Use Of An Automated Temperature Data Logger (ATDL) During Falling Weight Deflectometer (FWD) Testing

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Abstract

There are various methods of designing an overlay for a flexible pavement. A common method presently used involves analyzing FWD data to assess the structural capacity of a pavement section and using the backcalculated data to determine the overlay thickness.

Deflection results of all pavements are dependent on seasonal variations that affect the underlying aggregate and subgrade. The results from asphalt pavements are also dependent on the temperature of the asphalt. In order to meaningfully analyze the deflections or the deflection results, the deflection analysis results, must be adjusted to account for seasonal and temperature effects.

A piece of equipment that has recently been introduced into the Long Term Pavement Performance (LTPP) program is the Automated Temperature Data Logger (ATDL). The ATDL is designed to provide continuous automated data collection for pavement temperature gradient measurements associated with FWD deflection tests on LTPP test sections.

This report will examine the effect of using different sources of temperature gradient data and temperature models used to adjust FWD deflection results on overlay design for pavement sections in the LTPP program. All data will be sourced from the LTPP database and will use data from various states in the North Atlantic and North Central regions of North America. Furthermore, this report will discuss the impacts of the findings on pavement management and sustainability.

1 Introduction

1.1 Background

By definition, the term engineering involves the creative applications of scientific principles to design, construct, maintain and manage structures, machines, apparatus or processes. Pavement engineering is a subcategory of civil engineering and primarily focuses on pavements. Pavement engineers use techniques to design, construct, maintain, predict performance and manage road and pavement structures [Tighe 2012].

This paper focuses on evaluating the structural capacity of pavements. Structural capacity can be measured using a Benkelman Beam, Dynaflect or by using a Falling Weight Deflectometer (FWD). The FWD is an automated device used to rapidly and non-destructively measure pavement deflection. An impulse load which reasonably simulates traffic loading is applied to a spring loaded baseplate on the pavement surface. Deflections are measured at the center of the baseplate and at six to eight (depending on sensor configuration) other pre-determined radial points from the baseplate by geophones or deflection sensors. The deflection response of a pavement to an applied load is an important indicator of structural capacity, material properties and subsequent pavement performance.

The FWD was introduced in the 1980's and was gaining wide acceptance in North America during the 1990's. The FWD has now become the industry standard for collecting pavement deflection data. This was due to its non-destructive nature, portability (towed by a trailer) and speed of testing. The FWD can perform in excess of 30 tests per hour [Dynatest 2010].

The data used in this report is sourced from the Long-Term Pavement Performance (LTPP) program. "The Long–Term Pavement Performance (LTPP) program began in 1987 as part of SHRP" [FHWA 2010a]. The primary purpose was to establish a national long–term pavement database to support pavement research and improved pavement performance. When SHRP ended as planned in 1992, the LTPP program continued under the Federal Highway Administration (FHWA) with the participation of highway agencies in all 50 States, the District of Columbia, Puerto Rico, and 10 Canadian Provinces. Since 1989, the LTPP program has monitored nearly 2,500 pavement test sections throughout the United States and Canada" [FHWA 2010b].

A piece of equipment that has recently been introduced into the LTPP program is the Automated Temperature Data Logger (ATDL). The ATDL is designed to provide continuous automated data collection for pavement temperature gradient measurements associated with FWD deflection tests on LTPP test sections.

Pavement temperature gradients include up to five measurements in holes drilled in the bound layers of the pavement at nominal depths of 25, 50, 100, 200 and 300 mm. Holes are drilled in the center of the outer wheel path but must be offset at least 0.5 m from each other in the longitudinal direction [FHWA 2008].

1.0 Purpose

The purpose of this report is to assess the implementation of an ATDL to record pavement temperature gradients during collection of FWD data. This report will evaluate the differences in using temperature data from several sources during deflection testing. Furthermore, this paper will investigate the impact on the design of a pavement by comparing designs using temperature data from different sources.

1.1 Objectives

1. Review the current practice of temperature adjustment factors for deflection data.
2. Evaluate AC layer moduli for a pavement structure using different sources of temperature data and multiple methods of temperature correction in order to synthesize the various procedures currently in practice.
3. Investigate the effect on the design process.
4. Highlight differences in findings and discuss how pavement management and sustainability are affected.

1.1.1 Data Source

All data used for analysis can be found on the latest LTPP Standard Data Release (SDR) 26.0, which was released in January 2012 at the Transportation Research Board (TRB) Annual Conference in Washington, DC.

1.1.2 Analysis Software

The backcalculation software used for analyzing the asphalt concrete (AC) layer stiffness in the pavement structures is called Elmod 6.0 from Dynatest.

2 Data Review & Background

2.0 Temperature Data Collection

There are many different ways of obtaining temperature data while performing FWD tests:

1. Automated collection using FWD infrared sensors
2. Manual collection using a handheld infrared thermometer
3. Drilling holes into the bound layers of the pavement to perform manual measurements
4. Drilling holes into the bound layers of the pavement to use an ATDL

Industry most commonly uses Method 1, shown above. Since the LTPP program is a research project, data collection adheres to a much higher standard as quality data is a number one priority. The LTPP program uses all four of the above listed methods of obtaining temperature measurements.

