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DESIGN AND EVALUATION OF COLD IN-PLACE RECYCLED PAVEMENTS

by

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and
M. E. Bienvenu

DEPARTMENT OF CIVIL ENGINEERING

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The report is a descriptive study focused on communities that identified access to food as an issue. Access is defined as availability of variety of nutritious and affordable food from stores and market that are well maintained and that offer products reflecting the cultural make-up of the community. This report intents 1)to determine the types of strategies these underserved communities employed to reduce or eliminate barriers and 2) to assess the effectiveness of each strategy. Information was obtained through an extensive review of the literature and a survey of individuals with some expertise in food access.
The Annotated Bibliography, a separate volume, is intended to serve as a guide to the literature on food access strategies, issues, and developments. It includes a list of over 300 printed documents.
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16. Abstract

Cold In-place Recycling appears to be a cost effective rehabilitation method for some flexible pavements. However, difficulties exist in constructing projects due to non-standard mixture design and analysis techniques. As a part of the cold mix recycling process, the old and aged mix is milled out and is mixed with a virgin aggregate and additional asphaltic rejuvenator to make the recycled mix meet the standard specifications. The recycling agents or rejuvenators for cold mixes are emulsified asphalts, either cationic (CMS) or anionic with high-float characteristics (HFRA). The mix design for cold mixes containing emulsified asphalts is significantly different from the conventional hot asphalt mixes. Currently, there is no standard procedure for design of cold emulsified asphalt recycled mixes. The objectives of this study were to: (1) Develop a methodology for cold mix design using CMS and HFMS emulsions as recycling agents; (2) Evaluate and characterize these cold mixes in the laboratory and compare the likely performance with the conventional hot asphalt mixes; (3) Through laboratory characterizations, evaluate the structural contribution of cold recycled mixes in pavements; and (4) Develop guidelines for design and construction of cold mix binder layers.

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ABSTRACT

The purpose of this research was to utilize state of the art technology to develop a design and analysis method for cold-recycled mixtures. There is no current universally accepted design method for cold recycling of asphalt pavements. The objective of this research was to devise a simplified method of determining the optimum asphalt content using the maximum allowable amount of reclaimed asphalt pavement (RAP) in the cold-recycled mix based on material properties. The maximum allowable RAP content of the recycled mix in this study was determined to be 75%. This study introduces adaptations of SUPERPAVE™ technology for binder characterization and develops a binder blending chart for use in cold-recycled mix design. The binder blending chart is valuable in determination of maximum allowable RAP content of the recycled mix, the acceptable ranges of recycling agent residue in the binder blend, and in selection of the most desirable recycling agent for the RAP being recycled. The study also recommends mix and curing procedures for the production of test specimens and the incorporation of mechanical tests (resilient modulus and indirect tensile strength) and pavement prediction models in the determination of the optimum recycled mix. As a means of supporting the results of the mixture testing methodology, SUPERPAVE™ volumetric and intermediate mix testing are utilized for comparison of results, though the SUPERPAVE™ mix tests are not components of the recommended design methodology. The procedure which results from this research provides a more reliable means of cold-recycled mix design and analysis by the incorporation of better analysis tools for material characterization, especially in selection and proportioning of the most appropriate recycling agent for a particular project.

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CHAPTER 1

INTRODUCTION

The basic concept of pavement recycling lies in the conservation of total energy required to rehabilitate deteriorated pavement, the conservation of materials through reuse of old pavement and the reduced need for new materials, and the preservation of the environment by eliminating the necessity for disposing of old materials. The recycling concept has long been deemed to be ideal but seldom realized due to the fact that pavement recycling was more expensive than traditional new construction. It is now time, however, for such an ideal concept to be realized because of changes in the economic climate, and in actuality, recycling construction techniques have been developed and widely used. Furthermore, recent advances in technology available for material characterization and mix analysis have improved the reliability of recycled mix design.

Cold In-Place Recycling (CIPR) appears to be a cost effective rehabilitation method for some flexible pavements. However, difficulties exist in constructing projects due to non-standard mixture design and analysis techniques. As a part of the cold recycling process, the old and aged mix is milled out and is mixed with a virgin aggregate and additional asphaltic rejuvenator to make the recycled mix meet the standard specifications. The recycling agents or rejuvenators for cold-recycled mixes are emulsified asphalts, such as cationic (CMS) or anionic with high-float characteristics (HFRA). The mix design for cold mixes containing emulsified asphalts is significantly different from the conventional hot asphalt mixes.

Currently, there is no standard procedure for design of cold emulsified asphalt recycled mixes. As the use of cold in-place recycling in North Carolina becomes widespread, a standard mix design procedure for emulsified cold mixes will need to be developed. In addition, laboratory characterization of such mixes will also be needed for employment in overlay design.

1.1 The Recycling Process

Asphaltic mixtures to be recycled usually contain aged and very much hardened or high viscosity asphaltic materials. Restoration of viscosity, known as rejuvenation, is required for the reuse of such mixtures from the standpoint of mechanical properties such as stability, flexibility, and durability as well as workability at the time of placement. One of the methods of rejuvenation is to add a material with a lower viscosity to old mixtures. Materials such as cutback asphalts and emulsified asphalts have been utilized as "rejuvenating agents" or "softening agents" for cold mix recycling. However, the emulsified asphalts are the most commonly used recycling agents employed for this purpose.

The reaction taking place inside a mixture upon application of a softening agent is a solution. The softening agent reacts with the aged and hardened film of a bituminous material which covers an aggregate and dissolves it, reducing the viscosity locally. Reaction at the interface may occur in a short time period, but the deeper portion of the film may react later than at the surface, creating a viscosity gradient within the mixture. This time dependency of the softening effect might account for unexpected deficiencies in the pavement such as lateral flow or lack of stability in the long run, even when the recycled mixture shows adequate properties initially.

1.1.1 Conceptual Model Of Softening Effect

A conceptual model of the long term effect of softening may be visualized in Figure 1.1 [1]. An assumption may be made that there are certain acceptable limits of viscosity, η_{min} and η_{max}. If the viscosity of asphalt becomes greater than η_{max}, the mixture may be too hard and crack inducement may result. If the viscosity becomes softer than η_{min}, instability and permanent deformation may result. A mixture with the viscosity falling within the limits of η_{min} and η_{max} is expected to be stable.

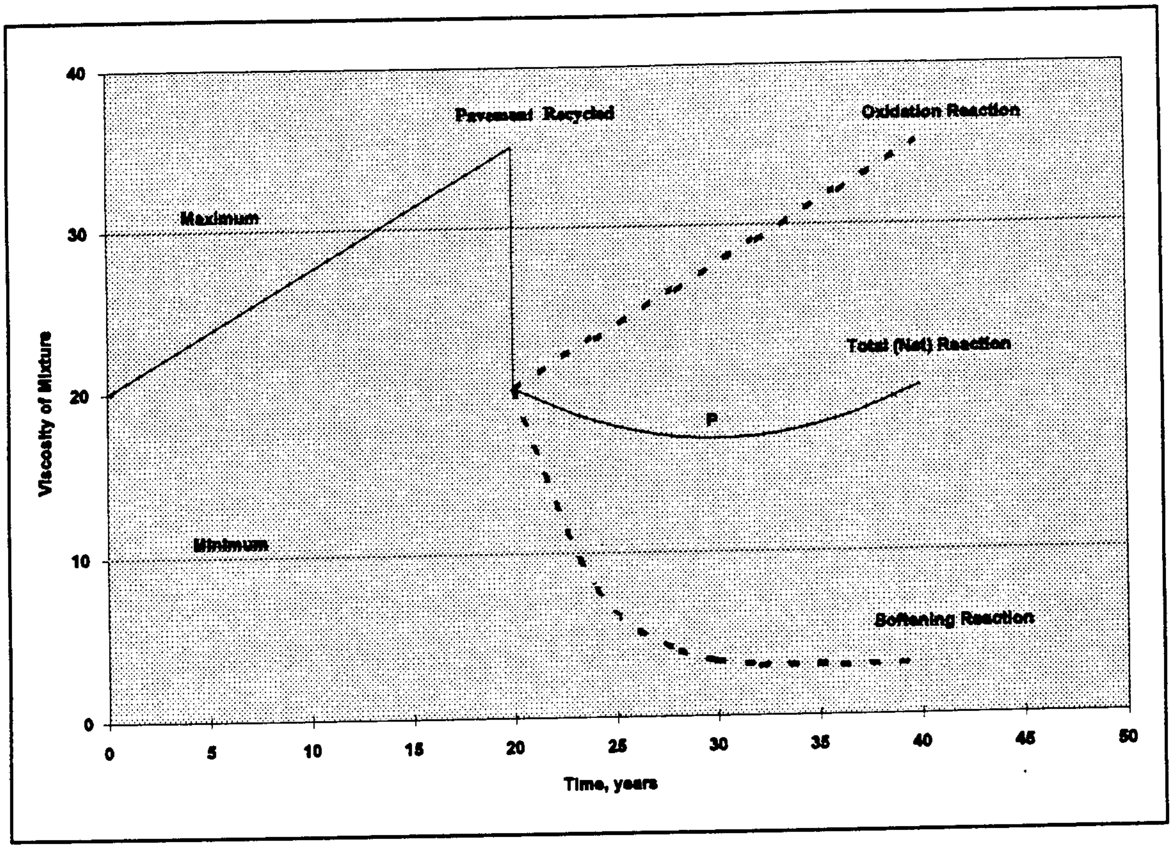

Figure 1.1 Conceptual Diagram of Softening Effect (from reference 1)

At the time of the initial construction, the viscosity may well be within the specified limits. However, as the oxidation of asphalt progresses, the viscosity increases, and may exceed η_{max} at some future time. As a result, deterioration may occur and restoration would be required. In Figure 1.1, the pavement is recycled at year 20.

Cold-recycling is one of the methods of restoration of deteriorated pavements. Upon application of a softening agent, the initial viscosity may be within the desirable limits and may continue to decrease over a long term. However, at the same time, oxidation is taking place within the mixture, resulting in a simultaneous increase in the viscosity. It can be expected that the viscosity of the binder after recycling is the result of a combination of the two reactions, softening and oxidation hardening. The softening will cease at a certain time depending upon the behavior and the amount of the softening agent applied, which means that the viscosity has a minimum point, designated P in Figure 1.1. If P stays within the limits, deterioration will not take place. Otherwise, the mixture will experience a lack of stability, and rutting may occur.

1.1.2 A Tool To Measure Viscosity

In order to measure the viscosity of the binder, it is possible to extract the asphalt from the recycled mix and conduct consistency tests on the recovered material. The problem with this approach is that it does not reflect the reaction as it occurs in cold-recycling.

The cold-recycling reaction may be schematically shown in Figure 1.2 [1]. Before being recycled, an aggregate in the mixture is coated with a film of old bituminous material with high viscosity (Figure 1.2-A). Upon application, the softening agent adheres to the old material and develops a very thin film (Figure 1.2-B). The softening agent then begins to penetrate into the old material (Figure 1.2-C). In the long run, the outer part of the softened material begins to be oxidized and becomes harder (Figure 1.2-D).

The steps can be understood by comparing Figure 1.2 with Figure 1.1. If the extraction and recovery procedure described above is used, the average viscosity of the blend may be measured and the difference between the steps B and C of Figure 1.2 may not be distinguished. It is this time-dependency of the softening and hardening effect which creates the difficulty in cold-recycling mix design.

Here it must be recalled that it is the mixture itself that makes up the pavement, not the bituminous material alone. One of the possibilities to relate the viscosity of the binder to a mixture property lies in the concept that bituminous mixtures are viscoelastic. This suggests the possibility of obtaining the change in the viscosity of asphalt through the viscosity of mixture. Since the viscosity of a binder is functionally related to the viscosity of mixture, the conceptual model in Figure 1.1 is applicable to the mixture as well.

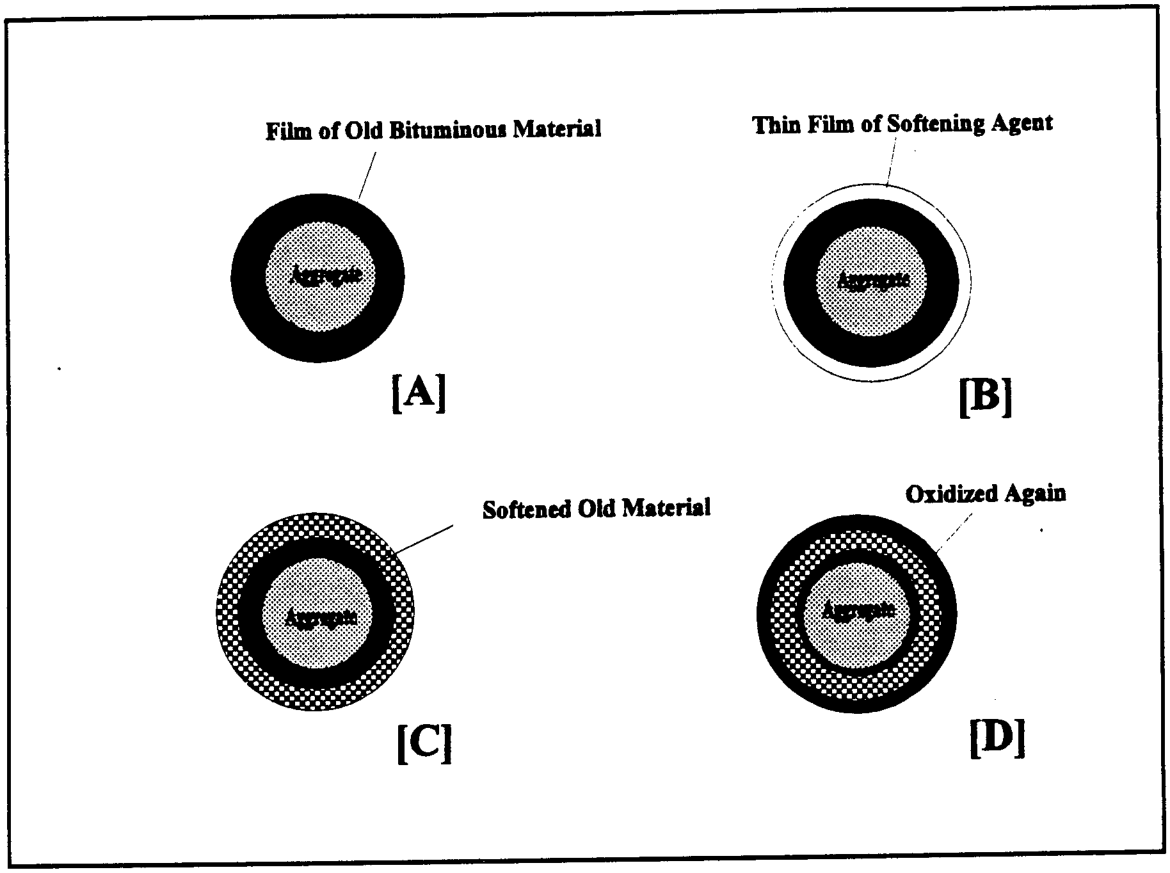

Figure 1.2 Schematic Diagram of Reactions (from reference 1)

A proper design procedure for recycled mixtures has not yet been established in a satisfactory manner. One of the deficiencies is that any proposed procedures do not take into account the long-term behavior of mixtures. As stated earlier, recycled mixtures behave in a way different from virgin mixtures in principle, and some evaluation procedure of the long-term behavior should be incorporated in the proper design.

It is a well known fact that a virgin mixture made through a cold process takes a longer curing time to obtain a certain level of strength than that made through a hot process. This experience means that a cold recycling process may be more critical in terms of the time to open a recycled road for use after construction. Thus as discussed, long-term observation is quite necessary in establishing a proper design method, especially in the case of cold-recycled mixtures.

To study the effect of curing on strength gain, a non-destructive test like "resilient modulus" is proposed to be utilized. An appropriate procedure will be utilized to simulate the curing that occurs in the field. The time at which the rate of gain in resilient modulus becomes minimal signifies the completion of curing and identifies the structural strength of this mix available in a pavement when opened to traffic (Figure 1.3). Therefore, the ultimate strengths of a recycled mix is assumed to be the strength exhibited when the measurable change in resilient modulus of the recycled mix approaches zero.

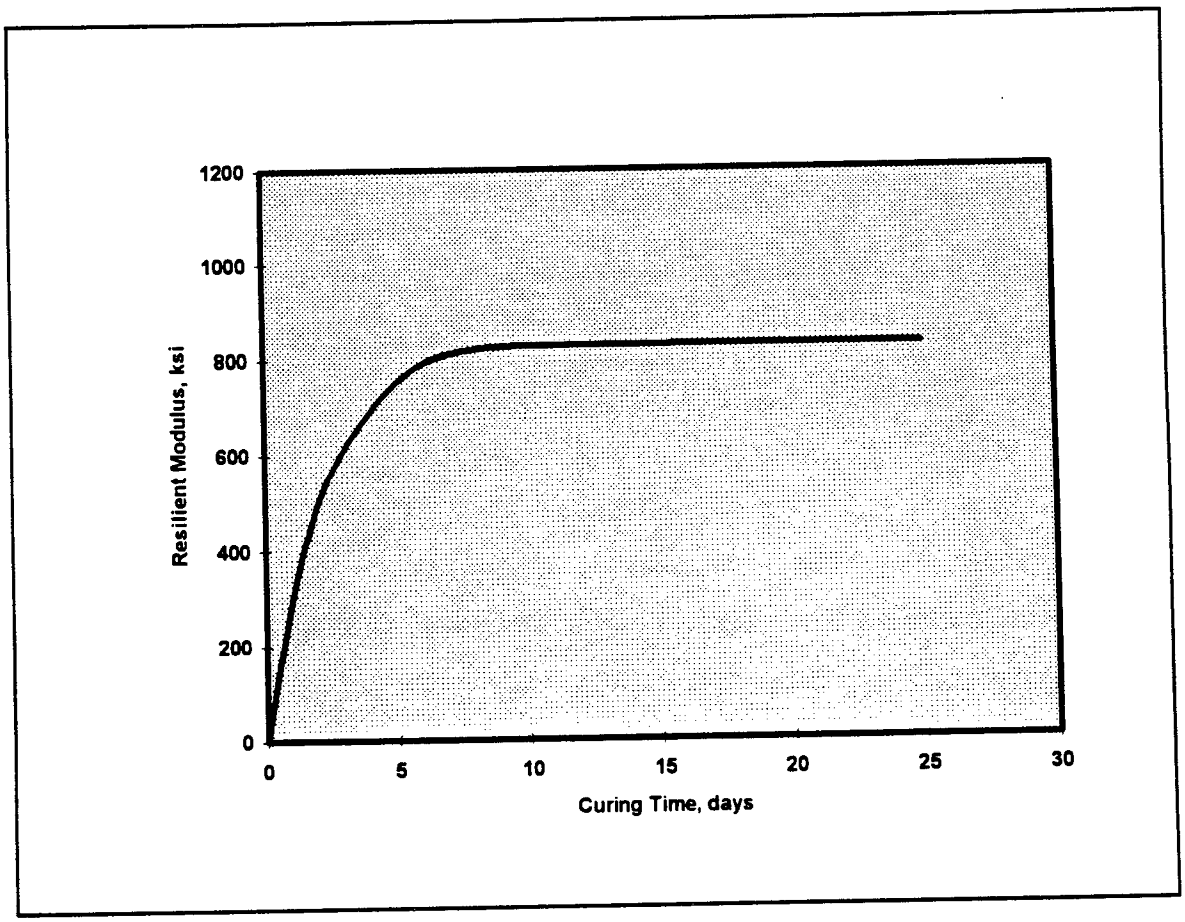

Figure 1.3 Effect of Curing Time on Resilient Modulus (from reference 1)

1.2 Objectives And Scope Of Study

The objectives of this study were to:

1) Present and validate a binder mixing chart for cold-recycled mix design and analysis;
2) Determine the maximum allowable RAP content of the cold-recycled mix;
3) Develop a reliable cold-recycled mix design method for the pavement binder layer utilizing new technology;
4) Evaluate the structural contribution of the cold-recycled mix to the pavement.

The scope of this project was limited to the following:

1) CMS-2 and HFRA recycling agents were used in the development of the design procedure.
2) A given RAP supply with unknown material characteristics was used.
3) Development of a cold-recycled mix design for the binder pavement layer was based on current design specifications for North Carolina.
4) The SUPERPAVE™ binder grade specification of PG64-16 was assumed.
5) A bending beam rheometer (BBR) was not available for low temperature PG grading, but an assumed low temperature grade of (-16) was used for determination of intermediate temperature fatigue characteristics.

1.3 Research Approach And Methodology

The general research approach to cold-recycled mix analysis and design can be summarized as follows:

1) Evaluate the RAP material;
2) Determine the new aggregate requirements for binder layer in North Carolina;
3) Characterize the recycling agent residues;
4) Develop binder blending charts for the RAP and recycling agents;
5) Prepare and test cold-recycled mix specimens with the maximum allowable RAP content;
6) Evaluate the results.

The following mix tests and analyses were used to characterize the cold-recycled mixtures:

1) Resilient modulus tests;
2) Indirect tensile strength tests
3) Shear stiffness tests;
4) Fatigue analysis;
5) Permanent deformation analysis.

An attempt was made to characterize the cold-recycled mixtures with respect to strength parameters recommended in SUPERPAVE™ mix design. Volumetric principles were used to develop mix specimens and SUPERPAVE™ Intermediate testing was performed at critical temperatures for rutting and fatigue analysis. In the absence of SUPERPAVE™ performance prediction models, comparisons of rutting performance among the different mixes tested and a surrogate fatigue model were used to predict relative fatigue performance. This analysis approach culminated in a procedure for design and evaluation of cold-recycled mixtures. Comparisons between cold recycled mixtures and conventional hot asphaltic mixtures were also made.

CHAPTER 2

LITERATURE REVIEW

Pavement recycling has been utilized since 1915 [2]. Much of the development of mix design procedures has occurred in the last 20 years due to increased emphasis on economic and environmental considerations in highway construction and pavement rehabilitation. In Allegheny County, Pennsylvania, the engineering services department reported that using 100% of the milled asphalt pavement in cold-in-place recycling not only eliminated the need for stockpiling RAP or sending it to a landfill, it also reduced project costs by 35% to 55% [3].

Design methods currently utilize a variety of procedures ranging from "in-the-field" design to more sophisticated methods which adapt Marshall and Hveem mix design. The goal of mixture design is to obtain the optimum asphalt content for a desired gradation.

Material characterizations normally include analysis of the RAP and virgin materials to be added during the recycling process. RAP analysis includes determination of existing asphalt content, existing aggregate gradation and grading or characterization of the reclaimed asphalt. Aggregate is added to RAP to achieve a desired gradation, to allow for the addition of new asphalt or rejuvenator, or to provide additional thickness [4]. Rejuvenators, recycling agents or emulsions are added to the RAP to soften the existing asphalt and to coat and bind new aggregate within the mix.

Aggregate blending is accomplished by traditional methods of achieving desired gradations. Selection of a recycling agent is dependent upon the condition of the existing asphalt, the asphalt content of the RAP, and the amount of recycling agent required, as well as, workability, availability, and cost. Traditionally, emulsion residues and RAP asphalt have been penetration graded at 77^0 F or viscosity graded at 140^0 F or both.

2.1 Design Procedures

A few state agencies have developed their own design procedures for cold recycling. Some of the states or agencies which have developed their own cold recycling procedures are California, Oregon, Chevron USA, Pennsylvania, and the Asphalt Institute. Brief reviews of these methods are discussed below. The remaining states which utilize cold recycling as a rehabilitation method either borrow design procedures from other states or agencies or allow contractors to design mixtures in the field based on experience.

2.1.1 California Method [2]

The California method includes evaluation of RAP, determination of asphalt demand and a viscosity determination of the recycling agent base asphalt. Evaluation of RAP includes asphalt content, gradation of aggregate and asphalt viscosity. Aggregate adjustments are made as needed. The asphalt demand is estimated using the aggregate surface area formula.

A viscosity blending chart is used for selection of recycling agent grade. The target blend is 4000 poises at 140^0 F. Figure 2.1 shows an example of the viscosity blending chart. The RAP asphalt viscosity is plotted on the left margin of the log viscosity vs. percent recycling agent in the asphalt blend. The viscosity of the recycling agent is plotted on the right margin. The design recycling agent is the one in which the lines connecting the viscosity of the RAP asphalt with the viscosities of the prospective recycling agents intersects the AR 4000 line at the percent recycling agent in the asphalt blend determined by the aggregate surface area method. The optimum asphalt content is the one with the highest percent emulsion which has a minimum 4% air voids and a minimum Hveem stability value of 30 at 60^0 C without flushing or bleeding.

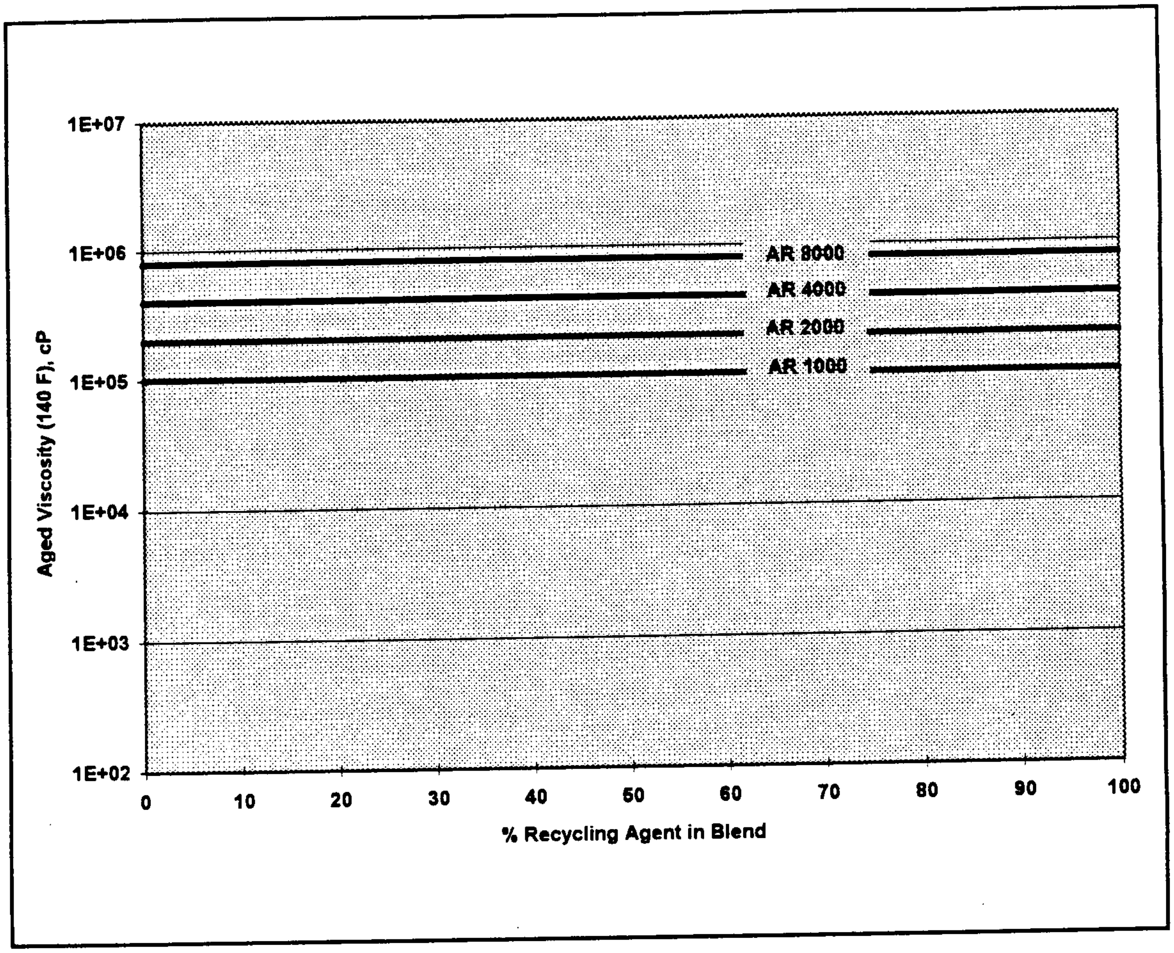

Figure 2.1 Viscosity Blending Chart - California Method (from reference 2)

2.1.2 Chevron USA Method [2]

The Chevron USA method requires evaluation of RAP and determination of asphalt demand by either the centrifuge kerosene equivalent (CKE) or the aggregate surface area method. The design procedure requires a minimum of 2% emulsified recycling agent in the mix. Recycling agent selection is based on the viscosity of the RAP asphalt, with softer emulsion residues recommended for highly aged RAP asphalt.

