USDOT Region V Regional University Transportation Center Final Report

NEXTRANS Project No. 094IY04

Development of Improved Pavement Rehabilitation Procedures Based on FWD Backcalculation

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Report Submission Date: January, 2015

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Funding for this research was provided by the NEXTRANS Center, Purdue University under Grant No. DTRT07-G-005 of the U.S. Department of Transportation, Research and Innovative Technology Administration (RITA), University Transportation Centers Program. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. This document is disseminated under the sponsorship of the Department of Transportation, University Transportation Centers Program, in the interest of information exchange. The U.S. Government assumes no liability for the contents or use thereof.

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TECHNICAL SUMMARY

NEXTRANS Project No. 094IY04 Final Report, September, 2014

Development of Improved Pavement Rehabilitation Procedures Based on FWD Backcalculation

Introduction

Hot Mix Asphalt (HMA) overlays are among the most effective maintenance and rehabilitation alternatives in improving the structural as well as functional performance of flexible pavements. HMA overlay design procedures can be based on: (1) engineering judgment, (2) pavement component analysis, (3) non-destructive testing (NDT) with limiting defection criteria, and (4) mechanistic-empirical analysis and design. Although different state highway agencies have different methodologies in designing HMA overlay thickness, design procedures are more or less following or modifying the 1993 AASHTO Pavement Design Guide procedure, which is an empirical based approach using the structural deficiency concept and generally listed in above categories 1 and 2. The lack of mechanical testing for evaluating the structural conditions of existing, in-service pavements often leads to unsafe and uneconomical practices as far as the rehabilitation of low volume roads is concerned. This research study presents a mechanistic-empirical (M-E) approach for overlay thickness designs of flexible pavements through a combination of NDT and pre-established pavement damage models. Structural conditions of a n umber of in-service pavement sections were tested in the field using a Falling Weight Deflectometer (FWD) test device. The required overlay thicknesses of the field pavement sections were then determined using two different methods currently used by local agencies, and the newly developed M-E Overlay Design method. The M-E Overlay Design Method mechanistically backcalculates pavement layer moduli and critical pavement responses due to FWD loading using advanced materials characterization and layered analysis solutions, and then compares them to threshold pavement responses for the fatigue cracking and rutting pavement damage criteria according to pre-established pavement damage algorithms.

Findings

In coordination with local agencies 5 different pavement sections located in 2 counties in the State of Illinois were selected in this research study to conduct FWD tests on these deteriorated pavements and 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. 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 in nature, predicted rather thicker overlays for the pavement sections when compared to the newly developed M-E Overlay Design method. The newly developed 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 compared to the IDOT modified layer coefficients method.

Recommendation

Pavement rehabilitation requires adequate overlay thickness designs critical to a local road agency's ability to maintain its pavement network. Such rehabilitation projects need to be encouraged to properly utilize FWD testing in the structural condition evaluations of existing, in-service pavements. The use of the M-E Overlay Design method developed in this project can prove to be a big step forward for local transportation agencies as far as overlay thickness designs of low volume flexible pavements are concerned. Improved road safety, design reliability and performance will be achieved since mechanistic analysis and design concepts will be fully implemented in the development of HMA overlay structural thickness designs.

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ACKNOWLEDGMENT, DISCLAIMER, MANUFACTURERS' NAMES

The authors would like to thank post-doctoral research associate Dr. Debakanta Mishra and research engineers, James Pforr, Jim Meister, and Dr. Aaron Coenen of the Illinois Center for Transportation, who helped greatly with the Falling Weight Deflectometer (FWD) testing in this project. Finally, we are also grateful to Dr. William Vavrik and Mr. Douglas Steele from Applied Research Associates, Inc. for providing local agency field test data and their insightful feedback on the FWD data analysis.

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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 collecting functional deficiencies, structural condition of a pavement needs to be evaluated 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 significantly improving the structural as well as 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 analysis approaches, such as energy-based and viscoelastic methods, the degree of implementation of such methods to real practice has been found to be often lagging. Some of the factors to have potentially contributed to such differences in the state of the art in research and state of practice in pavement technology are: (a) initial costs associated with the procurement of FWD devices and (b) 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 transportation agencies using highly empirical approaches without any mechanistic analyses. The benefits of using NDT based overlay design methods can be summarized as follows (Kinchen and Temple 1980) :

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

Although the NDT-based overlay thickness design method specified by the 1993 AASHTO Pavement Design Guide (AASHTO 1993) 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 pavement structural condition evaluation methods 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.

Incorporating advanced pavement material characterization and finite element (FE) analysis into mechanistic-empirical (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. The previous NEXTRANS research project of the PI, No.010IY01: “Nondestructive Pavement Evaluation using Finite Element Analysis Based Soft Computing Models,” found that the developed ANN-Pro and SOFTSYS, Soft Computing Based Pavement and Geomaterial System Analyzer, programs was a quick and accurate method to backcalculate in-service pavement layer moduli and thicknesses from the measured FWD deflection basins of flexible pavements analyzed in Illinois, Indiana and Ohio (Tutumluer et al. 2009). The major advantage of using these advanced backcalculation programs was that the most accurate FWD backcalculation analysis results could be obtained at the push of a button based on the sophisticated ILLI-PAVE FE solutions. Note that the validated ILLI-PAVE FE program, developed by (Thompson and Elliott 1985), analyzes full depth and conventional flexible pavements by properly taking into account the nonlinear, stress dependent behavior of subgrade soils and granular base materials.

1.2 Research Objectives

The main objectives of this research study are to (1) identify and evaluate the HMA overlay design procedures currently used by local and state highway agencies in Illinois, Ohio and Indiana by conducting sensitivity analyses to investigate the effect of each input design parameter in the final HMA overlay thickness, (2) develop improved pavement rehabilitation procedures based on FWD test results collected from in-service flexible pavements for layer modulus and critical pavement response backcalculation, and finally, (3) prepare cost comparisons for several overlay design projects summarizing the technical adequacies/inadequacies of the current HMA overlay design methods and the newly developed improved procedure based on FWD testing and backcalculation. The proposed study therefore aims to (i) demonstrate advantages/disadvantages of HMA overlay design procedures currently in use, (ii) document and compare the estimated construction and life cycle costs of the different design alternatives, and finally, (iii) develop an advanced procedure for HMA overlay design that can incorporate critical pavement responses achieved by performing FWD testing on pavement sections. The developed methods will be mechanistic-empirical in nature, and will rely on the analysis of FWD-based NDT results. Results from the newly developed methods will be compared to other methods currently used by transportation agencies, such as the Illinois Department of Transportation (IDOT) Modified AASHTO method (based on Structural Numbers and Layer Coefficients), 1993 AASHTO NDT method, and the Asphalt Institute deflection method.

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 in Illinois and Ohio will be studied for their applicability to local roads to investigate 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 – *Development of Improved Overlay Procedures Based on FWD Testing & M-E Concepts:* 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. IDOT's Dynatest FWD machine currently at the University of Illinois Advanced Transportation Research and Engineering Laboratory (ATREL), which houses Illinois Center for Transportation (ICT), will be re-assembled and used in this research for the FWD testing of local agency rehabilitation projects. Nondestructive field FWD data will be collected together with pavement geometry and materials data and all other details specific to each rehabilitation project to be studied. The field FWD data collected will be analyzed using the previously developed M-E approaches for layer modulus and critical pavement response backcalculation. This 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 type transfer functions established by the IDOT Bureau of Materials and Physical Research (BMPR) and Bureau of Local Roads and Streets (BLRS). 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 gained from the FWD testing and M-E based pavement layer moduli and response backcalculation.

Task 3 – Cost Comparisons of Design Alternatives: Cost comparisons will be established for the overlay thickness designs developed in Tasks 2 and 3 for the studied local road and street rehabilitation projects in order to contrast adequacies/inadequacies of the currently used pavement rehabilitation design practices/procedures and the newly developed FWD testing and backcalculation based HMA overlay thickness design alternative. Since many local agencies are resistant to using the FWD testing because of initial cost of paying a consulting engineer and/or 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 are also presented in Chapter 2 along with the outcomes of the sensitivity analyses 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. Chapter 3 presents the details of the selected case studies and 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 4 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, repeatable, and they can be performed much more rapidly than destructive tests. In addition, an overall cost reduction is typically achieved through these benefits 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. This method is commonly referred to as backcalculation of layer moduli in transportation / pavement engineering. 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 lbs.) on 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 (see Figure 2.1). The load is then transmitted from the rubber buffers to 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 on 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 lbs. in 25-30 milliseconds) (Ulliditz and Stubstad 1985). A typical test configuration is shown in Figure 2.3.

FWD's ability to best replicate the load histories and deflections of a moving vehicle among all the other testing equipment has made it a widely accepted tool worldwide (Hoffman and Thompson 1981, Roesset and Shao 1985, Ulliditz 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 FWD's described in the literature, the three most commonly used and commercially available ones are the following:

1. Dynatest Model 8000 (Dynatest Consulting, Inc.);
2. KUAB FWD Models 50 and 150 (KUAB America);
3. JILS FWD (Foundation Mechanics, Inc.).

2.3 Backcalculation Methods

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

- Simplified methods;
- Gradient relaxation methods; and
- Direct interpolation methods.

Figure 2.1: Dynatest Falling Weight Deflectometer (FWD) device at the University of Illinois

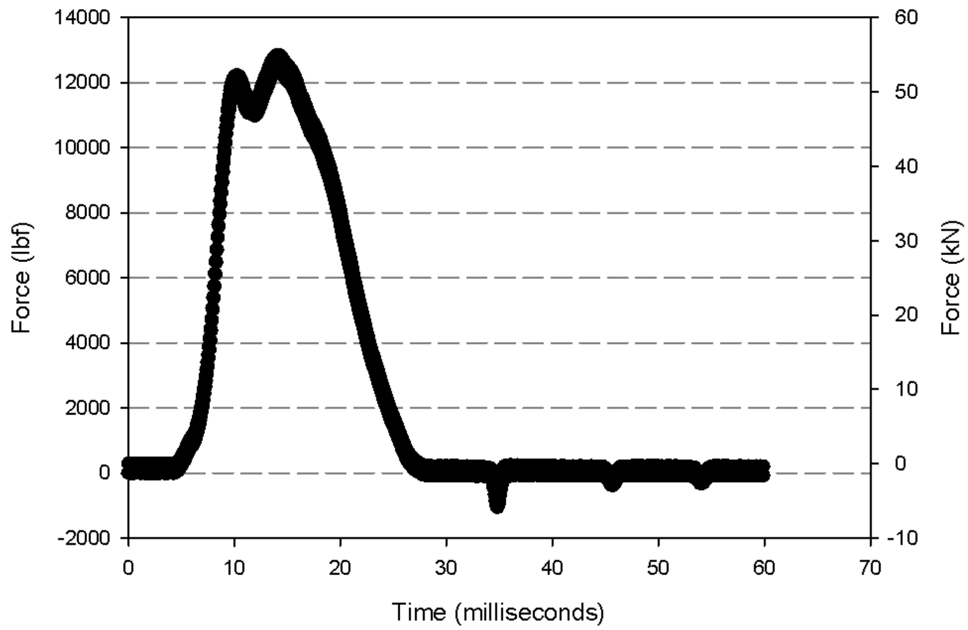


Figure 2.2: Haversine Loading Applied by FWD device

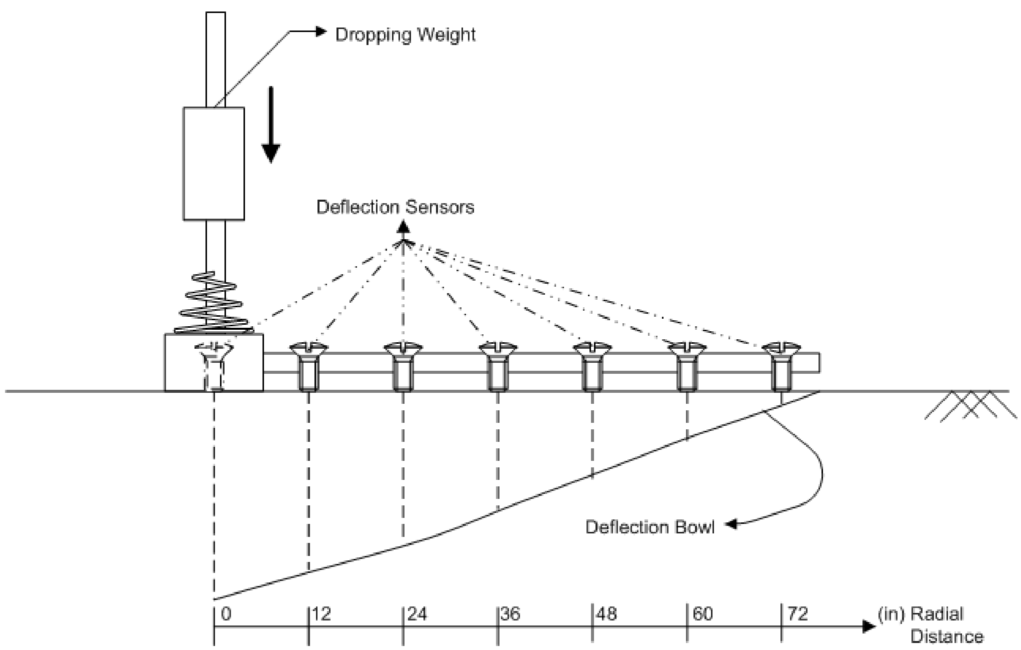


Figure 2.3: Locations of FWD Sensors and Schematic Drawing

These approaches have been used to develop many software applications which 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 that are 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. However, in spite of the drawbacks, 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 in hand and then to compare the estimated value with the experimental values from FWD testing (see 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 that it takes 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, ELMOD are the most commonly used ones. 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 US Army Corps of Engineers included in MODULUS, performs the forward calculation for building a database of 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. Similar to MODULUS, EVERCALC also uses an iterative approach incorporating WESLEA as the forward engine to calculate 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 Augmented Gauss-Newton algorithm, EVERCALC can provide evaluations of layer moduli for up to five layer pavement structures. Unlike EVERCALC and MODULUS, that use the WESLEA elastic layered program, ELMOD, another commonly used backcalculation software program, uses the Odemark equivalent thickness approach. Table 2.1 provides a summary of the key features of some of these backcalculation software programs.

