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DEVELOPMENT OF IMPROVED OVERLAY THICKNESS DESIGN ALTERNATIVES FOR LOCAL ROADS

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16. Abstract

In this research study, 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 both the existing and HMA overlaid pavement sections. Then the corresponding required overlay thicknesses were determined using three different methods—the AASHTO 1993 nondestructive testing (NDT) method, the IDOT modified layer coefficients method, and the Asphalt Institute deflection approach—that are currently used by local agencies including municipalities, counties and townships. Inadequacies of the currently available methods to properly account for the pavement structural conditions were highlighted. 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 with 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 subsequently resulted in reliable and cost-effective overlay solutions, as compared with the IDOT modified layer coefficients method.

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EXECUTIVE SUMMARY

The objective of this research project is to demonstrate the advantages of nondestructive testing (NDT) and pavement structural evaluation and to develop improved overlay thickness design alternatives for local roads in Illinois. 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.

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. In this research study, 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 both the existing and HMA overlaid pavement sections. Then the corresponding required overlay thicknesses were determined using three different methods—the AASHTO 1993 NDT method, the IDOT modified layer coefficients method, and the Asphalt Institute deflection approach—that are currently used by local agencies. Inadequacies of the currently available methods to properly account for the pavement structural conditions were highlighted.

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 with 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 subsequently resulted in reliable and cost-effective overlay solutions, as compared with the IDOT modified layer coefficients method.

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CHAPTER 1 INTRODUCTION

1.1 BACKGROUND AND MOTIVATION

Each year, local and state agencies make substantial investments in evaluating the conditions of existing, in-service pavements. In addition to enumerating functional deficiencies, these agencies need to evaluate the structural condition of a pavement through the use of proper nondestructive testing and sensor technologies so that adequate rehabilitation options can be formulated with maximum cost savings. Adequate maintenance of existing pavement structures and design/implementation of suitable rehabilitative approaches through structural capacity assessments are critical to ensuring long-lasting, cost-effective pavement systems.

One of the most common maintenance and rehabilitation approaches for flexible pavements involves the placement of 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 follow: (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 is the modified layer coefficient–based approach used by the Illinois Department of Transportation (IDOT).

Based on the daily traffic volume, low-volume roads in Illinois can be subdivided into two classes: (1) Class III: daily traffic volume ranging between 400 and 2,000; and (2) Class IV: daily traffic volume under 400. The modified layer coefficient–based overlay thickness design method used in Illinois for low-volume roads relies on the use of empirical layer coefficients described in the *1972 AASHTO Interim Guide for Design of Pavement Structures*. Although this empirical approach is fairly simple to use, it has been established by researchers to be inefficient in characterizing modern construction materials such as recycled and nontraditional aggregates. Moreover, accurate assessment of the current structural conditions of existing pavement structures is essential for economical design of HMA overlays. The benefits of using NDT-based overlay design methods can be summarized as follow (Kinchen and Temple 1980):

- Less relying on human judgment for estimating pavement strength and structural capacity
- Direct estimation of existing pavement layer moduli without laboratory testing
- Lower cost, as the expenses and inaccuracies associated with destructive testing of pavement designs components are no longer required
- Provision of HMA overlay designs that more accurately match the expected design life

Although the NDT-based overlay thickness design method specified by the *1993 AASHTO Guide for Design of Pavement Structures* uses FWD deflection data, it is primarily based on the concept of structural numbers (SN), which is inherently empirical in nature and developed from the AASHO road test field study conducted nearly six decades ago. With the increased prevalence of mechanistic-empirical 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 to take into account local conditions and pavement damage mechanisms can constitute the empirical component of such methods.

1.2 RESEARCH OBJECTIVE

The primary objective of this research study is to develop and compare HMA overlay thickness designs for a number of local road and street projects using (1) the currently used modified AASHTO *(Bureau of Local Roads and Streets Manual),* (2) Asphalt Institute, and finally, (3) nondestructive FWD testing and layer modulus backcalculation-based, mechanistic design procedures, such as the *1993 AASHTO Guide for Design of Pavement Structures* procedure, to demonstrate advantages and disadvantages of each approach and to document the estimated construction cost of each design alternative. Related to the third category, this research study also aims to develop an improved overlay thickness design method for low-volume roads in the state of Illinois using the concepts of mechanistic-empirical (M-E) pavement design and FWD testing. The method developed, referred to as the M-E overlay design method, uses critical pavement response parameters obtained from FWD testing, along with a calibrated pavement damage algorithm, to estimate mechanistically the required overlay thicknesses. Results from the newly developed method will be compared with other methods currently used by local agencies, such as the modified AASHTO method (based on structural numbers and layer coefficients), the Asphalt Institute method, and the AASHTO 1993 NDT method. By formulating an improved HMA overlay design solution for resurfaced, recycled, and reclaimed pavement alternatives to be incorporated into the Illinois Department of Transportation (IDOT) *Bureau of Local Roads and Streets (BLRS) Manual,* this research is also intended to aid the design and construction of sustainable economic pavement structures.

Because the modified AASHTO coefficients are outdated and do not reflect modern materials, a goal in this research is to demonstrate that, through proper structural evaluation using FWD testing, significant cost savings can be realized in HMA overlay thickness designs for local roads. Accurate structural condition assessment of existing, in-service pavements through FWD or rolling wheel deflectometer (RWD) testing is anticipated to result in thinner overlays for roads deemed structurally adequate. For those found to be structurally inadequate or failing through FWD testing, the need for thicker overlays will be revealed through the use of the M-E overlay design method, thus making the current modified AASHTO approach less reliable when selecting a rehabilitation strategy. Accordingly, improved design reliability and performance will be achieved because mechanistic analysis and design concepts will be fully implemented in the development of HMA overlay structural thickness designs through the use of the M-E overlay design method. Based on the results of design evaluations and research findings, Chapter 46 of the BLRS manual can be revised to provide better guidance for local agencies in designing cost-effective overlays.

1.3 RESEARCH METHODOLOGY

The research was performed following the major tasks for reaching the study goals:

- Task 1—*Evaluate Existing Overlay Design Procedures*: Current rehabilitation design procedures used by the local and state highway agencies will be studied for their applicability to local roads in accordance with the IDOT BLRS manual and how structural conditions of existing pavements are evaluated for overlay design. Using advanced statistical methods, sensitivity analyses will be performed to determine the effect of each input design parameter on the final HMA overlay thickness in any specific design method.

- Task 2—*Establish Local Road and Street Field Demonstration Projects*: Several local agency rehabilitation project sites will be selected to conduct FWD testing and collect test data for evaluating structural conditions of in-service, hot-mix asphalt (HMA) pavements in Illinois. IDOT's Dynatest FWD machine currently at ATREL will be reassembled and used in this research for the FWD testing of local agency rehabilitation projects. Nondestructive field FWD data will be collected, along with pavement geometry and materials data and all other details specific to each rehabilitation project to be studied. This task will also include developing HMA overlay thickness designs for each project to be studied, using currently available design procedures, such as the modified AASHTO, in the IDOT BLRS manual.
- Task 3—*Develop Improved Overlay Procedures Based on FWD Testing and M-E Concepts:* The field FWD data collected in Task 2 will be analyzed using the previously developed M-E approaches for layer modulus and critical pavement response backcalculation. This approach will facilitate a proper assessment of the structural conditions of the existing, in-service pavements, which is a key step for estimating the pavement structural capacity and developing improved overlay thickness designs, using HMA fatigue and rutting transfer functions established by the IDOT Bureau of Materials and Physical Research and Bureau of Local Roads and Streets. Final overlay thicknesses will be compared with thicknesses determined from other currently available design procedures, such as the modified AASHTO in the IDOT BLRS manual, to highlight the benefits from the FWD testing and M-E based pavement layer moduli and response backcalculation.
- Task 4—*Compare Costs of Design Alternatives*: Cost comparisons will be established for the overlay thickness designs developed in Tasks 2 and 3 for the local road and street rehabilitation projects studied, to contrast adequacies and inadequacies of the current pavement rehabilitation design practices and procedures and the newly developed FWD testing and backcalculation-based HMA overlay thickness design alternative. As many local agencies are resistant to using the FWD testing because of the initial cost of paying a consulting engineer and/or a lack of understanding of the process, the findings of this task will determine which design method provides overall the most economical design.

1.4 REPORT ORGANIZATION

Chapter 2 of this report includes an introduction on FWD testing as the most popular pavement nondestructive testing and evaluation approach and discusses backcalculation analysis approaches for FWD data. An overview of the current overlay procedures is also presented in Chapter 2, along with the outcomes of the sensitivity analyses conducted on existing overlay design procedure, to determine the effect of each input design parameter on the final HMA overlay thickness in any specific design method. Details of selected case studies are presented in Chapter 3. Chapter 4 presents the research approach adopted in the development of the M-E overlay design method and compares the determined overlay thicknesses from different rehabilitation procedures. Chapter 5 includes a summary of conclusions and recommendations based on the research study findings.

CHAPTER 2 BACKGROUND AND LITERATURE REVIEW

2.1 INTRODUCTION

The structural evaluation of existing, in-service pavements depends heavily upon an accurate determination of the layer properties, i.e., pavement layer moduli, evaluated either by destructive or nondestructive means. In recent years, NDT methods have established themselves as a reliable means to assess the structural condition of an existing pavement, as they are quite easy to use, they are repeatable, and they can be performed much more rapidly than destructive tests. In addition, an overall cost reduction is typically achieved through this approach in the long run, thus making them advantageous over destructive testing of pavements. However, an accurate determination of pavement layer stiffness or modulus and layer thickness from the test results depends on the reliability of NDT methods. One of the most popular NDT methods to evaluate pavements is falling weight deflectometer (FWD) testing. FWD basically simulates the deflection of pavement caused by a fast-moving highway truck by means of dropping a certain weight on the pavement and measuring surface deflections. These surface deflections are later used to evaluate the structural capacity of the existing pavement system by determining pavement layer properties. In transportation/pavement engineering, this method is commonly referred to as backcalculation of layer moduli. This chapter presents an overview of FWD testing and the state-of-the-art backcalculation analysis approaches, followed by a summary of the currently available HMA overlay thickness design procedures. Sensitivity analyses are then conducted on existing overlay design procedures to determine the effect of each input design parameter on the final HMA overlay thickness in any specific design method.

2.2 OVERVIEW OF FALLING WEIGHT DEFLECTOMETER TESTING

Falling weight deflectometer (FWD) test equipment is a field NDT device that applies an impulsive load (usually between 110 and 660 lb) to pavement while recording the resulting vertical deflections on the pavement surface at different offset locations from the dropped load. It drops the specified weight from a given distance (up to 16 in.) to strike a buffered plate resting on the pavement surface (Figure 2.1). The load is then transmitted from the rubber buffers to the pavement through a 5.91 in.–radius steel plate underlain by a rubber pad. The rubber pad is installed to facilitate a uniform application of the load to the pavement surface. As shown in Figure 2.2, it simulates the same load duration of a vehicle travelling at 40 to 50 mph by producing a peak dynamic force (typically between 1,500 and 24,000 lb in 25 to 30 milliseconds) (Ullidtz and Stubstad 1985). A typical test configuration is shown in Figure 2.3.

Figure 2.1: Photo of the Dynatest falling weight deflectometer device at the University of Illinois.

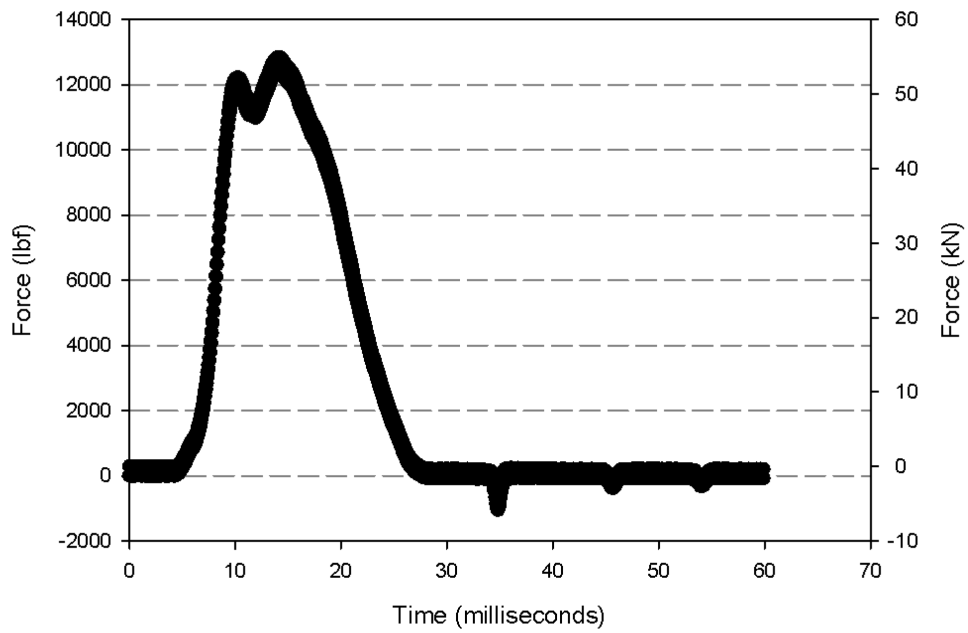


Figure 2.2: Haversine loading applied by FWD device.

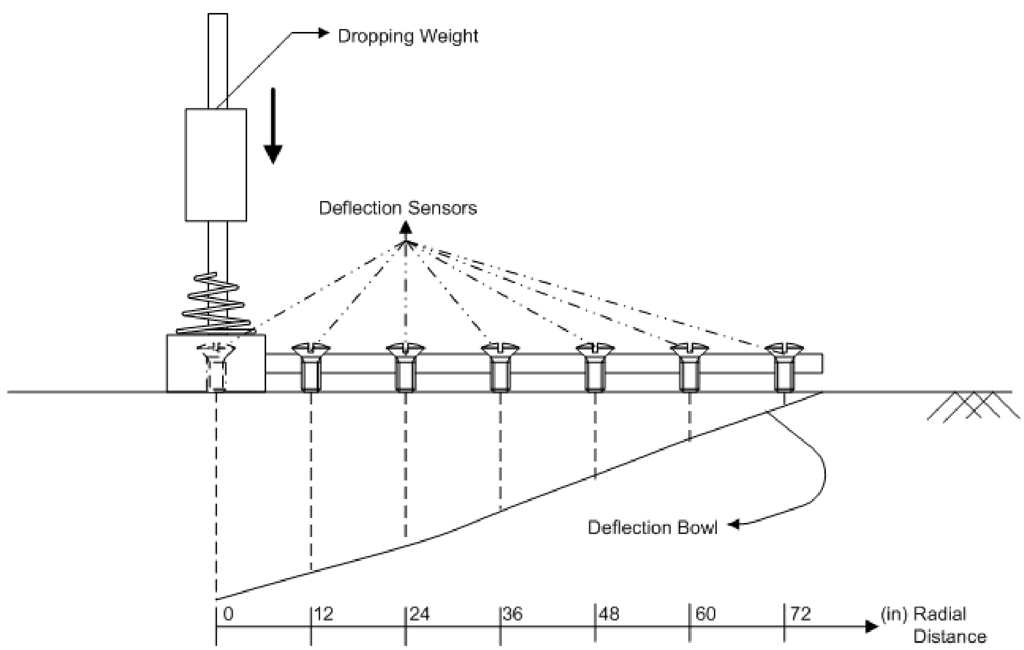


Figure 2.3: Locations of FWD sensors and schematic drawing.

Among all the other testing equipment, FWD's ability to best replicate the load histories and deflections of a moving vehicle among has made it a widely accepted tool worldwide (Hoffman and Thompson 1981, Roesset and Shao 1985, Ullidtz and Stubstad 1985). The magnitude and frequency of the loading are the two key parameters that can affect the deflection profile or basin obtained from FWD testing (Shahin 2005). Among many FWDs described in the literature, the three most commonly used and commercially available are the following:

- Dynatest Model 8000 (Dynatest Consulting, Inc.)
- KUAB FWD Models 50 and 150 (KUAB America)
- JILS FWD (Foundation Mechanics, Inc.)

2.3 BACKCALCULATION METHODS

Backcalculation in pavement analysis is a process in which NDT test results, such as those of FWDs, are used to infer layer properties, including the layer thickness and layer moduli. Though empirical methods are popular as well, backcalculation analysis approaches may be classified as follows:

- Simplified methods
- Gradient-relaxation methods
- Direct interpolation methods

These approaches have been used to develop many software applications that actually can reasonably accomplish backcalculation from FWD test results, using different assumptions of the elastic layered systems. Simplified and direct interpolation approaches are not popular because the typical numerical routines used for backcalculation may not properly iterate the moduli, as the local minima for the solution for a system can be numerous and global optimization may be required. These methods also pose the possibility of inaccurate solutions if the pavement layer properties are not in accord with the assumptions made. In spite of the drawbacks, however, the problem—if formulated correctly—leads to very reasonable solutions.

Gradient-relaxation methods are the most popular ones due to their nonlinear behavior in formulation of the algorithm. They employ mathematical models to describe the pavement condition. The process is to use a set of seed moduli (from experience or known values for standard layers) to determine deflections from a formulated model for the problem at hand and then to compare the estimated value with the experimental values from FWD testing (Figure 2.4). The trial-and-error method leads to extraction of reasonable layer properties in cases where the assumptions about the layer thickness, homogeneity, and other properties are quite in accord with the situation of the pavement. Hence, it is very important to design the algorithm in such a way as to take care of the variation from standard layer properties. Also, the nature of the problem should be understood thoroughly before designing the scheme for solution.

Flexible pavement layer moduli calculations can be performed using several well-known software programs among which MODULUS, EVERCALC, and ELMOD are the most commonly used. MODULUS and EVERCALC were developed by the Texas Transportation Institution and the Washington State Department of Transportation (WSDOT), respectively. WESLEA, a layered elastic solution platform by the U.S. Army Corps of Engineers included in MODULUS 6.0, performs the forward calculation for building a database of the computed deflection basin. This database is compared with measured deflections, using a pattern-search routine to determine the layer moduli in the pavement system. Flexible pavements with up to four unknown layers can be processed using MODULUS 6.0. Similar to MODULUS 6.0, EVERCALC also uses an iterative approach incorporating WESLEA as the forward engine to calculate the deflection basin, based on a given set of layer moduli. The measured and computed deflections are matched within a pre-specified root mean square (RMS) error range.

Using an optimization technique known as the augmented Gauss-Newton algorithm, EVERCALC can provide evaluations of layer moduli for up to five-layer pavement structures. Unlike EVERCALC and MODULUS 6.0, which use the WESLEA elastic layered program, another commonly used backcalculation software program, ELMOD4, uses the Odemark equivalent-thickness approach. Table 2.1 provides a summary of the key features of some of these backcalculation software programs.

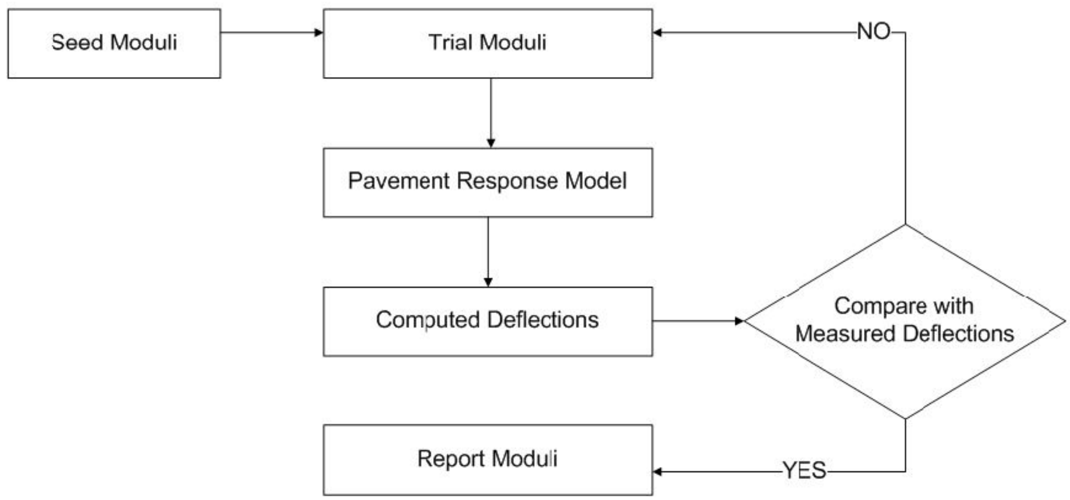


Figure 2.4: Traditional iterative backcalculation procedure (Meier 1995).

Table 2.1: Key Features of Popular Backcalculation Software Programs

Software Program	Forward Calculation Routine	Convergence Rule	Backcalculation Approach
MODULUS (Scullion et al. 1990)	Linear elastic approach, WESLEA	Root mean squared (RMS) error	Minimize the difference between the predicted and the measured basin by adjusting the modulus of the various layers through searching a database.
MICHBACK (Harichandran et al. 1993)	Linear elastic approach, CHEVRON	Root mean squared (RMS) error	Minimize the difference between the predicted and the measured basin by adjusting the modulus of the various layers through a number of iterations
MODCOMP (Irwin 2001)	Linear elastic approach, CHEVRON		
ELMOD	Odemark equivalent-thickness approach		
EVERCALC (Sivaneswaran et al. 1991)	Linear elastic approach, WESLEA		
WESDEF (Van Cauwelaert et al.1989)	Linear elastic approach, WESLEA		

To determine the pavement layer moduli, however, most of these traditional software programs use linear elastic solutions, which do not take account the nonlinear, stress-dependent behavior of fine-grained soils and aggregates. The recent ICT research study (R39-002) entitled "Nondestructive Pavement Evaluation Using ILLI-PAVE based Artificial Neural Network Models" developed a field-validated, nondestructive pavement evaluation toolbox that can be used for rapidly and accurately backcalculating in-service HMA pavement layer properties and thicknesses, as well as predicting critical stress, strain, and deformation responses of these in-service pavements from the measured FWD deflection basins (Pekcan et al. 2006, 2008, and 2009). The major advantage of using this toolbox is that the most accurate FWD backcalculation analysis results can be obtained at the push of a button, based on the sophisticated ILLI-PAVE finite element (FE) solutions. Note that the validated ILLI-PAVE FE program, developed by Thompson and Elliott (Thompson and Elliott 1985), analyzes full-depth and conventional flexible pavements by taking into account the nonlinear, stress-dependent behavior of subgrade soils and granular base materials. Incorporating advanced pavement material characterization and FE analysis into M-E overlay design methodology can essentially optimize the final HMA overlay thickness to ensure pavement infrastructure sustainability and provide substantial cost savings for local and state highway agencies.

Linear regression methods, ANNs (artificial neural networks), GAs (genetic algorithms), and other fuzzy systems are the primary nontraditional computational methods used for backcalculation. They are known as soft-computing methods and have become popular, as they provide nonuniversal, problem-specific solutions derived from artificially intelligent, self-learning computation capability. The recent ICT research study (R29-002) used ANNs and GAs in the software packages ANN-Pro and SOFTSYS developed for determining the most accurate FWD backcalculation analysis results, based on the sophisticated ILLI-PAVE FE solutions.

2.3.1 Artificial Neural Network (ANN)

ANN plays the role of an efficient pavement parameter analysis platform, and GA is a robust search and optimization system; in combination, they provide a process that is very fast (because of ANN) and stochastic (because of GA) to determine the parameters from FWD tests. A very powerful regression analysis system, ANN has been in use as both a forward analysis platform and backcalculation methods to provide useful information about the layer thickness and layer moduli including other parameters (Meier 1995, Meier et al. 1997, Ceylan et al. 2005, Pekcan et al. 2006 and 2008). FE methods such as the ILLI-PAVE program actually generate inputs and outputs to train ANN models for capturing the nonlinear behavior of the pavements (of various grades and characters) from FWD test results. At first, a broad range of input parameter space is generated and fed to the FE analysis module. The analyses help to establish a nonlinear relationship between the input parameters (layer properties) and the output variables (layer deflection values). These FE solutions are used to train the ANN model to capture nonlinear behavior of the system in a simulation environment. As advanced FE analysis by itself is slow, the simulation from a trained ANN model helps to generate rapidly the results with specified low errors in the estimations. ANN-Pro is one software program that uses ANN models to provide back-analysis solutions of measured FWD surface deflection data (Figure 2.5) (Pekcan et al. 2006, 2008, and 2009).

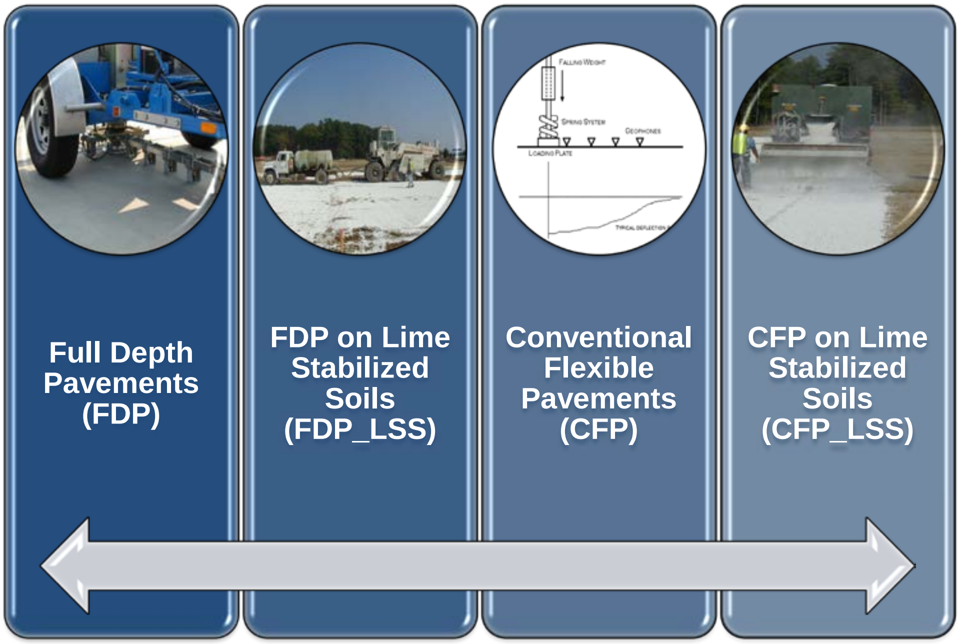


Figure 2.5: ANN-Pro software (Pekcan et al. 2009).

2.3.2 Genetic Algorithm (GA)

Nature-inspired, evolution-based GA is used to provide the optimization platform and sorting procedure for the inputs in a deflection-calculation model. Robust and imprecision-tolerant GA actually provides a solution space from structural model simulation and optimizes the parameter values to best match the experimental results through a fitness function:

$$Fitness = \frac{1}{1+\sum_{i=1}^{n}\frac{(D_{FWD,i}-D_{ANN,i})^2}{D_{FWD,i}}} \tag{2.1}$$

where D_{FWD} and D_{ANN} are deflection values obtained from FWD testing and ANN simulations of ILLI-PAVE FE solutions, respectively. The number *n* is the number of deflectometers used in the FWD testing and simulation.

Though ANN and GA approaches have been in use individually as powerful backcalculation methods for a period of time and provided reliable data analyzing capability, Pekcan and colleagues (Pekcan et al. 2006, 2008, and 2009) employed a combination of ANN, GA, and FEM (finite element method) in a software platform called soft computing–based system analyzer (SOFTSYS). This approach provided a way to analyze the condition and layer properties of various geomechanical systems. The algorithm of the hybrid model used in SOFTSYS is presented in Figure 2.6.

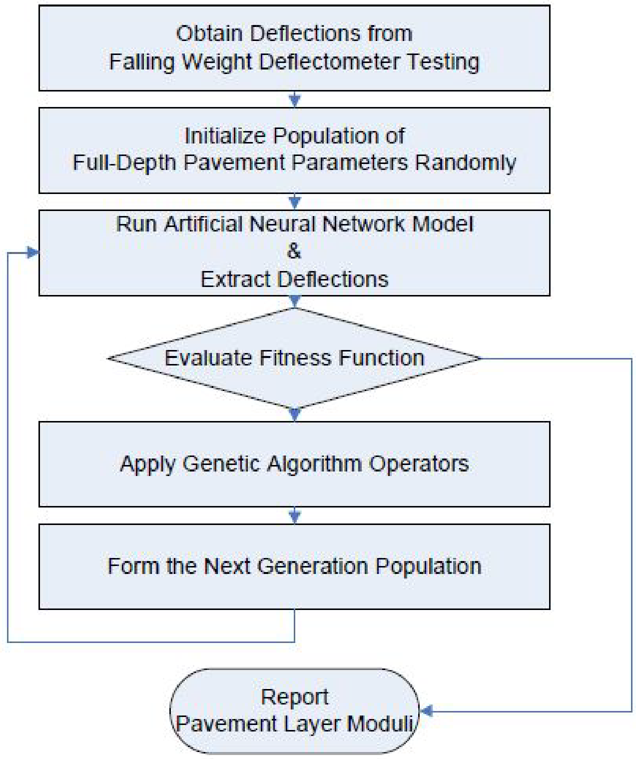


Figure 2.6: SOFTSYS algorithm flowchart (Pekcan et al. 2009).

