6.0 PERFORMANCE TEST RESULTS FOR FIELD CORES AND LAB MIXES

6.1 Introduction

The main objective of this task was to evaluate the performance of the field cores subjected to the traffic loading and the laboratory mixes. In this section, the bond strength of the field cores and the performance of the laboratory mixes compacted using the Superpave Gyratory Compactor (SGC) was evaluated using the state of the art SHRP (Strategic Highway Research Program) tests. Test methods utilized for measuring the performance were Shear Frequency Sweep at Constant Height (FSCH) Test, Repeated Shear at Constant Height (RSCH) Test, Uniaxial Tensile Test (VRAMP) and Axial Frequency Sweep Test (AFST). Testing parameters, test description, methodology, and results are presented in the following sections.

6.2 Test parameters

For a given test system, the results of the performance test are governed by several parameters including reliability and repeatability of the test system, and the mix and test parameters. For the mix parameters, the asphalt type and content, and the aggregate type and gradation was fixed based on the job mix formula for the given pavement section. The only mix parameter that varied was the air void content of the cores and laboratory mixes. Tables 6.1 and 6.2 shows the air void contents of the field cores used for the shear and uniaxial tests, respectively. The average air void contents for the field cores were 6.7-percent and 7.2-percent for the cores from Buncombe and Rutherford Counties, respectively. Table 6.3 shows the air void content of the laboratory prepared specimens with and without the baghouse fines. For these mixes, the air void content of 5±0.5-percent was targeted based on the JMF requirement of 5-percent voids.

The major test parameters considered in this study were: 1) test temperature, 2) applied stress or strain, 3) test frequency, and 4) test duration. As per the research methodology presented in Chapter 2, the following tests were conducted on the cores and laboratory mixes from both the counties with testing broadly classified in the following two categories:

- Shear Testing

FSCH (Frequency Sweep at Constant Height Test)

RSCH (Repeated Shear at Constant Height Test)

- Axial Testing

  AFST (Axial Frequency Sweep Test)

  VRAMP (Vertical Ramp Test – Uniaxial Tensile Test)

The shear testing consisted of a shear frequency sweep at constant height (FSCH) and repeated shear at constant height (RSCH) tests, whereas the axial testing consisted of uniaxial frequency sweep test (AFST) and a uniaxial tensile test (VRAMP). Each of these test methods is described in the latter sections. The field core samples were sawed to a height of 50±2-mm with precautions that the bonded interface between the two layers of interest was at the mid-height. Laboratory specimens 150-mm in diameter were compacted using the SGC and were sawed to the required height. No axial testing was conducted on these laboratory specimens.

6.3 Test temperature

6.3.1 Selection of testing temperature

Temperature plays an important role in the design of asphalt mixes. The properties of binder depend significantly on the temperature and, consequently, the mix properties such as resistance to rutting and fatigue vary with temperature. In order to evaluate the load associated performance of the pavement it is imperative that the testing be carried out at a proper temperature representing the actual field conditions. One of the procedures for determining the pavement temperatures is recommended by AASHTO TP7 - Procedure F (Repeated Shear at Constant Height Test) [1]. This procedure requires conducting the RSCH test at the maximum seven-day pavement temperature at the selected pavement depth. The recommended depth at which the maximum seven-day pavement temperature is calculated is 20-mm from the top surface. The data for this temperature is normally obtained from the weather data at the paving site using the SHRPBIND program [19] developed within the SUPERPAVE™ program.

6.3.2 Temperature zones

SHRP report (SHRP-A-415) [18] outlines an elaborate procedure for computing the critical and maximum pavement temperatures. It has divided the continental United States into nine climatic regions based on the temperature and humidity of the soils. The nine temperature zones are shown in Figure 6.1. Table 6.4 lists the effective, maximum, and critical temperatures for the nine zones as reported in SHRP-A-415 [18]. The effective temperature is the temperature at which loading damage accumulates at the same average rate in service as in laboratory. Thus, there is a one-to-one correspondence between the laboratory and in-service loading cycles at the effective temperature. The critical temperature is the temperature at which the maximum amount of damage occurs in service. This temperature can be considered as an ideal temperature for laboratory testing because it minimizes errors due to variations in the mix temperature sensitivity due to its accelerated rate of damage accumulation. North Carolina falls in regions IB and IC with both Buncombe and Rutherford counties being in region IC with critical and maximum temperatures in the range of 35 to 38^{o}C.

6.3.3 Selection of depth for computation of testing temperature

The job mix formulae for the mixes from both the counties indicated that there were two 50-mm lifts HDS course. Ideally, the testing temperatures for mixes from both the counties should have been 38^{o}C, but the actual layer thickness' are much lower than 50-mm. The average depths to the uppermost interface measured from the core surfaces are summarized in Table 6.5 for both the counties. Since the parameter under investigation was the tack coat properties, it was necessary that the laboratory test temperature corresponded to that of the tack coats in the field. Consequently, testing temperatures were selected corresponding to the depth of the tack coat (approximately 33-mm for both counties).

6.3.4 Reliability factors

AASHTO provisional standard TP-7 [1] specifies that the RSCH test be conducted at the maximum seven-day pavement temperature for the selected depth. However, it does not specify the reliability level at which this temperature should be computed. A reliability level of 50-percent was selected for this study.

6.3.5 Temperature selection method

The seven-day maximum air temperatures were computed based on the following equations used within the SHRPBIND [19] software:

$$T_{surf} - T_{air} = -0.00618 \times (lat.)^2 + 0.2289 \times (lat.) + 24.4 \qquad (6.1)$$

Where T_{surf} and T_{air} are the air and surface temperatures respectively in degree Celsius and *lat.* is the latitude in degrees. From the surface temperature, the pavement temperature is computed using:

$$T_d = T_{surf} \times \left(1 - 0.063 \times d + 0.007 \times d^2 - 0.0004 \times d^3\right) \qquad (6.2)$$

Where T_d and T_{air} are the temperatures at depth d and at surface, respectively, in ^{o}F with the depth, d, in inches. In this study, the pavement temperatures were calculated by two different ways. In the first method, the temperature was calculated at the required depth from the air temperature using Equations 6.1 and 6.2. In the second method, the pavement temperature was calculated using the SHRPBIND program. It was found that the temperatures calculated by the two different methods differed by approximately 3^{o}C. Hence, an average of the two was taken as the critical test temperature. Table 6.5 summarizes the temperatures calculated by the two methods. The output from the SHRPBIND program is enclosed in Appendix C.

Based on an average value, the testing temperatures for Buncombe and Rutherford counties were 50.2°C and 54.0^{o}C, respectively at 33-mm depth and 50-percent reliability. However, in order to compare different tack coats (CRS-2 and PG64-22) a single test temperature of 50.2^{o}C was selected.

6.4 Performance test results for field cores

6.4.1 Introduction

Objectives of this task were to evaluate the bond strength and performance of the field cores containing different tack coats – CRS-2 emulsion for the cores from Buncombe County, and PG64-22 binder for the cores from the Rutherford County. Field cores description, volumetric and stability analysis was presented in Chapter 4. Performance tests description and test results are described in the following sections.

6.4.2 Frequency sweep test at constant height (FSCH)

6.4.2.1 Test Description

The FSCH test measures the viscoelastic shear properties (dynamic shear modulus, |G*| and the phase shift, δ) over a range of testing frequencies and at different temperatures. Testing is conducted in a semi-confined condition in which the specimen dilation due to application of shear load is prevented by an axial force – hence, the acronym "constant height" test.

In this study, testing was conducted in accordance with AASHTO TP7 Procedure E [1] in which a sinusoidal shearing strain of amplitude ±0.005-percent (0.0001 mm/mm peak-to-peak strain) was applied at frequencies of 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz. At each frequency, the stress response is measured along with the phase shift between the stress and strain. The dynamic shear modulus (|G*|) was computed as the ratio of the peak stress over the peak strain. It should be noted that in this study, the measured dynamic response of the core is a composite response of the two asphalt layers separated by a thin film of tack coat. For this reason the |G*| value is termed as the 'dynamic composite shear modulus.' Testing was conducted at 50.2°C.

6.4.2.2 Test results and observations

Tables 6.6 to Table 6.11 show the FSCH test results for the field cores from Buncombe and Rutherford counties. These results are graphically presented in Figure 6.2 through Figure 6.7. It should be noted that the prefixes "BG and BB" refer to the field cores from the "good and bad" pavement sections from Buncombe County; and "RG" and "RB" refer to the "good" and "bad" pavement sections from Rutherford County.

The measured parameters from the FSCH test were the |G*| and δ values of the composite cores at 50.2^{o}C test temperature for both counties. From Figures 6.2 through 6.4 it may be observed that the cores from 'good' Buncombe County pavement section have higher |G*| values compared to their counterparts from 'bad' section. The average difference over all frequencies based on |G*| and |G*|/sinδ values (Table 6.8) between the good and bad sections is 40 and 54-percent (percentages based on higher value), respectively.

For Rutherford County, Figures 6.5 through 6.7 and Table 6.11 show that the differences in the |G*| and |G*|/sinδ values between the good and bad sections is not as pronounced as was the case for the Buncombe County. These differences are only 5 and 11-percent, respectively.

In Chapter 4, it was stated that there was not much visible difference between the so-called good and bad field cores for Rutherford County. This observation is in agreement with the shear frequency sweep test results. For the Buncombe County, the volumetric and stability test results presented in Chapter 4 indicated that the differences in the air voids and Marshall stability of the good and bad field cores were minor. However, the flow value of the good cores was 12 as compared to 14 for the bad cores. Nevertheless, the percentage difference in the flow value is only 14-percent.

The lower |G*| and |G*|/sinδ values for the field cores from bad performing pavement section in Buncombe County, is clearly indicative of the pavement sections susceptibility to rutting and/or shoving distresses. However, it should be noted that at this juncture, the failure mechanism can not be identified, i.e., is the deficiency due to tack coat or the mixture? Interestingly, it may be noted that based on the results presented in Tables 6.8 and 6.11, the results for the good field cores from Buncombe County are very similar to those from the Rutherford County (good and bad).

6.4.3 Repeated shear at constant height (RSCH) test

6.4.3.1 Test Description

The RSCH test measures the rutting potential of the mix over a range of temperatures. In this study, the RSCH test was conducted in accordance with AASHTO TP-7, Procedure F [1]. A controlled cyclic haversine shearing stress was applied for a period of 0.1 s followed by a rest period of 0.6 s with a peak shear stress of 68 ± 5 kPa. The test duration was defined to correspond with permanent shear strain accumulation of 5-percent, or 100,000 loading cycles. The measured response was in terms of permanent shear strain accumulation as function of the number of loading cycles.

6.4.3.2 Test results and observations

The RSCH test results are shown in Figures 6.8 and 6.9 for the Buncombe and Rutherford counties, respectively. From these data, Tables 6.12 and 6.13 were prepared which show the performance comparison in two formats. Table 6.12 shows the number of load cycles corresponding to 5-percent terminal shear plastic strain. It can be observed that for Buncombe County the good cores reached failure at an average of 41,000 loading cycles compared with 6900 cycles for the bad cores — a difference of 83-percent. It may be noted that these results are consistent with the results of the FSCH test where the $|G^*|$ and $|G^*|/\sin \delta$ values were higher for the cores which endured more loading cycles.

Examination of the failed core samples showed a distinct pattern of cracking with diagonal cracks in both upper and lower asphalt concrete layers with a horizontal crack joining the diagonal cracks indicating distinct failure in the interface (tack coat) layer. Note that for these cores CRS-2 emulsion was used as a bonding agent.

For the Rutherford County, all core samples show higher number of cycles to failure compared with the cores from Buncombe County, a difference of 72-percent. However, for the Rutherford County, the good field cores performance is lower as compared to the bad cores, confirming earlier observations that there is really no difference between these cores. In essence, RSCH test results are in line with the FSCH test results as well as field observations where no significant differences were noted for the good and bad cores.

For the Rutherford County cores that failed before reaching 100,000 loading cycles, the crack formation was unlike that observed for the Buncombe County cores — diagonal cracks from top to bottom as shown in Figure 6.10 — a pattern consistent with those observed in monolithic single layer specimen. For the failed sample, no separation or a horizontal crack at the interface layer was evident. Note that for Rutherford County cores, PG64-22 asphalt cement was used as a bonding agent.

Table 6.13 shows the permanent shear strain at 5,000 loading cycles. Consistent with earlier observations for the Buncombe County, the good cores have lower accumulated shear plastic strain compared to the bad cores – a difference of 38-percent. For Rutherford County, there is a difference of 14-percent between the performance of the good and bad cores with good cores showing a slightly higher accumulated plastic shear strain.

6.4.3.3 Conclusion

The RSCH test results are in agreement with the earlier FSCH test results. For Buncombe County, a distinct difference between the good and bad cores was noted. Horizontal cracking in the core samples at the interface was evident indicating failure in CRS-2 tack coat. For Rutherford County, failure of core samples were more in line with those that may be anticipated for monolithic specimens. No cracking of the PG64-22 tack coat interface was noted in any of the specimens. It may, therefore, be concluded that PG64-22 binder appears to provide a more effective bonding compared to the CRS-2 emulsion.

6.4.4 Axial frequency sweep test (AFST)

In the previous two sections, the performance of the composite field cores was evaluated using the FSCH and RSCH test. In those tests, the performance of the tack coat was an integral part of the total response of the composite core and could not be separated from the mix response. In the following sections, field cores from both counties were tested in uniaxial tensile mode that, perhaps, tests the contribution of the tack coat more directly.

6.4.4.1 Test description

The AFST test is similar to the FSCH test except that a dynamic uniaxial loading is applied as opposed to shear. Although taller samples would have been ideal in this case, the test was carried out with cores 50-mm in height. This test was conducted in a controlled strain mode of loading with a sinusoidal axial strain of amplitude ±0.005-percent (0.0001 mm/mm peak-to-peak strain) applied at 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02 and 0.01 Hz frequencies. At each frequency, the stress response was measured along with the phase shift (δ) between the stress and strain and the dynamic axial composite modulus (|E*|) was computed as the ratio of the peak-stress over peak-strain.

6.4.4.2 Results and observation

Tables 6.14 to Table 6.17 and Figures 6.11 to 6.14 show the AFST test results for the cores from Buncombe and Rutherford Counties. From Figures 6.11 and Figure 6.13 it can be seen that the response of the mix to the sinusoidal axial loading is very similar to the response obtained during the shear frequency sweep testing (FSCH), with |E*| values showing similar pattern that was obtained for the |G*| values for both counties.

Table 6.18 shows the average values of |E*| for both counties. For the Buncombe County, the average |E*| values for 'good' cores are 56-percent higher than the average |E*| values for 'bad' cores. It may be noted that this result is consistent with the 40-percent difference observed in FSCH test. For Rutherford County, the difference between the 'good' and 'bad' cores is only 18-percent. More importantly, the average difference between the performance of the Buncombe (CRS-2 tack coat) and Rutherford (PG64-22 tack coat) County cores is 40-percent which is a clear indication of the higher bond strength provided by the PG64-22 tack coat over CRS-2 emulsion.

