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Evaluating Performance of Asphalt Pavement Based on Data Collected During IRP

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16. Abstract

In 2010 portions of the asphalt concrete surface layers of Arkansas' interstate system began deteriorating prematurely. Depending on the geographic location the deterioration varied significantly. The primary objective of this study was to determine why some of the pavements constructed experienced significant amounts distresses (cracking, potholes, and roughness), while other pavements have performed as expected. This objective was achieved by collecting and analyzing historical data and testing field cores in the laboratory in an attempt to establish correlations with in-place performance and the collected and analyzed data. Over the ten sections (four good, two medium, and four poor performing), it was found that higher initial IRI may predict poor performing sections, while a higher F/A ratio could also cause premature cracking. In the lab, dynamic modulus, semi-circular bend fracture, and dynamic shear rheometer provided strong confirmation of pavement performance, while there were issues with bond strength and penetration. This research confirmed the importance of collecting comprehensive and meticulous data during production and construction (i.e. job diaries), performing testing of materials before pavement failure, and the recommendation of implementing the torsion bar dynamic modulus test in specifications.

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Evaluating Performance of Asphalt Pavement Based on Data Collected During IRP (TRC1404, April 2017)

RESEARCH

PROJECT OBJECTIVES

The research project included 1) conduct a comprehensive literature search to document other state's experience with premature pavement failure, 2) compile construction data on the eight asphalt pavement sections included in this study, 3) develop a sampling and testing plan, 4) compare field data to laboratory data, and 5) produce deliverables.

SCOPE

After only 9-12 years of service, ten sections of interstates within Arkansas were prematurely deteriorating. Four poor, two fair, and four good performing sections were compared in order to evaluate potential causes of the premature deterioration. Several key properties, including structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage, were examined.

FINDINGS

The major findings of the study are summarized below.

1. There was no clear relationship between relative humidity, maximum temperature, minimum temperature, or precipitation and pavement performance.
2. There was not any direct relationship between the soil classification, moisture content, resilient modulus, or R-value and pavement performance.
3. The penetration test did not do a good job of ranking field performance.
4. A significantly higher number of layers were either debonded before coring or the coring process sheared the layers apart in the field during sampling on the poor sections.
5. The SC(B) test clearly showed higher fracture energy for good performing sections versus poor performing sections.
6. Both IDT and torsion bar dynamic modulus were generally able to predict higher values for the better sections (a stiffer, more cohesive material) and lower values for the poorer sections (a softer, less cohesive material).
7. After evaluating all ten sections, it was determined that the bond strength, in-place voids, and moisture damage contributed to the premature deterioration

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1.1 Introduction

Prior to 1999, several pavement sections of the Interstate system in Arkansas were constructed out of Portland Cement Concrete (PCC). During the 1999-2004 Interstate Rehabilitation Program (IRP), about 270 miles of these pavements were rubblized and asphalt concrete (AC) surface layers were placed on top as overlays. In 2010, however, portions of the AC surface layers began deteriorating prematurely. Depending on the geographic locations, however, the deterioration of surface layers varied significantly. For example, several sections in the west of Little Rock on I-40 have experienced severe cracking, potholes, and roughness, whereas the pavements in the southern (I-30) and eastern (east of Little Rock on I-40) parts of the state have performed as designed. These trends are shown in Figure 1.1.

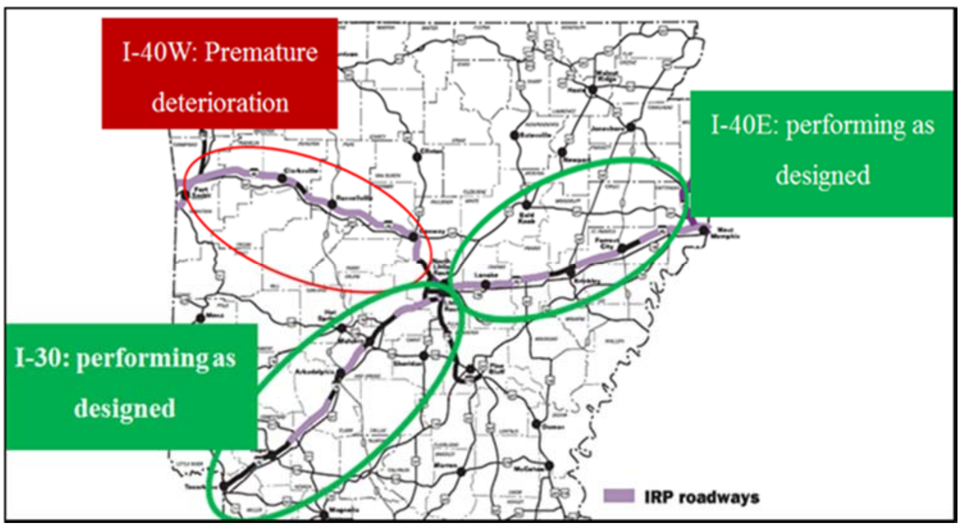

Figure 1.1 - General Pavement Conditions on Arkansas Interstates (credit: Chowdhury)

Since both I-40 and I-30 were rehabilitated approximately at the same time, and both have similar pavement structures, the question becomes why pavements of I-40 in the west are experiencing severe deterioration, while others are performing as designed. In order to answer this question, this study contains two stages of investigations. The first stage involved performing a forensic evaluation of ten rehabilitation projects to determine if there were any design, production, or construction issues, which contributed to the premature deterioration of some sections. The second stage of this study included conducting a mechanistic investigation of collected core samples from ten pavement sections (Figure 1.2) and extracted materials (aggregates and binders).

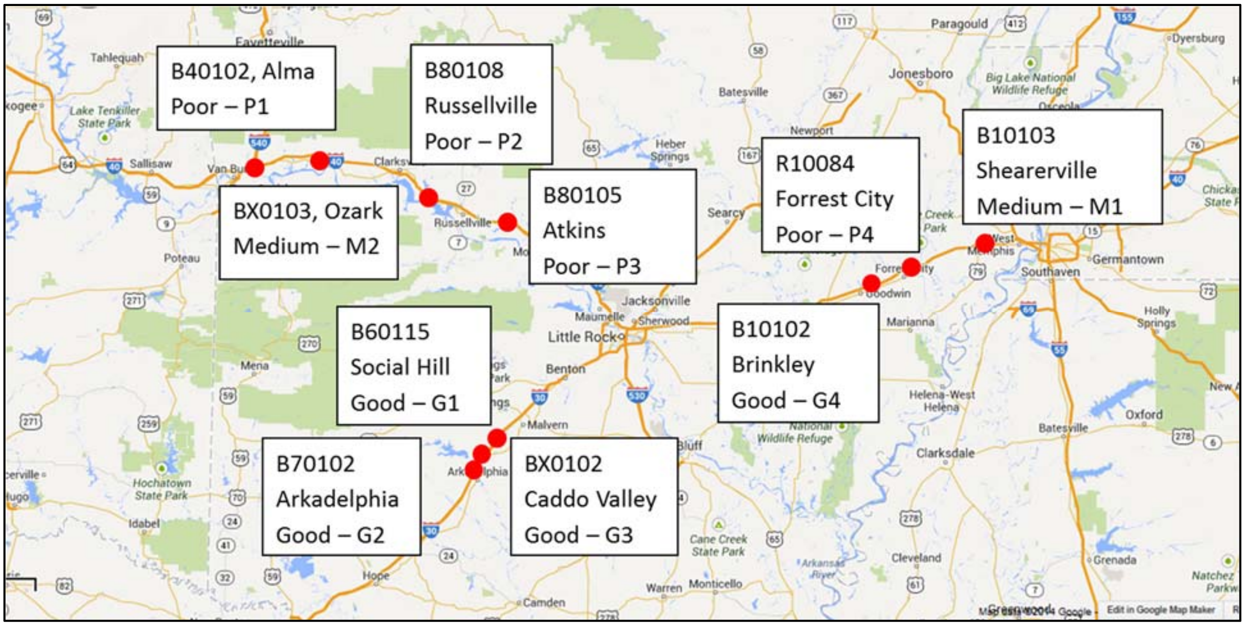

Figure 1.2 - Four Good, Four Poor, and Two Medium Performing Sections in Arkansas.

Cylindrical core samples were taken from four “Good,” four “Poor” and two “Medium” performing sections. The three “Good” pavement sections are located at Interstate 30, near Arkadelphia, where three “Poor” and one “Medium” pavement sections are located at I-40, west of Conway and one “Good,” one “Poor” and one “Medium” pavement sections are situated at I-40, East of Little Rock. At least twenty seven cores were collected from each of these sections from the right wheel path, left wheel path, and between wheel paths in order to execute the laboratory tests. Details of ten sections discussed in this study are given in Table 1. In Table 1, the designation for each section is also provided as “good” (G), “medium” (M), and “poor” (P). These initial pavement condition evaluations were based on qualitative description only by state personnel.

Table 1.1 - Detail of the Ten Pavement Sections.

Job Number	Report Designation	Interstate	Traffic Direction	Mile Marker	Nearest City
B60115	G1	I-30	West Bound	242.8,246.0,248.0	Social Hill
B70102	G2	I-30	West Bound	87.6,87.8,88.0	Arkadelphia
BX0102	G3	I-30	West Bound	84.6,84.8,85.0	Caddo Valley
B10102	G4	I-40	East Bound	223.0,223.2,223.4	Brinkley
B10103	M1	I-40	East Bound	266.3,266.5,266.7	Shearerville
BX0103	M2	I-40	West Bound	38.6, 38.7, 38.8	Ozark
B40102	P1	I-40	West Bound	14.8,15.0,15.2	Alma
B80108	P2	I-40	West Bound	64.6,64.8,68.9,70,71.5	Russellville
B80105	P3	I-40	West Bound	85.8, 89.9	Atkins
B10103	P4	I-40	West Bound	75.6,75.8,76.0	Forrest City

1.2 Objectives

The primary objective of this study was to determine why some of the pavements constructed during 1999-2004 IRP experienced significant amounts distresses (cracking, potholes, and roughness), while other pavements have performed as expected. The detail objectives are given below:

- The first objective of this study was to collect and analyze the historical data such as mix design data, construction and design, weather data during the construction, job diaries, and long term pavement performance data from Automated Road Analyzer (ARAN) for good, medium, and poor performing sections.
- The second objective was to collect cylindrical cores from good, medium, and poor performing sections in order to first run mixture test on the top two surface layers, and then extract and recover binders.
- The third objective was to evaluate the mixture and binder properties and establish correlations, if any, with the candidate parameters influencing the aforementioned distresses by conducting laboratory experiment.

1.3 Tasks

The tasks that were conducted to meet the objectives of this study were:

1. Literature review: A comprehensive literature review from several sources was done to understand the state of art practice for evaluating forensic investigation. Notable sources of literature include technical articles, from Transportation Research record, Association of Asphalt Pavement Technologies, national level reports from the Federal Highway Administration and the National Cooperative Highway Research Program (NCHRP).
2. Collect field data: Field data including mix design, construction and design, soils, weather and long term pavement performance data was collected for all ten sections.
3. Collect core samples: Two hundred and seventy (270) core samples of 4 inch and 6 inch diameter were collected from all of the ten sections.
4. Perform laboratory testing: Two sets of testing was performed on the field cores – mixture and binder testing.
    a. Asphalt mixture tests included volumetric tests, bond strength tests, IDT dynamic modulus, torsion bar dynamic modulus, SC(B) fracture tests, SC(B) stripping ratio, and ignition oven binder content.
    b. Asphalt binders tests included Rotavapor recovery, Dynamic Shear Rheometer at intermediate temperatures, penetration, and rotational viscosity.
5. Analyze field and laboratory results: Field and laboratory data were analyzed in order to establish correlations, if any, between the different asphalt mixture and asphalt binder testing and pavement distresses.

2.1 Literature Review

This literature review discusses the state of art practices of conducting forensic investigations by reviewing notable sources and finding reasons behind premature pavement failures. To this end, pertinent literature from well reputed journals such as Transportation Research Board, Federal Highway Authority, National Cooperative Highway Research Program reports, and other sources were reviewed. The main focus while reading these research articles and summarizing them was to find the possible causes of pavement deterioration and how other states or highway agencies have investigated or solved these kinds of problems.

While conducting the literature review on forensic investigation, the main scope of study was to find out how other highway agencies or research groups carried out their investigation, such as, visual investigation, collection of core samples, survey of construction feature and materials etc. Afterwards, the focus was shifted towards asphalt mixture and binder performance testing and comparing field data with laboratory test results. The test results from different highway agencies and research groups on various asphalt mixture and asphalt binder tests to determine if other analysis could lead to insight for this study.

2.2 Forensic Investigation

Clyne *et al.* (2008) performed forensic analyses of four small sections in MnRoad in order to determine the mechanism of pavement failures such as deterioration along and underneath transverse cracks, rutting on low volume road, general pavement structural failure on low volume road in the state of Minnesota. These researchers undertook a wide variety of field testing tasks such as preparing forensic trenches, collecting cores, performing dynamic cone penetrometer test, and rod and level survey as well as some laboratory testing activities that incorporated determining the reasons of deterioration along transverse cracks, rutting, structure failure and faulting. From the visual inspection and laboratory test results, the authors recommended proper compaction and selection of material, add dowels in concrete pavements that will improve the long term performance. For asphalt pavements, material properties such as unbound material moisture content, and asphalt binder content and grade were reported to be contributing factors for rutting and stripping. In regard to preventive maintenance, it was suggested to prevent the cracking in the first place. The authors also emphasized on proper compaction, selection of material, and pavement design for preventing cracks and premature deterioration.

Sargrandet *et al.* (2010) reported that local material properties and design factors should be considered most prevailing parameters in predicting performance and durability of future pavement projects in Ohio. Among 20 test sections these researchers investigated, Gallia County (State Route -7) and Athens County (US Route 33) exhibited exceptional performance compared to other 18 test sections. It was reported that use of low cementitious material content, large aggregate content, 19 mm diameter dowel bar in control joints with hot mix asphalt bituminous joint filler material was the main reason of such exceptional longevity.

Chen *et al.* (2006) conducted a forensic investigation on highway systems in Texas and evaluated pertinent materials for gradation, moisture content, capillary action, and indirect tensile strength. Further extensive field measurements were taken using the following devices: (i) a dynamic cone penetrometer (DCP) to measure the variation in strength with depth, (ii) a seismic pavement analyzer (SPA) to estimate the layer thickness and modulus with depth as well as the overall condition, and (iii) a portable seismic pavement analyzer (PSPA) to measure the stiffness of the base and subgrade stiffness after the overlying layers were removed. Among the four trenches opened to conduct tests directly on the top of each pavement layers, two sections (T 240 and T 260) demonstrated higher FWD deflections and severe distress while other two sections (T 218 and T 234) deflected less and have no distress. Based on the SPA and DCP test results, the authors also observed that the base was stiff at location T218. Further, trenches at locations T240 and T260 exhibited deflections three to four times higher than those at locations T 218 and T 234.

A follow up study by Chen and Scullion (2007) investigated three asphalt concrete (AC) field projects, namely, Projects 1, 2 and 3, to illustrate an integrated approach used widely in Texas. A combination of field non-destructive testing, trenching, coring, lab testing, and a thorough review of construction records and rehabilitation history was reported to enable engineers to determine the root causes of the pavement failures. In particular, the application of nondestructive testing such as ground penetrating radar (GPR) and FWD, as well as field testing such as DCP, coring, and laboratory testing were found to be critical to these forensic investigations on AC pavements. In that study, the extent of stripping and porosity that caused delamination for projects 1 (US 281) and project 2 (US 69) was determined by GPR, FWD and evaluation of core samples. These researchers found that moisture entered in the base of Project 3 (a detour section that contained Interstate traffic) through the poorly compacted AC layer and longitudinal joints after testing from GPR, lab density and permeability tests. Further it was reported that rehabilitation strategies were aimed at strengthening the pavement sections because FWD data demonstrated that the pavement structure for Projects 1 and 2 were inadequate.

Battaglia *et al.* (2010) investigated two perpetual pavement test sections on the entrance ramp to I-94 from the Kenosha Safety and Weigh Station Facility in Southeastern WI, both constructed in 2003. It is observed that FWD test data showed little difference in pavement layer moduli, while the Superpave shear tester and simple performance test results indicated that both test sections had sufficient rutting resistance. The authors recommended using a stiffer binder (PG 76- 28) that had better rutting properties. It was also mentioned that the maximum allowable strain ($\varepsilon_{allowable}$) value should be $70x10^{-6}$, which was a conservative value for the HMA endurance limit.

Toward performing forensic analysis, Victorine *et al.* (1997) developed a database that contained useful information such as: critical design, construction, and laboratory information for identifying premature pavement failures. These authors also recommended protocols for forensic analysis for TxDOT. The protocols included review of past work, field data conditions, existing databases and data collection procedures followed by the development of conceptual framework, and preparation of interim and final reports. Finally these researchers introduced a GIS- oriented forensic information and analysis system (ForenSys) that would provide pavement engineers with the ability to find with pavements with exact or similar characteristics within a certain tolerable range through GIS.

Kandhal and Rickards (2001) documented the effect of pavement saturation on stripping from four case histories: two from Oklahoma, one from Pennsylvania, and one from Australia. In this study, stripping was investigated from a global perspective, looking at the relative permeability of the pavement components, subsurface drainage system, and the interaction between different pavement layers. These researchers documented the details of construction, visual observation of pavement distress and testing of pavement. These researchers suspected the pavement stripping in Will Rogers Parkway in Oklahoma was due to the presence of open-graded friction course (OGFC) at the top. Moreover, there was no outlet for the drainage of subsurface water or the water coming from the cracks and joints from the pavement structures. It is recommended that patch repairs could be successful if the deteriorated (stripped) OGFC (Oklahoma Type B) mix was also removed and replaced with a dense-graded Oklahoma Type B mix along with a positive subsurface drainage system was installed. In the case of the second site (I-40), several potholes were reported on the outside wheel track of westbound lane. These potholes were continuously fed with water from the remaining width of the pavement towards the median. The selection of course graded pervious F binder mix underneath OGFC was considered as the primary cause of stripping in this section. In the case of pavement section in New South Wales, Australia, the authors recommended to provide asphalt treated pavement material (APTM) base course with relatively fine graded surface course mix with not more than 12.5 mm. In case of Pennsylvania, the researchers suggested to provide 100-mm thick layer of APTM drainage course right over the rubblized PCC pavement and 1-1/2% of hydrated lime (by weight of aggregate) as an antistripping agent. The authors reported that under saturated conditions all asphalt mixes may fail as a consequence of cyclical hydraulic stress physically scouring the asphalt binder from the aggregate. Besides high air void content, these researchers reported three essential factors to promote stripping: the presence of water (degree of saturation), high stress, and high temperature. Among these factors, the degree of saturation of the pavement and asphalt layers was reported to be a critical element to appraise stripping failures. Thus, it was reported to include a measure of the moisture conditions in failed and non-failed sections of each project to ascertain the degree of saturation while performing forensic examinations of stripping failures

2.3 Asphalt Mixture Testing

For this literature review, three asphalt mixture tests were explored: the bond strength test, the SC(B) fracture test, and dynamic modulus.

2.3.1 Bond Strength Test

At the University of Arkansas, Hall and RamakrishnaReddy developed the bond strength testing device in mid-2000's (Hall and RamakrishnaReddy, 2012), which will be applied in this project. The effects of emulsion types, application rates, testing temperatures, and normal stresses on bond strength were studied. The concept of a bond strength test is important, as flexible pavements are designed to work as a single robust pavement structure. If the layers are not bonded together well, the single pavement structure acts more like several unattached thin pavement layers, significantly reducing the strength of the pavement.

In order to investigate the bonding between asphalt concrete and Portland Cement Concrete, Leng developed a direct shear test device (Leng, 2008). Different mixtures, tack coats, tack coats applicant rate and testing temperature were studied for the effect on bond strength.

2.3.2 SC(B) Fracture Test

Cracking is one of the primary distress mechanisms in asphalt concrete, and crack could introduce further damage to the pavement. There are three mechanism of cracking: fatigue, low temperature, or reflective mechanisms. The development of cracks in the pavement surface is undesirable. Currently, there are several performance tests that quantify cracking performance, including the four-point bending beam (fatigue), the Superpave Indirect Tension Test (low temperature), and the Texas Overlay Tester (reflective). However, fracture testing has gained traction in quantifying cracking of asphalt concrete in recent years.

As one the fracture tests, the Semi-Circular Bend test, or SC(B), has been successfully applied to investigate the fracture resistance of HMA (Li and Marasteanu, 2004). Although the stress states in an SC(B) test is complicated, it is easy to obtain SC(B) samples from field core and gyratory compacted samples. This test utilizes a three point bending load configuration, which is easy to be applied (Wagoner *et al.* 2005).

2.3.3 Dynamic Modulus

Dynamic modulus is an important fundamental input in the Mechanistic-Empirical Pavement Design Guide (MEPDG) or DARwin-ME. It is a function of temperature, load frequency, and mixture properties. The laboratory tested dynamic modulus is and necessary inputs for the level 1 in flexible pavement design. Pellinen and Witczak (2002) investigated the correlation between dynamic modulus and field performance including rutting, thermal and fatigue cracking, and dynamic modulus was recommended to be the Simple Performance Test parameter for rutting and fatigue cracking.

In this project, dynamic modulus will be tested with the field cores. Gedafa *et al.* (2010) compared the dynamic modulus among filed cores, laboratory-compacted samples, and drew the conclusion that the dynamic modulus from those filed cores are comparable to laboratory- compacted samples at 4˚C , variation increases as the test temperature increasing. As field cores will be composited by surface, binder and base courses, axial compression dynamic modulus test configuration cannot be used for the material from each layer if the thickness of the layer is lower than 6 inch. North Carolina Department Transportation studied the dynamic modulus test of hot mix asphalt using the indirect tension (IDT) mode, and evaluated the accuracy. The solution of dynamic modulus was evaluated as validated from the experimental data from axial compression and IDT test. (Kim *et al.* 2007).

2.4 Asphalt Binder Testing

In a laboratory study, Boriack *et al.* (2014) investigated the performance (stiffness, rutting and fatigue resistance)of asphalt concrete mixes using three different percentages (0%, 20%, and 40%) of reclaimed asphalt pavement (RAP) with three different percentages (design, design+0.5% and design+1%) of added asphalt binder. These researchers found a significant decrease in dynamic modulus values comparing the design mix to the mix with the additional 1% binder for the 0% RAP, 20% RAP and 40% RAP mixes. For both 0% and 20% RAP mixes it was reported that about 100% increase in the flow number for the design+0.0% binder and the design+0.5% binder, respectively. However, the highest flow number was observed in 40% RAP when added to the base binder. Finally, the researchers observed the slope of stiffness versus fatigue cycle milder as the binder content increased in the 0%, 20% and 40% RAP mixes.

Kannan *et al.* (2014) studied two PG binders (PG 46-34 and PG 64-22) with three levels (0%, 2.5% and 7.1%) of recycled asphalt shingles (RAS) to evaluate the characteristics of RAS by determining the viscoelastic modulus, shear strength and fatigue resistance. As expected, it was observed that both binders with 7.1% RAS exhibited higher complex modulus than the others. Similarly, the mix of PG 64-22 and PG 46-34 with 7.1% RAS showed the highest shear strength followed by the mix with PG 64-22 with 0% RAS. The researchers observed that strain controlled fatigue test (0.15% strain rate) showed a reduction in fatigue life as the RAS content increased. On the other hand, when a stress controlled fatigue test (200,000 Pa and 300,000 Pa) was used the fatigue life was improved as RAS content increased.

