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13. Abstract

The objective of this forensic investigation was to determine possible causes of the early pavement deterioration at Ponderosa Drive in East Baton Rouge Parish. The study comprised of pavement and geotechnical evaluation to determine structural strength of the pavement and subsurface layers. Further, asphalt field cores were obtained for subsequent laboratory evaluation. The pavement research group conducted rutting and structural evaluation of the pavement section utilizing the profiler and falling weight deflectometer (FWD), respectively. Rut depths were found to be greater than 0.75 in. at specific locations along the pavement section. FWD deflection data showed that the pavement structure is in poor condition with possible large variations in subgrade support. In addition, the asphalt layer for the pavement section was found to exhibit stiffness values within the range of values reported by other researchers for a typical asphalt concrete. Possible causes of the rapid deterioration of the pavement section include higher levels of traffic, non-homogeneous cement treated base, weakening of pavement structure due to the August 2016 floods, and the subsequent use of the road section by heavier waste disposal trucks to remove flood debris. It is recommended that the entire pavement section be reconstructed with a design that ensures that the prevailing traffic is considered.

Forensic Evaluation of Pavement Structure: Ponderosa Drive in East Baton Rouge Parish

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The objective of this forensic investigation was to determine possible causes of the early pavement deterioration at Ponderosa Drive in East Baton Rouge Parish. The study comprised of pavement and geotechnical evaluation to determine structural strength of the pavement and subsurface layers. Further, asphalt field cores were obtained for subsequent laboratory evaluation. The pavement research group conducted rutting performance and structural evaluation of the pavement section utilizing the profiler and falling weight deflectometer (FWD), respectively. Rut depths were found to be greater than 0.75 in. at specific locations along the pavement section. FWD deflection data showed that the pavement structure is in poor condition with possible large variations in subgrade support. In addition, the asphalt layer for the pavement section was found to exhibit stiffness values within the range of values reported by other researchers for a typical wearing course. Possible causes of the rapid deterioration of the pavement section include higher levels of prevailing traffic, non-homogeneous cement treated base, weakening of pavement structure due to the August 2016 floods, and the subsequent use of the road section by heavier waste disposal trucks to remove flood debris. It is recommended that the entire pavement section be rehabilitated with a design that ensures that the prevailing traffic is considered.

Acknowledgments

The research was completed with the assistance of lab technicians. Significant contributions to this investigation were performed by Terrell Gorham, Benjamin Key, Biyuan Zheng, Renee Cosse, Sonda Brown, and Hend Alyousef. In addition, Kevin Gaspard is acknowledged for providing valuable input to this investigation.

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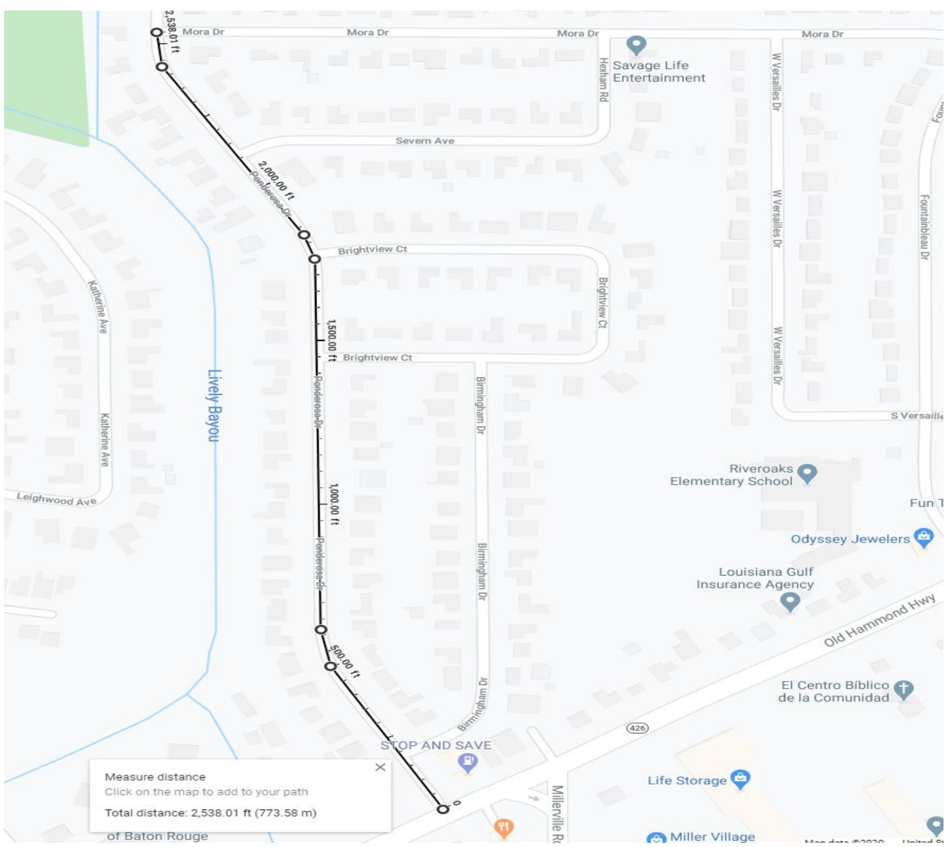
Introduction

The Transportation and Drainage Department of East Baton Rouge Parish contacted Louisiana Transportation Research Center (LTRC) regarding the rapid deterioration of a pavement section at Ponderosa Drive. Ponderosa Drive is a 1-mile subdivision arterial road directly between I-12 interstate highway and US 190/Florida Boulevard in Baton Rouge, Louisiana. This road is a cut through from Florida Boulevard to I-12 via Old Hammond Highway and Millerville Road and likely experiences higher levels traffic than the design traffic volume. The section of interest is located between Old Hammond Highway and Mora Drive and extends approximately 2,500 ft. (≈0.5 mile) in length; see Figure 1.

Figure 1. Pavement section location



The pavement section was rehabilitated in 2014. The rehabilitation treatment consisted of milling off approximately 2 in. of existing asphalt wearing course, and the treatment of 10 in. of existing base layer with 10 percent Portland cement. Further, the treated base layer was overlaid with a 2-in. asphalt wearing course. The roadway boring report indicates that the pavement structure prior to rehabilitation consisted of 2 to 4-in. of asphalt wearing course supported by 12 to 13-in. of soil cement base; see Table 1. According to the East Baton Rouge Transportation and Drainage Department, the road has performed poorly over the years. Based on preliminary investigation, excessive rutting and fatigue cracks were observed in the wheel paths along the road segment; see Figure 2a. Several patched areas were also detected on the asphalt surface with white streaks, which was an indication of early wear and problems likely associated with moisture damage; see Figure 2b. The white streaks at the surface indicate that material is pumping up from below the asphalt layer. In addition, transverse and longitudinal cracks were also seen along the pavement section; see Figure 2c.

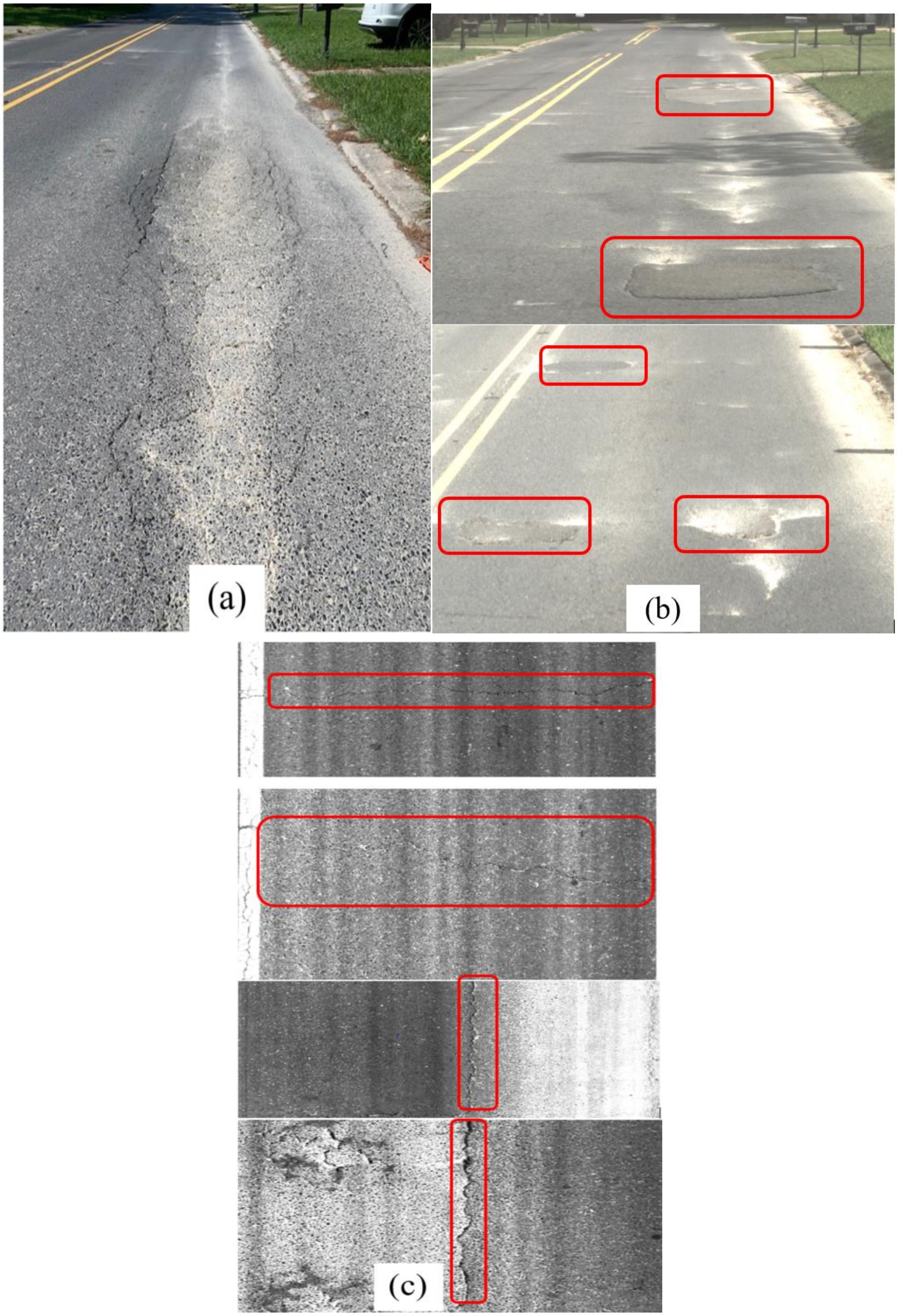
Table 1. Roadway boring report

DATE: August 13, 2013 PSI REPORT No.: 0193501-01

STREET NAME AND BORING NUMBER	LOCATION (GPS Coordinates)	DEPTH	PAVEMENT/BASE/SUBGRADE MATERIAL ENCOUNTERED
980 Ponderosa Dr. B-1	N 30˚ 27' 6.81" W 91˚ 1' 32.7"	0-2"	Asphalt
		2-14"	Soil Cement
		14-24"	Gray and Tan Clay
867 Ponderosa Dr. B-2	N 30˚ 27' 14.1" W 91˚ 1' 34.4"	0-3"	Asphalt
		3-15"	Soil Cement
		15-24"	Gray Clay
624 Ponderosa Dr. B-3	N 30˚ 27' 21.6" W 91˚ 1' 34.7"	0-4"	Asphalt
		4-17"	Soil Cement
		17-24"	Tan Clay
522 Ponderosa Dr. B-4	N 30˚ 27' 26.5" W 91˚ 1' 38.3"	0-4"	Asphalt
		4-17"	Soil Cement
		17-24"	Tan Clay

Note: The following report details the field results of the roadway borings performed for the above referenced project.

