Technical Memorandum

To: Jeff Uhlmeyer

From: Gary Elkins, Gonzalo Rada, Kevin Senn and David Jones

cc: Mustafa Mohamedali

Date: Original: June 9, 2019, Revised: June 19, 2019

Re. Forensic Desktop Study Report: LTPP Test Section 531005, Field Evaluation Update

The Long-Term Pavement Performance (LTPP) General Pavement Studies (GPS) test section 531005[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations" since it is scheduled for mill and overlay in the near future. The purpose of this document is to review the test section history, examine distress manifestations, and make recommendations on the need for forensic evaluation related work to explain the performance of the test section over time.

SITE DESCRIPTION

LTPP test section 531005 is located on I-90, east bound at milepost 208.6 in Adams County, Washington State. I-90 is a rural interstate highway with two lanes in the direction of traffic. It is classified as being in a Dry Freeze climate zone with an average annual precipitation of 10.6 inches and an annual average air freezing index of 389 Deg-F degree-days. The coordinates of the test section are 47.09633, -118.6293. Photograph 1 shows the test section in 2013, while Map 1 shows the geographical location of the test section within the State of Washington.

BASE-LINE PAVEMENT HISTORY

The information included in this portion of the document presents the baseline data on history of pavement structure, climate, traffic and pavement distresses, roughness and deflection.

[1] First two digits in test section number represent the State Code [53 = Washington]. For LTPP GPS test sections, the final four digits are unique within each State/Province and they were assigned at the time the test section was accepted into the LTPP program. For LTPP Specific Pavement Studies (SPS) test sections, the second set of two numbers indicates the Project Code (e.g., 02 = SPS-2) and the final set of two numbers represents the test section number on that project (e.g., 13).

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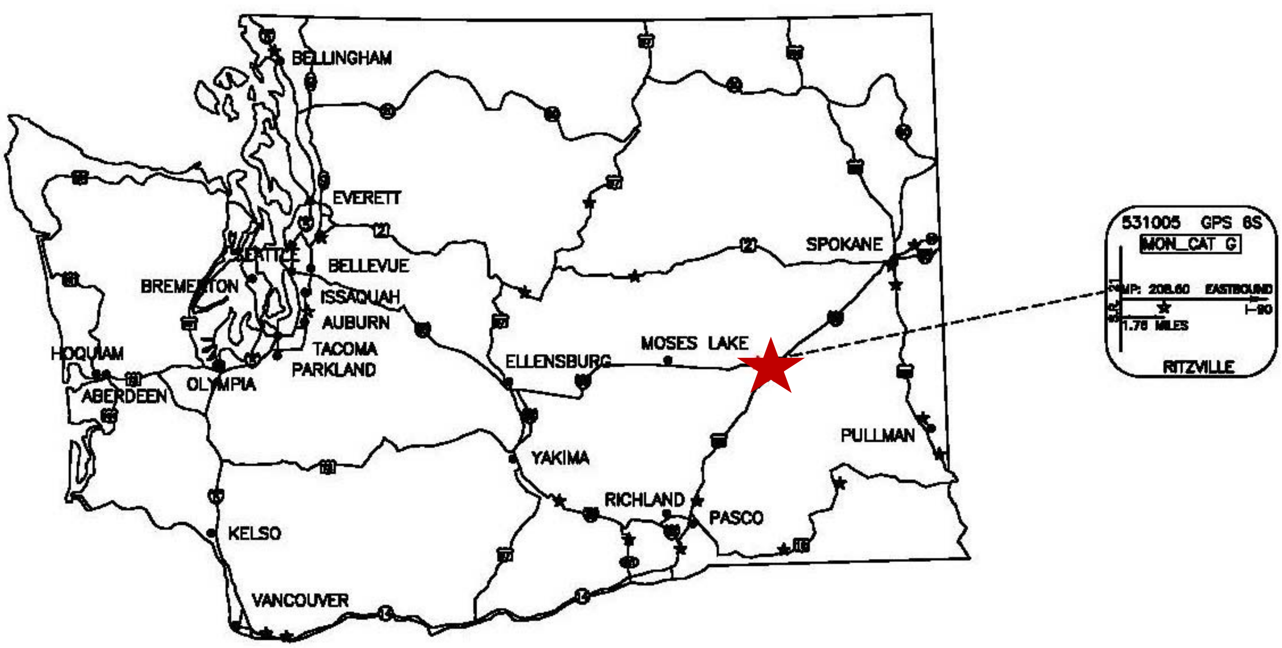
Source: FHWA

Photograph 1. Picture of test section 531005 in 2013 (from start of section looking west).



Source: FHWA

Map 1. Geographical location of test section within the State of Washington.

Pavement Structure and Construction history

The initial pavement structure was constructed in 1973. This was the original pavement structure that was included in the LTPP program in 1988. The original layer structure is detailed in Table 1. This corresponds to CONSRTUCTION_NO = 1 (CN1).

Table 1. Pavement structure from 1973 to 1989.

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		254-Coarse-Grained Soil: Poorly Graded Gravel with Silt
2	Unbound (granular) subbase	6.5	304-Crushed Gravel
3	Unbound (granular) base	3.0	304-Crushed Gravel
4	Asphalt concrete layer	5.9	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	3.6	1-Hot Mixed, Hot Laid AC, Dense Graded

In July 1989, the test section was overlaid with a 2.3-inch dense graded asphalt concrete mixture. No milling was performed. This corresponds to CN = 2. The resulting pavement structure is detailed in Table 2.

Table 2. Pavement Structure from 1989 to 2001

Layer Number	Layer Type	Thickness (in)	Material Code Description
1	Subgrade (untreated)		254-Coarse-Grained Soil: Poorly Graded Gravel with Silt
2	Unbound (granular) subbase	6.5	304-Crushed Gravel
3	Unbound (granular) base	3.0	304-Crushed Gravel
4	Asphalt concrete layer	5.9	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	3.6	1-Hot Mixed, Hot Laid AC, Dense Graded
6	Asphalt concrete overlay layer	2.3	1-Hot Mixed, Hot Laid AC, Dense Graded

In October 2001, approximately 2 inches of milling was performed, and a nominal new 2-inch overlay was placed. This corresponds to CN = 3. The resulting pavement structure is detailed in Table 3. Note that the thickness of layer 6 has been reduced.

Table 3. Pavement Structure from 2001 to Date

Layer Number	Layer Type	Thickness (in)	Material Code Description
1	Subgrade (untreated)		254-Coarse-Grained Soil: Poorly Graded Gravel with Silt
2	Unbound (granular) subbase	6.5	304-Crushed Gravel
3	Unbound (granular) base	3.0	304-Crushed Gravel
4	Asphalt concrete layer	5.9	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	3.6	1-Hot Mixed, Hot Laid AC, Dense Graded
6	Asphalt concrete overlay layer	0.2	1-Hot Mixed, Hot Laid AC, Dense Graded
7	Asphalt concrete overlay layer	2.1	1-Hot Mixed, Hot Laid AC, Dense Graded

Patching was performed in August 2008 and June 2013. These events correspond to CN = 4 and CN = 5.

Climate History

The time history for annual average precipitation since 1973 is shown in Figure 1. In 1996, the amount of precipitation appears to be a local high at the site, albeit low by national standards. However, there appears to be no drastic changes in pavement distresses or pavement response due to this year of higher than normal precipitation.

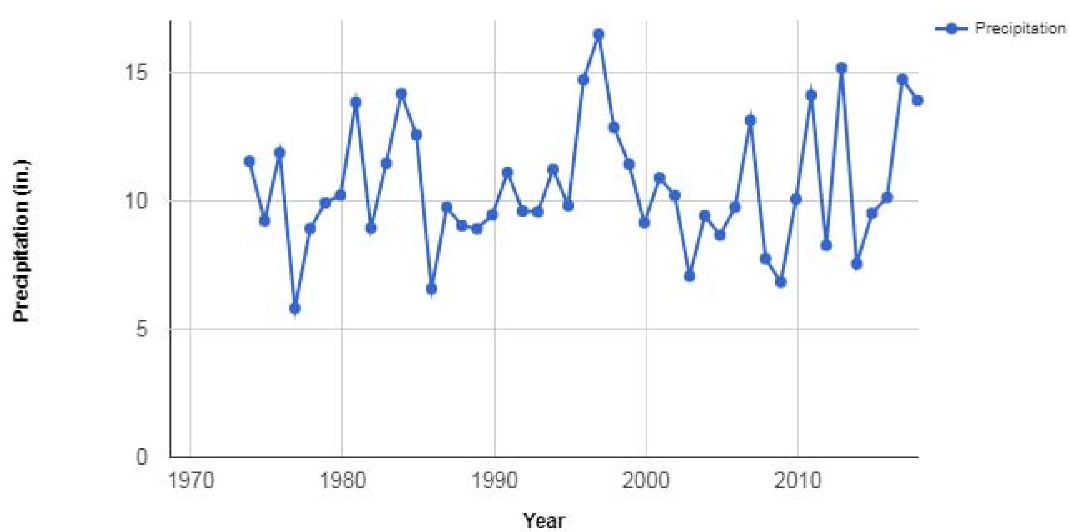


Figure 1. History of annual precipitation starting in 1973.

Figure 2 shows the time history of the annual freezing index over the history of this project. The freezing index is the sum of the difference between 32 degrees F and when the average air temperature is less than freezing and 32 degrees F for each day, which is summed 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. A large spike in extreme low temperatures can be observed in 1985. Because the test section came into LTPP study in 1988, it can only be postulated that maybe the extreme low temperature in 1985 might have been the cause of the transverse cracks in the original pavement structure constructed in 1973.

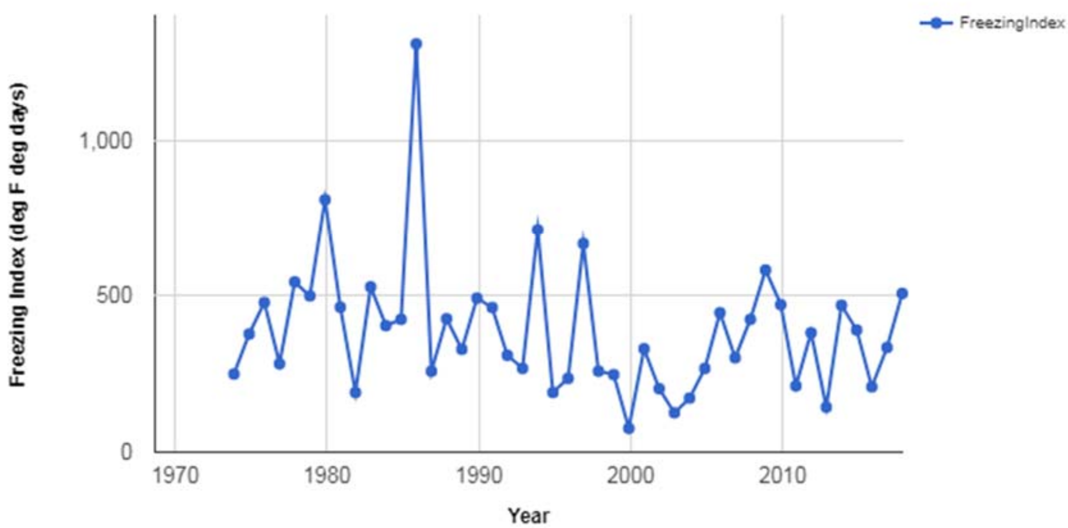


Figure 2. Time history of annual air temperature freezing index.

