Technical Memorandum

To: Jeff Uhlmeyer

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cc: Mustafa Mohamedali

Date: April 23, 2019 (original)

Re. Forensic Desktop Study Report: Arizona SPS-2 Test Sections 0214, 0215, 0217 and 0262

The Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS) project 040200[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations" due to the presence of test sections that have demonstrated much better performance than other sections within the same project. Two sections with relatively good performance and two with relatively poor performance were selected for analysis and comparison. The purpose of this document is to review the history examine performance over time for four Arizona SPS-2 test section (040214, 040215, 040217 and 040262). Further, on the basis of the resulting information, recommendations are provided regarding the need for a field forensic evaluation to better explain the performance of the test sections over time.

SITE DESCRIPTION

The SPS-2 project site is located on eastbound Interstate 10, starting at milepost 105.95, in Maricopa County, Arizona. This is a two-lane rural interstate highway in the direction of travel. It is classified as being in a Dry-No Freeze climate zone with an average annual precipitation of 8.2 inches and an annual average air freezing index of zero 32 Deg-F degree-days. The coordinates for the start of the project are 33.453080, -112.73961. Figure 1 shows the geographical location of the project within the State of Arizona, while Figure 2 shows the actual location for each section within the project. The identified sections for this desktop study include a red star on the layout. Pictures showing the general location and surrounding landscape of each test section are presented in Figures 3 through 6, which were obtained in 2019, from the start of the section looking east.

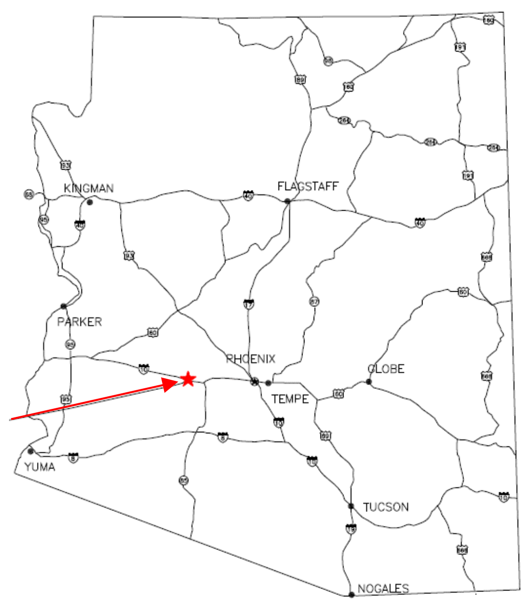

Figure 1. Geographical location of test sections within Arizona

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[1] *First two digits in test section number represent the State Code [04 = Arizona]. 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., 14).*

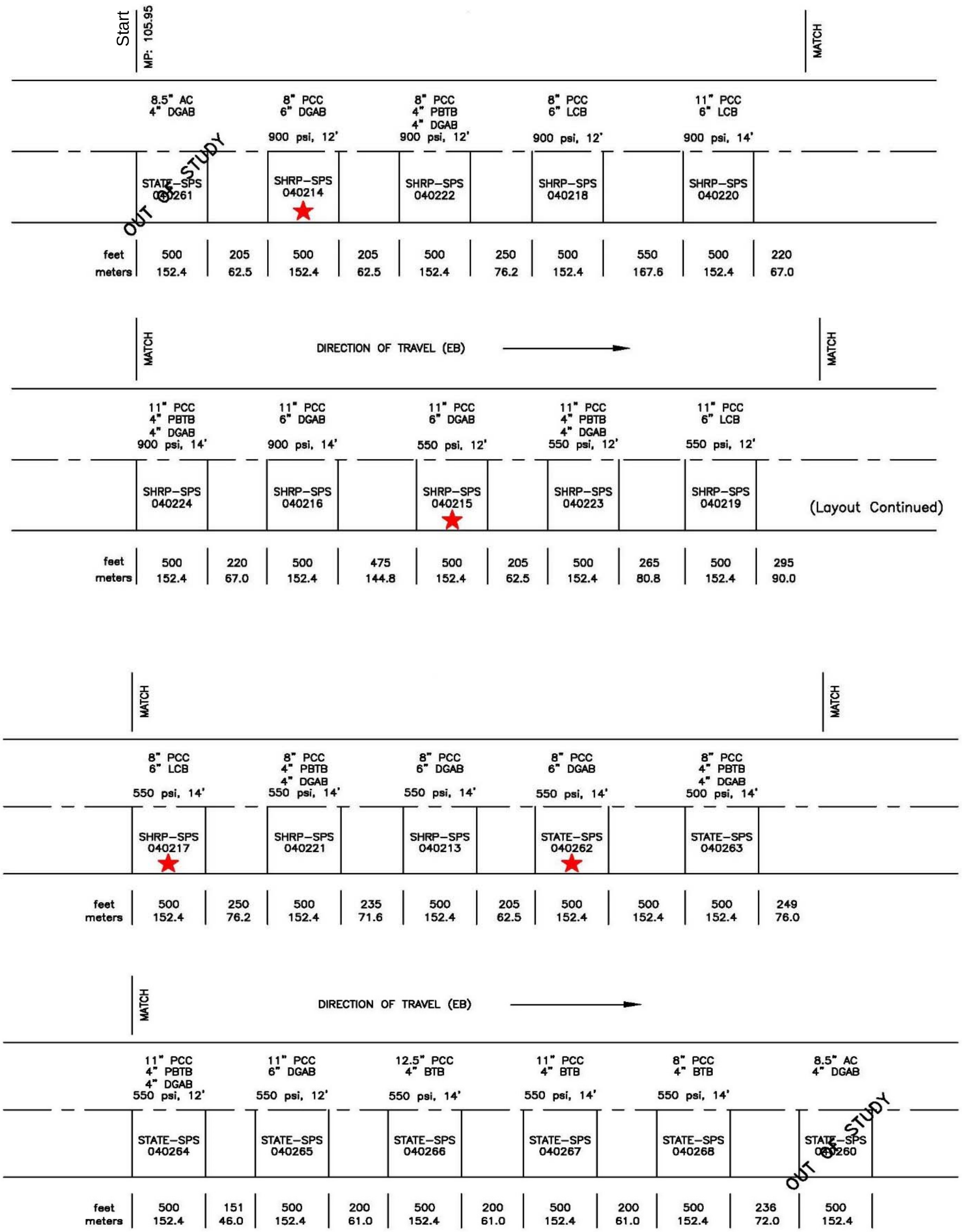

Figure 2. Location of selected test sections within the Arizona SPS-2 project.

Figure 3. Test section 040214.

Figure 4. Test section 040215.

