Falling Weight Deflectometer Testing Based Mechanistic-Empirical Overlay Thickness Design Approach for Low-Volume Roads in Illinois

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Abstract

A recent Illinois Center for Transportation (ICT) research project at the University of Illinois has aimed at evaluating the use of Falling Weight Deflectometer in the assessment of structural conditions of in-service low volume roads that are in need of rehabilitation. Ten different pavement sections were selected from four counties in Illinois with varying structural and traffic characteristics to conduct Falling Weight Deflectometer (FWD) tests. A neural-network based pavement analyzer, ANN-Pro, previously developed at the University of Illinois at Urbana-Champaign based on ILLI-PAVE finite element program solutions, was used to analyze the FWD data in order to determine and monitor the structural adequacies of existing pavement sections. This paper presents a new Mechanistic-Empirical (M-E) Overlay Design method that was introduced as part of the ICT research project to adequately assess the structural conditions of existing pavements and subsequently recommend required overlay thickness values from critical pavement responses computed from FWD field deflections. The M-E Overlay Design methodology compares the critical pavement responses to threshold values for the pre-established fatigue and/or rutting damage algorithms.

INTRODUCTION

One of the most common maintenance and rehabilitation approaches for flexible pavements involves the placement of a hot-mix asphalt (HMA) overlay on the existing pavement structure, thus improving the structural as well as the functional condition of the pavement. Proper assessment of the current structural condition of existing pavements is critical for this process and can be accomplished using nondestructive testing (NDT) equipment such as the Falling Weight Deflectometer (FWD). Although the state of the art in deflection-based pavement structural evaluation has advanced significantly with incorporation of modern analytic approaches, such as energy-based and viscoelastic methods, the degree to which such methods are used in real practice has been found to be suboptimal. Some factors that have potentially contributed to differences in the state of the art in research and the state of practice in pavement technology are as follows: (1) initial costs associated with the procurement of FWD devices and (2) inconveniences associated with the application of complex analysis procedures requiring significant time and knowledge of practicing engineers. These obstacles and the availability of limited resources become particularly significant during the rehabilitation of low volume roads. Accordingly, overlay thickness design for low volume flexible pavements is often carried out by local agencies using highly empirical approaches without any mechanistic analyses. One example of such an empirical approach which is based on empirical layer coefficients described in the *1993 AASHTO Guide for Design of Pavement Structures* is the modified layer coefficient–based approach used by the Illinois Department of Transportation (IDOT) in Illinois (IDOT 2012). Although this empirical approach is fairly simple to use, it has been found to be inefficient in characterizing modern construction materials (Sarker *et al*. 2015). Moreover, accurate assessment of the current structural conditions of existing pavement structures is essential for economical and proper design of HMA overlays.

The NDT-based overlay thickness design method specified by the *1993 AASHTO Guide for Design of Pavement Structures,* which uses FWD deflection data is primarily based on the concept of structural numbers (SN), is inherently empirical in nature and developed from the AASHO road test study conducted nearly six decades ago (AASHTO 1993). Addressing the limitations associated with using empirical approaches to design an overlay for low volume roads in Illinois, a recent research study was undertaken at the Illinois Center for Transportation (ICT) to develop a mechanistic-empirical design approach in an effort to improve the overlay thickness design methods for low volume roads in Illinois. This paper will present an overview of the recently proposed mechanistic-empirical (M-E) design approach for low volume roads in Illinois. With the increased prevalence of M-E pavement design approaches, it is important for the overlay thickness design methods for low volume roads to have a mechanistic foundation as well. Deflection-based evaluation methods of pavement structural condition, along with the calculated critical pavement response parameters, can provide the required inputs for such a mechanistic-based overlay thickness design method. Pre-established calibrated damage algorithms that take into account local conditions and pavement damage mechanisms can constitute the empirical component of such methods. The advantages of the developed method are demonstrated through case studies conducted with local agencies in the state of Illinois and will be presented in the following sections.

