Technical Memorandum

To: Kim Schofield

From: Lauren Gardner, Gonzalo Rada, Kevin Senn, and Jose Medina

cc: Mustafa Mohamedali

Date: January 23, 2021, May 11, 2021 (Addendum)

Re. Forensic Desktop Study Report: Pennsylvania LTPP Test Section 42_1597

The Long-Term Pavement Performance (LTPP) General Pavement Studies-1 (GPS-1) Asphalt Concrete (AC) Pavement on Granular Base test section 42_1597[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." This test section was originally constructed in 1980 and was incorporated into the LTPP program in 1988 as 6.5 inches of asphalt concrete (split between two layers) and 16.8 inches of unbound granular base over a fine-grained subgrade soil. During the first 20 years of the pavement's history, the test section received five treatments—crack sealing in June 1990, June 1996, and May 1999, patching in August 1997, and a shoulder restoration and mill and overlay in July 2000, which moved the test section to GPS-6S AC Overlay of Milled AC Pavement Using Conventional or Modified Asphalt. Following the mill and overlay in 2000, the pavement section received crack sealing in June 2011, a slurry seal in June 2015, and crack sealing again in June 2019. The overlay event in 2000, which occurred approximately 20 years after the construction of the pavement section, provides an opportunity to assess and compare the condition and performance of the pavement prior to and following the rehabilitation event. Accordingly, the purpose of this study was to investigate 1) the cause(s) of the fatigue cracking following the mill and overlay in 2000, 2) whether any of the cracking observed prior to the mill and overlay (specifically longitudinal and transverse cracking) was reflected following the mill and overlay, 3) the cause(s) of the high IRI and rutting values (256 in/mi and 0.31 in, respectively, in 2000) prior to the mill and overlay event, and 4) differences in the initiation and propagation of cracking prior to and following the mill and overlay.

SITE DESCRIPTIONS

LTPP test section 42_1597 is located on State Route 49, eastbound, in Tioga County, Pennsylvania. State Route 49 is a rural minor arterial with one lane in the direction of traffic. The test section is classified as being in a Wet, Freeze climate zone. The coordinates (in degrees) of the site are 41.97236, -77.2385. Photograph 1 shows the section at Station 0+00 looking eastbound in 2016, while Map 1 shows the geographical location of the test section. Although the test section was in active status at the time it was

[1] First two digits in test section number represent the State Code [42 = Pennsylvania]. 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.

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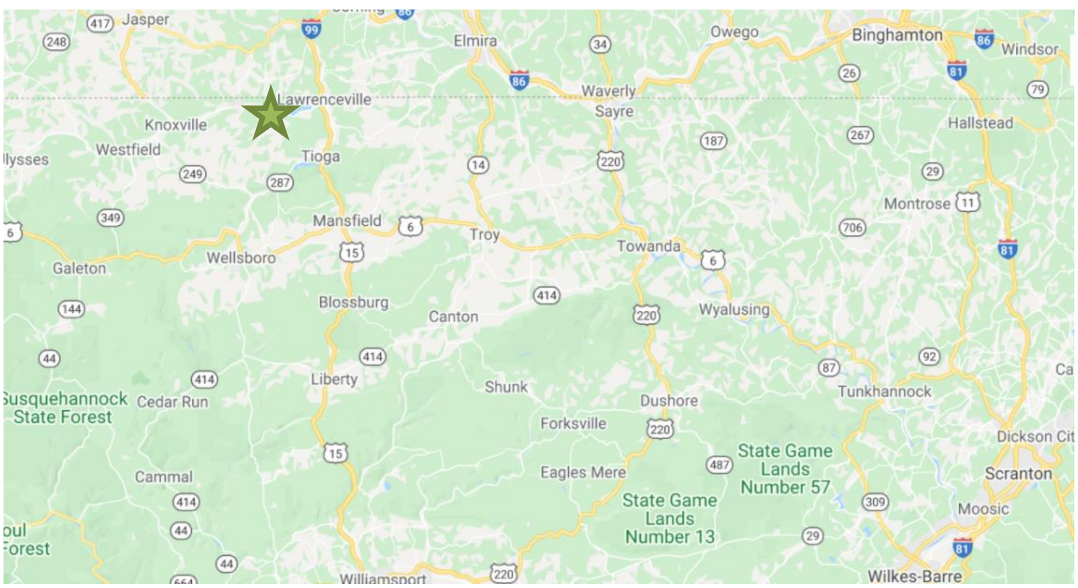
recommended for a desktop study, closeout monitoring occurred on the test section in September 2020. Therefore, the section is anticipated to officially go out of study soon.

Photograph 1. LTPP Section 42_1597 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 1980 and was accepted into the LTPP Program as part of the GPS-1 experiment in August 1988. The pavement structure at the time of its incorporation into the LTPP program consisted of 6.5 inches of asphalt concrete (split between two layers) and 16.8 inches of unbound granular base over a fine-grained subgrade soil. This pavement structure is summarized in Table 1 and corresponds to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. The next major construction event occurred in July 2000, when the test section received a shoulder restoration and a 1.5-inch mill and 6.6-inch AC overlay (over three layers with three different mix types), moving the test section to the GPS-6S: AC Overlay of Milled AC Pavement Using Conventional or Modified Asphalt experiment. Table 2 summarizes the pavement structure following the mill and overlay which corresponds to CONSTRUCTION_NO = 6 (CN = 6). An additional construction event in June 2015 (CN=8), a slurry seal, also resulted in 0.3-inch increase to the pavement structure as shown in Table 3. Other minor construction events that occurred on the test section included crack sealing in June 1990 (CN=2), June 1996 (CN=3), May 1999 (CN=5), June 2011 (CN=7), and June 2019 (CN=9) and patching in August 1997 (CN=4). Additionally, it is important to note, while not changing the overall pavement structure, rumble strips were placed in the centerline of the roadway sometime between 2003 and 2007.

Table 1. Pavement structure for 42_1597 (CN=1)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Gravelly Lean Clay
2	Unbound (granular) base	16.8	Gravel (Uncrushed)
3	AC-Asphalt concrete layer	5	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	1.5	Hot Mixed, Hot Laid AC, Dense Graded

Table 2. Pavement structure for 42_1597 (CN=6)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Gravelly Lean Clay
2	Unbound (granular) base	16.8	Gravel (Uncrushed)
3	AC-Asphalt concrete layer	5	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	0	Hot Mixed, Hot Laid AC, Dense Graded
5	AC-Asphalt concrete layer	2.2	Hot Mixed, Hot Laid AC, Dense Graded
6	AC-Asphalt concrete layer	2.5	Hot Mixed, Hot Laid AC, Dense Graded
7	AC-Asphalt concrete layer	1.9	Hot Mixed, Hot Laid AC, Dense Graded

Table 3. Pavement structure for 42_1597 (CN=8)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Gravelly Lean Clay
2	Unbound (granular) base	16.8	Gravel (Uncrushed)
3	AC-Asphalt concrete layer	5	Hot Mixed, Hot Laid AC, Dense Graded
4	AC-Asphalt concrete layer	0	Hot Mixed, Hot Laid AC, Dense Graded
5	AC-Asphalt concrete layer	2.2	Hot Mixed, Hot Laid AC, Dense Graded
6	AC-Asphalt concrete layer	2.5	Hot Mixed, Hot Laid AC, Dense Graded
7	AC-Asphalt concrete layer	1.9	Hot Mixed, Hot Laid AC, Dense Graded
8	AC-Asphalt concrete layer	0.3	Slurry Seal

Pavement Structural Properties

Error! Reference source not found. shows the average FWD deflections under the nominal 9,000-pound load plate. 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 **Error! Reference source not found.**, the deflections reported ranged from 9.7 mils (May 1990) to 15.6 mils (June 1994 and April 2000) prior to the mill and overlay and from 4.6 mils (November 2000) to 10.6 mils (July 2003) following the mill and overlay; the reported deflections were higher prior to the mill and overlay event in 2000. Following this construction event, when the overall AC thickness increased, the deflections subsequently decreased. As shown in the figure, the fluctuations in the deflections reported prior to and following the mill and overlay event appear to be affected by the air temperature at the time of testing. Generally, as the air temperature at the time of testing increased, the deflections reported increased. Notable exceptions to this trend included the deflections reported in September 1995, April 2000, and June 2009.

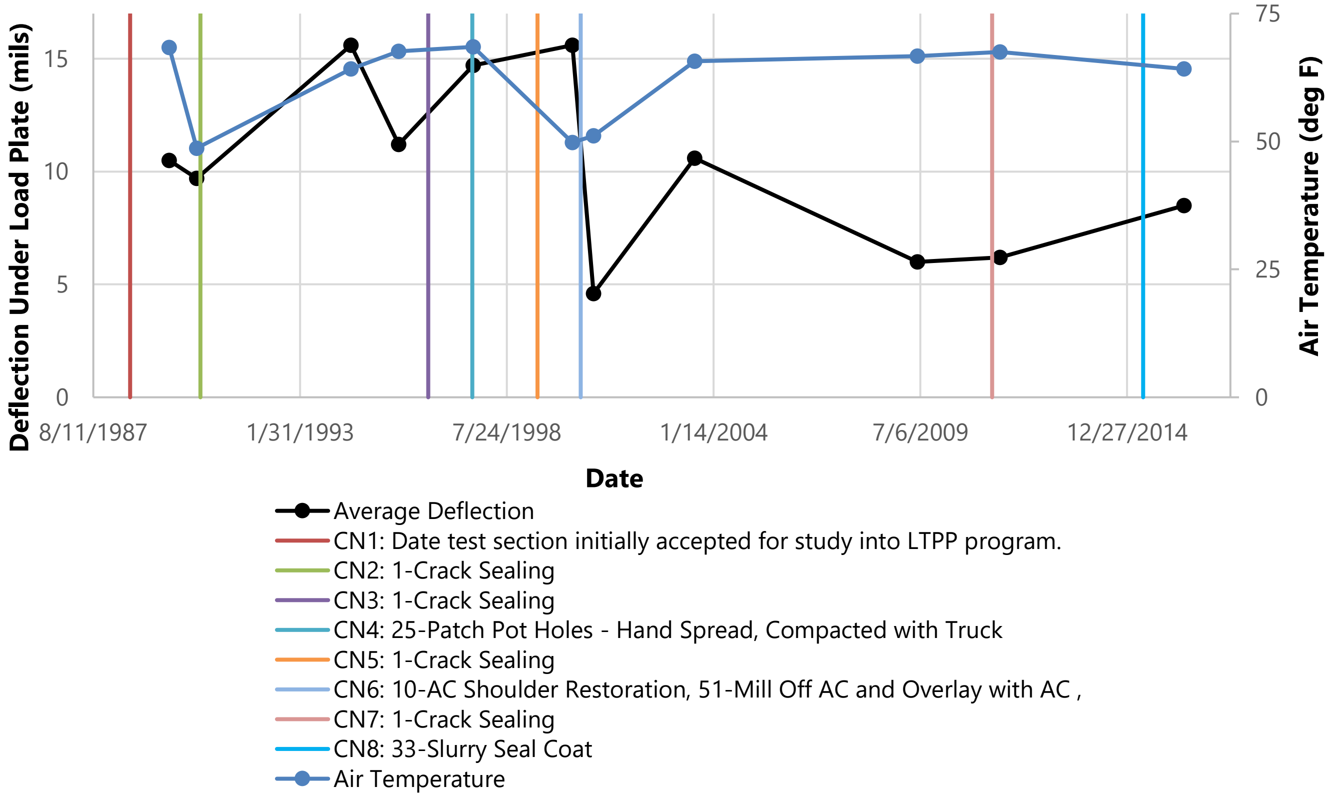


Figure 1. FWD deflections under the load plate over time with air temperature and the 7-day precipitation at test section.

The layer moduli backcalculated from the deflection data were also assessed for the test section. Prior to the 2000 mill and overlay, the pavement structure for test section 42_1597 was modeled as 6.4 inches of AC (0.1-inch less than reported thickness) and 16.4 inches of typical granular base (0.4-inch less than the reported thickness) over subgrade (divided into two layers). Following the mill and overlay in 2000, the pavement structure was modeled as 11.2 inches of AC (0.4-inch less than the reported thickness) and 16.4 inches of typical granular base (0.4-inch less than the reported thickness) over subgrade (divided into two layers). It is important to note that for each construction event, the representative thicknesses used for the backcalculations were less than the reported thicknesses. While the difference in the reported thicknesses and the thicknesses used for backcalculations are small, additional information on the reason(s) for the deviation should be pursued. The backcalculated moduli for each layer for ten collection dates between August 1989 and August 2011 and cumulative precipitation reported the 7 days leading up to the FWD collection dates (based on MERRA data) are shown in Figure 2 through Figure 5. The collection of FWD data in 2016 was performed after the completion of the LTPP contract to backcalculate moduli data; therefore, backcalculated moduli for this test date are not included in the LTPP database.

