Field Performance Monitoring and Modeling of Instrumented Pavement on I-35 in McClain County

ANNUAL REPORT FOR FY 2011

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Submitted to:

John Bowman, P.E.
Planning and Research Division Engineer
Oklahoma Department of Transportation
200 N.E. 21st Street
Oklahoma City, Oklahoma 73105

Submitted by:

Musharraf Zaman, K.K. Muraleetharan,
Marc Breidy, Pranshoo Solanki
School of Civil Engineering and Environmental Science
The University of Oklahoma
202 W. Boyd Street
Norman, OK 73019

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1. Introduction

This combined laboratory and field study is conducted to better understand the mechanisms that cause pavement failure under actual traffic loading and environmental conditions. A 1,000-ft. long experimental pavement section was constructed on I-35 in McClain County and instrumented in collaboration with the National Center for Asphalt Technology (NCAT) and the Oklahoma Department of Transportation (ODOT) for field data collection. The field data collection is focused on pavement dynamic response data (e.g., distribution of stresses within the pavement structure, longitudinal and transverse strains at the bottom of the asphalt layer), environmental data (e.g., air temperature, variation of temperature within the pavement structure), traffic data (e.g., axle load, position, speed), and field performance data (e.g., fatigue cracking, rutting). From the field data, necessary correlations, namely rut and fatigue transfer functions, will be developed. From the laboratory data, rutting and fatigue cracking susceptibility will be analyzed to address the behavior of asphalt concrete mixes used in the construction of the test section. Activities performed in Fiscal Year 2011 included dynamic data collection from weekly visits to the test section, pavement performance data collection from quarterly field visits**,** analysis of dynamic and environmental data, analysis of the Falling Weight Deflectometer (FWD) data, development of fatigue transfer functions, and maintenance of the test section and instrumentation. An overview of these activities is given in the following sections.

2. Overview of Work Done

2.1 Field Rut Measurements

Three field trips and distress surveys were conducted during the reporting period (FY2011): November 22, 2010, February 14, 2011, and June 07, 2011 to address pavement distresses namely, rutting and fatigue cracking. During each survey, field testing was conducted at six stations, namely, Station No. 144, 235, 319, 540, 738 and 900, located at approximately 100-ft. intervals along the outer wheelpath. Rut data was collected across the wheel paths at each station using a Face Dipstick® using 6-in. moon-foot spacing. The rutting progressions in all the test sections are presented in Figure 1, where there are six rutting progression curves, each curve representing the rutting progression at a specific station. The first three points of each curve (pertaining to August 21, 2008, December 3, 2008 and January 8, 2009) present the highest rut depth measured with the straight edge/rut gauge combination method. The last nine points of each curve (from May 19, 2009 to June 07, 2011) present the highest rut values of the two wheelpaths measured with the Face Dipstick® using 6-in. moon-foot spacing. From Table 1, it can be seen that rut depths data, collected during the reporting period (FY 2011), has increased on all stations, except on Station No. 235. The rut increase varied between stations; on Station No. 144, 319 and 540 the increase was between 0.012-in. and 0.032-in., and on Station No. 738 and 900, rut increased more significantly, between 0.051-in. and 0.060-in. The highest recorded rut value is 0.678-in. (17.22 millimeter), recorded on November 22, 2010, and corresponding to Station No. 738. The highest recorded rut value corresponding to the last field trip conducted on June 07, 2011 is 0.663-in. (16.84 millimeter) and was recorded on Station No. 738. From these observations, in general, it can be concluded that the rut depths increased between August 10, 2010 and June 07, 2011, especially during warmer months. Similar type of rut behavior was observed in the AASHO road test (Finn. et al., 1977) and NCAT test track (Selvaraj, 2007). Finn et al. (1977) and Selvaraj (2007) reported visible increase in rut depth values during summer and fall months, but not in winter months. Thus, the observations from the present study are in agreement with those from the AASHO road test and the NCAT studies. Further discussions of field rut test results are presented in Hossain (2010).

Further, dynamic cone penetration (DCP) values were collected on the shoulder near the six stations on June 07, 2011. Before conducting DCP tests, approximately 15-in. deep hole was drilled to reach the surface of stabilized subgrade layer. The hole was drilled using HILTI TE 55 driller. Then, the DCP tests were performed down to a depth of between 20 – 25-in. The DCP profiles for all stations are summarized in Figure 2 in terms of incremental cone index (ICI), which represents the depth of penetration per blow of the DCP hammer (SHT, 1992). A lower ICI value indicates a stronger of stiffer material, while a higher ICI value indicates a weaker subgrade. From these plots several interesting observations are made.

1. The ICI values for all the stations showed higher stiffness (ICI< 15 mm/blow) for a depth of approximately 5-in. One of the explanations could be presence of approximately 8-in.

of stiff stabilized subgrade layer on the top of comparatively soft natural subgrade layer. This is also consistent with the higher FWD back-calculated modulus values obtained for stabilized subgrade soil as compared to natural subgrade.

2. For Station No. 738, the ICI values revealed significant increase at depth greater than 5-in. This can be attributed to higher moisture content of natural subgrade layer at Station No. 738. This observation is also consistent with higher rut depths obtained at Station No. 738, as discussed earlier. The higher moisture content values of natural subgrade at Station No. 738 might have attributed to higher rutting.