SELECTED CASE STUDIES

Ten pavement sections in four counties in Illinois were selected for FWD-based structural condition evaluations and subsequent overlay thickness designs. Pavement layer configurations, design traffic levels, and maintenance schedules of local agencies were all carefully reviewed during the development of the field FWD test matrix. Primary emphasis was given to pavement sections that displayed high-severity distresses and had already been selected by the local agencies for rehabilitation. Table 1 presents the locations and study details of the selected pavement test sections. Typically, FWD tests along a given road segment were conducted at 200-ft intervals on the outer wheel paths. The trailer-mounted Dynatest FWD was used in this study with a standard configuration of geophones placed at 0, 12, 24, 36, 48, 60, and 72 in., respectively, from the center of the loading plate (plate radius = 6 in.). Pavement surface temperature was collected during the testing at every 2000-ft interval along the testing lane. Figure 1 shows photos taken during the FWD testing in local road pavement sections.

Table 1. Details of the Selected Case Studies.

Location in Illinois	Road Name	No. of Sections	Section Number	Pavement Condition
McHenry Coral County	East Coral Road	2	1–2	Severely Cracked; Overlay Needed
	Church Road	2	3–4	Severely Cracked; Overlay Needed
City of Dekalb	Twombly Road	1	5	Severely Cracked; Overlay Needed
Village of Tinley Park	Normandy Lane	1	6	Moderate Cracks
	Dorothy Lane	1	7	Moderate Cracks
Champaign County	CH 1 Dewey–Fisher Road	3	8–10	Very Few Cracks

Figure 1. Conducting FWD tests in selected local road pavement sections in Illinois.

Sections 1 through 4 were located in McHenry County, Section 5 was located in DeKalb County, whereas Section 6 and 7 in Village of Tinley Park, and Sections 8 through 10 were located in Champaign County. The layer configurations of the selected pavement sections and photos showing the pavement surface distresses are given in Figure 2 and Figure 3, respectively. As can be seen in Figure 3, Sections 1 through 5 had signs of severe surface cracking and needed to be rehabilitated urgently. Figure 4 shows aerial locations of pavement sections tested in this project. As shown, Sections 1 and 2 represent contiguous sections on the same road segment (East Coral Road). Sections 3 and 4, on the other hand, represent lanes carrying traffic in opposite directions along the other road segment (Church Street). Such division of the tested road segments into different sections was necessary considering the varying pavement layer profiles and substructure (base, subbase, and subgrade) support conditions. Sections 8, 9, and 10 were also part of the same road segment (CH1 Dewey-Fisher Road) but with varying traffic conditions.

