Technical Memorandum

To: Kim Schofield

From: Lauren Gardner, Gonzalo Rada, and Kevin Senn

cc: Mustafa Mohamedali

Date: June 1, 2021

Re. Forensic Desktop Study Report: Washington State LTPP SPS-8 Test Sections

The Long-Term Pavement Performance (LTPP) Specific Pavement Studies-8 (SPS-8) Study of Environmental Effects in the Absence of Heavy Loads test sections 53_0801, 53_0802, 53_A809 and 53_A810 [1] were nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." Test sections 53_0801 and 53_0802 are asphalt concrete (AC) pavements located in Columbia County, Washington while test sections 53_A809 and 53_A810 are jointed plain concrete pavements (JPCP) located in Walla Walla County, Washington. Although the pavement structures vary in terms of layer material types and thicknesses, they are exposed to similar truck traffic (very little) and climatic conditions. Therefore, these test sections provide an opportunity to assess the effects of varying layer material types and thicknesses on the performance of pavements in the absence of heavy loads and similar climatic conditions. The desktop study is intended to assess and compare the performance of the test sections with a focus on the differences in pavement deflections, IRI, and pavement surface distresses over time – recognizing the AC and PCC pavement test sections were independent projects constructed five years apart.

SITE DESCRIPTIONS

LTPP test sections 53_0801 and 53_0802 are located on North Touchet Rd, northbound, in Columbia County, Washington State, while test sections 53_A809 and 53_A810 are located on Smith Springs Rd, eastbound, in Walla Walla County. All four test sections are located on rural local collectors in a Dry No-Freeze climate. The coordinates (in degrees) of test sections 53_0801, 53_0802, 53_A809, and 53_A810 are (46.2707, -117.88281), (46.27224, -117.88609), (46.41105, -118.43233), and (46.41101, -118.42958), respectively. Photograph 1 and 2 shows test sections 53_0801 and 53_A809 at Station 0+00 in 2018, while Map 1 shows the geographical location of the test sections.

[1] First two digits in test section number represent the State Code [53 = Washington]. For LTPP Specific Pavement Studies (SPS) test sections, the second set of two numbers indicates the Project Code (e.g., 08/A8= SPS-8), and the final set of two numbers represents the test section number on that project (e.g., 01).

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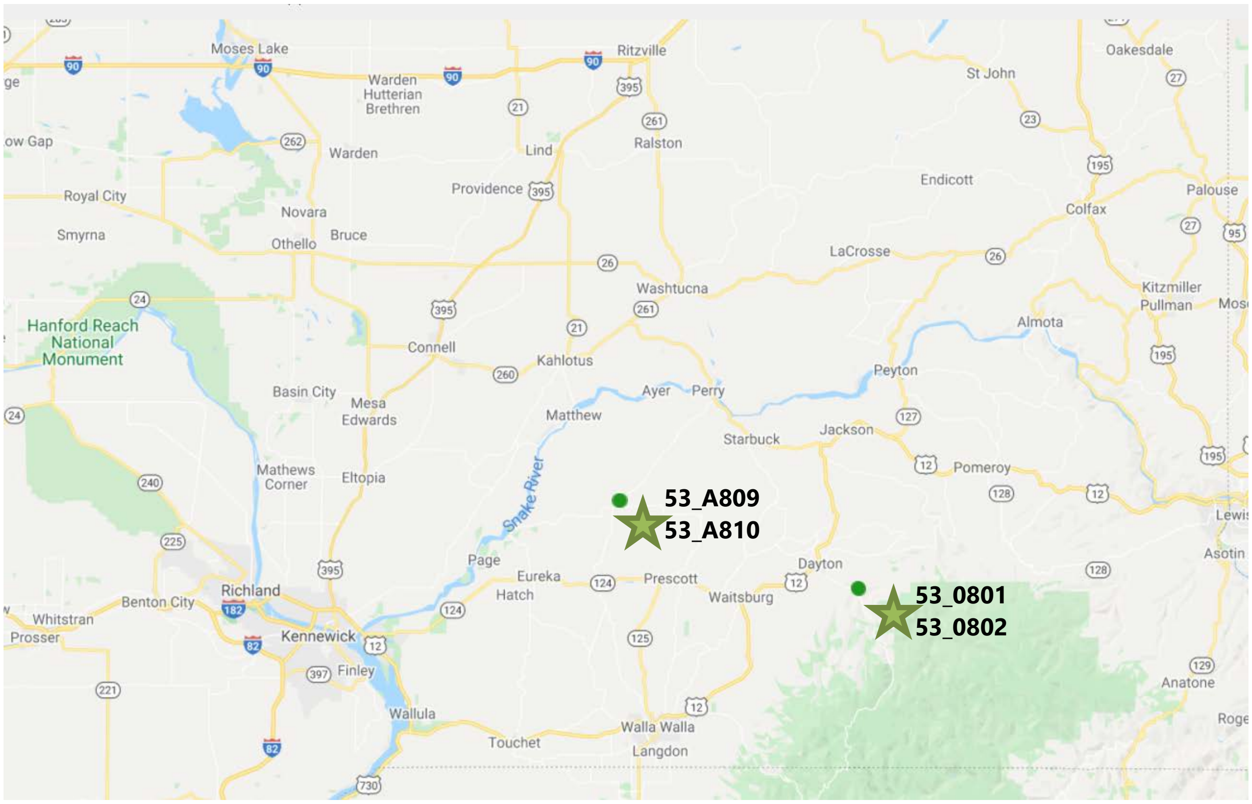
Photograph 1. LTPP Section 53_0801 at Station 0+00 looking northbound in 2018.

Photograph 2. LTPP Section 53_A809 at Station 0+00 looking eastbound in 2018.



Map 1. Geographical location of test sections.

BASELINE PAVEMENT HISTORY

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

Pavement Structure and Construction History

The four test sections were constructed as part of the SPS-8 Study of Environmental Effects in the Absence of Heavy Loads experiment; this experiment was developed to better understand the effect of varying pavement structures (in terms of surface and base thicknesses, specifically) in the absence of heavy loading. SPS-8 AC test sections were to be designed with a surface thickness of 4 inches and a base thickness of 6 inches and a surface thickness of 7 inches and a base thickness of 12 inches, the same design used for SPS-1 Strategic Study of Structural Factors for Flexible Pavements test sections. For rigid test sections included in the SPS-8 experiment, test sections were designed to have an 8-inch and an 11-inch surface over a 6-inch base, similar to the design of SPS-2 Strategic Study of Structural Factors for Rigid Pavements test sections. In Washington State, two SPS-8 projects were conducted—one consisting of two AC test sections—53_0801 and 53_0802— constructed in 1995 and the other consisting of two jointed plain concrete pavement (JPCP) test sections—53_A809 and 05_A810—constructed in 2000.

At the time of construction in 1995, test section 53_0801 consisted of 3.7 inches of dense-graded asphalt concrete (0.3-inch less than the specified design thickness) and 8 inches of unbound granular base (2 inches greater than the specified design thickness) over 38.4 inches of unbound subbase and a fine-grained subgrade soil. Test section 53_0802, also constructed in 1995, consisted of 6.8 inches of dense-graded asphalt concrete (0.2-inch less than the specified design thickness) and 11.7 inches of unbound granular base (0.3-inch less than the specified design thickness) over 38.4 inches of granular subbase and a coarse-grained subgrade soil. Both test sections were crack sealed in 2000 (CN=2), 2003 (CN=3), and in

June and October of 2015 (CN=6 and CN=7 for 53_0801 and CN=5 and CN=6 for 53_0802). Test section 53_0801 also received 0.3-inch chip seal in 2005 (CN=4) and a 0.1-inch fog seal in 2011 (CN=5) while test section 53_0802 only received a 0.3-inch chip seal in 2005 (CN=4). These pavement structures are summarized in Table 1 and correspond to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. While the test sections have not received maintenance recently despite increases in distress, no additional maintenance events are planned within the next year or two.

