Technical Memorandum

To: Jeff Uhlmeyer

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cc: Mustafa Mohamedali

Date: July 9, 2019 (original), June 9, 2020 (revised)

Re. Forensic Desktop Study Report: Colorado SPS-2 Test Sections 0216, 0218, 0223 and 0224

The Long-Term Pavement Performance (LTPP) Specific Pavement Studies (SPS) project 080200[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations" to investigate the role of locked-in surface curvature, temperature and PCC pavement structure properties that potentially influence changes in IRI over the time of a single day, as shown with existing LTPP diurnal measurements on four JPCC test sections.

The purpose of this document is to review the history and examine performance over time for four Colorado SPS-2 test section (080216, 080218, 080223 and 080224). These four sections were selected for this analysis because 080223 has a trend of increasing IRI with increasing temperature, whereas the remaining three have a trend of decreasing IRI with increasing temperature. This memorandum has been reviewed by the Pooled Fund's Technical Advisory Committee (TAC), and revisions were made based on both written comments and feedback provided during the July 17, 2019 TAC Meeting.

BACKGROUND

Review of diurnal profile testing performed on June 23, 2013 at LTPP project 080200 showed that some sections had little change in IRI with temperature, other sites had a decrease in IRI with increase in temperature, and one section had an increase in IRI with increase in temperature. These changes along with key experimental factors are summarized in Table 1. As shown in this Table 1, the difference in IRI versus temperature trend appears to be related to the mix type—all of the sites with decreasing IRI with increasing temperature had the high strength mix, whereas the sites with the low strength mix had little change in IRI on increasing IRI with increasing temperature. The correlation would be perfect, except that site 080224 had no significant change in IRI with increasing temperature although it did have the high strength mix. No other experimental factor is correlated with the change in IRI versus temperature.

[1] First two digits in test section number represent the State Code [08 = Colorado]. 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., 16).

Table 1.IRI vs. temperature trends and experimental factors.

Section	Change in IRI with Increasing Temp			Concrete Mix	Slab Thick., in.	Slab Width, ft.	Base Type
	No Change	Increase	Decrease				
080213	X			550 PSI	8	14	DGAB
080214			X	900 PSI	8	12	DGAB
080216			X	900 PSI	11	14	DGAB
080218			X	900 PSI	8	12	LCB
080219	X			550 PSI	11	12	LCB
080220			X	900 PSI	11	14	LCB
080222			X	900 PSI	8	12	PATB
080223		X		550 PSI	11	12	PATB
080224	X			900 PSI	11	14	PATB

It is reasonable to expect the high-strength mix to have increased sensitivity to moisture-related curl as high strength mixes typically have significantly higher paste content. Depending on the local weather at the date of test, there could have been a relationship between slab moisture and temperature over the course of the day. Temperature related warp would be expected to be similar if the mixes have the same aggregate type, but different otherwise.

SITE DESCRIPTION

The SPS-2 project site is located on eastbound Interstate 76, starting at milepost 18.4, in Denver, Colorado. This is a two-lane rural interstate highway in the direction of travel. It is classified as being in a Dry, Freeze climate zone with an average annual precipitation of 14 inches and an annual average air freezing index of 544 32 Deg-F degree-days. The coordinates for the start of the project are 39.92894, -104.79875. Figure 1 shows the geographical location of the project within the State of Colorado, 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 2017, from the start of the section looking east.

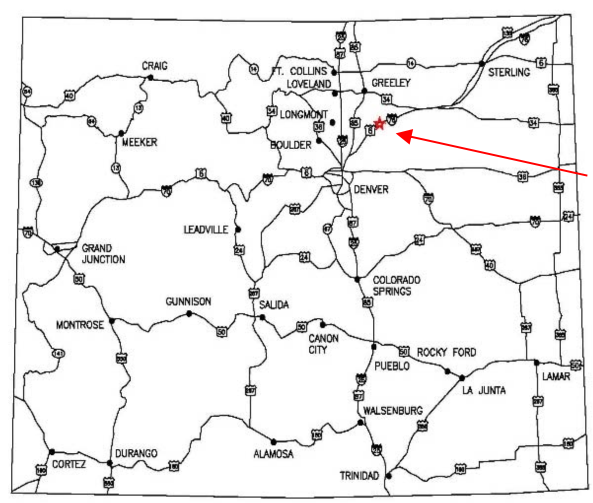


Figure 1. Geographical location of SPS-2 test sections within Colorado.

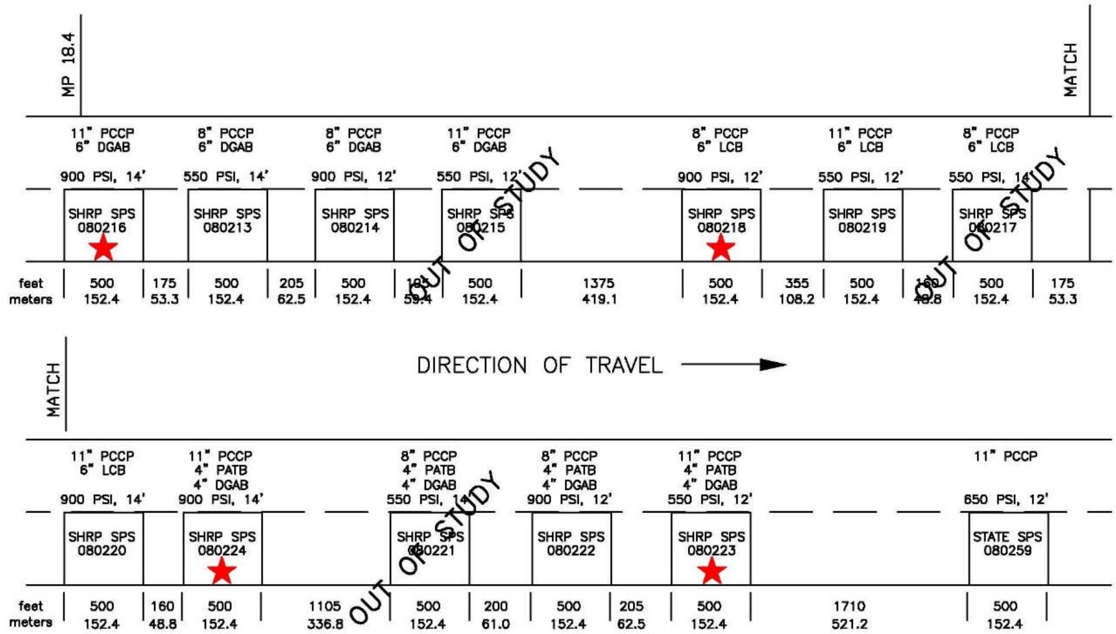


Figure 2. Location of selected test sections within the Colorado SPS-2 project.

Figure 3. Test section 080216.

Figure 4. Test section 080218.

Figure 5. Test section 080223.

Figure 6. Test section 080224.

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 Colorado 080200 project began in March 1993 and the concrete pavement sections were opened to traffic in November 1993. The original layer structure for each section selected for the forensic desktop study is detailed in Table 2. This corresponds to CONSRTUCTION_NO = 1 (CN1). Lane width and flexural strength were also experimental variables in the SPS-2 project and are detailed in Table 3. The core SPS-2 sections, included in this desk study (080216, 080218, 080223 and 080224) are doweled concrete pavement. The mix data for the two mixes employed at the Colorado SPS-2 test sections are shown in Table 4. Mix component weights are from the mix design submittal, whereas air content and Coefficient of Thermal expansion (CTE) are from drilled cores. The aggregate source is not specified; however, the reported specific gravity and gradation of both fine and coarse aggregate is the same for the two mixes, indicating that the same aggregate was used, and that the mixes do not differ in components, only in their proportions.

Table 2. Pavement structure from 1993 to date.

Section	Layer Number	Layer Type	Thickness (in.)	Material Code Description
080216	1	Subgrade (untreated)		Coarse-Grained Soils: Poorly Graded Sand with Silt
	2	Unbound (granular) base	5.9	Crushed Gravel
	3	Portland cement concrete layer	11.9	Portland Cement Concrete (JPCP)
080218	1	Subgrade (untreated)		Fine-Grained Soils: Clay
	2	Bound (treated) base	6.2	Lean Concrete
	3	Portland cement concrete layer	7.6	Portland Cement Concrete (JPCP)
080223	1	Subgrade (untreated)		Coarse-Grained Soils: Well-Graded Sand with Silt
	2	Unbound (granular) base	4.7	Crushed Gravel
	3	Bound (treated) base	4.2	Open Graded, Hot Laid, Central Plant Mix
	4	Portland cement concrete layer	11.7	Portland Cement Concrete (JPCP)
080224	1	Subgrade (untreated)		Coarse-Grained Soil: Clayey Sand
	2	Unbound (granular) base	3.1	Crushed Gravel
	3	Bound (treated) base	4.6	Open Graded, Hot Laid, Central Plant Mix
	4	Portland cement concrete layer	11.6	Portland Cement Concrete (JPCP)

Table 3. Additional experimental factors.

Section	Slab Width (ft)	Concrete Mixture 14-day Design Flexural Strength (psi)
080216	14	900
080218	12	900
080223	12	550
080224	14	900

Table 4. PCC Mix Design Properties

Component	Weight per Cubic Yard, lbs.	
	High Strength Mix	Low strength Mix
Cement – Southwestern Type I/II Low Alkali	749	399
Flyash – Type F	187 (20%[1])	133 (25%[1])
Water	257	236
Coarse Aggregate	1865	1720
Fine Aggregate	935	1430
Air Entraining Admixture[2]	0.9	0.5
Total Weight	3994	3919
Total Cementitious Content	936	532
w/cm	0.27	0.44
Air Content	6.5%	7.3%
CTE (in/in/°F) x 10^{-6}	4.81	4.66

Note 1: Percent by weight of total cementitious
Note 2: Reported as 0.1% by weight of cement

Section 080223 only has one CN, which corresponds to the date when the section was accepted into the LTPP experimental program. The 080216 construction events are as follows:

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

CN2 – Partial depth patching other than at joints, June 2005.

CN3 – Partial depth patching at joints and other than at joints, July 2010.

CN4 – Partial depth patching at joints and other than at joints, September 2016.

The 080218 construction events are as follows:

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

CN2 – Partial depth patching at joints, June 2005.

CN3 – Partial depth patching at joints, June 2008.

CN4 – Partial depth patching at joints, June 2014.

CN5 – Partial depth patching at joints, June 2015.

CN6 – Partial depth patching at joints, September 2016.

The 080224 construction events are as follows:

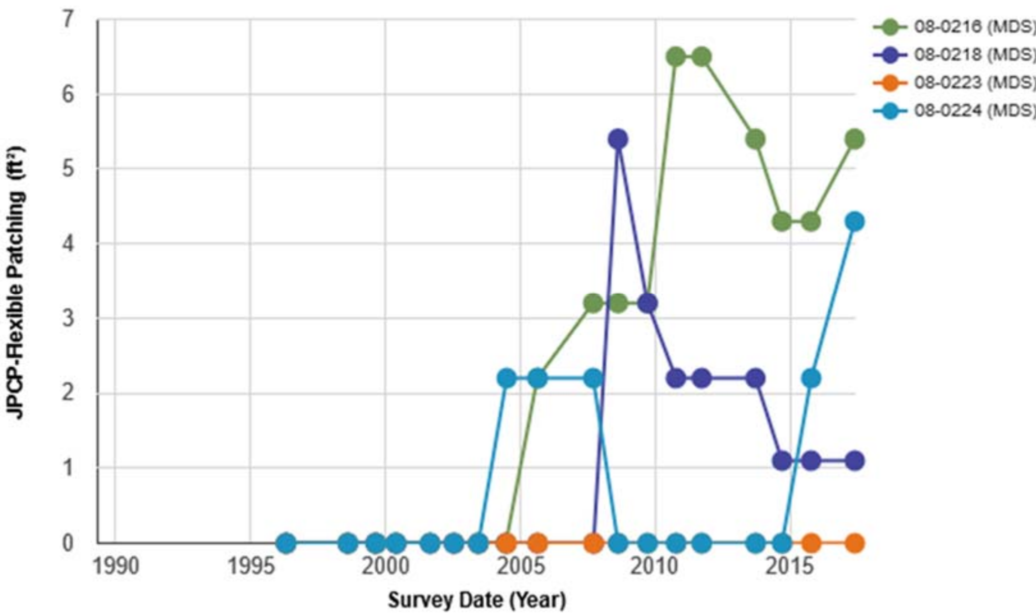
CN1 – Test section accepted into the LTPP program, January 1993.

CN2 – Partial depth patching at joints, July 2003.

CN3 – Partial depth patching at joints, July 2014

CN4 – Partial depth patching at joints, September 2016.

Figure 7 displays the recorded quantities of flexible patching for the four sections in this study. In Figure 7 section 080224 has one 2.2ft² flexible patch not displayed in September 2014. Section 080218 is the only section in this study that has received rigid patching. In June 2015 a 2.2ft² rigid patch at the centerline and transverse joint was added to section 080218.



Note: There was a patch in 2014 that was not recorded on the MDS. The patch has been added to the manual distress survey (MDS) and the LTPP pavement performance database (PPDB), but since graph is a reflection of what is currently in LTPP InfoPave, the patch is not displayed in the figure.

Figure 7. Time history of flexible patching.

Climate History

The time history for annual average precipitation since 1993 is shown in Figure 8. The annual average precipitation between 1993 and 2007 was 14 inches and in 2013 the year diurnal measurements were collected for the Colorado SPS-2 precipitation was 16 inches. In 2012 the year prior to the diurnal data collection, precipitation was at about 8 inches which was the year that had the least amount of precipitation between 1993 and 2017.

Figure 9 shows the time history of the annual freezing index over the history of this project. The freezing index is determined by an equation that computes the negative of the sum of all average daily air temperatures that are less than 32 degrees F within a year. The freezing index in Figure 9 can be an indicator of the severity of a winter season. From 1993 to 2007 the freezing index was between 370 and 760 and in 2003 during the diurnal collection there was a freezing index of 403.

Table 5 displays daily temperature and precipitation data for 080200 collected from the Virtual Weather station (VWS) the day before the diurnal profile data were collected and the day those data were collected. Maximum temperatures on June 22, 2013 were almost 10 degrees higher than the high temperatures on June 23, 2013 and had less precipitation.

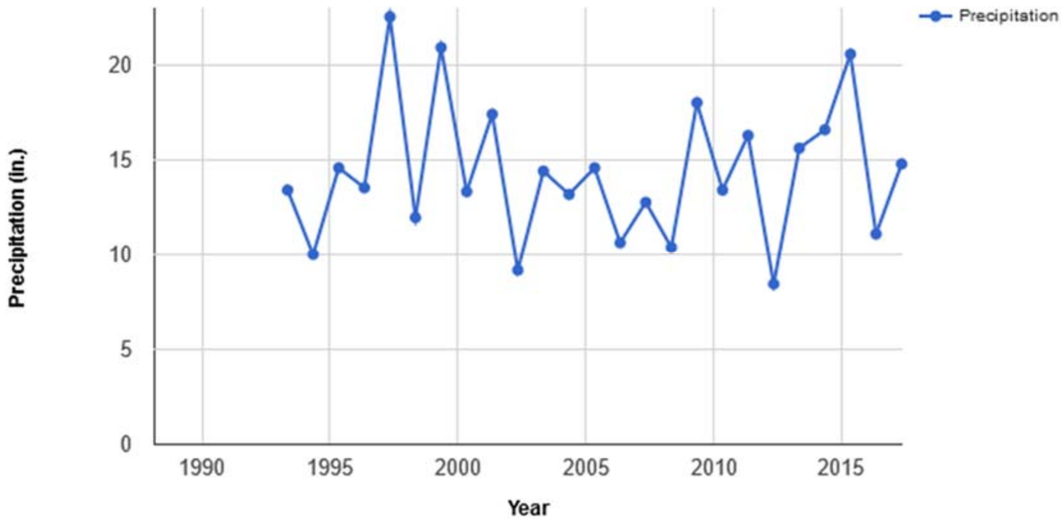


Figure 8. History of annual precipitation starting in 1993.

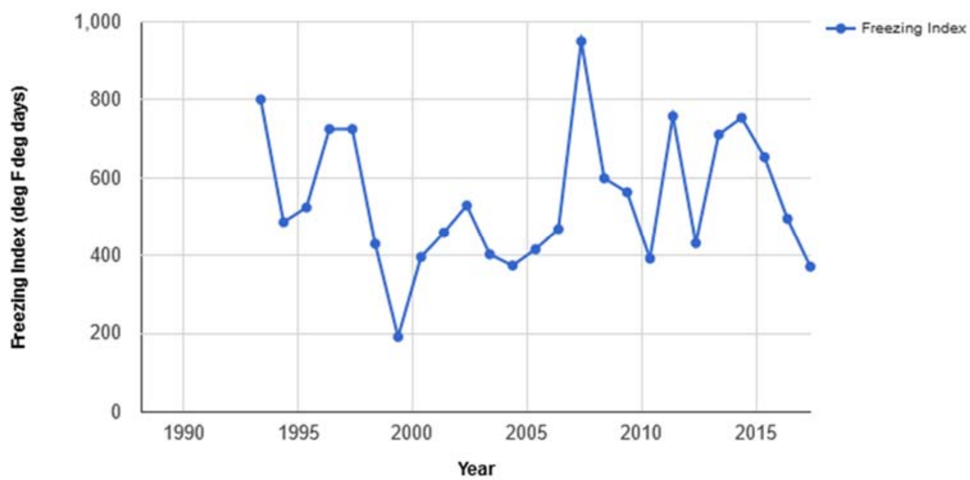


Figure 9. Time History of annual air temperature freezing index.

Table 5. Daily temperature and precipitation data from virtual weather station for 08000

VWS	Date	Mean Temp. (deg F)	Max. Temp. (deg F)	Min. Temp. (deg F)	Precipitation (in.)	Snowfall (in.)
080200	6/22/2013	74.8	95.7	54.1	0.004	0.0
	6/23/2013	69.1	86.2	52.0	0.024	0.0

Table 6 includes average temperature, precipitation and average solar data for 080200 in June 2013 from the MERRA data, which explains the difference in the mean temperature compared to the mean temperature from the VWS data in Table 4.

Table 6. 080200 MERRA data in June 2013

Temperature		Precipitation			Solar Data	
Temp. Avg. (deg F)	Mean Temp Avg. (deg F)	Evaporation (in.)	Precipitation (in.)	Days with Precip.	Cloud Coverage Avg. %	Shortwave Surface Avg.
71.6	71.4	2.15724	0.30075	30	0.25	775.491

Truck Volume History (Traffic)

Figure 10 shows annual truck volume data in the LTPP test lane by year. The triangles represent estimated data provided by Colorado State DOT. The blue diamonds are truck counts based on monitoring data, reported to or collected by the LTPP program. Estimated truck count data from 2002 to 2015 were provided by the agency; they were not collected directly from the Weigh-In-Motion (WIM) stations. For analysis purposes, it is recommended that truck count estimates from 2006 to 2015 be ignored due to data not coming from the WIM, but from agency estimated information. The average annual daily truck traffic on the Colorado SPS-2 has slowly, but steadily increased from approximately 750 to 1650from 1993 to 2017.

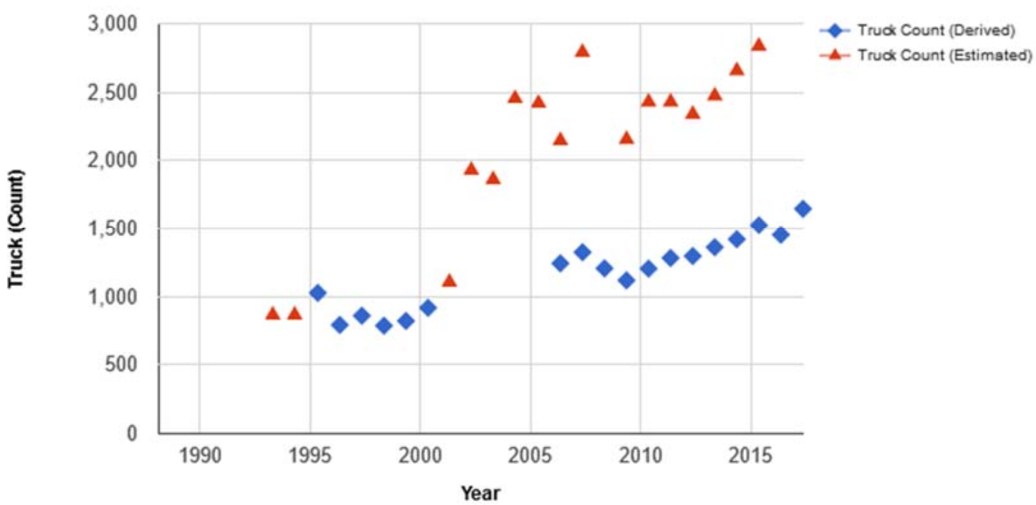


Figure 10. Average annual daily truck traffic on project 080200 based on state supplied estimates and monitoring measurements.

Table 7 presents the representative Average Annual Daily Truck Traffic (AADTT) and percentage per vehicle class for the Colorado SPS-2 (project (080200) in 2018.

Table 7. Representative traffic data: average annual daily truck traffic (AADTT) and percentage per vehicle class

Project	Representative AADTT	Representative Percentage Per FHWA Vehicle Class									
		4	5	6	7	8	9	10	11	12	13
080200	1,311	1.07	22.9	3.95	0.22	3.75	62.33	0.8	3.34	1.48	0.16

IRI Roughness Time Histories

The time history of roughness measurements for the four test sections in the study are shown in Figure 11. 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 shown in Figure 11, as of 2017 all four sections had an IRI value in the “Fair” category with IRI values ranging between 101 and 130. The average IRI values in 2013, for regular data collection and diurnal data collection, put all four sections in the “Fair” category with IRI values ranging between 99 inches/mile and 130 inches/mile.

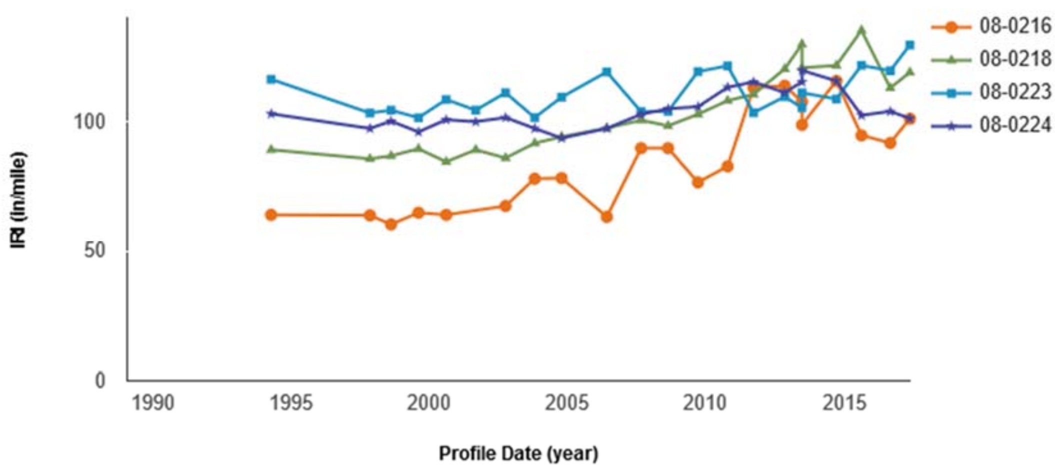


Figure 11. Time history plot of pavement roughness for 080216, 080218, 080223 and 080224.

In Figures 12 through 15 the diurnal longitudinal profile measurements, collected on June 23, 2013, display the changes in IRI over the time of a single day for each section. The diurnal data were collected at three separate times of the day; early morning, mid-morning and the afternoon. Figures 12 through 15 shows data for three runs during each time of the day for the left wheel path, right wheel path and the center lane. Section 080216, shown in Figure 12, had a decrease in IRI as average surface temperatures increased throughout the day. IRI values during the early morning and mid-morning fluctuated within the “Fair” category ranging between 99 inches/mile and 122 inches/mile, but as the temperatures increased the IRI values in the afternoon were at their lowest, and fluctuated within the “Good” category with IRI values ranging between 82 and 91 inches/mile.

This behavior of decreasing IRI versus increasing temperature could be due to built-in upward curling of the concrete slabs at the time of paving. This can occur when the temperature of top of the PCC is lower than the temperature at the bottom of the slab due to high base layer temperature. During the use-phase of these slabs, when the temperature increases, the slabs will begin to flatten out, resulting in lower IRI values. This phenomenon was first discovered in a 1998 report[1].