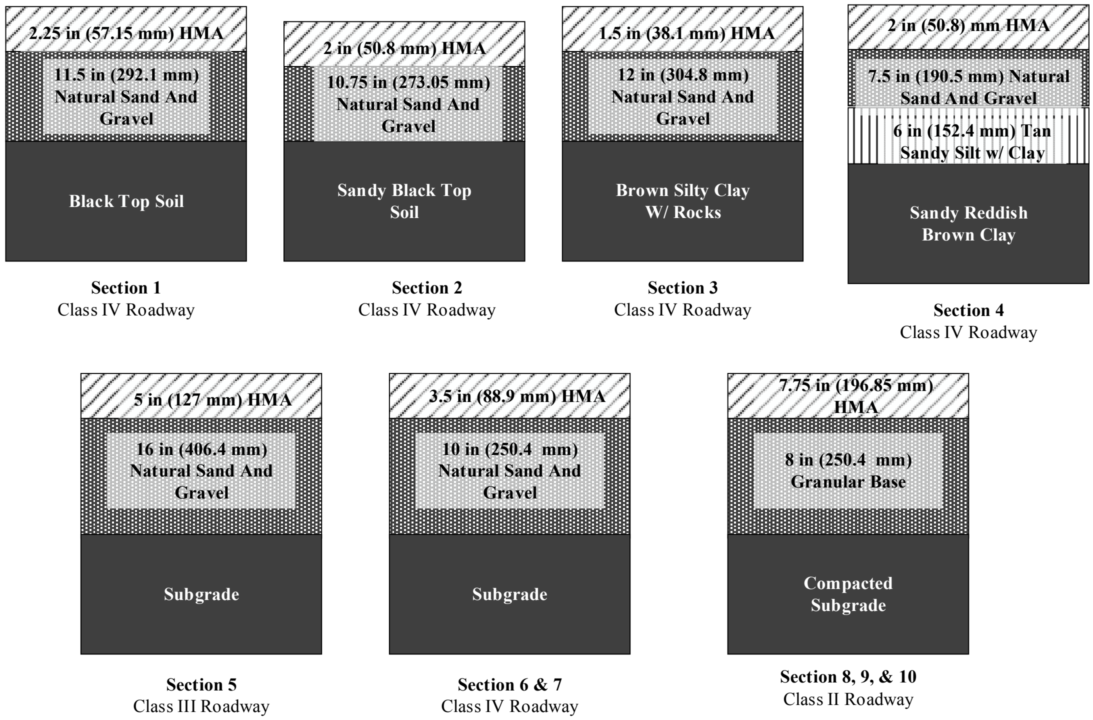

Figure 2. Layer configurations of the selected local road pavement sections in Illinois.

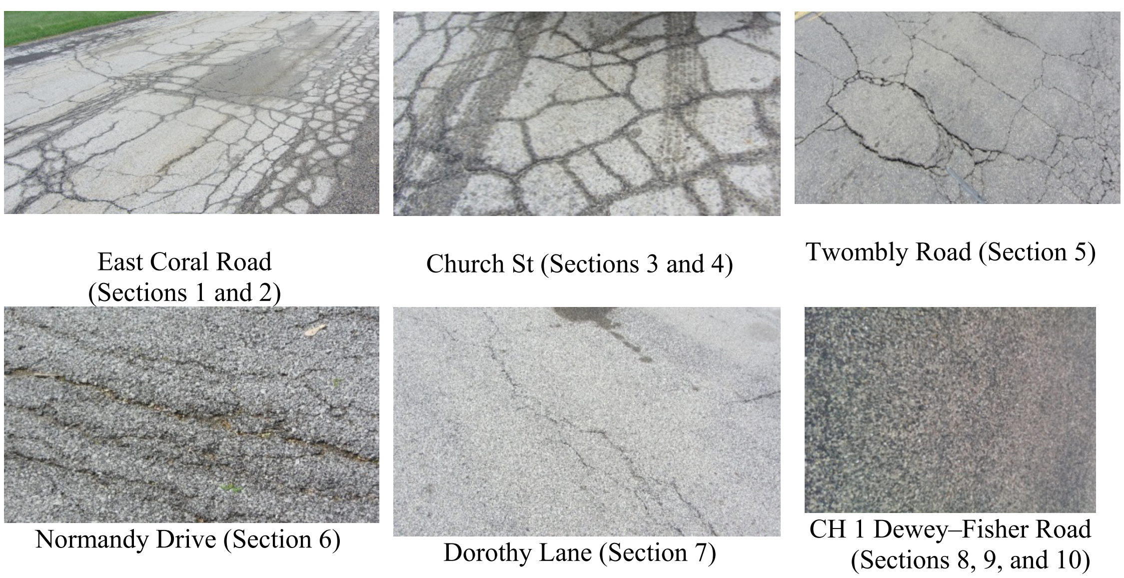

Figure 3. Photos of the selected local road pavement sections in Illinois.