Since data from all four sources is available, a comparison can be made using different methods. For the purpose of this report, Methods 1 and 4 were used for comparison. It is important to note that Method 1 only provides surface temperature, while Method 4 provides surface temperature and asphalt temperature at various depths within the layer.

2.1 Temperature Correction

Deflection results of all pavements are dependent on seasonal variations that affect the underlying aggregate and subgrade. The results from asphalt pavements are also dependent on the temperature of the asphalt. In order to meaningfully analyze the deflections or the deflection results the deflection analysis results, must be adjusted to account for the seasonal and temperature effects [FHWA 2000].

Over the years, a number of methods have been developed to measure the asphalt temperature and to adjust the deflection results for the effects of temperature. The Elmod backcalculation software has the flexibility to use three sources of data to correct temperatures:

1. Asphalt temperature measured at ⅓ or ½ the depth of the layer (can be manual or automated)
2. Surface temperature (can be manual or automated)
3. Average air temperature the day before testing (BELLS equation)

The BELLS equation was refined by the FHWA using temperature models created by using LTPP data from all of the experimental pavement sections across North America.

The version of the BELLS equation used in analysis by the Elmod software is a variation of the original equation called the BELLS2 equation [FHWA 2000] and can be expressed as the following:

$$T_{\mathrm{d}} = 2.9 + 0.935 \times IR + \{log_{10}(d) - 1.25\}\{-0.487 \times IR + 0.626 \times (1-day) + 3.29 \times sin(hr_{18} - 15.5)\} + 0.037 \times IR \times sin(hr_{18} - 13.5)$$

where:

T_d	=	Pavement temperature at depth d, °C
IR	=	Infrared surface temperature, °C
log	=	Base 10 logarithm
d	=	Depth at which mat temperature is to be predicted, mm
1-day	=	Average air temperature the day before testing
sin	=	Sine function on an 18-hr clock system, with 2π radians equal to one 18-hr cycle
hr_{18}	=	Time of day, in 24-hr clock system, but calculated using an 18-hr asphalt concrete (AC) temperature rise- and fall-time cycle

2.2 Test Section Details

For this report, three different experimental test sections were analyzed. The first section, 510115, is located on the SR-29 Expressway near Danville, Virginia. The second pavement section, 340506, is located on the Westbound lanes of I-195 in Upper Freehold, New Jersey. The third section, 447401, is located on the Northbound lanes of the N Smithfield Expressway near Providence, Rhode Island [FHWA 2010c].

2.2.1 510115 – SR-29 near Danville, Virginia

Section 510115 is part of the SPS-1 project. The SPS-1 experiment is a study of the influence of strategic factors that affect performance of asphalt concrete pavement [FHWA 2011]. Section 510115 is 1 of 13 individual 152.4m long, 3.7m wide pavement sections on SR-29.

Construction was completed in September of 1995 and the section is performing well with a minimal amount of maintenance activity. The only maintenance done on this section has been some spot patching and was done in October of 2004 [FHWA 2010c].

Section 510115 has had 7 FWD testing visits. This report will analyze one of those visits dated July 15th, 2010. Only one visit was analyzed because the ATDL was first implemented in 2009. The previous visit was in 2004, well before implementation of the ATDL equipment. This pavement section was also recently overlaid in the summer of 2011. The new layer thickness data is not yet available from the recent overlay, but it would be interesting to compare to the data analyzed as part of this report [FHWA 2010c].

The layer structure of Section 510115 is as follows:

163mm AC

219mm HMAC Treated Base

152mm Cement Treated Subbase

Fine-Grained Clay with Gravel Subgrade

Figure 1: Pavement Structure – Section 510115 [FHWA 2010c]

2.2.2 340506 – I-195 near Upper Freehold, New Jersey

Section 340506 is part of the SPS-5 project. The SPS-5 experiment is a study of the rehabilitation of an asphalt concrete pavement which is in fair condition [FHWA 2011]. Section 340506 is 1 of 11 individual 152.4m long, 3.7m wide pavement sections on I-195.

Construction was completed in August of 1994 and the section is performing well and has not required any maintenance up to the recent FWD testing visit in March of 2010. This section has recently been overlaid in the summer of 2010 [FHWA 2010c].

Section 340506 has had 17 visits where FWD testing was conducted. This report will analyze two of those visits dated April 25th, 2009 and March 9th, 2010. All previous visits were prior to 2009, and before implementation of the ATDL equipment [FHWA 2010c].

The layer structure of Section 340506 is as follows:

287mm AC

254mm Granular Base

Coarse-Grained Silty Sand Subgrade

Figure 2: Pavement Structure – Section 340506 [FHWA 2010c]

2.2.3 447401 – N Smithfield Expressway near Providence, Rhode Island

Section 447401 is part of the GPS-7S set of experiments. The GPS-7S experiment is a study of an AC overlay on a Portland Cement Concrete (PCC) pavement [FHWA 2011]. Section 447401 is a 152.4m long and 3.7m wide pavement sections on the N Smithfield Expressway.