Optimum asphalt content selection is based on surface coating of aggregate and mix analysis including resilient modulus and Hveem tests. After final curing, resilient modulus values must be between 150 and 600 ksi at 23^0 C, Hveen stability must be a minimum of 30, and Hveem cohesiometer values must be a minimum of 100 at 60^0 C.

2.1.3 Oregon Method [2, 5]

The RAP is evaluated for asphalt content, asphalt softness and RAP gradation. A table based on the RAP characteristics determined from the evaluation with a base emulsion content of 1.2% by weight of the recycled mix is used for estimation of required emulsion content.

Test samples are compacted in a Hveem compactor at emulsion contents equal to the estimated value and ±0.3%, ±0.6% and ±0.9%, cured for 96 hours and Hveem stability and resilient modulus are determined. The optimum emulsion content is the peak of the stability and resilient modulus curves.

2.1.4 Pennsylvania Method [2]

RAP evaluation includes asphalt content, aggregate gradation and asphalt stiffness. The design incorporates up to 50% new aggregate and the addition of over 2% to 3% emulsion. The estimated asphalt demand is determined using the aggregate surface area formula. Resilient modulus at 77^0 F and Marshall stability and flow are used to determine the optimum asphalt content.

2.1.5 Asphalt Institute Method [4]

Material evaluations using the Asphalt Institute method include RAP aggregate gradation and asphalt content. Suitability of RAP aggregates for cold recycling is dependent upon the plasticity index or the sand equivalent test. New aggregate is added to allow for gradation adjustments or increasing pavement thickness. Selection of recycling agent is based on fines content of the RAP and workability of the recycled mix during construction.

The amount of recycling agent required is determined by the aggregate surface area method or the centrifuge kerosene equivalent. Some mixture tests for determining the optimum asphalt content recommended by the Asphalt Institute for cold mix design include resilient modulus and modified Hveem or Marshall test methods, or the McConnaughay method [6].

2.2 Research at North Carolina State University

The Highway Materials Laboratory at NCSU conducted analyses of cold recycled mixes using CMS-2 and HFRA recycling agents. The goal was to develop a mix design method that would simulate field curing and allow for laboratory determinations of optimum asphalt contents and maximum mix strengths [7].

It was discovered that the optimum curing procedure included 72 hours in a forced-draft oven followed by curing under an infra-red lamp for 4 hours per day for 10 days with the lamp height set so that the surface of the sample was 60^0 C (140^0 F). The resilient moduli of the specimens approached maximum values after 10 days under the infra-red lamp. Longer curing times did not have an appreciable effect on strength. Some of the tests used in the research project included resilient modulus, creep and fatigue analysis.

2.3 SUPERPAVE™ Binder and Mixture Testing

SUPERPAVE™ binder testing allows for the predictability of the rutting potential and fatigue resistance of the binder at several test temperatures under a variety of load frequencies [8]. Mixture characterizations are also possible using the SUPERPAVE™ shear test system. Shear test results are intended to be input into mechanistic models which can predict rutting and fatigue life. Glitches in the models have forced a delay in the release of the software programs. However, the test results are still valuable in comparing mixtures with variable levels of asphalt content and different recycling agents.

2.3.1 Binder Characterization

The binder characterization uses the Dynamic Shear Rheometer (DSR). Binder analysis allows researchers to determine if a binder is suitable for the environmental and loading conditions of a specific geographic location. This performance grading method characterizes the binder under dynamic loads simulating in-service conditions.

A recent research project at NCSU using the DSR analysis studied the effects of fines on the performance and aging characteristics of binders. The study indicated that binders with high fines contents had increased stiffness over the same binder with no fines [9]. It was also found that the presence of fines accelerated the long-term aging behavior of the binder. This could account for the findings in the literature that RAP with high fines contents perform better with a softer recycling agent and that stiffer recycling agents are preferred with larger gradations.

The DSR is useful in determining the aged condition of the reclaimed RAP asphalt which, in turn, allows for the selection of the appropriate recycling agent for mix design, i.e., softer recycling agents for more highly aged RAP. Additionally, the RAP binder can be long-term aged using SUPERPAVE™ aging methods and the aging potential of the RAP asphalt can be determined. The extent of and potential for aging are useful in determining the RAP asphalt's propensity for fatigue failure under in-situ conditions.

2.3.2 Mix Characterization

Once the initial trial material compositions of the cold recycled mix are determined, specimens prepared at varying asphalt contents can be tested with the SUPERPAVE™ Shear Test System. This system allows for dynamic shear testing under selected environmental conditions. These tests are intended to simulate field stress conditions and replace non-mechanistic testing procedures. The slope of the log complex shear modulus versus log frequency from the frequency sweep at constant height test (FSCH) is the same as the slope of the creep compliance curve used to determine the relative temperature susceptibility and rutting potential of mixes [10]. Therefore, the FSCH can be used in place of the standard creep test to evaluate relative rutting potential of the mixes. In addition, the complex shear modulus is a measure of the mix stiffness under a dynamic shear load which simulates the tire load in pavements. Even though SUPERPAVE™ rutting prediction software is not available, the test procedures are still reliable indicators of relative performance at different asphalt contents.

The frequency sweep at constant height test also provides the values for shear complex modulus, shear storage modulus and shear loss modulus. As a current substitute for the SUPERPAVE™ fatigue prediction model, an empirical fatigue model developed in conjunction with the SUPERPAVE™ testing system can be used to predict the fatigue life of a mix in terms of equivalent single axle loads (ESALs) or numbers of supply loads. This fatigue model uses the shear loss modulus from the frequency sweep at constant height test to estimate the flexural loss modulus and then estimate the N_{supply} in terms of ESALs of the mix under a predetermined initial strain and certain volumetric properties of the mix sample [11].

The SUPERPAVE™ Volumetric mix design method is intended for pavement life traffic levels less than 1 million ESALs [12]. In addition to establishing gradation requirements the development of the design procedure related the number of a specified number of gyrations of the SUPERPAVE™ Gyratory Compactor to pavement performance [13]. The initial design criteria were developed for hot-mix asphalt design using unmodified asphalt binders. Adaptation of Volumetric design procedures to cold mix pavement design or cold recycled pavement design is questionable. However, cognizance of gradation requirements and other mixture qualities must be maintained throughout the design and analysis of any asphalt concrete pavement.

SUPERPAVE™ Intermediate mix design is intended for pavements with design ESALs of 1 million to 10 million [12]. The frequency sweep at constant height test and the simple shear test at constant height are performed at T_{eff} for permanent deformation and fatigue in the specific geographical location in which the pavement is to be constructed [10, 12, 14]. SUPERPAVE™ Intermediate mix analysis and design was intended to utilize rutting, fatigue and low-temperature performance models for the purpose of selection of pavement mixes which best suit the loading and environmental conditions anticipated for the pavement being designed. Though the software is not yet available, Intermediate test methods are still valuable tools for selection and design of asphalt concrete pavements or for validation of other mix test results.

CHAPTER 3

MATERIAL CHARACTERIZATION

The first step in cold recycled mixture design is material characterization. Cold-recycled pavements are basically mixtures of RAP, virgin aggregate and asphalt emulsion recycling agents. Each material was characterized in this study in order to provide a basis for cold recycled mixture design.

3.1 Reclaimed Asphalt Pavement (RAP)

The RAP used in this study was milled surface mixture. It was obtained from a single stockpile provided by a hot-mix asphalt supplier in Raleigh, NC. In order to ensure sample consistency, the RAP was stored in two 55-gallon drums in the materials laboratory of the Civil Engineering Department at North Carolina State University (NCSU). For binder testing and RAP aggregate gradation, the RAP asphalt was extracted by the North Carolina Department of Transportation (NCDOT) materials laboratory.

The test method described in *ASTM D2041* (*Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures*) was used to determine the specific gravity of the RAP. The RAP specific gravity (G_{rap}) was 2.41.

3.1.1 Aggregate Gradation

After extraction of the asphalt, the NCDOT materials laboratory graded the RAP aggregate. The gradations are reported in Table 3.1

Table 3.1 RAP Aggregate Gradation

Sieve Size	Percent Passing (%)				
	1	2	3	4	Average
1 "	100	100	100	100	100
3/4 "	100	99	98	100	99
1/2 "	97	97	93	96	96
3/8 "	92	91	89	89	90
# 4	76	74	74	72	74
# 8	59	58	58	56	58
# 16	45	45	44	43	44
# 30	34	33	33	32	33
# 50	23	23	22	22	22
# 100	15	14	14	14	14
# 200	8.6	8.3	8.1	8.4	8.5

3.1.2 Binder Characterization

The average asphalt content of the RAP was 4.5%. The recovered RAP binder was characterized at the NCSU materials laboratory using the Bohlin Dynamic Shear Rheometer (DSR) in accordance with *AASHTO Test No. TP5 (Determining the Rheological Properties of Asphalt Binder Using a Dynamic Shear Rheometer)*.

One modification in the test procedure was that the binder was not short-term aged in the Rolling Thin Film Oven (RTFO) prior to long-term aging in the Pressure Aging Vessel (PAV). RTFO aging is a laboratory procedure used to simulate short-term aging of an asphalt cement due to volatilization associated with the high mixing temperatures of hot-mix asphalt construction. Such short-term aging is not present in cold-mix recycling and was omitted from the characterization protocol.

To determine the aged condition of the recovered asphalt cement, testing was conducted on the recovered and PAV-aged specimens. Long-term aging of the recovered asphalt was in accordance with *AASHTO Test No. PP1 (Accelerated Aging of Asphalt Binder Using a Pressure Aging Vessel)*. Investigations into the effects of long-term aging on the recovered asphalt were necessary to determine the condition of any RAP asphalt that may not blend with the recycling agent. Fatigue susceptibility would be of particular interest in any unsoftened RAP asphalt.

The testing format included frequency sweeps in the controlled-strain mode at temperatures ranging from 40^0 C to 10^0 C in six-degree decrements and from 40^0 C to 70^0 C in six-degree increments. The dynamic viscosities of the RAP binder in the unaged (recovered) and PAV-aged conditions are shown in Figure 3.1. The curves were constructed using the principle of superposition and the time-temperature correspondence principle of thermorheologically simple materials. The reference temperature was 28^0 C.

The curves indicated that some oxidative aging of any unblended RAP asphalt would occur over the life of the recycled pavement. At lower load frequencies (or higher temperatures), the PAV-aged RAP asphalt had higher viscosities, but the curves converged at higher load frequencies (or lower temperatures).

Based on SUPERPAVE™ criteria which were established at the time of this study, the necessary performance grade of asphalt cement for 98% reliability in North Carolina is PG64-16[8]. This corresponds to DSR testing in the unaged condition at 64^0 C and in the PAV-aged condition at 28^0 C.

Fatigue is the critical failure mode in aged asphalt cement. The fatigue cracking factor, $G^*\sin(\delta)$, is used to indicate the susceptibility of aged asphalt cement to fatigue cracking.

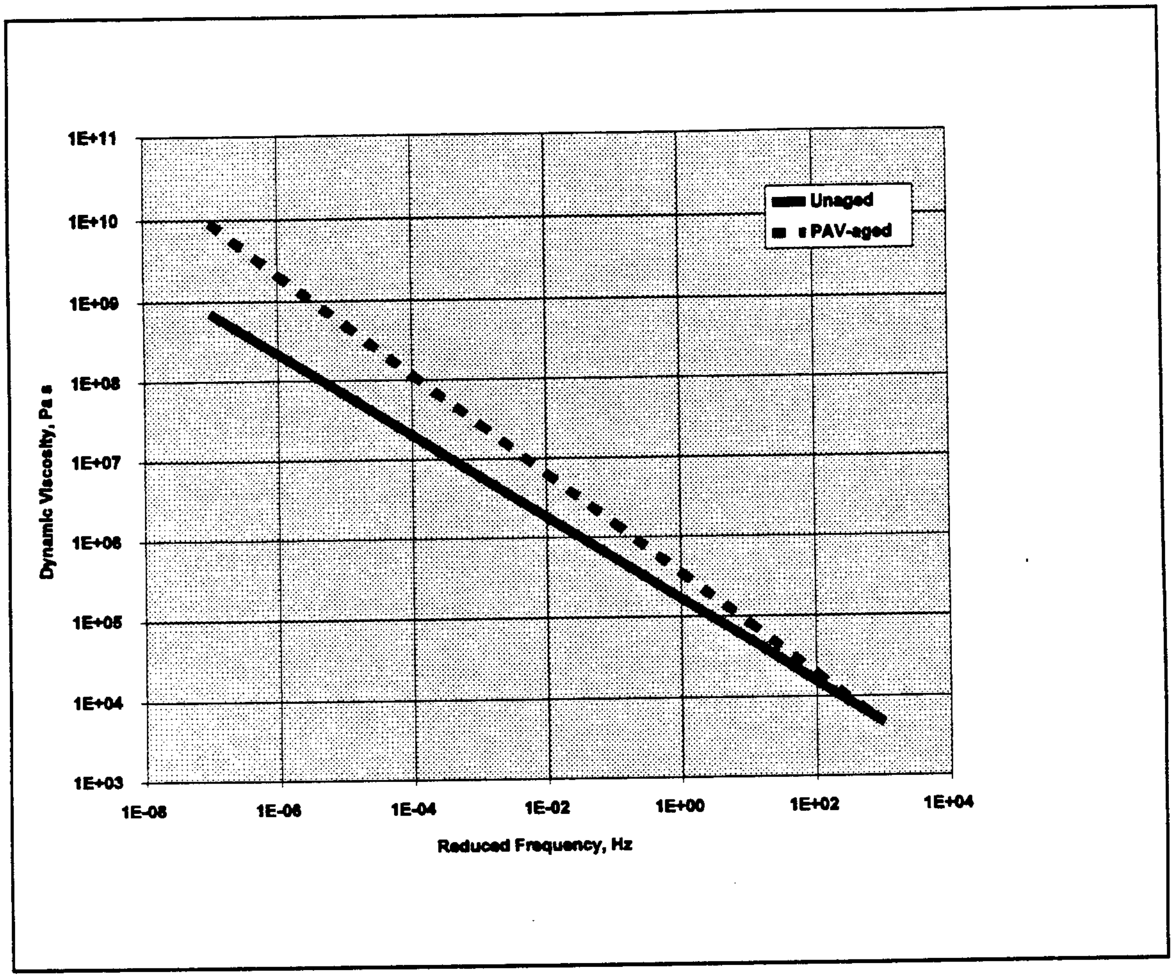

Figure 3.1 RAP Asphalt Dynamic Viscosity vs. Reduced Frequency Reference Temperature 28⁰ C

Figure 3.2 is a master curve of G*sin(δ) at a reference temperature of 28⁰ C, the binder test temperature for intermediate temperature testing for a binder performance graded at PG64-16. As with the dynamic viscosity, the reclaimed asphalt cement was tested without any additional laboratory aging and then again after PAV-aging to determine if the asphalt cement would be subject to additional stiffening and increased susceptibility to fatigue if placed in service. Any RAP asphalt that had not been softened by the addition of recycling agent would be subject to the aging measured by the G*sin(δ) of the PAV-aged specimen.

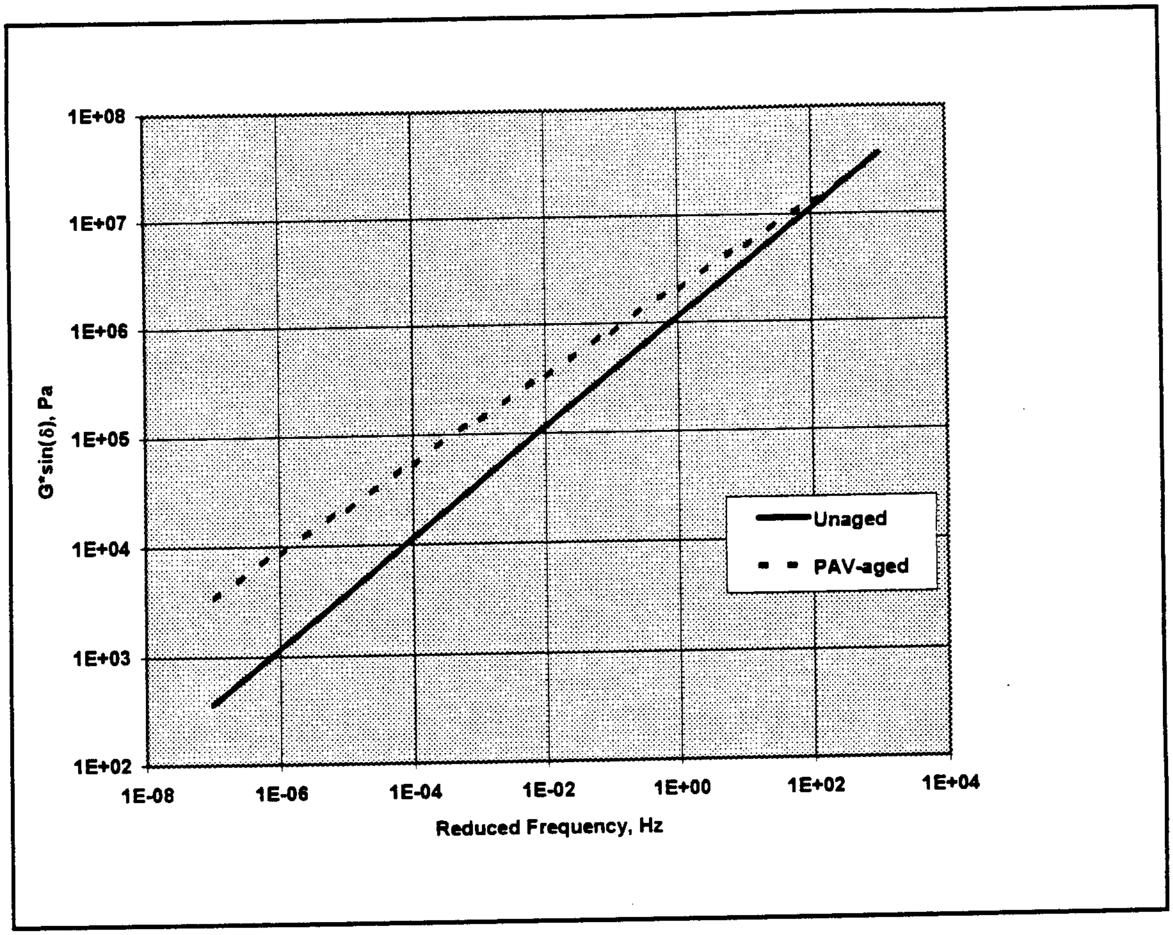

Figure 3.2 RAP Asphalt G*sin(δ) vs. Reduced Frequency
Reference Temperature 28^0 C

As indicated by Figure 3.2, any RAP asphalt which is not softened during the recycling process, is subject to further stiffening. This result was supported by the stiffening of the asphalt cement measured by the dynamic viscosity of the PAV-aged reclaimed asphalt shown in Figure 3.1. The RAP asphalt cement was susceptible to further hardening from oxidation.

Figure 3.3 shows the intermediate temperature test results for an asphalt cement which is graded at PG64-16. The solid curve represents G*sin(δ) versus temperature in degrees Celsius. The broken line represents the maximum G*sin(δ) allowable, 5000 kPa, for PAV-aged samples. The G*sin(δ) of the PAV-aged RAP asphalt cement was less than 5000 kPa at 22^0 C and higher temperatures. For a PG64-16 asphalt cement, the fatigue requirement is maximum G*sin(δ) of 5000 kPa at the intermediate test temperature of 28^0 C. Based on DSR analysis, the unsoftened and PAV-aged RAP asphalt met the fatigue criteria for a PG64-16 binder.

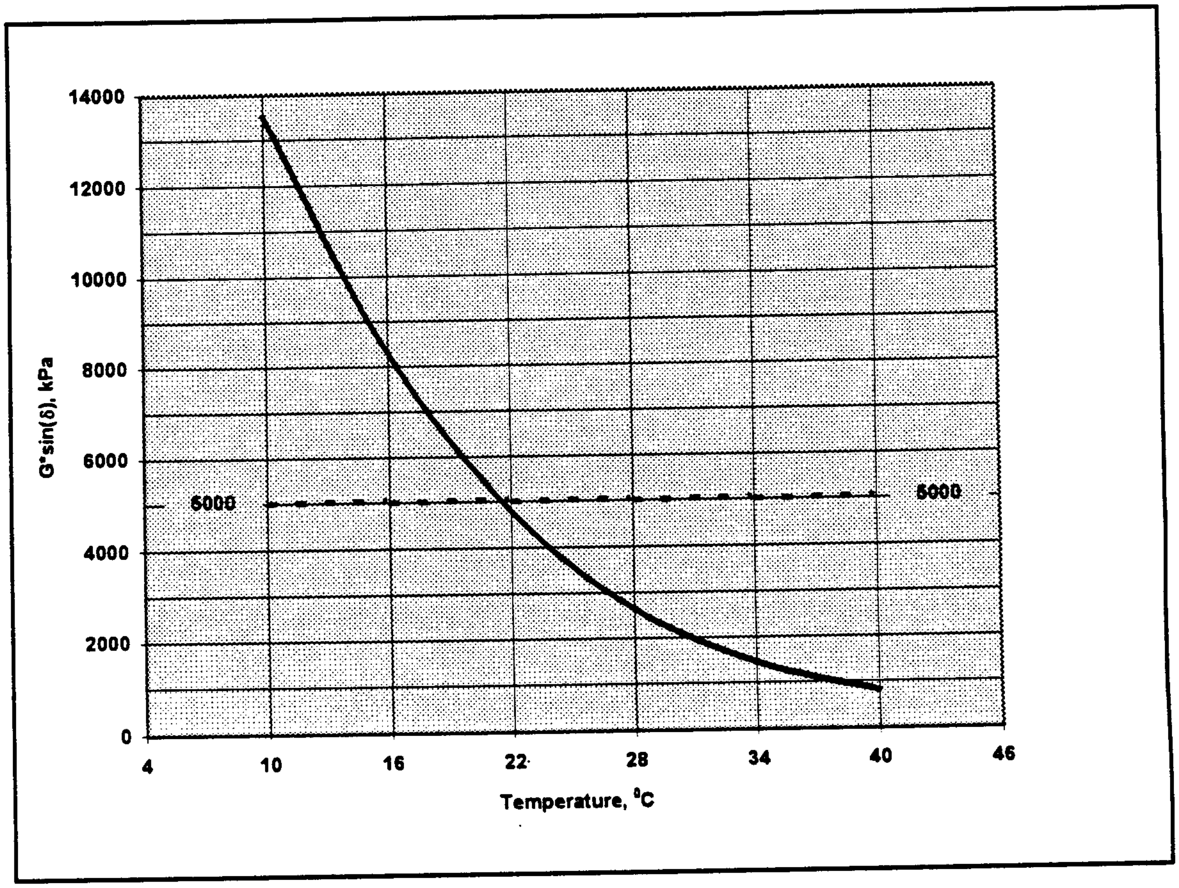

Figure 3.3 RAP Asphalt G*sin(δ) vs. Temperature
PAV-aged - Frequency 1.59 Hz

Permanent deformation is the critical failure mode in high temperature unaged asphalt concrete. G*/sin(δ) at the high reference temperature represents the high temperature rutting resistance of the unaged asphalt binder. Because the RAP asphalt had already been subjected to both short-term and long-term aging during its previous pavement life, permanent deformation was not expected to be a major concern for the RAP asphalt.

G*/sin(δ) of the reclaimed asphalt binder for the high temperature DSR tests are shown in Figure 3.4. The performance grading minimum for G*/sin(δ) is 1.0 kPa in the unaged condition. However, the RAP asphalt was aged and not subject to short-term aging from volatilization. The performance grade was determined from the RTFO-aged minimum of 2.2 kPa since the short-term aged condition is the condition at which asphalt cement is placed in service. As expected, the high temperature performance grade of the reclaimed asphalt binder exceeded the requirement for PG64-16.

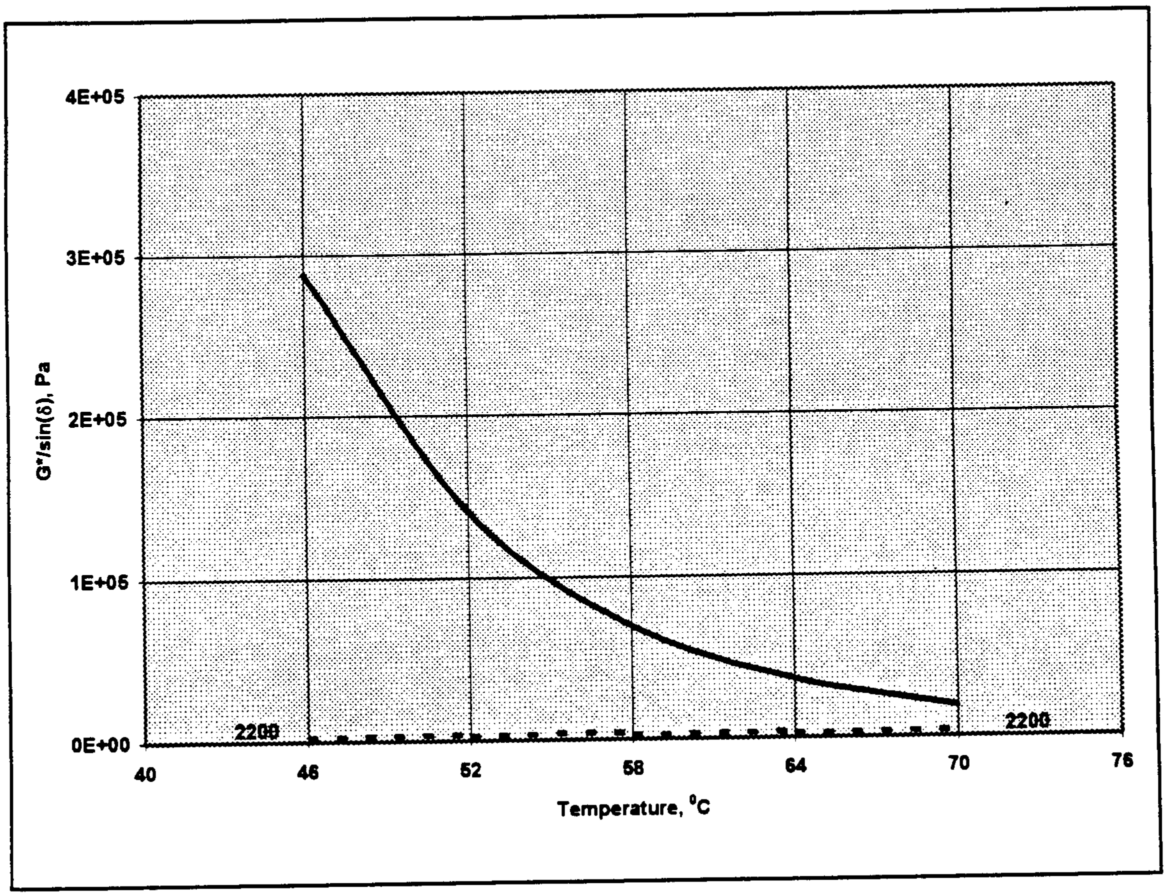

Figure 3.4 RAP Asphalt G*/sin(δ) vs. Temperature
Unaged - Frequency 1.59 Hz

Figure 3.4 shows that the reclaimed asphalt binder was determined to be performance graded at least PG70-y, the highest temperature at which the binder was tested.

3.2 Virgin Aggregate Gradation

The percentage of RAP allowable in cold recycled pavements has traditionally been limited by agency policy. An objective of this project was to determine the maximum allowable RAP in the cold recycled binder mix based on material properties.

