Technical Memorandum

To: Jeff Uhlmeyer

From: Lauren Gardner, Gonzalo Rada, Gary Elkins and Kevin Senn

cc: Mustafa Mohamedali

Date: August 13, 2020 (original)

Re. Forensic Desktop Study Report: Maine LTPP Test Section 23_1028

The Long-Term Pavement Performance GPS 1 Asphalt Concrete (AC) on Granular Base test section 23_1028 [1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." The test section was incorporated into the LTPP program in 1988. In 1992, there was full-depth patching, and in 1994, 22-years after it was originally constructed, the test section was overlaid with 1.9 inches of AC, moving it to the GPS-6B (Planned AC Overlay of AC) experiment. The performance of the test section has been mixed as high amounts of non-wheel path longitudinal cracking, transverse cracking, and rutting have been reported, but no fatigue/alligator cracking, wheel path longitudinal cracking, block cracking, or patching after the overlay in 1994. While the IRI on the section prior to the 1994 AC overlay was close to 100 inches/mile, it decreased prior to the overlay and has remained below 80 inches/mile as of the last survey in 2015. The deflection data is also indicative of a pavement that has remained structurally sound throughout its entire life. The primary objective of this investigation is to determine what is driving the performance of the pavement. The investigation will also look into the performance metric values, such as fatigue/alligator cracking, longitudinal cracking inside and outside the wheel-path, transverse cracking, rutting, and IRI, which at times do not appear to be reasonable for the pavement section.

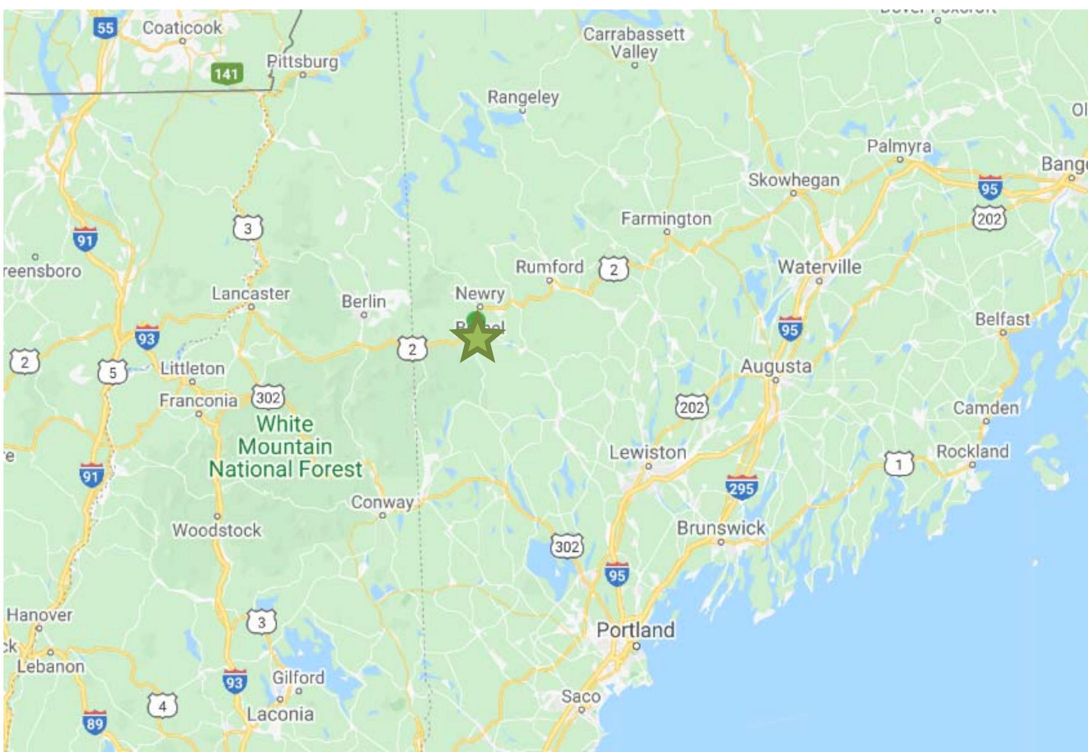
SITE DESCRIPTION

LTPP test section 23_1028 is located on U.S. 2, eastbound, in Oxford County, Maine. U.S. 2 is a rural principal arterial with one lane in the direction of traffic. The site is approximately 800 feet from the Androscoggin River. It is classified as being in a Wet, Freeze climate zone with an average annual precipitation ranging between 36 inches (2001) and 78 inches (2005) prior to 2019. The test section has an annual average air freezing index ranging between 1,161 deg-F deg-days (2006) and 2,533 deg-F deg-days (1989) during the performance period in question. The coordinates (in degrees) of the test section are 44.42956, -70.79961. Photograph 1 shows the test section at Station 0+00 looking eastbound in 2016, while Map 1 shows the geographical location of the test section.

[1] First two digits in test section number represent the State Code [23 = Maine]. 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.

Wood Environment & Infrastructure Solutions, Inc.
12000 Indian Creek Ct, Suite F
Beltsville, MD 20705
+1 301-210-5105
www.woodplc.com

Photograph 1. LTPP Section 23_1028 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 test section and its structural capacity, climate, traffic, and pavement distresses.

Pavement Structure and Construction History

The pavement at this test section was constructed in 1972 and consisted of 7.1 inches of asphalt concrete (AC) (over two layers) on 18.2 inches of a soil aggregate mixture base, on a poorly graded sand with gravel subgrade. The test section was incorporated into the LTPP program in 1988, as part of the GPS-1 (AC on Granular Base) experiment. The original pavement structure for the test section at the time of its incorporation into the LTPP program is summarized in Table 1; this information corresponds to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. In 1992, there was full-depth patching on the test section (CN=2), and in 1994, 22-years after originally constructed, the test section was overlaid with 1.9 inches of AC, moving the test section to the GPS-6B (Planned AC Overlay of AC) experiment. The pavement structure of CN=3 is depicted in Table 2.

Table 1. Pavement structure for CN =1

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Coarse-Grained Soils: Poorly Graded Sand with Grave
2	Unbound (granular) base	18.2	Soil-Aggregate Mixture (Predominantly Coarse-Grained)
3 and 4	Asphalt concrete layer	7.1	Hot Mixed, Hot Laid AC, Dense Graded

Table 2. Pavement structure for CN =3

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Coarse-Grained Soils: Poorly Graded Sand with Grave
2	Unbound (granular) base	18.2	Soil-Aggregate Mixture (Predominantly Coarse-Grained)
3 and 4	Asphalt concrete layer	7.1	Hot Mixed, Hot Laid AC, Dense Graded
5 and 6	Asphalt concrete layer	1.9	Hot Mixed, Hot Laid AC, Dense Graded

Following the 1994 overlay, cores were taken on each end outside of the section (before Station 0 and after Station 5) and used to determine the average thickness of each layer of the pavement. While there was limited variability in the thicknesses reported at each end of the section for most layers, the interlayer (Layer 5) of the overlay was reported to be 0.3 inches on one side of the section and 3 inches on the other side of the section. As it seems unlikely the layer thickness would vary that greatly from one end of the section to the other, for the purposes of this report, it was assumed that Layer 5 was 0.3 inches, the reported overlay design thickness, throughout the section. However, a more in-depth follow-up of this issue by the LTPP program is recommended.

Pavement Structural Properties

Figure 1 shows the average Falling Weight Deflectometer (FWD) deflection under the nominal 9,000-pound load plate over time. 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 and moisture at the time of testing, and by other factors. As depicted in Figure 1, the deflections observed on the site fluctuate over time. Prior to the overlay in 1994, the reported deflections over the load plate decreased modestly from 10.4 mils in July 1989 to 9.3 mils in May 1994. Following (and as a direct result of) the overlay in September 1994, the deflection reported drops to 6.3 mils. However, after 1994 the deflections fluctuate between 7.1 mils (2016) and 10.2 mils (2001). As depicted in Figure 1, some of the highest and lowest deflections values reported on the section, especially in 2001 and 2016, appear to be related to the temperatures leading up to the testing day – the highest temperature of 71°F and lowest temperature of 33°F, respectively—as shown in Figure 1.