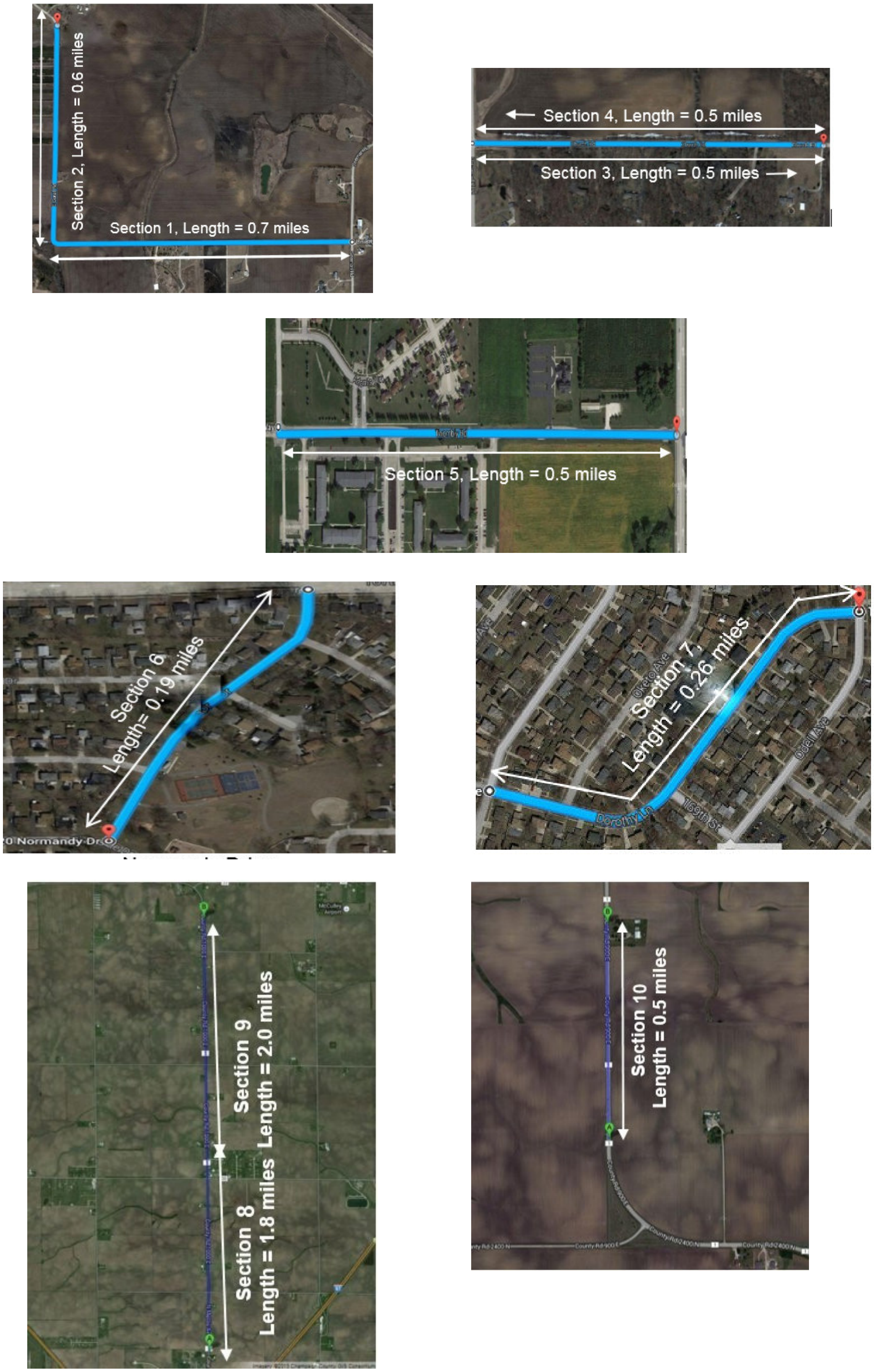

Figure 4. Aerial photos of the selected local road pavement sections in Illinois.

PROPOSED MECHANISTIC-EMPIRICAL DESIGN APPROACH

The following section presents an overview of the mechanistic-empirical design approach introduced to design HMA overlays for low volume roads in Illinois. As previously mentioned, this proposed approach is based on proper structural evaluation of the existing pavements that relies on the fatigue and deflection responses of the pavement as the design criteria.