The amount of RAP allowable in a recycled mixture is either gradation limited, binder limited or both gradation and binder limited. Binder limitations are due to a highly aged condition of the RAP asphalt binder and the resulting rejuvenator requirements for softening. The RAP asphalt exceeded the rutting and fatigue requirements for a PG64-16 asphalt. Therefore, the cold-recycled mix was not binder limited and was therefore gradation limited. The amount of RAP used in the recycled mix was based on the gradation requirements for pavement binder layers in North Carolina. Based on the gradation requirements for the binder layer in North Carolina, a #6 aggregate stockpile was chosen to provide the gradation necessary for proper aggregate blending to meet those specifications. The aggregate used in this project was crushed granite obtained from an aggregate supplier in the Raleigh, NC area.

Table 3.2 lists the aggregate gradations of the RAP, virgin aggregate and the aggregate gradation specifications for binder layers in North Carolina. The maximum amount of RAP in the cold recycled mix based on gradation was 75%. The cold recycled mixes used in this project were therefore composed of 75% RAP aggregate and 25% virgin aggregate.

The specific gravities and absorption of the #6 virgin aggregate were determined in accordance with *ASTM C127 (Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate)*. The bulk specific gravity (G_{agg}) was 2.61; the saturated surface dry specific gravity (G_{SSD}) was 2.63; the apparent specific gravity (G_{APP}) was 2.66; and the percent absorption (Abs) of the virgin aggregate was 0.65%.

Table 3.2 Virgin Aggregate Gradation Requirements

Sieve Size	Percent Passing (%)			
	RAP	#6 Virgin Aggregate	Blended 75% RAP & 25% Virgin Aggregate	North Carolina Binder Layer Specification
1 "	100	100	100	100
3/4 "	99	95	98	90-100
1/2 "	96	38	82	67-88
3/8 "	90	9	70	-
# 4	74	3	56	-
# 8	58	0	43	25-45
# 16	44	0	33	-
# 30	33	0	25	-
# 50	22	0	17	-
# 100	14	0	11	-
# 200	8.5	0	6.0	1-7

3.3 Asphalt Emulsion / Recycling Agent

The recycling agents used in this project were a cationic medium setting (CMS-2) emulsion and a high-float recycling agent (HFRA). Table 3.3 provides basic information on the recycling agents obtained from the supplier.

Table 3.3 Standard Properties of Recycling Agents

Property	CMS-2	HFRA
Oil (%)	8.0	2.0
Asphalt Content (%)	65.25	66.75
Penetration @ 25^0 C	210	550-600
Float	-----	1200+

The recycling agent residues were obtained using the evaporation method of *ASTM D244 (Standard Test Methods for Emulsified Asphalts)*. As with the RAP asphalt cement, the recycling agent residues were tested using the DSR in order to determine the rheological properties and to attempt to performance grade the residues.

There are no SUPERPAVE™ performance grading criteria for emulsion residues. One consideration made in the performance grading of a recycling agent was that the emulsion residues are not subject to short-term aging from the high temperatures associated with construction of hot-mix asphalt pavements. The unaged residue is the condition at which a cold-mix pavement is placed in service. In order to properly performance grade the recycling agent residues, the minimum permanent deformation criterion of $G^*/\sin(\delta)$ greater than 2.2 kPa was used. This minimum is used by SUPERPAVE™ to performance grade the initial in-service short-term aged condition of asphalt cement.

3.3.1 Unaged Condition

The dynamic viscosities of the unaged recycling agent residues are plotted in Figure 3.5 The master curve was constructed using the time-temperature superposition principle of thermorheologically simple materials. The reference temperature was 64^0 C. The graph shows that the CMS-2 residue was stiffer and less temperature susceptible than the HFRA.

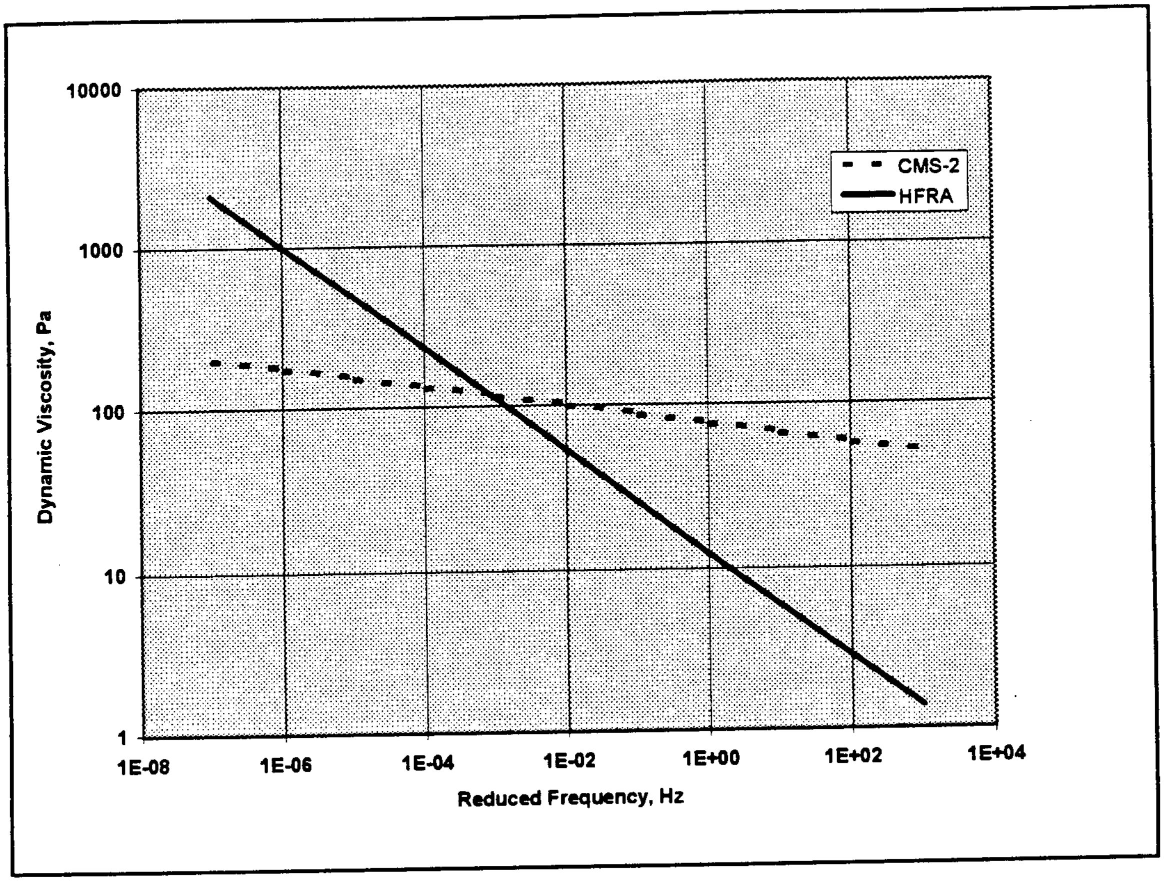

Figure 3.5 Unaged Recycling Agent Residues Dynamic Viscosity vs. Reduced Frequency Reference Temperature 64^0 C

The major problem with soft emulsion mixes is the propensity for permanent deformation, especially in the early stages of curing. G*/sin(δ) measures the rutting potential of an asphalt binder. The G*/sin(δ) curves of the unaged residues at temperatures ranging from 46^0 C to 70^0 C in 6^0 C increments are shown in Figure 3.6. The high temperature performance grade of asphalt binders in SUPERPAVE™ is the test temperature at which G*/sin(δ) is greater than or equal to 1.0 kPa. However, the residues were graded using the minimum in-service condition of 2.2 kPa. The CMS-2 residue met the criteria for a PG52-y asphalt cement. The HFRA residue could not be performance graded.

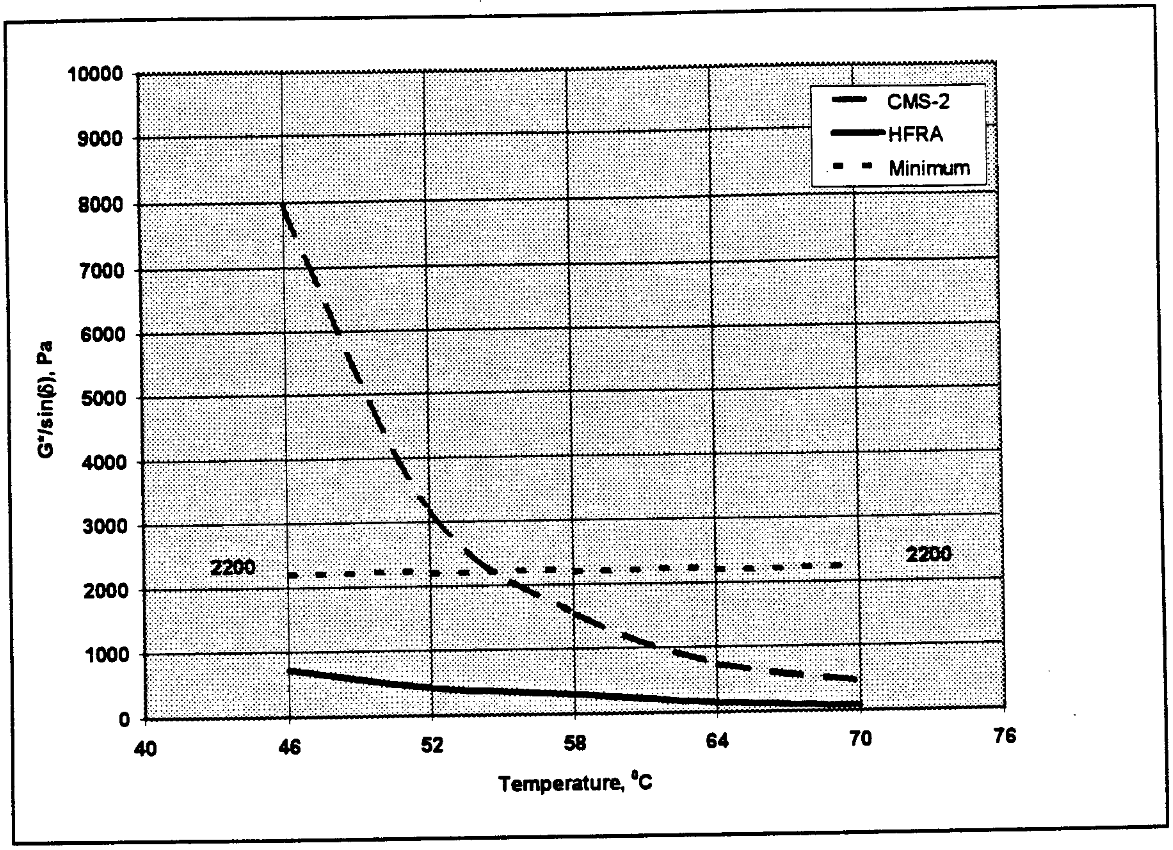

Figure 3.6 Unaged Recycling Agent Residues
G*/sin(δ) vs. Temperature
Frequency 1.59 Hz

3.3.2 PAV-aged Condition

Long-term aging of the residues was simulated in accordance with *AASHTO PP1 (Accelerated Aging of Asphalt Binder Using Pressure Aging Vessel).* The PAV-aged specimens were tested with the DSR at temperatures ranging from 40^0 C to 10^0 C in 6^0 C decrements. G*sin(δ) is used to measure the fatigue potential of the aged asphalt binder in SUPERPAVE™. Figure 3.7 shows the G*sin(δ) of the PAV-aged residues in kPa at the six test temperatures.

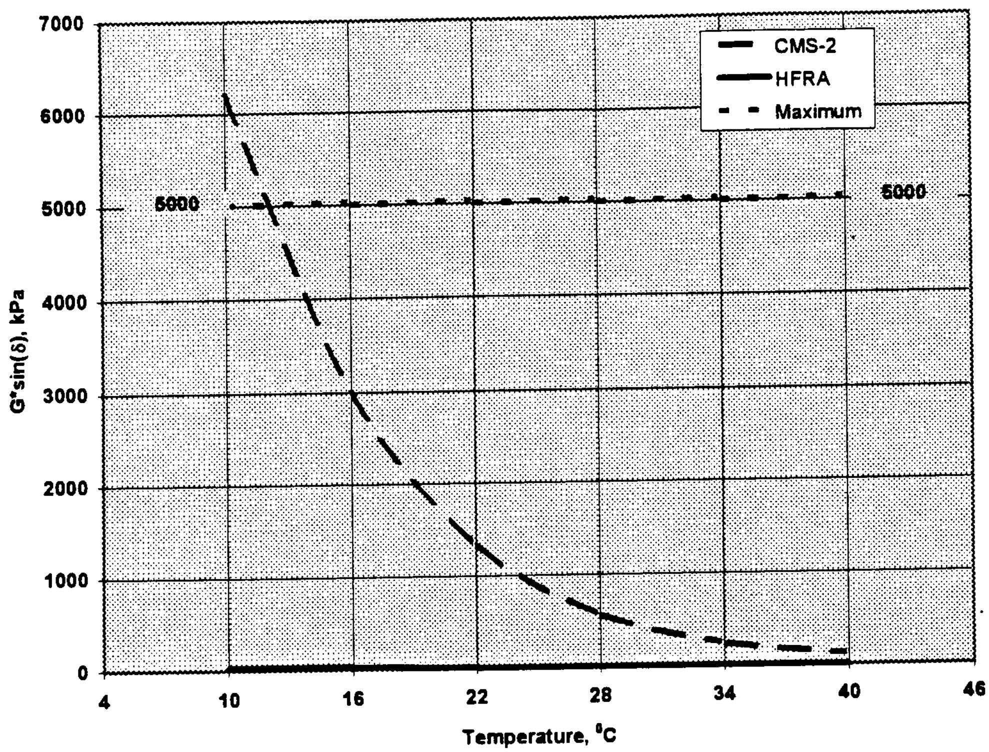

Figure 3.7 **PAV-aged Recycling Agent Residues**
G*sin(δ) vs. Temperature
Frequency 1.59 Hz

The residues were not short-term aged prior to PAV aging. Short-term aging of asphalt cement is the process of volatilization resulting from the high temperatures used in the mixing and construction of hot-mix asphalt pavements. Since cold recycling does not utilize high mixing and construction temperatures, laboratory simulated short-term aging in the rolling thin-film oven would be contrary to in-situ aging.

The SUPERPAVE™ criteria requires that PAV-aged asphalt binders have a maximum G*sin(δ) value of 5000 kPa. At all test temperatures, the G*sin(δ) of the HFRA aged residue was well below the 5000 kPa maximum. The G*sin(δ) of the CMS-2 aged residue was below the 5000 kPa maximum at all test temperatures except 10^0 C. As a result, the CMS-2 met the rutting and fatigue criteria for a PG52-28 binder.

CHAPTER 4

RECYCLED MIXTURE CHARACTERIZATION

4.1 Mix Design Methodology

Once the materials to be used in the cold-recycled mix were characterized, the next step was to design a mixture with each recycling agent that would provide optimum performance. The components of the recycled mixtures were aggregate gradation, asphalt binder blending, and optimum asphalt content. Aggregate gradation and binder blending yielded the maximum allowable percentage of RAP which could be used in the cold recycled mixtures. Since the PAV-aged condition of the RAP asphalt cement indicated that fatigue failure of the binder was not the critical mode and its stiffness in the reclaimed condition indicated a high resistance to permanent deformation, it was deduced that optimum asphalt blending would utilize as high a percentage of RAP asphalt that the gradation requirements would allow. Therefore, aggregate gradation requirements dictated the percentage RAP used in the cold recycled mixtures.

Once the aggregate specifications were determined, estimates of the recycling agent requirements were made. The recycling agent requirements were dependent upon three variables. The first variable considered was adequate binder content to coat and bind the virgin aggregate to the RAP. The second variable was the asphalt content of the RAP. The third variable considered was based upon the rheological properties of the RAP asphalt and the resulting rejuvenator requirements.

After the recycling agent requirements were estimated, test mixtures were prepared, cured and tested. Resilient modulus and indirect tensile tests were used to determine the optimum asphalt contents of the mixtures and to select the most favorable recycling agent for the RAP being recycled. The SUPERPAVE™ mix tests were used to support resilient modulus and indirect tensile test results.

4.2 Aggregate Blending

The composition of the recycled mixture was determined to be aggregate limited. The amount of RAP in the mixture was limited by the binder layer aggregate specifications of the North Carolina Department of Transportation (NCDOT). Table 4.1 shows the aggregate gradations for the RAP, virgin aggregate and the blended aggregate along with the NCDOT binder layer aggregate gradation requirements. The aggregate blend of 75% RAP and 25% virgin aggregate met the NCDOT binder layer specification.

Table 4.1 Aggregate Blending of Recycled Mix

Sieve Size	Percent Passing (%)			
	RAP	#6 Virgin Aggregate	Blended 75% RAP & 25% Virgin Aggregate	NCDOT Binder Layer Specification
1 "	100	100	100	100
3/4 "	99	95	98	90 - 100
1/2 "	96	38	82	67 - 88
3/8 "	90	9	70	-
# 4	74	3	56	-
# 8	58	0	43	25 - 45
#16	44	0	33	-
# 30	33	0	25	-
# 50	22	0	17	-
# 100	14	0	11	-
# 200	8.5	0	6.0	1 - 7

4.3 Determination of Trial Asphalt Content

The trial asphalt content was determined using the aggregate surface area method and by considering the upper and lower boundaries of percent recycling agent in the total asphalt content. Material characterization of the asphalt binders from the RAP and recycling agents indicated that while fatigue failure was not a major concern. However, permanent deformation of the mixture was of major concern due to the soft nature of the recycling agent residues. Therefore, the amount of recycling agent in the total asphalt content was determined to be a critical consideration and should be limited to minimal amounts.

Borrowing from the general concept of the cold-mix recycling blending charts in use by the California Department of Transportation [2], a blending chart was constructed for the RAP asphalt and recycling agents. Knowing that the blending chart is not completely reliable in cold-mix recycling due to the time dependency of the softening process, the blending chart provided an estimated range of acceptable percentages of recycling agent in the total recycled mix asphalt content.

4.3.1 Asphalt Institute Aggregate Surface Area Method

The Asphalt Institute recommends an aggregate surface area determination of initial asphalt content for cold mixes and cold recycled mixes [2]. The surface area empirical formula for determining percent asphalt demand of the combined aggregates is as follows [4].

$$Pc = \frac{0.035a + 0.045b + Kc + F}{R} \tag{4.1}$$

where: Pc = Percent of asphalt material by weight of total mix
K = 0.18 for 6% to 10% passing 75 μm (No. 200) sieve
a = percent of mineral aggregate retained on 2.36 mm (No. 8) sieve
b = percent of mineral aggregate passing 2.36 mm (No. 8) sieve and retained on 75 μm (No. 200) sieve
c = percent of mineral aggregate passing 75 μm (No. 200) sieve
F = 0 to 2.0% based on absorption of light or heavy aggregate. The formula is based on an average specific gravity of 2.60 to 2.70. In the absence of other data 0.7 to 1.0 should cover most conditions.
R = 0.60 to 0.65 for asphalt emulsions

The amount of recycling agent necessary for the mix is the difference between the total asphalt demand and the asphalt content of the RAP. The recycling agent requirement as determined by the Asphalt Institute method is as follows:

$$Pr = Pc - \frac{Pa * Pp}{R} \tag{4.2}$$

where: Pr = percent new asphalt in the recycled mix
Pc = percent of asphalt by weight of total mix
Pa = percent of asphalt in the RAP
Pp = decimal percent RAP in the recycled mix
R = 0.60 to 0.65 for asphalt emulsions

Substituting the measured values into Equation 4.1 gives the calculation of percent asphalt demand of the combined aggregates, Pc:

$$Pc = \frac{0.035(57) + 0.045(37) + (0.18)(6) + 0.7}{0.65} = 8.37\%$$

This is the emulsion equivalent demand by weight of mix not the amount of asphalt cement by weight of mix.

The percent of new asphalt in the mix, Pr, is determined by substituting the values for the variables into Equation 4.2:

$$Pr = 8.37 - \frac{(4.5) * (0.75)}{0.65} = 3.18\%$$

The amount of recycling agent emulsion required by weight of mix was 3.18%.

The Chevron Method [2] indicates that a minimum of 2% emulsified recycling agent is recommended for use. Comparatively, the Oregon Method [2] indicated that experience had indicated that the final estimated design emulsion content for CMS-2 can be as high as 2.6% and HFE-150 can be as high as 1.8% The Pennsylvania Method [2] indicated that the desirable amount of emulsion added should exceed 2 to 3% for adequate adhesion and cohesion. Therefore, the aggregate surface area method used was considered to be a good estimation of the initial emulsion content for the mix.

4.3.2 Development of Binder Blending Charts

The California Method of cold-recycled mix design incorporates a blending chart in which the log-viscosity of the RTFO-aged binder is used to determine the trial asphalt content. The desired initial percent recycling agent in the total asphalt content is that which is determined from an aggregate surface area method of asphalt demand. The recycling agent to be used is the one which provides a blended viscosity estimated to be closest to an AR4000 [2].

With the availability of DSR testing, the method utilized at NCSU incorporated a blending chart which determined an acceptable blending range within the boundaries of $G^*/\sin(\delta)$ for the unaged conditions as the upper limit and $G^*\sin(\delta)$ of the aged conditions as the lower limit. This range was used to determine whether the percent of recycling agent in the recycled mixture as determined by the surface area method of approximation was reasonable or unreasonable with regards to the rheological properties of the RAP asphalt and the recycling agent residues.

4.3.2.1 RAP and CMS-2 Blending Charts

The RAP and CMS-2 blending chart was constructed by overlaying the unaged properties with the aged properties of the binders. The first step in developing the blending chart was to construct the unaged portion. This gave the theoretical maximum amount of recycling agent residue in the total asphalt content. Once this was done, the aged portion of the blending chart was constructed. This relationship between the PAV-aged RAP asphalt and the PAV-aged recycling agent residue indicated a theoretical minimum amount of recycling agent residue in the total asphalt content.

4.3.2.1.1 G*/sin(δ)

The unaged condition of the RAP asphalt is the reclaimed condition which exists after extraction. The measure of rutting potential in the unaged state is G*/sin(δ) at the design temperature and a load rate of 1.59 Hz (10 rad/sec). For North Carolina, the performance grade of asphalt which provides 98% reliability in design is PG64-16 [8]. The acceptable rutting and fatigue characteristics of a PG64-16 blend were used as guidelines in establishing the target binder blend.

As determined from DSR testing of the unaged RAP asphalt, G*/sin(δ) of the unaged RAP asphalt was 36.3 kPa. The value of the RAP G*/sin(δ) was compared with the G*/sin(δ) of the CMS-2 residue to determine the maximum recycling agent content of the blended asphalt cement. This method assumed a fully blended condition even though, in reality, softening is a time-dependent process. From DSR testing of the unaged CMS-2 residue, G*/sin(δ) of the unaged CMS-2 residue at 64^0 C and 1.59 Hz was 0.73 kPa.

Figure 4.1 shows the plot of log G*/sin(δ) versus percent recycling agent. The left extreme of the graph represents 100% RAP asphalt and 0% recycling agent. It is the value of G*/sin(δ) for the unaged RAP. The right extreme of the graph represents 0% RAP asphalt and 100% recycling agent. This is the value of G*/sin(δ) for the CMS-2 unaged residue. A line connects the two points to predict G*/sin(δ) of the blended unaged asphalt at various recycling agent residue contents by weight of total asphalt in the mix. SUPERPAVE™ requires a minimum G*/sin(δ) of 1.0 Pa in the unaged condition at the appropriate PG grade. However, since short-term aging simulated by RTFO aging in the laboratory is not applicable to cold recycling and to provide a margin of safety, the blended asphalt binders were required to meet the minimum G*/sin(δ) of 2.2 kPa.

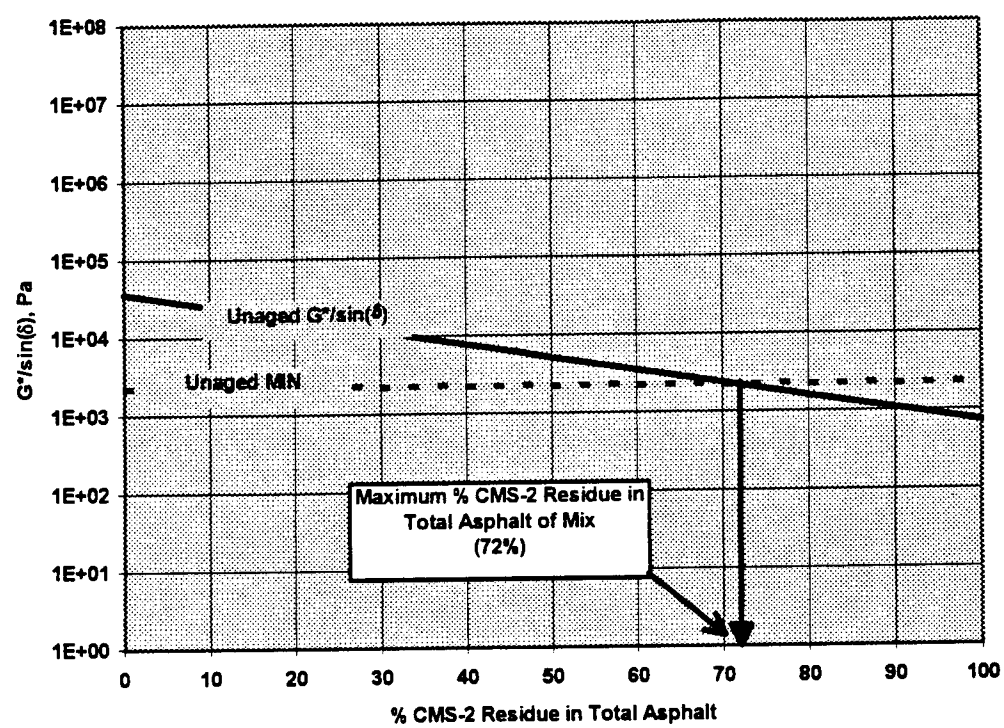

Figure 4.1 Blending Chart for RAP and CMS-2 Recycling Agent Unaged Condition - G*/sin(δ) at 64° C

In Figure 4.1, the maximum percentage of CMS-2 residue in the total asphalt of a recycled mix is the intersection of the G*/sin(δ) blend line with 2.2 kPa. The lines intersect at approximately 72% recycling agent residue and 28% RAP asphalt. The theoretical maximum amount of recycling agent residue allowable in the total asphalt content of the recycled mix is 72%. Any blend which is to the left of the point of intersection on the graph is a theoretically acceptable binder blend.

This chart assumes a fully blended condition. Since blending in cold recycling is time-dependent and difficult to predict, the optimum blend should contain something less than 72% recycling agent residue and a higher percentage of RAP asphalt. The reasoning was that any unblended recycling agent residue would be highly susceptible to permanent deformation because of its inherently soft character.

4.3.2.1.2 G* sin(δ)

The PAV-aged RAP asphalt and the PAV-aged CMS-2 residue were tested with the DSR to predict the fatigue resistance of the binders after long-term aging. The SUPERPAVE™ criterion for fatigue resistance of a PG64-16 binder is G*sin(δ) at 28^0 C. G*sin(δ) of the PAV-aged RAP asphalt was 2,653 kPa. G*sin(δ) of the PAV-aged CMS-2 residue was 55.5 kPa. The maximum allowable value of G*sin(δ) at the test temperature of 28^0 C is 5,000 kPa.

Figure 4.2 shows the plot of log G*sin(δ) versus percent recycling agent. The left extreme of the graph represents 100% RAP asphalt and 0% recycling agent. It is the value of G*sin(δ) for the PAV-aged RAP. The right extreme of the graph represents 0% RAP asphalt and 100% recycling agent. It is the value of G*sin(δ) for the CMS-2 PAV-aged residue. A line connects the two points to predict G*sin(δ) of the blended asphalt at varying recycling agent residue contents by weight of total asphalt in the mix. The minimum percentage of CMS-2 residue in the total asphalt of a recycled mix is the intersection of the blend line with 5000 kPa. The lines do not intersect. The theoretical lower limit of recycling agent residue allowable in the total asphalt content of the recycled mix is 0% and any blend which is to the right of this value is a theoretically acceptable blend for fatigue resistance. Fatigue failure was not considered the critical failure mode for the asphalt binders.