2.3.3 ILLI-PAVE Finite Element Modeling and FWD Simulation Adopted by ICT Study R39-002

The ILLI-PAVE 2005 finite element (FE) program, the most recent version of this extensively tested and validated ILLI-PAVE pavement analysis program for over three decades, was used by the researchers at the University of Illinois, Pekcan et al. 2009) as an advanced structural model for solving deflection profiles and responses of the typical Illinois full-depth pavements (FDP) and conventional flexible pavements (CFP), full-depth pavements on lime-stabilized soils (FDP-LSS), and conventional flexible pavements on lime-stabilized soils (CFP-LSS). ILLI-PAVE uses an axisymmetric revolution of the cross section to model the layered flexible pavement structure. Unlike the linear elastic theory commonly used in pavement analysis, nonlinear unbound aggregate base and subgrade soil-characterization models are used in the ILLI-PAVE program to account for the typical hardening behavior of base-course granular materials and the softening nature of fine-grained subgrade soils under increasing stress. Among the several modifications implemented in the new ILLI-PAVE 2005 FE code are these:

- An increased number of elements (degrees of freedom)
- New/updated material models for the granular materials and subgrade soils
- Enhanced iterative solution methods
- Fortran 90 coding and compilation
- A new user-friendly Borland Delphi pre-/post-processing interface to assist in the analysis (Thompson et al. 2002) (Figure 2.7).

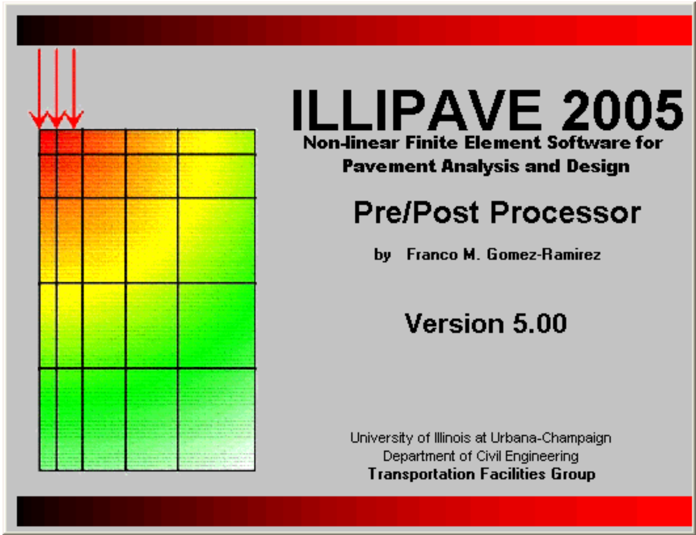


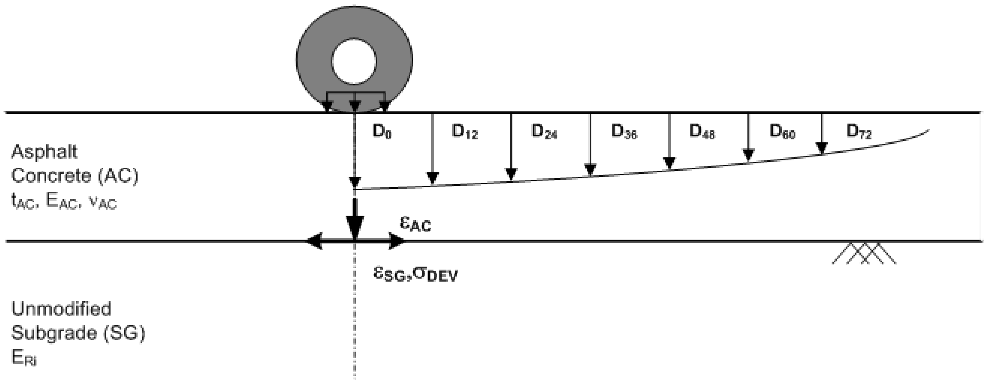
Figure 2.7: ILLI-PAVE 2005 finite element software for pavement analysis.

Pavement FE modeling was performed in ICT study R39-002 using an axisymmetric FE mesh for all pavement sections considered. Using the ILLI-PAVE FE program, FWD tests on flexible pavements were modeled with the standard 9 kip-equivalent, single-axle loading applied as uniform pressure of 80 psi over a circular area of 6-in. radius. The FE mesh was selected according to the uniform spacing option of the FWD sensors as follows: 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. away from the center of the FWD plate. The surface deflections corresponding to the locations of these FWD sensors were abbreviated as D_0, D_8, D_{12}, D_{18}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72}, respectively.

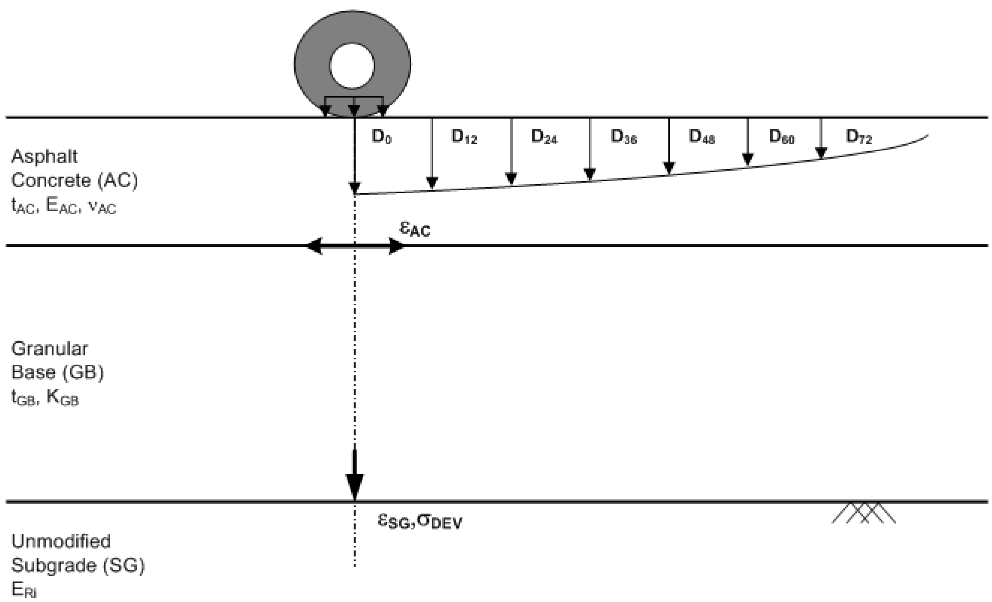
These deflections conform to the uniform spacing commonly used in FWD testing by many state highway agencies, including Illinois (Table 2.2). Typically, finer-mesh spacing was used in the loaded area, with the horizontal spacing adjusted according to the locations of the geophones used in FWD tests. In addition to the deflections, the critical pavement responses (i.e., horizontal strain at the bottom of AC layer (ε_{AC}), vertical strain at the top of the subgrade (ε_{SG}), and the vertical deviator stress on top of the subgrade (σ_{DEV}) directly at the centerline of the FWD loading) were also extracted from ILLI-PAVE results. Figure 2.8 (a to d) shows the locations of these responses obtained from different types of flexible pavements. These critical pavement responses play a crucial role in the context of mechanistic-empirical asphalt pavement design procedures, as they directly relate to major failure mechanisms because of excessive fatigue cracking and rutting in the wheel paths.

Table 2.2: Falling Weight Deflectometer Sensor Spacing

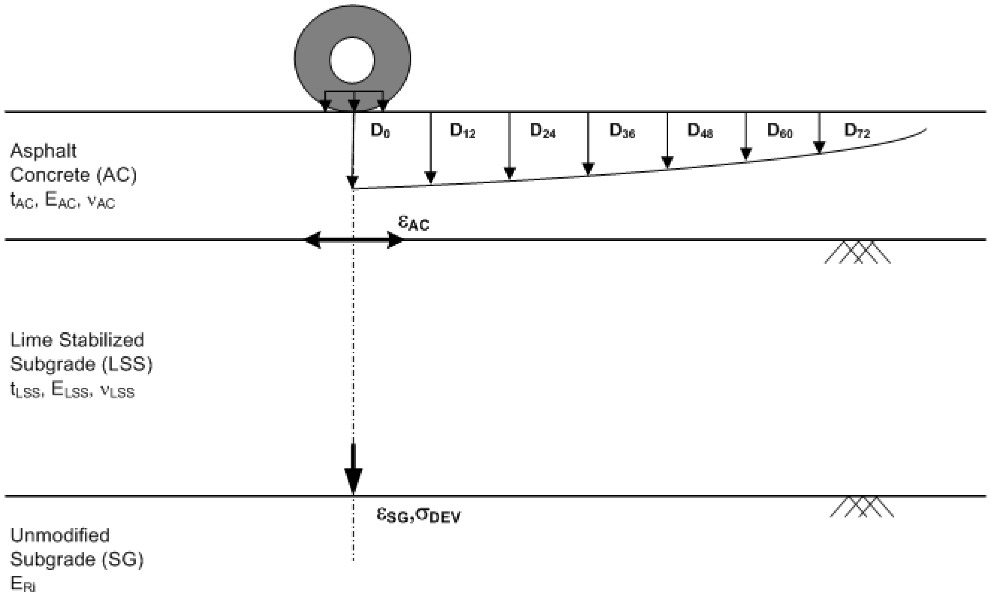
Sensor Spacing (in.)	0	8	12	18	24	36	48	60	72
Uniform (used in this and ICT study R39-002)	+		+		+	+	+	+	+
State Highway Research Program (SHRP)	+	+	+	+	+	+		+	



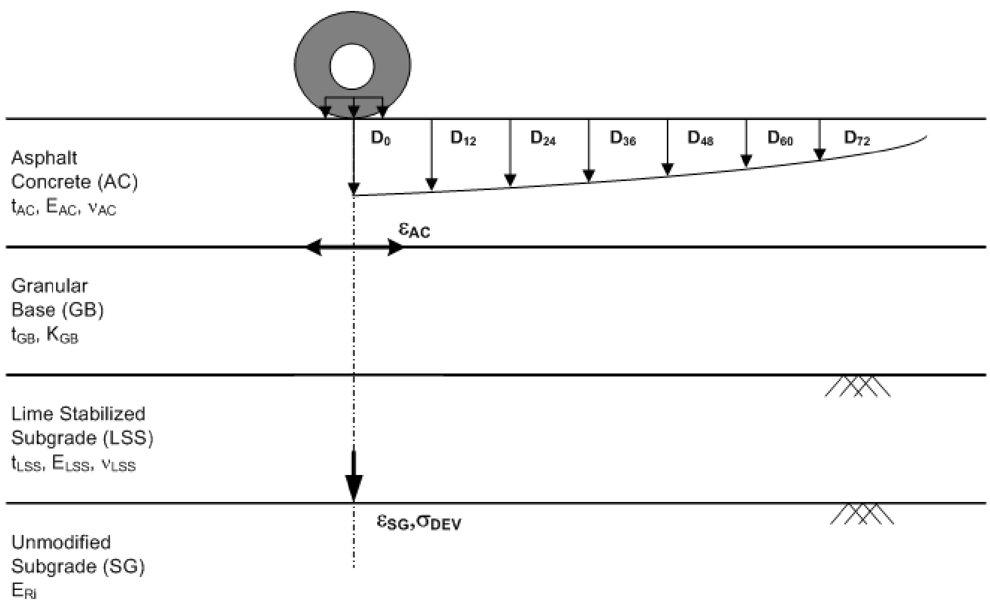
(a) full-depth asphalt pavements



(b) conventional flexible pavements



(c) full-depth asphalt pavements built on lime-stabilized soils



(d) conventional flexible pavements built on lime-stabilized soils

Figure 2.8: Locations of critical pavement responses and deflections.

A total analysis depth of 300 in. was selected for all pavements analyzed in ICT study R39-002. Depending on the thicknesses of the layers, an aspect ratio of 1 was mainly used in the finite elements, with a limiting value of 4 to get consistent pavement response predictions from ILLI-PAVE FE analyses

(Pekcan et al. 2006). The vertical and horizontal spacings in the FE mesh were chosen appropriately so there was neither numerical instability nor inconsistency in the results because of meshing. Figure 2.9 shows a sample ILLI-PAVE FE mesh that was used in the analyses of FDP-LSS. The thicknesses of all layers were selected to have appropriate ranges encountered for most flexible pavements in Illinois.

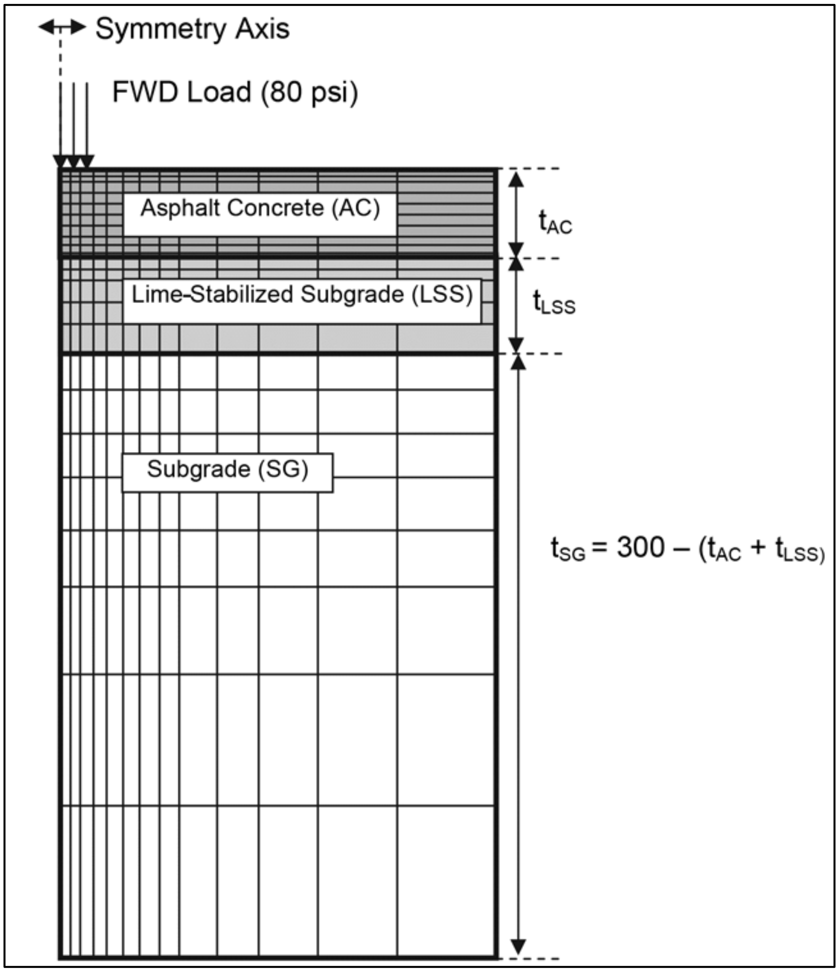


Figure 2.9: Example of FE mesh used for full-depth pavements on lime-stabilized subgrade.

Adequately characterizing pavement layer behavior plays a crucial role in an accurate backcalculation of the layer moduli. Accordingly, modeling of FDP and CFP requires accurate material characterizations for the asphalt concrete, granular base, and fine-grained subgrade soil layers. After material shakedown has taken place due to construction loading and early trafficking of the pavements, most of the deformations under a passing truck wheel are recoverable and hence considered resilient or elastic. The resilient modulus (M_R), defined as repeated wheel load stress divided by recoverable strain, is therefore the elastic modulus (E) often used to describe flexible pavement layer behavior under traffic loading.

In ILLI-PAVE FE models of the various flexible pavements analyzed in ICT study R39-002, the asphalt concrete (AC) surface course was always represented with elastic properties, layer modulus E_{AC} and

Poisson's Ratio ν_{AC}, for the instant loading during FWD testing. A constant of 0.35 was used for the value of ν_{AC}.

The modeling of fine-grained subgrade soils, mainly encountered in Illinois, has received more attention in the last three decades because it has a major impact on all the responses predicted under traffic loading within the context of M-E design. Fine-grained subgrade soils exhibit nonlinear behavior when subjected to traffic loading (Thompson and Robnett 1979, Ceylan et al. 2005). The subgrade stiffness characterized by the resilient modulus (M_R) is usually expressed as a function of the applied deviator stress through nonlinear modulus response models. These models were developed based on the results of repeated load triaxial tests, which form the basis of evaluating resilient properties of fine-grained soils (AASHTO-T307-99, 2000).

Illinois subgrade soils are mostly fine-grained, exhibit stress softening behavior, and can be characterized using the bilinear arithmetic model (Thompson and Robnett 1979, Thompson and Elliott 1985), with the modulus-deviator stress relationship shown in Figure 2.10. The upper-limit deviator stress in the bilinear model, σ_{dul}, is dependent on the breakpoint modulus, E_{Ri}, which is also a function of the unconfined compressive strength, Q_u, expressed by Equation 2.2 (Thompson and Robnett 1979). E_{Ri} is a characteristic property of the fine-grained soil that is often computed for Illinois soils at a breakpoint deviator stress σ_{di} of 6 psi. The corresponding values and parameters of the bilinear model used in the analyses are also given in Figure 2.10.

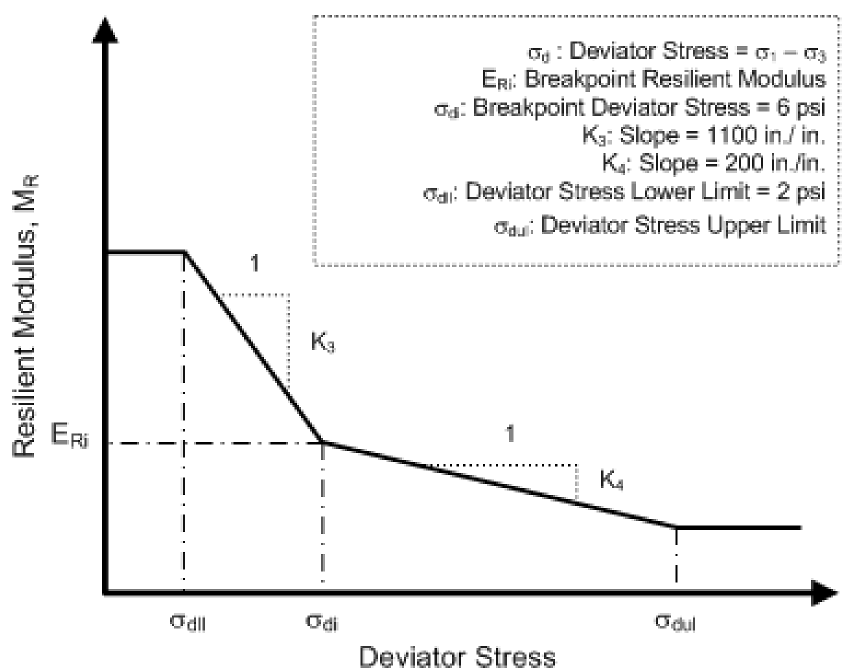


Figure 2.10: Bilinear model to characterize stress dependency of fine-grained soils.

$$\sigma_{dul}(psi) = Q_u(psi) = \frac{E_{RI} \cdot (ksi) - 0.86}{0.307} \quad (2.2)$$

The granular base (GB) layer provides the essential load transfer in a conventional flexible pavement. The effect of this layer is predominant in determining the fatigue behavior of AC layer. The well-known *K-θ* model (Hicks and Monismith 1971) was used in our modeling study to characterize the stress dependency of elastic (i.e., resilient) modulus in ILLI-PAVE analyses. In this model, the modulus stress dependency is accounted for by the use of two model parameters, *K* and *n.* The model parameter *n* is correlated to the *K* parameter according to Equation 2.3, where *K* is given in psi. A major advantage of the given equation is that the unbound aggregate modulus characterization model then requires only one model parameter. *K-θ* model parameters of different granular materials (*K* and *n* values) are also given in Table 2.3. Typical *K* values range from 3 ksi to 12 ksi, based on the comprehensive granular material database compiled by Rada and Witczak (1981) (Figure 2.11). Poisson's ratio was taken as 0.35 when $K \geq 5$ ksi; otherwise it was assumed to be 0.40.

$$\log_{10}(K) = 4.657 - 1.807 * n \quad (2.3)$$

Table 2.3: Typical Resilient Property Data for Granular Materials
(after Rada and Witczak 1981)

Granular Material Type	Number of Data Points	K (psi) *		n *	
		Mean	Standard Deviation	Mean	Standard Deviation
Silty Sands	8	1620	780	0.62	0.13
Sand-Gravel	37	4480	4300	0.53	0.17
Sand-Aggregate Blends	78	4350	2630	0.59	0.13
Crushed Stone	115	7210	7490	0.45	0.23

*$E_R = K\theta^n$ where E_R is resilient modulus and K, n are model parameters obtained from multiple-regression analyses of repeated load triaxial test data.

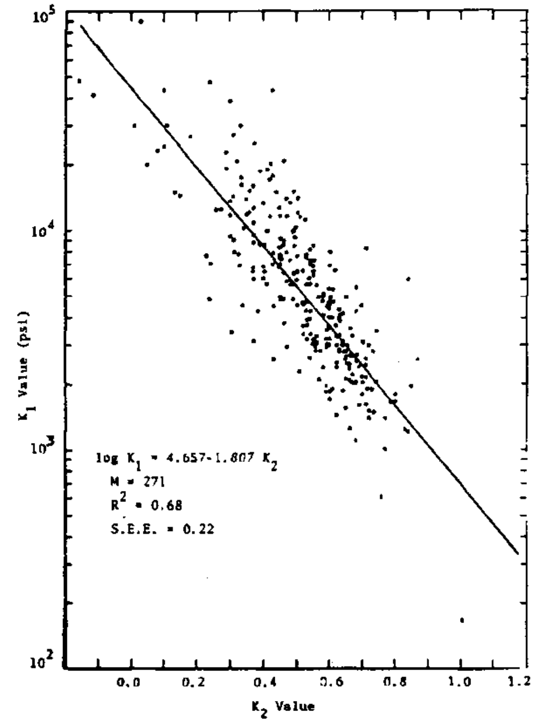


Figure 2.11: Relationship between *K* (shown as K_1) and *n* (shown as K_2) values for granular materials identified by Rada and Witczak (1981).

2.4 OVERVIEW OF CURRENT OVERLAY DESIGN PROCEDURES FOR FLEXIBLE PAVEMENTS

An extensive review of published literature was carried out to gather information on the state of the art and current state of practice in overlay thickness designs of flexible pavements. Based on the underlying principle, commonly used overlay thickness design methods can be classified into three broad categories:

- Methods based on the concept of structural deficiency
- Methods based on the concepts of maximum deflection and effective thickness
- Methods based on rutting and/or fatigue damage algorithms

Several state departments of transportation (DOTs) routinely conduct FWD testing for structural evaluation of in-service pavement structures. Some examples of state DOTs include Alabama, Arkansas, California, Idaho, Maryland, Minnesota, Mississippi, North Carolina, Ohio, South Carolina, Texas, and Washington (Kassabian 1992, Bayomy et al. 1996, Scullion and Michalak 1998, Skok et al. 2003, Wu and Gaspard 2009, WSDOT 2011). For local roads and streets, IDOT uses a modified version of the method recommended by the *1972 AASHTO Interim Guide for Design of Pavement Structures* incorporating the use of empirical layer coefficients for structural number (SN) calculations (AASHTO 1972). The following subsections present overviews of the most commonly used methods in each category.

2.4.1 Methods Based on the Concept of Structural Deficiency

2.4.1.1 The 1993 AASHTO NDT Method

The 1993 AASHTO NDT–based method uses FWD-obtained deflection basin information; subsequently, the subgrade resilient modulus (M_R), the required structural number (SN_{req}), and the projected traffic are determined using available charts. The effective structural number (SN_{eff}) of the existing pavement is calculated, and the difference between SN_{eff} and SN_{req} is used to determine the required overlay thickness using layer empirical coefficients. Equations 2.4 through 2.6 illustrate the steps in this design process. More details on the design approach can be found elsewhere (AASHTO 1993).

$$M_R = \frac{0.24 \times P}{d_r \times r} \tag{2.4}$$

$$d_0 = 1.5 \times p \times a \left\{ \frac{1}{M_R \left[1 + \left\{ \left(\frac{D}{a} \right) \left(\frac{E_p}{M_R} \right)^{\frac{1}{3}} \right\}^2 \right]^{\frac{1}{2}}} + \frac{1 - \frac{1}{\left(1 + \left\{ \frac{D}{a} \right\}^2 \right)^{\frac{1}{2}}}}{E_p} \right\} \tag{2.5}$$

$$SN_{eff} = 0.0045 * h_p * \left(E_p\right)^{0.33} \tag{2.6}$$

where

- M_R: backcalculated subgrade modulus
- d_0: center deflection normalized to P = 40 kN (9,000 lb) load and adjusted 20°C (68°F)
- d_r: deflection at r sensor distance from the center of the loading plate
- p: pressure (stress) on load plate
- a: radius of the load plate
- E_p: composite pavement modulus representing all layers above the subgrade
- D and h_p: total thickness of all layers above the subgrade

Once the SN_{eff} and SN_{req} are obtained, the required overlay thickness can be calculated using Equations 2.7 and 2.8.

$$SN_{ol} = SN_{req} - SN_{eff} \tag{2.7}$$

$$D_{ol} = \frac{SN_{req} - SN_{eff}}{a_{ol}} \tag{2.8}$$

where

SN_{ol}: required overlay structural number
a_{ol}: structural coefficient of the overlay material
D_{ol}: required overlay thickness

This procedure basically estimates the structural impact of the overlay in terms of effective structural number by adding the value of the overlay structure to the structural capacity of the existing pavement, as if the overlay were part of the original structure. However, if SN_{eff} is used to depict a pavement's structural condition, it does not necessarily portray the individual pavement layer moduli, meaning a layer with a higher modulus may not have a greater SN_{eff} than a layer with a lower modulus.

2.4.1.2 Illinois Department of Transportation (IDOT) Procedure

According to Chapter 46 of the IDOT *Bureau of Local Roads and Streets Manual,* the following steps are used to determine the thickness of an HMA overlay (BLRS 2012).

1. Determine traffic factor based on the facility class, average daily traffic, and design period.
2. Determine immediate bearing value (IBV; similar in concept to unsoaked CBR) based on the type of roadbed soil support.
3. Determine the required structural number (SN_f) using appropriate nomographs based on estimated traffic factor and existing soil support.
4. Determine the existing structural number using the following equation:

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3 \tag{2.9}$$

where, a_1, a_2, a_3 are empirical layer coefficients for the surface, base, and subbase layers, respectively; and D_1, D_2, and D_3 are the thicknesses for the surface, base, and subbase layers in the existing pavement. Although this approach is fairly simple to use, its primary limitation is the premise of the 1972 AASHTO approach, which assumes the statistically derived SN governs the structural capacity of the pavement associated with the use of empirical and often ambiguous layer coefficients.

2.4.2 Methods Based on the Concepts of Effective Thickness and Maximum Deflection

2.4.2.1 Asphalt Institute (AI) Method – I

The Asphalt Institute (AI) provides two design methods for the design of an HMA overlay on a conventional asphalt pavement (AI 1996). The first method, known as the effective thickness method, determines the required overlay thickness by subtracting the effective thickness of the existing pavement from the required thickness of a new full-depth asphalt pavement that carries the same traffic volume. Equation 2.10 illustrates the underlying concept for this method:

$$h_{ol} = h_n - h_e = h_n - \sum_{i=1}^{n} C_i h_n \tag{2.10}$$

where

h_{OL}: required asphalt overlay thickness
h_n: thickness of new full-depth asphalt pavement
h_e: effective thickness of the existing pavement
h_i: thickness of the i^{th} layer of the existing pavement
C_i: conversion factor associated with the i^{th} layer in the existing pavement structure
n: number of layers in the existing pavement structure

Although the effective thickness method is fairly simple to apply, the estimated required overlay thickness varies greatly, depending on the design conversion factors used, as these conversion factors are somewhat subjective.

