

**Forensic Evaluation - Southwest Anthony Henday Drive's Portland Cement Concrete Pavement**

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## **ABSTRACT**

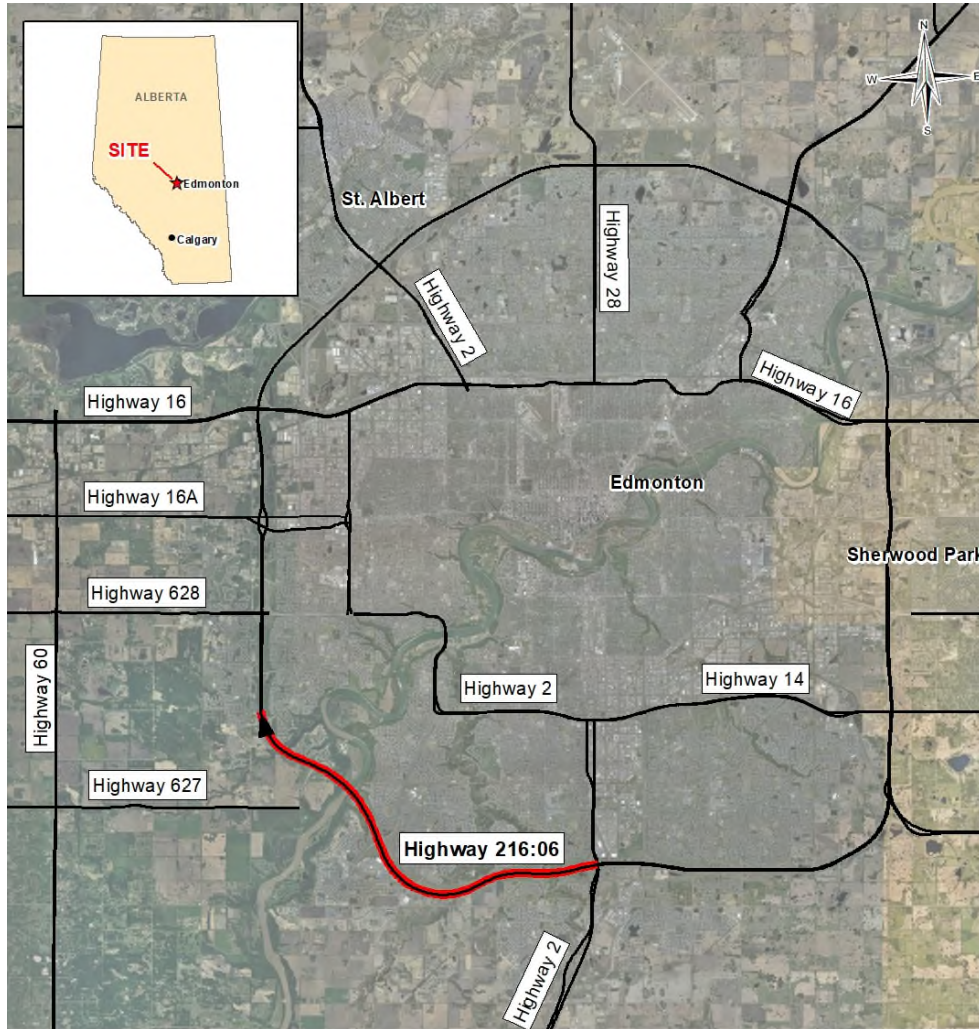
A 14-kilometre-long section of Portland cement concrete (PCC) pavement along Southwest Anthony Henday Drive (SWAHD) in Edmonton, Alberta was opened to traffic in 2006. Since opening, the roadway has experienced a significant increase in traffic volume. The PCC pavement consists of a doweled jointed plain concrete and presently the performance of the pavement is poor relative to expectations of a longer life pavement. The condition of the pavement is fair to poor, with smoothness (ride quality) a major concern.

The pavement exhibited joint sealant loss soon after construction and, six years after construction, major rehabilitation activities were undertaken to address various areas exhibiting distresses and ride quality issues. The completed rehabilitation activities included partial and full depth panel replacement, cross stitching of longitudinal cracks, dowel bar placement at mid-panel transverse cracks, slab jacking, and diamond grinding. Despite the rehabilitation activities, ride quality has continued to be an issue and ongoing maintenance activities have been required to address cracked panels, drainage along the roadway, and joint resealing.

Subsequently, additional investigations were undertaken in 2017. This paper presents the forensic methodology used to investigate the causes of the premature development of distresses and poor ride quality issues. Site inspections, geotechnical borehole drilling, pavement coring, ground penetrating radar (GPR) testing to determine the location of the dowel bars and presence of voids underneath the pavement, analysis of Laser Crack Measurement System (LCMS) data, historical pavement smoothness data, Falling Weight Deflectometer (FWD) data, drainage analysis, profile analysis using LiDAR data, along with other analyses, were undertaken for this investigation. The results of the additional investigations are presented along with potential causes of the poor performance and recommendations for rehabilitation and rehabilitation sequencing.

## 1.0 INTRODUCTION

Southwest Anthony Henday Drive (SWAHD) between Calgary Trail and Lessard Road is a 14 kilometre long, four-lane divided Portland Cement Concrete (PCC) pavement section of the Edmonton Ring Road. The alignment (shown in Figure 1) is generally westbound from Calgary Trail, then turns northwards crossing the North Saskatchewan River, and finishes northbound at Lessard Road. This PCC pavement was constructed between 2004 and 2006 and has seven interchanges within the project limits. The interchanges at Rabbit Hill Road, Cameron Heights Road and Lessard Road were constructed after the opening of the mainline.



**Figure 1: Project Location**

To date, the performance of the pavement has been relatively poor compared to expectations for a longer life pavement. Presently, the pavement condition is fair to poor, due mostly to smoothness (ride quality). The pavement started exhibiting joint sealant loss soon after construction and it is believed that the poor performance of the pavement might be attributed to the presence of subsurface water.

## **2.0 REVIEW OF BACKGROUND INFORMATION**

### **2.1 As-built Pavement Structure**

The pavement structure within the project limits consists of a Jointed Plain Concrete Pavement (JPCP) with a thickness of 230 mm of PCC over 150 mm of granular base course (GBC). The pavement design included 32 mm smooth steel dowel bars at transverse joints to facilitate load transfer and 15M deformed steel tie bars at longitudinal joints to mitigate longitudinal joint opening. Tie bars were omitted from longitudinal joints where the pavement width exceeded 15 m (such as entrance and exit ramps) to reduce the potential for longitudinal shrinkage cracks in these areas.

### **2.2 Traffic Data**

At the design stage for the initial construction of the PCC, the Annual Average Daily Traffic (AADT) was projected to be 30,000 vehicles in 2005 and 40,000 in 2025. Truck traffic of 10% was assumed and used in the calculation of the design ESALs. Based on this, rigid design ESALs of 36.7 million were calculated in the design lane over the design life of 30 years.

A review and analysis of actual traffic data since the opening of SWAHD to traffic indicated AADT of 92,000 and cumulative observed ESALs of 23.2 million in the design lane to 2017. Commercial traffic on the roadway comprise of approximately 4.4% Single Unit Trucks (SUT) and 4.5% Tractor Trailer Combinations (TTC). Truck factors of 0.881 (for SUT) and 2.073 (for TTC) were used in the calculation of the design traffic as per the Alberta Transportation methodology.

### **2.3 Completed Maintenance and Rehabilitation Treatments**

The maintenance contractor has been performing sealant joint repairs of the transverse joints since 2009. Drainage improvements were also completed around the Terwillegar Drive interchange and Lessard Road interchanges by the maintenance contractor in 2017 to address poor surface and sub-surface drainage.

A major rehabilitation project that included full depth panel replacements, partial depth repair of spalled concrete areas, cross stitching of the panels, spall repairs and diamond grinding for select areas exhibiting distresses and rough ride quality was completed in 2012/13. As part of that project, approximately 9 lane-km of the pavement surface was diamond ground, 106 panels were replaced (full depth replacement), 30 panels with longitudinal cracking were repaired using cross stitching, 12 panels with transverse cracks were repaired using dowel bar retrofits (DBR) and 5,600 m<sup>2</sup> of the panels were jacked using polyurethane injection.