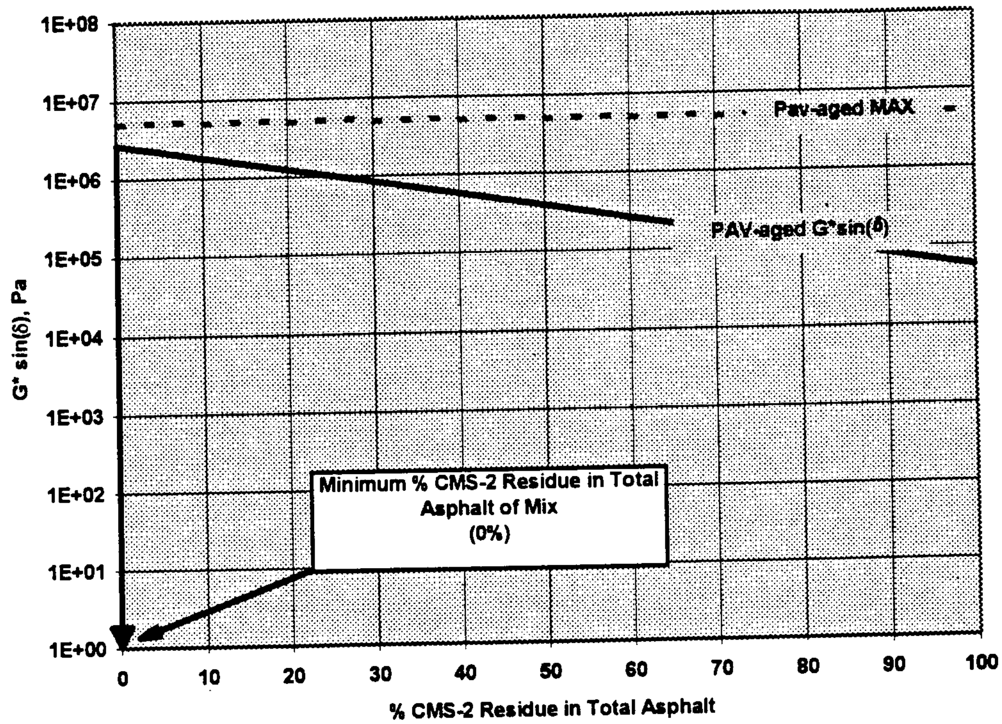

Figure 4.2 **Blending Chart for RAP and CMS-2 Recycling Agent PAV-aged Condition - G*sin(δ) at 28° C**

4.3.2.1.3 Determination of the Design Asphalt Blend Window

The RAP asphalt / CMS-2 residue blending window was developed by overlaying the unaged G*/sin(δ) blend chart (Figure 4.1) with the PAV-aged G* sin(δ) blend chart (Figure 4.2). This is shown in Figure 4.3. The window is the theoretically acceptable range of recycling agent residue in the binder blend. The upper limit of CMS-2 in the asphalt blend was 72% based on the intersection of the G*/sin(δ) blend line with the 2.2 kPa line in the unaged condition. The minimum percent of CMS-2 in the asphalt blend was the intersection of the G* sin(δ) blend line with 5000 kPa. Because the blend line never intersects the 5000 kPa line, the lower boundary of the window was 0% CMS-2 residue in the asphalt blend.

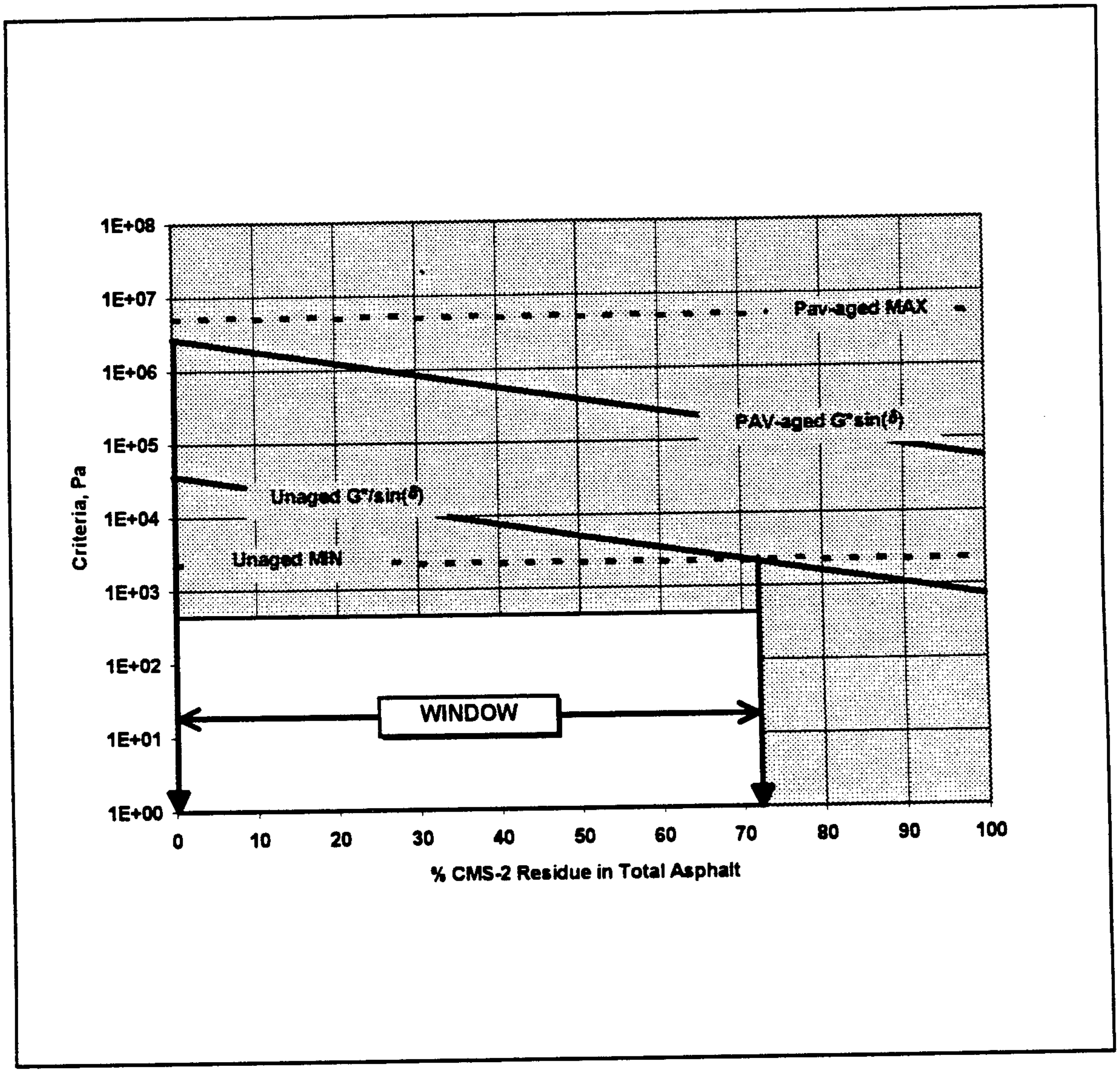

Figure 4.3 **Blending Chart for RAP and CMS-2 Recycling Agent G*/sin(δ) at 64° C and G*sin(δ) at 28° C**

From Figure 4.3, the range of CMS-2 residue contents of the recycled asphalt blend was determined to be 0% at the lower extreme and 72% at the upper extreme. Any gradation-based asphalt requirement that would fall within this window would be acceptable. Because fatigue was not the critical failure mode with the binders tested, the optimum CMS-2 content was considered to be a blend which was near the lower extreme as gradation and mixing allowed.

4.3.2.2 RAP and HFRA Blending Chart

The same procedure used to develop the RAP / CMS-2 blending chart was used to develop the RAP / HFRA blending chart. In the unaged condition, $G^*/\sin(\delta)$ of the RAP asphalt was 36.3 kPa and $G^*/\sin(\delta)$ for the HFRA residue was 0.15 kPa. In the PAV-aged condition, $G^*\sin(\delta)$ of the RAP asphalt was 2,653 kPa and $G^*\sin(\delta)$ of the HFRA residue was 2.7 kPa.

Figure 4.4 is the RAP / HFRA blending chart. The design window was lower bound by 0% HFRA in the blend and upper bound by 50% HFRA in the recycled asphalt blend. The HFRA residue was inherently soft and any unblended recycling agent creates a weak link in the rutting resistance of a mixture. Therefore, the target HFRA content of the asphalt blend was somewhat less than the 50% upper boundary of the design window.

Comparing the RAP / CMS-2 blending chart in Figure 4.3 and the RAP / HFRA blending chart in Figure 4.4, the differences in the unaged $G^*/\sin(\delta)$ values of the two recycling agent residues become obvious when considering the maximum allowable recycling agent residues in the binder blends. The softer nature of the HFRA recycling agent residue limits the amount of recycling agent which may be used in the mixture. The blending chart comparisons indicate that the optimum asphalt content of the HFRA-recycled mixtures will be lower than the optimum asphalt content of the CMS-2-recycled mixtures since the variations in the asphalt contents are due to changes in the recycling agent residue contents of the binder blends.

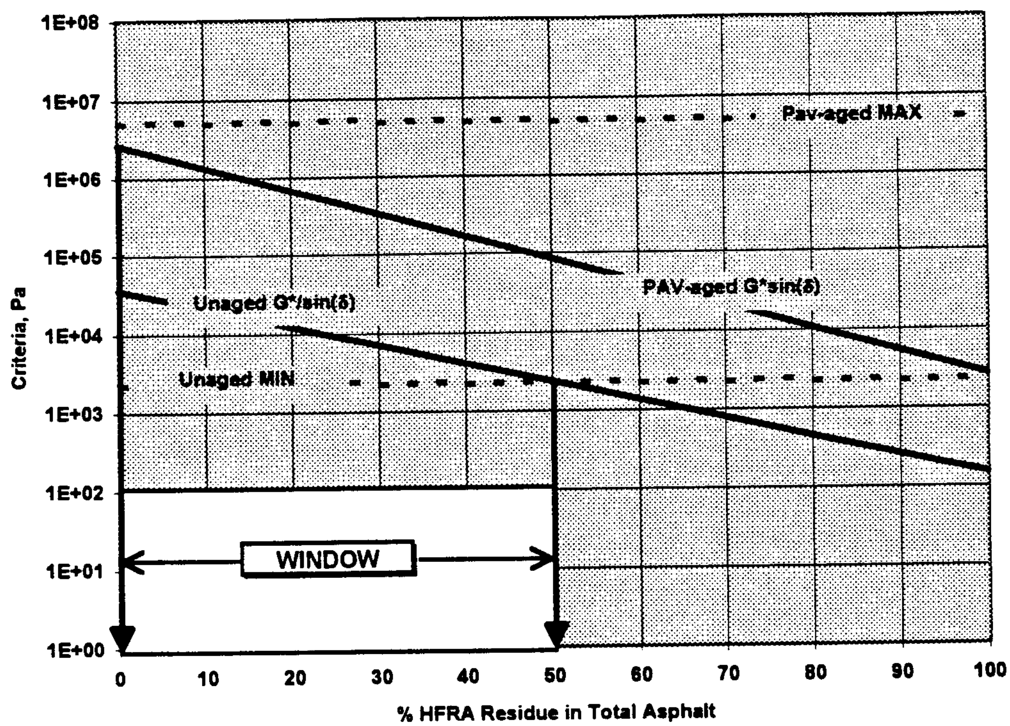

Figure 4.4 **Blending Chart for RAP and HFRA Recycling Agent $G^*/\sin(\delta)$ at 64^0 C and $G^*\sin(\delta)$ at 28^0 C**

4.4 Cold Recycled Mix Preparation

Based on Figures 4.3 and 4.4, the acceptable ranges of recycling agents were determined as indicated earlier. Mixtures were prepared using the determined blends, and then the specimens were cured and tested. The mix preparation protocol used in this study followed a procedure developed earlier at NCSU [7]. After several trial runs, the procedure was slightly modified to correct some problems encountered with sample preparation.

Preparation and curing of the specimens required approximately sixteen days before tests could be conducted. The mix design procedure was as follows:

1. The necessary portions of RAP and virgin aggregate were weighed in separate pans. Then the virgin aggregate was placed into a mixing bowl. Water was added to the virgin aggregate equal to 4% of the weight of the aggregate and hand mixed making certain all aggregate had been moistened. The water and aggregate were allowed to stand for 5 minutes.
2. The recycling agent was poured into the mixing bowl and hand stirred making certain that all of the virgin aggregate was well coated. Next, the RAP was poured into the mixing bowl and hand stirred for 30 seconds. After hand stirring, the materials were mixed in a mechanical mixer for 60 seconds, then hand stirred for 30 seconds to make certain that the recycling agent and smaller RAP particles were not sticking to the bottom and sides of the mixing bowl and were free to mix with the larger particles. Finally, the materials were mixed once more in the mechanical mixer for 30 seconds.
3. The mix was placed into a pan and heated in a forced draft oven at 60^0 C (140^0 F) for one hour. After one hour, the mix was removed from the oven and immediately placed into a compaction mold by spooning to avoid segregation of the particles.
4. The mixture was compacted in the gyratory compactor. After compaction, the specimens were allowed the to cool to room temperature in the mold overnight.
5. The specimens were extruded from the mold and allowed to cure at room temperature for 24 hours. The specimens were placed in a forced draft oven and allowed to cure at 60^0 C (140^0 F) for 72 hours. After the 72-hour period, the specimens were allowed to cool to room temperature overnight. After cooling, resilient modulus tests and indirect tensile tests were performed on one-half of the oven-cured specimens.
6. The remaining specimens were cured 4 hours-a-day for 10 days under an infra-red lamp adjusted to a height so that the surface temperatures of the specimens were 60^0

C (140^0 F). The specimens were turned each day to ensure even curing throughout. After the 10th curing day, the specimens were allowed to cool to room temperature overnight, then resilient modulus tests were performed on the infra-red lamp cured specimens.

The first samples mixed and tested utilized the CMS-2 emulsion recycling agent. The initial trial asphalt content was 5.1% as determined by the aggregate surface area formula. In addition to the initial trial asphalt content of 5.1%, samples were also prepared with asphalt contents of ±0.5% and +1.0% (i.e. 4.6%, 5.1%, 5.6% and 6.1%).

Considering its softer nature, adjustments were made to the trial asphalt contents of the HFRA recycled mixtures. The trial asphalt contents of the HFRA recycled mixes prepared were 3.9%, 4.4%, 4.9%, and 5.4%. Specimens with lower asphalt contents appeared very dry and crumbly upon extrusion from the molds. However, as the specimens cured they became "wetter" and less fragile.

4.5 Determination of Optimum Asphalt Content

The optimum asphalt content of the cold recycled mixes were determined using resilient modulus tests and indirect tensile tests on the oven-cured and resilient modulus tests on the infra-red cured samples.

4.5.1 Resilient Modulus

The resilient modulus tests were performed in accordance with *ASTM D4123 (Test Methods for Indirect Tensile Test for Resilient Modulus of Bituminous Mixtures)* using the MTS Model 810 testing apparatus. The tests were conducted in the diametral mode at 20^0 C. A 0.1-second loading and 0.9-second recovery cycle was used. The specimens were conditioned for 100 cycles prior to data collection. Each specimen was tested twice with one test on each perpendicular axis. The resilient moduli from the two axes were averaged to determine a single resilient modulus of each test specimen. Four specimen replicates were prepared and tested for each asphalt content treatment and for each curing level.

The resilient modulus tests were conducted on the oven-cured specimens and the specimens which had been oven-cured and then cured under the infra-red lamp. The oven-cured samples were the condition at which, in the field, the excess water would have been evaporated off and minimal curing of the recycled mix would have occurred.

The samples which were both oven-cured and infra-red cured were those in which the resilient modulus had reached its theoretical peak. Based on earlier studies at NCSU on cold-recycled mixes, it was determined that the infra-red curing procedure used in this project cured samples to the point that the resilient modulus was at or near its maximum value. Additional curing would not have significantly increased the resilient moduli of the cold-recycled mixes.

Figure 4.5 shows the average resilient moduli of the CMS-2 recycled mixes at the different asphalt contents under both curing conditions. Those specimens which were tested after the three-day oven curing (oven-cured) procedure had lower resilient moduli than those which were allowed to cure an additional 40 hours under the infra-red light (infra-red cured). The optimum asphalt content based on resilient modulus was determined to be 4.6%. This equated to an emulsion content in the mix of 1.9%. It was sightly lower than the minimum suggested by the Pennsylvania and Chevron Methods, but was within the acceptable range for the Oregon Method. The percent recycling agent residue which was contained in the 4.6% asphalt content was 27%. The recycling agent content was within the acceptable range as determined by the CMS-2 blending chart shown in Figure 4.3.

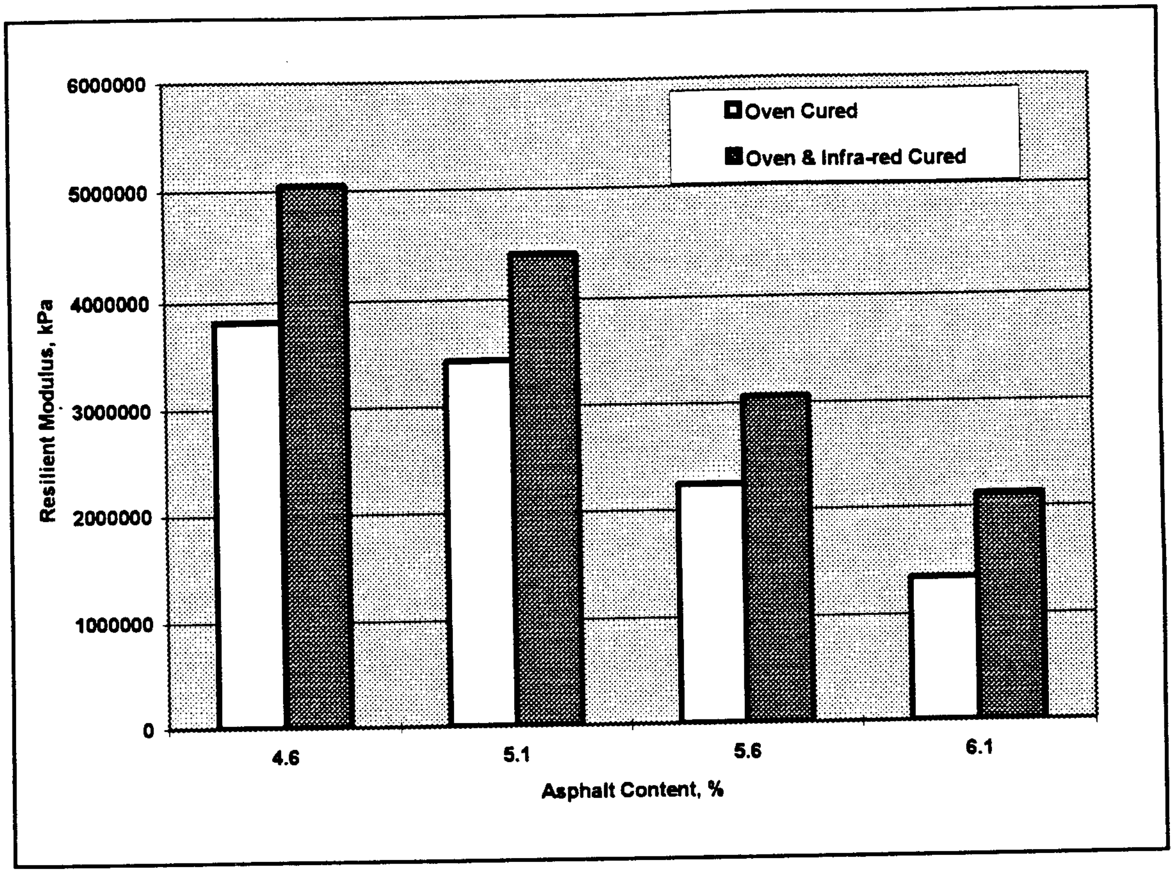

Figure 4.5 CMS-2 Recycled Mix
Resilient Modulus vs. Asphalt Content at 20^0 C

The results of resilient modulus tests on the HFRA recycled mixes are shown in Figure 4.6. The testing was done in the same manner as described for the CMS-2 recycled mixes. Slight adjustments were made in the asphalt contents of the test specimens because of the softer nature of the HFRA residue. The optimum asphalt content of the HFRA recycled mixes based on resilient modulus tests was 4.4% considering the 10-day cured condition. This equated to 1.6% emulsion in the mixture which was low based on the Chevron and Pennsylvania Methods but acceptable by the Oregon Method. The percent recycling agent residue which was contained in the 4.4% asphalt content was 23%. The HFRA residue content of the total binder was within the acceptable range as determined by the HFRA blending chart shown in Figure 4.4.

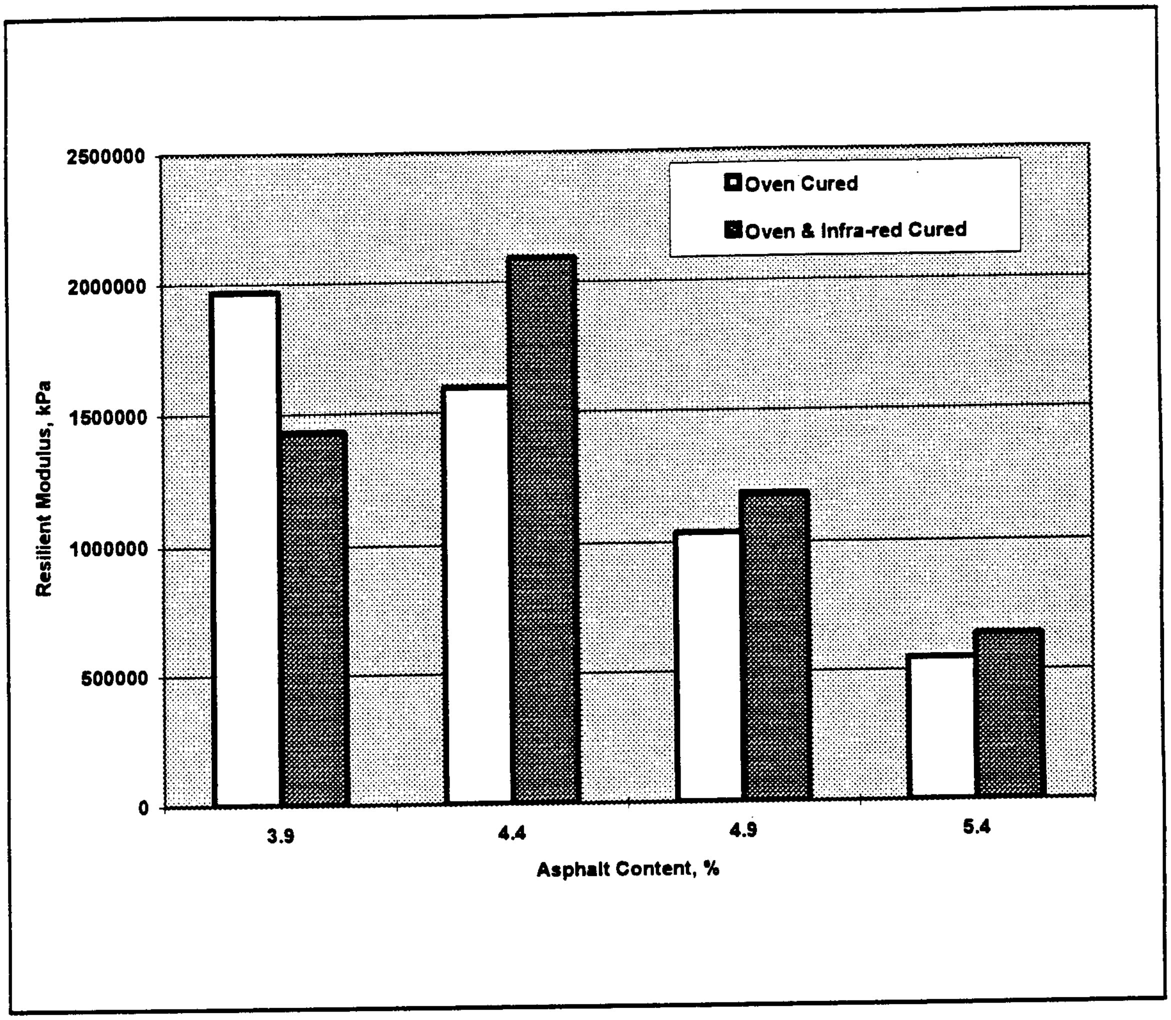

Figure 4.6 HFRA Recycled Mix
Resilient Modulus vs. Asphalt Content at 20^0 C

4.5.2 Indirect Tensile Strength

The indirect tensile tests were conducted on the oven-cured samples only. The results supported those obtained from the resilient modulus tests. Figure 4.7 contains the results of the resilient modulus tests shown in Figure 4.5 with a line graph of the results of the indirect tensile strengths vs. asphalt content for the CMS-2 samples. Figure 4.7 shows that the trend of the indirect tensile strength results follows the strength trend of the resilient modulus tests. The optimum performance was exhibited by the mix with 4.6% asphalt content.

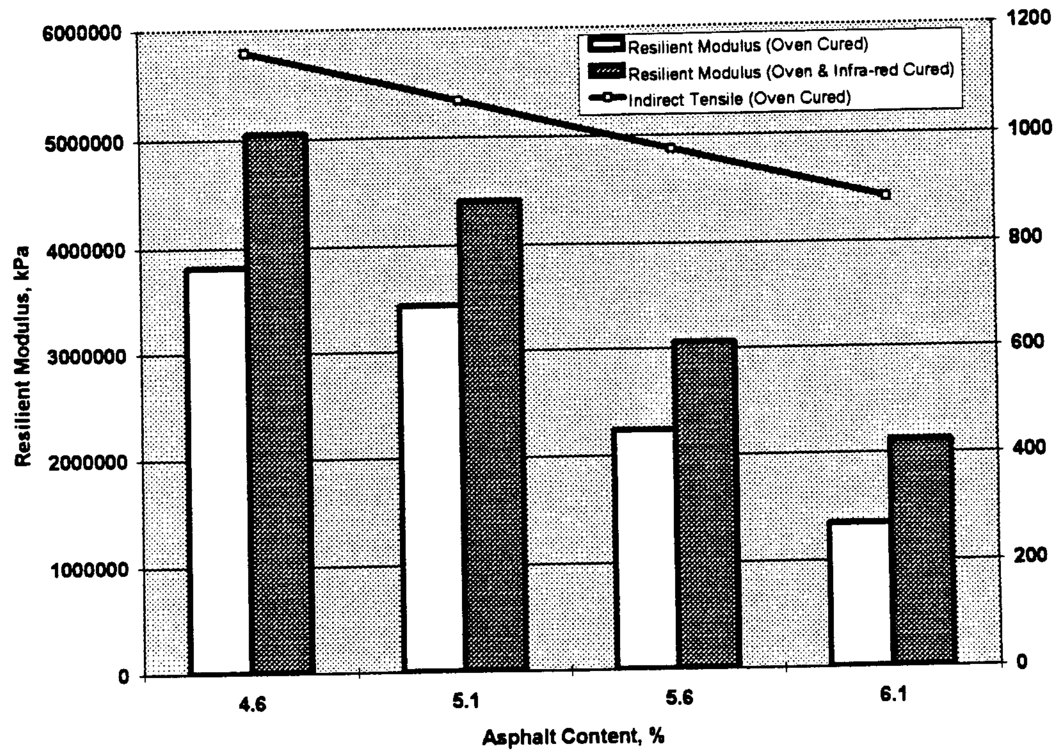

Figure 4.7 CMS-2 Recycled Mix
Indirect Tensile Strength and Resilient Modulus vs. Asphalt Content

The results of the HFRA indirect tensile tests are shown in Figure 4.8. The optimum asphalt content for the HFRA recycled mixes based on indirect tensile strength was 4.4%. As with the CMS-2 specimens, the indirect tensile strength trend mirrors the results of the resilient modulus tests.