Additionally, soil samples were collected from the same drill holes used for collecting DCP data. These samples were used for determining in-situ moisture content and results are presented in Table 2. The lowest (12.2%) and highest (16.1%) moisture contents were recorded at Station No. 540 and 319, respectively.

2.2 Field Crack Mapping

Crack mapping was also performed during the distress survey for the entire test section. For the Station No. 144, 319, 540, 738 and 900, crack mapping was performed at 50-ft. both way of each station. To eliminate overlapping of mapping area, crack mapping was performed at 41-ft. north and 34-ft. south of Station No. 235. No crack is observed, so far, at any station, except along the construction joint and localized pot holes near the LPS sensors. Also, loss of aggregates (or raveling) was noticed on the pavement surface, as shown in Figures 3 (a) and (b). Further, Figures 4 (a) and (b) show a comparison of pavement surface condition at Station No. 144 in the form of photograph taken on June 05, 2009 and February 14, 2011, respectively. It is clear from Figures 4 (a) and (b) that the pavement has undergone noticeable deterioration along the edges (between driving lane and shoulder). Additional freeze-thaw cycles and precipitation are likely to cause formation of potholes, if cracks are not sealed.

2.3 FWD Analysis

A Dynatest model 8000 series (8002-057) type FWD was used in this study. The testing pattern was designed for a series of six stations located at approximately 100-ft. intervals along the outer wheel path. For conducting tests on the top of asphalt concrete layer, a plate of 11.8-in. diameter was used with seven deflection sensors spaced at 8-, 12-, 24-, 36-, 48-, and 72-in. from the center, as recommended by the ASTM D 4694 test method. The loading pattern included three seating drops plus one load drop from different heights in progressive order. The FWD testing was conducted on the top of asphalt concrete layer by including four different loads (6, 9, 12 and 15 kips). The collected data was analyzed for layer modulus values using MODULUS 6.0 software. The asphalt concrete modulus-temperature correlation obtained by collecting data up to October 28, 2009 is presented in Figure 5. The regression analysis on back-calculated data from FWD yielded an exponential best-fit line of the form presented in Equation (1) with α_1 = regression constant (18,841 ksi), α_2 = regression constant (–0.045), and T = mid-depth temperature of asphalt concrete from temperature sensors.

$$E = \alpha_1 e^{\alpha_2 T} \tag{1}$$

In general, Equation (1) is good predictor (R^2 = 0.863) of modulus value of asphalt concrete at different temperatures. At low temperature (50^{o}F) the average back-calculated modulus value is approximately 1,792 ksi with a 40% coefficient of variation. On the other hand, at higher temperature of approximately 105^{o}F the average back-calculated modulus value and coefficient of variation is approximately 131 ksi and 4%, respectively. Additionally, Figure 6 shows variation of asphalt concrete modulus values with temperature for all the FWD data collected until June 07, 2011. It is evident from Figure 6 that the modulus values back-calculated from FWD data collected before opening the lane for traffic is lower than the modulus values at corresponding temperature collected after opening the lane for traffic. For example, the average modulus value collected at a temperature of approximately 95^{o}F is 191 and 412 ksi before opening the lane for traffic (May, 2008) and after opening the lane for traffic on August 11, 2010. Since the test section had relatively high initial air voids (approximately 8% to 10%) in the asphalt concrete layers, it is expected that compaction of asphalt concrete layer took initially by compaction due to traffic and thus increased modulus values.

2.4 Development of International Roughness Index (IRI)

The IRI for the test section was developed using previous and recent collected IRI field data (up to June 7, 2011) using the Face Dipstick. The data are collected from Station No. 319, going 50-ft. north then 50-ft. south at three different locations: inner wheel path, outer wheelpath and mid-lane. The IRI results are presented in graphical and tabular form in Figure 7 and Table 3, respectively. Based on the graph presented in Figure 7, the average IRI value at the section started around 70s and increased until it reached mid-80s. In general, values are increasing with time, which means that the road surface is getting rougher. Based on the Federal Highway Administration (IRI between 60 to 94), the pavement at the test section is still considered in good conditions.

2.5 Development of Fatigue Transfer Functions

Fatigue transfer functions have been development by the OU research team, covering the first three years of the test section damage life (May 2008 to May 2011). First, the strain-temperature relationships used in the fatigue transfer function have been finalized for both steering and tandem axles (Figures 8 and 9). Also used in developing transfer functions, is the asphalt concrete modulus-temperature relationship developed from back-calculated FWD data (Figure 5).

The current state of practice for fatigue transfer functions, including AI MS-1, Shell Oil Design Guide and the MEPDG, is in the form of (Timm and Priest, 2006):

$$N_f = k_1\left(\frac{1}{\varepsilon_t}\right)^{k_2}\left(\frac{1}{E}\right)^{k_3} \qquad (2)$$

where:

N_f = Number of load cycles until fatigue failure

ε_t = Applied horizontal tensile strain (from strain-temperature relationship equation)

E = HMA mixture stiffness (from stiffness-temperature relationship equation)

k_1, k_2, k_3 = Regression constants

Based on the strain-temperature relationship, the modulus-temperature relationship, and on the observed surface performance (crack mapping), the fatigue transfer functions was established, for both steering and tandem axles:

For steering axles: $N_f = 0.8(\frac{1}{\varepsilon_t})^{4.0}(\frac{1}{E})^{1.0}$ (3)

For tandem axles: $N_f = 0.8(\frac{1}{\varepsilon_t})^{4.0}(\frac{1}{E})^{1.0}$ (4)

Both equations correspond to an assumed damage ratio of 0.2, since the pavement didn't fail and no cracks had been observed. Also note that the regression constants (k_1, k_2, k_3) used in Equations (3) and (4) are same. Figure 10 shows the accumulation of damage over time, from May 30, 2008 to May 31, 2011, for the I-35 test section. It is clear that the damage at the terminal date is equal to 0.2, as assumed. The red line represents a damage ratio of 1, where the curve will reach when the pavement fails in terms of fatigue cracking. Once the pavement start showing cracks and begin to deteriorates, the transfer functions will have to be recalibrated.

2.6 Meeting with ODOT

On November 29, 2010 and May 20, 2011, meetings were held with ODOT personnel at the Planning and Research Conference Room, ODOT. In these meetings an update related to the progress of I-35 project was presented. Further details are presented in Appendix A and B.

2.7 Problems and Maintenance

- The surface of the LPS was grouted, using a flexible material, during lane closure activities (November 22, 2010; February 14, 2011; June 07, 2011). The work was completed by ODOT and OU teams.

- Currently, five strain gauges (numbered: 1, 6, 9 and 11) are giving erroneous readings. This problem was first encountered on June 6, 2010.

- From April 15 through April 30 of 2011, a construction zone was located south of the test section, which created traffic congestion on the test section and WIM station, making it impossible to collect dynamic and traffic data.

3. Plan for Fiscal Year 2012

Overall, the project is on track. The FY 2012 activities will include the following:

a) Collection of dynamic data, environmental data, traffic data, and performance data, will continue.

b) Field performance testing and distress survey will be conducted periodically (at least quarterly).

c) Analysis of data for updating and calibrating ‘Rut transfer function’ and ‘Fatigue transfer function’.

d) Bi-annual meeting with ODOT personnel to discuss the progress of the project.

e) Maintenance of the instrumentation such as temperature probes, axle sensors, and strain gauges, as needed.

f) Monitoring of pavement distress including formation of pot holes that are likely with additional freeze-thaw cycles in winter 2012, particularly near/along the longitudinal joint between the driving lane and the shoulder.

g) Weekly visual observation and data collection to detect any possible changes in asphalt strain gauge readings that might be an indicator of fatigue crack initiation.

h) Comparison of observed and predicted fatigue behavior, in case of fatigue cracking.

i) Prediction and measurement of rut values for increased ESALs.

j) Capture increased rutting during the summer months of 2012.

k) Observation of rut profile and contribution of different layers to rutting from trenching.

l) Documentation of five-year data and field performance of the test section, and a summary of lessons learned that can be used in improved design of highway pavements in Oklahoma