Section 447401 was overlaid in July of 2002 and the section is performing well and has not required any maintenance up to the recent FWD testing visit in August of 2011 [FHWA 2010c].

Section 447401 has had 9 visits where FWD testing was conducted. This report will analyze two of those visits dated November 30th, 2010 and August 3rd, 2011. All previous visits were prior to 2009, and before implementation of the ATDL equipment [FHWA 2010c].

The layer structure of Section 447401 is as follows:

132mm AC

208mm PCC

Poorly Graded Gravel with Sand Subgrade

Figure 3: Pavement Structure – Section 447401 [FHWA 2010c]

3 Data Analysis

The Elmod 6.0 software used for the analysis in this report is able to utilize three different sources of pavement temperature.

- **Source #1**: Asphalt temperature recorded with an ATDL
- **Source #2**: Asphalt Surface temperature recorded by the infrared sensors on the FWD during testing
- **Source #3**: Asphalt temperature predicted using the BELLS temperature prediction model

The LTPP FWD testing protocol consists of a total of 16 drops at each station and four drops per load level. The four different load levels are 26.7kN, 40kN, 53.4kN and 71.2kN [FHWA 2000]. For the purpose of this report, only the drops at the 40kN load level were analyzed as this is the standard design load.

Typical modulus values at a reference temperature of 20°C are provided below in Table 1.

Table 1: Typical Modulus Values for AC at 20°C

Material	Modulus (MPa)
Bituminous at 20°C	3,000 to 7,000

1.1 FWD Testing & Analysis for Section 510115

FWD testing was completed in the outer wheelpath on SR-29 for Section 510115 on July 15th, 2010. This cycle of testing was chosen from the database because of the high asphalt surface temperatures measured during testing.

It is well documented that the stiffness of an asphalt pavement is directly related to its temperature. When collecting FWD data in the hot summer months, the pavement will produce a greater deflection at higher temperatures. Consequently, all backcalculated stiffness moduli for the AC layer must be temperature corrected to account for this material property.

Table 2, shown below, provides the collected surface and air temperatures from the FWD, the ATDL temperature data recorded during testing, as well as the previous day`s average air temperature for input into the BELLS equation.

Table 2: Temperature Data - Section 510115

Date	Surface Temp (°C)				Air Temp (°C)				ATDL Temp (°C)	Previous Day Avg Temp (°C)
	Min	Max	Avg	St Dev	Min	Max	Avg	St Dev		
15-Jul-10	34.3	45.5	40	3.7	28.6	33.1	30.7	1.4	29.2	23

Table 3, shown below, presents a summary of the output of the backcalculation analysis. The first column summarizes the backcalculated AC layer moduli at the temperature during testing. The second column summarizes the backcalculated layer moduli corrected to a reference temperature of 20°C using the three different sources. It is evident that there is a substantial difference between the three different sources of temperature. When correcting the layer moduli to a reference temperature of 20°C using surface temperature data from the infrared (IR) sensors on the FWD unit, the moduli is over-corrected by approximately 1000MPa when compared to Source 1 or 3.

Table 3: Stiffness Moduli - Section 510115

Source	AC Layer Moduli (MPa)			*AC Layer Moduli @ 20°C (MPa)		
	Avg	St Dev	84th Percentile	Avg	St Dev	84th Percentile
1 – ATDL	1409	1.231	1734	1962	1.231	1594
2 – Surface Temp	1412	1.238	1748	3237	1.227	2638
3 – BELLS	1409	1.242	1750	2070	1.234	1678

*Note: Moduli are corrected to a reference temperature of 20°C

3.0 FWD Testing & Analysis for Section 340506

FWD testing was completed in the outer wheelpath on April 25th, 2009 and on March 9th, 2010. This section was chosen to be analyzed because testing was done in the spring time when temperatures are usually below 20°C.

Table 4, shown below, provides the collected surface and air temperatures from the FWD, the ATDL temperature data recorded during testing, as well as the previous day`s average air temperature for input into the BELLS equation.

In this case, the temperature data allows for an interesting comparison as the testing done in April was completed on a day with high temperatures, while the testing completed in March, the following year, was done on a day with relatively low temperatures.

Table 4: Temperature Data - Section 340506

Date	Surface Temp (°C)				Air Temp (°C)				ATDL Temp (°C)	Previous Day Avg Temp (°C)
	Min	Max	Avg	St Dev	Min	Max	Avg	St Dev		
25-Apr-09	29.2	42.2	37.0	3.4	25.0	29.6	27.3	1.5	19.4	12
9-Mar-10	6.4	8.5	7.3	0.71	6.6	8.6	7.6	0.64	16.7	9

Table 5, shown below, presents a summary of the output of the backcalculation analysis. The first column summarizes the backcalculated AC layer moduli at the temperature during testing. The second column summarizes the backcalculated layer moduli corrected to a reference temperature of 20°C using the three different sources.

