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16. Abstract

Within six months of construction, areas with excessive premature rutting were noticed on a 4.95 mile long asphaltic concrete overlay project on the I-20 corridor in Madison Parish, Louisiana. The scope of the project included full depth patching of the existing 8-inch Continuously Reinforced Concrete Pavement and providing a structural overlay with a 2-inch Stone Matrix Asphalt (SMA) layer on top of a 7.5-inch minimum layer of Superpave Level 2 Binder Course. Prior to this report, at the request of District 05, LTRC Asphalt Material Lab performed two rounds of forensic evaluation of the Stone Matrix Asphalt (SMA) surface layer of that job. From the preliminary investigations, the blended aggregate gradations of SMA mixture at isolated areas seemed to be the cause of the failures. However, since the occurrence of the initial rutting, the problem kept progressing with time and rutting was detected for almost the entire length of the job. Twenty one full depth asphaltic concrete roadway cores were sampled from various locations and a suite of laboratory tests were conducted to evaluate the mixture. Tests include various volumetric tests along with Loaded Wheel Tests (LWT) and boil tests to quantify the moisture susceptibility of the mixture. The SMA layer was excluded from this round of testing as that mix was already included in the first and second rounds. Investigation into design features revealed lift thicknesses being placed that were thinner than adequate for the nominal maximum aggregate size used in the mixture. Even though a small percentage of aggregate gradations and asphalt contents of Superpave BC mixes were found to be out of the validation tolerance limits in the latest round of evaluation, no correlation was noticed between materials and performance of distressed and non-distressed areas. However, field investigations, subsurface drainage inspection, and laboratory evaluation of mixtures demonstrated the occurrence of moisture damage in the leveling and binder course lifts. High Speed Profiler and FWD data were collected and evaluated in an effort to define areas of weaker pavement for estimating the amount of full depth patching that would be required. LTRC recommends modifying the existing drainage system to help eliminate the moisture from the existing pavement structure. Milling the existing surface course of each outside travel lane and replacing with a new HMA mixture is recommended.

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Evaluation of Rutting Distresses on I-20 near Mound to Delta Scales

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January 2014

ABSTRACT

Within six months of construction, areas with excessive premature rutting were noticed on a 4.95 mile long asphaltic concrete overlay project on the I-20 corridor in Madison Parish, Louisiana. The scope of the project included full depth patching of the existing 8-inch Continuously Reinforced Concrete Pavement and providing a structural overlay with a 2-inch Stone Matrix Asphalt (SMA) layer on top of a 7.5-inch minimum layer of Superpave Level 2 Binder Course. Prior to this report, at the request of District 05, LTRC Asphalt Material Lab performed two rounds of forensic evaluation of the Stone Matrix Asphalt (SMA) surface layer of that job. From the preliminary investigations, the blended aggregate gradations of SMA mixture at isolated areas seemed to be the cause of the failures. However, since the occurrence of the initial rutting, the problem kept progressing with time and rutting was detected for almost the entire length of the job. Twenty one full depth asphaltic concrete roadway cores were sampled from various locations and a suite of laboratory tests were conducted to evaluate the mixture. Tests include various volumetric tests along with Loaded Wheel Tests (LWT) and boil tests to quantify the moisture susceptibility of the mixture. The SMA layer was excluded from this round of testing as that mix was already included in the first and second rounds. Investigation into design features revealed lift thicknesses being placed that were thinner than adequate for the nominal maximum aggregate size used in the mixture. Even though a small percentage of aggregate gradations and asphalt contents of Superpave BC mixes were found to be out of the validation tolerance limits in the latest round of evaluation, no correlation was noticed between materials and performance of distressed and non-distressed areas. However, field investigations, subsurface drainage inspection, and laboratory evaluation of mixtures demonstrated the occurrence of moisture damage in the leveling and binder course lifts. High Speed Profiler and FWD data were collected and evaluated in an effort to define areas of weaker pavement for estimating the amount of full depth patching that would be required. LTRC recommends modifying the existing drainage system to help eliminate the moisture from the existing pavement structure. Milling the existing surface course of each outside travel lane and replacing with a new HMA mixture is recommended.

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INTRODUCTION

Moisture damage in pavement structures depends on material properties, design factors, and subsurface drainage capabilities. Water can enter the pavement through surface infiltration, joints, cracks, pores, and through movement by subsurface water *[1]*. Among these, the most common water movement is upward by capillary under a pavement. The subbase or subgrade in an existing roadway can get saturated with the capillary moisture whenever the layer lacks in desired permeability *[2]*. Excessive moisture in a pavement structure can cause faulting and associated pumping in rigid pavements. It may reduce strength, strip-off asphalt from the mixture, and develop extensive cracking from loss of subgrade support in flexible pavement. Since the beginning of asphaltic pavements, drainage of water has been given a very important consideration, as many premature pavement failures are found to be tied to inadequate subsurface drainage systems. Kandhal et al. reported case histories of projects with inadequate subsurface drainage that led to localized premature failure of over-saturated pavements *[3]*. In the case of an insufficient drainage system, moisture may flood the base and rise through the pavement and deteriorate the pavement structure through premature stripping. Proper drainage of surface and subsurface water can prevent the accumulation of moisture in pavement and consequently enhance the pavement performance, reducing the premature failures and associated maintenance cost.

Premature failures in asphalt pavement are not very common in Louisiana. However, recently a premature pavement failure was noticed in the form of excessive rutting in a 4.95 miles long asphaltic concrete overlay job on the I-20 corridor in Madison Parish (State Project No. 451-08-0078). As shown in Figure 1, the project started close to the Mound interchange (Log mile: 27.880) and ended west of the Vicksburg Bridge (Log mile: 32.851). Two different sections were involved in this project. The first section consisted of a Continuously Reinforced Concrete Pavement (CRCP) as the existing pavement. The second section consisted of Jointed Plain Concrete Pavement (JPCP) in lieu of CRCP. This investigation focuses solely on the pavement distresses observed in the first section of the project. The project scope of the inspected section included full depth patching of the existing 8-in. CRCP and providing a structural overlay with a 2-in. Stone Matrix Asphalt (SMA) layer on top of a 7.5-in. (minimum) of Superpave Level 2 Binder Course (BC). Subsequently, this project provided a change in the existing cross slope from 2.0 percent to 2.5 percent. Typical sectional drawings showed an existing soil cement base underneath both shoulder and mainline of this section, which was to remain. The project was designed for the 20-year ESALs of 22,212,185 (2025 ADT of 42300 with 40 percent truck).

Initially, the premature rutting was noticed only at localized areas within six months of construction. Soon after, at the request of the LADOTD District 05 engineers, the Louisiana

Transportation Research Center (LTRC) asphalt research group performed a small scale forensic evaluation in December 2011 followed by the second round completed in February 2012, assuming the top lift (a 2-in. SMA layer) was the suspect. Loaded Wheel Test (LWT), percent asphalt content, and blended aggregate gradations tests were performed on the collected roadway cores. The cores were reported to be taken from good, bad, and very bad areas of the roadway. From the combination of blended gradations and LWT test results, it was noticed that the SMA mixture representing the "good section" was produced according to LADOTD's specification and, consequently, met the performance criteria. Alternatively, mixtures that did not meet the gradations requirements of LADOTD failed to show the desired performance. The detailed reports of those two evaluations are attached in Appendix A of this report. However, the rutting issues kept progressing; within the next six months, the problem was detected in the entire length of the outside lanes of the project. Numerous sections in the inside lanes also showed signs of excessive rutting. At this point, it was realized that the rutting problem was not limited to the top layer of the pavement and a more in-depth investigation was necessary. On the basis of previous experience, historical reports of the job, and initial site investigation, the LTRC research team anticipated that the presence of moisture in the pavement structure was possibly the major contributor to the premature rutting. This paper presents the case history that identified the root cause of the failure mechanism.

Figure 1
Project location on I-20 in Madison Parish

OBJECTIVE

The principal objective of this study was to conduct a forensic evaluation, as a follow up of two initial preliminary evaluations, of the aforementioned I-20 corridor, where excessive premature rutting was noticed within six months of the construction of the project. In particular, the objectives included the following:

- Field evaluation, which included visual inspection and measuring the extent of rutting.
- Laboratory evaluation of the roadway cores.
- Checking the roadway design data for any potential issues related to rutting.
- Determining pavement smoothness throughout the project.
- Evaluating pavement structure to determine severity and extent of full depth deterioration for use in estimating full depth rehabilitation limits.

SCOPE

For laboratory performance evaluation, 21 full depth asphaltic concrete roadway cores were sampled from various locations of the problematic I-20 corridor. Several of those roadway cores were separated into individual lifts; Superpave binder course (BC) lifts were tested to determine the compliance with specifications for asphalt content and blended aggregate gradations. Additionally, Loaded Wheel Tests (LWT) and boil tests were conducted to quantify the moisture susceptibility of the mixture. The SMA layer was excluded from this round of testing as that mix was already included in the first and second rounds. The High Speed Profiler was used to measure the extent of premature rutting throughout the project. The Falling Weight Deflectometer (FWD) was also used to evaluate the pavement structure in an effort to identify areas that would require full depth re-habilitation.

METHODOLOGY

To assess the overall condition of the pavement, a visual condition survey and a nondestructive distress condition survey were conducted at first. As shown in Figure 2, rut depths were measured randomly using a traditional straight edge and measuring gauge at severely rutted areas. Afterwards, an automated laser profiler was utilized to record the rut depth for the entire length of the project. Historical records such as design details, preliminary engineering reports, and construction records were also reviewed to find the possible clues to the pavement failure. After the preliminary site investigation, 21 full depth 6-in. diameter asphaltic concrete roadway cores were taken from four strategic locations to evaluate the current in situ pavement condition. Roadway cores were sampled from sites that rutted (wheel paths) along with samples from non-distressed areas (center line). The locations of cored test sections and the schematics of coring are illustrated in Figures 3 and 4, respectively; whereas, a brief description of those cores is presented in Table 1. All cores had 4-lifts of Superpave mixture with approximately 2-in. thickness per lift in addition to the 2-in SMA. The arrangement of various lifts found in the roadway cores is illustrated in Figure 5. According to the plant production data and roadway reports, the bottom two lifts, Lift A and Lift B were deemed as the leveling courses; whereas, the top two lifts, Lift C and Lift D were the binder courses. It should be noted that all Superpave lifts were produced in accordance with the same Job Mix Formula (JMF).

The roadway cores were brought back to the LTRC asphalt laboratory, separated into individual lifts, and subsequently tested to determine specification compliance for asphalt content and blended aggregate gradations. In addition, LWT and boil tests were conducted to quantify the moisture susceptibility of the mixture. As the SMA layer of this project has been tested in the first and second round to forensic evaluation, the objective of this evaluation was set to examine the Superpave binder course layers for the possible cause of the rutting failure.

Figure 2
Localized rut measurement using traditional straight edge

Figure 3
Locations of test sections

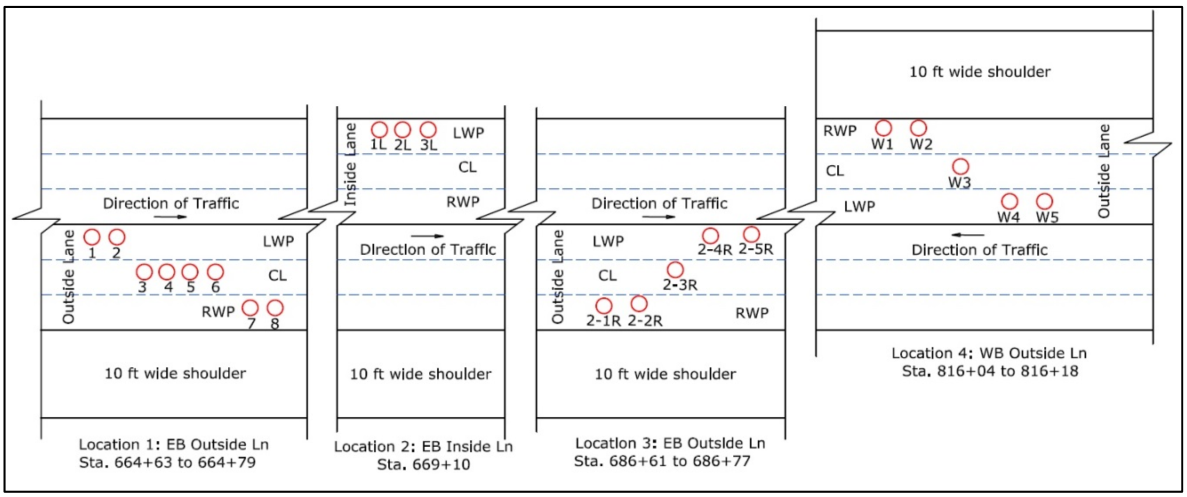


Figure 4
Schematics of coring

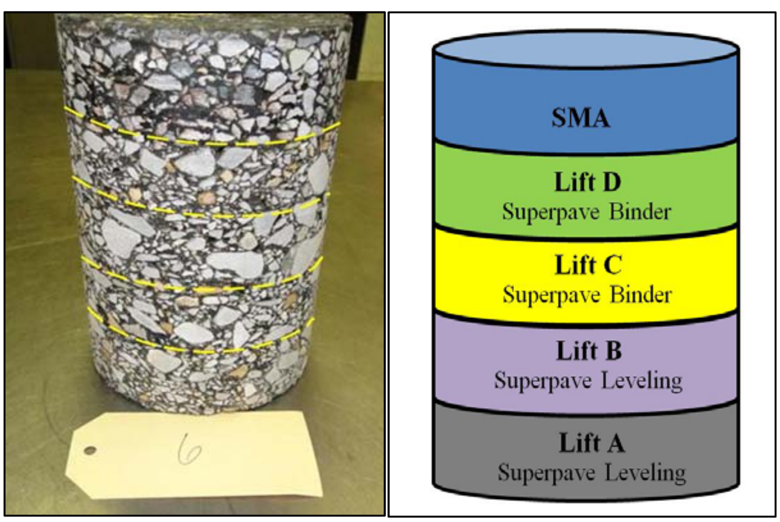


Figure 5
Layers in a roadway core

Table 1
Descriptions of roadway cores

Section	Sample ID	Location Of Core	Rut Depth (in)	Ht. of Total Core (in)	Ht. of SMA Layer (in)
EB, Outside Lane Sta. 664+63 to 664+79	1	LWP	5/8	8-3/8	1-5/8
	2	LWP			
	3	CL		9-0	2-0
	4	CL		8-5/8	2-0
	5	CL		8-3/4	2-0
	6	CL		8-3/4	2-0
	7	RWP	3/8	8-3/8	1-7/8
	8	RWP		8-1/2	1-7/8
EB, Inside Lane Sta. 669+10 (apprx.)	1L	LWP		10-3/8	2-0
	2L	LWP		10-3/8	2-0
	3L	LWP		10-3/8	2-0
EB, Outside Lane Sta. 686+61 to 686+77	2-1R	RWP	3/8	9-1/2	1-7/8
	2-2R	RWP		9-1/2	1-7/8
	2-3R	CL		10-1/4	2-1/8
	2-4R	LWP		9-9/16	1-3/4
	2-5R	LWP		9-1/2	1-3/4
WB, Outside Lane Sta. 816+04 to 816+18	W-1	RWP		9-3/8	1-3/4
	W-2	RWP			
	W-3	CL			
	W-4	LWP		9-0	1-11/16
	W-5	LWP		9-0	1-11/16

High Speed Profiler

The high speed profiler was used to collect rutting and roughness data [IRI (in/mile)] for the entire project in 0.1-mile intervals. According to FHWA, the IRI limits presented in Table 2 can be used to define ride quality *[4]*.

Table 2
Ride Quality

Ride Quality	IRI (in. per mile)
Smooth	0 to 80
Moderate	80 to 130
Rough	>130

FWD

The Falling Weight Deflectometer is a device used to access the strength of the pavement structure. From its tests, the pavement layer moduli, and the in-place structure number (SN_{eff}) were determined *[5-11]*.

FWD tests were conducted on two occasions at similar locations on July 9 and 10, 2013, on the inside and outside lanes in the east and west bound directions. Tests were conducted at 0.05-mile (264-ft.) intervals on the outside lanes and 0.1-mile (528-ft.) intervals on the inside lanes. Tests were conducted at 15,000 lbs and 25,000 lbs and the results presented in this report were backcalculated from the 25,000 lbs load.

Pavement Layer Moduli

The pavement layers for this project consisted of approximately 10-in. asphaltic concrete (AC), 8-in. continuously reinforced concrete pavement (CRCP), 8-in. soil cement base course, and subgrade. Moduli were determined for the 10-in. AC layer, 8-in. CRCP layer, and composite 8-in. soil cement /subgrade layer. With the issues occurring on this project, namely in the AC, only that layer will be elaborated on. The moduli values for all layers can be found in Appendix B.

AC Modulus

The AC modulus was backcalculated to determine if it met or exceeded national guidelines for good AC pavement (450 ksi) as well as to determine if the differences existed between travel lanes *[12-13]*. A statistical analysis was conducted comparing the travel lanes in both the east and west bound directions using Tukey's method *[11]*.

Structural Number

The structural number (SN) is a dimensionless number that indicates the strength of a pavement structure as well as its ability to sustain the design traffic loading *[5-6]*. The term “structural number” is typically used in two ways in transportation engineering: The structural number required to endure future projected traffic loadings (SN_f) and the in-place structural number (SN_{eff}) of a pavement structure. The SN_f is determined through a DARwin analysis and the SN_{eff} is determined from a device such as an FWD *[5-6]*.