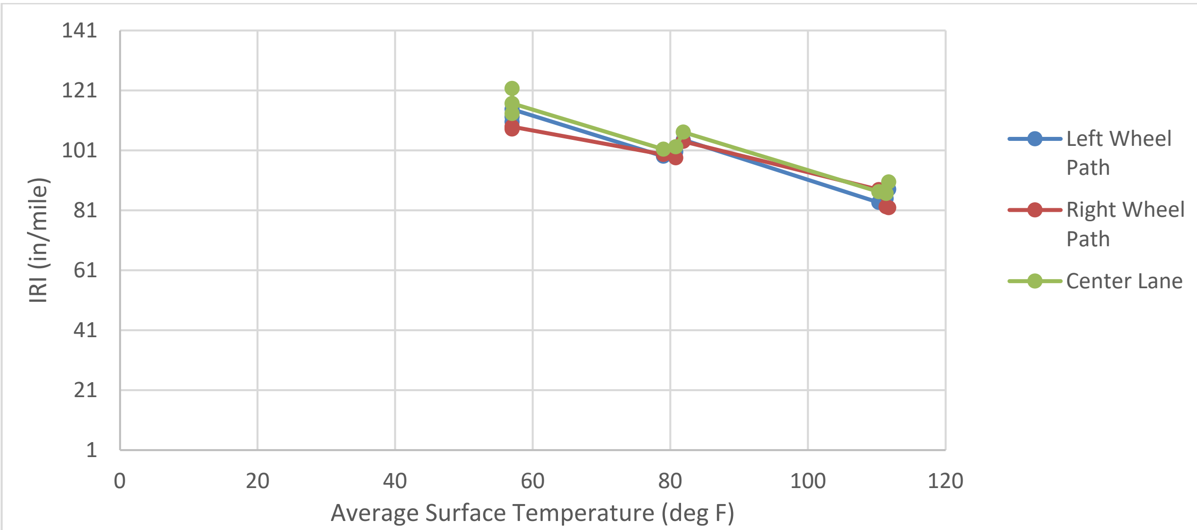


Figure 12. Section 080216 IRI versus temperature, June 23, 2013.

For section 080218, shown in Figure 13, IRI values slightly decreased as average surface temperatures increased, but unlike 080216 it never moved into the “Good” category and fluctuated within the “Fair” category, with IRI values ranging from 100 inches/mile to 151 inches/mile. During the afternoon, the lowest IRI value for 080218 was 100 inches/mile in the left wheel path, which is close to the “Good” category, but still puts it in the “Fair” category.

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[1] Yu, Thomas H.; Khazanovich, Lev; Darter, Michael I., Ardani, Ahmad. *Analysis of Concrete Pavement Responses to Temperature and Wheel Loads Measured from Intrumented Slabs.* Transportation Research Board, Washington D.C., 1998.

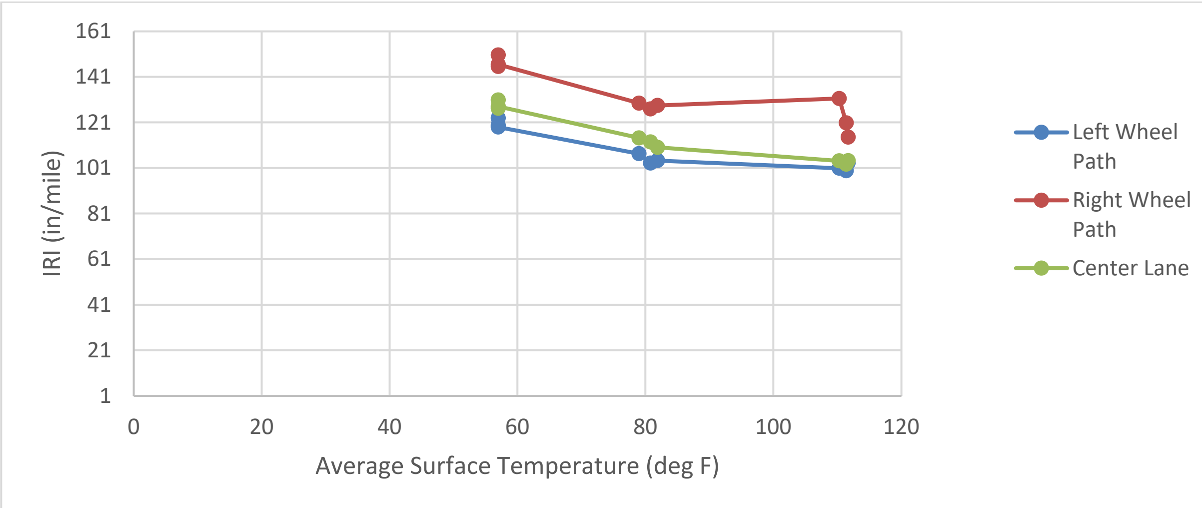

Figure 13. Section 080218 IRI versus temperature, June 23, 2013.

Unlike 080218, test section 080223, shown in Figure 14, experienced the opposite IRI value change, where the IRI values increased as temperatures increased throughout the day. The IRI values fluctuated within the "Fair" category, ranging between 98 inches/mile and 124 inches/mile. The lowest IRI values that were closest to the "Good" category were during the morning run when average surface temperatures were at their lowest.

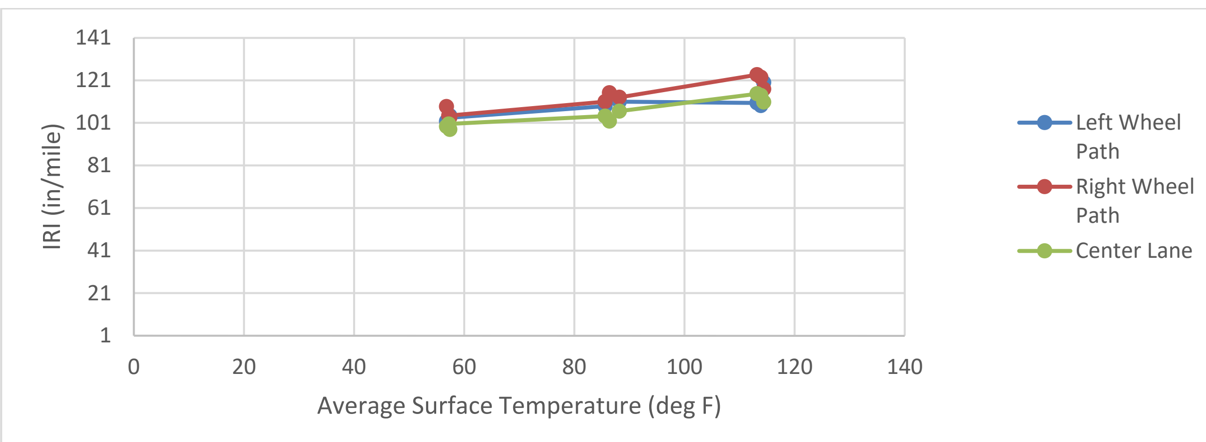

Figure 14. Section 080223 IRI versus temperature, June 23, 2013.

Section 080224, shown in Figure 15, had higher IRI values in the beginning of the day and as temperatures began to increase the IRI values began to slightly decrease. The lowest IRI values for 080224 were in the afternoon. The IRI values fell within the "Fair" category throughout the entire day except in the left wheel path where one of the runs produced an IRI value at 94 inches/mile, which is considered "Good". However, the other two runs had IRI values that would put it on the low end of the "Fair" category.

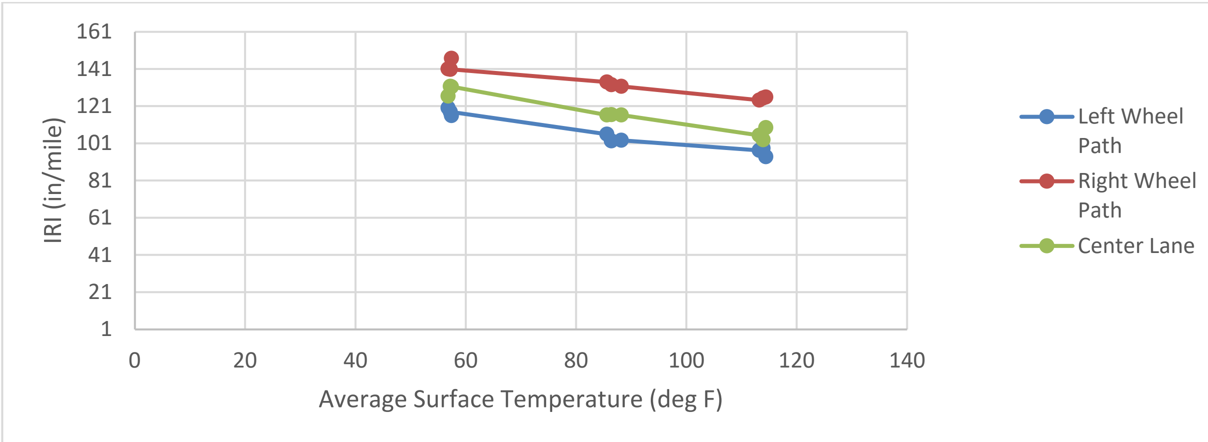

Figure 15. Section 080224 IRI versus temperature, June 23, 2013.

Diurnal Profile Changes

Raw elevation and power spectral density (PSD) are plotted for each section in Attachments A through D for three of the profile runs performed on June 23, 2013. Both the raw elevation plots and the PSD plots show a trend of flattening of the slabs with increased temperature for sections 080216, 080218 and 080224. In the raw profile plots, this can be seen directly, whereas for the PSD plots this shows up as a decrease in the spike at a wavelength of 15 feet, corresponding to the slab length. Again, 080223 has different behavior, with an apparent increase in concave-down curvature, and an increase in roughness with a wavelength of 15 feet.

Deflection Under Load Plate Time Histories and Load Transfer Efficiency

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 16. 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 080223 and 080224 were very low (less than 4 mils) and remained constant throughout the 20+ year monitoring period to date. This was likely due to the presence of a thicker pavement slab (almost 12-inches) and the presence of a treated base (with an almost 5-inch thickness) and a granular base (3 inches for 080223 and almost 5-inches for 080224). Section 080216 had low deflections, fluctuating between 3 and 5 mils, and at the end of the monitoring period in 2015, deflection values were 2 mils higher than 080223 and 080224. Section 080218 had the highest deflections out of the four sections in this study. Deflection values for 080218 started off with steadily increasing deflection values, until 1999 and 2001 where it dropped to 6 mils. From 2002 to 2015 deflection values fluctuated between 4 and 8 mils, except in 2013 when there was a significant increase to 11 mils, which then dropped to 6 mils during the 2015 deflection testing. An explanation for the high deflections for 080218, is that the PCC layer thickness is 4-inches thinner than the PCC layer in the other three sections, leaving it with a PCC layer pavement structure that has less overall stiffness compared to 080216, 080223 and 080224. However, 0218 is also the only section that has a lean concrete base (LCB). Further investigation, including whether the deflection data is consistent with a bond between the slab and LCB, would require more sophisticated analysis on the deflection data, such as backcalculation. Section 080216 had the second highest deflections which is not surprising as, opposed to 0223 and 0224, 0216 only has a DGAB below the PCC surface.

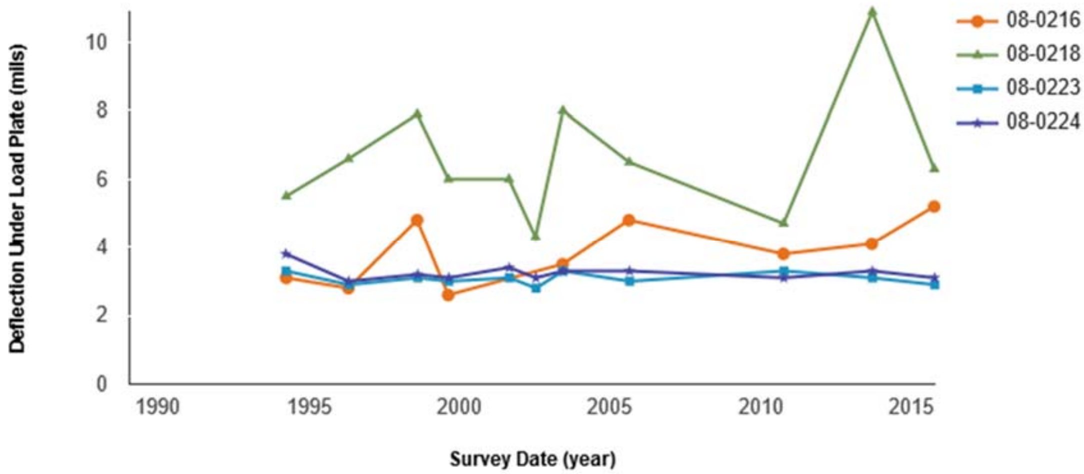

Figure 16. Time history plot of average deflection for the sensor located in the load plate normalized to 9,000 lb. drop load for 080216, 080218, 080223 and 080224.

Figure 17 provides a comparison of the load transfer efficiency (LTE) for the four sections in this study. Sections 080216 and 080224 maintained a good LTE throughout the monitoring period, staying between 89% and 98%. Section 080218 maintained a good LTE starting at 85% in 1994 and increasing to 93% in 2002. In 2003 the LTE for 080218 dropped to 80%, increased slightly in 2005 and gradually decreased until the end of the monitoring period. The LTE for 080223 started at 75% in 1994, dropped to 48% in 1996 and fluctuated between 50% and 89%, until 2005 when it dropped down to 36%. From 2005 on the LTE decreased gradually until the last deflection survey was conducted in 2015 where the LTE dropped to 19%. Values of LTE this low indicate that the dowels are not providing load transfer. The reason cannot be inferred from the data examined in this study. Section 0223 and 0216 both have the same nominal slab thickness and granular base. The differences are that 0223 had a narrower slab and the low strength mix. The narrower slab has two fewer dowels, and the lower strength mix possibly is less capable of resisting loads at the interface with the dowel, leading to local crushing of the concrete and "play" or "socketing" in the dowels. 0223 is the only low-strength mix section in this study, further investigation would require review of the data from the other low-strength sections. Interestingly, 0218, the other 12 foot wide slab in this study also shows lower LTE than the two 14 foot wide slabs, indicating that a combination of effects may be operative.

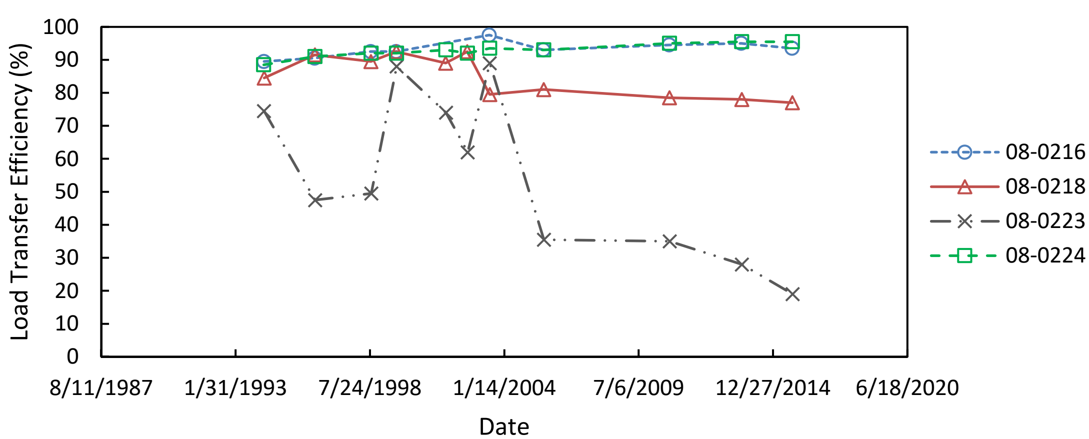

Figure 17. Load transfer efficiency summary.

Longitudinal and Transverse Cracking Time Histories

Figure 18 shows the time history for longitudinal cracking lengths for the four Colorado SPS-2 sections in the study. Section 080216 has the largest amount of longitudinal cracking at 659ft, 080218 has 177ft of cracking, 080224 has about 1ft of longitudinal cracking and 080223 has no longitudinal cracking present.

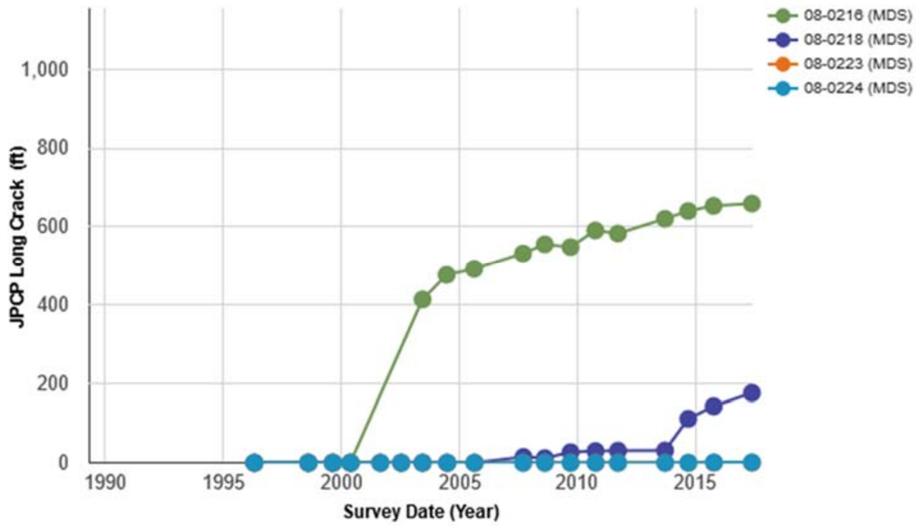

Figure 18. Time history of longitudinal cracking length.

In Figure 19, transverse cracking lengths are displayed for the four sections. For section 080216, not only does it have the most longitudinal cracking, but it also has the most transverse cracking at 65ft. Section 080216 has 6 cracked slabs (slab with a transverse crack greater than or equal to 1.9ft) out of 33, which converts to 18% of slabs cracked. This section also exhibits map cracking primarily in the outer wheel path. Section 080218 also has 6 cracked slabs out of 33, which is 18% of slabs cracked. 080218 was the only section that exhibited corner breaks, durability cracking and it had the most map cracking out of the four test sections (5,757ft^2). Section 080223 has 5 cracked slabs out of 33 which is 15% of slabs cracked and although this section had no longitudinal cracking the section exhibits 3,971ft^2 of map cracking throughout the section, the most polished aggregate (6,070ft^2) out of the four test sections in this study and the section with the most scaling present (651ft^2). Section 080224 has no transverse cracking, a small amount of longitudinal cracking and had the least amount of surface deformations; only a limited scaling (13ft^2), a small amount of map cracking (18ft^2) and some polished aggregate (349ft^2). According to the FHWA performance standard, 080224 is considered “Good” with less than 5% cracked slabs, 080223 is considered “Fair” with cracked slabs between 5%-15% and 080216 and 080218 have greater than 15% cracking putting them in the “Poor” category.

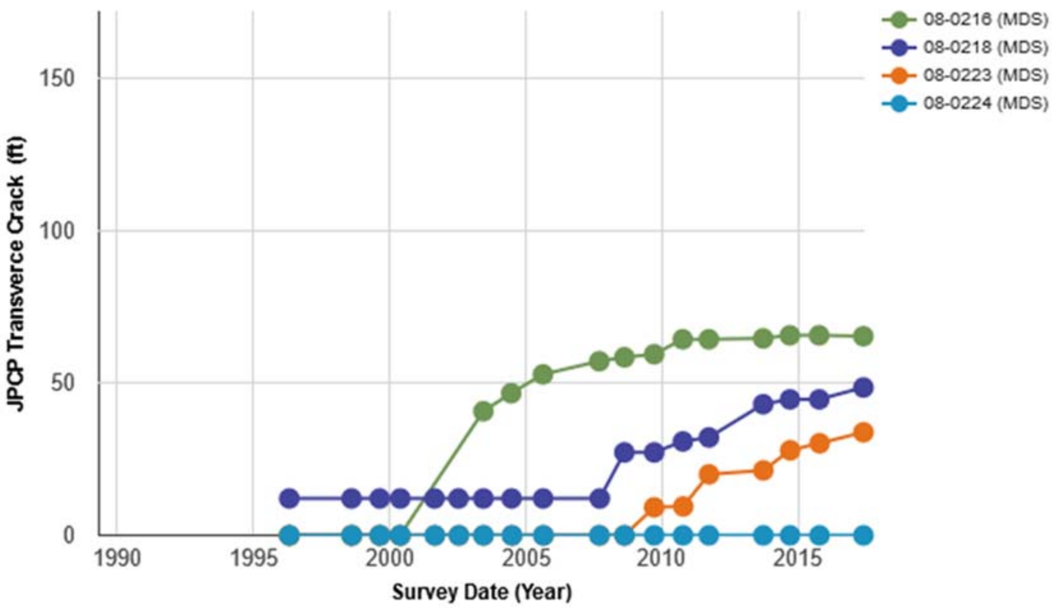

Figure 19. Time history of transverse cracking length.

Figure 20 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. Section 080224 displayed the least amount of spalling with three affected transverse joints. For sections 080218 and 080223, affected joints began to gradually increase after 2005. At the end of the monitoring period 080218 had 17 affected joints and 080223 had 14 affected joints. Of all four sections 080216 had the most dramatic increase from 2001 to 2003, jumping from two affected joints to 18 affected joints. The data point for section 080216 in July 2003 is from an Automated Distress Survey, which is analyzed from photos. For the purpose of this study, it is recommended that the July 2003 data point be ignored due to variability from data not being analyzed in the field. Beginning in 2004, the number of affected joints for 080216 steadily increased. At the end of the monitoring period, 080216 had twenty-four affected joints, the most affected joints of the four test sections in this study.

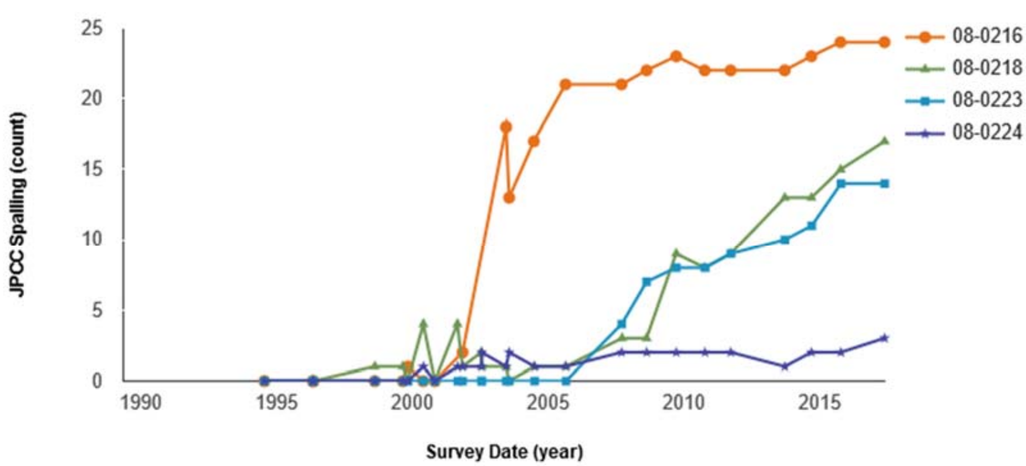

Figure 20. Time history of total number of JPCC spalling at transverse joints.

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 21 shows the rutting time history from Dipstick® data. Rut depths measured in March 1996, three years after construction began, ranged from 0.16 to 0.35 inches. Some questions have been raised regarding whether these measurements are a true indication of rutting.

According to the rutting FHWA performance standard on asphalt pavement, less than 0.2 inches is considered "Good", rutting between 0.2 inches and 0.4 inches is considered "Fair" and rutting greater than 0.4 inches is considered "Poor". Section 080224 is the only section out of the four sections that was considered "Good" when rut depths were first measured in 1996. 080224 stayed in the "Good" category up until 2017 when it moved to "Fair" at 0.2 inches. As seen in Figure 21, section 080216 started in the "Fair" category in 1996, but then it moved to the "Good" category until the last monitoring year in 2017 when it moved back to the "Fair" category. Sections 080218 and 080223 had the highest rutting values throughout the years and stayed within the "Fair" category over the twenty-year monitoring period. In 2017 all four sections are considered to be "Fair" according the FHWA performance standard for AC surfaces with rutting between 0.2 inches and 0.35 inches. Variability in apparent rutting with time at this site seems more likely to be due to measurement error than wear, or other deterioration effect.

Faulting Time Histories

Figure 22 shows the faulting time histories on the four Colorado SPS-2 sections. Faulting values at the end of the monitoring period for 080218 and 080223 are positve fault heights, which occur when the leave side of the joint has a higher elevation than the approach. Sections 080216 and 080224 ended with no faulting, but throughout it's monitoring life they fluctuated from positive and negative faulting values. As seen in Figure 22, faulting values for all four sections have fluctuated throughout the twenty plus years they have been monitored. Section 080223 is the only section that did not have negative faulting values and steadily increased after 2013. 080223 ended with the highest faulting values at .09 inches in 2017. Section 080223 is the only section with a design flexural strength of 550 psi unlike the three other sections that have a design flexural strength of 900 psi. Although the faulting values have fluctuated throughout the years and section 080223 ended with the hightest faulting value of 0.09 inches, all sections have stayed in the "Good" category accoridng to the FHWA performance standard. The FHWA performance standard for faulting is as follows; faulting less than 0.1 inches is "Good", faulting between 0.1 inches and 0.15 inches is "Fair" and faulting greater than 0.15 inches is considered "Poor".