Figure 2. (a) Rutting and fatigue cracks, (b) patched areas, and (c) transverse and longitudinal cracks



Objective and Scope

The purpose of this forensic investigation was to determine possible causes of the early pavement deterioration at Ponderosa Drive in East Baton Rouge Parish.

The scope of the forensic investigation consisted of pavement and geotechnical evaluation to determine structural strength of the pavement and the subsurface layers. In addition, asphalt field cores were obtained for laboratory evaluation.

Methodology

Detailed description of the investigation techniques utilized in this study are presented in subsequent sections.

Pavement Evaluation

LTRC's roadway profiler and imaging vehicle (hereafter referred to as profiler) and falling weight deflectometer (FWD) were utilized for the pavement evaluation. Prior to performing the field evaluation of the pavement structure, the pavement research group obtained six years of Google Earth images of the pavement section to ascertain the progressive deterioration of the pavement section over the years; see Figures 3 and 4. Figures 3a and 3b show images of the road section before and during rehabilitation in 2013 and 2014, respectively. It is worth noting that the white spots, which are an indication of pavement deterioration and pumping failure, existed in 2013 before rehabilitation; see Figure 3a. Figures 4a, 4b, and 4c show the road section after rehabilitation (with some white spots appearing in January 2016, then followed by an apparent patching activity; see Figures 4b and 4c) and before the August 2016 flood. In addition, Figures 4d, 4e, and 4f present images of the road section after the August 2016 floods. It is noted that the white spots started appearing again on the road section from November 2017 and intensified in May 2019; see Figures 4e and 4f. Figure 5 shows detailed images of the deteriorated spots observed in May 2019, which are marked with red quadrilateral solid lines. Based on the preliminary analysis of the Google Earth images, the early deterioration of the road section may be attributed to the presence of weak spots in the pavement structure. This observation indicates that 2014 rehabilitation may not have improved the underlying weaker spots in the pavement. Additional causes of the early deterioration include weakening of the base and subgrade layers due to the August 2016 flood and the subsequent use of the road section by heavier waste disposal trucks to cart away flood debris.

Figure 3. Google Earth images of road section in (a) March 2013 and (b) March 2014

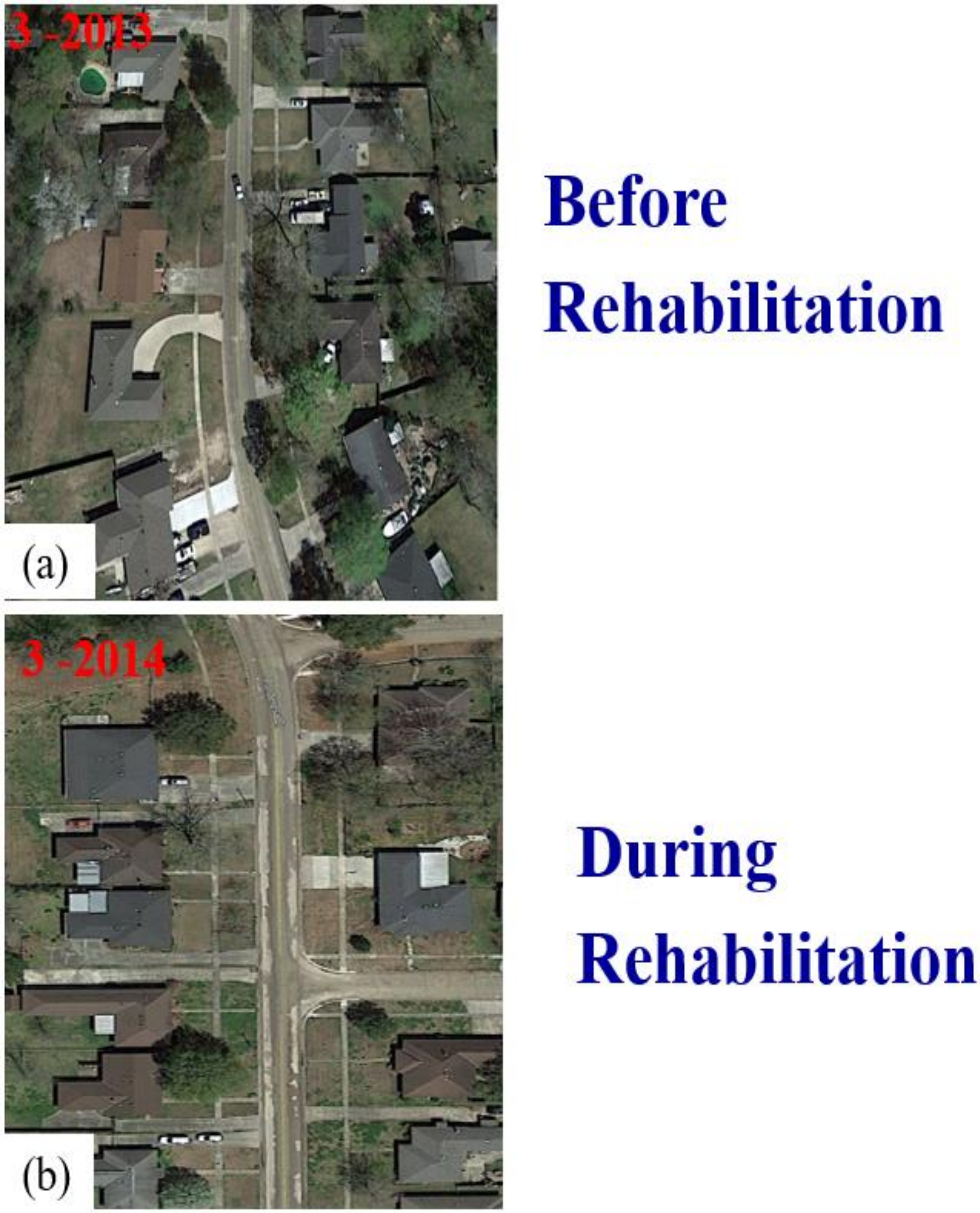


Figure 4. Google Earth images of pavement section taken in (a) August 2015, (b) January 2016 (before patching of areas with white streaks), (c) January 2016 (after patching of areas with white streaks), (d) October 2016, (e) November 2017, and (f) January 2019

Figure 5. Distress locations in May 2019

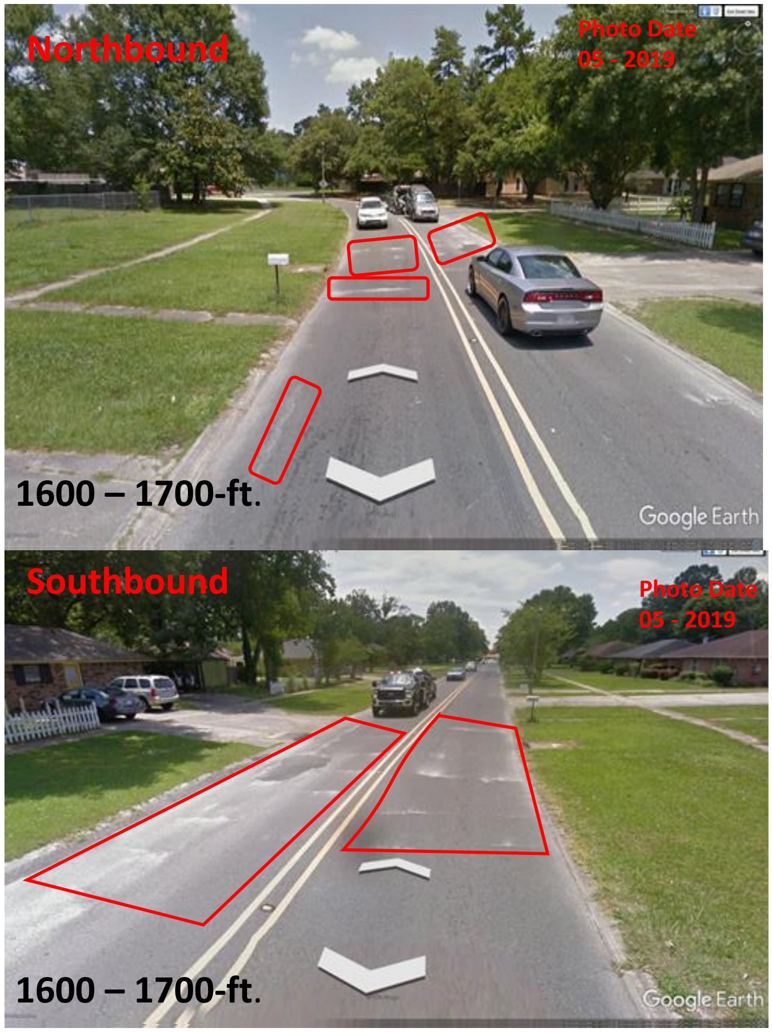
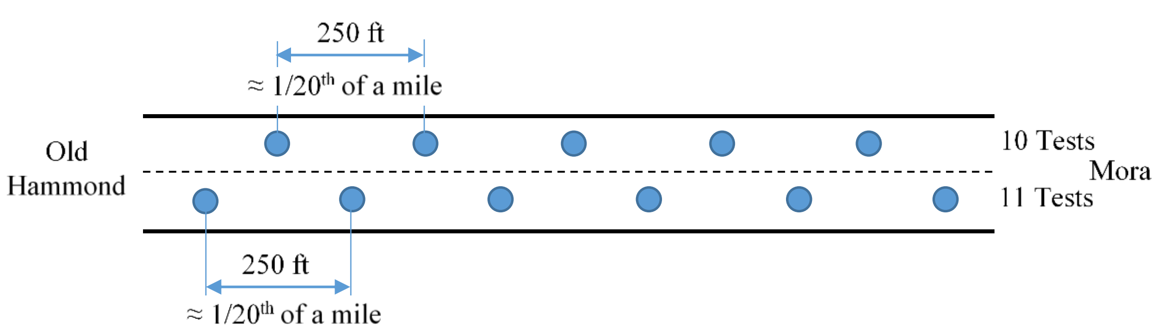


Figure 6 shows the FWD testing locations/frequency used in this investigation.

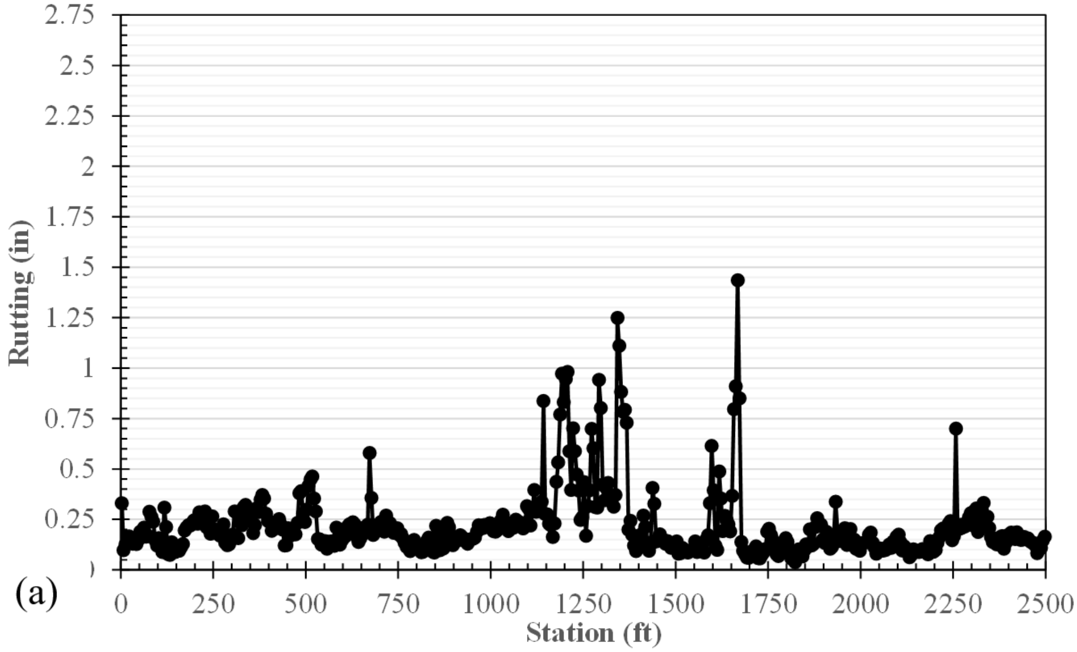
Figure 6. FWD testing locations/frequency