FWD data analyses for layer characterization

Structural evaluation of the pavement sections and subsequent development of an improved overlay thickness design approach involved backcalculation of individual layer moduli from the FWD data. This task was accomplished using the extensively tested and validated ILLI-PAVE finite element (FE) pavement analysis program (Raad and Figueroa 1980) in conjunction with the linear elastic theory–based software program BISAR (1989) to carry out modulus backcalculation for the individual pavement layers. In addition, ANN-Pro, a neural network based backcalculation software program, developed as a surrogate to ILLI-PAVE with easy to use features for practitioners, was also used when applicable as an alternative to perform the backcalculation analyses (Pekcan *et al*. 2009). ANN-Pro aims to evaluate the current structural conditions of the pavement by analyzing the FWD deflection data implementing the advanced ILLI-PAVE FE solutions in backcalculation analyses. For the ILLI-PAVE analyses, layered elastic analyses using BISAR were first carried out to calculate typical stress states (represented by the sum of principal stresses or bulk stress; $\theta = \sigma_1+\sigma_2+\sigma_3$) at the mid-height of the unbound aggregate base layer. Later, the bulk stress θ values were used in a stress-dependent resilient modulus model (K-θ model) in ILLI-PAVE to calculate the critical pavement responses. ILLI-PAVE uses nonlinear, stress-dependent resilient modulus characterizations in the subgrade and granular base/subbase layers.

FWD tests on the test pavement sections were modeled as a standard 40-kN (9-kip) equivalent, single-axle loading applied with a uniform pressure of 551 kPa (80 psi) over a circular area of 152.4-mm (6-in.) radius. In accordance with the locations of FWD geophones, the surface deflection values were extracted from the ILLI-PAVE analysis results at 0, 12, 24, and 36 in., respectively, away from the center of the loading plate. The purpose of using ILLI-PAVE or, alternatively, the forward analysis module in ANN-Pro when applicable, was to adjust the layer moduli in such ways that the original field deflection basins could be modeled properly. Individual layer moduli in the pavement sections being analyzed were then iteratively adjusted until the deflection values predicted from ILLI-PAVE were sufficiently close to the median values obtained from the field test results. Although the actual FWD test configuration comprised seven geophones to capture the pavement deflection basin, this iterative calculation step aimed, for convenience, to match the deflections at the first four sensor locations. The surface deflections corresponding to the locations of these FWD sensors were abbreviated as D_0, D_{12}, D_{24}, and D_{36}, respectively. Next, the backcalculated layer moduli were further adjusted using ILLI-PAVE and BISAR software programs and ANN-Pro neural network based pavement analyzer. Figure 5 shows a fairly good match between the field-measured (median) and predicted deflection values.

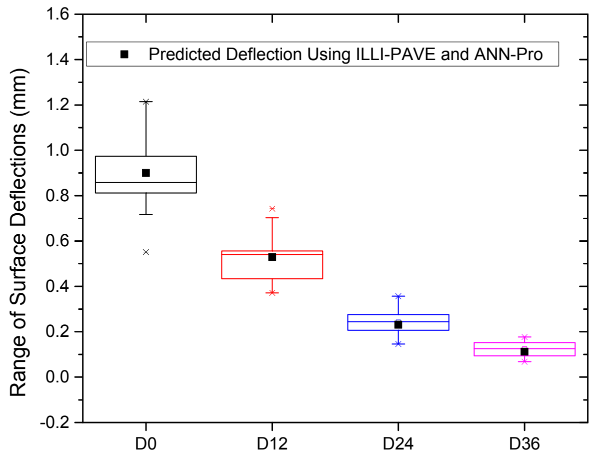

Figure 5. Deflection matching with ILLI-PAVE finite element program and its easier to use surrogate ANN-Pro neural network analysis platform

