Technical Memorandum

To: Jeff Uhlmeyer

From: Lauren Gardner, Gonzalo Rada, and Kevin Senn

cc: Mustafa Mohamedali

Date: November 20, 2020

Re. Forensic Desktop Study Report: Indiana LTPP Test Section 18_1037

The Long-Term Pavement Performance GPS-1 Asphalt Concrete (AC) Pavement on Granular Base test section 18_1037[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." Test section 18_1037 was originally constructed in 1983 and was incorporated into the LTPP program in 1987 as a full-depth AC pavement with 14.7 inches of asphalt concrete (split between 4 layers) over a fine-grained subgrade soil. Despite the low levels of truck traffic observed on the test section and the additional structural capacity provided by AC overlays in September 1994 and May 2003, the test section has reported relatively high levels of rutting from 1987 to present, particularly prior to the first overlay event. In terms of other distresses, the pavement section also reports an increase in cracking (fatigue and non-wheel path (NWP) longitudinal) following the second overlay in 2003. The Falling Weight Deflectometer (FWD) deflections and IRI measurements on this test section were relatively low. Accordingly, the purpose of this study was to 1) investigate the cause(s) of the rutting depths observed, particularly prior to the first overlay in 1994, 2) investigate the reason(s) for increased cracking following the second overlay in 2003, and 3) further investigate the reason(s) for the performance of the pavement in terms of low FWD deflections and IRI.

SITE DESCRIPTIONS

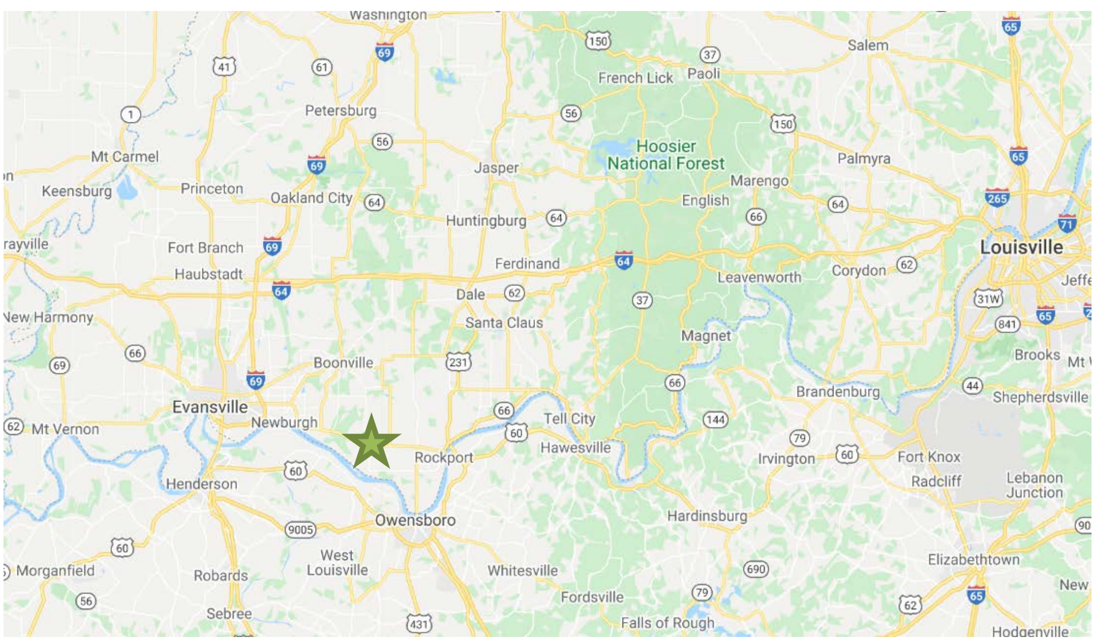
LTPP test section 18_1037 was located on State Route 66, eastbound, in Spencer County, Indiana. State Route 66 is a rural minor arterial with one lane in the direction of traffic. The test section was classified as being in a Wet, No-Freeze climate zone[2]. While at the time of its nomination the site was considered

[1] First two digits in test section number represent the State Code [18 = Indiana]. The final four digits are unique within each State/Province and were assigned at the time the test section was accepted into the LTPP program.

[2] In the current LTPP dataset, the test section is classified as being in a No-Freeze climate based on the location of the section rather than the freezing index at the test site. However, starting with the 2021 LTPP data release, the climatic region for each LTPP test section will be assigned using the MERRA-2 dataset. In the updated web portal, the average annual freezing index will be used to classify the climatic region of each test section. Through this update, the threshold between Freeze and No Freeze regions will be an average annual freezing index of 83.3 degree-Celsius days. Therefore, test section 18_1037 will be classified as a being in a Freeze climatic region.

active, based on information in the LTPP database, the test section has since been found to be milled and overlaid and therefore has been placed Out of Study (OOS). The coordinates (in degrees) of the site were 37.90013, -87.22045. Photograph 1 shows the section at Station 0+00 looking eastbound in 2016, while Map 1 shows the geographical location of the test section.

Photograph 1. LTPP Section 18_1037 at Station 0+00 looking eastbound in 2016.



Map 1. Geographical location of test section.

BASELINE PAVEMENT HISTORY

This section of the document presents historical data on the pavement structure and its structural capacity, climate, traffic, and observed surface distresses.

Pavement Structure and Construction History

The test section was constructed in 1983 and was accepted into the LTPP Program as part of the GPS-1 experiment in January 1987. The pavement structure at the time of its incorporation into the LTPP program consisted of 14.7 inches of asphalt concrete (AC) (comprised of four distinct layers) over a fine-grained subgrade. This pavement structure is summarized in Table 1 and corresponds to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. The next construction event occurred in September 1994, when the test section received a mill and a 2.4-inch AC overlay, moving the test section to the GPS-6S: AC Overlay on AC Pavement with Milling and/or Fabric experiment. Table 2 summarizes the pavement structure following the mill and overlay which corresponds to CONSTRUCTION_NO = 2 (CN = 2). A second overlay event occurred in May 2003, during which the test section received a 1.5-inch overlay (CN=4), moving the section into the GPS-6D AC Overlay of AC Pavement using a Conventional AC overlay; the pavement structure following the overlay event is depicted in Table 3. Prior to and following the second overlay, in June 2000 and June 2014, the test section received crack sealing (CN=3 and CN=5). While no additional construction events were reported following 2014, the test section was found to be milled and overlaid sometime after the last survey date in 2016 (the specific year of the event is still being determined), and therefore, the site has been placed Out of Study (OOS).

Table 1. Pavement structure for 18_1037 (CN=1)

Layer Number(s)	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Sandy Silty Clay
2	AC-Asphalt concrete layer	4.3	Hot Mixed, Hot Laid AC, Dense Graded
3	AC-Asphalt concrete layer	7.4	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	2.3	Hot Mixed, Hot Laid AC, Dense Graded
5	AC-Asphalt concrete layer	0.7	Hot Mixed, Hot Laid AC, Dense Graded

Table 2. Pavement structure for 18_1037 (CN=2)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soil: Silt
2	AC-Asphalt concrete layer	4.3	Hot Mixed, Hot Laid AC, Dense Graded
3	AC-Asphalt concrete layer	7.4	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	2.3	Hot Mixed, Hot Laid AC, Dense Graded
5	AC-Asphalt concrete layer	0	Hot Mixed, Hot Laid AC, Dense Graded
6	AC-Asphalt concrete layer	0.7	Hot Mixed, Hot Laid AC, Dense Graded
7	AC-Asphalt concrete layer	1.7	Hot Mixed, Hot Laid AC, Dense Graded

