FINAL REPORT

RESEARCH PROJECT 1999-03

Delamination and Shoving of Asphalt Concrete Layers Containing Baghouse Fines

by

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May 15, 2000

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Technical Report Documentation Page

1. Report No. FHWA/NC/2002-011	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Delamination and Shoving of Asphalt Concrete Layers Containing Baghouse Fines		5. Report Date May 15, 2000 6. Performing Organization Code
7. Author(s) A. A. Tayebali, M. B. Kulkarni, H. F. Waller		8. Performing Organization Report No.
9. Performing Organization Name and Address Center for Transportation Engineering Studies Department of Civil Engineering North Carolina State University Raleigh, NC 27695-7908		10. Work Unit No. (TRAIS) 11. Contract or Grant No.
12. Sponsoring Agency Name and Address North Carolina Department of Transportation Research and Analysis Group 1 South Wilmington Street Raleigh, NC 27601		13. Type of Report and Period Covered Final Report 6/1/1998 – 12/31/1999 14. Sponsoring Agency Code 1999-03
15. Supplementary Notes		

16. Abstract

This study investigated the cause(s) of the excessive delamination and shoving distresses observed in NCDOT Division 13. Two possible causes of these distresses were identified to be: 1) the intermittent purging of the baghouse fines in in-situ asphalt mixtures, and 2) improper selection and/or application of the tack coat, i.e. the use of CRS-2 emulsion versus the PG64-22 asphalt binder.

Laboratory performance test results on field cores and asphalt mixtures showed that baghouse fines had a stiffening effect on mixtures and these mixtures were more resistant to rutting. However, mixtures containing baghouse fines were found to be moisture sensitive with tensile stress ratios below that specified by NCDOT. The performance test results for the evaluation of the bond strength of the in-situ cores, showed that the PG64-22 binder used as tack coat provided a better interfacial bonding compared to the CRS-2 emulsion.

Results of this investigation suggest that the delamination and shoving distresses in NCDOT Division 13 could be attributed to the combined effect of intermittent purging of baghouse fines in asphalt mixtures and the use of CRS-2 emulsion as tack coat. Due to intermittent purging of baghouse fines, some in-situ mixtures may contain significantly higher proportion of baghouse fines compared to regular fines. Although the NCDOT JMF requires use of an anti-strip additive, the dosage does not appear to be sufficient to offset the increased moisture damage leading to in-situ mixture deterioration and, consequently, loss of strength and stability. Once the moisture damaged mixture is susceptible to shoving under traffic loading, the CRS-2 emulsion may not provide the tacking strength necessary for the surface layer to remain bonded to the lower layer, hence, leading to delamination.

17. Key Words Delamination, shoving, baghouse fines, moisture sensitivity, repeated shear, frequency sweep, rutting, dynamic shear modulus	18. Distribution Statement		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 151	22. Price

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

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Acknowledgments

The authors extend their sincere appreciation to the authorities of the North Carolina Department of Transportation (NCDOT) for funding this project and the assistance provided by the Federal Highway Administration, Turner Fairbank Research Facility. This research investigation was based on a research project need statement submitted to NCDOT by Mr. R. M. Crisp, P.E.

This research endeavor would not have been possible without the support of the numerous NCDOT and FHWA personals. The authors would like to thank Mr. R. M. Crisp, P.E. and the following members of the Research Project Steering Committee: Mr. G. Gibson, P.E., Mr. C. Jones, P.E., Mr. J. E. Grady, Jr., P.E., Mr. C. A. Clippard, P.E., Mr. J. E. Cowsert, P.E., Mr. J. E. Travis, P.E., Mr. J. Trogdon, P.E., Mr. M. P. Strong, P.E., and Dr. M. Biswas, P.E.; for their continuous assistance in this study. Thanks are also due to Mr. Tom Harmon, P.E. and Dr. Pedro Romero of FHWA Turner Fairbank Research Facility.

Executive Summary

This study investigated the cause(s) of the excessive delamination and shoving distresses observed in NCDOT Division 13. Two possible causes of these distresses were identified to be: 1) intermittent purging of the baghouse fines in in-situ asphalt mixtures, and 2) improper selection and/or application of the tack coat, i.e. use of CRS-2 emulsion versus the PG64-22 asphalt binder.

Cores and raw materials were obtained from pavement sections in Buncombe and Rutherford counties for forensic analysis of the in-situ materials and to evaluate the laboratory performance of mixtures containing baghouse fines. The core samples obtained were subjected to the volumetric and stability analysis, and laboratory performance testing to evaluate the tack coat bond strength of the CRS-2 emulsion versus the PG64-22 asphalt binder.

The results of the gradation, volumetric and stability analysis, indicated that the in-situ asphalt mixtures used in Buncombe and Rutherford counties were within acceptable NCDOT mixture design specifications and should have performed well in-situ under normal traffic loading. It was originally hypothesized that the one of the contributory factor to the delamination and shoving was the intermittent purging of baghouse fines in the field asphalt mixes. Results of the gradation analysis using the particle analyzer showed that the baghouse fines had similar or in some cases coarser gradation as compared to the regular mineral filler used in these respective counties. Laboratory performance test results mixes indicated that:

1. Baghouse fines have a stiffening effect on mixtures from both counties;
2. Mixtures containing baghouse fines are more resistant to rutting as compared to mixtures without baghouse fines;
3. Respective mixtures from both counties show similar dynamic shear stiffness and rutting characteristics.

The APA test results showed that the accumulated rut depths for mixtures from Buncombe and Rutherford counties were approximately 6.15-mm (1/4-inch) and 12.5-mm (1/2-inch), respectively, for both mixtures with and without baghouse fines. Although, these

rut depths suggest excessive rutting susceptibility for mixes based on the NCDOT specification, it confirmed findings based on other tests that indicated that the performance of mixtures with and without baghouse fines are very similar. However, the modified AASHTO T283 test clearly indicated that the mixtures containing baghouse fines were moisture sensitive as compared to the mixtures containing regular mineral filler even though, an anti-strip additive was used for both mixtures. The performance test results for the evaluation of the bond strength of the in-situ cores, showed that the PG64-22 binder used as tack coat provided a better interfacial bond than CRS-2 emulsion.

Based on the results of this investigation, it is the opinion of the authors that the intermittent purging of baghouse fines in combination with the use of CRS-2 emulsion, could be a contributory factor in the delamination and shoving distress observed in NCDOT Division 13. Due to intermittent purging of baghouse fines, some in-situ mixtures may contain very high proportion of baghouse fines in relation to regular fines. Although the NCDOT JMF requires use of an anti-strip additive, the dosage does not appear to be adequate to offset the increased moisture-induced damage due to baghouse fines, leading to in-situ mixture deterioration and, consequently, loss of strength and stability. Once the moisture damaged mixture is susceptible to shoving under traffic loading, the CRS-2 emulsion may not provide the tacking strength necessary for the surface layer to remain bonded to the lower layer, hence, leading to delamination. In Rutherford County where some pavement sections may contain relatively higher amount of baghouse fines due to intermittent purging, the PG64-22 binder used as tack coat may be providing sufficient bonding which may prevent asphalt layer from delaminating even though mixtures may undergo slight moisture damage.

Based on the findings of this investigation, it is recommended that:

1. The introduction of baghouse fines in asphalt mixtures be metered rather than purged intermittently.
2. The amount of baghouse fines in relation to the amount of regular mineral filler should be restricted based on the tensile strength ratio to minimize the moisture damage in asphalt mixtures.
3. It is imperative that baghouse fines be used from the onset in the design of asphalt mixtures and development of job mix formula.

4. In cases where marginal or moisture sensitive materials are used for asphalt concrete or composite pavements, PG64-22 binder used as tack coat may provide superior bonding compared to CRS-2 emulsion.

Key Words: delamination, shoving, baghouse fines, moisture sensitivity, repeated shear, frequency sweep, rutting, dynamic shear modulus

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Notations

AC	asphalt cement
AFST	axial frequency sweep test
AI	Asphalt Institute
APA	Asphalt Pavement Analyzer
BFB	Buncombe County, field cores, rutting
BFG	Buncombe County, field cores, no rutting
BLW	Buncombe County, lab cores, with baghouse fines
BLWO	Buncombe County, lab cores, without baghouse fines
CBR	California bearing ratio
CRS-2	cationic rapid setting emulsion
DSR	dynamic shear rheometer
ESAL	equivalent single axle load
FSCH	frequency sweep test at constant height
GaLWT	Georgia loaded wheel tester
HDS	High density surface
HMA	hot mix asphalt
PRT	pavement rutting tester
PG	performance graded
RFB	Rutherford County, field cores, rutting
RFG	Rutherford County, field cores, no rutting
RFW	Rutherford County, lab cores, with baghouse fines
RFWO	Rutherford County, lab cores, without baghouse fines
RSCH	repeated shear constant height
SST	simple shear testing machine
SUPERPAVE	Superior performing pavements
VRAMP	uniaxial tension
wp	wheel-path
σ	axial stress
τ	shear stress
γ	shear strain
$\|G^*\|$	magnitude of dynamic shear modulus
δ	phase angle

1. BACKGROUND

1.1 Introduction

Asphalt pavements constitute 96 percent of the hard surfaced roads in the US. In terms of distance, approximately 2.2 million miles of roads have asphalt surfaces and approximately 91 percent of the 2 trillion annual vehicular miles of travel occur on these asphalt pavements. With ever increasing number of vehicles on the roads, the need for proper maintenance of the existing infrastructure cannot be overemphasized. Today, with a virtual saturation in the construction of newer roads, most of the work is primarily confined to maintenance and rehabilitation of existing pavements.

Pavement rehabilitation often involves an increase in the capacity of the pavements, whereas maintenance restores the pavement structurally and functionally to its original level. The structural soundness refers to the ability of the pavement to carry the design load, whereas the functional ability refers to the ride quality and safety. The source of functional and structural inadequacy can be either load associated or non-load associated. Distresses such as fatigue, rutting, potholes, etc. are load associated while bleeding, corrugations, depressions, raveling, thermal cracking, etc. are non-load-associated distresses.

For thick asphalt pavements, the construction is done in layers. Before paving a new hot mix asphalt (HMA) layer, the top surface of the existing layer is cleaned and a tack coat is applied to bond the new surface being paved to the underlying course. The tack coat consists of a light application of asphalt cement, usually asphalt emulsion or liquid asphalt. For optimal performance, it is important that the tack coat be thin and uniform, and 'breaks' just before the new HMA is laid [14]. For the pavement to be structurally and functionally sound, there should be proper interface bonding between the upper and lower AC (asphalt concrete) layers. Lack of interface bonding may lead to several premature distresses of which slippage cracking and distortion are the most prominent. The Asphalt Institute MS-16 [3] manual indicates that slippage cracks result from a lack of bond between the surface and the layer beneath. Distortion, a result of asphalt layer instability, can take a number of different forms such as shoving, pushing, corrugation, rutting, etc. Corrugation is a form of plastic movement typified by ripples across the asphalt surface. It occurs usually at intersections where there is acceleration or deceleration of vehicles. This distress is a functional failure of the pavement and can affect the ride quality and safety. The development of slippage cracks, crescent or half-moon shaped, is also a result of poor interfacial bond. In this distress, under the shearing action of the traffic, the asphalt mix moves laterally away from the rest of the surface. Some reasons for a lack of bonding between the asphalt layers are:

- Poor condition of the old pavement — presence of dust, oil, rubber, dirt, water or any other non-adhesive materials;
- Use of excessive or inadequate tack coat, or a non-uniform application of the same;
- Highly polished aggregate on existing surface which may be water sensitive and / or use of tack coat that may not be compatible with the polished aggregates;
- Use of mixture having a high sand content, especially with rounded particles;
- Use of improper construction technique and lack of proper degree of compaction of the AC layer.

The debonding or delamination could be caused by any one or a combination of any of the factors listed above. In addition to the above criteria, the following factors contribute equally: (a) improper consideration of temperature and field conditions, (b) excessive load repetitions and vehicular accelerations and, (c) very thin surface layer thickness. In practice, most of the delamination distresses can be attributed to either improper construction techniques or choice of inappropriate of tack coat.

This project involved an evaluation of some field sections undergoing similar distresses. It was found that pavement sections in Division 13 of the North Carolina Department of Transportation ("NCDOT") were experiencing large scale shoving and pushing of asphalt concrete mat. After preliminary investigations concerning the construction techniques and materials used, it was found that major factors likely to initiate the debonding process were: (a) improper selection and application of tack coat, (b) adverse effect of moisture on the tack coat, and (c) improper use of baghouse fines.

Another factor for that could accelerate debonding was the use of baghouse fines. The collected baghouse fines presently are purged into the AC mixtures intermittently rather than being stored in a silo and metered into the mix in a uniform, controlled manner as a mineral filler. Due to this, the job-mix gradation may be highly variable, and the mix stability and the volumetric properties may change from batch to batch or hour to hour. Those batches containing higher percentage of baghouse fines in the mineral filler may produce "doughy" or "tender" mixtures susceptible to pushing or shoving.

1.2 Objectives and scope

The objective of this study was to evaluate and identify the cause(s) of delamination (loss of bonding) and distortion (shoving and pushing) of asphalt concrete layers directly applicable to NCDOT Division 13. As indicated in the earlier section, there were a number of factors that affect debonding and distortion of mixtures. Two factors – tack coat and asphalt mixtures containing baghouse fines – were chosen for evaluation in this study.

For the pavement sections mentioned above, NCDOT used CRS-2 (Cationic Rapid Setting) emulsion as tack coat. The level of distresses in the sections tacked with CRS-2 emulsion was so high that in some sections NCDOT decided to consider using PG64-22 (Performance Graded) binder as an alternative to CRS-2 emulsion. The contribution of both, the emulsion and binder as tacking material was evaluated in this study. In addition, other factors such as construction technique used for placement and compaction of the hot mix asphalt were briefly evaluated to assure that these were not the major factors contributing to the distresses under investigation.

The specific work tasks included in this study were the following:

1. Evaluate the effect of the presence of baghouse fines on the mixture volumetric properties and stability.
2. Evaluate the effect of the presence of baghouse fines on mastic rheology.
3. Evaluate the effect of the type of tack coat used on the AC layer interface bond strength for mixtures containing baghouse fines.
4. Evaluate the effect of moisture on the stability of mixtures containing bag-house fines.

This study is divided into three specific tasks. The first task was aimed at verifying whether the mix in the field complied with the NCDOT specifications and to evaluate the characteristics of asphalt mixtures containing bag-house fines. Volumetric tests, gradation analysis, Marshall testing and laboratory performance testing (FSCH and RSCH) were done to evaluate the field mixture properties. In addition to the mix evaluation, the bag-house fines' gradation was evaluated using the particle analyzer. Two sets of laboratory specimens were prepared – the first set consisted of samples with bag-house fines and the other set consisted of samples made without bag-house fines. The laboratory specimens were then subjected to FSCH, RSCH, APA, and TSR performance tests to evaluate the differences in their properties caused due to bag-house fines. The second task consisted of evaluating the tack coat bond strength of field cores. These cores were subjected to FSCH, AFST, RSCH, and VRAMP tests. In the final task, mastics were prepared from the fines and tested using DSR to evaluate the contribution of fines to rheological properties of asphalt cement.

1.3 Significance

NCDOT Division 13 was experiencing severe distresses in the form of asphalt concrete layer delamination and distortion in the form of shoving and pushing of mat layers. If the causes of these distresses could be identified and remedied, it would result in better riding quality. Also, there would be a direct cost benefit to NCDOT in terms of a reduced repair and maintenance cost of existing distressed pavements, and will prolong the in-service life of the new or rehabilitated pavements. This research work provides a comparison of the effectiveness of emulsified asphalt versus a performance-graded binder used as a tack coat.