Table 1. Pavement structure for test sections 53_0802 and 53_0802 (CN=1)

Layer Number	Layer Type	Test Section 53_0801		Test Section 53_0802	
		Thickness (in.)	Material Code Description	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Sandy Lean Clay		Coarse Grained Soil: Silty Sand
2	Unbound (granular) Subbase	36	Crushed Stone	36	Crushed Stone
3	Unbound (granular) Subbase	2.4	Crushed Stone	2.4	Crushed Stone
4	Unbound (granular) Base	8	Crushed Stone	11.7	Crushed Stone
5	Asphalt Concrete Layer	3.7	Hot Mixed, Hot Laid AC, Dense Graded	6.8	Hot Mixed, Hot Laid AC, Dense Graded

Notably, the subgrade material classification of the two test sections differed, with one subgrade classified as a fine-grained and the other coarse-grained. Table 2 confirms these differences in the material properties of the two subgrades; however, neither of the two test sections meet the 35% passing or less on the No. 200 sieve used to classify coarse-grained soils using the AASHTO Soil Classification System.[2] Both original subgrade materials were also considered active clays and prone to swelling. However, due to abnormally high amounts of precipitation and excessive pumping in the year of construction, the subgrades at both test sections were excavated a minimum of three feet and replaced with shot-rock and a thin layer of borrow material. Washington State DOT staff familiar with this area confirmed that the LTPP-defined subgrade types for these test sections aligned with the typical subgrades in the area. It was also noted that the subgrades within the area typically contain a lot of larger stones.

Test sections 53_A809 and 53_A810 were both constructed in 2000 as four layers: 8.5 and 10.9 inches of Portland Cement Concrete (PCC) (0.5-inch greater than and 0.1-inch less than the specified design thicknesses, respectively) and 4.5 and 4.7 inches of unbound granular base (1.5 and 1.3 inches less than the specified design thickness, respectively) over an unbound granular subbase and fine-grained subgrade soil for test sections 05_A809 and 05_A810, respectively. Neither section reported any additional construction

[2]American Association of State Highway and Transportation Officials. (1991). *AASHTO M 145, 1991 Edition - Standard Specification for Classification of Soils and Soil–Aggregate Mixtures for Highway Construction Purposes*.

events following their incorporation into the LTPP program. Table 3 summarizes the pavement structure of the two JPCP test sections when they were first incorporated into the LTPP program (CN=1).

Table 2. Material properties of test sections 53_0801 and 53_0802

Property	Test Section 53_0801	Test Section 53_0802
No. 4 Passing	82	66
No. 200 Passing	61	42
Liquid Limit	31	-
Plastic Limit	25	-
Plasticity Index	6	Non-Plastic

Table 3. Pavement structure for test sections 53_A809 and 53_A810 (CN=1).

Layer Number	Layer Type	Test Section 53_A809		Test Section 53_A810	
		Thickness (in.)	Material Code Description	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		Fine-Grained Soils: Sandy Silt		Fine-Grained Soils: Sandy Silt
2	Unbound (granular) Subbase	90.8	Fine-Grained Soils: Silt with Sand	35.8	Fine-Grained Soils: Sandy Silt
3	Unbound (granular) Base	4.5	Crushed Stone	4.7	Crushed Stone
4	Portland Cement Concrete Layer	8.5	Portland Cement Concrete (JPCP)	10.9	Portland Cement Concrete (JPCP)

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 for the JPCP test sections were lower than those reported for the AC test sections, as expected. Test section 53_A810, the thicker JPCP test section, reported the lowest deflection values, which ranged from 2.4 to 3.9 mils. The other JPCP test section, test section 53_A809, reported the second lowest deflection values, which ranged from 3.6 to 6.1 mils. Of the two AC test sections, the test section with the thicker pavement structure, 53_0802, reported the third lowest deflections over time, ranging from 6.8 to 10.2 mils, while test section 53_0801 reported the highest deflections, ranging from 10.2 to 14.4 mils. For all four test sections, the reported deflections were relatively constant over time, indicating structurally sound pavement structures.

The layer moduli backcalculated from the deflection data were also assessed for the test sections as depicted in Figure 2 through Figure 5. The pavement structure for test section 53_0801 was modeled as 3.7 inches of AC, 10.4 inches of base, and 36 inches of granular subbase over a fine-grained subgrade and bedrock. Test section 53_0802 was modeled as 6.8 inches of AC, 14.1 inches of base, and 36 inches of granular subbase over a coarse-grained subgrade and bedrock. Test section 53_A809 were modeled 8.5 inches of PCC, 4.5 inches of typical granular base, and 91 inches of granular subbase over a fine-grained subgrade and bedrock. Finally, test section 53_A810 was modeled as 10.9 inches of PCC, 4.7 inches of typical granular base, and 36 inches of granular subbase over a fine-grained subgrade and bedrock. All four test sections were modeled as five layers, where the assumed value of the bedrock layer, Layer 5, was 500 ksi. Backcalculated moduli for FWD data collected after 2012 were not calculated and therefore, not included in the LTPP database.

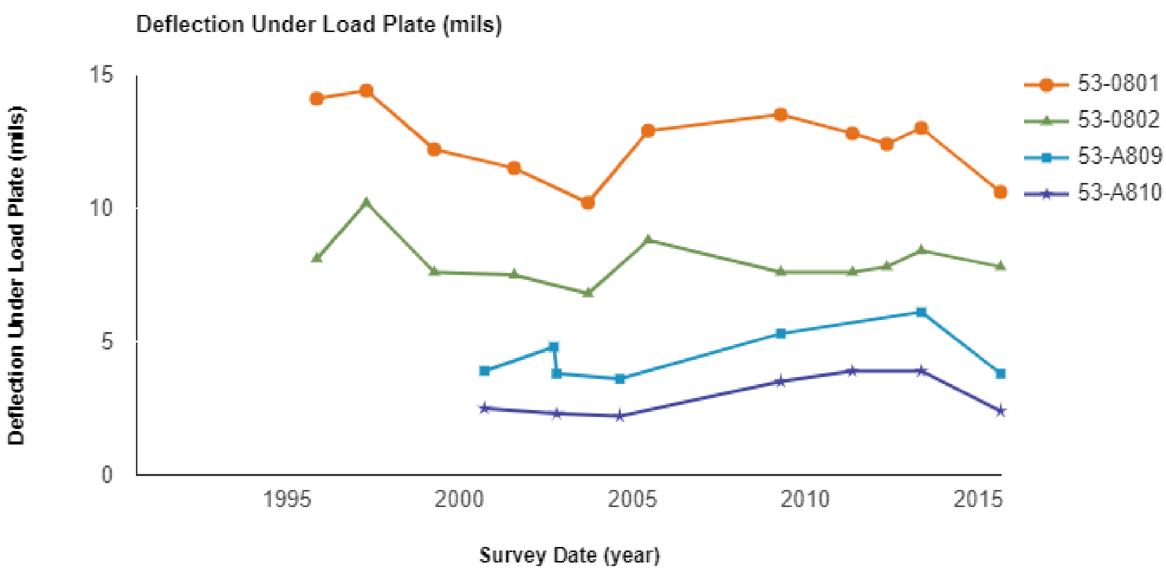


Figure 1. FWD deflections under the load plate over time.