Table 3. Pavement structure for 18_1037 (CN=4)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soil: Silt
2	AC-Asphalt concrete layer	4.3	Hot Mixed, Hot Laid AC, Dense Graded
3	AC-Asphalt concrete layer	7.4	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	2.3	Hot Mixed, Hot Laid AC, Dense Graded
5	AC-Asphalt concrete layer	0	Hot Mixed, Hot Laid AC, Dense Graded
6	AC-Asphalt concrete layer	0.7	Hot Mixed, Hot Laid AC, Dense Graded
7	AC-Asphalt concrete layer	1.7	Hot Mixed, Hot Laid AC, Dense Graded
8	AC-Asphalt concrete layer	1.5	Hot Mixed, Hot Laid AC, Dense Graded

Pavement Structural Properties

Figure 1 shows the average FWD deflections under the nominal 9,000-pound load plate during the entire period the test section was in study. 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. As shown in Figure 1, the deflections reported were relatively low, ranging from 3.9 mils (1993) to 6.1 mils (2005), likely because of the overall thickness of AC layer of the pavement. The deflections observed appear to fluctuate based on the temperature at the time of collection. As the temperature increased, the deflections reported also increased, with the highest deflections observed in July and August. This relationship, which is further confirmed in Figure 2, helps explain the increase in the deflections observed following the overlay events. However, this observation still fails to explain why the reported deflections in 1994 and 1995 remained the same despite an increase in temperature between the two test dates.

The layer moduli backcalculated from the deflection data were also assessed for the test section. The pavement structure for test section 18_1037 was modeled as 14.5 inches of AC over subgrade (divided into two layers). Following CN=2 in 1994, the pavement structure for the test section was modeled as 16.2 inches of AC over subgrade (divided into two layers). Finally, after CN=4 in 2003, the test section was modeled as 17.7 inches of AC over subgrade (divided into two layers). It is important to note that for each construction event, the representative AC thicknesses used for the backcalculations were 0.2-inch less than the reported thickness of the section in the TST_L05B table. While the difference in the reported thickness and the thickness used for backcalcuations is small, additional information on the reason(s) for the deviation should be pursued. The backcalculated moduli for each layer for six collection dates between March 1993 and October 2011 (March 1993, February 1999, March 2003, August 2005, August 2010, and October 2011) are shown in Figure 3 through Figure 5. The collection of FWD data in 2016 was performed after the completion of the LTPP contract to backcalculate moduli data, and therefore, backcalculated moduli for this test date are not included in the LTPP database.

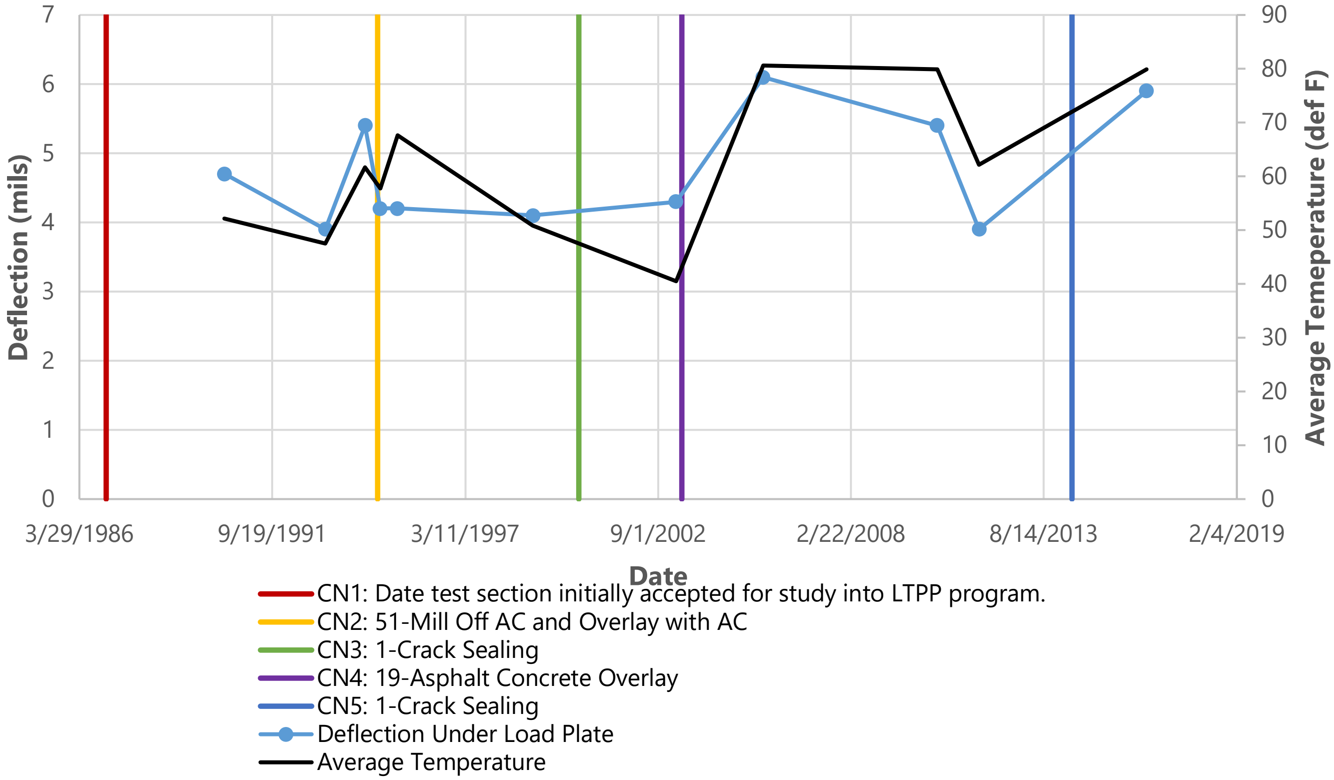


Figure 1. FWD deflections under the load plate and average daily temperature (using Modern-Era Retrospective analysis for Research and Applications, or MERRA data) over time.

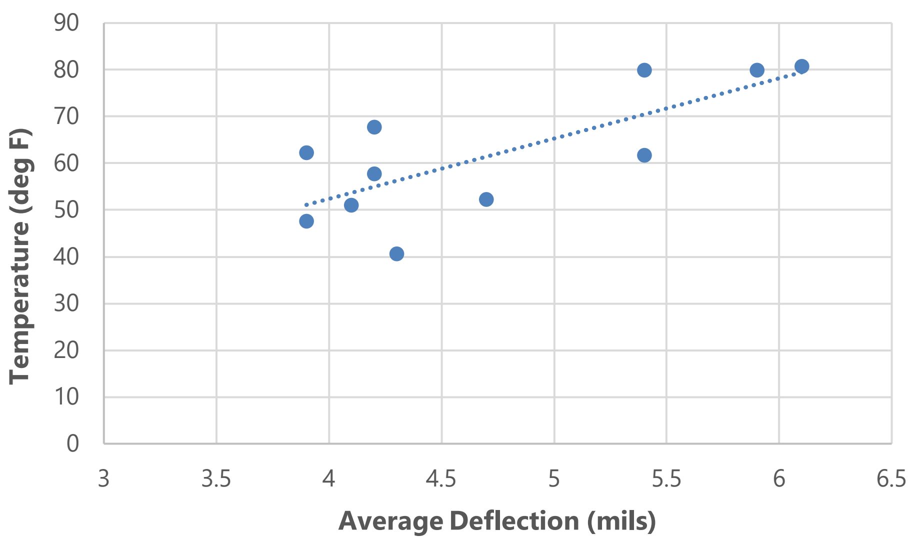


Figure 2. Relationship between deflection under the load plate and average daily temperature (using MERRA data).