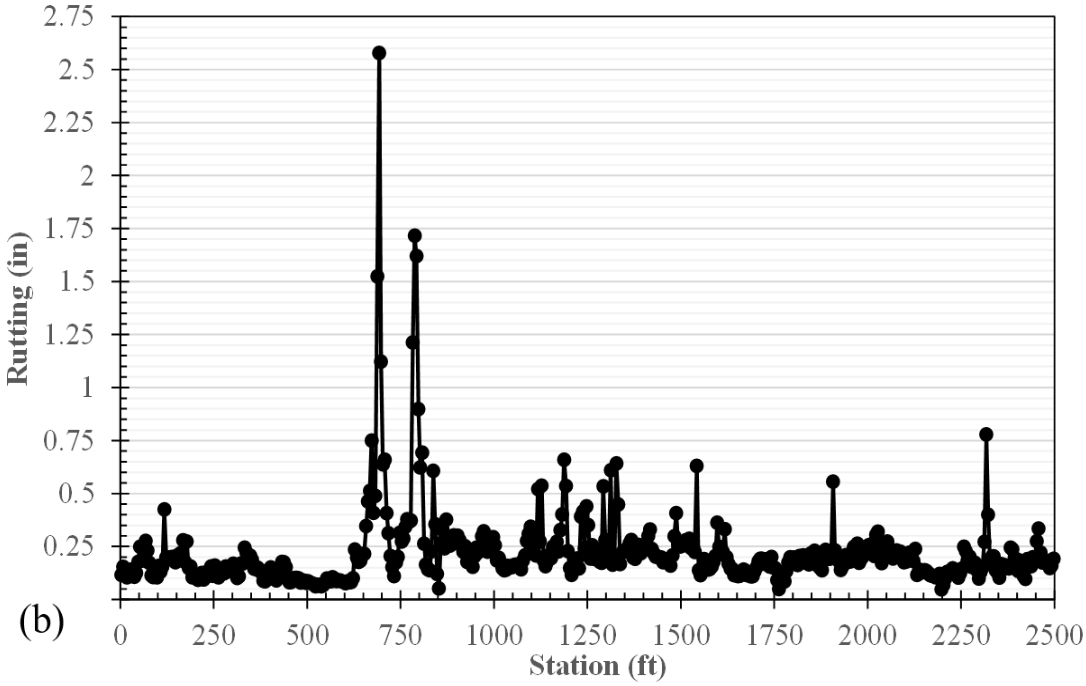


Profiler and FWD Results and Discussions

Rut depths were measured using LTRC's road profiler, which uses a 5-point rut bar system for pavement rut depth measurements. The rut depths were continuously measured as the survey van drove on the pavement section. The measured rut depths are presented in Figure 7. A couple of locations along the section exhibited rut depths greater than 0.75 in. The rut depths at a few locations were greater than 1 in. Such high rut depths are not typically observed in asphalt pavements with soil cement base. This observation may be attributed to a weakened base or subgrade layer.

Figure 7. Rutting values along (a) Northbound and (b) Southbound lanes

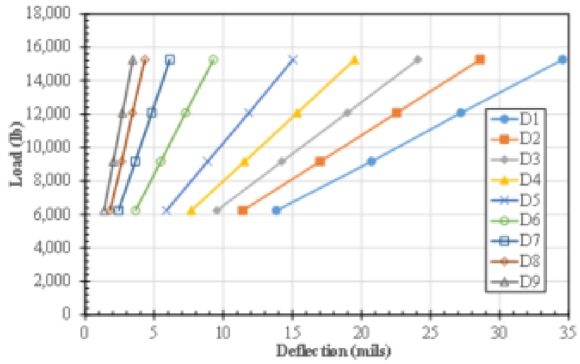




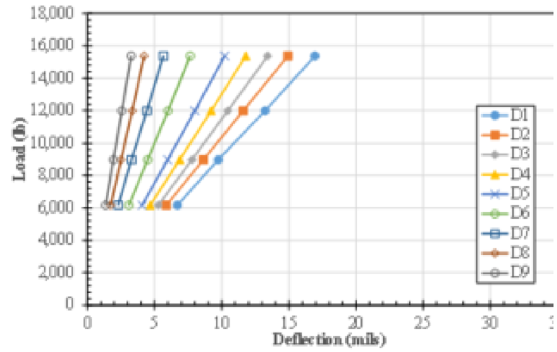
Processing Deflection Data from FWD

In addition to general data checks, FWD deflection data were checked for linearity. Figure 8 compares the load versus sensor deflection for selected locations in both North and Southbound lanes. The measurements show the data is good for linear analysis.

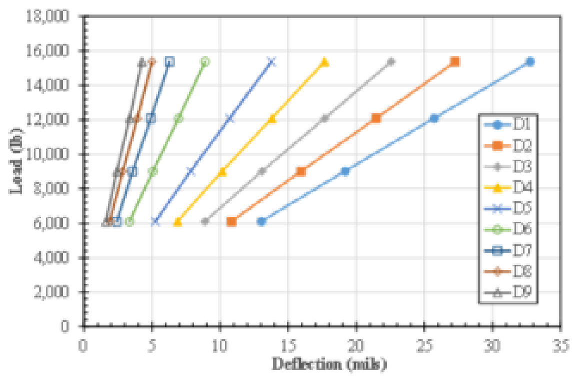
Figure 8. FWD load versus sensor deflection



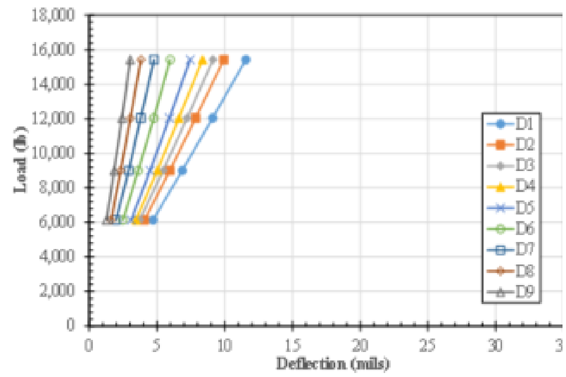
(a) Station 0, Northbound



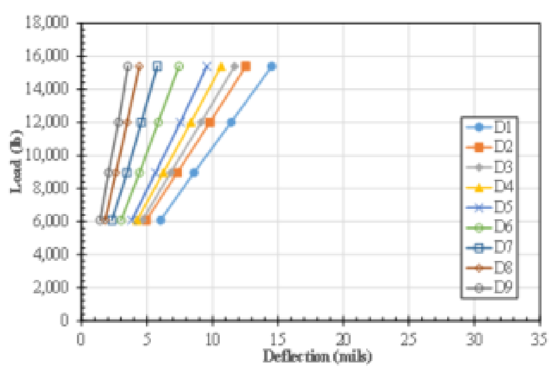
(b) Station 125, Southbound



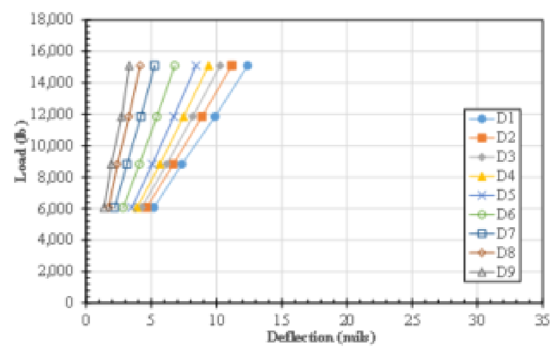
(c) Station 1250, Northbound



(d) Station 1125, Southbound



(e) Station 2500, Northbound



(f) Station 2375, Southbound

Deflection-Based Indices

A number of deflection-based indices were computed and are presented in Tables 2 and 3. The pavement structural conditions were rated by comparing these computed indices to the standard benchmark values [1]. It is noted that lower deflection (D_0), surface curvature index (SCI), base curvature index (BCI), and base damage index (BDI) values are preferred. On the other hand, the higher the radius of curvature (RoC) value, the better the pavement structure condition [1]. It was observed that the pavement structure is in poor condition (rated as severe) at a couple of locations along the section. The radius of curvature (RoC) results (>150 μm) show that the upper part of the pavement structure is in a sound structural condition, whereas the base curvature or damage index (BCI/BDI) values suggest possible large variations in subgrade support along the section. It is worth noting that BCI and BDI values greater than 100 and 80 μm, respectively represent pavement structure in poor condition (rated as severe) [1].

Table 2. Deflection-based indices and pavement structure conditions for Northbound

Station	D_0 (μm)	RoC (μm)	SCI (μm)	BCI (μm)	BDI (μm)	D_0 (Rating)	RoC (Rating))	SCI (Rating))	BCI (Rating))	BDI (Rating))
0	514	230	158	134	85	Severe	Sound	Warning	Severe	Severe
250	251	538	60	45	35	Warning	Sound	Sound	Sound	Sound
500	323	383	93	65	45	Warning	Sound	Sound	Warning	Warning
750	132	1544	18	11	14	Sound	Sound	Sound	Sound	Sound
1000	166	908	34	19	19	Sound	Sound	Sound	Sound	Sound
1250	486	253	153	133	71	Severe	Sound	Warning	Severe	Warning
1500	175	953	32	23	23	Sound	Sound	Sound	Sound	Sound
1750	601	239	175	171	84	Severe	Sound	Warning	Severe	Severe
2000	196	677	44	27	26	Sound	Sound	Sound	Sound	Sound
2250	246	730	45	41	36	Warning	Sound	Sound	Sound	Sound
2500	220	618	46	30	31	Warning	Sound	Sound	Sound	Sound

D_0: Defelction at center of loading plate; RoC: Radius of curvature; SCI: Surface curvature index; BCI: Base curvature index; BDI: Base Damage Index; μm: micrometer

Table 3. Deflection-based indices and pavement structure conditions for Southbound

Station	D_0 (μm)	RoC (μm)	SCI (μm)	BCI (μm)	BDI (μm)	D_0 (Rating))	RoC (Rating))	SCI (Rating))	BCI (Rating))	BDI (Rating)
125	250	691	52	46	38	Warning	Sound	Sound	Sound	Sound
375	192	820	37	26	24	Sound	Sound	Sound	Sound	Sound
625	109	1728	14	10	13	Sound	Sound	Sound	Sound	Sound
875	363	362	99	77	54	Warning	Sound	Sound	Warning	Warning
1125	173	872	36	24	22	Sound	Sound	Sound	Sound	Sound
1375	203	623	47	30	28	Warning	Sound	Sound	Sound	Sound
1625	181	1413	29	24	23	Sound	Sound	Sound	Sound	Sound
1875	258	1019	37	36	37	Warning	Sound	Sound	Sound	Sound
2125	193	1088	31	29	27	Sound	Sound	Sound	Sound	Sound
2375	191	1090	33	28	25	Sound	Sound	Sound	Sound	Sound

D_0: Defelction at center of loading plate; RoC: Radius of curvature; SCI: Surface curvature index; BCI: Base curvature index; BDI: Base Damage Index; μm: micrometer