Truck Volume History

Figure 3 shows the annual truck volume data in the LTPP test lane by year. The triangles are data provided by Washington State DOT (WSDOT) from historical records. The blue diamonds are truck counts based on monitoring data reported to LTPP by WSDOT. Mount St. Helen's volcano, located in the south western portion of Washington, erupted in 1980, which might explain the bump in truck volumes just after this time period. In terms of trends, since about 1990, annual truck volumes in the LTPP lane, have almost doubled.

Pavement Distress History

One of the more significant distresses on this test section appears to have been transverse cracking. Figure 4 shows the time history of the number of transverse cracks on the test section. In this figure, the construction events are shown with vertical lines. The distress data shown are from manual distress surveys. Note that a manual distress survey was not performed immediately after the July 1989 overlay. Photographic based automated distress survey (ADS) information shows that by September 1989, one transverse crack had reappeared, by July 1991 13 transverse cracks were present, by June 1993 14 transverse cracks, and then the data from the manual distress surveys as shown in figure 1 picks up the long-term history of transverse cracking as a function of time and CN events.

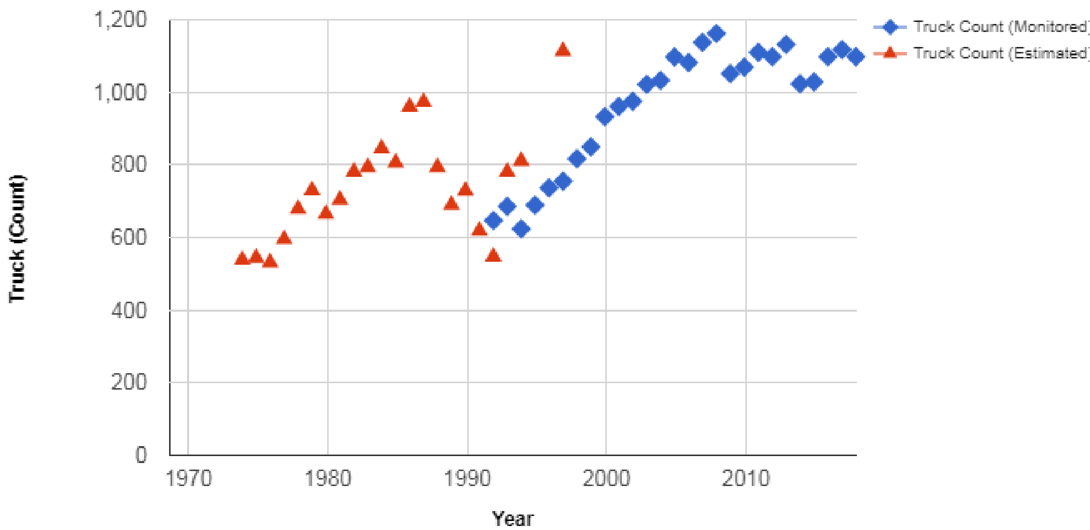


Figure 3. Average annual daily truck traffic on test section 531005 based on state supplied estimates and monitoring measurements.

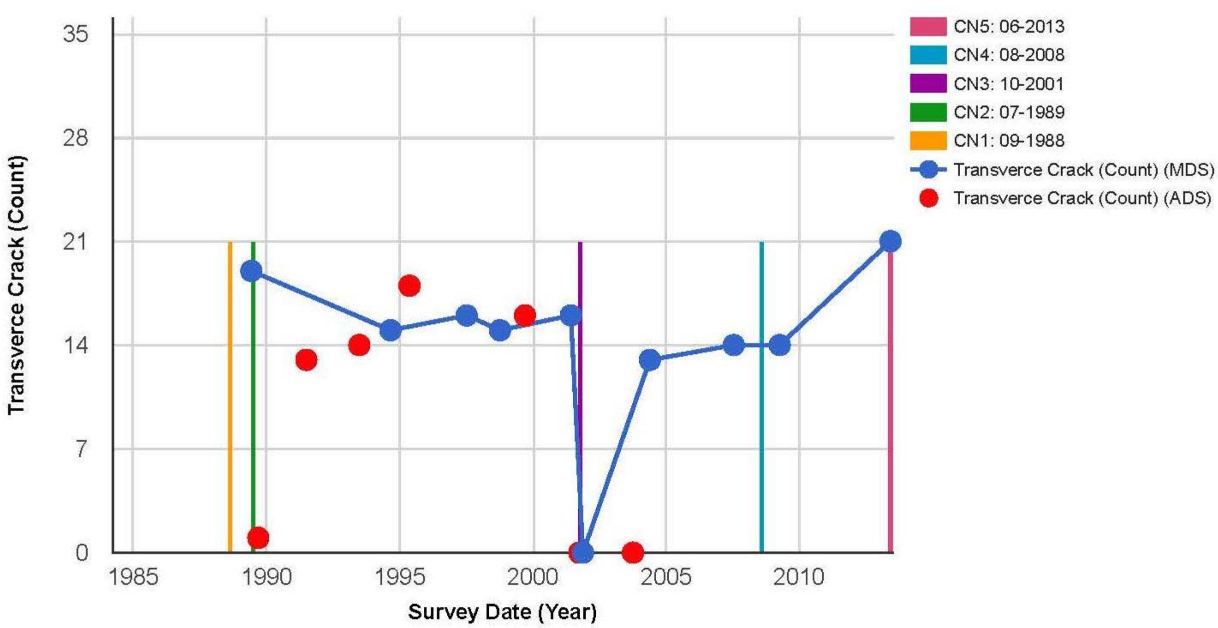


Figure 4. Time history of the number of transverse cracks.

Based on the location of the test section and the spacing between transverse cracks, it appears that these transverse cracks are related to low temperature cracking. After each overlay, it is postulated that it takes time for the transverse cracks in the previous pavement structure to reflect through the overlay layer. In Figure 4, the symbols connected with lines are from manual distress surveys (MDS), while the symbols not connected with lines are from automated distress surveys. Thus, after application of the overlays the number of transverse cracks visible on the pavement surface drop to zero, then they progressively reappear over time. During the most recent distress survey performed on this test section in 2013, 3 more transverse cracks had formed than were present in the original pavement structure in 1988, just prior to the 1989 overlay.

Figure 5 shows the time history plot of alligator cracking data on LTPP test section 531005. While the plot is labeled fatigue cracking, in the LTPP distress rating system, this is alligator cracking that is not limited to the wheel path. The amounts of alligator cracking on this test section are relatively low.

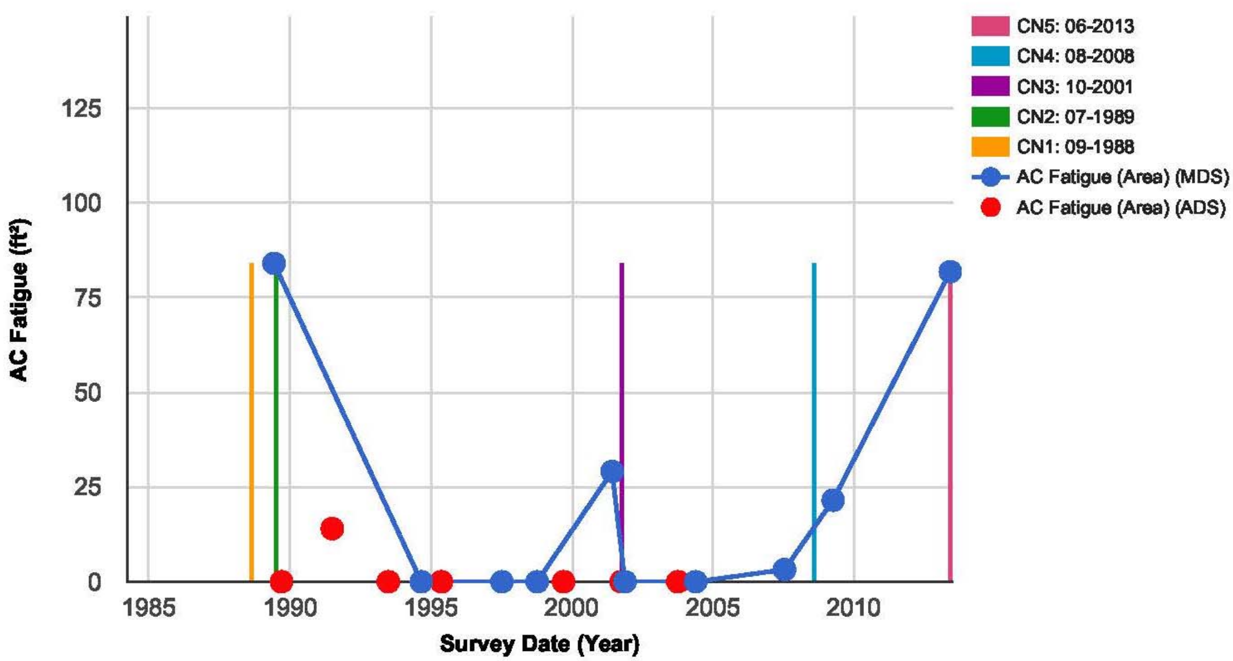


Figure 5. Time history of area of alligator cracking.

Figure 6 shows the time history plot of longitudinal cracking in the wheel path which can be considered as an indicator of fatigue cracking on a pavement. Except for the one minor rating of 6.9 feet in 1997, there essentially does not appear to be any wheel path cracking related to a fatigue mechanism on this test section.

The time history plot of rutting on the test section is shown in Figure 7. LTPP does not appear to have transverse profile measurements prior to the 1989 overlay. However, the rut depth during the first overlay started at about 6 mm and progressed to 14 mm prior to the October 2001 mill and fill overlay. The noise in this graph is probably due to the way LTPP performs transverse profile measures using a Dipstick. The starting point of the measurement at the outside lane edge can affect the computed rut depths. A maximum average rut depth of 0.4 inches is currently considered as being greater than the maximum limit for design purposes on Interstates according to the 2015 version of the AASHTO Mechanistic-Empirical Pavement Design Guide and may have been the trigger for the mill and fill overlay performed in 2001. It is not clear if the rutting is due to plastic deformation in the pavement layers or ablative wear on the surface potentially due to use of winter traction control devices, such as studded tires or chains.