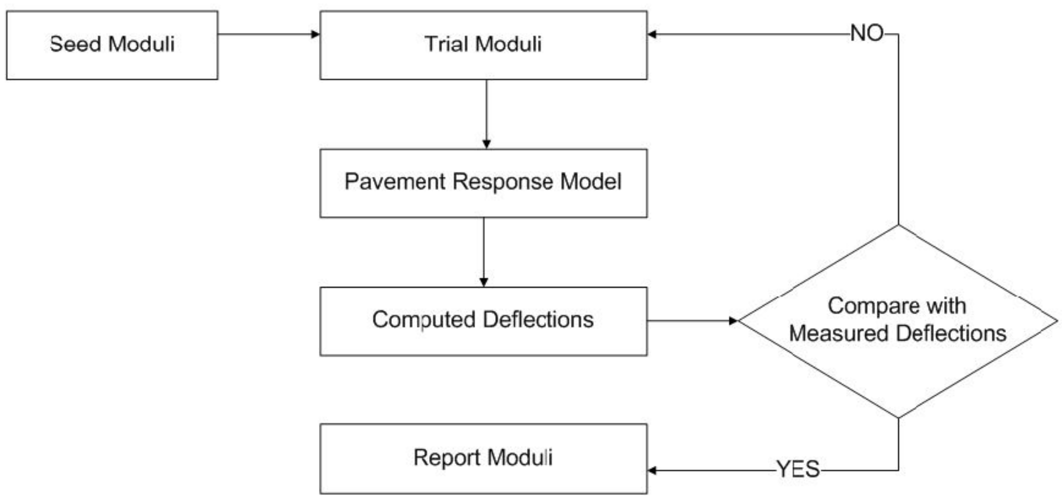


Figure 2.4: Traditional Iterative Backcalculation Procedure (Meier 1995)

However, most of these traditional software programs use linear elastic solutions to determine the pavement layer moduli which do not take account the nonlinear, stress dependent behavior of fine grained soils and aggregates. The recent Illinois Center for Transportation ICT R39-2 research study, 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 properly 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.

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		

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 with non-universal problem specific solution derived from artificially intelligent self-learning computation capability. The recent ICT R39-2 research study utilized ANNs and GAs in the developed software packages ANN-Pro and SOFTSYS 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 role of efficient pavement parameter analysis platform and GA is a robust search and optimization system; in combination they provide a very fast (due to ANN) and stochastic process (due to GA) to determine the parameters from FWD tests. A very powerful regression analysis system, ANN, has been in use as both 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 rapidly generate the results with specified low errors in the estimations. ANN-Pro is one software program, which uses ANN models to provide back analysis solutions of measured FWD surface deflection data (see Figure 2.5) (Pekcan et al. 2006, 2008, and 2009)

2.3.2 Genetic algorithm (GA)

Nature-inspired evolution-based GA is used to provide with 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.

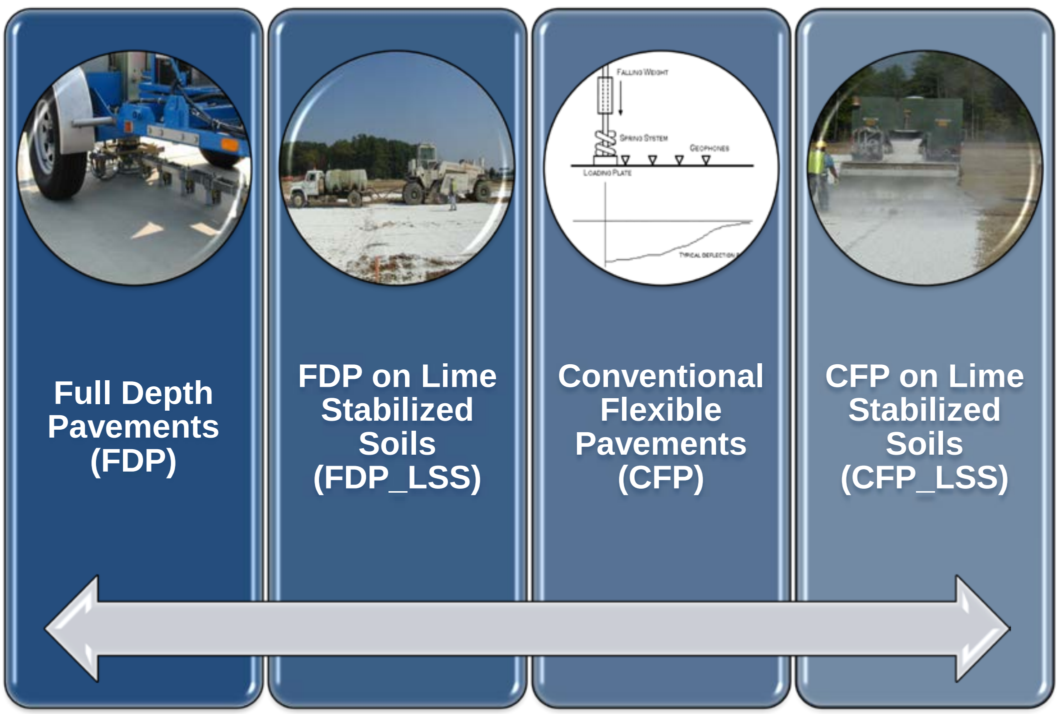


Figure 2.5: ANN-Pro Software (Pekcan et al. 2009)

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 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 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.

2.3.3 ILLI-PAVE finite element modeling and FWD simulation adopted by the ICT R39-2 study

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 (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 typical hardening behavior of base course granular materials and softening nature of fine-grained subgrade soils under increasing stress states. Among the several modifications implemented in the new ILLI-PAVE 2005 FE code are:

1) increased number of elements (degrees of freedom);
2) new/updated material models for the granular materials and subgrade soils;
3) enhanced iterative solution methods;
4) Fortran 90 coding and compilation, and
5) a new user-friendly Borland Delphi pre-/post-processing interface to assist in the analysis (Thompson et al. 2002) (see Figure 2.7).

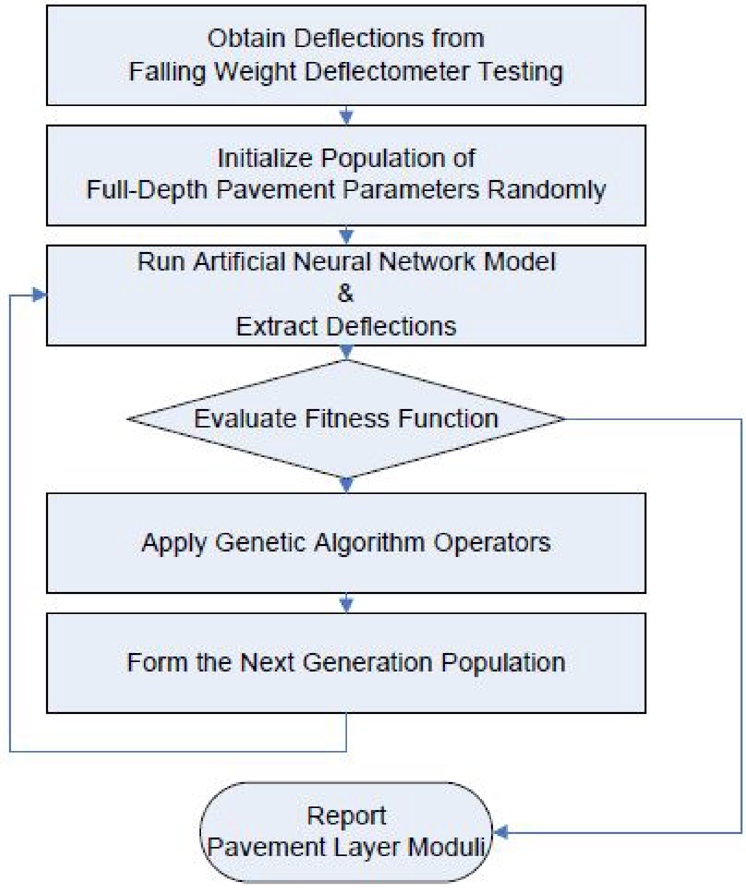


Figure 2.6: SOFTSYS Algorithm Flowchart (Pekcan et al. 2009)

Pavement FE modeling was performed in the ICT R39-2 study using an axisymmetric FE mesh for all pavement sections considered. Using 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 in., 8 in., 12 in., 18 in., 24 in., 36 in., 48 in., 60 in. 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.

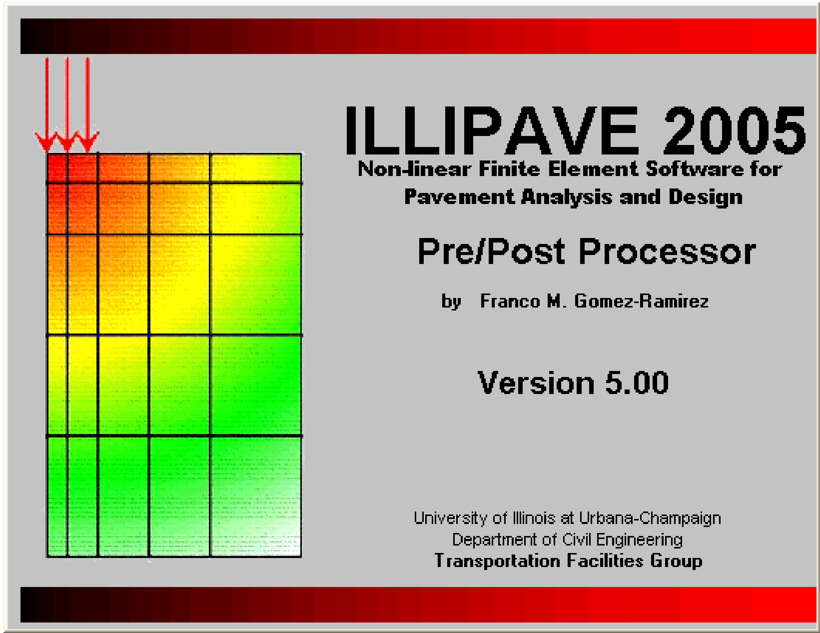
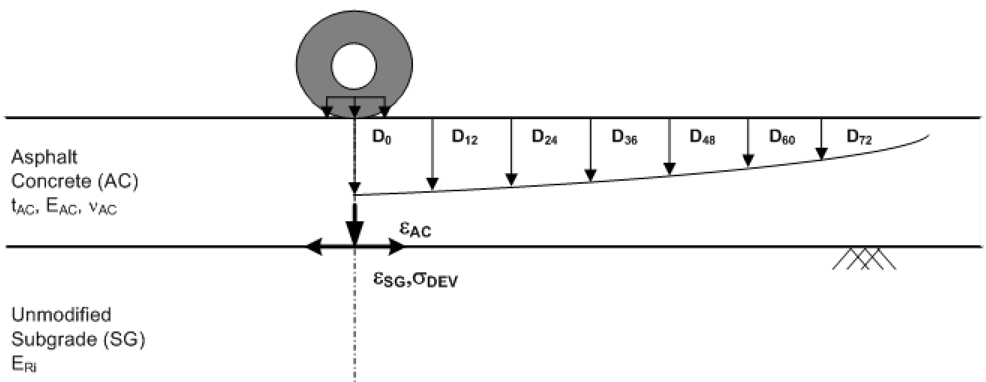


Figure 2.7: ILLIPAVE 2005 Finite Element Software for Pavement Analysis

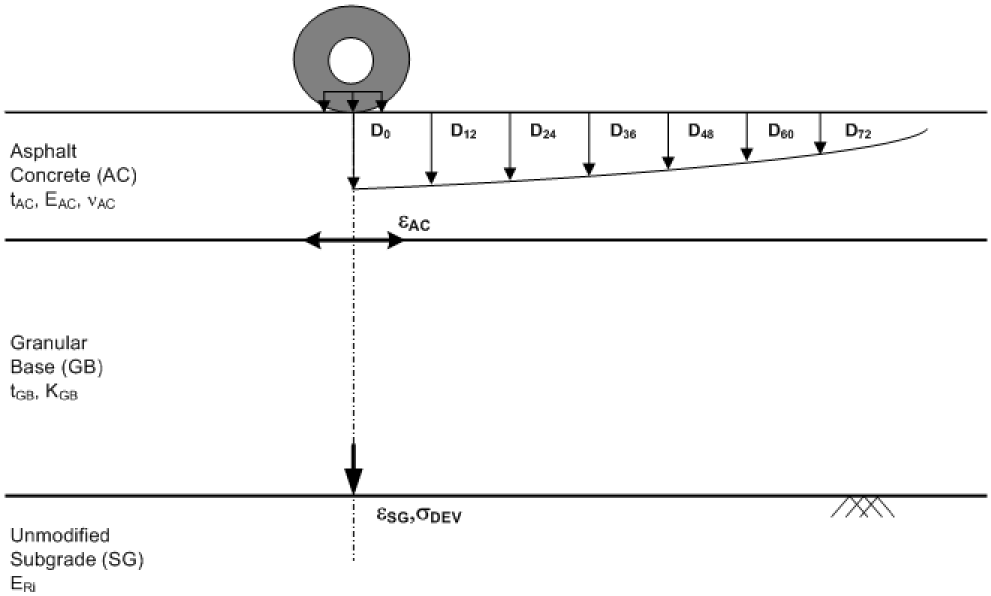
These deflections are in conformity with the uniform spacing commonly used in FWD testing by many state highway agencies including Illinois (Table 2.2). Typically, finer mesh spacing was used in the loaded area with the horizontal spacing adjusted according to the locations of the geophones used in FWD 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) show 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 due to excessive fatigue cracking and rutting in the wheel paths.