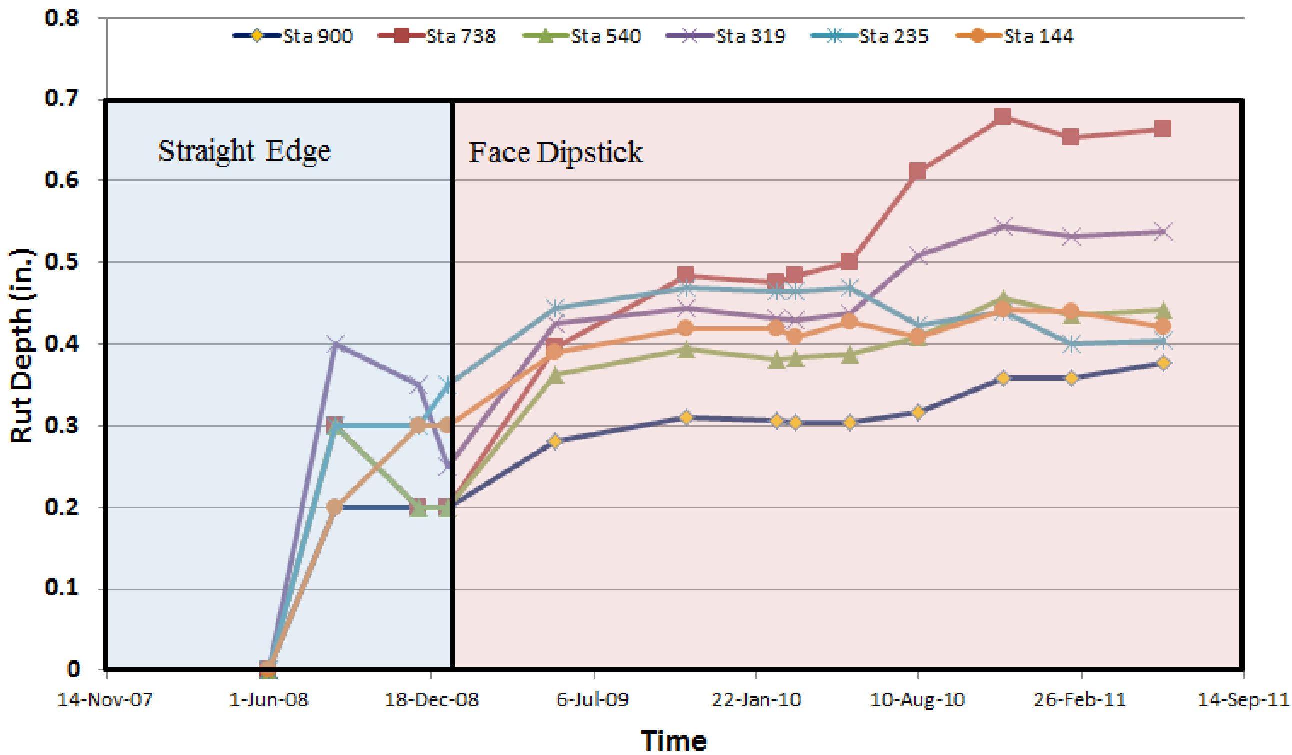

Figure 1: Rut Progression on the Test Section

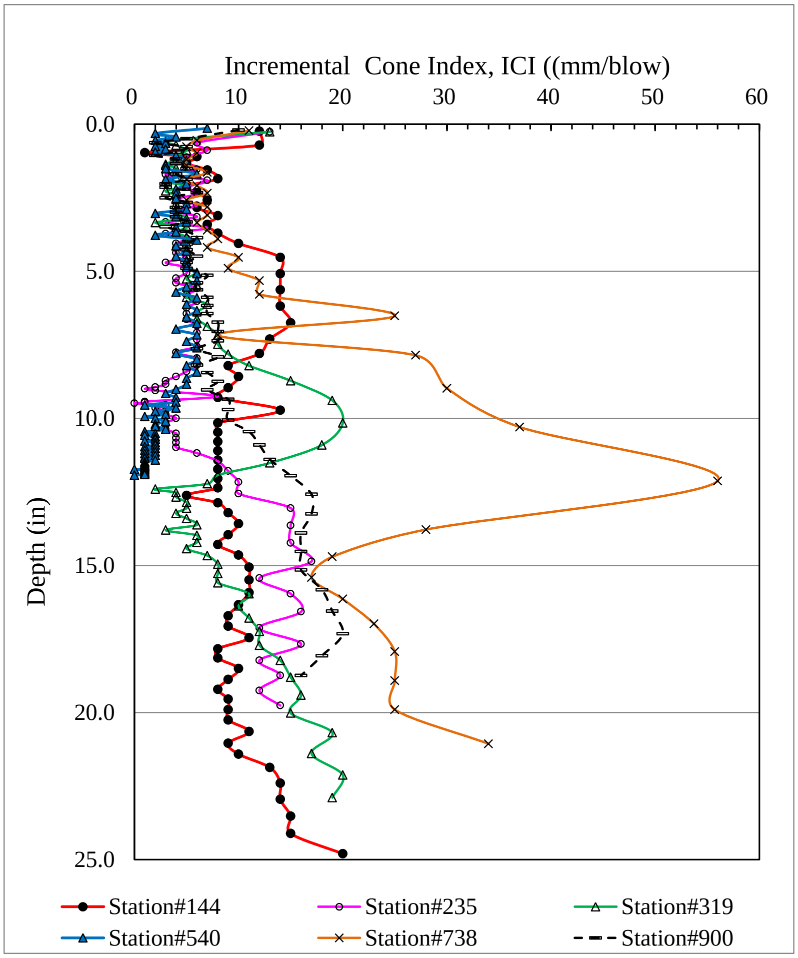

Figure 2: Summary of Dynamic Cone Penetrometer (DCP) Test Results (June 07, 2011)

(a) (b)

Figure 3: Photographic View of Loss of Aggregates from Pavement at a Distance of (a) 318 ft and (b) 741 ft from North End of the Test Section

(a) (b)

Figure 4: Photographic View of Pavement Surface at Station No. 144 taken on (a) June 05, 2009, and (b) February 14, 2011