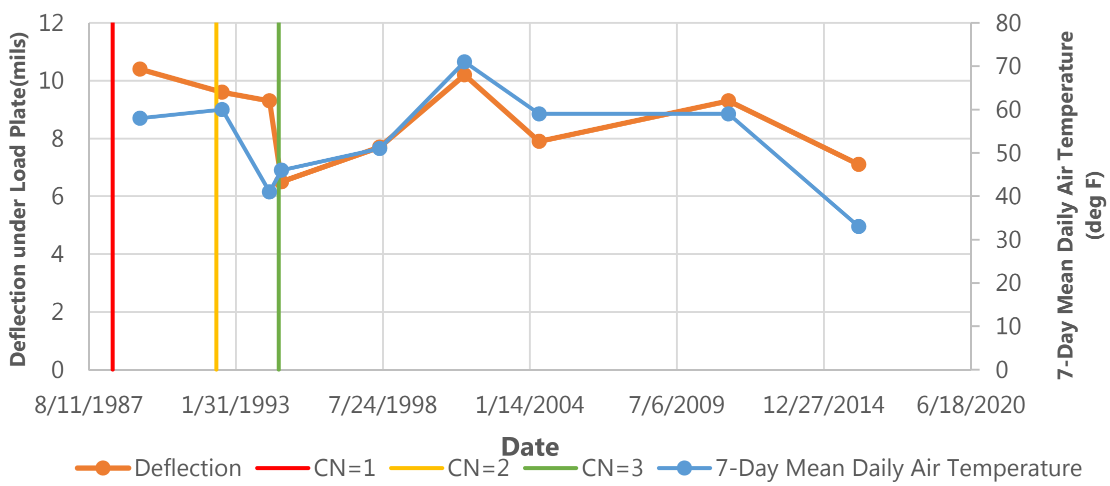


Figure 1. Time history of average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load.

The layer moduli backcalculated from the deflection data were also assessed for the test section. Prior to the overlay in 1994, the pavement structure was modeled as a 6.6-inch AC layer (Layer 1) over 24 inches of coarse granular base (Layer 2) and a semi-infinite subgrade layer (Layer 3). Following the overlay, the pavement structure was modeled as an 8.2-inch AC layer (Layer 1) over 24 inches of coarse granular base (Layer 2) and a semi-infinite subgrade layer (Layer 3). The thicknesses used in the backcalculations likely varied from the reported layer thicknesses of the test section due to a lack of strong results (with low RSME) when using the reported layer thicknesses for this section. Within the backcalculation database, it was noted the thicknesses of the layers used for the analysis differed from the actual thicknesses of the section layers, however, further information on these differences should be pursued. The backcalculated modulus for each layer in July 1989, July 1992, May 1994, October 1994, June 1998, August 2001, May 2004, and June 2011 (eight FWD testing dates) are shown in Figures 2-4. Backcalculations were not performed on the deflection data collected in April 2016.

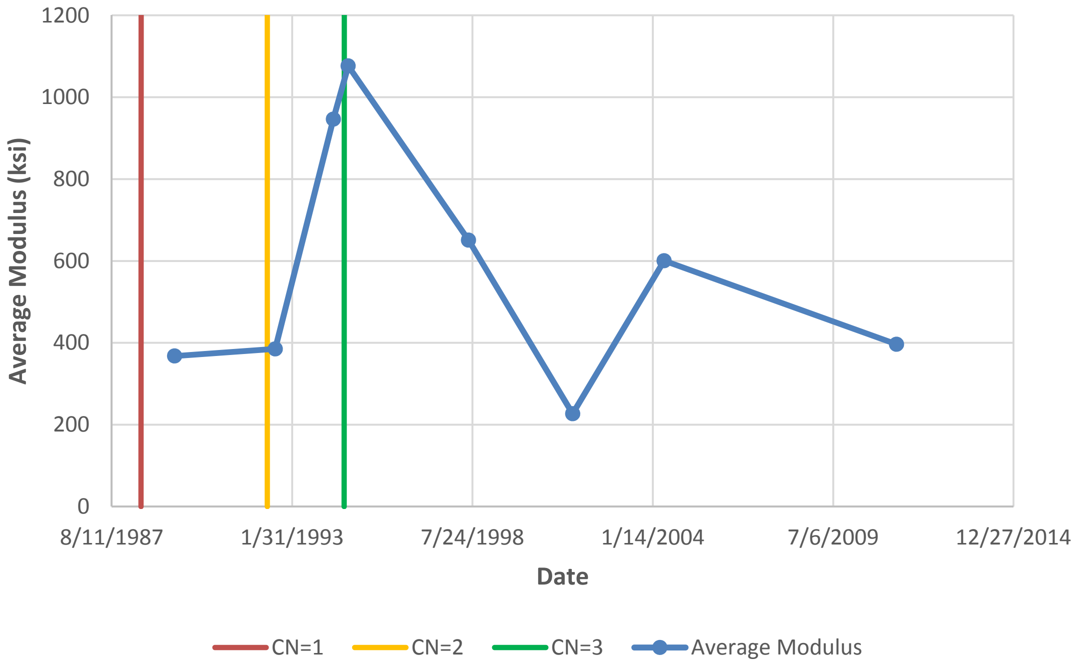


Figure 2. Average backcalculated modulus for AC layer (Layer 1).

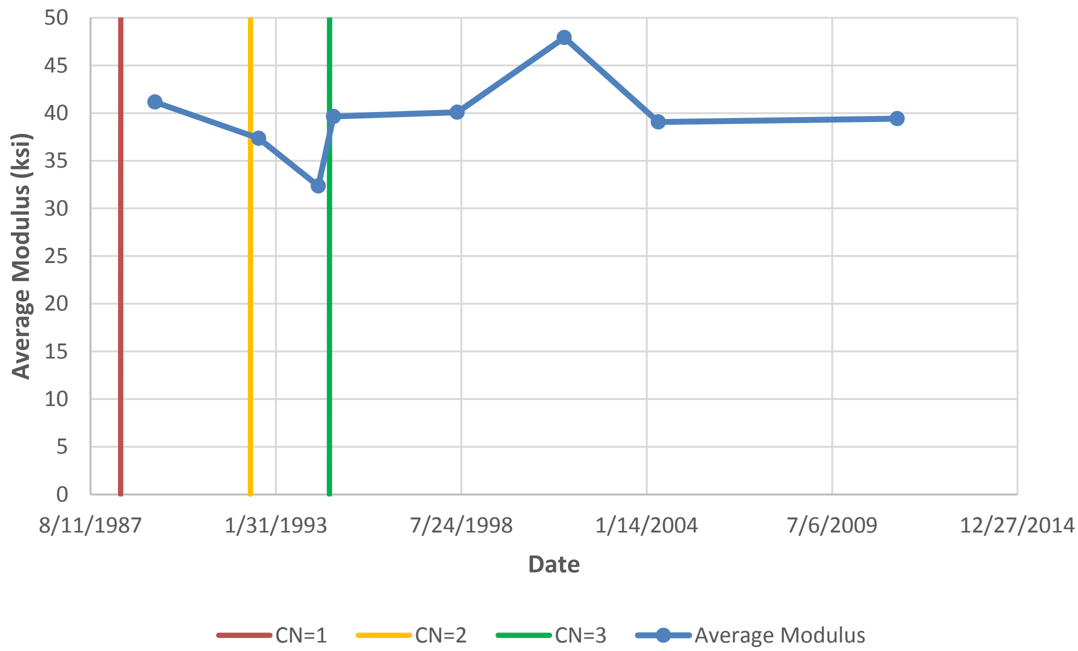


Figure 3. Average backcalculated modulus for granular base (Layer 2).

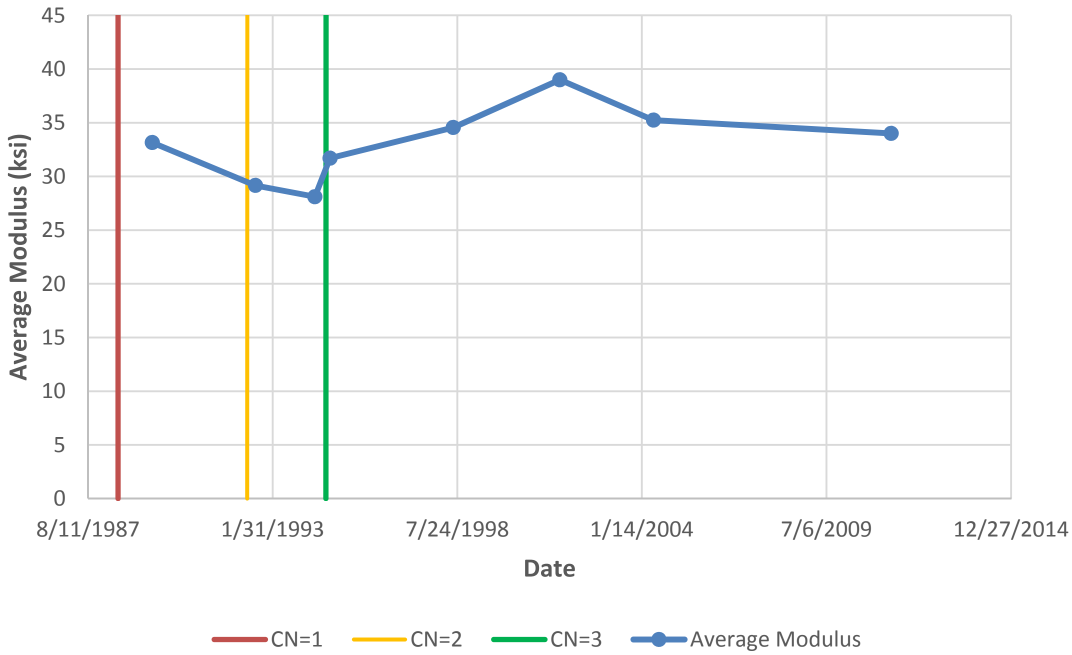


Figure 4. Average backcalculated modulus for subgrade (Layer 3).

