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Nondestructive Deflection Testing based Mechanistic-Empirical Overlay Thickness Design Approach for Low Volume Roads: Case Studies

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Abstract
Illinois Department of Transportation (IDOT) uses an empirical design approach to conduct thickness designs of structural overlays for low volume roads. Modified layer coefficients for a limited number of material types are utilized to assign the structural capacity of in-service pavements. Despite the ease in use and simplicity, such an empirical approach is outdated and lacking in many aspects to characterize recycled and/or nontraditional construction materials nowadays more commonly used in pavements. As far as the rehabilitation of low volume roads is concerned, the lack of testing for evaluating the structural condition of existing, in-service pavements often results in uneconomical and unreliable practices. This paper presents a mechanistic-empirical approach for overlay thickness designs of low volume pavements through a combination of nondestructive deflection testing and pre-established pavement damage models. Twenty different pavement sections were selected from six counties in Illinois with varying structural and traffic characteristics. Falling Weight Deflectometer (FWD) tests were conducted on these road segments and FWD data were analyzed with appropriate temperature correction procedures to determine and monitor the structural conditions of existing, in-service pavement sections. Then, the corresponding required overlay thicknesses were determined for these twenty case sections using three different methods commonly used by local agencies such as, AASHTO 1993 NDT method, IDOT modified layer coefficient method, and Asphalt Institute deflection approach. The M-E Overlay Design method successfully identified structural deficiencies in the original pavement configurations through FWD tests and subsequently led to more economical or safer and more reliable overlay solutions for low volume roads.

Keywords: Falling Weight Deflectometer, Pavement Overlays; Low Volume Roads, Mechanistic-Empirical Pavement Design

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1 Background

Local agencies, including municipalities, counties and townships, often use empirical approaches based on layer coefficients for designing the hot-mix asphalt (HMA) overlay thickness for low volume pavements. For example, the Illinois Department of Transportation (IDOT) Bureau of Local Roads and Streets Manual (2012) provides a modified layer coefficients method based on a purely empirical approach with assumed layer coefficient values for a limited number of material types. Although such empirical approaches are fairly simple to use, they are often not suitable for considering the effects of recycled/reclaimed and/or nontraditional construction materials currently considered with sustainable pavement applications and has been found to be inefficient in characterizing modern construction materials (Sarker et al. 2015).

The lack of mechanical testing for evaluating the pavement structural condition often leads to uneconomical practices as far as the rehabilitation of low volume roads is concerned. Although the NDT-based overlay thickness design method specified by the 1993 AASHTO Pavement Design Guide (AASHTO 1993) uses Falling Weight Deflectometer (FWD) deflection data, it is primarily based on the concept of Structural Numbers (SN), which were developed from the AASHO Road Test conducted more than five decades ago and are inherently empirical in nature. Also, Asphalt Institute (AI) Deflection Method, requires several parameters such as Benkelman beam deflection measurements and projected overlay traffic to determine the design overlay thickness using available design charts (AI 1996). The AI deflection-based approach also requires an additional critical season conversion adjustment factor, a parameter that may not be available due to lack of yearly records of measured deflections. However, with the increased prevalence of mechanistic-empirical pavement design approaches, it is necessary for the overlay thickness design methods for low volume roads to have a mechanistic foundation as well.

The objective of this paper is to demonstrate the advantages of nondestructive testing (NDT) and pavement structural evaluation and to develop improved overlay thickness design alternatives for low volume roads in Illinois. In a recent research study at the University of Illinois, 20 pavement sections were selected from six counties in Illinois with varying structural and traffic characteristics. Falling weight deflectometer (FWD) tests were conducted on these road segments to determine and monitor the structural conditions of existing, in-service pavement sections. Accordingly, a new mechanistic-empirical (M-E) overlay design method was developed to adequately assess the structural conditions of existing pavements to 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. This paper present the results obtained using the newly developed M-E overlay design method, which successfully identified structural deficiencies in the original pavement configurations through FWD NDT and subsequently resulted in reliable overlay solutions, as compared to the AASHTO 1993 NDT method (AASHTO 1993), the IDOT modified layer coefficients method (IDOT BLRS Manual 2012), and the Asphalt Institute deflection approach (AI 1996)—that are currently used by the local agencies.