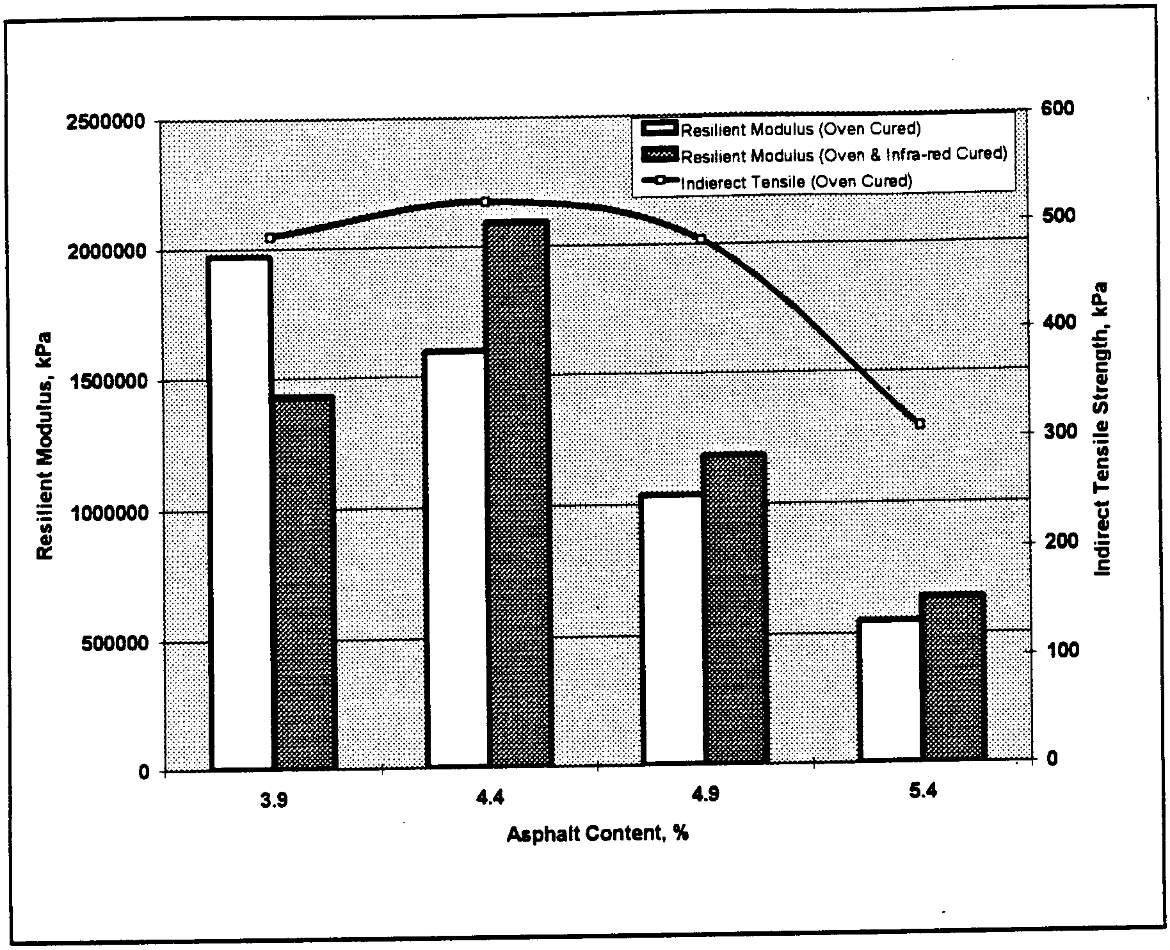

Figure 4.8 HFRA Recycled Mix
Indirect Tensile Strength and Resilient Modulus vs. Asphalt Content

Table 4.2 summarizes the results of the resilient modulus and indirect tensile tests for the CMS-2 and the HFRA recycled mixes at each asphalt content considered. Based on the initial testing, the CMS-2 recycled mixes performed best with 4.6% asphalt by weight of mix with 27% of the binder contributed by the CMS-2 residue. The optimum asphalt content of the HFRA recycled mixes was 4.4% by weight of mix with 23% of the total asphalt contributed by the HFRA residue. Comparing the two recycling agents, the CMS-2 recycling agent yielded a mix with a much higher resilient modulus and higher indirect tensile strength. These results indicate that the CMS-2 recycled mixes were considerably stiffer than the HFRA recycled mixes.

Table 4.2 Optimum Asphalt Contents of the CMS-2 and HFRA Recycled Mixes Based on Resilient Modulus and Indirect Tensile Tests.

Recycling Agent	Optimum Asphalt Content of Recycled Mix	% Recycling Agent Residue in Asphalt	Resilient Modulus (Oven Cured)	Resilient Modulus (Oven & Infra-red Cured)	Indirect Tensile Strength (Oven Cured)
CMS-2	4.6 %	27 %	3816 MPa (553 ksi)	5051 MPa (733 ksi)	1158 kPa (168 psi)
	5.1 %	34 %	3433 MPa (498 ksi)	4409 MPa (639 ksi)	1067 kPa (155 psi)
	5.6 %	40 %	2235 MPa (324 ksi)	3061 MPa (444 ksi)	975 kPa (141 psi)
	6.1 %	45 %	1342 MPa (195 ksi)	2121 MPa (308 ksi)	883 kPa (128 psi)
HFRA	3.9 %	13 %	1969 MPa (286 ksi)	1430 MPa (207 ksi)	491 kPa (71.2 psi)
	4.4 %	23 %	1595 MPa (231 ksi)	2093 MPa (304 ksi)	521 kPa (75.6 psi)
	4.9 %	31 %	1036 MPa (150 ksi)	1184 MPa (172 ksi)	484 kPa (70.2 psi)
	5.4 %	38 %	549 MPa (80 ksi)	639 MPa (93 ksi)	310 kPa (44.9 psi)

4.6 SUPERPAVE™ Mixture Tests

SUPERPAVE™ mix design consists of three levels of testing. Volumetric design controls material selection and volumetrics. Intermediate testing includes volumetric material selection along with mechanical testing which predicts pavement performance. Complete Mix Analysis includes Intermediate testing with additional mix characterization provides the highest level of performance testing and mix characterization. One problem with SUPERPAVE™ Intermediate and Complete test analyses is that the SUPERPAVE™ software used to predict rut depth and percent cracking were not available at the time this study was completed. Some indications of relative performance of the test mixtures were obtained from the mechanical tests even though the SUPERPAVE™ analysis models were unavailable.

4.6.1 Volumetric Properties of the Mixture

Problems associated with incorporation of volumetric mix design into a testing protocol for cold mix recycling were immediately obvious. First, material selection of a mix which includes 75% RAP is difficult. At best, standards can be established which would determine whether or not a particular RAP is desirable for a recycling project. In this project, the RAP was selected based on availability. Comparisons of RAP sources were not within the scope of this study. Therefore, it was assumed for design and analysis purposes, that the material selection was adequate.

The second problem with volumetric design was the variability of water contents at the point of compaction. Following material mixing, the sample was placed in a forced-draft oven at 60^0 C for one hour prior to compaction. The actual water content of the mixture at the point of compaction was found to vary and no method of predicting the water content was established.

The third problem encountered with volumetric mix design was that a table of gyrations was not appropriate for mix compaction as the recycled mixture was not cured, blending of recycling agent and RAP asphalt was incomplete, and the number of gyrations had no logical relationship to estimated field performance. Further study into the adaptation of a volumetric mix design for cold mix recycling may yield a reliable method but development of such a method was not within the scope of this project.

In SUPERPAVE™, the nominal maximum size is one size larger than the first sieve to retain more than 10% on the metric gradation scale. Maximum size aggregate is one sieve size larger than the nominal maximum size. For the aggregate blend in this project, the nominal maximum size was 19.0 mm (3/4"). The maximum size aggregate was 25.0 mm (1.0"). Table 4.3 shows the actual gradation and the SUPERPAVE™ requirements for 19.0 mm nominal size aggregate.

Table 4.3 Recycled Mixture Gradation and SUPERPAVE™ Requirements

Sieve Size (in)	Sieve Size (mm)	% Passing	Control Points		Restricted Zone Boundary	
			Minimum	Maximum	Minimum	Maximum
1"	25	100		100.0		
3/4"	19	98	90.0	100.0		
1/2"	12.5	82				
3/8"	9.5	70				
No. 4	4.75	56				
No. 8	2.36	43	23.0	49.0	34.6	34.6
No. 16	1.18	33			22.3	28.3
No. 30	0.600	25			16.7	20.7
No. 50	0.300	17			13.7	13.7
No. 200	0.075	6.0	2.0	8.0		

Figure 4.9 shows the 0.45 Power gradation curve and the SUPERPAVE™ gradation requirements for 19.0 mm nominal maximum size aggregate mixtures. The blended gradation meets the SUPERPAVE™ requirements for mixes with 19.0 mm nominal maximum size aggregate. The gradation falls between the control points and does not intersect the restricted zone. SUPERPAVE™ recommends that the gradation curve should fall below the restricted zone for best performance but requires only that the gradation does not intersect its boundaries. Therefore, the aggregate blend meets the SUPERPAVE requirements.

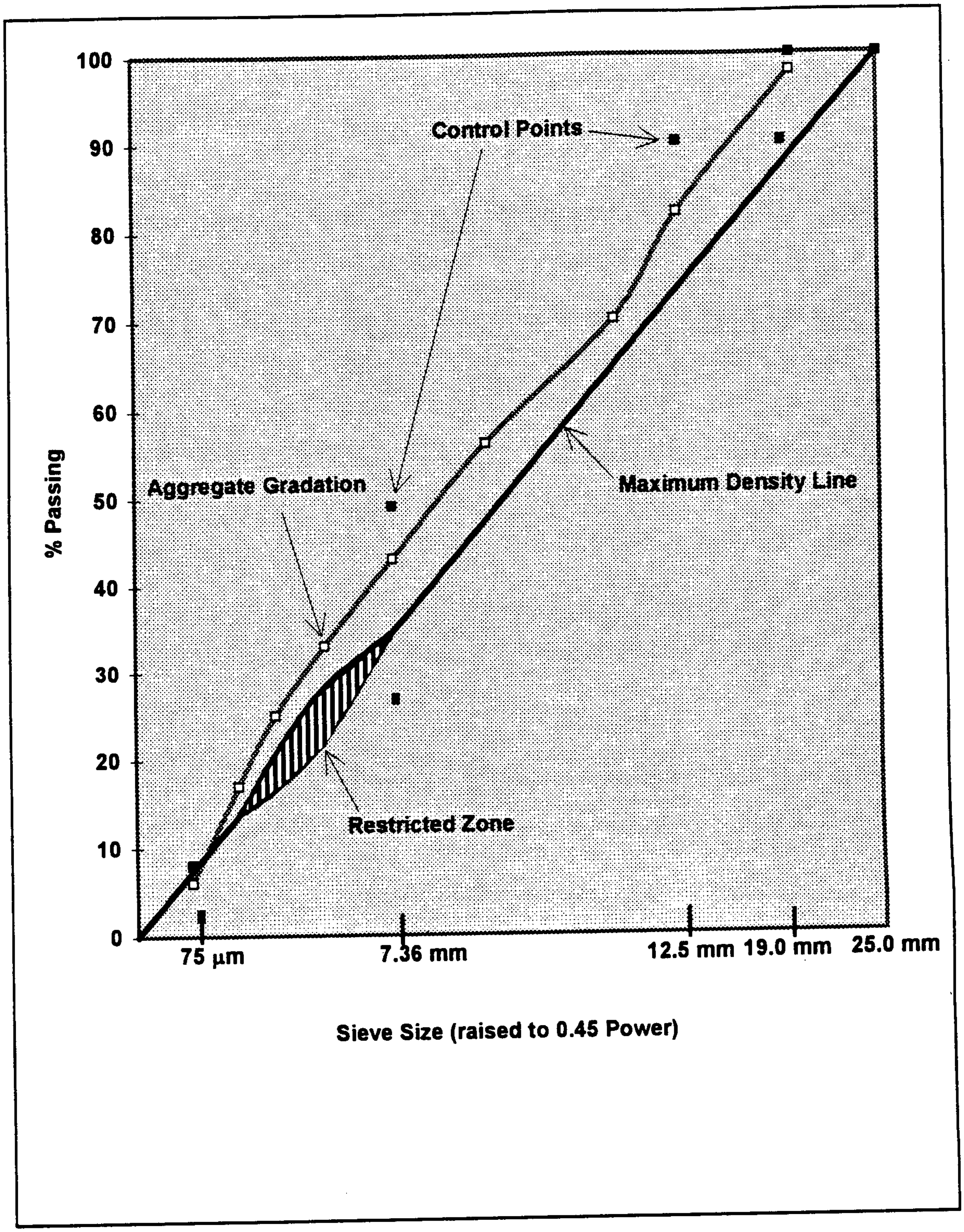

Figure 4.9 Gradation of Recycled Mix Aggregate Blend with 75% RAP 0.45 Power Curve - 19.0 mm Nominal Maximum Size

The volumetric properties of the recycled mixes were determined in accordance with the SUPERPAVE™ Volumetric specifications. The measured properties included specific gravities of the RAP (G_{rap}) and the virgin aggregate (G_{agg}), maximum theoretical specific gravities of the recycled mixtures (G_{mm}) and the bulk specific gravity of the compacted mixtures (G_{mb}) at 4% air voids. The specific gravity of the virgin aggregate was determined in accordance with *ASTM Test Method C127 (Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate)*. Maximum theoretical specific gravities of the RAP and mixtures were determined using *ASTM Test Method D2041 (Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures)*. The bulk specific gravity (G_{agg}) of the virgin aggregate was 2.61. The maximum specific gravity of the RAP (G_{rap}) was 2.41. The theoretical maximum specific gravities (G_{mm}) were determined on mixes which had cured in the forced draft oven for 72 hours at 60^0 C. Tables 4.4 and 4.5 contain the volumetric properties of the CMS-2 and HFRA recycled mixes at 4% air voids and 7% air voids, respectively.

Table 4.4 Volumetric Properties of the Recycled Mixes at 4% Air Voids

Recycling Agent	Asphalt Content	G_{sb}	G_{mm}	G_{mb}	G_{se}	P_{ba} (%)	P_{be} (%)	VMA (%)	Va (%)	VFA (%)
CMS-2	4.1	2.46	2.38	2.29	2.52	1.10	3.04	10.7	4.0	62.8
	4.6	2.46	2.37	2.28	2.53	1.22	3.43	11.6	4.0	65.5
	5.1	2.46	2.36	2.27	2.54	1.34	3.83	12.4	4.0	67.8
HFRA	3.6	2.46	2.40	2.31	2.53	1.26	2.38	9.3	4.0	57.2
	4.1	2.46	2.37	2.27	2.50	0.87	3.26	11.2	4.0	64.3
	4.6	2.46	2.32	2.23	2.47	0.30	4.31	13.3	4.0	70.0

Table 4.5 Volumetric Properties of the Recycled Mixes at 7% Air Voids

Recycling Agent	Asphalt Content	G_{sb}	G_{mm}	G_{mb}	G_{se}	P_{ba} (%)	P_{be} (%)	VMA (%)	Va (%)	VFA (%)
CMS-2	4.1	2.46	2.38	2.21	2.52	1.10	3.04	13.7	7.1	47.7
	4.6	2.46	2.37	2.20	2.53	1.22	3.43	14.5	7.2	50.6
	5.1	2.46	2.36	2.19	2.54	1.34	3.83	15.3	7.2	53.1
HFRA	3.6	2.46	2.40	2.24	2.53	1.26	2.38	12.2	7.0	42.6
	4.1	2.46	2.37	2.20	2.51	0.87	3.26	14.0	7.0	49.9
	4.6	2.46	2.32	2.17	2.47	0.30	4.31	15.8	6.8	57.3

The values calculated were the bulk specific gravity of the blended aggregate (G_{sb}), effective specific gravity of the blended aggregate (G_{se}), asphalt absorption (P_{ba}), effective asphalt content of the mixtures (P_{be}), percent voids in mineral aggregate in the compacted mixtures (VMA), percent air voids in the compacted mixtures (Va), and percentage of the voids in the mineral aggregate filled with asphalt (VFA or VFB).

The SUPERPAVE™ formula for determining the bulks specific gravity of the blended aggregate is:

$$Gsb = \frac{Prap + Pagg}{[Prap/Grap] + [Pagg/Gagg]} \tag{4.3}$$

where *Gsb* = bulk specific gravity of the total aggregate
Prap = percentage RAP aggregate in the blended aggregate
Papp = percentage virgin aggregate in the blended aggregate
Grap = RAP aggregate specific gravity
Gagg = virgin aggregate specific gravity

Substituting the measured values into Equation 4.3 gave the following:

$$Gsb = \frac{78.5+25}{[78.5/2.41]+[25/2.61]} = 2.455$$

The effective specific gravity of the recycled mixture includes the void spaces in the aggregate which do not absorb asphalt. The formula for the effective specific gravity is:

$$Gse = \frac{100-Pb}{[100/Gmm]-[Pb/Gb]} \tag{4.4}$$

where Gse = effective specific gravity of aggregate
Gmm = maximum theoretical specific gravity of the recycled mix at no air voids
Pb = asphalt content by total mass of the recycled mixture
Gb = specific gravity of the asphalt (assumed 1.03)

As an example of the Gse calculation, the measured Gmm for the CMS-2 recycled mixture was determined to be 2.37 at 4.6% asphalt content. Substituting these values into Equation 4.4 gives:

$$Gse = \frac{100-4.6}{[100/2.37]-[4.6/1.03]} = 2.53$$

The estimation of asphalt absorption by the aggregate is reported as a percentage of total aggregate mass. The formula for determination of asphalt absorption in the recycled mixture is:

$$Pba = 100 * \frac{[Gse-Gsb]}{[Gsb Gse]} * Gb \tag{4.5}$$

where Pba = absorbed asphalt

Substituting the values calculated earlier and assuming an asphalt content of 4.6% for the recycled mixture into Equation 4.5 gives:

$$Pba = 100 * \frac{[2.529 - 2.455]}{[(2.529)(2.455)]} * 1.03 = 1.22\%$$

The effective asphalt content is the percentage of asphalt in the mixture minus the percent absorbed asphalt. The effective asphalt content was determined by Equation 4.6.

$$Pbe = Pb - \frac{Pba}{100} * Ps \qquad (4.6)$$

where ***Pbe***	=	effective asphalt content as a percent of the mass of the mixture
Ps	=	aggregate as a percent of total mass of the mixture (95.4%)

Substituting the values into the formula for an asphalt content of 4.6% asphalt in the CMS-2 recycled mixture gives:

$$Pbe = 4.6 - \frac{1.222}{100} * 95.5 = 3.43\%$$

Calculation of the percent voids in mineral aggregate was made using Formula 4.7.

$$VMA = 100 - \frac{Gmb * Ps}{Gsb} \qquad (4.7)$$

where ***VMA*** = voids in mineral aggregate (percent of bulk aggregate volume)

For the CMS-2 recycled mixture with 4.6% asphalt in the mixture at 7% air voids, the VMA was:

$$VMA = 100 - \frac{[2.20 * 95.4]}{2.455} = 14.5\%$$

The estimated air voids in the mixture was 7%. The formula for calculation of air voids based on measured and calculated values is:

$$\boldsymbol{Va}=100*\frac{[Gmm\text{-}Gmb]}{Gmm} \tag{4.8}$$

where Va = percent air voids in the compacted mixture

Substituting the measured Gmm and Gmb for the CMS-2 recycled mixture with 4.6% total asphalt gave:

$$Va=100*\frac{[2.37\text{-}2.20]}{2.37}=7.2\%$$

The percentage of VMA filled with asphalt (VFA or VFB) is estimated with Equation 4.9:

$$VFA=100*\frac{[VMA\text{-}Va]}{VMA} \tag{4.9}$$

where VFA = voids in mineral aggregate filled with asphalt

For the CMS-2 recycled mixture with 4.6% asphalt, the VFA was:

$$VFA=100*\frac{[14.5\text{-}7.2]}{14.5}=50.6\%$$

The voids filled with asphalt (VFA) is synonymous with voids filled with bitumen (VFB) as used in some literature.

SUPERPAVE Volumetric specifications reflect 4% air voids. Calculations of volumetric properties at 4% air voids for the CMS-2 and HFRA recycled mixes are given in Table 4.4, while calculated values of volumetric properties at 7% air voids for the CMS-2 and

HFRA recycled mixes are given in Table 4.5.

The SUPERPAVE™ VMA criterion for mixtures with 4% air voids and the 19.0 mm nominal maximum size aggregate is a minimum 13.0%. Only the HFRA mixtures with 4.6% asphalt met this requirement. The VFA range of 65% to 75% was satisfied by the CMS-2 recycled mixtures with 4.6% and 5.1% asphalt contents and the HFRA recycled mixtures with 4.6% asphalt content. At 4% air voids, the HFRA met both SUPERPAVE™ VMA and VFA requirements at 4.6% asphalt content. This compares favorably to the results from the resilient modulus and indirect tensile tests on the HFRA recycled mixes in which the optimum asphalt content was determined to be 4.4% by weight of mix. The CMS-2 recycled mix did not meet the SUPERPAVE™ VMA criterion at any of the asphalt contents considered. However, the CMS-2 recycled mix did meet the SUPERPAVE™ VFA requirement at both 4.6% and 5.1% asphalt contents by weight of the mix. The results from the resilient modulus and indirect tensile tests on the CMS-2 recycled mixes indicated that the optimum asphalt content was 4.6%, which compared favorably with the VFA but not with the VMA requirements of SUPERPAVE™ volumetric analysis.

4.6.2 SUPERPAVE™ Shear Tests

Intermediate mix analysis involves testing of the specimens at the effective permanent deformation temperature, T_{eff}(PD), and the effective fatigue cracking temperature, T_{eff}(FC). Low temperature cracking tests were not conducted. T_{eff}(FC) used in this testing was 20^0 C. T_{eff}(PD) was 40^0 C. These test temperatures provided generally accepted conditions for standard intermediate mix analysis.

The test protocol included simple shear at constant height and frequency sweep at constant height at the effective temperatures. The asphalt contents used in the testing were 4.1%, 4.6% and 5.1% for the CMS-2 mixes and 3.6%, 4.1%, and 4.6% for the HFRA mixes. These asphalt contents were selected based on the results from the resilient modulus and indirect tensile tests. The results were used to compare fatigue and permanent deformation characteristics of the test mixtures.

The SUPERPAVE™ gyratory compactor was used to prepare the 150 mm (6 ") diameter specimens used in the shear tests. Compaction of the recycled mixes with 7% air voids was accomplished in accordance with *AASHTO TP4 (Preparation of Compacted Specimens of modified and Unmodified Hot Mix Asphalt by Means of the SHRP Gryatory Compactor)* with modifications.

The mix procedure used in the specimen preparation for resilient modulus was duplicated for this procedure with the exception that the SUPERPAVE™ Gyratory Compactor manufactured by Troxler Electronic Labs, Inc. was used instead of the GTM for specimen preparation. The angle of gyration of 1.25^0 and gyration rate of 30 RPM along with an end pressure of 0.6 MPa were maintained as specified in *AASHTO TP4*. The mass of the mixture necessary for a height of 130 mm was determined using the previously measured G_{mm} values and a specimen air void content of 7%. Estimations of G_{mb} were made using Equation 4.10.

$$\boldsymbol{Gmb = Gmm * [1-(.01 * Va)]} \tag{4.10}$$

where **Gmb** = bulk specific gravity of the compacted mix
Gmm = maximum theoretical specific gravity of the recycled mix
Va = air voids (%)

The actual G_{mb} of the compacted mixes were determined after the specimens were dried in a forced-draft oven at 60^0 C for 72 hours. The specimen heights were adjusted using the corrected G_{mb} values. The bulk specific gravities of the compacted recycled mixes at the various asphalt contents and 7% air voids for the CMS-2 and HFRA specimens are given in Table 4.5.

The specimens were compacted to a height slightly greater than 130 mm. The compacted specimens were allowed to cool in the mold over night and then extruded. After curing in a forced-draft oven at 60^0 C for 72 hours and allowed to cool, the specimens were sawn into two test specimens with heights of 50 to 55 mm.

In place of SUPERPAVE™ aging methods, the curing method developed at NCSU was used. The specimens were cured under the infra-red lamp for 4 hours a day for 10 days. The height of the light was adjusted so that the surface temperature of the specimens while the light was on was 60^0 C. The specimens were turned each day to ensure even curing.

4.6.2.1 Frequency Sweep at Constant Height

Frequency sweep at constant height (FSCH) is a controlled-strain test. The specimen is dynamically loaded to a controlled shear strain of ±0.05. The testing software returns the values for the shear complex modulus in pascals, shear storage modulus in pascals, shear loss modulus in pascals and the shear phase angle. FSCH tests were conducted on specimens at 20^0 C and 40^0 C.

SUPERPAVE™ analysis software was designed to input the values from the FSCH into rutting and fatigue models to predict pavement performance. The SUPERPAVE™ rutting and fatigue analysis programs were not available at the time this project was completed. The data from the FSCH at 20^0 C were used in a surrogate fatigue model which predicts fatigue life in terms of the number of supply ESALs (N_{supply}) [11]. The N_{supply} values estimated by the surrogate fatigue model were used to determine the relative fatigue performance of the mixtures at different asphalt contents.

The surrogate fatigue model uses the shear loss modulus from the FSCH and calculates the equivalent flexural loss stiffness. The empirical relationship was:

$$So'' = 81.125 * [Go'']^{0.725} \tag{4.11}$$

where So'' = initial flexural loss stiffness at 50th loading cycle (psi)
Go'' = shear loss stiffness at 10 Hz (psi)

N_{supply} was calculated using the empirical formula in Equation 4.12.

$$N_{supply}=2.738*10^5*[e^{0.077*VFB}]*[\epsilon_o^{-3.624}]*[So''^{-2.720}] \tag{4.12}$$

where N_{supply} = the number of load repetitions to 50% reduction in stiffness (crack initiation)
e = base of the natural logarithm
So'' = the initial flexural loss stiffness at 50th loading cycle (psi)
VFB = voids filled with bitumen (%)
ε_o = initial flexural strain at the bottom of a pavement layer

The R^2 for Equation 4.11 was 0.512 and the R^2 for Equation 4.12 was 0.79 [11]. The reliability of the prediction of N_{supply} was not considered high but the relative values of the fatigue life predictions were reliable indications of comparative fatigue susceptibility. In order to use the Go" values obtained from the FSCH test, the values which were obtained in pascals were converted to pounds per square inch (psi).

A multi-layered analysis of a pavement using ELSYM5 with a 4-inch binder layer with the shear moduli obtained from the FSCH tests at 20^0 C was used to estimate a reasonable value of ε_o. R^2 for Equation 4.13 was 0.712.

$$So=8.560*[Go]^{0.913} \tag{4.13}$$

where So = initial flexural stiffness at 50th loading cycle (psi)
Go = shear stiffness at 10 Hz (psi)

Table 4.6 contains the values for Go" in pascals and psi, the estimation of So" in psi from Equation 4.11 and So in psi from Equation 4.13 for CMS-2 and HFRA.

Table 4.6 So and So" Values Calculated from Go and Go" from Frequency Sweep at Constant Height Test at 20^0 C

Recycling Agent	Asphalt Content	Go" (Pa)	Go" (psi)	So" (psi) [1]	Go (Pa)	Go (psi)	So (psi) [2]
CMS-2	4.1	4.34x10^8	6.29x10^4	2.44x10^5	1.15x10^9	1.67x10^5	5.02x10^5
	4.6	4.69x10^8	6.80x10^4	2.59x10^5	1.13x10^9	1.63x10^5	4.92x10^5
	5.1	4.26x10^8	6.17x10^4	2.41x10^5	0.95x10^9	1.38x10^5	4.22x10^5
HFRA	3.6	3.42x10^8	4.96x10^4	2.06x10^5	0.90x10^9	1.31x10^5	4.02x10^5
	4.1	3.16x10^8	4.58x10^4	1.94x10^5	0.72x10^9	1.04x10^5	3.26x10^5
	4.6	3.12x10^8	4.53x10^4	1.93x10^5	0.64x10^9	0.93x10^5	2.94x10^5

[1] from Equation 4.11
[2] from Equation 4.13

Table 4.7 contains the predicted N_{supply} values from Equation 4.12. The values of VFA were obtained from Table 4.5 for the various asphalt contents of the CMS-2 and HFRA recycling agents.