Figure 5. Test section 040217

Figure 6. Test section 040262

BASE-LINE PAVEMENT HISTORY

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

Pavement Structure and Construction History

Construction of the Arizona 040200 project began in February 1993 and the concrete pavement sections were opened to traffic in October 1993. The original layer structure for each section selected for the forensic desktop study is detailed in Table 1. This corresponds to CONSRTUCTION_NO = 1 (CN1). Lane width and flexural strength were also experimental variables in the SPS-2 experiment and are detailed in Table 2. The core SPS-2 sections, included in this desk study (040214, 040215 and 040217) are doweled concrete pavement, while the State supplemental section (040262) is undoweled.

Table 1. Pavement structure from 1993 to date.

Section	Layer No.	Layer Type	Thickness (in.)	Material Code Description
040214	1	Subgrade (untreated)		215 - Coarse-Grained Soil: Silty Sand with Gravel
	2	Unbound (granular) base	6.1	304 - Crushed Gravel
	3	Portland cement concrete layer	8.3	4 - Portland Cement Concrete (JPCP)
040215	1	Subgrade (untreated)		215 - Coarse-Grained Soil: Silty Sand with Gravel
	2	Unbound (granular) base	6.3	304 - Crushed Gravel
	3	Portland cement concrete layer	11	4 - Portland Cement Concrete (JPCP)
040217	1	Subgrade (untreated)		215 - Coarse-Grained Soil: Silty Sand with Gravel
	2	Bound (treated) base	6.1	334 - Lean Concrete
	3	Portland cement concrete layer	8.1	4 - Portland Cement Concrete (JPCP)
040262	1	Subgrade (untreated)		215 - Coarse-Grained Soil: Silty Sand with Gravel
	2	Unbound (granular) base	6.1	304 - Crushed Gravel
	3	Portland cement concrete layer	8.1	4 - Portland Cement Concrete (JPCP)

Table 2. Additional experimental factors.

Section	Slab Width (ft)	Concrete Mixture 14-day Design Flexural Strength (psi)
040214	12	900
040215	12	550
040217	14	550
040262	14	550

Sections 040214 and 040215 only have one CN, which corresponds to the date when the sections were accepted into the LTPP experimental program.

The 040217 construction events are as follows:

CN1 – Test section accepted into the LTPP program, January 1993.

CN2 – Partial-depth patching at joints and other than at joints, August 2009. Patching did not meet minimum LTPP size requirement (≥ $1ft^2$) and were not rated on the distress surveys.

CN3 – Partial-depth patching at joints, February 2016.

The 040262 construction events are as follows:

CN1 – Test section accepted into the LTPP program, January 1993.

CN2 – Partial-depth patching other than at joints, August 2009. Patching did not meet minimum LTPP size requirement (≥ $1ft^2$) and were not rated on the distress surveys.

CN3 – Partial-depth patching at joints, January 2013. Patching did not meet minimum LTPP size requirement (≥ $1ft^2$) and were not rated on the distress surveys.

Figure 7 displays the recorded quantities of patching. Only Section 040217 (CN3, February 2016) had patching recorded that met the LTPP minimum size ($0.1m^2$). The quantity of patching (2.2 ft^2) was small considering the overall area of the test section.

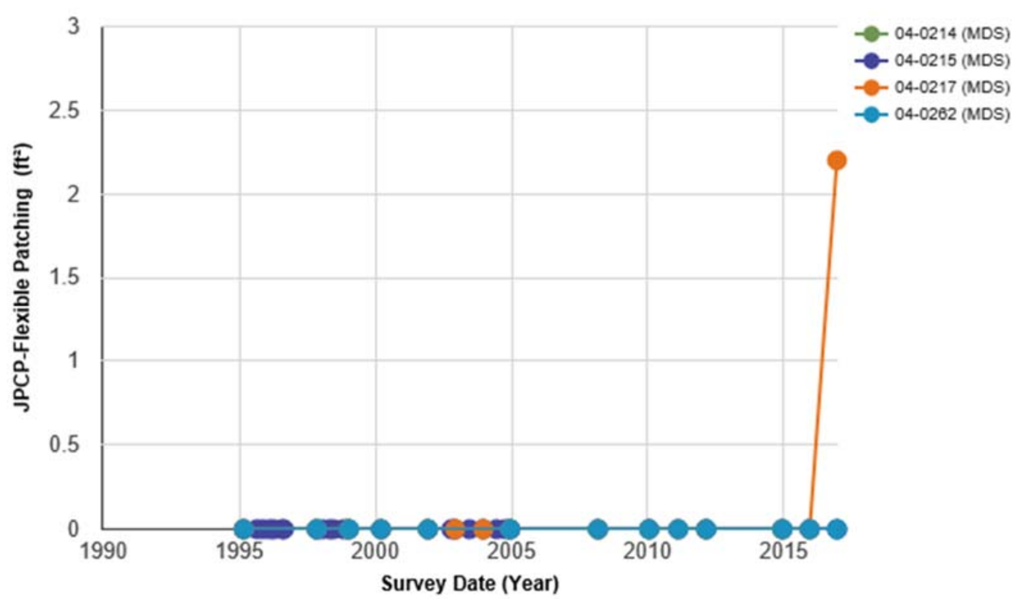

Figure 7. Time history of flexible patching.

Climate History

The time history for annual average precipitation since 1993 is shown in Figure 8. Except for 1993 and 1998, average precipitation was between 4 and 10 inches. Annual precipitation was nearly 15 inches in 1993 and 12 inches in 1998. These data indicate that the construction year experienced a higher than usual precipitation. All other years have annual precipitation values that are expected for an arid, desert climate (dry, no-freeze). This location does not experience freeze-thaw cycles.

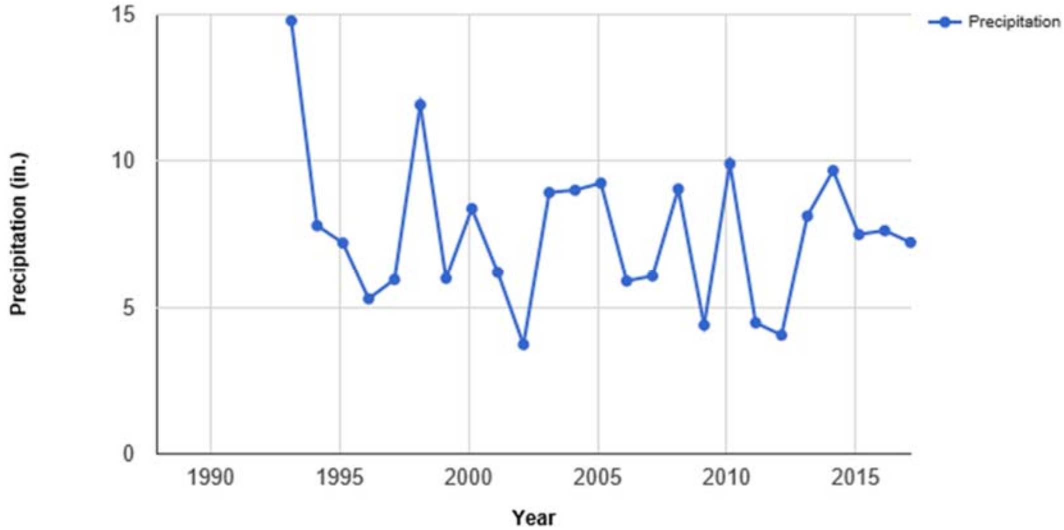

Figure 8. History of annual precipitation starting in 1993.