Mohseni *et al.* (2014) studied a new repeated load test referred as Incremental Repeated Load Permanent Deformation (iRLPD) in lieu of the multi stress creep recovery (MSCR) test for high temperature (PG 82-22 to PG 58-28) characterization of highly modified asphalt binders. Twelve binders were tested according to iRLPD and MSCR test methods to observe the difference. The researchers reported that the loading time (0.1 second), mode of the test and method of calculating test parameters for iRLPD are different from MSCR test. The researchers reported that binder samples tested at a high loading time tended to flow and the stress strain relationship became unstable and increased binder modification increased stress strain nonlinearity that also affects the permanent strain at high loading time of the MSCR test.

Tan *et al.* (2014) studied five neat and two modified binders to establish a unified evaluation index (R_j) for the high and low temperature performance of asphalt binder. Dynamic shear test and repeated creep recovery test were done to evaluate high temperature performance while the Bending Beam Rheometer (BBR) test was done to correlate low temperature performance of binders. It was reported that in regards to the G*/sinδ value of actual rut depth, the high-temperature performance the neat asphalt binders correlated well with that of the neat asphalt binder mixtures. However, the G*/sinδ value of the modified asphalt binder does not correlate well with rutting of modified binder mixes.

Johnson and Hesp (2014) investigated the effect of waste engine oil (WEO) modification on hardening tendencies for a set of well-defined asphalt from Strategic Highway Research Program (SHRP) Materials Reference Library (MRL). To this end, five SHRP MRL binders and one commercial binder from Ontario were modified with 15% WEO. The unaged, RTFO-aged and PAV-aged residues were tested in the DSR and BBR to obtain high temperature, intermediate and low temperature properties. The researchers observed that for all temperature and frequencies, the complex modulus increased by a modest amount and the phase angle decreased more significantly. These researchers reported the tendencies to become more elastic than viscous when modified with WEO suggested less ability to heal micro cracks at high temperature during summer.

McDaniel *et al.* (2000) investigated the effects of the hardened RAP binder on the blended binder and mix properties by testing binder and mixes prepared from two virgin binders and three RAP binders recovered from three RAP sources from Florida, Connecticut and Arizona. The aging of mixes was accomplished by heating at 155-160°C for four hours (short-term aging). The mixtures were compacted in a gyratory compactor to reach a specific air voids level. Engineering properties and critical temperatures were determined by Superpave tests (DSR, BBR). The high critical temperatures were reported to be 53.9°C, 67.8°C for virgin binders and 82.2°C, 82.4°C and 89.0°C for RAP binders. The authors also compared the high and low strain test results of PG 52-34 mixtures with RAPs from Connecticut and Arizona. It was observed that the low strain samples exhibited a constantly higher stiffness than the high strain samples. It was reported that the addition of RAP would increase the stiffness of mix. Hence, RAP mix would decrease the life of an asphalt pavement if no adjustment is made to the virgin binder grade.

Stephens *et al.* (2001) extracted and recovered the RAP binders (obtained from Connecticut) using the Abson recovery method to evaluate the effects of high RAP contents in the PG grade of the virgin binder. It was reported that the recovered binder was blended with the virgin PG 64-28 binder and after that the blend of recovered and virgin binder was tested outlined by AASHTO MP-1. The authors found that the completely blended binder produced by the plant mix blending is stiffer than the laboratory blending. These researchers reported possible problems of using higher percentages (more than 25% RAP) of RAP as the RAP modified binders may produce undesirable reactions between the virgin binder and RAP binder. The authors observed increases of compression and tensile strengths of the mix by one third when using 15% RAP compared to the virgin mix using the same virgin binder. It was also reported that specimens containing a virgin PG 64-28 binder and 15% RAP produced the most effective PG grade.

As noted earlier, to evaluate the stiffness of RAP binders, they are often examined for viscosity and performance grading after they are extracted and recovered from RAP samples. Among available recovery techniques, the Abson recovery method (AASHTO T 170) is reported to produce samples with the highest variability in test results among the recovery procedures studied (Anderson 2003). On the other hand, the Rotavapor method (AASHTO T 319) is reported to show less influence on binder grading as the solvent-asphalt mixture is heated more gently in a rotating flask in an oil bath. The current study is planning to pursue the Rotavapor technique to recover binders from core samples collected from the project sites. A combination toluene (85%) and alcohol (15%) will be used as the solvent during the extraction process.

3.1 Collect Field Data and Field Cores

In order to troubleshoot the ten sections of pavement, several characteristics of the jobs were explored in a full forensic analysis, including a review of the job diaries, weather data around the documented placement of pavement layers, the rutting and cracking data over the life of the pavement, a review of the mix designs, and soil characteristics. In addition, samples were collected from all ten sections for laboratory asphalt mixture and asphalt binder testing. This section will review the characteristics of the jobs and some examples of sampling of the field cores.

3.2 Review of Job Diaries

Unfortunately, not all job diaries and other paper documents were available for review from AHTD. Table 3.1 indicates which sections had access and which did not. Note, if the job diaries were not available, weather data and mix designs were not generally available either. This is a significant danger in performing a forensic analysis up to fifteen years after the pavement was designed and constructed, as often documents are lost over time.

Table 3.1 - Detail of the Availability of Job Diaries.

Job Number	Report Designation	Paper Documents Available?	Paving Dates	Water Added to Tack Coat (%)	Tack Coat Application (gal/yd^2)
B60115	G1	No	-	-	-
B70102	G2	No	-	-	-
BX0102	G3	No	-	-	-
B10102	G4	Yes	Jul. 2001 – Jun. 2002	50	0.017 – 0.051
B10103	M1	Yes	Oct. 2001 – Sept. 2002	50	0.018 – 0.073
BX0103	M2	Yes	Mar. 2004 – Aug. 2004	50	0.01 – 0.05
B40102	P1	Yes	Apr. 2002 – Jan. 2004	30	0.03 – 0.05
B80108	P2	Yes	May 2002 – Sept. 2004	50	0.011 – 0.048
B80105	P3	Yes	Apr. 2001 – Apr. 2004	50	0.012 – 0.081
B10103	P4	No	-	-	-

For all of the pavement sections, a slow-set, low viscosity anionic emulsion was used as tack coat (SS-1). No information was provided on the supplier of the emulsion. No emulsion was made available for testing for this research, therefore, no performance tests were run on the material itself. The material was only tested indirectly through the bond strength test. Table 3.1 shows that there appeared to be no relationship between either the water added to the tack coat or the tack coat application. It was anticipated that perhaps the poor performing sections did not have as much tack coat application, but that is not the case. The range of tack coat shows the minimum and maximum observed application rate during the duration of the project.

3.3 Weather Data

Once the paving dates were established, weather data was collected from the National Oceanic and Atmospheric Administration (NOAA). Data collected included relative humidity, minimum temperature, maximum temperature, and precipitation. Figures 3.1 – 3.6 show the final surface course paving along with weather events in the vicinity.

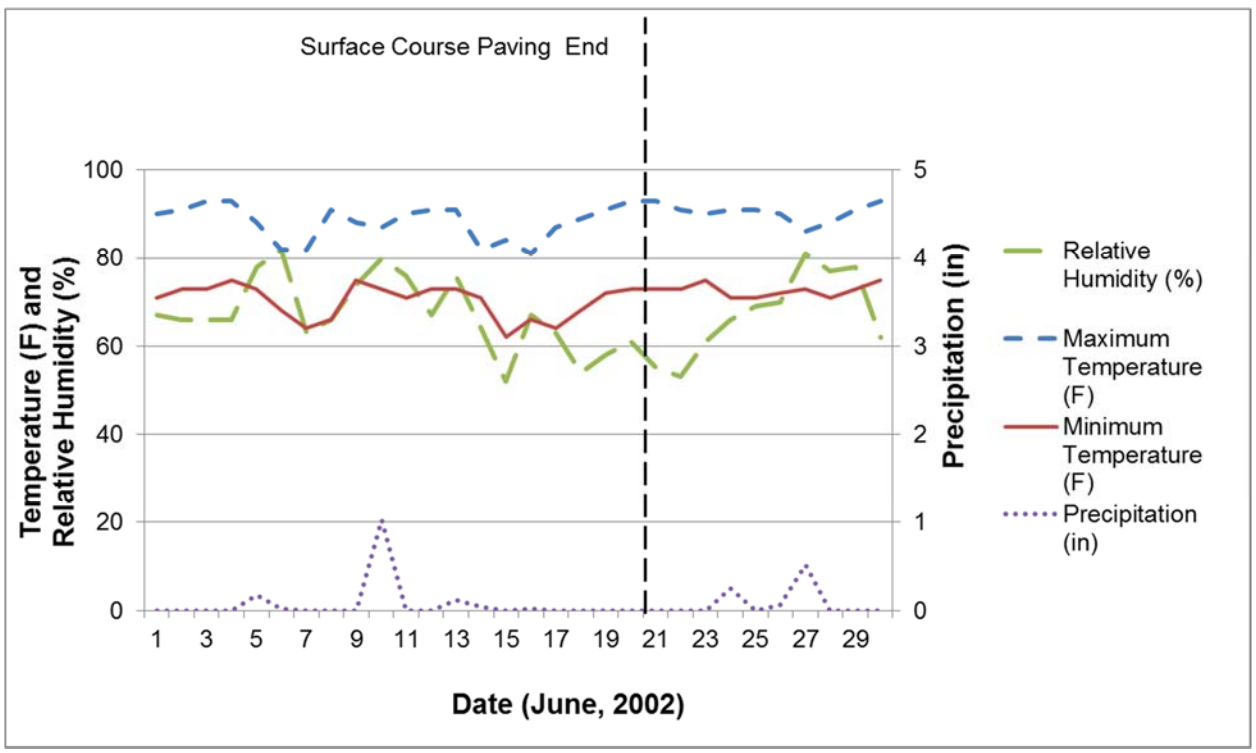

Figure 3.1 – Weather Summary at End of Surface Course Paving (B10102, Section G4)

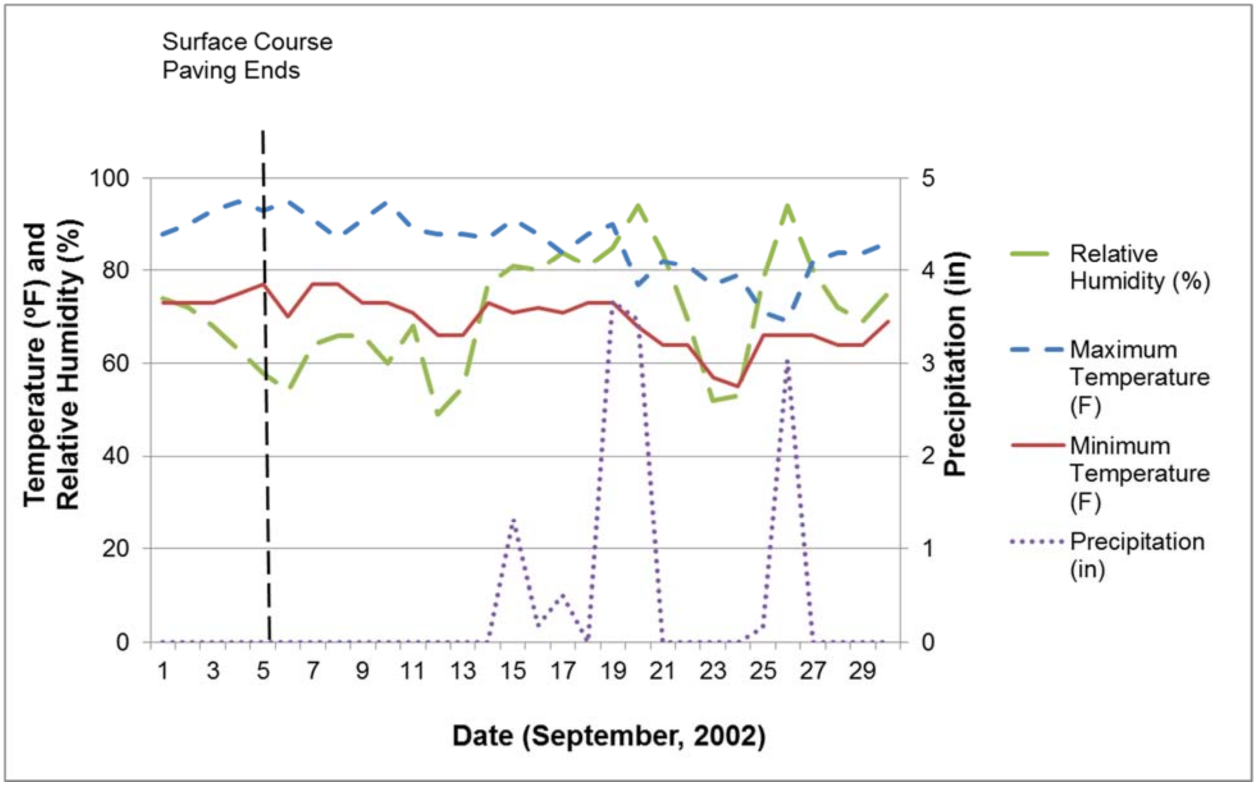

Figure 3.2 – Weather Summary at End of Surface Course Paving (B10103, Section M1)

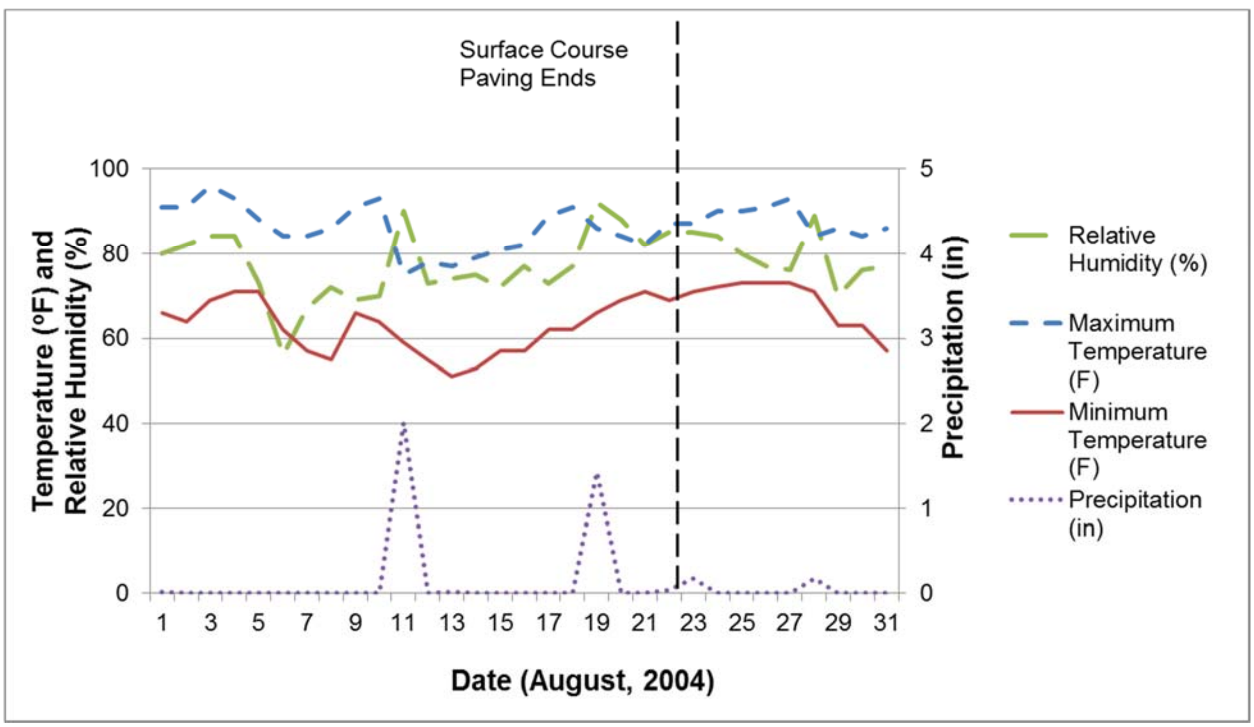

Figure 3.3 – Weather Summary at End of Surface Course Paving (BX0103, Section M2)

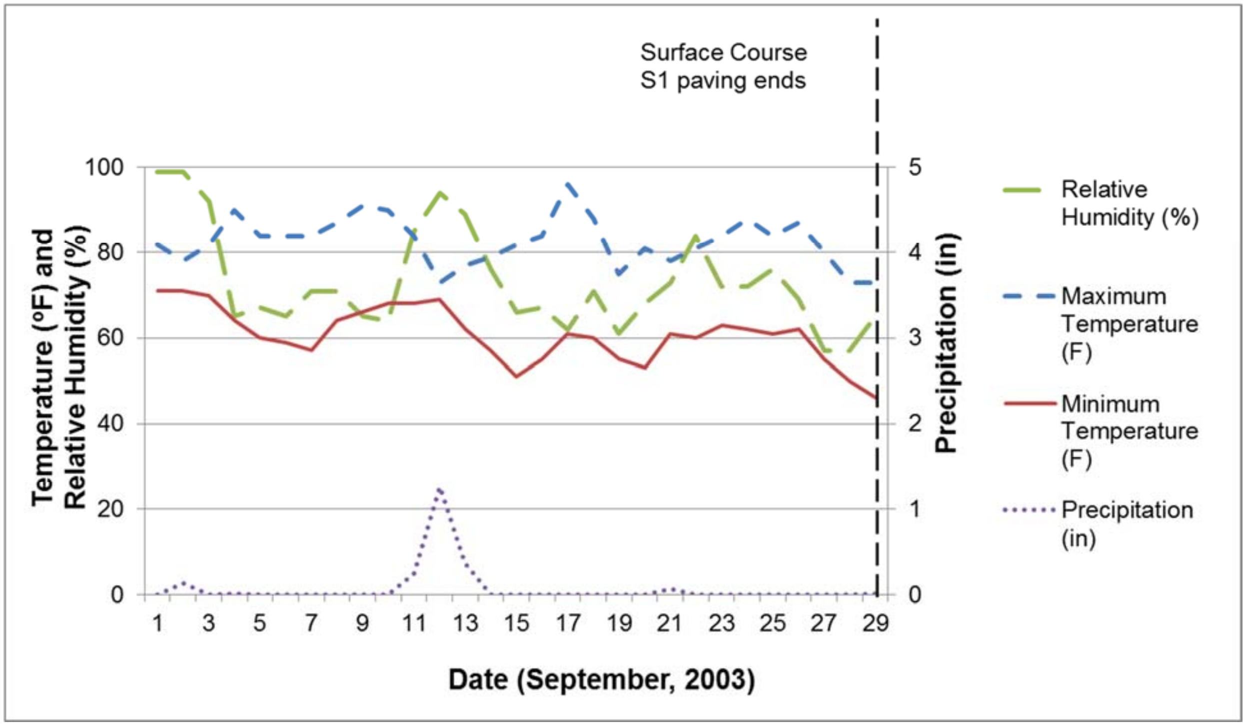

Figure 3.4 – Weather Summary at End of Surface Course Paving (B40102, Section P1)

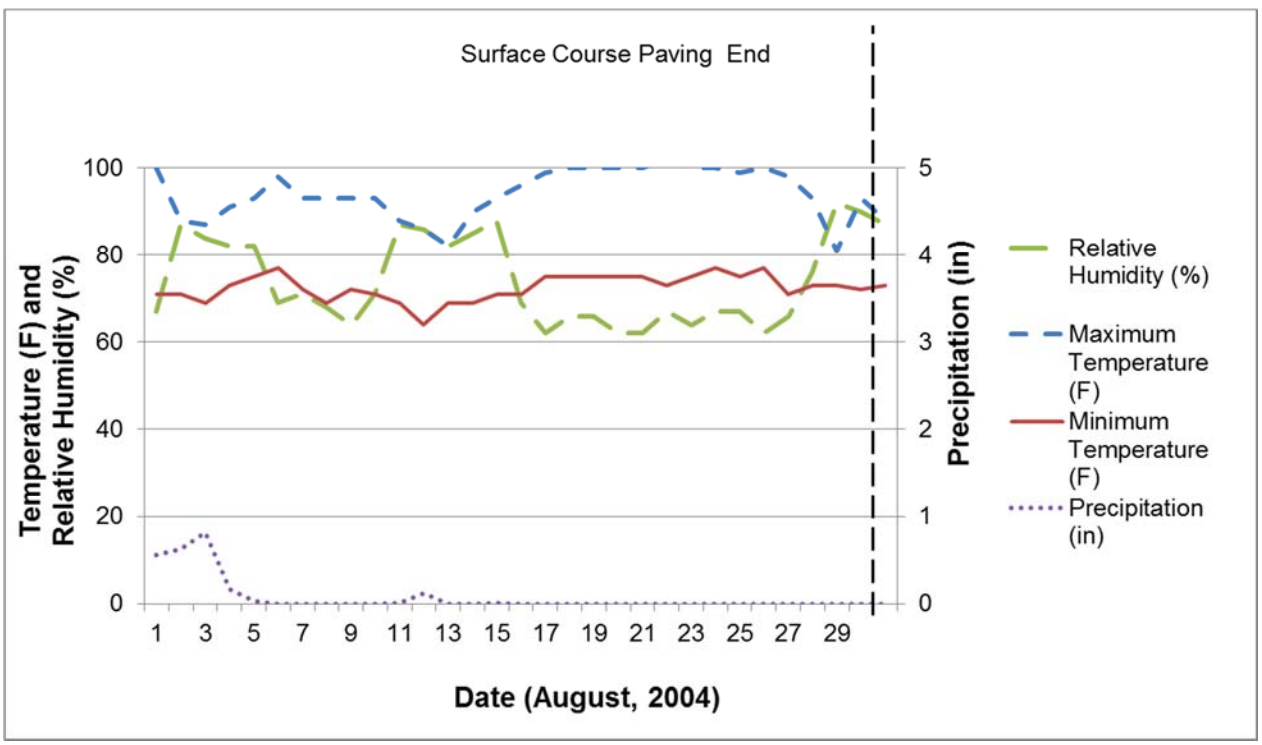

Figure 3.5 – Weather Summary at End of Surface Course Paving (B80108, Section P2)

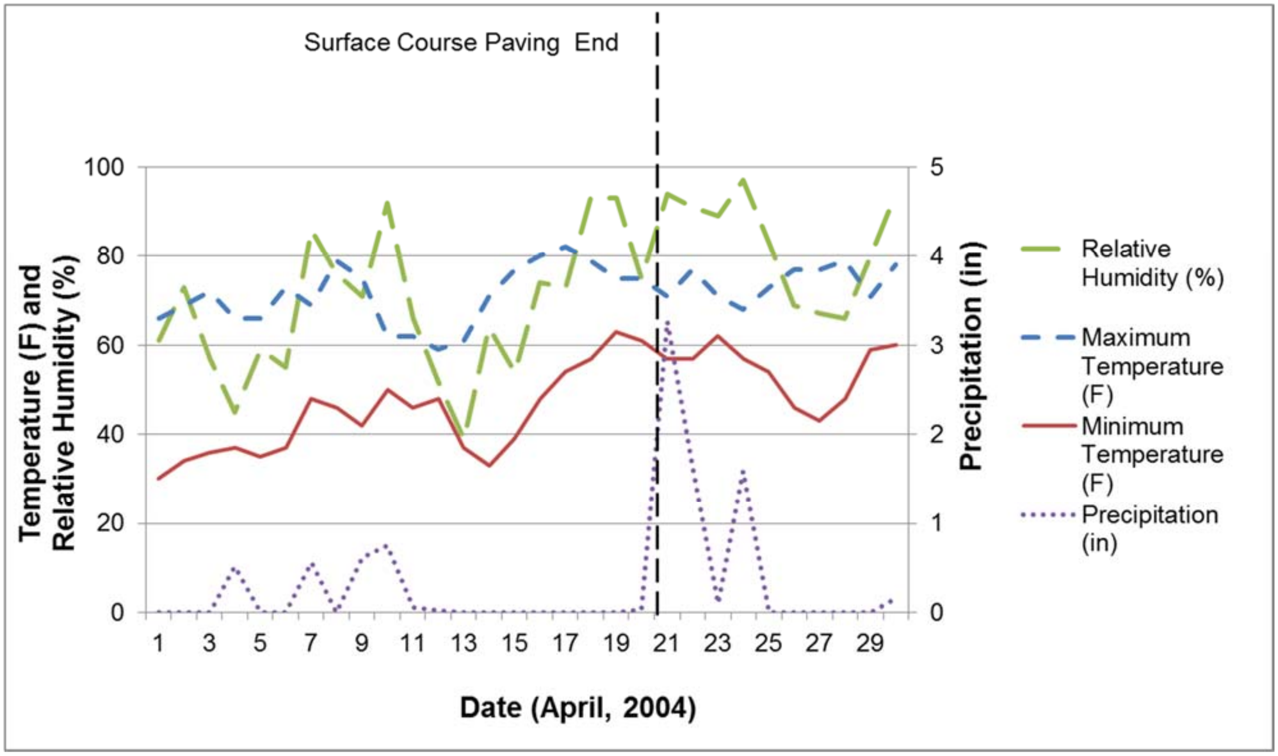

Figure 3.6 – Weather Summary at End of Surface Course Paving (B80105, Section P3)

While there are some potential issues with the weather around paving, for example, there was significant rain immediately after the surface course paving ended in Section P3, there was no clear relationship between relative humidity, maximum temperature, minimum temperature, or precipitation and pavement performance. While it is unfortunate that no relationship was found between weather and performance, it is still important to investigate as many avenues as possible for a complete research effort.