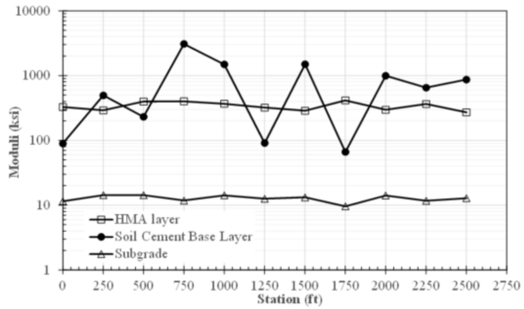
Backcalculation Analysis

Backcalculation was performed using the method of equivalent thickness. The program used for this analyisis is ELMOD6. It is worth noting that the asphalt layer thickness is only 2 in. for the pavement section investigated. It can be difficult to obtain reasonably backcalculated moduli for asphalt surface layers less than 3 in. thick. As such, in a recent FHWA publication of FWD testing and data analysis guidelines, Pierce et al. [1] recommended that if the total thickness of the asphalt layer is less than 3 in., the modulus of the asphalt layer should be fixed to allow backcalculation of the base and subgrade moduli. It is also noted, from the borings report, that a few inches of old soil cement were left in place. However, the condition of this old soil cement is unknown. As such, the following two layer combinations were used in backcalculation analysis:

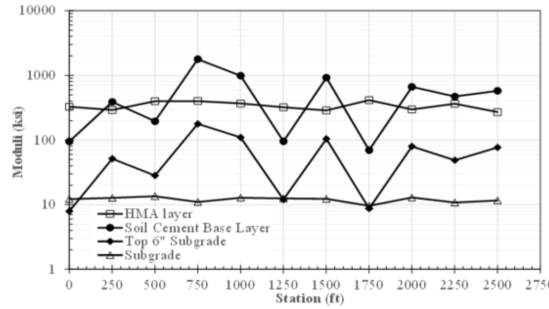
- Three-layer system (HMA, soil cement base, and subgrade)
- Four-layer system (HMA, soil cement base, top 6 in. of subgrade by considering the possible improvement provided by in-place old soil cement, and subgrade)

Figure 9 shows the backcalculation results for the various stations. Introducting a 6-in. top subgrade layer generally led to lower and more reasonbale soil cement base modulus values. However, its effect on the backcalculated base modulus at stations with low modulus values (i.e., station 0, 1250, and 1750 Northbound) is minimal (within approximately 8-percent difference). The backcalculated soil cement base values at these stations were below 100 ksi (200 ksi is a typical value for stabilized soil cement [2]).

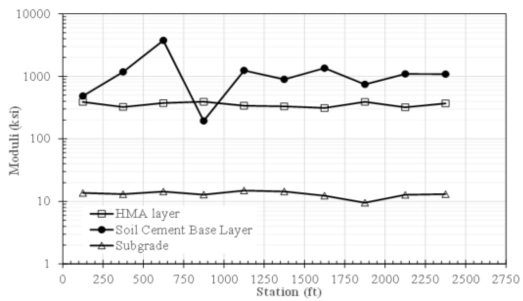
Figure 9. Backcalculated moduli for different layer combinations along the section



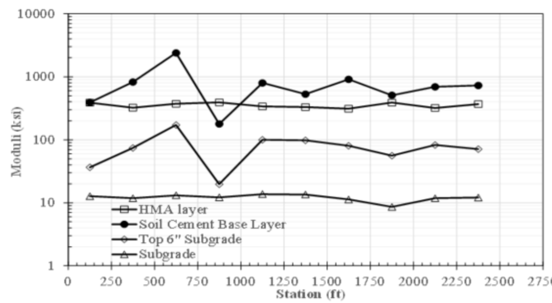
(a) Three-Layer System (Northbound)



(b) Four-Layer System (Northbound)



(c) Three-Layer System (Southbound)

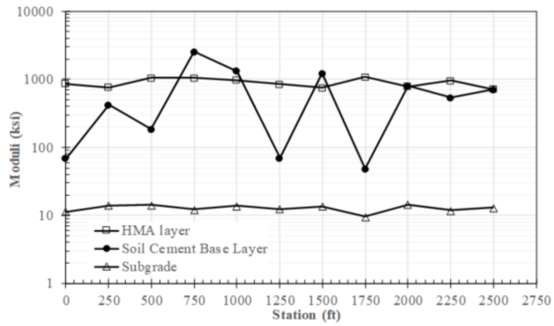


(d) Four-Layer System (Southbound)

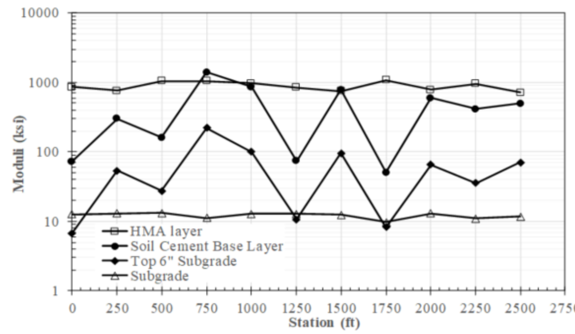
Repeated Backcalculation Analysis

The backcalculaton analysis was re-peformed based on the recommended asphalt layer modulus value of 1180 ksi determined from the laboratory evalaution of field cores. Figure 10 illustrates the repeated backcalculation results for the various stations. Similar observations were made in the repeated backcalculation as compared to the initial backcalculation analsysis; see Figures 9 and 10.

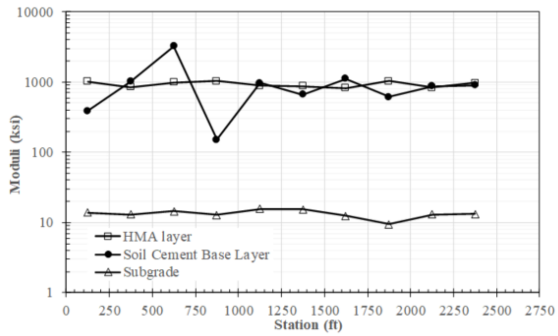
Figure 10. Repeated backcalculated moduli for different layer combinations along the section



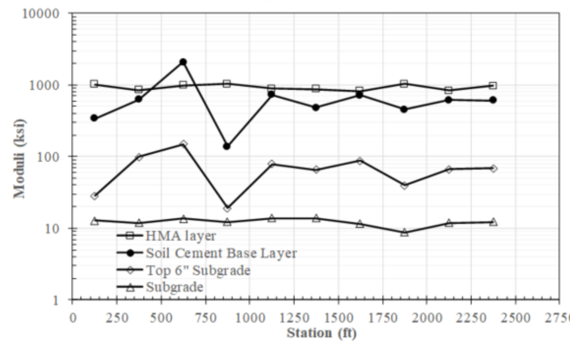
(a) Three-Layer System (Northbound)



(b) Four-Layer System (Northbound)



(c) Three-Layer System (Southbound)



(d) Four-Layer System (Southbound)

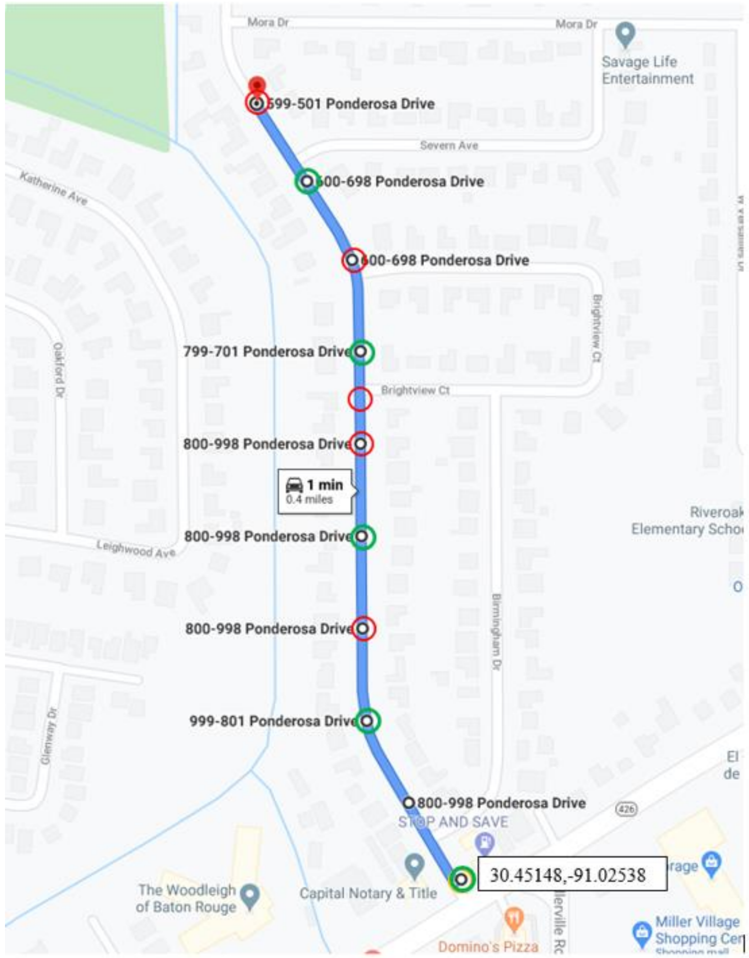
Summary and Recommendation from Pavement Evaluation

The FWD deflection data showed that the pavement structure is in poor condition at a couple of locations along the section with possible large variations in subgrade support. The backcaluated layer moduli show that the cement-treated soil base is also in poor condition at a couple of locations, but it is not easy to pinpoint the exact cause of the distress due to the limitations of the backcalculation method. It is worth noting that FWD tests are typically performed at areas without severe cracking and rutting.

Geotechnical Evaluation

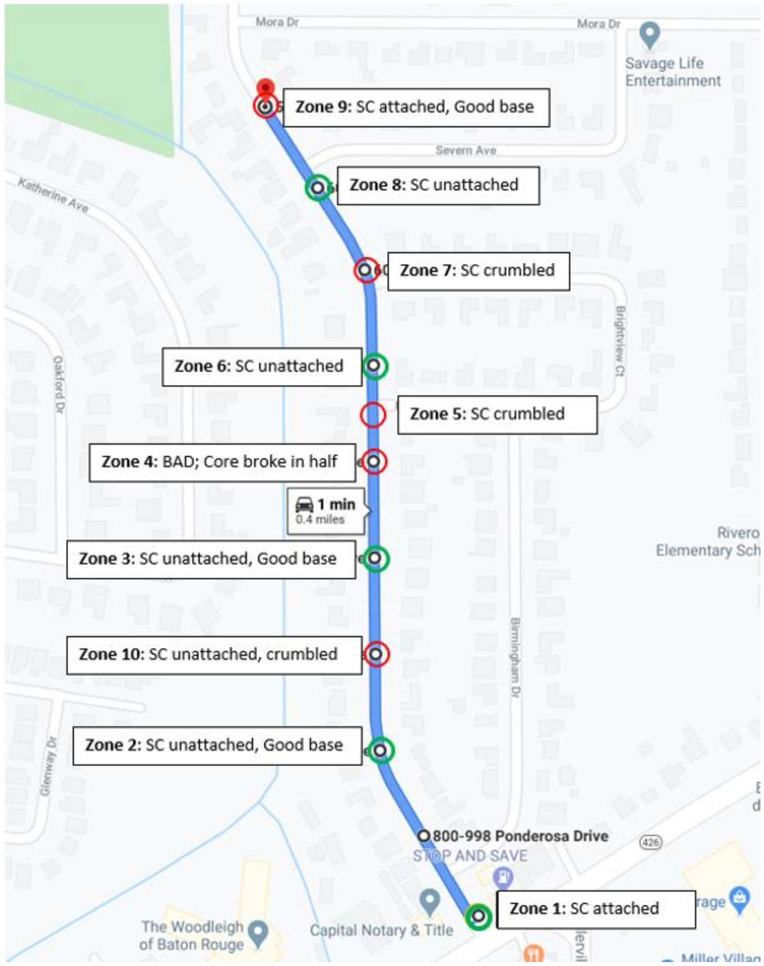
Based on the FWD tests, LTRC's pavement research group identified 10 locations along the pavement section, which were classified as "good" and bad "spots" based on the severity of the distress and the moduli of base and subgrade layers. These 10 locations were marked for further investigations by the Geotechnical and Asphalt groups. Figure 11 shows a map of 10 preliminary forensic investigation zones; good zones are denoted by the green circles and bad zones by red circles.