As shown in the figures below, in addition to temperature affecting the structural properties of the section at the time of testing, moisture (measured as the cumulative precipitation reported the 7 days leading up to the FWD collection dates) also played a role in the reported moduli over time. For each of the four layers, increases in the 7-day precipitation typically led to increases in the reported moduli. For the AC layer (Layer 1), prior to the mill and overlay event in 2000, the moduli reported ranged from 357 ksi to 1,263 ksi. Following the mill and overlay event, the moduli reported for Layer 1 ranged from 213 ksi to 985 ksi. The range of values reported for Layers 2, 3, and 4 prior to the 2000 mill and overlay event was 15-20 ksi, 12-30 ksi, and 54-70 ksi, respectively. Following the mill and overlay, the range of values reported for Layers 2, 3, and 4 was 25-38 ksi, 17 to 30 ksi, and 72-90 ksi. For all four layers, the moduli values reported directly after the mill and overlay increased despite an increase in the overall pavement thickness. This may be the result of stress hardening; a thicker pavement structure may have led to an increase in stress and therefore, a higher modulus.

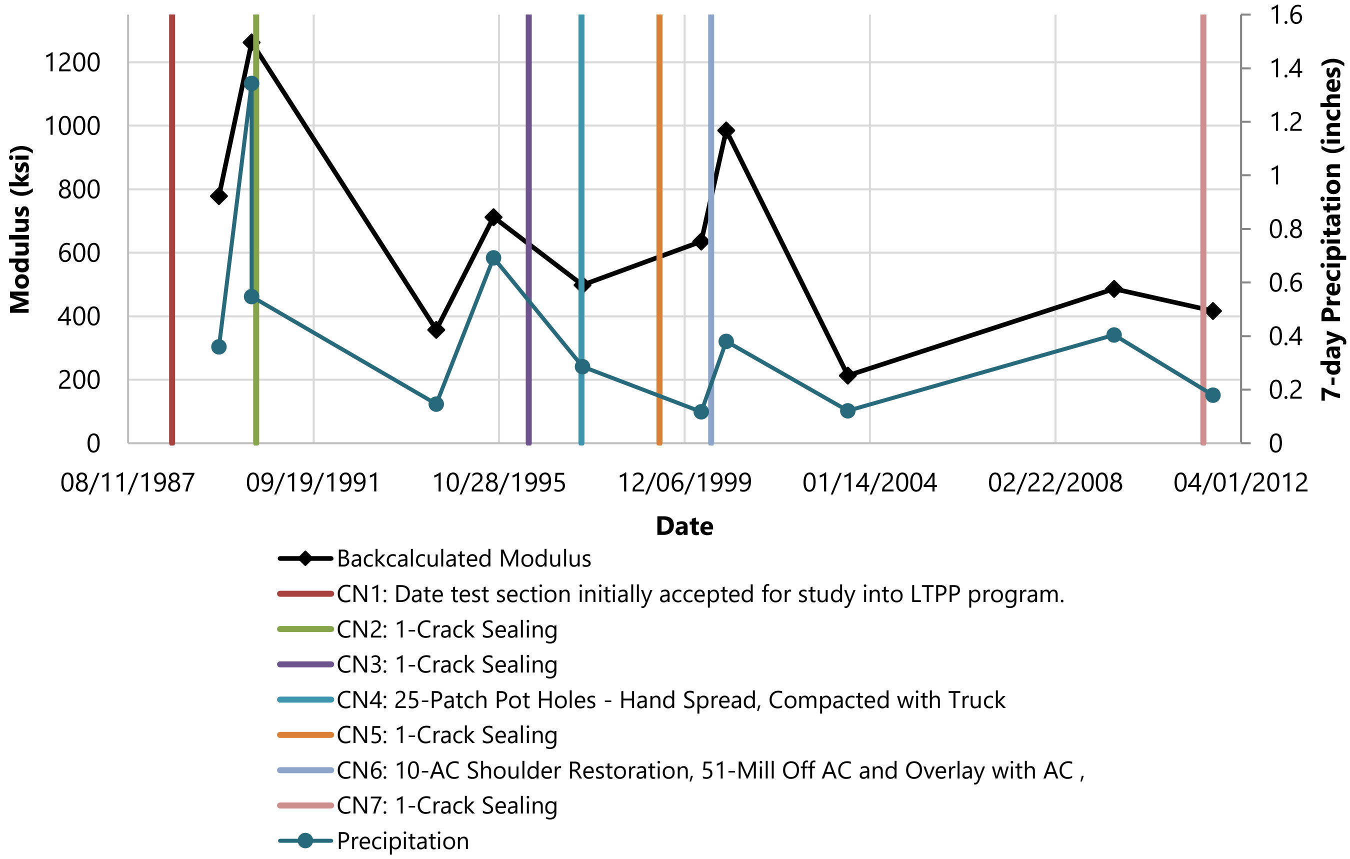


Figure 2. Average backcalculated modulus for AC (Layer 1) and the 7-day precipitation at test section.

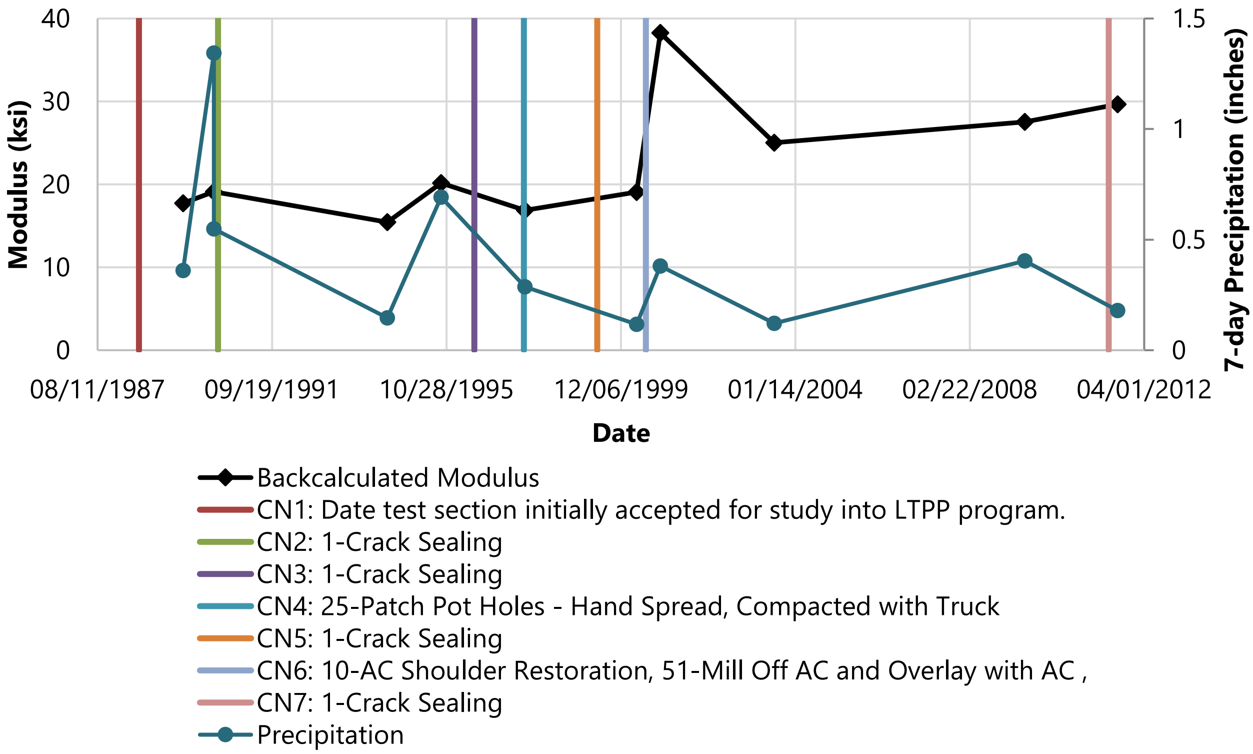


Figure 3. Average backcalculated modulus for base layer (Layer 2) and the 7-day precipitation at test section.

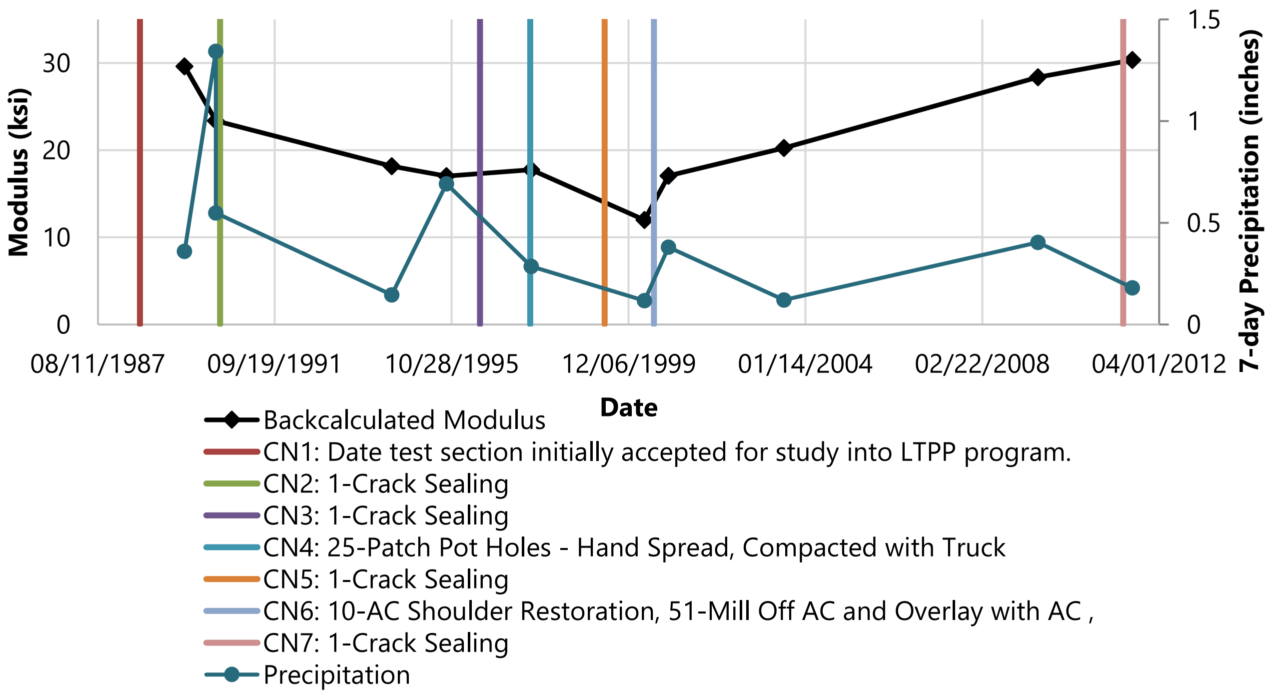


Figure 4. Average backcalculated modulus for the first 24-inches of subgrade (Layer 3) and the 7-day precipitation at test section.