Truck Volume History (Traffic)

Figure 9 shows annual truck volume data in the LTPP test lane by year. The triangles represent estimated data provided by Arizona State DOT. The blue diamonds are truck counts based on monitoring data, reported to or collected by LTPP. Estimated truck count data from 1999 to 2002 were provided by the agency; they were not collected directly from the Weigh-In-Motion (WIM) stations. For analysis purposes, it is recommended that the truck counts from 1999 to 2002 be ignored due to data not coming from the WIM, but from agency estimated information.

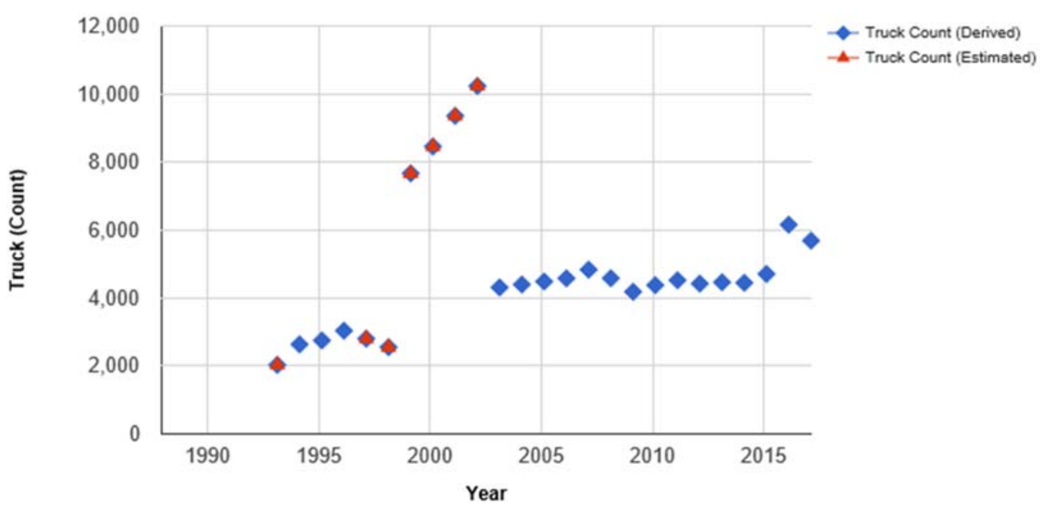

Figure 9. Average annual daily truck traffic on project 040200 based on state supplied estimates and monitoring measurements.

In March 2005, "*Estimating Cumulative Traffic Loads, Volume II: Traffic Data Assessment and Axle Load Projection for the Sites with Acceptable Axle Weight Data, Final Report for Phase 2*, was published. This report provided a summary of traffic projections; with an excerpt provided in Table 3 for the 040200 experiment. As displayed in this table, the AZ SPS-2 project highway functional class is a Rural Principal Arterial – Interstate (1) with traffic counts at 3,290, an annual growth rate of 6% and 1,706,650 ESALs in 1998.

Table 3. Excerpt from Summary of Traffic Projection Results from Estimating Cumulative Traffic Loads, Volume II: Traffic Data Assessment and Axle Load Projection for the Sites with Acceptable Axle Weight Data, Final Report for Phase 2.

Agency	SHRP ID	Traffic ID	GPS/ SPS	Exp. No.	Highway Functional Class	Projections			Pav. Type	Last Pr. Year	Traffic Characteristics		
						Cat.	Code	Status			Traffic Volume	Annual Growth Rate (%)	ESALs
AZ	0200	040200	S	2	1	2	A	R	R	1998	3,290	6.00	1,706,650

IRI Roughness Time Histories

The time history of roughness measurements for the four test sections in the study are shown in Figure 10. In Figures 12 through 15, IRI is broken up by section with the vertical lines indicating construction events, which are detailed under the Pavement Structure and Construction History section of this report. According to the FHWA performance standard, IRI is considered in the "Good" category if the IRI is less than 95 inches/mile. If the IRI is between 95 and 170 inches/mile it is considered "Fair", and an IRI greater than 170 inches/mile is considered "Poor". As of 2016 the only section with an IRI in the "Good" category is 040217. Sections 040214 and 040215 are in the "Fair" category, and 040262 is considered "Poor".

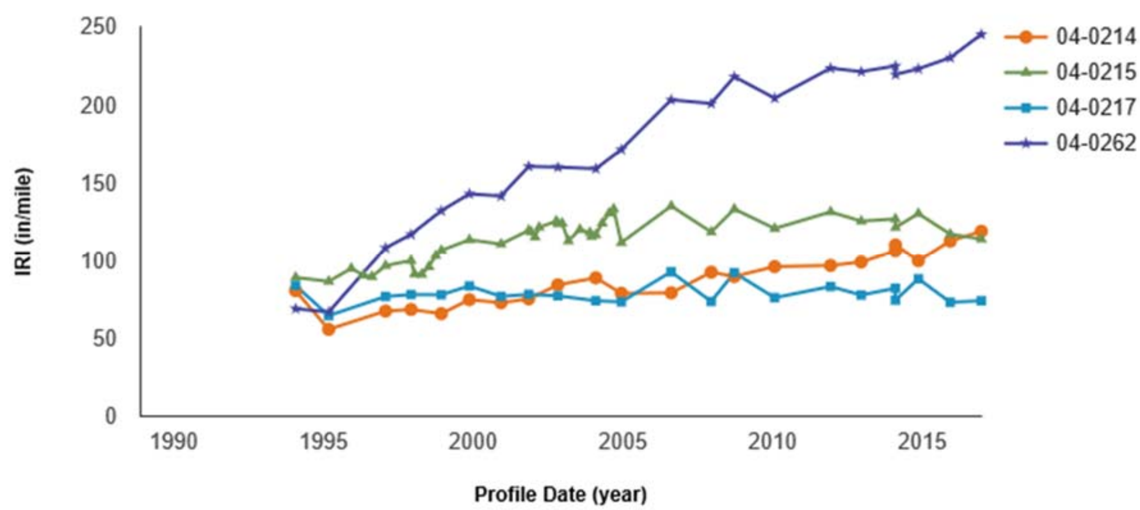

Figure 10. Time history plot of pavement roughness for 040214, 040215, 040217 and 040262.