Darwin analysis was conducted for this project, which is presented in Appendix C. Because it was an overlay analysis, structural support credit was given to the CRCP. The results from the analysis indicated that the total AC thickness should be 12.59 in. to handle the projected traffic loadings. Since credit was given to the CRCP, only 8 in. of AC was required.

The SN_f can be inferred by multiplying 12.59 in. AC * 0.44 SN/in., which equates to 5.53. In turn, this was compared to measured SN_{eff} values obtained from FWD readings to determine if the existing pavement had enough structure to carry the projected traffic loadings.

The data obtained from cores on this project indicated that approximately 3-in. of AC was stripped. Stripped AC is similar to stone base course, which has a layer coefficient of 0.14. So the difference in structural number between AC and stripped AC would be approximately 3* (0.44-0.14) = 0.9 SN. Differences in SNeff values between the travel lanes can imply that stripping has weakened certain locations more than others.

RESULTS AND DISCUSSION

Field Evaluation

During the recent coring operation, it was noticed that the permanent deformation was only occurring at the wearing course or SMA lift. However, moisture damage was noticed at a couple of locations where the corresponding cores fell apart completely (Figure 6). The moisture damage was mainly noticed in the binder/ leveling course lifts and the mixtures of those layers seemed to be soft and lacking cohesion. It should be noted that the soil cement base underneath the CRCP is generally highly impermeable to water. It is anticipated that the water or water vapor was getting into the pavement structural system from underneath and finding no place to escape due to an inadequate subsurface drainage conditions. In addition, the differential thermal expansion of water vapor during day-night cycle and the cyclic stresses from moving traffic increases the pore water pressure which may rupture the asphalt-aggregate bond and cause stripping *[3]*.

Figure 6
Observation of moisture damage during coring

In an effort to measure the extent of premature rutting, the LTRC pavement research group collected rut measurements at every 0.10 mile interval for the entire length of the project. As shown in Figure 7, it is clear that the outside lanes in both directions rutted more than their inside counterparts. The highest rut depths of 0.61 inch and 0.41 inch were noticed in the outside lanes in the eastbound and westbound directions respectively. The outside lanes usually experience more and heavier traffic which may be the possible reason behind the higher rutting results. Interestingly, the scenario of having higher rutting in the outside lanes also supports the underneath moisture damage phenomenon. The roadway was built with a slope towards the outside lane, which may allow the moisture to flow in that direction and eventually deposit under those lanes.

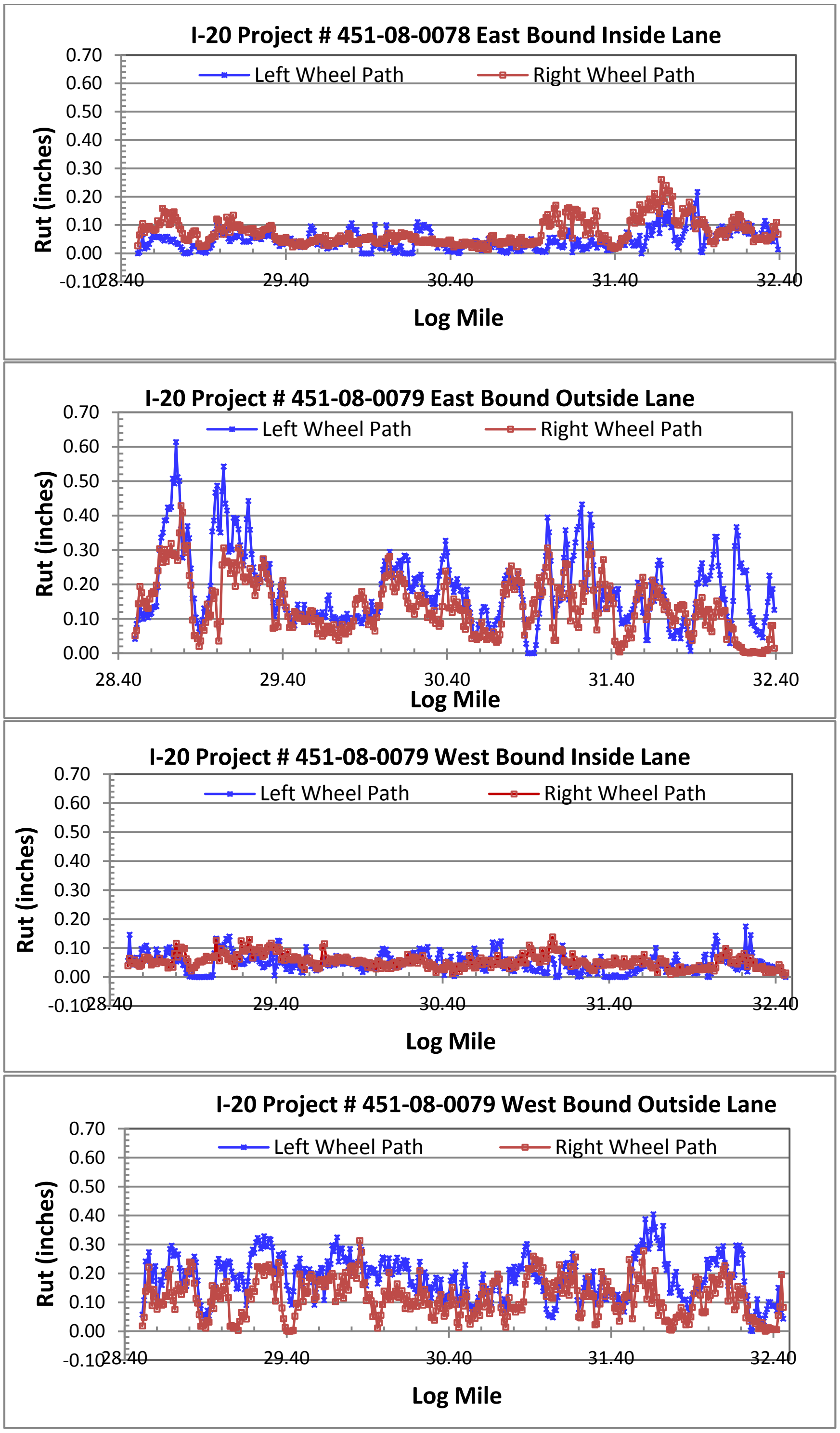


Figure 7
Rut measurements of I-20 sections

The IRI values as presented in Figures 8a to 8d indicate that, for the most part, the project is smooth (IRI < 80) according to the FHWA criteria. The east bound inside lane (EBIL) had IRI values generally less than 50 except for the locations near the weigh-in-motion (WIM) station at CSLM 31as presented in Figure 8a. The east bound outside lane (EBOL) had a similar trend to the EBIL except for the first mile of the project, which had IRI values up to 150 as presented in Figure 8b. The west bound inside lane (WBIL) had IRI values around 40 indicating a very smooth ride as presented in Figure 8c. The west bound outside lane (WBOL) had a different trend than the WBIL with more variation in IRI values and notably higher IRI values at CSLM 29 to 29.1 and CSLM 31 to 32.4.

In both the east and west bound directions, the outside lanes had rougher trends than inside lanes, though, generally, the IRI values were within FHWA's criteria for a smooth roadway.

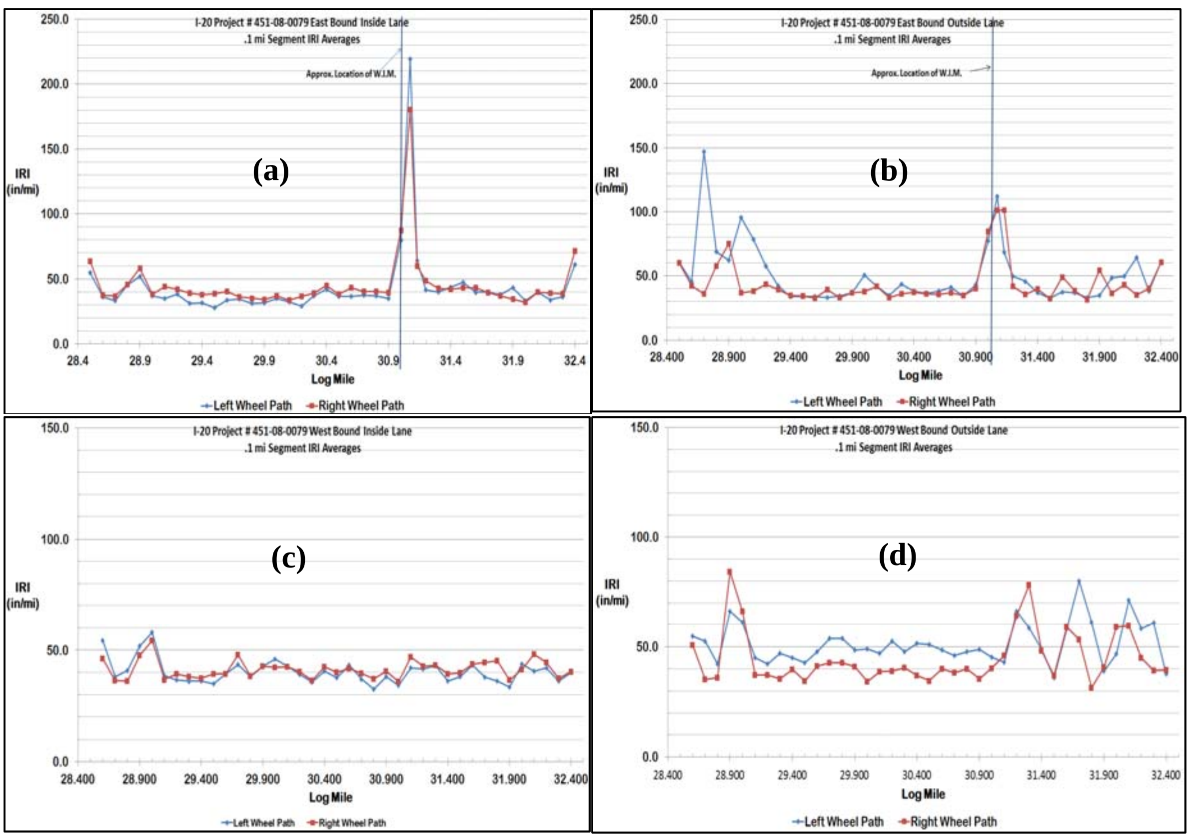


Figure 8
IRI data

The asphalt mixture (AC) moduli values for the west bound outside (WBOL) are presented in Figures 9 to 10. Outliers, values which were far out of range, were not removed. Even though the average AC modulus was 534 ksi, which was greater than the 450 ksi benchmark, many test locations were below the 450 ksi benchmark as presented in Figure 9. Approximately 45 percent of the AC modulus values were less than 450 ksi as presented in

Figure 10. That could indicate that stiffness of the AC layer was significantly impacted by stripping at those locations.

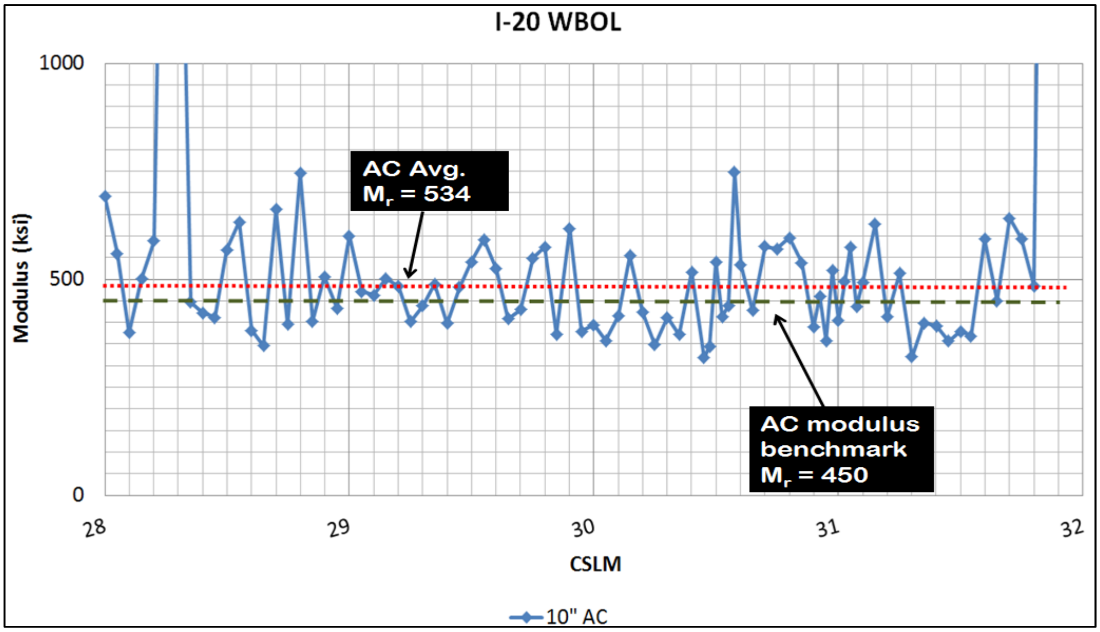


Figure 9
Westbound outside lane

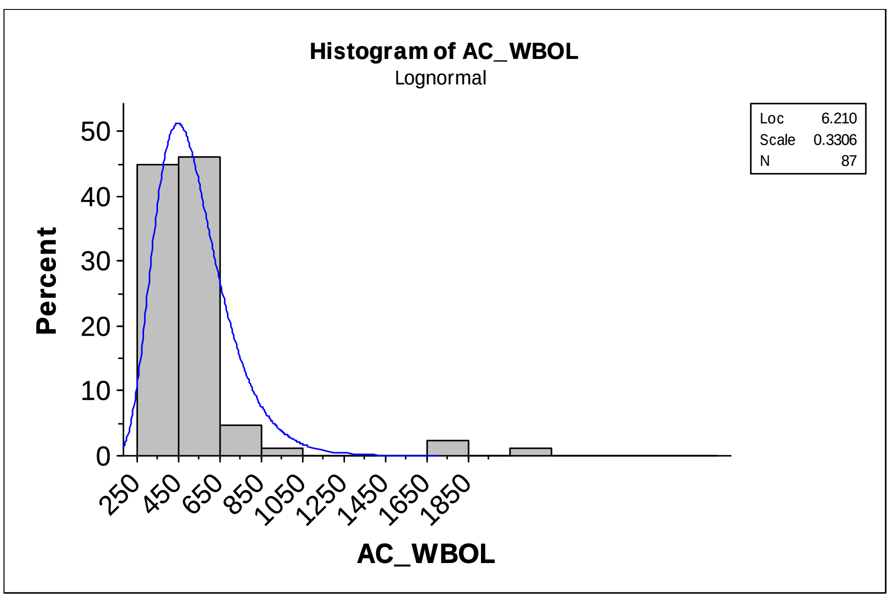


Figure 10
Westbound outside lane histogram

Figures 11 and 12 present the AC modulus results for the WBIL. The average AC modulus was 672 ksi as presented in Figure 11. Unlike the WBOL, only a few test locations were below the 450 ksi benchmark. The histogram for the WBIL indicated that approximately 15 percent were below the 450 ksi benchmark as presented in Figure 12.

