Technical Memorandum

To: Kim Schofield

From: Lauren Gardner, Gonzalo Rada, and Kevin Senn

cc: Mustafa Mohamedali

Date: June 4, 2021

Re. Forensic Desktop Study Report: Idaho LTPP Test Section 16_1020

The Long-Term Pavement Performance GPS-1 Asphalt Concrete (AC) on Granular Base test section 16_1020[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." The test section was constructed in September 1986 and incorporated into the LTPP program as a GPS-1 study in July 1988. In June 2011, twenty-five years after it was constructed, the test section was moved to the GPS-6C AC Overlay Using Modified Asphalt of AC Pavement-No Milling study after receiving a 2.4-inch AC overlay. Overall, the test section has performed well over time with respect to longitudinal cracking, which remained minimal, IRI, which remained below 55 in/mi throughout the entire analysis period, and average deflection under the center load plate, which remained constant prior to the overlay. The level of rutting reported on the test section prior to the overlay was notable, reaching 0.35-inch in 2011. However, between 1990 (the first year where rutting data was available) and 2011, the average rut depth only increased by 0.15-inch, indicating that more than half of the observed rutting occurred prior to 1990. Accordingly, the objectives of this study were to examine: (1) the key reasons for the relatively good performance of the test section and (2) the key cause(s) of rutting at this test section.

SITE DESCRIPTIONS

LTPP test section 16_1020 is located on U.S. Route 93, northbound, in Jerome County, Idaho. U.S. Route 93 is a rural principal arterial with one lane in the direction of traffic. The test section is classified as being in a Dry, Freeze climate zone. The coordinates (in degrees) of the site are 42.73878, -114.4381. Photograph 1 shows the section at Station 0+00 looking northbound in 2013, while Map 1 shows the geographical location of the test section.

[1] First two digits in test section number represent the State Code [16 = Idaho]. 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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Photograph 1. LTPP Section 16_1020 at Station 0+00 looking northbound in 2013.

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 1986 and was accepted into the LTPP Program as part of the GPS-1 experiment in July 1988. The pavement structure at the time of its incorporation into the LTPP program consisted of 0.2 inches of chip seal, 3.6 inches asphalt concrete, 12.3 inches of aggregate base, and 8.2 inches of subbase over a fine-grained subgrade soil. In an interview with staff, the Idaho Transportation Department (ITD) confirmed the application of a chip seal within one year of a pavement's construction was common practice at the time the test section was built. 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 June 2011, when the test section received a 2.4-inch AC overlay moving the test section to the GPS-6C: AC Overlay Using Modified Asphalt of AC Pavement-No Milling experiment. Table 2 summarizes the pavement structure following the overlay, which corresponds to CONSTRUCTION_NO = 4 (CN = 4). An additional construction event in July 2013 (CN=5), a chip seal, also resulted in 0.2-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 April 1993 (CN=2) and patching in August 2008 (CN=3). While ITD does have work planned on a nearby area of U.S. 93, it was confirmed that the test section was not included in the planned work, and was not scheduled to be rehabilitated or reconstructed in the immediate future

Table 1. Pavement structure for 16_1020 (CN=1)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	93.0	Fine-grained soils: silt
2	Unbound granular subbase	8.2	Soil-aggregate mixture
3	Unbound granular base	12.3	Crushed gravel
4	Asphalt concrete layer	3.6	Hot mixed, hot laid AC, dense graded
5	Asphalt concrete layer	0.2	Chip seal

Table 2. Pavement structure for 16_1020 (CN=4)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	93.0	Fine-grained soils: silt
2	Unbound granular subbase	8.2	Soil-aggregate mixture
3	Unbound granular base	12.3	Crushed gravel
4	Asphalt concrete layer	3.6	Hot mixed, hot laid AC, dense graded
5	Asphalt concrete layer	0.2	Chip seal
6	Asphalt concrete layer	2.4	Hot mixed, hot laid AC, dense graded

Table 3. Pavement structure for 16_1020 (CN=5)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	93.0	Fine-grained soils: silt
2	Unbound granular subbase	8.2	Soil-aggregate mixture
3	Unbound granular base	12.3	Crushed gravel
4	Asphalt concrete layer	3.6	Hot mixed, hot laid AC, dense graded
5	Asphalt concrete layer	0.2	Chip seal
6	Asphalt concrete layer	2.4	Hot mixed, hot laid AC, dense graded
7	Asphalt concrete layer	0.2	Chip seal

Pavement Structural Properties

Figure 1 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 Figure 1, the deflections reported increased from 15.3 mils (May 1989) to 16.1 mils (April 2011) prior to the overlay and from 11.9 mils (October 2011) to 11 mils (July 2013) following the overlay. The drop in deflection reported following the 2011 overlay is aligned with expectations; as the AC thickness increased, the deflections subsequently decreased. As shown in the figure, the fluctuations in the deflections reported prior to and following the overlay event appear to be affected by the pavement temperature at the time of testing. Generally, as the pavement temperature at the time of testing increased, the deflections reported increased. Notable exceptions to this trend included the deflections reported in August 1993, July 1997, and July 2004. However, despite the changes in temperature, the deflections remained fairly consistent overall. There was no clear relationship between the reported deflections and the 7-day cumulative precipitation at the test section, which is not surprising as little to no precipitation was reported on the test section prior to each FWD test date.

