complex modulus. Complex modulus has two components: the storage modulus (G’) or elastic portion, and the loss modulus (G’’) or viscous portion. The phase angle must also be determined because it is possible for two asphalt cements to have the same numerical complex modulus, yet exhibit different amounts of elastic and viscous behavior. Figure 2.1 illustrates the relationship between G*, G’, G’’ and d.

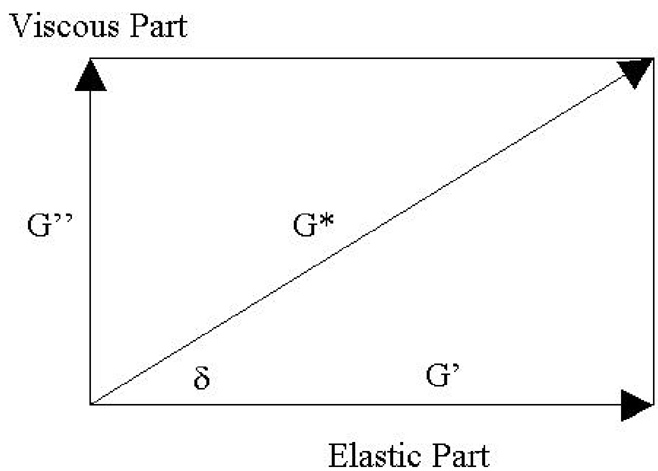

Figure 2.1 Complex Modulus Components

DSR data can be used to compare fatigue life and rutting susceptibility of recycled and virgin asphalt mixtures. A high complex modulus and low phase angle provide a stiff, yet elastic mix, which increases resistance to the permanent deformation visible in rutting. Roberts et al. (25) state that for increased fatigue life, a low complex modulus (less stiffness) and low phase angle are desirable.

As a result of the properties listed above, Superpave imposes requirements on G* and d. The rutting parameter, $G^*/\sin\delta$, is required to be a minimum of 1 kPa for unaged binder and 2.2 kPa for RTFO aged binder. The fatigue parameter requires $G^* \sin\delta$ to be a maximum of 5,000 kPa for PAV aged binder. These values have been set in an effort to maximize asphalt binder quality.

Binder Quality

A major issue with RAP mixtures is the question of durability. Will the effects of aging negatively influence performance, even when combined with virgin asphalt or with a rejuvenator? A Georgia study described by Kandhal et al. (27) evaluated five projects using both recycled and virgin mixtures. The project was designed so that the recycled and virgin sections contained the same virgin aggregates, were produced by the same plant, constructed by the same crews, and subjected to the same traffic and environment. The virgin mixtures used AC-30 asphalt, while the recycled mixtures used AC-20, AC-20S, or AC-30, with RAP contents varying from 10 to 25 percent.

The asphalt was extracted from six cores taken from each of the five project locations, varying in age from 1.5 to 2.25 years. Penetration, viscosity, and DSR testing were performed on the recovered asphalt. The penetrations of the binders in the recycled mixture and the virgin mixture were not found to be statistically different. The average penetration was 20. The average viscosity of the recycled mixture was less than the virgin mixture (4,688 Pa-s vs. 5,466 Pa-s at 60 °C), though these were not statistically different when a paired t-test was conducted. Additionally, there was no significant difference in the rutting and fatigue factors as determined by the DSR data. The Georgia study seems to indicate that RAP mixtures can be made which meet or exceed the performance of virgin mixtures.

Solaimanian and Tahmoressi (28) describe a research program was conducted in Texas to compare the variability of RAP mixtures to virgin HMA. The study examined four RAP projects, containing between 35 and 50 percent RAP. The materials from the RAP projects were compared to a control plant mix not containing RAP. The type of virgin asphalt binder used is not mentioned in the paper, but the study showed RAP had significantly higher viscosity and lower penetration, and was more variable than the control HMA.

A Louisiana study similar to the Georgia project is described by Paul (29). Five RAP projects and five virgin mixtures were compared, all ranging in age from 6 to 9 years. The recycled and virgin projects were paired in order to have their construction in the same time frame, by the same contractor if possible, similar mix design, section, and traffic. Current Louisiana specifications state that the an AC-30 asphalt is required for mixtures containing 20 percent or less RAP, while an AC-10 asphalt is to be used for mixtures containing between 20 and 30 percent RAP. Louisiana also specifies that recovered RAP binders shall not have a plant-produced viscosity which exceeds 12,000 poises. As in the Georgia study, the penetration and viscosity of the RAP binders showed no significant difference (using a paired t-test with 0.05 significance level) from the virgin asphalt binders. Ductility testing was also performed in this study, and no significant difference was found.

These research projects indicate that while RAP binders do have increased viscosity due to age hardening, this can be mitigated. When the binder is properly blended with a soft asphalt or other recycling agent, acceptable penetration, viscosity, and ductility can be obtained and the mixture can perform similar to a virgin asphalt mixture.

Rejuvenation

Before RAP material is recycled and again placed, it must be mixed with some form of recycling agent or asphalt to lower its viscosity and attempt to restore its properties to those of a virgin asphalt cement. Roberts et al. (25) report that most states use relatively low viscosity soft asphalt cements as recycling agents. Some states, especially in the western U.S., permit the use of softening agents or rejuvenating agents as well as soft asphalts. Softening agents consist of flux oils and lube oils, which lower the viscosity of the aged asphalt cement. Terrel and Epps (30) describe rejuvenating agents as a combination of lubricating oil extracts and extender oils, which contain a high proportion of the napthenic or polar aromatic fractions (maltene component), and are used to restore the chemical and physical properties of the aged asphalt cement. As mentioned previously, maltenes are necessary in order to keep the asphaltenes dispersed. Without the presence of enough aromatics, the binder will increase in viscosity, and decrease in ductility.

Not only should a recycling agent be high in aromatics, but it must also be compatible with the aged binder. Bullin et al. (24) point out that asphaltenes and saturates are highly incompatible, and as a result the recycling agent should have a low saturate content. Many researchers such as Davidson et al. (31), Dunning and Mendenhall (32), Epps et al. (33), and Peterson et al. (34) have suggested that a recycling agent should have a controlled composition. For example, Dunning and Mendenhall recommend that the agent have a minimum polar aromatic content of 9 percent, a minimum napthene aromatic content of 60 percent, and a maximum saturate content of 31 percent. More recent work by Peterson et al. (34) and Bullin et al. (24) support these conclusions.

A study in 1984 by Newcomb et al. (35) suggested that rejuvenating agents with a high polar to saturate ratio (P/S) yielded increased penetration and ductility and a decrease in the aging index of asphalt blends after aging in the RTFO. Additional work has been done by Noureldin and Wood (36) using rejuvenating agents and TFOT aging. Out of three recycled blends they tested, two yielded aging indexes lower than the virgin asphalt.

Aggregate

Gradation

With regard to the importance of aggregate in an HMA mixture, Roberts et al. (25) state:

> Gradation is perhaps the most important property of an aggregate. It affects almost all the important properties of a HMA, including stiffness, stability, durability, permeability, workability, fatigue resistance, frictional resistance, and resistance to moisture damage.

Paul (29) compared the gradations from the extracted cores of five recycled projects in Louisiana. After reviewing the original construction data they determined little or no degradation of materials had occurred. Other gradation research has also shown little to no change in the gradation of recycled aggregates when taken from core samples.

However, different results have been obtained when the aggregate gradation is determined from RAP materials after the pavement has been milled. Beam and Maurer (37) describe a 1991 Pennsylvania Department of Transportation RAP study which found that while core sampling was beneficial to identifying the existing pavement layers and their condition, they did not accurately represent the aggregate gradation after the milling process. On the six of the projects observed, the milling process resulted in a finer aggregate gradation than the cores indicated. Brownie and Hironaka (38) showed a similar reduction in aggregate gradation during a RAP crushing process. It was cited that the reduction in size is dependent upon the hardness of the aggregate.

A large amount of fines is detrimental as it can result in a thin asphalt film thickness, which has been associated with poor mixture durability. The size reduction of the larger aggregate also increases the mixture susceptibility to rutting and decreases fatigue life. The potentially adverse effects of the milling operation can present a problem in meeting Superpave fine gradation requirements as well. Currently, this problem is addressed by placing restrictions on the maximum amount of RAP that may be used in the mixture and blending in virgin aggregate. It has also been suggested by Stroup-Gardiner and Wagner (39) that RAP could be split into a coarse and fine fraction in order to keep the large amount of the dust fraction out of the mix, thereby allowing a higher percentage of RAP to be used.

Shape and Hardness

No literature was found citing aggregate shape or hardness (with the exception of Brownie and Hironaka's comment on hardness and the crushing process), and the relationship with RAP materials. Bukowski (6) lists FHWA Mixture Expert Task Group recommendations for the use of RAP in Superpave. The recommendations are based on discussions of industry professionals, and not experiments. This group recommends treating RAP as an aggregate stockpile and does not recommend measurement of aggregate angularity, sand equivalent, or the flat and elongated criteria outlined by Superpave.

Mix Design

Marshall/Hveem

One of the first comprehensive methods of RAP mix design was published by Epps et al. (40) in a 1980 National Cooperative Highway Research Program (NCHRP) report entitled "Guidelines for Recycling Pavement Materials." This reference was intended to be a source of information regarding recycling processes and RAP mix design incorporating asphalt modifiers. A detailed mix procedure is outlined in the appendix report, which was modeled after the work of Davidson et al (31), Dunning (32), Canessa (41), and Terrel and Fritchen (42).

A very similar recycled mix design procedure is presented in the Asphalt Institute (43) MS-2 Marshall and Hveem mix design methods manual. The Asphalt Institute recommended procedure is as follows:

1. Determine RAP aggregate gradation.
2. Determine RAP asphalt content and asphalt binder viscosity.
3. Blend RAP and virgin aggregate to obtain a gradation which meets specifications.
4. Approximate the asphalt demand of the combined aggregates.
   This may be done by the Centrifuge Kerosene Equivalent test or by the empirical formula in the manual. The formula is dependent on the proportion of aggregate retained on the No. 8 sieve, passing the No. 8 sieve, and passing the No. 200 sieve, with a constant given for each proportion.
5. Estimate the percent of new asphalt in the mix.
   This is estimated with a formula in the manual.

6. Select the grade of the new asphalt (or recycling agent).
   This is determined by using a target viscosity, the viscosity of the virgin asphalt, the viscosity of the asphalt in the RAP, and a viscosity blending chart.
7. Perform trial mix design using the Marshall or the Hveem method.
   Brownie and Hironaka (38) report that the addition of recycling agents may bring the asphalt content above optimum, resulting in a mix with lower stability. For this reason, it is important to use try a range of asphalt contents, both above and below the estimated asphalt demand.
8. Select the job-mix formula.

Superpave

The Superpave Mixtures Expert Task Group guidelines described by Bukowski (6) suggest that Superpave mixtures containing RAP should generally follow the same mix design requirements as conventional mixtures. The process of binder selection is further split into three tiers, depending on the RAP content:

1. Less than 15% RAP
   The asphalt binder grade should remain the same as what would be chosen for a mix design using only virgin materials.
2. 15% to 25% RAP
   Guidelines suggest to use a binder one grade lower for both the high and low temperature required for virgin binder. It also suggests that the low temperature grade may not need to be adjusted for temperate climates, and that binder grade can be selected using a blending chart if the designer compensates for the stiffness of the RAP binder.
3. More than 25% RAP
   A blending chart for high and low temperatures should be used to select the grade for the new asphalt binder.

In a study recently completed at the National Center for Asphalt Technology, Kandhal and Foo (44) recommend using the three-tier binder selection criteria set forth by the Superpave Mixtures Expert Task Group.

If the three-tier system is not used, sweep blending charts may be used. A high temperature sweep blending chart is constructed by determining the temperature at which $G^*/\sin\delta = 1.0$ kPa (unaged binder) or 2.2 kPa (RTFO aged binder) for a range of virgin binder percentages. The temperature values are plotted and an iso-stiffness curve is drawn. A similar method is used to construct the intermediate temperature blending chart, except that the temperature at which $G^* \sin\delta = 5.0$ kPa is used. Three such charts are necessary to determine the low temperature value.

Superpave places requirements on G* sinδ and G*/ sinδ for each grade of binder. For example, a PG 64-22 binder must have G*/ sinδ = 1.0 kPa between 64 and 70 C. As shown in Figure 2.2, the required new binder content is between 57 and 79 percent (21 to 43 percent RAP).

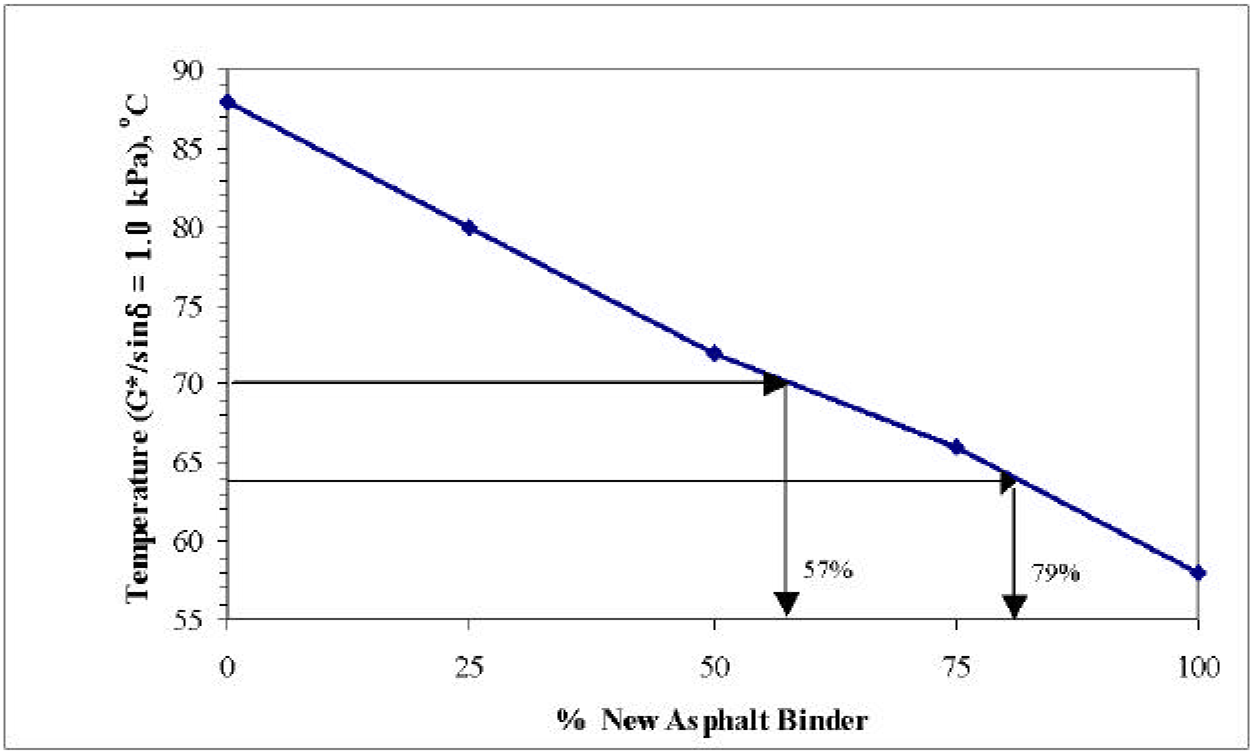

Figure 2.2 High Temperature Sweep Blending Chart (after Kandhal and Foo (44))

When a high temperature sweep blending chart is used, Kandhal and Foo (44) recommend using only the G*/sinδ = 1 kPa chart since it is easier to construct (the RTFO test is not needed) than the G*/ sinδ = 2.2 kPa chart. They also concluded that the intermediate temperature sweep blending chart (the chart which is expected to determine the maximum amount of RAP allowed in the mixture), G* sinδ = 5 MPa, allowed unusually high percentages of RAP when compared to what field experience has shown to be acceptable. The study did not look at the low temperature sweep blending charts.

In order to avoid performing temperature sweeps, and to overcome the potential error in using the intermediate temperature sweep blending chart, Kandhal and Foo (44) recommend using a specific grade blending chart for RAP mixture binder selection. This blending chart has 1.0 and 2.0 kPa stiffness lines. The 1.0 kPa line is used to determine the maximum amount of virgin asphalt binder (i.e. the minimum amount of RAP) to be used in the recycled asphalt binder. The 2.0-kPa line is used to determine the minimum amount of binder (i.e. the maximum amount of RAP) allowed in the recycled binder.