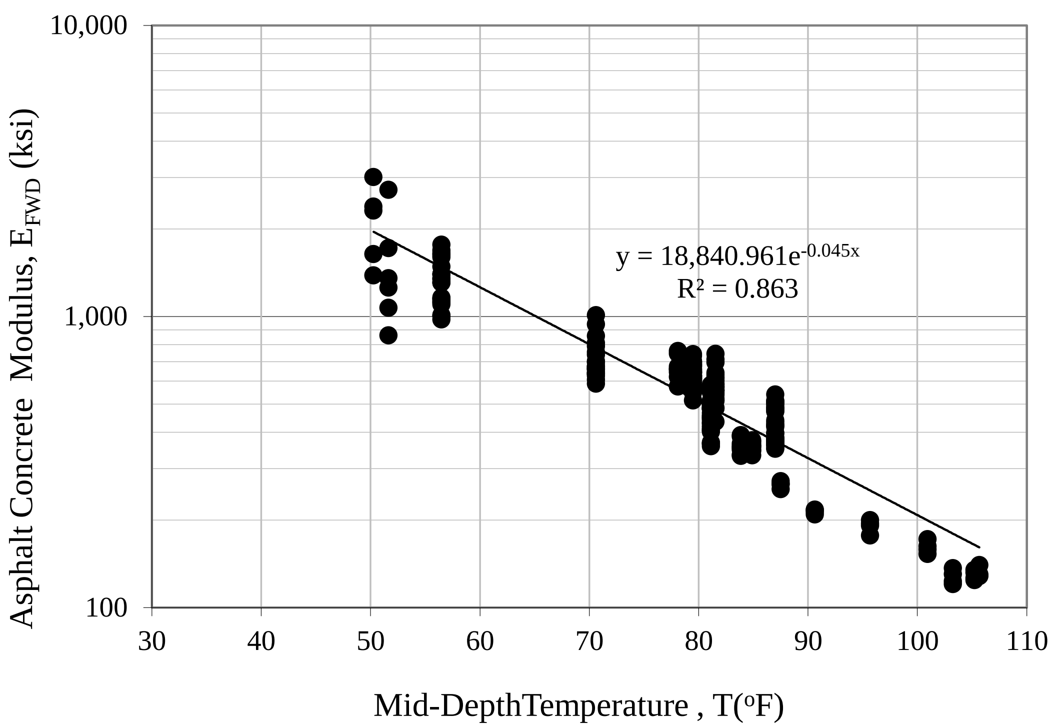

Figure 5: Asphalt Concrete Modulus-Temperature Relationship

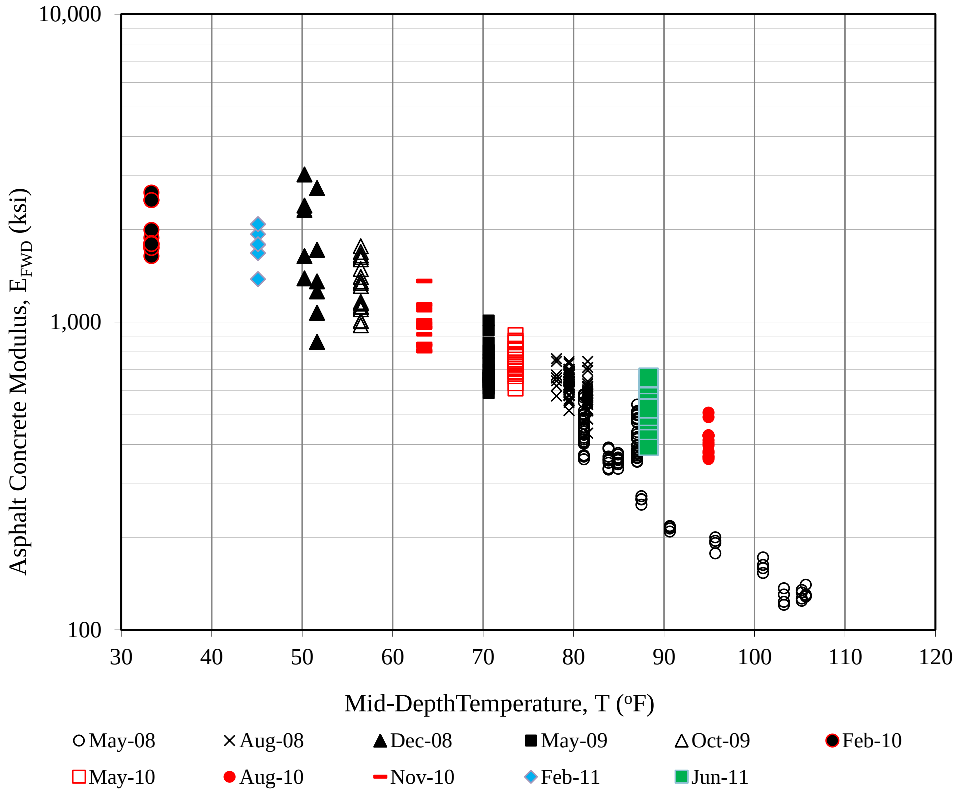

Figure 6: Variation of Asphalt Concrete Modulus with Mid-Depth Temperature (Last Data: June 07, 2011)

