Using Falling Weight Deflectometer Data with Mechanistic-Empirical Design and Analysis, Volume I: Final Report

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FOREWORD

This report documents a study conducted to investigate the use of the falling weight deflectometer (FWD) as part of mechanistic-empirical pavement design and rehabilitation procedures incorporated within the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) developed by the National Cooperative Highway Research Program and subsequently adopted by the American Association of State Highway and Transportation Officials. The first volume of this three-volume report documents general pavement deflection-testing procedures and commonly used deflection analysis approaches and a review of backcalculation programs for flexible, rigid, and composite pavement structures. The relevance of the different procedures and approaches to the MEPDG were explored through examination of six case studies evaluated using FWD testing results in the MEPDG, and the findings are presented in the second volume. Based on the case study findings and information from the literature, best practice guidelines for effective testing of existing pavement structures and interpretation of those results as part of a mechanistic-empirical pavement evaluation and rehabilitation process were developed and are presented in the third volume. This report is intended for use by pavement engineers as well as researchers involved in rehabilitation design and management of agencies' pavements.

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Director, Office of Infrastructure
Research and Development

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16. Abstract
The need to accurately characterize the structural condition of existing pavements has increased with the recent development, release, and ongoing implementation of the *Mechanistic-Empirical Pavement Design Guide* (MEPDG). A number of different material inputs are required in the procedure, and it is important to adequately characterize and define them. The analysis of deflection data collected by the falling weight deflectometer (FWD) provides a quick and reliable way to characterize the properties of the paving layers as well as to assess the load-carrying capacity of existing pavement structures. With the release of the new MEPDG, there is a pressing need to identify and evaluate the way that FWD testing is integrated into the new design procedure. Moreover, as highway agencies continue to implement the MEPDG, best practices guidance is needed on how to effectively test existing pavement structures and interpret the results as part of a mechanistic-empirical pavement evaluation and rehabilitation process.

This document is part of a three-volume report investigating the use of the FWD as part of mechanistic-empirical pavement design and rehabilitation procedures. In this volume, general pavement deflection-testing procedures and commonly used deflection analysis approaches and backcalculation programs are reviewed for flexible, rigid, and composite pavement structures. The relevance of the different procedures and approaches to the current MEPDG are explored through examination of six case studies evaluated using FWD testing results in the MEPDG. These six case studies used pavement sections from the Long-Term Pavement Performance database containing adequate design, construction, and testing data results as a means of assessing the way that FWD deflection data are used in the rehabilitation portion of the MEPDG. Based on the case study findings, and on information from the literature, recommendations for continued improvements and developments in the analysis and interpretation of pavement deflection data were developed.

This is volume I of a three-volume report. The other volumes in the series are FHWA-HRT-16-010, *Volume II: Case Study Reports*, and FHWA-HRT-16-011, *Volume III: Guidelines for Deflection Testing, Interpretation, and Analysis.*

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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LIST OF ACRONYMS AND ABBREVIATIONS

3-D	three-dimensional
AASHTO	American Association of State Highway and Transportation Officials
AI	Asphalt Institute
ANN	artificial neural network
CRCP	continuous reinforced concrete pavement
EBITD	effective built-in temperature difference
EICM	Enhanced Integrated Climatic Model
ESAL	equivalent single axle load
FAA	Federal Aviation Administration
FEA	finite element analysis
FFT	fast Fourier transform
FHWA	Federal Highway Administration
FWD	falling weight deflectometer
GPR	ground-penetrating radar
HMA	hot-mix asphalt
HMA/PCC	Hot-mix asphalt overlaid portland cement concrete
ISM	impulse stiffness modulus
JPCP	jointed portland cement concrete pavements
JRCP	jointed reinforced concrete pavement
LTE	load transfer efficiency
LTPP	long-term pavement performance
MEPDG	*Mechanistic-Empirical Pavement Design Guide*
MET	method of equivalent thickness
NCAT	National Center for Asphalt Technology
NCHRP	National Cooperative Highway Research Program
NDT	nondestructive testing
PCC	portland cement concrete
QA/QC	quality assurance/quality control
RDD	rolling dynamic deflectometer
RWD	rolling wheel deflectometer
SHRP	Strategic Highway Research Program
SID	system identification
SPS	Specific Pavement Studies
SVD	singular value decomposition
TELTD	total effective linear temperature difference
USACE-WES	U.S. Army Corps of Engineers—Waterways Experiment Station

CHAPTER 1. INTRODUCTION

BACKGROUND

The collection and analysis of pavement deflection data is a relatively quick and easy way to assess the structural condition of an existing pavement. Early work done by Hveem clearly indicated a relationship between the magnitude of pavement deflections and pavement performance, with "weaker" pavements exhibiting larger deflections and "stronger" pavements exhibiting smaller deflections.[1] In later work, Hveem et al. presented safe limits for maximum deflections to preclude cracking for different pavement types subjected to several million repetitions of a 6,800-kg (15,000-lb) axle load.[2]

With the development of more rapid and sophisticated deflection-measuring equipment and improvements in computer technology, deflection-testing results can now be analyzed to provide a more complete portrayal of pavement behavior. Not only can deflection testing be used to assess the structural condition of existing pavements, but it can also be used to assist in the design of structural overlays, to appraise seasonal variations in pavement response, to assess structural variability along a project, and to characterize paving layer properties and subgrade support conditions. On rigid pavements, deflection testing can also be used to determine load transfer across joints and cracks and to detect underlying voids. Today, pavement deflection testing plays a significant role in the pavement monitoring and evaluation activities of many transportation agencies.

Over the years, a number of different testing devices have been used to obtain pavement deflection measurements, and currently impulse load testing using the falling weight deflectometer (FWD) has become the accepted worldwide standard. The FWD imparts a dynamic load to a pavement structure that is similar in magnitude and duration to that of a moving wheel load, thus producing a representative pavement deflection response. In addition, the FWD provides a relatively rapid (and nondestructive) test procedure that allows a greater sampling frequency than typically possible when using more traditional material sampling methods. The greater sampling frequency allows the engineer to better define both the average and standard deviation of the design inputs while also providing the opportunity to perform the testing under in situ temperature, moisture, and confinement conditions.[3] Although most commonly conducted for project-level testing (and primarily to assist in the development of overlay designs), some State highway agencies have incorporated FWD testing as part of their network-level pavement evaluations (e.g., Texas and Virginia).[4,5]

The need to accurately characterize the structural condition of the pavement has increased with the development of the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) prepared under National Cooperative Highway Research Program (NCHRP) Project 1-37A and now available in an interim edition from the American Association of State Highway and Transportation Officials (AASHTO).[6,7] In the MEPDG, the performance of the designed pavement is projected by simulating the expected accumulated damage on a monthly or semimonthly basis over the selected design period. The amount of incremental damage occurring during each computation interval (either monthly or semimonthly) varies as the effects of prevailing environmental conditions, changes in material properties, and effects of traffic loading are directly considered.

Ultimately, the incremental damage accumulated during each computation interval is converted into physical pavement distresses and projected roughness levels using calibrated models that relate the damage to observable.[6,7]

An integral part of this process is the accurate characterization of material properties of each layer in the pavement structure. Deflection data collected by the FWD can be quickly and easily used to characterize the properties of the paving layers through a methodology called "backcalculation." This is merely a process whereby the fundamental engineering properties of the paving layers (elastic modulus, E) and underlying soil (resilient modulus M_R or modulus of subgrade reaction k) are estimated based on the measured surface deflections, the magnitude of the load, and information on the pavement layer thicknesses. In essence, the set of characteristics for the paving layers and subgrade material is determined such that it produces a pavement response that best matches the measured deflections under the known loading.

Backcalculation has come a long way since the pioneering work performed by Scrivner, Michalak, and Moore, which produced a graphical solution for a simple two-layer system.[8] Since that time, numerous methods have been developed to determine the material properties in each layer of a pavement structure. Flexible pavement systems are typically modeled using a static, linear (or quasi-nonlinear) elastic layered analysis. The material properties of each layer can be determined using either forward calculation or backcalculation. Sometimes both methods are employed. The forward calculation is used first to determine the seed moduli for the backcalculation analysis or to check the "reasonableness" of the backcalculated moduli. However, it is not uncommon to get very different results when using the programs available for analyzing FWD data collected for a specific pavement even though a similar analytical approach is applied by each of these programs. Discrepancies between actual and backcalculated models arise as the result of a departure of the true pavement behavior from the idealized theoretical models. For example, a static analysis is typically performed even though the FWD testing typifies a dynamic loading condition.

Rigid and composite (hot-mix asphalt (HMA) over portland cement concrete (PCC)) pavements are typically modeled as semirigid plates on top of either a dense liquid or elastic solid foundation. The two approaches used for evaluating the support conditions are the Best-Fit and AREA methods. The Best-Fit method is used to define the support layer conditions in the Long-Term Pavement Performance (LTPP) database, but the cumbersome nature of the calculations required for the Best-Fit method has led many researchers and practitioners to use the AREA method. Fortunately, the two methods appear to produce very similar results for specific sensor configurations. Regardless of the method used, the accuracy of the results is limited by the inability to accurately account for variables such as the effective slab size (which is influenced by the level of load transfer efficiency (LTE) present at the longitudinal and transverse joints) and inherent temperature gradients.

PROBLEM STATEMENT

As noted in the previous discussion, FWD testing is a routine pavement evaluation method, and testing results play an integral role in the critical determination of in-place structural characteristics. With the release of the new MEPDG, there was a pressing need for a comprehensive review of the current state of the art/state of the practice of FWD testing, backcalculation, and interpretation. Moreover, there was a need to identify how FWD testing was integrated into the new MEPDG and to provide best practices guidance on how to effectively test existing pavement structures and interpret those results as part of a mechanistic-empirical pavement evaluation and rehabilitation process.

PROJECT OBJECTIVES

This project was initiated to address many of the issues noted above. Specifically, the overall objectives for this project can be summarized as follows:

1. Review the current state of the practice/state of the art of FWD testing and backcalculation, including its use with the new MEPDG.(7)

2. Demonstrate the use of the FWD testing and analysis as it pertains to the MEPDG.

3. Provide recommendations for improvements in FWD testing and interpretation, particularly ones relevant to the rehabilitation procedures in the MEPDG.

4. Develop best practices guidelines for testing with the FWD and for analyzing/ interpreting testing results, particularly as they pertain to the MEPDG or other mechanistic-empirical design processes.

This project addressed FWD data analysis and interpretation of flexible, rigid, and composite pavement systems.

REPORT ORGANIZATION

The final report is presented in three s: volume I (Final Report), volume II (Case Studies), and volume III (Guidelines for Deflection Testing, Interpretation, and Analysis). This report (volume I), which documents the entire research effort that was conducted under the project, contains five chapters in addition to this introduction:

- Chapter 2 is an overview of deflection testing, including the reasons for performing deflection testing and the types of deflection-measuring equipment.

- Chapter 3 describes basic backcalculation concepts for various pavement systems and the use and application of the backcalculation results.

- Chapters 4, 5, and 6 describe specific FWD data analysis and interpretation procedures for flexible, rigid, and composite pavement structures, respectively.

- Chapter 7 summarizes the work activities and the findings from the pavement evaluation/backcalculation case studies that were conducted under the project.

- Chapter 8 is an overview of recommended FWD data analysis and interpretation procedures, and chapter 9 provides an overall report summary. This volume concludes with a bibliography of reports, articles, and other technical documents on FWD testing and analysis.

CHAPTER 2. OVERVIEW OF PAVEMENT DEFLECTION TESTING

INTRODUCTION

As described in chapter 1, pavement deflection testing is a quick and easy way to assess the structural condition of an in-service pavement in a nondestructive manner. Over the years, a variety of deflection-testing equipment has been used for this purpose, from simple beam-like devices affixed with mechanical dial gauges to more sophisticated equipment using laser-based technology. Nevertheless, all pavement deflection-testing equipment basically operates in the same manner. A known load is applied to the pavement and the resulting maximum surface deflection (or an array of surface deflections located at fixed distances from the load, known as a deflection basin) are measured. Figure 1 shows a schematic of a deflection basin.

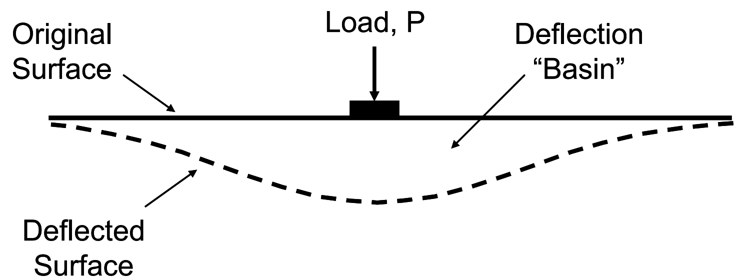


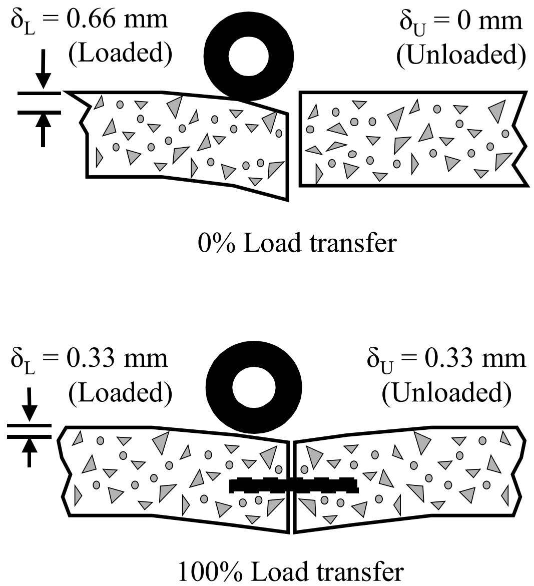
Figure 1. Diagram. Typical pavement deflection basin.

This chapter reviews the reasons for conducting deflection testing, provides a summary of commonly used deflection-measuring devices, describes common deflection-testing patterns, and discusses important factors influencing deflection measurements.

PURPOSE OF DEFLECTION TESTING

The primary purpose of deflection testing is to determine the structural adequacy of an existing pavement and to assess its capability of handling future traffic loadings. As observed in the work by Hveem, there is a strong correlation between pavement deflections (an indicator of the structural adequacy of the pavement) and the ability of the pavement to carry traffic loadings at a prescribed minimum level of service.(1) Early work attempted to identify maximum deflection limits below which pavements were expected to perform well, and these limits were based on experience and observations of performance of similar pavements. This concept quickly lent itself to overlay design, in that required overlay thicknesses could be determined based on trying to reduce maximum pavement deflections to tolerable levels.

When complete deflection basins are available, deflection testing can provide key properties for the existing pavement structure through backcalculation of the measured pavement responses. Specifically, for HMA pavements, the elastic modulus (E) of the individual paving layers can be determined, along with the resilient modulus (M_R) of the subgrade. For PCC pavements, the elastic modulus (E) of the slab and the modulus of subgrade reaction (k or k-value) can be determined. In addition, deflection testing conducted on PCC pavements can be used to estimate the LTE across joints or cracks (see figure 2) as well as to identify loss of support at slab corners.



©National Highway Institute
δ_L = Deflection at loaded slab edge.
δ_U = Deflection at unloaded slab edge.
1 mm = 0.039 inches.

Figure 2. Diagram. Comparison of LTE.[(9)]

These properties of the pavement layers and of the subgrade are used in pavement design procedures or in performance prediction models to estimate the remaining life or load-carrying capacity of the pavement. They can also be used in elastic layer or finite element programs to compute stresses and strains in the pavement structure and are also direct inputs in many overlay design procedures to determine the required overlay thickness needed for the current pavement condition and the anticipated future traffic loadings.

Deflection data can also be used in a number of other ways to help characterize the condition of the existing pavement. For example, plots of deflection data along a pavement project can be examined for nonuniformity, which may suggest areas that require further investigation using destructive means. In addition, daily or seasonal deflection data can provide insight regarding a pavement's response to environmental forces, including the effects of thermal curling, frozen support conditions, and asphalt stiffening. Some agencies also use deflection criteria to establish seasonal load restrictions for certain low-volume roads. Deflection testing has also seen some limited use as a means of monitoring the quality of a pavement during construction.[(3)] Finally, a few agencies conduct deflection testing at the network level to provide a general indication of the structural capacity of the pavement structure.

As alluded to in chapter 1, pavement deflection testing provides some distinct advantages over destructive testing, including the following:[(10)]

- More rapid testing operation.
- Relative ease of operation.
- Lower operating cost.
- Reduced manpower requirements.

- Less intrusive procedure.
- Increased number of test points.

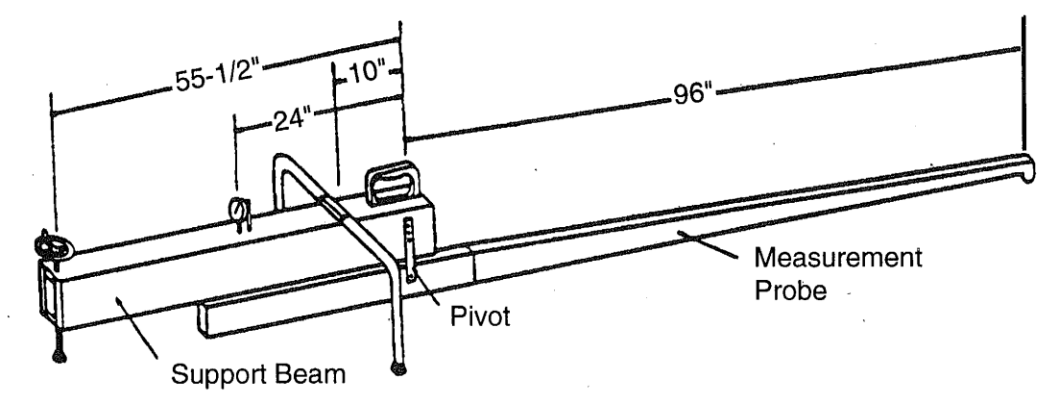
PAVEMENT DEFLECTION MEASUREMENT DEVICES

At the time of this report, there were many different commercially available deflection-testing devices. These devices could be generally grouped based on the type of loading imparted on the pavement (static, steady-state vibratory, and impulse). More recently, a fourth type of deflection device was introduced, one in which dynamic deflections were continuously measured at highway speeds. This section describes each of these types of devices, including their principles of operation, advantages, and disadvantages.

Static Deflection Devices (Benkelman Beam)

Static deflection devices measure the pavement's response under a static or slow-moving wheel load and include equipment such as the Benkelman Beam, plate bearing tests, and curvature meters.(11) Of these, the Benkelman Beam is the most commonly used, and in fact has a long history of use as a deflection-measuring device. The Benkelman Beam was developed by A.C. Benkelman while assigned at the Western Association of State Highway Officials Road Test in the 1950s.(12) It was also used at the American Association of State Highway Officials Road Test in the late 1950s and by the Asphalt Institute (AI) in the 1960s and 1970s for HMA overlay design.

The Benkelman Beam consists of a support beam, a probe arm, and a dial gauge (see figure 3). The device is used by placing the tip of the probe between the dual tires of a loaded truck, typically with an 8,172-kg (18,000-lb) axle load; as the loaded vehicle moves away from the beam, the upward movement of the pavement (termed the rebound deflection) is recorded by a dial gauge.(10)



1 inch = 25.4 mm.

Figure 3. Diagram. Schematic of Benkelman Beam device.(13)

The advantages of the Benkelman Beam include its ease of use, low equipment cost, and the existence of a large database from its use over many years.(9) Disadvantages of this device include the following:(9)

- Difficulty ensuring that the front supports are not in the deflection basin (particularly on stiff pavement structures).

- Difficulty or inability to determine the shape and size of the deflection basin (only maximum deflection is obtained).

- Poor repeatability of measurements obtained by the device.

- Labor-intensive and cumbersome nature of the device.

In the 1970s, automated deflection beams, such as the Lacroix deflectograph and the California Traveling Deflectometer, were developed to overcome the labor and speed disadvantages of the Benkelman Beam, but these still had the other limitations associated with the use of the Benkelman Beam.(10)

Steady-State Vibratory Deflection Devices

In this category of deflection devices, a relatively large static preload is applied to the pavement, with a sinusoidal force superimposed to simulate a dynamic loading condition. A typical loading series is shown in figure 4, in which the static load is constant and a dynamic force is produced at a fixed driving frequency.(9) The amplitude of the peak-to-peak dynamic force must be less than the static force to preclude the possibility of the device bouncing off of the pavement surface. However, the presence of the static preload may make it difficult to interpret the resultant deflection data because it could close voids beneath the surface or it could influence deflections on particularly stress-sensitive materials.(9)

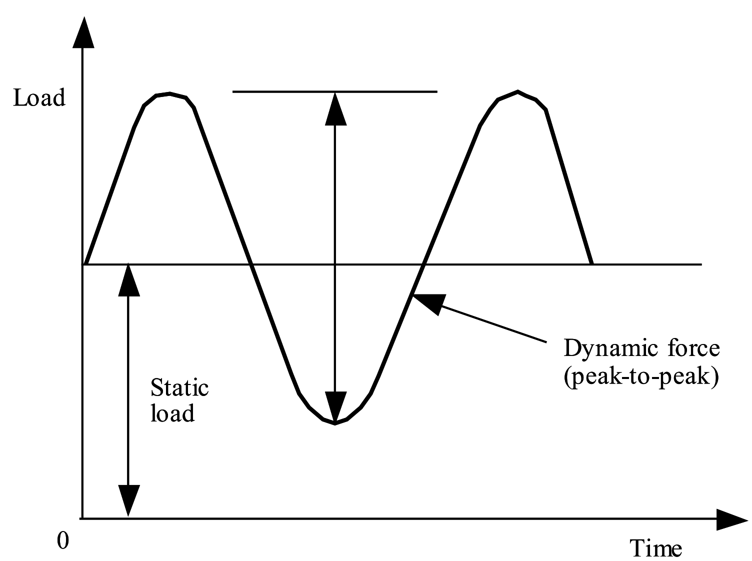
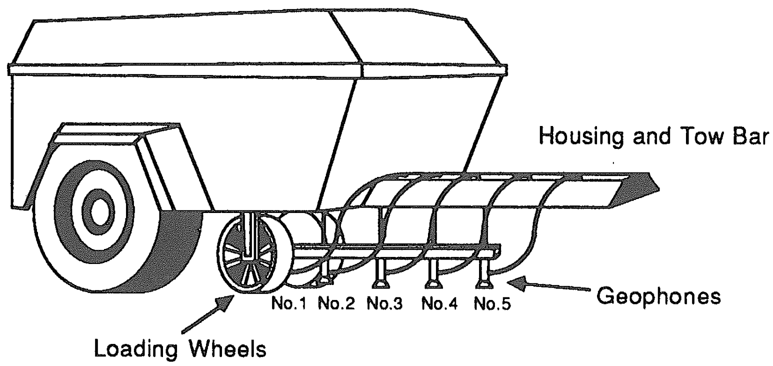


Figure 4. Graph. Typical output of vibrating steady-state force generator.(9)

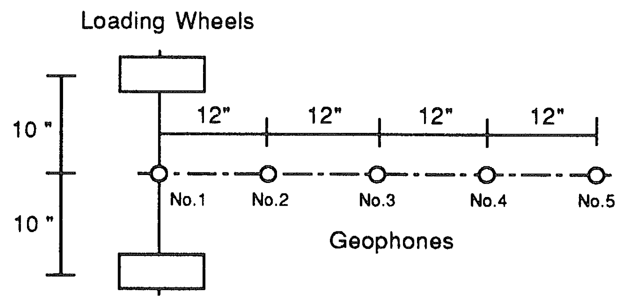
The most common steady-state vibratory deflection devices are the Dynaflect and the Road Rater™. These are discussed separately in the following sections.

Dynaflect

The Dynaflect device is second to the Benkelman Beam in terms of its longevity of use for deflection measurement. It is a trailer-mounted device that is quick and relatively simple to operate. Figure 5 shows the schematic of the Dynaflect device, including the loading wheels and geophones and the geophone configuration.



(a) The Dynaflect with Load Wheels in the Test Position



(b) Configuration of Dynaflect Load Wheels and Geophones

1 inch = 25.4 mm.

Figure 5. Diagram. Schematic of Dynaflect device.[10]

The Dynaflect device is a light-load, fixed-frequency device, with a static weight of 900 kg (2,000 lb), that produces a 4.5-kN (1,000-lbf) peak-to-peak dynamic force at a fixed frequency of 8 cycles/s.[10] The load is applied through two counter-rotating eccentric masses, and the resulting deflections are recorded by five velocity transducers. The transducers are suspended from a placing bar and are normally positioned with one located between the two wheels and the remaining four placed at 300-mm (12-inch) intervals away from the wheels.

The Dynaflect device has the following technical limitations:[14]

- The peak-to-peak force is 4.5 kN (1,000 lbf), which is insufficient for testing thicker pavements and inadequate for characterizing pavements exposed to heavy vehicle loadings.

- Neither the magnitude nor the frequency of the dynamic load can be varied.
- The static preload may affect resultant deflection measurements.

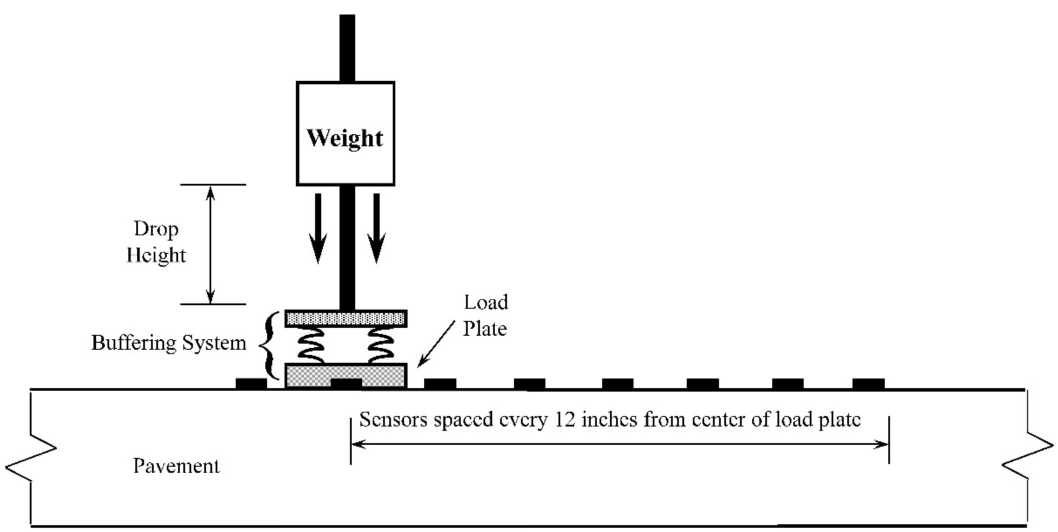
However, the Dynaflect does provide a deflection basin, which allows more meaningful interpretation of the deflection data. In addition, a number of agencies have used the Dynaflect for many years, and it has demonstrated a high level of ruggedness and dependability.

Road Rater™

The Road Rater™ is the other common type of steady-state vibratory deflection equipment. It is similar to the Dynaflect device in that a vibratory load is applied to the pavement, but it has the capability of applying greater loadings than the Dynaflect, depending on the model. For example, the static loads range from 10.7 to 25.8 kN (2,400 to 5,800 lbf), and the peak-to-peak dynamic loads range from 2.2 to 35.6 kN (500 to 8,000 lbf).[14] Moreover, the Road Rater™ applies the load to the pavement via a load plate, as opposed to the rigid wheels on the Dynaflect. Four velocity transducers are used to measure the deflections, one in the center of the plate and the other three placed at 300-mm (12-inch) intervals away from the load plate. The advantages and disadvantages of the Road Rater™ are similar to those of the Dynaflect.

Impulse Load Deflection Devices (FWD)

Under the category of impulse load deflection devices is the FWD, which is the most common deflection-measuring device in use today and is, therefore, the emphasis of this project. As shown in figure 6, the FWD releases a known weight from a given height onto a load plate resting on the pavement structure, producing a load on the pavement that is similar in magnitude and duration to that of a moving wheel load. A series of sensors are located at fixed distances from the load plate so that a deflection basin can be measured. Variations in the force applied to the pavement are obtained by varying the weights and the drop heights; force levels from 13 to more than 222 kN (3,000 to more than 50,000 lbf) can be applied, depending on the equipment type.



1 inch = 25.4 mm.

Figure 6. Diagram. FWD testing schematic.

Developed in the 1970s, the FWD emerged in the 1980s as the worldwide standard for pavement deflection testing. The equipment of two FWD manufacturers—Dynatest® and KUAB—are described in the following sections as illustrative examples, but there are also several other manufacturers of FWD equipment.

Dynatest® FWD

The Dynatest® FWD is a trailer-mounted system (see figure 7) with an operations control computer located in the tow vehicle. The computer controls the complete operation of the FWD, including the lowering and raising of the load plate and deflection sensor bar as well as the sequencing of drop heights. Many FWDs are fitted with external cameras to help operators precisely align on selected testing locations.

Figure 7. Photo. Dynatest® heavyweight FWD.

Dynatest® currently offers two FWD trailer-mounted models, the 8000 and the 8081. The 8000 model applies peak impact loads in the range of 7 to 120 kN (1,500 to 27,000 lbf), whereas the 8081 model (termed the "heavy weight deflectometer") applies peak impact loads in the range of 30 to 320 kN (6,500 to 71,800 lbf).[15] Typical testing production rates range from about 200 to 300 points per day, depending on traffic control requirements and specific testing locations (e.g., basin testing versus joint/crack load transfer testing).

Two different plate sizes can be used with the Dynatest® FWD: a 300-mm- (11.8-inch-) diameter plate or a 450-mm- (17.7-inch-) diameter plate. The smaller plate is typically used for street and highway pavements, whereas the larger plate is commonly used on airfield pavements (and generally on the heavyweight FWD model).

The Dynatest® FWD is used in the Federal Highway Administration's (FHWA) LTPP Program, for which pavement deflection measurements have been routinely collected on more than 900 pavement sections since the late 1980s. FHWA has established four regional FWD calibration centers across the United States to provide annual calibrations on the FWD equipment to ensure it is operating within allowable tolerances. Dynatest® also performs calibrations at its facilities in Florida and California, and at least one other State has its own calibrating facility.

KUAB FWD

Like the Dynatest® FWD, the KUAB FWD is a trailer-mounted device with a loading system and series of deflection sensors. However, it has its own defining characteristics, including a metal housing completely enclosing the loading system (see figure 8). Other characteristics include the following:

- A two-mass system in which the initial load mass is dropped onto an intermediate buffering system. That buffering system transmits the force to another buffering system, which in turn transmits the load to the load plate. The use of the two-mass system creates a smoother load pulse than can be delivered by a single mass system.[14]

- A segmented load plate in which each quarter circle of the plate is capable of conforming to the shape of the pavement surface being tested.

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Figure 8. Photo. KUAB 2m-FWD.[16]

Several KUAB models are available, with the primary difference being the magnitude of the load that can be applied. The heaviest KUAB device can impart a load of 294 kN (66,000 lbf), making it suitable for use in airfield applications.

As with the Dynatest® FWD, the KUAB has two loading plates available: 300 or 450 mm (11.8 or 17.7 inches) diameter. Also, the testing operation is completely automatic, so productivity levels of 200 to 300 points/h can be achieved.

Advantages and Disadvantages of Impulse Load Equipment

In the preceding discussions on impact load deflection devices, a number of advantages were cited for the equipment, including the following:[14]

- Realistic simulation of actual wheel loading.
- High productivity.

- Ability to measure deflection basin.
- Ability to measure joint/crack load transfer.

At the same time, however, the FWD does have some disadvantages, such as the following:(14)

- High initial cost.
- Need for traffic control.
- Relatively complex electromechanical system.

When interpreting FWD deflection data, the loading time is important to consider when evaluating differences in backcalculated moduli of viscoelastic materials because shorter loading times generally result in higher backcalculated modulus values for HMA.(14) The Dynatest® FWD produces a loading time of about 28 to 30 ms, whereas the KUAB produces a loading time of about 80 ms.(14)

Continuous Deflection Profiling Equipment

In the last decade, considerable work has been conducted on the development of deflection-measuring equipment capable of collecting continuous deflection data along the length of a project. Continuous deflection profiles are noted to provide the following advantages over discrete deflection measurements:(17)

- The entire length of the pavement project can be investigated. Thus, there is no danger of missing critical sections and no uncertainty about a test section being representative of the entire pavement system.

- The spatial variability in deflections owing to pavement features such as joints, cracks, patches, and changing constructed or subgrade conditions are identified.

- Testing and measurement operations are more efficient because there is no time lost stopping and starting.

At the time of this report, two such devices were under development, the rolling dynamic deflectometer (RDD) and the rolling wheel deflectometer (RWD). Although both of these devices were still in the prototype stage with no production models available, the following sections describe some of the characteristics of each device.

RDD

The RDD, developed at the University of Texas in the mid-1990s, is a truck-mounted deflectometer that applies large cyclic loads to the pavement and measures the induced cyclic deflections as it moves along the roadway.(18) Several deflection sensors are used on the RDD to measure deflections at different distances from the loaded areas. Often, however, only the maximum deflection is collected and studied to provide an indication of the overall stiffness of the pavement so the pavement can be divided into areas of similar response.(17) Deflection testing can be performed while the RDD vehicle travels at speeds of up to 2.4 km/h (1.5 mi/h).

Figure 9 shows a schematic drawing of the RDD. The truck has a gross weight of about 20,000 kg (44,000 lb). A large diesel engine on the rear of the truck powers a hydraulic pump. This hydraulic system powers the loading system, which applies a combined static and dynamic sinusoidal force to the pavement through two loading rollers. The hydraulic system is capable of generating dynamic forces up to 154 kN (34,700 lbf) at frequencies from 5 to 100 Hz. The dynamic forces are transferred down the stilt structure, to the loading frame, and then through the loading rollers to the pavement (see figure 10). The force applied to the pavement is measured with load cells located between the loading frame and the bearings of the loading rollers. The displacements induced by the applied dynamic force are measured with multiple rolling sensors that are pulled along with the truck.

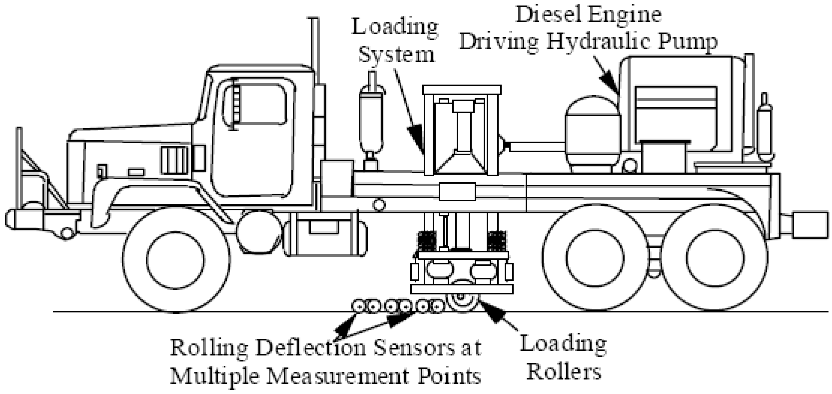


Figure 9. Diagram. Schematic of an RDD.[(18)]

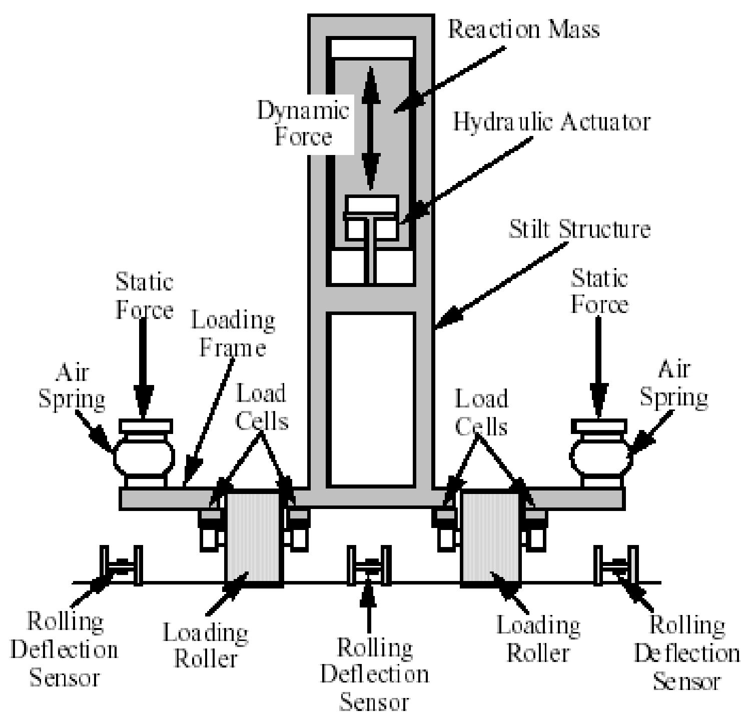


Figure 10. Diagram. RDD loading and deflection measurement systems.[(18)]

Several applications are especially well suited for RDD testing. One is quality assurance and quality control (QA/QC) because continuous profiling can help identify all sections of the pavement system not conforming to specifications.(18) Another application is the development of load ratings for pavements because RDD testing can be performed along the entire pavement to identify critical sections.(18) Continuous profiling can also eliminate or limit the need for traffic control and associated costs.

RWD

The RWD is a dual-wheel, single-axle semitrailer equipped with four spot lasers mounted on an aluminum beam beneath the trailer to measure deflections (see figure 11). The trailer is 16 m (53 ft) long and can vary the single axle load from 8,160 to 10,890 kg (18,000 to 24,000 lb) through the use of water tanks permanently installed over the rear axle.(19) The long trailer was selected to minimize differential bouncing from the front to the rear of the trailer and to allow for the long beam length so that the forward lasers are sufficiently away from the rear tractor axle.(19) The aluminum beam is 7.8 m (25.5 ft) long and is outfitted with four spot lasers mounted 2.6 m (8.5 ft) apart with the rearmost laser placed 152 mm (6 inches) behind the axle centerline.(20)

Figure 11. Photo. RWD collecting deflection data (aluminum beam beneath trailer contains laser sensors).(20)

The RWD configuration allows collection of deflection data at speeds up to 88 km/h (55 mi/h) at intervals of 12.2 mm (0.5 inches). In a field trial, such a high productivity allowed the collection of deflection data from more than 483 km (300 mi) of pavement in a single day.(20) Evaluation studies of those data found that the deflection data collected by the RWD compared favorably with that collected by an FWD; moreover, multiple RWD passes on several days for the same section produced results that were reasonable in magnitude and showed fair repeatability.(19) Other items of note from the field trials include the following:(19,20)

- The RWD prototype was physically limited from being able to measure deflections directly at the axle centerline between the dual tires. The laser used to calculate deflection was located 277 mm (10.9 inches) forward of the axle centerline, which meant that, when combined with the delay in the peak deflection (which occurred approximately 150 mm (6 inches) behind the axle centerline), the RWD effectively measured deflections about 406 to 457 mm (16 to 18 inches) forward of the actual peak deflection. This resulted in smaller deflections and provided less contrast between pavements of varying stiffnesses.

- RWD results were sensitive to factors that did not affect FWD-measured deflections, such as driver habits (uniform speed and minimizing sudden steering corrections), pavement texture, and roughness.
- The RWD experienced a warming up effect prior to stabilization of readings in that the first one or two runs of the repeated measurements showed systematically higher deflections than the others.