## **3.0 FIELD INVESTIGATION AND DATA ANALYSIS**

Despite the major rehabilitation project in 2012/13, the PCC pavement continued to exhibit performance issues related to ride quality and joint sealant failures. As such additional work was undertaken in 2017 to further investigate the causes of these problems. The investigations and analysis included field reconnaissance, traffic data, subsurface exploration, ground penetrating radar (GPR) surveys, coring, strength testing, multi-year roughness data, longitudinal profile, LCMS data, LiDAR data, and drainage assessment. A brief summary of the findings from these activities are provided in Table 1. A more in-depth analysis of roughness and drainage is discussed later.

**Table 1: Key findings from the investigation and analysis**

Investigation / Analysis	Findings
<b>Field reconnaissance</b>	<ul style="list-style-type: none"> <li>▪ Overall, the pavement was observed to be in fair to poor condition.</li> <li>▪ The ride quality offered by the pavement has deteriorated and the pavement is exhibiting a variety of other distresses that need further rehabilitation intervention.</li> <li>▪ Distresses exhibited by the pavement include cracked panels, lane and shoulder drop offs, faulting of the panels, failed joint sealant, failed joint repairs, joint failures at the intersection of longitudinal and transverse joints and spalling of concrete.</li> <li>▪ Partial depth repairs of the spalled concrete completed in 2012/2013 are performing poorly. Majority of the partial depth repairs have failed and will need to be repaired again.</li> <li>▪ Drainage along the roadway needs improvement. Median ditches were observed to be flat; ponding water was observed adjacent to the pavement (in the ditches and the central median) and there was no place for the water in many of the gore areas to escape.</li> <li>▪ Majority of the on and off ramps were observed to be in good to fair condition with a few cracked panels on some of the ramps.</li> </ul>
<b>Traffic data</b>	<ul style="list-style-type: none"> <li>▪ Shortly after opening, rapid increase in AADT of about 75% from 2007 to 2008.</li> <li>▪ Between 2008 and 2015, the compound annual traffic growth rate was calculated as 7%.</li> <li>▪ Four-lane AADT capacity<sup>1</sup> of 80,000 reached in 2015. On track to reach six-lane capacity in 2021. (Note: only four lanes exist throughout most of the project).</li> <li>▪ Original 30-year design ESALs of 36.7 million to be exceeded in 2021. Actual ESAL projected to reach 83 million at 30-years (this analysis assumed addition of a third lane in each direction and then no growth after capacity is reached in 2021).</li> </ul>
<b>Subsurface exploration</b>	<p>The findings are based on regional geology and 18 boreholes which were advanced to depths ranging from 3.05 m to 5.33 m, sampling and testing of the granular base course (GBC) materials underneath the PCC, the bottom ash insulation layer (where present) and subgrade soils, and water table depth during the borehole drilling. It is also noted that sub-surface water was observed in shallow excavations (trenches dug by maintenance contractor for installing edge drains) immediately adjacent to the road.</p> <ul style="list-style-type: none"> <li>▪ At depth, hard silty clay with observable silt laminations was typically encountered. In some locations, clay-rich till-like materials were observed. Overlying these materials were either silty clay fill (likely reworked or replaced, glaciolacustrine or till-like soil) or road structure materials. The materials are all relatively low hydraulic conductivity materials which have a limited capacity to transmit water. The presence of relatively low hydraulic conductivity materials near and at surface is likely one of the primary contributors to observed pooling of water at surface.</li> <li>▪ The road structure consists of a GBC of between 0.1 m and 0.2 m and PCC between 0.225 m and 0.240 m in thickness. Although the GBC is described as “granular” it is not necessarily “free draining” (with fines content varying from 8.3% to 13.2%).</li> <li>▪ Groundwater levels were observed between 0.29 to 5.24 m below the ground surface with the majority of the water level readings being greater than 1 m below ground surface. This would suggest that the observed water at surface is decoupled from the groundwater i.e. there is partially saturated soil underlying the drainage ditches.</li> <li>▪ Subsequently, the shallow surface water is present due to poor surface drainage and because the geological materials do not have sufficient capacity to allow infiltration to the groundwater.</li> </ul>
<b>GPR Surveys</b>	<p>GPR survey was completed at two locations to locate the dowel bars in the pavement with the intent of extracting cores over the dowel bars (to determine the efficiency of the dowel bars at a location with faulting and a control location).</p>

<sup>1</sup> Alberta Transportation considers AADTs of 80,000 and 120,000 to be the capacity of four-lane and six-lane highways, respectively.

Investigation / Analysis	Findings
	<ul style="list-style-type: none"> <li>▪ At a faulting location, GPR indicated a pattern where the dowel bars were misplaced such that they were entirely on the leave slab at alternating joints (i.e. did not cross the joint). The joints where the dowel bars were misplaced had faulting and the joints at which the dowel bars were placed correctly did not have faulting. This location was 165 m long and had faulting in the outside lane only.</li> <li>▪ At a control location (location without faulting), GPR scanning indicated that the dowel bars were placed centred on the joints and at the specified spacing.</li> </ul>
<b>Road Radar</b>	<ul style="list-style-type: none"> <li>▪ Road Radar testing was completed at four detailed pavement evaluation locations. Review of the collected and analyzed Road Radar data indicated presence of some anomalies at the interface of the PCC pavement and granular base. These anomalies indicated presence of excess moisture and/or voids.</li> </ul>
<b>Coring</b>	<p>Pavement coring was completed at 12 locations within the project limits.</p> <ul style="list-style-type: none"> <li>▪ At locations without diamond grinding (7 cores) – average thickness of 231 mm.</li> <li>▪ At locations with diamond grinding (5 cores) – average thickness of 220 mm.</li> <li>▪ Extracted cores were visually inspected, polished core surface were reviewed under microscope, were tested to determine compressive strength and density, and were also reviewed microscopically for requirements of CSA A23.1-14 for freeze/thaw durability.</li> <li>▪ It was concluded that the concrete was strong and durable.</li> </ul>
<b>Strength testing</b>	<p>FWD testing (one test at the joint and one test at mid slab) was completed for every 40 panels in the outer lanes and every 80 panels in the inner lanes. Additional FWD testing was also completed at closer spacing at the locations selected for detailed pavement evaluation and at a few other locations exhibiting distresses.</p> <ul style="list-style-type: none"> <li>▪ For the regular testing, mid-panel deflection in the outside lane averaged 28% higher than the inside lane. Deflection at joints averaged 44% higher in the outside lane than the inside lane.</li> <li>▪ For the regular testing, void detection analysis conducted for the joint testing indicated 4% to 10% of the outside lanes may have voids. None of the tests in the inside lanes indicated the presence of voids.</li> <li>▪ For the regular testing, 98% of tests at joints had good load transfer efficiency (LTE).</li> <li>▪ For the detailed evaluation site testing, 7% of joints had poor LTE; all joints with poor LTE were noted to have faulting.</li> </ul>
<b>Multi-year International Roughness Index (IRI)</b>	<ul style="list-style-type: none"> <li>▪ General consistency in IRI between lanes.</li> <li>▪ Linear growth in IRI of 0.06 mm/m to 0.07 mm/m per year.</li> <li>▪ Improvement due to diamond grinding of 0.5 mm/m to 1.2 mm/m.</li> </ul>
<b>Longitudinal profile</b>	<p>Inertial profile data from the Province’s network level data collection program was analyzed.</p> <ul style="list-style-type: none"> <li>▪ Warping was identified as the primary cause of roughness and the analysis is discussed in further detail in a following section. The warping is stable from year-to-year (2014-2016) and did not reoccur in areas that were diamond ground.</li> <li>▪ Faulting extent and severity increased from 2014 to 2016, was more extensive in the outside lanes than the inside lanes. However, it was relatively infrequent except for a few isolated areas and it was concluded that faulting was not the primary cause of roughness.</li> </ul>
<b>LCMS (Laser Crack Measurement System)</b>	<ul style="list-style-type: none"> <li>▪ Used to delineate the extent of lane / shoulder drop-off greater than 10 mm. These locations were confined to areas without tie-bars at on and off ramps.</li> <li>▪ The percentage of cracked slabs was 2.2%.</li> </ul>
<b>Mobile Terrestrial LiDAR Survey</b>	<p>LiDAR (light detection and ranging) data collected as part of Alberta Transportation’s network-level surveys was analyzed and ditch depths and slope were calculated:</p> <ul style="list-style-type: none"> <li>▪ Ditch depths were classified as good (Ditches greater than 1 m in depth), fair (ditch depths ranging from 0.6 m to 1.0 m), poor (ditch depths ranging from 0.3 m to 0.6 m) and non-existent (ditch depths of less than 0.3 m). Median ditches were mostly poor or non-existent.</li> </ul>