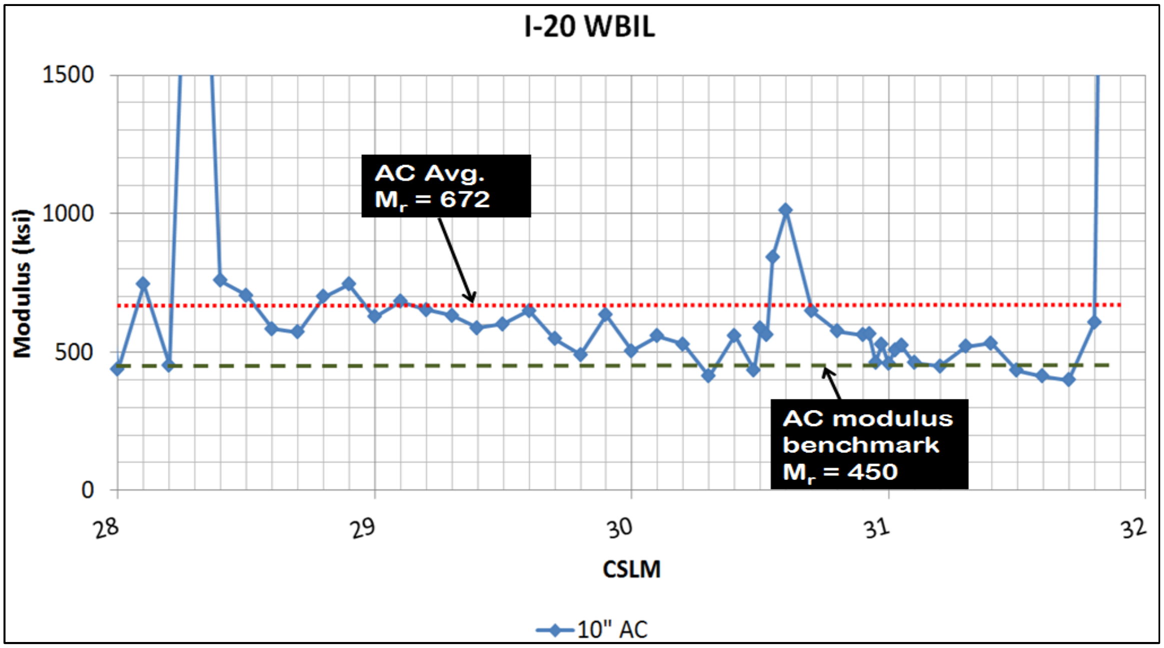


Figure 11
Westbound inside lane AC modulus

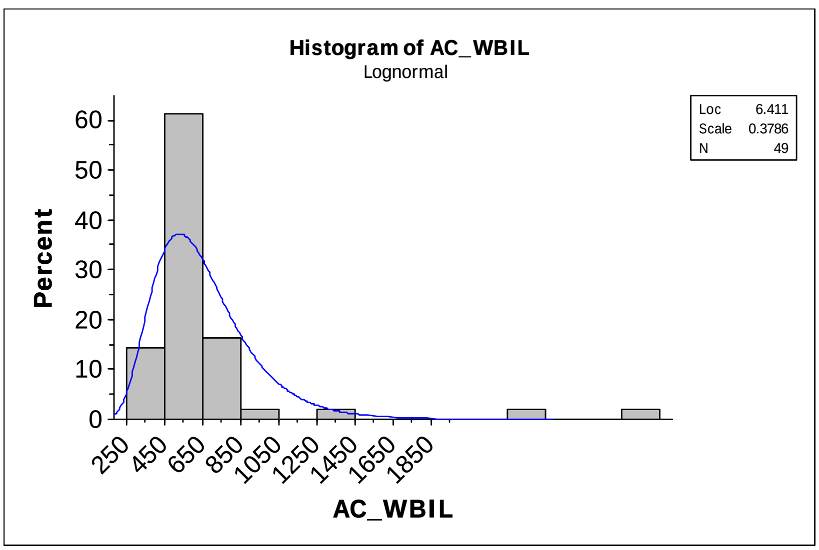


Figure 12
Westbound inside lane histogram

The AC moduli values for the east bound outside lane (EBOL) are presented in Figures 13 to 14. Outliers, values which were far out of range, were not removed. The results indicated that the average M_r value was 641 ksi with some points below the 450 ksi benchmark as presented in Figure 13. The histogram of the AC modulus data indicated that approximately 9 percent of the points were below the 450 ksi benchmark as presented in Figure 14. This implies that a significant portion (9 percent) of the pavement was impacted by the stripping.

Figures 15 to 16 present the AC modulus values for the east bound inside lane (EBIL). As presented in Figure 15, the average AC modulus is 611 ksi with some points below the 450 ksi benchmark. The histogram for the AC modulus data indicated that approximately 18 percent of the points were below the 450 ksi benchmark as presented in Figure 16. This implies that a significant portion (18 percent) of the pavement was impacted by the stripping.

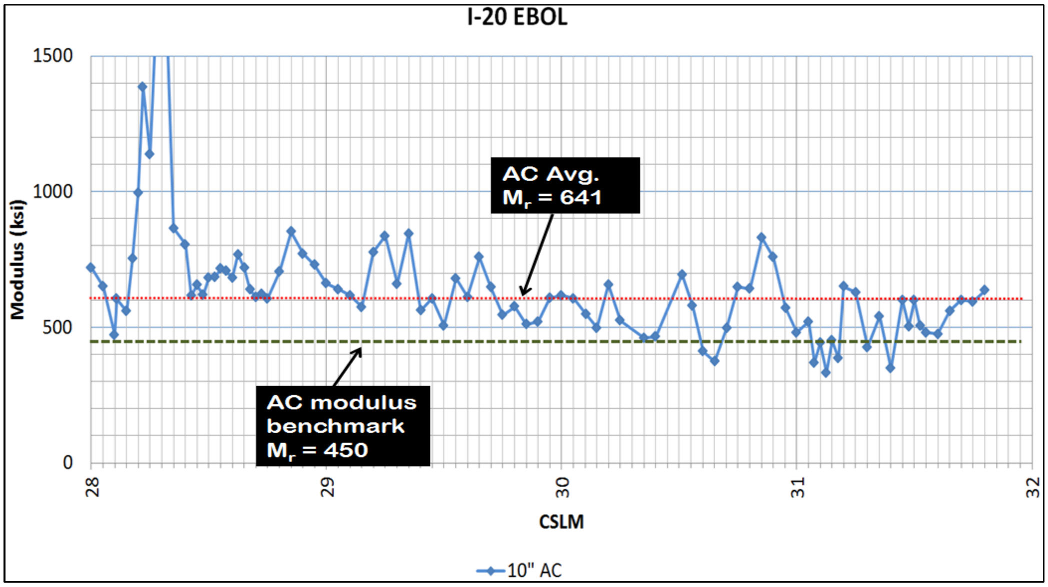


Figure 13
Eastbound outside lane AC modulus

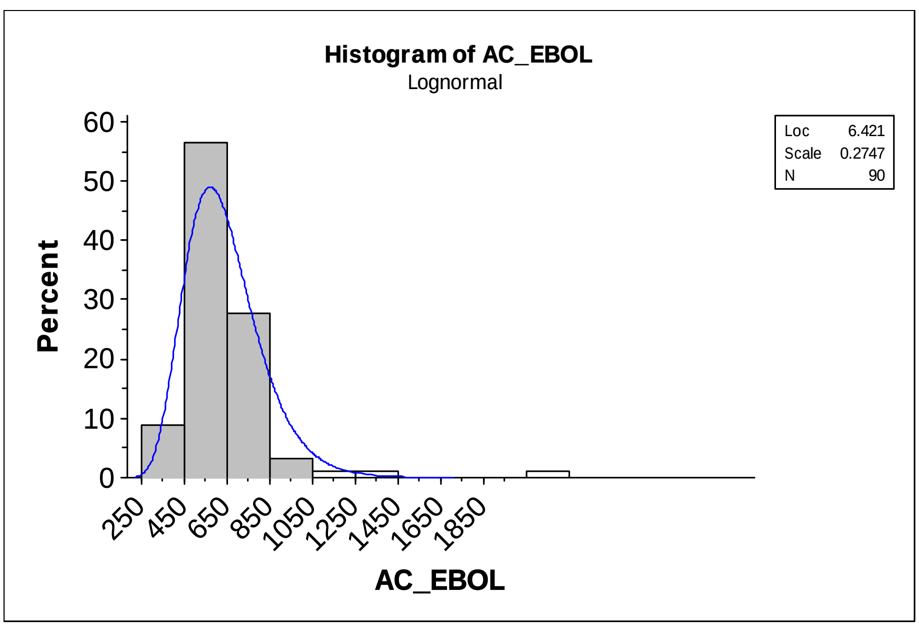


Figure 14
Eastbound outside lane histogram

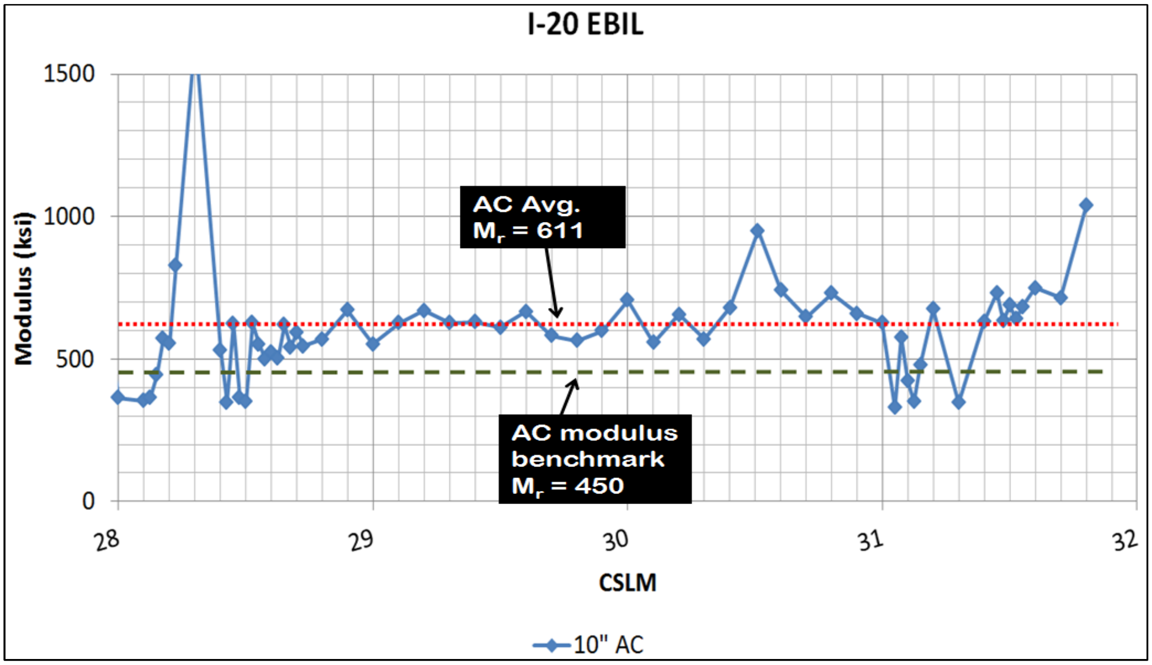


Figure 15
Eastbound inside lane AC modulus

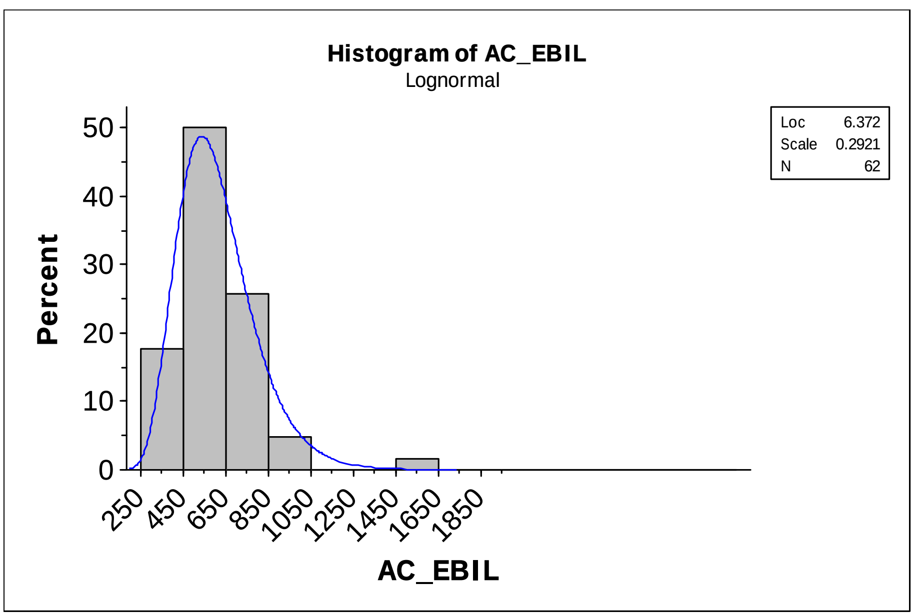


Figure 16
Eastbound inside lane histogram

Statistical Analysis and Summary

A statistical analysis was conducted to compare the results from the WBOL, WBIL, EBOL, and EBIL. As shown in Table 3, the results indicated that the WBIL, EBOL, and EBIL were statistically similar with the EBOL and EBIL sharing a statistical similarity with the WBOL.

The WBIL and WBOL are statistically different. The fact that the EBOL and EBIL are in the same groups with the WBOL and WBIL makes it difficult to discern which lanes are the most distressed. However, a review of the histograms (Figures 10, 12, 14, and 16), indicate that all lanes have been impacted by the stripping with the WBOL showing the largest distress.

Table 3
Statistical analysis of AC modulus

Direction_Lane	Mean / Stdev. (ksi)	Statistical Grouping[1]
WBIL	671.8 / 441.2	A
EBOL	641.3 / 227.2	AB
EBIL	611.1 / 199.1	AB
WBOL	534.3 / 280.1	B

[1] Similar letters indicate statistical similarity while different letters indicate statistical differences.

Structural Number

The SN_{eff} values for the west bound outside lane (WBOL) are presented in Figures 17 and 18. The results indicated that the average SN_{eff} was 5.14 and that only a few test locations were near the 5.53 benchmark as presented in Figure 17. The histogram of the SN_{eff} data indicated that all points were below the 5.53 SN_f benchmark as presented in Figure 18. The difference between the average SN_{eff} (5.14) and the benchmark SN_f (5.53) is -0.39, which equates to approximately to 1-in. of AC.

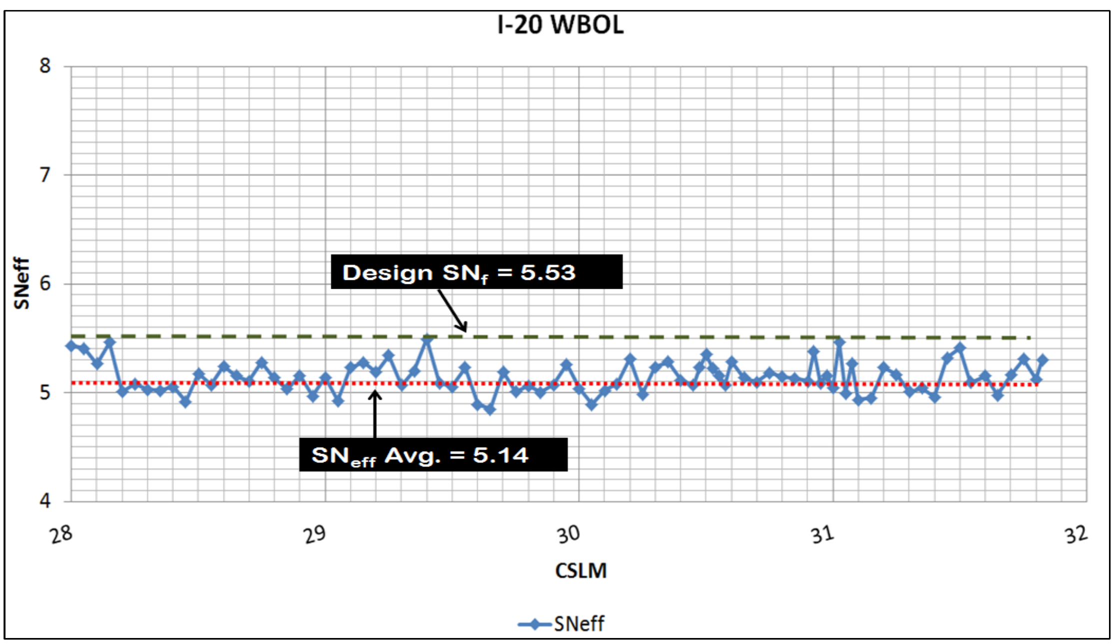


Figure 17
Westbound outside lane SN_{eff}

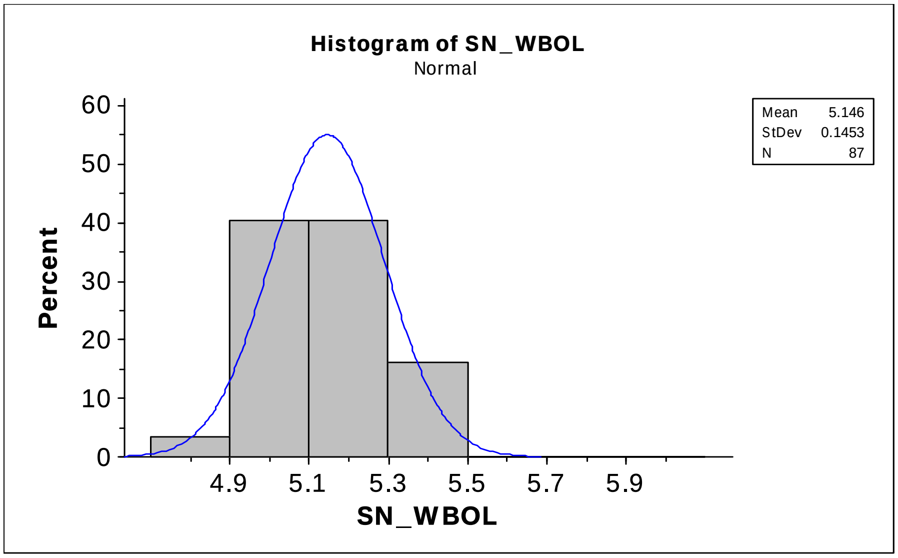


Figure 18
Westbound outside lane SN_{eff} histogram

The SN_{eff} values for the west bound inside lane (WBIL) are presented in Figures 19 and 20. The results indicated that the average SN_{eff} was 5.38 and that only a few test locations were above the 5.53 benchmark as presented in Figure 19. The histogram of the SN_{eff} data indicated that approximately 86 percent of the test locations were below the 5.53 SN_f benchmark as presented in Figure 20. The difference between the average SN_{eff} (5.38) and the benchmark SN_f (5.53) is -0.15, which equates to approximately 0.34-in. of AC.