Although the RWD was still in the phase of prototype testing and improvements, the potential benefit of the RWD would be that it would help highway officials prioritize and target funding and projects to those segments of the highway network that needed structural improvement and rehabilitation.[20]

CORRELATIONS BETWEEN DEFLECTION-MEASURING EQUIPMENT

Because the pavement deflections measured with the different devices reflect different loading conditions (static versus dynamic) and load duration, the pavement deflections obtained from the various deflection devices cannot be universally substituted for one another. However, sometimes there is a need to convert deflections from one device into deflections obtained from another; for example, the AI overlay design procedure is based on Benkelman Beam deflections, but many agencies have moved to the FWD device while still using the AI design procedure.[21] To address this need, some very rough correlations have been developed, but these should be used cautiously because they are often based on limited data and are valid only for the specific set of conditions under which the procedure was developed.[14] Some of these general relationships are provided in figure 12 through figure 17.

$$BB = 1.33269 + 0.93748(FWD)$$

Figure 12. Equation. Conversion of FWD deflection to Benkelman Beam deflection—method 1.[14]

Where:

BB = Benkelman Beam deflection, 0.001 inches.
FWD = FWD maximum deflection, 0.001 inches (normalized to a 40-kN (9,000-lbf) load applied on a 300-mm (11.8-inch) diameter plate).

$$BB = 1.61(FWD)$$

Figure 13. Equation. Conversion of FWD deflection to Benkelman Beam deflection—method 2.[21]

Where:

BB = Benkelman Beam deflection, 0.001 inches.
FWD = FWD maximum deflection, 0.001 inches (normalized to a 40-kN (9,000-lbf) load applied on a 300-mm (11.8-inch) diameter plate).

$$BB = 22.5(D)$$

Figure 14. Equation. Conversion of Dynaflect deflection to Benkelman Beam deflection—method 1.[14]

Where:

BB = Benkelman Beam deflection, 0.001 inches.
D =Dynaflect maximum deflection, 0.001 inches.

$$BB = 22.3(D) - 2.73$$

Figure 15. Equation. Conversion of Dynaflect deflection to Benkelman Beam deflection—method 2.[21]

Where:

BB = Benkelman Beam deflection, 0.001 inches.
D = Dynaflect maximum deflection, 0.001 inches.

$$BB = 2.57 + 1.27(RR)$$

Figure 16. Equation. Conversion of Road Rater™ deflection to Benkelman Beam deflection—method 1.[14]

Where:

BB = Benkelman Beam deflection, 0.001 inches.
RR = Road Rater™ maximum deflection, 0.001 inches (at 36 kN (8,000 lbf)).

$$FWD = -3.40 + 1.21(RR)$$

Figure 17. Equation. Conversion of Road Rater™ deflection to FWD deflection.[14]

Where:

FWD = Maximum FWD deflection, 0.001 inches (under 36 kN (8,000 lbf) on a 300-mm (11.8-inch) diameter plate).
RR = Road Rater™ maximum deflection, 0.001 inches (at 36 kN (8,000 lbf) on a 300-mm (11.8-inch) load plate).

FWD TESTING PATTERNS

Typical testing patterns for FWD testing vary, depending on the purpose of the testing and on the type and condition of the pavement. For network-level testing, deflection testing is conducted at greater intervals, commonly 150 to 450 m (500 to 1,500 ft) in a single traffic lane.(14) This level of testing is normally sufficient to provide a general indicator of structural adequacy of the pavement network.

For project-level testing, the spatial location of the deflection points should be adequate to capture the variability in structural capacity of the pavement; pavements with greater variability in structural condition should be subjected to a greater number of deflection measurements.(11) Typical project-level testing intervals for both HMA and PCC pavements are between 30 and 150 m (100 and 500 ft), with the shorter testing interval warranted for pavements in poorer condition and the larger testing interval appropriate for pavements in better condition. Even shorter testing intervals are sometimes used for research projects. If needed, a testing pattern can be set up to stagger the tests across traffic lanes, although often, traffic control constraints may prevent that. Recommended testing locations for specific pavement types include the following:

- **HMA pavements**: Outer wheelpath of the outer traffic lane.

- **PCC pavements**: Basin testing in the center slab of the outer traffic lane (to minimize edge effects) and joint load transfer testing in the outer wheelpath of the outer traffic lane. In addition, testing at selected slab corners locations may also be conducted to detect the presence of voids.

FWD CALIBRATION

Routine FWD calibration is a vital component to ensure accurate loading and deflection measurements. As outlined in AASHTO R32-09, *Calibrating the Load Cell and Deflection Sensors for a Falling Weight Deflectometer*, FWD calibration should include the following:(22)

- Annual calibration of the load cell and deflection sensors using an independently calibrated reference device (referred to as “reference calibration”). Deflection sensors are also compared with each other (referred to as “relative calibration)”. Annual calibration should also be conducted as soon as possible after load cell or deflection sensor replacement. Annual calibration is performed by a certified technician.

- Monthly relative calibration of the deflection sensors. Monthly deflection sensor calibration is conducted using a relative calibration stand supplied by the FWD manufacturer and is different than the relative calibration conducted during annual calibration. Relative calibration should also be conducted immediately after replacement of a deflection sensor. Relative calibration does not require a certified technician.

FACTORS AFFECTING DEFLECTION VALUES

A number of factors affect the magnitude of measured pavement deflections, which makes the interpretation of deflection results difficult. To the extent possible, direct consideration of these factors should be an integral part of the deflection-testing process so that the resultant deflection data are meaningful and representative of actual conditions. For example, conducting load transfer testing on PCC slabs in the afternoon of a warm day (when the slabs have expanded and the joints are tight) produces very high load transfer results, which likely are not representative of the load transfer capabilities during cooler temperatures (when the slabs have contracted and the joints are open). Recognizing and accounting for these factors before the establishment of a field testing program helps ensure the collection of useful deflection data that can be used in subsequent backcalculation analyses.

The major factors that affect pavement deflections can be grouped into categories of pavement structure (type and thickness), pavement loading (load magnitude and type of loading), and climate (temperature and seasonal effects). Each of these is discussed in the sections that follow.

Pavement Structure

In essence, the deflection of a pavement represents an overall system response of the surface, base, and subbase layers, as well as the subgrade itself. Thus, the properties of the surface layer (thickness and stiffness) and of the supporting layers (thickness and stiffness) all affect the magnitude of the measured deflections. Generally speaking, "weaker" systems deflect more than "stronger" systems under the same load, with the exact shape of the deflection basin related to the stiffness of the individual paving layers.[9] As a general rule, pavements of similar materials (flexible or rigid) exhibiting greater deflections typically have shorter service lives. Figure 18 illustrates typical flexible and rigid pavement deflection responses to loading.

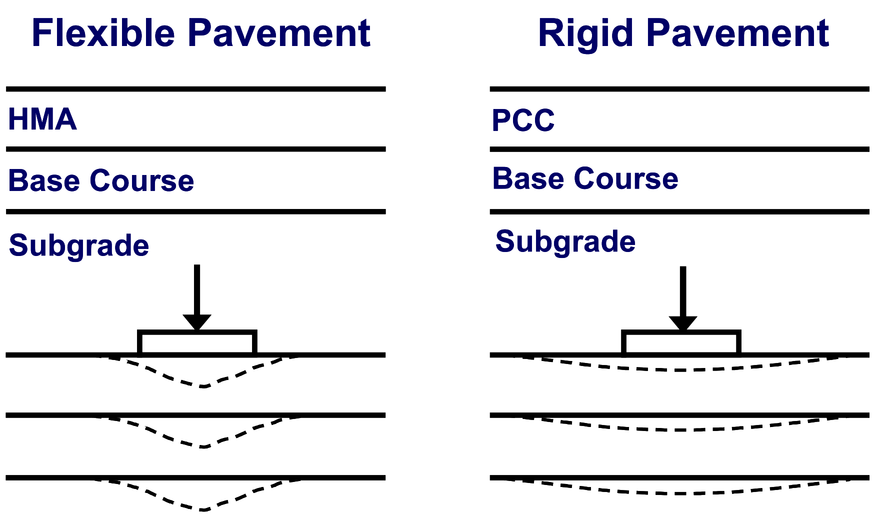


Figure 18. Diagram. Comparison of typical flexible and rigid pavement deflection responses.

Many other pavement-related factors can affect deflections, including the following:

- Testing near joints, edges, cracks, or in areas containing structural distress (such as alligator cracking), can produce higher deflections than testing at interior portions of the pavement.
- Random variations in pavement layer thickness can create variabilities in deflection.

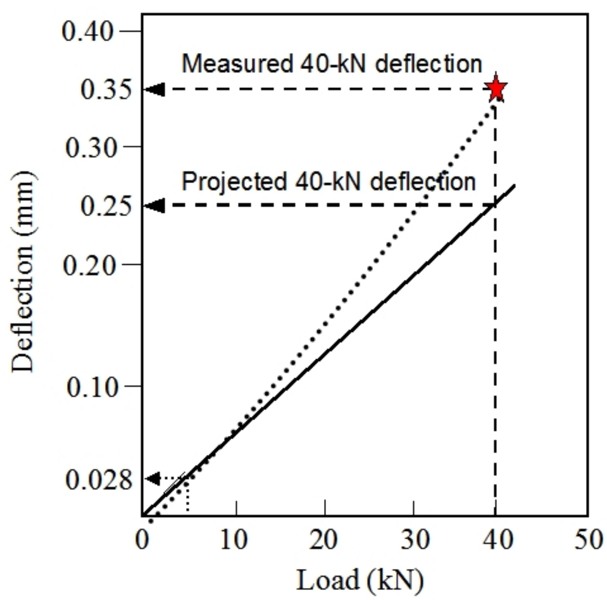
- Variations in subgrade properties and the presence of underlying rigid layers (such as bedrock or a high water table level) can provide significant variability in deflections.

Loading

Load Magnitude

One of the most obvious factors that affects pavement deflections is the magnitude of the applied load. Most modern deflection equipment can impose load levels from as little as 13 kN (3,000 lbf) to more than 245 kN (55,000 lbf), and it is important that appropriate load levels be targeted for each application. For example, for most highway pavement testing, a nominal load level of 40 kN (9,000 lbf) is often used because this is representative of a standard 80-kN (18,000-lbf) axle load. On the other hand, load levels of 156 to 200 kN (35,000 to 45,000 lbf), selected to match the wheel loads of commercial aircraft, may be needed on heavy-duty airfield pavements.

An important reason for selecting test loads as close as possible to the design loads is the nonlinear deflection behavior exhibited by many pavements. This is generically shown in figure 19, in which a pavement structure exhibits a deflection of 0.028 mm (0.001 inches) under a 4.4-kN (1,000-lbf) loading, and a deflection of 0.35 mm (0.014 inches) under a 40-kN (9,000 lbf) load. Had the 40-kN (9,000 lbf) deflection been projected based on the 4.4-kN (1,000 lbf) load, a deflection of 0.25 mm (0.01 inches) would have been erroneously projected. Nonlinear pavement response can result from a number of factors, including viscoelastic behavior, stress sensitive materials, and nonuniform support conditions.



Measured deflection:
$\delta_{40\text{-kN}} = 0.35$ mm
Projected deflection:
$\delta_{40\text{-kN}} = 0.25$ mm
25.4 mm =1 inch
1 kN = 224.8 lbf

Figure 19. Graph. Nonlinear pavement deflection response.[(9)]

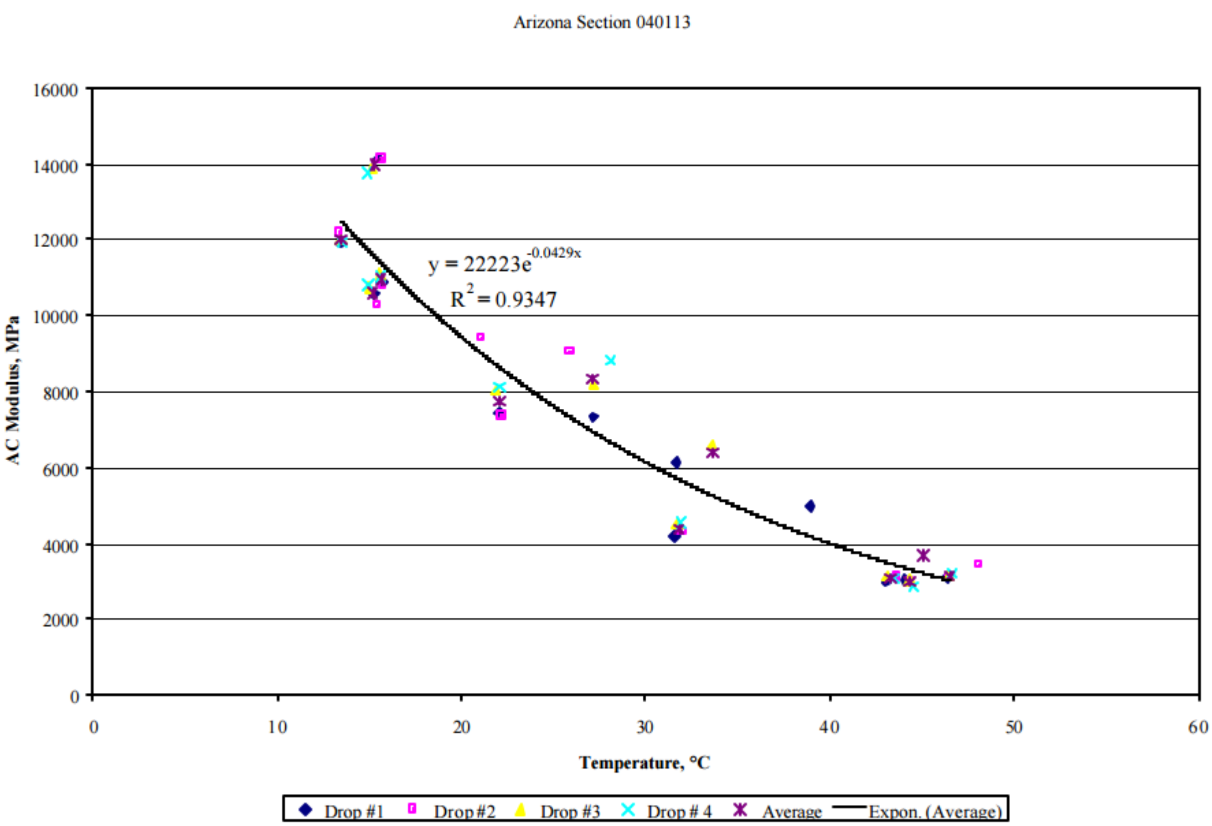
Type of Loading

Pavement deflection response can also be affected by the type of loading; a slow, static loading condition produces a different response than a rapid, dynamic loading condition. In general, the more rapid the loading (i.e., the shorter the load pulse), the smaller the deflections; this is why the static load devices (such as the Benkelman Beam) tend to produce deflections larger than those produced by dynamic loading devices (such as the FWD).

Climate

Pavement Temperature

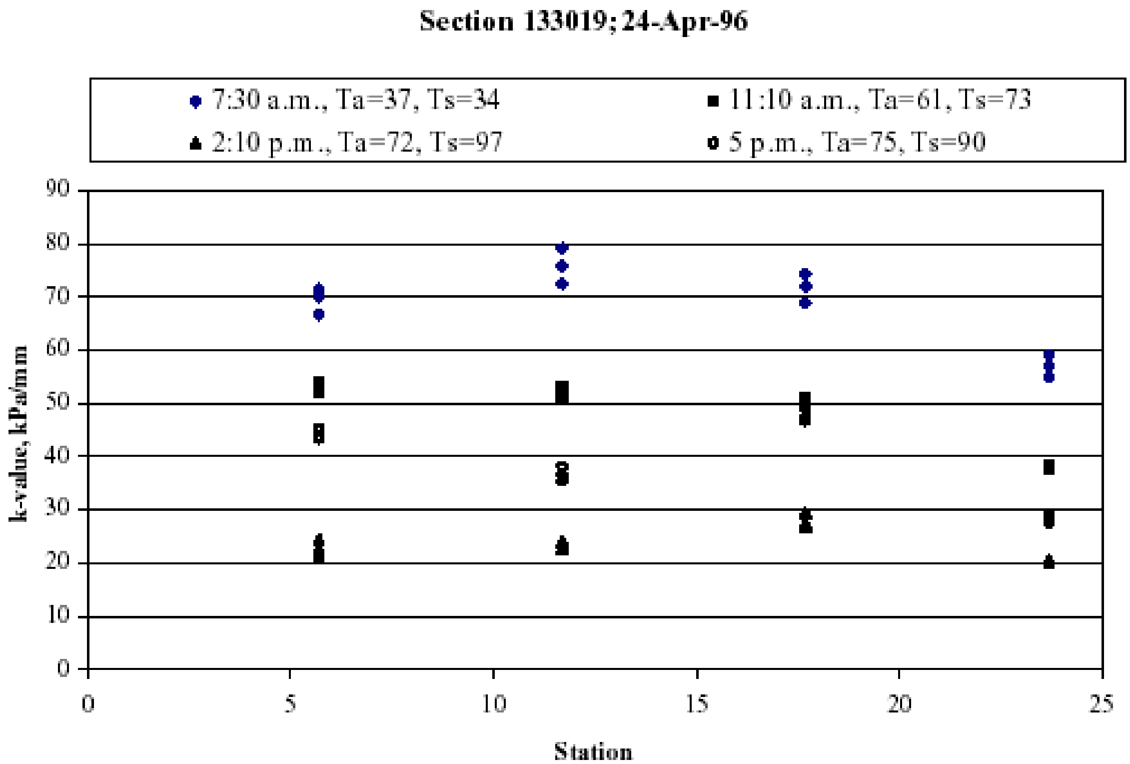
Temperature is a very important factor that must be considered as part of any pavement deflection-testing program. For HMA pavements, the stiffness of the asphalt layer decreases as the temperature increases, resulting in larger deflections (see figure 20). Therefore, correction of the measured deflections to a standard temperature is required to perform meaningful interpretations of the data. Thickness design procedures also typically assume a standard HMA temperature. Correction factor charts are available to assist in converting deflections to a standard temperature, generally 20 °C (68 °F). When correcting to a standard temperature, FWD testing should ideally occur within a reasonable range of the standard temperature.



°C = (°F -32)/1.8.
1 MPa = 145 lbf/inch^2.

Figure 20. Graph. HMA elastic modulus as a function of middepth pavement temperature.[23]

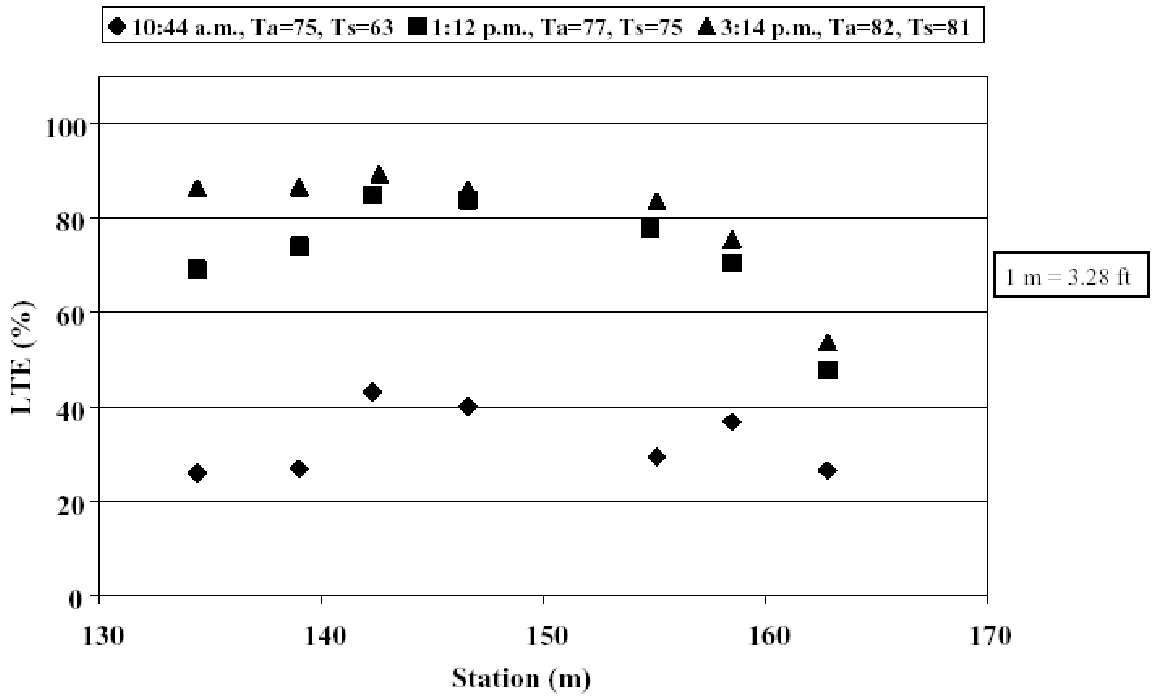
PCC pavement deflections are also affected by temperature, in both basin testing and in joint and corner testing. Differences in temperature between the top and bottom of the slab cause the slab to curl either upward (slab surface is cooler than the slab bottom) or downward (slab surface is warmer than the slab bottom). If basin testing is conducted when the slab is curled down, or if corner testing is conducted when the slab is curled up, the slab could be unsupported and greater deflections may result. Figure 21 shows the effect of daily temperature variations on backcalculated modulus of subgrade reaction (*k*-value).



°C = (°F -32)/1.8.
1 MPa = 145 lbf/inch^2.

Figure 21. Graph. Variation in backcalculated *k*-value due to variation in temperature gradient.[(24)]

Temperature also affects joint and crack behavior in PCC pavements. Warmer temperatures cause the slabs to expand and, coupled with slab curling effects, may "lock up" the joints. Deflection testing conducted at joints when they are locked up results in lower joint deflections and higher LTEs that are misleading regarding the overall load transfer capabilities of the joint. Figure 22 shows the variation in computed LTEs throughout the day, with the higher values computed from data collected in mid-afternoon.[(24)] Because of these effects, it is normally recommended to conduct FWD testing early in the morning or during cold periods of the year on PCC pavements.

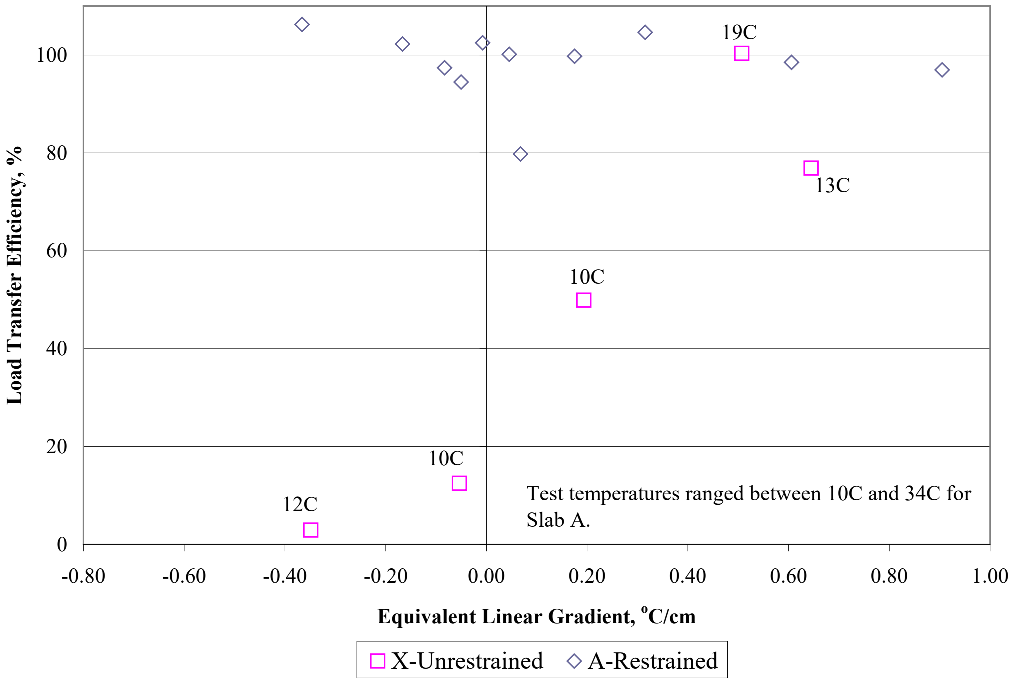


°C = (°F – 32)/1.8.

Figure 22. Graph. Daily variation in the calculated LTEs (leave side of joint).[24]

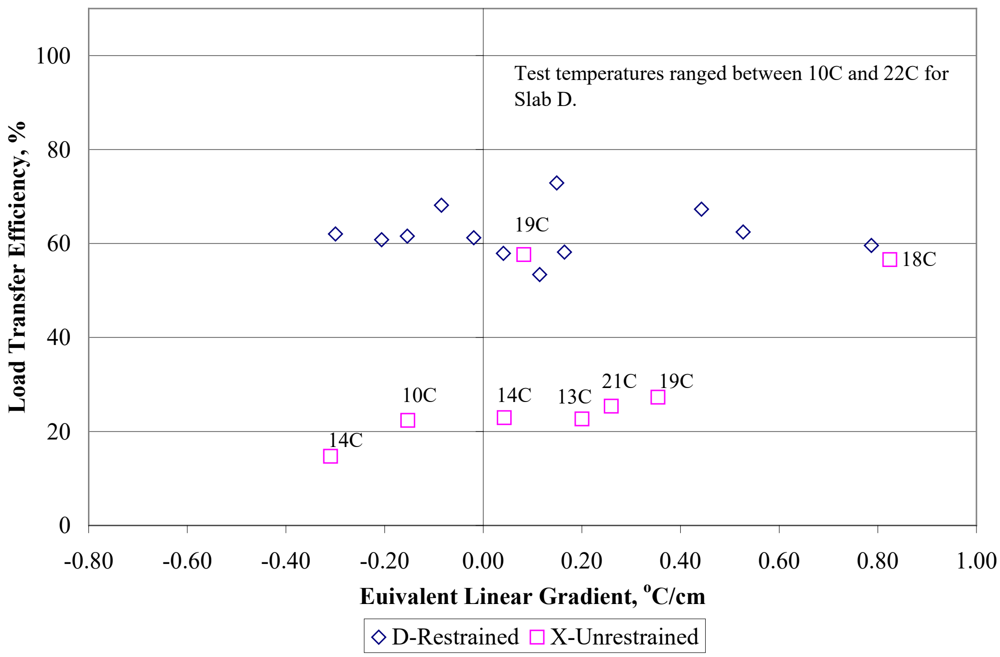
Further insight on the effects of both the weighted average slab temperature and temperature gradients on the LTEs for doweled (restrained) and undoweled (unrestrained) joints can be obtained from figure 23 and figure 24. Figure 23 provides LTEs for two 240-mm (9.5-inch) slabs (one slab is doweled and the other is undoweled) with a 4.6-m (15-ft) joint spacing. The graph shows the equivalent linear gradient and average slab temperature present at the time of testing significantly influenced the LTE of an undoweled joint. The LTE for the doweled joint was not influenced by either the equivalent linear gradient or average slab temperature.

Figure 24 shows the relationship between load transfer and equivalent linear temperature gradient at the time of testing for two 190-mm (7.5-inch) slabs with a 6.1-m (21-ft) joint spacing. This figure shows that even when the LTE of a doweled joint was low (approximately 60 percent), it was still not significantly influenced by the weighted average temperature or the equivalent linear gradient.[25]



°C = (°F – 32)/1.8.
1 cm = 0.39 inches.

Figure 23. Graph. Variation in the calculated LTEs for two slabs tested at different temperature gradients and weighted average slab temperatures.[25]



°C = (°F − 32)/1.8.
1.81 cm = 0.39 inches.

Figure 24. Graph. Relationship between LTEs and equivalent linear temperature gradients for two joints with low LTEs.(25)

Testing Season

Seasonal variations in temperature and moisture conditions also affect pavement deflection response. Generally speaking, deflections are greatest in the spring because of saturated conditions and reduced pavement support and lowest in the winter when the underlying layers and subgrade are frozen (see figure 25). This is the reason that many agencies located in seasonal frost areas place spring load restrictions on their secondary flexible pavements; otherwise, a significant amount of damage could be inflicted on the pavements when the pavement layers are in a weakened and saturated state. PCC pavements are less affected by seasonal variations in support conditions.

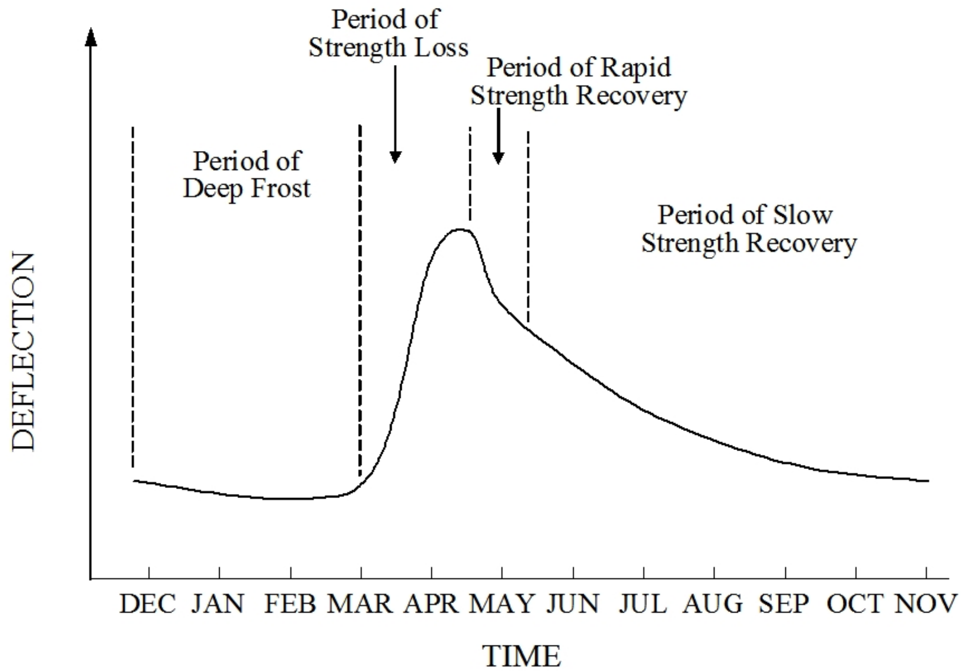
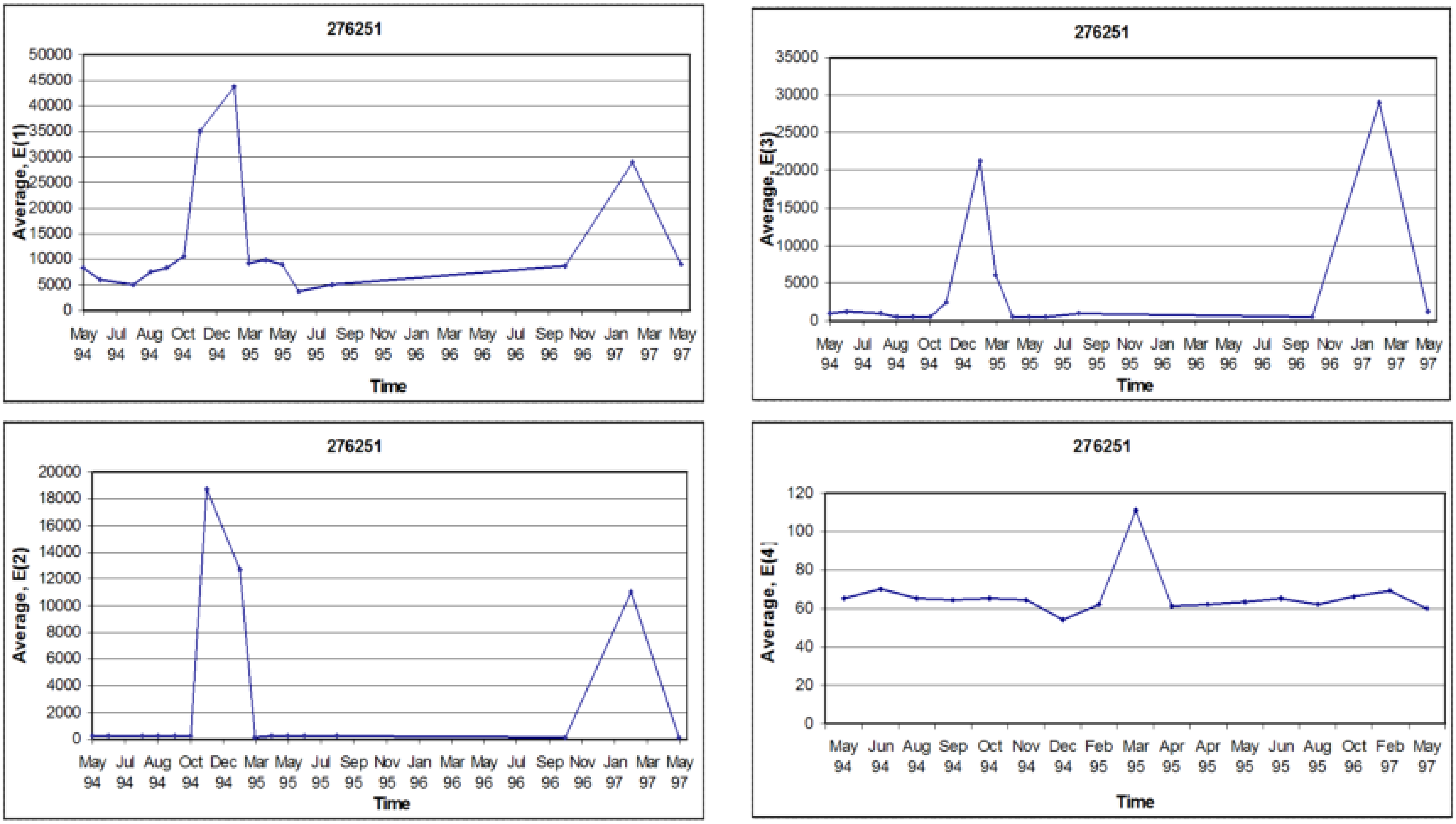


Figure 25. Graph. Seasonal effects on pavement deflection.[26]

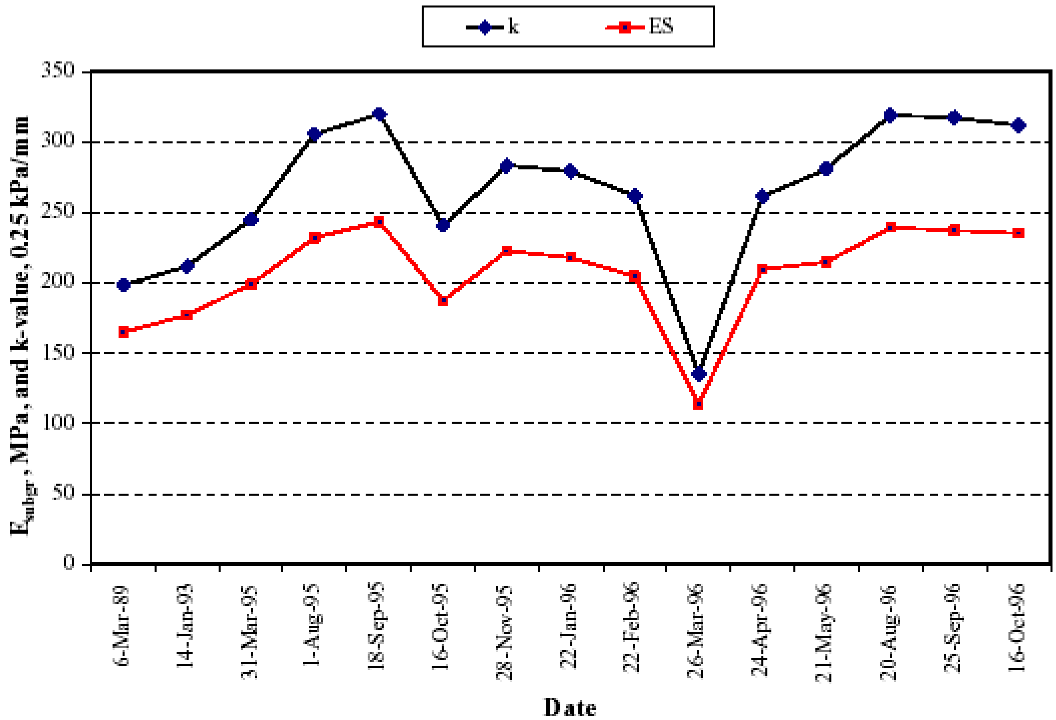
Backcalculated modulus values can also vary seasonally. For example, figure 26 illustrates the variation of the computed elastic modulus values for the different layers of a pavement test section (276251) located in Minnesota.[23] The modulus values were provided over a 3-year period, and several abrupt spikes were observed during winter periods in which the HMA layer became very stiff and the prevailing frozen conditions resulted in higher modulus values for the base, subbase, and subgrade. (Note that the time scale on the x-axis in figure 26 is categorical not continuous, based on when the FWD testing was performed.)

Seasonal variations are also apparent on PCC pavements. For example, figure 27 shows seasonal variations in the backcalculated support properties of the subgrade.[27] Both the backcalculated *k*-value and the backcalculated elastic modulus of the subgrade are shown in that figure, with a noticeable decrease in the support conditions observed during the springtime. As another example of seasonal effects on PCC pavements, figure 28 shows the average LTEs over a 2-year period.[24] As a general trend, the LTE parallels the surface temperature, generally decreasing with the decreases in the surface temperature and increasing with increases in the surface temperature.



1 MPa = 145 lbf/inch^2.

Figure 26. Graphs. Comparison of monthly variation in elastic modulus (in MPa) for pavement layers and subgrade.[23]



ES = Elastic modulus of the subgrade.
1 MPa = 145 lbf/inch^2.

Figure 27. Graph. Seasonal variation in backcalculated subgrade modulus.(27)

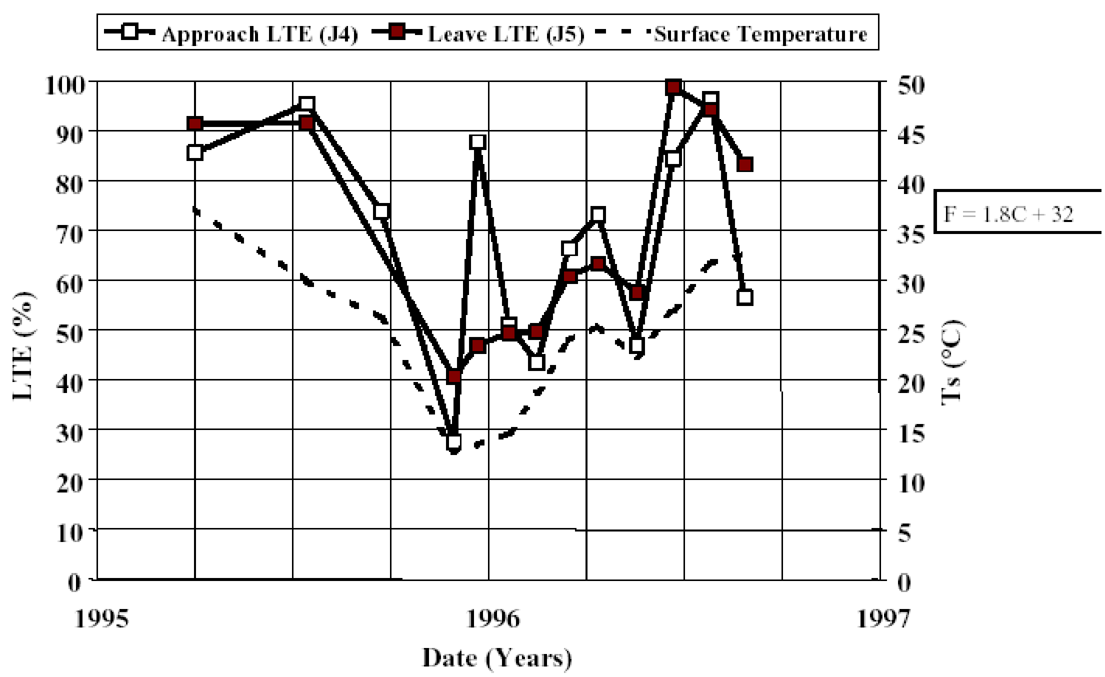


Figure 28. Graph. Seasonal variation in LTE and PCC surface temperature.(24)

SUMMARY

Pavement deflection testing is recognized as a reliable, quick, and cost-effective method for determining the structural condition of existing pavements. Specifically, deflection measurements can be used for backcalculating the elastic moduli of the pavement structural layers and for estimating the load-carrying capacity for both HMA and PCC pavements. In addition, in PCC pavements, identification of loss of support at slab corners and the evaluation of the joint or crack load transfer can be performed using deflection testing.