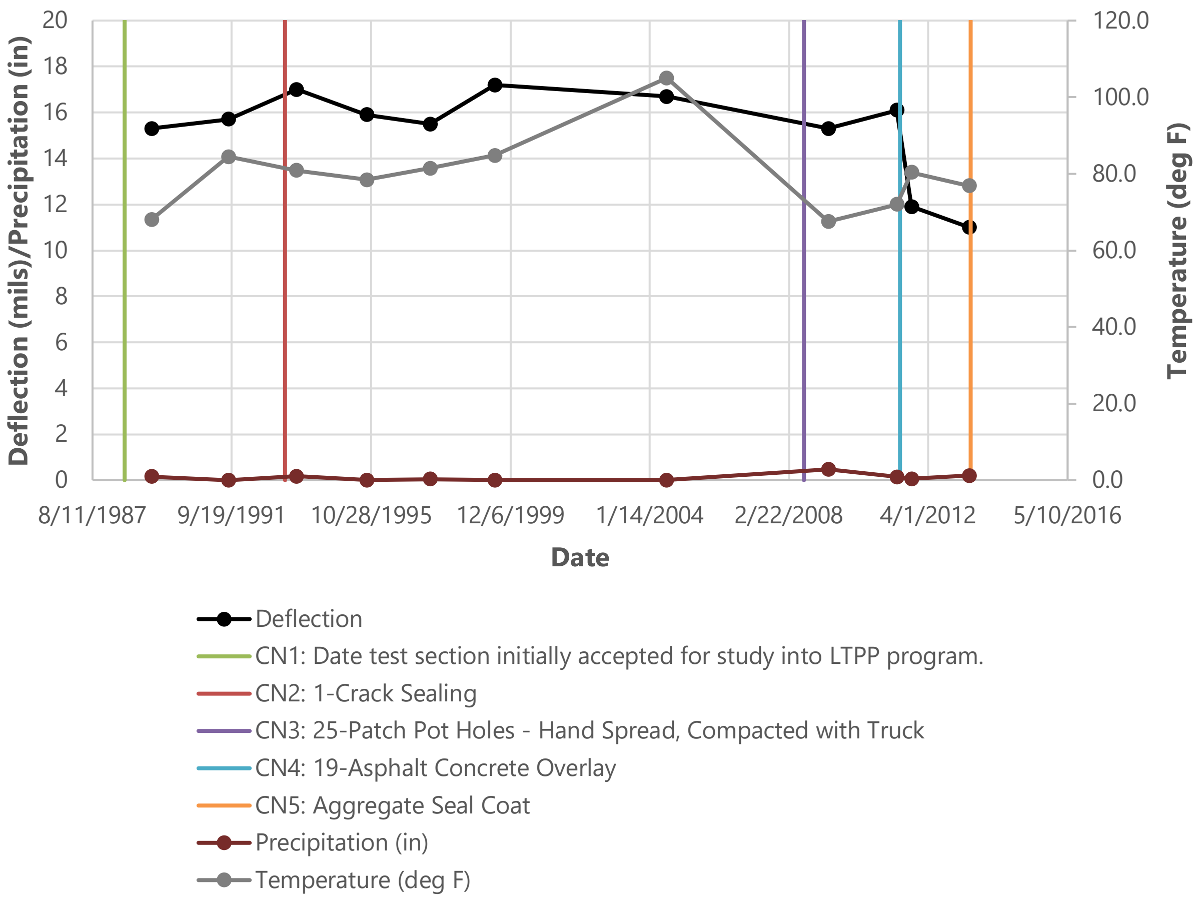


Figure 1. FWD deflections under the load plate over time with pavement temperature and the 7-day precipitation at test section.

The layer moduli backcalculated from the deflection data were also assessed for the test section. The pavement structure for test section 16_1020 was modeled as 3.6 inches of AC (original AC surface without the 0.2-inch chip seal),12.3 inches of typical granular base, and 8.2 inches of coarse granular subbase over subgrade (divided into two layers). While the test section received an AC overlay in 2011, the assumed backcalculation thicknesses did not change over time (likely due to a lag in capturing the rehabilitation event in the LTPP database compared to when the data was extracted for use by the analysis contractor), which likely impacted the accuracy of the moduli calculated for October 2011. For this reason, backcalculated data for October 2011 was removed from the analysis. Additionally, backcalculated moduli for FWD data collected after 2012 were not calculated, and therefore were not included in the LTPP database. Figure 2 through Figure 6 show the backcalculated moduli of Layers 1 through 5.

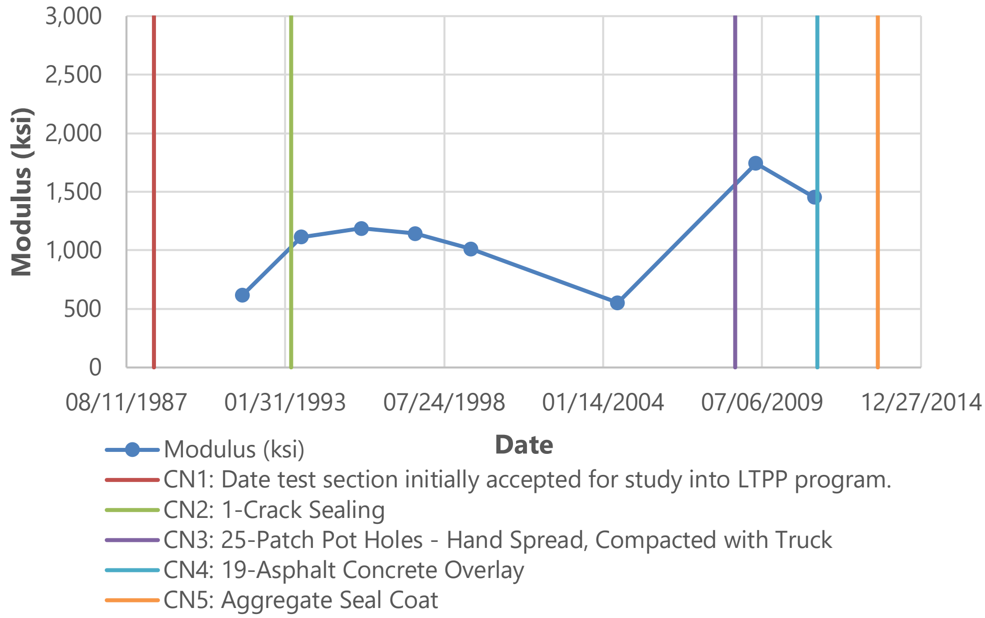


Figure 2. Average backcalculated modulus for AC (Layer 1).

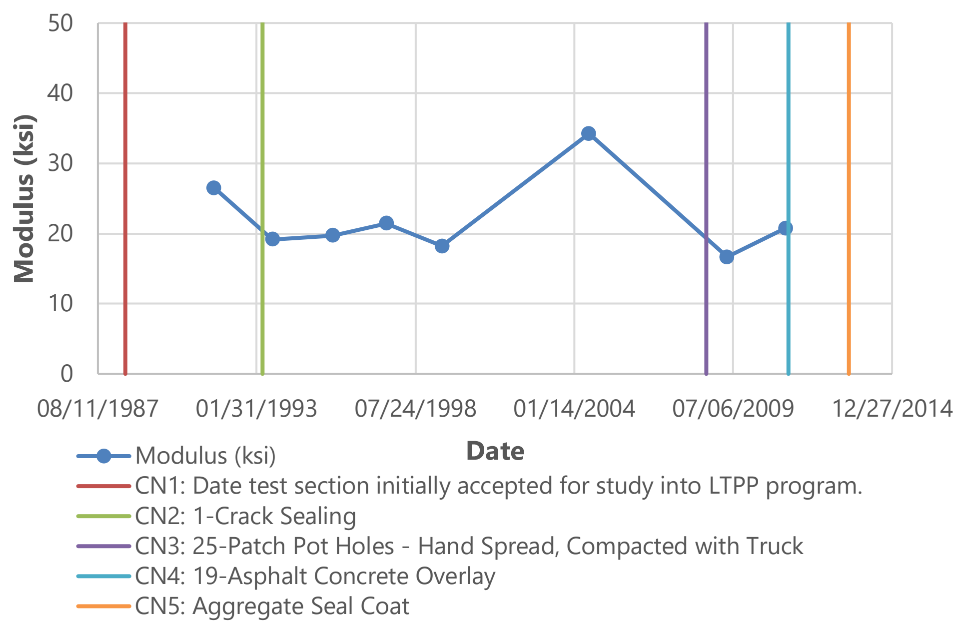


Figure 3. Average backcalculated modulus for base layer (Layer 2).