Figure 11. Preliminary map for Ponderosa Project



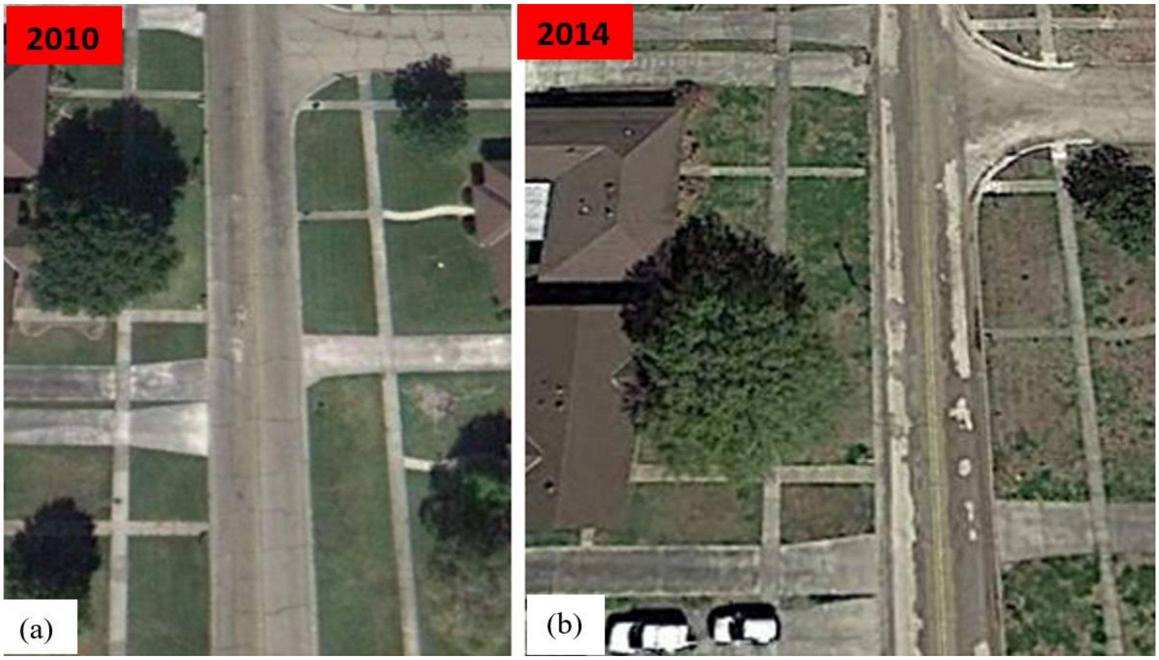
Prior to the geotechnical investigation, the asphalt crew performed 1-ft. coring operations at each of the 10 locations after ensuring that there were no underground utilities underneath the coring locations. Figure 12 indicates the asphalt core status (i.e., whether the soil cement base course was intact and attached to the asphalt layer). It is noted that the asphalt layer thickness varied from 1.5 to 2.8 in.

Figure 12. Core description map



A preliminary investigation utilizing Google Earth identified historical issues along the road. Figure 13 shows two photos of Ponderosa Drive from 2010 and 2014, respectively. In 2010, visible distress can be seen with white spots visible like those currently seen at the site; see Figure 13a. In March 2014, more white spots can be seen (possible evidence of milling operations); see Figure 13b. Figure 13a shows evidence of road distresses prior to rehabilitation in 2014, and the catastrophic flooding event of August 2016 (a non-named storm) that affected 21 parishes spanning from Lake Charles to the Louisiana-Mississippi border. In addition, January 2016 images show white spots after rehabilitation, indicating problems were recurring prior to the August 2016 flood; see Figures 4b and 4c.

Figure 13. Google Earth photos from (a) 2010 and (b) 2014



The geotechnical research team conducted dynamic cone penetration (DCP) tests at each identified location; see Figure 12. The 2-in. asphalt layer was drilled with a dry hammer-drill to access the top of the base course. The depth of each drilling was recorded prior to the start of DCP testing.

DCP tests were conducted to a depth of 3 ft. into the ground through the cement-treated soil base layer and the subgrade layers. The DCP consists of two steel rods coupled by an anvil. An 8-kg (17.2-lb.) sliding hammer is lifted and dropped to strike the anvil with a repeatable amount of energy. One additional note on Table 4 is that each of the results negates the first two blows of DCP testing to compensate for the falling debris caused by the drilling into the asphalt. The debris was not as compacted as the rest of the layer and would produce weaker results. This technique is in line with the Minnesota DOT DCP specifications for cutting out the first two blows.

DCP refusal is defined by LTRC as when there are 10 consecutive blows without advancement, or if the DCP hammer bounces due to soil that is too stiff. When DCP refusal occurred, another hole was drilled deeper to test the lower half of the base course and/or the subgrade underneath.

Zones 1-3 were “good zones” based on the FWD results, and they were located closest to the intersection of Ponderosa Drive and Old Hammond highway; see Figure 12. Zones 1-

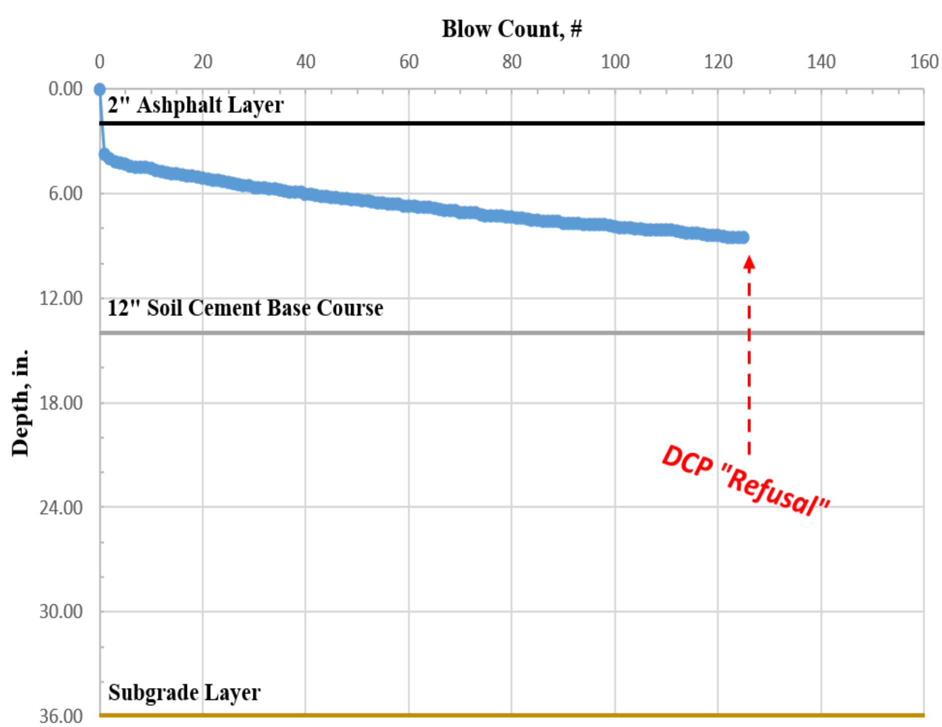
3 were not tested due to the time required for each “good” base course DCP test and the presence of the adjacent, often turning high-speed traffic, which was hazardous to the testing crew.

LTRC tested Zones 4-10 over a span of two days. Stiff base courses should be difficult to penetrate through and likely will be “refused” by the DCP. Weaker base course material can be penetrated, but it can take many blows with slow progression. Each tested zone is described below.

Zone 4

Zone 4 was initially deemed a “bad spot” based on the FWD results; however, the conducted DCP met refusal and was stopped. In response, a second test was conducted right next to the asphalt core hole, and the DCP test was performed the full 3 ft. Figure 14 shows “Zone 4 Take 1,” where a DCP refusal was recorded. The graph is a representation of the extent of DCP steel rod’s penetration due to each blow of the hammer. There are reference lines representing the presumed thickness of the asphalt and soil cement base layers, and the total rod length into the subgrade at 36 in.

Figure 14. Zone 4 Take 1 blow count versus depth



Figures 15 and 16 show “Zone 4 Take 2” blow count versus depth, and the location where the DCP was conducted to the full depth, respectively. The “Take 2” DCP at Zone 4 was initiated because “Take 1” DCP refusal contradicted the FWD results that indicated this area as a “bad zone. This DCP location is closer to the coring location, which was cored utilizing water. The coring technique may have affected the DCP results. This “Take 2” DCP test was able to penetrate through the soil cement in contrast to “Take 1” that reached refusal just 4-ft. away. This shows a possible inconsistency in the distribution of cement across the soil cement base.

Figure 15. Zone 4 Take 2 blow count versus depth

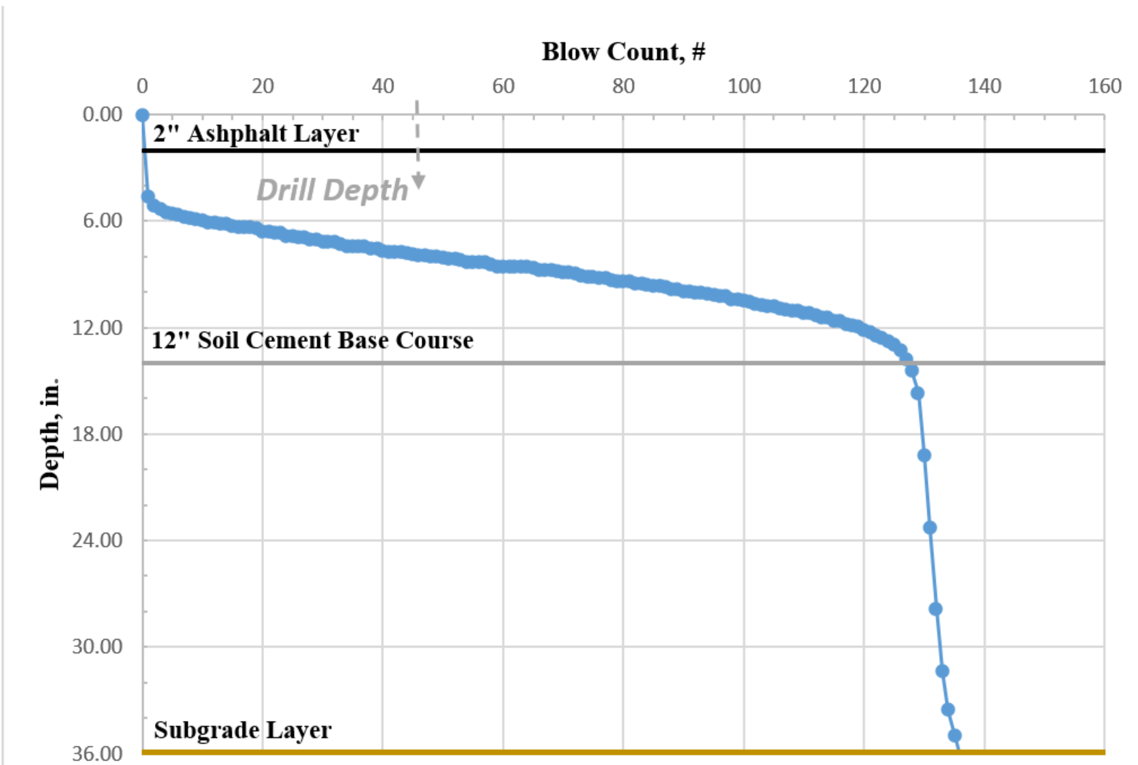
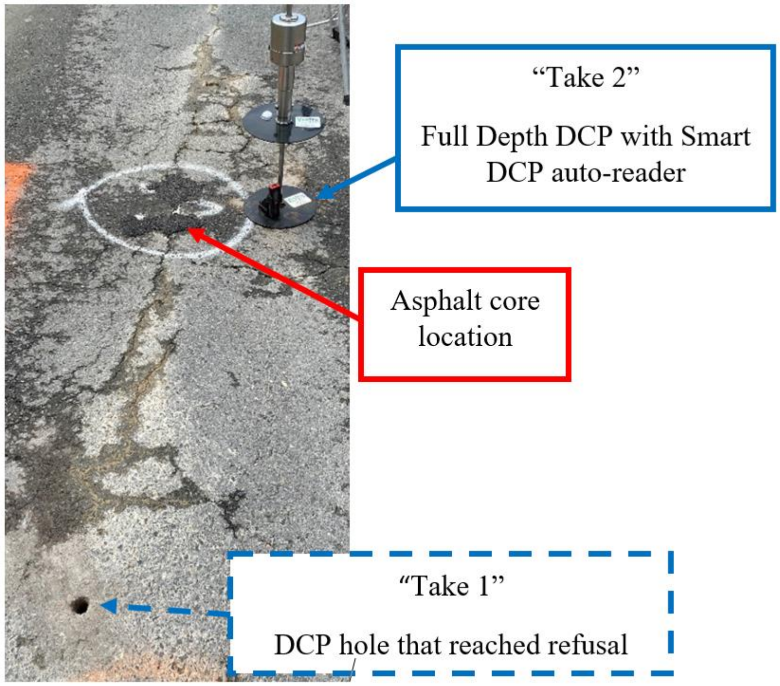