6.4.4.3 Conclusion

The AFST test results are in agreement with the FSCH test results. The average value of |E*| for good Buncombe County cores was much higher than bad cores. However, the average value for the Buncombe County cores is 40-percent lower than the Rutherford County cores. Based on these results and the results of the shear frequency sweep and repeated loading tests, it is expected that the bond strength of the cores tacked using PG64-22 binder will be higher than the cores tacked using CRS-2 emulsion. The comparison of the bond strength using the uniaxial tension test is discussed in the next section.

6.4.5 Uniaxial tensile test (VRAMP)

6.4.5.1 Test Description

Uniaxial Tensile Test is a controlled strain test in which the core sample is pulled apart axially at a rate of 2.5 mm/minute [24]. The measured parameters are the axial load as function of displacement (and time) with the strength defined as the stress corresponding to the peak load. For layered composite core specimen, if the mix is stronger than the tack coat, it is expected that the tensile failure will occur due to the failure of the joint. Therefore, this test is a measure of the tensile strength of the interfacial joint or bond strength.

6.4.5.2 Results and observations

Tables 6.19 and 6.20 show the results at peak load for Buncombe and Rutherford counties. The axial stress and strain as function of time is shown in Figures 6.15 through 6.18 for Buncombe and Rutherford counties. It can be seen that the average strength for ‘good’ and ‘bad’ cores from Buncombe County is 28.5 kPa and 22.8 kPa, and 56.6 kPa and 38.4 kPa for Rutherford County, respectively. The average difference between the Buncombe County and Rutherford County cores is 46-percent indicating that the PG64-22 provides a much stronger bond compared to the CRS-2 emulsion, a result consistent with not only the AFST test but also with the shear tests.

Figures 6.20 through 6.24 show the failure mechanism for the cores tested from Buncombe and Rutherford counties. From Figures 6.20 to 6.22 it is clear that for Buncombe County, the failure is through the CRS-2 tack coat at the interface of the two layers. For Rutherford County (Figure 6.23 and 6.24), the failure was observed to occur within the mix, and not at the interface indicating that PG64-22 provided bond strength stronger than the mix strength.

6.4.5.3 Conclusions

The VRAMP test results clearly show the contribution of the tack coat in relation to that of the mix. For the Buncombe county cores where a CRS-2 emulsion was used as tack coat, failure occurred at the interface layer indicating a weak interface bond in relation to the mix strength. For Rutherford County, the failure was observed to be in the core sample itself indicating that PG64-22 as a tack coat provided a more stronger bond in relation to the mix.

Considering that both the Buncombe and Rutherford County mixes are within the NCDOT specifications, it may be concluded that the PG64-22 binder as a tack coat provides a better interfacial bond compared to the CRS-2 emulsion.

6.5 Performance test results of lab mixes containing baghouse fines

The objective of this task was to evaluate the effect of baghouse fines on laboratory mixes. Laboratory specimens were fabricated using the SGC. Performance of the specimens containing baghouse fines versus crushed mineral filler (passing #200 sieve) was evaluated using the FSCH and the RSCH tests.

6.5.1 Specimen fabrication

The specimens for this task were fabricated at NCSU materials laboratory. All the specimens were fabricated using the SGC (Superpave gyratory Compactor). The raw materials received from NCDOT were separated into various fractions depending on their sieve sizes and were then blended to the appropriate NCDOT specified JMF gradations. The exception to this procedure was that the Rutherford County sand and all the baghouse fines, were added in bulk as received. Specimens with zero percent baghouse fines were fabricated with mineral filler (fraction passing #200 sieve) whereas, specimens with 100% baghouses had their fraction passing #200 sieve substituted completely by the baghouse fines. For Rutherford County, there were two types of baghouses: the 'fine' baghouse fines and the 'coarse' baghouse fines. For the purpose of laboratory testing, only the 'fine' baghouse fines were used. The asphalt contents for Rutherford and Buncombe Counties were 6.2 and 5.7-percent, respectively, and the non-strip additive requirement was 0.5-percent for both the counties.

The mixing and compaction was carried out at a temperature of 285^{o}F, and before compaction, the mixes were aged at a temperature of 275^{o}F for 2 hours. The 6-inch diameter RSCH test specimens were compacted to a height of approximately 3-inches with target air voids of 5±1-percent. Both ends were then sawed to achieve the required height of 2-inches. Table 6.3 shows the air voids content of the specimens used for the RSCH test. It may be noted that the specimen identification for Buncombe and Rutherford counties consists 'W' for the specimens containing baghouse fines, and 'WO' for specimens without the baghouse fines.

6.5.2 Performance test results for laboratory specimens

The laboratory compacted specimens with and without the baghouse fines were first subjected to the FSCH test described earlier in this chapter. Following the FSCH test, these specimens were then subjected to RSCH test to evaluate the mixture resistance to rutting. Testing was conducted at 50.2°C.

Tables 6.21 through 6.26 and Figures 6.24 through 6.29 show the FSCH test results for the mixtures from Buncombe and Rutherford counties. Based on these figures, and in particular Tables 6.23 and 6.26 it may be noted that the baghouse fines has a stiffening effect on the mixtures. That is, on an average, specimens containing baghouse fines have higher shear modulus values $|G^*|$ and $|G^*|/\sin\delta$ compared to those specimens without the baghouse fines. The percentage difference is approximately 30-percent for the Buncombe County mixes and 20-pecent for the Rutherford County mixes. These results are consistent with the results obtained for the mastics using the DSR presented in Chapter 5. Moreover, for both mixes with and without the baghouse fines, the Buncombe County mixes generally show very similar performance to the Rutherford County mixes for the air voids and test temperature used in this study. Based on these results, it is expected that rutting performance will also be in line with the results obtained from FSCH test.

Table 6.27 and Figures 6.30 to 6.31 show the RSCH test results. These tests were conducted to 100,000 loading cycles. The accumulated plastic shear strain at 100,000 cycles shown in Table 6.27 confirm the results from FSCH test: 1) for both counties specimens containing baghouse fines show lower accumulated plastic shear strain compared to specimens without the bag house fines with a percentage difference of approximately 15-percent; and 2) respective mixes from Buncombe and Rutherford counties show similar performance. As the accumulated plastic strain for all mixtures are less than 5-percent, it is expected that these mixtures should not show in-situ accumulated rut depth more than 0.5-inch under normal traffic loading.

6.6 Summary and conclusion

The main objective of this task was to evaluate the performance of field cores and the laboratory mixes containing baghouse fines. Of particular interest was 1) the effect of baghouse fines on mixture performance, and 2) the bond strength of the CRS-2 emulsion and PG64-22 tack coats.

FSCH and RSCH test results for laboratory mixes containing baghouse fines show the following:

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 not containing baghouse fines;
3. Respective mixtures from both counties show similar dynamic shear stiffness and rutting characteristics.

Performance testing on in-situ cores shows the following:

1. The FSCH, RSCH, and AFST tests clearly show the difference between the cores from the 'good' and 'bad' pavement sections in Buncombe County as identified visually during the coring operation. Consistent with the visual field observation, these tests do not show any significant difference in performance of the 'good' and 'bad' pavement sections in Rutherford County.
2. RSCH test results show that for the Buncombe County cores that were tacked with CRS-2 emulsion, horizontal crack through the tack coat was observed. For the Rutherford County cores that were tacked with PG64-22, cracking pattern was diagonal, a pattern more in line with that usually observed for monolithic sections. Moreover, Buncombe County cores in general failed at much lower number of loading cycles compared to the cores from Rutherford County with overall difference of 40-percent.
3. Similar to RSCH test, the uniaxial tensile (VRAMP) test also clearly showed that the bond strength of CRS-2 emulsion to be 46-percent lower as compared to the cores from Rutherford County. For the Buncombe County cores, failure was observed at the

interface of the asphalt layers with clear separation of the two layers. For the Rutherford County Cores, failure was observed in the asphalt mix layers.

Considering that both the Buncombe and Rutherford County mixes were found to adhere to NCDOT specifications, and that the laboratory performance of the mixtures with and without baghouse fines were similar, it may be concluded that the PG64-22 tack coat provided a better interfacial bonding compared to the CRS-2 emulsion.

Table 6.1 Air void content of field cores used in shear testing

'Good' Cores		'Bad' Cores		Average Air Voids (%)
Sample ID	Air Voids (%)	Sample ID	Air Voids (%)	
BG13	6.2	BB20	7.5	6.7
BG14	6.2	BB21	6.7	
		BB24	6.6	
RG01	7.0	RB16	7.2	7.2
RG02	8.0	RB17	6.7	

Table 6.2 Air void content of field cores used in axial testing

'Good' Cores		'Bad' Cores		Average Air Voids (%)
Sample ID	Air Voids (%)	Sample ID	Air Voids(%)	
BG11	6.2	BB22	7.6	6.7
BG12	6.1	BB23	6.7	
RG04	6.9	RB19	7.4	7.2
RG05	7.2	RB20	7.1	

Table 6.3 Air voids and G_{mm} of 150-mm diameter laboratory mix specimens

Buncombe County				Rutherford County			
Sample ID	Height (mm)	Air voids (%)	Avg. Air Void (%)	Sample ID	Height (mm)	Air void (%)	Avg. Air Void (%)
BW11	50.2	4.9	4.7	RW41	50.5	5.5	5.5
BW12	49.3	4.5		RW42	47.5	5.5	
BWO11	50.4	4.9	4.9	RWO41	47.4	5.6	5.5
BWO12	48.6	4.8		RWO42	48.6	5.3	
G_{mm} – mixes w/baghouse fines			2.511	G_{mm} – mixes w/baghouse fines			2.513
G_{mm} – mixes wo/baghouse fines			2.505	G_{mm} – mixes wo/baghouse fines			2.509

Table 6.4 Nationwide pavement temperatures, [18]

Region	Temperature in ^{o}C		
	Effective	Critical	Maximum
I A	27.7	35	37.6
I B	**33.0**	**40**	**41.8**
I C	**29.3**	**35**	**37.5**
II A	28.3	36	38.4
II B	34.2	42	43.7
II C	36.0	43	45.7
III A	30.1	36	38.6
III B	37.2	44	46.6
III C	35.1	42	44.3
Mean	32.3	39.2	41.6

Table 6.5 Average depths and test temperatures for field cores

County	Weather Station.	Depth (mm)	Equation Temp.	SHRPBIND Temp.	Average Temp.
Buncombe	Asheville	33	51.4^{o}C	48.9^{o}C	50.2^{o}C
Rutherford	Caroleen	32	55.4^{o}C	52.5^{o}C	54.0^{o}C

Table 6.6 |G*| (Pa) versus frequency (Hz) for field cores, 50.2^{o}C, Buncombe County

Frequency	BG13	BG14	BB20	BB21	BB24
10	1.13E+08	9.28E+07	7.34E+07	6.00E+07	7.91E+07
5	9.50E+07	7.21E+07	5.69E+07	4.47E+07	6.25E+07
2	7.75E+07	5.46E+07	4.14E+07	3.22E+07	4.66E+07
1	6.70E+07	4.23E+07	3.45E+07	2.59E+07	3.93E+07
0.5	5.90E+07	3.76E+07	2.89E+07	2.14E+07	3.37E+07
0.2	5.15E+07	3.16E+07	2.39E+07	1.71E+07	2.82E+07
0.1	4.86E+07	2.77E+07	2.17E+07	1.30E+07	2.46E+07
0.05	4.34E+07	2.77E+07	1.93E+07	1.22E+07	2.35E+07
0.02	4.04E+07	2.43E+07	1.83E+07	1.27E+07	2.10E+07
0.01	3.99E+07	1.90E+07	1.42E+07	1.16E+07	1.89E+07

Table 6.7 δ (degrees) versus frequency (Hz) for field cores, 50.2^{o}C, Buncombe County

Frequency	BG13	BG14	BB20	BB21	BB24
10	34.90	42.73	46.83	48.96	43.15
5	32.60	40.85	44.42	47.01	40.35
2	28.74	38.32	41.16	45.05	38.47
1	27.36	36.04	38.23	42.64	35.78
0.5	25.27	33.88	36.45	40.72	33.21
0.2	22.60	30.31	31.98	39.61	32.13
0.1	21.13	27.31	30.60	37.53	30.36
0.05	20.39	25.09	26.91	35.67	29.87
0.02	16.28	13.29	27.20	23.77	26.76
0.01	17.27	28.19	25.10	31.78	25.58

Table 6.8 Average |G*|, δ, and |G*|/sin δ values, 50.2°C, Buncombe County

Frequency	|G*| (Pa.) (Good)	|G*| (Pa.) (Bad)	δ (deg.) (Good)	δ (deg.) (Bad)	|G*|/sin δ (Good)	|G*|/sin δ (Bad)
10	1.03E+08	7.09E+07	38.82	46.31	1.68E+08	9.86E+07
5	8.36E+07	5.47E+07	36.73	43.93	1.43E+08	7.96E+07
2	6.61E+07	4.00E+07	33.53	41.56	1.25E+08	6.11E+07
1	5.46E+07	3.32E+07	31.70	38.88	1.09E+08	5.37E+07
0.5	4.83E+07	2.80E+07	29.58	36.79	1.03E+08	4.77E+07
0.2	4.15E+07	2.31E+07	26.45	34.57	9.83E+07	4.17E+07
0.1	3.82E+07	1.98E+07	24.22	32.83	9.76E+07	3.75E+07
0.05	3.55E+07	1.83E+07	22.74	30.82	9.49E+07	3.69E+07
0.02	3.24E+07	1.73E+07	14.78	25.91	1.25E+08	3.94E+07
0.01	2.95E+07	1.49E+07	22.73	27.49	8.74E+07	3.31E+07
Average	5.33E+07	3.20E+07	28.1	35.9	1.15E+08	5.29E+07

Table 6.9 |G*| (Pa) versus frequency (Hz) for field cores, 50.2°C, Rutherford County

Frequency	RG01	RG02	RB16	RB17
10	1.27E+08	1.48E+08	1.44E+08	1.29E+08
5	9.46E+07	1.09E+08	1.04E+08	9.38E+07
2	6.54E+07	7.44E+07	6.95E+07	6.34E+07
1	5.21E+07	5.62E+07	5.17E+07	4.79E+07
0.5	4.01E+07	4.47E+07	3.99E+07	3.72E+07
0.2	3.11E+07	3.28E+07	2.97E+07	2.76E+07
0.1	2.61E+07	2.77E+07	2.56E+07	2.27E+07
0.05	2.32E+07	2.29E+07	2.05E+07	1.89E+07
0.02	2.01E+07	1.69E+07	1.79E+07	1.74E+07
0.01	1.76E+07	1.69E+07	1.65E+07	1.50E+07

Table 6.10 δ (degrees) versus freq. (Hz) for field cores, 50.2°C, Rutherford County

