Technical Memorandum

To: Jeff Uhlmeyer

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Re. Forensic Desktop Study Report: New Mexico LTPP Test Sections 35_0801 and 35_0802

INTRODUCTION

The Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS) test sections 35_0801 and 35_0802[1] were nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." Both sections are a part of the SPS-8 experiment, which studies the environmental effects on pavement in the absence of heavy loads. In general, the SPS-8 test sections have a structural capacity much greater than the amount of heavy truck loading expected at each project site. Therefore, with low levels of traffic reported, the loading conditions at the site should not lead to the development of load related fatigue cracking. However, cracking patterns in the wheel path typically associated with the fatigue cracking mechanism have been identified on both sections and, contrary to intuition, more so on the thicker test section (35_0802). Although there are differences in the level of reported cracking, both test sections have comparable amounts of longitudinal (wheel path and non-wheel path) and transverse cracking as well as similar IRI and rutting values. The focus of the proposed investigation is to (1) identify the reason(s) for the presence of (actual or apparent) fatigue cracking in the two test sections and (2) to compare the performance (surface distresses, rutting, IRI and deflections) of the two test sections, which are subjected to the same traffic and climatic conditions and constructed with the same materials and on similar subgrade soil.

[1] First two digits in test section number represent the State Code [35 =New Mexico]. For LTPP GPS test sections, the final four digits are unique within each State/Province, and they were assigned at the time the test section was accepted into the LTPP program. For LTPP Specific Pavement Studies (SPS) test sections, the second set of two numbers indicates the Project Code (e.g., 08= SPS-8), and the final set of two numbers represents the test section number on that project (e.g., 01).

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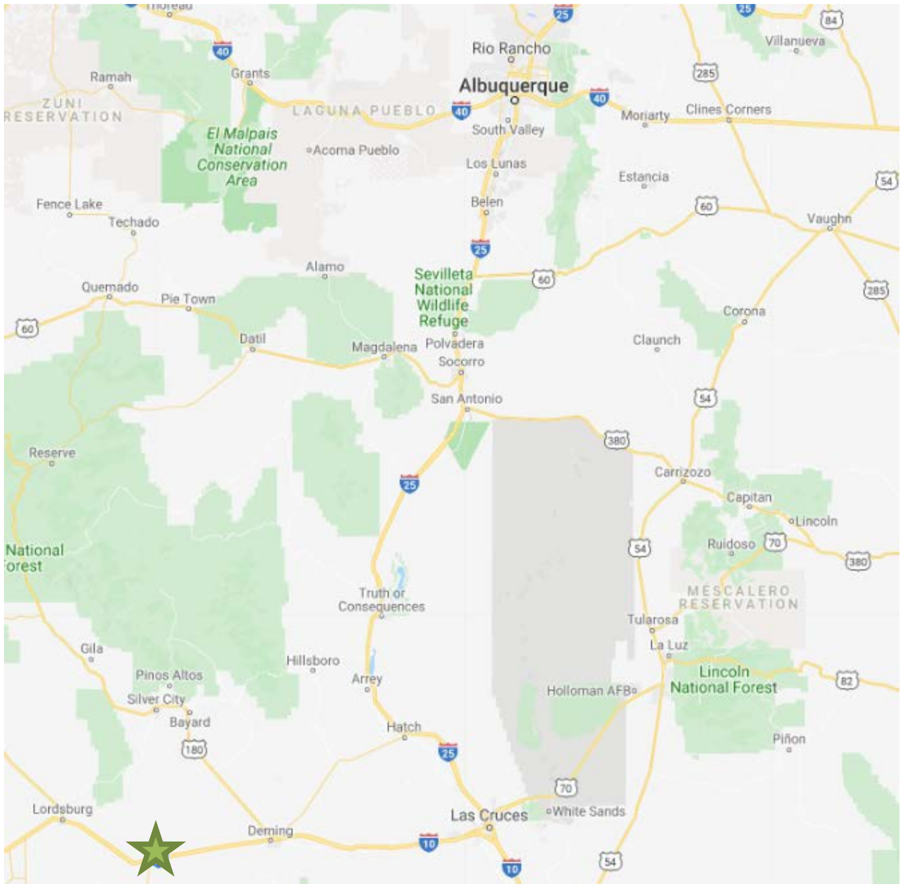
SITE DESCRIPTION

LTPP test sections 35_0801 and 35_0802 are located on the Interstate Highway 10 (I-10) Frontage Road (FR), eastbound, in Grant County, New Mexico. I-10 FR is a remote rural local collector with one lane in each direction of traffic. Both sections are in a Dry-No Freeze climatic zone with an average annual precipitation ranging between 7.3 inches (2003) and 15.4 inches (2011) and an annual average air freezing index ranging between 0 deg F deg days (multiple years) and 83 deg F deg days (2011) during the performance period being assessed (1996 to 2017). The coordinates of the test sections 35_0801 and 35_0802 are 32.19315, -108.30111, and 32.19354, -108.29852, respectively. The elevation of the test sections is over 4,000 feet. Photograph 1 and Photograph 2 show test sections 35_0801 and 35_0802 looking eastbound in 2014, while Map 1 shows the geographical location of the test sections relative to Albuquerque, New Mexico.

Photograph 1. Picture of test section 35_0801 (start of section) in 2014 looking eastbound.

Photograph 2. Picture of test section 35_0802 (beginning of section) in 2014 looking eastbound.



Map 1. Geographical location of test sections relative to Albuquerque, New Mexico.

BASELINE PAVEMENT HISTORY

In this section, data related to the base-line pavement history of the test sections—including the history of the pavement structure and its structural capacity, climate, traffic, pavement distresses, rutting and roughness—are provided.

Pavement Structure and Construction History

The initial pavement structure for both test sections was constructed in 1995 and incorporated into the LTPP program that same year as part of the SPS-8 Study of Environmental Effects in the Absence of Heavy Loads. The original layer structures for each section are detailed in Table 1; this information corresponds to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. The layer structure has remained as is for both test sections (i.e., no maintenance or rehabilitation applied) since their construction in 1995, and consequently there are no additional CN events beyond CN = 1.

Table 1. Pavement structure.

Layer Number	Layer Type	Test section 35_0801		Test section 35_0802	
		Thickness (in.)	Material Code Description	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	Semi-infinite	216—Coarse-Grained Soil: Clayey Sand	0.0	217-Coarse-Grained Soil: Clayey Sand with Gravel
2	Unbound (granular) subbase	9.7	308—Soil-Aggregate Mixture (Predominantly Coarse-Grained)	12.7	308—Soil-Aggregate Mixture (Predominantly Coarse-Grained)
3	Asphalt Concrete (AC) layer	4.2	1—Hot Mixed, Hot Laid AC, Dense Graded	7.0	1—Hot Mixed, Hot Laid AC, Dense Graded

Pavement Structural Properties

Figure 1 shows the average Falling Weight Deflectometer (FWD) deflection under the nominal 9,000-pound load plate over time for each test section. The deflection of the sensor located in the center of the load plate is a general indication of the total "strength" or response of all layers in the pavement structure to a vertically applied load. This deflection can be influenced by pavement temperature at the time of testing, precipitation, and changes in pavement structure. As shown, the original pavement structure of the test sections had a maximum deflection of 9.6 mils (for section 35_0801) and 6.3 mils (for section 35_0802) in 1997, which increased over time to 21.1 mils (for section 35_0801) and 16.1 mils (for section 35_0802) by 2012 when deflection measurements reached a maximum for both sections. These deflections show the pavement structure is deteriorating with time under local climatic and traffic conditions. However, while an increase in deflection due to pavement deterioration is expected, the average deflections observed more than doubled between the time the pavement was constructed in 1995 and the last time an FWD test was conducted in 2016. The magnitude of deflection reported for each test section is reflective of each section's pavement structure; section 35_0801, which is less thick than the second test section, has higher measured deflections throughout the study's duration as the section has less structural support than section 35_0802. An exception to this observation occurred in 2002 when section 35_0802 reported higher deflections than section 35_0801; this may be a result of the conditions at the time of testing as temperature and environmental elements may have influenced the deflections recorded.

Figure 1. Average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load over time.

Table 2, Figure 2, and Figure 3 show the layer moduli backcalculated (using EVERCALC 5.0 software) from the deflection data measured between February 1997 and April 2005 (five rounds of testing), which were available in the LTPP database. The pavement structure for section 35_0801 was modeled as a 4.2-inch AC layer over an unbound layer made up of 9.7 inches of coarse granular base, the top 24 inches of coarse subgrade (intended to address issues such as shallow depth to bedrock or groundwater table) and a semi-infinite layer of coarse subgrade. Section 35_0802 was modeled as a 7-inch AC layer over 24 inches of coarse granular base and a semi-infinite coarse subgrade. Although the pavement structure for section 35_0802 included 12.7 inches of granular unbound subbase, for modeling purposes it was combined with and modeled to be 24 inches of coarse granular base. It is hypothesized that section 35_0802 was modeled in this way by the LTPP data analysis contractor to address unrealistic backcalculated moduli that resulted from modeling the pavement as four layers.

Based on the backcalculation results, the following observations can be made about the test sections.

1. While the average modulus of the asphalt layer (Layer 1) of section 35_0801 is initially less than section 35_0802, by 2005, the average modulus of section 35_0801 is nearly double the modulus of section 35_0802. The average modulus of section 35_0801 and section 35_0802 was 458 ksi and 751 ksi in 1997 and 1,724 ksi and 937 ksi in 2005 respectively. The average modulus for section 35_0801 is unreasonably high by 2002 indicating the asphalt is stiffening over time or that a compensating layer effect is occurring in the backcalculation.

2. The average modulus of the coarse granular base (Layer 2) of section 35_0801 is consistently greater than the average modulus for the same layer in section 35_0802, and the moduli reported for the section are unreasonably high. The average modulus during the 1997 collection date was 108 ksi and 68 ksi for section 35_0801 and section 35_0802, respectively, while in 2005 the average modulus was 113 ksi and 26 ksi, respectively. Additionally, as depicted in Figure 3, the average modulus for the granular base of section 35_0802 drops after the 2000 data collection. This behavior may indicate an increase in moisture in this layer over time.

Table 2. Backcalculated layer moduli over time.