3.4 Rutting and Cracking Data

AHTD utilizes an ARAN in order to collect field data of Arkansas' highway network. Specifically, the ARAN collected International Roughness Index (IRI) data and rutting data. This is a valuable tool for the department in order to track the quality of the roadways. Figures 3.7-3.16 shows the data for the ten sections.

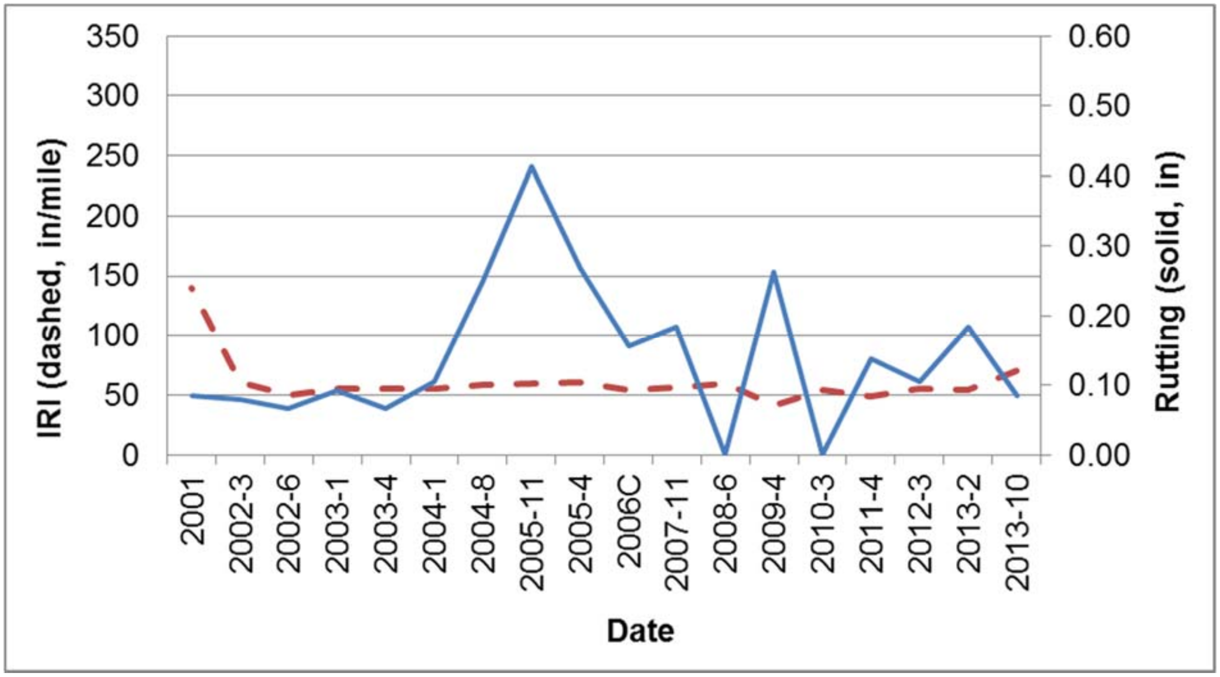

Figure 3.7 – Rutting and Cracking Data (B60115, Section G1, no construction dates available)

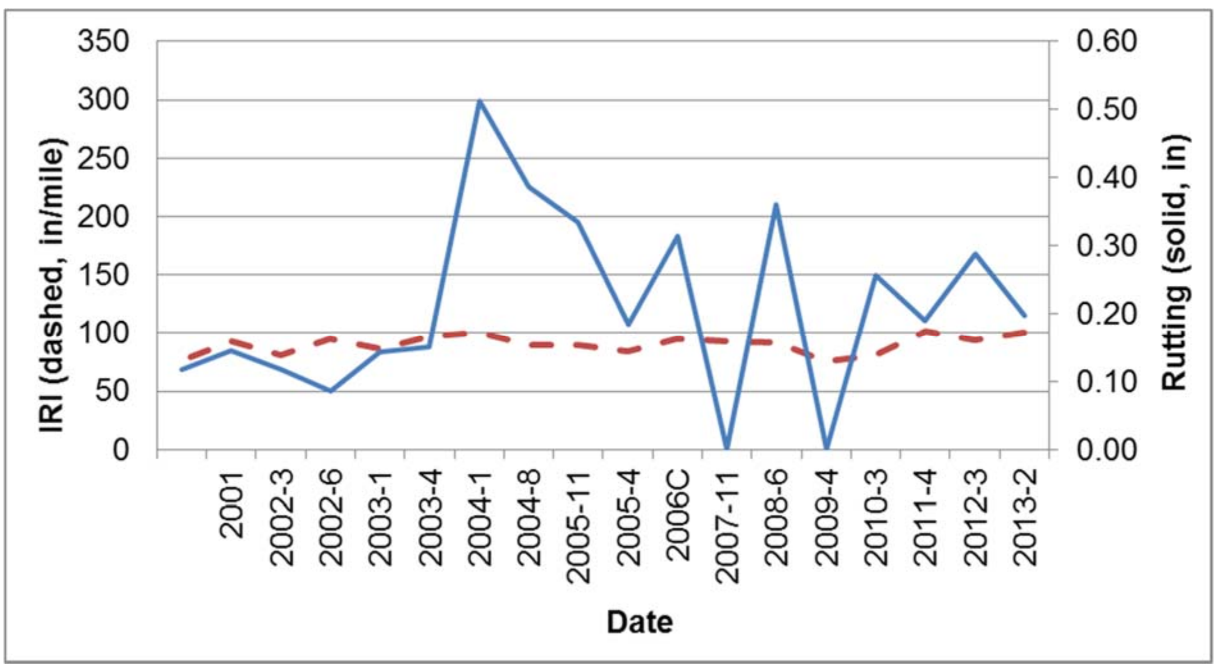

Figure 3.8 – Rutting and Cracking Data (B70102, Section G2, no construction dates available)

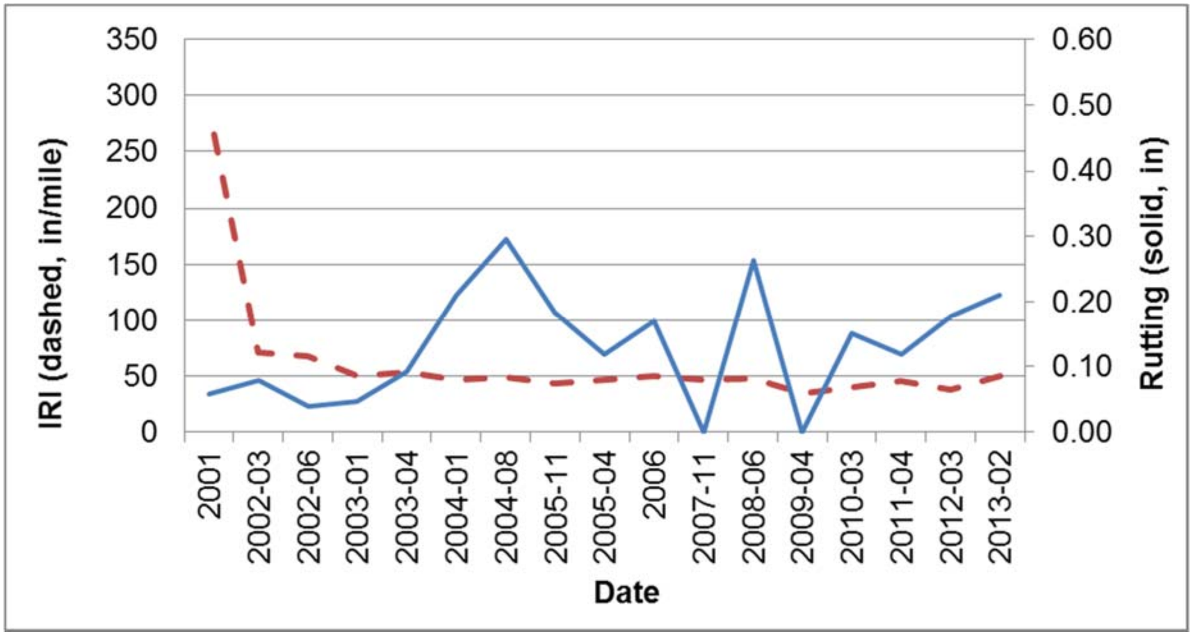

Figure 3.9 – Rutting and Cracking Data (BX0102, Section G3, no construction dates available)

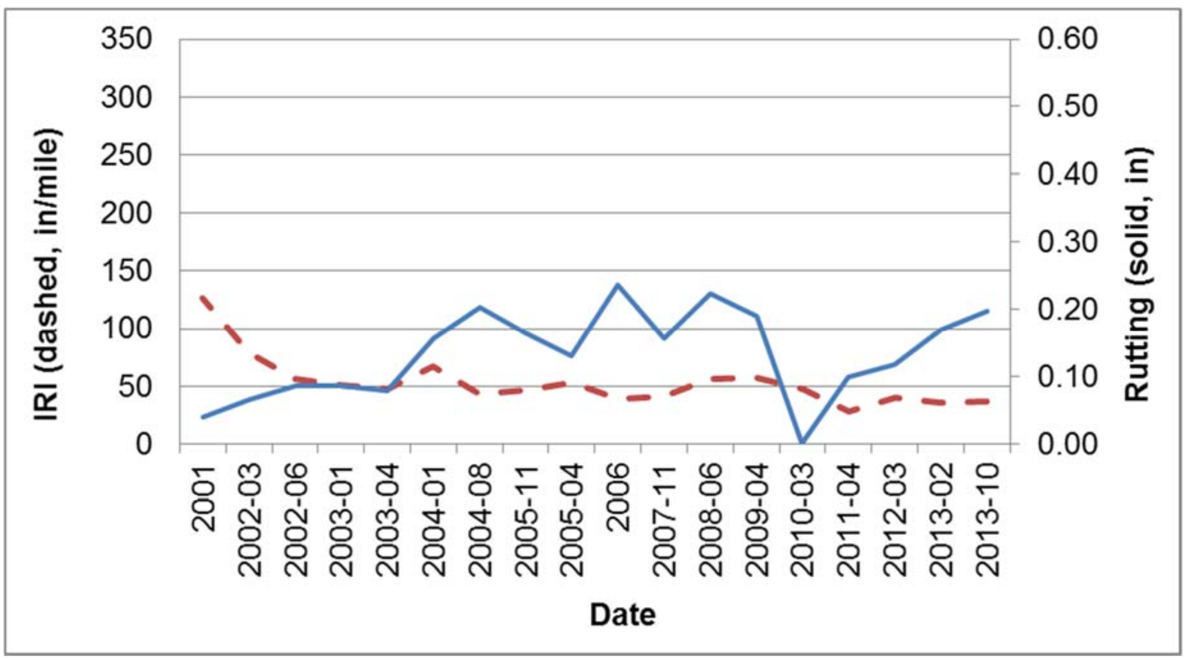

Figure 3.10 – Rutting and Cracking Data (B10102, Section G4, constructed Jul. 2001 – Jun. 2002)

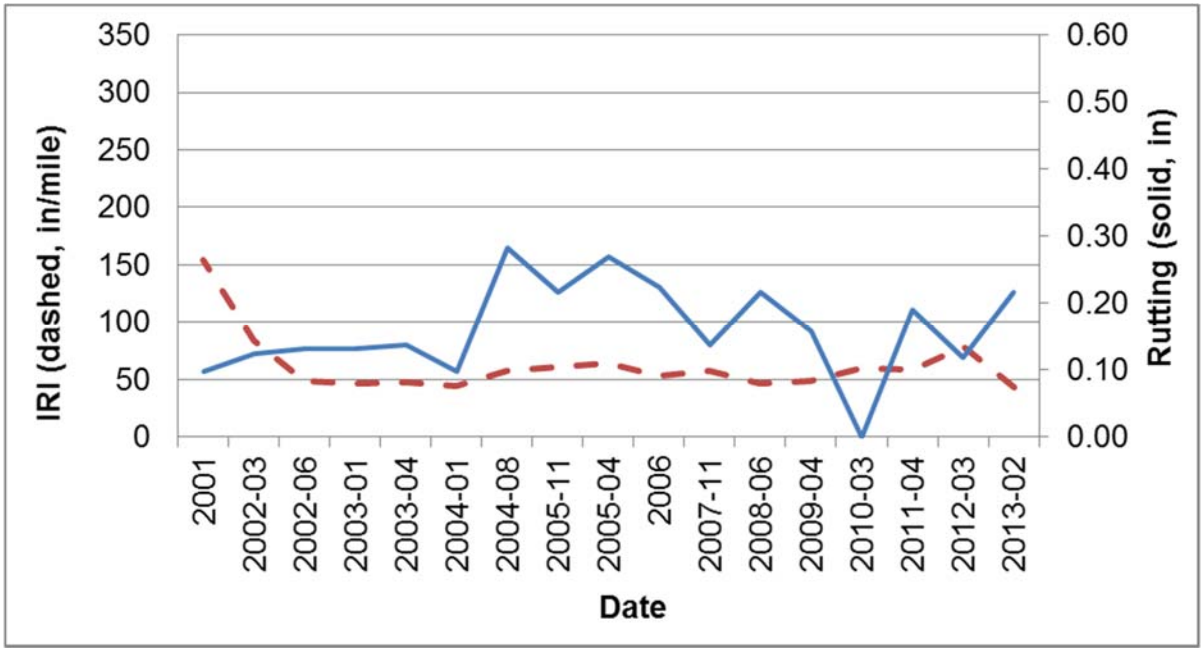

Figure 3.11 – Rutting and Cracking Data (B10103, Section M1, constructed Oct. 2001 – Jul. 2002)

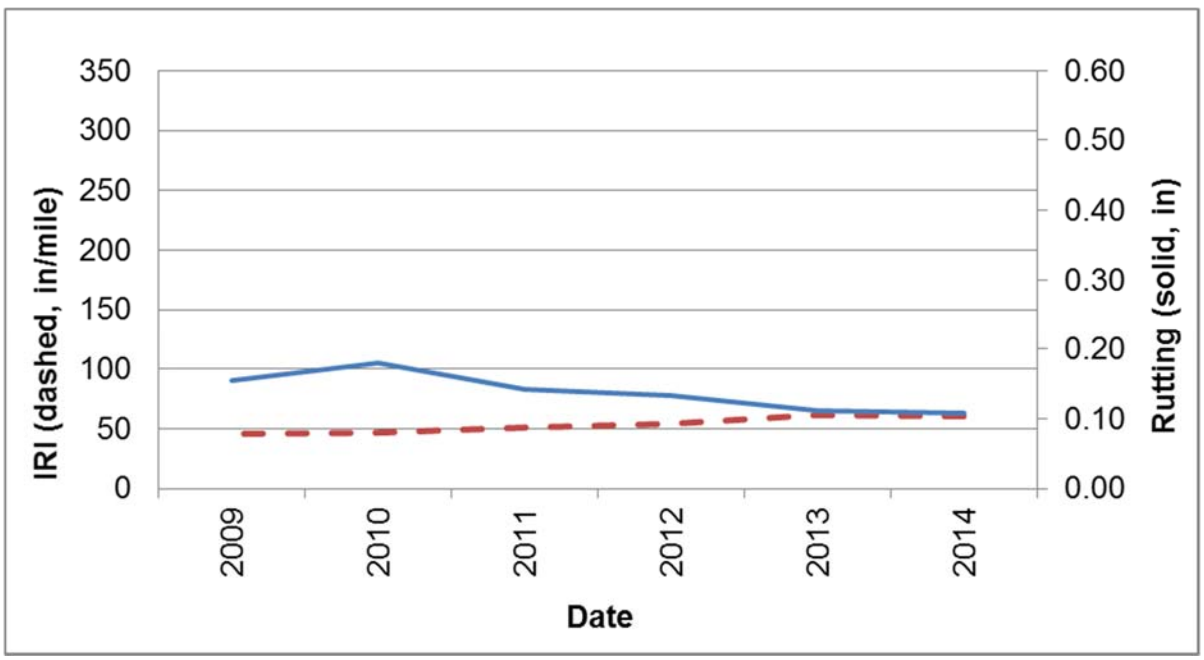

Figure 3.12 – Rutting and Cracking Data (BX0103, Section M2, constructed Mar. 2004 – Aug. 2004) (note, pre-2009 data not available)

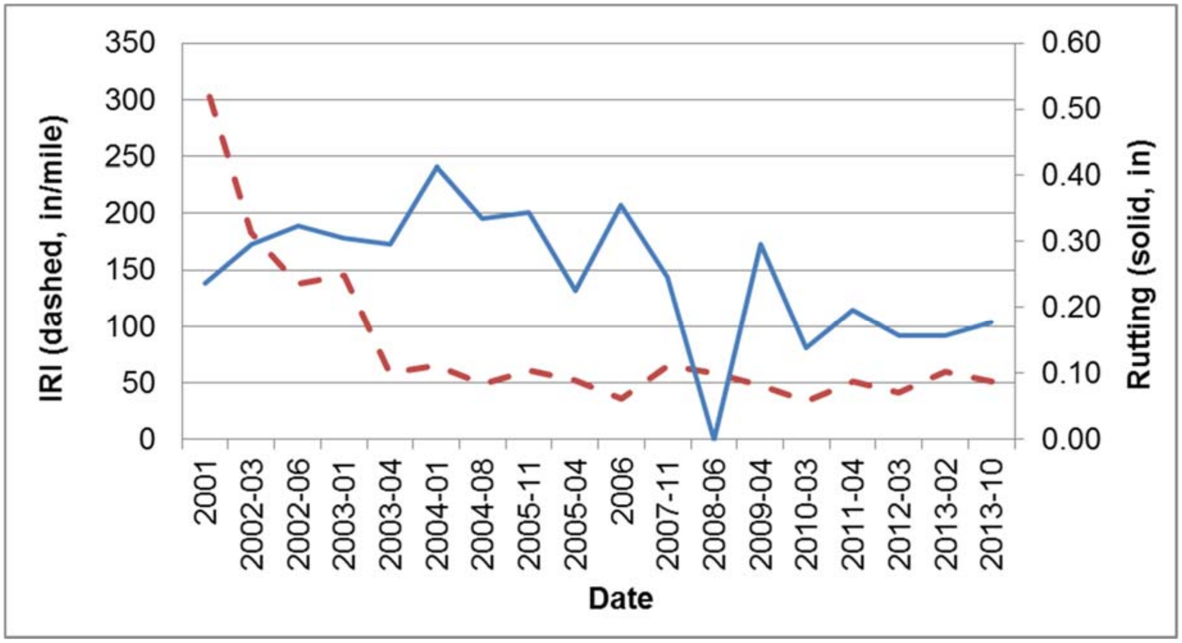

Figure 3.13 – Rutting and Cracking Data (B40102, Section P1, constructed Apr. 2002 – Jan. 2004)

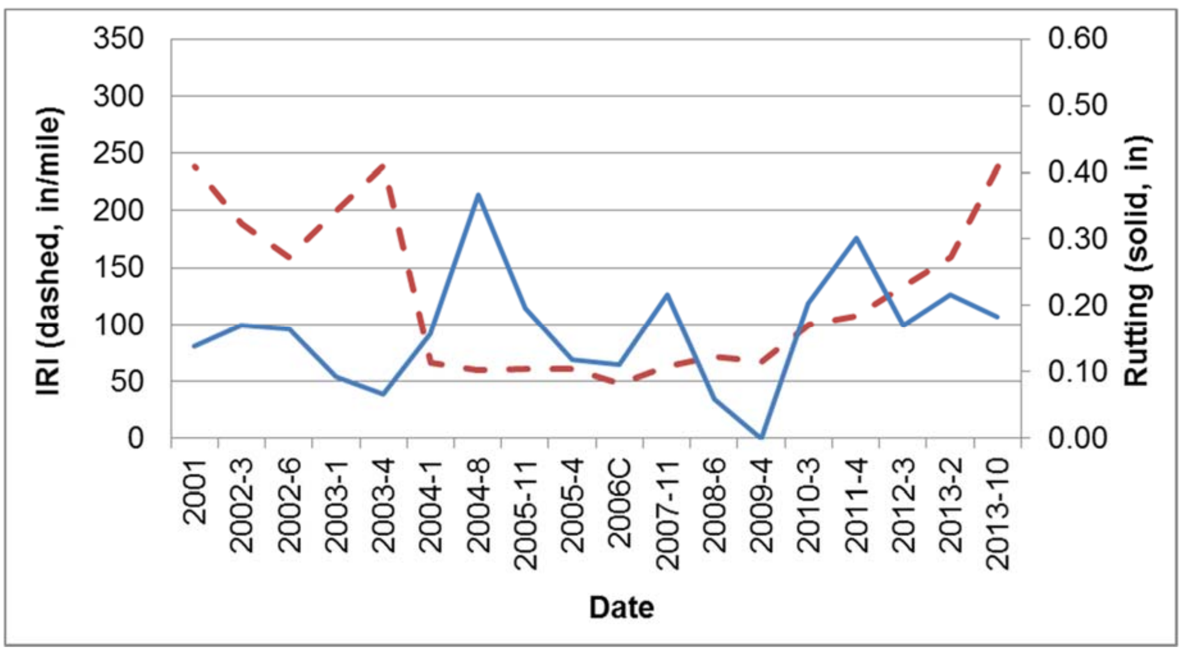

Figure 3.14 – Rutting and Cracking Data (B80108, Section P2, constructed May 2002 – Jul. 2002)

