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BACKCALCULATION OF PAVEMENT LAYER PROPERTIES FROM DEFLECTION DATA

presented by

Tariq Mahmood

has been accepted towards fulfillment
of the requirements for

Doctor of Philosophy degree in Civil Engineering

Ronald Harichandran
Gilbert Baladi

Major professor

Date 11/17/93

BAC

BACKCALCULATION OF PAVEMENT LAYER PROPERTIES FROM DEFLECTION DATA

By

Tariq Mahmood

A DISSERTATION

Submitted to
Michigan State University
in partial fulfillment of the requirements for the degree of

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1993

ABSTRACT

BACKCALCULATION OF PAVEMENT LAYER PROPERTIES FROM DEFLECTION DATA

BY

TARIQ MAHMOOD

A computer program named MICHBACK has been developed for the backcalculation of pavement layer properties. The program uses a modified Newton algorithm to backcalculate pavement layer moduli and thicknesses from measured surface deflections. It is shown that the newly developed algorithm to predict the roadbed soil modulus at the cost of only a single call to a mechanistic analysis program is accurate. An iterative modified Newton method to calculate the stiff layer depth from deflection data is presented and its accuracy is discussed. The ability of MICHBACK to predict any one of the layer thicknesses along with layer moduli from deflection data is presented. An extensive sensitivity analysis of the backcalculated results to the initial seed moduli is included. Some of the user-friendly features of the MICHBACK program are also presented.

The program has been extensively tested using theoretical deflection basins generated by using a multilayer linear elastic program, as well as field data obtained by using a falling weight deflectometer (FWD). The results of these tests which validate the accuracy and robustness of the program are included. The sensitivity of the results to many factors known to affect the backcalculation results are also explored. The capabilities of the program have been compared with other leading backcalculation programs and the results indicate the superiority of the MICHBACK program in many aspects. The effect of temperature on the backcalculated layer moduli has also been examined.

TO MY PARENTS AND FAMILY

ACKNOWLEDGEMENTS

First of all I want to thank Allah Almighty for providing me with the opportunity to undertake and complete this task. Thanks are also due to the Pakistan Army Corps of Engineers for sponsoring my study program. I also want to thank the Michigan Department of Transportation and the University of Michigan Transportation Research Institute for sponsoring this study.

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I acknowledge with gratitude the dedicated and professional assistance provided by Cynthia Ramon and Weijun Wang in writing the MICHBACK program. Hamid Mukhtar deserves special thanks for sharing the data prepared by him, and his advise during the course of this study. Efforts of Stuart S., Guy N., John A.,

Erich T

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Erich T., and others in data acquisition and reduction is appreciated.

The patience, understanding, and continuous prayers of my wife and my family deserve a special mention and thanks. I am grateful to my elder brother Mumtaz Ahmad Javaid for never letting my absence from home being felt. My children Iqra and Umar deserve special thanks for their understanding and for providing much needed moments of pleasure and relief.

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TABLE OF CONTENTS

LIST OF TABLES

Table 4.1

Table 4.1

Table 4.1

Table 4.1

Table 4.1

Table 4.2

Table 4.

Table 4.2

Table 4.

Table 4.

Table 4.

Table 4.

Table 4.2

Table 4.2

Table 4.2

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 GENERAL

A pavement system subjected to a vehicular or other load input produces a measurable output response in the form of surface deflections. Hence, pavement deflections represent an overall "system response" of the paving layers and the roadbed soil to an applied load. Pavement surface deflections have traditionally been used as an indicator of its structural capacity. Emphasis on mechanistic design and analysis led to the search for efficient schemes to backcalculate layer properties needed for the analysis. The assessment of layer properties and their variation along the road is essential for accurate evaluation of existing pavements and for the design of the asphalt overlays.

A review of existing backcalculation schemes and their characteristics suggest that, in general, they can be grouped into one of three categories: those based on regression equations, those based on pre-calculated database of deflections basins, and those based on iterative methods. The accuracy of the methods based on statistical equations is generally low and problem dependent. Methods based on a database of deflection basins received a better acceptance because of the MODULUS program (Scullion, et al., 1990). However the accuracy of the backcalculated results are affected by the seed values of the layer moduli especially that of the roadbed soil. Another shortcoming is that the estimation of the stiff layer depth is not very accurate which in turn can contribute considerable error to the backcalculated results. Existing iterative methods are relatively slow, share the disadvantage of dependence on seed modulus values, and are unable to mechanistically predict the stiff layer depth. In many cases the accuracy of these methods decreases with the increasing number of pavement layers.

Most existing iterative programs seek to minimize an objective function for the estimation of layer moduli. The objective function is normally the weighted sum of squares of the difference between calculated and measured surface deflections (Uzan, et al., 1989). One of the problems of this approach is that the multi-dimensional surface represented by the objective function may have many local minima. The minimum to which a numerical solution may converge depends on the seed moduli supplied by the analyst and may yield unacceptable results. Also, for many cases, the method may fail to converge on a solution within a reasonable time.

The exact layer thicknesses at the point of FWD testing are seldom known. Pavement coring is one direct method of measuring the layer thicknesses. A typical core, however, yields layer thickness information at a point. The FWD test provides deflection information over much wider distance (60 - inch for the MDOT FWD). Unfortunately, variation in construction and terrain make the variation in layer thicknesses inevitable. Inaccuracies in the layer thicknesses can contribute a significant error in the backcalculated layer properties. Therefore, backcalculation methods which can compute layer thicknesses along with the layer moduli from the deflection data will have the advantage of increased accuracy.

Detection of the depth to the stiff layer is also essential for accuracy in the backcalculated layer moduli. Estimation of stiff layer depth by existing methods is not very accurate and can induce considerable error in the backcalculated properties. Improvement of this estimate would represent a major contribution to this profession.

1.2 PROBLEM STATEMENT

Most existing backcalculation of pavement layer moduli methods seek to minimize an objective function for the estimation of layer moduli. This approach makes the backcalculated results dependent on the values of the seed modulus. None of the existing programs appear to determine the stiff layer depth mechanistically, they are capable of only providing a rough estimate which can adversely affect the backcalculated layer moduli. Hence, there is a need to produce a backcalculation algorithm such that its outputs are not affected by the values of the seed moduli, and is able to provide a mechanistic estimate to the stiff layer depth along with the layer moduli.

1.3 RESEARCH OBJECTIVES

The main objective of this research is to develop a robust backcalculation program whose results are not sensitive to the seed values of the layer moduli. Also, the algorithm should have the capability to accurately compute the stiff layer depth.

The program should be user-friendly, provide various options to the user to view and pre-process the deflection basins if necessary, to be of benefit to local State Highway Agencies (SHA) and able to directly process the format of the deflection output files of the Michigan Department of Transportation (MDOT) operated version of the KUAB Falling Weight Deflectometer.

1.4 THESIS LAYOUT

This thesis is organized in eight chapters as follows:

Chapter 2 - Literature review - Nondestructive deflection testing (NDT) methods and related equipment along with their limitations and capabilities are briefly discussed. Various uses of the deflection data, backcalculation of layer moduli methods and their merits and limitations, and pavement material properties are introduced. Also some of the difficulties related to the backcalculation process, error sources, and various methods for converting the backcalculated properties to standard load and temperature conditions are presented.

Chapter 3 - Efficient iterative methods for backcalculation of pavement layer

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properties - The Newton method and its application to the backcalculation of pavement layer properties is presented. A new method to estimate the modulus of the roadbed soil in a single call to an elastic layer program is introduced. Also, the modified Newton method and a logarithmic transformation to speed up convergence is discussed.

Chapter 4 - Verification of MICHBACK algorithm using theoretical deflection basins - Verification of the MICHBACK backcalculation algorithm using theoretical deflection basins are presented. The backcalculated results are compared to those obtained from MODULUS 4.0 and EVERCALC 3.0 computer programs. Important aspects of convergence characteristics and uniqueness of the solutions are tested. Sensitivity of backcalculated results to Poisson's ratios, inaccuracies in deflections at different sensor locations and accuracy of elastic layer programs is also examined.

Chapter 5 - Michback program structure and features - The features and the structure of the MICHBACK program are presented.

Chapter 6 - Validation using field data - Validation of the MICHBACK program using FWD test data from pavements across the State of Michigan is presented.

Chapter 7 - Temperature effects on the backcalculated layer moduli - The effects of the pavement temperature on the AC modulus are discussed.

Chapter 8 - Summary, conclusions, and recommendations.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

A pavement system subjected to a vehicular or other load input produces an output response in the form of deflection. Hence, pavement deflections represent an overall "system response" of the paving layers and the roadbed soil to an applied load. A load applied at a point (or an area) on the pavement surface will attenuate with depths and radial distances thereby, causing all pavement layers to deflect as a consequence of the introduced stresses and the resulting strains. The amount of deflection in each layer will generally decrease with depth and radial distance and will vary depending on the layer properties. Beyond a certain depth and radial distance, the induced stresses become negligible and the materials are not affected by the applied load. Further, stronger pavements (i.e., those with good quality materials and thick layers) deflect less under a given load than do weaker pavements. Hence, pavement deflections are being used as indicators of pavement quality and as inputs (along with the layer thicknesses) to a mechanistic analysis routine to backcalculate the properties of the various pavement layers.

Pavement deflection can be measured by using nondestructive deflection tests (NDT). An NDT consists of applying a known force to a pavement surface and measuring its response (deflection). In some test methods (e.g., the Benkelman beam), the pavement deflection (or more precisely pavement rebound) is typically measured at a point located between the two tires of the back axle of a single axle truck. In other methods (e.g., dynaflect, falling weight deflectometer (FWD), road rater), the pavement deflection profile (deflection at several points located under, and at various radial distances away from the center of the loaded area) is typically measured. Analysis of the measured pavement deflection provides a quantitative basis for evaluation of the pavement structural conditions at any time during its service life. In addition, important information regarding rehabilitation and maintenance requirements can be inferred from the deflection profile (deflection basin). Because they are nondestructive in nature, NDT are easily and quickly performed, the tests cause minimal hindrance to the normal flow of traffic, and they are less hazardous and more economical to perform. In addition, the measured deflections are representative of the actual pavement response to the applied load.

As stated earlier, pavement surface deflection has traditionally been used as an indicator of the structural capacity of pavements. One of the earliest uses of pavement deflection was that made in California in 1938 and reported by Haveem (1938). He concluded that flexible pavements would have satisfactory performance if they deflect less than 20 mils under a 15000 lb axle load. The WASHO Road Test (conducted in Huba Valley on flexible pavements) results showed that for a satisfactory pavement performance, pavement deflections should be limited to 30 to 40 mils for pavements located in cold and warm regions, respectively (WASHO, 1954; WASHO, 1955).

Presently, NDT data are being used in conjunction with the pavement distress survey for evaluation, rehabilitation, and pavement management purposes. A proper analysis of the NDT data can provide information regarding:

1. The need of a particular pavement section for an overlay and perhaps, the required thickness of the overlay.
2. The degree of variability of the materials along the roadway.
3. Potential locations of voids beneath the surface layer.
4. The load transfer efficiency across joints in concrete pavements.
5. The elastic properties of the various pavement layers and the roadbed soil.
6. The ability of the pavement structure to support traffic loads at the posted speed limits.

7. The effect of seasonal variations on the load carrying capacity of the pavements.
8. The need for, and perhaps the type of rehabilitation activities.
9. The in situ stress sensitivity of the paving materials.
10. The effectiveness and benefits of various rehabilitation techniques.

The mechanistic analysis of pavement deflections to infer the structural properties (moduli and Poisson's ratios) of the various layers is referred to as "backcalculation of layer moduli". The task of backcalculation of layer moduli, however, is a difficult one. This difficulty is related to several reasons including:

1. The variability of the pavement materials along a given stretch of roadway.
2. The changing characteristics of the pavement materials due to seasonal changes, time, and temperature.
3. The nonlinear behavior of the paving materials.
4. The lack of accurate information concerning layer thicknesses and depth to stiff layer (e.g., bedrock).
5. The existence (in some cases) of a very thin layer (e.g., a one-inch debonding layer between an original pavement and an overlay).

Nevertheless, backcalclation of layer moduli techniques have been developed and have been successfully applied to both flexible and rigid pavements. Most of these techniques are based on elastic layered concepts and are directed at the incorporation of NDT data collected on flexible pavements and at mid-slabs of rigid pavements. Elastic layered analysis does not adequately model discontinuities such as joints and cracks in rigid pavements.

The NDT equipment used to measure pavement deflections differ in the methods used in applying loads to pavement and in the number and location of sensors needed for measuring the pavement response. The various types of NDT equipment can generally be divided into five categories as presented in the next section.

2.2 NONDESTRUCTIVE DEFLECTION TESTING DEVICES

NDT devices can be categorized as follows:

1. **Static Deflection Equipment** - Static deflection equipment measure pavement deflections or rebound due to the application of a gradually increasing or decreasing load. This type of equipment includes the Benkelman Beam (Moore, et al., 1978; Asphalt Institute, 1977; Epps, et al., 1986), Plate bearing test (Moore, et al., 1978; Nazarian, et al., 1989), Dehlen Curvature Meter (Guozheng, 1982), Pavement Deflection Logging Machine (Keneddy, et al., 1978), and C.E.B.T.P. Curviameter (Paquet, 1978). Several technical problems are associated with this type of equipment including:
   a) It is time consuming and laborious.
   b) The test requires closing the pavement section to traffic.
   c) Deflection can be measured only at one point (special arrangements need to be made to measure the deflection profile).
   d) The test presents hazardous conditions to both the traveling public and the test operators.

2. **Automated Deflection Equipment** - Automated deflection equipment delivers a gradually applied load to the pavement structure in an automated mechanism. This type of equipment includes the La Croix Deflectograph (Hoffman, et al., 1982; Keneddy, 1978) and the California Travelling Deflectometer (Roberts, 1977). The technical problems associated with this type of equipment are

similar to those associated with the static deflection equipment.

3. **Steady-State Dynamic Deflection Equipment** - Steady-state dynamic deflection equipment (also called vibrators) produce a sinusoidal vibration in the pavement with a dynamic force generator (typically rotating eccentric load). The most popular devices include the Dynaflect, the Road Rater, the Cox Device, the Waterways Experiment Station (WES) Heavy Vibrator, and the Federal Highway Administration (FHWA) Thumper (Scrivner, et al., 1969; Smith, et al., 1984, Moore, et al., 1978). Each of these devices has some limitations and advantages that are addressed elsewhere (Bush, 1980; May, 1981).

4. **Impulse Deflection Equipment** - Impulse deflection equipment delivers a transient force impulse to the pavement surface by means of dropping a weight. The most popular devices include the Dynatest Falling Weight Deflectometer (FWD), the KUAB Falling Weight Deflectometer, and the Phoenix Falling weight Deflectometer (Nazarian, et al., 1989; Hoffman, et al., 1981; Bohn, et al., 1972; Crovetti, et al., 1989; Claessan, et al., 1976). Recently, FWD devices have become popular and they are being used by an increasing number of State Highway Agencies (SHA).

5. **Wave Propagation Equipment/Method** - The wave propagation method (also called surface wave testing) has been used with some success to backcalculate the layer moduli of pavements. The method is not very widely used because of the complex data analysis procedures associated with it (Thomas, 1977).

A study carried out by Lytton et al. (1990) lists in detail the characteristics, operational costs, and data analysis techniques associated with different types of NDT equipment. The NDT equipment have been rated after assigning weights to various important factors associated with the use of the equipment. The study concluded that the FWD is the best equipment available for simulating the actual traffic loading and further ranked the Dynatest Model 8000 as the best FWD equipment available.

Since the backcalculation technique presented in this thesis is based on NDT, only a brief summary of the wave propagation method is presented in the next section. The concepts regarding deflection testing and backcalculation of layer moduli, on the other hand, are presented in greater detail in subsequent sections.

2.3 SURFACE WAVE TESTING

The surface wave testing technique involves the measurement of the velocity and length of the surface waves propagating away from the point of application of an impact load (Nazarian, et al., 1983; Nazarian, et al., 1984; Robert, et al., 1986). This technique was pioneered by the German Society of Soil Mechanics in the late 1930's (Bernhard, 1939). The wave propagation theory is based upon the fact that wave velocities in a homogeneous and isotropic half space subjected to external impact load can be expressed by the following equation:

$$V = \alpha \sqrt{E/\rho} \tag{2.1}$$

where E = modulus of elasticity;

ρ = mass density; and

α = a coefficient that is a function of Poisson's ratio of the medium.

Upon excitation by an impact or a vibratory load, three types of waves are generally transmitted through and along the pavement. The three wave types and the percent of

the applied energy that is dissipated by each type are tabulated below (Miller, et al., 1955).

Wave type	Energy dissipation (% of applied energy)
Compression (P)	7
Shear (S)	26
Rayleigh (R)	67

For a multi-layer pavement structure, analysis of these waves is a complex proposition. Extensive mathematical analysis of these waves can be found elsewhere (Thomas, 1977). The remaining part of this section summarizes two techniques that can be used to induce the three types of waves in a pavement structure and the advantages and disadvantages of the surface wave method.

2.3.1 Impulse Load Testing

In this technique, an impulse (impact) load is applied at a point (called the source) on the surface of the pavement section. Any impact device (e.g., sledge hammer or impact hammer) can be used for this purpose. Upon impact, the three types of waves will propagate away from the source either along the pavement surface or with depth. At an interface between two pavement layers, the wave fronts undergo both, reflection and refraction. During the test, the time of arrival of several wave fronts at different points along the pavement surface is recorded in order to measure the travel paths of these waves through different layers and to deduce their velocities in each layer. This method is not applicable when high-velocity stiff layers, as is the case for flexible pavements, are encountered at the top and the layers grow progressively weaker with depth (Moore, et al., 1978). As such the method cannot be applied to backcalculate the layer moduli of pavements.

2.3.2 Steady-State Vibration

In the steady-state vibration technique, a vibratory source is placed at an arbitrary point on the pavement surface and a vibrating load (typically sinusoidal) is applied at the source. In this technique, the phase lag (the time delay which occurs between the motion of the pavement and that of the source as the waves travel away from the source) is measured. High frequency waves have shorter wave lengths and hence can penetrate only the surface layer. On the other hand, if low frequency waves are used they will have lengths several times that of the pavement thickness and their speed will be determined by the properties of the roadbed soils. Intermediate wave lengths can be used and their speeds can be related to the elastic properties and thicknesses of the underlaying layers (Jones, 1960).

2.3.3 Advantages and Limitations

The surface wave test method has several advantages over other NDT methods. These include:

1. Layer thicknesses of the various pavement layers can be calculated and as such no assumptions or approximations are required (Nazarian, et al., 1989). Therefore, the method is more pertinent when the dimensions of the structure is not known (Robert, 1986).
2. The depth to bedrock as well as the bedrock modulus can be accurately calculated. These two factors represent a major source of error for the deflection based backcalculation technique (Heukelom, et al., 1962).
3. The elastic modulus of a thin or a thick asphalt concrete (AC) layers can be estimated. Variations of the modulus within any paving layer can also be estimated. The method has the potential of being fully automated at a later

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time (Hiltunen, et al., 1989).

4. The test method and equipment are simple to operate, but sophisticated data analysis is required (Robert, et al., 1986). The test result pertain to a wide area and not necessarily to the local pavement properties in the vicinity of the applied load (Watkins, et al., 1974).

The major disadvantage of this method is that the testing and data reduction cannot be performed rapidly. It takes about 20 minutes to perform one test whereas a deflection test can be performed in less than 2 minutes (Wang, et al., 1989). Also the moduli obtained are for low strain levels which may not be an accurate estimate of the moduli under actual traffic loading (large load may cause the pavement materials to exhibit stress-dependent behavior). At best, the method is presently suited for project level surveys only (Nazarian, et al., 1989). Interpretation of the data is an extremely complex process which can only be undertaken by experts. The test results are not necessarily unique and the method works only for a few special structures (Thomas, 1977).

2.4 NONDESTRUCTIVE DEFLECTION TESTING

Applying a known force to measure the deflection response of a pavement structure is the essence of NDT. The earliest device used in the United States (U.S.) for measuring pavement deflection is the General Electric Travel Gage in 1938 (Moore, et al., 1978). The tests showed that the pavement deflection can be measured to a depth of 21 feet and that the main contribution to the total deflection comes from the upper 3 feet of the structural section (Haveem, 1938).

During the WASHO Road Test (1954; 1955) an improved version of the General Electric Travel Gage, incorporating a Linear Variable Differential Transformer (LVDT), was used. In 1952, A.C. Benkelman developed a simple and easy to use instrument for measuring pavement deflections called the Benkelman Beam. The Benkelman Beam is still widely used in many countries across the world for measuring pavement deflections. Recent years have witnessed the development of equipment with better capabilities than the Benkelman Beam and as a result new NDT methods have been developed. Currently used techniques can be placed into one of the following categories based on the method by which the deflections are induced:

1. static force-deflection
2. dynamic steady-state vibrations
3. dynamic impulse force-deflection

2.4.1 Static Force-Deflection

In this procedure, the response of a pavement structure to gradually applied or gradually removed loads is measured. The force may be applied by a slow moving vehicle of known weight or through a rigid plate of specified diameter that is part of a stationary loading frame. Static or quasi-dynamic measurements of either rebound or loading deflection are made by making the load vehicle pass a point located on the pavement surface at a creep speed. Deflection and rebound deflection testing procedures have been published by AASHTO (1982) and the Asphalt Institute (1977), respectively.

2.4.1.1 Advantages and Limitations

The basic advantage of the static deflection method can be attributed to its simplicity, cheaper equipment, and low maintenance costs. The disadvantages are:

1. It is difficult to obtain an immovable reference point for making deflection measurements.
2. All the devices measure only a single deflection making it difficult to obtain valuable information regarding the shape and size of the deflection basin.

Moreover, no information on the critical strain in the upper layer is obtained.

3. The automated beam equipment is further handicapped by the fact that it is difficult to test a specific point on the pavement. Further if the deflection basin is large, the reference point may be located within the basin itself.
4. The method is suitable only for use with static analysis and dynamic effects cannot be analyzed.

2.4.2 Dynamic Steady-State Vibrator

Essentially all steady-state vibrator equipment induce a steady state sinusoidal vibration in the pavement using a dynamic force generator. The dynamic force is superimposed on the static force exerted by the weight of the force generator (Figure 2.1). The dynamic load causes the pavement system to vibrate at the same frequency as the load. The deflection response of the pavement is usually measured with inertial sensors. Velocity sensors (called geophones) are commonly used, although some equipment make use of accelerometers as well. Many of the devices can vary both the amplitude and the frequency of the excitation (Moore, et al., 1978).

2.4.2.1 Steady-State Dynamic Response of Pavements

Under static loads, pavement deflection is normally proportional to the applied forces, and substantial recovery is obtained when the load is removed. Dynamic response is no different in that, at any specific driving frequency, the amplitude of the dynamic deflection is approximately proportional to the amplitude of the applied force (Moore, et al., 1978). Green and Hall (1974) obtained results using a 16 kip vibrator at three different driving frequencies (Figure 2.2). The test results showed that the deflection is almost proportional to the loads at 15 Hz and 40 Hz and somewhat non linear at 10 Hz.

The overall rigidity of road construction, S, defined by Van der Poel (1951)

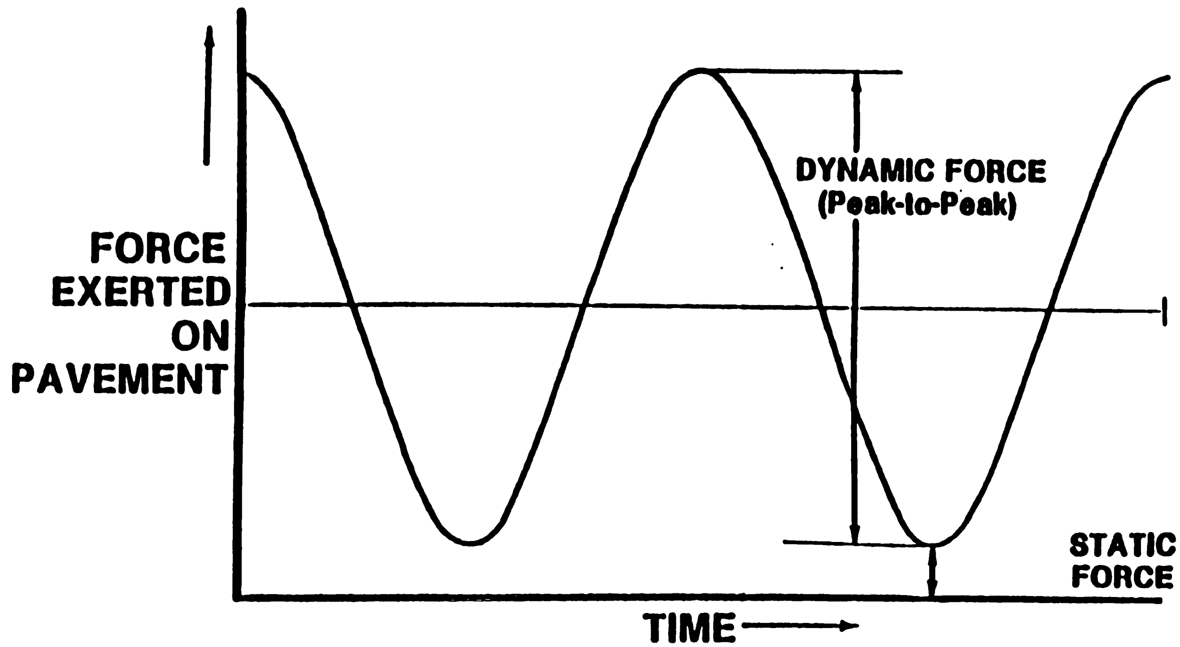

Figure 2.1. Typical output of a vibrating steady state force generator (Moore, et al., 1978).

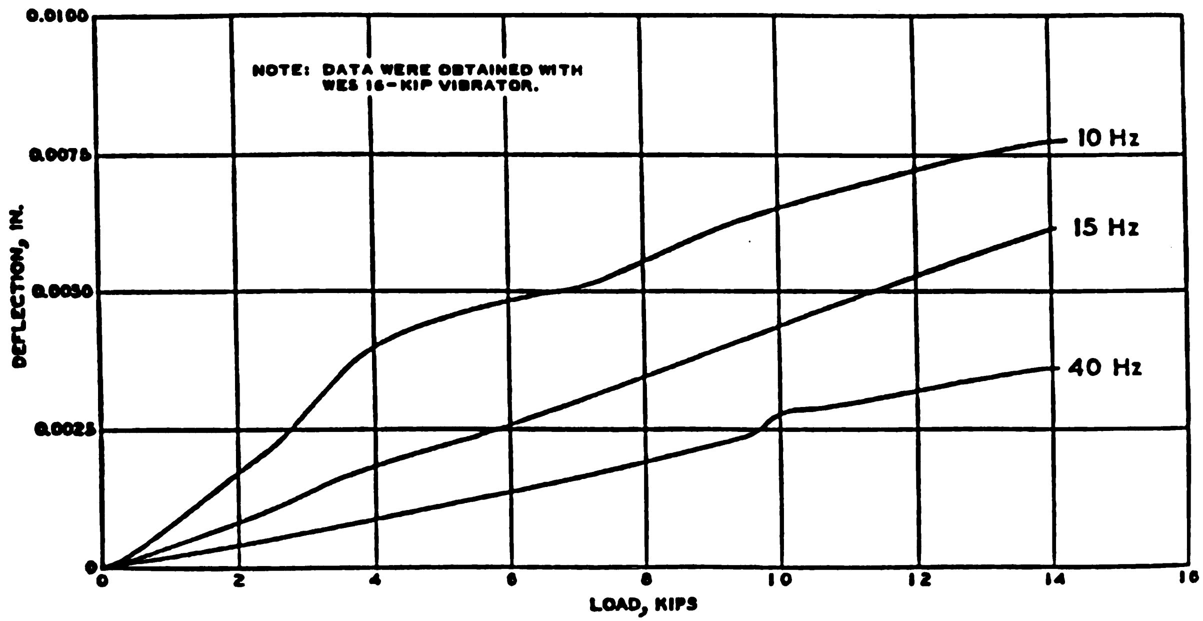

Figure 2.2. Typical load-deflection curves for frequencies of 10, 15, and 40 Hz. (Green, et al., 1974).

as the amplitude of the dynamic force required to produce a unit amplitude in the deflection of the pavement surface. He pointed out that S is not constant, it depends upon the driving frequency.

The Kelvin model has been used to represent the pavement response to dynamic loading as shown in Figure 2.3 (Lorenz, et al., 1953; Van der Poel, 1953). The equation of the motion and significance of different parameters involved in the model can be found elsewhere (Baladi, 1976; Baladi, 1979; Taylor, 1978; Thompson, 1972; Heukelom, et al., 1960; Heukelom, 1961; Szendirei, et al., 1970).

2.4.2.2 Advantages and Limitations

The advantages of NDT are:

1. Accurate deflection basin measurements can be made with respect to an inertial reference.
2. Steady state dynamic deflection devices correlate well with the static deflection measurements. Many agencies make use of these correlations for pavement evaluation.

The disadvantages are:

1. These types of measurements represent the stiffness of an entire pavement structure. The separation of the effects of all the pavement components with measurement of the deflection basin has not yet been accomplished (Nazarian, et al., 1989).
2. The commercially available machines operate at light loads and hence the pavement is not stressed to traffic loads. As a result, the effect of any non-linearity in the paving materials is neglected (Pell, et al., 1972).
3. The steady state deflections are observed to be greater in magnitude than rebound deflections for Bankelman Beam (Hoffman, et al., 1981), while

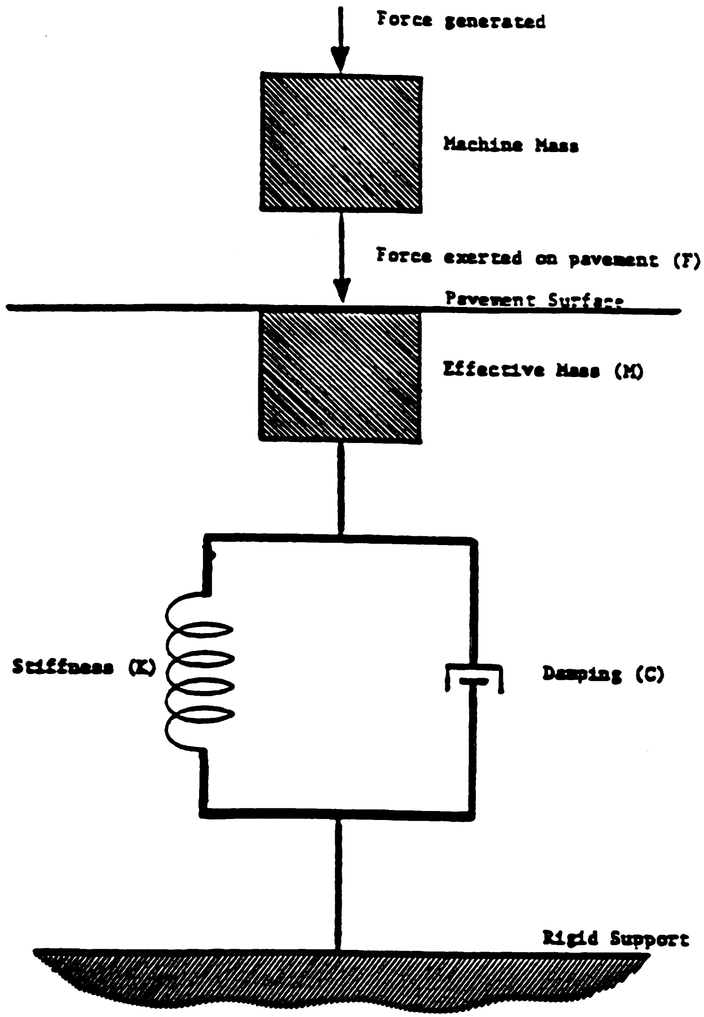

Figure 2.3. Mass-spring-dashpot representation of a pavement structure subjected to a forced dynamic vibration (Lorenz, et al., 1953).

the deflections under moving vehicle are found to be smaller than those for equivalent static loads (Lister, 1967).

2.4.3 Impact Loading

Impact loading devices deliver an impulse force to the pavement surface and measure the transient response. Force impulses are normally generated by dropping a known weight from a known height on a plate placed on the pavement surface. Inertial motion sensors are normally used to record the pavement response. In the U.S., the Cornell Aeronautical Laboratory (CAL) was the first agency to use a trailer mounted force generator for impulse loading (Moore, et al., 1978).

The pavement properties can be investigated by using the classical Kelvin single degree of freedom (SDOF) system. Fourier transform of an instantaneous impulse response deflection can provide complete information regarding the steady-state frequency response. Practically, however, it is impossible to generate an instantaneous impulse. Based on their test results, Szendrei and Freeme (1970) and Moore et al. (1978) concluded that a load pulse duration must be less than 1 msec to be considered instantaneous. Longer force impulses do not contain all the steady-state frequency response. Analytical treatment of the instantaneous force impulse of the classical model can be found in the literature (Richart, 1970; Tayabji, et al., 1976; Hansen, et al., 1956).

2.4.3.1 <u>Advantages and Limitations</u>

The advantages of the impact load test include:

1. The loading most closely resembles actual traffic loading (Hoffman, et al., 1982). Therefore the shape of the deflection basin and hence the developed strains closely reflect those due to actual traffic load.
2. The actual duration of the test is only a few minutes and the measured data

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gives sufficient information regarding the deflection basin to investigate the layer properties (Moore, et al., 1978).

3. Results can easily be correlated with those of the static load test.
4. The test equipment is simple to operate and can be maintained at reasonable costs.

The disadvantages are:

1. Complex analysis is required to model the response because of the difficulty of producing an instantaneous impulse (Taylor, 1978). Analysis of longer force impulses are even more complex.
2. It is a problem to obtain the response in the low frequency range because of the low output characteristics of the motion sensors.

2.5 DEFLECTION RESPONSE OF PAVEMENTS

Based on review of the literature, certain concepts regarding the deflection response of the pavements can be expressed (Moore et al., 1978; Taylor, 1978):

1. A maximum tolerable deflection level can be assigned to each pavement structure. This level is typically a function of the layer thicknesses and properties and the traffic load and volume.
2. Overlaying a pavement will reduce its deflections.
3. The deflection history of a well designed pavement can be traced through three phases (Figure 2.4):
   a. Initial phase: Just after construction, the pavement undergoes consolidation and the deflections show a slight decrease.
   b. Functional phase: The deflections remain constant or increase slightly.
   c. Failure phase: The deflections increase rapidly.

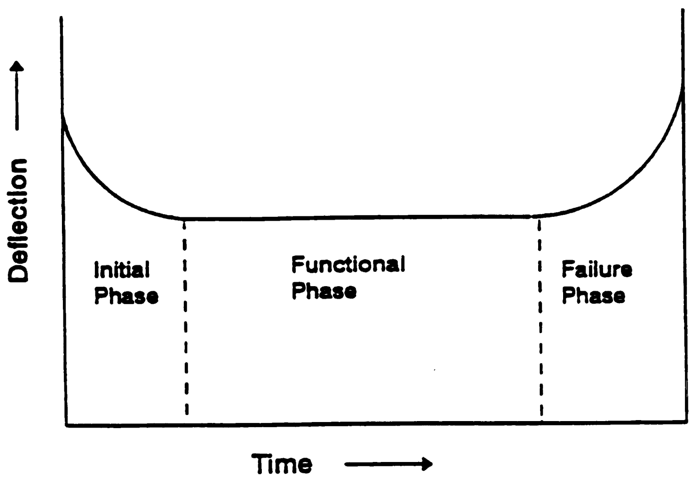

Figure 2.4. Well-designed pavement deflection history curve (Moore, et al., 1978).

4. The deflections of a flexible pavement increase with the increase in temperature of the bituminous surface depending upon the thickness of the bituminous layer.

5. The deflection history of the pavement varies throughout the year depending upon the environmental factors (Izada, 1966). A typical annual deflection history of a pavement subjected to frost and spring-thaw actions can be divided into four periods (Figure 2.5):

   a. A deep frost period when the pavement is frozen.
   b. A spring-thaw period during which the pavement deflections rise rapidly.
   c. A rapid strength recovery period during which water from melting frost starts draining and evaporating from the pavement.
   d. A relatively dry period during which the pavement deflections level off.

2.6 INTERPRETATION OF DEFLECTION DATA

Interpretation of NDT deflection data is a complex and difficult task. The techniques used to extract useful information from deflection data can be divided into three basic groups

1. Empirical Analysis
2. Rational Analysis
3. Mechanistic Analysis

2.6.1 Empirical Analysis

In the past most pavement design procedures were empirical in nature. An empirical approach relies upon the results of past experiments and experience. Generally, deflections are directly related to the pavement conditions and other

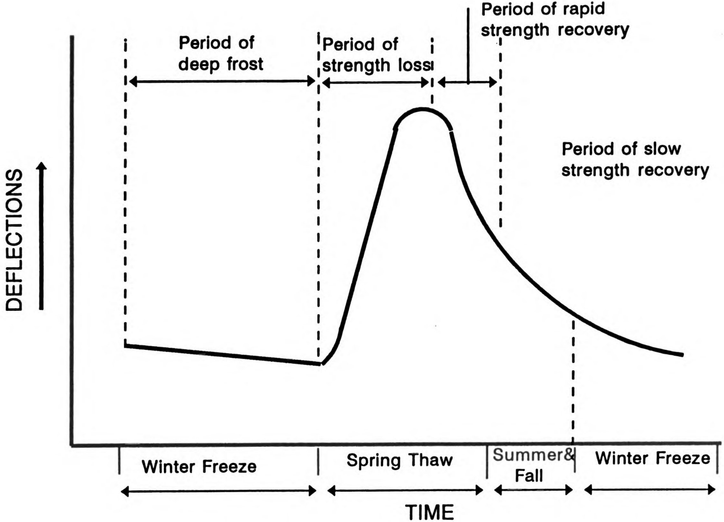

Figure 2.5. Influence of season on pavement deflections (Izada, 1966).

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variables such as traffic, pavement type, and environment factors. Results of a number of experiments are used to obtain a relationship between the variables and the outcomes. The relationship is typically not supported by theory. Statistics rather than the phenomena shaping the results are given more importance.

The Utah Department of Transportation (UDOT) procedure (Molenaar, et al., 1982) is one such example. The maximum deflection (DMD), the Surface Curvature Index (SCI), and the Base Curvature Index (BCI) are compared to the acceptable values of these parameters which are inferred from a long term pavement performance (LTPP) (Figure 2.6; Table 2.1). The California DOT (1973), Asphalt Institute (1974), Oklahoma DOT (AASHTO, 1972), Louisiana DOT (Kinchen, et al., 1977), and Texas DOT (Brown, et al., 1970) methods are other such examples where the pavement deflections have been utilized directly to infer information regarding the pavement condition and its capability to carry projected future traffic.

2.6.2 Rational Methods

Rational methods utilize basin properties such as spreadability or representative structural properties to describe the pavement strength (Lytton, et al., 1990). The representative structural properties of a pavement are normally taken as the effective thickness of the pavement (McComb, et al., 1974; Kinchen, et al., 1980), effective thickness of the asphalt concrete and base courses (Vaswani, 1971), or the effective modulus of the pavement (Asphalt Institute, 1977; Lytton, et al., 1990).

2.6.3 Mechanistic Analysis

A mechanistic approach refers to the calculation of induced stresses and strains to determine the response of the pavement structure to applied load. In order to calculate the response of the pavement structure, certain fundamental material properties along with the layer thicknesses must be known. The results obtained can

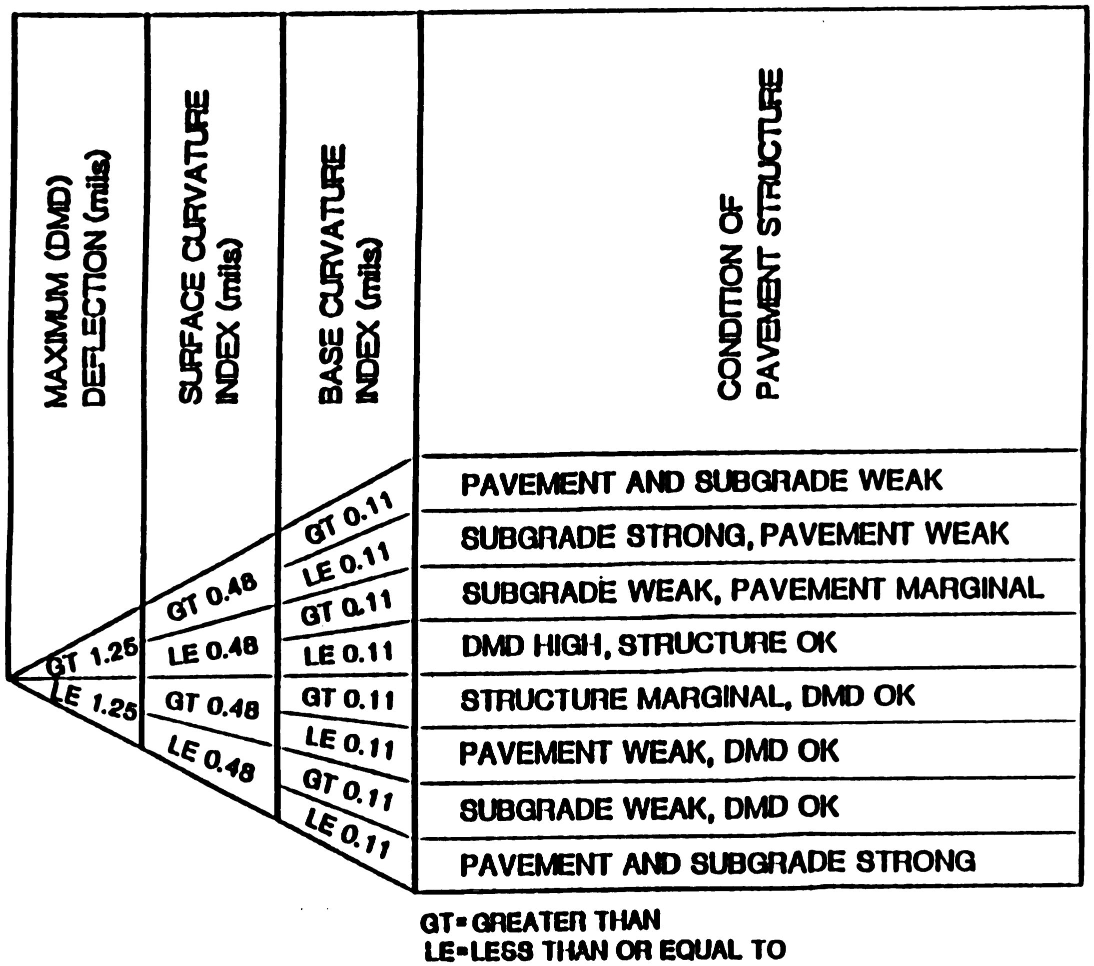


Figure 2.6. Use of deflection basin parameters to analyze pavement structural layers, from Utah overlay design procedure (Molenaar, et al., 1982).

Table 2.1. Summary of deflection basin parameters (Mahoney, et al., 1991).

Table 2.1. Summary of deflection basin parameters (Mahoney, et al., 1991).

Parameter	Formula	Measuring device
Maximum deflection	D_0	Benkelman Beam, Lacroux deflectometer, FWD
Radius of curvature	$R = \dfrac{r^2}{2 D_0(D_0/D_r - 1)}$ $r = 127$ mm	Curvaturemeter
Spreadability	$S = \dfrac{[(D_0 + D_1 + D_2 + D_3)/5]100}{D_0}$ $D_1 \ldots D_3$ spaced 305 mm	Dynaflect
Area	$A = 6[1 + 2 (D_1/D_0) + 2 (D_2/ D_0) + D_3/D_0)$	FWD
Shape factors	$F_1 = (D_0 - D_2) / D_1$ $F_2 = (D_1 - D_3) / D_1$	FWD
Surface curvature index	$SCI = D_0 - D_r$, where $r = 305$ mm or $r = 500$ mm	Benkelman Beam Road Rater FWD
Base curvature index	$BCI = D_{610} - D_{915}$	Road Rater
Base damage index	$BDI = D_{305} - D_{610}$	Road Rater
Deflection ratio	$Q_r = D_r/D_0$, where $D_r = D_0/2$	FWD
Bending index	$BI = D/a$, where a = Deflection basin	Benkelman Beam
Slope of deflection	$SD = \tan^{-1} (D_0 - D_r)/r$ where $r = 610$ mm	Benkelman Beam

thus be explained by theory and changes in response due to changes in any variables can be predicted more rationally. The advantages of such an approach include (Mahoney, et al., 1991):

1. The accommodation of changing loads.
2. The ability to account for changes in materials and environmental conditions.
3. Improvement in the reliability of performance prediction models.
4. Better assessment of the performance of various paving materials.

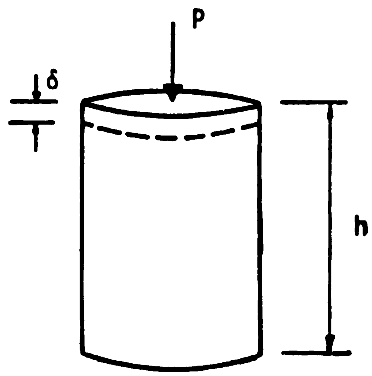
The aim of backcalculation methods based on mechanistic analysis, is to backcalculate the layer moduli of different paving layers. Which in turn are used to calculate stresses and strains induced in the pavement structure due to a given load. Mechanistic properties can be used for both the design and evaluation of pavement structures. The mechanistic design procedures use laboratory determined material properties to calculate a layered system response to an applied load. The mechanistic evaluation procedures, on the other hand, uses the measured pavement response under a known load to backcalculate the layer properties (Robert, et al., 1986). The two procedures are illustrated in Figure 2.7.

In general mechanistic-based backcalculation procedures require computer based solution. Some of the better known mechanistic models are discussed in detail in Section 2.8.

2.7 SPECIAL PROPERTIES OF PAVING MATERIALS

Stress sensitivity of unbound materials and temperature dependency of asphalt concrete have considerable affects on the backcalculated layer moduli. These two properties of paving materials are discussed next.

Laboratory Testing of a Specimen

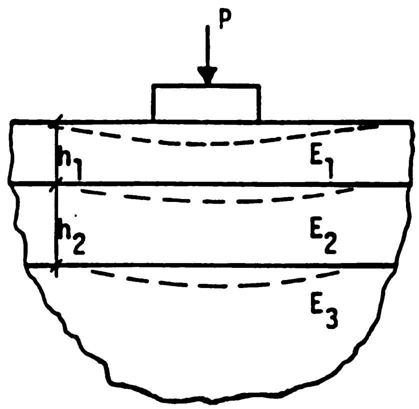


1. Apply axial load;
2. Measure axial deformation;
3. Compute "E" based on the ratio:

$$E = \frac{\text{Deviator Stress}}{\text{Recoverable Strain}}$$

Component Characterization ➡ System Response

Full-Scale Testing of a Pavement



1. Apply NDT load;
2. Measure surface deflections;
3. Compute "E's" based on a Layered Model

System Response ➡ Component Characterization

Figure 2.7. Component versus system analysis of pavement structures.

2.7.1 Material Non-Linearity

The response of common highway materials to traffic loading typically includes elastic, viscoelastic, and plastic components. During the initial cycles of stress, at slow loading rates or high stress levels, the viscous and plastic components may be dominant (Dehken, 1978).

The stress-strain response of different highway materials is different under changing stress conditions depending upon their properties and is discussed separately for each material type.

2.7.1.1 Granular Materials

The resilient modulus of sands and gravel is reported to increase with confining pressure. The magnitude of repeated deviator stress, unless high enough to induce shear failure, has no effect on the resilient modulus (Dehken, 1978). Biarez (1962) suggested a relation for the resilient modulus (M_R) after performing tests on uniform sand in a triaxial apparatus:

$$M_R = K_1 \, (\theta)^{K_2} \tag{2.2}$$

where K_1 and k_2 are material constants, and θ is the sum of the principal stresses (identical to the stress invariant I_1). If θ is increased in a manner that $(\sigma_1 - \sigma_3)$ are constant essentially there will be no change in the modulus but the model fails to address this condition.

Trollope, et al. (1963) observed that the rebound modulus increased with confining pressure and also confirmed earlier observation that as long as failure conditions are not approached the modulus was not affected by the axial stress. Morgan (1966) performed repeated load triaxial tests on two sands and observed marked increase in resilient modulus with increasing confining pressure and a slight decrease with increasing deviator stress. He also stated that the resilient Poisson's ratio remained unaffected by both the deviator stress and the confining pressure.

Mitray and Seed, et al. (1964; 1969) found the M_R of sand subjected to triaxial repetitive load tests can be expressed as:

$$M_R = K_3 * \sigma_3^{K_4} \tag{2.3}$$

where σ_3 is the confining pressure and K_3 and K_4 are material constants. Equations 2.2 and 2.3 were also found to apply to gravel with changed values of exponents K_2 and K_4. Jones (1960) observed stress-dependency of the modulus of sand subjected to dynamic (vibratory) tests. Hardin and Black (1966) found that the shear modulus, G, could be represented by:

$$G = K_5 (\theta)^{K_6} \tag{2.4}$$

where θ is the sum of normal stresses. The exponent K_6 was 0.6 for $\theta < 42$ psi. and 0.5 for greater values. Stress sensitivity of cohesionless roadbed soils of Michigan was confirmed by Boker (1978).

2.7.1.2 Cohesive Soils

Resilient modulus of cohesive soils is reported to decrease with increasing deviator stress and is little affected by the transverse stresses (George, 1969). Seed, et al. (1972) reported results of repeated load triaxial tests on silty clay. The resilient modulus was found to decrease rapidly with increase in the deviator stress up to 15 or 20 psi, any further increase in deviator stress resulted in an increase of the modulus. The variation in modulus between deviator stress values of 3 and 15 psi was found to be about 400 %.

Kallas and Rilley (1977) found that in repeated compression tests on silty clay, the values of M_R increase slightly with increasing confining pressure and decrease more markedly with increasing deviator stress. Sparrow and Tory (1966) performed in-situ tests on medium plastic clay and reported an increase in modulus with increase in depth and offset radius. The secant modulus was found to decrease with increase of the major principal stresses, the effect being more pronounced at lower stresses. These results were confirmed by Brown and Pell (1967).

The most commonly used model representing the stress dependency of fine grained soils is reported by Young and Baladi (1977) as follows:

$$M_R = K_1 * \sigma_D^{-K_2} \tag{2.5}$$

where σ_D = deviator stress; and

K_1 , K_2 = material constants.

Some mechanistic pavement design procedures use non-linear models. For example, MICHPAVE and ILLIPAVE computer programs use a bilinear model for cohesive soils. The relationship is expressed as:

$$M_r = \begin{cases} k_2 + k_3[k_1 - (\sigma_1 - \sigma_3)] & \textit{for } k_1 > (\sigma_1 - \sigma_3) \\ k_2 + k_4[(\sigma_1 - \sigma_3) - k_1] & \textit{for } k_1 \leq (\sigma_1 - \sigma_3) \end{cases} \tag{2.6}$$

where $(\sigma_1 - \sigma_3)$ = deviator stress; and

k_1, k_2, k_3, and k_4 = material constants.

The relationship is illustrated in Figure 2.8.

2.7.1.3 Treated Materials

Fossberg (1969) conducted tests on highly plastic clay stabilized with lime and concluded that the resilient modulus increased with increasing confining pressure and decreased with increasing deviator stress. Mitchel, et al. (1977) observed a decrease in M_R of cement stabilized sand with increase in deviator stress. Wang (1968) found that the resilient modulus of a silty clay stabilized with cement increase with increasing confining pressure and decreased with increasing deviator stress. Both researchers concluded that cement stabilized materials behave essentially linearly

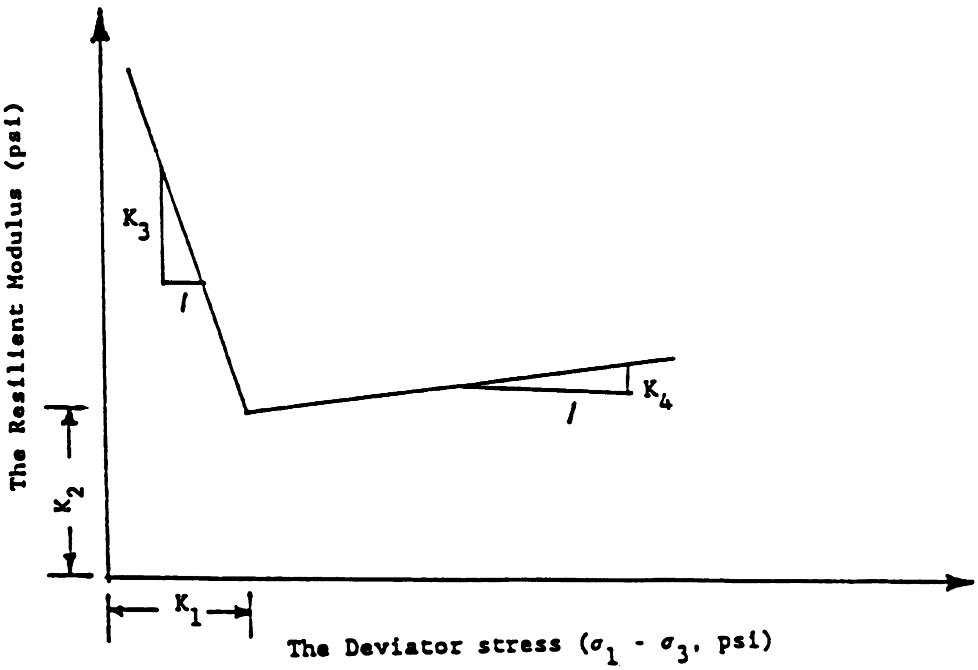


Figure 2.8. Typical variation of resilient modulus with repeated stress for cohesive roadbed soil (Ming-Shan, 1989).