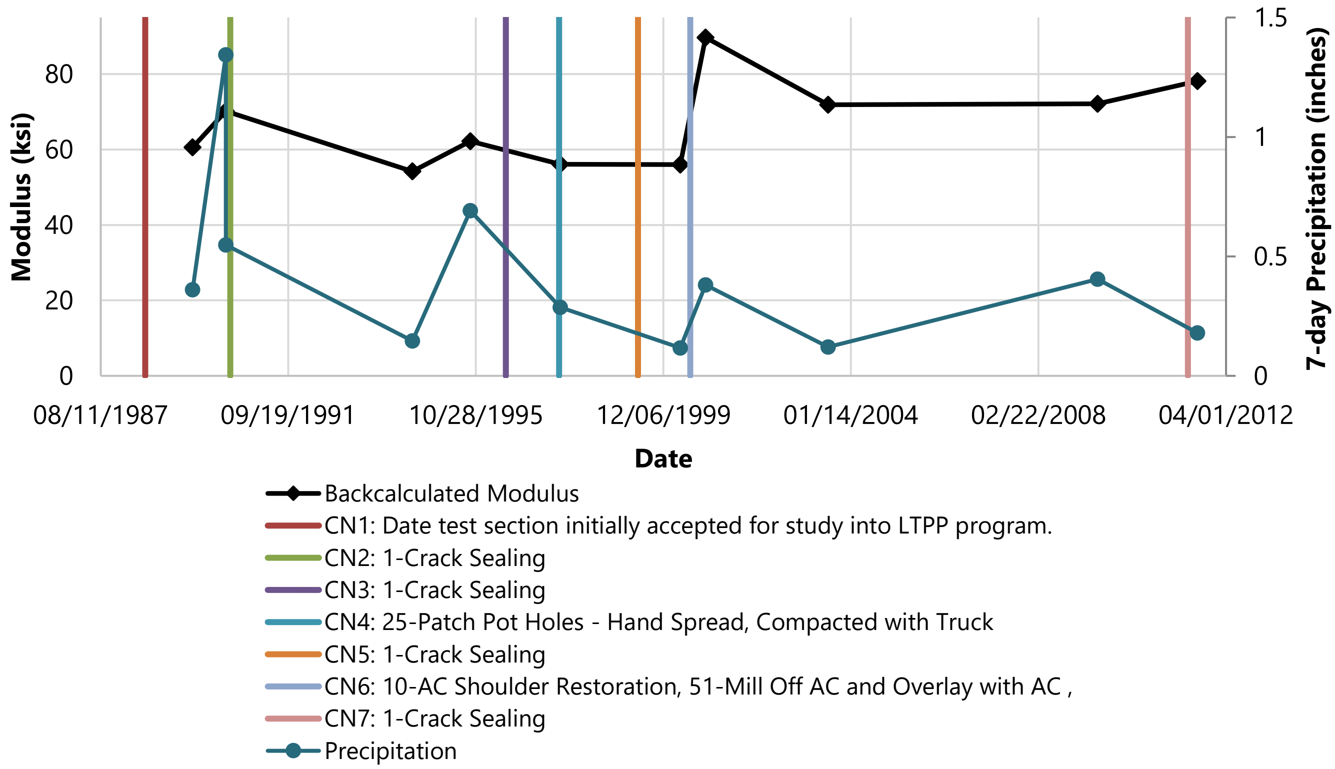


Figure 5. Average backcalculated modulus for remaining subgrade (Layer 4) and the 7-day precipitation at test section.

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 two AC layers (Layer 5 and Layer 6 in the LTPP database) and the subgrade layer. 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-measured 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 slightly lower than the backcalculated moduli reported for the top 24 inches of subgrade.

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 6 (2.2 in)	41	1 sample (2 tests)	1,209-1,305	N/A	N/A
	77	1 sample (2 tests)	322-397	N/A	N/A
	104	1 sample (2 tests)	68-113	N/A	N/A
AC-Layer 5 (2.5 in)	41	1 sample (2 tests)	1,101-1,466	N/A	N/A
	77	1 sample (2 tests)	389-418	N/A	N/A
	104	1 sample (2 tests)	113-116	N/A	N/A
Subgrade	N/A	2 samples (15 test results each)	9.4 to 17.1 (Average of 14.4)	2 to 6	2 to 10

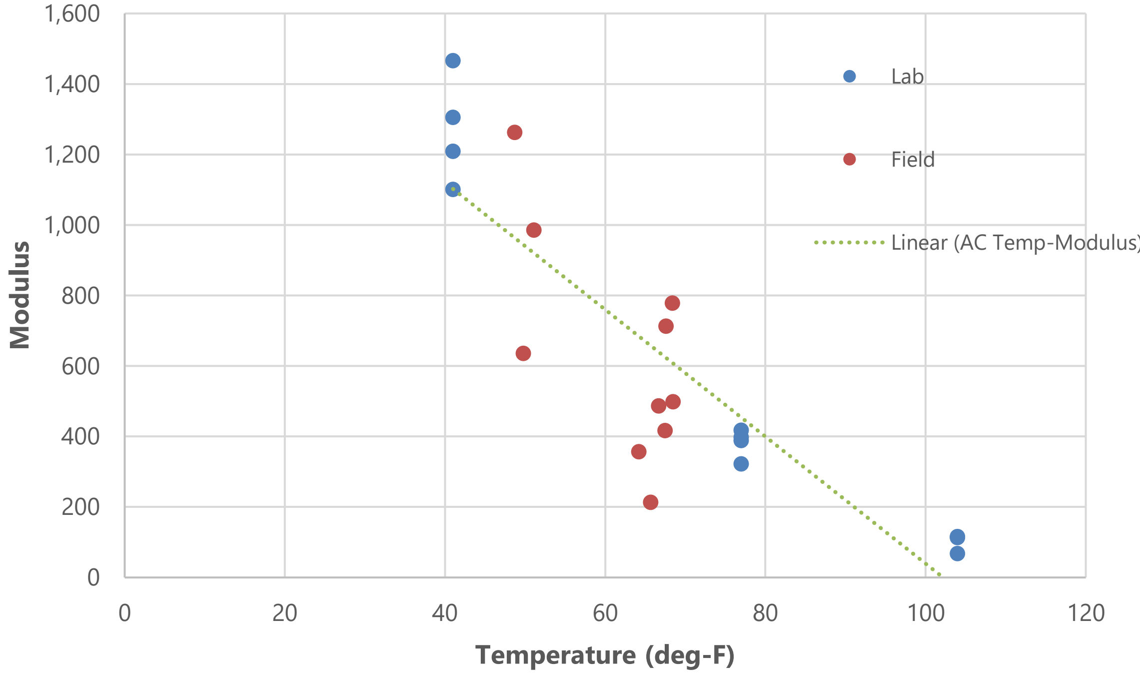


Figure 6. Field- and lab-derived AC resilient modulus values.

Climate History

The time history for average annual precipitation (from MERRA) since 1980 is shown in Figure 7. While the average annual precipitation observed over the analysis period fluctuated on a year-to-year basis, the average yearly precipitation observed increased over time. As shown in the figure, between 1988, when the test section was incorporated into the LTPP program, and 2019, the average annual precipitation at the test section increased from 33 inches to 51 inches. Additionally, notable spikes in precipitation were observed in 2003, 2004, 2011, and 2018, when 59, 61, 66, and 69 inches of precipitation were reported, respectively. These notable spikes are likely related to extreme weather events reported in central Pennsylvania including Tropical Storm Henri, Hurricane Isabel, and the President's Day Storm II in 2003, Hurricane Ivan in 2004, and Hurricane Irene, Hurricane Agnes, and Tropical Storm Lee in 2011 as well as abnormally wet spring and summer seasons in 2018.

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 516 deg F deg days (1998) to 1,548 deg F deg days (1994) during the analysis period. As shown in the figure, the overall trend of the freezing index is decreasing overtime, indicating the climate is getting warmer. This trend could have implications on the overall performance of the pavement overtime.

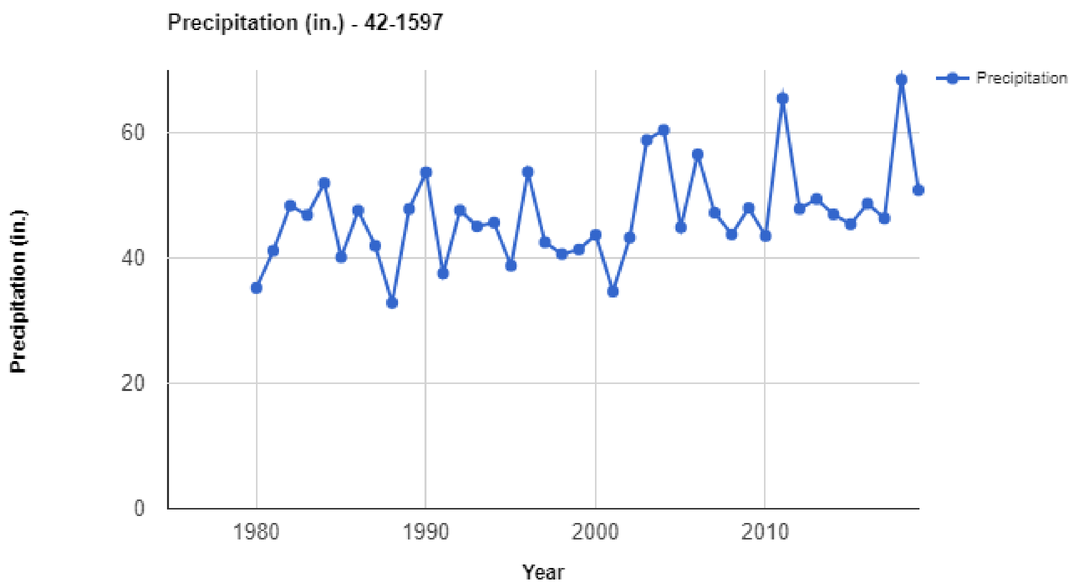


Figure 7. Average yearly precipitation over time.

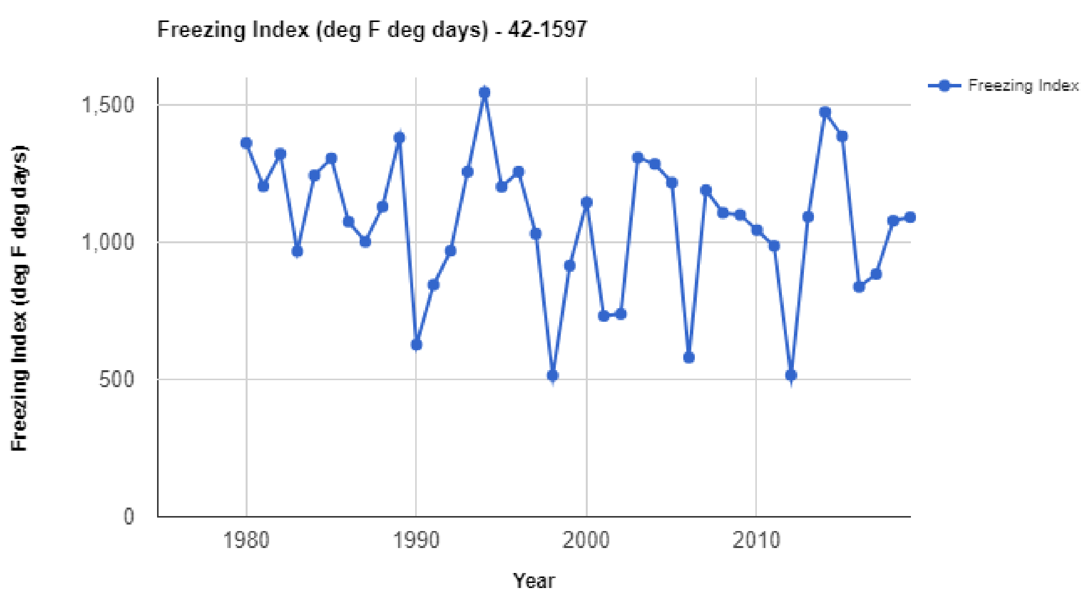


Figure 8. Average annual freezing index over time.

Truck Volume History

Figure 9 shows the annual average daily truck traffic (AADTT) data in the LTPP test lane by year. The annual truck traffic counts increase from 73 in 1980 to 158 in 2017. The average number of ESALs reported on the section also increased over time as depicted in Figure 10. The number of ESALS increased from 5,280 in 1980 to 29,364 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 (1980-1989), state provided AADTT values (1990, 1992, and 1996), monitored values (1991, 1993-1995, 1997, and 2000-2003), monitored values calculated from class data (1998-1999 and 2004-2011), and values calculated using a compound growth function (2012-2017) were used to report traffic along this test section. Overall, the test section showed relatively low truck volumes and loading throughout time.

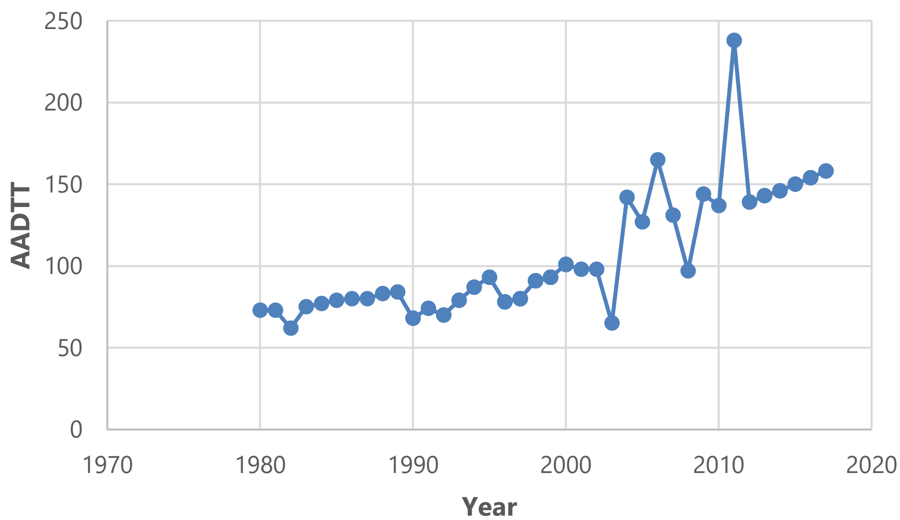


Figure 9. Average annual daily truck traffic (AADTT) history.

