Technical Memorandum

To: Jeff Uhlmeyer

From: Gonzalo Rada, Gary Elkins, Nick Weitzel, and Kevin Senn

cc: Mustafa Mohamedali

Date: September 28, 2019 (original), September 15, 2020 (Revised)

Re. Forensic Desktop Study Report: California LTPP Test Section 06_7452

The Long-Term Pavement Performance (LTPP) General Pavement Studies (GPS) test section 06_7452[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations" to investigate the 43-year performance of an asphalt concrete (AC) over bound base pavement test section that has received two AC overlays. The test section appears to have performed well when viewed in terms of IRI, rutting and deflections, but not so in terms of longitudinal and transverse cracking; this cracking appears to have driven the application of the AC overlays. The focus of the investigation is on the cracking of the test section, especially the amount of transverse cracking prior to application of the first AC overlay (on average, one transverse crack every 5 feet). Given the test section location, this cracking does not appear to be a low-temperature related mechanism.

SITE DESCRIPTION

LTPP test section 06_7452 is located on State Route 29, northbound at milepost 44.5 in Lake County, California. State Route 29 is a rural minor arterial with two lanes in the direction of traffic. It is classified as being in a Wet-No Freeze climate zone with an average annual precipitation ranging between 9.8 inches (2013) and 74.8 inches (1983) and an annual average air freezing index ranging between 0 Deg-F degree-days (most years) and 23 Deg-F degree-days (1990) during the performance period in question of 1972 to 2017. The coordinates of the test section are 39.0766, -122.93076. Photograph 1 shows the test section at Station 0+00 looking north in 2015, while Map 1 shows the geographical location of the test section relative to the City of Sacramento within the State of California.

The test section is located one mile to the north west of Clear Lake. Clear Lake currently has a 68 square miles of surface area. It is the oldest lake in North America, and the site has been the location of lakes dating back more than 2,500,000 years.[2]

[1] First two digits in test section number represent the State Code [06 = California]. 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).
[2] https://en.wikipedia.org/wiki/Clear_Lake_(California)#cite_ref-:0_3-1

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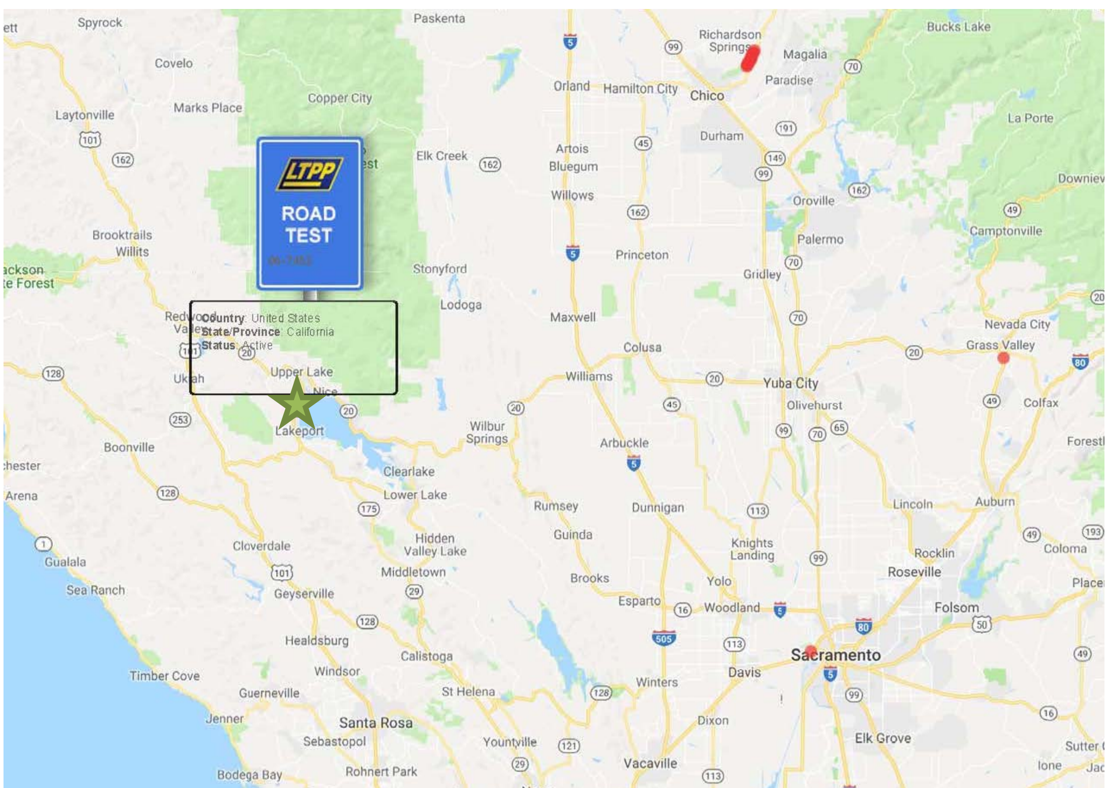
Photograph 1. Picture of test section 06_7452 in 2014 (from start of section looking east).



Map 1. Geographical location of test section relative to Sacramento, California.

BASELINE PAVEMENT HISTORY

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

Pavement Structure and Construction history

The initial pavement structure was constructed in 1972, and it was incorporated into the LTPP program in 1989 as part of the GPS-2 (AC over Bound Base) experiment. The original layer structure is detailed in Table 1. This corresponds to CONSRTUCTION_NO = 1 (CN = 1).

Table 1. Pavement structure from 1972 to 1999.

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		114-Fine-Grained Soils: Sandy Lean Clay
2	Unbound (granular) subbase	9.8	302-Gravel (Uncrushed)
3	Bound (treated) base	6.7	334-Lean Concrete
4	Asphalt concrete layer	3.4	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	0.5	71-Chip Seal

A 4.0-inch AC overlay was applied to the test section in July 1999, which corresponds to CN = 2, and it was moved to the GPS-6B (Planned AC Overlay of AC Pavement) experiment. In August 2010, milling and another AC overlay was applied to the test section, which corresponds to CN = 3, and it was moved to the GPS-6C (Modified AC Overlay of AC Pavement) experiment. The resulting CN = 2 and CN = 3 pavement structures are detailed in Tables 2 and 3, respectively. As shown in Table 3, the 2010 mill and overlay effectively added 1.2 inches of AC to the pavement structure.

Table 2. Pavement Structure from 1999 to 2010.

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		114-Fine-Grained Soils: Sandy Lean Clay
2	Unbound (granular) subbase	9.8	302-Gravel (Uncrushed
3	Bound (treated) base	6.7	334-Lean Concrete
4	Asphalt concrete layer	3.4	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	0.5	71-Chip Seal
6	Asphalt concrete layer	2.6	1-Hot Mixed, Hot Laid AC, Dense Graded
7	Asphalt concrete layer	1.4	1-Hot Mixed, Hot Laid AC, Dense Graded

Table 3. Pavement Structure from 2010 to date.

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		114-Fine-Grained Soils: Sandy Lean Clay
2	Unbound (granular) subbase	9.8	302-Gravel (Uncrushed
3	Bound (treated) base	6.7	334-Lean Concrete
4	Asphalt concrete layer	3.4	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	0.5	71-Chip Seal
6	Asphalt concrete layer	2.6	1-Hot Mixed, Hot Laid AC, Dense Graded
7	Asphalt concrete layer	1.4	1-Hot Mixed, Hot Laid AC, Dense Graded
8	Asphalt concrete layer	1.2	1-Hot Mixed, Hot Laid AC, Dense Graded with partial milling

Pavement Structural Properties

Figure 1 shows the time history average FWD deflection plot under the nominal 9,000 lb. load from tests performed in the wheel path for the deflection sensor position in the center of the load plate. 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. As shown, the original pavement structure had maximum deflections in the 6 to 8 mils range, which decreased after the first overlay to between 5 and 6 mils, and which again decreased after the second overlay to between 4 and 5 mils. These deflections are relatively low and appear reasonable for a pavement structure with a strong lean concrete base – i.e., not as low as those for a typical PCC pavement, but not as high as a typical AC pavement.

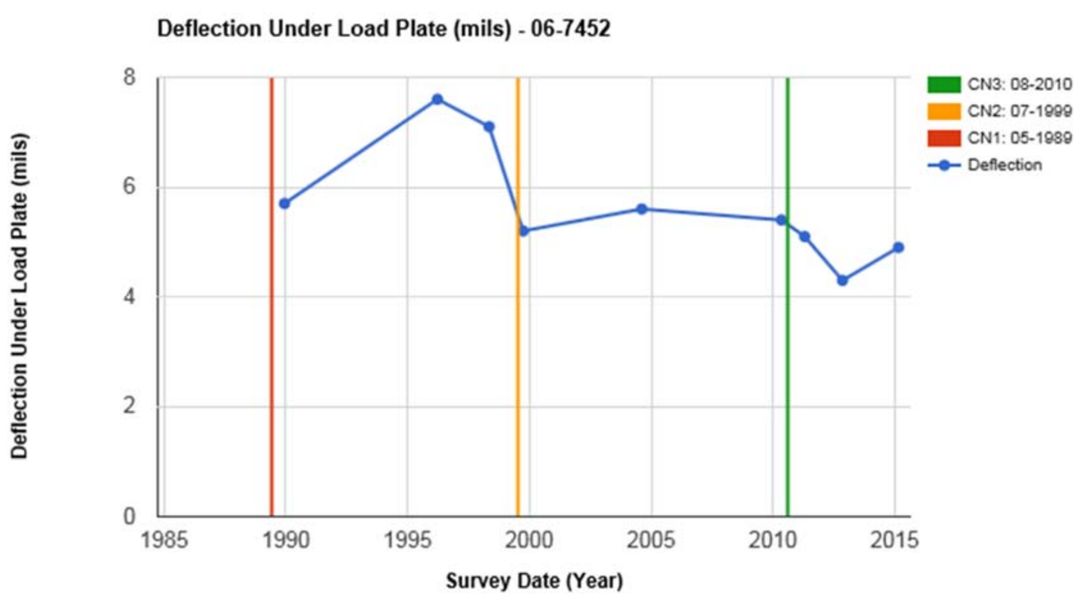


Figure 1. Time history of average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load.

Table 4 shows the layer moduli backcalculated (using EVERCALC 5.0 software) from the deflection data measured between December 1989 and April 2011; i.e., seven rounds of FWD testing – three under CN = 1. Three under CN = 2 and one under CN = 3. The pavement structure was modeled as consisting of an AC layer whose thickness varied depending on CN, over 6.7 inches of lean concrete, 9.8 inches of unbound granular subbase, 24 inches of top subgrade, and semi-infinite subgrade.

Table 4. Backcalculated layer moduli over time.

Layer Type	Thickness (inches)	Test Date	Modulus (ksi)
Asphalt Concrete	3.9 (CN = 1)	12/11/1989	1,018
		03/22/1996	1,016
		05/06/1998	2,156
	7.9 (CN = 2)	09/28/1999	764
		08/11/2004	366
		05/06/2010	718
	8.4 (CN =3)	04/19/2011	769
Lean Concrete Base	6.7	12/11/1989	2,382
		03/22/1996	791
		05/06/1998	747
		09/28/1999	504
		08/11/2004	1,210
		05/06/2010	685
		04/19/2011	634
Unbound Granular Subbase	9.8	12/11/1989	43
		03/22/1996	35
		05/06/1998	30
		09/28/1999	30
		08/11/2004	22
		05/06/2010	36
		04/19/2011	30
Subgrade (top)	24.0	12/11/1989	21
		03/22/1996	26
		05/06/1998	22
		09/28/1999	34
		08/11/2004	19
		05/06/2010	25
		04/19/2011	26
Subgrade (bottom)	Semi-Infinite	12/11/1989	61
		03/22/1996	42
		05/06/1998	45
		09/28/1999	46
		08/11/2004	45
		05/06/2010	45
		04/19/2011	46

As shown in Table 4:

- The backcalculated modulus values for the AC layer appear to be high for CN = 1, but reasonable for CN = 2 and 3. The former may be due to the thickness of the AC layer (i.e., 3.9 inches) for CN = 1 – backcalculation software often have a more difficult time analyzing thinner layers, which is often influenced by the location of the geophones. This is further compounded by the presence of the stiff lean concrete layer underneath, which violates an important linear elastic assumption in the backcalculation software – i.e., stiffness of layer on top is higher than that of layer underneath.
- The backcalculated layer moduli for the lean concrete appears reasonable if the layer has developed a significant amount of transverse cracks. The 1989 modulus value seem high for a lean concrete layer constructed 17 years earlier, in 1972, but it is possible that the extent of transverses cracking had not yet fully developed (which is also supported by the transverse cracking data presented later in the memorandum) and therefore was not as high as in the ensuing years – seven years passed until the next round of deflection testing was performed in 1996.
- The backcalculated layer moduli for the unbound granular subbase, top subgrade and semi-infinite subgrade appear to be reasonable. However, it is somewhat surprising that the values remain so stable over the 22 years of deflection testing (1989 to 2011), despite pavement deterioration and changing climatic conditions. Further investigation into the stability of these layer moduli is warranted and it should consider the effects of climatic, subsurface moisture conditions, and stress sensitivity.