Table 4.7 N_{supply} from Frequency Sweep at Constant Height Test at 20^0 C

Recycling Agent	Asphalt Content	So" (psi)	So (psi)	VFA (%) [1]	ε_o	N_{supply} [2]
CMS-2	4.1	2.44x10^5	5.02x10^5	47.7	1.72x10^{-4}	1.05x10^6
	4.6	2.59x10^5	4.92x10^5	50.6	1.74x10^{-4}	1.07x10^6
	5.1	2.41x10^5	4.22x10^5	53.0	1.86x10^{-4}	1.23x10^6
HFRA	3.6	2.06x10^5	4.02x10^5	42.6	1.91x10^{-4}	0.77x10^6
	4.1	1.94x10^5	3.26x10^5	49.9	2.09x10^{-4}	1.14x10^6
	4.6	1.93x10^5	2.94x10^5	57.3	2.18x10^{-4}	1.78x10^6

[1] from Table 4.5
[2] from Equation 4.12

The following input assumptions were constant. The wearing course was assumed to be 2 inches thick with an elastic modulus of 500 ksi. The aggregate base course was assumed to be 8 inches thick with an elastic modulus of 35 ksi. The subgrade was assumed to be semi-infinite with an elastic modulus of 10 ksi. All Poisson's ratios were assumed to be 0.35 except the subgrade which was assumed to be 0.40.

As an example, using the CMS-2 recycled mix with 4.6% asphalt content, Go" was 68.0 ksi. Substituting this value into Equation 4.11 gives the value of 259 ksi for So".

$$\boldsymbol{So''=81.125*[6.80*10^{4}]^{0.725}=2.59*10^{5}}$$

Inserting this value of So" and the VFA from Table 4.5 into Equation 4.12 gives the following:

$$\boldsymbol{N_{supply}=2.738*10^{5}*[e^{0.077*(50.6)}]*[(1.74*10^{-4})^{-3.624}]*[(2.59*10^{5})^{-2.720}]=1.07*10^{6}}$$

N_{supply} for the CMS-2 recycled mixture with 4.6% asphalt was estimated to be slightly greater than one-million ESALs.

Figure 4.10 shows the graph of N_{supply} vs. asphalt content for the CMS-2 and HFRA recycled mixtures. The higher asphalt content yielded a slightly longer fatigue life. The DSR tests on the binders predicted fatigue failure was not a critical concern. The poor mix performance below 4.1% asphalt content was due to an insufficient asphalt content and, furthermore, the VFA was too low, as indicated by the volumetric analyses.

The FSCH test was also performed at 40^0 C to evaluate the relative rutting resistance of the six mixtures. As with the SUPERPAVE™ fatigue prediction model, the SUPERPAVE™ rutting model was not available.

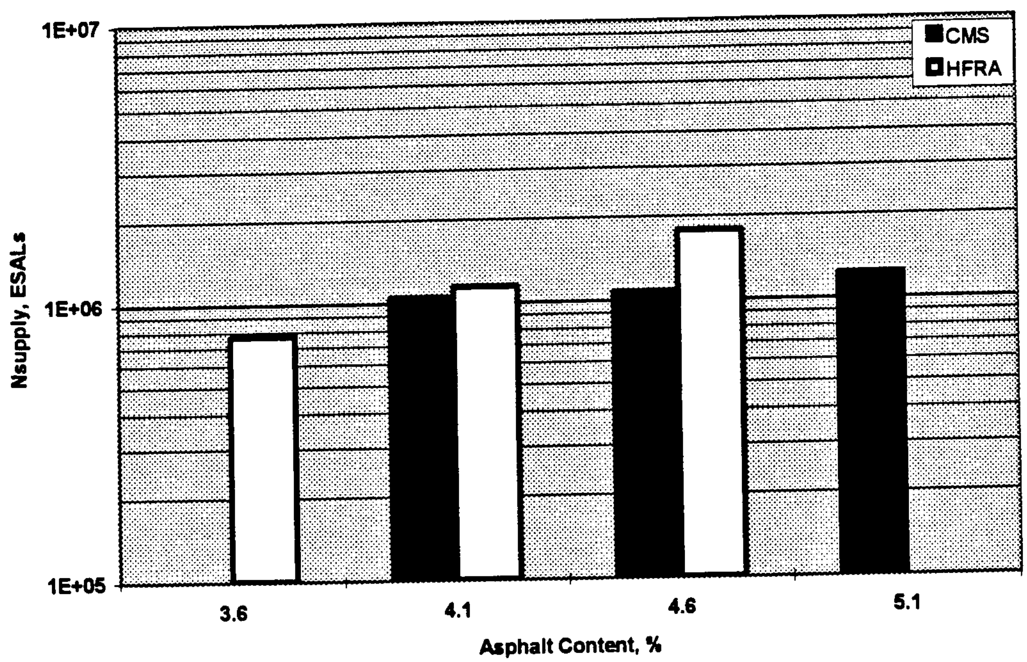

Figure 4.10 N_{supply} vs. Asphalt Content (Surrogate Fatigue Model)

The FSCH tests provided values for the shear complex moduli (G*) of the mixes at the various loading frequencies. Figure 4.11 is a log-log graph of the shear complex modulus vs. frequency for the CMS-2 recycled mixes at 40^0 C. Figure 4.12 is the graph of the log shear complex modulus vs. log frequency for the HFRA recycled mixes.

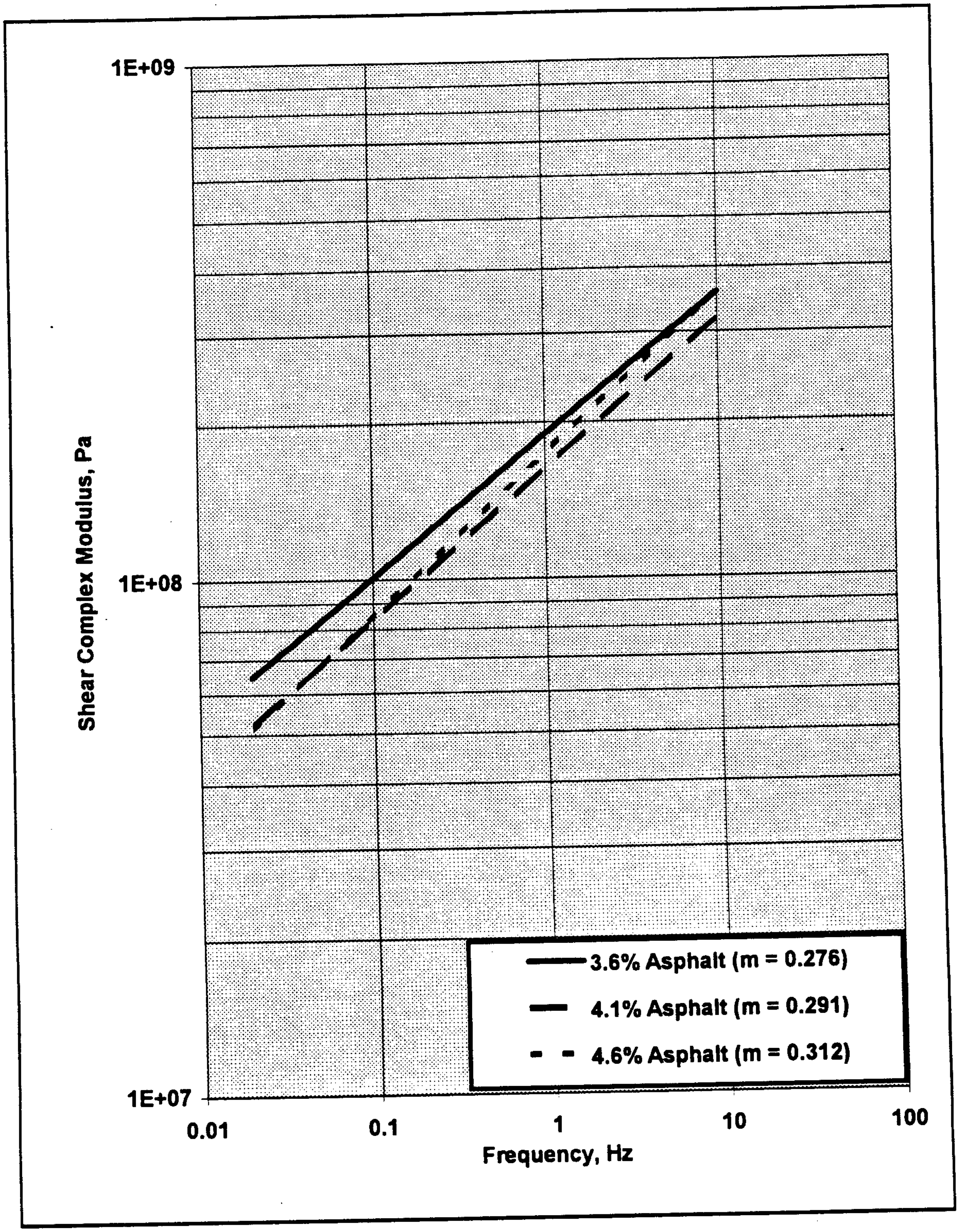

Figure 4.11 CMS-2 Recycled Mix
Shear Complex Modulus vs. Frequency at 40^0 C

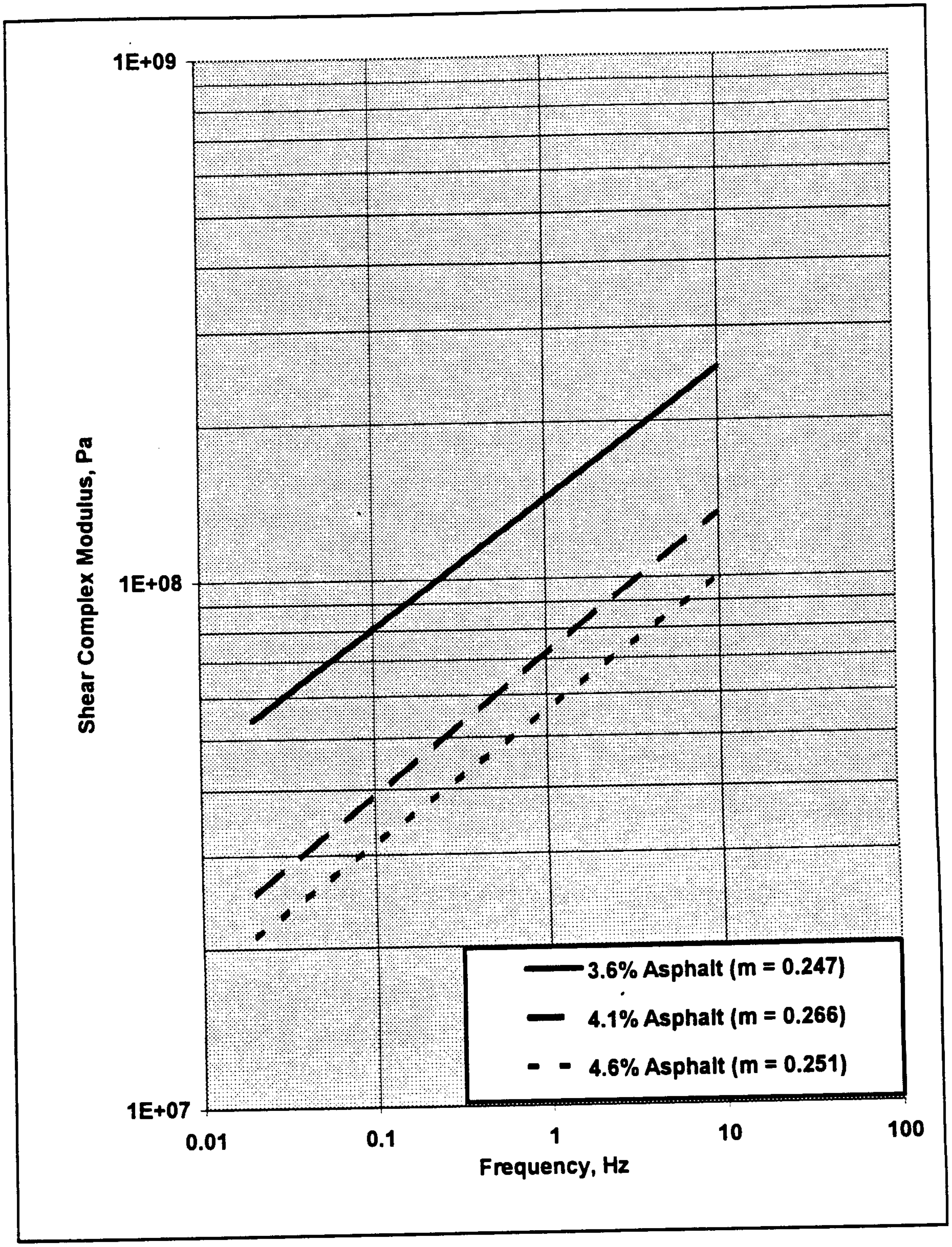

Figure 4.12 HFRA Recycled Mix
Shear Complex Modulus vs. Frequency at 40^0 C

A characteristic of the curves was that the G* increased with increasing load frequency. This was expected due to the viscoelastic nature of asphalt and asphalt concrete mixes. Another characteristic observed was that the mixes with the lower asphalt contents had the higher shear complex moduli at all frequencies. The differences in G* among the CMS-2 mixes were minor, while the differences among the HFRA recycled mixes were greater.

The G* of the CMS-2 mix at 4.1% asphalt content was also considerably larger than the HFRA mix with 4.1% asphalt. However, the HFRA mix with 3.6% asphalt content exhibited similar shear characteristics to that of the CMS-2 mixture with 4.1% asphalt. This trend was expected since the DSR results indicated that the HFRA residue was more viscous than the CMS-2 residue, especially at higher temperatures.

The relative rutting potential of asphalt concrete samples have traditionally been determined by creep compliance from the creep test. The slope of the creep compliance curve has been determined to be the same as the slope of the log G* vs. log frequency curve (m) obtained from the FSCH test [10]. The greater the slope of the log G* vs. log frequency curve, the greater the temperature susceptibility of the mixture and the more susceptible to permanent deformation at higher temperatures.

Table 4.8 gives the regression slopes of the curves from Figures 4.11 and 4.12. The slopes indicate that, in general, with increasing asphalt content, G* decreases and m increases. However, no great differences in m were exhibited. As the data in Table 4.8 indicate, the higher the asphalt content of the mixes tested, the more susceptible the mixes were to permanent deformation.

Table 4.8 Slopes of the Log Shear Complex Modulus vs. Log Frequency

Recycling Agent	Asphalt Content (%)	G* at 10 Hz (Pa)	Slope of Log G* vs. Log Frequency Curve
CMS-2	4.1%	3.75×10^8	0.276
	4.6%	3.39×10^8	0.291
	5.1%	3.73×10^8	0.312
HFRA	3.6%	2.67×10^8	0.247
	4.1%	1.46×10^8	0.266
	4.6%	1.13×10^8	0.251

4.6.2.2 Simple Shear at Constant Height

SUPERPAVE™ Intermediate Mix Tests also incorporates the simple shear at constant height (SSCH) test at T_{eff}(PD) and T_{eff}(FC). T_{eff}(PD) was 40° C and T_{eff}(FC) was 20° C for this project. Table 4.9 shows typical data obtained from the SSCH at 40° C.

Table 4.9 Data from the Simple Shear at Constant Height Test at 40° C

Recycling Agent	Asphalt Content (%)	Maximum Shear Stress (Pa)	Maximum Shear Strain	Plastic Strain
CMS-2	4.1%	31352	1.15×10^{-3}	0.67×10^{-3}
	4.6%	32580	2.11×10^{-3}	1.33×10^{-3}
	5.1%	32867	2.05×10^{-3}	1.35×10^{-3}
HFRA	3.6%	33480	2.06×10^{-3}	1.25×10^{-3}
	4.1%	33275	3.43×10^{-3}	2.27×10^{-3}
	4.6%	33112	3.97×10^{-3}	2.45×10^{-3}

Table 4.9 shows the expected trend that the higher the asphalt content considered and the softer the recycling agent in a mixture, the greater the maximum shear and plastic strains

exhibited. Plastic strain is permanent deformation and may be indicative of the rutting potential of a mix.

In general, the SUPERPAVE™ Intermediate test data supported the results from the resilient modulus and indirect tensile tests. They also supported the predictions from the DSR tests relative to rutting and fatigue. SUPERPAVE™ performance predictions were not available, but comparative performance predictions among asphalt contents and recycling agents were determined.

The CMS-2 was slightly more fatigue resistant at 5.1% asphalt content, while the CMS-2 mix with 4.1% asphalt content exhibited a greater resistance to permanent deformation. The HFRA recycled mix with 4.6% asphalt content was less susceptible to fatigue failure than the other asphalt contents considered. The HFRA recycled mix with 3.6% asphalt content exhibited a higher permanent deformation resistance. Absent specific design criteria, maximizing both the fatigue and permanent deformation resistant properties of the mixtures would provide the optimum design mix. Therefore, based on the shear test results, the optimum asphalt content of the CMS-2 recycled mix was between 4.1% and 5.1% which agrees with the results of the resilient modulus and indirect tensile strength test results of 4.6% optimum asphalt content. Likewise, the optimum asphalt content of the HFRA recycled mix was between 3.6% and 4.6% which supports the resilient modulus and indirect tensile test results of 4.4% optimum asphalt content.

CHAPTER 5

ANALYSIS OF TEST RESULTS

The testing and analysis of cold-recycled mixes included characterization of materials used in the mixes, as well as mechanical testing of mix specimens at several asphalt contents. A main objective of this study was to develop a simplified method of determining the optimum asphalt content while utilizing the maximum amount of RAP in the mixes. The optimum asphalt content is determined by maximizing pavement performance in terms of resistance to permanent deformation and fatigue.

The RAP sample asphalt exhibited desirable rutting and fatigue characteristics for a PG64-16 binder without rejuvenation. The reclaimed binder performance grading exceeded the requirements of PG64-16. Addition of recycling agent was necessary for blending and binding of virgin aggregate with the RAP. Under these conditions, it was assumed and verified that the stiffest recycling agent emulsion residue available would provide the most desirable mixture properties. Mix testing showed that increasing the recycling agent content reduced the mixture's resistance to permanent deformation. The higher the RAP content of the recycled mixes, the better the performance in relation to permanent deformation.

In this project, the amount of RAP utilized in the recycled mixtures was dependent upon the gradation requirements for the pavement binder layer. Addition of aggregate from a #6 stockpile allowed for mixtures with 75% RAP. The resulting gradation which met the binder specifications for North Carolina also met the SUPERPAVE™ gradation requirements for 19.0 mm nominal maximum size aggregate mixtures. With this as the basis of the design, the determination of the optimum asphalt content in the mix became the focus of the research.

The blend chart in Figure 5.1 shows that the DSR characterization of the RAP asphalt and the recycling agent residues indicated that in the fully blended condition, the theoretical amount of CMS-2 recycling agent allowable in the total asphalt was within a range of 0% to 72%.

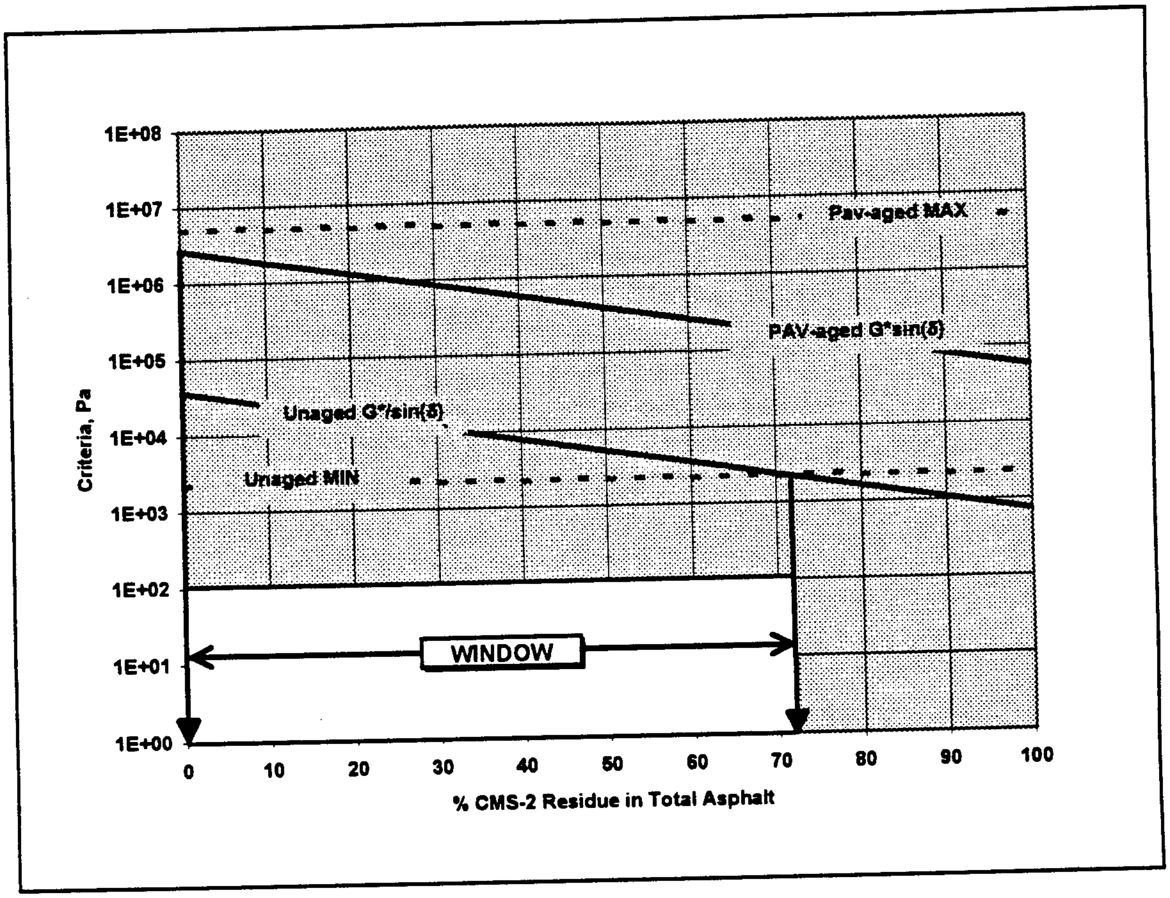

Figure 5.1 CMS-2 / RAP Blending Chart

Figure 5.2 shows that the amount of HFRA recycling agent allowable was between 0% and 50%. Knowing that any unblended recycling agent would increase the propensity for permanent deformation in the mixes, it was assumed that the optimum recycling agent content was closer to the lower limit than to the upper limit of the blend chart.

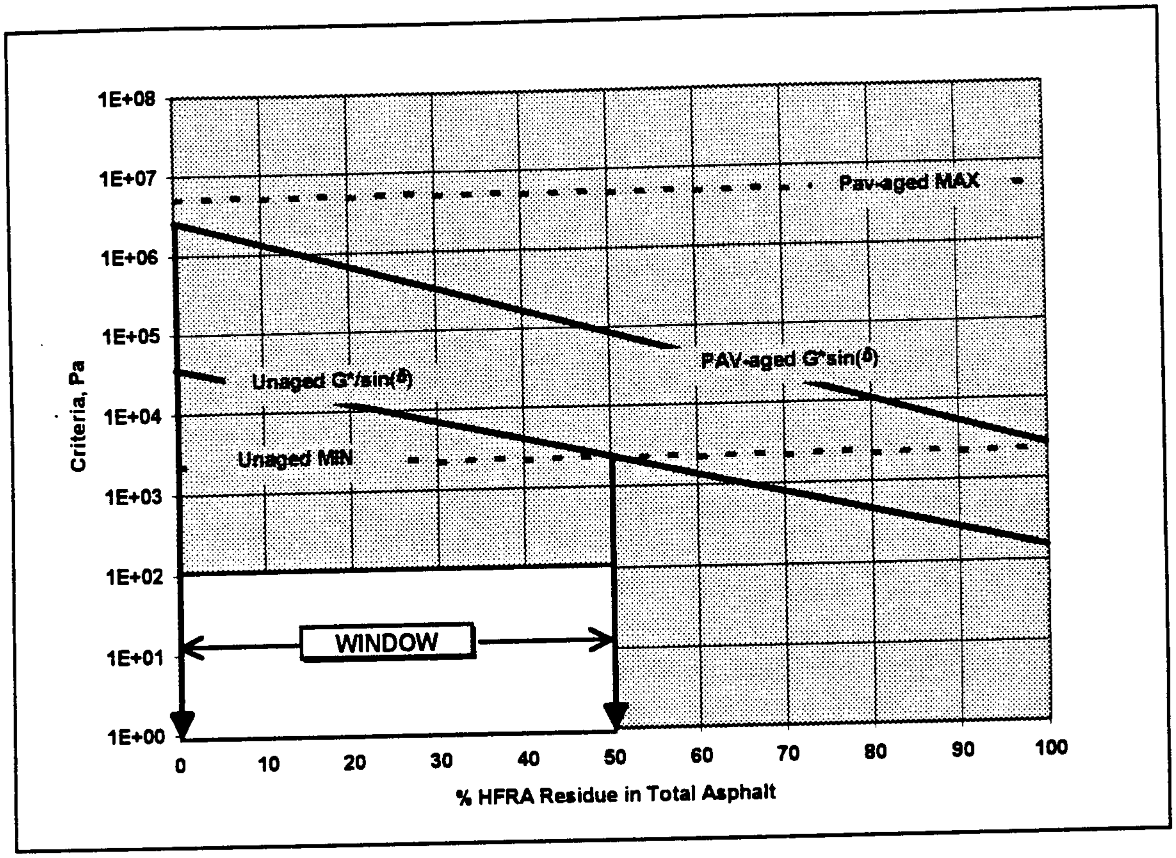

Figure 5.2 HFRA / RAP Blending Chart

The relationships between the asphalt content of the mixtures and the percent recycling agent residue in the total asphalt content and the asphalt content of the mixtures are given in Table 5.1. The range of asphalt contents shown in Table 5.1 reflects the range of asphalt contents tested in determination of the optimum for this project.

Table 5.1 Relationship Between Percent Recycling Agent Residue in Total Asphalt and Asphalt Content of the Recycled Mixtures Containing 75% RAP

Asphalt Content of Mix (% by wt. of mix)	3.6	3.9	4.1	4.4	4.6	4.9	5.1	5.4	5.6
Emulsion Residue Content. (% of total asphalt)	6	13	18	23	27	31	34	38	40

The estimation of the asphalt binder requirement for the recycled mixtures was initially made using the aggregate surface area method. For the CMS-2 recycled mixtures, the initial asphalt content was calculated to be 5.1%, which equated to 34% CMS-2 residue in the asphalt and was within the acceptable range of 0% to 72% determined by DSR testing. Resilient modulus and indirect tensile tests were conducted on CMS-2 recycled mixes with 5.1% asphalt content, and at asphalt contents of 5.1% ± 0.5% and 5.1% + 1%. These tests revealed that the aggregate surface area method provided a binder-rich mixture. The HFRA recycled mixes were therefore prepared with asphalt contents of 3.9%, 4.4%, 4.9% and 5.4% to account for the richness of the surface area prediction and the lower viscosity of the HFRA residue.

The results of the resilient modulus and indirect tensile tests are shown in Table 5.2. These results supported the assumption that the optimum recycling agent residue content for the recycled mixes was closer to the lower limit of the acceptable range (0%) as determined from the DSR testing than to the upper limit of the acceptable range (72% for CMS-2 and 50% for HFRA). The results also supported the assumption that the optimum percent of CMS-2 residue in the recycled mix would be higher than the optimum percent HFRA residue in the recycled mix due to the lower viscosity of the HFRA residue.

Table 5.2 Optimum Asphalt Content Determinations Based on Resilient Modulus and Indirect Tensile Tests

Recycling Agent	Resilient Modulus Test (Oven cured Specimens)		Resilient Modulus Test (Oven cured & Infra-red cured Specimens)		Indirect Tensile Test (Oven cured Specimens)	
	Optimum Asphalt Content	Recycling Agent Residue in Asphalt	Optimum Asphalt Content	Recycling Agent Residue in Asphalt	Optimum Asphalt Content	Recycling Agent Residue in Asphalt
CMS-2	4.6%	27%	4.6%	27%	4.6%	27%
HFRA	3.9%	13%	4.4%	23%	4.4%	23%

The resilient modulus and the indirect tensile tests on the CMS-2 recycled mixes indicated that the optimum asphalt content of the recycled mixes was 4.6%, the lowest asphalt content tested. The resilient modulus tests on the HFRA specimens oven cured for 72 hours at 60^0 C in a forced draft indicated that the optimum asphalt content was 3.9%. The resilient modulus tests on the HFRA specimens which had been infra-red cured for 4 hours-per-day for 10 days following 72 hours in the forced draft oven indicated that the optimum asphalt content was 4.4%. This was supported by the indirect tensile tests conducted on specimens which had only been cured in the forced draft oven for 72 hours. In all cases, mix stiffness rapidly decreased at higher asphalt contents.