Figure 11 shows that section 040214 stayed within the "Good" category according to FHWA's performance standard, until January 2010 when it reached 96 inches/mile. From there the IRI has gradually increased to 119 inches/mile in December 2016. The unexpected drop in post-construction IRI was likely related to data collection during different seasons and time-of-day as these can affect profile measurements used to calculate IRI. Ambient temperature impacts the temperature gradient within a slab and causes a warp or curl effect. The initial survey in January 1994 was collected from 6:00pm to 8:00pm while the next survey was collected in March 1995 at 11:20 am.

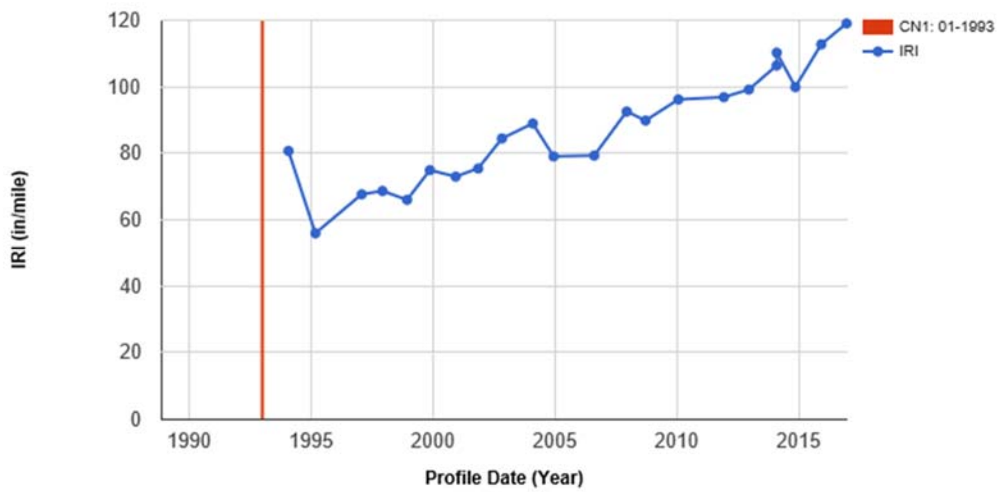

Figure 11. Section 040214 IRI and Construction Time History Plot.

Section 040215 IRI, shown in Figure 12, fluctuated within the "Good" category from January 1994 to July 1998, going a little bit above the 95 inches/mile "Good" category range in 1997 maxing at 100 inches/mile. In September 1998, IRI entered the "Fair" category and has fluctuated within this category. The IRI in December 2016 was 114 inches/mile.

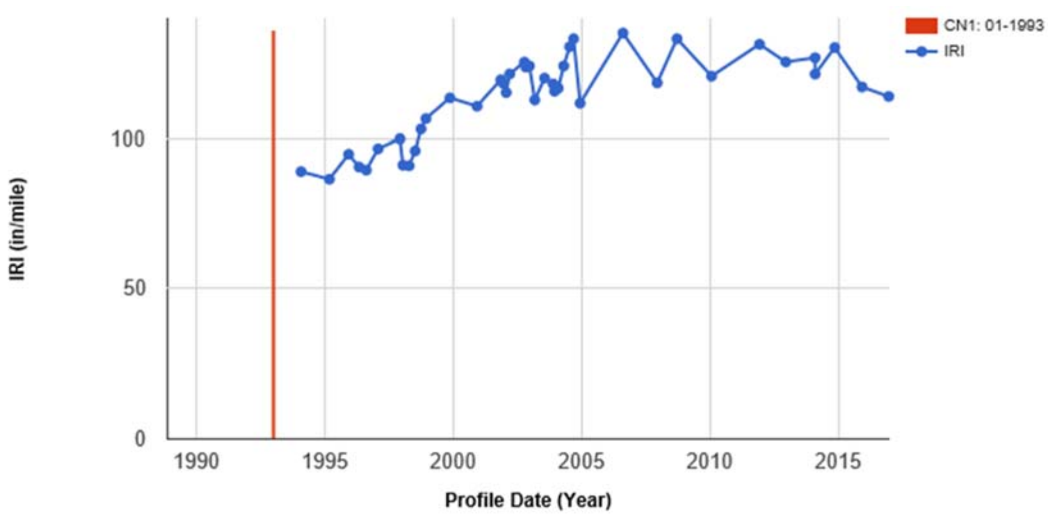

Figure 12. Section 040215 IRI and Construction Time History Plot.

Section 040217 shown in Figure 13 is the only section in this study that has continuously stayed within the "Good" category since it was accepted into the program. From 1994 to 2004 the IRI has been somewhat linear except for in 1995 where it dropped down 19 inches/mile to 65 inches/mile. This highest IRI value of 93 inches/mile was recorded in August 2006. In 2010 CN2 occurred and the IRI value decreased to 76inches/mile. After CN2 the value fluctuated from 76 up to 88 inches/mile. After CN3 it stayed relatively the same and in 2016 IRI was 74 inches/mile.

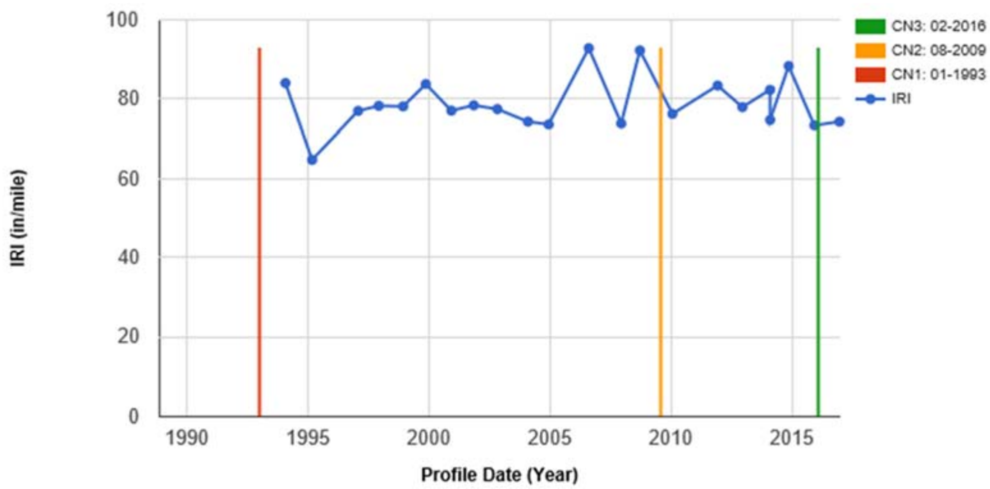

Figure 13. Section 040217 IRI and Construction Time History Plot.