Climate History

The time history for annual average precipitation since 1973 is shown in Figure 2. In 1983, the high amount of precipitation was recorded (74.8 inches) at the site, while the low (9.8 inches) was recorded in 2013. These measurements deviate significantly from the mean at the site (38.1 inches for time period in Figure 2); however, performance data are only available starting in 1989 and limited data are available after 2013, hence it is difficult to assess the impact of these two precipitation extremes on pavement distresses or pavement response at the test section.

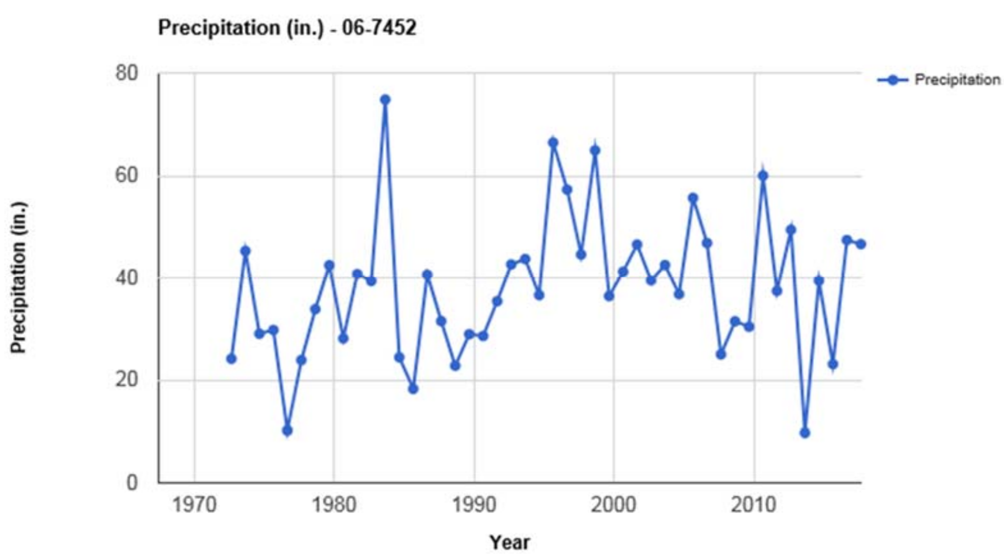


Figure 2. Time history of annual precipitation.

Figure 3 shows the time history of the annual freezing index over the history of this test section. 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. Except for minor spikes in 1972, 1989 and 1990, the annual freezing index at the site has remained at or close to zero, which implies low temperature issues are not major contributors to the transverse cracking observed on the test section.

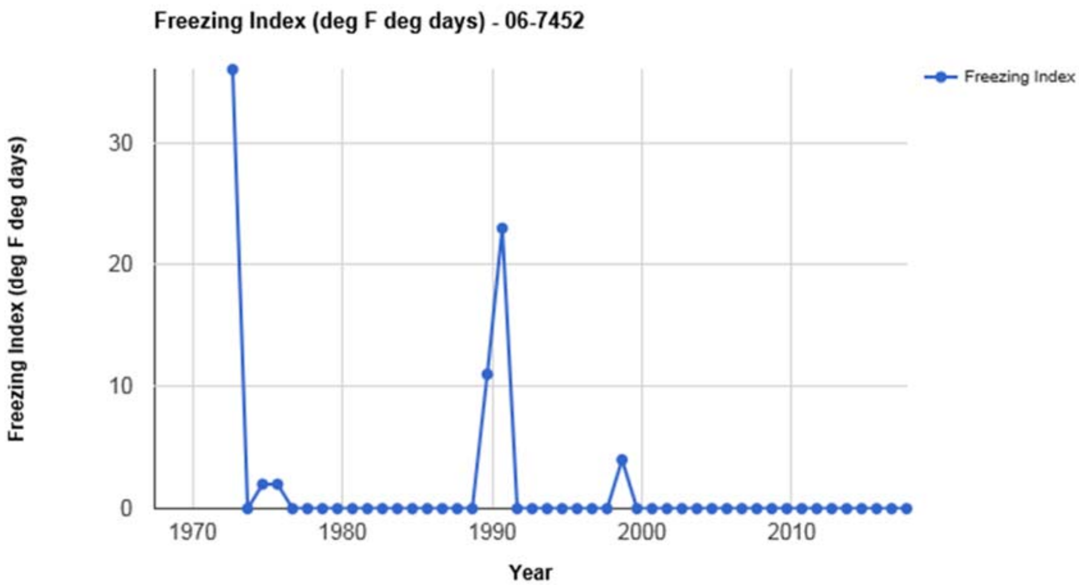


Figure 3. Time history of annual air temperature freezing index.

Truck Volume History

Figure 4 shows the annual truck volume data in the LTPP test lane by year. The red triangles are data provided by the California Department of Transportation (Caltrans) from historical records. The blue diamonds are truck counts derived based on monitoring data reported to LTPP by Caltrans. While not perfect, there appears to be agreement between the two counts. The figure also shows that truck volumes have steadily increased from around 100 trucks per day in the mid-1970s to around 400 trucks per day in the mid-2010s; i.e., it has quadrupled, but nonetheless they are low volume counts.

Pavement Distress History

This section summarizes the distresses observed at the test section during the period of 1989 to 2015 (CN =1, 2 and 3), which is when the last round of measurements was performed. The pavement surface cracking histories shown in the ensuing figures include data derived from automated distress surveys (35-mm Black & White continuous photographs) performed in 1989, 1991, 1993, 1995 and 2002, and from ten rounds of manual distress surveys between 1996 and 2015, the year of the last survey. Both data sets were included in this desktop study since there were no manual distress surveys prior to 1996, as automated distress surveys were the primary method in use by LTPP in the early years of the program.

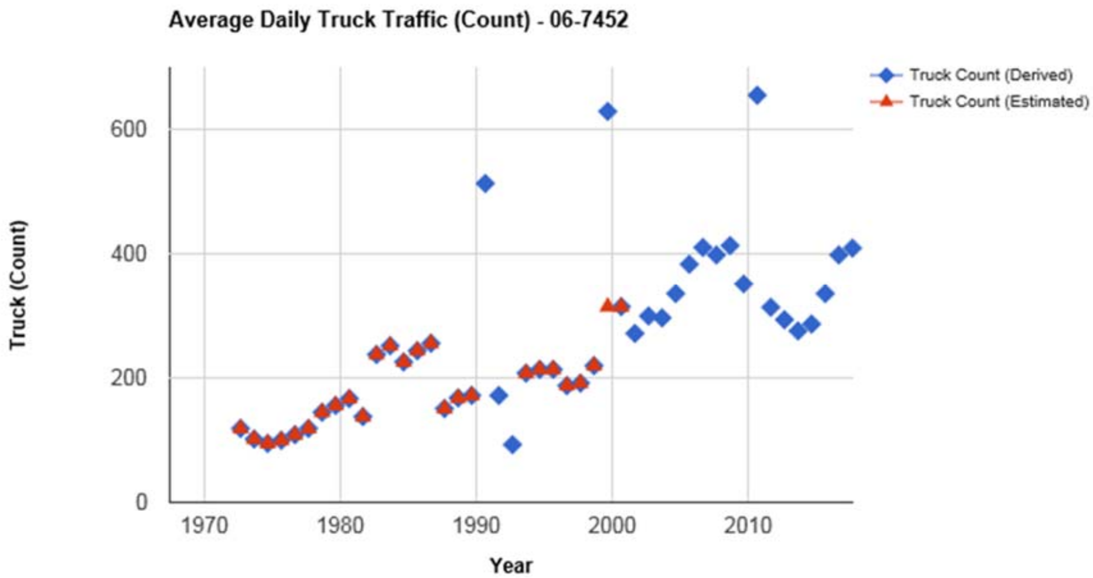


Figure 4. Average annual daily truck traffic history.

Figures 5 and 6 show the time history of the number and length of transverse cracks. As shown in these two figures, the number of transverse cracks increases from 39 in 1989 to 100 in 1998, while the length of that cracking increases from 195 ft to 610 ft over the same time period. This would support the earlier theory that the backcalculated layer moduli for the lean concrete appears reasonable if the layer has developed a network of transverse cracks, which appears to be the case. It also supports the theroy that the 1989 modulus value is high because the extent of transverses cracking had not yet fully developed, which again is supported by the transverse cracking data.

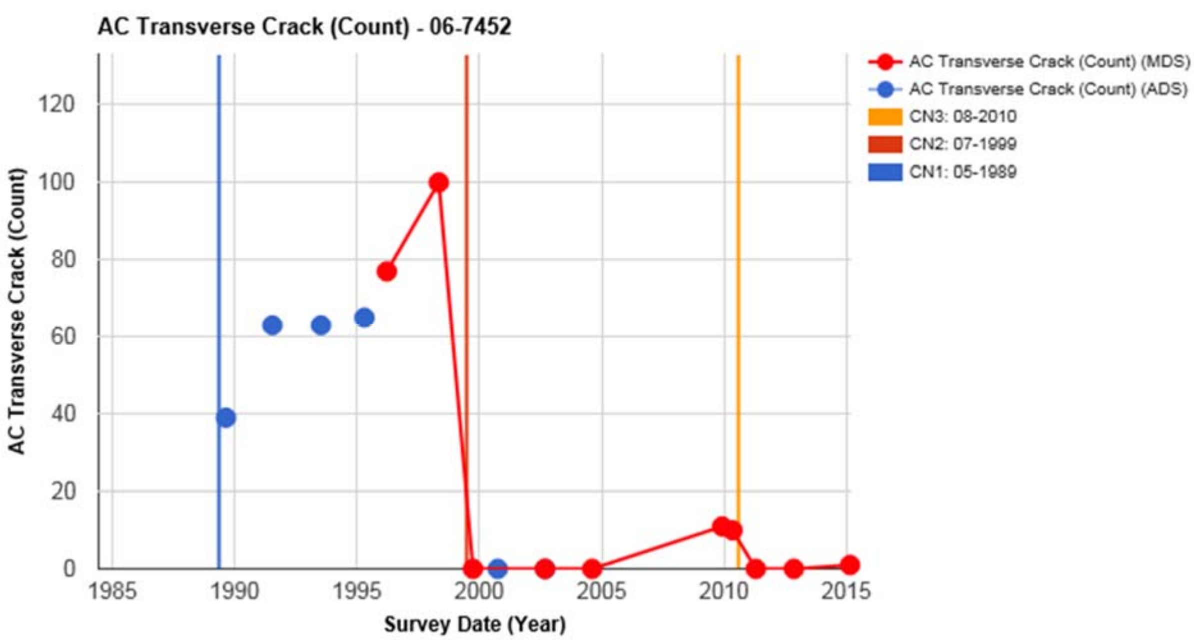


Figure 5. Time history of the number of transverse cracks.

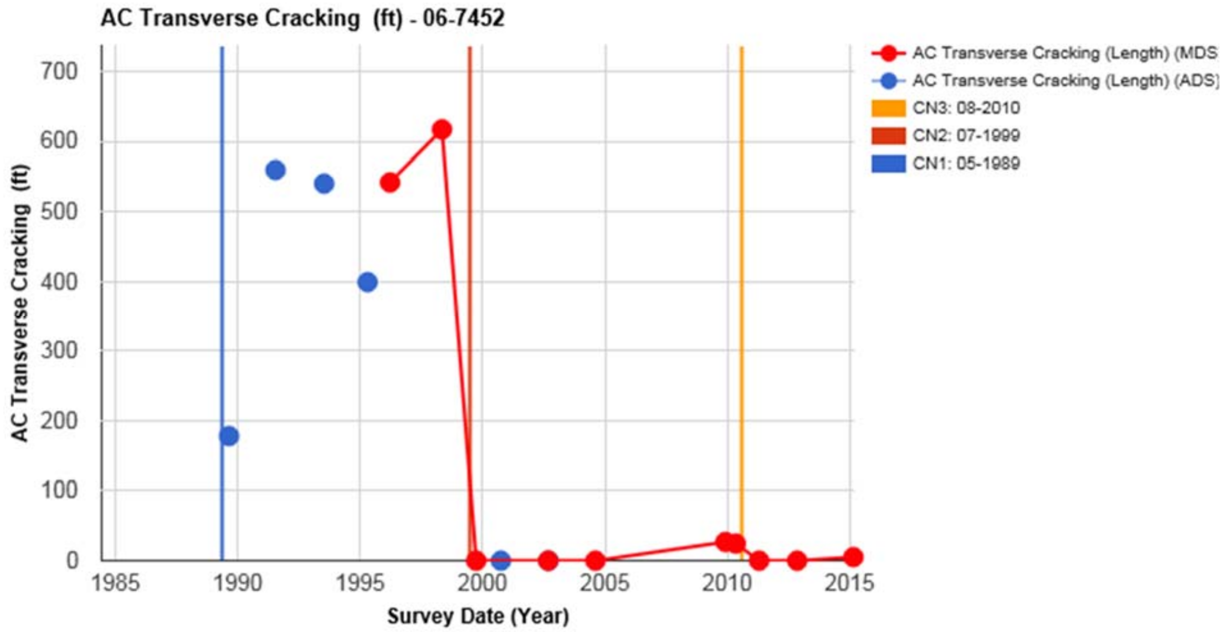


Figure 6. Time history of the length of transverse cracks.