Frequency	RG01	RG02	RB16	RB17
10	44.59	45.15	47.18	46.96
5	44.64	45.37	47.89	47.23
2	43.03	45.35	48.10	47.15
1	42.68	44.94	47.40	45.45
0.5	41.23	43.42	46.36	44.12
0.2	36.84	40.47	42.74	41.36
0.1	34.82	37.79	42.64	38.86
0.05	33.43	33.16	43.32	30.43
0.02	21.59	32.73	27.20	35.27
0.01	25.60	31.88	38.17	33.40

Table 6.11 Average |G*|, δ, and |G*|/sin δ values, 50.2^{o}C, Rutherford County

Frequency	|G*| (Pa.) (Good)	|G*| (Pa.) (Bad)	δ (deg.) (Good)	δ (deg.) (Bad)	|G*|/sin δ (Good)	|G*|/sin δ (Bad)
10	1.37E+08	1.36E+08	44.87	47.07	1.95E+08	1.86E+08
5	1.02E+08	9.90E+07	45.00	47.56	1.44E+08	1.34E+08
2	6.99E+07	6.64E+07	44.19	47.63	1.00E+08	8.99E+07
1	5.41E+07	4.98E+07	43.81	46.42	7.82E+07	6.87E+07
0.5	4.24E+07	3.86E+07	42.32	45.24	6.29E+07	5.43E+07
0.2	3.20E+07	2.86E+07	38.66	42.05	5.12E+07	4.27E+07
0.1	2.69E+07	2.42E+07	36.30	40.75	4.54E+07	3.70E+07
0.05	2.31E+07	1.97E+07	33.29	36.88	4.20E+07	3.36E+07
0.02	1.85E+07	1.77E+07	27.16	31.23	4.30E+07	3.47E+07
0.01	1.73E+07	1.57E+07	28.74	35.79	3.64E+07	2.69E+07
Average	5.23E+07	4.96E+07	38.4	42.1	7.98E+07	7.08E+07

Table 6.12 Number of RSCH cycles for each specimen, 50.2^{o}C

'Good' Cores			**'Bad' Cores**		
County	Sample ID	# of cycles	County	Sample ID	# of cycles
Buncombe	BG13	22000	Buncombe	BB20	10000
Buncombe	BG14	60000	Buncombe	BB24	3800
Average # of cycles		**41000**	**Average # of cycles**		**6900**
Rutherford	RG01	65000	Rutherford	RB16	100000
Rutherford	RG02	80000	Rutherford	RB17	100000
Average # of cycles		**73000**	**Average # of cycles**		**100000**

Table 6.13 Permanent strains at 5000 RSCH cycles, 50.2^{o}C

'Good' Cores			**'Bad' Cores**		
County	**Sample ID**	**ε (%)**	**County**	**Sample ID**	**ε (%)**
Buncombe	BG13	1.40	Buncombe	BB20	2.53
Buncombe	BG14	1.82	Buncombe	BB24	2.59
Rutherford	RG01	1.12	Rutherford	RB16	0.91
Rutherford	RG02	0.93	Rutherford	RB17	0.86

Table 6.14 |E*| (Pa) versus frequency (Hz) for field cores, 50.2°C, Buncombe County

Frequency	BG11	BG12	BB22	BB23
10	3.16E+08	2.28E+08	2.04E+08	2.17E+08
5	2.31E+08	1.04E+08	1.50E+08	1.56E+08
2	1.39E+08	7.94E+07	1.13E+08	1.12E+08
1	1.00E+08	6.58E+07	8.60E+07	8.20E+07
0.5	**	5.52E+07	6.71E+07	6.34E+07
0.2	6.14E+07	5.43E+07	6.23E+07	4.38E+07
0.1	6.16E+07	3.73E+07	4.87E+07	4.27E+07
0.05	4.38E+07	3.84E+07	4.20E+07	3.99E+07
0.02	4.48E+07	2.89E+07	3.52E+07	2.98E+07
0.01	3.81E+07	4.10E+07	3.22E+07	3.34E+07

Table 6.15 δ (degrees) versus frequency (Hz) for field cores, 50.2°C, Buncombe County

Frequency	BG11	BG12	BB22	BB23
10	45.46	50.19	46.90	49.62
5	44.96	44.36	44.44	48.53
2	44.90	49.23	39.73	43.85
1	35.41	47.34	32.90	38.93
0.5	**	45.74	30.61	37.75
0.2	37.14	29.59	26.49	22.52
0.1	16.01	33.40	31.49	33.98
0.05	21.76	30.27	28.91	40.16
0.02	20.45	10.79	25.06	18.69
0.01	21.69	41.71	12.57	42.67

Table 6.16 |E*| (Pa) versus frequency (Hz) for field cores, 50.2°C, Rutherford County

Frequency	RG04	RG05	RB19	RB20
10	4.49E+08	4.39E+08	3.17E+08	2.69E+08
5	3.16E+08	1.38E+08	2.25E+08	1.33E+08
2	1.71E+08	1.20E+08	1.47E+08	1.11E+08
1	1.32E+08	1.13E+08	1.08E+08	8.14E+07
0.5	9.71E+07	8.55E+07	8.18E+07	5.92E+07
0.2	6.36E+07	5.56E+07	5.82E+07	5.64E+07
0.1	4.81E+07	4.12E+07	4.66E+07	3.40E+07
0.05	2.63E+07	1.97E+07	3.83E+07	1.97E+07
0.02	2.44E+07	9.76E+06	3.26E+07	2.26E+07
0.01	1.61E+07	9.91E+06	2.92E+07	5.92E+07

Table 6.17 δ (degrees) versus freq. (Hz) for field cores, 50.2°C, Rutherford County

Frequency	RG04	RG05	RB19	RB20
10	50.00	50.33	48.82	48.48
5	50.06	39.46	48.30	43.25
2	46.23	61.87	48.26	53.69
1	52.99	54.67	45.54	50.24
0.5	54.74	60.37	43.39	52.68
0.2	51.07	60.86	33.69	66.43
0.1	45.62	55.09	29.81	37.46
0.05	43.14	36.68	26.77	28.21
0.02	36.17	50.07	19.25	21.62
0.01	30.39	47.29	24.97	17.50

Table 6.18 Average values of |E*| (Pa.), 50.2°C, Buncombe and Rutherford

Frequency	Buncombe County		Rutherford County	
Hz	|E*| (Good)	|E*| (Bad)	|E*| (Good)	|E*| (Bad)
10	2.72E+08	1.02E+08	4.44E+08	2.93E+08
5	1.68E+08	7.51E+07	2.27E+08	1.79E+08
2	1.09E+08	5.67E+07	1.45E+08	1.29E+08
1	8.31E+07	4.30E+07	1.23E+08	9.45E+07
0.5	5.52E+07	3.36E+07	9.13E+07	7.05E+07
0.2	5.79E+07	3.11E+07	5.96E+07	5.73E+07
0.1	4.94E+07	2.43E+07	4.46E+07	4.03E+07
0.05	4.11E+07	2.10E+07	2.30E+07	2.90E+07
0.02	3.69E+07	1.76E+07	1.71E+07	2.76E+07
0.01	3.96E+07	1.61E+07	1.30E+07	1.42E+07
Average	9.12E+07	4.0E+07	11.87E+07	9.64E+07
Difference	**56%**		**18%**	

Table 6.19 Values at peak load, 50.2°C, Buncombe County

Sample ID	Time (s)	Axial Load (kN)	Axial Disp. (mm)	Axial Stress (Pa)	Axial Strain (mm/mm)
BG11	2.1565	0.57191	0.0868	32363.5	0.001778
BG12	4.6825	0.43528	0.1736	24644.3	0.003711
BB22	2.4505	0.35614	0.0869	20153.2	0.001824
BB23	4.5085	0.45170	0.1737	25561.2	0.003607

Table 6.20 Values at peak load, 50.2°C, Rutherford County

Sample ID	Time (s)	Axial Load (kN)	Axial Disp. (mm)	Axial Stress (Pa)	Axial Strain (mm/mm)
RG03	4.8025	0.86460	0.1984	48925.5	0.003808
RG04	5.4095	1.07686	0.2294	61051.0	0.004517
RG05	5.1095	1.05647	0.1922	59784.0	0.003703
RB19	2.7440	0.66225	0.1302	37475.7	0.002573
RB20	6.0095	0.69360	0.2294	39250.2	0.004255

Table 6.21 |G*| (Pa) versus frequency (Hz) for lab mixes, 50.2°C, Buncombe County

Frequency	BW11	BW12	BWO11	BWO12
10	1.44E+08	1.90E+08	9.18E+07	1.62E+08
5	1.10E+08	1.41E+08	6.69E+07	1.17E+08
2	7.94E+07	9.88E+07	4.66E+07	8.02E+07
1	6.38E+07	7.71E+07	3.67E+07	6.17E+07
0.5	5.23E+07	6.15E+07	2.93E+07	5.12E+07
0.2	4.24E+07	4.83E+07	2.30E+07	3.66E+07
0.1	3.78E+07	4.11E+07	2.00E+07	3.08E+07
0.05	3.16E+07	3.49E+07	1.65E+07	2.72E+07
0.02	2.87E+07	3.30E+07	1.48E+07	2.44E+07
0.01	2.71E+07	2.66E+07	1.38E+07	2.10E+07

Table 6.22 δ (degrees) versus frequency (Hz) for lab mixes, 50.2°C, Buncombe County

Frequency	BW11	BW12	BWO11	BWO12
10	41.11	45.80	49.68	47.84
5	40.46	45.47	48.20	47.79
2	39.37	44.07	46.92	46.45
1	38.41	42.59	43.19	45.93
0.5	36.08	40.72	44.05	43.55
0.2	33.70	37.97	40.55	41.28
0.1	32.40	36.07	37.35	39.46
0.05	29.00	33.47	35.30	35.01
0.02	29.26	34.25	34.74	34.25
0.01	25.68	25.39	29.56	29.78

Table 6.23 Average |G*|, δ, and |G*|/sin δ values, 50.2°C, lab mixes Buncombe County

Frequency	|G*| (Pa.) (With)	|G*| (Pa.) (W/o)	δ (deg.) (With)	δ (deg.) (W/o)	|G*|/sin δ (With)	|G*|/sin δ (W/o)
10	1.67E+08	1.27E+08	43.46	48.76	2.42E+08	1.69E+08
5	1.25E+08	9.21E+07	42.97	48.00	1.83E+08	1.24E+08
2	8.91E+07	6.34E+07	41.72	46.69	1.34E+08	8.73E+07
1	7.04E+07	4.92E+07	40.50	44.56	1.08E+08	6.97E+07
0.5	5.69E+07	4.03E+07	38.40	43.80	9.15E+07	5.82E+07
0.2	4.53E+07	2.98E+07	35.83	40.91	7.74E+07	4.54E+07
0.1	3.94E+07	2.54E+07	34.23	38.40	7.02E+07	4.07E+07
0.05	3.32E+07	2.19E+07	31.24	35.16	6.42E+07	3.80E+07
0.02	3.08E+07	1.96E+07	31.75	34.50	5.87E+07	3.47E+07
0.01	2.68E+07	1.74E+07	25.54	29.67	6.23E+07	3.52E+07
Average	6.85E+07	4.86E+07	3.66E+01	4.10E+01	1.09E+08	7.02E+07

Table 6.24 |G*| (Pa) versus frequency (Hz) for lab mixes, 50.2°C, Rutherford County

Frequency	RW41	RW42	RWO41	RWO42
10	2.25E+08	2.02E+08	1.74E+08	1.86E+08
5	1.64E+08	1.46E+08	1.23E+08	1.34E+08
2	1.09E+08	9.68E+07	7.83E+07	8.85E+07
1	8.00E+07	7.16E+07	5.68E+07	4.39E+07
0.5	5.98E+07	5.41E+07	4.19E+07	4.47E+07
0.2	4.18E+07	3.83E+07	2.95E+07	3.68E+07
0.1	3.21E+07	3.09E+07	2.36E+07	2.91E+07
0.05	2.56E+07	2.31E+07	1.80E+07	1.15E+07
0.02	2.00E+07	1.98E+07	1.50E+07	1.21E+07
0.01	1.77E+07	1.84E+07	1.33E+07	1.54E+07

Table 6.25 δ (degrees) versus frequency (Hz) for lab mixes, 50.2°C, Rutherford County

Frequency	RW41	RW42	RWO41	RWO42
10	43.97	45.61	48.26	46.45
5	45.36	46.81	49.51	47.29
2	46.72	47.28	50.07	47.98
1	47.66	45.66	54.41	28.08
0.5	47.79	47.09	50.10	59.49
0.2	45.93	45.22	47.63	57.29
0.1	43.61	42.52	46.53	48.85
0.05	40.86	39.49	43.78	35.35
0.02	39.28	34.86	37.73	32.52
0.01	37.73	32.08	37.85	33.17

Table 6.26 Average |G*|, δ, and |G*|/sin δ values, 50.2°C, lab mixes Rutherford County

Frequency	|G*| (Pa.) (With)	|G*| (Pa.) (W/o)	δ (deg.) (With)	δ (deg.) (W/o)	|G*|/sin δ (With)	|G*|/sin δ (W/o)
10	2.14E+08	1.80E+08	44.79	47.36	3.04E+08	2.44E+08
5	1.55E+08	1.28E+08	46.08	48.40	2.15E+08	1.72E+08
2	1.03E+08	8.34E+07	47.00	49.03	1.41E+08	1.11E+08
1	7.58E+07	5.03E+07	46.66	41.24	1.04E+08	8.15E+07
0.5	5.70E+07	4.33E+07	47.44	54.80	7.73E+07	5.32E+07
0.2	4.00E+07	3.31E+07	45.58	52.46	5.60E+07	4.18E+07
0.1	3.15E+07	2.64E+07	43.07	47.69	4.61E+07	3.56E+07
0.05	2.44E+07	1.48E+07	40.18	39.57	3.78E+07	2.30E+07
0.02	1.99E+07	1.36E+07	37.07	35.12	3.31E+07	2.35E+07
0.01	1.81E+07	1.44E+07	34.91	35.51	3.18E+07	2.49E+07
Average	7.38E+07	5.87E+07	4.33E+01	4.51E+01	1.05E+08	8.10E+07

Table 6.27 Strain at the end of RSCH test, 50.2^{o}C, lab mixes

Specimens 'With' Baghouse Fines			Specimens 'Without' Baghouse Fines		
County	**Sample ID**	**% Strain**	**County**	**Sample ID**	**% Strain**
Buncombe	BW11	2.10	Buncombe	BWO11	2.70
Buncombe	BW12	1.59	Buncombe	BWO12	1.57
Average % Strain		**1.85**	**Average % Strain**		**2.14**
Rutherford	RW41	1.70	Rutherford	RWO41	2.18
Rutherford	RW42	1.89	Rutherford	RWO42	2.15
Average % Strain		**1.80**	**Average % Strain**		**2.17**