2.4.2.2 Asphalt Institute Method – II

The second method proposed by the Asphalt Institute, known as the deflection method, requires the following parameters:

- Benkelman beam (static) deflection measurements
- Representative rebound deflection
- Projected overlay traffic
- Temperature adjustment factor
- Critical period adjustment factor

These parameters are used to determine the design overlay thickness by using a design chart that has a unique relationship established among the overlay thickness, the projected overlay traffic, and a corrected elastic deflection referred to as the representative rebound deflection.

2.4.3 Methods Based on Rutting and/or Fatigue Damage Algorithms

Several agencies such as the Idaho Transportation Department (ITD), Texas Department of Transportation (TxDOT), Minnesota Department of Transportation (MnDOT), and Washington Department of Transportation (WSDOT) have developed specialized software programs based on the combined use of pavement deflection data and damage algorithms (Bayomy et al. 1996, Scullion and Michalak 1998, Skok et al. 2003, 2011). The damage algorithms used by all the agencies mentioned above are primarily based on the empirical equations developed by the Asphalt Institute (AI) for asphalt cracking–based fatigue and subgrade rutting.

Although different state highway agencies have different methodologies for designing HMA overlay thicknesses, these design procedures essentially incorporate some form of modification to *1993 AASHTO Guide for Design of Pavement Structures* procedure, which is an empirical approach based on the concept of structural deficiency. Further, most of these design standards have been developed for high-volume roads; and very few pavement design procedures have been developed specifically for local roads and streets with low traffic volume (Zhao and Dennis 2007).

2.5 SENSITIVITY OF DESIGN PARAMETERS IN OVERLAY DESIGN PROCEDURES

Sensitivity analysis plays a crucial role in studying the behavior of a complex model to determine the variation of each input parameter's influence on the response of the model. It primarily observes how sensitive a system is to the variations of the system input parameters around their typical values.

Similar to many other pavement design problems, overlay thickness design may not have a unique solution. In other words, numerous design alternatives are possible even with the same input parameters. Therefore, for each overlay design approach, the effect of variability of the input factors, such as pavement layer properties, needs to be evaluated. Sensitivity analyses need to be performed to investigate the effect of each input design parameter on the final HMA overlay thickness in any specific design method.

Sensitivity analyses were performed to determine the effect of each input design parameter on the final HMA overlay thickness for the following design methods:

- Modified AASHTO design for overlays on existing flexible pavement (used by IDOT BLRS)
- 1993 AASHTO NDT method (used by the Ohio Department of Transportation, ODOT)
- Asphalt Institute deflection method.

2.5.1 Sensitivity Analysis: Modified AASHTO Layer Coefficients Design for Overlays on Existing Flexible Pavement

IDOT BLRS uses the modified AASHTO layer coefficients method to design overlays for the rehabilitation of deteriorated flexible pavements. This approach is based on determining the structural number (SN_f) of the pavement, i.e., structural capacity, based on the layer thickness and material properties. SN_f is basically used to express a pavement's load-carrying capacity for a certain combination of soil strength, known traffic volume, terminal serviceability, and environment factors.

A sensitivity analysis was performed to determine the effect of each input variable on the final HMA overlay thickness. The following input variables are essentially taken into consideration:

- Existing pavement layer thicknesses
- Structural design traffic (ADT)
- Immediate bearing value (IBV) of subgrade
- Layer coefficients

For convenience, pavement design period and type of highway were kept constant throughout the sensitivity analyses at 20 years and Class I, respectively. The pavement configuration presented in Figure 2.12, an example taken from the IDOT BLRS manual, was chosen as a base case. In addition, an ADT of 10,000, a design period of 20 years, and an IBV of 3 for subgrade were used in this base case scenario. Accordingly, Table 2.4 lists all the cases that were included in the analyses. Note that these layer coefficient values taken from the IDOT BLRS manual (Table 2.5) were used to calculate the structural number of an in-service pavement. The manual provides structural coefficients for a limited number of materials; and certainly, it is not adequate to address the structural capacity of a pavement built with nonconventional materials. Also, depending on the required structural number, the manual sets minimum requirements of the thickness of the overlay (2 to 4 in.) that must be installed on an existing pavement section regardless of the current structural condition of the pavement.

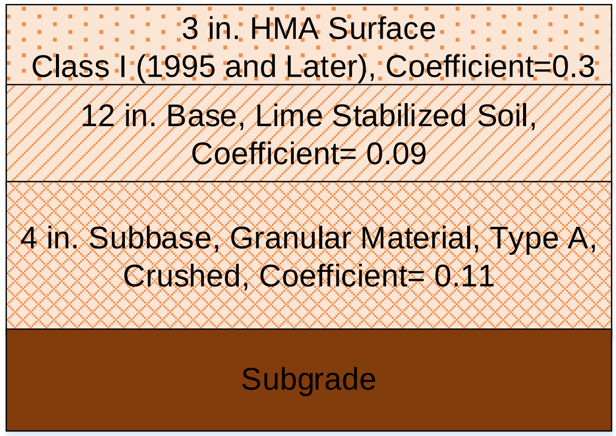


Figure 2.12: Pavement layer configuration used as a base case.

The methodology adopted to perform the sensitivity analyses was fairly simple. The effect of a unit change of the sensitivity variable on the final overlay thickness was calculated by changing one variable at a time while keeping all the other variables constant. Accordingly, the overlay thicknesses calculated for the various cases listed in Table 2.4 are presented in Figure 2.13. Note that the HMA overlay thicknesses required varied the most with changes in the layer coefficients, which were used to calculate the pavement load-carrying capacity.

Table 2.4: Case Studies Used in the Sensitivity Analyses

Case Numbers	Sensitivity Variable	Range of Values Considered
1–4	Surface Layer Coefficient	0.15–0.3
5–19	Base Layer Coefficient	0.08–0.25
20-22	Subbase Layer Coefficient	0.09–0.11
23–26	Surface Layer Thickness	3"–6"
27–31	Base Layer Thickness	9"–13"
32–36	Subbase Layer Thickness	4"–8"
37–41	Traffic Factor (TF)	0.4–1.5
42–45	Immediate Bearing Value (IBV)	3–9

Table 2.5: Structural Layer Coefficients from the IDOT BLRS Manual

STRUCTURAL MATERIALS	MINIMUM STRENGTH REQUIREMENTS			COEFFICIENTS[3]		
					Existing Material at the time of	
	MS[1]	IBV	CS[2]	In-Place Recycling[4]	1st Resurfacing	2nd Resurfacing or Recycling
Bituminous Surface				a_1	a_1'	a_1''
Road Mix (Class B)					0.15	0.11
Plant Mix (Class B): Liquid Asphalt					0.16	0.12
Plant Mix (Class B): Asphalt Cement	900				0.23	0.17
Class I (1954 and before)					0.23	0.17
Class I (1955 and later)	1700				0.30	0.23
HMA IL9.5 & IL12.5 (4% voids)				0.40	0.30	0.23
Base Course				a_2	a_2'	a_2''
Aggregate, Type B, Uncrushed		50			0.08	0.06
Aggregate, Type B, Crushed		80			0.10	0.08
Aggregate, Type A		80			0.10	0.08
Waterbound Macadam		110			0.11	0.09
Bituminous Stabilized Granular Material	300				0.12	0.09
	400				0.14	0.11
	800				0.17	0.13
	1000				0.19	0.15
	1200				0.21	0.16
	1500				0.23	0.17
	1700				0.25	0.20
CIR Recycling with Asphalt Products	1250			0.28		
FDR with Asphalt Products	1250			0.25	0.19	0.15
HMA Base Course					0.23	0.17
HMA IL19.0 (4% voids)				0.33	0.25	0.20
Pozzolanic, Type A			600		0.22	0.16
Lime Stabilized Soil			150		0.09	0.07
Select Soil Stabilized			300		0.12	0.09
with Cement			500		0.15	0.11
Cement Stabilized Granular Material			650		0.17	0.13
			750		0.19	0.15
			1000		0.22	0.16
Subbase Course				a_3	a_3'	a_3''
Granular Material, Type B		30			0.09	0.07
Granular Material, Type A, Uncrushed		50			0.10	0.08
Granular Material, Type A, Crushed		80			0.11	0.09
Lime Stabilized Soil			100		0.10	0.08

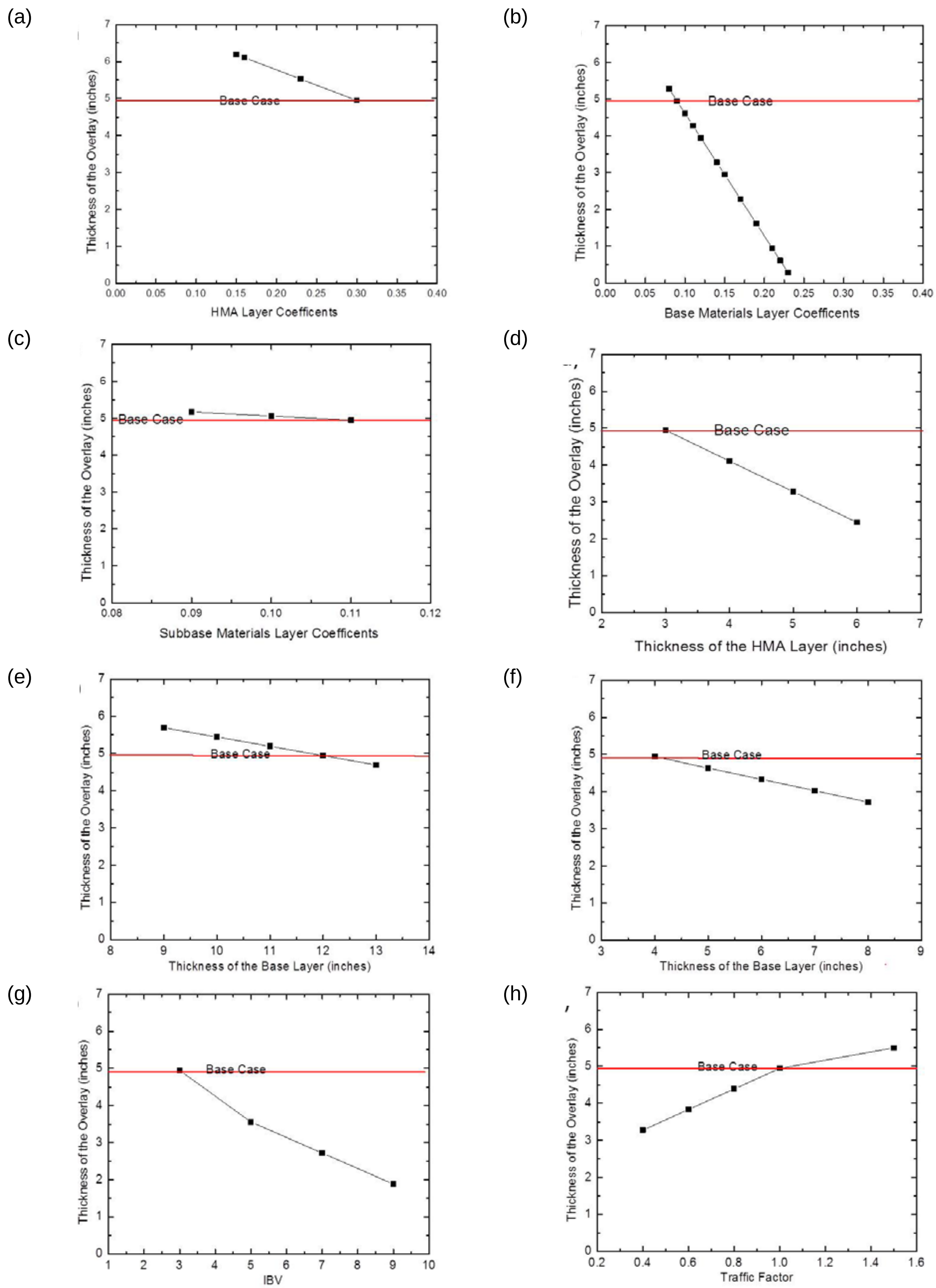


Figure 2.13: Overlay thicknesses calculated for various cases studied as listed in Table 2.4.

Note that the pavement layer coefficients used above are all empirical and therefore limited in their ability to characterize properly the structural contributions of the many recycled/reclaimed, stabilized and large-sized construction materials, as well as asphalt mixes more commonly used in today's sustainable pavement design and construction practices. Further, the concept of assigning layer coefficients is deficient because of its lack of consideration of the lifetime degradation of the layer materials and how the pavement functionality and performance degrade over time with the repeated traffic loading and climatic effects.

Based on the results of the sensitivity analyses, input parameters required to perform an overlay design according to the modified AASHTO layer coefficients method can be ranked as follows:

- HMA and base layer coefficients—most sensitive
- Layer thicknesses—sensitive
- IBV value and traffic factor—sensitive

2.5.2 Sensitivity Analysis: 1993 AASHTO NDT–Based Method for Overlay Design (Used by ODOT)

Ohio Department of Transportation (ODOT) (1999) uses the 1993 AASHTO NDT method to design the required HMA overlay thicknesses for flexible pavements. For the sensitivity analysis, the schematic of the pavement profile shown in Figure 2.14 was taken as the base case because this pavement configuration is one of the most commonly built configurations found in the local roads and streets in Illinois. The pavement layer configuration and the range of input values considered in the analyses were taken from a test section in Ogle County, Illinois, discussed in Chapter 3.

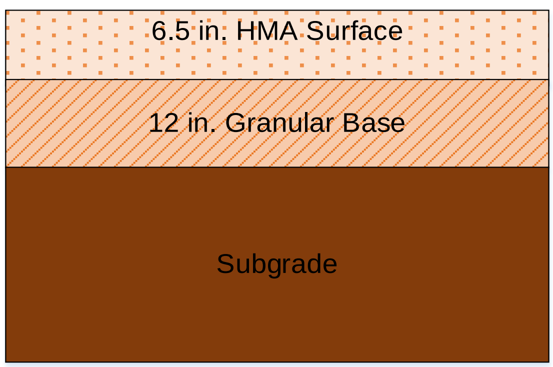


Figure 2.14: Pavement layer configuration used as a base case.

Table 2.6 lists all the cases studied, including the ranges of input values considered. The input parameters that were taken into consideration in performing the sensitivity analyses are listed below.

- FWD center deflection (d_0)
- Pavement temperature at the time of testing
- Traffic in terms of ESALs
- Layer thicknesses

Table 2.6: Case Studies Used in the Sensitivity Analyses

Cases	Sensitivity Variable	Range of Values Considered
1–5	FWD Center Deflection	17 mils to 25 mils
6–9	Pavement Temperature	94°F to 100°F
10–13	Surface Layer Thickness	3.0–6.5 in.
14–17	Base Layer Thickness	9–12 in.
18–22	Traffic	8 million to 12 million ESALs

Figure 2.15 (a to e) shows the HMA overlay thicknesses calculated for the cases listed in Table 2.6. Note that the HMA overlay thicknesses required varied the most with changes in FWD center deflections followed by the traffic inputs in ESALs. Based on the results of the sensitivity analyses, the input parameters required to perform an overlay design according to the 1993 AASHTO NDT method can be ranked as follows:

- FWD center deflection—most sensitive
- Traffic in ESALs—very sensitive
- HMA and base layer thicknesses—sensitive

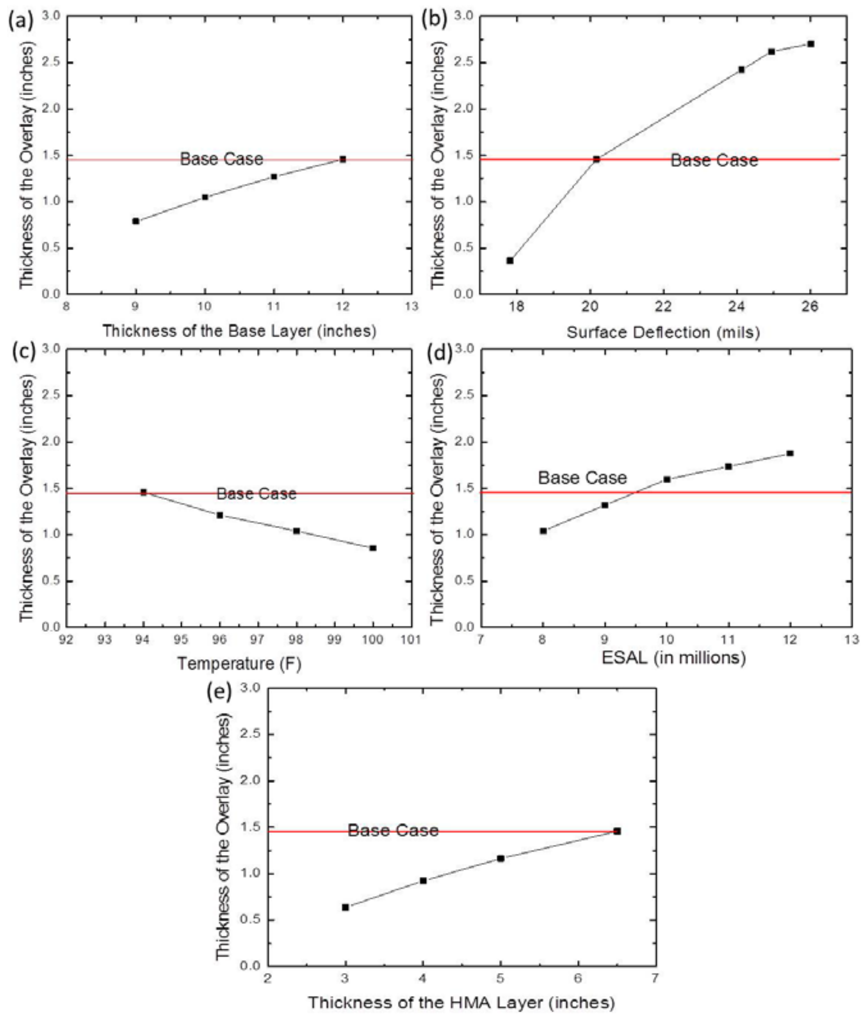


Figure 2.15: Overlay thicknesses calculated for various cases listed in Table 2.6.

2.5.3 Sensitivity Analysis: Asphalt Institute Deflection Method

The Asphalt Institute deflection method of overlay design is based on the representative rebound deflection (RRD), which is computed from the Benkelman beam test static deflection measurements. When FWD NDT testing is conducted instead, a conversion factor of 1.61 often is multiplied by the FWD center deflection to use in the calculation of the rebound deflection. A design chart as shown in Figure 2.16 establishes a pre-constructed unique relationship between the design rebound deflection and the allowable ESALs to determine the design overlay thickness. Note that the projected overlay traffic, temperature adjustment factor for the deflection measured, and critical period adjustment factor for the high deflections during spring thaw are all considered for determining the rebound deflection and the HMA overlay thickness.

The step-by-step procedure of the Asphalt Institute deflection method is as follows:

1. Determine the rebound deflections using Benkelman beam tests on the pavement in need of an overlay, with a truck weight of 80 kN or 18 kips on a single axle.
2. Determine the representative rebound deflection (RRD) using Equation 2.11.

$$RRD = (\bar{x} + 2s)\,\mathrm{c} \tag{2.11}$$

where

$\bar{x}$: mean of the temperature adjusted rebound deflections
s: standard deviation of rebound deflections
c: critical period adjustment factor

3. Estimate the required ESAL that needs to be supported by the overlaid pavement.
4. Determine the required overlay thickness according to the RRD and the design ESAL by using an overlay thickness chart (Figure 2.16 and Figure 2.17).

To perform the sensitivity analyses, the input design variables taken into consideration in the AI deflection-based method are as follow:

- Representative rebound deflection, RRD = 0.03 – 0.10 in. (0.01-in. increment) (0.06 in. chosen as base condition)
- Projected traffic, ESALs = 2, 3, and 5 million (2 million chosen as base condition)

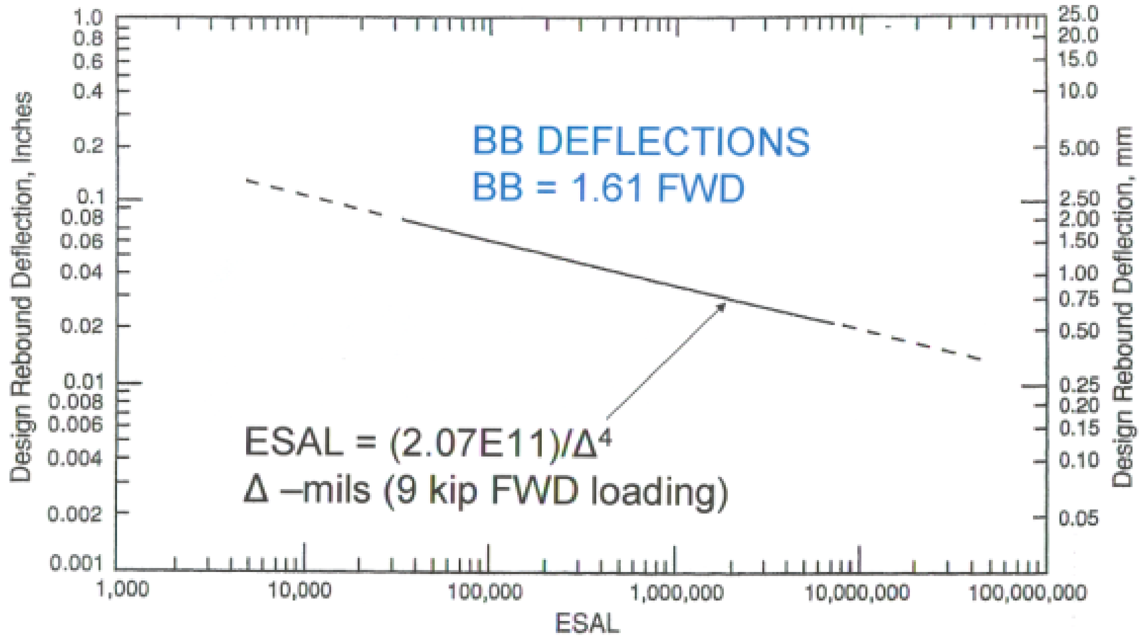


Figure 2.16: Design rebound deflection chart (AI 1996).

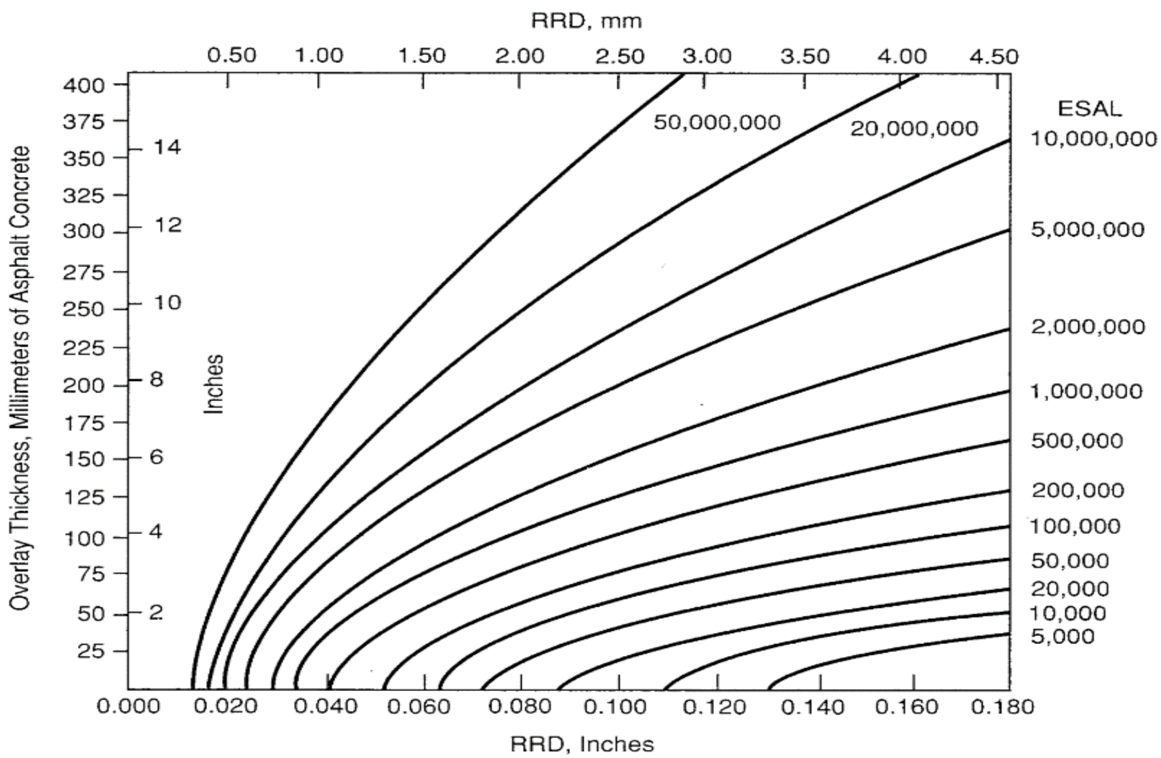


Figure 2.17: Asphalt concrete overlay thickness required to reduce pavement deflections to representative rebound deflection value (AI 1996).

Based on the results of the sensitivity analyses, Figure 2.18 (a and b) shows the HMA overlay thicknesses calculated for the various traffic ESAL counts studied and the RRD values, respectively. Note that the variation in the RRD values has a much more significant impact on the required overlay thickness when compared with the projected traffic inputs, which varied within the 2- to 5-million ESAL range. This finding further confirms how HMA structural overlay thicknesses can be determined adequately from NDT-based pavement deflection measurements.

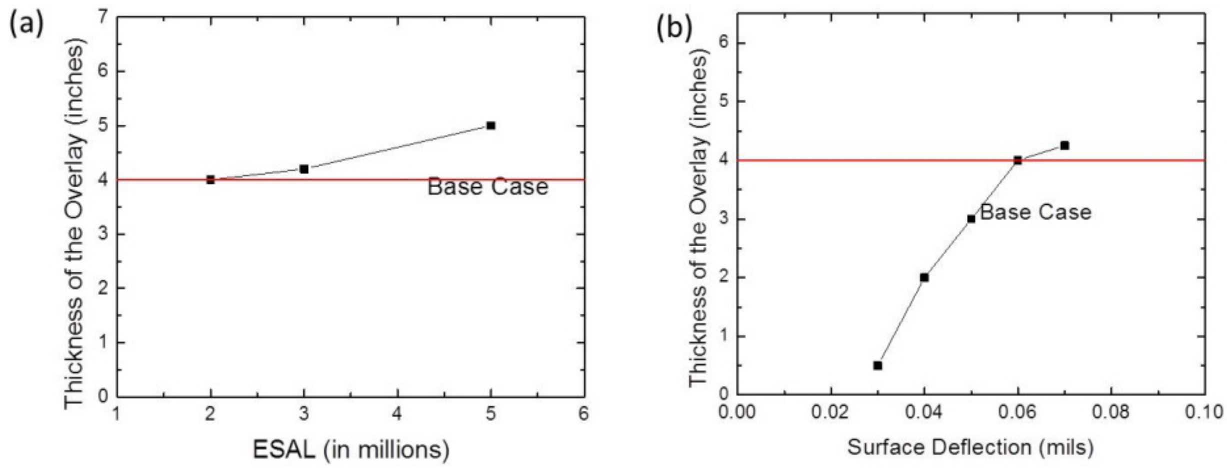


Figure 2.18: Calculated overlay thicknesses for the Asphalt Institute deflection method.

2.6 SUMMARY

Backcalculation in pavement analysis is a process in which results of NDT tests such as the FWD are used to infer layer properties, including layer thickness and layer moduli through a number of engineering approaches, such as simplified search methods, gradient-relaxation methods, and direct interpolation methods. Some key features of the available and commonly used software programs that employ these approaches were highlighted in this chapter. In a recent ICT study (R39-002), the development of new toolboxes, ANN-Pro and SOFTSYS, were also discussed, to indicate advantages of using artificial intelligence–based methods, such as the artificial neural networks and genetic algorithms, for predicting flexible pavement layer properties such as thicknesses, as well as critical stress and strain and deformation responses of these in-service pavements, which can be accurately and rapidly determined from the field FWD deflection basins. An overview was provided for the current pavement overlay procedures of flexible pavements, i.e., the 1993 AASHTO NDT method, IDOT modified layer coefficient method, and the Asphalt Institute method. Further, sensitivity analyses were conducted to determine effects of input properties on the calculated HMA overlay thicknesses. For both the 1993 AASHTO NDT and AI deflection methods, the magnitude of pavement deflection most influenced the overlay thickness. Whereas in the modified layer coefficients method used by IDOT, the assigned layer coefficients most influenced the overlay thickness. These modified AASHTO layer coefficients are outdated and inadequate to characterize structural contributions of in-service pavements.