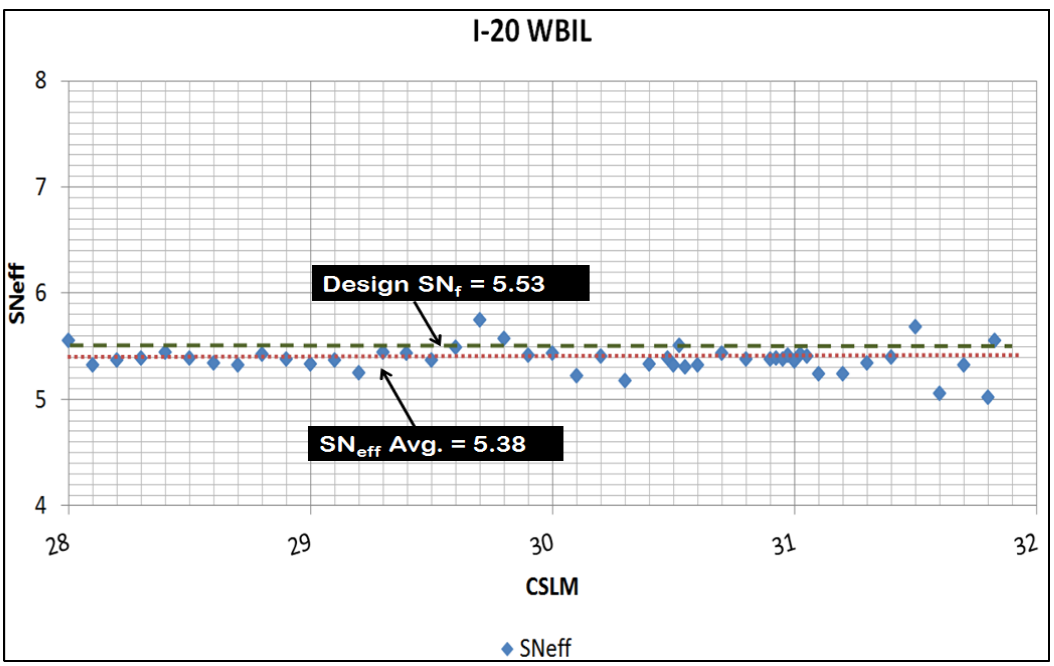


Figure 19
Westbound inside lane SN_{eff}

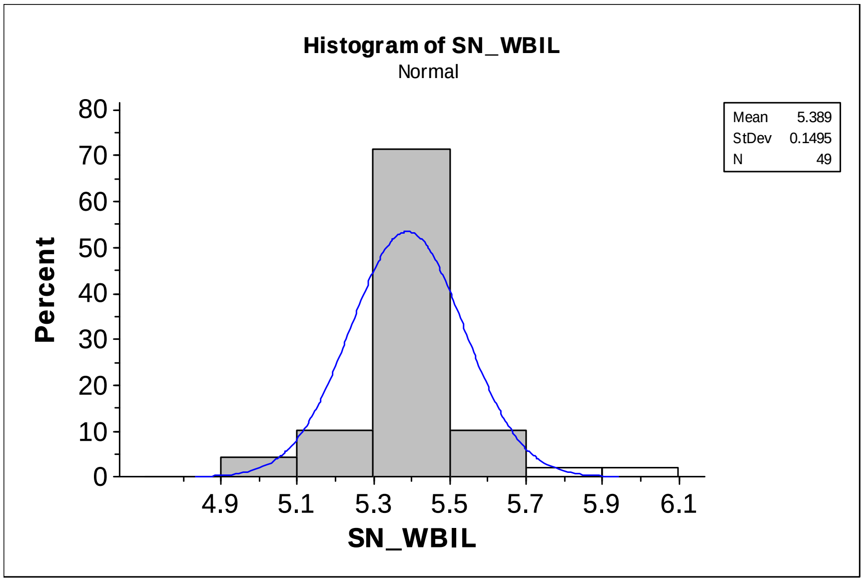


Figure 20
Westbound inside lane SN_{eff} histogram

The SN_{eff} values for the east bound outside lane (EBOL) are presented in Figures 21 and 22. The results indicated that the average SN_{eff} was 5.46 and that many test locations were near the 5.53 benchmark as presented in Figure 21. The histogram of the SN_{eff} data indicated that approximately 58 percent of the test locations were below the 5.53 SN_f benchmark as presented in Figure 22. The difference between the average SN_{eff} (5.46) and the benchmark SN_f (5.53) is -0.07, which equates to approximately 0.15-in. of AC.

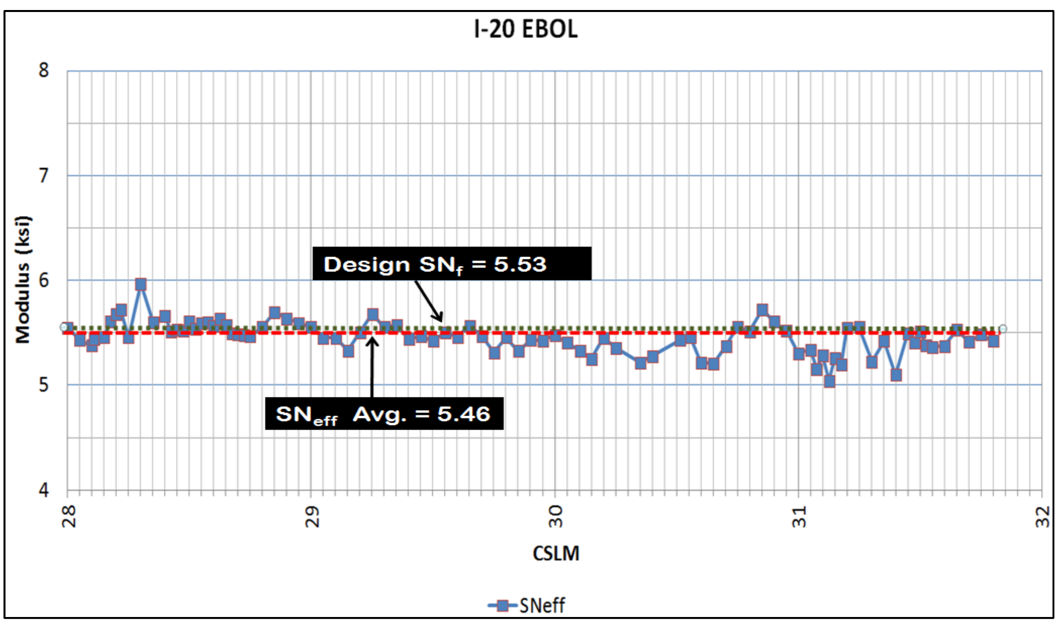


Figure 21
Eastbound outside lane SN_{eff}

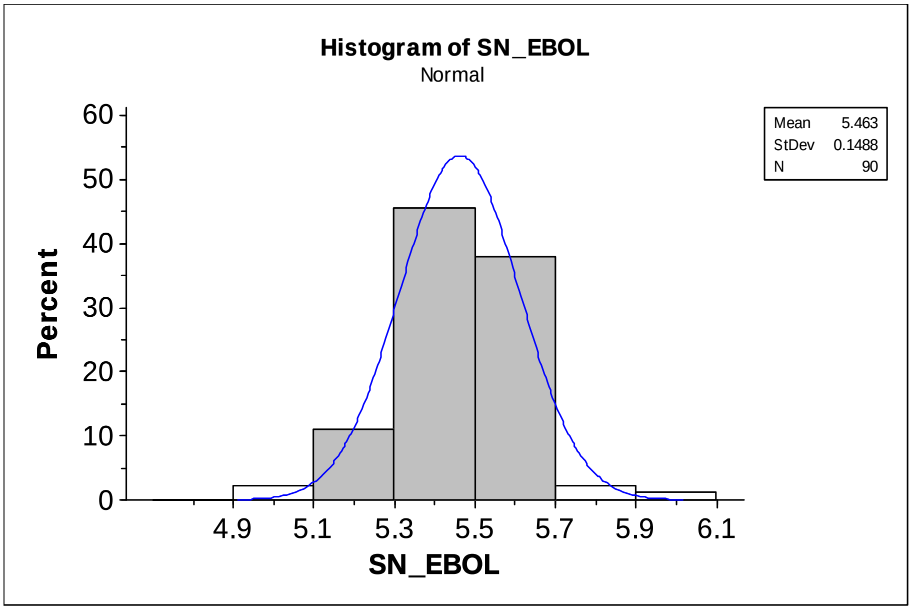


Figure 22
Eastbound outside lane SN_{eff} histogram

The SN_{eff} values for the east bound inside lane (EBIL) are presented in Figures 23 and 24. The results indicated that the average SN_{eff} was 5.62 and that many test locations were above the 5.53 benchmark as presented in Figure 23. The histogram of the SN_{eff} data indicated that approximately 18 percent of the test locations were below the 5.53 SN_f benchmark as presented in Figure 24. The difference between the average SN_{eff} (5.62) and the benchmark SN_f (5.53) is +0.09, which equates to approximately 0.2-in. of AC.

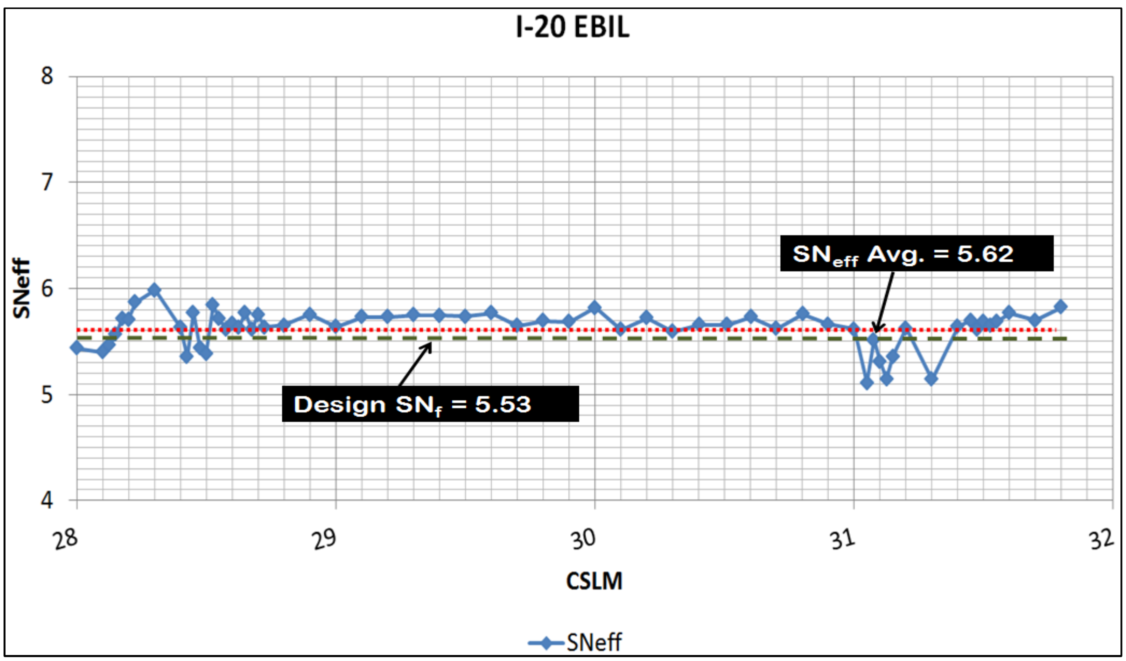


Figure 23
Eastbound inside lane SN_{eff}

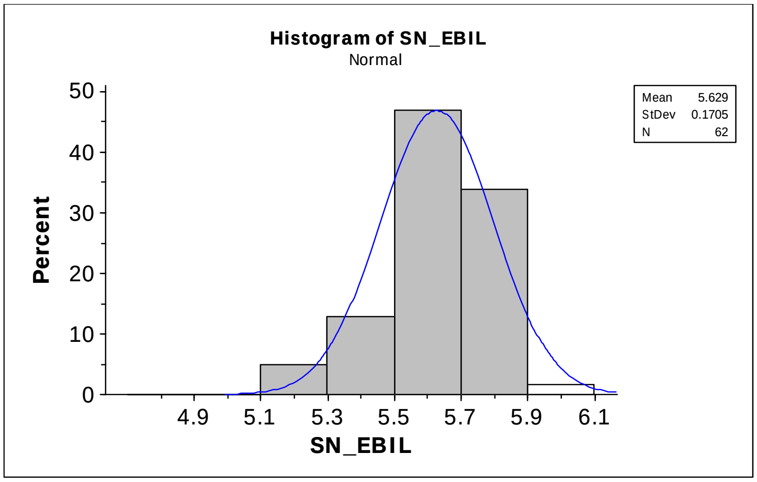


Figure 24
Eastbound inside lane SN_{eff} histogram

Statistical Analysis and Summary

A statistical analysis was conducted to compare the results from the WBOL, WBIL, EBOL, and EBIL. The results shown in Table 4 indicated that a statistical difference existed between all groups. According to the Darwin analysis, an SN_f of 5.53 is required to handle the projected loadings. The WBOL has an SN deficiency of approximately 0.39, which is equivalent to about 1-in. of AC pavement. The remaining travel lanes, appear to have adequate SN_{eff} values when compared to the required SN_f of 5.53.

Table 4
Statistical analysis of SN_{eff}

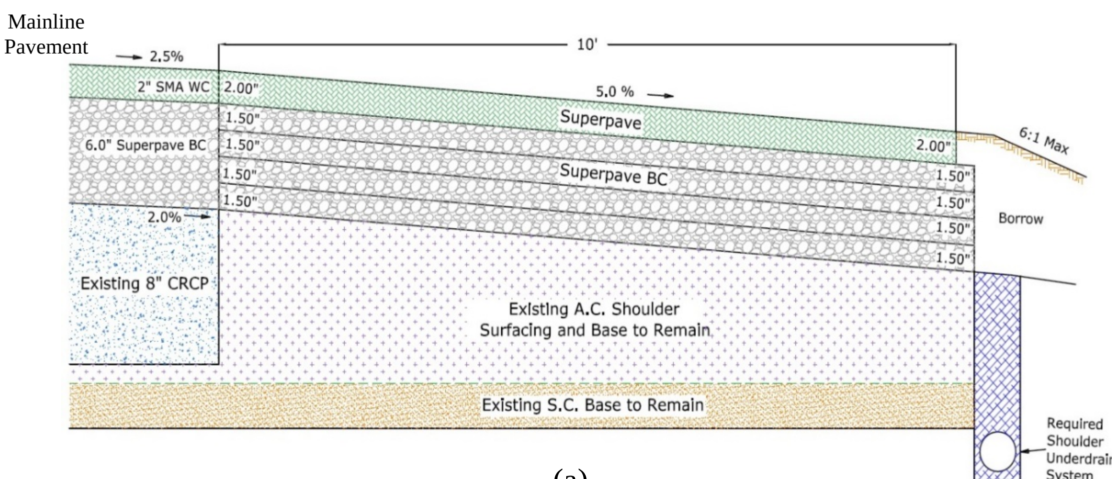
Direction_Lane	SN_{eff} Mean / Stdev. (ksi)	Statistical Grouping[1]
EBIL	5.62 / 0.1705	A
EBOL	5.46 / 0.1488	B
WBIL	5.38 / 0.1495	C
WBOL	5.14 / 0.1453	D

[1]Similar letters indicate statistical similarity while different letters indicate statistical differences.

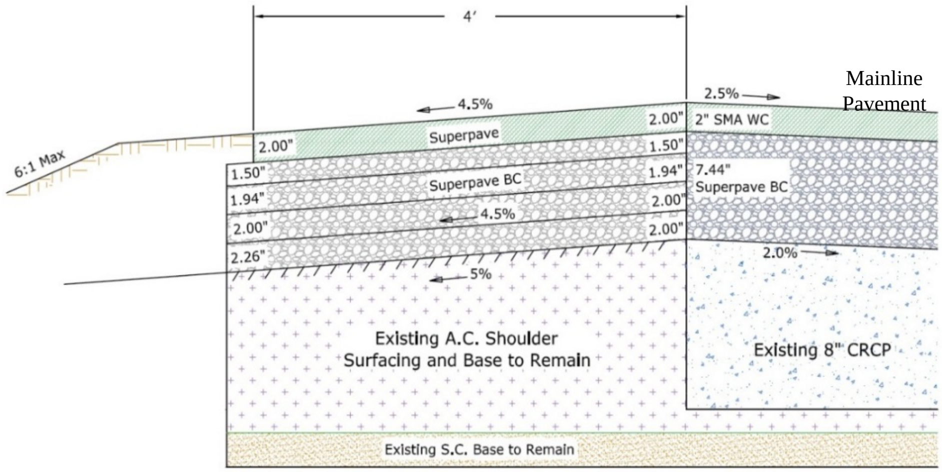
Design Issues

Subsurface Drainage Issues

Moisture damage in pavement structures depends on material properties, design factors, and subdrainage capabilities. Excess free water in PCC pavement can cause erosion of the subbase which may essentially create a “Bath-tub” effect without drainage underneath the pavement. Before being overlaid, the PCC pavement structural system is assumed to lose moisture by evaporation through the cracks, joints, and disintegrated portion of the pavement *[3]*. After the overlay, severe stripping may be observed in the newly laid layers due to the entrapped water and moisture vapor that stays there for a long period. In this type of scenario, a proper subsurface drainage system with a lateral outlet is necessary for successful removal of the water and moisture from the base and subgrade layers. Figure 25 shows the existing under/subsurface edge drainage system for this project. It was evident during the inspection survey by LTRC research engineers that the existing drainage system was not placed at the proper location to drain water and moisture effectively from underneath the pavement structure. The edge drain was found to be placed at the outside edge of the shoulder rather than at the edge of the mainline pavement. Therefore, the entrapped water and moisture under the mainline PCC pavement had nowhere to escape and eventually attributed to create the bath-tub underneath the pavement as mentioned above. To rectify this problem, a subsurface edge drainage system needs to be placed against the outside edge of the original concrete pavement, as shown in Figure 26. The depth of the underdrain system should be placed deep enough to go through the treated base course. LTRC personnel have experienced these type failures before, where the presence of moisture in the pavement structure appears to be a major contributor to the premature rutting. One such project was Interstate 10 in West Baton Rouge Parish. Forensic evaluations were conducted on mixture problems as reported in a technical assistance report for state project no. 450-07-0031 and 450-08-0022 *[14]*. Subsequently, edge drains were properly installed on this project as a mitigation after the pavement failed.