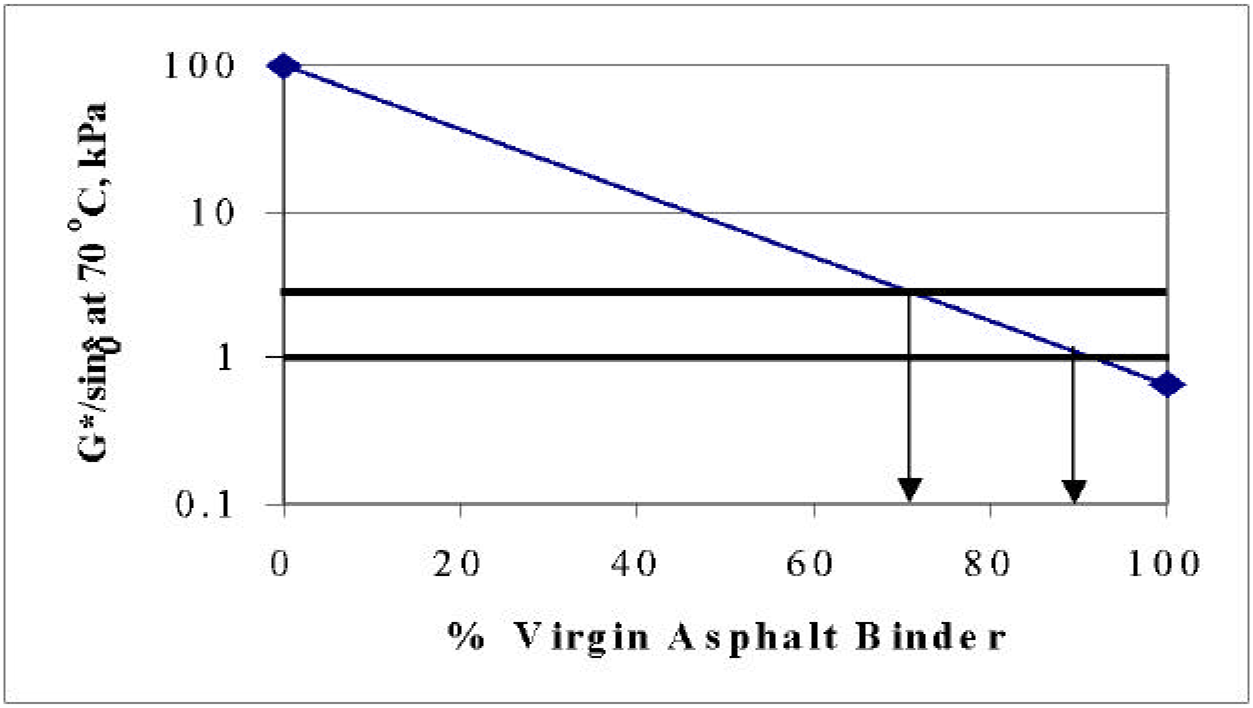

Figure 2.3 Specific Grade Blending Chart (after Kandhal and Foo (44))

Figure 2.3 is an example of a specific grade blending chart. The G*/sinδ of the aged asphalt binder is 100 kPa and the virgin PG 58-34 is 0.65 kPa, with a line drawn between these two points. The line intersects the 2.0 kPa line at 72 percent virgin binder, which corresponds to a maximum of 28 percent RAP. The 1.0 kPa line is intersected at 89 percent virgin binder, or a minimum of 11 percent RAP. If a different binder was to be used, the virgin binder G*/ sinδ would be different, and a different percentage of RAP would be required. Kandhal and Foo recommend using this type of a specific grade blending chart to perform a RAP mix design under the Superpave system (44).

Mn/DOT Practice

The study by Johnson and Han (4) showed most RAP is used for surface courses, with 30% stockpiled for future use, 12 percent used as base, and 11 percent wasted. These statistics are based on the responses of 60 counties and 39 cities to a survey conducted by the Mn/DOT State Aid Office.

Mn/DOT currently allows the use of recycled mixtures in Type 2350 base, leveling, shoulder, and wear courses. RAP percentages as high as 30 percent may be used in the base and binder courses and shoulders, and 15 percent in wear courses. All mixtures containing RAP are required to use PG 58-28 asphalt cement. The maximum particle size in any dimension is 75 mm for a Type 32 mixture and 19 mm in any dimension for Type 42 or 48 mixtures. Provided the above specifications are followed, and no objectionable material (glass, wood, rubber, etc.) is present in the RAP, a RAP mixture may be substituted for any virgin mixture, with the exception of Minnesota Department of Transportation (45) Specification 2360 mixtures.

Mix Characteristics

Strength

The tensile strength of asphalt mixtures is often tested with the indirect tensile test. For this test, a cylindrical specimen is loaded in compression parallel to and along the vertical diametral plane. This results in a relatively uniform tensile stress along the vertical plane of the sample, and the sample ultimately fails in tension.

Kennedy and Perez (46) obtained tensile strengths between 559 and 2,200 kPa (81 and 319 psi) for recycled mixtures. They reported that these values were slightly higher than what they normally observed with conventional mixtures.

Kandhal et al. (27) also tested some recycled mixes for indirect tensile strength and compared recycled and virgin mixtures. Their data show a tensile strength of 1,393 kPa (202 psi) for the control mix and 1,289 kPa (187 psi) for the recycled mix. This was shown to be statistically different with a paired t-test. The paper did not provide an explanation why the tensile strength was lower for the recycled mixtures.

Stroup-Gardiner and Wagner (39) performed tensile testing on mixes with gradations both above and below-the-restricted-zone Superpave gradations. In general, the RAP tensile strengths for the below-the-restricted-zone gradation were higher than the virgin mixes, for both moisture conditioned and unconditioned samples.

Durability

Another issue to be considered with a RAP mixture is durability. Moisture susceptibility is generally the cause of poor mixture durability. It may be caused by the loss of cohesive bond between binder and aggregate, usually due to moisture intrusion. This is called stripping, and it often starts at the top of the pavement and progresses downward, resulting in raveling. It is primarily a function of aggregate type, although it can be caused by other factors such as poor drainage or inadequate compaction (47). Moisture susceptibility can be evaluated in the laboratory by performing stability, resilient modulus, or tensile strength testing on unconditioned and moisture conditioned samples.

Laboratory Testing

Epps et al. (33) did Marshall stability testing on mixtures containing RAP. The conditioned samples were subjected to 2 hours of vacuum saturation followed by 7 days of soaking at 24 °C. Many of the samples tested retained about the same stability before and after conditioning, and some stabilities increased, leading Epps et al. to question whether the recycling process may make RAP mixtures less moisture susceptible.

Brownie and Hironaka (38) also used Marshall stability and stability retained to evaluate the stripping potential of RAP mixtures. They obtained RAP samples from 3 airfields and two civilian airports. The RAP mixtures were combined with varying degrees of Paxole recycling agent. Original Marshall stabilities were obtained and samples were immersed in a 60 °C (140 °F) water bath for 24 hours. The retained stabilities ranged from 66 to 100 percent. According to the authors, 75 percent is the minimum recommended retained stability. The material which did not pass this criterion was from the Fallon airfield in Nevada. Samples of this mixture were tested with an antistripping agent, but the 75 percent retention was still not achieved. Brownie and Hironaka theorize that the antistripping agent, which has been used successfully in many virgin asphalt paving studies, could not effectively coat and chemically alter the surface of the RAP aggregate. They recommended additional research to effectively treat hydrophilic aggregates during recycling operations.

More recently, Kallas et al. (48) studied the moisture damage and stripping behavior of 5 recycled mixtures, one of which used an aggregate known to have stripping problems when it was originally in place. The four other mixtures showed no moisture damage would take place after short-term moisture conditioning (Lottman method), and two of the mixtures would strip after long-term conditioning. The moisture susceptible aggregate was tested with and without antistripping agent. The mixture with the agent did not strip after long term testing, but the one without the agent did. These results suggest that RAP mixtures should be tested for moisture susceptibility, and an antistripping agent should be used where needed.

Moisture sensitivity testing by Stroup-Gardiner and Wagner (39) showed that the tensile strength retained ratio (TSR) for Minnesota and Georgia RAP mixtures was similar to the TSR of the virgin control mixture, with all three retaining near 50 percent. Superpave recommends a minimum TSR of 80 percent, so the RAP mixtures and the control mixture examined in this project have stripping potential.

Observed Performance

A potential problem with using a high percentage RAP mixture is raveling shortly after construction. The New York State Department of Transportation experienced such a problem while conducting recycled pavement research. They used a binder course and a wear course, both with a RAP content of 35 percent. The pavement was placed in 1977, and within three months of construction, extensive delamination and deterioration of coarse aggregate particles was noticed. Raveling progressed to the point where the pavement needed to be resurfaced in 1982.

Engineers from New York State DOT attributed the raveling to the virgin asphalt physically coating the old asphalt, but not chemically bonding with either the old asphalt or the aggregate. As a result, there was a layer of old asphalt sandwiched between the aggregate and the virgin asphalt, resulting in an area of brittle failure (49).

Though raveling is a potential problem when dealing with a RAP mixture, there have been numerous projects where raveling was slight or nonexistent. A Vermont project described by Frascoia (47) and a the Pennsylvania project described by Beam and Maurer (37) reportedly had no raveling, and the Louisiana project described by Paul (29) experienced only slight raveling on 11 of 95 sites surveyed. These projects show that while durability with a RAP mixture is a concern, it is possible to construct a high quality pavement with these materials.

Stiffness

According to Roberts et al. (25), the most common method of measuring HMA stiffness is the resilient modulus test (ASTM D4123). The resilient modulus test is a nondestructive test where the asphalt concrete sample is dynamically loaded. Sample loading is generally between 5 and 20% of the sample indirect tensile strength. The applied load and recovered strain are measured, and the resilient modulus is defined as:

$$M_R = \frac{P(0.27 + u)}{\Delta U t} \tag{2.1}$$

Where:

M_R = resilient modulus, Pa

P = applied load, Newtons

ΔU = horizontal deformation, mm

t = sample thickness, mm

ν = Poisson's ratio

A number of researchers have performed resilient modulus testing to compare the stiffness of RAP mixtures to virgin mixtures. Kennedy and Perez (46) conducted a study using different soft asphalts and rejuvenators, as well as RAP with no treatment. They obtained resilient moduli ranging from 1,720 MPa to 6,915 MPa at 25 °C. They indicated that these results were slightly higher than previously evaluated conventional mixtures.

Kandhal et al. (27) performed resilient modulus testing on cores taken from 5 paired RAP and conventional paving jobs in service for 1.5 to 2.25 years. They determined the average resilient modulus of the conventional mixtures and the recycled mixtures at 25 °C to be 6,530 MPa and 6,150 MPa, respectively. However, these moduli were shown to not be statistically different using a paired t-test with a significance level of 0.05.

Noureldin and Wood (48) conducted a laboratory study comparing the resilient moduli of four mixtures, all having the same gradation. The only difference between the mixtures was that one was a virgin mixture which used AC-20 binder, and the other three were RAP (AC-20) blended with three different types of rejuvenators. The mixtures were tested at asphalt contents of 5.5, 6.0, and 6.5% percent. The results indicated that all three of the mixtures with rejuvenators yielded a lower resilient modulus than the virgin mixture, which is an indication the rejuvenators were functioning to both decrease the asphalt binder viscosity and lower the mixture stiffness.

Stroup-Gardiner and Wagner (39) examined the use of RAP with Superpave guidelines. Three mixtures with above-the-restricted-zone Superpave gradations were used. The first was a virgin mixture which used a PG 64-22 (used as the virgin asphalt for all three mixtures) asphalt cement and 100 percent crushed granite (used as the virgin aggregate for all three mixtures). The second mixture contained 15 percent Georgia RAP and the third contained 15 percent Minnesota RAP. No rejuvenator was used in these mixtures. Resilient modulus testing was conducted on these mixtures at temperatures of 4, 25 and 40 °C, as shown in Table 2.1:

Table 2.1 Resilient Modulus Results (after Stroup-Gardiner and Wagner (39))

Mixture	Resilient Modulus, MPa (ksi)		
	4 °C	25 °C	40 °C
Control	8,037 (1,165)	4,593 (666)	1,203 (186)
15% Georgia RAP	9,223 (1,337)	7,007 (1,016)	2,655 (385)
15% Minnesota RAP	10,385 (1,500)	7,538 (1,093)	3,034 (440)

The data in Table 2.1 show the addition of 15 percent RAP more than doubled the resilient modulus at 40 °C, which is beneficial in terms of resistance to rutting at high temperatures. The authors did in fact see a reduction of in-laboratory rut depths with the RAP mixtures. The RAP did not show much difference form the virgin mix at the lowest test temperature.

Resilient modulus testing results appear to be dependent on the amount of RAP and the grade of virgin asphalt or rejuvenator used. A mixture of 100 percent RAP will have a higher resilient modulus than a virgin mixture of the same original asphalt grade. However, if an excess of soft asphalt or rejuvenator with an extremely low viscosity is used, the resilient modulus may be decreased. The research by Stroup-Gardiner and Wagner (39) shows that mixtures with RAP content as low as 15 percent can affect the mixture properties, and it may be necessary to use a lower binder grade at these RAP percentages.

Complex Modulus

Witczak and Root (49) describe the complex modulus of asphalt concrete as the relationship between stress and strain during sinusoidal loading. Since it is a complex number, it is composed of a real part and an imaginary part. The real part is referred to as the storage modulus, and measures the elastic response of the sample. The imaginary part is the loss modulus, which measures the viscous response. In these respects, it is similar to dynamic shear rheometer testing performed on asphalt binder. Figure 2.4 illustrates the concept of strain lag and phase angle, denoted by the symbol δ. The magnitude or absolute value of the complex modulus is defined as the ratio of the peak stress (σ_o) to the peak strain (ε_o).

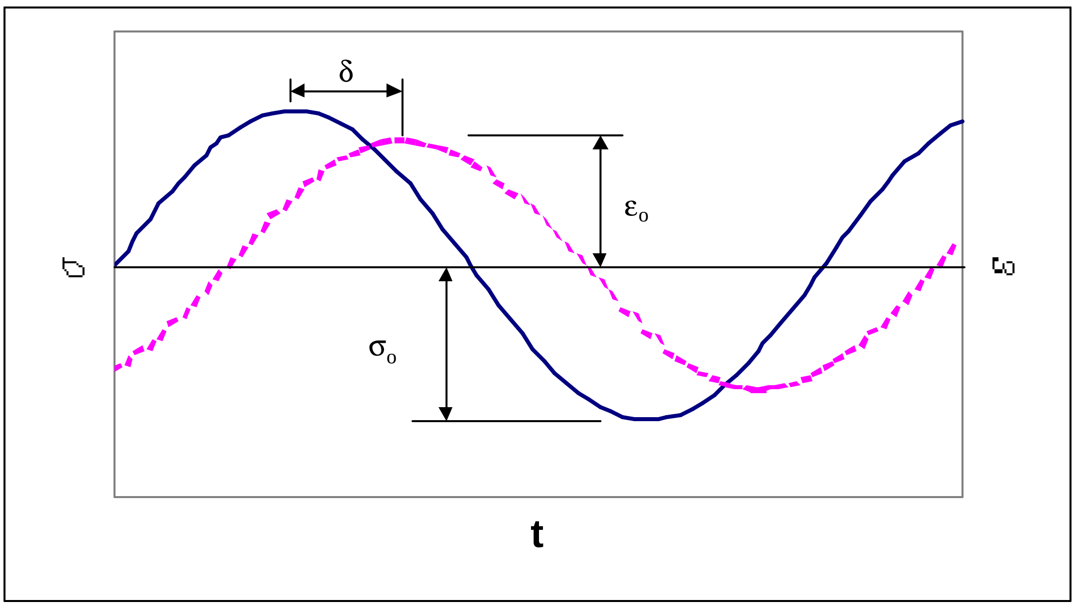

Figure 2.4 Illustration of Phase Angle

Fonseca and Witczak (52) have shown the complex modulus test to be affected by many factors. Dynamic modulus or absolute value and phase angle are a function of both test temperature and loading frequency. As temperature increases, dynamic modulus decreases and phase angle increases. When the loading frequency is increased, dynamic modulus increases and the phase angle decreases.

The asphalt concrete sample can be loaded in three ways: compression, tension or tension-compression. If the material is isotropic, the value of the complex modulus will be independent of testing method. Kallas (50) conducted tests using all three loading methods and obtained similar values of dynamic modulus. Coffman (51) has also shown asphalt concrete to be isotropic.

While no research was found regarding complex modulus and RAP samples, Fonseca and Witczak (52) did develop a model to predict the dynamic modulus of field aged asphalt concrete. They did so by using a database of dynamic modulus and viscosity values. Similarly, studies could be performed using the complex modulus to measure the amount of aging experienced, and predict future aging of RAP mixtures.

The effect of aging on asphalt concrete mixtures using complex modulus was also evaluated by Daniel et al. (53). Samples were compacted and subjected to one of four aging processes: short term aging, or three different levels of long term aging. They found that the complex modulus at all aging levels decreased with frequency and higher temperatures. The more aging of the mixture, the higher the value of dynamic modulus. Phase angles were found to increase with temperature and decreasing frequency, as expected. In addition, Daniel et al. found that as aging progresses, the phase angle decreases.

Although the study by Daniel et al. (53) evaluated mixture aging, their results may have been similar if the effect of RAP was studied. RAP has essentially been subjected to long term aging. Therefore, it is expected that a higher RAP content will yield a mixture with a higher dynamic modulus and lower phase angle than a mixture with a lower RAP content. Recent work by Francken et al. (54) also has shown aged mixtures to have a lower phase angle than unaged mixtures.

RELATION TO THIS STUDY

As mentioned earlier, one of the objectives of this study was to sample RAP stockpiles from around the state to characterize typical Minnesota RAP gradation and binder properties. Past experience has shown it is important to characterize the aggregate gradation in the stockpile, rather than as cores, since the gradation may significantly change during the milling process. The first objective was accomplished by determining the gradation of the RAP as taken from the stockpile. Prior to the gradation analysis, the binder was removed from the aggregate by both the solvent method and by the ignition method. Aggregate gradation was obtained after each method and the results compared. The ignition oven was also used to determine the asphalt cement content.

The second objective involved the development of a mix design incorporating RAP and following Superpave criteria. Rather than running an array of tests on extracted asphalt from RAP stockpiles, RAP mixtures were be tested. The effects of aged asphalt can be assessed through the comparison of different RAP percentages and different virgin binder grades.

Mixture characterization was accomplished using the resilient modulus, retained indirect tensile strength, and complex modulus procedures. Resilient modulus testing ensured adequate mixture stiffness and acceptable temperature susceptibility, and provided a reference to conventional mixture test results. The retained indirect tensile strength tests provided information about the strength and stripping potential of RAP mixtures. Complex modulus testing determined the elastic and viscous characteristics of the RAP mixture, analogous to the Superpave specifications for an asphalt binder. The final result is an assessment of Minnesota RAP mixtures with respect to the Superpave mix design methodology.