Test Date	Average Daily Temperature on Day of Testing (deg F)	Layer	Test section 35_0801		Test section 35_0802	
			Thickness (inches)	Modulus (ksi)	Thickness (inches)	Modulus (ksi)
2/5/1997	46.4	1—Asphalt Concrete Layer	4.2	458	7	751
		2—Coarse Granular Base	9.7	108	24	68
		3—Coarse Subgrade	24	34	Infinite	35
		4—Coarse Subgrade	Infinite	31		
5/26/2000	75.1	1—Asphalt Concrete Layer	4.2	277	7	795
		2—Coarse Granular Base	9.7	145	24	91
		3—Coarse Subgrade	24	29	Infinite	27
		4—Coarse Subgrade	Infinite	31		
5/3/2002	63.1	1—Asphalt Concrete Layer	4.2	1,818	7	1,080
		2—Coarse Granular Base	9.7	118	24	49
		3—Coarse Subgrade	24	26	Infinite	22
		4—Coarse Subgrade	Infinite	24		
3/17/2004	60.38	1—Asphalt Concrete Layer	4.2	1,090	7	734
		2—Coarse Granular Base	9.7	129	24	27
		3—Coarse Subgrade	24	27	Infinite	19
		4—Coarse Subgrade	Infinite	28		
4/9/2005	54.29	1—Asphalt Concrete Layer	4.2	1,724	7	937
		2—Coarse Granular Base	9.7	113	24	26
		3—Coarse Subgrade	24	33	Infinite	19
		4—Coarse Subgrade	Infinite	27		

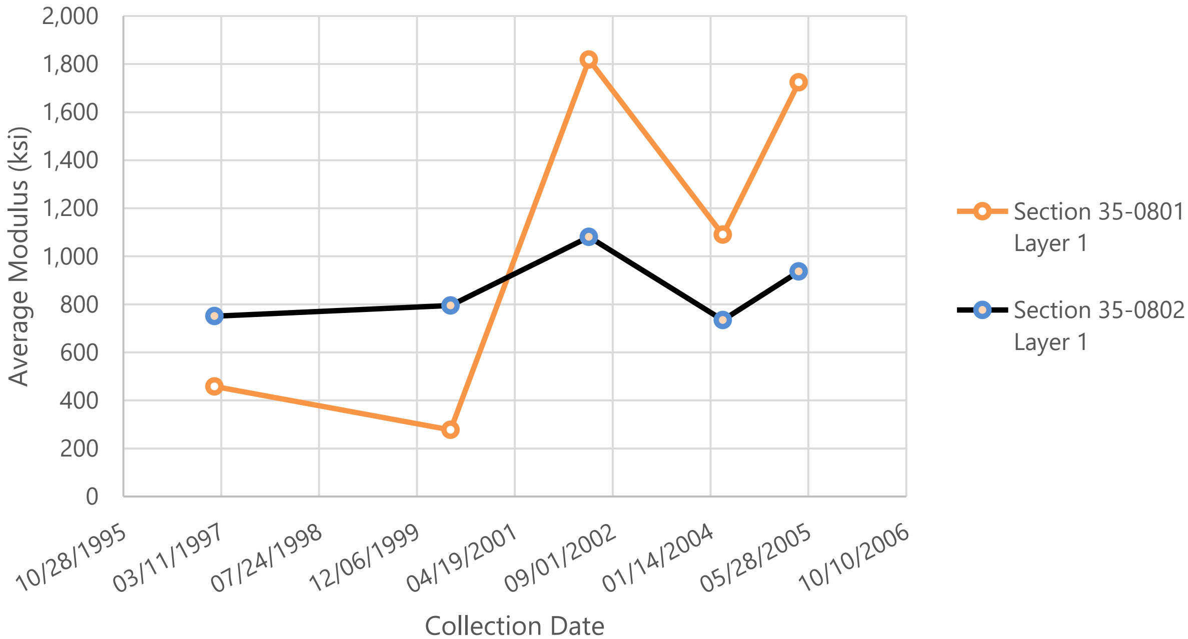


Figure 2. Average modulus of AC layers for test sections over time.

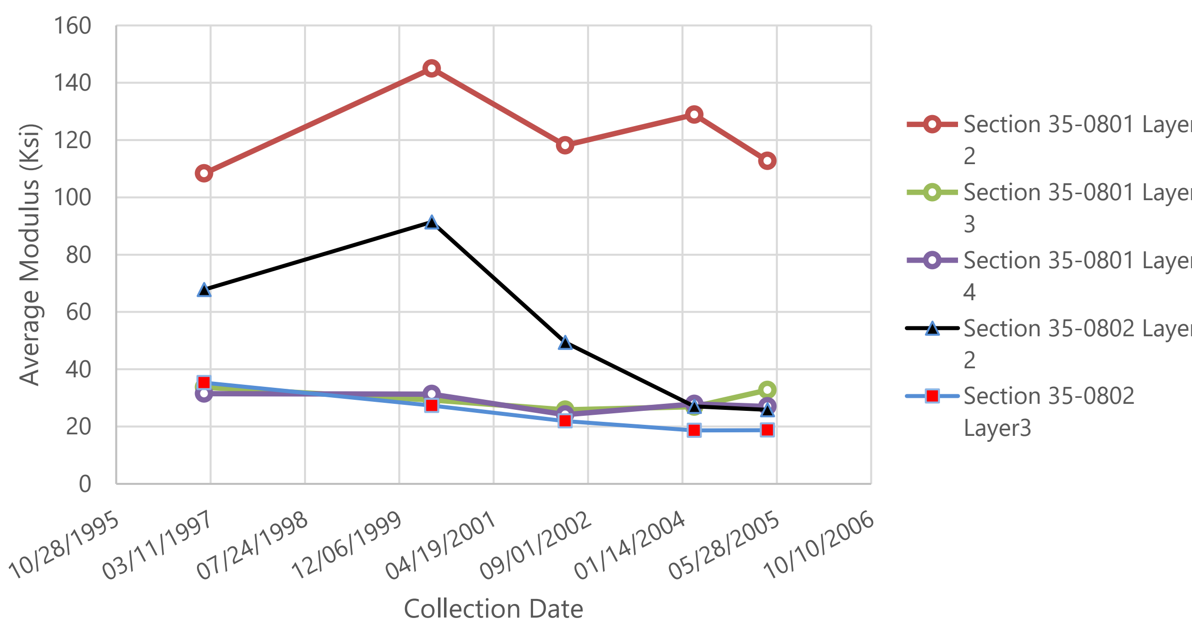


Figure 3. Average modulus of base and subgrade layers for test sections over time.

3. The average moduli of the two test sections for the coarse subgrade layers is similar throughout time. The reported subgrade modulus of section 35_0801 was 34 ksi and 31 ksi in 1997, while for section 35_0802 it was 35 ksi. The reported subgrade modulus of section 35_0801 was 33 ksi and 27 ksi in 1997, while for section 35_0802 it was 19 ksi.

Based on the results provided, one potential cause for the unreasonable values reported for the asphalt and base layers is that compensating layer effects occurred during backcalculation. The compensating layer effect is the result of the way the modulus of each layer is calculated, from the subgrade upward. While the values of the subgrade moduli appear to be reasonable, the values collected in subsequent layers (i.e. base and asphalt layers) result in calculated values that over or underestimate the modulus as the model aims to compensate for errors incurred through the iterative backcalculation process. Therefore, high values for a base layer may result in low moduli values for the asphalt layer as is the case for section 35_0802 between 1997 and 2000. However, the compensating layer effect does not describe all behaviors described by the data after 2000. Another round of FWD testing and moduli analysis is recommended to better understand and confirm these results.

Material Properties

In addition to assessing the structural performance of the test sections over time, the material properties at the construction of the test sites were also assessed to ensure the AC mix was consistent between the two sections being assessed. To do so, both the resilient modulus and the E^* of the asphalt layers of section 35_0801 and 35_0802 were analyzed.

The resilient modulus is critical in understanding the structural response to loading within pavement structures. Resilient modulus testing for LTPP sites adheres to LTPP Protocol P07, which assesses the resilient modulus of a pavement under different stress states and temperatures. Both test segments were assessed at 41°, 77°, and 104° Fahrenheit resulting in the resilient moduli detailed in the Table 3. While the resilient moduli of the two sections are not the same, the values of each are reasonably close to each other. When comparing the resilient modulus calculated at 77 degrees Fahrenheit with the backcalculated moduli in the previous section (where temperatures ranged from 46.4 to 75.1 degrees Fahrenheit), the resilient modulus values are lower than the backcalculated values. Except for 1997 and 2000, the moduli calculated for the asphalt layer using the backcalculation method are more aligned with or higher than the resilient moduli that were calculated for 41 degrees Fahrenheit. The discrepancy between these values may further indicate the backcalculated moduli and the compensating layer effect that the method results in are skewed to moduli values that are higher than expected.

Table 3. Laboratory measured resilient moduli for section 35_0801 and section 35_0802 at different temperatures.

Temperature (deg F)	Resilient Modulus (ksi)	
	Section 35_0801	Section 35_0802
41	1,126.65	1,065.75
77	345.10	397.30
104	136.30	137.75

The E^* or dynamic moduli of the two test sections was also assessed. The dynamic modulus describes a pavement's strain response at different loading rates and temperatures. The dynamic modulus for both test sections was calculated using varying temperatures, varying frequencies of loading, and a resilient modulus Artificial Neural Network (ANN) as summarized in Table 4. In each case, the dynamic modulus of section 35_0801 and 35_0802 was the same under each set of conditions. When comparing the dynamic modulus with the backcalculated moduli values in the previous section, the dynamic modulus values calculated at 70 degrees Fahrenheit are lower than the backcalculated values of the asphalt layer. The dynamic modulus is, however, similar to the resilient modulus calculated for 77 degrees Fahrenheit. This further suggests the compensating layer effect in the backcalculation process is skewing the moduli, making the values higher than expected.

Table 4. Estimated dynamic moduli for section 35_0801 and section 35_0802 at different temperatures and frequencies.

Temperature	Frequency (Hz)	Dynamic modulus (psi)
14	0.1	2,400,596
	0.5	2,748,515
	1	2,880,292
	5	3,143,091
	10	3,238,384
	25	3,349,119
40	0.1	986,513
	0.5	1,379,575
	1	1,560,077
	5	1,982,504
	10	2,158,904
	25	2,381,321
70	0.1	186,636
	0.5	318,226
	1	396,634
	5	637,754
	10	767,724
	25	961,923
100	0.1	41,778
	0.5	66,814
	1	83,132
	5	141,437
	10	178,659
	25	242,919
130	0.1	17,891
	0.5	24,309
	1	28,349
	5	42617
	10	51,854
	25	68,336

Climate History

The time history for average annual precipitation since 1995 is shown in Figure 4. In 2011, the amount of precipitation appears to be a local high (15.4 inches) at the sites, while the low (7.0 inches) was recorded in 2003. These measurements, as well as the remaining measurements, do not greatly deviate from the mean precipitation recorded at the sites (11.4 inches for the time period shown in Figure 4), and hence there are no specific precipitation events at the annual level that might have a great effect on the performance of the pavement.