Investigation / Analysis	Findings
	<ul style="list-style-type: none"> <li>▪ Ditch slopes were classified as good (slope of greater than 2%), fair (ditch slope ranging from 1% to 2%), and poor (ditch slopes of less than 1%). Median and roadside slope were generally poor, except near the North Saskatchewan River where the roadway descends into the river valley.</li> </ul>
<b>Drainage summary</b>	<ul style="list-style-type: none"> <li>▪ Visual inspection of the PCC showed that the cracks and joints (longitudinal and transverse) allow a direct hydraulic connection between the road surface and the GBC. This connection is likely the primary cause of water being present beneath the PCC pavement.</li> <li>▪ Surface and subsurface drainage conditions were identified as a primary concern for the roadway segment and primary contributing factor to the poor performance of the pavement.</li> </ul>

**3.1 IRI Data**

International Roughness Index (IRI) data is regularly collected with an inertial profiler as part of the Alberta Transportation’s network-level pavement condition surveys. The data was first collected in all lanes in 2007 and then biennially from 2010 to 2016. To analyze the progression of roughness on the project, the IRI data was filtered to remove roadway sections containing bridge decks, bridge approaches, and asphalt pavement sections. To analyze the impact of diamond grinding on roughness, the IRI data was further filtered and segmented based on observed locations of the diamond grinding.

The progression of IRI was analyzed for two data sets: 50 m roadway segments with no apparent rehabilitation treatments; and 50 m segments with diamond grinding apparent over their entirety. 50 m segments that had diamond grinding over part of their length were excluded from this analysis.

For the roadway segments observed to have received no rehabilitation treatments, the progression of IRI over time appears to be approximately linear, as illustrated in Figure 2 for each lane of the roadways. Table 2 presents the average IRI values for each year as well as the calculated average annual increase in IRI.

**Table 2: Average IRI for Roadway Segments without Rehabilitation**

Roadway	Lane	IRI (mm/m)					Average Annual Increase
		2007	2010	2012	2014	2016	
Westbound	Inside	1.83	1.96	2.20	2.21	2.37	0.06
	Outside	1.72	1.89	1.90	2.20	2.33	0.07
Eastbound	Inside	1.79	1.89	2.04	2.19	2.30	0.06
	Outside	1.70	1.80	1.83	2.18	2.37	0.08
<b>All Lanes Combined</b>		<b>1.76</b>	<b>1.88</b>	<b>2.00</b>	<b>2.20</b>	<b>2.34</b>	<b>0.07</b>





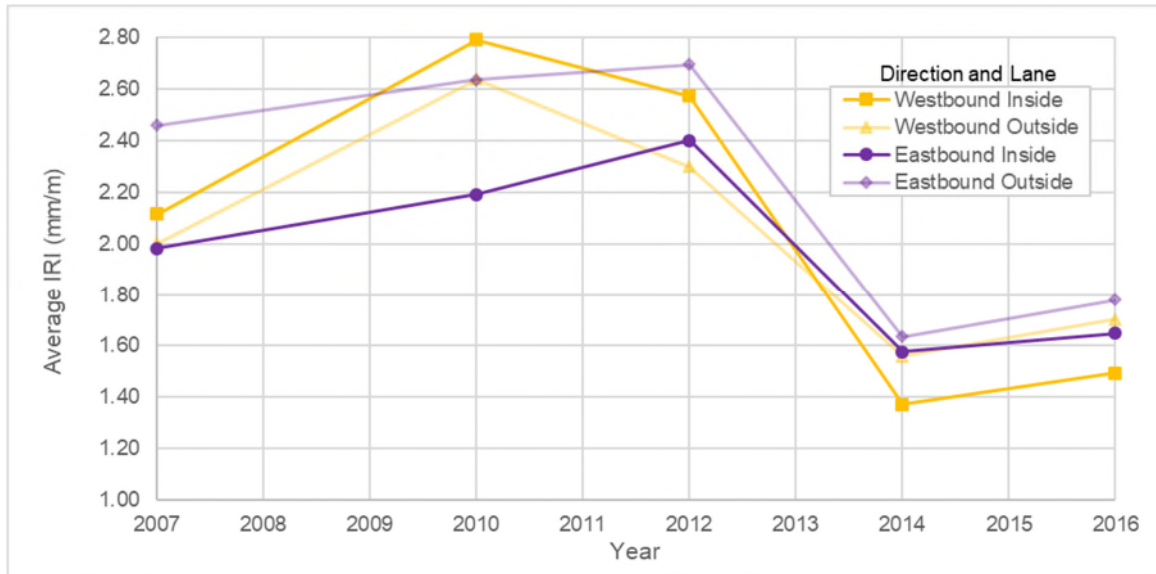
**Figure 2: IRI Progression (Segments without Rehabilitation)**

For the roadway segments that received diamond grinding in 2013, the progression of IRI is illustrated in Figure 3. Table 3 presents the average IRI values for each year; the average annual increase in IRI from year 2007 to 2012; the decrease in IRI from year 2012 to 2014 (i.e. the decrease in IRI associated with the diamond grinding rehabilitation); and the average annual increase in IRI from year 2014 to 2016.

**Table 3: Average IRI for Roadway Segments with Rehabilitation**

Roadway	Lane	IRI (mm/m)							
		Pre - Rehabilitation			Post - Rehabilitation		Average Annual Increase 2007-2010	Decrease 2012-2014	Average Annual Increase 2014-2016
		2007	2010	2012	2014	2016			
Westbound	Inside*	2.12	2.79	2.57	1.37	1.49	0.06	-1.20	0.06
	Outside*	2.00	2.64	2.30	1.56	1.70	0.02	-0.74	0.07
Eastbound	Inside	1.98	2.19	2.40	1.58	1.65	0.09	-0.82	0.04
	Outside	2.46	2.64	2.70	1.64	1.78	0.04	-1.06	0.07
<b>All Lanes Combined</b>		<b>2.14</b>	<b>2.55</b>	<b>2.49</b>	<b>1.54</b>	<b>1.66</b>	<b>0.05</b>	<b>-0.95</b>	<b>0.06</b>

\* IRI in 2012 was lower than in 2010 because the 2012 data may contain some segments that had some pavement repairs completed prior to data collection in 2012.



**Figure 3: IRI Progression and Improvement (Segments with Diamond Grinding)**

Diamond grinding treatments, on average, provided between a 0.5 and 1.2 mm/m decrease in IRI. The rate of IRI increase before and after diamond grinding are approximately the same. Therefore, following diamond grinding, roughness would expect to return at approximately the same rate as an untreated segment of roadway, i.e. approximately 0.06 mm/m increase in IRI per year. Generally, roughness is consistent between the inner lanes and outer lanes throughout the project. In areas without rehabilitation to improve smoothness, the average annual increase in IRI has been approximately 0.07 mm/m (per year) since 2007.

### 3.2 Curling / Warping

The extent and contribution of the curling/warping to the poor ride quality was investigated through the analysis of the inertial profiler data.