Analyzing this set of temperature data confirms the fact that at colder temperatures asphalt stiffness increases and at higher temperatures its stiffness decreases. So how does this affect our temperature correction?

Analysis of the previous test section, 510115, illustrated an over-correction of AC layer moduli in high temperatures when using Source 2. This data set is no different, and the outcome is the same. When testing at the higher temperatures experienced in April of 2009, average air temperature measured at 27.3°C, the AC layer moduli is again over-corrected to a much higher value when using Source 2 for temperature correction than when using Sources 1 or 3.

When testing at relatively low temperatures experienced during testing in March of 2010, average air temperature measured at 7.6°C, the AC layer moduli values are all relatively close together and almost match the 84th percentile values for Sources 1 and 3 from the previous year.

Table 5: Stiffness Moduli - Section 340506

Source	Date	AC Layer Moduli (MPa)			*AC Layer Moduli @ 20°C (MPa)		
		Avg	St Dev	84th Percentile	Avg	St Dev	84th Percentile
1 – ATDL	25-Apr-09	3403	1.359	4622	3656	1.359	2691
2 – Surface Temp	25-Apr-09	3398	1.358	4613	6888	1.346	5120
3 – BELLS	25-Apr-09	3396	1.358	4611	3990	1.349	2959
1 – ATDL	9-Mar-10	5945	1.358	8073	4688	1.358	3452
2 – Surface Temp	9-Mar-10	5945	1.359	8080	3851	1.357	2839
3 – BELLS	9-Mar-10	5949	1.359	8082	4353	1.355	3213

*Note: Moduli are corrected to a reference temperature of 20°C

The table above illustrates that temperature correction is more consistent when there is not as much variation in temperature. During FWD testing on March 9th, 2010 the air temperature and surface temperature only varied by a maximum of two degrees.

3.1 FWD Testing & Analysis for Section 447401

FWD testing was completed in the outer wheelpath on November 30th, 2010 and on August 3rd, 2011. This section was chosen to be analyzed because testing was done in the fall and in the summer within a year of previous visit. Also, it would be interesting to see if previous trends were to continue, especially since this section is also a composite pavement.

Table 6, shown below, provides the collected surface and air temperatures from the FWD, the ATDL temperature data recorded during testing, as well as the previous day`s average air temperature for input into the BELLS equation.

Similar to the situation in New Jersey, testing was done at relatively consistent cooler temperatures during the visit in 2010, while the visit in 2011 was tested during a very warm day in August with large variations in temperature.

Table 6: Temperature Data - Section 447401

Date	Surface Temp (°C)				Air Temp (°C)				ATDL Temp (°C)	Previous Day Avg Temp (°C)
	Min	Max	Avg	St Dev	Min	Max	Avg	St Dev		
30-Nov-10	6.9	9.2	8.4	0.59	6.9	7.6	7.2	0.20	8.3	3.0
3-Aug-11	35.2	43.4	39.2	2.47	23.5	26.6	24.9	0.73	33.3	26.0

Table 7, shown below, presents a summary of the output of the backcalculation analysis. The first column summarizes the backcalculated AC layer moduli at the temperature during testing. The second column summarizes the backcalculated layer moduli corrected to a reference temperature of 20°C using the three different sources.

Similar to the trend seen in New Jersey, temperature correction was relatively consistent when analyzing the 2010 FWD data which was collected at relatively consistent temperatures. However, the moduli values for Source 2 were not over-corrected as much as in previous cases. This may be due to several factors: the section is composed of a composite pavement, or the structural adequacy of the section has deteriorated rapidly within the past year. The backcalculated modulus values for the 2011 visit are very low and may indicate a drastic change in the structural support of the pavement structure. Data from subsequent visits should be analyzed for comparison when available.

Table 7: Stiffness Moduli - Section 447401

Source	Date	AC Layer Moduli (MPa)			*AC Layer Moduli @ 20°C (MPa)		
		Avg	St Dev	84th Percentile	Avg	St Dev	84th Percentile
1 – ATDL	30-Nov-10	5998	1.464	8791	3933	1.464	2689
2 – Surface Temp	30-Nov-10	6022	1.453	8761	3960	1.460	2714
3 – BELLS	30-Nov-10	6107	1.464	8950	3982	1.467	2716
1 – ATDL	3-Aug-11	663	1.201	796	1066	1.146	931
2 – Surface Temp	3-Aug-11	667	1.201	800	1330	1.163	1144
3 – BELLS	3-Aug-11	666	1.198	798	1030	1.167	882

*Note: Moduli are corrected to a reference temperature of 20°C

The next section of the report will discuss how the differences in the corrected AC layer moduli affect the design process.

3.2 The Effect of AC Layer Moduli on Selecting a Suitable Design

Elmod has an Overlay Design feature where a user can input the existing layer structure and the desired layer structure and the software will output a design thickness based on the backcalculated layer moduli as well as the AADT or ESAL input for traffic.