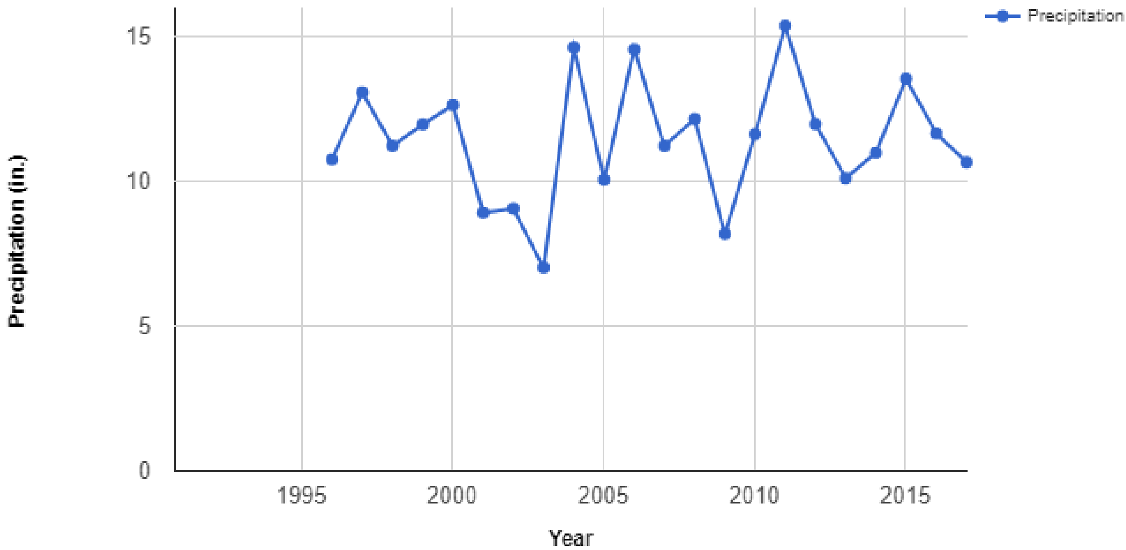


Figure 4. Average annual precipitation over time.

Figure 5 shows the time history of the average annual freezing index for both test sections. The freezing index is the sum of the difference between 32 degrees F and the average air temperature when it is less than freezing and 32 degrees F for each day, which is summed over a year's time. This index is an indicator of the harshness of the winter season relative to issues such as ground frost and low temperature cracking in pavements. From the time when the test sections were constructed to present, the sites' freezing indices have been variable. As depicted in Figure 5, the freezing index values ranged from 0 (multiple years) to 83 (in 2011) indicating variability in the harshness of winter at the location of the sections. As winter weather often contributes to a decrease in pavement performance, the annual freezing index is a factor of interest. However, based on the plot's lack of extreme values (except for the spikes in freezing index in 2001 and 2003), there is little evidence to support its significance, and therefore, the winter conditions appear to have little effect on the performance of the test sections.

Truck Volume History

For both sections, no truck traffic was reported by the highway agency from 1996 to present. The only structure served by the access road is a remote pipeline pumping station. Thus, it is expected that the route receives some traffic, however according to the highway agency the amount of traffic is so low that annual number of trucks at the two sections can be estimated as zero. Therefore, truck traffic is not a significant factor in terms of understanding the performance of the two pavement test sections. Moreover, since they are collocated, they are both subjected to the exact same traffic.

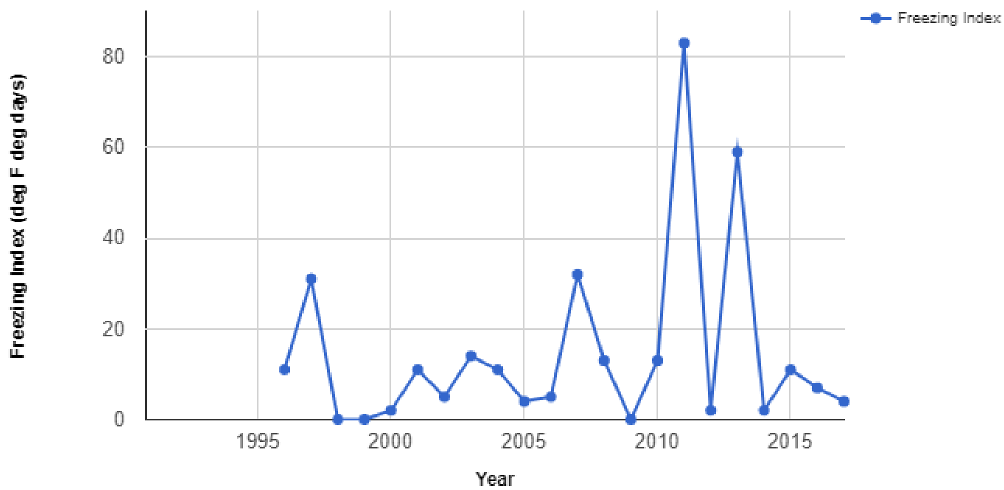


Figure 5. Average annual air temperature freezing index over time.

Pavement Distress History

This section summarizes the distresses observed on the test sections between the time the sections were constructed and 2016, which is when the last distress survey was performed. Fatigue cracking, longitudinal cracking (inside and outside the wheelpath), transverse cracking, IRI, and rutting were assessed for these sections. No patching or block cracking was observed.

Fatigue Cracking

Figure 6 shows the total area of fatigue related cracking observed for sections 35_0801 and 35_0802 between 1996 and 2016. While the graph obtained from InfoPave is labeled fatigue cracking, it might be better thought of alligator cracking since the area is not constrained to the wheel paths. The name fatigue cracking implies a mechanism, where-as the LTPP distress surveyors were reporting only observations of cracking on the surface of the pavement test sections following LTPP guidelines. For both pavement sections, the area of roadway with fatigue/alligator cracking was observed to increase over time. For test section 35_0801, 338 ft^2 of fatigue/alligator cracking or nearly six percent of the total section area was observed in 2016, while for test section 35_0802, 935.40 ft^2 or nearly sixteen percent of the total section area was observed in the last year of collection. In both test sections, the increasing area of fatigue/alligator cracking that appears over time plateaued in 2014 when the observed distress increased at a reduced rate.

Figure 7 shows the time history trend for cracking in the wheelpaths using the 2016 Highway Performance Monitoring System (HPMS) percent cracking metric that all states now report to the Federal Highway Administration (FHWA). This is computed by the total length of cracking in the wheel paths, assumed to be 39 inches wide, by the total area of the test section. The maximum value for a 12-foot-wide lane is approximately 55%. Wheelpath cracking is one of the traditional methods used to assess fatigue response of an AC pavement. Figure 7 shows a similar time history trend to the fatigue/alligator cracking shown in Figure 6, with the thicker test section, 35_0802, exhibiting greater cracking. The plateau in the rate of percent cracking after 2013 is more consistent between the two test sections than that shown in Figure 6.

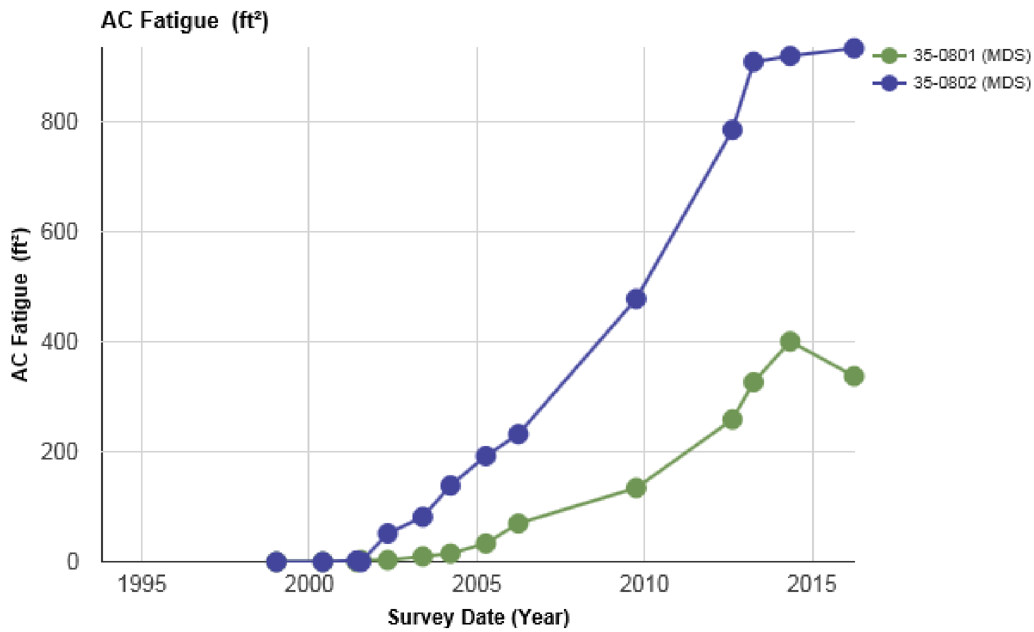


Figure 6. Total area of alligator cracking over time.

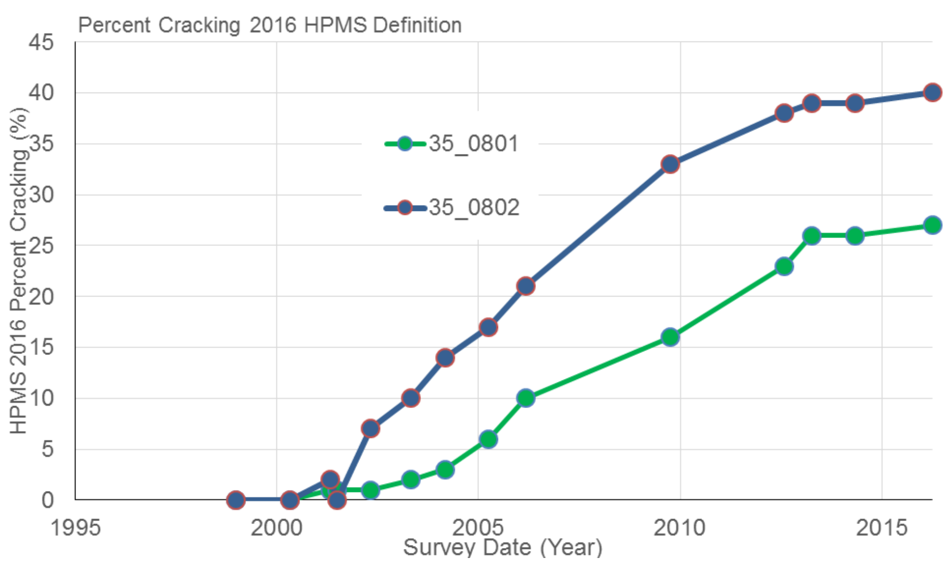


Figure 7. HPMS 2016 percent cracking definition using cracking limited to the wheel paths.

While no freeze climates typically exhibit more fatigue cracking due to the use of softer AC mixes[2], the amount of fatigue cracking exhibited at the test sections is still increasing at an abnormal level for a low-traffic roadway. One potential reason for the observed increase in fatigue cracking over time is that the AC pavement surface layer is undergoing age hardening (or oxidation); hardening leads to a more brittle asphalt surface layer and the development of raveling. Oxidation is influenced by the asphalt binder used,

[2] Titus-Glover, L., Darter, M., and Von Quintus, H. (2019). *Impact of Environmental Factors on Pavement Performance in the Absence of Heavy Loads* (FHWA-HRT-16-084). Washington, DC: Federal Highway Administration.

film thickness, aggregate gradation, in situ moisture content, and AC-mix percentage of air voids. To better understand whether oxidation played a role in increased levels of fatigue cracking, it is suggested that each of the factors mentioned be assessed using information collected on the sections at the time of construction as well as through lab tests of the existing pavement material.

Another possible reason for an increase in fatigue cracking on a low-traffic roadway is the development of stripping, which occurs when the aggregate and the binder in the AC layer begin debonding. Stripping is typically caused by water infiltration or inadequate surface or subsurface drainage and is common in hot, dry locations like the one where the test sections are located. Stripping is also associated with high levels of raveling, or progressive asphalt disintegration, which is confirmed by high levels of raveling reported at these sites. Additional lab tests to assess if striping is occurring on these test sections is recommended to confirm this hypothesis.