Figures 5 and 6 also show that the amount of transverse cracking remained very low after the applciation of the 1999 AC overaly; i.e., the transverse cracking did not reflect through either the 1999 or 2010 AC overlays. This raises an important issue that, if possible, needs to be investigated: why did transverse cracking reflect from the lean concrete base through the original AC surface layer, but not the overlays? One option, for example, would be to core at locations where transverse cracks were observed at the end of the CN =1 period, but where transverse (reflection) cracks were not obseved after placement of the first AC overlay in 1999.

Figure 7 is the hand drawn distress map of the test section in 1998 prior to the application of the first overlay. This map illustrates the amount of transverse cracking on the test section. There is also a significant amount of non-wheel path longitudinal crack near the left edge of the test lane; this distress type is addressed later in the memorandum.

Figure 8 shows the time history plot of alligator cracking data. 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. For the CN = 1 period, alligator cracking increased from 25 ft^2 in 1989 to 182 ft^2 in 1991, then decreased of 30 ft^2 in 1993 and to zero in 1995, then increased to 50 ft^2 1996 and back to zero in 1998. When this trend is compared with the trend in wheelpath (WP) longitudinal cracking shown in Figure 9 it is apparent that during this time period the rating of these distress types were changing within the LTPP program. During the same time period, the WP longitudinal cracking decreased from 149 ft in 1989 to 35 ft in 1991, then increased to 198 ft in 1993, then decreased to 175 ft in 1995, then increased to 235 ft in 1996 and 431 ft in 1998 – almost a reverse image to the alligator cracking trend.

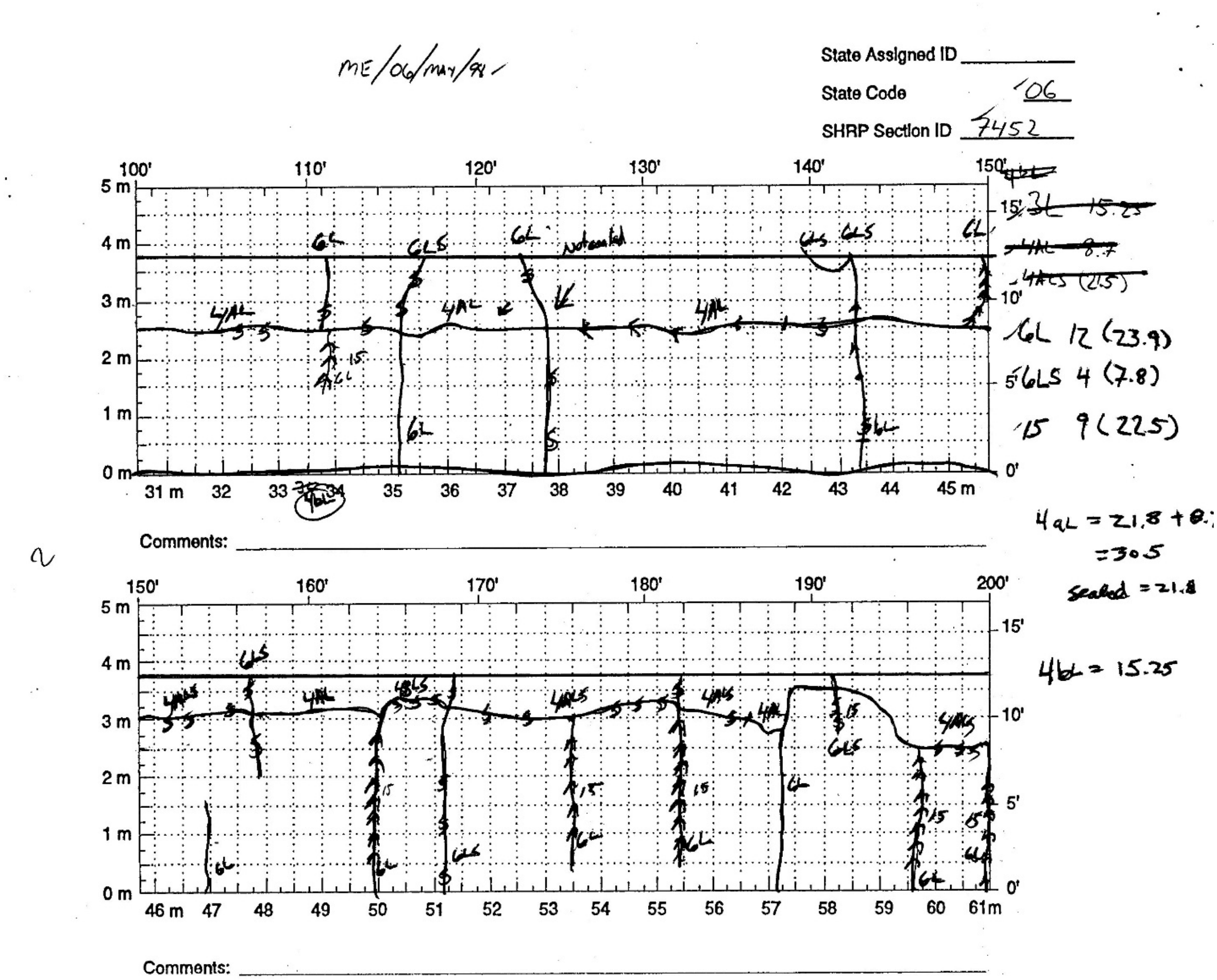


Figure 7. May 1998 Distress map from station 100 to 200 on test section 06_7452.

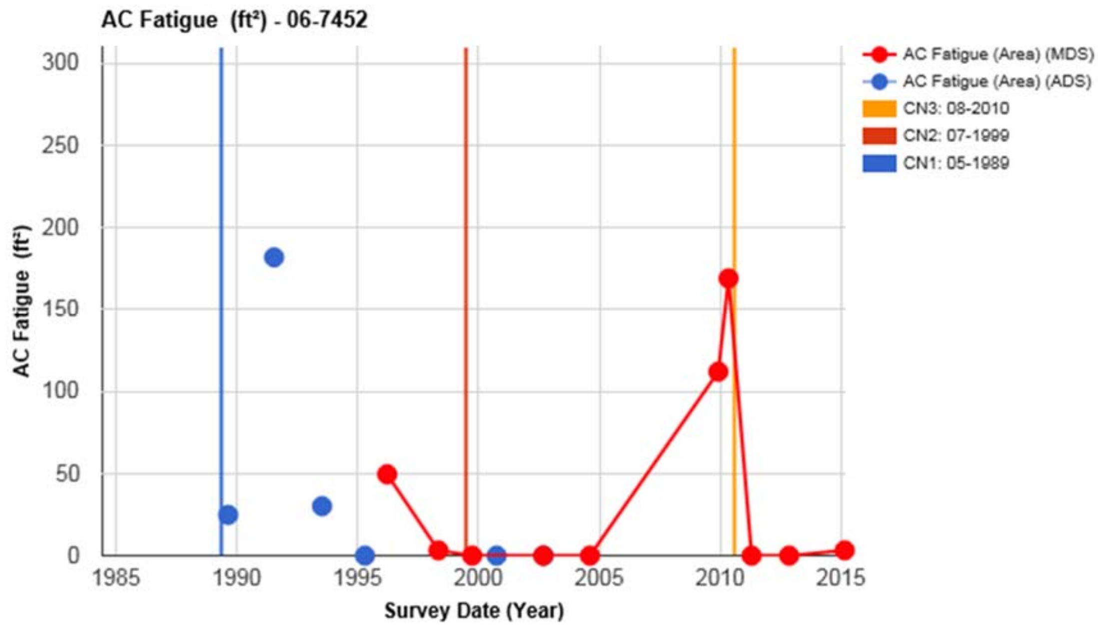


Figure 8. Time history of area of alligator cracking.

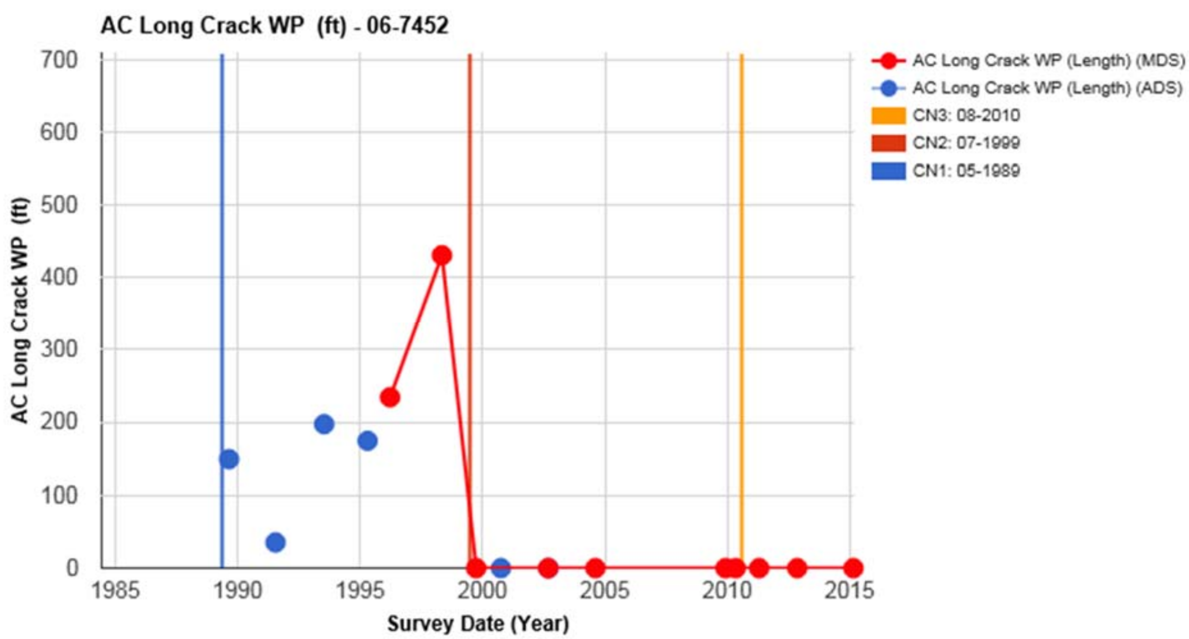


Figure 9. Time history of WP longitudinal cracking.

Figure 8 also shows that alligator cracking after application of the 1999 AC overlay was non-existent, but it re-appeared and increased between 2005 and 2010, prior to application of the 2010 AC overlay. In 2015, alligator appears to be re-appearing, albeit only a small quantity was observed.

Figures 9 shows the time history plot of longitudinal cracking in the wheel path (WP). As indicated earlier, WP longitudinal cracking has the reverse trend of alligator cracking during the CN = 1 period, which appears to be related to changes in the rating of these two distress types in the early years of the LTPP program. After application of the 1999 AC overlay, WP longitudinal cracking was eliminated and it has not re-appeared through 2015, which is when the last distress survey was performed.

Figure 10 shows the time history plots of longitudinal cracking not in the wheel path (NWP). The trend in NWP longitudinal cracking during CN =1 appears odd (decreasing with time), but this is simply the effect of a high value (586 ft) in 1989, and then dropping to values between 125 ft and 292 ft during the 1991 to 1998 time period. It is possible that the high initial value of 586 ft in 1989 is influenced by:

- How the cracks at the lane edge were judged, i.e., there were changes in the way these cracks were rated over the years.
- Width of the distress survey which changed from 14.8 ft in 1989 to 13.8 ft in 1991, 13.0 ft in 1993, and 12.5 ft in 1995.

Moreover, without the 1989 value, the test section would show a more logical trend that increases with time, reaching a maximum value of 292 ft in 1998, prior to application of the 1999 AC overlay. After the overlay, NWP longitudinal cracking remained low, only reaching a maximum value of 74 ft prior to application of the 2010 AC overlay. Starting in 2011, however, NWP longitudinal cracking re-appears and its presence accelerates, reaching a value of 465 ft in 2015.