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under repeated flexure, but non-linearly under compression.

2.7.1.4 Asphalt Mixes

Asphalt cement exhibits a linear viscoelastic response, but asphalt aggregate mixtures were found to display non-linear viscoelastic behavior, or even behavior which was not viscoelastic (Krokosky, et al., 1963). Based on uniaxial creep tests, Monismith, et al. (1966) concluded that for uniaxial strain of less than 0.1%, asphalt aggregate mixes behavior is linear.

Terrel (1967) observed that the M_R of asphalt mixtures increases slightly with increase in lateral pressure and it decreases as the deviator stress was increased. Chatti (1987) and Baladi (1988) also found similar trends in another study. Through repetitive triaxial confining tests Trollope, et al. (1962) found that the rebound modulus increased almost linearly with increase in the confining pressure.

2.7.1.5 Pavement Structures

Sparrow, et al. (1966) performed plate bearing tests on a roadbed soil consisted of a homogeneous silty clay of medium plasticity in a test pit. The test results showed stress-softening material non-linearity. This observation was also confirmed by others through independent tests on silty clay (Mitray, 1964; Wang, 1968; Terrel, 1967). The most marked non-linear effects were observed at pressures below 3 to 6 psi, at higher pressures a linear behavior was observed.

Mitray (1964) observed stress hardening behavior for pavement structures consisting of a gravel base over a highly plastic roadbed soil. The response of the roadbed soil was observed to be of the stress softening type but the overall results were controlled by the over riding stress hardening behavior of the base material. Shifley (1967) performed similar test on an actual pavement structure. During its construction, different paving layers were subjected to plate bearing tests. The clay road

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roadbed was found to be of the highly stress softening type, but an addition of 11 in. thick crushed base aggregates changed the behavior to a stress hardening type, and the subsequent addition of a 2.4 in. thick asphalt concrete (AC) layer rendered the pavement response almost linear. The pavement when completed with a further addition of 4.8 in. AC layer showed markedly stress softening behavior.

The test results at the AASHO road test (1962) confirmed the pattern of non-linearity described above under actual traffic loading. The non-linearity of the measured pavement deflections was found to be much less pronounced than those of the constituent materials in the laboratory. This phenomenon can be explained by the fact that in a layered system consisting of stress-hardening and stress-softening materials, the opposite effects counter each other, reducing the overall effect of material non-linearity.

2.7.2 Temperature Dependency

Asphalt cement and asphalt-aggregate mixes are known to behave elastically at low temperatures, whereas the behavior tends to be viscoelastic at higher temperatures. Temperature dependency of the asphalt cement behavior was related to its stiffness characteristics (Heukelom, 1969; Mcleod, 1969). Van der Poel (1954) devised the Shell nomograph for estimating the asphalt layer stiffness with changing temperatures. Heukelom (1969) modified the Shell method to make it applicable to North American asphalts. Further, modification was suggested by McLeod (1969), who proposed the Penetration-Viscosity-Number (PVN) as a means of characterizing the asphalt. As a result of a study conducted at the state of Washington (Bubusait, et al., 1974), an empirical relationship was suggested for temperature adjustment of the asphalt cement modulus. The results of the study also indicated that the sensitivity of the backcalculated asphalt layer modulus to temperature depends on the condition of the pavement. New pavements being affected more than distressed ones.

The problem of temperature measurement and application of temperature correction to the asphalt layer modulus is discussed in more detail in section 2.9.1.

2.8 MECHANISTIC ANALYSIS MODELS

The load-carrying capacity of a flexible pavement is enhanced by the load-distribution characteristics of its layered system. The system consists of various paving layers with the highest quality material placed at the top (Yoder, et al., 1975). The load distribution over the roadbed soil is achieved by building up thick layers of paving materials. Early calculations of stresses in flexible pavements were based on linear elastic theory. Since then efforts have been made to improve the basic models to incorporate non-linear and material damping effects under traffic loads. Although more complicated techniques to model the pavement response are now available, (e.g., dynamic, viscoelastic) layered elastic analysis is still widely used because of its simplicity and ease with which the required input data can be acquired in practice. Elastic layer analysis and some other models which have traditionally been used are reviewed in this section.

2.8.1 Layered Elastic Model

Multi-layered elastic theory has been extensively used to model the stresses and strains in flexible pavements. The basic multi-layered system as pictured by Yoder, et al. (1975) is shown in Figure 2.9. The analytical solution based on elastic theory has several inherent assumptions including:

1. The material properties of each layer are homogeneous and isotropic.
2. Each layer has finite thickness except the roadbed soil, and all are infinitely wide in the lateral directions.
3. Full friction between the paving layers is developed at each interface .

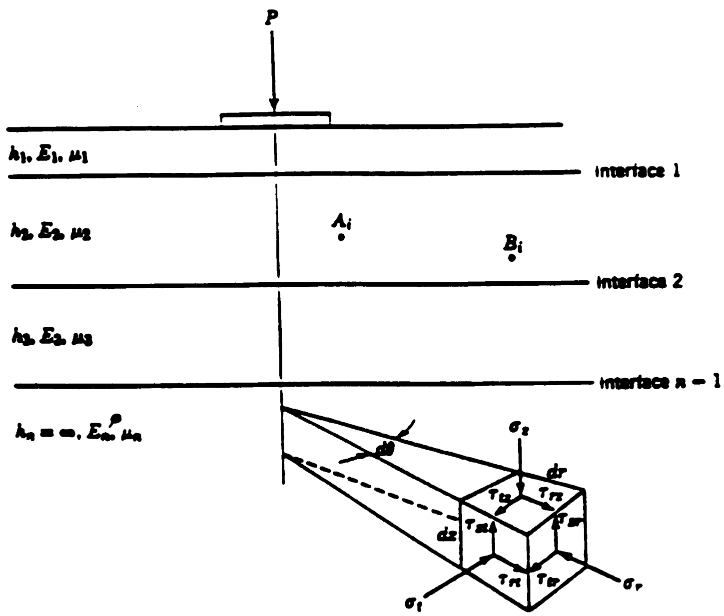

Figure 2.9. Multi-layered elastic system (Yoder, et al., 1975).

4. There are no shearing forces at the pavement surface.

Early calculations of stresses and strains in flexible pavements were based on Boussinesq's equations originally developed for a homogeneous, isotropic, and elastic half-space subjected to a point load. The vertical stress at any point below the earth's surface due to a point load at the surface is given by Boussinesq's formula as

$$\sigma_z = k \frac{P}{Z^2} \tag{2.7}$$

$$k = \frac{3}{2\pi} \frac{1}{[1+(r/z)^2]^{5/2}} \tag{2.8}$$

where r = radial distance from the point load; and

z = depth.

According to the above equation the vertical stress is independent of the properties of the medium and depends only on the vertical depth and radial distance from the load. By treating the whole pavement as a homogeneous and isotropic half-space, it is assumed that the contribution of pavement layers above the subgrade towards the total surface deflection is negligible (Yoder, et al., 1975).

Burmister (1943; 1958) developed a solution for a two-layered elastic system. The materials in both layers are assumed to be homogeneous, isotropic and elastic. The surface layer has finite depth and is assumed to be infinite in the lateral direction, whereas the lower layer is infinite in both the lateral and vertical directions. Both layers are assumed to have a full contact and the surface layer is free of shear and normal stresses outside the loaded area. This model takes into account the properties of the materials above the subgrade. It also accounts for a uniformly distributed circular load which is a better representation of the wheel load than a point load. Further, the high stiffness of the surface layer has a pronounced effect on the vertical str

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stresses and strains. In contrast to Boussinesq's solution, the stress gradients obtained by two layer theory are appreciably different from those obtained using a homogeneous half-space.

Burmister and other researchers (Acum, et al., 1951; Jones, 1962; Peattie, 1962) expanded the solutions to three layer systems and presented the radial and vertical stresses in tabular and graphical forms.

The advent of microcomputers made it possible to extend the linear elastic layer analysis to systems with more than three layers. Many computer programs are now available which can handle up to ten paving layers. Mathematical derivations and comprehensive explanation of concepts and assumptions pertaining to the elastic layer theory can be found elsewhere (Higdon, 1967; Westergaard, 1964; Timoshenko, 1987). Despite the availability of more advanced and complicated analysis models, the elastic layer solution is still widely employed for pavement analysis because of its practicality and simplicity.

2.8.1.1 Advantages and Limitations of Elastic Layer Theory

The basic advantages of elastic layer theory are:

1. It satisfies the laws of mechanics and is thus capable of making consistent calculations.
2. It is a relatively simple method and the calculation effort and capabilities required of computers are small.
3. Iterative variations of elastic layer theory that can approximately account for the non-linear variation of material properties in the vertical direction have also been developed, but for highly stress sensitive materials these may be inadequate.
4. Only two parameters, resilient modulus E and Poisson's ratio μ, are required in elastic layer theory. The results are not too sensitive to the value of

Poisson's ratios and reasonable values are assumed in most cases. This leaves E as the only required material property, along with the layer thicknesses, as an input to perform the analysis.

The limitations and inconsistencies related to the elastic layer assumptions are:

1. The behavior of the paving materials is not purely elastic. It has plastic, and visco-elastic components.
2. The stress-strain relationships for the materials are not linear for the range of stress levels encountered in pavements. Linear elastic theory may be inadequate to model highly stress sensitive materials when encountered.
3. Most paving materials are particulate in nature, hence, they are neither homogenous nor isotropic.
4. The stress-strain characteristics of most materials vary over time in all three dimensions.
5. Boundary conditions are quite complex and different from those assumed by elastic layer theory.
6. Actual traffic and the FWD apply dynamic loads to the pavement, elastic layer analysis is typically used with static loading.

2.8.2 Hogg's Model

Hogg (1938; 1944) modeled the pavement as a thin plate resting on an elastic subgrade. This model assumes that the vertical stresses within the pavement structure are small and can be neglected. The model has yielded good results for the backcalculation of roadbed modulus values. A major advantage of this model is that the roadbed modulus can be estimated without a prior knowledge of the characteristics of the pavement layers (Wiseman, et al., 1985). The model was used to evaluate three layer pavements using deflection data obtained by the La Crox-L.C.P.C.

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Deflectograph. The results were satisfactory (Hoyinck, 1982). Details about the model have been presented elsewhere (Wiseman, 1975; Wiseman, 1983).

2.8.3 Equivalent Thickness Model

All equivalent layer models developed for estimating deflections of multilayered pavements share Odemark's assumption (Odemark, 1949). Odemark's assumption is used to convert a multilayered elastic system to a single layer elastic model. He suggested that deflections of multilayered pavement with moduli E_i, and layer thicknesses h_i, can be approximated by a single layer thickness, H, and a single modulus E_o, if the thickness H is calculated as;

$$H = \sum_{i=1}^{m} Ch_i \, (E_i \, / \, E_o)^{1/3} \tag{2.9}$$

where H = the equivalent thickness;

h_i = actual thickness of the ith layer;

E_i = the elastic modulus of the ith layer;

E_o = the modulus of a single layer to which the multilayered system has been converted; and

C = constant.

The equivalent thickness method has limitations in its application and for certain types of pavements is known to give erroneous results (Kuo, 1979; Lytton, et al., 1979; Hung, et al., 1982). The method has the advantage of simplicity and speed of computation.

Ulliditz (1978) used the equivalent layer model successfully for pavements with all linear elastic materials and also for pavements with non-linear roadbed soils. Lytton (1989) also used the model with some modifications for the analysis of flexible pavements.

2.8.4 Finite Element Method

This is the only method which theoretically can adjust the stiffness of each element according to its own stress state. The method recognizes that for a nonlinear material the modulus is not characteristic of the whole layer but instead pertains to a point within that layer.

MICHPAVE (Ming-Shen, 1989) is one such program which uses finite element method for linear and non-linear elastic analysis of layered system. In this program the limitation of modeling an infinite subgrade by deep fixed boundary has successfully been overcome by the incorporation of a flexible boundary concept.

There are no known backcalculation programs directly based on finite element analysis because of computational time required. Instead, the data generated from finite element programs has been used to generate regression equations in order to account for the non-linearity of the paving materials (Hoffman, et al., 1982). Such an approach requires lesser time for backcalculation but all the limitations of regression type analysis of deflection data are applicable.

2.8.5 Dynamic Analysis

Static analysis are generally used to backcalculate the layer moduli of pavements regardless of the load application mode. A load applied dynamically is not equivalent to the static loading and so should not be the stress and strain fields induced by each loading mode (Wiseman, et al., 1972; Stolle, et al., 1989). However, conflicting views regarding the magnitude of the error induced by the inertial response of pavements to dynamic loading can be found in the literature. Some researchers have reported significant error (Davies, et al., 1985), while others found the error to be insignificant (Roesset, et al., 1985).

Davies and Mamlouk (1985) argued that the single-degree of freedom (SDF) models employed by most researchers are inadequate. They stressed the need to use elastodynamic solution for the analysis of pavement deflection data, which can account for loading from multi-directions. In elastodynamics, Helmholtz equation for steady-state harmonic motion is used (Erigen, et al., 1975):

$$C_p^2 \text{ grade(dive } u) - C_s^2 \text{ curl(curl } u) + \rho^2\omega^2 u = 0 \qquad (2.10)$$

where c_p and c_s = the pressure and shear wave velocities, respectively;

u = displacement vector; and

ω = the circular frequency of the excitation.

The displacement vector **u** can be expressed in the form:

$$u(t) = u^* e^{i\omega t} \qquad (2.11)$$

where $\mathbf{u}^*$ = complex amplitudes of the displacement vector;

t = time; and

i = unit imaginary number.

The wave velocities are related to the stiffness and mass density of the material by:

$$C_p = \left[\frac{E(1-\mu)}{(1+\mu)(1-2\mu)\rho}\right]^{\frac{1}{2}} \qquad (2.12)$$

$$C_s = \left[\frac{E}{2(1+\mu)\rho}\right]^{\frac{1}{2}} \qquad (2.13)$$

where E, μ and ρ are Young's Modulus , Poisson's Ratio, and mass density, respectively.

A closed form solution for equation 2.12 is available only for a point load excitation on a homogeneous half-space. Numerical solutions, must be obtained for a multi-layered system. The usual assumptions of linear elastic material and isotropy are invoked. The soil and pavement layers are assumed to be unbounded laterally, bedrock is assumed at a finite depth and full bonding is assumed at all layer interfaces (Davies, et al., 1985).

Using the numerical solution technique presented by Kausel and Peek (1982) the in-phase and out-of-phase displacements at any location throughout the pavement can be obtained (Sebally, et al., 1986). Davies, et al. (1985), however, have stressed the complexity and difficulties associated with the analysis despite making a number of simplifying assumptions.

2.8.5.1 Extension of Dynamic Analysis to FWD

Loads applied by the FWD are transient in nature and not harmonic. The Fourier transform is used to represent the transient load by the sum of the harmonic load over different frequencies and amplitudes (Roesset, et al., 1985). Sebaaly et al. (1986) assumed a periodic loading impulse with period T, which they divided into a loading pulse-width, t_p, and a rest period, T_R, (Figure 2.10). The loading pulse width is a function of the loading device and pavement system properties, with typical values ranging between 25 and 60 msec for most FWD devices. The rest period T_R is chosen to be large enough such that the pavement fully recovers from deformation and hence the response of every drop is independent of the earlier one.

The Fourier coefficients for the load impulse expansion are analyzed and then the phase lag, frequency, and amplitude of each harmonic response component are obtained. The harmonic responses are summed in the time domain to obtain the complete response due to the impulse. Similar solution was sought by Roesset and Young (1985).

2.8.5.2 Comparison of Static and Dynamic Analyses

1. Static analysis of dynamically loaded pavements result in a significant error if the frequency of the applied load is approximately equal to the resonant frequency of the pavement system, or if the resonant frequency is so high that the inertial forces become dominant (Davies, et al., 1985).

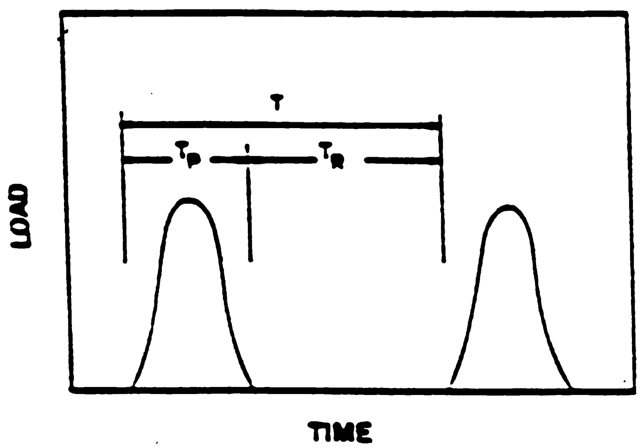

Figure 2.10. Assumed periodicity of FWD impulses (Sebaaly, et al., 1986).

2. Static interpretation of the deflections measured by an FWD test is reasonable when the depth of the bed rock is more than 60 feet. The resulting error increases when the subbase material is not homogeneous and/or when its stiffness increases with depth (Roesset, et al., 1985).
3. Dynamic effects are less important for FWD loading because its load covers a wide band of frequencies. The results obtained by Roesset, et al. (1985) using dynamic and static analysis of FWD deflections showed that the difference in backcalculated moduli using two methods was small. In an independent study conducted on three different pavement structures in the United Kingdom, Tam, et al. (1989) concluded that the differences in the results of backcalculated moduli, for static and dynamic analysis using FWD deflection basins, were insignificant.
4. Stolle and Hein (1989) observed from the results obtained by Sebaaly, et al. (1986) that better agreement exists between the deflection basins measured by FWD and that predicted by static analysis, than between that predicted by dynamic analysis (Figure 2.11). They also pointed out from a previous study (McCullough, et al., 1982) that while the accuracy of the measured deflection values is important to evaluate the roadbed modulus, the shape of the deflection basin is more important to accurately evaluate the pavement layer moduli.

2.8.6 Nonlinear Elasticity

Stress-strain curve for many paving materials are nonlinear. One simple method of dealing with such materials is to replace the elastic constants in the linear stress-strain relations with tangent moduli dependent upon stress or strain. The elastic constants can be obtained by using piece wise linear models. Such an approach is called a Cauchy elastic formulation (Chen, et al., 1985).

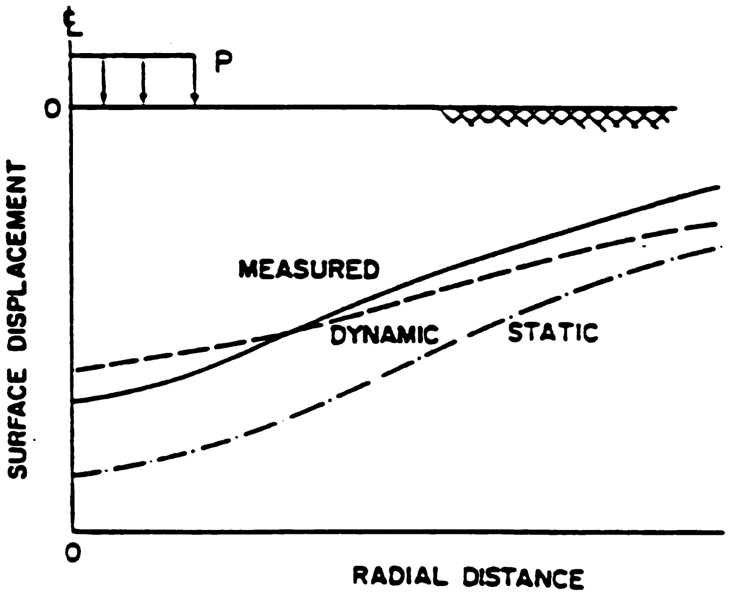

Figure 2.11 Schematic showing measured and predicted surface deflection basins (Stolle, et al., 1989)

The Cauchy type of elastic models may generate energy for certain types of loading-unloading cycles. Hyperelastic models do not suffer from this drawback. Both of these models suffer from the disadvantage that they are independent of the stress or strain path, which is not true for soils in general.

A more realistic and rational description is provided by the hypoelastic formulation in which the incremental stress and strain tensors are linearly related through variable material response moduli that are functions of the current state of stress or strain. Details of the three formulations and their respective characteristics can be found elsewhere (Chen, et al., 1985). Here, only the second order stress-strain relationships are reproduced (Uzan, 1993).

The constitutive relationship for the Cauchy model is of the form

$$\sigma_{ij} = (C_1 I_1 + C_2 I_1^2 + C_3 I_2)\delta_{ij} + (C_4 + C_5 I_1)\epsilon_{ij} + C_6 \epsilon_{ik}\epsilon_{kj} \tag{2.14}$$

while for the hyperelastic model it is

$$\sigma_{ij} = (2C_1 I_1 + 3C_2 I_1^2 + C_3 I_2)\delta_{ij} + (C_4 + C_3 I_1)\epsilon_{ij} + C_5 \epsilon_{ik}\epsilon_{kj} \tag{2.15}$$

In the hypoelastic model, the stress rate is expressed in terms of the stresses and strain rates by:

$$\dot{\sigma}_{ij} = C_o \dot{\epsilon}_{kk}\delta_{ij} + C_1 \dot{\epsilon}_{ij} + C_2 \sigma_{nn}\dot{\epsilon}_{kk}\delta_{ij} + C_3 \sigma_{nn}\dot{\epsilon}_{ij} + C_4 \sigma_{ij}\dot{\epsilon}_{kk}$$

$$+ C_5(\sigma_{im}\dot{\epsilon}_{mj} + \dot{\epsilon}_{im}\sigma_{mj}) + C_6 \sigma_{mn}\dot{\epsilon}_{nm}\delta_{ij} \tag{2.16}$$

where σ_{ij} , ϵ_{ij} = components of stress and strain tensor;

$\dot{\sigma}_{ij}$, $\dot{\varepsilon}_{ij}$ = components of stress and strain rate tensor;

I_1, I_2 = stress invariants; and

C_0, to C_6 = material parameters.

These formulations, while having a strong mechanistic basis, are not widely used because the material constants have little or no physical interpretation (Uzan, 1993). Uzan (1985) presented a simplified and general model for the secant resilient modulus of paving materials as:

$$MR = k_1 p_a \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3} \tag{2.17}$$

where p_a = atmospheric pressure;

τ_{oct} = octahedral shear stress;

θ = bulk stress; and

k_1 , k_2 , and k_3 = material constants.

This model is a simplified form of the non-linear Cauchy model, and though appealing, violates the laws of thermodynamics. In a recent paper Uzan (1993) presented a modification of this model where two more material constants have been added to account for the non-liner behavior of Poisson's ratio and are derived by imposing the path independence of the strain energy density function.

The five parameters for each paving layer cannot be directly backcalculated from the set of deflection data measured by FWD equipment. Hence, Uzan suggests that only k_1 be backcalculated, while k_2 - k_5 be estimated from laboratory testing or existing data banks. This partially defeats the purpose of backcalculation, making non-linear models unappealing at present.

2.8.7 Viscoelastic Model

Asphalt displays viscoelastic behavior, and since FWD loadings are essentially dynamic, some researchers have used viscoelastic models to obtain a more accurate representation. In general, even granular and cohesive material can be modeled as being viscoelastic. Viscoelastic material can be easily modeled by using a complex-valued modulus (Wolf, 1985) as:

$$E^*(\omega) = E'(\omega) + iE''(\omega) = E'(\omega)[1 + i2\xi] \tag{2.18}$$

where E^*, E', E'' = complex-valued modulus and its real and imaginary parts;

ω = circular frequency; and

ξ = damping ratio.

Granular materials are normally assumed to have a constant damping ratio (Uzan, 1993). Nonlinear viscoelastic models are used more and more for dynamic backcalculation but are relatively complex and hence have not been used widely in practice. The increased number of parameters makes it difficult to backcalculate all of them solely from FWD measurements.

2.8.8 Pavement Material Type and Choice of Analysis Model

Differences in opinion regarding the degree of non-linearity of pavement materials can be found throughout the literature. Uzan (1993) recommended the use of complex moduli to model the viscoelastic behavior of pavements and recommended that data of the last few deflection sensors not be used in the backcalculation if the complex moduli tended to increase away from the applied load. He further emphasized the importance of using linear dynamic analysis when the bedrock is present at a shallow depth. For deep bedrock Uzan recommends the use of the static non-linear backcalculation as the state-of-the-art improves. Presently the use of non-linear or viscoelastic models require some of the material properties to be inferred from existing data banks or from laboratory tests. As a result, they have not found much use in practice, and backcalculation based on simple linear elastic models are still the most popular.

2.9 BACKCALCULATION METHODS

Existing backcalculation routines can be classified into three major groups depending on the techniques used to reach the solution. These three techniques may have any of the forward analysis methods, discussed earlier, embedded in them. The first group is based on iteration techniques, which repeatedly use a forward analysis method within 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 data base 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. Some of the known backcalculation computer programs and their characteristics (adopted from Mahoney, et al., 1991) are presented in Table 2.2.

2.9.1 Iterative Methods

The ultimate objective of most backcalculation methods is to find a set of moduli such that the calculated deflection basin match the measured one within a specified tolerance. This is usually achieved by minimizing an objective function which is commonly defined as the weighted sum of squares of the differences between calculated and measured surface deflections (Uzan, et al., 1989) i.e.,

$$minimize \;\; f = \sum_{j=1}^{m} \alpha_j [\omega_{j_m} - \omega_{j_c}]^2 \qquad (2.19)$$

where w_{jm} = the measured deflection at sensor j;

w_{jc} = the calculated deflection at sensor j; and

a_j = a weighing factor for sensor j.

The flow chart (Lytton, 1989) presented in Figure 2.12, illustrates this process. The main steps of the iteration process are:

Table 2.2. **Backcalculation programs compiled by SHRP as of November 1990 (modified after ref. Mahoney, 1991).**

Program Name	Developed By	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Subroutine	Non-linear Analysis	Seed Modulus	Comments
BISDEF	A. Bush USACE-WES	Multi-Layer Elastic Theory	BISAR	ITERATIVE	No	Required	Sensitive to seed modulus. Uses gradient Search method.
BOUSEDEF	Zhou, et al.	Equivalent Layer Thickness	MET	ITERATIVE	Yes	Required	Program logic similar to BISDEF.
CHEVDEF	A. Bush USACE-WES	Multi-Layer Elastic Theory	CHEVRON	ITERATIVE	No	Required	Sensitive to seed modulus.
COMDEF	M. Anderson	Multi-Layer Elastic Theory	DELTA	DATA BASE	No	Required	For composite pavements only.
DBCONPAS	M. Tia, et al.	Finite Element	FEACONSIII	DATA BASE	Yes		For rigid pavements only.
ELMOD	P. Ulidtz	Equivalent Layer Thickness	MET	ITERATIVE	Yes roadbed only	Not required	Fast, but has limitations inherent to MET program
ELSDEF	Texas A&M University	Multi-Layer Elastic Theory	ELSYM5	ITERATIVE	No	Required	Sensitive to seed modulus.
EMOD		Multi-Layer Elastic Theory	CHEVRON	ITERATIVE	Yes roadbed only	Required	

Table 2.3. *Backcalculation* programs compiled by SHRP as of November 1990 (modified after ref. Mahoney, 1991) (continued).

Program Name	Developed By	Forward Calculation Method	Forward Calculation Subroutine	Backcalculation Subroutine	Non-linear Analysis	Seed Modulus	Comments
EVERCALC	Mahoney, J., et al.	Multi-Layer Elastic Theory	CHEVRON	ITERATIVE	Yes	Not required for up to 3 layers	Primarily for flexible pavements.
FPEDDI	W. Uddin	Multi-Layer Elastic Theory	BASNIF	ITERATIVE	Yes	Not required	
ISSEM4	P. Ulidtz	Multi-Layer Elastic Theory	ELSYM5	ITERATIVE	Yes	Required	Uses deflections at five point to calculate moduli for three layers.
MODCOMP2	L. Irvin	Multi-Layer Elastic Theory	CHEVRON	ITERATIVE	Yes	Required	More oriented for research work.
MODULUS	J. Uzan	Multi-Layer Elastic Theory	WESLEA	DATA BASE	Yes	Required	Used in an expert system frame work.
PADAL	S. F. Brown	Finite Element		ITERATIVE	Yes	Required	
RPEDDI	W. Uddin	Multi-Layer Elastic Theory	BASINR	ITERATIVE	Yes	Not required	For rigid pavements only.
WESDEF	USACE-WES	Multi-Layer Elastic Theory	WESLEA	ITERATIVE	No	Required	Sensitive to seed modulus.

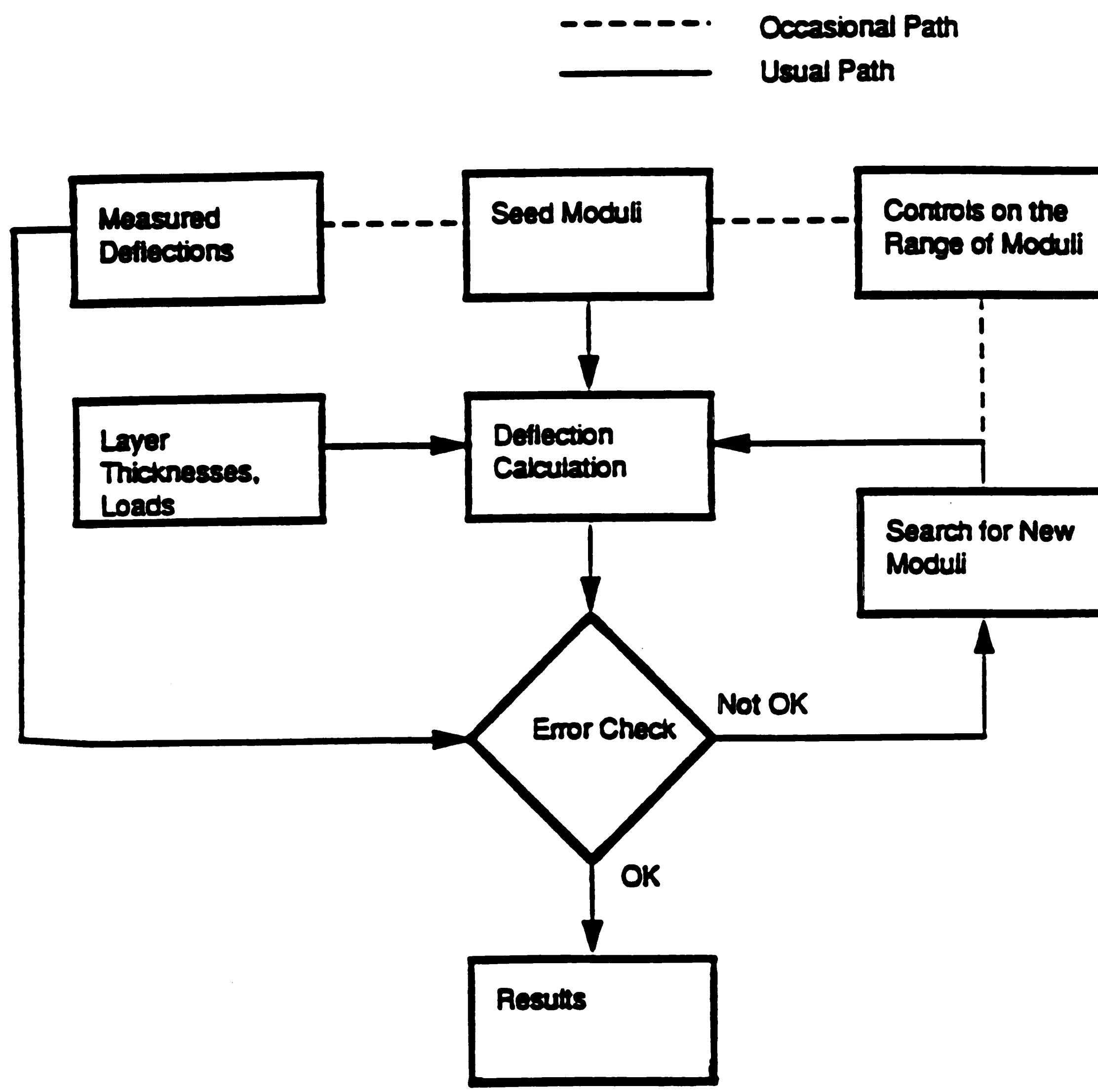

Figure 2.12. Typical flow chart for an iterative program (Lytton, 1989).

Step 1. Surface deflections at known distances away from the applied load are measured.

Step 2. Layer thicknesses, load application characteristics, and Poisson's ratios for each layer are required to be input by the user. In almost all programs constant values of Poisson's ratios are used.

Step 3. In order to start the forward calculation process, approximate layer modulus values (seed moduli) are required as input. Seed moduli are sometimes generated by the program using measured deflections and regression equations, or else they must be specified by the user. Some programs use a database approach at this stage to obtain seed moduli.

Step 4. The data specified in step 2 and the latest set of layer moduli are used to calculate surface deflections at the same radial offsets at which the deflections were measured.

Step 5. An error check is performed to assess if the measured and calculated surface deflections are within a specified tolerance limit. Different techniques are used at this stage to adjust the set of layer moduli so that 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 procedure based programs. Steps 4 and 5 are repeated until the value of the objective function is sufficiently small or the adjustments to the layer moduli are very small.

One of the problems faced with this approach is that the multi-dimensional surface represented by the objective function may have many local minima. As a result the program may converge to different solutions for a different set of seed moduli. Some programs overcome this problem by automated assignment of the 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 (Sivaneswaran, et al., 1991), which uses an efficient and general minimization method (Levenberg-Marquardt algorithm). The program seeks to minimize an objective function formed as the sum of squared relative difference 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 "---DEF" series of programs use an assumed linear variation in logarithmic space 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 (Bush, 1980) is one such example in which the CHEVRON program (Michelow, 1963) is used for forward calculations. A set of seed moduli are required to be input by the user in this program to start the iteration process. The simplified description of the process to find new layer modulus from an initial guess for one layer and one deflection is shown in Figure 2.13. For multiple deflections and layers a set of equations defining the slope and intercept for each deflection and each unknown layer modulus is developed as follows (Van Cauwelaert, et al., 1989):

$$\log\,(\text{deflection}_j) = A_{ji} + S_{ji}\,(\log E_i) \tag{2.20}$$

where A_{ji} = intercepts;

S_{ji} = slope;

j = 1,2,...ND (ND=No. of deflections); and

i = 1,2,...NL (NL=No. of layers with unknown moduli).

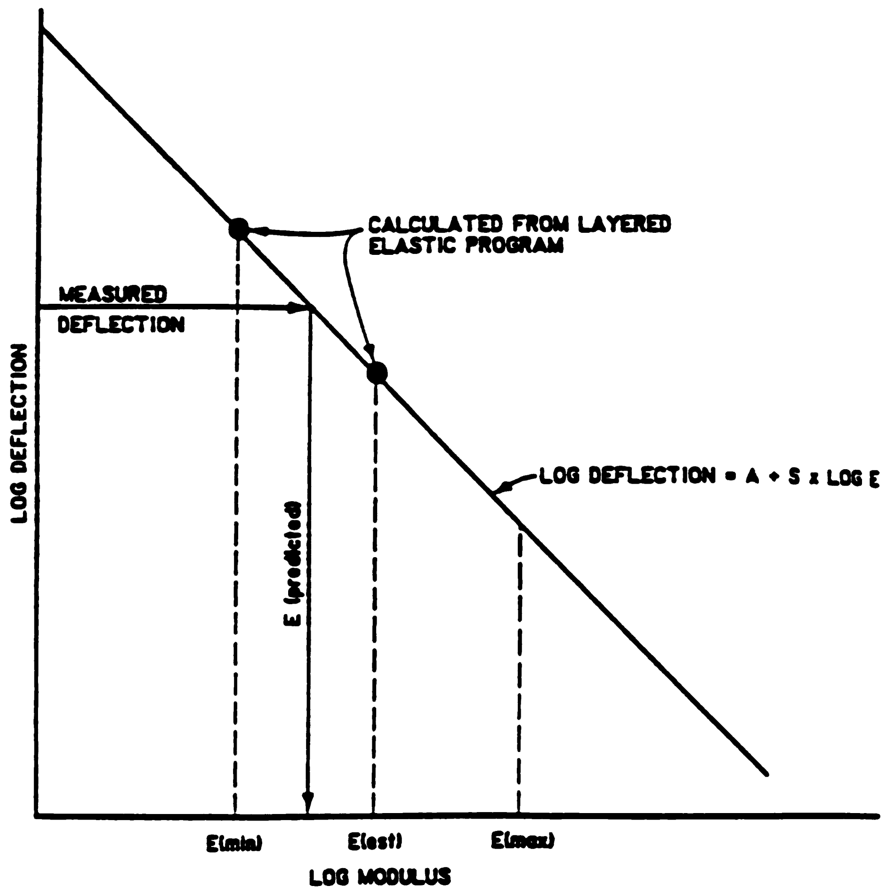

Figure 2.13. Basic process for matching deflection basins (Van Cauwelaert, 1989).

The linearization of the model in logarithmic space simplifies the search for new set of moduli. However the results obtained by these programs are highly dependent on the initial seed moduli.

Some programs such as ELMOD (Ullidtz, et al., 1985) use an equivalent layer thickness concept along with Bousinesq's equation in an iterative program to backcalculate the layer moduli. There are some other programs which are based on the elastic layer concept but which first estimate the subgrade modulus based on the deflections measured by the outermost senors. The subgrade modulus is then fixed and intermediate layer moduli are estimated using the middle sensor deflections and finally the AC modulus is inferred from the set of inner-most sensor deflections. The error in the estimated moduli of lower layers thus contribute to the errors in the moduli of the upper layers.

2.9.2 Database Approach

In this method, a forward calculation program is used to generate a data base of deflection basins for different combinations of layer moduli, and 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 which produced the closest calculated deflection basins in the database.

The MODULUS backcalculation program (Uzan, 1985) is one such example which uses databases generated by WESLEA (Van Cauwelaert, 1989) program. The number of basins required to obtain a suitable database depends upon the number of layers and the expected moduli ranges provided by the user. Wide ranges require a greater number of basins to be generated 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 moduli. The program seeks to obtain a set of moduli which will minimize an objective function defined as the relative sum of squared differences between the measured and calculated surface deflections. The program is known to converge always, although the chances of converging to a local minimum cannot be ruled out (Scullion, et al., 1990). 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 suited when a large number of pavements with similar configuration are to be tested in continuation. For these situations the data bank once generated can be used repeatedly to backcalculate moduli for all similar pavements, and the time required to generate the database can be minimized. This technique can be used with database generated from any linear or nonlinear program (Chua, et al., 1984). The results obtained are moderately accurate, but the accuracy of the results is sensitive to the expertise of the user and his or her knowledge of pavement materials.

The COMPDEF (Chua, 1989) program also uses a database approach to backcalculate the layer moduli. The program uses a precalculated database of composite pavement deflection basins stored in a matrix, which is searched by an interpolation technique to find the layer moduli.

2.9.3 Statistical Analysis

This method is similar to the database technique, the only difference being in how the database is used. The database is created by using any forward calculation routines, and then statistical analysis is performed to generate regression equations. These equations take the deflections as independent variables and attempt to predict the values of the layer moduli. Pavements of different configuration 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 have to be treated separately.

The LOADRATE program (Chua, 1989) belongs to this category and uses regression equations generated from a database obtained by using the ILLIPAVE non-linear finite element program (1982). This technique is best suited for agencies which deal with a limited and known type and configuration of pavements. Data bank generation to include all the expected combinations of pavement layers in the initial stages can offset this disadvantage to a large extent. Proper statistical interpretation of data can give reasonably accurate results. 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 which was used to generate the statistical equations represents the pavement being analyzed.

2.9.4 Conversion of Backcalculated Layer Moduli to Standard Conditions

The modulus of the asphalt concrete surface course is significantly affected by the temperature and frequency of loading. On the other hand, the base, subbase and roadbed soil moduli are more affected by confining pressure, moisture, and stress levels. For a better interpretation of the modulus values, the backcalculated moduli should be converted to some standard conditions (moisture and temperature). The conversion to standard conditions is referred to as corrections to backcalculated moduli.

2.9.4.1 Temperature and Frequency Corrections

The temperature correction procedure suggested by the Asphalt Institute is most commonly used for pavements with more than 2 in. thick AC layer (1977). The mean temperature of the pavement must be calculated for the same time when the pavement deflections are measured. This procedure requires exhaustive data including:

1. The maximum and minimum temperatures for five days prior to the day the test is performed.
2. The pavement surface temperature at the time of NDT is conducted.
3. The frequency of loading or in case of FWD loading the time duration of the load impulse .
4. The percent asphalt cement content by weight.

The chart in Figure 2.14 is used to determine the temperature at the top, middle, and bottom of the asphalt layer. The data in steps 1, 2, and 3 described above are required to use the figure. The mean of the three temperatures is considered as the mean temperature of the pavement.

Southgate (1968) presented a slightly different procedure than the one described above to find the mean temperature of pavements having an asphaltic concrete layer thickness of less than or equal to 2 in. This procedure stresses that for thin asphaltic layer pavements, the hour of the day and the amount of heat absorption is more important than the maximum and minimum temperatures used in the Asphalt Institute method. Figure 2.15 is used to find the temperature on the underside of a thin asphaltic concrete layer. This temperature and that of the surface of the layer at the time of testing are then averaged.

The frequency of loading is also required to use the Asphalt Institute equation (1982). The frequency in the case of a cyclic loading device, such as the Dynaflect or Road Rater, is the actual frequency of loading. In the case of a FWD device the frequency is obtained as:

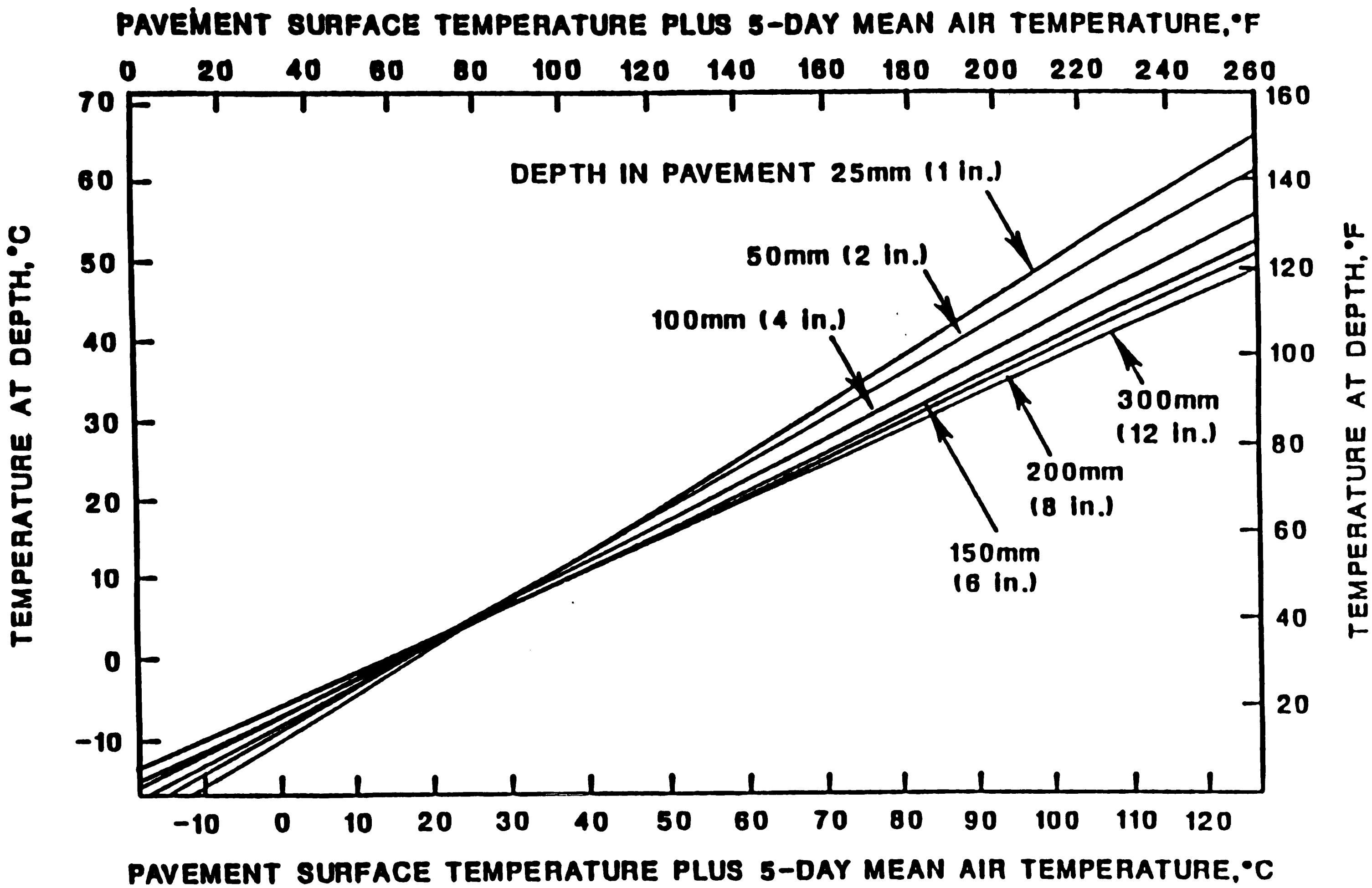

Figure 2.14. Predicted pavement temperatures (Asphalt Institute, 1977).

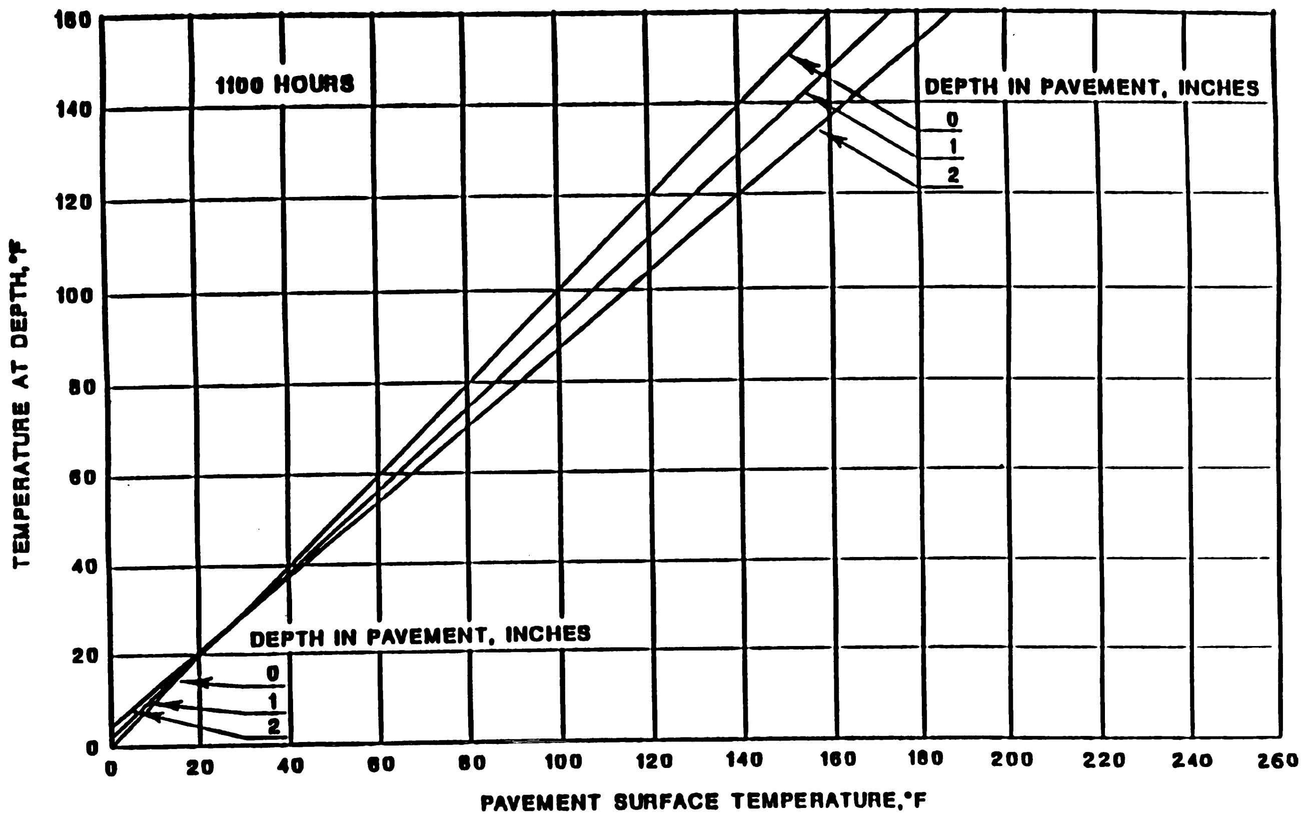

Figure 2.15. Temperature prediction graphs for pavements equal to or less than 2 inches thick (Asphalt Institute, 1977)

$$f=\frac{1}{2t}$$

where t = time duration of the impulse load in seconds.

The frequency and temperature corrected modulus is then calculated using the following equation:

$$\log E_o=\log E+P_{200}\left[\frac{1}{(f_o)^l}-\frac{1}{(f)^l}\right]+0.000005\sqrt{P_{ac}}\left[(t_o)^{r_o}-(t)^r\right]$$

$$-.00189\sqrt{P_{ac}}\left[\frac{(t_o)^{r_o}}{(f_o)^{1.1}}-\frac{(t)^r}{(t)^{1.1}}\right]+.9317\left[\frac{1}{(f_o)^n}-\frac{1}{(f)^n}\right] \qquad (2.21)$$

where l = 0.17033;

n = 0.02774;

E = backcalculated uncorrected modulus;

t,t_o = test and reference temperatures in °F;

f,f_o = loading and reference frequency in Hz;

P_{ac} = percentage of asphalt cement in the mix;

E_o = the corrected modulus;

r_o = 1.3 + 0.49825 log (f_o);

r = 1.3 + 0.49825 log (f); and

P_{200} = the percent passing the No. 200 sieve.

Germann and Lytton (1989) evaluated the accuracy of this correction procedure. They concluded that the procedure gives better results when correcting a modulus measured at a low temperature to a higher standard temperature, than vice versa.

2.9.4.2 Load Level Corrections

Load level corrections are necessary only when the applied load, under which the modulus is required to be calculated is outside the elastic range for any of the paving layers. Also the load level correction is only required for base and roadbed materials since the AC layer behaves almost linearly. Basically two corrections are required (1) for confining pressure level, and (2) for the strain level (Germann, et al., 1989).

All the layered elastic analysis methods make some assumptions regarding the use of an average modulus for the entire paving layer, which technically is incorrect. The FHWA-ARE method assumptions (1975) have been found to provide a suitable basis to apply load level corrections to the average modulus values.

Richard and Abbot (1975) proposed the following equation for the general stress-strain curve (Figure 2.16) for the base, subbase, and roadbed soils:

$$\sigma = E_p \epsilon + \frac{E_r \epsilon}{\left[1 + \left[\frac{E_r \epsilon}{\sigma_y}\right]^m\right]^{\frac{1}{m}}} \qquad (2.22)$$

where σ, ϵ = any stress and its corresponding strain levels;

E_i = initial tangent modulus;

E_p = plastic modulus;

E_r = E_i - E_p ;

σ_y = maximum plastic yield stress; and

m = a material constant.