CHAPTER 3
- RESEARCH METHODOLOGY –

OVERVIEW

The characteristics of both the binder and aggregate present in recycled asphalt pavement (RAP) materials have been discussed in chapter 2. The Superpave mixture design system, which was used in a modified form in this work, was presented. This chapter includes a description of the equipment and materials used to produce the specimens for mix design and testing. In addition, the procedures and equipment used in the testing program are presented. Due to high variability in the RAP, a modified mix design was used to determine the optimum asphalt content. Samples were created at this asphalt content and subsequently tested for complex modulus, resilient modulus, and moisture susceptibility using the indirect tensile strength test.

MIX DESIGN

Summary of Laboratory Mixtures

It was initially decided that this work would follow the Superpave mix design framework and use materials standard to the asphalt pavement laboratory at the University of Minnesota. As a result, a gradation passing above the restricted zone on the 0.45 power chart was used. Fine aggregate from the Lakeland, Minnesota pit and coarse aggregate from the Granite Falls, Minnesota quarry were used for the initial gradation. However, the initial gradation did not satisfy the Superpave 14 percent VMA requirement (for a 12.5 mm nominal maximum size). Subsequent coarser and finer gradations were tried with the same aggregates, though these also fell short of the VMA requirement.

It was then decided to locate a source of fine aggregate which was highly angular, unlike the rounded Lakeland gravel, which has a fine aggregate angularity (FAA) of only 40. A fine aggregate source of angular crushed traprock was located in nearby Dresser, Wisconsin. This material had an FAA of 48. The Dresser and Lakeland fine aggregates were blended to achieve a higher FAA. Since the Lakeland aggregate yielded small amounts of material retained on the 2.36 and 1.18-mm sieve, it was decided to use the Dresser aggregate only for these two sieve sizes. The gradation of the Dresser aggregate yielded the highest amount of material on these two sieve sizes, and so the blending decision was one of efficiency as well as increased angularity.

The blend produces a fine aggregate with an FAA of 42. The FAA would be higher, except this test takes into account only the aggregate between the 1.18 and 0.150 mm sieves. Therefore, the use of the Dresser fine aggregate increases the overall angularity by an amount greater than indicated by the FAA test.

Samples were compacted at an asphalt content of 5.1 percent using this blended gradation, shown on Figure 3.1 and Table 3.1. The VMA was improved by blending these aggregates, but it still fell short of the Superpave requirement. The minimum VMA for a 12.5 mm nominal maximum sieve size is 14 percent, and the average VMA of the 18 different mixtures was 12.9 percent. It was decided to continue with this gradation for the mix design and test samples since the mix was intended for a low volume roadway. On a low volume roadway, durability can sometimes be ensured using a higher asphalt content (subsequently yielding a lower air void content) even though the VMA criteria is not met.

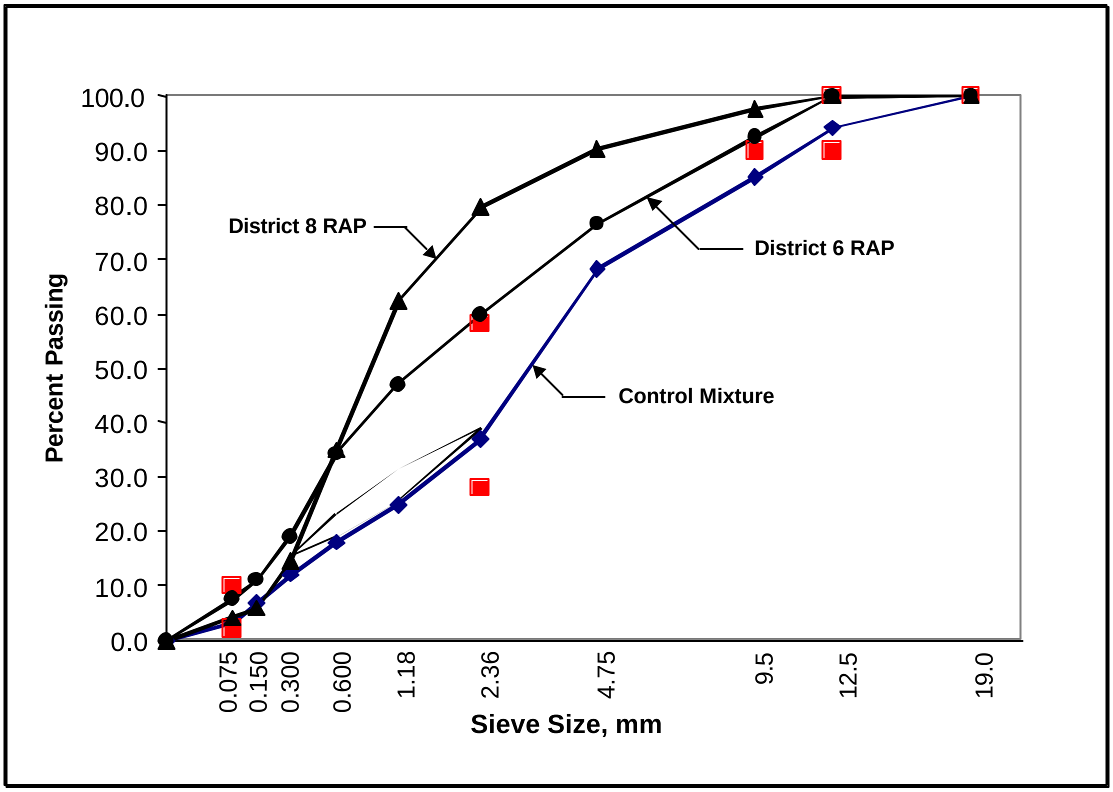

Figure 3.1 Control and RAP Gradations

Table 3.1 Percent Passing for Control and RAP Gradations

Sieve Size, mm	Control Mixture	District 6 RAP	District 8 RAP
19.0	100.0	100.0	100.0
12.5	94.0	100.0	100.0
9.5	85.0	92.4	97.7
4.75	68.0	76.5	90.2
2.36	37.0	60.0	79.5
1.18	25.0	47.2	62.2
0.600	18.0	34.4	35.1
0.300	12.0	18.9	14.6
0.150	7.0	10.9	5.9
0.075	3.0	7.5	4.0

The mixture described above was used as the control mixture for the testing program. When RAP was added to the mixes, the RAP gradation after toluene extraction (shown on Figure 3.1) was blended with the control aggregate to achieve a gradation that was similar to the gradation of the control mixture. It was possible to blend 40 percent District 6 RAP and virgin aggregate to produce a gradation similar to the control gradation, but this could not be done with the District 8 RAP. Due to large amount of fine aggregate in the District 8 RAP, a maximum of 30 percent could be used while still maintaining the control gradation.

Virgin Aggregate Properties

As previously mentioned, the rounded Lakeland gravel was blended with the more angular Dresser traprock to increase the fine aggregate angularity. Granite Falls Granite was used as the coarse aggregate source. The properties of the aggregates are listed in Tables 3.2 and 3.3.

Table 3.2 Fine Aggregate Properties

Test	Lakeland Gravel	Dresser Traprock
G_{sb}	2.602	2.848
G_{sa}	2.770	3.026
Water Absorption, %	2.3	1.8
Fine Aggregate Angularity, %	40	48
Sand Equivalent, %	48	20

Table 3.3 Coarse Aggregate Properties

Test	Granite Falls Granite
G_{sb}	2.757
G_{sa}	2.797
Water Absorption, %	0.46
Flat/Elongated Particles %, (1:3 Ratio)	14.0
Fractured Faces %, (≥ 1 / ≥ 2)	100/100

RAP Binder and Aggregate Properties

The asphalt content of the RAP was determined by both the ignition oven and solvent extraction methods. It was decided to use the ignition oven results since Brown and Mager (15) showed it to be more precise than solvent methods. Solvent extraction was used to recover the RAP binder since this is not possible with the ignition oven. The Minnesota Department of Transportation determined the PG grade of the recovered binder. RAP asphalt content and PG grade are in Table 3.4.

Table 3.4 RAP Asphalt Properties

RAP Source	Asphalt Content	PG Grade
District 6	7.1%	67-24
District 8	4.7%	78-11

Once the asphalt binder was separated from the aggregate, the properties of the RAP aggregate were determined. The gradation of both RAP sources was determined after solvent extraction and was shown in Figure 3.1. The aggregate gradation after solvent extraction was used because of the possibility of substantial loss of fines during the ignition oven process. The bulk specific gravity, effective specific gravity, fine aggregate angularity (FAA), and absorption of the RAP aggregates were then determined, as given in Table 3.5.

Table 3.5 RAP Aggregate Properties

RAP Source	G_{sb}	G_{se}	FAA	Absorption
District 6	2.588	2.811	39.8	1.55%
District 8	2.528	2.858	39.6	2.49%

It was originally intended that the mix design would be performed using PG 52-34 asphalt with 0 and 30 percent RAP. Once the optimum asphalt content was determined for these two mixtures, the optimum asphalt content would be interpolated for 15 percent RAP and extrapolated for 40 percent RAP. Since optimum asphalt content is generally not affected by binder grade, the values found for the PG 52-34 would be used for the PG 46-40 and PG 58-28.

However, a high level of variability in the air void content was noted early on in the mix design procedure. The target air void content was 4 percent, but the results for RAP mixtures varied from 3 to 5 percent. At the same time, the control mixture (no RAP) exhibited less variability in air void content.

Solaimanian and Tahmoressi (28) have documented the increased variability of mixtures containing high percentages of RAP material. They looked at laboratory and field compacted mixtures with RAP contents ranging from 0 percent to 50 percent. It was found that as RAP content is increased, the variability in asphalt content, gradation, and air voids also increased.

As a result of the observed variability in air voids for RAP mixtures, it was decided to use the control mixture optimum asphalt content for the virgin material portion of all mixtures. The optimum asphalt content for the control mixture was determined to be 5.1% percent. No virgin asphalt binder was added to the RAP portion of the mixtures.

This procedure ensured that the virgin material had the same asphalt content for all mixtures. However, the total asphalt content varies for each mixture due to the contribution of the aged asphalt binder already present in the RAP. The total asphalt content for each mixture is shown in Table 3.6.

Table 3.6 True Asphalt Contents

Mixture	Control	D6 15%	D6 30%	D6 40%	D8 15%	D8 30%
Asphalt Content	5.1%	5.4%	5.7%	5.9%	5.0%	5.0%

Gyratory Compactor

The compactor used throughout the project was the Brovold gyratory compactor, a Superpave approved Gyratory Compactor manufactured by Test Quip. The Brovold compactor produces 150-mm diameter samples.

A piston pushing down on a plate within the compaction mold supplies the vertical pressure. The Superpave standard of 600 kPa (87 psi) was used throughout the project. The gyratory angle of the compactor creates the compactive shear force. Increasing the angle of gyration increases the shear force created by the compactor. Superpave guidelines call for a gyratory angle of 1.25 ± 0.02°.

The height of the sample is continually recorded by a linear variable displacement transducer (LVDT). The density of the sample at each gyration is calculated using the current height and mass of the sample. The operator is able to control the compaction energy transmitted to the sample by inputting the desired number of gyrations. Superpave specifies the number of gyrations as a function of temperature and anticipated traffic. The design number of gyrations was based on low-volume, level two, traffic: 300,000 to 1,000,000 ESALs. Therefore, $N_{ini} = 7$, $N_{des} = 76$, and $N_{max} = 117$.

Compaction Procedure

Prior to mixing, pre-batched, 12,000 gram aggregate samples were placed in a forced-draft oven for a minimum of four hours to ensure they were dry. The asphalt binder was pre-heated to the appropriate mixing temperature, 125 C for PG 46-40, 138 C for PG 52-34 and 145 C for PG 58-40. The virgin aggregate was poured into a bucket mixer followed by the appropriate amount of RAP. The two materials were given about one minute to thoroughly mix, and then the proper amount of asphalt cement was added. After adequate mixing in the bucket mixer, the mixture was placed in a large pan and mixed by hand to prevent segregation. Each batch was then split into two 4,800-gram samples for compaction, and two 1,000-gram samples for theoretical maximum specific gravity tests. Mix design samples were prepared according to the matrix shown in Table 3.7.

Table 3.7 Project Mixture Matrix

RAP Content		**0 %**			**15 %**			**30 %**			**40 %**		
PG Grade		**46-40**	**52-34**	**58-28**	**46-40**	**52-34**	**58-28**	**46-40**	**52-34**	**58-28**	**46-40**	**52-34**	**58-28**
RAP SOURCE	District 6	X	X	X	X	X	X	X	X	X	X	X	X
	District 8				X	X	X	X	X	X			

The samples were aged for four hours at 135 °C as specified by the Asphalt Institute (5). After aging, the PG 46-40 samples were compacted at 115 °C, the PG 52-34 samples at 128 °C, and the PG 58-28 samples at 135 °C.

Moisture Sensitivity

Moisture sensitivity in an asphalt mixture is typically manifested by a gradual loss of strength over a period of years resulting in the development of rutting and shoving in the wheel paths. Loss of strength is due to the weakening of the bond between the asphalt cement and aggregate. Stripping may occur, which can lead to aggregate particles becoming detached from the asphalt concrete matrix. To help protect against moisture damage it is necessary to determine if a mixture is susceptible to water damage in the event of water penetration.

Moisture sensitivity tests were conducted in accordance with ASTM D4867. One set of four specimens for each mixture was compacted to between 6 and 8 percent air voids. A subset of two specimens remained unconditioned while the other subset was partially saturated with water and moisture conditioned. The samples were vacuum saturated to between 55 and 80 percent. After being partially saturated, the conditioned samples were placed in a 60 °C water bath for 24 hours and then cooled to 25 °C for 4 hours. Both subsets were then subjected to the tensile splitting test and loaded at a rate of 50 mm per minute until failure. The tensile strength of each subset was determined by Equation 3.1.

$$S_t = \frac{2000P}{ptD} \tag{3.1}$$

Where:

S_t = tensile strength, kPa
P = maximum load, N
t = specimen height before tensile test, mm
D = specimen diameter, mm

The potential for moisture damage is indicated by the tensile strength ratio (TSR): the ratio of the tensile strength of the conditioned subset to that of the unconditioned subset. The TSR for each mixture is calculated by Equation 3.2.

$$TSR = \frac{S_{tw}}{S_{td}} \times 100 \tag{3.2}$$

Where:

S_{tw} = tensile strength of moisture-conditioned sample, kPa
S_{td} = tensile strength of unconditioned (dry) sample, kPa

COMPLEX MODULUS

The complex modulus test was used to evaluate the viscoelastic properties of the asphalt concrete mixtures. When a harmonic sine wave load is applied to the sample the complex compliance may be determined directly. The form of the harmonic loading function is shown in Equation 3.3.

$$P(t) = P_0 \cos wt \tag{3.3}$$

Where:

$P(t)$ = applied load

P_0 = amplitude of the harmonic load
w = angular frequency

However, a harmonic load requires the application of a cyclic tension-compression load, which is difficult to perform on asphalt concrete. As a result, it is common to apply a haversine load, which is a constant load superposed with a harmonic load. In this manner, the sample is always in compression during the test. Equation 3.4 shows the form of the haversine loading function.

$$P(t) = P_0(1 - \cos wt) \tag{3.4}$$

The material response will be the sum of the responses caused by the harmonic and constant loads. In order to determine the complex compliance, the response caused by the harmonic load must be separated from the total response. If it is assumed the harmonic response has reached steady-state, the total response may be represented by Equation 3.5.

$$\Delta(t) = at + b + c\cos(wt - d) \tag{3.5}$$

Where:
$\Delta(t)$ = total response
$at + b$ = response due to constant load
c = amplitude of the harmonic response
d = phase angle

A regression routine was developed by Zhang (8) to separate and determine the harmonic response. Once this is known, the real and imaginary portions of the deviatoric and volumetric complex compliance may be determined from Equations 3.6a through 3.6d.

$$J_{d1}(w) = [0.745\Delta U_0(w)\cos d_u(w) + 1.635\Delta V_0(w)\cos d_v(w)]\frac{L}{P_0} \tag{3.6a}$$

$$J_{d2}(w) = -[0.745\Delta U_0(w)\sin d_u(w) + 1.635\Delta V_0(w)\sin d_v(w)]\frac{L}{P_0} \tag{3.6b}$$

$$J_{v1}(w) = [3.448\Delta V_0(w)\cos d_v(w) - 2.537\Delta U_0(w)\cos d_u(w)]\frac{L}{P_0} \tag{3.6c}$$

$$J_{v2}(w) = -[3.448\Delta V_0(w)\sin d_v(w) - 2.537\Delta U_0(w)\sin d_u(w)]\frac{L}{P_0} \tag{3.6d}$$

Where:
$J_{d1}(w)$ = real (storage) portion of the deviatoric complex compliance

$J_{d2}(w)$ = complex (loss) portion of the deviatoric complex compliance
$J_{v1}(w)$ = real portion of the volumetric complex compliance
$J_{v2}(w)$ = complex portion of the volumetric complex compliance
ΔV_0 = amplitude of the vertical response
ΔU_0 = amplitude of the horizontal response
d_v = phase angle of the vertical response
d_u = phase angle of the horizontal response

The above equations are valid only for measurements taken over the entire diameter of the sample for horizontal deformation and over the center 25 percent of the sample for vertical deformation. Additional equations for vertical measurements over the entire diameter are presented by Zhang (7). The sample must be tested in indirect tension and the ratio of the width of the loading strips to the sample diameter must be 1:8.