(a)



(b)

Figure 25

Existing shoulder cross-section: (a) outside shoulder, (b) inside shoulder

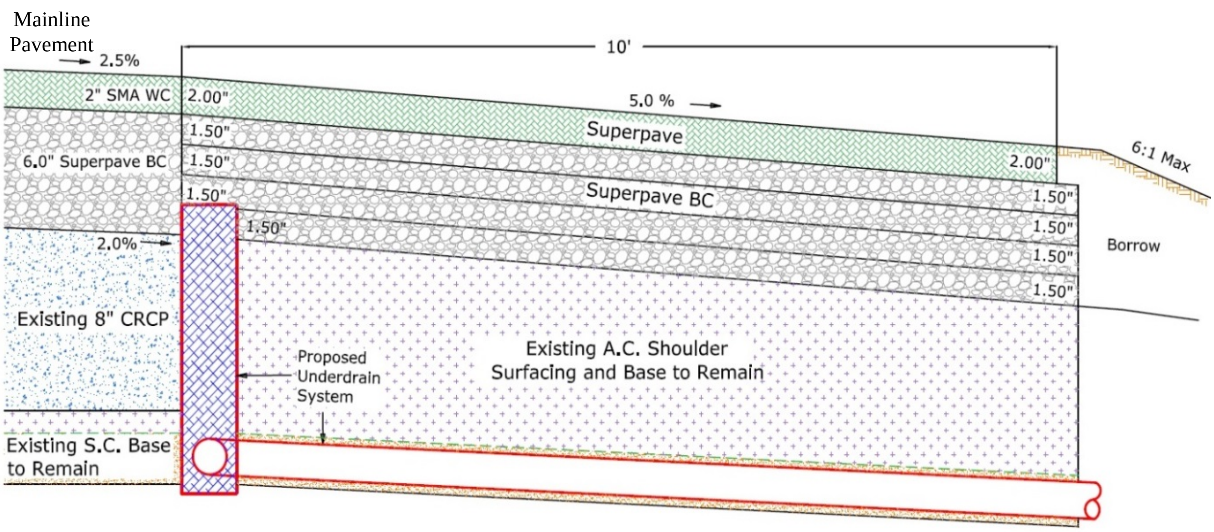


Figure 26

New proposed cross sections showing edge drain

Lift Thickness Issues

It was observed from LADOTD roadway reports that the shoulder and mainline pavements were constructed at the same time. From the typical cross-sections of inside and outside shoulders presented in Figure 25, it can be seen that the shoulders were constructed in different layers. As a regular construction practice, it is anticipated that the same layer thicknesses were maintained also for mainline roadway construction to avoid drop off and facilitate safe traffic movement during construction. However, it should also be noted that the lift thickness of asphaltic pavements is influenced by the nominal maximum aggregate size (NMAS). Generally, the minimum lift thickness is at least 2.5 to 3 times the NMAS to ensure the proper alignment of the aggregates during compaction. According to the current LADOTD superpave specifications, the lift thickness of a level 2 binder course mixture with NMAS size of 1.0 inch (25 mm) which was used in this project has to be within 2.5 to 4.0 in. Interestingly, all the layers shown in Figure 25 had a thickness of 2-in. or less. This clearly indicates that the proper construction practice was not followed during the construction of this job.

Laboratory Performance Evaluation

Aggregate Gradations and Asphalt Content

Figure 27 shows the percent asphalt content obtained from various cores and the corresponding Sublots respectively. Except for a couple of the specimens (Core 6: Lift D and Core 2L: Lift C), the percent asphalt for all roadway samples were within the validated JMF tolerance limits. Even though LTRC asphalt lab computed the extracted aggregate gradations and percent asphalt content for leveling course mixtures (Lift A and Lift B), a comparison between plant production data and roadway samples could not be performed due to the unavailability of plant production data at this time.

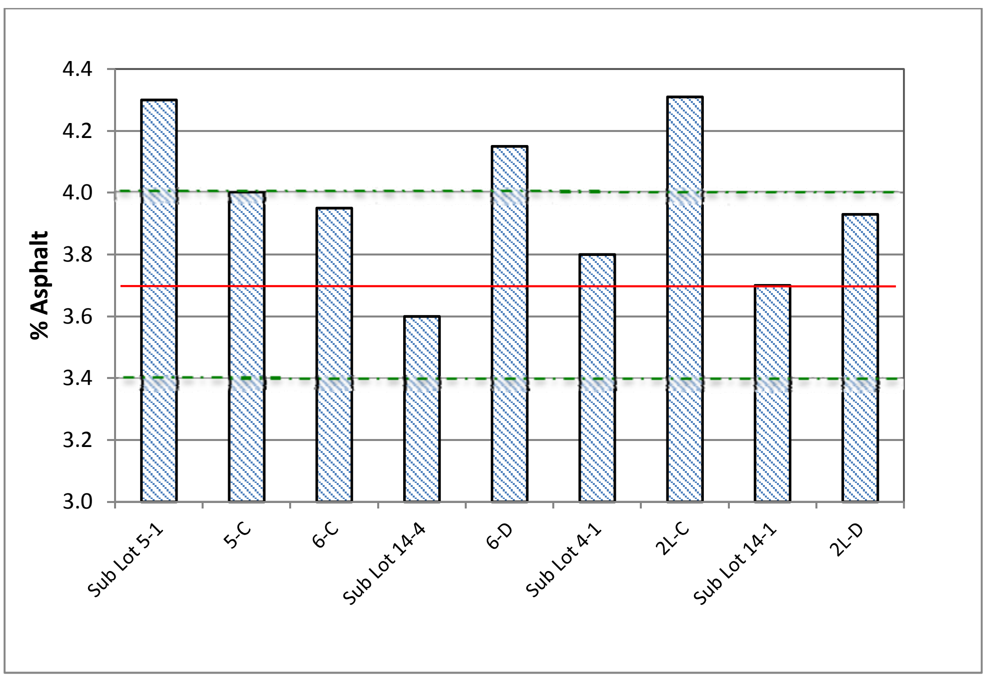
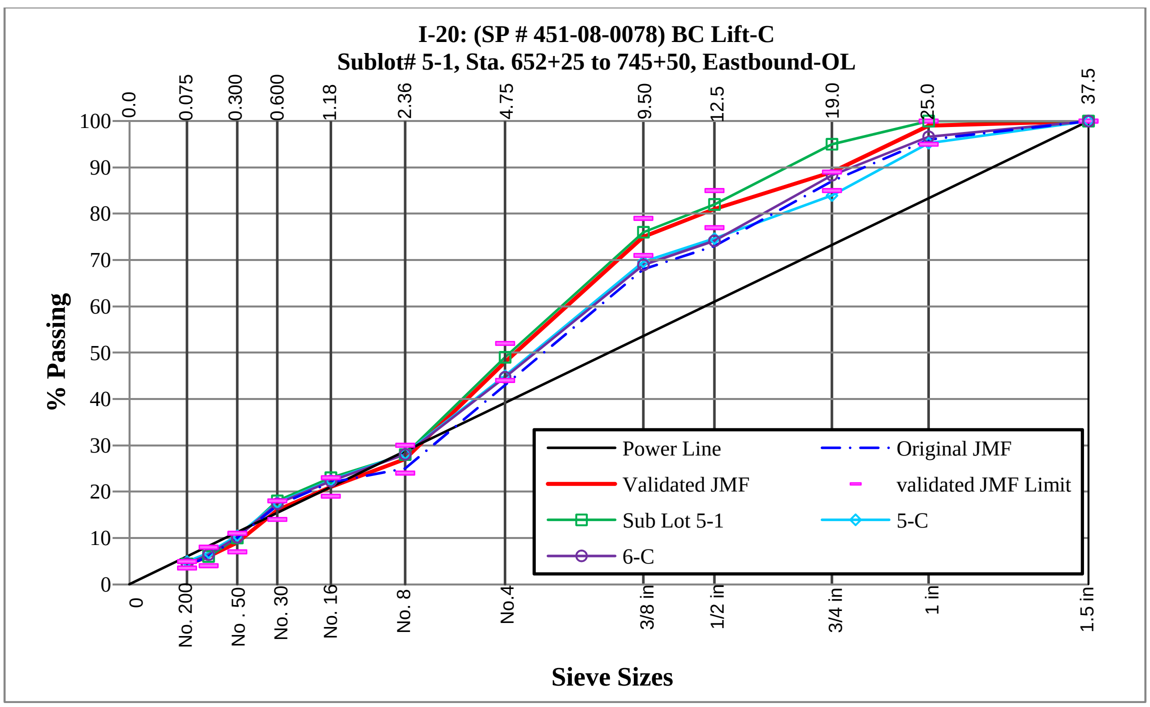


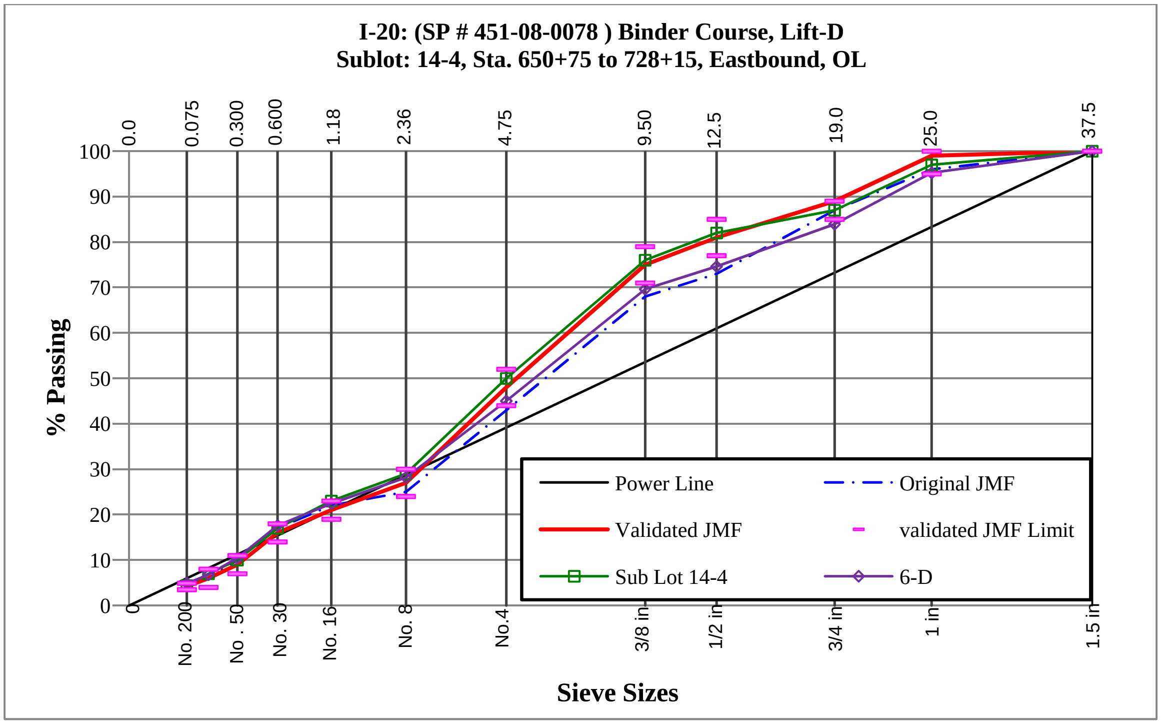
Figure 27
Comparisons of asphalt content

Blended aggregate gradations and percent asphalt contents were obtained from the ignition oven method and compared to the validated JMF and the current LADOTD specification tolerances. In doing so, two centerline samples, Core 5 and Core 6, and one wheel path sample, Core 2L, were separated into different lifts and used for burn gradations. It should be noted that Core 5 and Core 6 were collected from station 664+63 to station 664+79 area (Approximate Logmile: 29.17) whereas, Core 2L was collected from station 669+10. The plant records indicate that Lift C for Core 5 and Core 6 represent mix from Sublot 5-1, Lift D for Core 5 and Core 6 represent mix from Sublot 14-4, Lift C for Core 2L represent mix from Sublot 4-1, and Lift D for Core 2L represent mix from Sublot 14-1, respectively.

Figures 28 and 29 show a comparison of blended aggregate gradations during production of Sublot 5-1, Sublot 14-4, Sublot 4-1, Sublot 14-1, and the gradations found from the corresponding roadway core lifts. It is evident that the extracted aggregate gradations of roadway cores did not always match the gradations reported in the plant data. In most cases, the aggregate structures from roadway cores were found to be slightly coarser especially on 3/8 inch and 1/2 inch sieves. However, except for Lift C cores 5 and 6, the extracted gradations are very comparable to the gradations reported during production.



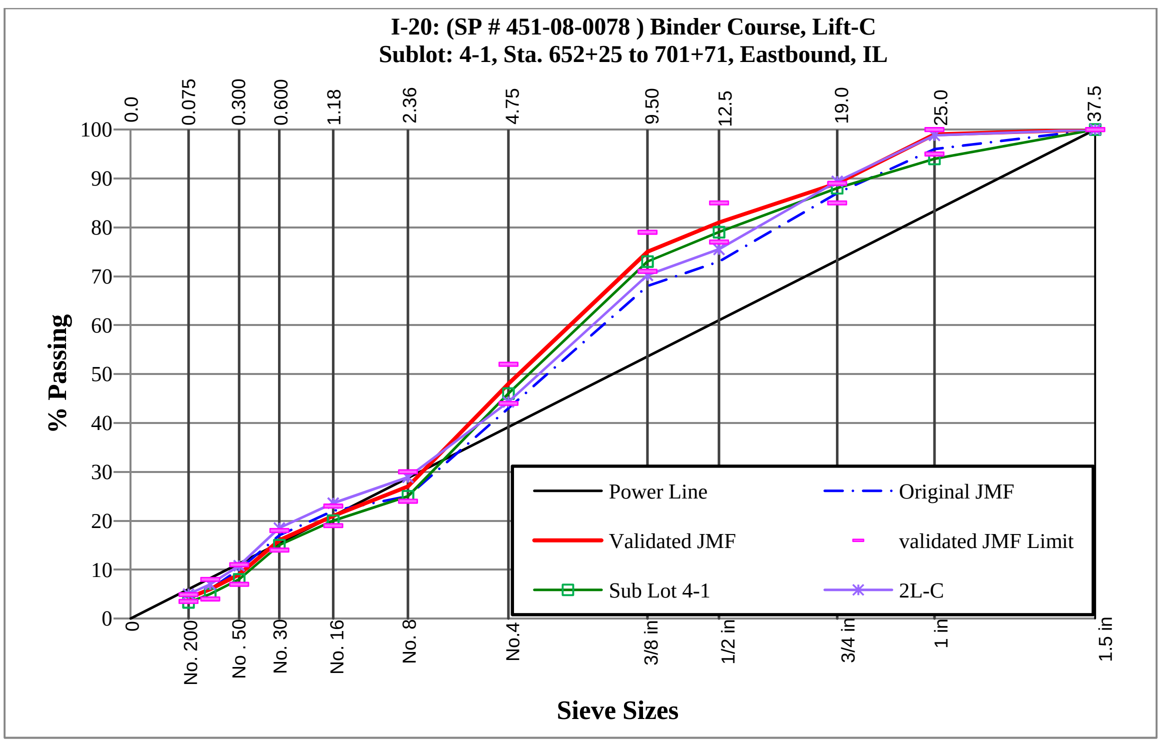
(a)