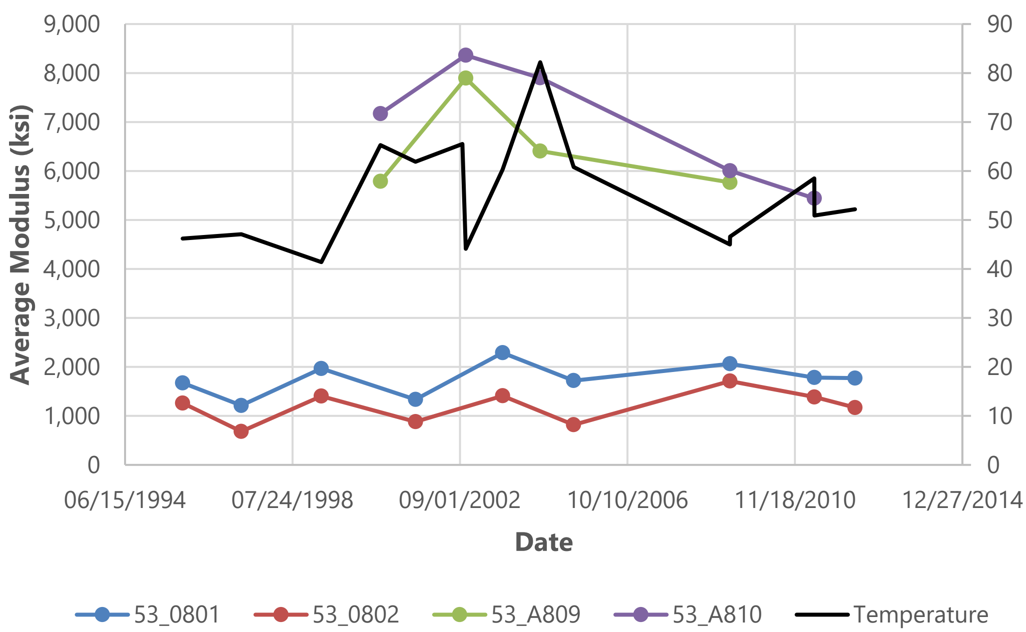


Figure 2. Average backcalculated modulus for surface layer (Layer 1).

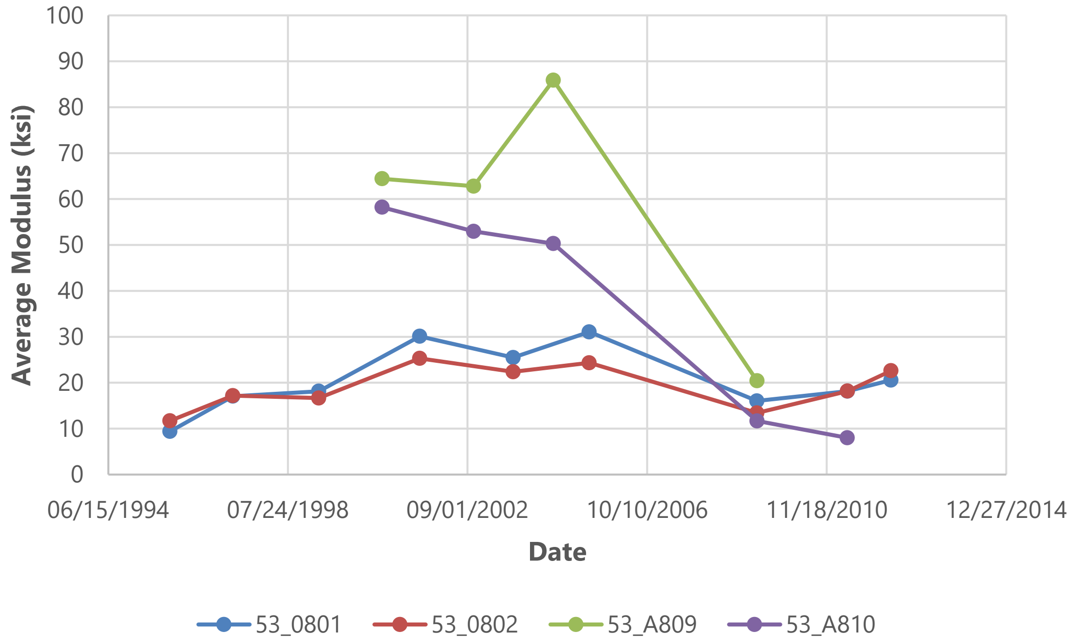


Figure 3. Average backcalculated modulus for base layer (Layer 2).

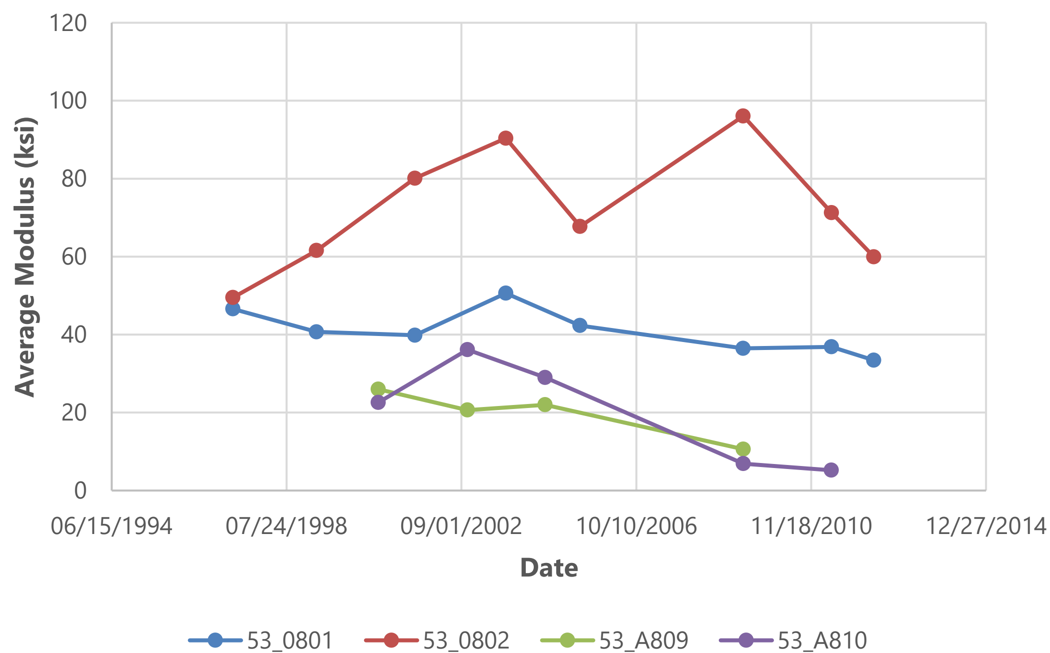


Figure 4. Average backcalculated modulus for top of subgrade/subbase (Layer 3).