As fatigue cracking indicates that a pavement is either structurally inadequate or has reached the end of its service life, the high percentage of what appears to be fatigue related cracking observed on the thicker pavement structure (section 35_0802) is counterintuitive, especially in light of the limited amount of traffic at the site. Because thicker layers of asphalt concrete and subgrade implicate increased strength to withstand loads and section 35_0802 is thicker than section 35_0801, further investigation of the high levels of what appears to be fatigue-related cracking observed on section 35_0802 is needed. One possible explanation is that section 35_0802 has worse drainage than Section 35_0801, which resulted in increased pavement stripping and subsequently increased cracking indices. This explanation can be supported by photographs of the sites. In the photographs from 2013 and 2014, both sections depict darker cracks and cracks where plants are growing out of them as shown in Photograph 3. Each of these observations may indicate increased moisture due to poor drainage resulting in pumping. Despite both sections exhibiting these conditions, section 35_0802 appears to have slightly more instances of cracking that appears to be wet and is the only section with reported pumping in the distress survey. An additional distress survey and analysis of drainage at the two sections is necessary to assess whether drainage issues caused additional fatigue cracking in section 35_0802.

Longitudinal Cracking

Data on longitudinal cracking, inside and outside the wheelpath of the test sections, was collected between 1999 and 2016 as shown in Figures 8 and 9. For both sections, minimal longitudinal cracking was observed inside the wheelpath. In test section 35_0801, 5.25 feet of longitudinal cracking was observed during the last survey of the site while 14.43 feet of longitudinal cracking was observed inside the wheelpath of test section 35_0802 during the last data collection at the site. In Figure 8, a few of the collection years indicate spikes in the amount of longitudinal cracking inside the wheelpath. However, these spikes may be attributed to the inconsistent interpretation of low severity longitudinal cracking within the wheelpath and fatigue cracking – fatigue cracking is associated with “random cracking and meandering,” which is difficult to objectively determine in some cases.

Photograph 3. Picture of what appears to be moisture at the surface of cracks on section 35_0802.

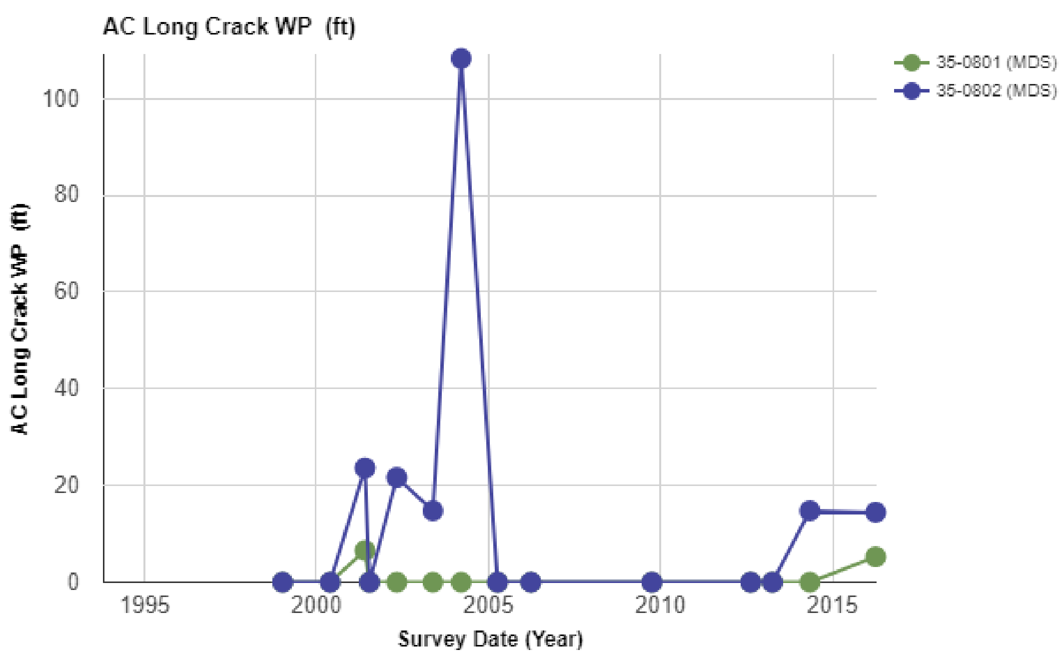


Figure 8. Total longitudinal cracking on the wheelpath over time.

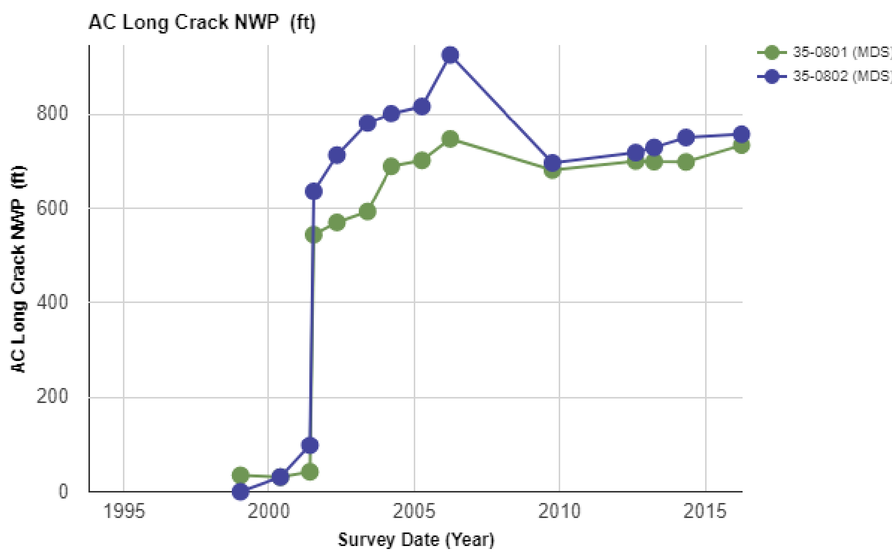


Figure 9. Total longitudinal cracking, non-wheelpath over time.

Unlike longitudinal cracking observed inside the wheelpath, the non-wheelpath (NWP) longitudinal cracking for both test sections increased over time. Figure 9 depicts the increase in NWP longitudinal cracking observed on both sections between 1999 and 2016. Both test sections experienced a steady increase in NWP longitudinal cracking between 1999 and 2001, followed by a sharp increase in July of 2001. By 2009, both test sections' NWP longitudinal cracking plateaued. The final measurement of NWP longitudinal cracking observed for test section 35_0801 was 733.74 feet, while the final measurement observed for section 35_0802 was 756.70 feet. Throughout the distress history of both sections, the NWP longitudinal cracking measurements are consistent between the two sections. The increase in NWP longitudinal cracking is likely a reflection of changes made to the distress survey definitions. In the early days of the LTPP program, non-wheelpath cracking was restricted to inside the edge of lane lines despite multiple cracks being observed on or very near the lane marking. The jump in longitudinal cracking observed is reflective of changes in the non-wheelpath longitudinal cracking definition where cracks on or near pavement markings were only included in the total count after 2001.

Transverse Cracking

Figures 10 and 11 depict the total transverse cracking reported for both sections of pavement throughout the analysis period; the transverse cracking observed increases over time in the test sections. For both sections, no transverse cracking was observed prior to 2006, but by 2006 and 2012 for section 35_0801 and section 35_0802 respectively, the transverse cracking observed began to increase rapidly. By 2016, 125.30 feet (37 total cracks) and 100.40 feet (35 total cracks) of transverse cracking are observed along test sections 35_0801 and 35_0802 respectively. The mechanism most often linked to transverse cracking in AC pavements is due to temperature changes in the AC layer. Unlike pavements in freeze zones that experience very low air temperatures, pavements in high-altitude no-freeze desert areas can experience higher diurnal changes in temperature between the maximum and minimum temperatures over a short duration than pavements in humid environments. These pavement test sections are at an altitude greater than 4,000 feet above sea level. This altitude also increases the solar radiation input energy into pavement structures. Another mechanism that is likely at play in the development of transverse cracking is the hardening of the AC layers, making them more brittle and more prone to thermal-induced cracking. The combination of age hardening with temperature-induced "fatigue" in the AC due to diurnal temperature-induced stresses is one potential reason for the development of transverse cracking on these test sections.

One way to assess AC hardening is to perform viscosity related tests on asphalts extracted from the test sections and compare the results to the results from initial construction.

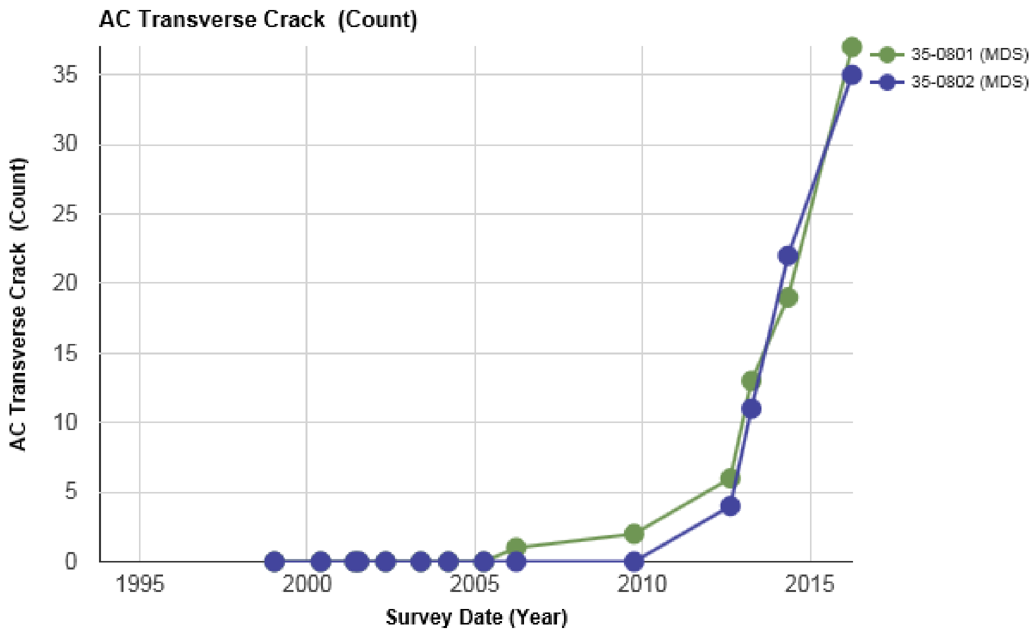


Figure 10. Total transverse cracking (count) observed over time.