A number of different types of equipment are available for the collection of pavement deflection data, including static devices (e.g., Benkelman Beam), steady-state vibratory devices (e.g., Dynaflect and Road Rater™), and impulse devices (e.g., FWD). The features and operating characteristics of these devices are described, and it is noted that the FWD has become the worldwide standard for pavement deflection testing, largely because of its ability to closely simulate the loading characteristics of a moving wheel load. Deflection devices capable of providing continuous deflection measurements are currently being developed.

Pavement deflections represent an overall system response of the pavement structure and subgrade soil to an applied load. The major factors that affect pavement deflections can be grouped into categories of pavement structure (type and thickness), pavement loading (load magnitude and type of loading), and climate (temperature and seasonal effects).

CHAPTER 3. BACKCALCULATION CONCEPTS AND APPROACHES

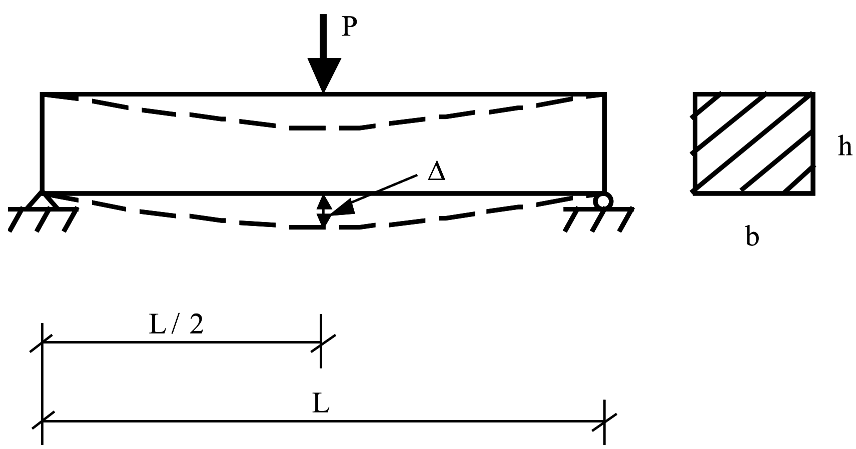
INTRODUCTION

As described in previous chapters, pavement deflection data can be used to determine material properties of pavement layers and the subgrade. In this process, known as "backcalculation," pavement layer stiffnesses are determined based on the deflection data and the assumed pavement cross section. Over the years, the evaluation of pavement deflection data has become increasingly more complex, driven largely by the interest in moving toward more mechanistic pavement analyses and the increasing power of today's sophisticated computers. Many different concepts and approaches are available to perform backcalculation of deflection data, which vary primarily by the type of pavement being analyzed.

This chapter presents an overview of backcalculation concepts for flexible, rigid, and composite pavements, as well as other uses of deflection data. Chapters 4 through 6 discuss FWD data analysis and interpretation for the three pavement types, respectively, in more detail.

GENERAL BACKCALCULATION OVERVIEW

The most widely used technique of determining the effective elastic moduli of the pavement structural layers and subgrade is backcalculation of elastic moduli based on measured surface deflections. The process can be described using the case of a simply supported beam, as depicted in figure 29.[28]



P = Load.
b = Width.
L = Length.
h = Height.
Δ = Maximum deflection.

Figure 29. Diagram. Simply supported beam with a concentrated midspan.[28]

From fundamental engineering mechanics, the maximum deflection occurs under the load (i.e., in the middle of the beam) and is calculated in figure 30.

$$\Delta = \frac{PL^3}{48EI}$$

Figure 30. Equation. Maximum deflection of a beam under a fixed load.

Where:

Δ = Midspan deflection of the beam.
P = Load applied to the surface.
L = Beam span.
E = Elastic modulus of the beam.
I = Moment of inertia for a rectangular beam.

The moment of inertia for a rectangular beam I can be determined from the width and height of the beam, as shown in figure 31.

$$I = \frac{bh^3}{12}$$

Figure 31. Equation. Moment of inertia of rectangular beam.

Where:

b = Beam width.
h = Beam height.

Finally, the elastic modulus of the beam (E) can be calculated by substituting the known values of P, L, b, and h.

Although similar in concept to the beam example just described, the process of backcalculating the elastic moduli for pavements is more complicated because multiple unknowns (i.e., the moduli of the various pavement layers and their interaction with one another) affect the total deflection measured on the surface. Over the years, researchers and practitioners have developed numerous approaches to backcalculate pavement layer and subgrade moduli as well as numerous programs to perform the calculations. Table 1 summarizes some of the available software programs that can be used for backcalculation of deflection data, along with the forward and backcalculation schemes and other characteristics of the programs. In the next sections of this chapter, methods for backcalculating individual layer moduli for flexible, rigid, and composite pavements are described, including a discussion of some of these programs, where appropriate.

Table 1. Summary of available backcalculation programs.

Program Name	Latest Version	Developer	Public Domain	Pavement Type	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Method	Nonlinear Analysis	Layer Interface Analysis	Maximum Number of Layers	Seed Moduli	Range of Acceptable Modulus	Ability to Fix Modulus	Convergence Scheme	Error Weighting Function
BAKFAA	N/A	Federal Aviation Administration (FAA)	Yes	Rigid/ Flexible	Multilayer Elastic Theory	LEAF	Iterative	No	Variable	5	Required	Required	Yes	Sum of squares of absolute error	Yes
BISDEF©	N/A	USACE-WES	No	Flexible	Multilayer Elastic Theory	BISAR	Iterative	No	Variable	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
BOUSDEF	2.0	Zhou et al. (Oregon State)	No	Flexible	Equivalent Thickness	MET	Iterative	Yes	Fixed (rough)	At least 4	Required	Required	NA	Sum of percent errors	N/A
CHEVDEF	N/A	USACE-WES	Yes	Flexible	Multilayer Elastic Theory	CHEVRON	Iterative	No	Fixed (rough)	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
COMDEF	N/A	USACE-WES	No	Composite	Multilayer Elastic Theory	BISAR	Database	No	Fixed (rough)	3	No	No	N/A	Various	No
DBCONPAS	N/A	Tia et al. (University of Florida)	No	Rigid	Finite Element	FEACONS III	Database	Yes	N/A	2	No	No	N/A	N/A	N/A
DIPLOBACK	N/A	Khazanovich and Roesler	No	Composite	Multilayer Elastic +Westergaard	Neural Networks	Closed Form Solution	No	N/A	3	No	No	No	Closed form solution	N/A
ELMOD®/ ELCON	5	Ullidtz (Dynatest®)	No	Flexible/ Rigid	Equivalent Thickness/ Finite Element	MET	Iterative	Yes (subgrade only)	Fixed (rough)	4 (exclusive of rigid layer)	No	No	Yes	Relative error of 5 sensors	No
ELSDEF	N/A	Texas A&M, USACE-WES	No	N/A	Multilayer Elastic Theory	ELSYM5	Iterative	No	Fixed (rough)	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
EMOD	N/A	PCS/Law	No	N/A	Multilayer Elastic Theory	CHEVRON	Iterative	Yes (subgrade only)	Fixed (rough)	3	Required	Required	Yes	Sum of relative squared error	No
EVERCALC©	5.0	Mahoney et al.	Yes	Flexible	Multilayer Elastic Theory	WESLEA	Optimization	Yes	Fixed (rough)	4 (exclusive of rigid layer)	Required if more than 3 layers	Required	Yes	Sum of absolute error	Yes
FPEDD1	NA	Uddin	No	NA	Multilayer Elastic Theory	BASINF	Iterative	Yes	Fixed (rough)	NA	Program Generated	NA	NA	NA	No
ILLI-BACK	NA	Ioannides	No	Rigid and	Closed Form	Closed Form	Closed Form	No	No	2	No	No	No	Closed	NA

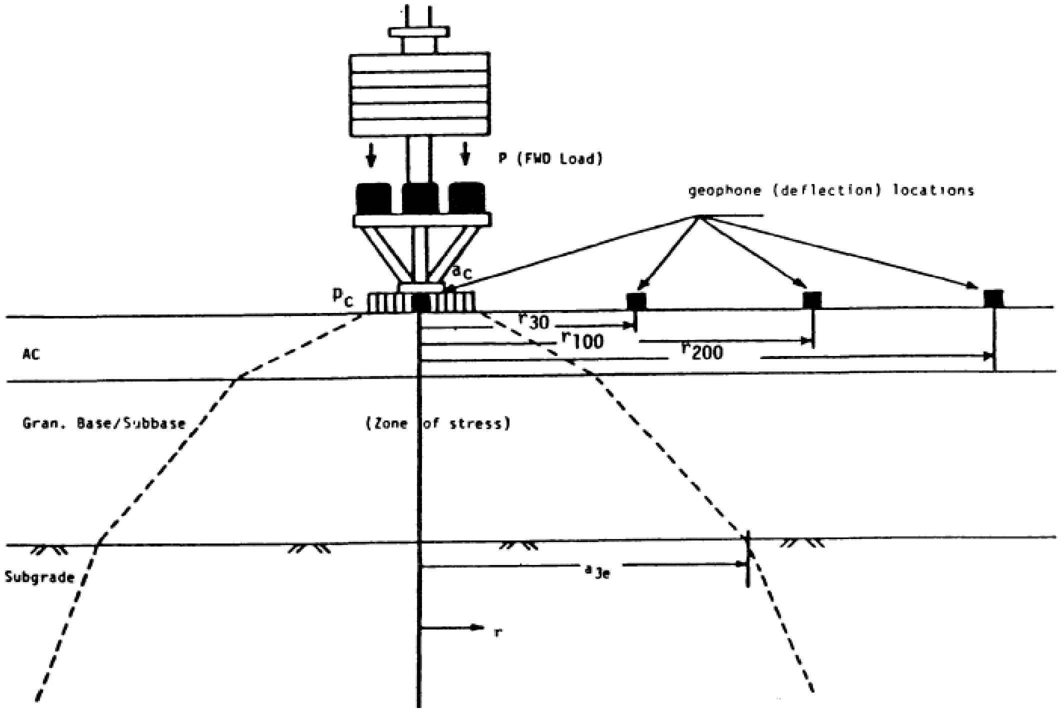
Program Name	Latest Version	Developer	Public Domain	Pavement Type	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Method	Nonlinear Analysis	Layer Interface Analysis	Maximum Number of Layers	Seed Moduli	Range of Acceptable Modulus	Ability to Fix Modulus	Convergence Scheme	Error Weighting Function
				Composite	Solution	Solution	Solution							form solution	
ISSEM4	NA	Ullidtz, Stubstad	No	NA	Multilayer Elastic Theory	ELSYM5	Iterative	Yes (finite cylinder concept)	Fixed (rough)	4	Required	Required	Yes	Relative deflection error	No
MICHBACK©	NA	Harichandran/ Michigan State	Yes	Flexible/ Composite	Multilayer Elastic Theory	CHEVRONX	Raphson-Newton Method	No	NA	3 + rigid layer	Required	Required	Yes	Least squares	Yes
MODCOMP©	6	Irwin, Szebenyi	Yes	Flexible	Multilayer Elastic Theory	CHEVRON	Iterative	Yes	Fixed (rough)	2 to 15 layers; maximum of 5 unknown layers	Required	Required	Yes	Relative deflection error at sensors	No
MODULUS	6.0	Texas Transportation Institute	Yes	Flexible	Multilayer Elastic Theory	WESLEA	Database (Optimization)	No	Fixed	4 plus rigid layer	Required	Required	Yes	Sum of relative squared error	Yes
PADAL	2	Brown et al.	No	NA	Multilayer Elastic Theory	NA	Iterative	Yes (subgrade only)	Fixed	NA	Required	NA	NA	Sum of relative squared error	NA
PCASE	2.08	USACE	Yes	Rigid/ Flexible/ Composite	Multilayer Elastic Theory	LEEP/ WESLEA	Iterative	No	Variable	5	Required	Required	Yes	Sum of squares of absolute error	Yes
RPEDD1	NA	Uddin	No	Rigid	Multilayer Elastic Theory	BASINR	Iterative	Yes	Fixed?	NA	Program Generated	NA	NA	NA	No
WESDEF	NA	USACE-WES	Yes	Flexible	Multilayer Elastic Theory	WESLEA	Iterative	No	Variable	4 + rigid layer	Required	Required	Yes	Sum of squares of absolute error	Yes

NA = Not applicable.
USACE-WES = U.S. Army Corps of Engineers—Waterways Experiment Station.

BACKCALCULATION OF FLEXIBLE PAVEMENTS

To help explain the concept of backcalculation for flexible pavements, consider figure 32, which shows a three-layer system with the surface deflections measured by sensors at five locations. The load is arbitrarily assumed to be distributed through the pavement layers according to the broken lines.[29] In this example, the deflections measured by sensors 4 and 5, which are located outside the stress zone of the HMA and base layers, depend solely on the subgrade. As a first step, any reasonable moduli can be assumed for the HMA and granular base, while the subgrade modulus is varied until the computed deflections at sensors 4 and 5 match measured deflections. Next, the deflection at sensor 3 depends on the moduli of the granular base and the subgrade, which is determined in the first step and is independent of the HMA modulus. Thus, the modulus of the granular base is varied until a satisfactory match between the computed and measured deflection at sensor 3 is obtained. Finally, the modulus of the HMA layer is determined by applying the same procedure to sensors 1 and 2.[29]

One limitation of this procedure is the assumption of the linear response of the pavement layers to the load. The problem is more complicated if the granular base and subgrade behave in a nonlinear fashion. Theoretically, to match the deflection at a given sensor, the stress points for computing the elastic moduli of all nonlinear layers should be located directly beneath that sensor. The use of different stress points for different sensors is not possible because elastic layer theory limits only one stress point in each layer. The best approach is to assume an average stress distribution with the stress points shown by the smaller circles in figure 32.[32] Other issues that complicate the analysis include a dynamic (instead of static) response, the effect of underlying bedrock layers, and the effect of temperature and moisture fluctuations.



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P = Applied load.
P_c = Contact pressure.
a_c = Contact area.
r_{30} = Distance of second geophone from load center.
r_{100} = Distance of third geophone from load center.
r_{200} = Distance of fourth geophone from load center.
a_{3c} = Radial distance in which stress zone intersects interface of subbase and subgrade layers.

Figure 32. Diagram. Stress zone under the FWD load.[32]

Many of the backcalculation approaches (and corresponding programs) for flexible pavements are similar, although the results obtained from the programs do differ because of the inherent assumptions, iteration techniques, and forward and backcalculation schemes built into the programs. The following sections briefly describe the various approaches used for backcalculation of flexible pavements.

Regression Equations

Several researchers have developed regression equations for predicting layer moduli from the deflection testing data. For example, Newcomb developed regression equations for two- and three-layer pavement systems to predict the subgrade modulus as part of an overall effort to develop a mechanistic-empirical overlay design procedure for the Wisconsin Department of Transportation.[30] Horak also developed a regression equation to predict the subgrade modulus using the deflection at 2,000 mm (79 inches) from the center of the load plate.[31]

1993 AASHTO Design Guide

The most commonly used equation for predicting the subgrade modulus is presented in the *1993 AASHTO Design Guide* and shown in figure 33.[32]

$$M_R = \frac{0.24\,P}{d_r\,r}$$

Figure 33. Equation. Backcalculation of subgrade modulus.[(32)]

Where:

M_R = Backcalculated subgrade resilient modulus, MPa (lbf/inch^2).
P = Applied load, kN (lbf).
d_r = Measured deflection at distance r from applied load, mm (inches).
r = Radial distance at which the deflection is measured, mm (inches).

The guide also presents equations to ensure that the deflection used in this equation is sufficiently far away from the load so that the deflection is primarily due to deformation of the subgrade, yet as close as possible to minimize errors associated with smaller deflections.

The guide also presents an equation, shown in figure 34, for predicting the effective modulus of the pavement structure (i.e., all layers above the subgrade).[(32)]

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

Figure 34. Equation. Computation of the effective modulus of the pavement structure.[(32)]

Where:

d_0 = Deflection measured under plate and adjusted to standard temperature, mm (inches).
p = FWD load plate pressure, MPa (lbf/inch^2).
a = FWD load plate area, mm^2 (inches^2).
D = Total thickness of pavement layers above the subgrade, mm (inches).
M_R = Subgrade resilient modulus, MPa (lbf/inch$^{2)}$.
E_p = Effective modulus of all pavement layers above the subgrade, MPa (lbf/inch^2).

Method of Equivalent Thickness (MET)

MET is based on Odemark's assumption, which is that the deflections of a multilayered pavement system with moduli E_i and layer thicknesses h_i can be obtained using a single layer of thickness H and modulus E provided that H satisfies the equation shown in figure 35 where C equals the layer coeifficent.[(33)]

$$H = \sum_{i}^{n} Ch_i \left(\frac{E_i}{E}\right)^{1/3} \qquad C = 0.8 \text{ to } 0.9$$

Figure 35. Equation. Methodology for computing equivalent thickness.[(33)]

Ullidtz reports that this method produces results that are as good as or better than those obtained from layered elastic and finite element solutions, when compared with measured data.[(33,34)] The method can also be adapted to handle nonlinear subgrade materials. Another advantage of this method is its simplicity, which leads to significant savings in computation time.

ELMOD3 is an example of a program that uses the Odemark-Boussinesq method of equivalent layer thickness concept and the radius of curvature method.[(33)] Initially, the subgrade material properties, stiffness and nonlinearity, are calculated using the deflections from the outer sensors. The radius of curvature from the central sensors can be used to assess the stiffness of the upper pavement layer. The stiffness of the remaining layers is then calculated based on the overall pavement response to the applied load. This ensures that the proposed pavement structure results in the correct central deflection under the measured load.

The surface modulus concept is very useful for estimating subgrade modulus and for diagnosing stress-sensitive subgrade material and the presence of stiff layers. It uses Boussinesq's original closed-form equations relating the vertical deflection on the surface of a homogeneous, isotropic, linearly elastic halfspace δ_z and the elastic modulus E (figure 36 and figure 37).

$$\delta_z = \frac{2P}{\pi E a}(1-\mu^2)$$

Figure 36. Equation. Vertical deflection under a uniformly distributed load.

$$\delta_z = \frac{P}{\pi E r}(1-\mu^2)$$

Figure 37. Equation. Vertical deflection under a point load.

Where:

P = Surface load, kN (lbf).
r = Radial distance from center of load, mm (inches).
a = Radius of loaded area, mm (inches).
μ = Poisson's ratio.

These equations can be used directly to backcalculate the surface modulus E_0, given a measured surface deflection due to a known load. For a multilayered pavement system, the calculated surface modulus at the center tends to underestimate the modulus of the surface layer and overestimate the modulus of the subgrade.[(35)] However, the surface modulus approaches a constant value at large radii; this value corresponds to the subgrade modulus, illustrating a basic principle of backcalculation, which is that the outer deflections can be used to determine the moduli of the deeper layers. If the surface modulus increases with increasing radial distance, then it is an indication of a stress-sensitive (nonlinear) subgrade or the presence of a stiff layer.[(33)]

BOUSDEF is another program that uses the MET. This program assumes a single, uniform layer of material and uses Boussinesq's equations to determine theoretical deflections. By matching the deflection basins measured in the field, the program calculates the moduli of the surface, base, and subgrade layer.

Optimization Method

In this method (sometimes referred to as a "basin search method"), a forward calculation program is used to generate a database of deflection basins for different combinations of layer moduli, specified layer thicknesses, material properties, pavement types, and loading conditions. The measured deflection basin is compared with the deflection basins in the database using a search algorithm, and a set of moduli are interpolated from the layer moduli that produces the closest calculated deflection basins in the database.

The MODULUS backcalculation program, which uses databases generated by the WESLEA program, is one program that uses this approach.(36,37) The number of basins required to obtain a suitable database depends on the number of layers and the expected moduli ranges provided by the user (i.e., wide ranges require generation of a greater number of basins than narrow ones). The generated deflection basins are then searched using an algorithm, and the moduli are interpolated using the various deflection basins. The program optimizes the solution by seeking to minimize the relative sum of squared differences between the measured and calculated surface deflections. The program always converges, although the chances of converging to a local minimum cannot be ruled out.(38) The program performs a convexity test to determine the likelihood of having converged to a local minimum, and the user is warned if this test is not satisfied.

WESDEF is another example of a backcalculation program that uses this approach. This program was developed at the USACE-WES. It is capable of handling up to five layers (the bottom layer is "fixed" to act as a rigid layer), although one study showed that reasonable results are obtained if the number of backcalculated layers is kept to three.(39)

This type of approach is best suited for cases when a large number of pavements with a similar configuration need to be tested in succession. For these situations, the generated database can be used repeatedly to backcalculate the pavement layer moduli for all similar pavements, and the time required to generate the database can be minimized. This technique can be used with a database generated from any linear or nonlinear program.(38)

Iterative Method

The iterative method is another approach used in the backcalculation of flexible pavements. In this approach, layer moduli are repeatedly changed until the calculated deflection basin matches the measured deflection basin within a specified tolerance. The primary problem associated with backcalculating elastic moduli for a layered pavement structure is that the equations for calculating pavement surface deflection are not closed-form solutions (i.e., the unknowns cannot be solved directly). Therefore, a rigorous iterative process involving some decision and convergence criteria is required. Figure 38 illustrates how this process works.

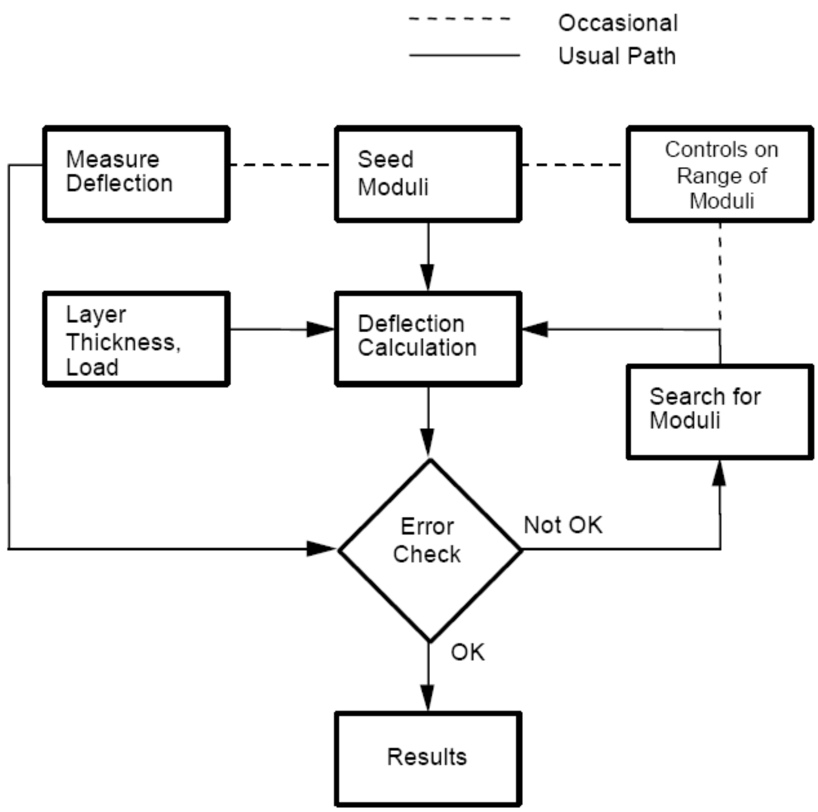

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Figure 38. Chart. Typical iterative backcalculation flow.[40,41]

The search algorithm is usually achieved by minimizing an objective function of any set of independent variables (i.e., layer moduli, thicknesses, etc.), which is commonly defined as the weighted sum of squares of the differences between calculated and measured surface deflections as shown in figure 39.

$$f = \sum_{j=1}^{m} a_j (w_{jm} - w_{jc})^2$$

Figure 39. Equation. Objective function to be minimized in the search algorithm used in iterative method.

Where:

w_{jm} = Measured deflection at sensor *j*, mm (inches).
w_{jc} = Calculated deflection at sensor *j*, mm (inches).
a_j = Weighing factor for sensor *j,* mm (inches).

This equation can be solved using nonlinear optimization methods, which locate the least value of the objective function. Many minimization techniques are available in the literature, including the factored secant update method, modified Levenberg-Marquardt algorithm, and modified Powell hybrid algorithm.[42] One issue with this approach is that the program result may converge to different solutions for different sets of seed moduli. Another issue is that the convergence can be very slow, requiring numerous iterations of the forward calculation program.

An example of an iterative program is EVERCALC©, which uses the Levenberg-Marquardt minimization algorithm.(43) The program seeks to minimize an objective function formed as the sum of squared relative differences between the calculated and measured surface deflections. EVERCALC© is a robust, efficient, and accurate program, and uses the CHEVRON computer program for forward calculations.

CHEVDEF is another example of a backcalculation program that uses the CHEVRON program for forward calculations.(44) This program uses an assumed linear variation in logarithmic space between layer moduli and surface deflections to revise the layer moduli after each iteration. It employs a gradient search technique, and the correct set of moduli is searched in an iterative manner. As previously described, one issue with programs that use this approach is they are highly dependent on the initial seed moduli and can provide different solutions for different initial seed moduli.

MODCOMP© is another program that uses an iterative method. MODCOMP4 was selected as the recommended program to calculate the elastic moduli of the pavement layers and subgrade for the LTPP test sections, considering the following factors:(23)

- Accuracy of the program.
- Operational characteristics.
- Ease of use of the program.
- Stability of the program.
- Probability of success.

MODCOMP4 was found to result in a reasonable solution in more than 90 percent of the initial study sections.(23) This program uses a nonlinear constitutive equation to represent the response of unbound pavement materials and soils, while also offering the use of linear elastic response models to estimate the nonlinear properties. It also converges reasonably fast and can be used in batch mode to analyze numerous deflection basins. The study also found that the root mean square (RMS) error for each solution varied between 0.1 and 1 percent, which was considered acceptable.(23)

MICHBACK© is yet another example of an iterative backcalculation program. It uses the CHEVRONX computer program for forward calculations and the modified Newton-Raphson (also called "secant") method for minimization. The method of least-squares is used to solve the over-determined system of equations (m equations in n unknowns, $m > n$). If desired, weighting factors can be used for each sensor measurement to emphasize some deflection measurements over others.

Dynamic Backcalculation Methods

The previous approaches consider the load applied to the pavement surface to be a static load. Computationally, this approach is very efficient, and when the depths of the layers are known and their properties are largely homogeneous with depth, the procedure is effective in backcalculating layer properties. However, when the depths are uncertain or when the moduli vary within a layer, the static backcalculation scheme may not yield reliable results. Moreover, impulse-loading devices, such as the FWD, impart a dynamic load to the pavement.

To overcome the limitation of static deflection analysis, extensive recent work has been conducted to interpret the dynamic response of FWD deflection time histories. To model the dynamic deflections in pavement layers, the vertical profile is discretized into a number of thin computational layers, and the moduli of these layers are used as variables in an optimization problem that seeks to minimize the residual error between computed and observed deflections over time. The solution is obtained by gradient-based numerical methods that require the repeated evaluation of the error (objective) function and its derivatives, which is also known as a "forward solution." The objective function to be minimized is the difference between the recorded response at the FWD sensors and the computed response of a pavement profile whose properties are encoded in the vector *x*. This represents the error in the model, and the minimization problem may be written as shown in figure 40.[45]

$$\min_{\mathrm{x}\in\Re^{n}} E(\mathrm{x}) = \left\| f(X) \right\|_2^2 \sum_{i=1}^{nr} \sum_{j=1}^{nt} \left\| f_{\mathrm{rec}}(i,j) - f_{\mathrm{com}}(i,j,\mathrm{x}) \right\|^2$$

Figure 40. Equation. Algorithm for minimization of the difference between FWD response and computed response.[45]

Where:

n = Number of computational layers.
nr = Number of receivers.
nt = Number of time steps in the recorded response.
f_{rec} = Recorded motion.
f_{com} = Computed motion.

The forward solution is the most computationally extensive and time-consuming portion of the computation. Various backcalculation computer programs available at the time of this report employ various methods to accelerate this computation. These programs include EVERCALC© (developed by the Washington State Department of Transportation), MICHBACK© (developed by Michigan Department of Transportation), and DYNABACK-F.

The DYNABACK-F program uses new algorithms for backcalculating layer parameters based on dynamic interpretation of FWD deflection time histories using frequency and time-domain solutions. The backcalculation procedure is based on the modified Newton-Raphson method originally adopted in the MICHBACK© program. Singular value decomposition (SVD), in conjunction with scaling and truncation techniques, is employed in solving for the inverse problem. The frequency-domain method uses real and imaginary deflection basins as the measured quantities, while the time-domain method uses either the peak deflections and corresponding time lags or traces of the deflection time histories as the measured quantities to be matched by the backcalculation procedure.[46]

The results indicated that dynamic backcalculation of layer parameters using field data would present some challenges. The frequency-domain method can lead to large errors if the measured

FWD records are truncated before the motions fully decay in time. The time-domain methods, when simultaneously backcalculating layer moduli and thicknesses, produce mixed results. Convergence is not ensured when using peak deflections and corresponding time lags. However, when matching traces of sensor time histories, the SVD method allows very good convergence and the backcalculation of the HMA layer thickness, in addition to the layer moduli and damping, even when using field-measured data.[46]

Artificial Neural Network (ANN) Analysis

As noted previously, the static analysis of deflection data can produce great errors, while the dynamic analysis using gradient search optimization procedures is very time consuming. To overcome limitations of both methods, ANN technologies can be applied to the backcalculation problem. DIPLOBACK is an example of a program that uses this approach.

ANNs are biologically inspired analogies of the human brain. They are composed of many operationally simple yet highly interconnected units. Similar to a human brain, certain types of ANNs can "teach themselves" to recognize common features within the data and to group the data accordingly through repeated exposure to a set of data.[47] ANNs can also generalize an ideal mapping from imperfect examples and extract essential information from input containing both relevant and irrelevant data. Their ability to "see" through noise and distortion to the underlying pattern has been exploited successfully for solving many problems related to pattern recognition.[47]

To solve the backcalculation problem as a pattern recognition task, multilayer, feed-forward ANNs are used (see figure 41). The interconnected units pass information in the form of signal patterns. The output signal patterns from a given input signal pattern are uniquely determined by the distribution of connection strengths throughout the network.

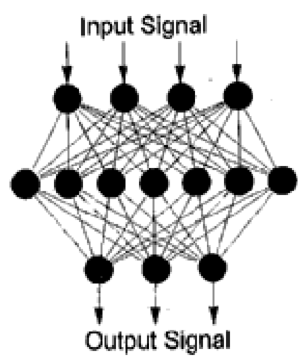


Figure 41. Diagram. ANN architecture.[47]

The excitation level of a processing element is modeled mathematically as a weighted sum of inputs from the neighboring elements, as shown in figure 42.[47]

$$N_j = \sum_{i-1}^{n} w_{ji} x_i$$

Figure 42. Equation. Excitation level of a processing element.[(47)]

Where:

N_j = Excitation level.
w_{ji} = Weight assigned to the connection.
x_i = Signal coming from the *i*th processing element in the preceding layer.

The response of a processing element to the net excitation N_j is modeled by the logistic function in figure 43.[(47)]

$$a_j = f(N_j) = \frac{1}{1 + e^{-N_j}}$$

Figure 43. Equation. Response of a processing element to the net excitation.[(47)]

Multilayered, feed-forward ANNs are commonly trained by a technique known as "error backpropagation." After each training example is presented to the network, the errors (i.e., differences between the calculated and target output patterns) are computed and propagated backward through the network.

Meier and Rix trained backpropagation networks to backcalculate HMA pavement layer moduli from deflection basins obtained using an FWD.[(47)] A three-layer pavement system consisting of an HMA layer, nonstabilized base, and subgrade was considered. Figure 44 illustrates the network architecture chosen to determine the layer moduli for the measured deflection basin and known HMA and base layer thicknesses.

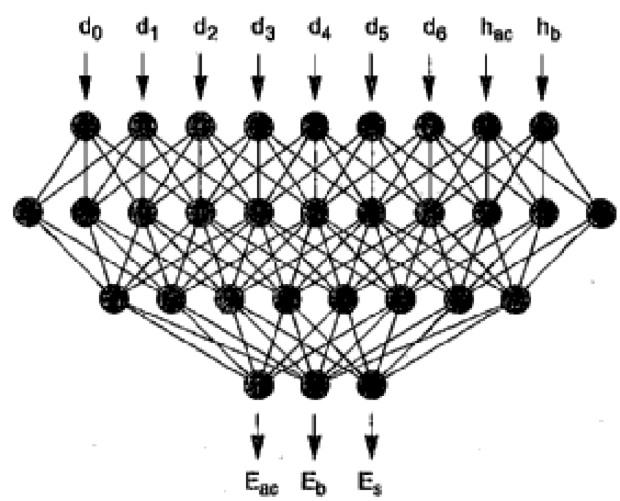


Figure 44. Diagram. ANN for backcalculating pavement moduli.[(47)]

The primary advantage of an ANN is that it is an accelerated backcalculation process. In addition, the ANN analysis does not require seed moduli and ranges, which means the backcalculation process is less dependent on user subjectivity. However, the applicability of the ANN is limited by the range of pavement layer properties that are included in the training process. In addition, increasing the number of layers to be analyzed increase the computation time.

Gucunski, Abdallah, and Nazarian recently developed an ANN for backcalculation of pavement profiles from data obtained from the Surface Analysis of Spectral Waves test.(48) They reported a significant improvement in accuracy of evaluation of pavement properties in comparison with previous ANN models. The improvement was primarily attributed to the use of a data transformation algorithm that generates smoother and more linear relations, thus enabling a better learning process.(48)

BACKCALCULATION OF RIGID PAVEMENTS

As previously mentioned, the deflections measured on a PCC slab are used for backcalculating the PCC elastic modulus and *k*-value of the supporting medium. The PCC elastic modulus can be used to evaluate the structural condition of the PCC slab, while the *k*-value can be used to evaluate supporting layers. Those two parameters are required inputs for both new and overlay design procedures using the new MEPDG.(7)

Traditionally, there are two basic approaches for backcalculation of pavement layer moduli and subgrade support conditions. One approach is based on layered elastic theory. This approach is commonly used for analysis of flexible pavements and has already been described in this chapter. The second approach was developed specifically for rigid pavements and is based on plate theory. With this approach, the pavement structure can be modeled as either a slab on an elastic solid foundation or on a dense liquid foundation. The plate theory approach can be based on either the AREA or Best-Fit method, both of which are briefly described in the following sections (and more thoroughly presented in chapter 5). Both methods are based on Westergaard's solution for an interior loading of a linear elastic, homogeneous, isotropic plate resting on a dense liquid foundation.

AREA Method

For backcalculation, rigid pavements are generally treated as two-layer systems because the base or subbase has little influence on the shape of the deflection basin compared with the influence of the PCC and subgrade.(14) Those systems are modeled as a rigid plate resting on an elastic solid or a dense liquid foundation. Such a configuration allows obtaining the closed-form solution based on the dimensional analysis of the deflection basin, which involves the *AREA* parameter and the radius of relative stiffness ℓ.(49) The procedure consists of the following steps:

- Step 1: Drop the weight and record the applied load (P) and the resulting deflections.
- Step 2: Calculate the normalized area of the basin (*AREA*).
- Step 3: Determine the radius of relative stiffness (ℓ).

- Step 4: Backcalculate subgrade support (*k* or *C* for dense liquid or elastic solid foundation, respectively).

- Step 5: Backcalculate the slab flexural stiffness (*D*), define the slab modulus of elasticity (*E*) if the slab thickness (*h*) is known, or, alternatively, define *h* if *E* is known.

The computational steps of the procedure above are described in more detail in following sections and are further illustrated in chapter 5.

AREA Parameter

The use of a parameter *AREA* was first proposed by Hoffman and Thompson for interpreting flexible pavement deflection basins.(50) This parameter combines the effect of several measured deflections in the basin, defined as shown in figure 45.(27)

$$AREA = \frac{1}{2W_0}\left[W_0 r_1 + \left(\sum_{i=1}^{n-1} W_i(r_{i+1} - r_i)\right) + W_n\,(r_n - r_{n-1})\right]$$

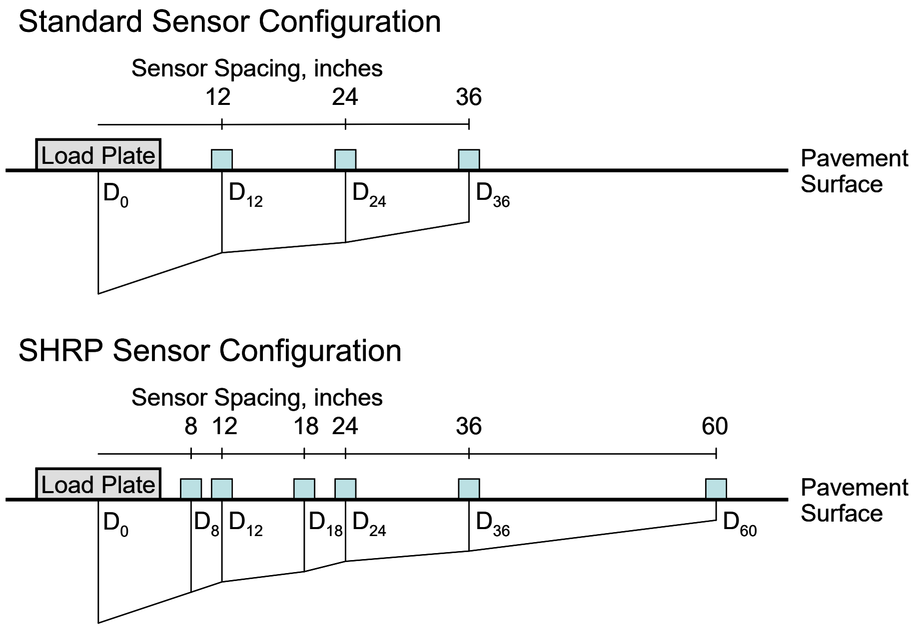
Figure 45. Equation. Parameter *AREA*.(27)

Where:

W_i = Measured deflections (i = 0, n–1), mm (inches).
n = Number of FWD sensors minus 1.
r_i = Distance between the center of the load plate and sensor, mm (inches).
W_n = Measured deflection at the last sensor n.
W_0 = Measured deflection at the load center.

The *AREA* algorithm has been used extensively to analyze PCC pavement deflection basins. Ioannides, Barenberg, and Lary identified the unique relationship between *AREA* and the radius of relative stiffness, and Hall et al. obtained simple approximations for this relationship for different sensor configurations.(49,51) This approach is used for rehabilitation design of rigid pavements in the *1993 AASHTO Design Guide*.(32)

The *AREA* parameter represents the normalized area of a slice taken through the deflection basin between the center of the load plate and an outer sensor.(14) Several methods of computing *AREA* are available, including, for PCC pavements, one using a standard sensor configuration and one using the Strategic Highway Research Program (SHRP) sensor configuration (see figure 46).



1 inch = 25.4 mm.

Figure 46. Diagram. Comparison of standard and SHRP sensor configurations for *AREA* computations.

Where:

d_r = Deflection at radial distance r from the load center, mm (inches).

Using the trapezoidal rule, *AREA* is computed in figure 47 and figure 48 for the different configurations.

$$AREA = 6 + 12\left(\frac{d_{12}}{d_0}\right) + 12\left(\frac{d_{24}}{d_0}\right) + 6\left(\frac{d_{36}}{d_0}\right)$$

Figure 47. Equation. *AREA* via trapezoidal rule for standard sensor configuration.