CHAPTER 3 SELECTED FWD TEST LOCATIONS

3.1 INTRODUCTION

This research study was undertaken to evaluate currently available HMA overlay thickness designs for low-volume roads and to develop improved overlay design procedures based on proper structural evaluation of existing, in-service pavements in Illinois through the use of NDT methods such as the FWD test. In an effort to select candidate in-service pavements to study in this research project, a survey questionnaire was prepared and distributed among local agencies, including municipalities, counties, and townships throughout the state. After a careful review of the responses collected from the agencies, 20 pavement sections in six 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. This chapter presents the details of the selected FWD test sites and the corresponding local road overlay design project considerations at these case study sites, including the existing pavement layer configurations, pavement distresses, and performance histories.

3.2 DETAILS OF SELECTED LOCAL ROAD SITES

According to the local agency survey responses, 20 pavement sections in six counties in Illinois were selected for FWD-based structural condition evaluations. Table 3.1 presents the locations and study details of the selected pavement test sections. The structural conditions of some of these pavement sections were monitored over a period of one year through three different sets of FWD testing (Table 3.2). Typically, FWD tests along a given road segment were conducted at 200-ft intervals on the outer wheel paths, except for Sections 8 through 14, where FWD testing was conducted at 400-ft intervals. 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.) (Figure 3.1). Pavement surface temperature was collected during the testing at every 200-ft interval along the testing lane. Further details about these selected test locations are discussed in the following subsections.

Table 3.1: Details of the In-Service Pavements Studied

Location in Illinois	Road Name	No. of Sections	Section Number	Pavement Condition
McHenry County, Coral Township	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 Drive	1	6	Moderately Cracked
	Dorothy Lane	1	7	Moderately Cracked
Vermilion County	Perrysville Road	9	8–14	Moderately Cracked
Champaign County	CH 1 Dewey–Fisher Road	3	15–17	Very Few Cracks
Ogle County	S. Pines Road	3	18–20	Moderately Cracked in Few Locations

Table 3.2: FWD Tests and Pavement Sections Studied

FWD Testing Effort	Pavement Sections Tested	Pavement Condition Notes
Set 1	1 through 20	Some of the sections were severely cracked
Set 2	1, 2, 3, 4, 6, 7	After the overlay placement
Set 3	1, 2, 3, 4, 6, 7	One year after Set 1 testing effort

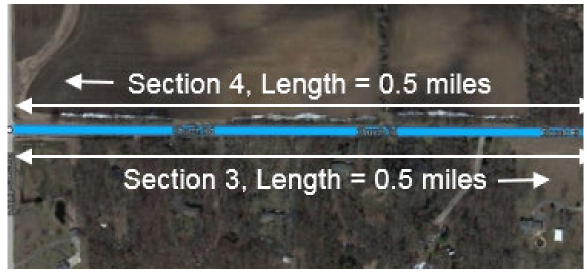
Figure 3.1: Conducting FWD tests in selected local road pavement sections in Illinois.

3.2.1 McHenry County, Coral Township

In total, three sets of FWD tests were conducted at two locations, East Coral Road and Church Street, in McHenry County. The first set (Set 1) of FWD tests was conducted when the pavement sections were severely deteriorated and in need of major rehabilitation work. The second set (Set 2) of FWD tests was conducted immediately after the overlay. Set 3 was conducted one year after Set 1 and can be used to assess the extent of pavement structural deterioration over time.

Figures 3.2, 3.3, and 3.4 show the aerial views, photos, and the layer configurations along with traffic information for the tested pavement sections, respectively. The pavement configurations after the overlay are referred to, as 1-b, 2-b, etc. Accordingly, the pavement configuration for Section 1 after the overlay is indicated as Section 1-b. As shown in Figure 3.2, Sections 1 and 2 represent contiguous sections on the 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 because of the varying pavement layer profiles and substructure (base, subbase, and subgrade) support conditions. Pavement surface temperatures were recorded to be around 45°F at both locations during Set 1 testing. During Set 2 testing, pavement surface temperature was recorded as 71°F for Sections 3 and 4; and the surface temperatures varied from 77°F to 88°F for Sections 1 and 2. During Set 3 testing, the surface temperatures ranged from 105°F to120°F for Sections 1 and 2 and 100°F to 103°F for sections 3 and 4. Note that Sections 1 and 2 were overlaid with 31.75 mm (1.25 in.) of HMA after the first set of FWD tests, whereas Sections 3 and 4 received a 38 mm (1.5 in.) thick overlay.

East Coral Road



Church St

Figure 3.2: Aerial views of sections 1, 2, 3, and 4 in McHenry County, Illinois.


East Coral Road (Sections 1 and 2)
before Overlay


Church St (Sections 3 and 4)
before Overlay


East Coral Road (Sections 1 and 2)
after Overlay


Church St (Sections 3 and 4)
after Overlay

Figure 3.3: Photos of the McHenry County pavement sections tested with FWD.

Figure 3.4: Layer configurations and traffic information for sections 1, 2, 3, and 4 in McHenry County, Illinois.

3.2.2 City of DeKalb, Illinois

FWD tests were carried out twice on Twombly Road (referred to as Section 5 in this study; about 0.5 mi long) in the city of DeKalb, Illinois. The first set (Set 1) of FWD tests was conducted when the pavement section was severely deteriorated and in need for major rehabilitation work. The pavement surface temperature was recorded to be 88°F. Set 3 was conducted one year after Set 1 to assess the rate and extent of pavement structural deterioration over time. The pavement surface temperature was recorded to be 109°F. Note that Section 5 received no overlay after the initial testing effort. Figures 3.5 and 3.6 show the aerial view and the layer configurations, along with traffic information and the field photos of the tested pavement sections, respectively.

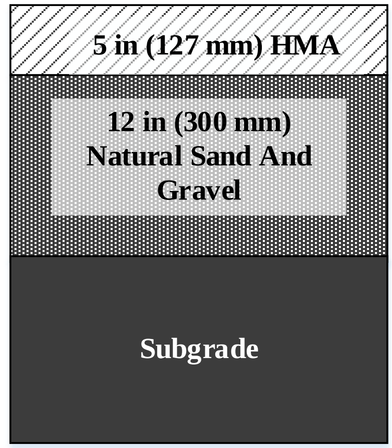
3.2.3 Village of Tinley Park, Illinois

FWD tests were carried out twice on Normandy Drive and Dorothy Lane (referred to as Sections 6 and 7, respectively, in this study) in the village of Tinley Park, Illinois. Pavement surface temperature at the time of testing was 83°F in both locations. Figures 3.7 and 3.8 show the aerial views and pavement photos, respectively. Figure 3.9 shows the pavement layer configurations, along with traffic information for the sections tested.

(a)

Twombly Road

(b)

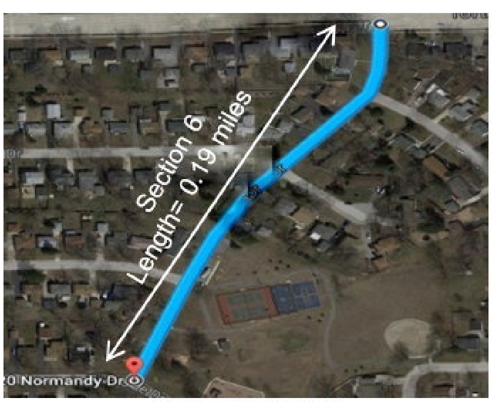


Section 5
Class III Roadway with
Daily Traffic <1500

Figure 3.5: (a) aerial view, (b) layer configuration and traffic information for section 5 in City of DeKalb, Illinois.

Twombly Road (Section 5)

Figure 3.6: Photo of Twombly road in City of DeKalb tested with FWD.



Normandy Drive

Dorothy Lane

Figure 3.7: Aerial views of sections 6 and 7 in Village of Tinley Park, Illinois.

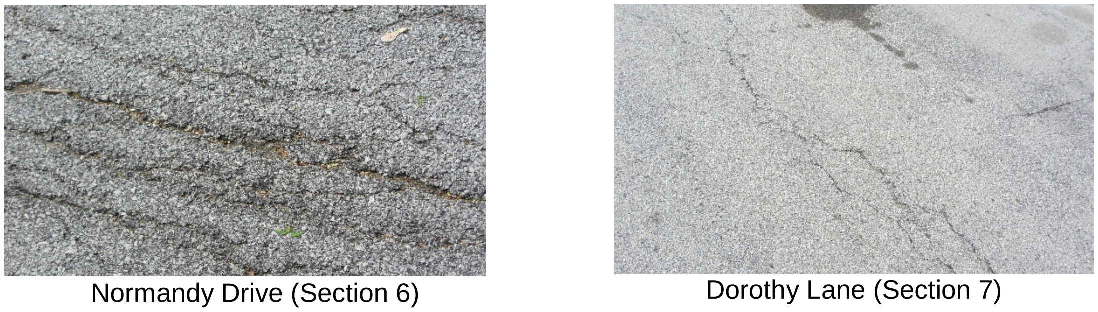


Figure 3.8: Photos of sections 6 and 7 in Village of Tinley Park tested with FWD.

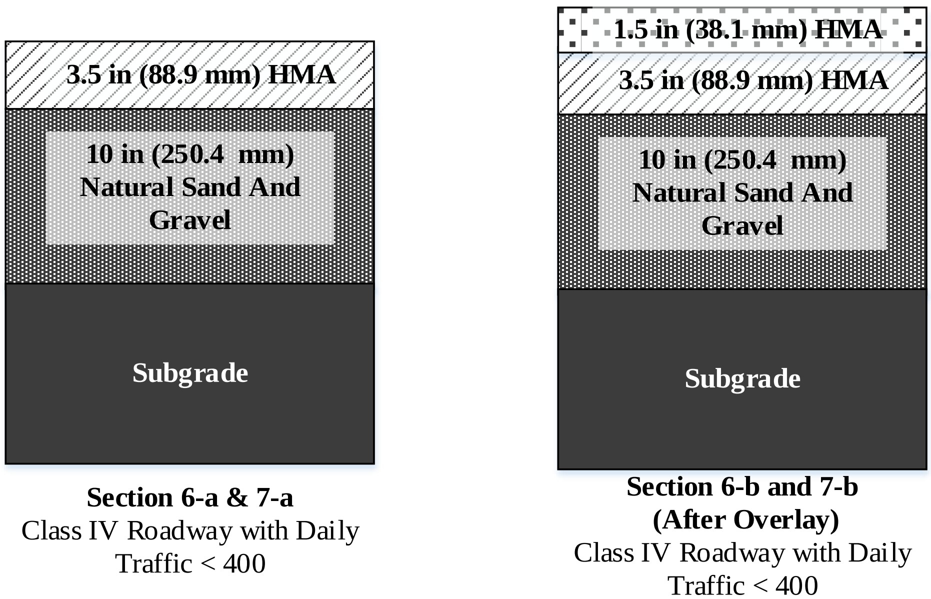


Figure 3.9: Layer configurations and traffic information for section 6 in Village of Tinley Park, Illinois.

The first set (Set 1) of FWD tests was conducted in the village of Tinley Park when the pavement sections were severely deteriorated and in need for major rehabilitation work. Set 3 FWD testing was conducted one year after Set 1, and the surface temperature was recorded to be around 111°F for Section 6 and 113°F for Section 7. These pavement surface temperatures were significantly higher than the temperatures recorded during the Set 1 testing effort. However, just before the Set 3 testing was undertaken, the pavements were overlaid with a 1.5 in.-thick HMA. The pavement configurations after the overlay are referred to as 6-b and 7-b in Figure 3.9.

3.2.4 Vermilion County, Illinois

FWD tests were carried out on Perrysville Road (CH 6) in Vermilion County, Illinois; and the testing was done in both eastbound and westbound directions at a 400-ft interval. Based on the layer configurations received from the county, the total road segment was divided into seven pavement sections. All the test sections had a composite pavement in the middle part of the pavement, and the outer sides were widened using HMA surface at the edge. Accordingly, the FWD loads were dropped at a 7-ft offset from the shoulder of the pavement to ensure that the composite pavement in the middle was not tested. However, 0.625-mile-long stretch of composite pavement section was also tested with FWD to collect

FWD deflection basins. The detailed pavement layer configuration of the tested composite section, along with the field deflection basins, is included in Appendix A of this report. Figure 3.10 shows the aerial views of the test locations, and Figures 3.11 and 3.12 present the field photos and the layer configurations, along with the traffic information, of the existing test sections. Measured pavement surface temperatures varied from 110°F to 133°F during testing.

Figure 3.10: Aerial views of sections 8 through 14 in Vermilion County, Illinois.

Figure 3.11: Photos of sections tested with FWD in Vermilion County, Illinois.

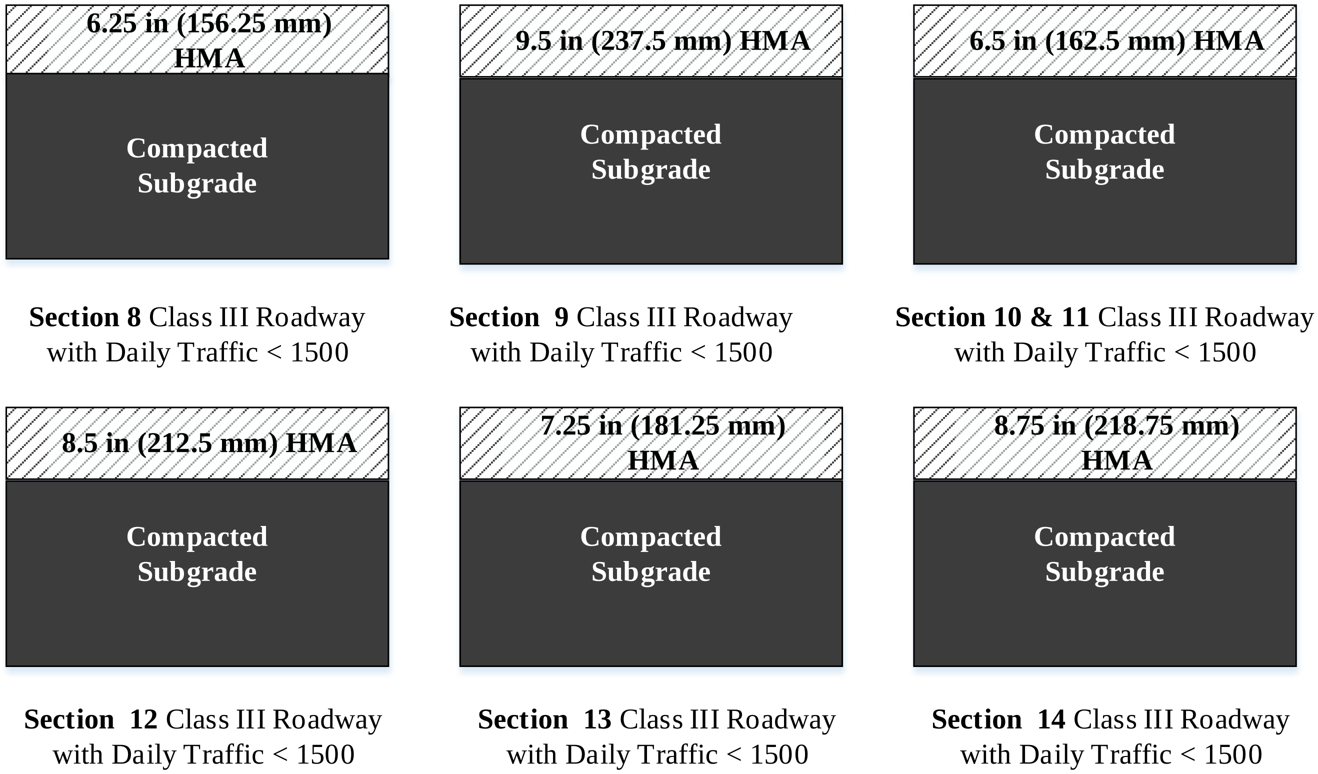
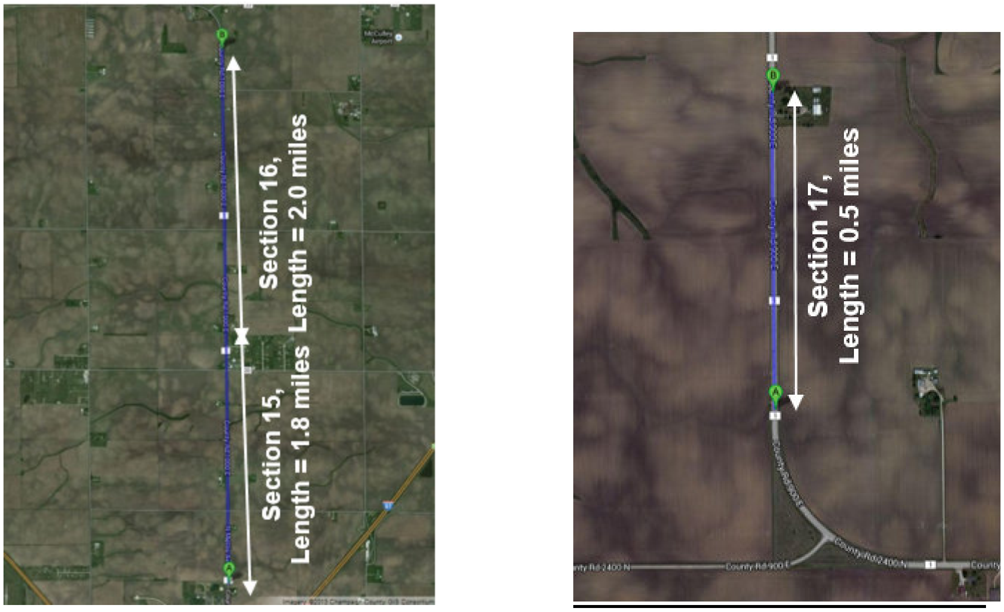


Figure 3.12: Layer configurations and traffic information for sections 8 through 14 in Vermilion County, Illinois.

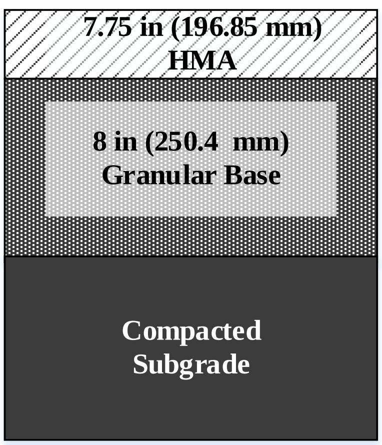
3.2.5 Champaign County, Illinois

FWD tests were carried out on Dewey–Fisher Road (CH 1) in Champaign County, Illinois; and the total length of the road segments tested was 4.3 mi. The testing was done only in the northbound direction, as the existing pavement section in the southbound direction consisted of a Portland cement concrete (PCC) pavement. However, a curved section also existed along the test location, which was a composite section (3-in. asphalt overlay on top of an 8-in. PCC) and hence was excluded from this analysis. The FWD test road was divided into two segments to exclude the curved composite section. Figures 3.13 and 3.14 show the aerial views and the layer configurations of the test sections along the Dewey–Fisher Road (CH 1), where the FWD tests were conducted, respectively. The test sections were divided into three sections based on the varying levels of traffic along the test section.



Dewey–Fisher Road CH 1

Figure 3.13: Aerial views of sections 15, 16, and 17 in Champaign County, Illinois.



Section 15, 16, & 17
Class II Roadway

Dewey–Fisher Road CH 1
(Section 15,16, and 17)

Figure 3.14: Layer configurations and photos for sections 15, 16, and 17 in Champaign County, Illinois.

3.2.6 Ogle County, Illinois

FWD tests were carried out on South Pines Road in Ogle County, Illinois. The road segment tested during this effort was first divided into three sections based on the layer configuration information. The FWD testing was done in both northbound and southbound directions, and pavement surface temperature varied from 93°F to 104°F. Figure 3.15 shows the layer configurations of the tested pavement sections along South Pines Road. Figures 3.16 and 3.17 show the field photos and the layer configurations of the test sections from Ogle County, respectively.

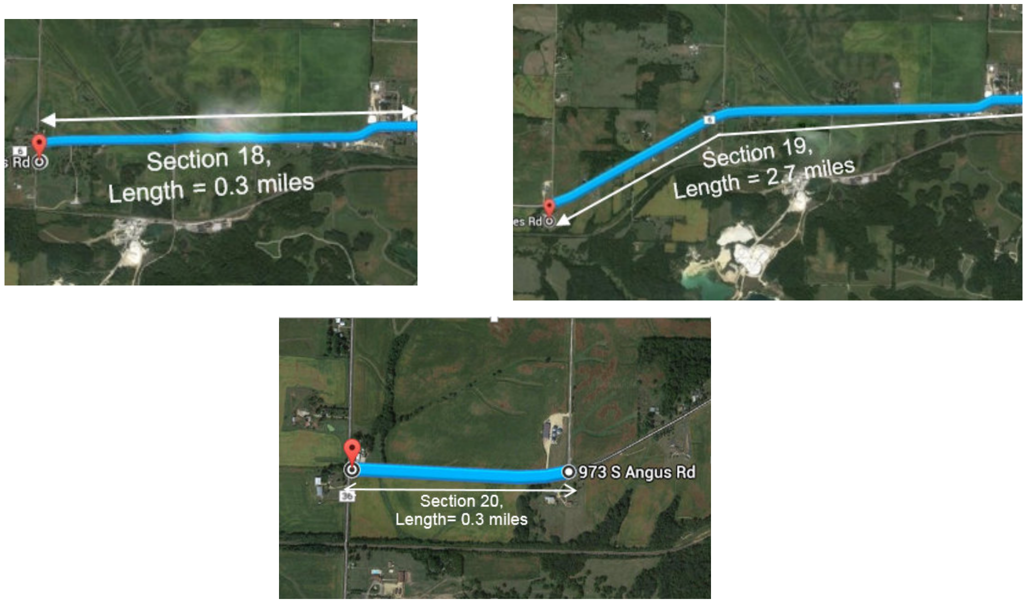


Figure 3.15: Aerial views of sections 18, 19, and 20 in Ogle County, Illinois.

S. Pines Road (Section 18,19, and 20)

Figure 3.16: Photos of pavement sections tested with FWD in Ogle County, Illinois.

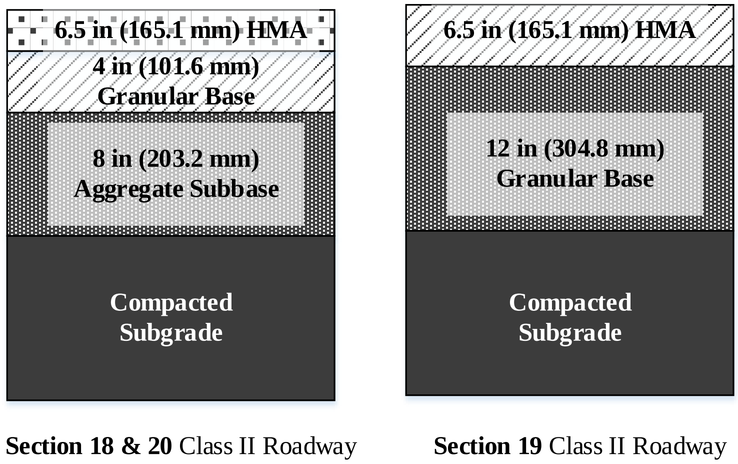


Figure 3.17: Layer configurations of sections 18, 19, and 20 in Ogle County, Illinois.

3.3 SUMMARY

Twenty pavement test sections were selected in six Illinois counties to conduct FWD testing on local agency roads for the purpose of providing NDT-based structural condition evaluations. The pavement test sections varied greatly in HMA overlay thickness design requirements in terms of the existing pavement layer profiles, traffic volumes, distress conditions, and the deflection basins at different surface temperatures collected during FWD testing. It was possible to collect up to three sets of FWD test data to assess the structural conditions of the pavement test sections (i.e., before, immediately after, and one year after the overlay placement). Four of the test sections rehabilitated with thin HMA overlays could be tested with the overlay immediately after the initial set of FWD testing. Two sections were tested one year after the initial set of FWD testing, which enabled evaluating the performance of the overlaid pavement. The rest of the pavement sections were never overlaid. The details about the FWD test results and the related analyses are presented in the next chapter.

CHAPTER 4 FWD TEST RESULTS, DATA ANALYSES, AND OVERLAY THICKNESS DESIGNS

4.1 INTRODUCTION

The details of the 20 pavement sections tested with the FWD equipment for evaluating structural conditions were described in detail in Chapter 3. Some of these sections were tested up to three times to evaluate structural conditions before rehabilitation, immediately after the overlay was placed, and after one year of service under traffic. This chapter first presents the test results obtained from these FWD test sites to compare and contrast differences in structural conditions of the case study pavement sections requiring rehabilitation, as identified by the local agencies in six Illinois counties. The analyses of the FWD data for backcalculating layer modulus properties will be described next. The previously introduced and currently available overlay thickness design procedures are used next to compute the HMA overlay thicknesses required for each pavement section. In addition, a newly developed M-E overlay design method is introduced to compute critical pavement responses obtained from FWD testing, along with calibrated pavement damage algorithms to mechanistically estimate the required overlay thicknesses. Finally, the summary recommendations of the HMA overlay thicknesses are presented; and their estimated unit costs for construction are compared in this chapter.

4.2 FWD TEST RESULTS

Among the 20 pavement sections tested in the field and evaluated for structural conditions in this study, FWD tests were conducted on sections located in McHenry County a total of three times over a period of one year. This subsection presents the FWD deflection basins of Sections 1 and 2, and the FWD test results of all other pavement sections are included in Appendix A. Figure 4.1 shows the deflection basins obtained for Section 1. During the Set 1 FWD testing on the deteriorated old pavement, the deflection values varied significantly among the test stations. For instance, at station 3,000 ft east direction and at 2,000 ft and 2,500 ft west direction, the center deflection values (D_0) were very close to that of the one obtained 12 in. from the center of the loading plate (D_{12}). This finding could be because these pavement sections were severely cracked at many locations along the road alignment, which resulted in such anomalies.

As shown clearly in Figure 4.1b for almost every station tested at 200-ft intervals along the total length of the section, surface deflection values were generally reduced and more uniform, with fewer fluctuations, after the placement of a 1.5 in.-thick HMA overlay. However, for some sections, as indicated in Figure 4.1, the center deflections are slightly larger than those obtained before the overlay. This happened because pavement surface temperatures were much higher during the Set 2 testing (varied between 71°F and 88°F), compared with the 45°F pavement surface temperature recorded during the Set 1 FWD testing. Nevertheless, although tested at higher temperature, the deflection values seemed not to vary too much from one station to another adjacent station. An interesting observation is that center deflection values seemed to get lower after one year with the overlay during the set 3 FWD testing. This finding could be because Section 1 had a thin HMA surface layer, so the pavement base might have become stiffer because of the traffic it was exposed to for about one year and eventually resulted in lower deflection values when compared with the measurement one year earlier, just after the overlay placement.

Trends similar to those mentioned above can be found for Section 2, as highlighted in Figure 4.2. In addition, in Figure 4.2a, several stations from the beginning of the FWD tests are missing deflection values. This finding is because these pavement sections were severely cracked at many locations along the test section, which eventually resulted in non-decreasing deflection bowls. Accordingly, stations with such questionable data were removed from the analyses and are not shown in the deflection basin curves.