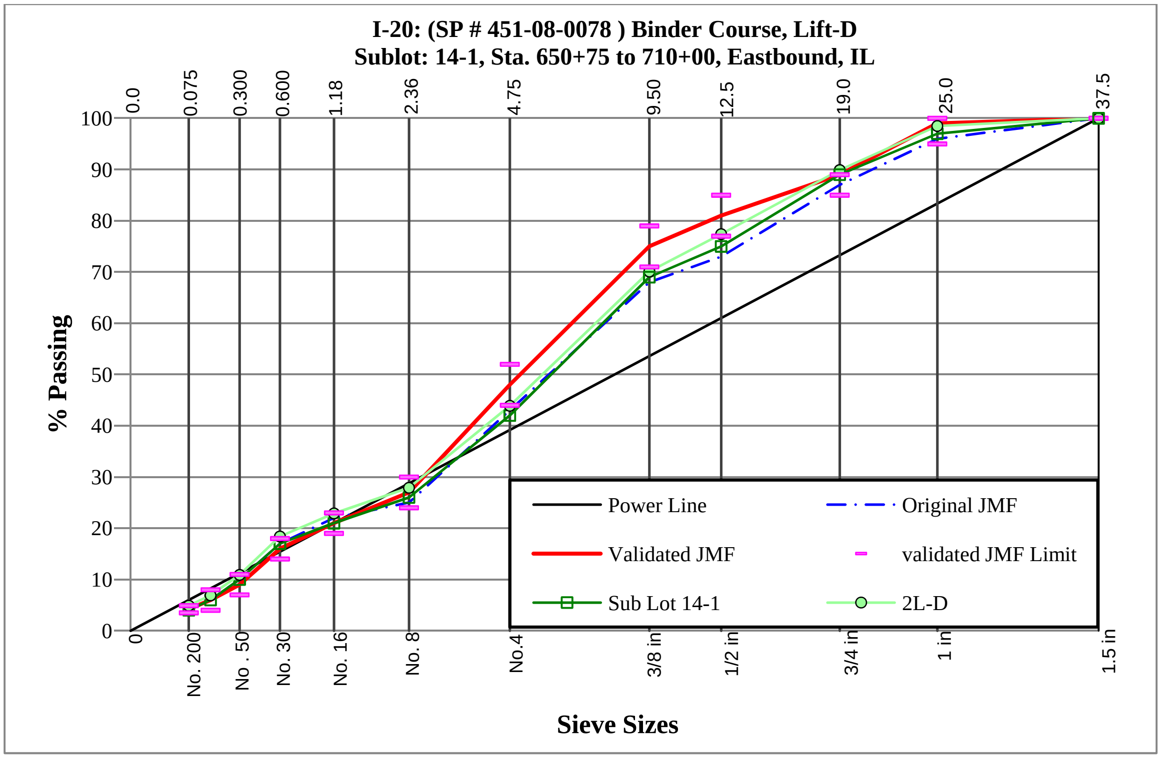
(b)

Figure 28

Comparisons of blended aggregate gradations for sublots 5-1 and 14-4



(a)



(b)

Figure 29

Comparisons of blended aggregate gradations for sublots 4-1 and 14-1

Loaded Wheel Test

To evaluate the rutting potential and moisture susceptibility of Superpave mixtures used in this project, roadway cores were evaluated with LWT (Hamburg) tests in accordance with the AASHTO T 324 test method. In this test, a steel wheel is rolled across the surface of an asphaltic specimen submerged at 50°C water and the deformation is presented as a function of the number of wheel passes. Each steel wheel passes for 20,000 cycles or until 20 mm deformation, whichever is reached first. An abrupt change in the rate of deformation indicates the stripping of asphalt binder from the aggregate in the asphalt mixture specimen. As shown in Figure 30, there are four different parts in a LWT test result plot: post-compaction, creep slope, stripping slope, and stripping inflection point. The creep slope is used to measure the rutting susceptibility, whereas the stripping slope and the stripping inflection points measure the moisture damage potential. Stripping inflection point, assumed to be the initiating point of moisture failure, is the number of wheel passes at the intersection of creep slope and stripping slope.

Results presented in Figure 31 clearly indicate that all roadway cores used in LWT test failed miserably and did not survive the whole 20,000 wheel passes. Regardless of the location of the cores (i.e. wheel path or centerline), none meet the current LADOTD specifications requirement of 6.0 mm rut depth after 20,000 passes for level 2 Superpave BC mixtures. The samples also showed very high stripping potential. However, the cores taken from inside lane showed slightly better performance than the cores from outside lane. Except Lift C samples, all samples from the inside lane showed stripping inflection points around 8000 cycles whereas, the samples from Lift C and Lift D from outside lane showed stripping inflection points at 6200 and 4100 cycles respectively. Note that Colorado DOT reported that a stripping inflection point below 10,000 cycles indicated moisture susceptibility potential of that mix *[13]*.

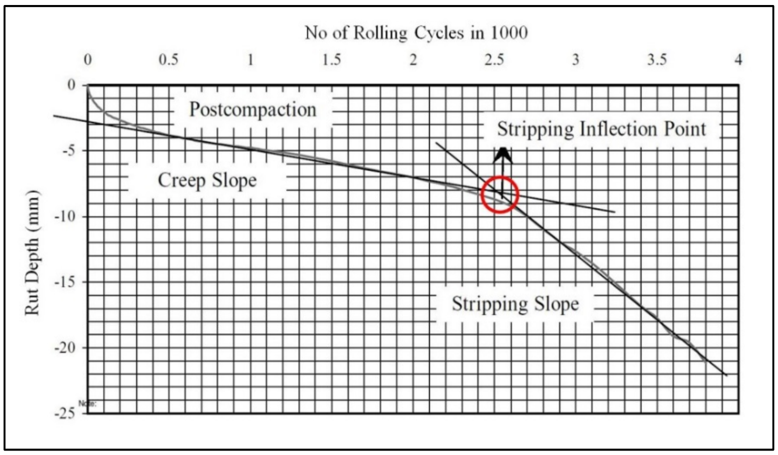
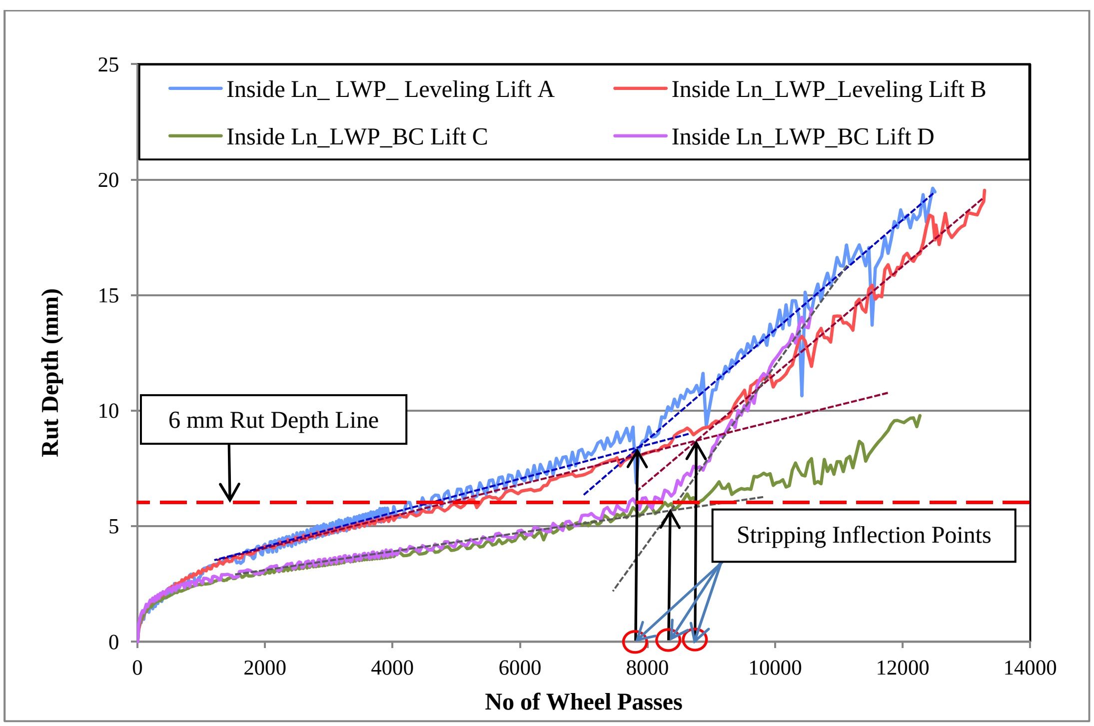


Figure 30
Typical LWT graph



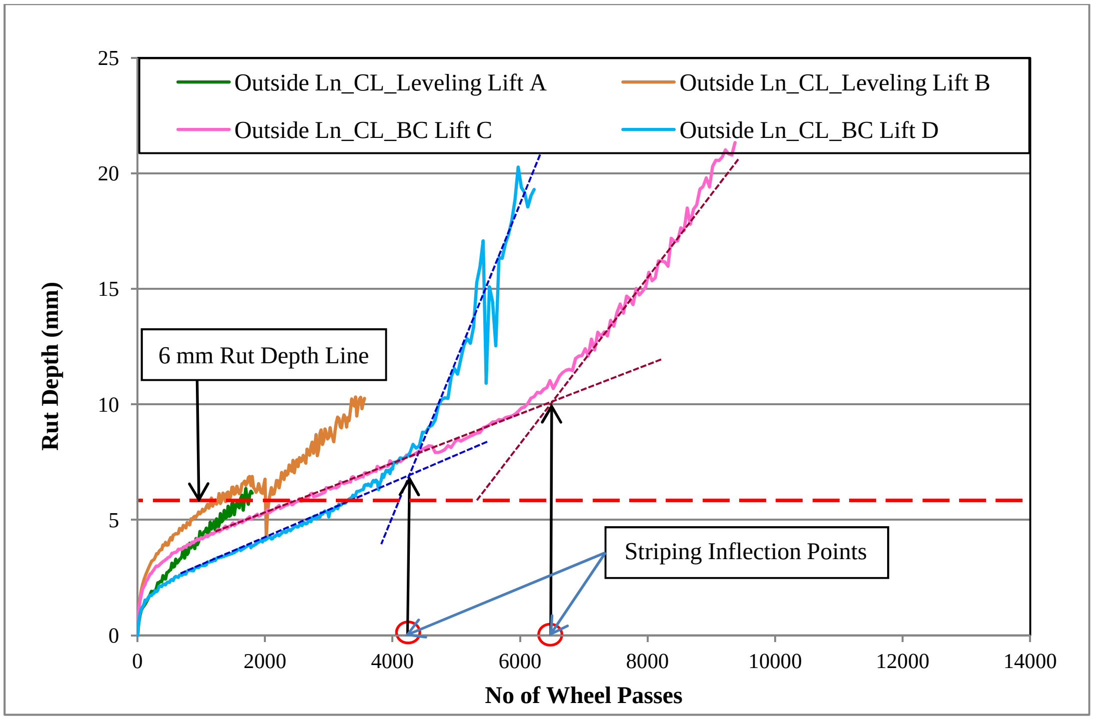


Figure 31
LWT test results

The graphs in Figure 31 also indicate that the lower the lift of the pavement the more moisture damage it showed. Interestingly, this also supports the underneath moisture damage phenomenon, as Lift A has the maximum potential to be exposed to the moisture. The higher traffic on the outside lane perhaps developed a pumping action of moisture in the lower lifts, which might be the reason behind the moisture damage revealed in LWT test results. The damaged samples were further investigated once the LWT test was completed. From visual inspection, the cores appeared to be completely stripped and fine sand appeared to be leaching as shown in Figure 32.

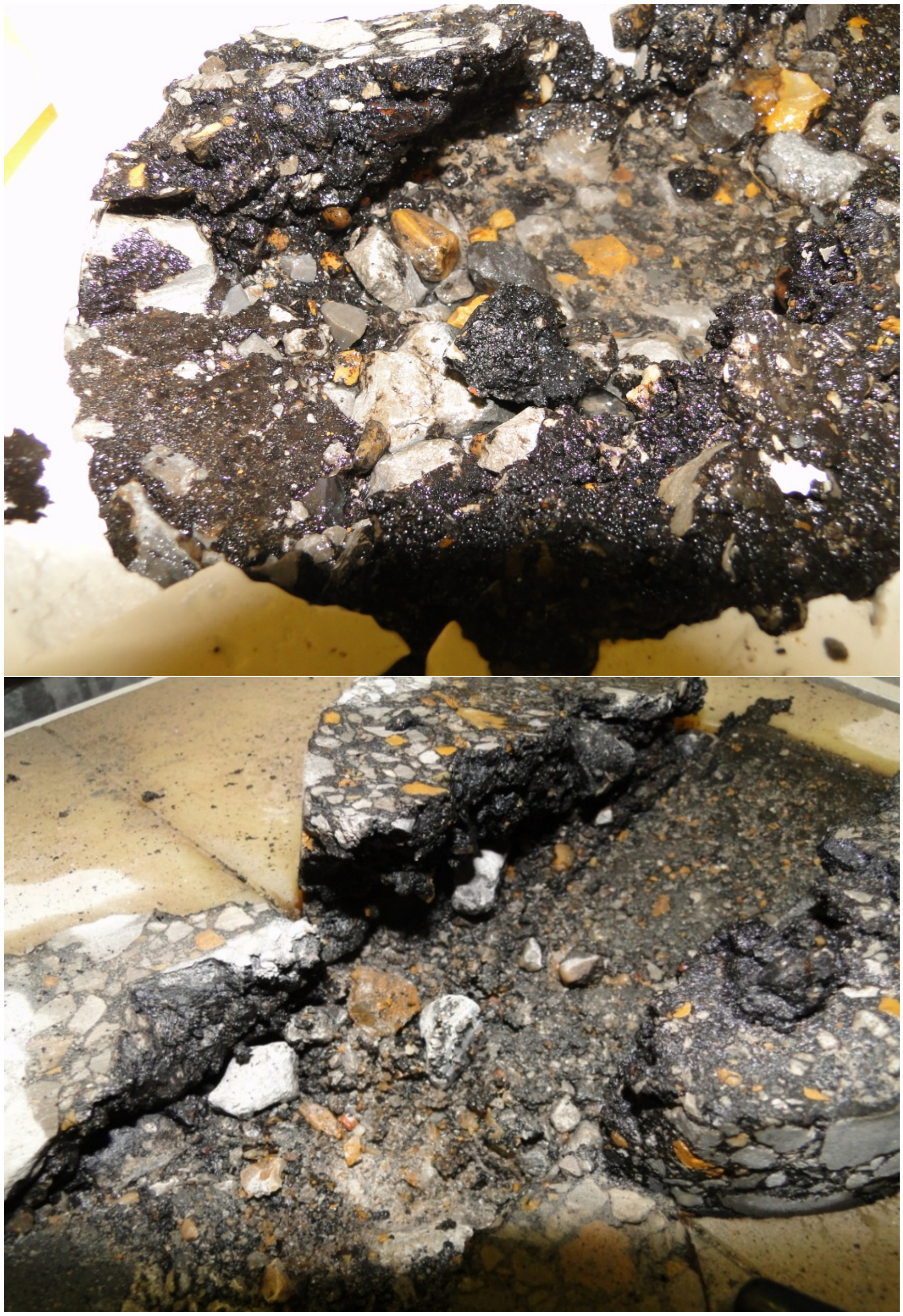


Figure 32
Moisture damage observed on cores after LWT test

Boil Test
The boil test is a quick and easy method to measure the compatibility of materials used in mixture in addition to quantifying the moisture susceptibility potential. However, this test is very subjective and only a visual rating is conducted to assess the moisture damage potential. The Louisiana boil test (TR 317M, Method B) was utilized in this study to evaluate the

moisture susceptibility of mixtures obtained from roadway cores. The boil test that was conducted for this study is presented in Figure 33. To obtain loose mixtures for the test, roadway cores were split into different lifts and then each lift was reheated at 135±5°C until they became soft enough to separate by hand without fracturing any aggregate. It should be noted that the test is meant to be conducted on the plus No. 4 fraction of the aggregates; however, the whole mixtures collected from roadway cores were utilized due to the difficulty in separating the plus No. 4 fractions from the mix. Mixtures from only the center parts of the cores were used to avoid the aggregates with the cut-faces that are found at the edge of the cores.

As seen in Figure 33, almost no-stripping of binder was evident from the boil test. The reviewers of the test judged that, on average, 98 percent of the aggregates retained the binder coating after 10 minutes of boiling. This also indicates that there may not be any compatibility issue with the materials used in this mixture. However, it should be noted that boil test neither reflects any mechanical properties of the mixture nor accounts for the effect of traffic action. Therefore, the stripping problem noticed in the LWT test was perhaps a result of the underneath moisture and pumping action from the traffic movement rather than the compatibility of the materials.

Figure 33
Boil test

CONCLUSIONS

Both field and laboratory investigations demonstrated the occurrence of moisture damage in the leveling and binder course lifts of this section of I-20. Based on these investigations the following conclusions are derived:

- The drainage system was constructed in an area that provided little or no benefit in removing moisture from underneath the pavement structure. A recommendation to correct this issue is described in the proceeding section.
- There was no evidence that the excessive rutting in the surface was solely the result of an inferior surface course mixture. In fact, the presence of moisture in the pavement structure appears to be a major contributor to the surface rutting.
- LADOTD reports revealed that the lift thicknesses were not properly constructed in accordance with the NMAS used in the JMF.
- Blended aggregate gradations and percent asphalt contents of the roadway cores indicated that the contractor met the specification requirements for Superpave binder course mixtures. Despite a small percentage of samples found to be out of the validation tolerance limits, no correlation was noticed between materials and performance of distressed and non-distressed areas.
- The High Speed profiler revealed there was a general trend toward higher rut depths in the outside lanes as compared to the inside lanes for both the east and west bound directions.
- The FWD indicated the amount of AC modulus values below 450 ksi ranged from 9 to 45 percent. This implies that the stiffness of the AC pavement has been impacted by stripping.
- With the exception of the WBOL, the in-place structure number, SN_{eff}, was generally close to the required structural number, SN_f, implying that sufficient strength was available to carry the projected loading.
- Taking into account the results from the modulus and SN_{eff} values, it can be concluded that stripping has affected the strength of the AC pavement and that sufficient structure generally exists to carry the projected loading, except for the WBOL, which may need improvement.