Germann and Lytton (1989), used this equation and assumed that the resilient modulus is a secant modulus of the curve shown in Figure 2.16, proposed a relation between the secant modulus, E, and the initial modulus, E_i as:

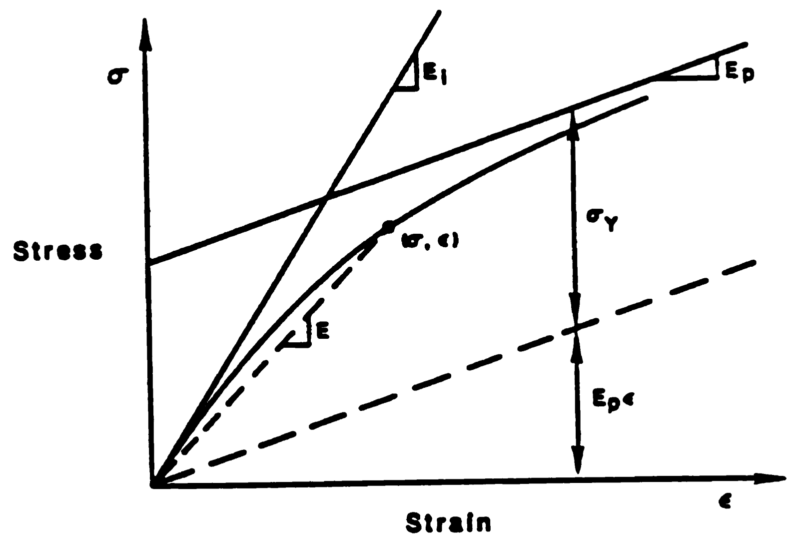

Figure 2.16. General hyperbolic stress-strain curve for base, subbase, and roadbed materials (Richard, et al., 1975).

$$\left[\frac{1-a}{\left(\frac{E}{E_i}-a\right)}\right]^m-\left[\frac{(1-a)\epsilon}{b}\right]^m=1 \tag{2.23}$$

where a = E_p / E_i;

b = σ_y / E_i;

E_i = initial tangent modulus; and

m = a material constant.

The unknowns of equation 2.23, a, b, m, and E_i can be found through regression analysis. The final correction equation is of the form (Lytton, et al., 1990)

$$\frac{E_k}{E_j}=\left[\frac{(\sigma_1+\sigma_2+\sigma_3)\ k}{(\sigma_1+\sigma_2+\sigma_3)\ j}\right]^{k_2}\left[\frac{\dfrac{(1-a)}{\left[1+\left[\dfrac{(1-a)\epsilon_k}{b}\right]^m\right]^{1/m}}}{\dfrac{(1-a)}{\left[1+\left[\dfrac{(1-a)\epsilon_j}{b}\right]^m\right]^{1/m}}}\right] \tag{2.24}$$

where E_k / E_j = correction factor to convert backcalculated moduli;

E_k = resilient (secant) modulus at a standard load level;

E_{ik} = initial tangent modulus at the standard load;

E_{ij} = initial tangent modulus at the NDT load level;

a,b,m = dimensionless constants;

ϵ_k = strain under standard load level; and

ϵ_j = strain under NDT load level.

Equation 2.24 is used in an iterative process. A mechanistic program is used to calculate the principal stresses and strains for standard and NDT load levels. Stresses are calculated at mid-depth for the paving layers and at one foot depth for the roadbed soil.

The iterative procedure is initiated by assuming a modulus for each layer under the standard load, the secant modulus for the non-standard load being known for each layer from the analysis of the NDT data. The confining pressure and strains are then calculated for both loading conditions. From known properties of the materials, the modulus for each layer under standard loading conditions are then calculated. If these calculated moduli are significantly different from those initially assumed, the procedure is repeated using the new values until all values are close enough to the values from which they were calculated.

2.9.5 Sources of Error In Backcalculated Layer Moduli

The sources of errors in the backcalculated layer moduli can be classified into two main groups: random errors that are mainly associated with the measuring devices and pavement structural geometry and condition; and the systematic errors associated with the load representation, theoretical model, and the analysis process (Uzan, et al., 1989).

2.9.5.1 <u>Measuring Devices</u>

Load cell and deflection sensors have manufacturer's accuracy specifications which for deflection sensors is ±2% of their full range. This error may be larger depending upon the pavement condition and the way the sensors rest on the pavement. The load cell error seemingly renders error of similar magnitude and nature to the backcalculated moduli. The overall error due to measuring devices may lead to inadmissible errors in the backcalculation. Being random in nature, this error can only be minimized by making a greater number of measurements at each test site and averaging the results.

2.9.5.2 Loading Mechanism

The relative rigidity of the pavement and the loading plate affects the pressure distribution on the loaded area. The pressure concentration will be higher at the edge when the pavement is relatively flexible while a relatively rigid pavement will result in pressure concentrations near the middle of the loading plate.

Replaceable ribbed rubber pads of different rigidities can be used to overcome this problem which has been discussed in detail elsewhere (Uzan, et al., 1989). Also the presence of a hole at the center of the plate is a source of error. Both of these errors are systematic in nature.

2.9.5.3 Pavement Condition and Geometry

Pavement condition, such as the presence of voids, cracks, surface texture of the loaded area and variable layer thicknesses all effect the backcalculated moduli.

The thickness of the upper layer has the maximum effect on the backcalculated moduli (Rwebangira, et al., 1987). Using average values from the cores or from design data may yield a random error. The effect of layer thickness accuracy has been discussed by Irwin, et al. (1989). Similarly the seating of the loading plate may be affected by the rough surface texture of the pavement which again will result in a random error. Seating errors can be minimized by dropping the weight at least two times before the data is recorded.

2.9.5.4 Material Model

Use of different material models affects the backcalculated results in a systematic way. The assumption of linear, homogeneous, isotropic materials can give rise to errors in the backcalculated moduli if the actual materials do not satisfy such assumptions.

2.9.5.5 Analysis Method

The analysis method must conform to the mode of loading. Most NDT devices impart a dynamic load to the pavement, while the analysis methods used are typically static (Mamlouk, 1987). This discrepancy introduces a systematic error in the backcalculated moduli.

2.9.6 Error Measures and Convergence Criteria

In order to assess how well the layer moduli are backcalculated, some measure of the error between measured and calculated surface deflections is required. In iterative methods, this error can be used as a criterion to determine convergence.

Most programs use one of three most frequently used error measures. These are the sum of absolute differences, the sum of squared differences and the sum of squared relative errors between the measured and calculated deflections.

The absolute arithmetic error (AAE) is defined as:

$$AAE,\ \% = \sum_{i=1}^{n} \left| \frac{100*(d_{ci}-d_{mi})}{d_{mi}} \right| \tag{2.25}$$

where n = number of measured deflections;

d_{ci} = calculated surface deflection at sensor i; and

d_{mi} = measured surface deflection at sensor i.

The root mean square (RMS) error is expressed as:

$$RMS,\ \% = \left[\frac{1}{n} \sum_{i=1}^{n} \left(\frac{d_{ci}-d_{mi}}{d_{mi}} \right)^2 \right]^{1/2} \tag{2.26}$$

Irwin, et al. (1989) suggested that the RMS is a better measure of the goodness of fit because its magnitude is unconstrained by the number of sensors, while the AAE is directly proportional to the number of sensors.

Uzan, et al. (1989) also argued that since the accuracy of the sensors, a major contributor to random errors, is normally specified in relative terms, the objective function to minimize the deflections at respective sensors should also be expressed in relative terms. The error measure they suggested takes the form:

$$\epsilon^2 = \sum_{i=1}^{n}\left[\sum_{j=1}^{d}\left(\frac{d_{ij}^{m}-d_{ij}^{c}}{d_{ij}^{m}}\right)\right]^2 w_i \tag{2.27}$$

where d_{ij}^{m}, d_{ij}^{c} = measured and calculated deflections at *ith* sensor and *jth* drop;

w_i = weight assigned to the *ith* sensor;

n = number of sensors; and

j = number of drops with approximately the same drop height.

This function has been used in the MODULUS program, and differs from the RMS only slightly.

The third measure, the average of AAE, is obtained by dividing the AAE by the number of sensors.

2.10 SUMMARY

Deflection testing is the most widely accepted and used method for evaluating pavement performance. The load applied by the FWD is considered to resemble actual traffic loading. The test is simple to perform and is automated. Dynatest is rated as the most efficient FWD equipment available.

Deflections are affected by season and temperature at the time of testing, the condition of the pavement and the moisture in the roadbed soil. Seasonal correction factors developed for a general geographic area must be used.

Paving material response to traffic loading is non-linear. Most studies, however, have highlighted the fact that in a layered system, the overall non-linear effect is reduced because of the interaction of different paving materials.

The linear layered elastic model is used most widely to analyze FWD data despite its shortcomings. A number of more complex models have been presented recently with claims of having overcome some of the limitations of the elastic layer model. However these models have not been tested extensively, they are quite complex to use, and may even require expert users. Further, the various material parameters required in these complex methods often cannot be inferred from the present NDT equipment. Hence, they must be inferred either from existing databases or from laboratory testing. As such the advantages associated with these models mostly come from their theoretical soundness.

Existing backcalculation programs are mostly engineered to minimize an objective function, which is constructed from the observed and measured surface deflections. These programs suffer from the disadvantage that the backcalculated results may be a consequence of convergence to a local minima rather than a global one. Some of these programs have incorporated some measures to guard against such a convergence, but the effectiveness of these measures vary from one program to another. Existing methods of predicting the stiff layer depth from FWD deflection data are restricted to using regression equations or trial and error methods. Further, no program can practically estimate layer thicknesses from measured deflection data.

There are no truly non-linear backcalculation programs, and most programs require deflection data from at least three load levels to infer non-linear response. Finally, the backcalculated results are affected by a variety of complex factors making backcalculation of layer moduli a difficult undertaking.

In light of the shortcomings of the existing backcalculation programs, some of the objectives that will be persued in the course of this research to offer enhanced capabilities in a new computer program include:

1. The ability to mechanistically estimate stiff layer depth from FWD deflection data.
2. The capability to predict the thickness of any layer other than the roadbed soil.
3. Eliminate or reduce the sensitivity of the backcalculated results to the seed values.
4. Enhance the user interaction in a manner not only to facilitate and simplify the use of the program but also to enhance the understanding of the backcalculation process and results.
5. Finally, since the project is sponsored by MDOT, verify the applicability of the existing temperature conversion factors to the pavements in the state of Michigan.

CHAPTER 3

EFFICIENT ITERATIVE METHODS FOR BACKCALCULATION OF PAVEMENT LAYER PROPERTIES

3.1 GENERAL

Elastic layer analysis of pavements is used to calculate the load induced strains and stresses in different regions of a pavement system whose properties are already known. Backcalculation, on the other hand, is the inverse problem related to elastic layer analysis in which some of the unknown layer properties are estimated from the measured pavement response (deflections) due to a known load. The two problems are depicted in Figure 2.7. The deflections are typically measured at various lateral distances away from the load and the number of deflection measurements must be greater than or at least equal to the number of parameters to be backcalculated.

For three or more layers, the inverse problem cannot be solved exactly and a numerical solution scheme must be used. A review of backcalculation methods that use elastic layer analysis has been presented in section 2.9, and most of them are based on the minimization of an objective function formulated in terms of the error between the measured and the calculated surface deflections.

In this study, a new and efficient method to substantially improve the predicted value of the roadbed modulus with a single call to a forward calculation program is developed. Along with this a backcalculation procedure based on the Newton method for the solution of non-linear equations is developed, and this method is known for its fast convergence (Dennis, et al., 1983). In addition the Newton method is extended to estimate layer thicknesses and the depth to the stiff layer along with the layer moduli. Finally a logarithmic transformation of the data to improve the speed of convergence is implemented.

3.2 IMPROVED ESTIMATION OF ROADBED MODULUS

It is well known that the roadbed soil often contributes up to 90% of the pavement peak surface deflection (Yoder, et al., 1978). Deflections measured by all sensors are affected substantially by the roadbed modulus with the farthest sensor from the applied load being affected the most. Hence, accurate estimation of the roadbed modulus is essential for the over all accuracy of the backcalculation results.

For an iterative backcalculation program, an early accurate estimate of the roadbed modulus will not only reduce the analysis time but will also reduce the possibility of divergence for complex problems. The algorithm developed below has been incorporated in the MICHBACK program in order to substantially improve the roadbed modulus at the beginning of the backcalculation. Further improvement to the roadbed modulus is carried out simultaneously along with the backcalculation of the other layer properties.

Recognizing that the roadbed soil contributes strongly to the deflection measured by all sensors, a technique is developed to improve the roadbed modulus using a single call to a mechanistic analysis program. This estimation at the beginning of the backcalculation procedure facilitates convergence.

Consider a pavement with n layers for which m surface deflections are measured ($m \geq n$). Let the vector $\{ \hat{w}_j \}$ contain the m deflections computed at the top of the jth layer using current estimates of the layer moduli ($\hat{E}_j$). The vertical compression under the sensors in the jth layer is $\{ \hat{w}_j \} - \{ \hat{w}_{j+1} \}$. For the last layer, one can take $\{ \hat{w}_{n+1} \} = \{ 0 \}$. The vertical compression in any layer is a result of the accumulated vertical strain ($\hat{e}_j$), which is inversely proportional to the layer modulus (i.e., proportional to $\hat{e}_j = 1/\hat{E}_j$). By scaling the compression in each layer by the layer modulus, one can obtain the following vector:

$$\{a_j\} = \hat{E}_j(\ \{\hat{w}_j\} - \{\hat{w}_{j+1}\}\ ) \tag{3.1}$$

The collection of all such vectors will be an $n \times m$ matrix of the form:

$$\mathbf{A} = [\ \{a_1\}\ \{a_2\} \ldots \{a_m\}\ ] \tag{3.2}$$

The sum of the compressions in each layer must add to the total surface deflection. that is:

$$\sum_{j=1}^{n} (\ \{\hat{w}_j\} - \{\hat{w}_{j+1}\}\ ) = \{\hat{w}_1\} \tag{3.3}$$

or equivalently

$$\mathbf{A}\ \{\hat{e}\} = \{\hat{w}_1\} \tag{3.4}$$

Equation 3.4 can be used as the basis to the following iterative scheme

$$\mathbf{A}^i\ \{\hat{e}\}^{i+1} = \{\mathbf{w}\} \tag{3.5}$$

in which $[A]^i$ is computed using the current moduli estimates $\{\hat{E}\}^i$, and $\{\mathbf{w}\}$ are the measured surface deflections. The over-determined system of equations (n equations and m unknowns) can then be solved using the method of least squares to obtain the revised inverse moduli $\{\hat{e}\}^{i+1}$.

It may appear that Equation 3.5 can be used to improve the estimates of all layer moduli. Unfortunately, the technique is very unstable for estimating all the unknown moduli. The method was found to be very sensitive to the ratios of the seed moduli (e.g., ratio of each layer moduli to the roadbed modulus). When the ratios of the seed moduli are close to the actual ones (even though the initial moduli estimates may be substantially different from the actual ones) the results for all the unknown moduli are predicted very well even after a single iteration. However, the sensitivity of the method to the ratios of the seed moduli often results in a negative modulus value of one of the layers other than the roadbed. Therefore, the method has been adopted to improve only the roadbed modulus. The values obtained for the other layer moduli are disregarded.

3.3 NEWTON'S METHOD

Consider the example of finding the roots of a nonlinear equation in one unknown

$$f(x) = x^2 - 3 \tag{3.6}$$

Suppose an initial estimate of the answer is $x_o=2$, a better estimate $\hat{x}_1$ can be obtained by drawing the line that is tangent to *f(x)* at $(2,f(2))=(2,1)$ and finding the point $\hat{x}_1$ where the line crosses the x axis as shown in Figure 3.1.

$$\hat{x}_1 = \hat{x}_o - \Delta x \tag{3.7}$$

It can be readily verified that

$$\hat{x}_1 = \hat{x}_o - \frac{f(\hat{x}_o)}{f'(\hat{x}_o)} \tag{3.8}$$

for this particular case

$x_1 = 2 - 1/4 = 1.75$

If two more additional iterations are performed in a similar manner, the result will be 1.7320508 which is accurate to the eight significant digits. This method is called the *Newton-Raphson method*, or simply *Newton's method*. The use of Newton's method for backcalculating layer moduli in flexible pavements was first suggested by Thomas Hou (1977), but the method has not been pursued since until recently (Wang, et al., 1993; Van Cauwelaert, et at., 1989).

Newton's method is a useful tool for solving nonlinear problems. It is an

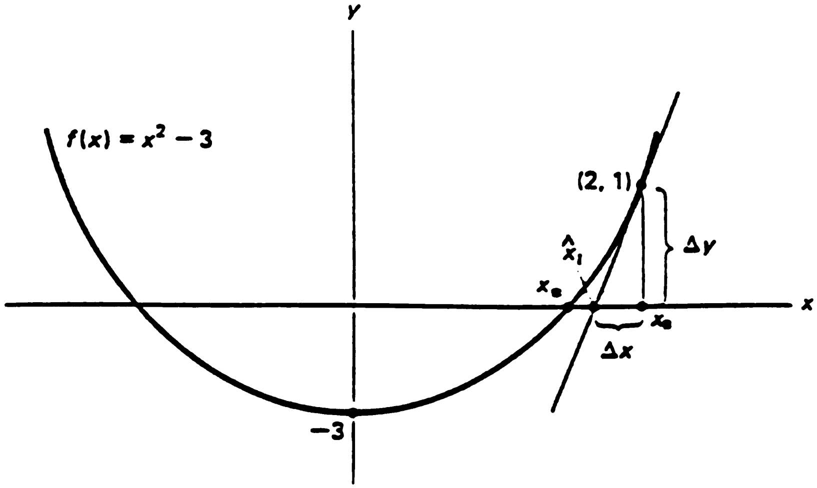

Figure 3.1. General illustration of Newton's method (Dennis, 1983).

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iterative method which generates a sequence of points that rapidly approach the true solution. The convergence of the method depends upon the nature of the problem itself. Given a good initial estimate, the method is known to converge quadratically for most problems but for some ill-behaved functions with poor initial estimates, it may not converge at all. Also local convergence for certain types of problems cannot be ruled out. The characteristics of Newton's method when used to backcalculate the layer moduli are highlighted in Chapter 4.

3.4 NEWTON'S METHOD WHEN THE DERIVATIVES OF THE FUNCTION ARE NOT AVAILABLE

In many practical problems, the closed-form of the function $f(x)$ is not known explicitly and is obtained as an output from some numerical or experimental procedure, as is the case in FWD testing. In such cases the derivatives are also not available in a closed-form and Newton's method must be modified to make use of the values of $f(x)$ only.

In the classic Newton's method the values of $f'(x)$ are used to model $f(x)$ near the current solution estimate x_o by the line tangent to $f(x)$ at x_o as shown in Figure 3.1. When the derivative of the function is not available, the model can be replaced by the secant line that passes through f at x_o and some nearby point $x_o + h_o$ as depicted in Figure 3.2. The slope of this line is given by the following equation:

$$a_o = \frac{f(x_o+h_o) - f(x_o)}{h_o} \tag{3.9}$$

Replacing $f'(x_o)$ by a_o in Equation 3.8 yields.

$$\hat{x}_1 = \hat{x}_o - \frac{f(x_o)}{a_o} \tag{3.10}$$

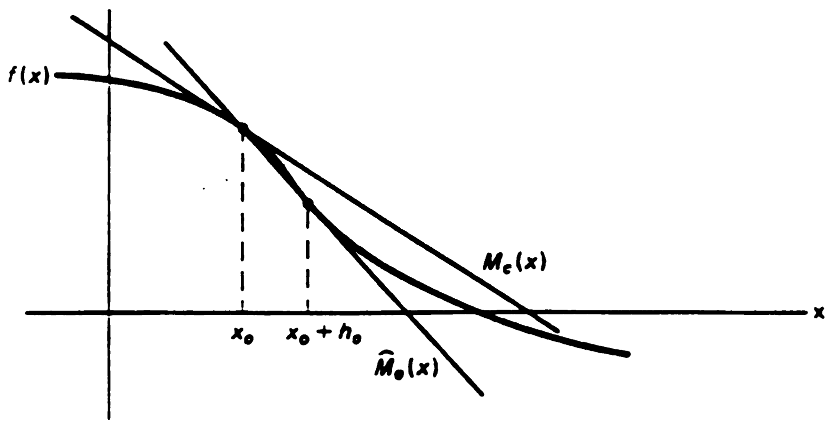

Figure 3.2. Secant approximation of Newton's method (Dennis, 1983).

It is apparent that in the limit as $h_o \rightarrow 0$, a_o will converge to $f'(x_o)$. When h_o is chosen to be small, a_o is called a finite-difference approximation to $f'(x_o)$. This modified quasi-Newton's method is known to work as well as the Newton's method and sometimes is referred to as the *Secant Method*. The characteristics of the quasi-Newton method is similar to the standard Newton's method, but convergence is usually slow.

3.5 MULTI-DIMENSIONAL FORM OF NEWTON'S METHOD

Newton's method can be easily expanded to accommodate the solution of a set of nonlinear equations to solve for more than one unknown. Consider the set of equations

$$\begin{aligned} f_1(x_1, x_2, \ldots, x_n) &= 0 \\ f_2(x_1, x_2, \ldots, x_n) &= 0 \\ \vdots \qquad & \quad \vdots \\ f_n(x_1, x_2, \ldots, x_n) &= 0 \end{aligned} \qquad (3.11)$$

or in the vector form

$$\mathbf{f}(\mathbf{x}) = 0 \qquad (3.12)$$

Given an estimate

$$\hat{\mathbf{x}}_i = \left[\hat{x}_1^i, \hat{x}_2^i, \ldots, \hat{x}_n^i\right]^T \qquad (3.13)$$

an improved estimate is obtained by

$$\begin{aligned} \hat{\mathbf{x}}_{i+1} &= \hat{\mathbf{x}}_i - \Delta\mathbf{x} \\ &= \hat{\mathbf{x}}_i - G^{-1}\,\mathbf{f}(\hat{\mathbf{x}}^i) \end{aligned} \qquad (3.14)$$

where

In generál, rather then inverting G, it is more efficient to solve the set of linear

$$G^{i} = \left[\frac{\partial \boldsymbol{f}}{\partial \boldsymbol{x}}\right]\Bigg|_{\{\boldsymbol{x}\} = \{\hat{\boldsymbol{x}}_i\}} = \left.\begin{bmatrix} \dfrac{\partial f_1}{\partial x_1} & \dfrac{\partial f_1}{\partial x_2} & \cdots & \dfrac{\partial f_1}{\partial x_n} \\ \vdots & & & \vdots \\ \dfrac{\partial f_n}{\partial x_1} & \dfrac{\partial f_2}{\partial x_2} & \cdots & \dfrac{\partial f_n}{\partial x_n} \end{bmatrix}\right|_{\{\boldsymbol{x}\} = \{\hat{\boldsymbol{x}}_i\}} \qquad (3.15)$$

equations as follows:

$$G\,\Delta \boldsymbol{x} = \boldsymbol{f}(\hat{\boldsymbol{x}}_i) \qquad (3.16)$$

3.6 USE OF THE NEWTON'S METHOD TO BACKCALCULATE PAVEMENT LAYER MODULI

Consider a pavement with n layers for which m surface deflections are measured ($m \geq n$). Let the vector w represents the measured surface deflections due to applied FWD load. The non-linear deflection versus modulus curve is approximated by a straight line which is tangent to it at the estimate $\hat{E}^i$. The slope of the straight line, $(dw\,/\,dE)|_{E = \hat{E}^i}$, is used to obtain the increment, ΔE^i, which is added to $\hat{E}^i$ to obtain the improved modulus estimate $\hat{E}^{i+1}$ as shown in Figure 3.3.

Since the slope is not known analytically, it is obtained numerically, as discussed in section 3.4, by using the following equation:

$$\frac{dw}{dE}\Big|_{E = \hat{E}^i} \approx \frac{w((1+r)\ \hat{E}^i) - w(\hat{E}^i)}{r\hat{E}^i} \qquad (3.17)$$

in which r is sufficiently small. This requires additional deflections arising from moduli values of $(1+r)\hat{E}^i$ to be computed.

For the described system of n layers and m sensors, the slope is represented by the following gradient matrix:

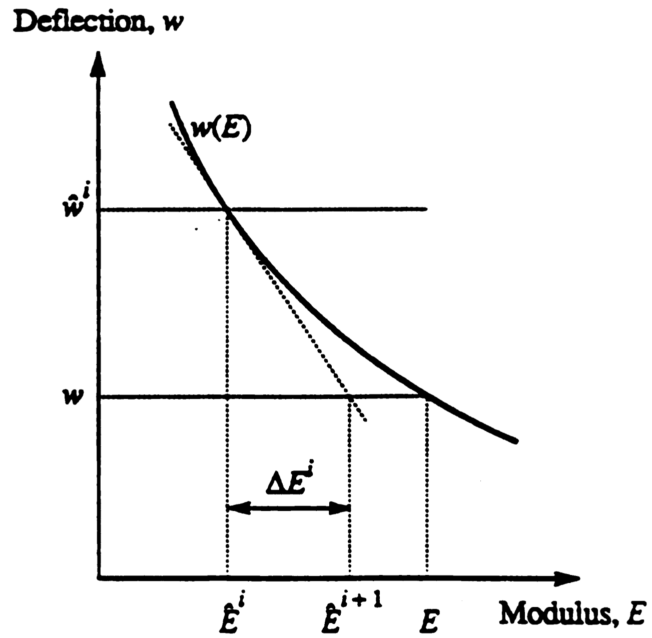

Figure 3.3. Graphical illustration of Newton's method iteration to find pavement layer moduli.

$$G^i = \left[\frac{\partial\{w\}}{\partial\{E\}}\right]\Bigg|_{\{E\}=\{E\}^i} = \left.\begin{bmatrix} \frac{\partial w_1}{\partial E_1} & \frac{\partial w_1}{\partial E_2} & \cdots & \frac{\partial w_1}{\partial E_n} \\ \vdots & & & \vdots \\ \frac{\partial w_m}{\partial E_1} & \frac{\partial w_2}{\partial E_2} & \cdots & \frac{\partial w_m}{\partial E_n} \end{bmatrix}\right|_{\{E\}=\{E\}^i} \tag{3.18}$$

The element of the jth row and kth column of the matrix can be estimated numerically as follows:

$$\frac{\partial w_j}{\partial E_k}\Big|_{E=E^i} \approx \frac{w_j([R]\{\hat{E}^i\}) - w_j(\{\hat{E}^i\})}{r\hat{E}_k^i} \tag{3.19}$$

in which $[R]$ is a diagonal matrix with kth diagonal element being $(1+r)$ and all other elements being 1. Thus the partial derivative is estimated numerically by taking the difference in the jth deflection arising from the use of a set of moduli $\hat{E}^i_1, \hat{E}^i_2, \ldots, (1+r)\hat{E}^i_k, \ldots, \hat{E}^i_n$ and the moduli $\hat{E}^i_1, \hat{E}^i_2, \ldots, \hat{E}^i_k, \ldots, \hat{E}^i_n$. Hence, a separate call to the mechanistic analysis routine is required to compute the partial derivatives in each column of the gradient matrix. The increments to the moduli, $\{\Delta E\}^i$, can then be obtained by solving the m equations in n unknowns:

$$\hat{w}^i + G^i\,\Delta E^i = w \tag{3.20}$$

The method of least-square may be used at this stage to solve the over-determined system of equations (m equations in n unknowns) to determine ΔE^i. If desired, weights can be used for each sensor measurement to emphasize some measurements over others. The revised moduli are obtained through

$$\{E\}^{i+1} = \{E\}^i + \{\Delta E\}^i \tag{3.21}$$

The iteration is terminated when the changes in layer moduli are smaller than a tolerance ϵ_1 specified by the analyst. That is,

$$\frac{\hat{E}_k^{i+1} - \hat{E}_k^{i}}{\hat{E}_k^{i}} \leq \epsilon_1 , \qquad K = 1,2,3\ldots n \tag{3.22}$$

When the computed and measured deflection basin match closely, the following root-mean-square (RMS) error criterion can also be used to terminate the iteration:

$$RMS = \sqrt{\frac{1}{m} \sum_{j=i}^{m} \left(\frac{\hat{w}_j^{i} - w_j}{w_j}\right)^2} \leq \epsilon_2 \tag{3.23}$$

The iteration will end if either of the two criteria is met at any stage of the process. It should be noted that the RMS error criterion will usually be met only if the model used for the forward calculation accurately represents the pavement system that produced the measured surface deflections. Equation 3.22, on the other hand will always be satisfied if the numerical algorithm converges.

3.7 LAYER THICKNESS ESTIMATION

The layer thicknesses at the locations of the FWD tests are seldom known exactly. One direct method of measuring the layer thicknesses is coring. But selective or random coring provides only a better estimate of the average layer thicknesses in the section of interest, since it will never be possible to core all the FWD test locations. Variation in construction and terrain make the variation in layer thicknesses inevitable. Estimation of layer thicknesses along with the layer moduli from the deflection data can enhance the accuracy of the backcalculated results.

Newton's method can be extended to backcalculate pavement layer thicknesses along with layer moduli. The restriction remains that the total number of unknown moduli and layer thicknesses together must not exceed the number of deflection measurements.

For improving l layer thicknesses in addition to n layer moduli Equation 3.8 is extended as follows:

$$\boldsymbol{w}^{i}+G^{i}\begin{Bmatrix}\Delta \boldsymbol{E}^{i}\\ \Delta \boldsymbol{t}^{i}\end{Bmatrix} = \boldsymbol{w} \tag{3.24}$$

in which Δt^i is the vector of thickness increments and the augmented gradient matrix G^i is given by

$$G^{i} = \left[\frac{\partial\{w\}}{\partial\{E\}}\ \frac{\partial\{w\}}{\partial\{t\}}\right]\Bigg|_{\substack{\{E\}=\{E\}^i\\ \{t\}=\{t\}^i}} = \left.\begin{bmatrix} \frac{\partial w_1}{\partial E_1} & \cdots & \frac{\partial w_1}{\partial E_n} & \frac{\partial w_1}{\partial t_1} & \cdots & \frac{\partial w_1}{\partial t_l} \\ \vdots & & \vdots & \vdots & & \vdots \\ \frac{\partial w_m}{\partial E_1} & \cdots & \frac{\partial w_m}{\partial E_n} & \frac{\partial w_m}{\partial t_1} & \cdots & \frac{\partial w_m}{\partial t_l} \end{bmatrix}\right|_{\substack{\{E\}=\{E\}^i\\ \{t\}=\{t\}^i}} \tag{3.25}$$

A column of the gradient matrix corresponding to a partial derivative with respect to a thickness is estimated numerically by computing the surface deflections due to a slight increase in that thickness. The number of forward calculations during each iteration now increases to $(n + l + 1)$.

During extensive testing of the method to backcalculate layer thicknesses and moduli it was observed that a better overall convergence is achieved if only the layer moduli are improved in the first few iterations. Additional iterations are then performed to improve both the layer moduli and thicknesses as outlined in this section.

Theoretically it should be possible to backcalculate the thicknesses of any number of layers simultaneously, provided the total number of unknowns do not exceed the number of deflection measurements. In practice however, backcalculating layer thicknesses for more than one layer often causes the scheme to diverge. Further investigations indicated that if the deflections calculated by the forward analysis program are used in the backcalculation without any truncation, then up to two layer thicknesses can be backcalculated simultaneously. However, if the deflections are truncated to imitate field measurements, then the scheme often diverges if more than one layer thickness is backcalculated.

Theoretically speaking, the technique outlined above can be extended to predict any layer property, including Poisson's ratio, as long as the number of unknown quantities does not exceed the number of sensors. However, convergence of the method must be studied rigorously to validate such a claim.

3.8 THE MODIFIED NEWTON METHOD

In the Newton method, the number of calls made to a mechanistic analysis program for each iteration is $(n + l + 1)$. In the modified Newton method the total number of calls to a forward calculation program can often be reduced. In the modified method several iterations are performed with a gradient matrix before it is revised. Although, the convergence in the modified approach is slower than the normal method, the n forward calculations required to calculate the gradient matrix during each iteration can be reduced. The total number of iterations required to reach a desired level of accuracy will become higher in the modified method but the total number of calls made to the forward analysis program may be reduced. Experience has shown that performing n iterations before revising the gradient matrix yields better results with fewer calls to the mechanistic analysis program.

3.9 STIFF LAYER EFFECTS AND ESTIMATION OF DEPTH TO STIFF LAYER

In elastic layer modeling of pavements the roadbed materials are assumed to be uniformly stiff and infinitely thick. However, in most real pavements, the roadbed stiffness increases with depth. This increase is mainly due to the increasing lateral stress with depth and is influenced by changes in the material or even the presence of a stiff layer (e.g., bedrock) within the zone influencing the FWD measurements.

Many researchers have acknowledged the need to incorporate a stiff layer at an appropriate depth since this can significantly affect the backcalculated results(Bush,

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1980; Chou, 1989). Various methods of estimating the depth to the stiff layer have been suggested.

In a layered elastic analysis the rigid layer can be incorporated by assigning a very high modulus to the lowest layer, but the depth to this layer may be unknown. Bush (1980) recommended the use of a rigid layer at 20 feet depth for all FWD analysis. Uddin, et al. (1986) suggested that the roadbed depth can be inferred from the velocity of compression waves in the roadbed and frequency of loading. Chou (1989) suggested an iterative approach based on a trial and error method. The depth which results in the least RMS error can be found which will presumably be the true depth. The method apart from being tedious is prone to error owing to the non-uniqueness of the solution.

Recently, Brown (1990) presented a set of regression equations to estimate the depth to stiff layer using the measured deflections and layer thicknesses. These equations have been incorporated in the MODULUS and EVERCALC programs to estimate the depth to the stiff layer (Rohde and Scullion, 1990; Mahoney, et al., 1993). The regression equations yield relatively inaccurate depth estimates for medium to deep stiff layer locations. EVERCALC allows the user to input the modulus of the stiff layer, while the MODULUS program assigns the modulus of the stiff layer automatically. The accuracy of the regression equations has been tested in Chapter 4.

The estimation of stiff layer depth based solely on regression equations can give rise to significant errors in the backcalculated moduli. In MICHBACK program the initial estimate of the stiff layer depth is obtained using regression model developed by Baladi (1994). Unlike the other two programs, however, an iterative process has been developed and is implemented in MICHBACK to improve the stiff layer depth, or equivalently the roadbed thickness. The iterative process is carried out in two distinct steps as follows:

<u>Step 1</u>. In this step only the values of the layer moduli are improved until they show some stability.

<u>Step 2</u>. In this step two distinct schemes are used to find the roadbed thickness. The first scheme is identical to the one explained in section 3.7.1. The roadbed thickness is included as an unknown in the gradient matrix along with the layer moduli (Equation 3.25). As reported earlier, this method can estimate the unknown thickness of any layer other than the roadbed along with the layer moduli. However, this method alone was insufficient in predicting the roadbed thickness accurately. When this method is used, the moduli are estimated much better and faster, compared with the method in which only the moduli are kept as unknowns. This method also works well in providing fine corrections to the roadbed thickness when some other method has been used to bring the estimate reasonably close to the actual one. The shortcoming of this method is overcome by adding another scheme to this step, involving iteration to derive the layer thickness of the roadbed solely based upon the deflections. The gradient matrix is constructed for one variable alone and then a least squares solution is sought. This method makes the estimation of the roadbed thickness fast and accurate. These two schemes are used iteratively until one of the convergence criterion specified by the user is met. It is ensured that the scheme where the layer moduli and roadbed thickness are both treated as unknowns, always occurs last. This guarantees the revision of layer moduli after an accurate estimate of the stiff layer thickness has finally been obtained using the scheme which only estimates the stiff layer depth.

3.10 LOGARITHMIC TRANSFORMATION

It has been reported (Bush, 1980) that the relationship between surface deflections and layer moduli display less non-linearity if the deflections and layer moduli are transformed to a logarithmic scale. This section explores this transformation.

3.10.1 Relationship between Surface Deflection and Layer Moduli

The relationship between surface deflections and layer moduli was probed extensively. The layer moduli of an arbitrary three-layer flexible pavement section of medium AC thickness were varied, one at a time, and their effects on the calculated surface deflections were analyzed. Figures 3.4 through 3.9 depict the pavement surface deflections plotted against the layer moduli using both arithmetic and logarithmic scales. To avoid clutter in the plots, only the deflections at three locations are shown. It can be seen that the relationship between the layer moduli and the surface deflections are non-linear for both the AC layer and the roadbed soil and nearly linear for the base layer. The logarithmic transformations substantially decrease these non-linearities.

Arithmetic and logarithmic plots of the pavement surface deflection as a function of the moduli of the AC and roadbed soil are shown in Figure 3.10 and 3.11 respectively. It can be seen that the use of logarithmic scale transformed an otherwise curved surface to a much flatter one. The analysis of such a flat surface results in computational savings that are highlighted in Chapter 4.

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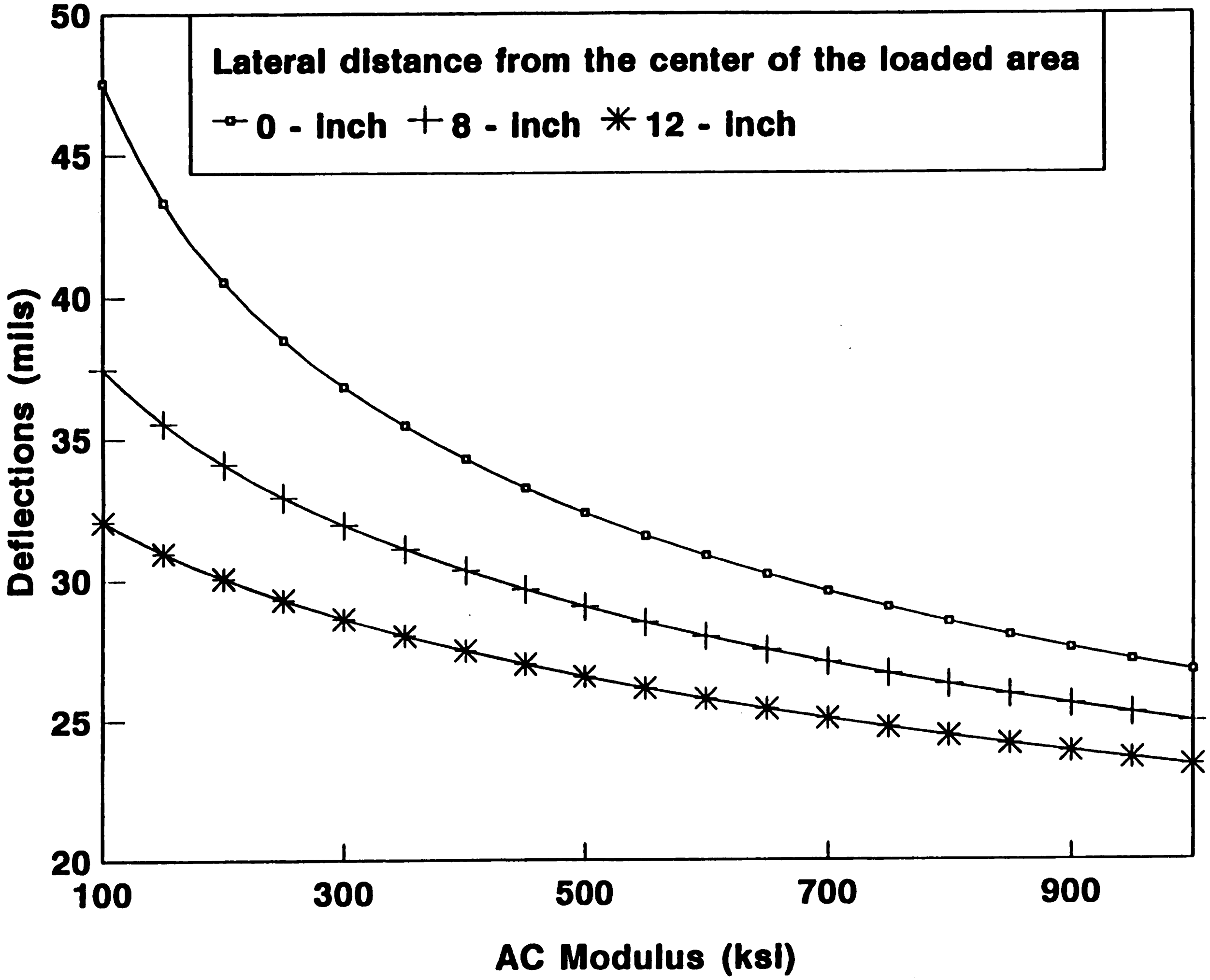

Figure 3.4. Effect of AC layer modulus on pavement surface deflections.

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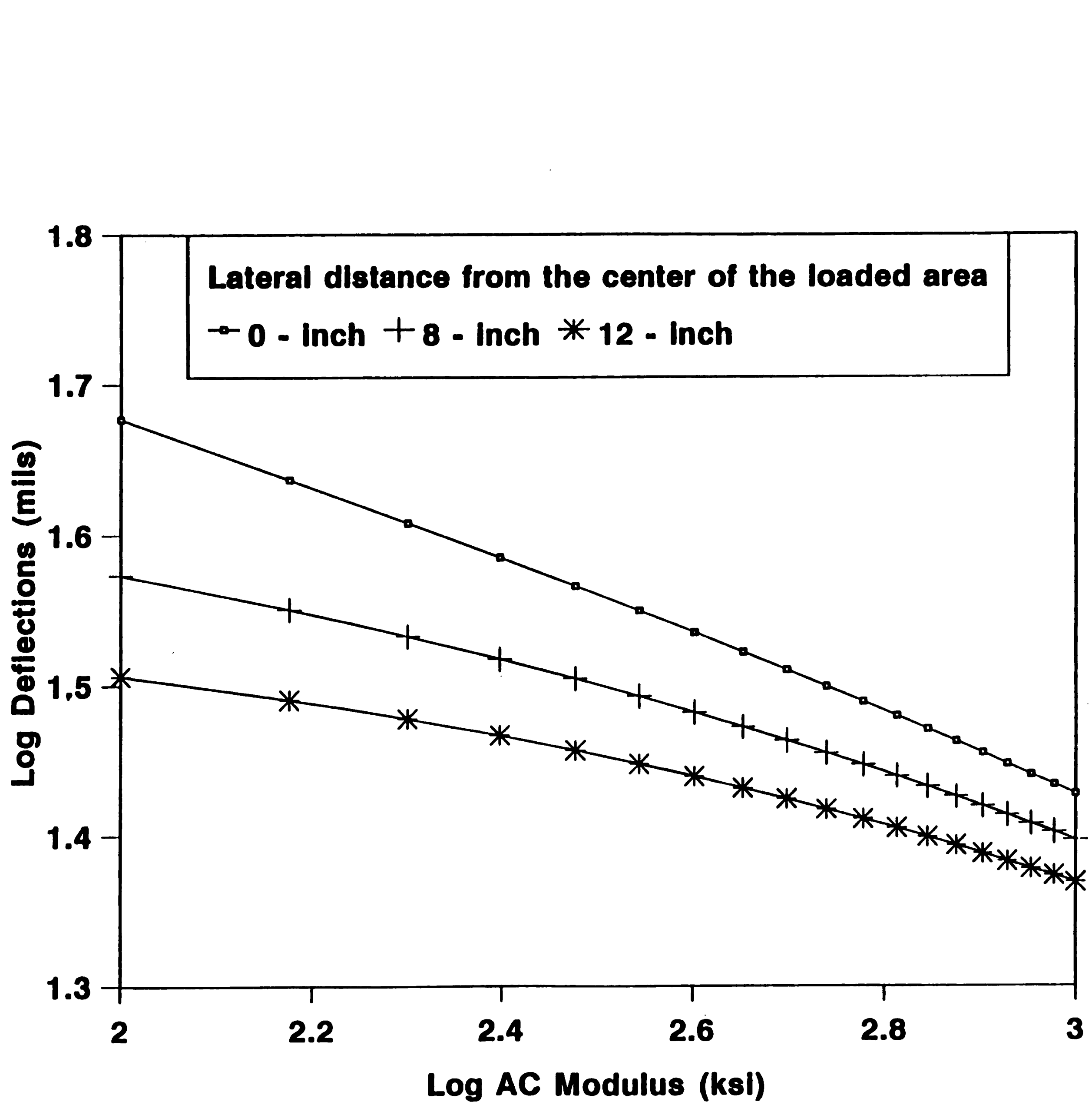

Figure 3.5. Effect of AC layer modulus on pavement surface deflections (logarithmic scale).

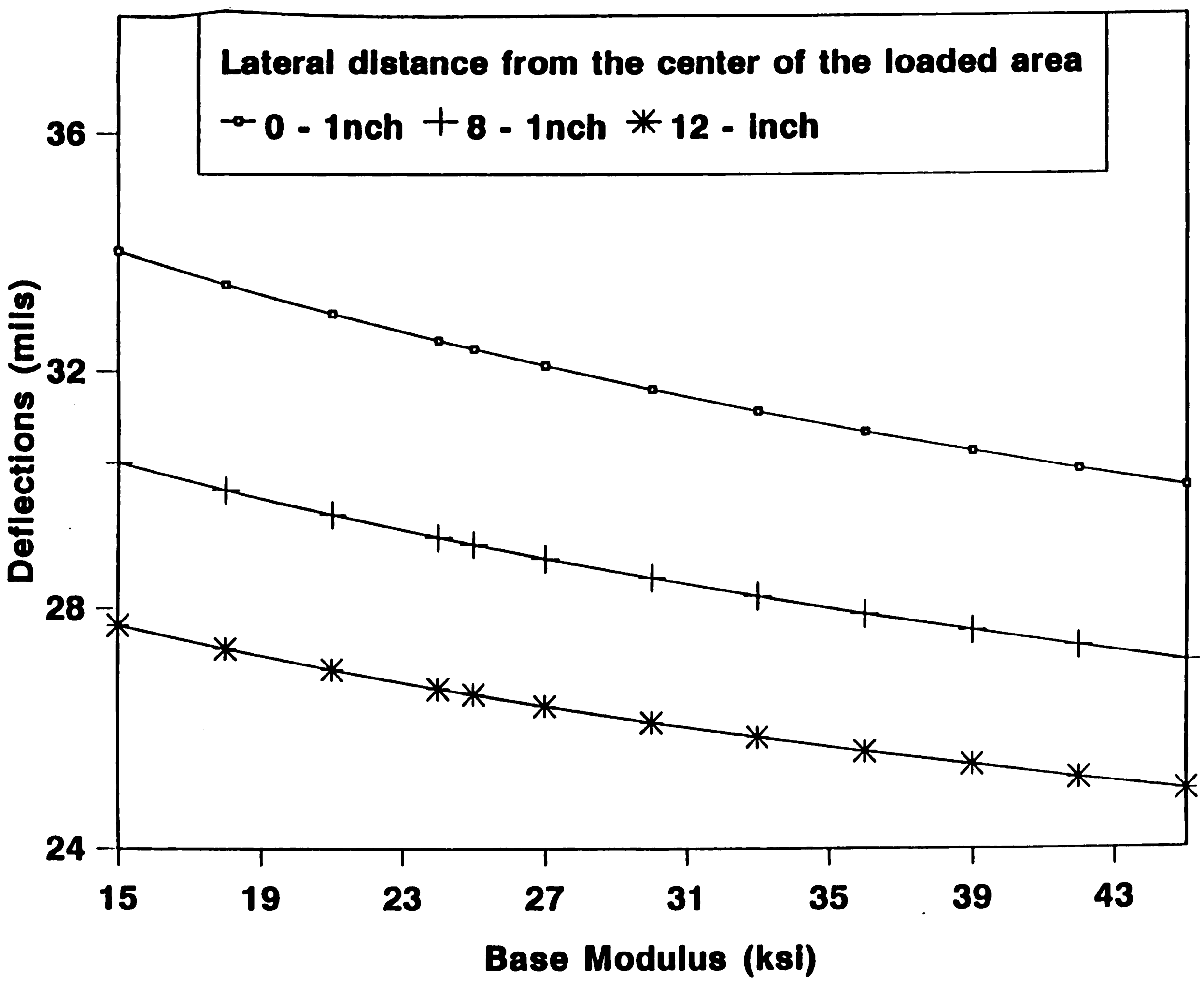

Figure 3.6. Effect of base layer modulus on pavement surface deflections.

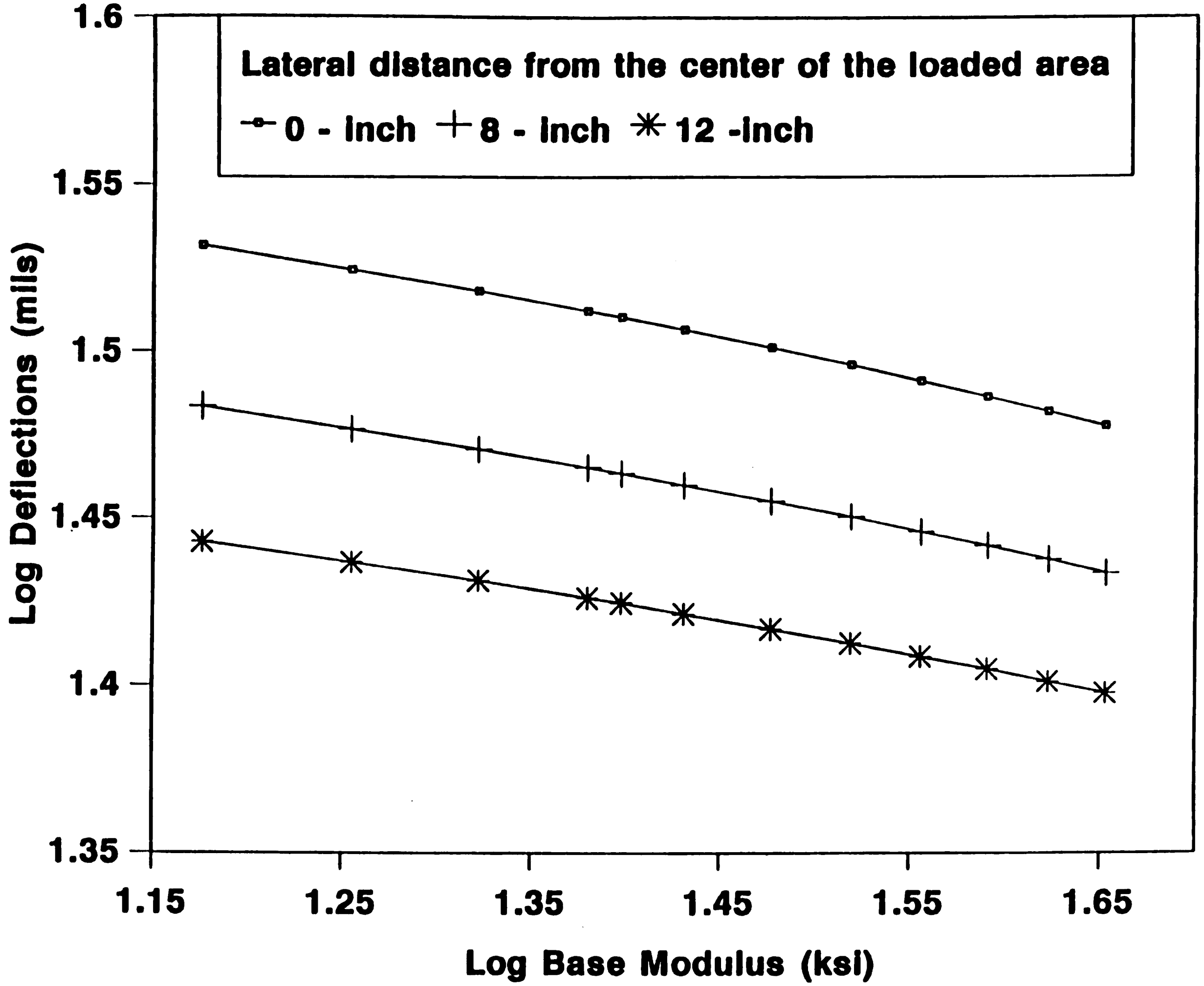

Figure 3.7. Effect of base layer modulus on pavement surface deflections (logarithmic scale).