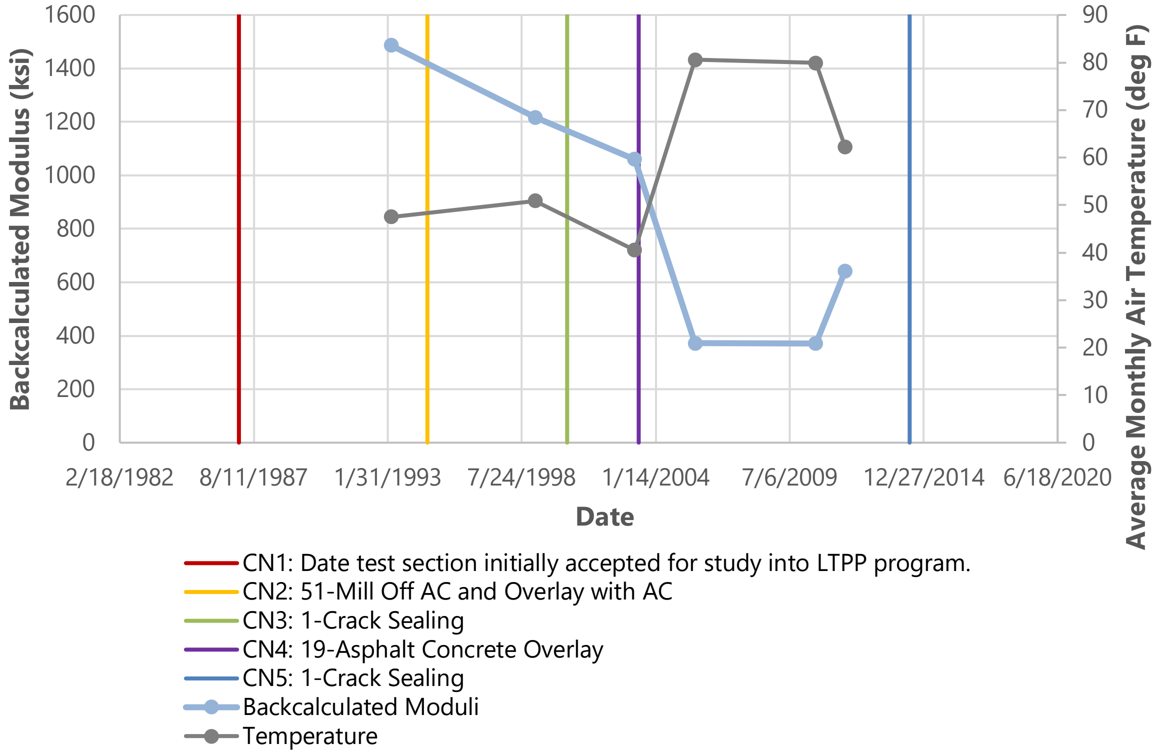


Figure 3. Average backcalculated modulus for AC (Layer 1) and average daily temperature (using MERRA data).

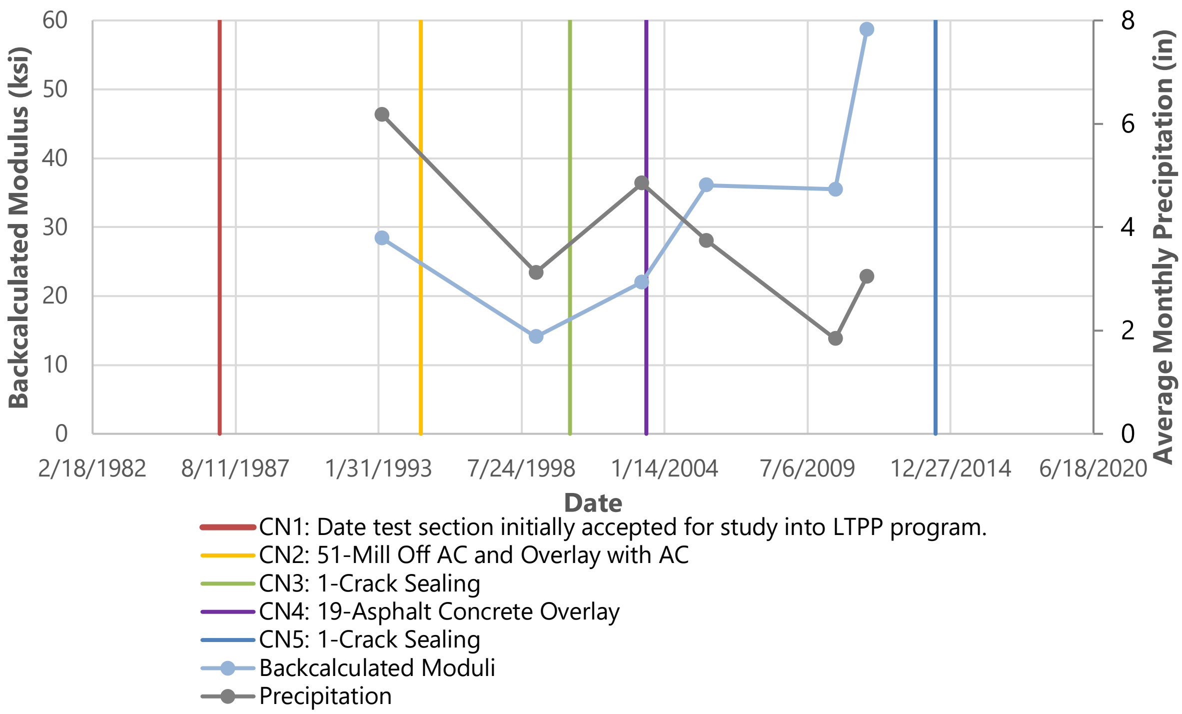


Figure 4. Average backcalculated modulus for top 24 inches of subgrade (Layer 2) and average monthly precipitation (using MERRA data).

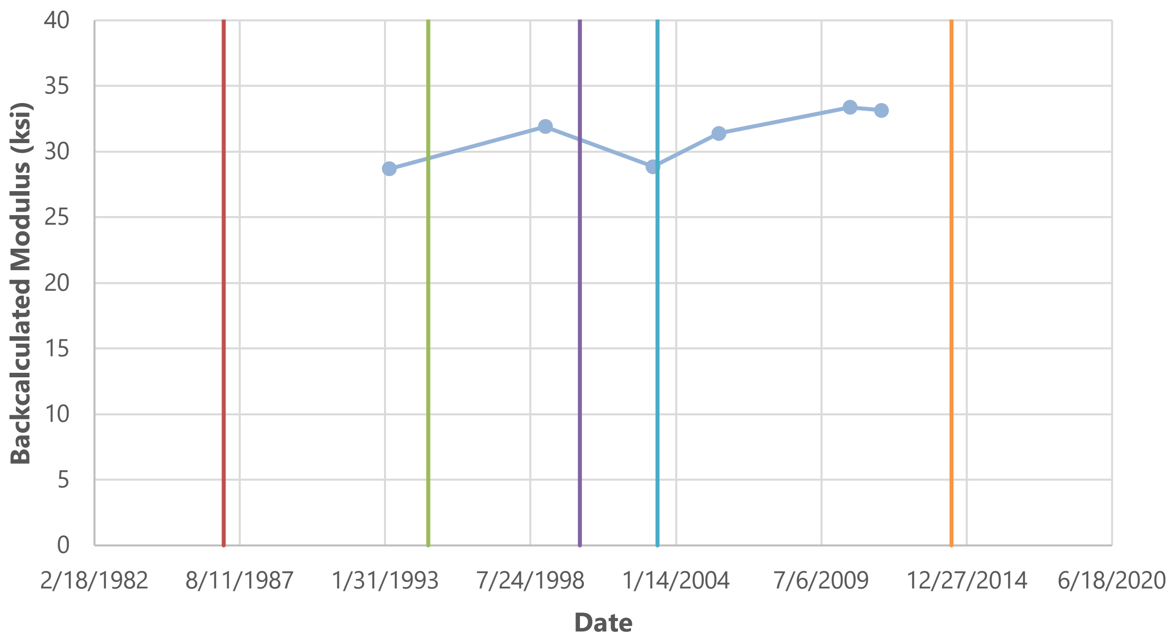


Figure 5. Average backcalculated modulus for subgrade (Layer 3).