As FWD deflection basins are obtained at different ambient and pavement temperatures, in order to properly backcalculate modulus of the asphalt pavement layer from FWD deflection data, it is necessary to adjust either the deflections or the backcalculated modulus to a reference temperature. In this study, this process was achieved through a two-step procedure. The first step required determining the HMA temperature at a desired depth of the HMA layer, followed by the second step of adjusting the backcalculated modulus to a reference temperature by applying temperature correction factors. As FWD testing has established itself as an effective means of structural evaluation of existing in-service pavements, many research studies have proposed models for the adjustment of asphalt moduli to a reference temperature by investigating the influence of pavement temperatures on backcalculated asphalt pavement moduli. This study used the temperature prediction model in Equation 1 developed by Park *et al*. (2001) to calculate mid-depth HMA temperatures from the pavement surface temperatures measured during FWD testing. Next, the asphalt temperature adjustment factor (ATAF) was obtained using the mid-depth asphalt HMA temperature developed by a Long-Term Pavement Performance (LTPP) program study (Lukanen *et al*. 2000) given in Equation 2.

$$T_z = T_{surf} + (-0.3451z - 0.0432z^2 + 0.00196z^3)sin(-6.3252t + 5.0967) \quad (1)$$

where
T_z : AC pavement temperature at depth z, °C
T_{surf} : AC pavement temperature at the surface, °C
Z : Depth at which temperature to be determined, (cm)
Sin : Sine functions, (radians)
T : Time when the pavement surface temperature was measured, days.

$$ATAF = 10^{slope(T_r - T_m)} \quad (2)$$

where
ATAF : Asphalt temperature adjustment factor
Slope : Slope of the log Modulus versus Temperature curve
T_r : Reference temperature of 21°C
T_m : Pavement temperature at mid-depth (°C).

Table 2 lists the iteratively calculated layer modulus values using ILLI-PAVE and ANN-Pro. The effect of temperature corrections is apparent on the backcalculated HMA modulus as sections tested (Sections 1 through 4) at low temperature showed a decrease in modulus while sections such as Sections 4 through 10 exhibited an increase.

Table 2. Iteratively Calculated Layer Moduli using ILLI-PAVE and ANN-Pro to Match FWD Deflection Basins.

Section Number	HMA Modulus (ksi)	HMA Modulus (ksi) (Temperature Corrected for 70°F)	Testing Temperature (°F)	Base/Subbase E_r (ksi) = K (ksi) $\left(\frac{\theta}{p_0}\right)^n$	Subgrade Modulus (ksi)
1	600	307	45	K=2.5, n=0.33	14
2	800	408	45	K=2, n=0.33	12
3	600	304	45	K=4, n=0.33	12
4	550	278	45	K_{base}=4.2, n_{base}=0.33 $K_{subbase}$=2.5, $n_{subbase}$=0.33	12
5	300	483	88	K=4, n=0 .33	11
6	200	279	82	K=4.5, n=0.5	6.8
7	425	590	83	K=4.9, n=0.5	8
8	250	279	75	Base: K=7, n=0.5 Subbase: K=5, n=0.5	15
9	300	358	79	K=6, n=0.5	15
10	200	257	82	Base: K=5.8, n=0.5 Subbase: K=2, n=0.5	17.9

Structural adequacy evaluation of the existing pavement

Upon completion of characterizations of the layer properties, the structural conditions of the pavement sections were evaluated using critical pavement responses (tensile strain at the bottom of the asphalt layer, ε_t; and vertical surface deflection under the load, δ_v) and the IDOT damage algorithms (see Equations 3 and 4). Design traffic information obtained from the local transportation agencies was used to calculate the total Equivalent Single Axle Loads (ESALs) over a design period of 20 years (N_f). This value of N_f was then used to calculate the threshold critical pavement response values for the different pavement sections.

$$N_f = \frac{8.78\times10^{-8}}{(\varepsilon_t)^{3.5}} \tag{3}$$

$$N_f = \frac{5.73\times10^{10}}{(\delta_v)^4} \quad (4)$$

Whether the pavement section requires an overlay or not was determined by comparing the ε_t and δ_v values under the current pavement configuration with the threshold values calculated using Equations 3 and 4. The threshold values of ε_t and δ_v, along with the corresponding values under FWD test efforts are listed in Table 3.

Table 3. Critical Pavement Responses Compared to the Threshold Values for Design Traffic Levels.