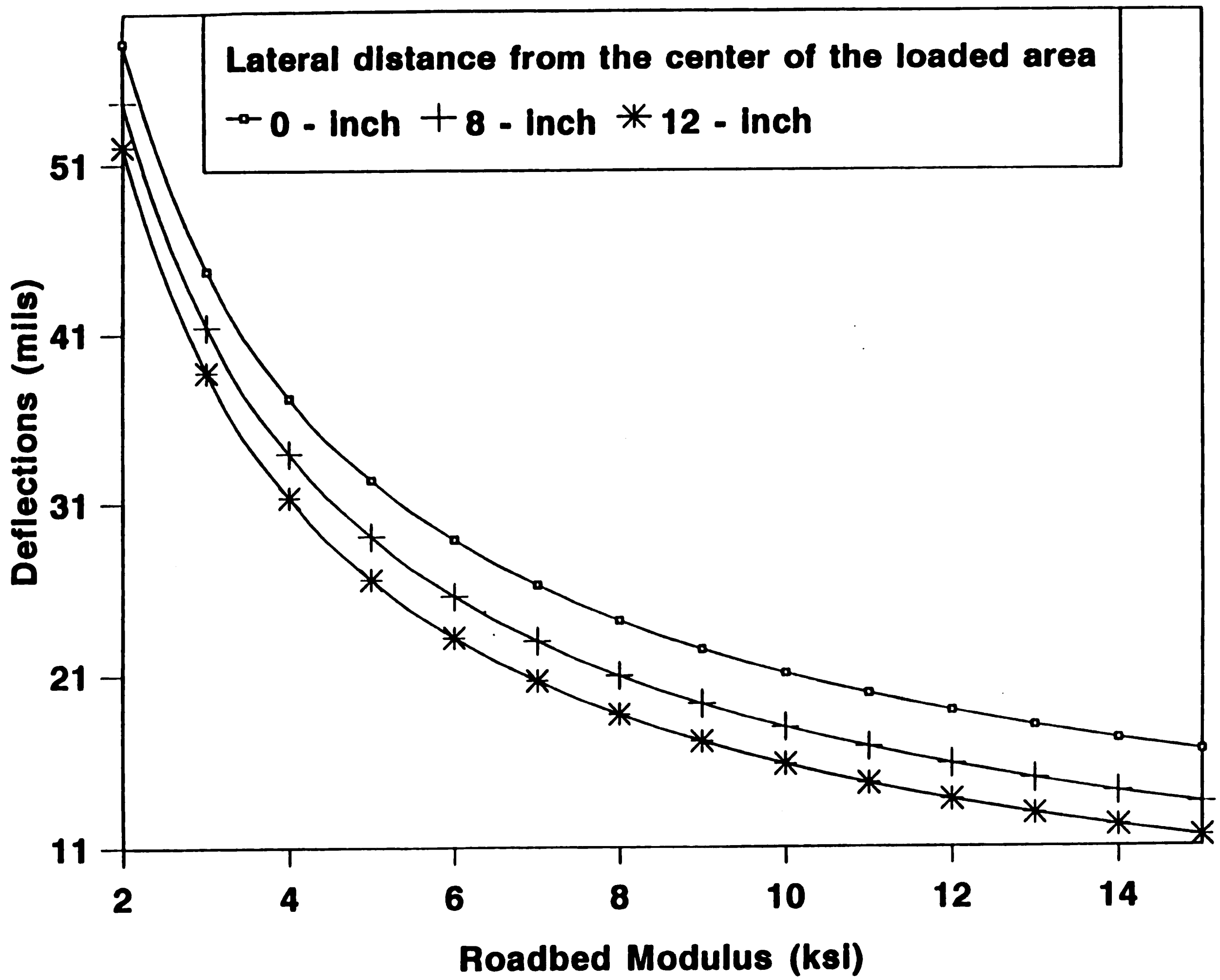

Figure 3.8. Effect of roadbed layer modulus on pavement surface deflections.

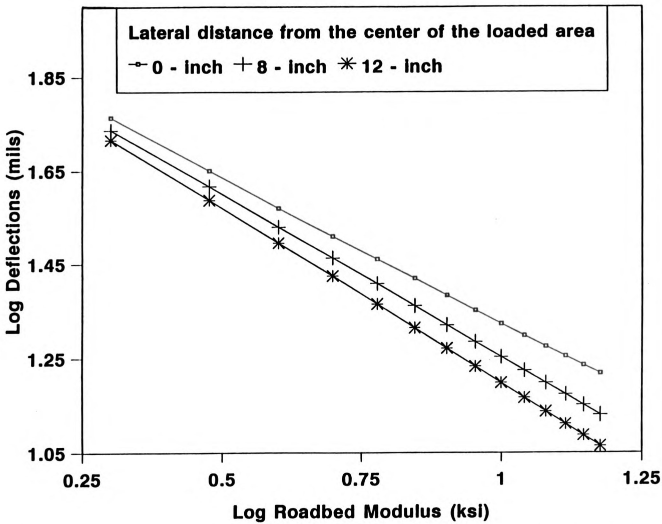

Figure 3.9. Effect of roadbed layer modulus on pavement surface deflections (logarithmic scale).

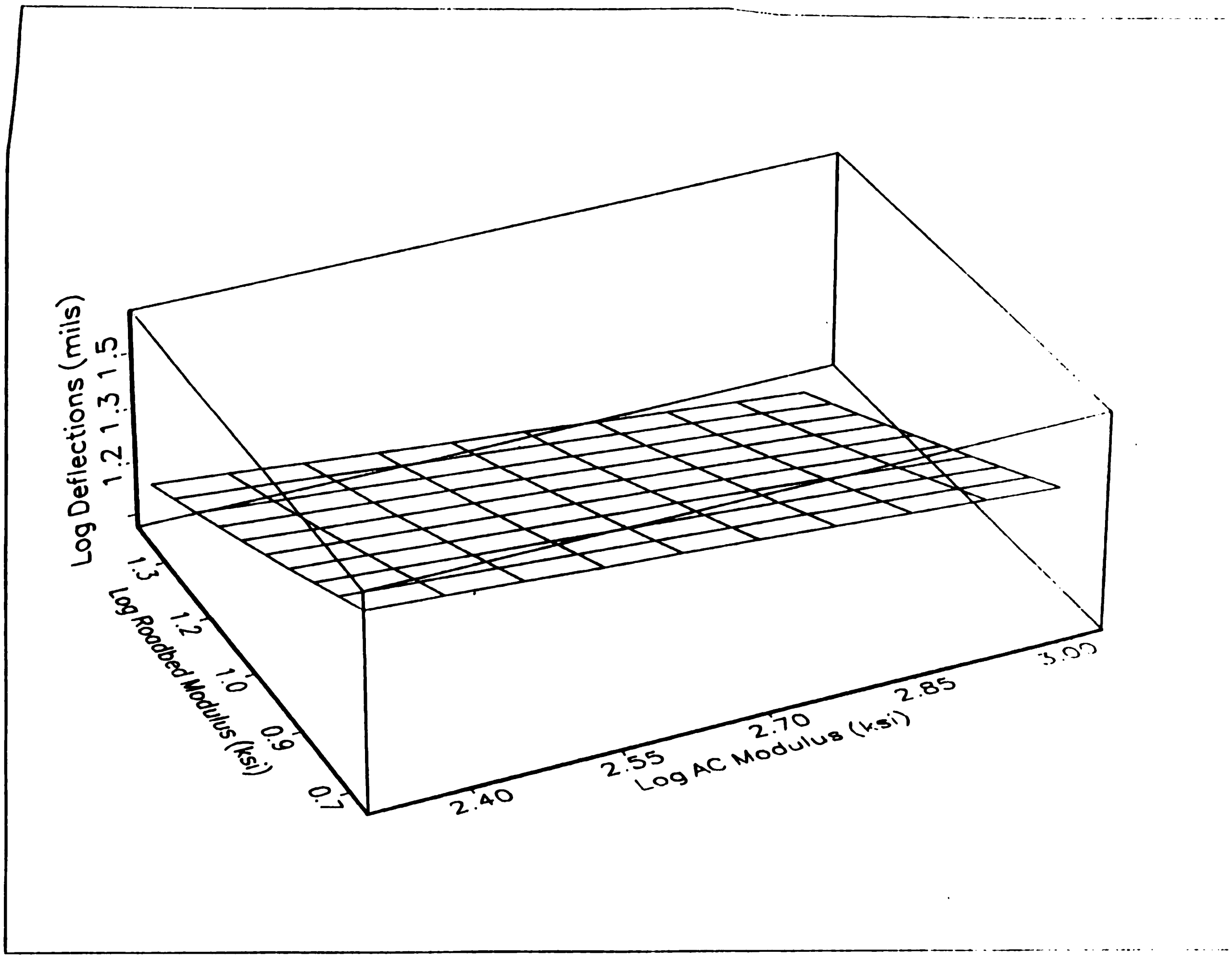

Figure 3.11. Graphic illustration of surface deflection variation with AC and roadbed modulus (logarithmic scale).

3.10.2 Implementation of Logarithmic Transformation

In order to shorten the computational time, logarithmic transformation was implemented to calculate the gradient matrix and the moduli increments in the logarithmic space by using the following equation:

$$\left.\frac{d(\log w)}{d(\log E)}\right|_{E=\hat{E}^i} \approx \frac{\log(w(10^{(1+r)*\log \hat{E}^i})) - \log(w(\hat{E}^i))}{r \log(\hat{E}^i)} \tag{3.26}$$

It this case the additional deflections arising from a modulus of $10^{((1+r)*\log(E))}$ have to be computed, where r is a sufficiently small number.

The set up of the gradient matrix G^i, representing the slope for a system of n layers and m sensors is essentially the same as in Equation 3.25. The difference in the formulation of this matrix in the logarithmic space is that the element in the jth row and kth column of the matrix is estimated numerically for columns related to the moduli as follows:

$$\left.\frac{\partial w_j}{\partial E_k}\right|_{E=\hat{E}^i} \approx \frac{\log\left(w_j\left(10^{\left([R]\left\{\begin{matrix}\log \hat{E}^i \\ \hat{t}^i\end{matrix}\right\}\right)}\right)\right) - \log\left(w_j\left\{\begin{matrix}\hat{E}^i \\ \hat{t}^i\end{matrix}\right\}\right)}{r \log(\hat{E}_k^i)} \tag{3.27}$$

For columns related to layer thicknesses, since the thicknesses are not transformed to the logarithmic's scale, the following equation is used:

$$\left.\frac{\partial w_j}{\partial t_k}\right|_{t=\hat{t}^k} \approx \frac{\log\left(w_j\left(10^{\left([R]\left\{\begin{matrix}\log \hat{E}^i \\ \hat{t}^i\end{matrix}\right\}\right)}\right)\right) - \log\left(w_j\left\{\begin{matrix}\hat{E}^i \\ \hat{t}^i\end{matrix}\right\}\right)}{r\, \hat{t}_k^i} \tag{3.28}$$

For these cases, the following equation is set up for optimum least square solution

$$\log(\hat{\boldsymbol{w}}^i) + G^i \left\{\begin{matrix}\log(\Delta E^i) \\ \Delta t^i\end{matrix}\right\} = \log(\boldsymbol{w}) \tag{3.29}$$

Finally, the revised moduli and thickness are calculated using the following equations:

$$\begin{aligned} \{E\}^{i+1} &= \{E\}^{i} * \{10^{(\Delta E^{i})}\} \\ t^{i+1} &= t^{i} + \Delta t^{i} \end{aligned} \tag{3.30}$$

After this step is performed the deflections and the revised moduli are both converted back to the arithmetic scale. Then the usual convergence checks as given by Equations 3.22 and 3.23 are performed to determine whether the iteration has converged.

For very shallow stiff layer depths (e.g., 36 in. or less), negative deflections may be measured by the outer sensors. Almost all existing backcalculation programs, do not accept negative deflections at any of the sensors. For such cases, a solution that ignores the deflections of the outer sensors is used. The MICHBACK program can analyze deflection basins containing negative measured deflections. However, since negative deflections cannot be transformed to the logarithmic scale the arithmetic scale is automatically used by the program.

CHAPTER 4

VERIFICATION OF THE MICHBACK ALGORITHM USING THEORETICAL DEFLECTION BASINS

4.1 GENERAL

In the previous chapter, a mechanistic based gradient method to backcalculate the layer moduli of a pavement structure using FWD data was described. The algorithm for improved estimation of the roadbed modulus, layer thickness and stiff layer depth were also introduced. This algorithm has been incorporated into a microcomputer based backcalculation program named MICHBACK. The MICHBACK program uses an enhanced version of the CHEVRON program (named CHEVRONX) to perform multilayer linear elastic analysis. The modification to the original CHEVRON program was done by Dr. Lynne Irwin of Cornell University. The structure of the MICHBACK program and its different components are illustrated in Chapter 5.

The theoretical aspects of the backcalculation program presented in Chapter 3 are validated in this chapter using theoretical deflection basins generated by CHEVRONX. Numerical examples have been incorporated to highlight the effects of incorrect layer thicknesses and stiff layer depth specifications on the backcalculated layer moduli. Two important properties of the program, the convergence characteristics and the uniqueness of the results are also examined. Analyses of the effect of errors in the deflections measured at different sensor locations on the backcalculated layer properties are presented. Sensitivity analysis of the backcalculated results is conducted and presented in both arithmetic and logarithmic scales. The sensitivity of the backcalculated moduli to Poisson's ratios is also highlighted.

Existing backcalculation programs use different analysis routines, as indicated in Table 2.2. Most of these routines are based on multilayer elastic analysis schemes. However, the deflections computed by these programs differ from each other. The difference in some cases is substantial. Consequently the backcalculated properties are affected not only by the backcalculation technique used but also by the analysis scheme.

In addition, the results of the MICHBACK program have been compared with those of two leading backcalculation programs, MODULUS 4.0 and EVERCALC 3.0.

4.2 TYPICAL PAVEMENT SECTIONS AND TEST PARAMETERS USED

The effectiveness of the MICHBACK program was studied by using four hypothetical pavement sections with known layer thicknesses and properties as shown in Tables 4.1 through 4.4. For each pavement section, its layer thicknesses and properties were used as inputs to the CHEVRONX computer program and the theoretical deflections at lateral distances of 0-, 8-, 12-, 18-, 24-, 36-, and 60-inch from the center of the loaded area were calculated. The other input factors to the CHEVRONX were as follows:

1. 9000-lb load.
2. A 5.91-inch radius of a circular contact area.
3. When incorporated, a stiff layer modulus of 5000 ksi, Poisson's ratio of 0.25, and depths from the pavement surface of 36, 144, and 240-inch.

The calculated theoretical deflection basins and the layer thicknesses were then used as inputs to the MICHBACK program and the layer moduli were calculated. A match between the backcalculated layer moduli and those listed in Tables 4.1 through 4.4 imply a perfect accuracy of the backcalculation routine.

The same calculated theoretical deflection basins and layer thicknesses were

Table 4.1. A typical three layer flexible pavement.

Layer	Thickness (in.)			Poisson's ratio	Modulus (ksi)
	Thin	Medium	Thick		
AC	2	5	9	.35	500
Base	8	8	8	.40	45
Roadbed				.45	7.5

Table 4.2. A typical four layer flexible pavement.

Layer	Thickness (in.)	Poisson's ratio	Modulus (ksi)
AC	6	.35	500
Base	10	.40	45
Subbase	6	.40	15
Roadbed		.45	7.5

Table 4.3. Typical four layer composite pavements.

Layer	Layer Type	Thickness in.	Poisson's ratio	Modulus Ksi
AC	AC	6	.35	500
		4	.35	500
Base	Unbound	8	.40	25
	PCC slab	8	.25	4500
Subbase	PCC slab	10	.25	4500
	Unbound	6	.40	25
Roadbed	Both		.45	7.5

Table 4.4. A typical five layer flexible pavement.

Layer	Thickness (in.)	Poisson's ratio	Modulus (ksi)
AC	4	.35	500
Treated Base	6	.35	100
Base	8	.4	45
Subbase	6	.4	15
Roadbed		.45	7.5

also used in two other leading backcalculation routines; MODULUS 4.0 and EVERCALC 3.0. The values of the seed moduli (the range for MODULUS) used in the three programs are provided below.

Computer program	Pavement system	Seed moduli (ksi)			
		AC	Base	Subbase	Roadbed soil
MICHBACK and EVERCALC	3-layers flexible	100	15	-	3
	4-layers flexible	100	15	7	3
	4-layers composite	100	7	1000	3
		100	1000	7	3
MODULUS	3-layers flexible	100-800	15-75	-	7
	4-layers flexible	100-800	15-75	7-50	7
	4-layers composite	100-800	7-50	1000-6000	7
		100-800	1000-6000	7-50	7

It should be noted that the above ranges in the seed moduli for the MODULUS program were expanded whenever the program had indicated that one of the limiting value had been reached. For the examples involving the stiff layer there was no change in the seed moduli or in the moduli ranges specified for the MODULUS program.

The MICHBACK and EVERCALC programs essentially require similar types of input parameters. The convergence criterion for the moduli for the two programs was specified as ϵ = .001 (.1%) (see Section 3.6). Further, the MODULUS program requires, as an input, the most expected value of the roadbed modulus, a value of 7 ksi (compared to the actual one of 7.5 ksi) was provided for all examples to make the comparison favorable to MODULUS. With the exception of problems involving a stiff layer at finite depths, a semi-infinite roadbed thickness was assumed for all other examples. The MODULUS program assigns the stiff layer modulus internally, but for the other two programs the actual value of the stiff layer modulus of 5000 ksi was specified. The MODULUS program was allowed to automatically assign weight factors to the different deflection locations and the "RUN A FULL ANALYSIS" option was used for all examples, so that material types were not required as input.

4.3 NUMERICAL ILLUSTRATION: IMPROVED ESTIMATION OF ROADBED MODULUS

An improved technique to accurately estimate the roadbed modulus at the onset of the analysis was presented in Chapter 3. The effectiveness of the method is illustrated in this section by using the pavement sections of Tables 4.1, 4.2, and 4.4.

The improved estimates of the roadbed moduli after the first iteration with a single call to the mechanistic analysis program (CHEVRONX) are presented in Tables 4.5 through 4.7. It can be seen that after only one call to the CHEVRONX program, the maximum error in the estimated roadbed value is 5 %. The importance of this improvement in the roadbed modulus after the very first iteration can be explained by the fact that the deflections at all locations are affected by the roadbed soil modulus. The accurate estimate of these effects at the onset of the analysis increases the accuracy and efficiency of the estimates of the moduli of the other pavement layers in the consequent iterations.

4.4 ESTIMATION OF LAYER THICKNESSES AND THEIR EFFECT ON THE BACKCALCULATED LAYER MODULI

The effect of incorrectly specified layer thicknesses on the backcalculated layer moduli is investigated using the MICHBACK, EVERCALC, and MODULUS

Table 4.5. Improvement of the roadbed modulus for a three layer flexible pavement.

Pavement type	Actual modulus (ksi)			Seed modulus (ksi)			Improved roadbed modulus	
	AC	Base	Roadbed	AC	Base	Roadbed	Modulus (ksi)	Error (%)
Thin	500	45	7.5	100	10	1	7.88	5.07
				1000	70	20	7.44	-0.8
Medium	500	45	7.5	100	10	1	7.54	0.53
				1000	70	20	7.5	0.0
Thick	500	45	7.5	100	10	1	7.38	-1.6
				1000	70	20	7.47	-0.13

Table 4.6. Improvement of the roadbed modulus for a four layer flexible pavement.

Actual modulus (ksi)				Seed modulus (ksi)				Improved roadbed modulus	
AC	Base	Subbase	Roadbed	AC	Base	Subbase	Roadbed	Modulus (ksi)	Error (%)
500	45	25	7.5	100	25	7	1	7.83	-1.6
				1000	100	45	30	7.37	-1.7

Table 4.7. Improvement of the roadbed modulus for a five layer flexible pavement.

Actual modulus (ksi)					Seed modulus (ksi)					Improved roadbed modulus	
AC	Base	Base	Subbase	Roadbed	AC	Base	Base	Subbase	Roadbed	Modulus (ksi)	Error (%)
500	100	45	15	7.5	100	50	15	7	1	7.7	2.67
					1000	300	70	50	20	7.33	-2.27

programs. The performance of the Newton's method to predict the actual layer thicknesses from incorrectly specified values is highlighted in this section.

The three layer flexible pavement with medium AC thickness (Table 4.1) was used to study the effect of incorrectly specified layer thicknesses on the backcalculated layer moduli. Either the AC or the base layer thickness was specified with an error range of ± 40 % relative to the true thickness. Two types of backcalculations were performed using MICHBACK: one with automatic correction of the incorrectly specified thickness (MICHBACK1), and the other with the thickness held fixed at the incorrect value as done by the other two programs (MICHBACK2). The percent errors in the estimated modulus values relative to the actual ones are calculated and presented below.

4.4.1 AC Thickness

The percent errors in the backcalculated layer moduli due to an incorrectly specified AC layer thickness are presented in Table 4.8 and Figures 4.1 through 4.3. Examination of the figures indicate that for all three pavements:

1. The effect of the errors in the AC thickness is most pronounced on the AC modulus.
2. The roadbed modulus is relatively insensitive to inaccuracies in the AC layer thickness.
3. A positive error in the AC layer thickness results in a lower prediction of the AC and base moduli and a negative error results in stiffening of the two moduli.
4. When the option to correct the erroneous AC thickness was used, MICHBACK produced accurate prediction of all layer moduli along with the AC thickness.

Table 4.8. The percent error in the backcalculated layer moduli due to percent errors in the AC thickness.

Error (%)	AC				Base				Roadbed			
	MB1	MB	EC	MOD	MB1	MB	EC	MOD	MB1	MB	EC	MOD
-40	-2.1	186	184.3	199.8	-1.6	55.2	52.8	49.1	0.0	-0.3	0.0	0.0
-20	-2.1	50.1	45.9	48.1	-1.6	27.3	27.2	27.8	0.0	-0.2	0.0	0.0
-10	-2.0	19.9	13.0	19.6	-1.6	13.4	15.4	13.8	0.0	-0.1	0.0	0.0
-5	-2.0	9.0	2.6	5.2	0.0	6.6	7.1	7.1	0.0	0.0	0.0	0.0
0	0.0	0.0	-4.4	0.6	-1.6	0.0	2.2	-0.9	0.0	0.0	0.0	0.0
5	-2.0	-7.6	-14.2	-9.4	-1.6	-6.5	-4.3	-7.1	0.0	0.1	0.0	0.0
10	-2.0	-14.0	-20.7	-14.8	-1.6	-12.8	-7.8	-12.2	0.0	0.1	0.0	0.0
20	2.2	-24.1	-28.4	-25.3	-1.6	-25.0	-20.4	-23.6	0.0	0.3	0.0	0.0
40	-2.1	-38.3	-42.6	-40.2	-1.6	-46.2	-40.0	-43.3	0.0	0.7	0.1	0.0

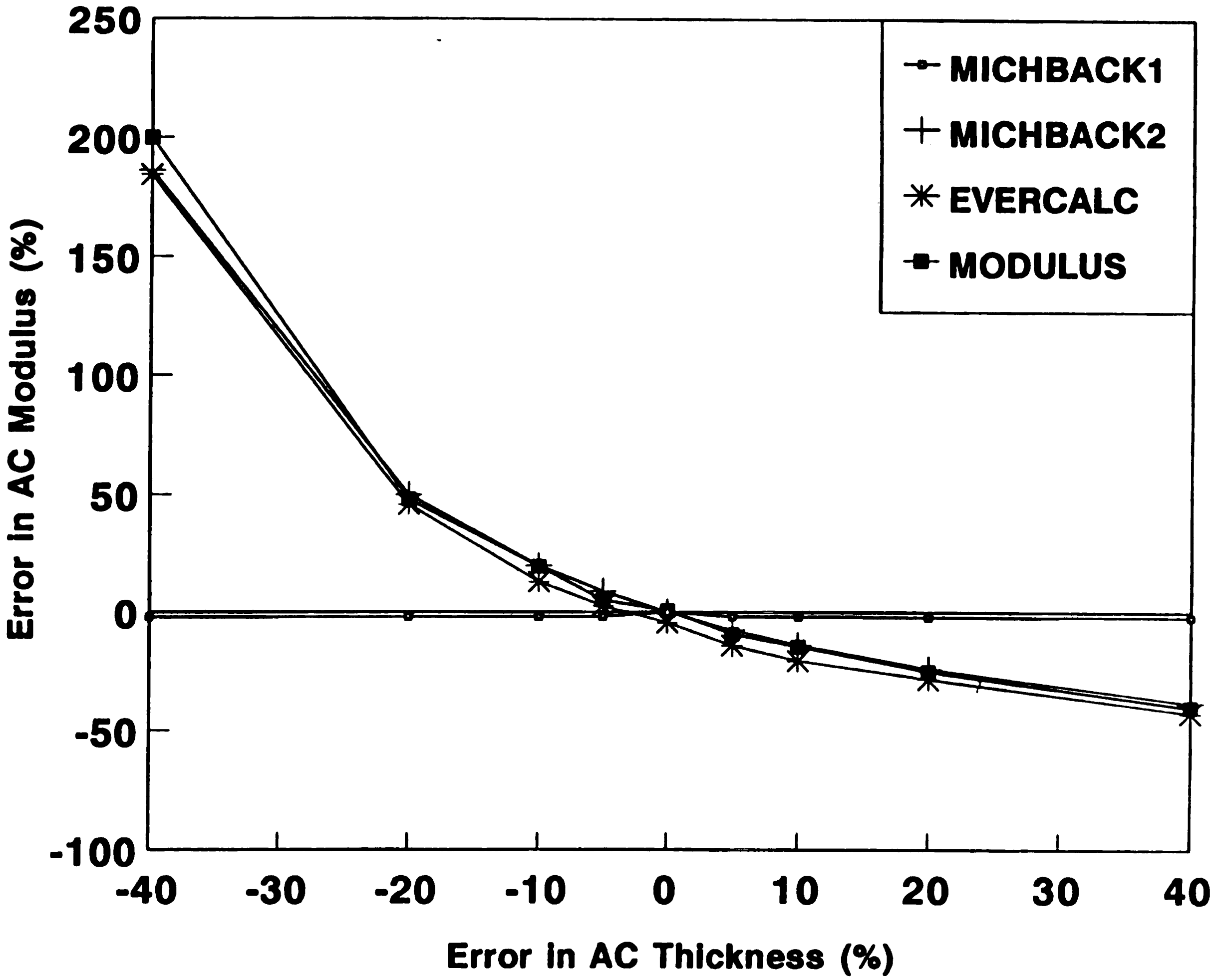

Figure 4.1. Errors in the backcalculated AC modulus due to an incorrect AC thickness specification.

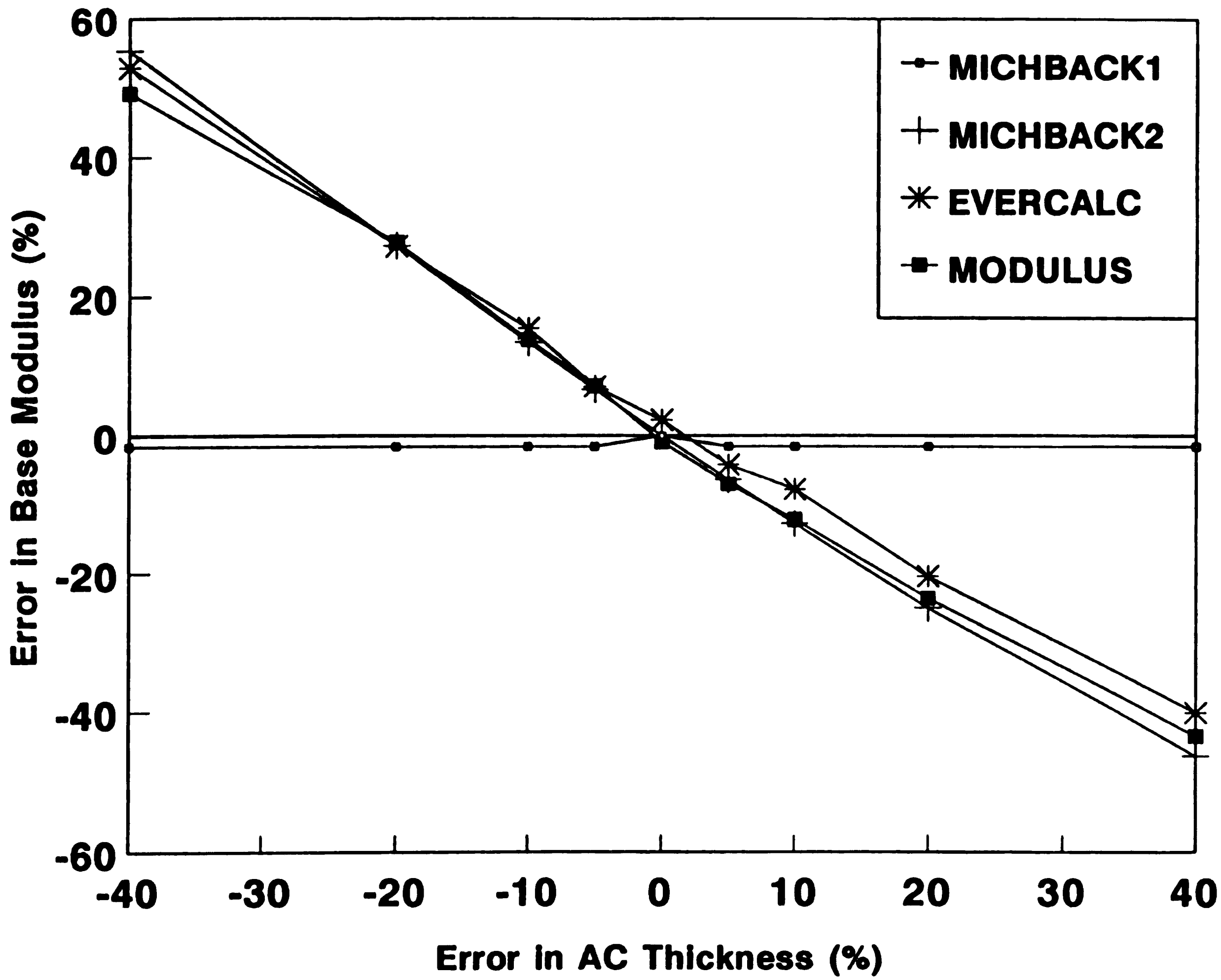

Figure 4.2. Errors in the backcalculated base modulus due to an incorrect AC thickness specification.

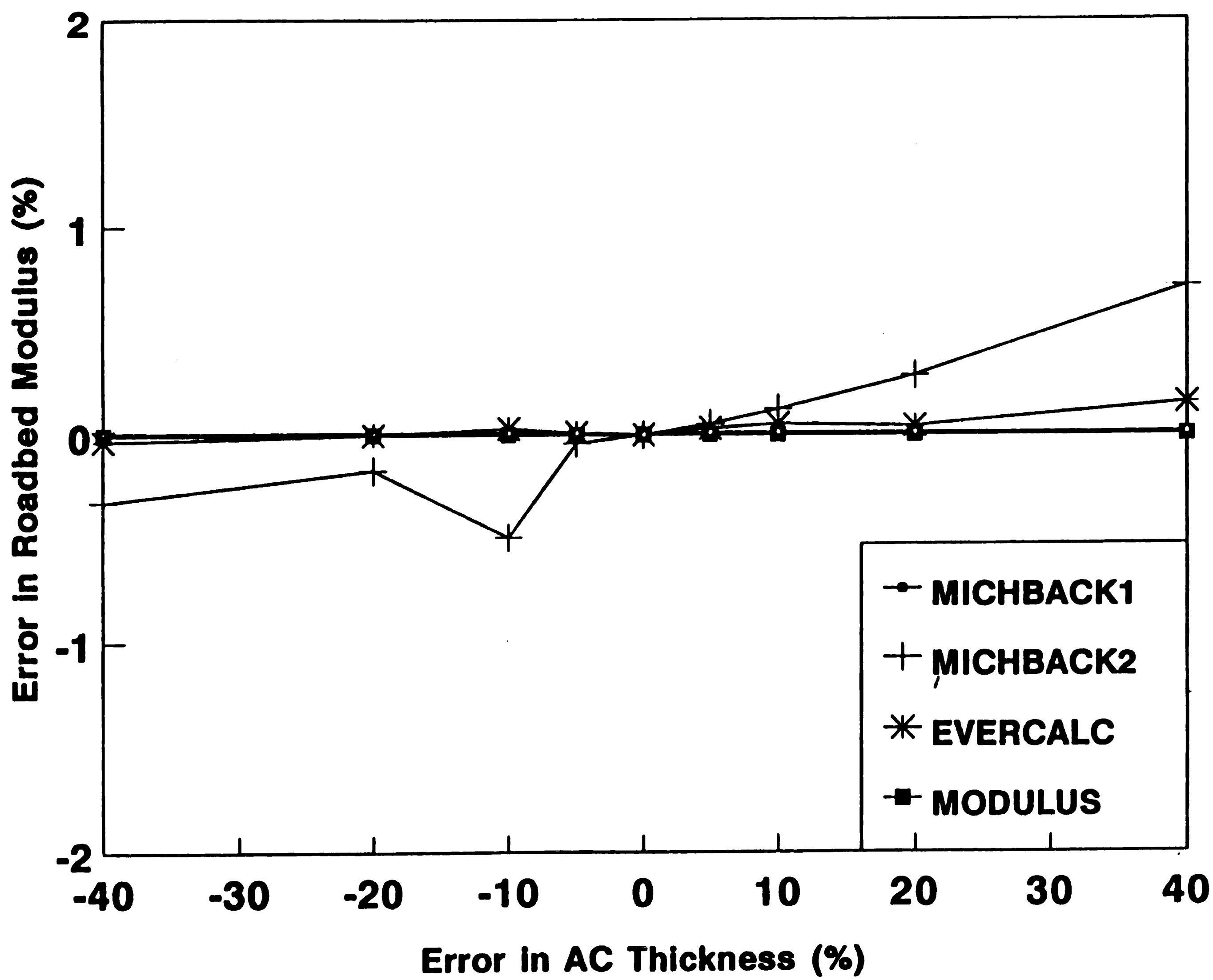

Figure 4.3. Errors in the backcalculated roadbed modulus due to incorrect AC thickness specification.

4.4.1.1 Effect of AC Thickness Error on Pavements with Different AC Thicknesses

The effect of incorrectly specified AC thicknesses on the layer moduli of pavements with different AC thickness was studied using three typical three-layer flexible pavements listed in Table 4.1. The AC thickness was varied between ± 10 % for all three programs. The results obtained from the programs are presented in Table 4.9. Examination of the results of the three programs indicate:

1. Inaccuracies in the AC thickness affect the AC modulus of thin AC layer pavements the most and thick pavements the least.
2. The base modulus was affected the most for thick pavements and the least for thin ones. This observation was further investigated by studying the change in the vertical stresses at the top of the base layer with inaccuracies in the AC layer thickness. The study reveals that owing to an inaccuracy of -10% in the AC thickness of the thin pavement, the increase in the vertical stress at the top of the base layer is about 6%, compared to about 20% for the thick pavement as shown in Figure 4.4.
3. For EVERCALC, it can be noted that for the thick pavement, even for cases in which the error in the AC thickness is negative, the predicted AC modulus is always smaller than the actual. This is because of the relative inaccuracy of the older version of CHEVRON (used in EVERCALC 3.0), which becomes more significant for stiffer pavements. This discrepancy is highlighted in Section 4.10 where the results of different elastic layer programs are compared.

Table 4.9. Errors in the backcalculated layer moduli due to errors in the specified AC thickness for different AC thicknesses.

Error	Program	AC			Base			Roadbed		
		Thick	Med	Thin	Thick	Med	Thin	thick	Med	Thin
-10	MB	32.6	19.9	9.0	2.2	13.4	45.5	-0.2	-0.5	-0.2
	EC	14.0	13.0	-1.4	1.1	15.4	57.0	-0.1	0.0	-0.1
	MOD	48.7	19.6	10.5	0.0	13.8	40.9	0.0	0.0	0.0
-5	MB	15.2	9.0	4.2	1.1	6.6	21.9	-0.1	0.0	-0.1
	EC	12.6	2.6	-5.6	1.28	7.1	34.2	-1.0	0.0	-0.1
	MOD	14.8	5.2	9.6	1.6	7.1	4.2	0.0	0.0	0.0
0	MB	-0.4	0.0	0.2	0.0	0.0	-0.8	0.0	0.0	0.0
	EC	0.7	-4.4	-12.1	-0.2	2.25	28.9	0.0	0.0	-0.1
	MOD	-2.9	0.6	-2.9	2.0	-0.9	-4.0	0.0	0.0	0.0
5	MB	-11.5	-7.6	-4.1	-1.1	-6.5	-19.7	0.0	0.1	0.2
	EC	-12.8	-14.2	-13.1	-0.7	-4.3	-5.6	0.0	0.0	0.0
	MOD	-9.2	-9.4	-3.9	-0.9	-7.1	-25.3	0.0	0.0	1.3
10	MB	-20.8	-14.0	-7.9	-2.2	-12.8	-37.6	0.1	0.1	0.4
	EC	-22.6	-20.7	-16.6	-1.6	-7.8	-22.8	0.0	0.1	0.1
	MOD	-19.0	-14.8	-9.7	-1.8	-12.2	-72.4	0.0	0.0	4.0

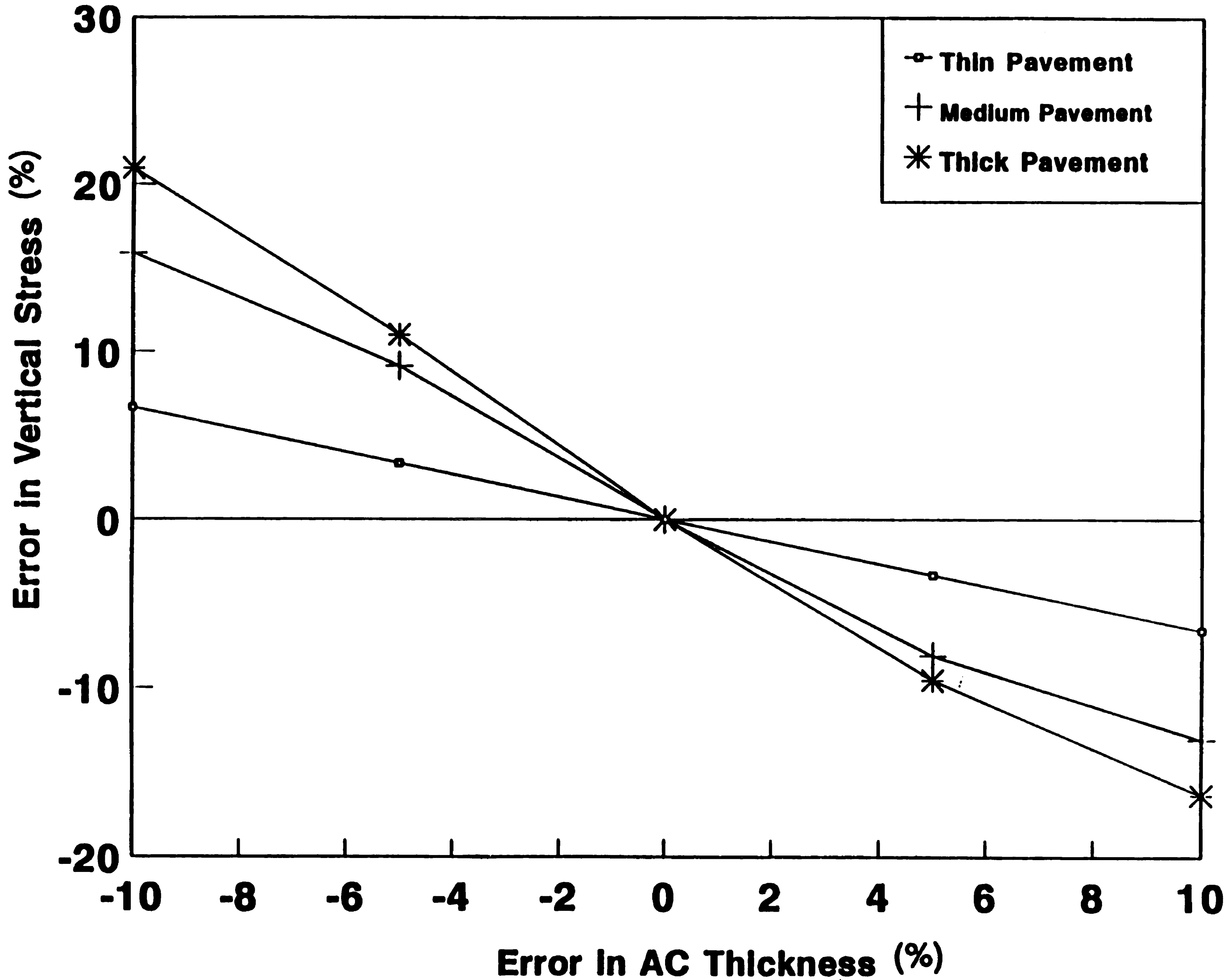

Figure 4.4. Errors in the vertical stress at the top of the base layer due to incorrect AC thickness specification.

4.4.1.2 <u>Effect of AC Thickness Error on Pavements with Different AC Stiffness</u>

The medium thickness pavement listed in Table 4.1 was used to study the effect of error in the AC thickness for pavements having different AC stiffness. The AC moduli of 300, 500, and 800 ksi was used to simulate soft, medium and stiff pavements. The resulting errors in the different layer moduli due to the error in the AC thickness are presented in Table 4.10.

As it was expected, for the same inaccuracy in the AC layer thickness, the error in the backcalculated AC layer modulus is higher for stiffer pavements for all three programs. The MICHBACK and EVERCALC programs have similar trends for the base modulus also (i.e., having greater error for stiffer pavements), but the MODULUS program gave erratic results not indicating any trends.

The roadbed modulus remained insensitive to inaccuracies in the AC thickness for all three programs as observed earlier.

4.4.2 Base Thickness

The percent errors in the backcalculated layer moduli due to an incorrectly specified base layer thickness are presented in Table 4.11, and Figures 4.5 through 4.7. The observations from these results for all three programs are:

1. The roadbed modulus is relatively insensitive to errors in the base thickness.
2. Both the AC and the base layer moduli are significantly affected by inaccuracies in the base layer thickness. With the base layer being affected the most.
3. The base layer modulus shows stiffening effect due to negative errors in the base layer thickness and vice versa.
4. The AC modulus was found to be positively related to the errors in the base layer thickness (i.e., negative errors in the specified thickness produce

Table 4.10. Errors in the backcalculated layer moduli due to errors in the specified AC thickness for different AC modulus.

Error (%)	Program	AC			Base			Roadbed		
		Lean	Med	Stiff	Lean	Med	Stiff	Lean	Med	Stiff
-10	MB	15.6	19.9	23.7	12.2	13.4	14.3	0.0	-0.5	0.0
	EC	10.1	13.0	17.3	13.5	15.4	17.3	0.1	0.0	0.0
	MOD	8.1	19.6	31.1	18.0	13.8	5.6	0.0	0.0	0.0
-5	MB	7.3	9.0	10.8	5.8	6.6	6.7	0.0	0.0	0.0
	EC	3.6	2.6	4.19	7.03	7.1	10.8	0.1	0.0	0.0
	MOD	4.4	5.2	8.6	6.9	7.1	3.6	0.0	0.0	0.0
0	MB	0.5	0.0	0.2	-0.4	0.0	-0.2	0.0	0.0	0.0
	EC	-2.9	-4.4	-5.8	1.0	2.25	4.0	0.0	0.0	0.0
	MOD	-2.0	0.6	-4.3	2.4	-0.9	-6.0	0.0	0.0	0.0
5	MB	-5.6	-7.6	-8.6	-6.1	-6.5	-7.2	0.0	0.1	0.1
	EC	-10.6	-14.2	-15.3	-5.5	-4.3	-1.2	0.0	0.0	0.0
	MOD	-8.4	-9.4	-11.0	-5.3	-7.1	-8.7	0.0	0.0	0.0
10	MB	-10.5	-14.0	-1.1	-12.2	-12.8	-14.1	0.1	0.1	0.2
	EC	-15.5	-20.7	-24.7	-9.6	-7.8	-5.2	0.0	0.1	0.1
	MOD	-9.2	-14.8	-13.7	-12.7	-12.2	-18.2	0.0	0.0	0.0

Table 4.11. The percent errors in the backcalculated layer moduli due to percent errors in the base thickness.

Error (%)	AC				Base				Roadbed			
	MBI	MB	EC	MOD	MBI	MB	EC	MOD	MBI	MB	EC	MOD
-40	-0.6	-32.8	-44.2	-41.0	1.5	181.7	246.8	234.7	0.0	3.0	1.9	2.7
-20	-0.6	-14.9	-26.1	-20.4	1.5	51.4	69.2	62.9	0.0	1.3	0.8	1.3
-10	-0.6	-7.0	-15.1	-10.2	1.5	20.7	28.7	25.3	0.0	0.6	0.4	0.0
-5	-0.6	-3.4	-9.1	-4.5	1.5	9.4	14.3	10.7	0.0	0.3	0.2	0.0
0	0.0	0.0	-4.4	0.6	0.0	0.0	2.2	-0.9	0.0	0.0	0.0	0.0
5	-0.6	3.1	0.6	5.1	1.5	-7.9	-7.6	-9.8	0.0	-0.3	-0.1	0.0
10	-0.6	6.0	5.3	8.2	1.5	-14.6	-15.7	-16.2	0.0	-0.5	-0.3	0.0
20	-0.6	11.3	13.6	13.3	1.5	-25.3	-28.3	-27.1	0.0	-0.9	-0.5	0.0
40	-0.6	20.1	27.0	23.5	1.5	-39.8	-44.1	-41.8	0.0	-2.23	-0.8	-1.3

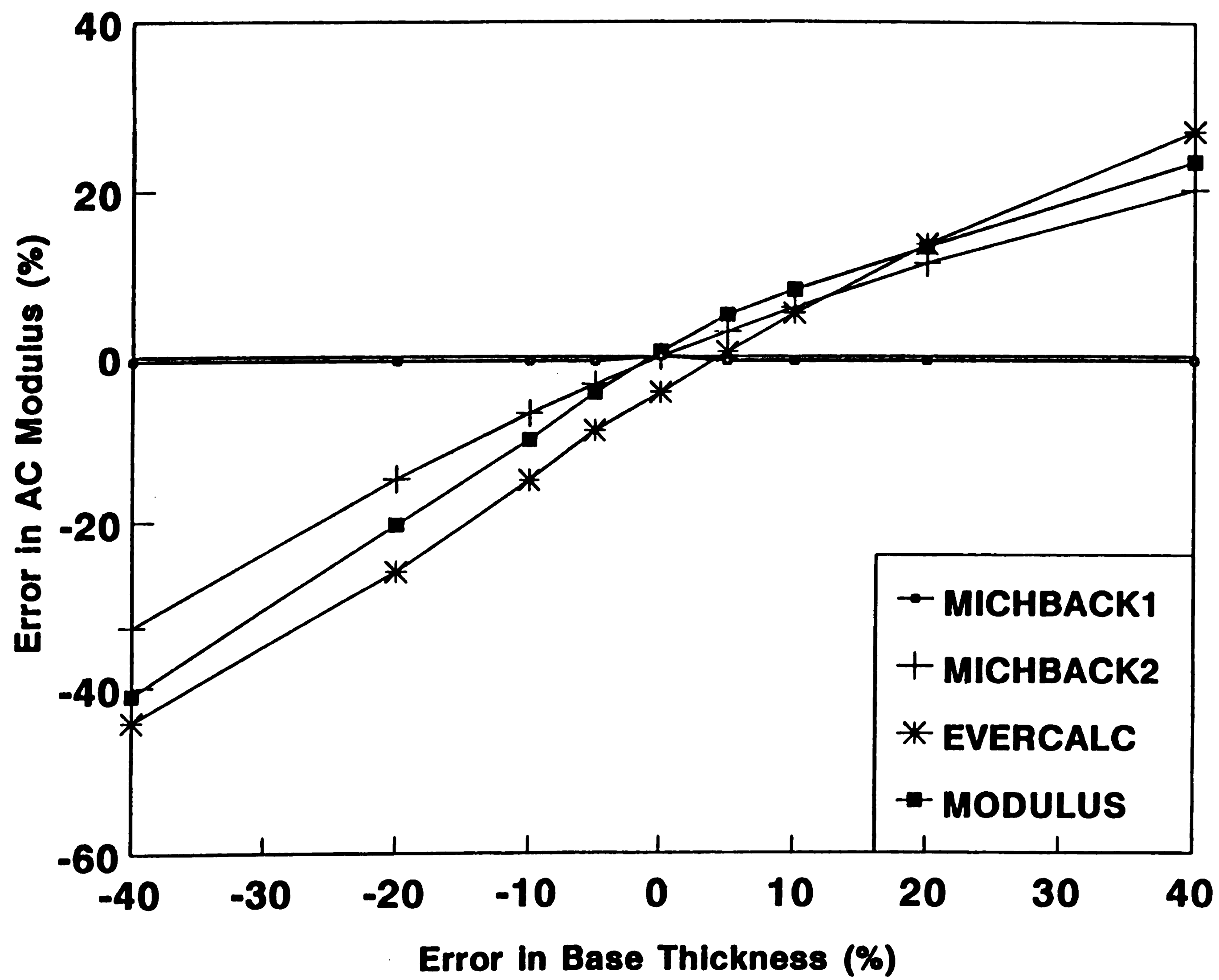

Figure 4.5. Errors in the backcalculated AC modulus due to incorrect base thickness specification.

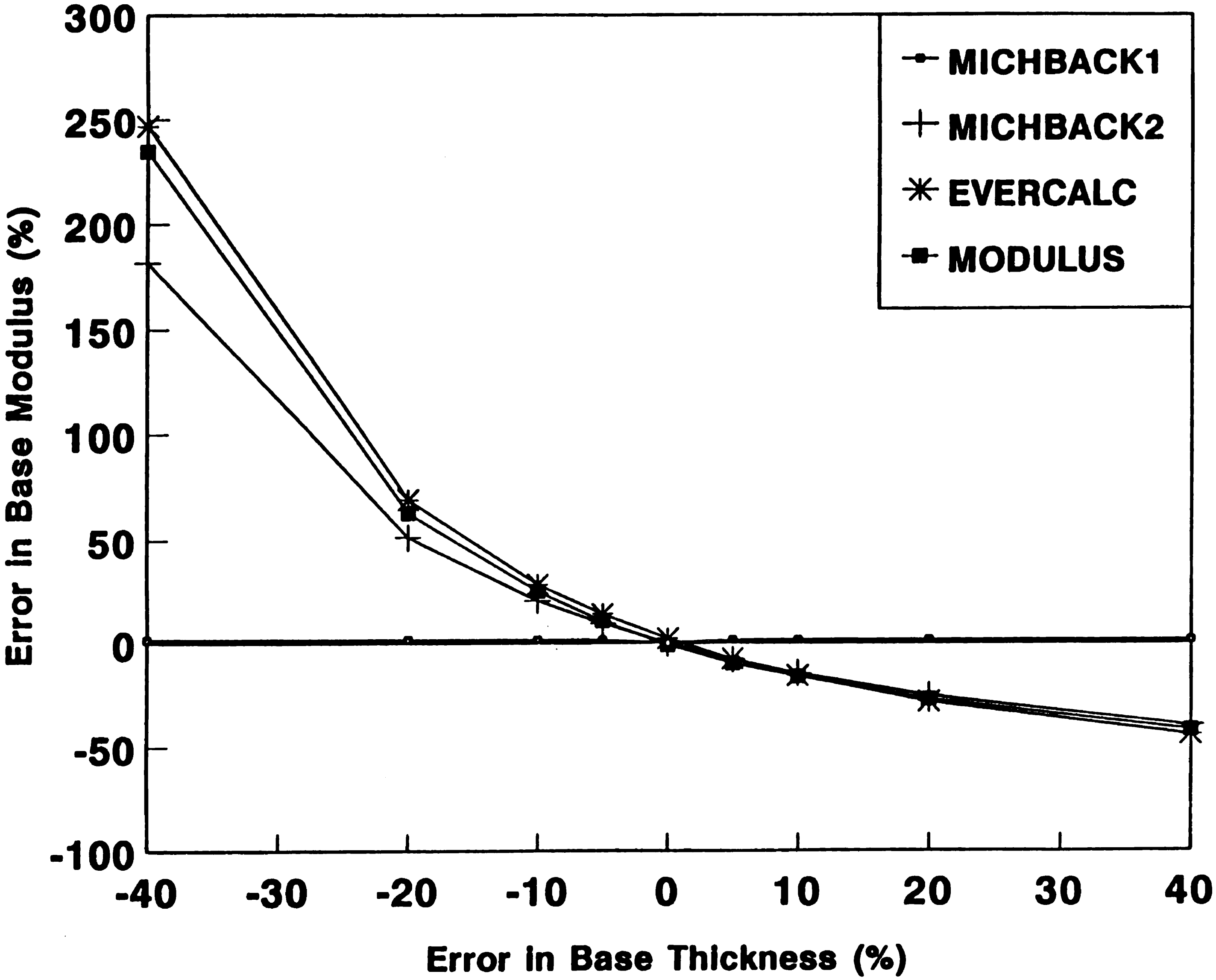

Figure 4.6. Errors in the backcalculated base modulus due to incorrect base thickness specification.