As shown in the figures above, while the backcalculated moduli for Layer 3 appear to be reasonable and consistent throughout time, the reported backcalculated moduli for Layers 1 and 2 fluctuate over time. The moduli for Layer 1, the AC layer, appears to be related to the change in temperature over time. As depicted in Figure 2, as the average daily temperature reported at the test section increases, the moduli reported for Layer 1 also increases and vice versa. This helps explain why the modulus reported continued to decrease between the two overlay events, as the temperature at the time of testing also decreased over this period. Exceptions to this relationship include the decrease in the modulus in August 2005 despite the increase in temperature, which is likely related to changes in the pavement structure, and the increase in the modulus observed in 2011 despite a decrease in temperature. The moduli for Layer 2, the top 24 inches of the subgrade layer, seem to be affected by the moisture at the test section. As shown in Figure 4, as the average monthly precipitation reported at the test section increases, the modulus reported for Layer 2 also increases and vice versa. One exception to this relationship is the increase in the modulus reported in August 2005, despite a decrease in the average monthly precipitation.

The reasonableness of the backcalculated layer moduli was compared to moduli derived from laboratory resilient modulus testing. Table 4 summarizes the laboratory test results for the two AC overlay layers, conducted in 2004, and the subgrade layer, conducted in 1995 and 1996. For the AC layers, moduli values are shown for three test temperatures – 41, 77, and 104°F, respectively. As shown in Figure 6, the AC modulus versus temperature relationship for the field- (FWD-derived backcalculated moduli) and lab-derived resilient moduli appears to be reasonable; there appears to be a clear trend between temperature and the pavement modulus. For the subgrade layer, various statistical analyses were conducted for the range of stress states (confining and deviatoric stresses) to which the laboratory samples were subjected. The laboratory values for the subgrade were lower than the backcalculated moduli reported.

Table 4. Laboratory Resilient Modulus Test Results

Layer	Temperature (°F)	Number of Samples/test results	Range of moduli values (ksi)	Range of Confining Stress (psi)	Range of Maximum Nominal Axial Stress (psi)
AC-Layer 8 (1.5 in)	41	1 sample (2 tests, AC overlay)	2,353-2,642	N/A	N/A
	77	1 sample (2 tests, AC overlay)	898-1,103	N/A	N/A
	104	1 sample (2 tests, original AC layer and AC overlay)	309-506	N/A	N/A
AC-Layer 7 (1.7 in)	41	1 sample (2 tests, original AC layer and AC overlay)	2,181-2,564	N/A	N/A
	77	1 sample (2 tests, original AC layer and AC overlay)	871-998	N/A	N/A
	104	1 sample (2 tests, original AC layer and AC overlay)	347-380	N/A	N/A
Subgrade	N/A	2 samples (15 test results each)	2.2 to 5.8 (Average of 3.5)	2 to 6	1.9 to 9.9

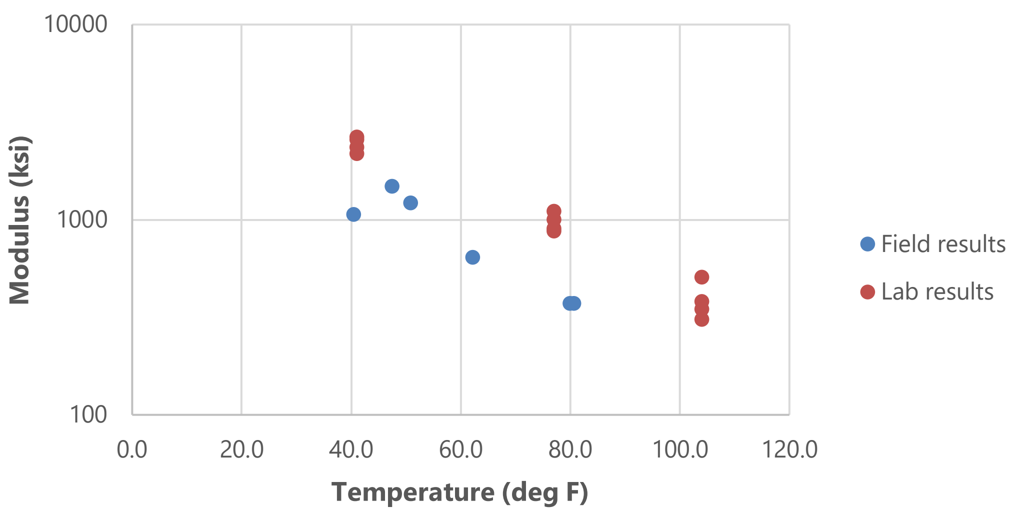


Figure 6. Field- and lab-derived AC resilient modulus values.

Climate History

The time history for average annual precipitation (from MERRA) since 1983 is shown in Figure 7. The mean precipitation recorded at the section was 55 inches for the period of analysis. Notable spikes in precipitation were observed in 2011 and 2018, when 75 and 81 inches of precipitation were reported, respectively.

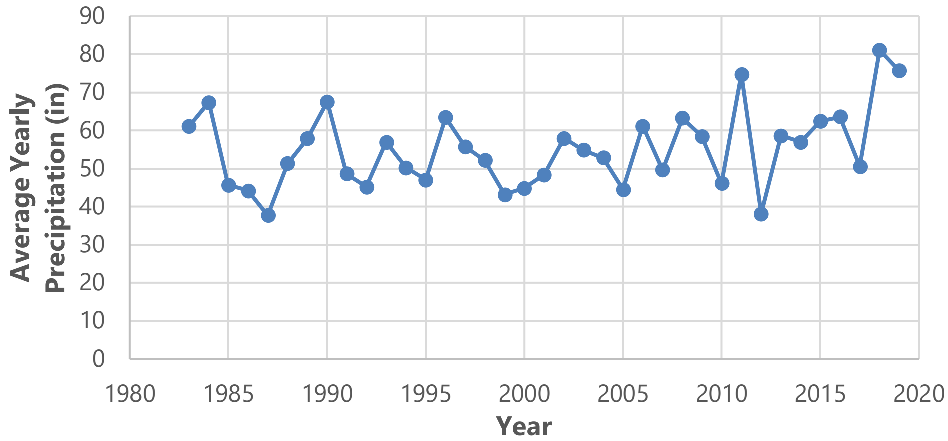


Figure 7. Average yearly precipitation over time.

Figure 8 shows the time history of the average annual freezing index (from MERRA) for the test site. The freezing index is the summation of the difference between freezing temperature and the average air temperature when it is less than freezing 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. As depicted in Figure 8, the freezing index values ranged from 97.2 deg F deg days (2006) to 581.4 deg F deg days (1989) during the analysis period. For the most part, all freezing indices reported during the analysis period are above the 150 deg F deg days used to classify a freeze region, and therefore, as stated earlier, this site would be classified as a being in a Freeze climate.