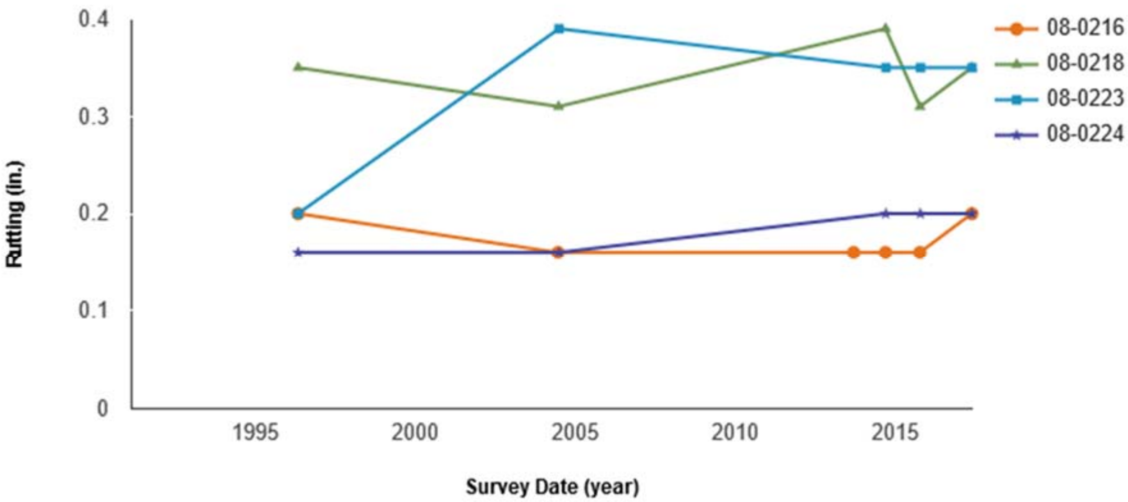

Figure 21. Time history plot of average rut depth computations.

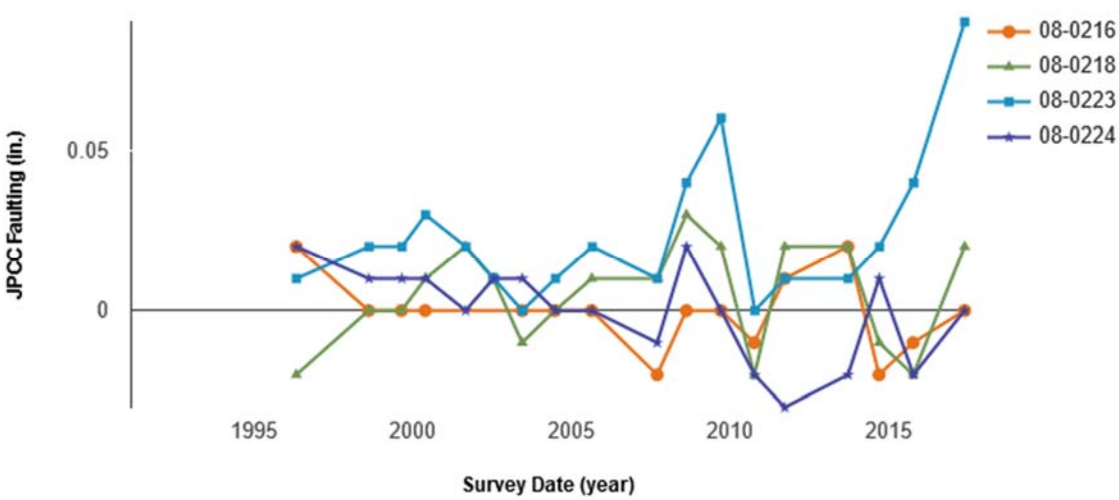

Figure 22. Time history plot of faulting.

SUMMARY OF FINDINGS

Table 8 provides a summary of the experimental factors for each section, along with the FHWA performance categories and other performance indicators for the 2017 distress levels.

Table 8. Summary of experimental factors and performance as of 2017.

Experimental Features					FHWA Performance				Other Performance				
Section	PCC (in.)	Base	Slab Width (ft)	Design Flexural Strength (psi)	IRI	Fault	Trans. Cracks	Rut	Diurnal IRI	Long. Cracks	Significant Map Cracks	Significant Polished Aggregate	LTE
080216	11.9	Crushed gravel (6-in.)	14	900	Fair	Good	Poor	Fair	Fair and Good	Many	No	No	Good
080218	7.6	Lean concrete (6 in.)	12	900	Fair	Good	Poor	Fair	Fair	Some	Yes	No	Good
080223	11.7	Crushed gravel (4.7-in.) Open Graded, Hot Laid, Central Plant Mix (4.2-in.)	12	550	Fair	Good	Fair	Fair	Fair	None	Yes	Yes	Poor
080224	11.6	Crushed gravel (3.1-in.) Open Graded, Hot Laid, Central Plant Mix (4.6-in.)	14	900	Fair	Good	Good	Fair	Fair	Minimal	No	No	Good

CONCLUSIONS

The observed difference amongst the sections in IRI versus temperature response on June 23, 2013 appears to be related to the differences in mixture properties, with the mixes with high cement contents having decreasing IRI with increasing temperature, whereas the mixes with low cement content having no or increasing IRI with increasing temperature. Observed differences in CTE should not be considered to be significant in light of CTE test procedure without further investigation. However, the difference in cement content between the mixes is significant, and will certainly result in the high strength mix having a much higher dimensional change due to changes in moisture. Tantalizingly, the available weather data shows a combination of high daily temperature variation along with a minor precipitation event on the day of testing, which should cause variations in both curl and warp during the day. The temperature-related curl would be expected to be similar for all the sections, whereas the moisture-related changes in warp would be expected to be more significant for the high-strength sections. Further investigation would require at least hour-by-hour weather data for the project location – this project did have an automated weather station, but it stopped collecting data in 2008. Perhaps further study would uncover a sufficiently adjacent weather station with hourly data availability. MERRA data shows a very minor precipitation event beginning about 1.5 hours after testing, which confusingly is accompanied by a drop in relative humidity. With the MERRA grid-scale (roughly 30 miles by 30 miles) and the prevalence of isolated summer thunderstorms in the area, the MERRA data is not fine-grained enough to determine if and exactly when a rain event would have happened at this site.

Section 080223 was the only site that showed a clear increase in IRI with increasing temperature, whereas the other low-strength sections showed no clear trend. Amongst the sections investigated in this study, 080223 is also the only one that shows clear failure of the dowel bars, with a sharp drop in LTE in 2004-2005 timeframe and an increase in faulting in about 2014. It is possible that loss of restraint from the dowel bars has allowed the joints to rotate more due to warp and curl, and thus show increased sensitivity of roughness.

FURTHER EVALUATION RECOMMENDATIONS

It is recommended that the desktop study be extended to further investigate the following:

- Potential sources of nearby hourly (or better) temperature, precipitation and ideally solar flux and humidity data that may further explain warping and curling behavior on June 23, 2013.
- Evaluate the statistical significance, or lack thereof, of the CTE differences between the mixes.
- Compare LTE and faulting behavior of 080223 to the other low-strength mixes (080213, 080219, 080224).
- Include earlier investigations into diurnal testing on SPS-2 projects (two papers were written in 2015 by L. Scofield, with one having J. Springer as co-author).
- Determine whether batch tickets from the original construction are available.

Further recommendations for field testing:

- Repeat diurnal profile measurements.
- Obtain additional cores for CTE testing and aggregate type identification.
- Core through dowels on 080223 and another section with apparent good LTE and faulting performance. Identify whether socketing is cause of poor load transfer.
- Core 080218 to determine whether slab and LCB are bonded, and, if practical, also core the other test sections constructed with LCB.
- Core at least three test sections each for the two different concrete mixes to support an analysis of strength gains over time, and also a petrographic assessment of hydration of the cement and w/c ratio for each mix.
- Take straight edge measurements to assess rutting.

Additional topics that could be considered as part of a broader investigation include:

- Compare raw profile measurements to faultmeter measurements and examine whether the faulting is the same.
- Attempt to determine the minimum perceptible fault level, and assess any correlation with IRI.

ADDENDUM TO MEMORANDUM: FOLLOW-UP INVESTIGATIONS

The desktop study analyzed available field data to explain the performance of multiple test sections for Colorado's SPS-2 experiment. 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 investigations detailed below were performed:

- The original LTPP construction report for the Colorado SPS-2 was reviewed to determine the weather conditions during the construction of the sections, which could influence the shape of the slabs as they hardened.
- FWD testing was performed in the early morning, late morning, and early afternoon of October 26, 2019 following standard LTPP FWD protocols; however, due to time and weather constraints, FWD drops were collected on two sections instead of all four. This data collection was originally scheduled for early September 2019 but had to be moved to October 2019 for traffic control reasons.
- Faulting was also performed in the early morning, late morning, and early afternoon of October 26, 2019 following standard LTPP protocols; however, due to time and weather constraints faulting measurements were only collected on a limited number of joints.
- Longitudinal profile measurements were performed in the early morning, late morning, and early afternoon of July 2019 using the LTPP High Speed Survey vehicle and following standard LTPP profiling protocols. A second set of measurements were collected on January 10, 2020.

080200 LTPP CONSTRUCTION REPORT

During an extensive review of documents available on InfoPave™, the original construction report for the Colorado SPS-2 was reviewed. The report discussed the construction practices and weather conditions during the PCC paving. A slip-form paver was used to place the concrete at the design thickness and was followed by a wet burlap drag. Dowel bars were placed at mid-depth and were held in place using baskets. Dowels measured 18 inches in length and were spaced at 12 inches on center; dowel diameter was 1 ¼ inch for the 8-inch thick pavement and 1 ½ inch diameter for the 11-inch thick pavement. A white wax-based curing compound was applied to the PCC pavement within 45 minutes of placing concrete.

Table 9 summarized the weather during PCC paving from the Colorado SPS-2 construction report, which included temperature measurements but not humidity measurements. The different test sections were all constructed on different days between September 7 and October 21, 1993. All four sections were paved during daytime hours with starting times no earlier than 07:30 and no later than 15:50. The difference between the high and low air temperatures during paving were from 7 to 15°F for sections 080218, 080223, and 080224. For section 080216, this difference in air temperatures during paving was 25°F. The weather during paving of Section 080224 became stormy in the afternoon with heavy rainfall occurring at 15:10. The paving crew immediately began to cover the fresh concrete, but due to high winds did not completely cover the section until 15:45.

Weather during construction can have a large influence on the initial roughness of new concrete pavements. These slabs will hold their shape as they begin to harden and cure, and it is possible for slabs that are cast in high or low temperatures to have built-in curling. The typical dynamic of slab curling is that as the surface temperature of the slabs increases, the slab edges will lower and the slab takes on a concave-down curvature. As the surface temperature of the slabs decreases and is cooler on top than on the bottom, the slab edges will raise up and take on a concave-up curvature.

The behavior of Sections 080216, 080218, and 080224 show the IRI values decrease as the temperature increases, possibly because the concrete slabs are flattening out and forming a smoother pavement throughout the sections. Section 080223 shows the IRI values increase as the temperature increases, which runs counter to typical PCC roughness behavior. This possibly could have been due to the initial shape of these slabs as a result of construction weather, but the air temperature ranges given in the construction report do not present any clear trends. What is interesting is the roughness behavior of Section 080224 do not appear to be influenced by the severe rainstorm that occurred during construction of this section. The profile data plots in Attachment A do not show a significant change in the slab shapes for Section 080216, 080218, and 080224 while slabs in Section 080223 appear to experience a small change in edge curling.

Table 9. Summary of Weather during Construction of LTPP Sections.

Section	Construction Date	Paving Start Time	Paving Ending Time	Air Temperature Range During Paving	General Weather Comments
080216	Oct. 11, 1993	09:30	14:10	45 to 70°F	None in Report
080218	Oct. 21, 1993	10:00	15:50	40 to 55°F	None in Report
080223	Sept. 3, 1993	07:50	10:35	55 to 68°F	None in Report
080224	Sept. 7, 1993 & Sept. 8, 1993	08:00 07:30	15:05 11:05	--- 45 to 60°F	Severe storm caused paving to stop mid-section

PREVIOUS RESEARCH FOR DIURNAL PROFILE TESTING ON SPS-2 TEST SECTIONS

A study[2] conducted in 2015 investigated daily changes in roughness of LTPP SPS-2 test sections. Between the summer of 2013 and the spring of 2014, profile measurements were collected at three different times during the day at each of the in-service SPS-2 test sections. The daily changes in mean roughness index (MRI) are shown in Figure 22, where a negative value indicates the pavement is rougher in the afternoon.

The results aligned with previous research that shows ride quality on concrete pavements can change throughout the day. Individual sections exhibited a wide range of daily MRI variations, from an increase of 25 in/mile to a decrease of 30 in/mile. Overall, the average percent change in MRI per state is between 7 to 17 in/mile. The average daily MRI variation amongst all test sections was 12 in/mile.

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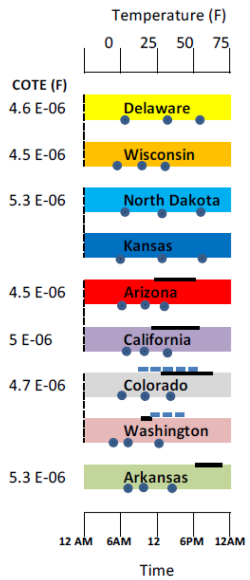
[2] Scofield, Larry; and Springer, Jack. *Daily Changes in Roughness on LTPP SPS-2 Test Sections*. Transportation Research Board, Washington D.C., 2015.



Pavement Structure					Climatic Zones, Subgrade															
Drainage	Base Type	PCCP		Lane Width (ft)	WET								DRY							
		Thickness (inches)	Flexural Strength 14-D (psi)		FREEZE				NO FREEZE				FREEZE				NO FREEZE			
					Fine		Coarse		Fine		Coarse		Fine		Coarse		Fine		Coarse	
					KS	ND	DE	WI	NC	O	P	AR	WA	CO	T	U	V	W	CA	AZ
No	DGAB	8	550	12	-22.0		30.0						4.5						3.2	
				14		-4.2		-13.6						11.9						18.0
			900	12		30		12.2				15.2		37.9						-12.1
				14	2.8		29.7						6						15.4	
		11	550	12		-26		-7.5				1.2		13.5						20.6
				14	-10.9		-9.9						7.3						11.2	
			900	12	13.6		-9.5						13.4						23.5	
				14		5.4		8.4				45.5		24.0						0.5
No	LCB	8	550	12	-24.1		8.6						2.8						6.4	
				14		-24		3.3				-93.3		-17.2						13.1
			900	12		26		4.3				6.3		19.1						9.2
				14	-25.2		15.7						-19.5						7.0	
		11	550	12		-20		-10.9				-12.9		5.2						22.8
				14	-18.4		-12.0						-3.7						14.4	
			900	12	-7.2		-4.4						-14.3						15.1	
				14		12		3.4				22.8		16.2						10.2
Yes	PATB	8	550	12	-14.1		7.0						2.7						9.2	
				14		-9.3		-11.0				-8.0		-1.4						16.1
			900	12		-8.5		-5.3				9.2		23.7						23.0
				14	14.3		3.3						1.6						25.8	
		11	550	12		-13		-6.8				-5.5		-12.1						24.4
				14	-16.9		4.2						2.2						5.6	
			900	12	4.1		0.5						-1.4						18.0	
				14		1.1		-1.4				6.6		18.9						25.9

Figure 22. Summary of Average Daily Roughness Change (MRI-in/mile)[1]

This paper also presented several conclusions, such as how the sections with 550 psi flexural strength mixes exhibited 30% less daily MRI change than the sections with 950 psi flexural strength mixes. Additionally, the sections with permeable asphalt treated base (PATB) generally had the lowest daily roughness changes when compared to sections with either lean concrete base (LCB) or dense graded aggregate base (DGAB).

Focusing on the Colorado SPS-2 test sections, the results presented in Figure 22 show that nearly all sections had a reduction in roughness as temperature increased. The two sections presenting an increase in roughness from morning to afternoon testing were Sections 080217 (17 in/mile increase) and 080223 (12 in/mile increase). Figures 12 through 15 illustrate how sections 080216, 080218, and 080224 all had decreasing IRI values with increases in temperature, whereas Section 080223 experienced an increase in IRI with increases in temperature.

2013 DIURNAL PROFILE COLLECTION WEATHER

The weather during the 2013 profile data collection was discussed in Tables 5 and 6 of this desktop study. However, historical hourly weather data from the Denver International Airport is available and is presented in Table 10 and illustrated in Figure 23. The results show the temperature ranged from 52 to 85°F during the day. Profile measurements were collected at 05:29 (orange highlight), 09:16 (green highlight), and 13:18 (purple highlight). The 2013 diurnal profile measurements can be viewed in Attachments A thru D from the desktop study. This weather data would indicate there may be a larger change in IRI between the mid-morning and early afternoon testing than between the early morning and mid-morning IRI values. Figures 12 through 15 show this is the case for Sections 080216 and 080223. Section 080218 has a large drop in IRI values between the early morning and mid-morning testing, but then experiences a small change between the mid-morning and afternoon testing. On section 080224, the IRI values appear to decrease at a constant rate between the various testing times, indicating the relative difference in test temperature does not result in corresponding relative differences in IRI.

Table 10. Average Air Temperature during 2013 Diurnal Profile Data Collection.

Time	Temperature	Time	Temperature
11:53 PM	63 F	3:53 PM	84 F
12:53 AM	59 F	4:16 PM	81 F
1:53 AM	59 F	4:24 PM	72 F
2:53 AM	52 F	4:28 PM	73 F
3:53 AM	54 F	4:53 PM	68 F
4:53 AM	55 F	5:53 PM	65 F
5:53 AM	53 F	5:58 PM	66 F
6:53 AM	55 F	6:10 PM	70 F
7:53 AM	58 F	6:53 PM	70 F
8:53 AM	62 F	7:53 PM	66 F
9:53 AM	66 F	8:46 PM	61 F
10:53 AM	71 F	8:53 PM	58 F
11:53 AM	76 F	9:53 PM	63 F
12:53 PM	81 F	10:07 PM	59 F
1:53 PM	85 F	10:53 PM	59 F
2:53 PM	83 F	11:35 PM	61 F

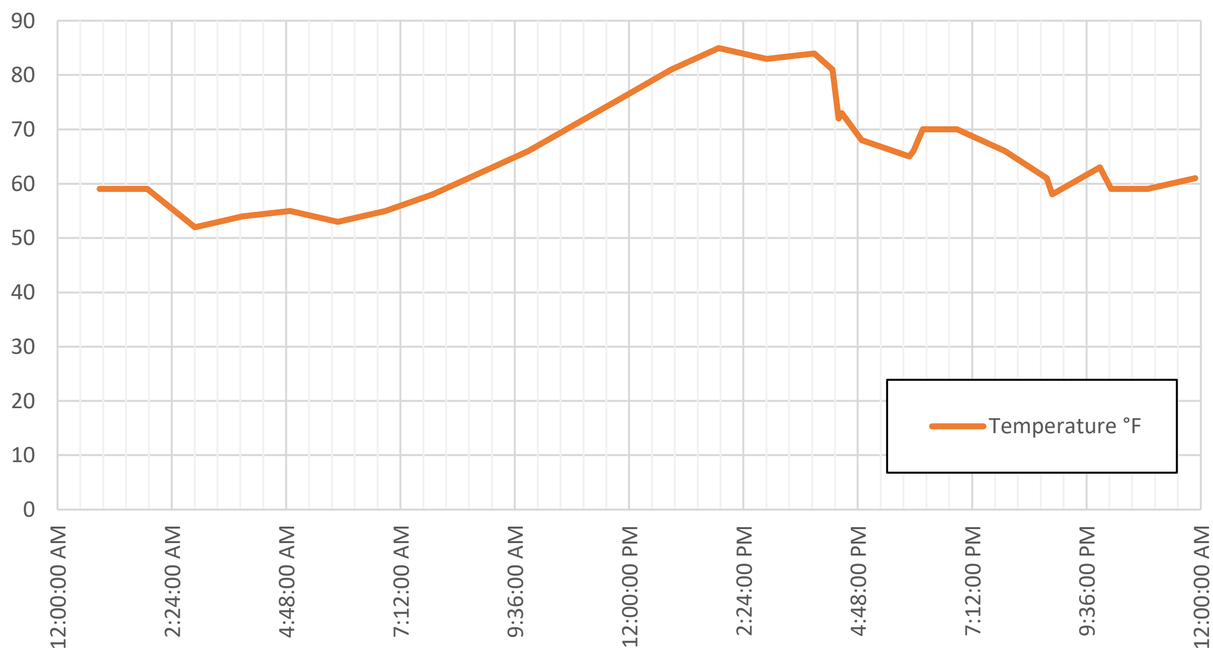


Figure 23. Average Air Temperature during 2013 Diurnal Profile Data Collection

2019 DIURNAL PROFILE TESTING

In Figures 24 through 27 the diurnal longitudinal profile measurements, collected in July 2019, display the changes in IRI over the time of a single day for each section. The diurnal data were collected at three separate times of the day; early morning, mid-morning and the afternoon. Figures 24 through 27 shows data for three runs during each time of the day for the left wheel path, right wheel path and the center lane. For section 080218, shown in Figure 12, IRI values decreased as average surface temperatures increased. with IRI values that start out in the "Fair" condition and remain that way throughout the day, with the exception of the Right Wheel Path in the afternoon approaching "Good" condition

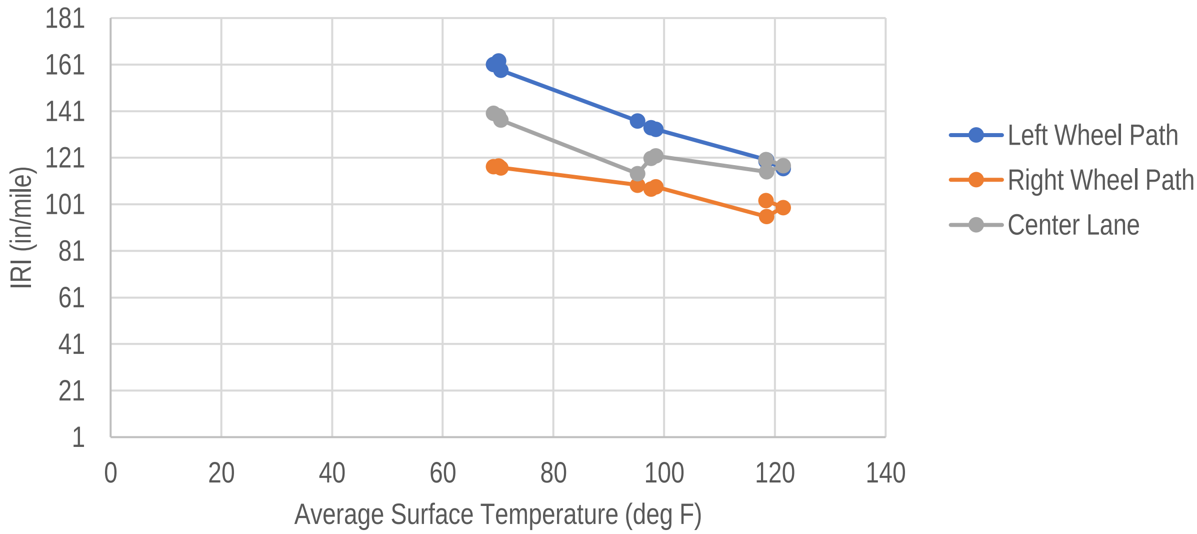


Figure 24. Section 080216 IRI versus temperature, July 13, 2019.

For section 080218, shown in Figure 25, IRI values slightly decreased as average surface temperatures increased, and fluctuated within the "Fair" category. During the afternoon, the lowest IRI value for 080218 was 106 inches/mile in the center of the lane, which is close to the "Good" category, but still puts it in the "Fair" category.