The AC moduli appear to be quite reasonable, varying between 200 and 1,100 ksi. The value of 200 ksi was determined when the temperature was 71°F, while values close to 1,100 ksi were determined when the temperature was in the low-to-mid 40s (°F). Perhaps the only surprising value is 400 ksi when the temperature was 33°F. Similarly, the backcalculated granular base layer moduli (includes 18.1 inches of granular material and 5.9 inches of top of subgrade) appear quite reasonable, ranging between 32 and 48 ksi over the 22-year period of 1989 to 2011. This is also the case for the remainder of the subgrade layer, with values ranging between 28 and 39 ksi over the same 22-year period.

The above conclusion about the reasonableness of the AC backcalculated layer moduli is further confirmed by comparing the results to those derived from laboratory resilient modulus testing. Table 3 summarizes the laboratory test results for the AC layer (lab results for the subgrade and base layers were not reported). For the AC layer, moduli values are shown for three test temperatures – 41, 77 and 104°F, respectively.

Table 3. Laboratory Resilient Modulus Test Results

Layer	Temperature (°F)	Number of Samples/test results	Range of moduli values (ksi)
AC	41	1 sample (2 tests)	1,288-1,435
	77	1 sample (2 tests)	315-323
	104	1 sample (2 tests)	61-80

As shown in Figure 5, the AC modulus-temperature trend defined by the two datasets appears to be reasonable. The exception is the field-derived values above 80°F, which appear unusually high.

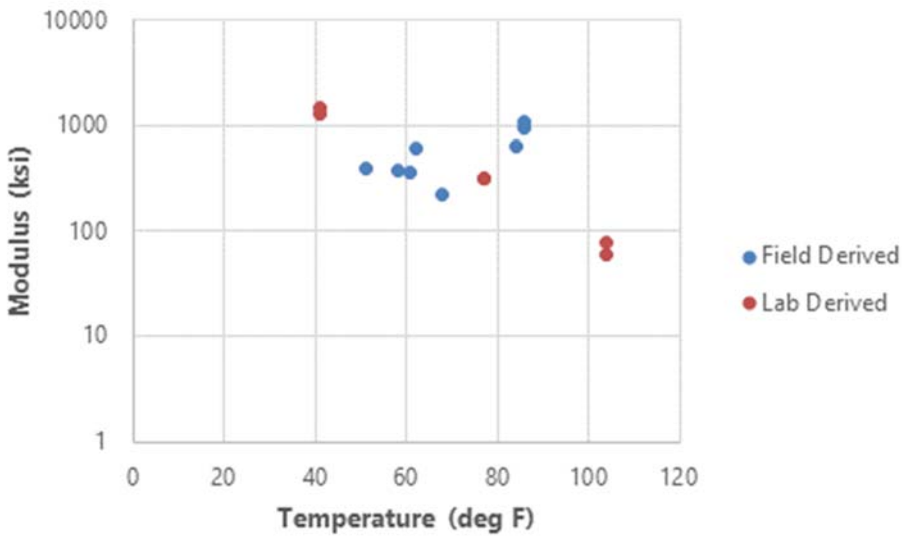


Figure 5. Field- and lab-derived AC resilient modulus values.

Climate History

The time history for average annual precipitation (from MERRA data) is shown in Figure 6. In 2005, the amount of precipitation appears to be a local high (78 inches), while the low (36 inches) was recorded in 2001. The mean precipitation recorded at the site is 55 inches for the period shown in Figure 6. Overall, the site has reported high amounts of precipitation over time, which is expected for a Wet climate. Most notable is from 2005 to 2014, when on average over 60 inches of precipitation was observed each year.

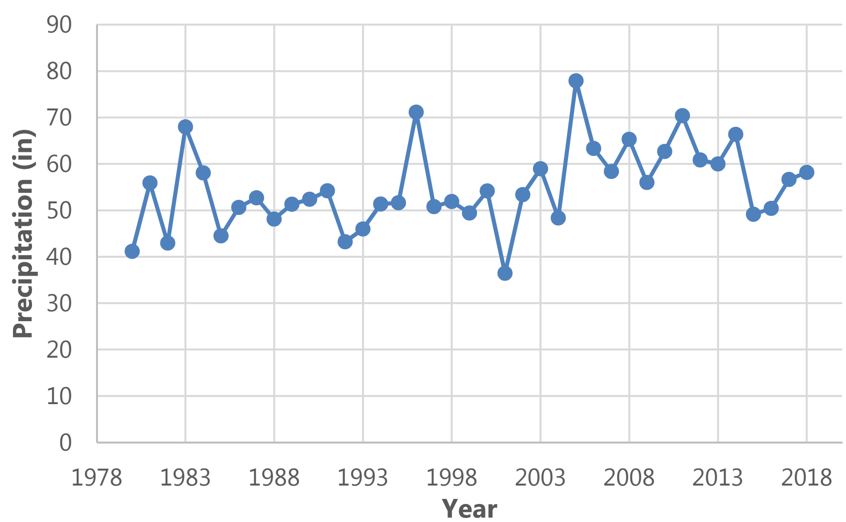


Figure 6. Average yearly precipitation over time.

Figure 7 shows the time history of the average annual freezing index (from MERRA data) 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 7, the freezing index values ranged from 1,161 (2006) to 2,353 (in 1989)—which is well above the 150 deg F deg days used to classify a freeze region—indicating it may be a factor affecting the performance of the test section.

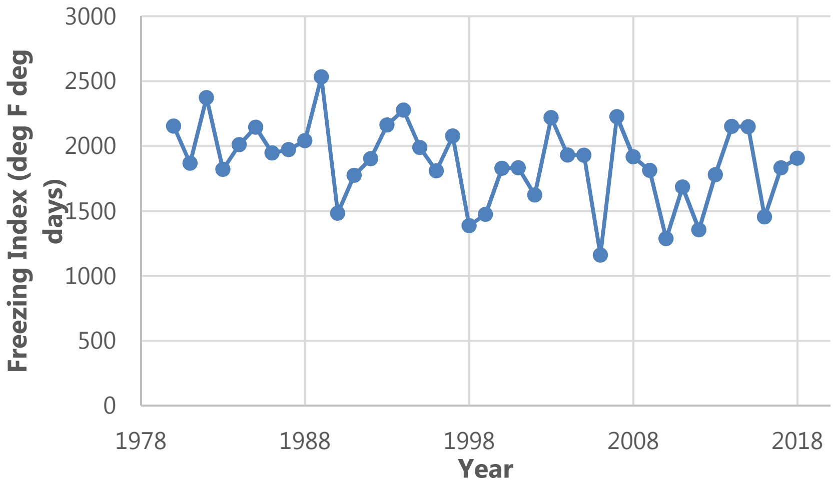


Figure 7. Average annual freezing index over time.

Truck Volume History

Figure 8 shows the annual average daily truck traffic (AADTT) data in the LTPP test lane by year from 1990 to 2017. While the section was incorporated into the LTPP program in 1988, the AADTT reported prior to 1990 relied on historical AADTT estimations from the State, which grossly overestimated the AADTT and ESALs during this time period and therefore, was not included in the figure. The annual truck traffic counts increase from 215 in 1,990 to 343 in 2017, or approximately 5 additional trucks per day per year. The overall increase in reported AADTT is not consistent throughout time. This is likely a result of source of AADTT data over time. Between 1990 and 1999, the reported AADTT values are state provided, from 2000 to 2010, monitored data is reported, and between 2011 and 2017, AADTT is computed using a compound growth function. The average number of ESALs reported on this section also increased over time from 165,000 in 1990 to 255,398 in 2017 as depicted in Figure 9. Like the AADTT, the reported ESALs fluctuated over time, likely as a result of the data source.

