Technical Memorandum

To: Kim Schofield

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Date: June 18, 2021

Re. Forensic Desktop Study Report: Oklahoma LTPP Test Section 40_4157

The Long-Term Pavement Performance GPS-3: Jointed Plain Concrete Pavement (JPCP) test section 40_4157[1] was nominated for a desktop study under TPF-5(332) "LTPP Forensic Evaluations." The pavement structure, which was originally constructed in 1986, consisted of 9.1 inches of JPCP and 3.8 inches of hot-mix asphalt concrete (AC) treated base over 42.0 inches of unbound silty sand subgrade soil. The last round of data collection was conducted in 2016, almost 30 years after construction, and at that time, the test section showed little, if any, distress, deflections remained low at around 3 mils, and faulting was under 0.1 inches. However, joint load transfer efficiency (LTE) appears to have been a problem at the test section. Except for values of ~90% in 1990 and 2003, the measured joint LTE remained in the 20% to 60% range until 2012, which is when joint load transfer restoration, surface diamond grinding and joint sealing were performed resulting in an improved joint LTE of ~80%. These treatments also resulted in a significant IRI improvement – IRI value decreased from 82 in/mi prior to treatments to 44 in/mi after the treatments – and faulting was reduced to zero inches. In light of the above, the objectives of the desktop study were to: (1) examine and identify those factors that have contributed to the excellent performance of the test section, (2) study the history of joint LTE of the test section prior to the application of the 2012 treatments and identify those factors contributing to the low joint LTE values, and (3) study the effects of the treatments applied in 2012 on the performance of the test section, with a particular focus on IRI and faulting.

SITE DESCRIPTIONS

LTPP test section 40_4157 is located on U.S. Route 69, northbound, in Mayes County, Oklahoma. U.S. Route 69 is a rural principal arterial with two lanes in the direction of traffic. The test section is classified as being in a Wet, No-Freeze climate zone. The coordinates (in degrees) of the site are (36.07647, -95.36436). Photograph 1 shows the section at Station 0+00 looking northbound in 2015, while Map 1 shows the geographical location of the test section.

[1] First two digits in test section number represent the State Code [40 = Oklahoma]. The final four digits are unique within each State/Province and were assigned at the time the test section was accepted into the LTPP program.

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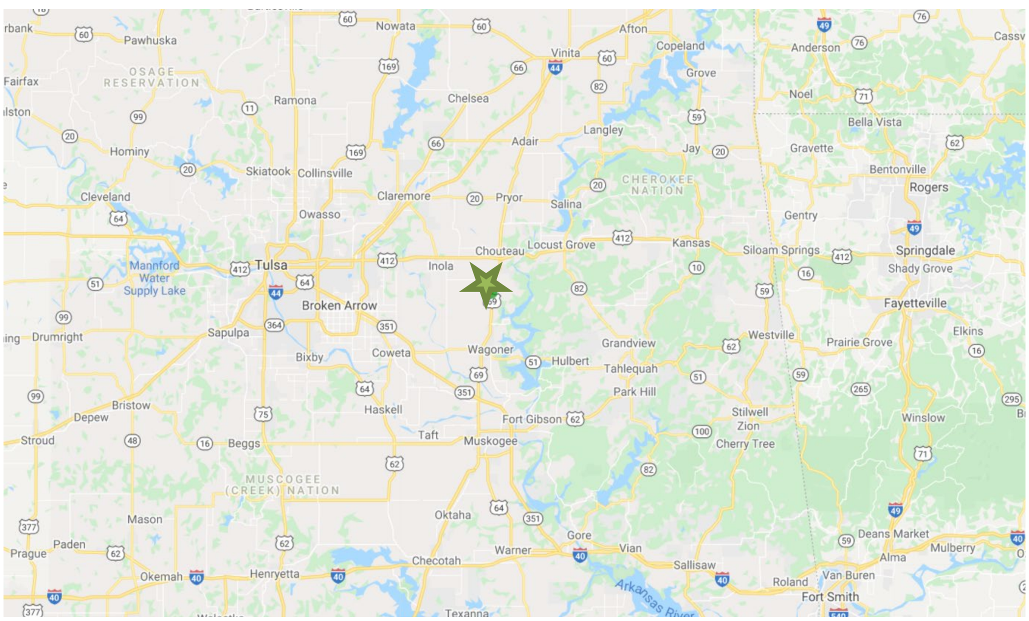
Photograph 1. LTPP Section 40_4157 at Station 0+00 looking northbound in 2015.



Map 1. Geographical location of test section.

BASELINE PAVEMENT HISTORY

This section of the document presents historical data on the pavement structure and its structural capacity, climate, traffic, and observed surface distresses.

Pavement Structure and Construction History

The test section was constructed in March 1986 and was accepted into the LTPP Program as part of the GPS-3 experiment in January 1987. The pavement structure at the time of its incorporation into the LTPP program consisted of 9.1 inches of JPCP and 3.8 inches of hot-mix AC treated base over 42.0 inches of unbound silty sand subgrade soil. This pavement structure is summarized in Table 1 and corresponds to CONSTRUCTION_NO = 1 (CN = 1) in the LTPP database. The original structure remained largely unchanged since the test section's original construction; however, the test section did receive joint load transfer restoration (CN=2), surface grind (CN=3), transverse joint sealing, and longitudinal joint sealing (CN=4) in September 2012. The surface grind resulted in the PCC surface layer being reduced from 9.1 inches to 8.9 inches, as shown in Table 2.

Table 1. Pavement structure for 40_4157 (CN=1)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	42	Coarse-Grained Soil: Silty Sand
2	Bound treated base	3.8	HMAC
3	Portland cement concrete layer	9.1	Portland Cement Concrete (JPCP)

Table 2. Pavement structure for 40_4157 (CN=3)

Layer Number	Layer Type	Thickness (in.)	Material Code Description
1	Subgrade (untreated)	42	Coarse-Grained Soil: Silty Sand
2	Bound treated base	3.8	HMAC
3	Portland cement concrete layer	8.9	Portland Cement Concrete (JPCP)

Pavement Structural Properties

Figure 1 shows the average FWD deflections under the nominal 9,000-pound load plate. The deflection of the sensor located in the center of the load plate is a general indication of the total "strength" or response of all layers in the pavement structure to a vertically applied load. As shown in Figure 1, the deflections reported ranged from 2.6 mils (December 1998) to 3.5 mils (May 1993) over the analysis period. The fluctuations in deflections were minimal over time and the overall deflections reported were low, which is consistent with a concrete pavement.