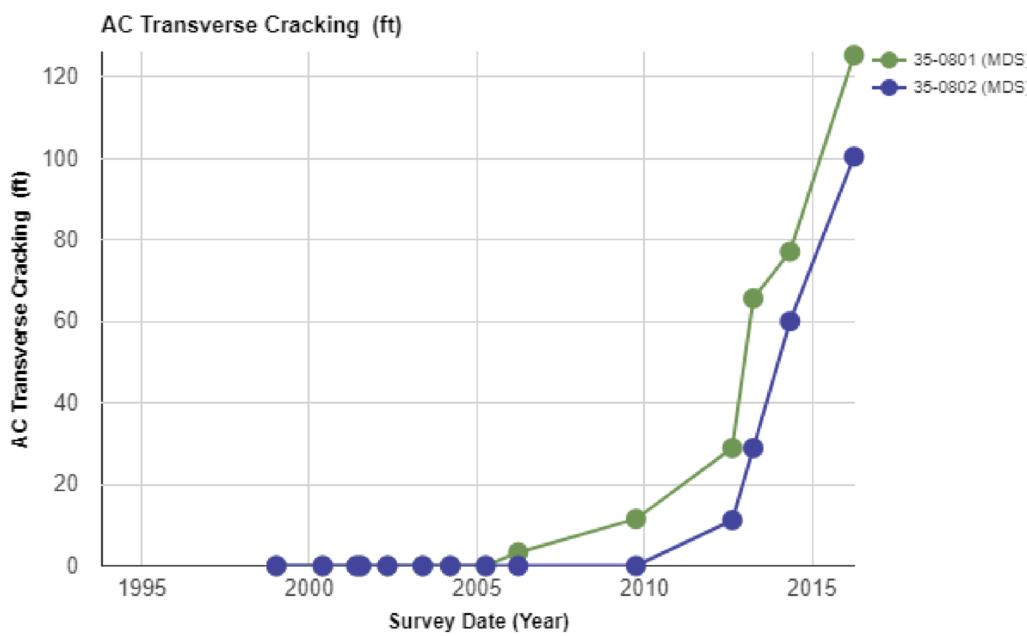


Figure 11. Total transverse cracking (length) observed over time.

International Roughness Index (IRI)

The average IRI measurements for each section over time are shown in Figure 12. The IRI of the test sections gradually increased over time. Section 35_0801's IRI increased from 73.56 in/mile in 1999 to 95.23 in/mile in 2016 while section 35_0802's IRI increased from 74.38 in/mile in 1999 to 94.72 in/mile in 2016. The gradual increase in roughness over time is reasonable as the amount of cracking observed on the pavement sections also increased over time. Despite the increase in cracking distresses observed, the pavement's IRI performance is classified as "Good" based on FHWA performance definitions. This type of pavement roughness history is also not consistent with heavy truck load related cracking. With truck load related cracking, there tends to be pot-hole formation as chunks of the pavement surface are dislodged. This observation tends to support the environmental aging hypothesis and age hardening of the AC materials in the pavement structure. It also suggests that low temperature cracking, as witnessed in low temperature environmental zones, is not happening since the transverse cracks are not building up surface "heaves" on each side of the transverse cracks, which will affect pavement roughness measurements.

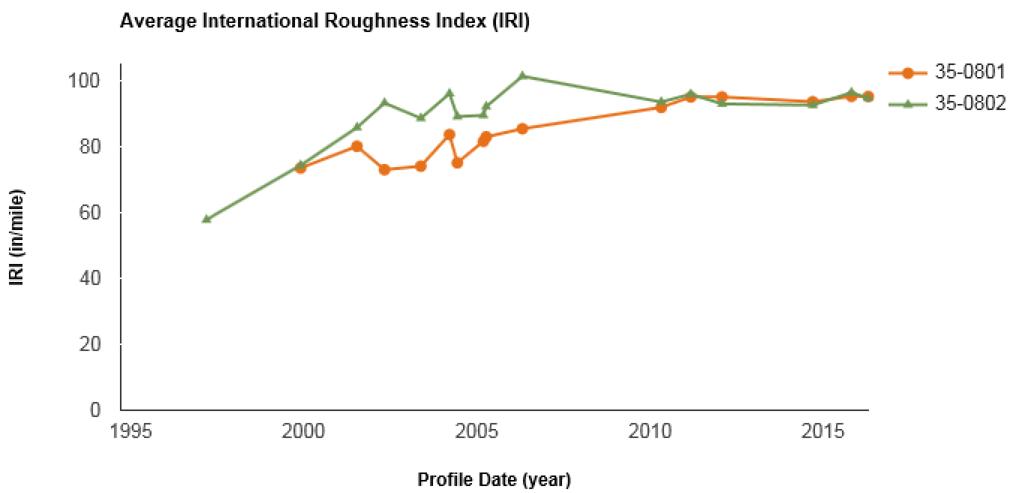


Figure 12. Average pavement roughness over time.

Rutting

The last distress assessed for both sections was rutting and is shown in Figure 13. While there appears to be some variation in rut depths over time, these ruts are considered steady and not increasing over time because LTPP reports rut depths to a single millimeter precision. From this information rutting is not a critical factor for these test sections, which reinforces the very low/nonexistent traffic volumes applied to the test sections. For both sections, the mode of the rutting measurements was 0.12 inches, which can be due to raveling.

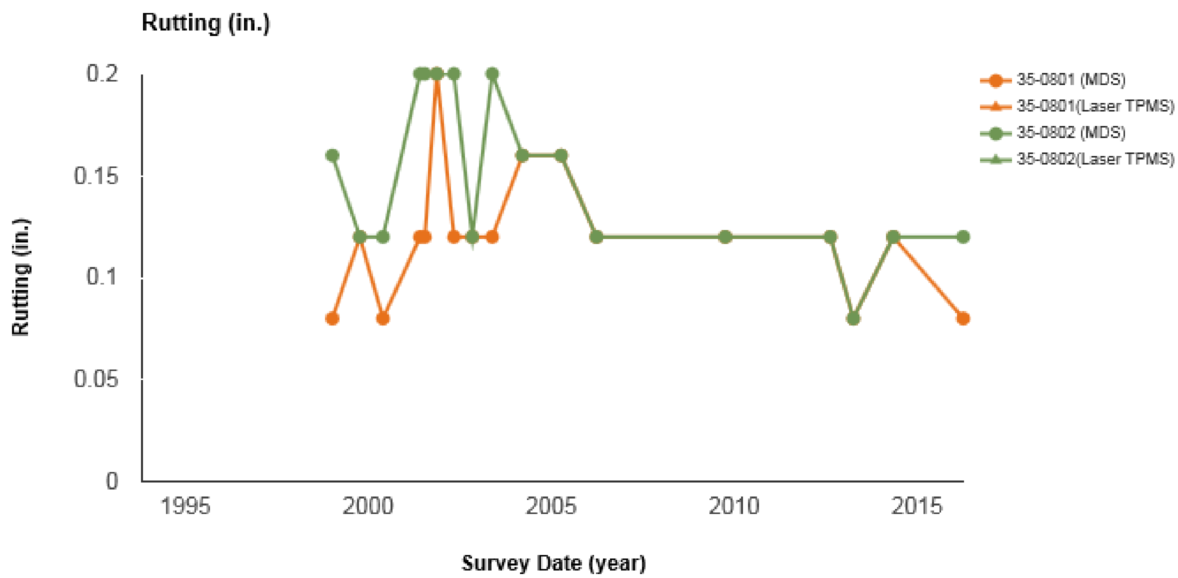


Figure 13. Rutting over time.

SUMMARY OF FINDINGS

In this review of information concerning the performance history of test sections 35_0801 and 35_0802 the following information was presented:

- The test sections were originally constructed in 1995; section 35_0801 consists of 4.2 inches of AC, 9.7 inches of granular subbase, and untreated subgrade while section 35_0802 consists of 7 inches of AC, 12.7 inches of granular subbase, and an untreated subgrade. Both were included in the LTPP program as part of the SPS-8 Study of Environmental Effects in the Absence of Heavy Loads. Except for the layer thicknesses (as expected based on the SPS-8 experiment design and as documented in the associate Construction Report), the structural and material properties of the test sections are comparable (as evidenced by similar backcalculated moduli, resilient moduli, and dynamic moduli).
- Table 5 provides a summary of the initial and final measurements of deflection, fatigue cracking, longitudinal cracking (both wheel path and non-wheel path), transverse cracking, IRI, and rutting. Most notable from the comparison of the two sections was the higher quantity of fatigue/alligator related cracking reported for the thicker pavement section (section 35_0802), which was almost triple that of the thinner section (35_0801). Using the HPMS 2016 percent cracking metric of cracks in the wheel paths, the relative difference between the two sections is much smaller, but still significant. The study team did confirm from the raw field data collection forms that the pavement thicknesses contained in the LTPP database are correct.
- From the assessment conducted and summarized in Table 5, while the deterioration of the pavement sections over time is expected, there is still uncertainty as to the cause of some of the deterioration exhibited in the two sections. In terms of the structural properties of the two sections, the abnormally high values for the backcalculated moduli of the asphalt and base layers is not aligned with expectations nor with the tripling of the deflections values from 1995 to 2016. The high values of backcalculated modulus of the AC layers is consistent with how age hardening of the asphalt affects the modulus of the AC layer. As for distresses, the high quantity of fatigue related cracking observed on this low-volume road requires additional investigation especially

since section 35_0802 exhibited more fatigue related cracking despite being thicker than section 35_0801. While the increased fatigue related cracking on the thicker section may be the result of asphalt hardening or stripping, increased subgrade restraint from the base layer due to increased overburden pressures and the increased fatigue related cracking in section-35_0802 could be due different drainage conditions than for section 35_0801, follow-up forensic investigations are strongly recommended in order to better understand the reason these pavement sections are behaving the way they are.

Table 5. Test sections attribute summary.

Attribute	First Measurement		Last Measurement	
	Section 35_0801	Section 35_0802	Section 35_0801	Section 35_0802
Deflection under Load plate (mils)	9.6	6.3	21.1	16.1
Fatigue/Alligator cracking (ft^2)	0	0	338	935.40
HPMS 2016 Percent Cracking (%)	0	0	27	40
Longitudinal WP Cracking (ft)	0	0	5.25	14.43
Longitudinal NWP Cracking (ft)	34.44	0	733.74	756.70
Transverse Cracking	0	0	125.30 (37 cracks total)	100.40 (35 cracks total)
IRI (in/mi)	73.56	74.38	94.72	95.23
Rutting (in)	0.08	0.16	0.08	0.12

FORENSIC EVALUATION RECOMMENDATIONS

It is recommended that the desktop study be extended to further investigate the abnormalities described by carrying out the following:

- Additional FWD testing to determine if deflections have continued to steadily increase over time and hence structural capacity of the pavements has continued to deteriorate. Additional investigation as to why section 35_0802 was modeled as three layers instead of four is recommended.