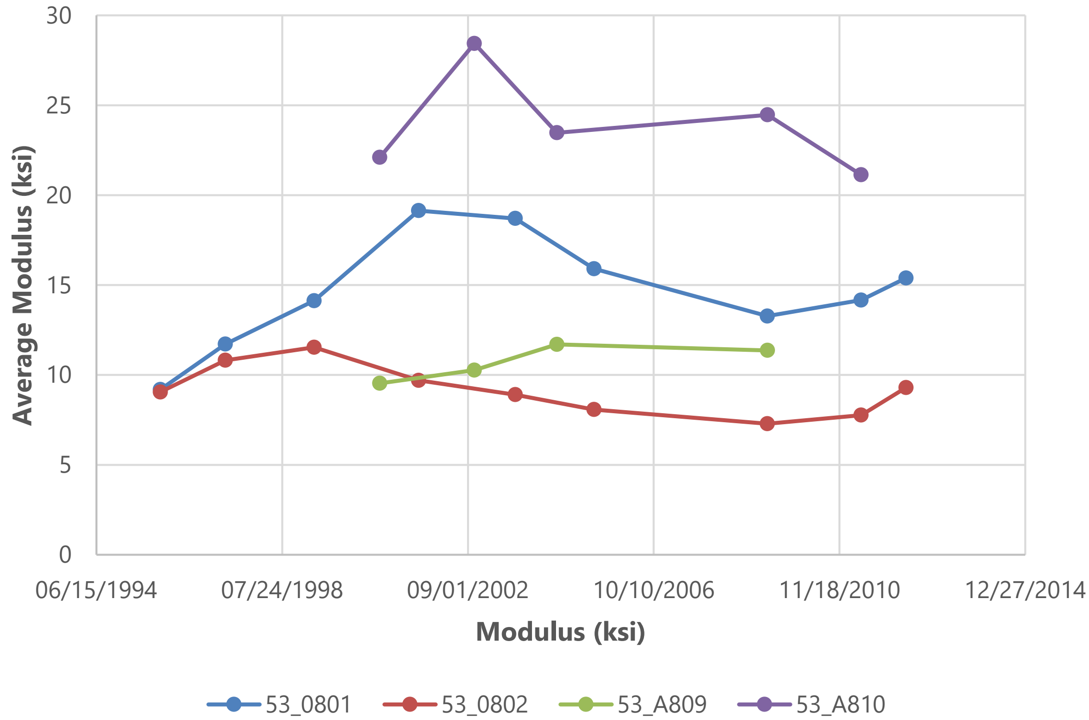


Figure 5. Average backcalculated modulus for subgrade (Layer 4).

The backcalculated moduli for the AC test sections 53_0801 and 53_0802 were similar over time. For the surface layer (Layer 1), the reported moduli fluctuated with test section 53_0801 reporting slightly higher average moduli values than test section 53_0802. The average modulus on the test sections was 1,757 ksi and 1,192 ksi for test section 53_0801 and 53_0802, respectively. These average values are slightly high for AC pavements; however, the reported temperature at the time of testing (especially low temperatures) may have impacted the moduli values reported. Trends reported for the base layer were also similar for both test sections. Again, test section 53_0801 reported a slightly higher average modulus of 21 ksi over the analysis period while 53_0802 reported an average modulus of 19 ksi. For both the subbase (Layer 3) and subgrade layers (Layer 4), there was a greater difference between the reported moduli for the two AC test sections. Test section 53_0802 reported an average backcalculated moduli of 84 ksi for Layer 3 while test section 53_0801 reported an average backcalculated moduli of 48 ksi. Notably, both test sections reported a higher backcalculated modulus in 1995 which appears to be an outlier and was removed from the figure above. For Layer 4, test section 53_0801 and 53_0802 reported an average backcalculated moduli of 15 ksi and 9 ksi, respectively.

The backcalculated modulus values reported for the two JPCP test sections were less consistent over time when compared to the AC test sections. Test section 53_A810—the thicker of the two JPCP test sections—reported higher modulus values for Layer 1 when compared to test section 53_A809. The average reported modulus of test section 53_A810 was 6,976 ksi while the average reported modulus of test section 53_A809 was 6,464 ksi, both which are higher than expected for a PCC pavement. Layer 2, the base layer, showed the greatest variability in moduli reported for the two JPCP test sections; test section 53_A809 fluctuated between 20.4 ksi and 86 ksi while test section 53_A810 fluctuated between 8 ksi and 58 ksi. For both test sections, the range of values reported for Layer 2 seems to indicate that there are outliers in the backcalculated values. The average backcalculated moduli for Layer 3 and Layer 4 were 18 ksi and 20 ksi and 14 ksi and 24 ksi for test sections 53_A809 and 53_A810, respectively. Test section 53_A810 reported a notably higher modulus value in October 2002; it is hypothesized the value was an outlier as the rest of the values reported on the test section were steady over time.

The reasonableness of the backcalculated layer moduli was compared to moduli derived from laboratory testing. Table 4 summarizes the laboratory test results for the AC layers, PCC layers, base layers, and subgrade layers. For the AC layers, moduli values are shown for three test temperatures – 41, 77, and 104°F, respectively. As depicted 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 is a trend between pavement modulus and temperature.

Table 4. Laboratory Resilient Modulus Test Results

Test Sections	Layer	Temperature (°F)	Range of moduli values (ksi)
AC Test Sections (53_0801 and 53_0802)	AC	41	1,685-1,781
		77	471-579
		104	149-270
	Base	N/A	7-51
	Subbase-Top Layer	N/A	6-13
	Subbase-Bottom Layer	N/A	15-47
PCC Test Sections (53_A809 and 53_0810)	PCC	N/A	5,004-6,872
	Base	N/A	5-35
	Subbase	N/A	5-12
	Subgrade	N/A	7-12

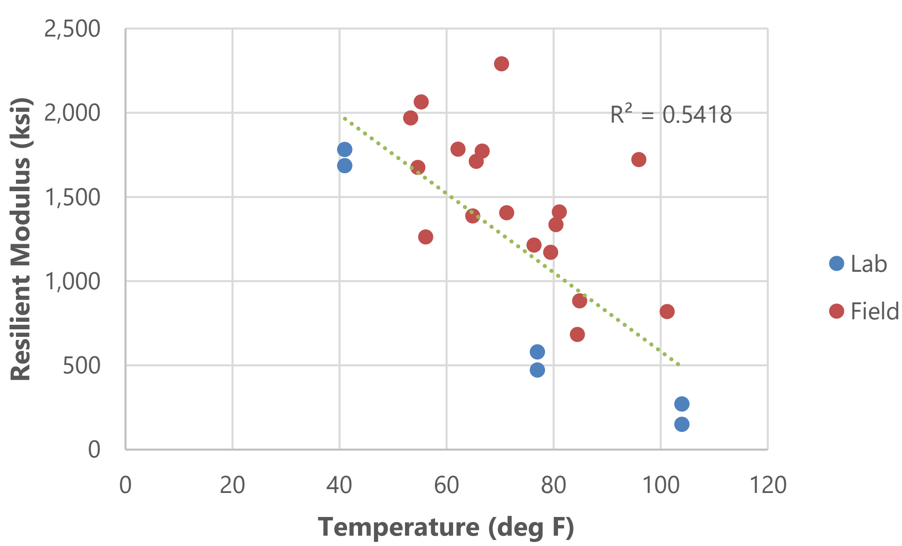


Figure 6. Field- and lab-derived AC resilient modulus values.

Climate History

The time history for total annual precipitation (using AWS) for all test sections is shown in Figure 7. For test section 53_0801 and 53_0802, the total annual precipitation observed fluctuated from year to year with an average annual precipitation of 15 inches between 1995 and 2008. For the JPCP test sections, the reported total precipitation was slightly less than the reported precipitation for the AC test sections. On average, test section 53_A809 and 53_A810 reported 8 inches of precipitation per year.