Before running the analysis on the different sections it was expected that if there was a large variation in the temperature corrected AC moduli, the resulting overlay design would also vary. However, this was not the case for Sections 340506 and 447401.

The Elmod 6 software uses a mechanistic-empirical based approach when determining the remaining service life and required overlay thickness.

For purposes of the analyses of the pavement sections, the most important equations and parameters assumed for the climatic zone and material conditions were as follows:

1. Three seasons (beginning January 1st) were used, where seasons 1, 2 and 3 are 17, 24 and 11 weeks long each, respectively.
2. Seasonal variation of the asphalt concrete modulus is assumed to be sinusoidal.
3. Flexible pavement design fatigue failure criteria ("structural" failure, fatigue cracking in AC):

$$\varepsilon = 261 \times N^{-0.304} \times (\frac{E}{435})^{-0.259}$$

   where: ε = Tensile strain at bottom of AC layer (microns)
   N = number of load repetitions in millions
   E = AC modulus (MPa)

   This performance model was used in the development of the Asphalt Institute's MS-1 Pavement Design Manual.

4. Permanent deformation on the base and subgrade materials ("functional" failure, roughness related):

$$\sigma_{1,p} = 0.0174 \times N^{-0.307} \times \frac{E}{E_0}^{C}$$

   where: $\sigma_{1,p}$ = Vertical stress on top of unbound layer (MPa)
   N = Number of load repetitions in millions
   E = Modulus of material (MPa)
   E_0 = 160 MPa
   C = 1.16 for $E < E_0$
   C = 1 for $E \geq E_0$

   This transfer function was derived from an analysis of the WASHO and AASHO road tests and from CBR designs by J.M. Kirk (Denmark).

5. Poisson's ratio = 0.35 for all layers
6. Asphalt concrete overlay modulus was assumed to be equal to 3000 MPa at 20°C
7. Assumed design period = 17 years
8. Temperature sensitivity:

   The asphalt concrete moduli vary with temperature and in order to compare these it is necessary to convert the calculated moduli to a reference temperature. As no complete information was available on the asphalt concrete material and no attempts have been made at deriving the master curve from repeated testing of representative points at different temperatures, the exponential equation of the form shown below was used to adjust the asphalt concrete moduli to a reference temperature of 20°C:

$$E = e^{A \times (t - t_{Ref})} \times E_{Ref} \qquad \text{[valid from 0 to 50°C]}$$

where: t_{Ref} = Reference temperature, 20°C
t = AC temperature at time of testing
A = -0.036
E = Backcalculated AC modulus at tested temperature
E_{Ref} = Adjusted AC modulus to reference temperature

For all designs and sources of temperature data, a 100mm mill and overlay was considered.

The traffic input was not taken from the LTPP database, but was selected to be artificially high so that the Elmod software would trigger a need for rehabilitation of the pavement section. The reasoning for this choice was to eliminate another variable from the analysis. The differences in design thickness output by the software could then be compared on an even-handed basis.

3.2.1 Rehab Design – Section 510115

The Elmod software used the backcalculated and adjusted stiffness moduli from Table 3 to be able to calculate the required overlay thickness.

The following design parameters were used in the analysis:

- Maximum effective pavement temperature occurs during Week No. 30, in July = 43°C. Minimum temperature = -13°C
- Mill 100mm of the existing AC layer
- Total traffic on the section per year = 200,000 ESALs

Table 8, shown below, presents the output from the Mill and Overlay design from Elmod.

Table 8: Mill and Overlay Design - Section 510115

Source	Remaining Service Life (yrs)	Recommended Overlay (mm)
1 – ATDL	6	186
2 – Surface Temp	9	138
3 – BELLS	6	178

Similar to the temperature corrected moduli values, the remaining service life and recommended overlays were fairly consistent when using Sources 1 and 3. The design and recommended overlay, when using Source 2, shows an overestimation of pavement strength.

3.2.2 Rehab Design – Section 340506

The Elmod software used the backcalculated and adjusted stiffness moduli from Table 5 to be able to calculate the required overlay thickness.

The following design parameters were used in the analysis:

- Maximum effective pavement temperature occurs during Week No. 29, in July = 45°C. Minimum temperature = -12°C
- Mill 100mm of the existing AC layer
- Total traffic on the section per year = 5 million ESALs

Table 9, shown below, presents the output from the Mill and Overlay design from Elmod.

Table 9: Mill and Overlay Design - Section 340506

Source	Date	Remaining Service Life (yrs)	Recommended Overlay (mm)
1 – ATDL	25-Apr-09	4	144
2 – Surface Temp	25-Apr-09	5	135
3 – BELLS	25-Apr-09	4	143
1 – ATDL	9-Mar-10	7	95
2 – Surface Temp	9-Mar-10	6	99
3 – BELLS	9-Mar-10	6	94

The results shown above were unexpected. The remaining service lives and recommended overlays are consistent when comparing the different sources. However, the results are inconsistent between the two test dates. It appears as though the section has gained strength between 2009 and 2010. However, it is highly unlikely that any rehabilitation was performed between the two visits. There is a possibility that the pavement structure in 2009 was weaker because of excess moisture in the subgrade caused by the spring thaw. This cannot be confirmed without additional data or a geotechnical investigation.