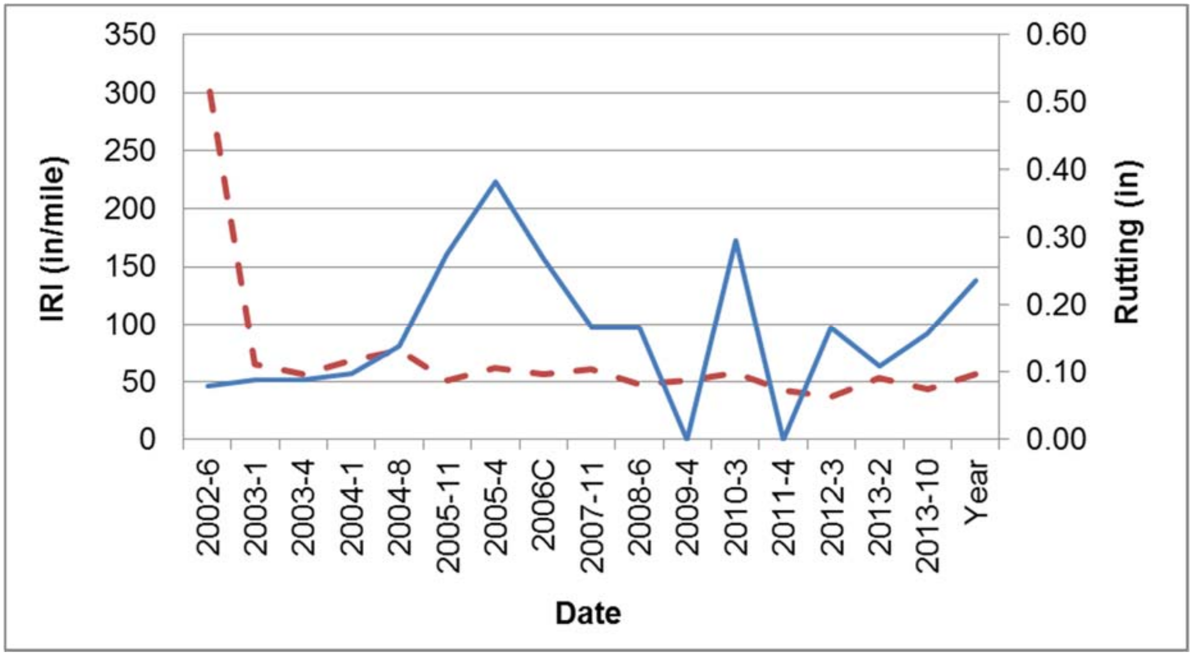

Figure 3.15 – Rutting and Cracking Data (B80105, Section P3, constructed Apr. 2001 – Apr. 2004)

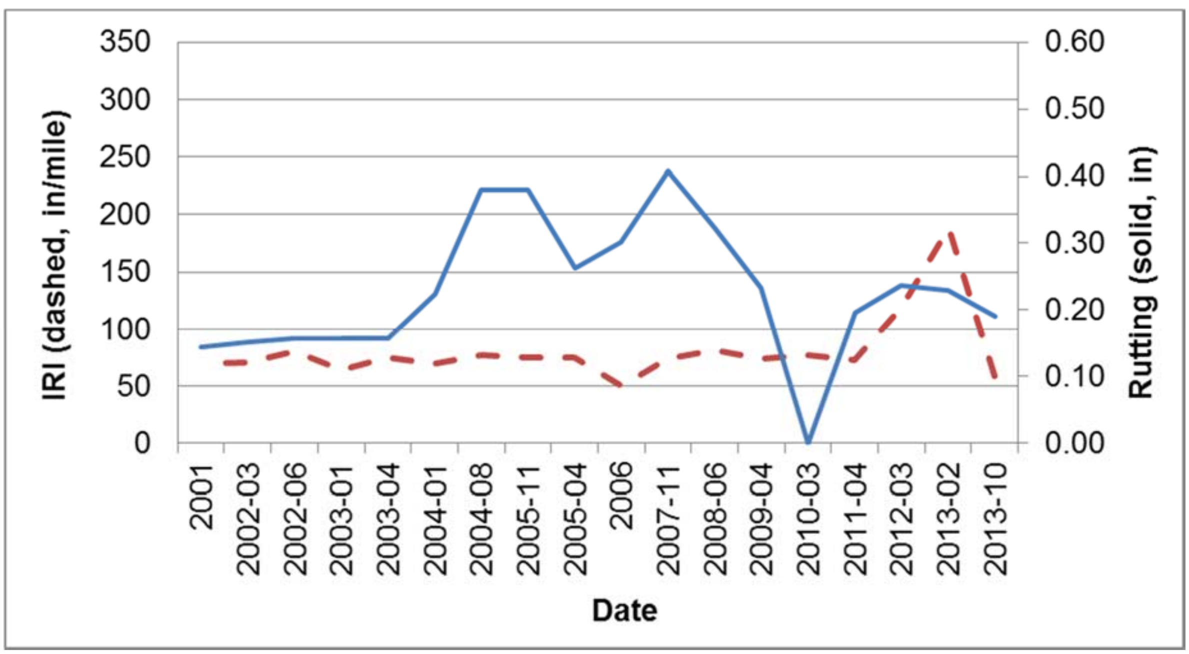

Figure 3.16 – Rutting and Cracking Data (B10103, Section P4, no construction dates available)

In Figures 3.7-16, two trends were apparent. First, the rutting data was low (less than 0.4 in), indicating that these ten mixtures were not susceptible to rutting. Second, in general (except for sections G3 and P4) the IRI for the poor performing sections started high while the good performing sections did not have this initial drop in IRI, indicating that a higher initial roughness could indicate a mixture that is more susceptible to deterioration. However, based on construction dates, this drop could also indicate that the sections were under construction, so some of the IRI data was actually collected on the older pavement sections. Third, there were situations where IRI data collected was zero, which either indicated malfunction equipment or sections were captured with zero roughness. Overall, it appeared that the only potential trend to follow from the ARAN was higher initial IRI (greater than 250 in/mile) immediately after construction could indicate a mixture that would prematurely deteriorate on the roadway. While this could have been a combination of older sections being recorded with the newer sections, the data was not able to segregate pre- versus post- construction, so this hypothesis could not be tested.

3.5 Mix Designs

Records that are kept by AHTD's field or resident engineers were examined to determine the mix design for each section. Since cracking was a significant issue with the material, several characteristics were specifically targeted that are often associated with cracking, including Nominal Maximum Aggregate Size (NMAS), the percentage passing the #200 sieve (P200), the asphalt binder content, the Voids in Mineral Aggregate (VMA), the Voids Filled with Asphalt (VFA), and the fines (or P200) to asphalt binder ratio (F/A ratio). Since different engineers collected the data, some of the data was found as a single point while other data was found as a range. Table 3.2 shows the data collected for the mix design of the six sections that had available data.

Table 3.2 - Details of the Mix Design

Job Number	Report Designation	NMAS (mm)	P200 (%)	Asphalt Content (%)	VMA (%, >14)	VFA (%, 65-75)	F/A Ratio (0.6-1.2)
B10102	G4	12.5	5.0	5.4	14.8	73.0	1.00
B10103	M1	12.5	4.6	5.0	14.7	78.9	1.17
BX0103	M2	12.5	5.0	5.3	15.3	74.2	1.01
B40102	P1	12.5	5.5 – 5.8	5.9 – 6.0	14.3 – 14.6	72.4 – 72.6	1.16 – 1.23
B80108	P2	12.5	5.3	5.8	14.6	71.9	1.14
B80105	P3	12.5	4.0 – 5.3	5.0 – 5.8	14.8 – 15.5	71.9 – 74.2	1.02 – 1.03

In Table 3.2, the NMAS, VMA, and VFA all appeared to be equal and would not affect the performance of the roadway sections. In addition, the VMA and VFA passed Superpave specifications, except for Section

M1's VFA, which was slightly above the acceptable range. For a 12.5mm NMAS, the VMA must be greater than 14%, and for roads with traffic greater than 10 million ESALs, the design VFA must fall between 65-75%. However, the upper range of the P200, or fines, was 10-20% higher for the poor performing sections. Having a higher percentage of P200 often leads to stiffer mixtures, which may be more susceptible to cracking. Even more important, however, it is the F/A ratio, which balances the amount of fines versus the asphalt binder content. When the fines are mixed with the asphalt binder, a mastic is formed, and the ratio of fines to asphalt binder gives an indication of the stiffness of the mixture. The Superpave specification for F/A ratio is 0.6 – 1.2. While all three of the good and medium performing sections fell within this range, one of the poor sections was out of the range, while the other was near the top of the specification. This could increase the stiffness of the mixture and lead to premature cracking. Overall, it appears that the higher percentage of P200, and the higher F/A ratio, could be a cause of the premature cracking on the poor performing sections, but the trends were not consistent across the mixtures.

3.6 Soil Characteristics

The final non-testing characteristic explored was the soil characteristics. Soil underneath the pavement structure forms the foundation of the pavement, and has the potential to heavily influence the performance of the pavement. Table 3.3 summarizes the soil types below the pavement structure.

Table 3.3 – Soil Characteristics of Each Pavement Section (n/a indicates not available)

Job Number	Report Designation	United Soil Classification System / AASHTO Classification	Moisture Content (%)	Resilient Modulus (psi)	R-Value
B60115	G1	A-4(4), A-6(7), A-4(4)	11.7	2400	1.33×10^6
B70102	G2	CH, CL	n/a	n/a	n/a
BX0102	G3	CH, CL / A-7	n/a	n/a	n/a
B10102	G4	CH, CL	23.1	2600	1.44×10^6
B10103	M1	CH, CL	32.8	2600	1.44×10^6
BX0103	M2	A-4	n/a	n/a	n/a
B40102	P1	A-4	n/a	2700	1.50×10^6
B80108	P2	A-4	n/a	n/a	n/a
B80105	P3	A-4	n/a	n/a	n/a
B10103	P4	CH, CL	n/a	n/a	n/a

In Table 3.3, there does not seem to be any direct relationship between the soil classification, moisture content, resilient modulus, or R-value and pavement performance. The majority of soils beneath the pavement structures were either an A-4 soil (a silty soil according to AASHTO) or a CH/CL soil (also a clay soil with low to high plasticity according to USCS). Only three sections had moisture content data available, so no clear conclusions were drawn, and the resilient modulus and R-Values did not appear significantly different between the good, medium and poor sections. Overall, no clear trends were found between soil characteristics and pavement performance.

3.7 Sample Collection

In all, over 270 cores were processed from the ten sections. However, in order to process the cores, they needed to be collected from the field. AHTD did a tremendous job in coordinating with the collection and organization of cores. The research team had the opportunity to assist with the collection of cores by visiting the field sites. Figure 3.17 shows several pictures from coring operations in the field.

Figure 3.17 – Collecting Samples in the Field

Coring operations were consistent in the field, with three sets of nine cores collected. Within each set of nine, six cores were taken from the wheel path and three from between the wheel path. Figure 3.18 diagrams the standard coring pattern, which was repeated two more times for a total of three sections across 0.4 miles and twenty-seven cores from each interstate section.

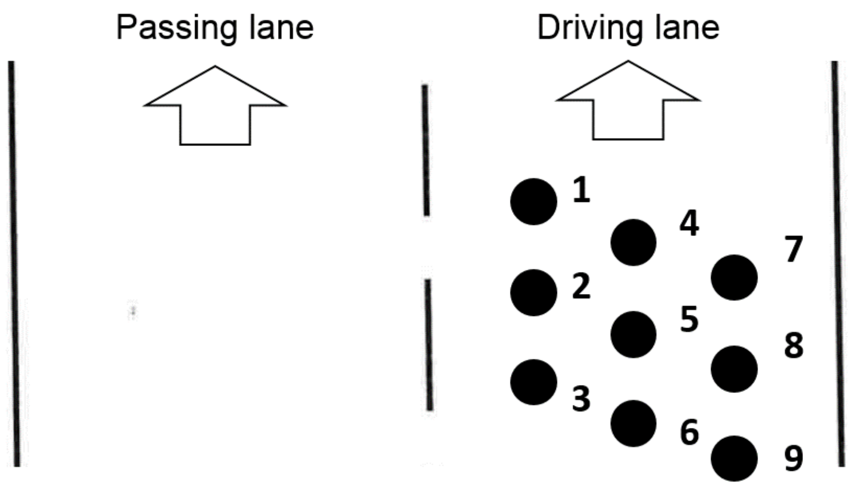

Figure 3.18 – Collecting Pattern for each Interstate Section

Once the samples were collected, they were brought back to the lab at the University of Arkansas for full processing. Each core was labeled and placed into a spreadsheet to determine which laboratory tests would be performed. Because of the wide range of conditions of the cores (Figure 3.19), the research team was not able to consistently test either within wheel-path or between wheel-path samples. This is unfortunate, as behavior may change depending on whether the material was under traffic or between wheels. However, this was another function of working with field sites and field samples.

Figure 3.19 – Various Conditions of Cores on Arrival at Lab

4.1 Asphalt Mixture Testing

The asphalt mixtures tests evaluated in this study included volumetric properties, bond strength tests, IDT dynamic modulus, torsion bar dynamic modulus, SC(B) fracture tests, SC(B) stripping ratio, and ignition oven binder content.

4.2 Volumetric Tests

The goal of volumetric tests of the cores in this research was to determine the air voids of each sample. The specific gravity of core were performed based on AASHTO T166, but the theoretical maximum specific gravity (G_{mm}) cannot be captured by a normal AASTHO T209 because the cores are compacted and the cores have saw-cut surfaces. However, several papers have discussed how to circumvent these two issues. Louisiana allows heating of the sample to 160±5°C until proper workability is obtained if the sample is not soft enough to be separated with spatula or trowel. Hall *et al.* (2000) reheated the field sample to equi-viscous compaction temperature for sample splitting. Based on these two reports, this research preheated the cores to 150°C to break the sample into a loose, G_{mm} form. In order to calculate the maximum specific gravity, a method developed by the Asphalt Institute was utilized (Blow, 2014). This method considers the impact of crushed, milled, or saw cut new surface. In short, this method adds 1% of new asphalt to coat all the saw cut surface, and removes the effects of new asphalt on the G_{mm} by calculation shown in Equation 1:

$$\text{Theoretical Maximum Specific Gravity} = A–J / (A+D) – (E+K) \tag{1}$$

where:

A = mass of the oven-dry sample in air, g;

D = mass of the container filled with water at 25°C (77°F), g;

E = mass of the container filled with the sample and water at 25°C (77°F), g;

J = mass of the added asphalt binder in air, g;

K = volume of the added asphalt binder.

The air voids for all ten sections, for both the top surface course (S2) and bottom surface course (S1) are shown in Table 4.1

Table 4.1 – Air Voids of Bottom Surface Course (S1) and Top Surface Course (S2)

Job Number	Report Designation	S1 G_{mm}	S1 Air Voids (%)	S2 G_{mm}	S2 Air Voids (%)
B60115	G1	2.411	4.5	2.391	6.2
B70102	G2	2.398	6.3	2.411	7.8
BX0102	G3	2.438	6.1	2.442	5.8
B10102	G4	2.404	6.4	2.417	6.4
B10103	M1	2.401	5.5	2.392	4.8
BX0103	M2	2.392	6.8	2.401	7.5
B40102	P1	2.361	9.2	2.372	11.1
B80108	P2	2.401	10	2.395	8.8
B80105	P3	2.442	7.2	2.431	9.5
B10103	P4	2.442	7.4	2.457	7.7

Overall, there did not appear to be any trend in maximum specific gravity. However, it appeared that the good and medium performing sections had lower air voids than the poor performing sections. The good and medium performing sections both had an average air voids of 6.2%, while the poor performing section had an average of 8.9%. In addition, the bottom surface course air voids averaged 6.9% and the top surface course averaged 7.6%.

4.3 Bond Strength Test

The bond strength test was run at two temperatures and two pressures, on the interface between both S2 and S1 (the two surface courses) and S1 and B2 (the bottom surface course and the top binder course). Figures 4.1a and 4.1b show the bond strength values. In these figures, it was apparent that higher test temperatures and lower confining pressures produced lower bond strength values. In addition, there did not seem to be a significant difference between the bond strength values at the two different pavement depths. A disadvantage of this test, however, is it can only be run on intact cores. Many of the poor sections arrived in the lab already debonded, as the layers were either debonded before coring or the coring process sheared the layers apart in the field during sampling. Therefore, Figure 4.2 shows the percentage of debonded cores upon arrival. A significantly higher number of the poor cores were debonded upon arrival, as were a decent number of medium sections, indicating that the good sections were strong in the field. In addition, more samples were debonded between the two surface layers versus the bottom surface and binder course.

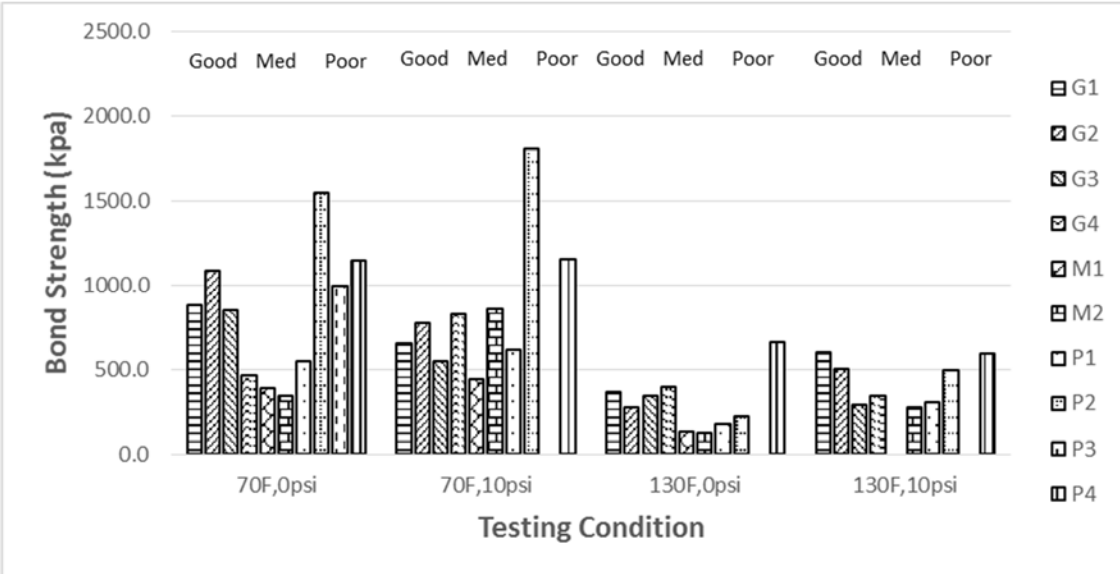

Figure 4.1a – Bond Strength Between S2 and S1

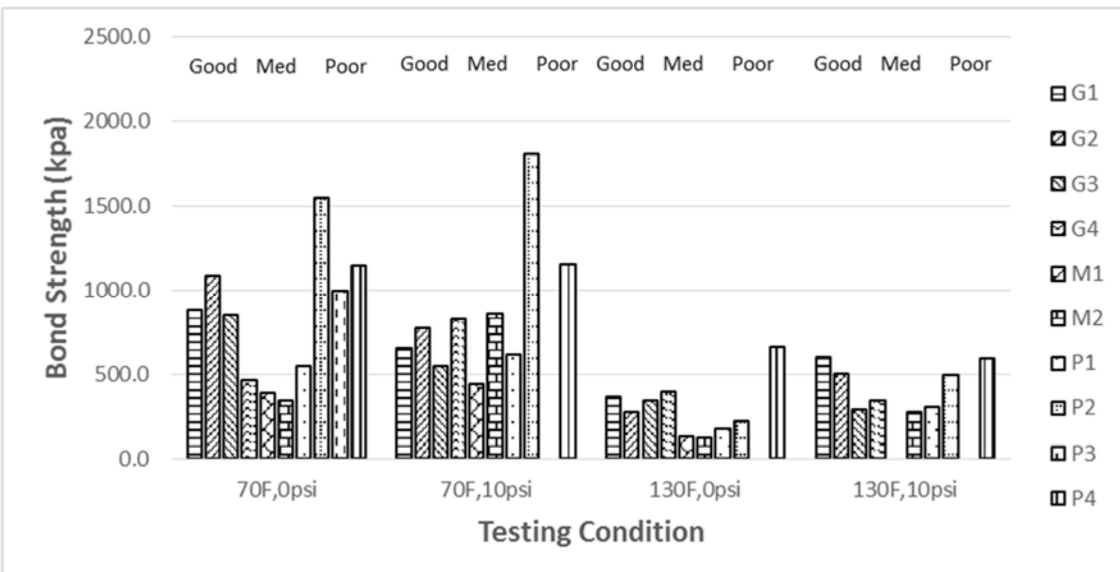

Figure 4.1b – Bond Strength Between S1 and B2

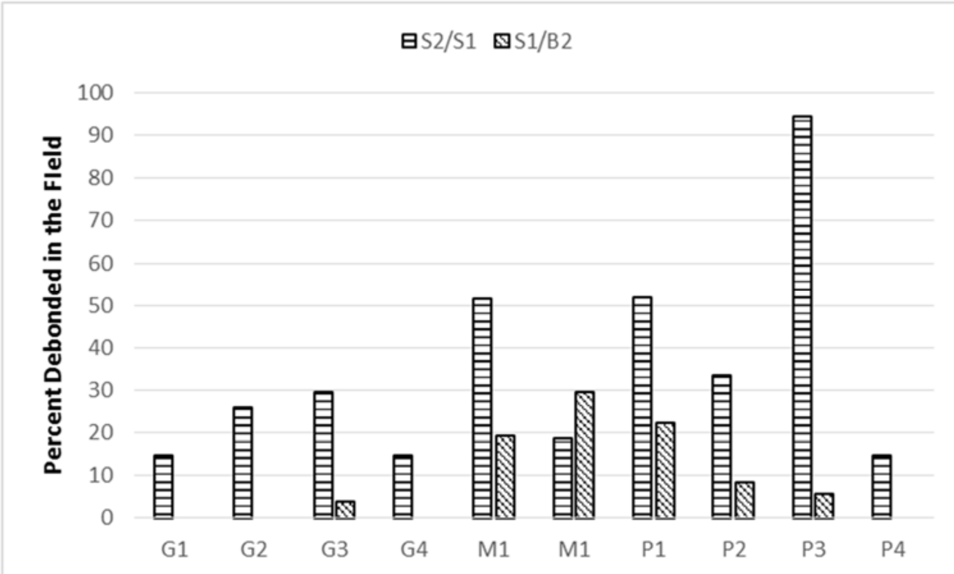

Figure 4.2 – Percentage of Samples Debonded on Arrival to Lab

4.4 IDT Dynamic Modulus

In general, the good sections had the highest IDT dynamic modulus, while the poor sections had lowest modulus, with medium sections also showing poor stiffness. However, there were several exceptions. For example, the poor section P4's modulus at low temperature or high frequencies was in-between a good section and medium section, for both surface courses. Also, the good section G4 had a lower modulus than the medium sections but higher than poor sections in top surface course S2. Although there were some exceptions, the dynamic modulus was able to predict higher values for the better sections (a stiffer, more cohesive material) and lower values for the poorer sections (a softer, less cohesive material). Figure 4.3a shows the dynamic modulus curves for the S2 (top surface) and Figure 4.3b shows the dynamic modulus curves for the S1 (bottom surface).