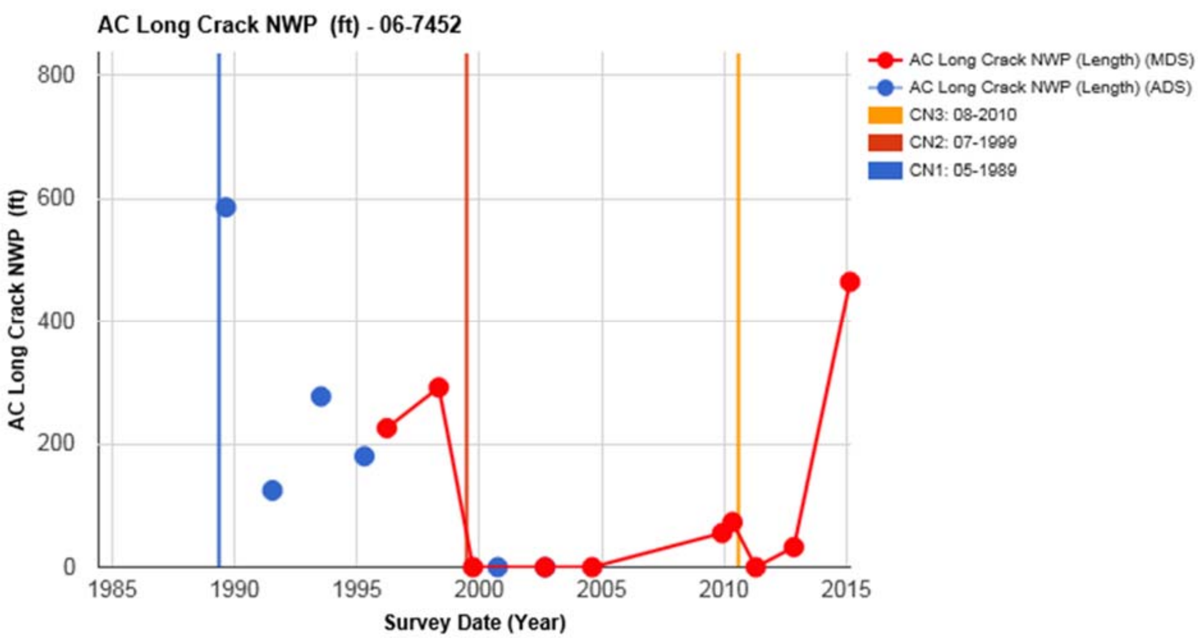


Figure 10. Time history of NWP longitudinal cracking.

The time history plot of rutting on the test section is shown in Figure 11. As shown, rutting of the pavement test section remained at around 0.2 inches during the CN =1 time period. After application of the 1999 AC overlay, rutting decreased slightly, but still remained close to 0.2 inches throughout the CN = 2 time period. After application of the 2010 AC overlay, rutting decreased more significantly, remaining close to 0.1 inches during the CN = 3 time period.

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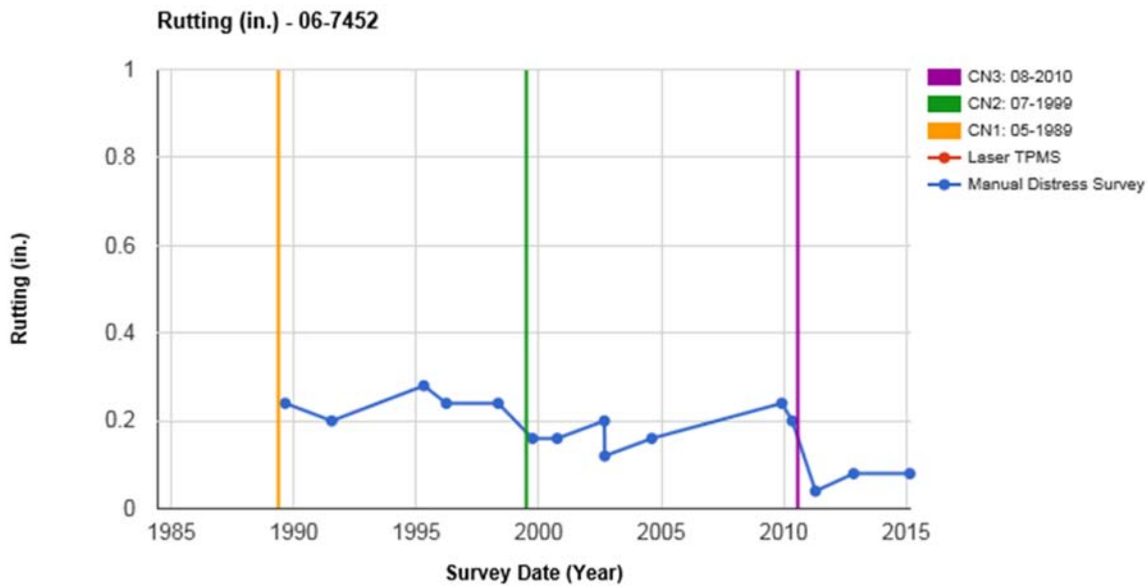

Figure 11. Time history plot of average rut depth computations.

Similarly, the time history of roughness measurements is shown in Figure 12. As shown, the IRI increased from 87 inches/mile in 1989 to 100 inches/mile in 1998. After placement of the 1999 AC overlay, roughness decreased to 48 inches/mile, but steadily increased to 85 in 2010, prior to placement of the second AC overlay. After the overlay, IRI decreased to 58 inches/mile and it remained close to that value through 2015, which is when the last survey was performed.

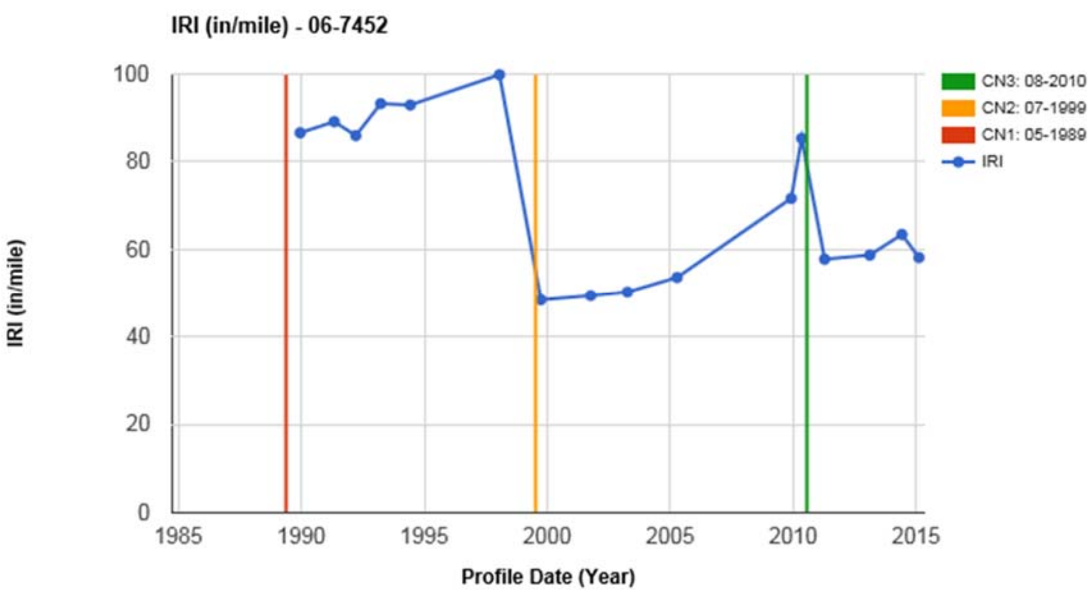

Figure 12. Time history plot of pavement roughness.

Figures showing the count and area of patching and block cracking are not included as part of the pavement distress history for the test section as they have remained at zero throughout the entire performance period of 1989 to 2015 (CN = 1, 2 and 3).

SUMMARY OF FINDINGS

In this review of information concerning the performance history of test section 06_7452 the following information was presented:

- The test section was originally constructed in 1972, consisting of 3.9 inches of AC on 6.7 inches of lean concrete and 9.8 inches of granular subbase (gravel) over a sandy lean clay. It was included in the LTPP program in 1989 as part of the GPS-2 (AC over Bound Base) experiment. In 1999, a 4.0 inch AC overlay was applied to the test section and it was classified in the GPS-6B (Planned AC Overlay of AC Pavement) experiment. In 2010, milling and another AC overlay were applied to the test section, effectively adding 1.2 inches of AC overlay to the structure, and it was reclassified to the GPS-6C (Modified AC Overlay of AC Pavement) experiment.
- The pavement structure has performed well over the 43 year period of 1972 to 2015 (date of last survey). The pavement condition metrics over the life of the pavement structure are summarized in Table 4. As shown, other than cracking (alligator, longitudinal and transverse) at the end of the CN =1 period (1972 to 1999) and alligator cracking at the end of the CN = 2 period of 1999 to 2010, the various pavement metrics are considered to be in good condition.

Table 5. Pavement Test Section 067452 Condition History

Pavement Condition Metric	Condition Prior to 1999 AC Overlay	Condition After 1999 AC Overlay	Condition Prior to 2010 AC Overlay	Condition After 2010 AC Overlay	Latest Condition (2015)
Alligator Cracking	3.2 ft^2	0 ft^2	169 ft^2	0 ft^2	3.2 ft^2
WP Longitudinal Cracking	431 ft	0 ft	0 ft	0 ft	0 ft
NWP Longitudinal Cracking	292 ft	0 ft	74 ft	0 ft	465 ft
Transverse Crack Count	100	0	10	0	1
Block Cracking	0 ft^2	0 ft^2	0 ft^2	0 ft^2	0 ft^2
Patching	0 ft^2	0 ft^2	0 ft^2	0 ft^2	0 ft^2
Rutting	0.2 inches	0.2 inches	0.2 inches	0.0 inches	0.1 inches
IRI	100 in/mile	48 in/mile	85 in/mile	58 in/mile	58 in/mile
Normalized Max. Deflection	7.1 mils	5.2 mils	5.4 mils	5.1 mils	4.9 mils

- What is surprising is that the fact that there is limited transverse cracking in the AC overlays. It is hypothesized that the observed transverse cracking during the CN = 1 period was due to the reflection of cracks in the lean concrete base to the AC surface layer. The application of the two AC overlays would certainly have delayed the re-appearance of transverse cracking. However, only 10 transverse cracks were observed at the end of CN = 2, where 4 inches of AC overlay were placed, and only one transverse crack was observed in 2015, five years after the second AC overlay.
- Traffic at the test section has steadily increased over the life of the test section, from and average daily truck count of approximately 100 in 1992 to a little over 400 in 2015. Other than a few data points that appear to be outliers, the derived and estimated truck counts agree with each other and their increasing trends appear reasonable.
- In general, the layer moduli backcalculated from the FWD deflection test data appear reasonable, but it is somewhat surprising that the values for the unbound granular layers remain so stable over the 22 years of deflection testing (1989 to 2011), despite pavement deterioration and variability in climatic conditions.

In summary, the original pavement structure and subsequent two overlays appear to have performed well over the 43 years. Moreover, since condition metrics at the test section have not been monitored since 2015, it is possible that the pavement may still be performing well, which would bring the total to 47 years.

FORENSIC EVALUATION RECOMMENDATIONS

While sufficient data appear to be available to explain the performance of the California 06_7452 test section from 1972 to 2015 (last survey), there are a few items that require confirmation or clarification in order to provide a better understanding of the performance of the test section. Accordingly, it is recommended that the desktop study be extended as follows:

- Obtain design information from Caltrans for the original pavement structure as well as for the two AC overlays to confirm the performance of the test section. This should include traffic, material strengths, layer thicknesses and drainage information. As part of this information gathering effort, it is also recommended that interviews of Caltrans staff familiar with the test section be conducted, if possible, to gather their thoughts as to why the test section has performed so well.
- Perform another round of distress and IRI surveys to investigate the performance of the second AC overlay through 2019-2020.
- Perform additional FWD testing to confirm the current structural soundness of the pavement as well as to further explore why the layer moduli for the unbound granular layers remained so stable over the 1989 to 2011 time period.
- Perform limited full-depth coring of surface cracks to (1) confirm transverse cracking and non-wheel path cracking prior to the 1999 overlay are the result of reflective cracking starting in the lean concrete base and propagating to the surface and (2) provide materials samples in support of the below. Also, take pictures of the subsurface layers, such as the lean concrete base, to assess its stability over the past 40 years.
- Perform a review of available materials data to identify data gaps and implement plan for addressing those data gaps. For example, while not fully explored in this desktop study, it appears that little if any materials data are available for the 2010 AC overlay.

ADDENDUM TO MEMORANDUM: FOLLOW-UP INVESTIGATIONS

The desktop study analyzed available field data to explain the performance test section 067452. Several interesting observations were made and led to multiple conclusions being drawn. Recommendations for future fieldwork were made as additional data could give a better insight into the performance of these sections.

The follow-up field activities took place in February 2020:

- Manual distress survey and FWD testing were performed on February 19, 2020 following standard LTPP protocols.
- Nine 6-inch cores were obtained from within the section on February 19 and 20, 2020.
- Longitudinal and transverse profile measurements were performed on February 21, 2020 following standard LTPP protocols.

Pavement Coring Summary

On the days of February 19 and 20 of 2020, a total of nine 6-inch cores were obtained from the within the test section. The cores varied in condition, with some in excellent condition and others showing the AC and LCB had cracked and deteriorated substantially. Overall, it appeared that any cracks in the LCB layer reflected up into the AC layers. Additionally, top-down cracking was observed in two cores, which was limited to layers L8 through L6. Core measurements are shown below in Table 6. Core photos can be viewed in Attachment A. The cracking observations for each core are summarized in Table 7.