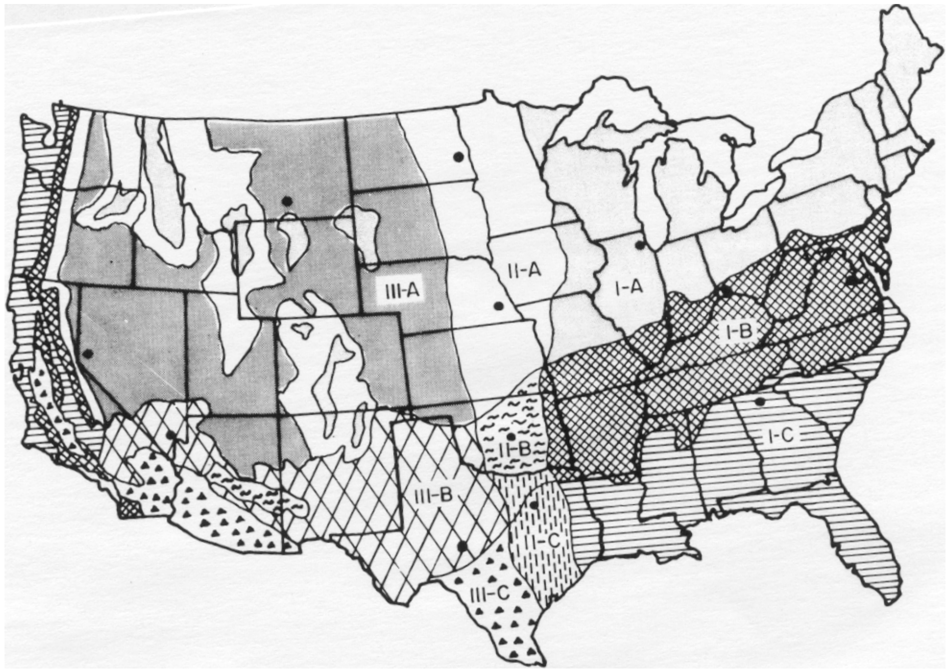

Figure 6.1 Nine climatic regions in US

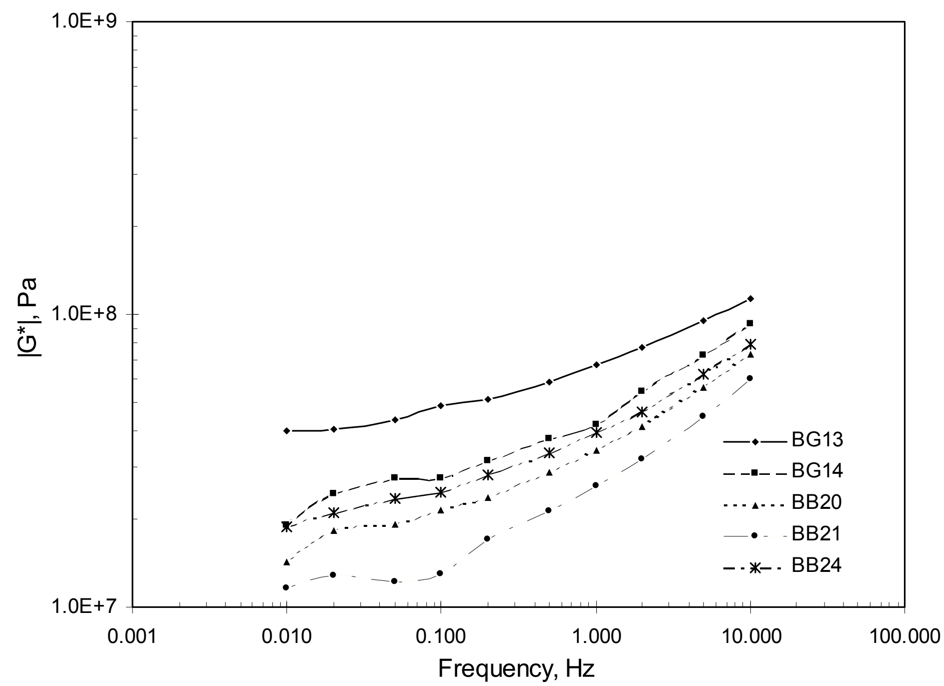

Figure 6.2 Dynamic shear modulus (|G*|) versus freq. 50.2^{o}C, Buncombe County

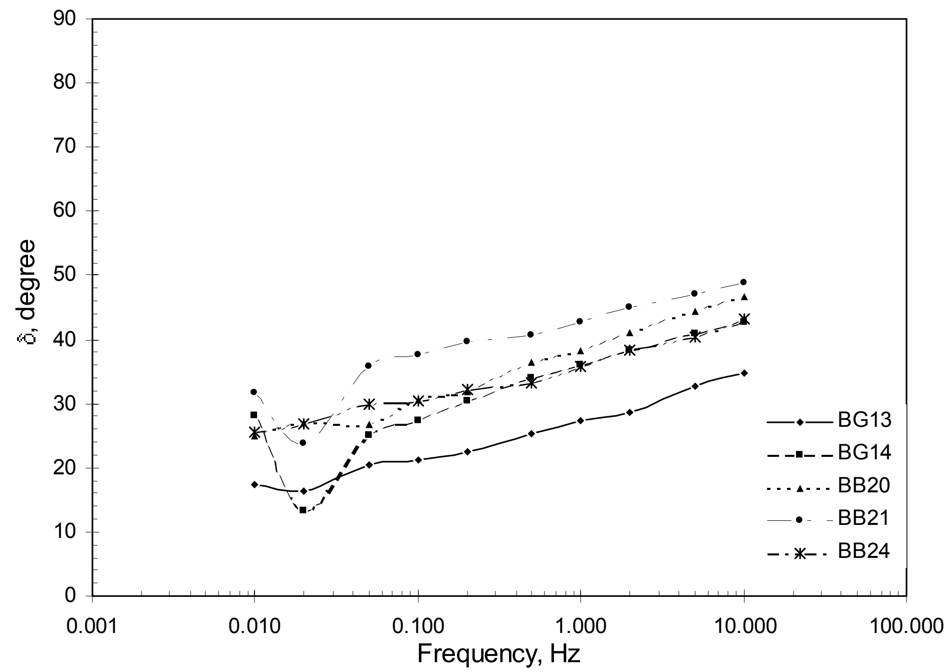

Figure 6.3 Phase angle (δ) versus frequency, 50.2^{o}C, Buncombe County

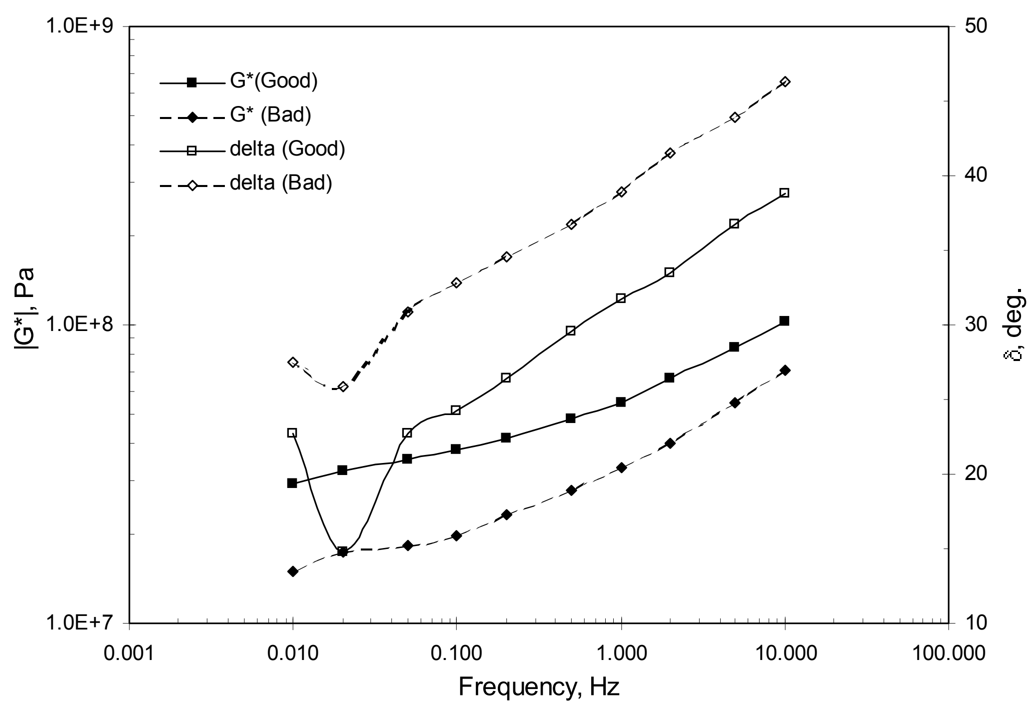

Figure 6.4 Average |G*| and δ values versus frequency, 50.2^{o}C, Buncombe County

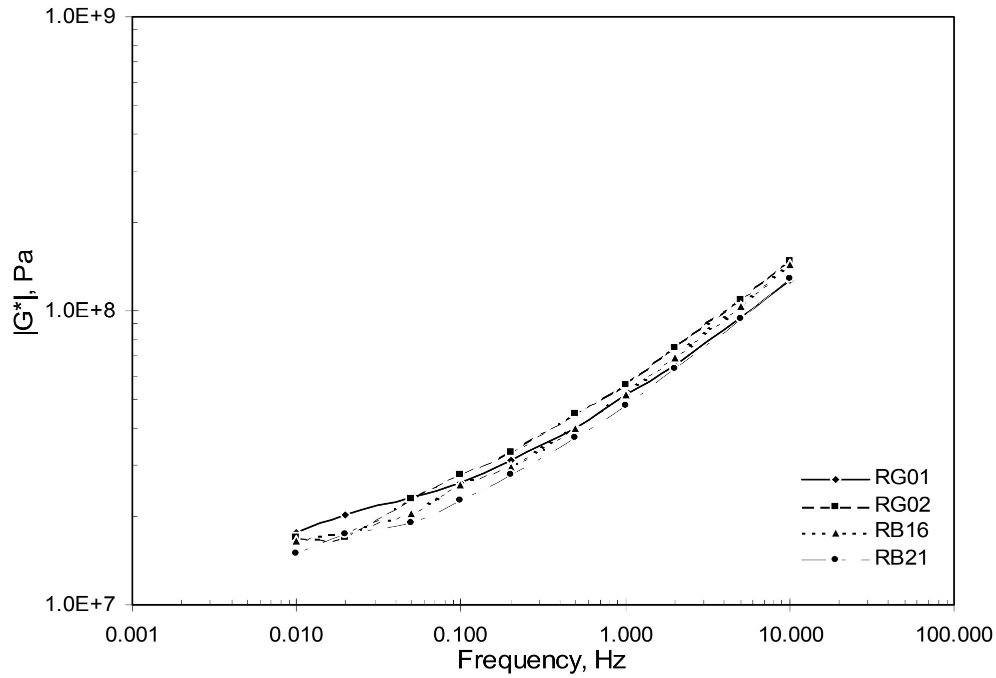

Figure 6.5 Dynamic shear modulus (|G*|) versus frequency, 50.2^{o}C, Rutherford County