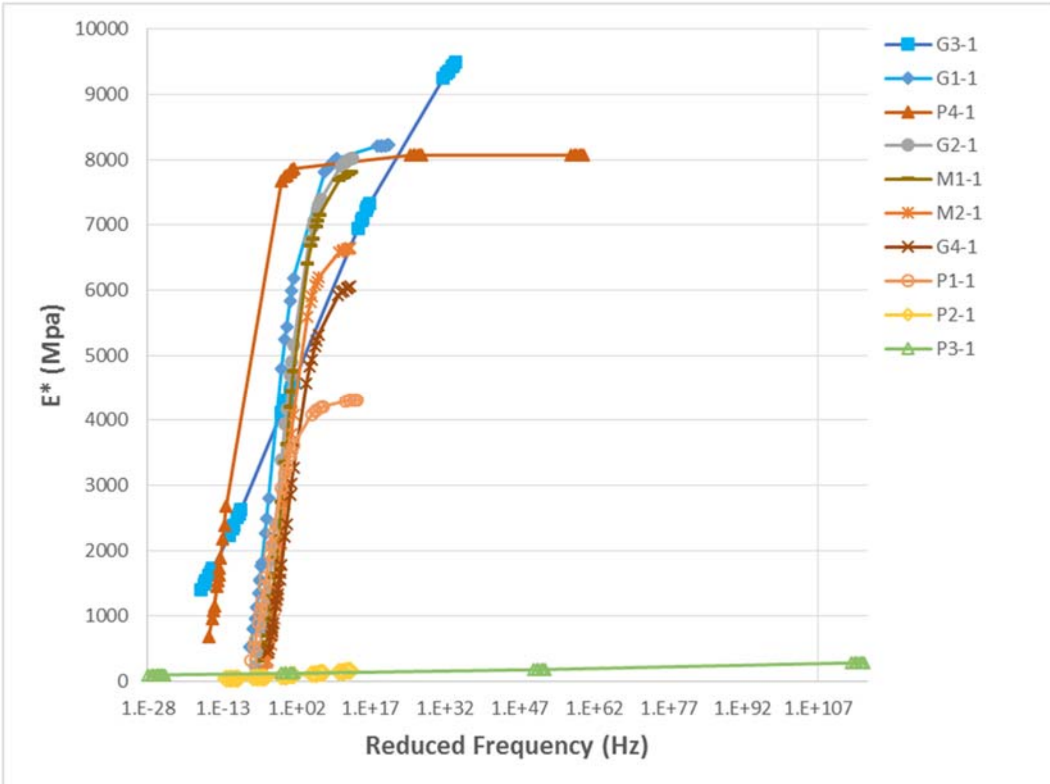

Figure 4.3a – IDT Dynamic Modulus Master Curve for S2 (top surface)

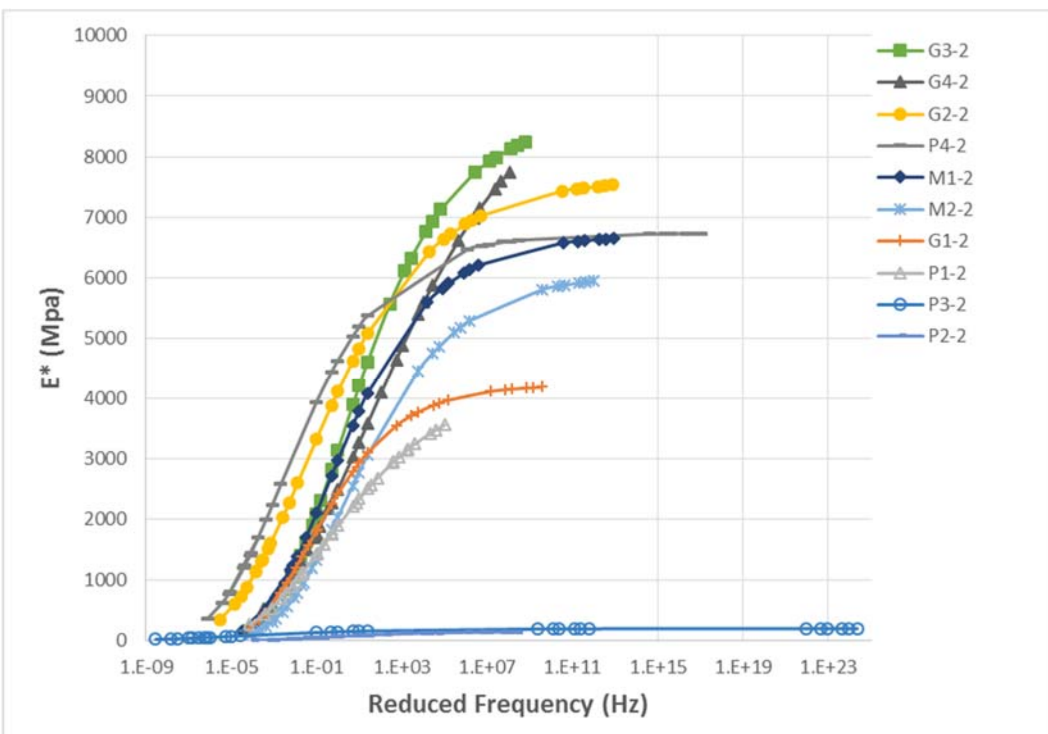

Figure 4.3b – IDT Dynamic Modulus Master Curve for S1 (bottom surface)

4.5 Torsion Bar Dynamic Modulus

In addition to the IDT dynamic modulus, torsion bar dynamic modulus tests were run. The advantage to the torsion bar dynamic modulus is that up to 15 torsion bar samples can be extracted from the typical size of an IDT dynamic modulus sample. Eight testing temperatures (-10, 0, 10, 20, 30, 40, 50, and 60°C) were used, with frequencies from 100rad/s to 0.03 rad/s at each temperature. The geometry of the sample was 50×12.5×6.5mm. The trends for the torsion bar were similar to the IDT dynamic modulus, with the majority of good samples having higher stiffness than the poor sections, but like the IDT configurations, there were some exceptions. From this data, it appears that the torsion bar dynamic modulus can be used as a substitute for the IDT dynamic modulus, creating significant savings in field extraction of samples. Figure 4.4a shows the dynamic modulus curves for the S2 (top surface) and Figure 4.4b shows the dynamic modulus curves for the S1 (bottom surface).

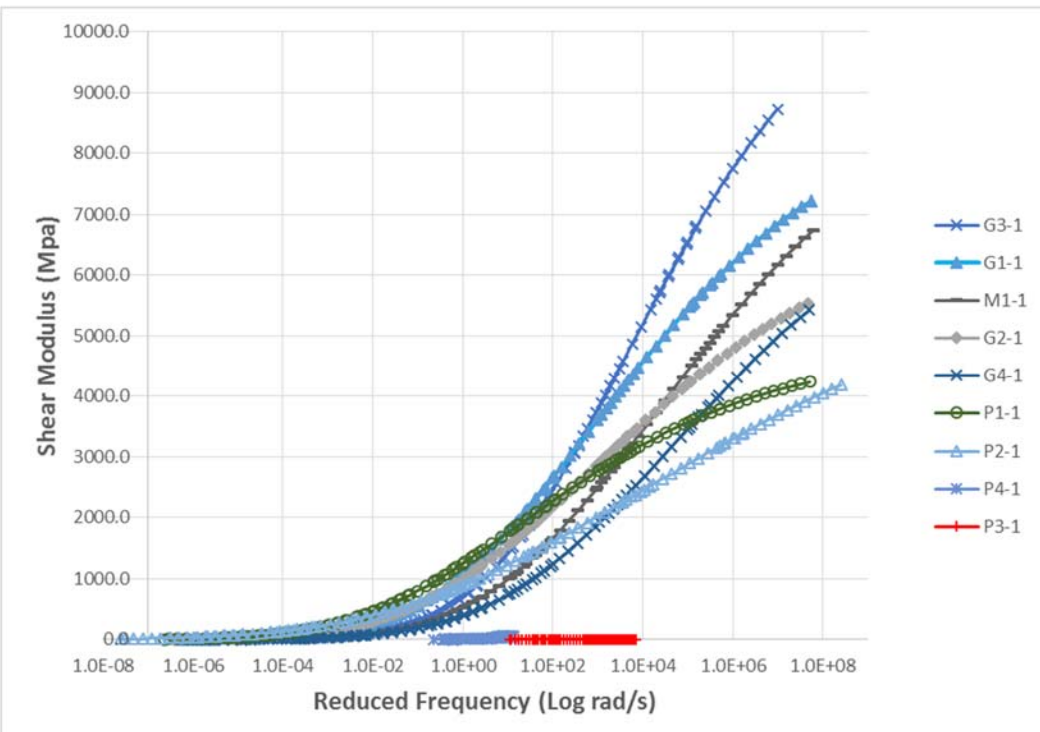

Figure 4.4a – Torsion Bar Dynamic Modulus Master Curve for S2 (top surface)

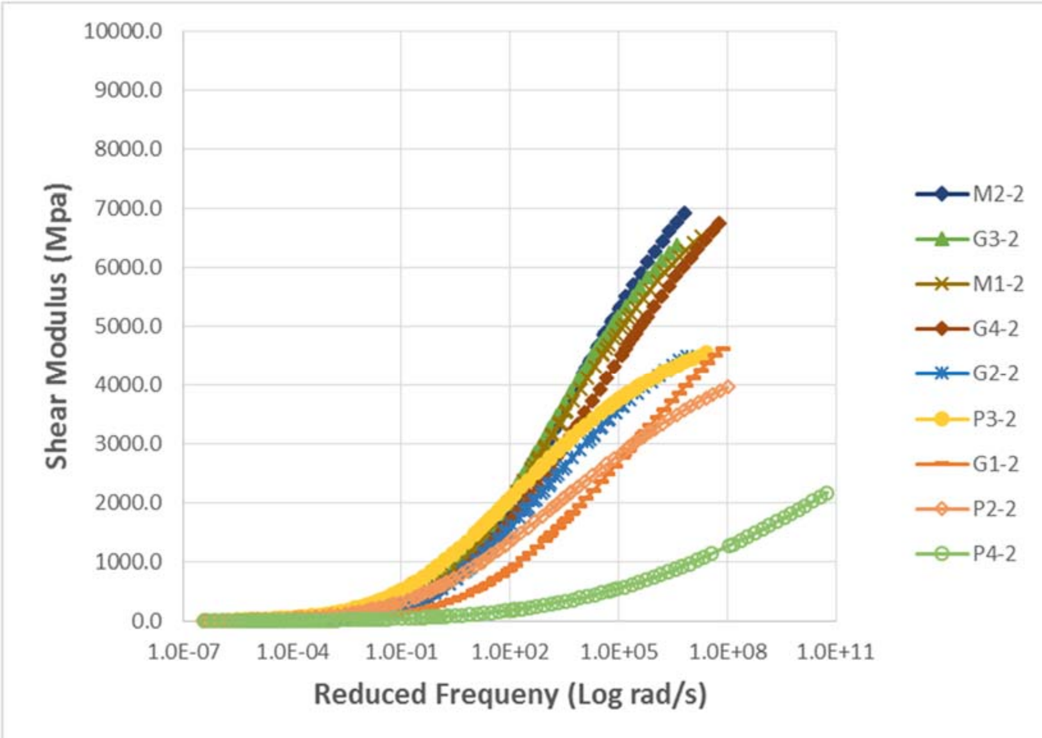

Figure 4.4b – Torsion Dynamic Modulus Master Curve for S1 (bottom surface)

4.6 Semi-Circular Bend [SC(B)] Fracture and Stripping Ratio

Fracture energy can be an indication of how easily cracks can form. The SC(B) fracture test clearly showed that the good sections had higher fracture energy versus the poor sections. A zero fracture energy indicates that the peak load did not reach the minimum requirement (0.5 kN) before the crack propagated through the entire sample. Figure 4.5 shows the fracture energy for each section and pavement layer. In addition to the calculating the fracture energy, a stripping ratio was calculated based on the face created by the crack. The stripping ratio was based on a one to five number, with one exhibiting no stripping and five exhibiting significant stripping. The stripping ratio in AASHTO T273 was used as a guide. Figure 4.6 shows the stripping ratios for each section and layer, and indicates that there was no clear correlation between good performance and the stripping of the asphalt binder off the aggregate.

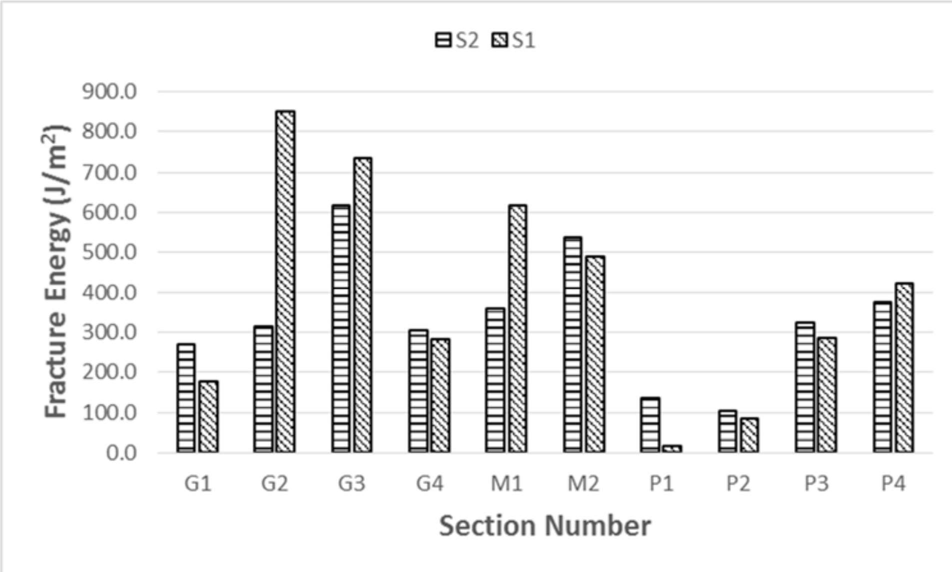

Figure 4.5 – Fracture Energy for Surface S2 and S1

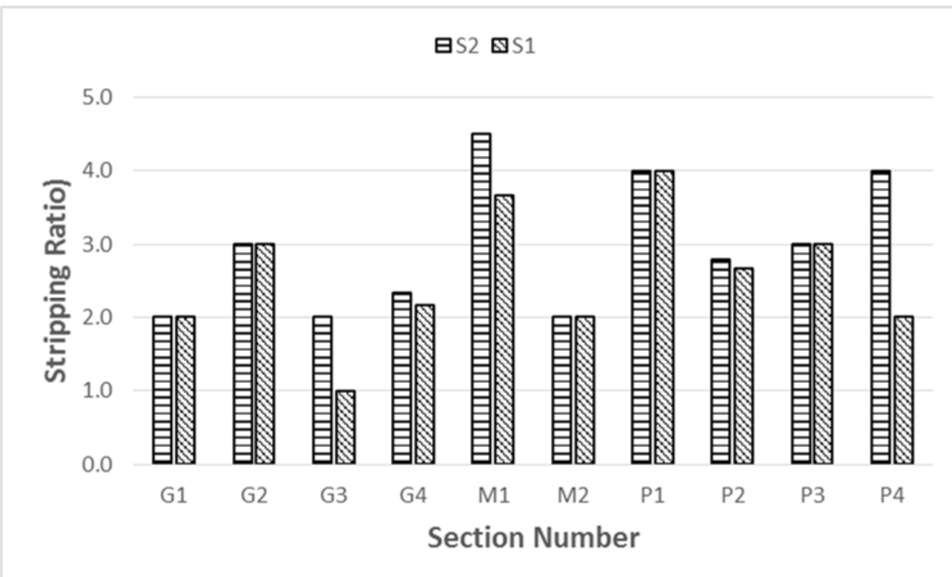

Figure 4.6 – Stripping Ratio for Surface S2 and S1

4.7 Ignition Oven Binder Content

The ignition oven tests, run according to AASHTO T308, did not show any clear trends between binder content and pavement performance, as seen in Figure 4.7.

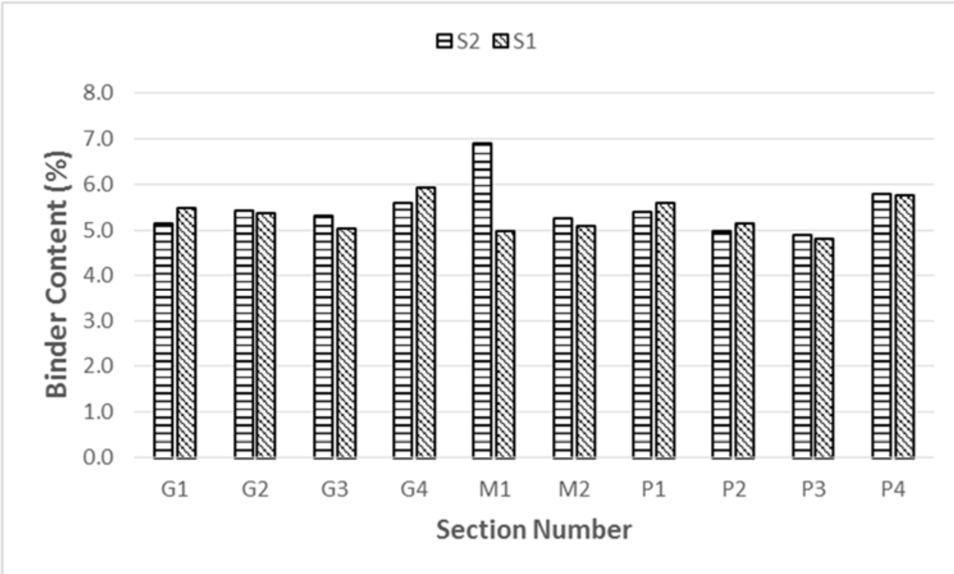

Figure 4.7 – Ignition Oven Binder Content for S2 and S1

5.1 Asphalt Binder Testing

The asphalt binder was extracted and recovered from samples that were collected from good, medium and poor performing sections. In general, the asphalt mixture samples were tested at the University of Arkansas Fayetteville, then transferred to Arkansas State University for asphalt binder testing. First, the binder needed to be separated from the aggregate. To do this, extraction and recovery was done by AASHTO T 64 and ASTM D5404 methods using a centrifuge and Rotavapor. Table 5.1 shows the binder content for each section as an average of all samples tested.

Table 5.1 – Asphalt Binder Content from Rotavapor Extraction of Bottom Surface Course (S1) and Top Surface Course (S2)

Job Number	Report Designation	S1 Binder Content (%)	S2 Binder Content (%)
B60115	G1	6.37	6.18
B70102	G2	6.30	6.40
BX0102	G3	6.10	5.70
B10102	G4	5.40	5.50
B10103	M1	5.10	5.20
BX0103	M2	5.20	5.00
B40102	P1	5.61	5.45
B80108	P2	5.61	5.27
B80105	P3	5.49	5.13
B10103	P4	5.70	6.90

Unlike the ignition oven, there did appear to be slightly higher asphalt binder contents in the majority of the good performing sections. Addition asphalt could reduce cracking, but too much asphalt may induce rutting. Since there was no significant rutting overserved, the additional asphalt binder in the good performing sections may have allowed for better long-term performance. With the binder extracted, the penetration test, Dynamic Shear Rheometer (DSR), and Rotational Viscosity (RV) tests were conducted according to AASHTO T 49, AASHTO T 315 and AASHTO T 316 methods, respectively.

5.2 Penetration Test

The penetration test is a historical test that gives an indication of the stiffness of an asphalt binder. In theory, harder (i.e. stiffer or less viscous) asphalt binder has a lower penetration number while softer asphalt binder has a higher penetration number. Table 5.2 summarizes the ten sections.

Table 5.2 – Penetration Values of Bottom Surface Course (S1) and Top Surface Course (S2)

Job Number	Report Designation	S1 Penetration (0.1mm)	S2 Penetration (0.1mm)
B60115	G1	5.0	5.0
B70102	G2	6.3	6.4
BX0102	G3	11.0	7.0
B10102	G4	5.4	5.5
B10103	M1	16.3	7.3
BX0103	M2	21.0	8.0
B40102	P1	6.0	5.0
B80108	P2	11.0	7.0
B80105	P3	6.9	4.1
B10103	P4	15.5	10.3

Overall, it was expected that the poorer performing sections would have stiffer, less viscous binders. This would be reflected with lower penetration numbers. However, Table 5.2 shows that the poor performing sections did not necessarily have lower penetration numbers, and in fact, several sections had higher penetration numbers than the good and medium performing sections. Therefore, it did not appear that the penetration test did an accurate job of ranking the performance of mixtures. However, the penetration number was able to tell the difference between the bottom surface course (S1) and the top surface course (S2) on the majority of mixtures, by showing lower penetration numbers on the S2 layers, indicating the ability to identify higher levels of oxidation and other weather related deterioration of the asphalt binder.

5.3 Dynamic Shear Rheometer (DSR) Test

The DSR test is the current Superpave test method to quantify the rutting (high temperature) and fatigue cracking (intermediate temperature) cracking. Since it appeared that many of the poor performing pavement sections were deteriorating due to a form of cracking, the intermediate temperatures were investigated in this study. The asphalt binder was tested at three temperatures: 25, 28, and 31°C. For simplicity sake, the average values for the good, medium, and poor sections are presented in Table 5.3.

Table 5.3 – Average G* × sin δ Values of Bottom Surface Course (S1) and Top Surface Course (S2)

	Good Performing		**Medium Performing**		**Poor Performing**	
	S1	**S2**	**S1**	**S2**	**S1**	**S2**
G* × sin δ at 25°C (kPa)	10,848	14,168	4,741	18,321	11,885	18,638
G* × sin δ at 25°C (kPa)	7,903	10,345	3,346	13,263	9,085	14,220
G* × sin δ at 25°C (kPa)	6,035	8,134	2,277	9,842	6,783	11,270

In general, higher G* × sin δ values indicate a stiffer, more fatigue cracking prone asphalt binder. Here, the poor performing sections have the highest G* × sin δ values, while the good performing sections have the lowest. This would indicate that the poor performing sections are more susceptible to fatigue cracking. In addition, the S2 layers all had higher G* × sin δ values than the S1 layers, showing that the DSR was able to identify the material exposed to more oxidation and weathering.