2 Research Methodology

2.1 Site Description

Twenty pavement sections in six Illinois counties were selected for FWD-based structural condition evaluations and subsequent overlay thickness designs after a careful review of the pavement layer configurations, design traffic levels, and maintenance schedules of local agencies. During the selection process, primary emphasis was given to pavement sections that displayed high-severity distresses and had already been selected by the local agencies for rehabilitation. The trailer-mounted Dynatest FWD was used in this study with a standard configuration of geophones placed at 0, 0, 12, 24, 36, 48, 60, and 72 in., respectively, from the center of the loading plate (plate radius = 6 in.). Typically, FWD tests along a given road segment were conducted at 200-ft intervals on the outer wheel paths. Pavement surface temperature was collected during the testing at every 2000-ft interval along the testing lane. Table 1 presents the locations and study details of the selected pavement test sections. Please note that Sections 10 and 11 in Vermilion County and Sections 15 through 17 in Champaign County were part of the same road segment, Perrysville Road and CH1 Dewey-Fisher Road, respectively, with constant layer thicknesses but with varying traffic conditions.

Location in Illinois	Road Name	No. of Sections	Surface (in.)	Base (in.)	Subbase (in.)	Pavement Condition
McHenry County, Coral Township	East Coral Road	1	2.25	11.5	-	Severely Cracked
		2	2	10.75	-	Severely Cracked
	Church Road	3	1.5	12	-	Severely Cracked
		4	2	7.5	6	Severely Cracked
City of DeKalb	Twombly Road	5	5	12	-	Severely Cracked
Village of Tinley Park	Normandy Drive	6	3.5	10	-	Severely Cracked
	Dorothy Lane	7	3.5	10	-	Moderately Cracked
Vermilion County	Perrysville Road	8	6.25	-	-	Moderately Cracked
		9	9.5	-	-	Moderately Cracked
		10-11	6.5	-	-	Moderately Cracked
		12	8.5	-	-	Moderately Cracked
		13	7.25	-	-	Moderately Cracked
		14	8.75	-	-	Moderately Cracked
Champaign County	CH 1 Dewey–Fisher Road	15-17	7.75	8	-	Very Few Cracks
Ogle County	S. Pines Road	18	6.5	4	8	Moderately Cracked
		19	6.5	12		Moderately Cracked
		20	6.5	4	8	Moderately Cracked

Table 1: Details of the in-service pavements studied

2.2 Proposed Mechanistic-Empirical Design Approach

The first step in the proposed Mechanistic-Empirical Design approach was to conduct an accurate evaluation to assess the current structural capacities of selected pavement sections. In order to achieve that, extensively tested and validated ILLI-PAVE finite element (FE) pavement analysis program (Raad and Figueroa 1980) was used to establish characteristics of the individual pavement 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 was to adjust the layer moduli in such a way that the original field deflection basin could be modeled properly. Individual layer moduli in the pavement sections 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. In addition, when applicable, ANN-Pro, a neural network based backcalculation software program developed by the researchers at the University of Illinois at Urbana-Champaign was also used as an alternative to ILLI-PAVE to characterize the pavement layers (Pekcan et al. 2008). 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 and can analyze a pavement system with up to 3 layers. The next step in the analyses was to adjust backcalculated HMA modulus to a reference temperature. In this study the temperature prediction model developed by Park et al. (2001) was employed to calculate mid-depth HMA temperatures from the pavement surface temperatures measured during FWD testing (Equation 1). 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.

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

Where,
Tz : AC pavement temperature at depth z, °C
Tsurf : 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.

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

Where,
ATAF : Asphalt temperature adjustment factor
Slope : Slope of the log Modulus versus Temperature curve
Tr : Reference temperature of 21°C
Tm : Pavement temperature at mid-depth (°C).