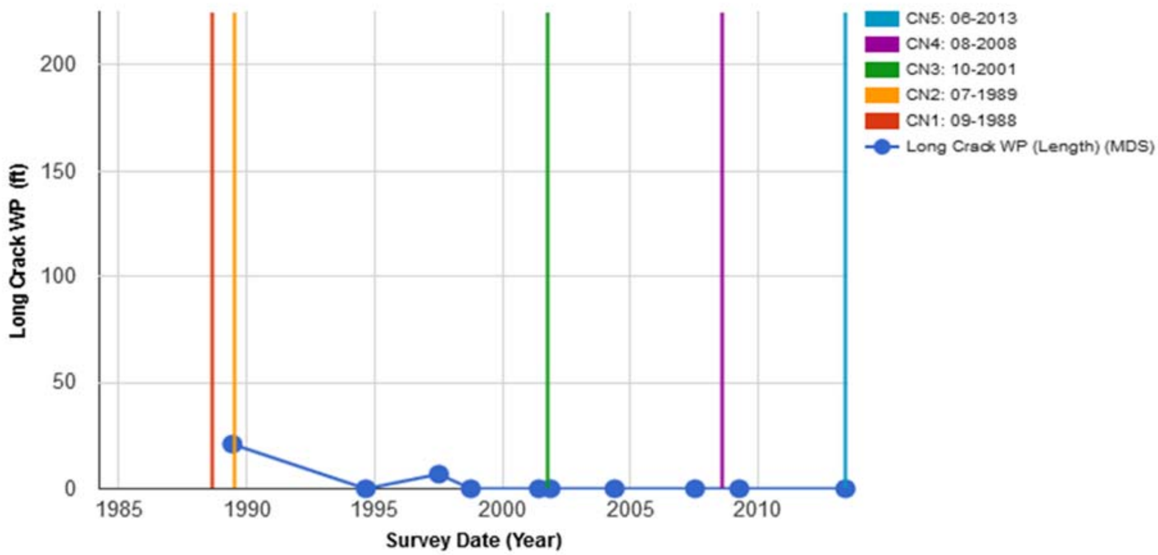


Figure 6. Time history of longitudinal cracking in the wheel path.

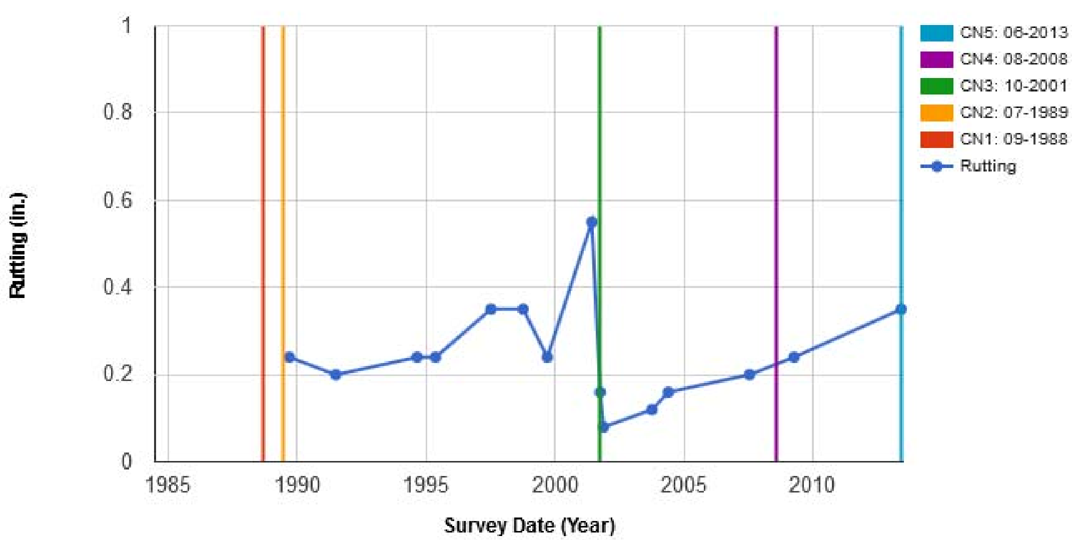


Figure 7. Time history plot of average rut depth computations.

The time history of roughness measurements is shown in Figure 8. An interesting feature of this graph is that the IRI before and after the mill and fill overlay October 2001 is nearly identical. From 1989 through 2013, the rate of increase in IRI has been relatively linear, even after the mill and fill overlay performed in 2001. However, since 2013, the rate of increase in IRI appears to have been accelerated. Since 2015, the IRI has exceeded FHWA's "Good" category for interstates of 95 inches/mile but has not yet reached the unacceptable limit of 170 inches/mile. It is possible that the section IRI values are being affected by the winter traction control devices, which do not wear the pavement out smoothly.

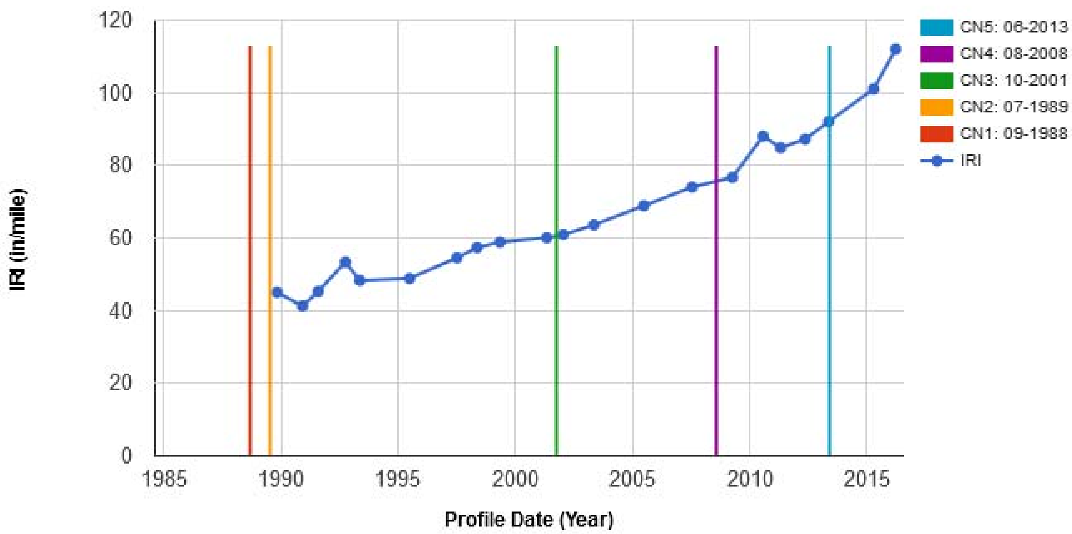


Figure 8. Time history plot of pavement roughness.

Figure 9 shows the time history average FWD deflection plot under the nominal 9,000 lb. load from the sensor position in the load plate on LTPP test section 531005. The deflection of the sensor located in the load plate is a general indication of the total "strength" or response of all layers in the pavement structure to a vertical applied load. This deflection can be influenced by pavement temperature at the time of testing, precipitation, and changes in pavement structure. In Figure 9, the short term decreases in deflection magnitude at the center sensor, resulting from what was an essentially a 2-inch mill and fill overlay construction event is interesting. One potential contributing factor to this minor decrease in deflection is that the deflection measurements prior to overlay were performed in June, while the measurements after overlay were performed in November when the pavement is generally colder. Overall the central sensor deflection time history magnitudes are judged to be relatively consistent with respect to natural variations in FWD measurements and very low indicating a "strong" pavement structure.

SUPPLEMENTAL DATA

In this portion of the desktop study, a more in-depth evaluation is made of the available data to try and explain the performance of the observed pavement structure.

While transverse cracking is a predominate distress feature of this pavement section, rutting, based on transverse profile measurements appears to be the only distress feature to have exceeded a national threshold limit on pavement performance indicators.

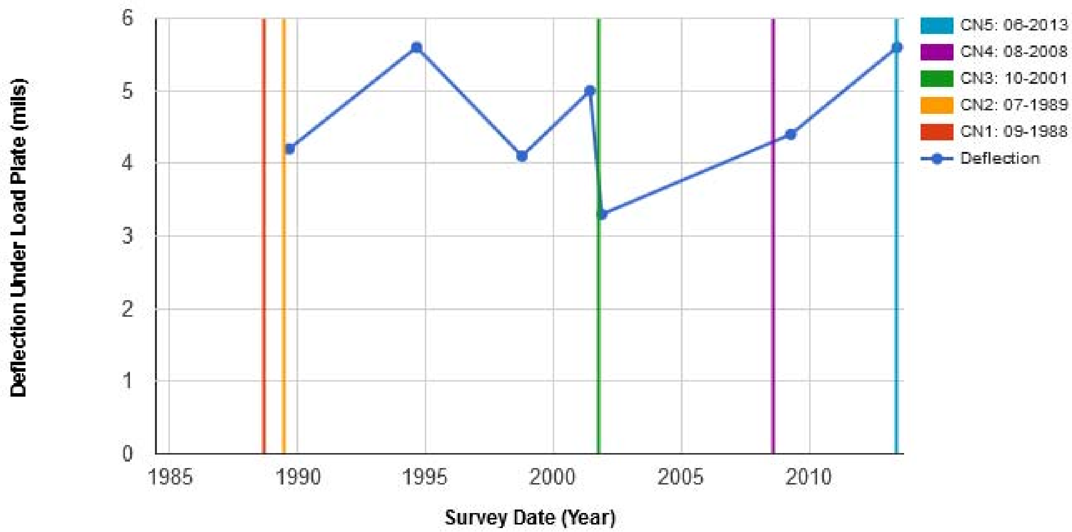


Figure 9. Time history of average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load.

Transverse Profile Measurements

The maximum average rut depth was recorded in the June 2001 distress survey just prior to the mill and fill construction activity performed in October 2001. Figure 10 shows the comparisons between normalized transverse profile elevation plot for station 0+00 in June 2001 and raw transverse profile measurement. A normalized transverse profile elevation plot, is one where the start and end points of the transverse profile are assigned a zero elevation, and the elevations of the pavement profile measurements between these points are assigned a relative elevation interpolated to the straight line between the end points. This portion of the test section has a negative cross slope, meaning that the outside edge of the lane is higher in elevation than the inside lane edge. This is reasonable since the test section is located on a beginning of a horizontal curve to the left in the direction of traffic and a negative pavement cross slope is the norm in this situation. Note that the deepest rut is in the inside wheel path.