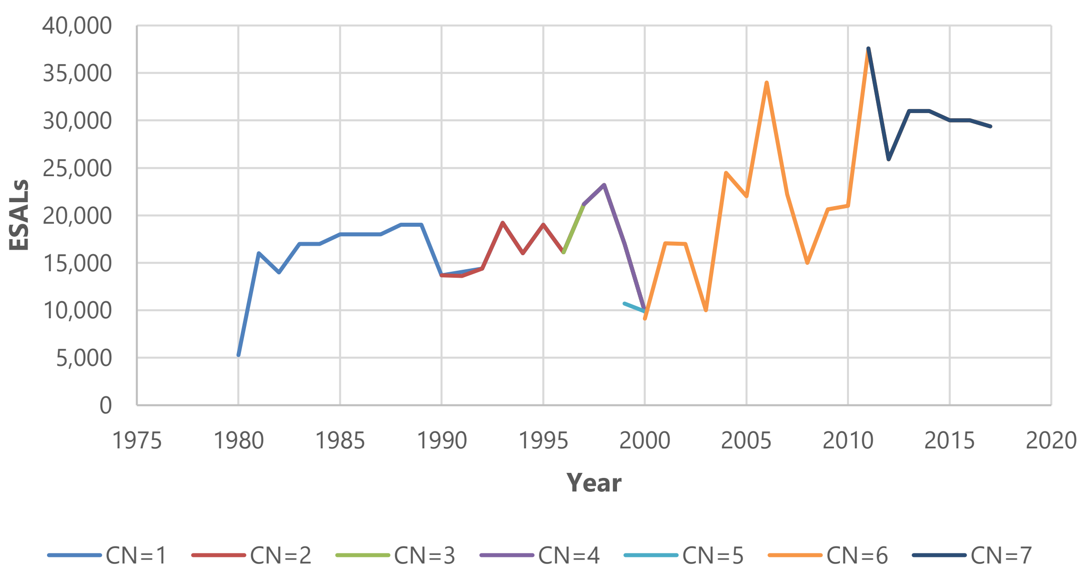


Figure 10. Estimated annual ESALs for vehicle classes 4-13 over time.

Pavement Distress History

The following summarizes the distresses observed on the test section between 1988 and closeout monitoring in September 2020. 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 1992 and 2020. While the graph obtained from InfoPave™ is labeled fatigue cracking, which implies a mechanism, the distress values reported includes both fatigue cracking (inside the wheel path) and alligator cracking (outside the wheel path).

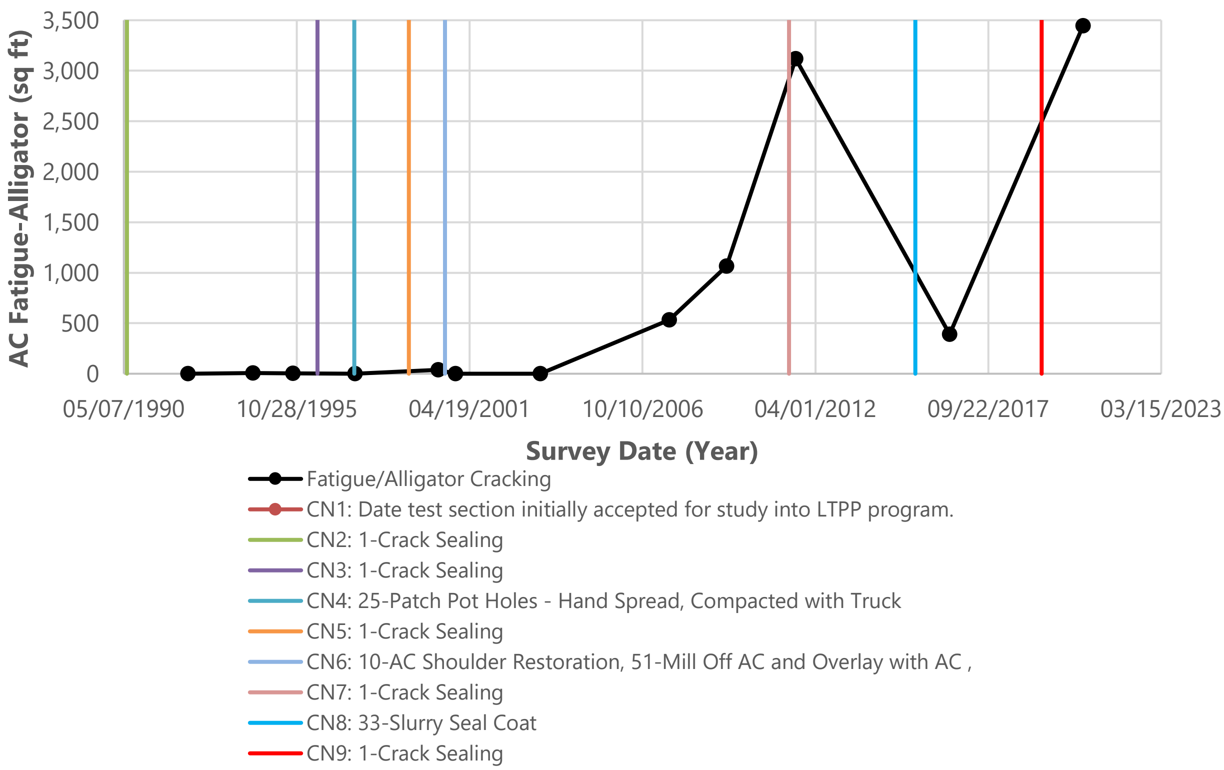


Figure 11. Time history of the length of fatigue cracking.

Fatigue/alligator cracking was first reported during the manual distress survey in June 1994, 14 years after the construction of the roadway, when 5.4 ft^2 was observed. However, fatigue/alligator cracking on the test section remained minimal between 1994 and April 2000, when 38.8 ft^2 of fatigue/alligator cracking was observed. Following the mill and overlay event in 2000, fatigue/alligator cracking is not reported again until the manual distress survey conducted in 2007, 7 years after the overlay, when 532.7 ft^2 of fatigue/alligator cracking was observed on the section. Between 2007 and 2011, the amount of fatigue/alligator cracking observed continued to increase at a rate of 647 ft^2/year, reaching 3,118.3 ft^2 by 2011. The test section received a slurry seal in 2015, which appears to have hid the amount of fatigue/alligator cracking observed. However, by 2020, the last year a manual distress survey was conducted, the cracking observed prior to the slurry seal reappeared, with 3,445.5 ft^2 reported.

As depicted in the figure, fatigue/alligator cracking was not significant on the test section until after the mill and overlay event. The increase in the fatigue/alligator cracking after 2000 is likely related to a combination of environmental and structural factors. One possible reason for the increase in fatigue/alligator cracking following the overlay is increased levels of precipitation reported during this period. As water infiltrates the pavement, unbound granular layers tend to weaken (especially when reaching saturation conditions), which can contribute to the observed fatigue cracking. Another factor that could have played a role in the increase in fatigue/alligator cracking reported following the overlay is a combination of loading and the freeze-thaw periods the test section undergoes. During the thaw period, the pavement's base layer tends to weaken which may have magnified the effect of any loading experienced on this test section during the thaw period. Lastly, as the AC overlay at this test section consisted of three separate AC mixes, how well the individual AC overlay layers bonded may also have played a role in the fatigue/alligator cracking observed. If the bonds between these three AC layers were weak or if they de-bonded, this could have ultimately led to a weakened pavement structure more apt to develop fatigue/alligator cracking. As the mechanism of the developed cracking (top-down versus bottom-up) is important for understanding the cause(s) of fatigue/alligator cracking, coring is recommended as a follow-up activity at this test section.

Longitudinal Cracking

Non-wheel path (NWP) longitudinal cracking, depicted in **Error! Reference source not found.**, was reported during the first manual distress survey in May 1992, 12 years after the construction of the roadway, when 73 feet of cracking was observed. Between May 1992 and April 2000, the NWP longitudinal cracking increased at a rate of 50 feet/year, reaching 475 feet by 2000. Following the mill and overlay in July 2000, NWP longitudinal cracking was not observed again until the July 2003 manual distress survey, three years after the mill and overlay event, when 42 feet of cracking was reported. The NWP longitudinal cracking continued to increase between July 2003 and June 2020, at a rate of 72 feet/year, reaching 1,266 feet by 2020. NWP longitudinal cracking was predominantly located on the centerline (between the section lane and the lane in the opposite direction) prior to the mill and overlay event in 2000 and on both the centerline and edge of the lane following the 2000 mill and overlay. Given the location of the cracking, it is hypothesized that the propagation of the NWP longitudinal cracking is construction-related; the cracking is likely a result of the construction joint locations. Following the mill and overlay event, the cracking developed on the centerline of the roadway appears to have been reflected to the overlay layers.

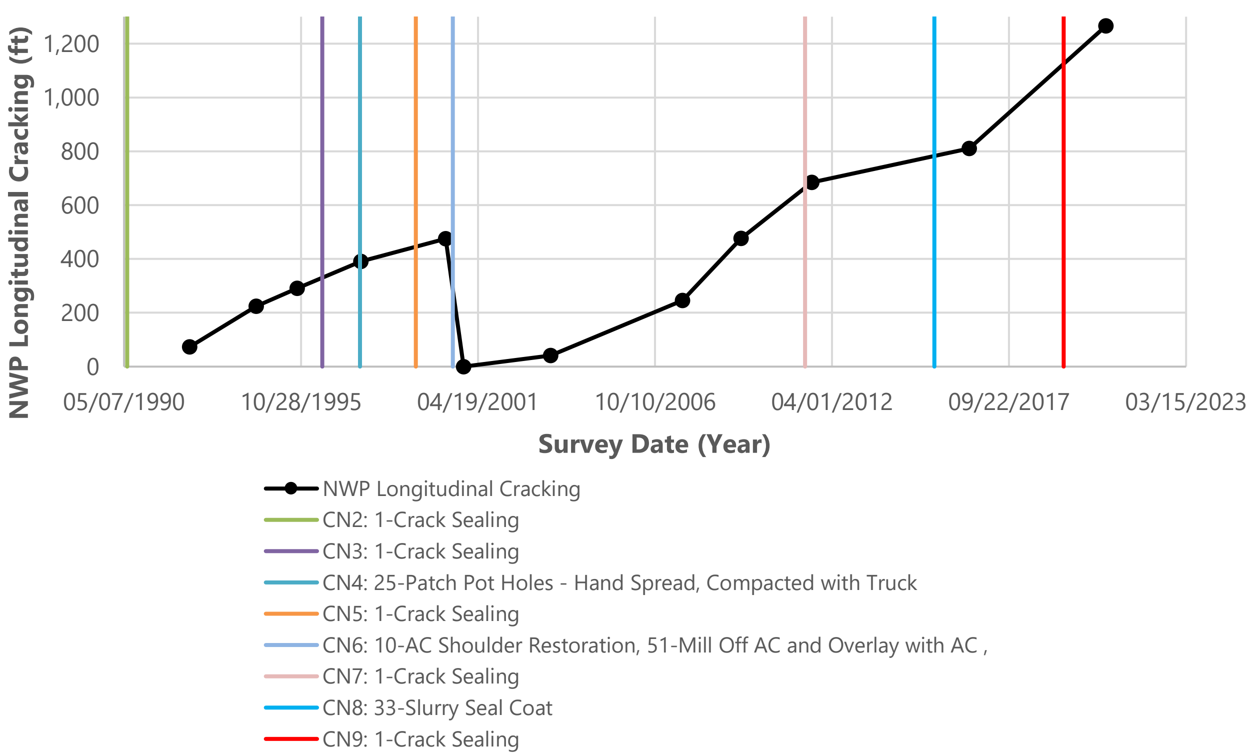


Figure 12. Time history of the length of NWP longitudinal cracks.

Minimal wheel path (WP) longitudinal cracking was observed on the test section as depicted in Figure 13. Prior to the mill and overlay in 2000, the WP longitudinal cracking observed increased at a rate of 2 feet/year, reaching 16 feet by April 2000. Following the mill and overlay in July 2000, WP longitudinal was reported between August 2007 and August 2011, increasing at a rate of 6 feet/year, reaching 33 feet by 2011. The test section received a slurry seal in 2015 that hid the amount of WP longitudinal cracking observed previously. During the 2020 manual distress survey, while most cracking types observed prior to the mill and overlay reappeared, WP longitudinal cracking was reduced as fatigue/alligator cracking inside the wheelpath increased (which likely captured the previously reported WP longitudinal cracking). Like fatigue/alligator cracking, the increase in the WP longitudinal cracking between 2000 and 2011 was likely related to a combination of the increased levels of precipitation, freeze/thaw, the ESALs reported during this period, and the de-bonding of the three mix types used in the AC overlay.