In the combination oven cured and infra-red lamp cured conditions, the average resilient modulus for the CMS-2 mixtures was 5051 MPa (733 ksi). Under the same curing conditions, the average resilient modulus for the HFRA mixtures was 2093 MPa (304 ksi). The average resilient modulus of the CMS-2 mixes at optimum asphalt content was nearly 59% greater than the average resilient modulus of the HFRA mixes at optimum asphalt content. Comparatively, the CMS-2 recycled mix performed better in the resilient modulus and indirect tensile tests than the HFRA recycled mix. At the respective optimum asphalt contents, CMS-2 mixtures were considerably stiffer than the HFRA mixtures.

The comparative results of the indirect tensile tests on the oven cured samples were similar. The indirect tensile strength of the CMS-2 mixes at the optimum asphalt content was 1158 kPa (168 psi). The indirect tensile strength of the HFRA mixes at optimum asphalt content was 521 kPa (75.6 psi). The average indirect tensile strength of the CMS-2 recycled mixtures at the optimum asphalt content was approximately 55% greater than the indirect tensile strength of the HFRA recycled mixtures at the optimum asphalt content.

Shear tests on the recycled mixtures in accordance with SUPERPAVE™ specifications utilized the results of the optimum asphalt contents from the resilient modulus and indirect tensile tests. For comparison purposes, the HFRA asphalt contents conformed with the of the CMS-2 asphalt contents. The CMS-2 recycled mixes were tested at asphalt contents of 4.1%, 4.6% and 5.1%. The HFRA recycled mixes were tested at asphalt contents of 3.6%, 4.1% and 4.6%. The lower asphalt contents of the HFRA mixes were based on the lower optimum asphalt content of the HFRA mixes from the resilient modulus and indirect tensile tests.

The shear test samples were prepared using the Troxler SUPERPAVE™ Gyratory Compactor at 7% air voids. SUPERPAVE™ intermediate mix analysis is recommended for design traffic levels in the range of 1 million to 10 million ESALs. The design ESALs for the pavements which will utilize the cold in-place recycling method was assumed to be somewhat less than 10 million. SUPERPAVE™ testing was used to support the resilient modulus and indirect tensile test results. The ability to conduct mechanical shear tests in controlled temperature environments greatly facilitated the analysis process. The test protocol included simple shear at constant height (SSCH) at 20^0 C and 40^0 C and frequency sweep at constant height (FSCH) at 20^0 C and 40^0 C. The data from the tests at 40^0 C were used for permanent deformation analysis while the data from the tests at 20^0 C were used for fatigue analysis.

The SUPERPAVE™ analysis software was intended to predict fatigue and rutting performance of pavements. The software was not reliable at the time of completion of this project. However, relative fatigue and rutting determinations were made for the purpose of defining optimum asphalt contents.

The fatigue analysis used a surrogate model for predicting N_{supply}. Though fatigue was not considered the critical failure mode, the analysis was undertaken for comparison purposes. Table 5.3 shows that the fatigue analysis results indicated that the higher asphalt contents provided better fatigue resistance.

Table 5.3 **Results of the FSCH Test at 20^0 C and the A-003 Fatigue Model**

Recycling Agent	Asphalt Content (%)	N_{supply} (ESALs)
CMS-2	4.1 %	1.05×10^6
	4.6 %	1.07×10^6
	5.1 %	1.23×10^6
HFRA	3.6 %	0.77×10^6
	4.1 %	1.14×10^6
	4.6 %	1.78×10^6

The SUPERPAVE™ rutting model was not available for prediction of rutting performance of the mix samples. The results of the FSCH test at 40^0 C were used to compare the complex shear moduli (G*) and to compare the slopes (m) of the log G* vs log frequency curves. Based on the time-temperature correspondence principle of asphalt cement and asphalt concrete mixes, the slope (m) of the log G* vs log frequency curve is indicative of the temperature susceptibility of the mixture and thus the relative rutting potential of the mixture at higher temperatures. The smaller the value of m, the lower the temperature susceptibility and the more desirable the mix. G* is a measure of the shear stiffness of the mix.

The results of this analysis are given in Table 5.4. The CMS-2 recycled mix with 4.1% asphalt content had higher G* at 10 Hz than all other samples. The HFRA recycled mix with 3.6% asphalt content had the highest G* of the HFRA samples. G* of the HFRA recycled mix at 3.6% asphalt content was lower than all of the CMS-2 mixes tested. The least slope of the log G* vs. log frequency curve was exhibited by the HFRA recycled mix with 3.6% asphalt content. The HFRA recycling agent residue makes up only 5% of the total asphalt content at 3.6% asphalt content of the mix by weight of mix. The stiffness and low temperature susceptibility of the RAP asphalt predominates at the lower emulsion residue contents. At the 4.1% asphalt content, the slope of the log G* vs. log frequency curve for the HFRA recycled mix was slightly less than the CMS-2 recycled mix at 4.1% asphalt content. However, the shear stiffness of the HFRA recycled mix at 4.1% asphalt content was much lower than the CMS-2 recycled mix at 4.1% asphalt content.

Table 5.4 Comparisons of G* and Slopes of the Log G* vs. Log Frequency Curves from the FSCH Test at 40^0 C

Recycling Agent	Asphalt Content (%)	G* at 10 Hz (Pa)	Slope of Log G* vs. Log Frequency Curve
CMS-2	4.1 %	3.75×10^8	0.276
	4.6 %	3.39×10^8	0.291
	5.1 %	3.73×10^8	0.312
HFRA	3.6 %	2.67×10^8	0.247
	4.1 %	1.46×10^8	0.266
	4.6 %	1.13×10^8	0.251

Figure 5.3 shows the shear complex moduli of the CMS-2 and HFRA recycled mixes at 10 Hz load rate, which is comparable to field loading conditions under SUPERPAVE™. The CMS-2 recycled mix complex shear moduli were higher than the HFRA recycled mix complex shear moduli at all asphalt contents. The differences among the CMS-2 complex shear moduli at different asphalt contents were minor.

The data from the simple shear test at constant height (SSCH) was not used in a rutting model, though the SUPERPAVE™ software will utilize the results of the SSCH in the rutting model when the analysis software is available. The SSCH is a stress controlled test which returns maximum shear strain and allows for determination of plastic shear strain (permanent strain), which may be indicative of rutting potential. The data obtained from the SSCH provided the same relative results as the FSCH. As asphalt content increased so did maximum shear strain and plastic shear strain, indicating a greater propensity for

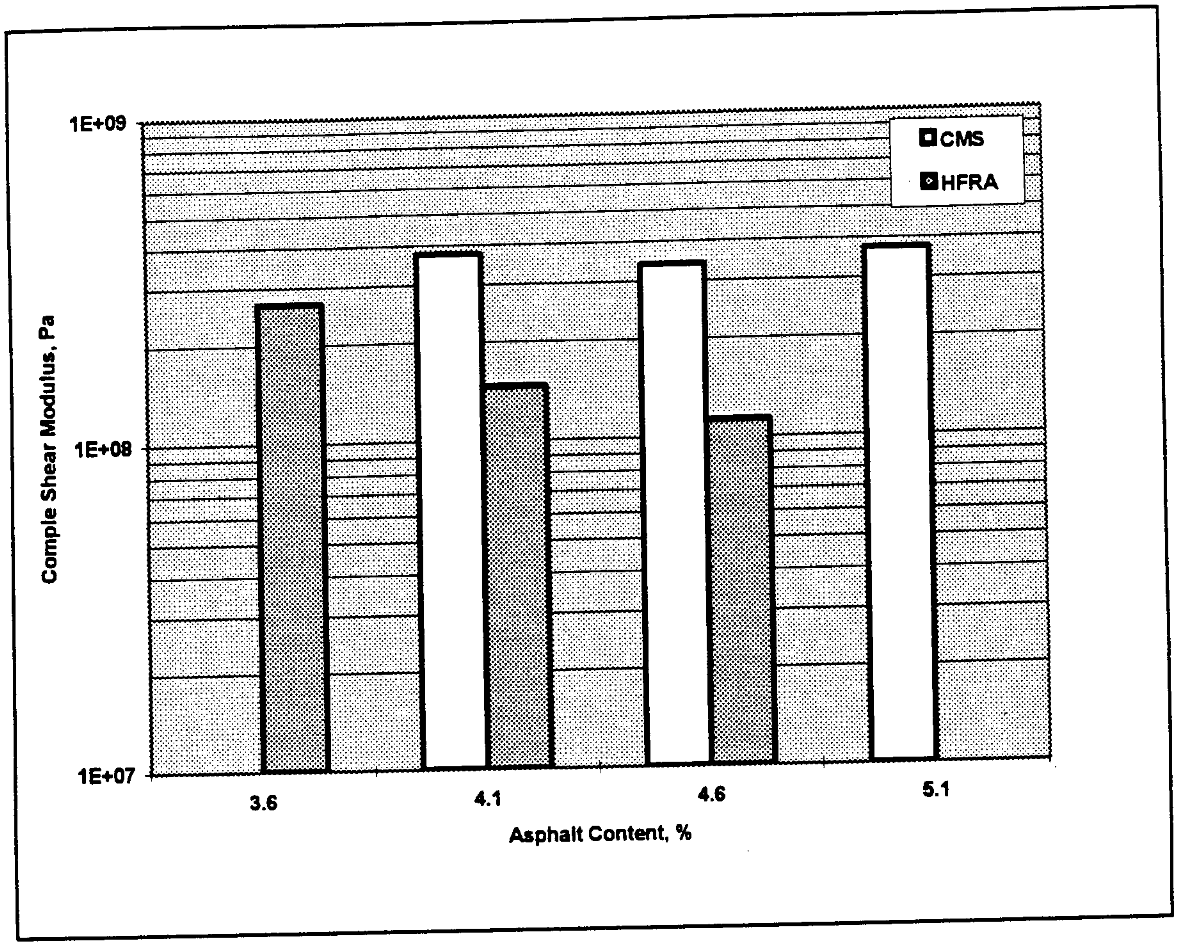

Figure 5.3 Shear Complex Modulus vs. Asphalt Content

permanent deformation. The CMS-2 recycled mix with 4.1% asphalt content deformed in shear less than the other specimens. The HFRA recycled mix with 3.6% asphalt content had lower strain levels than the HFRA samples with 4.1% and 4.6% asphalt.

The Asphalt Institute formula for permanent deformation analysis was used [15]. The normal compressive strain, ε_c, at the top of the subgrade based on the wheel configuration of an 18-wheel tractor trailer with 18-kip single axle loads was used in the mechanistic analysis. The rutting model used was:

$$N_d = (1.365 \times 10^{-9})\ (\varepsilon_c^{-4.477}) \qquad (5.1)$$

where: N_d = supply ESAL load repetitions for rutting
ε_c = normal compressive strain at top of subgrade

The normal compressive strains at the top of the subgrade were obtained from the ELSYM5 analyses. The resilient moduli from Table 4.2 for the oven and infra-red cured conditions were used as the binder layer elastic moduli. The pavement binder layer was assumed to be four inches thick in each analysis. The other input variables were constant as well. These included a two-inch surface course with an elastic modulus of 500 ksi; an eight-inch aggregate base course with an elastic modulus of 35 ksi; and a semi-infinite subgrade with an elastic modulus of 10 ksi. All Poisson's ratios were assumed to be 0.35 except for the subgrade Poisson's ratio which was assumed to be 0.40.

Table 5.5 gives the data from the permanent deformation analysis. Though all layers affect the strain magnitude, the only variable was the stiffness of the recycled binder layer. Therefore, the differences in N_d reflect differences in the asphalt contents of the mix specimens.

Table 5.5 Rutting Analysis Using Asphalt Institute Mechanistic Model

Recycling Agent	Asphalt Content	ε_c	N_d (ESALs)
CMS-2	4.1 %	4.38×10^{-4}	1.48×10^{6}
	4.6 %	4.50×10^{-4}	1.32×10^{6}
	5.1 %	4.80×10^{-4}	0.98×10^{6}
	5.6 %	5.12×10^{-4}	0.74×10^{6}
HFRA	3.9 %	5.47×10^{-4}	0.55×10^{6}
	4.4 %	5.13×10^{-4}	0.73×10^{6}
	4.9 %	5.64×10^{-4}	0.48×10^{6}
	5.4 %	6.25×10^{-4}	0.30×10^{6}

Overall, the CMS-2 recycled mix samples performed better with regards to stiffness, shear stiffness and resistance to permanent deformation. The HFRA samples performed slightly better in fatigue and exhibited a greater softening effect. The HFRA was more workable because of its fluid consistency.

It was anticipated from the DSR G*/sin(δ) results on the unaged emulsion residues that the CMS-2 samples would be more resistant to rutting than the HFRA samples. It was also predicted that fatigue would not be a critical failure mode of the recycled mixes. In general, the results of the resilient modulus, indirect tensile, and shear tests supported the results of the DSR tests. Figure 5.4 shows the relationship between the fatigue-life predictions from the surrogate fatigue model and the permanent deformation-life predictions from the permanent deformation analyses for the CMS-2 recycled mixes.

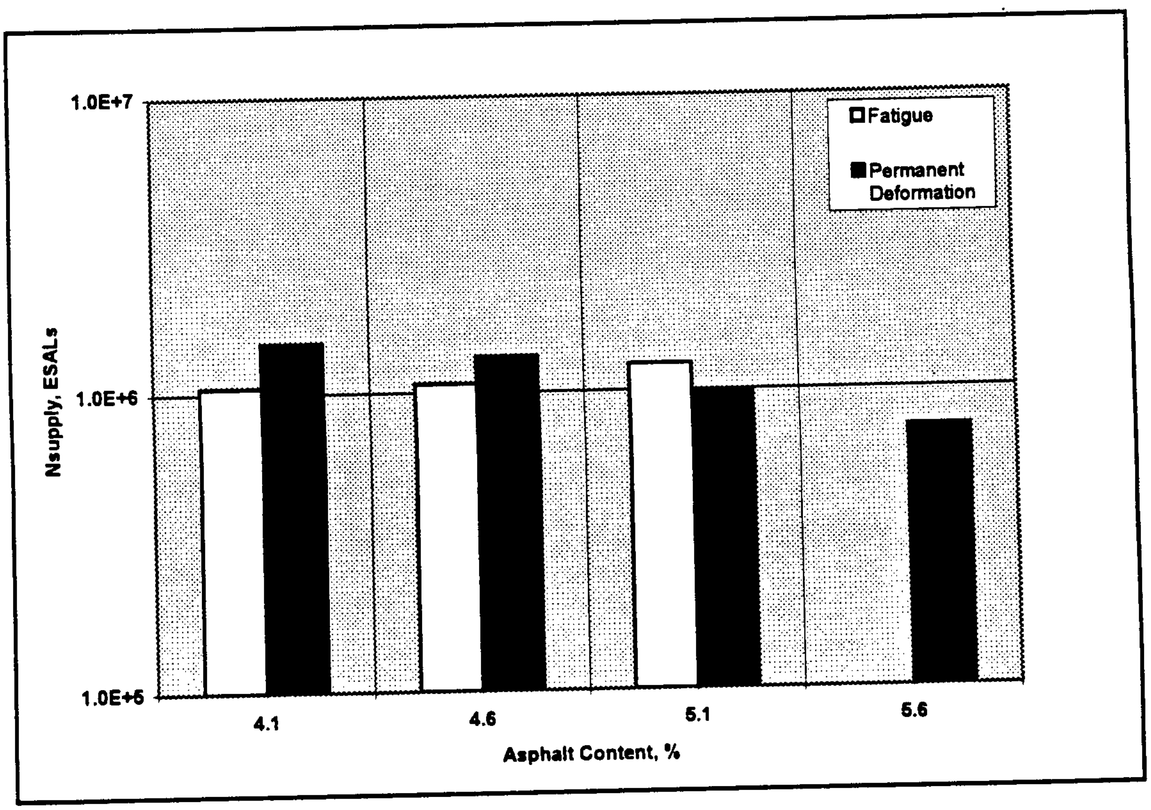

Figure 5.4 CMS-2 Permanent Deformation and Fatigue-Life Predictions

Increasing the asphalt content above 4.6 % by 0.5% reduced the rutting supply ESALs and decreasing the asphalt content below 4.6% by 0.5% reduced the fatigue supply ESALs. Since the fatigue and rutting predictions were maximized at 4.6% asphalt content, the CMS-2 recycled mix with 4.6% asphalt content was selected as the optimum mix.

Figure 5.5 shows the relationship between the fatigue-life predictions from the surrogate fatigue model and the permanent deformation-life predictions from the permanent deformation analyses for the HFRA recycled mixes. Since rutting was especially critical with the HFRA recycled mixes, those mixes with 4.4% asphalt content were considered optimum. 4.4% was the asphalt content exhibiting the highest number of permanent deformation supply ESALs.

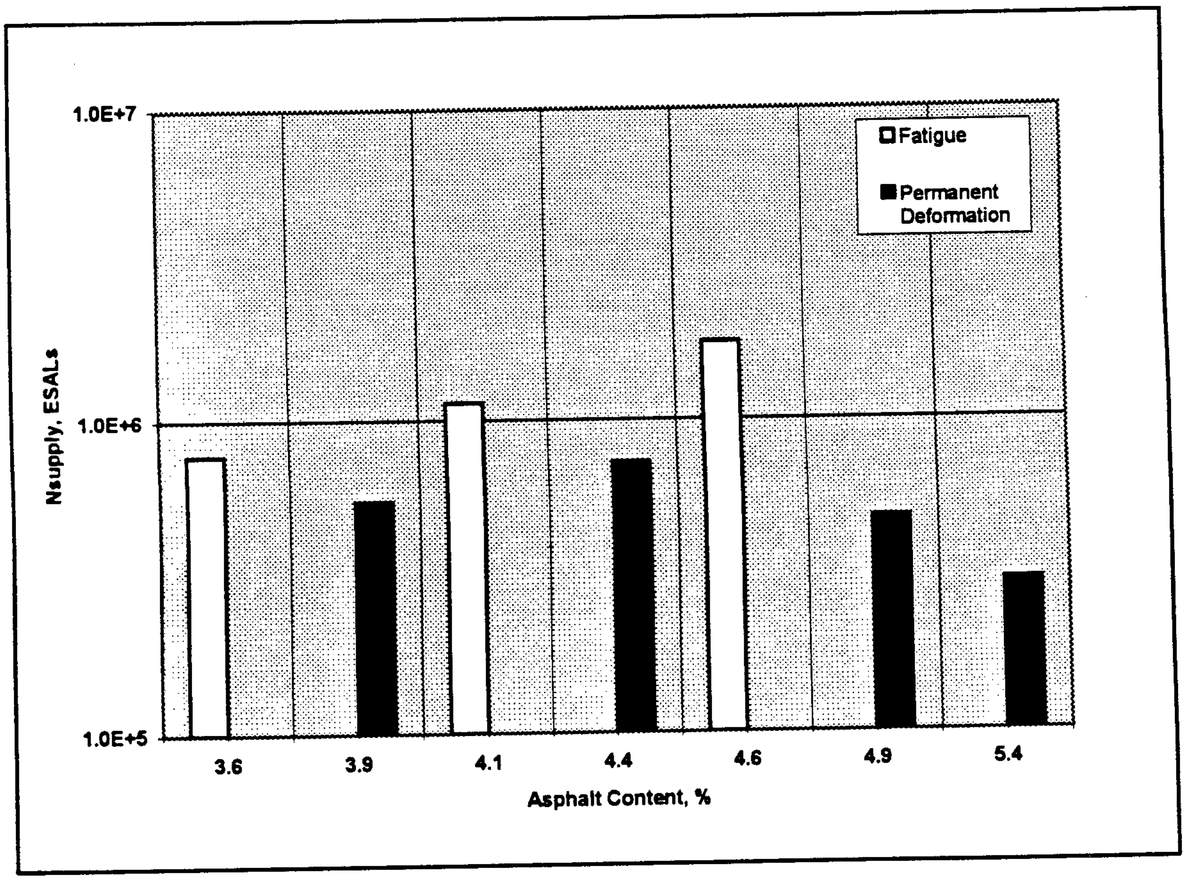

Figure 5.5 HFRA Permanent Deformation and Fatigue-Life Predictions

The CMS-2 emulsion was the more desirable of the two tested because of the material characteristics of the RAP asphalt. Though it was a design consideration, fatigue was not considered the critical failure mode. Therefore, for the RAP used in this study, the stiffer recycling agent residue was more desirable.

Table 5.6 shows the optimum asphalt contents based on each test analysis.

Table 5.6 Optimum Asphalt Contents from Test Analyses

Recycling Agent	Resilient Modulus (oven cured)	Resilient Modulus (oven & infra red light cured)	Indirect Tensile (oven cured)	Fatigue & Permanent Deformation
CMS-2	4.6 %	4.6 %	4.6 %	4.6 %
HFRA	3.9 %	4.4 %	4.4 %	4.4 %

Table 5.7 shows the optimum percent recycling agent residue in the total asphalt.

Table 5.7 Optimum Percent of Recycling Agent Residue in Total Asphalt from Test Results

Recycling Agent	Resilient Modulus (oven cured)	Resilient Modulus (oven & infra red light cured)	Indirect Tensile (oven cured)	Fatigue & Permanent Deformation
CMS-2	27 %	27 %	27 %	27 %
HFRA	13 %	23 %	23 %	23 %

In general, the results indicated that the lower the asphalt content, the better the mixtures performed with regards to rutting resistance. The opposite was true when considering fatigue resistance. A general design approach would be to consider both failure modes and then determine which asphalt content satisfies the design criteria for rutting and fatigue.

Figure 5.6 is the binder blending chart for the CMS-2 residue and RAP asphalt DSR tests. The optimum results from the resilient modulus, indirect tensile, rutting and fatigue analyses have been added to the chart. The optimum asphalt contents for the test analyses were all within the acceptable range predicted by the DSR tests.

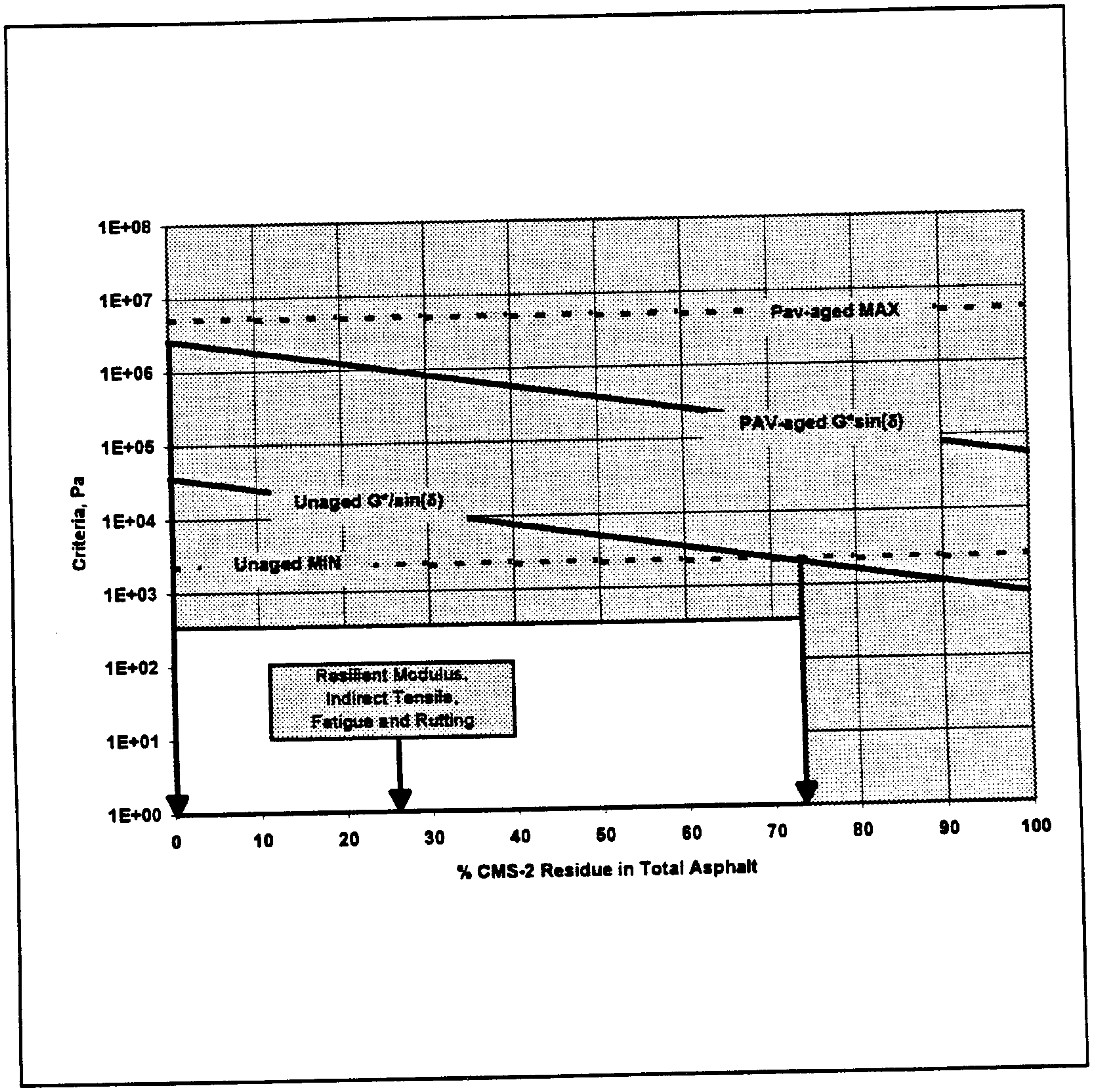

Figure 5.6 **CMS-2 / RAP Blending Chart with Mix Test Results**

Figure 5.7 provides the same information for the HFRA recycled mixes. The resilient modulus and indirect tensile tests were shown to be reliable tools for optimum mix selection along with the results of the DSR binder tests.