The more significant issue is that the normalized rut depth at station 0+00 is a little over an inch, based on the lane-width wire line method. This is enough to hold perhaps .25 inches of water looking at the raw transverse profile plot.

Figure 11 shows the same transverse profile elevation information performed in November 2001 after placement of the nominal 2-inch mil and fill overlay in October 2001. As seen in the figure, the transverse profile is relatively flat and the rutting in the inside lane has been removed. This shows a very successful application of a mill and fill overlay treatment.

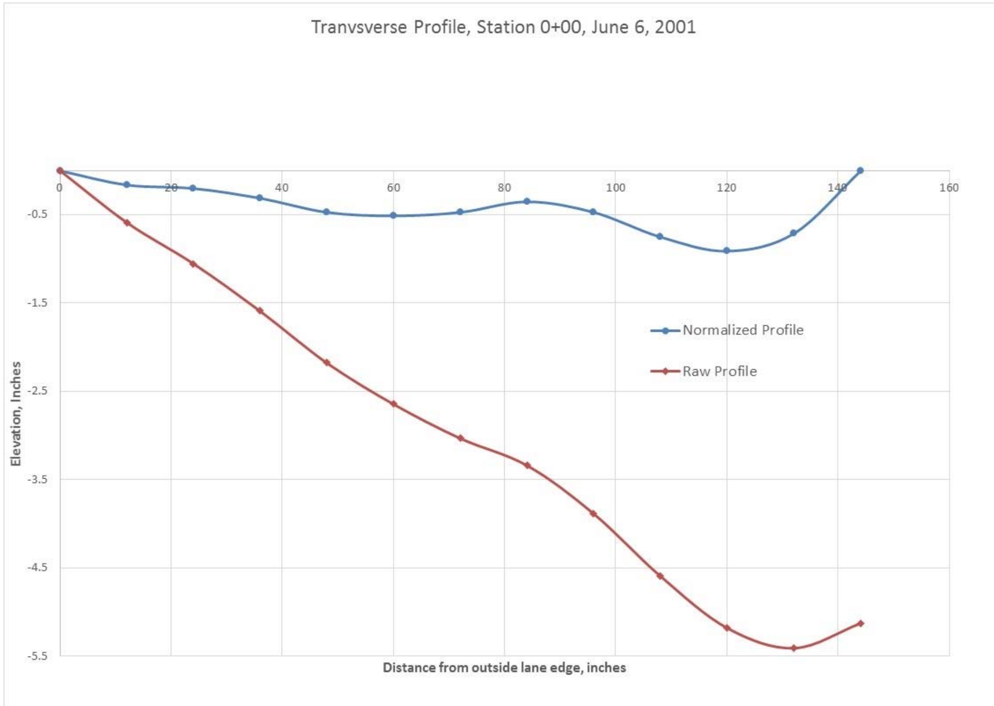


Figure 10. Normalized and raw transverse pavement profile at station 0+00 performed on June 6, 2001 prior to the mill and fill construction event.

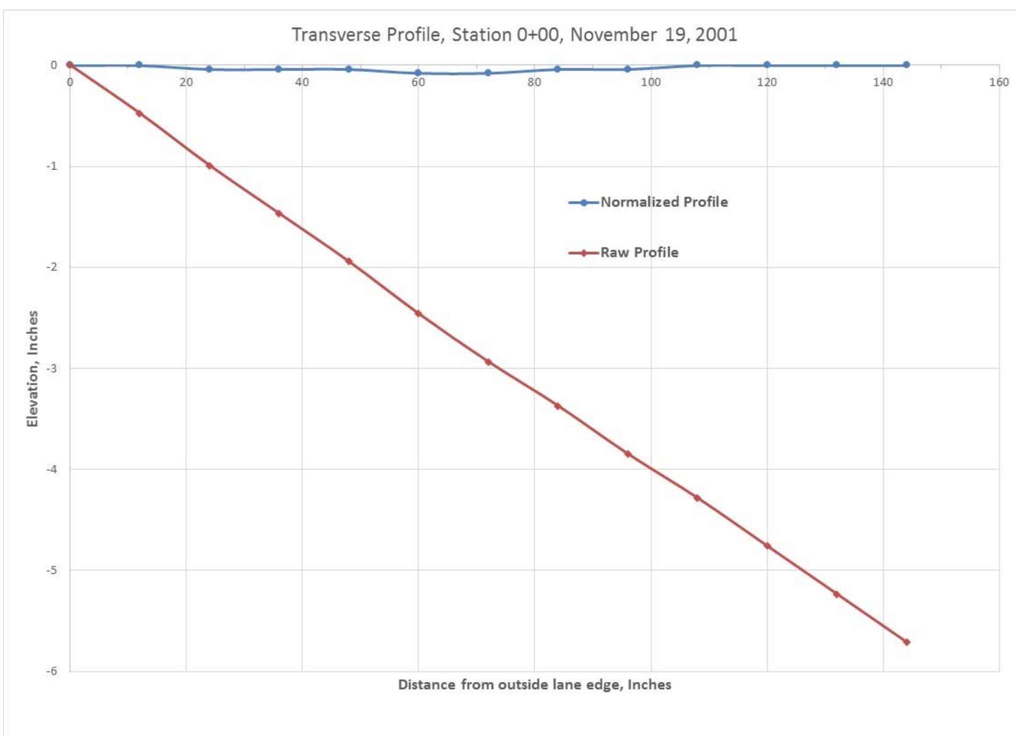


Figure 11. Normalized and raw transverse profile at stat 0+00 measured November 19, 2001, after the mill and fill overlay placed in October 2001.

Figure 12 shows an exaggerated vertical scale image of the normalized transverse profile plot in 2001 just after the mill and fill overlay event and the last transverse profile measurement at station 0+00 performed in 2013. Station 0+00 was selected for this plot since it had the deepest rut depth prior to the 2001 overlay. These measurements are more than 12 years apart. More significantly, the deformation patterns in the pavement surface now appear to be significantly different than those shown in Figure 10 where the most significant rut depths were those in the inside lane. This evidence suggests that the rutting phenomena is contained within the upper layers of the pavement structure, i.e. the overlay surface layers.

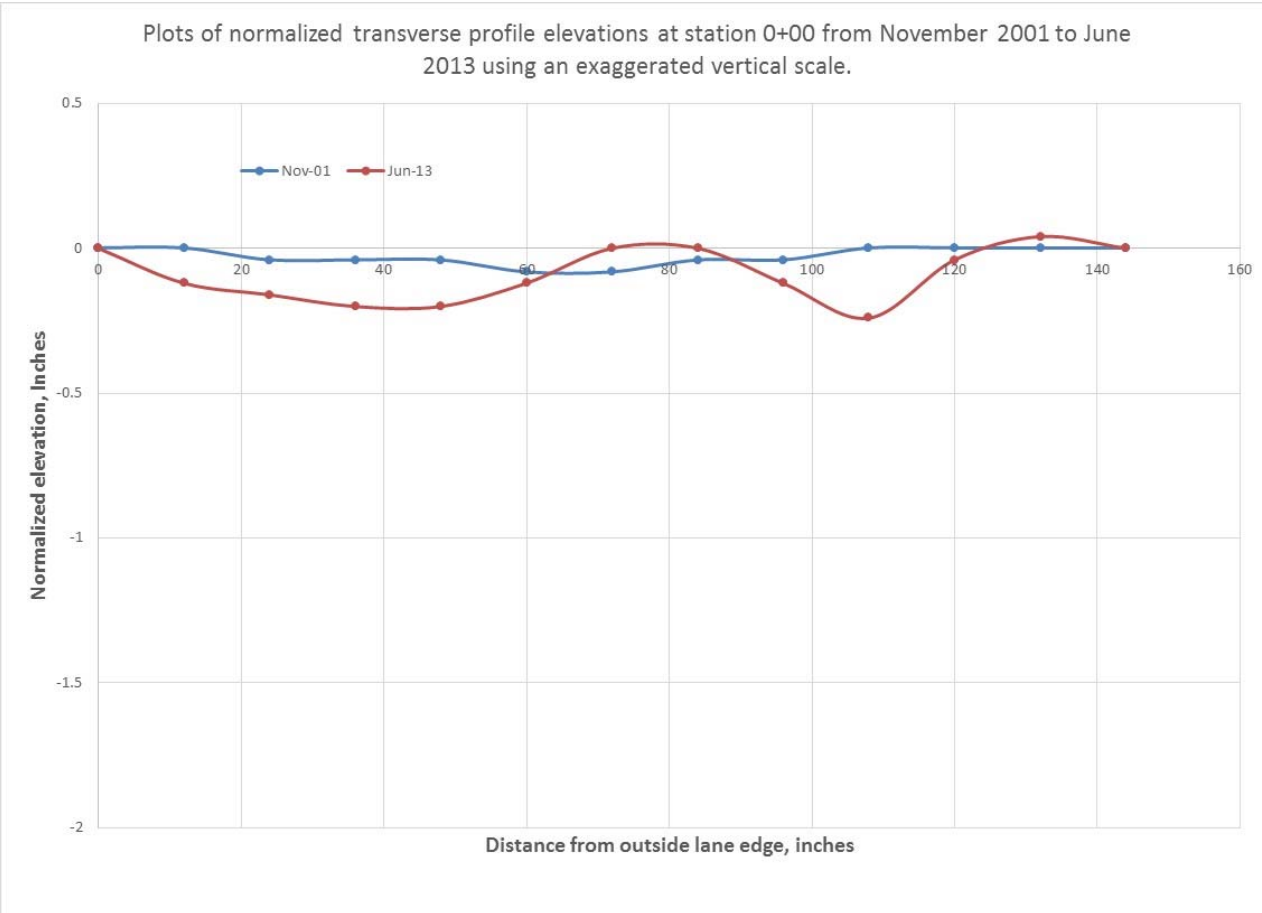


Figure 12. Exaggerated vertical scale of the normalized transverse profile at station 0+00 after the mill and fill overlay placed in October 2001.

SUMMARY OF FINDINGS

In this review of information concerning the performance history of test section 531005 the following information was presented:

- The pavement structure was initially constructed in 1973 as part of Interstate 90 in Washington State.
- Within the LTPP program research study, the test section was initially accepted into the

program as a GPS-1 test section in 1988 and has only one monitoring measurement prior to the overlay placed in 1989. The 1989 overlay placed this test section into the LTPP GPS-6B experiment classification, i.e., AC overlay of an AC pavement, no milling,

- This pavement has exhibited a good pavement performance history between construction events
  - The first pavement performance cycle of the original pavement structure on this test section existed 16 years before application of the first nominal 2-inch overlay in 1989. Transverse cracking appears to be the most prevalent distress manifestation in the original pavement structure prior to overlay.
  - The second pavement performance cycle consisted of an additional nominal 2.3-inch AC overlay with no milling added to the pavement 1989. This pavement structure lasted another 12 years before the next overlay in 2001. In this pavement performance cycle, rutting in the overlay appears to be the most critical distress manifestation on this test section. Transverse cracks existing in the pavement structure prior to overlay, appear to have reflected through this overlay over time.
  - In the third pavement performance cycle on this test section, which can be considered a mill and fill construction event, where the 2.3 overlay placed in 1989 was milled off and replaced with a nominal 2-inch thick AC surface occurred in 2001. This pavement structure has now lasted another 17 years, in good condition, with what might be considered minimal preservation treatments. Milling of the initial pavement overlay in 2001 appears to have reduced the rate of rutting and changed its location across the pavement structure.