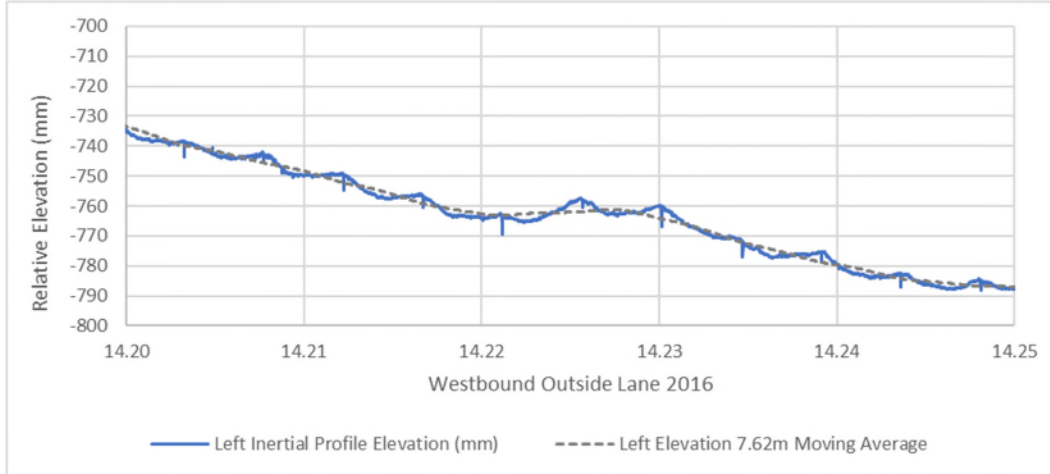
#### 3.2.1 Background

The presence of curling and/or warping in concrete pavement manifests as slab curvature. Curling is defined as curvature induced by a temperature gradient in which the surface of the slab has a temperature that differs from the bottom of the slab. The direction of curvature may be upwards (as at night when the top is cooler than the bottom and contracts relative to the bottom) or downwards (as in daytime when the top is warmer and expands relative to the bottom). Warping is curvature in the slab that results from a moisture gradient. Warping is almost always upward and starts as the surface begins to dry following placement of the concrete. The bottom of the slab may often remain at or near saturation and shrinks less than the top as the concrete cures. (Van Dam, 2015)

#### 3.2.2 Analysis

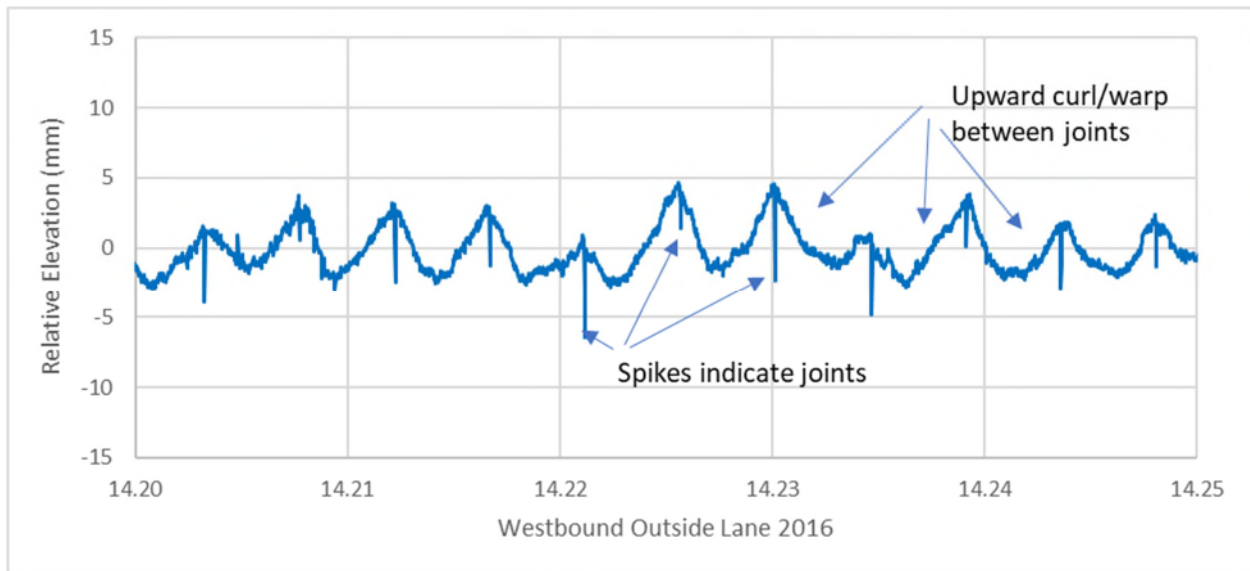
A moving average filter was used to remove roadway grades and long wavelength roughness from the profiles. A 7.62 m (25 foot) base length for the moving average was chosen because that length proved effective in the analysis of curling and warping at the LTPP SPS2 (Long-Term Pavement Performance, Specific Pavement Study 2) site in Arizona (Karamihas and Senn, 2012). The 7.62 m moving average was

first calculated to represent the grade and longer wavelength roughness. An example of the moving average plotted with the raw inertial profile data is shown in Figure 4.



**Figure 4: Example Inertial Profile Data and 7.62 m Moving Average**

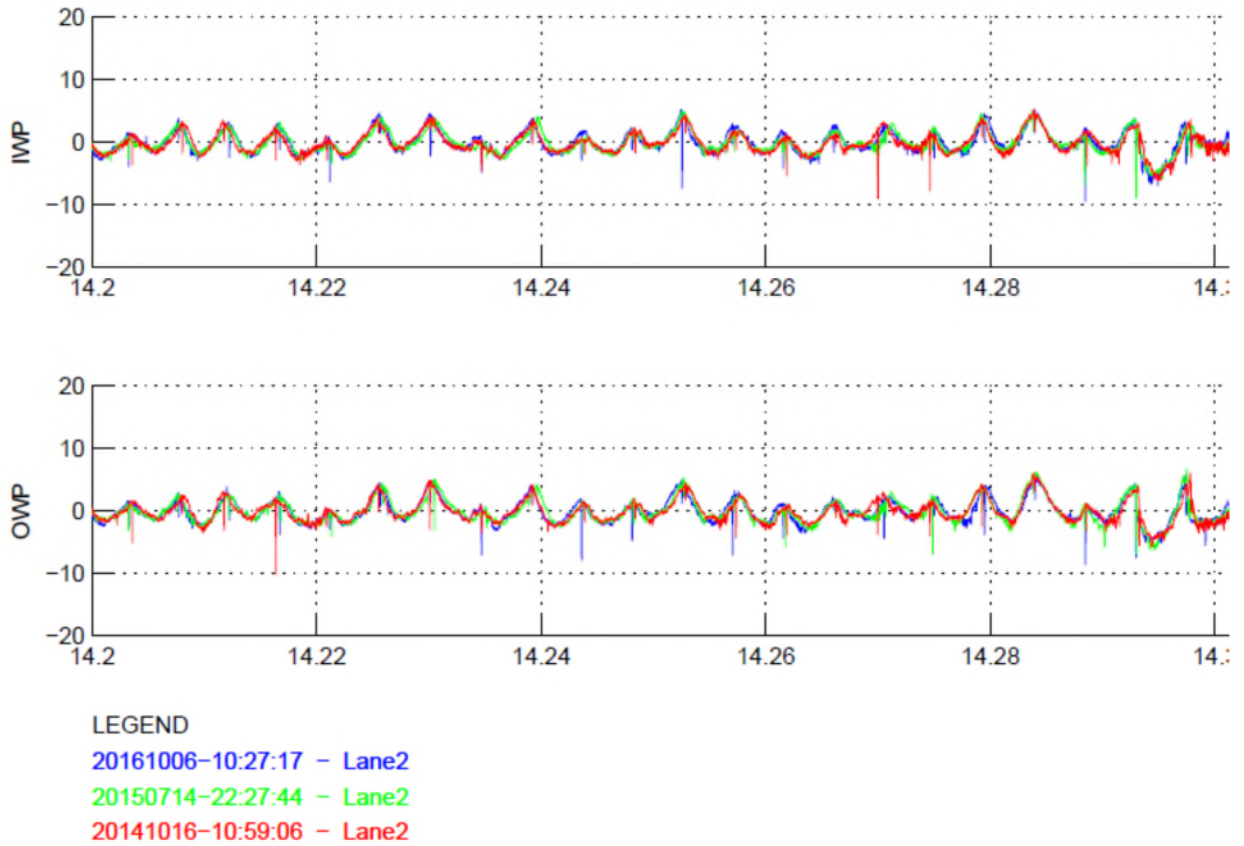
This averaged profile was then subtracted from the original profile to show surface features with wavelength generally shorter than 7.62 m. This difference between the original profile and smoothed profile is known as the ‘anti-smoothed’ profile. An example is shown in Figure 5. In this figure, downward spikes indicate the location of transverse joints where a laser entered the joint. The presence of upward curl/warp can clearly be seen in this example.