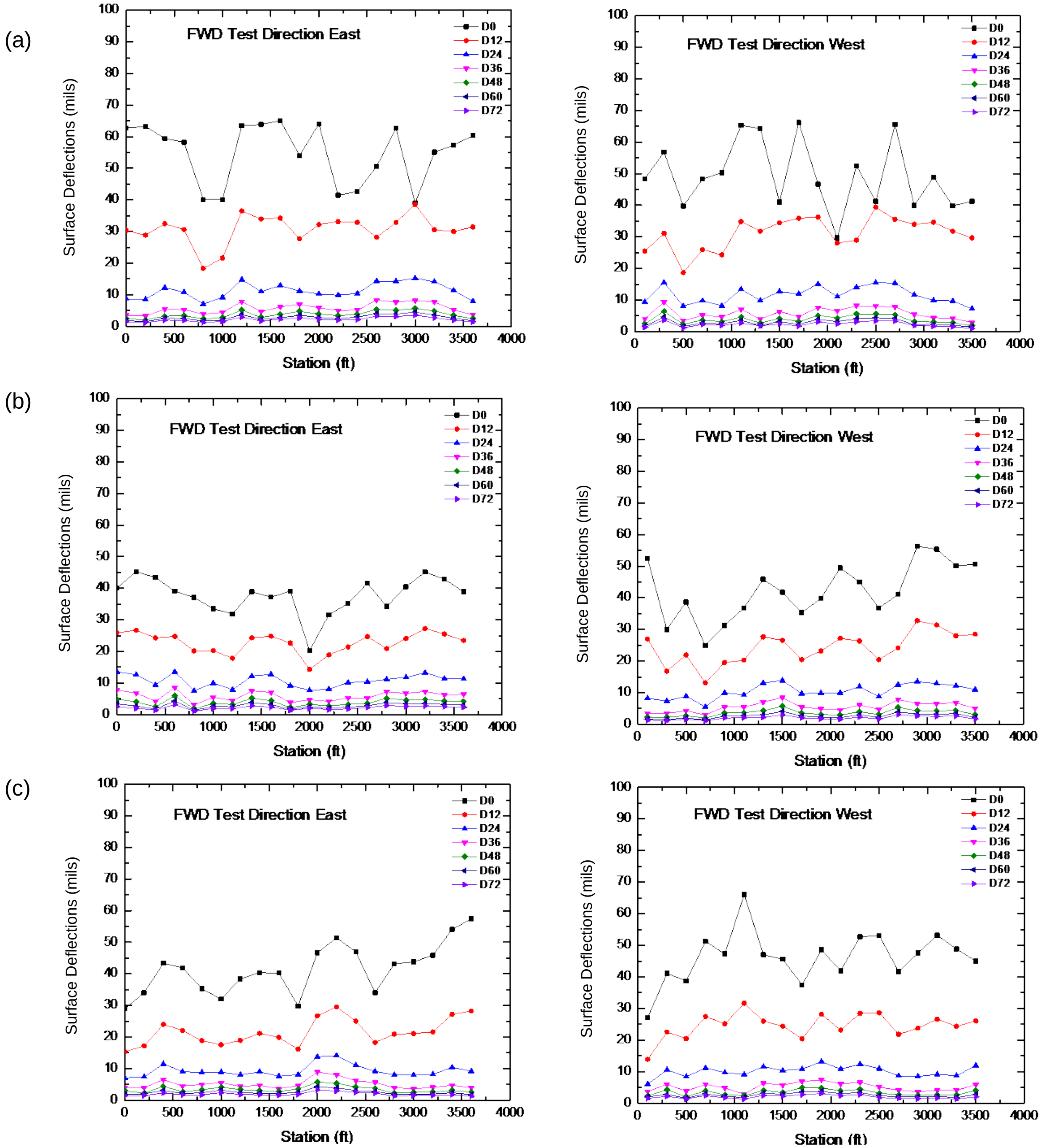


Figure 4.1: Deflection basins obtained from the field during (a) Set 1, (b) Set 2, and (c) Set 3 FWD testing efforts for pavement section 1.

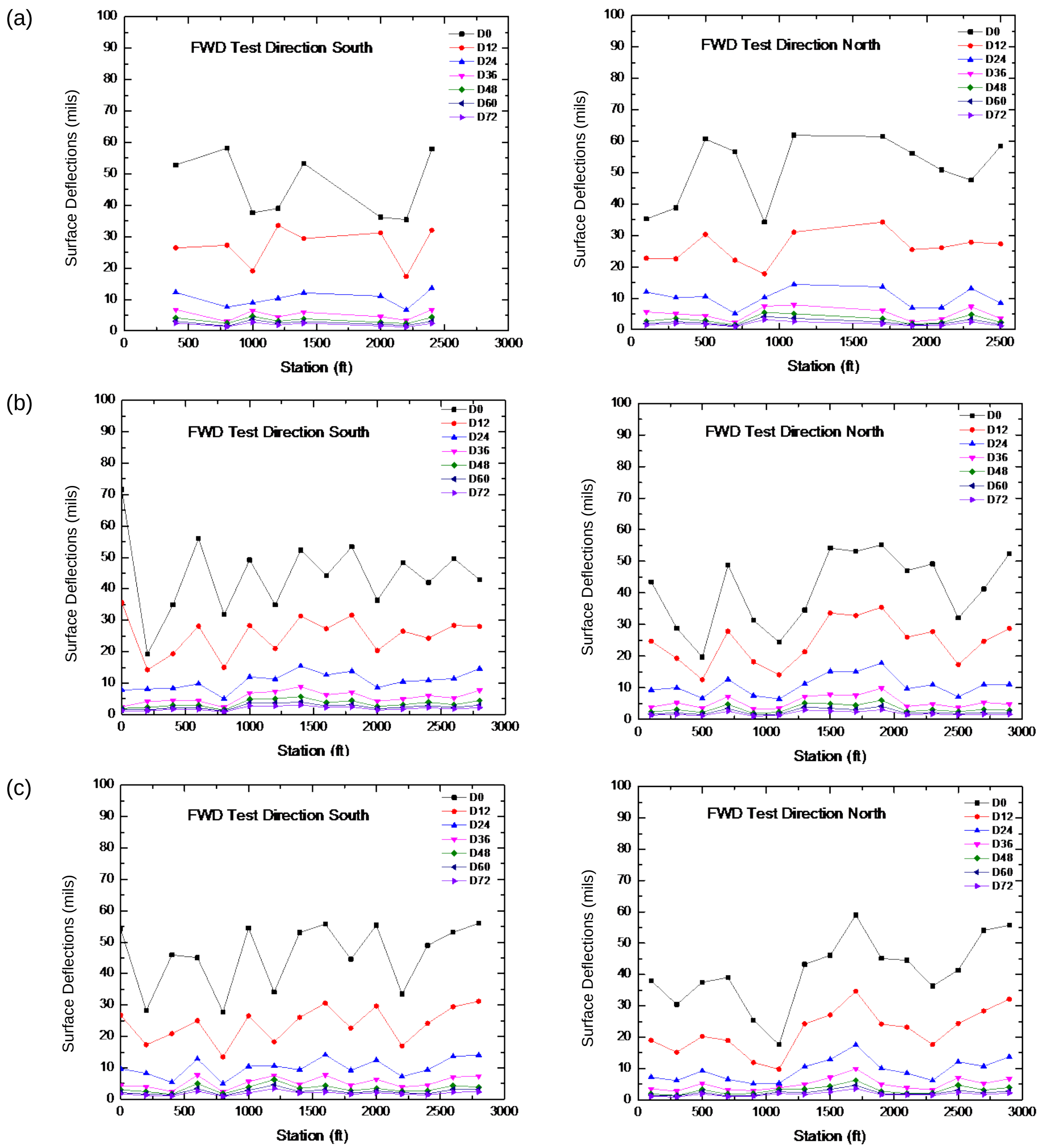


Figure 4.2: Deflection basins obtained from the field during (a) Set 1, (b) Set 2, and (c) Set 3 FWD testing efforts for pavement section 2.

4.3 BACKCALCULATION ANALYSES FOR LAYER MODULI

The first task in 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 several backcalculation analysis software programs described in Chapter 2. Among these programs, MODULUS 6.0 (backcalculation software developed at the Texas Transportation Institute) (Liu and Scullion 2001) was available for free to state and local transportation agencies. The ANN-Pro, a neural network–based backcalculation software program, and the SOFTSYS program were developed during a previous ICT research project (R39-002) at the University of Illinois at Urbana-Champaign. Note that both ANN-Pro and SOFSYS solutions take advantage of the advanced ILLI-PAVE FE solutions in backcalculation analyses.

Layer configurations for the pavement sections were obtained in coordination with the local transportation agencies. The FWD deflections were analyzed; significant variations were observed in the backcalculated layer modulus values, even within a single pavement section. This finding was primarily because of varying support conditions; and also, different severity levels of cracking were observed along the road segments. Moreover, at several stations, severe cracking on the pavement surface resulted in deflection profiles that were unsuitable for backcalculation purposes and therefore were not used in the analysis. For example, inadequate contact of geophones with the cracked pavement surface sometimes led to non-decreasing deflection profiles with radial distance from the FWD load drop location. Such stations with questionable data had to be eliminated from the analyses. Accordingly, several stations with weak support conditions also had to be excluded from the moduli backcalculations, sometimes resulting in higher backcalculated layer moduli compared than if results from all test stations were included in the analyses.

The pavement layer moduli backcalculated after set 1 of FWD testing are presented in Figures 4.3, 4.4, and 4.5 in the form of box plots for Sections 1 through 20 evaluated in this study. (Note: the pavement layer moduli shown in Figures 4.3, 4.4, and 4.5 are backcalculated from the original field deflection basins before any temperature correction was applied to them). The upper and lower values of each box are the 25th and 75th percentiles of the layer moduli, respectively. The square symbol inside each box depicts the mean value of the layer moduli for individual section. The backcalculation of the layer moduli were completed with the help of MODULUS and ANN-Pro (in lieu of ILLI-PAVE FE) programs. The MODULUS layer moduli obtained from linear elastic layered solutions were used to determine typical stress states in the pavement layers. The stress states obtained were then used in the ILLI-PAVE finite element (ANN-Pro forward calculation) program to verify the surface deflection profiles measured in the field. For the pavement sections, the surface moduli values shown here are the average values computed by these two programs.

Figure 4.6 shows the layer moduli backcalculated after Set 2 of FWD testing for Sections 1 through 4 in McHenry County. After the overlay placement, the new and old surface courses were considered together as one layer; and accordingly, the overall surface moduli values decreased. It is important to note that this trend should not be misinterpreted as a reduction in the layer modulus upon application of the overlay. This finding is primarily because results from several of the “weak” test locations had to be eliminated from the analyses of the Set 1 test results. As already mentioned, this trend was the outcome of excessive cracking of the pavement surface and the subsequent non-decreasing deflection basins. The primary aspect to notice when comparing Figures 4.3 and 4.6 is the significant improvement in distribution of layer modulus values (reduction in the range in test results) after application of the overlay.

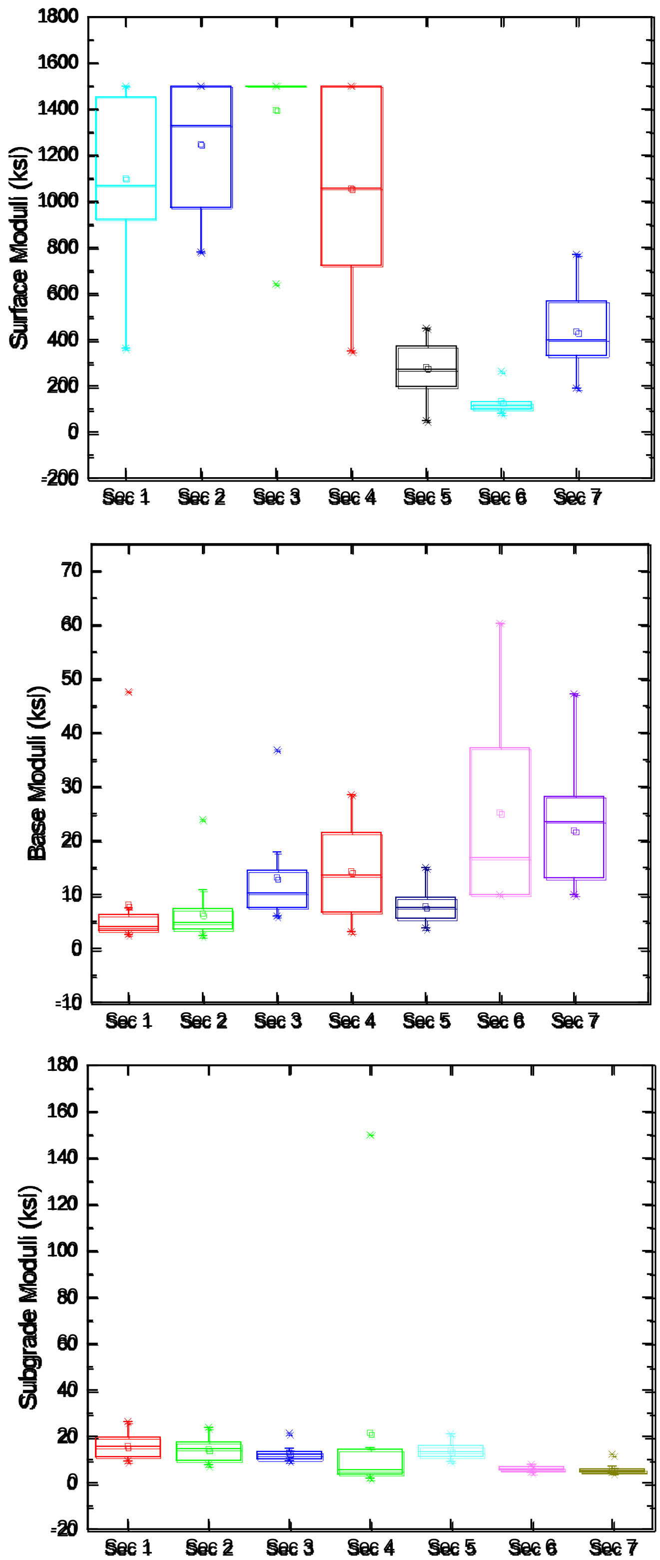


Figure 4.3: Backcalculated layer moduli for pavement sections 1 through 7.

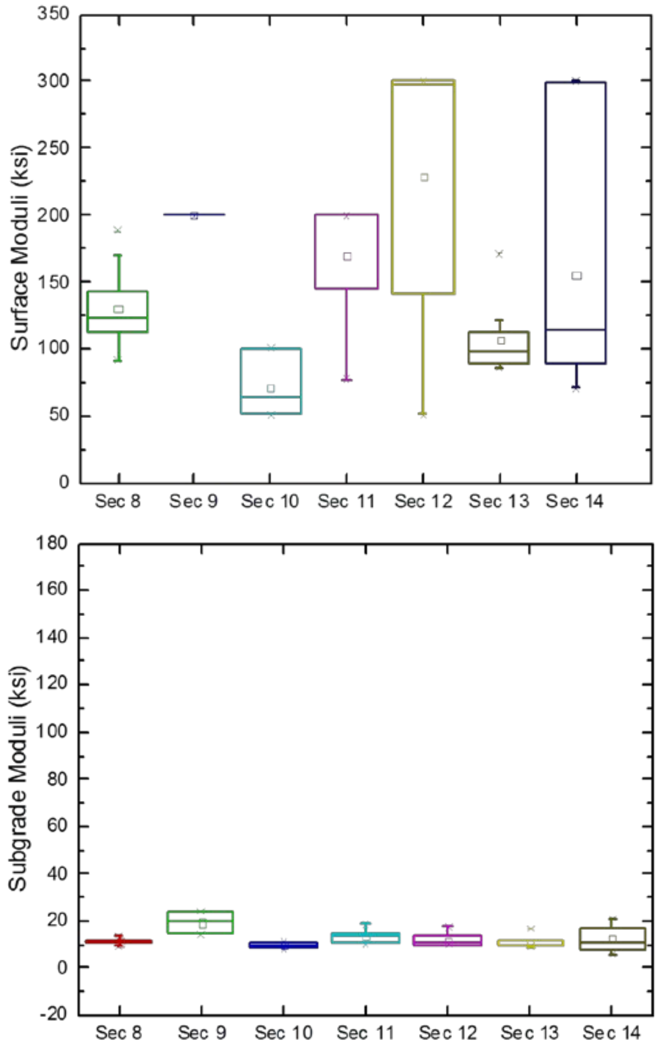


Figure 4.4: Backcalculated layer moduli for pavement sections 8 through 14.

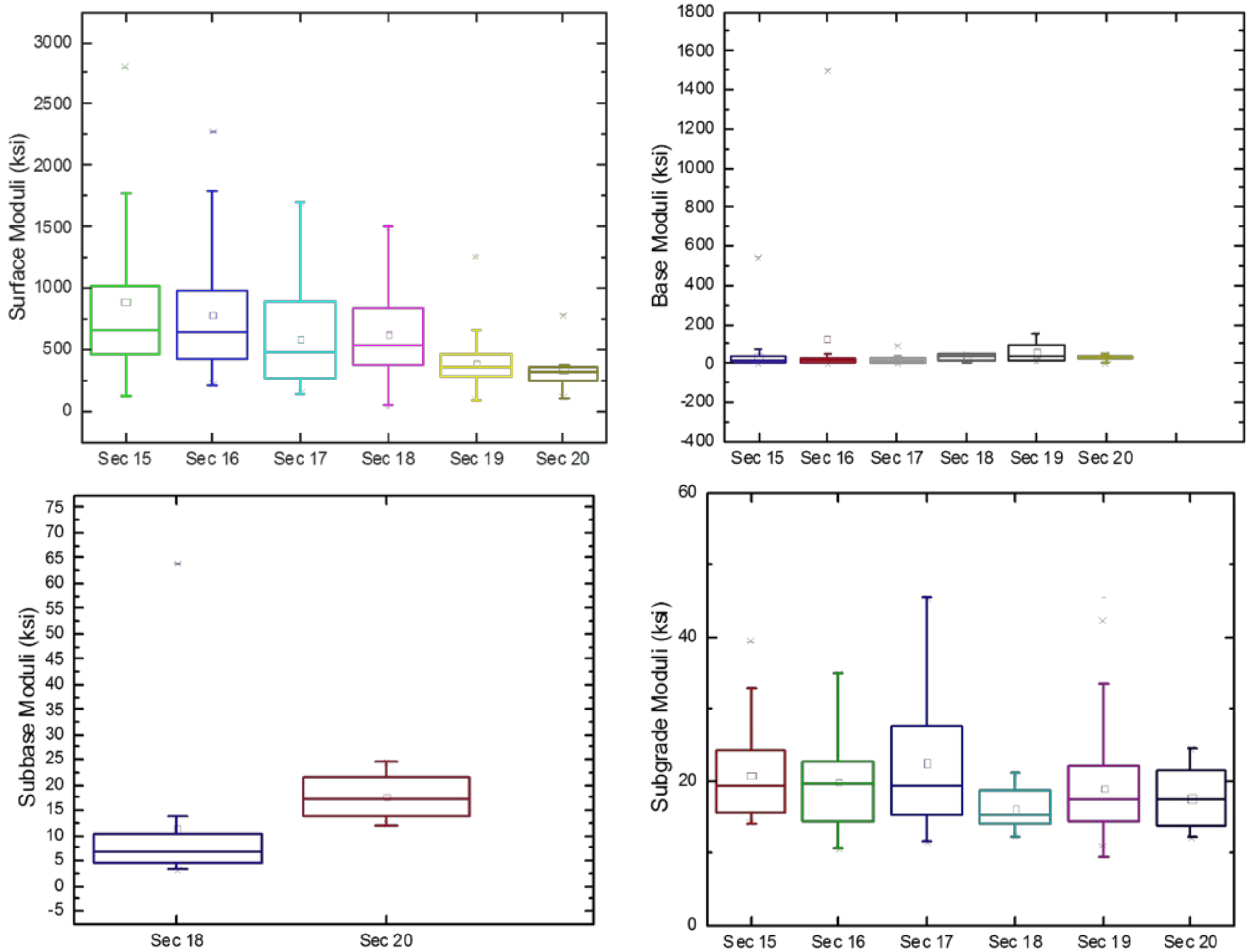


Figure 4.5: Backcalculated layer moduli for pavement sections 15 through 20.

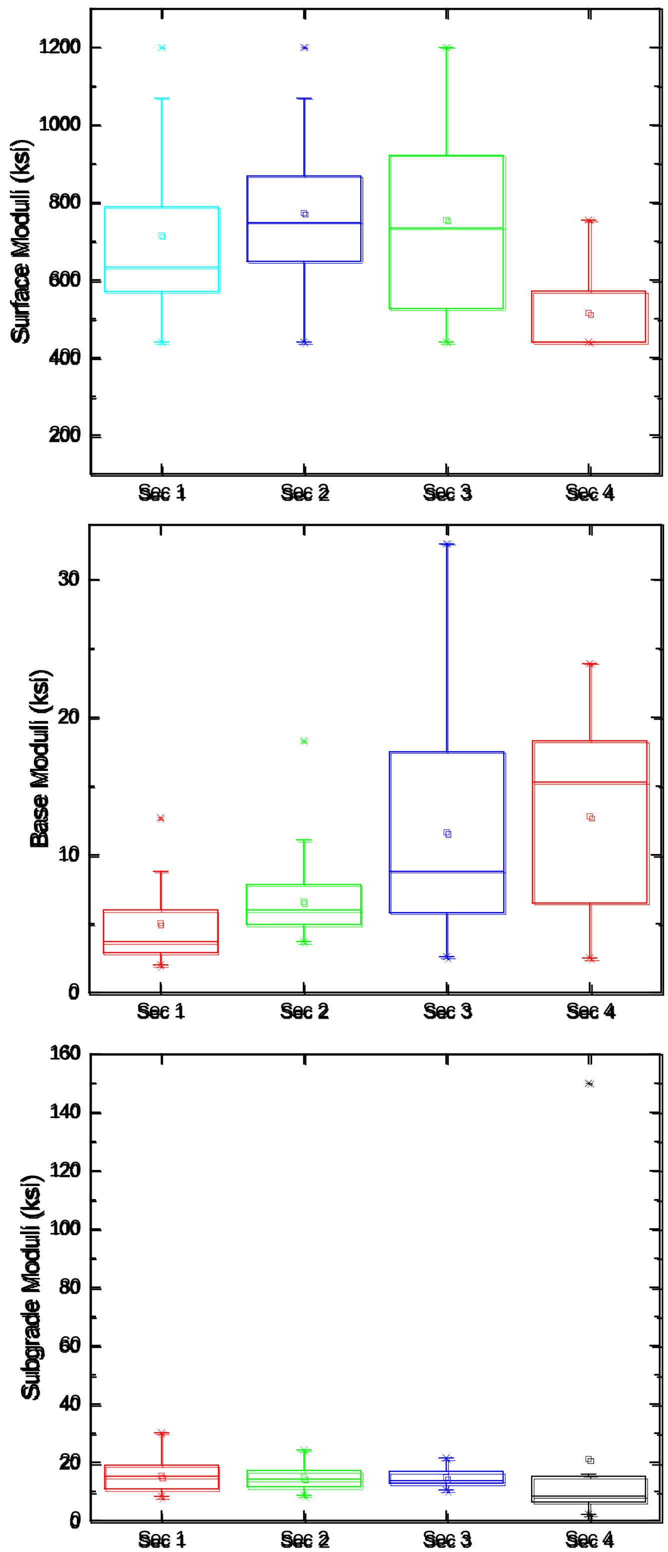


Figure 4.6: Backcalculated layer moduli for pavement sections 1 through 4 immediately after application of overlay.

4.4 OVERLAY THICKNESS DESIGNS USING 1993 AASHTO NDT, IDOT, AND AI DEFLECTION PROCEDURES

The next step was to determine the required overlay thicknesses for the tested pavement sections, based on commonly available design methods. The AASHTO 1993 NDT, IDOT, and AI deflection methods were used for this purpose. Please note that the IDOT method here refers to Modified Layer Coefficients method based on the AASHTO procedure for overlay design as discussed earlier in Chapter 2. Besides using the Modified Layer Coefficients approach, IDOT also recommends other available overlay design procedures such as the AI deflection method when applicable. However, the Modified AASHTO procedure is the one that is used most commonly by local agencies including municipalities, counties and townships. Therefore, the IDOT method refers to the Modified Layer Coefficients approach from here on.

Traffic factors were calculated using the equations provided in the Illinois *Bureau of Local Roads and Streets Manual* (BLRS 2012). The layer coefficients for the IDOT method were also obtained from the BLRS manual. The subgrade strength was kept constant at an IBV (similar in concept to the unsoaked California bearing ratio or CBR) value of 6%. Note that this figure corresponds to the minimum required bearing value in Illinois for the construction of flexible pavements without subgrade replacement and pavement working-platform construction. Calculation steps involved in these methods are trivial in nature and, for brevity, are not given here in detail. A summary of the design parameters and layer coefficients used in these design approaches is presented in Table 4.1 together with the determined HMA overlay thicknesses.

In the 1993 AASHTO NDT method, 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; 50th percentile) compared with the corresponding SN_{req} (= 3.1). Accordingly, all pavement sections except for Section 5 would not require any structural overlay. However, as previously mentioned, all pavement sections demonstrated a severe degree of fatigue cracking during the first set of 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.

Significant differences were found between the recommended overlay thicknesses determined from the 1993 AASHTO NDT and the IDOT methods. This finding can potentially be attributed to assumptions associated with the values of the empirical layer coefficients. As already mentioned, layer coefficients for the HMA and base layers in the IDOT method were selected from a range of values presented in the IDOT BLRS manual (2012). Additionally, the layer coefficients used in the IDOT method are empirical in nature and have been established for a limited number of materials. Accordingly, using the IDOT method for structural evaluation of pavements constructed with recycled and/or nontraditional materials is questionable at best.

Most of the sections did not require an overlay according to the calculations based on the AI deflection method, except for Sections 5, 8, and 10 through 14. Note that the AI deflection-based approach requires an additional critical season conversion adjustment, which was neglected in this analysis because yearly records of measured deflections for the test sections were not available. Also, when compared with the IDOT method, the AI deflection method often predicted thicker HMA overlays, based on proper structural condition assessment through the use of the FWD-measured center deflections.

Table 4.1: Overlay Thickness Designs Using 1993 AASHTO NDT, IDOT, and AI Deflection Methods

		Sec 1	**Sec 2**	**Sec3**	**Sec 4**	**Sec 5**	**Sec 6**	**Sec 7**	**Sec 8**	**Sec 9**	**Sec 10**	**Sec 11**
1993 AASHTO NDT	Traffic Factor	0.014	0.014	0.014	0.014	0.41	0.014	0.014	0.25	0.31	0.31	0.31
	90^{th} Percentile SN_{eff}	2.56	2.61	2.66	2.64	3.22	3.33	3.73	4.85	7.013	4.42	4.21
	Median SN_{eff}	2.08	2.19	2.16	2.28	2.96	2.81	3.08	3.73	5.37	4.11	3.80
	10^{th} Percentile SN_{eff}	1.84	1.90	1.85	1.95	2.64	2.46	2.95	3.04	4.69	4.0	2.98
	SN_{req} (IBV=6)	1.90	1.90	1.90	1.90	3.1	1.9	1.9	2.8	2.9	2.9	2.9
	Overlay Requirement (in.), for 50^{th} Percentile SN_{eff}	0	0	0	0	0.35	0	0	0	0	0	0
IDOT Method	Existing HMA Layer Coefficient	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Base Layer Coefficient	0.09	0.09	0.09	0.09	0.09	0.09	0.09	N/A	N/A	N/A	N/A
	Subbase Layer Coefficient	N/A	N/A	N/A	0.07	N/A	N/A	N/A	N/A	N/A	N/A	N/A
	SN_{eff}	1.71	1.57	1.53	1.7	2.58	1.95	1.95	1.875	2.85	1.95	1.95
	SN_{req} (IBV=6)	1.90	1.90	1.90	1.90	3.1	1.9	1.9	2.8	2.9	2.9	2.9
	HMA Overlay Requirement (in.)	0.48	0.84	0.92	0.5	1.3	0	0	2.31	0.125	2.375	2.375
AI Method	HMA Overlay Requirement (in.)	0	0	0	0	2	0	0	2.3	0	2.3	2

Table 4.1: Overlay Thickness Designs Using 1993 AASHTO NDT, IDOT, and AI Deflection Methods (cont'd)

		Sec 12	Sec 13	Sec 14	Sec 15	Sec 16	Sec 17	Sec 18	Sec19	Sec 20
1993 AASHTO NDT	Traffic Factor	0.31	0.31	0.31	1.52	1.56	1.35	0.43	0.43	0.43
	90^{th} Percentile SN_{eff}	6.18	3.98	3.61	7.72	7.29	6.17	5.04	4.75	4.36
	Median SN_{eff}	5.5	3.21	3.06	5.54	5.05	4.79	4.12	3.9	3.63
	10^{th} Percentile SN_{eff}	4.63	2.7	2.67	4.04	4.16	3.74	3.41	3.33	2.97
	SN_{req} (IBV=6)	2.9	2.9	2.9	3.68	3.70	3.60	3.2	3.2	3.2
	Overlay Requirement (in.), for 50^{th} Percentile SN_{eff}	0	0	0	0	0	0	0	0	0
IDOT Method	Existing HMA Layer Coefficient	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
	Base Layer Coefficient	N/A	N/A	N/A	0.09	0.09	0.09	0.09	0.09	0.09
	Subbase Layer Coefficient	N/A	N/A	N/A	N/A	N/A	N/A	0.07	N/A	0.07
	SN_{eff}	2.55	2.175	2.625	3.045	3.045	3.045	2.87	3.03	2.84
	SN_{req} (IBV=6)	2.9	2.9	2.9	3.68	3.70	3.60	3.2	3.2	3.2
	HMA Overlay Requirement (in.)	0.875	1.81	0.69	1.59	1.64	1.39	0.825	0.425	0.9
AI Method	HMA Overlay Requirement (in.)	1.5	2.6	2.3	0	0	0	0	0	0

4.5 PROPOSED MECHANISTIC-EMPIRICAL (M-E) OVERLAY DESIGN METHOD

Addressing the issues associated with using the empirical layer coefficients in the IDOT BLRS methodology, this research study aimed to develop an improved M-E overlay design method for local roads and streets in Illinois. This proposed overlay design approach is based on proper structural evaluation of the existing pavements through the use of FWD testing and adopts asphalt-fatigue (bottom-up alligator) cracking and pavement rutting as the design criteria. This section provides an overview of the proposed M-E overlay design method.