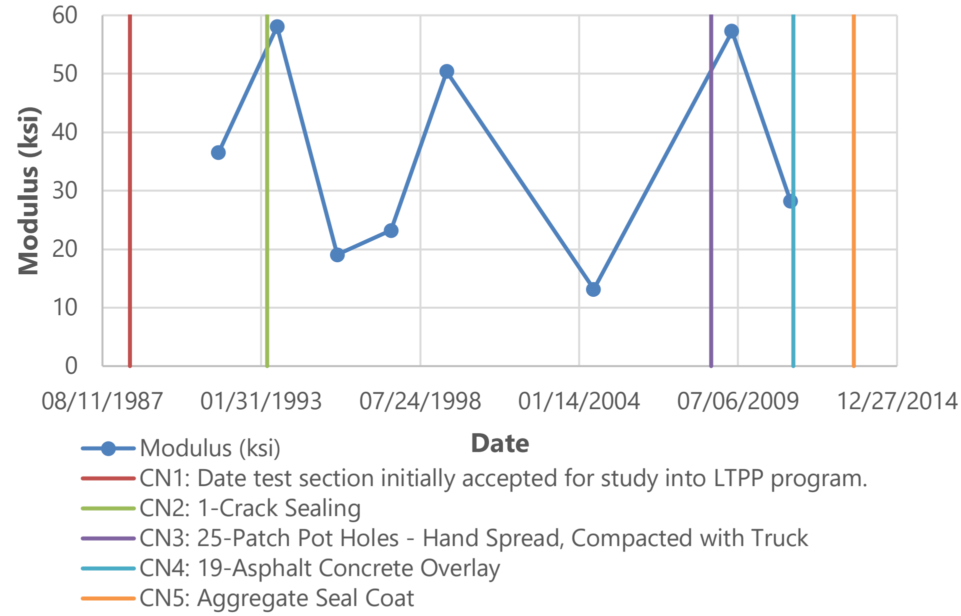


Figure 4. Average backcalculated modulus for subbase layer (Layer 3).

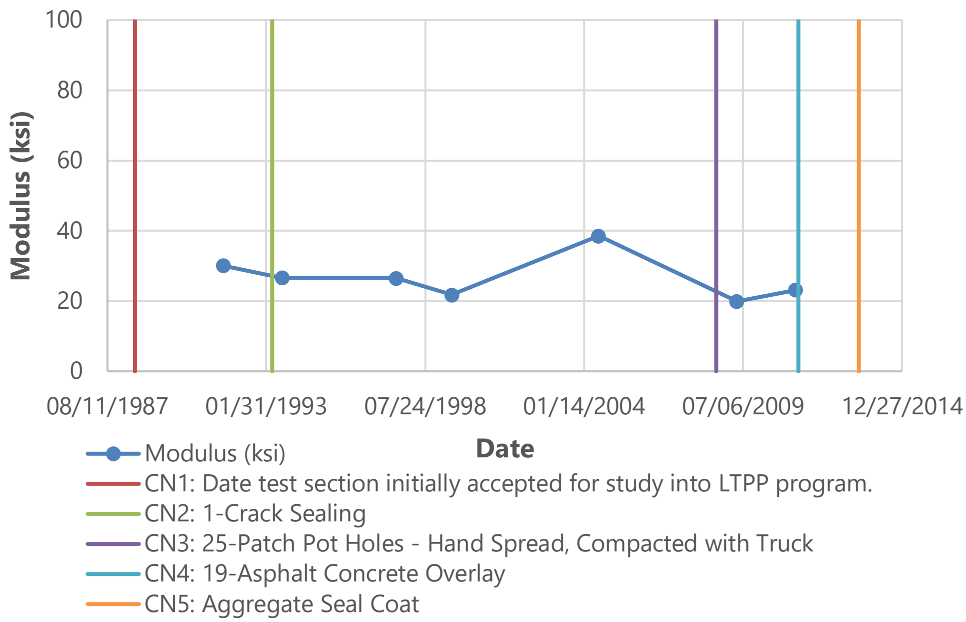


Figure 5. Average backcalculated modulus for the first 24-inches of subgrade (Layer 4).

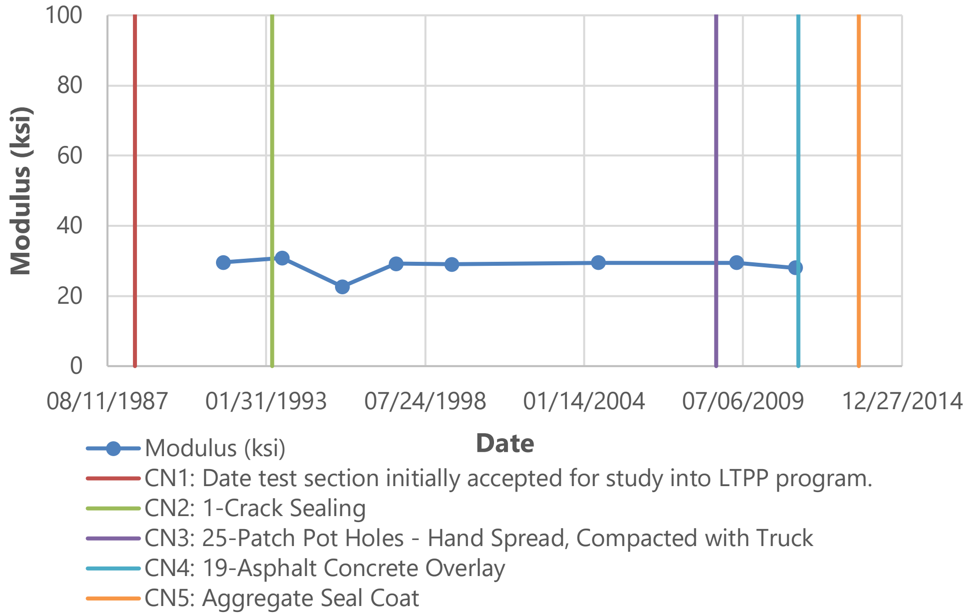


Figure 6. Average backcalculated modulus for remaining subgrade (Layer 5).

The backcalculated moduli reported for the AC layer (Layer 1) varied over time. Between 1991 and 2004, the reported modulus of the test section ranged from 553 ksi to 1,186 ksi. However, in 2009, the reported modulus value for Layer 1 spiked. The increase in modulus reported in 2009 was not explained by a change in the pavement structure. Instead, this deviation in the reported moduli values may be related to the compensating layer moduli effect; i.e., during backcalculation the modulus value for one layer went up so the value of another layer had to go down. In this case, in 2009, the reported modulus value for the base layer decreased while the modulus of the AC layer increased. Overall, the moduli values reported for the AC layer appear reasonable.

The reported backcalculated moduli values for the base layer (Layer 2) followed a similar trend to the AC layer. Between 1991 and 1999, the reported modulus was consistent, ranging from 18 ksi to 26 ksi. However, by 2004, the test section reported a peak in the average modulus of 34 ksi before decreasing and leveling out again. The range of values reported for Layers 3 and 4 greatly fluctuated over time with values from 13 to 58 ksi and 20 to 38 ksi, respectively. However, an outlier of 85 ksi in 1995 was removed from the plot and analysis of Layer 3. Layer 5, the remaining subgrade layer, was the most consistent over time, with modulus values ranging from 23 to 33 ksi. Overall, the moduli values of Layers 2 through 5 appear to have been affected by the way in which the backcalculation layers were created. While the pavement structure did have five layers at the time of construction, the use of five layers in the backcalculations appeared to underestimate the base modulus and overestimate the subbase modulus.

The reasonableness of the backcalculated layer moduli was to be compared to moduli derived from laboratory resilient modulus testing. However, resilient modulus testing was only performed on the subgrade of the test section as there were insufficient unbound base and subbase materials to perform the testing. The lab-reported modulus values for the subgrade ranged from 30-74 ksi which aligns with the reported backcalculated moduli.

Climate History

Figure 7 shows the time history of average annual precipitation (from MERRA-2) at the test section since 1986. The average annual precipitation observed over the analysis period fluctuated on a year-to-year basis with a notable increase in precipitation reported in 1995. However, overall, the reported average annual precipitation remained low at an average of 13.4 inches per year.