This report, first gives an overview of the mineral fillers and baghouse fines; and tack contribution and evaluation of its effectiveness. The research approach and methodology are discussed in Chapter 2. Chapter 3 deals with the selection of appropriate field sections for laboratory testing based on the results of the survey conducted. The evaluation of mechanical properties, volumetrics, Marshall stability results, and performance testing of laboratory mixes are presented in Chapter 4. Chapters 5 and 6 cover the evaluation of tack coat bond strength and rheological characterization of mastics using DSR, respectively. Chapter 7 deals with the results from the APA and moisture sensitivity tests. Finally, summary and conclusions are presented in Chapter 8.

1.4 Literature review

The probable causes for debonding chosen for investigation were improper use of baghouse fines; inadequacy of emulsion based tack coat and an adverse effect of water. This chapter presents a summary of the previous research done in this field and some of the recent developments.

1.4.1 Mineral filler and baghouses

1.4.1.1 Definition

The stone crushing industry has problems in marketing aggregates with large amount of material passing 75 microns. Typically, 50-percent of fine aggregate production is washed screenings and out of which 20-percent is the mineral filler. Considerable research has been done on the mineral filler and its effect on asphalt concrete mixtures since 1900. Mineral filler is defined as finely divided mineral matter and includes material such as rock dust, slag dust, portland cement, hydrated lime and loess. Baghouse fines can also be considered as mineral filler; but since they are a recent development, not much research has been done on them. The difference between the traditionally used mineral fillers and baghouse fines is that baghouse fines contain a much large percentage of very fine 10-micron material. As a first step, the precise definition of mineral filler should state [21]:

- What is filled?
- What does filling?
- Why the filling is done?

One of the officially accepted definitions used by Asphalt Institute is [13]: *"The mineral filler shall consist of limestone dust, portland cement or other suitable (inert) mineral matter. It shall be thoroughly dry and free of lumps consisting of aggregation of finer particles."* It is a requirement that the mineral filler added to mineral aggregate passes the 600-micron sieve completely and at least 65-percent of it passes the 75-micron sieve. However, such a definition is inadequate because the maximum permissible size of 600 micron could overlap with the fine aggregate sizes. This not only encroaches on the scope of fine aggregate definition but also makes the aggregate proportioning more cumbersome. The other proposed definition is [21]: *"Filler is that portion of the mineral aggregate generally* *passing number 200 sieve and occupying void spaces between the coarser aggregate particles in order to reduce the size of these voids and increase the density and stability of the mass."* Thus, void space in the coarse mineral aggregate is filled with mineral particles passing the number 200 sieve because it is desirable to make the size of these voids smaller and the density of the mass greater. Another proposed definition is: *"Filler is the mineral material that is in colloidal suspension in the asphalt cement and results in a cement with stiffer consistency."* Thus, asphalt cement is filled with a colloidal mineral matter because it is desirable to increase its viscosity.

1.4.1.2 Sources of mineral filler [15]

The process of HMA manufacture involves heating of aggregates. The heating is necessary to remove the moisture and for proper mixing and compaction temperatures. Heating on such a large scale is done by subjecting the aggregates to a draft of hot air under pressure. During the process of heating, due to the high pressure and velocity of the air draft, the air currents pick up a large amount of extremely fine particulate matter. The amount of exhaust dust depends primarily on the size and weight of the material being dried and the velocity of air. In case of drum mixers, the amount of airborne dust is also affected by the location of asphalt cement inlet. The closer the asphalt cement inlet to the flame, the lower is the amount of dust that gets airborne. These particles are fine enough to be carried by a current of air or inhaled by humans and are, therefore, a source of significant air pollution. The exposure of these fines to combustible fuels makes them more hazardous. Hence, the Environmental Protection Agency has established strict regulations for the particulate emissions from drying units in batch and drum mix facilities. All new plants are restricted to 0.04 grains of particulate matter per standard cubic foot of exhaust gas. As it is not possible to do away with the process of air drying, the only way was to control the exhaust from such plants is by installing dust collectors. Primarily, there are three types of dust collectors found in practice:

Primary dry collectors

The method of dust collection adopted by dry collectors is by causing a sudden drop in the velocity of exhaust gases. There are two subtypes of such dust collectors: the knockout box and the centrifugal box used mostly by the batch plants. The dust collectors are placed at the end of the dryer where the exhaust gases leave and enter the stack. The knockout box consists of an expansion chamber where there is a significant enlargement of the cross-sectional area causing a drop in the velocity. The reduction of velocity causes the heavier particles to settle down. The centrifugal box collector consists of a tangential inlet which forces the exhaust on the outer wall slowing down the flow and causing the heavier dust to settle. The efficiency of these collectors is 60 to 85-percent.

Primary wet collectors

The wet collectors cause the airborne dust to fall out from the stream of exhaust by increasing the weight of the particles. This is achieved by spraying the particles with small water droplets. The heavily loaded dust particles are carried into a separator and are made to follow a helical path. The particles settle down on the walls due to centrifugal action and flow down into settling ponds. The deposited fines are in the form of sludge and are disposed off differently (primarily wasted). The supernatant water is recycled back into the system. Use of wet collector results in a loss of fines. If the amount of dust collected is large, then there is a significant change in the proposed gradation. For gradations where external mineral filler is added, the use of wet collectors is not recommended.

Secondary fabric filters (baghouses)

This is the most efficient way of removal of dust from exhaust gases. The efficiency of these types of filters is about 99-percent. The exhaust gases are made to pass through a filter cloth that traps the dust on one side but allows clean air to pass through. The dust collected is called as baghouse fines. The bags are made up of felted nylon fabric, which is temperature resistant up to 450^{o}F and can withstand flexing cycles of the cleaning process. The bags are stitched in the form of a cylinder closed at one end and are fixed to metal frames to prevent them from collapsing. The gases are made to pass through the filter bag and the dust is trapped on the outside letting the clean air out. Continual deposition of dust on the bag causes the formation of dust cake, which improves the efficiency of the system as more finer particles get trapped. If the dust cake becomes too thick, then it is removed by pulsing. The dust that falls off from these bags is either fed back into the HMA or wasted. The efficiency of these bags is dependent on the pressure drop that takes place across the walls of the bag. The lower the pressure drop the higher is the efficiency.

1.4.1.3 Significance of mineral filler

Filler has dual functionality. It can either be a part of the aggregate structure or form mastic. This duality of function is peculiar to mineral filler, and this property distinctly separates it from the other aggregate fractions. A question may arise with regard to the dividing line at which the filler ceases to be a part of the mineral aggregate and becomes a part of the asphalt film, thereby changing the properties of the binder. It is obvious that many factors influence the location of such a dividing line e.g. aggregate gradation, asphalt content, air voids and their structure, shape and texture of mineral filler particles, etc. Inspite of knowing all this, it has not been possible, so far, to predict the exact performance of the filler and correlate the properties to the performance characteristics.

The properties of filler affect the rheology of the binders. It means that various binders which formerly could not meet the requirements for a particular type of specification can now qualify because of modification of some of their physical properties. This can be true otherwise also. Such modification of properties can have an effect on the performance of the mix. The effect is not restricted to the modification of performance, which includes moisture resistance, fatigue, permanent deformation, cracking, adhesion, but also affects different production phases such as mix design, aggregate drying, proportioning, mixing, and compaction. It has been observed [10] that some baghouse fines make the mixture "critical" (very sensitive to a slight increase or decrease of asphalt content) resulting in either flushing or raveling. In many cases, the amount of asphalt needs to be reduced to prevent a loss of stability or bleeding.

Commercial considerations such as non-acceptance of aggregates with large amount of fines resulting in stockpiling of a large amount of fine aggregates is also a significant factor contributing towards the study of mineral fillers. The extreme fineness of dust particles, especially baghouse fines, poses a big environmental hazard. Therefore, for safe disposal, it is necessary that alternative commercially viable uses are found out (most prominent use would be in the HMA industry itself).

1.4.1.4 Characterization of fillers [11]

Typically, satisfactory mineral filler should:

- Not have an adverse chemical reaction with bitumen.
- Not possess hydrophilic surfaces so that there is good adhesion between the binder and aggregates.
- Not be porous and have a high affinity for bitumen.
- Contain a particle size gradation close to a dense gradation.

In early times, the evaluation of mineral filler in bituminous mixes was dependent heavily on routine laboratory tests. These tests were mostly from soil mechanics and included liquid limit, plasticity index, cementation, shrinkage, and water-bitumen preferential test (to eliminate hydrophilic fillers). However, with more experience, the following tests are performed nowadays:

Particle size analysis [5]

There are different methods by which this can be performed. Various gradation analysis methods are summarized in Table 1. The following list enumerates the different properties related to the shape and texture.

1. Fineness Modulus (F.M.)

$$= \frac{\sum \left(\text{Cumulative \% retained at the 75, 50, 30, 20,10, 5, 3, 1 } \mu\text{m levels}\right)}{100}$$

$$= \frac{\sum_{i=a}^{h} \left[100 - \text{Cumulative \% passing the i}^{\text{th}}\ \mu\text{m level}\right]}{100} \qquad (1.1)$$

where $i \in (75, 50, 30, 20, 10, 5, 3, 1)$

The fineness modulus is a single value used to represent the fineness of a graded aggregate sample. The lower the value, the finer is the sample.

2. Coefficient of Uniformity (C_U)

$$= \frac{\left(\text{Maximum particle diameter corresponding to 60\% passing}\right)}{\left(\text{Maximum particle diameter corresponding to 10\% passing}\right)}$$

$$= \frac{(D_{60\%})}{(D_{10\%})} \tag{1.2}$$

The coefficient of uniformity (C_U) is a measurement of the range of particle sizes that are present in the sample. The greater the value of C_U, wider is the range of particle sizes.

3. Coefficient of Curvature (C_C)

$$= \frac{(\text{Particle diameter at 30\% passing})^2}{(\text{Particle diameter at 10\% passing}) \times (\text{Particle diameter at 60\% passing})}$$

$$= \frac{(D_{30\%})^2}{(D_{10\%}) \times (D_{60\%})} \tag{1.3}$$

It is an indicator of the shape of the gradation curve.

4. Skewness Indicator (σ_1)

$$= \frac{\text{Particle diameter corresponding to 84.13\% passing}}{\text{Particle diameter corresponding to 50\% passing}}$$

$$= \frac{\text{Maximum particle diameter corresponding to 84.13\% passing}}{\text{Median particle diameter}}$$

$$= \frac{(D_{84.13\%})}{(D_{50\%})} \tag{1.4}$$

"Sigma 1," σ_1, gives an indication of the skewness of the distribution of the individual percent fractions of gradation. If the particle sizes and frequencies in a sample were normally distributed among the median particle size, $D_{50\%}$, then σ_1 would be the standard deviation of the distribution. $D_{84.13\%}$ corresponds to one probit distance form $D_{50\%}$.

5. Skewness Indicator (σ_2)

$$= \frac{\text{Particle diameter corresponding to 50\% passing}}{\text{Particle diameter corresponding to 15.87\% passing}}$$

$$= \frac{\text{Median particle diameter}}{\text{Particle diameter corresponding to 15.87\% passing}}$$

$$= \frac{(D_{50\%})}{(D_{15.87\%})} \tag{1.5}$$

"Sigma 2," σ_2, gives an indication of the skewness of the distribution of the individual percent fractions of gradation. If the particle sizes and frequencies in a sample were normally distributed among the median particle size, $D_{50\%}$, then σ_2 would be the standard deviation of the distribution. $D_{15.87\%}$ corresponds to one probit distance form $D_{50\%}$.

6. Median Particle Size ($D_{50\%}$) in μm

   The median particle size ($D_{50\%}$) is calculated from the gradation curve; linear interpolation is used to find the size of the particle that corresponds to the cumulative 50% passing.

7. Specific Surface Area *(cm^2/cm^3)*

   The specific surface area is an indication of the fineness of the sample; the finer the sample, the higher surface area per unit volume it will have, which means a higher specific surface area.

8. Regression Model

   Using the values obtained from the calculations above, the expected performance of filler (the probability of filler being "good") is calculated using the full linear regression mode as shown in Equation 1.6. If the value of E is greater than 0.5 then the filler is considered good for mixes. The value of R^2 obtained is 0.70.

$$\begin{aligned} E\{Y\} &= 11.62697 - 0.04545 \times \%\text{ Rigden Voids} \\ &- 0.08871 \times \text{Stiffening power at 35\% filler by volume} + 0.54707 \times |\sigma_1 - \sigma_2| \\ &- 0.07684 \times C_U - 1.22693 \times \text{F.M.} - 0.00040 \times \text{Specific surface area} \end{aligned} \tag{1.6}$$

Rigden voids test [6]

This test enables the determination of the air voids in dry compacted filler powder. The apparatus consists of a cylindrical container and a ram of specified dimensions and weight. The filler is compacted by repeated dropping of the whole apparatus over a specified height. This method measures the volume of a unit weight of filler and is a standard method. The result is expressed in volume percentage, V_{fR}, of filler granules present in the observed bulk volume. Due to the dropping of the apparatus, this test poses a problem with fillers having fragile granules. This test was modified by Anderson to obtain the percent voids in a compacted sample of dry mineral filler. As will be discussed later, the volume of voids in a compacted sample of dry mineral filler consists of volume of binder in the mastic that is fixed and hence, unavailable to participate in the mix as a whole. When the percent voids in a compacted sample containing mineral filler is too high; not enough binder is free leading to excessively stiff mix susceptible to cracking. In cases where the voids are excessively low, implying a large amount of "free" binder, the mixes are more prone to rut or bleed.

Kerosene absorption test [6]

In this test, kerosene is gradually added to dry filler powder and is shaken in a dish until a ball is formed that can just take up all the filler granules. The compaction effort is delivered by the surface tension of the kerosene. It has been found that this method gives almost the same values for the voids (= volume of kerosene) as found by dry compaction. The following relationship has been found:

$$V_{fK} = 0.97 \times V_{fR} \tag{1.7}$$

1.4.1.5 Effect on binders

Filler–bitumen systems [21]

The definition of mineral filler is based on the fact that certain fine aggregate particles get suspended in the binder and modify the consistency of the binder. The modification results in altering the properties of the pavement, primarily through surface attraction or adsorption between the mineral surface and the bitumen. The suspension is called colloidal, veracity of which has still not been known. The suspended filler particle offers a large surface area with respect to its volume. This, in turn, results in the predominance of adsorption over other effects that may be associated with that particle. In a filler-bitumen system, the filler particle is surrounded by an adsorbed layer of bitumen as shown in Figure 1.1. This layer can be typically distributed in two parts, the inner, and the outer. The inner layer thickness T_1 represents the optimum film thickness or the layer under the maximum influence of adsorption. The outer layer of thickness, T_2, represents the additional bitumen influenced by adsorption. There exists an energy gradient, as shown by the darkly shaded region from the inner surface of the inner layer to the outer boundary of the outer layer. This gradient primarily represents the surface energy or the stiffening effect. It is primarily constant in the inner layer and decreases progressively in the outer layer. The presence of such a gradient can explain the stiffening of the binder on addition of the filler.