The layer moduli backcalculated from the deflection data were also assessed for the test section. The pavement structure for test section 40_1597 was modeled as 9.1 inches of PCC and 3.8 inches of asphalt-treated base over coarse subgrade and bedrock. The backcalculated moduli for the five collection dates between June 1990 and March 2012 are shown in Figure 2 through Figure 4. The assumed modulus for the bedrock layer was 500 ksi and therefore, was not plotted. The collection of FWD data in 2015 was performed after the completion of the LTPP contract to backcalculate moduli data; therefore, backcalculated moduli for this test date were also not included in the LTPP database.

The moduli reported for each layer ranged from 5,666 ksi to 7,414 ksi, 17 ksi to 29 ksi ,and 73 ksi to 722 ksi for the PCC, asphalt treated base, and subgrade respectively. The modulus values of the asphalt-treated base were also considered within the context of the average daily air temperature (based on MERRA data). As depicted in Figure 3, there appeared to be an inverse relationship between the backcalculated modulus values and the average daily air temperature; as the temperature increased, the backcalculated modulus tended to decrease. This is aligned with expectation—pavements tend to stiffen in cold temperatures leading to an increased modulus. The relationship between moisture (measured as the cumulative precipitation reported the 7 days leading up to the FWD collection dates) and the subgrade layer backcalculated moduli was also assessed. Figure 4 shows no clear relationship between the backcalculated moduli and the 7-day precipitation.

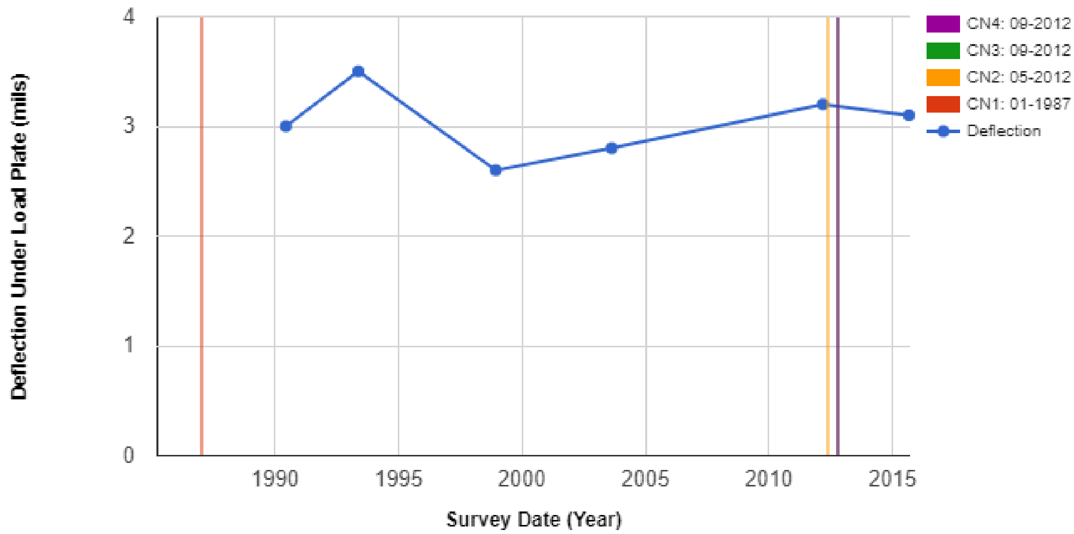


Figure 1. FWD deflections under the load plate over time.

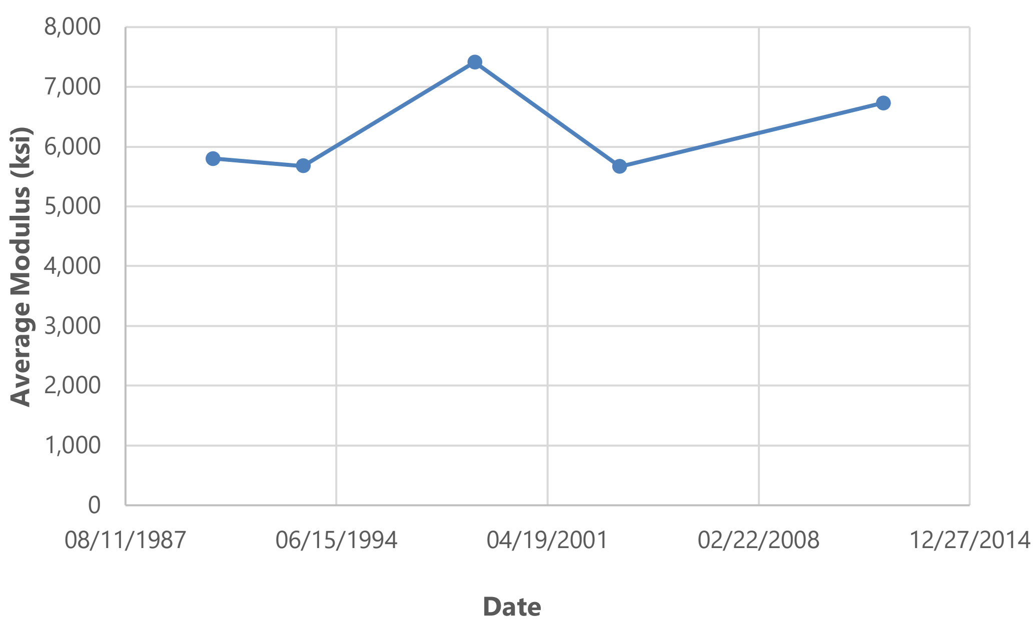


Figure 2. Average backcalculated modulus for PCC (Layer 1).