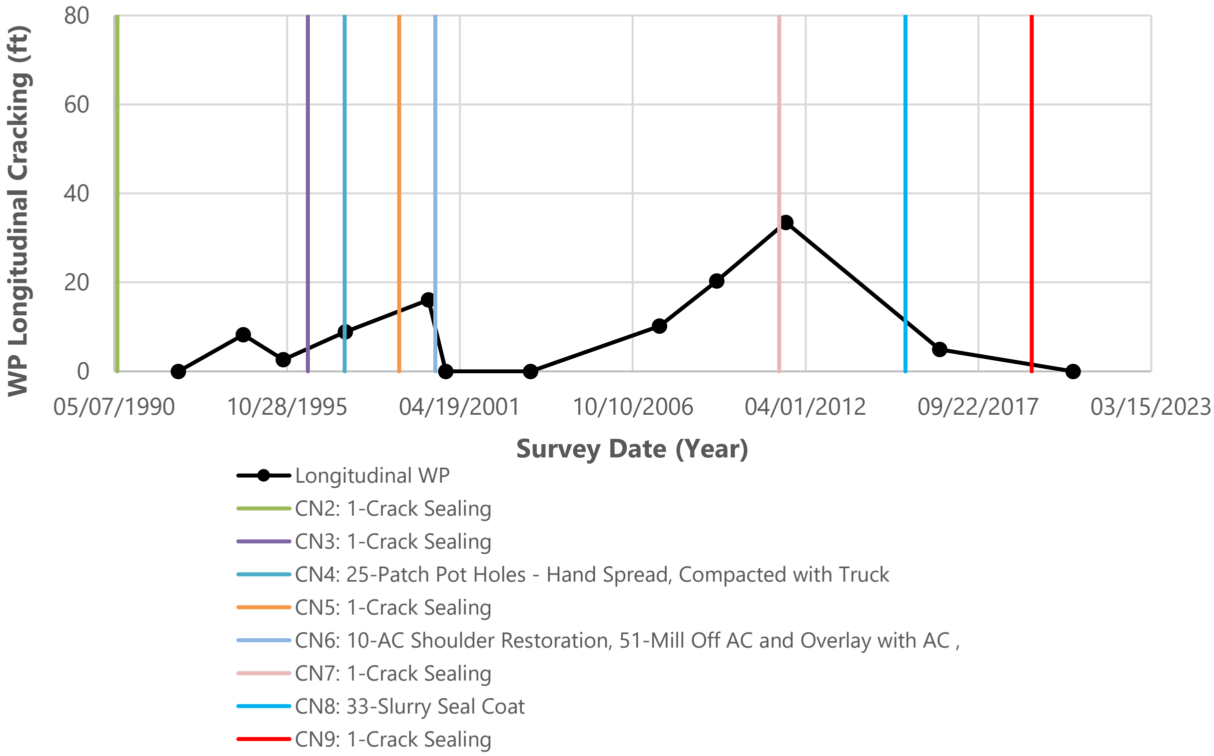


Figure 13. Time history of the length of WP longitudinal cracks.

Transverse Cracking

Data on transverse cracking was collected between 1992 and 2020, as shown in **Error! Reference source not found.** and **Error! Reference source not found.**. Transverse cracking was first reported during the manual distress survey in May 1992, 12 years after the construction of the roadway, when 86 feet of transverse cracking (9 cracks) was observed. The transverse cracking continued to increase between 1992 and April 2000, at a rate of 50 feet/year, reaching 487 feet (78 cracks) by 2000. Prior to the mill and overlay event, transverse cracking was a mix of low, medium, and high severity cracking. Following the mill and overlay event in 2000, transverse cracking was not observed again until the August 2007 manual distress survey, seven years after the construction event, when 30 feet (8 cracks) of transverse cracking was reported. The transverse cracking continued to increase between 2007 and June 2016 at a rate 45 feet/year of reaching 441 feet (188 cracks) by 2016, despite the application of a slurry seal in 2015. During the last distress survey, conducted in 2020, the observed transverse cracking decreased to 195 feet (87 cracks) due to the initiation of block cracking on the test section in 2020 (1,177 ft^2 of block cracking observed in 2020). Transverse cracking reported after the mill and overlay event was strictly low and medium severity.

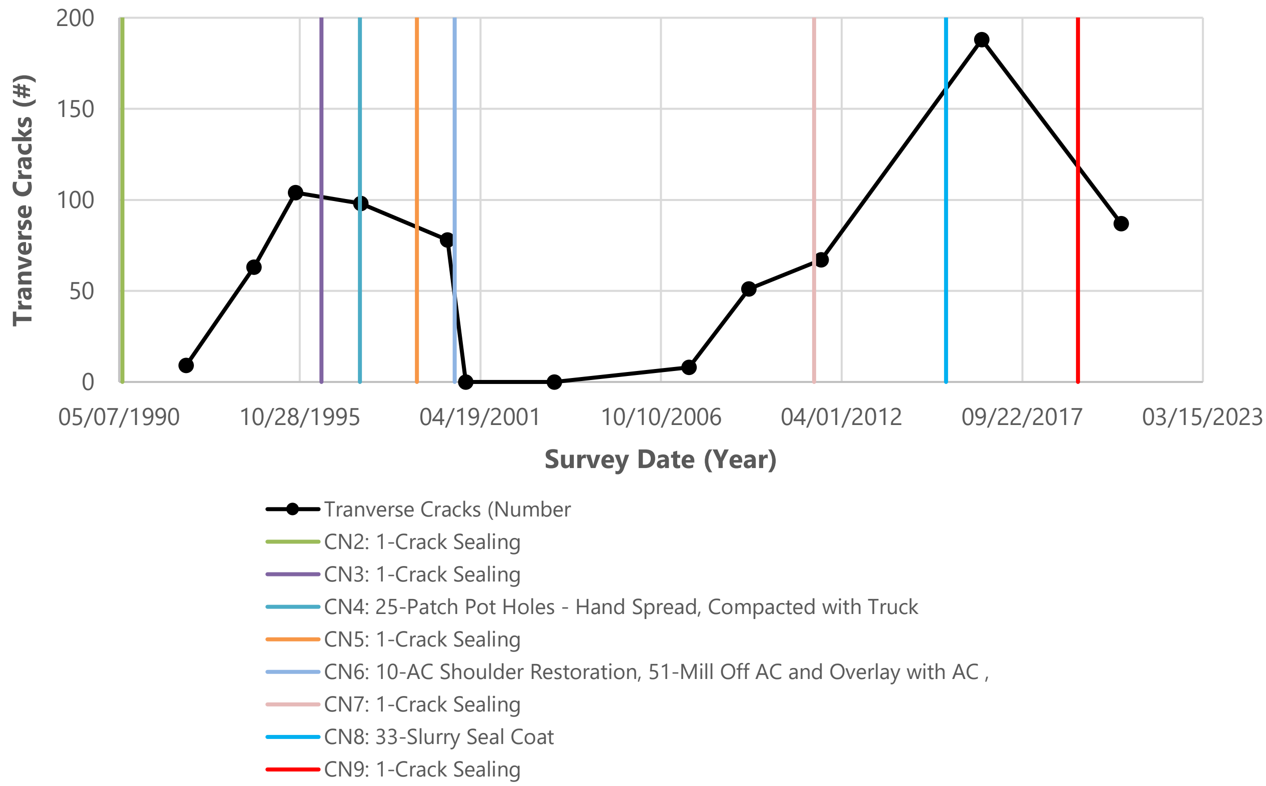


Figure 14. Time history of the number of transverse cracks.

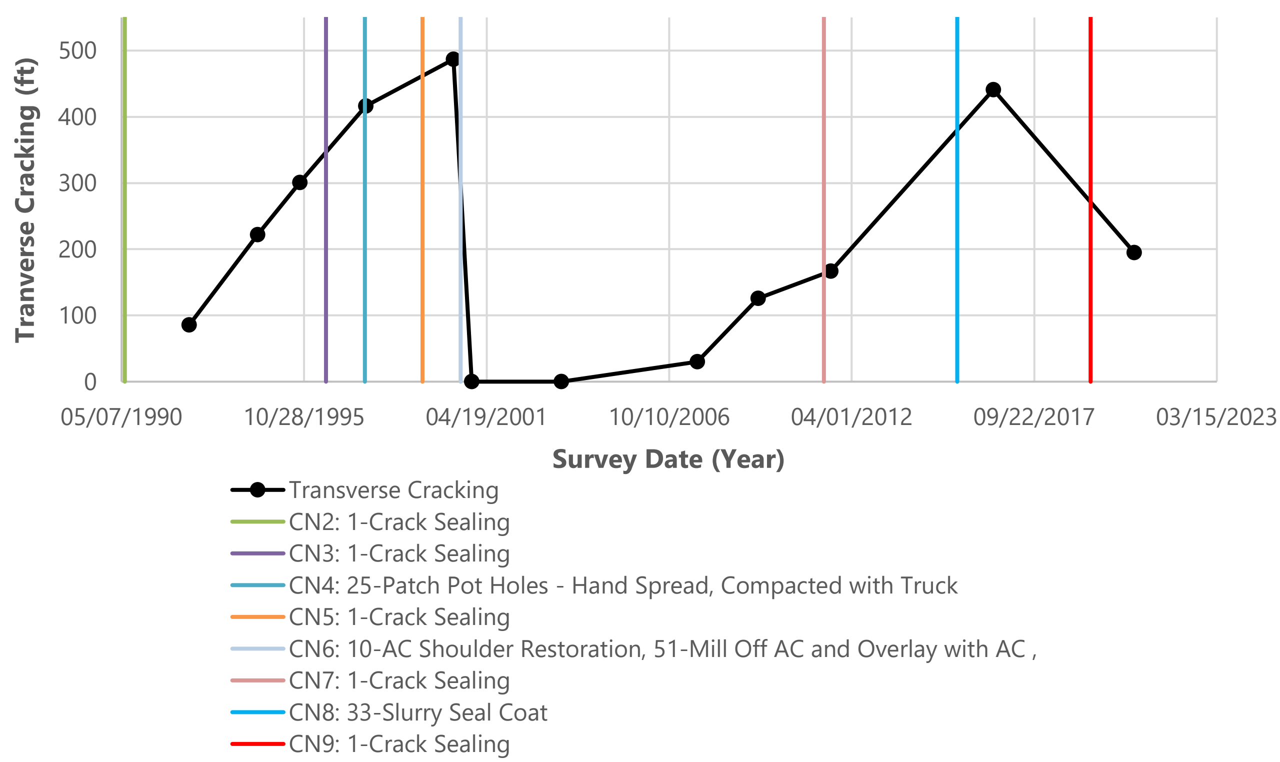


Figure 15. Time history of the length of transverse cracking.

The rate of propagation before and after the mill and overlay was similar; however, the location of the transverse cracking before and after the event was not consistent. While it was hypothesized there was a shift in stationing following the application of the mill and overlay, due to the construction event, a comparison of distress surveys conducted before and after the mill and overlay event did not show consistent shifts in transverse crack locations. As shown in Figure 16, prior to and following the overlay, there were some cracks that were likely reflected through the overlay (e.g. at 57 meters), but there were also transverse cracks that were located in different locations prior to and following the mill and overlay. In addition, after the mill and overlay event, there appeared to be more partial width cracks perpendicular to NWP longitudinal cracking at the centerline and edge of the lane.

Both prior to and following the 2000 mill and overlay, it is hypothesized that the propagation of transverse cracking was related to the freeze-thaw periods (evidenced by the high freezing indices) of the pavement section over time. Additionally, the presence of rumble strips along the centerline of the pavement section after the 2003 manual distress survey may have also contributed to the transverse cracking reported after 2003.