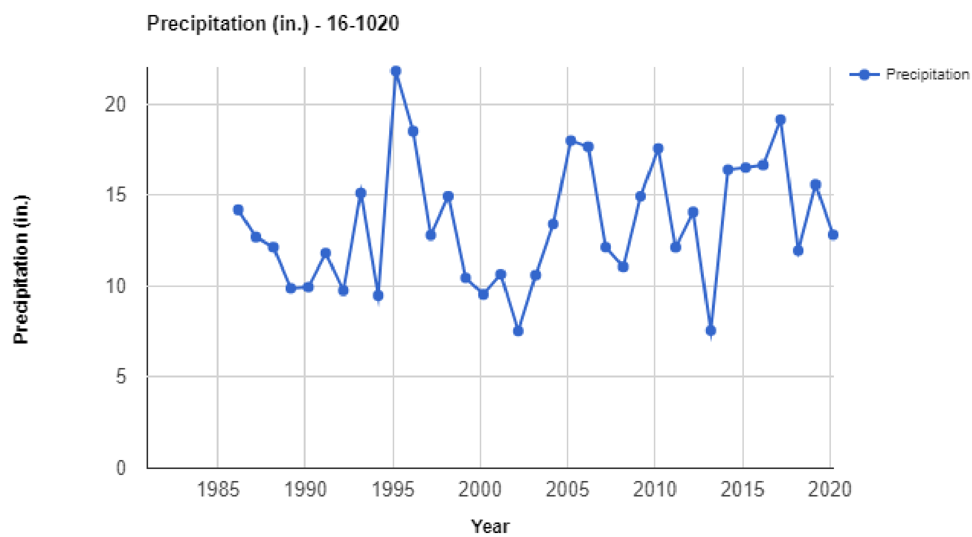


Figure 7. Average yearly precipitation over time.

Figure 8 shows the time history of the average annual freezing index (from MERRA-2) 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 178 deg F deg days (2003) to 635 deg F deg days (1990) during the analysis period. The overall trend of the freezing index fluctuated over time with an average freezing index of 406 deg F deg days. During an interview with ITD staff, it was noted that freeze-thaw should not have played a major role at this test section due to the subgrade not being susceptible to freeze-thaw changes.

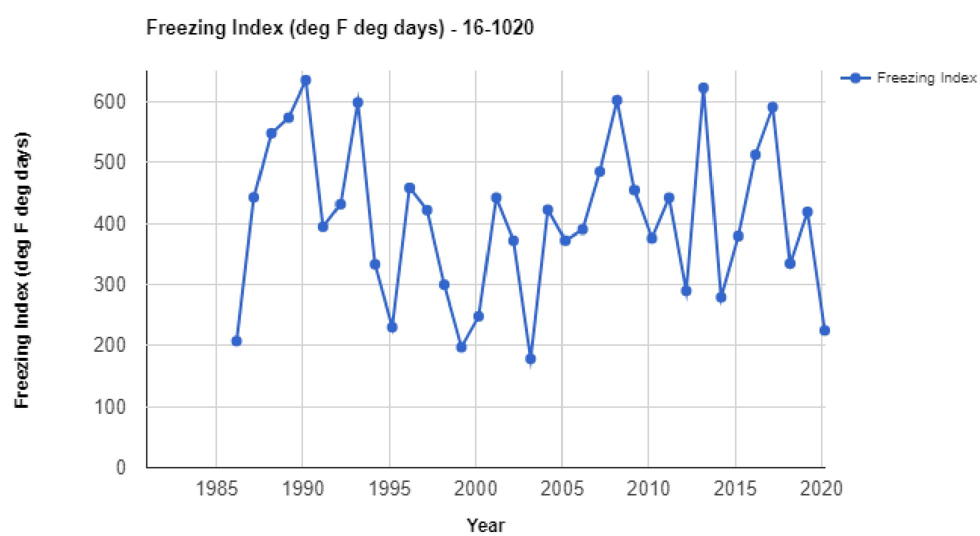


Figure 8. Average annual freezing index over time.

Truck Volume History

Figure 9 shows the annual average daily truck traffic (AADTT) data in the LTPP test lane by year. The average annual daily truck traffic increased from 175 in 1986 to 438 in 2009. The average number of ESALs reported on the section also increased over time as depicted in Figure 10. In 1986, 18,574 ESALS were reported annually on the test section whereas by 2009, the average annual ESALs increased to 139,087. While the figures that follow only report the values associated with historical, state provided, and monitored values, additional traffic data were reported in the LTPP database. These data, which was calculated using a linear growth function (2004-2008 and 2010-2017), was not included in the analysis because it appeared to underestimate the traffic experienced on this test section over time. Based on an interview with ITD staff familiar with this test section, traffic on the test section has increased significantly since the early 2000s due to an increase in agricultural and construction-related truck traffic in the area. Additional data, provided by the ITD, confirmed the accuracy of the AADTT reported in 2009; the data showed the initial two-way ADDTT on the test section was 871. The discrepancy between the non-monitored traffic reported (which was approximately ¼ of the actual AADTT) in the LTPP database and the monitored traffic counts is significant finding of this study and will submitted as a Data Analysis/Operations Feedback Report (DAOFR) to be further addressed by the LTPP program. Overall, in spite of the sharp increase in traffic loading on this test section, the test section performed well as will be discussed in the sections to follow.

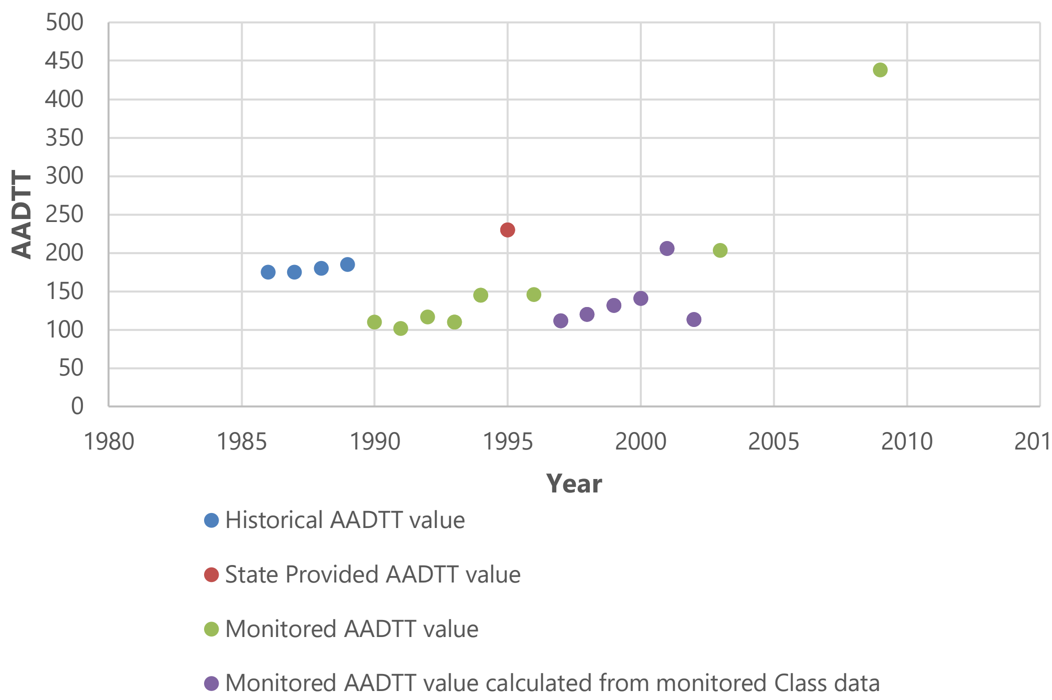


Figure 9. Average annual daily truck traffic (AADTT) history.