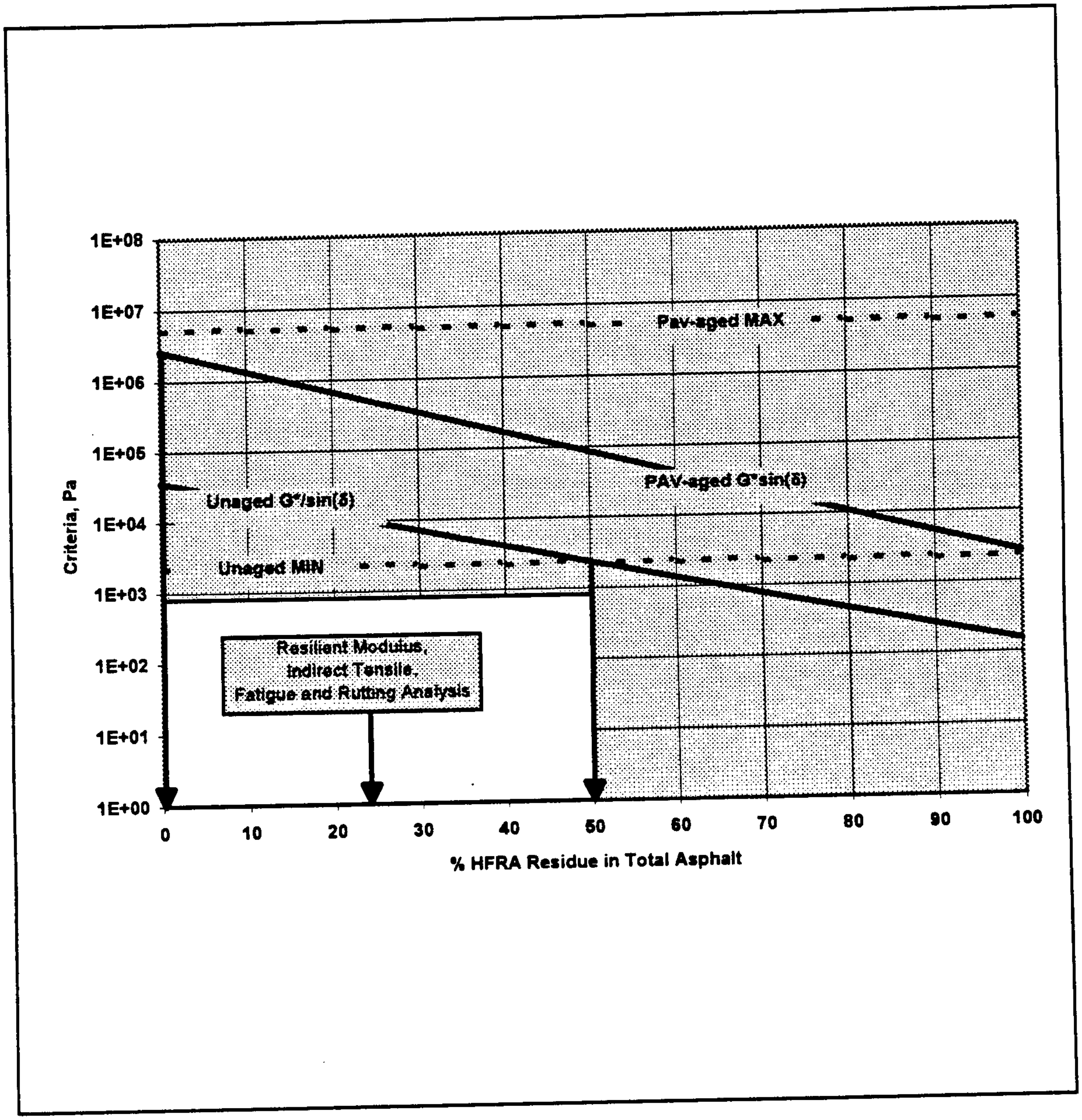

Figure 5.7 HFRA / RAP Blending Chart with Mix Test Results

Based on the AASHTO structural design, the contribution of the binder layer in terms of 18-kip single axle loads, W_{18}, in the design lane were determined [16]. The design criteria used were:

Standard normal deviate, Z_R:	-1.282	(Reliability, R = 0.90)
Standard deviation, So:	0.45	
Change in pavement serviceability, ΔPSI:	2.0	
Binder layer thickness, D:	4 inches	

The structural layer coefficient for the recycled mixtures is based on the resilient modulus values at the optimum asphalt contents. Table 5.8 below gives the relationship between the resilient moduli and the AASHTO structural layer coefficients.

Table 5.8 AASHTO Structural Layer Coefficients for Recycled Mixtures

Recycling Agent	Aged Condition	Resilient Modulus (psi)	AASHTO Structural Layer Coefficient [16]
CMS-2	72 hours in forced draft oven	520,000	0.45
	72 hours in forced draft oven & infra red lamp	733,000	0.45
HFRA	72 hours in forced draft oven	180,000	0.28
	72 hours in forced draft oven & infra red lamp	304,000	0.36

The structural number used was the sum of structural number contributions of the wearing course, the aggregate base course, and the binder layer. The 2-inch wearing course contribution to the structural number was 0.9. The 8-inch aggregate base course contribution was 1.20. The elastic modulus of the surface course was assumed to be constant and was not included in the calculation. The W_{18} contributions of the binder layer of the CMS-2 recycled mixes are given in Table 5.9.

Table 5.9 AASHTO Structural Layer Contributions of the CMS-2 Recycled Binder Mixes

Asphalt Content (%)	Resilient Modulus (ksi)	Binder Structural Layer Coefficient, a_2	Structural Number, SN	W_{18} (ESALs)
4.6	733	0.45	3.9	5.06×10^6
5.1	639	0.45	3.9	5.06×10^6
5.6	444	0.43	3.8	4.41×10^6
6.1	308	0.37	3.6	2.87×10^6

The W_{18} contributions of the binder layer of the HFRA recycled mixes are given in Table 5.10.

Table 5.10 AASHTO Structural Layer Contributions of the HFRA Recycled Binder Mixes

Asphalt Content (%)	Resilient Modulus (ksi)	Binder Structural Layer Coefficient, a_2	Structural Number, SN	W_{18} (ESALs)
3.9	207	0.30	3.3	1.69×10^6
4.4	304	0.37	3.6	2.87×10^6
4.9	172	0.26	3.1	1.23×10^6
5.4	93	0.17	2.8	0.56×10^6

As expected, the optimum asphalt content determined from the AASHTO analyses provided the same results as the resilient modulus analyses. This was obvious since the W_{18} is directly related to the resilient modulus of the mix. The AASHTO analyses predicted pavement lives much greater at the optimum asphalt content than the surrogate fatigue model and the permanent deformation model discussed earlier.

Comparison of resilient moduli for the recycled mixes with hot-mix standard H-binders and hot-mix large stone binder mixes tested at NCSU showed that the recycled mixes exhibited stiffness values approaching those of the non-recycled hot-mix binders. The resilient modulus of the CMS-2 recycled mix at 4.6% asphalt content was 81% to 92% of the average hot-mix H-binder resilient modulus and 73% to 81% of the large stone binder mix resilient moduli at 20^0 C.

The resilient modulus of the HFRA recycled mix at 4.4% asphalt content was less than one-half of the hot-mix H-binder resilient modulus and about one-third of the large stone binder mix resilient modulus at 20^0 C. Table 5.11 shows the comparisons of the maximum resilient moduli of the recycled mixes with the average H-type binder.

Table 5.11 Resilient Modulus Values for Cold Recycled Mixes and H-type Binder Mixes

Binder Mix Type	H-Binder Mix	Large Stone Binder Mix	CMS-2 Recycled Mix	HFRA Recycled Mix
Resilient Modulus @ 22^0 C (ksi)	800 - 900	900 - 1,000	733	304

These comparisons reflect the strength contribution of the RAP to the recycled mixtures. The aged condition of the RAP when rejuvenated with the stiffer CMS-2 recycling agent provided a mix strength near that of the unaged hot-mix binders. The HFRA recycled mix exhibited a resilient modulus which was much less than those of the hot-mix binders. Because the HFRA was softer than the CMS-2, the lower stiffness of the HFRA was considered to be a weak link in the strength of these particular mixes.

CHAPTER 6

RECOMMENDED METHOD FOR COLD-RECYCLED MIX DESIGN

The results of this research indicated that asphalt pavements which have exhausted their service life are not necessarily due to highly aged binder. Factors other than asphalt oxidation may have contributed to the pavement failure, such as inadequate underlying layers or traffic loads and levels far in excess of those anticipated in the design. A highly damaged pavement may be an ideal candidate for recycling without the need for rejuvenation. The amounts of recycling agent necessary in cases such as these are limited to adequate coating and binding of the loose materials with associated blending from the chemical reactions which occur during the softening process. If the RAP was originally designed as a surface course, then it might logically contain a higher asphalt content than is required for a binder layer due to the larger total surface area of a smaller aggregate gradation relative to a binder layer gradation. This would account for seemingly low emulsion requirements for recycling of RAP containing asphalt that is not highly oxidized. As this research indicates, the amount of RAP in the final recycled mixture may be limited by the gradation requirements for the given layer, not necessarily the viscosity of the RAP asphalt.

6.1 Design Methodology

The first step in cold recycling of asphalt pavements is characterization of the RAP materials. This characterization includes aggregate gradation, binder characterization, and determination of asphalt content. Recent advances in asphalt binder characterization as a result of the Strategic Highway Research Program (SHRP) have improved the ability to predict the future performance of the recycled RAP asphalt and to allow for adjustments in the rutting and fatigue characteristics of the recycled mixtures.

The gradation of the RAP aggregate dictates the virgin aggregate requirements for a given pavement layer and, as in this project, can be the determining factor in RAP content of the recycled mix. Determining the asphalt content of the RAP is a key to estimating the recycling agent demand of the recycled mix. Initial recycling agent content is the difference between the estimated asphalt demand of the mix and the asphalt contribution of the RAP to the recycled mix.

After characterization of the RAP, the next step in the design process is determination of virgin aggregate supply, the development of a final gradation and the selection of a recycling agent. Virgin aggregate should meet the requirements of any pavement design standard, including angularity, toughness, durability, soundness and gradation. The stockpile selected should provide consistency of gradation to ensure that the aggregate blend meets the criteria for the intended pavement layer.

Characterization of the recycling agent residues provides insight into the reasonable boundaries of recycling agent content of the mix. The amount of an extremely soft residue in a mix will be less than the allowable amount of a stiffer residue. Because any unblended recycling agent residue will function as a weak link in the strength of the recycled pavement, optimum contents will probably be considerably lower than the upper limit determined by the binder blending chart. It is imperative that such considerations be made during the design process.

After material selection is complete, optimum asphalt content is determined by mix analysis. The mix analysis protocol should be based on characterization of the RAP asphalt in the reclaimed and PAV-aged condition. For instance, if the RAP asphalt does not exhibit a propensity for fatigue problems as determined by SUPERPAVE™ binder analysis, then mixture stiffness and rutting resistance should be considered major design criteria. Even if fatigue failure is a major concern, the mix stiffness is a major variable in tensile strains and initial cracking as determined from multi-layered elastic or visco-elastic analyses.

The general procedure for cold-recycled pavement design is:

1. Characterize the RAP;
2. Characterize the prospective recycling agents;
3. Develop G*sin(δ) and G*/sin(δ) blending charts (for each prospective recycling agent);
4. Determine the new aggregate gradation requirements and the allowable RAP content;
5. Estimate recycling agent content of trial mixes;
6. Prepare test specimens;
7. Perform mechanical tests on samples to determine the optimum asphalt content of the recycled mix;
8. Analyze the results.

6.1.1 RAP Characterization

The characterization of RAP includes determination of the asphalt content, aggregate gradation, and characterization of the extracted RAP asphalt. The elements of this step are:

- Extract binder from RAP samples in accordance with *ASTM D2172 (Quantitative Extraction of Bitumen form Bituminous Paving Mixtures)*;
- Determine the RAP aggregate gradation in accordance with *ASTM C136 (Sieve Analysis of Fine and Coarse Aggregates)*;
- Determine the specific gravity of the RAP coarse aggregates in accordance with *ASTM C127 (Standard Test Method for Specific Gravity and Absorption of Coarse Aggregates)*;
- Determine the specific gravity of the RAP fine aggregates in accordance with *ASTM C128 (Standard Test Method for Specific Gravity and Absorption of Fine Aggregates)*;
- Determine the RAP maximum theoretical specific gravity in accordance with *ASTM D2041 (Test Method for Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures)*;
- Determine the RAP asphalt content by weight of mix;

- Determine $G^*/\sin(\delta)$ of the reclaimed (unaged) RAP asphalt at 64^0 C (or the appropriate PG high test temperature);
- PAV-age the residue in accordance with *AASHTO PP1 (Accelerated Aging of Asphalt Binder Using Pressure Aging Vessel)* (omit RTFO procedures);
- Determine $G^*\sin(\delta)$ of the PAV-aged RAP asphalt at 28^0 C (or the appropriate PG intermediate test temperature);

6.1.2 Recycling Agent Characterization

The procedure for recycling agent characterization is similar to the RAP asphalt characterization. This step includes:

- Obtain recycling agent residue in accordance with *ASTM D244 (Standard Test Methods for Emulsified Asphalts)*;
- Determine $G^*/\sin(\delta)$ of the unaged residue at 64^0 C (or the appropriate PG high test temperature);
- PAV-age the residue in accordance with *AASHTO PP1 (Accelerated Aging of Asphalt Binder Using Pressure Aging Vessel)* (omit RTFO aging and testing procedures);
- Determine $G^*\sin(\delta)$ of the PAV-aged residue at 28^0 C (or the appropriate PG intermediate test temperature);

6.1.3 Develop Binder Blending Charts

With the RAP asphalt and recycling agent residue DSR data, the binder blending charts are developed using the following steps:

- Plot $G^*\sin(\delta)$ of the PAV-aged RAP asphalt corresponding to 0% Recycling Agent on the blend chart (Figure 6.1);
- Plot $G^*\sin(\delta)$ of the PAV-aged recycling agent residue corresponding to 100% recycling agent on the blend chart;
- Connect the points with a straight line;

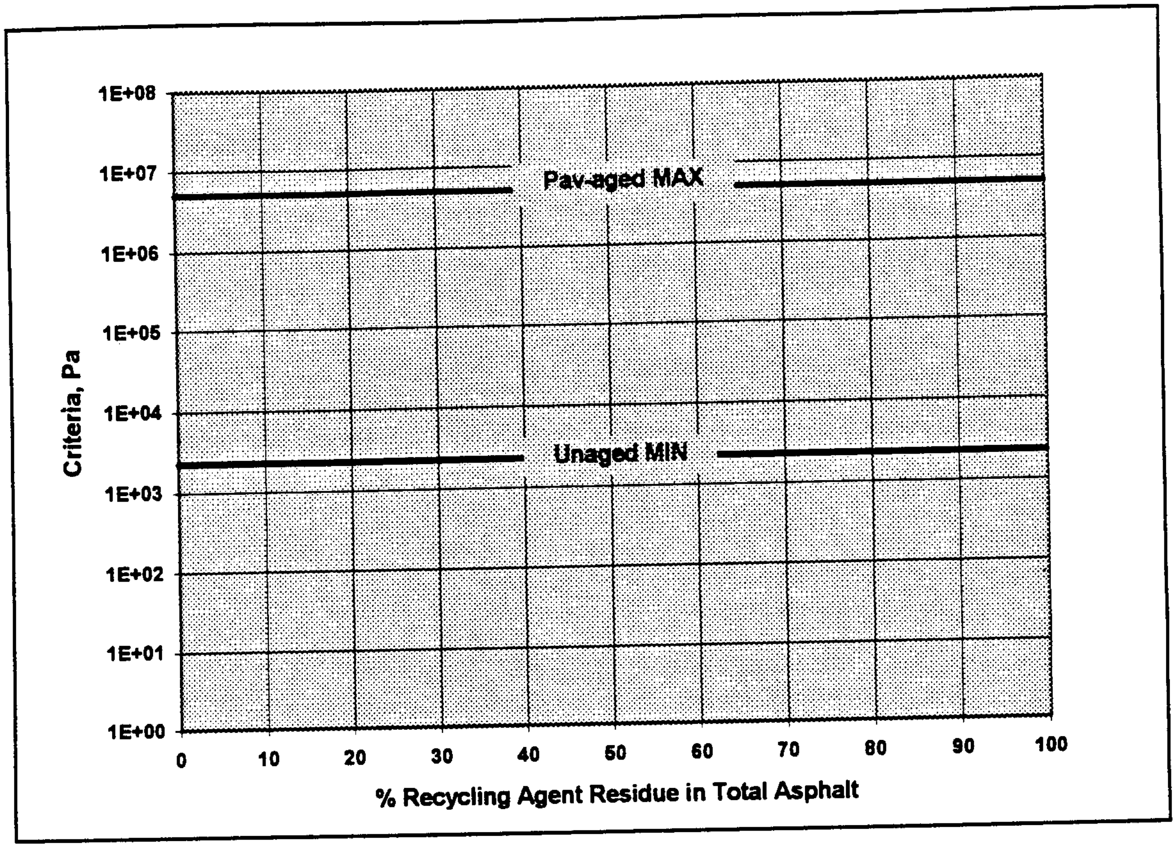

Figure 6.1 Binder Blending Chart

- The intersection of the plotted line with the 5000 kPa line is the theoretical lower limit of the recycling agent residue content in the binder blend of the recycled mix;
- Plot $G^*/\sin(\delta)$ of the unaged RAP asphalt corresponding to 0% recycling agent on the blend chart;
- Plot $G^*/\sin(\delta)$ of the unaged recycling agent residue corresponding to 100% recycling agent on the blend chart;
- Connect the points with a straight line;
- The intersection of the plotted line with the 2.2 kPa line is the theoretical upper limit of the recycling agent residue content in the binder blend of the recycled mix;
- The design window for the blending of the RAP asphalt with the recycling agent residue is bound by the lower limit (based on the PAV-aged condition) and the upper limit (based on the unaged condition).

6.1.4 Gradation Requirement of Virgin (New) Aggregate

The virgin aggregate requirement of the recycled mix is dependent on the RAP aggregate gradation, the required gradation for the pavement binder layer, and the upper and lower bounds of the binder blending chart. This is accomplished as follows:

- Using standard methods of aggregate blending, determine the gradation requirements and stockpile of the new aggregate;
- Using Equation 6.1 and an assumed initial asphalt content of 4.5% by weight of mix, determine the recycling agent residue content of the total asphalt of the mix.

$$\boldsymbol{Per} = \frac{[(\boldsymbol{Pac})x(100)]-[(100-\boldsymbol{Pac})x\dfrac{(\boldsymbol{Prap})}{(100-\boldsymbol{Pb})}x(\boldsymbol{Pb})]}{\boldsymbol{Pac}} \tag{6.1}$$

where ***Per*** = the percent of recycling agent residue in the total asphalt content of the recycled mix (whole number percent)

Prap = percent of aggregate from RAP (whole number percent)

Pac = asphalt content of the trial mix (4.5%)

Pb = asphalt content in of RAP (whole number percent)

Per for ***Pac*** = 4.5% should fall within the upper and lower limits of the blending chart or the percentage RAP should be altered so that the estimated ***Per*** is within the theoretical limits. If ***Per*** is less than the lower limit of the allowable recycling agent residue, then either a softer recycling agent should be considered or the gradation should be adjusted using a lower RAP content or both. If ***Per*** is larger than the upper limit of the blending chart, then a stiffer recycling agent should be considered or the gradation should be adjusted, if possible, using a higher RAP content or both.

- Determine the specific gravity of the new aggregate in accordance with *ASTM C127 (Standard Test Method for Specific Gravity and Absorption of Coarse Aggregate)*;
- Determine the specific gravity of the new aggregate in accordance with *ASTM C128 (Standard Test Method for Specific Gravity and Absorption of Fine Aggregate)*;

6.1.5 Trial Mix Asphalt Contents

The asphalt contents of the trial mix specimens are varied by increasing or decreasing the amount of recycling agent added to the mixes.

- Trial mix specimens should be made with 3.5%, 4.0%, 4.5%, 5.0%, and 5.5% asphalt contents. This equates to an initial trial asphalt content of 4.5% and additional trial asphalt contents of 4.5 ± 0.5% and 4.5 ± 1.0%.
- The amount of recycling agent emulsion to be added to the mix at a desired asphalt content is determined by the formula:

$$\boldsymbol{Memul} = \frac{[(\boldsymbol{Mmix}) x (\frac{\boldsymbol{Per}}{\mathbf{100}}) x (\frac{\boldsymbol{Pac}}{\mathbf{100}})]}{\boldsymbol{R}} \qquad (6.2)$$

where ***Memul*** = mass of emulsion in mixture
Mmix = mass of mix
Per = the percent of recycling agent residue in the total asphalt content of the recycled mix (whole number percent)
Pac = asphalt content of the trial mix (3.5%, 4.0%, 4.5%, 5.0%, 5.5%)
R = the decimal bitumen content of the emulsion (approximately 0.65)

6.1.6 Recycled Mix Specimen Preparation

The SUPERPAVE™ Gyratory Compaction method which produces 150-mm diameter specimens allows for adjustment of specimen height and the specific gravity of the compacted specimen. Adjusting the height of the specimen allows for compaction to a desired air void content which can be held nearly constant for all specimens to be tested. Determination of the resilient modulus of a sample at an in-service air void content of 4% provides a more consistent method of comparison of mixture properties. Variations in the RAP used in each specimen theoretically have less of an effect on the larger 150-mm diameter specimens than on the 100-mm diameter GTM specimens.

Once the volumetric parameters have been satisfied, specimens should be prepared and cured in accordance with the method below:

1. Weigh-out the necessary portions of RAP and virgin aggregate in separate pans, then place the virgin aggregate into a mixing bowl. Add water to the virgin aggregate equal to 4% of the weight of the aggregate and hand mix making certain all aggregate has been moistened. Let the water and aggregate stand for 5 minutes.
2. Pour the recycling agent into the mixing bowl and hand stir making certain that all of the virgin aggregate is well coated. Next, pour the RAP into the mixing bowl and hand stir for 30 seconds. After hand stirring, mix the materials in a mechanical mixer for 60 seconds. Remove the mixing bowl from the mixer and hand stir for 30 seconds to make certain that the recycling agent and smaller RAP particles are not sticking to the bottom and sides of the mixing bowl and are free to mix with the larger particles. Mix once more in the mechanical mixer for 30 seconds.
3. Place the mix into a pan and place the pan in a forced draft oven at 60^0 C (140^0 F) for 1 hour. After one hour, remove the mix from the oven and immediately place the mix into a 150-mm (6-inch) SUPERPAVE™ gyratory compaction mold by spooning to avoid segregation of the particles.
4. Compact the mixture to a height of approximately 63.5 mm (2.5 inches) and 4% air voids using the SUPERPAVE™ gyratory compactor in accordance with standard SUPERPAVE™ compaction specifications. Allow the samples to cool to room temperature in the mold overnight.
5. Extrude the samples from the mold and allow them to cure at room temperature for 24 hours. Place the samples in a forced draft oven and allow them to cure at 60^0 C (140^0 F) for 72 hours. Remove the samples and allow them to cool to room temperature overnight. Conduct resilient modulus tests and indirect tensile tests on one-half of the oven-cured samples.
6. Cure the remaining samples for 4 hours-per-day for 10 days under an infra-red lamp adjusted to a height so that the surface temperatures of the samples are 60^0 C (140^0

F). Make certain to turn the samples each day to ensure even curing throughout the sample. After the 10th curing day, allow the samples to cool to room temperature overnight. Conduct resilient modulus and indirect tensile tests on the infra-red lamp cured samples.

6.1.7 Mechanical Testing of Mix Specimens

For design purposes, the resilient modulus of the mixture is used in pavement structural design, such as Asphalt Institute Damage Analysis [15] or AASHTO Design of Pavement Structures [16]. For purpose of optimum mix selection only, the indirect tensile test at was shown to provide adequate results. The agency or designer responsible for the cold-recycled mix design must determine the depth of the mix analysis required for the particular project under consideration.

The resilient modulus testing should be conducted in accordance with *ASTM D4123 (Test Method for Indirect Tensile Test for Resilient Modulus of Bituminous Mixtures)*. The results of the resilient modulus tests should be used to determine the mix stiffness for optimum mix selection and structural layer design. It was found that the results of the indirect tensile test tend to reflect the results from the resilient modulus tests in determinations of optimum asphalt content. Therefore, the indirect tensile test using the Geotest Instrument Group Marshall Stability Machine at 20^0 C is adequate for optimum mix selection. However, resilient modulus data which is necessary for structural layer design is not provided by the indirect tensile test.

6.1.8 Analysis of Test Data

The resilient modulus of a mixture after oven curing and infra-red lamp curing is a measure of the ultimate mix stiffness. However, for structural design purposes, the resilient modulus after 72 hours in the forced draft oven will more closely predict the pavement stiffness at the time the recycled pavement is placed into service. Engineering judgment must be exercised in determining which modulus value is most appropriate for structural design.

This would include the anticipated traffic loadings within the 10-day curing period following the final compaction of the binder layer. Furthermore, if a structural analysis is not contemplated, then the indirect tensile test was shown to be a good indicator of optimum asphalt content in mix design.

6.2 Use of Data in Pavement Performance Prediction Models

The pavement structural design model used is dependent upon agency or user preference. The Asphalt Institute permanent deformation model and the A-003 fatigue model provided more conservative supply ESAL estimates than the AASHTO method for the materials and assumptions used in this study. Other models like the Asphalt Institute fatigue model [15] may also be used to determine the optimum mix selection and for purposes of structural layer design.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

The cold-recycled mixes developed in this project exhibited properties which met or exceeded specifications for recycled binder layers. The RAP material had desirable characteristics for recycling. The aggregate gradation was easily corrected for the binder layer gradation requirement in North Carolina. The RAP asphalt was stiff and quite elastic but was not susceptible to severe aged-fatigue failure.

The higher the percentage of RAP used in a recycled mixture, the more economically beneficial the option of cold-recycling becomes in comparison to new pavement construction. For the materials used in this research, increasing RAP contents improved mixture properties. This is due to the characteristics of the RAP material and is not a general statement about recycled mixes. For this reason, material characterization of RAP is an essential first step in cold-recycled mix design.

The percentage of RAP used in a recycled mix is dependent on the aggregate gradation of the RAP, the gradation requirements of the recycled mixture, the binder characteristics of the RAP asphalt, and the binder characteristics of the recycling agents. DSR tests on the RAP asphalt indicated that fatigue was not a critical failure mode and therefore, gradation requirements dictated the maximum amount of RAP material which could be incorporated into the mix. It was determined that using virgin aggregate from a # 6 stockpile provided the specified binder layer aggregate gradation while utilizing the highest percentage of RAP. The mixes in this project contained 75% RAP and 25% virgin aggregate.

The recycling agents considered in this research were CMS-2 and HFRA. DSR tests on the residues indicated that the HFRA was much softer than the CMS-2. While exhibiting fatigue resistant properties comparable to the HFRA mixes, the CMS-2 recycled mixtures were superior in strength and rutting resistance.

The binder blending chart developed in this study proved to be a reliable design tool. It is valuable in determining the maximum allowable RAP in the recycled mix and in establishing an acceptable range of recycling agent residue content of the total binder in the recycled mix.

7.1 Conclusions

Some of the conclusions drawn from this project were:

1. DSR testing of the reclaimed binder and the recycling agent residues and the development of the binder blending charts provided a reliable acceptable range of recycling agent contents in the total asphalt of the recycled mix.
2. The maximum allowable RAP content of the aggregate blend was determined to be 75% and was based on aggregate gradation limitations specified for pavement binder layers in North Carolina.
3. For binder layer mixes, an initial asphalt content of 4.5% was an adequate prediction of initial asphalt content of the recycled mix.
4. The resilient modulus and indirect tensile strength tests were valuable in determining the optimum asphalt contents as long as the binder mix contents remained within the acceptable range of the binder blending charts.
5. The SUPERPAVE shear test system supported the results of the resilient modulus, indirect tensile strength and DSR tests in selection of optimum asphalt content of the recycled mix.
6. The CMS-2 recycling agent yielded stronger recycled mixes than the HFRA recycling agent. The ultimate resilient modulus of the CMS-2 recycled mixes was 733 ksi. The ultimate strength of the HFRA recycled mixes was 304 ksi.
7. The AASHTO structural layer coefficients of the recycled mixes in this project were between 0.28 and 0.45. These coefficients were within the ranges reported by other researchers in the literature on cold-recycled mixtures [2].
8. The CMS-2 recycled mixes were as much as 90% as strong as the non-recycled hot-mix binder mixtures based on resilient modulus tests. The HFRA recycled mixes had strengths about one-third those of the non-recycled hot-mix binders mixtures.

9. Much of the strength of the recycled mixture was provided by the RAP.

7.2 Recommendations

Some recommendations for cold-recycled mix design include:

1. Though the DSR binder tests provided reliable predictions of binder mixture parameters, the results of the binder mixture tests with the DSR should not be relied upon as the sole means of mixture design. The compatibility of the materials should be considered in the mix design.
2. Lab samples should be compacted using the SUPERPAVE™ Gyratory Compactor. Resilient modulus tests and indirect tensile tests should be performed on samples compacted to a height of approximately 63.5 mm (2.5 in) at 4% air voids.
3. Addition of recycling agent to enhance workability in the field should be avoided. Reductions in the strengths of the recycled mixtures at asphalt contents even 0.5% higher than the optimum could be considerable.
4. The mix properties of other recycling agents should be investigated along with the effects of additives such as hydrated lime and cement on mixture properties.
5. The actual effect of fines on the performance of different recycling agents should be investigated.
6. In order to simplify the design process, a correlation between indirect tensile strength and resilient modulus may be established, especially for those who do not need modulus values for structural layer design.
7. Investigations into reliable means of optimizing cold-recycled mix design using the SUPERPAVE™ Gyratory Compactor by adapting SUPERPAVE™ volumetric design procedures should be considered.

7.3 Implementation

The methodology developed in this study can be effectively applied and implemented for design and evaluation of cold-recycled mix design. The DSR tests along with the SUPERPAVE Gyratory Compactor can be used for these mix designs. The application of this methodology to several projects involving different RAP materials would provide valuable experience which may result in further refinement of the cold-recycled mix design practice in North Carolina.

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