Table 6. February 2020 Core Measurements

Core Number	Station (ft)	Offset from Fogline (ft)	Layer Thickness (in)					
			L8: AC Overlay	L7: AC Overlay	L6: AC Overlay	L5: Seal Coat	L4: AC Surface	L3: LCB Base
CA01	200	3	1.3	0	1.9	0.4	3.4	7.1
CA02	375	6	1.3	0	2.4	0.6	2.9	6.9
CA03	425	2	1.3	0	2.2	0.6	3.1	7.2
CA04	13	6	1.3	0	2.3	0.5	3.2	6.8
CA05	13	2	1.3	0	2.3	0.4	3.2	7.4
CA06	90	2.4	1.3	0	2.1	0.7	2.9	8.2
CA07	175	6.2	1.3	0	2.2	0.5	3.2	6.7
CA08	267	0.8	1.3	0	1.6	0.5	3.1	7.3
CA09	280	2.4	1.3	0	1.6	0.4	3.3	6.8

Table 7. February 2020 Core Observations

Core Number	Core Location	Core Observations
C01	Outer Wheelpath on Transverse Crack with Pumping	Full-depth crack that likely started in LCB and reflected into AC; AC and LCB were not well-bonded.
C02	Midlane	LCB material is deteriorated and portions of the core were loose material that was not recovered; AC layers L4 and L5 showed moisture damage.
C03	Outer Wheelpath	Core shows material is in good condition with no cracks or deterioration.
C04	Midlane	Core shows material is in good condition, portion of LCB was not well-compacted; AC and LCB were not well-bonded.
C05	Outer Wheelpath on Small Transverse Crack	Core shows material is in good condition with no cracks or deterioration.
C06	Outer Wheelpath on Transverse Crack	Top-down crack in AC layers L8 through L6; AC and LCB were not well-bonded.
C07	Midlane on Small Transverse Crack	Core shows material is in good condition, portion of LCB was not well-compacted; AC and LCB were not well-bonded.
C08	0.8 feet from Outside Shoulder on Transverse Crack with Pumping	Top-down crack in AC layers L8 through L6; bottom-up crack in LCB did not reflect due to top of LCB layer being severely deteriorated.
C09	Outer Wheelpath at Intersection of Fatigue and Transverse Cracking	Core shows extreme deterioration of all layers; both top-down cracking in AC layers L8 and L6, and bottom-up cracks in LCB that have reflected into AC layers L4 through L6.

The cores obtained in February 2020 showed the pavement structure within the test section differed from what was reported in the LTPP database. The LTPP database shows the 2010 treatment as being an overlay adding a 1.3-inch AC layer on top of the existing 1999 AC overlay. However, examination of the cores showed layer L7 had been completely milled off and layer L6 was reduced in thickness from 2.6 inches to 2.1 inches. Therefore, the updated pavement structure from within the section is presented in Table 8.

As described in Table 7, six cores were obtained from on top of cracking and four showed the LCB layer was either cracked or deteriorated, with three of these cores showing the LCB cracking had reflected into the AC layers. This shows that while the AC surface was renewed through overlays, the original AC and LCB layers were deteriorating and cracking, which slowly propagated through the overlays to the surface.

Manual Distress Survey (MDS)

Table 9 summarizes the current condition of section 067452 based on the data collected in February 2020. In previous years, manual distress surveys showed the progression of several key distresses, such as alligator, longitudinal, and transverse cracking, have taken years to manifest. In the case of the 1999 overlay, there was 169 ft^2 of alligator cracking, 74 ft of longitudinal and 10 transverse cracks measured on the surface of the overlay 11 years after the test sections first came into study. For the 2010 overlay, there was almost no alligator cracking after 5 years of service, but 465 ft of longitudinal cracking and 1 transverse crack were measured. However, after 10 years of service, a more significant amount of distress developed on the pavement surface. The 2020 survey recorded over 1,000 ft of longitudinal cracking and over 56 transverse cracks. Alligator cracking was also prevalent with 323 ft^2 observed, the most ever measured throughout the history of this section.

Table 8. Updated Pavement Structure from 2010 to date.

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)		114-Fine-Grained Soils: Sandy Lean Clay
2	Unbound (granular) subbase	9.8	302-Gravel (Uncrushed
3	Bound (treated) base	6.7	334-Lean Concrete
4	Asphalt concrete layer	3.4	1-Hot Mixed, Hot Laid AC, Dense Graded
5	Asphalt concrete layer	0.5	71-Chip Seal
6	Asphalt concrete layer	~~2.6~~ 2.1	1-Hot Mixed, Hot Laid AC, Dense Graded
7	Asphalt concrete layer	~~1.4~~ 0.0	1-Hot Mixed, Hot Laid AC, Dense Graded
8	Asphalt concrete layer	~~1.2~~ 1.3	1-Hot Mixed, Hot Laid AC, Dense Graded with partial milling

Table 9. Pavement Test Section 067452 Condition History

Pavement Condition Metric	Condition Prior to 2010 AC Overlay	Condition After 2010 AC Overlay	Condition in 2015	Condition in 2020
Alligator Cracking	169 ft^2	0 ft^2	3.2 ft^2	323 ft^2
WP Longitudinal Cracking	0 ft	0 ft	0 ft	0 ft
NWP Longitudinal Cracking	74 ft	0 ft	465 ft	1,016 ft
Transverse Crack Count	10	0	1	56
Block Cracking	0 ft^2	0 ft^2	0 ft^2	0 ft^2
Patching	0 ft^2	0 ft^2	0 ft^2	0 ft^2
Rutting	0.2 inches	0.0 inches	0.1 inches	0.16 inches
IRI	85 in/mile	58 in/mile	58 in/mile	69 in/mile
Normalized Max. Deflection	5.4 mils	5.1 mils	4.9 mils	4.7 mils

The most obvious reason for the recent spike in distress is the overlay thicknesses. The 1999 overlay consisted of two lifts totaling 4 inches of AC, which effectively doubled the thickness of the total AC. The 2010 overlay involved a 2-inch mill and 1.3-inch overlay, which reduced the total AC thickness from 7.9 inches to 7.3 inches. The 1.3-inch overlay would be expected to delay the propagation of existing cracking to the surface, but to a significantly lesser degree than a 4-inch overlay would have done.

The cores showed some of the cracks were full-depth cracks that most likely originated in the LCB and reflected into the AC. The cores also showed some spots where the AC layer exhibited some moisture damage. The accumulated degradation of the LCB and lower AC layers meant that cracking was most likely still present in the milled surface prior to the 1.3-inch overlay. The manual distress surveys conducted prior to the 4-inch overlay showed widespread longitudinal and transverse cracking, which took 12 years until it started to propagate to the AC surface. It is expected, though some cracking was removed through the milling process, these distresses would reflect through a 1.3-inch overlay much faster. The MDS data shows this cracking started showing up on the AC surface between 2 to 5 years after construction, and the most recent MDS shows widespread cracking at a comparable level to the section prior to the 4-inch overlay.

While the section has experienced widespread cracking in recent years, rutting has remained low, only increasing 0.06 inches since the previous transverse profile survey in 2015. Similarly, the section has remained relatively smooth since the 2010 mill and overlay with an IRI hovering around 58 in/mile. The 2020 longitudinal profile data showed the IRI had increased to a value of 69 in/mile, an increase of 11 in/mile compared to the most recent IRI measurement of 58 in/mile in 2015. Despite the rapid increase in cracking, the ride quality has not been significantly impacted. In addition, the current IRI value of the section is less than the IRI of the section prior to both the 1999 and 2010 overlays, which had values of 100 in/mile and 85 in/mile, respectively.

Pavement Overlay Structural Assessment

Falling Weight Deflectometer Data

All available FWD data was downloaded from InfoPave™ and combined with the FWD data collected in February 2020. Moduli values for section 067452 calculated using EVERCALC were available for FWD data collected from 1989 to 2011, however the 2011 results are based on an incorrect AC thickness. EVERCALC was used to determine moduli values for the FWD data collected in or after 2011, the results are shown in Table 10.

In general, the backcalculated moduli values appear reasonable and compare favorably to the historical EVERCALC values in the LTPP database. The unbound granular layer moduli values are higher than the historical values, but this is most likely because the upper limit for this layer was set to 75 ksi; the upper limit used for the historical moduli values is not known at this time. Different backcalculation models will produce different moduli values, and without knowing the specific details about the backcalculation model used for the LTPP EVERCALC moduli values, direct comparisons between the two sets of moduli values should be conducted with some caution.

AASHTO 1993 Overlay Design Analysis

In addition to the backcalculated moduli values, the FWD data were used to perform remaining life analyses for section 067452 in conjunction with the vast amount of data available on InfoPave™. The AASHTO 1993 Pavement Design Guide presents a methodology to calculate the structural capacity needed from an AC overlay, resulting in the required thickness of AC. There are three methods to determine the effective structural number of the pavement: reduced layer coefficients based on surface distresses, reduced structural capacity based on condition factor, and structural capacity based on backcalculation of FWD data. The backcalculation method will likely over-estimate the pavement’s structural number as it is suited for pavement sections without cement-treated layers. Accordingly, the remaining life analysis was conducted using both the reduced layer coefficient and the Structural Number (SN) reduction based on condition factor methods. A detailed write-up of the analysis methodology is presented in Attachment B.

Table 10. Backcalculated layer moduli over time.

Layer Type	Thickness (inches)	Test Date	Modulus (ksi)
Asphalt Concrete	3.9 (CN = 1)	12/11/1989	1,018
		03/22/1996	1,016
		05/06/1998	2,156
	7.9 (CN = 2)	09/28/1999	764
		08/11/2004	366
		05/06/2010	718
	7.3 (CN =3)	04/19/2011	817
		11/7/2012	1,015
		2/24/2015	817
		2/19/2020	928
Lean Concrete Base	6.7	12/11/1989	2,382
		03/22/1996	791
		05/06/1998	747
		09/28/1999	504
		08/11/2004	1,210
		05/06/2010	685
		04/19/2011	577
		11/7/2012	625
		2/24/2015	616
		2/19/2020	573
Unbound Granular Subbase	9.8	12/11/1989	43
		03/22/1996	35
		05/06/1998	30
		09/28/1999	30
		08/11/2004	22
		05/06/2010	36
		04/19/2011	42
		11/7/2012	55
		2/24/2015	61
		2/19/2020	54
Subgrade (top)	24.0	12/11/1989	21
		03/22/1996	26
		05/06/1998	22
		09/28/1999	34
		08/11/2004	19
		05/06/2010	25
		04/19/2011	24
		11/7/2012	31
		2/24/2015	24
		2/19/2020	31
Subgrade (bottom)	Semi-Infinite	12/11/1989	61
		03/22/1996	42
		05/06/1998	45
		09/28/1999	46
		08/11/2004	45
		05/06/2010	45
		04/19/2011	46
		11/7/2012	50
		2/24/2015	47
		2/19/2020	43

The initial structural number for the original pavement in 1972 was determined and served as the baseline for this analysis. For the condition factor method, the number of applied ESALS between the years 1972 and 1989 were summed up and divided by the remaining ESALs of the pavement in 1972. This ratio was used to determine the condition factor, which was 0.84. This factor was multiplied with the effective structural number of the pavement in 1972 to determine the effective structural number in 1989. Because of the lack of information on the section prior to 1989, the age and condition of the AC in 1989 is unknown, so it was assumed it was the AC was in good condition but the LCB and granular layer had accumulated damage[3]. This cycle was repeated for the 1996 FWD data, where the number of applied ESALs between 1989 and 1996 were summed and divided by the remaining ESALs of the pavement in 1989. This yielded a condition factor of 0.86, which was multiplied by the effective structural number of the pavement in 1989 to obtain the effective structural number in 1996. This process continued for each FWD test date.

In the AC reduction method, the effective structural number was determined based on surface condition and the amount of pavement distress. If no or little distress is present, the AC layer coefficient can be on par with new AC material, whereas a heavily cracked AC layer may get a very low coefficient. Since it can take several years for structural cracks to propagate from the bottom of an AC layer to the surface, this method can often times result in an overestimation of the effective structural number and thus an overestimation of the remaining ESALs of the pavement.

Remaining Life Analysis Conclusions

The results of the two analyses show large differences in both the effective structural number and consequently the remaining ESALs, as illustrated in Figures 13 and 14 and summarized in Tables 11 and 12, respectively. Because the surface distresses disappear following any overlay or milling event, the AC layer coefficient can be increased, resulting in likely over-estimated effective SN values. Conversely, with the condition factor, the SN values will continue to decrease even as the surface distresses are removed, meaning only an increase in (or replacement of) pavement structure can cause an increase in SN. This is easily apparent after the 2011 mill and inlay: the removal of the surface distress means a higher AC coefficient can be selected, which is applied to all 7.3 inches of the pavement. In the condition factor analysis, the 3.9 inches of original AC and the 2.1 inches of the remaining 1999 overlay have been subjected to the cumulative reduction in structural capacity of the original AC surface, meaning the reduction in AC thickness from 7.9 inches to 7.3 inches actually results in a drop in structural number.