In summary, this pavement section on a major interstate highway has performed well over its 45 years in service with just 2 minor overlay construction events. This appears to reinforce Washington State DOT practice of performing “preservation” and “rehabilitation” construction treatments in advance of the time normal PMS threshold limits are reached.

FIELD FORENSIC EALUATION RECOMMENDATIONS

Based on this investigative office review of available data, it appears that sufficient information is available to adequately explain the observed performance of the test section, which is summarized below:

1. The original pavement structure on an interstate highway performed well for 16 years.
2. The nominal 2.3-inch overlay layer placed during the CN=2 cycle, appears to have extended the pavement life by another 12 years. During this time frame the most significant distresses were transverse cracking and rutting. Since that layer was milled off in the next construction cycle, there is no way to test its material properties. This 2.3-inch overlay lasted more than 10 years in service on a major interstate highway, which is longer than most AC overlays this thin are expected to last.
3. Deflection measurements at this location show that the pavement structure and subgrade

are rigid, i.e. low deflections over every change in pavement structure. This suggests that there is a good pavement foundation that supports this pavement structure.

4. The second overlay, which was a nominal 2-inch mill and fill event, has now lasted 17 more years without exceeding "failure" pavement performance threshold limits. This again is a long performance cycle for such a thin overlay.
5. While the expected drop in IRI after the 2001 mill and fill overlay event was not observed, there are no field measurements that can be made at this time to determine why the expected drop in IRI after placement of the new overlay was not observed. It is possible that local WSDOT staff may be able to provide information that would explain the observed behavior. Accordingly, such information will be pursued as part of the follow-up investigations recommended at the end of this section, if such investigations are conducted.
6. This test section has been programmed to go out of study in the LTPP program after the next distress measurement performed by the LTPP. This decision was based on the section having consistent performance through 2 overlay cycles and the need to reduce the number of sections monitored due to budgetary cut backs in LTPP program funding.
7. Based on the photographs of the test section, there appears to be more distress in the adjacent left lane than in the LTPP study lane, however LTPP does not monitor the adjacent lane and this casual observation has no formal metrics to make a conclusion one way or the other if the left lane has accumulated more damage than the right lane. Again, it is possible that local WSDOT staff may be able to provide information that would explain the observed behavior. Accordingly, this information will be pursued as part of the follow-up investigations recommended at the end of this section, if such investigations are conducted.

In conclusion, while the performance of the test section and the reasons for it appear clear, the following two investigations are recommended to address specific performance issues not fully addressed by the available data or for which assumptions were made:

- A trench study – single location within the test section running the full transverse pavement width – is recommended to determine if the observed rutting is due to plastic deformation in the pavement layers or ablative wear on the surface potentially due to use of winter traction control devices, such as studded tires or chains.
- FWD testing performed at two distinct times on the same day – early in the morning and then mid to late afternoon – to confirm the assumption made earlier that the minor decrease in deflections in 2001 was due to the decrease in temperature from June (prior to overlay) to November (after overly). Also, since the last FWD testing was performed in 2013, the resulting data will help confirm the structural capacity of the pavement has remained "strong" as reflected by low measured deflections.

Attachment A to this memorandum details the results of the follow-up field investigations outlined above.

ATTACHMENT A: FOLLOW-UP FIELD INVESTIGATIONS

The follow-up field investigations were performed the week of April 1, 2019. The following activities were performed in the field:

- Five cores across the width of the pavement were obtained at stations 200 and 400. The cores were obtained in lieu of cutting a trench to look for indications of rutting in the AC surface layer.
- FWD testing was performed in early morning following standard LTPP FWD protocols. It was not possible, however, to obtain a second set of deflection measurements as originally planned.
- A manual distress survey was performed following standard LTPP distress protocols, including the LTPP Distress Identification Manual (DIM).
- Longitudinal, transverse profile and surface texture measurements were performed using the LTPP High Speed Survey vehicle and following standard LTPP profiling protocols.

CORE INSPECTION

At station 200, five 6-inch diameter cores were cores were obtained. Figure 12 shows the condition of the pavement at station 200 prior to coring. Patches can be seen in the wheel paths, which appear to be skin patches using something that approximates a slurry seal type of material. It was subsequently discovered based on discussions with WSDOT staff that they performed wheel path chip seals on this section in 2016.

Table 4 show the results of the core examination and thicknesses of the layers in the table from station 200 using LTPP core thickness measurement protocol. The LTPP layer thickness protocol requires thicknesses measured at 6 locations location around the circumference of a core, with the average thickness reported. The thicknesses from these cores show a uniform decrease in thickness moving from the right lane edge to the center line. This decrease in thickness appears to be attributed to layer 4, the deepest AC layer in the pavement structure. (Note that this pavement has a negative cross slope, where the outside edge has a higher elevation than the inside edge [See Figures 10 and 11]). While the 0.2-inch-thick patches in the wheel paths indicates that rutting was present, it is not clear from the cores if the mechanism was surface ablative wear from traction control devices or plastic deformation in the AC layers.

Figures 13 through 17 are images of the cores from station 200 obtained on April 3, 2019. The images were taken in a standard LTPP core box with scales at each side, and markings on the core to show individual layers within each core. The layer numbers as well as the change in layer strata are painted on each core. In general, the aggregate partials in all the layers look to contain many fractured faces, indicative of a crushed stone. The size of the aggregate in layer 4 looks to be larger than that in layer 5, while in turn appears to be larger than that in layer 7. The use of crushed stone in these pavement layers supports the general theory that the aggregate interlock provided by the angular interface from the crushed stone faces enhances the resistance to plastic deformation in the AC layers.

Table 4. Summary of core measurements at station 200.

Core ID	1A	1B	1C	1D	1E
Station	200	200	200	200	200
Offset (ft)	0.8	3.1	5.8	9.2	13
Wheelpath Chip Seal	0	0.2	0	0.2	0
Top AC Lift (Layer 7) (inch)	2.2	2.2	2.2	2.2	2.4
AC Lift (Layer 6) (inch)	0	0	0	0	0
AC Lift (Layer 5) (inch)	3.8	3.7	3.9	3.9	4
Bottom AC Lift (Layer 4) (inch)	6.3	6	5.7	5.1	4.7
Total Thickness (inch)	12.3	12.1	11.8	11.4	11.1

Figure 12. Image of station 200 prior to coring in 2019. The white dot paint marks are locations for FWD measurements.

In Figure 13, there is an obvious separation between layers 4 and 5. This is the core at the upper most edge of the test section, nearest the right edge longitudinal joint.

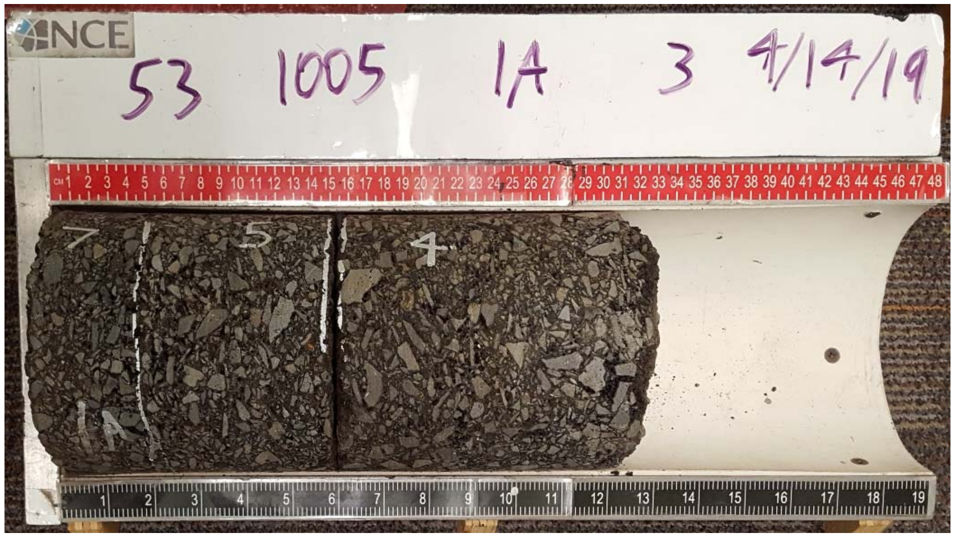


Figure 13. Image of core 1A obtained at station 200 at an offset of 0.8 feet from the outside lane edge.

Figure 14. Image of core 1B, obtained at station 200 at an offset of 3.1 feet. This core was taken from the right-side wheel path.

Figure 15. Image of core 1C, obtained at station 200 and an offset of 5.8 feet.

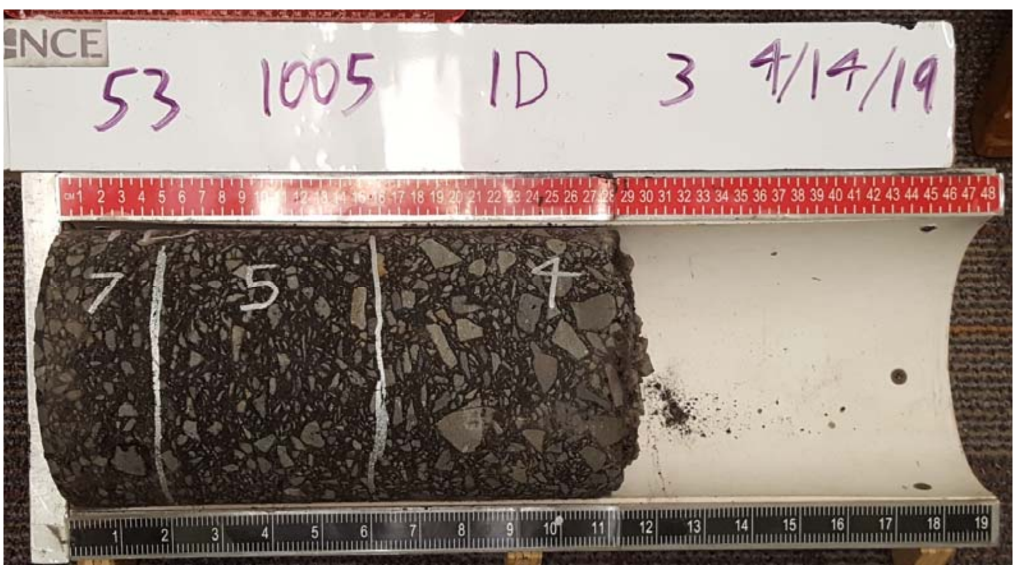


Figure 16. Image of core 1D, obtained at station 200 at an offset of 9.2 feet.