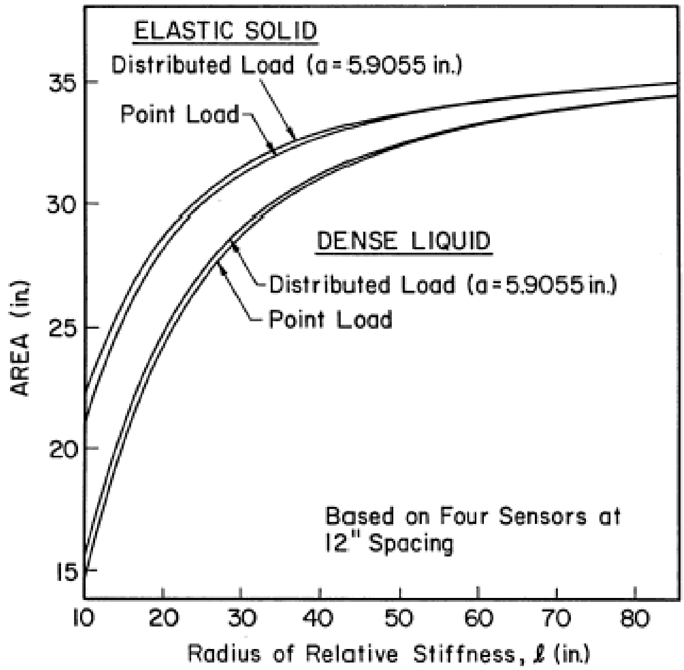
$$AREA = 4 + 6\left(\frac{d_8}{d_0}\right) + 5\left(\frac{d_{12}}{d_8}\right) + 6\left(\frac{d_{18}}{d_0}\right) + 9\left(\frac{d_{24}}{d_0}\right) + 18\left(\frac{d_{36}}{d_0}\right) + 12\left(\frac{d_{60}}{d_0}\right)$$

Figure 48. Equation. *AREA* via trapezoidal rule for SHRP sensor configuration.

The standard formulation of *AREA* was used by Ioannides, Barenberg, and Lary to develop the procedure based on the *AREA* method.(49)

Determining the Radius of Relative Stiffness

The *AREA* parameter provides a fairly good indication of relative stiffness of the pavement structure, particularly the bound layer(s), because it is largely insensitive to subgrade stiffness.(14) Ioannides, Barenberg, and Lary found a unique relationship between radius of relative stiffness and *AREA*, which is shown in figure 49.(49) A regression equation was also developed to define this relationship (see chapter 5).



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1 inch = 25.4 mm.

Figure 49. Graph. Variation in *AREA* with ℓ.(49)

For a dense liquid foundation, the radius of relative stiffness can be calculated as shown in figure 50, assuming PCC and subgrade properties are known.(49)

$$\ell = \left(\frac{Eh^3}{12(1-\mu^2)k} \right)^{1/4}$$

Figure 50. Equation. Radius of relative stiffness for dense liquid foundation.

Where:

ℓ = Radius of relative stiffness, mm (inches).
E = PCC elastic modulus, MPa (lbf/inch^2).
h = PCC thickness, mm (inches).
μ = PCC Poisson's ratio.
k = Modulus of subgrade reaction (k-value), MPa/mm (lb/inch^2/inch).

For an elastic solid foundation, the relationship shown in figure 51 exists.(49)

$$\ell = \left(\frac{Eh^3}{6(1-\mu^2)C} \right)^{1/3}$$

$$C = \frac{E_s}{1-\mu_s^2}$$

Figure 51. Equation. Radius of relative stiffness for elastic solid foundation.[(49)]

Where:

E_s = Subgrade modulus of elasticity, MPa (lbf/inch^2).
μ_s = Subgrade Poisson's ratio.

Estimation of Subgrade Support

According to Ioannides, Barenberg, and Lary, the k-value (for a dense liquid foundation) and C (for an elastic solid foundation) can be found knowing the load (P), radius of relative stiffness (ℓ), and measured deflections (d_r), as shown in figure 52.[(49)]

$$k = \frac{P\,d^*_r}{d_r\,\ell^2}$$

$$C = \frac{2P\,d^*_r}{d_r\,\ell}$$

Figure 52. Equation. Estimation of subgrade support for dense liquid and elastic solid foundations.[(49)]

Where:

k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2).
C = Subgrade constant, MPa (lbf/inch^2).
P = Applied load, N (lbf).
d^*_r = Nondimensional deflection coefficient for deflection at radial distance r from load.
d_r = Measured deflection at radial distance r from the load, mm (inches).
ℓ= Radius of relative stiffness, mm (inches).

Knowing the subgrade constant (C) and Poisson's ratio (μ_s), the subgrade modulus of elasticity (E_s) can be backcalculated from the second part of the equation in figure 51.

Backcalculation of PCC Slab Parameters

If the slab thickness is known, the PCC elastic modulus can be backcalculated from equations in figure 50 or figure 51 for a dense liquid or elastic solid foundation, respectively. Alternatively, if the PCC elastic modulus is known, the slab thickness can be computed. The flexural stiffness of the slab (D) is defined in figure 53:

$$D = \frac{Eh^3}{12\left(1-\mu^2\right)}$$

Figure 53. Equation. Flexural stiffness of the slab.

Using the radius of relative stiffness from the relationship in figure 53, figure 50 can be rearranged to solve for the PCC elastic modulus, as shown in figure 54.

$$E_{PCC} = \frac{12\,\ell^4\left(1-\mu^2\right)k}{h^3}$$

Figure 54. Equation. Computation of the PCC elastic modulus for the dense liquid foundation.

Where:

E_{PCC} = Elastic modulus of the slab, MPa (lbf/inch^2).
ℓ = Radius of relative stiffness, mm (inches).
μ = PCC Poisson's ratio.
k = Modulus of subgrade reaction, MPa/mm ($\text{lbf/inch}^2\text{/inch}$).
h = Slab thickness, mm (inches).

The principles of backcalculation described here were used to develop the ILLI-BACK computer program. The method was validated with field data, and the coefficient of variation varied between 1 and 3 percent.(49) This closed-form solution can also be programmed into a spreadsheet format.

Best-Fit Method

Another commonly used backcalculation procedure is the Best-Fit method. With this method, the PCC elastic modulus and subgrade *k*-value are found by identifying the best combination of the two parameters that produces a calculated deflection profile that best matches the measured profile.(51) This is performed through the minimization of the error function provided in figure 55.

$$F\left(E,k\right) = \sum_{i=0}^{n} \alpha_i \left(w(r_i) - W_i\right)^2$$

Figure 55. Equation. Minimization of the error function.

Where:

E = Elastic modulus of the slab, MPa (lbf/inch^2).
k = Modulus of subgrade reaction, MPa/mm ($\text{lbf/inch}^2\text{/inch}$).
α_i = Weighting factors.
$w(r_i)$ = Calculated deflection at sensor location i, mm (inches).
W_i = Measured deflection at sensor location i, mm (inches).

The calculated deflection (w_i) is based on Westergaard's solution for the interior loading of a plate consisting of a linear elastic, homogenous, and isotropic material on a dense liquid foundation. The ability to control the weights given to the various deflection measurements adds some flexibility to the Best-Fit process.(27)

Through a series of assumed conditions and substitutions (described more fully in chapter 5), the equation in figure 56 can be used to determine the radius of relative stiffness.

$$\frac{\sum_{i=0}^{n} \alpha_i f_i(\ell) f_i^{'}(\ell)}{\sum_{i=0}^{n} \alpha_i (f_i(\ell))^2} = \frac{\sum_{i=0}^{n} \alpha_i W_i f_i^{'}(\ell)}{\sum_{i=0}^{n} \alpha_i W_i f_i(\ell)}$$

Figure 56. Equation. Determination of radius of relative stiffness.

Where:

f_i and f'_i=Function of f, distance from the load, and parameters of applied load.

The radius of relative stiffness can then be used to determine the elastic modulus of the PCC slab using the equation in figure 54.

Likewise, for an elastic solid foundation, the PCC elastic modulus can be determined by the equation in figure 57.

$$E_{PCC} = \frac{6(1-\mu_{PCC}{}^2)\ell_e{}^3 E_s}{h_{PCC}{}^3 \left(1-\mu_s{}^2\right)}$$

Figure 57. Equation. Computation of the PCC elastic modulus for the elastic solid foundation.

Where:

E_{PCC} = Elastic modulus of the PCC slab, MPa (lbf/inch^2).
μ_{PCC} = PCC Poisson's ratio.
ℓ_e = $(D/C)^{1/3}$ = Radius of relative stiffness, mm (inches).
E_s = Modulus of elasticity of the subgrade, MPa (lbf/inch^2).
h_{PCC} = PCC slab thickness, mm (inches).
μ_s = Poisson's ratio of the subgrade.

A recent LTPP study compared results from the Best Fit and AREA methods of backcalculation for both dense liquid and elastic solid subgrade models.(27) Both the SHRP sensor configuration (seven sensors) and standard sensor configuration (four sensors spaced 305 mm (12 inches) apart) were also studied.

A critical assumption of plate theory is that there is no compression in the upper layer, such that the entire deflection is attributed to compression of the subgrade and bending of the plate.

Khazanovich, Tayabji, and Darter observed a greater discrepancy between the Best-Fit and AREA methods when the deflection directly under the load was used in backcalculation.[27] At that location, the deflections predicted by plate theory and layered elastic theory differ the most because of compression in the PCC pavement. Therefore, it is reasonable to suggest that deviation of the PCC slab behavior from the plate theory prediction is a significant source of discrepancy between different backcalculation methods.[27] Further investigation using the DIPLOMAT program confirmed this discrepancy between the two methods. These findings, coupled with previous work performed by Hall et al., led the researchers to recommend the Best-Fit method for backcalculation of rigid pavements.[51] The Best-Fit method yields a lower coefficient of variation in backcalculated *k*-values from multiple drops, is less sensitive to the randomness in measured maximum deflections, and provides better correspondence between measured and calculated deflection basins. It is also able to provide the best fit between calculated and measured deflections for any sensor configuration.

Dynamic Backcalculation of Rigid Pavements

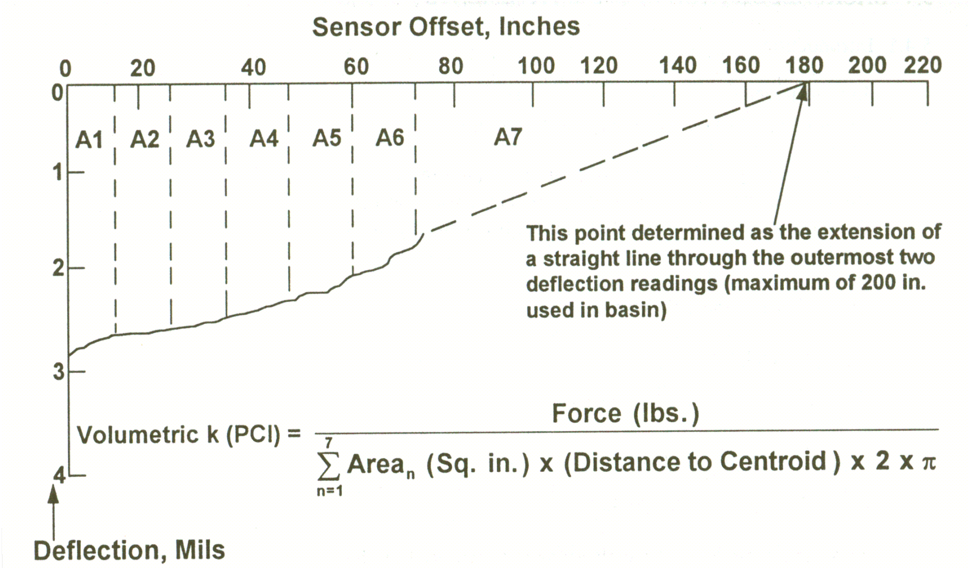
Historically, the pavement is assumed to exhibit a quasi-static behavior. However, some evidence negates this assumption because there is a lag between the peak of the applied load and among the peaks of the sensor deflections. This dynamic behavior may explain the differences between laboratory and backcalculated moduli as well as between moduli obtained at different times of the year. Recently, researchers have been investigating the dynamic response and developing procedures and programs for dynamic backcalculation; these programs are evaluated as part of this study. However, these programs must be acceptable for routine use and should be able to efficiently analyze significant quantities of FWD data.

Khazanovich presented a closed-form solution to describe the dynamic behavior of a linear elastic, homogeneous, and isotropic plate on a dense-liquid foundation based on Westergaard's solution for an interior loading.[52] The damping effects of the foundation are characterized by a damping parameter, and a dimensionless mass parameter is used to adjust the dynamic pavement response as a result of the effects of the inertia of the pavement. A backcalculation procedure can be developed based on this solution.

Likewise, Chatti and Kim developed the DYNABACK-R program for backcalculation of the dynamic subgrade stiffness and damping coefficients from the FWD deflection basin.[53] The program can also be used to detect a stiff layer beneath the pavement system.

To accomplish this effort, the transient deflection signal of each FWD sensor was first decomposed into a series of harmonic motions by the fast Fourier transform (FFT) algorithm. Then, for each frequency of interest, the real and imaginary components of the displaced volume underneath the slab were calculated from the complex deflection basin. The dynamic force-displacement relationship were decomposed into real and imaginary parts, leading to a simple system of equations that could be solved for the *k*-value and the radiation damping coefficient (*c*).[53] The *AREA* parameter was calculated using the formulation described in figure 45. To estimate the *k*-value, Chatti and Kim used the volumetric method illustrated in figure 58, where the *k*-value was determined by computing the volume of the deflection basin.[53,14]

The method was checked against field data obtained by the Michigan Department of Transportation. Good agreement between measured and predicted deflections was obtained at five of the six sites, with RMS values ranging from 2.6 to 9.8 percent.(53) The static backcalculation of the same deflection data yielded RMS values that varied between 2.5 and 10.8 percent, which suggests that dynamic backcalculation does not drastically improve consistency of the results.(53)



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$Area_n$ = Area of *n*th segment.
A1–A7 = Area segments.
PCI = Pounds per cubic inch.
1 inch = 25.4 mm.

Figure 58. Graph. Computing the *k*-value.(14)

BACKCALCULATION OF COMPOSITE PAVEMENTS

For the purposes of this study, "composite pavements" refer to HMA overlays on PCC pavements. Other types of composite pavement can generally be handled using the methods discussed for flexible or rigid pavements. For example, a PCC pavement on an HMA pavement can be analyzed as a rigid pavement on a stabilized base course. Likewise, an HMA pavement on a rubblized PCC pavement can be analyzed as a flexible pavement on a stiff base course.

Most distresses in composite pavements occur because of deterioration of the PCC pavement below the HMA overlay. Distresses most responsible for the PCC pavement deterioration are slab cracking, punchouts, joint deterioration, materials-related deterioration (such as D-cracking and alkali-silica reaction), and deterioration of patches. Deflection testing can be used to evaluate the condition of the PCC pavement not visible under the HMA overlay and to obtain the *k*-value below the surface.(14)

Although a composite (HMA-overlaid PCC (HMA/PCC)) pavement generally behaves as a rigid pavement, analysis is complicated by the effect of compression of the HMA layer directly under the load plate. Ignoring the effects of compression in the HMA layer can result in significant errors in the backcalculated moduli.(54) There are two approaches for addressing the compression under the load: (1) ignore the deflection directly under the load and base the calculations on the remaining deflections, or (2) subtract out the deflection under the load due directly to compression within the HMA layer and then use the same backcalculation procedure as for a bare PCC pavement.

The most commonly used methods for analyzing composite pavements are the AREA method and the Best-Fit method, which have already been discussed for rigid pavements. A slight adjustment to each method is made to account for compression in the HMA layer, as discussed in the following sections.

Outer-AREA Method

The outer-AREA method follows the same approach as the AREA method with an adjustment to minimize the compression effect in the HMA layer.(51) Using the outer-AREA method, the deflection directly under the load (d_0) is ignored, and remaining deflections are normalized to the deflection obtained 305 mm (12 inches) away from the load plate (d_{12}). Figure 59 is the equation to calculate outer-AREA for seven sensors spaced uniformly 305 mm (12 inches) apart.

$$Outer\ AREA = 6 \cdot \left(1 + 2 \cdot \frac{d_{24}}{d_{12}} + 2 \cdot \frac{d_{36}}{d_{12}} + 2 \cdot \frac{d_{48}}{d_{12}} + 2 \cdot \frac{d_{60}}{d_{12}} + \frac{d_{72}}{d_{12}} \right)$$

Figure 59. Equation. Outer-AREA method for backcalculation of composite pavements.

Where:

d_r = FWD deflections at distance r from the center of the load plate, mm (inches).

Beyond this calculation, the approach is the same as for the AREA method, although different coefficients are used for the determination of the radius of relative stiffness. The effective elastic modulus of the composite pavement (HMA and PCC layers) is then solved using the same equation, shown in figure 60.

$$E_e = \frac{12\ \ell^4 \left(1 - \mu^2\right) k}{h^3}$$

Figure 60. Equation. Effective elastic modulus of the composite pavement.

Where:

E_e = Effective elastic modulus of combined HMA and PCC layers, MPa (lbf/inch^2).
ℓ = Radius of relative stiffness, mm (inches).
μ = PCC Poisson's ratio.
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).
h = Slab thickness, mm (inches).

In this approach, the backcalculated elastic modulus represents the combined stiffness of the HMA and PCC layers. This effective modulus is converted to individual layer moduli using the approach outlined in chapter 5 for a PCC pavement on a stabilized base (equations for bonded and unbonded layers are presented).

Best-Fit Method

In this approach, the pavement layer moduli and subgrade k-value are estimated by finding the combination of material properties that provides the best match between the calculated and measured deflections. As noted for rigid pavements, the problem is formulated as a minimization of the error function, F, as shown in figure 61.[51]

$$F(E,k) = \sum_{i=0}^{n} \alpha_i (w_i - W_i)^2$$

Figure 61. Equation. Minimization of the error function.[51]

Where:

E = Effective elastic modulus of combined HMA and PCC layers, MPa (lbf/inch^2).
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).
α_i = Weighting factors.
w_i = Calculated deflection at sensor location i, mm (inches).
W_i = Measured deflection at sensor location i, mm (inches).

For the case of composite pavements, the weighting factor for the deflection directly under the load (α_0) is set to 0, thus excluding it from the calculation, while the weighting factor for the remaining sensors is set to 1. Again, the modulus obtained from this approach represents the effective pavement modulus of the HMA and PCC layers and must be converted to individual layer moduli.

Available Programs

It is difficult to achieve a solution when using programs based on multilayer elastic theory or iterative elastic layer backcalculation programs for composite pavements. These programs have difficulty with the upper layers being stiff when compared with the underlying materials, and they under-predict the modulus of the HMA surface while they overpredict the modulus of the PCC pavement.[14]

Anderson developed the computer program COMDEF that backcalculates moduli for composite pavements.[55] COMDEF backcalculates the moduli of a three-layer system consisting of an HMA layer, PCC layer, and a uniform subgrade. This program can only backcalculate moduli based on deflections measured by an FWD using seven sensors spaced 305 mm (12 inches) apart. It cannot accommodate fewer sensors or different spacing. The program uses precalculated solutions stored in database files to backcalculate the moduli, which are calculated using elastic layer theory. Interpolation techniques are used by COMDEF in the database of precalculated solutions to obtain deflections for cases not covered in the database. COMDEF uses 33 database files that contain deflections corresponding to fixed HMA layer thickness.[14] Two techniques are applied in COMDEF to backcalculate moduli: a stepwise direct optimization and an iterative relaxation technique using gradient matrices. An option allows the user to enforce to reasonable limits of the HMA modulus based on temperature.[14]

Several other computer programs can be used to backcalculate moduli for composite pavements, as listed in table 1. Some include using closed-form solutions for backcalculation of bare PCC pavements and adjusting the measured deflections to account for the influence of the HMA layer.[14]

OTHER USES OF FWD DATA

Load Transfer at Joints and Cracks

Nondestructive deflection testing can also be used to evaluate the load transfer at joints and cracks in rigid pavements. The test is conducted by applying a load, such as from an FWD, near the joint and measuring deflections on the loaded and unloaded slabs.

LTE is a measure of the percent of the load (or deflection) that is transferred across a joint or crack. The load transfer is most significantly affected by the following factors:

- **Aggregate interlock:** The interlocking of aggregate particles at the joint or crack interface helps transfer the load from the loaded side of the discontinuity to the unloaded side. The degree of aggregate interlock is based on factors such as the width of the crack opening, material strength, and the shape, size, and texture of the coarse aggregate particles. For example, larger aggregates with an angular, rough surface (such as crushed stone) generally provide better aggregate interlock than smaller aggregates with a rounded, smooth surface (such as a natural gravel).

- **Load transfer devices:** Load transfer devices, such as dowel bars, provide an effective means of transferring load across joints.

- **Underlying support conditions:** The stiffness of the underlying layer(s) also affects the amount of load transferred across a discontinuity. A stiff underlying layer (such as a stabilized base course) provides greater LTE than a less stiff layer (such as a gravel base or subgrade material).

- **Temperature**: Pavements expand and contract with changes in temperature. A joint or crack opens as the temperatures decrease and the pavement contracts, thereby reducing LTE provided by aggregate interlock. Conversely, a joint or crack closes as temperatures increase and the pavement expands, providing greater aggregate interlock load transfer.

LTE is calculated based on deflections of unloaded and loaded slabs using the equation in figure 62.

$$LTE = \beta \times \frac{d_u}{d_l} \times 100$$

$$\beta = \frac{d_{0center}}{d_{12center}}$$

Figure 62. Equation. LTE calculation.

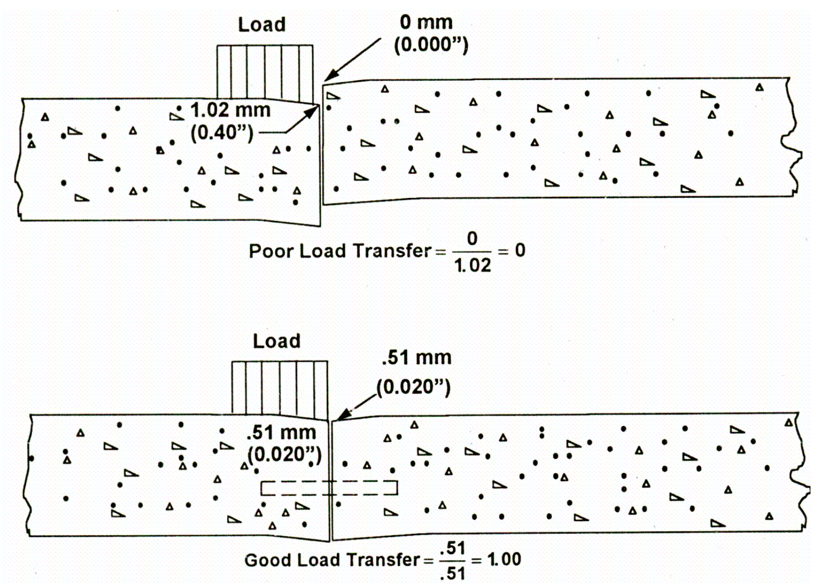
Where:

LTE = Load transfer efficiency, percent.
β = Slab bending correction factor.
d_u = Deflection on the unloaded slab, mm (inches).
d_l = Deflection on the loaded slab, mm (inches).

In theory, the slab bending correction factor (β) is necessary because the deflections d_0 and d_{12}, measured 305 mm (12 inches) apart, would not be equal even if measured in the interior of the slab. However, this correction factor is not widely used by researchers and practitioners.

The theoretical LTE ranges from 0 percent (no deflection on the unloaded slab) to 100 percent (equal deflections on the loaded and unloaded slabs). These two conditions are illustrated in figure 63.(14) Generally speaking, the following guidelines can be used to define different levels of LTE:(6)

- **Excellent**: 90–100 percent.
- **Good**: 75–89 percent.
- **Fair**: 50–74 percent.
- **Poor**: 25–49 percent.
- **Very Poor**: 0–24 percent.



1 mm = 0.039 inches.

Figure 63. Diagram. Comparison of examples of poor and good load transfer.(14)

Void Detection in PCC Pavements

Voids are generally created below slab corners due to pumping and erosion of subbase/subgrade material from repeated loading cycles.(14) Voids represent an area of poor support, which can lead to accelerated cracking and other distresses. Deflection testing can be used to detect the presence of voids by measuring deflections at the slab corner using a series of load levels. However, slab curling (due to temperature differences between the top and bottom of the slab) and warping (due to moisture level differences between the top and bottom of the slab) greatly affect the deflection of the slab during the testing; this concept is illustrated in figure 64 for slab curling.(14) Corner testing should be avoided when the slab is experiencing significant curling or warping because this can lead to misleading indications as to whether a void is actually present.

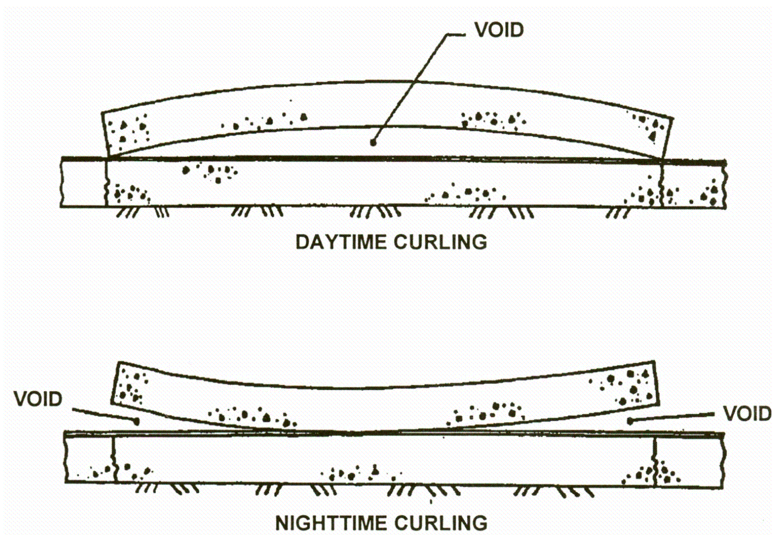
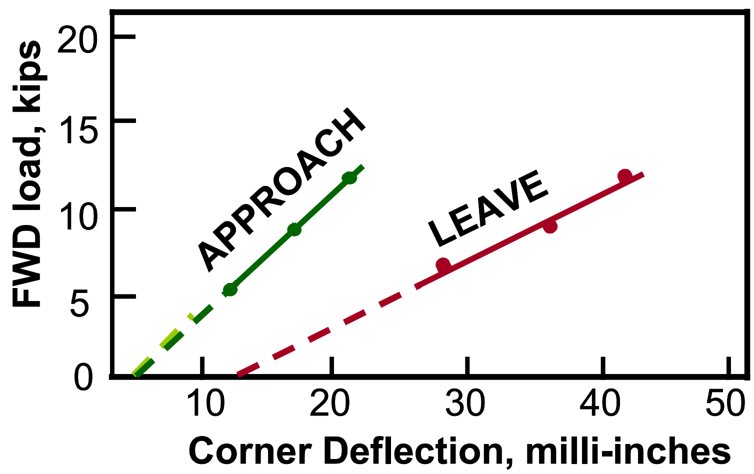


Figure 64. Diagram. Comparison of slab curling due to temperature differentials in the slab.(14)

There are a number of ways to detect the presence of voids beneath a slab corner. One of the simplest is the comparison of the magnitude of the corner deflection against a project average or pre-established threshold. However, a single maximum value used in the analysis of the corner deflection profile may not be appropriate if load transfer varies significantly from joint to joint. Because of this factor, as well as the influence of test temperature on the results, this method must be viewed as merely a general indicator of potential loss of support.

A more robust method of detecting voids is based on the analysis of corner deflections under variable loads.(56) In this method, corner deflections are measured at three load levels (such as 40, 53, and 67 kN (9,000, 12,000, and 15,000 lbf)). The results are plotted to establish a load-deflection relationship at each corner, as shown in figure 65. The figure illustrates that for the approach joint, the load-deflection line crosses the x-axis close to 0 at 0.051 mm (0.002 inches). For the leave joint, the load-deflection line crosses the deflection axis at a much greater distance away from the origin, indicating greater deflections under the same load. A line crossing the deflection axis at a point greater than 0.076 mm (3 mils) suggests the potential for a void under the slab.



1 kip = 0.45 metric tons

1 milli-inch = 0.0254 mm

Figure 65. Graph. Example void detection plot using FWD deflection data.[57]

Several mechanistic-based approaches have been developed to determine the presence of voids at slab corners. Shahin developed an approach to detect voids by comparing the measured corner deflections with theoretical corner deflections determined through finite element analysis (FEA), with the difference indicating a potential void.[58] Ullidtz presented a method to detect voids by comparing the modulus of subgrade reaction (*k*-value) at the corner of the slab to the *k*-value at the center of the slab, with poor corner support indicated when the *k*-value at the corner is between 60 and 80 percent of the *k*-value at the center of the slab.[33]

Rehabilitation Design and Structural Analysis

Nondestructive deflection testing is an integral part of the structural evaluation and development of rehabilitation designs for existing pavement structures. As previously described, deflection testing data can be used to determine the following pavement parameters:

- Elastic moduli of the pavement layers.
- Subgrade support conditions.
- Load transfer across joints and cracks.
- Void detection.

These properties can then be used to more accurately model the existing pavement structure and to develop better rehabilitation designs than by assuming typical material properties. In addition, FWD testing can help identify localized areas of weakness or changes in the pavement cross section or material properties, all of which lead to better designs and reduce the risk of premature failure, but without overdesigning the pavement structure.

The material properties obtained from backcalculation can also be used to more accurately model the pavement structure when analyzing its structural capacity or structural remaining life. When layer moduli are backcalculated from deflection data, stresses and strains under traffic can be determined, and the pavement's fatigue life can be estimated. Once the layer moduli are estimated, the threshold strain value is assigned to control fatigue, and the allowable number of load repetitions is determined from the previous equation.[59]

For inclusion as inputs for the MEPDG, nondestructive testing (NDT) results provide the current in situ layer properties for structural materials.[7] These data can prove to be invaluable in creating a better rehabilitation design rather than estimating material properties and possibly overdesigning the new pavement structure.

Effective Built-In Curl Analysis

Thermal curling and moisture warping can have a tremendous effect on the deflections and stresses in PCC pavements and the designs developed from these inputs. A portion of the slab curvature can be attributed to transient temperature and moisture gradients while the other contributing factors include the built-in temperature gradient and irreversible shrinkage. The latter component is referred to as the effective built-in temperature difference (EBITD). The magnitude of the EBITD is a function of the temperature gradient in the pavement at the time of concrete set, the ultimate drying shrinkage, and the creep characteristics of the concrete. A procedure is available to estimate its magnitude and effect, as described in the following paragraphs.[60,61]

In the procedure, the finite element program ILLI-SLAB is used to analyze the PCC pavement response to the combined effect of loading and temperature differential. The pavement layer and support conditions are first determined through backcalculation of FWD testing data obtained at midslab locations. A range of temperature differentials is then analyzed using ILLI-SLAB. The predicted deflection due to the FWD load is the difference between the loaded slab deflection (load and temperature) and the unloaded slab deflection (temperature only). From this analysis, a plot is developed showing the predicted deflections versus total effective linear temperature difference (TELTD) through the slab depth.

The interpolated deflections as measured in the field, along with the results of the FEA, are then used to estimate the TELTD for each load location. The EBITD is then the difference between the TELTD and the slab's measured temperature difference at the time of FWD testing.

Materials-Related Distress Assessment

Materials-related distress or durability problems in PCC pavements are often worse along the joints or at slab corners where moisture can more readily penetrate. Likewise, the deterioration can often be worse near the bottom of the slab compared with the slab surface. The latest FAA advisory circular on NDT includes an approach to assess the severity of materials-related distress below the surface or in a PCC pavement with an HMA overlay.[62] This approach, which is equally applicable to highway pavements, uses the ratio of the impulse stiffness modulus (ISM) at the slab center to the ISM at a joint or corner, where ISM is defined in figure 66.

$$ISM = \frac{P}{d_0}$$

Figure 66. Equation. ISM.

Where:

ISM = Impulse stiffness modulus, kN/mm (lbf/inch).
P = Applied load, kN (lbf).
d_0 = Maximum deflection under the load plate, mm (inches).

The impulse stiffness modulus ratio (ISM_{ratio}) is then defined in figure 67:

$$ISM_{ratio} = \frac{ISM_{slab\_center}}{ISM_{slab\_joint}} \quad or \quad ISM_{ratio} = \frac{ISM_{slab\_center}}{ISM_{slab\_corner}}$$

Figure 67. Equation. ISM ratio.

Where:

ISM_{ratio} = Impulse stiffness modulus ratio, kN/mm (lbf/inch).
ISM_{slab_center} = Impulse stiffness modulus at slab center, kN/mm (lbf/inch).
ISM_{slab_joint} = Impulse stiffness modulus at slab joint, kN/mm (lbf/inch).
ISM_{slab_corner} = Impulse stiffness modulus at slab corner, kN/mm (lbf/inch).

An ISM_{ratio} less than 1.5 indicates that the pavement is likely in good condition, with little or no materials-related distress at the joint or corner. (Note that the ratio will not be equal to 1 even for a slab in perfect condition because slab deflections are higher at a joint or corner compared with midslab.) An ISM_{ratio} between 1.5 and 3.0 signifies a pavement where the durability is questionable. Finally, an ISM_{ratio} greater than 3.0 indicates that the durability at the slab joint or corner is poor.

SUMMARY

The process to analyze deflection data is commonly referred to as "backcalculation." In the backcalculation process, pavement layer stiffnesses are determined based on the deflection data and the pavement cross section. Backcalculation derives its name from the fact that it solves for pavement layer properties by knowing the deflection response, which is the reverse of what is commonly done in analyzing a pavement structure using forward calculation. This chapter has presented an overview of backcalculation concepts for flexible, rigid, and composite pavements; a more detailed discussion of the available approaches and their applicability for use with the MEPDG is provided in chapters 4 through 6 for the three pavement types.(7)

This chapter also has discussed other potential uses and benefits of FWD testing and deflection data analysis. For example, FWD data can be used for analyzing the load-carrying capacity and structural remaining life of pavement structures, for determining LTEs, for evaluating the extent of materials-related distress, and for determining the presence of built-in curling.

CHAPTER 4. FWD DATA ANALYSIS AND INTERPRETATION—FLEXIBLE PAVEMENTS

This chapter discusses various approaches in FWD data analysis and interpretation for flexible pavements, including a review of common backcalculation programs in terms of their underlying assumptions, required inputs and resulting outputs, overall advantages and disadvantages, and user friendliness. Flexible pavement modeling issues, such as the effects of temperature and moisture, stiff layer, layer thicknesses; and nonlinear, viscoelastic, and dynamic material behavior, are also discussed. In addition, this chapter includes recommendations on the use of FWD data for mechanistic-empirical pavement design, along with suggestions for future research needs.

APPROACHES

There are various approaches for FWD data analysis and interpretation. These can be broken down according to (1) the methods of analysis for calculating pavement response (forward analysis) and (2) the methods for interpretation of pavement response (backcalculation). Each of these is described in the following sections.

Forward Analysis

Methods of calculating pavement response (forward calculation) include (1) closed-form solutions based on Boussinesq's original halfspace solution, (2) layered elastic solutions based on Burmister's original two- and three-layer solutions, and (3) finite element solutions.[63–65] Some of the common computer programs for flexible pavement analysis are presented in table 2, along with their basis and characteristics.

Table 2. Flexible pavement analysis programs.(66)

Software Name	Method Used in Response Model	Type[1]	Nonlinearity	Rheology	Anisotropy	Interface	Climate Effects	Dynamic Loading	Axle Spectrum	Tyre Characteristics	Stochastic	Crack Propagation	Thermal Effects	Cumulated Damage	Fatigue	Permanent Def.
APAS-WIN	Multilayer	3	—	—	—	—	Y	—	Y	Y	—	—	Y	—	Y	—
AXYDIN	Axi-symmetric FEM	1	—	—	—	—	—	Y	—	—	—	—	—	—	—	—
BISAR/SPDM	Multilayer	3	—	—	—	Y	Y	—	Y	—	—	—	—	—	Y	Y
CIRCLY	Multilayer	3	—	—	Y	Y	—	—	Y	Y	—	—	—	Y	Y	—
CAPA-3D	3D-FEM	3	Y	Y	Y	Y	—	Y	—	Y	—	Y	Y	Y	Y	Y
CESAR	3D-FEM	3	Y	Y	Y	Y	Y	Y	—	Y	Y	Y	—	Y	Y	Y
ECOROUTE[2]	Multilayer	1	—	—	—	Y	—	—	—	Y	—	—	—	Y	—	—
ELSYM 5	Multilayer	1	—	—	—	—	—	—	—	—	—	—	—	—	—	—
KENLAYER	Multilayer	2	Y	Y	—	Y	—	Y	—	Y	—	—	—	Y	Y	Y
MICHPAVE	Axi-symmetric FEM	1	Y	—	—	—	—	—	—	—	—	—	—	—	Y	—
MMOPP	Multilayer	2	Y	—	—	—	Y	Y	Y	Y	Y	Y	—	Y	Y	Y
NOAH	Multilayer	3	—	—	Y	Y	Y		Y		Y	—	—	—	Y	Y
ROADENT/ WESLEA[3]	Multilayer	2	—	—	—	Y	Y	—	Y	Y	—	—	—	—	—	—
SYSTUS	3D-FEM	2	Y	Y	Y	Y	—	Y	—	Y	—	Y	—	—	—	—
VAGDIM 95	Multilayer	3	—	—	—	—	Y	—	—	—	—	—	Y	Y	Y	Y
VEROAD	Multilayer	1	—	Y	—	—	—	—	Y	Y	Y	—	—	—	—	—
VESYS	Multilayer	3	—	—	—	—	Y	—	Y	Y	Y	—	—	Y	Y	Y

Def. = Deformation.
Y = Yes.
— Indicates properties not considered in the program.
[1]Type: 1 = Response only, 2 = Response + Partial Performance, and 3 = Full Design Procedure. See section 2.4 of reference 66.
[2]Model was not used in the project because of lack of availability.
[3]Only the response model WESLEA was evaluated in the project.

MET

The most useful solution in the first category is MET, which is based on Odemark's assumptions.(33) In this method, the deflections of a multilayered pavement system with moduli E_i and layer thicknesses h_i can be obtained using a single layer of thickness H and modulus E provided that H satisfies the equation in figure 68.

$$H = \sum_{i}^{n} C h_i \left(\frac{E_i}{E}\right)^{1/3} \qquad C = 0.8 \text{ to } 0.9$$

Figure 68. Equation. Odemark transformation.

This method is reported to produce results that are as good as or better than those from layered elastic and finite element solutions, when compared with measured data.(33,34) The method can also be adapted to handle nonlinear subgrade materials. Another clear advantage of this method is its simplicity, which leads to significant savings in computation time, a feature that is particularly useful in backcalculation because many iterations are usually required.

Layered Elastic Solutions

The layered elastic solutions are by far the most commonly used among all methods. They are generally restricted to linear elastostatic analysis and have been shown to produce good results if material behavior remains in the linear range. Numerous pavement-specific computer programs (such as CHEVRON, ELSYM5, and BISAR) have been developed using the *n*-layer solution provided by Schiffman.[35] The basic assumptions in this category of solutions are the following:

- Surface load is uniformly distributed over a circular area.
- All layers are homogeneous, isotropic, and linearly elastic.
- Upper layers extend horizontally to infinity.
- Bottom layer is a semi-infinite halfspace.

Some of the available computer programs do allow for nonlinear response (e.g., NELAPAV, PADAL) or viscoelastic response (e.g., VESYS, KENPAVE).

FEA Method

A number of computer programs based on FEA have also been used for pavement analysis, including three-dimensional (3-D) general purpose (structural mechanics) programs such as SAP®, ABAQUS®, and ANSYS®, and pavement-specific programs for two-dimensional axisymmetric (e.g., ILLIPAVE, MICHPAVE) and 3-D solutions (CAPA-3D). The main advantage of using FEA is the ability to handle material variability and nonlinearity in both vertical and horizontal directions, and to include any number of sophisticated constitutive models. However, most of these programs involve a large number of elements and input parameters, and therefore are much more time-consuming to set up and to run. Hence, their use has been limited mainly to the pavement research community, although there has been a growing interest in their use among pavement engineers in recent years.