The difference in remaining ESALs is remarkably different. This difference is small during CN 1, where the AC thickness is only 3.9 inches, but as the AC thickness increases, an increase in AC layer coefficient results in a substantial increase in remaining ESALs for the AC coefficient method. As discussed above, the condition factor method is constantly discounting the effective SN at the start of each analysis period, which prevents these large jumps in remaining ESALs where the pavement structure changes. Looking at the CN = 3 test dates, the AC coefficient method shows a rapid drop in remaining ESALs while the condition factor method shows a less-aggressive drop.

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[3] While it is unlikely that the original pavement structure would be essentially unchanged from 1972 to 1989, with minimal cracking present, the InfoPave™ database does not contain any documentation of historical maintenance and rehabilitation data. The condition of the AC surface in 1989 is not known, and in the absence of this data, it was assumed the AC was in good condition.

Table 11. Summary of AC coefficient reduction overlay analysis.

FWD Test Date	80% Percentile Subgrade MR (psi)	AC Thickness (in.)	AC Layer Coefficient	LCB Layer Coefficient	AB Layer Coefficient	SN Provided	Remaining ESALs	Actual ESALs carried after FWD Test Date
1/1/1972	8,061	3.9	0.40	0.15	0.10	3.545	641,144	414,183
12/11/1989	8,061	3.9	0.40	0.12	0.08	3.148	371,062	220,413
3/22/1996	7,147	3.9	0.38	0.12	0.08	3.070	224,976	57,895
5/6/1998	7,071	3.9	0.35	0.12	0.08	2.953	166,925	32,178
9/28/1999	9,150	7.9	0.40	0.12	0.08	4.748	10,031,959	219,598
8/11/2004	8,210	7.9	0.35	0.12	0.08	4.353	4,151,050	322,767
5/6/2010	8,835	7.9	0.30	0.12	0.08	3.958	1,405,189	21,499
4/19/2011	9,047	7.2	0.40	0.12	0.08	4.508	5,984,122	62,969
11/7/2012	10,526	7.2	0.35	0.12	0.08	4.143	5,002,354	149,653
2/24/2015	9,554	7.2	0.35	0.12	0.08	4.143	3,967,052	274,528
2/19/2020	9,189	7.2	0.25	0.12	0.08	3.413	1,163,287	--

Table 12. Summary of AC coefficient reduction overlay analysis.

FWD Test Date	80% Percentile Subgrade MR (psi)	AC Thickness (in.)	SN Provided	Remaining ESALs	Actual ESALs carried after FWD Test Date	Remaining Life (%)	Condition Factor
1/1/1972	8,061	3.9	3.545	641,144	414,183	35.4	0.84[1]
12/11/1989	8,061	3.9	3.227	416,471	220,413	47.1%	0.86
3/22/1996	7,147	3.9	2.776	142,319	57,895	59.3%	0.92
5/6/1998	7,071	3.9	2.554	87,154	32,178	63.1%	0.92
9/28/1999	9,150	7.9	3.949	3,103,454	219,598	92.9%	0.98
8/11/2004	8,210	7.9	3.870	2,022,157	322,767	84.0%	0.97
5/6/2010	8,835	7.9	3.754	1,088,299	21,499	98.0%	0.99
4/19/2011	9,047	7.2	3.521	1,375,822	62,969	95.4%	0.99
11/7/2012	10,526	7.2	3.486	1,832,826	149,653	91.8%	0.98
2/24/2015	9,554	7.2	3.416	1,298,957	274,528	78.9%	0.95
2/19/2020	9,189	7.2	3.246	877,309	--	--	--

Note 1: Applies to LCB and AB only. Assumed AC was in New Condition in 1989.

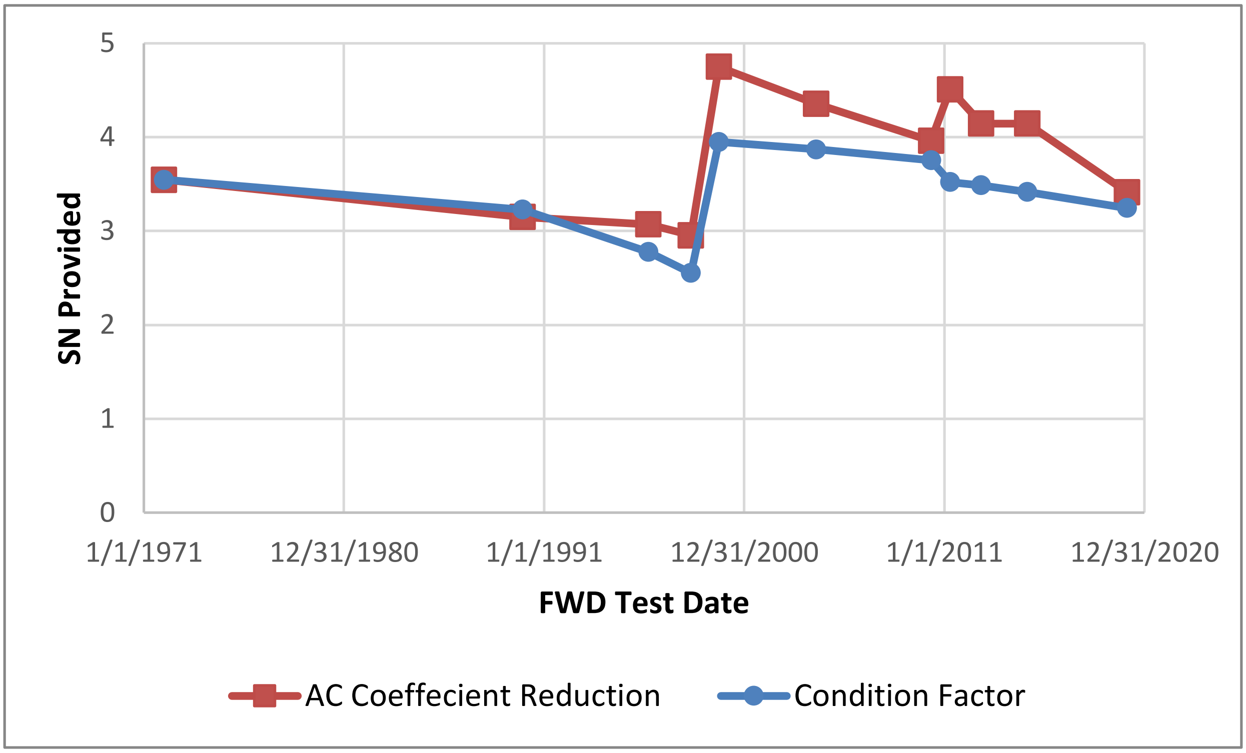


Figure 13. Effective pavement structural number for each FWD test date.

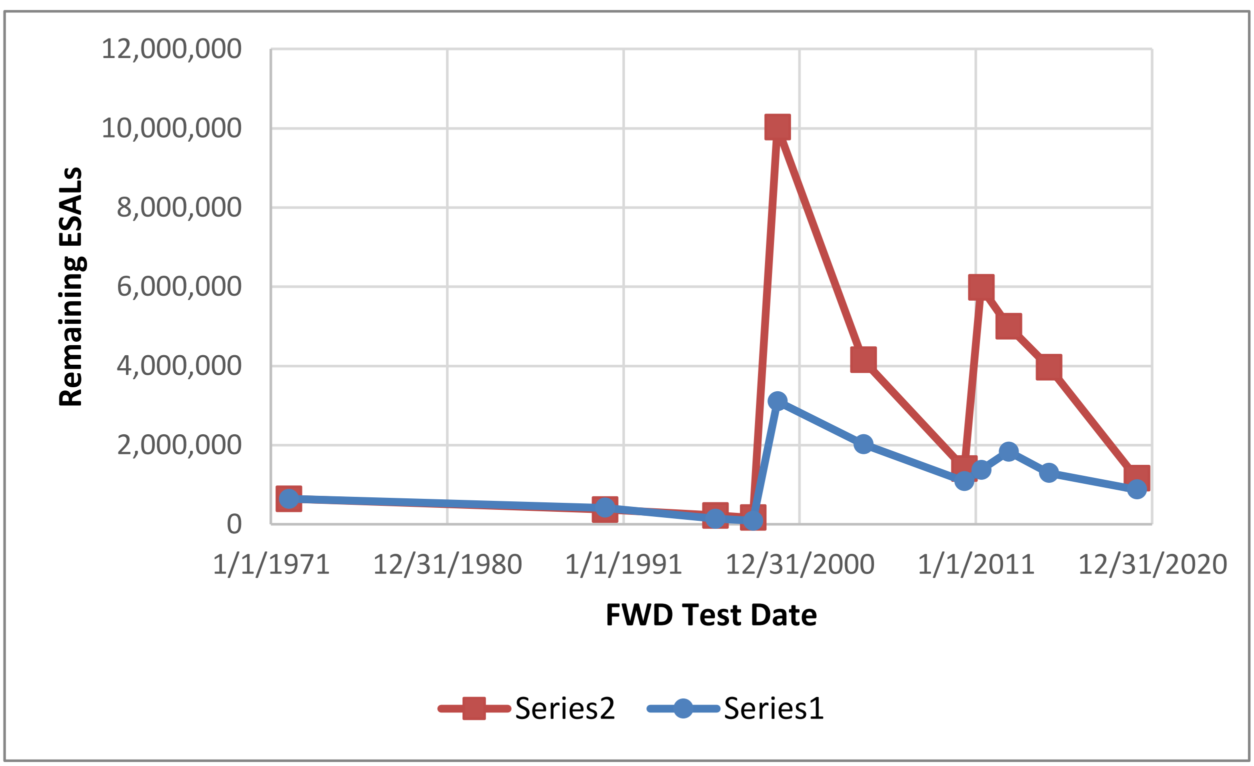


Figure 14. Remaining ESALs of pavement for each FWD test date.

The stated observations can indicate several things, including:

- The condition factor method appears to be the preferred option for designing the thickness of an overlay compared to the AC coefficient method. Use of the AC coefficient method could result in a design that does not provide sufficient additional structure or could show no additional structure is needed.
- The condition factor method is also the preferred option for determining the remaining ESALs (and remaining life) of a pavement. In instances where only a single set of FWD data will be collected, the AC coefficient method has the potential of drastically overestimating the remaining life of the pavement compared to the condition factor method.
- However, in instances where there are several years of FWD data, the AC coefficient method could give a better indication that a pavement has reached failure. The remaining life values from 2011 on show a steady drop in remaining ESALs that, if projected forward, would reach 0 on 5/19/2022. The remaining ESAL values from the condition factor method, if projected forward, show the pavement will reach 0 in the year 2032. This is likely related to the development of surface cracking since the 2011 overlay, and in the absence of cracking, the AC coefficient would not show these large drops in remaining ESALs.
- The 2020 cores showed the LCB layer was heavily distressed and oftentimes cracked, with these cracks reflecting into the AC layers. Because this section has received multiple overlays, and given the advanced age of the LCB, the condition factor method may be overestimating the structural capacity of the LCB layer. The AC coefficient method is based on surface distresses, but the layer coefficient for the LCB layer could be reduced over time based on the apparent decreasing LCB moduli values with time. Furthermore, neither method can explicitly account for cracks on the milled surface of the AC prior to the 2011 overlay.

OBSERVATIONS AND CONCLUDING REMARKS

The following observations are based on the follow-up investigations on test section 06_7452:

- 2020 manual distress data shows widespread cracking has developed since 2015. There are significant amounts of longitudinal and transverse cracking throughout the section. With the test section going out-of-study, not additional monitoring of the project will be performed, but it is expected cracking will continue to propagate until another rehabilitation event occurs.
- The rutting and roughness progressions, however, have been quite minimal over time, and both remain in the “Good” range as defined by FHWA.
- Cores obtained in 2020 show the LCB is distressed with multiple cracks that have reflected through the entire AC layer. These cracks will continue to reflect if new AC material is placed. Additionally, the LCB moduli values appear to be decreasing with time, which would indicate increased cracking in this layer over time and could result in further increased in reflective cracking into the AC layer.
- 2020 core measurements show the previous layer structure for CN = 3 were incorrect and the correct layer information has been provided to the LTPP Program (this will be updated in the next LTPP Standard Data Release). It should be noted that existing computations (e.g., backcalculated moduli) in InfoPave™ based on the layer structure for CN = 3 were done using the assumed structure, and should be redone based on the actual layer thicknesses.
- Considering the above, the structural assessment showed the moduli values for the section have remained stable through the life of the pavement and show little seasonal variation. Although the

climate data shows variations in yearly temperatures and precipitation, the pavement remained relatively unaffected.

- Overlay design analyses showed the section's remaining ESALs are dropping over time, and projecting forward indicate the pavement would reach failure between 2022 and 2032.