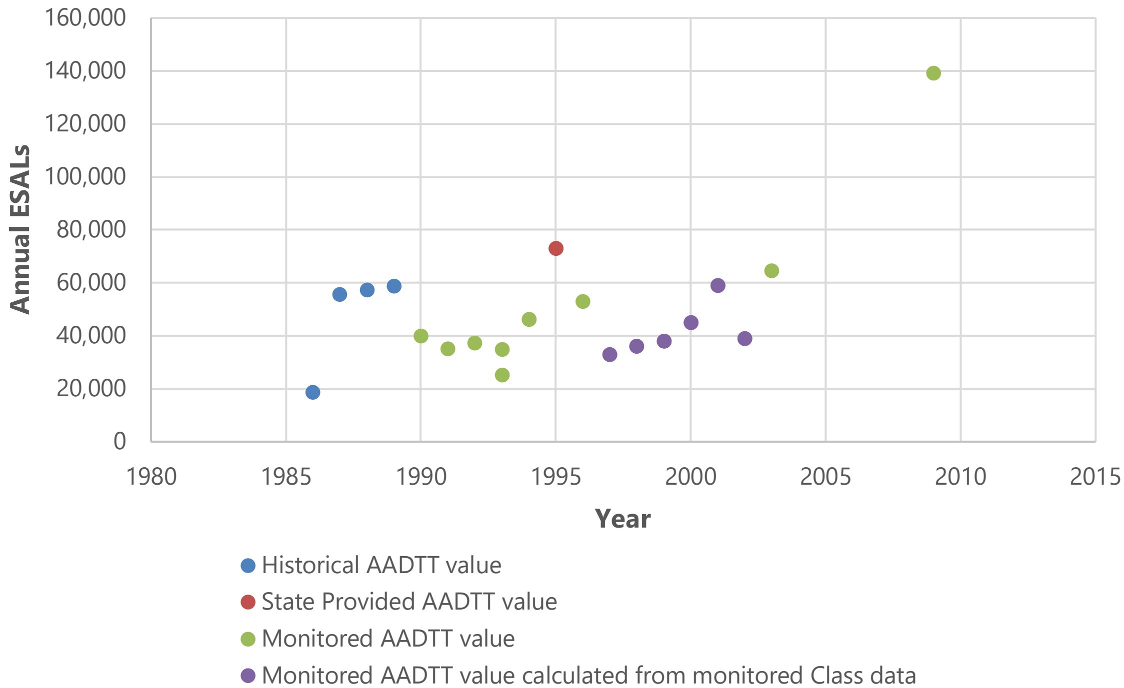


Figure 10. Estimated annual ESALs for vehicle classes 4-13 over time.

Pavement Distress History

The following summarizes the distresses observed on the test section between 1990 and the last manual distress survey of the test section in August 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 1990 and 2020. Fatigue/alligator cracking was first reported during the manual distress survey in October 2002, 16 years after the construction of the roadway, when 17.2 ft^2 was observed. Between 2002 and 2011, fatigue/alligator cracking increased at a rate of 6.4 ft^2/year, reaching 75 ft^2 of fatigue/alligator cracking by 2011. Following the overlay event in 2011, fatigue/alligator cracking was not reported again until the manual distress survey conducted in 2020, 9 years after the overlay, when 79 ft^2 of fatigue/alligator cracking was observed on the section or slightly more than pre-overlay conditions. Both prior to and following the overlay, the reported fatigue/alligator cracking was predominantly in the wheelpath. Despite the age of the test section, only small quantities and predominantly low severity fatigue cracking was reported on the section. It is hypothesized that the delay in the development of fatigue cracking is related to the minimal precipitation and traffic loading at the section in the early years of the test section's life. The development of the fatigue cracking may also have been related to issues with tack coats between the AC layers. Following the overlay, the reported fatigue/alligator cracking is likely a result of the increase in truck traffic along the test section.

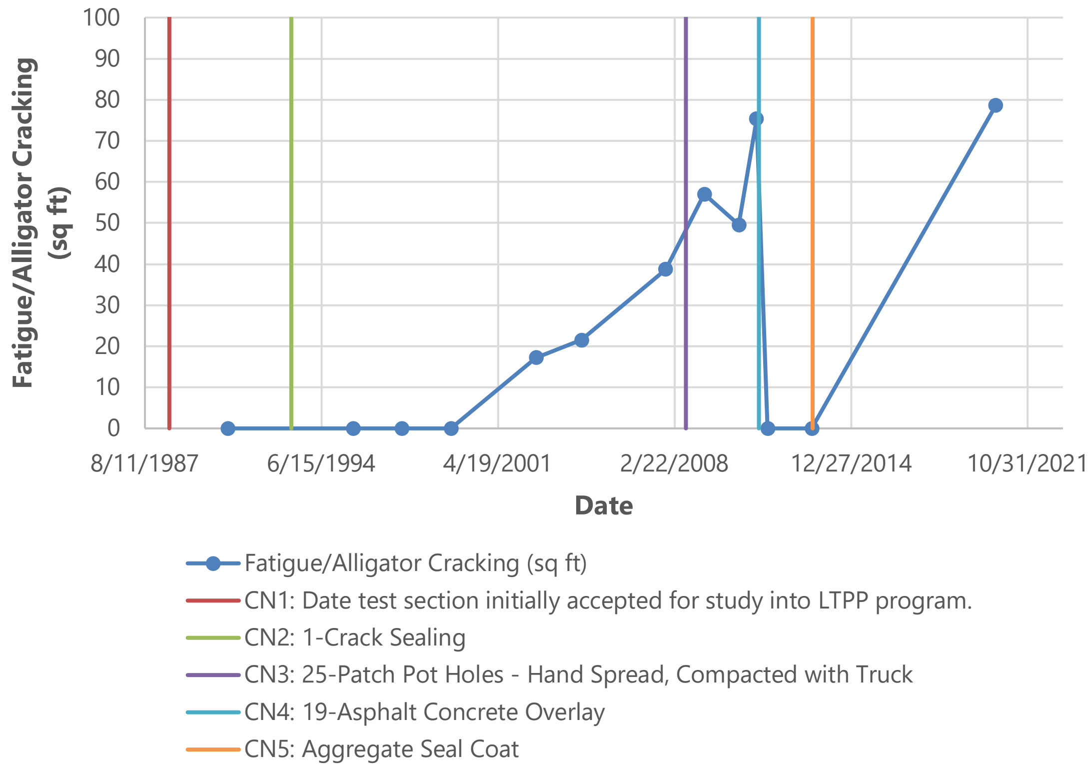


Figure 11. Time history of the length of fatigue cracking.