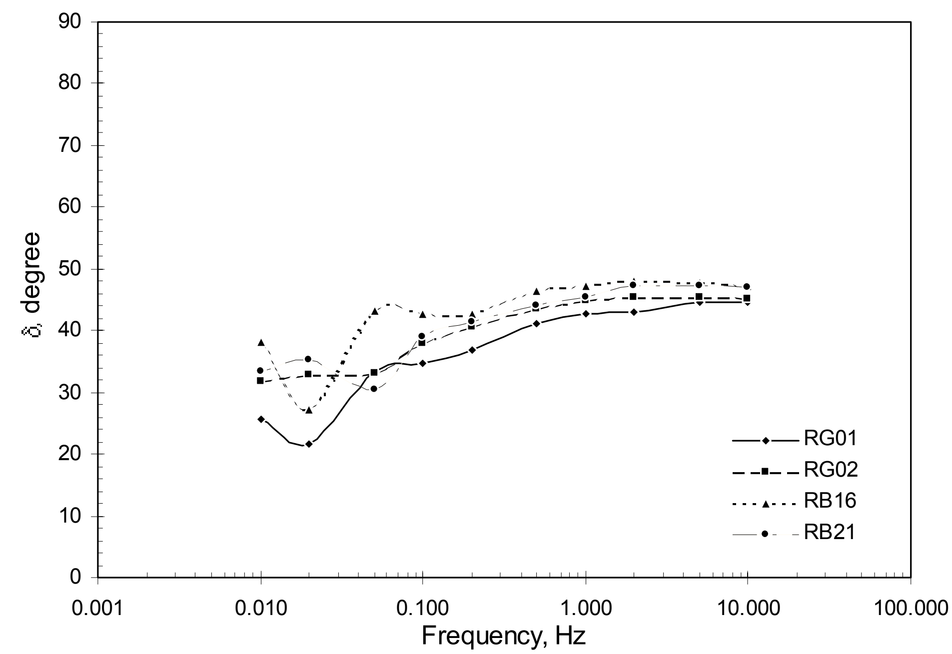

Figure 6.6 Phase angle (δ) versus frequency, 50.2^{o}C, Rutherford County

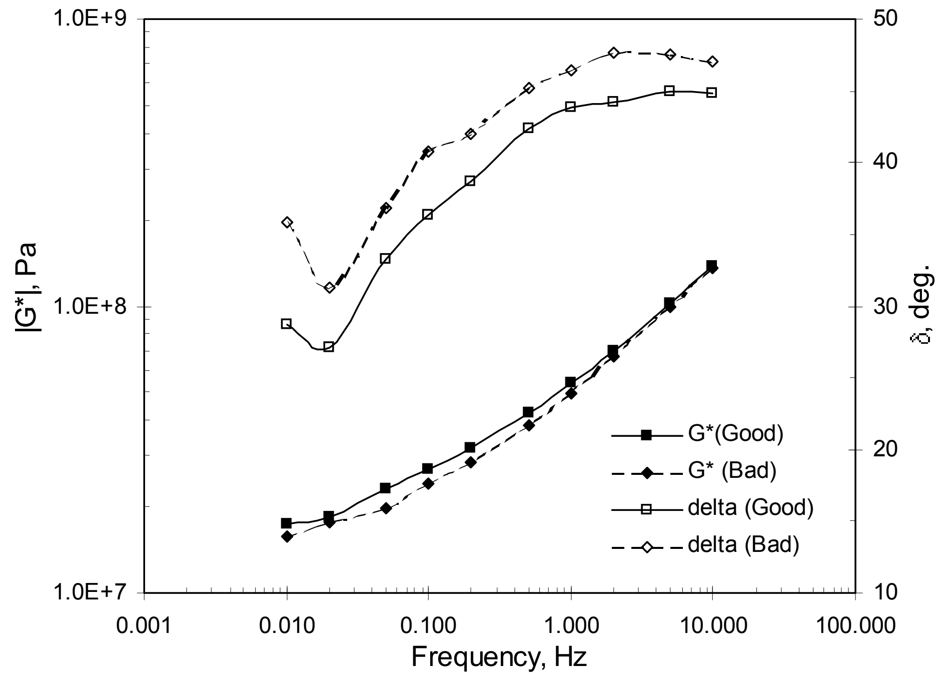

Figure 6.7 Average |G*| and δ values versus frequency, 50.2°C, Rutherford County

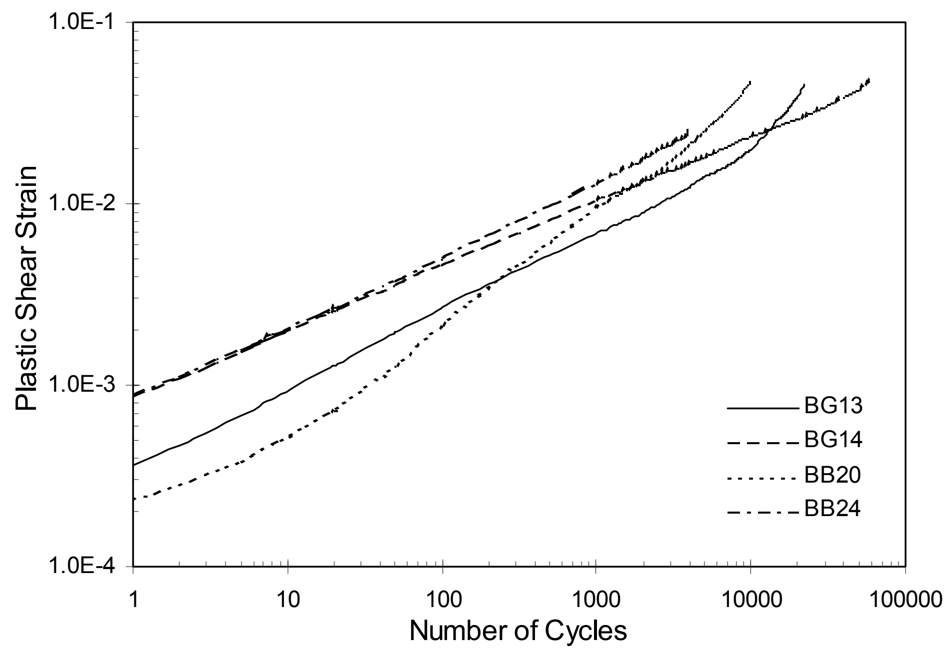

Figure 6.8 Plastic shear strain vs. number of RSCH cycles, 50.2°C, Buncombe County

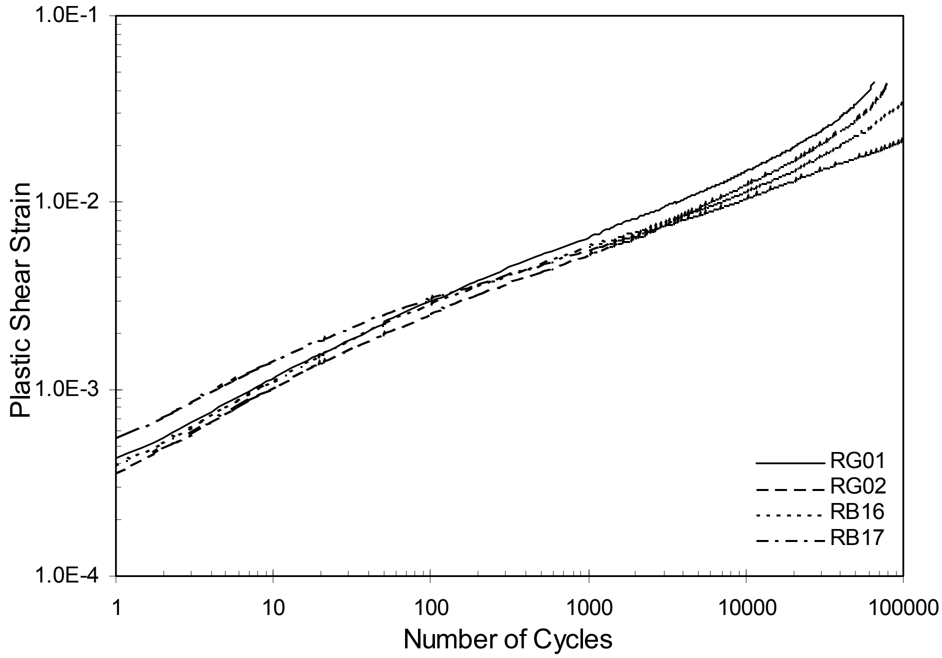

Figure 6.9 Plastic shear strain vs. number of RSCH cycles, 50.2°C, Rutherford County

Figure 6.10 Failed RSCH specimen, Rutherford County

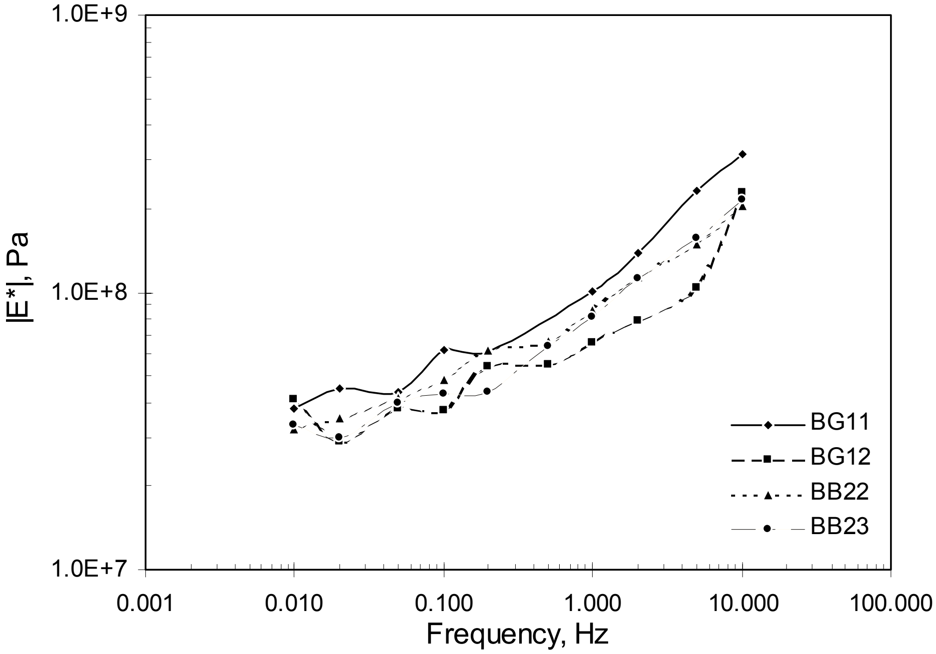

Figure 6.11 Dynamic axial modulus (|E*|) versus frequency, 50.2^{o}C, Buncombe County

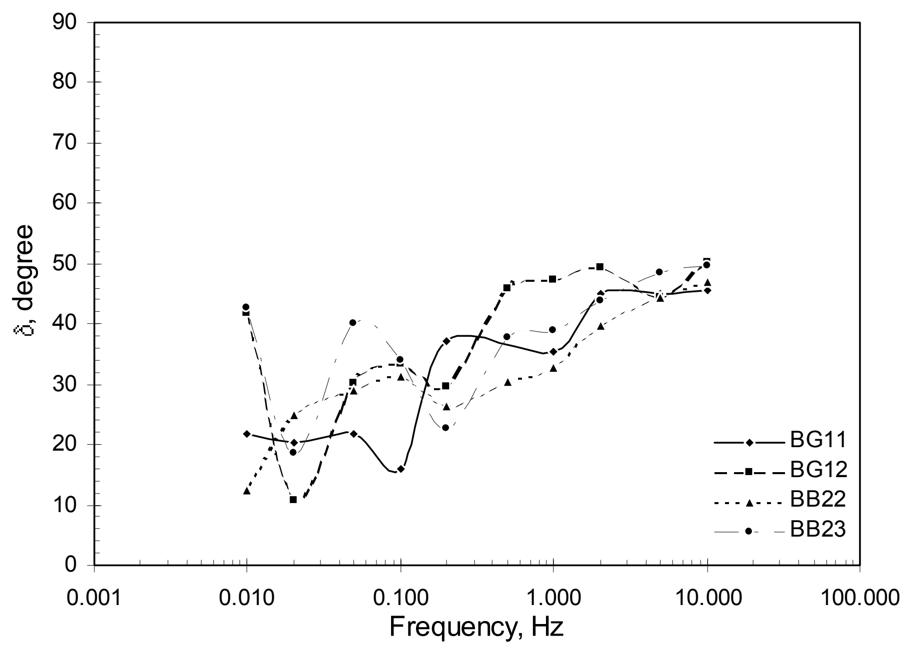

Figure 6.12 Phase angle (δ) versus frequency, 50.2^{o}C, Buncombe County

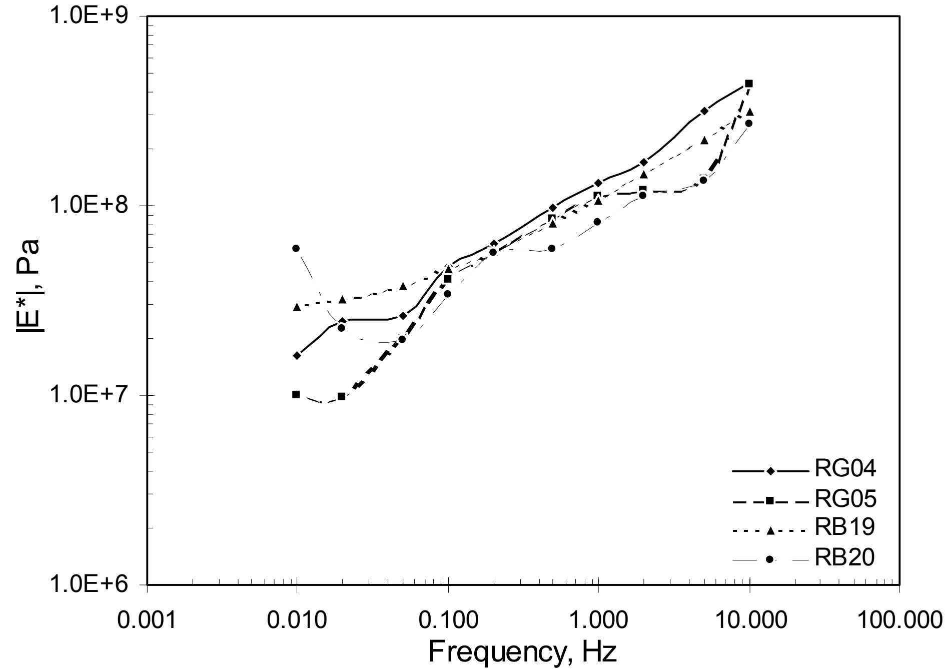

Figure 6.13 Dynamic axial modulus (|E*|) vs. frequency, 50.2°C, Rutherford County