Rheological properties

The addition of filler to bitumen results in the modification of its properties. Properties such as viscosity, penetration, ductility, aging characteristics, etc, are modified, most of which relate to the performance of HMA in field. The proportioning of filler in the filler-bitumen mixes is done based on volume rather than on weight for a meaningful comparison between different fines. To be precise, it is expressed as the volumetric ratio of filler to binder – where the volume of binder is equal to the volume of filler plus the volume of asphalt. The mixing is carried out by heating both, the asphalt and filler, to 163^{o}C. The fines are added slowly in intervals and the entire procedure is completed in about 30 minutes. For the current project, the proportioning was carried out on weight basis instead of volumetric basis. This was done because the aim was to evaluate the effect of fines on the binder properties at a fixed percentage of fines proportioned based on weight in the AC mix. The following tests can be used to measure the various properties of the filler-bitumen mixes:

Penetration

Typically, this test is performed at temperature of 39.2, 60 and 77^{o}F. The detailed procedure for this test is available in ASTM D5. In this test, a standard needle, carrying a standard weight, is made to penetrate in the mastic for a specified amount of time. The amount of penetration is noted and converted into penetration index using the appropriate formulas. Higher the penetration, softer is the material. This test is not sufficiently sensitive at 60^{o}C and lower, with a 100g weight. With increasing amount of filler concentration, there is decrease in the penetration almost at a linear rate. The linearity is dominant for a filler-bitumen ratio from 0.1 to 0.4.

Viscosity

The addition of filler to the bitumen significantly alters the viscosity of the bitumen. The viscosity measurements at lower temperatures are carried out by sliding plate micro-viscometer and are done using the capillary viscometer at higher temperatures. The capillary viscosity tests, if carried out at compaction temperatures, can be used to estimate the compaction effort. Puzinauskas [13] has stated that increase in viscosity is not only related to the specific surface area but is also related to the affinity between the filler surface area and the asphalt. The problem with the viscometer test is that at high filler concentrations, there is settlement of filler in the capillary tubes, which might affect the results. In addition to this, the rate of shear becomes an overriding factor over the concentration of fines. This problem is not prevalent with the dynamic shear rheometer (DSR) and the softening point apparatus where better repeatability for high filler-bitumen ratio mixes is obtainable.

Huschek [7] attributes rutting by external forces at high temperatures to the viscosity of the filler-bitumen mixture. The adjacent filler particles, separated by a thin film of binder butt against each other squeezing the film out. This causes a permanent change in the matrix structure, which cannot be reversed after the removal of load. The same principle is used in the measurement of viscosity using a Squeeze Film Viscometer. In this method, a filler-bitumen mixture is squeezed between two parallel plates by applying axial pressure for a given time; the final and the initial heights are measured and using a mathematical relationship the dynamic viscosity is obtained. The dynamic viscosity is dependent on the amount of filler and its volumetric properties.

Dynamic shear rheometer

Commonly known as the DSR, it is used to characterize the linear viscoelastic behavior of the asphalt binder over a range of temperatures, frequencies, strain, and stress levels. It consists of two parallel plates, one stationary and another free to rotate about a vertical axis. The asphalt specimen is sandwiched into the two plates and is subjected to either rotational strain or stress. The levels of strains, stresses, and temperatures can be controlled by computer software. It measures the resistance offered by the asphalt binders to deformation, in terms of the applied torque for oscillation.

The DSR characterizes both the elastic and the viscous components of the resistance offered by measuring the complex shear modulus (G^*) and the phase angle (δ). G^* is the total resistance offered by a material to deformation and δ represents the relative proportion of the elastic and viscous components of G^*. The elastic component is given by $|G^*|\cos\delta$ and represents the modulus corresponding to the recoverable deformation. The viscous component is given by $|G^*|\sin\delta$ and it represents the modulus corresponding to permanent (non-recoverable) deformation. The values of the phase angle depend on the material properties and temperature and are in the range of zero to 90^o.

It is expected that with an increase in the fines in the filler-bitumen mixture, the stiffness is likely to increase. However, in some cases, the filler also acts as an 'asphalt extender' resulting in lowering of stiffness and increase in the δ values. This transition from 'stiffener' to 'extender' depends on the size of the particles, their concentration and the type of binder used. Another method has been suggested by Harris [5], in which the concept of 'stiffening power' is described. It defines stiffening power as the increase in the temperature of mastic needed to match the stiffness ($|G^*|$) of neat binder at 50^oC. This test failed to identify the so-called good and bad fillers and no correlation could be obtained using this method.

<u>Other properties</u>

The softening point test measures the temperature at which the asphalt cement cannot support the weight of the standard steel ball and starts flowing. Performed using the ring and ball apparatus, the purpose of this test is to measure the temperature at which the phase change occurs in the asphalt cement. Kandhal [10] has shown that the temperature characteristics obtained by methods such as penetration, viscosity, softening point are all identical. Also, notable is the fact that the difference in the temperature susceptibilities increases with an increase in the fines. Lower ductility values are synonymous with higher stiffness (viscosity) and or lack of adhesion (tackiness). It has also been shown by Kandhal [10] that there is maximum reduction in ductility of samples with highest viscosity.

Volumetric Analysis of filler-bitumen systems [6, 10]

The fractional voids in the filler at its closest packing are of major importance to understand the behavior of filler-bitumen systems. This concept has been shown in Figure 1.2 and has been discussed by Heukelom [6]. If the amount of asphalt used for filler-bitumen mix is much lower than required to fill the voids of the filler then a stiff dry product is obtained. Excessive filling of voids with asphalt makes the mixture more fluid. In a filler-bitumen mix, the asphalt trapped in between the inter-granular spaces gets 'fixed' whereas the remaining is ‘free.’ Due to this, the functional volumes of solid and fluid phases are different from the compositional volume percentages of filler and bitumen. Filler-bitumen systems are typically proportioned based on volume of solid fine particles (V_f) and asphalt (V_b). The bulk volume (V_{fa}) is different for every filler and is dependent on the Rigden voids ($100-V_{fR}$). It can be seen from Figure 1.2 that higher bulk volume of fines results in more ‘fixed’ asphalt and less ‘free’ asphalt. Typically, there is a rapid increase in viscosity after the bulk volume of fines increases above 40-percent. This increase in viscosity makes the mix difficult to compact unless the temperature of compaction is raised. Insufficient compaction can lead to rutting and cracking. Kandhal has suggested that an upper limit of 60% on V_{fa} be put on the bulk volume of fines to avoid undue stiffening and loss of stability. It has been proved that the tensile strength of a mix reaches a maximum at 60-percent bulk volume concentration. Further, Kandhal [10] suggests that up to 50-percent concentration there is not much effect on the binder but above it, a detailed analysis needs to be performed.

1.4.1.6 Effect on mixes

Effect on compaction

Relative compaction effort can be measured in two ways [10]:

1. Keeping the same compaction effort but compacting at different temperatures
2. Compacting at same temperature but with different compaction efforts.

The presence of filler increases the viscosity of the filler-bitumen mixture and this leads to an increased effort for compaction. Tests [10, 12] indicate that there is a good and significant correlation between the binder viscosity and the compaction effort needed to densify a paving mixture. It has been suggested that a substantial increase in the temperature may be needed when compacting paving mixtures containing high viscosity filler-bitumen binder.

Marshall stability

Marshall stability is the most widely used test to measure the strength of the mix. Due to increased stiffness of the mastic, the stiffness of the HMA also increases. Apart from this, the filler serves to make the mix denser by filling in the voids and creating tighter aggregate interlock. This leads to an increase in the Marshall stability values. The increase in stability is more due to the aggregate interlock than stiffening of the binder [9]. Less filler is needed to develop maximum stability in dense graded asphalt concrete than in poorly graded sheet asphalt mixtures. It has been shown [19] that even for crushed aggregates with natural sand the stability increases with increase in the amount of mineral filler. That is, however, true only till a certain limit. Excess amount of mineral filler produces improper compaction and therefore there is a drop in the stability values. If the void content is kept the same (implies varied compaction effort), then there is a continued increase in stability.

Flow values

Marshall flow values are considered equal in importance to stability for judging the suitability of mixtures by the Marshall method. These values should depend primarily upon the surface characteristics of the filler and the ductility of the filler-bitumen system. They increase with the increase in the amount of fines. However, no explicit correlation has been obtained relating it to the amount of filler [20].

Asphalt content

The asphalt in the mixture primarily is distributed in absorption, coating aggregates, forming a filler-bitumen system, and occupying the voids. It has been found that increasing the amount of filler decreases the asphalt requirement [20]. This has been found so because filler also acts as an asphalt extender. The smaller filler particles are embedded in the film around the larger aggregate particles thus increasing the thickness of the film causing an apparent increase in the asphalt content.

Density

It has been observed that there is an increase in the unit weight of the mixes at the optimum bitumen content containing more fines. Also, there is a wide variation in the asphalt content required for achieving maximum densities for various fillers.

Fatigue and permanent deformation

No definite relationship has been found regarding fatigue. However, if the stiffness increases the number of cycles to failure will increase due to lower tensile strains. The permanent deformation can be measure using an RSCH test [20]. It has been shown that increasing the amount of mineral filler in the mixtures reduces the accumulation of permanent strain. The difference in performance of two mixes is dependent on the amount of filler. Greater the amount of mineral filler, higher is the difference in the performance. Presence of excessive amount of fines impedes compaction making the mix more susceptible to rutting.

Moisture sensitivity

Moisture sensitivity depends on the affinity of mix to water. Presence of asphalt always leads to decrease in moisture related damage. Increase in asphalt above the optimum always decreases the stability. It has been proved that the mineral fillers affect cohesion (i.e., apparent viscosity) of filler-asphalt mixtures and of asphalt paving mixtures. The mechanism of the filler in promoting adhesion in bitumen-aggregate systems is both mechanical and physico-chemical. On one hand, high viscosity can reduce the 'coat-ability' and wetting of the aggregate during the coating phase. On the other hand, with good initial coating and wetting, the resistance to stripping is increased with the increasing viscosity of the binder [8]. With the use of large quantities of filler, or with the selection of improper filler, such poor adhesion may become critical, particularly, when the paving mixture is exposed to the action of liquid water or water vapor. In such an environment, the bond between binder films and mineral aggregate may be weakened or even destroyed leading to a weak, poorly performing unstable paving mixture. It has been shown that different fillers require different amounts of asphalt for the satisfactory water proofing of the paving mixture, however these contents do not necessarily coincide with the optimum asphalt contents from regular mix design procedures. Pre-mixing of filler with bitumen or normal mixing may not make any difference in the moisture susceptibilities. However, the moisture-related damage can be kept to a low level if the filler to asphalt ratio is kept at a low level. Generally, fillers containing more hygroscopic aggregates have higher moisture susceptibilities. The water resistance of the mix along with the optimum amount of filler required could be determined using the Immersion-Compression tests [9, 13], or using the procedure outlined in AASHTO T283.

Low temperature performance

The stiffening of the mix due to the filler increases the amount of thermal stresses induced in it due to extreme low temperature. In addition, lower ductility means more chance of cracking. In cases where the asphalt material is 'extended' by the fines, there is a lower chance of thermal cracking due to the apparent increase in the asphalt content.

1.4.2 Tack coat evaluation

1.4.2.1 What is tack coat?

A tack coat is usually a light application of asphalt, usually emulsions, to ensure a bond between the surface being paved and the overlying course [2]. Currently, most of the states in the US use SS (Slow Setting) or CSS (Cationic Slow Setting) emulsions diluted with equal amount of water for tack coat. The tack coat is applied on the paving surface but before the application of tack coat, the paving surface is 'prepared.' 'Preparing' the existing surface involves removal of all loose dirt, clay, asphalt material, leveling off the depressions, potholes and milling of high spots. After preparation, the emulsions are either sprayed or painted at the rate of 0.05–0.15 gallon per square yard. For optimal effect, it is necessary that the tack coat is thin, uniformly applied, and must be allowed to break (cure) before the hot-mix asphalt (HMA) is paved. Tack coats having excessive asphalt over the surface sometimes form a slip plane. Excessive asphalt can also cause bleeding and flushing. Usually, a light tack coat is preferable as it bonds the surfaces and does not have any adverse effects. In addition, it is recommended that the tack coat does not remain exposed to air for longer than a day.

1.4.2.2 Summary of previous research

Currently, the design and evaluation of flexible pavements is based on an elastic multi-layered analysis. For the design of pavements, the interfaces are assumed rough with no slippage occurring between the two layers. This, however, is not the case in practice. The state of adhesion at the interfaces between various layers affects the performance of flexible pavements by influencing the stressing level of the materials. It also affects the distribution of stresses in the pavement structure. The stress distribution is more influenced by the interfacial condition of the upper layers than the lower ones. Hence, the knowledge of the interfacial conditions in upper layers is important.

Study by Shahin et al. [16]

Shahin et al [16] have discussed the effect of layer slippage on the performance of asphalt pavements. Using an example of an airfield pavement and with BISAR (Bituminous Structures Analysis in Roads) and the French Shell model for analysis, various scenarios were evaluated about the fatigue life of the pavement. The pavement section that was assumed consisted of a 2-inch thick overlay over a 4-inch thick AC surface course. The base course was 25-inches thick with elastic modulus of 75000 psi and a CBR value of 80. The subgrade was quite weak with CBR (California Bearing Ratio) value of 5 and stiffness of 7500 psi. The tensile stress at the bottom of the asphalt layers (overlay and the original surface course) and the vertical compressive strain on the subgrade were the criteria for failure. It was found that for full friction between the interfaces, the maximum tensile strain in the section is located at the bottom surface of the original asphalt layer. If the slippage was allowed below the uppermost layer, the tensile strain also existed at the bottom of the overlay. In addition, the following observations were made:

- Only a small amount of slippage is sufficient to produce strains in the pavement that approach those of the free slippage case.
- The tensile stress at the bottom of the overlay causes a compressive stress to develop on the upper surface of the asphalt surface layer. This causes a relative movement of points on the either side of the interface. This distortion further weakens the bond between the asphalt layers, allowing more slippage leading to higher strains.
- The subgrade strains increase with increasing slippage. Because two thinner layers are not as stiff as single layer of the same overall thickness, the compressive vertical strain on the subgrade increases.
- Further, under the action of horizontal loads, the study found that for no friction, the horizontal strains are much higher than those for without friction.

The principal normal tensile strains, developed by the horizontal loads along the back edge of the contact area, are of the same magnitude and cause progressive failure along the rear edge. This tensile failure would cause slippage cracks in the overlay. If the overlay is not properly bonded to the underlying layers, the overlay moves resulting in the opening of the cracks. These cracks are crescent shaped. In order to fix these cracks, either the existing layer needs to be removed and re-paved, or a thicker well-bonded overlay be placed on the existing overlay. In addition to strong interlayer bonding, it was recommended to have an overlay stiffness of at least 500,000-psi.

Study by Uzan et al [23, 24]

A research to evaluate the adhesion between asphalt mixes was conducted by Uzan et al. Using the Goodman's constitutive law [4]:

$$\tau = K \times \Delta u \tag{1-8}$$

Where:

τ denotes the shear stress at interface,

Δu the relative horizontal displacement of the two faces at the interface, and

K is the horizontal interface reaction modulus,

the interface behavior was described, which formerly was restricted only to perfectly rough or perfectly smooth conditions. The analysis was carried out using BISAR program for a test section at different levels of adhesion. It was observed that for perfectly smooth interfaces (K = 0) the tensile radial strain at the bottom of the uppermost layer was higher than for the perfectly rough interface. The top of the second layer also changed to compressive strain when K approached zero. Further, it has been shown that even an adherence of 90 percent was very close to the smooth condition as has been shown by Shahin et al [16].