- Perform within test section coring in order to address the following three items that could help explain one or more of the issues raised in this memorandum:
  - Confirm within test section thicknesses match those measured outside the section when the test sections were incorporated into the LTPP program.
  - Determine whether or not stripping of the AC layer is a problem on the test sections.
  - Provide asphalt concrete layer material for use in laboratory testing of binder as detailed in the below bullet item.
- Obtain new material field samples from the sections for the following purposes:
  - Test extractions from the field aged binders to compare against initial laboratory tests related to viscosity to examine issues related to asphalt hardening.
  - Obtain cores at selected crack locations for inspection to determine the potential influence of asphalt stripping.
  - Conduct Hamburg wheel rutting tests on potential existing samples of the AC layer from the original construction and cores from the current pavement structure. The purpose of the Hamburg test for this investigation is to determine the stripping potential of the AC mixture and not necessarily rutting related issues.
  - Obtain samples of the AC materials from initial construction and cores from the current AC pavement layer to perform dynamic modulus tests using the AMPT test procedure to investigate stiffening of the AC surface layer due to aging.
- Continued monitoring of these test sections is recommended. Another round of deflection measurements is recommended in the next monitoring cycle performed by LTPP. Also, during the next round of monitoring, the LTPP field staff should also be asked to investigate features related to test section drainage, changes in the surrounding land use, and condition of the pavement outside the test sections to determine if other factors are affecting the performance of these two pavement test section can be observed,
- A limited comparison study with the two SPS-8 test sections constructed in North Carolina is recommended. These test sections, 37_0801 and 37_0802, have the same pavement structure and also exhibit high levels of fatigue related cracking. The North Carolina test sections are in a wet no-freeze zone and at much lower elevation. The intent of expanding this investigation to these LTPP test sections is to look at similarities in the cracking patterns.
- A discussion should be held with current New Mexico DOT staff and past LTPP Southern Regional contract staff relative to the zero traffic estimates on the test sites.

ADDENDUM TO MEMORANDUM: FOLLOW-UP INVESTIGATIONS

Based on the findings and recommendations of the desktop memorandum, follow-up investigations were conducted on New Mexico SPS-8 test sections 35_0801 and 35_0802 in an effort to better understand the performance of the test sections over time, particularly with regards to fatigue/alligator cracking and deflections reported on the section. The follow-up activities, which consisted of both office and field work, included:

- ***FWD testing***: FWD testing was performed in the early morning for section 35_0801 and late morning for section 35_0802 on July 15, 2020 and followed standard LTPP FWD protocols. The objective of collecting additional deflection information was to determine if there was a steady increase in deflections over time and therefore, the structural capacity of the pavement has continued to deteriorate.
- ***Coring***: A total of 8 cores were collected on July 15, 2020 just outside each test section (at the start and end). Cores were collected to confirm that the test section thicknesses matched those reported when the sections were first incorporated into the LTPP program and to determine whether or not AC stripping was a problem on the test sections.
- ***Manual distress survey and profile measurements***: Manual distress surveys were conducted on July 15, 2020 for the test sections. The collected distress information was used to assess the performance of the pavement since the last manual distress survey used in the desktop study was in 2016 (2019 data was released after the desktop memorandum was finalized). Longitudinal profile measurements were collected during the morning of July 11, 2020 using the LTPP High Speed Survey vehicle and following standard LTPP profiling protocols. Transverse profile measurements were also reported using a Profilometer.
- ***Comparison to other SPS-8 test sections****:* A comparison of the test sections to other SPS-8 test sections showing similar fatigue/alligator cracking was conducted.
- ***Examination of traffic estimates****:* Additional information on the estimated zero traffic reported on the test sections was pursued.

Other activities recommended in the desktop memorandum included gathering field samples and conducting material testing. However, due to time constraints, material sampling was not conducted. It is recommended that material testing be pursued as a follow-up activity to this extended investigation.

This addendum provides a summary of the findings of the follow-up activities described above.

FWD Testing

The objective of collecting additional FWD data on the test sections was to determine if there was a steady increase in the deflections reported over time. The desktop study showed that between the first time FWD data was collected on the test sections in 1997 and the 2016 data collection, the deflections reported more than doubled. The increase in deflections reported over time indicated that the structural capacity of the test sections was deteriorating.

The additional FWD testing, conducted in July 2020, showed an increase in the deflection reported for test section 35_0801 and a decrease in the deflection reported for 35_0802 when compared to 2016 deflection data; however, the change in deflections between 2016 and 2020 was small (+1.4 mils for test section 35_0801 and -1.2 mils for test section 35_0802). As depicted in Figure 14, the average daily temperature reported on the day of FWD testing (using MERRA data) was also investigated to better describe the reported deflections on the test sections over time. The relationship between the average daily temperature and reported deflections is unclear between 2000 and 2004. Before 2000 and after 2004, there is generally a positive relationship between the reported temperature and the reported deflections at the test sections: the higher the daily temperature, the higher the reported deflection. The relationship between temperature and deflection helps to explain the increase in deflection reported on both test sections over time. However, it is still expected that the aging of the pavements played some role in the increase in deflections. Aging, which would make the AC layer stiffer, can also lead to an increase in cracking, which reduces the effective modulus of the AC layer and leads to higher deflections. As evidenced later in this addendum, there is an increase in cracking (particularly transverse and fatigue/alligator cracking) reported on the sections throughout time which helps support this hypothesis.

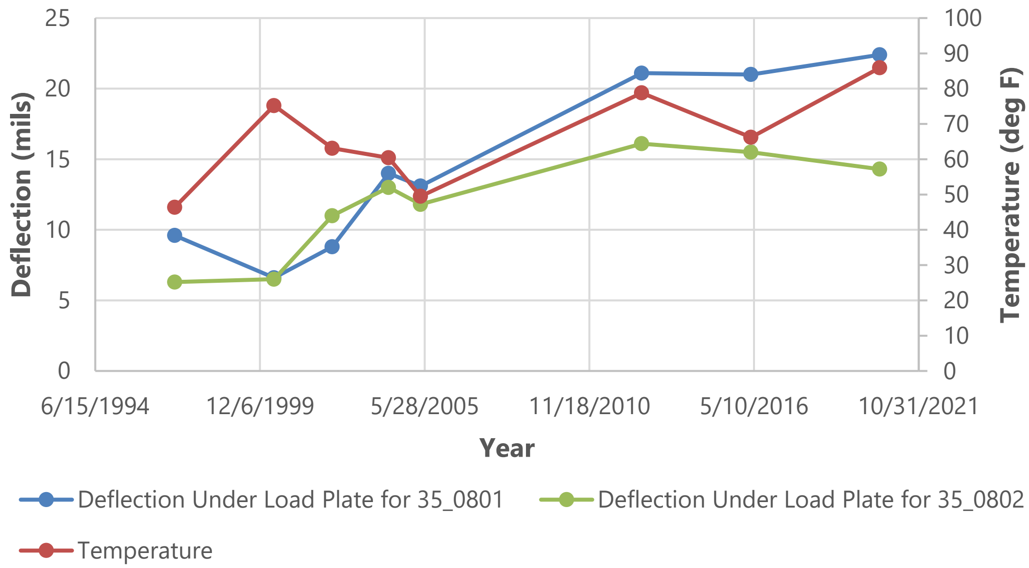


Figure 14. FWD deflections under the load plate and average daily temperature over time.

Coring

The purpose of collecting cores on the New Mexico SPS-8 test sections was two-fold. The first objective was to confirm that the thicknesses of the test sections, reported at the time of their inclusion into the LTPP program, were consistent with the actual test section thicknesses. This was of particular interest, because despite having a thicker pavement structure, test section 35_0802 reported higher amounts of fatigue/alligator cracking over time. The second objective was to determine whether or not stripping was present in the AC layers. As reported in the desktop study, one possible reason for an increase in fatigue cracking on these low-traffic test sections is the development of stripping. Stripping occurs when the aggregate and the binder in the AC layer begin debonding and is typically caused by water infiltration or inadequate surface or subsurface drainage. It is common in hot, dry locations like the one where the test sections are located.

Four cores were obtained from each of the sections: two cores before and two cores after the monitoring areas. The AC thicknesses reported using the cores were less than the thicknesses reported on the test sections at the time of their incorporation into the LTPP program. The average AC thickness reported from the cores for test section 35_0801 was 3.9 inches. This is 0.3-inch less than the 4.2 inches reported for the AC layer at the time of the test section's incorporation into the LTPP program and 0.1-inch less than the planned thickness of 4 inches from the construction report for the section. Similarly, the average AC thickness reported from the cores for test section 35_0802 was 6.8 inches. This is 0.2-inch less than 7 inches reported for the AC layer at the time of its incorporation into the LTPP program and the planned thickness from the construction report for the section. Table 6 provides a summary of the 8 cores obtained from the New Mexico SPS-8 sections.

Table 6. Summary of 2020 core samples from New Mexico SPS-8 sections.

Core Number	Station	Offset	AC Thickness (in)	Section	Average AC Thickness (in)
C01	0-35	2.5	4.3	350801	3.9
C02	0-35	6	3.8		
C03	5+30	2.5	3.9		
C04	5+30	6	3.6		
C01	0-20	2.5	6.1	350802	6.8
C02	0-20	6	6.8		
C03	5+23	2.5	7.1		
C04	5+23	6	7.1		

The cores were also used to assess whether stripping was present in the AC layer. As depicted in Appendices A and B, there were no obvious signs of moisture damage or other kinds of material breakdown in the cores for either test section. This indicates that stripping likely was not the primary cause of the fatigue cracking observed on the test sections and that other potential causes of the fatigue/alligator cracking (such as age hardening/oxidation) should be considered.

As an extension of the examination of the thicknesses reported for the AC layers, the original construction report for sections 35_0801 and 35_0802 was also reviewed. The construction report for the test sections did not report any major abnormalities that occurred during construction; there was no mention of low laydown and compaction temperatures of the AC material, poor compaction efforts, paver stops, or non-homogenous AC material. However, the report did note that traffic was allowed to drive on the exposed subgrade during construction after a rainstorm occurred, which caused rutting in the wheel paths. Before the aggregate base (AB) layer was placed and compacted, the subgrade was dried and recompacted.

The report also noted that despite several attempts to provide a uniform AB thickness, the aggregate base layer of test section 35_0801 ranged from 7.3 to 11 inches (despite having a planned thickness of 8 inches), when measured with a rod and level. Similarly, the AC thickness of test section 35_0801 also varied, ranging from 2.6 to 4.6 inches (compared to a planned thickness of 4 inches and the average thickness of 3.9 inches estimated from the cores). Test section 35_0802 showed similar variation in layer thicknesses. The AB thickness of test section 35_0802 ranged from 10 inches to 14 inches (despite having a planned thickness of 12 inches), while the AC thickness ranged from 5.6 inches to 7.5 inches (compared to a planned thickness of 7 inches and the average thickness of 6.8 inches estimated from the cores). The variation in the AC layer thicknesses was compared to the observed fatigue/alligator cracking reported in the manual distress surveys for the test sections to determine if the variations in layer thicknesses played a role in the fatigue cracking observed or specifically, if areas of fatigue/alligator cracking were associated with areas where the AC layer was thinnest. However, it was found that there was no clear relationship between the amount and severity of the fatigue/alligator cracking in areas with thinner AC layers. Figures 15 and 16 illustrate the layer thicknesses for each section at 50-foot intervals.