RECOMMENDATIONS

LTRC recommends placing an underdrain system throughout the length of the project against the outside edge of the original concrete pavement to help eliminate the moisture from the existing pavement structure. Ensure the top of the underdrain system is placed at the top or slightly above the first lift of asphalt mixture. The depth of the underdrain system should be enough to go through the treated base coarse. Outlets should be placed according to the standard plan for edge drains. A full depth 8- to 10-ft wide permeable asphalt drainage mixture placed in the shoulder and day lighted through the cross slope could be utilized for the outlets.

The EB outside lane has the worst rutting followed by the WB outside lane. FWD data shows the WB outside lane to have lower structural strength than the design or other lanes to the equivalent of about one inch of hot mix. The FWD data can be used to identify weak areas to estimate potential full depth patching in the EB outside lane. The FWD data has too much variation in the WB outside lane to determine potential full depth patching. It is suggested that the best way to estimate potential full depth patching in the WB outside lane would be areas of the worst rutting especially if cracking is beginning to be observed in or around the ruts.

Once the drainage is placed, it is recommended that the outside lanes in both directions be milled two inches and replaced with either SMA or SuperPave mix. After milling, any areas of visibly weak or moisture damaged pavement should be replaced full depth. Estimates for bidding can be made as indicated in the paragraph above. It is additionally proposed that all four lanes should be topped with an Open Graded Friction Course. The OGFC will add safety from both a driving public perspective and factor of safety structural perspective. The researchers have seen OGFCs provide a bridging effect on other sections of I-20 where there was significant stripping.

LTRC remains willing to assist the district in any material testing and monitoring of the reconstruction of this roadway.

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APPENDIX A

Forensic Evaluation Report: 1st Round
I-20, SP No. 451-08-0078

Introduction

At the request of District 05, the LTRC Asphalt Material Lab performed a forensic evaluation of the above titled I-20 site. The exact location of the project is shown below in Figure 34. The scope of this project included a full depth patching of the existing 8" continuously reinforced PCC pavement and placing a 2" Stone Matrix Asphalt (SMA) layer on top of a 6" (minimum) of Superpave Level 2 Binder Course (BC). According to the District 05 some areas of this project showed excessive premature rutting.

Figure 34
Project location

A total of 40 roadway cores were collected by the District 05 materials lab and sent to LTRC for forensic evaluation. Eight of those cores were from "good" locations, six were from "very bad" areas and the rest were from the "bad" areas.

SMA mixtures consist of a high percent of predominantly gap-graded coarse aggregates. These type of mixtures are designed in such a way that the permanent deformation resistance capacity relies on the stone on stone contact of the coarse aggregate skeleton. Longitudinal sectional views of a roadway core collected from a very bad section of this project are presented in Figure 35. It is evident there is lack of proper stone on stone contract for the SMA mixture used in this project.

Figure 35
Aggregate arrangements in the SMA mixture

Aggregate Gradations from Roadway Cores

Blended aggregate gradations and percent asphalt contents were obtained from the ignition oven method and compared with the original job mix formula (JMF) for this project and the current LADOTD specification limits for SMA mixtures. Investigation showed that the extracted aggregate gradations obtained at the LTRC Asphalt Lab were different than the approved JMF (Figure 36). Moreover, the "percent passing" values for several sieves exceeded the specification limits for mixtures representing bad and very bad sections. The use of finer graded aggregates might be the possible reason of losing stone on stone contact of aggregates as discussed previously. The percent asphalt content obtained from the roadway cores representing good, bad, and very bad sections are 6.18, 6.37, and 6.16 percent respectively, whereas the asphalt content in the approved JMF was reported to be 6.0 percent. The extracted asphalt content was within the JMF and the specifications requirement of 6.0 ± 0.4 percent.

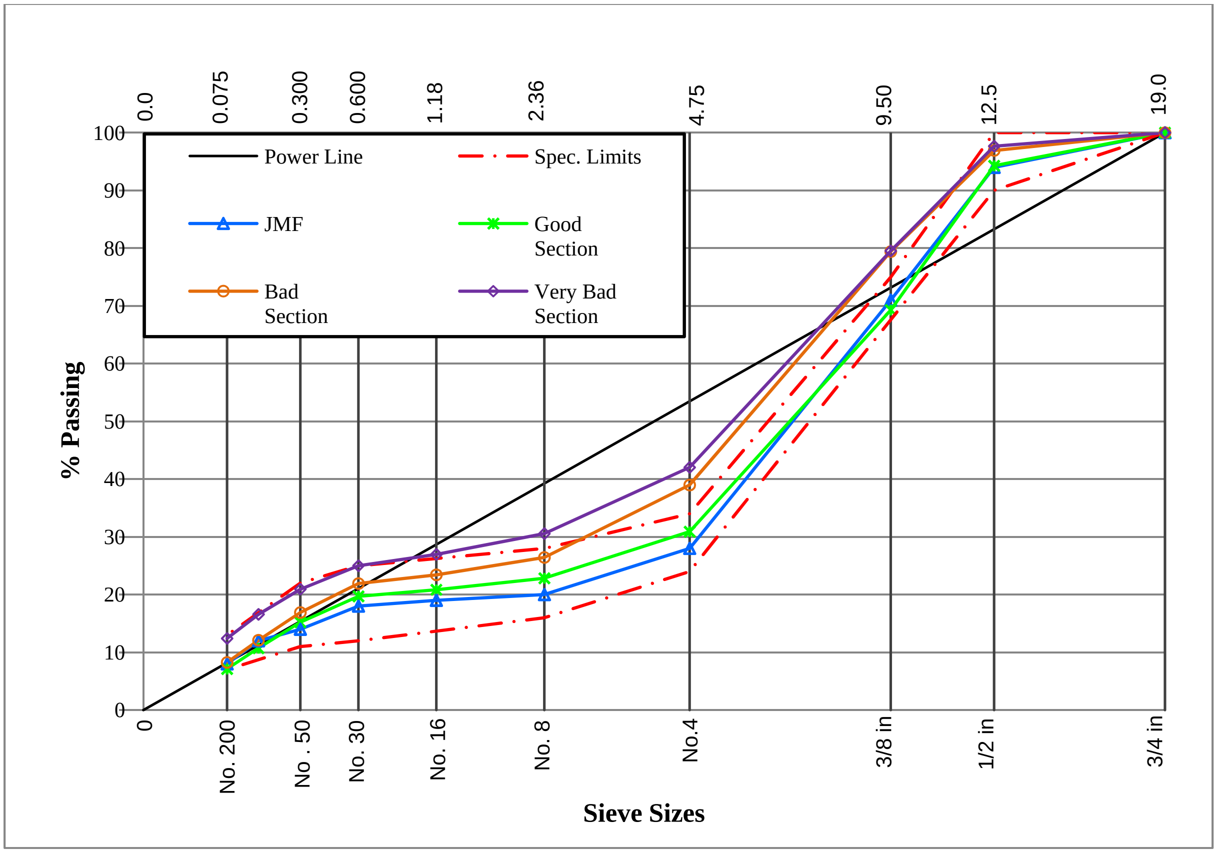

Figure 36
Aggregate blend gradations

Loaded Wheel Tracking (LWT) Test

LWT (Hamburg) tests were conducted in accordance with AASHTO T 324 in an effort to evaluate the rutting potential and moisture susceptibility of SMA mixture used in this project. Roadway cores from good and very bad sections were tested in addition to laboratory fabricated superpave gyratory compacted (SGC) specimens supplied by the contractor, D & J Construction. It should be noted that the SGC specimens were prepared to replicate the plant production of the mixtures from the actual aggregate stockpiles during the construction of the project. Results presented in Figure 37 indicate that roadway cores from the good section and laboratory fabricated SGC specimens met the maximum allowable rut depth requirement of 6.0 mm after 20,000 passes. Also, no sign of stripping was observed for those specimens. Alternatively, roadway cores from the very bad section failed immediately after 6300 wheel passes and showed sign of severe stripping. As shown in Figure 38, the roadway cores from very bad section were crashed and completely destroyed after the test.

Aggregate Gradations from Gyratory Specimens

Once again the ignition oven method was used to determine the blended aggregate gradations and percent asphalt contents for laboratory fabricated SGC samples supplied by the contractor. Although SGC specimens passed the LWT test, the burn gradations for that mixture failed to meet the current LADOTD aggregate blend specifications for SMA mixtures (Figure 39). Additionally, the average asphalt content for SGC samples was

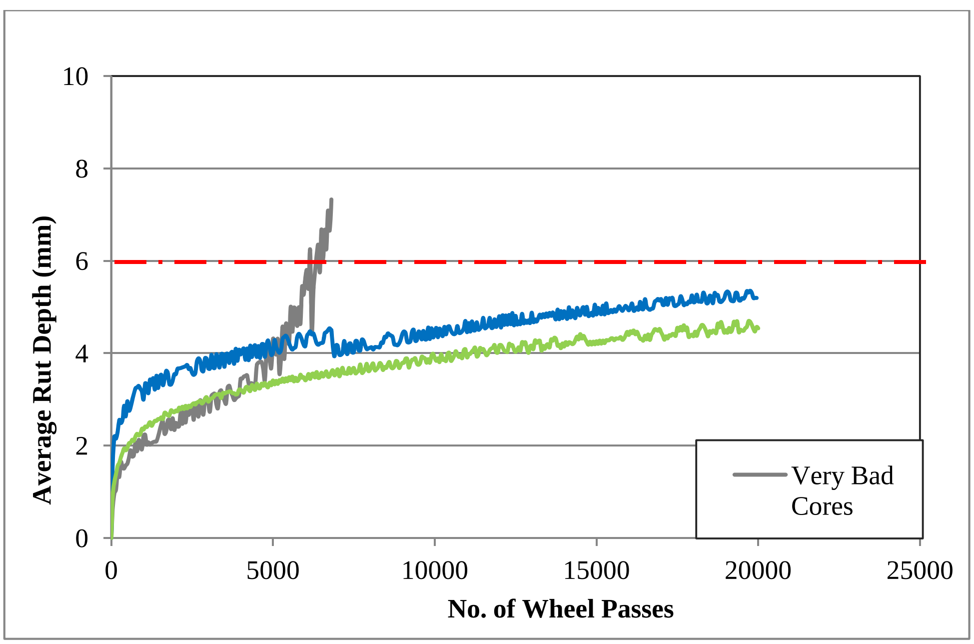

Figure 37
LWT test results

found to be 5.45 percent which failed to meet the minimum asphalt content requirement of 5.6 percent for SMA mixtures.

Interface Shear Strength (ISS) Test

ISS tests were performed on roadway cores collected from sections identified as good, bad, and very bad. The mean interface shear strengths for those sections were 42.1-, 52.6-, and 38.6-psi, respectively. In a previous study Dr. Louay Mohammad recommended a minimum ISS of 40-psi for satisfactory performances. Test results indicate that the very bad section marginally failed to meet this criterion.

Figure 37
Roadway Core from Very Bad Section after LWT Test

Figure 38

Condition of roadway core specimens after LWT testing

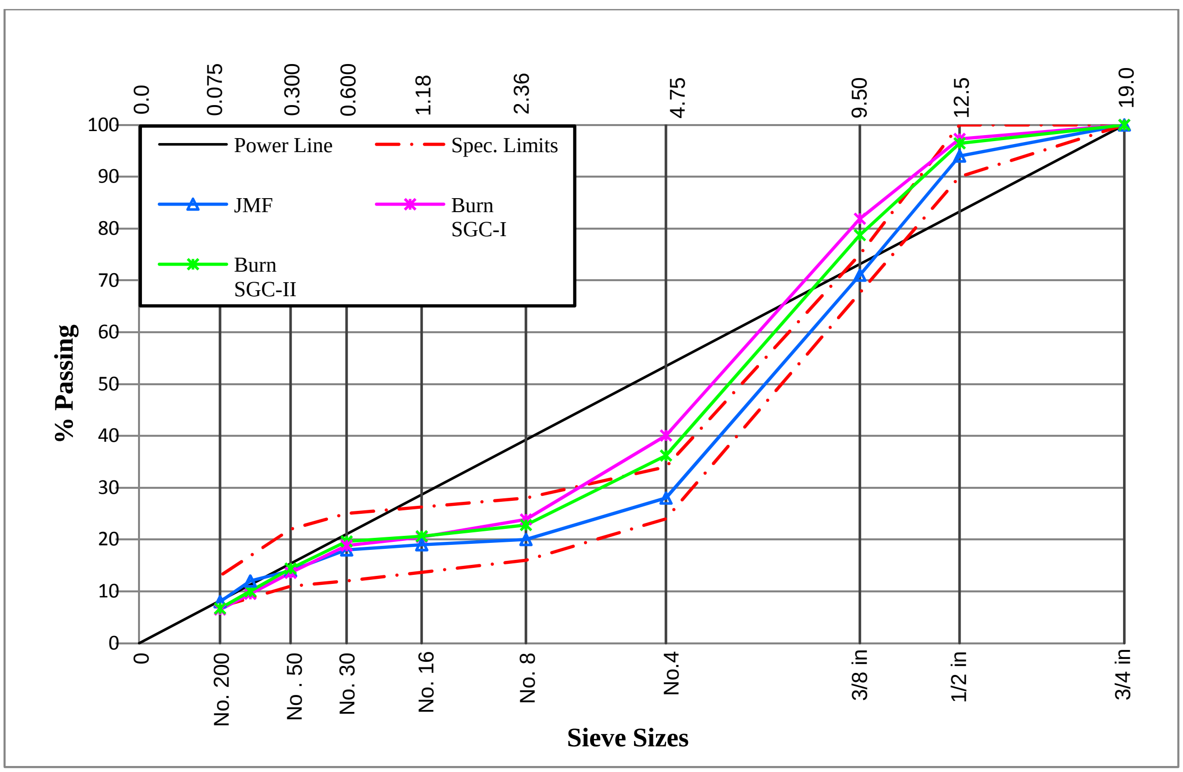

Figure 39

Aggregate blended gradations for lab-fabricated SGC specimens

Summary

It is evident from the laboratory tests performed at LTRC that the failing mixtures did not meet the gradation requirements of LADOTD. As a result, the stone on stone contact is compromised, which is critical to the performance of SMA mixtures.

Acknowlegements

The authors would also like to thank the Willie Gueho, Patrick Frazier, and Jeremy Icenogle at the LTRC Asphalt Laboratory for their hard work and assistance.

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Forensic Evaluation Report: 2nd Round
I-20, SP No. 451-08-0078

Introduction

In mid-December 2011, the LTRC Asphalt Materials Lab performed a first-round of forensic evaluation of the above titled I-20 site at the request of District 05. In mid-February, LTRC received another request from District 05 to perform a second forensic evaluation of six roadway cores collected from the same project. The locations and sampling dates of the second set of roadway cores are presented in Table 6. These new samples were taken from the wheel-path region of the mainline roadway that was indicated to LTRC as a good roadway section. Longitudinal sectional views and relative aggregate arrangements in two sample cores are shown in Figure 40.

Table 6
Roadway core locations

Sample ID	Direction	Location	Date Sampled
Core # 1	Westbound	Inside Lane, 10 feet left of C.L. At mile marker 187.	1/31/12
Core # 2	Westbound	Inside Lane, 10 feet left of C.L. 1043 feet west of mile marker 187.	1/31/12
Core # 3	Eastbound	Inside Lane, 10 feet left of C.L. 1043 feet west of mile marker 187.	02/07/12
Core # 4	Eastbound	Inside Lane, 10 feet left of C.L. At mile marker 187.	02/07/12
Core # 5	Eastbound	Outside Lane, 10 feet right of C.L. 1043 feet west of mile marker 187.	02/09/12
Core # 6	Eastbound	Outside Lane, 10 feet right of C.L. At mile marker 187.	02/09/12

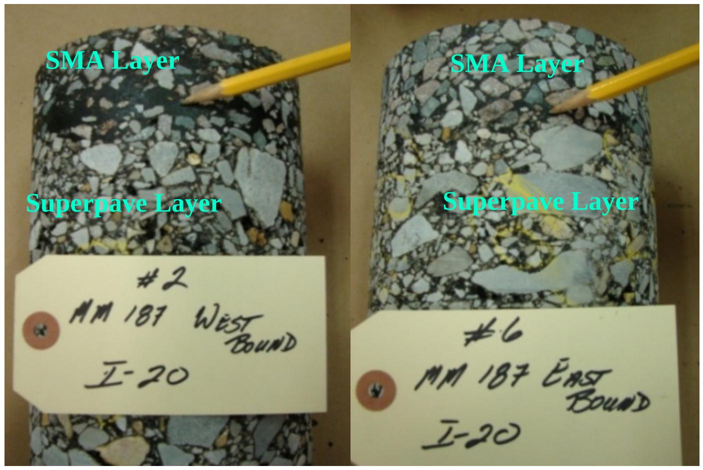

Figure 40
Aggregate arrangement in the SMA mixture

Aggregate Gradations from Roadway Cores

Core # 5 and Core # 6 were used in obtaining the blended aggregate gradations and percent asphalt contents from ignition oven method. Figure 41 shows a comparison of blended gradations with the original job mix formula (JMF), specified tolerances for SMA mixtures, and the blended aggregate gradations found from the roadway cores previously submitted (roadway cores represented the good, bad, and very bad sections of the first submission). It is evident that the extracted aggregate gradations obtained at the LTRC Asphalt Lab did not match the gradations reported in the approved JMF. Also, the "percent passing" values for several sieves exceeded the tolerance limits specified in current LADOTD specifications for SMA mixtures. Interestingly, for the No. 4, 3/8 inch, and 1/2 inch sieves, the "percent passing" values from second submission of roadway cores are similar to the mixtures representing bad and very bad sections in the first submission of cores. The percent asphalt content obtained from Core # 5 and Core # 6 were 6.41 and 6.51 percent respectively whereas, the asphalt content in the approved JMF was reported to be 6.0 percent. The asphalt content for the first set of roadway cores from good, bad, and very bad sections are 6.18, 6.37, and 6.16 percent respectively. Note that the current LADOTD specifications tolerate 6.0 ± 0.4 percent of extracted asphalt in SMA mixtures.