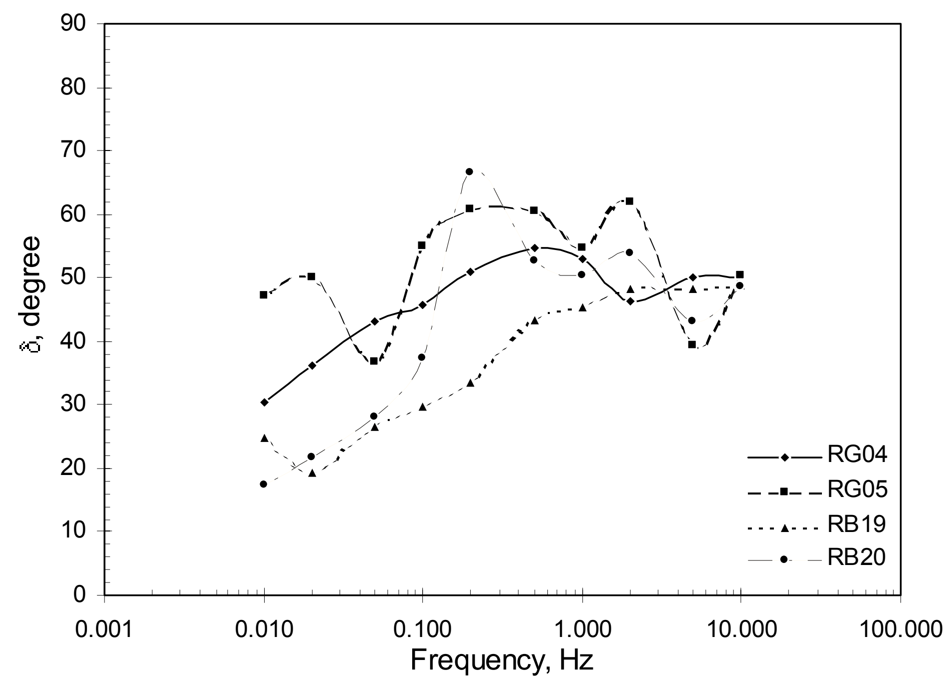

Figure 6.14 Phase angle (δ) versus frequency, 50.2°C, Rutherford County

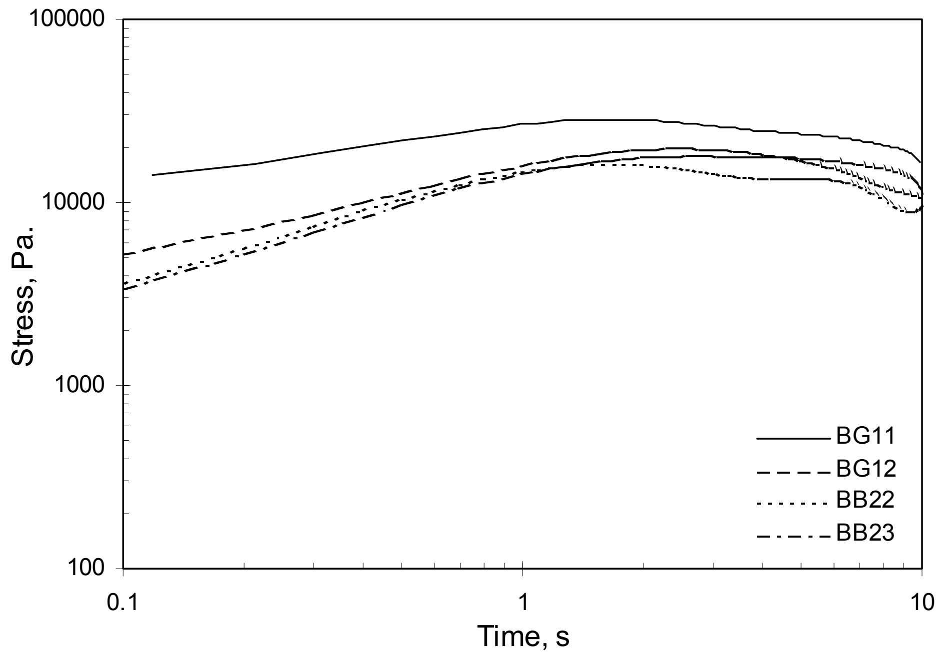

Figure 6.15 Axial stress versus time, 50.2^{o}C, Buncombe County

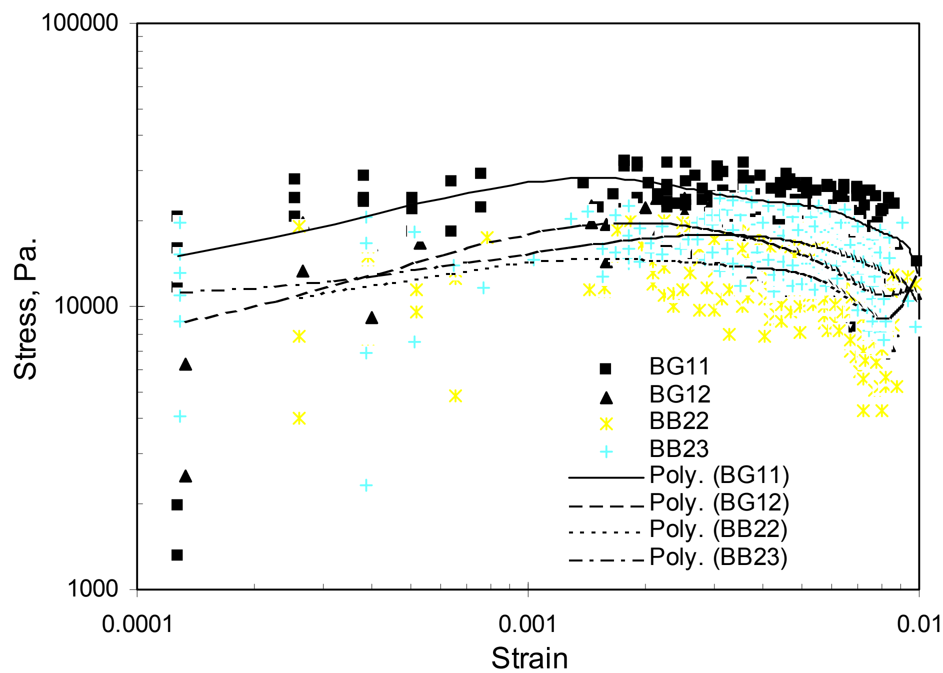

Figure 6.16 Axial stress versus axial strain, 50.2^{o}C, Buncombe County

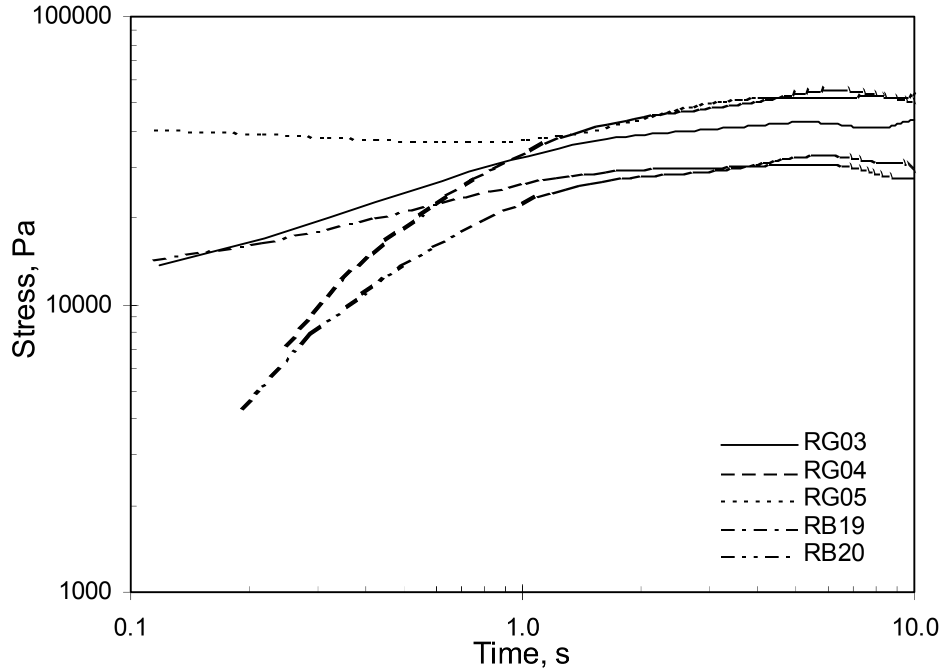

Figure 6.17 Axial stress versus time, 50.2°C, Rutherford County

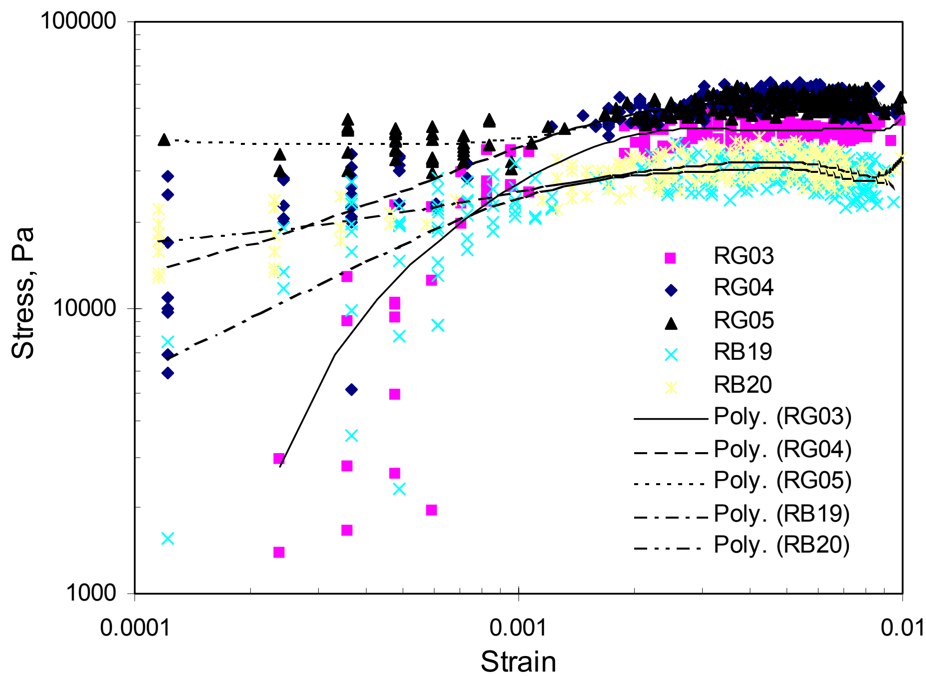

Figure 6.18 Axial stress versus axial strain, 50.2°C, Rutherford County

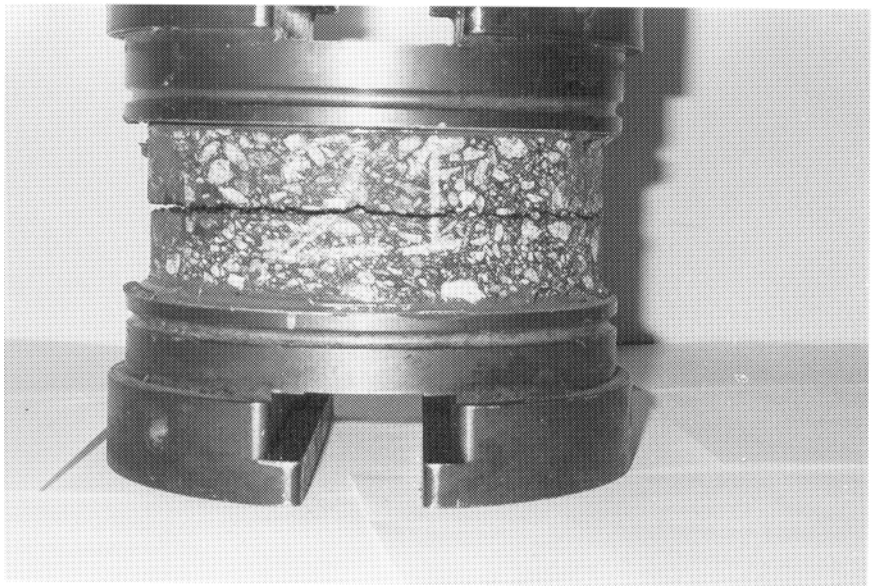

Figure 6.19 Uniaxial test specimen BB21, 50.2°C, Buncombe County (CRS-2)

Figure 6.20 Uniaxial test specimen BB21, 50.2°C, Buncombe County (CRS-2)

Figure 6.21 Uniaxial test specimen BB22, 50.2^{o}C, Buncombe County (CRS-2)

Figure 6.22 Uniaxial test specimen RG03, 50.2^{o}C, Rutherford County (PG64-22)

Figure 6.23 Uniaxial test specimen RG03, 50.2^{o}C, Rutherford County (PG64-22)