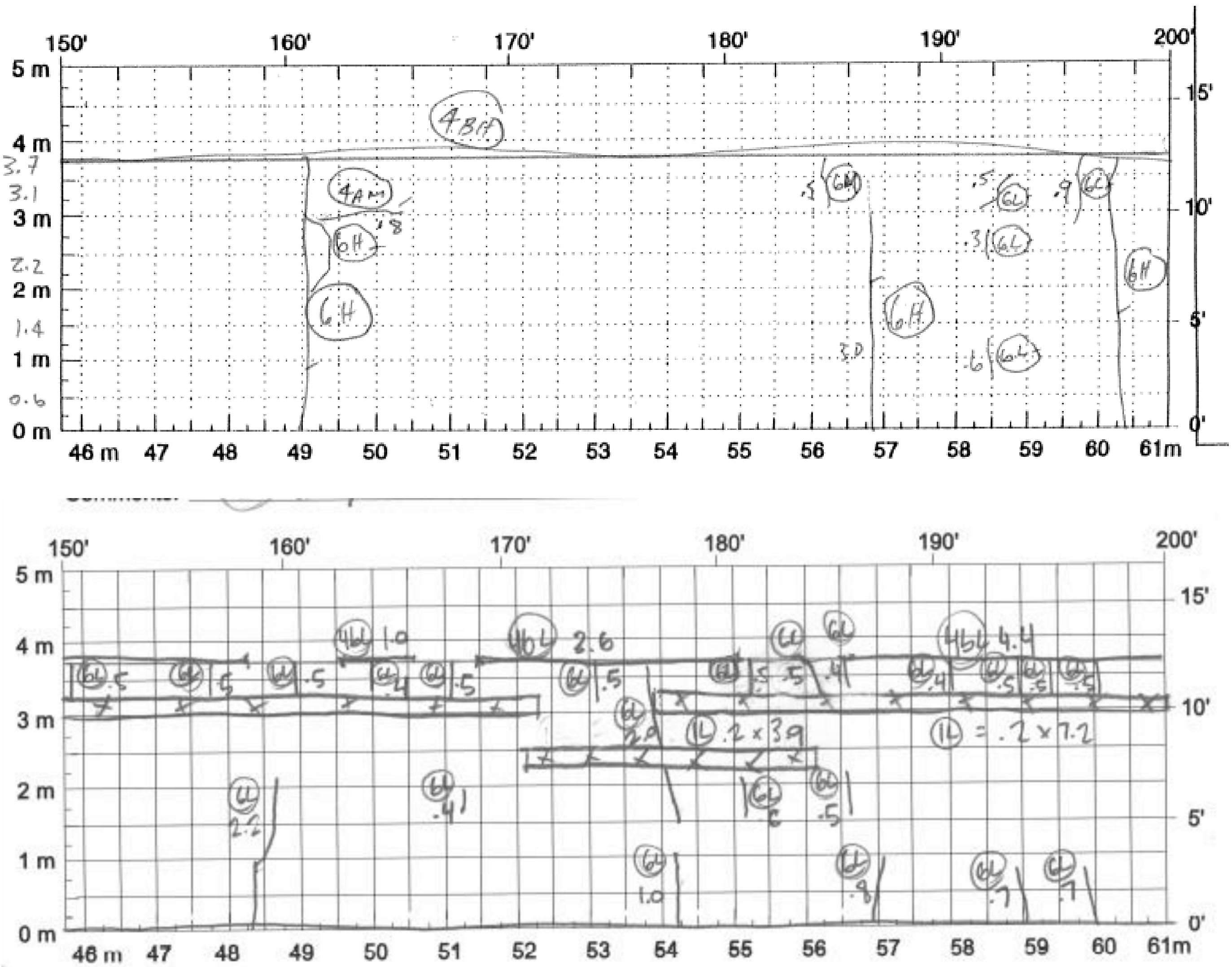


Figure 16. Manual distress surveys from April 2000 (top) and June 2016 (bottom) showing transverse cracking in both similar and different locations before and after the overlay.

IRI

The average IRI measurements for the section over time are shown in Figure 17. During the first performance period of the test section, from its incorporation into the LTPP program to the mill and overlay event in 2000, the IRI on the test section increased substantially over time. The IRI on the section prior to the mill and overlay event in 2000 averaged 256 in/mi, which means the performance of the pavement was classified as “Poor” based on FHWA performance definitions. During the second performance period, after the 2000 mill and overlay event, the IRI of the test section dropped to 33 in/mile in November 2000 and remained low, reaching 47 in/mi by 2020. The average IRI during this performance period is classified as “Good” based on FHWA performance definitions.

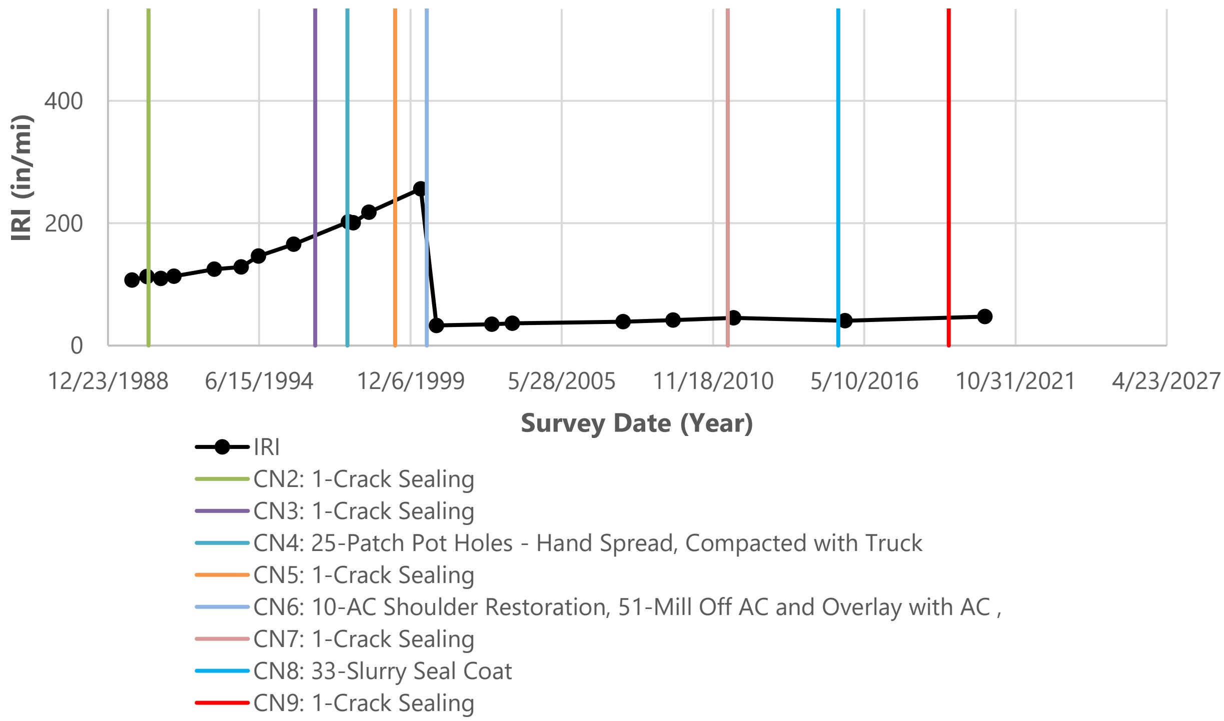


Figure 17. Time history plot of pavement roughness.

The IRI reported during the two 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 equal or higher quantities following the mill and overlay in 2000 than prior to the overlay event despite the significantly lower average IRI reported post-overlay. This may be related to the severity of the cracking observed on the section following the mill and overlay—predominantly low and medium—which plays less of a role in the roughness of the test section. The higher IRI values reported prior to the mill and overlay may also be related to the initial smoothness of the road when it was constructed in 1980 (at which point the specification for pavement smoothness may have differed from current specifications); the thick overlay placed in 2000 may have helped correct this smoothness. The smoother a pavement is during the construction of the pavement, the slower the expected deterioration rate. Therefore, if the pavement’s starting IRI was higher, the IRI pre-overlay may have deteriorated at a faster rate.[2] Additionally, as will be described in the section to follow,

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[2] R.W. Perera, S.D. Kohn, *LTPP Data Analysis: Factors Affecting Pavement Smoothness. National Cooperative Highway Research Program (NCHRP),* Washington, D.C., United States, Project Report 20-50[8/13], 2001.

the increase in rutting during the first performance period may have also affected the IRI reported on the section prior to the mill and overlay event.

Rutting

The average rut depths observed for the section between 1989 and 2016 are shown in Figure 18. While transverse profiles were collected as a part of the closeout monitoring conducted in 2020, they were not readily available during the preparation of this desktop study memorandum and therefore, will not be considered until the follow-up investigation related to this test section. The rutting on the section prior to the mill and overlay in 2000 increased from 0.16 in in 1989 to 0.28 in in 2000. Following the mill and overlay in 2000, the average rut depth dropped to 0.04 in. The average rut depth began to slightly increase following the overlay, at a rate of 0.01 in/year between 2000 and 2016. It is hypothesized the majority of rutting observed prior to 2000 occurred within the top 1.5-inch AC layer (which was entirely removed during the mill and overlay event in 2000, leading to lower rutting values following the overlay).

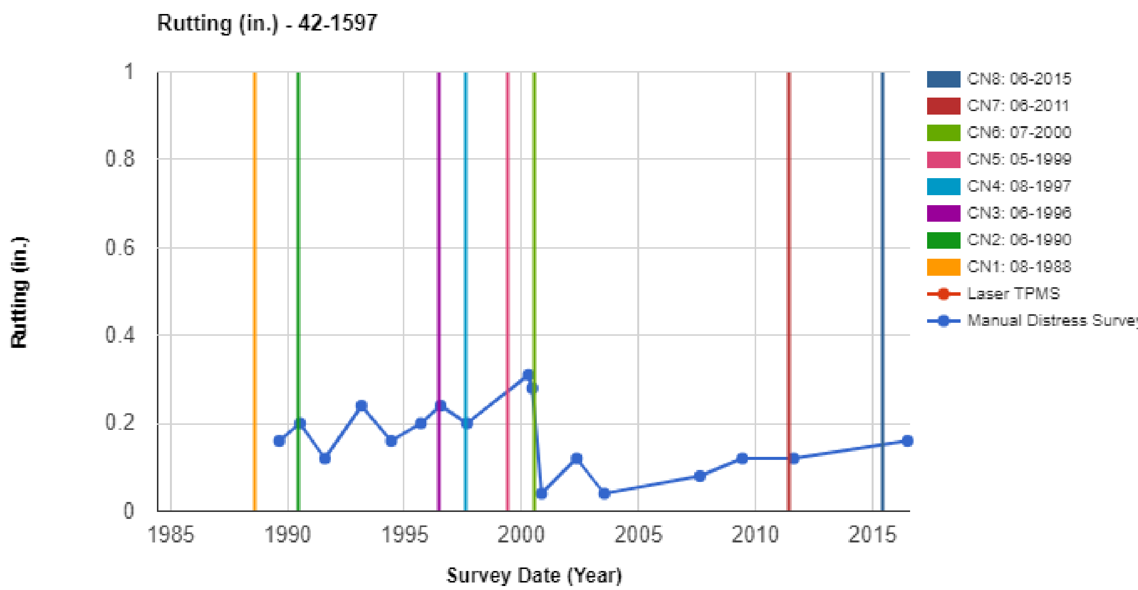


Figure 18. Time history plot of average rut depth.

In addition to the average rut depth observed over time, the change in the transverse profile of the test section was also investigated. Using the transverse profiles of the test section at multiple locations, an analysis of the predominant layer in which plastic deformation occurs was assessed using the method developed in NCHRP 01-34a.[3] The NCHRP method, which was derived using finite element analyses of rutting mechanisms in the HMA surface, base, and subgrade, is focused on the transverse profile characteristics indicative of permanent deformation such as densification, shear failure, or shear flow.

The methodology consists of two key steps: calculation of distortion parameters and the use of criteria to classify the lowest layer in the pavement structure contributing to the ruts. Distortion parameters include the maximum rut depth (D), positive area, and negative area of a transverse profile. For each profile, the wire method is used to assess the maximum rut depth, which is the greatest perpendicular distance

[3] White, T., J. Haddock, A.J.T. Hand, & H. Fang. NCHRP 468: *Contributions of Pavement Structural Layers to Rutting of Hot Mix Asphalt Pavements*. National Cooperative Highway Program, Washington D.C., 2002.

measured from the pavement surface to the wire reference line as depicted in Figure 19. Similarly, the positive area (A_P) and negative area (A_N) are the sum of the areas above and below the transverse profile reference line, respectively. Using these parameters, the ratio of positive area to negative area (R), total area (A_T), and the theoretical total areas for the HMA, base, and subgrade failure (C_1,C_2, and C_3, respectively) are calculated and used to assess the failed layer. The assessment of the parameters used to determine the lowest layer contributing to the pavement's surface deformation is described in Figure 20.