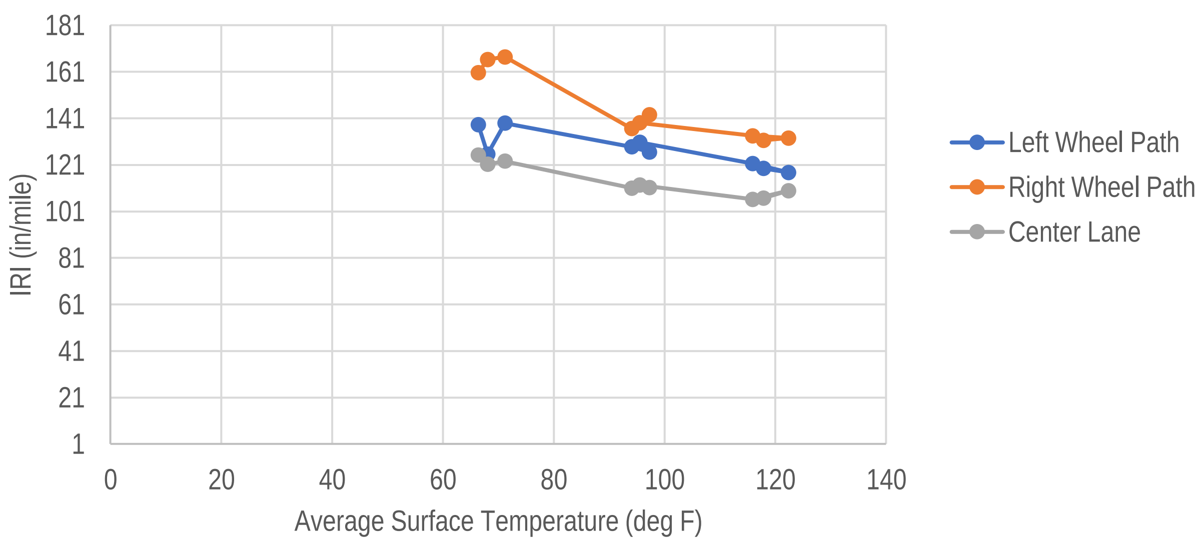


Figure 25. Section 080218 IRI versus temperature, July 13, 2019.

For section 080223, shown in Figure 26, IRI values actually increase as average surface temperatures increased, going from an average value of 114 inches/mile up to 128 inches/mile. There was a slight increase for both the left wheel path and center lane IRI values, but the right wheel path values increases a bit more between the mid-morning and afternoon testing. During the early morning, the lowest IRI value for 080223 was 108 inches/mile in the left wheel path.

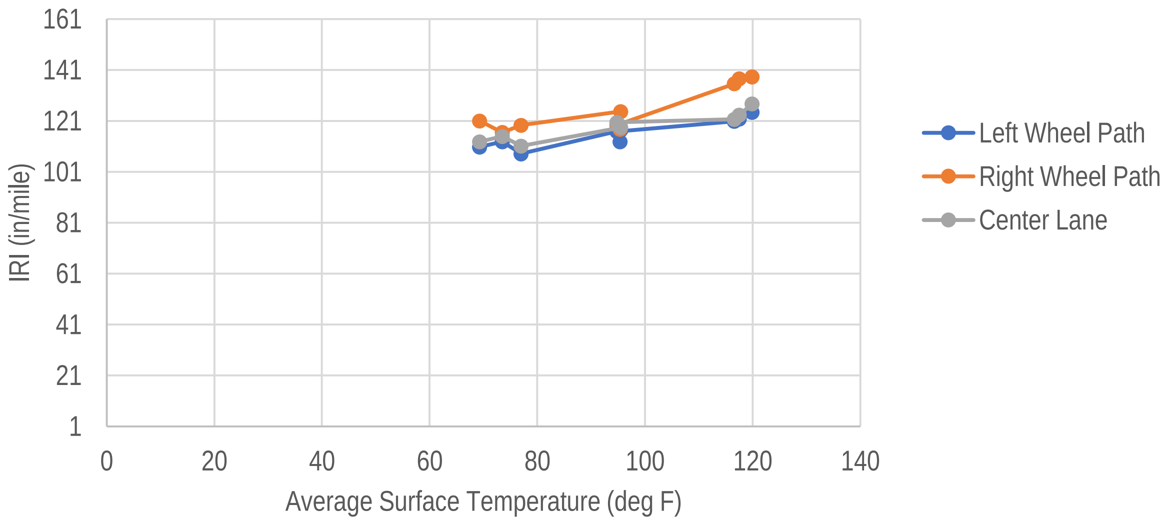


Figure 26. Section 080223 IRI versus temperature, July 13, 2019.

For section 080224 (Figure 27), the IRI values slightly decreased as average surface temperatures increased, decreasing from an average value of 132 inches/mile in the early morning to a value of 118 inches/mile in the afternoon. The IRI values decrease at a roughly steady rate for the center lane throughout the day. However, both wheel paths experience a larger decrease between the early morning and mid-morning testing and then a smaller decrease between the mid-morning and afternoon testing.

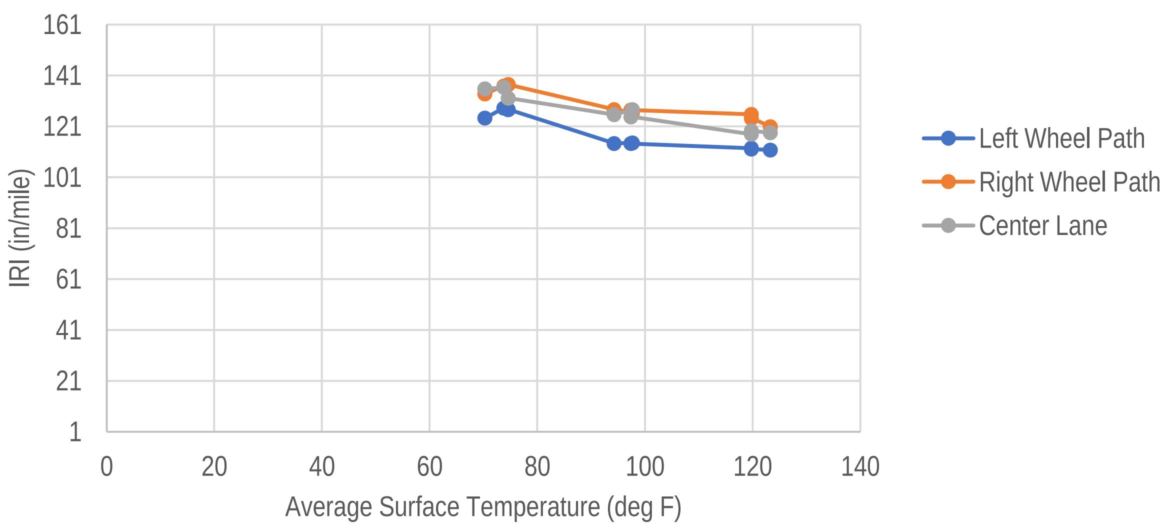


Figure 27. Section 080224 IRI versus temperature, July 13, 2019.

The historical hourly weather data from the Denver International Airport during the July 2019 diurnal profile testing is available and is presented in Table 11 and illustrated in Figure 28. The results show the temperature ranged from 61 to 96°F during the day. Profile measurements were collected from 05:08 to 06:55 (orange highlight), 10:00 to 10:55 (green highlight), and 13:20 to 14:23 (purple highlight). The 2013 diurnal profile measurements can be viewed in Attachments A thru D from the desktop study. This weather data would indicate there may be a larger change in IRI between the mid-morning and early afternoon testing than between the early morning and mid-morning IRI values. Figures 24 through 27 show this is the case for Sections 080218, 080223, and 080224. On section 080216, the IRI values appear to decrease at a constant rate between the various testing times, indicating the relative difference in test temperature does not result in corresponding relative differences in IRI.

The diurnal profile data was plotted in ProVAL in the same fashion as the 2013 profile data in Attachments A through D. The 2019 diurnal raw elevation profiles were very similar to those shown in Attachments A through D. The raw elevation plots showed that for sections 080216, 080218, and 080224 the slabs showed a trend of flattening as the temperature increased from the early morning to the afternoon runs. Section 080223 had an apparent increase in concave-down curvature as temperature increased, and the change in slab shape was more apparent between the early morning and mid-morning profile runs and became less apparent between the mid-morning and afternoon profile runs.

Table 11. Average Air Temperature during July 2019 Diurnal Profile Data Collection.

Time	Temperature	Time	Temperature
12:53 AM	63 F	4:58 PM	87 F
1:53 AM	63 F	5:05 PM	87 F
2:53 AM	67 F	5:29 PM	79 F
3:53 AM	67 F	5:37 PM	81 F
4:53 AM	65 F	5:53 PM	80 F
5:53 AM	61 F	6:19 PM	78 F
6:53 AM	64 F	6:53 PM	75 F
7:53 AM	67 F	7:16 PM	75 F
8:53 AM	72 F	7:37 PM	75 F
9:53 AM	78 F	7:53 PM	73 F
10:53 AM	85 F	8:20 PM	71 F
11:53 AM	89 F	8:28 PM	70 F
12:53 PM	92 F	8:53 PM	70 F
1:53 PM	96 F	9:53 PM	69 F
2:53 PM	96 F	10:53 PM	72 F
3:53 PM	92 F	11:53 PM	72 F

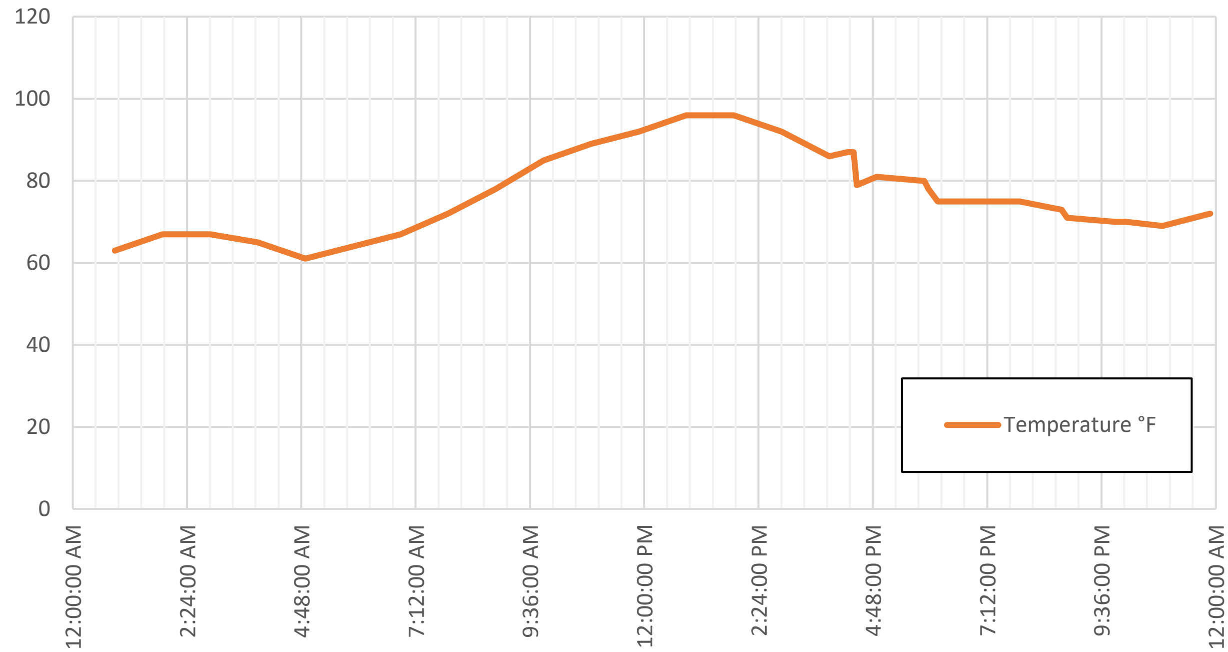


Figure 28. Average Air Temperature during July 2019 Diurnal Profile Data Collection

2020 PROFILE TESTING

Additional profile testing was conducted In January 2020 but was only collected following standard LTPP protocols where nine runs were collected between 13:52 and 15:42. The roughness values and average surface temperature for each section is summarized below in Table 12. There is a large difference in average surface temperature when comparing the 2019 and 2020 profile data, with the 2020 surface temperature about 44°F compared to a low temperature of about 70°F for the 2019 diurnal profile testing. Before continuing with this information, it should be noted that the original Table 1 indicated section 080224 did not change with temperature, whereas the subsequent data collection showed a decrease in IRI. This has been addressed in Revised Table 1.

Revised Table 1.IRI vs. temperature trends and experimental factors.

Section	Change in IRI with Increasing Temp			Concrete Mix	Slab Thick., in.	Slab Width, ft.	Base Type
	No Change	Increase	Decrease				
080213	X			550 PSI	8	14	DGAB
080214			X	900 PSI	8	12	DGAB
080216			X	900 PSI	11	14	DGAB
080218			X	900 PSI	8	12	LCB
080219	X			550 PSI	11	12	LCB
080220			X	900 PSI	11	14	LCB
080222			X	900 PSI	8	12	PATB
080223		X		550 PSI	11	12	PATB
080224			X	900 PSI	11	14	PATB

The 2019 diurnal profile testing and 2020 profile data were plotted and compared for each section. On sections 080216 and 080224 (Figures 29 and 32), the 2020 IRI values decrease as the temperature decreases when compared to the 2019 IRI values. This would mean the IRI values follow a slight concave down trend, where the IRI values start low at low temperatures and increase as temperatures increase, then start to decrease as temperature further increases. For section 080218 (Figure 30), the center lane and left wheel path IRI values remain constant as the temperature decreased from about 70°F in 2019 to 44°F in 2020 while the right wheel path IRI decreases from about 165 inches/mile in 2019 to 132 inches/mile in 2020. Lastly, for section 080223 (Figure 31) the IRI values for the center lane and both left and right wheel path remain the same in 2020 when compared to the early morning IRI values from 2019, indicating the IRI could remain low at low temperatures, but then being to increase at temperatures above 80°F.

Table 12. Average Air Temperature and IRI from 2020 Profile Data Collection.

Section	Average Surface Temperature (°F)	IRI (inches/mile)		
		LWP	RWP	Center
080216	41.2	107.9	111.8	122.7
080218	43.8	135.6	132.2	120.4
080223	44.1	109.6	122.7	121.8
080224	43.5	119.6	125.3	124.9

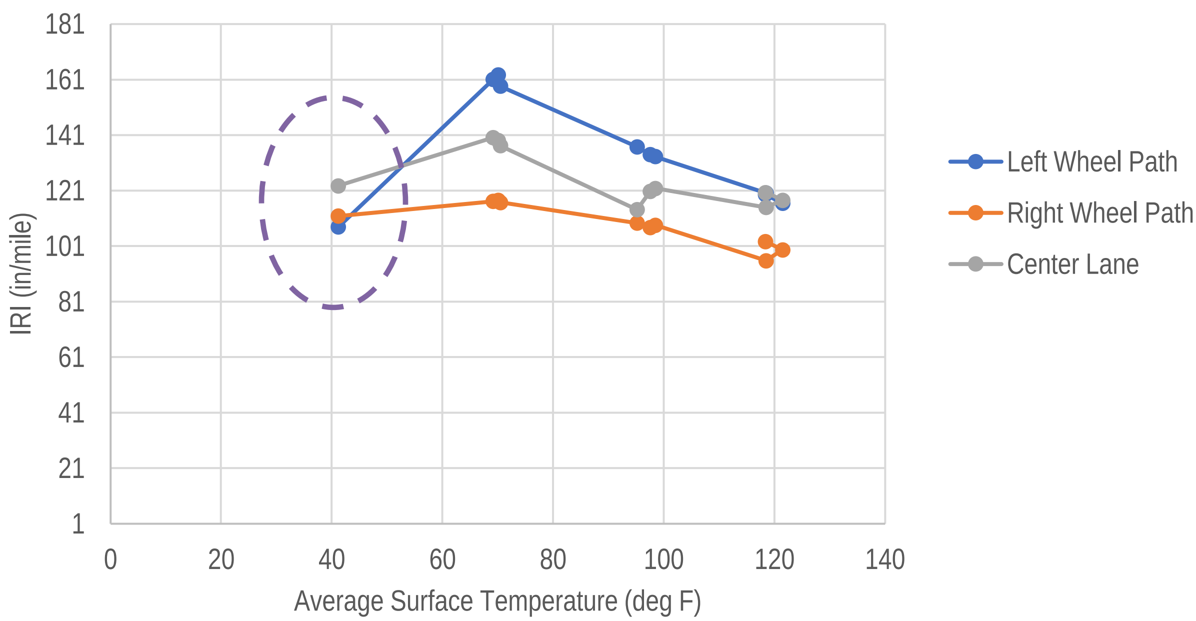


Figure 29. Section 080216 IRI versus temperature, 2019 and 2020 (circled) profile data.

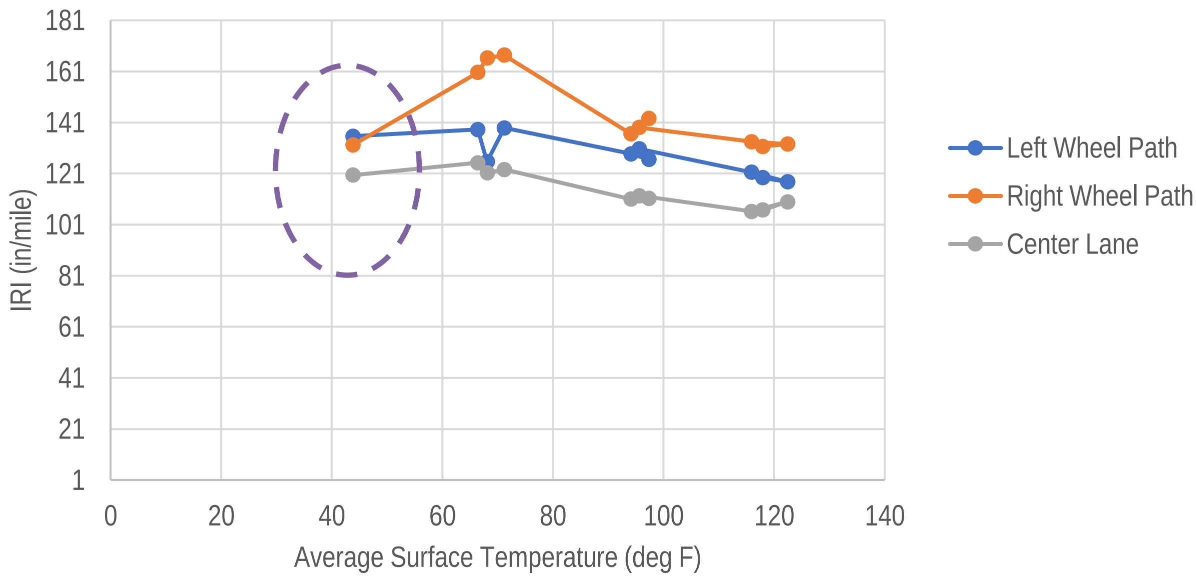


Figure 30. Section 080218 IRI versus temperature, 2019 and 2020 (circled) profile data.

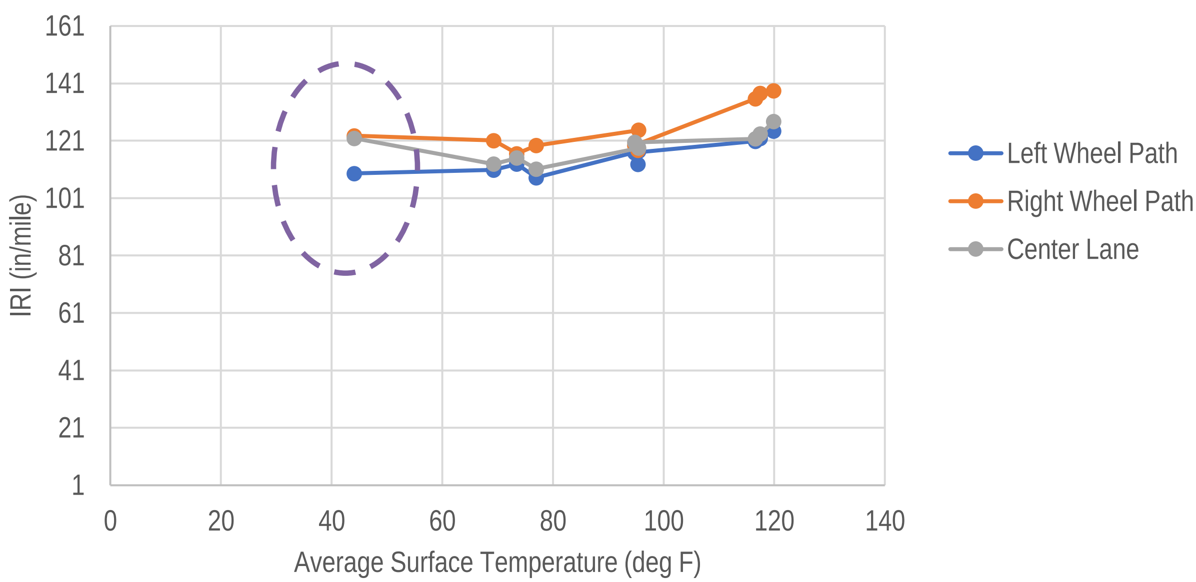


Figure 31. Section 080223 IRI versus temperature, 2019 and 2020 (circled) profile data.

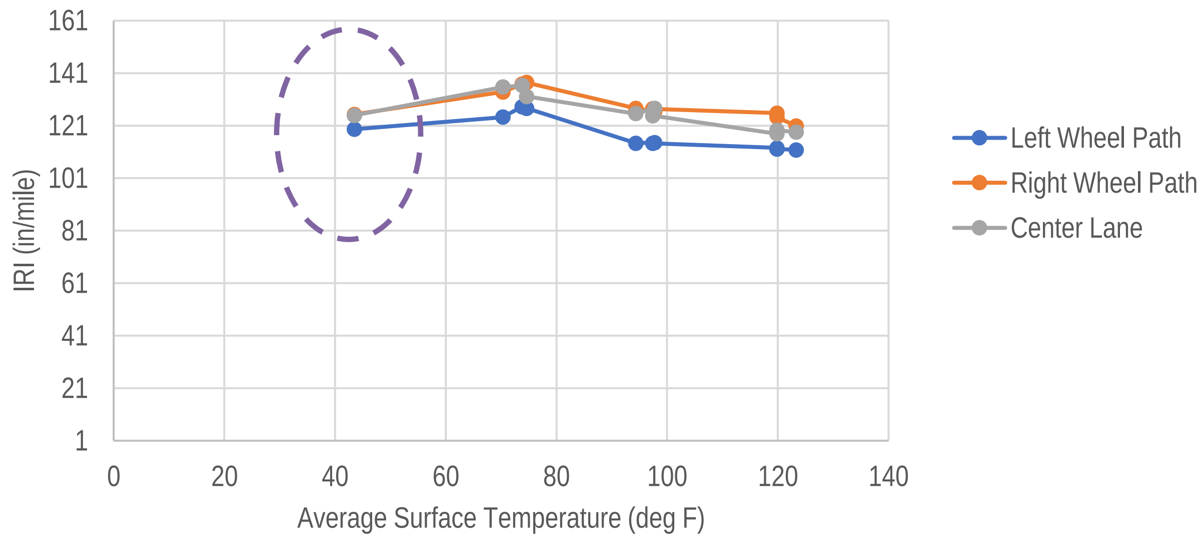


Figure 32. Section 080224 IRI versus temperature, 2019 and 2020 (circled) profile data.

DIURNAL DEFLECTION TESTING

Deflection measurements were collected on October 26, 2019. While originally planned on all four sections, due to time and weather constraints, FWD data were only collected on sections 080223 and 080224. The test procedures were specific to this investigation, wherein the approach and leave side of five joints and five slab edge tests were conducted for each section. Maximum deflections for each of the 15 FWD test points were normalized to a 9,000-pound load and plotted versus test time for sections 080223 and 080224 as illustrated in Figure 33 and Figure 34, respectively. Deflections on section 080223 are higher than those collected on section 080224. For both sections, the deflections drop as temperature increases.

Nominal maximum deflections were also plotted per FWD test point for sections 080223 and 080224 as illustrated in Figure 35 and Figure 36, respectively. JA-X represents an approach side of a joint, JL-X represents the leave side of a joint, E-X represents a slab edge, and X representing the joint or slab edge number. On both sections, the larger deflections occur at the joints, with section 080223 experiencing a larger drop from the morning to early afternoon tests compared to section 080224. The deflections then remain unchanged from the early afternoon to late afternoon for both sections.

An observation of note is the deflections at the slab edges remain virtually identical across all three test times for both sections. It is possible the slab edges are still tied to the shoulder and have limited curling movement associated with changing temperature. The decrease in deflections at the joints could be due to the slab having a downward curvature as the slab temperature increases and the slabs come into higher contact with the base layer. This would provide additional stiffness that would cause a reduction in deflections with this change in temperature.