Section Number	Predicted ESALs Over Pavement Design Life	Threshold-Critical Pavement Response Parameters based on Damage Algorithms		Critical Pavement Response Parameters under Original Pavement Configuration		Overlay Required?
		ε_t	δ_v (mil)	ε_t *	δ_v** (mil)	
1	13,524	6.36E-4	45.36	6.13E-4	46.33	YES
2	13,524	6.36E-4	45.36	6.06E-4	52.21	YES
3	13,524	6.36E-4	45.36	4.52E-4	48.47	YES
4	13,524	6.36E-4	45.36	5.32E-4	47.88	YES
5	404,787	2.40E-4	19.40	4.53E-4	29.51	YES
6	13,524	6.36E-4	45.36	4.49E-4	41.7	NO
7	13,524	6.36E-4	45.36	3.49E-4	32.89	NO
8	1,519,234	1.64E-4	13.94	1.19E-4	11.21	NO
9	1,556,746	1.63E-4	13.85	1.19E-4	11.21	NO
10	1,350,430	1.71E-4	14.35	1.19E-4	11.21	NO

As can be observed from Table 3, the M-E overlay design method adequately captures the structural inadequacies of the pavement sections for the existing pavement configurations. Section 5 fails under the fatigue as well as rutting algorithms. Sections 1 through 4 on the other hand, prove to be adequate for the fatigue performance but fail under the rutting criterion. Sections 6 through 10 are found to be structurally adequate to carry on the 20 year projected traffic load.

Once the requirements of overlay for the sections were established, the overlay thickness was iteratively adjusted using both ILLI-PAVE and ANN-Pro forward calculations (when applicable) to ensure that the new pavement system will meet the threshold critical pavement response parameters as presented in Table 3. Table 4 lists HMA overlay thickness requirements determined by the M-E overlay design method and Figure 6 presents a flow chart illustrating the procedure of the proposed Mechanistic Empirical (M-E) overlay design procedure.

Table 4. HMA Overlay Thicknesses Determined from the Proposed M-E Overlay Design Method.

Section Number	Required Thickness of HMA Overlay (in.)	Critical Pavement Responses After Overlay		Capacity > Required (Design Period = 20 Years)
		ε_t	δ_v (mil)	
1	2	5.55E-4	34.74	YES
2	2	5.35E-4	36.49	YES
3	2	5.46E-4	34.05	YES
4	2	5.47E-4	34.03	YES
5	2.5	1.96E-4	17.78	YES
6	Not Required	N/A	N/A	YES
7	Not Required	N/A	N/A	YES
8	Not Required	N/A	N/A	YES
9	Not Required	N/A	N/A	YES
10	Not Required	N/A	N/A	YES