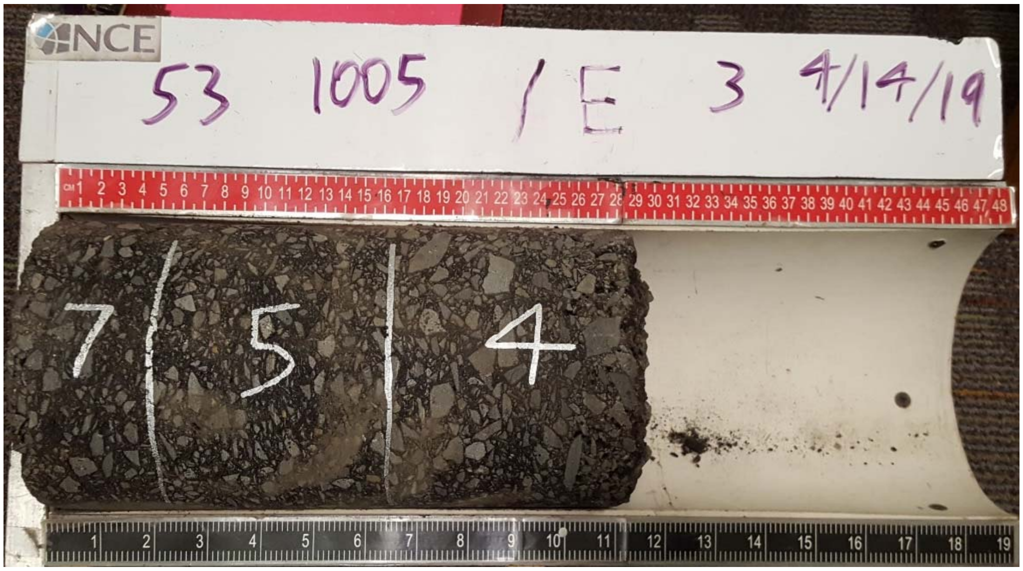


Figure 17. Image of core 1E, obtained at station 200 and on offset of 13.1 feet.

Figures 18 shows the condition of the pavement at station 4+00 where the other set of five cores was taken. The patches in the wheelpaths are visiable in the picture. Table 6 contains the results of core examinations from the five cores extracted at this location. Figures 19 through 23 show images of the cores obtained from this station.

Figure 18. Condition of the pavement at station 400 prior to coring. The white dots are FWD test locations.

Table 6. Summary of Core Thickness measurements at station 4+00.

Core ID	1A	1B	1C	1D	1E
Station	400	400	400	400	400
Offset (ft)	0.7	3	6	8.5	13
Wheelpath Chip Seal	0	0.2	0	0.2	0
Top AC Lift (Layer 7) (inch)	2.3	2.3	2.2	2.4	2.3
AC Lift (Layer 6) (inch)	0	0	0	0	0
AC Lift (Layer 5) (inch)	3.7	3.7	4.1	3.8	3.5
Bottom AC Lift (Layer 4) (inch)	6.7	6.4	5.5	5.7	5.97
Total Thickness (inch)	12.7	12.6	11.8	12.1	11.7

Figure 19. Image of core 2A obtained from station 400 with an effset of 0.7 feet.

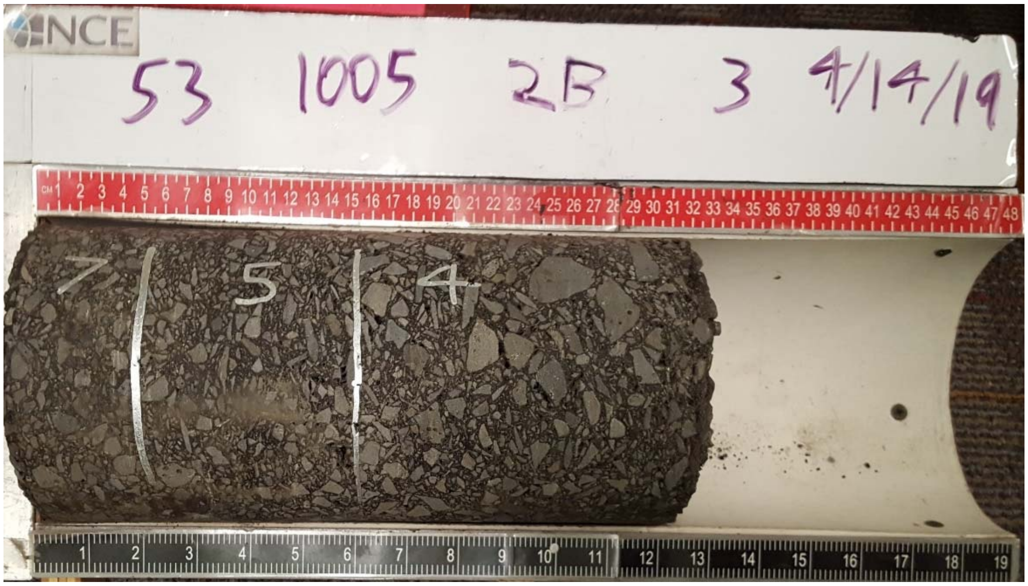


Figure 20. Image of core 2B from station 400 with an offset of 3-feet.

Figrue 21. Imge of core 2C from station 400 with an offset of 6 feet.

Figure 22. Image of core 2D from station 400 with an offset of 8.5 feet.

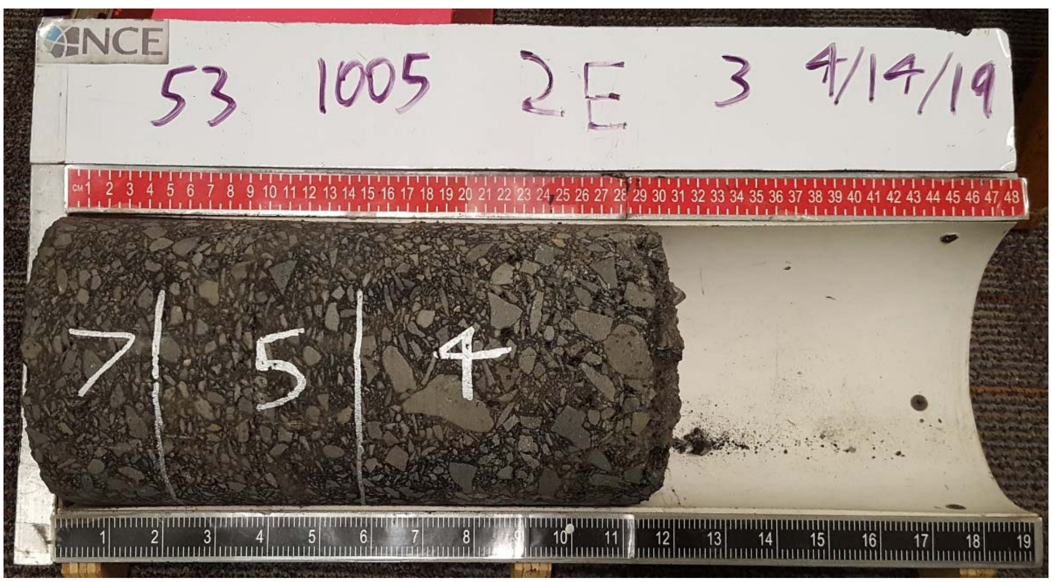


Figure 23. Image of core 2F, from station 400 with an offset of 13 feet.

PAVEMENT ROUGNESS

Figure 24 is the updated time history plot of pavement roughness, which includes breaks for rehabilitation and matinenance events. This is a plot of the combined mean IRI from both wheel paths. The new measurement in 2019 appears to show the effect of the skin patch placed in the wheel path circa 2016 on improving pavement smoothness.

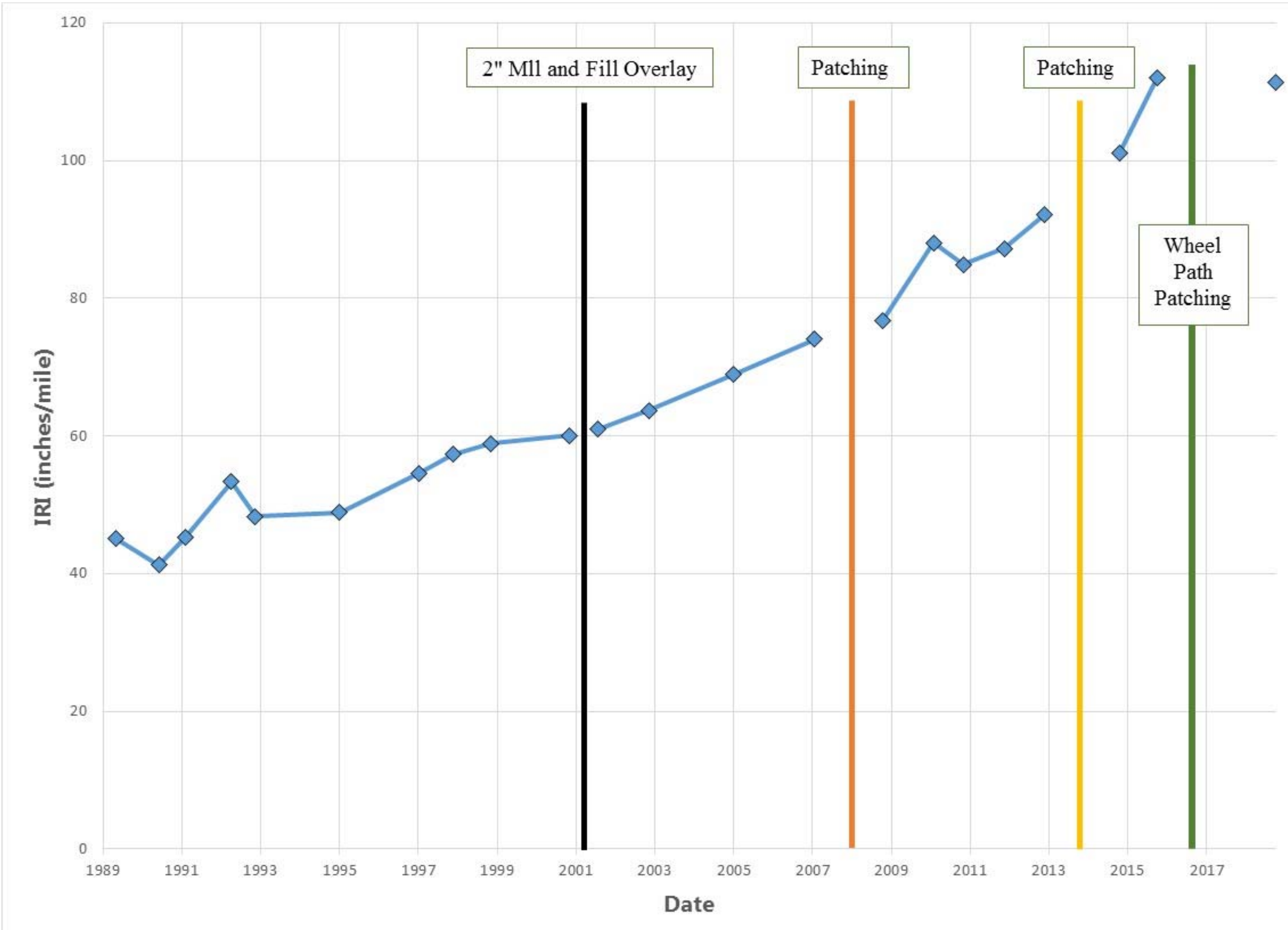


Figure 24. Updated pavement roughness time-history adding the April 2019 LTPP measurement. The wheel path skin patches appear to arrest the further increase in roughness, within the time frame of these measurements.