The magnitude of the complex compliance may be calculated using Equation 3.7.

$$|J^*| = \sqrt{J_1^2 + J_2^2} \tag{3.7}$$

Where:
J^* = complex compliance
J_1 = storage compliance
J_2 = loss compliance

The magnitude of the complex modulus is the inverse of the magnitude of the complex compliance as shown in Equation 3.8.

$$|E^*| = \frac{1}{|J^*|} \tag{3.8}$$

Where:
E^* = complex modulus

If the complex modulus and the phase angle are known, it is possible to determine the elastic and viscous properties of the material. The elastic and viscous components are known as the storage modulus (E_1) and loss modulus (E_2), respectively. Their relationships to the complex modulus and phase angle are shown in Equations 3.9 and 3.10, respectively.

$$E_1 = E^* \cos d \tag{3.9}$$

$$E_2 = E^* \sin d \tag{3.10}$$

Complex modulus testing was performed using an MTS 810 electrohydraulic test system, operated by a 458 series console and controllers. Load was applied using a hydraulic actuator with a capacity of 100 kN. Deformation was measured across the horizontal diameter with an MTS 632.94 extensometer with lateral attachment kit. Vertical deformation was measured across the center 25 percent of the diameter using an in-house modified MTS 632.06 clip-on extensometer.

The test was performed at four different temperatures in order to evaluate how the viscoelastic properties changed with temperature: -18, 1, 25, and 32 °. It was not possible to test at a temperature higher than 32 °C since the vertical extensometer points would creep due to the clamping force of the clip-on extensometer and eventually be pulled from the face of the sample.

Load frequency was varied from 0.03 Hz to 30 Hz . However, it was found that there was too much scatter in the data above 5 Hz at the –18 and 1 °C temperatures. As a result, data is only presented up to 5 Hz. The test was not performed at 0.03 Hz at 32 °C because of excessive creep during the longer loading time at this frequency.

The load amplitude was chosen to produce enough deformation to be accurately measured by the extensometers without causing so much deformation that the response was no longer linear viscoelastic and the samples were damaged. A stress level similar to that used by Zhang (7) was used at the 25 °C test temperature. Higher load amplitudes were used at the colder temperatures and a lower amplitude at the higher temperature, as shown in Table 3.8:

Table 3.8 Load Amplitudes (N) for Complex Modulus Tests

Temp. (°C)	**-18**		**1**		**25**		**32**	
Freq. (Hz)	**0.03 – 1**	**5**	**0.03 – 1**	**5**	**0.03 – 1**	**5 – 30**	**0.1 – 1**	**5 – 30**
PG 46-40	4000	4500	700	1000	250	350	5	7.5
PG 52-34	4000	4500	700	1000	350	550	15	25
PG 58-28	4000	4500	700	1000	400	600	25	40

RESILIENT MODULUS

Resilient modulus tests were conducted on all samples to evaluate mixture temperature susceptibility and for use as a reference to earlier testing. Though it was once believed stiffer pavements had greater resistance to permanent deformation, Roberts et al. (25) caution that there currently is no solid correlation between resilient modulus and rutting. However, they have concluded that resilient modulus at low temperatures is somewhat related to cracking as stiffer mixes (higher MR) at low temperatures tend to crack earlier than more flexible mixtures (lower MR).

Resilient modulus testing was conducted in accordance with ASTM D 4123-82, “Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures.” In order to produce enough deformation for accurate measurement without damaging the samples, applied load levels were similar to those used in the complex modulus test sequence. The test was performed at 0.33, 0.5, and 1 Hz. In a 1-Hz test the applied cycles consisted of a 0.1-second load followed by a 0.9-second rest period. Upon test completion, the resilient modulus was calculated using Equation 3.11.

$$M_R = \frac{P(0.27+u)}{\Delta U t} \tag{3.11}$$

Where:

M_R = resilient modulus, Pa
P = applied load, N
ΔU = horizontal change in length, mm
t = sample thickness, mm
u = Poisson’s Ratio

Poisson’s Ratio for a 150-mm sample was calculated using Equation 3.12.

$$u = \frac{4.09\Delta U}{\Delta V} - 0.27 \tag{3.12}$$

Where:

ΔU = horizontal change in length, mm
ΔV = vertical change in length, mm

Since the determination of Poisson’s Ratio from horizontal and vertical measurements is often inaccurate, typical assumed values from Yoder and Witczak (25a) were used. The values used are shown in Table 3.9.

Table 3.9 Typical Values of Poisson's Ratio for Asphalt Concrete

Temperature (°C)	Poisson's Ratio
- 18	0.20
1	0.25
25	0.35
32	0.40

The same samples used for complex modulus testing were also used for resilient modulus testing. As a result, two samples from each mixture were tested. Each sample was tested at both zero and 90-degree orientations. The samples were given a 2-hour waiting period between zero and 90-degree tests to allow for recovery from the previous test. All 36 samples were tested at -18, 0, 25, and 32 °C. Prior to testing, the samples were placed in temperature controlled environmental chambers for a minimum of 24 hours to ensure equilibrium at the test temperature. Isopropyl alcohol was used to remove any ice accumulation from the extensometers at cold temperatures.

CHAPTER 4
- RESULTS AND DISCUSSION –

INTRODUCTION

The results of the laboratory testing for resilient modulus, complex modulus, and the evaluation of moisture sensitivity using the indirect tensile strength test are discussed in this chapter. Data were collected from 18 different mix designs, incorporating three different asphalt binders, two sources of RAP, and varying amounts of RAP. The resilient modulus tests were completed to provide a comparison with past testing performed on conventional mixtures, as well as testing performed on RAP mixtures. The complex modulus test allows for characterization the elastic and viscous properties of a mixture, making it more informative than the resilient modulus test. Finally, the indirect tensile strength was analyzed for the various mixtures to determine how the addition of RAP affects mixture moisture susceptibility.

RESULTS

Moisture Sensitivity

Tables 4.1, 4.2, and 4.3 summarize the moisture sensitivity results. Superpave criteria require a minimum tensile strength ratio (TSR) of 80 percent. The TSRs for the 18 mixtures evaluated were all above 95 percent. Although still debated, high TSR values such as these may indicate a lower susceptibility to moisture damage.

Table 4.1 PG 58-28 Moisture Sensitivity Results

Sample ID	Measurement	PG 58-28					
		Control	**D6 15%**	**D6 30%**	**D6 40%**	**D8 15%**	**D8 30%**
Unconditioned Samples	Load (N)	23,776	23,674	23,678	23,660	23,825	23,847
	Dry Strength (kPa)	1,145	1,135	1,118	1,138	1,139	1,130
Unconditioned Samples	% Air Voids	7.24	6.88	7.33	7.58	6.89	6.77
Conditioned Samples	% Air Voids	7.16	6.69	7.49	7.39	6.62	6.92
After Vacuum Saturation	% Saturation	65.3	60.8	71.5	59.8	66.5	59.8
	% Swell	3.18	2.94	4.15	3.74	3.84	3.44
After 140°F 24-hr Water Bath	% Saturation	86.4	79.8	89.8	84.5	91.7	81.9
	% Swell	5.32	4.87	5.46	5.26	5.61	4.98
	Load (N)	23,972	23,612	23,434	23,603	23,803	23,700
	Wet Strength (kPa)	1,131	1,128	1,107	1,131	1,128	1,123
Tensile Strength Ratio	TSR, %	98.8	99.4	99.0	99.4	99.0	99.4

Table 4.2 PG 52-34 Moisture Sensitivity Results

Sample ID	Measurement	PG 52-34					
		Control	**D6 15%**	**D6 30%**	**D6 40%**	**D8 15%**	**D8 30%**
Unconditioned Samples	Load (N)	24,511	24,453	24,439	24,386	24,364	24,348
	Dry Strength (kPa)	1,170	1,186	1,182	1,212	1,161	1,150
Unconditioned Samples	% Air Voids	7.59	7.68	6.55	6.79	6.81	7.35
Conditioned Samples	% Air Voids	7.42	7.72	6.69	6.60	6.90	7.27
After Vacuum Saturation	% Saturation	62.8	59.9	57.6	60.4	65.8	62.3
	% Swell	3.41	3.13	2.97	4.09	3.80	4.24
After 140°F 24-hr Water Bath	% Saturation	84.9	86.9	81.70	83.6	89.4	91.8
	% Swell	5.16	4.44	4.29	5.10	4.97	5.39
	Load (N)	24,261	24,257	24,204	24,166	24,148	24,154
	Wet Strength (kPa)	1,154	1,173	1,168	1,194	1,149	1,145
Tensile Strength Ratio	TSR, %	98.6	98.9	98.8	98.5	98.9	99.5

Table 4.3 PG 46-40 Moisture Sensitivity Results

Sample ID	Measurement	PG 46-40					
		Control	D6 15%	D6 30%	D6 40%	D8 15%	D8 30%
Unconditioned Samples	Load (N)	24,026	23,997	23,959	23,971	23,910	23,916
	Dry Strength (kPa)	1,146	1,151	1,165	1,155	1,130	1,127
Unconditioned Samples	% Air Voids	6.68	7.19	7.27	6.89	6.92	7.16
Conditioned Samples	% Air Voids	6.78	7.14	7.39	6.75	7.10	7.38
After Vacuum Saturation	% Saturation	71.5	69.2	71.4	61.8	65.4	74.6
	% Swell	3.18	4.62	3.32	4.04	3.62	3.84
After 140°F 24-hr Water Bath	% Saturation	88.9	84.5	91.7	80.6	89.3	92.3
	% Swell	4.97	5.38	4.82	5.22	4.95	5.51
	Load (N)	23,921	23,956	23,925	23,912	23,892	23,898
	Wet Strength (kPa)	1,143	1,136	1,156	1,141	1,113	1,124
Tensile Strength Ratio	TSR, %	99.8	98.7	99.2	98.7	98.5	99.8

The tensile strength retained values were higher and less variable than expected. In addition, there was very little difference in TSR value when asphalt binder or RAP content was changed. Figure 4.1 shows that a strong relationship between TSR and asphalt binder or RAP content does not seem to be present in this data. As a result, it may be said that these data indicate the addition of RAP to a mixture had no positive or negative influence on the mixture tensile strength or moisture susceptibility.

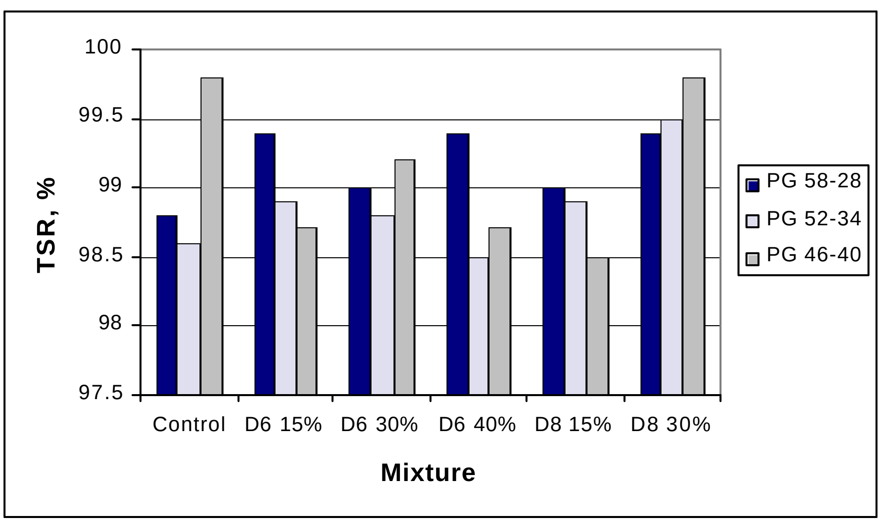

Figure 4.1 Tensile Strength Ratios for RAP Mixtures

High TSR values were noted by McGennis et al. (55). They concluded specimens compacted with a Superpave gyratory compactor resulted in significantly higher TSR values. They also cited the test's low reliability and lack of a satisfactory relationship between laboratory and field results.

Resilient Modulus

Two samples from each mixture were tested in accordance with ASTM D 4123. The same samples that had been used for the complex modulus testing were used for the resilient modulus testing. The average resilient modulus at 1 Hz and the coefficient of variation for each mixture are shown in Table 4.4.

Table 4.4 Resilient Modulus Results, 1.0 Hz

Asphalt Grade	Test Temperature, °C	-18		0		25		32	
	Frequency = 1.0 Hz	Res Mod (Mpa)	Coef. of Var. (%)	Res Mod (Mpa)	Coef. of Var. (%)	Res Mod (Mpa)	Coef. of Var. (%)	Res Mod (Mpa)	Coef. of Var. (%)
PG 46-40	Control	8119	2.2	2885	6.6	308	1.4	68	2.2
	D6 15%	7426	5.4	4143	0.9	513	7.5	105	4.2
	D6 30%	7172	11.4	4411	15.5	767	3.8	116	2.9
	D6 40%	6796	4.6	5109	14.9	811	1.3	151	4.8
	D8 15%	7283	8.6	4810	5.5	711	0.3	139	4.2
	D8 30%	6057	2.8	4793	10.2	827	1.4	178	0.3
PG 52-34	Control	5897	31.7	4000	13.6	756	6.4	153	9.9
	D6 15%	6686	21.2	4264	8.4	1463	13.4	279	13.7
	D6 30%	12719	8.7	6966	3.7	2010	1.6	408	5.3
	D6 40%	6907	9.8	5589	4.3	2325	9.7	426	6.3
	D8 15%	5444	16.0	4402	1.0	2055	0.9	368	6.4
	D8 30%	9824	6.7	6466	1.3	2827	4.5	504	2.0
PG 58-28	Control	5040	8.9	3343	17.6	1615	1.6	266	0.9
	D6 15%	5811	15.6	3937	6.9	2273	1.6	435	11.6
	D6 30%	7989	13.0	5011	0.8	2611	2.4	522	9.3
	D6 40%	8725	30.5	4597	3.7	2806	3.2	613	16.3
	D8 15%	11852	27.6	5064	7.2	2426	0.8	441	1.5
	D8 30%	9909	13.5	4120	5.9	2980	19.8	818	11.8

D6 and D8 denote District 6 and District 8, respectively

There were three primary objectives to this sequence of resilient modulus testing:

1. Examine the effect of asphalt binder grade
2. Examine the effect of the addition of RAP
3. Examine the effect of the RAP source

Effect of Asphalt Binder on Resilient Modulus

In general, the data at –18 and 1 °C do not clearly show how resilient modulus is affected by asphalt grade and RAP content. This is attributed to increased variability in the resilient modulus test at the lower temperatures. In general, the coefficient of variation is highest for the two coldest temperatures, as listed in Table 4.4. It is believed part of the variability at lower temperatures stems from how the loading strips contact the sample. The sample surface is not always uniform and at low temperatures the sample may not be compliant enough to ensure complete contact with the loading strips. Lower deformations are also measured colder temperatures, making electronic noise in the sensors a more significant source of error in the measurement. The coefficient of variation generally was within or below 10 to 20 percent.

At the higher temperatures of 25 and 32 °C the results were as expected and the relationships observed are consistent with those of past research efforts. Noureldin and Wood (48) have shown a mixture with a stiffer asphalt binder will have a higher resilient modulus than a mixture with a softer asphalt binder when other variables are the same. At the two upper test temperatures a stiffer asphalt binder (PG 58-28) always yielded a higher resilient modulus than a mixture with a softer asphalt binder (PG 46-40), with all other variables the same. Figure 4.2 shows the relationship observed between resilient modulus and asphalt binder grade for the District 6 15 percent RAP mixtures. These data can be considered to exhibit a relationship typical of all the mixtures tested. The resilient modulus rapidly decreases with increasing temperature. This is due to the softening of the asphalt binder as the temperature is increased.

Effect of RAP Content on Resilient Modulus

The addition of RAP to the mixture also had a pronounced effect on the resilient modulus. As found by other researchers such as Stroup-Gardiner and Wagner (39), the addition of RAP will result in a stiffer mixture. Again, this relationship is clear at the higher test temperatures, but not at the lowest temperature. Figures 4.2 and 4.3 illustrate the increase in resilient modulus with the addition of RAP. At 25 °C, adding 40 percent District 6 RAP to a PG 58-28 control mixture resulted in a 74 percent increase in stiffness and a 164percent increase with a PG 46-40 control mixture. Increases in stiffness were also observed with the addition of District 8 RAP.