Section 040262, shown in Figure 14, is the only section that has reached the unacceptable IRI limit according to FHWA's standard. In 1994, the roughness was at 69 inches/mile with a steady increase until the patching event occurred in August of 2009. IRI was considered "Good" for only the first two years after construction, 1994 and 1995. In 2004 the section transitioned from "Fair" to "Poor" category with an IRI of 171 inches/mile. After the August 2009 patching event (CN2) IRI dropped 14 inches/mile from 218 to 204 inches/mile and continued to increase thereafter. More patching occurred (CN3) in January 2013, but IRI after the CN event was nearly identical.

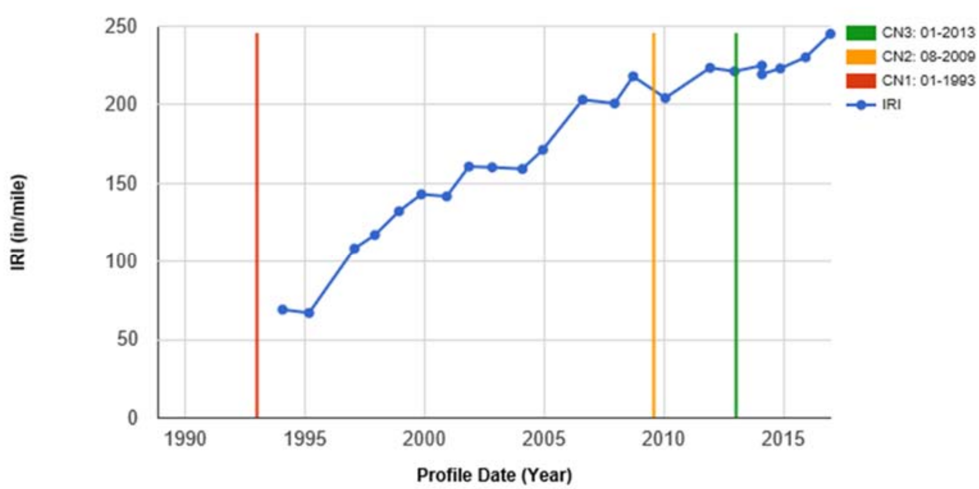

Figure 14. Section 040262 IRI and Construction Time History Plot.

Deflection Under Load Plate Time Histories

The time history of average deflection under the load plate, normalized to a 9,000 lb. drop load, for the four test sections in the study are shown in Figure 15. The patching CN events do not affect slab deflection and therefore are not included in this discussion. FHWA does not have performance categories for deflection. Deflections for Sections 040215 and 040217 were very low (generally less than 4 mils) and remained constant throughout the 20+ year monitoring period. This was likely due to the presence of a thicker pavement slab (11 inches) in Section 040215 and the presence of a treated base (with an 8-inch slab) in Section 040217. Sections 040214 had steadily increasing deflection values with a sudden increase around 2012. This data point cannot be explained with available data. Section 040262 exhibited a trend of increased deflection until 2005 when values remained constant or dropped slightly. An explanation for this trend was that both sections were constructed with an 8-inch PCC slab over approximately 6-inches of unbound granular base and the pavement structures had less overall stiffness (compared to Sections 040215 and 040217).

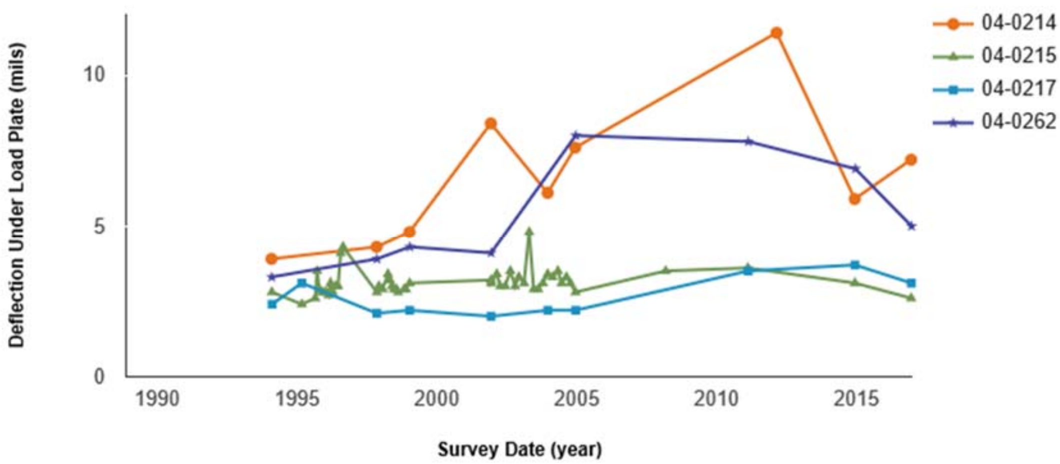

Figure 15. Time history plot of average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load for 040214, 040215, 040217 and 040262.

Figure 16 provides a comparison of the load transfer efficiency (LTE) for the four sections. The time history of LTE confirms that dowels are an important aspect for load transfer. The undoweled section (040262) exhibited poor load transfer (as of 2014) compared to sections 040214 and 040215, which maintained good LTE throughout the monitoring period. It is not immediately clear why LTE started at 46% in 1994 and then increased to 77% in 1999. The continual decrease from 1997 is related to the lack of dowel bars. Immediately following construction, LTE was good for Section 040217, decreased rapidly until 1997, and then remained constant at round 50% for the remainder of the study period. Longitudinal and transverse cracking occurred early on in this section and increased throughout the 20+ year monitoring period and this was the only section constructed on treated base material. The exact cause of the sudden drop in LTE could not be determined from these data.

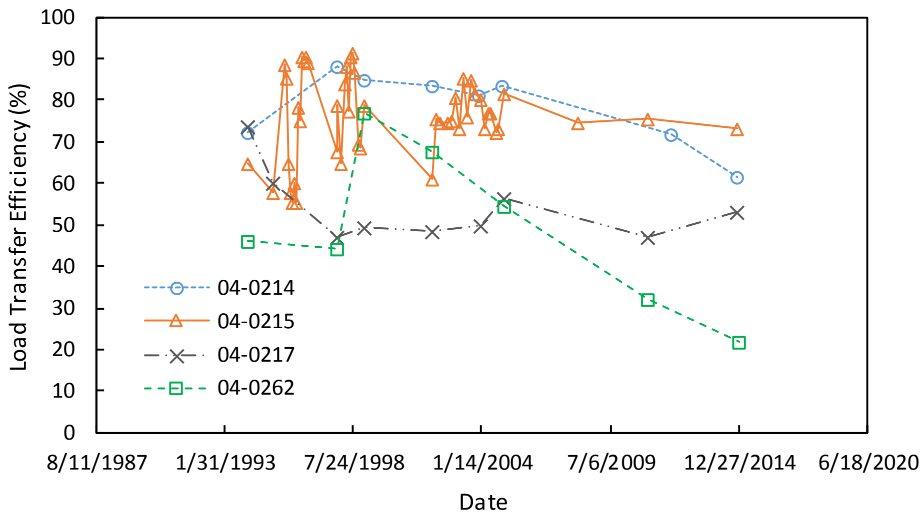

Figure 16. Load transfer efficiency summary.