Direct shear tests were performed on the layered asphalt concrete specimens. The shearing was done along the tack coat and the variables were temperature, vertical pressure, and rate of application of tack coat (Figure 1.3). It was concluded that the components influencing the strength of the interface were (a) adhesion, represented by the tensile properties of the slip plane (b) interlocking, from the penetration of aggregates into the voids of the other layer; and (c) friction, from rugosity of the two faces. Further, the friction component was included in the other two components. It was suggested that measurement of the adhesion component, which is indicated by rupture of the bond between layers in the bitumen or mastic phase, could be done by a tensile test. (The interlocking effect would be absent for pure tension.) The interlocking component depends on the texture of the surfaces in contact and properties of the asphalt mix. The following factors largely affect the interface shear strength:

Temperature

It is known that temperature affects the asphalt properties. The stiffness decreases with increase in the temperature and vice versa. The effect of higher temperatures is more dominant while testing in tension than in compression. In order to offset the effect of increased temperature, higher vertical pressures are applied. With increasing vertical pressures, the interlocking component gets more dominant than the adhesion component. For tensile testing, the tensile strength decreases rapidly with increase in testing temperature.

Tack Coat Rate

The tack coat bonding the two layers usually functions in two ways: (a) fill the voids on the surface, (b) increase the interface film thickness or get absorbed in the adjacent layers. The filling of voids on the surface of the mixes increases the contact area and consequently the adhesion. However, excessive film thickness decreases the adhesion and aggregate interlock. Very low tack coat rate could mean loss of the adhesion component. Hence, it is required that the tack coat be applied at an optimum rate.

Rate of Deformation

The rate of shear deformation is an important factor controlling the strength and deformation ability of the interface. Generally, with increasing rate of deformation the values of the stress increase. The rate of shear was 2.5-mm per minute.

Study by Tschegg et al. [22]

A common method for measuring the bond strength of asphalt cores is the pull-off test, Tschegg [22]. For this test, cores with a diameter of 100-mm were drilled from the top surface down through the overlay, through the interface, and about 50-mm into the base layer. Steel plates were glued to the top surface of the cores. Then the drill core was pulled off with a tension machine in axial direction of the base layer. The maximum load is registered during the pull-off test. This is a simple test method but gave only the adhesive tensile strength and showed extensive scattering of results. The reasons for wide scattering of results were: eccentricity of load, small core diameter and large aggregate size, notches at the surface of the cores by drilling or burst out aggregates, stress concentrations, uncontrolled temperature, and indentation effects owing to rough surfaces. In addition, the test was useless if the tensile strength of the mix was lower than the interface bond strength.

For avoiding such drawbacks, a ‘Wedge Splitting Test’ was developed. In this test, a block of asphalt concrete was made to crack along a predetermined joint at a steady rate. The splitting was done by a wedge that was located in a groove between the two blocks of asphalt. The force and the displacements were recorded during stable crack propagation until complete separation of the specimen took place. Based on the shape of the force-displacement curve, a differentiation between brittle and ductile behavior is possible. Figure 1.4 and Figure 1.5 show the test setup and the specimens used for testing purposes. It was found that with increasing temperature, the plastic behavior of the asphalt increased. There was a decrease in the peak load values with an increase in the temperature. At low temperatures, it was found that the relationship between the force and the crack opening displacement was linear. However, this test could not distinguish between the two different types of tack coats used in that study.

1.4.3 Relevance to pavement design

The design of flexible pavements is based on the tensile stress at the bottom of the surface course or the compressive stress at the top of the subgrade. Full adhesion implies lower stress at the bottom of the top layer and consequently, lower vertical stress on the subgrade. Another approach based on the smooth interfaces indicates that tensile stresses exist at the bottom of each asphalt layer. For partial adhesion, the stress distribution is complex; this could create a problem of under-design of the pavement based on the stressing of subgrade. In addition, at high temperatures, crescent shaped cracks may develop under vertical and horizontal loads where the interface bond is weak due to poor construction. Existence of a strong bond between the layers would prolong the life of the pavement requiring lesser maintenance and lower costs. Hence, it is important that the tack coat and its effect on interface properties be investigated.

Table 1.1 Particle size analysis methods [5]

Method	Means	Measurement Basis	Parameter Measured	Size Range (μm)
Screening	Woven wire or electroformed	Weight of fractions	Cross sectional area	> 38
Sedimentation	Air-gravity	Settled volume	Hydraulic drag	1 to 50
	Liquid-centrifuge		Diameter	0.02 to 30
Microscopy	Optical electron	Projected area	Area or chord length	> 1 > 0.001
Light scattering		Diffused light	Projected area	0.1 to 200
Electrical conductivity		Electrolyte streaming	Volume	0.4 to 400
Light blocking		Light interruption	Projected area	2 to 400
X-ray	Small angle	Scattering	Volume	< 0.05
	Large angle	Broadening		< 0.02

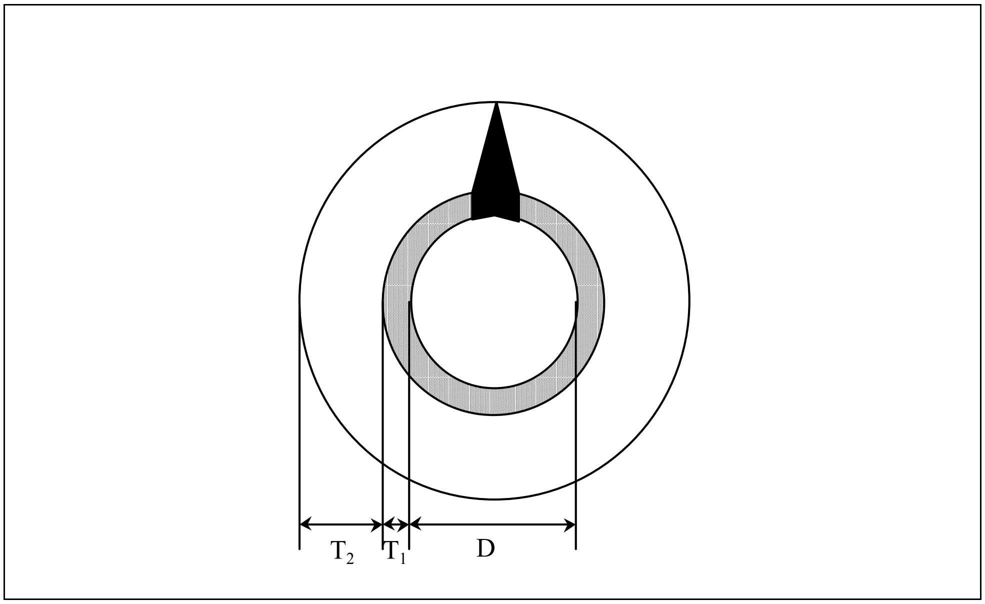

Figure 1.1 Suspended mineral particle in bitumen, [20]

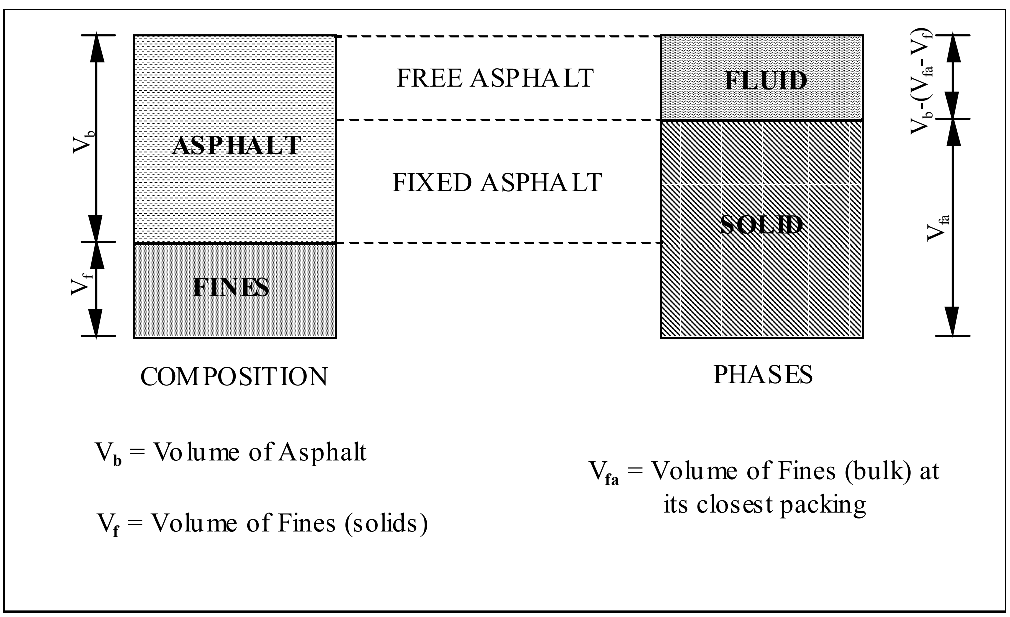

Figure 1.2 Basic concept of fractional voids, [6]

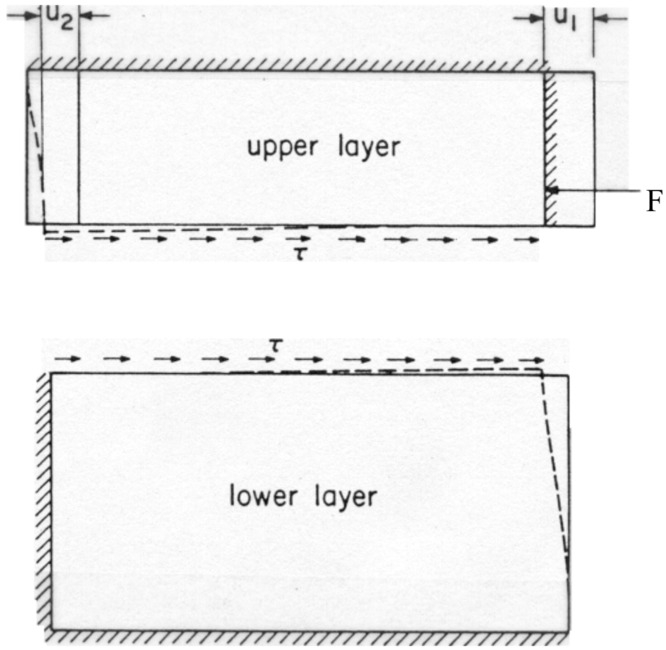

Figure 1.3 Schematic of specimen deformation during shear testing, [24]

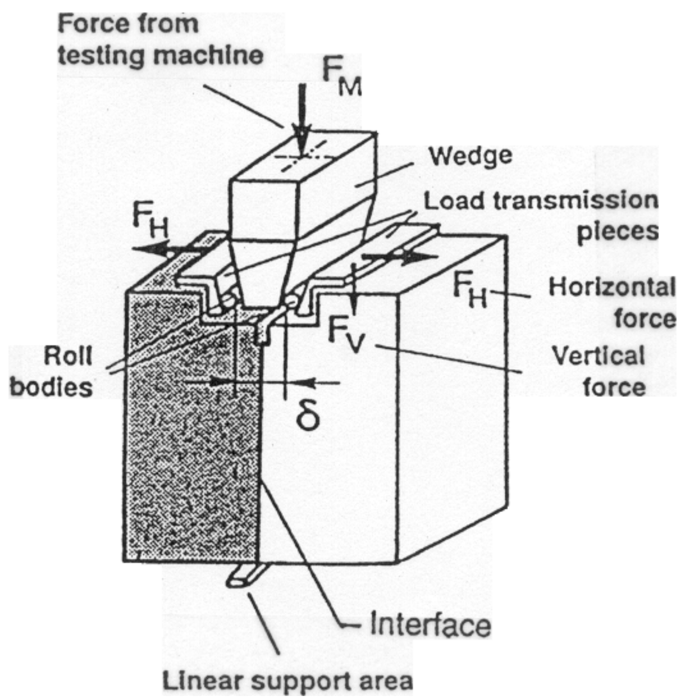

Figure 1.4 Setup of wedge splitting test, [22]

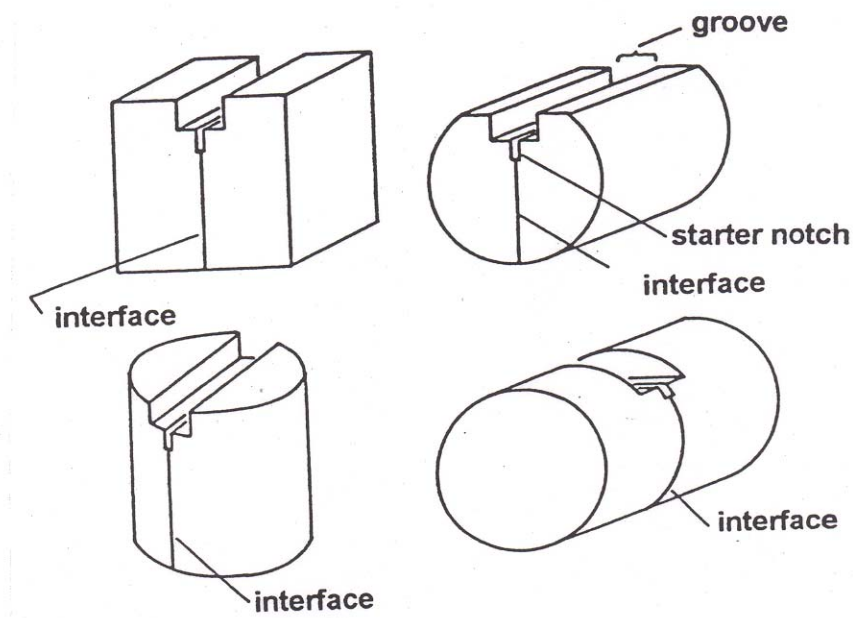

Figure 1.5 Specimen shapes for wedge splitting tests, [22]

2. RESEARCH APPROACH AND METHODOLOGY

2.1 Introduction

Asphalt layer delamination was one of the distress modes occurring in NCDOT Division 13. The suspected cause was the poor bond between the interfaces of the asphalt layers tacked using CRS-2 emulsion. Due to high level of distress, NCDOT allowed CRS-2 emulsion to be substituted with PG64-22 binder. Hence, it was necessary to evaluate the interface bond strength of the field cores where CRS-2 and PG64-22 were used as tack coats. Field cores were obtained from existing pavement sections that were in service for several years and had shown poor as well as good field performance in terms of slippage delamination distresses. A survey was undertaken to determine the location of pavement sections to be sampled. Details of the survey questionnaire and the results of the survey are presented in chapter 3.

Research approach in this investigation was divided into separate tasks as shown in Figure 2.1. The work plan for each task is briefly discussed in the following sections.

2.2 Research tasks

2.2.1 Characteristics of asphalt mixtures containing baghouse fines

The Asphalt Institute Manual Series MS-16 [3], recognizes the detrimental effect of the rounded particles and its contribution to mixture instability. Although, present job-mix in NCDOT Division 13 may not contain high sand content, it was hypothesized that the variable amount of baghouse fines purged intermittently into the asphalt mixture may account for the mix instability and significant changes in volumetric properties. Therefore, the objective of this task was to evaluate the contribution of baghouse fines relative to the total mineral filler content in the mix. This task was divided into the following subtasks:

2.2.1.1 Evaluation of field mixes

In this subtask, cores from NCDOT Division 13 were obtained for forensic analysis based on the results of the survey. Asphalt content, air void content, Marshall stability [2], and the aggregate gradation were evaluated based on 4-inch diameter field cores. The asphalt content determination was done by the NCDOT M&T Unit. Residual aggregates from the ignition oven test were then subjected to gradation analysis. The mineral filler (material passing #200 sieve) portion from the gradation analysis was sent to the FHWA laboratory for further analysis using the state-of-the-art "particle materials analyzer" to determine the specific nature of mineral filler gradation, i.e., particle size distribution of material passing the number 200 sieve. At the same time bag-house fine samples collected from the field were also sent for analysis to FHWA.

Following the Marshall stability testing of the cores, the material from these cores was recompacted using 75 blows Marshall hammer to verify the job-mix with the NCDOT required mix design criteria.