Table 2.2: Falling Weight Deflectometer Sensor Spacing

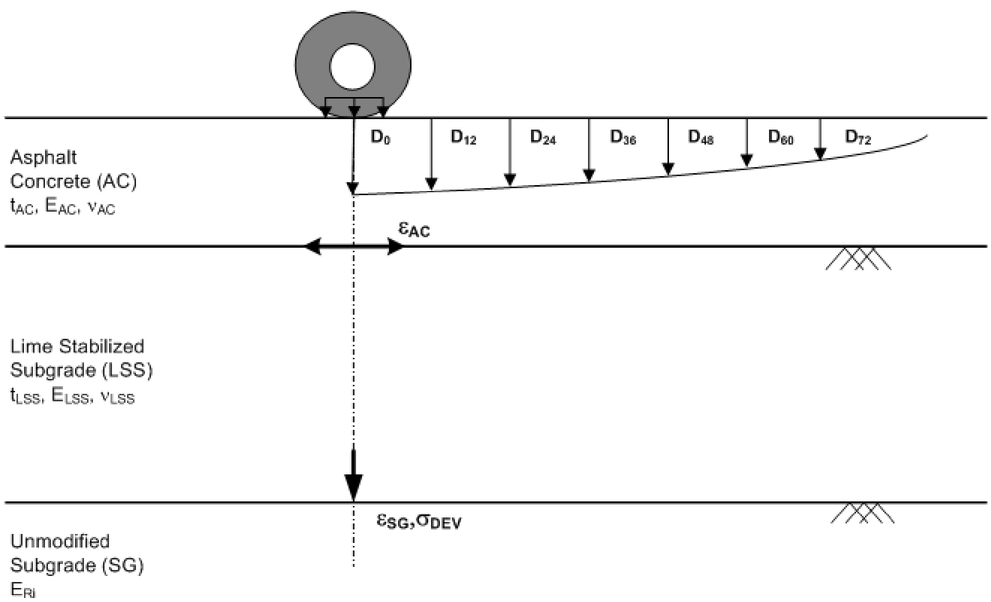
Sensor Spacing (in.)	0	8	12	18	24	36	48	60	72
Uniform (used in this and the ICT R39-2 study)	+		+		+	+	+	+	+
State Highway Research Program (SHRP)	+	+	+	+	+	+		+	



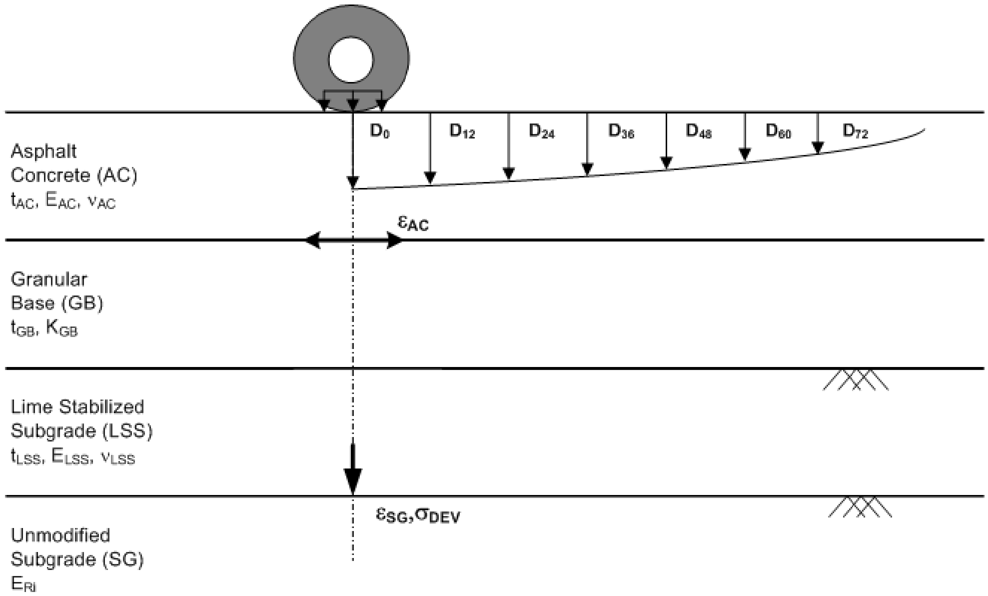
(a) full-depth asphalt pavements



(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 the ICT R39-2 study. 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 that there was neither numerical instability nor inconsistency in the results due to 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.

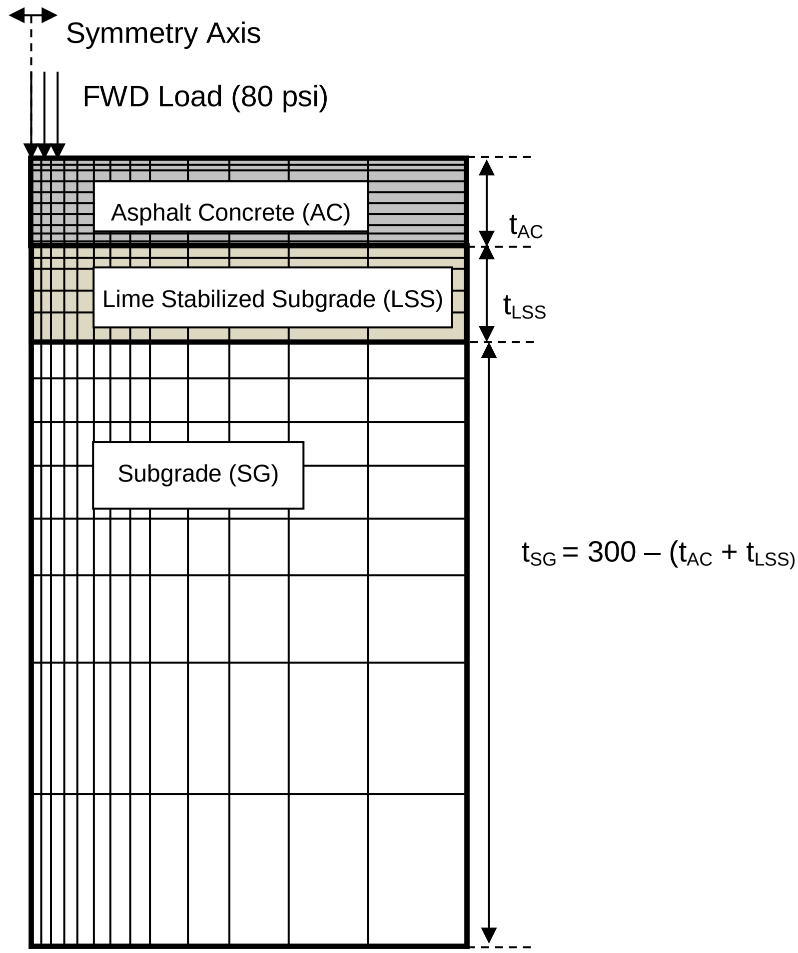


Figure 2.9: Example of FE Mesh used for Full-depth Pavements on Lime Stabilized Subgrade

Adequately characterizing pavement layer behavior plays a crucial role for 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 by 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 different flexible pavements analyzed in the ICT R39-2 study, 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. The value of ν_{AC} was taken constant as 0.35.

The modeling of fine-grained subgrade soils, mainly encountered in Illinois, has received more attention in the last three decades since 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 the deviator stress through nonlinear modulus response models. These models were developed based on the results of repeated load triaxial tests, which forms 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 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.

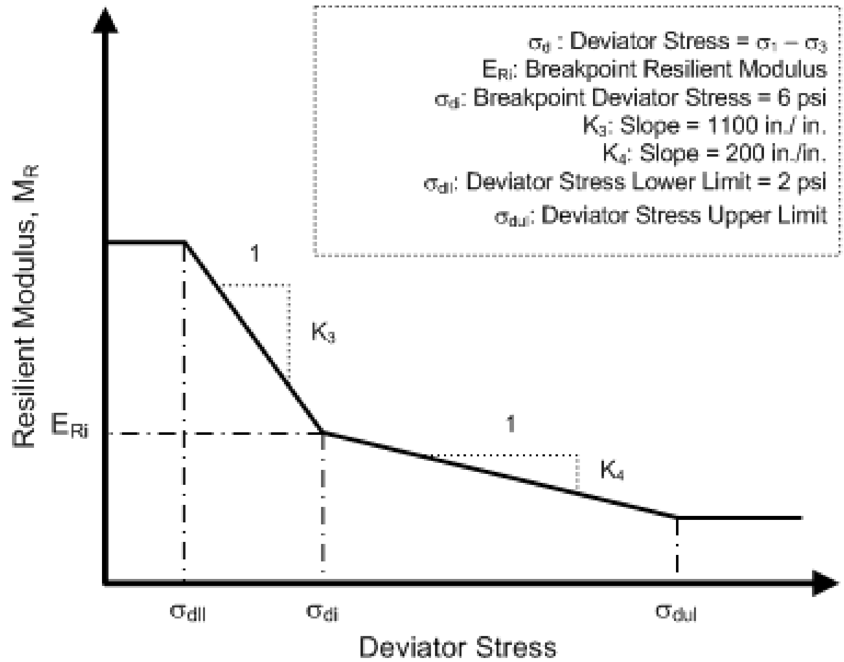


Figure 2.10: Bilinear Model to Characterize Stress Dependency of Fine-Grained Soils

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

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 considered by the use of two model parameters, “K” and “n.” The model parameter “n” is correlated to K-parameter according to Equation 2.3, where K is in psi. A major advantage of the given equation is that the unbound aggregate modulus characterization model then only requires 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) (see Figure 2.11). Poisson’s ratio was taken as 0.35 when K ≥ 5 ksi otherwise it was assumed 0.40.

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

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.

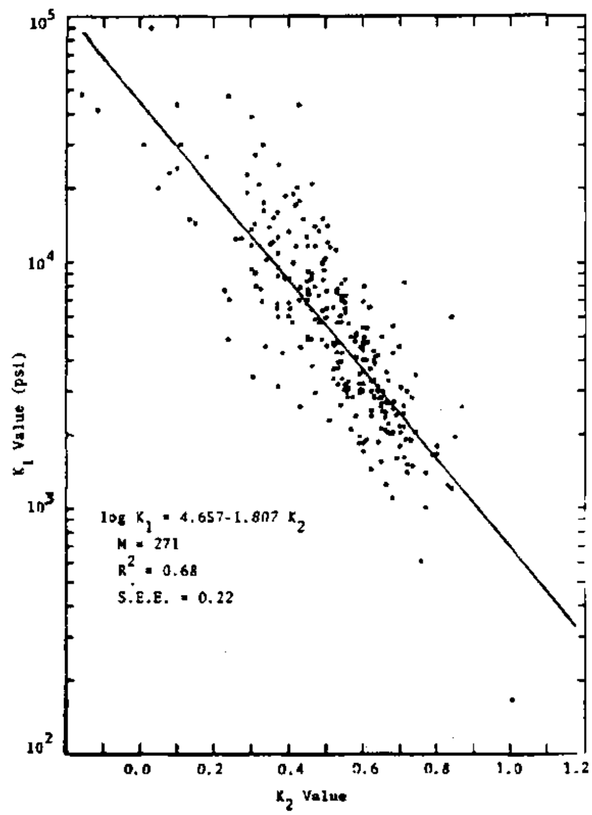


Figure 2.11: Relationship between K (shown as K_1) and n (shown as K_2) Values for Granular Materials Identified by Rada and Witczak (1981)

2.3.4 Field validation of SOFTSYS

This section presents two case studies demonstrating field validation of SOFTSYS (Pekcan, 2011).