Dynamic Solutions

Computer programs for dynamic analysis of pavement systems use either dynamic damped-elastic finite-layer or finite element models for their forward solutions. The finite layer solutions are based on Kausel's formulation, which subdivides the medium into discrete layers that have a linear displacement function in the vertical direction and satisfy the wave equation in the horizontal direction.[67] Examples of programs containing such solutions include UTDYNAF, UTFWD, GREEN, and SAPSI. (See references 68 through 71.) The computer program SCALPOT models the asphalt concrete layer as a viscoelastic material using a two-parameter power law model, while the SAPSI program allows the layer material properties to be complex and frequency-dependent.[72] Al-Khoury et al. developed an efficient forward solution for the dynamic analysis of flexible pavements using the spectral element technique for the simulation of wave propagation in layered systems.[73] The method can model each layer as one element without the need for subdivision into several sublayers.

Backcalculation Analysis

Existing backcalculation routines can be categorized in three major groups, depending on the techniques used to reach the solution. These three techniques can be applied to any of the forward analysis methods previously discussed. The first group is based on iteration techniques, which repeatedly use a forward analysis method in an iterative process. The layer moduli are repeatedly adjusted until a suitable match between the calculated and measured deflection basins is obtained. The second group is based on searching a database of deflection basins. A forward calculating scheme is used to generate a database, which is then searched to find a best match for the observed deflection basin. The third group is based on the use of regression equations fitted to a database of deflection basins generated by a forward calculation scheme. Each of these schemes is described in the following sections.

Closed-Form Method

The original ELMOD3 program uses the Odemark-Boussinesq method of equivalent layer thickness concept and the radius of curvature method.(32) Initially, the subgrade material properties, stiffness and nonlinearity, are calculated using the deflections from the outer sensors. The radius of curvature from the central sensors can be used to assess the stiffness of the upper pavement layer. The stiffness of remaining layers is then calculated based on the overall pavement response to the applied load. This ensures that the proposed pavement structure results in the correct central deflection under the measured load.

The surface modulus concept is very useful for estimating the subgrade modulus and for diagnosing stress-sensitive subgrade material and the presence of stiffer layers. It uses Boussinesq's original closed-form equations (see figure 69 and figure 70.) relating the vertical deflection on the surface of a homogeneous, isotropic, linearly elastic halfspace δ_Z and the elastic (Young's) modulus E as follows:

$$\delta_z = \frac{2P}{\pi E a}(1-\mu^2)$$

Figure 69. Equation. Deflection for uniformly distributed load.

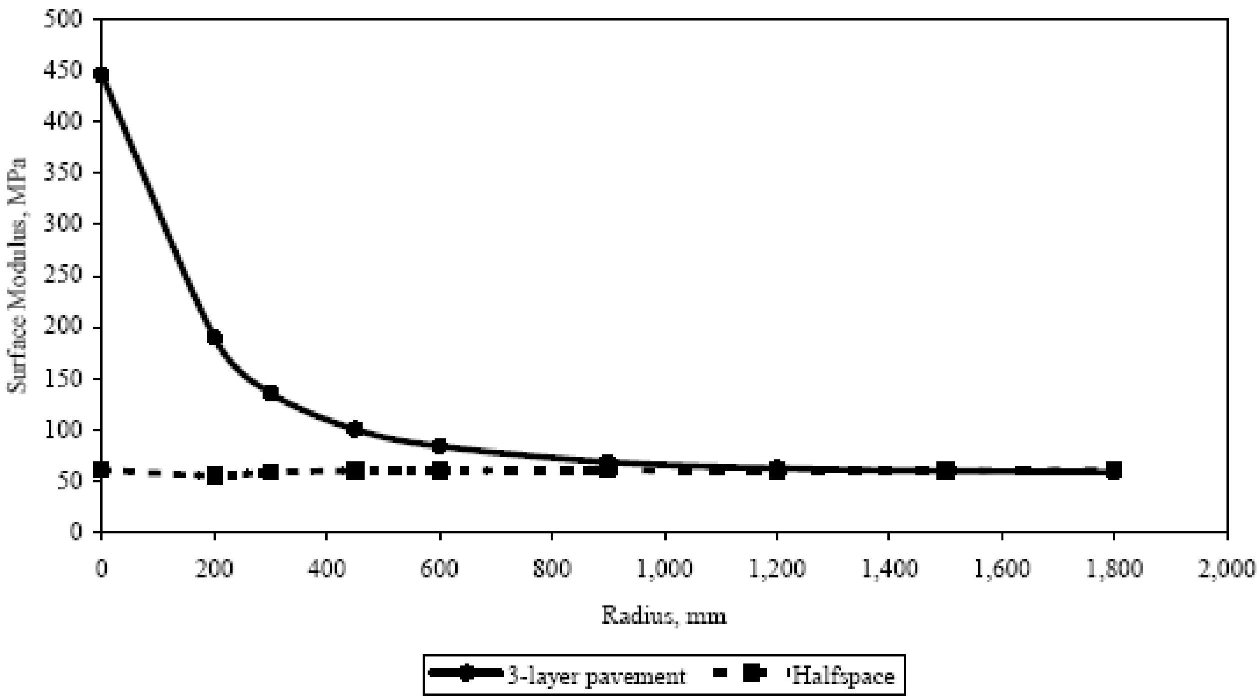
$$\delta_z = \frac{P}{\pi E r}(1-\mu^2)$$

Figure 70. Equation. Deflection for point load.

Where:

P = Surface load (force).
r = Radial distance from center of load.
a = Radius of loaded area.
μ = Poisson's ratio.

These equations can be used directly to backcalculate the surface modulus E_o given a measured surface deflection due to a known load. For a multilayered pavement system, the calculated surface modulus at the center underestimates the modulus of the surface layer, and it overestimates the modulus of the subgrade. However, as illustrated in figure 71, the surface modulus approaches a constant value at large radii; this value corresponds to the subgrade modulus, illustrating a basic principle of backcalculation, which is that the outer deflections can be used to determine the moduli of the deeper layers. If the surface modulus increases with increasing radial distance, then it is an indication of a stress-sensitive (nonlinear) subgrade or the presence of a stiff layer.(33) The effect of Poisson's ratio is relatively small for most typical values.



By permission of D.P. Orr, Director, Cornell Local Roads Program
1 mm = 0.039 inches.
1 MPa = 145 lbf/inch^2.

Figure 71. Graph. Surface modulus for a three-layer pavement and a halfspace.(35)

Iterative Deflection Basin Fit Methods

In this approach, layer moduli are repeatedly changed until the calculated deflection basin matches the measured one (see figure 72) within a specified tolerance. The flowchart in figure 38 (see chapter 3) illustrates this process.

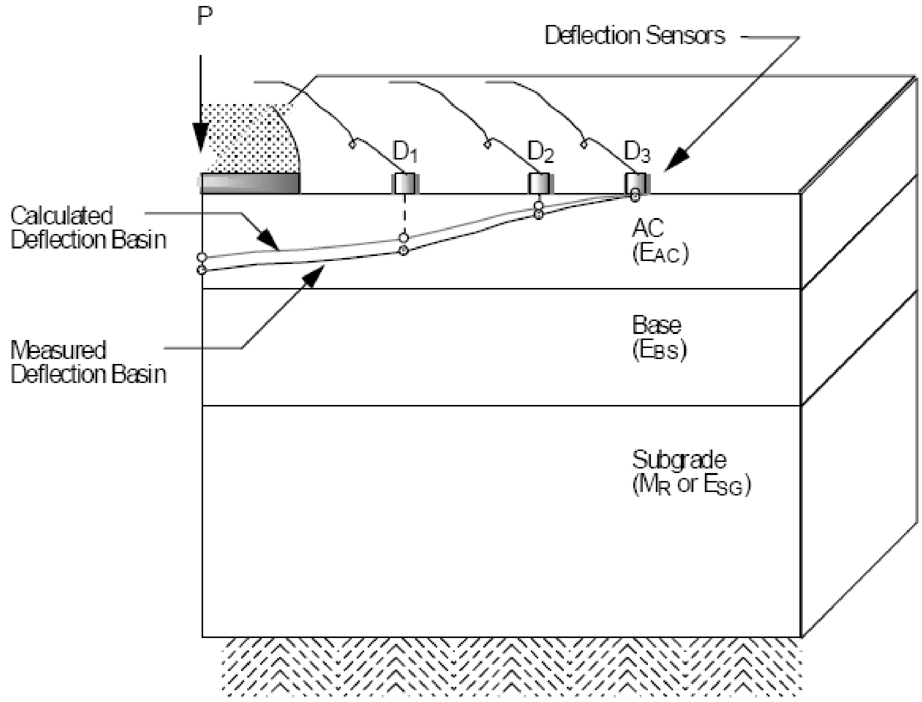


Figure 72. Diagram. Matching measured and calculated deflection basins.[(40)]

The main steps of the iteration process are as follows:[(37)]

- **Step 1:** Measure surface deflections at known radial distances from the center of the loaded area.

- **Step 2:** Enter layer thicknesses, load application characteristics, and Poisson's ratios for each layer.

- **Step 3:** Start the forward calculation process using initially assumed layer modulus values (seed moduli), which are required as input. Seed moduli are sometimes generated by the program using measured deflections and regression equations; otherwise, the user must specify them. At this stage, some programs use a database approach to obtain seed moduli.

- **Step 4:** Use data specified in step 2 and the latest set of layer moduli to calculate surface deflections at the same radial offsets at which the deflections were measured.

- **Step 5:** Perform an error check to assess whether the measured and calculated surface deflections are within specified tolerance limits. At this stage, different techniques are used to adjust the set of layer moduli so the new set of moduli reduces the error quantified by the objective function. The method by which the moduli are adjusted is the main differentiating factor between most iterative-based programs. Repeat steps 4 and 5 until the value of the objective function is sufficiently small or the adjustments to the layer moduli are very small.

The search algorithm is usually achieved by minimizing an objective function of any set of independent variables (i.e., layer moduli, thicknesses, etc.), which is commonly defined as the weighted sum of squares of the differences between calculated and measured surface deflections, shown in figure 73:

$$f = \sum_{j=1}^{m} a_j (w_{jm} - w_{jc})^2$$

Figure 73. Equation. Objective function to be minimized in the search algorithm for minimizing differences between measured and computed deflections.

Where:

w_{jm} = Measured deflection at sensor j.
w_{jc} = Calculated deflection at sensor j.
a_j = Weighing factor for sensor j.

The equation in figure 73 can be solved using nonlinear optimization methods, which locate the least value of the objective function. Many minimization techniques are available in the literature, including the following:(42)

- Factored secant update method.
- Modified Levenberg-Marquardt algorithm.
- Modified Powell hybrid algorithm.

One of the problems of this approach is that the multidimensional surface represented by the objective function may have many local minima. As a result, the program may converge to different solutions for different sets of seed moduli. Another problem is that the convergence can be very slow, requiring numerous calls to a mechanistic analysis program.

An example of an iterative program is EVERCALC©, which uses the Levenberg-Marquardt minimization algorithm.(43) The program seeks to minimize an objective function formed as the sum of squared relative differences between the calculated and measured surface deflections. EVERCALC© is a robust, efficient, and accurate program, and uses the CHEVRON computer program for forward calculations.

The series of programs with names ending in "DEF" use an assumed linear variation in logarithmic space between layer moduli and surface deflections to revise the layer moduli after each iteration. These programs employ a gradient search technique, and the "correct" set of moduli is searched in an iterative manner. The CHEVDEF program is one such example in which the CHEVRON program is used for forward calculations. A set of seed moduli are required to be entered by the user in this program to start the iteration process.(44,74) The linearization of the model in logarithmic space simplifies the search for a new set of moduli. However, the results obtained by these programs are highly dependent on the initial seed moduli.

In addition to the original radius of curvature method, the newer ELMOD5 program has options where the deflection basins (calculated using the Odemark-Boussinesq MET, WESLAYER, or finite element method) are matched iteratively with the convergence criteria based on the degree of fit between the measured and calculated deflection basins.

The search method can also take the form of solving the linear set of equations shown in figure 74.

$$[F]^k \{d\}^k = \{r\}^k$$

Figure 74. Equation. Deflection basins matched iteratively with the convergence criteria.

Where:

$[F]^k$ = kth iteration of the m x n matrix of partial derivatives $\partial fj/\partial E_i$, where j = 1 to m and m is the number of deflections measured, and i = 1 to n where n is the number of layers in the pavement.
$\{d\}^k$ = kth iteration difference vector, $E_i^{k+1} - E_i^k$, between the new and old moduli.
$\{r\}^k$ = kth iteration residual vector, $w_{jc} - w_{jm}$, between the most recently calculated and the measured surface deflections.

An example of an iterative program using the above search method is MICHBACK©, which uses the modified Newton-Raphson (also called secant) method. The method of least-squares is used to solve the overdetermined system of equations (m equations in n unknowns, $m > n$) in the figure 74 equation. If desired, weighting factors can be used for each sensor measurement to emphasize some deflection measurements over others. MICHBACK© also uses the CHEVRONX computer program for forward calculations.

Database Approach

In this method, a forward calculation program is used to generate a database of deflection basins for different combinations of layer moduli, specified layer thicknesses, material properties, pavement types, and loading conditions. The measured deflection basin is compared with the deflection basins in the database using a search algorithm, and a set of moduli are interpolated from the layer moduli that produces the closest calculated deflection basins in the database.

The MODULUS backcalculation program, which uses databases generated by the WESLEA program, is one such example.(36,37) The number of basins required to obtain a suitable database depends on the number of layers and the expected moduli ranges provided by the user. Wide ranges require generation of a greater number of basins than narrow ones. The generated deflection basins are then searched using the Hookes-Jeeves algorithm, and a three-point Lagrangian method is used to interpolate the values of the moduli between the various deflection basins. The program seeks to obtain a set of moduli that minimize an objective function defined as the relative sum of squared differences between the measured and calculated surface deflections. The program always converges, although the chances of converging to a local minimum cannot be ruled out.(38) The program performs a convexity test to determine the likelihood of having converged to a local minimum, and the user is warned if this test is not satisfied.

Backcalculation based on a database search is especially suitable when a large number of pavements with a similar configuration need to be tested in succession. For these situations the database, once generated, can be used repeatedly to backcalculate values of the modulus of the various pavement layer for all similar pavements, and the time required to generate the database can be minimized. This technique can be used with a database generated from any linear or nonlinear program.[38]

Statistical Analysis

This method is similar to the database approach, the only difference being how the database is used. The database is created by using any forward calculation routine, and then statistical analysis is performed to generate regression equations. These equations use the deflections as independent variables and attempt to predict the values of the layer moduli. Pavements of different configurations can be grouped separately to yield different equations for more accurate predictions. Different prediction equations are required for each pavement layer, and pavements with a different number of layers must be treated separately.

This technique is best suited for agencies that deal with limited and known types and configurations of pavement. Database generation to include all the expected combinations of pavement layers in the initial stages can offset this disadvantage to a large extent. Once the regression equations are obtained, this technique is simple and extremely quick. The results, on the other hand, vary in accuracy depending on how well the database used to generate the statistical equations represents the pavement being analyzed.

Dynamic Backcalculation Methods

Most dynamic backcalculation methods use dynamic, damped-elastic finite-layer or finite element models for their forward solutions, as previously discussed. Dynamic backcalculation methods are based on either frequency or time domain solutions. For the former procedure, the applied load and measured deflection time histories are transformed into the frequency domain by using FFT. Backcalculation of layer parameters is done by matching the calculated steady-state (complex) deflection basin with the frequency component of the measured sensor deflections at one or more frequencies. In time domain backcalculation, the measured deflection time histories are directly compared with the predicted results from the forward program. One of the advantages of this method is that matching can be achieved for any time interval desired. Uzan compared the two methods and concluded that time domain backcalculation was preferred over frequency domain backcalculation.[75]

BACKCALCULATION COMPUTER PROGRAMS

Static Backcalculation Programs

Numerous computer programs for performing automated backcalculation have been written. Some of the known static backcalculation computer programs and their characteristics are presented in table 3. Different versions of these programs exist, with improved and/or updated editions being released periodically. Most of the automated backcalculation programs rely on static analysis and a linear elastic layer program. Notable exceptions include ELMOD®, which can use either Odemark's method or the finite element method in addition to the layered elastic solution, and MODCOMP©, which can handle nonlinear material properties.

Table 3. Commonly available backcalculation computer programs for flexible pavements.

Program Name	Developer	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Method	Nonlinear Analysis	Layer Interface Analysis	Maximum Number of Layers	Seed Moduli	Range of Acceptable Modulus	Ability to Fix Modulus	Convergence Scheme	Error Weighting Function
BISDEF©	USACE-WES	Multilayer Elastic Theory	BISAR	Iterative	No	Variable	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
BOUSDEF	Zhou et al. (Oregon State)	MET	MET	Iterative	Yes	Fixed (rough)	At least 4	Required	Required	N/A	Sum of percent errors	N/A
CHEVDEF	USACE-WES	Multilayer Elastic Theory	CHEVRON	Iterative	No	Fixed (rough)	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
COMDEF	USACE-WES	Multilayer Elastic Theory	BISAR	Database	No	Fixed (rough)	3	No	No	N/A	Various	No
DBCONPAS	Tia et al. (University of Florida)	Finite Element	FEACONS III	Database	Yes?	Yes?	N/A	No	No	N/A	NA	N/A
ELMOD®/ ELCON	Ullidtz (Dynatest®)	MET	MET	Iterative	Yes (subgrade only)	Fixed (rough)	4 (exclusive of rigid layer)	No	No	Yes	Relative error of 5 sensors	No
ELSDEF	Texas A&M, USACE-WES	Multilayer Elastic Theory	ELSYM5	Iterative	No	Fixed (rough)	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of squares of absolute error	Yes
EMOD	PCS/Law	Multilayer Elastic Theory	CHEVRON	Iterative	Yes (subgrade only)	Fixed (rough)	3	Required	Required	Yes	Sum of Relative squared error	No
EVERCALC©	Mahoney et al.	Multilayer Elastic Theory	CHEVRON	Iterative	Yes	Fixed (rough)	3 (exclusive of rigid layer)	Required	Required	Yes	Sum of absolute error	No
FPEDD1	Uddin	Multilayer Elastic Theory	BASINF?	Iterative	Yes	Fixed (rough)	N/A	Program Generated	NA	N/A	NA	No
ISSEM4	Ullidtz, Stubstad	Multilayer Elastic Theory	ELSYM5	Iterative	Yes (finite cylinder concept)	Fixed (rough)	4	Required	Required	Yes	Relative deflection error	No

Program Name	Developer	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Method	Nonlinear Analysis	Layer Interface Analysis	Maximum Number of Layers	Seed Moduli	Range of Acceptable Modulus	Ability to Fix Modulus	Convergence Scheme	Error Weighting Function
MICHBACK©	Harichandran et al.	Multilayer Elastic Theory	CHEVRONX	Newton method	No	Fixed (rough)	Number of deflections; best for 3 unknowns	Required	Required	Yes	Sum of relative squared error	NA
MODCOMP5	Irwin, Szebenyi	Multilayer Elastic Theory	CHEVRON	Iterative	Yes	Fixed (rough)	2 to 15 layers; maximum of 5 unknown layers	Required	Required	Yes	Relative deflection error at sensors	No
MODULUS	Texas Transportation Institute	Multilayer Elastic Theory	WESLEA	Database	Yes?	Fixed?	4 unknowns plus stiff layer	Required	Required	Yes	Sum of relative squared error	Yes
PADAL	Brown et al.	Multilayer Elastic Theory	n/a	Iterative	Yes (subgrade only)	Fixed ?	n/a	Required	N/A	N/A	Sum of relative squared error	N/A
RPEDD1	Uddin	Multilayer Elastic Theory	BASINR ?	Iterative	Yes	Fixed ?	n/a	Program Generated	N/A	N/A	n/a	No
WESDEF	USACE-WES	Multilayer Elastic Theory	WESLEA	Iterative	No	Variable	5	Required	Required	Yes	Sum of squares of absolute error	Yes

N/A = Not applicable.

Accuracy and Reliability

Many of the programs written for production purposes are intended to get to an accurate solution reliably. While most static backcalculation programs usually converge to a solution reasonably quickly and reliably, one cannot assert the uniqueness of the set of layer moduli derived from any search method. For this reason, many programs use various controls to guide the iterative search toward an "acceptable" set of layer moduli. These include (1) making some assumptions about the type of pavement system being analyzed (e.g., assuming that layer moduli decrease with depth, that the subgrade modulus is constant with depth, that a rigid layer exists a certain depth, and so on), and (2) limiting the acceptable range of moduli for each individual layer type.

Required Inputs

Required inputs typically include peak sensor deflections and their location, peak load values, the number of layers in the pavement system and their thicknesses, and assumed values for Poisson's ratios. Most programs also require seed moduli as input, although some have methods that generate these from the measured deflections or from regression equations.

Resulting Outputs

Typical outputs include the measured and calculated deflections, the differences and percent differences, the final set of layer moduli, and the error sums. Most of the existing backcalculation programs allow for 3 to 5 layers; a notable exception is the MODCOMP5 program, which allows up to 15 layers (with a maximum of 5 unknown layers).

User Friendliness

Because many of the backcalculation programs are written for production purposes, they are user friendly, require minimum involvement from the user, and provide various features intended to be useful for project-level analysis. Conversely, those programs written for use in research tend to lack the features needed for production. They also usually allow and require significant involvement from the user. These include dynamic backcalculation programs that rely on dynamic analysis to calculate the deflection time histories, and those that use available, general-use finite element method programs.

Advantages and Disadvantages

Attempting to do a one-on-one comparison of different backcalculation programs for the purpose of identifying the best one is a difficult task. All of these programs have different pros and cons, and each may be particularly useful in a specific situation. Before making such comparisons, one should first define the purpose in doing backcalculation and the evaluation criteria that one will use. In general, the advantage of using simpler methods is that they are very fast and easy to use. Their disadvantage is that they are limited in their interpretation of the FWD data. For example, most static backcalculation programs are limited to three layers. This may not be sufficient to characterize realistic pavement profiles, which may comprise five or more layers and cannot be used to allow for variation of subgrade modulus with depth, for example. On the other hand, more advanced methods of backcalculation, which will theoretically allow for the backcalculation of a larger number of parameters, are computationally expensive and time consuming. Also, they are not guaranteed to converge when using real field-measured data.

Dynamic Backcalculation Programs

A number of computer programs have been developed for dynamic backcalculation of flexible pavement layer parameters. Each program employs a particular forward model and a specific backcalculation scheme. All of these programs require the time histories of the load and deflection sensors. Theoretically, because these time histories contain more information than just the peak values of load and deflection, dynamic backcalculation programs can backcalculate a larger number of parameters when using synthetically generated deflection time histories. However, there are serious challenges when using measured field data. For example, the frequency-domain solutions can lead to large errors if the measured FWD records are truncated before the motions fully decay in time.

Time-domain backcalculation solutions present another set of challenges. For example, the time synchronization between the load and sensor records and the digitization of the response can be problematic. Noise in the data and the ill-posed nature of the inversion problem can be amplified when matching traces of time histories, requiring special filtering and regularization techniques that are not easy to implement. In addition, unlike frequency-domain analysis where the properties are backcalculated at each frequency independently, time-domain backcalculation precludes making a choice about the behavior of material properties with frequency; that is, it either assumes a constant HMA modulus (similar to static backcalculation) or a prescribed function of the HMA layer modulus with frequency (e.g., linear relation in the log-log space). While this assumption may be acceptable for unbound materials, it may significantly affect the predicted response of the HMA layer because of its viscoelastic nature. Finally, none of these programs are considered ready for production mode because they usually require a lot of involvement from the user, are computationally very expensive, and have not been fully evaluated for use with field-measured data. Some of the dynamic backcalculation computer programs and their characteristics are presented in table 4. A brief overview of the programs developed to date is also provided.

Table 4. Dynamic backcalculation programs for flexible pavements.

Program	Domain	Inverse Method	Forward Program	Reference
BKGREEN	Frequency	Nonlinear least-square optimization	GREEN	70
No name	Frequency/ Time	Newton's method	UTFWIBM	76
PAVE-SID	Frequency	System Identification (SID)	SCALPOT	72
FEDPAN	Time	Linear least squares	SAP IV	77
No name[1]	Frequency	Levenberg-Marquardt	SAPSI	78
No name[1]	Frequency	Secant Update Levenberg-Marquardt Powell Hybrid	LAMDA	79
No name[1]	Time	Gauss-Newton method	FEM	80
DYNABACK	Frequency/ Time	Newton's method with least-square or SVD method	SAPSI	81, 82, 46
EVERCALCII	Time	Nonlinear least square optimization with Tikhonov regularization and continuation method	FEM	83

[1]Program was developed for internal study and was never assigned a label.

Uzan presented two dynamic linear backcalculation procedures, one in the time domain and the other in the frequency domain.(76) Both approaches use the program UTFWIBM as the forward model and Newton's method as the backcalculation solution.

PAVE-SID is a computer program that uses the SCALPOT program to generate frequency response curves; a system identification technique is applied for matching computed frequency data to extract pavement properties.(72) SCALPOT computes the dynamic response of a horizontally layered viscoelastic halfspace to a time dependent surface pressure distribution.

BKGREEN models the pavement as a layered elastic system in terms of dynamic Green flexibility influence functions using Kausel's formulation of discrete Green functions for dynamic loads in linear viscoelastic layered media.(70,67) Backcalculation is done at multiple frequencies, and the set of layer moduli is determined using a nonlinear least squares technique. The calculation can experience some computational difficulties at certain frequencies because of the numerical complications associated with implementing infinite integration in computer codes.

Al-Khoury et al. developed an axisymmetric layered solution as a forward model using the spectral element technique, and used the modified Levenberg-Marquardt and Powell hybrid methods for solving the resulting system of nonlinear equations.(73,79,84)

Losa used SAPSI as the forward program and a nonlinear least squares optimization technique (Levenberg-Marquardt method) for multifrequency backcalculation.(78,71) The HMA and subgrade materials were assumed to be frequency dependent while the base/subbase material was assumed to be frequency independent.

FEDPAN is a finite element program that can perform both static and dynamic backcalculation for three-layer pavement systems using the CHEVDEF backcalculation algorithm.(77,44) This program can simulate the effects of pavement inertia and damping in the dynamic analysis, and material nonlinearity in the static analysis.

Meier and Rix developed an ANN solution that has been trained to backcalculate pavement layer moduli for three-layer flexible pavement systems using synthetic dynamic deflection basins.[47] The dynamic pavement response was calculated using an elastodynamic Green function solution based on Kausel's formulation.[67]

Work by Chatti developed the DYNABACK computer program, which allows for different dynamic backcalculation algorithms for both frequency-based and time-based solutions.[81,82,46] The DYNABACK program uses the SAPSI program as its forward solution and an expanded version of the modified Newton-Raphson algorithm in the MICHBACK© program as its backcalculation solution.[71,85] The solution uses the least squares minimization technique to solve the real-valued gradient matrix equation. The DYNABACK program includes two basic solutions with several options for backcalculating different layer parameters: (1) frequency-domain backcalculation at one or multiple frequencies and (2) time-domain backcalculation using peak responses or time history traces. Theoretically, single frequency backcalculation can be used to backcalculate up to 8 parameters while multiple frequency backcalculation can be used to backcalculate up to 15 parameters. The same is true for time domain backcalculation using peak responses and traces, respectively. However, when using measured deflection time histories, the number of backcalculated parameters must be reduced to fewer than eight.

Finally, Turkiyyah has been developing an improved EVERCALCII program that uses the complete FWD sensor time histories to recover pavement layer moduli distribution and thicknesses using thin "computational layers" that discretize the profile.[83] In this solution, physical layer thicknesses may be obtained, after backcalculation of thin computational layer moduli, by grouping thin layers of similar moduli values. Two regularization techniques are employed: one involves the absolute values of the moduli to prevent physically unrealistic solutions with large layer moduli, while the second controls the gradient of the moduli in the vertical direction to prevent convergence to profiles with neighboring layers that alternate between high and low moduli. In addition, a continuation scheme is used to control the weights on the regularization terms to overcome the ill-posed nature of the optimization problem. Because this solution relies on backcalculating the moduli of a relatively large number of elements that make up these thin computational layers, the computational effort for solving the inverse problem is very significant. Efforts are underway to speed up the forward (finite element method) solution.

DATA-RELATED ISSUES

The primary issues related to FWD data analysis and interpretation are (1) errors in measurement (relevant to static and dynamic backcalculation) and (2) signal noise and truncation (relevant for dynamic analysis only). These topics are discussed in the following sections.

FWD Data Errors

According to Irwin, there are three main sources of errors in FWD data: (1) seating errors, (2) random errors, and (3) systematic errors.[35] Irwin, Yang, and Stubstad showed that even very small deflection errors (on the order of 2 μm or less) can lead to very large errors in the backcalculated moduli.[86]

Seating Errors

Seating errors are caused by the rough texture of pavements and are of more critical importance in testing HMA pavements. These errors can be eliminated by applying one or two drops at each new test point. This causes the deflection sensors to become seated.

Random Errors

Random errors are associated with the analog-to-digital conversion of the deflections and are on the order of ±2 μm.[86] Random errors cannot be completely eliminated, but they can be reduced by taking multiple readings and averaging the results. This reduction is proportional to the square root of the number of observations used in computing the mean. For example, if four replicate FWD drops (with the same height) at the same point were averaged, the random error would be reduced by half. Care must be taken to ensure that no liquefaction or compaction has taken place because of the additional drops.[35]

Systematic Errors

Systematic errors are associated with the particular FWD equipment and its specific sensors. Systematic errors are on the order of ±2 percent. FWD specifications therefore call for an accuracy of ±2 percent or ±2 μm, whichever is larger. This specification combines the systematic error and the random error. Systematic errors can be reduced to 0.3 percent or less for each individual sensor, including the load cell, through calibration.[35]

Noise and Truncation of FWD Sensor Signals

The sensitivity of dynamic backcalculation solutions to signal noise is high. Basically, noisy data alters the error function surface enough to lead optimization astray. This can cause the search algorithm to diverge, or to converge to a different modulus when regularization techniques are used. The remedy to noise is to preprocess the raw data by filtering out the high frequency content of the signal (anything above 100 Hz) in deflection and load pulse data.

Another issue that is relevant to dynamic analysis is the existence of a nonphysical time lag between the load and deflection pulses that may be caused by synchronization problems in the data acquisition system. In addition, most FWD load and deflection pulses are cut off before the complete decay of the time histories. This generates a discontinuity in the slope of the function leading to significant errors in the frequency content of the signals. This truncation problem can lead to large errors in the backcalculated layer parameters when using a frequency-based solution.[81]

MODELING ISSUES

Several specific modeling issues are in play when considering backcalculation solutions, as described in the following subsections.

Static Versus Dynamic Response

As described in chapter 2, the FWD test consists of dropping a large weight from a specified height, which creates a 20- to 60-ms impulse load, simulating a moving wheel load. This creates waves in the pavement system and underlying subgrade soil. These elastic waves propagate with distance and are partly reflected at the interface between any given two successive layers, with the remaining wave energy penetrating and propagating to the next layer, and the process is repeated. These waves bounce up and down a few hundred times in a given test. The deflection time histories lag the load pulse, with the time lag increasing as the distance between the load plate and the sensor increases. So, clearly, the FWD test is dynamic.

The difference between static response and dynamic response can be defined in terms of the internal forces involved. In static analysis, only elastic forces are considered. On the other hand, viscous and inertial forces are considered in addition to the elastic forces in dynamic analysis. The question therefore is whether the effects of viscous and inertial forces are significant enough that one cannot afford to ignore them when characterizing the in situ conditions of a pavement system under an FWD test. Most pavement engineers argue that backcalculation is an exercise that determines pavement parameters, and not properties, that are to be used within a given mechanistic framework. Therefore, it is acceptable to use static analysis and to backcalculate parameters that are compatible with the current mechanistic-empirical design framework that is grounded in static and not dynamic analysis. However, advocates for dynamic analysis maintain that such an approach takes advantage of more information provided by the test, which allows for backcalculating more parameters such as layer thicknesses or the modulus versus frequency curve of the HMA layer. Also, in certain cases, such as the existence of a stiff layer or water table at shallow depth, the effect of dynamics on pavement response may be more important.

Linear Versus Nonlinear Material Response

When pavement structures are thin enough or the applied loads and corresponding stresses are high enough, the subgrade material likely exhibits stress-softening, nonlinear behavior (i.e., its response increases at a higher rate than the load or stress increases). This translates to the subgrade modulus changing with depth and radial distance from the load. If the forward model uses a layered solution that assumes linear material behavior, it can only use one modulus value for an entire layer. Consequently, the backcalculated modulus that is required to match the measured deflections is an averaged value. Typically, the backcalculated subgrade modulus is higher than the laboratory-based value by a factor of about 1.3 to 3.[87]

On the other hand, granular (cohesionless) materials used in bases and subbases are stress-dependent in a different (positive) way (i.e., their modulus increases with increasing confinement). Similar to the subgrade modulus, this leads to a base/subbase modulus that varies with depth and radial distance from the load, and any linear backcalculation exercise can only lead to an averaged modulus value, assuming linear behavior in the forward model leads to an underestimation of the base/subbase modulus. The combination of the above phenomena often leads to a base modulus that is lower than the subgrade modulus despite the fact that the base material is superior to that of the subgrade. One way of addressing this problem is to introduce an artificial layer. However, a more direct way of addressing the problem is to treat the subgrade as a nonlinear elastic material with stress-dependent modulus as shown in figure 75.[88]

$$E = C\left(\frac{\sigma_1}{p}\right)^n$$

Figure 75. Equation. Stress-dependent elastic modulus.[(88)]

Where:

E = Modulus value.
C = Positive constant.
σ_1 = Stress.
p = Reference stress.
n = Negative constant.

Ullidtz argues that the effect of the positive nonlinearity in granular base/subbase layers on backcalculation results is less important.[(88)]

Ideally, only the finite element method can model the variation of moduli with depth and radial distance. However, there are models based on layered elastic theory that can handle nonlinear behavior approximately (e.g., NELAPAVE and KENPAVE). Ullidtz combines MET described previously with a stress-dependent subgrade modulus, as described in figure 75, to handle material nonlinearity, and reports that this approach is superior to the finite element method.[(88)]

Bedrock or Stiff Layer Effect

A stiff layer condition can exist if there is shallow bedrock, a stiff clay layer, or a groundwater table. The effect of a stiff layer at a shallow depth can be very significant. Assuming the subgrade layer to be a semi-infinite halfspace, while in reality the subgrade layer is only a few meters thick, causes the backcalculated moduli for the upper pavement layers to be incorrect. Generally, when the stiff layer is deeper than about 12 m (39 ft), its presence has little or no influence on the backcalculated moduli. The depth to the stiff layer can be evaluated by using a relationship between the deflection, δ_z, and $1/r$, where r is the radius at which it occurs (see figure 76). Several regression equations for different HMA layer thicknesses can be used as a function of r_o and deflection basin parameters.[(40)]

An alternative, and arguably better, way to determine the depth to the stiff layer is to use the free vibration response from FWD deflection sensor measurements and one-dimensional wave propagation theory.[(89)] Chatti, Ji, and Harichandran modified Roesset's equations to account for different conditions, as shown in the equations in figure 77 and figure 78.[(46)]

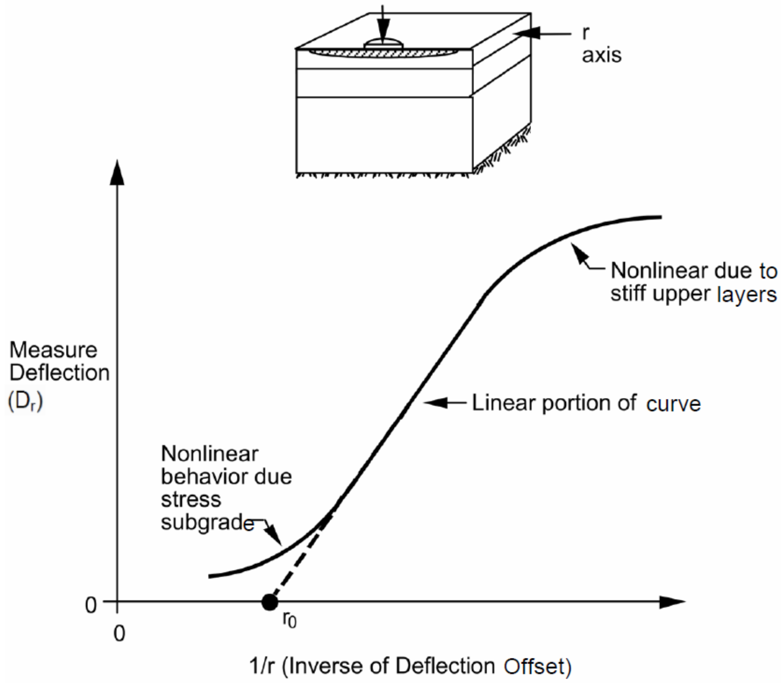


Figure 76. Graph. Plot of the inverse of deflection offset versus measured deflection.[40]

$$D_b = \frac{V_s * T_d}{1.35}$$

Figure 77. Equation. Saturated subgrade with bedrock.[46]

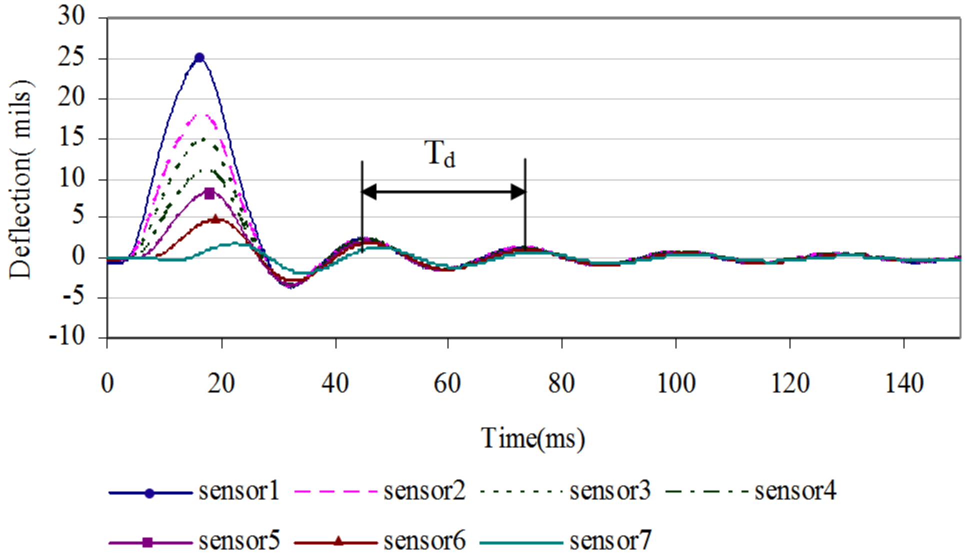
$$D_b = \frac{V_s * T_d}{(\pi - 2.24 * u)}$$

Figure 78. Equation. Nonsaturated subgrade with bedrock or groundwater table.[46]

Where:

D_b = Depth of bedrock.
V_s = Shear-wave velocity of subgrade = $[(E_{sg}/(2(1-u^2))/\rho]^{0.5}$.
E_{sg} = Modulus of the subgrade.
ρ = Unit weight of the subgrade.
u = Poisson's ratio of subgrade.
T_d = Natural period of free vibration (see figure 79).

As defined here, the shear wave velocity of the subgrade, V_s, is a function of the subgrade modulus, the subgrade unit weight, and the subgrade Poisson's ratio. This value can be determined using an iterative procedure developed by Lee, Kim, and Ranjithan.[90]



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1 mil = 0.0254 mm.