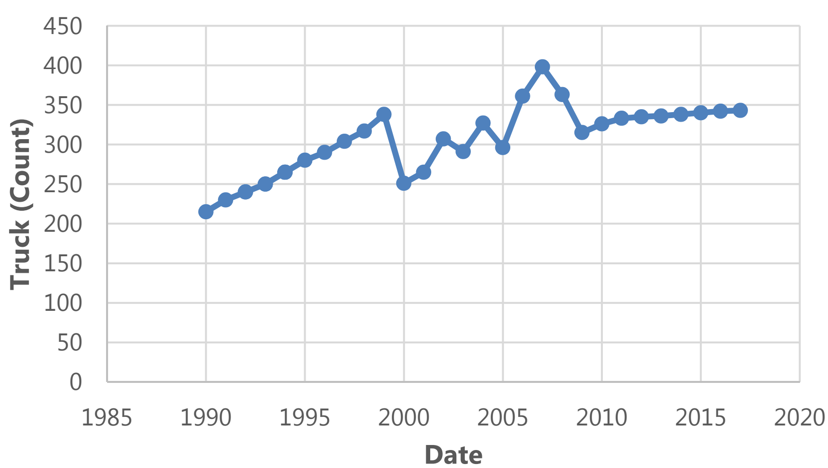


Figure 8. Average annual daily truck traffic (AADTT) history.

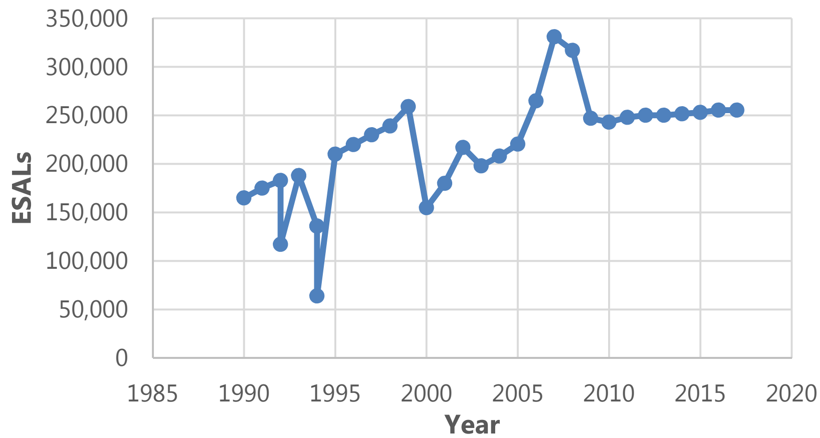


Figure 9. Estimated annual ESALs for vehicle classes 4-13 over time.

Pavement Distress History

The following section summarizes the distresses observed on the test section between 1989 and 2016, which is when the most recent manual distress survey was performed on the test section. Fatigue/alligator cracking, longitudinal cracking (inside and outside the wheel path), transverse cracking, IRI, and rutting were assessed. Little to no block cracking (0 ft^2) or patching (14 ft^2 in 1994) was observed on this section and therefore, these distresses were not included in this analysis.

Fatigue/Alligator Cracking

Figure 10 shows the total area of fatigue-related cracking observed on the section. Prior to the AC overlay in 1994, fatigue cracking was not observed until 1993 when 7 ft^2 was reported on the section. A spike in fatigue cracking was reported in May 1994 when 358 ft^2 of fatigue cracking is observed.

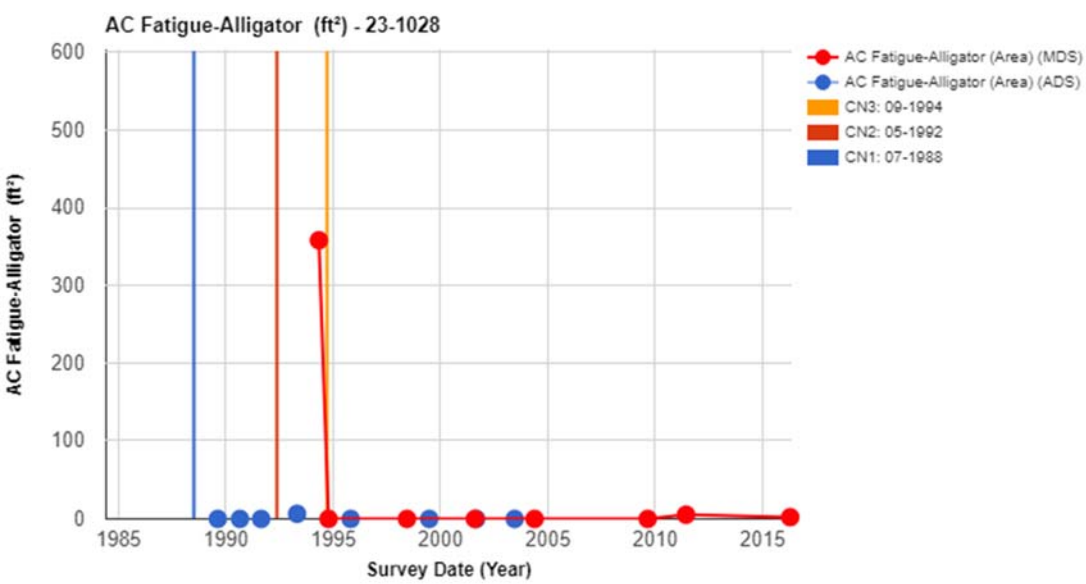


Figure 10. Time history of the area of bottom-up cracking.

As shown in Figure 10, the fatigue cracking reported in 1994 corresponds to the first manual distress survey conducted on the section; all surveys prior to this date were automated distress surveys. As automated distress surveys relied on images to assess distresses along test sections, some cracking, especially low severity cracking like the fatigue cracking observed on this section, was not adequately captured and therefore, underreported. The fatigue cracking that was observed in the 1994 manual distress survey was mostly near the centerline of the section. Following the overlay in 1994, the area of roadway where fatigue cracking had been observed is no longer notable. Fatigue cracking is first observed in 2011, 17 years after the overlay, when 5 ft^2 was reported on the section. Minimal fatigue cracking is observed in the subsequent distress survey in 2016.

It is hypothesized that the cause of the fatigue cracking prior to the overlay in 1994 is likely a result of the high levels of moisture and the freeze-thaw cycles experienced at this test section. Because this area is in a wet, freeze climate with thaw in the spring, the fatigue cracking may be related to weakened base and subgrade layers. However, the reason why fatigue cracking is occurring near the centerline is not clear. Is it because the base and subgrade layers are weaker near the centerline? Is it because the layers in the pavement structure are thinner near the centreline? Is winter traffic more damaging and are the wheel paths being shifted towards the centerline because of snow cover? This fatigue cracking location issue will need to be addressed as part of follow-up investigations.

Longitudinal Cracking

Data on longitudinal cracking, inside and outside the wheel path, were collected between 1989 and 2016 as shown in Figures 11 and 12. In May 1994, the date of the first manual distress survey for this section, 575 feet of non-wheel path (NWP) longitudinal cracking was reported. Prior to the overlay in 1994, the NWP longitudinal cracking observed fluctuated. This is likely due to the source of the data reported. As discussed previously, the first manual distress survey was conducted on this site in 1994. Prior to 1994, automated distress surveys were used to report the longitudinal cracking observed on this site. The NWP longitudinal cracking observed prior to the overlay was mostly located at the edges of the lane and in between the wheel paths. Following the overlay, the NWP longitudinal cracking observed on the section dropped to zero in 1994. However, NWP longitudinal cracking was observed again in 1998, four years after the overlay, when 136 feet of NWP longitudinal cracking was observed. Once cracking was initiated, it propagated at a rate of 49 feet/year between 1998 and 2016. The NWP longitudinal cracking observed following the overlay is predominantly located at the edges of the lane.