Longitudinal and Transverse Cracking Time Histories

Figure 17 shows the time history for longitudinal cracking length for the four Arizona SPS-2 sections of interest. Section 040217 has the largest amount of longitudinal cracking at 699 ft, 040262 has 144ft of longitudinal cracking and 040214 and 040215 have a very small amount of longitudinal cracking, at less than 3ft of longitudinal cracking. Sections 040214, 040217 and 040262 all exhibit a large amount of map cracking. This observation was logical for 040214 given a higher amount of total cementitious content to achieve a 900-psi design strength. However, sections 040217 and 040262 used a 550-psi design strength and the extent of map cracking was more significant than observed on other 550-psi sections in this experiment. The development of map cracking could have been an artifact of poor curing during construction or the onset of alkali-silica reactivity.

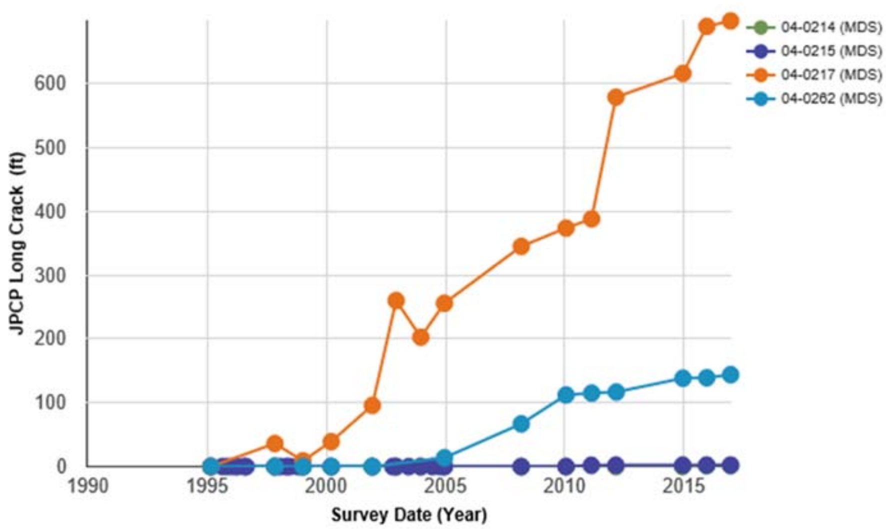

Figure 17. Time history of longitudinal cracking length.

Similarly, Figure 18 shows that 040217 has the most transverse cracking, whereas the other sections had little to no transverse cracks. Section 040217 has 25 cracked slabs (transverse crack greater than or equal to 1.9ft) out of 34 slabs which translates to 73% of slabs cracked. Sections 040214 and 040262 have one cracked slab with less than 3% slabs cracked and 040215 has no cracked slabs. According to the FHWA performance standard, 040214, 040215 and 040262 are considered “Good” and 040217 has greater than 15% cracking, putting it in the “Poor” category.

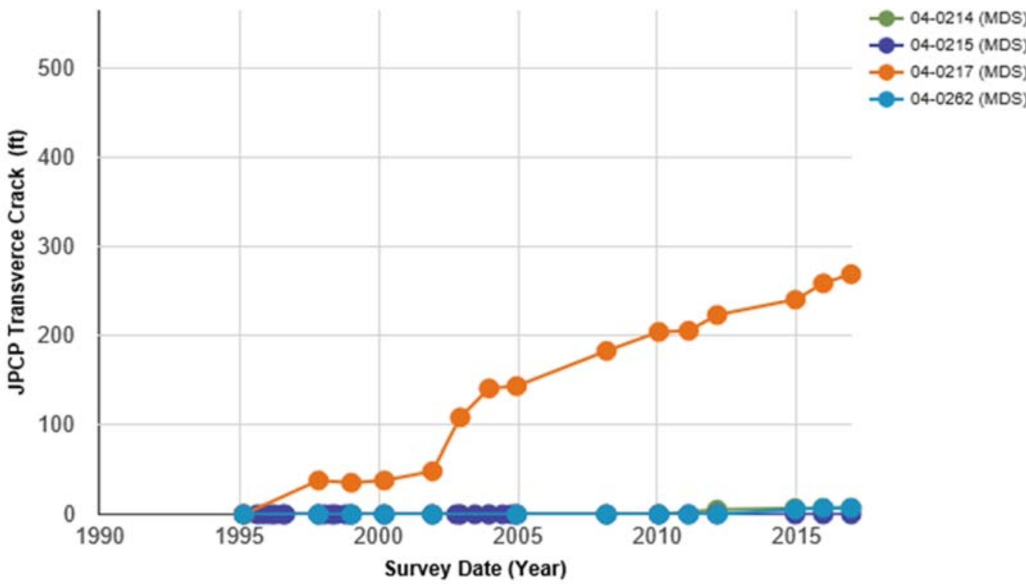

Figure18. Time history of transverse cracking length.

Figure 19 shows the time histories for spalling at transverse joints, indicating the number of affected joints. Based on the LTPP Distress Identification Manual (DIM), a transverse joint is considered affected if spalling is present for 10% or more of the length of the joint. Minor spalling at transverse joints is present on all sections with 040262 having the most spalling with five affected transverse joints. This undoweled section exhibited poor load transfer and resulting joint movement was a likely factor in the steady increase in spalled joints. Sections 040214 and 020215 did not exhibit spalling until approximately 18 years after construction. At that point, only one to two joints were affected. Section 040217 had a CN in 2009 that included partial-depth patching at the joints. This explains the drop in affected joints around that time period. The spike in spalled joints in 1997 was likely rater variability as these data points for 040214 and 020215 do not follow the trends for those sections.