4.5.1 Layer Moduli Adjustment Using Layered Elastic, and Finite Element–Based Pavement Analyses

The extensively tested and validated ILLI-PAVE finite element (FE) pavement analysis program (Raad and Figueroa 1980) was used together with the linear elastic theory–based software program BISAR (1989) to carry out modulus backcalculation for the individual pavement layers. 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 response parameters. As discussed in detail in Chapter 2, 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. 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 basin 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. Table 4.2 lists the iteratively calculated layer modulus values using ILLI-PAVE. Figure 4.7 shows a fairly good match between the field-measured (median) and ILLI-PAVE predicted deflection values.

Table 4.2: Iteratively Calculated Layer Moduli using ILLI-PAVE to Match FWD Deflection Basins

Section Number	HMA Modulus (ksi)	Base/Subbase E_r (ksi) = K (ksi) $\left(\frac{\theta}{p_0}\right)^n$	Subgrade Modulus (ksi)
1	600	K=2.5, n=0.33	14
2	800	K=2, n=0.33	12
3	600	K=4, n=0.33	12
4	550	K_{base}=4.2, n_{base}=0.33 $K_{subbase}$=2.5, $n_{subbase}$=0.33	12
5	300	K=4, n=0 .33	11
6	200	K=4.5, n=0.5	6.8
7	425	K=4.9, n=0.5	8
8	100	N/A	7.9
9	100	N/A	11
10	80	N/A	7.8
11	90	N/A	8.5
12	90	N/A	8.5
13	80	N/A	7.5
14	80	N/A	6.8
15	775	K=7, n=0.5	14
16	775	K=7, n=0.5	14
17	775	K=7, n=0.5	14
18	250	Base: K=7, n=0.5 Subbase: K=5, n=0.5	15
19	300	K=6, n=0.5	15
20	200	Base: K=5.8, n=0.5 Subbase: K=2, n=0.5	17.9

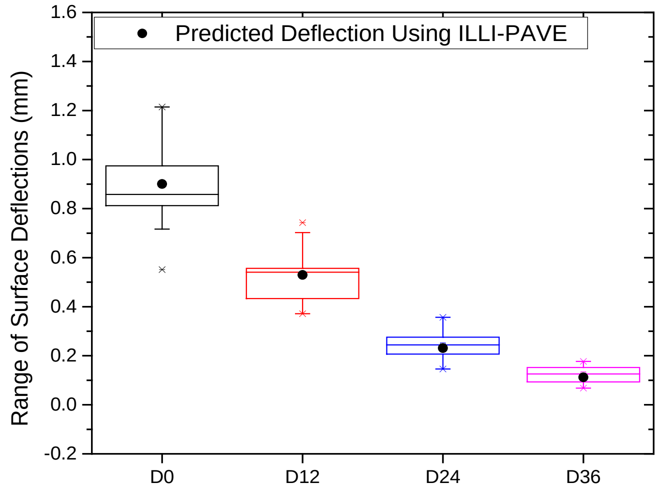


Figure 4.7: Deflection matching with ILLI-PAVE and ANN-Pro.

4.6 OVERLAY THICKNESS DETERMINATION

Upon completion of the layer moduli estimation, 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 4.1 and 4.2). 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 N_f value was then used to compute the threshold-critical pavement responses for the different pavement sections.

$$N_f = \frac{8.78 \times 10^{-8}}{(\varepsilon_t)^{3.5}} \tag{4.1}$$

$$N_f = \frac{5.73 \times 10^{10}}{(\delta_v)^4} \tag{4.2}$$

Whether or not the pavement section requires an overlay was determined by comparing the ε_t and δ_v values under the current pavement configuration, with the threshold values calculated using Equations 4.1 and 4.2. The threshold values of tensile strain at the bottom of asphalt layer *(ε_t)* and vertical surface deflection *(δ_v),* along with the corresponding values under different FWD tests are listed in Table 4.3.

Table 4.3: Critical Pavement Responses Compared with 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 (FWD Set 1)		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

As indicated in Table 4.3, the M-E overlay design method adequately captures the structural inadequacies of the pavement sections for the original, before-rehabilitation pavement configurations. Sections 5 and 20, and Sections 9 through 14 fail both under the fatigue as well as rutting algorithms (Table 4.3). 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. Table 4.4 presents HMA overlay thickness requirements determined by the M-E overlay design method.

Table 4.4: 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	1.25	4.33E-4	33.42	YES
2	1.25	4.44E-4	38.50	YES
3	1.5	4.24E-4	34.22	YES
4	1.5	4.56E-4	37.22	YES
5	3	2.36E-4	18.34	YES
6	Not Required	N/A	N/A	YES
7	Not Required	N/A	N/A	YES
8	4.5	2.72E-4	20.17	YES
9	2.5	2.45E-4	17.49	YES
10	5.5	2.34E-4	18.72	YES
11	5	2.01E-4	16.72	YES
12	3.5	2.58E-4	19.46	YES
13	5	2.48E-4	19.63	YES
14	5	2.21E-4	19.28	YES
15	Not Required	N/A	N/A	YES
16	Not Required	N/A	N/A	YES
17	Not Required	N/A	N/A	YES
18	1.5	2.20E-4	15.16	YES
19	2.0	1.91E-4	13.63	YES
20	3.5	2.00E-4	14.16	YES

Table 4.5 presents information similar to that of Table 4.3 one year after the original FWD testing. Sections 1 through 4 and Section 7 appear to be performing adequately, satisfying both the fatigue and rutting criteria, thus confirming the accuracy of the overlay thickness requirements provided by the proposed M-E overlay design method. Note that these threshold-critical pavement responses were calculated using future traffic demand for a design period of 20 years. But Section 5, which did not receive an overlay in a timely fashion, deteriorated further; and the structural inadequacy of Section 5 is also visible in Table 4.5. However, Section 6 failed under excessive rutting and fatigue damage. This finding could be because the overlay was installed one year after the initial testing and immediately before the Set 2 FWD tests were carried out. Apparently, the pavement condition deteriorated more during that one-year period due to both traffic loading and a possibly weaker subgrade that could have accumulated considerable damage after a harsh winter season. Further, during FWD testing in July 2014, the pavement surface temperature was 113°F, which also resulted in significantly high deflections due to low surface moduli. Note that coring information was not available for Sections 5, 6, and 7; instead, the layer configurations were estimated by the local agency.

Table 4.5: Critical Pavement Responses for Sections FWD Tested One Year After the Initial Testing

Section Number	Critical Pavement Response Parameters One Year After Initial Set of Testing		Capacity > Demand (Design Period = 20 Years)
	ε_t	δ_v (mil)	
1	5.07E-4	35.72	YES
2	4.79E-4	38.58	YES
3	3.61E-4	30.20	YES
4	3.37E-4	28.37	YES
5	4.76E-4	30.74	NO
6	1.034E-5	55.08	NO
7	3.49E-4	32.89	YES

Various features of the 1993 AASHTO NDT, the IDOT, the AI, and the proposed M-E overlay design methods are compared in Table 4.6. The M-E method presents a significant improvement over the 1993 AASHTO NDT and the IDOT modified layer coefficients methods by combining mechanistically computed pavement responses along with pre-established pavement damage algorithms. A flowchart of steps involved in the new M-E overlay design method is presented in Figure 4.8.

Table 4.6: Comparisons of 1993 AASHTO NDT, IDOT, and Proposed M-E Overlay Design Methods

<table>
<tr><th>Characterization Approach</th><th>1993 AASHTO NDT Method</th><th>IDOT Method</th><th>M-E Overlay Design Method</th><th>AI Method</th></tr>
<tr><td>Surface Layer (HMA)</td><td rowspan="2">One structural number (SN_{eff}) assigned to the entire pavement structure above the subgrade</td><td rowspan="2">Empirical layer coefficients available for limited number of material types</td><td>Constant modulus assigned based on iterative calculation using ILLI-PAVE to match deflection basin obtained from FWD testing</td><td rowspan="3">Elastic modulus is assigned assuming the whole existing pavement section as a homogenous half-space with a Poisson's ratio of 0.5</td></tr>
<tr><td>Base Layer</td><td>Stress-dependent resilient modulus (K-θ model) assigned based on typical stress states estimated at layer mid-depth</td></tr>
<tr><td>Subgrade Layer</td><td>Resilient modulus, M_R from equation</td><td>IBV Based</td><td>Constant modulus assigned based on iterative calculation using ILLI-PAVE to match deflection basin obtained from FWD testing</td></tr>
<tr><td>Pavement Structural Number (SN)</td><td>Current and required SN values represented by SN_{eff} and SN_{req}</td><td>Current and required SN values represented by SN_{eff} and SN_{req}</td><td>N/A</td><td>N/A</td></tr>
<tr><td>Load Characterization Parameter</td><td>ESAL</td><td>Traffic Factor (ESAL in millions)</td><td>ESAL</td><td>ESAL</td></tr>
<tr><td>Consideration of New and Nontraditional Materials</td><td>Effect indirectly incorporated through FWD center deflection</td><td>Effect of nontraditional materials cannot be incorporated</td><td>Can be directly incorporated through K-θ Model</td><td>Effect indirectly incorporated through FWD center deflection</td></tr>
</table>

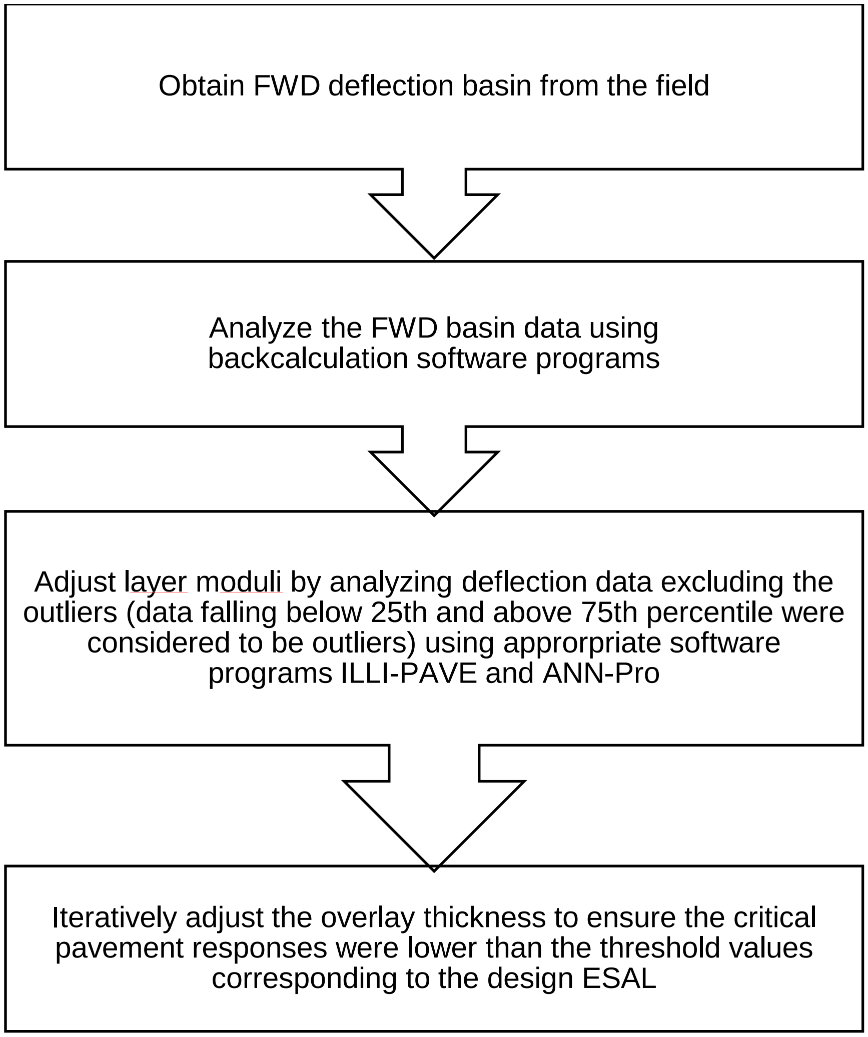


Figure 4.8: Flowchart of the proposed mechanistic-empirical (M-E) overlay design procedure.

Table 4.7 summarizes all the HMA overlay thickness calculations based on the 1993 AASHTO NDT, the IDOT, the AI, and the proposed M-E overlay design methods. Please note that the overlay thicknesses presented for the IDOT method are the minimum thicknesses as suggested by the BLRS manual (2012), based on the required traffic numbers of each pavement section.

Table 4.7: Summary of the Required Overlay Thicknesses for All the Methods

Section	Proposed M-E Overlay Method (in.)	IDOT Modified Layer Coefficients Method* (in.)	1993 AASHTO NDT Method (in.)	AI Deflection Method (in.)
1	1.25	2	No Overlay Required	No Overlay Required
2	1.25	2	No Overlay Required	No Overlay Required
3	1.5	2	No Overlay Required	No Overlay Required
4	1.5	2	No Overlay Required	No Overlay Required
5	3	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	4.5	3	No Overlay Required	2.3
9	2.5	3	No Overlay Required	No Overlay Required
10	5.5	3	No Overlay Required	2.3
11	5	3	No Overlay Required	2
12	3.5	3	No Overlay Required	1.5
13	5	3	No Overlay Required	2.6
14	5	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	1.5	3	No Overlay Required	No Overlay Required
19	2	3	No Overlay Required	No Overlay Required
20	3.5	3	No Overlay Required	No Overlay Required

*Minimum thicknesses as suggested by the IDOT BLRS manual (2012).

As indicated in Table 4.7, many pavement sections, such as Sections 1 through 4, required lower thickness requirements than those calculated by the IDOT method. However, both the 1993 AASHTO NDT and the AI deflection methods characterized these sections as structurally sound to carry the intended traffic volume and subsequently resulted in no thickness requirements. Sections 8 through 14 required higher thicknesses, mostly because these pavement sections were tested at high pavement temperatures, and further adjustments to HMA backcalculated moduli to accommodate temperature changes could result in thinner thickness requirements. Sections 6, 7, 15, 16, and 17 did not require any form of overlay according to the proposed M-E overlay design method.

4.6.1 Effect of Temperature Corrections on FWD Deflections and HMA Overlay Thicknesses

As FWD deflection basins are obtained at different ambient and pavement temperatures, in order to 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. This process can be achieved through a two-step procedure. The first step requires determining the HMA temperature at a desired depth of the HMA layer, followed by the second step of adjusting either the FWD center deflection or 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. Table 4.8 provides a summary of these past studies that present temperature correction models found in the literature.

Table 4.8: Summary of Temperature Correction Models Found in the Literature

Temperature Correction Model	Developed by	Model Parameters
$\frac{E_{ref}}{E_{AC}} = \frac{1}{1 - 2.2log(\frac{T_{AC}}{T_{ref}})}$	Stubstad et al. 1994	E_{ref} and E_{AC} = Reference and Backcalculated Asphalt Moduli T_{ref} = Reference Temperature (20°C) T_{AC} = Temperature at 1/3 of Pavement Thickness (°C)
$\frac{E_{ref}}{E_{AC}} = 10^{-0.018(20 - T_{AC})}$	Baltzer and Jansen 1994	
ATAF = $10^{slope(T_r - T_m)}$	Lukanen et al. 2000	*ATAF* = Asphalt Temperature Adjustment Factor *Slope* = Slope of the log Modulus Versus Temperature Curve T_r = Reference Temperature (°C) T_m = Pavement Temperature at Mid-Depth (°C)
$\frac{E_{68}}{E_T} = 10^{-0.0153(68 - T)}$	Kim et al. 1995	E_{68} = Asphalt Modulus at 68°F (20°C) E_T = Backcalculated Asphalt Modulus at Temperature *T* *T* = Temperature of the Asphalt Pavement at Mid-Depth
$\frac{E_{std}}{E_{field}} = 10^{-0.0002175(70^{1.886} - T^{1.886})}$	Johnson and Baus 1992	E_{std} = AC Modulus at Standard (Reference) Temperature (70°F) E_{field} = AC Modulus at Field Temperature *T* = Measured Temperature (°F)
$\frac{S_T}{S_{15}} = 1 - 1.384\log(\frac{T}{15})$	Ullidtz and Peattie 1982	S_T and S_{15} = Asphalt Moduli at Temperatures of T (°C) and 15°C
$\frac{E_{T0}}{E_T} = \frac{1}{3.177 - 1.673\, log(T)}$	Ullidtz 1987	E_{T0} and E_T = Asphalt Moduli at Temperatures of T_0 (°C) and *T* °C
$\frac{E_{T1}}{E_{T2}} = \frac{1.635 - 0.0317\, T_1}{1.635 - 0.0317\, T_2}$	Antunes 1993	E_{T1} and E_{T2} = Asphalt Moduli at Temperatures of T_1 (°C) and T_2 °C
$\frac{E_{Tw}}{E_{Tc}} = \frac{(1.8\, T_c + 32)^{2.4462}}{(1.8\, T_w + 32)^{2.4462}}$	Chen et al. 2000	E_{Tw} and E_{Tc} = Asphalt Moduli at Temperatures of *Tw* (°C) and T_c °C (Mid-Depth Temperature)
$\frac{E_r}{E_0} = 10^{-0.02822\,(25 - T_c)}$	Chang et al. 2002	E_r = Measured Modulus at Temperature T_c E_0 = Adjusted Modulus to 25°C T_c = Asphalt Mid-Depth Temperature (°C)

In addition, the research team contacted Applied Research Associates to document their temperature correction procedures implemented in FWD testing in Illinois. They are mainly adopted from AASHTO and Asphalt Institute methods and are included in Appendix B.

To study the effect of temperature correction on overlay thickness designs, three cases from the previously discussed pavement sections were chosen to reanalyze the corresponding FWD test data for interpreting the pavement layer moduli and to determine the final overlay thicknesses using the proposed M-E overlay design method. These case study pavements are Section 2 in McHenry County, Section 10 in Vermilion County, and Section 20 in Ogle County. These sections were selected because FWD testing was conducted on Section 2 at a low pavement temperature (45°F), on Section 10 at a very high pavement temperature (highest temperature was recorded as 133°F), and on Section 20 at a high pavement temperature (98°F). The layer thickness profiles, along with the adjusted layer moduli values, are presented in Figure 4.9.

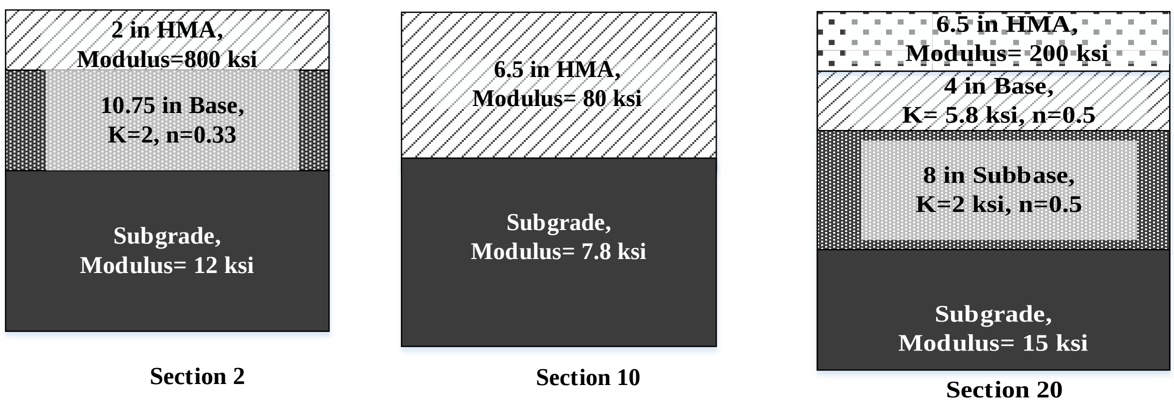


Figure 4.9: Selected pavement sections for studying the influence of temperature correction.

The temperature prediction model in Equation 4.1 developed by Park et al. (2001) was used 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 model developed in a Long-Term Pavement Performance (LTPP) program study (Lukanen et al. 2000), as shown in Equation 4.2.

$$T_z = T_{surf} + (-0.3451z - 0.0432z^2 + 0.00196z^3) sin(-6.3252t + 5.0967) \quad (4.1)$$

where

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

$$ATAF = 10^{slope(T_r - T_m)} \tag{4.2}$$

where

ATAF: Asphalt temperature adjustment factor
Slope: Slope of the log modulus versus temperature curve (used here, the default of –0.021 is suggested)
T_r: Reference temperature of 21°C
T_m: Pavement temperature at mid-depth (°C)

Table 4.9 summarizes the temperature correction procedure applied to the selected case studies and lists the HMA moduli adjusted to 21°C. After the temperature correction was incorporated, the HMA moduli ranged from 347 ksi to 394 ksi, which are slightly lower than the range of moduli values recommended by the backcalculation software program Modulus 6.0 (HMA moduli ranged from 460 ksi to 1,240 ksi according to MODULUS 6.0). Accordingly, Table 4.10 lists the new overlay thicknesses determined for the three pavement sections. The overlay thicknesses determined significantly decrease for Sections 10 and 20, which were FWD tested at high pavement temperatures and therefore gave rather thick HMA overlay requirements earlier. Note that for Section 2, the overlay thickness increased only slightly after applying the temperature correction, which caused a significant decrease in the HMA modulus. This finding is probably due to the low predicted ESALs over the design life, as such low traffic volume increased the limit of the acceptable critical pavement responses and resulted in subsequent low overlay thickness requirement.

Table 4.9: Temperature Correction of the Backcalculated HMA Modulus

Case Studies	T_{surf} (°C)	T_z (°C) at Mid-Depth (Eq. 4.1)	ATAF (Eq. 4.2)	Backcalculated HMA Modulus (ksi)	Adjusted HMA Modulus to 21°C (ksi)
Section 2	7.2	6.36	0.49	800	394
Section 10	56.11	51.73	4.42	80	354
Section 20	36.67	32.4	1.73	200	347

Table 4.10: Overlay Thickness Determination with Temperature Correction

Case Studies	Predicted ESAL	Adjusted HMA Modulus (ksi)	Overlay Thickness (in.)	
			Before Temperature Adjustment	After Temperature Adjustment
Section 2	13,524	394	1.25	1.5
Section 10	310,336	354	5.5	2
Section 20	437,311	347	3.5	1.5

4.7 COST COMPARISONS

The cost of installing an HMA overlay over a one-mile-long pavement section was estimated based on the typical costs associated with FWD testing (including mobilization) and material costs listed in Table 4.11 and Table 4.12, respectively. Note that, interpretation of FWD test data also usually requires agencies to do coring and/or GPR testing to determine layer thicknesses that could add to the cost.

Table 4.11: FWD Testing, One Lane-Mile (27 data points)

Item	Unit	Average Unit Price* ($/hour)
FWD Testing	Hour	$300
Analysis of FWD data	Hour	$125
Traffic Control	Hour	$125
Total Cost, $/hour		$550

* Phone communication with Douglas Steele of Applied Research Associates.

Table 4.12: Material Type, Cost, and Quantity Calculation

Mix Type	IL 9.5-mm dense graded HMA
G_{mm}	2.5
AC, %	6.0
N_{design}	50
Binder	PG 64-22
Quantity of Material Required, Tons per lane-mile/inch	394.0
Total Cost, $/ton	92.00

Table 4.13 summarizes the cost of constructing the required HMA overlay over a one-mile-long section. As indicated in Table 4.11, the cost of conducting an FWD analysis is only $550 per lane-mile per hour, including the mobilization cost, which decreases when greater lengths of road segments are FWD tested. Typically, it takes about an hour to conduct FWD testing every 200 ft on a mile-long road segment. According to Table 4.13, the cost of implementing the M-E overlay design method seems to be the most expensive, followed by the IDOT method, AI deflection method, and the 1993 AASHTO NDT method. For about ten pavement sections tested, the M-E overlay design method gives a lower-cost overlay alternative than the requirement from the IDOT modified layer coefficients method (Table 4.13). Also, note that the thicknesses presented in Table 4.7 were taken as the basis for arriving at these cost numbers in Table 4.13. Accordingly, proper temperature correction procedures adopted in FWD testing and asphalt surface layer modulus backcalculation could result in lower HMA overlay requirements, as highlighted for Sections 10 and 20 in Table 4.10.

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. 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 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.

Table 4.13: Cost of Constructing HMA Overlay ($/lane-mile/in.)

Section	Proposed M-E Overlay Design Method	IDOT Modified Layer Coefficients Method	1993 AASHTO NDT Method	AI Deflection Method
1	***$45,860****	$72,496	No Overlay Required	No Overlay Required
2	***$45,860****	$72,496	No Overlay Required	No Overlay Required
3	***$54,922****	$72,496	No Overlay Required	No Overlay Required
4	***$54,922****	$72,496	No Overlay Required	No Overlay Required
5	$109,294	$108,744	$13,237	$73,046
6	***No Overlay* Required***	$72,496	No Overlay Required	No Overlay Required
7	***No Overlay* Required***	$72,496	No Overlay Required	No Overlay Required
8	$163,666	$108,744	No Overlay Required	$83,920
9	***$91,170****	$108,744	No Overlay Required	No Overlay Required
10	$199,914	$108,744	No Overlay Required	$83,920
11	$181,790	$108,744	No Overlay Required	$73,046
12	$127,418	$108,744	No Overlay Required	$52,922
13	$181,790	$108,744	No Overlay Required	$94,795
14	$181,790	$108,744	No Overlay Required	$83,920
15	***No Overlay* Required***	$144,992	No Overlay Required	No Overlay Required
16	***No Overlay* Required***	$144,992	No Overlay Required	No Overlay Required
17	***No Overlay* Required***	$144,992	No Overlay Required	No Overlay Required
18	***$54,922****	$108,744	No Overlay Required	No Overlay Required
19	***$73,046****	$108,744	No Overlay Required	No Overlay Required
20	$127,418	$108,744	No Overlay Required	No Overlay Required

*A lower-cost rehabilitation option when compared with the current IDOT method.

CHAPTER 5 SUMMARY OF FINDINGS AND RECOMMENDATIONS

Local and state highway agencies dedicate a significant portion of their annual pavement management and pavement rehabilitation budget toward assessing the condition of in-service pavements. However, an accurate evaluation of the functional as well as the structural deficiencies of the existing pavement structure is necessary to select an adequate, effective, and economical rehabilitation strategy. Accordingly, the structural conditions of existing pavements should be investigated through the use of proper nondestructive testing (NDT) and sensor technologies. This project was initiated to demonstrate the advantages of NDT testing and pavement evaluation for local agency (municipalities, counties, and townships) pavement rehabilitation. The intent was to develop improved hot-mix asphalt (HMA) overlay thickness design alternatives for low-volume roads, based on proper structural evaluation of existing in-service pavements through NDT methods, such as the falling weight deflectometer (FWD) test.

The following are the summary highlights, major observations, and important findings of this research study:

- In coordination with local agencies and the Illinois DOT (IDOT) Bureau of Local Roads and Streets (BLRS), 20 pavement sections located in six counties in the state of Illinois were selected in this research study to conduct FWD tests on deteriorated pavements and to evaluate their structural conditions for pavement design and rehabilitation.
- FWD tests were conducted just before the HMA overlay placement in all the pavement sections. Some of the sections were also tested immediately after the overlay placement and one year after the overlay placement to monitor the structural conditions and condition deteriorations of the pavement sections.
- Three commonly used overlay thickness design approaches (i.e., the 1993 AASHTO NDT method, the IDOT modified layer coefficients method, and the Asphalt Institute (AI) deflection method) were used with the specific data gathered from the tested pavement sections to design and recommend HMA overlay thicknesses.
- Because of the empirical nature and other limitations of the currently used overlay design methods, a mechanistic-empirical (M-E) overlay design method was developed to design HMA overlays for low-volume flexible pavements in Illinois. 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 with 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 with 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 with those based on the minimum thickness requirement by the IDOT method—except for Section 8 and Sections 10 through 14.