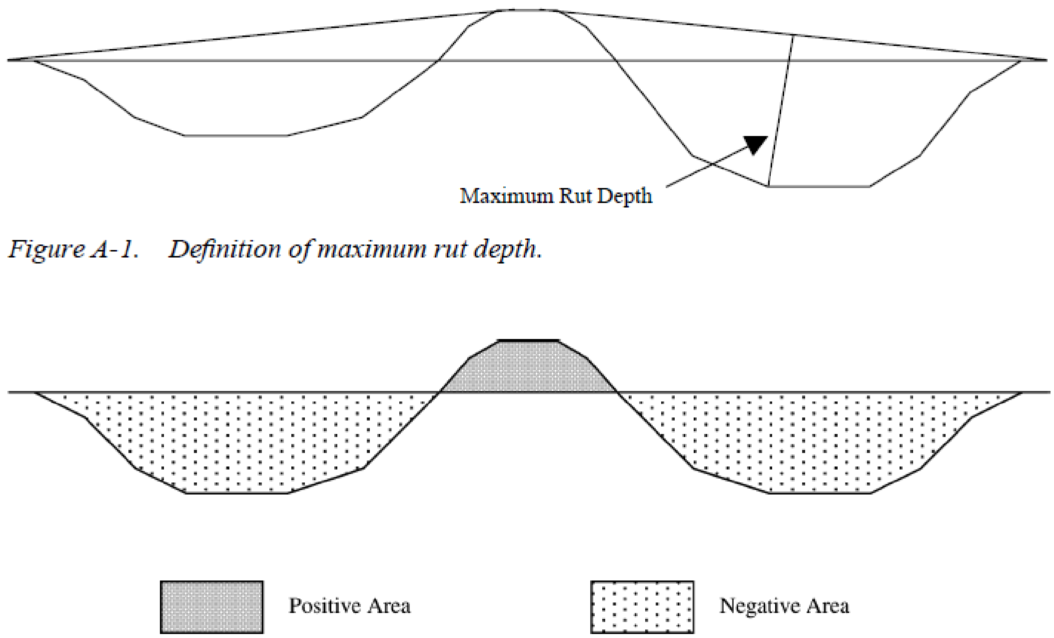


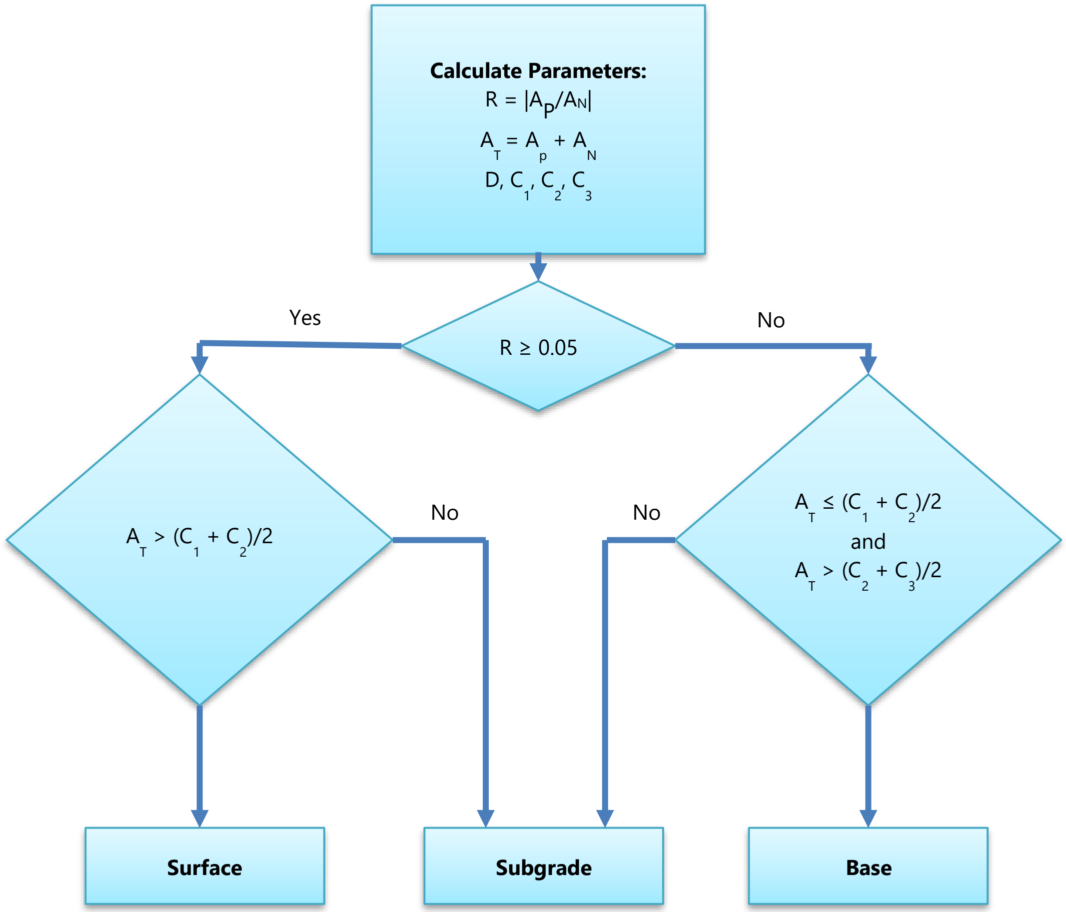
Figure 19. Transverse profile maximum rut depth and positive and negative areas (White et al., 2002)

Based on the analysis conducted for each of the transverse profiles of the test section (between 10 and 11 profiles spaced at 50 ft) for the 17 collection dates between August 1989 and June 2016, the predominant lowest layer contributing to rutting was calculated for each date of collection at multiple locations along the section. For all years, the predominant layer contributing to rutting was the surface layer (except one profile in June 2009 which was classified as a base failure). This helps to support the claim that the rutting observed prior to 2000 mostly occurred within the top 1.5-inch AC layer (which was effectively removed during the mill and overlay event in 2000), leading to lower rutting values following the overlay.

SUMMARY OF FINDINGS

LTPP test section 42_1597 is located on State Route 49, eastbound, in Tioga County, Pennsylvania. State Route 49 is a rural minor arterial with one lane in the direction of traffic. The test section was constructed in 1980 and was accepted into the LTPP Program as part of the GPS-1 experiment in August 1988. Although the test section was in active status at the time it was recommended for a desktop study, closeout monitoring occurred on the test section in September 2020. Therefore, the section is anticipated to officially go out of study soon. The pavement structure at the time of its incorporation into the LTPP program consisted of 6.5 inches of asphalt concrete (split between two layers) and 16.8 inches of unbound granular base over a fine-grained subgrade soil. The next major construction event occurred in July 2000, when the test section received a shoulder restoration and a mill and a 6.6-inch AC overlay (using three different mix types), moving the test section to the GPS-6S: AC Overlay of Milled AC Pavement Using

Conventional or Modified Asphalt experiment. An additional construction event in June 2015 (CN=8), a slurry seal, also resulted in 0.3-inch increase to the pavement structure. Other minor construction events that occurred on the test section included crack sealing in June 1990, June 1996, May 1999, June 2011, and June 2019 and patching in August 1997.



D= Maximum rut depth
A_p= Positive area (area above pavement surface line of a transverse profile)
A_n= Negative area (area below pavement surface line of a transverse profile)
C_1= (-858.21) D + 667.58, theoretical total area for HMA failure
C_2= (-1509) D -287.78, theoretical average total for base/subbase failure
C_3= (-2120.1) D – 407.95, theoretical average for subgrade failure

Figure 20. Failure layer determination using methodology by White et al. (2002)

The memorandum was focused identifying the differences in the overall performance of the test section before and after the mill and overlay event in 2000. Specifically, it focused on:

1. **Investigating the cause(s) of the fatigue cracking following the mill and overlay event in 2000**. Fatigue/alligator cracking on the test section was not significant until after the mill and overlay in 2000. The increase in the fatigue/alligator cracking after the mill and overlay is likely related to a combination of environmental and structural factors. Specifically, it is hypothesized

that an increase in precipitation following the overlay, increased loading on the test section over time during thaw periods (when the base layer is weakened), and the bond (or lack thereof) between the AC mixes used for the overlay may have played a role in the increase of fatigue/alligator cracking observed.

2. **Identifying whether any of the cracking observed prior to the mill and overlay (specifically longitudinal and transverse cracking) was reflected following the mill and overlay.** NWP longitudinal cracking was predominantly located on the centerline (between the section lane and the lane in the opposite direction) prior to the mill and overlay event in 2000 and on both the centerline and edge of the lane following the 2000 mill and overlay. Given the location of the cracking, it is hypothesized that the propagation of the NWP longitudinal cracking is construction related. Following the mill and overlay event, the cracking developed on the centerline of the roadway appears to have been reflected to the overlay layers. For transverse cracking, the rate of propagation before and after the mill and overlay was similar; however, the location of the transverse cracking before and after the event was not consistent. Prior to and following the overlay, there were some cracks that were likely reflected through the overlay, but there were also transverse cracks that were located in different locations prior to and following the mill and overlay. Additionally, the presence of rumble strips along the centerline of the pavement section after the 2003 manual distress survey may have also contributed to the transverse cracking reported after 2003.
3. **Further investigate the cause(s) of the high IRI and rutting values (256 in/mi and 0.31 in in 2000) prior to the mill and overlay event in 2000**. The IRI reported on the section did not seem to be correlated to the cracking reported throughout time; the predominant cracking types that purportedly affect the overall IRI of the test section—fatigue/alligator and transverse cracking—were present in equal or higher quantities following the mill and overlay in 2000 than prior to the overlay event despite the lower average IRI reported during this period. This was likely the result of the severity of the cracking reported, the initial smoothness of the roadway during construction and its effect on the deterioration of smoothness prior to the overlay, and the rutting reported prior to the mill and overlay event. The rutting reported on the section followed a similar trend to the IRI— the rutting on the section prior to the mill and overlay in 2000 increased from 0.16 in in 1989 to 0.28 in in 2000, and following the mill and overlay in 2000, the average rut depth dropped to 0.04 in and increased at a rate of 0.01 in/year. It is hypothesized that the majority of the rutting observed prior to 2000 occurred within the top 1.5-inch AC layer (which was removed during the mill and overlay event in 2000, leading to lower rutting values following the overlay) which was supported by the analysis conducted for each of the transverse profiles using the NCHRP 01-34a method.
4. **Determine the differences in the initiation and propagation of cracking prior to and following the mill and overlay event**. The initiation and propagation of fatigue/alligator cracking, NWP longitudinal cracking, and transverse cracking was assessed prior to and following the mill and overlay event in 2000. As shown in Table 5, for both fatigue/alligator cracking and NWP longitudinal cracking, cracking both initiated and propagated more quickly following the mill and overlay than prior to the mill and overlay. Transverse cracking initiated more quickly after the overlay but propagated at a similar rate prior to and following the mill and overlay. Ultimately, following the mill and overlay, transverse cracking decreased as block cracking initiated on the test section. These trends are supported by the environmental and structural conditions of the test section; a combination of an increase in precipitation, an increase in ESALs, freeze-thaw, and the strength of the bond between the AC layers in the overlay are hypothesized to have affected the performance of the pavement following the mill and overlay event.

Table 5. Initiation and propagation of cracking prior to and following 2000 mill and overlay

Cracking Type	Prior to Mill and Overlay		Following Mill and Overlay	
	Initiation (Years since construction)	Propagation	Initiation (Years since mill and overlay)	Propagation
Fatigue/Alligator	14	6 ft^2/year	7	647 ft^2/year
NWP Longitudinal	12	50 ft/year	3	72 ft/year
Transverse	12	50 ft/year	7	45 ft/year

FORENSIC EVALUATION RECOMMENDATIONS

Although the test section was considered active at the time it was recommended for a desktop study, closeout monitoring occurred on the test section in September 2020. Therefore, the section is anticipated to officially go out of study soon. Because of this, the following activities are recommended:

1. Coring to enable the following activities:
    a. Confirm that layer thicknesses match those reported when the test section was incorporated into the LTPP program.
    b. Investigate whether some of the NWP longitudinal and transverse cracking at the test section is reflection cracking.
    c. Identify issues with bonding between the AC lifts via visual inspection.
    d. Identify the rutting failure layer via visual inspection.
2. Investigation of the layer thicknesses used for the moduli backcalculation analyses, as the reported thicknesses used in the analysis differed from the reported layer thicknesses.
3. Determine the location of the construction joints of the pavement section to determine whether they are aligned with the observed NWP longitudinal cracking.
4. Analysis of transverse profile information collected during the September 2020 data collection.
5. Investigate the evolution of smoothness specifications within PennDOT, particularly at the time the test section was constructed and at the time the test section was rehabilitated, to better understand the trend in IRI over time.