In addition to the four inch diameter cores, six inch diameter cores were also obtained to evaluate the shear susceptibility of the composite cores using the Superpave™ repeated shear constant height (RSCH) test at a single temperature corresponding to critical temperature for the site under investigation.

2.2.1.2 Laboratory mix evaluation

The objective of this subtask was to evaluate the effect of baghouse fines on mix characteristics using controlled mixes, and investigate their sensitivity to moisture exposure. Raw materials including the baghouse fines and the job-mix were obtained from NCDOT Division 13. Two mixes were prepared in the laboratory based on the job-mix formula with materials passing #200 sieve (mineral filler) containing:

a) zero percent baghouse fines; and

b) 100-percent baghouse fines.

All other mix parameters such as asphalt content and aggregate gradation were kept constant. These two mixes were subjected to the following laboratory testing:

- Laboratory RSCH test on 6-inch diameter specimens compacted using SGC;
- Georgia wheel rutting test at NCDOT M&T Unit on six inch diameter specimens prepared using SGC;
- Moisture sensitivity testing using the modified AASHTO T283 procedure.

2.2.2 Tack coat bond strength evaluation

The objective of this task was to evaluate the strength of AC layer interface bond consisting of CRS-2 and PG64-22 tack coats. Six-inch diameter cores obtained from NCDOT Division 13 were subjected to the following performance tests:

- Shear frequency sweep (FSCH) test;
- Repeated shear at constant height (RSCH) test;
- Uniaxial frequency sweep test (AFST);
- Uniaxial tension test (VRAMP).

2.2.3 Characterization of rheological properties of asphalt-fines mastic using DSR

The objective of this task was to evaluate the rheological properties of the binder in the presence of baghouse fines using a dynamic shear rheometer (DSR). This task was subdivided into following three subtasks:

1) in sub-task 1, the properties of aged and virgin binders from both the counties were evaluated;
2) in subtask 2, rheological properties of the mastic containing baghouse fines and virgin binders was evaluated; and
3) in subtask 3, the mastic was aged and its rheological properties evaluated.

Figure 2.1 summarizes the organization of this report. The first three chapters present introduction and background, research methodology, and survey results, respectively. Volumetric and stability analysis results are presented in Chapter 4. Chapter 5 deals with the fines and mastic evaluation using DSR. Results of performance testing of field cores and laboratory mixes are presented in Chapter 6, followed by results of wheel track and moisture sensitivity tests in Chapter 7. Summary and conclusions are presented in Chapter 8.

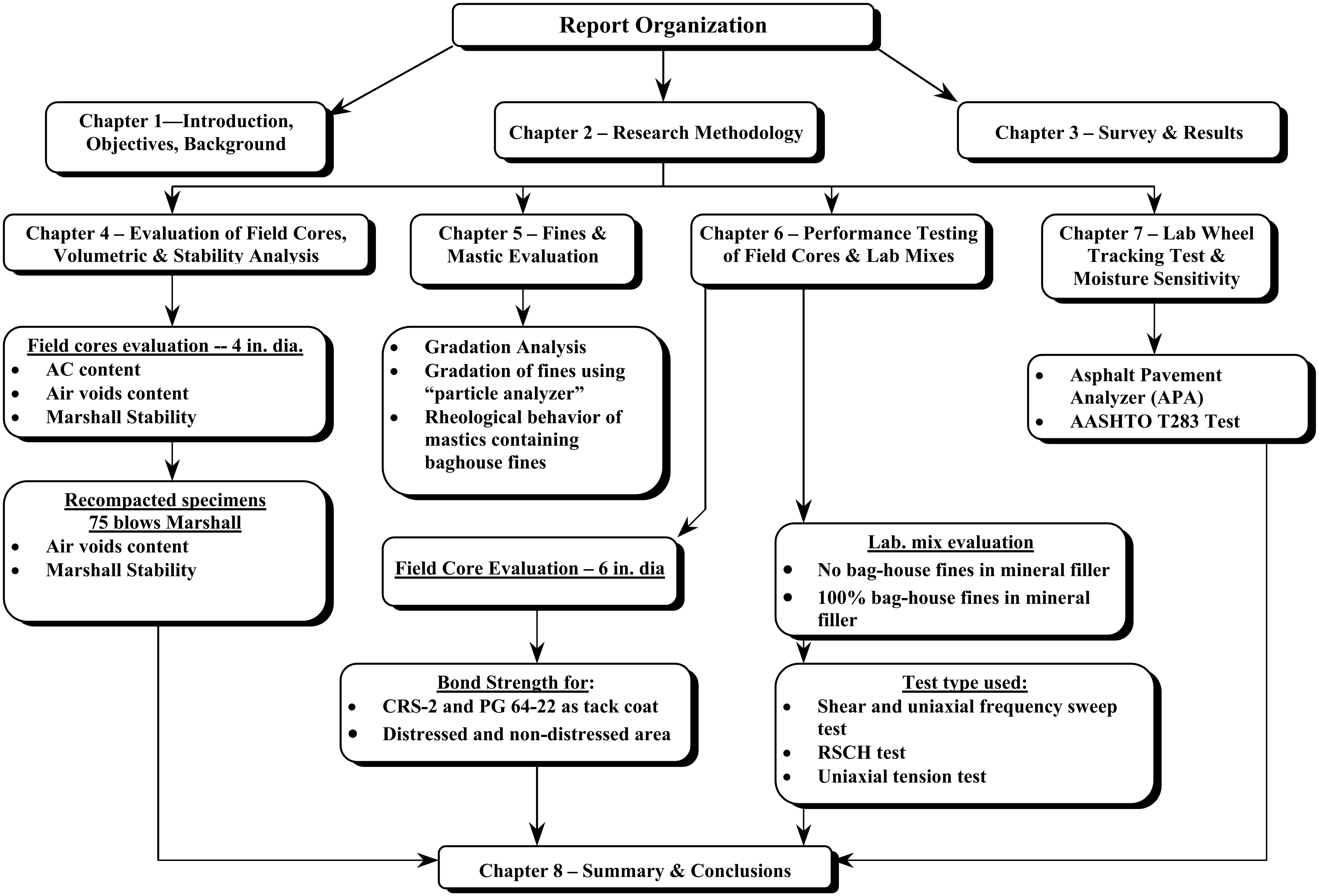

Figure 2.1 Summary of research approach and methodology

3. SURVEY ANALYSIS

3.1 Development of questionnaire

Concerning the specific distress mechanism of delamination and shoving encountered in Division 13 of NCDOT, the purpose of the research program was to investigate the effect of using different types of tack coats.

In order to acquire information as to the use of different tack coats and the type of material used in the field, it was necessary to develop a questionnaire so that appropriate field sections could be identified for evaluation. Table 3.1 shows the questionnaire that was sent to the NCDOT Division 13 to identify viable field sections with other pertinent HMA plant and mix information.

Information about the dust collector system(s), method of introduction of fines especially the baghouse fines, the use of recycled asphalt pavement (RAP) and the job mix formula (JMF) used for the production of HMA was also collected. In all, NCDOT Division 13 identified six field locations where different HMA plant mixes or tack coat types were used. These six plants surveyed were:

1. ASTEC, Rutherford County, located at Rutherfordton, NC.
2. GENCORE, Buncombe County, located at Enka, NC.
3. BARBER - GREEN, Burke County, located at Morganton, NC.
4. McCARTER, Buncombe County, located at Swannanoa, NC.
5. APAC – Consta Flow, Buncombe County, located at Weaverville, NC.
6. BARBER - GREEN, Yancey County, located at Burnsville, NC.

3.2 Questionnaire analysis

Results of the questionnaire obtained from the six plants indicated above are attached in Appendix 1. Based on the analysis of these questionnaires the following observations could be made:

- All plants produced mixes for base courses, binder courses, and surface courses.
- All plants used baghouses for dust collection purposes.
- All plants reintroduced the baghouse fines into the mix.

- No baghouse fines were wasted, implying that all fines were reintroduced into the mix.
- The method of introduction of fines into the mix was by auguring, and was not controlled or systematic.
- Those plants that used RAP in pavements restricted it to an upper limit of 15-percent.
- The binder used by all the plants was PG64-22.
- With the exception of Rutherford County plant, all plants used CRS emulsions from SECO, South Carolina as tack coat.
- Most of JMF's added non-strip additives (0.25-0.60 percent).
- The mixing and paving temperatures for all plants were 310 and 290^{o}F, respectively.

3.3 Survey results

Based on the responses to the questionnaire and in consultation with NCDOT, two field sections were identified. As one of the objective was to study the effect of two different tack coats, the following two plants were chosen: Enka, Buncombe County and Rutherfordton, Rutherford County. CRS-2 emulsion was used as tack coat for pavement section in Buncombe County, whereas PG64-22 binder was used as tack coat for the pavement section in Rutherford County. Both the Enka and Rutherfordton are drum mixer plants, and the Enka plant used RAP in their mixes. Field cores from these two counties were obtained from the distressed and non-distressed (if they could be identified) sections of the pavements. For the laboratory mixes the raw materials were obtained from the respective plants. The analysis of the cores obtained is described in subsequent chapters.

Table 3.1 Questionnaire delamination and shoving of asphalt concrete layers

1. Date:			
2. County:			
3. Location of asphalt mix plant:			
4. Type of plant:	**Please circle one**	**Batch Plant**	**Drum Mixer**
5. Plant brand name:			
6. Types of mixes produced by plant:			
7. Source(s) of aggregates used by plant:			
8. Type and source(s) of asphalt cement used:			
9. Mixing and lay down temperature:			
10. Type and source of tack coat used:			
11. Type of dust collector used:			
12. How is dust introduced into the mix:			
13. Are all bag house fines used in mix or is part wasted:			
14. Is RAP used and percentage:			
15. Rated capacity of plant (tons/day):			
16. Attach all current NCDOT approved Job Mix Formula:			

4. EVALUATION OF FIELD CORES

Based on the results of the survey presented in Chapter 3, the asphalt plants selected for investigation were Rutherfordton, Rutherford County and Enka, Buncombe County. In order to evaluate the in-situ mixes, it was necessary that cores be obtained from the existing paved sections. The details of the coring operation and the field sections are described in the following sections.

4.1 Buncombe County cores

Cores were obtained from distressed and non-distressed sections of I-26 in Asheville, Buncombe County. All the cores were approximately 10 inch in depth and were drilled from the wheel path of the southbound lane. In all, sixteen (16) 4-inch and twenty (20) 6-inch diameter cores were received. The cores were labeled from 9 through 28. Cores 9 through 18 represented the cores from non-distressed section – 'good cores' – and cores 19 through 28 represented cores from distressed section – 'bad cores.' There was a little visible difference between the good and bad sections of the pavement. Core no. 9 was drilled 400 ft north of the north end of the French Broad River Bridge. Core no. 10 was 50 ft north of core no. 9 and subsequent cores through no. 18 were spaced at 50-ft intervals. The spacing between cores 18 and 19 was 1620 ft and the spacing between each of cores from 19 through 28 was 50 ft. The 6-inch and 4-inch cores having the same numbers were taken from the same location and were spaced about 6-inch longitudinally. The pavement section consisted of two 2-inch thick lifts of hot mix asphalt surface course (BCSC, Type HDS, and JMF # 93-447-052) placed over a portland cement concrete pavement. This pavement had been rubbelized before the placement of the hot mix asphalt. The JMF for the above mix is attached in Appendix B.

4.2 Rutherford County cores

This pavement section consisted of two 2-inch thick lifts of HDS mixture (BCSC, Type HDS, JMF # 90-840-053, 93-903-051) placed over a 4-inch thick binder course. The samples were taken from wheel paths on a section of outermost westbound lane of US 74, NC Bypass. In all twelve (12) 4-inch (labeled 1 through 12) and twenty-four (24) 6-inch (labeled 1 through 24) diameter cores were obtained. The 4 and 6-inch diameters cores having the same numbers were immediately adjacent to each other. It was found that there was not much difference between the surfaces of 'good' and 'bad' pavement sections; also the coring was done in a period of moderate to heavy rainfall which made it more difficult to visually differentiate the surface condition of the pavement at various locations. The project began at the intersection of US 74 and US 221 South. The first core sample was obtained just beyond the end of the off ramp from US 221 at the point where the two outbound lanes begin. The cores were spaced approximately 50 ft apart and sampling extended to the crossover road identified as Oakland Road.

4.3 Nomenclature for field cores and laboratory specimens

The following nomenclature was adopted for identifying the field cores. The field core names consisted of two characters and a number at the end. The first character stands for the county from which the core was obtained – **R** for Rutherford County and **B** for Buncombe County. The second character stands for the type of cores – **G** for cores from pavement section with 'good' performance and **B** for 'bad' cores from pavement section showing relatively poor performance. The numbers at the end represented the core number. For example, core numbered BB23 indicated a Buncombe County core from a 'bad' section with serial number 23. For the laboratory specimens a slightly different system was adopted. As an example, for specimen labeled RWO41; **R** stands for a specimen made from Rutherford County materials, **WO** stands for specimens *without* baghouse fines and **41** denotes the serial number of the specimen.

4.4 Visual inspection of field cores

The cores obtained from the pavement sections were observed visually before sawing them. All cores had the direction of traffic marked on them. The following two subsections describe in detail the description of the cores obtained from each county.

4.4.1 Buncombe County cores

As mentioned earlier, the pavement structure consisted of two 2-inch thick layers of HMA surface course placed over a portland cement concrete pavement. The tack joint between the two 2-inch thick layers was clearly visible for all the cores. The thickness' of the upper and lower HMA layers were much less than the originally specified lift thickness of 2-

inches. The thickness of the upper layer ranged from about 1-inch to less than 1.5-inches. This indicated either occurrence of compaction due to traffic loading or construction variance from a specified lift thickness of 2-inches, or both. The bottom of the lower layer for most of the cores was clean, smooth, and dry, which was expected as the underlying layer was of portland cement concrete pavement. The lower layer had asphalt mix or hardened mastic sticking at its bottom, which looked dry and had a large amount of fines in it. The bottom ¼-inch layer looked slightly darker than the upper portions for all cores. Other notable features included a taper for upper ¼-inch for all the 6-inch cores.

Cores BG09 and BG10 had thin upper layer and were not used. Core BG13 had large number of pop outs from the bottom whereas for core BG14 the joint was not clearly visible. Cores BG11-12 had clearly visible interfaces with smooth surfaces. The lower half of the core BB19 seemed to have broken while drilling and the break seemed to have occurred just below the tack coat. Cores BB22-28 had slight taper due to drilling but were fine otherwise.

4.4.2 Rutherford County cores

The structure of the pavement was the same as mentioned earlier with two 2-inch thick layers of HDS mixture placed on 4-inch binder course. All cores at their bottom had some amount of base/subbase material, which could be removed easily by scraping. The material seemed to be cementatious, had a large amount of fines, and loosely bound large sized aggregate, which was easy to pop out. There were two interfaces, one between the upper HDS layers and the other between the middle HDS and the lower binder layer. The lower interface could be clearly distinguished because of the larger aggregate size in the binder layer compared to the aggregate size in the HDS layer.

Cores RG01 through 06 had very clear interfaces. In fact, most of the 'good' cores had clearly visible interfaces especially the upper ones. For cores RB13 through 15, the lower interface was quite fuzzy. In addition, the interfaces and the top surfaces were not parallel. Cores RB16 through 18 did not have clearly visible interfaces. Cores RB19 through 22 had their upper surfaces inclined slightly.