Figure 16. Photo of Zone 4 Take 2



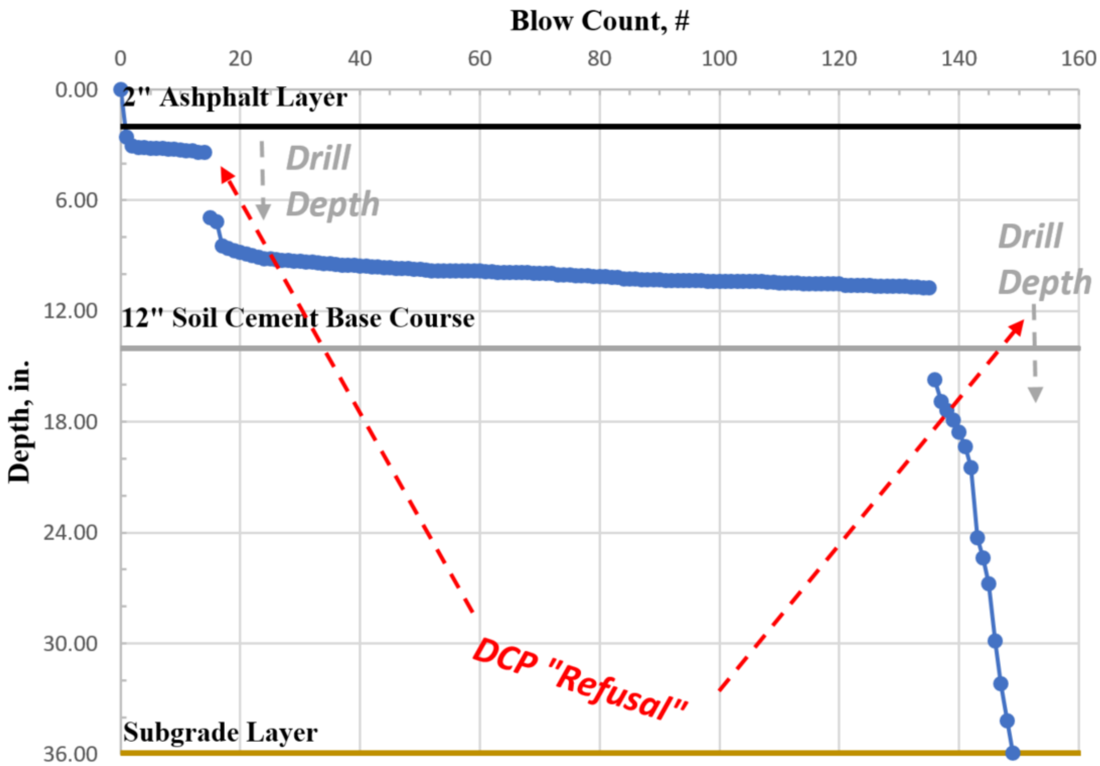
Zone 5

Zone 5 was initially deemed a "bad spot" based on FWD results, and the asphalt group confirmed that the core at this location broke in half, perhaps due to poor quality of the base course. The graph of blow count versus depth for this zone can be found in the Appendix. The DCP was performed 3 ft. away from the asphalt coring location. A photo of the test and coring locations is presented in the Appendix.

Zone 6

Zone 6 was initially deemed a "good spot" based on the FWD results. The DCP was performed within 2 ft. of the asphalt coring location. The soil cement base at this location was so stiff that it caused two DCP refusals. When the top of the soil cement reached refusal, the geotechnical group drilled to test the top of the lower half of the soil cement base course. When that also reached refusal, another hole was drilled to test the subgrade. Figure17 presents a combination of three DCP tests plotted together on the same graph. The two steep sections/gaps represent drilling efforts.

Figure 17. Zone 6 blow count versus depth



Zone 7

Zone 7 was initially deemed a "bad spot" based on the FWD results. The geotechnical group conducted the DCP test within 2 ft. of the asphalt coring location. The graph of blow count versus depth for this zone can be found in the Appendix. The soil cement base at this location was not as stiff, and the DCP test was conducted to its full depth without reaching refusal.

Zone 8

Zone 8 was initially deemed a "good spot" based on the FWD results. The geotechnical group conducted the DCP test within 2 ft. of the asphalt coring location. The graph of blow count versus depth for this zone can be found in the Appendix. The DCP test conducted at the top of the soil cement met refusal after 5 in. An adjacent hole was drilled to test the lower half of the soil cement base. This second test penetrated the lower portion of the base course and continued into the subgrade.

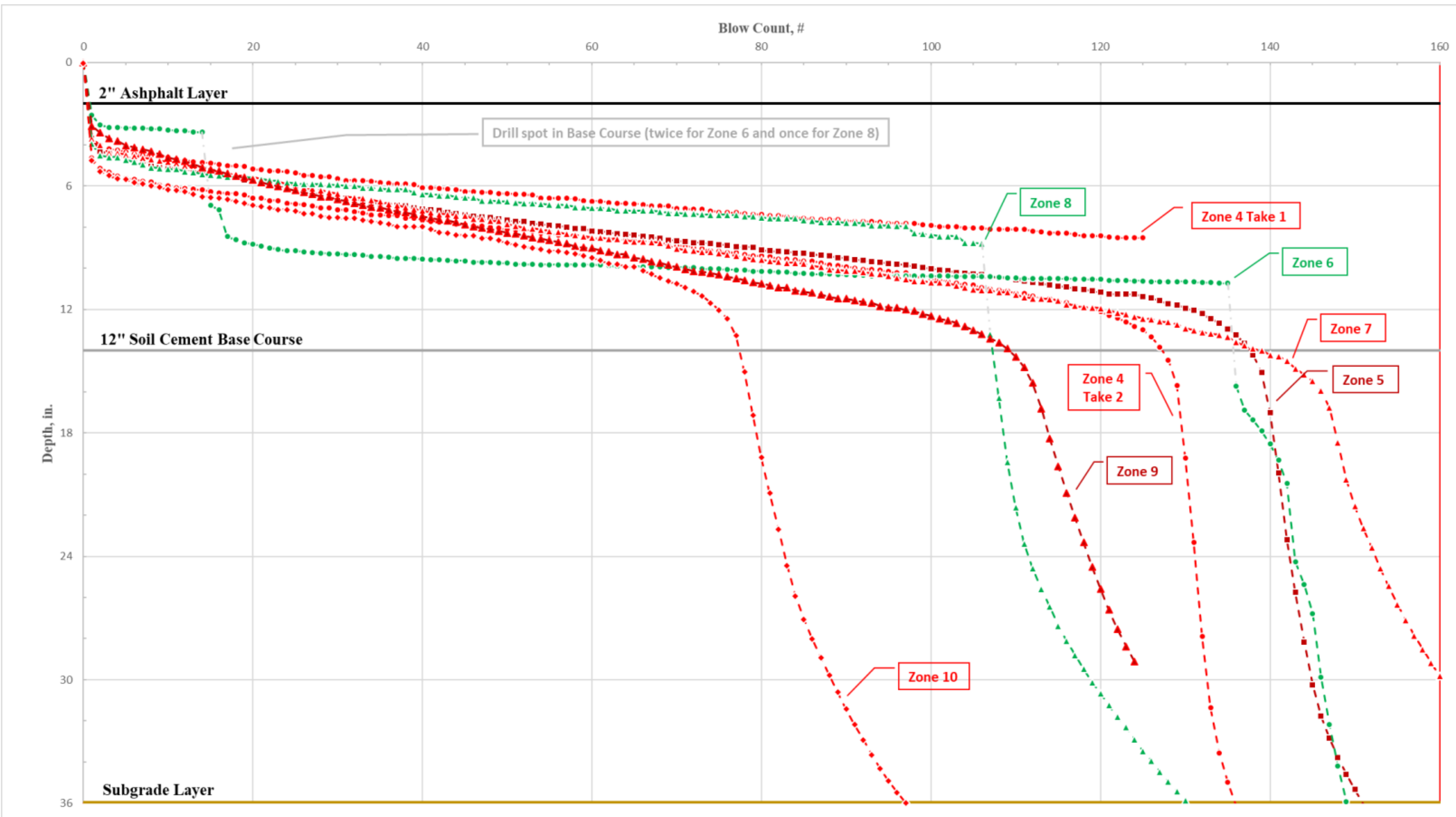
Zone 9

Zone 9 was initially deemed a “bad spot” based on the FWD results. Zone 9 is the first of two locations tested in the southbound lane on the Ponderosa Drive project. This was the geotechnical group’s first DCP performed on site, and it was within 2 ft. away from the asphalt coring location. The graph of blow count versus depth for this zone can be found in the Appendix. The soil cement base at this location was not as stiff as compared to other locations and the DCP test was conducted to the full depth without reaching refusal. This DCP only penetrated 2.5 ft. into the ground due to the initial concerns of possibly getting the device stuck into the ground.

Zone 10

Zone 10 was initially considered a “bad spot” based on the FWD results. The graph of blow count versus depth for this zone can be found in the Appendix. The DCP was performed about 4 ft. away from the asphalt coring location in the deepest depression of the right wheel path rut as seen in Appendix. The soil cement base at this location was the weakest of all locations tested in this investigation. The DCP test was conducted to the full depth without reaching refusal. This test did not reach 100 blows in total and crossed from soil cement to subgrade in under 80 blows. In contrast, all other full-depth DCPs crossed over into the subgrade at over 100 blows; see Figure 18.

Figure 18. DCP analysis for all zones



One distinctive value calculated by the DCP is the Dynamic Cone Penetration Index (DCPI). This is the average depth after each blow for a standalone DCP test, and the values for each zone are highlighted in the yellow columns of Table 4. Table 4 also shows the total drill depth, total depth tested in base and subgrade layers, and each of the blows counts per DCP test. Drill depth varied because the asphalt layers varied. The asphalt layer thickness was not exactly 2 in. throughout the roadway. The asphalt group indicated that asphalt cores had thicknesses that ranged from 1.5 in. to 2.8 in.