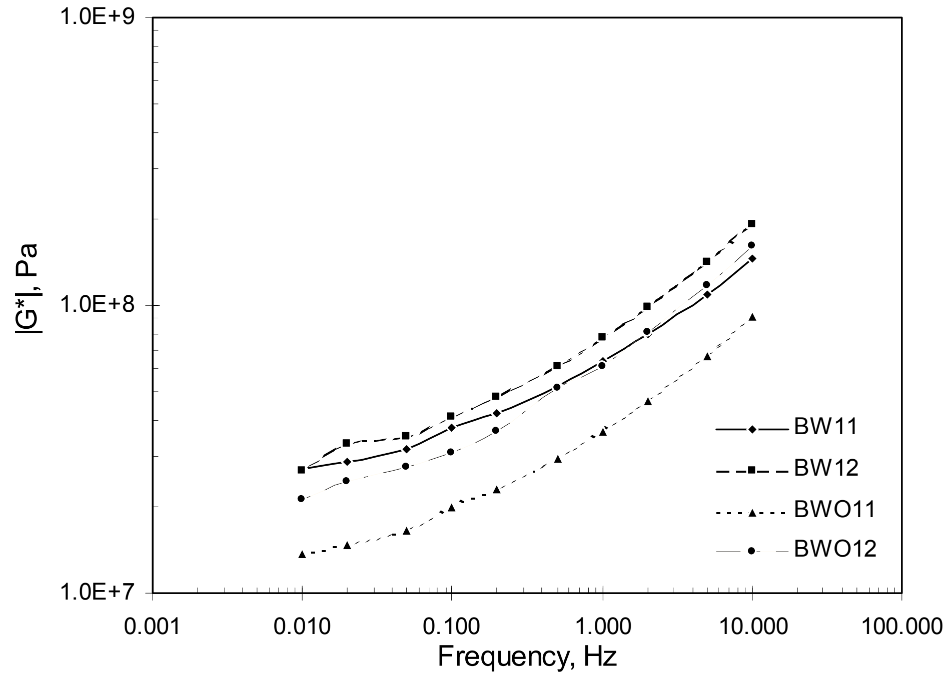

Figure 6.24 Dynamic Shear Modulus (|G*|) vs. freq., 50.2°C, Buncombe, lab mixes

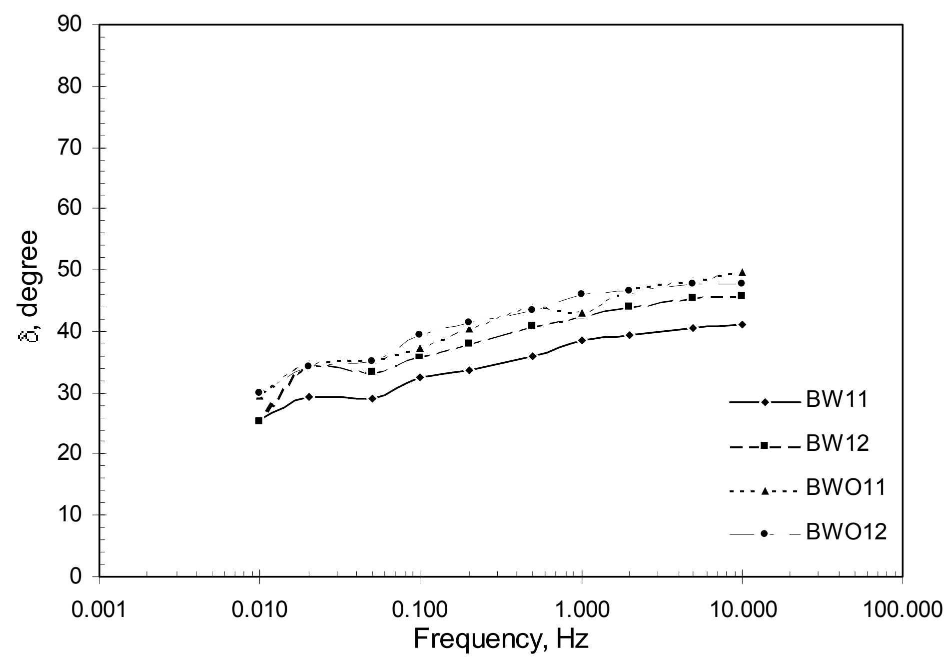

Figure 6.25 Phase angle (δ) versus frequency, 50.2°C, Buncombe, lab mixes

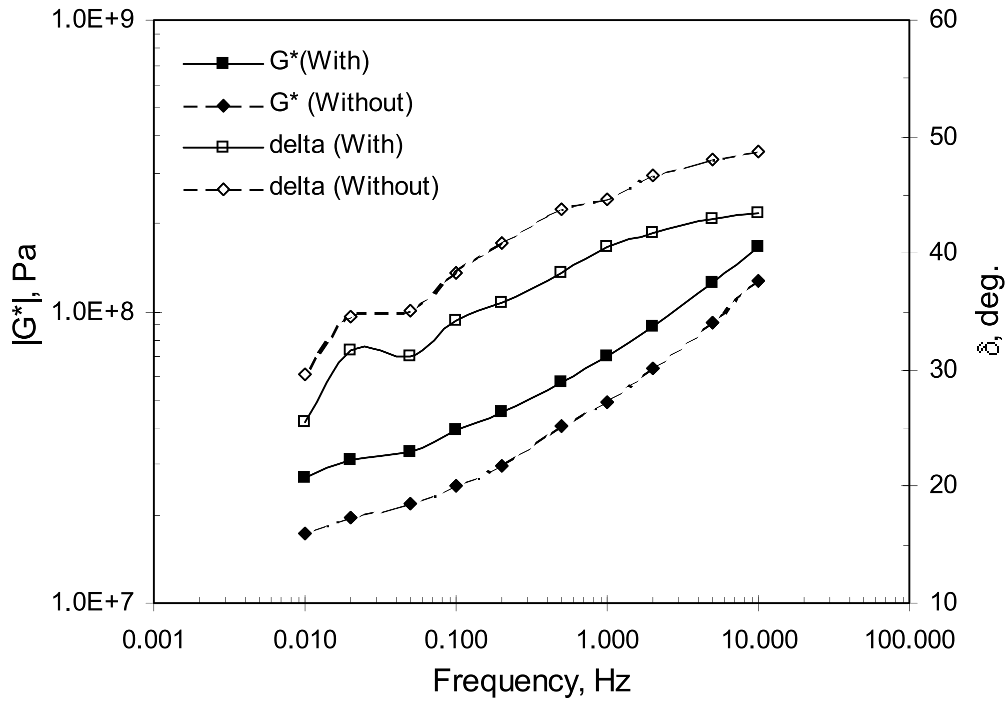

Figure 6.26 Average |G*| and δ values vs. freq., 50.2°C, Buncombe, lab mixes

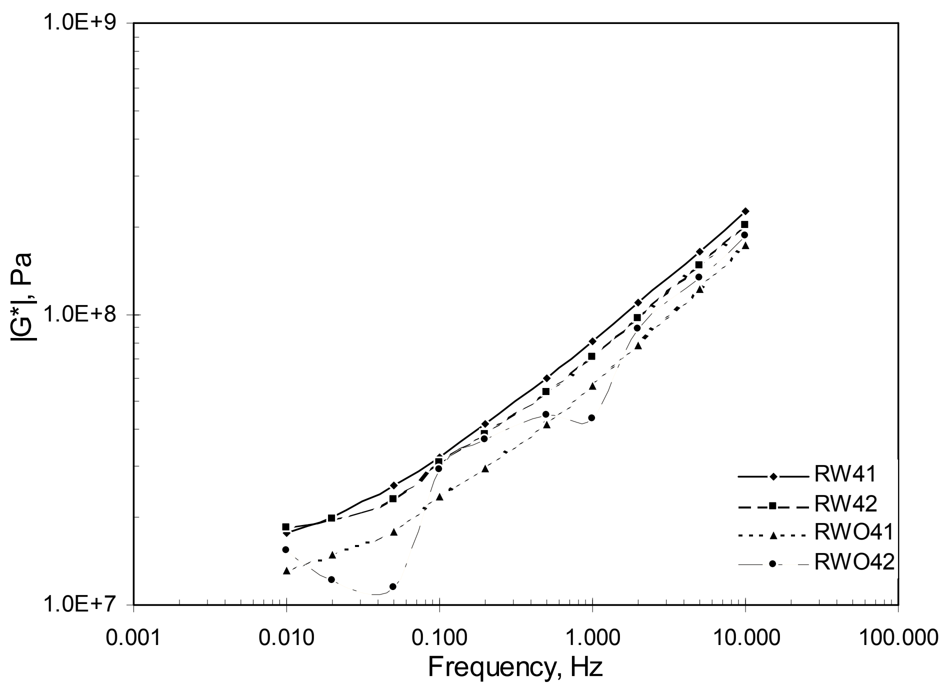

Figure 6.27 Dynamic Shear Modulus (|G*|) vs. freq., 50.2°C, Rutherford, lab mixes

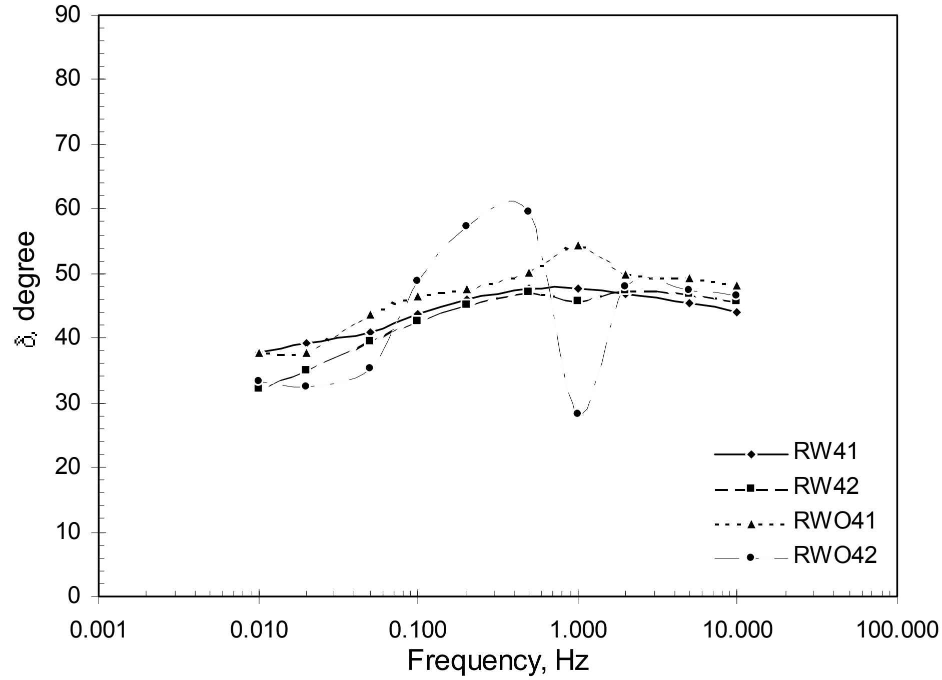

Figure 6.28 Phase angle (δ) vs. frequency, 50.2°C, Rutherford, lab mixes

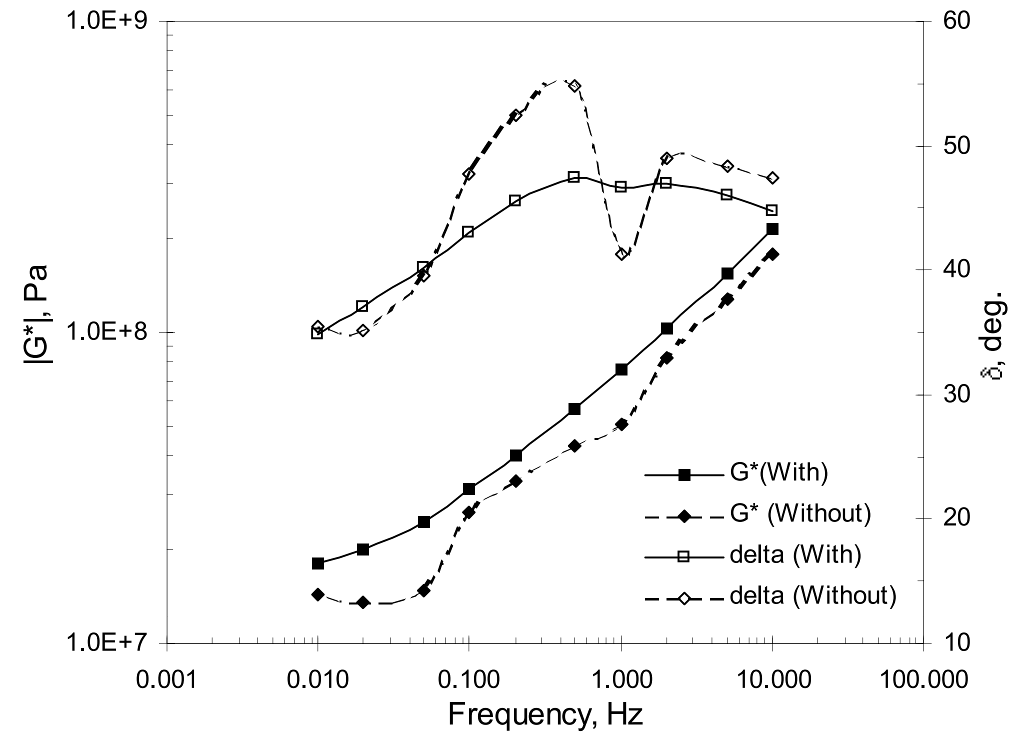

Figure 6.29 Average |G*| and δ values vs. freq., 50.2°C, Rutherford, lab mixes

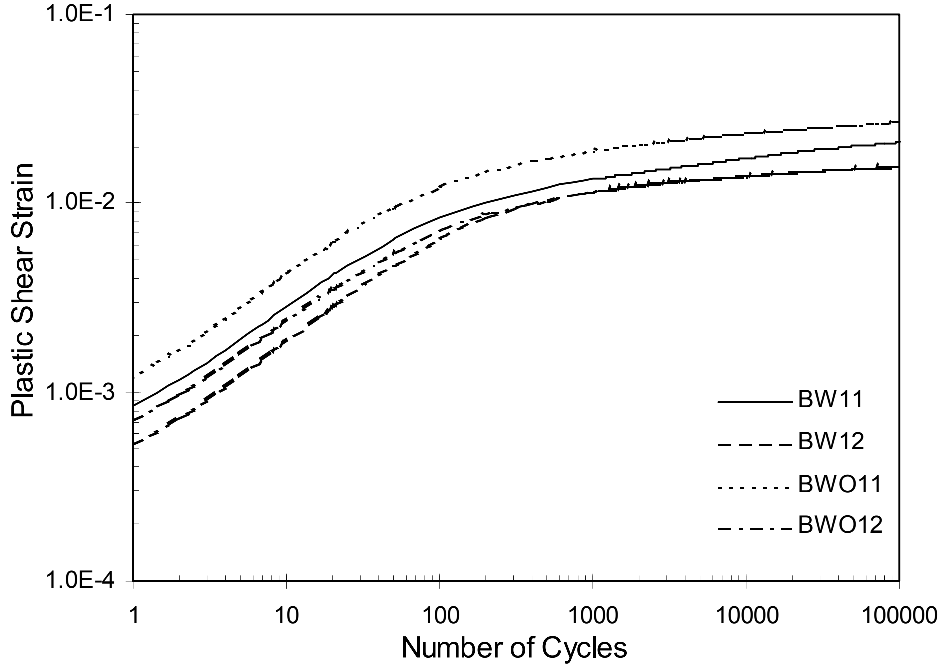

Figure 6.30 Plastic shear strain vs. RSCH cycles, 50.2^{o}C, Buncombe County, lab mixes

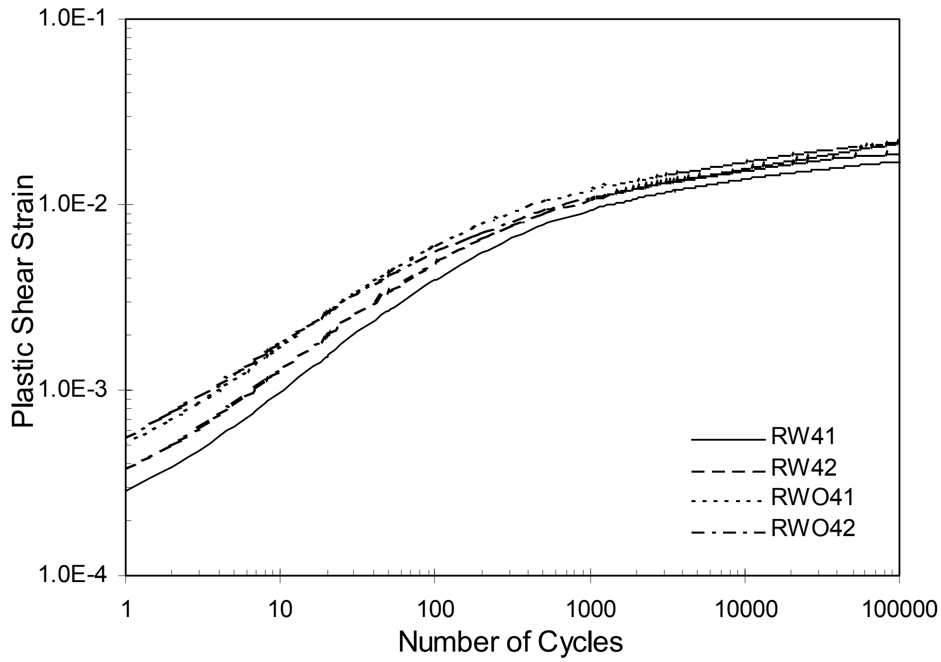

Figure 6.31 Plastic shear strain vs. RSCH cycles, 50.2^{o}C, Rutherford County, lab mixes

7. APA AND TSR TEST RESULTS

In this section, the effect of baghouse fines on moisture sensitivity and the rutting resistance of asphalt mixtures is evaluated using the modified TSR test and Asphalt Pavement Analyzer (APA), respectively. NCDOT Materials and Test Unit conducted both tests. The APA test results are discussed first followed by the TSR test results.

7.1 Rutting resistance of mixtures using APA test

"Accelerated pavement testing is defined as the controlled application of a prototype wheel loading, at or above the appropriate legal load limit to a prototype or actual, layered, structural pavement system to determine pavement response and performance under a controlled, accelerated, accumulation of damage in a compressed time period [12]."

The APA measures rutting susceptibility by rolling a steel wheel over pressurized rubber hose that is positioned across a rectangular asphalt concrete slab or a 6-inch diameter circular specimen. The test is normally performed at 40.6°C and with the rubber hoses pressurized to 0.69 MPa (100 psi). The wheel passes over the hoses and slab at approximately 2.0 km/h (33±1 cycles/min) and the specimen is subjected to 8,000 cycles with each cycle defined as two passes of the wheel back and forth across the specimen. The deformation of the slab or specimen is measured at three points across the specimen and averaged. The Georgia Department of Transportation (GDOT) defines a mixture as susceptible to rutting if the average rut depth for replicate specimens is greater than 7.6-mm. However, the FHWA recommends that the maximum rut depth criteria be set to 5-mm.

Since the APA is a 'proof' test or a 'pass or fail test', many variations of the test temperatures and rut depth acceptance criteria exists based on local experience. NCDOT normally conducts these tests corresponding to the asphalt cements high PG rating with rut depth acceptance criterion of 0.25-inches (6.25-mm). In this study, APA test temperature of 50°C was selected for consistency with the temperatures used for other performance tests.

7.1.1 Specimen fabrication and air voids

The 6-inch diameter specimens for APA test were fabricated at NCSU materials laboratory using the SGC (Superpave Gyratory Compactor). The raw materials received from NCDOT were separated into various fractions depending on their sieve sizes and were then blended to the appropriate NCDOT specified JMF gradations. The exception to this procedure was that the Rutherford County sand and all the baghouse fines, were added in bulk as received. Specimens with zero percent baghouse fines were fabricated with mineral filler (fraction passing #200 sieve) whereas, specimens with 100-percent baghouse fines had their fraction passing #200 sieve substituted completely by the baghouse fines. For Rutherford County, there were two types of baghouses: the 'fine' baghouse fines and the 'coarse' baghouse fines. For the purpose of laboratory testing, only the 'fine' baghouse fines were used. The asphalt contents for Rutherford and Buncombe Counties were 6.2 and 5.7-percent, respectively, and the non-strip additive requirement was 0.5-percent for both the counties.

The mixing and compaction was carried out at a temperature of 285^{o}F, and before compaction, the mixes were aged at a temperature of 275^{o}F for 2 hours. The specimens were compacted to a height of 3 inches with a target air void content of 7±1-percent. Table 7.1 shows the air voids content of specimens used for the APA tests. Two cylindrical specimens were used for each test and an average rut depth was determined.