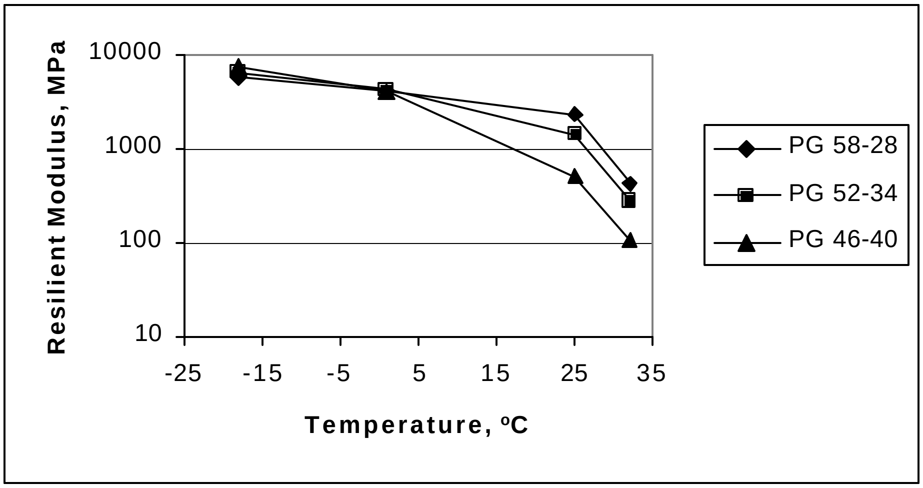

Figure 4.2 Effect of Asphalt Grade on Resilient Modulus for District 6 15% RAP Mixtures, 1.0 Hz

Effect of RAP Content on Resilient Modulus

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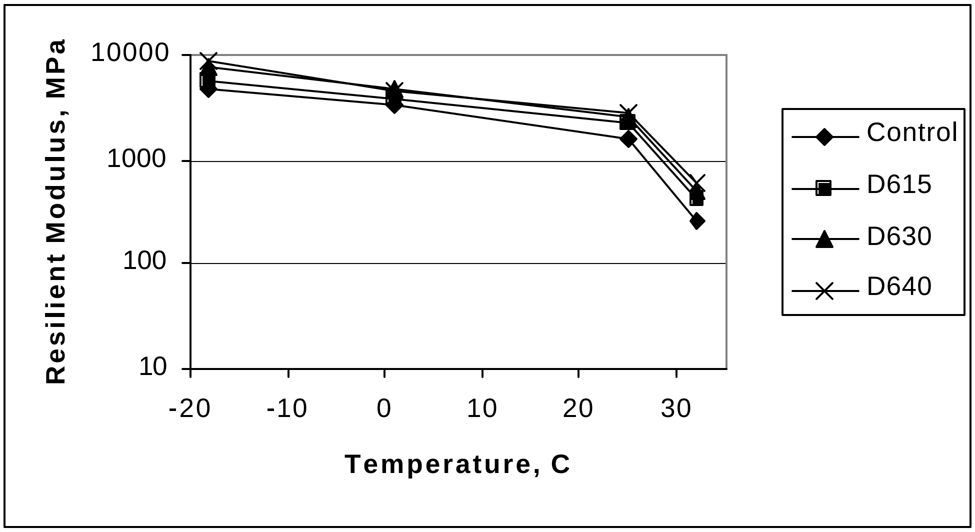

Figure 4.3 Effect of RAP on Resilient Modulus for District 6 RAP PG 58-28 Mixtures, 1.0 Hz

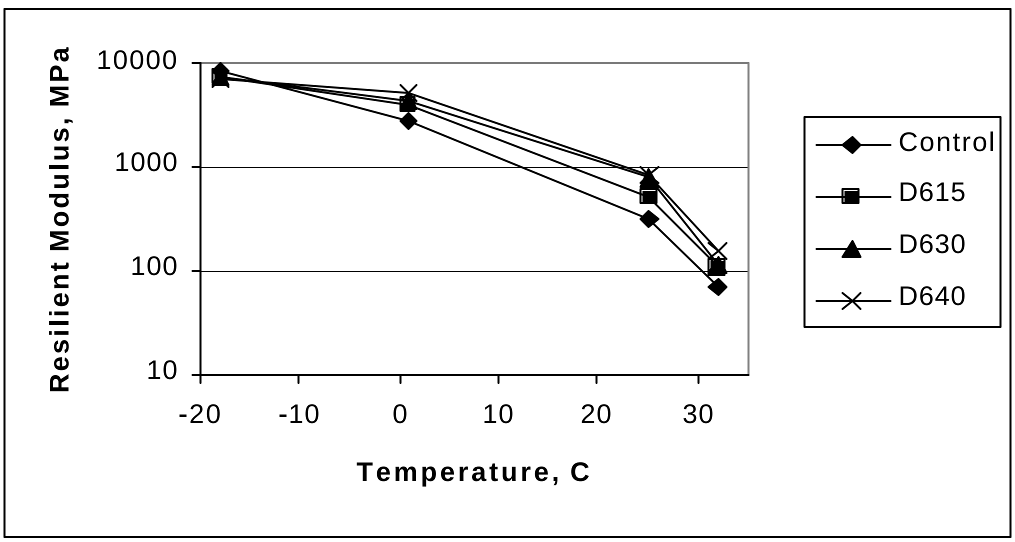

Figure 4.4 Effect of RAP on Resilient Modulus for District 6 PG 46-40 Mixtures, 1.0 Hz, 25 °C

The increased stiffness brought about by the addition of RAP may be offset by the use of a softer asphalt binder. Figure 4.5 shows that a mixture with a PG 52-34 asphalt binder and about 20 percent District 6 RAP will have a similar stiffness as a virgin mixture with PG 58-28 asphalt binder. Also, a mixture with a PG 46-40 asphalt binder and 35 percent RAP will have a similar stiffness as a virgin mixture with a PG 52-34 asphalt binder.

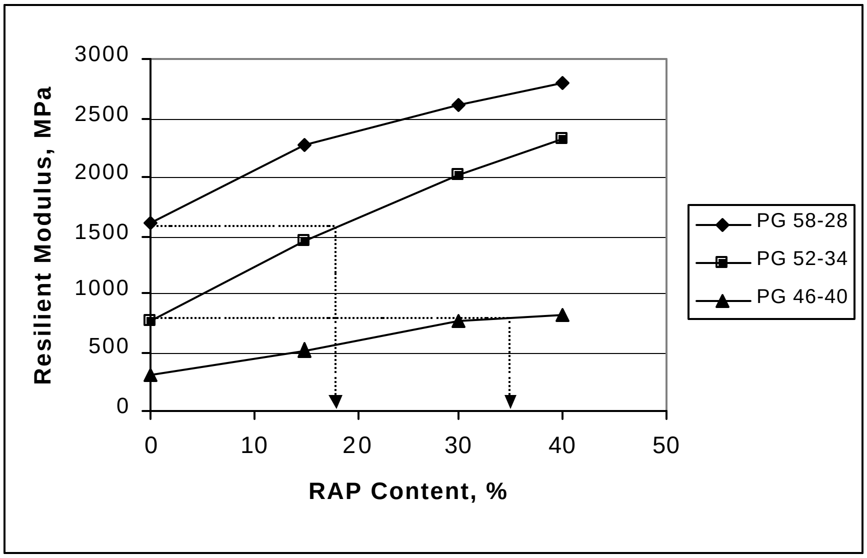

Figure 4.5 Amount of District 6 RAP to Obtain Stiffness Similar to Virgin Mixture, 1.0 Hz, 25 °C

Effect of RAP Source on Resilient Modulus

The source of the RAP material is important when examining the mixture stiffness. Stiffness will vary since RAP from different sources may have different asphalt binders. In addition, the actual RAP binder properties at the time of testing can vary depending on the age of the material. A 10-year old PG 52-34 asphalt will generally be stiffer than a 2-year old PG 52-34 asphalt.

When asphalt binder extractions were completed and the direct shear rheometer (DSR) testing performed, it was found that the District 6 RAP asphalt binder was equivalent to a PG 67-24 and the District 8 RAP asphalt binder was equivalent to a PG 78-11. Therefore, it is expected that a mixture with 15 percent RAP from District 8 will be stiffer than a mixture with 15 percent District 6 RAP. This relationship was exhibited by the resilient modulus results, as shown in Figure 4.6. At 25 °C, the District 8 30 percent RAP mixtures averaged a resilient modulus that was 20 percent higher than the District 6 30 percent RAP mixtures.

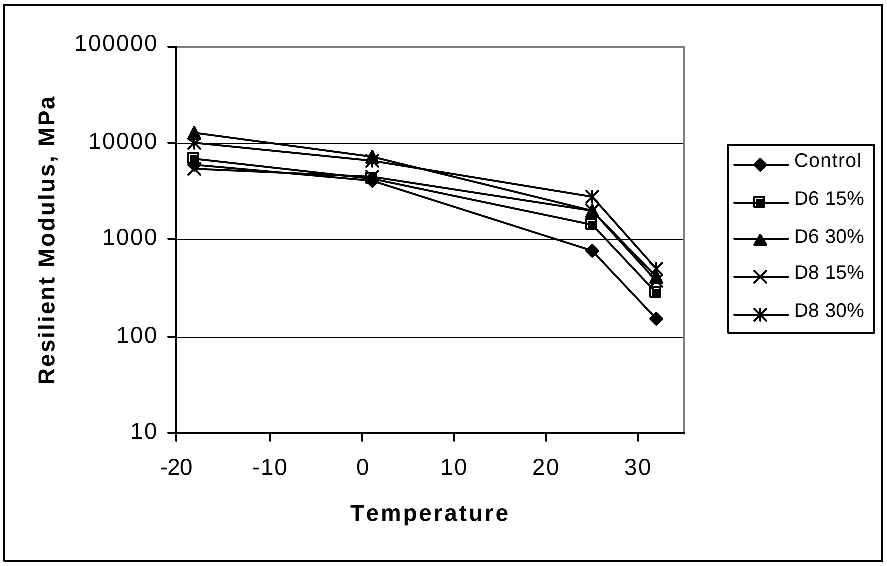

Figure 4.6 Effect of RAP Source on Resilient Modulus for PG 52-34 Mixtures with District 6 and District 8 RAP, 1.0 Hz

Effect of RAP Content on Variability

As discussed in Chapter 3, variability in the RAP seemed to have an effect on the mix design process. Obtaining consistent air voids was difficult when using the RAP, even when the asphalt content was held constant. It was expected that this variability would also become evident in the test results. However, only two samples from each mixture were tested, making it difficult to determine if RAP content affected the variability between resilient modulus tests.

Therefore, it was decided to average the coefficient of variation for all of the control mixture samples. Since three asphalt grades were used, this was an average of six samples. The same thing was done with the RAP samples, resulting in an average taken from twelve 15 percent RAP samples, twelve 30 percent RAP samples and six 40 percent RAP samples. Figure 4.7 shows that, in general, the addition of RAP does increase the variability of resilient modulus tests.

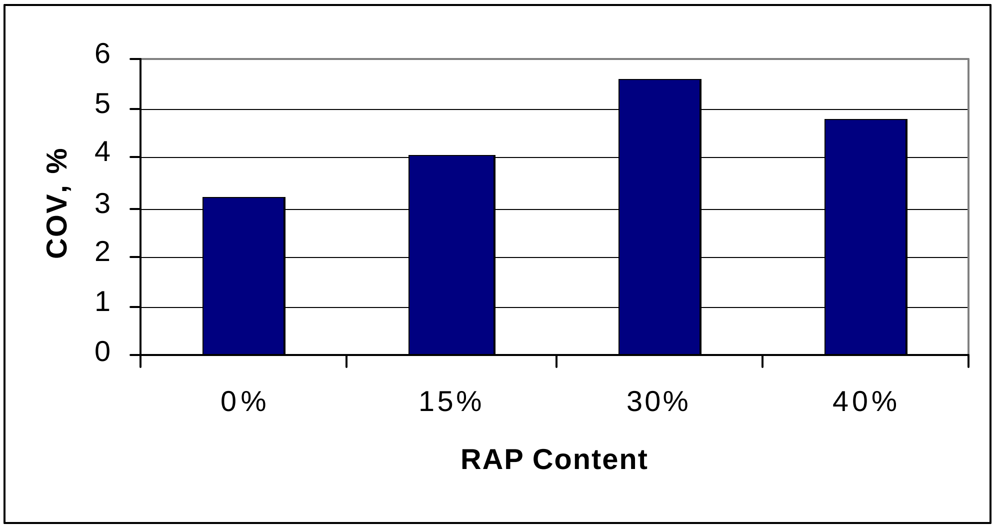

Figure 4.7 Effect of RAP Content on Variability of Resilient Modulus Tests

Complex Modulus

The complex modulus of asphalt concrete defines the relationship between stress and strain during haversine loading. This research was performed in order to determine how:

1. Complex modulus is affected by asphalt binder grade
2. Complex modulus is affected by changes in testing temperature
3. Complex modulus is affected by changes in loading frequency
4. Complex modulus changes with the addition of RAP

The complex modulus and phase angle results at the 1.0 Hz loading frequency are contained in Tables 4.5 through 4.8.

Table 4.5 Complex Modulus and Phase Angle at 1.0 Hz, -18 °C

Asphalt Binder	RAP Source/Content	Dynamic Complex Modulus |E*|, MPa	Phase Angle, degrees
PG 58-28	Control	13,614	6.13
	District 6 15%	18,424	5.19
	District 6 30%	26,227	4.40
	District 6 40%	32,298	5.96
	District 8 15%	28,797	6.14
	District 8 30%	20,204	7.79
PG 52-34	Control	17,905	9.18
	District 6 15%	10,358	7.21
	District 6 30%	20,928	6.65
	District 6 40%	15,599	7.01
	District 8 15%	34,919	8.46
	District 8 30%	38,659	4.21
PG 46-40	Control	8,275	12.97
	District 6 15%	17,248	9.56
	District 6 30%	26,734	9.00
	District 6 40%	19,661	9.10
	District 8 15%	23,668	10.71
	District 8 30%	28,127	5.65

Table 4.6 Complex Modulus and Phase Angle at 1.0 Hz, 1 °C

Asphalt Binder	RAP Source/Content	Dynamic Complex Modulus |E*|, MPa	Phase Angle, degrees
PG 58-28	Control	7,098	20.71
	District 6 15%	7,698	18.89
	District 6 30%	9,654	18.20
	District 6 40%	12,875	14.40
	District 8 15%	10,981	15.47
	District 8 30%	21,797	12.13
PG 52-34	Control	5,493	24.65
	District 6 15%	6,125	20.52
	District 6 30%	8,977	19.93
	District 6 40%	11,072	15.60
	District 8 15%	10,831	14.86
	District 8 30%	16,977	11.93
PG 46-40	Control	2,788	34.50
	District 6 15%	4,946	31.82
	District 6 30%	5,610	27.83
	District 6 40%	6,236	25.50
	District 8 15%	5,247	30.90
	District 8 30%	8,377	24.20

Table 4.7 Complex Modulus and Phase Angle at 1.0 Hz, 25 °C

Asphalt Binder	RAP Source/Content	Dynamic Complex Modulus |E*|, MPa	Phase Angle, degrees
PG 58-28	Control	1,292	32.74
	District 6 15%	1,412	32.09
	District 6 30%	2,536	31.80
	District 6 40%	2,892	29.49
	District 8 15%	1,765	32.50
	District 8 30%	3,073	31.32
PG 52-34	Control	542	28.48
	District 6 15%	915	31.68
	District 6 30%	1,332	30.58
	District 6 40%	1,747	29.30
	District 8 15%	1,737	28.98
	District 8 30%	2,703	24.73
PG 46-40	Control	252	19.36
	District 6 15%	394	25.81
	District 6 30%	545	29.85
	District 6 40%	654	27.54
	District 8 15%	577	28.20
	District 8 30%	770	30.12

Table 4.8 Complex Modulus and Phase Angle at 1.0 Hz, 32 °C

Asphalt Binder	RAP Source/Content	Dynamic Complex Modulus |E*|, MPa	Phase Angle, degrees
PG 58-28	Control	42	35.56
	District 6 15%	50	33.99
	District 6 30%	78	32.64
	District 6 40%	112	39.47
	District 8 15%	84	33.78
	District 8 30%	107	33.43
PG 52-34	Control	35	28.35
	District 6 15%	66	28.82
	District 6 30%	74	27.82
	District 6 40%	99	30.51
	District 8 15%	76	31.14
	District 8 30%	133	30.54
PG 46-40	Control	33	15.84
	District 6 15%	59	15.01
	District 6 30%	67	22.30
	District 6 40%	N/A	N/A
	District 8 15%	67	19.69
	District 8 30%	76	27.27

General Observations and Comments Regarding Complex Modulus Testing

Some conclusions concerning the complex modulus test can be drawn from the data in Tables 4.5 through 4.8. Data collected at the –18 °C test temperature had more variability than data collected at the other temperatures. This variability can be seen in a plot of the load and deformation response, which shows more scatter at the lower test temperatures. Deformations are lower at the coldest temperatures, and electronic noise in the sensors may have a significant effect on the measured response. Higher variability at the colder test temperatures may also be a result of non-uniform contact of the loading strips. The sample surface may not be compliant enough at the low temperatures to ensure good contact, and therefore the stress distribution may vary. The data collected at 25 °C contained the least variability, followed by 32 and 1 °C.

Another important note is that phase angles decreased as loading frequency increased at –18 and 1 °C. However, at 25 and 32 °C, the phase angles tend to increase as the loading frequency is increased. This is not evident from Tables 4.5 through 4.8 since they only contain data at the 1-Hz loading frequency. However, these data do show a change of trend. At –18 and 1 °C the phase angle tends to decrease as RAP content increases. At 25 °C this trend is less evident, especially for the mixtures made with the PG 46-40 asphalt. At 32 °C, the phase angle values become erratic, showing almost no trend at all. The reason why the phase angle measurements became unreliable at higher temperatures is explained later in this section.

Effect of Asphalt Binder and Temperature on Complex Modulus

Fonseca and Witczak (52) have shown that the dynamic modulus should increase as test temperature is decreased. This was observed for all mixtures tested. Figure 4.8 shows the typical behavior of complex modulus with temperature. The asphalt binder grade appears to have a significant effect on complex modulus up to the 25 °C test temperature. The effect is not nearly as pronounced at 32 °C.