Figure 79. Graph. Natural period, T_d, from sensor deflection time histories.(46)

Temperature and Moisture Effects

Temperature and moisture conditions in the pavement vary over time. This variation occurs daily as well as seasonally. A pavement is strongest during the freezing season (in a freezing climate) because of the frozen state of the underlying materials. On the other hand, even in a freezing environment, the pavement can be at its weakest state during a thaw period, even if that period is short and temporary (e.g., on a sunny day in late winter, during the warmest hours around midday). In areas where there is little or no freezing, seasonal variations can be very important in terms of moisture changes, which affect the modulus of the subgrade and to a lesser extent that of the base layer. For the HMA layers, hourly temperature variations during a given day need to be taken into account because temperature gradients exist in the pavement, which can lead to modulus variation with depth. Also, seasonal variations can have a major effect on the modulus of an HMA layer. These effects must be considered when performing backcalculation. It is crucial to test the pavement at different times of the year to gain information about the seasonal variation. Testing should also be conducted at different times during the day to account for daily temperature variations.

Other Effects

Several other issues may need to be addressed in backcalculation analysis, including the following:

- Major cracks in the pavement, or testing near a pavement edge or joint, can cause the deflection data to depart drastically from the assumed conditions.
- Layer thicknesses are often not known, and subsurface layers can be overlooked.
- Layer thicknesses are not uniform, and materials in the layers are not homogeneous.

- Some pavement layers are too thin to be backcalculated in the pavement model.

Relevance to MEPDG Use

The required input material properties for HMA pavements in the new MEPDG that are relevant to the use of FWD data and backcalculation results are (1) the time-temperature dependent dynamic modulus E^* for the HMA layer(s), (2) the resilient moduli for the unbound base/subbase and subgrade materials, and (3) the elastic modulus of the bedrock, if present.(7) The MEPDG also provides an option for considering nonlinear material parameters for the unbound layers for level 1 analysis. However, the performance models used in the software have not been calibrated for nonlinear conditions; therefore, this option is not considered further in this report.

HMA Materials

For new HMA design, level 1 analysis requires conducting E^* laboratory testing (ASTM D3496) at loading frequencies and temperatures of interest for the given mixture.(91) Level 2 analysis does not require E* laboratory testing; instead, the user can enter asphalt mix properties (gradation parameters) and laboratory binder test data (from G^* testing or other conventional binder tests). The MEPDG software calculates the corresponding asphalt viscosity values; it then uses the modified Witczak equation to predict E^* and develops the master curve for the HMA mixture.(7) This same procedure is used for level 3 analysis to estimate the HMA dynamic modulus except no laboratory test data are required for the binder.

For rehabilitation design, determination of the HMA layer dynamic modulus follows the same general concepts described above, except the software allows use of a modified procedure to account for damage incurred in the HMA layer during the life of the existing pavement.(7) The procedure therefore determines a "field damaged" dynamic modulus master curve as follows:

For level 1 analysis, the MEPDG calls for the following procedure:(7)

- **Step 1:** Conduct FWD tests in the outer wheelpath over the project to be rehabilitated; calculate the mean backcalculated HMA modulus, E_{dam}, for the project, combining layers with similar materials and including cracked as well as uncracked areas; record the HMA layer temperature at the time of testing and determine the layer thickness along the project using coring or ground-penetrating radar (GPR) testing.

- **Step 2:** Determine mix volumetric parameters (air void content, asphalt content, and gradation) and asphalt viscosity parameters (regression intercept (A) and regression slope of viscosity temperature susceptibility (VTS)) from cores and follow the same procedure for determining binder viscosity-temperature properties as for new or reconstruction design.

- **Step 3:** Develop an undamaged dynamic modulus master curve using the modified Witczak equation and the data from step 2 at the same temperature recorded in the field and at an equivalent frequency corresponding to the FWD pulse duration.

- **Step 4:** Estimate the fatigue damage in the HMA layer (d_{ac}) using the E_{dam} obtained from step 1 and the undamaged dynamic modulus, E^*, obtained from step 3.
- **Step 5:** Calculate $\alpha' = (1 - d_{ac})\,\alpha$; where α is a function of mix gradation parameters.
- **Step 6:** Determine the field-damaged dynamic modulus master curve using α' instead of α in the modified Witczak equation.

For level 2 and level 3 analyses, no FWD testing is required. The level 2 procedure is similar to the level 1 procedure, in that field cores are used to obtain the undamaged modulus; however, estimates for fatigue damage of the existing asphalt layer is determined through a detailed pavement condition survey and the calibrated MEPDG distress models. For the level 3 procedure, no coring or testing is required; instead, typical estimates of HMA mix parameters (typical volumetric and binder properties) are entered, and the program calculates the undamaged master curve.

Unbound Materials

For unbound materials (and bedrock), only level 1 analysis calls for FWD testing in rehabilitation and reconstruction designs. The resilient modulus, M_r, for each unbound layer (including the subgrade) can be either determined in the laboratory using cyclic triaxial tests or backcalculated using standard backcalculation procedures. As discussed previously, while the MEPDG does allow for the generalized nonlinear, stress-dependent model in the design procedure, this approach is not recommended at this time because the performance models in the software have not been calibrated for nonlinear conditions; therefore, the option of backcalculating the k_1, k_2, and k_3 parameters in the nonlinear model is not discussed.[(7)] Consequently, the discussion only includes the backcalculation and use of "effective" moduli that would account for any stress sensitivity, cracks, or any other anomalies in any layer within the existing pavement. For level 2 analysis, correlations with strength test data are used. For level 3, the MEPDG lists typical modulus values based on soil classification, but warns that they are very approximate and strongly recommends some form of testing, especially using FWD testing and backcalculation (as in level 1).[(7)]

The MEPDG notes that the reason for caution is related to using the wrong assumptions: either a fairly strong subgrade material may be erroneously assumed to be semi-infinite while it may actually be less than 1 m (3 ft) thick (e.g., as part of an embankment), or conversely, a weak subgrade soil may be assumed to be semi-infinite while it may, in reality, be overlying a stronger soil or bedrock.[(7)]

Chemically Stabilized Materials

Similar to unbound materials, only level 1 analysis calls for FWD testing in rehabilitation and reconstruction designs. The modulus, E or M_r, for any chemically stabilized layer (including lean concrete and cement stabilized base, as well as lime/cement/flyash stabilized soils) can be either determined in the laboratory or backcalculated using standard backcalculation procedures. Layer thicknesses can be obtained by coring or using NDT techniques such as GPR. The MEPDG recommends performing limited testing on cored lime stabilized soil specimens to verify/confirm the backcalculated values and notes that backcalculation of modulus values for layers less than

150 mm (6 inches) thick located below other paving layers may be problematic, thus requiring laboratory testing.[7]

For level 2 analysis, correlations with strength test data are used. For level 3, the MEPDG calls for estimating the moduli based on experience or historical records and lists typical modulus values.[7] The MEPDG also notes that semirigid cementitiously stabilized materials are more prone to deterioration owing to repeated traffic loads when used in HMA pavements and suggests some typical (minimum) values for such deteriorated materials.[7]

Using Static Backcalculation in the Current MEPDG Procedure

It should be clear from the previous discussion that the analysis in the MEPDG software always uses an E^* master curve and therefore does not accept a constant modulus value for the HMA layer(s). This is necessary because the analysis calculates different HMA moduli for the different sublayers comprising the HMA layer(s) as a function of depth, speed, and axle type, as explained in appendix CC of the MEPDG.[7] For rehabilitation of existing pavements, the current MEPDG procedure (level 1) calls for (static) backcalculation of layer moduli, which leads to constant backcalculated moduli for all layers, including the HMA layer. To maintain compatibility of backcalculated layer moduli with the forward analysis in the software, the MEPDG procedure calls for adjusting the HMA dynamic modulus using the damage factor d_{ac} (ratio of backcalculated HMA modulus to predicted E^* value using the Witczak equation).[7] This effectively shifts the undamaged master curve down while essentially maintaining the variation with frequency as predicted by the Witczak equation. The procedure also calls for adjusting the master curve using the aged viscosity value in the predictive E^* equation, which would shift the master curve upward; however, this upward shift will be negligible compared with the downward shift using the backcalculated modulus for the damaged HMA layer, E_i (as explained in previous subsection on HMA Materials).

Feasibility of Using Dynamic Backcalculation for Future Versions of the MEPDG

Ideally, one should be able to determine a curve of HMA layer modulus as a function of frequency using a (dynamic) frequency-based backcalculation algorithm. This would give a more direct estimation of the HMA layer modulus with frequency from actual field conditions as opposed to relying on a laboratory-derived curve such as the Witczak equation. However, care should be taken in interpreting and using such data with the existing MEPDG performance predictions because they have been calibrated using laboratory-derived moduli. Also, recent analyses show that while dynamic backcalculation methods can backcalculate layer moduli and thicknesses accurately from synthetically generated FWD data for pavement systems with three or more layers, they present some serious challenges when using field data.[46] The frequency-domain method can lead to large errors if the measured FWD records are truncated before the motions fully decay in time. Dynamic, time-domain backcalculation algorithms present another challenge in that they cannot directly determine the HMA modulus as a function of frequency. They either assume a constant HMA modulus (similar to static backcalculation) or a prescribed function of the HMA layer modulus with frequency (e.g., a linear relation in the log-log space).

CHAPTER 5. FWD DATA ANALYSIS AND INTERPRETATION—RIGID PAVEMENTS

INTRODUCTION

FWD testing has been traditionally performed on rigid pavements to assess the condition of the pavement to identify the most effective rehabilitation strategy and to establish inputs for an overlay design. FWD data are very useful in determining structural deficiencies before the point when they manifest as a distress. Although FWD data by itself can be quite useful, it is commonly supplemented with a distress survey and possibly GPR testing or destructive testing. Use of the FWD to assess the condition of a rigid pavement includes testing at the corner of the slab for void detection and adjacent to the form transverse joint to monitor joint performance. The MEPDG does not include a direct input for the presence of voids, but this information should be used to determine whether subsealing or rubblization should be performed before the placement of an overlay. The LTE measured at the joints/cracks is also not a required input for the MEPDG but should still be evaluated to determine whether dowel retrofits are necessary before placing the overlay or determining the potential for reflective cracking if a HMA overlay is applied.

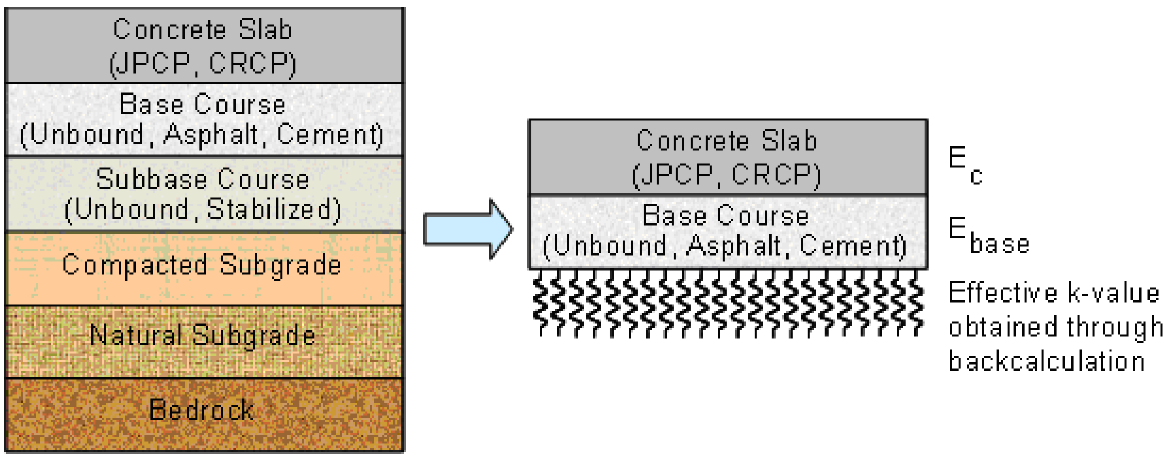
FWD testing is also performed at midpanel to backcalculate the material properties for designing an overlay. Historically, the modulus of subgrade reaction (*k*-value) has been required to characterize the support condition beneath the slab for rigid pavements. When using the MEPDG to design overlays placed on existing PCC pavements, the pavement structure can be characterized by either the modulus of each layer or the elastic modulus of the slab and the modulus of subgrade reaction.

This chapter discusses the approaches available for backcalculating the material properties of a PCC pavement for designing an overlay, analysis and modeling issues associated with these approaches, recommendations for using these approaches with the MEPDG, and future research needs in the area.

APPROACHES

Several approaches are available for backcalculating the moduli of the PCC slab, base, subbase, and subgrade, as well as the modulus of subgrade reaction. The two primary approaches are a function of how the pavement structure is modeled. The first approach, based on the principles of elastic layered analysis, is typically used for flexible pavements but has also been applied to rigid pavements. The second approach, in which the pavement structure is modeled as either a slab on an elastic solid or on a dense liquid foundation, was specifically developed for rigid pavements based on plate theory.

When the layer moduli are used to define the characteristics of the pavement structure in the MEPDG, the software uses an internal conversion process that automatically determines the effective *k*-value corresponding to the layer moduli. (As a side note, traditionally, an effective *k*-value refers to a *k*-value that is adjusted for seasonal effects, and the composite *k*-value refers to the composite stiffness of all layers beneath the slab. The MEPDG documentation refers to an effective *k*-value as being the composite stiffness of all layers beneath the base, as shown in figure 80.[7]) The process involves backcalculating the effective *k*-value from the theoretical deflection basin produced using the elastic layer program JULEA. However, in this process, the subgrade resilient modulus is adjusted to reflect the lower deviator stress under PCC pavements (compared with that used in laboratory resilient modulus testing) before generating the deflection basin. Therefore, the subgrade resilient modulus backcalculated from FWD testing cannot be used directly in the MEPDG for PCC or composite pavements because the backcalculated moduli values would reflect the state of stress under PCC pavements, not laboratory testing conditions, which is expected in the MEPDG. Although adjustments could again be made to obtain “laboratory” resilient moduli, the *k*-value directly backcalculated from the FWD data best represents the true foundation stiffness.



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JPCP = Jointed portland cement concrete pavement
CRCP = Continuous reinforced concrete pavement

Figure 80. Illustration. *E*-to-*k* conversion process incorporated in the MEPDG.[6]

Because the linear elastic layered analysis approach was already discussed in chapter 4 and the preferred input is actually the effective *k*-value, the focus here is on the methods that are based on the plate theory approach: the Best-Fit and AREA methods. The Best-Fit method minimizes the difference between the predicted and measured deflections at each sensor by solving for the best combination of the radius of relative stiffness and the modulus of subgrade reaction. The AREA method estimates the radius of relative stiffness based on the AREA of the deflection basin. Closed form equations along with the estimated radius of relative stiffness can then be used to estimate the modulus of subgrade reaction and the elastic modulus of the slab. Both methods are based on Westergaard’s solution for interior loading of a linear elastic, homogenous, isotropic plate resting on a dense liquid foundation.

Available Programs

The two backcalculation procedures based on plate theory (the AREA method and the Best-Fit method) are described in the following sections.

AREA Method

Hoffman and Thompson first introduced the *AREA* parameter to characterize the deflection basin for a simple two-parameter backcalculation procedure for flexible pavements.(50) Foxworthy and Darter were the first to apply the AREA concept to the backcalculation for PCC pavements.(92,93) Others refined this approach through the development of efficient, closed-form solutions that replaced the graphical procedure developed by Foxworthy and Darter.(94,49,95) This method has been used quite extensively, in part, because it is the procedure adopted in the 1993 AASHTO *Guide for Design Pavement Structures*.(32)

The AREA method is based on the unique relationship between the *AREA* parameter and the radius of relative stiffness, ℓ, which is defined by the equation in figure 81.

$$\ell = \sqrt[4]{\frac{E_{PCC} \cdot h^3}{12 \cdot (1 - \mu^2) \cdot k}}$$

Figure 81. Equation. Computation of radius of relative stiffness.

Where:

ℓ = Radius of relative stiffness, mm (inches).
E_{PCC} = PCC elastic modulus, MPa (lbf/inch^2).
h = Slab thickness, mm (inches).
μ = PCC Poisson's ratio.
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).

The *AREA* parameter is calculated based on the trapezoidal rule. It is not the actual area under the deflection profile but rather an area normalized with respect to one of the measured deflections. Therefore, *AREA* has the unit of length and not area. The normalization helps reduce the effects of load magnitude. The *AREA* definitions developed for three common sensor configurations are provided in the equations in figure 82 through figure 84.(95)

$$AREA_{36} = 6\left(1 + 2\frac{d_{12}}{d_0} + 2\frac{d_{24}}{d_0} + \frac{d_{36}}{d_0}\right)$$

Figure 82. Equation. *AREA* parameter for four-sensor configuration.(95)

$$AREA_{60} = 4 + 6\cdot\frac{d_8}{d_0} + 5\cdot\frac{d_{12}}{d_0} + 6\cdot\frac{d_{18}}{d_0} + 9\cdot\frac{d_{24}}{d_0} + 18\cdot\frac{d_{36}}{d_0} + 12\cdot\frac{d_{60}}{d_0}$$

Figure 83. Equation. *AREA* parameter for seven-sensor configuration.(95)

$$AREA_{72} = 6\cdot\left(1 + 2\cdot\frac{d_{12}}{d_0} + 2\cdot\frac{d_{24}}{d_0} + 2\cdot\frac{d_{36}}{d_0} + 2\cdot\frac{d_{48}}{d_0} + 2\cdot\frac{d_{60}}{d_0} + \frac{d_{72}}{d_0}\right)$$

Figure 84. Equation. *AREA* parameter for eight- to nine-sensor configuration.(95)

The relationship between AREA and ℓ is given by the equation in figure 85, with the regression coefficients provided in table 5.[95] These coefficients were defined based on *AREA* parameters provided in U.S. customary units.

$$\ell = \left[\frac{\ln\left(\frac{k_1 - AREA}{k_2}\right)}{-k_3} \right]^{1/k_4}$$

Figure 85. Equation. Relationship between AREA and ℓ . [95]

Table 5. Regression coefficients for AREA versus radius of stiffness relationship.[51]

***AREA* Parameter**	k_1	k_2	k_3	$1/k_4$
$AREA_{36}$	36	1,812.597	2.559	4.387
$AREA_{60}$	60	289.708	0.698	2.566
$AREA_{72}$	72	242.385	0.442	2.205

Note: Coefficients are for use with U.S. customary units.

The radius of relative stiffness is also related to a nondimensional deflection coefficient (d^*). Once the radius of relative stiffness is determined, the nondimensional deflection coefficient for any sensor can be determined using the equation in figure 86 and the regression coefficients given in table 6.[95]

$$d_r^* = a \cdot e^{-b \cdot e^{-c \cdot \ell}}$$

Figure 86. Equation. Nondimensional deflection coefficient.[95]

Where:

d_r^* = Nondimensional deflection coefficient for deflection at radial distance r from load.
a, b, c = Regression coefficients (see table 6).
ℓ = Radius of relative stiffness, mm (inches).

Table 6. Regression coefficients for nondimensional deflection coefficient.[51]

Nondimensional Deflection Coefficient	*a*	*b*	*c*
d_0^*	0.12450	0.14707	0.07565
d_8^*	0.12323	0.46911	0.07209
d_{12}^*	0.12188	0.79432	0.07074
d_{18}^*	0.11933	1.38363	0.06909
d_{24}^*	0.11634	2.06115	0.06775
d_{36}^*	0.10960	3.62187	0.06568
d_{48}^*	0.10241	5.41549	0.06402
d_{60}^*	0.09521	7.41241	0.06255
d_{72}^*	0.08822	9.59399	0.06118

Note: Coefficients are for use with U.S. customary units.

These values can then be used to calculate the *k*-value based on any sensor deflection using the equation in figure 87.[95]

$$k = \frac{P \cdot d_r^*}{d_r \cdot \ell^2}$$

Figure 87. Equation. Calculation of *k*-value.[95]

Where:

k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).
P = Applied load, N (lbf).
d_r* = Nondimensional deflection coefficient for deflection at radial distance r from the load.
d_r = Measured deflection at radial distance r from the load, mm (inches).
ℓ= Radius of relative stiffness, mm (inches).

The subgrade *k*-value is then used to determine the PCC modulus by rearranging the equation in figure 81 into the equation shown in figure 88.

$$E_{PCC} = \frac{12 \cdot \ell^4 \cdot (1 - \mu^2) \cdot k}{h^3}$$

Figure 88. Equation. Computation of PCC elastic modulus.

Where:

E_{PCC}= Elastic modulus of the slab, MPa (lbf/inch^2).
ℓ = Radius of relative stiffness, mm (inches).
μ = PCC Poisson's ratio.

k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inches).
h = Slab thickness, mm (inches).

The subgrade k-value and E_{PCC} for the entire basin can be determined by averaging the values calculated for each deflection sensor.

Best-Fit Method

The second backcalculation procedure is the Best-Fit method. With this method, the PCC elastic modulus and subgrade k-value are found by identifying the best combination of the two parameters that produces a calculated deflection profile that best matches the measured profile.(51) This is performed through the minimization of the error function provided in figure 89.

$$F(E,k) = \sum_{i=0}^{n} \alpha_i (w_i - W_i)^2$$

Figure 89. Equation. Minimization of the error function.

Where:

E = Elastic modulus of the slab, MPa (lbf/inch^2).
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/in).
α_i = Weighting factor for deflection measured by sensor i.
w_i = Calculated deflection at sensor location i, mm (inches).
W_i = Measured deflection at sensor location i, mm (inches).
n = number of sensors.

The calculated deflection (w_i) is based on Westergaard's solution for interior loading of a plate consisting of a linear elastic, homogenous, and isotropic material on a dense liquid foundation. The weighting factor α_i can be set to 1, $(1/W_i)^2$, or any other number as a means of providing flexibility to the Best-Fit solution process.(27) Deflection (w) for sensor i located a distance r from a uniformly distributed circular load can be determined using the equation in figure 90.(96,27)

$$\boldsymbol{w_i(r) = \frac{p}{k} f_i(\ell)}$$

$$f(r) = 1 - C_1(a_\ell)ber(s) - C_2(a_\ell)bei(s) \qquad \textit{for } 0 < r < a$$

$$f(r) = C_3(a_\ell)\ker(s) + C_4(a_\ell)kei(s) \qquad \textit{for } r > a$$

Figure 90. Equation. Calculated deflection at specified location.(96,27)

Where:

w_i = Calculated deflection at sensor location i, mm (inches).
r = Radial distance of sensor i from the applied load, mm (inches).
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).
$f_i(\ell)$ = A function of ℓ, distance from the load, and parameters of applied load.
ℓ = Radius of relative stiffness, mm (inches).

$a_\ell = (a/\ell)$ = Dimensionless radius of the applied load.
$s = (r/\ell)$ = Normalized radial distance.
$\ell = (D/k)^{0.25}$ = Radius of relative stiffness of plate-subgrade system for dense-liquid foundation.
$D = Eh^3/12(1 - \mu^2)$ = Flexural rigidity of the plate.
E = Plate elastic modulus.
m = Plate Poisson's ratio.
h = Plate thickness.
p = Applied load intensity (pressure) = $P/(\pi a^2)$.
P = Total applied load.
a = Radius of the applied load.

Note that *ber*, *bei*, *ker*, and *kei* are Kelvin Bessel functions that can be evaluated using appropriate series expressions.[95]

Korenev provided the solutions for the constants C_1, C_2, C_3, and C_4 shown in figure 91.[97]

$$C_1 = -a_\lambda \ker' a_\lambda$$
$$C_2 = a_\lambda kei' a_\lambda$$
$$C_3 = -a_\lambda ber' a_\lambda$$
$$C_4 = -a_\lambda bei' a_\lambda$$

Figure 91. Equation. Solutions for *C* constants.[96]

Where *ker′*, *kei′*, *ber′*, and *bei′* are the first derivatives of the Kelvin Bessel functions *ker*, *kei*, *ber*, and *bei*, respectively.

Based on equations in figure 89 and figure 90, the error function can be defined as shown in figure 92.

$$F(E, k) \equiv F(\ell, k) = \sum_{i=0}^{n} \alpha_i \left(\frac{p}{k} f_i(\ell) - W_i \right)^2$$

Figure 92. Equation. Error function.

The error function is minimized by satisfying the conditions shown in figure 93.

$$\frac{\partial F}{\partial k} = 0$$
$$\frac{\partial F}{\partial \ell} = 0$$

Figure 93. Equation. Minimization of error function.

The *k*-value (figure 94) is determined by substituting the error function equation (figure 92) into the first condition of figure 93.

$$k = p \cdot \frac{\sum_{i=0}^{n} \alpha_i \left(f_i(\ell) \right)^2}{\sum_{i=0}^{n} \alpha_i W_i f_i(\ell)}$$

Figure 94. Equation. *k*-value determination.

Substitution of the error function equation (figure 92) into the second condition of figure 93 yields the equation in figure 95 for the radius of relative stiffness.(27)

$$\frac{\sum_{i=0}^{n} \alpha_i f_i(\ell) f_i'(\ell)}{\sum_{i=0}^{n} \alpha_i \left(f_i(\ell) \right)^2} = \frac{\sum_{i=0}^{n} \alpha_i W_i f_i'(\ell)}{\sum_{i=0}^{n} \alpha_i W_i f_i(\ell)}$$

Figure 95. Equation. Radius of relative stiffness determination.(27)

The radius of relative stiffness can then be used to determine the elastic modulus of the slab using the equation in figure 88.

The Best-Fit and AREA methods have both been adopted for a pavement structure modeled as a plate on an elastic solid foundation. The procedure was developed based on Losberg's solution for the distribution of deflections under a load distributed uniformly over a circular area for a plate on an elastic solid.(98) In the case of the Best-Fit method, the minimization of the error function is performed with respect to the elastic modulus of the foundation and the radius of relative stiffness for an elastic solid foundation.(27) It was also found that a unique solution exists between AREA and the elastic solid radius of relative stiffness, making it possible to develop closed-form solutions for the backcalculation of the modulus of an elastic solid foundation using the AREA method.(99) Further details on these methods are not provided because the foundation is not modeled as an elastic solid in the MEPDG.

Both the AREA and Best-Fit methods are relatively easy to use even though software programs are not available for either method. The AREA method has the advantage that it uses simple, closed-form solutions, so a spreadsheet can be easily set up to backcalculate the *k*-value and the elastic modulus of the slab. A simple computer program can be written to solve for these parameters using the Best-Fit method. The advantage of the Best-Fit method is that backcalculation can be performed for any sensor configuration.

Base Layer

The methods described are applicable for PCC pavements modeled as slab-on-grade structures. This assumes no structural contribution is provided by the base or subbase layer. In fact, these layers can have a significant effect on the pavement response when the base layer is stiff. This results in an artificially high modulus of the PCC slab. Ioannides and Khazanovich developed an approach for the backcalculation of a two-layered slab-on-grade system.(100) The two layers are modeled as either bonded or unbonded plates, and therefore, these layers are considered incompressible. Both the unbonded and bonded cases are discussed in the following subsections.

Unbonded Case

For the unbonded case, the two plates act independently even though the deflected shapes of the two plates are the same. The effective stiffness of the two plates combined (D_e) is equal to the sum of the stiffnesses of the upper plate (D_1) and the lower plate (D_2), as given by the figure 96 equation. D_e can be determined using the backcalculated elastic modulus from the slab-on-grade analyses presented above using either the AREA or Best-Fit method. The effective stiffness is then determined using the figure 97 equation, which assumes that the thickness of the effective plate, h_e, is equal to the thickness of the upper plate, h_1.(27)

$$D_e = D_1 + D_2$$

Figure 96. Equation. Concept of effective stiffness.(27)

$$D_e = \frac{E_e \cdot h_e^3}{12 \cdot (1 - \mu_e^2)}$$

Figure 97. Equation. Effective stiffness determination.(27)

The Poisson's ratio of the effective plate, μ_e, the upper plate, μ_1, and the lower plate, μ_2, are assumed to be equal (figure 98) and, therefore, the equation in figure 99 is valid.

$$\mu_e = \mu_1 = \mu_2$$

Figure 98. Equation. Equivalency of Poisson's ratio for all plates.(27)

$$E_e h_e^3 = E_e h_1^3 = E_1 h_1^3 + E_2 h_2^3$$

Figure 99. Equation. Equivalency of plate stiffnesses.(27)

Where:

E_1 = Elastic modulus of the upper plate, MPa (lbf/inch^2).
E_2 = Elastic modulus of the lower plate, MPa (lbf/inch^2).

By introducing an additional input parameter β, the stiffness of the two plates (figure 100) can be determined using the equations in figure 101 and figure 102. β, known as the moduli ratio, is the ratio of the stiffness of the lower plate with respect to that of the upper plate.

$$\beta = \frac{E_2}{E_1}$$

Figure 100. Equation. Moduli ratio.(27)

$$E_1 = \frac{h_1^3}{h_1^3 + \beta h_2^3} \cdot E_e$$

Figure 101. Equation. Computation of upper plate elastic modulus.[(27)]

$$E_2 = \frac{h_1^3 \beta}{h_1^3 + \beta h_2^3} \cdot E_e$$

Figure 102. Equation. Computation of lower plate elastic modulus.[(27)]

Bonded Case

When the two plates are bonded, the effective stiffness of the two plates is no longer a summation of the upper and lower plates, but can be defined as shown in figure 103.

$$\frac{E_e h_e^3}{12} = \frac{E_1 h_1^3}{12} + E_1 h_1\left(x - \frac{h_1}{2}\right)^2 + \frac{E_2 h_2^3}{12} + E_2 h_2\left(h_1 - x + \frac{h_2}{2}\right)^2$$

$$x = \frac{\frac{h_1^2}{2} + h_2 \beta\left(h_1 + \frac{h_2}{2}\right)}{h_1 + \beta h_2}$$

Figure 103. Equation. Bonded case effective stiffness.

With the thickness of the effective plate, h_e, equal to the thickness of the upper plate, h_1, the elastic modulus backcalculated based on the slab on-grade pavement structure, E_e, can be used to determine the elastic modulus of the upper plate, E_1, as shown in figure 104.

$$E_1 = \frac{h_1^3}{h_1^3 + \beta h_2^3 + 12 h_1\left(x - \frac{h_1}{2}\right)^2 + 12 \beta h_2\left(h_1 - x + \frac{h_2}{2}\right)^2} E_e$$

Figure 104. Equation. Elastic modulus of the upper plate.

The stiffness of the lower plate, E_2, can then be found by multiplying the stiffness of the upper plate by β.

This procedure requires that two pieces of information be known at the time FWD testing is performed: (1) the bond condition between the slab and the base, and (2) the moduli ratio. Engineering judgment is needed in selecting an appropriate moduli ratio. Guidance in selecting a moduli ratio is provided by Khazanovich, Tayabji, and Darter, where a table of typical values for β is provided for a range of base materials.[(27)]

Fortunately, it has been found that the influence of the moduli ratio on E_{PCC} is not significant for most projects. The modulus of the base is more sensitive to changes in β. The use of an inaccurate moduli ratio that does not affect the backcalculated modulus of the slab could still produce an inaccurate elastic modulus for the base. Again, previous studies have shown that as long as the ratio is within reasonable limits, the results of backcalculation are insensitive to the ratio.(101,27)

As previously noted, the bond condition between the base and the slab must be known to use the two-layered, slab-on-grade analysis. If the slab is assumed to be fully bonded when it actually is unbonded, then unrealistically high moduli will be obtained for the PCC and the base. Likewise, if the slab is assumed to be fully unbonded when it actually is bonded, then unrealistically low moduli will be obtained. The assumed bond condition should be reevaluated and adjusted accordingly if unrealistic moduli are determined.

Applicability for MEPDG Use

The Best-Fit method was determined to provide more consistent results and has been recommended by other researchers.(51,27) The *k*-values and slab moduli used to populate the LTPP database were also determined using the Best-Fit method. These values were then used in the development of the MEPDG. The advantage the Best-Fit method has over the AREA method is that the equations can easily be placed in a spreadsheet, making the backcalculation of the moduli very simple.

Another matter to consider when performing backcalculation is defining which sensor configuration will provide the best results based on the backcalculation method to be used. When using the Best-Fit method, it has been found that the four-sensor configuration yielded more reasonable results a greater percentage of the time than the seven-sensor configuration (the seven- and four-sensor configurations are listed in table 7).(51) Therefore, it is recommended that the four-sensor configuration be used for the Best-Fit method.

Table 7. Distances of each sensor from the applied load for the seven- and four-sensor configurations.(51)

Sensor Configuration	Sensor Position (mm)
Seven	0, 203, 457, 610, 914, and 1,524
Four	0, 305, 610, and 914

25.4 mm = 1 inch

The AREA and Best-Fit methods give comparable results, although the AREA method tends to produce *k*-values that are slightly higher than the values obtained from the Best-Fit method. This is largely because of the compressibility of the PCC slab.(27) However, given that good agreement was found between the Best-Fit method with the four-sensor configuration and the AREA method with the seven-sensor configuration, the latter method is also recommended for use in backcalculating moduli.(51,27)

Accuracy and Reliability

The reliability of the backcalculation results can be checked by comparing the predicted deflection basin with the measured deflection basin. The relative error between the measured deflection at each sensor and deflections calculated using the backcalculated parameters can be found using the equation in figure 105. The average of the absolute relative errors for all of the sensors is the absolute relative error of the deflection basin (figure 106).

$$\varepsilon_i = \frac{w_i - W_i}{W_i} 100\, percent$$

Figure 105. Equation. Relative error between the measured and calculated deflections.

Where:

ε_i = Relative error for sensor i, percent.
w_i = Calculated deflection at sensor location i, mm (inches).
W_i = Measured deflection at sensor location i, mm (inches).

$$\varepsilon_b = \frac{\sum_{i=1}^{n} \varepsilon_i}{n}$$

Figure 106. Equation. Mean absolute relative error for deflection basin.

Where:

ε_b = Mean absolute relative error for a deflection basin, percent.
ε_i = Relative error for sensor i, percent.
n = Number of sensors used to define the deflection basin.

A tolerance level of 2 percent for the mean absolute relative error for the deflection basin is considered acceptable.[27]

A check on the validity of the backcalculation data should include not only a comparison of the measured and calculated deflection basins at each test location but also the variability of the backcalculated parameters along the test section. The coefficient of variation for the backcalculated *k* should be less than 20 percent along any particular section once reasonable outliers have been removed.[51] The coefficient of variation for the radius of relative stiffness should be less than 10 percent.[27] Factors that might result in coefficients of variation greater than this include changes in subgrade type, depth to bedrock, layer thicknesses, layer conditions, and slab curling conditions.[51,27]

ANALYSIS AND MODELING ISSUES

Various analysis and modeling issues affect the ability to accurately backcalculate *E* and *k*-values. The most prominent of these issues are discussed in following subsections.

Temperature Effects

Slab curling and/or warping can significantly influence the deflection response of PCC pavements, but none of the existing methods account for the effects of slab curling.[27,102] Khazanovich, Tayabji, and Darter showed *k*-values backcalculated at one location to be up to three times as high as a result of temperature gradients.[27] The magnitude of the effect of the gradient is a function of the pavement structure being tested. Vandenbossche found that temperature gradients have a greater influence on backcalculated *k*-values for thinner slabs compared with thicker slabs because the deflections measured on thicker slabs are less sensitive to changes in support conditions.[102] Currently, there is no method available for correcting for the effects of slab curvature when backcalculating *k*-values. It is recommended that FWD testing be performed when there is no significant temperature gradient present to avoid any influence slab curling might have on the backcalculated E_{PCC} and *k*-values.

While temperature and moisture gradients can have an effect on the backcalculated E_{PCC} and *k*-values, researchers have found that uniform changes in temperature have little to no effect.[102,103] Only large temperature fluctuations (temperatures outside of 7 to 32 °C (45 to 90 °F)) substantially influence the backcalculated E_{PCC} and *k*-values.[103] In general, changes in the uniform temperature produce very little variation above and beyond what is typically inherent to the equipment and pavement materials.

Slab Size

The backcalculation procedure presented is based on Westergaard's solution for an infinite plate, but pavements have a finite length and width. Correction factors have been developed for determining the elastic modulus and *k*-value for the interior loading of a square slab by Crovetti using the following procedure:[104]

- **Step 1:** Estimate ℓ (ℓ_{est}) using either the Best-Fit or AREA method.
- **Step 2:** Calculate L/ℓ_{est}, where L is the length of the sides of the square slab.
- **Step 3:** Calculate adjustment factors for the deflection directly under the load plate (d_o) and radius of relative stiffness (ℓ) using equations in figure 107 and figure 108.

$$AF_{\ell_{est}} = 1 - 0.89434e^{-0.61662\left(\frac{L}{\ell_{est}}\right)^{1.04831}}$$

Figure 107. Equation. Adjustment factor for radius of relative stiffness.[104]

$$AF_{d0} = 1 - 1.15085e^{-0.71878\left(\frac{L}{\ell_{est}}\right)^{0.80151}}$$

Figure 108. Equation. Adjustment factor for the deflection directly under the load plate.[104]

- **Step 4:** Calculate adjusted d_0, where adjusted d_0 is the measured $d_0 \times (AF_{do})$.
- **Step 5:** Calculate adjusted ℓ, where adjusted $\ell = \ell_{est} \times (AF_{\ell est})$.
- **Step 6:** Backcalculate E_{PCC} and *k*-value using the adjusted d_0 and the adjusted ℓ.

This method was expanded upon to address rectangular slabs by using an L that represents a square slab having the same area as the rectangular slab, as shown in the figure 109 equation.[27]

$$L = \sqrt{L_{act} W_{act}}$$

Figure 109. Equation. Conversion to equivalent square slab.[(27)]

Where:

L = Length of square slab, m (ft).
L_{act} = Actual slab length, m (ft).
W_{act} = Actual slab width, m (ft).

For longer slabs, where the slab length is greater than twice the width, the equivalent slab length is equal to the value calculated using the equation in figure 110.

$$L = \sqrt{2} L_{act}$$

Figure 110. Equation. Conversion to equivalent square slab (slab length greater than twice the width).

The k-value is then calculated using the figure 111 equation instead of the adjusted d_0.

$$k = \frac{k_{est}}{\left(AF_{\ell_{est}}\right)^2 AF_{d_0}}$$

Figure 111. Equation. Calculation of *k*-value.

The k-value and the adjusted ℓ can then be used to determine E_{PCC} using the figure 88 equation.

Corrections for slab length are not frequently used because it is difficult to define the effective length and width of the slab. The effective length and width are a function of the LTE at the adjacent joints. While this information might be available for the adjacent transverse joints, the load transfer across the lane/shoulder and centerline joints are typically not known. Therefore, defining the effective width becomes difficult.

Load Level

The load magnitude for roadways is typically sufficient that varying load levels does not affect the backcalculation results for PCC pavements. Evidence of this is provided in the study by Khazanovich, Tayabji, and Darter.(27) Larger loads would be necessary for airport pavements.