US 50

Highway US 50 is located in both St. Clair County and Clinton County in Illinois. The pavement section tested is 9.5 in. of HMA built on unmodified subgrade. The pavement temperature on the day of FWD tests was recorded as 95 degrees F for both sections. Figure 2.12 (a) to (d) show the FDP SOFTSYS model predictions for US 50 using SOFTSYS model FDP-PM1-FWD4. The SOFTSYS derived AC modulus and RI modulus are of the same order as those derived from Hills equation and Thompson's algorithms, with very less average absolute error (AAE) (around 15%), except for the E_{RI} with Hill's equation (52.9%) (Figure 2.12a-d). The overall performance in Hill's algorithm compared to SOFTSYS was more deviated when compared to Thompson's algorithm derived modulus values. The best fitness value (0.996251) for SOFTSYS analysis was found after 22 generations. Looking into the dataset, three stations stand out with unreasonably high moduli values and that can be taken care of by considering the temperature variations of the stations during the FWD tests. The variations can be better predicted by refining initial EAC ranges in accord with the temperatures observed.

US 20

Highway US 20 is located in Stephenson County in Illinois. The design pavement section is 13 in. of HMA built on unmodified subgrade. The FWD tests were performed on both sections A and B, which are approximately 200 ft. in length. The pavement temperature was reported to be 99oF for both sections on the day of FWD tests. Figure 2.13 (a) to (d) show the SOFTSYS model predictions for US 20. SOFTSYS model FDP-PM1-FWD4 was used in the analyses and the C modulus obtained in the analysis had quite the similar accuracy for both the Hill's and Thompson's algorithms, the AAE values being 16.5% and 12.4% respectfully. 10.1% and 6.0% AAE values were obtained for E_{RI} similarly. So in overall comparison Thompson's algorithm provides the best fitness. The number of generations for the best fitness was 24 and fitness value was 0.982988. But the SOFTSYS analysis underestimates EAC values as it does not account for temperature as reason for higher deflection values from FWD tests. This can be taken care of by claiming the data point as an outlier or by refining the precursors of GA search.

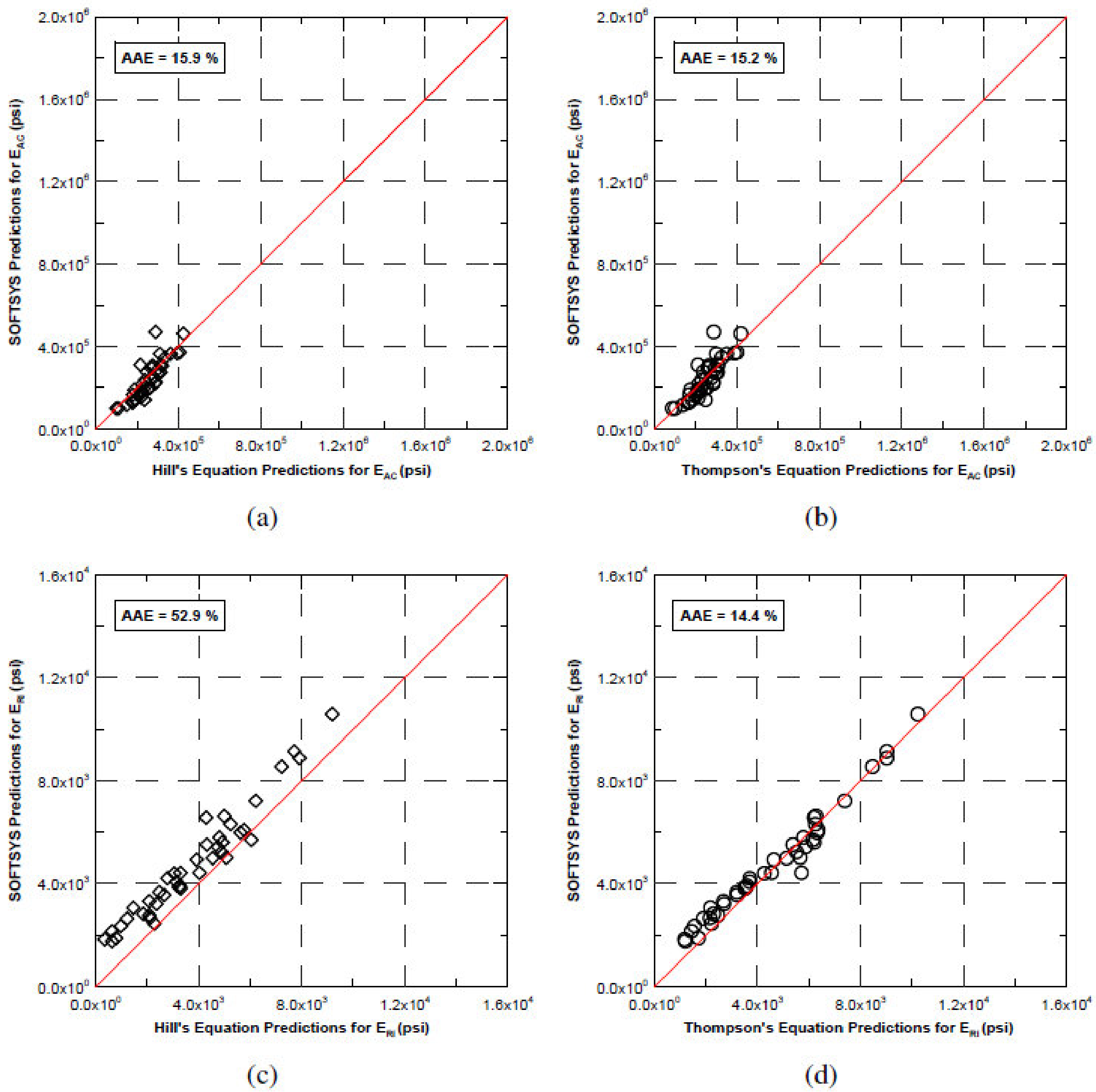


Figure 2.12: Performances of a SOFTSYS Model FDP-PM1-FWD4 for US 50 (Pekcan 2011)

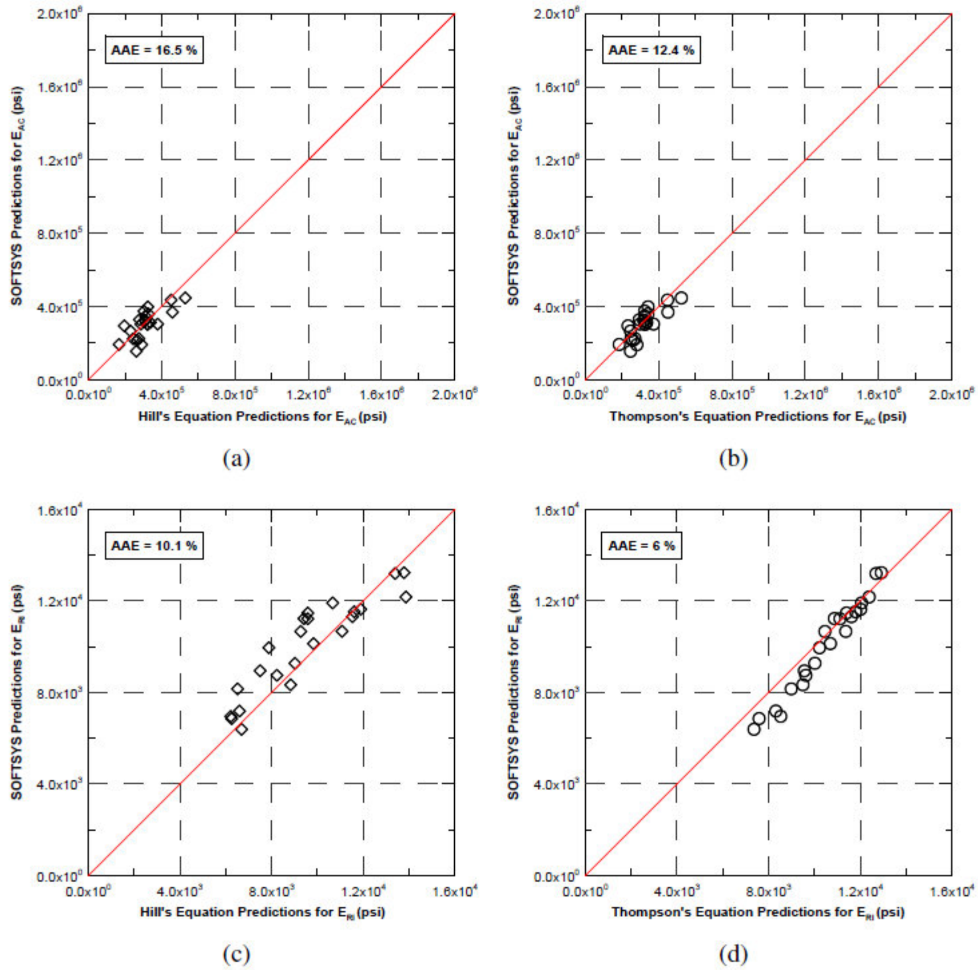


Figure 2.13: Performances of a SOFTSYS Model FDP-PM1-FWD4 for US 20 (Pekcan 2011)

Both case studies serve well as the validation of SOFTSYS platform's ability to handle deflection data in real time and its analyzing capability within the limits of capturing nonlinear behavior of the system.

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:

1. Methods based on the concept of structural deficiency;
2. Methods based on the concepts of maximum deflection and effective thickness; and
3. Methods based on rutting and/or fatigue damage algorithms.

Several state Departments of Transportation (DOT) routinely conduct FWD testing for structural evaluation of in-service pavement structures. Some examples to state DOTs include: Alabama, Arkansas, California, Idaho, Maryland, Minnesota, Mississippi, North Carolina, Ohio, South Carolina, Texas, 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 1993 AASHTO Pavement Design Guide incorporating the use of empirical layer coefficients for structural number (SN) calculations (AASHTO 1993). The following subsections will present overviews of the most commonly used methods in each category.

2.4.1 *Methods based on the concept of structural deficiency*

The 1993 AASHTO NDT Method

The 1993 AASHTO NDT-based method uses FWD-obtained deflection basin information; subsequently, the subgrade resilient modulus (M_R), and the required structural number (SN_{req}); and the projected traffic is 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 different 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 (9000 lbs.) 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; and

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; and

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.

Illinois Department of Transportation (IDOT) Procedure

According to Chapter 46 of the IDOT Bureau of Local Roads and Streets (BLRS) 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; and
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. 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 1993 AASHTO approach assuming 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

Asphalt Institute (AI) Method – I

The Asphalt Institute (AI) provides with 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; and

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 used design conversion factors, as these conversion factors are somewhat subjective.

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; and
- Critical period adjustment factor.

These parameters are used to determine the design overlay thickness using a design chart that has a unique relationship established among the overlay thickness, 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 usage 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 above mentioned agencies are primarily based on the empirical equations for asphalt cracking based fatigue and subgrade rutting developed by the Asphalt Institute (AI).

Although different state highway agencies have different methodologies for designing HMA overlay thicknesses, these design procedures essentially incorporate some form of modification to the 1993 AASHTO Pavement Design Guide 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 specifically developed for local roads and streets for 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:

1. Modified AASHTO Design for Overlays on Existing Flexible Pavement (used by IDOT BLRS);
2. 1993 AASHTO NDT Method (used by Ohio Department of Transportation, ODOT); and
3. 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 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 (see Table 2.4):

1. Existing pavement layer thicknesses;
2. Structural design traffic (ADT);

3. Immediate bearing value (IBV) of subgrade; and

4. 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. Pavement configuration presented in Figure 2.14 taken from IDOT BLRS Manual example was chosen as a base case. In addition, an ADT of 10,000, 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 (see 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 non-conventional materials. Also, depending on the required structural number, the manual sets minimum requirements of the thickness of the overlay from 2 to 4 in. that must be installed on an existing pavement section regardless of the current structural condition of the pavement.