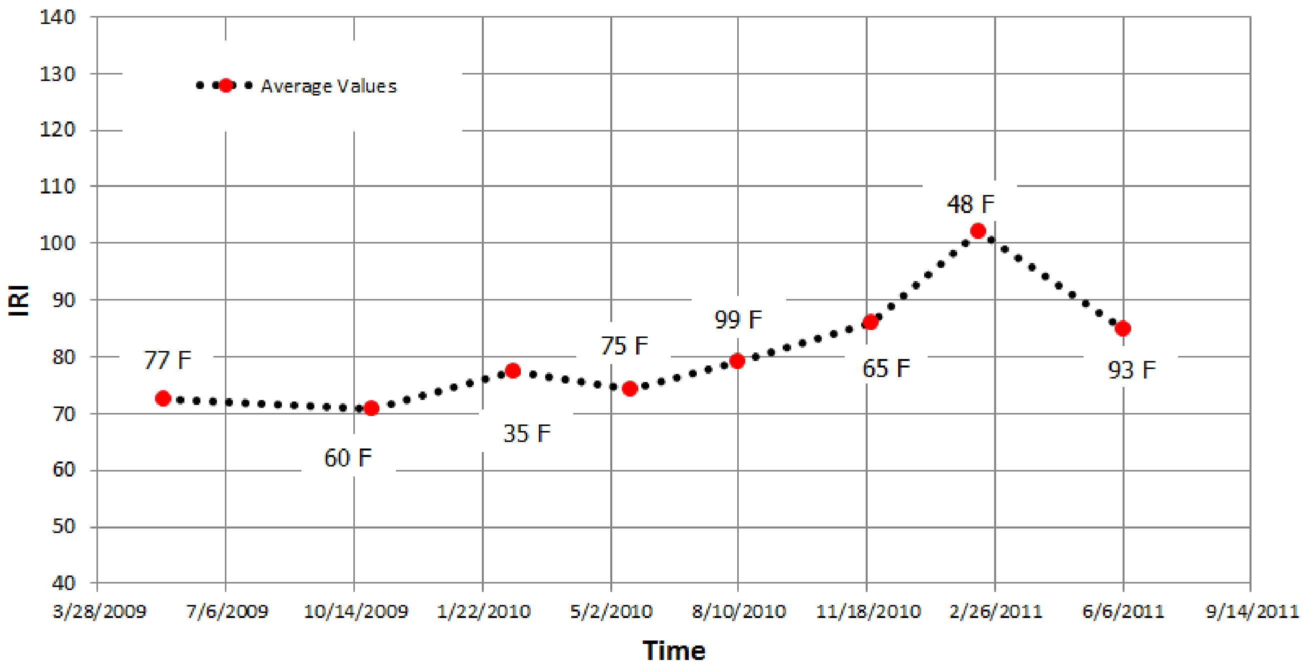

Figure 7: Average IRI Values for the Test Section

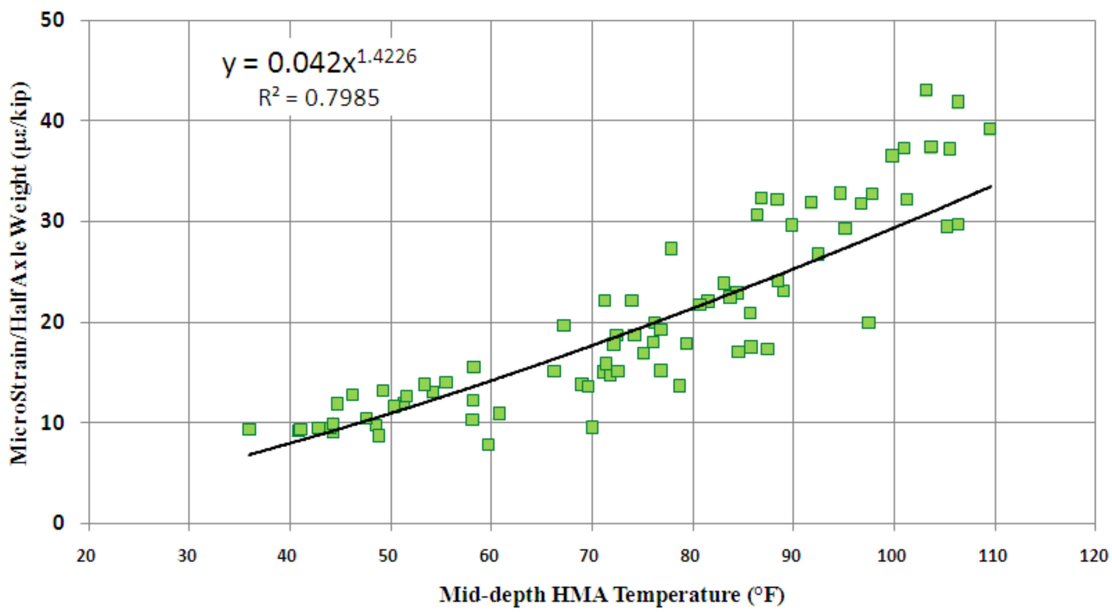

Figure 8: Strain-temperature Correlation for Single Tires (May 30, 2008 to May 27, 2011)

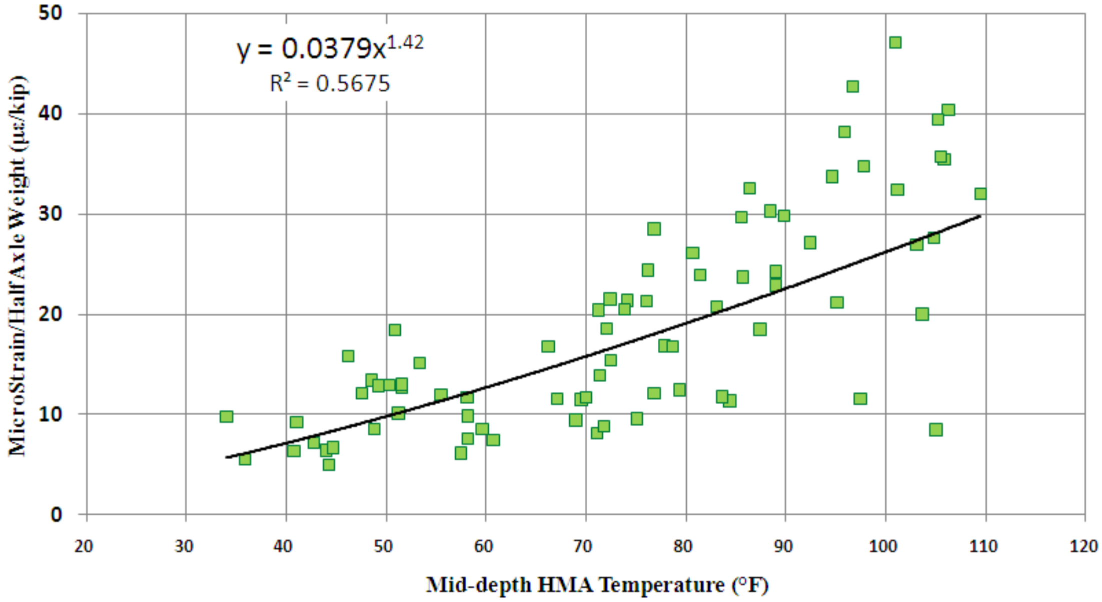

Figure 9: Strain-temperature Correlation for Dual Tires (May 30, 2008 to May 27, 2011)