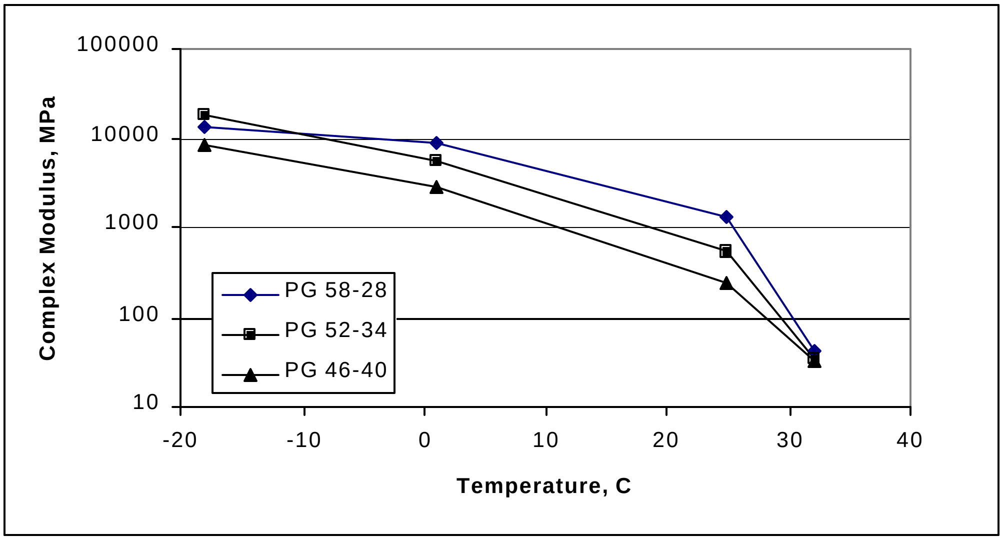

Figure 4.8 Effect of Asphalt Binder and Temperature on Complex Modulus, Control Mixtures. 1.0 Hz

Figure 4.8 shows that all control mixtures become softer as the test temperature increases. This was also true for the mixtures incorporating RAP. In addition, a stiffer asphalt binder yielded a higher dynamic modulus than a mixture with a softer asphalt binder. As mentioned before, an increase in test variability was noted at the two coldest temperatures, and especially at the cold extreme. It is likely that the mixture made with the PG 58-28 asphalt is not actually softer than the mixture made with the PG 52-34 asphalt at –18 °C, but that this is an artifact of testing variability.

Effect of Loading Frequency on Complex Modulus and Phase Angle

As loading frequency was increased the dynamic modulus increased for all mixtures. A dramatic increase in dynamic modulus is observed from 0.03 Hz to 20 Hz. For the PG 58-28 control mixture shown at 25 °C, the modulus increased an order of magnitude in this frequency range, an increase was typical for all mixtures. Once the frequency was increased above 20 Hz the dynamic modulus continued to increase, but at a less dramatic rate. In addition to an increase in dynamic modulus, an increase in loading frequency caused the phase angle to decrease. Figures 4.9 and 4.10 show this relationship. These results are in agreement with those of Daniel et al. (53) and Zhang (7).

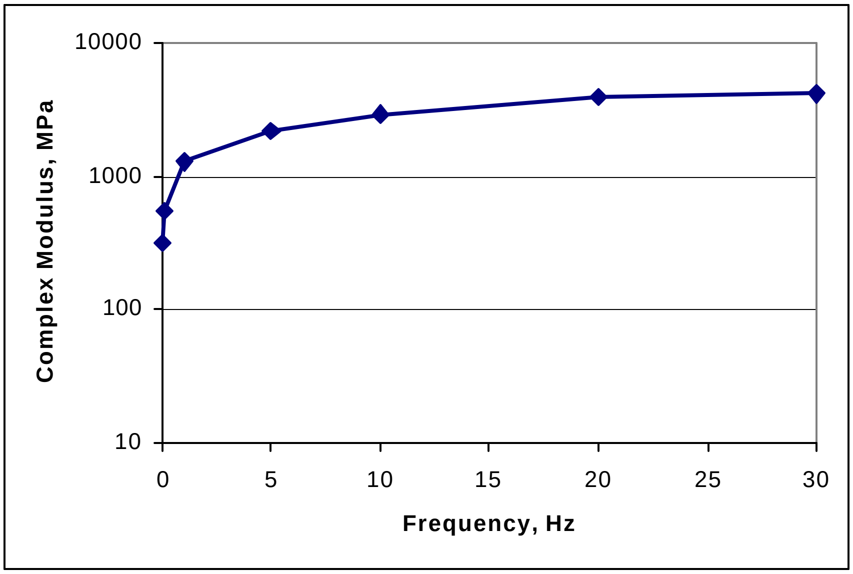

**Figure 4.9 Effect of Frequency on Complex Modulus,
PG 58-28 Control Mixture, 1 °C, 1.0 Hz**

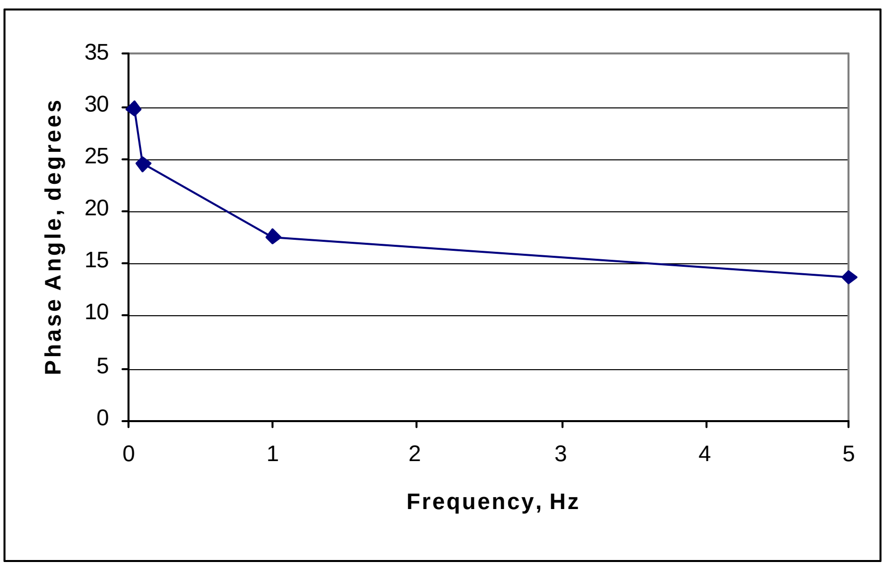

**Figure 4.10 Effect of Frequency on Phase Angle,
PG 58-28 Control Mixture, 1 °C, 1.0 Hz**

It is important to note that Figure 4.10 contains phase angle data at the 1 °C test temperature. At 25 and 32 °C, the results indicate that the phase angle increased as the frequency increased. Such a result is incorrect, and the cause is explained later in this section. Phase angle data collected at temperatures above 1 °C were not used for the purposes of mixture analysis because of these errors.

Effect of RAP Content on Complex Modulus and Phase Angle

Similar to what was observed during resilient modulus testing, the addition of RAP to a mixture caused an increase in the complex modulus. This result was observed at all test temperatures and for all mixtures. A typical response is shown in Figure 4.11.

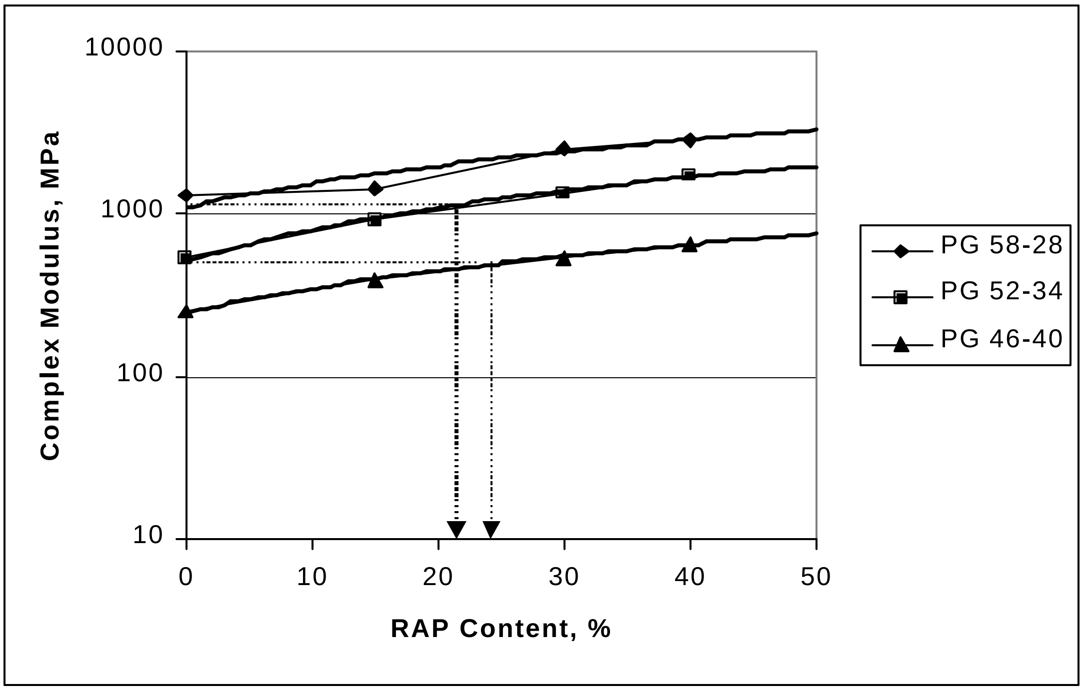

Figure 4.11 Change in Complex Modulus with RAP Content, District 6 RAP Mixtures, 25 °C, 1.0 Hz

A best-fit line was drawn through the data for each control mixture. Since the slope of the best-fit line is steepest for the PG 58-28 mixtures and has the lowest slope for the PG 46-40 mixtures, it may be that a stiffer asphalt binder will be more sensitive to the addition of RAP than a softer binder. In other words, adding RAP to a PG 58-28 mixture will cause a more dramatic increase in dynamic modulus than the addition of RAP to a PG 46-40 mixture. The District 8 RAP mixtures show the same relationship of increasing dynamic modulus with the addition of RAP, as shown in Figure 4.12.

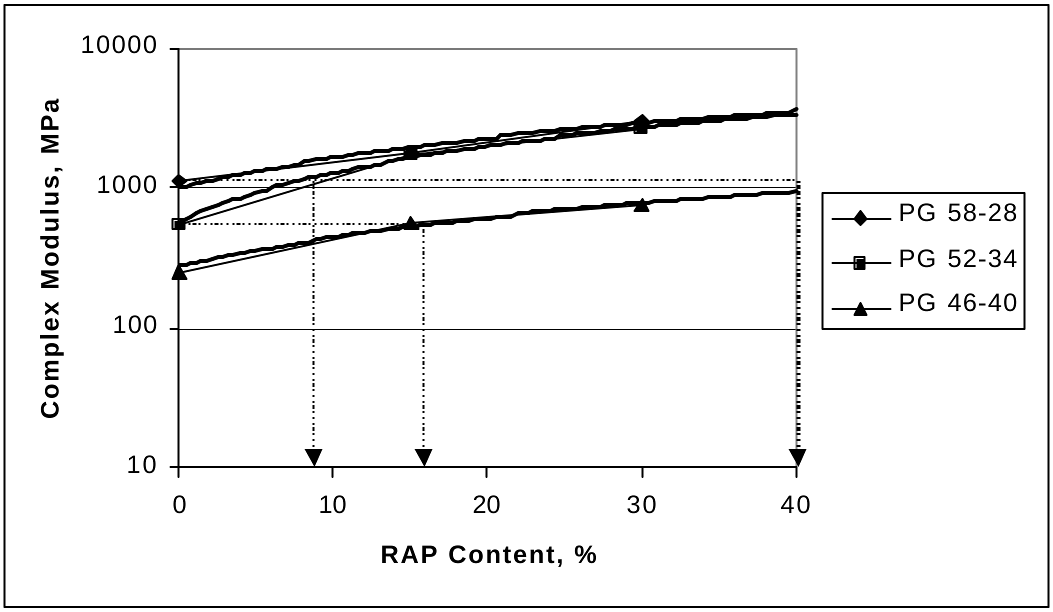

Figure 4.12 Change in Complex Modulus with RAP Content, District 8 RAP Mixtures, 25 °C, 1.0 Hz

Figures 4.11 and 4.12 also are useful for the determination of how much RAP may be added to a mixture while maintaining a dynamic modulus similar to a mixture composed of entirely virgin material. For District 6 RAP mixtures at 25 °C, about 23 percent of RAP may be used with a PG 52-34 asphalt binder to get a complex modulus similar to a virgin mixture made with PG 58-28 asphalt binder. About 28 percent RAP may be used with PG 46-40 asphalt binder to get the same complex modulus as a mixture composed of virgin material and PG 52-34 asphalt binder.

For a District 8 RAP mixture at 25 °C, about 8 percent RAP may be used with a PG 52-34 asphalt binder to get a complex modulus similar to a virgin mixture made with PG 58-28 asphalt binder. About 16 percent RAP may be used with PG 46-40 asphalt binder to get the same complex modulus as a mixture composed of virgin material and PG 52-34 asphalt binder.

When the test temperature is changed, the percentages of RAP material that may be added to a mixture change as well. Figure 4.13 shows how the complex modulus varies with the addition of RAP for District 6 RAP mixtures at 1 °C. This figure shows that about 16 percent District 6 RAP may be added to a PG 52-34 asphalt binder to obtain a similar complex modulus to a virgin mixture with PG 58-28 asphalt binder. About 26 percent District 6 RAP may be used with a PG

46-40 asphalt binder to obtain about the same complex modulus as a virgin mixture with PG 52-34 asphalt binder.

Figure 4.14 shows how the complex modulus varies with the addition of RAP for District 8 RAP mixtures at 1 °C. This figure shows that about 7 percent District 8 RAP may be added to a PG 52-34 asphalt binder to obtain a similar complex modulus to a virgin mixture with PG 58-28 asphalt binder. About 14 percent District 8 RAP may be used with a PG 46-40 asphalt binder to obtain about the same complex modulus as a virgin mixture with PG 52-34 asphalt binder.

It should be noted that the difference in the log of the modulus at 1 °C is less than the difference in the log of the modulus at 25 °C. In other words, the effect in changes of modulus at greater values are less than those at lower modulus values.

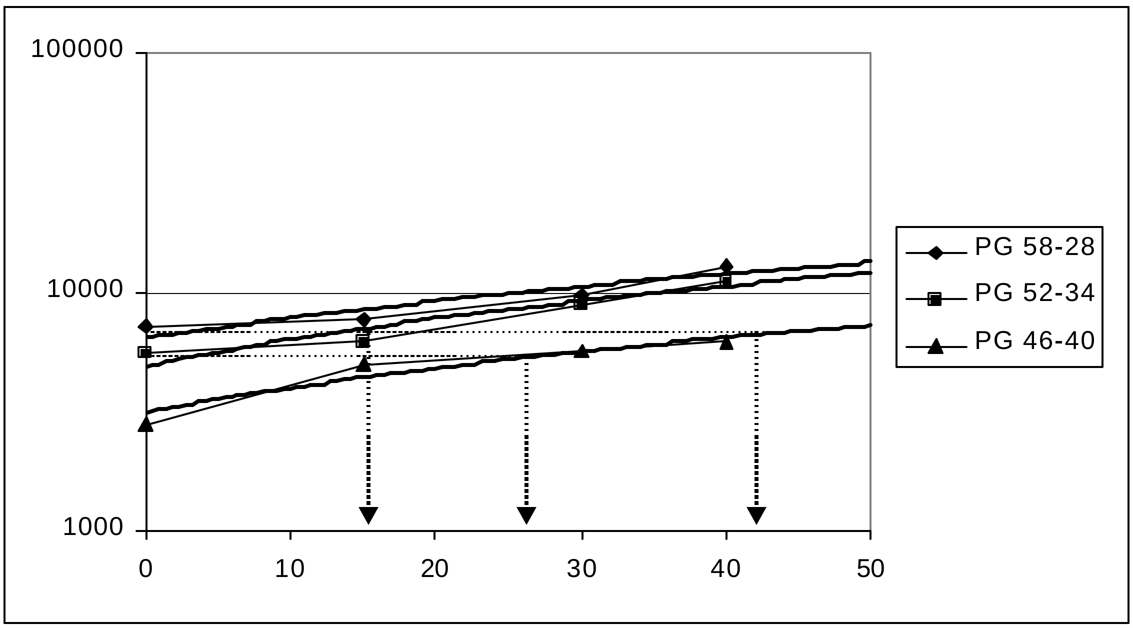

Figure 4.13 Change in Complex Modulus with RAP Content, District 6 RAP Mixtures, 1 °C, 1.0 Hz

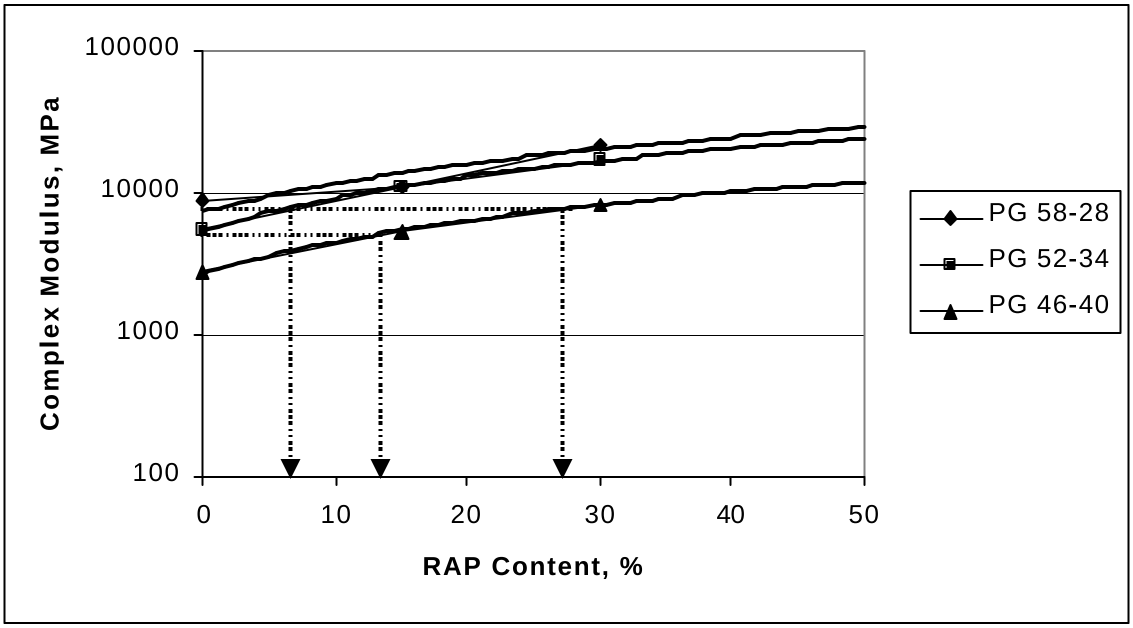

Figure 4.14 Change in Complex Modulus with RAP Content, District 8 RAP Mixtures, 1 °C, 1.0 Hz

When the test temperature was increased to 32 °C, it became more difficult to distinguish between mixtures made with different asphalt binders. However, the increase in complex modulus with increasing RAP content was still very pronounced, as illustrated in Figure 4.15. All of the mixtures were very soft at 32 °C. However, the addition of 40 percent RAP still nearly tripled the complex modulus when compared to virgin mixtures.