**Figure 5: Example Filtered Profile Showing Upward Curl / Warp**

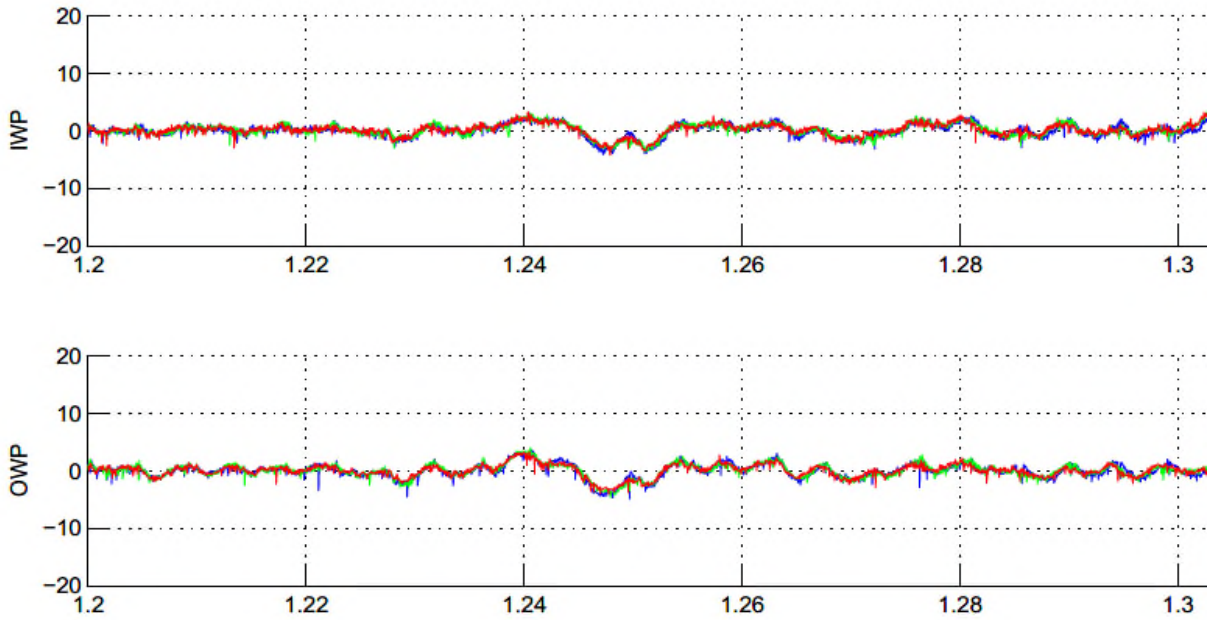
This moving average filtering procedure was applied to the inertial profile data from 2014 to 2016. The surveys took place in mid-October, mid-July, and early October for the three years. Surveys also varied from late evening to mid-morning. The resultant plots, an example of which is shown in Figure 6, showed that the upward curl/warp was present nearly throughout the project limits and relatively stable from year-to-year. This stability is also prevalent throughout the project limits regardless of date or time of day.

This stability leads to the conclusion that the upward curvature is primarily due to moisture, rather than temperature, and is therefore primarily warping, rather than curling.



**Figure 6: Example of Multi-Year Filtered Profiles (section without diamond grinding)**

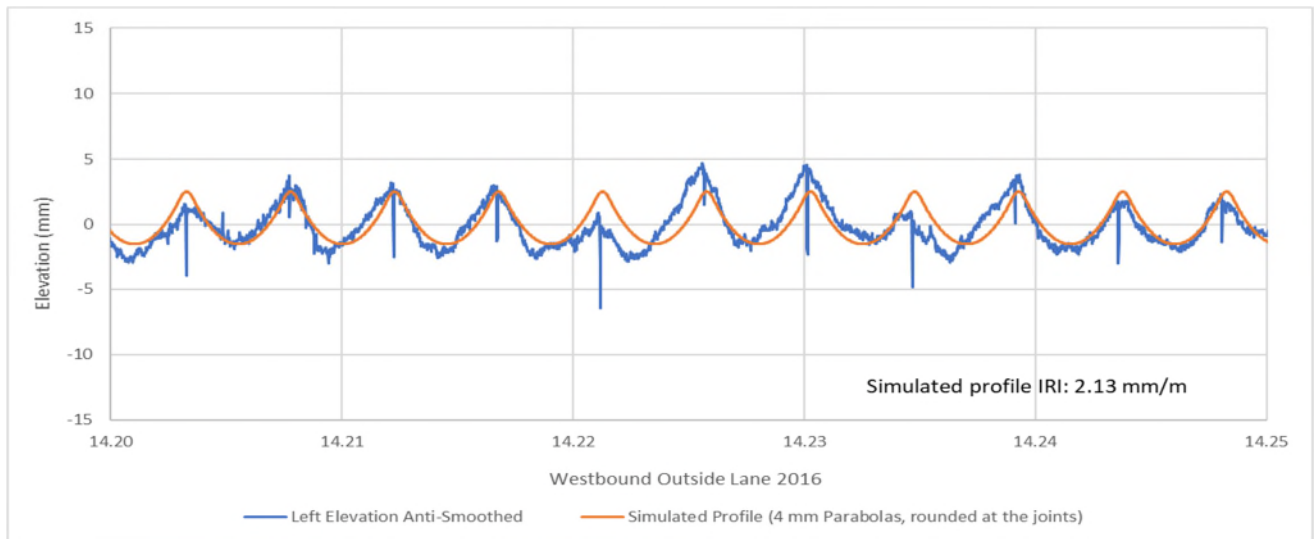
The stability in warping was also evident in sections that had diamond grinding completed in 2013. The diamond grinding was effective in removing the warping and the curvature appeared to be negligible in 2016. An example of longitudinal profile in an area that had diamond grinding in 2013 is shown in Figure 7.



**Figure 7: Example of Multi-Year Filtered Profiles (section with diamond grinding)**

### 3.2.3 Roughness Due to Warping

The portion of the westbound outside lane as shown in Figure 6, from km 14.20 to km 14.25, exhibits warping in the range of 3 mm to 7 mm. The shape of warping resembles a parabola centered on each panel with rounding of the parabola edges between panels. A simulation was conducted on a theoretical profile with 4 mm of curl / warp to develop an understanding of the roughness associated with curl / warp. The example profile, and simulated profile with 4 mm of repetitive parabolic curl / warp, rounded at the joints is also shown in Figure 8.



**Figure 8: Example Profile and Simulated Warping**

The level of curling / warping observed in the multi-year analysis from 2014 to 2016 would result in a poor ride, even in the absence of other defects. Therefore, warping was considered to be a primary cause of the poor ride quality offered by the pavement.

### **3.3 Drainage Assessment**

#### **3.3.1 Surface Drainage**

Observations made during the field reconnaissance indicated presence of ponding water at the edge of the pavement, in the ditches and the gores. In addition to this, deep tire ruts in the granular material adjacent to the pavement shoulder were also observed indicating weak soils/ poor drainage along the roadway. Water discharge from transverse joints at various locations during dry weather indicate the presence of water in the joint or at the interface of the PCC and the granular layer.

Based on the observations made during the field reconnaissance and a review of the LiDAR data, it was noted that the ditches are either shallow or non-existent at several locations. At a few locations, the slope in the ditches is not enough to take the water away from the pavement structure. Surface water was observed entering the pavement structure via cracks, unsealed joints, and other discontinuities in the PCC.