DEFLECTION TIME HISTORY

Figure 25 shows the updated FWD wheelpath average 9,000 lb level drop height measurements on section 53_1005. These are the average deflection measurements from the center of the load plate. As shown, the average deflection in 2019 is lower than the one in 2013, but it is still within the range of deflection variations on this test section over 20 years. Moreover, as with previous deflection measurements, the 2019 deflections confirm that this is a very stiff pavement structure, i.e. “strong” pavement structure.

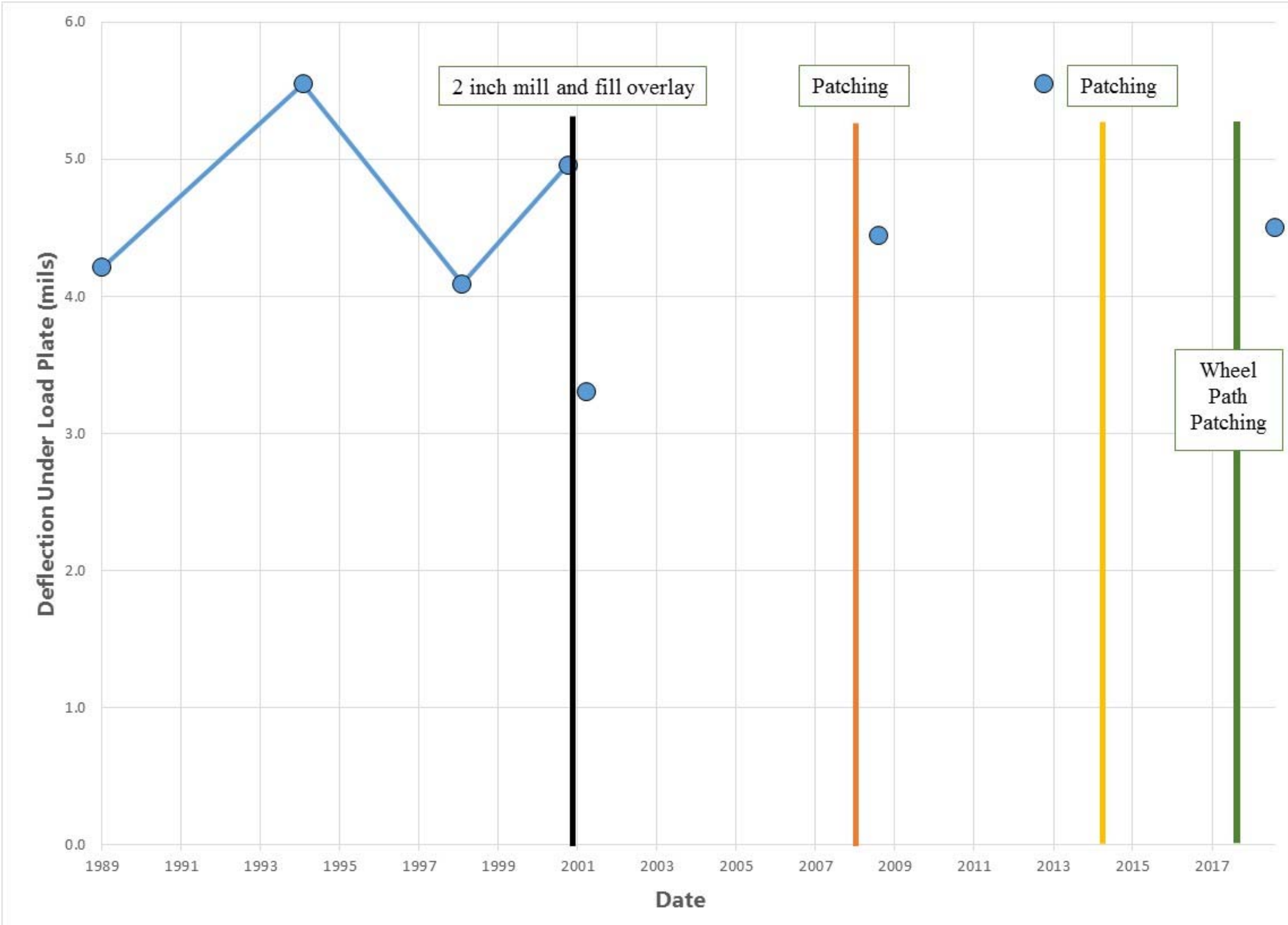


Figure 25. Average FWD wheelpath deflection at the 9,000-lb drop level over time.

TRANSVERSE PROFILE

The final set of transverse profile measurement were performed on April 5, 2019, using the new high-speed LTPP survey measurement vehicle. LTPP is in the transition stage of shifting to the new high-speed transverse profile measurements using laser-based technology, which can provide up to 2,000 elevation measurements at each station versus the older Dipstick method, which provides measurements at 1-foot intervals.

Figure 26 is a plot of the normalized transverse profile elevations at or within 1 foot of station 0+00 on section 53_1005. The plot shows the normalized transverse elevations starting in 2001 just after placement of the 2-inch mill and fill overlay and includes measurements through April 2009. Just after overlay placement in 2001, the maximum displacement in the surface profile, was less than 0.1 inches from a straight line through the lane's edges. The transverse profile distortions in 2001 look like differences in compaction due to movement of the compaction rollers over the AC mixture. The distortions in the elevation of transverse profile are relatively flat across the pavement profile. The level of displacement from a straight line is within the range of the thickness of a penny (.06 inches). The 2004 transverse profile measurements at this location show a deepening of the transverse profile by about another 0.05 inches in the wheel paths. By June 2013, the depressions in the wheel paths are more pronounced. The measurement in 2019 using the new laser transverse measurement technology, shows a significant increase in the deformation of the pavement surface, even after application of a skin patch in the wheelpaths in 2016.

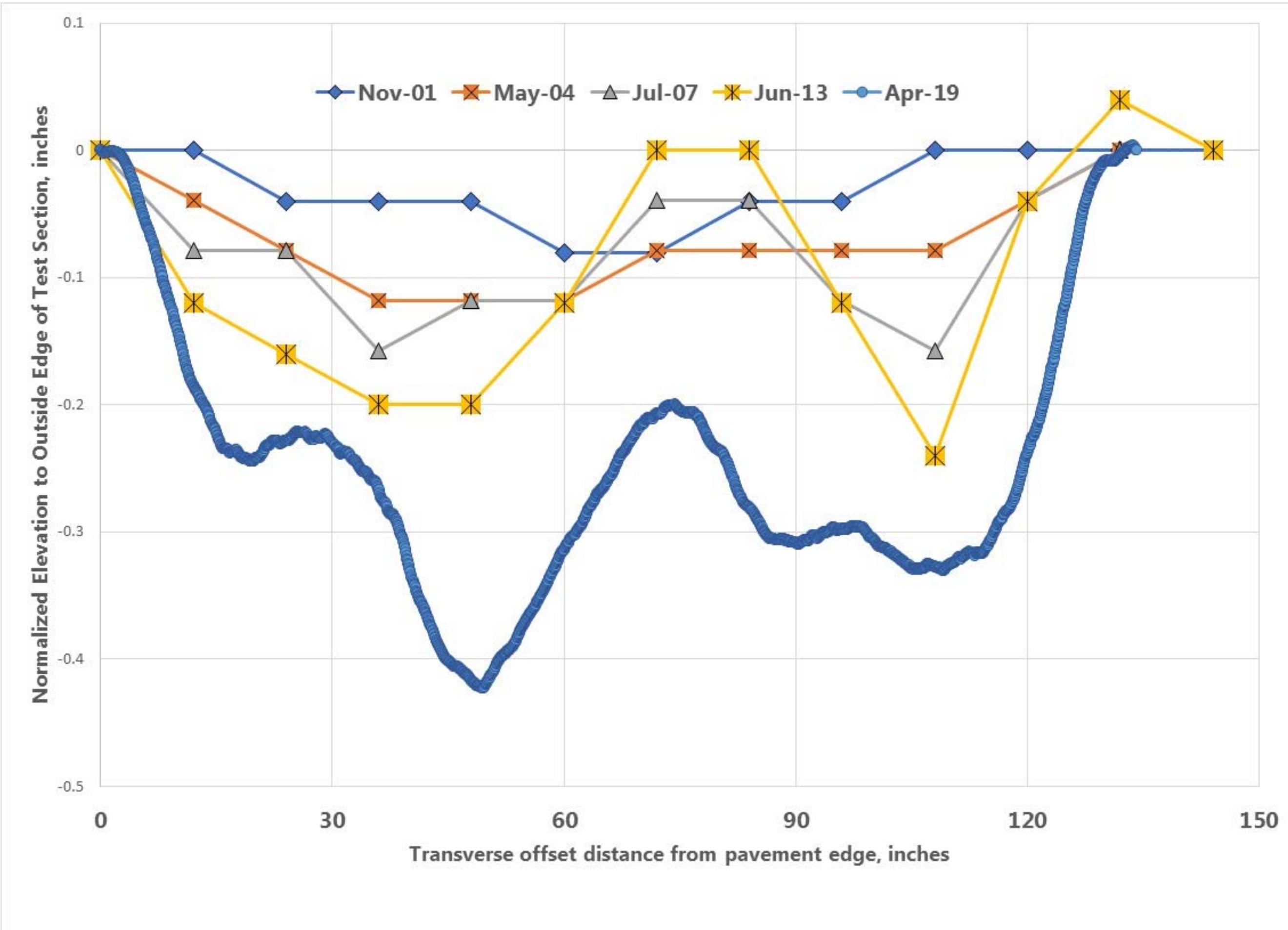


Fig 26. Comparison of historic normalized transverse profile plots at station 0+00 on section 53_1005.