Rutting (Surface Wear) Time Histories

Although typically collected on asphalt pavements, a Dipstick® was occasionally used to measure transverse profile on the four concrete SPS-2 sections in this study. For concrete pavements, wheel path ruts develop as a result of surface wear (paste and aggregate) and the distress is classified as rutting in LTPP. Figure 20 shows the rutting time history from Dipstick® data. Rut depths measured in March 1995, two years after construction began, ranged from 0.16 to 0.2 inches. This initial rutting is a result of the “weaker” tining ridges wearing away and once coarse aggregate is exposed, the rate of wear decreases. Sections 04-0215 and 04-0217 did not exhibit a change in rutting over the 20-year monitoring period. Data show a decrease in Section 04-0214 and 04-0217 rutting between 1995 and 2003. This trend could be due to rater variability or changes in the elevation of the starting point to collect transverse profile data (i.e., if the tining between the wheel path and fog line wore away at a faster rate than the wheel path itself [between 1995 and 2003], the Dipstick starting elevation would have been lower on subsequent measurements and would have underestimated rut depths compared to the original datum point (with tining present at fog line).

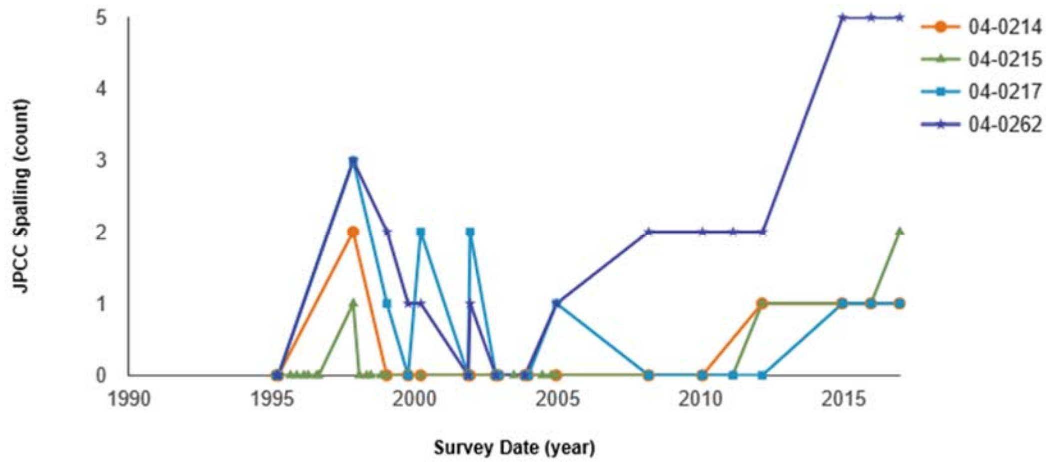

Figure 19. Time history of total number of JPCC spalling at transverse joints.

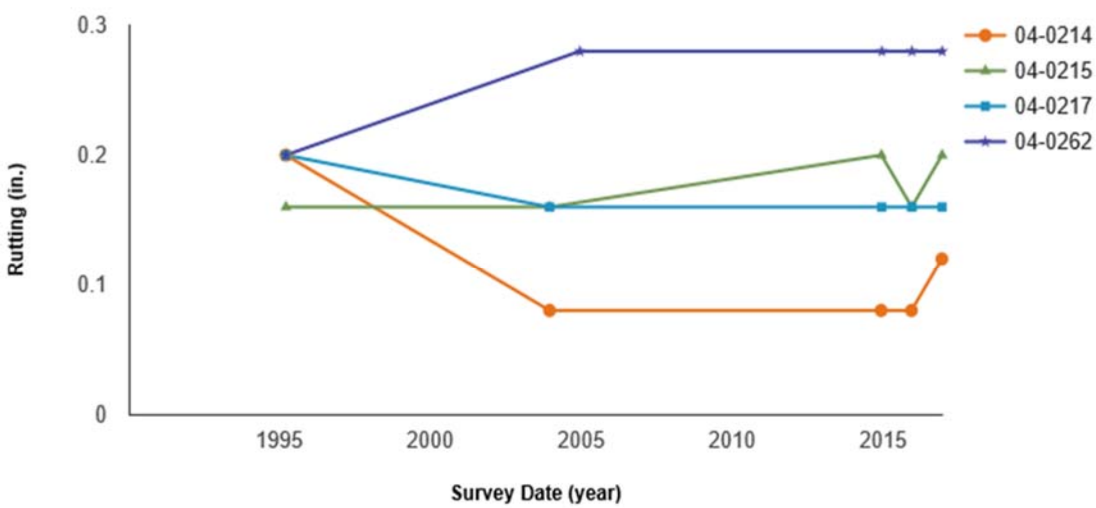

Figure 20. Time history plot of average rut depth computations.

According to the rutting FHWA performance standard on asphalt pavement, less than 0.2 inches is considered “Good”, rutting between 0.2 and 0.4 inches is considered “Fair” and rutting greater than 0.4 inches is considered “Poor”. Section 040262 was the only section considered to be “Fair” with rutting between 0.2 inches and 0.3 inches in every survey, and in 2014 section 040215 moved into the “Fair” category.

Faulting Time Histories

Figure 21 shows the faulting time histories on the four Arizona SPS-2 sections. Most of the faulting values are positive fault heights, which occur when the leave side of the joint has a higher elevation than the approach. According to Figure 25, section 040262 had the most faulting present while the other sections have minimal to no faulting. The FHWA performance standard for faulting is as follows; faulting less than 0.1 inches is "Good", faulting between 0.1 and 0.15 inches is "Fair", and faulting greater than 0.15 inches is considered "Poor". According to the faulting performance standards 040262 has been in the "Poor" category since 2001. The three other sections remained below 0.1 inches, putting them all into the "Good" category. The steady increase in faulting is thought to be due to Section 040262 being undoweled; the other three sections were constructed with dowel bars.

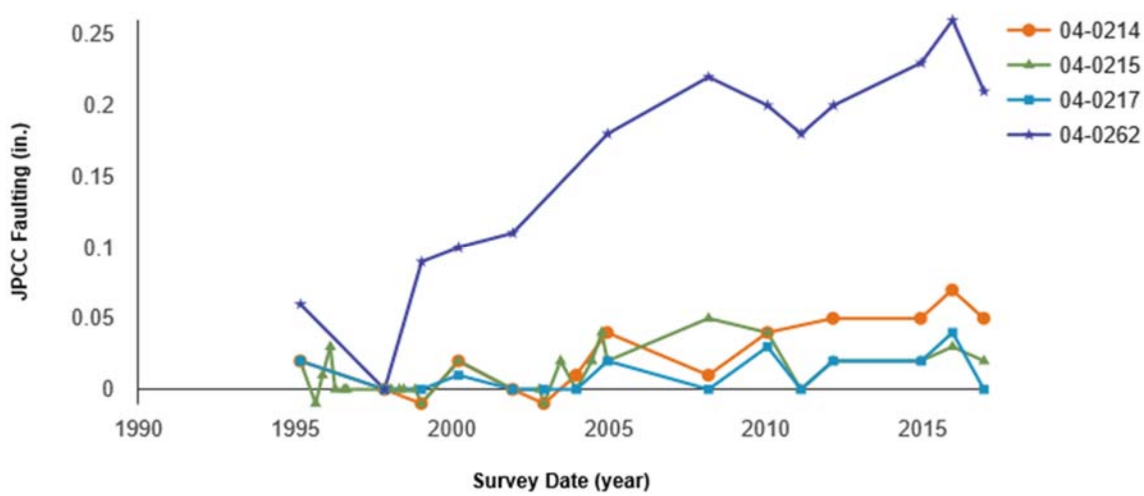

Figure 21. Time history plot of faulting.