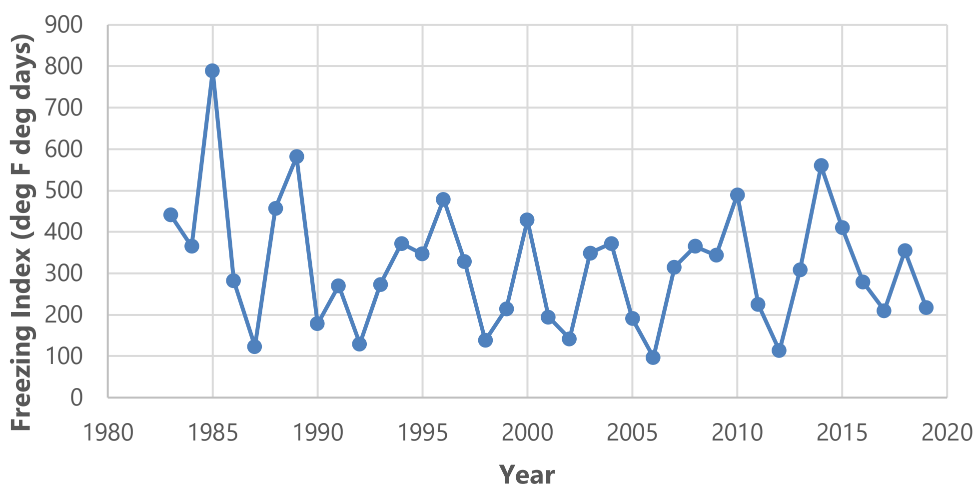


Figure 8. Average annual freezing index over time.

Truck Volume History

Figure 8 shows the annual average daily truck traffic (AADTT) data in the LTPP test lane by year. The annual truck traffic counts increase from 168 in 1983 to 596 in 2017, or approximately 13 additional trucks per day per year. The average number of ESALs reported on the section also increased over time as depicted in Figure 9. The number of ESALS increased from 29,434 in 1983 to 104,419 in 2017. The fluctuations in both the AADTT and ESALs reported for the test section are likely a result of the source of the data used over time. A combination of historical AADTT values (1983-1989), state provided AADTT values (1990-1991 and 2002), monitored values (1994, 1997, 1999, 2000, 2003, 2010, 2014), monitored values calculated from class data (1992, 1993, 1995, 1996, 1998, 2001, 2004, 2006-2009, 2011-2013), and values calculated using a compound growth function (2005 and 2015-2017) were used to report traffic along these test sections. Additionally, for the ESAL data reported in years when a major construction event occurred (such as the mill and overlay in 1994 and the overlay in 2003), the average annual ESALs was reported twice—once using data collected prior to the construction event and a second time using monitored data collected after the construction event. For example, in 1994, when the mill and overlay event occurred on the test section, the average annual ESALs for the section was calculated using monitored data collected before the September mill and overlay (monitored data collected between January and September of 1994) and separately using data collected after the mill and overlay event (monitored data collected between September and December of 1994). While in the figure below, the average annual ESALs reported in 1994 and 2003 is based on the data collected for a larger proportion of the year (the January to September monitored data for 1994), the average annual ESALs may be underrepresented because it is based on data collected for only a part of the year.

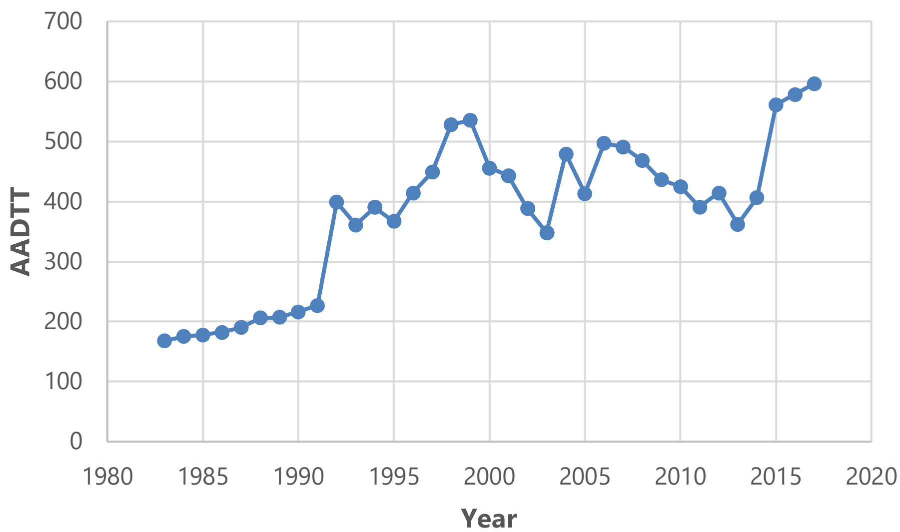


Figure 9. Average annual daily truck traffic (AADTT) history.

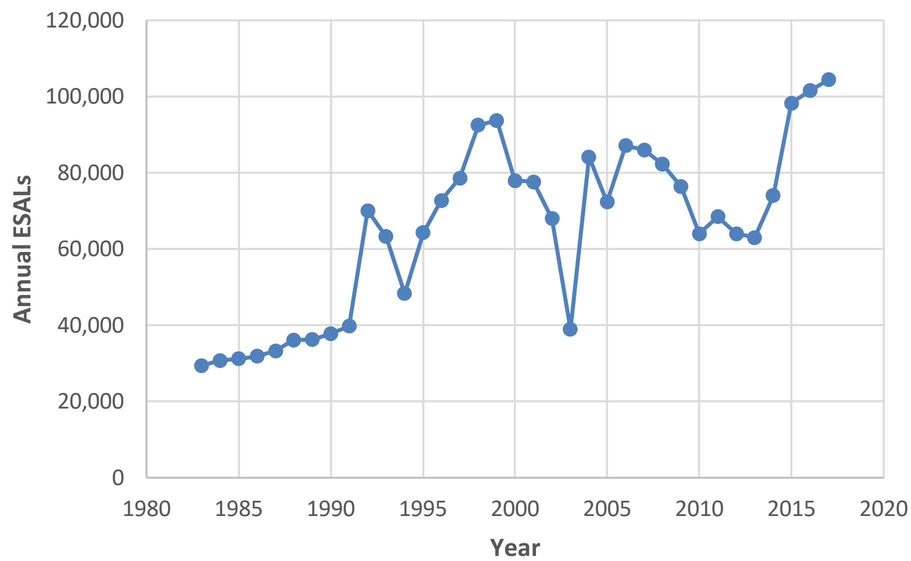


Figure 10. Estimated annual ESALs for vehicle classes 4-13 over time.

Pavement Distress History

The following summarizes the distresses observed on the test section until the last time it was monitored in 2016. Fatigue/alligator cracking, longitudinal cracking, transverse cracking, IRI, and rutting were assessed.

Fatigue/Alligator Cracking

Figure 11 shows the total reported area of fatigue/alligator cracking between 1993 and 2016. While the graph obtained from InfoPave™ is labelled fatigue cracking, which implies a mechanism, the distress values reported includes both fatigue cracking (inside the wheelpath) and alligator cracking (outside the wheelpath).