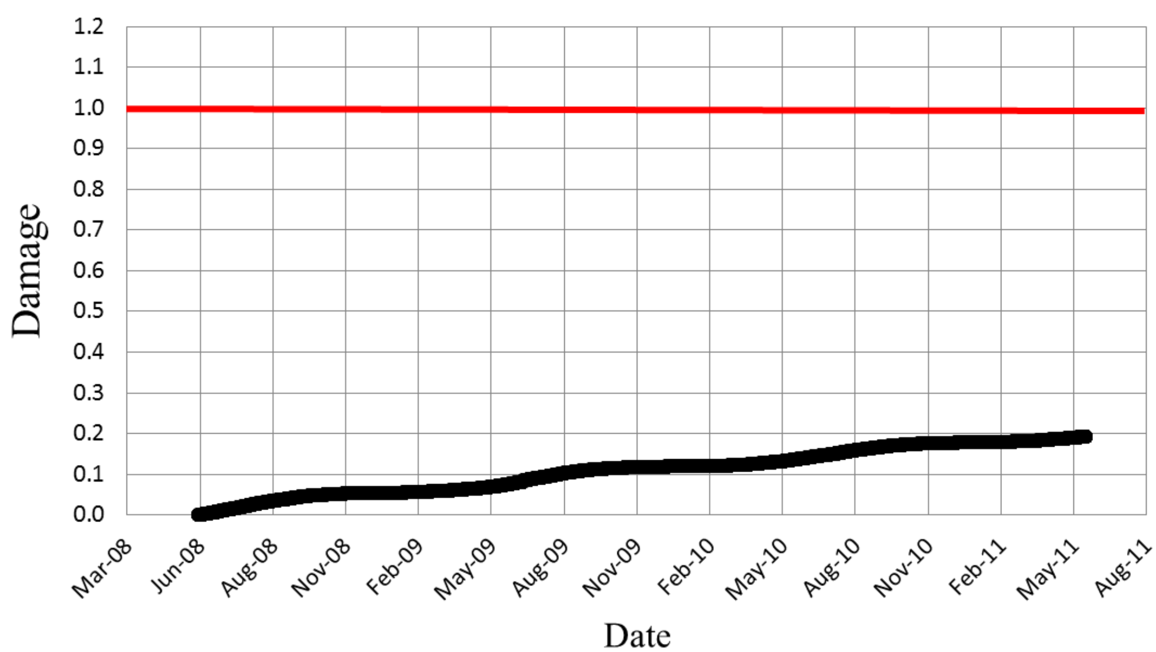

Figure 10: Damage Accumulation for the I-35 Test Section

Table 1: Tabular Form of the Rut Progression **(bold number denotes highest value recorded at that particular date of data collection)**

	Highest Rut (in.)					
Date	**Sta 144**	**Sta 235**	**Sta 319**	**Sta 540**	**Sta 738**	**Sta 900**
31-May-08	0	0	0	0	0	0
21-Aug-08	0.2	0.3	***0.4***	0.3	0.300	0.200
3-Dec-08	0.3	0.3	***0.35***	0.2	0.200	0.200
8-Jan-09	0.3	***0.35***	0.25	0.2	0.200	0.200
19-May-09	0.390	***0.444***	0.425	0.363	0.395	0.280
28-Oct-09	0.418	0.468	0.444	0.393	***0.483***	0.310
16-Feb-10	0.419	0.465	0.431	0.381	***0.476***	0.307
10-Mar-10	0.409	0.465	0.429	0.384	***0.483***	0.304
18-May-10	0.427	0.469	0.437	0.388	***0.501***	0.303
10-Aug-10	0.409	0.424	0.509	0.409	***0.612***	0.317
22-Nov-10	0.441	0.439	0.545	0.457	***0.678***	0.359
14-Feb-11	0.440	0.400	0.532	0.435	***0.653***	0.358
7-Jun-11	0.421	0.405	0.538	0.441	***0.663***	0.377

Table 2: Summary of In-Situ Moisture Content Results from DCP Drill Holes (June 07, 2011)

Station No.	Tin Weight gm	Tin+Wet Soil gm	Tin+Dry Soil gm	Moisture Content %	Average %
144	31.3	75.6	69.9	14.8	14.9
	31.3	69.6	64.6	15.0	
235	31.0	68.9	64.4	13.5	13.7
	30.9	69.5	64.8	13.9	
319	30.5	64.3	59.7	15.8	16.1
	31.3	68.2	63.0	16.4	
540	30.5	55.6	53.0	11.6	12.2
	30.6	54.4	51.7	12.8	
738	30.4	77.7	71.7	14.5	14.4
	31.1	80.5	74.3	14.4	
900	30.7	70.1	64.8	15.5	15.4
	30.8	77.1	71.0	15.2	