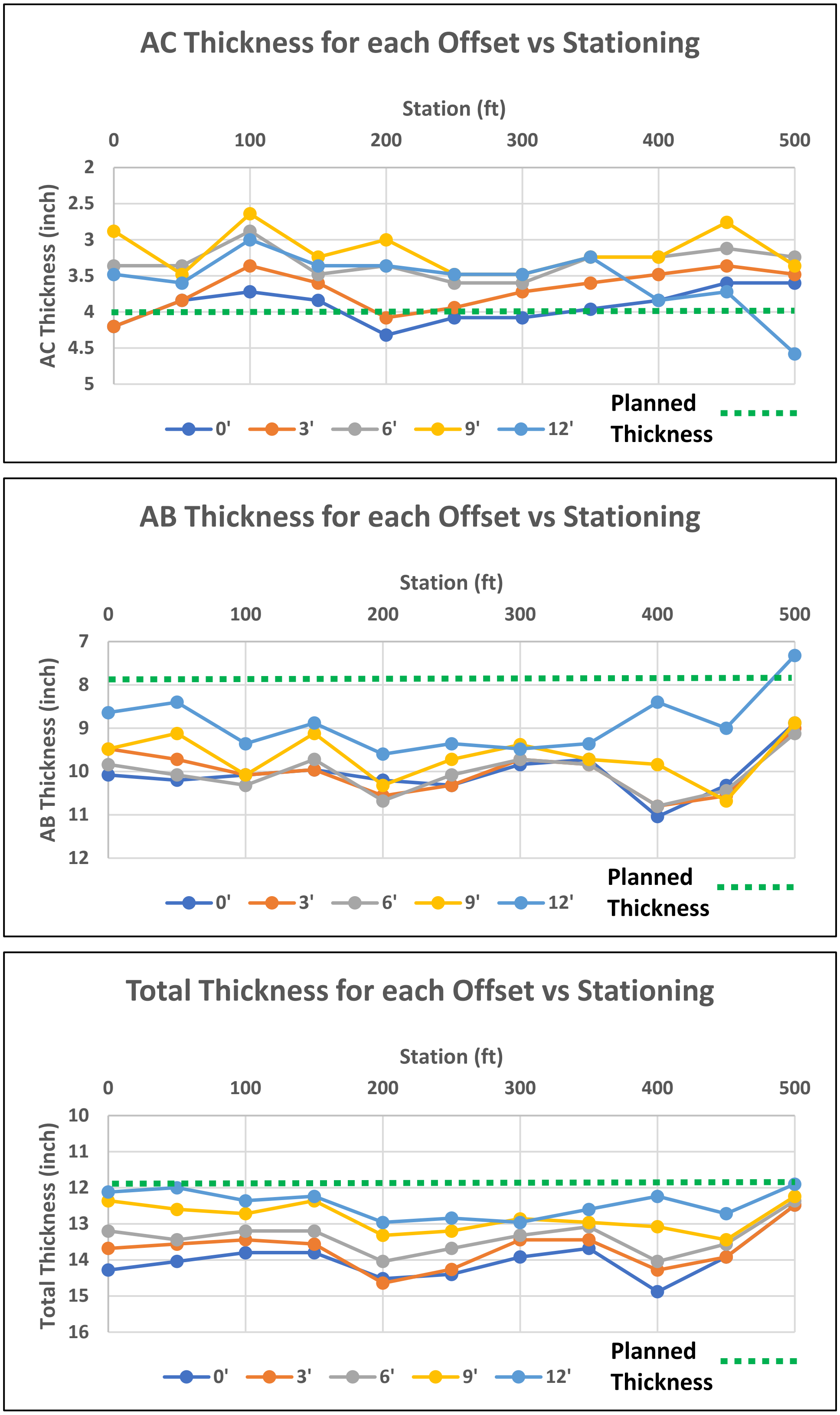
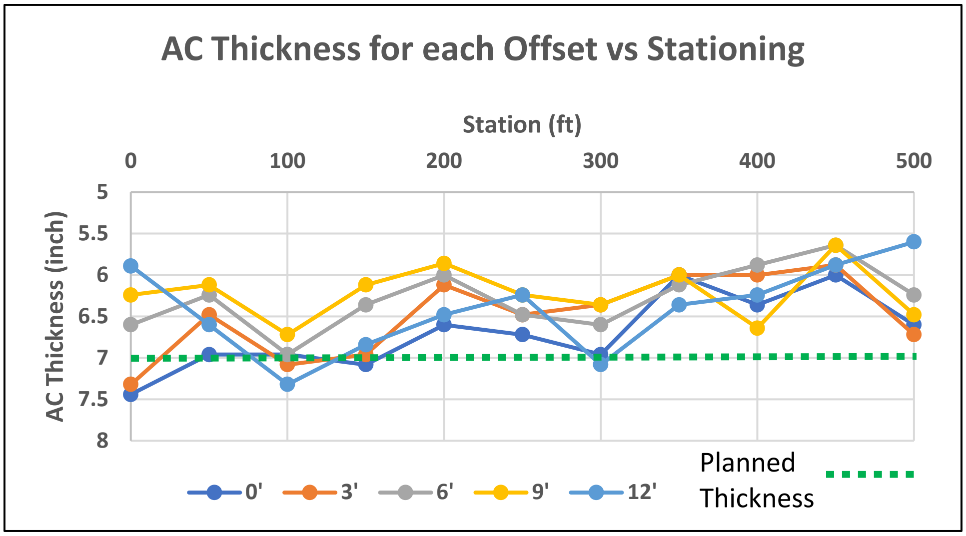
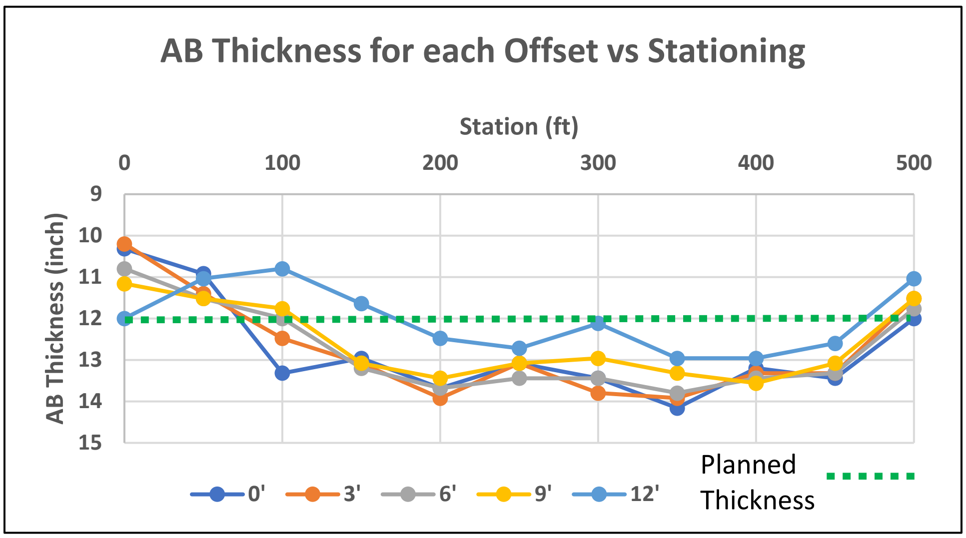


Figure 15. Summary of AC (top), AB (middle), and Total Pavement Thickness (bottom) for test section 35_0801, with the green line representing the planned thickness for each layer.





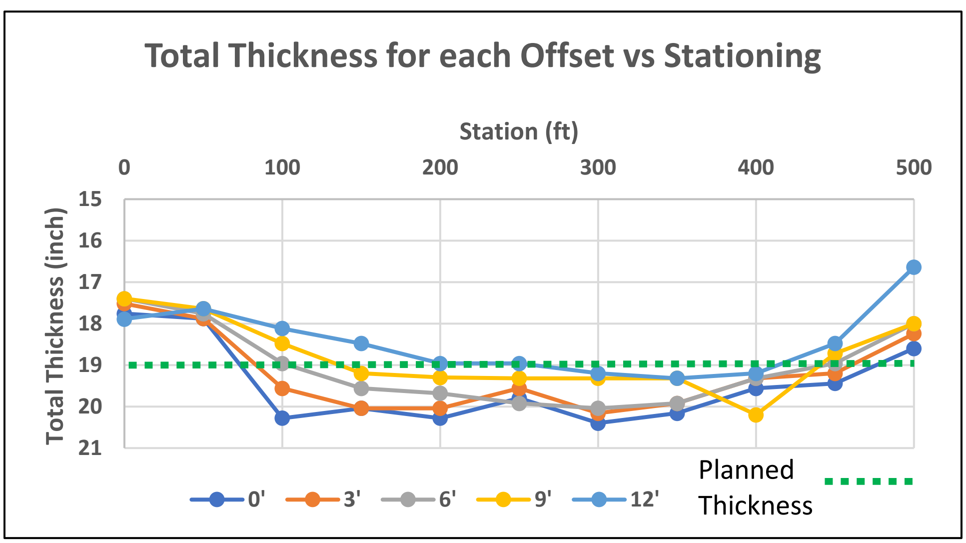


Figure 16. Summary of AC (top), AB (middle), and Total Pavement Thickness (bottom) for test section 35_0802, with the green line representing the planned thickness for each layer.

Manual Distress Survey and Profile Measurements

As a part of the follow-up field investigations, transverse and longitudinal profiles and manual distress surveys were conducted on both sections. The objective of collecting this information was to identify changes in the performance of the test sections over time. A summary of the data collected during a previous collection date in April 2016 and during the most recent data collection in July 2020 is shown in Table 7 below. Metrics for each test section are color coded based on the change in performance—green indicates an increase in the metric between 2016 and 2020 and yellow indicates little to no change.

Table 7. Summary of test sections current conditions.

Metric	Section 35_0801		Section 35_0802	
	2016	2020	2016	2020
Fatigue/Alligator cracking (ft^2)	338	556	935	870
HPMS Percent Cracking (%)	27	43	40	47
Longitudinal WP Cracking (ft)	5	1	14	0
Longitudinal NWP Cracking (ft)	734	740	757	797
Transverse Cracking	125 (37 cracks total)	157 (47 cracks total)	100 (35 cracks total)	180 (50 cracks total)
IRI (in/mi)	95	97	95	100
Rutting (in)	0.08	Pending[3]	0.12	Pending[3]

As shown in the table, notable increases in transverse cracking and HPMS percent cracking were reported for both test sections between 2016 and 2020, while only test section 35_0801 reported an increase in fatigue cracking during the same period (an increase of 218 ft^2). Increases in transverse and fatigue cracking are aligned with the assumption that age hardening, or oxidation is playing a role in the performance of the pavements. Oxidation causes the asphalt binder to stiffen and become brittle, making

[3] Rutting values associated with the 2020 data collection had not yet been computed at the time this memorandum was finalized. The project team will routinely check for these values until posted, and if significant changes have occurred (i.e., large increase in rutting), then the project team will update the memorandum to include updated rut data analysis and review.

it prone to environmental stresses and the development of full-width transverse cracking and block cracking.

Comparison to Other SPS-8 Test Sections

Another recommendation of the desktop study was to compare the performance of the two New Mexico SPS-8 test sections with the performance of the other SPS-8 test sections in the LTPP program. To do so, findings from the previously referenced FHWA report[4] were considered. The study conducted was focused on identifying and quantifying the effects of environmental and design factors on the performance of pavements in the absence of heavy loads by comparing the performance of SPS-8 test sections to heavily trafficked test sections from other LTPP experiments. Specifically, the study considered the impact of factors such as freeze versus non-freeze climates, freeze-thaw cycles, frost depth, AC temperature, annual precipitation, in situ moisture content, subgrade type, clay content, and others. Both New Mexico SPS-8 test sections were included in the analysis dataset.