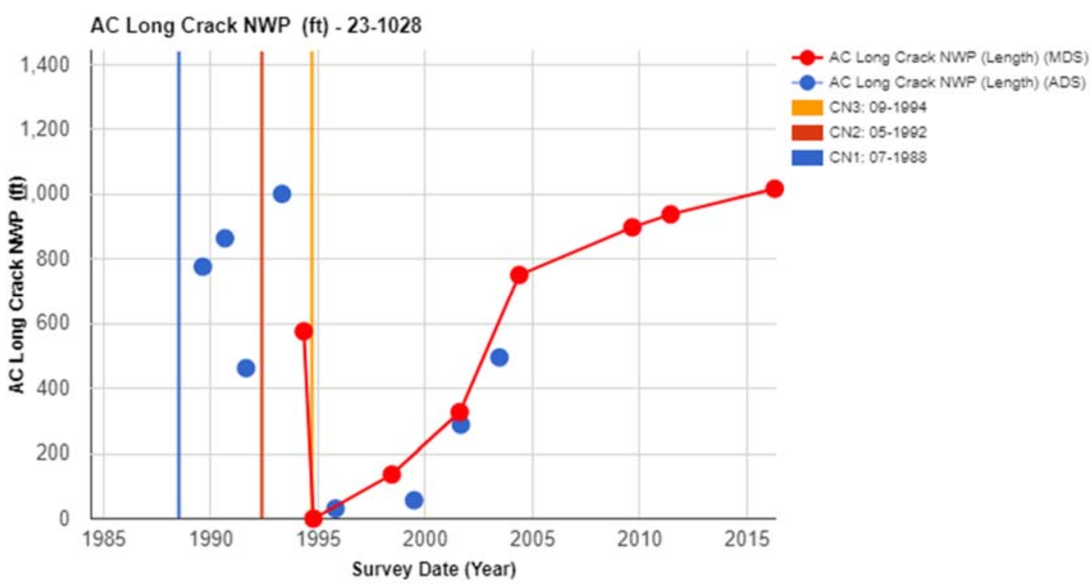


Figure 11. Time history of the length of NWP longitudinal cracks.

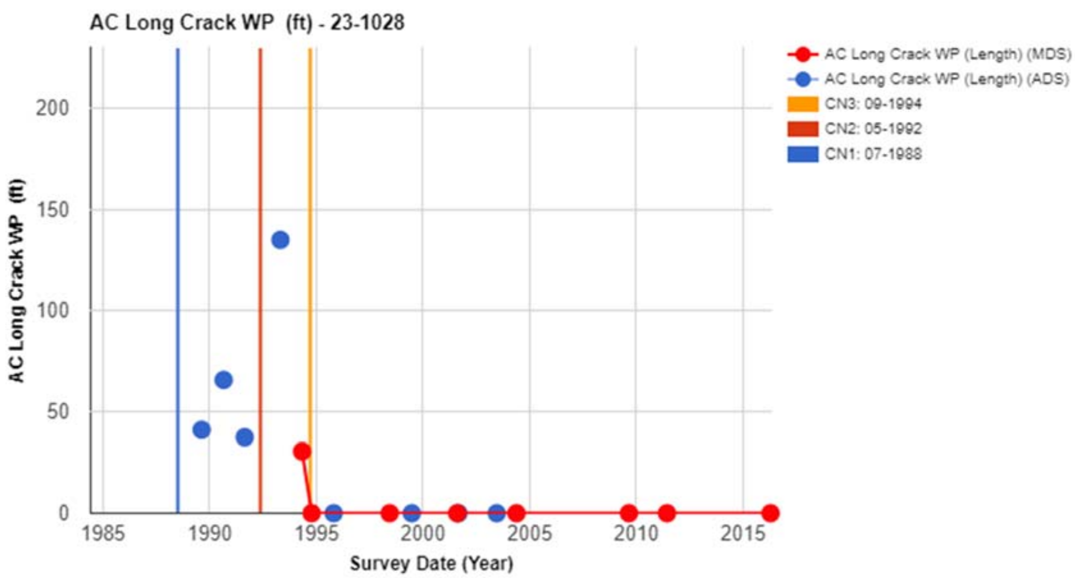


Figure 12. Time history of the length of WP longitudinal cracks.

It is hypothesized that some (not all) of the NWP longitudinal cracking observed on this section is likely related to the fatigue cracking observed near the centerline of the test section; i.e., it is probable that it is low severity fatigue cracking and not NWP longitudinal cracking. This is further confirmed by the state of the test section's shoulder over time which shows similar fatigue-related distresses as well as pumping. The remaining NWP longitudinal cracking observed at the test section, as depicted in Figure 13, appears to be reflection cracking of the NWP longitudinal cracking observed prior to the overlay. While the amount of cracking observed following the overlay has surpassed the amount of NWP longitudinal cracking observed in 1994, the level of severity of the cracking following the overlay is mostly low while prior to the overlay the severity ranged from low to high.

Minimal longitudinal cracking inside the wheel path (WP) is observed on this section. In 1994, 9.30 feet of longitudinal cracking inside the wheel path (WP) appeared along the section during the first reported manual distress survey. Prior to the overlay in 1994, the observed WP longitudinal cracking fluctuates. This is likely due to the source of the data reported. As discussed previously, the first manual distress survey was conducted on this site in 1994. Prior to 1994, automated distress surveys were used to report the longitudinal cracking observed on this site. After the application of the AC overlay in 1994, WP longitudinal cracking was not reported again.

Transverse Cracking

Data on transverse cracking was collected between 1989 and 2016 as shown in Figures 14 and 15. Transverse cracking appeared along the section during the first reported automated distress survey in 1989, when 76 feet (23 cracks) of cracking was reported. The transverse cracking observed increases prior to the 1994 overlay reaching 309 feet (154 cracks) in 1994. Following the overlay, the transverse cracking observed on the section dropped to zero. However, transverse cracking was observed again in 1998, four years after the overlay, when 17 feet (3 cracks) were observed. Once cracking initiated, it propagated at a rate of 9 feet/year between 1998 and 2016.

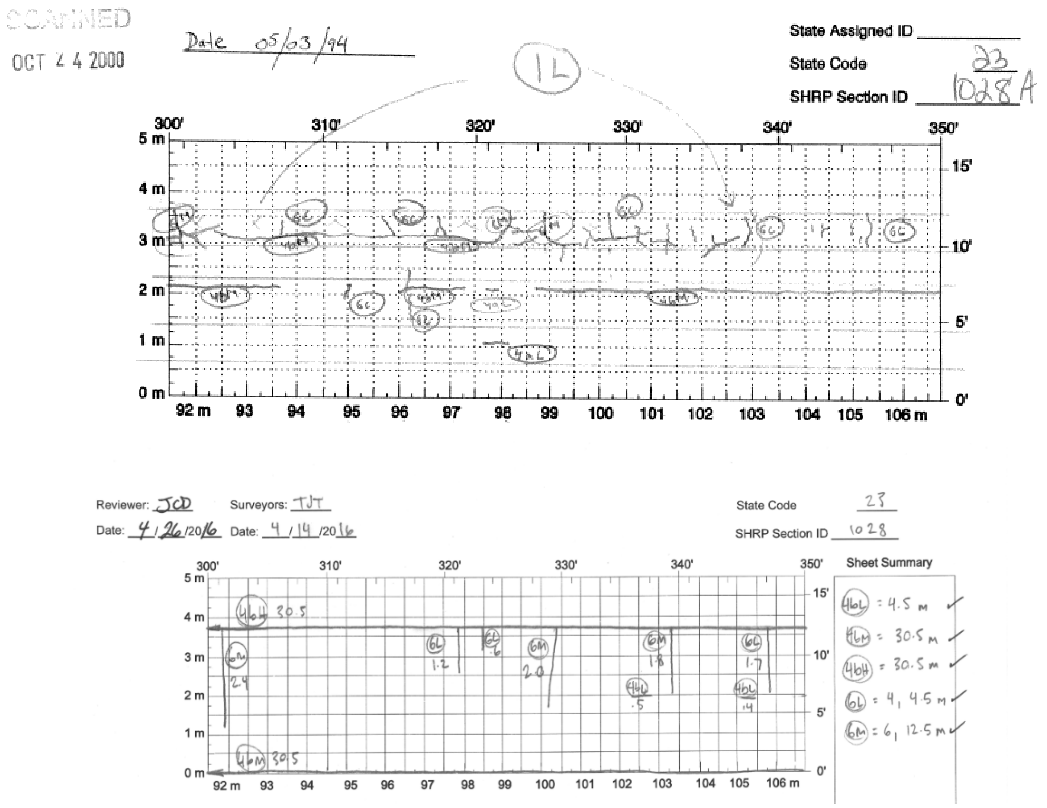


Figure 13. NWP longitudinal cracking on the test section between Station 3 and Station 3.5 in 1994 prior to the overlay (top) and 2016 (bottom)