#### **3.3.2 Sub-Surface Drainage**

The discontinuities in the PCC (i.e. cracks and non-sealed joints) form a conduit for flow from the surface directly to the GBC of the road structure. The GBC is underlain by low hydraulic conductivity fine grained clay subgrade which limits the potential for infiltrating water to recharge the groundwater. There is also limited potential for lateral drainage of the GBC as the adjacent ditches are prone to collect water and the GBC itself is not a free draining material. In short, water penetrates the road surface and tends to accumulate within the GBC. This will be most severe where the GBC is contaminated with fines, where the road is at the bottom of a sag curve, and where drainage ditches are shallow and contain water at an elevation at, or within, the elevation of the GBC.

### **3.4 Remaining Service Life**

Including as part of the investigation work was an analysis of the remaining service life (RSL) of the PCC. The RSL was calculated based on two methods: comparison of actual traffic volumes to design traffic volumes, and calibration of a slab cracking model to the observed performance.

#### **3.4.1 Remaining Service Life Based on Design**

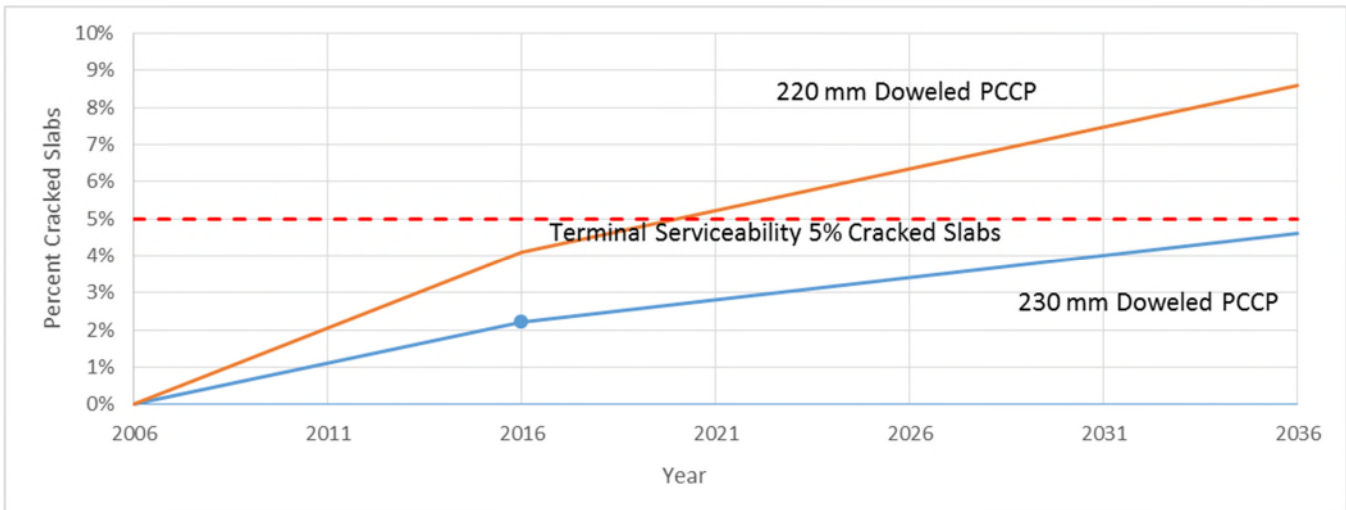
A comparison of the design ESALs calculated during the design stage with the actual ESALs over the pavement following opening to traffic indicates that the actual ESALs on the roadway have been significantly higher than originally anticipated. A review of the analyzed data indicated that the design ESALs calculated for the 30-year pavement design life (during the design stage) will be exceeded in year 2021 (15 years after the opening of the highway to traffic). Based on this comparison, it is concluded that theoretically the pavement would reach the end of its service life in year 2021, or alternatively, it has a remaining service life of 3 years (from 2018).

### 3.4.2 Remaining Service Life Based on Slab Cracking Performance

The American Concrete Pavement Association’s (ACPA) StreetPave12 software has incorporated an enhanced implementation of the Portland Cement Association (PCA) design method originally used to design the SWAHD concrete thickness. The two main enhancements were the inclusion of additional concrete fatigue experience since the PCA method was developed in 1984 along with replacement of the PCA method Load Safety Factor with Reliability. The method is used in thickness design to determine the minimum design thickness that would result in the designer’s selected percentage of cracked slabs.

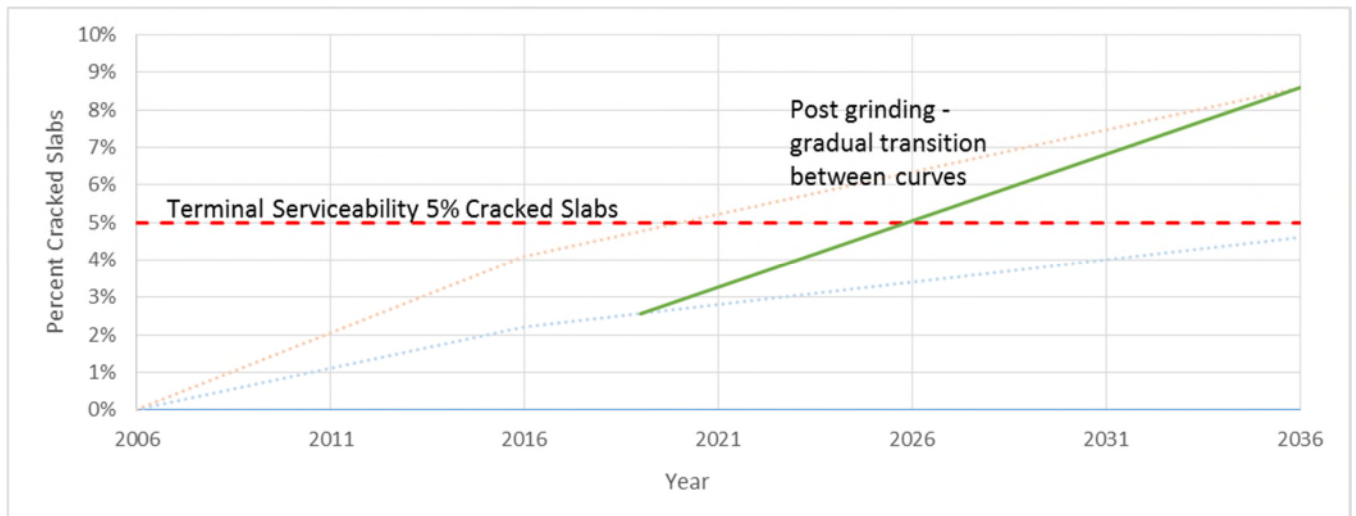
To apply StreetPave12, the prediction for cracked slabs was calibrated to observed conditions to determine the remaining service life. The calibration involved iteratively changing the reliability and modulus of rupture to result in approximately the observed amount of cracking after 11 years of service. This percentage of cracked slabs was estimated at 2.2 percent (%) based on the total number of cracked panels repaired in 2012/2013 and the number cracked panels observed in 2016. It was found that the software predicts that a 230 mm thick concrete should have 2.2% cracked slabs with 83% reliability and a concrete modulus of rupture of 4.5 MPa. The number of cracked slabs in 2016 was based on the LCMS data and the 4.5 MPa modulus of rupture was based on the design. The 83% reliability is effectively used as a calibration factor. The same reliability and modulus of rupture were then used to determine that 5% cracked slabs would be reached after 2036 (year 30). The percentage of cracked slabs of 5% also corresponds to typical terminal serviceability for a freeway such as SWAHD.

To determine the expected service life after grinding, the calibrated cracking model was used with a thickness of 220 mm (following removal of approximately 10 mm of PCC thickness resulting from diamond grinding). The cracking models for both 230 mm and 220 mm thick concrete are shown in Figures 9 and 10.