3.2.3 Rehab Design – Section 447401

The Elmod software used the backcalculated and adjusted stiffness moduli from Table 7 to be able to calculate the required overlay thickness.

The following design parameters were used in the analysis:

- Maximum effective pavement temperature occurs during Week No. 29, in July = 46°C. Minimum temperature = -16°C
- Mill 100mm of the existing AC layer
- Total traffic on the section per year = 250,000 ESALs

Table 10, shown below, presents the output from the Mill and Overlay design from Elmod.

Table 10: Mill and Overlay Design - Section 447401

Source	Date	Remaining Service Life (yrs)	Recommended Overlay (mm)
1 – ATDL	30-Nov-10	10	126
2 – Surface Temp	30-Nov-10	9	127
3 – BELLS	30-Nov-10	9	133
1 – ATDL	3-Aug-11	12	126
2 – Surface Temp	3-Aug-11	12	133
3 – BELLS	3-Aug-11	12	131

Similar to the results from the section analyzed in New Jersey, the remaining service lives and recommended overlays are fairly consistent throughout for Section 447401.

3.3 Additional Analysis

The Mechanistic-Empirical Pavement Design Guide (MEPDG) approach considers the input parameters that influence pavement performance – including traffic, climate and pavement layer thickness and properties – and applies the principles of engineering mechanics to predict critical pavement responses [AASHTO 2008]. Similar to the new DarWin-ME software, although not as stringent, the Elmod mechanistic-empirical design calculations can account for large variations in temperature when using FWD data collected at various times and weather conditions of the year.

The MEPDG approach presents a major change from the pavement design methods in the 1993 AASHTO *Guide for Design of Pavement Structures*, which are based on limited empirical performance equations developed from the AASHO Road Test in the late 1950s [Hanna 2010]. Additional analyses comparing the Elmod mechanistic-empirical method and the AASHTO 93 design method were completed using the New Jersey data.

The New Jersey data was only selected since a master temperature correlation curve could be developed by using data from 2002 since FWD testing was done in four separate months that year. Section 3.4 of this paper provided the equation used for temperature correction:

$$E = e^{A\times(t-t_{Ref})} \times E_{Ref}$$

A master curve was developed by using the accumulated difference method using backcalculated FWD moduli at their respective testing temperatures from data collected February 3rd, March 9th, September 28th and December 21st in 2002.

Once the data was plotted, the unknown constant, A, was solved for. Figure 4, shown below presents the master curve.