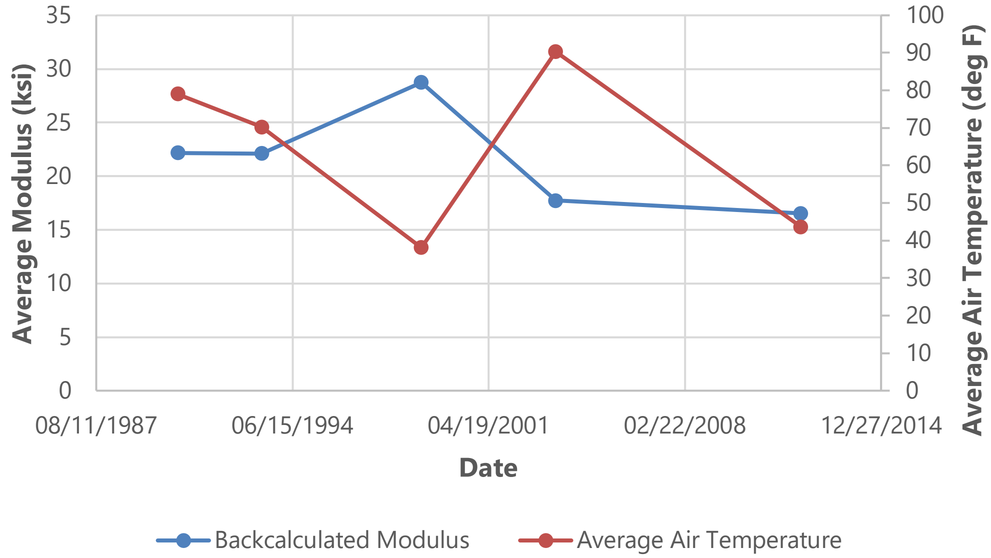


Figure 3. Average backcalculated modulus for asphalt-treated base layer (Layer 2).

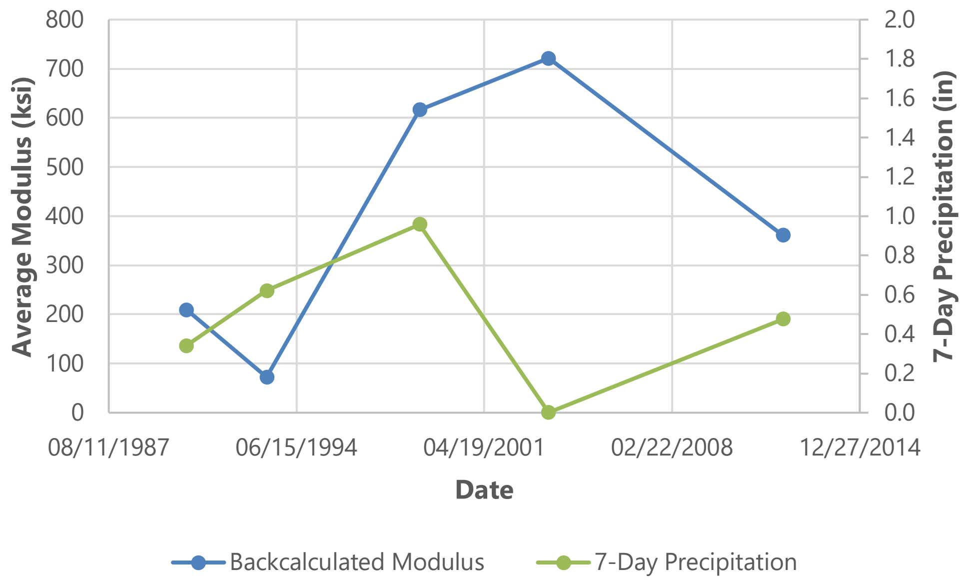


Figure 4. Average backcalculated modulus for the subgrade (Layer 3).

The reasonableness of the backcalculated layer moduli was compared to moduli derived from laboratory resilient modulus testing. On this project, only the subgrade and PCC layers were tested. Table 2 summarizes the laboratory test results. For the subgrade layer, various statistical analyses were conducted for the range of stress states (confining and deviatoric stresses) to which the laboratory samples were subjected. The laboratory values for the subgrade were significantly lower than the backcalculated modulus values. For the PCC layer, the elastic modulus was calculated using cores taken of the PCC layer. The range of values reported for the layer was lower than the backcalculated moduli (4,725 ksi vs. approximately 6,000 ksi). In addition, the compressive strength of the tested cores ranged from 7,660 psi to 8,590, and the split tensile strength ranged from 648 to 817 psi.

Table 2. Laboratory resilient modulus test results

Layer	Range of moduli values (ksi)
PCC	4,700 to 4,750 (Average of 4,725)
Subgrade	6.2 to 12 (Average of 9.3)

Load Transfer Efficiency

Prior to the dowel bar retrofit that occurred in 2012, the test section generally reported poor load transfer efficiencies. Table 3 shows the average LTE over time on both the approach and leave slabs. During the warmer months of the year, it is hypothesized that expansion of the slabs occurred, and the resulting locked joints provided sufficient load transfer of the joints. In colder months, the load transfer was quite poor, which typically results in faulting, pumping, and—if left untreated—corner breaks. The relationship between average daily air temperature (MERRA) and LTE is quite apparent in Figure 5; as the temperature increased, the load transfer efficiency increased. The only measurement after the dowel bar retrofit, in September 2015, showed a substantially improved load transfer, although this also aligned similarly with the temperature/LTE trend prior to the construction event.

Table 3. Load Transfer Efficiency for approach and leave.