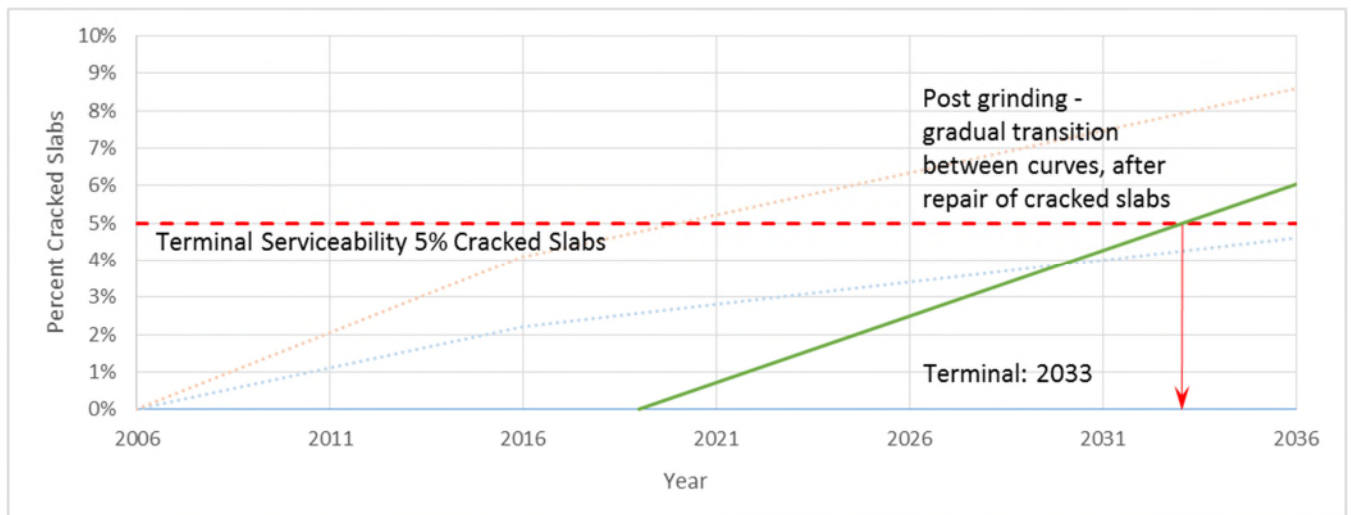
**Figure 9: Calibrated cracking Models Based on 83% Reliability and 4.5 MPa Modulus of Rupture**

Without grinding, a 230 mm thick doweled PCCP would be expected to reach terminal serviceability after 2036. Similarly, a pavement with a thickness of 220 mm (for its entire life) would be expected to reach terminal serviceability in 2019, after 28 million ESALs. In the case of Highway 216:06, the concrete thickness would change during the structural service life, an input option not available in StreetPave12. It is hypothesized that grinding would result in gradual transition from the cracked slabs curve for 230 mm thick concrete to the curve for 220 mm thick concrete. This transition is shown in Figure 10.



**Figure 10: Hypothesized transition of cracking from 230 mm thick curve to 220 mm thick curve**

Based on the transition from the lower to the higher percentage cracked slabs curve, the PCC would reach terminal serviceability in 2026, after 47 million ESALs. However, any future rehabilitation project with diamond grinding would include the repair of existing cracking; this would reset the cracked slabs to zero. This reset curve, is shown in Figure 11.



**Figure 11: Post-repair transition of cracking from 230 mm thick curve to 220 mm thick curve**

The cracked slabs of this reset curve will reach terminal serviceability in 2033, after 76 million ESALs. PCC thinner than 220 mm would be expected to have progressively higher risk of cracking and therefore grinding this PCC more than once is not advisable.



## **4.0 OTHER PERFORMANCE OBSERVATIONS**

### **4.1 Performance of Joint Sealants**

Based on the information received from the Alberta Transportation maintenance contractor and observations made during the field reconnaissance, the originally constructed joint sealants have performed poorly. Observations included the sealant being pushed out of the joints during rain events or after the rain events. This is likely caused by water trapped in the base layer underneath the PCC pavement. As with pumping, this water would get pressurized under heavy wheel loads as vehicles travel from one panel to another and force the sealant out of the joints.

### **4.2 Performance of Longitudinal Joints**

Tie bars were not installed at longitudinal joints at the locations of the lane merges and on and off ramps over concerns of excessive stresses resulting in cracking in the pavement. It was observed that at such locations, the outside concrete panels of the merge lanes have drifted resulting in wide longitudinal joints. These joints were observed to be as wide as 50 mm at some locations and the joint sealant in these locations was ineffective. These joints were observed to be filled with soil, and vegetation was growing at a few locations indicating that these joints are performing poorly, allowing the surface water to get into the pavement structure and pumping fines through these joints. A lane/shoulder drop off (slow lane to outside shoulder) adjacent to wider longitudinal joints was observed at a few localized locations.

## **5.0 FACTORS CONTRIBUTING TO POOR PAVEMENT PERFORMANCE**

### **5.1 Drainage**

The poor ride quality was attributed to extensive warping caused by moisture differential between the top (relatively dry) and bottom (relatively wet) of the PCC.

#### **5.1.1 Poor Sub-Surface Drainage**

The unsealed longitudinal and transverse joints in the PCC provide a conduit for surface water to flow into the GBC. The low hydraulic conductivity soils surrounding the GBC, as well as the limited capacity of the GBC to transmit water, means that water will tend to accumulate rather than being drained. This is exacerbated by the shallow drainage ditches which tend to have standing water in them.

Improvements to the sub-surface drainage can be made by minimizing the infiltration of water through the pavement structure and creating a hydraulic connection from the GBC to the surface.

#### **5.1.2 Surface Drainage**

Surface drainage was considered to be poor for the majority of the project length. Surface run-off water (from rain and/or melting snow) makes its way into the pavement structure through the open longitudinal joints, transverse joints and through the transverse joints from the edge of the pavement. This water has nowhere to escape and saturates the granular base and subgrade underneath the PCC pavement. Wet and saturated base and subgrade layers result in pumping of fines through the joints, formation of voids underneath the pavement, loss of support to the pavement and pavement failures.

Ponding of water adjacent to the pavement was also observed at numerous locations within the project limits. This water made its way into the pavement structure through the transverse joint and saturated the granular base and subgrade layer underneath the PCC pavement. This ponding water stays within the pavement structure as it has no easy escape route (owing to impermeable subgrade, flat ditches and clayey soil adjacent to the pavement) saturates and weakens the subgrade.

## **5.2 Absence of Tie Bars at Tie-In Locations**

Tie bars were not placed to tie the outside lanes with the merging lanes at the locations of the on and off ramps at various interchange locations within the project limits. This was done to avoid the generation of excessive cracking as the total concrete width would have become too wide (greater than 15m) at these locations.

The absence of the tie-bars at these locations has allowed the on and off ramp slabs to drift outwards resulting in longitudinal joints as wide as 50 – 60 mm. These joints were observed to be unsealed and filled with dirt. These wide-open longitudinal joints also allow an easy path for the water ingress into the pavement structure, pumping of fines and formation of voids underneath the pavements. The adjacent through lane was observed to be at a lower elevation compared to the adjacent shoulder / on and off ramp pavement at a number of such locations. The elevation difference at these locations was determined to be as high as 35 mm.

## **5.3 Misplaced Dowel Bars at Transverse Joints**

Misplaced dowel bars were identified at isolated locations during the detailed GPR survey. At these locations, the PCC was exhibiting faulting and exhibited very low LTE at the joints. Misplaced dowel bars do not provide any load transfer, which resulted in faulting of the pavement and contributed to poor ride quality and poor pavement performance at these locations.

Although, LTE at the remainder of the FWD test locations was considered good, some other locations exhibiting faulting were identified from the analysis of the profile data.

## **5.4 Traffic Loading**

The pavement structure design for the roadway was completed for a 30-year design traffic of 36.7 million ESALs. However, the actual traffic has grown at a faster growth rate and the actual traffic is projected to exceed the 30-year design traffic in year 2021 (15 years following the opening of the highway).

Although the existing pavement structure is considered adequate to support the traffic loadings in the interim, the need for diamond grinding of the pavement to address the rough ride quality offered by the pavement will result in thinning of the pavement structure. This thinner pavement structure combined with the higher design ESALs than originally anticipated will result in a pavement structure that will not be adequate for the traffic for an extended period.