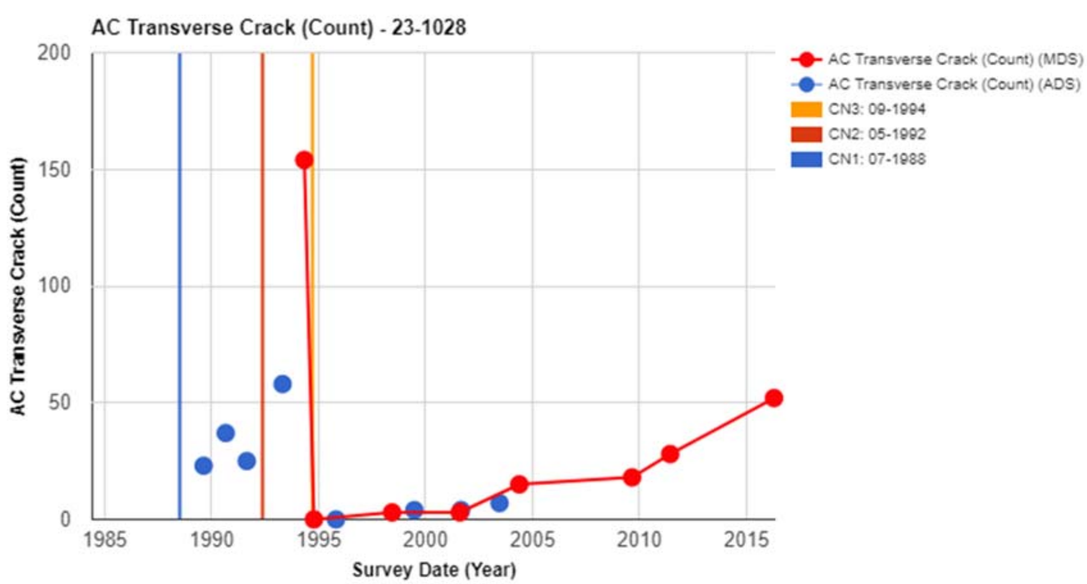


Figure 14. Time history of the number of transverse cracks.

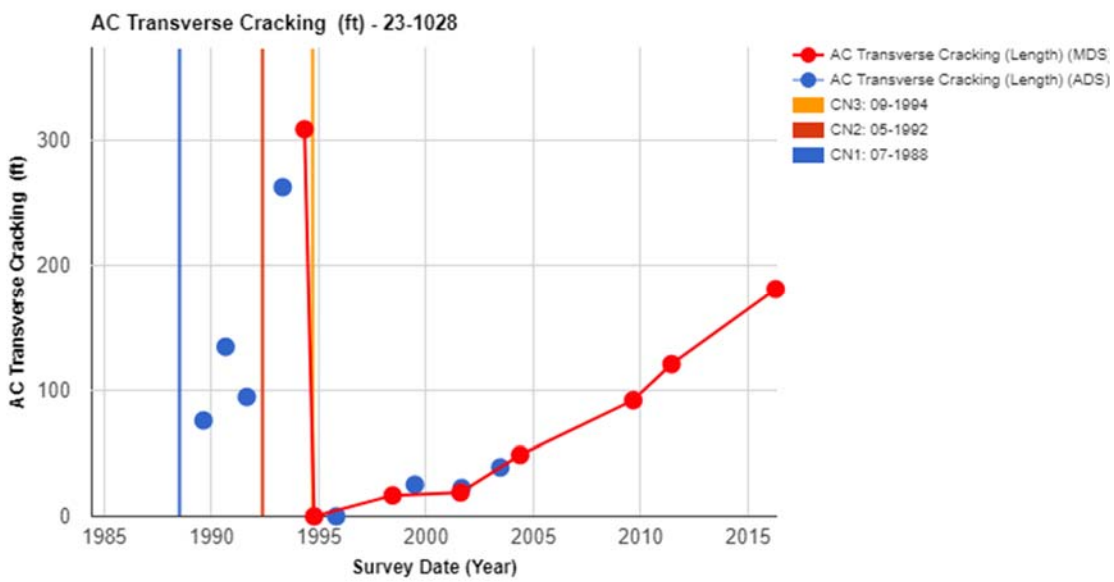


Figure 15. Time history of the length of transverse cracking.

The transverse cracking observed on this section was mostly short cracks near the edges of the lane rather than full width transverse cracks. This appears to indicate that the observed transverse cracking is not likely low temperature cracking.

IRI

The average IRI measurements for the section over time are shown in Figure 16 and the standard deviation of the IRI measurements for each test date is shown in Table 4.

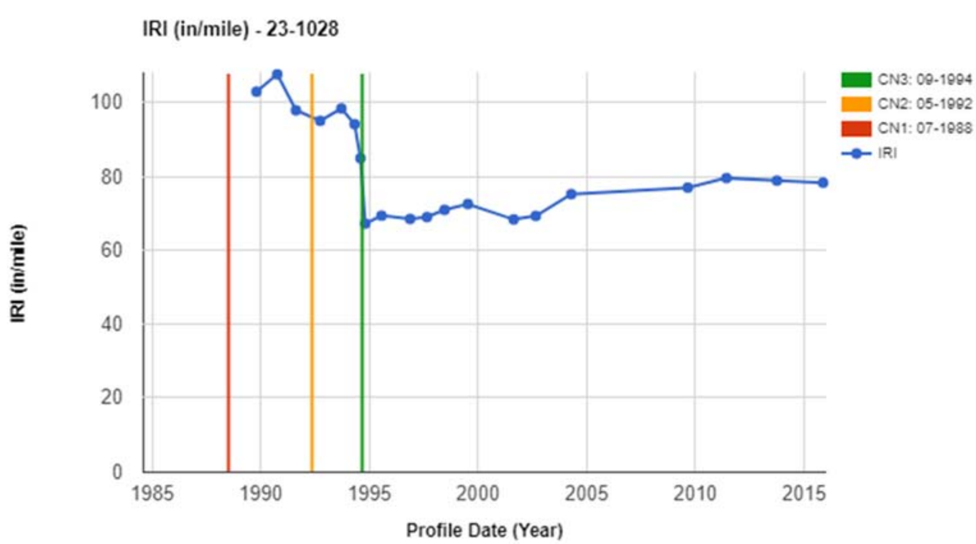


Figure 16. Time history plot of pavement roughness.

Table 4. Standard deviation of IRI values over time.

Date	Average IRI (in/mi)	Standard Deviation
10/21/1989	103.0	1.3
10/05/1990	107.6	2.6
08/16/1991	97.9	1.4
09/26/1992	95.0	1.1
09/17/1993	98.4	0.9
04/28/1994	94.1	1.0
08/04/1994	85.1	0.6
10/22/1994	67.3	0.7
07/24/1995	69.5	0.7
11/15/1996	68.6	0.5
08/26/1997	69.1	0.9
06/17/1998	71.1	1.1
07/15/1999	72.7	0.7
08/22/2001	68.5	0.4
08/29/2002	69.5	0.8
04/16/2004	75.3	1.0
08/26/2009	77.1	0.6
06/08/2011	79.7	0.5
10/02/2013	79.0	0.4
11/17/2015	78.4	2.0

Prior to the AC overlay in 1994, the IRI reported on the test section fluctuated modestly with values ranging from 108 in/mi in 1990 to 94 in/mi in 1994. The reported IRI appears to decrease overall prior to the overlay with the pavement's IRI performance starting at "Fair" and ending at "Good" based on FHWA performance definitions. This downward trend in IRI prior to the overlay may be related to the variability in the measurements used to calculate the average IRI of the test section over time. As shown in Table 4, while subtle, the standard deviations of the average IRI values recorded per run per test date prior to the overlay were on average greater than after the overlay. This indicates a higher variability in the IRI values reported prior to the overlay, which could explain the downward trend in IRI observed. Following the AC overlay in 1994, the IRI on the section dropped to 67 in/mile in 1994. The IRI reported increases to 78 in/mile by 2015, increasing at a rate of just 0.5 in/mile over 21 years. The pavement's IRI performance is classified as "Good" during this period based on FHWA performance definitions.

Rutting

The rutting observed over time on the test section is shown in Figure 17. Prior to the overlay in 1994, the rut depths observed increased from 0.47 inches in 1989 to 0.55 inches in 1993. These large rutting values were likely the driver for the application of the overlay in 1994, but this needs to be confirmed by the LTPP program as such information could not be found in the LTPP database. Following the overlay in 1994, the rutting observed on the section decreased to 0.12 inches in 1995. The rutting increased between 1995 and 2016 reaching a rut depth of 0.43 inches in 2016.

In addition to the average rut depth observed over time, the change in the transverse profile of the test section at one location (station 150) was also investigated. Figure 18 depicts the changes in the profile over time with red lines representing rutting profiles of the roadway prior to the overlay in 1994. From the figure, the transverse profiles observed prior to the AC overlay in 1994 showed deeper depressions along the wheel path over time whereas the rutting following the overlay was less pronounced in the wheel-path and is instead concave down. It should also be noted that the transverse profiles prior to the overlay, while normalized, show the edge of the lane (offset equal to 0) is more than 0.4 inches below the next point of the transverse profile (to the left of the edge). This seems to indicate that the values calculated on the lane edge, which showed material degradation, may be abnormally lower than the rest of the surface. The use of the deteriorated edge as the first point of the transverse profile may have also impacted the overall rut depths reported on this section.