Survey Date	Approach (%)	Leave (%)	Average Daily Air Temperature (deg F)
6/12/1990	88	92	79
5/17/1993	36	60	70.3
12/10/1998	24	21	38.1
8/21/2003	91	87	90.3
3/9/2012	23	26	43.7
9/2/2015	78	85	79

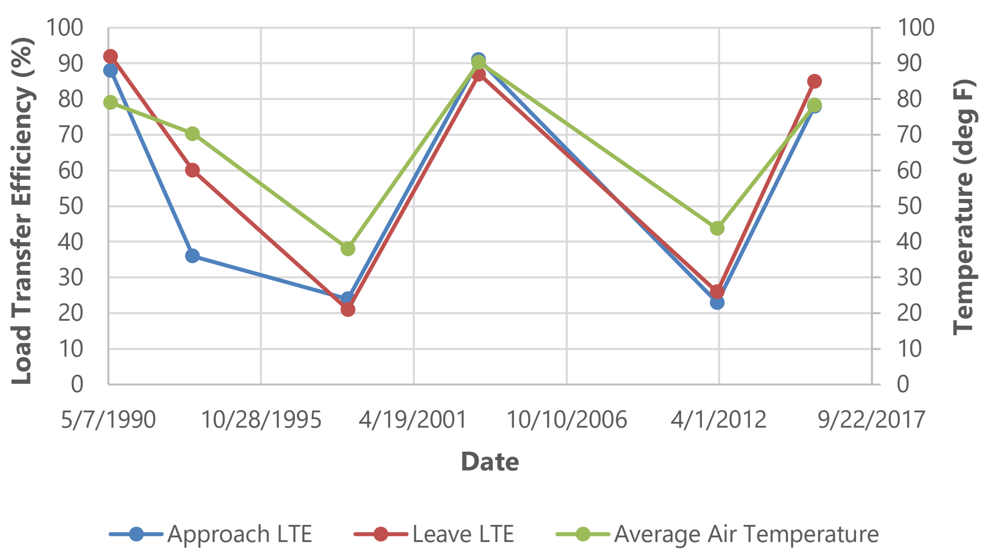


Figure 5. Load Transfer Efficiency over time.

Climate History

The time history for average annual precipitation (from MERRA) since 1986 is shown in Figure 6. As shown in the figure, between 1987, when the test section was incorporated into the LTPP program, and 2020, the average annual precipitation at the test section was essentially unchanged (48.2 inches in 1987 and 47.6 inches in 2020). However, the average annual precipitation observed from year-to-year, was more consistent prior to 2005. The average annual precipitation during this period ranged from 25 inches (2005) to 54 inches (1999), whereas after 2005, the average annual precipitation ranged from 24 inches (2012) to 72 inches (2015). Additionally, notable spikes in precipitation were observed in 1999, 2007, 2015, and 2019, when 54, 58, 72, and 66 inches of precipitation were reported, respectively. These notable spikes may be tied to extreme weather events such as flooding, and snow and ice storms reported during those years. Overall annual average precipitation is 44 inches and overall average annual temperature is 59.6 F

Figure 7 shows the time history of the average annual freezing index (from MERRA) for the test site. The freezing index is the summation of the difference between freezing temperature and the average air temperature when it is less than freezing over a year's time. This index is an indicator of the harshness of the winter season relative to issues such as ground frost and low temperature cracking in pavements. As depicted in Figure 7, the freezing index values ranged from 32 deg F deg days (1992) to 351 deg F deg days (1989) during the analysis period. As shown in the figure, the overall trend of the freezing index is decreasing over time, indicating the climate is getting warmer. This trend could have implications on the overall performance of the pavement over time. Overall annual average freeze index is 138 deg F deg days. This lack of freezing may contribute to the observed excellent performance of the test section.

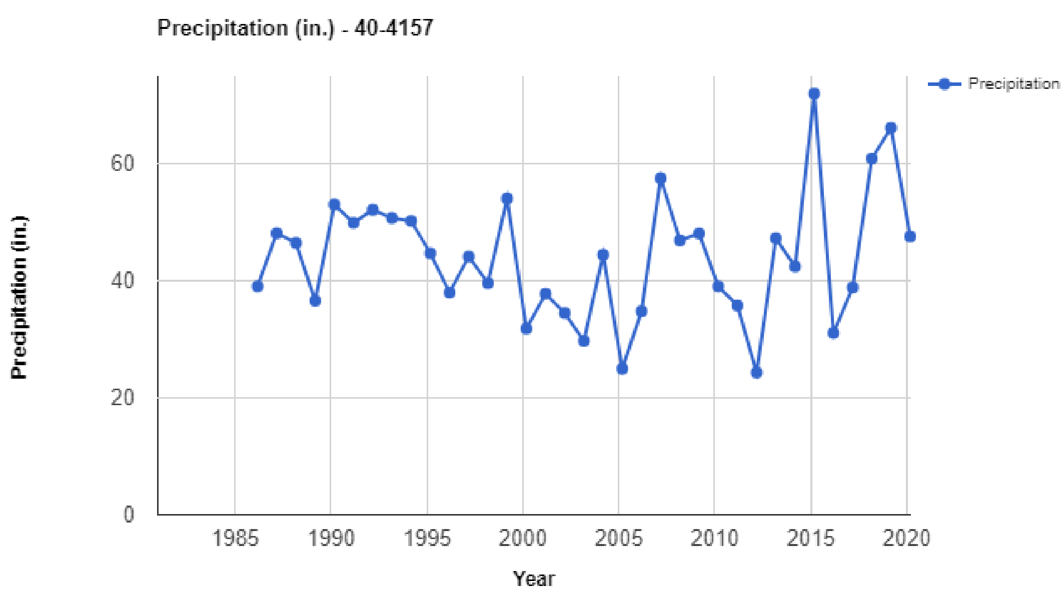


Figure 6. Average yearly precipitation over time.