## **6.0 REHABILITATION RECOMMENDATIONS**

Based on the results of the investigations, it was concluded that, before undertaking project wide rehabilitation, preliminary treatments to address the existing issues would need to be completed to

improve the pavement performance and extend the service life of any recommended rehabilitation treatment.

## **6.1 Preliminary Treatments**

The recommended preliminary treatments to be completed prior to any major rehabilitation were:

### **Drainage Improvements:**

It was recommended that drainage improvements including re-grading of the median, shoulders adjacent to the pavements, and the ditches, be carried out to improve the performance of the pavement. The shoulders were recommended to be re-graded to re-establish positive drainage and prevent ponding of the water adjacent to the pavement. Edge drains should also be constructed to drain the water away from the edge of the pavement.

### **Slab Stabilization:**

Stabilization / jacking of the slabs should be undertaken at locations of lane / shoulder drop offs or at faulting locations to restore the elevation with respect to the adjacent slabs.

### **Panel Replacements:**

The replacement of the panels with multiple full width cracks should be completed (including dowel and tie bar replacement). Dowel Bar Retrofit (transverse cracks) or cross-stitching (longitudinal cracks) of panels with a single full width crack would be enough.

### **Dowel Bar Retrofits:**

Dowel bar retrofit should be completed at all locations identified as having dowel bar misalignments. A total of three 450 mm long, 32 mm dowel bars spaced at 300 mm spacing should be installed in each wheelpath at each location.

### **Slot-Stitching of Longitudinal Joints:**

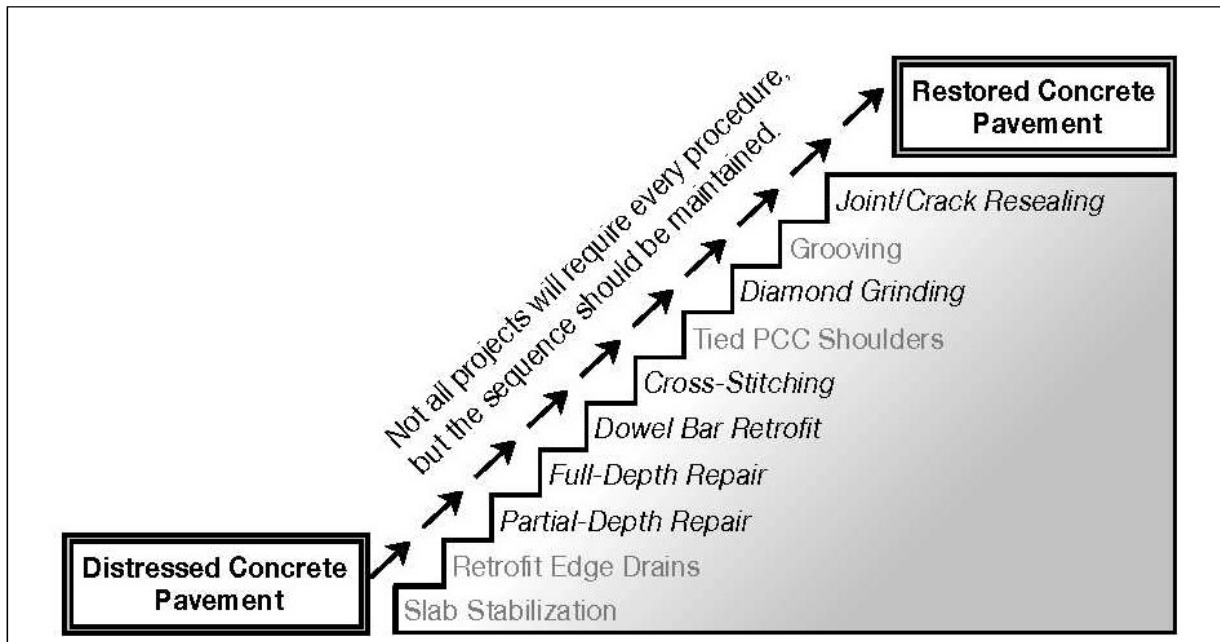
Tie bars (800 mm long, 15M deformed bars placed at 900 mm spacing) should be installed at all longitudinal joint locations exhibiting open longitudinal joints.

### **Joint Sealant Repairs:**

Sealing of the transverse joints and longitudinal joints should be undertaken to prevent the ingress of surface water into the pavement structure. It was further recommended that the dropped off lanes / shoulders be mud jacked prior to sealing the longitudinal joints (especially at the lane and ramp merge points) to block off the ingress of surface water into the pavement structure.

### **Sequence of Repairs**

A logical sequence of repairs is critical to prevent damage to the previously completed repairs. Figure 12 shows the recommended sequence of the preliminary rehabilitation treatments.



**Figure 12: Sequence of Concrete Pavement Repairs<sup>1</sup>**

<sup>1</sup> Source – Concrete Pavement Field Reference Preservation and Repair by American Concrete Pavement Association

## 6.2 Rehabilitation Options

Given the average IRI of the PCC, which was 2.34 mm/m in 2016, rehabilitation options were also examined to restore the serviceability of the PCC. Placement of a PCC overlay was considered as a rehabilitation treatment. However, it was not included as a feasible option given the expense, construction challenges and lengthy traffic disruptions. Similarly, rubblization of the pavement was not considered appropriate as the existing pavement was only 12 years old and limited extent of fatigue cracking.

### 6.2.1 Option 1 - Diamond Grinding

Completion of the preliminary repair treatments identified above followed by diamond grinding of the roadway was considered as a rehabilitation option. This treatment would result in improvement of the ride quality of the roadway. Consideration would need to be given to limiting diamond grinding to the areas that were not diamond ground in 2013 unless those areas were exhibiting a high IRI.

Based on the review of the IRI data for the roadway segments that were previously diamond ground in 2013, it is estimated that it would take approximately 14 to 16 years for the pavement to come back to current IRI levels following diamond grinding.

### 6.2.2 Option 2 - Asphalt Concrete Overlay

Placement of a 100 mm asphalt concrete pavement (ACP) overlay after the completion of all the repairs to the PCC pavements was also considered as a rehabilitation of the pavement. Given that the transverse and longitudinal joints from the PCC pavement would reflect through to the new ACP surface shortly after

the placement, this repair option also recommended saw cutting of the asphalt overlay at the existing concrete joint locations right after paving to simplify crack sealing and reduce the amount of the raveling due to wandering of the reflective cracking. The anticipated service life of this treatment was 14 years based on the high traffic volumes.

### 6.2.3 Life Cycle Cost Analysis of Rehabilitation Options

A 30-year Life Cycle Cost Analysis (LCCA) of the two options, using a four percent discount rate, was conducted and is presented in Table 4.

**Table 4: Summary of LCCA Results**

Alternative (Service Life)	Initial Capital Cost (Million \$)	Net Present Value (NPV, Million \$)	Rank	NPV Comparison
Option 1: Diamond Grinding (16)	12.02	21.47	1	1.00
Option 2: 100 mm asphalt overlay (14)	15.84	23.18	2	1.08

The LCCA excluded the costs of the recommended preliminary repair treatments (because they were common to both recommended options) and user delays. However, the LCCA did include future anticipated repair costs for both options (e.g. additional concrete repairs, re-sealing of joints). The results showed that the two options vary by a net present value of approximately eight percent.

## 7.0 CONCLUSIONS

Based on the results of the 2017 investigations into the performance of the PCC pavement, it was concluded that poor drainage was the leading cause of the poor pavement performance. The lack of drainage is holding water adjacent to the PCC and contributed to the warping of the concrete panels. This in turn is negatively impacting the ride quality. Other findings from the investigation indicate localized issues (e.g. misaligned dowel bars, panel drifting) but generally acceptable performance of the PCC (e.g. cracked slabs of 2.2%, generally adequate LTE, and good concrete durability).

An analysis of the traffic data showed that the pavement is nearing the end of its original design ESAL. However, with appropriate repairs to both the PCC and drainage elements, it is anticipated that another 14 to 16 years of performance could be obtained. It is also anticipated that Alberta Transportation will undertake the preliminary treatment repairs and rehabilitation of the PCC starting in 2019.

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