Section Number	HMA Modulus (ksi)	Adjusted HMA Modulus (ksi) (Temperature Corrected for 70°F)	Average 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	100	287	109	N/A	7.9
9	100	280	109	N/A	11
10	80	354	132	N/A	7.8
11	90	320	118	N/A	8.5
12	90	312	117	N/A	8.5
13	80	409	133	N/A	7.5
14	80	399	133	N/A	6.8
15	775	279	75	K_{base}=7, n_{base}=0.5 $K_{subbase}$=5, $n_{subbase}$=0.5	15
16	775	358	79	K=6, n=0.5	15
17	775	257	82	K_{base}=5.8, n_{base}=0.5 $K_{subbase}$=2, $n_{subbase}$=0.5	17.9
18	250	526	99	K_{base}=7, n_{base}=0.5 $K_{subbase}$=5, $n_{subbase}$=0.5	15
19	300	537	93	K=6, n=0.5	15
20	200	472	104	K_{base}=5.8, n_{base}=0.5 $K_{subbase}$=2, n_{base}=0.5	17.9

Table 2: Iteratively calculated layer moduli using ILLI-PAVE to match FWD deflection basins

Table 2 lists all the iteratively calculated layer modulus values using ILLI-PAVE and ANN-Pro along with the necessary temperature corrections applied. Upon completion of the layer property characterizations, the structural conditions of the pavement sections were evaluated using critical pavement responses (tensile strain at the bottom of the asphalt layer, εt; and vertical surface deflection under the load, δv) 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).

$$N_f = \frac{8.78 \times 10^{-8}}{(\varepsilon_t)^{3.5}} \tag{3}$$

$$N_f = \frac{5.73 \times 10^{10}}{(\delta_v)^4} \tag{4}$$

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	256,365	2.74E-4	21.74	7.65E-4	40.34	YES
9	310,336	2.60E-4	20.73	4E-4	24.37	YES
10	310,336	2.60E-4	20.73	8.43E-4	42.84	YES
11	310,336	2.60E-4	20.73	7.60E-4	38.9	YES
12	310,336	2.60E-4	20.73	5.54E-4	32.26	YES
13	310,336	2.60E-4	20.73	7.55E-4	40.71	YES
14	310,336	2.60E-4	20.73	6.27E-4	37.97	YES
15	1,519,234	1.64E-4	13.94	1.19E-4	11.21	NO
16	1,556,746	1.63E-4	13.85	1.19E-4	11.21	NO
17	1,350,430	1.71E-4	14.35	1.19E-4	11.21	NO
18	437,311	2.36E-4	19.03	2.75E-4	18.39	YES
19	437,311	2.36E-4	19.03	2.58E-4	17.26	YES
20	437,311	2.36E-4	19.03	3.71E-4	22.63	YES

Table 3: Critical pavement responses compared to the threshold values for design traffic levels along with required HMA thicknesses

Whether the pavement section requires an overlay or not was determined by comparing the εt and δv values under the current pavement configuration with the threshold values calculated using Equations 3 and 4. The threshold values of tensile strain at the bottom of asphalt layer (εt) and vertical surface deflection (δv), along with the corresponding values under different FWD tests are listed in Table 3. As can be seen, the M-E overlay design method adequately captures the structural inadequacies of the pavement sections for the original pavement configurations. Sections 5 and 20, and Sections 9 through 14 fail both under the fatigue as well as rutting algorithms. Sections 1 through

4 and Section 8, on the other hand, prove to be adequate for the fatigue performance but fail under the rutting criterion. Sections 18 and 19, however, seem to be adequate for rutting performance, but fail under the fatigue criterion. Section 6, 7, and Section 15 through 17 are found to be structurally adequate to carry on the projected traffic load. After the requirement of overlay for the sections were established, the overlay thickness was iteratively adjusted using both ILLI-PAVE and ANN-Pro (when applicable) to ensure that the new pavement system will meet the threshold critical pavement responses as presented in Table 3.