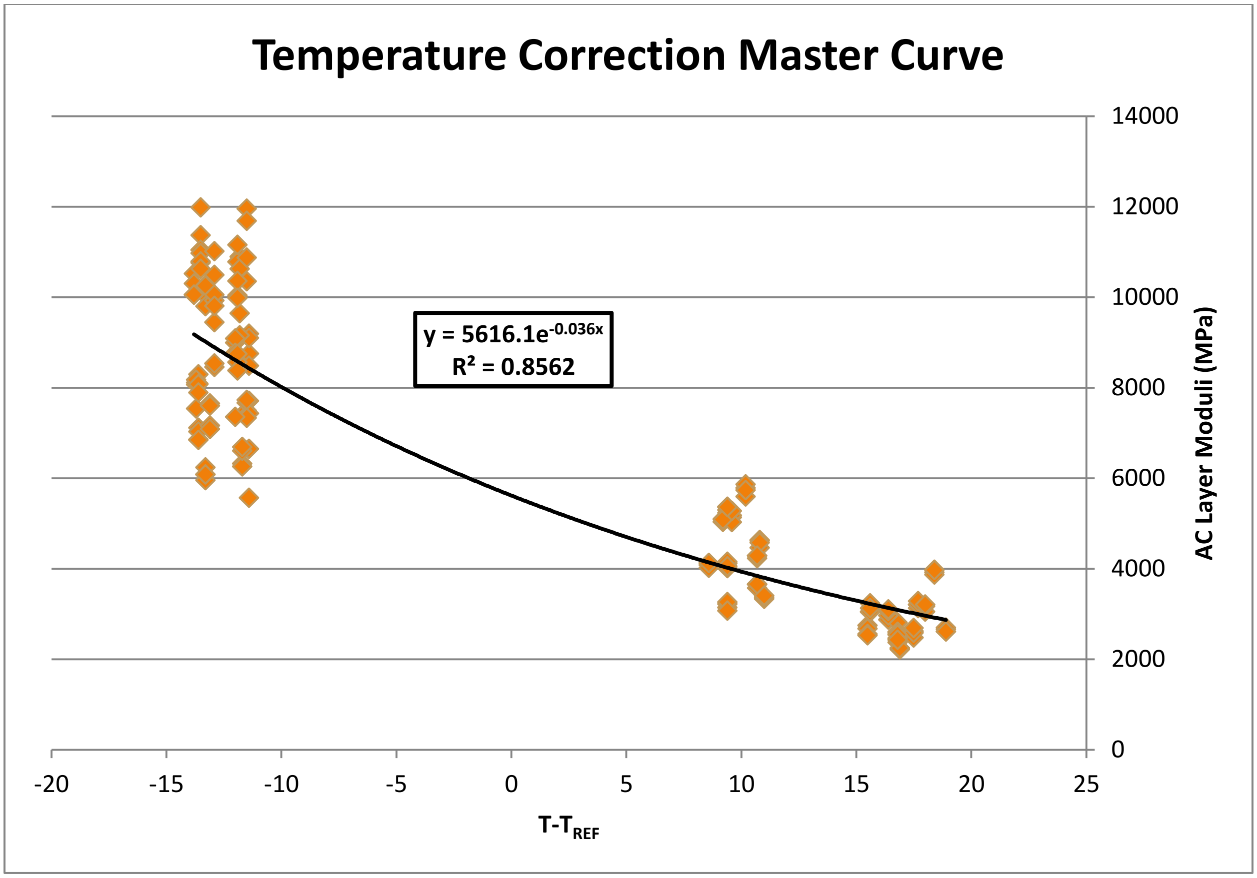

Figure 4: Section 340506 Master Curve

From the above exponential regression curve equation, the unknown constant was found to be:

A = -0.036 (same as default constant in Elmod 6)

The constant was found to be identical to the default constant already in the Elmod software. Since this constant was used in the initial analysis, no further analysis was required in Elmod.

An AASHTO 1993 analysis was run on the New Jersey 340506 data collected in 2009 and 2010. The results were normalized to 40kN load levels and temperature corrected only using the IR surface temperature collected directly from the FWD unit. The same traffic values were used as in the Elmod analysis; 5 million ESALs/year.

Using the SN_{EFF} value calculated in the AASHTO '93 process, the structural number at the time of testing can be compared to the structural number of the section immediately after construction. For the purpose of this report, the structural number immediately after construction will be labeled SN_{NEW}. Section 340506 is composed of 287 mm of asphalt concrete over a 254 mm granular base, therefore:

$$SN_{NEW} = 287 \times 0.42 + 254 \times 0.14 \times 0.9$$

$$SN_{NEW} = 153$$

The following parameters were used in the AASHTO 93 analysis:

- Initial Serviceability – 4.2
- Terminal Serviceability – 2.2
- Reliability (%) – 95
- Overall Standard Deviation – 0.44

Table 11, shown below, presents the results including the minimum, maximum, average and standard deviation of the SN_{EFF} value backcalculated using the AASHTO '93 method.

Table 11: Section 340506 AASHTO '93 Results

Section	Date	SN_{EFF}				SN_{NEW}
		Min	Max	Avg	St Dev	
340506	25-Apr-09	166	209	185	12.8	153
340506	09-Mar-10	288	367	322	25.4	

As shown in the table above, it is evident that the AASHTO 1993 backcalculation method is not reliable for determining design parameters for Section 340506. In both cases, the in-situ structural number was calculated to be higher than the structural number immediately after construction. The results indicate that the pavement section is structurally adequate and does not require rehabilitation. However, Elmod analysis showed that the asphalt concrete layer moduli were approaching the minimum modulus of a bituminous material and the overlay design resulted in a minimum requirement of a 100 mm mill and overlay rehabilitation treatment.

Section 340506 was rehabilitated in the summer of 2010 by the NJDOT. The performance measure that triggered the rehabilitation event is unknown at this time, but will be documented in future releases of the LTPP SDR.

4 Summary of Main Findings

Backcalculation analysis has been conducted on three different LTPP pavement sections located in the states of Virginia, New Jersey and Rhode Island in the United States. Section 510115, Section 340506 and Section 447401 are part of the LTPP SPS–1, SPS–5 and GPS-7S experiments respectively.

Within the past three years, the North Atlantic and North Central Regional Contractor has added a new method of temperature measurement using an ATDL. In addition, the NARC/NCRC also collects temperature data using the FWD unit's IR temperature sensors, as well as measuring the asphalt temperature manually using a hand-held sensor.

The goal of this report was to examine the use of the ATDL and its role in affecting the design process compared to using two other sources of temperature data.

In general, the temperature corrected moduli using Sources 1 and 3 were relatively close in every case. All three sources were consistent only when FWD testing was done in the spring or fall when temperatures are cooler and when the surface temperature is close to the air temperature during testing.

1.1 Section 510115

A backcalculation analysis was completed using FWD data from July 15th, 2010 from LTPP test section 510115 located on the SR-29 Expressway near Danville, Virginia. Air temperature during testing was approximately 30°C, with surface temperatures ranging from 40°C to 45°C.

An overlay design was conducted using the different temperature sources, Source 1, 2 and 3. The Elmod software used the adjusted backcalculated layer moduli to calculate the required overlay design thickness.