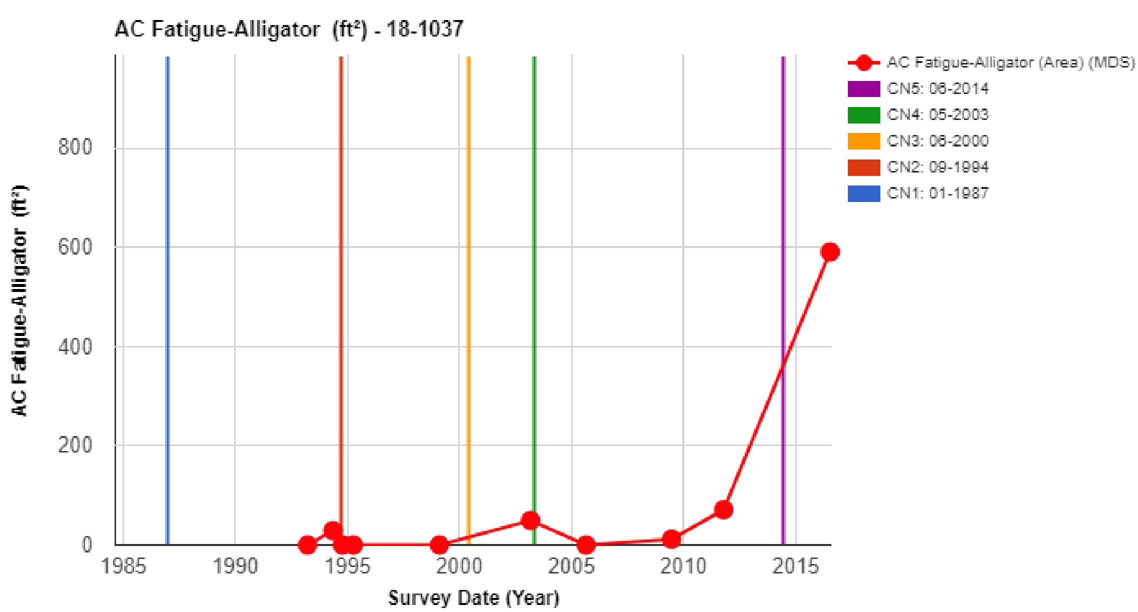


Figure 11. Time history of the length of fatigue cracking.

Fatigue/alligator cracking was first reported during the manual distress survey in May 1994, when 29.1 ft^2 was observed. After the mill and overlay event in September 1994, no fatigue/alligator cracking was reported again until the March 2003 distress survey, when 49.5 ft^2 of fatigue/alligator cracking was reported. Following the overlay event in May 2003, no fatigue cracking was reported again until the June 2009 distress survey when 11.8 ft^2 of fatigue/alligator cracking was observed. Once initiated, the fatigue/alligator cracking propagated at a rate of 82.7 ft^2 between 2009 and 2016, reaching 590.9 ft^2 in 2016.

As depicted in the figure, fatigue cracking was not significant on the test section until after 2010. It is hypothesized that delay in the appearance of significant fatigue cracking is related to the combination of the AC thickness of the test section and the application of AC overlays just as fatigue cracking appeared to propagate. The increase in the fatigue/alligator cracking between 2010 and 2016, may also be related to the increased levels of precipitation during this period, particularly in 2011 when 75 inches of precipitation was recorded in the area. As water infiltrates the pavement, unbound granular layers (subgrade only for test section in question) tend to weaken (especially when reaching saturation conditions), which can contribute to the observed fatigue cracking. It would have been interesting to see if the cracking was top-down or bottom-up (or some combination), but this will not be possible since the test section has been milled and overlaid.

Longitudinal Cracking

Non-wheel path (NWP) longitudinal cracking, depicted in Figure 12, was first reported during the manual distress survey in May 1994, when 554 feet of cracking was observed. After the mill and overlay event in September 1994, NWP longitudinal cracking was again reported during the distress survey conducted in February 1999, when 205 feet of NWP longitudinal cracking was observed. The NWP longitudinal cracking continued to increase between 1999 and March 2003, prior to the second overlay event in May 2003, at a rate of 170 feet/year between 1999 and 2003. Following the overlay event in May 2003, NWP longitudinal cracking was observed again during the June 2009 distress survey when 1,000 feet of NWP longitudinal cracking was observed. Once initiated, the NWP longitudinal cracking propagated at a rate of 31 feet/year between 2009 and 2016, reaching 1,219 feet in 2016.

Between the first and second overlay events (in 1994 and 2003, respectively) and after the second overlay event, NWP longitudinal cracking was predominantly located on the edge and centerline of the lane, indicating NWP longitudinal cracking observed prior to the second overlay event was reflected to the overlay surface. Given the location of the cracking, it is hypothesized that the propagation of the NWP longitudinal cracking is constructed-related as it appears the cracking may be located along construction joints.

No longitudinal cracking was observed inside the wheel path (WP) on the test section.

Transverse Cracking

Data on transverse cracking was collected between 1993 and 2016 as shown in Figure 13 and Figure 14. Transverse cracking was first reported during the manual distress survey in May 1994, when 301 feet of transverse cracking (47 cracks) was observed. Shortly after the mill and overlay event in September 1994, transverse cracking was observed again during the April 1995 distress survey when 115 feet (35 cracks) of transverse cracking was reported. The transverse cracking continued to increase between 1995 and March 2003, at a rate of 28 feet/year between 1995 and 2003. Following the overlay event in May 2003, transverse cracking was reported again during the June 2009 distress survey when 175 feet (30 cracks) of transverse cracking was observed. Unlike after the mill and overlay event, there was more time before transverse cracking appeared after the second overlay event and fewer cracks once cracking was initiated. Once initiated, the transverse cracking propagated at a rate of 26 feet/year between 2009 and 2016, reaching 354 feet (44 cracks) in 2016. The increase in the transverse cracking length and count between

2011 and 2016 was slowed as some cracks were sealed during 2014. The rate of propagation between the first and second overlay events (in 1994 and 2003) and after the second overlay event was similar. In both cases, transverse cracking was located in similar locations, indicating transverse cracking observed prior to the second overlay event was reflected to the overlay surface. It is hypothesized that the propagation of the transverse cracking is primarily related to the freeze-thaw periods (evidenced by the high freezing indices) of the pavement section over time.

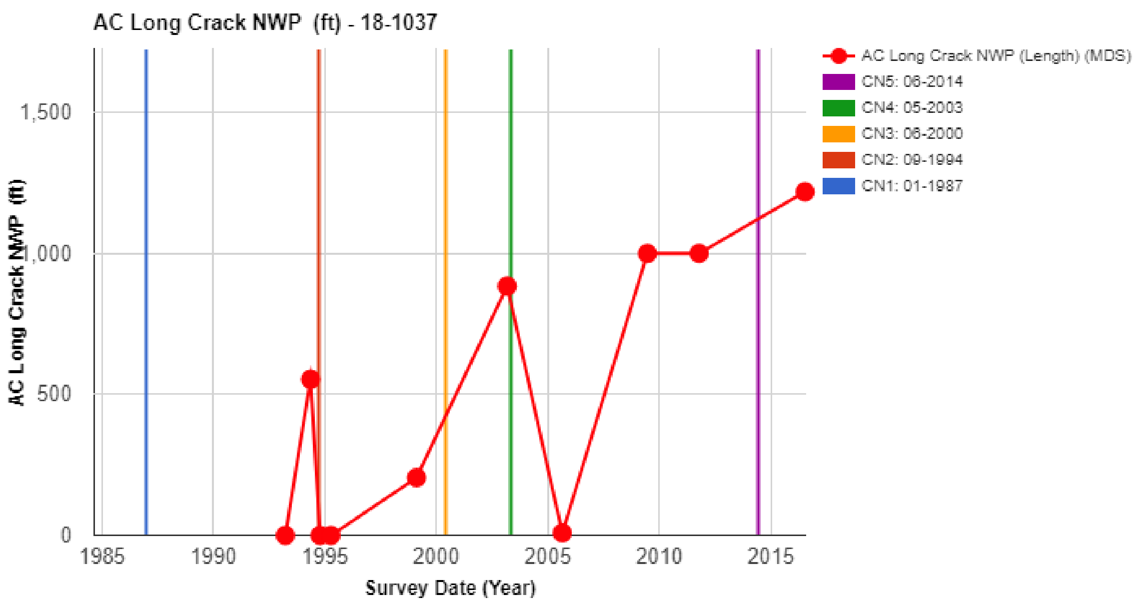


Figure 12. Time history of the length of NWP longitudinal cracks.