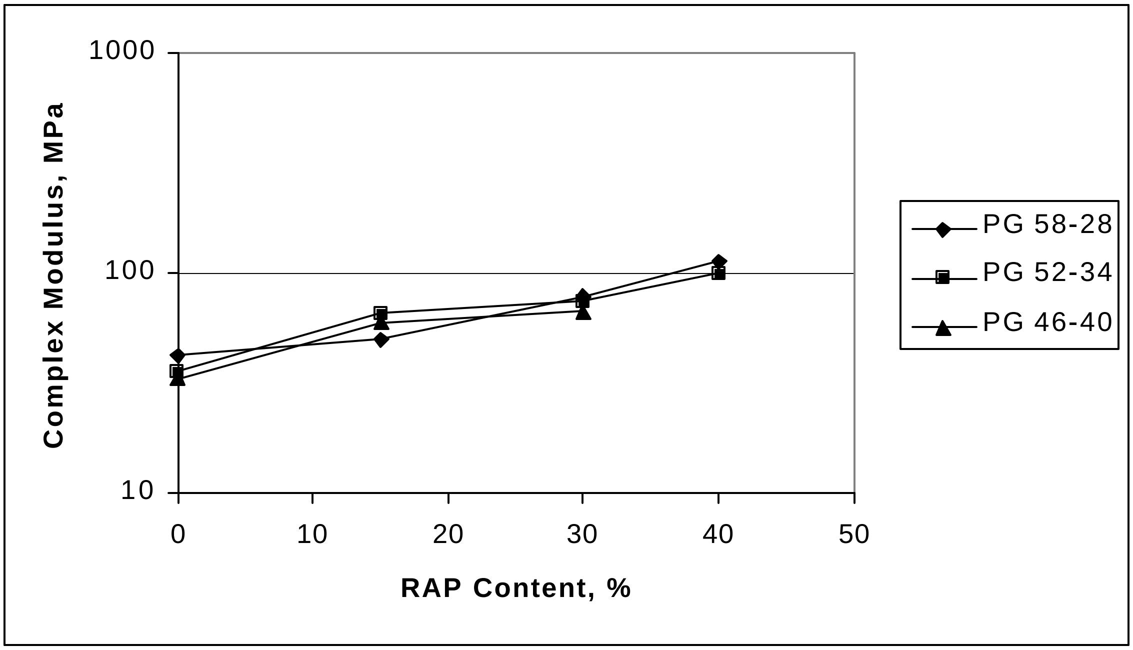

Figure 4.15 Change in Complex Modulus with RAP Content, District 6 RAP Mixtures, 32 °C, 1.0 Hz

This may be an indication that the addition of RAP to a mixture will significantly increase mixture stiffness at elevated temperatures. It is possible this increased stiffness at higher temperatures will be beneficial in terms of the resistance of the mixture to rutting. Stroup-Gardiner and Wagner (39) investigated the rutting potential of mixtures containing 15 percent RAP from Minnesota and 15 percent RAP from Georgia. They used 8,000 cycles with a loaded wheel Asphalt Pavement Analyzer in the evaluation. It was found that rutting in both RAP mixtures was about 20 percent less than the rutting observed in the control mixture.

The phase angle is also affected by the amount of RAP in the mixture Figures 4.16 and 4.17 show how the phase angle changes with the addition of RAP at –18 and 1 °C. A higher phase angle indicates a mixture has a larger viscous component.

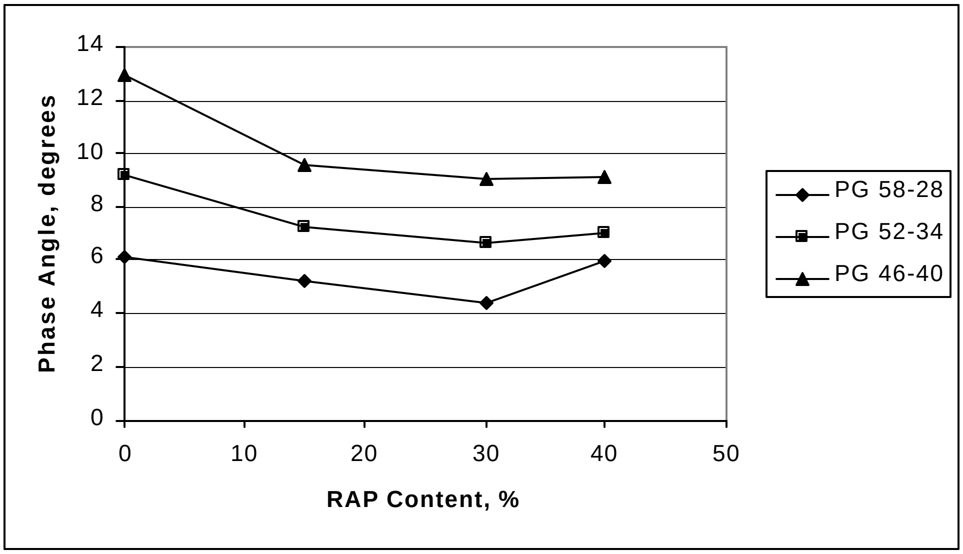

Figure 4.16 Change in Phase Angle with RAP Content, District 6 RAP Mixtures, -18 °C, 1.0 Hz

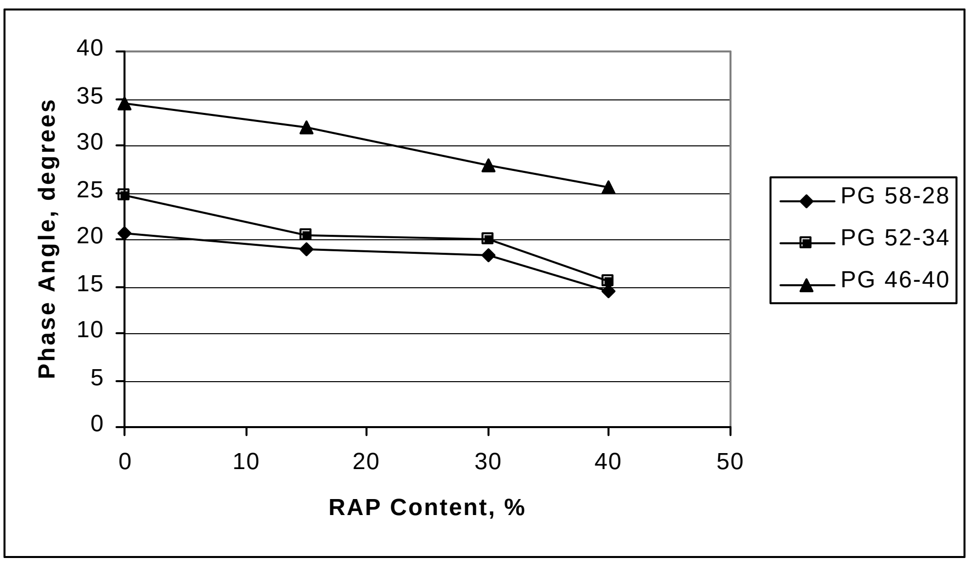

Figure 4.17 Change in Phase Angle with RAP Content, District 6 RAP Mixtures, 1 °C, 1.0 Hz

These figures show that mixtures with the stiffer asphalt binder will have a lower phase angle than those constructed with the softer asphalt binder. It would therefore be expected that as RAP is added to a mixture, the phase angle would decrease. This relationship is shown in the plots above at both –18 and 1 °C.

Effect of RAP Source on Complex Modulus and Phase Angle

Just as the asphalt binder grade and RAP content affect the mixture complex modulus and phase angle, so does the source of the RAP. Figures 4.18 and 4.19 show that the stiffer District 8 RAP asphalt produces a mixture with a higher complex modulus and lower phase angle than the District 6 RAP mixtures. Taking an average of the three asphalt grades, the District 8, 30 percent RAP mixtures had a 55 percent higher complex modulus than the District 6, 30 percent RAP mixtures at 25 °C, and a phase angle that was about 30 percent lower. This supports the results obtained in the resilient modulus testing, which showed the District 8, 30 percent RAP mixtures had a stiffness about 20 percent higher than the District 6, 30 percent RAP mixtures. It also shows the importance of knowing the stiffness of the RAP binder when considering a mixture made with RAP.

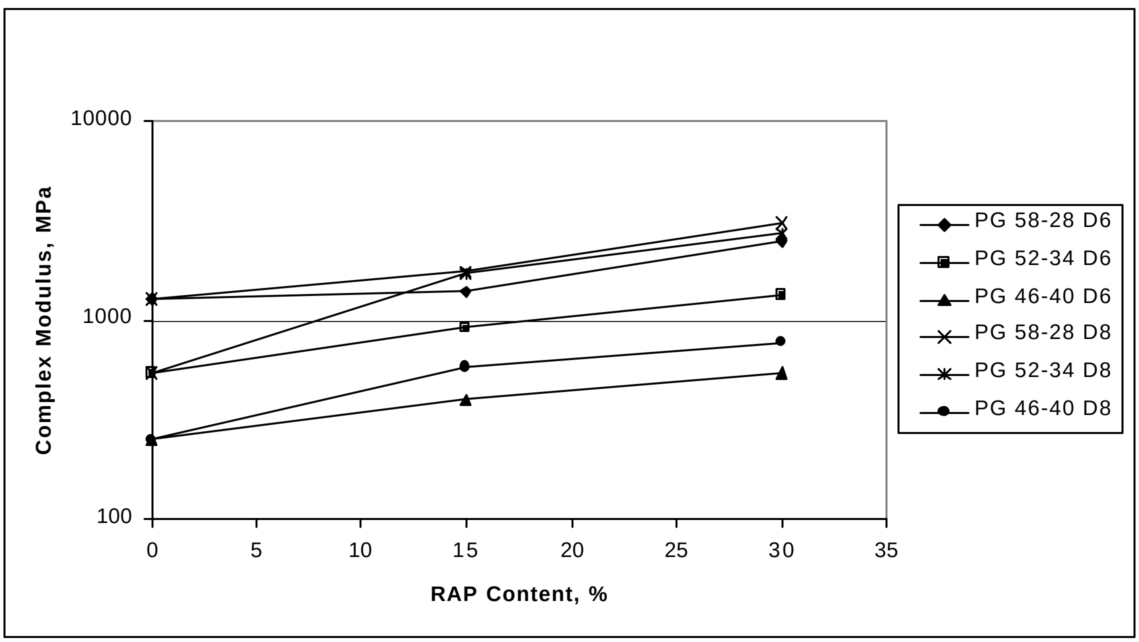

Figure 4.18 Effect of RAP Source on Complex Modulus, District 6 and 8 RAP Mixtures, 25 °C, 1.0 Hz

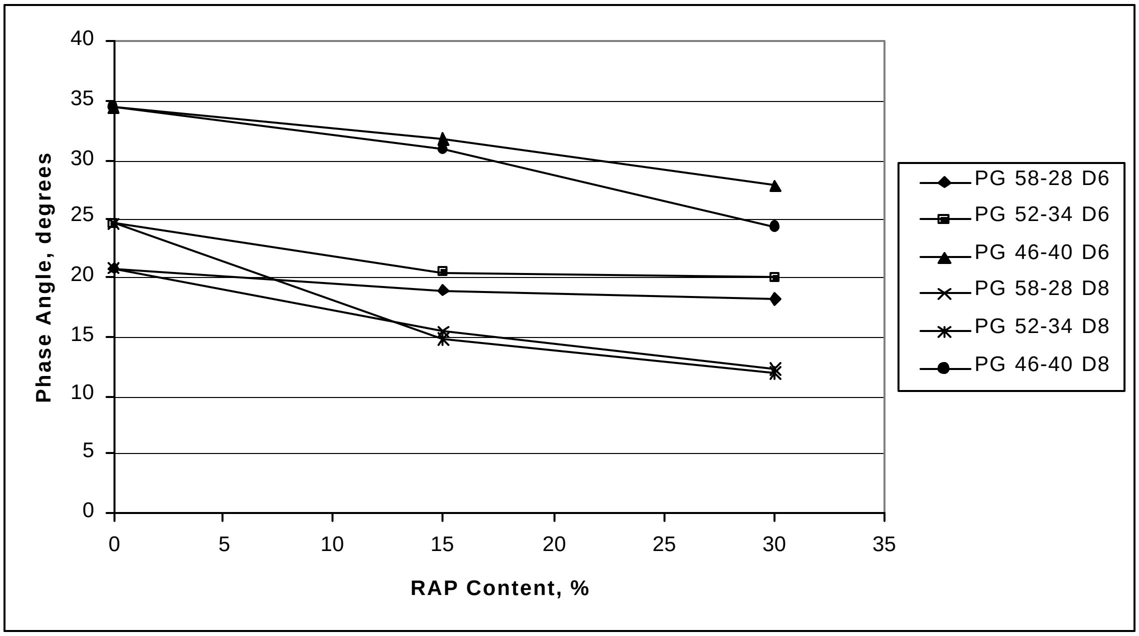

Figure 4.19 Effect of RAP Source on Phase Angle,
District 6 and 8 RAP Mixtures, 25 °C, 1.0 Hz

Examination of Load History and Effect on Phase Angle

When complex modulus testing was completed, a regression program developed by Zhang (7) was used to determine the vertical and horizontal displacements, as well as the phase angle. The regression program gave reasonable values for the phase angle at –18 and 1 °C. However, the results at 25 and 32 °C appeared to be in error. The phase angles were thought to be erroneous at the higher temperatures for two reasons. When loading frequency increased the phase angle increased. Past research by Daniel et al. (53) showed that as loading frequency increases the phase angle decreased. Second, a phase angle of nearly 90 degrees was obtained at the highest temperature with the softest mixtures. A phase angle of 90 degrees would indicate a perfectly viscous material, which is not reasonable for an asphalt concrete mixture.

It was therefore decided that the regression program should be checked for accuracy. Phase angles were calculated graphically in an attempt to determine if the erroneous phase angle measurements were a product of the test equipment or the regression program. A PG 58-28 control mixture was chosen for the graphical evaluation. Phase angles were calculated at all four test temperatures and at two or three frequencies for each temperature.

The phase angles calculated by the regression program and graphically agreed well with each other in most cases, as shown in Table 4.9. In all cases, the graphically determined phase angle estimates exceed the angle calculated by the regression program. It is important to note that both methods show the phase angle decreases with increasing frequency at –18 and 1 °C, but show the opposite at 25 and 32 °C. After making this observation, it was concluded that the regression program was not the source of the error in the phase angle determination.

Table 4.9 Comparison of Regression and Graphical Phase Angles

Temperature, °C	Frequency, Hz	Regression Angle, degrees	Graphical Angle, degrees
-18	0.1	8.3	13.1
-18	1.0	6.1	10.5
1	0.1	24.5	27.8
1	1.0	17.7	26.7
25	1.0	31.4	35.9
25	10.0	32.7	40.0
25	30.0	39.0	60.5
32	1.0	35.6	41.3
32	10.0	54.4	64.5
32	30.0	80.0	81.6

Upon closer observation of the printouts used for the graphical phase angle determination, it became evident that a true haversine waveform was not always being achieved during the complex modulus testing. In fact, the waveform was observed to degrade the most as both the test temperature and loading frequency were increased. The waveform degradation occurred in sync with the erroneous phase angle results. Figure 4.20 shows the load and deformation data at 1 °C. This plot is typical of the load and response that is expected in the complex modulus test.

When the applied load was reduced for testing at higher temperatures, the waveform deviated from a true haversine load, as shown in Figure 4.21. Since the phase angle was measured as the distance between the load and deformation peaks, the lack of a true haversine waveform was believed to be the source of the problem.