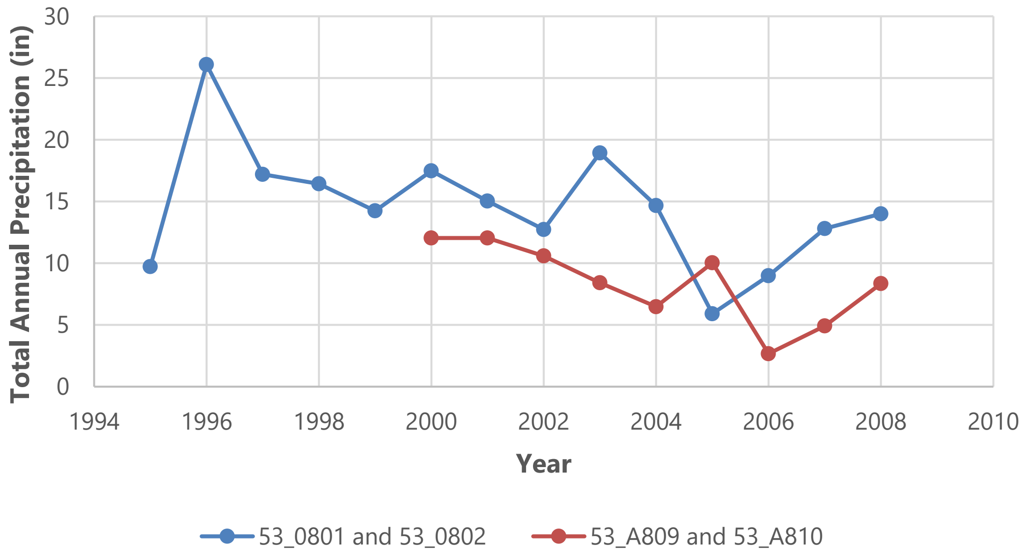


Figure 7. Average yearly precipitation over time.

Figure 8 shows the time history of the maximum monthly freezing index (from MERRA) for the test sites. 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. The freezing index of the test sections was variable throughout time. Values ranged from 31 deg F deg days in 1999 and 225 deg F deg days in 2008 for test sections 53_0801 and 53_0802 and from 18 deg F deg days in 2007 to 274 deg F deg days in 2008 for test section 53_A809 and 53_A810. While the test sections typically do not experience a deep freeze lasting throughout the winter months, the test sections do experience multiple freeze-thaw cycles per year.

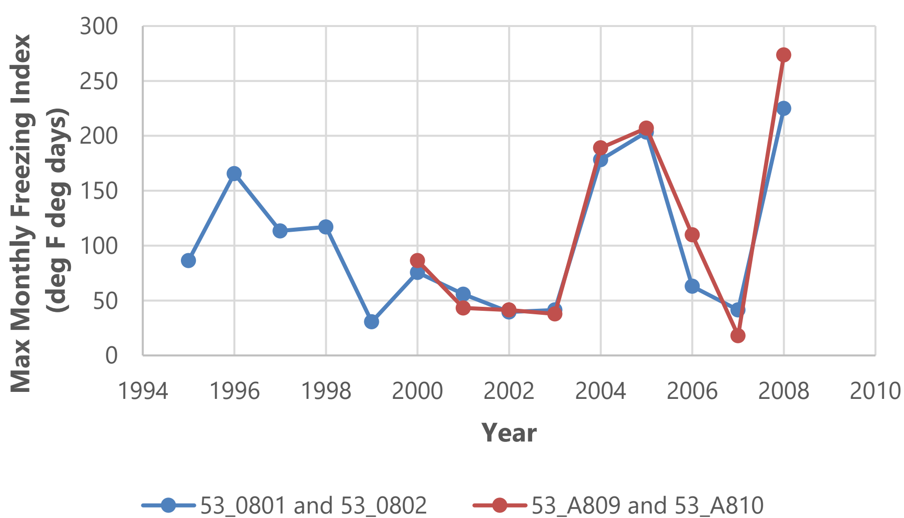


Figure 8. Maximum monthly freezing index over time.

Truck Volume History

The annual truck traffic counts for the test sections were minimal, as expected for SPS-8 test sections. For most years, the reported AADTT ranged from 0 to 31. Similarly, the average number of ESALs reported on the test sections was minimal for all years. In 2007, the test section 53_0801 and 53_0802 did have higher than normal logging traffic due to a local fire, but overall, traffic remained low. Therefore, traffic loading is not expected to play a damaging role in the overall performance of the test sections over time; rather, the absence of traffic should lead to longer service lives for the pavement test sections as compared to pavements receiving higher loading.

Pavement Distress History

The following summarizes the distresses observed on the test sections between 1995 and 2020, the year the last manual distress survey was conducted. Fatigue/alligator cracking, longitudinal cracking, transverse cracking, IRI, rutting, and faulting were assessed. While longitudinal and transverse cracking were evaluated for both the AC and JPCP test sections, there was no cracking observed on the JPCP test sections during the analysis period. However, the AC shoulders of the PCC test sections did show high amounts of cracking as these shoulders received minimal maintenance (typically crack sealing).

Fatigue/Alligator Cracking

Figure 9 shows the total reported area of fatigue/alligator cracking between 1995 and 2020 for the two AC test sections. Fatigue/alligator cracking was only reported on test section 53_0802, despite it being the thicker of the two AC test sections. It was first reported during the manual distress survey in 2004 when 3.20 ft^2 of fatigue/alligator cracking was observed. However, fatigue cracking was not observed again until 2008 due to the chip seal applied on the test section in 2005. Between 2008 and 2020, the fatigue/alligator cracking observed continued to increase from 14 ft^2 to 156 ft^2 at an average rate of 11.8 ft^2/year. The observed cracking was predominantly located in the wheel path.

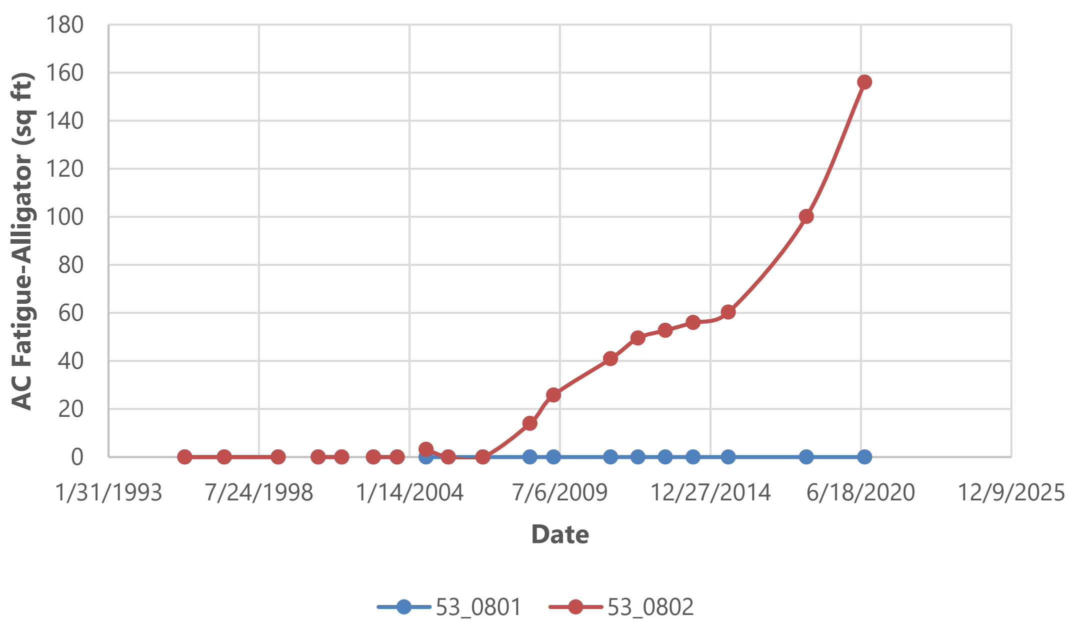


Figure 9. Time history of the length of fatigue cracking.