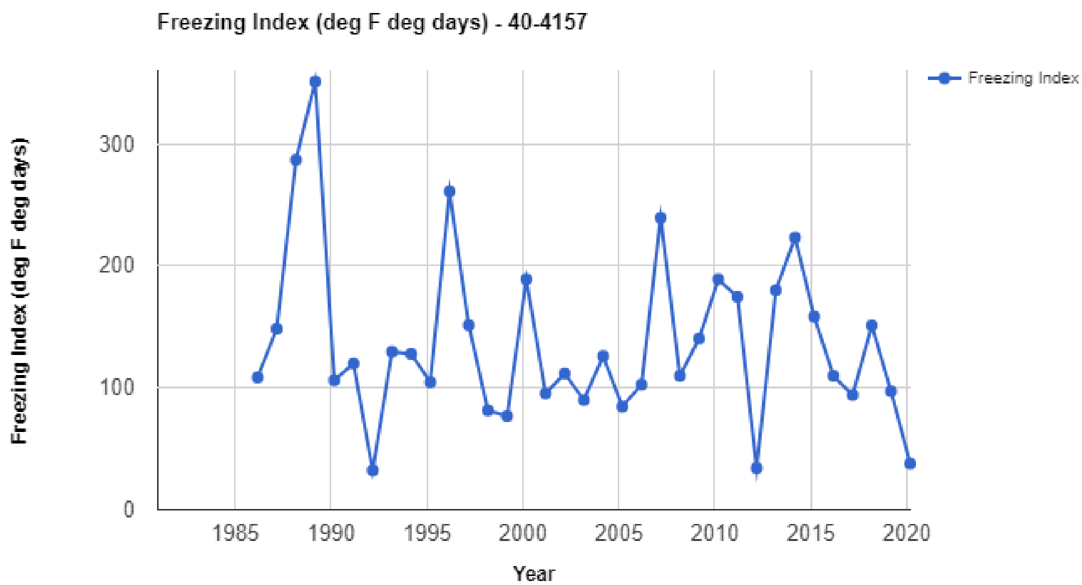


Figure 7. Average annual freezing index over time.

Truck Volume History

Figure 8 shows the annual average daily truck traffic (AADTT) data in the LTPP test lane by year. The AADTT increased between 1986 and 2017 from 1,040 to 1,888. The figure also shows the fluctuation in the reported traffic data which is likely a result of the source from which the data was gathered. The average number of ESALS reported on the test section followed a similar trend, increasing from 445,083 ESALs in 1986 to 985,442 in 2017 as depicted in Figure 9. Overall, the test section reported a high amount of truck traffic and ESALs over time.

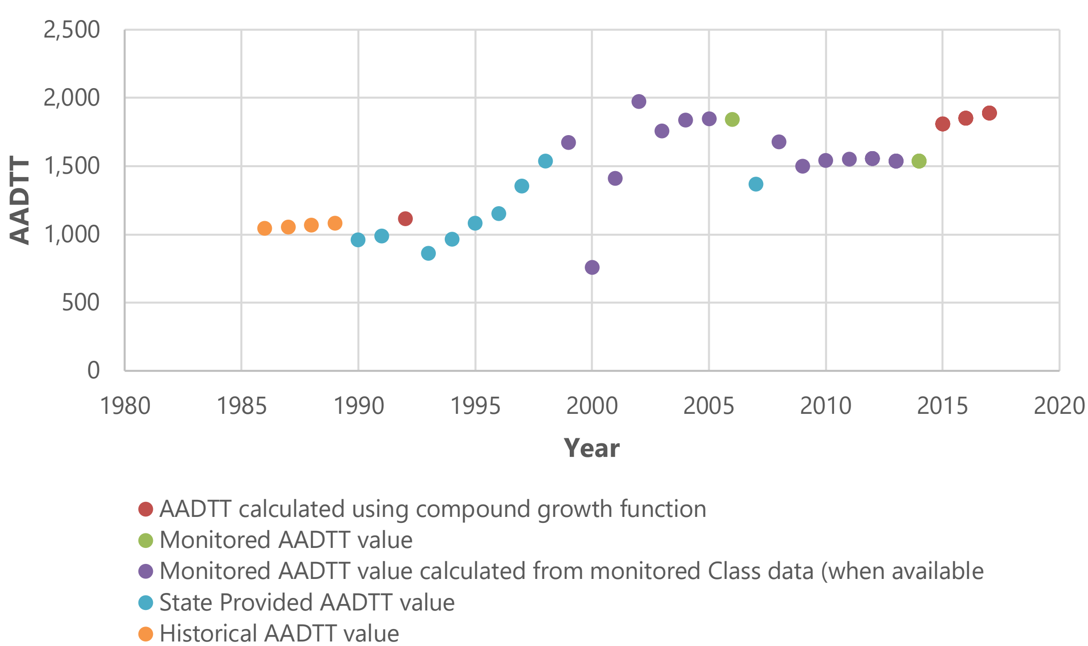


Figure 8. Average annual daily truck traffic (AADTT) history.

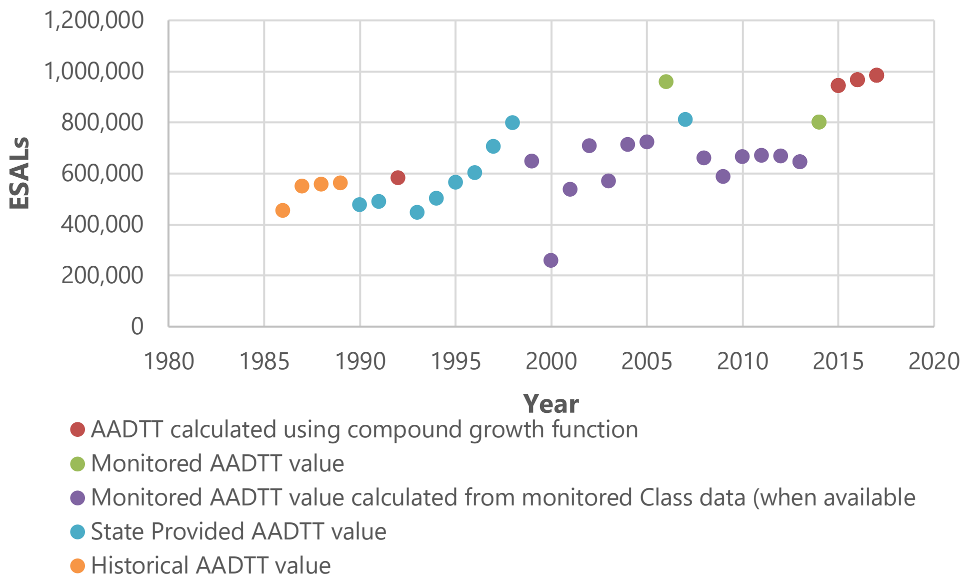


Figure 9. Estimated annual ESALs for vehicle classes 4-13 over time.