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 2.15. 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-.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

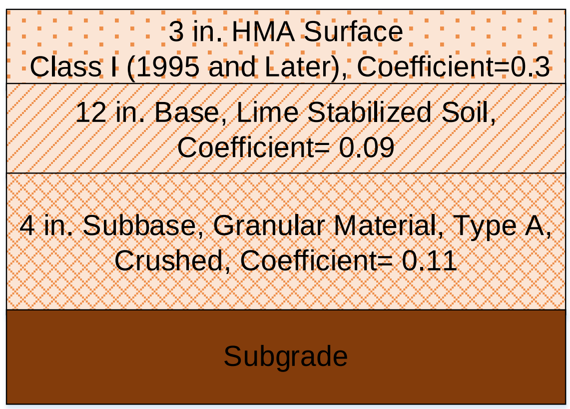


Figure 2.14: Pavement Layer Configuration Used as a Base Case

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
	300				0.12	0.09
	400				0.14	0.11
	800				0.17	0.13
Bituminous Stabilized Granular Material	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

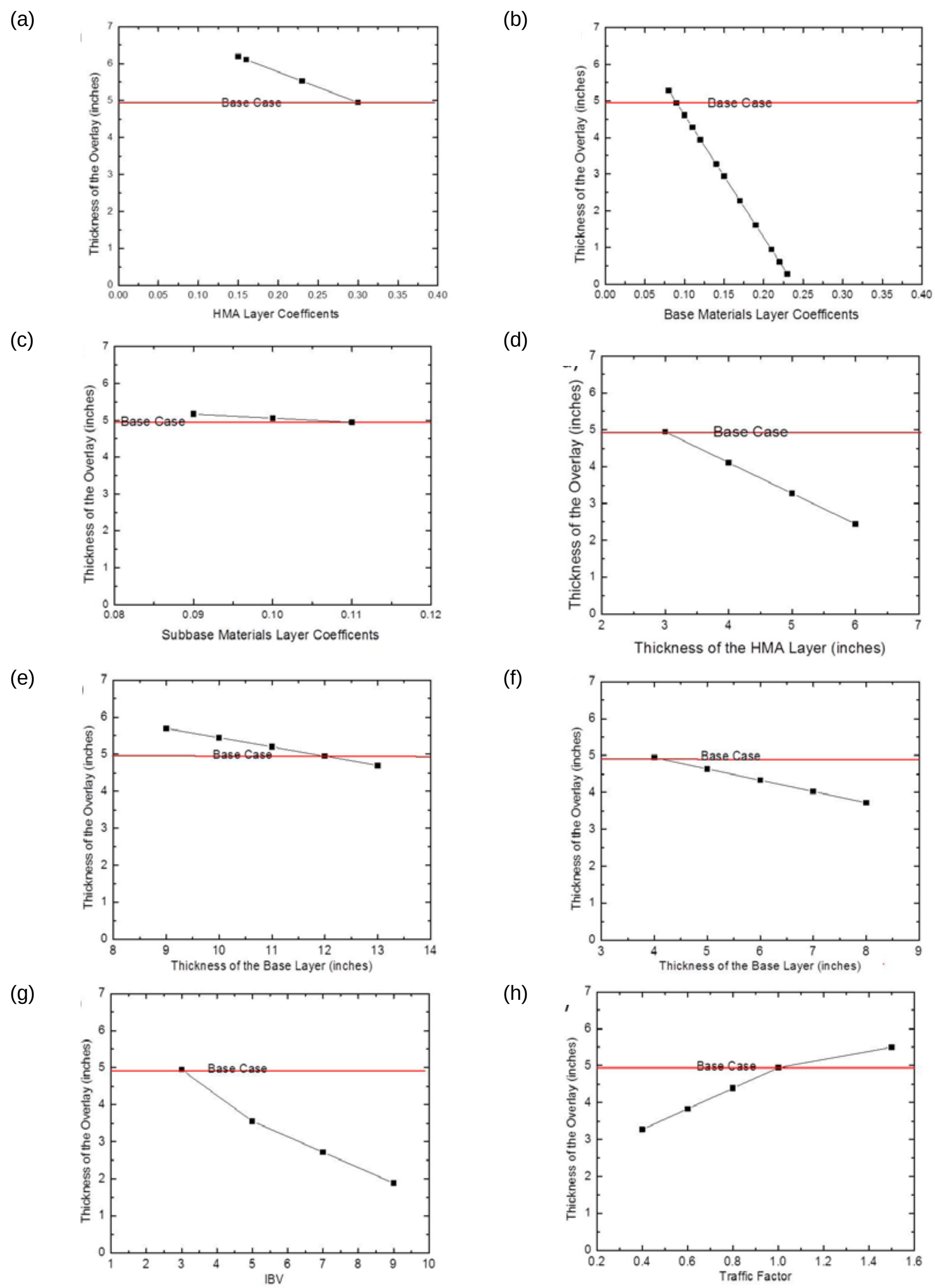


Figure 2.15: Overlay Thicknesses Calculated for Various Cases Studied as Listed in Table 2.4

Note that the pavement layer coefficients used above are all empirical and therefore limited in their ability to properly characterize the structural contributions of the many recycled/reclaimed, stabilized and large-sized construction materials as well as asphalt mixes more commonly utilized in today's sustainable pavement design and construction practices. Further, the concept of assigning layer coefficients is deficient due to its lack of consideration of the lifetime degradation of the layer materials and how the pavement functionality and performance degrade in 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; and
- 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 2.16 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 to be discussed later in Chapter 3.

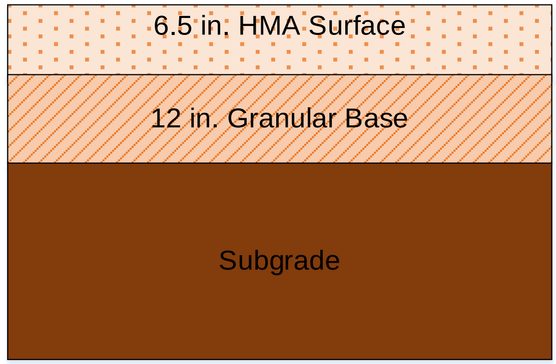


Figure 2.16: Pavement Layer Configuration Used as a Base Case

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

1. FWD center deflection (d_0);
2. Pavement temperature at the time of testing;
3. Traffic in terms of ESALs; and
4. 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 degrees F to 100 degrees 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

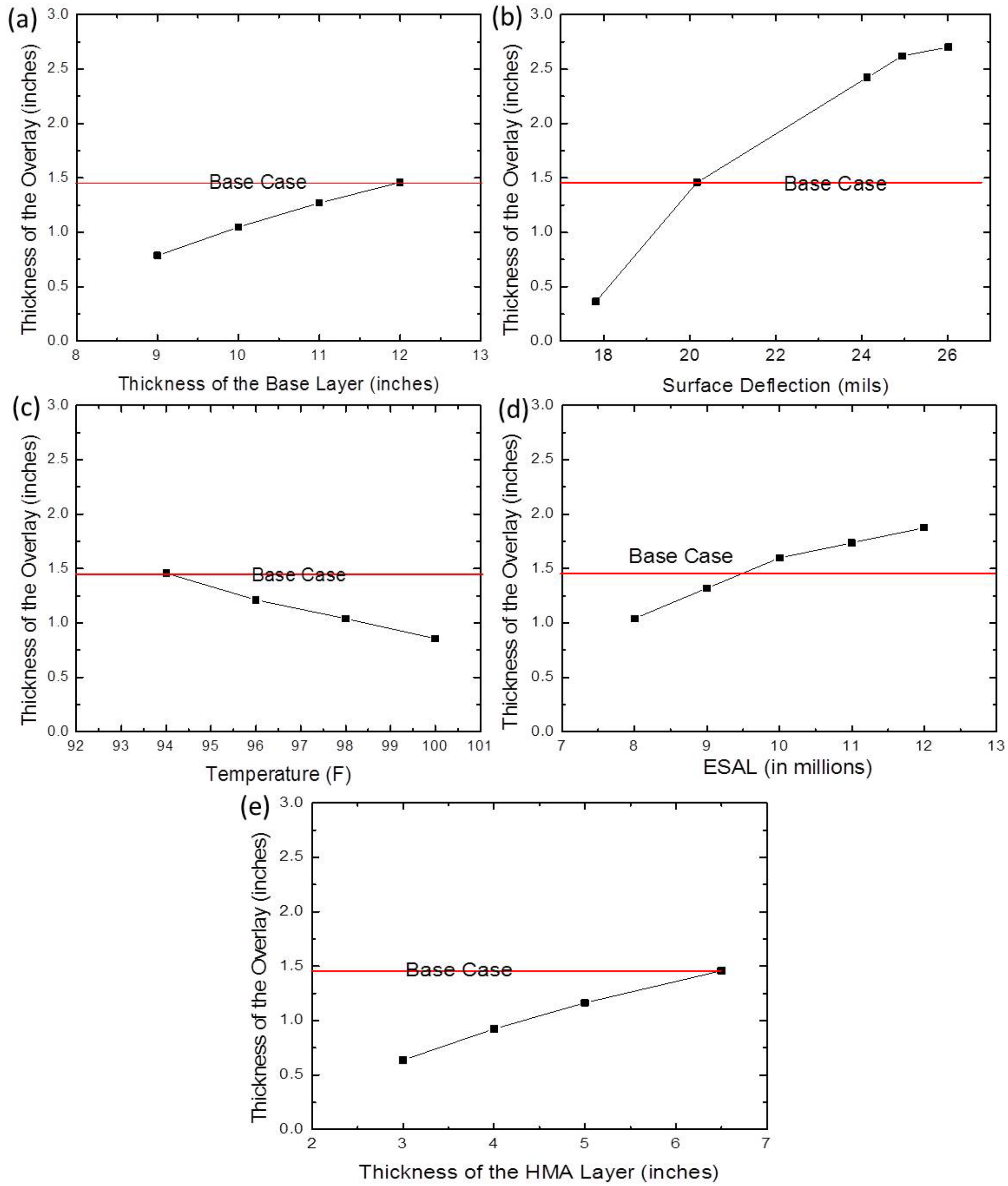


Figure 2.17: Overlay Thicknesses Calculated for Various Cases Listed in Table 2.6

Figure 2.17 (a-e) show 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; and
- HMA and base layer thicknesses – sensitive.

2.5.3 Sensitivity Analysis: Asphalt Institute Deflection Method

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, there is often a conversion factor of 1.61 that 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; and

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 using an overlay thickness chart (See Figure 2.18 and Figure 2.19).

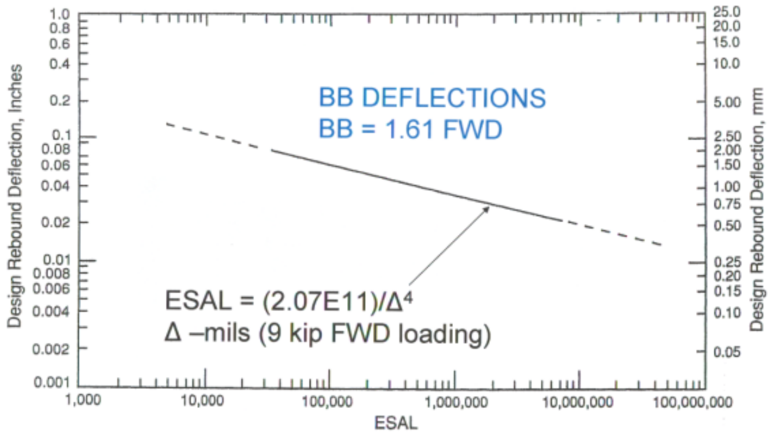


Figure 2.18: Design Rebound Deflection Chart (AI 1996)

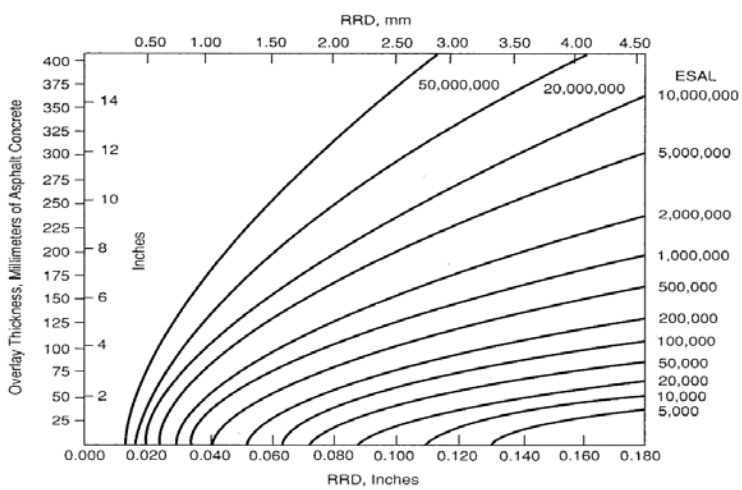


Figure 2.19: Asphalt Concrete Overlay Thickness Required to Reduce Pavement Deflections to Representative Rebound Deflection Value (AI 1996)

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

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

Based on the results of the sensitivity analyses, Figure 2.20 (a-b) show the HMA overlay thicknesses calculated for the studied various traffic ESAL counts 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 to the projected traffic inputs which varied within the 2 to 5 million ESAL range. This further confirms how HMA structural overlay thicknesses can adequately be determined from NDT based pavement deflection measurements.

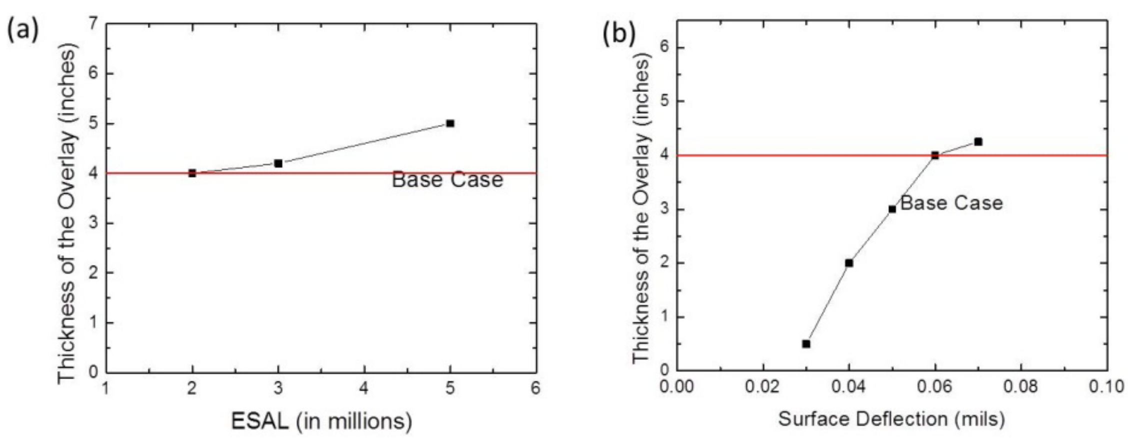


Figure 2.20: Calculated Overlay Thicknesses for the AI Deflection Method

2.6 Summary

Backcalculation in pavement analysis is a process where NDT tests such as FWD test results are used to infer layer properties including the layer thickness and layer moduli through a number of engineering approaches such as simplified search methods, gradient relaxation methods, and direct interpolation methods. Some of the key features of the available and commonly used software programs that employ these approaches were highlighted in this chapter. The development of new toolboxes in a recent ICT R39-2 study, named 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, thicknesses, as well as critical stress, strain and deformation responses of these in-service pavements, which can be accurately and rapidly determined from the field FWD deflection basins. Case studies to validate the application of SOFTSYS were also presented. 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 pavement deflection influenced the overlay thickness the most. Whereas, in the Modified Layer Coefficients method used by IDOT, the assigned layer coefficients influenced the overlay thickness the most. These modified AASHTO layer coefficients are outdated and inadequate to characterize structural contributions of in-service pavements.