- Many temperature correction procedures are available to adjust either the center deflection of the FWD deflection data or the backcalculated HMA moduli to a reference temperature. A recently proposed temperature prediction model was used to calculate mid-depth HMA temperatures from the pavement surface temperatures measured during FWD testing and to successfully demonstrate the importance of temperature correction on the overlay thicknesses.
- The thicker overlay requirements from the proposed M-E overlay design method often resulted from the fact that these pavements were tested with FWD at high daytime pavement temperatures. Further temperature adjustments to the backcalculated HMA moduli reduced the thickness requirement significantly.

Local agencies should be encouraged to use FWD testing to assist in the determination of rehabilitation strategies for low-volume roadways in Illinois. Such testing, as highlighted in this report, will allow the agency 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 in the methods currently used to determine rehabilitation strategies on low-volume roadways in Illinois.

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APPENDIX A FIELD DEFLECTION BASINS OBTAINED DURING FWD TESTING

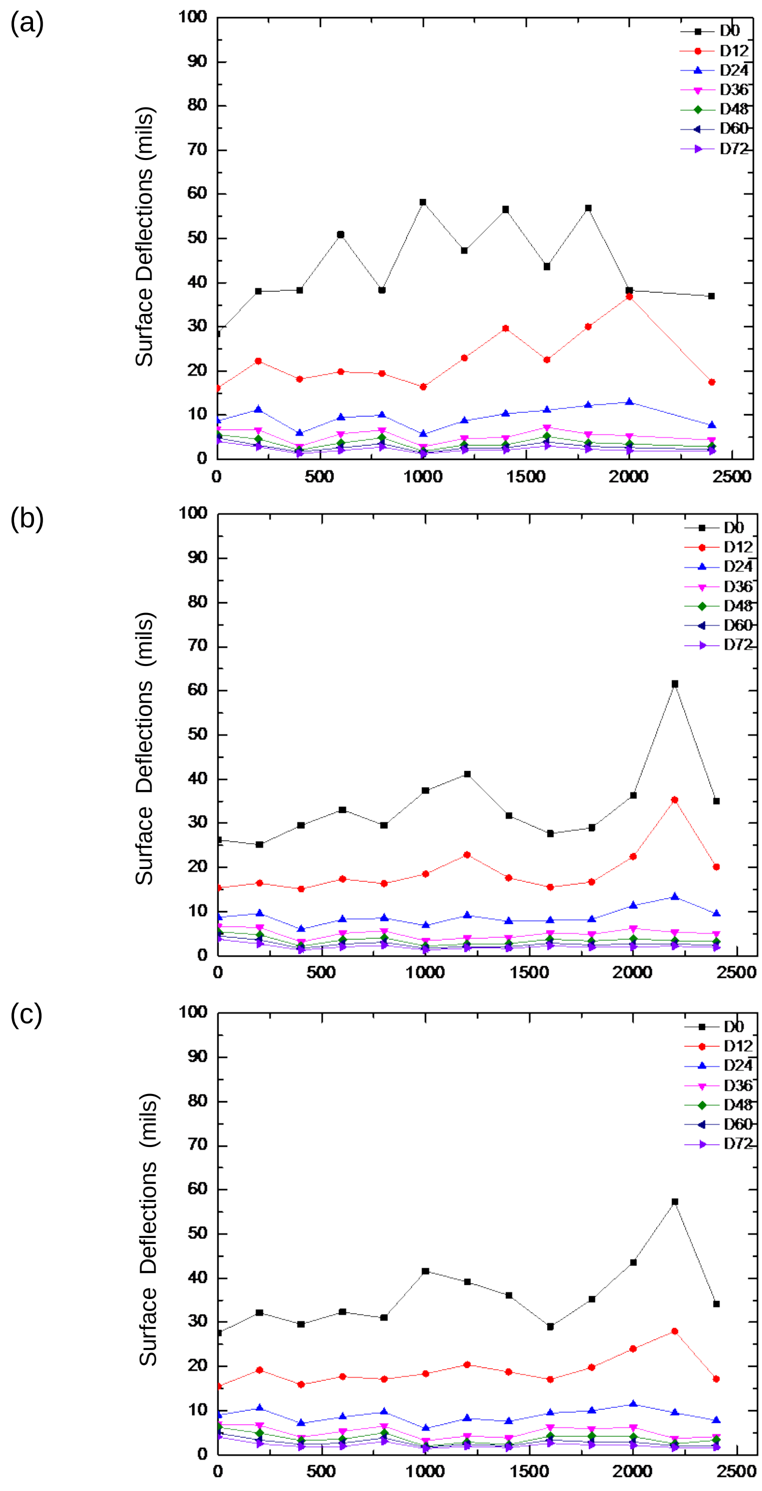


Figure A.1: Deflection basins obtained from the field during (a) Set 1, (b) Set 2, and (c) Set 3 FWD testing efforts for pavement section 3.

(a)

(b)

(c)

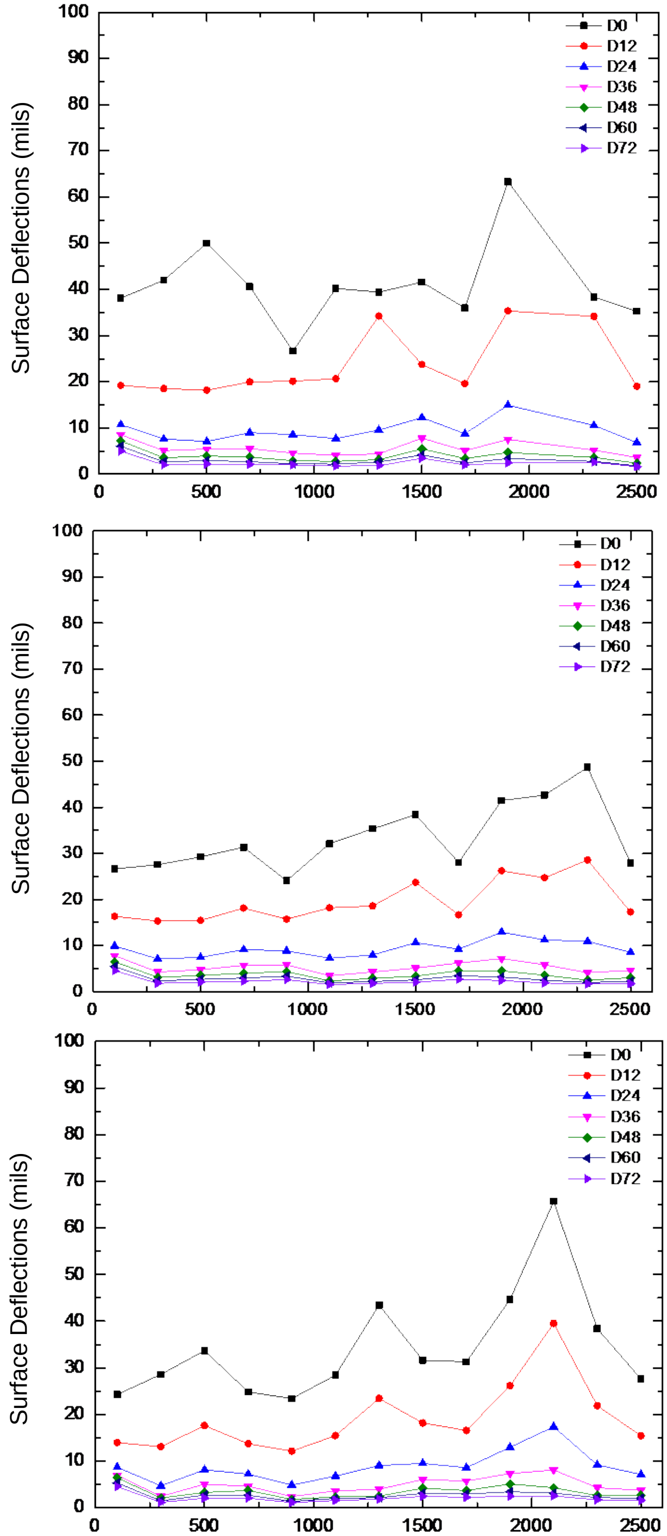


Figure A.2: Deflection basins obtained from the field during (a) Set 1, (b) Set 2, and (c) Set 3 FWD testing efforts for pavement section 4.

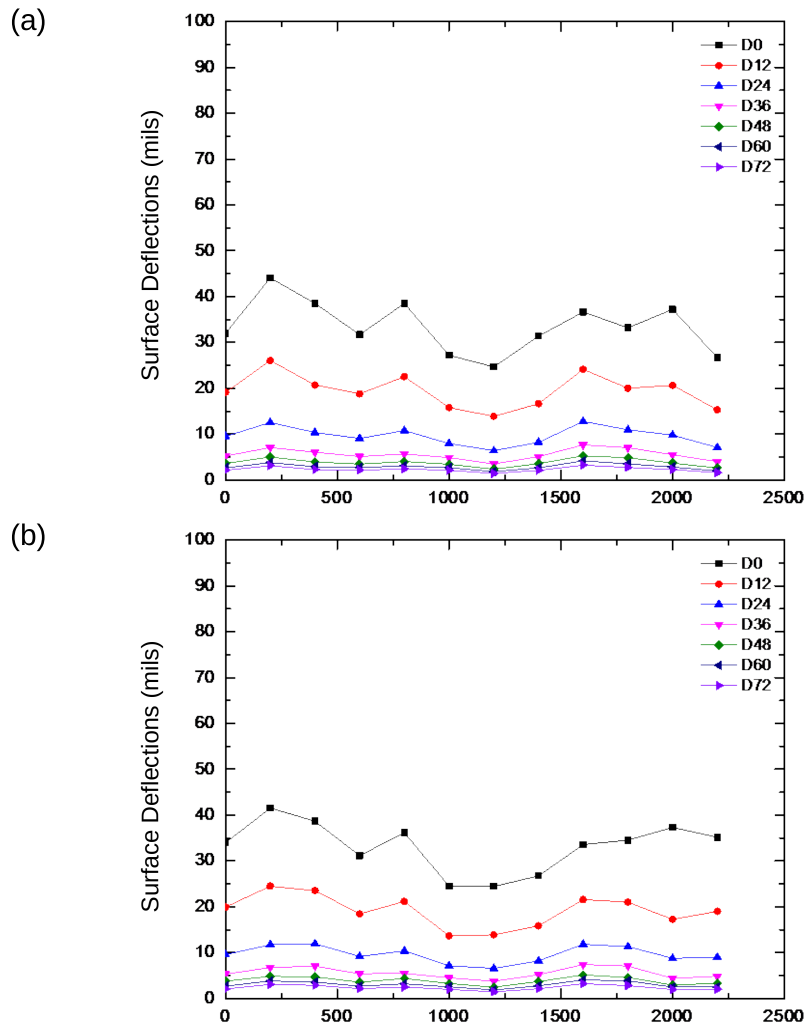
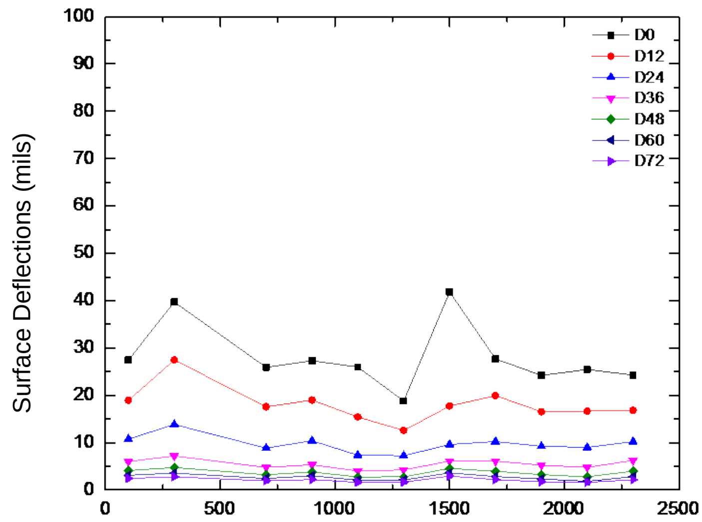


Figure A.3: Deflection basins obtained in east direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 5.

(a)



(b)

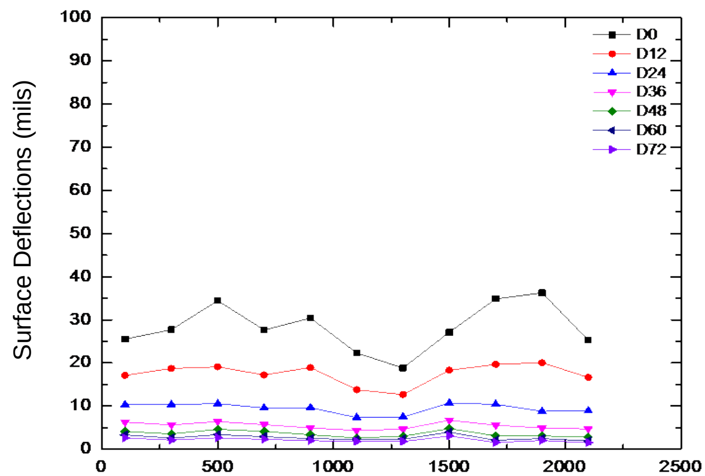
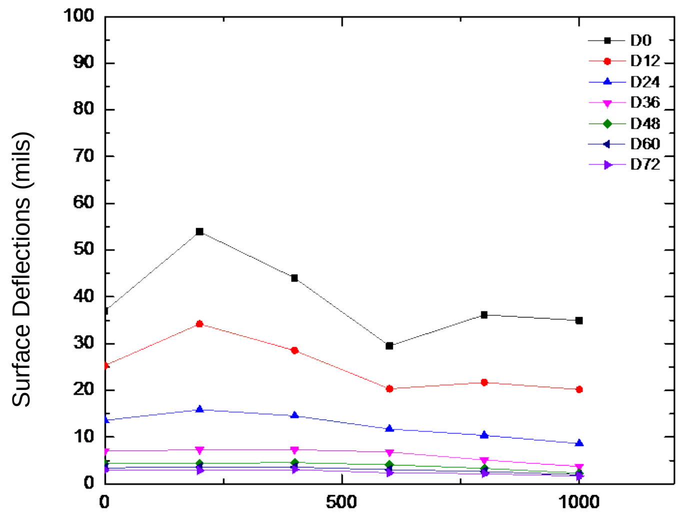


Figure A.4: Deflection basins obtained in the west direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 5.

(a)



(b)

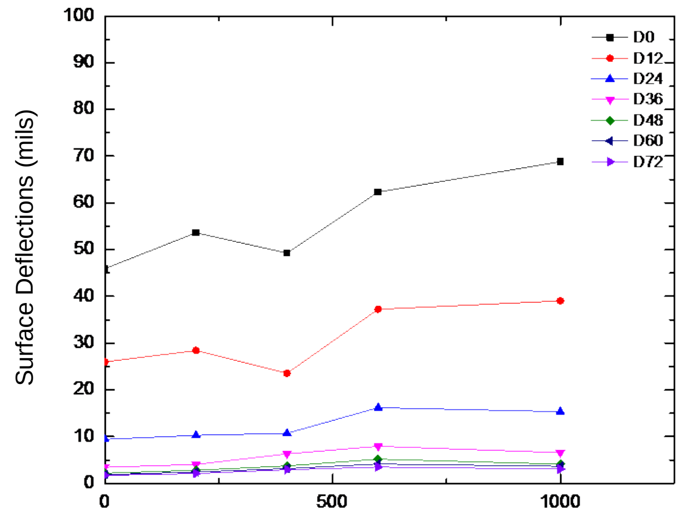


Figure A.5: Deflection basins obtained in north direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 6.

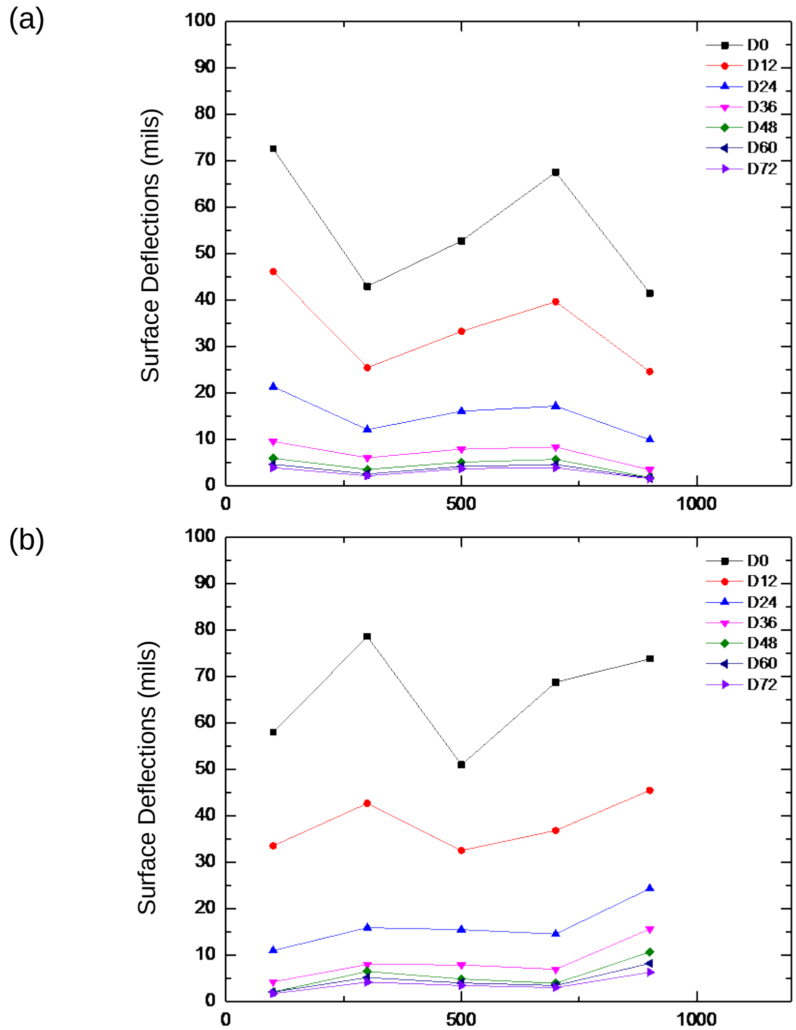
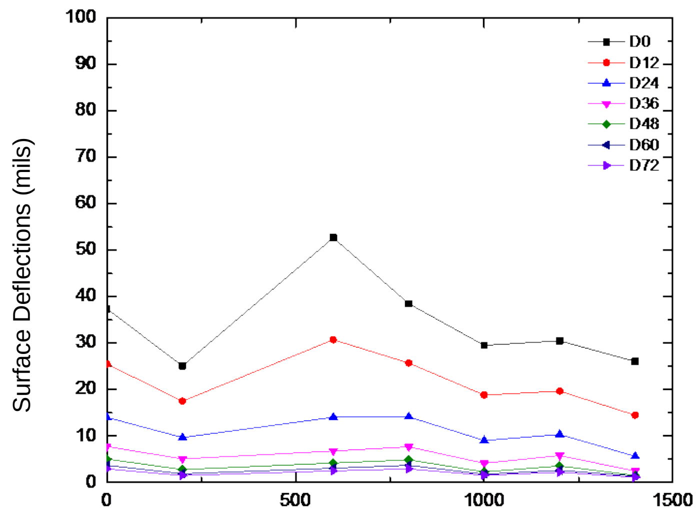


Figure A.6: Deflection basins obtained in south direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 6.

(a)



(b)

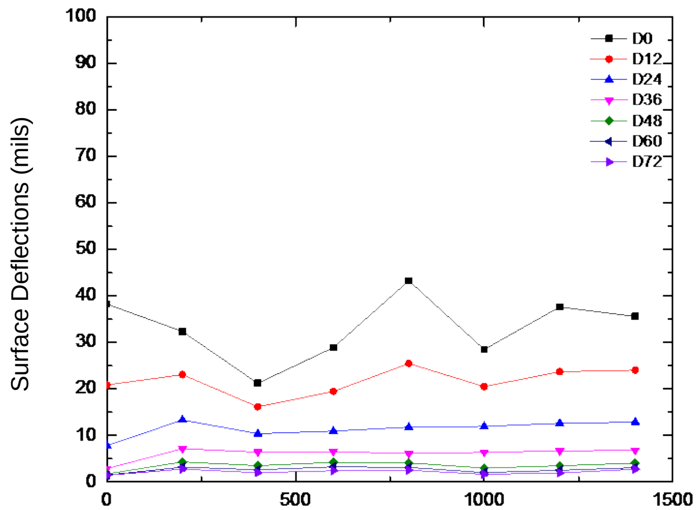
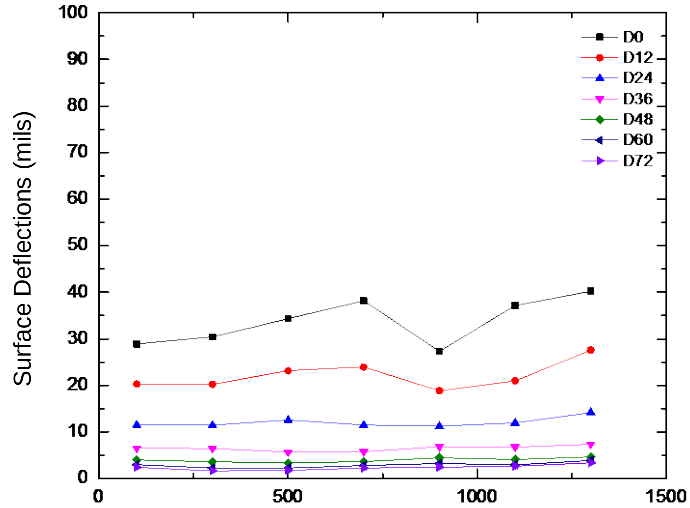


Figure A.7: Deflection basins obtained in north direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 7.

(a)



(b)

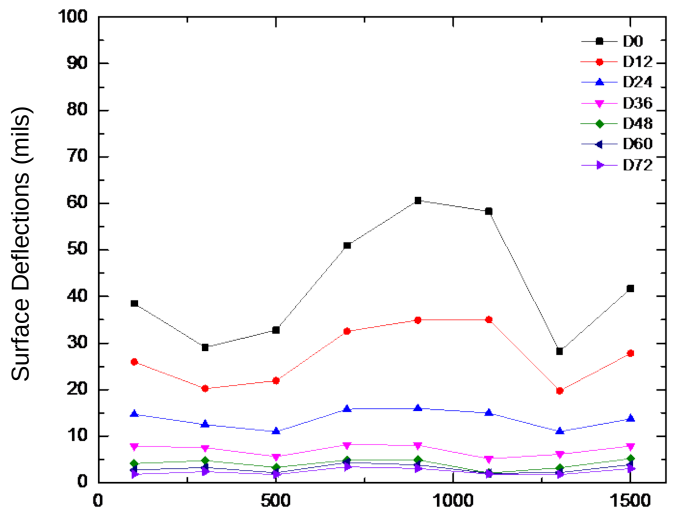
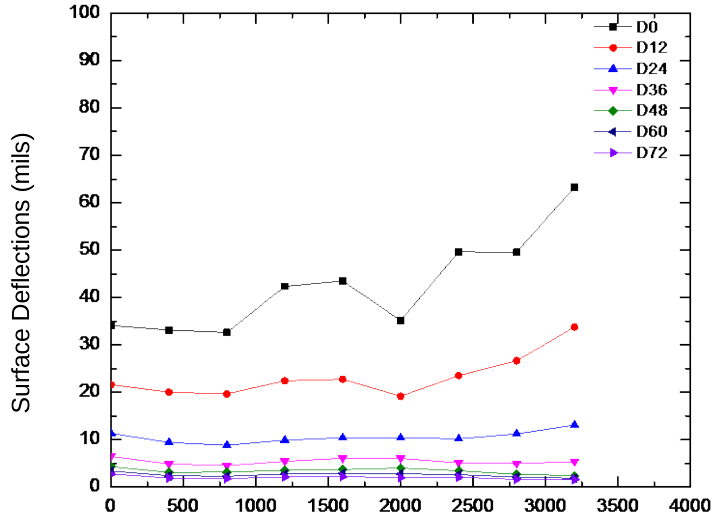


Figure A.8: Deflection basins obtained in south direction from the field during (a) Set 1, and (b) Set 3 FWD testing efforts for pavement section 7.

(a)



(b)

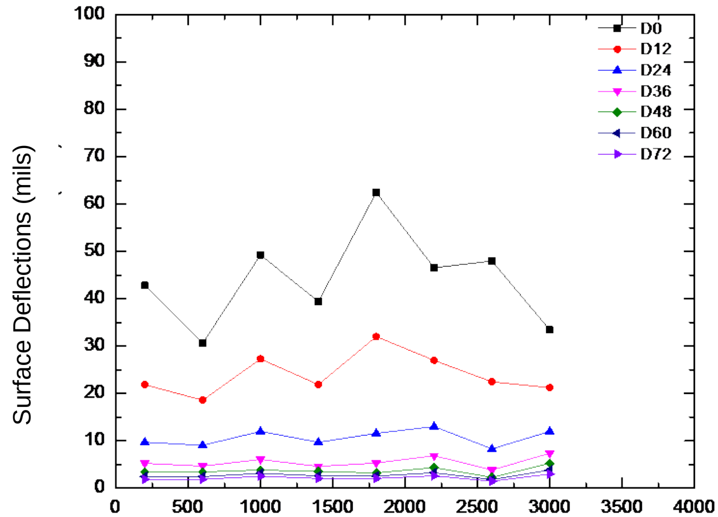


Figure A.9: Deflection basins obtained from the field during (a) in east direction, and (b) in west direction during Set 1 FWD testing efforts for pavement section 8.

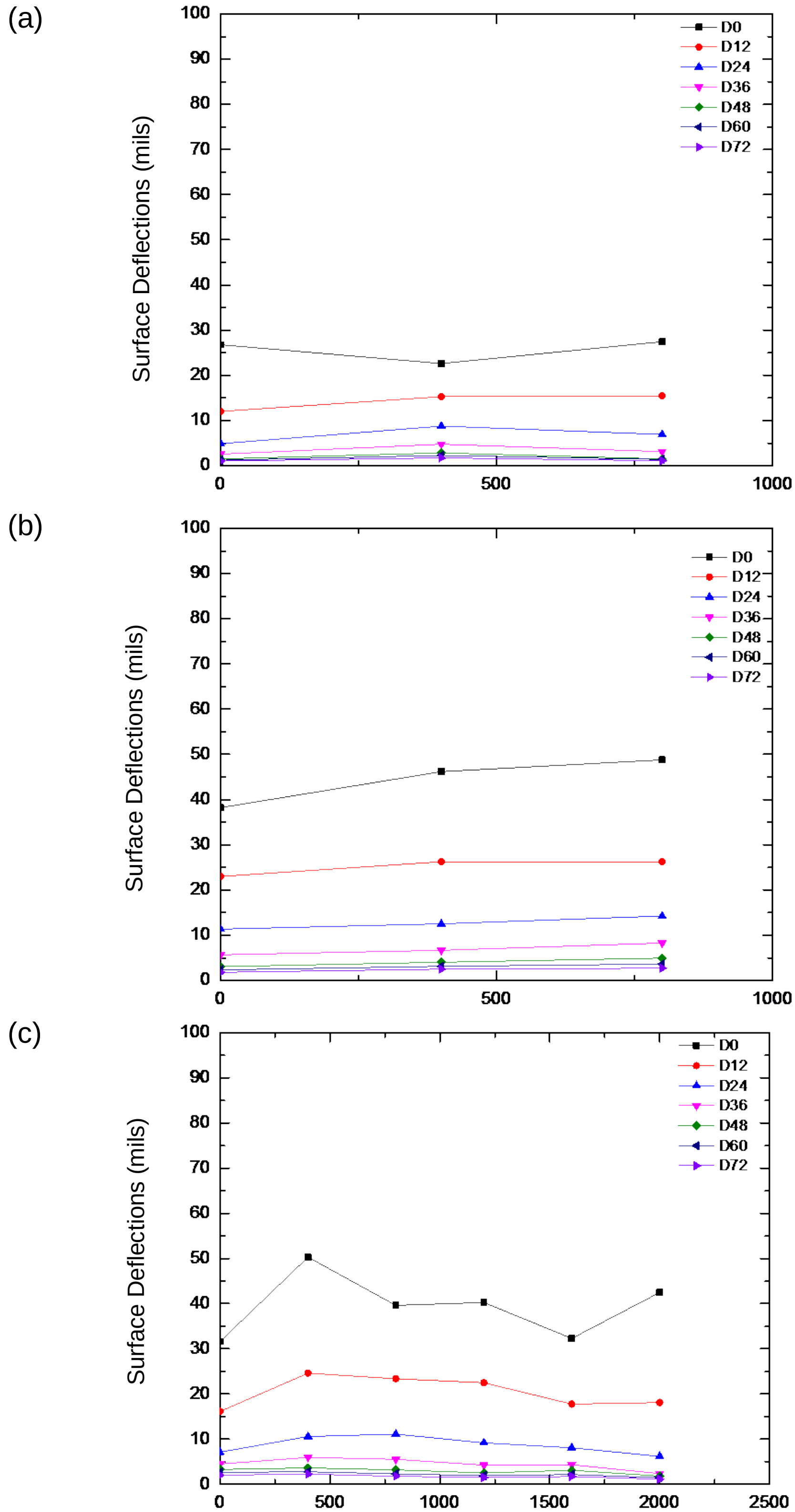


Figure A.10: Deflection basins obtained from the field during Set 1 FWD testing efforts for pavement section (a) 9, (b) 10, and (c) 11.