Longitudinal Cracking

Minimal non-wheel path (NWP) longitudinal cracking, depicted in Figure 12, was reported during the first manual distress survey in July 1997, 11 years after the construction of the roadway, when 2.6 ft of low severity cracking was observed. However, between 1999 and 2004, NWP longitudinal cracking was not reported on the test section. Instead, this location of cracking was rated as fatigue cracking. In 2007, 2.6 ft of low severity cracking was reported again (although in a different location), and the NWP longitudinal cracking continued to increase between 2007 and 2011. By April 2011, 29 ft of NWP longitudinal cracking was reported on the test section. Following the overlay event in late 2011, NWP longitudinal cracking was not reported again. All NWP longitudinal cracking in 2011 was observed on the right edge of the test section lane in multiple locations along the test section. There was no wheel path (WP) longitudinal cracking observed on the test section throughout the analysis period.

Transverse Cracking

Data on transverse cracking was collected between 1990 and 2020, as shown in Figure 13 and Figure 14. Transverse cracking was first reported during the manual distress survey in October 2007, 21 years after the construction of the roadway, when 6 ft of transverse cracking (3 cracks) was observed. The transverse cracking continued to increase between 2007 and April 2011, at a rate of 17.8 feet/year, reaching 77 feet (25 cracks) by 2011. The reported transverse cracking was predominantly of low severity. Following the overlay event in 2011, transverse cracking was not observed again until the August 2020 manual distress survey, nine years after the construction event, when 9 feet (3 cracks) of transverse cracking was reported. The transverse cracking reported on the test section, while minimal, was predominantly partial width cracking rather than the full width of the lane. It is hypothesized the transverse cracking observed was related to thermal expansion or the rumble strips in the shoulder of the section. However, during an interview, ITD noted rumble strips were not a typical cause of transverse cracking and were likely a nonfactor in the observed performance. Regardless, given the age of the pavement, the test section performed well in terms of transverse cracking.

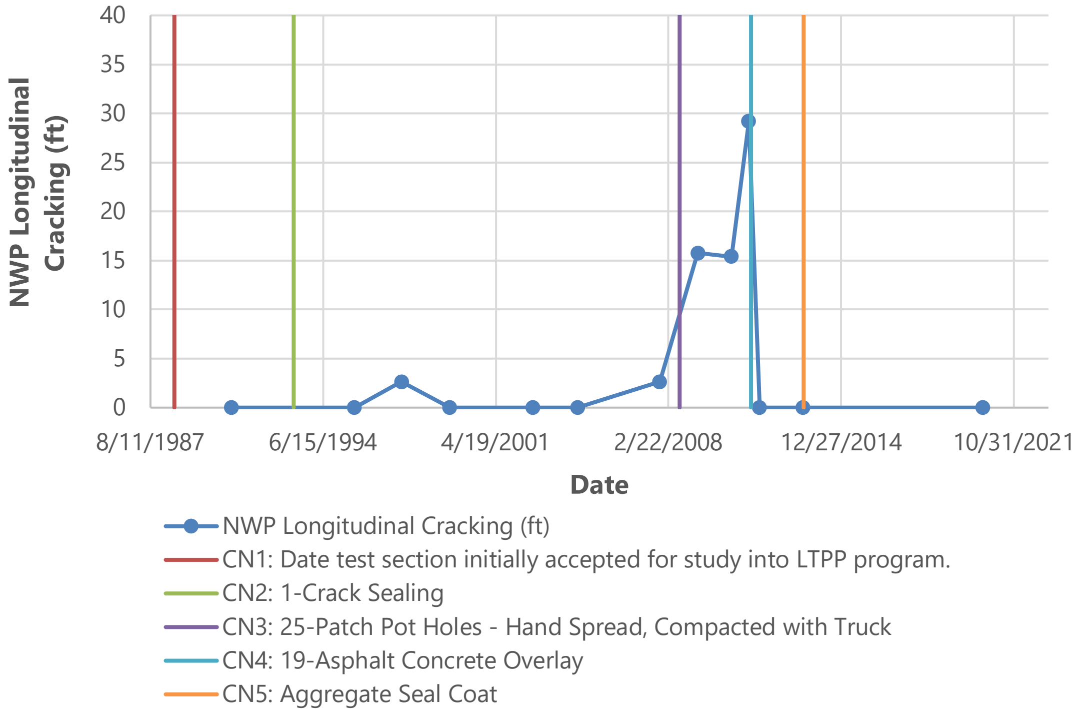


Figure 12. Time history of the length of NWP longitudinal cracks.

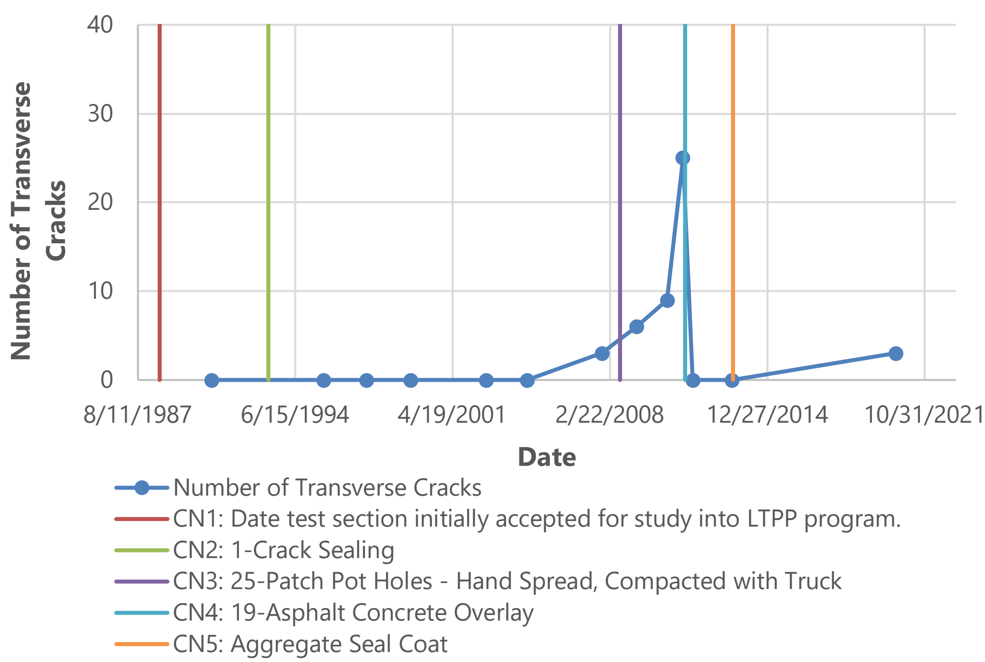


Figure 13. Time history of the number of transverse cracks.