One key finding of the study was that the higher the annual average temperature of the AC layer, the higher the amount of AC alligator cracking. The reason for this is that higher temperatures cause the AC material to oxidize at a faster rate. However, this finding was in relation to all test sections studied (both high and low traffic), rather than for low-traffic roads only. Age hardening or oxidation was one of the suggested hypotheses for the amount of fatigue cracking observed on the test sections. Consequently, it is recommended that the originally sampled binder and binder extracted from the recent cores be tested in accordance with AASHTO M320 to determine the binders' PG grade. These tests should include at a minimum:

- Dynamic Sheer Rheometer (DSR) testing on original, RTFO-aged, and PAV-aged binder,
- Bending Beam Rheometer on PAV-aged binder, and
- Direct Tension Tester on PAV-aged binder.

Determining the PG grade of the AC binder will be used to assess the validity of the hypothesis that age hardening, or oxidation is one of the main causes for the fatigue cracking observed on the New Mexico SPS-8 test sections.

Examination of Traffic Estimates

The final activity conducted as a part of the follow-up investigation was the pursual of additional information on traffic estimated on the test sections. As discussed in the desktop study, no truck traffic was reported on the test sections throughout the analysis period. While low levels of traffic were expected as the test sections are a part of the SPS-8 experiment, the fatigue cracking reported on the test sections indicated some truck traffic may exist. Further investigation of both the truck traffic reported in the LTPP database as well as the geographical location of the test sections did not provide any evidence that the reported truck traffic on the test section was incorrect and therefore, no additional actions were taken.

Conclusions and Recommendations

The purpose of the follow-up investigation of New Mexico test sections 35_0801 and 35_0802 was to pursue additional information on the change in performance of the test sections over time. Specifically, the investigation was used to gather additional information on the potential cause of the fatigue/alligator

[4] Titus-Glover, L., Darter, M., and Von Quintus, H. (2019). *Impact of Environmental Factors on Pavement Performance in the Absence of Heavy Loads* (FHWA-HRT-16-084). Washington, DC: Federal Highway Administration.

cracking reported on the test sections despite a lack of truck traffic and examine whether the pavement sections were continuing to deteriorate over time. To do so, a series of follow up field work—including FWD testing, coring, a manual distress survey, and transverse and longitudinal profiling—and office work—such as a comparison of the New Mexico SPS-8 test sections' performance to other SPS-8 test sections and the further investigation of the traffic reported on the test sections—was conducted.

The key finding from the activities conducted was that the performance of the test sections was likely related to age hardening or oxidation as stripping, variation in test section thicknesses, and truck traffic were not found to play a clear role in the performance of these test sections. While material testing of the test sections could not be conducted due to time constraints, it is recommended that PG grading be performed on the extracted cores. In doing so, information on whether the test sections have oxidized can be inferred.

Appendix A
2020 Core Photos from LTPP Test Section 35_0801

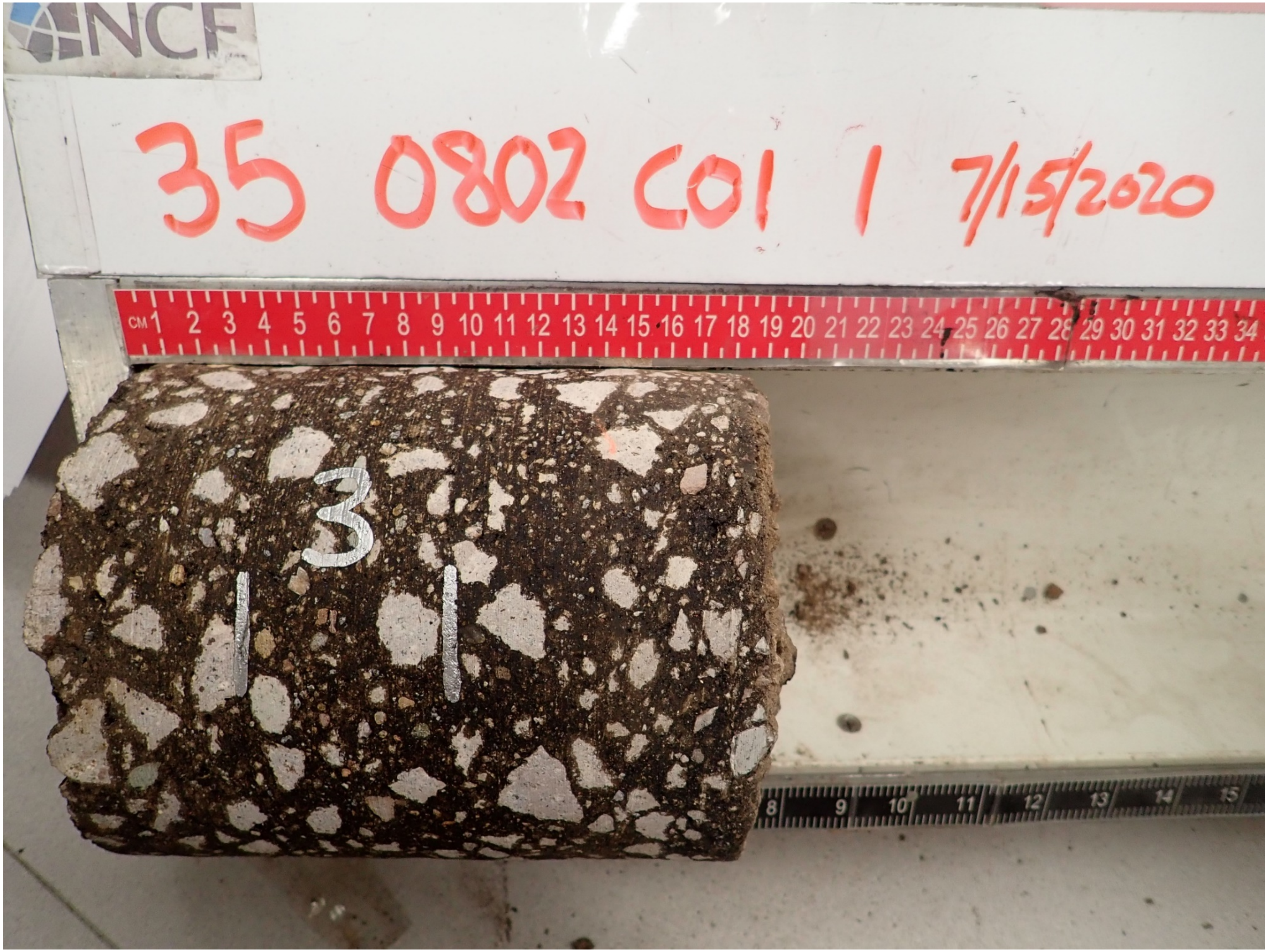
NCF
35 0801 C01 1 7/15/2020
3


35 0801 C02 1 7/15/2020
3


NCF
35 0801 C03 2 7/15/2020
3


NCF
35 0801 C04 2 7/15/2020
3
1

Appendix B
2020 Core Photos for LTPP Test Section 35_0802


NCF
35 0802 C01 1 7/15/2020
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35 0802 C02 1 7/15/2020
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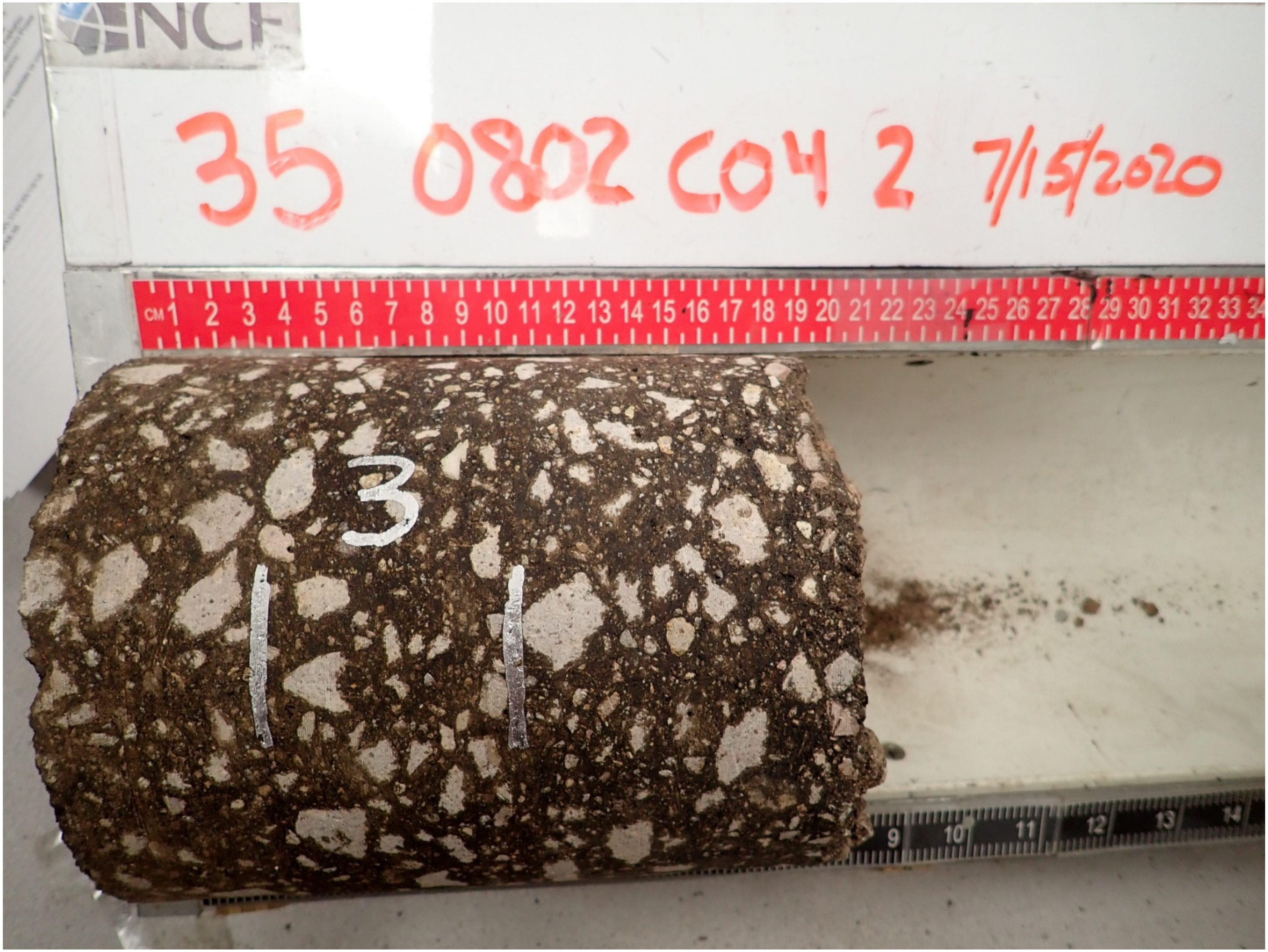
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35 0802 C04 2 7/15/2020
CM 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34
3
9 10 11 12 13 14