Through an investigation of the traffic at this test section, the project team identified an issue with the reported traffic plots being displayed as a part of the LTPP InfoPave™ Section Summary Report module. The plot produced, when compared with the data in the TRF_TREND table for the test section, was found to have to have been reporting the wrong traffic data. Upon further investigation, the issue was found to have been widespread and the traffic reported for other test sections were also found to be incorrect for the plot. The LTPP program has been notified about this issue with InfoPave™.

Pavement Distress History

The following summarizes the distresses observed on the test section between 1988 and monitoring in September 2015. Durability cracking (D cracking), joint seal damage, spalling, faulting, and IRI were assessed. No cracking, patching or corner breaks occurred during the monitoring period.

Durability Cracking

Figure 10 shows the total reported number and area of D cracking between 1991 and 2015. The D cracking did not manifest until after the 2009 manual distress survey. By 2012, 32.4 ft^2 of D cracking (11 cracks) was reported on the test section. Following the construction events in 2012, D cracking returned to zero, but was observed again in 2015 when 20.4 ft^2 (10 cracks) was reported. D cracking is typically associated with freeze and thaw damage to the aggregates within the PCC layer, and therefore it is not unexpected the cracking would resume following the construction events, given the pavement materials were unchanged following the milling and dowel bar retrofit.

Joint Seal Damage

Table 4 shows the joint seal condition on the test section (both transverse and longitudinal joints) over time. Per LTPP protocols, well-sealed transverse cracks are rated as having low level joint seal damage. The severity of the joint seal damage has to do with the length of the sealant that is effective. Low level transverse joints have less than 10% damaged, moderate severity transverse joints have 10-50% damaged, and high severity has greater than 50% damaged or missing. Between 1991-2000, the transverse joint condition on the test section deteriorated, and all joints were in poor condition (high severity) by 2003. Following the 2012 construction events, the joint sealant (for transverse joints) returned to good condition (low severity).

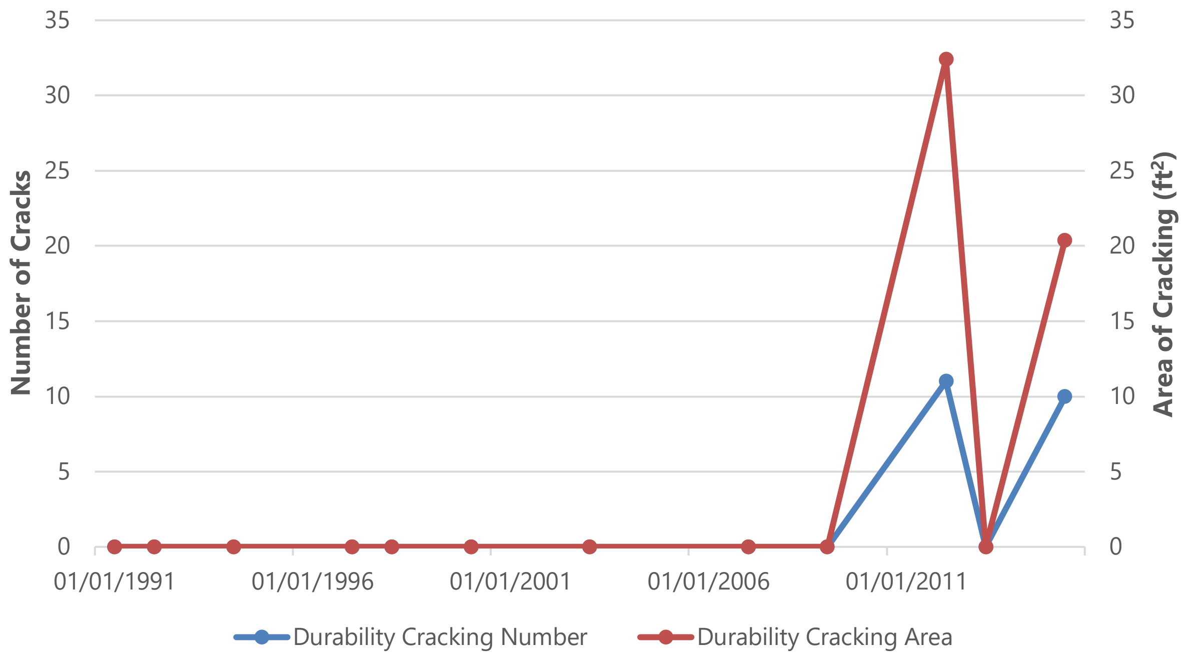


Figure 10. Durability cracking on the test section over time.

Table 4. Joint seal damage over time.

Survey Date	Transverse Joints Sealed—Low Severity	Transverse Joints Sealed—Medium Severity	Transverse Joints Sealed—High Severity	Longitudinal Joints Sealed	Longitudinal Joint Sealant Damage (ft)
10/09/1991	32	2	0	-	-
11/04/1992	8	26	0	1	0.0
11/01/1994	7	27	0	1	0.0
05/21/1997	7	21	6	1	0.0
12/10/1998	0	12	22	1	41.0
09/12/2000	0	8	26	1	292.6
08/21/2003	0	0	34	1	358.2
10/11/2007	0	0	34	1	500.2
11/19/2009	0	0	34	1	500.2
03/09/2012	0	0	34	1	500.2
03/20/2013	33	0	0	2	0.0
09/02/2015	33	0	0	1	4.6

For the longitudinal joints, aside from 2013, only one joint was rated as sealed. Only sealed joints were evaluated, and the total length of damage was recorded. The longitudinal joint on the test section was well sealed until 1998, at which point deterioration began. This continued until 2007, when the entire length of the section had longitudinal joint seal damage. Following the longitudinal joint sealing in 2012, the sealant condition improved for the entire length of the section, and only a minor amount of deterioration was recorded in 2015. It is not clear why two joints were recorded as being sealed in 2013, and only one was recorded as being sealed in 2015. The project team will submit a Data Analysis/Operations Feedback Report (DAOFR) to further investigate the data reported in 2015.