Source 1 – Average AC layer modulus (adjusted to 20°C) = 1962 MPa, 186 mm overlay
Source 2 – Average AC layer modulus (adjusted to 20°C) = 3237 MPa, 138 mm overlay
Source 3 – Average AC layer modulus (adjusted to 20°C) = 2070 MPa, 178 mm overlay

4.0 Section 340506

A backcalculation analysis was completed using FWD data from April 25th, 2009 and March 9th, 2010 from LTPP test section 340506 located on the westbound lanes of I-195 near Upper Freehold, New Jersey. Air temperature during testing was approximately 29°C in April 2009 and 8°C in March 2010, with surface temperatures ranging from 29°C to 42°C and 6°C to 9°C respectively.

An overlay design was conducted using the different temperature sources, Source 1, 2 and 3. The Elmod software used the adjusted backcalculated layer moduli to calculate the required overlay design thickness.

April 2009:

Source 1 – Average AC layer modulus (adjusted to 20°C) = 3656 MPa, 144 mm overlay
Source 2 – Average AC layer modulus (adjusted to 20°C) = 6888 MPa, 135 mm overlay
Source 3 – Average AC layer modulus (adjusted to 20°C) = 3990 MPa, 143 mm overlay

March 2010:
Source 1 – Average AC layer modulus (adjusted to 20°C) = 4688 MPa, 95 mm overlay
Source 2 – Average AC layer modulus (adjusted to 20°C) = 3851 MPa, 99 mm overlay
Source 3 – Average AC layer modulus (adjusted to 20°C) = 4353 MPa, 94 mm overlay

The master temperature correction curve was plotted using FWD data collected in 2002 and the ‘A’ coefficient was found to be equal to -0.036, which is the same as the value that Elmod uses by default for any design calculations.

4.1 Section 447401

A backcalculation analysis was completed using FWD data from November 30th, 2010 and August 3rd, 2011 from LTPP test section 447401 located on the northbound lanes of the N Smithfield Expressway near Providence, Rhode Island. Air temperature during testing was approximately 7°C in November 2010 and 26°C in August 2011, with surface temperatures ranging from 6°C to 9°C and 35°C to 43°C respectively.

An overlay design was conducted using the different temperature sources, Source 1, 2 and 3. The Elmod software used the adjusted backcalculated layer moduli to calculate the required overlay design thickness.

November 2010:
Source 1 – Average AC layer modulus (adjusted to 20°C) = 3933 MPa, 126 mm overlay
Source 2 – Average AC layer modulus (adjusted to 20°C) = 3960 MPa, 127 mm overlay
Source 3 – Average AC layer modulus (adjusted to 20°C) = 3982 MPa, 133 mm overlay

August 2011:
Source 1 – Average AC layer modulus (adjusted to 20°C) = 1066 MPa, 126 mm overlay
Source 2 – Average AC layer modulus (adjusted to 20°C) = 1330 MPa, 133 mm overlay
Source 3 – Average AC layer modulus (adjusted to 20°C) = 1030 MPa, 131 mm overlay

5 Conclusions

In December of 2009, the FHWA published an article in their monthly publication, known as Focus, regarding their vision on strategic, safe and sustainable pavements [FHWA 2009]. The FHWA outlined six focus areas to help achieve the high-performing, safe and cost-effective pavement network that the traveling public and the Nation's economy depends on every day:

1. Pavement design and analysis
2. Pavement materials and construction technology
3. Pavement management and preservation
4. Pavement surface characteristics
5. Construction and materials quality assurance
6. Environment stewardship

The MEPDG, developed under the National Cooperative Highway Research Program (NCHRP) Project 1-37A, "provides a uniform basis for the design of flexible, rigid and composite pavements, using mechanistic-empirical approaches that more realistically characterize in-service pavements and improve the reliability of designs" [FHWA 2009].

Overall, this paper shows that the differences in temperature corrected moduli can be significant and depending on the design analysis chosen, can cause either underestimation or overestimation of pavement strength as a result. However, when using a mechanistic-empirical based approach to the design, the quality of temperature data can be compensated for with a properly developed seasonal temperature model.

In the past five years, North America has been challenged by difficult financial times. As Federal, Provincial and Municipal governments cut annual infrastructure budgets, and as natural resources continue to increase in price and supplies become more limited, agencies must find new ways in maintaining the level of service of our infrastructure at an acceptable level.

The adoption of mechanistic-empirical design methods by agencies throughout North America will aid in effectively managing pavement networks and improving sustainability by providing more reliable pavement designs. As discussed in section 3.3, the AASHTO 1993 analysis provided results that indicated that the pavement section could be in service for another 15+ years without any rehabilitation. However, the Elmod mechanistic-empirical analysis indicated that the pavement section would need to be rehabilitated within the next 5 to 7 years. Trusting the AASHTO 1993 analysis, an agency could allow the pavement to degrade in quality until a reconstruction would be needed. This would be a significant misuse of funds and could be compounded when extrapolated to the rest of the pavement network. For this reason, a mechanistic-empirical design approach could potentially save an agency millions of dollars every year.

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