Table 4. DCPI analysis on Ponderosa Drive

Zone:	**Drill Depth (in.)**	**Test Layer**							
		Base Course (Soil Cement)				**Subgrade**			
Good/**Bad** (Based on FWD results)		**DCP Testing**		**DCPI (Δ)**		**DCP Testing**		**DCPI (Δ)**	
		Depth (in.)	Blow Count	in./blow	mm/blow	Depth (in.)	Blow Count	in./blow	mm/blow
Zone 4	3.7	4.3	122	0.04	1.0	Not Tested/DCP Refusal			
Zone 4 (2)	4.6	8.7	125	0.07	1.8	23.1	9	2.57	65.3
Zone 5	3.7	9.6	135	0.07	1.8	23.3	15	1.55	39.4
Zone 6	2.6	0.2	11	0.02	0.5	Not Tested/DCP Refusal			
	6.9	2.3	118	0.02	0.5	Not Tested/DCP Refusal			
	15.7	Drilled through to test subgrade >				21.5	13	1.65	41.9
Zone 7	3.7	9.6	137	0.07	1.8	22.6	34	0.66	16.8
Zone 8	3.9	4.2	104	0.04	1.0	Not Tested/DCP Refusal			
	13.2	Drilled through to test subgrade >				21.9	23	0.95	24.1
Zone 9	3.1	10.3	107	0.10	2.5	15.2	15	1.01	25.7
Zone 10	4.8	8.5	75	0.11	2.8	22.0	20	1.10	27.9

Smaller DCPI values indicate stiffer material. Cement-treated soil base course analysis shows that Zones 4 (Take 1), 6, and 8 all produced the smallest average DCPIs with a range from 0.02 – 0.04 in./blow (0.5 – 1.0 mm/blow). For the rest of the zones, the average DCPIs of the base course ranged from 0.07 – 0.11 in./blow (1.78 - 2.79 mm/blow). The subgrade DCPIs varied inconsistently for both "good" and "bad" zones.

Summary and Conclusions from Geotechnical Evaluation

After analysis and discussion, the geotechnical research team found that, when comparing the Pavement group's labeled "good" and "bad" zones, the soil cement base course was highly non-homogeneous along the roadway. Zone 4 location showed evidence of widely differing results within 4 ft. This likely shows that the amount and/or distribution of cement was not consistent across the road's base course. Google Earth aerial photos, show pavement distresses in 2010 and 2014. The two photos show evidence of road distresses prior to rehabilitation and the catastrophic flooding event of August 2016 (a non-named storm) that affected 21 parishes spanning from Lake Charles to the Louisiana-Mississippi border; see Figure 13. The white spots and apparent patching activity in the January 2016 images show distresses after rehabilitation and prior to the catastrophic flood; see Figures 4b and 4c. The cut-through nature of this road (from Florida to Millerville), especially after the flood of 2016, may have exacerbated pavement distresses.

Stiff base courses should be difficult to penetrate through, and they would likely "refuse" the DCP advancement. The DCP reached refusal criteria at Zones 4 (Take 1), 6, and 8, which indicated the base was a very stiff soil cement. This is not the case for the other zones: 4 (Take 2), 5, 7, 9, and 10. Typically, all soil cement base courses should be strong enough to prevent the DCP penetration. The average DCPI of the base course for the zones that reached DCP refusal was less than 0.04 in./blow (1.02 mm/blow).

Asphalt Mixture Evaluation

A total of eight asphalt field cores were obtained for the characterization of the visco-elastic properties of the asphalt layer utilizing the indirect tensile dynamic modulus (IDT|E*|) test.

Indirect Tensile Dynamic Modulus (IDT |E*|) Test

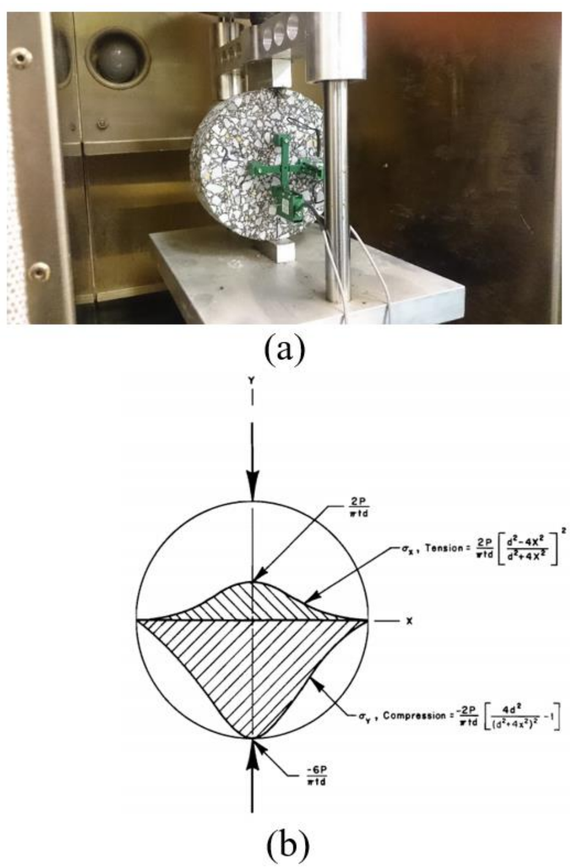
The IDT |E*| test was conducted according to AASHTO TP 131, "Proposed Standard Test Method for Determining the Dynamic Modulus of Asphalt Mixtures Using the Indirect Tension Test" [3]. The IDT|E*| test applies a sinusoidal compressive stress to the diametric axis of an unconfined cylindrical field core specimen; see Figure 19. This test was conducted at three temperatures: -10, 10, and 30°C (14, 50, and 86°F). For each test temperature, asphalt mixture specimens were tested at five loading frequencies: 10, 5, 1, 0.5, and 0.1 Hz. The compressive stress applied to the test specimen results in tensile stress and strain along the horizontal axis of the specimen. A target tensile strain level of 40 to 60 microstrain was maintained to keep the specimens in the linear viscoelastic region. The dynamic modulus was computed using the following equation:

$$|\boldsymbol{E}^*| = 2\left(\frac{\boldsymbol{P_0}}{\boldsymbol{\pi a d}}\right)\left(\frac{\boldsymbol{\beta_1 \gamma_2 - \beta_2 \gamma_1}}{\boldsymbol{\gamma_2 V_0 - \beta_2 U_0}}\right) \tag{1}$$

Where,
|E*| = Dynamic complex modulus
P_0 = Load amplitude,
U_0 = Horizontal displacement amplitude,
V_0 = Vertical displacement amplitude,
a = Loading strip width,
d = Specimen diameter, and
$\beta_1, \beta_2, \gamma_1$, and γ_2 = geometric constants

The geometric constants are functions of gauge length, specimen diameter, and loading strip width [4]. The dynamic modulus of asphalt mixtures obtained at various frequencies and temperatures were combined into a master curve using the time-temperature superposition principle [4]. Four asphalt mixture specimens were evaluated. The other four specimens were discarded because of non-uniform thickness and uneven cored surface.

Figure 19. (a) IDT |E*| test setup and (b) stress distribution along X-axis



Results and Discussion

Figure 20 presents the dynamic modulus master curves for asphalt field cores obtained from the pavement section evaluated. The master curves were constructed at a reference temperature of 10°C. It is noted that the dynamic modulus for the pavement section evaluated is within the range of values reported by other researchers for Louisiana level 2 asphalt mixtures ([5]- [6]). Figure 21 shows the $|E^*|_{54.4°C,\ 5Hz}$ values for the test section evaluated. It is worth noting that $|E^*|_{54°C,\ 5Hz}$ values were extrapolated from the dynamic modulus master curves. The $|E^*|_{54.4°C,\ 5Hz}$ parameter is a good indicator of mixture rutting performance [7]. Higher $|E^*|_{54.4°C,\ 5Hz}$ values indicate higher rutting performance and vice versa. For the field cores evaluated, $E^*|_{54.4°C,\ 5Hz}$ values were found to be higher than values reported by Mohammad et al. after characterizing commonly used level 1 and 2 asphalt mixtures in Louisiana [7]. This observation is attributed to the fact the asphalt layer was placed in 2014 and may have undergone substantial level of long-term aging [8].

Figure 20. Dynamic modulus master curves

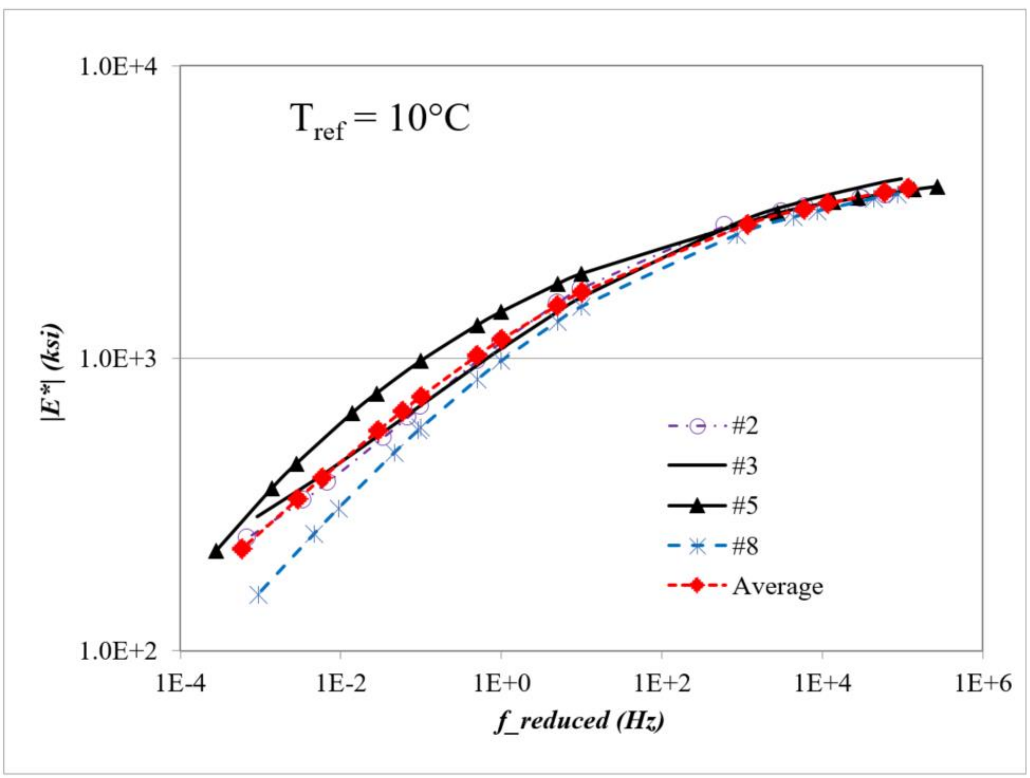
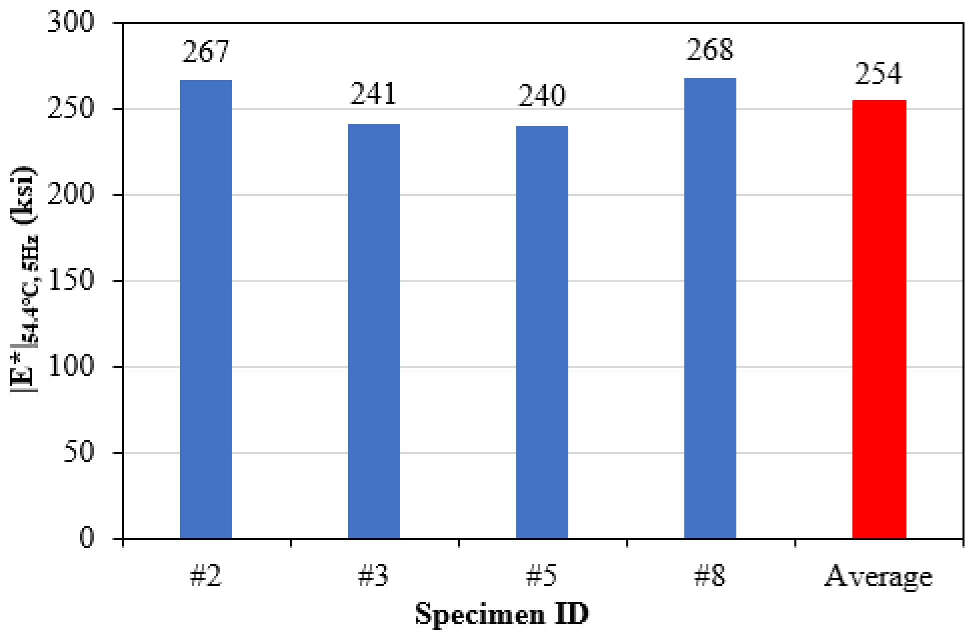


Figure 21. $|E^*|_{54.4°C,\ 5Hz}$ values



Estimation of Average Modulus of Asphalt Layer

The average dynamic modulus master curve was used to estimate the modulus of asphalt layer at the prevailing traffic speed and temperature. A loading frequency was computed to simulate the actual occurrence in the field utilizing a technique documented elsewhere [9]. The stress pulse was assumed to be a haversine with duration, which depends on traffic speed on the pavement section [9]. Based on a traffic speed of 30 mph and an assumed axle load of 9 ksi, the modulus of the asphalt layer was estimated to be 1180 ksi for a pavement surface temperature of 25°C (77°F) [9]. It is noted that a subgrade modulus of 8413 psi, which is a typical modulus value for subgrades in the East Baton Rouge Parish was used in the computation of the asphalt modulus [10].