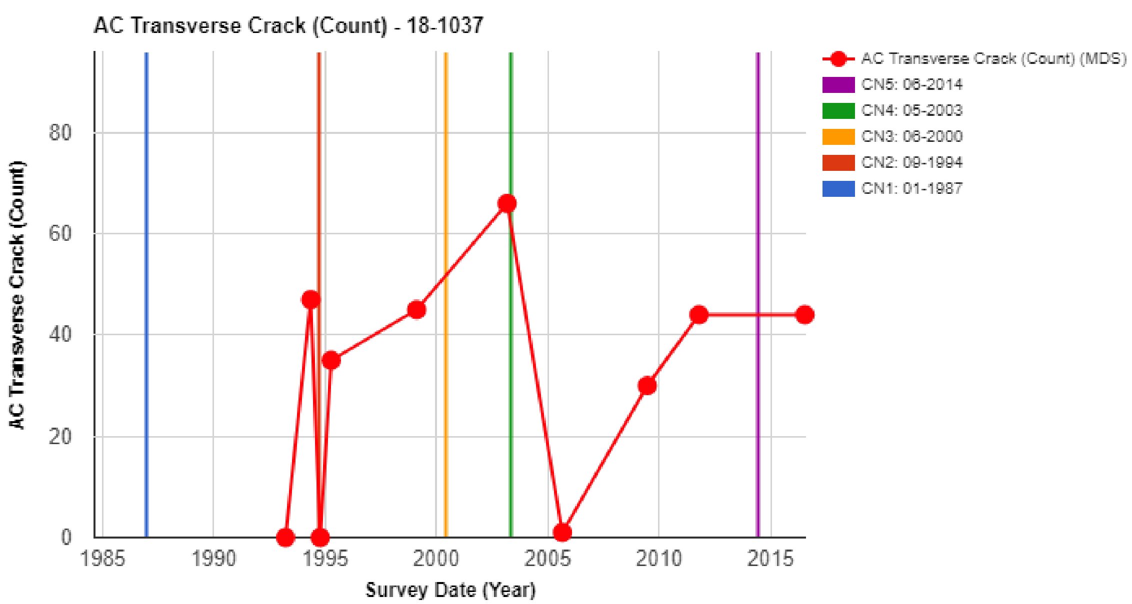


Figure 13. Time history of the number of transverse cracks.

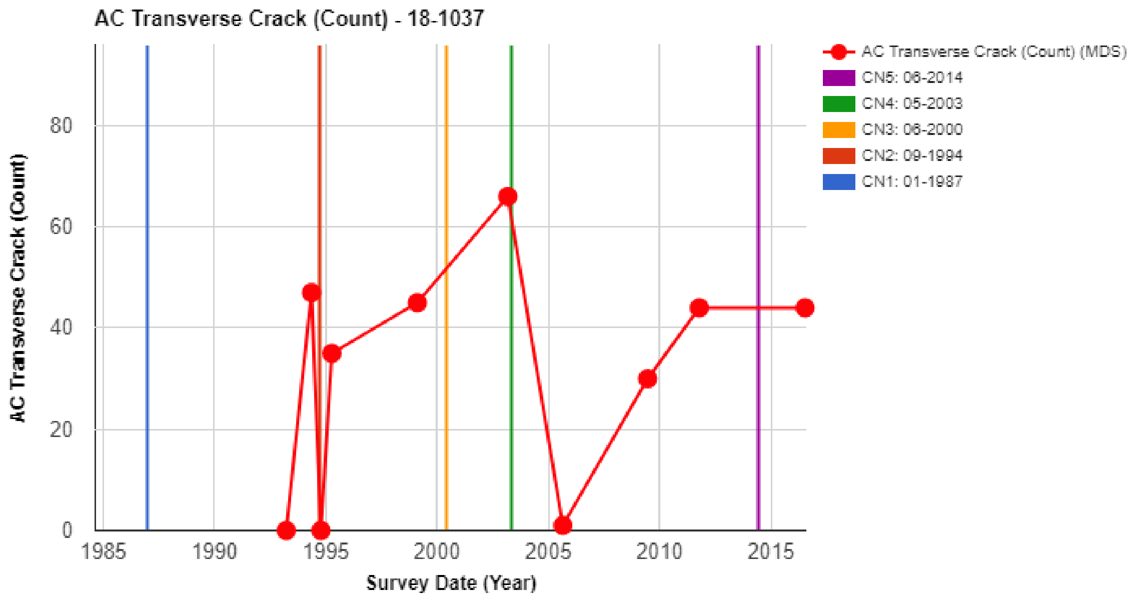


Figure 14. Time history of the length of transverse cracking.

IRI

The average IRI measurements for the section over time are shown in Figure 15. During the first performance period of the test section, from its incorporation into the LTPP program to the mill and overlay event in 1994, the IRI on the test section slightly increased over time. The IRI on the section prior to the mill and overlay event in 1994 averaged 125 in/mi, which means the performance of the pavement is classified as “Fair” based on FHWA performance definitions. During the second performance period, between the 1994 mill and overlay and the 2003 overlay event, the IRI of the test section dropped to 46 in/mile in 1995 before increasing again at a steady rate of 6 in/mi. The average IRI during this performance period is classified as “Good” based on FHWA performance definitions. Finally, during the third performance period, following the 2003 overlay event, the IRI again dropped to 42 in/mile before increasing at a rate of 1 in/mi between 2008 and 2017. The average IRI during this performance period is classified as “Good” based on FHWA performance definitions. It is important to note that after the 2003 overlay event IRI is not reported on the section again until 2008 and increases at a rate that is noticeably slower than the rate of IRI increase during both the first and second performance periods. It is hypothesized that there may have been an additional unreported M&R event between the 2003 overlay and the next time IRI is reported on the section. Further investigation of the reason for this gap between IRI collection dates is recommended.

The IRI reported during the three performance periods does not correlate with the observed cracking over time. Specifically, the predominant cracking types that purportedly affect the overall IRI of the test section—fatigue/alligator and transverse cracking—are present in higher quantities between the two overlay events in 1994 and 2003 (for transverse cracking only) and following the 2003 overlay event (for transverse and fatigue/alligator cracking) despite the lower average IRI reported during these periods. Prior to the 1994 mill and overlay event, when less fatigue/alligator and transverse cracking is present, IRI is at its highest. This may be related to the severity of the cracking observed on the section—predominantly low and medium—which plays less of a role in the roughness of the test section. Additionally, since the initial IRI of test section (at the time of its incorporation in the LTPP program in 1987) is unknown, it is also possible the initial roughness or IRI of the test section was not as smooth as the IRI reported after CN=2 and CN=4.