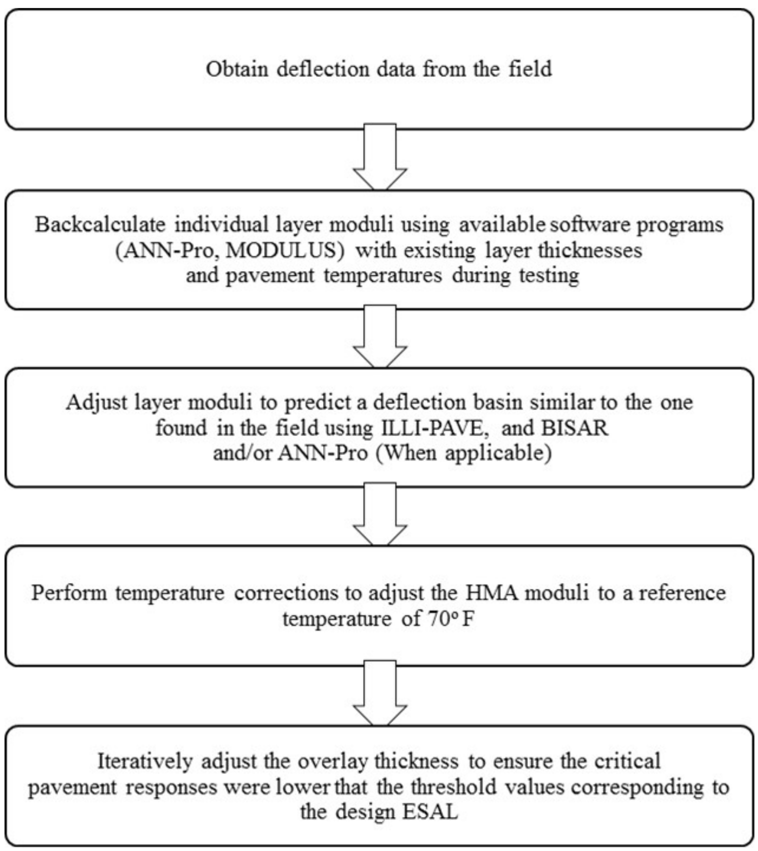

Figure 6. Flow Chart of the Proposed Mechanistic Empirical (M-E) Overlay Design Procedure.

COMPARISONS WITH THE AASHTO 1993 NDT METHOD

AASHTO 1993 NDT-based method uses FWD-obtained deflection basin information; subsequently, the subgrade resilient modulus (M_R), and the required structural number (SN_{req}) to carry the projected traffic is determined using available charts (AASHTO 1993). The effective structural number (SN_{eff}) of the existing pavement is calculated, and the difference between SN_{eff} and SN_{req} determines the required overlay thickness using empirical layer coefficients. Table 5 presents HMA overlay thickness requirements as determined by the AASHTO 1993 NDT-based method.

Table 5. HMA Overlay Thicknesses Determined by the AASHTO 1993 NDT Method.

	Sections									
	1	2	3	4	5	6	7	8	9	10
Traffic Factor	0.014	0.014	0.014	0.014	0.41	0.014	0.014	1.52	1.56	1.35
Median SN_{eff}	2.08	2.19	2.16	2.28	2.96	2.81	3.08	5.54	5.05	4.79
SN_{req} (IBV=6)	1.90	1.90	1.90	1.90	3.1	1.9	1.9	3.68	3.70	3.60
Overlay Requirement (in.), for 50^{th} Percentile SN_{eff}	0	0	0	0	0.35	0	0	0	0	0

Table 5 clearly indicates that when the median of the SN_{eff} values were considered, the required structural number (SN_{req}) was often found to be lower than the current structural number (SN_{eff}) of the pavement sections. Only Section 5 demonstrated a lower SN_{eff} value (SN_{eff} = 2.96; 50^{th} percentile) compared to the corresponding SN_{req} (= 3.1). Accordingly, all pavement sections except for Section 5 would not require any structural overlay. However, as previously mentioned, most of the pavement sections demonstrated a severe degree of fatigue cracking during the FWD testing, indicating an inadequate structural condition. The somewhat erroneous categorization of these pavements as structurally adequate by the AASHTO method can be attributed to the significantly low design traffic volumes for these pavement sections. Given identical material properties and layer configurations, increased traffic would also increase the required structural capacity, thus making the current pavement inadequate structurally as well.

SUMMARY AND CONCLUSIONS

This paper presented partial findings from a recently completed research study at the University of Illinois aimed at improving overlay thickness design methods for low volume road pavements in Illinois. Ten pavement sections were selected from four counties in Illinois, with varying structural and traffic characteristics. FWD tests were conducted on these road segments to determine the structural conditions of the existing HMA pavement sections. Accordingly, a new mechanistic-empirical (M-E) overlay design method was developed to adequately assess the structural conditions of existing pavements and subsequently recommend required thickness values from FWD-based critical pavement responses computed and compared to threshold values for the pre-established fatigue and/or rutting damage algorithms. The M-E overlay design method successfully identified structural deficiencies in the original pavement configurations through FWD NDT and resulted in reliable overlay solutions, as compared to the AASHTO 1993 NDT method. Such testing, as highlighted in this paper, will allow the local agencies to more accurately determine the most economical rehabilitation method and the anticipated service life of the improvement. The use of the proposed M-E overlay design method can prove to be a significant improvement over the methods currently used to determine rehabilitation strategies in low volume roadways in Illinois.

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