As fatigue/alligator cracking indicates that a pavement is either structurally inadequate or has reached the end of its service life, the fatigue/alligator cracking observed on the thicker pavement structure is counterintuitive, especially in light of the limited amount of traffic at the site. One potential reason for the differences in the fatigue/alligator cracking observed on the test sections is the difference in construction of the two test sections. It was noted in the construction report that the AC surface of test section 53_0802 was constructed in multiple lifts. However, between the first and second lift, the contractor did not allow time for the first lift to cool before the second lift was applied. Instead, an emulsified tack coat was added. It is therefore hypothesized that the lack of bonding between these two lifts may have resulted in fatigue/alligator cracking observed on test section 53_0802. An interview with Washington State DOT staff familiar with the test sections revealed that fatigue/alligator cracking is typical within this area. The State DOT has previously tried to mitigate the problem by chip sealing the roadway every 4-5 years but has since shifted to placing fog seals and chip sealing every 12-14 years instead.

The cause(s) of fatigue/alligator cracking observed on the test section was also considered in terms of overall SPS-8 experiment. A study by Glover et al.[3] focused on identifying and quantifying the effects of environmental and design factors on the performance of pavements in the absence of heavy loads by comparing the performance of SPS-8 test sections to heavily trafficked test sections from other LTPP experiments. Specifically, the study considered the impact of factors such as freeze versus non-freeze climates, freeze-thaw cycles, frost depth, AC temperature, annual precipitation, in situ moisture content, subgrade type, clay content, and others on the development of different distress types including fatigue cracking. The analysis found that for low-traffic test sections, most of the assessed factors had little impact on the development and progression of fatigue/alligator cracking. However, the study did find that while base and subgrade in situ moisture content was 3 percent or less for low traffic test sections, the higher the in-situ moisture content, the increased impact on the development of fatigue cracking after 15 years. The study also found that for both low traffic and high traffic test sections in non-freeze climates and test sections with higher in situ temperatures, typically had increased amounts of fatigue/alligator cracking. While this helps describe why fatigue/alligator cracking may have developed on this test section, it fails to explain why fatigue/alligator cracking was only observed on the thicker AC test section.

Longitudinal Cracking

Non-wheel path (NWP) longitudinal cracking, depicted in Figure 10, was first reported during the manual distress survey in 2000 for both AC test sections. Between 2000 and 2004, length of cracking observed on the test section increased at of a rate of 69.5 feet/year and 77.7 feet/year for test section 53_0801 and 53_0802, respectively. However, following the chip seal applied on the test sections in 2005, NWP longitudinal cracking was not observed again until 2008, at which point the reported amounts of NWP longitudinal cracking was similar to the levels observed prior to the chip seal; 297 feet of NWP cracking was reported on test section 53_0802 while 500 feet of cracking was reported on test section 53_0801. Between 2008 and 2020, the amount of NWP cracking reported on test section 53_0801 remained consistent while the cracking observed on test section 53_0802 increased levels similar to test section 53_0801. By 2020, test section 53_0801 reported 500 feet of NWP longitudinal cracking while test section 53_0802 reported 489 feet of NWP longitudinal cracking. The observed NWP longitudinal cracking reported on both test sections was located on the centerline of the roadway. Therefore, the reported cracking is likely construction-related as the cracking appears to be located on a roadway joint.

Wheel path (WP) longitudinal cracking was minimal on the AC test sections; less than 5 feet of WP longitudinal cracking was reported on the test sections over time.

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

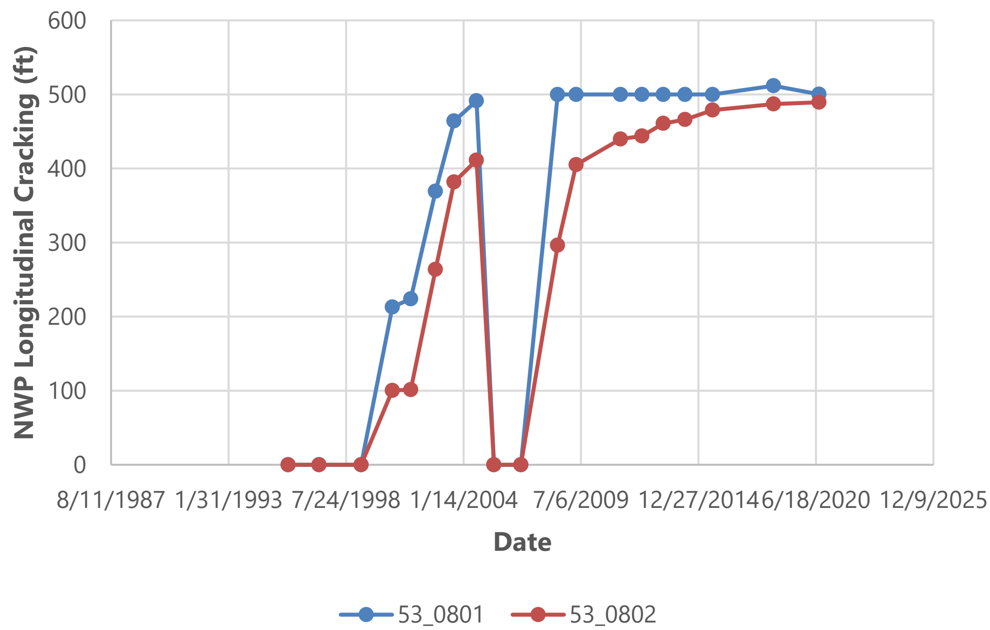

Figure 10. Time history of the length of NWP longitudinal cracks.

Transverse Cracking

Data on transverse cracking was collected between 1995 and 2020, as shown in Figure 11 and Figure 12. Test section 53_0802 first reported transverse cracking during the manual distress survey in 2011, 16 years after the construction of the test sections, when 5 feet of transverse cracking (3 cracks) were observed. Between 2011 and 2020, the reported transverse cracking increased at a rate of 19 feet/year, reaching 176 feet of cracking (55 cracks) in 2020. Transverse cracking was first reported on test section 53_0801 in 2014, when 48 feet (6 cracks) was reported on the test section. Test section 53_0801 reported an increase in transverse cracking between 2014 and 2020, at which point 110 feet (11 cracks) of transverse cracking was observed. For test section 53_0801, the observed transverse cracking was a mix of both full width (length of the lane) and partial width cracks while for test 53_0802, the reported cracking was predominantly partial width cracks, some in which crack sealing was applied but failed. While the reported transverse cracking on both sections remained minimal, it is hypothesized that it was thermal induced cracking or related to freeze-thaw cycles. Washington State DOT staff familiar with the test sections noted that transverse cracking was typical on this roadway and that the cracking would likely develop to full width over time. The State DOT also mentioned that they had spent a lot of effort on crack sealing to help mitigate this issue.

The transverse cracking observed on the test sections was also considered in terms of overall SPS-8 experiment. Glover et al.[4] found that for low traffic, thin AC test sections (such as test section 53_0801) transverse cracking was more prevalent in freeze climates, on test sections with high amounts of freeze-thaw cycles, on sections with high frost depths, on test sections with low in situ AC temperatures, and on sections that were in dry climates. For thick, low-traffic AC test sections (such as test section 53_0802), the study showed transverse cracking was more prevalent in no-freeze climates, on test sections with high

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

amounts of freeze-thaw cycles, on sections with high frost depths, on test sections with high in situ AC temperatures, and on sections that were in wet climates.