7.1.2 APA test results

Test results obtained from NCDOT (Appendix E) indicate that the materials from Buncombe County with and without the baghouse fines had an average rut depth of 6.15-mm and 6.12-mm, respectively. For the Rutherford County, specimens with and without the baghouse fines had an average rut depth of 12.33-mm and 12.78-mm, respectively, two times those observed for the Buncombe County.

Based on the test results obtained, it appears that the Buncombe County mixes would be acceptable based on the GDOT criterion but would fail based on the NCDOT criterion. It should be noted that GDOT requires testing to be conducted at 40.6°C, whereas NCDOT requires a testing temperature of 64°C as these mixtures contain a PG64-22 asphalt binder. For Rutherford County, both mixtures with and without the baghouse fines would fail.

APA test results indicate that mixtures from both counties are susceptible to excessive rutting. However, it should be noted that pavements sections in these counties have not shown excessive rutting to date. Pavement sections in Buncombe County were observed to have slightly more rutting (which was also evident from the field cores received) compared to the cores from Rutherford County, contrary to the APA test results. Nevertheless, the objective in this study was not to estimate the rutting susceptibility of the mixtures per-se, but to evaluate the effect of baghouse fines on the mixture performance. In this regards, the APA test shows that the baghouse fines used in this study, do not have any effect on the performance of the asphalt mixtures from either counties, a result consistent with all prior performance test results presented in earlier sections.

7.2 Effect of baghouse fines on moisture sensitivity

NCDOT Materials and Test Unit in accordance with their procedure conducted the TSR tests. It may be noted that NCDOT does not require the specimens to be subjected to freeze-thaw cycle as required under AASHTO T283 procedure. Four inch diameter specimens compacted using Marshall procedure were manufactured at NCSU materials laboratory and supplied to NCDOT for testing. In all, 8 specimens were made for each asphalt mixture with and without the baghouse fines for both counties. The results of the TSR tests are presented in Tables 7.2 through 7.5.

Table 7.6 shows the summary of TSR test results for the asphalt mixtures with and without baghouse fines for the Buncombe and Rutherford counties. Test results show that the tensile strength ratio for asphalt mixtures containing baghouse fines for Buncombe and Rutherford counties are 78-percent and 84-percent, respectively, which fails the NCDOT 85-percent tensile strength ratio requirement for surface mixtures. It may be noted that these mixtures do contain anti-strip additive with a dosage suggested in the respective NCDOT JMF’s. Mixtures without the baghouse fines meet or exceed the NCDOT requirement with mixtures from Buncombe and Rutherford counties showing an 85-percent and 92-percent tensile strength ratio, respectively.

7.3 Conclusion

The APA test results indicate that mixtures with and without baghouse fines from both counties are susceptible to excessive rutting. However, for both Buncombe as well as Rutherford counties, it was observed that the baghouse fines did not have an effect in comparison to the mixtures containing regular mineral filler materials. This observation is in agreement with other previous performance test results presented in earlier sections. However, the TSR test results clearly show that mixtures containing baghouse fines are sensitive to moisture and fail the NCDOT tensile strength ratio requirement. The mixture moisture sensitivity may therefore be one of the contributory factor in the shoving distress observed in the Buncombe County.

Table 7.1 Air voids and heights of 6-inch diameter laboratory specimens for APA test

Buncombe County				**Rutherford County**			
Sample ID	**Height (mm)**	**Air voids (%)**	**Avg. Air Void (%)**	**Sample ID**	**Height (mm)**	**Air void (%)**	**Avg. Air Void (%)**
BW02	75.6	6.5	6.5	RW05	75.6	7.7	7.6
BW03	75.5	6.5		RW06	75.4	7.4	
BWO1	75.5	6.4	6.5	RWO5	75.4	6.9	6.8
BWO2	75.5	6.6		RWO6	75.5	6.6	

Table 7.2 Buncombe County (With baghouse fines) TSR results (4-inch specimens)

Unconditioned Specimens				Conditioned Specimens			
Sample ID	Height (mm)	Air voids (%)	Max. Load (N)	Sample ID	Height (mm)	Air voids (%)	Max. Load (N)
BW01	63.9	6.9	2200	BW02	64.0	7.0	1600
BW03	63.9	6.8	2060	BW06	63.8	6.6	1750
BW05	63.8	6.7	2040	BW08	63.7	6.9	1550
BW11	63.8	7.0	2270	BW10	63.8	6.9	1700
Average		**6.9**	**2142**			**6.9**	**1650**

Table 7.3 Buncombe County (W/out baghouse fines) TSR results (4-inch specimens)

Unconditioned Specimens				Conditioned Specimens			
Sample ID	Height (mm)	Air voids (%)	Max. Load (N)	Sample ID	Height (mm)	Air voids (%)	Max. Load (N)
BWO03	64.0	6.6	1980	BWO01	63.6	6.9	1600
BWO06	63.7	6.8	2050	BWO02	63.8	6.5	1750
BWO08	63.9	6.3	2080	BWO05	63.8	6.3	1760
BWO09	63.9	6.8	1900	BWO07	63.8	6.7	1810
Average		**6.6**	**2002**			**6.6**	**1730**

Table 7.4 Rutherford County (With baghouse fines) TSR results (4-inch specimens)

Unconditioned Specimens				Conditioned Specimens			
Sample ID	Height (mm)	Air voids (%)	Max. Load (N)	Sample ID	Height (mm)	Air voids (%)	Max. Load (N)
RW03	63.7	6.9	2450	RW01	63.8	7.1	2050
RW06	63.7	7.1	2400	RW02	63.8	6.8	2050
RW07	63.8	7.0	2450	RW04	63.7	6.9	2050
RW08	63.8	6.7	2500	RW10	63.8	6.8	2050
Average		**6.9**	**2450**			**6.9**	**2050**

Table 7.5 Rutherford County (W/out baghouse fines) TSR results (4-inch specimens)

Unconditioned Specimens				Conditioned Specimens			
Sample ID	Height (mm)	Air voids (%)	Max. Load (N)	Sample ID	Height (mm)	Air voids (%)	Max. Load (N)
RWO02	63.9	6.5	2150	RWO01	63.7	6.4	1950
RWO04	63.9	6.4	2100	RWO03	63.9	6.4	2050
RWO07	63.8	6.2	2300	RWO05	64.0	6.3	2025
RWO08	63.8	6.4	2250	RWO06	63.9	6.4	2100
Average		**6.4**	**2200**			**6.4**	**2031**

Table 7.6 Summary of TSR results

County	Type of Mix	QA/QC Comparative TSR	Average Tensile Strength (kPa)		Tensile Strength Ratio (%)
			Dry	Wet	
Buncombe	With bag-fines	Minor	209.3	162.9	77.8
	Without bag-fines	Minor	203.0	172.8	85.1
Rutherford	With bag-fines	Minor	244.6	204.7	83.7
	Without bag-fines	Minor	219.4	202.5	92.3

8. SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This study investigated the cause(s) of the excessive delamination and shoving distress observed in NCDOT Division 13. Two potential 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.

Through the use of a questionnaire developed for the purpose of this study, two pavement sections in Buncombe and Rutherford counties were identified for this investigation. In both counties, the baghouse fines are intermittently purged into asphalt mixtures. In Buncombe County, the tack coat material used was a CRS-2 emulsion. However, due to severe delamination and shoving distresses observed in the pavement sections, a PG64-22 asphalt binder was used as tack coat in some pavement sections in Rutherford County.

Cores and raw materials were obtained from pavement sections in both counties for forensic analysis of the in-situ materials and to evaluate the laboratory performance of the mixtures containing baghouse fines. The core samples obtained were subjected to the volumetric and stability analysis, and to 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 generally within the NCDOT mixture design specifications and should have performed well in-situ under normal traffic loading. Rutherford County mixes showed slightly higher air void content and flow values, however, no excessive distresses were observed in-situ. For Buncombe County, the mixtures appeared to be designed within specifications, and although, the pavement sections were not expected to show any excessive distresses, delamination and shoving had been a major problem.

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. The dynamic mechanical analysis of the mastics using the DSR suggested 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 appeared to enhance the rut resistance of the asphalt mixtures. This finding was in agreement with the laboratory performance test results.

Based on the FSCH and RSCH test results for laboratory mixes containing baghouse fines, the following may be concluded:

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 not containing baghouse fines;
3. Respective mixtures from both counties show similar dynamic shear stiffness and rutting characteristics.

Mixtures containing regular mineral filler and baghouse fines were subjected to APA testing at NCDOT Materials and Test Unit. 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 confirms 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 are moisture sensitive as compared to the mixtures containing regular mineral filler even though, an anti-strip additive was used for both mixtures. The TSR ratios for the Buncombe County mixtures were 78 and 85-percent for mixtures with and without baghouse fines, respectively. The TSR ratios for Rutherford County mixtures were 83 and 92-percent for mixtures with and without baghouse fines, respectively.

Based on the performance test results for the evaluation of the bond strength of the in-situ cores, the following may be concluded:

1. The FSCH, RSCH, and AFST tests clearly demonstrate the difference between the cores from the 'good' and 'bad' pavement sections in Buncombe County as identified visually during the coring operation. Consistent with the visual field observation, these tests do not show any significant difference in performance of the 'good' and 'bad' pavement sections in Rutherford County.
2. RSCH test results show that for the Buncombe County cores that were tacked with CRS-2 emulsion, horizontal crack through the tack coat was observed. For the Rutherford County cores that were tacked with PG64-22, cracking pattern was diagonal, a pattern more in line with that usually observed for monolithic sections. Moreover, Buncombe County cores, in general, failed at much lower number of loading cycles compared to the cores from Rutherford County with overall difference of 40-percent.
3. Similar to RSCH test, the uniaxial tensile (VRAMP) test also clearly showed that the bond strength of CRS-2 emulsion to be 46-percent lower as compared to the cores from Rutherford County. For the Buncombe County cores, failure was observed at the interface of the asphalt layers with clear separation of the two layers. For the Rutherford County Cores, failure was observed in the asphalt mix layers.
4. Considering that both the Buncombe and Rutherford County mixes were found to adhere to NCDOT specifications, and that the laboratory performance of the mixtures with and without baghouse fines were similar, it may be concluded that the PG64-22 tack coat provided a better interfacial bond compared to the 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 the contributory factor in the delamination and shoving distress observed in NCDOT Division 13. It appears that the mechanism by which this distress is manifested is the following:

- Some in-situ mixtures may contain very high proportion of baghouse fines in relation to regular fines due to intermittent purging of the baghouse fines. Although the NCDOT JMF requires use of an anti-strip additive, the dosage does not appear to be sufficient to counter act 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 is not able to 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 appears to provide 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.

9. REFERENCES

1. American Association of State Highway and Transportation Officials, "Standard Test method for determining the Permanent Deformation and Fatigue Cracking Characteristics of Hot Mix Asphalt using a Simple Shear Test Device," AASHTO Designation TP7, (1996).
2. Asphalt Institute, "The Asphalt Handbook," *Manual Series No. 4 (MS-4),* 1989.
3. Asphalt Institute, "Asphalt in Pavement Maintenance," *Manual Series No. 16 (MS-16), Third Edition.*
4. Goodman, R. E., Taylor, R. L., and Brekke, T. L., "A Model for the Mechanics of Jointed Rock," *Journal of the Soil Mechanics and Foundations Division,* Vol. 94 (1967), pp. 637-659.
5. Harris, B. M., and Stuart, K. D., "Analysis of Mineral Fillers and Mastics used in Stone Matrix," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 64 (1995), pp. 54-95.
6. Heukelom, W. "The Role of Filler in Bituminous Mixes," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 34 (1965), pp. 396-429.
7. Huschek, S., and Angst, C., "Mechanical Properties of Filler-Bitumen Mixes at High and Low Service Temperatures," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 49 (1980), pp. 440-475.
8. Ishai, I., and Craus, J., "Effect of the Filler on Aggregate-Bitumen Adhesion Properties in Bituminous Mixtures," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 46 (1977), pp. 228-258.
9. Kallas, B., and Puzinauskas, V., "A Study of Mineral Fillers in Asphalt Paving Mixtures," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 30 (1961), pp. 493-528.
10. Kandhal, P. S., "Evaluation of Baghouse Fines in Bituminous Paving Mixtures," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 50 (1981), pp. 150-210.
11. Kavussi, A., and Hicks, R., "Properties of Bituminous Mixtures containing Different Fillers," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 66 (1997), pp. 153-186.

12. Metcalf, J. B., "Accelerated Pavement Testing, a Brief Review Directed Towards Asphalt Interests," Proceedings of the Association of Asphalt Paving Technologists, vol. 67 (1998), pp. 553-572.
13. Puzinauskas, V., "Filler in Asphalt Mixtures," *The Asphalt Institute Technical Report RR-69-2,* (April -1983), pp. 493-528.
14. Paul, H. R., and Scherocman, J. A., "Friction Testing of Tack Coat Surfaces," *Transportation Research Record,* No. 1616 (1998), pp. 6-12.
15. Roberts, F. L., Kandhal, P. S., Brown, E. R., Lee, D., and Kennedy, T. W., "Hot Mix Asphalt Materials, Mixture Design, and Construction," *NAPA Education Foundation* (1996), pp. 331-337.
16. Shahin, M. Y., Kirchner, K., Blackmon, E. W., and Tomita, H., "Effect of Layer Slippage on Performance of Asphalt Concrete Pavements," *Transportation Research Record,* No. 1095 (1986), pp. 79-85.
17. Shook, J. F., Finn, F. N., Witczak, M. W., and Monismith, C. L., "Thickness Design of Asphalt Pavements – The Asphalt Institute Method," *Proceedings of the Fifth International Conference on the Structural Design of Asphalt Pavements,* Vol. I (1982), pp. 17-53.
18. "SHRP-A-415 Permanent Deformation of Asphalt Aggregate Mixes*," Strategic Highway Research Program,* National Research Council, Washington, DC (1994).
19. "SHRP Superpave Binder Selection Program – SHRPBIND," version 2.1*, Federal Highway Administration,* Pavement Performance Division, McLean, Virginia (1985).
20. Tayebali, A. A., Khosla, N. P., and Malpass, G. A., "Impact of Fines of Asphalt Mix Design," *Department of Civil Engineering, North Carolina State University, Technical Report,* (June -1997).
21. Tunnicliff, D. G., "Mineral Filler," *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 31 (1962), pp. 118-150.
22. Tschegg, E. K., Kroer, G., Tan, D., M., Stanzl-Tschegg, S. E., and Litzka, J., "Investigation of Bonding between Asphalt Layers on Road Construction," *Journal of Transportation Engineering,* Vol. 121, No. 4 (1995), pp. 309-316.
23. Uzan, J. "Influence of the Interface Condition on Stress distribution in a Layered System," *Transportation Research Record,* No. 616 (1976), pp. 71-73.

24. Uzan, J. Livneh, M., and Eshed, Y., “Investigation of Adhesion Properties between Asphaltic-Concrete Layers,” *Proceedings of the Association of Asphalt Paving Technologists,* Vol. 47 (1978), pp. 495-521.