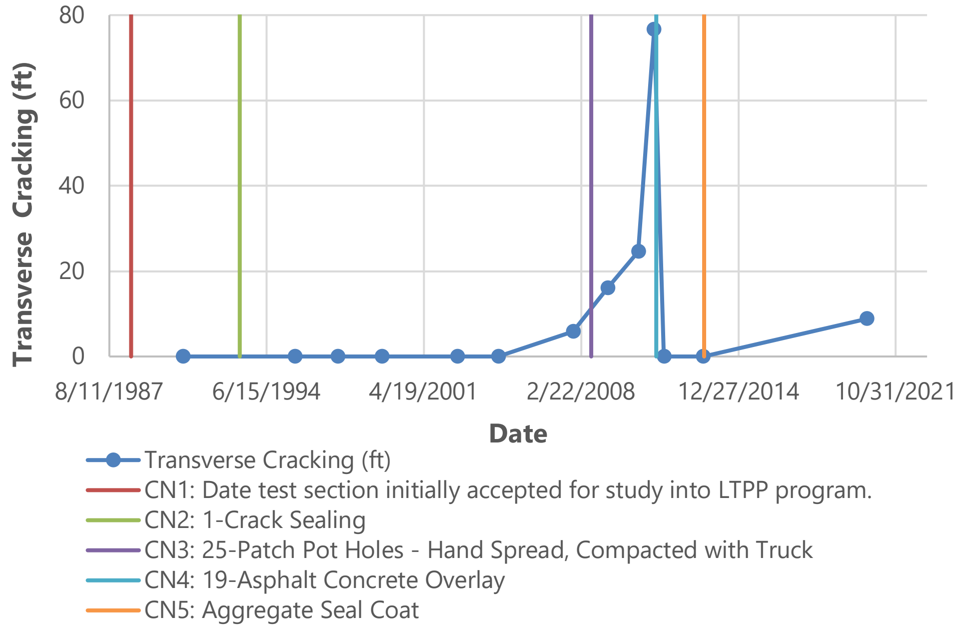


Figure 14. Time history of the length of transverse cracking.

IRI

The average IRI measurements for the test section over time are shown in Figure 15. During the first performance period of the test section, from its incorporation into the LTPP program to the overlay event in 2011, the IRI on the test section slightly increased over time from 43.4 in/mile in 1989 to 54 in/mile in 2010. Throughout this period, the performance of the pavement was classified as "Good" based on FHWA performance definitions. During the second performance period, after the 2011 overlay event, the IRI of the test section dropped to 37 in/mile in October 2011. With the application of an aggregate seal coat in 2013, the IRI again increased to 45.1 in/mile in 2014 before slightly decreasing to an average IRI of 40.7 in/mile in 2020. Again, the average IRI during this performance period was classified as "Good" based on FHWA performance definitions.

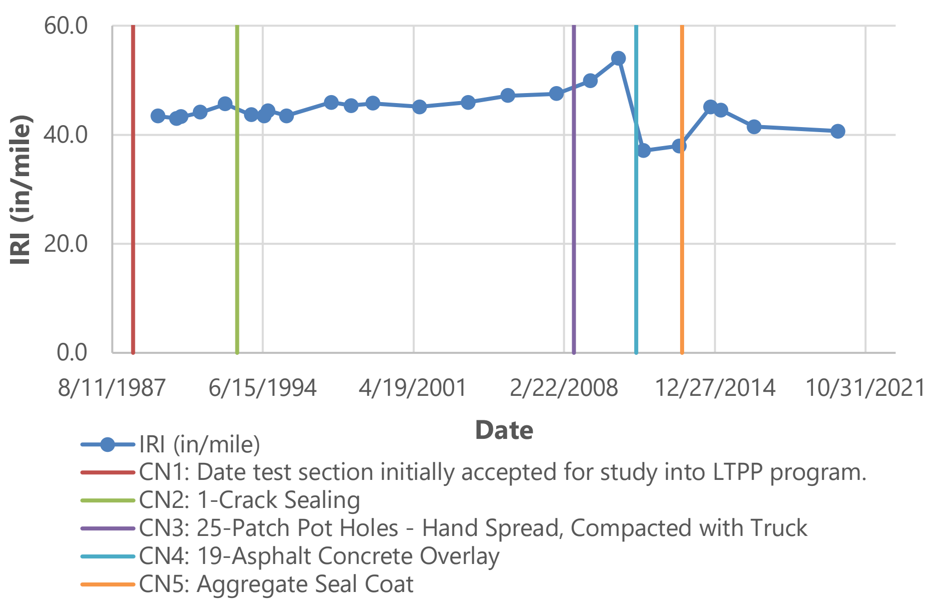


Figure 15. Time history plot of pavement roughness.

Rutting

The average rut depths observed for the test section between 1989 and 2020 are shown in Figure 16. The rutting on the section prior to in 2011 increased from 0.2 in in 1989 to 0.35 in in 2011. Following the overlay in 2011, the average rut depth dropped to 0.04 in. The average rut depth began to slightly increase following the overlay, at a rate of less than 0.01 in/year between 2011 and 2020.

Due to the amount of rutting observed on the test section prior to the overlay, the change in the transverse profile of the test section was 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.[2] 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.

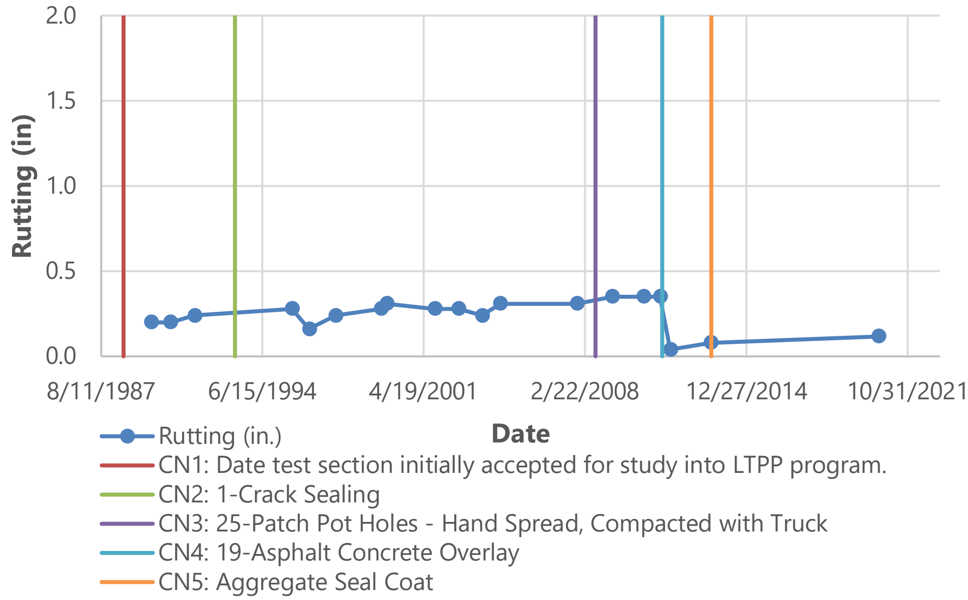


Figure 16. Time history plot of average rut depth.

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 17. 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 18.

[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.