Backcalculated Versus Laboratory Modulus

Khazanovich, Tayabji, and Darter compared the backcalculated E_{PCC} with the static chord modulus measured using ASTM C469 for cores pulled from LTPP rigid pavement sections with aggregate bases.(27,105) A very poor correlation was found, which can at least partly be attributed to curling/warping of the slab and the inability of the subgrade models to capture the actual subgrade response. The backcalculated moduli are also substantially greater than the measured static values. This is not surprising because a dynamic modulus is being compared with a static modulus. The difference between a static and dynamic modulus is discussed in more detail in the following section. Part of this discrepancy might also be attributable to the difference in the magnitude of the stress where the stress-strain relationship is being defined. PCC is modeled as a linear elastic material, but it actually is a nonlinear inelastic material in both tension and compression. The magnitude of the backcalculated modulus lies somewhere between the initial tangent modulus obtained using ASTM C469 and the chord modulus because the stress magnitude used to define the initial tangent modulus is substantially lower than that used for the chord modulus.[105]

Although the MEPDG requires a dynamic *k*-value, the input for PCC modulus is the static chord modulus measured based on ASTM C469.[7,105] The MEPDG recommends multiplying the backcalculated PCC elastic modulus by 0.8 to convert from a backcalculated modulus to a value measured in the laboratory using ASTM C469.[7,105]

Dynamic Backcalculation

Backcalculation methods often assume a quasi-static pavement response during FWD testing. This assumes the peak deflection at each sensor occurs at the same time as the peak load. What actually happens is that the peak load occurs first, followed by the peak deflections. There is a significant lag in time between the peak load and the peak deflection for the sensor under the load plate and among the peaks of the sensor deflections. The lag can be attributed to the inertia of the pavement system and damping effects.[52]

Some of the seasonal variation in backcalculated moduli can be attributed to the use of static backcalculation procedures for the analysis of a dynamic problem. It has been found that not only the backcalculated *k*-value, but also the backcalculated PCC moduli, show seasonal variation for pavements in colder climates. The PCC elastic modulus increases when temperatures are colder. This increase is larger than what would typically be expected as a result of the change in the temperature. Part of this increase in moduli can be attributed to an increase in the mass of the plate moved by the FWD load and variation in the subgrade damping due to the effects of the temperature and moisture on the lower layers.[52]

Khazanovich presented a closed-form solution to describe the dynamic behavior of a linear elastic, homogeneous, and isotropic plate on a dense liquid foundation based on Westergaard's solution for an interior loading.[52] The damping effects of the foundation are characterized by a damping parameter, and a nondimensional mass parameter is used to adjust the dynamic pavement response as a result of the effects of the inertia of the pavement. A backcalculation procedure can be developed based on this solution.

RECOMMENDATIONS FOR MEPDG USE

The input parameters needed for the design of an overlay on top of a PCC pavement using the MEPDG that can be extracted from FWD data include the elastic modulus of the existing PCC and base layers, the subgrade *k*-value, and the PCC flexural strength. The moduli for each layer can also be backcalculated using layer elastic theory as described in chapter 4, but as previously discussed, it is better to define the stiffness of the lower layers with the backcalculated *k*-value.

Recommendations to consider when determining these inputs, based on FWD data, are provided in the following subsections.

Effective *k*-Value

As previously discussed, the ideal method for characterizing the subgrade in the MEPDG is by backcalculating the effective *k*-value, which represents the stiffness of all layers beneath the base. The user will find that the program still requires the moduli of each layer to be entered as well.(7) This is a glitch in the program. Even though the layer moduli are required to be entered, the values entered will not affect the design. The design will only consider the effective *k*-value. It is important to correctly enter in the other material characterization properties, such as the gradations of these layers, because this information is used along with the Enhanced Integrated Climatic Model (EICM) to estimate the seasonal effects on the *k*-value. When entering the *k*-value, the designer must also enter the month in which the *k*-value was measured. Seasonal corrections are then applied to the *k*-value based on the moisture conditions predicted through the EICM. It is assumed that the correction factors for the *k*-value are the same as the adjustment ratios that would be used for the moisture corrections of the resilient moduli if these values were entered instead of a *k*-value.

It is important to note that the subgrade *k*-value determined from backcalculation of FWD data is a dynamic *k*-value, which may be up to three times higher than a static value.(106) Although the MEPDG requires a dynamic *k*-value, many design procedures require the use of a static *k*-value. For instance, in the AASHTO 1993 design procedure, the backcalculated *k*-value is divided by 2.0 to obtain a static *k*-value for use in design.(32)

PCC Elastic Modulus

The elastic modulus of the existing slab must be determined for overlay designs and existing JPCP being considered for restoration. The elastic modulus can be determined by pulling a core and measuring the chord modulus based on ASTM C469 or by using FWD data to backcalculate the modulus.(105) A backcalculated modulus must be multiplied by 0.8 to convert from a dynamic to a static elastic modulus.

For an unbonded overlay, the static elastic modulus of the PCC pavement that is determined using backcalculation or laboratory testing must be adjusted to reflect the overall condition of the pavement. The modulus is adjusted based on the condition of the pavement by multiplying it by the appropriate condition factor. Condition factors for a range of pavement conditions are provided in table 8. The pavement condition is defined based on the accumulated damage. The accumulated damage for a JPCP is a function of the percent slabs cracked (see table 9) and for a CRCP, it is based on the number of punchouts per mile (see table 10). The damage factor can then be used along with table 11 to provide an estimate of the pavement condition.

For restoration, the elastic modulus is assumed not to increase over time because the strength and stiffness of old concrete will not significantly change.

Table 8. Recommended condition factor values used to adjust moduli of intact slabs.[6]

Qualitative Description of Pavement Condition	Recommended Condition Factor, *C*
Good	0.42 to 0.75
Moderate	0.22 to 0.42
Severe	0.042 to 0.22

Table 9 through table 11 provide guidelines to assess pavement condition.

Table 9. Damage estimates for JPCP based on percent slabs cracked.[6]

Percent Slabs Cracked	Damage
0	0.100–0.250
10	0.270
20	0.438
30	0.604
40	0.786
50	1.000

Table 10. Damage estimates for CRCP based on punchouts per mile.[6]

Number of Punchouts Per Mile	Damage
0	0.10–0.15
2	0.22
4	0.34
6	0.44
8	0.53
10	0.62
> 10	> 0.62

1 mile = 1.61 km.

Table 11. Pavement condition rating based on damage estimates for JPCP and CRCP.[6]

Category	Damage
Excellent	0.10–0.25
Good	0.50–0.67
Fair	1.00
Poor	> 1.00
Very Poor	>> 1.00

PCC Flexural Strength

The PCC flexural strength of the existing slab must also be determined for an overlay design. This is best determined by measuring the split tensile strength of cores and then using the split tensile strength to estimate the flexural strength. These cores can also be used to determine the slab thickness because backcalculation is highly sensitive to layer thicknesses. As a possible level 3 input, Foxworthy's correlation between the backcalculated modulus of the slab, as presented in the figure 112 equation, can be used to estimate the flexural strength.[103] This relationship was established based on limited data (n = 13) and should therefore be used with caution. The coefficient of determination for this relationship is $R^2 = 0.71$.

$$FS = 43.5\left(\frac{E}{10^6}\right) + 488.5$$

Figure 112. Equation. Estimate of flexural strength based on elastic modulus.[103]

Where:

FS = Flexural strength estimated for indirect tensile strength, lb/inch^2.
E = In situ modulus of elasticity, backcalculated from FWD data, lb/inch^2.

FUTURE RESEARCH NEEDS

Substantial improvements could be made in the current backcalculation process to help reduce the variability found between backcalculated moduli calculated at the same location at different times. The development of correction factors that account for dynamic effects would help reduce seasonal variability. These correction factors would be used to account for the effects of changes in the inertia of the pavement system and damping of the subgrade. Curling/warping of the slab, on the other hand, can increase the variability of the backcalculated moduli calculated at the same test location throughout the day, although the magnitude of the variability fluctuates seasonally. With the large quantity of rehabilitation work that needs to be performed by many State agencies, it is not feasible to limit FWD testing to those time periods when gradients are not likely to be present. For this reason, the development of correction factors that account for the effects of temperature/moisture gradients in the slab on backcalculated moduli would be very useful. Some steps have already been taken to help address these issues.[52,102,104]

CHAPTER 6. FWD DATA ANALYSIS AND INTERPRETATION—COMPOSITE PAVEMENTS

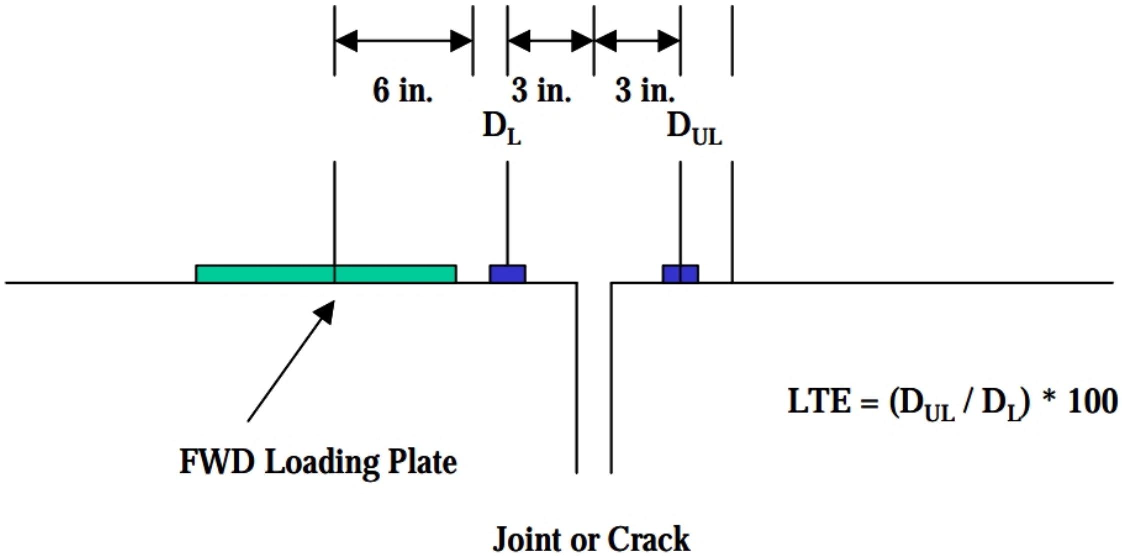
INTRODUCTION

In this chapter, FWD data analysis and interpretation of HMA/PCC pavements are discussed. Only pavements consisting of an intact PCC pavement overlaid with HMA are considered. Other types of pavements behave more as either a flexible or rigid pavement system; therefore, those methods discussed in chapter 4 or 5 are applicable. For example, if an existing PCC pavement is rubblized prior to overlaying with HMA, the resulting pavement behaves as a flexible pavement. Similarly, if an existing HMA pavement is overlaid with PCC (whitetopping), the resulting pavement is equivalent to a PCC pavement with an HMA base. Although the behavior of an HMA/PCC pavement is dominated by the underlying PCC pavement, the evaluation of FWD data collected on HMA/PCC pavements warrants special considerations because of the compression of the HMA layer that occurs when the load plate impacts the pavement surface.

Asphalt resurfacing is one of the more commonly used methods for PCC pavement rehabilitation. As of 2008, composite pavements comprised almost 23 percent of all pavement types on the National Highway System, most of which were originally PCC pavements.(107) Thus, the evaluation of HMA/PCC pavements is an important topic for State departments of transportation, especially because the life of the composite pavements can be extended even further through the application of timely and effective maintenance and rehabilitation strategies.

FWD testing plays an important role in the evaluation of HMA/PCC pavements. As with other pavement types, deflection testing can be used to evaluate the structural integrity of the underlying PCC slabs, the degree of foundation support, and the LTE across joints and cracks. As with PCC pavements, effective LTE across transverse joints and cracks is important for good performance of composite pavements. Poor LTE across joints and cracks leads to premature development of reflection cracking in the HMA overlay and faster deterioration of the reflected cracks.(108)

The single factor that distinguishes the analysis of FWD testing conducted on composite pavements from that conducted on conventional PCC pavements is the effect of compression of the HMA layer. Backcalculation analyses conducted without considering the effects of the compression that occurs in the HMA layer leads to significant errors.(54) Moreover, compression of the HMA layer also poses a problem in LTE testing for composite pavements. For example, the additional apparent deflection resulting from the compression of the HMA layer leads to lower apparent LTE than actually exists. One way to minimize this effect is to take the loaded-side deflection away from the load plate, as shown in figure 113. A similar approach is used in backcalculation to cope with the effects of the compression in the HMA layer. In this case, the recommended methods involve excluding the deflection under the load plate in the analysis.



1 inch = 25.4 mm.
D_L = Deflection loaded.
D_{UL} = Deflection unloaded.

Figure 113. Diagram. Layout of joint LTE testing on HMA/PCC.[108]

Presented in this chapter are the approaches, available software, and modeling issues for backcalculation of composite pavements, along with recommendations on the use of FWD data with the MEPDG. Note that one way to avoid all complications caused by the presence of the HMA overlay is to remove all HMA before FWD testing, but this may not always be practical or cost effective, depending on the thickness of the overlay. Thus, for planning purposes, the ability to evaluate the FWD data from composite pavements is desirable.

APPROACHES

Backcalculation for composite pavements can be accomplished using methods similar to those discussed in chapter 5 for rigid pavements. As for bare PCC pavements, the available backcalculation methods include the AREA method and the Best-Fit method.[51] Again, the main factor that complicates backcalculation of composite pavements is the compression that takes place within the HMA layer. Methods have been developed for handling the compression of the HMA layer, and it has been demonstrated that when the compression of the HMA layer is properly handled, both the pavement layer stiffness and the subgrade *k*-value can be reliably backcalculated from FWD data.

Several researchers have also demonstrated the feasibility of accomplishing backcalculation for composite pavements using ANNs to facilitate the calculation process.[109,110] The principal advantage of the ANN approach is that, based on the results of a limited number of training cases, the pavement response for any combination of input values can be determined almost instantaneously. The training cases are analyzed using the analysis tool that is capable of modeling all effects that are important for the subject pavement type. For example, for composite pavements, an analysis tool that is capable of modeling the compression in the HMA layer would be used. The ANN developed in this way can then be used as the rapid-solution tool to identify the pavement layers and subgrade moduli that best fit the deflection data. Although this is a promising technology, no software is yet available for general use.

AVAILABLE METHODS

This section presents a description of two methods of backcalculation for composite pavements that are available for general use: the outer-AREA method and the Best-Fit method. While no ready-to-use software is available for either method, they are simple to implement. The outer-AREA method, for example, can be easily implemented in a spreadsheet.

Outer AREA Method

The AREA method (discussed in chapter 5) is perhaps the most widely used method of backcalculation for rigid pavements. As described in chapter 5, AREA is the normalized area of the deflection basin.(50) Because the sensor deflections are first normalized by the deflection under the load plate (d_0) in calculating the AREA, the results of backcalculation by the AREA method are sensitive to the d_0 deflection. Therefore, the compression that occurs in the HMA layer is significant and must be taken into account. In one approach, the deflection under the load plate is corrected for the compression of the HMA layer to simulate the deflection profile of plain concrete pavement.(111) It was later found that excellent results can be obtained by performing the backcalculation without the deflection under the load plate (d_0) and instead using only the deflections from sensors 305 mm (12 inches) and farther away from the load plate.(54) This approach, which is commonly called the outer-AREA method, was first introduced by Hall et al.(51) McPeak et al. also showed that the outer-AREA method works well in determining the *k*-value from FWD testing conducted on flexible pavements for the evaluation of PCC overlays of existing HMA pavements.(112)

The calculation procedure for the outer-AREA method is the same as that for the AREA method, except that *AREA* is calculated without the d_0 deflection, with the deflections normalized by the d_{12} deflection. The *outer-AREA* for seven sensors at uniform 305-mm (12-inch) spacings is determined as shown in figure 114.

$$OuterAREA = 6 \cdot \left(1 + 2 \cdot \frac{d_{24}}{d_{12}} + 2 \cdot \frac{d_{36}}{d_{12}} + 2 \cdot \frac{d_{48}}{d_{12}} + 2 \cdot \frac{d_{60}}{d_{12}} + \frac{d_{72}}{d_{12}} \right)$$

Figure 114. Equation. *Outer-AREA* for seven-sensor configuration.

Where:

d_r = FWD deflections at distance r from the center of the load plate, mm (inches).

Following the approach introduced by Hall et al., McPeak and Khazanovich derived the relationship (shown in figure 115) between the radius of relative stiffness (ℓ) and the *outer-AREA* for the seven-sensor configuration.(51,54)

$$\ell = \left[\frac{ln\left(\frac{60 - OuterAREA}{301.8}\right)}{-0.6223} \right]^{2.501}$$

Figure 115. Equation. Relationship between the radius of relative stiffness (ℓ) and the *outer-AREA* for the seven-sensor configuration.[51,54]

Once the radius of relative stiffness is determined, the *k*-value can be determined from any of the measured deflections using the relationship shown in figure 116.[51]

$$k = \frac{\mathrm{P} \cdot d_r^*}{d_r \cdot \ell^2}$$

Figure 116. Equation. *k*-value determination.[51]

Where:

k = Modulus of subgrade reaction, MPa/mm, (lbf/inch^2/inch).
P = Applied load, N (lbf).
d_r^* = Nondimensional deflection coefficient for deflection at radial distance r from the center of the load plate.
d_r = Measured deflection at radial distance r from the load, mm (inches).
ℓ = Radius of relative stiffness, mm (inches).

The nondimensional deflection coefficient for deflection at radial distance r from the center of the load plate is given by the figure 117 equation.[51]

$$d_{\mathrm{r}}^* = a \cdot e^{\left(-b \cdot e^{\left(-c \cdot \ell\right)}\right)}$$

Figure 117. Equation. Nondimensional deflection coefficient for deflection.[51]

Where:

a, b, c = Regression coefficients (see table 12).
ℓ = Radius of relative stiffness, mm (inches).

Table 12. Coefficients for nondimensional deflection coefficients.[51]

Radial distance, inches	*a*	*b*	*c*
12	0.12188	0.79432	0.07074
24	0.11634	2.06115	0.06775
36	0.10960	3.62187	0.06568
48	0.10241	5.41549	0.06402
60	0.09521	7.41241	0.06255
72	0.08822	9.59399	0.06118

1 inch = 25.4 mm.
Note: Coefficients are for use with U.S. customary units.

Once the radius of relative stiffness (ℓ) and k-value are determined, the effective elastic modulus of the surface layers (PCC and HMA overlay combined) can be determined by rearranging the definition of ℓ as follows in figure 118 and figure 119 equations.

$$\ell = \sqrt[4]{\frac{E_e \cdot h_e^3}{12 \cdot (1-\mu^2) \cdot k}}$$

Figure 118. Equation. Radius of relative stiffness.

$$E_e = \frac{12 \cdot \ell^4 \cdot (1-\mu^2) \cdot k}{h_e^3}$$

Figure 119. Equation. Determination of elastic modulus.

Where:

E_e = Effective elastic modulus of the combined HMA and PCC layers, MPa (lbf/inch^2).
ℓ = Radius of relative stiffness, mm (inches).
h_e = Effective thickness of the HMA and PCC layers, mm (inches).
μ = PCC Poisson's ratio.
k = Modulus of subgrade reaction, MPa/mm (lbf/inch^2/inch).

Note that the backcalculated radius of relative stiffness reflects the combined stiffness of both the HMA and PCC layers. The backcalculation of a two-layer plate on grade is based on the approach proposed by Ioannides and Khazanovich.[100] In this approach, the two layers are modeled as a single equivalent plate that has the same stiffness as the original system. The equivalent plate has the thickness equaling the combined thickness of the two layers, and the modulus is selected such that the equivalent plate has the same flexural stiffness as the original system. In this situation, either a fully bonded or fully unbonded interface can be modeled under the assumptions of plate theory; thus, no through-thickness compression is assumed. In the case of HMA/PCC pavements, the two surface layers are assumed to be bonded.

For two bonded layers, the following relationship (figure 120) exists between the equivalent plate and the two actual pavement layers:[(100)]

$$\frac{E_e \cdot h_e^3}{12} = \frac{E_1 \cdot h_1^3}{12} + E_1 h_1 \cdot \left(x - \frac{h_1}{2} \right)^2 + \frac{E_2 \cdot h_2^3}{12} + E_2 h_2 \cdot \left(h_1 - x + \frac{h_2}{2} \right)^2$$

$$x = \frac{\frac{h_1^2}{2} + \beta \cdot h_2 \left(h_1 + \frac{h_2}{2} \right)}{h_1 + \beta \cdot h_2}$$

Figure 120. Equation. Relationship between the equivalent plate and the pavement layers.[(100)]

Where:

E_e = Effective elastic modulus of the combined HMA and PCC layers, MPa (lbf/inch²).
h_e = Effective thickness of the HMA and PCC layers, mm (inches) = $h_{HMA} + h_{PCC}$.
E_1 = Elastic modulus of the upper plate (HMA), MPa (lbf/inch²).
E_2 = Elastic modulus of the lower plate (PCC), MPa (lbf/inch²).
h_1 = Thickness of the upper plate (HMA), mm (inches).
h_2 = Thickness of the lower plate (PCC), mm (inches).
x = Depth to the neutral axis, mm (inches).
β = Modular ratio = E_2/E_1.

In the equation in figure 120, a similar Poisson's ratio was assumed for the two bonded layers as a matter of convenience.

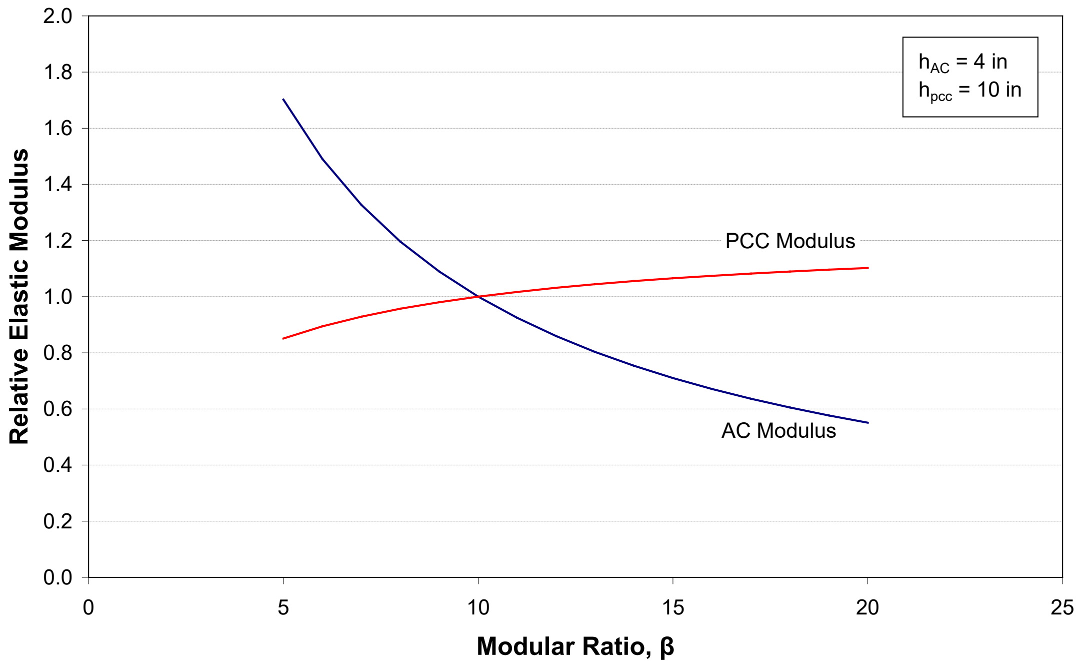
The moduli of the HMA (E_{AC}) and PCC layers can be determined from the backcalculated E_e by rearranging the figure 120 equation and substituting βE_1 for E_2 and ($h_1 + h_2$) for h_e (figure 121).

$$E_{AC} = E_1 = \frac{E_e (h_1 + h_2)^3}{h_1^3 + \beta \cdot h_2^3 + 12 \cdot h_1 \cdot \left(x - \frac{h_1}{2} \right)^2 + 12 \cdot h_2 \cdot \left(h_1 - x + \frac{h_2}{2} \right)^2}$$

$$E_{PCC} = E_2 = \beta \cdot E_1$$

Figure 121. Equation. Determination of moduli of the HMA and PCC layers.

One limitation of this approach is that the modular ratio β is unknown and must be assumed. However, as discussed in chapter 5, the backcalculated PCC modulus is relatively insensitive to the assumed value of β for a fairly wide range of values. This is illustrated in figure 122. However, the backcalculated HMA modulus could be off significantly, if the assumed value of β is incorrect.



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1 inch = 25.4 mm.

Figure 122. Graph. Sensitivity of the backcalculated moduli values to the assumed value of the modular ratio β.[100]

For the evaluation of the structural adequacy of the PCC layer, only the overall stiffness of the pavement layers and PCC modulus are significant; the HMA modulus does not directly affect the PCC stress calculation. In fact, for a composite pavement, the modulus of the existing PCC layer is the only bound layer for which backcalculated data are used as an input in the MEPDG.[7]

Best-Fit Method

In this approach, the modulus of the pavement layers and subgrade *k*-value are estimated by finding the combination that provides the best match between the calculated and measured deflections. As discussed in chapter 5, the problem is formulated as minimization of the error function, *F*, as shown in figure 123.[51]

$$F(E,k)=\sum_{i=0}^{n}\alpha_i\left(w_i - W_i\right)^2$$

Figure 123. Equation. Minimization of the error function, *F*.[51]

Where:

E = Effective elastic modulus of the combined HMA and PCC layers, MPa (lbf/inch²).
k = Modulus of subgrade reaction, MPa/mm (lbf/inch²/inch).
α_i = Weighting factors.

w_i = Calculated deflection at sensor location i, mm (inches).
W_i = Measured deflection at sensor location i, mm (inches).

The weighting factors could be set equal to 1, a function of measured deflection (W_i), or any other number. For the case of composite pavements, setting α_0 equal to 0 for the d_0 deflection and α_i equal to 1 for the remaining sensors gives equal weight to the sensors used and excludes the d_0 deflection from the calculation. The details of the Best-Fit method are given in chapter 5.

The radius of relative stiffness backcalculated by this approach again reflects the combined stiffness of the HMA and PCC layers. The same steps described for the outer-AREA method must be followed to obtain E_{HMA} and E_{PCC} from the combined pavement layer stiffness, based on the assumed value of the modular ratio β . The Best-Fit method was determined to provide more consistent results and recommended for use in an LTPP data analysis study.[51]

APPLICABILITY FOR MEPDG USE

The MEPDG evaluates HMA/PCC pavements in two steps. First, the pavement system is analyzed as a rigid pavement to model continued cracking of the underlying PCC pavement. The HMA distresses are then modeled, including thermal cracking, fatigue cracking, and rutting, as well as the International Roughness Index. Assuming that the analysis is to be conducted for a new HMA overlay, the key input parameters for this analysis obtained from FWD data are the subgrade *k*-value, E_{PCC}, and PCC modulus of rupture. Although, the PCC modulus of rupture can be estimated from backcalculated E_{PCC} using an empirical correlation, limited core testing is highly recommended to verify the values.[103] The joint LTE is also used in reflection cracking prediction, but only qualitative results are used (good or poor).

The backcalculation results for HMA/PCC pavements may contain greater variability than those for other pavement types, largely because the data may contain the results for tests conducted over joints or cracks in the underlying PCC pavement. For valid results, the locations of the joints in the underlying pavement should be identified and the testing conducted should be performed at midslab. Any significant deviations from the representative values may be an indication that the testing was conducted too close to underlying cracks or joints, and those results should be excluded in determining the average *k*-value and average *E*. For the evaluation of the structural adequacy of the underlying PCC pavement, the elastic modulus determined over the intact portion of the slab is needed.

For rehabilitation design, the MEPDG allows the dynamic backcalculated *k*-value to be entered directly.[7] Both the representative *k*-value and month of testing are entered. However, the backcalculated *k*-value is an optional input; the user is still required to enter resilient moduli for all unbound layers and subgrade. The MEPDG processes the input as usual (for new design) and determines the seasonal *k*-values based on EICM results and using the *E*-to-*k* conversion procedure. If the backcalculated *k*-value is entered, the seasonal adjustment is made using the relative *k*-values obtained through the *E*-to-*k* conversion process as the scaling factors.

For the HMA analysis, the seasonal resilient moduli are used, but no adjustment is made to account for any difference between the *k*-value from the *E*-to-*k* conversion process and the backcalculated *k*-value. To ensure consistency between the backcalculated *k*-value and the resilient moduli used in the HMA analyses, an iterative procedure may be used to adjust the subgrade resilient modulus input, rather than simply entering the backcalculated *k*-value. This involves adjusting the input subgrade resilient modulus up or down to match the *k*-value from the *E*-to-*k* conversion process and the backcalculated *k*-value for the month of the FWD testing. If the backcalculated *k*-value is entered directly, there may be some discrepancy between the *k*-value used in the PCC analysis and the resilient moduli used in the HMA analysis; however, this minor discrepancy is not likely to have any significant effect on the predicted HMA overlay performance.

RECOMMENDATIONS FOR MEPDG USE

The evaluation of FWD data is perhaps more difficult for HMA/PCC pavements than for other pavement types because of the complications introduced by the compression of the HMA layer. On the other hand, the structural adequacy is not always the principal concern for HMA/PCC pavements. Placing even a relatively thin layer of HMA (75 to 100 mm (3 to 4 inches)) has the effect of reducing the critical stresses in the underlying PCC pavement, because of the following:

- Additional stiffness provided by the HMA overlay. The HMA overlay bonds to the PCC layer to provide a significant increase in the structural capacity.
- Significant reduction in temperature gradients. On PCC pavements, the curling stresses can make up more than 50 percent of the combined stresses during the critical periods, when the pavement is subjected to high temperature gradients (positive or negative). The most severe temperature changes occur in the top 75 mm (3 inches) of the pavement. Placement of an HMA overlay significantly reduces the magnitude of the temperature gradients.
- Reduction in contact pressure. The HMA layer reduces the contact pressure on PCC pavement by spreading the load over a larger area.

The combined effect of these factors is that the critical stresses in the PCC pavement drop to a fraction of those prior to overlaying, even for a minimum-thickness HMA overlay (e.g., 75 mm (3 inches)). Thus, the performance life of HMA/PCC is typically governed by material and functional factors, not structural failure of the underlying PCC pavement. The key distresses include rutting, reflection cracking, and deterioration of the reflected cracks. The structural evaluation of HMA/PCC pavements is mainly a design check to ensure that the stresses are well within the tolerable limits. Therefore, the backcalculation procedure for HMA/PCC does not need to be as comprehensive.

Among the backcalculation procedures reviewed (including the most recent ANN-based procedures), the convenience and accessibility of the outer-AREA method outweigh the benefits of any advantages in accuracy the other methods may offer. For HMA overlay thicknesses up to about 254 mm (10 inches), the backcalculated *k*-values obtained using the outer-AREA method closely match those obtained by other, more sophisticated methods. For the backcalculated elastic moduli values, the ANN-based method offers a slight advantage in accuracy, and that approach does not involve estimating the modular ratio β (E_{PCC}/E_{HMA}) in advance.[109] For the backcalculated moduli, the Best-Fit method does not offer a significant advantage over the outer-AREA method. Both methods involve estimating the modular ratio β to resolve the backcalculated effective *E* into component moduli values; therefore, the slight advantage of the Best-Fit method in accuracy is not very meaningful.

One issue not previously discussed is the handling of the presence of a stabilized base under the PCC pavement. For the HMA surface layer, the compression of the HMA actually provides additional information that could be used to determine the HMA modulus, if a structural model that considers through-thickness compression were used (e.g., an elastic layer program). However, for the stabilized layers beneath the PCC layer, there may be no way to precisely determine the layer modulus from the surface deflection data. Given that the bending stiffness of multiple pavement layers (plates) can be represented by an equivalent plate with an effective thickness (h_e) and modulus (E_e), it is not possible to resolve the backcalculated effective modulus into component moduli without having additional information, namely the modular ratio and interface bonding condition. Because these ratios can only be estimated, the precise determination of individual layer moduli is not possible. This may be an inherent limitation of the backcalculation process for composite pavements, but because PCC pavement properties are relatively consistent over the length of a project, a limited amount of core testing can be conducted to obtain the additional information needed to verify backcalculation results.

In light of the discussion above, the following steps are recommended for the backcalculation of HMA/PCC pavements:

- Use the outer-AREA method.
- Conduct limited core testing to verify backcalculation results and the modular ratios assumed for backcalculation.

As shown in figure 122, the backcalculated E_{PCC} is not highly sensitive to the modular ratio β over a relatively wide range of β values. The following empirical correlation between HMA modulus and temperature and typical PCC moduli for the region may be used to obtain a good estimate of β (figure 124):(111)

$$\log\left(E_{AC}\right) = 6.451235 - 0.000164671 \cdot t_p^{1.92544}$$

Figure 124. Equation. Empirical correlation between HMA modulus and temperature.(111)

Where:

E_{HMA} = Elastic modulus of HMA lbf/inch²/inch.
t_p = HMA mix temperature °F.

The slab size correction need not be made for backcalculation results of HMA/PCC pavements. William showed that the deflection behavior of doweled pavements is similar to that of the infinite slabs assumed in the structural models used in backcalculation.(113) For HMA/PCC pavements, the HMA overlay contributes to the continuity of the pavement structure across transverse joints. Thus, no correction for the slab size is recommended for the backcalculation results for HMA/PCC pavements.

FUTURE RESEARCH NEEDS

Although the available analysis tools are lacking in their ability to fully model the behavior of HMA/PCC pavements, this limitation does not seriously impair the ability to evaluate FWD data for HMA/PCC pavements. That is, the limitations of the available tools do not have a significant practical impact on the ability to evaluate the characteristics of HMA/PCC pavements. The subgrade *k*-value can be determined reliably using the simplest of the available tools, the outer-AREA method. Reasonable estimates can also be made for the PCC elastic modulus using the outer-AREA method. As with the other pavement types, a limited amount of core testing is highly recommended to evaluate the material properties and to validate the pavement layer moduli determined from FWD data. The typical reasons for rehabilitating HMA/PCC pavements are commonly related to material and functional issues. The continued deterioration of the underlying PCC pavement is rarely the principal mode of deterioration for HMA/PCC pavements. The results obtained using the available analysis tools are adequate for the purposes of conducting what is essentially a design check on rehabilitation design of HMA/PCC pavements.

Nevertheless, improved models for both forward analysis and backcalculation would help more accurately model the behavior of composite pavements. The structural models based on plate theory do not correctly model the behavior of HMA/PCC pavements because the through-thickness deformations are ignored. Elastic layer programs do not yield the *k*-value typically needed for the analysis of PCC pavements. An elastic layer program that allows the subgrade to be modeled as a Winkler foundation has been developed, which may be ideal for the backcalculation of HMA/PCC pavements.[109] Researchers have also demonstrated the advantages of using ANNs for backcalculation. Development of software specifically designed for the backcalculation of HMA/PCC pavements (complete with an automated process for data screening to identify problem data points) would facilitate the process of analyzing the FWD data.

CHAPTER 7. USING FWD DATA IN THE MEPDG—CASE STUDIES

INTRODUCTION

As part of this project, case studies featuring existing pavement sections were conducted to showcase the use of FWD data for the characterization of pavement layer properties for analysis with the MEPDG. The case studies demonstrate the use of FWD and laboratory testing in deriving strength-related input properties for the MEPDG and also allow comparisons of the input properties derived from the FWD and laboratory testing. These case studies help outline the pros and cons of laboratory versus FWD testing to develop strength-related input properties of in-service pavement layers for use in the MEPDG rehabilitation design.(7)

Six case study projects were evaluated, representing the following six pavement types:

- Flexible pavement.
- Flexible pavement on rubblized PCC.
- Rigid pavement on a granular base.
- Rigid pavement on a stabilized base.
- Rigid pavement on an existing HMA pavement.
- Composite pavement (flexible pavement overlay on rigid pavement).

Although several data sources were considered (i.e., MnROAD, LTPP Program, National Center for Asphalt Technology (NCAT), and sections of roadways that have been used for special studies by State highway agencies), ultimately, data from existing LTPP pavement test sections were used in the case studies.

AVAILABLE CASE STUDY DATA—LTPP DATABASE

To accurately assess how well FWD testing results can estimate the true in situ material properties for different pavement types, backcalculated material properties needed to be compared with material properties determined by more traditional field and laboratory testing procedures. Two basic approaches for accomplishing this task were to (1) find existing data sources (databases) or past special study projects that contain both material testing and FWD results, or (2) conduct sampling and testing at new project locations to collect data that can be analyzed under this study. Because six different pavement types are included in this study, a very large research effort would be required to conduct both FWD testing and field and laboratory testing at multiple projects. Consequently, known existing data sources, including the LTPP database, the MnROAD test sections, NCAT test sections, and various special State studies, were evaluated for possible use in this study. Ultimately, it was determined that the LTPP database provided the most complete and comprehensive data source for this project, and appropriate sections were identified to fit within the established pavement categories.

Specifically, the sites in the Specific Pavement Studies (SPS)-1, -2, -5, and -6 experimental sections in the LTPP Program were reviewed for use as case study projects. Along with new flexible and rigid pavement designs (SPS-1 and -2), these sections also contained rehabilitated HMA and PCC pavements (SPS-5 and -6). Specific descriptions for each of the aforementioned

SPS categories (presented in the following sections) are reproduced here verbatim from the *Long-Term Pavement Performance Information Management System: Pavement Performance Database User Reference Guide*:(114)

> **SPS-1: Structural Factors for Flexible Pavements.** The experiment on the structural factors for flexible pavements study (SPS-1) examines the performance of specific AC-surfaced pavement structural factors under different environmental conditions. Pavements within SPS-1 must start with the original construction of the entire pavement structure or removal and complete reconstruction of an existing pavement. The pavement structural factors in this experiment include the in-pavement drainage layer, surface thickness, base type, and base thickness. The experiment design stipulates a traffic loading level in the study lane in excess of 100,000 80-kN (18-kip) [equivalent single axle loads] ESALs per year. The combination of the study factors in this experiment results in 24 different pavement structures. The experiment is designed using a fractional factorial approach to enhance implementation practicality, permitting the construction of 12 test sections at one site and a complementary 12 test sections to be constructed at another site within the same climatic region on a similar subgrade type.
>
> **SPS-2: Structural Factors for Rigid Pavements.** The experiment on the structural factors for rigid pavements study (SPS-2) examines the performance of specific JPCP structural factors under different environmental conditions. Pavements within SPS-2 must start with the original construction of the entire pavement structure or removal and complete reconstruction of an existing pavement. The pavement structural factors included in this experiment are in-pavement drainage layer (edgedrains or no edgedrains), PCC surface thickness (254 to 279 mm [10 or 11 in]), base type (dense-graded untreated granular, lean concrete, and permeable asphalt treated), PCC flexural strength (3.8 or 6.2 MPa [550 to 900 lb/in^2]), and lane width (3.66 and 4.27 m [12 to 14 ft]). The experiment requires that all test sections be constructed with perpendicular doweled joints at 4.9-m (15-ft) spacing and stipulate a traffic loading level in the lane in excess of 200,000 ESALs per year. The experiment is designed using a fractional factorial approach to enhance implementation practicality, permitting the construction of 12 test sections at one site and a complementary 12 test sections to be constructed at another site within the same climatic region on a similar subgrade type.
>
> **SPS-5: Rehabilitation of Asphalt Concrete Pavements.** The experiment on the rehabilitation of HMA pavements (SPS-5) examines the performance of eight combinations of HMA overlays on existing HMA-surfaced pavements. The rehabilitation treatment factors included in the study are the intensity of surface preparation, recycled versus virgin HMA overlay mixture, and overlay thickness. The experiment design includes all four climatic regions and the condition of the existing pavement. The experiment design stipulates a traffic loading level in the study lane in excess of 100,000 80-kN (18-kip) ESALs per year.

SPS-6: Rehabilitation of Jointed Portland Cement Concrete Pavements (JPCP). The experiment on the rehabilitation of JPCP pavements (SPS-6) examines the performance of seven rehabilitation treatment options as a function of the climatic region, type of pavement (plain or reinforced), and the condition of the existing pavement (note: an eighth scenario looks at just applying routine maintenance). The rehabilitation methods include surface preparation (limited preparation or full concrete pavement restoration) with a 102 mm- (4 in-) thick HMA overlay or without an overlay, crack/break and seat with two HMA overlay thicknesses (102 or 203 mm [4 or 8 in]), and limited surface preparation with a 102 mm- (4 in-) thick HMA overlay with sawed and sealed joints. (p. 142–143)

An assessment of the LTPP data availability using the LTPP DataPave (both online and disk-based) interface revealed 18 SPS-1 sections, 14 SPS-2 sections, 18 SPS-5 sections, and 14 SPS-6 sections that met all of the outlined LTPP criteria. In addition to those documented sections, the LTPP database also contained many "supplemental" sections that were also included in the LTPP database. Supplemental sections were typically State transportation department experimental projects that focused on investigating only one or two of the variables rather than all of the specific variables outlined in the LTPP project. The LTPP data also included key material testing results, performance monitoring data, climatic information, traffic loading data, and seasonal testing information.