4.5 Description of the test program

The field cores obtained were sawed to a height of 2.5-inches and subjected to two categories of tests – volumetric analysis and Marshall stability testing. The 4-inch diameter cores specimens were sawn to a height of 2.5-inches from the asphalt concrete layer of interest. For the Marshall Stability tests, the layer of interest was the uppermost one. The cores were then subjected to bulk specific gravity test (ASTM D 2726) followed by the Marshall Stability test (ASTM D 1559). They were then broken down and recompacted using the Marshall hammer (75 blows). The bulk specific gravity and the Marshall Stability tests were repeated on all recompacted specimens from both the counties.

Following the Marshall tests, the theoretical maximum specific gravity was determined using the residue (ASTM D 2041). The samples were then subjected to determination of the asphalt content using the ignition furnace at the NCDOT Materials and Tests Unit. The skeleton aggregates from the ignition furnace test were then subjected to gradation analysis (ASTM C 136).

4.6 Volumetric and stability analysis for field cores

For Buncombe County, cores numbered BG11, BG13, BG14, BB20, BB21, and BB24 were selected for Marshall Stability tests. For Rutherford County, cores numbered RG01, RG02 and RG03 were selected for the same. As there was not much visual difference between the cores from the so-called good and bad pavement sections, only three cores from Rutherford County were chosen for this task. Table 4.1 shows the specifications for Marshall testing and the JMF requirement by NCDOT.

4.6.1 Gradation results

Table 4.2 and Figure 4.1 show the results of the gradation analysis performed on the skeletal aggregates obtained from the ignition oven test. Table 4.3 shows the gradation specification limits for mixes based on the NCDOT recommendations and from the respective JMFs for comparison with the actual results. The bulk specific gravity values for the aggregate materials from the respective counties are shown in Table 4.4.

Based on Table 4.2 and Figure 4.1 it may be noted that gradation for the aggregate materials from Buncombe County were slightly coarser as compared to the materials from the Rutherford County. For the Buncombe County, the materials from the so called 'good' and 'bad' pavement sections are identical and the materials from both counties are generally within the NCDOT gradation and JMF specification requirements.

Figure 4.2 shows the aggregate gradation and their respective JMF gradation requirements on a 0.45 power chart. The aggregate gradations for both counties are within the Superpave specified control points. However, gradations for 'good' and 'bad' cores from Buncombe County, pass through the restricted zone.

4.6.2 Volumetric analysis results

The results for the volumetric tests are shown in Tables 4.5 and 4.6, and Figure 4.3. The asphalt content evaluated using the ignition furnace was found to be 5.2-percent and 5.3-percent for the 'good' and 'bad' mixes from Buncombe County, as opposed to the JMF requirement of 5.7-percent. For cores from Rutherford County, the asphalt content was found to be 6.0-percent versus JMF requirement of 6.2-percent. Overall, the asphalt contents are well within the tolerance of ±0.5-percent that is normally expected for in-situ mixtures and also as required by the NCDOT JMF specifications.

Air void contents for field cores from Buncombe County are slightly higher than the JMF specified values but are within the NCDOT specifications of 5±1-percent as shown in Figure 4.3. Moreover, the air void content after recompaction using 75 blows of Marshall hammer is also similar to the in-situ air void content. In case of Rutherford County, the in-situ cores air voids are 2-percent higher than the JMF values, and exceed the HDS specifications. The air void after recompaction, in general, are lower and within the specification limits.

The VFA and VMA requirements for both counties in general are within the NCDOT requirements of 60-75-percent and more than 15-percent, respectively, for the in-situ cores as well as the recompacted specimens.

4.6.3 Marshall stability and flow values

The stability and flow values for the in-situ cores and 75 blows recompacted Marshall specimens are presented in Tables 4.5 and 4.6, and Figures 4.4 and 4.5. For Buncombe County, the in-situ cores' stability is lower as compared to the JMF and HDS specification values. However, the stability of the recompacted cores is higher than the required JMF specification value of 3000 lbs. The flow values are within the required specification of 7 to 14 for both the in-situ cores as well as for the recompacted specimens.

For Rutherford County, although, the in-situ cores and recompacted specimens have higher stability compared to the specifications values, the flow values are higher than the requirement of 7 to 14. However, it may be noted that neither excessive rutting or delamination and shoving was observed in Rutherford County.

4.7 Conclusions

The results of the gradation, volumetric and stability analysis, indicated that the in-situ asphalt mixtures used in Buncombe and Rutherford counties were generally within the NCDOT mixture design specifications and should have performed well in-situ under normal traffic loading. Although, Rutherford County mixes show higher air voids content and flow values, no excessive distresses were observed in-situ. For Buncombe County, since the mixtures appear to be designed within specifications, the pavement sections were not expected to show any excessive distress. However, delamination and shoving has been a major problem in Buncombe County. Since the asphalt mix itself does not seem to be a contributory factor per-se, the baghouse fines and/or use of inappropriate tack coat might be a contributory factor to the distress observed in Buncombe County. The effect of baghouse fines and tack coat type are evaluated further in the following chapters.

Table 4.1 4" Marshall design specifications (NCDOT)

Requirement	HDS Limits	Buncombe JMF	Rutherford JMF
Number of Blows, each end (4-inch specimen)	75	75	75
Stability, Minimum (lbs.)	1500	3000	1900
Flow (0.01 in.)	7 – 14	11	9
Air Voids Content, (%)	4 – 6	4.9	5.0
Voids in Mineral Aggregate, VMA, (% Minimum)	15		
Voids Filled with Asphalt, VFA, (%)	60 – 75		
Retained Tensile Strength, TSR, (% Minimum)	85		
Bulk Specific Gravity		2.364	2.378
Maximum Theoretical Gravity		2.486	2.502

Table 4.2 Gradation analysis for skeleton aggregates

Sieve Size	Buncombe 'Good' (% Passing)	Buncombe 'Bad' (%Passing)	Rutherford (%Passing)
¾"	100	100	100
½"	100	100	100
3/8"	94	95	96
No. 4	67	67	74
No. 8	46	47	57
No. 16	33	34	46
No. 40	22	22	28
No. 80	13	13	14
No. 200	6.4	6.4	6.6

Table 4.3 NCDOT gradation limits for HDS mixes

Sieve Size	Recommended Percent Passing (wt.)	JMF Tolerance	Buncombe JMF	Rutherford JMF
¾"	100	0	100	100
½"	96 – 100	± 2	98	98
3/8"	85 – 100	± 5	95	95
No. 4	50 – 85	± 7	72	69
No. 8	33 – 67	± 5	50	53
No. 16	22 – 53	± 5	39	43
No. 40	9 – 40	± 5	24	24
No. 80	2 – 20	± 5	12	11
No. 200	2.0 – 8.0	± 2.0	5.0	5.9
AC Content	4.0 – 7.5%	± 0.5%	5.7	6.2
Mix Temperature	250 – 325^{o}F	± 15^{o}F	285^{o}F	285^{o}F

Table 4.4 Bulk specific gravity values

Buncombe County			Rutherford County		
Aggregate Material	**% by wt.**	**G_{sb}**	**Aggregate Material**	**% by wt.**	**G_{sb}**
#78 M	42	2.667	#78 M	47	2.623
Dry. Screenings	28	2.720	Screenings	33	2.802
Wet Screenings	30	2.764	Sand	20	2.582

Table 4.5 Volumetric and Marshall properties (Buncombe County)

Sample ID	Sample Type	AC (%)	Avg. Ht. (in.)	BSG	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	Flow (0.01 in)	Stability (lb.)
BG11	Orig.		2.47	2.382					10	1108
BG13	Orig.		2.44	2.371					12	1054
BG14	Orig.		2.49	2.400					15	1390
Avg.		**5.2**		**2.384**	**2.528**	**5.7**	**16.6**	**65.7**	**12**	**1180**
BB20	Orig.		2.48	2.370					13	1130
BB21	Orig.		2.60	2.377					15	1199
BB24	Orig.		2.47	2.372					13	1030
Avg.		**5.3**		**2.373**	**2.525**	**6.0**	**17.0**	**64.7**	**14**	**1120**
BG	Recomp		2.58	2.391					13	3374
BG	Recomp		2.60	2.376					13	3110
Avg.		**5.2**		**2.383**	**2.528**	**5.7**	**16.6**	**65.5**	**13**	**3240**
BB	Recomp		2.59	2.385					12	3376
BB	Recomp		2.63	2.383					10	3380
Avg.		**5.3**		**2.384**	**2.525**	**5.6**	**16.7**	**66.4**	**11**	**3380**
JMF	**Specs**	**5.7**		**2.364**	**2.486**	**4.9**			**11**	**3000**
HDS	**Specs**					**4 - 6**	**> 15**	**60–75**	**7-14**	**1500**

Table 4.6 Volumetric and Marshall properties (Rutherford County)

Sample ID	Sample Type	AC (%)	Avg. Ht. (in.)	BSG	G_{mm}	Air Voids (%)	VMA (%)	VFA (%)	Flow (0.01 in)	Stability (lb.)
RG01	Orig.		2.49	2.352					18	1753
RG02	Orig.		2.46	2.336					17	1628
RG03	Orig.		2.59	2.362					15	1896
Avg.		**6.0**		**2.350**	**2.527**	**7.0**	**17.3**	**59.5**	**17**	**1760**
RG	Recomp		2.60	2.369					14	4022
RG	Recomp		2.59	2.375					15	4429
Avg.		**6.0**		**2.372**	**2.527**	**6.1**	**16.5**	**62.9**	**15**	**4230**
JMF	**Specs**	**6.2**		**2.378**	**2.502**	**5.0**			**9**	**1900**
HDS	**Specs**					**4 - 6**	**> 15**	**60–75**	**7-14**	**1500**