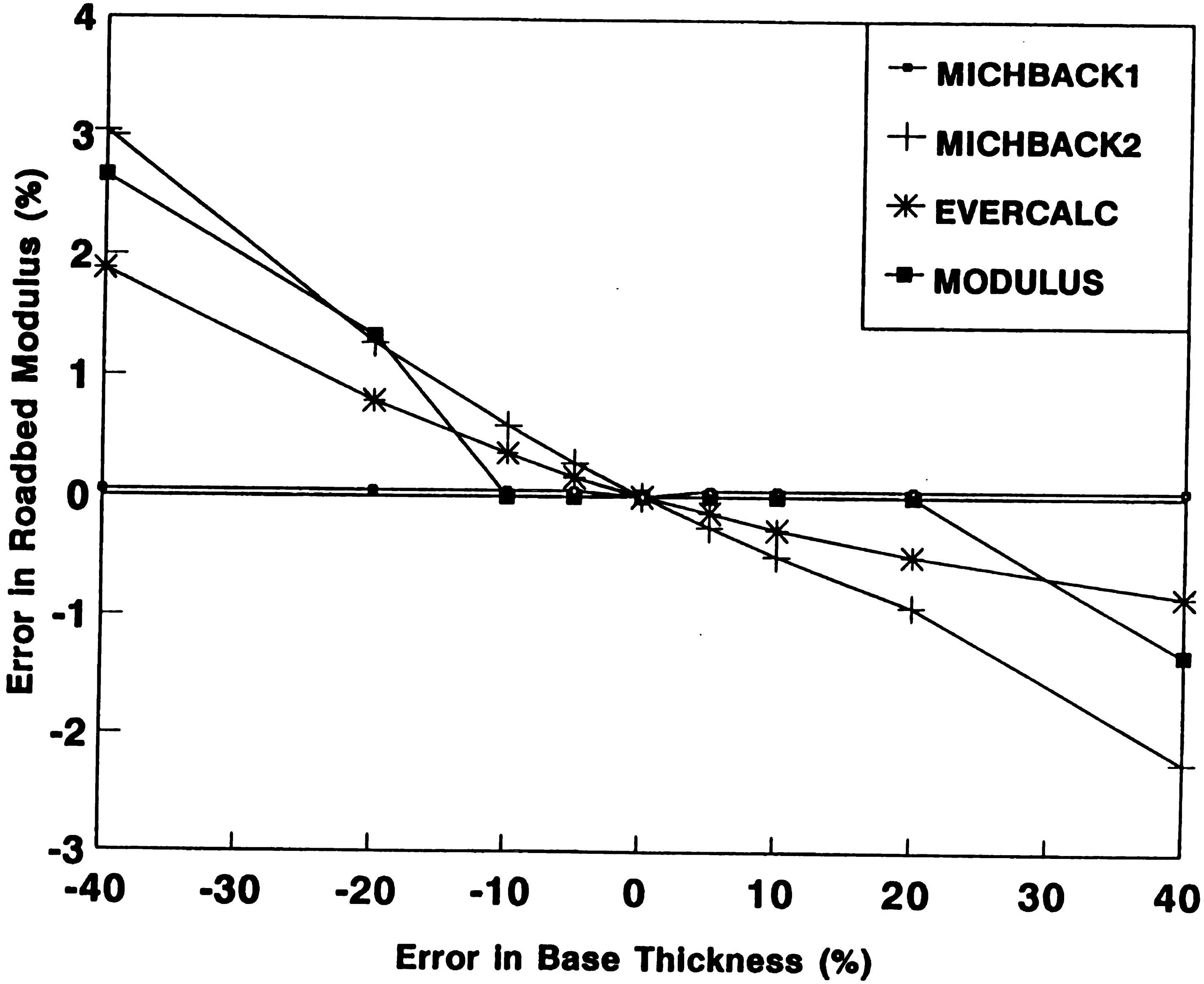

Figure 4.7. Errors in the backcalculated roadbed modulus due to incorrect base thickness specification.

negative errors in the AC modulus). This is probably because of the sensitivity of the base modulus to the errors in base thickness. Small errors in the base thickness result in higher errors in the base modulus which are compensated by corresponding errors in the AC layer moduli.

5. Again, the results of MICHBACK1 (where layer thickness option was used) indicate excellent prediction of all layer moduli and the base layer thickness.

4.5 STIFF LAYER EFFECTS AND DEPTH ESTIMATION

The effect of the stiff layer depth may be observed either by the presence of a stiff layer under the roadbed soil or even by the soil overburden and stress hardening. The need to incorporate the stiff layer at an appropriate depth has been recognized and various methods have been developed and discussed in Section 3.9. A mechanistic based method introduced in Chapter 3 has been incorporated in the MICHBACK program.

The capability of the three programs to estimate the stiff layer depth along with pavement layer moduli was tested using the three-layer medium thick flexible the four layer flexible, and the composite pavement sections listed in Tables 4.1, 4.2, and 4.5 respectively. The deflection data was generated using the CHEVRONX program. For the stiff layer, a modulus of 5000 ksi was used for the MICHBACK and EVERCALC programs, whereas the MODULUS program assigns the modulus internally. The stiff layer depth was varied between 36 and 240 inch. The results are presented below.

4.5.1 Backcalculation of Layer Moduli and Stiff Layer Depth

For the three layer pavements, the results are presented in Table 4.12, the salient points observed are:

Table 4.12. Backcalculation of stiff layer depth and moduli for a three layer flexible pavement.

Stiff Layer depth	Program	Percentage error in backcalculated properties			
		Stiff layer depth	AC modulus	Base modulus	Roadbed modulus
Shallow (36 in.)	MICHBACK	-0.3	-0.5	0.7	-0.6
	EVERCALC	5.6	23.6	-19.7	15.2
	MODULUS	5.6	10.5	-20.4	21.3
Medium (144 in.)	MICHBACK	-0.1	0.5	-0.6	0.1
	EVERCALC	-41.7	-56.2	179.7	-42.8
	MODULUS	-41.6	-23.7	59.6	-32.0
Deep (240 in.)	MICHBACK	0.3	1.0	-1.1	0.15
	EVERCALC	-56.6	-53.9	144.3	-36.7
	MODULUS	-52.5	-39.6	118.9	-33.3

1. The regression equations used by MODULUS and EVERCALC yield quite accurate results for the shallow stiff layer cases, but as the depth to the stiff layer increases so does the error. However, as shown in the following section the backcalculated layer moduli are more sensitive to the inaccuracies in the depth for a shallow stiff layer.
2. For the MODULUS and EVERCALC programs the error in the predicted stiff layer depth ranges from about 6% to 50% for the shallow and deep stiff layer cases.
3. MICHBACK on the other hand, with an initial start from the regression equations (Baladi, 1993) can converge to accurate results by refining the stiff layer depth as discussed in Section 3.9.1. The maximum error in the backcalculated depth to stiff layer remained below 0.5% for all examples.
4. Hence, incorporation of the stiff layer had no impact on the results backcalculated by MICHBACK, whereas the results of other two programs are significantly affected.

For the four layer flexible pavement the results presented in Table 4.13 indicate:

1. The regression equations yield results quite similar to those of the three layer pavement. Hence, the errors in the backcalculated moduli for the MODULUS and EVERCALC programs are about the same as they were for the three layer pavement.
2. Results from MICHBACK program are way better than the other two programs.

The stiff composite pavement listed in Table 4.4 was used to study the capability of the three programs to predict the layer properties of composite pavements with a stiff layer present at a finite depth. The highlights of the results

Table 4.13. Backcalculation of stiff layer depth and moduli for a four layer flexibile pavement.

Stiff layer depth	Program	Percent Error in Backcalculated Properties				
		Stiff layer depth	AC modulus	Base modulus	Subbase modulus	Roadbed modulus
Shallow (48 in.)	MICHBACK	2.5	0.8	-2.6	3.3	6.4
	EVERCALC	6.7	0.37	20.4	-80.6	139.9
	MODULUS	6.9	4.3	18.2	72.7	9.3
Medium (144 in.)	MICHBACK	-5.3	0.04	0.1	1.4	-0.7
	EVERCALC	-31.5	-31.6	-37.0	565.1	-41.9
	MODULUS	-31.3	2.6	-20.4	221.3	-36.0
Deep (240 in.)	MICHBACK	2.2	-0.2	1.1	-3.9	0.4
	EVERCALC	-35.2	12.8	-50.3	464.7	-29.3
	MODULUS	-35.0	-3.9	-4.7	118.7	-24.0

presented in Table 4.14 are:

1. The regression equations were probably not designed to account for composite pavements. The errors in the stiff layer depth prediction for the EVERCALC and MODULUS programs are over 300% and induce errors higher than 400% in the backcalculated moduli.
2. For the shallow stiff layer depth the prediction error for the two programs was almost the same, the depth being over predicted by about 130%. But for medium and deep stiff layer cases the MODULUS program predicted the depths as 300 inch. and EVERCALC as 600 inch. for both cases. This suggests that MODULUS has 300 inch. and EVERCALC 600 inch. as the maximum allowed depth to a stiff layer. The regression equations must have predicted values higher than these and the programs must have fixed the depth to the stiff layer at the arbitrarily set maximum limit. As a result the error in the predicted depths to the stiff layer for the two programs is different for these two cases.
3. It is highly recommended that whenever a stiff layer is used with composite pavements, the results from MODULUS and EVERCALC programs should be scrutinized carefully.
4. Results from MICHBACK, however, were comparable to those of the four layer flexible pavement and consistently better than those of the two leading programs.

4.5.2 Sensitivity of Backcalculated Moduli to Stiff Layer Characteristics

Having established the ranges of error in the stiff layer depth estimation by the three programs, the effect of inaccuracies in the depth to stiff layer on the backcalculated moduli was studied. Coupled with depth, the sensitivity of the backcalculated results to the stiffness of the stiff layer is also studied.

Table 4.14. Backcalculation of stiff layer depth and moduli for a four layer composite pavement.

Stiff Layer depth	Program	Percent Error in Backcalculated Properties				
		Stiff layer depth	AC modulus	Base modulus	Subbase modulus	Roadbed modulus
Shallow (48 in.)	MICHBACK	-3.1	-1.6	1.9	-13.7	5.9
	EVERCALC	172.95	-15.78	-28.55	-18.21	661.8
	MODULUS	172.3	-14.92	-24.45	35.96	353.3
Medium (144 in.)	MICHBACK	0.5	-1.1	0.5	6.8	0.1
	EVERCALC	329.0	40.9	-17.38	-95.75	201.0
	MODULUS	108.3	26.97	-36.33	15.28	83.03
Deep (240 in.)	MICHBACK	-0.7	0.2	-2.0	54.6	-1.0
	EVERCALC	150.0	198.8	-37.5	-96.0	78.8
	MODULUS	25.1	17.36	-25.7	279.8	12.4

4.5.2.1 Effect of the Stiff layer Depth on the Backcalculated Layer Moduli

The backcalculation was performed with an incorrectly specified depth to the stiff layer depth. As discussed in the previous section, many researchers have advocated the use of a stiff layer at an arbitrarily fixed depth. The medium pavement listed in Table 4.1 was used to study the effect of the depth to stiff layer. With all other parameters kept constant, the depth of the stiff layer with a modulus of 5000 ksi was varied between 36 and 600 inch.

The results are presented in Figure 4.8. It can be seen that the deflections obtained by an elastic layer program are affected even by a stiff layer located as deep as 600 inch. However, as the stiff layer depth increases, the effect of the stiff layer becomes smaller relative to the solutions where presence of the stiff layer was ignored.

The effect of using an incorrect stiff layer location on the backcalculated moduli was studied by using all three programs. Here the stiff layer was deliberately specified incorrectly. The stiff layer at medium depth (144 inch.) along with the medium three layer flexible pavement (Table 4.1) was used, and the error in the stiff layer depth was varied between ± 40 %. The errors in the backcalculated moduli are presented in Table 4.15 and Figures 4.9 through 4.11.

The trends observed for all three programs are:

1. The roadbed modulus backcalculated by all three programs is sensitive to errors in the stiff layer depth, since the error directly affects the roadbed thickness and hence its stiffness.
2. When the roadbed thickness is over-estimated, the roadbed modulus is also over-predicted as to reduce the total compression of the roadbed soil, and vice versa.
3. In general, the backcalculated moduli for the AC and the base layers interact

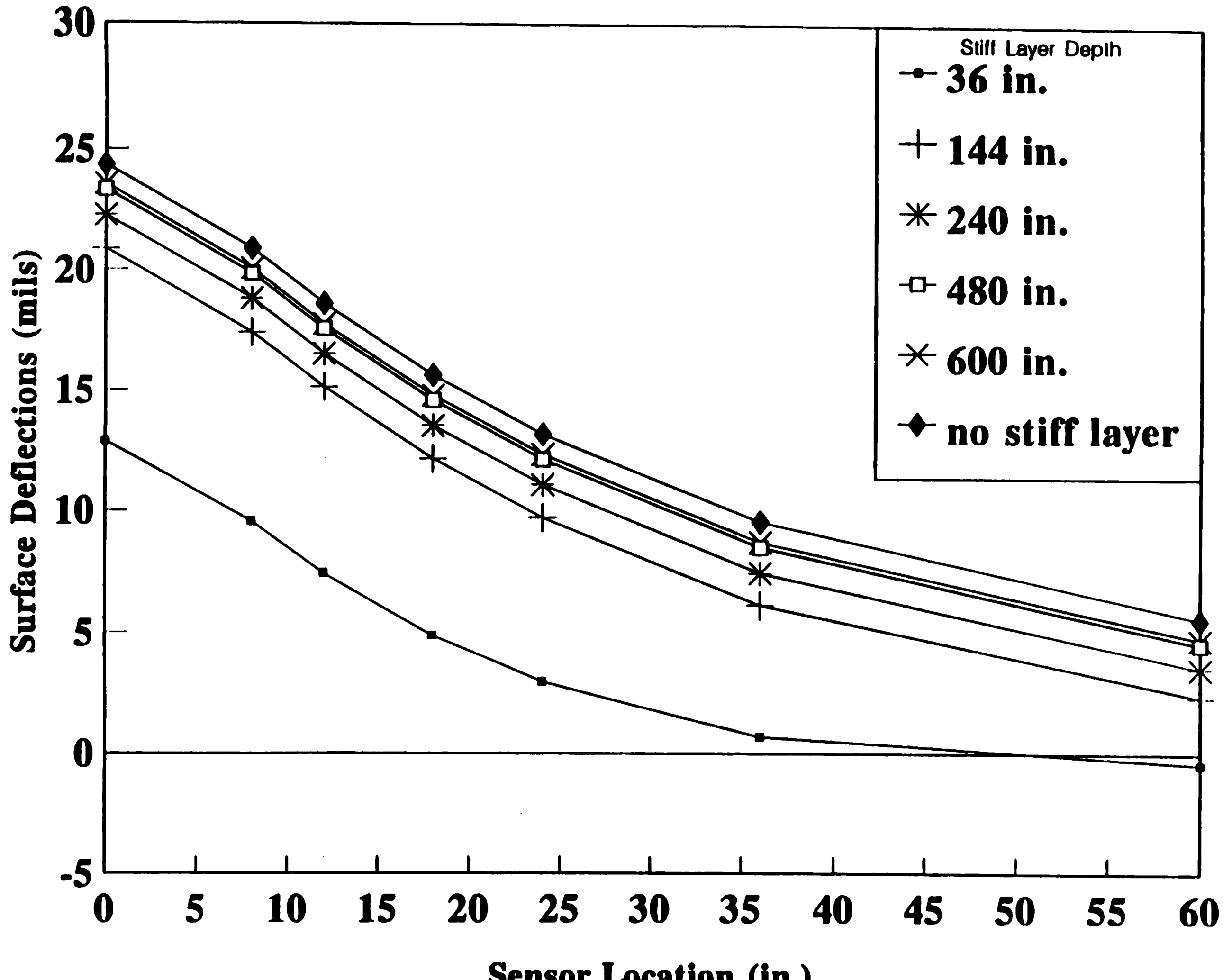

Figure 4.8. Effect of the depth to stiff layer on surface deflections.

Table 4.15. Error in the backcalculated layer moduli due to percent error in the depth to the stiff layer.

Error (%)	Percent error in the backcalculated results for the indicated programs								
	AC			Base			Roadbed		
	MB	EC	MOD	MB	EC	MOD	MB	EC	MOD
-40	-30.7	-56.7	-23.6	72.4	170.5	57.1	-33.4	-41.8	-30.7
-20	-14.2	-55.8	-11.3	27.8	82.5	22.2	-13.4	-16.6	-12.0
-10	-7.02	-45.0	-3.6	12.7	43.1	8.0	-6.1	-7.6	-5.3
-5	-3.22	-34.9	-0.4	5.8	24.2	2.0	-2.9	-3.7	-1.3
0	0.4	-21.6	2.8	-.4	6.6	-3.3	0.1	0.0	1.3
5	3.92	-6.0	4.6	-6.1	-9.6	-7.1	2.8	0.0	4.0
10	7.3	11.2	7.1	-11.3	-24.4	-11.1	5.3	6.8	5.3
20	13.5	47.2	7.2	-20.3	-49.5	-14.0	9.7	13.1	9.3
40	24.1	104.5	15.5	-34.4	-78.3	-26.0	17.1	25.7	16.0

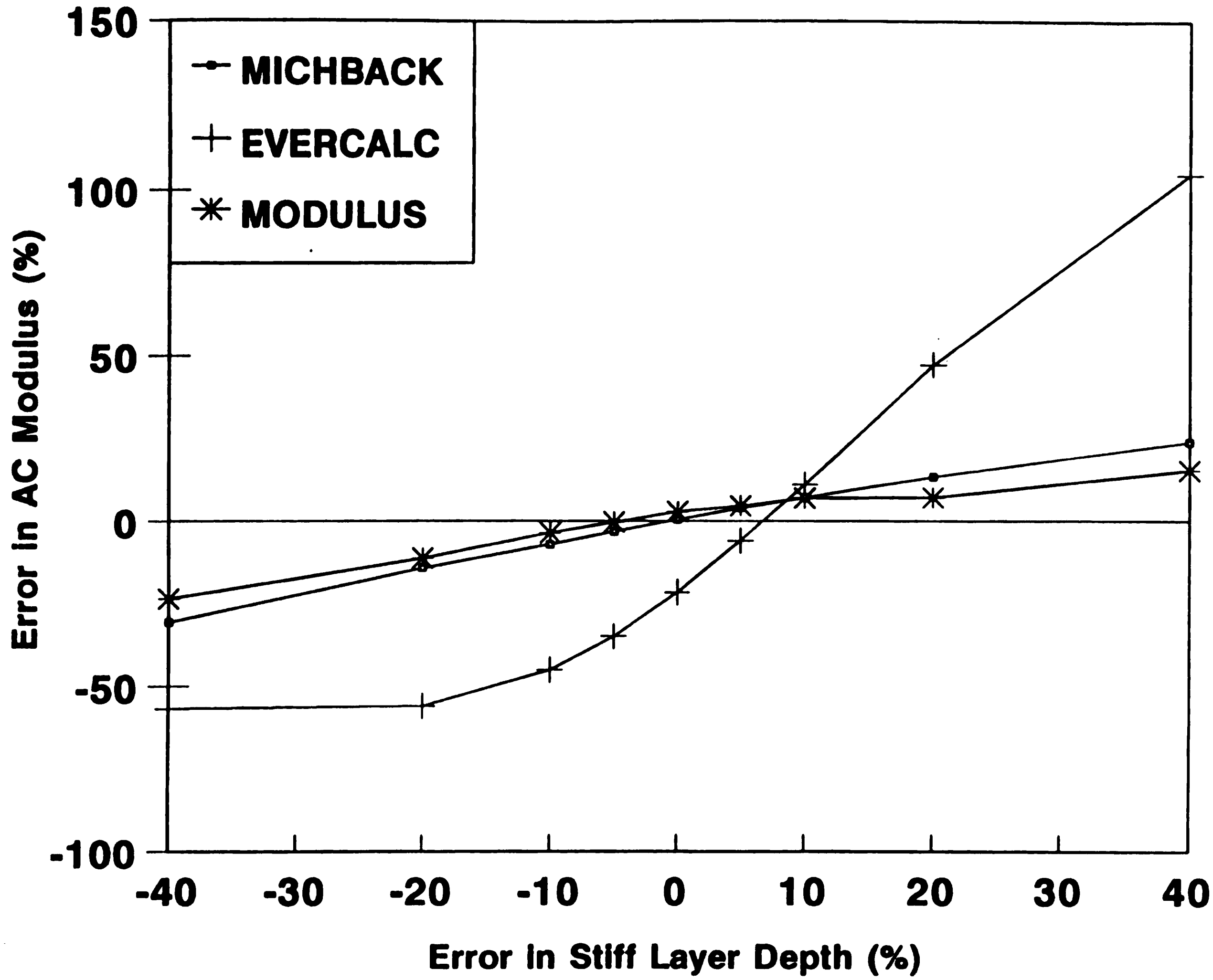

Figure 4.9. Errors in backcalculated AC modulus due to incorrect stiff layer depth specification.

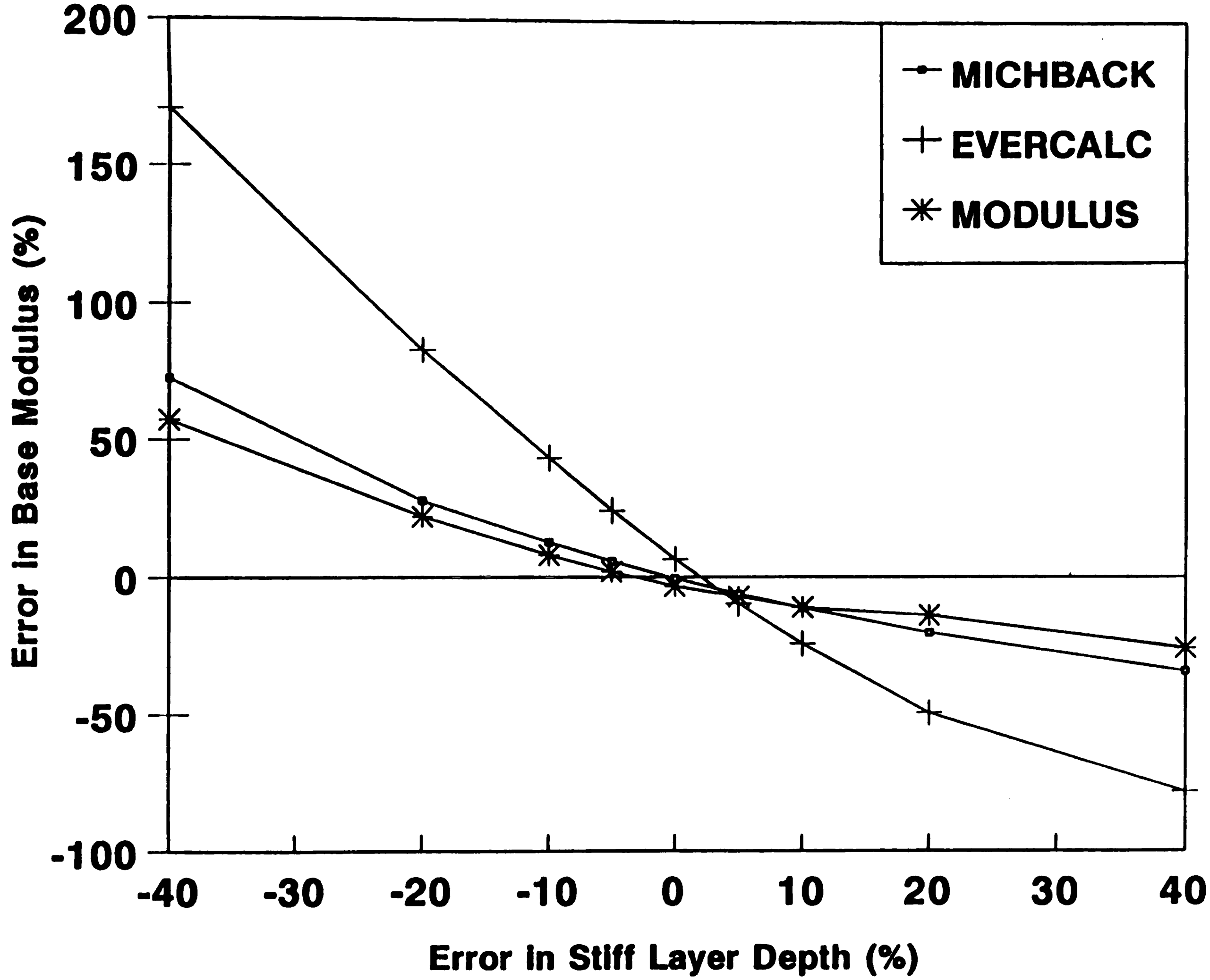

Figure 4.10. Errors in the backcalculated base modulus due to incorrect stiff layer depth specification.

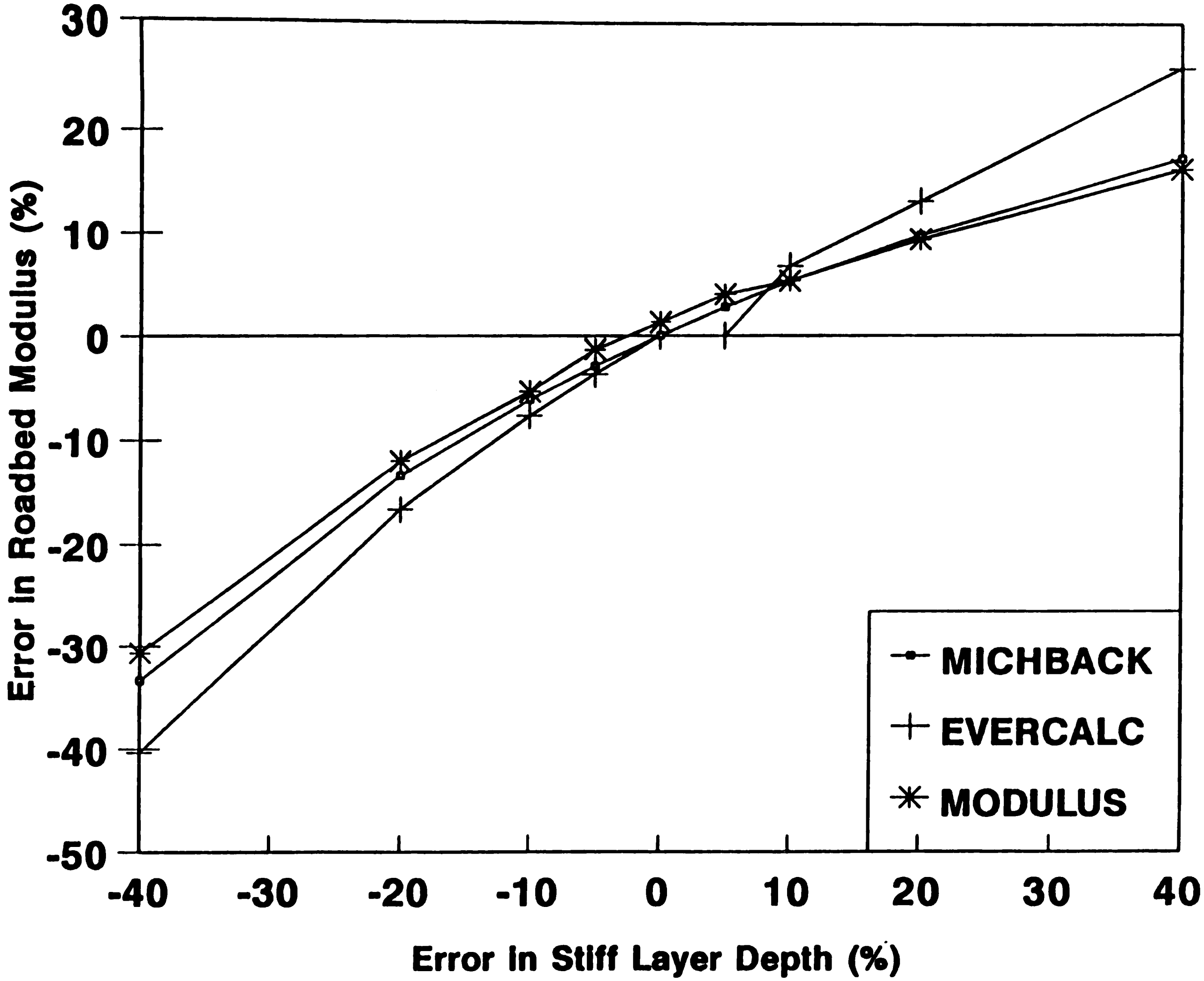

Figure 4.11. Errors in the backcalculated roadbed modulus due to incorrect stiff layer depth specification.

with each other, and the over-estimation of one results in the under-estimation of the other.

4. For all three programs the roadbed modulus was found to be very sensitive to error in the stiff layer depth. Therefore, the base modulus always showed an opposite trend than that of the roadbed compensating for the sensitivity of the adjacent layer. Thus, an over-estimation of the roadbed thickness resulted in the base modulus being under-predicted and the AC modulus being over-predicted, respectively.

4.5.2.2 Effect of Stiff layer Modulus on the Backcalculated Layer Moduli

Generally, the stiff layer modulus is more difficult to predict than the depth. This is especially true when the stiff layer effect is being observed not due to the presence of a hard layer but because of the soil overburden or stress stiffening characteristics of the roadbed soil. In the latter case the roadbed modulus gradually increases with depth, making the prediction of its modulus difficult.

The effect of the modulus of the stiff layer on the backcalculated layer moduli are illustrated in this section. The three layer medium thick pavement (Table 4.1) was used to study this effect, and the depth to stiff layer was varied between 36 and 240-inch. In the first case the correct depth to the stiff layer (i.e. the same as that used to generate the deflection data using CHEVRONX) was used. The data was generated using a value of 1000 ksi for the modulus of the stiff layer, while for the purposes of backcalculation, the modulus was varied between 500 and 7000 ksi (error range of -600% to 50%). The results presented in Table 4.16 indicate that:

1. Within the wide range of error in the stiff layer modulus, the observed error in the backcalculated layer moduli was reasonably small for the shallow stiff layer and almost insignificant for the medium and deep stiff layers.

Table 4.16. The effect of stiff layer modulus on the backcalculated layer moduli (stiff layer depth fixed).

Stiff layer depth	Stiff layer modulus (ksi.)	Error in stiff layer modulus (%)	RMS	Error in backcalculated moduli (%)		
				AC	Base	Roadbed
Shallow (36 in.)	7000	-600	3.7	-5.3	8.3	-4.9
	5000	-400	3.5	-4.9	7.7	-4.6
	1000	0	0.03	0.0	0.0	0.0
	500	50	4.5	6.8	-10.1	6.5
Medium (144 in.)	7000	-600	0.1	-3.2	3.8	-0.8
	5000	-400	0.1	-3.0	3.6	-0.7
	1000	0	0.0	-2.7	0.7	-0.1
	500	50	0.1	2.4	-2.8	0.7
Deep (240 in.)	7000	-600	0.1	-1.8	2.0	-0.4
	5000	-400	0.1	-1.7	1.9	-0.3
	1000	0	0.1	-0.6	0.6	0.0
	500	50	0.03	0.7	-1.0	0.4

2. The RMS error decreases as the stiff layer depth increased for the same degree of error in the stiff layer modulus.

The example above was repeated for all cases, but this time instead of providing the actual stiff layer depth and fixing it MICHBACK was allowed to find the stiff layer depth iteratively along with the layer moduli. The stiff layer modulus was deliberately varied between the above mentioned limits, and the seed value for the stiff layer depth was provided by the regression equations.

The results presented in Table 4.17 indicate:

1. The MICHBACK program compensates for higher values of the stiff layer modulus by reducing its depth keeping the stiffness of the other layers almost the same.
2. In the case of an incorrectly specified stiff layer depth, the backcalculated moduli have a slightly larger error than that when the true depth is specified. This observation is more pertinent to the shallow stiff layer. The reason for this is that as the stiff layer depth decreases, the backcalculated results become more sensitive to the stiff layer properties. At shallow depths, small errors in the backcalculated thickness coupled with the error in the stiff layer modulus can produce moderate errors in the backcalculated layer moduli.
3. The above exercise indicates that the stiff layer modulus has a negligible effect on the backcalculated layer properties, especially when the stiff layer depth is not very shallow.
4. It is further emphasized that the error range for the stiff layer modulus tested in the above exercise was extremely wide. Hence, making a reasonable guess regarding the stiff layer modulus will not have a significant effect on the backcalculated layer moduli. Also interaction between the stiff layer depth and modulus will only make the iterative process more complicated and prone to

Table 4.17. Effect of stiff layer modulus on the backcalculated layer moduli and stiff layer depth.

Stiff layer depth	Stiff layer modulus(ksi)	Error in modulus (%)	RMS	Error in backcalculated properties (%)			
				Stiff layer depth	Moduli		
					AC	Base	Roadbed
Shallow (36 in.)	7000	-600	1.8	10.8	3.0	-8.5	20.7
	5000	-400	1.8	10.3	2.8	-8.0	19.4
	1000	0	1.7	-1.0	0.0	0.0	0.01
	500	50	4.4	-7.7	-0.7	3.5	-13.8
Medium (144 in.)	7000	-600	0.07	1.7	0.4	-0.6	0.32
	5000	-400	0.07	1.6	0.4	-.6	0.32
	1000	0	0.0	0.0	0.0	0.0	0.0
	500	50	0.02	-1.3	-0.1	0.1	-1.
Deep (240 in.)	7000	-600	0.0	-0.2	1.0	-1.1	0.2
	5000	-400	.03	1.3	-0.21	0.1	0.1
	1000	0	0.0	-0.3	-.2.0	0.1	0.1
	500	50	1.3	-5.4	-0.23	0.16	0.1

error.

5. Based on the above observations, no effort has been made in the MICHBACK program to include the stiff layer modulus as an unknown in the backcalculation process.

4.6 CONVERGENCE CHARACTERISTICS

Newton's method is a rapidly convergent and accurate optimization technique. The speed of convergence, in MICHBACK, has been enhanced by the logarithmic transformation of the gradient matrix as explained in Chapter 3. The convergence characteristics have been tested in this section using the deflection data generated by CHEVRONX.

The MODULUS and EVERCALC programs being used for comparison are limited to a maximum of four pavement layers, and therefore no comparison could be made for the five layer example. The EVERCALC program uses the original version of the CHEVRON program for forward calculations, which being less accurate affects the backcalculated results. In order to make the comparison fair for EVERCALC program, the backcalculation was conducted by using theoretical deflection basins generated by both CHEVRONX and CHEVRON. The results where, CHEVRON generated data was used in the backcalculation, are denoted by EC-ALT.

For all examples, surface deflections were rounded to the nearest hundredth of a mil. An improved accuracy was obtained for MICHBACK when the surface deflections were input to a greater precision, especially for the composite pavements. The other two programs do not allow the surface deflections to be input to a precision greater than hundredth of a mil. Although, such a precision is unrealizable in the field, this observation with MICHBACK indicates the sensitivity of the backcalculated results for composite pavements to even small changes in the measured deflections.

4.6.1 Three-Layer Flexible Pavements

The properties of the three layer flexible pavements used in the analysis are listed in Table 4.1. The backcalculated results along with the maximum error in the moduli and the RMS error specified by Equation 3.23 are given in Table 4.18.

The MICHBACK program yields accurate results for all three layers. MODULUS, on the other hand, has comparatively larger error for the base modulus, 4% being the largest. For other layers, the errors are smaller. The EVERCALC program has the largest error of the three programs, mainly because the CHEVRON program is used in the backcalculation algorithm. Also it can be seen that EVERCALC progressively calculates poorer results as the pavement becomes stiffer. This indicates that the difference between the modified and older versions of the CHEVRON program increase for the stiffer pavements. When the deflections generated by the old version of the CHEVRON program is analyzed EVERCALC (EC-ALT) also yields excellent results for three layer pavements (similar to MICHBACK).

4.6.2 Four-Layer Flexible Pavement

The actual properties of the pavement are given in Table 4.2. The backcalculated results (Table 4.19) indicate that as the number of layers in the pavement increases MICHBACK clearly produces better results than the other two programs. EC-ALT results are comparable to those of MICHBACK but the largest error is more than 4% compared to less than 1% for MICHBACK. For pavements with more than three layers the MODULUS program yields a poorer result at least for one of the layers. MICHBACK produced consistently accurate results for all other four layer pavements tested as well.

Table 4.18. Comparison of the results of three programs for a three layer pavement.

Program	Pavement type	Backcalculated modulus (ksi)			Max. error in moduli (%)	RMS error (%)
		AC	Base	Roadbed		
MICHBACK	Thin	497.9	45.0	7.5	0.42	.02
	Medium	499.9	45.0	7.5	0.02	.03
	Thick	501.2	44.6	7.5	0.84	.01
MODULUS	Thin	485.4	45.9	7.5	2.92	.37
	Medium	503.1	44.6	7.5	0.89	.11
	Thick	485.4	46.8	7.5	4.00	.14
EVERCALC	Thin	503.6	44.9	7.5	0.73	.02
	Medium	477.7	46.0	7.5	4.45	.06
	Thick	439.6	58.0	7.5	28.87	.13
EC-ALT	Thin	500.2	44.9	7.5	0.11	.02
	Medium	502.9	44.8	7.5	0.57	.02
	Thick	500.5	45.8	7.5	0.17	.15

Table 4.19. Comparison of the results of three programs for a four layer pavement.

Program	Backcalculated modulus (psi)				Max. error in moduli (%)	RMS error in deflections (%)
	AC	Base	Subase	Roadbed		
MICHBACK	500.1	45.1	14.9	7.5	0.6	.01
MODULUS	544.9	36.0	22.3	7.6	48.7	.16
EVERCALC	476.2	46.3	14.7	7.5	4.8	.09
EC-ALT	495.0	46.3	14.3	7.5	4.6	.06

4.6.3 Four-Layer Composite Pavements

The properties of the two composite pavements analyzed are listed in Table 4.3. One composite pavement (composite 1) had a separation layer between the AC overlay and the PCC slab, whereas the other pavement (composite 2) had no such layer.

The results shown in (Tables 4.20 and 4.21) indicate that while MICHBACK converges reasonably well for both composite pavements, the other two programs yielded considerable errors especially for the composite 2. It has been pointed by various researchers that for composite pavements, the modulus of the layer immediately under the slab is the most difficult to predict unless the lower layer is the roadbed soil. For this pavement the maximum error for MICHBACK is 8% compared to about 61% for MODULUS and 131% for EVERCALC (using the data generated by old CHEVRON (EC-ALT)). This indicates that MODULUS and EVERCALC do not produce accurate results for composite pavements. MICHBACK has also shown some problems in predicting the modulus of the layer immediately under the slab. However the magnitude of the error is comparatively smaller. For all other composite pavement examples, the MICHBACK produced better results than both EVERCALC and MODULUS programs.

4.6.4 Three-Layer Pavements Over a Stiff layer

The medium thickness three layer flexible pavement (Table 4.1) was underlain by a stiff layer at two different depths, 36 and 240 inch. All other parameters were the same as for the three layer flexible medium thickness pavement analyzed without the stiff layer. The results are presented in Table 4.22. It can be seen that the results of MICHBACK and EVERCALC (EC-ALT) are comparable and better than those of MODULUS.

Table 4.20. Comparison of the backcalculated results for a composite pavement section.

Program	Backcalculated modulus (ksi)				Max. error in moduli (%)	RMS error (%)
	AC	Slab	Base	Roadbed		
MICHBACK	499.4	4516.3	23.0	7.5	8.0	0.01
MODULUS	527.7	4471.1	9.8	7.6	60.8	0.07
EVERCALC	1582.2	2297.1	13.2	7.5	216.4	1.53
EC-ALT	494.7	4217.1	57.8	7.4	131.1	0.06

Table 4.21. Comparison of the backcalculated results for a composite pavement section consisting of a granular separation layer.

Program	Backcalculated modulus (psi)				Max. error in moduli (%)	RMS error (%)
	AC	Base	PCC Slab	Roadbed		
MICHBACK	499.4	24.8	4472.9	7.5	0.6	0.03
MODULUS	492.1	25.4	4402.5	7.5	2.2	0.08
EVERCALC	622.7	27.7	3959.7	7.5	24.5	0.75
EC-ALT	491.5	25.5	4412.6	7.5	2.1	0.11

Table 4.22. Comparison of the results of three programs for a three layer pavement over stiff layer.

Program	Stiff layer location	Backcalculated modulus (psi)			Max. error in moduli (%)	RMS error in deflections (%)
		AC	Base	Roadbed		
MICHBACK	Deep	501.7	44.9	7.5	0.34	0.04
	Shallow	499.8	44.9	7.5	0.19	0.09
MODULUS	Deep	502.0	44.8	7.5	0.44	0.19
	Shallow	508.9	43.9	7.7	3.55	0.07
EVERCALC	Deep	796.8	31.2	7.5	59.36	1.15
	Shallow	598.0	40.3	7.6	19.60	0.60
EC-ALT	Deep	498.3	45.2	7.5	0.41	0.02
	Shallow	501.4	44.8	7.5	0.34	0.04

4.6.5 Five-Layer Flexible Pavement

For the five layer pavement, the results of MICHBACK are presented in Table 4.23. The other two programs cannot analyze a five layer pavement. The five layer pavement configuration used for the analysis is specified in Table 4.4. It can be seen that the maximum error produced by MICHBACK for the modulus of any layer is less than 1%. It indicates that, unlike the other programs, there is no decrease in the accuracy of the backcalculated results with the increase in the number of pavement layers for MICHBACK.

4.6.6 Performance Comparison

Performance comparison has mostly been restricted to the examples presented in this section only and to MICHBACK and EVERCALC programs because MODULUS is not an iterative program. For MODULUS, the number of deflection bowls generated depends upon the number of layers in the pavement as well as on the range of the moduli provided by the analyst. The range in turn affects the backcalculated results and a closer range was provided to keep the convergence performance of the program compatible. The MODULUS program has, therefore, not been included in the performance comparison.

The results of the comparison are provided in Table 4.24 and Figure 4.12. The number of calls for EVERCALC are based on deflections generated by the old CHEVRON (the number of calls for data generated by CHEVRONX were a little higher). The designation MICHBACK(N) refers to the use of arithmetic scale, MICHBACK(M) represents the results for the modified Newton method and MICHBACK(L) refers to the use of the logarithmic scale together with the modified Newton method, respectively.

The results indicate that after logarithmic transformation, the performance of MICHBACK is somewhat better than that of EVERCALC. The effect of logarithmic

Table 4.23. The MICHBACK backcalculation results for a five layer pavement.

Backcalculated modulus (psi)					Max. error in moduli (%)	RMS error in deflections (%)
AC	Treated base	Base	Subbase	Roadbed		
497.0	100.4	45.1	14.9	7.5	-0.96	0.02

Table 4.24. Comparison of the performance of MICHBACK and EVERCALC programs.

Pavement type	Program	Number of times operation was performed		
		CHEVRON called	Gradient computed	Iterations
Three layer (Thin)	MICHBACK(N)	22	5	5
	MICHBACK(M)	19	3	8
	MICHBACK(L)	14	2	6
	EVERCALC	17	4	4
Three layer (Medium)	MICHBACK(N)	26	6	6
	MICHBACK(M)	24	4	10
	MICHBACK(L)	13	2	5
	EVERCALC	17	4	4
Three layer (Thick)	MICHBACK(N)	25	5	5
	MICHBACK(M)	20	3	5
	MICHBACK(L)	14	2	6
	EVERCALC	17	4	4
Four layer	MICHBACK(N)	37	7	7
	MICHBACK(M)	23	3	9
	MICHBACK(L)	16	2	6
	EVERCALC	21	4	4
Composite	MICHBACK(N)	32	6	6
	MICHBACK(M)	28	4	10
	MICHBACK(L)	14	2	4
	EVERCALC	21	4	4
Composite (with granular separation layer)	MICHBACK(N)	32	6	6
	MICHBACK(M)	28	4	10
	MICHBACK(L)	21	3	7
	EVERCALC	21	4	4
Five Layer	MICHBACK(N)	32	5	5
	MICHBACK(M)	26	3	9
	MICHBACK(L)	18	2	6
Three Layer (Deep Stiff Layer)	MICHBACK(N)	36	7	7
	MICHABCK(M)	24	4	10
	MICHBACK(L)	12	2	5
	EVERCALC	17	4	4

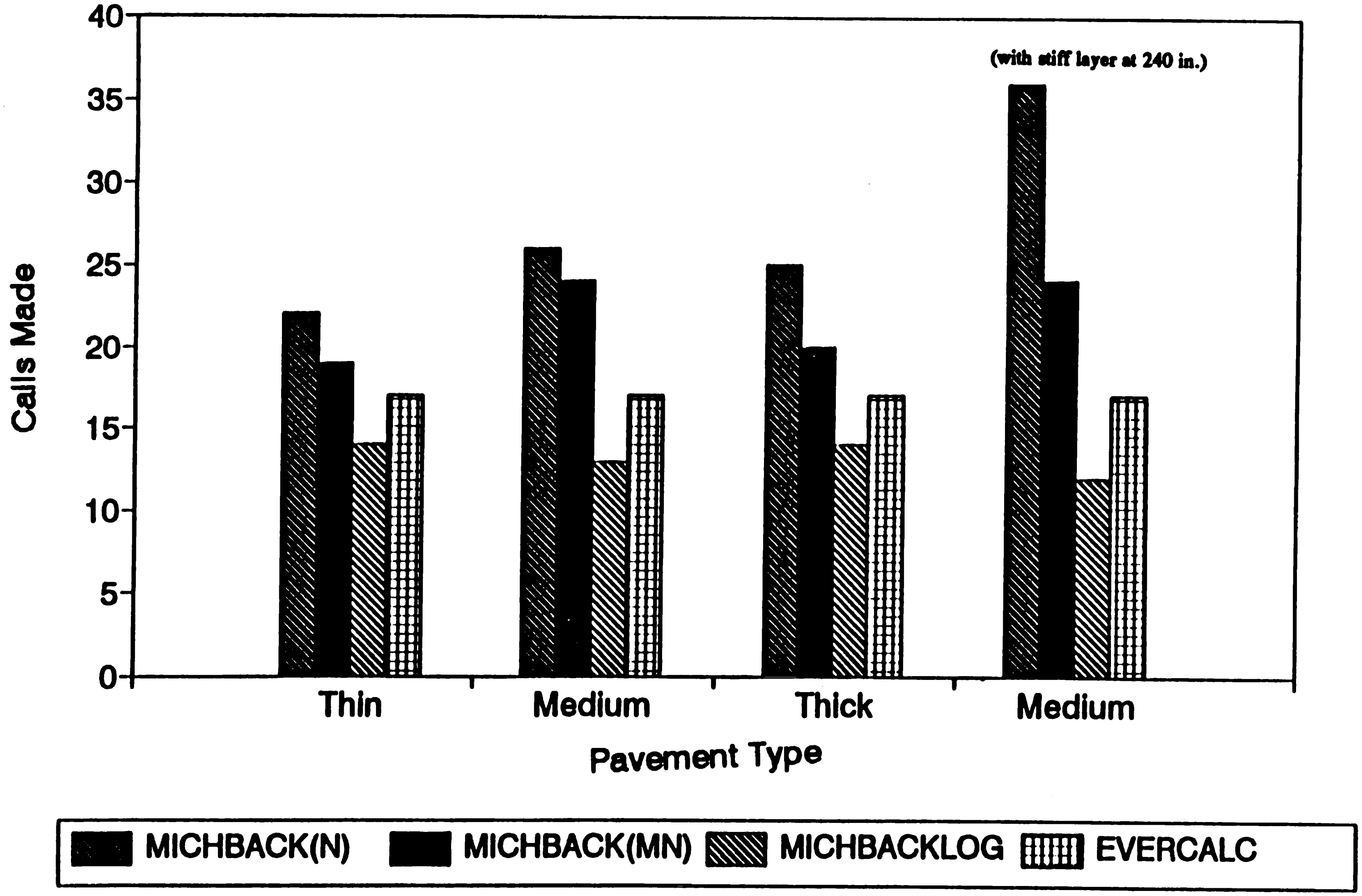

Figure 4.12. Comparison of the performance of MICHBACK and EVERCALC programs.

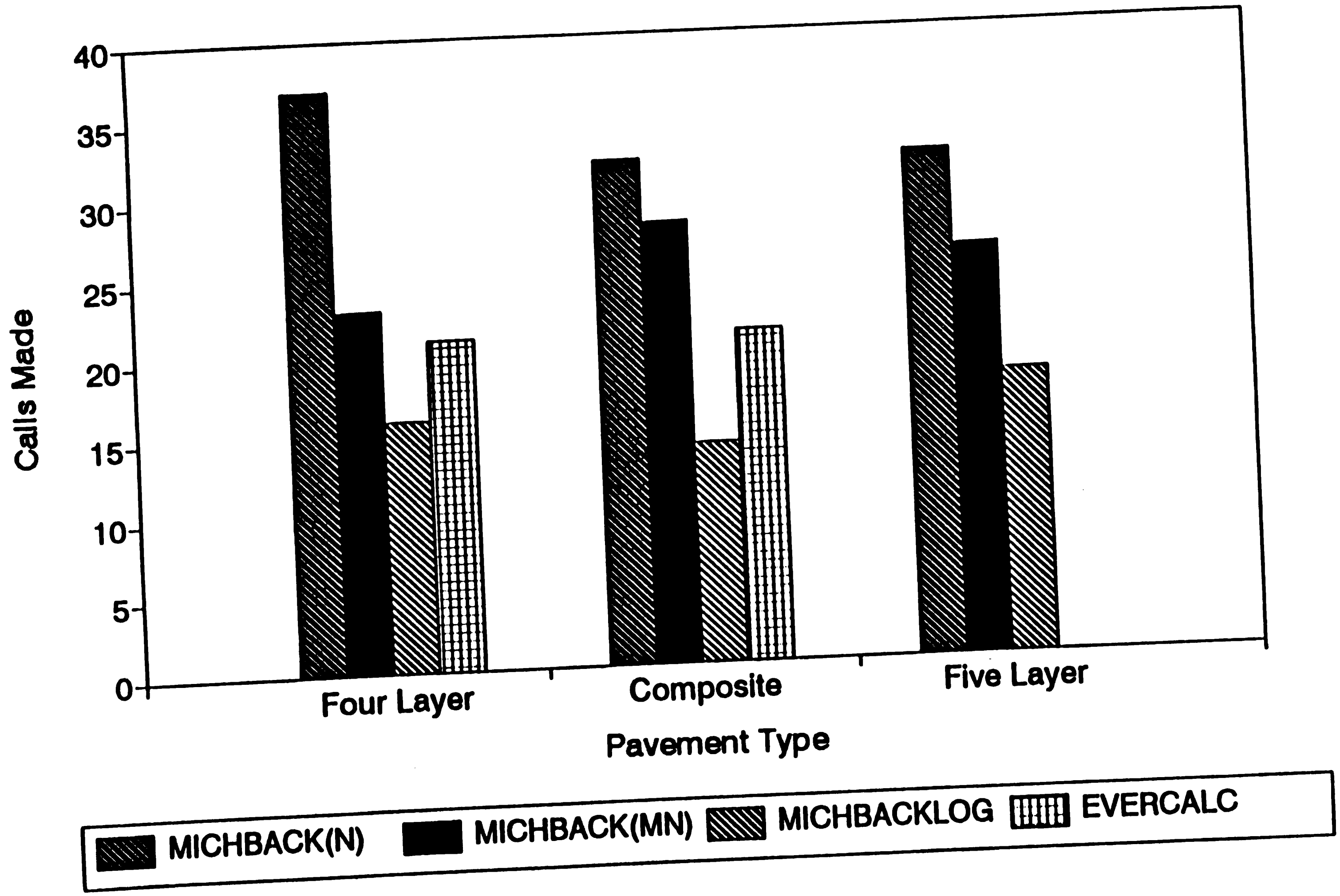

Figure 4.12. Comparison of the performance of MICHBACK and EVERCALC programs (continued).

transformation becomes more significant as the nature of the problem becomes more complicated.

4.7 UNIQUENESS OF THE BACKCALCULATED RESULTS

Many backcalculation programs suffer from the disadvantage that the backcalculated results are highly dependent on the seed modulus values provided by the user. The farther the guess is from the true values, the higher are the chances of converging to a wrong solution. This is especially true for the methods which seek the minimization of an objective function where the chances of converging to a local minimum are higher. The convergence of Newton's method in general is also not global, but is problem dependent. For many complicated problems, the solutions are reported to be governed by the starting values. However, backcalculation of layer properties from FWD deflection data appears to be a well behaved problem, especially for flexible pavements, and the results obtained using Newton's method seem to be independent of the starting values. Many researchers (Sivaneswaran, 1991) have also pointed out, though not with absolute certainty, that the criterion function constructed by minimizing the squared difference between measured and calculated deflections is convex in shape and hence will have a unique minimum.

For the flexible pavement examples, it was observed that the results obtained by MICHBACK are independent of the seed values. The results for three, four, and five layer pavements are presented in Tables 4.25 through 4.27, respectively. The deflection data was generated by using the pavement cross sections introduced earlier but the layer moduli were changed as shown in the respective tables. The seed moduli were chosen randomly but far from the true values to test the capability of the program to converge to the correct solution even for unreasonable seed values.