ADDENDUM TO MEMORANDUM: FOLLOW-UP INVESTIGATION

Based on the findings and recommendations of the desktop memorandum, a follow-up investigation was conducted on Pennsylvania GPS-6S test section 42_1597 to better understand the performance of the test section over time. Specifically, the follow-up study assessed the performance of the test section with regards to fatigue/alligator cracking, NWP longitudinal cracking, and transverse profile measurements and aimed to explain the difference in the roadway smoothness prior to and following the overlay. The follow-up activities included:

- ***Manual distress survey and profile measurements***: When coordinating with the LTPP Data Collection Contractor (DCC) to perform field work, the DCC shared that the monitoring conducted in September 2020 was the close-out testing for this project. Longitudinal profile measurements were collected during the same date using the LTPP High Speed Survey vehicle and following standard LTPP profiling protocols. Transverse profile measurements were also collected. Most of these data were part of the desktop study, so the rut depth computations are the only item included as part of this addendum.
- ***Interview with PennDOT personnel***: On April 29, 2021, a meeting with PennDOT personnel was held virtually. The intent of the meeting was to gain a better insight on potential causes of the observed cracking and overall condition on this test section. The meeting was well attended and informative.

Manual Distress Survey and Profile Measurements

As noted above, both the 2020 distress and roughness data were integrated into the desktop study. While the transverse profiles were collected as part of the September 2020 closeout monitoring, the computed rutting parameters was not available at the time of the original memorandum, and hence, an update to the transverse profile analysis has been included in the follow-up investigation. Figure 21 is an updated graph showing the average rut depth which includes the September 2020 measurements. The average rutting reported was 0.24 inches, which follows the same trend as previously shown – i.e., a rutting rate of 0.01 in/year between 2000 to 2020.

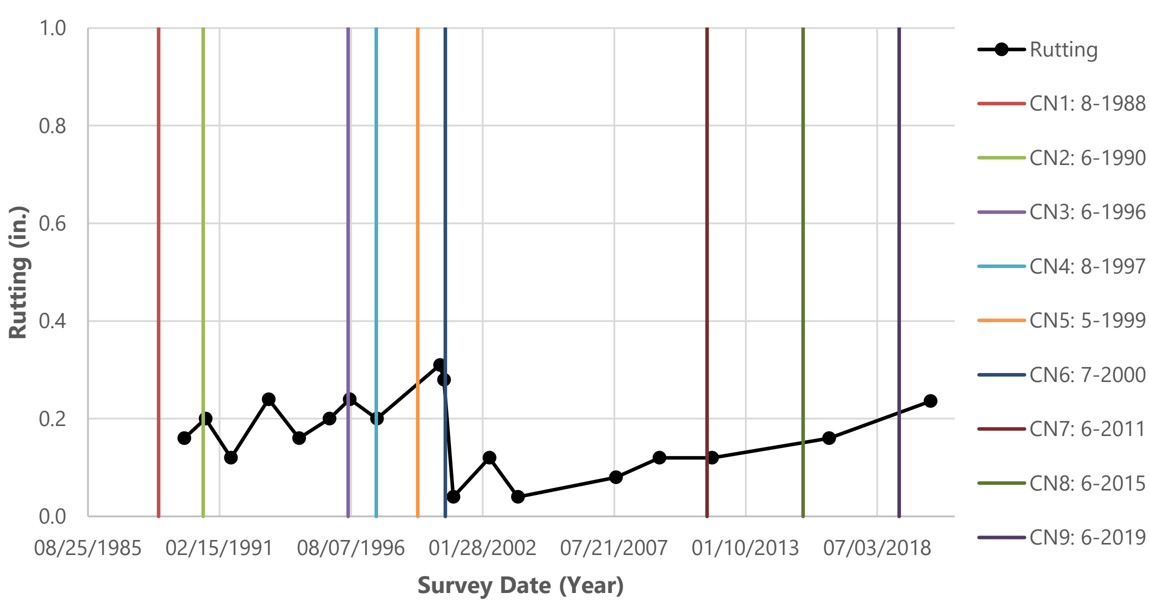


Figure 21. Time history plot of average rut depth.

Interview with PennDOT Personnel

The objective of the interview with PennDOT personnel was to gain a better understanding of the potential causes of the fatigue cracking observed, the longitudinal and transverse cracking trends, and the differentials in roughness progressions pre- and post-2000 overlay and to consider other relevant information that would be provided. A questionnaire was developed and sent to the PennDOT attendees two weeks prior to the meeting to assist in preparing for the meeting.

Some of the items for discussion included:

- Mix types used, whether they used tack coat, records showing original and overlay thickness, pavement density records, cores, or other quality control data.
- Paving practices between 1980 and 2000 in terms of AC mix specifications/material properties, and with regards to IRI smoothness specifications.
- PennDOT's opinion on the observed fatigue/alligator cracking after the overlay was placed. Relate the performance of this test section to other nearby pavements.

The interview was held virtually on April 29, 2021 and had a good participation from PennDOT. A total of 11 attendees were present in the interview which included 7 participants from the pavement design, materials and maintenance groups at PennDOT's District 3, the chief pavement design and LTPP coordinator from PennDOT central office, four participants from the pavement forensic study contract, and one participant from the data collection contractor. The interview lasted for 1 hour and the feedback and contributions from PennDOT attendees were positive and useful. Some of the insights from the interview are summarized in the subsequent paragraphs.

Regarding the mix types for the three-layer overlays, PennDOT personnel provided their historical record for this location as shown in Table 6, and a cross section of the typical pavement structure as shown in Figure 22. The historical record shown in the table is very similar to the layer structure described in the beginning of this study, with the only difference being these historical records do not show the 1.5 in milling prior to the overlay. Based on discussion with the DCC, from an LTPP perspective, there is a high degree of confidence milling was performed as part of the 2000 construction.

Table 6. Pavement historical records

Layer No.	Layer Description	Year	Depth (in.)
01	Surface Treatment Type A Single Application	2015	+0.30
02	Asphalt Mix HMA Wearing, 12.5 mm	2000	+1.50
03	Asphalt Mix HMA Binder, 19 mm	2000	+2.50
04	Asphalt Mix HMA Scratch 9.5 mm	2000	+0.75
05	Bituminous Wearing Course ID-2	1980	+1.50
06	Bituminous Concrete Base Course	1980	+5.00
07	2A Subbase	1980	+17.00

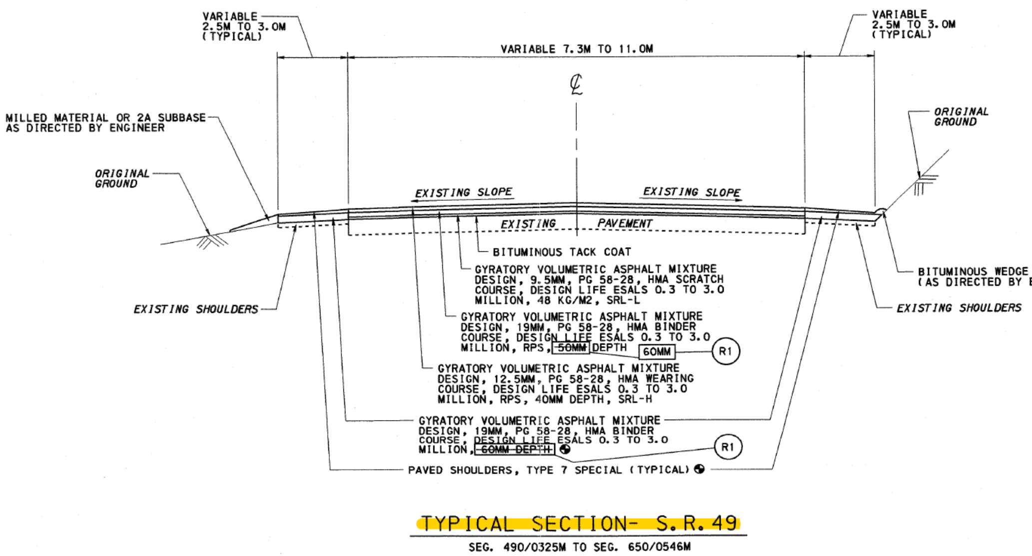


Figure 22. Typical pavement and overlay design in 2000s.

One assumption made in this forensic investigation study was that a potential cause for the increase of fatigue cracking observed after the 2000 overlay was the lack of bonding between the existing pavement and the overlay. The interview with PennDOT further confirmed this as a potential cause. Based on the discussion, PennDOT only applied a tack coat between the existing pavement and the "scratch" mix; no tack coats were applied between the other two layers. The omission of the tack coat application between each layer may explain the early fatigue/alligator cracking. By not creating a good bond between the layers, the pavement structure can become weaker, and with an increase of traffic during the freeze and thaw periods, could expedite the pavement fatigue mechanism. PennDOT has since changed this approach and now applies tack coat between every layer to further ensure the pavement structure will be a monolithic structure and provide more load capacity. It was also noted that the cracking could also be a by-product of the early days of SuperPave mixes. Neither the agencies nor the contractors were experienced with these mixes in 2000 when the overlay was applied and therefore, this unfamiliarity could have played a role in performance.

Another important outcome of the discussion was the observation by PennDOT attendees that the top 12.5 mm of wearing course mix likely had oxidized. Early oxidation of the top layer can contribute to the surface becoming brittle, thereby resulting in potential top-down cracking. To resolve this issue, PennDOT switched from the 12.5 mm mix to a 9.5 mm mix for wearing courses. It was also noted this change would help seal the pavement and keep water from getting further into the pavement structure. This change in wearing course design has seemed to solve issues related to early oxidation.

Another point of discussion was the longitudinal cracking in the middle of the lane. PennDOT noted this was commonly caused by a Blaw-Knox paver dragging material under the gear box. It was not definitive whether the 2000 overlay was constructed using that paver, but that was an issue observed in other projects. Another issue PennDOT frequently had in the early 2000s was longitudinal joints opening. PennDOT used to use notch wedge joints and has since changed the specifications to overband the longitudinal joints (a practice adopted about 5 years ago) and take densities at the joints. With these specifications, PennDOT has minimized the appearance of construction joint longitudinal cracking.

Regarding smoothness specifications, PennDOT does not have smoothness specifications for low volume roads; specifications are required to be met on high volume roads. For low volume roads, the contractor only has to meet a relatively generous straightedge specification. It is hypothesized that the improvement in smoothness following the 2000 overlay is related to the change in specification for the high-volume roadways, which contractors likely adopted for use on all roads.

Finally, the test section was compared to other pavement sections in the area, it terms of its overall performance. PennDOT noted that AC sections in this area of Pennsylvania typically provide a service life of 12 years and can be extended to 20-year cycle if a surface treatment is applied. The test section's performance was slightly worse than typical with more severe cracking than other AC sections in the area.

Conclusions and Recommendations

The purpose of the follow-up investigation of Pennsylvania test section 42_1597 was to pursue additional information on the change in performance of the test section over time. Specifically, the investigation was conducted to gather additional information on the potential cause(s) of the fatigue/alligator cracking reported on the test sections, the cause(s) of NWP longitudinal and transverse cracking, and the trend in rutting observed on the test section over time. To accomplish this, a series of follow-up analyses using data collected on the test section in 2020 and an interview with PennDOT personnel were conducted.

The key findings from the follow-up investigation supported some of the hypotheses presented in the desktop study. With regards to the fatigue/alligator cracking observed on the test section following the 2000 mill and overlay, PennDOT suggested that it may have been a result of the way in which the AC overlay was constructed. During the time of the overlay, there was no tack coat applied between AC layers, which could have made the overlay weaker and caused the propagation of fatigue/alligator cracking on the test section. Furthermore, the oxidation of the 1/2 in wearing course could have also contributed to the increase in cracking observed. PennDOT later changed their specifications to use a 3/8 in wearing course because of this issue. NWP Longitudinal cracking may have been caused by two issues: 1) the paver dragging material under the gear box, and 2) the use of notch wedge joints. In response to these issues, PennDOT now uses overband joints and density specifications at the joints.

Future recommendations (to be implemented after the conclusion of the forensic pooled fund study) include coring before the test section receives its next rehabilitation. This activity will help confirm layer thicknesses, further investigate the NWP longitudinal and transverse cracking to verify if they are reflection cracking, evaluate the bonding between AC layers, and identify the rutting layer(s) via visual inspection.

In closing, the project team wants to recognize the strong participation from Pennsylvania Department of Transportation in this investigation. PennDOT's staff's willingness to cooperate the valuable information provided during the interview improved the findings regarding test section performance over time.