The endpoint of the normalized transverse profile measurements should be noted. The June 2013 measurement went 13 feet wide, whereas most of the other measurements were only 12 feet wide. If the June 2013 measurement had been normalized to the 12-foot measurement width, this would have had the effect of moving down the normalized transverse profile elevations at most by 0.04 inches. This type of adjustment is in the range of a small stone sitting on the pavement surface.

While Figure 26 is at only one location in the test section, the application of the wheel path chip seals should have reduced the apparent rut depths by at least 0.2 inches based on the April 3, 2019 core thickness measurements.

The laser transverse profile measurement in 2019 shows differences in the profile from the historic transverse profiles measured in the past. This could be due to the increased number of measurements in 2019, which were approximately 0.08 inches apart. The LTPP program is currently assessing this issue.

Note that in Figure 26 the right wheel path is shown on the left side of the figure. This is because the outside edge of the pavement is assigned station zero.

RUT DEPTHS

Rut depths are computed using the normalized transverse profile data. Two types of ruts depths are computed by LTPP. One is the 6-foot straight edge method. In this method, a 6-foot straight edge is placed on the high points in each lane half, and the maximum distance from the bottom of the straight edge is computed. The second method is the lane width wire line. In this method a lane width wire line is simulated starting at the end points of the lane, resting on high points that extend above the end points. The maximum distance from the bottom of the wire line on each side of the lane are then computed.

Figures 27 and 28 shows the time history of the average rut depths, in the left and right wheel paths over the entire pavement section for the 6-foot straight edge and wire line methods, respectively. Breaks in the plot are shown for the application of overly and maintenance treatments.

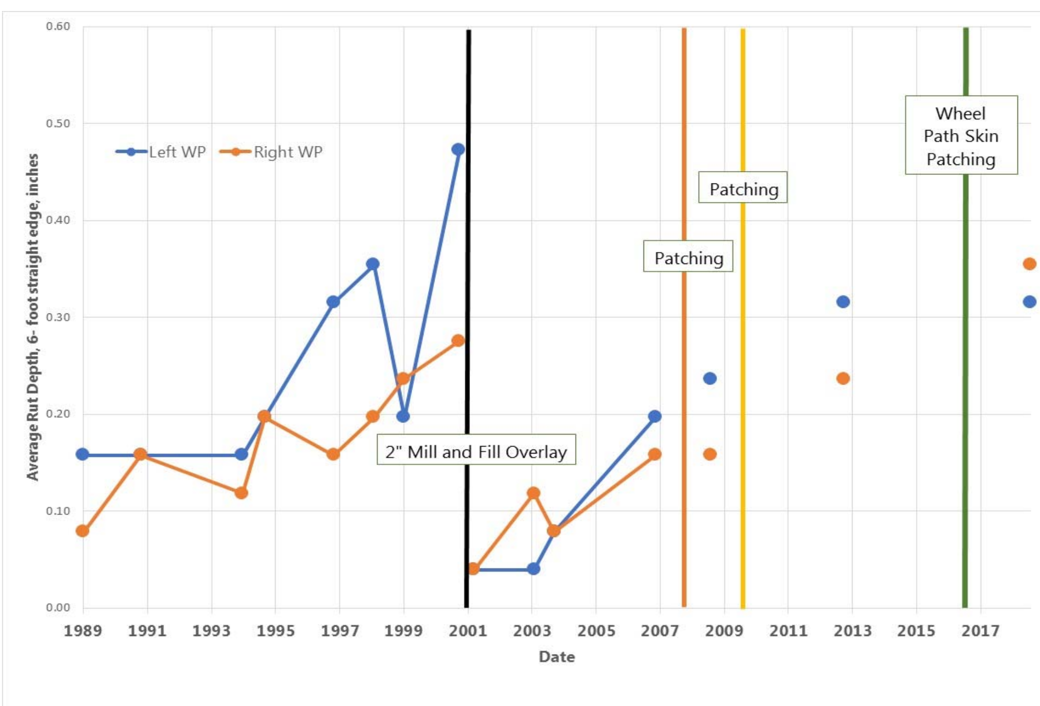


Figure 27. Time history plot of average rut depths in left and right wheel paths based on the 6-foot straight edge measurement method.

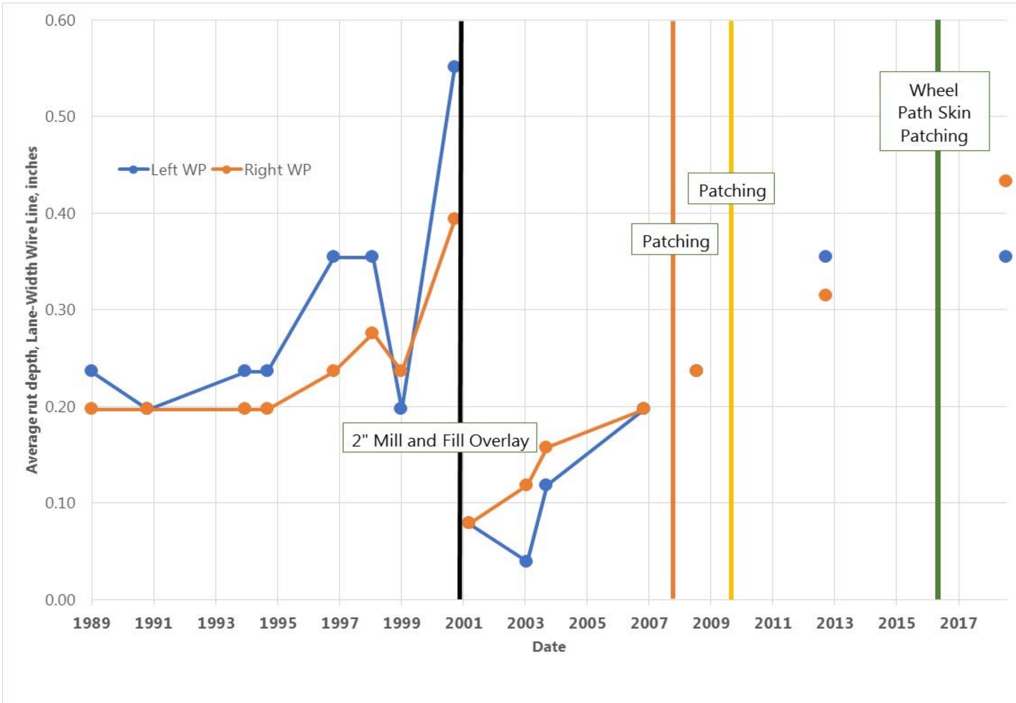


Figure 28. Time history plot of average rut depths in left and right wheel paths based on the lane width wire line edge measurement method.

The time history average rut depth shows the influence of the rehabilitation and maintenance treatments. The biggest change is the 2001 2-inch mill and fill overlay. The calculated ruts depths just after construction appeared to be more related to rolling patterns since the transverse profile is relatively flat with plateaus that match typical compaction roller widths. From 2001 there is a gradual increase in rut depths and the patching events 2008 and 2013 did not appear to slow down the rutting rate. It is surprising that the wheel path skin patches placed in 2016 kept the rutting in the left wheel path the same, but rutting in the right wheel path increased by a 0.1 inch when comparing the 2013 and 2019 measurements.

The 6-foot straight edge and wire line rut depths can be different depending on the shape of the normalized transverse profile. When the transverse profile has a bowl shape, such as shown in for 2019 measurements in Figure 26, the wire line method will show deeper ruts because the reference line connects the ends of the transverse profile bridges most of the other high points in the transverse profile. This can also be seen in Figures 27 and 28, where the 2001 ruts depth just prior to the overlay shows the wire line value is greater than that from the 6-foot straight edge. This can also be observed in the 2019 measurements, which were taken prior to planned rehabilitation of the test section by the WSDOT.

SUMMARY OF OBSERVATIONS FROM THE FOLLOW-UP STUDY

The following observations are based on the follow-up field investigation on LTPP test section 53_1005:

- The core examinations did not display any identifiable trend in AC layer thickness that would provide an explanation of what pavement structure layer the rutting was occurring. Even accounting for the thin wheel path skin patches, the cores at station 2+00 showed a consistent linear pattern in decrease in thickness from the outside lane edge to the inside lane edge. The cores of the AC layers at station 4+00 show the thinnest core from the middle of the lane. The influence of the 2016 wheel path patch appears to be more prevalent in these core thickness measurements.
- Since trenches were not used, there is no direct observation of deformation in the base and subgrade layers on this test section.
- The low deflections from the 2019 FWD measurements indicate a very strong test section and little variability over the years that LTPP measurements have been performed on this site. One inference from this observation is that the support from the base and subgrade layers are not changing too much over time. This implies that densification of the subgrade, i.e. volume changes are not occurring in the base and subgrade that would indicate they are contributing to pavement rutting.
- The interesting part of the IRI pavement roughness time history plot is that the 2001 2-inch mill and fill overlay and patches do not appear to have had significant impacts. The observation of roller compaction issues from the transverse profile measurements after construction of the 2001 overlay might contribute to the why. However, there is a steady increase in IRI on this test sections starting in 1993 and continuing through 2013. Only the wheel path chip seals placed in 2016 appears to have resulted in a reduction in pavement roughness as observed in the 2019 measurements.
- The best evidence available for the cause of rutting on this test sections comes from the transverse profile measurements. In Figure 26, the differences in the transverse profile of the surface of this pavement appear to be due to gradual eroding of the pavement surface. While the depths of the elevation in the wheel path have decreased, the depths in mid-path portion of the pavement structure have also decreased. This is the type of behavior that would be expected from test sections that are snow covered for portions of a year and traffic wandering when drivers cannot see the edges of the lane and use of traction control devices is the greatest. This is one of the more significant truck routes in Washington. The measurements reveal more rapid degradation in the wheel paths, which is consistent with the above theory.
- The above observation is supported by the wire line rut depths becoming greater than the 6-foot straight edge rut depths as the pavement aged.

In summary, based on the evidence collected as part of the follow-up field investigations, it appears that ablative wear in the wheel paths may be the causative factor that explains the overall performance of this test section. Furthermore, the findings from the field investigations further confirm and support the two major conclusions from the desktop study, which are:

- The pavement test section has performed well over its 45 years in service with just two minor overlay construction events, which appears to reinforce WSDOT practice of performing "preservation" and "rehabilitation" construction treatments in advance of the time normal PMS threshold limits are reached.
- Sufficient information is available to adequately explain the observed performance of the pavement test section.