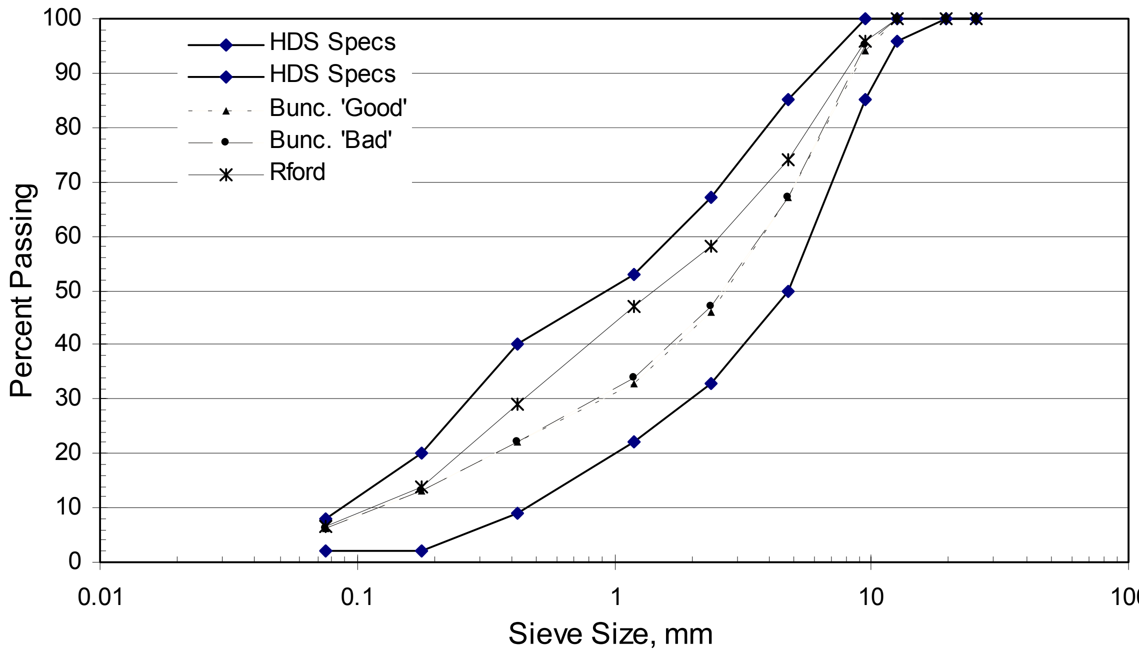

Figure 4.1 Core specimen gradations compared to NCDOT HDS specifications

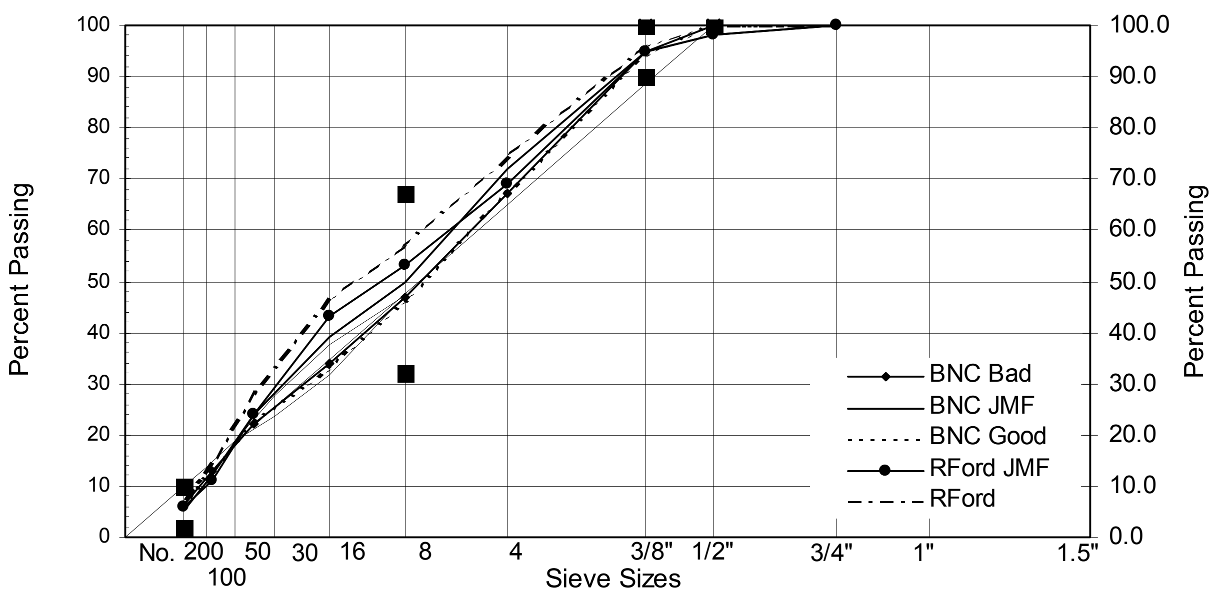

Figure 4.2 Gradations of cores from Buncombe and Rutherford Counties

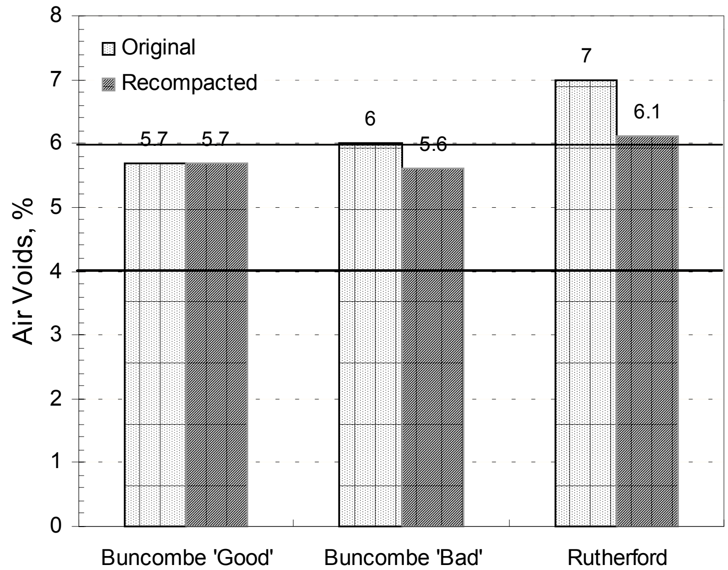

Figure 4.3 Air void contents for field cores and lab recompacted specimens

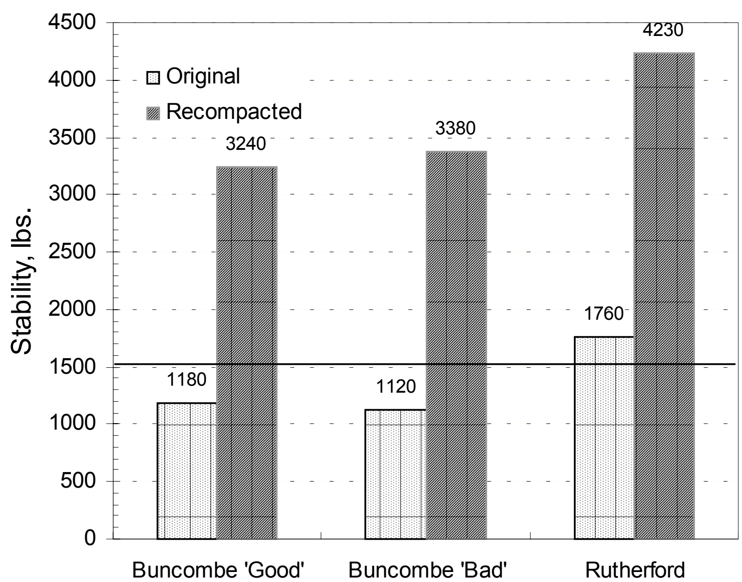

Figure 4.4 Marshall Stability for field cores and lab recompacted specimens

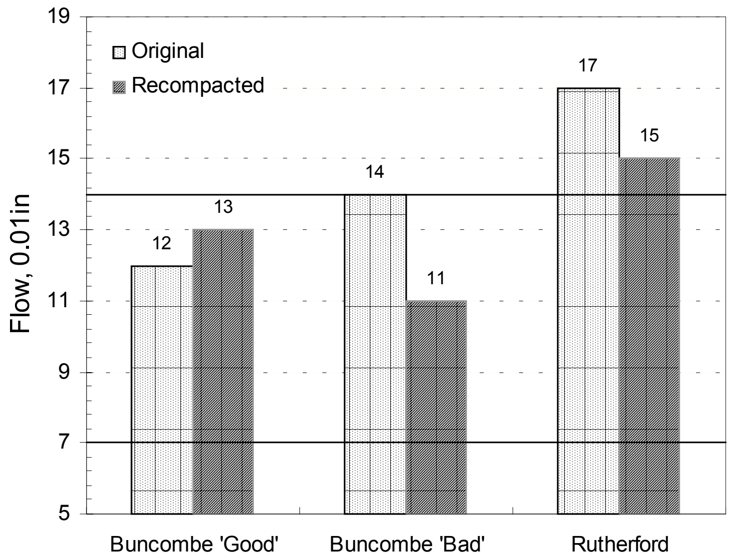

Figure 4.5 Marshall flow values for field cores and lab recompacted specimens

5. GRADATION ANALYSIS USING PARTICLE ANALYZER AND RHEOLOGICAL CHARACTERISATION OF MASTICS

5.1 Introduction

One of the concerns with respect to asphalt mixtures in NCDOT Division 13 was with regards to the variable amount of baghouse fines purged intermittently in the production of the field mixtures. This section deals with the evaluation of the gradation of baghouse fines using the state-of-the-art particle analyzer at FHWA Turner Fairbank Highway Research Center. The influence of fines on the rheological properties of asphalt cement using mastics was also evaluated using the DSR.

5.2 Selection of fines

For this study, the mineral filler and baghouse fines with materials passing #200 sieve were obtained from Buncombe and Rutherford counties. The fines selected for analysis were:

- Buncombe County:
    - passing #200 sieve fraction from wet screenings
    - baghouse fines
- Rutherford County
    - Passing #200 sieve fraction from dry screenings
    - 'fine' baghouse fines
    - 'coarse' baghouse fines.

In all, five different types of fines were analyzed using the particle analyzer.

5.3 Gradation analysis of fines using FHWA particle analyzer

5.3.1 Method description

The gradation analysis of mineral fillers was carried out at the Turner Fairbank Highway Research Center, McLean, Virginia. The material was separated using a small splitter then further separated using a micro splitter to obtain a representative sample. To run the test, a small amount of material was mixed in a medium to create a suspension. For baghouse fines, distilled water with 1-percent sodium hexametaphosphate was used. Experience showed that this was adequate for most of the mineral fillers.

The testing process consisted of taking a ‘blank’ measurement of the medium to establish a baseline. The material was then slowly introduced and mixed with the medium until an ‘optimum’ intensity was found. In order to prevent particle agglomeration, agitation by cavitation was induced by a high intensity ultrasonic processor for 2 minutes. The particle analyzer automatically converted different light intensity measurements into particle size distribution. The average of three different samples was obtained and the results were found to be consistent with each other. The results, however, necessarily need not match with the sieve analysis. The main reason for this is the fact that the gradation obtained from the laser particle analyzer is a volume gradation based on the projection of particles. The device showed the differences that were otherwise difficult to capture.

5.3.2 Results and discussion

In all two sets of analyses were performed within the duration of this study. Figures 5.1 and 5.2 show the gradations of the mineral fillers and bag-house fines obtained from the two counties. Figure 5.1 presents the particle analysis performed on the first set of fines received before the entire set of materials was requested so that a decision could be made about the further analysis to be performed. Figure 5.2 presents the particle size analysis that was carried out on fines received in the latter batch of materials from which the samples for laboratory testing were actually fabricated.

For Rutherford County, the ‘coarse’ baghouse fines appeared to be ‘sandy’ and hence their size distribution was measured using a slightly more viscous medium. The two media used for Rutherford County ‘coarse’ bag-house fines were distilled water (W) and 90-percent distilled water plus 10-percent glycerin (G). In Figure 5.1, the particle analysis results using the two different media are fairly different. However, both media show that this sample is the coarsest.

Figures 5.1 and 5.2 show that the ‘coarse’ baghouse fines from Rutherford County is the coarsest material passing #200 sieve. Although it was anticipated that the baghouse fines, in general, would be finer than the regular fines passing #200 sieve, the particle size analysis indicated for both set of materials that the baghouse fines from both counties had more or less similar particle size distribution as the regular fines. Table 5.1 shows the properties of the fines based on the second set of materials which was used for mixture performance testing in this study. The mean particle size of regular fines from Buncombe and Rutherford counties are both 30.5-μm. The baghouse fines from Buncombe County has the mean particle size 40-μm and the Rutherford County 'fine' baghouse fines has a mean particle size of 31-μm. The 'coarse' baghouse fines from Rutherford County has a mean particle size of 88-μm, more than twice the mean size of any other regular or baghouse fines.

The particle size analysis suggests that based on gradation, the mixture performance should not be affected whether the mineral filler used is regular fines or from baghouses. However, as the particle shape and/or perhaps the mineral content distribution may be different for the baghouse fines, it may have different effect on the rheological behavior of the asphalt cement and mixtures in general. This effect is investigated in the following section where the performance of baghouse and regular fines are studied using asphalt-fines mastics in an aged and unaged condition. In Chapters 6 and 7, this effect is also evaluated based on the performance of the asphalt mixtures.

5.4 Analysis of asphalt-fines mastics using DSR

The objective of this task is to evaluate the rheological properties of the binders and mastics containing baghouse fines at intermediate to high temperatures. Testing was conducted in accordance with AASHTO TP5-93.

5.4.1 Specimen preparation

The asphalt cement used for the preparation of mastics was a PG64-22 supplied by NCDOT. Four mastics were prepared using the fines received from Buncombe and Rutherford counties. The asphalt cements and each of the mastics were tested in an unaged and aged condition with at least three replicates. The following asphalt and mastic combinations were used:

- Binder (virgin and RTFO-aged, PG64-22 for both counties)
- Mastic (virgin and aged) from the baghouse fines. Only the 'fine' baghouse fines was used from Rutherford County.

- Mastic (Aged and virgin) from the regular mineral filler (fraction passing #200 sieve).

The prepared mastic was a mixture of the filler/fines with asphalt from the corresponding county. The proportion of asphalt was 50-percent by weight of the total mastic. For preparation of mastics, the fines and the asphalt binder were pre-heated to a temperature of 160^{o}C. The fines were then added slowly to the heated binder with constant stirring until a uniform, consistent agglomerate free 'batter' was obtained.

The asphalt binders were aged in an RTFO oven, while the mastics were aged in an air draft oven due to the higher consistency of the mastics. The mastics were poured in a PAV dish and were kept in an oven for a period of 85 minutes at 163^{o}C for aging. One problem encountered with handling the mastics was segregation of fines. Hence while pouring into the molds, it was necessary to stir vigorously so as to have a uniform consistency for the DSR specimens.

5.4.2 Test parameters and type

The asphalt cement and mastics were evaluated at three temperatures – 58^{o}C, 64^{o}C, and 70^{o}C. For the asphalt binder, the DSR spindle diameter of 25-mm with 1-mm gap was used. As the mastics were more viscous, a spindle diameter of 8-mm with 2-mm gap was used.

Testing at 10 rad./second and at different strain levels (strain sweep) was conducted to establish for each asphalt binder and mastic, the linear viscoelastic region in the strain domain (defined by AASHTO TP5 to be a range of strain values where the test measurement |G*| does not vary more than 95-percent of the |G*| estimated at zero strain). Based on the strain sweep test, the strain levels for unaged and aged binders selected was 12 and 10-percent, respectively. For the mastics, the strain levels selected were typically in the range of 1 to 2-percent for both aged and unaged binders.

Frequency sweep tests were conducted on the asphalt binders and mastics at the frequencies of 0.01, 0.05, 0.1, 0.15, 0.5, 1.0, 1.59, 5.0, 10.0 and 20.0 Hz. The measured parameters from the test results were the dynamic shear modulus $|G^*|$ and phase angle δ. These results were then used to generate master curves at 64°C for dynamic shear modulus $|G^*|$ and $|G^*|/\sin\delta$.

5.4.3 Test results and discussion

Figures 5.3 and 5.4 show the dynamic shear modulus as function of reduced frequency at 64°C for Buncombe and Rutherford counties. These figures show that in general:

1) aging tends to increase the $|G^*|$ values for both the asphalt binder and the mastics as compared to the unaged condition;
2) $|G^*|$ values for the mastics are much higher than the asphalt binder (this trend is expected as the mastics contain 50-percent fines);
3) in case of Buncombe County, the mastic containing baghouse fines have higher $|G^*|$ values over the range in frequency as compared to the mastic containing regular mineral filler material;
4) in case of Rutherford County, the mastic containing baghouse fines show similar $|G^*|$ values over the range in frequency as compared to the mastic containing regular mineral filler material.

Figures 5.5 through 5.8 show the Superpave rutting parameter $|G^*|/\sin\delta$ as function of reduced frequency at 64°C. For Buncombe County, mastic containing baghouse fines shows higher $|G^*|/\sin\delta$ values both in aged and unaged condition as compared to the mastic containing regular fines. For Rutherford County, the mastic containing baghouse fines shows higher $|G^*|/\sin\delta$ values in unaged condition but about similar values in aged condition compared to the mastic containing regular fines.

5.5 Conclusion

It was originally hypothesized that the one of the contributory factor to the delamination and shoving distress observed in Buncombe County was the intermittent purging of the baghouse fines in the field asphalt mixes. Results of the gradation analysis using the particle analyzer show that the baghouse fines have similar or in some cases coarser gradation as compared to the regular mineral filler used in these respective counties. The dynamic mechanical analysis of the mastics using the DSR suggests that inclusion of baghouse fines in asphalt mixtures may not have any detrimental effect. On the contrary, for Buncombe County, the inclusion of baghouse fines appears to enhance the rut resistance of the asphalt mixture as will be shown to be the case in Chapter 6. However, the effect of moisture sensitivity on the mixtures containing baghouse fines needs to be evaluated before any final conclusion can be made.

Table 5.1 Properties of fines from particle size analysis (set 2) [5]

Particle Property	**Buncombe Baghouses**	**Buncombe Passing #200**	**Rutherford 'Coarse' Fines**	**Rutherford 'Fine' Fines**	**Rutherford Passing #200**
Fineness Modulus (F.M.)	5.72	5.40	7.10	5.43	5.31
Coefficient of Uniformity (C_U)	10.20	7.60	4.00	9.11	8.67
Coefficient of Curvature (C_C)	1.73	2.15	1.72	1.96	2.28
Skewness Indicator (σ_1)	2.25	2.31	1.14	2.60	2.30
Skewness Indicator (σ_2)	4.40	2.90	2.75	3.88	3.59
Mean Particle Size (μm)	40.0	30.5	88.0	31.0	30.5