CHAPTER 3 RESEARCH APPROACH AND CASE STUDIES

3.1 Introduction

This research study was undertaken to develop advanced methods for Hot Mix Asphalt (HMA) overlay thickness designs for flexible pavements roads based on proper structural evaluation of existing in-service pavements through NDT method such as FWD test. In order to select candidate in-service pavements, a questionnaire was prepared and distributed among the local transportation agencies. After a careful review of the responses collected from the agencies, five different pavement sections in two different counties in Illinois were selected for FWD-based structural condition evaluation and subsequent overlay thickness design. Pavement configurations, design traffic levels, and maintenance schedule of local agencies were carefully reviewed during the development of the FWD test matrix. Please note that no response was received from the local agencies in Indiana and Ohio. Primary emphasis was given to pavement sections that displayed high degrees of distresses, and had been selected by the local agencies for rehabilitation.

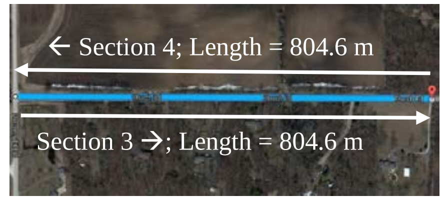
Structural conditions of the pavement sections were monitored over a period of one year through three different sets of FWD testing. The first set (Set 1) of FWD tests were conducted on severely deteriorated pavement sections in need for major rehabilitation work. The second set (Set 2) of FWD tests were 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. FWD tests along a given road segment were conducted at 200 ft intervals on the outer wheel paths. The trailer-mounted Dynatest FWD was used in this study with a standard configuration with geophones placed at 0, 12, 24, 36, 48, 60, and 72 inches, respectively from the center of the loading plate (plate radius = 152 mm or 6 in.). Pavement surface temperature was collected during the time of the testing at every 2000 ft interval along the testing lane. This chapter will include the details of the selected pavement sections tested, and the research methodology adopted to calculate the required overlay thickness of the pavement sections tested.

3.2 Details of the Selected Case Studies

Figure 3.1a shows the layout of the pavement sections tested in this project. Sections 1 through 4 were located in McHenry County, whereas Section 5 was located in DeKalb County. As shown in Figure 3.1, Sections 1 and 2 represent contiguous section on the same road segment (East Coral Road). Sections 3 and 4 on the other hand, represent lanes carrying traffic in opposite directions along the another road segment (Church Street). Such division of the tested road segments into different sections was necessary considering the varying pavement layer profiles, and substructure (base, subbase, and subgrade) support conditions. Note that Sections 1 and 2 were overlaid with 31.75 mm (1.25 in.) of HMA after the first set of FWD testings, whereas Sections 3 and 4 received a 38 mm (1.5 in.) thick overlay. No overlay was applied to Section 5. Figure 3.2 shows the layer configurations and traffic information for the tested pavement sections. Note that pavement configurations after the overlay have been referred to, as 1-b, 2-b, etc. Accordingly, the pavement configuration for Section 1 after the overlay has been referred to, as Section 1-b. Table 3.1 lists the pavement sections tested during each FWD testing effort, along with the corresponding pavement condition information. Accordingly, results from Sets 1 and 3 for Section 5 represent any change in pavement condition over one year of service.

(a)

Sections 1 and 2



Sections 3 and 4

Section 5

(b)

Section 1 and 2

Section 3 and 4

Section 5

Figure 3.1: Relative Locations and Photos of Selected Pavement Sections Tested with FWD in this Study

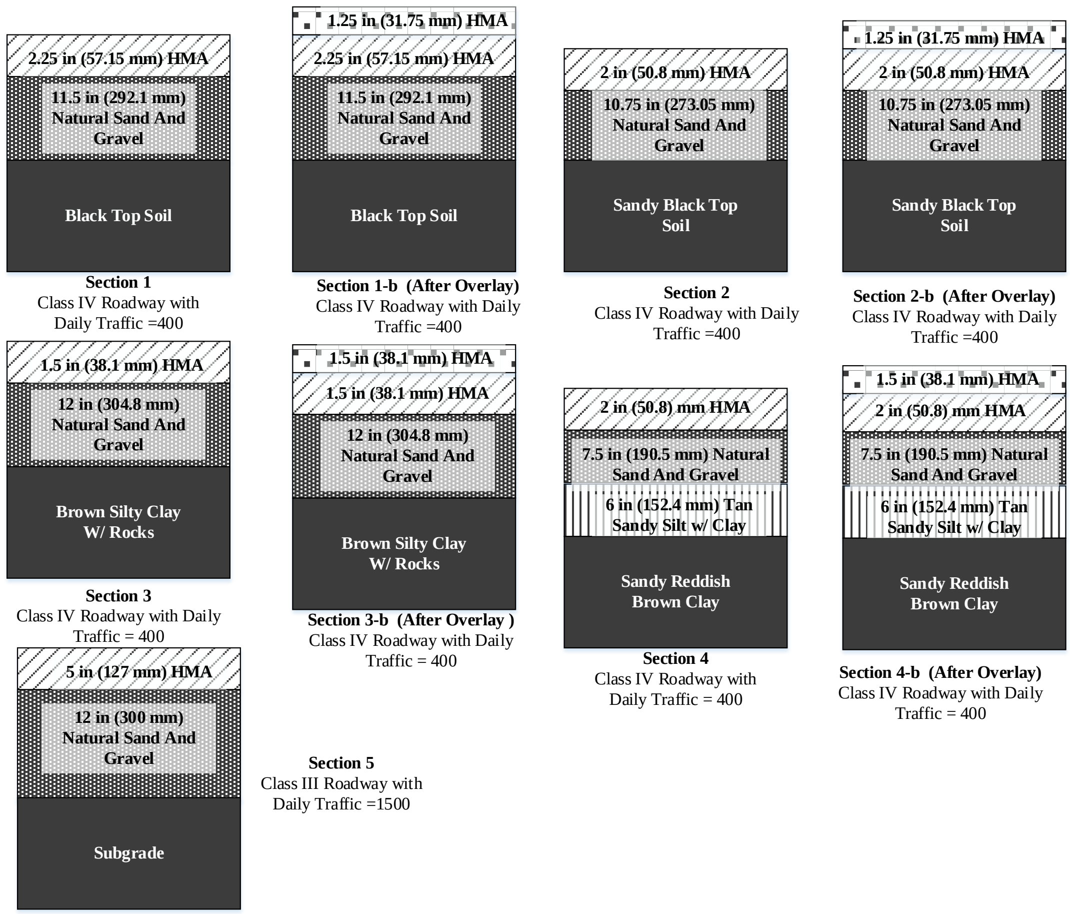


Figure 3.2: Layer Configurations and Traffic Information for Pavement Sections Selected for Current Study

Table 3.1: FWD Tests and Pavement Sections Studied

Testing Effort	Sections Tested	Pavement Condition Notes
Set 1	1, 2, 3, 4, 5	Severely Cracked; Overlay Needed
Set 2	1, 2, 3, 4	Immediately after the Overlay
Set 3	1, 2, 3, 4, 5	One Year after the Set 1 Testing Effort

3.3 FWD Test Results

Among the 5 pavement sections tested in the field and evaluated for structural conditions in this study, Sections 1 to 4 in McHenry County were the ones only tested for a total of 3 times. This subsection presents the FWD deflection basins of Sections 1 and 2. Figure 3.3 shows the deflection basins obtained for Section 1. During the set 1 FWD testing effort on the deteriorated old pavement, the deflection values varied significantly among all the test stations. For instance, at station 3000 ft. East direction and at 2000 ft. and 2500 ft. West direction, the center deflection values (D_0) were very close to that of the one obtained at 12-in. away from the center of loading plate (D_{12}). This could be due to the fact that these pavement sections were severely cracked at many locations along the road alignment which resulted in such anomalies.

As shown clearly in Figure 3.3b at almost every station tested at 200 ft interval 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, there are still some sections, as indicated in Figure 3.3, where the center deflections are slightly larger than those obtained before the overlay. This is because pavement surface temperatures were much higher during the set 2 testing efforts (varied between 71 and 88 degrees F) when compared to the 45 degrees F pavement surface temperature recorded during the set 1 FWD testing effort. 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 to note here is that center deflection values seemed to get lower after one year with the overlay during the set 3 FWD testing effort. This could be due to the fact that Section 1 had a thin HMA surface layer, so pavement base actually became stiffer due to the traffic that it was exposed to for about one year and eventually resulted in lower deflection values when compared to the just after the overlay placement one year earlier.

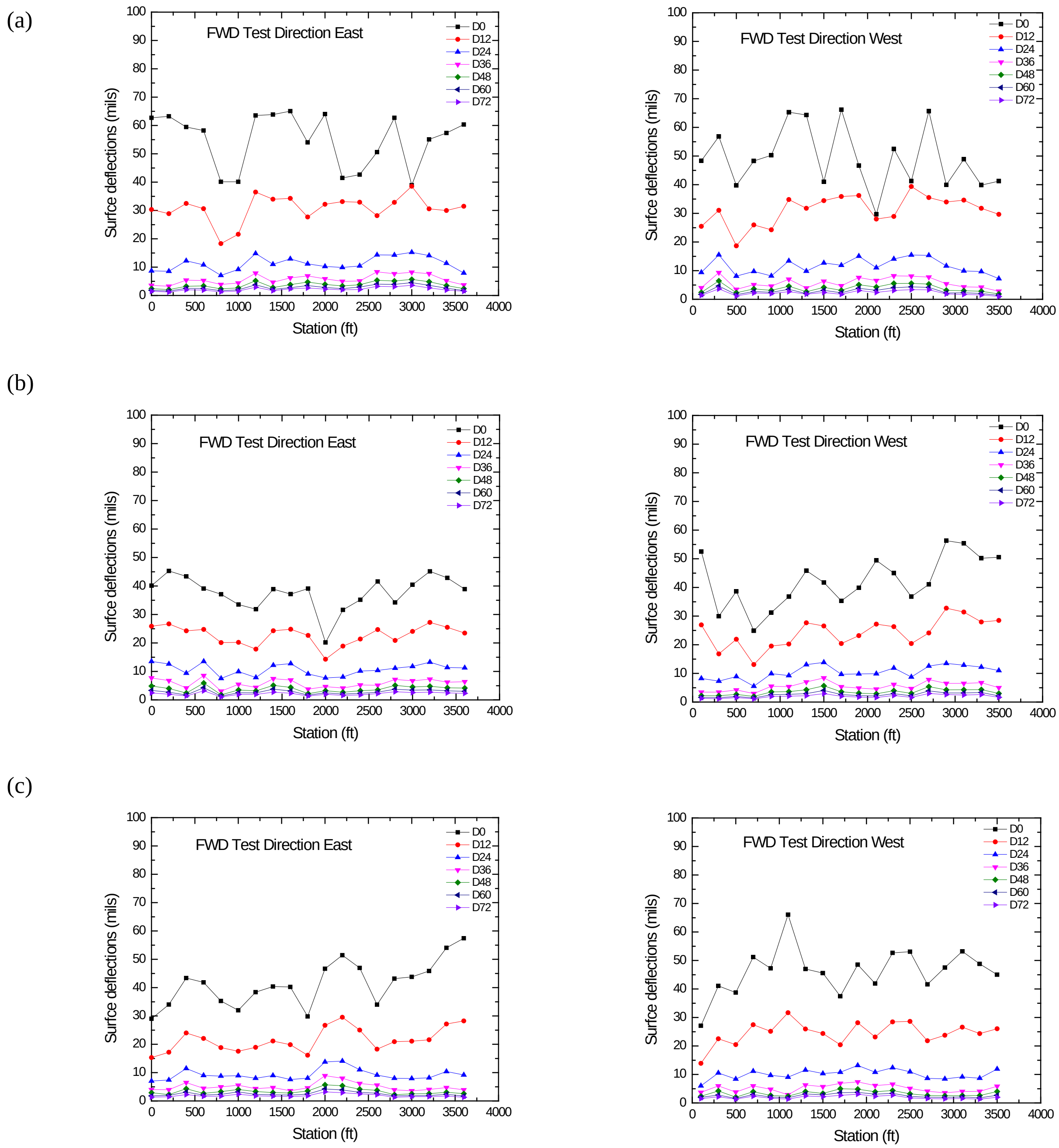


Figure 3.3: Deflection Basins Obtained from the Field during (a) Set 1, (b) Set 2, and (c) Set 3 FWD Testing Efforts for Pavement Section 1