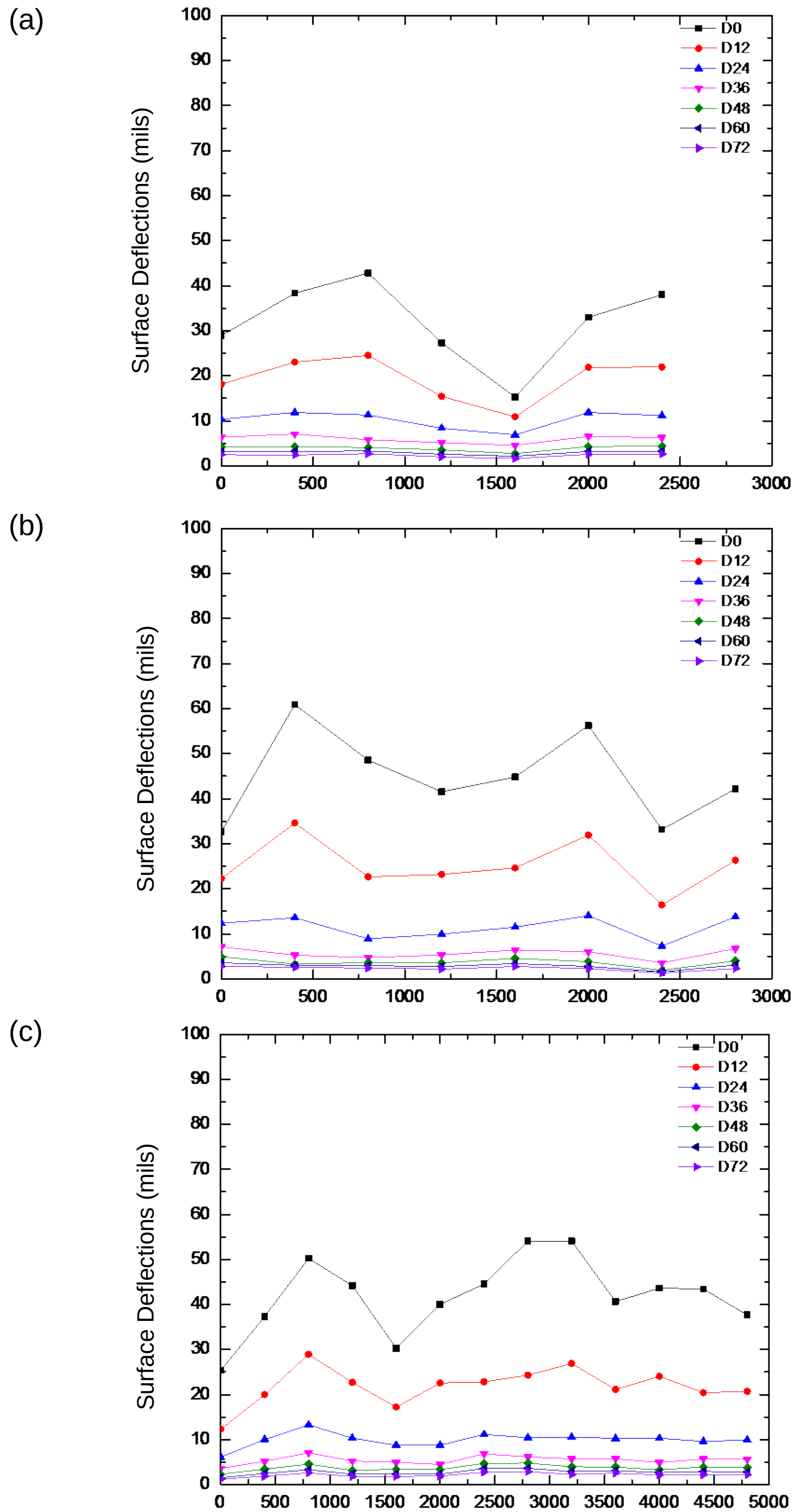


Figure A.11: Deflection basins obtained from the field during Set 1 FWD testing efforts for pavement section (a) 12, (b) 13, and (c) 14.

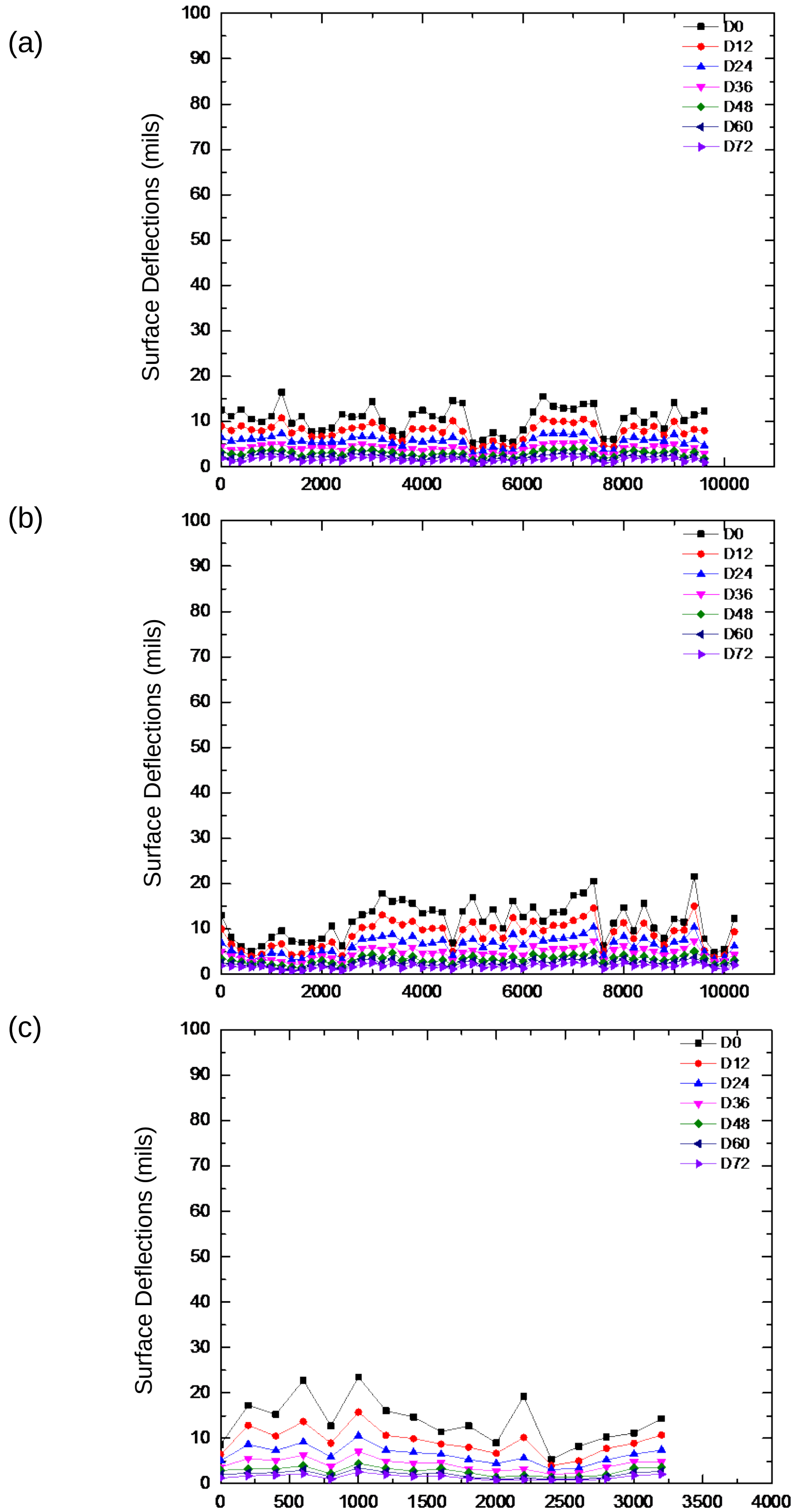


Figure A.12: Deflection basins obtained from the field during Set 1 FWD testing efforts for pavement section (a) 15, (b) 16, and (c) 17.

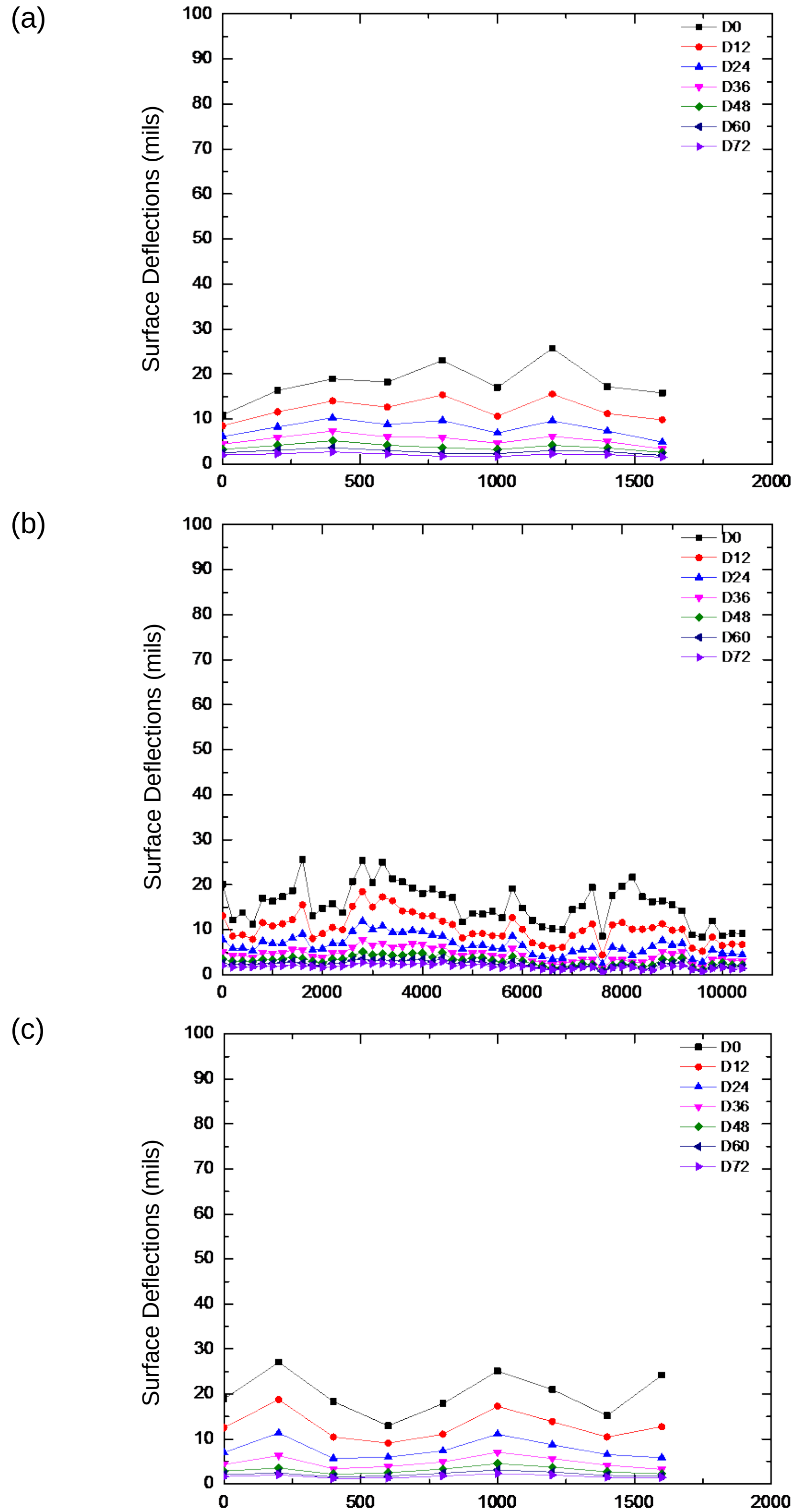
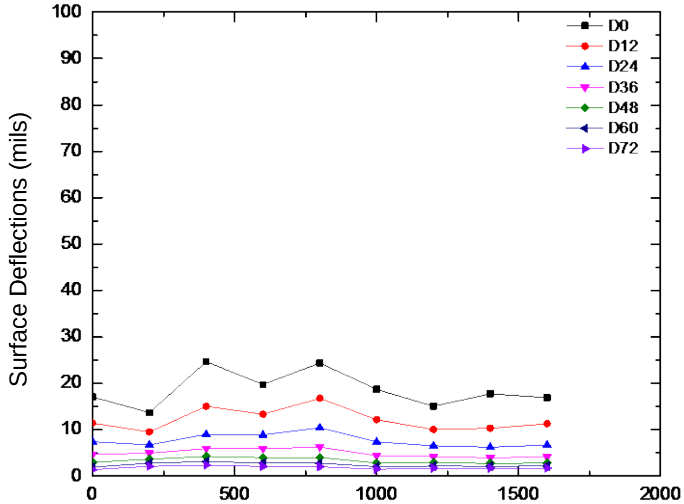
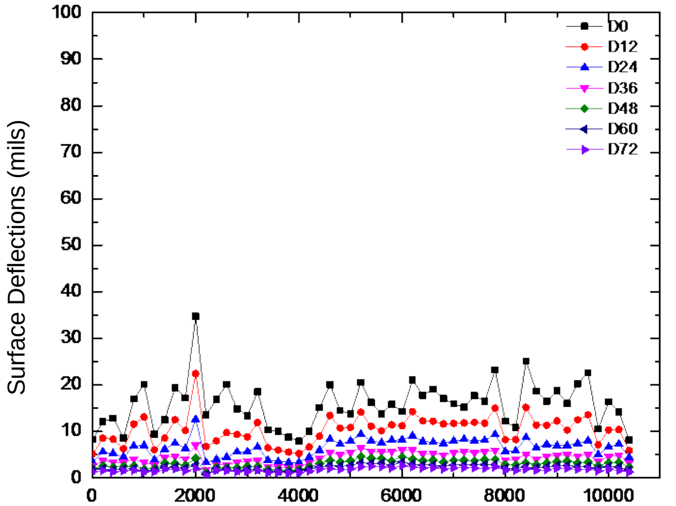


Figure A.13: Deflection basins obtained in the south direction from the field during Set 1 FWD testing efforts for pavement section (a) 18, (b) 19, and (c) 20.

(a)



(b)



(c)

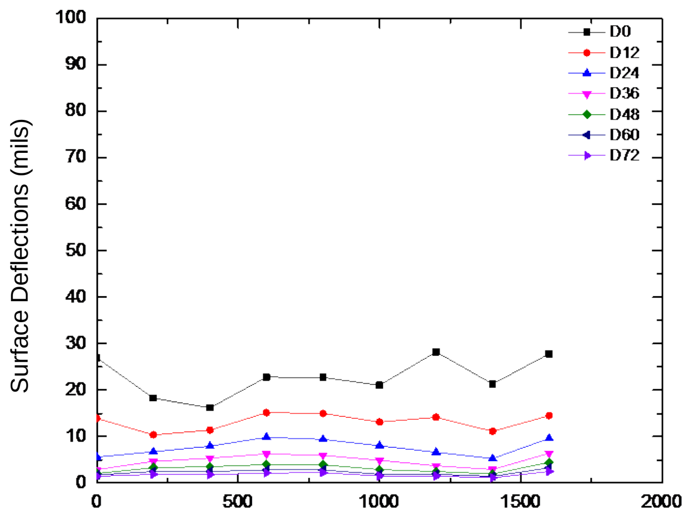
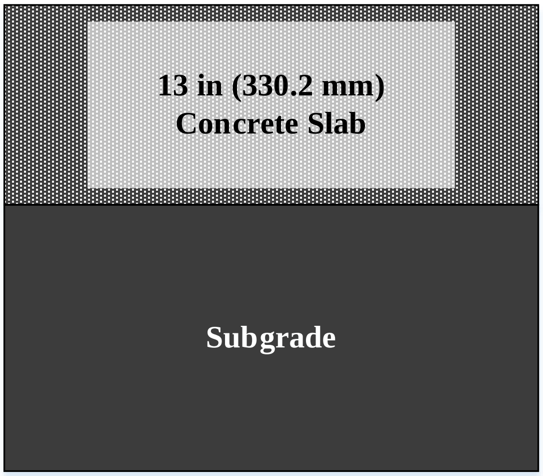
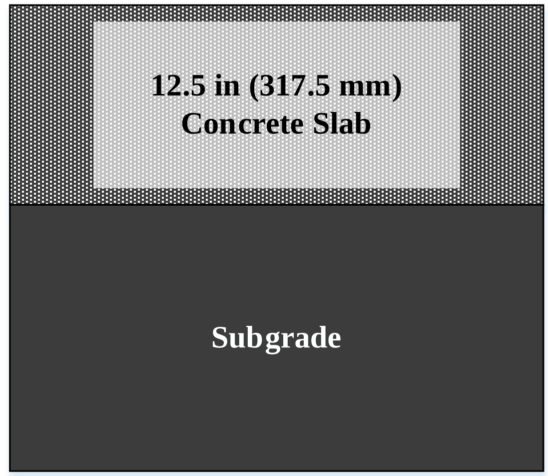


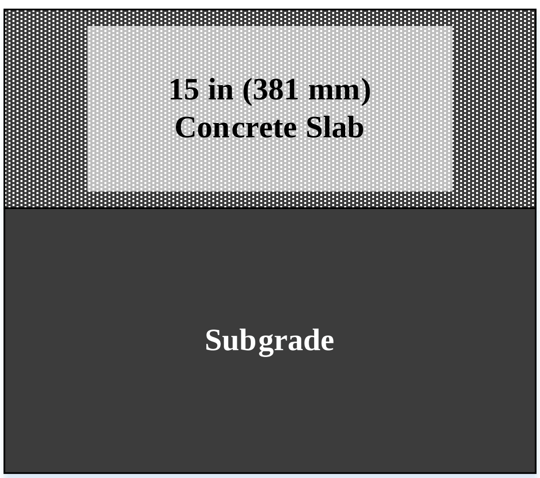
Figure A.14: Deflection basins obtained in the north direction from the field during Set 1 FWD testing efforts for pavement section (a) 18, (b) 19, and (c) 20.



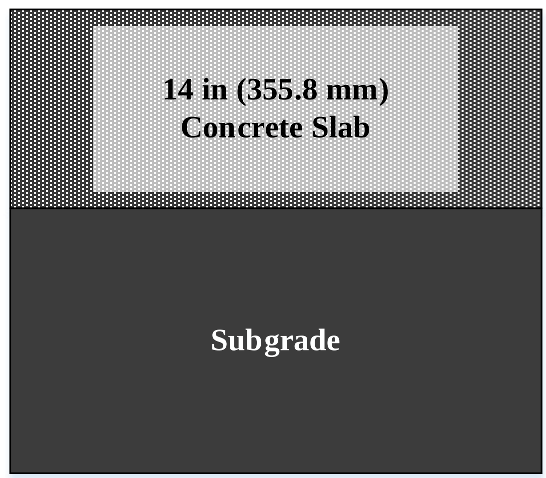
Composite Section-1
(East Direction)
Class III Roadway
with Daily Traffic <1500



Composite Section-1
(West Direction)
Class III Roadway
with Daily Traffic <1500



Composite Section-2
(East Direction)
Class III Roadway
with Daily Traffic <1500



Composite Section-2
(West Direction)
Class III Roadway
with Daily Traffic <1500

Figure A.15: Layer configurations and traffic information for composite sections in Vermilion County, Illinois.

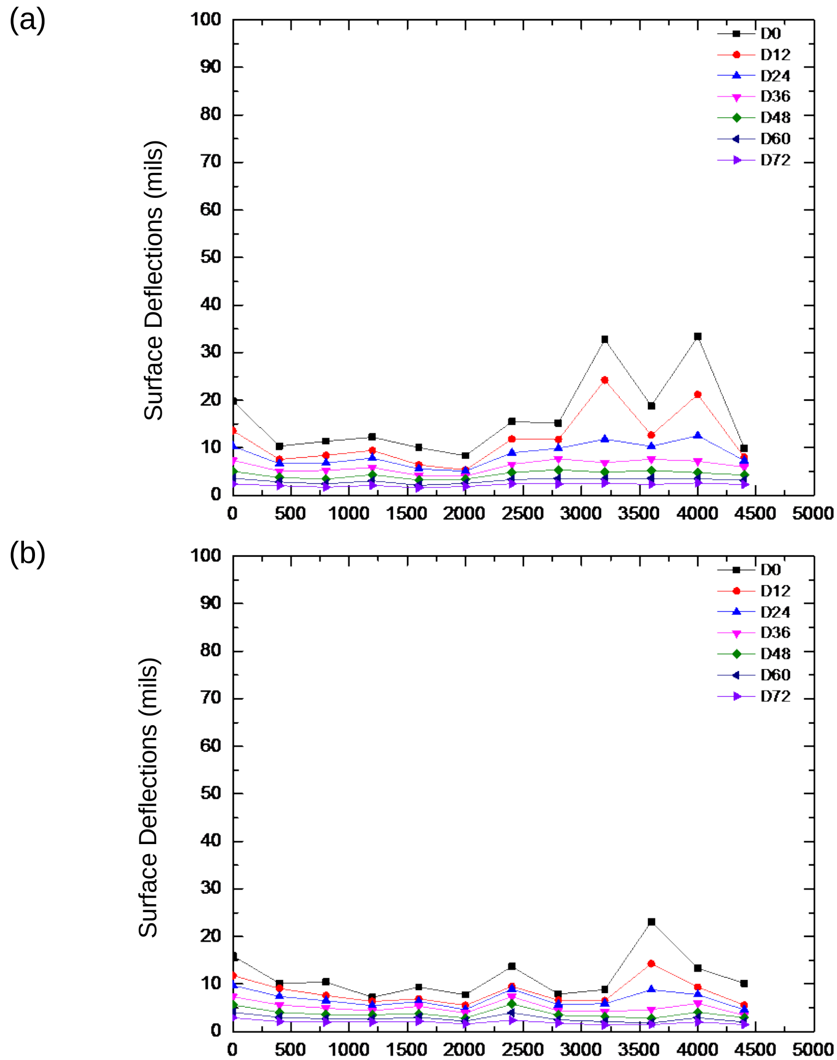


Figure A.16: Deflection basins obtained from the field during Set 1 FWD testing efforts for pavement composite section 1 (a) in east direction, and (b) in west direction.

(a)

(b)

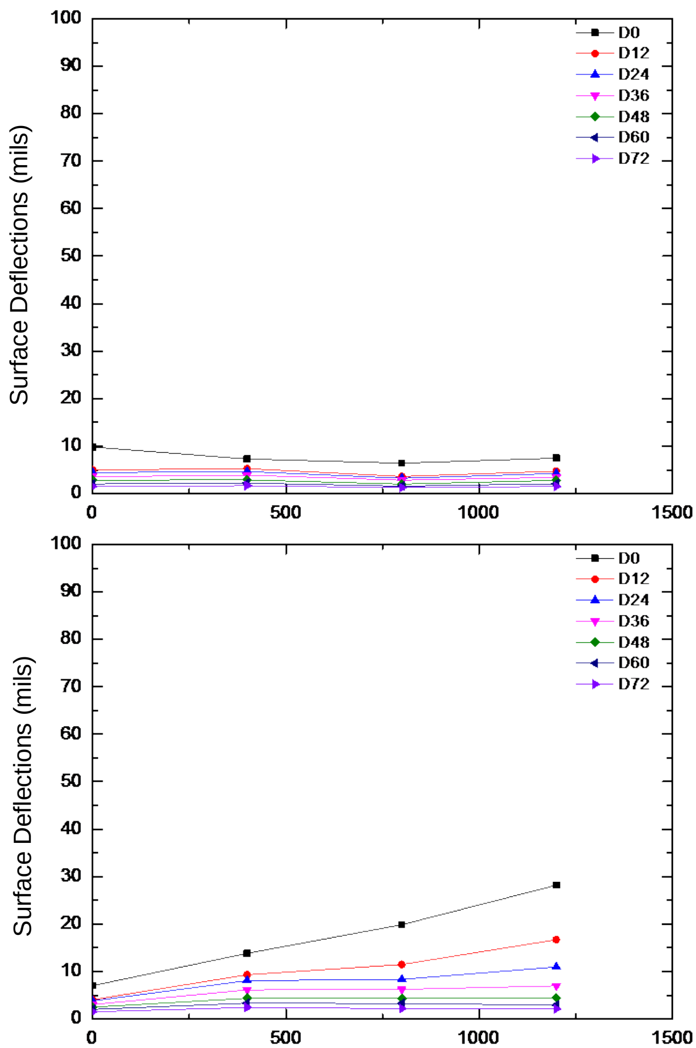


Figure A.17: Deflection basins obtained from the field during Set 1 FWD testing efforts for pavement composite section 2 (a) in east direction, and (b) in west direction.

APPENDIX B FWD TEMPERATURE CORRECTION PROCEDURES

Doug Steele, Senior Engineer, Applied Research Associates (ARA), Inc.
(Personal Communication, January 2, 2015)

BACKGROUND

ARA adjusts FWD data in two methods to account for variable field temperatures and the effect of temperature variations on flexible pavement deflections.

Method 1—Normalize Maximum Deflection to a Standard Temperature Prior to Backcalculating with the 1993 AASHTO Flexible Pavement Procedure

The AASHTO method models the pavement as two layers only—the subgrade, plus the combination of all layers above the subgrade. It uses an outer sensor to determine the subgrade modulus and the maximum deflection (D_0) to determine the effective pavement modulus (E_p) and effective structural number (SN_{eff}). While the outer sensors are considered to reflect subgrade stiffness only and are not temperature dependent, D_0 is normalized to 68°F prior to backcalculation.

- Step 1—Estimate the asphalt concrete (AC) layer mid-depth temperature using the BELLS method (AASHTO T317-2004). BELLS requires the pavement surface temperature and time of day of the testing, the previous day's mean air temperature, and the asphalt layer thickness.
- Step 2—Use the AC mid-depth temperature from BELLS and the AC layer thickness to determine a temperature adjustment factor from the 1993 AASHTO guide (Figure 5.6 for granular bases).
- Step 3—Multiply the adjustment factor by the FWD-measured D_0 to get a temperature adjusted D_0.
- Step 4—Perform backcalculation with the temperature adjusted D_0 and the non-temperature adjusted outer deflections. The backcalculated E_p and SN_{eff} are normalized to 68°F and used in the AASHTO design procedure.

Method 2—Backcalculate Using Raw Deflections and Normalize the Backcalculate AC Moduli to a Standard Temperature

Most multi-layer, linear elastic backcalculation programs perform analysis on the raw (non-temperature adjusted) FWD deflections, using all available sensors to determine the best fit between theoretical and FWD-measured deflections. Therefore, the backcalculated moduli reflect the temperature conditions at the time of testing. To account for varying temperatures due to daily or seasonal effects, the backcalculated AC moduli are normalized to a standard temperature.

- Step 1—Backcalculate using the program of your choice (e.g., Evercalc, Modulus, MODCOMP, Elmod).
- Step 2—Estimate the AC mid-depth temperature using the BELLS method described above.
- Step 3—Input the AC mid-depth temperature, AC mix parameters, and the FWD loading frequency into the Asphalt Institute equation (MS-1) for AC modulus prediction (E_{AC}).
- Step 4—Determine the difference between the predicted E_{AC} and the backcalculated E_{AC}, based on field temperatures.

- Step 5—Determine the predicted E_{AC} at a standard temperature (e.g., 70°F). Add (or subtract) the difference determined in Step 4 to the predicted E_{AC} at the standard temperature. This value is the backcalculated E_{AC} adjusted to a standard temperature, used for design.

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