5.4 Rotational Viscometer Test

Rotational viscosity is used to measure the viscosity of asphalt binder at high temperatures. Usually the workability of binder during pumping and mixing is determined by rotational viscosity test. According to AASHTO specification rotational viscosity of neat binder should be less than 3.0 Pa.s at 135°C. In this study, the rotational viscosity test of recovered asphalt binder from one good, one medium, and one poor section was performed at four temperatures, starting from 135°C to 180°C at 15°C intervals. Rotational viscosity test result for S1 and S2 layer of good, medium, and poor sections are shown in Figure 5.1 and Figure 5.2.

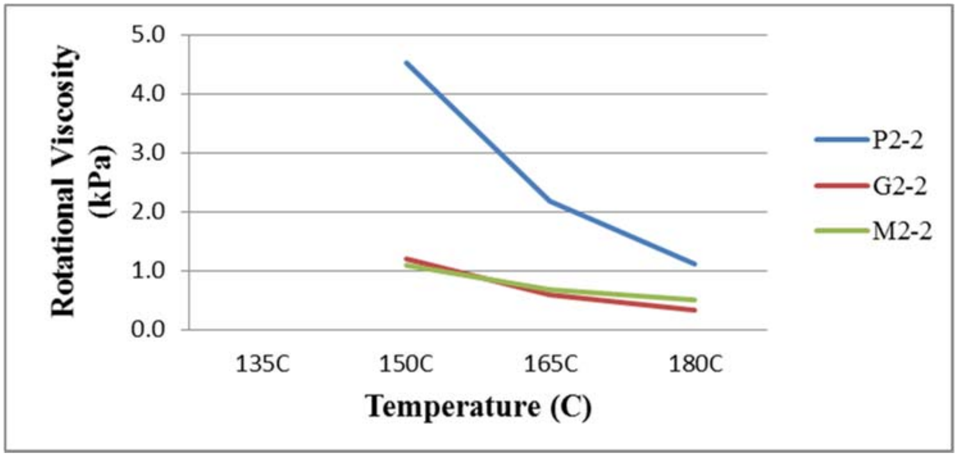

Figure 5.1 – Average Rotational Viscosity of Bottom Surface Course (S1)

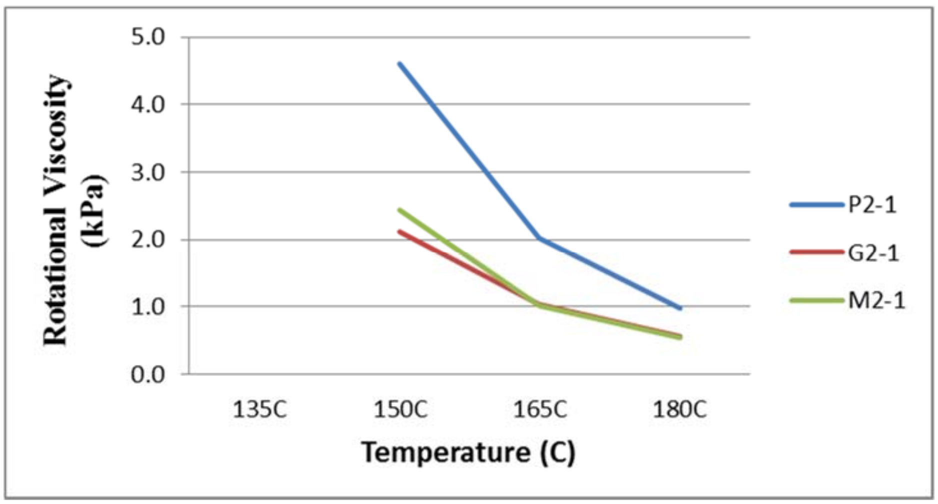

Figure 5.2 – Average Rotational Viscosity of Top Surface Course (S2)

For both the S1 and S2 layers, it was apparent the poor performing section had a higher viscosity, indicating a stiffer mix compared to the good and medium section. In addition, the top surface course was stiffer than the bottom surface course, indicating the rotational viscosity test was able to identify the pavement layers that were exposed to oxidation and weather.

6.1 Conclusions

After less than fifteen years of in-place service, several of the pavement surfaces on Arkansas' Interstate system were prematurely deteriorating. While the literature review showed that Arkansas is not alone with prematurely failing pavements, this research looked to determine the cause of premature deterioration. Therefore, job diaries and other paper documents were examined, and asphalt mixture and asphalt binder testing was performed on over 270 cores from four "good" performing sections, two "medium" performing sections, and four "poor" performing sections. Based on extensive document review and laboratory testing, the following conclusions were made:

- There appeared to be no relationship between either the water added to the tack coat or the tack coat application and pavement performance.
- There was no clear relationship between relative humidity, maximum temperature, minimum temperature, or precipitation and pavement performance.
- The only potential trend to follow from in-service rutting and cracking data was higher initial IRI (greater than 250 in/mile) could indicate a mixture that would prematurely deteriorate on the roadway. However, this could also be a function of pre-construction data being combined with post-construction data, but the information was not able to segregate these data sets.
- A higher percentage of P200 or F/A ratio could be a cause of the premature cracking on the poor performing sections, but the trends were not consistent across the mixtures.
- There was not any direct relationship between the soil classification, moisture content, resilient modulus, or R-value and pavement performance.
- There did not appear to be any trend in maximum specific gravity, but it appeared that the good and medium performing sections had lower air voids than the poor performing sections.
- While there did not seem to be a significant difference between bond strength values between layers of the pavement sections, a significantly higher number of layers were either debonded before coring or the coring process sheared the layers apart in the field during sampling on the poor sections.
- Both IDT and torsion bar dynamic modulus were generally able to predict higher values for the better sections (a stiffer, more cohesive material) and lower values for the poorer sections (a softer, less cohesive material).

- The SC(B) fracture test clearly showed that the good sections had higher fracture energy versus the poor sections, but there was no clear correlation between good performance and the stripping of the asphalt binder off the aggregate.
- The ignition oven tests did not show any clear trends between asphalt binder content and pavement performance, but the Rotavapor or recovery method showed a slightly higher asphalt binder contents in the majority of the good performing sections.
- The penetration test did not do an accurate job of ranking the performance of asphalt binders or asphalt layers.
- The Dynamic Shear Rheometer at intermediate temperatures and the Rotational Viscometer test did an acceptable job of identifying the performance of asphalt binders and asphalt layers.

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TRC1404A

Evaluating Performance of Asphalt Pavement Based on Data Collected During IRP - addendum to final report

Andrew Braham, Tim Aschenbrener, Zahid Hossain

Final Report

2016

Technical Report Documentation Page

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9. Performing Organization Name and Address University of Arkansas 4190 Bell Engineering Center Fayetteville, AR 72701		**10. Work Unit No. (TRAIS)** **11. Contract or Grant No.**
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16. Abstract

After only 9-12 years of service, multiple sections of various interstates within Arkansas were prematurely deteriorating. Four of these poor performing sections were compared to two medium (or fair) performing sections and four good performing sections in order to evaluate potential causes of the premature deterioration. Cylindrical cores were taken in order to determine why some sections were performing as designed while the others were not. Several key properties, including structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage, were examined. The last three of these properties were identified as impacting performance of the ten interstate sections. After evaluation, no section was found to be good, medium, or poor performing in all three properties. However, a composite rating of key properties was developed that provided strong correlations between the results from the forensic evaluation and actual field performance. Existing tests such as the bond strength test and the in-place air voids test were complemented by new metrics developed specifically from the data collected during this study. The four new metrics were debonded lifts, the bond strength factor, the core degradation, and the stripping rating. By averaging the good, medium, and poor performance in each of these six tests and metrics, an overall quantitative rating was established. In addition to the development of new metrics to quantify the importance of the relationship of these key properties to field performance, it was recommended to the Arkansas State Highway and Transportation Department (AHTD) that the moisture performance test and the mix design process be reevaluated in order to ensure long-lasting pavements that perform as designed.

17. Key Words		18. Distribution Statement	
Forensic evaluation, design mixture properties, bond strength, air voids, moisture damage		No restrictions. This document is available to the public through NTIS: National Technical Information Service 5285 Port Royal Road Springfield, Virginia 22161	
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Background

In 1999, the Arkansas State Highway and Transportation Department (AHTD) began a program to rehabilitate over 300 miles of Interstate in 5 years. This program was called the Interstate Rehabilitation Program, or IRP. As part of the IRP, approximately 270 miles of deteriorated concrete pavement was rubblized and overlaid. Less than ten years after construction, several of the pavements constructed exhibited a severe level of surface distresses for pavements with a 20-year design life. Observed distresses included top-down cracking, bottom-up cracking, and pop-outs, all of which increased the International Roughness Index (IRI). Most of these severely distressed asphalt pavements are located west of Conway on Interstate 40; while the pavements east of Little Rock on I-40 and west of Little Rock on I-30, which were constructed at virtually the same time, exhibit much less or no cracking.

In order to quantify the performance of the IRP, ten test sections were chosen. Of these ten sections, four (G1 through G4) were considered “good” performing, two (M1 and M2) sections were considered “medium” (or fair) performing, and four (P1 through P4) sections were considered “poor” performing. These qualifications were provided by AHTD’s research engineers based on AHTD’s pavement rating system. The section IDs (e.g., B40102 for P1), geographical locations, and conditions of the ten sections are shown in Figure 1.

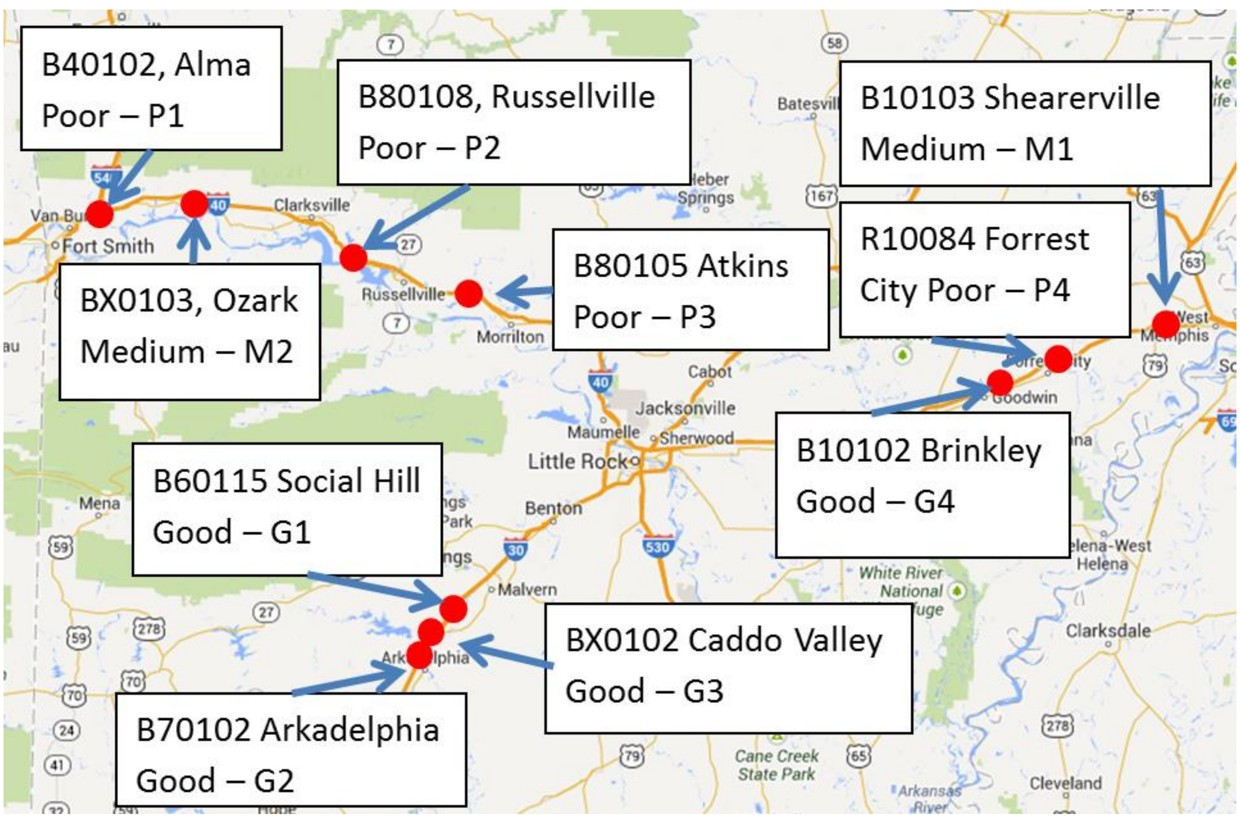

Figure 1 – Location of evaluated test sections

Initial research found that higher initial IRI values may predict poor performing sections, while a higher dust to total asphalt binder (F/A) ratio could cause premature cracking (Braham et al., 2015). Further, the dynamic modulus from the indirect tension test (IDT) and torsion bar geometries provided decent correlations to field performance (Yang et al., 2016). By utilizing the field core testing regime developed by Wagoner et al. (2008), tensile fracture tests were run after dynamic modulus testing. This not only allowed for the collection of fracture data, but also provided samples for an investigation of stripping rating on the newly exposed fracture faces. However, test results did not identify any fundamental issues in pavement design, material design, construction, or service life that could have caused the premature deterioration. Therefore, the laboratory test and field performance data was reanalyzed. After reanalyzing the data, five potential causes for premature deterioration were identified as areas of focus: structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage.

Previous Studies of Premature Deterioration

Previous research was reviewed with an emphasis in structure capacity, design mixture properties, bond strength, in-place air voids, and moisture damage, to provide guidance for this additional research. The National Cooperative Highway Research Program (NCHRP) Study (Report No. 747) conducted by Rada et al. (2001), suggested that visual inspections of roadways may often yield good results but they are unable to detect any structural deficiencies. For a thorough forensic investigation, the NCHRP study suggested incorporating three phases: preliminary investigation, non-destructive testing (NDT), and destructive and/or laboratory testing. It is stated that if sufficient data and information, gathered during the preliminary phase, lead to a conclusion no further investigation is required. An NDT plan can include tests such as falling weight deflectometer (FWD), ground penetrating radar (GPR), profilometer, skid friction tester, and noise measurements. If the NDT results provide a well-supported explanation of the issue being investigated, no further testing is required. Whereas, if the results are inconclusive, further testing requirements need to be determined. The recommended destructive testing requirements for AC pavemennts from NCHRP report 747 were:

- Routine and/or specialized laboratory tests encompassing physical properties
  - For example: gradations, asphalt content, air voids, and specific gravity
- Mechanical properties from Superpave performance tests
  - For example: Superpave binder testing, Superpave IDT, etc.
- Chemical analyses
  - For example: microscope analyses, SARA analysis, etc.
- CT scans.

For this research, AHTD felt that the preliminary investigation and NDT stages of the study had not provided adequate information or data to identify the cause for the premature deterioration of the highway sections in question; therefore, a laboratory testing analysis was pursued.

Another forensic analysis of interest, performed by Anderson et al. (2001), investigated early transverse (reflective) and longitudinal (surface-initiated) cracking on I-25 in Denver, CO. In this study, cylindrical cores and slab samples from two selected highway sections (the prematurely distressed section on I-25 and the other one without distress on I-70) of the same age were collected and tested in the laboratory to compare their mechanical and physical properties. These researchers focused on three major pavement deterioration factors: percent air voids, effective asphalt binder volume, and physical properties of asphalt binders. Binder test results revealed that the characteristics of asphalt binder from I-25 and I-70 were similar prompting the researchers to conclude that the binder was not the cause of the premature cracking. Furthermore, these researchers found that the recovered asphalt binder content of the I-25 mixture was 4.1% and considering the absorption of asphalt binder by the aggregate this percentage falls down to a 3.7% whereas, the design asphalts binder content was 4.6%. On the other hand the recovered asphalt binder content of the I-70 mixture was 4.8%, which was 0.7% higher than that of the I-25 mixture. These asphalt binder levels indicated that perhaps enough asphalt was not supplied in the mixture. Further, the percentage of air voids found in the I-25 mixture ranged from 7.2% to 8.3%; with an initial air void content after construction of 6.4%. This range of air voids was determined to be relatively high and the initial air void content failed to meet the quality control-quality assurance requirements. Several recommendations were made to minimize the occurrence of similar performance issues in the future including the evaluation of the following: alternate surface mixtures, methods used to determine in-place density and asphalt content, and methods for determining the mechanical properties of the mixtures.

A forensic study involving cold in- place recycled (CIR) asphalt on U.S. Highway 34 near Union County in Iowa showed localized areas of severe loss of stability under traffic over a 2-mile section of the roadway. The loss of stability along with structural strength resulted in non-uniform deep wheel path rutting and shoving (Heitzman, 2007). This study considered eleven parameters as possible root causes for the excessive rutting. However seven of these factors; namely, moisture, change in CIR age and RAP size, asphalt stabilizer content, steep grades, and construction staging were eliminated after careful consideration. The researcher observed an apparent correlation between the distress and high CIR compaction (i.e., low air voids). The high CIR density combined with high volume truck traffic and high temperatures created a very low air void condition and an unstable CIR layer. Based on the findings of this study, the Iowa DOT issued a restriction on the use of CIR on roadways with high volume truck traffic.

All three studies considered all or a portion of the five potential causes of premature deterioration identified for the early deterioration of sections of interstates in Arkansas.

Analysis of IRB Sections

With the five potential causes identified, all ten sections were evaluated based on the following properties: structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage. The following sections present in-depth analyses of the aforementioned five potential causes.

Coring Plan

Twenty seven cores were taken from each of the ten sections as part of the forensic investigation. Collection of the cores was performed by AHTD's Materials Division. Coring operations are shown in Figure 2.

Figure 2 – Coring operations

Coring operations were consistent in the field, with three sets of nine cores collected from each section. Within each set of nine, six cores were taken within the right and left wheel paths and three from cores were taken from between the wheel paths. Figure 3 illustrates the standard coring pattern. This coring pattern was repeated a total of three times for each 0.2-mile section; with 27 cores recovered for each section. Table 1 provides a summary of the ten sections.

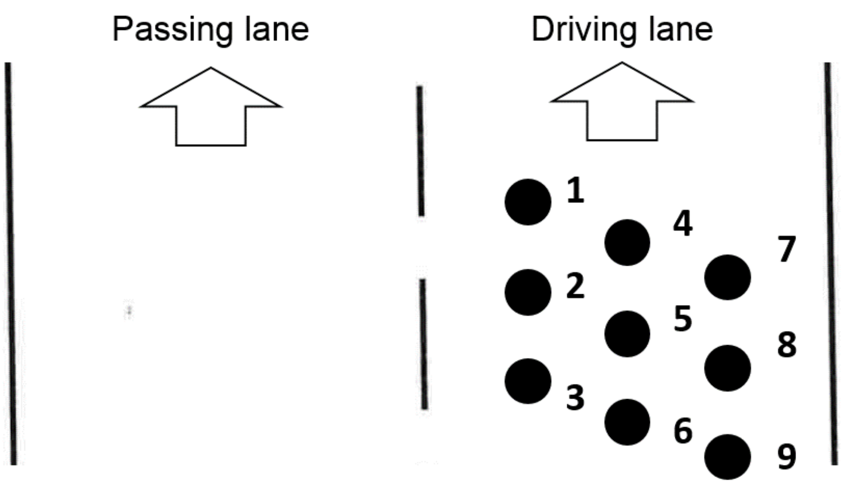

Figure 3 – Standard coring pattern (arrows indicate direction of traffic)

Table 1 – Project section summary

Section	Paving dates	Coring date	Age of pavement (years)
G1	Aug. 2002	February 2014	11.5
G2	Oct. 2003	February 2014	10.5
G3	Sept. 2003	February 2014	10.5
G4	Jul. 2001-Jun. 2002	November 2013	11
M1	Oct. 2001-Sept. 2002	November 2013	11
M2	Mar. 2004-Aug. 2004	January 2014	10
P1	Apr. 2002-Jan. 2004	January 2014	10
P2	May 2002-Sept. 2004	2013	9
P3	Apr. 2001-Apr. 2004	2013	9
P4	March 2002*	2014	12

*P4 date estimated from historical pavement condition surveys

<u>Structural capacity</u>

Upon arrival at the University of Arkansas - Fayetteville, pictures were taken of each core and lift thicknesses measured. Figure 4 is an image of a typical core, with the surface courses, the binder courses, and the base course labeled. Table 2 provides a summary of the average thicknesses of the twenty-seven cores from each of the ten sections. The base course thickness was not recorded due to the fact that not all cores arrived at the lab with the base course attached to the core. In these instances the base course was left in the coring hole as the material wasn't necessary to complete the testing plan.

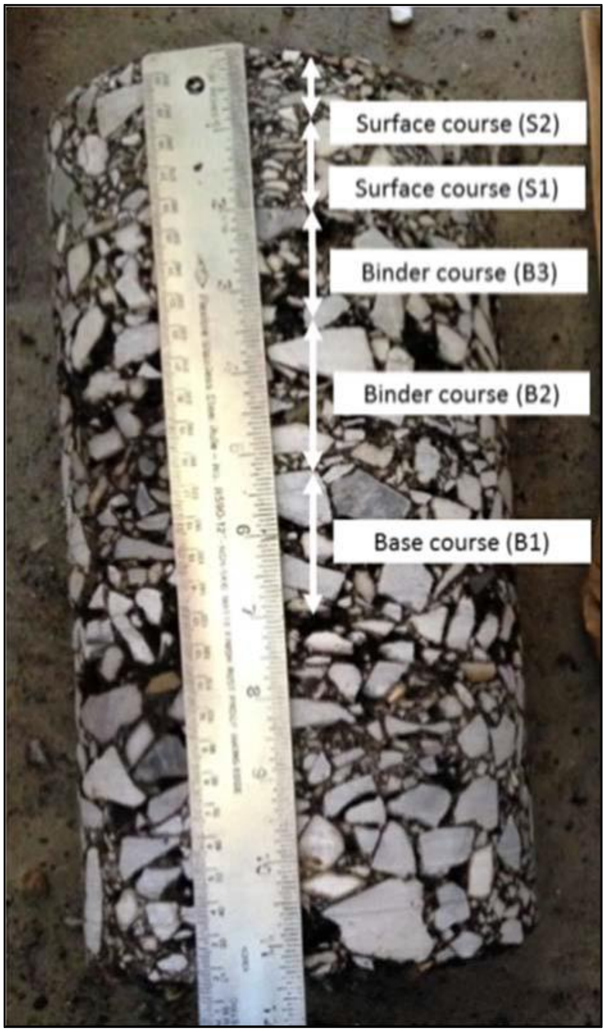

Figure 4 – Typical core cross section

Table 2 – Average pavement layer thickness

Section	Average core diameter (in)	Surface Course		Binder Course		Total thickness (in)
		(S2) (in)	(S1) (in)	(B3) (in)	(B2) (in)	
G1	6	1.95	2.76	3.40	3.72	11.83
G2	6	2.37	2.71	3.40	2.67	11.14
G3	6	2.16	2.32	3.65	4.76	12.89
G4	6	2.10	1.57	3.00	2.96	9.65
M1	6	2.61	2.61	3.15	4.99	13.35
M2	6	2.31	2.14	2.94	5.07	12.44
P1	6	2.19	2.29	3.32	5.50	13.31
P2	4	2.15	2.22	3.55	4.45	12.37
P3	4	1.64	2.33	2.63	2.29	8.89
P4	6	1.79	2.92	3.21	1.94	9.87
Average		2.15	2.39	3.23	3.84	11.57

In Table 2, it appears that all ten sections have structural capacity, as ten inches of pavement structure is no atypical for the interstate system. All ten pavements were designed using the AASHTO 1993 Design Guide procedures, and were design for a 20 year life span. The total thickness of each of the ten sections was approximately nine inches or greater but does not include the base course or the rubblized concrete below the flexible overlays. Therefore, along with traffic information provided in the main final report, any deficiencies associated with the structural design were eliminated.