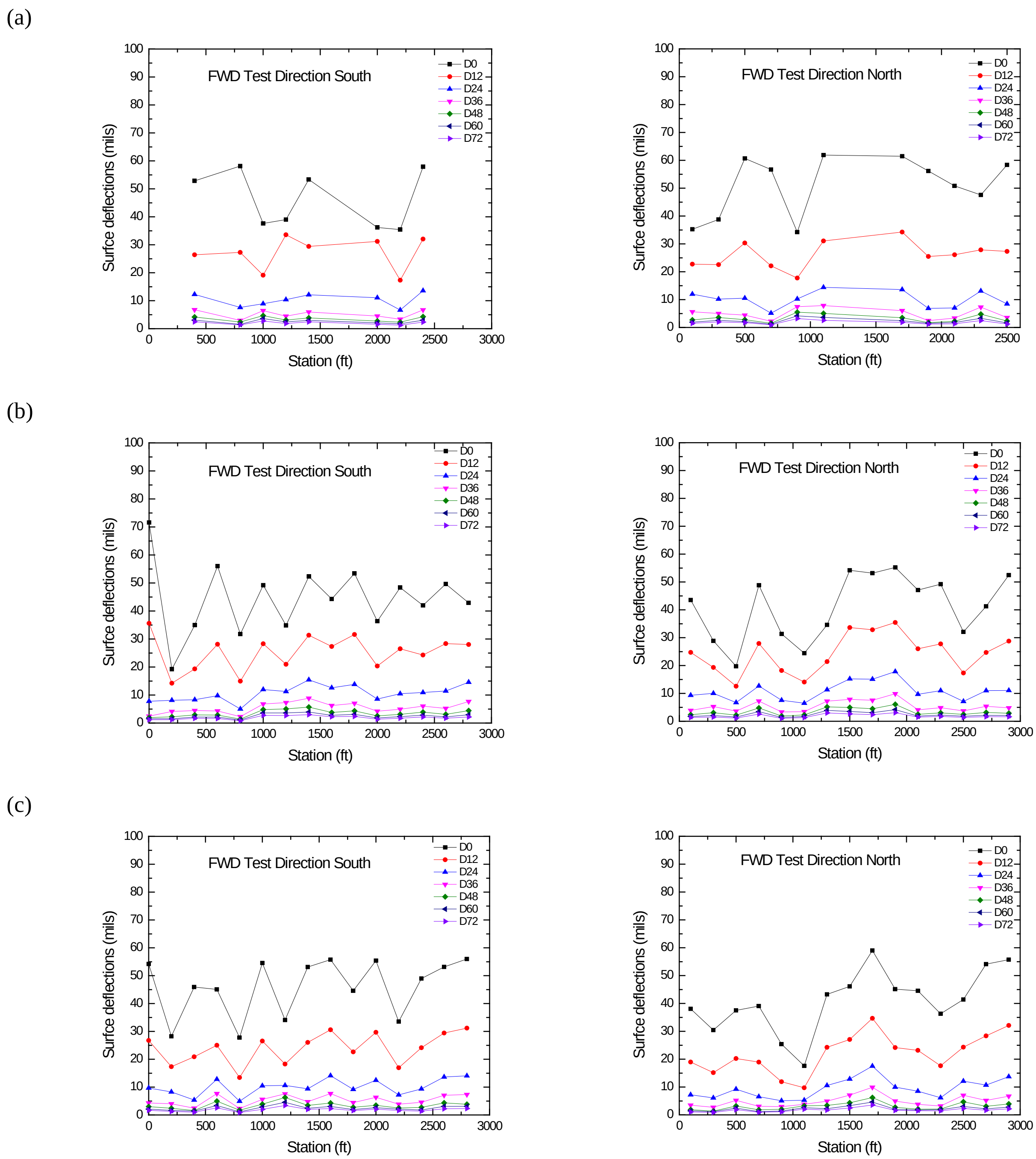


Figure 3.4: Deflection Basins Obtained from the Field during (a) Set 1, b) Set 2, and c) Set 3 FWD Testing Efforts for Pavement Section 2

Similar trends to those mentioned above can be found for Section 2 as highlighted in Figure 3.4. In addition, in Figure 3.4a, several stations from the beginning of the FWD tests are missing deflection values. This is due to the fact, that these pavement sections were severely cracked at many locations along the test section which eventually resulted in a non-decreasing deflection bowls. Accordingly, these stations with such questionable data were removed from the analyses and are not shown in the deflection basin curves.

3.4 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 back-calculation 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, a back-calculation 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 SOFTSYS program were developed during the previous ICT R39-2 research project efforts 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 pavements were obtained in coordination with the local transportation agencies. Significant variations were observed in the back-calculated layer modulus values even within a single pavement section. This was primarily because of varying support conditions, and also different degrees of cracking along the road segment. Moreover, severe cracking on the pavement surface resulted in deflection profiles at several stations that were unsuitable for back-calculation purposes. For example, inadequate contact of geophones with the cracked pavement surface sometimes led to non-decreasing deflection profiles as the distance from the load was increased. Such stations with questionable data had to be eliminated from the analyses. Accordingly, several stations with weak support conditions were excluded from the moduli back-calculations, resulting in higher back-calculated layer moduli compared to those 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 Figure 3.5 in the form of box plots for Sections 1 through 5 evaluated in this study. 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 3.6 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 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 was the outcome of excessive cracking of the pavement surface, subsequent non-decreasing deflection basins. The primary aspect to notice when comparing Figures 3.5 and 3.6 is rather the significant improvement in distribution of layer modulus values (reduction in the range in test results) after the application of the overlay.

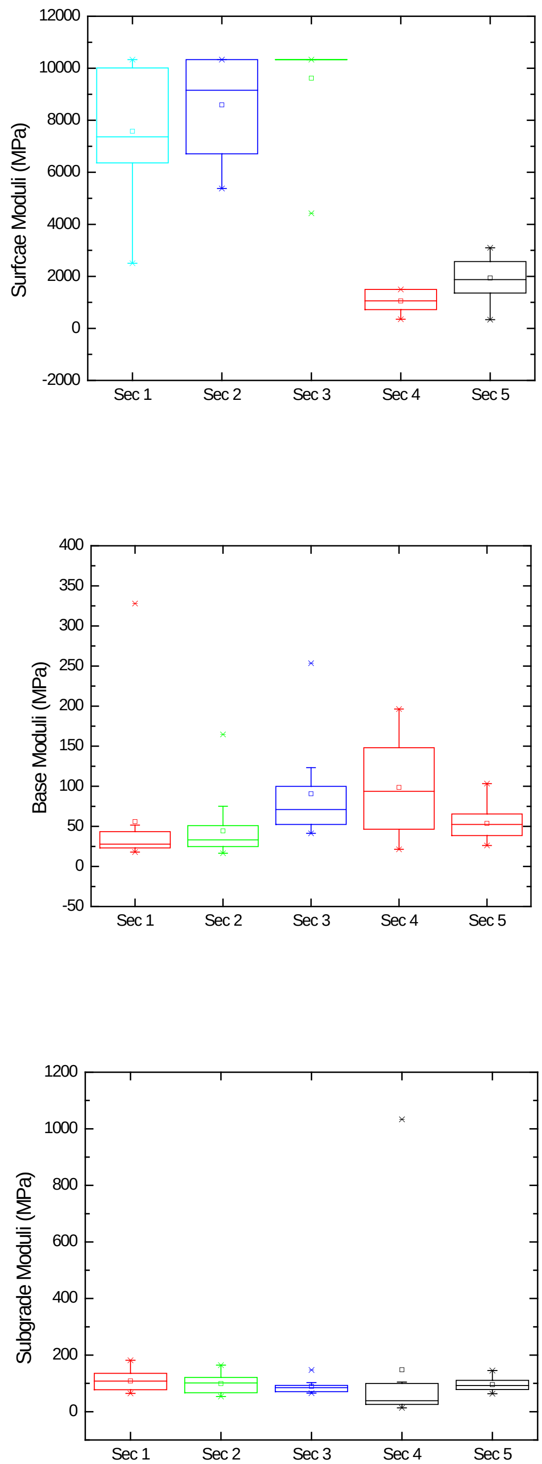


Figure 3.5: Back-Calculated Layer Modulus Values for Different Pavement Sections

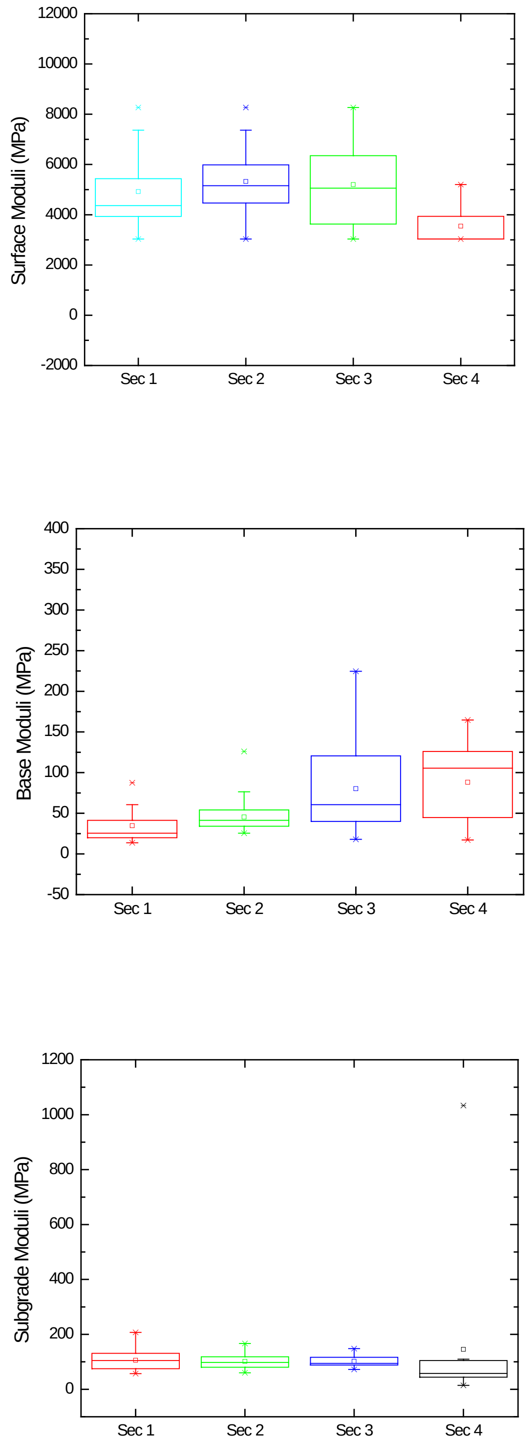


Figure 3.6: Back-Calculated Layer Modulus Values for Different Pavement Sections after Application of Overlay

3.5 Overlay Thickness Design using AASHTO and IDOT Procedures

The next step in the process involved determining the required overlay thicknesses for the tested pavement sections based on commonly available design methods. The AASHTO 1993 and IDOT methods were used for this purpose. Traffic factors were calculated using the equations provided in the Illinois Bureau of Local Roads and Streets (BLRS) Manual (2012), 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 Unsoaked California Bearing Ratio or CBR) value of 6%. Note that this corresponds to the minimum required bearing value in Illinois for the construction of flexible pavements without subgrade replacement. Calculation steps involved in these methods are trivial in nature, and are beyond the scope of the current manuscript. A summary of the parameters and coefficients used in the two design approaches is presented in Table 3.2.

In most of the cases, when the median of the SN_{eff} values are considered, the required structural number (SN_{req}) was found to be lower than the current structural number (SN_{eff}) of the pavement sections. Only Section 5 demonstrated a lower SN_{eff} value (SN_{eff} = 2.96; 50^{th} Percentile) compared to the corresponding SN_{req} (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 severe degree of fatigue cracking during the first set of FWD testing, indicating inadequate structural condition. Significant differences between the recommended overlay thicknesses determined from the AASHTO 1993 and the IDOT method 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).

Table 3.2: Overlay Thickness Design Using 1993 AASHTO NDT and IDOT Methods

		Section 1	**Section 2**	**Section 3**	**Section 4**	**Section 5**
1993 AASHTO NDT	Traffic Factor	0.014	0.014	0.014	0.014	0.41
	90^{th} Percentile SN_{eff}	2.56	2.61	2.66	2.64	3.22
	Median SN_{eff}	2.08	2.19	2.16	2.28	2.96
	10^{th} Percentile SN_{eff}	1.84	1.90	1.85	1.95	2.64
	SN_{req} (IBV=6)	1.90	1.90	1.90	1.90	3.1
	Overlay Requirement (in.), For 50^{th} Percentile SN_{eff}	0	0	0	0	0.35
IDOT Method	Existing HMA Layer Coefficient	0.3	0.3	0.3	0.3	0.3
	Base Layer Coefficient	0.09	0.09	0.09	0.09	0.09
	Subbase Layer Coefficient	N/A	N/A	N/A	0.07	N/A
	SN_{eff}	1.71	1.57	1.53	1.7	2.58
	SN_{req} (IBV=6)	1.90	1.90	1.90	1.90	3.1
	Overlay Requirement (in.)	0.48	0.84	0.92	0.5	1.3

The somewhat erroneous categorization of these pavements as structurally adequate by the AASTHO method can be attributed to the significantly low design traffic volumes for these pavement sections. Given identical material properties and layer configurations, with increasing traffic the required structural number will also increase, 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 recycled and/or non-traditional materials is questionable at best.