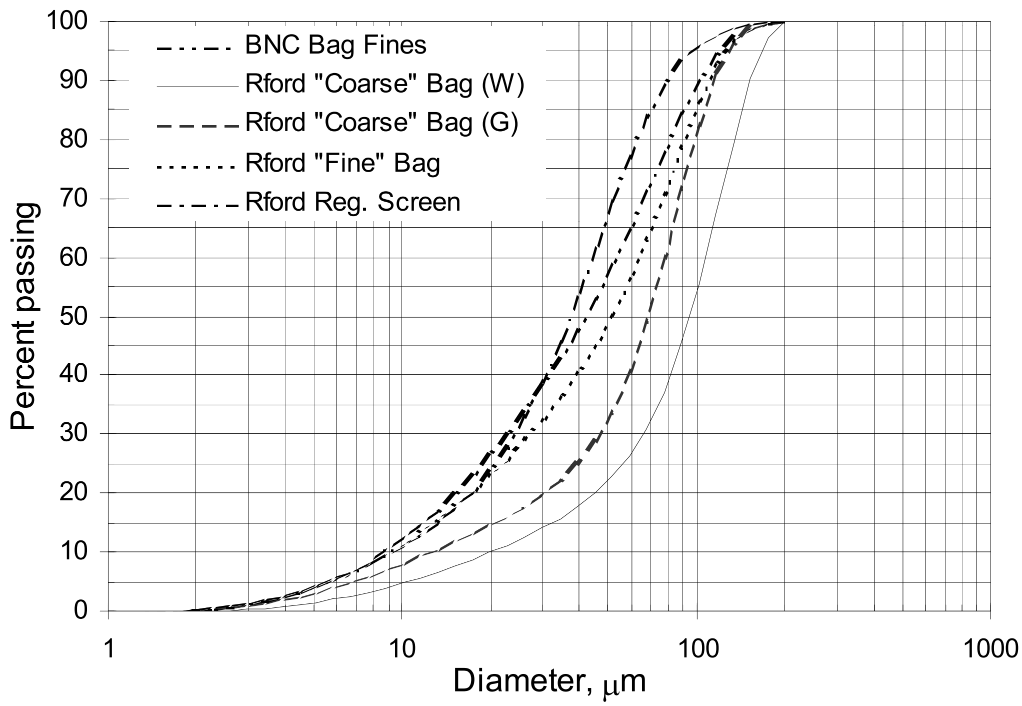

Figure 5.1 Gradation analysis of fines using FHWA particle analyzer, set 1

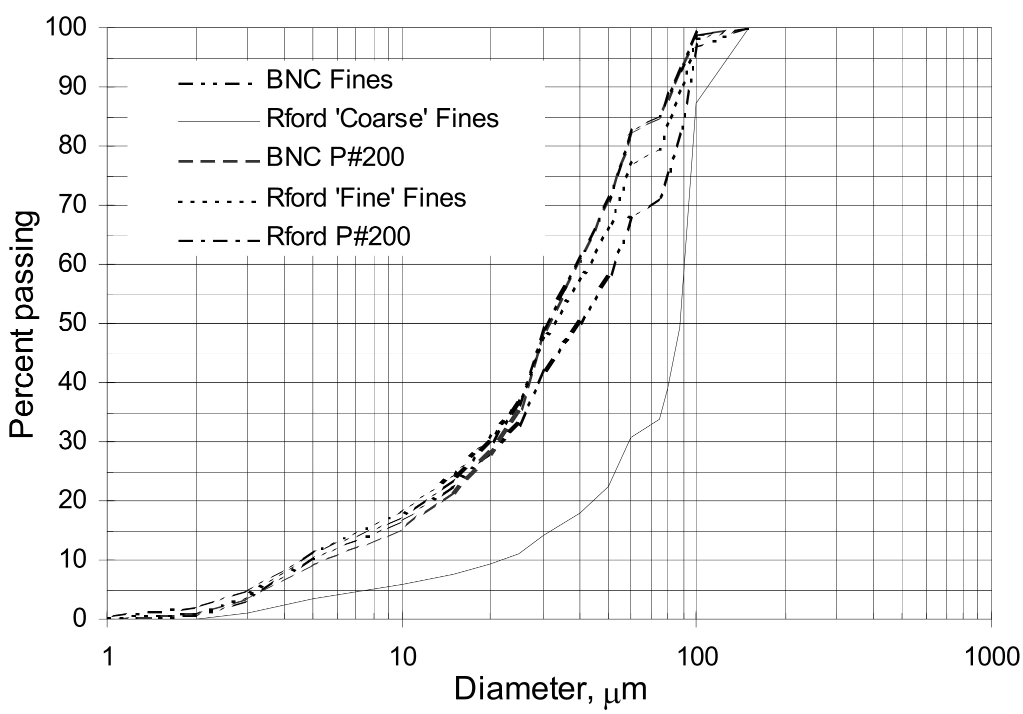

Figure 5.2 Gradation analysis of fines using FHWA particle analyzer, set 2

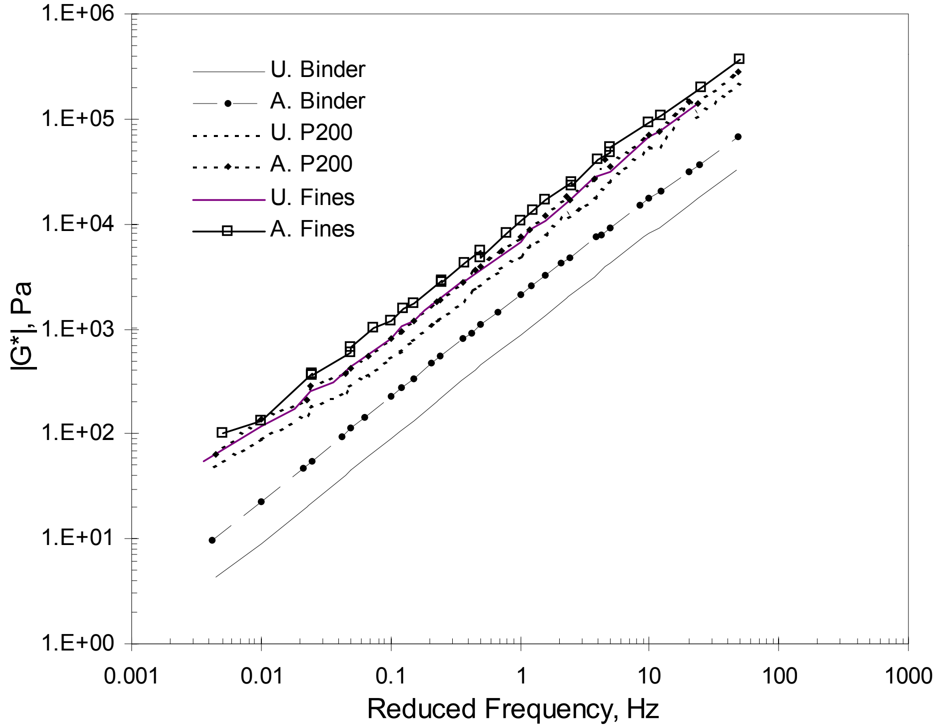

Figure 5.3 Master curves for Buncombe County, 64°C

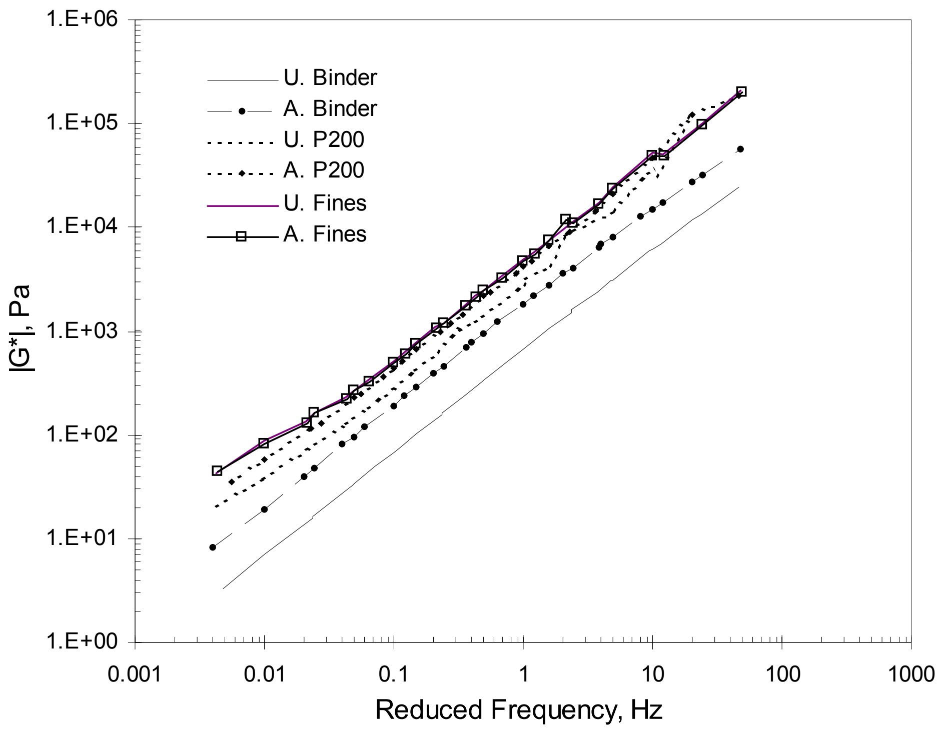

Figure 5.4 Master curves for Rutherford County, 64°C

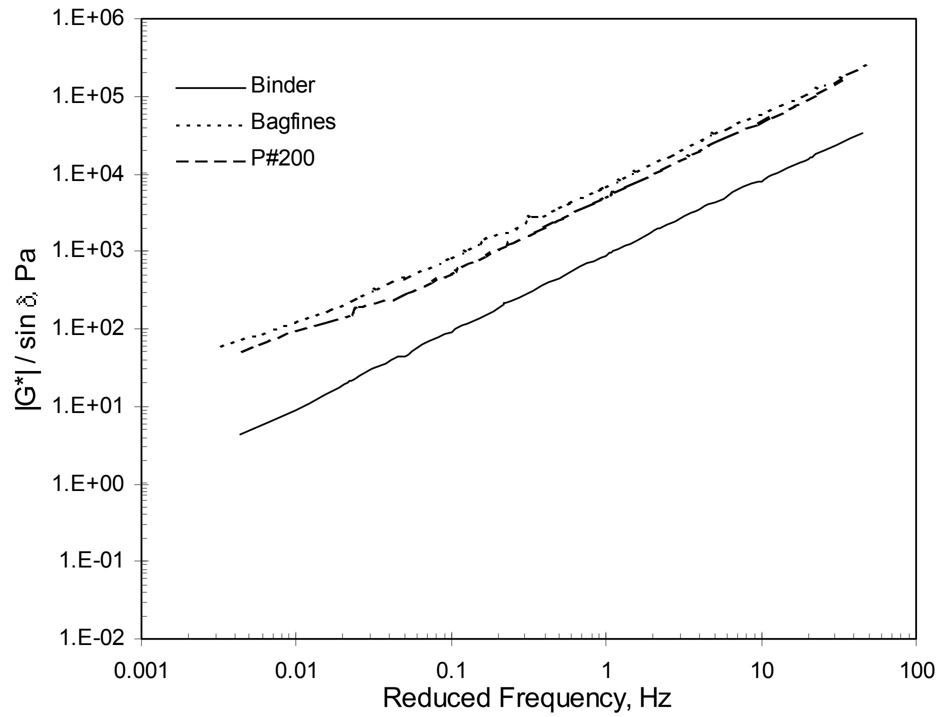

Figure 5.5 Master curves (|G*|/sin δ) for Buncombe County, unaged, 64°C

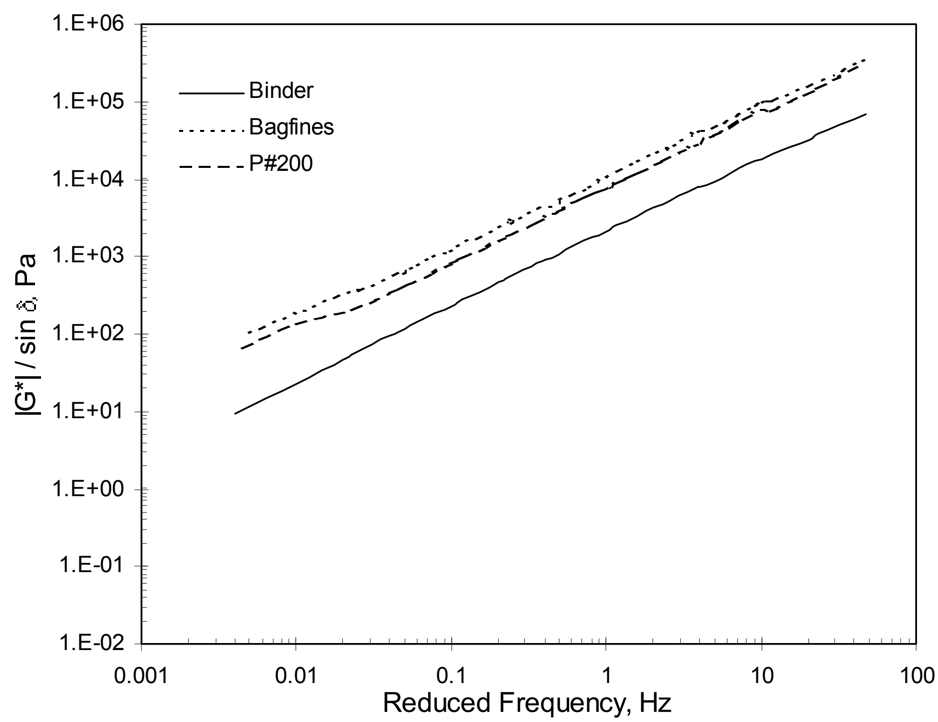

Figure 5.6 Master curves (|G*|/sin δ) for Buncombe County, aged, 64°C

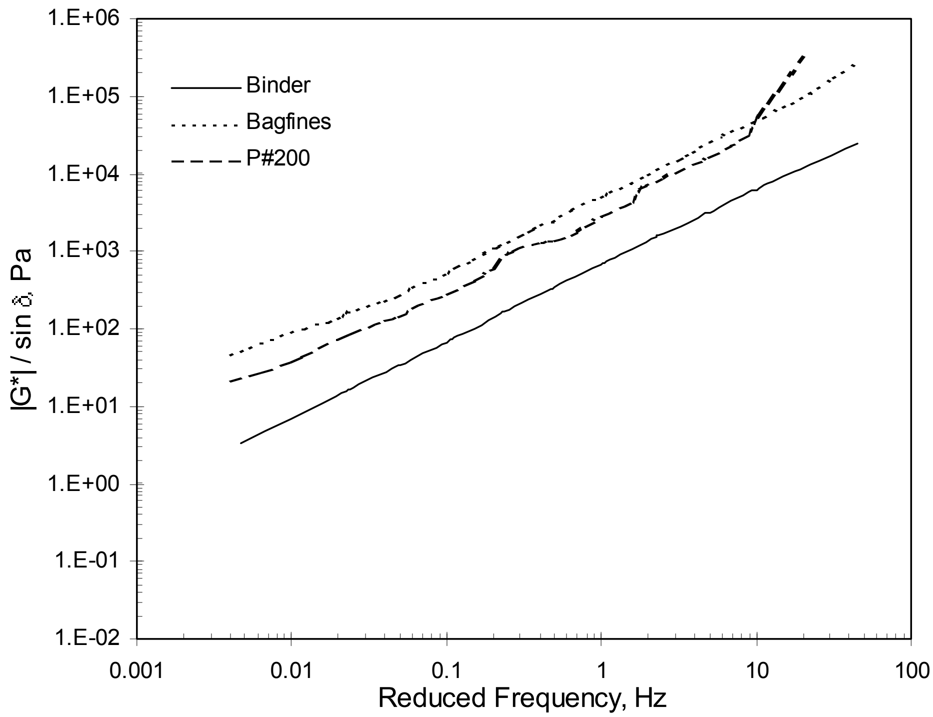

Figure 5.7 Master curves (|G*|/sin δ) for Rutherford County, unaged, 64°C

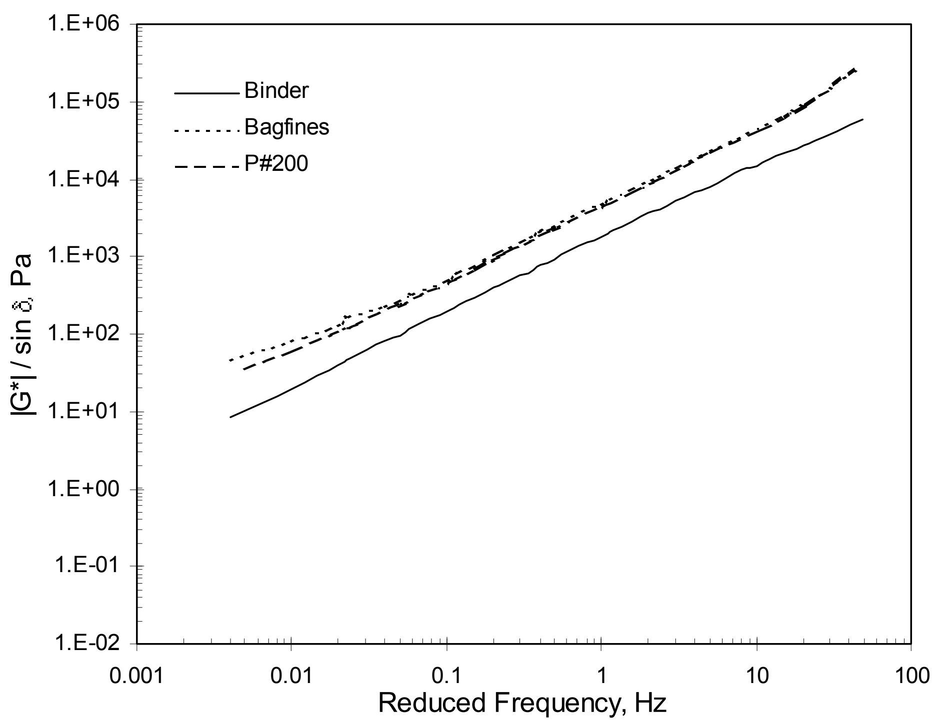

Figure 5.8 Master curves (|G*|/sin δ) for Rutherford County, aged, 64°C