It can be seen from these tables that the results obtained by MICHBACK are not affected by the seed moduli at all. The only difference is in the number of

Table 4.25. Uniqueness of the MICHBACK solution for a three layer flexible pavement.

Ex. No.	Actual modulus (ksi)			Seed modulus (ksi)			Bbackcalculated modulus (ksi)		
	E1	E2	E3	E1	E2	E3	E1	E2	E3
1	300	45	7.5	1000	100	100	301.4	44.83	7.5
				1	1	1	301.4	44.83	7.5
2	500	75	15	1000	500	100	499.2	75.10	15.0
				1	1	1	499.2	75.10	15.0
3	800	45	7.5	2000	100	100	801.3	44.91	7.5
				1	1	1	801.3	44.91	7.5

Table 4.26. Uniqueness of the MICHBACK solution for a four layer flexible pavement.

Ex. No.	Actual modulus (ksi)				Seed modulus (ksi)				Backcalculated modulus (ksi)			
	E1	E2	E3	E4	E1	E2	E3	E4	E1	E2	E3	E4
1	500	45	15	7.5	2000	100	100	100	500.1	45.1	14.9	7.5
					1	1	1	1	500.1	55.1	14.9	7.5
2	800	75	25	15	2000	500	50	50	803.0	74.6	25.3	15.0
					1	1	1	1	803.1	74.6	25.3	15.0

Table 4.27. Uniqueness of the MICHBACK solution for a five layer flexible pavement.

Seed Modulus (ksi)					Backcalculated Modulus (ksi)				
E1	E2	E3	E4	E5	E1	E2	E3	E4	E5
1	1	1	1	1	511.0	96.1	48.1	13.3	7.52
100	1000	15	5	1.5	510.8	96.2	48.0	13.3	7.52

iterations required to meet the given convergence criterion. The upper and lower limits for all the layer moduli were set at 10,000,000 and 1 psi respectively. This capability has been provided in the MICHBACK program so that the analyst can specify bounds for the backcalculated moduli. This further ensures that the solution should remain within the expected or even known moduli ranges for the pavement materials. In the case of flexible pavements this capability was not invoked.

For the composite pavements convergence to the correct results from excessively erroneous seed values can only be achieved by intelligent use of the bounding values. However, even for composite pavements, convergence from reasonable seed values (expected even from a novice) is not a problem. Unreasonable seed values were used only to check the robustness of the program. This problem has been partially addressed in the program by automatically setting a lower bound of 1 million on the backcalculated results whenever the analyst recognizes the pavement to be composite and not built on a rubbled slab. No upper bound is required to be set. The pre-specified lower limit assures convergence to the correct results even from unrealistic seed values.

4.8 EFFECT OF INACCURACIES IN DEFLECTIONS AT SIMULATED SENSOR LOCATIONS ON BACKCALCULATED RESULTS

The accuracy of each sensor of FWD is about ± 2% of the sensor's range. Hence, similar range in the accuracy of the backcalculated results should also be expected. Also it is a general belief that since the deflections at the outer sensors are comparatively smaller, inaccuracies at these sensors have a larger contribution towards the overall error especially for the lower layers. In this section, the effect of inaccuracies in deflections at different sensor locations on the backcalculated layer moduli are examined. It was feared that logarithmic transformation may make the backcalculated results more sensitive to the inaccuracies in the measured deflections.

Therefore, a comparison has also been made between the sensitivity of the backcalculated results to the surface deflection for both arithmetic and logarithmic scales.

The thin, medium, and thick three layer flexible pavements (Table 4.1) were used to study the sensitivity of the backcalculated results to the surface deflections. The medium three layer pavement was used to examine and compare the sensitivity of the backcalculated results when working in logarithmic scale. The surface deflections were generated using the CHEVRONX program. An error of $\pm$ 2% was deliberately introduced in each sensor deflection individually.

The results for the arithmetic and logarithmic scales are presented in Tables 4.28 and 4.29, respectively. The results indicate:

1. Inaccuracies in the deflections of these locations close to the load (for fixed percent error) induce larger error in the backcalculated layer moduli than the other locations.
2. For the same absolute magnitude of the error, negative errors induce greater errors in the backcalculated results than the positive ones.
3. For almost all locations, the AC modulus is the most affected followed by the base modulus. The roadbed modulus is least affected by inaccuracies in the surface deflections at any locations. Although errors in the last few locations do affect the roadbed modulus slightly more than errors in the others, the maximum error in the roadbed modulus remains well below 1.5%.
4. The AC modulus of thin layers is affected the most by deflection inaccuracies. This is because pavements with a thick AC layer can compensate for the erroneous deflections by smaller changes in the AC modulus, whereas to redress the same amount of error, the AC modulus must undergo a bigger change for pavements with a thin AC layer.
5. The base modulus of a pavement with a thick AC layer is affected more than

Table 4.28. Percent errors in the backcalculated layer moduli due to errors in deflections - arithmetic scale.

Location No.	Error in deflections (%)	Error in Backcalculated Moduli (%)								
		AC			Base			Roadbed		
		Thin	Med	Thick	Thin	Med	Thick	Thin	Med	Thick
1	+2	-40.4	-24.4	-19.8	5.5	19.0	36.8	0.7	.4	-0.6
	-2	54.4	29.8	24.6	-5.2	-17.2	-38.7	-0.3	.5	1.0
2	+2	48.6	10.3	4.6	-10.5	-12.3	-16.6	0.1	0	0.8
	-2	-64.1	-11.0	-4.7	23.7	14.0	17.9	1.2	-.6	-0.8
3	+2	20.2	0.7	8.6	-3.6	-4.4	-21.9	-0.6	1.1	0.7
	-2	-14.0	-13.6	-8.9	2.7	14.0	24.5	0.6	-.3	-0.7
4	+2	-5.7	7.4	7.9	5.4	-5.0	-16.5	-0.7	-.4	0.2
	-2	10.5	-7.0	-7.6	-3.5	4.8	16.8	0.8	.4	-0.1
5	+2	-12.3	0.4	4.4	4.2	2.3	-6.3	-0.6	-.8	-0.5
	-2	12.1	-0.3	-0.2	-3.7	-2.3	0.2	0.6	-.3	0.0
6	+2	-6.0	-4.8	-1.3	2.2	7.0	9.3	-0.4	-.9	-1.2
	-2	6.0	4.8	1.1	-2.0	6.7	-8.6	0.3	1	1.2
7	+2	-0.9	-3.2	-4.2	0.5	4.1	15.0	-0.1	-.5	-1.2
	-2	1.1	3.1	4.1	-0.4	-4.0	-14.1	0.1	.5	1.2

Table 4.29. Percent errors in the backcalculated layer moduli due to errors in deflections - logarithmic scale.

Location No.	Error in deflections (%)	Error in AC modulus (%)	Error in base modulus (%)	Error in roadbed modulus (%)
1	+2	-20.52	11.91	-0.04
	-2	23.38	-11.9	0.15
2	+2	2.29	-4.72	0.09
	-2	-1.68	4.23	-0.04
3	+2	9.96	-9.13	0.09
	-2	-9.16	8.99	0.0
4	+2	11.84	-9.09	-0.07
	-2	-10.72	8.78	0.15
5	+2	8.5	-5.65	-0.27
	-2	-7.5	5.0	0.35
6	+2	-0.5	2.56	-0.61
	-2	1.24	-3.34	0.69
7	+2	-9.75	11.05	-0.89
	-2	10.7	-11.6	1.0

that of a pavement with a thin AC layer. This is probably because the inaccuracies in the AC modulus for a pavement with a thick AC layer affects the overall strength of the pavement more than for a thin pavement. Therefore, in trying to adjust the overall stiffness the base modulus for thick AC pavement is affected more.

Results of a similar test for the medium thick pavement performed after the logarithmic transformation are presented in Table 4.29. It can be observed that this change does not significantly affect the sensitivity of the backcalculated results to the surface deflections. Except for the last deflection location, for most other sensors the error in the backcalculated layer moduli is slightly less than those arising from the arithmetic scale.

The above exercise was repeated by fixing the induced difference in the measured deflections at $\pm$ 0.5 mils for all the sensors. The results are presented in Table 4.30. The errors in the backcalculated moduli are a little higher than in the case where the errors were induced as percentages of the measured deflections. This is because of the fact that the $\pm$ 0.5 mil error is larger than the 2% error.

4.9 EFFECT OF POISSON'S RATIO ON THE BACKCALCULATED LAYER MODULI

Several studies were conducted to assess the effects of Poisson's ratios of the various pavement layers on the calculated deflections. Pichumani (1972) concluded that Poisson's ratio of only the roadbed soil has some appreciable effect on the surface deflections. Variations in the Poisson's ratios of the other layers were found to have little effect on the surface deflections. Another study, conducted at the University of Utah (Hou, 1977), suggested that for three-layer pavements, variations in Poisson's ratio of each layer including the roadbed soil from 0.25 to 0.45 have no

Table 4.30. Percentage error in backcalculated layer moduli due to ± 0.5 mil error in deflections-log scale.

Sensor No.	Error	AC	Base	Roadbed
1	+0.5	-25.8	17.7	-0.2
	-0.5	32.7	-18.6	0.4
2	+0.5	19.5	-19.3	0.4
	-0.5	-21.7	25.3	-0.6
3	+0.5	12.4	-10.9	0.1
	-0.5	-11.7	11.2	-0.1
4	+0.5	9.9	-7.1	-0.4
	-0.5	-8.3	5.9	0.5
5	+0.5	3.6	-0.2	-0.9
	-0.5	-3.0	-0.5	1.0
6	+0.5	7.2	12.4	-1.9
	-0.5	7.4	-12.2	2.0
7	+0.5	-15.8	24.9	-3.0
	-0.5	25.3	-29.6	3.8

significant effect on the surface deflections. These and other similar findings have led to a general consensus that since Poisson's ratios of the paving layers have little influence on the surface deflections, their effect on the backcalculated layer moduli must also be negligible. No study appears to have investigated the direct effect of Poisson's ratios on the backcalculated layer moduli. In this study, this issue was investigated and the results are presented in this section.

The deflection basins used in the previous sections were generated by using constant Poisson's ratios of 0.35, 0.4, 0.45, and 0.45 for the AC, base, and subbase layers and for the roadbed soil, respectively. To assess the effect of Poisson's ratio on the backcalculated layer moduli, the value of Poisson's ratio of one layer at a time was varied by $\pm$ 0.05 from the true value. The results of this analysis for the medium thick flexible pavement of Table 4.1 are listed in Table 4.31. Results of similar tests for composit pavement (Table 4.3) are presented in Table 4.32. Examination of the results indicate that:

1. An error of $\pm$ .05 in Poisson's ratio of the AC layer introduces about a 4% error in the backcalculated modulus of the AC layer. The errors in the moduli of the other two layers are comparatively small and the roadbed soil modulus is the least affected.
2. As the stiffness of the AC layer increases so does the effect of Poisson's ratio.
3. Poisson's ratio of the base layer has some impact on its modulus and the least effect on the backcalculated results of the other layers.
4. An error of $\pm$ 0.05 in Poisson's ratio of the roadbed soil has the largest effects on the backcalculated results. It introduces a 10 % error in the AC modulus, a 9 % error in the base modulus, and a small error in the roadbed soil modulus.
5. For composite pavements, the backcalculated layer moduli appear to be very sensitive to errors in the value of Poisson's ratio. For example, an error of

Table 4.31. Percent errors in the backcalculated layer moduli due to error in Poison's ratio for flexible pavements.

Pavement type	Layer	Error in Poisson's ratio	Error in backcalculated moduli (%)		
			AC	Base	Roadbed
Three layer medium	AC	+.05	-3.9	-1.06	-0.01
		-.05	3.96	0.39	0.07
	Base	+.05	0.61	1.85	-0.01
		-.05	-.03	-2.63	0.07
	Roadbed	+.05	-9.3	10.90	-3.20
		-.05	8.94	-13.95	1.89
Three layer thick	AC	+.05	-4.26	-0.84	-0.01
		-.05	3.68	0.52	0.07
	Base	+.05	0.14	1.68	0.03
		-.05	-0.14	-2.10	0.01
	Roadbed	+.05	-2.92	12.95	-3.12
		-.05	0.14	1.68	0.03

Table 4.32. Percent errors in the backcalculated layer moduli due to error in Poison's ratio for composite pavement.

Layer	Error in Poisson,s ratio	Error in backcalculated layer moduli (%)			
		AC	Base (slab)	Subbase	Roadbed
AC	+.05	-8.53	-20.60	344.10	0.50
	-.05	9.60	7.89	-82.50	2.44
PCC Slab (base)	+.05	0.18	-1.02	-24.02	0.15
	-.05	-0.56	1.54	25.59	-0.12
Subbase	+.05	-0.15	0.31	1.73	-0.08
	-.05	-0.14	0.35	-5.40	0.09
Roadbed	+.05	0.31	-2.81	87.60	-4.0
	-.05	-0.50	2.40	-44.16	2.49

300 % in the subbase layer modulus (the layer immediately beneath the PCC slab) was observed. The errors in the modulus values of the other layers were less than 20 %.

6. The effects of Poisson's ratios on the calculated deflection basins is negligible.

The above observations do not negate the past findings, they only emphasize that small changes in the deflection basins can significantly affect the values of the backcalculated layer moduli.

It should be noted that the results presented above are pertinent to the MICHBACK program (which uses the CHEVRONX as the forward analysis program). The sensitivity of the various backcalculation techniques to Poisson's ratios or to small changes in the deflection basins may vary. One point can be made here is that, given the present state of the equipment (deflections at only seven sensor locations are measured), and the number of unknowns that need to be estimated, the inclusion of Poisson's ratios of the various layers in the pool of unknown to be calculated will only make the backcalculation more complicated and prone to higher errors. Furthermore, the intention of the analysis presented above is to warn the analyst that reasonable ranges of Poisson's ratios of the different paving materials must be known and must be used cautiously.

4.10 COMPARISON OF DEFLECTION OUTPUT OF DIFFERENT ELASTIC LAYER PROGRAMS

The advent of fast micro-computers, made automated backcalculation of layer moduli possible. Most backcalculation programs make use of a multi-layer elastic routine as a forward analysis programs in one way or another. Hence, the backcalculated results are not only affected by the backcalculation technique, but also by the precision of the forward analysis routine. Surface deflections are the common output used from these routines in the backcalculation process. The accuracy of the multi-layer elastic routines is generally established by comparing the deflection results to those of an established and reputed one such as the "BISAR".

MICHBACK initially used CHEVRON as the forward analysis program. At the onset of this study, the analysis of deflection data received from various agencies have indicated that differences between the deflections obtained from the CHEVRON program and those from the BISAR program exist. Consequently, a corrected version "CHEVRONX" of the CHEVRON program was obtained from Dr. Lynne Irwin at Cornell University and it was embedded in the current version of MICHBACK. It should be noted that, originally, results of the CHEVRON program was validated by Lee, et. al. (1988). They concluded that the output of the CHEVRON program differs slightly from that of the BISAR program for flexible pavements and that the program should not be used for the analysis of stiff flexible and composite pavements.

In this section surface deflections from four elastic layer programs, (CHEVRON, ELSYM5, WESLEA, and CHEVRONX) have been compared with those of BISAR. Once again, CHEVRONX is the enhanced version of the CHEVRON program and it is used in MICHBACK. The five pavement sections listed in Tables 4.1 through 4.4 are used in this comparison. Further, an AC modulus of 800 ksi (instead of 500 ksi) was used for the medium thick three-layer flexible pavement. The stiff layer depth for the pavements was set at 144-inch to represent the presence of bedrock.

4.10.1 Comparison of Deflection Output

The deflections from all the five programs are presented in Table 4.33. The deflections were rounded to the nearest hundredth of a mils to represent the FWD readings. It can be seen that the outputs of BISAR, WESLEA and CHEVRONX are essentially the same. Table 4.34 provides a list of the percent differences in the

Table 4.33. Deflections from different elastic layer programs.

Pavement Type	Program	Deflections (mils)						
		d0	d1	d2	d3	d4	d5	d6
Three Layer Medium	BISAR	24.400	20.900	18.600	15.600	13.100	9.500	5.530
	CHEVRONX	24.370	20.870	18.570	15.580	13.120	9.498	5.534
	CHEVRON	24.260	20.870	18.570	15.580	13.120	9.498	5.534
	ELSYM5	24.260	20.870	18.570	15.580	13.120	9.497	5.534
	WESLEA	24.369	20.872	18.570	15.578	13.123	9.498	5.533
Three Layer Medium Stiff	BISAR	22.200	19.600	17.700	15.100	12.900	9.500	5.600
	CHEVRONX	22.200	19.570	17.700	15.120	12.900	9.497	5.595
	CHEVRON	22.070	19.560	17.700	15.120	12.900	9.497	5.595
	ELSYM5	22.070	19.560	17.700	15.120	12.900	9.497	5.595
	WESLEA	22.203	19.572	17.700	15.121	12.905	9.497	5.599
Three Layer Thick	BISAR	16.600	14.900	13.900	12.500	11.200	8.980	5.810
	CHEVRONX	16.650	14.900	13.910	12.530	11.230	8.977	5.813
	CHEVRON	16.470	14.790	13.920	12.530	11.230	8.977	5.813
	ELSYM5	16.470	14.790	13.920	12.530	11.230	8.977	5.813
	WESLEA	16.649	14.899	13.913	12.530	11.234	8.978	5.813
Four Layer	BISAR	19.600	17.000	15.400	13.300	11.600	8.930	5.660
	CHEVRONX	19.630	17.030	15.420	13.330	11.590	8.929	5.662
	CHEVRON	19.530	16.960	15.420	13.330	11.590	8.929	5.662
	ELSYM5	19.530	16.960	15.420	13.330	11.590	8.929	5.662
	WESLEA	19.634	17.032	15.421	13.331	11.593	8.929	5.658
Three Layer With Stiff Layer	BISAR	20.900	17.400	15.100	12.100	9.710	6.140	2.340
	CHEVRONX	20.910	17.420	15.120	12.150	9.709	6.140	2.345
	CHEVRON	20.310	16.700	14.950	12.240	9.739	6.134	2.345
	ELSYM5	20.310	16.700	14.950	12.240	9.739	6.134	2.345
	WESLEA	20.910	17.420	15.120	12.150	9.709	6.140	2.345
Composite Stiff	BISAR	9.480	8.800	8.600	8.270	7.890	7.080	5.520
	CHEVRONX	9.479	8.800	8.599	8.268	7.890	7.080	5.521
	CHEVRON	10.040	8.766	8.479	8.272	7.896	7.090	5.520
	ELSYM5	10.040	8.766	8.479	8.272	7.896	7.090	5.520
	WESLEA	9.479	8.800	8.599	8.268	7.890	7.080	5.521

Table 4.34. Percent difference in the deflections of different elastic layer programs.

Pavement Type	Program	Difference in Deflections (%)						
		d0	d1	d2	d3	d4	d5	d6
Three Layer Medium	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	-0.123	-0.144	-0.161	-0.128	0.153	-0.021	0.072
	CHEVRON	-0.574	-0.144	-0.161	-0.128	0.153	-0.021	0.072
	ELSYM5	-0.574	-0.144	-0.161	-0.128	0.153	-0.032	0.072
	WESLEA	-0.128	-0.133	-0.162	-0.140	0.173	-0.026	0.060
Three Layer Medium Stiff	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	0.000	-0.153	0.000	0.132	0.000	-0.032	-0.089
	CHEVRON	-0.586	-0.204	0.000	0.132	0.000	-0.032	-0.089
	ELSYM5	-0.586	-0.204	0.000	0.132	0.000	-0.032	-0.089
	WESLEA	0.013	-0.141	0.000	0.139	0.036	-0.028	-0.010
Three Layer Thick	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	0.301	0.000	0.072	0.240	0.268	-0.033	0.052
	CHEVRON	-0.783	-0.738	0.144	0.240	0.268	-0.033	0.052
	ELSYM5	-0.783	-0.738	0.144	0.240	0.268	-0.033	0.052
	WESLEA	0.293	-0.008	0.094	0.238	0.299	-0.023	0.049
Four Layer	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	0.153	0.176	0.130	0.226	-0.086	-0.011	0.035
	CHEVRON	-0.357	-0.235	0.130	0.226	-0.086	-0.011	0.035
	ELSYM5	-0.357	-0.235	0.130	0.226	-0.086	-0.011	0.035
	WESLEA	0.172	0.191	0.135	0.232	-0.057	-0.007	-0.028
Three Layer With Stiff Layer	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	0.048	0.115	0.132	0.413	-0.010	0.000	0.214
	CHEVRON	-2.823	-4.023	-0.993	1.157	0.299	-0.098	0.214
	ELSYM5	-2.823	-4.023	-0.993	1.157	0.299	-0.098	0.214
	WESLEA	0.048	0.115	0.132	0.413	-0.010	0.000	0.214
Composite Stiff	BISAR	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRONX	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	CHEVRON	5.907	-0.386	-1.407	0.024	0.076	0.141	0.000
	ELSYM5	5.907	-0.386	-1.407	0.024	0.076	0.141	0.000
	WESLEA	0.000	0.000	0.000	0.000	0.000	0.000	0.000

deflection at each sensor location relative to the BISAR calculated deflection. It can be seen that:

1. Except for rounding-off precision, there are no significant differences in the outputs of the CHEVRONX, WESLEA, and BISAR programs.
2. The deflection outputs of the CHEVRON and ELSYM5 programs are the same. However, the following observations can be made relative to the differences between these two outputs and that of the BISAR program:
    a) The differences are more pronounced at the first two sensor locations and negligible at the outer sensors.
    b) The differences increase as the stiffness of the pavement increases. For the medium thick pavements with a stiff layer, the differences at all sensor locations are higher than those for the same pavement without a stiff layer. The maximum difference of 4% is observed at the second sensor location. For flexible pavements, the CHEVRON deflections are generally lower than those of BISAR.
    c) For the composite pavements, significant differences were found between the CHEVRON and the BISAR deflections.

4.10.2 Comparison of Backcalculated Results

The deflections from the CHEVRONX and CHEVRON programs were used in MICHBACK to backcalculate the layer moduli. The results are presented in Table 4.35. It can be seen that:

1. The backcalculated results are appreciably different for the two sets of generated data. The difference increases as the overall stiffness of the pavement increases.
2. For the flexible pavement with a stiff layer, the maximum error is about 34%.
3. For the composite pavement, the results are quite erroneous, with a maximum

Table 4.35. Backcalculated results of MICHBACK for deflection data generated by different elastic layer programs.

Pavement type	Data generated by program	Error in backcalculated layer moduli (%)				RMS error
		AC	Base	Subbase	Roadbed	
Three layer medium	CHEVRON	5.39	-3.05	-	0.05	0.05
	CHEVRONX	-.02	-0.07	-	0.0	0.03
Three layer thick	CHEVRON	11.48	-16.1	-	0.2	0.15
	CHEVRONX	.25	-0.84	-	0.04	0.01
Three layer medium stiff	CHEVRON	5.74	-3.71	-	-0.01	0.09
	CHEVRONX	-.28	0.69	-	-0.06	0.02
Three layer with stiff layer (144 in.)	CHEVRON	34.05	-12.01	-	0.2	1.04
	CHEVRONX	.43	-0.44	-	0.04	0.03
Four layer	CHEVRON	4.26	-0.48	-2.83	0.09	0.08
	CHEVRONX	.02	0.22	-.05	0.03	0.03
Composite	CHEVRON	66.1	77.13	-99.4	231.7	0.01
	CHEVRONX	-.14	0.34	-7.9	0.07	0.89

error of about 231%.

The scenario presented in this and in the previous section demonstrates that the results of the backcalculation are, in general, affected by the accuracy of the employed forward analysis program. Based on these results, the use of the CHEVRON and the ELSYM5 programs for the backcalculation of layer moduli of any pavement type is not recommended.

4.11 COMPARISON OF MICHBACK RESULTS WITH SHRP STUDY

In the previous section, the backcalculated results of MICHBACK were compared with two leading programs and various performance aspects were highlighted. In this section, additional comparison of the MICHBACK results with three programs (MODCOMP, MODULUS, and WESDEF) is presented. In this comparison, the pavement cross sections and the deflection basins that reported by Rada, et al., 1992 and listed in Table 4.36 were used. The true values of the layer moduli and those backcalculated by using the three programs are listed in Table 4.37 (Rada, et al., 1992). Table 4.38 provides a list of the layer moduli of the first six pavement sections of Table 4.37 that were obtained by using the MICHBACK program. Table 4.39 provides a summary of the error in each layer modulus as well as the accumulated absolute error in the backcalculated results for all four programs.

It should be noted that the errors the MODCOMP, MODULUS, and WESDEF programs were obtained from Rada et al. study (1992). The accumulated absolute errors of the four programs for the six pavement sections are shown in Figure 4.13. It can be seen that the MICHBACK results have consistently the lower cumulative error.

Table 4.40 provides a list of minimum, maximum, average, and standard deviations of the accumulated absolute errors in the moduli values of the six pavement sections for the four programs. It can be clearly seen that the results of MICHBACK are much more consistent than those of the other programs. This comparison of the

Table 4.36. Deflection and cross sectional data for nine test sections (after Rada, et al., 1992).

ID	Layer	Material	Thickness (inches)	Surface Deflection (mils) r=0"	r=8"	r=12"	r=18"	r=24"	r=36"	r=60"
1	1	Asphalt Concrete	3	32.90	23.20	17.80	12.60	9.51	6.15	3.57
	2	Granular Base	6							
	3	Subgrade								
2	1	Asphalt Concrete	6	30.10	24.50	21.70	18.50	15.90	12.20	7.73
	2	Granular Base	12							
	3	Subgrade								
3	1	Asphalt Concrete	8	8.97	7.95	7.56	7.07	6.57	5.61	4.00
	2	Cement Stab. Base	6							
	3	Subgrade								
4	1	PCC Slab	9	8.94	8.41	8.05	7.48	6.91	5.80	4.02
	2	Lime Stab. Base	6							
	3	Subgrade								
5	1	PCC Slab	6	18.10	16.60	15.50	13.70	11.90	8.97	5.26
	2	Subgrade								
6	1	PCC Slab	12	8.38	8.06	7.91	7.71	7.50	7.05	6.10
	2	Cement Stab. Base	6							
	3	Subgrade								
7	1	Asphalt Concrete	3	7.50	6.13	5.87	5.43	4.97	4.09	2.72
	2	PCC Slab	9							
	3	Subgrade								
8	1	Asphalt Concrete	5	6.56	5.48	5.27	5.01	4.73	4.14	3.08
	2	PCC Slab	10							
	3	Lime Stab. Base	8							
	4	Subgrade								
9	1	Asphalt Concrete	4	6.89	5.87	5.74	5.59	5.42	5.05	4.28
	2	PCC Slab	12							
	3	Asphalt Stab. Base	8							
	4	Subgrade								

Load = 16,000 lbs Load Radius = 5.91 inches

Table 4.37. Comparison of the true and backcalculated moduli values (after Rada, et al., 1992).

Material Type	Layer Thickness (inches)	Assumed Moduli (BISAR) (psi)	Backcalculated Moduli			% Difference		
			Modcomp	Modulus	Wesdef	Modcomp	Modulus	Wesdef
AC	3	500000	553118	481700	480658	-10.62%	3.66%	3.87%
GB	6	50000	49520	51100	60371	0.96%	-2.20%	-20.74%
SG	---	20000	19973	20000	18723	0.14%	0.00%	6.39%
AC	6	300000	288840	311000	254599	3.72%	-3.67%	15.13%
GB	12	60000	65227	59100	75465	-8.71%	1.50%	-25.77%
SG	---	10000	9854	10000	9359	1.46%	0.00%	6.41%
AC	8	1000000	1100365	969700	960804	-10.04%	3.03%	3.92%
CTB	6	2000000	1610059	2130900	2436563	19.50%	-6.54%	-21.83%
SG	---	20000	19826	20100	18433	0.87%	-0.50%	7.84%
PCC	9	4000000	3572500	3118800	3645284	10.69%	22.03%	8.87%
LTB	6	60000	118685	237300	176950	-97.81%	-295.50%	-194.92%
SG	---	20000	19808	19700	18268	0.96%	1.50%	8.66%
PCC	6	3000000	2622308	2970800	3257227	12.59%	0.97%	-8.57%
SG	---	15000	15495	15100	13978	-3.30%	-0.67%	6.81%
PCC	12	4000000	4000000 *	4246400	3757139	0.00%	-6.16%	6.07%
CTB	6	2000000	1794517	1829600	2406254	10.27%	8.52%	-20.31%
SG	---	10000	9907	10000	8594	0.93%	0.00%	14.06%
AC	3	300000	298462	304000	256023	0.51%	-1.33%	14.66%
PCC	9	4000000	3240565	3883500	4899980	18.99%	2.91%	-22.50%
SG	---	30000	31381	30300	26828	-4.60%	-1.00%	10.57%
AC	5	600000	946524	649300	591949	-57.75%	-8.22%	1.34%
PCC	10	4000000	2223404	2764400	3871242	44.41%	30.89%	3.22%
LTB	8	100000	234554	254900	174140	-134.55%	-154.90%	-74.14%
SG	---	25000	24056	24700	22760	3.78%	1.20%	8.96%
AC	4	500000	500000 *	447900	493080	0.00%	10.42%	1.38%
PCC	12	4000000	4000000 *	7096600	3733876	0.00%	-77.42%	6.65%
STB	8	1000000	940431	367300	1300568	5.96%	63.27%	-30.06%
SG	---	15000	14796	15100	12996	1.36%	-0.67%	13.36%

Note: (1) AC = Asphalt Concrete, GB = Dense Graded Aggregate, PCC = Portland Cement Concrete, SG = Subgrade CTB = Cement Stabilized Base, LTB = Lime Stabilized Base, ATB = Asphalt Stabilized Base

* Fixed Modulus Value

Table 4.38. MICHBACK results for nine pavement sections.

Section	Backcalculated moduli (psi)				Error (%)			
	AC	Base	Subbase	Roadbed	AC	Base	Subbase	Roadbed
1	498769	51587	-	19953	-.25	3.17	-	-.24
2	306649	61170	-	9997	2.2	1.95	-	-.03
3	1010554	1946624	-	20003	1.06	-2.67	-	.02
4	3856337	59607	-	19973	-3.59	.65	-	-.14
5	2938099	-	-	15000	-2.06	-	-	0
6	3774276	2001021	-	9983	-5.6	.05	-	-.18
7	302930	3835586	-	30009	.98	-4.1	-	.03
8	589970	4329429	66263	24725	-1.67	8.23	-33.74	-1.1
9	489229	4499496	849430	14969	-2.15	12.49	-15.06	-.21

Table 4.39. Comparison of errors in the modulus values.

Section	Program	Error in backcalculated results (%)				RMS (%)
		AC	Base	Subbase	Roadbed	
1	MICHBACK	-0.25	3.17	-	-0.24	.11
	MODCOMP	10.62	.96	-	0.14	2.36
	MODULUS	3.66	-2.2	-	0.0	.16
	WESDEF	3.87	-20.74	-	6.39	2.32
2	MICHBACK	2.2	1.95	-	-0.03	.17
	MODCOMP	3.72	-8.71	-	1.46	.34
	MODULUS	-3.67	1.5	-	0.0	.17
	WESDEF	15.13	-25.77	-	6.41	1.12
3	MICHBACK	1.06	-2.67	-	0.02	.04
	MODCOMP	-10.04	19.5	-	0.87	.81
	MODULUS	3.03	-6.54	-	-0.5	.22
	WESDEF	3.92	-21.83	-	7.74	.26
4	MICHBACK	-3.59	.65	-	-.14	.03
	MODCOMP	10.69	-97.81	-	0.96	.81
	MODULUS	22.03	-295.5	-	1.5	.52
	WESDEF	8.87	-194.92	-	8.66	.23
5	MICHBACK	-4.37	-	-	0.0	.03
	MODCOMP	12.59	-	-	-3.3	3.68
	MODULUS	.97	-	-	-0.67	.2
	WESDEF	-8.57	-	-	6.81	.91
6	MICHBACK	-5.6	.05	-	-0.18	.03
	MODCOMP	0	10.27	-	0.93	6.8
	MODULUS	-6.16	8.52	-	0.0	.12
	WESDEF	6.07	-20.31	-	14.06	.21
7	MICHBACK	.98	-4.1	-	.03	.03
	MODCOMP	.51	18.99	-	-4.6	3.24
	MODULUS	-1.33	2.91	-	-1.0	.15
	WESDEF	14.66	-22.5	-	10.57	1.47
8	MICHBACK	-1.67	8.23	-33.74	-1.1	.03
	MODCOMP	-57.75	44.41	-134.55	3.78	.37
	MODULUS	-8.22	30.89	-154.9	1.2	.37
	WESDEF	1.34	3.22	-74.14	8.96	.58
9	MICHBACK	-2.15	12.49	-15.0	-.21	.01
	MODCOMP	0	0	5.96	1.36	5.42
	MODULUS	10.42	-77.42	63.27	-.67	.19
	WESDEF	1.38	6.65	-30.06	13.36	.57

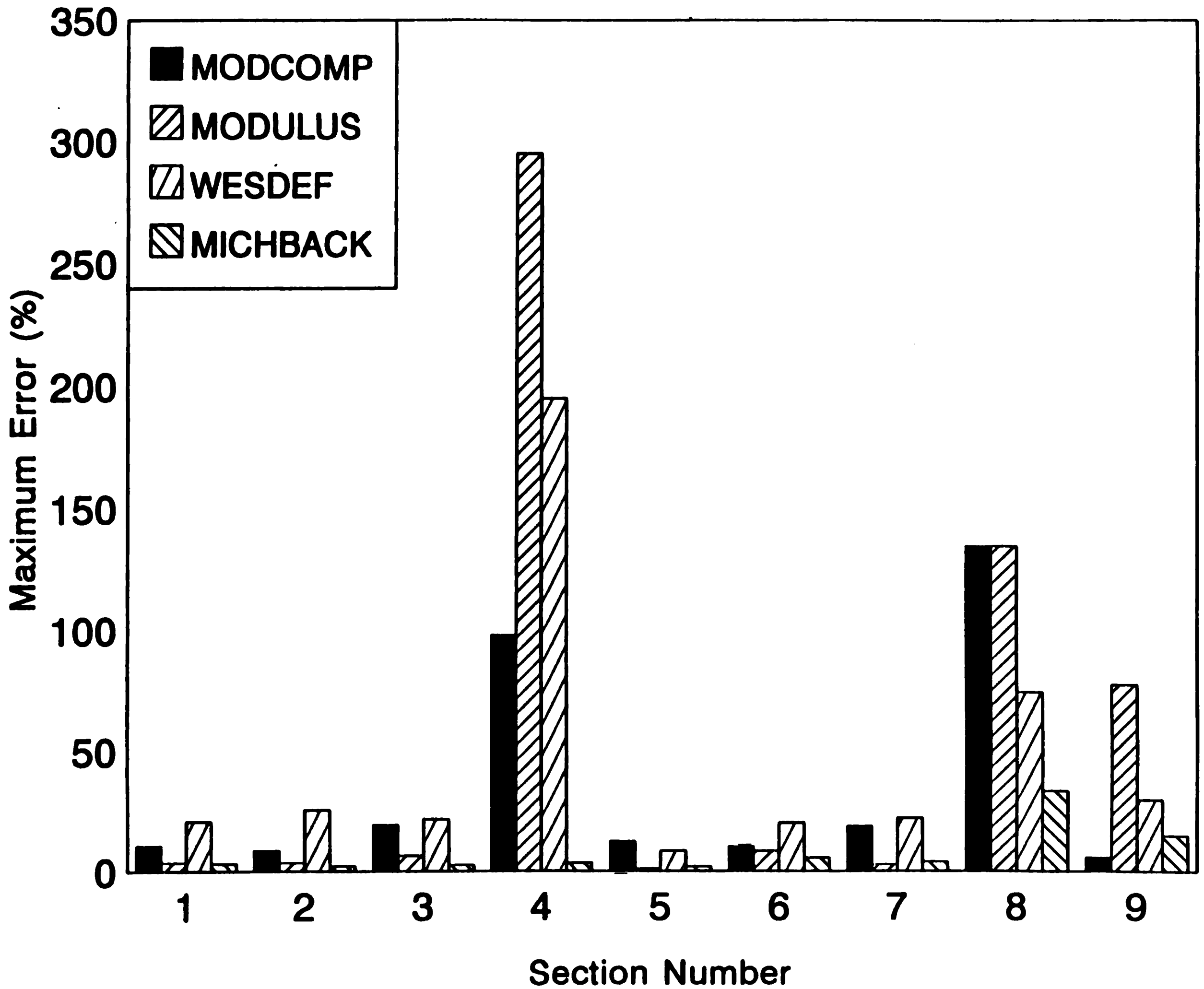

Figure 4.13. Maximum absolute relative percent error in layer moduli for different programs.

Table 4.40. Statistics of the maximum relative error for four computer programs.

Program	Statistics of maximum error			
	Minimum	Maximum	Average	Standard Deviation
MODCOMP	5.96	134.55	35.44	44.1
MODULUS	0.97	295.5	59.3	94.23
WESDEF	8.57	194.9	46.51	55.23
MICHBACK	2.06	33.74	8.02	9.85

resulting observations confirm once more the accuracy and consistency of the MICHBACK program relative to those programs.

4.12 SUMMARY

Several analysis for the validation of a new technique to assess the roadbed modulus, and to compare the effects of several factors on the backcalculated results are presented. Based on the analysis, the following conclusions were drawn:

1. The new technique to estimates the roadbed soil modulus effectively and, at the cost of only one call to CHEVRONX program, it yields a relatively accurate prediction.
2. The Newton's algorithm is incorporated in the MICHBACK program in a manner that can predict layer moduli accurately.
3. For flexible pavements without stiff layer, the accuracy of the MICHBACK results relative to other programs increases with increasing number of pavement layers.
4. For composite pavements, the MICHBACK results are significantly better than the other leading programs.
5. Inaccuracies in the layer thicknesses can induce large errors in the backcalculated layer moduli. The MICHBACK program, at the desire of the user, can correct any one layer thicknesses while estimating the layer moduli.
6. Inaccuracies in the stiff layer depth can induce large errors in the backcalculated results. The advantage of the MICHBACK program is its ability to accurately predict the stiff layer depth using mechanistic analysis.
7. The modulus of the Stiff layer has an insignificant effect on the backcalculated results.
8. The values of Poisson's ratios of the various pavement affect the accuracy of the backcalculated layer moduli.

9. The deflection outputs of the CHEVRON, ELSYM5, and EVERCALC programs differ from that of the BISAR (the accuracy of the BISAR output has been well established). The difference increases with the stiffness of the pavements.

CHAPTER 5

MICHBACK PROGRAM STRUCTURE AND FEATURES

5.1 GENERAL

The MICHBACK program has been written in Fortran 77. The structure and the user friendly features have been designed to facilitate the use of the program even by amateurs. The program can read the output files of the MDOT operated Falling Weight Deflectometer (KUAB), and minor changes in one of the subroutines can enable it to read the output of any other type of FWD. When processing the data from an FWD output file the program provides a wide range of options to the user to view and process the deflection data before it is analyzed.

In this chapter, the general structure and some of the features of the program are described. The function of each of the program subroutine is briefly introduced in Appendix A (Mahmood, 1993). Detailed explanation of the inputs required in each window would be available in a User's Manual (Harichandran, et al., 1994).

5.2 DATA INPUT

Deflection data can be entered using the keyboard or if the deflection output file format is that of the MDOT operated KUAB FWD, the file can be read and processed by the program automatically. The cross-sectional data, Poisson's ratios, type of the pavement being analyzed, the desired convergence criteria, expected ranges of the layer moduli, pavement temperature, and information regarding load application arrangement must be entered using the keyboard. Two options of the more frequently encountered deflection sensor layout schemes are provided. Other layouts can be specified by the users by the keyboard input. The default weight allocated to each sensor is 1.0, but can be changed by the user if desired. The input information is stored in a data file which can be edited on the interactive screen at any stage.

When entering the deflection data using the keyboard, a total of 4 deflection basins can be entered and processed by the program simultaneously. These basins must pertain to the same pavement cross section, although the test load may be different. The deflection basins are displayed on the screen in graphical form for the user to view and subsequently edit if an incorrect input is detected. Analyst can view the entered deflection basins one at a time or even can choose to look at all four basins simultaneously. All the deflection basins are analyzed without interruption and the backcalculated results of one basin are taken as seed moduli for the next basin. When processing the deflection data from a file, comprehensive keyboard input is required only at the start of the analysis. All the essential information required to operate the program is stored in easily accessible data files. When changes in the pavement cross section are encountered, required changes in the data file can be made by editing the existing data file. The various options provided to view, process, and analyze the deflection data from FWD files are discussed in the next section

5.3 PROCESSING FWD DEFLECTION DATA FILE

The MICHBACK program can read output files containing deflection data generated by the MDOT operated FWD (KUAB). The system is flexible and only minor changes would be required in one of the subroutines to customize the program to read other formats. The FWD output files normally contain deflection data for tests conducted over long pavement sections. Hence, a wide range of variability in the deflection data is expected. Even if the section length is small it will be beneficial to the user to view and understand the variability of the deflection data before starting the backcalculation process. The analyst should be able to identify and if necessary remove any outliers indicative of erroneous sensor measurements. Also, sub-dividing a long test section can be done in a more effective manner after reviewing the data. The MICHBACK program provides features which make the pre-processing of the deflection data very easy and efficient. The highlights of these features are covered in the next section.

5.3.1 Reviewing and Preprocessing the Deflection Data

Most backcalculation programs can read the FWD deflection output files of the agencies for which they were developed. Some of the programs use statistical methods to subdivide the pavement test section into uniform subsections, mostly based on the variations in the pavement peak deflections. The MODULUS program allows the user to review the measured surface deflections at each sensor location along the length of the section. But the program does not allow more than one sensor readings to be viewed simultaneously. Review of all the sensor readings along the pavement length of interest allows the user to observe the deflection trends and identify outliers. The analyst is not required to remember the deflection trends at all the stations for previously viewed sensors. The ability to drop an entire set of deflection readings at any station or even only few readings at some of the sensors, identified as outliers, is also essential for the accuracy and dependability of the backcalculated results.

In the MICHBACK program, deflections at all the sensors are plotted simultaneously along the length of the pavement section being analyzed (Figure 5.1). For a long pavement section the user can zoom-in on a smaller section for a more detailed viewing of the data (Figure 5.2).

Coupled with this feature is a "deletion" option which can be used while viewing the deflection data without having to exit the view mode. For each deflection basin, if the deflection at one or more sensor locations are identified as outliers, they can be deleted prior to performing the backcalculation at that location. Likewise, a deflection basin can be deleted if the user identifies such a basin as an outlier. Further, an option is provided so that the analyst can quit the system without implementing/saving the deleted data.

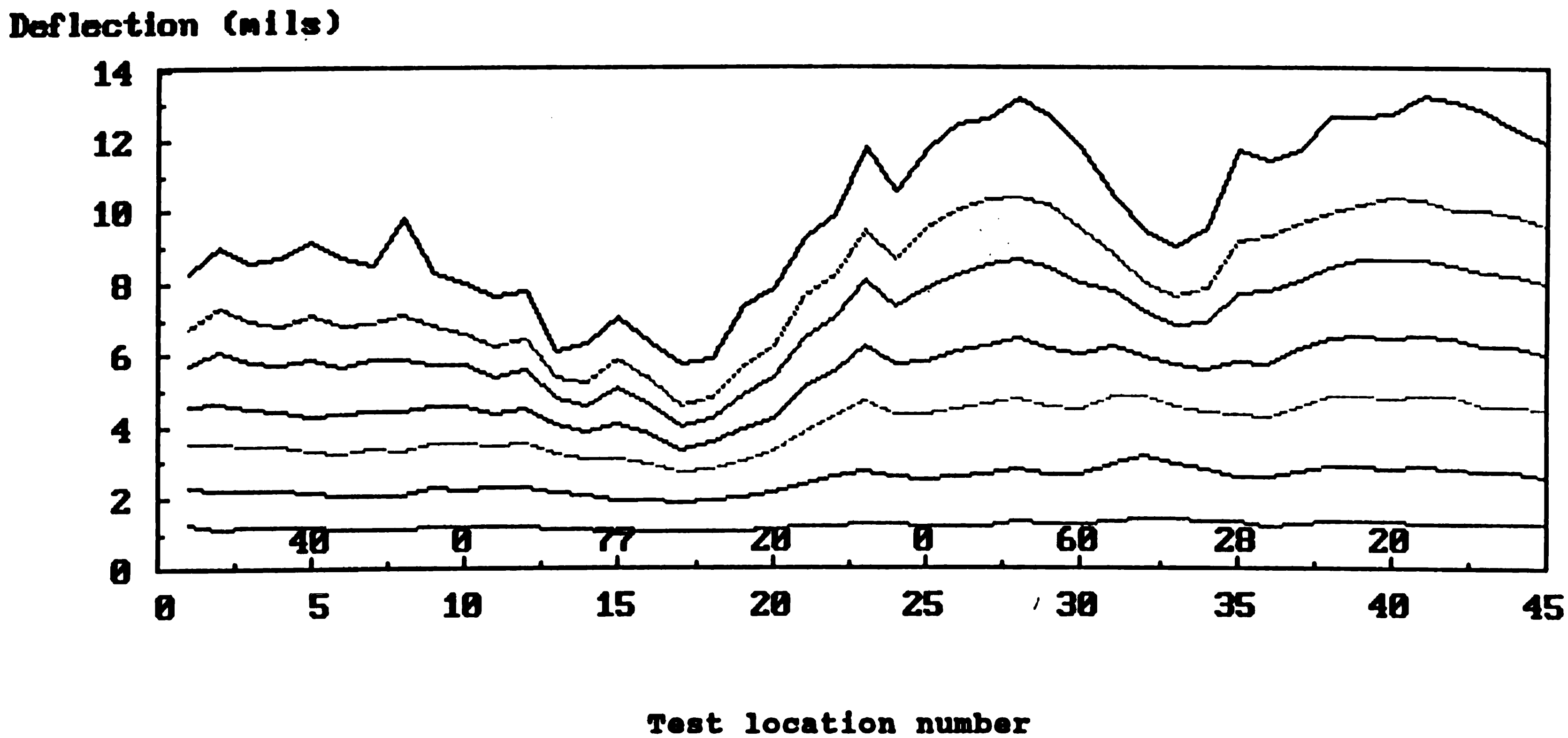

Figure 5.1. Deflection profile for pavement section MSU07F.

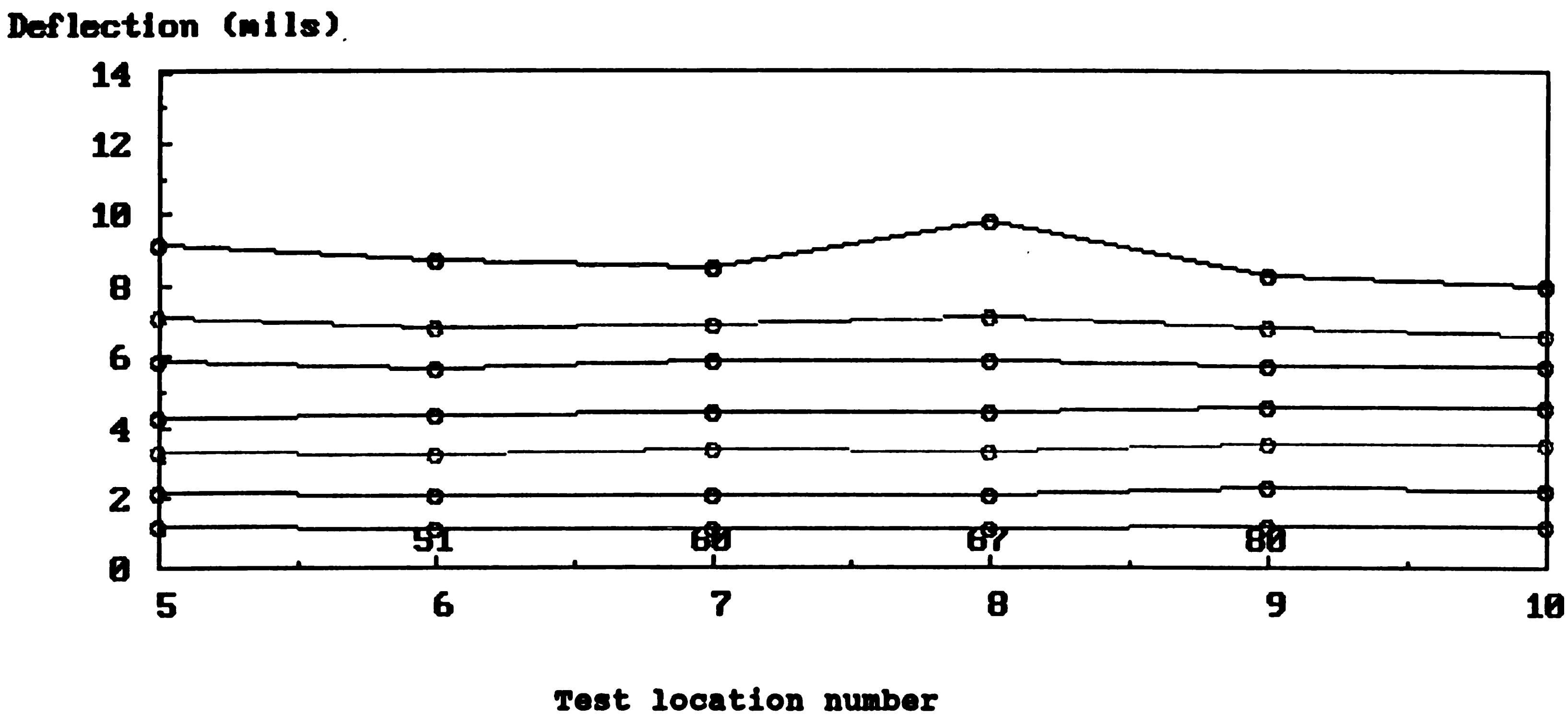

Figure 5.2. Zoom-in function between stations 10 and 15 of pavement section MSU07F.

5.3.2 Data Analysis Options

Most backcalculation programs provide the option, to either analyze deflection data at all the test stations or to backcalculate the layer moduli values based on averaged data for similar test sections. Sections that are similar are normally chosen by the programs internally without any user interaction. In MICHBACK the data analysis can be undertaken in a variety of possible methods, ensuring maximum user interaction.

Like most programs, for a test section where the cross section of the pavement does not change, the data can be analyzed at all the test locations. The user can view and process the FWD data that is read from a file and is required to enter the cross sectional data through the keyboard. The program can analyze the required data without any interference after this stage. To improve the program efficiency the results of previous location are taken as seed values for backcalculation at the next location. All the backcalculated results are saved in two different files, one file contains the summary of the results and the other contains a more detailed output.

The second available option is to choose a section with relatively uniform deflection measurements after visual inspection, and then backcalculate the results on the averaged deflection basin over the length of the chosen section. In this option, the average values of the cross section should normally be used for backcalculation. The results represent the average values of layer moduli for the section. This option may yield inaccurate results when the variation in the deflection data along the section is considerable. If the variability in the deflection data along the length of the section is moderate, this option may provide stability to the backcalculated results by reducing the random errors in the deflection measurements.

In the third option the program automatically selects the most representative deflection basin for analysis, from the uniform section chosen by the user. The concept of choosing the representative basin was first presented by Alexander et al.

(1989). The method has been adopted with some modification in MICHBACK. If all the test locations cannot be individually analyzed then a representative basin for the section being analyzed can be chosen. In this way an actual basin, which was physically measured, is selected to represent the section rather than an artificially computed basin, as would be the case if an averaged basin is analyzed. In MICHBACK the representative basin of a pavement section is chosen in four steps:

1. Step 1. Average of the peak deflections is computed (PKD).
2. Step 2. Average of measured deflections for respective sensors is computed (DF).
3. Step 3. Average of the area for all deflection basins is computed (AREA). The area is computed as the area under the measured portion of the deflection basin. The area between two sensors is assumed to be trapezoidal.
4. Step 4. An error function (Δ) is computed for each basin in the section as follows:

$$\Delta = \left(\frac{\overline{PKD}-PKD}{\overline{PKD}}\right)^2 + \sum_{x=1}^{n}\left(\frac{\overline{DF}-DF}{\overline{DF}}\right)^2 + \left(\frac{\overline{AREA}-AREA}{\overline{AREA}}\right)^2$$

where PKD = peak deflection;
DF = measured deflection;
AREA = computed area; and
n = number of sensors.

The measured deflection basin with the least error is then chosen as the representative basin.

Extensive study by Anderson (1989), showed that the use of the representative basin concept for flexible pavements yields good results. But for composite pavements, due to the wide variability in the backcalculated moduli, the use of a representative basin to estimate the moduli for an entire section is not recommended. In MICHBACK all three options have been provided. After gaining some experience with the deflection data pertaining to pavements in their area the users may choose the method best suited to their needs.