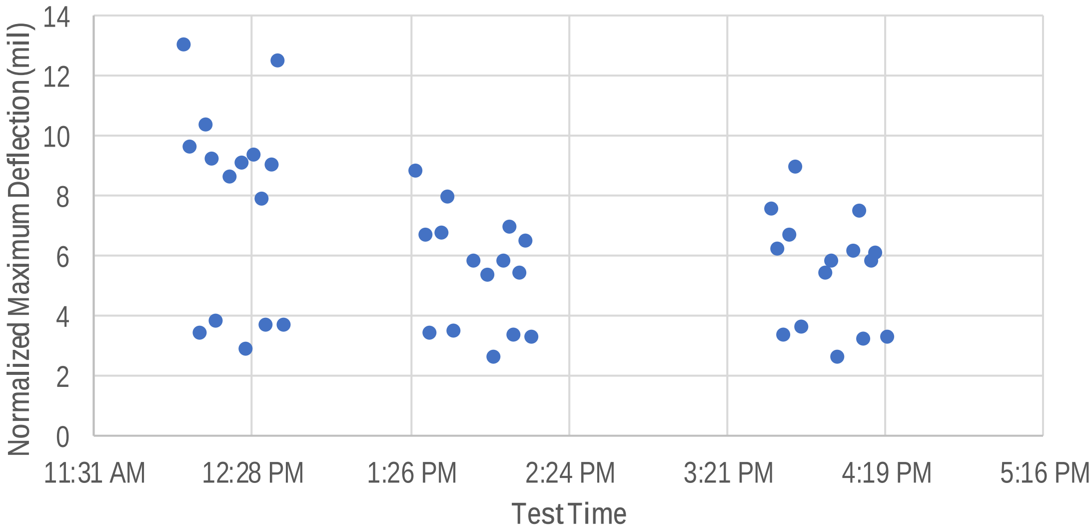


Figure 33. Normalized maximum deflection for 080223.

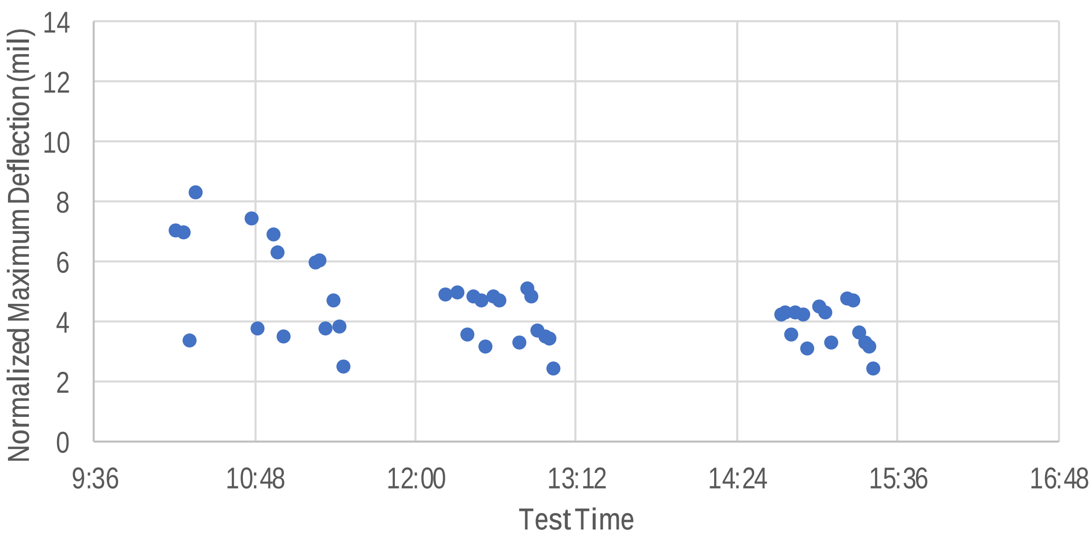


Figure 34. Normalized maximum deflection for 080224.

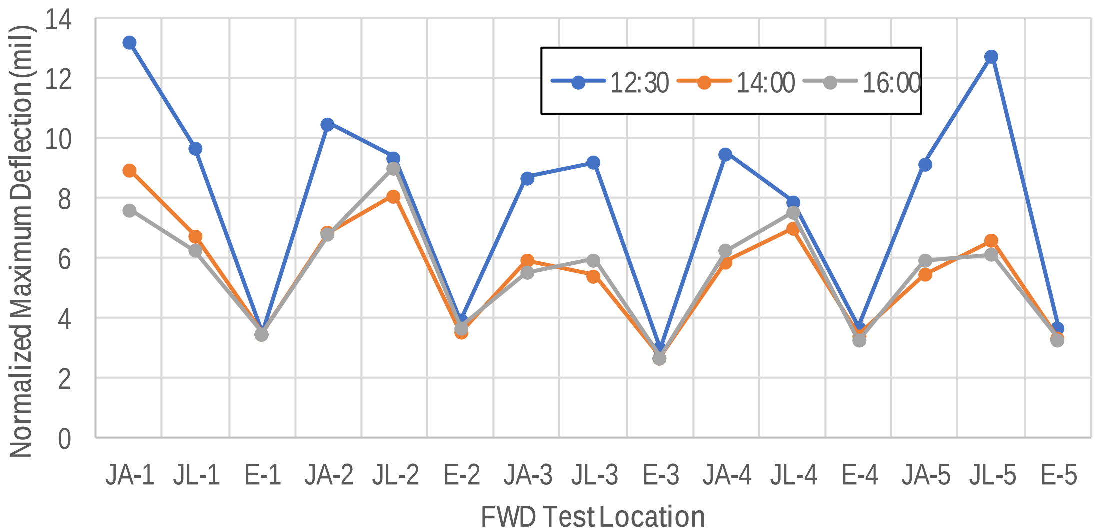


Figure 35. Normalized maximum deflection per test point for 080223.

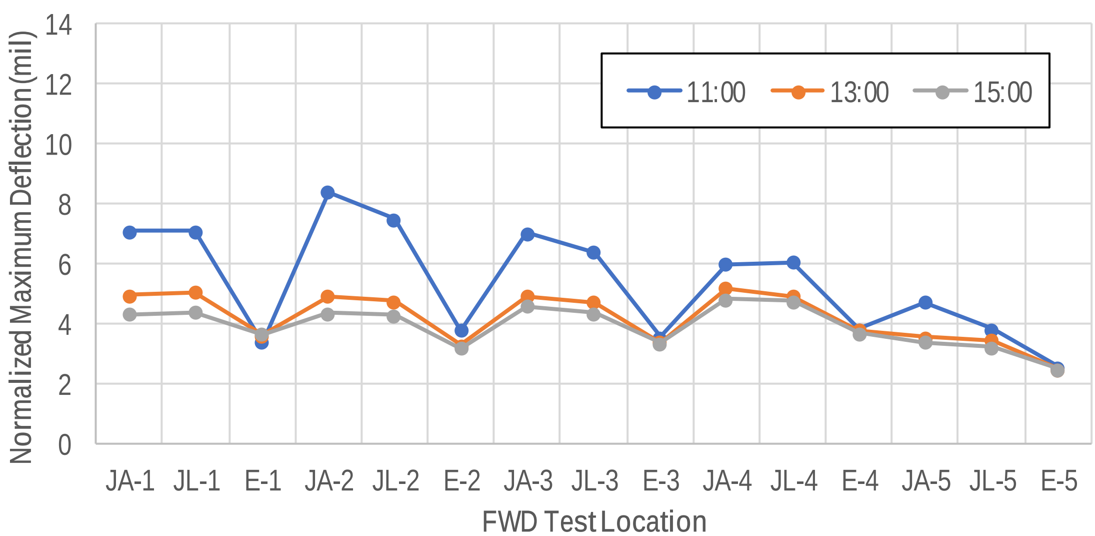


Figure 36. Normalized maximum deflection per test point for 080224.

Load transfer across each joint was calculated on both the approach and leave sides of each joint. Figures 37 and 38 illustrate the load transfer for sections 080223 and 080224, respectively. The average load transfers for both sections compare favorably to the values shown in Figure 17. On section 080223, the load transfer values are low in the morning and increase throughout the day. On Section 080224, the load transfer values are very high in the morning and slightly decrease throughout the day.

For section 080223, the load transfer values would indicate the dowel bars are no longer functioning as intended and the joints have excessive movement the dowel bars do not constrain. Thus, the increased load transfer efficiency values could be a result of the slabs curling as temperature increases and the slab edges come further into contact with the base layer. At this stage, the load transfer is being influenced by the additional support provided by the base layer, resulting in lower deflections on the loaded joint and causing the load transfer to increase.

For section 080224, the load transfer values start out extremely high and slightly decrease throughout the day. This behavior can't be easily explained, but given nearly all of the load transfer values are greater than 90%, the small decrease is not expected to cause any change in the pavement's performance.

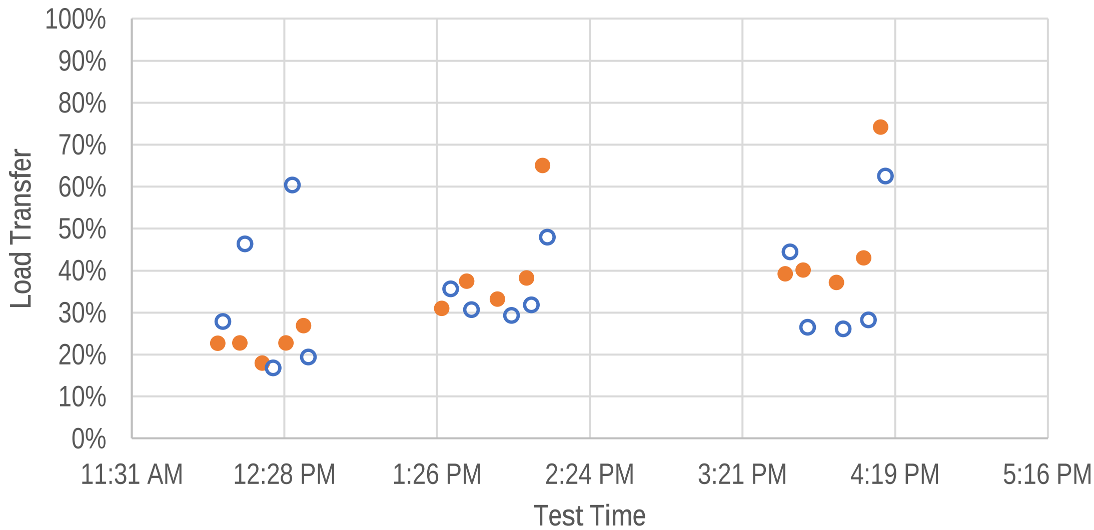


Figure 37. Time history plot of load transfer efficiency at approach (solid orange) and leave (blue) joints for 080223.

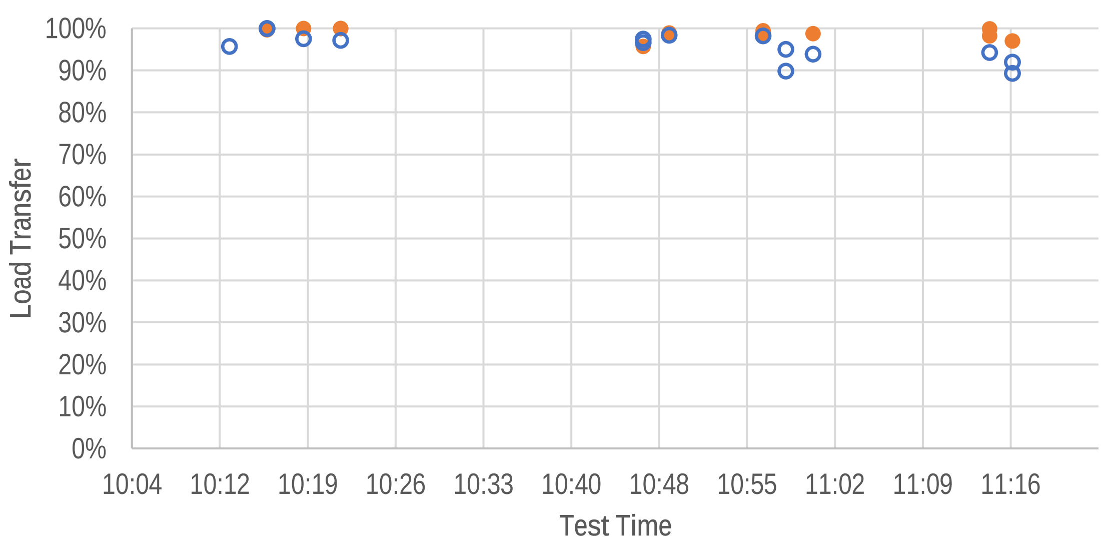


Figure 38. Time history plot of load transfer efficiency at approach (solid orange) and leave (blue) joints for 080224.

Lastly, a void-detection analysis was conducted at all FWD test locations. Maximum load was plotted on the Y-axis with applied load plotted on the X-axis for each load level at each FWD test location. A best-fit linear regression is then applied to the datapoints. The x-intercept should be close to zero in cases where there are no voids and an x-intercept that is farther away from zero indicates a higher potential for underslab voids. Figure 39 and Figure 40 illustrate the x-intercepts from the void analysis for sections 080223 and 080224, respectively. The results show that section 080224 has small x-intercept values that are close to zero and slightly decrease from the morning towards the afternoon. Section 080223 shows a different behavior with several locations possibly having underslab voids as based on the morning FWD data. However, as the temperature increases, the x-intercept values decrease in magnitude and are roughly similar to the x-intercept values observed in section 080224.

The void detection offset values for 080223 show there are possible voids in the morning hours, but as the temperature increases, the slab will curl and the joints will come further into contact with the underlying base material. This would result in lower void offset values as the temperature increases, which is supported by the data in Figure 35. The data cannot explain if the voids formed first and the excessive movement deteriorated load transfer, or if poor load transfer allowed the slabs to move excessively and slowly wear the base material away.

For section 080224, the void offset values do not change throughout the day and do not show any potential voids at any of the 15 test locations. The data can't explain whether or not the high load transfer has mean less potential for voids to occur, of if the lack of voids underneath the joints has kept the load transfer from eroding on section 080224, but both show the section is performing very well.

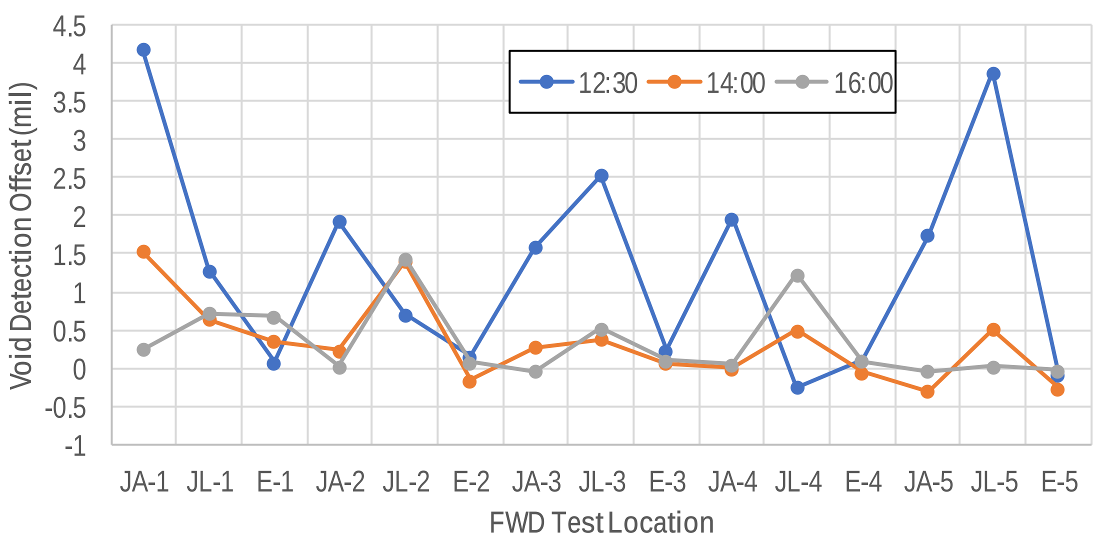


Figure 39. Void detection offset per test point for 080223.

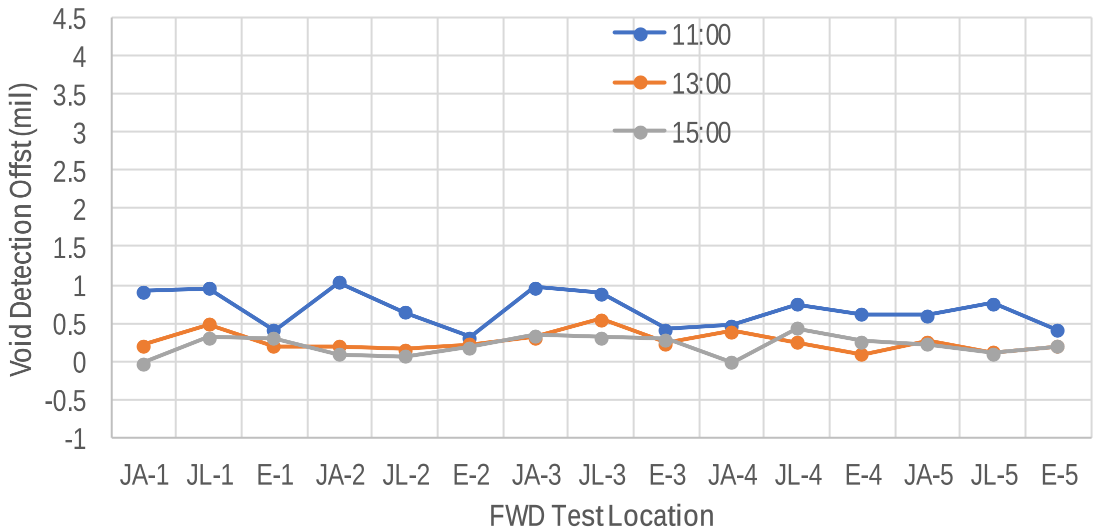


Figure 40. Void detection offset per test point for 080224.

DIURNAL FAULTING MEASUREMENTS

Faulting measurements were obtained on five joints from throughout sections 080223 and 080224 at three different times on October 26, 2019 in conjuncture with the FWD testing. Table 13 shows the faulting measurements for each section/time, which were performed at both a 0.3-meter and 0.7-meter offset from the edge. The results show there is negligible faulting in the morning, but as the temperatures increase, the faulting values become more negative. The faulting values on 080223 level out between the mid-day and afternoon testing. The faulting values on 080224, however, continue to decrease throughout the day.

These faulting measurements were compared to the values shown in Figure 22. For section 080223, the 12:30 measurements are similar to the last collected measurements in 2018, but the afternoon and early evening measurements are negative whereas historically the average faulting has never been below zero. For section 080224, none of the three recorded faulting measurements compared favorably with the historical average faulting values. These comparisons are not very powerful, as time constraints resulted in faulting only being measured for five joints rather than following LTPP data collection procedures and measuring faulting from all joints throughout the section. With only a limited number of faulting measurements collected, comparisons with historical faulting data should be done with care.

Table 13. Summary of Diurnal Faulting Measurements.

080223			080224		
Time	Station (m)	Faulting (mm)[1]	Time	Station (m)	Faulting (mm)[1]
12:30	2	0	11:00	2.6	0
	29.4	0.5		43.5	0
	61.4	1		71.1	0
	93.2	1		98.6	0.5
	125.2	0		130.5	1
Average Faulting (mm):		0.5	Average Faulting (mm):		0.3
14:00	2	-2	13:00	2.6	-2.5
	29.4	-2		43.5	-3
	61.4	-1.5		71.1	-2.5
	93.2	-0.5		98.6	-3.5
	125.2	-1.5		130.5	-0.5
Average Faulting (mm):		-1.5	Average Faulting (mm):		-2.4
16:00	2	-2	15:00	2.6	-4
	29.4	-1.5		43.5	-4.5
	61.4	-1		71.1	-3
	93.2	-0.5		98.6	-3.5
	125.2	-2		130.5	-1
Average Faulting (mm):		-1.4	Average Faulting (mm):		-3.2

Note 1: Reported faulting is the average of measurements taken at 0.3-meter and 0.7-meter offset from edge.

SUMMARY OF OBSERVATIONS AND CONCLUDING REMARKS

The following observations are based on the follow-up investigations on the Colorado SPS-2 test sections:

- 2019 diurnal longitudinal profile data confirms the behaviors observed in the original desktop study regarding the changes in ride quality with respect to temperature. This shows the behavior observed in 2013 was not an anomaly and instead was representative of the actual field performance of these sections.
- Diurnal FWD testing shows that section 080224 experiences little change in both load transfer and underslab voids with respect to temperature changes. This would indicate the pavement performance was not impacted by temperature and the section is structurally still in good condition.
- Section 080223 had increased load transfer efficiency values and decreased potential for underslab voids with an increase in temperature. This could indicate there are voids underneath the joints of this section, and as the slabs curl with rising temperature, the slab edges come into contact with the base layer.
- The limited number of diurnal faulting measurements conducted on sections 080223 and 080224 showed the faulting measurements for both sections decreased as temperatures increased. However, these values do not compare with historical faulting measurements, possibly due to the

limited number of joints tested for faulting. The negative faulting measurements should correspond to an increase in IRI, yet the IRI values drop as the "bump" at the joint gets bigger.

- It should be noted that the LCB received two coats of a wax-based curing compound prior to the PCC paving, and the PCC should not have bonded with the LCB.

It is recommended that additional field testing be conducted by the Colorado DOT, which would perform coring; and LTPP Program, which would include FWD testing by the current LTPP Data Collection Contractor and laboratory testing performed by the Turner Fairbanks Highway Research Center:

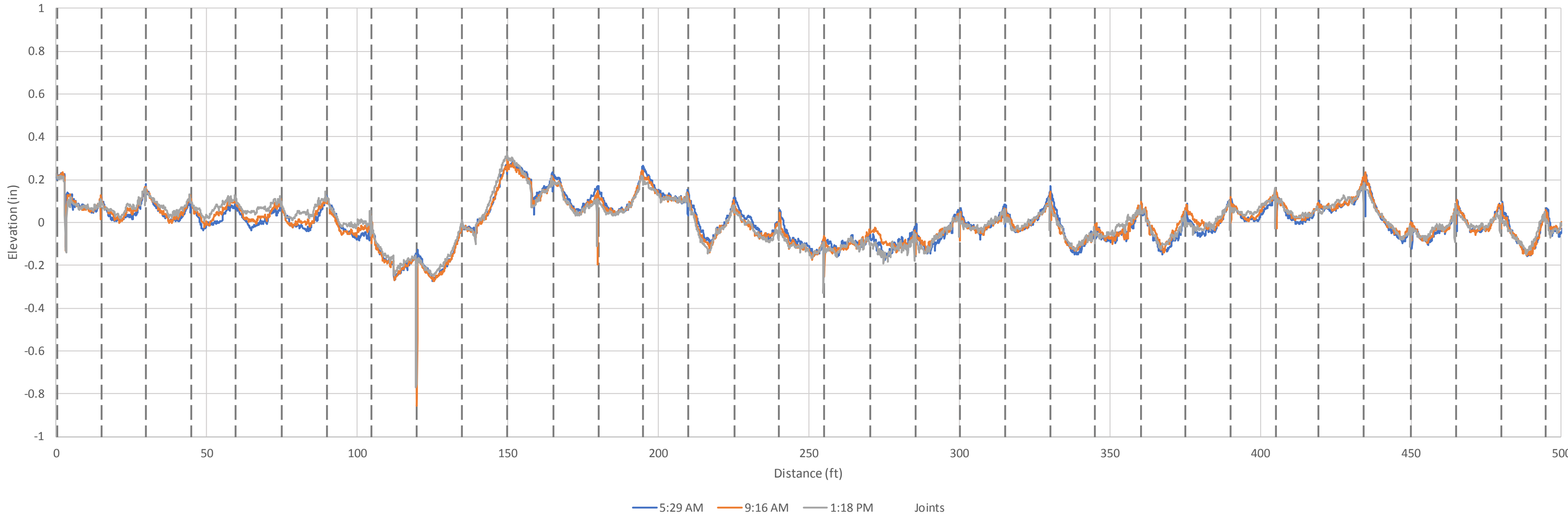
- Obtain additional cores from within the test sections for CTE testing and aggregate type identification.
- Core through dowels on section 080223 and another section with apparent good LTE and faulting performance. Identify whether socketing is the cause of poor load transfer and if possible, determine presence of underslab voids at joints.
- Core section 080218 to determine whether slab and LCB are bonded, and, if practical, also core the other test sections constructed with LCB.
- Core at least three test sections each for the two different concrete mixes to support an analysis of strength gains over time, and a petrographic assessment of hydration of the cement and w/c ratio for each mix.
- Collect additional FWD data from sections 080223 and 080224 in accordance with LTPP protocols to capture data throughout the test sections.

Additional topics that could be considered as part of a broader investigation include:

- Compare raw profile measurements to faultmeter measurements and examine whether the faulting is the same.
- Attempt to determine the minimum perceptible fault level and assess any correlation with IRI.

Aside from the aforementioned testing recommendations, the LTPP database appears to have adequate data to explain the performance of the Colorado SPS-2 projects in terms of the various measures considered in this study. There are years of historical FWD, IRI, and surface distress data. Additionally, there are currently two sets of diurnal profile testing collected in 2013 and again in 2019. Diurnal FWD testing was collected in 2019 and is recommended to be performed again. Additional coring from within the sections would confirm layer thicknesses in addition to supporting the topics already described.