Table 3: Tabular Form of the IRI Values

	Outer Wheel Path		Mid-Lane		Inner Wheel Path	
Date	North	South	North	South	North	South
May 19 2009	63.5	67.66	102.9	48.53	73.01	78.91
Oct 28 2009	60.49	67.51	83.24	47.77	71.92	93.6
Feb 16 2010	62.79	75.73	81.67	66	74.17	103.79
May 18 2010	70.78	62.28	89.51	48.33	79.01	96.26
Aug 10 2010	69.33	70.14	124.2	57.68	75.71	78.61
Nov 22 2010	76.3	79.58	117.97	67.98	99.63	75.27
Feb 14 2011	78.1	77.99	124.49	137.2	86.02	109.12
Jun 07 2011	74.9	78.93	130.69	60.57	80.68	84.04

Appendix A

I-35 Project Meeting

Time: 10:00 a.m. – 11:00 a.m., November 29, 2010

Location: Planning and Research Division Conference Room, Oklahoma Department of Transportation

Attendees: Bryan Hurst, Jeff Dean, Chris Westlund, Bryan Cooper, Chris Clarke (ODOT); Musharraf Zaman, K. K. (Muralee) Muraleetharan, Pranshoo Solanki (OU)

1. An update related to the progress of I-35 project was presented.
2. Based on previous meetings discussion, the presentation was mainly focused into comparison of modulus values back-calculated from Falling Weight Deflectometer (FWD) and strain gauges.
3. Jeff suggested developing one new correlation by using the FWD data collected after October 28, 2009. The developed correlation should be again compared with the modulus back-calculated from strain gauges.
4. Jeff showed interest in studying the damage in the back-calculated modulus values of all the pavement layers since the opening of lane for traffic.
5. Collection of FWD data by orienting FWD machine in transverse direction was an important issue of discussion. Chris and Bryan suggested that this may be conducted by driving the van towards the shoulder area. The collected FWD data in transverse direction could be further used for comparing modulus values back-calculated from strain gauges and FWD data (longitudinal direction)
6. Jeff also mentioned use of temperature correction factor in modulus values back-calculated FWD data.

Appendix B

I-35 Project Meeting

Time: 10:00 a.m. – 11:00 a.m., May 20, 2011

Location: Planning and Research Division Conference Room, Oklahoma Department of Transportation

Attendees: Bryan Hurst, Jeff Dean, Chris Westlund, Chris Clarke, Ken Hobson (ODOT); Musharraf Zaman, K. K. (Muralee) Muraleetharan, Pranshoo Solanki, Marc Breidy (OU)

7. An update related to the progress of the I-35 project was presented.
8. Based on the discussion at the last meeting, the presentation was mainly focused on the responses/analyses to address the following issues raised in previous meeting: (a) to develop new correlations (modulus versus temperature) using the entire Falling Weight Deflectometer Data (FWD), (b) to use temperature correction factor in modulus values, and (c) to study the variation of back-calculated modulus values of all layers with time.
9. Jeff Dean showed concern regarding the back-calculated modulus values for stabilized subgrade. According to Jeff, values were higher than expected and compared to values reported in previous projects.

   Action Taken: At the end of meeting, Pranshoo Solanki discussed about this in more detail with Jeff. Pranshoo e-mailed excel file having processed stabilized subgrade modulus values to Jeff on May 30, 2011. Also, it is important to note back-calculated resilient modulus values are similar to the laboratory resilient modulus values reported in the previous reports (Solanki et al., 2009).

10. Jeff recommended conducting dynamic cone penetration test on the test section.

    Action Taken: Dynamic cone penetration tests will be conducted in the next field trip on June 07, 2011.

11. Jeff Dean showed interest in studying the rut performance of the test section using Mechanistic Empirical Pavement Design Guide (MEPDG) software and comparing with the actual rut values in the field.

12. Additionally, the method used for measuring rut values and difference between measurements taken by using Dipstick® and rut gauge/straight edge combination was discussed.
13. A good part of the discussion was focused on one-year extension of the project.

1. Finn, F., Saraf, C., Kulkarni, R., Nair, K., Smith, W., and Abdullah, A. (1977)., “The Use of Distress Prediction Subsystems for the Design of Pavement Structures,” Proceedings of the Fourth International Conference on the Structural Design of Asphalt Pavements, Ann Arbor, Michigan, pp. 3-37.

2. Hossain, N. (2010). “Observed and Predicted Rut Behavior of an Instrumented Test Section on I-35,” M.S. Thesis, University of Oklahoma, Norman, OK.

3. Selvaraj, S.I. (2007)., “Development of Flexible Pavement Rut Prediction Models from the NCAT Test Track Structural Study Sections Data,” Ph.D. Dissertation, Auburn University, Auburn, Alabama.

4. Saskatchewan Highways and Transportation (SHT) (1992), “Evaluation of Bearing Capacities of Subbase and Subgrade Using Dynamic Cone Penetration Test,” Standard Test Procedure Manual, STP 240-20,

5. Timm, D.H., and Priest, A.L. (2006). “Methodology and calibration of fatigue transfer functions for mechanistic-empirical flexible pavement design,” *Technical Report*, National Center for Asphalt Technology (NCAT), Auburn University.