5.4 PRESENTATION OF THE BACKCALCULATED RESULTS

The backcalculated results are saved in a file which can be printed, or can be viewed on the screen. Results can also be seen graphically to observe the variation along the desired section length. The backcalculated layer moduli for all layers are plotted separately to present the variation clearly. The test locations for which the backcalculated results have touched either of the higher or lower bounds, specified by the user, are posted by a warning in the output for the user to consider.

5.5 PROGRAM STRUCTURE

The flow diagram of the program is presented in Figure 5.3. Some of the features related to the diagram have already been explained. Seed values for the layer moduli and the stiff layer depth are furnished by regression equations. The user is required to provide a set of seed values for the layer moduli as well. Just prior to the start of the backcalculation the two sets of seed moduli values are displayed and the user may select one or the other, or enter new values altogether.

Similarly, the seed value for the stiff layer depth estimated by the regression equations is displayed so that the user may decide whether or not to incorporate a stiff layer. If a stiff layer is desired the user can choose the seed value suggested by the program or can enter the most probable depth if known. When the stiff layer is incorporated, the program also requires its modulus and Poisson's ratio to be entered. Finally the user decides if estimation of the stiff layer depth along with layer moduli is also desired or the stiff layer depth may be fixed. Backcalculation is performed as

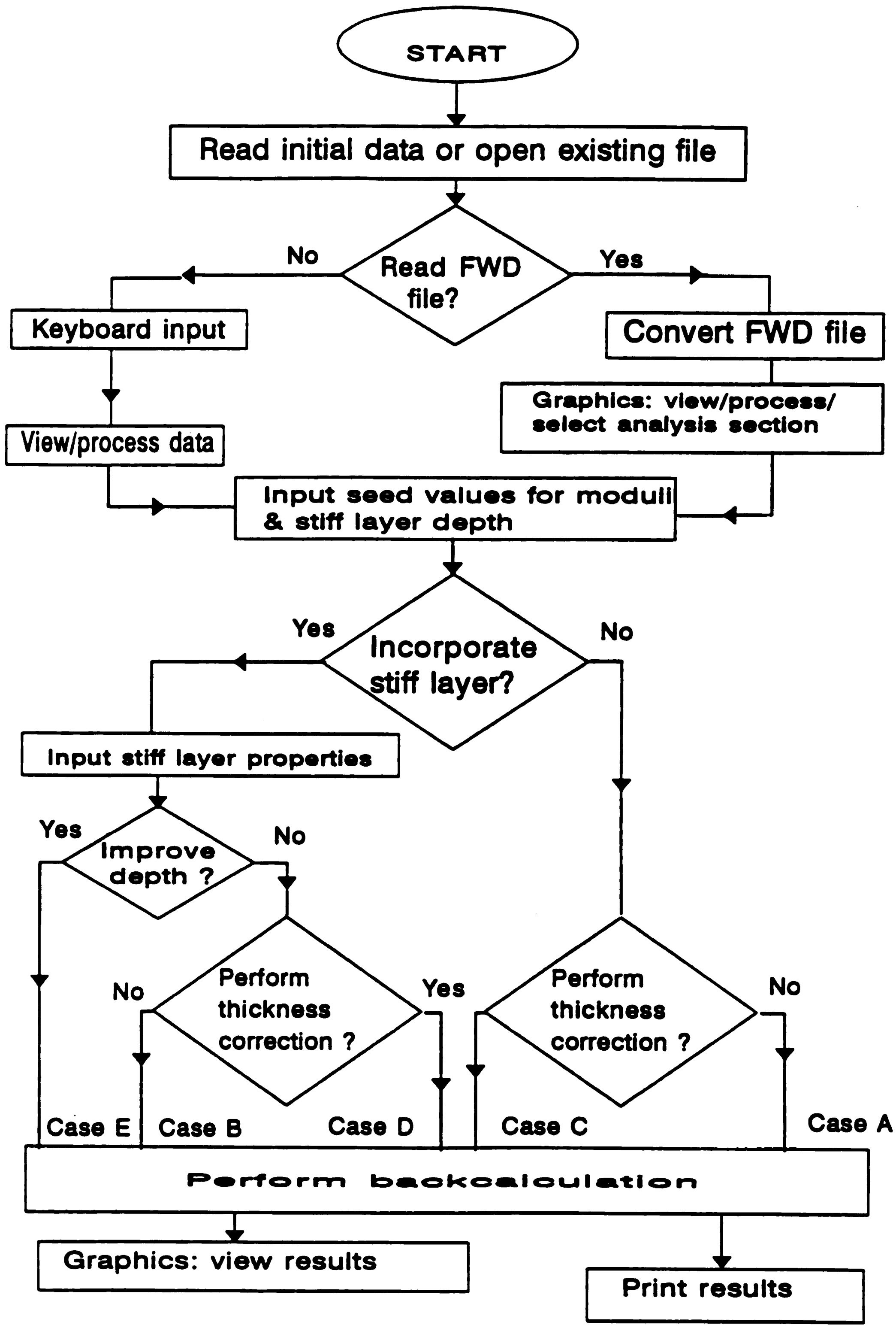

Figure 5.3. Flow chart for MICHBACK program.

desired by the user and the results are saved in a file. These backcalculated results can be viewed graphically as explained earlier. Finally the summary or the detailed results can be printed as desired.

5.6 BACKCALCULATION OF LAYER PROPERTIES

The backcalculation tasks performed by the program can be divided into three major groups:

1. Backcalculation of layer moduli without incorporation of stiff layer (Case A), or with stiff layer depth fixed at the probable value chosen by the user(Case B).
2. Backcalculation of layer moduli and the layer thickness of one of the layers when a stiff layer is not incorporated (Case C), or with the stiff layer depth fixed at the seed value (Case D).
3. Simultaneous backcalculation of layer moduli and the stiff layer depth (Case E).

5.6.1 Cases A and B

For cases A and B the layer moduli are backcalculated with or without a stiff layer (Figure 5.4), and if a stiff layer is incorporated then its depth is fixed at the user-specified value. A three layer pavement being analyzed as Case A (without stiff layer) will be treated as a four layer pavement under case B (with the stiff layer at a fixed depth). But the properties of fourth stiff layer are assumed to be known. This is the only capability available with existing backcalculation programs.

5.6.2 Cases C and D

These two problems require one of the layer thicknesses to be backcalculated along with the layer moduli. Like Case B above, Case D has a stiff layer incorporated

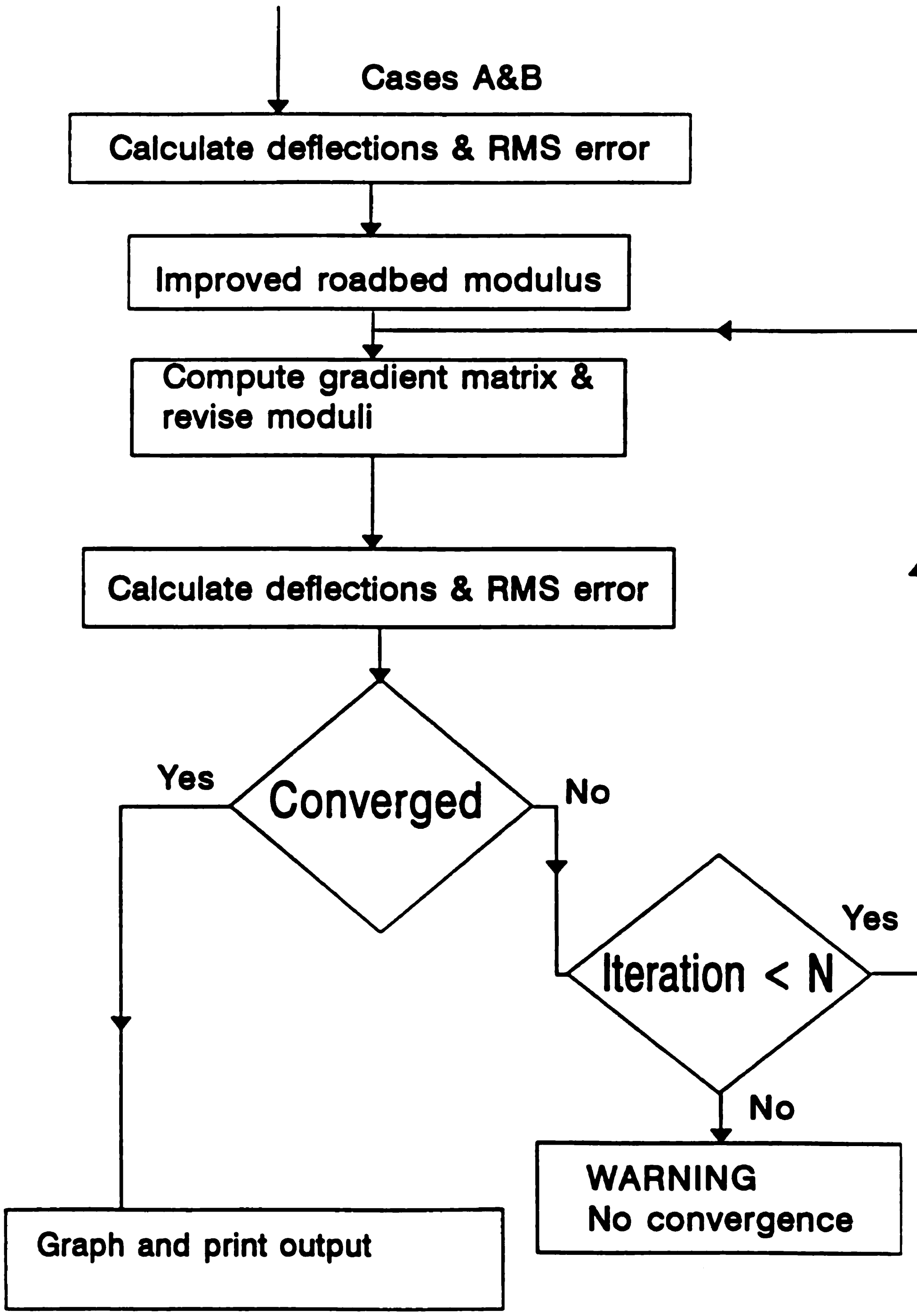

Figure 5.4. Details of backcalculataion procedure for cases A & B.

at a fixed depth for this option (Figure 5.5).

Initially the layer moduli estimates are improved. When the RMS error falls below 10% or when the number of iterations exceeds 2, only then is the improvement of the layer thickness undertaken along with the layer moduli.

5.6.3 Case E

Like Cases A and D, the backcalculation of layer moduli is the only desired objective. But for this case the depth to the stiff layer is not known and must be estimated by the program from the deflection data. The same criteria as outlined for Cases C and D is used to switch between the improvement of layer moduli alone to the improvement of layer moduli along with the stiff layer depth (Figure 5.6).

This option works like Cases C and D with a minor change. Each time after layer moduli and stiff layer depth are improved simultaneously the stiff layer depth is improved by itself. This alternating scheme has the best convergence characteristics.

For all the cases described the backcalculation is terminated with a warning if at any time during the backcalculation process the number of iterations equals the maximum number of iterations specified by the user. In such an event, the layer properties for which the RMS error was lowest during the entire backcalculation process is assumed to be the closest to the actual value and are printed as the backcalculated results along with a warning. When any of the specified convergence criteria is met, the results along with the convergence criteria are recorded. Various options to examine the backcalculated results have been presented in Section 5.4.

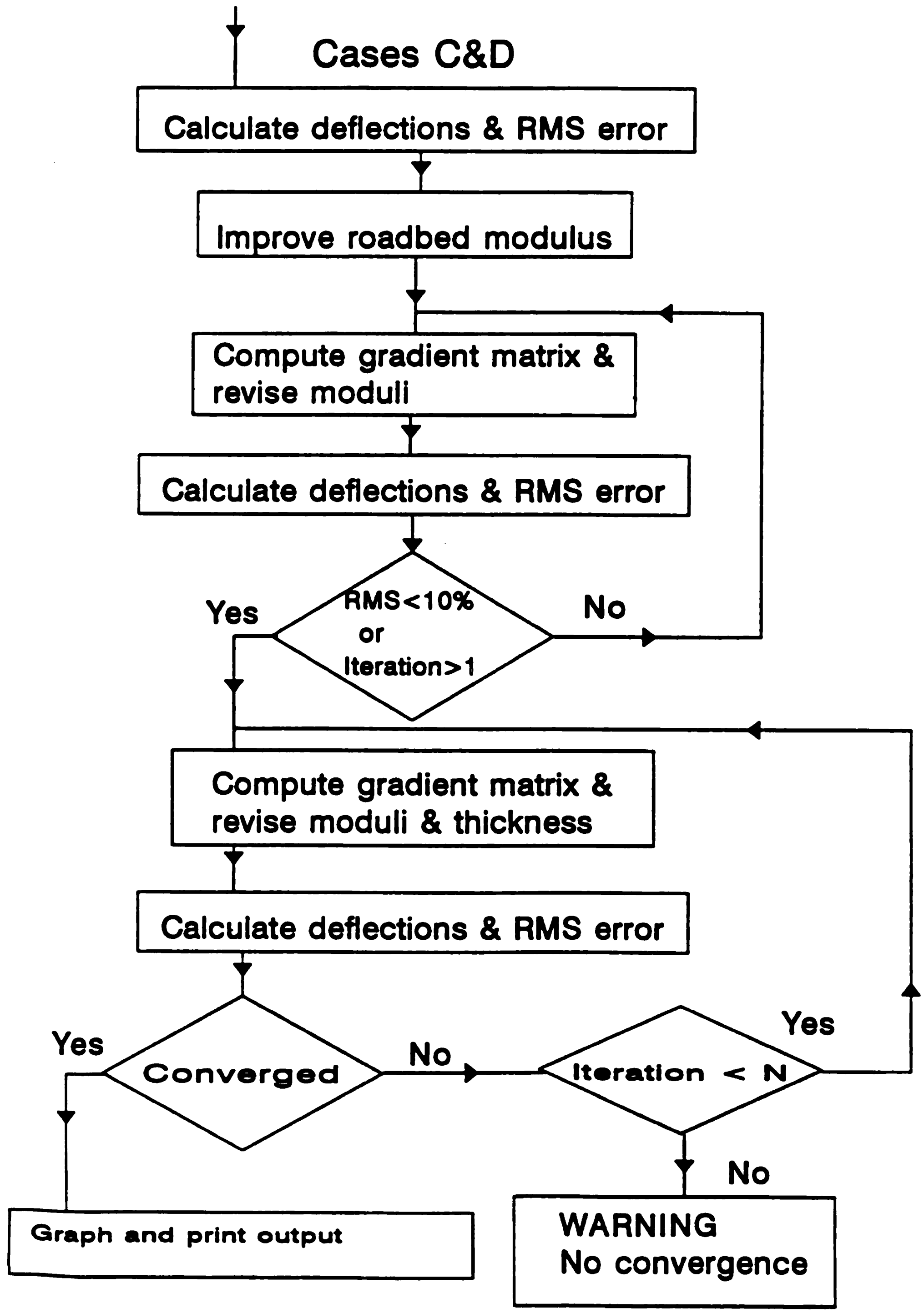

Figure 5.5. Details of backcalculataion procedure for cases C & D.

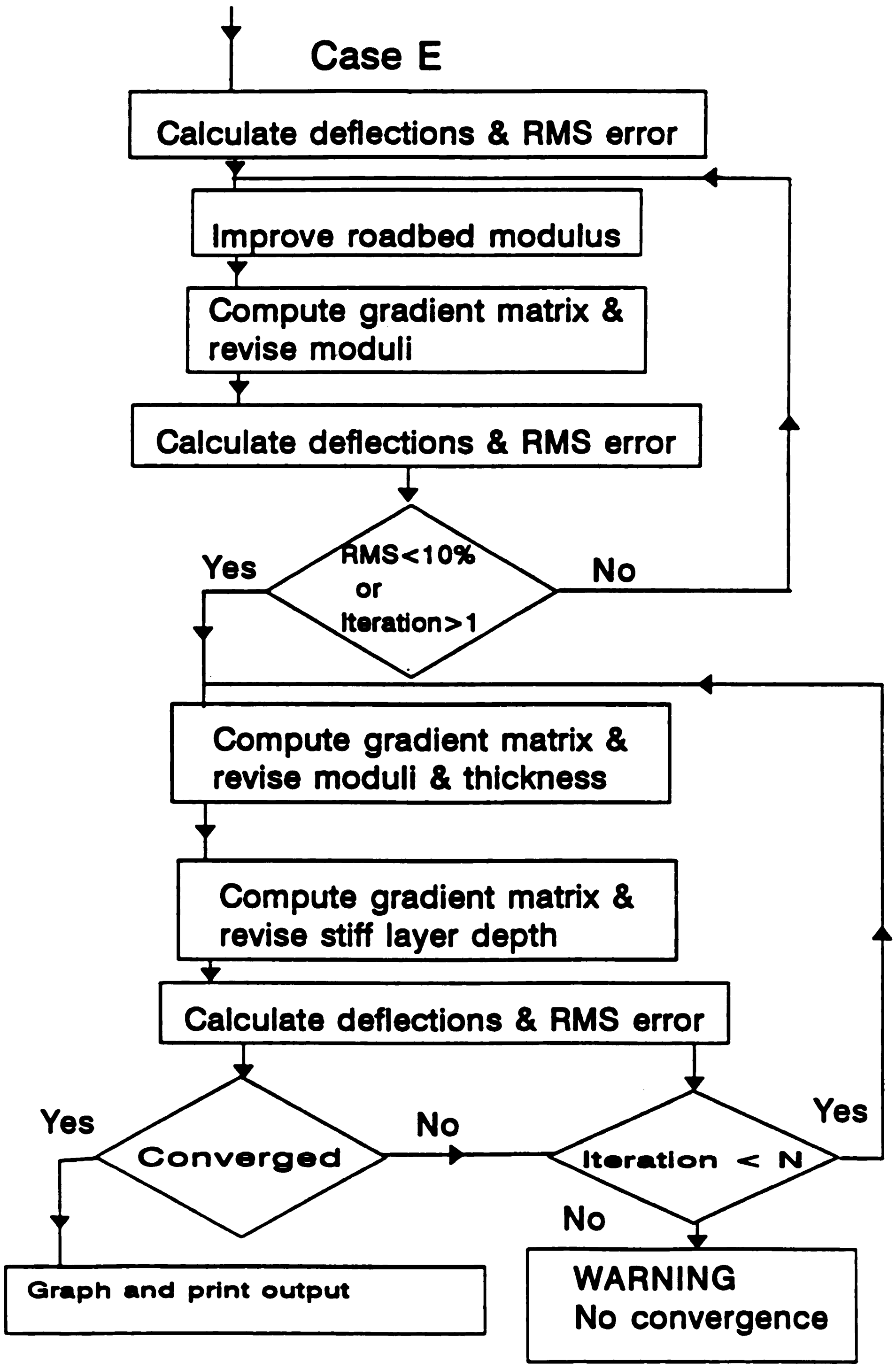

Figure 5.6. Details of backcalculataion procedure for case E.

CHAPTER 6

VALIDATION USING FIELD DATA

6.1 GENERAL

As stated earlier a comprehensive FWD testing plan was undertaken by MDOT as a part of this study. The FWD test data were analyzed to evaluate the capabilities and characteristics of the MICHBACK computer program. The analyses include the backcalculation of layer moduli values for various pavement sections tested by using varying load levels.

6.2 PAVEMENT SELECTION

The selection of the pavement test sections was accomplished in consultation with personnel from the Michigan Department of Transportation (MDOT). The main criterion used in this selection is that the pavement sections be representative of the spectrum of pavement cross-sections, paving materials, and traffic volume and load found throughout the State of Michigan. Also, the selected pavement sections should serve the needs of another separate but concurrent study (Mukhtar, 1993). In this regard, the following variables were identified and prioritized prior to the selection of the pavement sections:

1. Asphalt Course Thicknesses - Thin (less than 3-inches), moderately thick (3- to 6-inches), and thick (more than 6-inches) asphalt surfaces.
2. Traffic Volume and Load - Heavy, moderate, and light traffic load and volume in terms of 18-kips equivalent single axle load (ESAL).
3. Pavement Types - Flexible pavements without overlays, flexible pavements with overlays, and PCC pavements with asphalt overlays.
4. Cross-Sections - One layer (AC only), two layers (AC and base), and three layers (AC, base and subbase).

5. AC Mixes - Stability based and standard type mixes.
6. Roadbed Type - Cohesive and cohesionless soils.
7. Pavement Surface Age - Newly constructed and/or rehabilitated (less than 3-year old) and older pavement sections.
8. Distress Types - Rut and fatigue cracking.

Table 6.1 provides a list of the combination of variables used to prioritize the various flexible and composite pavement sections and the weights assigned to each variable. The weights are based on the importance of the variable in question. For example, a heavy traffic load (high percent commercial vehicles travelling the pavement section) was assigned a weight of 4 while a light traffic load was assigned a weight of 3. Likewise, a weight of 2 was assigned to each of two types of distress, fatigue cracking and rut. That is, the weight assigned to pavement section showing either rut or fatigue cracking was 2, the weight for a pavement showing both fatigue cracking and rut was 4, and the weight for a pavement section with no fatigue cracks and/or rut was zero.

Based on the above criteria and the various variables, 200 pavement sections (150 flexible and 50 composite) of variable lengths (one to several miles) were initially selected. For each pavement section, the MDOT pavement management system data base and the MDOT 1987 sufficiency rating book were used to obtain the location reference point, construction and rehabilitation history, rut, fatigue cracking, stripping, other distress data, traffic volume and load, cross-sectional data (layer thicknesses and types), and other general information. The data was then tabulated in a spreadsheet.

For each of the 150 flexible and 50 composite pavement sections, a score (based on the variables and their weights) was then calculated. A pavement

Table 6.1. Criteria for finial section selection.

Flexible Pavements		Composite Pavements	
Factor	**Weight**	**Factor**	**Weight**
Traffic (ESAL)		Traffic (ESAL)	
Light	3	Light	3
Heavy	4	Heavy	4
Thickness (inches)		Thickness (inches)	
Thin < 3"	3	Thin < 3"	2
Medium 3 To 6"	2	Medium 3 To 6"	3
Thick > 6"	2	Thick > 6"	4
Cross Section		AC Overlay	
2 Layers	3	1 Course	2
3 layers	2	2 Course	2
4 layers	2	3 or more Courses	3
AC Courses (no overlay).		Number of Overlays	
Less than 3	1	1 Overlay	2
More than 3	1	2 or more Overlays	2
AC (overlay)	1	Overlay Age (years)	
Roadbed Soil		Less than 3	2
		New Mix	2
Sand	2	Old Mix	2
Clay	2		
Pavement Age (years)		Pavement Distress	
Less than 3	2	Fatigue	2
New Mix	2	Rut	2
Old Mix	2	fatigue and Rut	2
Pavement Distress			
Fatigue	2		
Rut	2		
Fatigue and Rut	2		

score consists of the sum of the weights assigned to each variable. The pavement section with the highest score was given the highest priority. Based on the pavement score (priority), the 49 flexible and 15 composite pavement sections with the highest priorities were chosen and they are included in this study. Tables 6.2 and 6.3 provide lists of these flexible and composite pavement sections. Other information such as pavement type, route number, direction (north, south, east, or west bound), district, control section number, and the beginning and ending mile post of each pavement section are also listed in the tables. Figure 6.1 shows the distribution of the selected pavement sections across the State of Michigan.

For each of the 49 flexible and 15 composite pavement sections, the pavement condition data (obtained from MDOT data bank) was examined. It should be noted that the MDOT data bank does not identify fatigue cracking as a separate distress category. Rather, it identifies the severity and extent of cracking which includes various types of cracks. Therefore, only the rut data in the MDOT data bank were considered.

Each pavement section was divided into several 100-feet long test sites. For some sections, the test sites were adjacent to each others while for some others, they were separate.

During the Summer of 1991, four members of the Michigan State University (MSU) research team visited each test site. The purposes of the visit were to:

1. Verify the location reference point, pavement type, and general conditions of the sites.
2. Mark the test sites.
3. Inspect, measure, and record the extent and severity of rutting, fatigue cracking and other types of distress.
4. Identify those test sites to be cored by MDOT.
5. Mark locations for nondestructive deflection tests (NDT) within each test site.

Table 6.2 Flexible pavement sections selected for study.

Section NO.	Route I =11 US=22 M =33	Dir. N=1 S=2 E=3 W=4	District	Control Sectio	Mile Post FROM	TO	Remarks
MSU01F	3366	1	3	40031	2.00	13.00	
MSU02F	3350	3	8	23052	1.30	14.00	
MSU03F	3334	3	8	46041	1.50	11.20	
MSU04F	3372	3	3	40023	0.00	8.00	
MSU05F	3328	3	1	66022	3.00	14.40	
MSU06F	3350	4	5	34021	4.50	8.00	
MSU07F	22131	1	5	54014	8.00	11.10	Cored
MSU08F	33138	3	6	79011	16.70	19.90	
MSU09F	3320	4	5	54022	0.00	0.60	
MSU10F	3328	3	1	31021	4.60	9.60	Cored
MSU11F	3344	3	5	41051	4.20	5.20	
MSU12F	1196	3	5	61152	1.20	5.40	
MSU13F	2227	1	3	18034	1.31	1.80	Cored
MSU14F	3377	1	2	75052	8.00	10.00	Cored
MSU15F	2227	2	4	20016	0.00	6.30	
MSU16F	22131	1	3	83031	2.50	3.00	
MSU17F	22131	1	5	54013	0.50	8.41	
MSU18F	2227	1	4	20016	0.00	6.30	
MSU19F	1175	1	4	69013	0.00	6.00	cored
MSU20F	3366	1	5	59051	11.90	13.20	
MSU21F	3319	1	6	74032	0.00	10.00	
MSU22F	3382	4	5	62041	0.00	5.90	
MSU23F	3328	3	1	66023	6.60	11.00	
MSU24F	3399	1	8	33011	4.30	4.60	
MSU25F	22131	2	3	83031	3.00	2.00	
MSU26F	3357	3	6	25102	2.85	2.95	
MSU27F	1175	1	4	72061	7.00	13.40	
MSU28F	1175	1	4	16093	0.00	5.90	
MSU29F	3399	1	8	23092	2.20	7.30	cored
MSU30F	1175	1	4	69013	6.00	7.00	
MSU31F	3326	4	1	66051	0.00	4.00	
MSU32F	22131	1	5	59012	11.00	11.10	

Table 6.2 Flexible pavement sections selected for study (continued).

Section Designation	Route I =11 US=22 M =33	Dir. N=1 S=2 E=3 W=4	District	Control Section	Mile Post FROM	TO	Remarks
MSU33F	22131	2	5	54014	8.50	8.60	
MSU34F	33129	1	2	17072	12.10	19.30	
MSU35F	1175	1	4	16091	0.00	1.50	
MSU36F	1175	2	4	72061	13.30	7.00	
MSU37F	1175	1	4	72061	19.00	23.66	
MSU38F	3366	1	3	57013	8.20	12.71	
MSU39F	222	3	2	21024	4.80	14.80	cored
MSU40F	1175	1	4	20015	4.10	9.10	
MSU41F	1175	1	4	20015	9.10	14.20	
MSU42F	1194	3	7	11015	13.00	15.00	
MSU43F	1194	3	7	11015	3.00	7.00	cored
MSU44F	2223	1	4	71073	24.70	26.71	
MSU45F	1175	1	4	20015	0.00	1.00	
MSU46F	3366	1	3	40031	0.00	1.22	
MSU47F	3361	3	3	18041	8.65	8.92	
MSU48F	3349	1	8	30011	11.00	17.00	
MSU49F	1175	2	4	72061	16.80	17.50	

Table 6.3 Composite pavement sections selected for study.

Section Designation	Route I =11 US=22 M =33	Dir. N=1 S=2 E=3 W=4	District	Control Section	Mile Post FROM	TO	Remarks
MSU01C	1194	4	8	81104	5.60	7.90	cored
MSU02C	1194	4	8	81104	1.90	5.60	
MSU03C	3350	3	8	46081	1.10	3.00	
MSU04C	22127	1	8	30071	0.00	4.90	cored
MSU05C	2241	1	1	7023	8.48	14.01	cored
MSU06C	22223	3	8	46061	2.80	3.60	
MSU07C	22127	1	8	46011	4.50	5.18	
MSU08C	3350	3	8	46081	8.30	13.20	cored
MSU09C	2241	1	1	7013	0.00	3.10	
MSU10C	3350	3	8	46081	3.00	8.30	
MSU11C	3325	3	6	32012	12.00	19.90	cored
MSU12C	3325	3	6	32012	19.90	27.90	
MSU13C	22127	1	8	30071	9.80	10.30	
MSU14C	22127	1	8	30071	4.90	9.80	
MSU15C	1175	1	9	63173	9.25	9.50	

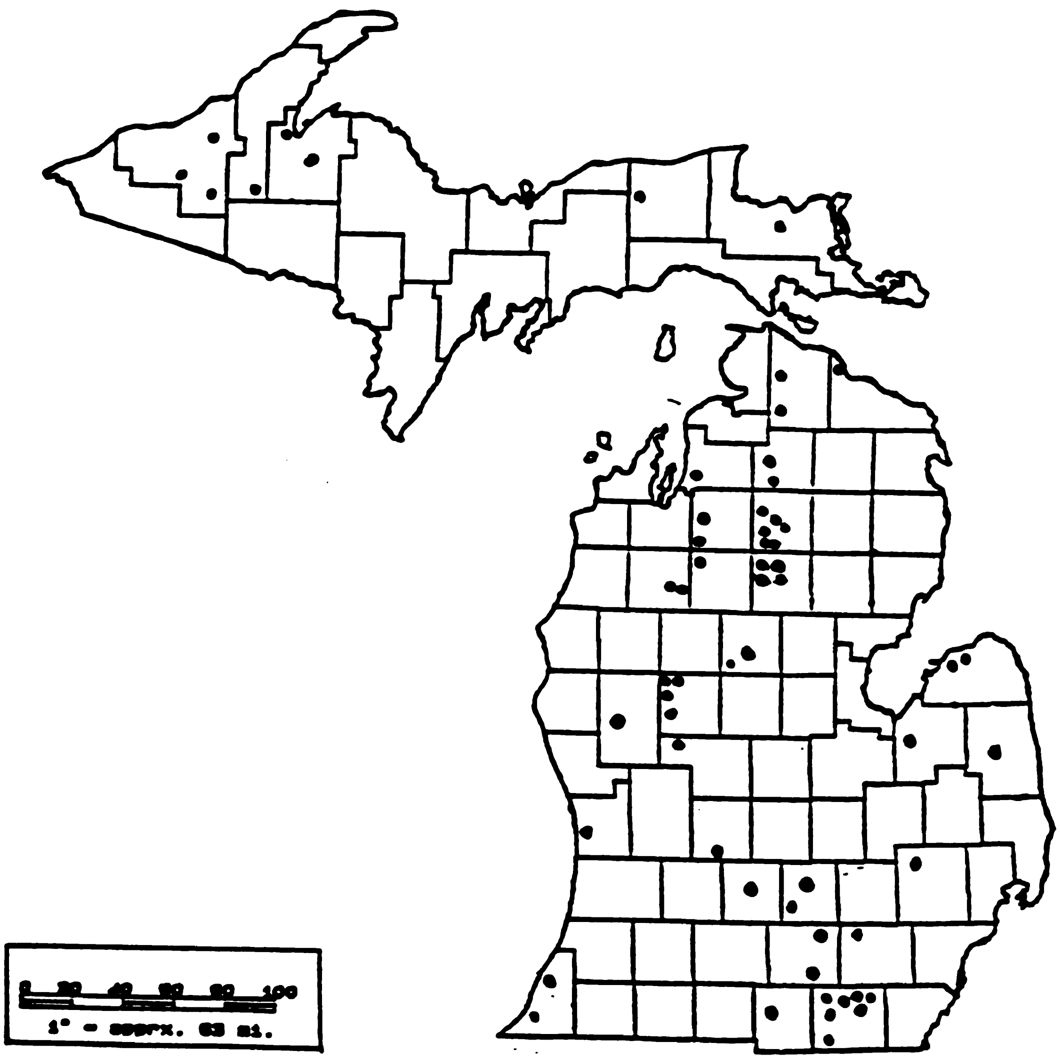

Figure 6.1. Distribution of selected pavement sections across the State of Michigan.

All sites were visited again during the summers of 1992 and 1993. During each visit, the rut and fatigue cracking data were collected and the general conditions of the sites were recorded. The rut depth was measured by using a six feet straight-edge leveling rod and a graduated triangular wedge with an accuracy of 0.025-inch. The rut was measured in the outer wheel path over each marked core location and at several other locations along the test site. The fatigue crack condition was recorded in terms of severity and the percent of the 100-feet long test site showing alligator cracking. Further, a total of 106 locations were designated for pavement coring. Each test site and NDT and core locations were given specific designation numbers. The coring method and the designation numbers are presented in the next section.

6.3 MARKING, CODING, CORING AND NDT

Each test site was designated by a two-number system. One number designates the pavement section and the other designates the test site. For example, a test site designation of 29-1 indicates the first test site of pavement section number 29. It should be noted that, for all pavement sections the test site number increases from south to north and from west to east.

Some test sections were selected for coring. The cores were located either in the outer wheel path, between the wheel paths, or in the inner wheel path of the traffic lane. In addition, some cores were located over an existing longitudinal crack. Each core location was designated using a seven digit number. Starting at the left most digit, the first two digits indicate pavement section number; the third digit indicates pavement type (1 for flexible and 2 for composite); the fourth digit designates the site number; the fifth and sixth digits indicate the distance of the core location from the beginning of the site; and the seventh digit indicates the core number within that site. For example, a core location designation of 2712402 indicates (left to right) that the core is obtained from section 27, flexible pavement, test site number 2, at 40 feet from the beginning of the site, and the second core of the site.

The cores were obtained by using a power auger equipped with a 6-inch coring bit. A hand auger was preferred over a power auger to obtain undisturbed samples from unbound bases, subbases and roadbed soils so that different layers could be easily identified. Inspite of this, for most pavement sections, separate and accurate identification of the base and subbase layer thicknesses was not possible because of the type of materials encountered (cohesionless and moist).

Non-destructive deflection tests (NDT) using a falling weight deflectometer (FWD) were conducted at several locations along each test site. The NDT tests were divided into two categories as follows.

1. Regular tests - Each test site (100-feet long) was subjected to five FWD tests (a test is 3 drops). The tests were conducted at equal intervals of 20-feet starting at the beginning of the test site.
2. Additional tests - For cored test sites, additional FWD tests were conducted over the core locations.

The NDT were performed during various seasons as follows:

1. Summer 1991 - All test sections at about 9000 lb load.
2. Fall 1991 - All test sections at about 9000 lb load.
3. Spring 1992 - All test sections at about 9000 lb load.
4. Summer 1992 - All test sections excluding core locations were tested at about 9000 lb load. For the cored pavement sections the FWD test were conducted at three load levels 4500, 9000, and 15000 lb.
5. Spring 1993 - Cored sections only at about 9000 lb load.

Each FWD test and the resulting deflection files were designated by using an eight digit number. Starting at the left most digit, the first two digits indicate pavement section number; the third digit indicates pavement type (1 for flexible and 2 for composite); the fourth digit designates the site number; the fifth and sixth digits indicate the distance of the test location from the beginning of the site; the seventh digit indicates the sequential drop number (drop number 1, 2 or 3); and the eighth (right most) digit indicates test location (0 for regular test in the outer wheel path, 1 for additional test in the outer wheel path, 2 for additional test at the center of the lane, 3 for additional test in the inner wheel path, 4 for a test at a joint in the outer wheel path, and 5 for a test at a joint in the inner wheel path). For example, an FWD test designation of 27124020 indicates (left to right) that the test is conducted on section 27, flexible pavement, test site number 2, at 40 feet from the beginning of the site, and the second drop of a regular test in the wheel path.

6.4 BACKCALCULATION OF LAYER MODULI FOR SELECTED PAVEMENT SECTIONS

As noted earlier, during the Summer of 1991, NDTs were conducted on all cored and uncored flexible and composite pavement sections. The measured deflection basins are tabulated in Appendix C and the pavement layer thicknesses are included in Appendix B of a research report (Mahmood, 1993). It should be noted that for the uncored pavement sections, the thicknesses of the pavement layers were obtained from the proper MDOT files. For the cored pavement sections, the layer thicknesses were measured from the cores. Since the layer thicknesses of the uncored pavement sections are not accurately known, the results of only typical cored flexible and composite pavement sections are presented and discussed in this section. Recall that each NDT test consisted of three drops. The target load for each drop was set at 9000 pounds. The actual load delivered to the pavement section and measured by the load cell of the FWD however, varied slightly from the target load. Therefore, the average of the three deflection basins and the average load of the three drops were used to perform the backcalculation of the layer moduli using the MICHBACK computer program.

For most flexible pavement sections there are no significant differences between the base and subbase layer materials (Chapter 4, Field Manual of Soil Engineering, MDOT). In addition, as stated earlier, the similarity in the texture of the base and subbase materials made it very difficult to identify the thickness of each layer (the materials were soaked by water during the coring). Because of this similarity and due to the lack of accurate base and subbase thickness data, the two layers were combined into a single layer and all flexible pavements were analyzed as three-layer systems (AC, base, and roadbed).

All of the composite pavement sections included in the study are listed in the MDOT proposed cross-section file as a three layer system. The construction procedure, however, recommends the inclusion of a 3-inch base layer under the PCC slab (Chapter 4, Field Manual of Soil Engineering, MDOT). During coring, a base layer of various thicknesses was detected under some of the PCC slabs. However, these thicknesses could not be accurately measured during coring. Therefore, all the composite pavements are also analyzed as three layer systems.

A second peculiarity of the composite pavement sections is that the history of the pavement was not available from the MDOT records. All sections were originally constructed as PCC pavements. Only after their deterioration were they overlaid by an AC layer. The conditions of the pavement or the treatment given to the pavement before the overlay (joint repair, crack seating, etc.), could not be verified. During the visual inspection, only the distresses that could be observed on the AC surface were recorded. The condition of the slab after the overlay has a significant influence on the backcalculated moduli. Hence, major factors that can be used for interpreting the deflection data and backcalculated moduli are missing from the pool of explanatory variables.

Tables 6.4 and 6.5 provide a summary of the general conditions (fatigue cracking, rut depth, and other types of distress) and layout (section and site numbers,

Table 6.4. Summary of the layout and conditions of the cored flexible pavement sections.

Pavement section designation	Pavement site No.	Distance between sites (miles)	Fatigue cracking (%)	Rut variation (in.)	Other distress type	Deflection along the pavement
MSU07F	1, 2, 3	Adjacent sites	50-100	0.19-0.50	Transverse cracking	Variable
		1.5				
	4, 5, 6	Adjacent sites	0-15	0.19-0.50		Variable
MSU10F	1, 2, 3	Adjacent sites		0.05-0.42	High severity transverse cracking, stripping	Uniform
MSU13F	1, 2, 3	Adjacent sites	50-100	0.12-0.25	Lane shoulder separation	Moderate Variation
		0.30				
	4, 5, 6	Adjacent sites		0.12-0.25	Lane shoulder separation	Moderate Variation
MSU14F	1, 2, 3, 4, 5, 6	Adjacent sites	100	0.10-0.50	High severity transverse cracks, lane shoulder separation, and bad local drainage conditions	High
MSU19F	1, 2, 3, 4	Adjacent sites		0.20-0.28		Uniform to mild variation
		1.0				
	5, 6, 7, 8	Adjacent sites		0.20-0.28		Uniform to mild variation

Table 6.4. Summary of the layout and conditions of the cored flexible pavement sections (continued).

Pavement section designation	Pavement site No.	Distance between sites (miles)	Fatigue cracking (%)	Rut variation (in.)	Other distress type	Deflection along the pavement
MSU29F	1, 2, 3, 4	Adjacent sites	100	0.06-0.12	Block and transverse cracking, and stripping	High variation
		3.32				
	5, 6, 7, 8	Adjacent sites	100	0.06-0.12	Block and transverse cracking, and stripping	High variation
MSU35F	1, 2, 3	Adjacent sites	100	0.12-0.30	Block and transverse cracking	High
		0.10				
	4, 5, 6	Adjacent sites	100	0.12-0.20	Block and transverse cracking	High
MSU43F	1, 2, 3	Adjacent sites	0.25-0.35			Medium
		0.30				
	4, 5, 6	Adjacent sites	0.38-1.0			Medium
		1.0				
	7, 8, 9	Adjacent sites	0.12-0.19			Medium

Table 6.5. Summary of the layout and condition of the cored composite pavement sections.

Pavement section designation	Pavement site No.	Distance between sites (miles)	Distress type	Rut variation (in.)	Deflection along the pavement
MSU01C	1, 2, 3	Adjacent sites	Edge cracking, high severity lane shoulder separation and transverse cracking	0.25-0.31	High variation
		1.0			
	4, 5, 6	Adjacent sites	Edge cracking, high severity lane shoulder separation and transverse cracking	0.25-0.31	High variation
MSU05C	1, 2, 3	Adjacent sites	Edge cracking, transverse cracks and longitudinal cracks	0.25-0.56	Very high variation
		1.0			
	4, 5, 6	Adjacent sites	Edge cracking, transverse cracks and longitudinal cracks	0.25-0.56	Very high variation
MSU08C	1, 2, 3	Adjacent sites	5 to 7 evenly spaced high severity transverse cracks in each site	0.25-0.56	High variation
		2.0			
	4, 5, 6	Adjacent sites	5 to 7 evenly spaced high severity transverse cracks in each site	0.25-0.56	High variation
MSU04C	1, 2, 3	Adjacent sites	Pavement in good condition		High variation

and distances between the test sites) of eight cored flexible and four cored composite pavement sections, respectively. Descriptive terms relative to the variability of the deflection data along the various pavement sites are also listed in the right-most column of the tables. It should be noted that the location of any FWD test (a test consists of three drops) is designated by a location/station number. Location or station number 1 is always located at the beginning of the first test site of each pavement section. For any given pavement section, regardless of whether the test sites are adjacent to each others or not, a continuous numbering system (e.g., 1, 2, ..., 8) is used to designate the location/station numbers. In the next subsections, examples of the actual variations of the various deflection values along the sites are presented. Further, typical deflection basins measured at various location/station numbers along various pavement sites are also presented.

6.4.1 Flexible Pavement Section MSU07F - Variable Deflections

There are a total of six test sites in this section, the first three and the last three sites are grouped together with a distance of 1.5 miles between the two groups. The first three sites have low severity fatigue cracking extending over 50 to 100 percent over each site, while the extent of cracking along the last three sites is between 0 to 15 percent. The second group of sites is located on a fill section. The variations of the deflections (measured by the seven FWD sensors) along the length of the entire pavement section is shown in Figure 6.2. The backcalculated layer moduli are presented in Table 6.6 and they are discussed below in detail as an example of typical results for pavements with a relatively high variation in the measured deflections.

Examination of the values of the backcalculated layer moduli listed in Table 6.6 indicates that:

1. The AC modulus varies from a low value of 711,248 psi at test location

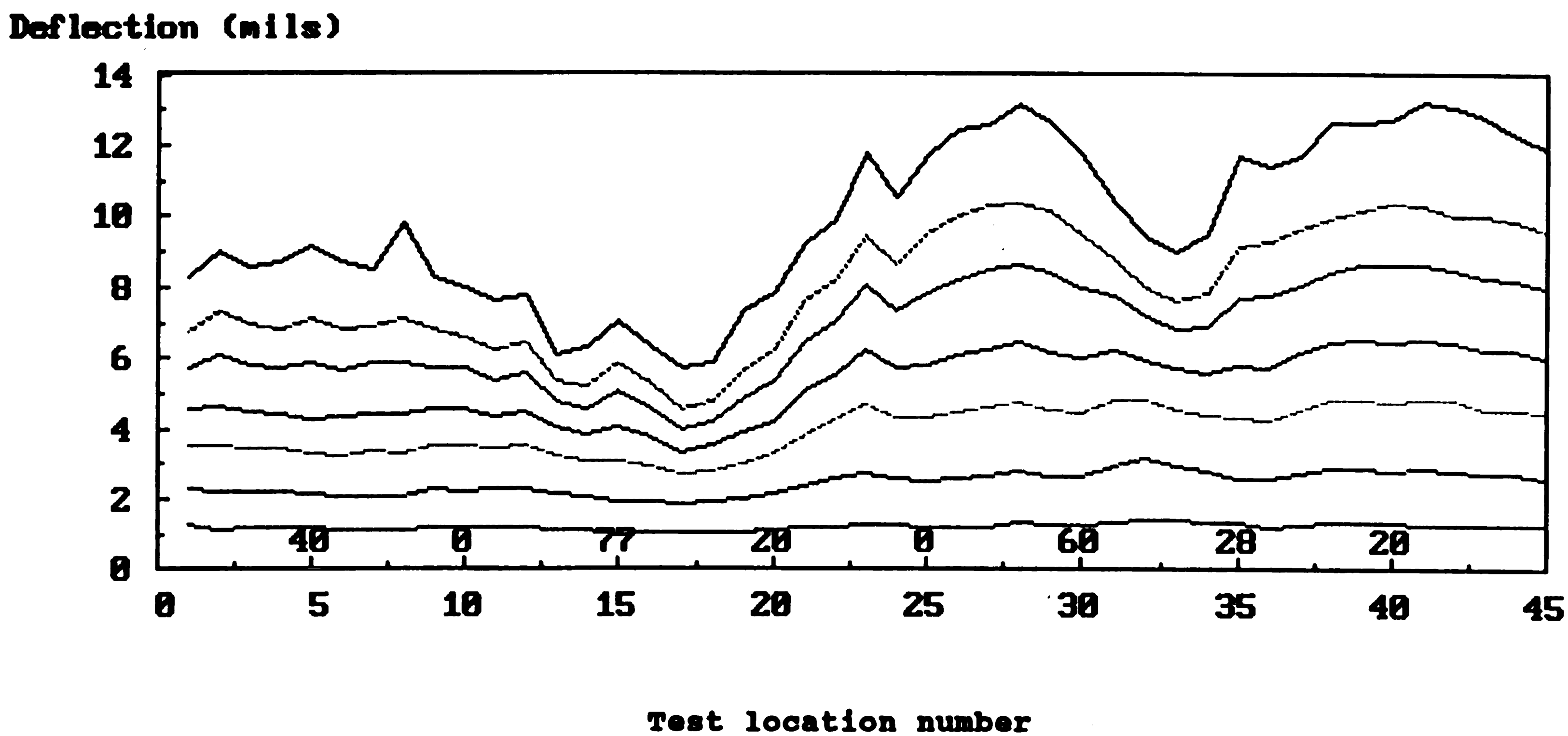

Figure 6.2. Deflection profile for pavement section MSU07F.

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Table 6.6. Backcalculated moduli for pavement section MSU07F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
07112011	3	711248	33538	30701	1.04
07122411	12	1106757	34646	29804	0.72
07127721	15	1131439	33336	36455	1.25
07128332	17	1320628	54731	34356	1.55
07131211	19	1028024	43254	32759	1.46
07137022	24	767833	21714	27864	0.36
07141211	26	909228	20197	27628	0.79

number 3 to a high value of 1,320,628 psi at test location number 17 (a factor of about 1.86).

2. The base modulus varies from a low value of 20,197 psi at test location number 26 to a high value of 54,731 psi at test location number 17 (a factor of about 2.71).
3. The roadbed modulus varies from a low value of 27,628 psi at test location number 26 to a high value of 36,455 psi at test location number 15 (a factor of about 1.32).
4. The maximum values of the root-mean-square of the errors between the seven calculated and measured deflection values is 1.55 %.

In order to discuss the variations in the values of the backcalculated layer moduli provided in Table 6.6, the measured pavement deflections at sensors 1 (the inner-most sensor) and 7 (the outer-most sensor) and the AC thickness for test location number 3 were designated as datum values and the percent differences between the datum values and the corresponding values at other test locations were calculated and are depicted in Figure 6.3. Examination of the figure indicates that:

1. The relatively high AC modulus values at test locations 12, 15, and 19 can be related to lower measured deflections (sensor 1) and AC thicknesses at these locations. Thin AC layers should lead to higher deflections. The low measured deflection values at these locations relative to those at test location number 3, imply that the AC layer is stiffer and hence its modulus is higher.
2. For test location number 17, the measured deflection at sensor 1 is lower than that at test location number 3, yet the AC thickness is almost the same (slightly higher). Again, the stiffer (higher modulus) of the AC is the main contributing factor to lower deflection.
3. For test locations 24 and 26, the measured deflections at sensor 1 are higher

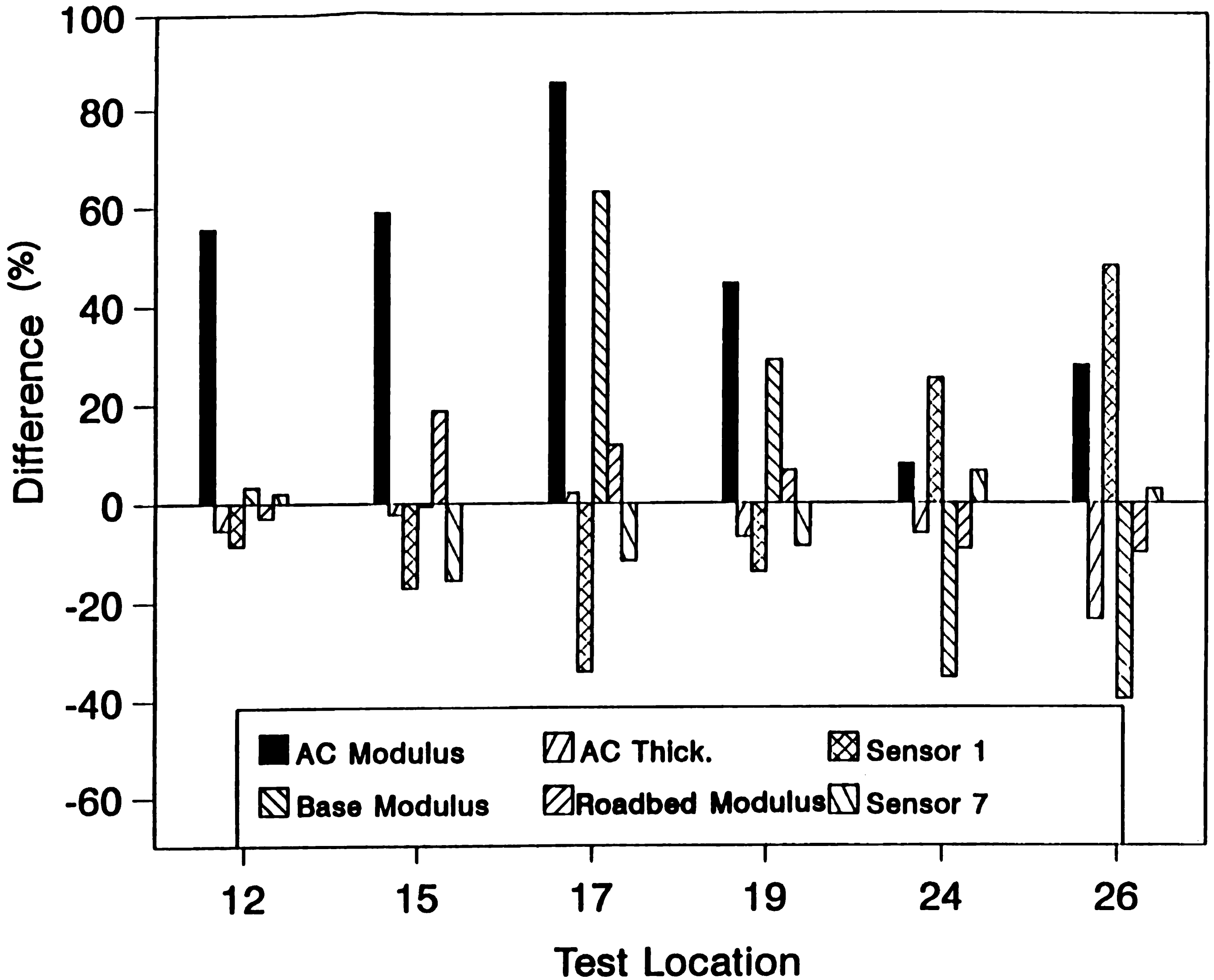

Figure 6.3. Variation of backcalculated layer moduli, the AC thickness, and the deflection of sensor 1 and 7 along pavement section MSU07F.

than those at test location number 3 and yet the backcalculated layer moduli are higher. One may expect that higher deflection values imply softer materials and lower moduli. Although this is true, the thicknesses of the AC layer at these two test locations are substantially lower than the AC thickness at test location number 3. Lower AC thicknesses cause higher deflections. The above scenario implies, as expected, that the deflection of a pavement structure is a function of both the modulus and the thickness of the AC layer. Variations in any one variable lead to variation in the measured deflection. At test location number 24 and 26, the thickness of the AC is the main cause of higher deflections.