Spalling

Figure 11 shows the total length of spalling for both longitudinal and transverse joints. For longitudinal joints, all spalling is counted, while for transverse joints, only joints that are at least 10% spalled are counted as having spalling. With the exception of one transverse joint rated as having spalled in 2001 (which is likely due to rater variability), 2009 was the first instance of spalling being rated, and this occurred to a low degree both longitudinally and transversely. By 2007, over 60 feet of longitudinal spalling was present, with four transverse joints possessing sufficient spalling as to be rated (only 6.6 feet of transverse spalls in total). Spall repairs were not a part of the construction work in 2012, but the amount of spalling still reduced substantially for both longitudinal and transverse joints following CN=3. This is assumed to be related to the 0.2 inches of milling that was performed. Low severity spalling was removed or became more difficult to perceive following the milling. The reduction in longitudinal spalling between 2013 and 2015 is attributed to rater variability.

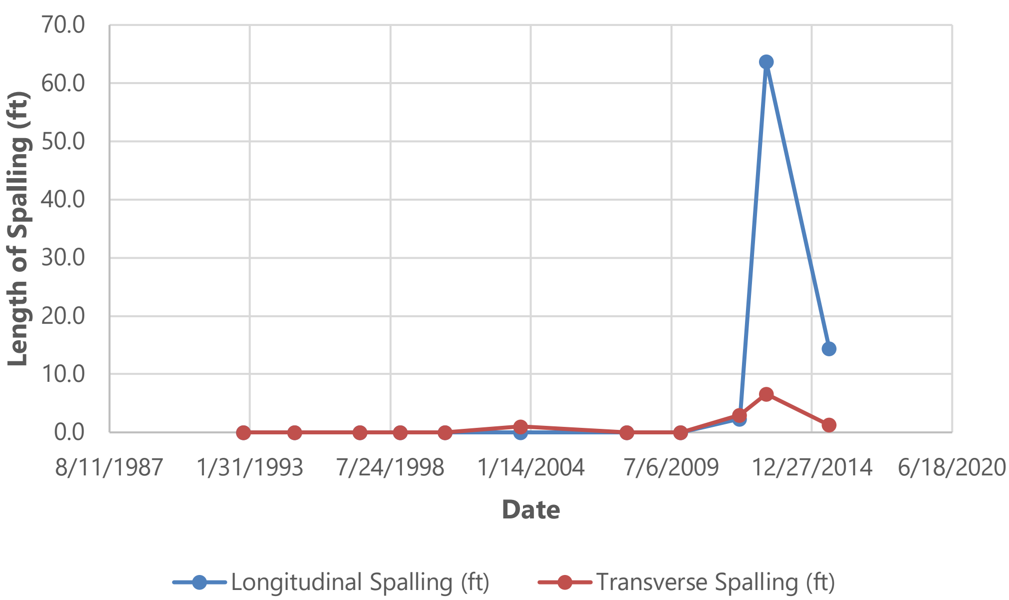


Figure 11. Length of spalling over time.

Faulting

Figure 12 shows the average amount of faulting present during each monitoring event. Per FHWA's Performance Measures, faulting less than 0.1 inches is categorized as "Good", 0.1-0.15 inches as "Fair" and faulting greater than 0.15 inches as "Poor." Faulting measurements on the test section were consistently less than 0.1 inches but were higher before the construction activities in 2012 than after. Following this work, the faulting on the section was essentially zero, which is to be expected with the combination of dowel bar retrofits and surface grinding.

IRI

The average IRI measurements for the section over time are shown in Figure 13. Again, based on FHWA's performance measures, IRI values below 95 inches/mile are categorized as "Good", between 95-170 in/mi as "Fair", and greater than 170 inches/mile as "Poor." While the roughness measurements were consistently in the "Good" range prior to the 2012 construction activities, following the dowel bar retrofit and surface grinding, the IRI dropped substantially (from around 80 in/mi to below 45 in/mi). Given the faulting present, as described above, approximately half of the measured roughness during CN=1 is likely attributed to faulting.

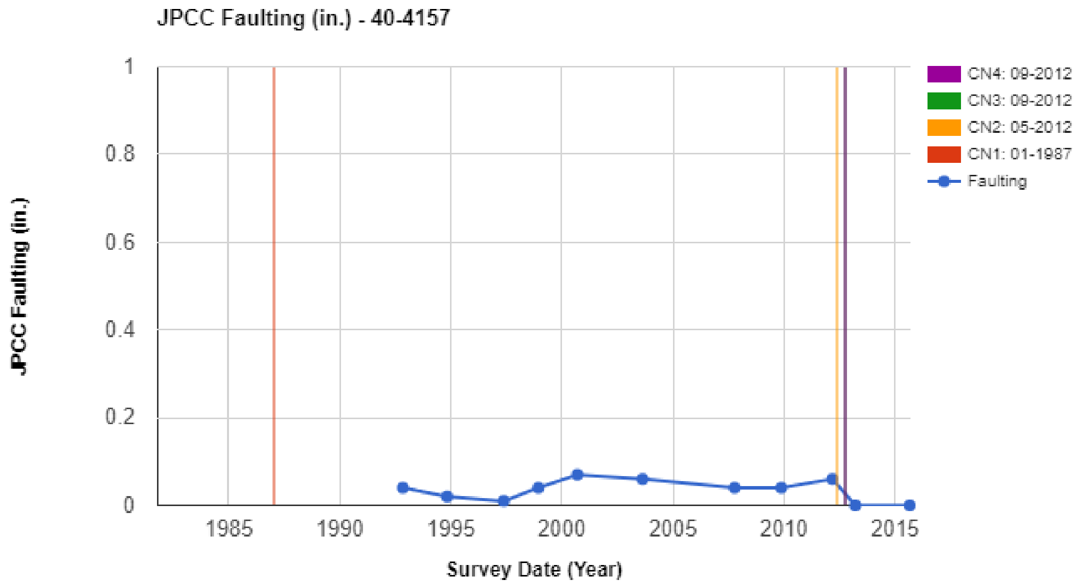


Figure 12. Faulting on test section over time.