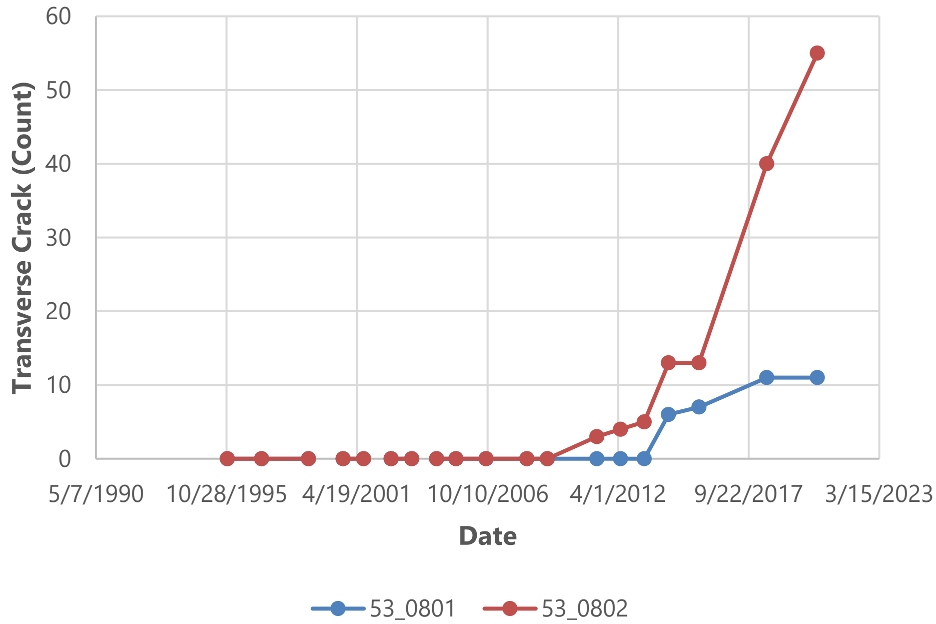

Figure 11. Time history of the number of transverse cracks.

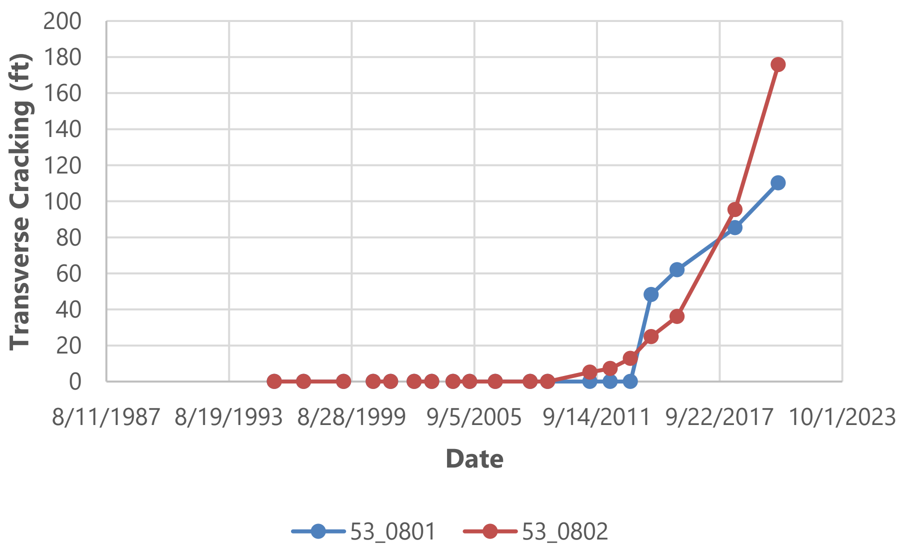

Figure 12. Time history of the length of transverse cracking.

Rutting

The average rut depths observed on the AC test sections between 1995 and 2020 are shown in Figure 15. Both test sections reported relatively low rut depths over time, ranging between 0.04- and 0.16-inch. Each AC test section reported variability in rut depths reported over time, but this variability was within error.[5]

[5] Simpson, A., Rada, G., Bryce, J., Serigos, P., Visintine, B. and Groeger, J. (2018). *Interstate Highway Pavement Sampling Data Quality Management Plan*. Washington, DC: Federal Highway Administration.

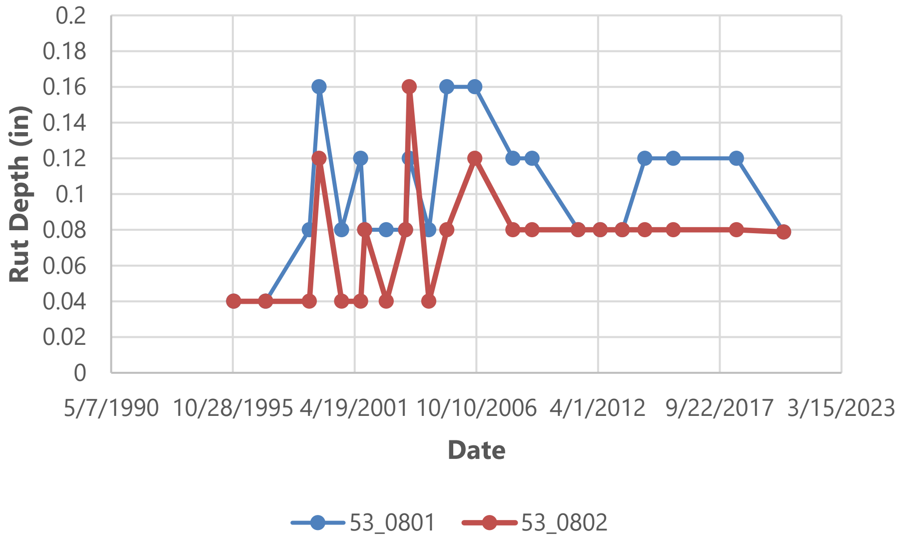

Figure 13. Time history plot of average rut depth.

Faulting

The average faulting observed over time on the JPCP test sections is shown in Figure 16. The faulting on both test sections was minimal. Test section 53_A809 reported faulting between 0 in and 0.02 in (2013). For test section 53_A810, faulting ranged from -0.01 in (2006) to 0.03 in (2013). The average faulting for the JPCP test sections is classified as "Good" based on FHWA performance definitions.

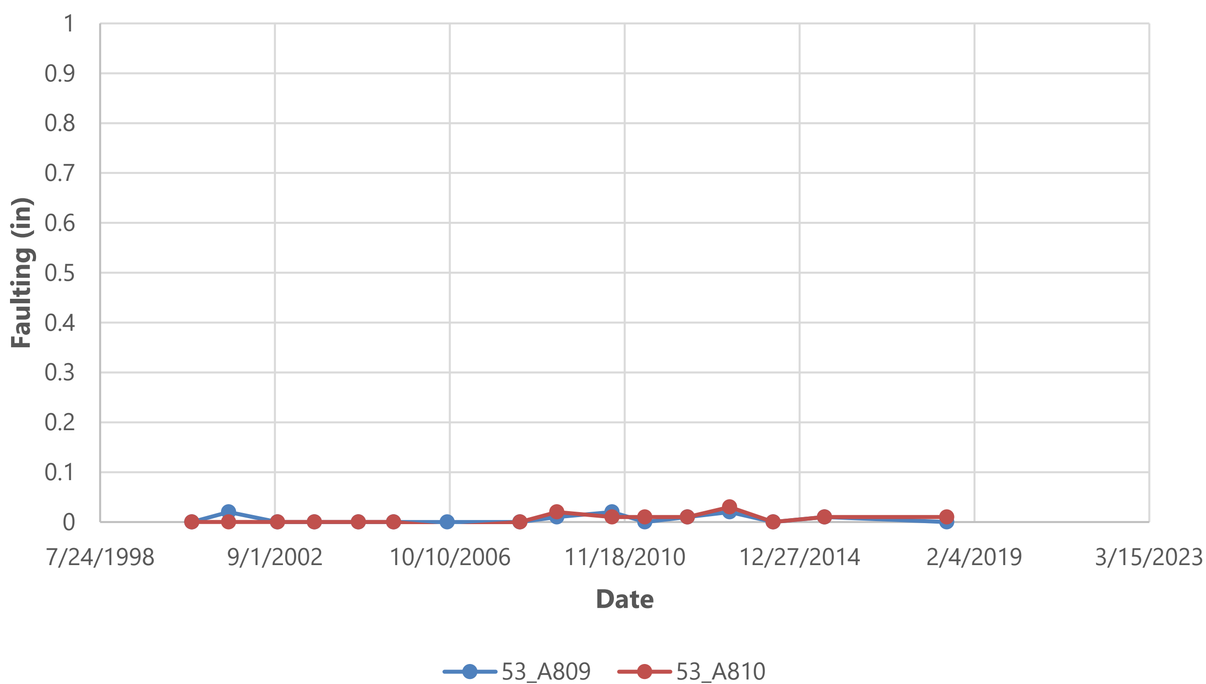

Figure 14. Time history plot of average faulting.