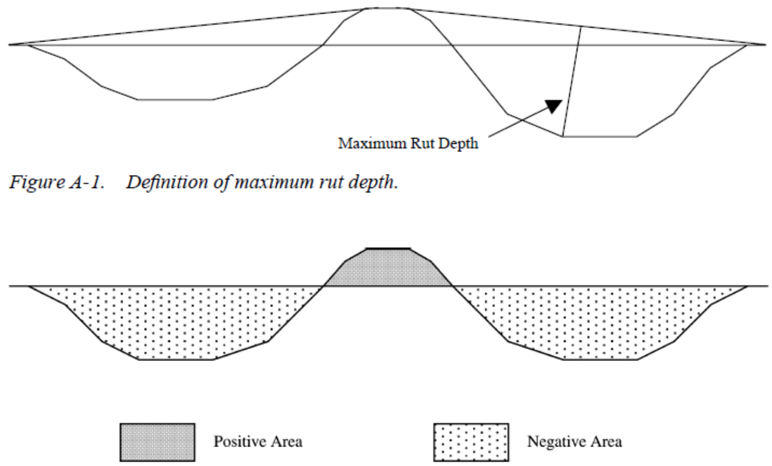
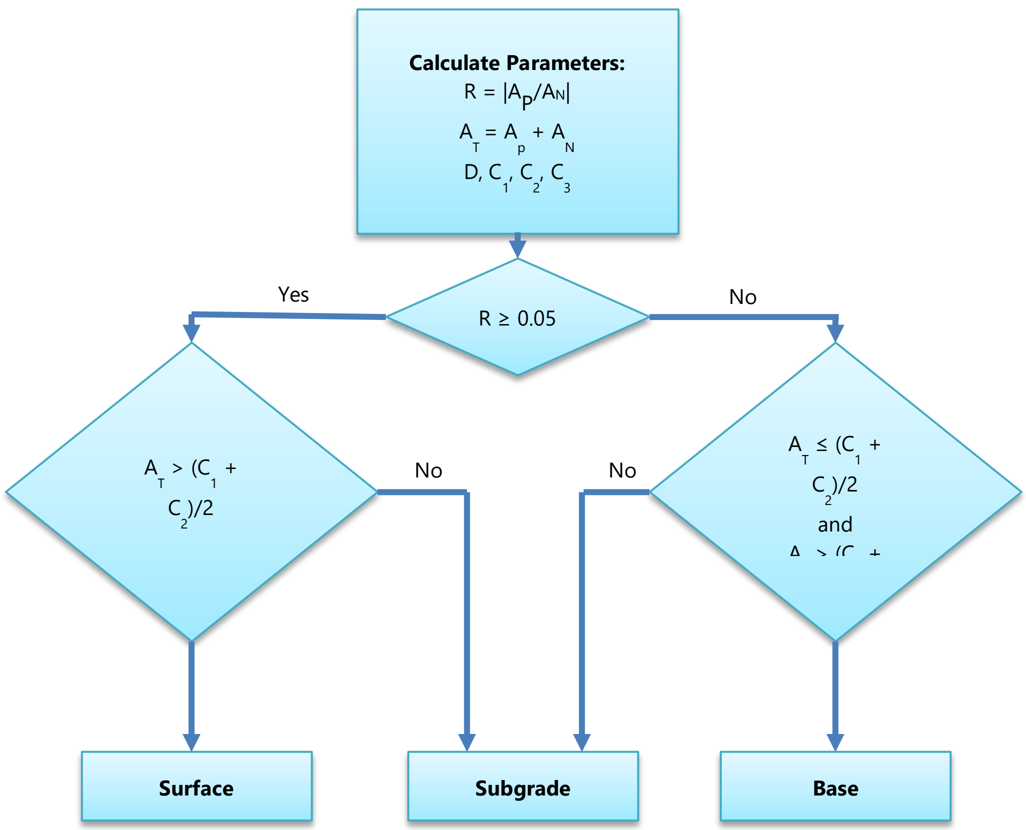


Figure 17. Transverse profile maximum rut depth and positive and negative areas (White et al., 2002)



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

Based on the analysis conducted for each of the transverse profiles of the test section (profiles spaced at 50 ft) for the collection dates between September 1989 and June 2013, the predominant lowest layer contributing to rutting was calculated for each date of collection at multiple locations along the section. For most years, the predominant layer contributing to rutting was the surface layer as shown in Table 4. Based on an interview with ITD staff, it is hypothesized that the rutting observed prior to 1990 may have been related to the AC thickness, annual pavement temperature, the binder used, and to a lesser extent, the increase in agricultural and constructed-related traffic along the roadway. Additionally, while there may have been some studded tires used on this road, studded tire and chain abrasion in the winter months were not considered as significantly impacting rut measurements.

Table 4. Predicted lowest layer contributing to rutting.

Survey Date	Number of transverse profiles where the following were estimated as the lowest layer contributing to rutting		
	Subgrade	Base	Surface
09/20/1989	-	7	3
07/19/1990	-	-	11
07/26/1991	-	-	11
09/13/1995	-	-	11
06/05/1996	-	-	11
07/24/1997	-	-	11
06/22/1999	-	-	11
09/25/1999	-	-	11
10/05/2001	-	-	11
10/10/2002	-	-	11
10/08/2003	-	-	11
07/16/2004	-	-	11
10/18/2007	-	2	9
04/21/2009	-	2	9
08/20/2010	-	3	8
04/28/2011	-	1	10
10/03/2011	-	-	11
06/19/2013	-	-	11

The rutting analysis conducted as a part of this desktop study was of particular interest to ITD. While the methodology used is not a definitive means of understanding rutting, it does help hypothesize the layer causing the rutting observed. Moving forward, the method could be adapted for use by ITD to get a network-level overview of the rutting observed on their pavement network.

SUMMARY OF FINDINGS

LTPP test section 16_1020 is located on U.S. Route 93, northbound, in Jerome County, Idaho. U.S. Route 93 is a rural principal arterial with one lane in the direction of traffic. The test section was constructed in 1986 and was accepted into the LTPP Program as part of the GPS-1 experiment in July 1988. The pavement structure at the time of its incorporation into the LTPP program consisted of 0.2 inches of chip seal, 3.6 inches asphalt concrete, 12.3 inches of aggregate base, and 8.2 inches of subbase over a fine-grained subgrade soil. The next major construction event occurred in June 2011, when the test section received a 2.4-inch AC overlay moving the test section to the GPS-6C: AC Overlay Using Modified Asphalt of AC Pavement-No Milling experiment. An additional construction event in July 2013 (CN=5), a chip seal, also resulted in 0.2-inch increase to the pavement structure. Other minor construction events that occurred on the test section included crack sealing in April 1993 (CN=2) and patching in August 2008 (CN=3). This memorandum was focused on understanding the good performance in terms of cracking and IRI as well as the cause of the high levels of rutting observed on the test section prior to the 2011 overlay. Specifically, it focused on:

1. **The key reasons for the relatively good performance of the test section.** The test section performed well in terms of cracking and IRI. While some cracking was observed on the test section, overall, the cracking observed was minimal, low severity, and slow to develop. It is hypothesized that the good performance of the test section is related to the pavement design and the lack of extreme environmental conditions.
2. **The key cause(s) of rutting at this test section.** 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. Based on the analysis conducted for each of the transverse profiles of the test section for the collection dates between September 1989 and June 2013, the predominant layer contributing to rutting was the surface layer. It was hypothesized that the rutting observed prior to 1990 may have been related to the AC thickness, annual pavement temperature, the binder used, and to a lesser extent, the increase in agricultural and constructed-related traffic along the roadway. Additionally, while there may have been some studded tires used on this road, studded tire and chain abrasion in the winter months were not considered as significantly impacting rut measurements.

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 August 2020. Therefore, the section is anticipated to officially go out of study soon. Because of this, the following activities are recommended to be conducted:

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. Determine if any of the cracking observed following the overlay event were reflected from the original AC surface.
    c. Identify the rutting failure layer via visual inspection.
2. Analysis of 2020 TSD data, collected at the direction of ITD, including:
    a. Computation of layer moduli.
    b. Computation of $SN_{eff.}$

c. Assessment of overlay thickness.

d. Calculation of remaining life.

Additionally, as discussed previously, the difference between the monitored and projected LTPP-reported truck traffic on the test section was significant. Therefore, a DAOFR will be submitted to the LTPP program to further investigate and correct this issue.