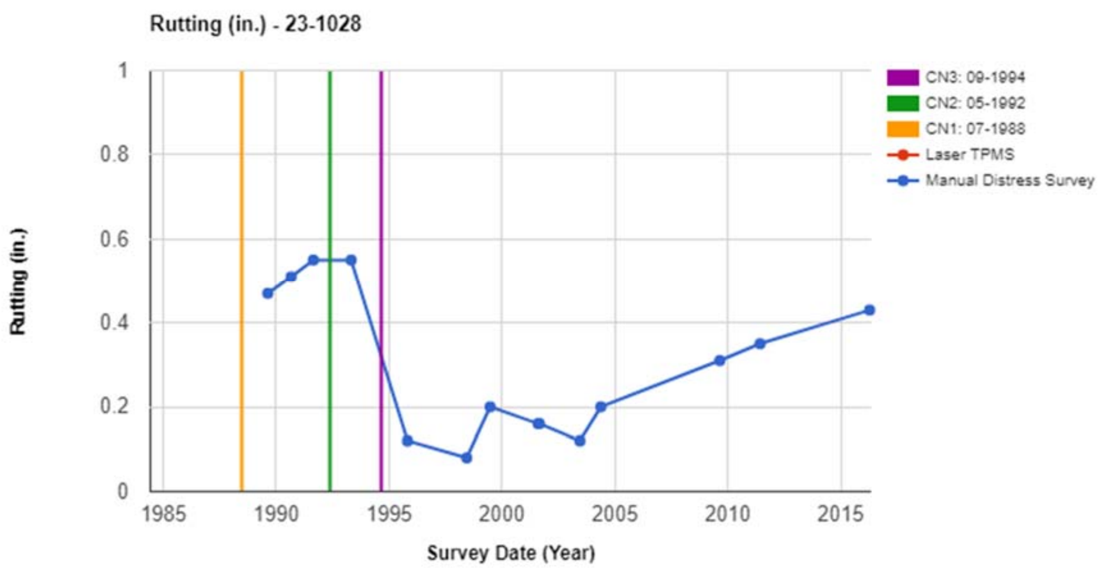


Figure 17. Time history plot of average rut depth computations.

.

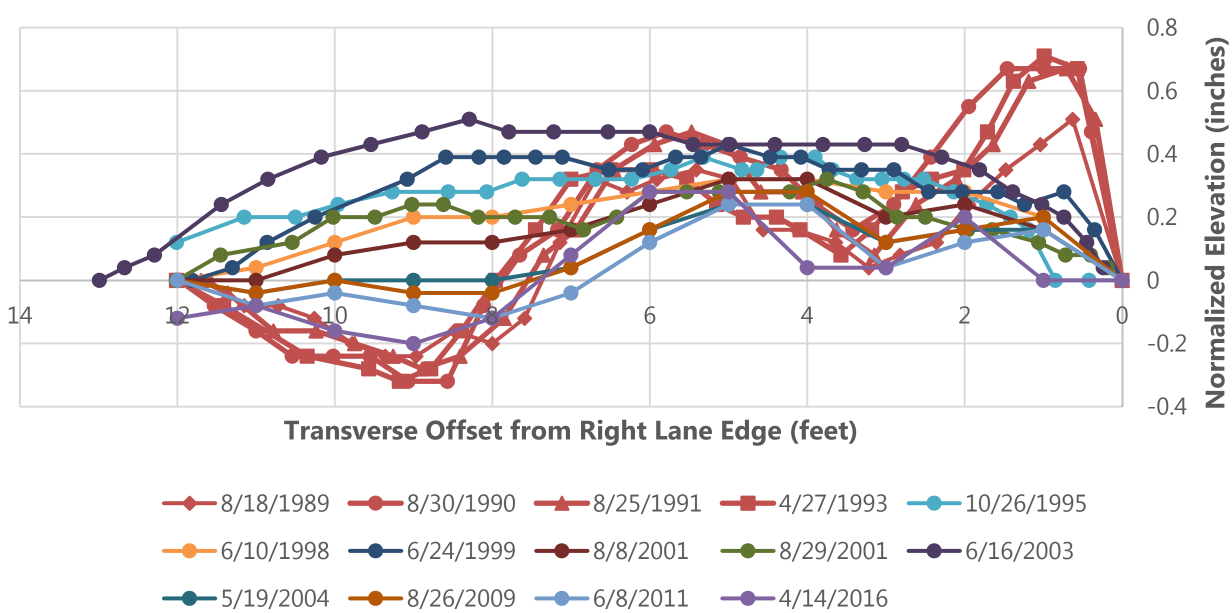


Figure 18. Plot of transverse profiles at Station 1.5 over time.

The change in the transverse profile of the test section at all locations over time was also investigated. Using the transverse profiles, an analysis of the predominant layer where the plastic deformation occurred was assessed using the method developed in NCHRP 01-34a.[2] The 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 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. Figure 20 describes the parameters used to determine the lowest layer contributing to the pavement's surface deformation.

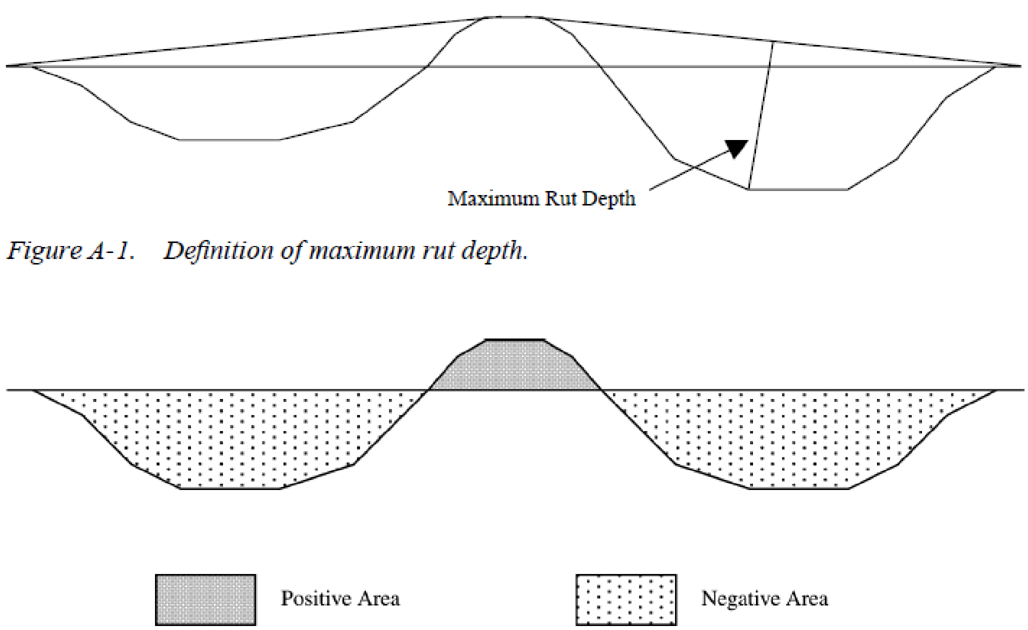
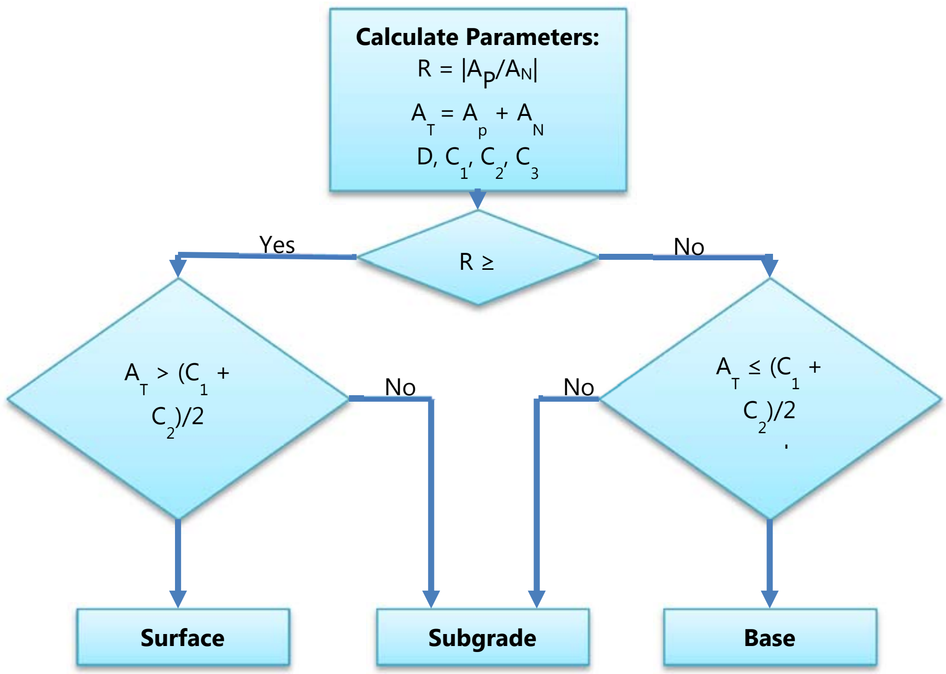


Figure 19. Transverse profile maximum rut depth and positive and negative areas (White et al., 2002)

[2] 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.