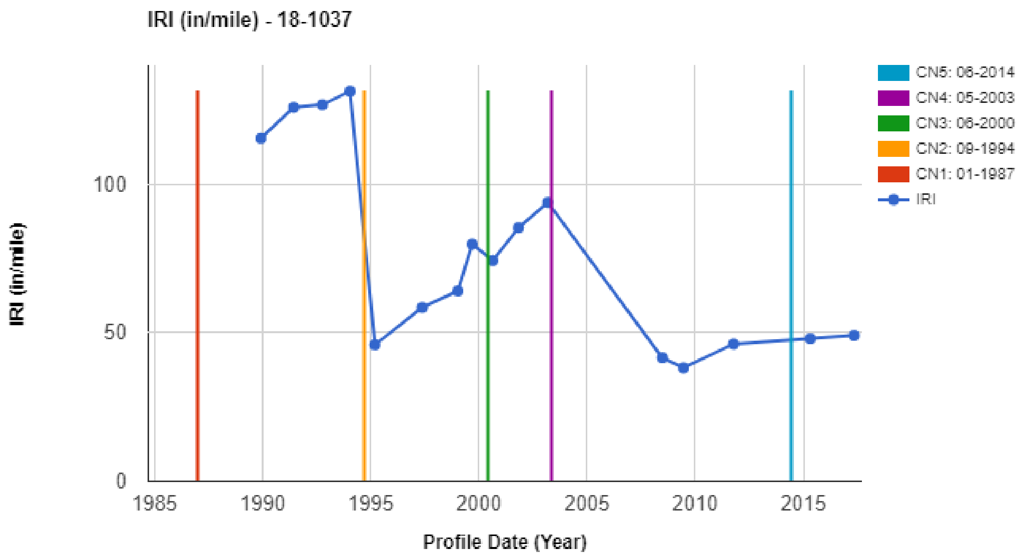


Figure 15. Time history plot of pavement roughness.

Rutting

The average rut depths observed for the section over time are shown in Figure 16. The rutting on the section prior to the first overlay in 1994 averaged 0.51 in. Following the mill and overlay in 1994, the average rut depth dropped to 0.04 in. However, the average rut depth began to increase following the overlay, at a rate of 0.03 in/year between 1995 and 2003. Following the second overlay event in 2003, the average rut depth again decreased this time to 0.12 in in 2005. The average rut depth slightly increased between 2005 and 2016 to 0.31 in. It is hypothesized that the rutting observed prior to 1994 occurred within the top 0.7-inch AC layer (which was effectively removed during the mill and overlay event in 1994, leading to lower rutting values following the overlay).

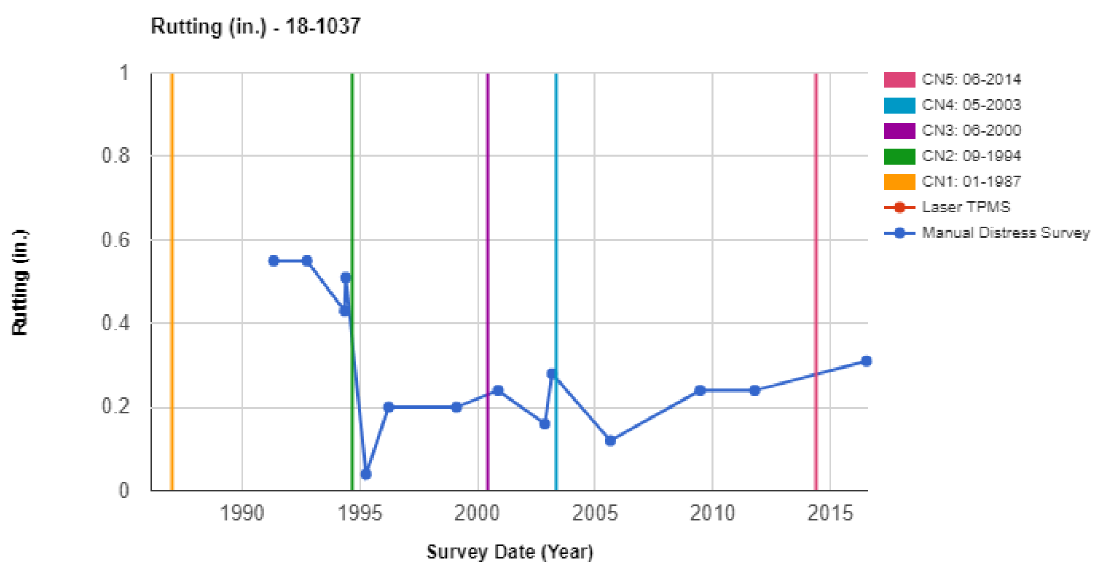


Figure 16. Time history plot of average rut depth.

SUMMARY OF FINDINGS

LTPP test section 18_1037 was located on State Route 66, eastbound, in Spencer County, Indiana. State Route 66 is a rural minor arterial with one lane in the direction of traffic. The test section was constructed in 1983 and was accepted into the LTPP Program as part of the GPS-1 experiment in January 1987. The pavement structure at the time of its incorporation into the LTPP program consisted of 14.7 inches of asphalt concrete (AC) (across 4 layers) over a fine-grained subgrade layer. The site received two overlays, a mill and 1.7-inch AC overlay in September 1994, and a 1.5-inch overlay in May 2003, moving the section first to the GPS-6S: AC Overlay on AC Pavement with Milling and/or Fabric experiment, and subsequently to the GPS-6D AC Overlay of AC Pavement using a Conventional AC overlay experiment. Prior to and following the second overlay, in June 2000 and June 2014, the test section received crack sealing. While no additional construction events were reported following 2014, the test section was found to be milled and overlaid sometime after the last survey date in 2016 (the specific year of the event is still being determined), and therefore, the site is now considered Out of Study (OOS).

The memorandum was focused on the following:

1. **Investigating the cause(s) of the rutting depths observed, particularly prior to the first overlay in 1994**. The rutting on the section prior to the first overlay in 1994 averaged 0.51 in. Following the mill and overlay in 1994, the average rut depth dropped to 0.04 in. It is hypothesized that the rutting observed prior to 1994 occurred within the top 0.7-inch AC layer and was effectively removed during the mill and overlay event in 1994, leading to lower rutting values following this event.
2. **Investigate the reason(s) for increased cracking following the second overlay in 2003.** The increase in the cracking is likely related to related to a combination of aging/structural deterioration of the pavement section over time and the increased levels of precipitation during this period, particularly in 2011 when 75 inches of precipitation was recorded in the area. As water infiltrates the pavement layers, the unbound granular layers (subgrade only for test section in question) tend to weaken (especially when reaching saturation conditions) causing the increase in cracking observed. It is also hypothesized that the propagation of the cracking is. For transverse cracking specifically, it is hypothesized that the propagation of cracking is primarily related to the freeze-thaw periods (evidenced by the high freezing indices) of the pavement section over time.
3. **Further investigate the reason(s) for the performance of the pavement in terms of FWD deflections and IRI**. The deflections reported on the test section were relatively low, ranging from 3.9 mils (1993) to 6.1 mils (2005), which correlates with the substantial overall thickness of the AC layers. The IRI reported on the section did not seem to be correlated to the cracking reported throughout time; while there was low IRI values reported on the test section, there was significant cracking observed. This may be related to the severity of the cracking observed on the section—predominantly low and medium—which plays less of a role in the roughness of the test section. Additionally, since the initial IRI of test section (at the time of its incorporation in the LTPP program in 1987) is unknown, it is also possible the initial roughness or IRI of the test section was not as smooth as the IRI reported after CN=2 and CN=4.

FORENSIC EVALUATION RECOMMENDATIONS

While the test section was reported as active when it was initially nominated for investigation, as noted earlier, this test section was found to have been milled and overlaid when preparing to schedule the field evaluation. For this reason, no follow-up field investigations are recommended for this test section. It is suggested, however, that the FHWA LTPP Team investigate:

- Small differences in the reported layer thicknesses and the thicknesses used for backcalculation to help clarify and inform users of the LTPP database of these deviations,
- Reason(s) for the 1994 mill and overlay and the 2003 overlay – it is hypothesized that NWP and transverse cracking together with IRI were the main drivers, but this needs to be confirmed.
- Reason(s) for the slowed increase in IRI after the 2003 overlay. It is hypothesized that there may have been an additional unreported M&R event between the 2003 overlay and the next time IRI is reported on the section in 2008.