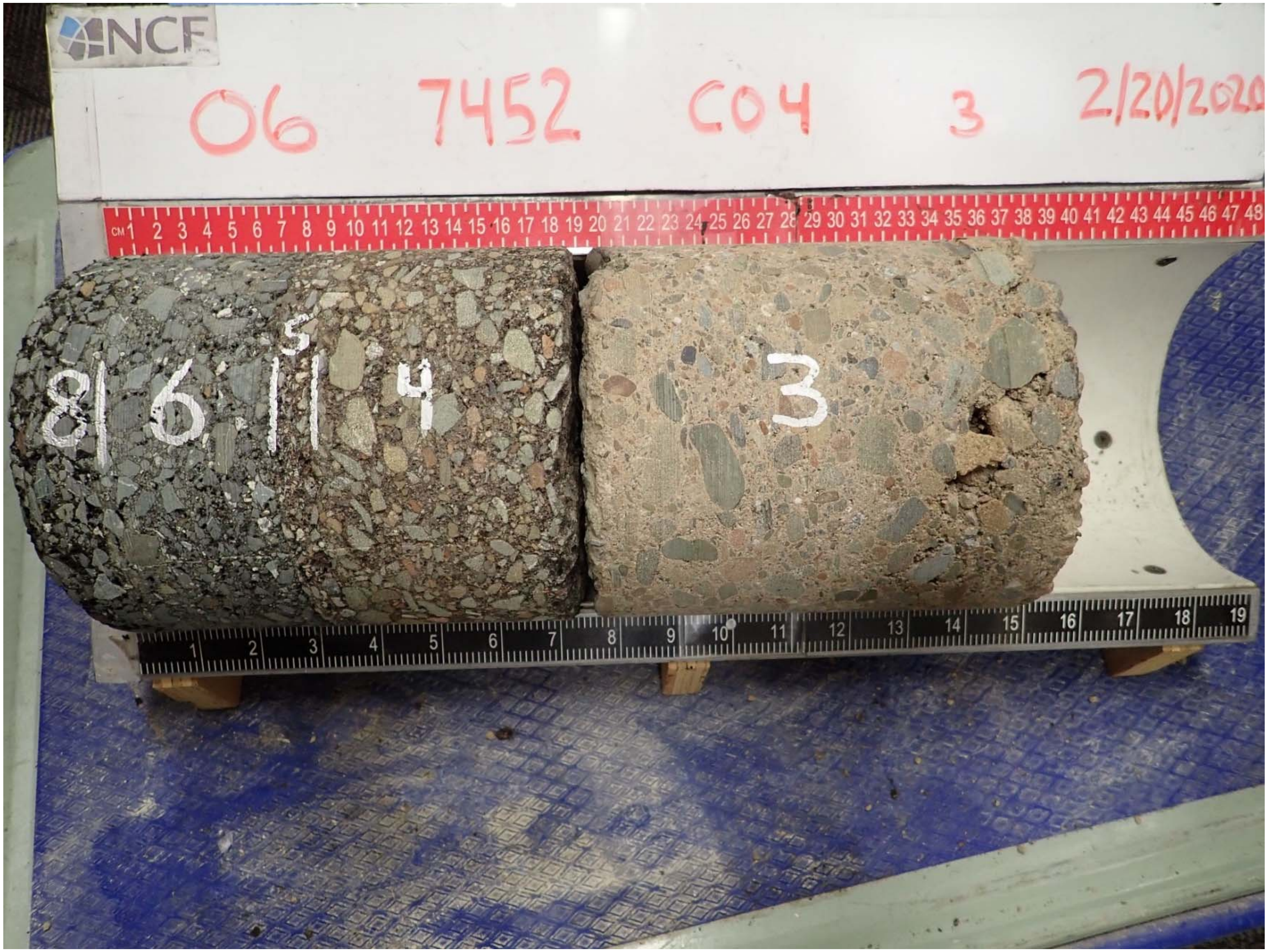
As noted previously, while this was the final monitoring associated with the LTPP Program, and the project will be placed out-of-study, additional project-related data collected by Caltrans as part of designing either another rehabilitation or a reconstruction would be of interest. Given the degradation of the LCB, treatments similar to the 2010 mill and overlay would not be expected to perform well.

ATTACHMENT A. PAVEMENT CORES FROM SECTION 06_7452 OBTAINED IN FEBRUARY 2020.