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)

Based on the analysis conducted for each of the transverse profiles (total of 9-11 profiles spaced at 50 ft), the layer contributing the most to rutting was determined for each data collection date at multiple locations along the section. It was determined that the AC surface was the lowest layer contributing to rutting for every transverse profile assessed except one in 1989. Based on both the findings of the analysis and the transverse profiles, the rutting observed is likely related to permanent deformations in the AC layer. This may be the result of a softer asphalt binder being used in Maine coupled with hot temperatures during the summer and traffic loadings.

SUMMARY OF FINDINGS

LTPP test section 23_1028 is located on U.S. 2, eastbound, in Oxford County, Maine. U.S. 2 is a rural principal arterial with one lane in the direction of traffic. The pavement at this test section was constructed in 1972 and consisted of 7.1 inches of asphalt concrete (AC) (over two layers) on 18.2 inches of a soil aggregate mixture base, on a poorly graded sand with gravel subgrade. The test section was incorporated into the LTPP program in 1988, as part of the GPS-1 AC on Granular Base experiment. In 1992, there was full-depth patching, and in 1994, 22-years after originally constructed, the test section was overlaid with 1.9 inches of AC, moving it to the GPS-6B (Planned AC Overlay of AC) experiment.

The primary objective of this investigation was to determine what is driving the performance of the pavement as well as to assess the performance metrics over time. Below is a summary of the key findings:

- **Fatigue/alligator cracking:** Prior to the AC overlay in 1994, fatigue cracking was not observed until 1993 when 7 ft^2 was reported on the section. A spike in fatigue cracking is then reported in May 1994, when 358 ft^2 of fatigue cracking was recorded. While there is minimal fatigue cracking reported prior to 1994, the apparent spike is likely related to the distress collection method rather than a sudden increase in fatigue cracking – i.e., while already present, fatigue cracking was not captured using the automated method used prior to 1994. Following the overlay in 1994, the area of roadway where fatigue cracking had been observed is no longer noticeable. It is hypothesized that the cause of the fatigue cracking prior to the overlay in 1994 was likely a result of the high levels of moisture and the freeze-thaw cycles experienced at this test section. Because this area is in a wet, freeze climate with heavy thaw likely in the spring, the cracking may be related to a weakened base and subgrade layers. However, the reason why fatigue cracking is occurring near the centerline needs to be investigated as part of the follow-up investigations.
- **NWP longitudinal cracking:** In May 1994, the date of the first manual distress survey for this section, 575 feet of non-wheel path (NWP) longitudinal cracking was reported. Following the overlay, the NWP longitudinal cracking observed on the section dropped to zero in 1994. However, NWP longitudinal cracking was again recorded in 1998, four years after the overlay, when 136 feet of NWP longitudinal cracking was observed. Once cracking was initiated, it propagated at a rate of 49 feet/year between 1998 and 2016. Prior to the overlay, some of the NWP longitudinal cracking observed appears to be related to the fatigue cracking observed near the centerline. Following the overlay in 1994, it appears the NWP longitudinal cracking observed prior to the overlay is reflected onto the new overlay surface.
- **WP longitudinal cracking:** In 1994, 9.30 feet of longitudinal cracking inside the wheel path (WP) appears along the section during the first reported manual distress survey. After the application of the AC overlay in 1994, WP longitudinal cracking was not reported again.
- **Transverse cracking:** Transverse cracking appears along the section during the first reported manual distress survey in 1994, when 309 feet of transverse cracking (154 cracks) was reported. Following the overlay, the transverse cracking observed on the section dropped to zero. However, transverse cracking was observed again in 1998, four years after the overlay, when 17 feet (3 cracks) was observed. Once cracking initiated, it propagated at a rate of 9 feet/year between 1998 and 2016. The transverse cracking observed on this section was mostly short cracks near the edges of the lane rather than full width transverse cracks. This would suggest that the observed transverse cracking is not likely low temperature cracking.
- **IRI:** Prior to the AC overlay in 1994, the IRI reported on the test section fluctuated with values ranging from 108 in/mi in 1990 to 94 in/mi in 1994. This downward trend in IRI reported prior to the overlay may be related to the variability in the measurements used to calculate the average IRI of the test section over time. Following the AC overlay in 1994, the IRI on the section dropped to 67 in/mile. The IRI then increased to 78 in/mile by 2015, increasing at a rate of 0.5 in/mile over 21 years, which is a notable achievement.
- **Rutting:** Prior to the overlay in 1994, the rut depths observed increased from 0.47 inches in 1989 to 0.55 inches in 1993. These large rutting values were likely the driver for the application of the overlay in 1994. Following the overlay in 1994, the rutting observed on the section decreased to 0.12 inches in 1995. The rutting then increased between 1995 and 2016 reaching a value of 0.43 inches in 2016. In addition to the average rut depth observed over time, the change in the transverse profile of the test section at one location (station 150) was also investigated. The transverse profiles observed prior to the AC overlay in 1994 show deeper depressions along the wheel path over time, whereas the rutting following the overlay is less pronounced in the wheel-path and is instead concave down. It was also noted that the transverse profiles prior to the overlay, while normalized, show that the edge of the

lane (offset equal to 0) is more than 0.4 inches below the next point of the transverse profile (to the left of the edge). This seems to indicate the values calculated on the lane edge, which showed material degradation, may be abnormally lower than the rest of the surface. Using the method defined in NCHRP 01-34a, the lowest layer contributing to the rutting was determined to be the AC surface layer. Based on both the findings of the analysis and the transverse profiles, the rutting observed is likely related to the permanent deformation of the AC layer given the combination of a softer asphalt binder coupled with hot temperatures during the summer and traffic loadings.

- **Other findings:** In 1994, 22-years after originally constructed, the test section was overlaid with 1.9 inches of AC (two layers). Following the 1994 overlay, cores were taken on each end outside of the section (before Station 0 and after Station 5) and used to determine the average thickness of each layer of the pavement. While there was limited variability in the thicknesses reported at each end of the section for most layers, the interlayer (Layer 5) of the overlay was reported to be 0.3 inches on one side of the section and 3 inches on the other side of the section. Similarly, differences in layer thicknesses were also reported for the backcalculations of layer moduli for this section. The thicknesses used in the backcalculations seem to have varied from the reported layer thicknesses of the test section due to a lack of reasonable results (e.g., low RSME) when using the reported layer thicknesses for the section. In both cases, further information on these differences in thicknesses should be pursued.

FORENSIC EVALUATION RECOMMENDATIONS

Based on the information gathered and analyzed in the above sections, these follow-up actions are recommended:

1. LTPP close-out monitoring (FWD testing is optional but desirable). This is considered a crucial activity as distress, IRI, and rutting data for this section have not been collected since 2016, which means an important point in time is missing for the performance metrics in question.
2. Coring and boring to enable the following activities:
    a. Confirm that layer thicknesses match those reported when the test section was incorporated into the LTPP program. In turn, this will enable the following:
        i. Investigation into differences in layer thicknesses reported for the test section in various LTPP data tables and, more specifically, clarification on the reported layer thickness of the AC interlayer (Layer 5).
        ii. 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.
        iii. Investigation into whether or not layer thicknesses decrease towards the centerline, which is where fatigue cracking was observed.
    b. Confirm that NWP longitudinal cracking at the test section is reflection cracking.
    c. Obtain adequate unbound granular base and subgrade layer material for purposes of carrying out laboratory resilient modulus testing.
    d. Allow for dynamic cone penetrometer (DCP) testing to be carried out to further characterize the unbound granular base and subgrade layers.

These recommended activities should be carried out by the LTPP program with assistance from the Maine DOT and the TPF-5(332) pooled fund study team.