Design Mixture properties

Several properties were tracked from the original mix designs used for construction for each of the sections. Field acceptance testing results were desired, but the data was not available. Therefore, design records were examined to capture properties from the mix design. Five properties were examined for the mixtures as part of this study: optimal asphalt content, voids in the mineral aggregate (VMA), AASHTO M 323, dust to total asphalt ratio, AASHTO M 323, retained stability, AHTD Test Method 455A-11, and design gyrations, AASHTO R 35.

After reviewing the available construction documents and electronic communications the data was found to be incomplete. Mixture property data was not available for G1, G2, G3, and P4. Of the data that was available all mixture design test results were within the specification limits as outlined in AHTD specification, year 2014, Division 400, Section 404 "Design and Quality Control of Asphalt Mixtures." AHTD follows AASHTO M 323, except for the following exceptions:

- Air void limits were as follows:
  - PG 64-22 and PG 70-22 mix designs: 4.5%
  - PG 76-22 mix designs: 4.0%
- Voids in the Mineral Aggregate (VMA) ranges were as follows:
  - Base course: 11.5% - 13.0%
  - Binder course: 12.5% - 14.0%
  - Surface course (12.5mm NMAS): 14.0% - 16.0%
  - Surface course (9.5mm NMAS): 15.0% - 17.0%
- Wheel tracking test results
- Water sensitivity was determined using AHTD Test Method 455A

These exceptions could have been due to specification changes since construction. The ten sections were constructed utilizing either the 1993 or 2003 specifications. While there is no specification for asphalt content, the levels seemed in line with expectations compared to successfully performing interstate sections and no particular grouping of sections was different than the others. AHTD specifications require the VMA to fall between 14% - 16% for 12.5mm NMAS surface mixtures. The dust to asphalt ratio has minimum and maximum limits of 0.6 and 1.2, respectively. The retained stability is required to be greater than 80%. As seen in Figure 5, all of the specifications were met for the available data.

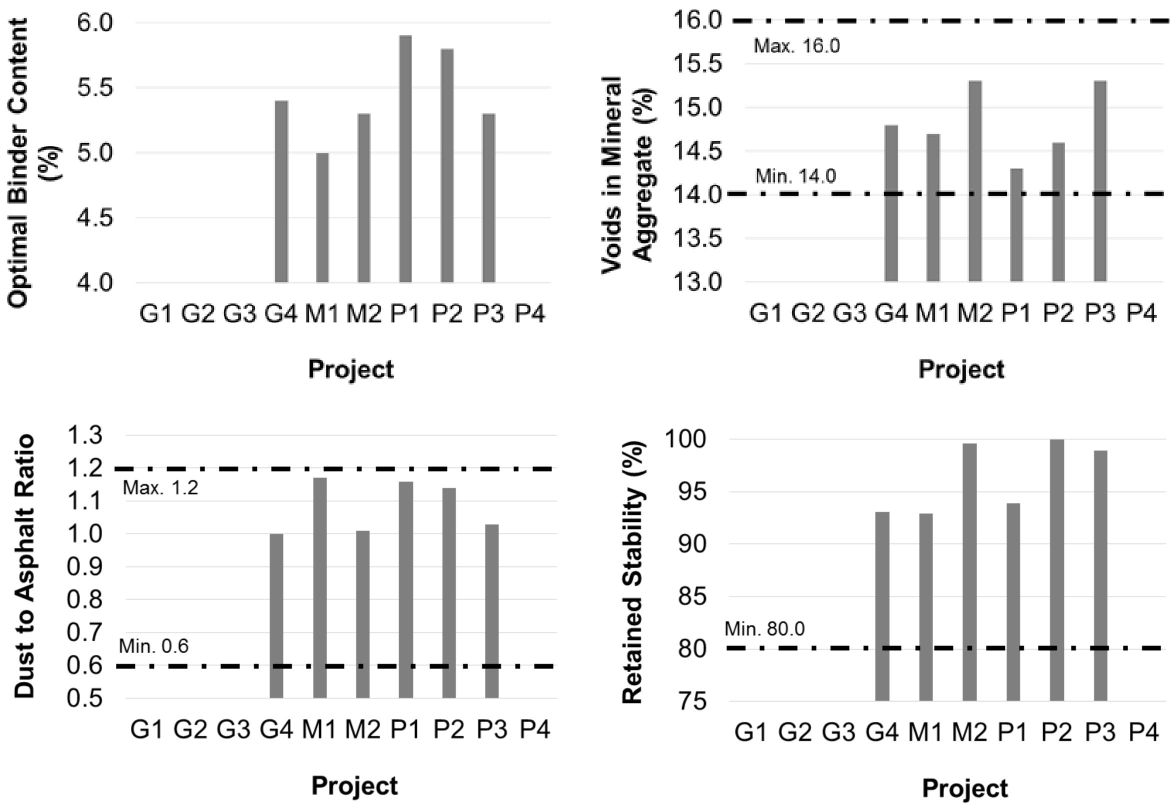

Figure 5 – Summary of laboratory mix design properties

In addition to the four aforementioned design mixture properties, the six sections, for which data was available, used an N_{des} = 125 gyrations per AASHTO R 35-15, as all of the sections had traffic greater than 30 million equivalent single axle loads (ESALs). Per specifications, the corresponding number of gyrations to 89% density (N_{init}) and number of gyrations to 98% density (N_{max}) were 9 and 205 gyrations, respectively. When considering potential changes to specifications, these numbers show that the gyrations are in line with expected Superpave design.

Mix design properties were not considered to be a contributing factor in identifying differences between sections with different field performance. According to a review done by the authors, over half of state DOTs have altered the AASHTO M 323 requirements for design air voids and VMA and/or altered the number of design gyrations specified in AAHTO R 35 in order to increase the optimum asphalt content. In addition, according to a review done by the authors, AHTD is the only state DOT that has made changes to AASHTO M 323 and AASHTO R 35 resulting in stiffer asphalt mixtures by increasing the design air void level.

Bond strength

After the cores were cataloged at the University of Arkansas, the bond strength between layers was determined based on the test method described by RamakrishnaReddy (2007). This test consists of a shearing a core sample at the layer interface after clamping it within a steel frame. The test can be run at two temperatures (70°F and 130°F) and two normal stresses (0 psi and 10 psi). More details on this test can be found in the full final report. All bond strength test results presented were performed on intact cores, at 70°F with zero normal stress. Tests were run at the interface of the top (S2) and bottom (S1) surface courses; and the bottom surface course (S1) and the top binder course (B3). Utilizing performance numbers based on National Center for Asphalt Technology (NCAT) Report 05-08 (West et al., 2005), bond strength was categorized into three levels of performance: “good” for strengths greater than 100 psi, “medium” for strengths between 50 psi and 100 psi, and “poor” for strengths less than 50 psi. Figure 6 shows the bond strength test results from the ten sections with a minimum of three replicates for each configuration. Error bars are shown to provide the variability for each set of replicates.

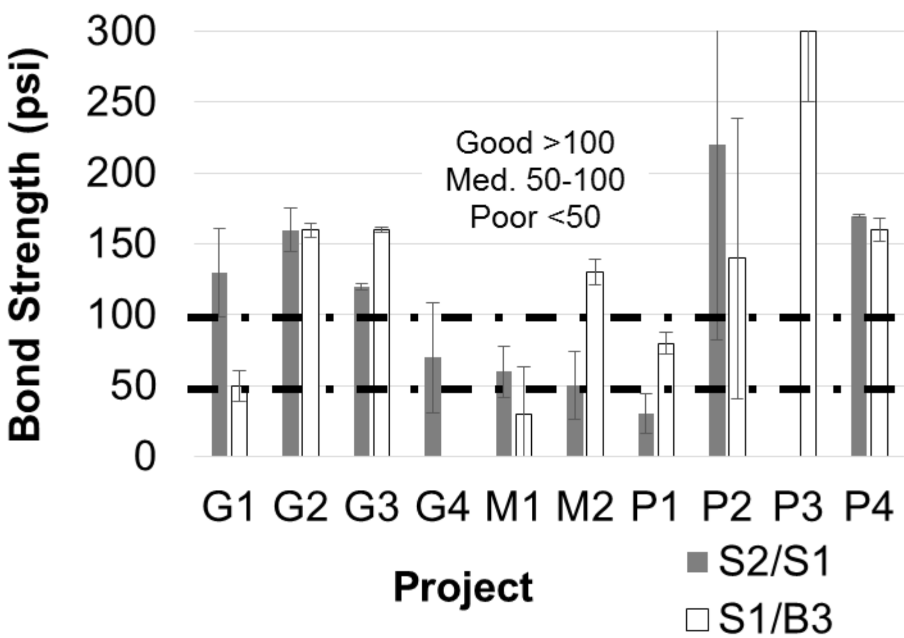

Figure 6 – Bond strength test results

The results in Figure 6 indicated that no clear trend exists between good, medium, and poor performing sections. However, these results could be misleading. If a core arrived in the lab intact, it was able to survive coring in the field. When taking a core in the field, a significant torqueing action is applied to the sample. The torque of the core barrel might be high enough to debond the sample while attempting to collect the sample. Or, the two layers could have not been bonded in the first place. Regardless, samples with adequate bond strength in the field were not affected, only samples with weak or no bond strength in the field were debonded by coring. Therefore, the number of cores that arrived in the lab along with the percent of debonded lifts was recorded. In general, 27 cores were taken per project.

Based on visual inspection and data in Figures 6 and 7, it was estimated that good performing sites had less than 20% debonded lifts upon arrival, medium (or fair) sites had 20-35%, and poor sites had more than 35% debonded lifts. Figures 7a and 7b summarize the percent of debonded lifts.

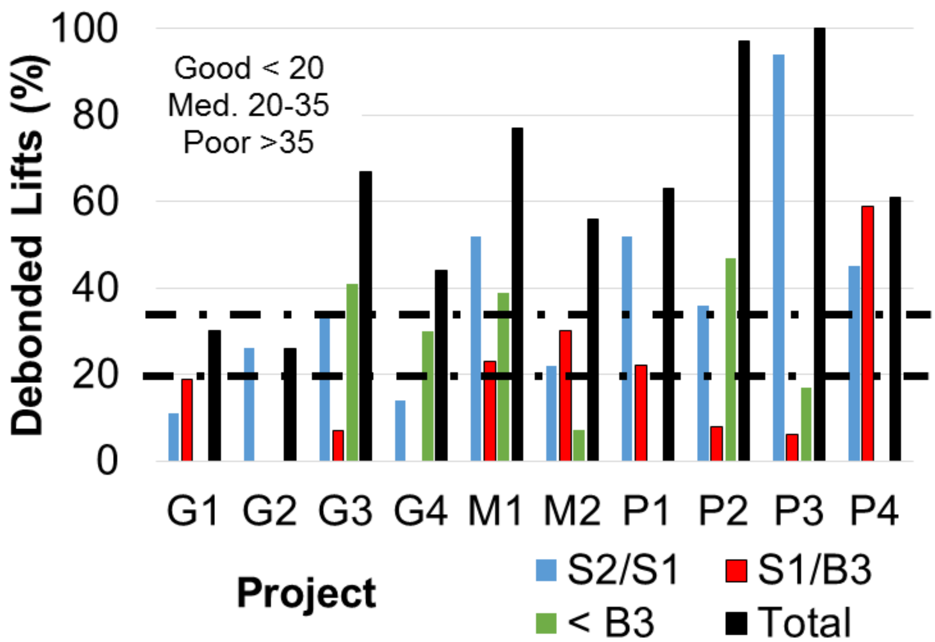

Figure 7 – Percentage of debonded lifts on arrival at lab

In Figure 7 it is apparent that the four poor performing sections have a higher percentages of debonded lifts than the good performing sections, meaning either a lack of or reduced effectiveness of the tack coat. This indicates two things. First, since the poor sections had fewer intact cores, there were fewer replicates to measure bond strength, which means that data may not be as robust as the medium and good performing sections. The bond strength could have been assumed to be zero, but it was unknown whether the bond strength was zero or simply lower than the torque applied during coring, therefore, the data was simply discarded. Second, quantifying the bond strength alone may not be appropriate, as any core that survives the coring process itself has enough bond strength to survive the coring process. Therefore, a new metric was developed that incorporated both bond strength and the number of surviving cores - the bond strength factor. The bond strength factor was calculated by multiplying the bond strength (as recorded in Figure 6) by the percentage, as a decimal, of the bonded cores (both S2/S1 and S1/B3, in Figure 7) as seen in Equation 1.

Bond Strength Factor = Bond Strength x Percentage of Bonded Cores Equation 1

This bond strength factor accounts for both the bond strength itself, and the number of cores that arrive in the lab that were still bonded together. Figure 8 shows the bond strength factor and potential specification limits for performance measures based on the ten sections analyzed.

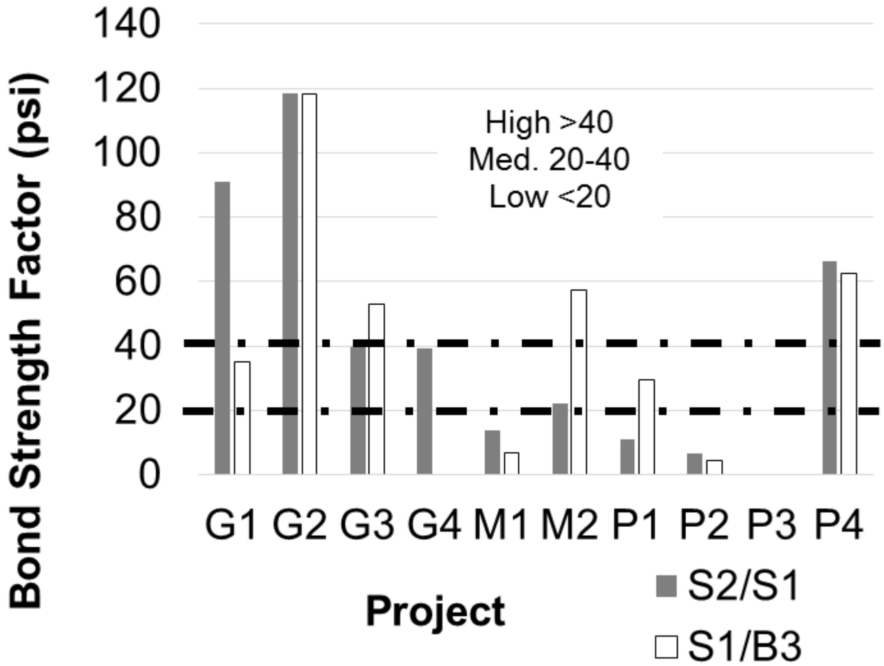

Figure 8 – Bond strength factor

In Figure 8, the three sets of sections with varying performances become better delineated. In general, the four good performing sections had either a medium or high bond strength factor, while the poor sections, in general, had a low bond strength factor. While the bond strength test is an excellent test to evaluate the performance of tack coat, it may give erroneous results if used on field cores without taking into account the number of cores that arrived in the lab intact. Overall, all three different metrics that capture different quantifications of bond strength were considered a contributing factor in identifying the performance differences between sections.

In-place air voids

After performing the bond strength test, the bulk specific gravity of the mixture (G_{mb}) was determined for each lift of each core per AASHTO T166. The air voids were determined per AASHTO T209; and were expected to be near the laboratory mix design air voids of 4.0% after approximately 9-12 years of interstate traffic. Figure 9 shows the average air voids for each project, along with proposed criteria for performance measures based on the ten sections analyzed.

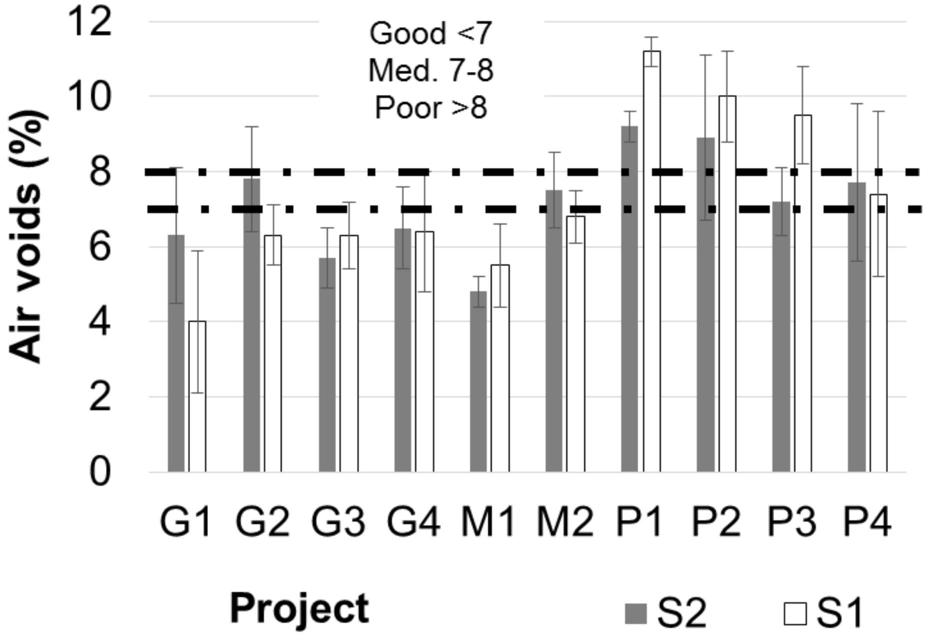

Figure 9 – Surface course air voids

In Figure 9, there is a clear trend of good performing sections having lower air voids versus the poor performing sections. For instance, on the wheel paths where the most truck traffic loads are expected, the average air voids for good performing sections varied from 5.3% to 6.7%, whereas poor performing sections ranged from 7.7% to 9.0%. A similar observation was made in Anderson et al., 2001, in which relatively high air voids (7.2% to 8.3%) were suspected to be one of the main causes of premature failure of HMA pavements on I-25. It was concluded that compaction efforts on poor performing sections of I-40 near Alma and Russellville may have contributed to excessive surface cracking. Based on the data from the ten sections, good performance was quantified by having less than 7% air voids, while poor performance was greater than 8%. These high levels of air voids were a significant surprise, especially after live traffic for 9-12 years, as pavements are designed to be in operation at 4% air voids. Table 3 summarizes the air voids data for the top and bottom surface course layer for the nine core locations with three replicates for each section; six which fell in the wheel path and three which fell in between the wheel paths.

Table 3 – Summary of air voids for each s

Project	S2 (Surface Top Lift)				S1 (Surface Lower Lift)			
	All Cores Avg. and (S.D.) [#Samples]	Left Wheel Path Avg.	Avg. Between Wheel Paths	Right Wheel Path Avg.	All Cores Avg. and (S.D.) [#Samples]	Left Wheel Path Avg.	Avg. Between Wheel Paths	Right Wheel Path Avg.
G1	6.3 (1.8) [10]	5.9	6.5	6.6	4.0 (1.9) [8]	4.6	4.8	3.4
G2	7.8 (1.4) [12]	9.2	8.7	6.7	6.3 (0.8) [12]	7.3	6.3	5.8
G3	5.7 (0.8) [12]	6.1	6.8	5.3	6.3 (0.9) [12]	6.6	6.5	6.2
G4	6.5 (1.1) [14]	6.2	7.3	6.6	6.4 (1.6) [14]	6.3	9.2	5.6
M1	4.8 (0.4) [7]	4.6	4.7	5.3	5.5 (1.1) [13]	5.0	6.2	5.8
M2	7.5 (1.0) [9]	8.2	6.4	7.0	6.8 (0.7) [13]	7.2	6.2	6.9
P1	9.2 (0.4) [10]	-	9.3	9.0	11.2 (0.4) [10]	-	11.2	10.8
P2	8.9 (2.2) [12]	8.9	9.6	8.2	10.0 (1.2) [12]	9.4	10.6	10.2
P3	7.2 (0.9) [12]	7.6	7.2	6.8	9.5 (1.3) [12]	9.1	9.1	10.3
P4	7.7 (2.1) [12]	7.6	7.8	7.7	7.4 (2.2) [12]	7.6	6.8	7.9

The in-place air voids were a considered a contributing factor in identifying differences between sections with different performance quality.

Moisture damage

The final potential cause investigated was the influence of moisture on the samples. Two methods were utilized in order to determine potential moisture damage using the cores. The first method was a visual inspection of the core upon arrival at the University of Arkansas, to observe any potential degradation of cores due to moisture. The second method was a stripping rating, outlined in AASHTO T 283, performed on the faces of samples tested in the Semi-Circular Bend (SCB) fracture test. Moisture damage is often identified by the formation of an hourglass shape in the core on each lift. This shape indicates the beginning of deterioration at the lift interfaces, which is associated with moisture damage. Figure 10 shows examples of four levels of core degradation.

Figure 10 – Core degradation, Left to right: good, medium, severe, very severe (rubble in bag)

In Figure 10, as moisture damage increased, the top and bottom of each lift deteriorated at the edges, forming a distinct hour glass shape. This shape was quantified with a good rating equaling one for those cores that had smooth sides and were intact, a medium rating equaling two for those cores that were separated and / or had an hourglass shape forming, and a poor rating equaling three for those cores that had a severe or very severe formed hourglass shape and/or substantial loose material. With these criteria, the core degradation was quantified. The results are shown in Figures 11, along with potential criteria for performance measures based on the ten sections analyzed. The quantification of the core degradation is a new metric designed to evaluate the condition of cores visually based on moisture damage after extraction from the field.

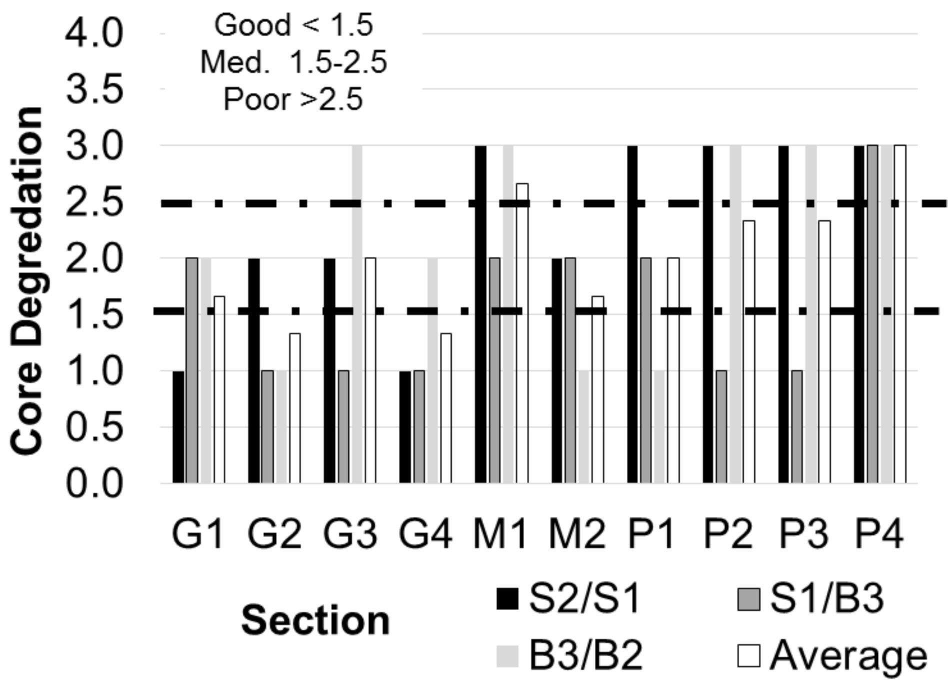

Figure 11 – Core degradation quantification between all interfaces

From the results seen in Figure 11, good, medium, and poor performing sections were quantified as less than 1.5, from 1.5 to 2.5, and greater than 2.5, respectively based on a visual inspection of the data. In Figure 11, the majority of good performing sections were either in the good or medium levels, while the poor were generally in the medium to poor performing levels. Moisture damage as identified by core degradation was considered a contributing factor in identification of differences between the sections with different performance quality.