SUMMARY OF FINDINGS

Table 4 provides a summary of the experimental factors for each section along with the FHWA performance categories and other performance indicators for 2016 distress levels.

The following observations are presented from the desktop study of four sections from the LTPP SPS-2 experiment (040214, 040215, 040217 and 040262):

- Section 040214 had minimal distress development across all categories; as of 2016, ride quality remained in the FHWA "fair" category at 119 inches/mile, cracking and faulting were minimal, rutting was less than 0.15 inches, deflection was around 7 mils, and LTE was around 62%. However, significant map cracking developed in this section and photos indicate these map cracks have become wider at the surface (with spalling) and some appear to have developed into full-depth cracks. This section was characterized by an 8-inch PCC, 900-psi flexural strength mixture, unbound granular base, and a 12-foot lane.

Table 4. Summary of experimental factors and performance as of 2016.

Experimental Features					FHWA Performance Category				Other Performance		
Section	**PCC (in.)**	**Base (6-in.)**	**Slab Width (ft)**	**Design Flexural Strength (psi)**	**IRI**	**Fault**	**Trans. Crack.**	**Rut**	**Long. Cracks**	**Significant Map Cracks**	**LTE**
040214	8.3	Crushed gravel	12	900	Fair	Good	Good	Good	Minimal	Yes	Good
040215	11	Crushed gravel	12	550	Fair	Good	Good	Fair	Minimal	No	Fair
040217	8.1	Lean concrete	14	550	Good	Good	Poor	Good	Some	Yes	Fair
040262[1]	8.1	Crushed gravel	14	550	Poor	Poor	Good	Fair	Many	Yes	Poor

[1]*undoweled*

- Section 040215 had minimal distress development across all categories; as of 2016, ride quality remained in the FHWA "fair" category at 114 inches/mile, cracking and faulting were minimal, rutting was 0.2 inches, deflection was very low, and LTE was 73%. Significant map cracking was not documented by raters. This section was characterized by an 11-inch PCC, 550-psi flexural strength mixture, unbound granular base, and a 12-foot lane.
- Section 040217 had moderate distress development across most categories except this section was the only one that remined in the FHWA "good" category for roughness with IRI at 74 inches/mile (2016). Longitudinal cracking was 699 linear feet and transverse cracking was 269 linear feet (most of the four sections). Faulting was minimal, rutting was less than 0.2 inches, deflection was very low, and LTE was 53%. Significant map cracking was noted. This section was characterized by an 8-inch PCC, 550-psi flexural strength mixture, lean concrete base, and a 14-foot lane.
- Section 040262 exhibited the most distress development in all categories except for transverse cracking. As of 2016, IRI was 245 inches/mile (FHWA "poor" category), deflection was 5 mils, LTE was 22%, longitudinal cracking was 144 feet, joint spalling was the highest of the four sections (5 affected joints), rutting was nearly 0.3 inches (FHWA "fair" category), and average faulting was over 0.2 inches. Significant map cracking was noted. This section was characterized by an 8-inch, undoweled PCC, unbound granular base, low flexural strength, and a 14-foot lane.
- It is unclear why Sections 040217 and 040262, with 550-psi flexural strength concrete, exhibited significant map cracking when other 550-psi sections in the experiment did not. This could have been the result of poor curing practice during construction or the onset

and progression of alkali-silica reactivity. The latter of which cannot be confirmed without additional study.

CONCLUSIONS

The following conclusions are presented from the desktop study of four sections from the LTPP SPS-2 experiment (040214, 040215, 040217 and 040262):

- The presence of a lean concrete base had a positive impact on IRI as section 040217 was the only section that maintained "good" IRI throughout the monitoring period. The IRI in 2016 was approximately 80 inches/mile, an excellent value for 20+ year old concrete pavement that has carried heavy truck traffic and received little maintenance.
- Sections with 14-foot slab widths were more prone to longitudinal cracking and the presence of LCB (Section 040217) had negative impact on transverse cracking. Sections 040214 and 040216, with 12-foot lane widths, had minimal cracking over the monitoring period.
- As expected, the undoweled pavement (Section 040262) had the highest IRI (nearly 250 inches/mile), the highest faulting (>0.2 inches), and lowest load transfer efficiency of all four sections. This finding wasn't surprising giving the current body of knowledge on doweled versus undoweled pavements.
- A conclusion regarding map cracking cannot be drawn without further study. Possible causes could be shrinkage from improper curing, alkali-silica reactivity, or combination of factors.
- Minimal distress developed and at least "fair" ride quality was maintained in two combinations of 12-foot-wide doweled concrete pavements over crushed aggregate base: 1) 11-inch concrete pavement with 550 psi flexural strength, and 2) 8-inch concrete pavement with 900 psi flexural strength.

FIELD FORENSIC EVALUATION RECOMMENDATIONS

This desktop study revealed a need for a follow-up forensic evaluation of Sections 040214, 040215, and 040217 to gain a better understanding of performance features that are not captured at the surface. As an example, why is LTE so low for Section 040217 that had a lean concrete, stabilized base? Section 040262 should not be included in this evaluation as poor performance can mainly be attributed to the lack of dowel bars. This follow-up survey should include the following elements:

- Site visit and field condition survey to document the performance since 2016 and to verify the performance ranking of Sections 040214, 040215, and 040217. During this survey, collect features such as slab settlement (lane to shoulder) and document surface wear condition.

- Petrographic analysis of extracted cores to understand map cracking, confirm ASR was not a factor, and understand the properties of the hardened concrete. Also, for Section 040217, to assess whether the PCC and LCB layers are bonded or unbonded.
- Deflection testing (FWD) to compare 20+year structural stiffness, measure load-transfer across joints, and to detect the presence of under-slab voids. This effort should include coring at the joints to assess condition of the dowel bars and surrounding concrete.
- Cut beams from concrete slabs to assess the 20+ year flexural strength and compare to post construction data. This may help to understand strength development and to see if the 550-psi sections achieved similar ultimate strengths as the 900-psi section.