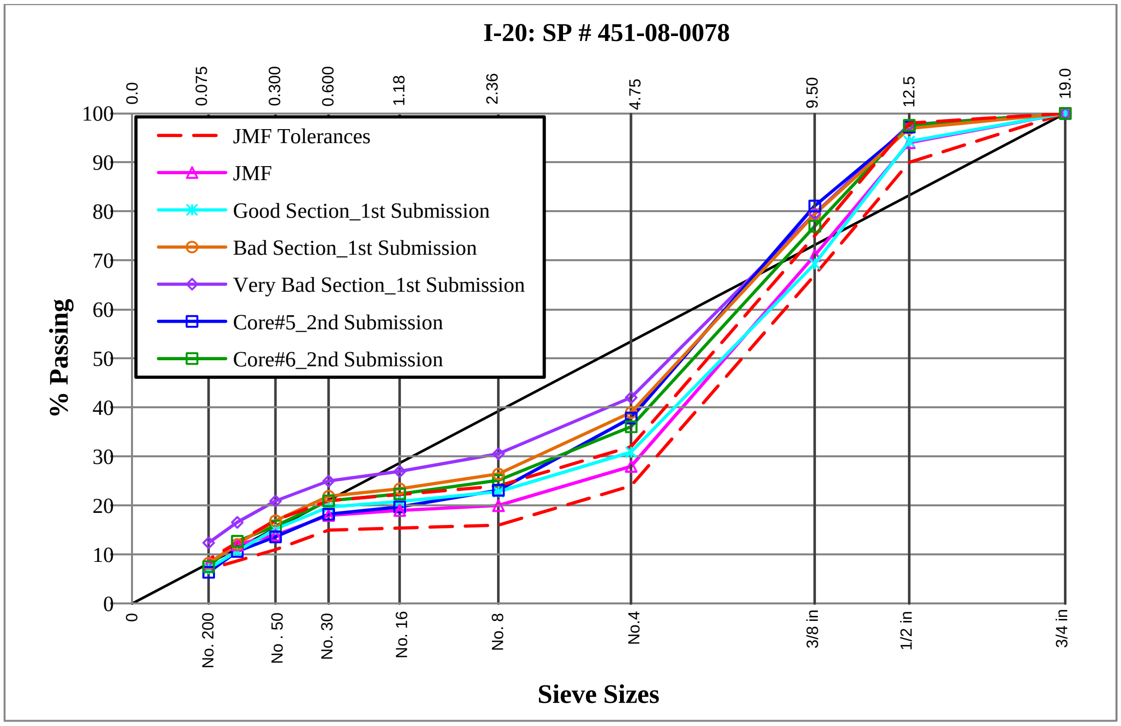

Figure 41
Aggregate blended gradations

Loaded Wheel Tracking (LWT) Test
To evaluate the rutting potential and moisture susceptibility of SMA mixture used in this project, cores from Westbound (Core # 1 and 2) and Eastbound (Core # 3 and 4) lanes were used in LWT (Hamburg) tests in accordance with AASHTO T 324. Results presented in Figure 42 indicate that roadway cores from both sections exceeded the maximum suggested rut depth requirement of 6.0 mm after 20,000 passes. Also, no sign of stripping was observed for those specimens. It should be noted that the roadway cores from very bad section of first submission failed immediately after 6300 wheel passes and showed sign of severe stripping.

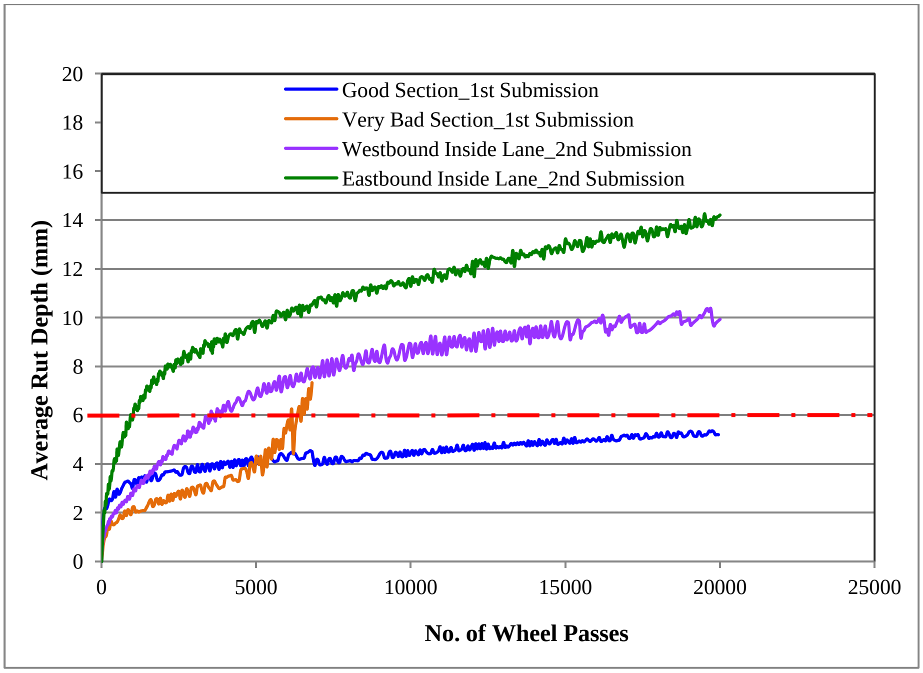

Figure 42
LWT test results

Conclusions

The combination of blended gradation and LWT test results indicates that the SMA mixture representing the good section of first submission, was produced according to LADOTD's specification and, consequently, meets the performance criteria. Alternatively, mixtures that did not meet the gradation requirements of LADOTD failed to show the recommended performance.

Acknowlegements

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APPENDIX B

Moduli Values

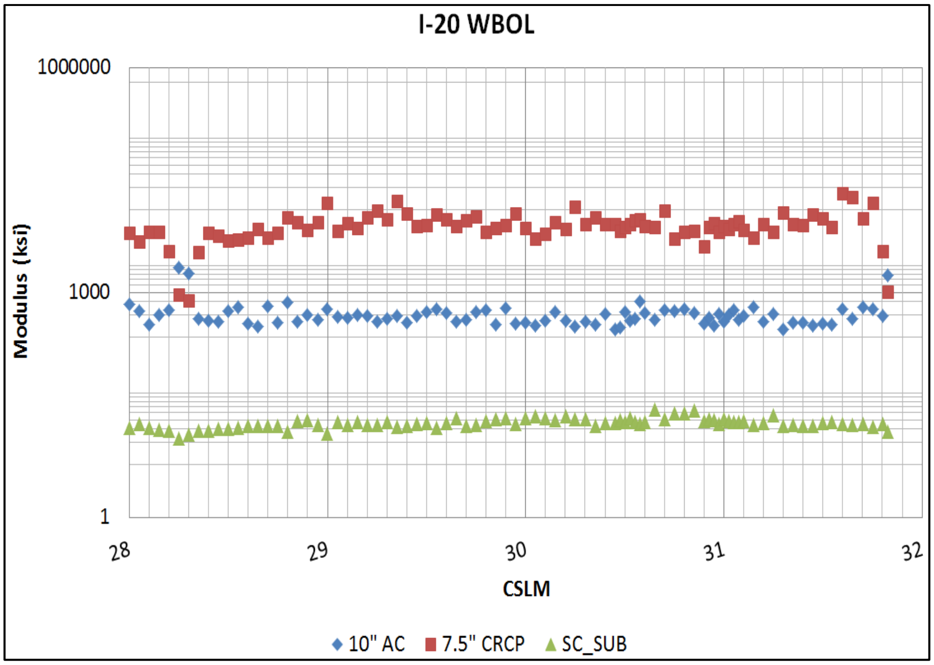

Figure 43
Westbound outside lane

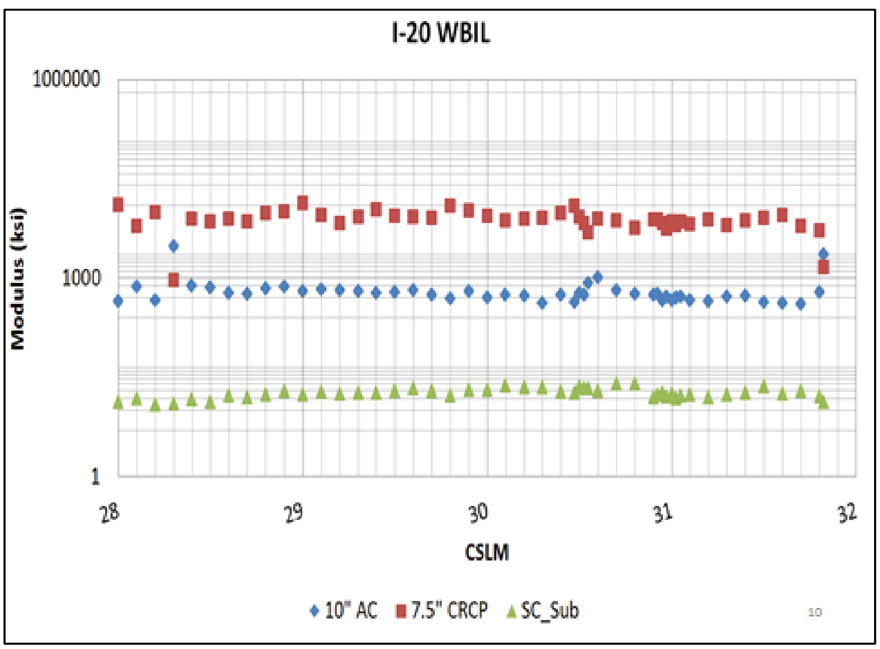

Figure 44
Westbound inside lane

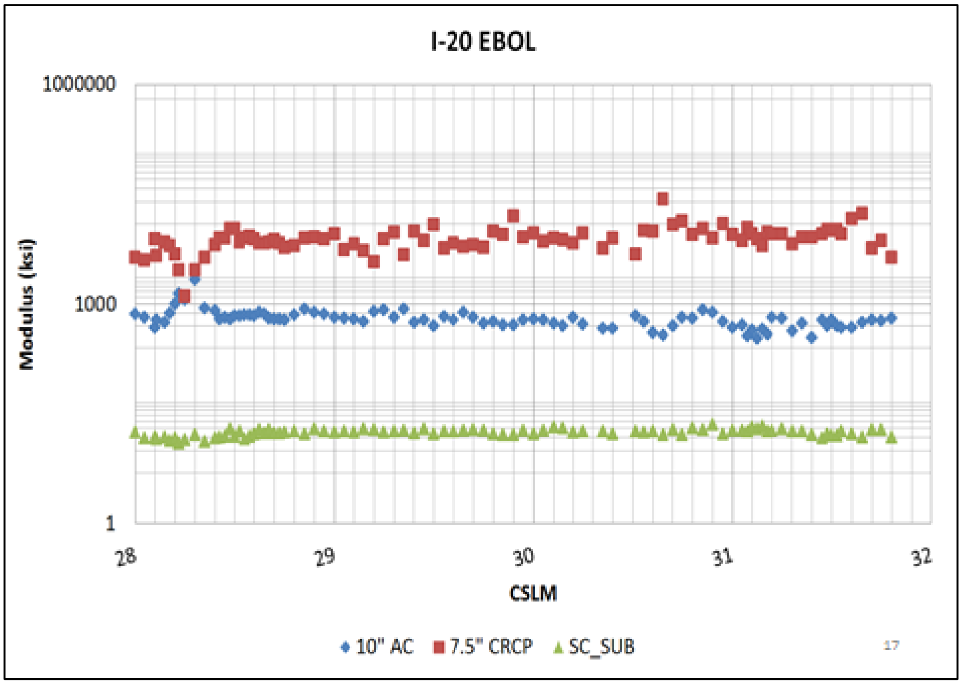

Figure 45
Eastbound outside lane

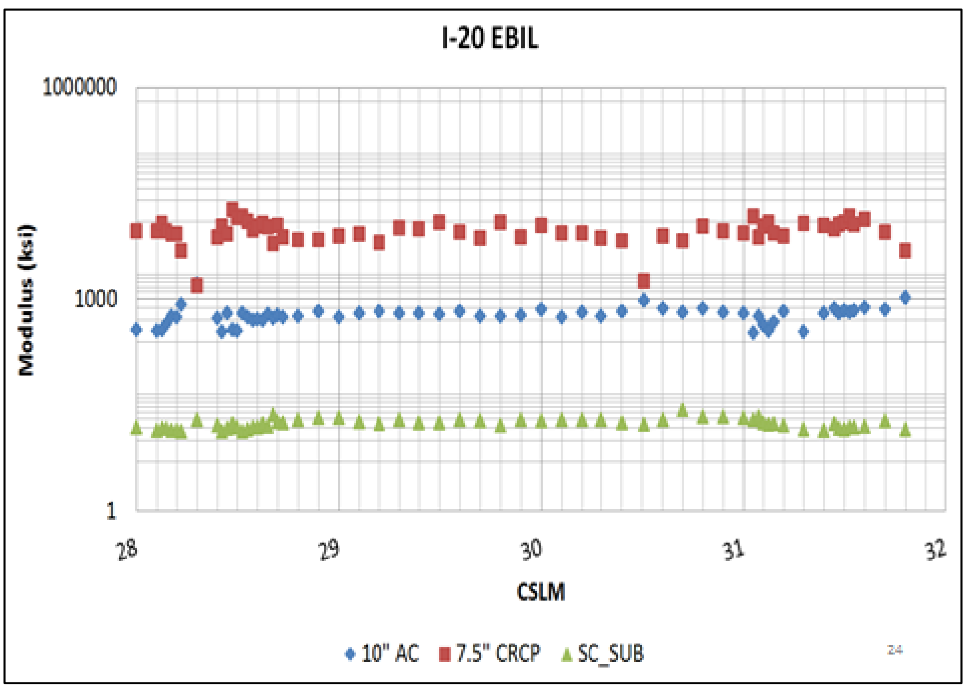

Figure 46
Eastbound inside lane

APPENDIX C

Darwin Analysis

1993 AASHTO Pavement Design

DARWin Pavement Design and Analysis System

A Proprietary AASHTOWare
Computer Software Product
PC User

Overlay Design Module

S.P. No. 451-08-0078
Route I-20
Mound - Delta
Madison Parish

AC Overlay of PCC Pavement

Pavement Thickness for Future Traffic 12.59 in

Design Method	Effective Existing Overlay Thickness (in)	Overlay Thickness (in)
Condition Survey	8	7.93
Remaining Life	-	-

Pavement Thickness for Future Traffic

Future 18-kip ESALs Over Design Period	24,285,840
Initial Serviceability	4.3
Terminal Serviceability	2.8
PCC Modulus of Rupture	600 psi
PCC Elastic Modulus	4,200,000 psi
Static k-value	575 psi/in
Reliability Level	97 %
Overall Standard Deviation	0.37
Load Transfer Coefficient, J	3.2
Overall Drainage Coefficient, Cd	1.05
Calculated Thickness for Future Traffic	12.59 in

Effective Pavement Thickness - Condition Survey Method

Existing PCC Thickness	8 in
Existing AC Thickness	- in
AC Milling Thickness	- in
Rut Depth	- in
Durability Adjustment Factor	1
Fatigue Damage Adjustment Factor	1
AC Quality Adjustment Factor	-
Number of Deteriorated Joints	0 per mi
Number of Deteriorated Cracks	0 per mi
Number of Unrepaired Punchouts	0 per mi
Number of Expansion Joints,	-
Exceptionally Wide Joints, or AC Full Depth Patches	0 per mi

Calculated Results

Calculated Joints and Cracks Adjustment Factor	1.00
Calculated Effective Pavement Thickness	8.00 in

Future Rigorous ESAL Calculation

Performance Period (years)	15
Two-Way Traffic (ADT)	20,700
Number of Lanes in Design Direction	2
Percent of All Trucks in Design Lane	80 %
Percent Trucks in Design Direction	50 %

Vehicle Class	Percent of ADT	Annual % Growth	Average Initial Truck Factor (ESALs/ Truck)	Annual % Growth in Truck Factor	Accumulated 18-kip ESALs over Performance Period
1	0.2	1	0.0004	0	39
2	45.2	1	0.0004	0	8,802
3	20.5	1	0.0142	0	141,711
4	0.5	1	0.1677	0	40,819
5	4.5	1	0.1677	0	367,374
6	1.2	1	0.5715	0	333,857
7	0.1	1	0.5715	0	27,821
8	3.2	1	0.9875	0	1,538,331
9	21.9	1	1.7686	0	18,855,429
10	1.1	1	2.873	0	1,538,477
11	0.8	1	1.84	0	716,590
12	0.4	1	1.84	0	358,295
13	0.4	1	1.84	0	358,295
Total	100	-	-	-	24,285,840

Growth	Compound
Total Calculated Cumulative ESALs	24,285,840

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