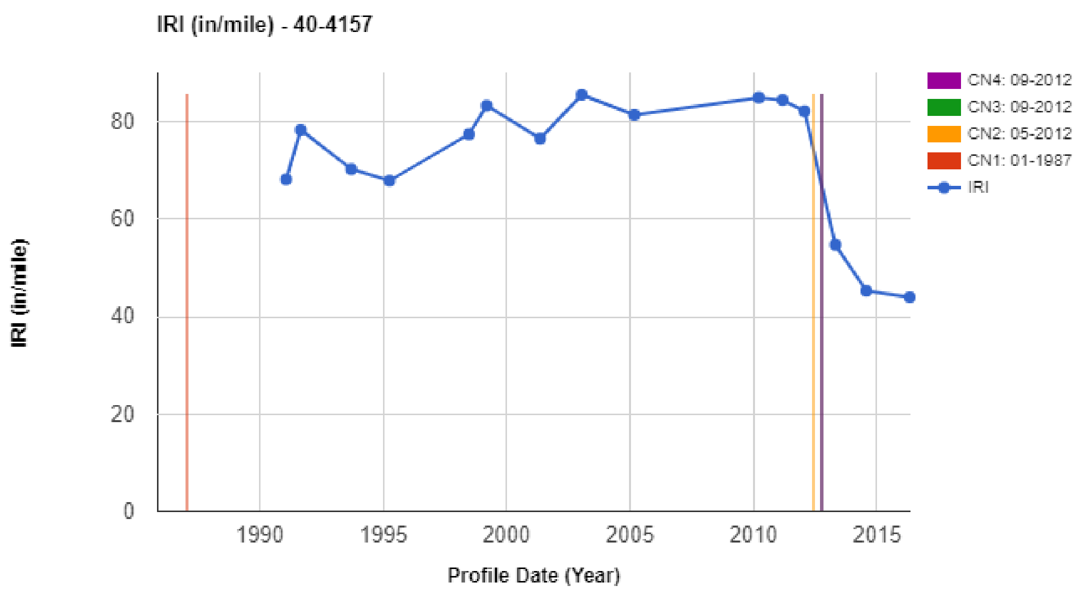


Figure 13. Time history plot of pavement roughness.

SUMMARY OF FINDINGS

LTPP test section 40_4157 is located on U.S. Route 69, northbound, in Mayes County, Oklahoma. U.S. Route 69 is a rural principal arterial with two lanes in the direction of traffic. The test section was constructed in 1986 and was accepted into the LTPP program as part of the GPS-3 experiment in January 1987. The test section is still active, with the most recent monitoring events taking place in either 2015 (FWD and distress) or 2016 (longitudinal profile). The pavement structure at the time of its incorporation into the LTPP program consisted of 9.1 inches of JPCP and 3.8 inches of hot-mix asphalt concrete (AC) treated base over 42 inches of unbound silty sand subgrade soil. The original structure remained largely unchanged since the test sections original construction; however, the test section did receive joint load transfer restoration, a surface grind, transverse joint sealing, and longitudinal joint sealing in September 2012. The surface grind resulted in the PCC surface layer being reduced from 9.1 inches to 8.9 inches.

The memorandum assessed the good performance of the test section over time. Specifically, it focused on:

1. **Examining and identifying factors contributing to the excellent performance of the test section**. Overall, the test section performed well in terms of distress (no cracking or patching), deflections, and faulting. The excellent performance of the test section is hypothesized to be attributed to the pavement design of the test section which includes a thick PCC layer over an asphaltic base layer, as well as the lack of freezing observed on the site. The excellent performance of the test section is aligned with findings from an early study of SPS-2 sections.[2] In the study, Jiang et al. (2005) found that sections with permeable asphalt treated bases (PATB) were often smoother, had low amounts of longitudinal cracking, a low percentage of cracked slabs, and low levels of faulting when compared to sections constructed on lean concrete base (LCB) or untreated base. Additional information on the predicted traffic loads and construction practices used at the test section would be helpful in further understanding the excellent performance of the test section.
2. **Studying the history of joint LTE of the test section prior to the application of the 2012 treatments and identifying those factors contributing to the low joint LTE values.** Prior to the dowel bar retrofit being applied in 2012, it was very common to have poor load transfer efficiencies on the test section. Based on an analysis of the average air temperature at the time of testing, the LTE reported appeared to be affected by the temperature. During the warmer part of the year, it is hypothesized expansion occurred and the resulting locked joints provided sufficient load transfer of the joints. Otherwise, the load transfer was quite poor, which typically results in faulting, pumping, and—if left untreated—corner breaks.
3. **Studying the effects of the of the treatments applied in 2012 on the performance of the test section, with a particular focus on IRI and faulting**. Faulting measurements on the test section were consistently less than 0.1 inches throughout the analysis period but were higher before the construction activities in 2012. Following this work, the faulting on the section was essentially zero, which is to be expected with the combination of dowel bar retrofits and surface grinding conducted. Similarly, the roughness measurements were consistently "Good" prior to the 2012 construction activities based on FHWA performance definitions. Following the dowel bar retrofit and surface grinding, the IRI dropped substantially (from around 80 in/mi to below 45 in/mi). It was hypothesized that approximately half of the measured roughness during CN=1 can be attributed to faulting.

FORENSIC EVALUATION RECOMMENDATIONS

The test section in question is part of LTPP's "Long Life" test sections, and therefore monitoring is intended to continue for as long as the test section remains in study. Because of this, destructive testing within the

[2] Jiang, J. and Darter, M. (2005*). Structural Factors of Jointed Plain Concrete Pavements: SPS-2—Initial Evaluation and Analysis. (Report No. FHWA-RD-01-167)*. Federal Highway Administration. https://rosap.ntl.bts.gov/view/dot/16454

test section is not recommended, but would be of interest once the test section has reached the end of its performance life. Future activities on this test section may include:

1. Coring to enable the following activities:
    a. Confirm that layer thicknesses match those reported when the test section was incorporated into the LTPP program.
    b. Identify whether any issues with bonding between the asphaltic base layer and PCC surface exist.
2. Performing additional FWD testing at the joints to track the effectiveness of the dowel bar retrofit with respect to load transfer efficiency.
3. Examining trends with regards to faulting, cracking, and roughness to assess the impact of the 2012 construction activities.
4. Conducting an interview with Oklahoma DOT staff familiar with the test section to gather additional information on the excellent performance of the test section.
5. Analyzing the performance of this test section compared with other LTPP JPCP test sections with similar ages, climatic regions, and traffic loading, particularly those within Oklahoma.