The second quantification of moisture damage was by observation of uncoated aggregates on the two fractured faces after performing the SCB tensile fracture test. The SCB fracture test, AASHTO TP 105, is a low-temperature fracture test (run at 10°F, or -12°C) intended to give cracking characteristics of mixtures. Two SCB fracture samples were obtained from each core's lift and tested. After testing, each sample resulted in two halves whose faces were visually examined for uncoated aggregates. Again, following the stripping rating established in AASHTO T 283, a rating was assigned for each sample. This stripping rating is one to five, with one exhibiting no stripping or exposed aggregates and five representing extensive stripping. The rating is very subjective, but in this study, all ratings were performed by the same person - Dr. Shu Yang. A thorough comparative process was used to ensure the rating was consistent and would provide a valid ranking. Figure 12 shows the stripping rating results including error bars for a minimum of three replicates, along with proposed specifications for performance measures based on the ten sections analyzed.

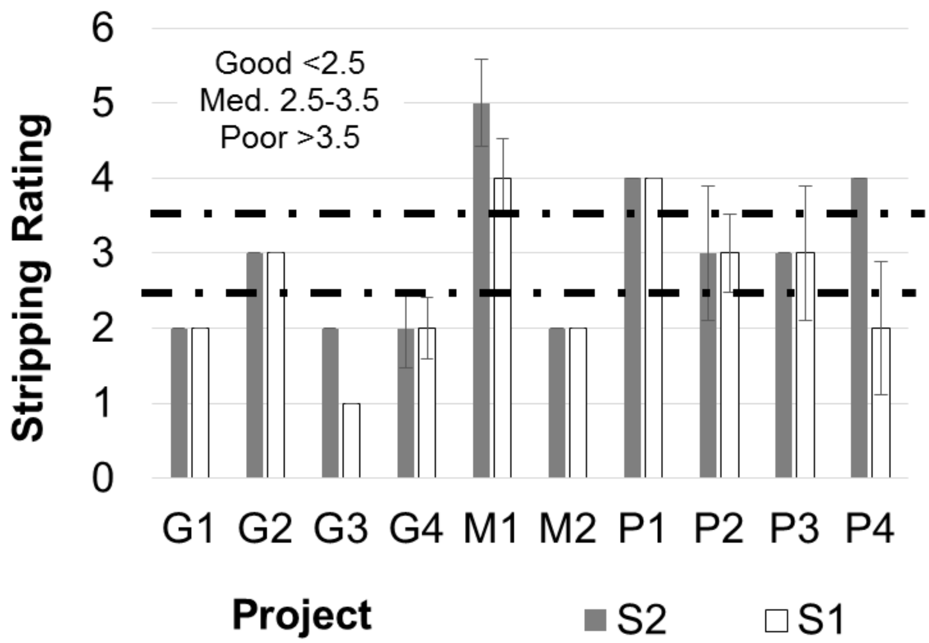

Figure 12 – Stripping rating of the ten sections

In Figure 12, the general trend is that the good performing sections had lower striping ratings than the poor performing sections. Moisture damage as identified by the stripping rating was considered a contributing factor in the identification of differences between the sections with different performance quality. In order to verify whether the three potential causes of premature deterioration could be correlated to actual field performance, each project is summarized and quantified for performance.

Project-by-Project Analysis

Five potential causes for the premature failure of Arkansas Interstates were identified and re- examined for each of the ten sections: structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage. The structural capacity and design mixture properties were not considered factors in differentiating the performance of the ten sections. However, bonding (bond strength test, percentage of bonded lifts, and bond strength factor), in-place air voids, and moisture damage (core degradation and stripping rating) were related to each of the sections performance. Each project was analyzed individually to identify specific properties associated with the cause for that particular pavement's performance.

Determining if these three properties were related to the pavement performance on a project-by-project basis was the next objective. Each property was given a qualitative rating of good, marginal, or poor. Each qualitative rating was also assigned a numerical value to establish a quantitative rating (good = 1, marginal = 2, poor = 3). Further, when possible, the properties most significant in each pavement's performance was determined.

G1 – I-30 at Social Hill

Bond strength between S2/S1 was 130 psi and 50 psi between S1/B3. Approximately 11% and 19% of S2/S1 and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The newly developed bond strength factors at the S2/S1 and S1/B3 interfaces were 91 psi and 35 psi, respectively. The average air voids of all cores for each of the two surface lifts were excellent per discuss in previous sections. The in-place air voids for S2 was 6.3% and for S1 was 4.0%. In terms of moisture damage, 85% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 2. Overall, the three properties were excellent and the pavement performance was good. It should be noted that a site visit in 2016 revealed the project was still in a good service condition. Table 4 provides a summary of G1's performance, with an average quantitative rating of 1.25.

Table 4 - Summary of critical properties, results, and ratings for project G1

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	130 and 50	Good and poor	2
Debonding: lifts debonded (%)	11 and 19	Good	1
Debonding: bond strength factor (psi)	91 and 35	Good and marginal	1.5
In-Place Air Voids (%)	6.3 and 4.0	Good	1
Moisture damage: core degradation (%)	85	Good	1
Moisture damage: stripping rating	2	Good	1
		Average quantitative rating	1.25

G2 – I-30 at Arkadelphia

The bond strengths between S2/S1 and S1/B3 were both 160 psi. 26% and 0% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 118 psi for both S2/S1 and S1/B3 interfaces. The average air voids for all cores in S2 was 7.8% which is marginal. The average air voids for all S1 lifts was 6.3%. In terms of moisture damage, 75% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 3 for both S2 and S1, which was a marginal rating. It should be noted that a site visit in 2016 revealed that this project had been milled and inlayed. Even though it was considered "good," it was not one of the better pavements based on discussion above. Table 5 provides a summary of G2's performance, with an average quantitative rating of 1.33.

Table 5 - Summary of critical properties, results, and ratings for project G2

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	160	Good	1
Debonding: lifts debonded (%)	26 and 0	Good and marginal	1.5
Debonding: bond strength factor (psi)	118 and 118	Good	1
In-Place Air Voids (%)	7.8 and 6.3	Good and marginal	1.5
Moisture damage: core degradation (%)	75	Good	1
Moisture damage: stripping rating	3 and 3	Marginal	2
		Average quantitative rating	1.33

G3 – I-30 at Caddo Valley

The bond strength between S2/S1 was 120 psi and 160 psi at the S1/B3 interface. 33% and 7% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 40 psi at the S2/S1 interface and 53 psi at the S1/B3 interface. The average air voids for all cores for each of the two surface lifts were good. The in-place air voids was 6.1% for S2 and 6.3% for S1. In terms of moisture damage, 67% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 2. Despite some debonding near the surface, the low air voids and moisture resistant material contributed to the pavement's overall good performance in the field. A site visit in 2016 revealed the project was still in service. Table 6 provides a summary of G3's performance, with an average quantitative rating of 1.25.

Table 6 - Summary of critical properties, results, and ratings for project G3

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	120 and 160	Good	1
Debonding: lifts debonded (%)	33 and 7	Good and marginal	1.5
Debonding: bond strength factor (psi)	40 and 53	Good	1
In-Place Air Voids (%)	6.1 and 6.3	Good	1
Moisture damage: core degradation (%)	67	Marginal	2
Moisture damage: stripping rating	1 and 2	Good	1
		Average quantitative rating	1.25

G4 – I-40 at Brinkley

The bond strength between S2/S1 was 70 psi. Bond strength testing was not possible at the S1/B3 interface. 14% and 0% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 39 psi at the S2/S1 interface. The average air voids for all cores for each of the two surface lifts were good. The in-place air voids for S2 was 6.5% and for S1 was 6.4%. In terms of moisture damage, 85% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 2 for both S2 and S1. Despite some debonding in the lower lifts, the low air voids and moisture resistant material contributed to the pavement overall good performance in the field. Table 7 provides a summary of G4's performance, with an average quantitative rating of 1.33.

Table 7 - Summary of critical properties, results, and ratings for project G4

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	70	Marginal	2
Debonding: lifts debonded (%)	14 and 0	Good	1
Debonding: bond strength factor (psi)	39	Marginal	2
In-Place Air Voids (%)	6.5 and 6.4	Good	1
Moisture damage: core degradation (%)	85%	Good	1
Moisture damage: stripping rating	2 and 2	Good	1
		Average quantitative rating	1.33

M1 – I-40 at Shearerville

The bond strength between S2/S1 was 60 psi and the bond strength between S1/B3 was 30 psi, for a marginal and poor rating. 48% and 74% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 14 psi at the S2/S1 interface and 7 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were good. The in-place air voids for S2 was 4.8% and for S1 was 5.3%. In terms of moisture damage, 45% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 4 for the S2 layer and 5 for the S1 layer. The only acceptable property was the in-place air voids. Moisture susceptibility and debonding were poor to marginal likely leading to the marginal performance. It is surprising the pavement performed as well as it did, but that may speak to the importance of in-place air voids. Table 8 provides a summary of M1's performance, with an average quantitative rating of 2.5.

Table 8 - Summary of critical properties, results, and ratings for project M1

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	60 and 30	Marginal and poor	2.5
Debonding: lifts debonded (%)	48 and 74	Marginal and poor	2.5
Debonding: bond strength factor (psi)	14 and 7	Poor	3
In-Place Air Voids (%)	4.8 and 5.3	Good	1
Moisture damage: core degradation (%)	45	Poor	3
Moisture damage: stripping rating	4 and 5	Poor	3
		Average quantitative rating	2.5

M2 – I-40 at Ozark

The bond strength between S2/S1 was 50 psi and the bond strength between S1/B3 was 130 psi, equating to a poor and good rating, respectively. 22% and 30% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 22 psi at the S2/S1 interface and 57 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were marginal. The in-place air voids for S2 was 7.5% and for S1 was 6.8%. In terms of moisture damage, 78% of the cores at the S2/S1 interface were in marginal condition, and the stripping rating was 2 for both the S2 layer and S1 layer. The aggregates seemed to be moisture resistant, but all the other properties were marginal. In this case, the tack coat failure was thought to be the leading cause of the pavement's marginal performance. Table 9 provides a summary of M2's performance, with an average quantitative rating of 1.83.

Table 9 - Summary of critical properties, results, and ratings for project M2

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	50 and 130	Good and poor	2.5
Debonding: lifts debonded (%)	22 and 30	Marginal	2
Debonding: bond strength factor (psi)	22 and 57	Good and marginal	1.5
In-Place Air Voids (%)	7.5 and 6.8	Marginal	2
Moisture damage: core degradation (%)	78	Marginal	2
Moisture damage: stripping rating	2 and 2	Good	1
		Average quantitative rating	1.83

<u>P1 – I-40 at Alma</u>

The bond strength between S2/S1 was 80 psi and the bond strength between S1/B3 was 80 psi, for a marginal rating. 52% and 22% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival at the lab. The bond strength factor was 11 psi at the S2/S1 interface and 30 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were poor. The in-place air voids for S2 was 9.2% and for S1 was 11.3%. In terms of moisture damage, 48% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 4 for both the S2 layer and S1 layers. With a moisture susceptible material and high in-place air voids, it was not a surprise that this was a poor performing pavement. Table 10 provides a summary of P1's performance, with an average quantitative rating of 2.67.

Table 10 - Summary of critical properties, results, and ratings for project P1

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	80 and 80	Marginal	2
Debonding: lifts debonded (%)	52 and 22	Marginal and poor	2.5
Debonding: bond strength factor (psi)	11 and 30	Marginal and poor	2.5
In-Place Air Voids (%)	9.2 and 11.2	Poor	3
Moisture damage: core degradation (%)	48	Poor	3
Moisture damage: stripping rating	4 and 4	Poor	3
		Average quantitative rating	2.67

<u>P2 – I-40 at Russellville</u>

The bond strength between S2/S1 was 220 psi and the bond strength between S1/B3 was 140 psi, equating to a good rating. 36% and 8% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival to the lab. The bond strength factor was 7 psi at the S2/S1 interface and 4 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were poor. The in-place air voids for S2 was 8.9% and for S1 was 10.0%. In terms of moisture damage, 64% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 3 for both the S2 layer and S1 layers. The aggregates were only marginally moisture susceptible.

However, the high in-place air voids and the amount of debonding likely led to the poor performance. Table 11 provides a summary of P2's performance, with an average quantitative rating of 2.33.

Table 11 - Summary of critical properties, results, and ratings for project P2

Property	**Result**	**Qualitative rating**	**Quantitative rating**
Debonding: bond strength test (psi)	220 and 140	Good	1
Debonding: lifts debonded (%)	36 and 8	Poor and good	2
Debonding: bond strength factor (psi)	7 and 4	Poor	3
In-Place Air Voids (%)	8.9 and 10.0	Poor	3
Moisture damage: core degradation (%)	64	Poor	3
Moisture damage: stripping rating	3 and 3	Marginal	2
		Average quantitative rating	2.33

<u>P3 – I-40 at Atkins</u>

It was not possible to test the bond strength between S2/S1 and the bond strength between S1/B3 was 300 psi, for a good rating. 94% and 6% of the S2/S1and S1/B3 interfaces, respectively, were debonded upon arrival to the lab. The bond strength factor was not possible to determine at the S2/S1 interface and 0 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were marginal and poor. The in-place air voids for S2 was 7.2% and for S1 was 9.5%. In terms of moisture damage, 4% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 3 for the S2 layer and 3 for the S1 layer. The aggregates were only marginally moisture susceptible. However, the high in-place air voids and a lot of debonding likely led to the poor performance. Table 12 provides a summary of P3's performance, with an average quantitative rating of 2.42.

Table 12 - Summary of critical properties, results, and ratings for project P3

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	Not possible and 300	Poor and good	2
Debonding: lifts debonded (%)	94 and 6	Poor and good	2
Debonding: bond strength factor (psi)	Not possible and 0	Poor	3
In-Place Air Voids (%)	7.2 and 9.5	Marginal and poor	2.5
Moisture damage: core degradation (%)	4	Poor	3
Moisture damage: stripping rating	3 and 3	Marginal	2
		Average quantitative rating	2.42

P4 – I-40 at Forrest City

The bond strength between S2/S1 was 170 psi and the bond strength between S1/B3 was 160 psi, for a good rating. 46% and 59% of the interfaces between lifts S2/S1and S1/B3, respectively, were debonded upon arrival to the lab. The bond strength factor was 66 psi at the S2/S1 interface and 62 psi at the S1/B3 interface. The average air voids of all cores for each of the two surface lifts were marginal. The in-place air voids for S2 was 7.7% and for S1 was 7.4%. In terms of moisture damage, 66% of the cores at the S2/S1 interface were in good condition, and the stripping rating was 4 for the S2 layer and 2 for the S1 layer. With a moisture susceptible material and marginal in-place air voids, it was not a surprise that this was a poor performing pavement. Table 13 provides a summary of P4's performance, with an average quantitative rating of 2.17.

Table 13 - Summary of critical properties, results, and ratings for project P4

Property	Result	Qualitative rating	Quantitative rating
Debonding: bond strength test (psi)	170 and 160	Good	1
Debonding: lifts debonded (%)	46 and 59	Poor	3
Debonding: bond strength factor (psi)	66 and 62	Good	1
In-Place Air Voids (%)	7.7 and 7.4	Marginal	2
Moisture damage: core degradation (%)	66	Poor	3
Moisture damage: stripping rating	4 and 2	Poor	3
		Average quantitative rating	2.17

Summary

A combination of interrelated factors including in-place bond strength, air voids, and moisture damage showed a relationship in distinguishing pavements with good, medium, and poor performance. The combination of these three factors demonstrates that often there is no single factor that leads to premature pavement failure, but usually a combination of factors. This research recommends that in order to evaluate the potential for premature failure, debonding (bond strength test, lifts debonded, and bond strength factor), in-place air voids, and moisture damage (core degradation and stripping rating) be examined together. By assigning a quantitative rating of one, two, and three for good, marginal, and poor, respectively, a direct comparison was made among the ten sections as shown in Figure 13.

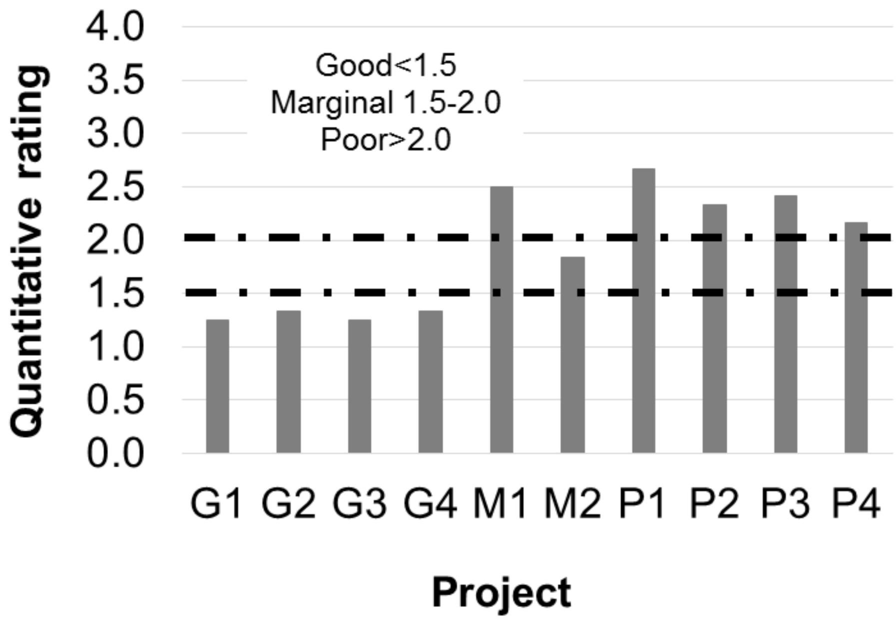

Figure 13 – Final ratings of ten sections over three properties

Based on these ten sections, it is recommended that if the average quantitative rating of the three properties is less than 1.5, the project is in good condition. If the rating is between 1.5 and 2.0, the project is in marginal condition and special attention should be given to the project. Finally, if the rating is above 2.0, the project is in poor condition and an evaluation should be made to fix either the bonding, air voids, or moisture damage.

Conclusions

The Arkansas State Highway and Transportation Department embarked on an ambitious interstate rehabilitation program starting in 1999 to rehabilitation all of the interstates within the state. However, after 9-12 years of service, several of the sections showed signs of premature deterioration. This research was performed in an attempt to determine reasons why some sections performed as designed while others showed premature deterioration. Ten sections were selected, four exhibiting good performance, two medium performance, and four poor performance. Cores were taken from the ten sections for evaluation. Five potential causes were identified that could have led to premature deterioration: structural capacity, design mixture properties, bond strength, in-place air voids, and moisture damage.

After evaluating all ten sections, it was determined that the bond strength, in-place air voids, and moisture damage contributed to the premature deterioration. Overall, not all good sections performed "good" in all of the properties evaluated: nor did all poor sections performed "poor" in all of the properties evaluated. There was a complex interrelationship of three properties: that differentiated the performance of the sections. Specifically, the three properties of importance were broken down into several subsets, with existing tests and metrics utilized and new metrics developed.

Existing tests used were the bond strength and in-place air voids tests. New metrics developed were the percentage of debonded lifts, the bond strength factor, the core degradation, and the stripping rating. In addition, a composite rating of all six tests was proposed to evaluate future sections.

Specific findings include:

- Bond strength
    - Bond strength test: the bond strength test was run at 70°C with no normal stress, was run and limits developed by NCAT were used for evaluation. Good performance was a bond strength value greater than 100 psi, medium performance was a bond strength from 50-100 psi, and poor performance was less than 50 psi.
    - Percentage of debonded lifts: the number of debonded lifts on arrival at the lab, indicating that debonding occurred either during the coring process, the transportation process, or the in situ pavement was not bonded. Analyzing the data gathered in this study, good performance was less than 20% debonded lifts, medium performance was 20-35%, and poor performance was greater than 35% debonded lifts.
    - Bond strength factor: the bond strength factor, a function of the bond strength test and percentage of debonded lifts, was also developed based on data gathered in this study. Good performance was defined as greater than 40 psi, medium performance was 20-40 psi, and poor performance of the bond strength factor was less than 20 psi.
- In-place air voids
    - All of the surface course mixtures were designed for in-service air voids of 4.0%. Based on data collected in this study, good performing pavements had air voids less than 7%, medium performing pavements had air voids from 7-8%, and poor was greater than 8%.
- Moisture damage
    - Core degradation: cores were examined, particularly at the lift interfaces, for signs of degradation due to moisture damage. Based on data collected during this study, good performing pavements had a core degradation less than 1.5, a medium rating was from 1.5 to 2.5, and poor performance was greater than 2.5.
    - Stripping rating: the stripping of asphalt binder from aggregates was quantified following the procedure found in AASHTO T283 on faces formed during the SCB fracture test. Based on data collected during his study, good performing pavements received a stripping rating less than 2.5, a medium rating was 2.5-3.5, and poor performance was greater than 3.5.
- Overall quantitative rating
    - The six metrics described under bond strength, in-place air voids, and moisture damage were averaged for each project. Based on the ratings from the four good performing sections, the two medium performing sections, and the four poor performing sections, an overall quantitative rating of less than 1.5 was classified as good, a medium rating was 1.5-2.0, and a poor overall quantitative rating was greater than 2.0.

Recommendations

Based on this research, three recommendations have been made to AHTD. First, the agency should improve their moisture susceptibility testing procedures. From the data available, all of the mixtures passed the existing moisture test, retained stability. However, Arkansas is the only state currently requiring this test, and it is recommended that AASHTO T 283 be implemented. Second, the air voids for all ten sections was higher than expected, and in many cases to the detriment of the pavement performance. Therefore, it is recommended to examine the mix design requirement to ensure adequate air voids are obtained after trafficking. This should involve an examination of air void requirements, VMA, and number of gyrations during mix design, which would result in better performing pavement. Third, while no incorrect practices were found in Section 401 of AHTD's specifications, it is recommended that a review of cleaning inspections of roadways (401.03a) and application of tack coat (401.03c) be conducted to ensure that these sections are properly being followed during construction. Potentially, AHTD could introduce a new field test to measure either tack rate or coverage, such as South Carolina SC-T-86 (Determination of Asphalt Tack Coat Roadway Placement Rate), or a field shear, torsion, or pull off test.

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