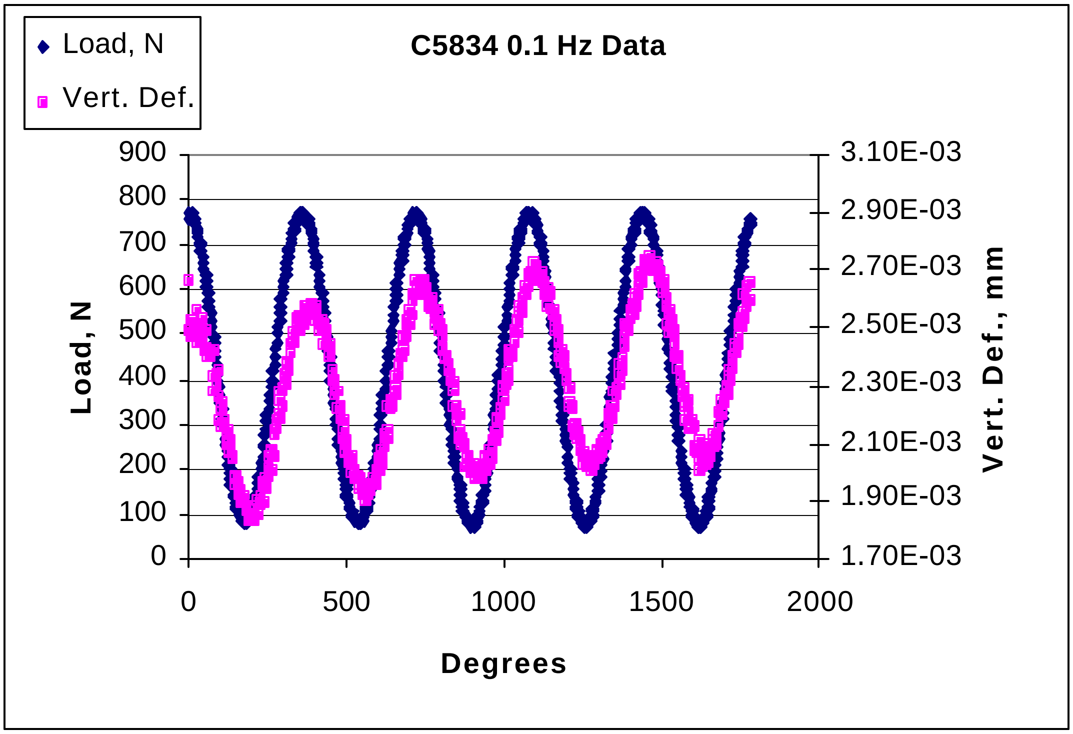

Figure 4.20 Load and Deformation Plot for PG 58-28 Control Mixture, 1 °C, 0.1 Hz

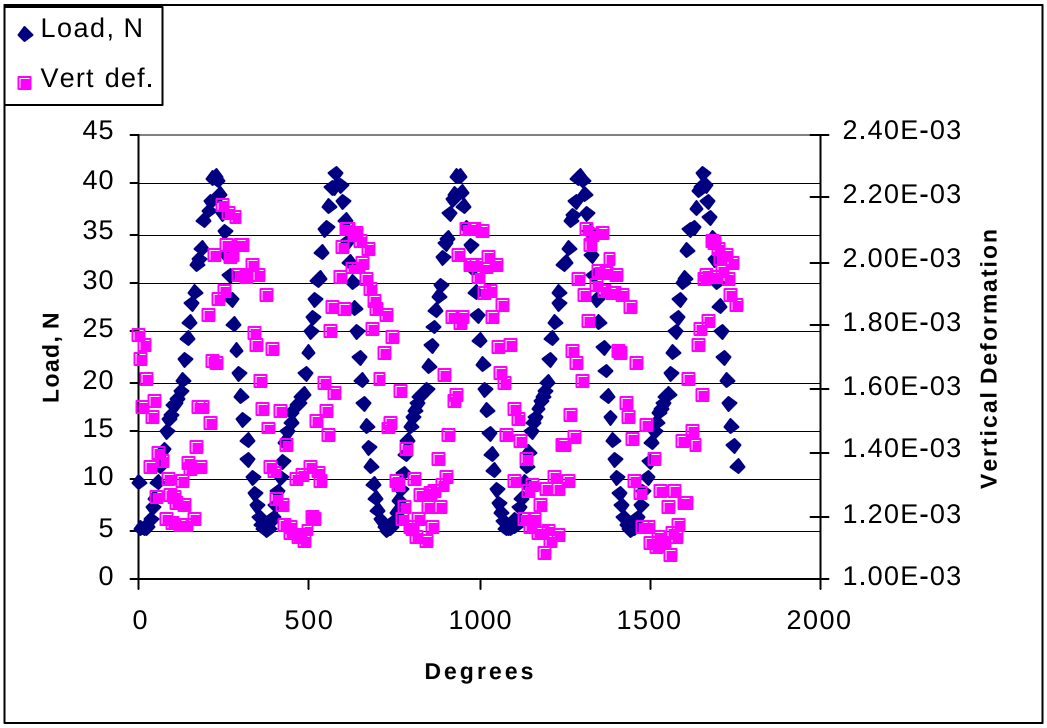

Figure 4.21 Load and Deformation Plot for PG 58-28 Control Mixture, 32 °C, 30 Hz

Since the waveform deviates from true haversine at lower load amplitudes and higher frequencies, it was concluded that the problem stemmed from the hydraulic actuator. The actuator used in this testing had a capacity of 100,000 N. At 32 °C, a load of only 40 N, or 0.04 percent of the actuator capacity, was being applied. It was likely that the actuator did not have an adequate resolution when very low loads were applied at high frequencies. An actuator with a lower load capacity should have been used for a series of complex modulus tests to confirm this hypothesis.

Comparison between 100 mm and 150 mm Diameter Samples

Earlier complex modulus testing and development of viscoelastic equations were accomplished by Zhang (7) using 100 mm diameter samples. This diameter was chosen since the Marshall compaction equipment in the laboratory was configured to produce a 100 mm sample. The recycled asphalt pavement (RAP) project used 150 mm diameter samples, as this is the size specified by Superpave and is the size produced by the Brovold gyratory compactor in the laboratory. In addition, a larger sample diameter should help to remove some of the effects of large aggregates and sample inhomogeneity.

Additional complex modulus testing was performed in order to investigate whether the test and equations developed by Zhang (7) were applicable to a 150 mm sample. Two 150 mm samples were tested using the diametral complex modulus testing procedure. The test temperature chosen was 25 °C since this temperature produced the most consistent data during earlier testing in the RAP project. The test was performed at different frequencies, ranging from 0.03 Hz to 20 Hz, as used in earlier testing.

Both samples were "control" mixtures, meaning only virgin material was used and no RAP was included. It was decided to use samples without RAP since the RAP would be another source of variability. One of the samples was made with PG 58-28 asphalt and the other with a softer PG 52-34 asphalt. The same asphalt contents were used for these samples as in the samples used for the remainder of the testing.

The 150 mm samples were tested with horizontal displacement data collected across the diameter of the sample and vertical displacement data collected across the central quarter (37.5 mm) of the sample. When these tests were completed, both 150 mm samples were cored to a diameter of approximately 100 mm. The same sequence of testing was performed on the cored samples, with a different vertical gage length. Since the central quarter of the sample was only 25 mm in length, the vertical gage clips were remounted at this distance.

Loading platens with a smaller contact radius were used for the 100 mm samples. This was done to preserve the ratio of loading strip radius to sample diameter. When this ratio is held constant, no modification of the constants in the viscoelastic equations is necessary. As a result, the data regression and analysis for the 100 mm and 150 mm samples was performed in the same manner as in earlier testing for the RAP project.

The data are presented in Figures 4.22 and 4.23. For both of the Figures C5234 100 denotes the sample was a control mixture with PG 52-34 asphalt and a diameter of 100 mm. The other names in the legend are to be interpreted accordingly.

Figures 4.22 and 4.23 show the results for a 100 mm and 150 mm to be reasonably similar. The results are said to be reasonably similar since sample diameter appears to have less of an effect on the results than the grade of asphalt used. Table 4.10 shows that the average difference in complex modulus between the 100 mm and 150 mm samples of the same asphalt binder was about 25 percent. The increase in complex modulus between a PG 52-34 and PG 58-28 sample was about 75 percent. In other words, the influence of the stiffer asphalt binder had three times the effect that the change in sample size had.

Table 4.10 Complex Modulus Results for Sample Size Comparison

Frequency, Hz	PG 52-34 100 mm, MPa	PG 52-34 150 mm, MPa	PG 58-28 100 mm, MPa	PG 58-28 150 mm, MPa
0.03	296.9	308.0	471.3	418.2
0.1	405.7	482.3	629.4	736.7
1	827.3	1076.1	1404.7	1866.4
5	1358.7	1758.1	2356.5	3218.5
10	1796.2	2298.0	3154.1	4213.4
20	2427.3	2820.3	4149.2	5179.8
Average	1185.4	1457.2	2027.5	2605.5

The problem of increased phase angle with increasing frequency is seen in the 100 mm samples. As explained earlier in this chapter, examination of the load history revealed that a true haversine load was not achieved by the hydraulic actuator at low loads and high frequencies. This is the same problem experienced in earlier testing. It is believed that this problem of increasing phase angle with increasing load frequency is not evident in the 150 mm samples because a higher load was used so that strains in both sample sizes would be roughly similar.

One potential reason why the complex modulus varies between the 100 mm and 150 mm is the sample itself. The 100 mm sample was cored from the 150 mm sample, so the sample properties should be roughly similar. However, if the gyratory sample has non-uniform density, one would expect the cored 100 mm sample to have a different modulus than the original 150 mm sample. Also, if the compactor causes a difference in particle orientation between the outer surface of the sample and the inner core, the results may again be affected.

Even though a testing program should be consistent in the sample size chosen, this small set of testing appears to show the procedure and equations used by Zhang (8) can be applied to sample diameters of either 100 or 150 mm.

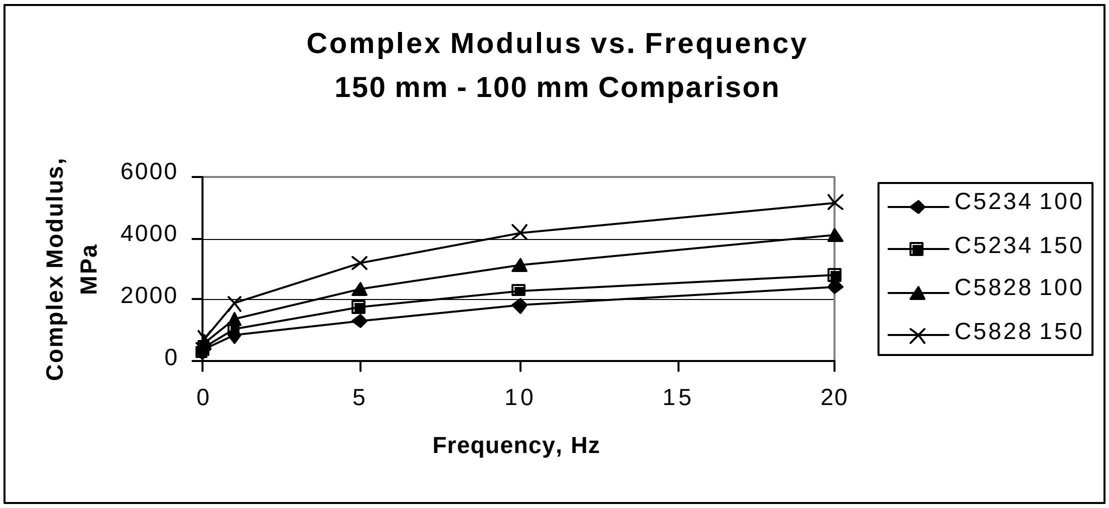

Figure 4.22 Comparison of Complex Modulus for 100 mm and 150 mm Diameter Samples

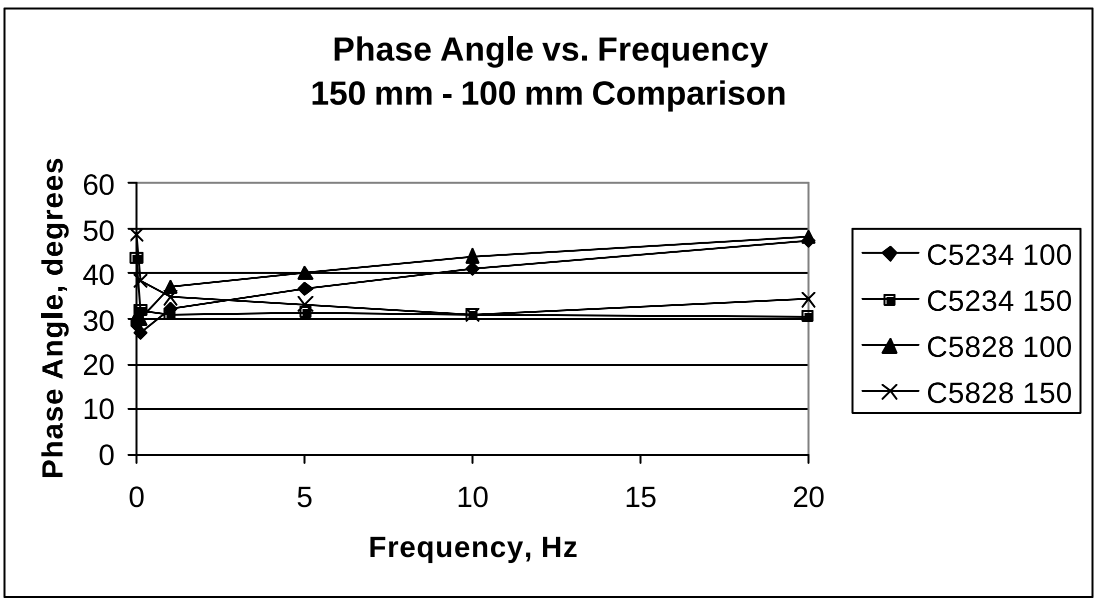

Figure 4.23 Comparison of Phase Angles for 100 mm and 150 mm Diameter Samples

CHAPTER 5
- CONCLUSIONS AND RECOMMENDATIONS -

CONCLUSIONS

1. Addition of RAP to a mixture increased the resilient modulus. At 25 °C, adding 40% percent District 6 RAP to a PG 58-28 control mixture resulted in a 74 percent increase in stiffness. A 164 percent increase was measured with a PG 46-40 control mixture. A similar increase was observed with the addition of District 8 RAP.

2. Resilient modulus tests indicated a mixture with a PG 52-34 asphalt binder and about 20 percent District 6 RAP will have a similar stiffness as a virgin mixture with PG 58-28 asphalt binder. It also showed a mixture with a PG 46-40 asphalt binder and 35 percent RAP would have a similar stiffness as a virgin mixture with a PG 52-34 asphalt binder.

3. The source of the RAP affected the resilient modulus results. The District 8 RAP binder had a higher PG grade than the District 6 RAP, and accordingly yielded a higher resilient modulus.

4. Complex modulus tests at 25 °C indicated 23 percent District 6 RAP and PG 52-34 asphalt would yield a complex modulus similar to a virgin mixture with PG 58-28 asphalt. About 28 percent District 6 RAP and PG 46-40 asphalt will give the same complex modulus as a virgin mixture with PG 52-34 asphalt. About 90 percent District 6 RAP and PG 46-40 asphalt will have a similar complex modulus as a virgin mixture with PG 46-40 asphalt.

5. Complex modulus tests at 25 °C indicated 10 percent District 8 RAP and PG 52-34 asphalt would yield a complex modulus similar to a virgin mixture with PG 58-28 asphalt. About 16 percent District 8 RAP and PG 46-40 asphalt will give the same complex modulus as a virgin mixture with PG 52-34 asphalt. About 40 percent District 8 RAP and PG 46-40 asphalt will have a similar complex modulus as a virgin mixture with PG 46-40 asphalt.

6. The source of the RAP affected the complex modulus results. The District 8 RAP binder had a higher PG grade than the District 6 RAP, and accordingly yielded a higher complex modulus and lower phase angle.

7. Equipment problems were responsible for the observed increase in phase angle with increased loading frequency. As a result, phase angle results at 25 and 32 °C are unreliable.

8. The same viscoelastic equations and test method used by Zhang (8) with 100 mm diameter samples may be applied to 150 mm diameter samples.

RECOMMENDATIONS

1. Based on resilient modulus and complex modulus test results, the RAP contents and respective asphalt binders in Table 5.1 will result in a stiffness similar to a virgin mixture:

Table 5.1 Recommended RAP Contents and Asphalt Binders

Original Asphalt Grade	Asphalt Grade with RAP	RAP Content with District 6 RAP	RAP Content with District 8 RAP
PG 58-28	PG 52-34	20 %	10 %
PG 58-28	PG 46-40	50 %	35 %
PG 52-34	PG 46-40	25 %	15 %

2. A testing system with a smaller capacity actuator should be used. The current hydraulic actuator has excessive capacity for the load amplitudes required. A true haversine waveform could probably be achieved at the low load levels and higher frequencies with a different actuator. This would likely resolve the problems experienced with phase angle measurement.

3. More samples from each mixture should be compacted and tested. This should help to reduce the variability and make the data more reliable, especially at the coldest test temperature.

4. Table 5.2 shows the test temperatures and loading frequencies recommended for diametral complex modulus testing with the current laboratory equipment. Loading frequency could likely be increased to 30 Hz if a smaller actuator was used.

Table 5.2 Recommended Test Temperatures and Frequency Range for Complex Modulus Testing

Test Temperature, °C	Minimum Loading Frequency, Hz	Maximum Loading Frequency, Hz
-18	0.03	5.0
1	0.03	5.0
25	0.03	5.0
32	0.1	5.0

5. Complex modulus and phase angle values should be related to pavement performance. Performance evaluation could be performed at accelerated loading facilities and full-scale test tracks such as Mn/ROAD.

6. Additional work should be performed to relate complex modulus and resilient modulus. Since the complex modulus test provides more information about mixture properties, it should replace resilient modulus as the standard test performed for mixture evaluation.

7. Moisture susceptibility of RAP mixtures should be evaluated in the field to verify that the addition of RAP does not adversely affect mixture durability. Field testing should also be performed to correlate field performance to laboratory testing.

8. Low-temperature creep compliance testing should be performed on all RAP mixtures. This would help to determine if the addition of RAP will cause the mixture to become brittle at low temperatures, possibly increasing the frequency and severity of thermal cracking in the field.

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