3.6 Proposed Mechanistic-Empirical (ME) Based Design Approach

Addressing the issues associated with using the empirical layer coefficients method by most of the local agencies, this research study aimed to develop a mechanistic-based overlay design system for flexible pavements. This proposed methodology is based on proper structural evaluation of the existing pavements that relies on the fatigue and deflection responses of the pavement as the design criteria. The following section will provide an overview of the proposed approach.

3.6.1 Layer Moduli Adjustment Using Layered Elastic, and Finite Element-Based Pavement Analysis Approaches

Extensively tested and validated Finite Element-based pavement analysis program ILLI-PAVE 2005 (Raad and Figueroa 1980), along with a linear elastic theory based software program BISAR (1989) were used to carry out modulus adjustments for the individual pavement layers. Layered elastic analyses using BISAR were first carried out to calculate the stress states (represented by the sum of principal stresses; $\theta = \sigma_1+\sigma_2+\sigma_3$) at the middle of the unbound aggregate base layer. Later the θ values were used in a stress dependent resilient modulus model (K-θ model) in ILLI-PAVE to calculate the critical pavement response parameters. ILLI-PAVE, unlike commonly used linear elastic programs, uses nonlinear stress dependent resilient modulus models to capture the typical hardening behavior of base course granular materials. 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 inch) radius in ILLI-PAVE.

In accordance with the location of FWD geophones, the surface deflections values were extracted from the ILLI-PAVE analysis results at 0, 12, 24, and 36 inches, respectively away from the center of the loading plate. The purpose of the using ILLI-PAVE 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 iteratively adjusted till the deflection values predicted from ILLI-PAVE were sufficiently close to the median value obtained from the field test results. Although the actual test configuration comprised 7 geophones to capture the pavement deflection basin, this iterative calculation step aimed to match the deflections at four locations for convenience.

Table 3.3: Iteratively Calculated Layer Moduli using ILLI-PAVE to Match FWD Deflection Basin

Section Number	HMA Modulus (ksi)	Base, 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

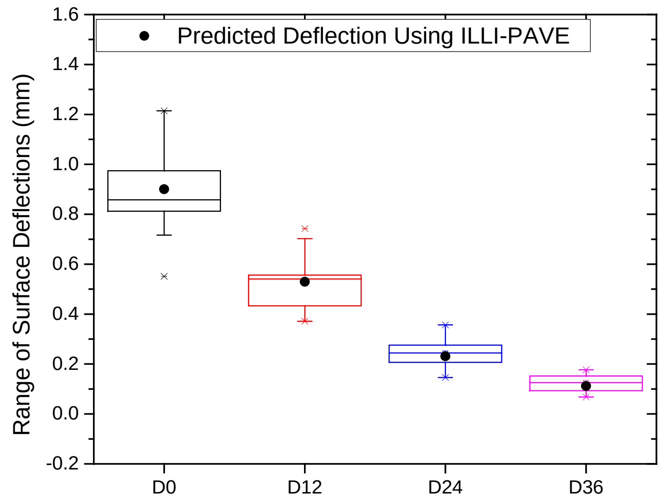


Figure 3.7: Deflection Matching with ILLI-PAVE and ANN-Pro

The surface deflections corresponding to the locations of these FWD sensors were abbreviated as D0, D12, D24, and D36, respectively. Then the back-calculated layer moduli were further adjusted using ILLI-PAVE and ANN-Pro, software programs. Table 3.3 lists the iteratively calculated layer modulus values using ILLI-PAVE. Figure 3.7 shows adequate match between the field-measured (median) and ILLI-PAVE predicted deflection values.

3.7 Overlay Thickness Determination

Upon completion of the layer modulus estimation, the current structural conditions of the pavement sections were evaluated using critical pavement response parameters (tensile strain at the bottom of the asphalt layer, ε_t; and vertical surface deflection under the load, δ_v) and the IDOT damage algorithms (see Equations 3.1 and 3.2). 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 value of N_f was then used to calculate the threshold critical pavement response parameter values for the different pavement sections.

$$N_f = \frac{8.78 \times 10^{-8}}{(\varepsilon_t)^{3.5}} \tag{3.1}$$

$$N_f = \frac{5.73 \times 10^{10}}{(\delta_v)^{4}} \tag{3.2}$$

Whether the pavement section requires an overlay or not, was determined by comparing the ε_t and δ_v values under the current pavement configuration with the threshold values calculated using Equations 3.1 and 3.2. The threshold values of ε_t and $\delta_{v,}$ along with the corresponding values under different FWD test efforts are listed inTable 3.4.

As can be seen from Table 3.4, the M-E Overlay Design Method adequately captures the structural inadequacy of the pavement sections under the original pavement configuration. Section 5 fails both under the fatigue as well as rutting algorithms. Sections 1 through 4, on the other hand, prove to be adequate for fatigue performance, but fail under the rutting criteria. Table 3.4-b presents the values of ε_t and δ_v immediately after application of the overlays (FWD Testing Set 2).

Table 3.4: Critical Pavement Responses for the Tested Pavement Sections Under FWD Loading: (a) Set 1; (b) Set 2; (c) Set 3

(a) Set 1

Section Number	Predicted ESALs Over Pavement Design Life	Threshold Critical Pavement Responses based on Damage Algorithms		Critical Pavement Responses 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	4.57E-4	30.24	YES

1 mil = 0.0254 mm
* Tensile Strain at the Bottom of the Asphalt Layer
** Vertical Surface Deflection Under Load
*** Overlay not required based on fatigue algorithm; but required based on rutting algorithm

(b) Set 2

Section Number	Critical Pavement Responses after Overlay		Capacity > Demand (Design Period= 20 Years)
	ε_t	δ_v (mil)	
1	4.33E-4	33.42	YES
2	4.44E-4	38.50	YES
3	4.24E-4	34.22	YES
4	4.56E-4	37.22	YES

(c) Set 3

Section Number	Critical Pavement Responses 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	30.87	NO

As expected, Sections 1 through 4 all pass the fatigue as well as rutting criteria. Note that no overlay was applied to Section 5. Please note that these threshold critical response parameters were calculated using future traffic demand for a design period of 20 years. Table 3.4(c) presents similar information one year after the original FWD testing. Although Sections 1 through 4 appear to be performing adequately under both fatigue and rutting criteria, Section 5 shows significantly higher ε_t and δ_v values compared to the thresholds and obviously requires overlay application. As mentioned, the M-E Overlay Design Method presents a significant improvement over the AASHTO and IDOT methods by combining mechanistic pavement response parameters along with pre-established pavement damage algorithms. A flow-chart of different steps involved in overlay thickness design using the M-E Overlay Design Method has been presented in Figure 3.8.

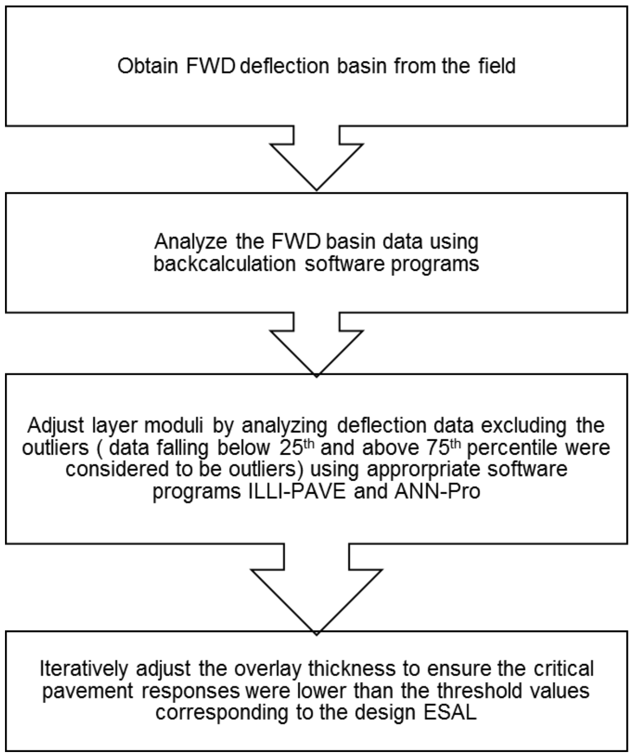


Figure 3.8: Flow Chart of the Developed M-E Overlay Design Procedure

Table 3.5: Summary of the Required Overlay Thicknesses for All the Methods

Section	Developed M-E Overlay Method (in.)	IDOT Modified Layer Coefficients Method (in.)*	1993 AASHTO NDT Method (in.)
1	1.25	2	No Overlay Required
2	1.25	2	No Overlay Required
3	1.5	2	No Overlay Required
4	1.5	2	No Overlay Required
5	3	3	0.35

*minimum requirement suggested by the manual based on required SN

As indicated in Table 3.5, there many pavements sections, such as Sections 1 through 4, which required lower thickness requirements than those calculated by IDOT method. However, 1993 AASHTO NDT methods characterized these sections as structurally sound to carry on the intended traffic volumes, and subsequently resulted in no thickness requirements.

3.8 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 listed in Table 3.6 and Table 3.7 respectively. Table 3.8 summarizes the cost of constructing the required HMA overlay over a one-mile long section. As indicated in Table 3.6, the cost of conducting an FWD analysis is only $550 per lane-mile per hour and this includes the mobilization cost, which decreases when greater lengths of road segments are FWD tested. Typically, it takes about an hour to conduct FWD testing at every 200 ft on a mile long road segment.

Table 3.6: FWD Testing - One Lane Mile (27 data points)

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

* Phone communication with Douglas Steele of Applied Research Associates.

According to Table 3.8, for about 4 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. Also, note that the thicknesses presented in Table 3.5 were taken as the basis for arriving at these cost numbers in Table 3.8.

Table 3.7: Material Type, Cost, and Quantity Calculation (Al-Qadi et al.2013)

Mix Type	9.5 mm dense graded HMA
G_{mm}	2.7
AC, %	5.3
N_{design}	90
Binder	PG 70-22
Quantity of Material Required, Ton/Lane-Mile/ inch	379.5
Total Cost, $/Ton	$90.63

Table 3.8: Cost of Constructing HMA Overlay ($/Lane-Mile)

	Cost, $/Lane-Mile		
	AASHTO 1993 NDT Method	IDOT Layer Coefficients Method	the M-E Overlay Design Method
Section 1	No Overlay Required	$68,788	***$43,543****
Section 2	No Overlay Required	$68,788	***$43,543****
Section 3	No Overlay Required	$68,788	***$52,141****
Section 4	No Overlay Required	$68,788	***$52,141****
Section 5	$12,416	$103,182	$103,732

*: A lower cost rehabilitation option when compared to the current IDOT method.

As can be observed from Table 3.8, the cost of implementing the M-E Overlay Design Method seems to be more expensive than that of the AASHTO 1993 NDT method. This is due to somewhat erroneous categorization of these pavements as structurally adequate by the AASHTO method which can be attributed to the significantly low design traffic volumes for these pavement sections. Given identical material properties and layer configurations, with increasing traffic the required structural number will also increase, 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 recycled and/or non-traditional materials is questionable at best.

CHAPTER 4 SUMMARY AND CONCLUSIONS

Local and state highway agencies dedicate a significant portion of their annual pavement management and rehabilitation budget towards the condition assessment of in-service pavements. However, an accurate evaluation of the functional as well as structural deficiencies of the existing pavement structure is necessary in order 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 pavement rehabilitation practices. 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 method 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 5 different pavement sections located in 2 counties in the State of Illinois were selected in this research study to conduct FWD tests on these deteriorated pavements and 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.

- Two commonly used overlay thickness design approaches, i.e., the 1993 AASHTO NDT method and IDOT Modified Layer Coefficients method were used with the specific data gathered from the tested pavement sections to design and recommend HMA overlay thicknesses.
- Due to 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 to threshold values for the pre-established fatigue and/or rutting damage algorithms.
- All but one of the tested pavement sections were erroneously categorized as structurally adequate by the 1993 AASHTO NDT method.
- Similarly, the modified layer coefficient-based IDOT method used in Illinois, being highly empirical in nature, predicted rather thicker overlays for the pavement sections when compared to the M-E Overlay Design method.
- Most of the sections had thinner overlay requirements following the developed M-E Overlay Design method when compared to those based on the minimum thickness requirement by the IDOT method.

Low volume road rehabilitation projects need to be encouraged to properly utilize FWD testing in the structural condition evaluations of existing, in-service pavements. Such projects, as highlighted in this report, will serve as demonstration projects to ensure proper implementation of the research findings related to the economical and safe pavement rehabilitation practices.

The developed M-E Overlay Design method is recommended for use in actual field overlay design projects to demonstrate its benefits through evaluating in-service pavements. Improved road safety, design reliability and performance will be achieved since mechanistic analysis and design concepts will be fully implemented in the development of HMA overlay structural thickness designs. Therefore, the use of the M-E Overlay Design method can prove to be a big step forward for local transportation agencies as far as overlay thickness designs of low volume flexible pavements are concerned.

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