IRI

The average IRI measurements for all four test sections over time are shown in Figure 14. While test section 53_0802 reported slightly higher IRI values for every collection date, likely due to the fatigue/alligator cracking reported on this section, the test sections generally performed similarly. The reported IRI on the test sections slightly increased or stayed the same between the first date of IRI data collection (1995 for test sections 53_0801 and 53_0802 and 2000 for test sections 53_A809 and 53_A810) and 2018: from 57 to 78 inches/mile for test section 53_0801, from 80 to 82 inches/mile for test section 53_0802, from 65 to 72 inches/mile for test section 53_A809, and it remained close to 67 inches/mile for test section 53_A810. For both AC test sections, the chip seal applied in 2005 resulted in a slight increase in IRI as shown in the figure below. However, the average IRI of all four test sections is classified as "Good" based on FHWA performance definitions. It is notable that while the JPCP test sections did not report any cracking and only minimum faulting, the smoothness of the two test sections was comparable to the AC test sections. This might be the result curl and warp on the JPCP test sections. Overall, the IRI reported on all test sections was consistent over the history of the test sections.

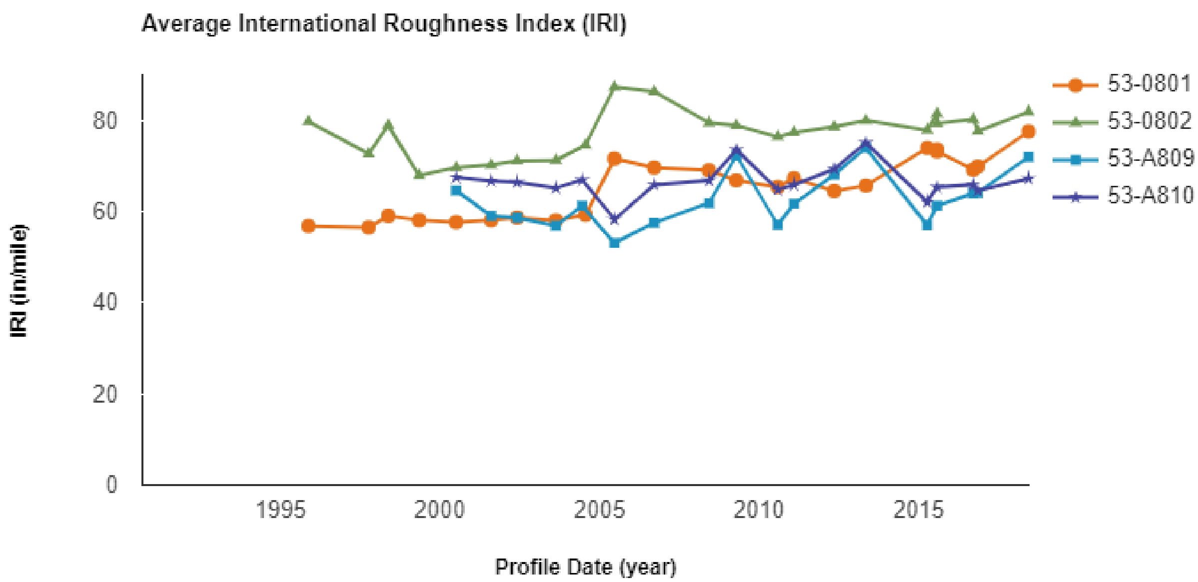

Figure 15. Time history plot of pavement roughness.

SUMMARY OF FINDINGS

LTPP test sections 53_0801 and 53_0802 are located on North Touchet Rd, northbound, in Columbia County, Washington State, while test sections 53_A809 and 53_A810 are located on Smith Springs Rd, eastbound, in Walla Walla County. The four test sections were constructed and accepted into the LTPP Program as part of the SPS-8 Study of Environmental Effects in the Absence of Heavy Loads experiment. At the time of construction in 1995, test section 53_0801 consisted of 3.7 inches of dense-graded asphalt concrete and 8 inches of unbound granular base over 38.4 inches of unbound subbase and a fine-grained subgrade soil. Test section 53_0802, also constructed in 1995, consisted of 6.8 inches of dense-graded asphalt concrete (0.2-inch less than the specified design thickness) and 11.7 inches of unbound granular base (0.3-inch less than the specified design thickness) over 38.4 inches of granular subbase and a coarse-grained subgrade soil. Both test sections were crack sealed in 2000, 2003, and in June and October of 2015. Test section 53_0801 also received a 0.3-inch fog seal in 2005 and a 0.1-inch fog seal in 2011 while test section 53_0802 only received 0.3-inch chip seal in 2005. Test sections 53_A809 and 53_A810 were both constructed in 2000 as four layers: 8.5 and 10.9 inches of Portland Cement Concrete (PCC) and 4.5 and 4.7 inches of unbound granular base over an unbound granular subbase and fine-grained subgrade soil for test sections 05_0809 and 05_0810, respectively. Neither section reported any additional construction events following their incorporation into the LTPP program. Key findings of the desktop study include:

1. **The cause(s) of the fatigue cracking observed on test section 53_0802.** The fatigue/alligator cracking observed on the thicker pavement structure is counterintuitive. One potential reason for the differences in the fatigue/alligator cracking observed on the test sections is the difference in construction of the two test sections. It was noted in the construction report that the AC surface of test section 53_0802 was constructed in multiple lifts. However, between the first and second lift, the contractor did not allow time for the first lift to cool before the second lift was applied. Instead, an emulsified tack coat was added. It is therefore hypothesized that the lack of bonding between these two lifts may have resulted in fatigue/alligator cracking observed on test section 53_0802.
2. **The difference(s) in the number of transverse cracks reported on the AC test sections (53_0801 and 53_0802).** Test section 53_0801 reported 110 feet (11 cracks) of transverse cracking in 2020 while test section 53_0802 reported 176 feet of cracking (55 cracks) in 2020. For test section 53_0801, the observed transverse cracking was a mix of both full width (length of the lane) and partial width cracks while for test section 53_0802, the reported cracking was predominantly partial width cracks, some in which crack sealing was applied but failed. While the reported transverse cracking on both sections remained minimal, it is hypothesized that it was thermal induced cracking or related to freeze-thaw cycles.
3. **The differences in the reported IRI of the AC and JPCP test sections over time.** All four test sections performed similarly in terms of IRI. It was notable that while JPCP test did not report any cracking and only minimal faulting that the smoothness of the PCC sections was comparable to the AC test sections. This might be the result curl and warp on the JPCP test sections.

FORENSIC EVALUATION RECOMMENDATIONS

It is recommended that the desktop study be extended to further investigate the various topics addressed in this technical memorandum by carrying out the following:

- Continued performance monitoring of test sections while each is active.
- Perform FWD testing on a two-year interval to measure the change in structural capacity.
- Coring outside of test sections 53_0801 and 53_0802 to examine whether observed cracking is top down or bottom up.