Summary and Conclusions from Asphalt Mixture Evaluation

Asphalt field cores obtained from the pavement section were found to exhibit stiffness values within the range of values reported in the literature ([7], [8]). In addition, the asphalt field cores exhibited $E^*|_{54.4°C,\ 5Hz}$ values higher than those reported for commonly used level 1 and 2 asphalt mixtures in Louisiana, probably due to the long-term aging of the pavement layer. It is recommended that asphalt modulus value of 1180 ksi is used to re-run the backcalculation analysis of the FWD test data.

Conclusions and Recommendations

Based on the forensic evaluation, the source of the rapid deterioration is possibly attributed to the following:

- relatively thin pavement structure and higher levels of traffic due to the cut through nature of the road (from Florida Boulevard to Millerville);
- non-uniform application of cement treatment across the road's base course resulting in non-homogeneous pavement structure with weak spots;
- the weakening of the pavement structure due to the August 2016 floods; and
- the subsequent use of the road section by heavier waste disposal trucks to cart away household items destroyed during the floods.

Therefore, it is recommended that the entire pavement section be reconstructed. In addition, the LTRC team recommends that a traffic survey is conducted to accurately estimate the traffic prior to the design and reconstruction of the pavement section.

Acronyms, Abbreviations, and Symbols

Term	Description
AASHTO	American Association of State Highway and Transportation Officials
BCI	Base Curvature Index
BDI	Base Damage Index
˚C	Degree Celsius
cm	centimeter(s)
D_0	Deflection at Center of Loading Plate
DCP	Dynamic Cone Penetrometer
DCPI	Dynamic Cone Penetration Index
˚F	Degree Fahrenheit
FHWA	Federal Highway Administration
ft.	foot (feet)
Hz	Hertz
in.	inch(es)
IDT|E*|	Indirect Tensile Dynamic Modulus
kg	Kilogram
ksi	Kilo pound per square inch
DOTD	Department of Transportation and Development
LTRC	Louisiana Transportation Research Center
lb.	pound(s)
m	meter(s)
mm	millimeter(s)
mph	miles per hour
RoC	radius of curvature
SCI	Surface Curvature Index

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of Laboratory Mixture Aging to Asphalt Mixture Performance," Transportation Research Circular E-C234, Washington DC, 2018.

[9] ARA, "Guide for the mechanistic-empirical design of new and rehabilitated pavement structures; Final Report. NCHRP 1-37A," Transportation Research Board, National Research Council, Washington DC, 2004.

[10] Louisiana Departement of Transportation and Development, "Soil Support and Resilient Modulus of Subgrade Soils in Louisiana, Baton Rouge, LA," Baton Rouge.

Appendix

Figure 22. Zone 5 blow count versus depth

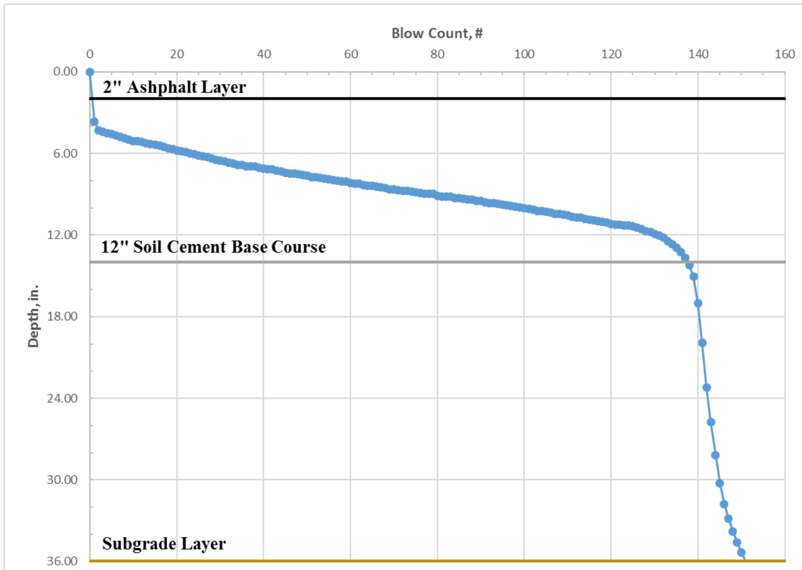

Figure 23. Photo of zone 5 DCP and core location

Figure 24. Zone 7 blow count versus depth

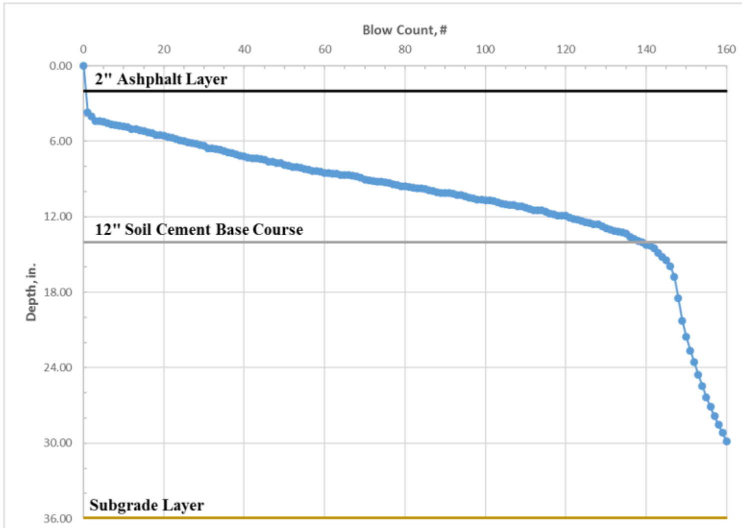

Figure 25. Zone 8 blow count versus depth

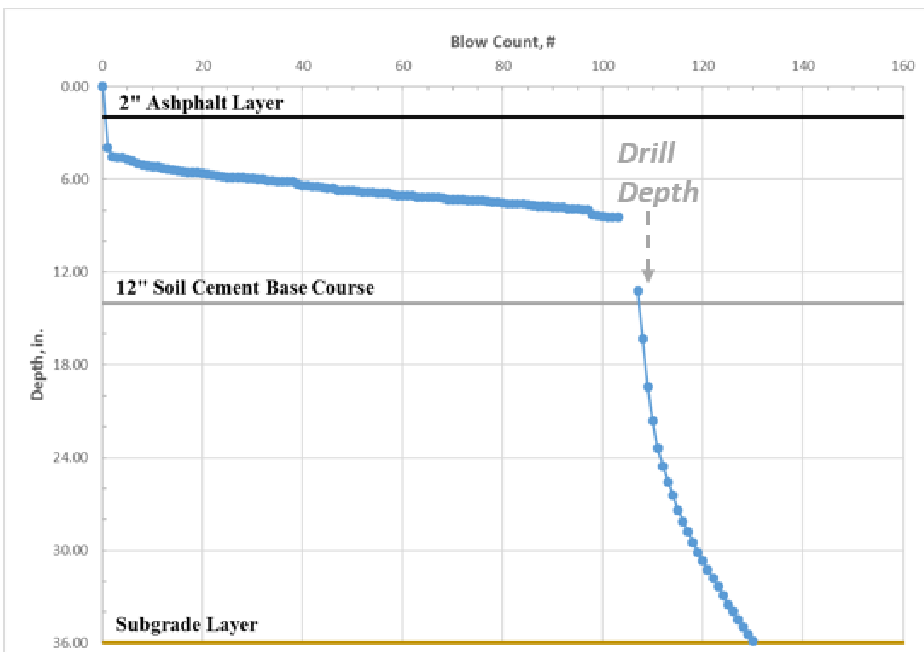

Figure 26. Zone 9 blow count versus depth

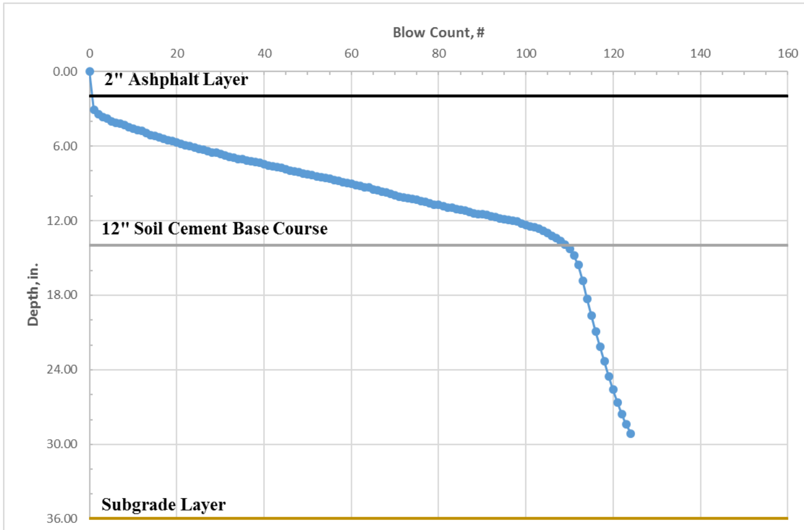

Figure 27. Zone 10 blow count versus depth

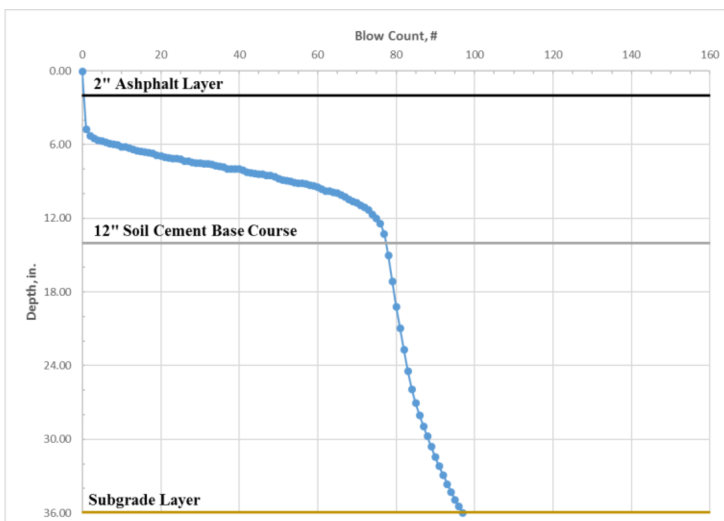

Figure 28. Photo of Zone 10 DCP and core location