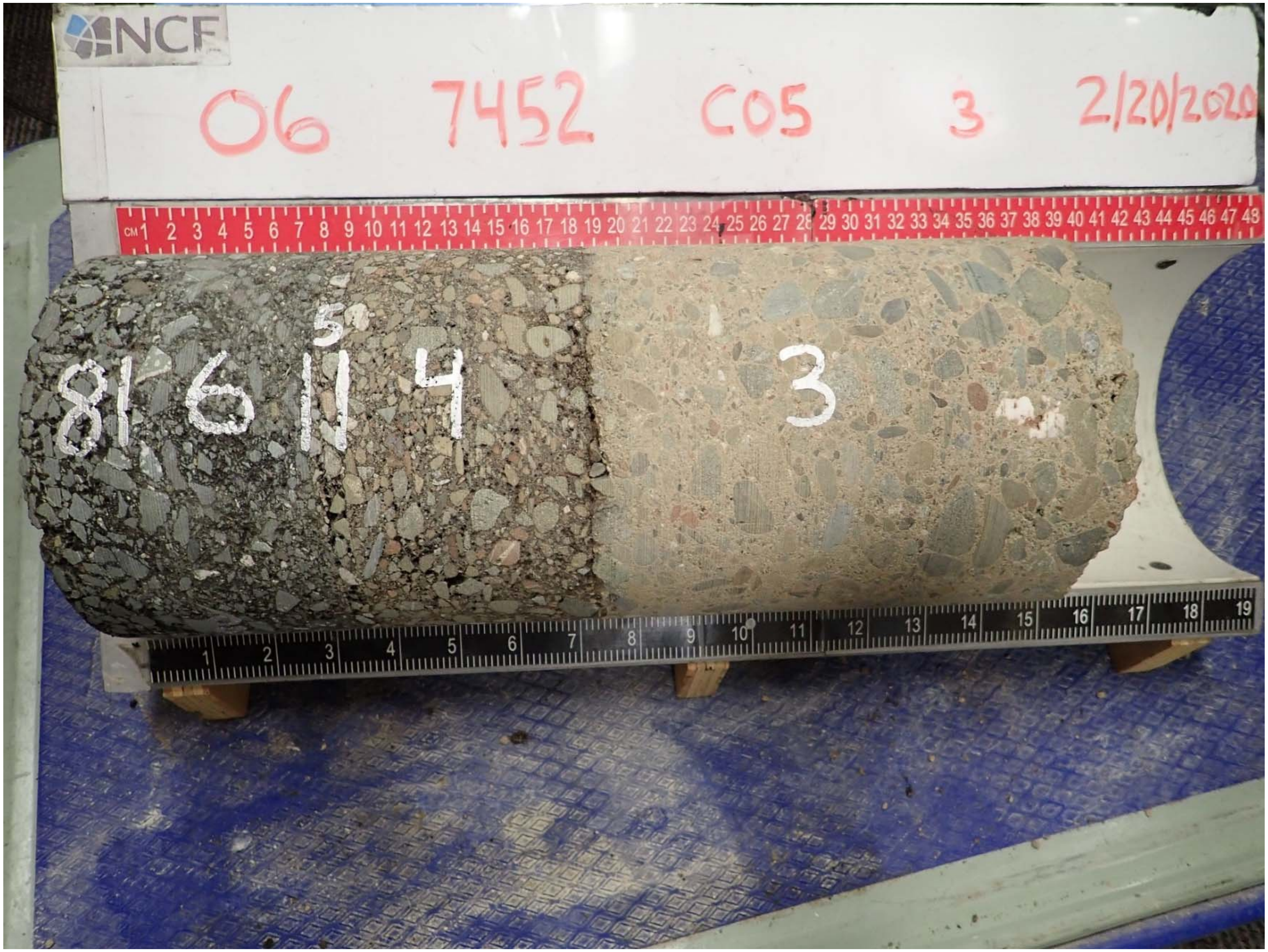
Core C01

Core C02

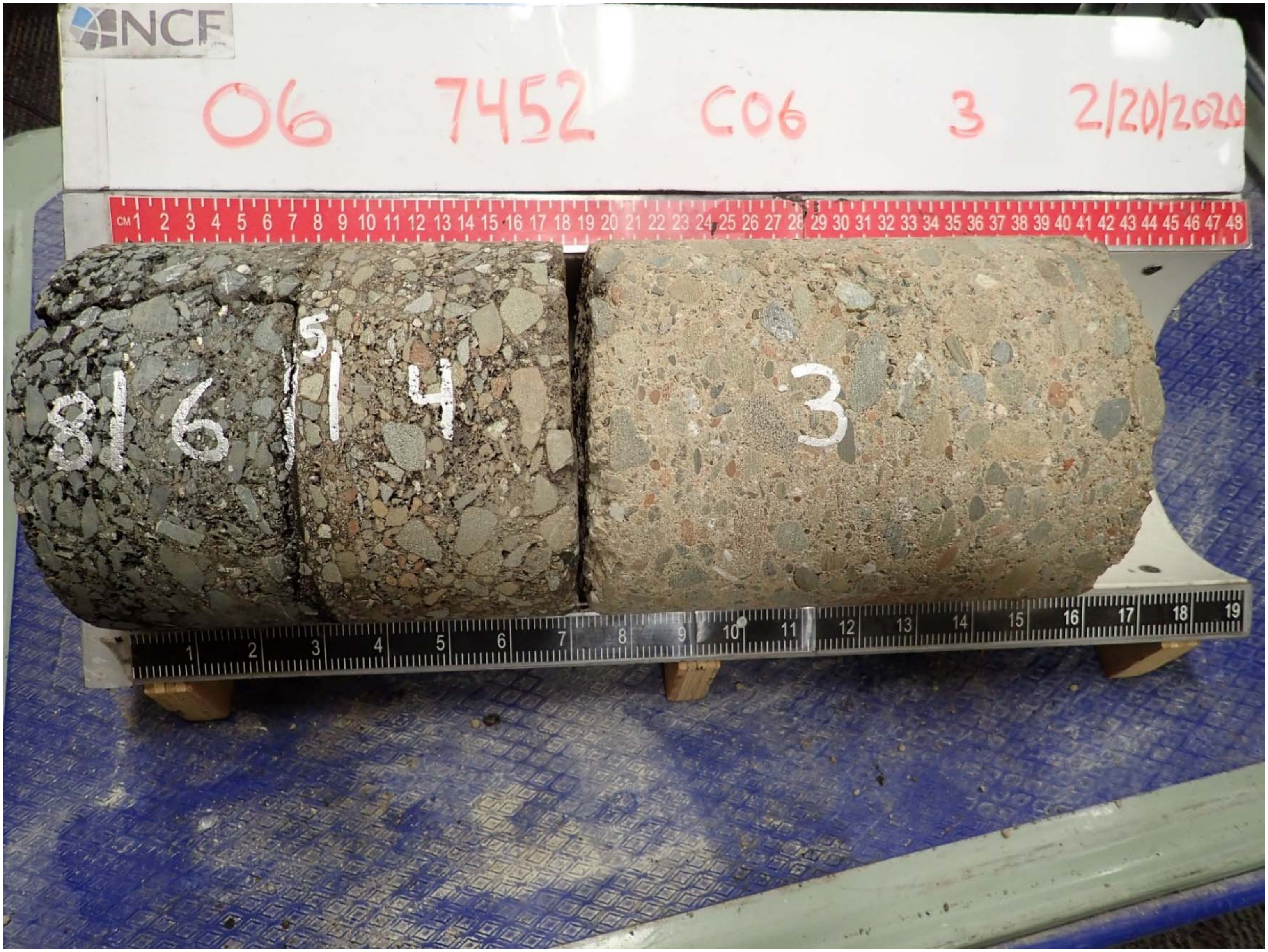
Core C03



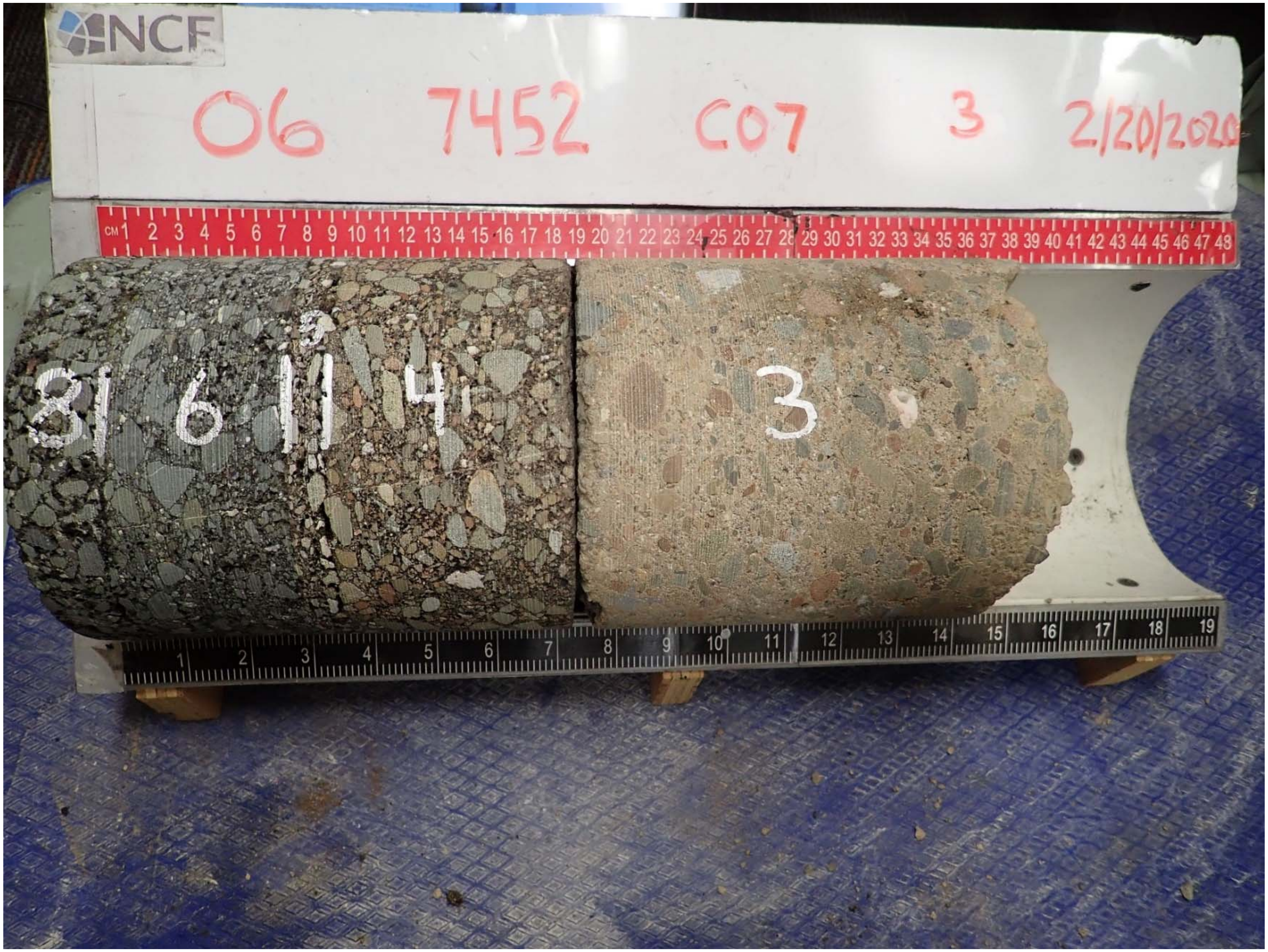
Core C04



Core C05



Core C06



Core C07

Core C08

Core C09

ATTACHMENT B. PAVEMENT REMAINING LIFE ANALYSIS METHODOLOGY

This document presents the methodology used to perform the overlay design analysis for the FWD data collected on California test section 06_7452.

As discussed in the revised technical memorandum, the AASHTO 1993 Pavement Design Guide presents a methodology to calculate the structural capacity needed from an AC overlay, resulting in the required thickness of AC. There are three methods to determine the effective structural number of the pavement: reduced layer coefficients based on surface distresses, reduced structural capacity based on condition factor, and structural capacity based on backcalculation of FWD data. Considering the backcalculation method will likely over-estimate the pavement's structural number as this method is best suited for pavement sections without cement-treated layers, the remaining life analysis was conducted using both the reduced layer coefficient and the structural number (SN) reduction based on condition factor methods.

As discussed in the memo report, the initial structural number for the original pavement in 1972 was determined and served as the baseline for this analysis. For the condition factor method, the number of applied ESALS between the years 1972 and 1989 were summed up and divided by the remaining ESALs of the pavement in 1972. This ratio was used to determine the condition factor, which was 0.84. This factor was multiplied with the effective structural number of the pavement in 1972 to determine the effective structural number in 1989. Because of the lack of information on the section prior to 1989, the age and condition of the AC in 1989 is unknown, so it was assumed it was the AC was in good condition but the LCB and granular layer had accumulated damage[4]. This cycle was repeated for the 1996 FWD data, where the number of applied ESALs between 1989 and 1996 were summed and divided by the remaining ESALs of the pavement in 1989. This yielded a condition factor of 0.86, which was multiplied by the effective structural number of the pavement in 1989 to obtain the effective structural number in 1996. This process continued for each FWD test date.

In the AC reduction method, the effective structural number was determined based on surface condition and the amount of pavement distress. If no or little distress is present, the AC layer coefficient can be on par with new AC material, whereas a heavily cracked AC layer may get a very low coefficient. Since it can take several years for structural cracks to propagate from the bottom of an AC layer to the surface, this method can often times result in an overestimation of the effective structural number and thus an overestimation of the remaining ESALs of the pavement.

Analysis Background Information

Before conducting the analyses, the first step was to determine the required design parameters that would be used, which are summarized in Table B1. The LCB material coefficient was selected based on a backcalculated modulus value of 600 ksi, and according to Figure 2.8 of the 1993 Design Guide, this corresponds to a layer coefficient of about 0.15. The original pavement was constructed in 1972, meaning the LCB and base layers were about 17 years old when the section was put into study in 1989. Given the lack of information on the section between 1972 and 1989, it was assumed the AC was in good condition when the section was put into study in 1989, however this analysis could be improved by obtaining (if possible) earlier condition data and redoing analysis per the approach presented in this attachment.

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[4] While it is unlikely that the original pavement structure would be essentially unchanged from 1972 to 1989, with minimal cracking present, the InfoPave™ database does not contain any documentation of historical maintenance and rehabilitation data. The condition of the AC surface in 1989 is not known, and in the absence of this data, it was assumed the AC was in good condition.

Table B1. Summary of Pavement Condition at each FWD Test Date

Design Parameter	Value
Reliability	90%
Standard Deviation	0.45
Terminal PSI	2.5
New AC Material Coefficient	0.40
New LCB Material Coefficient	0.15
New Aggregate Base Material Coefficient	0.10

Another important consideration is the pavement serviceability index at the beginning of each analysis period. The data from InfoPave™ was used to quantify the critical distresses for use in calculating the percent cracking. A 2018 report[5] using LTPP data presented a methodology to determine cracking percentage based on the recorded distresses. Table B2 summarizes the recorded cracking and calculated pavement serviceability index for each FWD collection date. The steps to calculate the percent cracking are listed below:

- Multiply the wheel path longitudinal cracking total by 2.5 ft to convert to area of cracking.
- Multiply the non-wheel path cracking and transverse cracking by 1 ft to convert to area of cracking.
- Total the above two values with the area of recorded alligator cracking and block cracking to obtain the total cracked area.
- Divide the total cracked area by the total area of test section to obtain cracking percentage.

The calculation of the pavement serviceability index is based on a report[6] for the Nevada Department of Transportation where the IRI, rut depth, and cracking percentage are used as inputs. The PSI values for each FWD data collection date was calculated using equation 1 below.

$$PSI = 5 * e^{-0.0041 * IRI} - 1.38 * (Rut\ Depth)^2 - 0.01 * (Cracking\ \% + Patching\ \%) \quad \text{(eq. 1)}$$

Reduced Layer Coefficients Based on Surface Distresses (AC Coefficient Method)

The reduced layer coefficient method uses the recorded surface distresses to determine the AC layer coefficient based on the criteria presented in the AASHTO 1993 pavement design guide, which is shown in Figure B1. The cracking observed in the section was limited prior to the 1999 AC overlay (CN 2), so the AC layer coefficient remained high. Between 1999 and the 2010 overlay, some of the distress had started to reflect into the overlay and become visible on the pavement surface, resulting in slightly lower AC coefficients. The 2020 field visit showed vast amounts of cracking, resulting in the lowest AC coefficient of all FWD test dates. Because of the unknown condition of the LCB and AB layers, the coefficients were not further reduced after 1989.

[5] Visintine, Beth A., Simpson, Amy L., and Rada, Gonzalo R. *Validation of Pavement Performance Measures using LTPP Data: Final Report.* Federal Highway Administration, Report No: FHWA-HRT-17-089. May 2018. https://www.fhwa.dot.gov/publications/research/infrastructure/pavements/ltpp/17089/17089.pdf

6 Sebally, Peter E., et. al. *Development of Pavement Performance Analyses and Procedures*. Nevada Department of Transportation. Report No: RTD-00-017. August 2001. https://www.nevadadot.com/home/showdocument?id=4038

Table B2. Summary of Pavement Condition at each FWD Test Date

FWD Date	Pavement Serviceability Index	IRI (in/ mile)	Rut Depth (in.)	Percent Cracking (%)	Longitudinal Cracking (ft)		Transverse Cracking (ft)	Alligator Cracking (ft^2)
					Wheel Path	Non-Wheel Path		
12/11/1989	3.43	86.67	0.24	0%	Not Available			
3/22/1996	3.32	93.01	0.24	23%	234.5	225.7	541.9	49.5
5/6/1998	3.22	99.92	0.24	33%	431.0	291.9	617.0	3.2
9/28/1999	4.06	48.50	0.16	0%	0.0	0.0	0.0	0.0
8/11/2004	4.01	51.96	0.16	0%	0.0	0.0	0.0	0.0
5/6/2010	3.46	85.40	0.20	4%	0.0	73.5	24.6	169.0
4/19/2011	3.94	58.00	0.04	0%	0.0	0.0	0.0	0.0
11/7/2012	3.93	58.00	0.08	1%	0.0	32.8	0.0	0.0
2/24/2015	3.92	58.00	0.08	8%	0.0	464.8	5.2	3.2
2/19/2020	3.91	58.00	0.12	26%	0.0	1016.0	251.6	323.0

Lastly, the InfoPave™ website has calculated the number of ESALs applied to the section for each year, which are summarized in Table B3.

Table B3. Summary of Yearly ESALs for Section 067452

Year	Annual ESALs	Year	Annual ESALs	Year	Annual ESALs
1972	9,932	1988	23,980	2003	45,000
1973	14,520	1989	24,484	2004	50,000
1974	13,523	1990	73,168	2005	60,000
1975	14,235	1991	24,484	2006	58,506
1976	15,559	1992	13,275	2007	64,000
1977	16,940	1993	29,609	2008	59,094
1978	20,641	1994	30,463	2009	57,000
1979	22,207	1995	30,463	2010	22,500
1980	23,838	1996	26,835	2011	42,000
1981	19,644	1997	27,331	2012	41,966
1982	33,879	1998	31,317	2013	32,000
1983	35,872	1999	22,604	2014	40,854
1984	32,259	2000	44,963	2015	41,000
1985	34,733	2001	37,000	2016	56,953
1986	36,442	2002	48,000	2017	58,364
1987	21,495	--	--	--	--

Table 5.2. Suggested Layer Coefficients for Existing AC Pavement Layer Materials

MATERIAL	SURFACE CONDITION	COEFFICIENT
AC Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0 35 to 0 40
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	0 25 to 0 35
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5–10 percent medium- and high-severity transverse cracking	0 20 to 0 30
	>10 percent medium-severity alligator cracking and/or <10 percent high-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	0 14 to 0 20
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	0 08 to 0 15

Figure B1. AC layer coefficient based on surface cracking.

Reduction of Structural Number Based on Condition Factor (Condition Factor method)

The second analysis method was the use of the condition factor, where the effective structural number (SN) of a pavement is discounted based on the remaining life of the pavement. As an example, a pavement with an SN of 5 that can handle 1,000 ESALS sees 600 ESALs over the next three years. The remaining life value is 40%, which corresponds to a condition factor of about 0.86, so the structural number of the pavement after three years would be 4.3. This process was conducted for each FWD test date, where the pavement SN at the time of FWD testing was determined and the cumulative ESALs carried by the pavement until the next FWD testing date was calculated. The relationship between remaining life (actual ESALs divided by remaining ESALs) with the condition factor is illustrated in Figure B2.

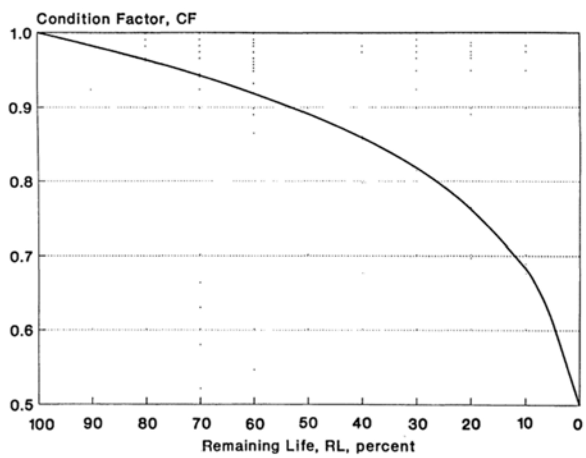

Figure 5.2. Relationship Between Condition Factor and Remaining Life

Figure B2. Determination of condition factor based on percent remain.