CASE STUDY SELECTION

Although a number of different potential sections were considered for the case studies, it was determined that a relatively small number of sections contained data for all of the input information required in the MEPDG. Even those ultimately selected for the case studies (which were those having the most complete data) often did not have all of the desired data.

This section describes the pavement projects that were selected for the case studies, as summarized in table 13. The detailed case studies themselves are included in volume II of this report.

Table 13. Summary of selected case study pavement sections.

Case Study	LTPP Section	Location	Highway/ Facility Type	Climate
Flexible	30-0100	Great Falls, MT	Interstate (rural)	Dry-freeze
Flexible on rubblized PCC	17-0600	Pesotum, IL	Interstate (rural)	Wet-freeze
PCC on granular	32-0200	Lander County, NV	Interstate (rural)	Dry-freeze
PCC on stabilized	05-0200	Saline County, AR	Interstate (rural)	Wet-nonfreeze
PCC on flexible	30-0100	Great Falls, MT	Interstate (rural)	Dry-freeze
Composite (HMA/PCC)	19-0600	Des Moines, IA	Interstate (rural)	Wet-freeze

Case Study 1: Flexible Pavement

As discussed in chapter 4, the required input material properties for HMA pavements in the MEPDG relevant to the use of FWD data and backcalculation results are the following:

- Time-temperature dependent dynamic modulus E^* for the HMA layer(s).
- Resilient moduli for the base/subbase and subgrade materials.
- Elastic modulus of the bedrock, if present.

The LTPP database contained 18 applicable SPS-1 sections representing all 4 climatic zones in the United States (i.e., wet-freeze, wet-nonfreeze, dry-freeze, and dry-nonfreeze). After reviewing the preliminary flexible pavement sections, LTPP section 30-0100 (30-0113 specifically) was selected as a flexible pavement rehabilitation case study. The test section, located on I-15 near Great Falls, MT, is a flexible pavement cross section with a HMA surface and aggregate base layer over subgrade. The original pavement, a 102-mm (4-inch) HMA surface on a 203-mm (8-inch) aggregate base, was constructed in 1997. Rehabilitation and repair work was performed in 2003 and 2004 (construction numbers 2 and 3, respectively).

This section was representative of the following selection factors:

- Dry-freeze climate zone.
- Principal interstate—rural functional class.
- “Fair” pavement condition.
- Sand subgrade classification.
- No (or deep) rigid layer.

Case Study 2: Flexible Pavement on Rubblized PCC

The MEPDG analysis of a flexible HMA overlay placed on a rubblized PCC pavement is very similar to the analysis of an HMA pavement placed on an aggregate base. The required MEPDG input material properties for this pavement type that are relevant to the use of FWD data and backcalculation results are the following:

- Time-temperature dependent dynamic modulus E^* for the HMA overlay layer.
- Elastic modulus of the rubblized PCC layer.
- Resilient moduli for the base/subbase and subgrade materials.
- Elastic modulus of the bedrock, if present.

There were 17 rubblized test sections (representing 7 States) included in the LTPP database as SPS-6 sections. Section 17-0600 was selected for the flexible pavement on rubblized PCC case study. Section 17-600, located on I-57 near Pesotum, IL, consists of 14 test sections, with test section 17-0663 being selected for use in the case study.

The test section 17-0663 PCC pavement was originally constructed in 1964 (construction number 1), with rehabilitation/repair work performed in 1990 (May and June) and 1997 (construction numbers 2, 3, and 4, respectively). Based on LTPP core data, the average original pavement cross section consisted of 254 mm (10 inches) of PCC (jointed reinforced concrete pavement (JRCP)) and 178 mm (7 inches) of aggregate base. The JRCP was rubblized and a HMA overlay was placed in 1990.

This section was representative of the following selection factors:

- Wet-freeze climate zone.
- Rural principal arterial–interstate functional class.
- “Fair” pavement condition.
- Fine-grained soil subgrade classification.
- No (or deep) rigid layer.

Case Study 3: Rigid Pavement on Granular Base

When using the MEPDG procedure to develop an overlay design for an existing rigid pavement on a granular base, the following four-layer strength-related properties are required:

- Elastic modulus of the existing PCC layer.
- Resilient modulus of the existing base layer(s).
- Subgrade *k*-value.
- PCC flexural strength.

The preliminary review of data sources indicated that such data were readily available in the LTPP database. For this pavement type, the SPS-2 database contained 14 applicable projects. Each of these SPS-2 projects contained four different test sections with aggregate bases. (Note that the four test sections represented unique combinations of pavement thickness, PCC flexural strength, and lane width.)

LTPP test section 32-0200 was selected for the rigid pavement on granular base case study. The test section was a part of I-80 in Lander County, NV. According to the LTPP data, the JPCP consists of a 295-mm (11.6-inch) PCC surface layer, a 145-mm (5.7-inch) dense graded aggregate base, a 513-mm (20.2-inch) granular subbase, and a subgrade of which the top 305 mm (12 inches) were treated with lime.

This section was representative of the following selection factors:

- Dry-freeze climate zone.
- Principal Interstate–Rural functional class.
- “Poor” pavement condition.
- Sandy silt subgrade classification.
- No (or deep) rigid layer.

Case Study 4: Rigid Pavement on Stabilized Base

When using the MEPDG procedure to design a rigid pavement on a stabilized base, the same layer strength-related properties required for the rigid pavement on a granular base pavement type are required. Those properties include the following:

- Elastic modulus of the existing PCC layer.
- Elastic modulus of the existing base layer(s).
- Subgrade *k*-value.
- PCC flexural strength.

Data required to verify the recommended FWD guidelines for this pavement type were also readily available from the SPS-2. For this pavement type, SPS-2 database contained 14 different projects, each of which contained 4 different test sections with a lean concrete base, and 4 test sections with a permeable asphalt treated base. Similar to the PCC pavement on a granular base, the four different test sections associated with each stabilized base type represented unique combinations of pavement thickness, PCC flexural strength, and lane width.

LTPP project 05-0200, located on I-30 in Saline County, AR, was selected for the rigid pavement on stabilized base case study. The section consisted of 12 test sections, with test section 05-0218 selected for the case study. The PCC pavement was originally constructed in September 1993. Based on LTPP core data, the typical cross section of section 05-0218 consisted of 203 mm (8 inches) of PCC (surface layer), 178 mm (7 inches) of a treated base (lean concrete), 102 mm (4 inches) of a granular layer, a woven geotextile interlayer, and the subgrade.

Section 05-0218 was a rigid pavement representative of the following selection factors:

- Wet-nonfreeze climate zone.
- Rural principal arterial–interstate functional class.
- "Poor" pavement condition.
- Gravel subgrade classification.

Case Study 5: Rigid Pavement on HMA Pavement

The design of a rigid pavement on top of an existing HMA pavement (i.e., "whitetopping") is equivalent to designing a PCC pavement with an HMA base, so the inputs required by the MEPDG are similar to those required by the other rigid pavement types. Specifically, the layer strength-related inputs required in the MEPDG approach include the following:

- Elastic modulus of the existing HMA layer
- Elastic modulus of the existing base layer(s)
- Subgrade *k*-value.

Data required for the rigid pavement overlay on an HMA pavement were obtained from the test section (30-0113) used in case study 1. Specific details on that pavement and data were provided in the previous description of case study 1.

Case Study 6: Composite Pavement

A *composite pavement* is defined as a PCC pavement that is overlaid with one or more HMA overlays. When analyzing composite pavements using the MEPDG procedure, the same input values required by the previously discussed cases studies were also required here.[7] Specifically, the required inputs for such an analysis were the following:

- Elastic modulus of the existing PCC layer.
- Flexural strength of the existing PCC layer.
- Resilient moduli for the base/subbase and subgrade materials.

Although the load bearing of an HMA/PCC pavement is dominated by the underlying PCC pavement, the evaluation of FWD data collected on HMA-overlaid PCC pavements warrants special considerations because of the compression of the HMA layer. Therefore, for this pavement type, it was important to find case study projects in which the materials properties and construction procedures were well documented.

Fourteen SPS-6 projects were included in the LTPP database. As mentioned previously, each SPS-6 project was used to investigate eight different rehabilitation options for JPCP pavements, three of which involved applying HMA overlays after completing various levels of pre-overlay repair. Specifically, the three rehabilitation options deemed useful to this study were the following:[115]

- **SPS-6 section 03:** Minimum restoration followed by a 100-mm (4-inch) HMA overlay. (Minimum restoration typically consists of routine maintenance, which includes limited patching (filling potholes), crack repair and sealing, and stabilization of joints.)

- **SPS-6 section 04:** Maximum restoration followed by a 100-mm (4-inch) HMA overlay with sawed and sealed joints. (Maximum restoration included activities such as subsealing, subdrainage, joint repair, full-depth repairs with restoration of load transfer, and shoulder rehabilitation.)

- **SPS-6 section 06:** Maximum restoration followed by a 100-mm (4-inch) HMA overlay.

In addition to the general SPS-6 sections, 13 additional supplemental SPS-6 sections representing 9 States were also available in the LTPP database.

Project 19-0600 from the LTPP SPS-6 database served as the basis for the composite pavement case study. The 19-0600 project was located on I-35 near Des Moines, IA, and test section 19-0659 was selected as a composite pavement rehabilitation case study. The test section was a composite pavement cross section with a HMA surface (overlay) on a JRCP and granular base layer over subgrade. The original JRCP pavement, a 254-mm (10-inch) PCC slab on a 203-mm (8-inch) base with 23.3-m (76.5-ft) joint spacing, was constructed about 1965. A 102-mm (4-inch) HMA overlay was placed in 1989.

This section was representative of the following selection factors:

- Wet-freeze climate zone.
- Principal arterial–Interstate (rural) functional class.
- “Fair” pavement condition.
- Clay subgrade classification.
- No (or deep) rigid layer.

SUMMARY OF CASE STUDY RESULTS

The selected project sites were used to backcalculate layer properties from deflection data and to develop design inputs in the MEPDG design procedure. This was done to illustrate the use of FWD-based data in the MEPDG and to evaluate the possible differences in results between FWD-based inputs and laboratory (and/or default) design inputs. Brief summaries of the significant findings from each case study are provided in the following sections, with details of the case studies presented in volume II of this report.

For the flexible backcalculation analyses, the following three backcalculation programs were used: MODTAG, MICHBACK©, and EVERCALC©. These programs were selected because they were widely used, readily available, and not proprietary. Three programs were employed to look into the effect of different inverse routines on backcalculated parameters and ultimately on rehabilitation design. For the rigid pavement backcalculation, the AREA method and outer-AREA method (for HMA/PCC pavements) were used because these had an established basis for use and the closed-form equations were easily implemented in a spreadsheet.

Case Study 1: Flexible Pavement

Three backcalculation programs (MODTAG, MICHBACK©, and EVERCALC©) were used to analyze FWD deflection data from various test locations (stations) within the project. The deflection data showed considerable variability within the project. In addition, the RMS values obtained from the three backcalculation programs were generally very high. It is recommended that agencies conduct FWD tests at multiple locations and use the average of backcalculated layer moduli for the section. This should provide consistent service life predictions irrespective of the backcalculation program used and therefore result in a similar recommended overlay design.

For this case study, the MEPDG results indicated that surface-down cracking was critical in the rehabilitation design of an HMA overlay over existing HMA pavements. Nevertheless, a 76-mm (3-inch) HMA overlay was satisfactory for nearly all of the input combinations. Within the ranges identified, the selection of inputs was more critical as one approaches the lower values for any layer.

In addition, the following recommendations are made:

- Until ongoing studies are completed, the correction factors for backcalculated properties of laboratory values developed by Von Quintus and Killingsworth should be applied to the backcalculation results.[87,116]
- While the procedure needs to be verified, for the time being, an equivalent frequency of 30 Hz should be used. This is calculated as 1/(FWD pulse duration) = 1/0.033 s = 30 Hz. Although this formula is technically incorrect, it is compatible with the equivalent frequency used for calculating $E*$ to be used in MEPDG.

Case Study 2: Flexible Pavement on Rubblized PCC

The same three backcalculation programs used for the flexible pavement case study were used to analyze FWD deflection data from various test locations (stations) within the project. The deflection data showed considerable variability within the project. In addition, the RMS values obtained from the MICHBACK© backcalculation program were very high. As with the previous case study, it is recommended that agencies conduct FWD tests at multiple locations and use the average of backcalculated layer moduli. This should provide consistent service life predictions irrespective of the backcalculation program used, and therefore result in a similar recommended overlay design.

For HMA overlays on rubblized concrete pavements, the critical performance measure in the MEPDG software was surface rutting. The required HMA overlay thickness to achieve a 90-percent reliability level was 178 mm (7 inches) for all analyses. However, if surface rutting was addressed through maintenance at some intermediate year (less than 20 years), a thinner HMA overlay would be appropriate.

The surface rutting predictive model was mainly sensitive to HMA overlay thickness and HMA and rubblized layer moduli. Therefore, care should be taken in selecting the modulus for the rubblized layer. In this particular case, the rubblized PCC modulus was slightly lower than the suggested level 3 input in the MEPDG. It may be useful to combine the rubblized layer with the existing unbound base layer when using some backcalculation programs (for example MICHBACK©, in this case). The remaining unbound layer moduli should be entered as the adjusted backcalculated values (until ongoing studies are completed, the correction factors developed by Von Quintus and Killingsworth should be applied to the backcalculation results), but the existing HMA modulus determined by backcalculation cannot currently be used.[115]

Case Study 3: Rigid Pavement on Granular Base

The backcalculation of PCC elastic modulus and modulus of subgrade reaction was only possible for the first half of section 32-0203 (station 0 to 75.6 m (0 to 248 ft)) because of the inconsistency in the deflection basin profiles for the rest of the stations. However, the information included in the LTPP database for this section was sufficient to determine the load transfer characteristics and the support conditions for the entire section.

The variation of the backcalculated *k*-value and the PCC elastic moduli along the section can be an indication that the backcalculation technique was capturing the overall stiffness, but it appeared to be overestimating the *k*-value and, therefore, underestimating the elastic modulus of the concrete for several of the test locations. The average laboratory-measured static PCC modulus 19,292,000 kPa (2.8 million lb/inch^2) showed a good correlation with the average backcalculated static PCC elastic modulus of 21,359,000 kPa (3.1 million lb/inch^2); however, the variation of the backcalculated values along the section was 22 percent, which was substantially higher than the 15 percent typically assumed acceptable.

The composite *k*-value for the section was obtained based on the modulus that was determined for each layer and compared with the composite *k*-value from the backcalculation. The resulting static composite *k*-value was approximately 80 kPa/mm (300 lb/inch^2/inch) for a base with 145-mm (5.7-inch) thickness, which was much higher than the corresponding static composite *k*-value obtained from the backcalculation (50 kPa/mm (190 lb/inch^2/inch). This was in keeping with other observations that the backcalculated modulus of the unbound materials tended to be smaller than the laboratory-determined modulus values.

The LTE values along the section were above a minimum acceptable level of 75 percent. This high level was constant over time, and it was not affected by cold temperatures, indicating the doweled joints were performing well. According to the void detection analysis, it appears that voids were not present beneath the slabs in this section of roadway. However, there was a possibility that some erosion began to develop in 2002.

The studied section was in poor condition, with a large number of transverse cracks. Not all of the necessary MEPDG design inputs were available in the LTPP database for the studied section 32-0203, or any other 32-0200 sections.[7] Therefore, based on the LTPP data, appropriate inputs for rehabilitation designs in the MEPDG are discussed in detail in the case study.

Three different overlay rehabilitation designs were developed for this project: an HMA overlay, an unbonded JPCP overlay, and a bonded JPCP overlay. The thinnest design thickness (102 mm (4 inches)) was obtained by using a bonded JPCP overlay because it used the remaining structural capacity of the existing road. The design thickness of the unbonded overlay (178 mm (7 inches)) was determined considering the fact that the unbonded JPCP overlays worked independently and thus some restraints in thickness must be provided to guarantee its structural capacity. The MEPDG provided an unreasonably thick HMA overlay design (305 mm (12 inches)), with the critical performance parameter being surface rutting. Even when various modifications were made to the HMA mix design, the resulting HMA overlay thickness was considered unreasonably thick. If surface rutting were addressed through maintenance at some intermediate year (less than 20 years), then a thinner HMA overlay would be appropriate.

No difference in the design thickness was found among the three design alternatives (laboratory/material default values, adjusted backcalculated *k*-value and backcalculated PCC elastic modulus, and backcalculated PCC elastic modulus and *k*-value), indicating the reliability of using both the backcalculated dynamic elastic modulus for the PCC layer and the dynamic *k*-value for the supporting layers in the MEPDG design.

The backcalculated *k*-value that represents the composite stiffness of all layers beneath the slab can be directly entered into the MEPDG without having a significant influence on the design thickness for the pavement structure analyzed. However, this does not definitively mean that the MEPDG takes the stiffness of the base layer into account in the *k*-value. It could be either due to the insensitivity of the design thickness on the input *k*-value or because the granular base contributes little to the composite stiffness of all layers beneath the slab. Other observations made regarding the calculation of *k*-value within the MEPDG program included the following:

- For the HMA overlay design, varying the subbase stiffness had very little influence on the determined *k*-value, which seemed to suggest the stiffness of the subbase layer was not taken into account in the calculated *k*-value. In addition, the reported *k*-values in the output file were identical for cases with varying base layer stiffness, indicating the stiffness of the base was likely not included in the *k*-value calculation. Furthermore, it appears that the MEPDG ignored the entered dynamic *k*-value and calculated the *k*-values based on the entered layer moduli because the summarized values were the same regardless of what dynamic *k*-value was entered or when a dynamic *k*-value was not entered.

- For the unbonded PCC overlay, additional design runs seemed to indicate that the stiffness of the interlayer and the existing PCC was not considered in the calculation of the *k*-value. It does appear that the base layer was taken into account by the difference in *k*-values when using a stiff and weak base layer. In addition, the *k*-values agreed well with the entered dynamic *k*-value, suggesting that the MEPDG used the entered value for unbonded PCC overlay designs.

- It appeared that the modulus of the base layer was considered in the calculation of the *k*-value for bonded JPCP overlay designs, which agreed with the assumptions made for bonded PCC overlays. It was also apparent that the calculated *k*-values matched the entered dynamic *k*-value.

To summarize the preceding discussion, no explicit conclusion could be drawn with respect to defining the layers used in the composite effective dynamic *k*-values calculated within the MEPDG for each type of overlay design because of the conflicting results obtained. However, it can be concluded that the manually entered *k*-value were used for unbonded JPCP and bonded JPCP overlay designs but not for the HMA overlay design. No appreciable difference in terms of the design thickness was found among the three design alternatives indicating the reliability of using the backcalculated dynamic (or static) elastic modulus for the PCC layer and the dynamic *k*-value for the supporting layers in the MEPDG design. Furthermore, it was also found that the backcalculated *k*-value that represented the composite stiffness of all layers beneath the slab could be directly entered into the MEPDG without significantly influencing the design thickness for the pavement structure analyzed. However, this does not definitively mean that the MEPDG took the stiffness of the base layer into account in the *k*-value. It could be either due to the insensitivity of the design thickness on the input *k*-value or because the unstabilized granular contributes very little to the composite stiffness of all layers beneath the slab.

Case Study 4: Rigid Pavement on Stabilized Base

The most recent FWD test data for section 05-0218 were from 2004. Using the LTPP database, input parameters required for the overlay design were compiled. Not all required data were available for section 05-0218; therefore, sometimes data from other sections of project 05-0200 were used.

Using the FWD test data, the PCC elastic modulus and the dynamic effective *k*-value were backcalculated for the section. These values were calculated to be 42,580,200 kPa (6.18 million lb/inch^2) and 100 kPa/mm (371 lb/inch^2/inch), respectively. The backcalculated PCC elastic modulus was later corrected for the effect of the stabilized base layer. The corrected value of the PCC elastic modulus obtained was 16,536,000 kPa (2.4 million lb/inch^2), which was closer to the measured value of 24,804,000 kPa (3.6 million lb/inch^2).

As with case study 3, the following three different overlay rehabilitation designs were developed for this project: an HMA overlay, an unbonded JPCP overlay, and a bonded JPCP overlay. The HMA overlay analysis produced an unreasonable overlay thickness of 381 mm (15 inches) for all underlying property scenarios. Several variations in mix properties were analyzed (such as varying binder grades, binder content, air void content, and so on) in efforts to minimize the resultant thickness, but rutting continued to control the design results. If rutting was not considered, an HMA overlay thickness of 102 mm (4 inches) was obtained that successfully met the other performance criteria.

The unbonded PCC overlay analysis produced a 178-mm (7-inch) PCC overlay using approaches based on the following: 1) laboratory-based inputs and 2) backcalculated PCC moduli and *k*-values. However, when using a modified PCC elastic modulus and base modulus (accounting for the stabilized base), a much thicker unbonded overlay (381 mm (15 inches)) was obtained. The modified elastic modulus value was as low as 16,536,000 kPa (2.4 million lb/inch^2), which did not provide enough structural capacity to adequately carry the future traffic loadings. The bonded PCC overlay analysis produced a 76-mm (3-inch) PCC overlay for all underlying property scenarios. Other observations made during the design analysis process included the following:

- For the HMA overlay design, it seemed impossible to draw a definite conclusion about the constituents contributing to the *k*-value being reported in the design output (and assumed to be used in the design calculations). The entered dynamic *k*-value did not appear to be used in the determination of *k*-value. The difference in the calculated *k*-values for low and high base stiffnesses was so slight that it appears that the base layer stiffness was not considered in the calculated *k*-value.

- With the unbonded JPCP overlay design, a noticeable difference was found in the calculated *k*-value between a low and high existing PCC modulus, which might indicate that the stiffness of the existing PCC was involved in the calculation of the *k*-value.

- The modulus of the base appeared to be considered in the calculation of the *k*-value for bonded PCC overlay designs, which agreed with the assumptions listed in the MEPDG. It was also apparent that the calculated *k*-values matched the entered dynamic *k*-values.

- For this case study, a low PCC modulus only influenced the unbonded PCC overlay thickness requirement, more than doubling the required thickness. The PCC modulus did not appear to influence the other overlay design options.

Case Study 5: Rigid Pavement on HMA Pavement

Rigid pavement on HMA pavement design was carried out essentially as a new rigid pavement design. The analysis produced a 178-mm (7-inch) thick concrete overlay for this section based on a 20-year design period.

Based on the results of the sensitivities conducted, it appeared that the MEPDG changed the original flexible pavement structure into an equivalent structure. The equivalent structure consisted of PCC with the same properties as the PCC overlay on top of a base layer with the same properties as the existing HMA; all of this was then supported by Winkler springs with a stiffness equal to the composite *k*-value established using all layers beneath the existing HMA.

Case Study 6: Composite Pavement

For this project, an HMA overlay thickness of 318 mm (12.5 inches) was required to achieve a 90-percent reliability, with top-down cracking acting as the controlling criterion. This was unreasonably thick and could be reduced by assuming a higher level of allowable top-down cracking distress or perhaps additional manipulation of the HMA mixture properties.

In this case study, the selection of layer property inputs from backcalculation values had minimal influence on the overall design results. It appeared that the input dynamic *k*-value was not considered in the design. However, the MEPDG documentation indicates the HMA overlay performance is based on flexible pavement design, so this was consistent in that the design was controlled by the HMA overlay properties. Although the design was controlled by HMA overlay properties, the research team makes the following recommendations:

- The design was insensitive to the trial PCC moduli. Therefore, the use of the dynamic backcalculation adjustment factor (0.8 for the PCC modulus) can continue until new ones are developed or an agency develops specific values.[6] The MEPDG program appeared to use static PCC elastic modulus values as entered inputs but the output files suggested it reverted to a dynamic value. Additional adjustment of the PCC modulus based on the overall pavement condition could be made, but it does not appear that this influence had an effect unless the pavement was in very poor condition.

- The established modular ratios should be used unless specific testing data are available to determine project specific ratios.[27]

- The subgrade modulus input should be correlated to the static backcalculated *k*-value. Both the flexible and rigid design analyses for a composite pavement appeared to use the input subgrade modulus, so this value should be based on the determined support conditions. The program did not appear to use the input dynamic *k*-value.

- Including an aggregate base layer and determining a corresponding dynamic subgrade *k*-value did not appear to have a significant effect on the design results. The addition of a layer would also seem to suggest a change in the climatic adjustment, but it did not appear to be significant, particularly when considering the overall design results.

SUMMARY

Six case studies, using data from in-service pavements, were used to evaluate the way that FWD deflection data were used in the rehabilitation portion of the MEPDG. Specifically, deflection data and backcalculation results were used to characterize the existing HMA, PCC, stabilized and unstabilized bases, and aggregate and subgrade properties in the MEPDG design program. When laboratory testing results were compared with FWD results, the final designs were relatively insensitive to the differences in characterization of existing layer inputs; new material properties tended to control design results. Details of these case studies are found in volume II of this report, but this chapter has presented some of the primary observations and trends. While many of the designs were controlled by new pavement material properties, recommendations on how to use backcalculation data in the MEPDG were discussed.

CHAPTER 8. FWD DATA ANALYSIS AND INTERPRETATION

INTRODUCTION TO THE GUIDELINES

One of the primary objectives of this project was to develop guidelines for conducting FWD testing, analyzing the resultant deflection data, and interpreting the results for pavement rehabilitation design. Volume III of this report is a standalone guide that addresses each of those topics, providing general guidance and direction throughout the entire testing and analysis process. It is organized into the following chapters:

- **Chapter 1. Introduction:** This chapter is a brief overview of the reasons for performing deflection testing and the information that can be obtained from deflection-testing data. It also explains the purpose and organization of the guidelines.

- **Chapter 2. Deflection Testing Guidelines:** This chapter offers specific guidance for conducting FWD testing, including recommendations for selecting sensor configuration/spacing, load levels, test locations and intervals, and measuring temperature (air, pavement surface, and in-pavement). It also presents some of the key factors affecting pavement deflections and data checks that can be performed to check the validity of the pavement deflection data.

- **Chapter 3. General Backcalculation Guidelines:** This chapter provides guidelines for backcalculating deflection data, including tips for modeling typical and atypical pavement structures (such as when to combine or separate pavement layers, when to set the layer moduli, how to handle different bonding conditions, and so on), typical/default input values, reasonable outputs, and how to identify outliers. This chapter also includes results of studies that have verified backcalculated results with instrumented pavement sections and an example illustrating and interpreting the results of a commonly used backcalculation program for flexible pavement.

- **Chapter 4. Use of Deflection Data in the MEPDG:** This chapter summarizes mechanistic-empirical pavement design principles, provides an overview of the MEPDG, and summarizes the inputs (including deflection data) for use in the MEPDG for the design of rehabilitated pavements.

- **Chapter 5. Summary:** This chapter briefly recaps the information contained in the guidelines.

The guidelines are structured to be a "how-to" or "step-by-step" guide for pavement engineering practitioners.

RECOMMENDATIONS FOR NEXT-GENERATION TOOLS AND ANALYSES

Backcalculation techniques and the available tools have evolved significantly since the early days of pavement deflection testing. Although analysis methods and tools continue to improve, a number of shortcomings have yet to be overcome, and several cutting-edge advancements have yet to make it to mainstream use. This section provides commentary on areas requiring improvements in the backcalculation and interpretation process, along with a discussion of impending advancements, as observed during the course of this project.

For flexible pavements, one should ideally be able to determine a curve of HMA layer modulus as a function of frequency using a (dynamic) frequency-based backcalculation algorithm, which would give a more direct estimation of the HMA layer modulus with frequency from actual field conditions rather than relying on a laboratory-derived curve such as the Witczak equation. However, care should be taken in interpreting and using such data with the existing MEPDG performance predictions because they were calibrated using laboratory-derived moduli. Although dynamic backcalculation methods can backcalculate layer moduli and thicknesses accurately from synthetically generated FWD data for pavement systems with three or more layers, they present some serious challenges when using field data.(46) The frequency-domain method can lead to large errors if the measured FWD records are truncated before the motions fully decay in time. Also, dynamic, time-domain backcalculation algorithms cannot directly determine the HMA modulus as a function of frequency. They either assume a constant HMA modulus (similar to static backcalculation) or a prescribed function of the HMA layer modulus with frequency (e.g., linear relation in the log-log space).

The procedures available for the evaluation of the structural support conditions under a rigid pavement could potentially be improved to enhance the current analytical capabilities. For example, most methodologies assume a flat slab, but temperature and moisture gradients have been shown to affect the interpretation of FWD data when evaluating support conditions as well as monitoring joint performance (particularly for nondoweled joints) and detecting voids.(102) Improved guidelines are needed to define when FWD testing can be performed so data interpretation is not influenced as much by temperature/moisture/construction gradients. However, a procedure to account for this influence when testing a slab that is not flat would also be helpful.

For rigid pavements, substantial improvements could be made in the backcalculation process to help reduce the variability found between backcalculated moduli calculated at the same location at different times. The development of correction factors that account for dynamic effects would help reduce seasonal variability. These correction factors would be used to account for the effects of changes in the inertia of the pavement system and damping of the subgrade. Curling/warping of the slab, on the other hand, can increase the variability of the backcalculated moduli calculated at the same test location throughout the day, although the magnitude of the variability will fluctuate seasonally. With the large quantity of rehabilitation work that needs to be performed by many State agencies, it is not feasible to limit FWD testing to the morning hours when gradients are not likely to be present. For this reason, the development of correction factors that account for the effects of temperature and moisture gradients in the slab on backcalculated moduli would be very useful. Some steps have already been taken to help address these issues.(104,52,102)

Alternatively, a dynamic analysis could be performed. As discussed, dynamic backcalculation has been applied successfully for flexible pavement analysis, and advances are being made for rigid pavements as well. However, the use of a static analysis should be evaluated to determine whether performing a dynamic analysis is necessary for rigid pavements. Although a static analysis is being applied to a dynamic load condition, it might not be necessary to backcalculate the *k*-value based on a dynamic analysis unless it were shown to help decrease unexplained variation. It should be noted that the dynamic *k*-value is required as input for the MEPDG program.

Another limitation in modeling rigid pavements when evaluating support conditions is that it is based on a slab of infinite length and width. Crovetti developed procedures, which were later modified by Hall et al., to adjust for the effective slab size but this still does not consider the effect the transverse or longitudinal joint load transfer and edge support have in increasing the effective slab size.[104,51] This is another potential area for which improvements are needed.

For HMA/PCC pavements, although the available analysis tools are lacking in their ability to fully model the behavior, this limitation does not seriously impair the ability to evaluate FWD data for these types of pavements. That is, the limitations of the available tools do not have a significant practical impact on the ability to evaluate the characteristics of HMA/PCC pavements. The subgrade *k*-value can be determined reliably using the simplest of the available tools, the outer-AREA method. Reasonable estimates can also be made for the PCC elastic modulus using the outer-AREA method. As with the other pavement types, a limited amount of core testing is highly recommended to evaluate the material properties and to validate the pavement layer moduli determined from FWD data. Core testing can also assist with determining appropriate modular ratios for the HMA and PCC used in the analysis to determine individual layer properties.

The typical reasons for rehabilitating HMA/PCC pavements are related to material and functional issues. The continued load-related deterioration of the underlying PCC pavement is rarely the principal mode of deterioration for HMA/PCC pavements. The results that can be obtained using the available analysis tools are adequate for the purposes of conducting what is essentially a design check on rehabilitation design of HMA/PCC pavements. Nevertheless, improved models for both forward analysis and backcalculation would help more accurately model the behavior of composite pavements. The structural models based on plate theory do not correctly model the behavior of HMA/PCC pavements because the through-thickness deformations are ignored. Elastic layer programs do not yield the *k*-value typically needed for the analysis of PCC pavements. An elastic layer program that allows the subgrade to be modeled as a Winkler foundation has been developed, which may be ideal for the backcalculation of HMA/PCC pavements, although it is currently a proprietary program.[109] Researchers have also demonstrated the advantages of using ANNs for backcalculation. Development of software specifically designed for the backcalculation of HMA/PCC pavements (complete with an automated process for data screening to identify problem data points) would facilitate the process of analyzing the FWD data.

CHAPTER 9. SUMMARY

The need to accurately characterize the structural condition of existing pavements has increased with the recent development and release of the MEPDG. An integral part of this process is the accurate characterization of material properties of each layer in the pavement structure, which can be determined either through laboratory testing procedures or through the testing of in situ pavement structures using various techniques, such as the FWD. In the past few decades, FWD testing has become a routine pavement evaluation method, and deflection data collected by the FWD can be quickly and easily used to characterize the properties of the paving layers, which are required inputs into the MEPDG for new flexible pavement design, new rigid pavement design, and rehabilitation design.

This document is part of a three-volume report investigating the use of the FWD as part of mechanistic-empirical pavement design and rehabilitation procedures. In this volume, general pavement deflection-testing procedures and commonly used deflection analysis approaches and backcalculation programs are reviewed. Specific procedures for interpreting and analyzing deflection data for flexible, rigid, and composite pavement structures are described, along with specific modeling issues unique to each pavement structure. The relevance of the different procedures and approaches to the current MEPDG are explored, giving rise to the examination of the use of FWD testing results in six case studies. These six case studies used pavement sections from the LTPP database containing sufficient design, construction, and testing data results (laboratory testing and FWD testing) as a means of assessing the way that FWD deflection data are used in the rehabilitation portion of the MEPDG. Specifically, deflection data and backcalculation results were used to characterize the existing HMA, PCC, stabilized and unstabilized bases, and aggregate and subgrade properties in the MEPDG design program. Laboratory testing results were compared with FWD testing results, and the final designs were found to be relatively insensitive to the differences in characterization of existing layer inputs; that is, new material properties tended to control design results. Some of the significant findings and recommendations from the specific case studies are summarized in the following sections.

CASE STUDY 1, FLEXIBLE PAVEMENT

- For this case study, the MEPDG results indicated that surface-down cracking was critical in the rehabilitation design of an HMA overlay over existing HMA pavements. Nevertheless, a 76-mm (3-inch) HMA overlay was satisfactory for nearly all of the input combinations. Within the ranges identified, the selection of inputs was more critical as one approached the lower values for any layer.

- Based on the design runs conducted for this case study, it is recommended that the correction factors developed by Von Quintus and Killingsworth for backcalculated properties of laboratory values should be applied to the backcalculation results until additional guidance becomes available.[87,116]

- When the existing HMA modulus is based on FWD testing, 30 Hz should be used for the testing frequency input in the design program. This is consistent with the defined equivalent frequency.

CASE STUDY 2, FLEXIBLE PAVEMENT ON RUBBLIZED PCC

- For HMA overlays on rubblized concrete pavements, the critical performance measure in the MEPDG software was surface rutting. The required HMA overlay thickness to achieve a 90-percent reliability level was 178 mm (7 inches) for all analyses. However, if surface rutting were addressed through maintenance at some intermediate year (less than 20 years), a thinner HMA overlay would be appropriate.

- The surface rutting predictive model was mainly sensitive to HMA overlay thickness and HMA and rubblized layer moduli. Therefore, care should be taken in selecting the modulus for the rubblized layer.

- In addition to the recommendations made for case study 1, it may be useful to combine the rubblized layer with the existing unbound base layer (for existing rubblized layers) when using some backcalculation programs.

CASE STUDY 3, RIGID PAVEMENT ON GRANULAR BASE

- The studied section was in poor condition, with a large number of transverse cracks. Three kinds of overlays (HMA overlay, unbonded JPCP overlay, and bonded JPCP overlay) were designed for the rehabilitation. The MEPDG produced the thinnest design for the bonded PCC overlay, while it produced an unreasonably thick HMA overlay, even when modifications were made to the HMA mix design properties.

- The manually entered *k*-value was used for unbonded JPCP and bonded JPCP overlay designs but did not appear to be used for the HMA overlay design. No appreciable difference in the design thickness was found among the three design alternatives by varying layer input values from laboratory or backcalculated results, indicating the reliability of using the backcalculated dynamic (or static) elastic modulus for the PCC layer and the dynamic *k*-value for the supporting layers in the MEPDG design. Furthermore, it was also found that the backcalculated *k*-value representing the composite stiffness of all layers beneath the slab could be directly entered into the MEPDG without significantly influencing the design thickness for the pavement structure analyzed.

CASE STUDY 4, RIGID PAVEMENT ON STABILIZED BASE

- Similar rehabilitation designs (for HMA overlays, unbonded overlays, and bonded overlays) were obtained when either the laboratory or backcalculated modulus values were used. The use of modified values to account for the stabilized base produced a thicker unbonded PCC overlay.

- For the HMA overlay design, it seemed impossible to draw a definite conclusion about the constituents contributing to the *k*-value being reported in the design output (and assumed to be used in the design calculations). The entered dynamic *k*-value did not appear to be used in the determination of *k*-value. The difference in the calculated *k*-values for low and high base stiffnesses was so slight that it appeared that the base layer stiffness was not considered in the calculated *k*-value.

- With the unbonded JPCP overlay design, a noticeable difference was found in the calculated *k*-value between a low and high existing PCC modulus, which might indicate that the stiffness of the existing PCC was involved in the calculation of the *k*-value.

- The modulus of the base appeared to be considered in the calculation of the *k*-value for bonded PCC overlay designs, which agreed with the assumptions listed in the MEPDG. It was also apparent that the calculated *k*-values matched the entered dynamic *k*-values.

CASE STUDY 5, RIGID PAVEMENT ON EXISTING HMA PAVEMENT

- Based on the results of the analyses, it appeared that the MEPDG changed the original flexible pavement structure into an equivalent structure. The equivalent structure consisted of PCC with the same properties as the PCC overlay on top of a base layer with the same properties as the existing HMA; all of this was then supported by Winkler springs with a stiffness equal to the composite *k*-value established using all layers beneath the existing HMA.

CASE STUDY 6, COMPOSITE PAVEMENT (HMA/PCC)

- The selection of layer property inputs from backcalculation values had minimal influence on the overall design results. It appeared that the input dynamic *k*-value was not considered in the design of an HMA overlay of an existing composite pavement. However, the MEPDG documentation indicates the HMA overlay performance is determined using the flexible pavement design, so this was consistent in that the design was controlled by the HMA overlay properties. Although the rigid performance was stated to have been analyzed using that modeling method, the output *k*-value did not appear to be based on the entered dynamic *k*-value. Therefore, the entered moduli should be based on the determined values from backcalculation.

- It was noted that the resultant design was relatively insensitive to the trial PCC moduli. Therefore, the use of the established dynamic backcalculation adjustment factor (0.8 for the PCC modulus) could continue until new factors are developed or an agency develops more specific values. The MEPDG program appears to use static PCC elastic modulus values as entered inputs but the output files suggest it then reverts back to a dynamic value.

- The established modular ratios (reported by Khazanovich, Tayabji, and Darter) to determine layer moduli from the backcalculated composite pavement modulus should continue to be used unless specific testing data are available to determine project specific ratios.[(27)]

- Including an aggregate base layer and determining a corresponding dynamic subgrade *k*-value did not appear to have a significant effect on the design results.

More details on the conduct of the case studies are found in chapter 7 of this volume and in volume II.

Based on the analyses that were conducted for the case study investigations, guidelines were developed for the conduct of FWD testing and the interpretation of the results. Specific guidance is provided on establishing FWD testing plans, performing backcalculation of deflection data (including useful tips on dealing with both routine and atypical situations), and using deflection data in the MEPDG. These guidelines are found in volume III of this report.

In addition, findings from the literature review and work on the case studies identified the need for continued improvements and developments in the analysis and interpretation of pavement deflection data. As described in chapter 8, these improvements lie in a number of specific areas, including the more direct consideration of climatic effects and slab size effects, the movement toward dynamic analyses (as warranted), the development of reliable corrections for dynamic loading conditions, and the development of improved models for both forward analysis and backcalculation for composite pavement structures.

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