Section	Proposed M-E Overlay Method (in.)	IDOT Modified Layer Coefficients Method (in.)	1993 AASHTO NDT Method (in.)	AI Deflection Method (in.)
1	2	2	No Overlay Required	No Overlay Required
2	2	2	No Overlay Required	No Overlay Required
3	2	2	No Overlay Required	No Overlay Required
4	2	2	No Overlay Required	No Overlay Required
5	2.5	3	0.35	2
6	No Overlay Required	2	No Overlay Required	No Overlay Required
7	No Overlay Required	2	No Overlay Required	No Overlay Required
8	2	3	No Overlay Required	2.3
9	1.5	3	No Overlay Required	No Overlay Required
10	2	3	No Overlay Required	2.3
11	2	3	No Overlay Required	2
12	2	3	No Overlay Required	1.5
13	1.25	3	No Overlay Required	2.6
14	1.25	3	No Overlay Required	2.3
15	No Overlay Required	4	No Overlay Required	No Overlay Required
16	No Overlay Required	4	No Overlay Required	No Overlay Required
17	No Overlay Required	4	No Overlay Required	No Overlay Required
18	2	3	No Overlay Required	No Overlay Required
19	1.25	3	No Overlay Required	No Overlay Required
20	2	3	No Overlay Required	No Overlay Required

Table 4: Summary of required overlay thicknesses by all the methods

Table 4 lists the summary of required overlay thicknesses by all the methods. As can be seen in Table 4, many pavement sections, such as Sections 5 through 20, required lower thickness requirements than those calculated by the IDOT method. Please note that thicknesses calculated by the IDOT method are the minimum thicknesses as suggested by the BLRS manual (2012). However, all but one of the tested pavement sections were erroneously categorized as structurally adequate by the 1993 AASHTO NDT method. The somewhat erroneous categorization of the pavement sections as structurally adequate by the AASHTO method can be attributed to the significantly low design traffic volumes for these pavement sections (Sections 1 through 4).

Given identical material properties and layer configurations, the required structural number would also increase with increasing traffic, thus making the current pavement inadequate structurally as well. Additionally, the layer coefficients used in the IDOT method are empirical in nature and have been established for a limited number of materials. Accordingly, the use of this method for structural evaluation of pavements constructed with new, recycled and/or nontraditional materials is questionable at best. Also the AI deflection methods characterized most of these sections (Sections 1 through 4, Sections 6, 7, and 9 and Sections 15 through 20) as structurally adequate to carry the intended traffic volume and subsequently resulted in no thickness requirements.

Please note that the AI deflection method requires the use of sophisticated conversion factors, which were simply assumed in this study because of the unavailability of a continuous record of yearly deflection data from the test sections. All of these factors could have attributed to erroneous characterization of the existing pavement structural capacity and resulted in an inaccurate overlay thickness requirement. Sections 6, 7, 15, 16, and 17 did not require any form of overlay according to the proposed M-E overlay design method. All of these sections were showing very few to moderate surface distresses during the time of testing and the M-E method successfully captured the structural adequacy of these pavement sections.

3 Summary and Conclusions

This paper presented findings from a recently completed research study at the University of Illinois aimed at improving the overlay thickness design methods for low volume road pavements in Illinois. FWD tests were conducted on twenty road sections in Illinois 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 evaluate the structural conditions of existing, in-service pavements and subsequently recommend required thickness values from FWD-based analyses. The M-E overlay design method was found to adequately assess the structural conditions of existing pavements and subsequently recommend required overlay thickness values from FWD-based critical pavement responses computed and compared to threshold values for the pre-established fatigue and/or rutting damage algorithms. All but one of the tested pavement sections were erroneously categorized as structurally adequate by the 1993 AASHTO NDT method. Similarly, the modified layer coefficient–based IDOT method used in Illinois, being highly empirical, predicted rather thicker overlays for approximately half of the pavement sections, when compared to the M-E overlay design method. The AI deflection method required the use of sophisticated conversion factors, which were assumed in this study because of the unavailability of a continuous record of yearly deflection data in the test sections. This approach made the proper use of the AI deflection method somewhat questionable when the periodic FWD deflection data were not available. Most of the sections had thinner overlay requirements following the proposed M-E overlay design method, when compared to those based on the minimum thickness requirement by the IDOT method. The M-E overlay design method was found to successfully identify structural deficiencies in the original pavement configurations through FWD NDT and led to reliable overlay solutions, as compared to the AASHTO 1993 NDT, AI Deflection, and IDOT methods. Local agencies should be encouraged to use FWD testing to assist in the determination of rehabilitation strategies for low volume roadways in Illinois as the use of the proposed M-E overlay design will determine the most economic and reliable rehabilitation solutions resulting in a significant improvement over the methods currently used to design overlays for low volume roads in Illinois.

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