ATTACHMENT A: PROVAL PLOTS FOR TEST SECTION 080216



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0.4
0.2
0
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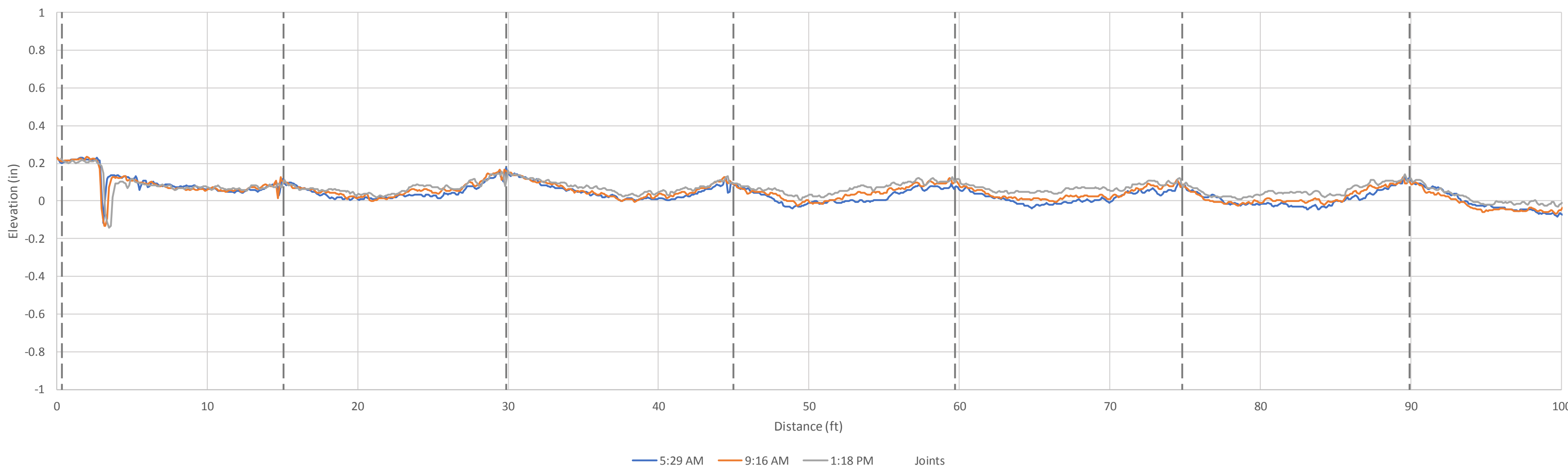
-0.4
-0.6
-0.8
-1
Elevation (in)
0
50
100
150
200
250
300
350
400
450
500
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints



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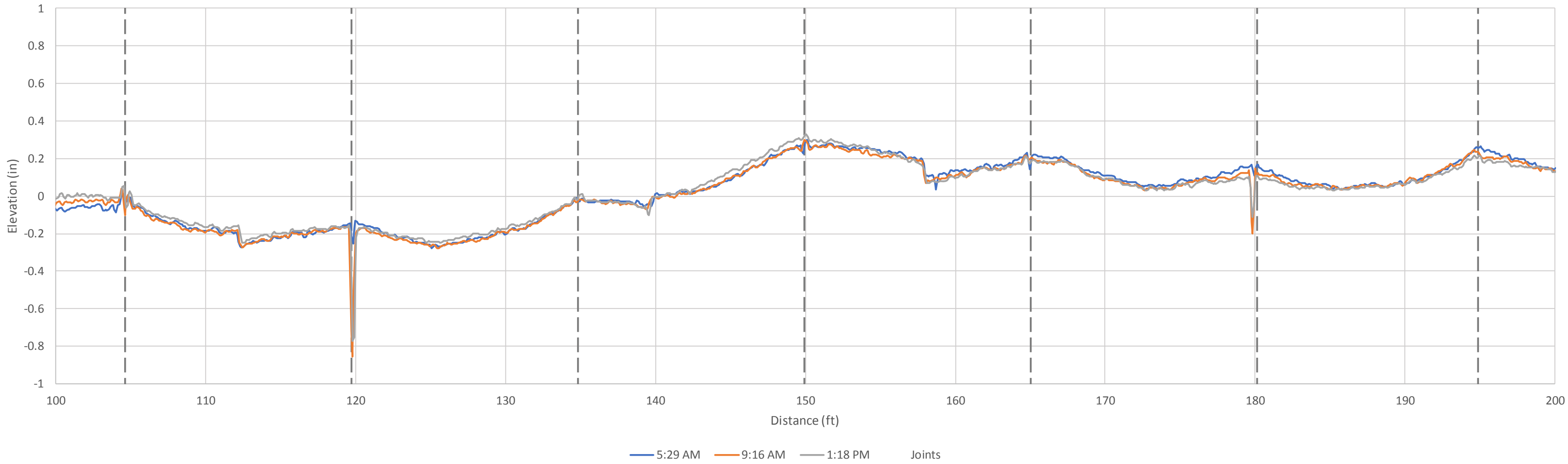
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
0
10
20
30
40
50
60
70
80
90
100
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints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0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
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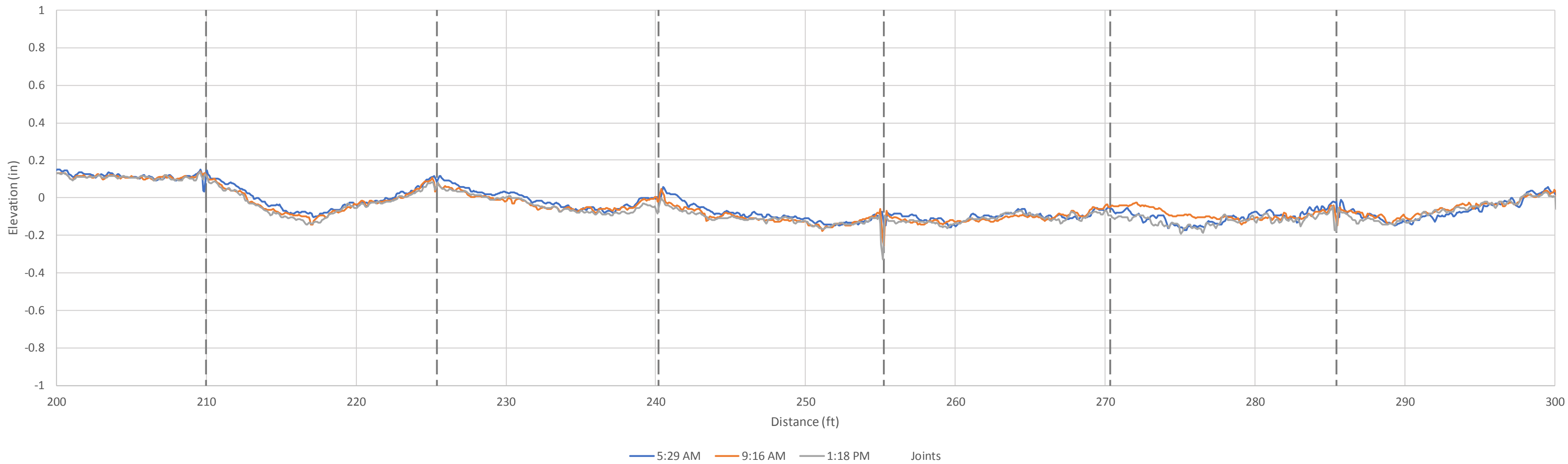
110
120
130
140
150
160
170
180
190
200
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints


1
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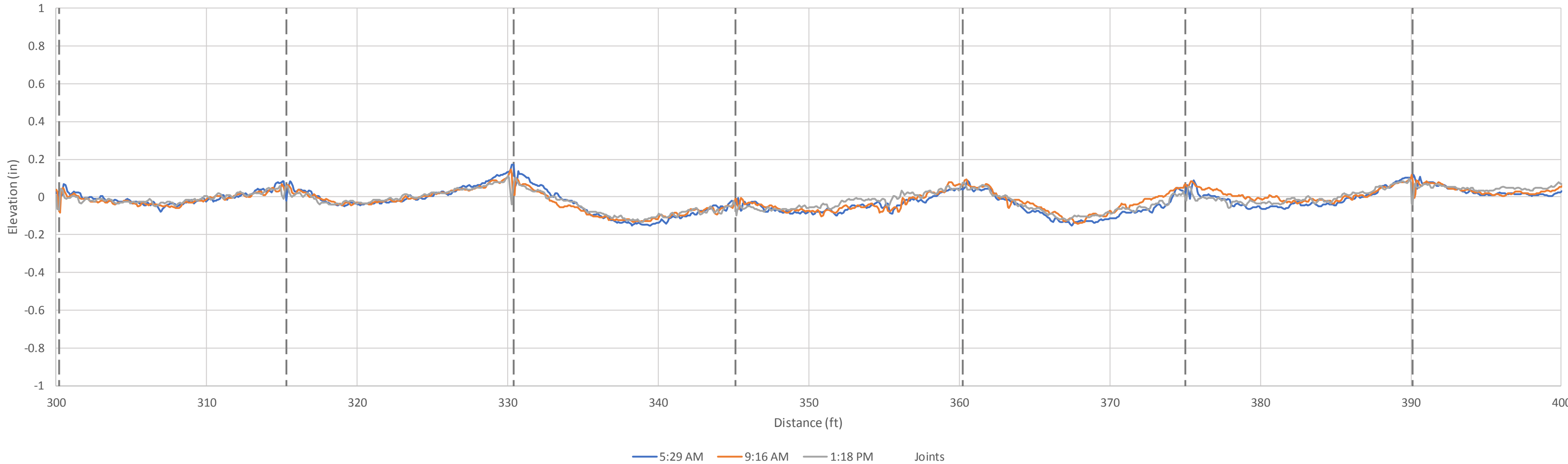
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
200
210
220
230
240
250
260
270
280
290
300
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints



1
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0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
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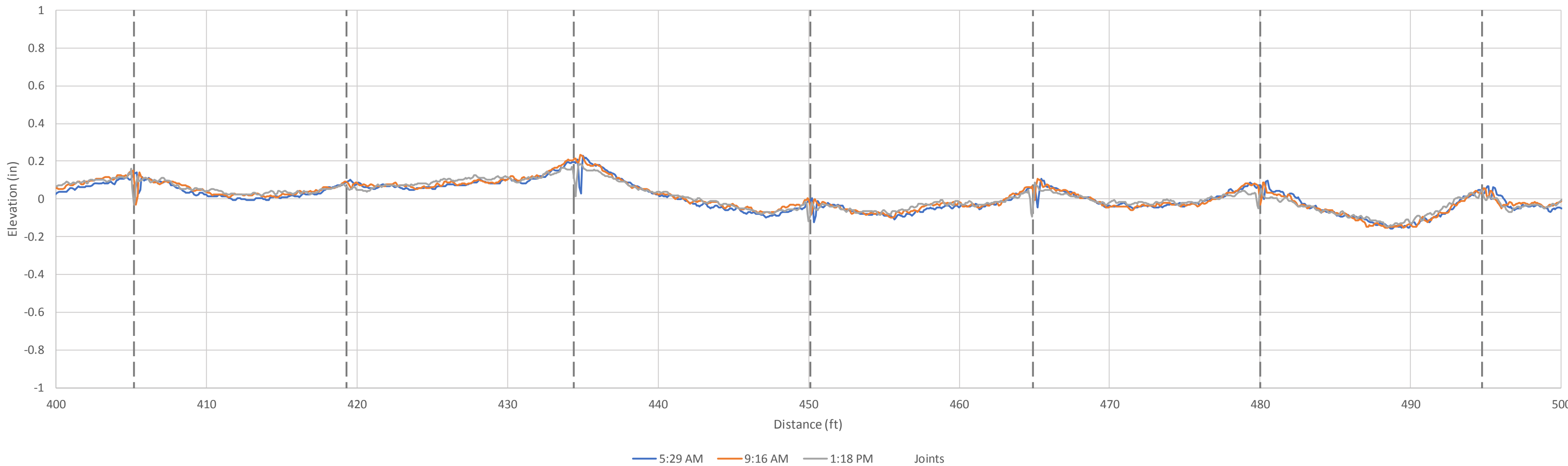
310
320
330
340
350
360
370
380
390
400
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints



1
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0.4
0.2
0
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-0.4
-0.6
-0.8
-1
Elevation (in)
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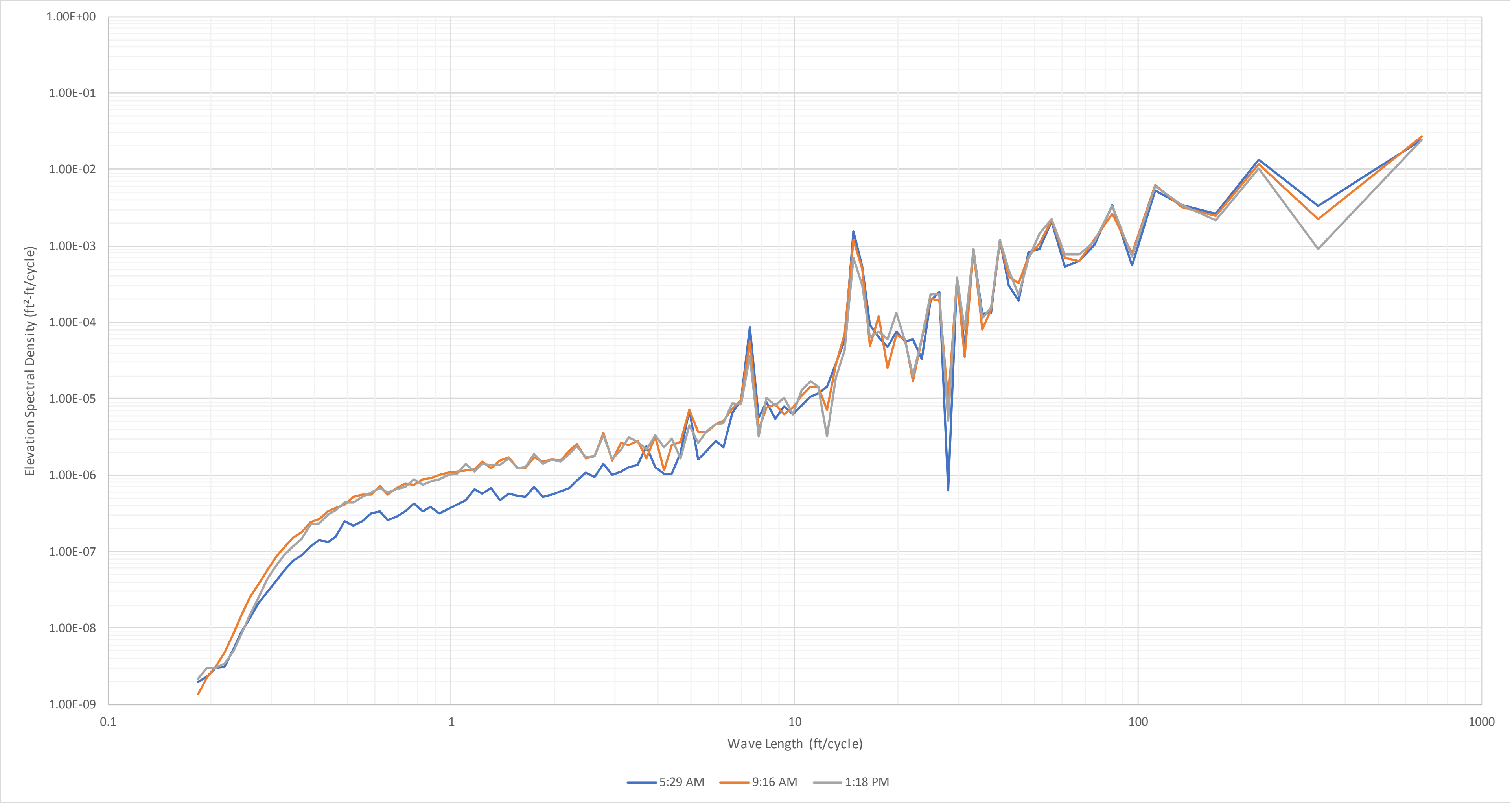
410
420
430
440
450
460
470
480
490
500
Distance (ft)
5:29 AM
9:16 AM
1:18 PM
Joints



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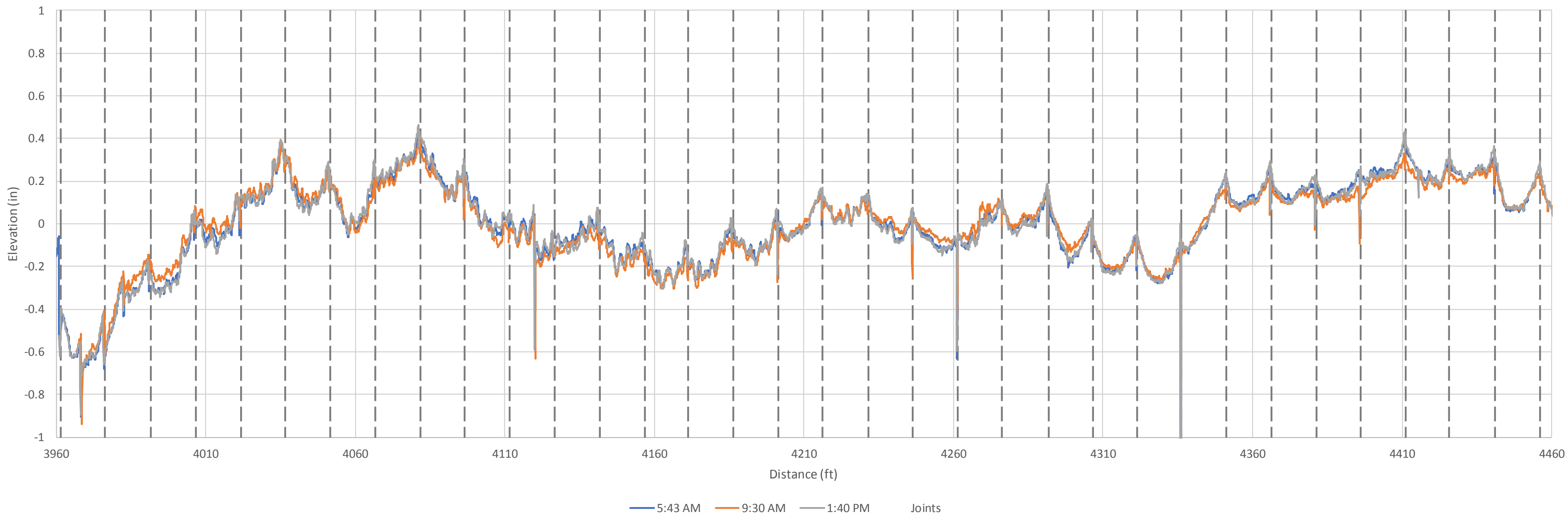
1.00E-02
1.00E-03
1.00E-04
1.00E-05
1.00E-06
1.00E-07
1.00E-08
1.00E-09
Elevation Spectral Density (ft²-ft/cycle)
0.1
1
10
100
1000
Wave Length (ft/cycle)
5:29 AM
9:16 AM
1:18 PM

ATTACHMENT B: PROVAL PLOTS FOR TEST SECTION 080218



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0.4
0.2
0
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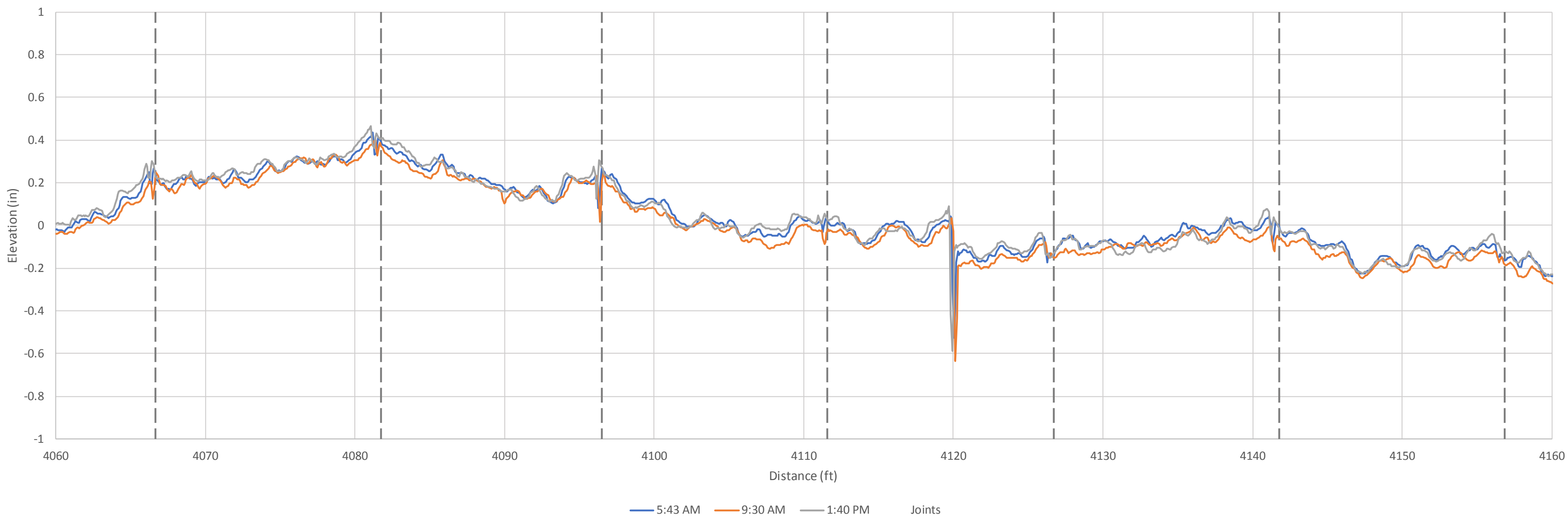
-0.4
-0.6
-0.8
-1
Elevation (in)
3960
4010
4060
4110
4160
4210
4260
4310
4360
4410
4460
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints

1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
3960
3970
3980
3990
4000
4010
4020
4030
4040
4050
4060
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints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0.6
0.4
0.2
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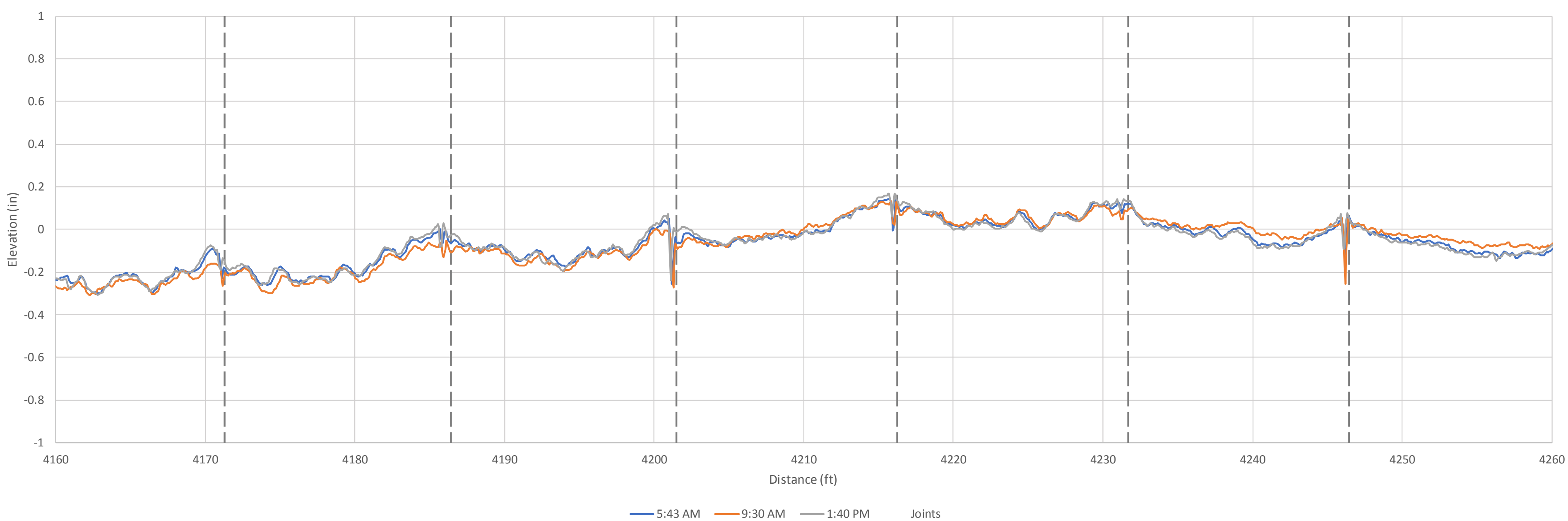
-0.6
-0.8
-1
Elevation (in)
4060
4070
4080
4090
4100
4110
4120
4130
4140
4150
4160
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints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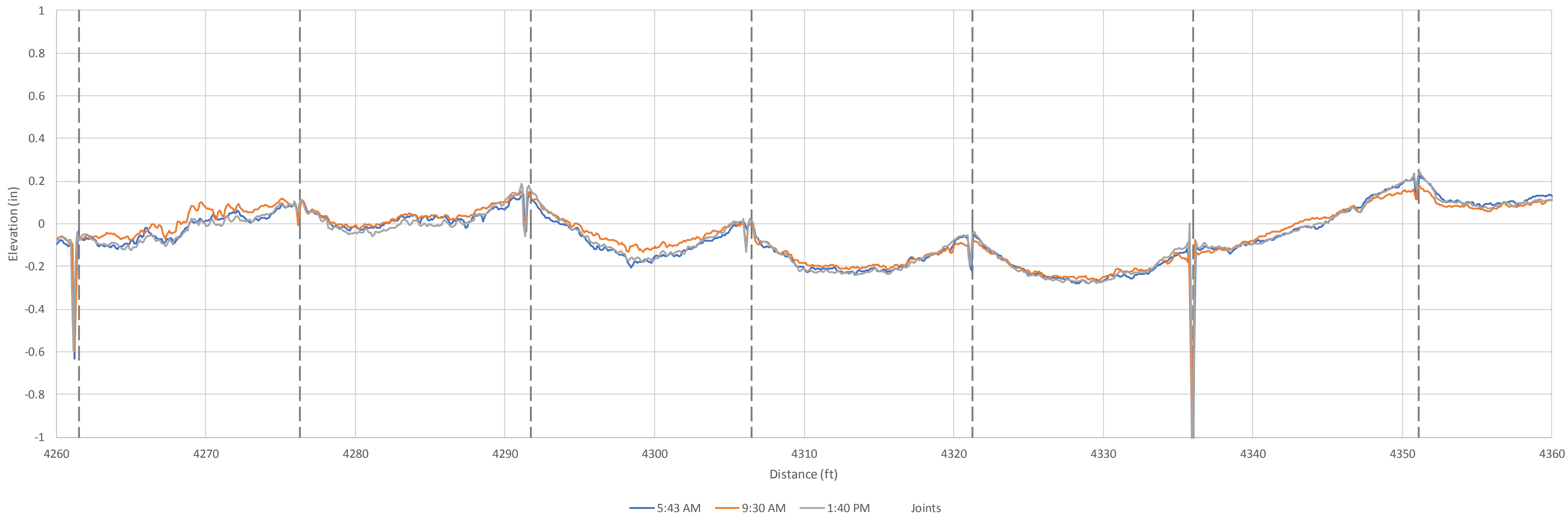
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
4160
4170
4180
4190
4200
4210
4220
4230
4240
4250
4260
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints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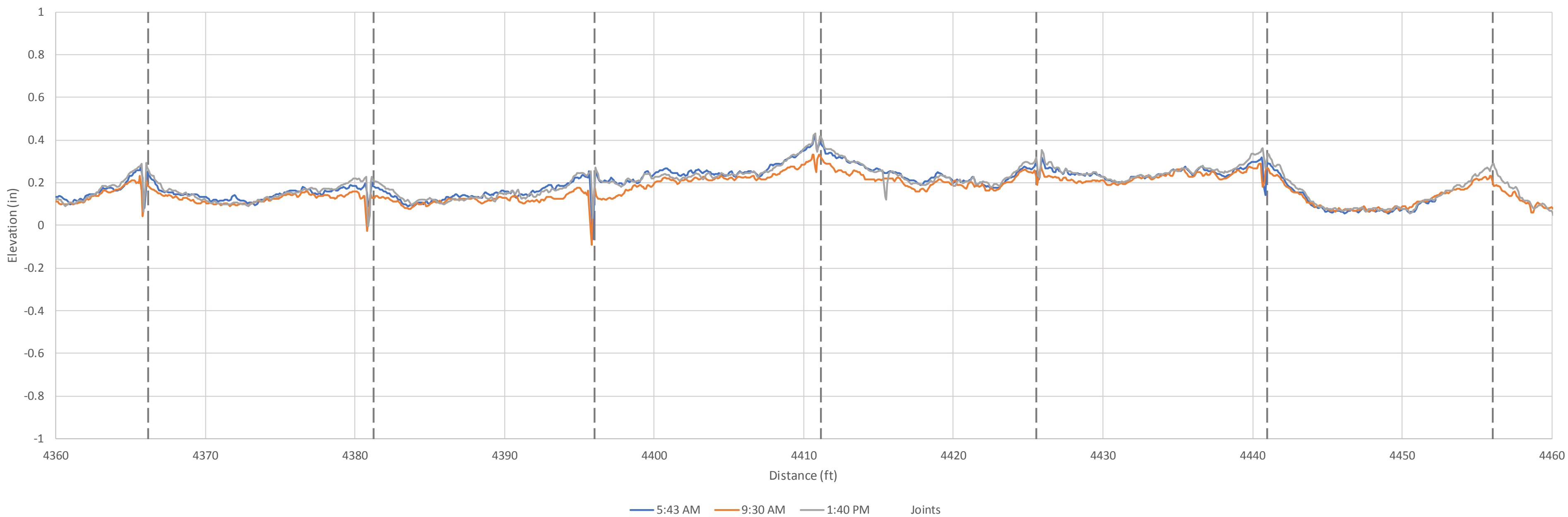
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
4260
4270
4280
4290
4300
4310
4320
4330
4340
4350
4360
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints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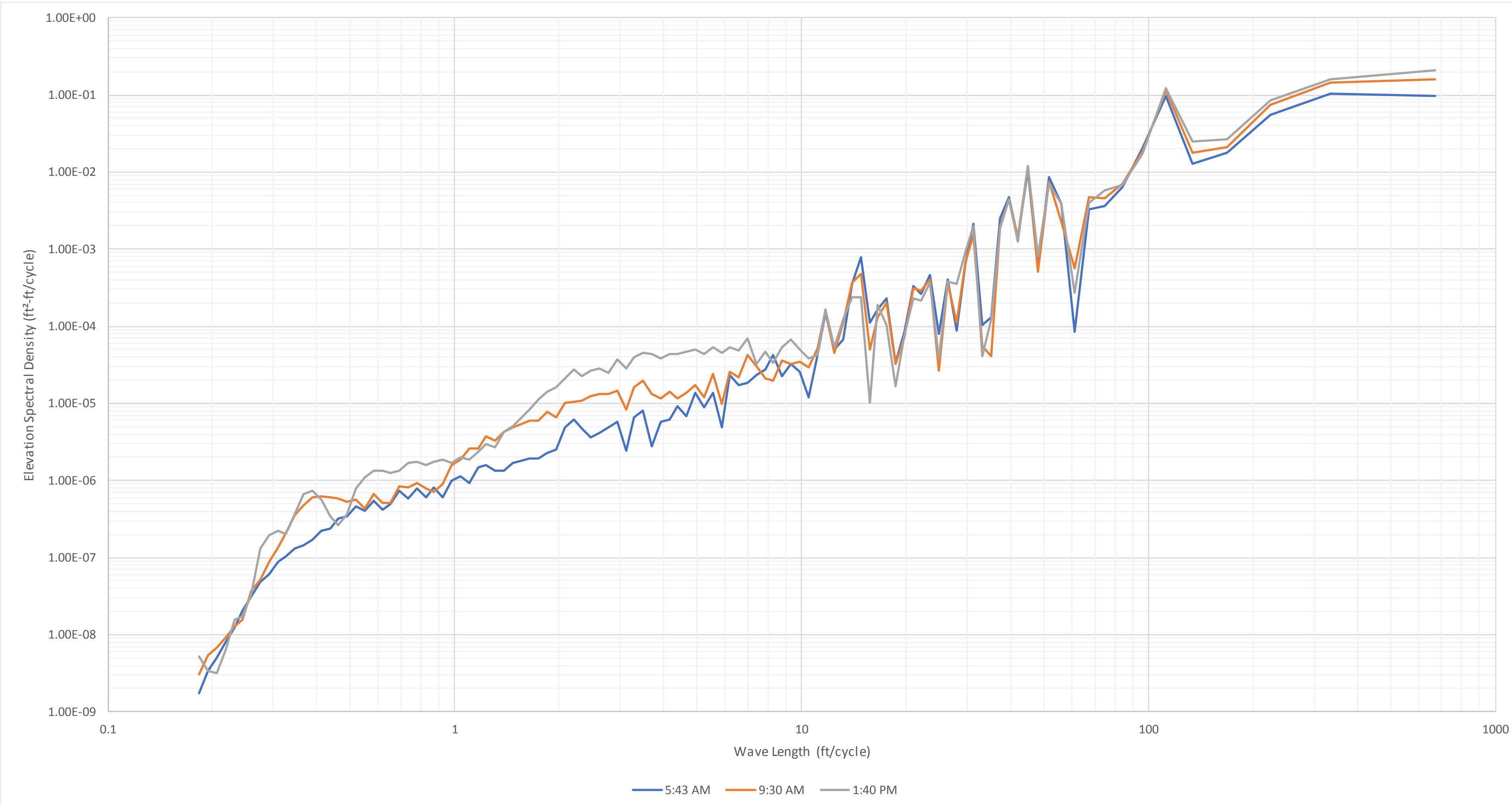
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
4360
4370
4380
4390
4400
4410
4420
4430
4440
4450
4460
Distance (ft)
5:43 AM
9:30 AM
1:40 PM
Joints



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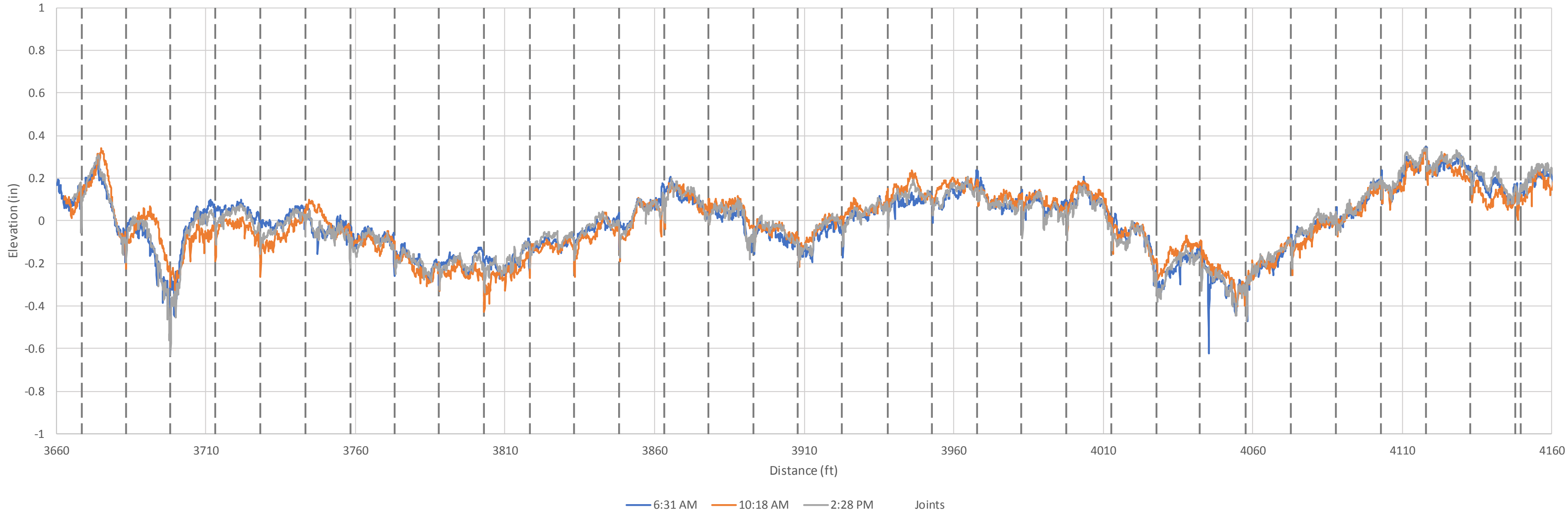
1.00E-01
1.00E-02
1.00E-03
1.00E-04
1.00E-05
1.00E-06
1.00E-07
1.00E-08
1.00E-09
0.1
1
10
100
1000
Elevation Spectral Density (ft²-ft/cycle)
Wave Length (ft/cycle)
5:43 AM
9:30 AM
1:40 PM

ATTACHMENT C: PROVAL PLOTS FOR TEST SECTION 080223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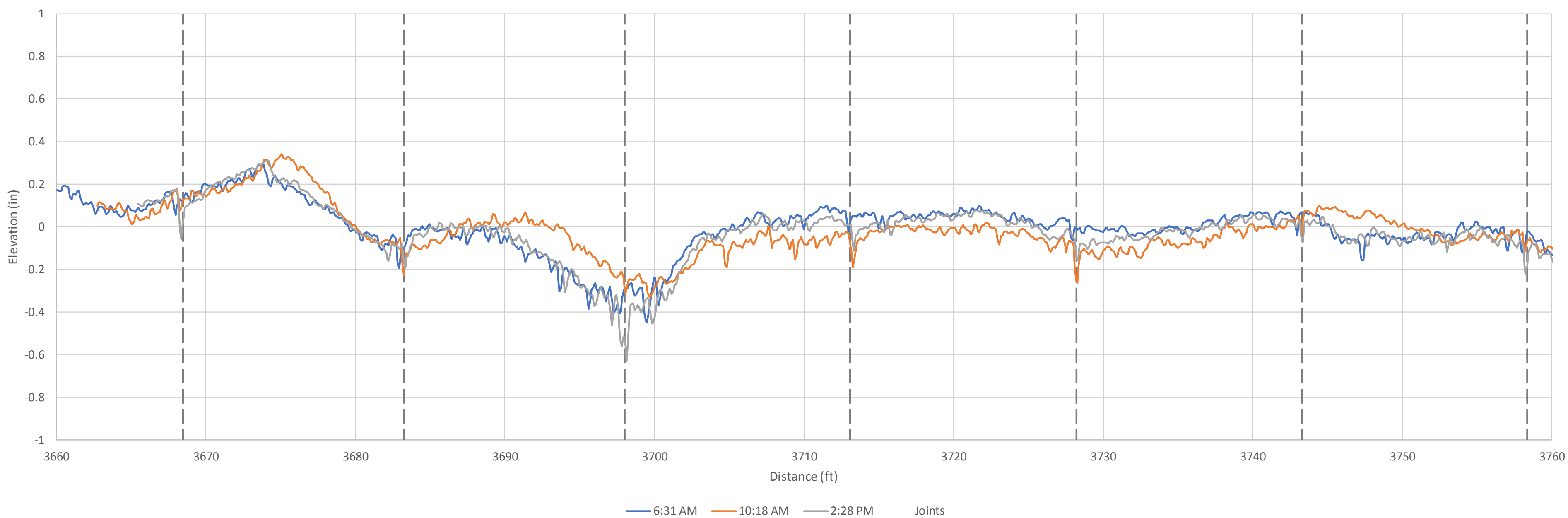
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
3660
3710
3760
3810
3860
3910
3960
4010
4060
4110
4160
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints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0.4
0.2
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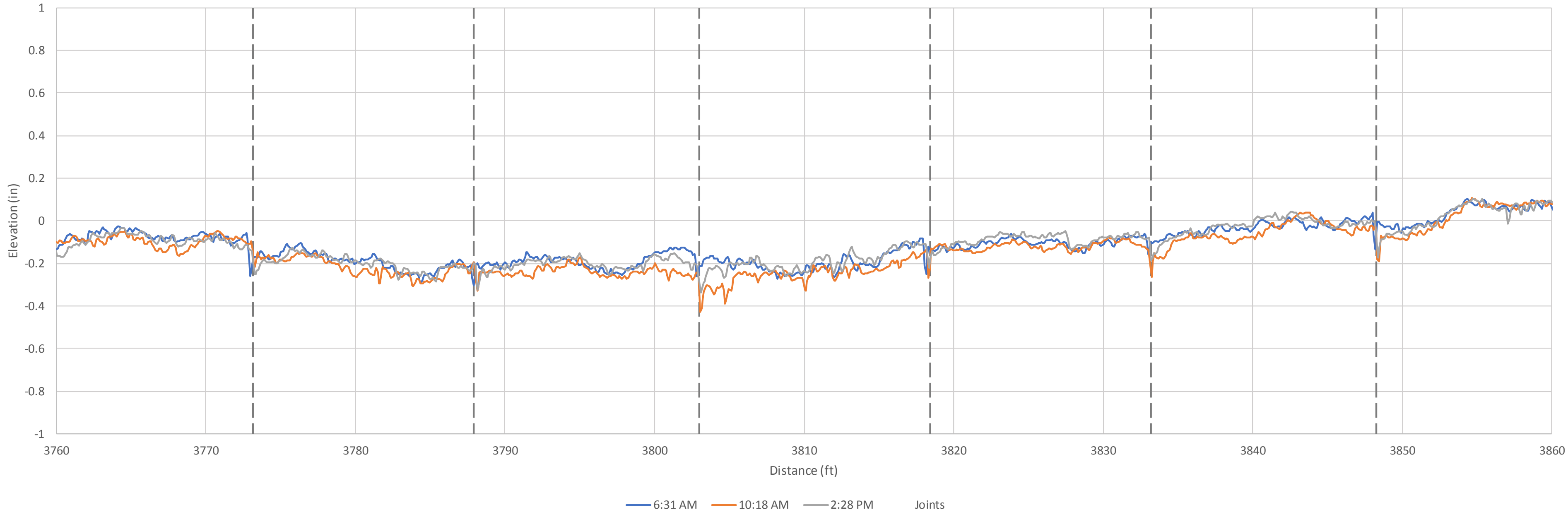
-0.8
-1
Elevation (in)
3660
3670
3680
3690
3700
3710
3720
3730
3740
3750
3760
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints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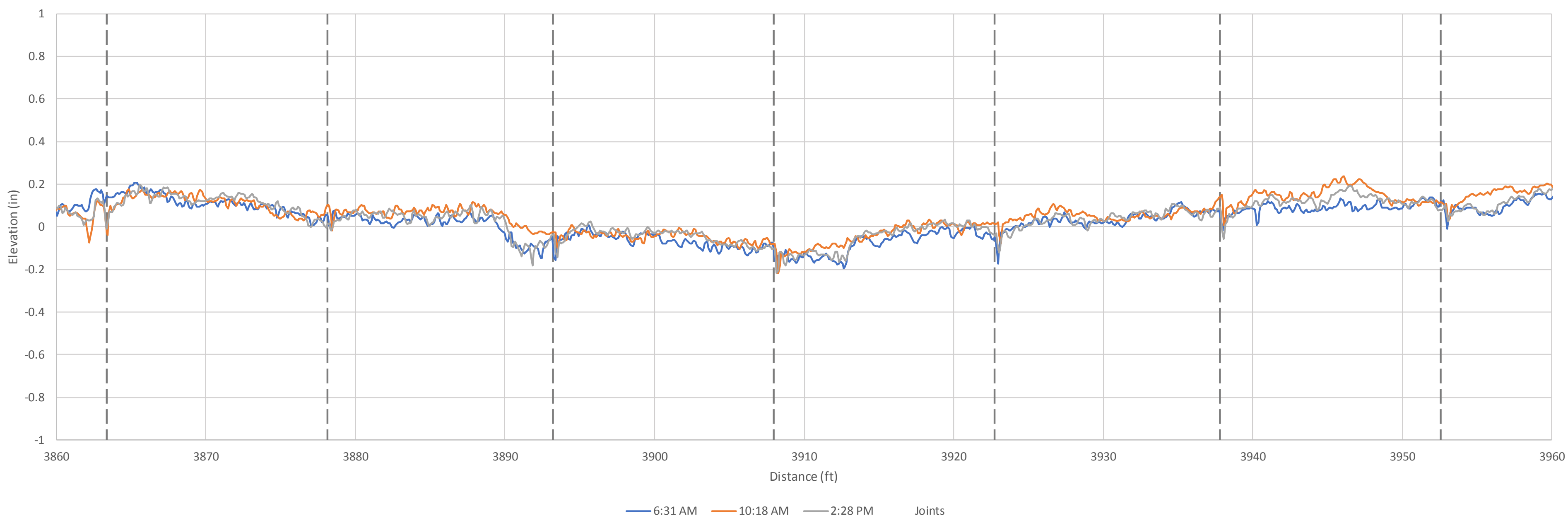
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
3760
3770
3780
3790
3800
3810
3820
3830
3840
3850
3860
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints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0.8
0.6
0.4
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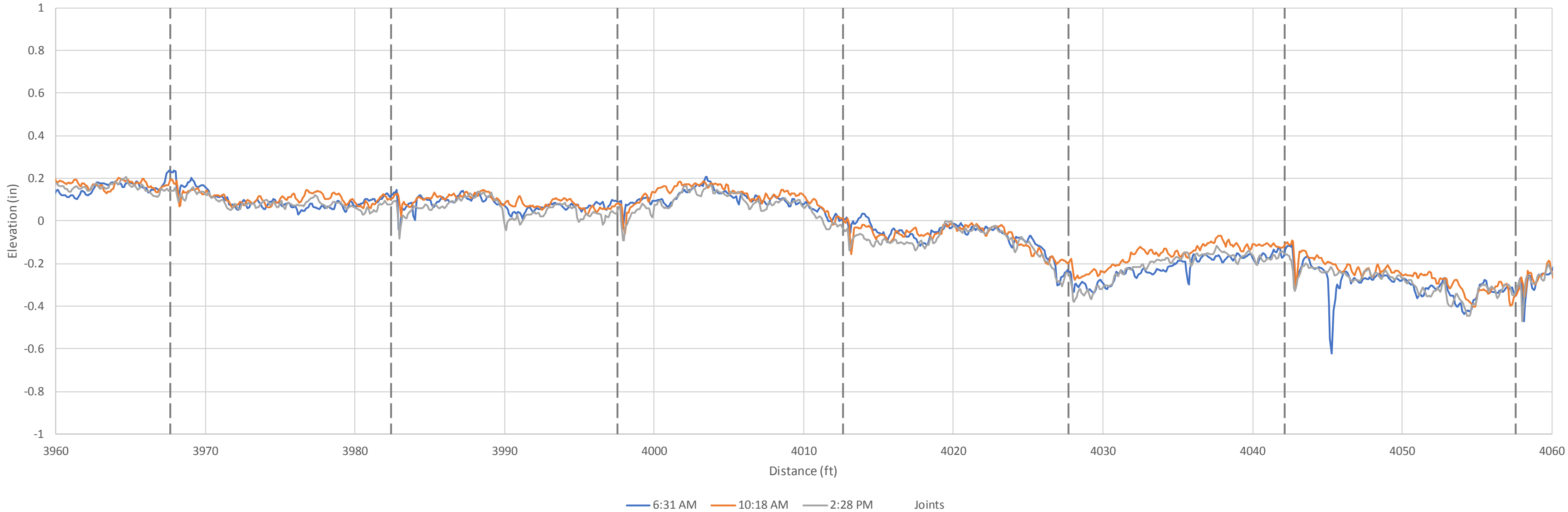
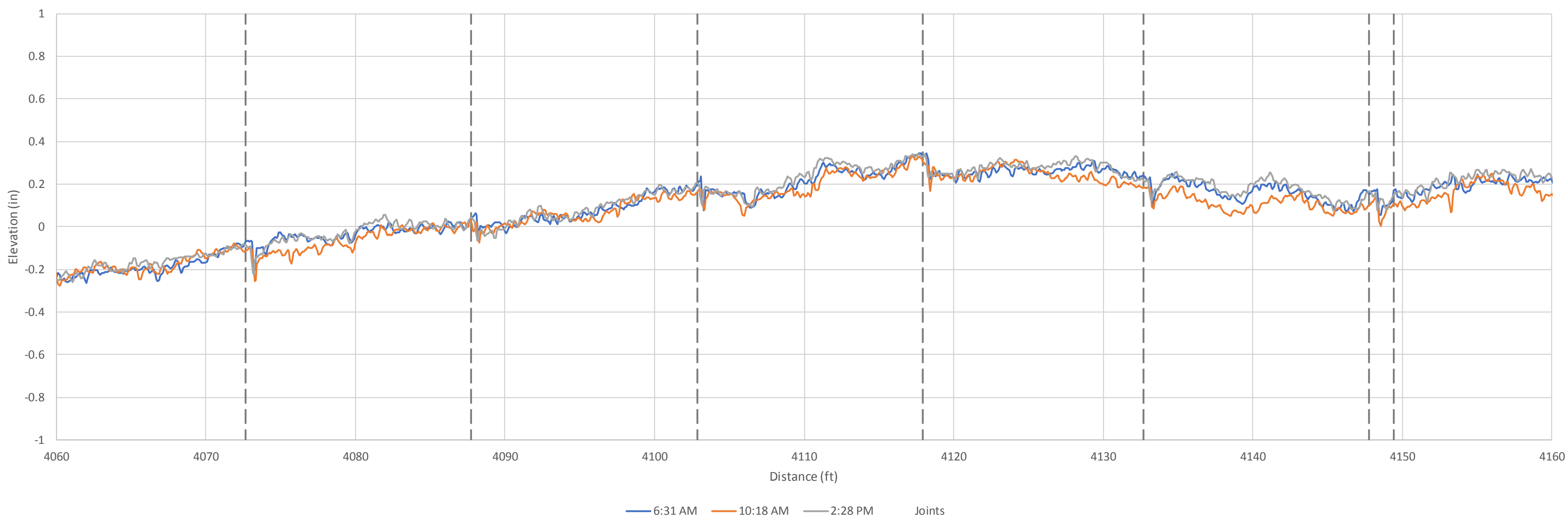
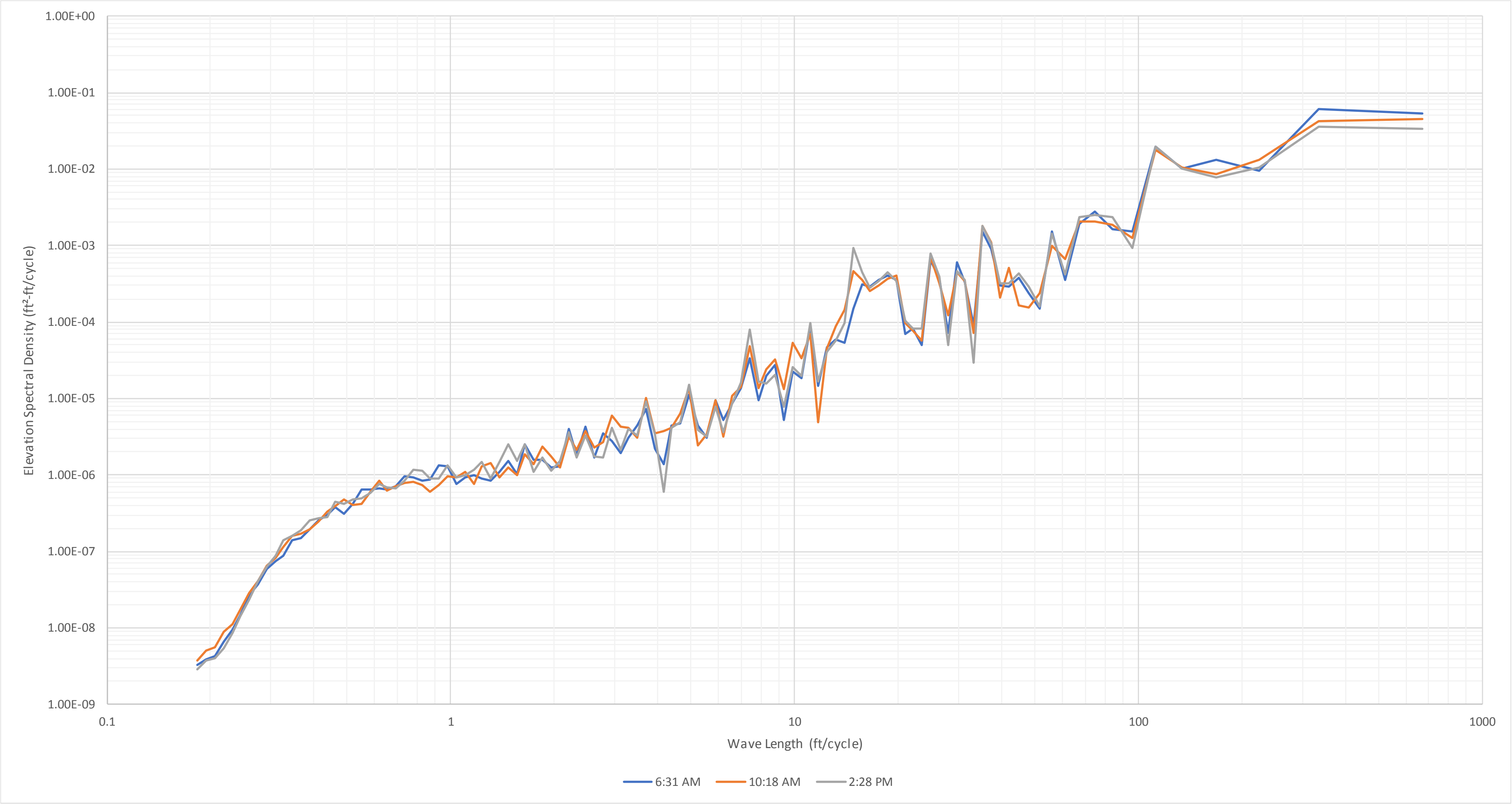
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
3860
3870
3880
3890
3900
3910
3920
3930
3940
3950
3960
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints



Elevation (in)
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints

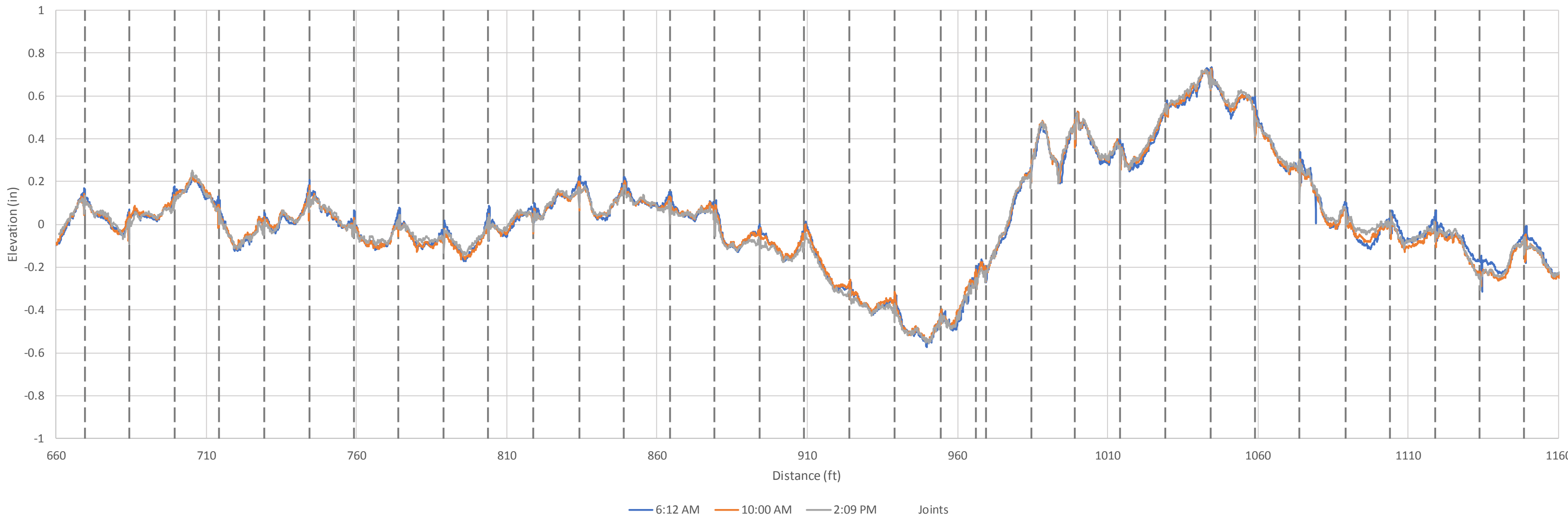


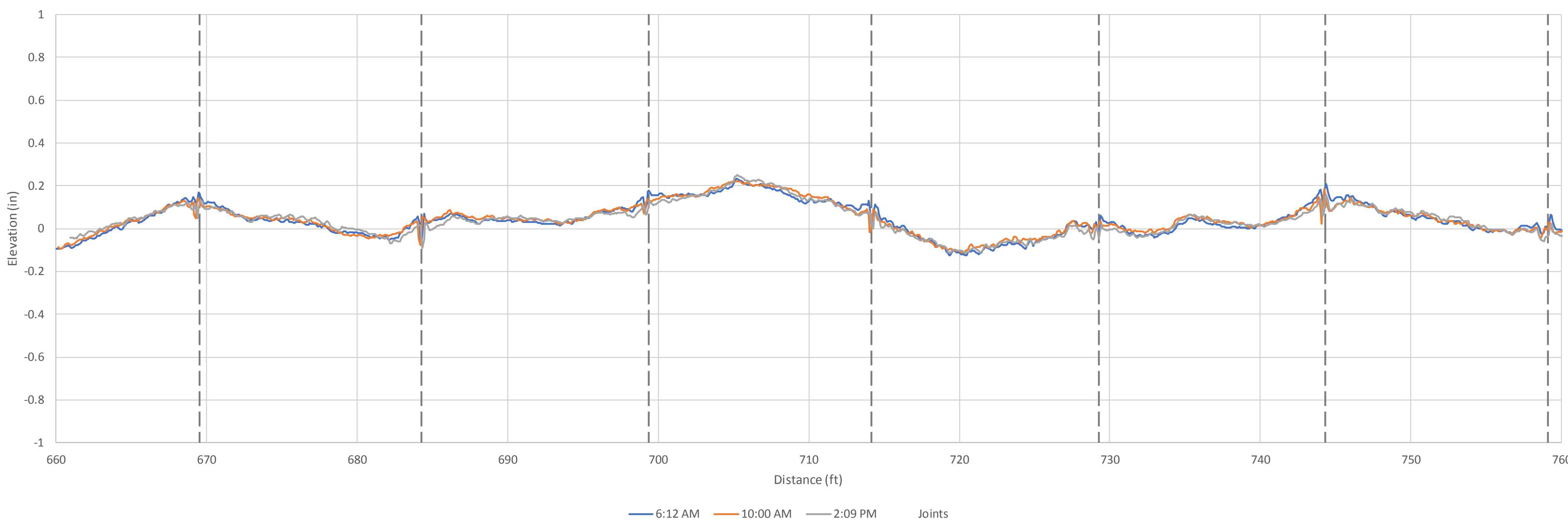
Elevation (in)
Distance (ft)
6:31 AM
10:18 AM
2:28 PM
Joints

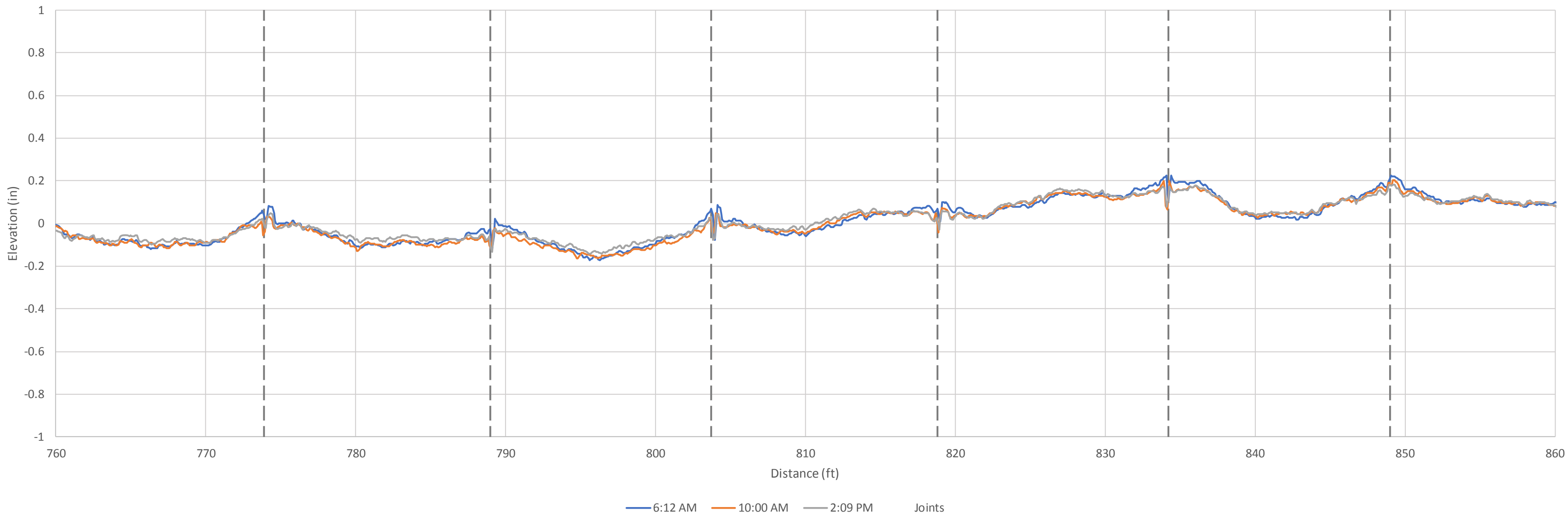


1.00E+00
1.00E-01
1.00E-02
1.00E-03
1.00E-04
1.00E-05
1.00E-06
1.00E-07
1.00E-08
1.00E-09
Elevation Spectral Density (ft²-ft/cycle)
0.1
1
10
100
1000
Wave Length (ft/cycle)
6:31 AM
10:18 AM
2:28 PM

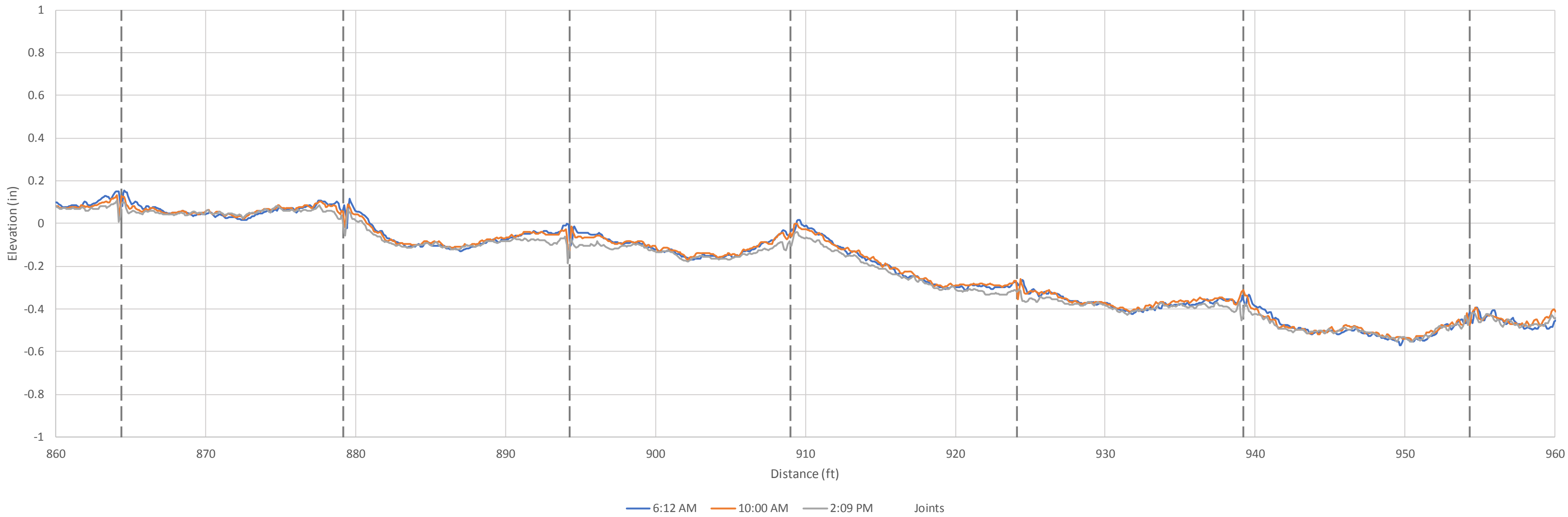
ATTACHMENT D: PROVAL PLOTS FOR TEST SECTION 080224


1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
660
710
760
810
860
910
960
1010
1060
1110
1160
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints

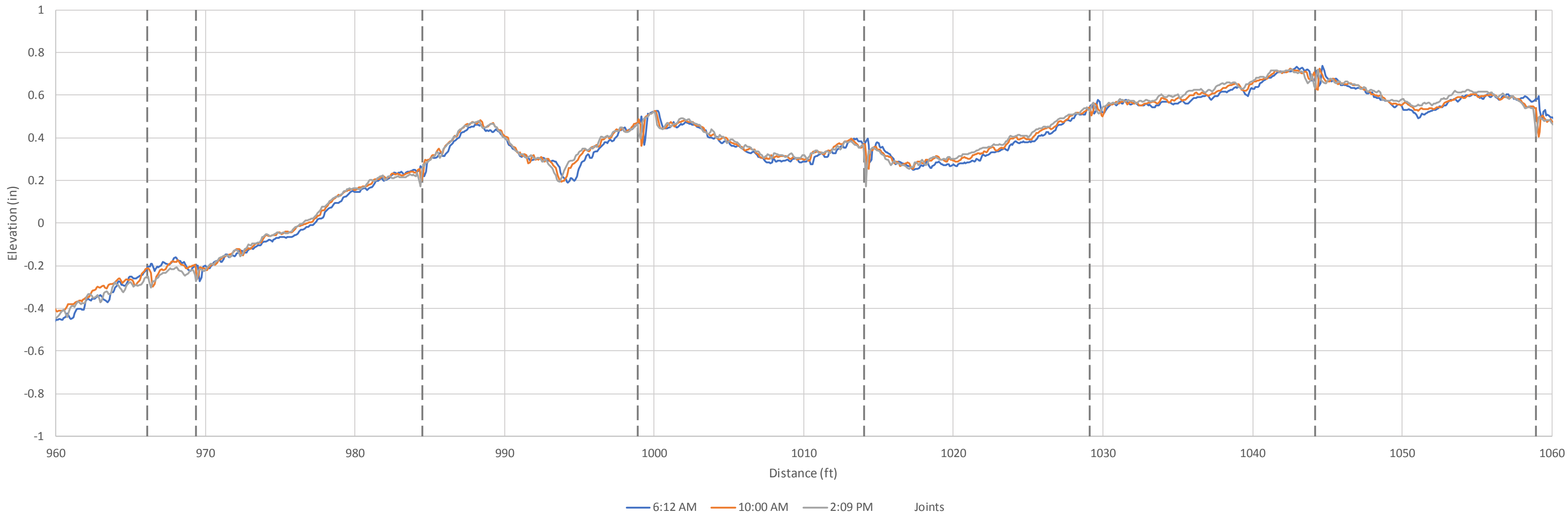

1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
660
670
680
690
700
710
720
730
740
750
760
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints

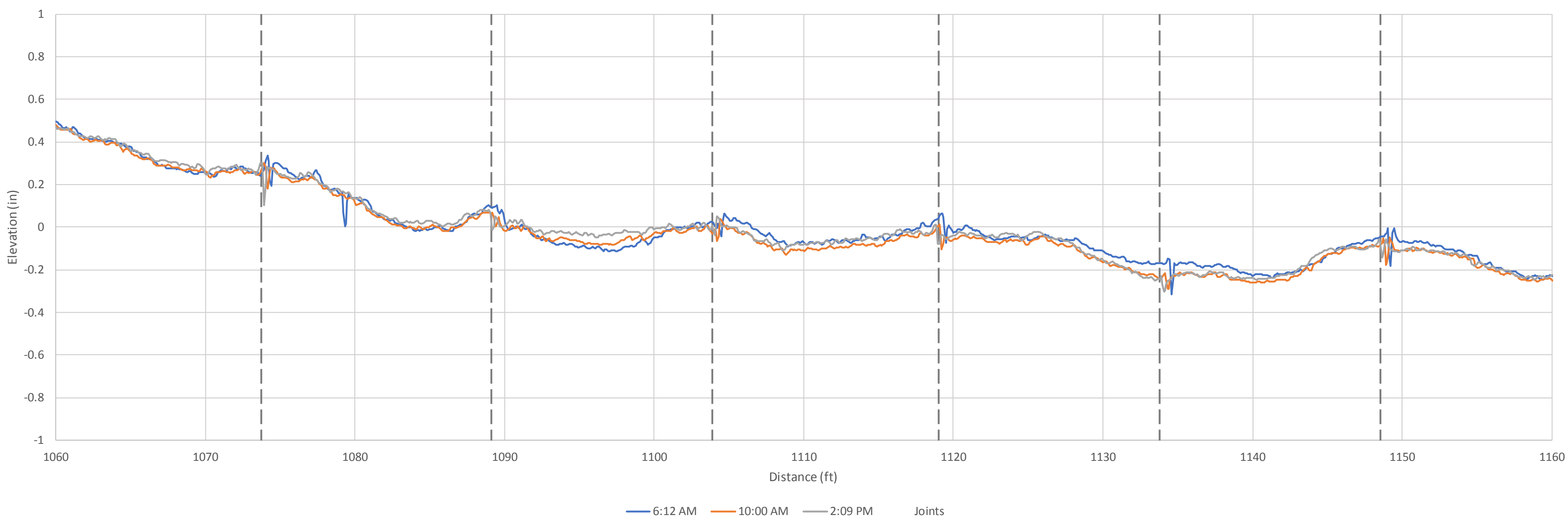


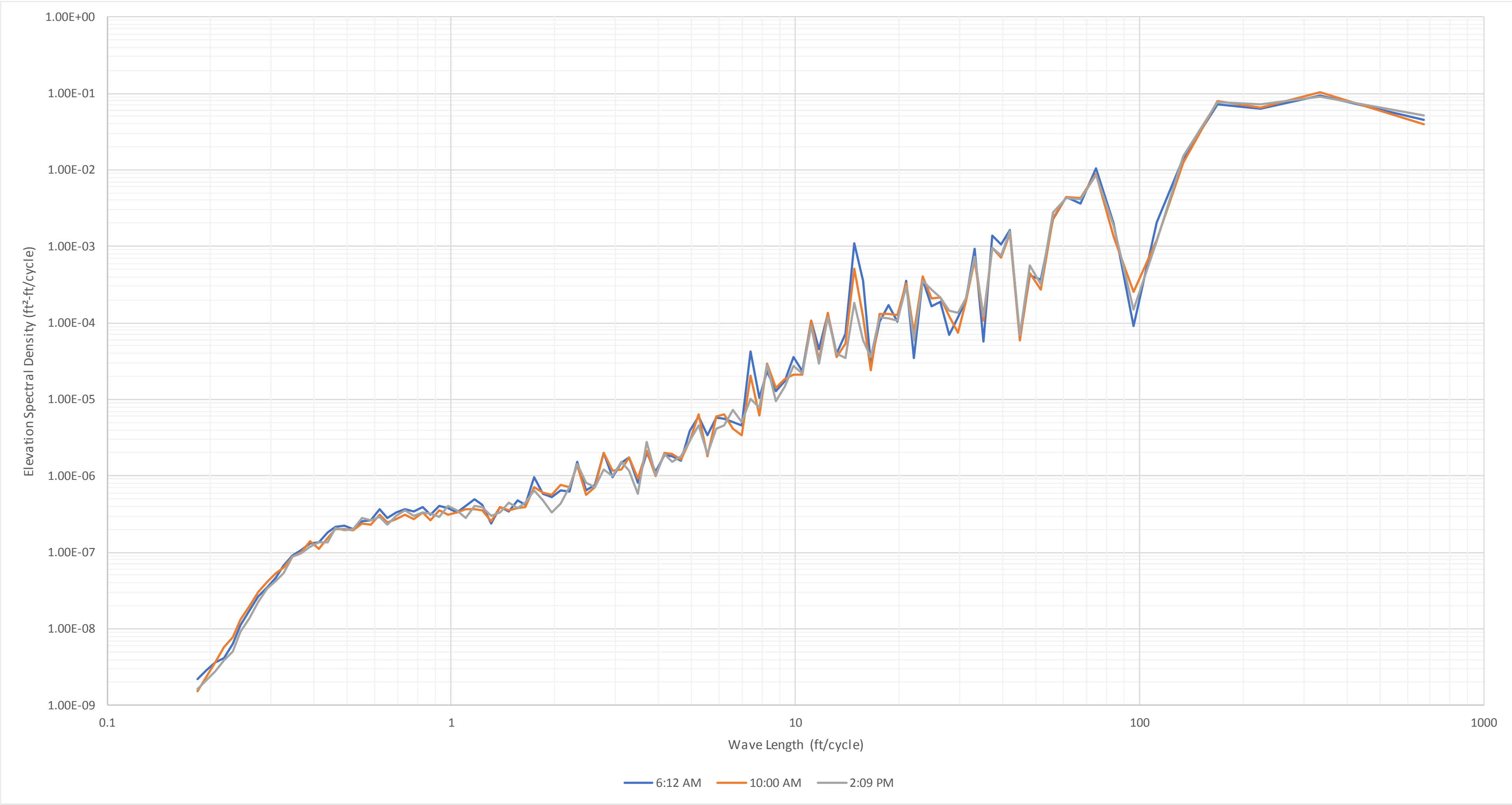
1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
760
770
780
790
800
810
820
830
840
850
860
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints



1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
860
870
880
890
900
910
920
930
940
950
960
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints


1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
960
970
980
990
1000
1010
1020
1030
1040
1050
1060
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints


1
0.8
0.6
0.4
0.2
0
-0.2
-0.4
-0.6
-0.8
-1
Elevation (in)
1060
1070
1080
1090
1100
1110
1120
1130
1140
1150
1160
Distance (ft)
6:12 AM
10:00 AM
2:09 PM
Joints


1.00E+00
1.00E-01
1.00E-02
1.00E-03
1.00E-04
1.00E-05
1.00E-06
1.00E-07
1.00E-08
1.00E-09
Elevation Spectral Density (ft²-ft/cycle)
0.1
1
10
100
1000
Wave Length (ft/cycle)
6:12 AM
10:00 AM
2:09 PM