4. The variation in the values of the roadbed modulus is well explained by the variation in the measured deflection at sensor 7 (located at a lateral distance of 60-inch from the center of the applied load). It can be seen from Figure 6.3 that a higher roadbed modulus corresponds to a lower deflection at sensor 7.

5. For test location numbers 12, 15, 17, and 19, the values of the base modulus is higher than that at location number 3, while they are lower for test location numbers 24 and 26. In order to probe further into the variation of the base modulus, the percent differences between the deflections measured at all seven sensor at test location number 3 and those at locations 15, 17, 19, and 26 are shown in Figure 6.4. It can be seen that for test location number 17, the deflections measured by all sensors are appreciably lower than those measured at other test locations. In particular, the deflections of the middle sensors (sensors 2 to 5) are much lower, which indicate the presence of a stiffer (higher modulus) middle layer (in this case, the base layer). Inversely, for test location number 26, the deflections measured by all middle sensors are appreciably higher, which indicate softer base layers.

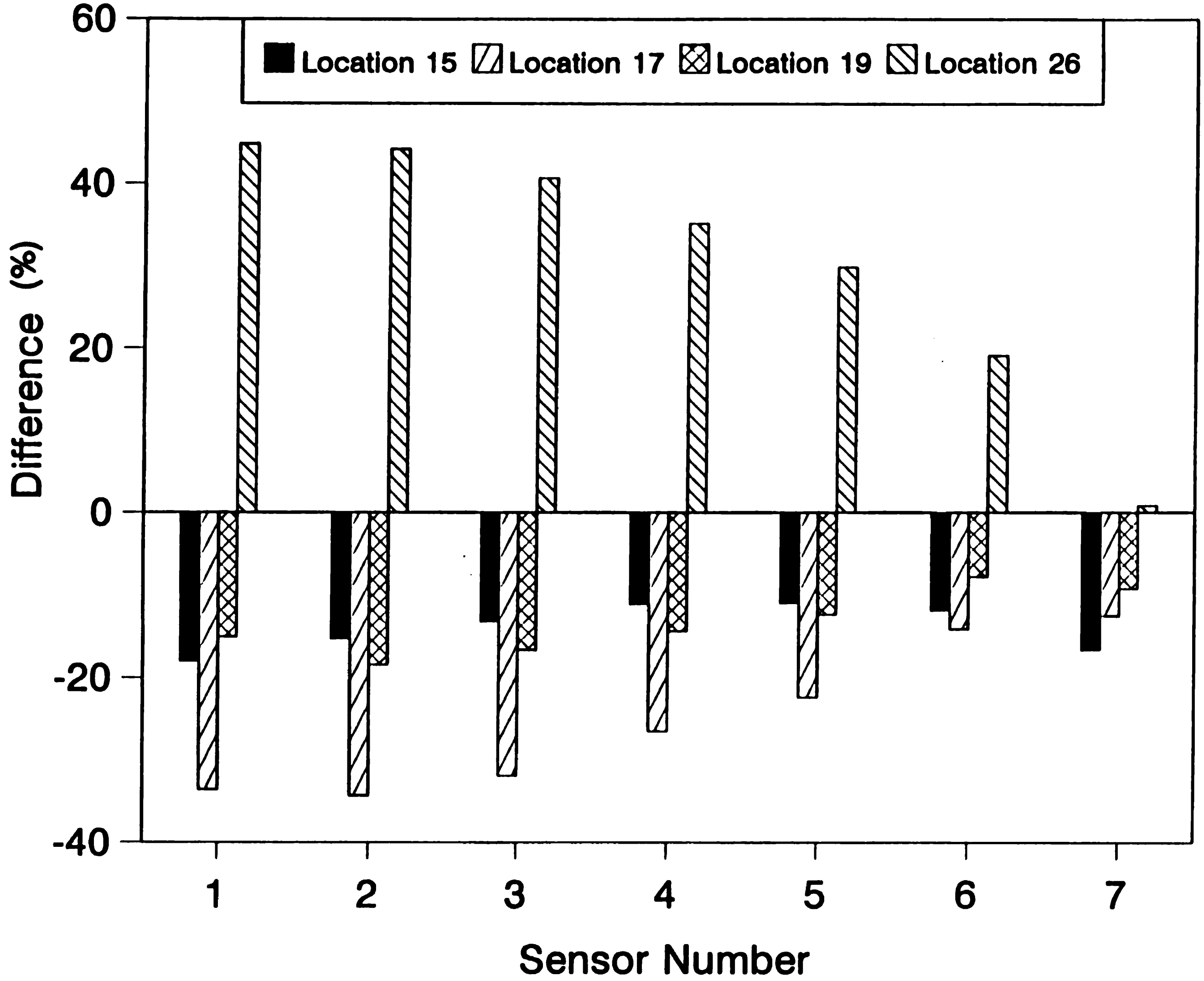

Figure 6.4. Variation of deflections for all seven sensors along pavement section MSU07F.

The above observations indicate that the variations in the backcalculated layer moduli along pavement section "MSU07F" are very reasonable and consistent with the variations of the measured pavement deflections and AC thicknesses.

6.4.2 Flexible Pavement Section MSU19F - Uniform Deflections

This pavement section consists of a total of eight test sites divided into two groups (each consists of four sites) which are separated by a one mile stretch of pavement. The pavement is in a good condition with no significant distress. The AC thickness within the section varies from 5.1 to 8.5-inch. The deflections measured by all seven sensor locations are shown in Figure 6.5 and the values of the backcalculated layer moduli are provided in Table 6.7. This pavement section is representative of those sections whose behavior is more or less uniform along the length of the pavement.

Examination of the values of the backcalculated layer moduli listed in Table 6.7 indicates that:

1. The AC modulus varies from a low value of 215,731 psi at test location number 45 to a high value of 391,656 psi at test location number 10 (a factor of about 1.82).
2. The base modulus varies from a low value of 56,760 psi at test location number 45 to a high value of 82,908 psi at test location number 17 (a factor of about 1.46).
3. The roadbed modulus varies from a low value of 40006 psi at test location number 40 to a high value of 50,857 psi at test location number 15 (a factor of about 1.27).
4. The maximum values of the root-mean-square of the errors between the seven calculated and measured deflection values is 3.54%.

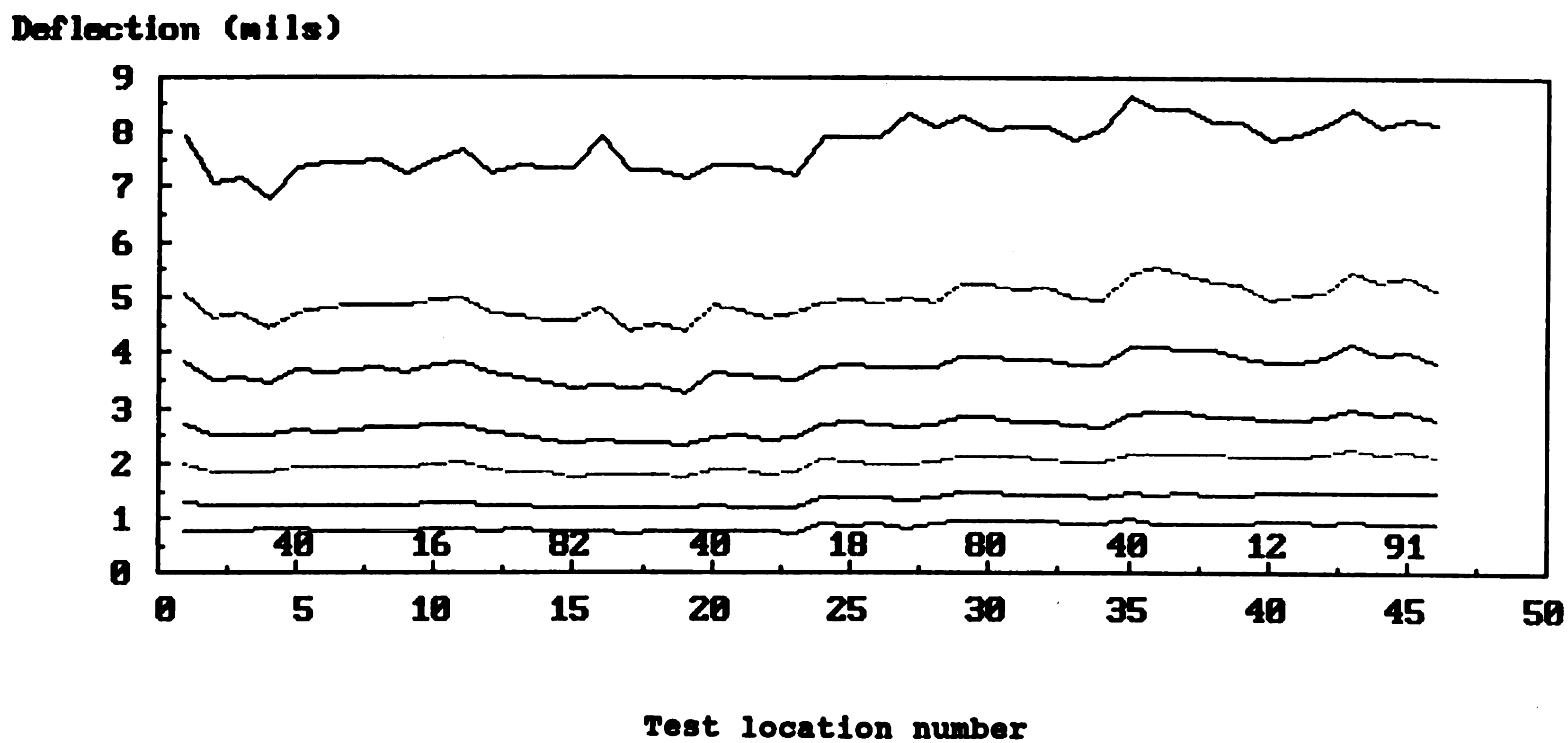

Figure 6.5. Deflection profile for pavement section MSU19F.

Table 6.7. Backcalculated moduli for pavement section MSU19F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
19113011	3	346848	71962	50190	2.99
19118311	8	321499	68224	48739	3.41
19121611	10	391656	69329	46408	2.03
19128211	15	271118	77296	50857	3.02
19131111	17	257348	82538	50809	1.71
19132312	19	253084	82908	50598	3.39
19137411	22	364790	76309	50215	3.19
19141811	25	258563	72029	43146	2.57
19148111	31	317965	76003	40839	3.22
19150911	33	263608	74919	41806	3.54
19158411	38	250851	77889	49048	2.45
19161211	40	227458	72986	40006	3.40
19169111	45	215731	56760	41228	3.39

As for the previous pavement section, the values of the backcalculated layer moduli provided in Table 6.7, the measured pavement deflections at sensors 1 (the inner-most sensor) and 7 (the outer-most sensor), and the AC thickness for test location number 3 were designated as datum values and the percent differences between the datum values and the corresponding values at other test locations were calculated and are depicted in Figure 6.6. Examination of the figure indicates that:

1. The variations in the measured deflections are small and so are the variations in the values of the backcalculated layer moduli.
2. For test location number 8, as it is expected, the deflection measured at sensor 1 and the AC layer thickness are higher which lead to a lower AC modulus.
3. For test location number 10, the deflection measured at sensor 1 is high because of the AC thickness which is substantially lower than that at location 3 (a difference of about 10 percent). Hence, for this test location high deflection is caused mainly by a lower AC thickness rather than by a softer AC layer.
4. An apparent discrepancy exists for test locations 15, 17, and 19. For example, at test location number 17, the AC thickness is almost the same as that at the datum (test location number 3) and the difference in the measured deflection at sensor 1 is moderately low, and yet an appreciable difference in the backcalculated AC modulus can be noted. This apparent discrepancy led to a further examination of the measured deflection records. It was noted that the NDT tests along this pavement section was conducted at noon time on a sunny summer day. Further, the temperature of the AC surface (which was recorded during the test) at test location number 17 was ten degrees higher than that recorded at test location number 3. Hence, the lower AC modulus is mainly due to the higher temperature of the AC. In order to confirm this, the percent differences between the deflections measured at all sensor and those measured at test location number 3 are plotted in Figure 6.7 for test location numbers 8,

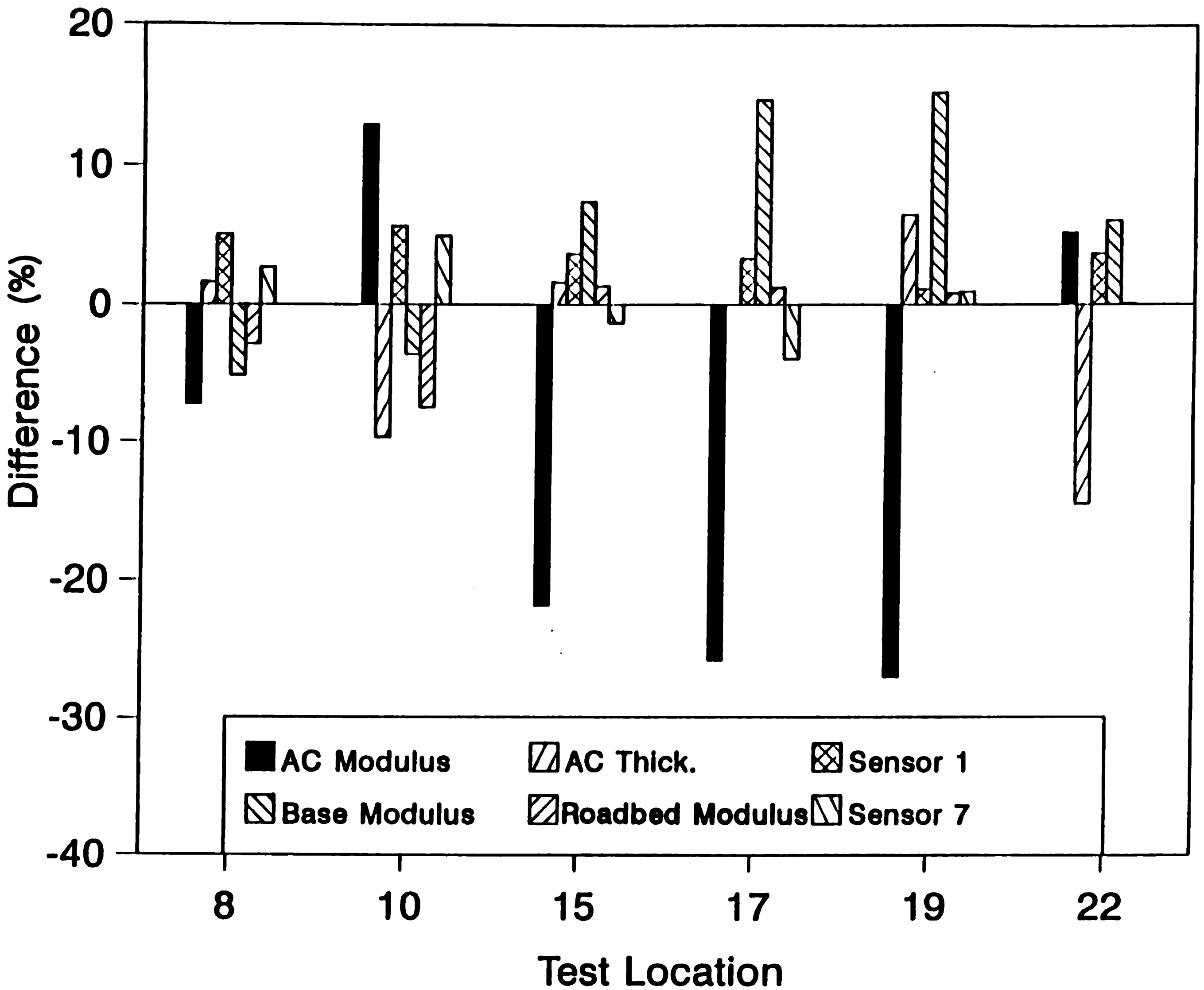

Figure 6.6. Variation of backcalculated layer moduli, the AC thickness, and the deflection of sensor 1 and 7 along pavement section MSU19F.

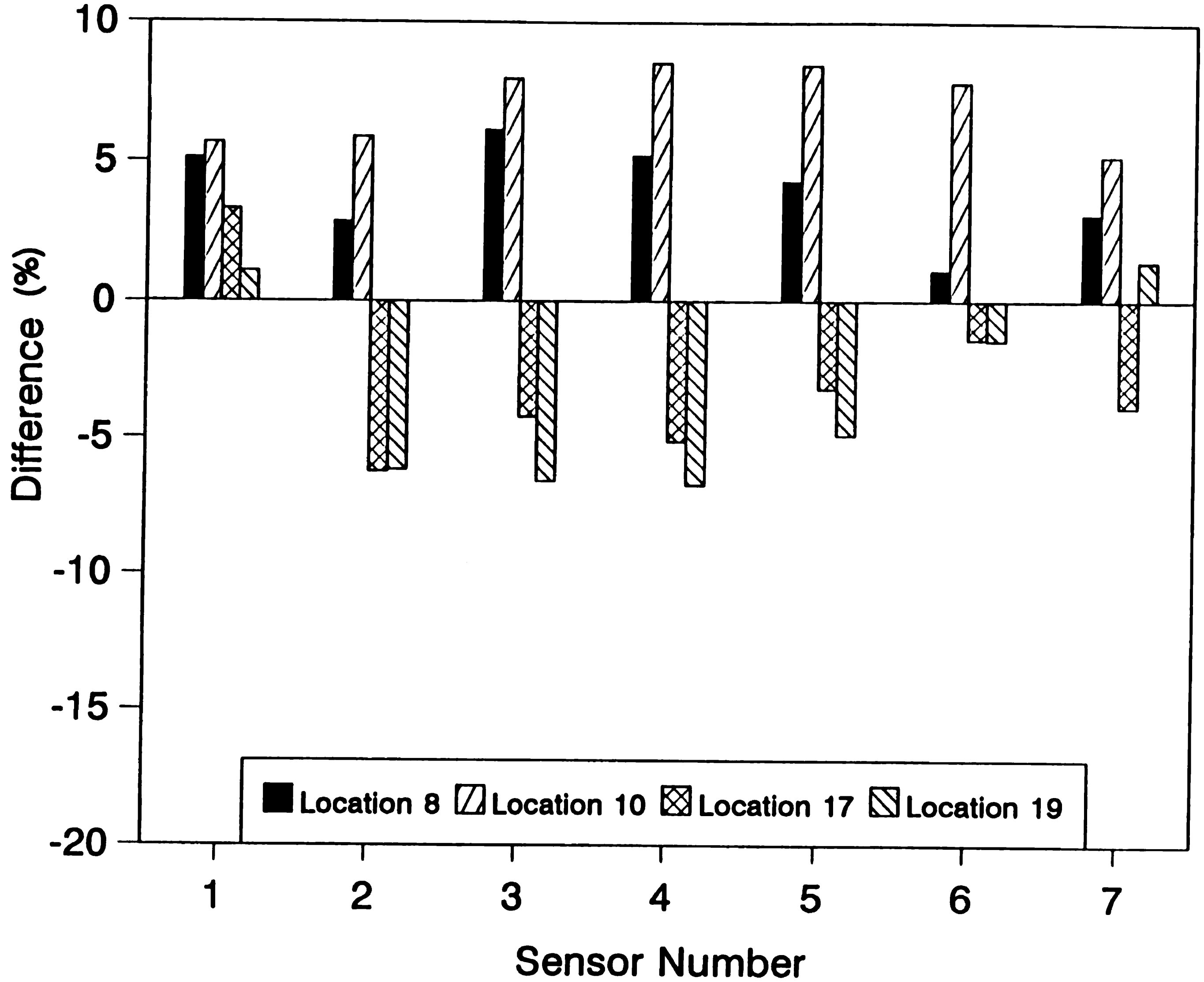

Figure 6.7. Variation of deflections for all seven sensors along pavement section MSU19F.

10, 17, and 19. It can be seen from the figure that for test location numbers 8 and 10, the deflections at all seven sensors are higher than those at test location number 3 which indicate softer materials. For test location numbers 17 and 19, however the deflection measured at sensors 1 is higher than that for test location number 3 and the deflections of all other sensors are lower. This trend is the signature of higher AC temperatures.

5. For test location number 22, the variation in the AC thickness has the largest impact on the measured deflection.

Once again, the above observations indicate that the variations in the backcalculated layer moduli along pavement section "MSU19F" are very reasonable and consistent with the variations of the measured pavement deflections and AC thicknesses.

6.4.3 Flexible Pavement Section MSU10F - Uniform Deflections

This pavement section has a total of three adjacent sites. Lane shoulder separation was the only noted distress; otherwise the pavement is in a very good state. The measured deflections are almost uniform along the length of the pavement section as shown in Figure 6.8. The typical deflection basin along the section is smooth as shown in Figure 6.9 and the deflection decreases gradually with increasing lateral distance.

The values of the backcalculated layer moduli are listed in Table 6.8. The variations in the base and roadbed moduli are consistent with the variations in the deflections measured along the pavement section. The variations in the AC modulus for the last three locations is higher than expected despite moderate variation in deflections. The thickness of the AC layer however, varied between 4.3 to 5.5-inch

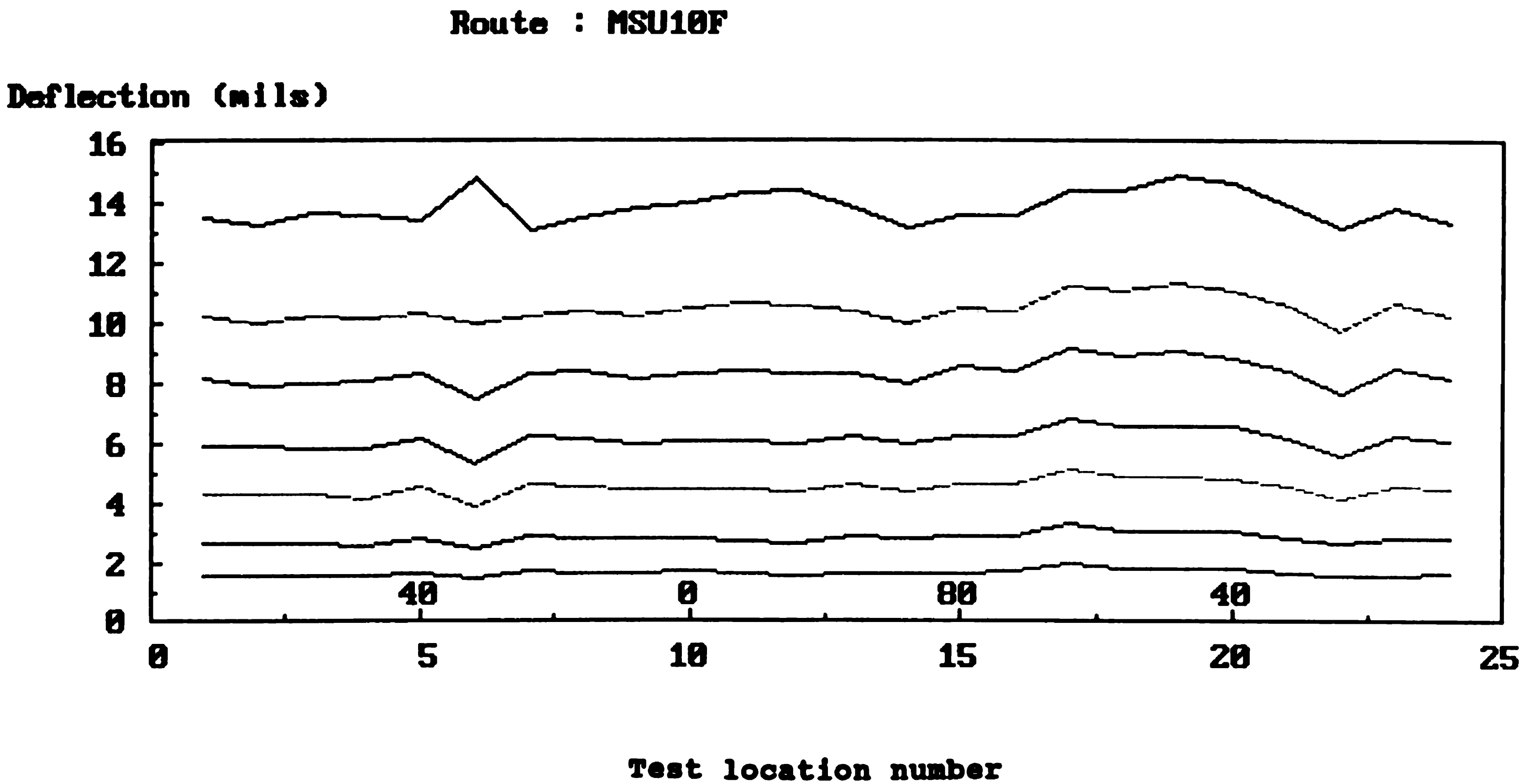

Figure 6.8. Deflection profile for pavement section MSU10F.

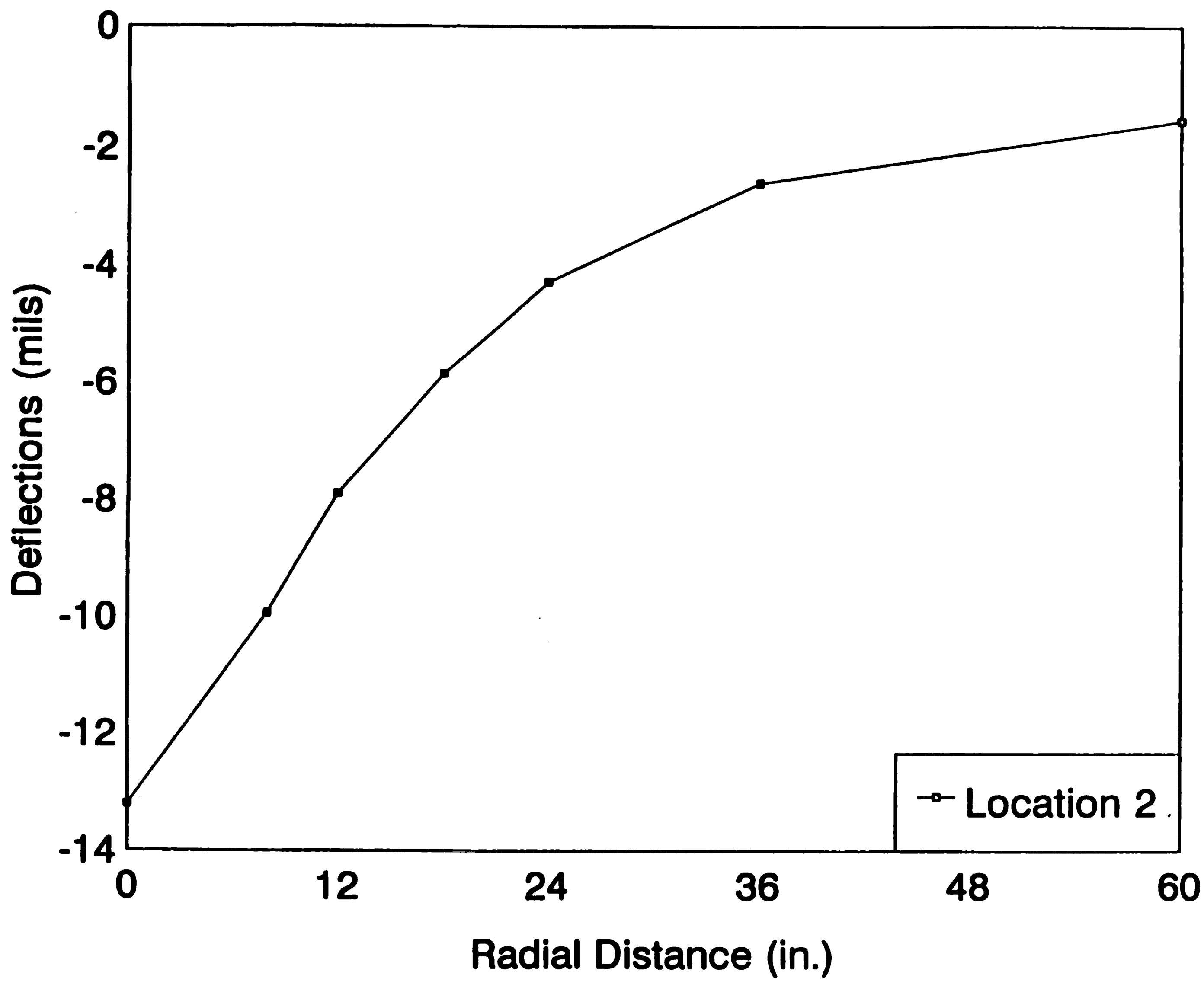

Figure 6.9. A typical deflection basin at test location No.2 of pavement section MSU10F.

Table 6.8. Backcalculated moduli for pavement section MSU10F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
10131011	2	440713	30169	24368	2.46
10131821	3	691145	31533	24070	1.74
10136641	8	630075	28078	23587	1.40
10121711	11	504689	29147	23422	1.72
10128121	16	594474	29148	22413	1.17
10111711	18	3743195	31735	20302	1.51
10117322	22	4226065	31953	20184	1.56
10118731	24	4525815	34564	21507	1.36

for the first four locations and from 2.3 to 2.4-inch for the last three locations. Given the almost uniform deflection along the pavement section, the abrupt and substantial change in the AC layer thickness implies a much stiffer AC as reflected in the backcalculated AC modulus values.

6.4.4 Flexible Pavement Section MSU13F - Variable Deflections

There are six test sites in this section located in two groups of three sites each which are approximately 0.4 mile apart. The first three sites have more distress (50 to 100 percent of each site with low to medium severity alligator cracking) than the last three sites (for which lane-shoulder separation is the only observed distress). Moderate variations in the measured deflections along the length of the section were observed and are shown in Figure 6.10.

The values of the backcalculated layer moduli are listed in Table 6.9. Variations in these values are consistent with those observed in the measured deflections and AC thickness.

6.4.5 Flexible Pavement Section MSU14F - Variable Deflection

This section consists of six adjacent test sites. The AC layer thickness varies between 2.4 and 3.1-inch. Hence, the section is placed in the thin pavement category. Low severity fatigue cracking was observed along almost all test sites. Some high severity transverse (temperature) cracks were also observed as well as low to medium severity lane-shoulder separation. In addition, poor drainage conditions were noticed along the sites.

Figure 6.11 depicts the considerable variation in the deflections measured at all sensors along the pavement section. This variation can be related in part to the high severity transverse cracks (except for test location numbers 22 and 26, all other test locations are in the vicinity of these cracks), and the poor but variable drainage

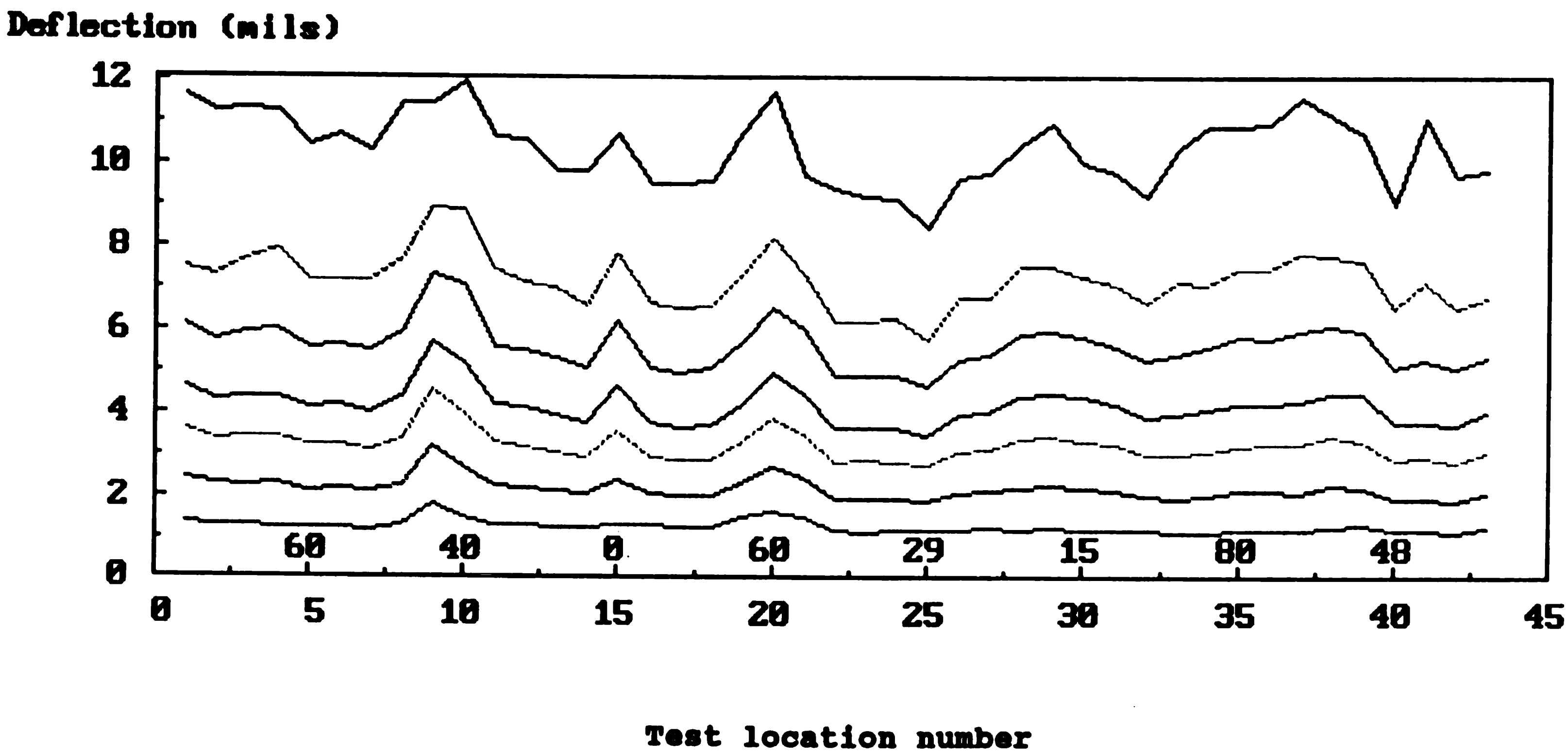

Figure 6.10. Deflection profile for pavement section MSU13F.

Table 6.9. Backcalculated moduli for pavement section MSU13F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
13117121	6	452995	48000	28829	1.34
13126311	12	801451	55078	27907	3.03
13132911	17	774716	65994	30021	1.33
13142311	24	7345505	62207	31031	1.56
13142922	25	668281	73135	31185	1.48
13151511	30	939812	50402	23574	1.09
13156621	34	573163	52322	26336	2.31
13164311	39	980012	44897	27913	1.86

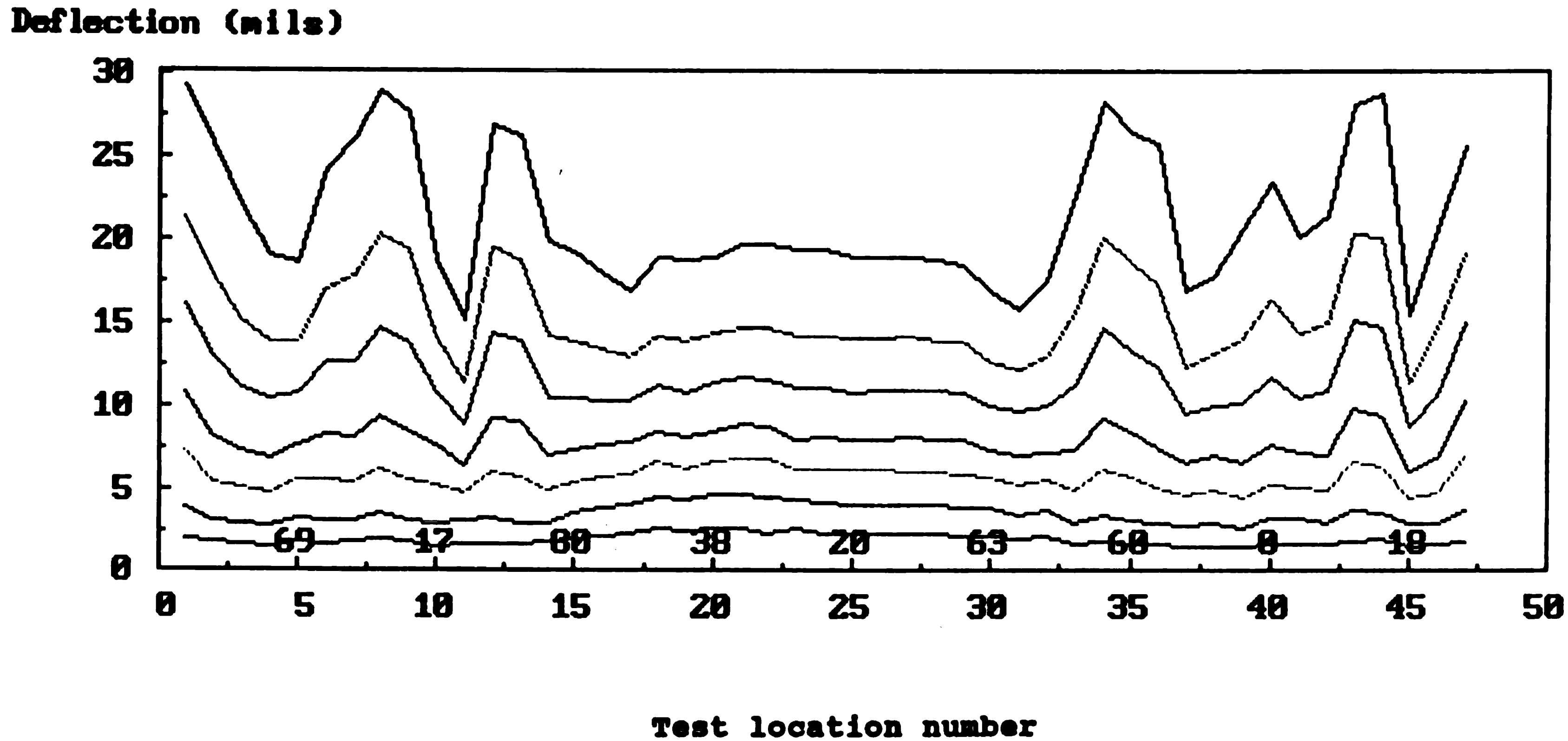

Figure 6.11. Deflection profile for pavement section MSU14F.

conditions along the section.

Table 6.10 provides a list of the values of the backcalculated layer moduli. It should be noted that some existing backcalculation routines (such as MODULUS 4.0) recommend the use of a fixed AC modulus for thin pavements. In this study, a fixed AC modulus was not used. All values were backcalculated using the MICHBACK computer program. Nevertheless, the trends between the values of the backcalculated layer moduli and the measured pavement deflections and AC thicknesses are reasonable and consistent.

6.4.6 Flexible Pavement Section MSU29F - Variable Deflections

In this section there are two groups, each group consisting of three test sites. The groups are separated by a distance of 3.2 miles. Difficulties in the analysis of the measured deflection basins for this section led to a further investigation of its conditions. It was noted that at least a portion of the outer pavement lane (the traffic lane) and the shoulder are located on an old portland cement concrete (PCC) pavement. The section has 100 percent medium to high severity cracking (mostly a combination of fatigue and block cracking) which had been sealed. The first three test sites are in an uphill fill section, whereas the last three sites are in a downgrade. A wide variation in the drainage conditions along the section was also observed. Some of the driveways to existing homes along the road were blocking the natural water drains.

High variations in the deflections measured at all sensors were observed and are shown in Figure 6.12. The variations follow a specific trend. The deflections between test location numbers 1 and 28 are comparatively low and are interspread with approximately equally spaced high blips. The deflections between stations 28 and 40 are however, consistently high for the first few sensors and consistently low for the last ones. After station number 40, the deflections taper off to lower values. In

Table 6.10. Backcalculated moduli for pavement section MSU14F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
14113013	4	1817913	21620	23158	1.21
14121713	10	2658550	18887	22458	1.36
14137811	22	1748687	31124	9476	1.21
14143711	26	2073122	29577	13147	0.75
14148213	31	1976844	29702	17113	0.38
14151513	37	1217635	24865	22264	0.72
14155113	38	1568918	23277	20567	1.22
14161813	45	1187707	30570	20315	0.62

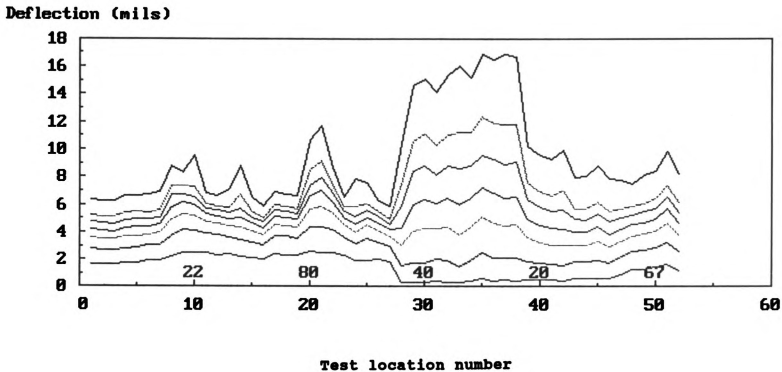

Figure 6.12. Deflection profile for pavement section MSU29F.

addition, the shapes of the measured deflection basins at the various stations are not consistent as depicted in Figure 6.13.

The values of the backcalculated layer moduli are provided in Table 6.11. It can be seen that the values are also highly variable and they follow the trends of the deflections along the road. For example, the values of the layer moduli are consistently high for all locations where the measured deflections are comparatively low. Further, the wide variation in the drainage condition has caused compatible variation in the base and roadbed moduli. Nevertheless, the backcalculated moduli values should be viewed with caution because of the presence of the PCC slab and the high distress conditions prevalent in the section.

6.4.7 Flexible Pavement Section MSU35F - Variable Deflections

This test section consists of six almost adjacent sites (the first three sites are only 0.1 mile away from the last three). The thickness of the AC measured from the cores varies from 5 to 7.3-inch. A combination of low severity fatigue and block cracking was observed along the entire test section. One to two high severity transverse cracks were also noted along each of the test sites.

Figure 6.14 displays the high variations in the deflections measured at all sensors along the pavement section. Except at a few locations, there is a recurring pattern of moderate deflection fluctuation which indicates the presence of equally spaced cracks. However, the shape of individual deflection basins was found to be consistent when compared to that of section 29F.

The values of the backcalculated layer moduli are listed in Table 6.12. The values are very much consistent with the variations in the measured deflections and AC thicknesses.

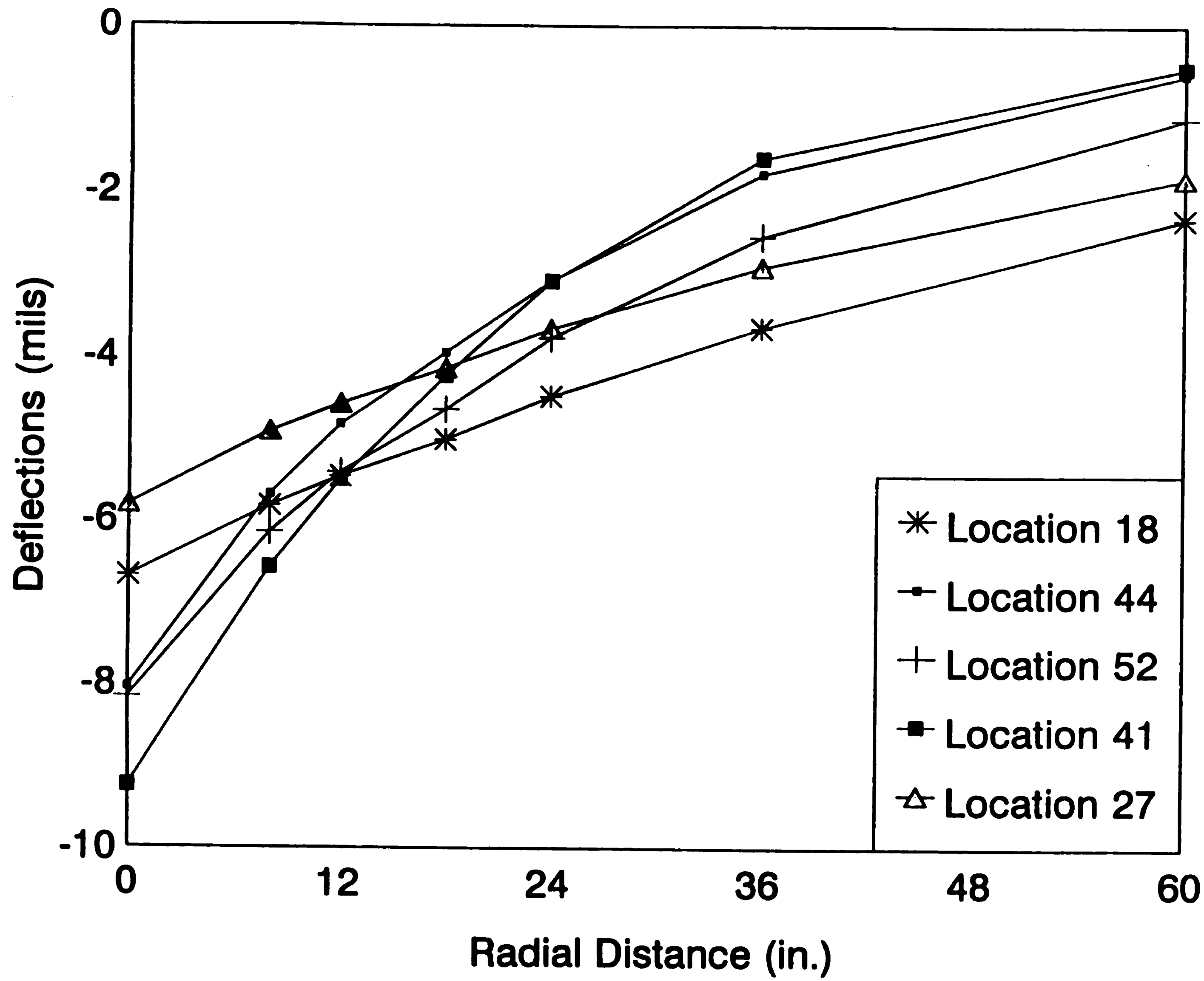

Figure 6.13. Typical deflection basins at test locations 18, 27, 41, 44, and 52 for pavement section MSU29F.

Table 6.11. Backcalculated modulus for pavement section MSU29F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
29111711	2	535056	30363	24682	1.04
29117721	6	471364	39576	20719	0.55
29134511	18	773636	11197	18025	0.40
29146911	27	719813	22708	21969	0.56
29172711	41	216579	6424	67347	1.89
29176321	44	283858	8025	46972	1.06
29186711	50	325096	27059	17819	0.60
29183321	52	336393	6417	38400	0.55

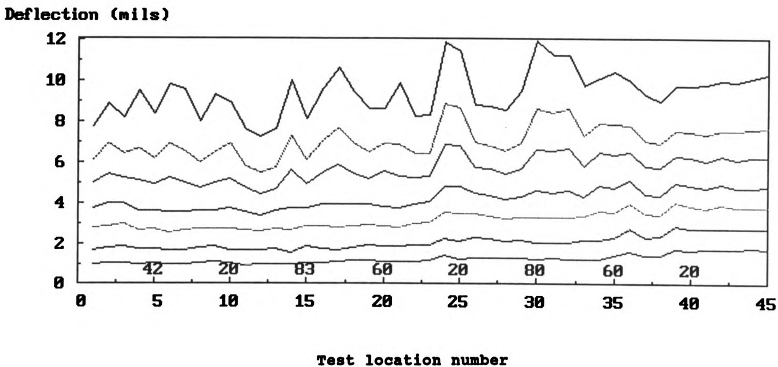

Figure 6.14. Deflection profile for pavement section MSU35F.

Table 6.12. Backcalculated modulus for pavement section MSU35F.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
35112211	3	685025	38914	36003	1.49
35114222	5	654514	51871	35720	1.05
35118931	8	580760	51552	35278	0.86
35123311	11	643382	46195	38451	0.88
35128321	15	862525	50443	34937	0.93
35132611	18	528756	45595	33322	2.91
35136221	22	718640	44710	32922	1.83
35143911	26	501497	36726	28339	1.54
35152311	33	463521	41488	29338	0.59
35159121	35	365961	36058	26835	3.01
35161411	39	472025	46138	21153	0.33
35167331	44	529678	56502	21318	1.53

6.4.8 Flexible Pavement Section MSU43F - Variable Deflections

This test section has a total of three groups, each consists of three test sites. The groups are separated by distances of 0.3 and 1 mile. The pavement section is characterized by very high rut depths, especially in the first three sites. Low severity cracking was also observed along the entire section.

Figure 6.15 depicts the high variations in the measured deflection profiles along the pavement section. It can be seen that continuously reoccurring blips of high deflections were measured. The shape of individual deflection basins however, is mostly smooth and consistent.

The backcalculated layer moduli listed in Table 6.13 have moderate variations in the base and roadbed moduli and high variation in the AC modulus. These variations are very much compatible with those of the measured deflections. For example, the deflections measured at test station number 29 are relatively higher than those at station 30. Consequently, the values of the backcalculate layer moduli at station number 29 are lower than those at station number 30.

6.4.9 Composite Pavement Section MSU01C - Variable Deflections

The eight test sites of this section are located in two groups having five and three sites, and are located 1 mile apart. Beside edge cracking, the main distress observed along this section is in the form of high severity sealed transverse cracks (reflective cracking). This section has very heavy truck traffic.

Figure 6.16 depicts the high variations of the measured deflection profile along the road. The towering blips of high deflections are indicative of the presence of high severity transverse cracks which are closely spaced in certain locations. The shapes of the measured deflection basins are not consistent and are not smooth as shown in Figure 6.17.

The values of the backcalculated layer moduli are listed in Table 6.14. The

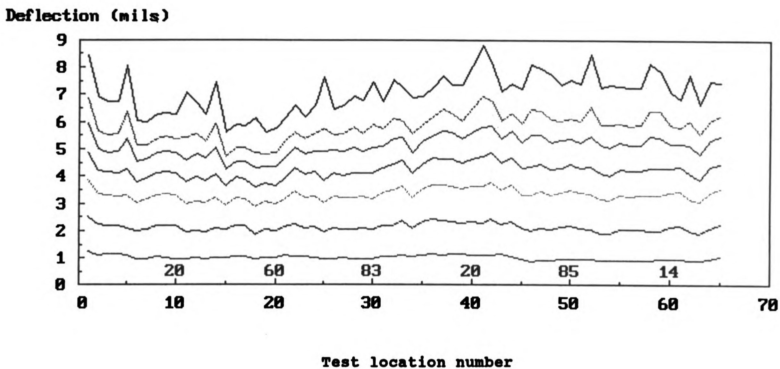

Figure 6.15. Deflection profile for pavement section MSU43F.

Table 6.13. Backcalculated moduli for pavement section MSU43F.

FWD Test Code	Test Location No.	Backcalculated Results (psi)			RMS Error (%)
		AC	Base	Roadbed	
43111711	2	580509	22886	36221	1.35
43121811	9	873327	13926	46995	2.26
43131911	16	890107	14774	44970	1.65
43141611	23	875335	17540	41522	3.06
43148313	29	597983	19410	42198	1.28
43148311	30	422440	24850	38947	1.47
43152311	33	713238	17787	36701	1.10
43161411	39	483639	20058	34977	2.12
43178511	50	589905	19907	39101	1.07
43178112	51	485790	20296	39462	1.60
43188111	57	531447	21100	40511	1.26
43191411	60	564445	19098	39715	1.64

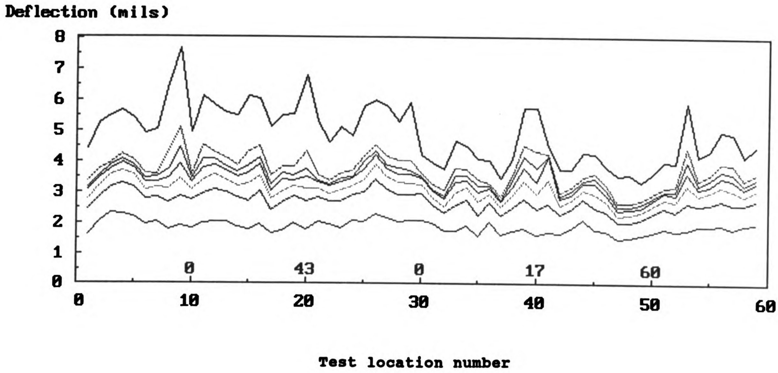

Figure 6.16. Deflection profile for pavement section MSU01C.

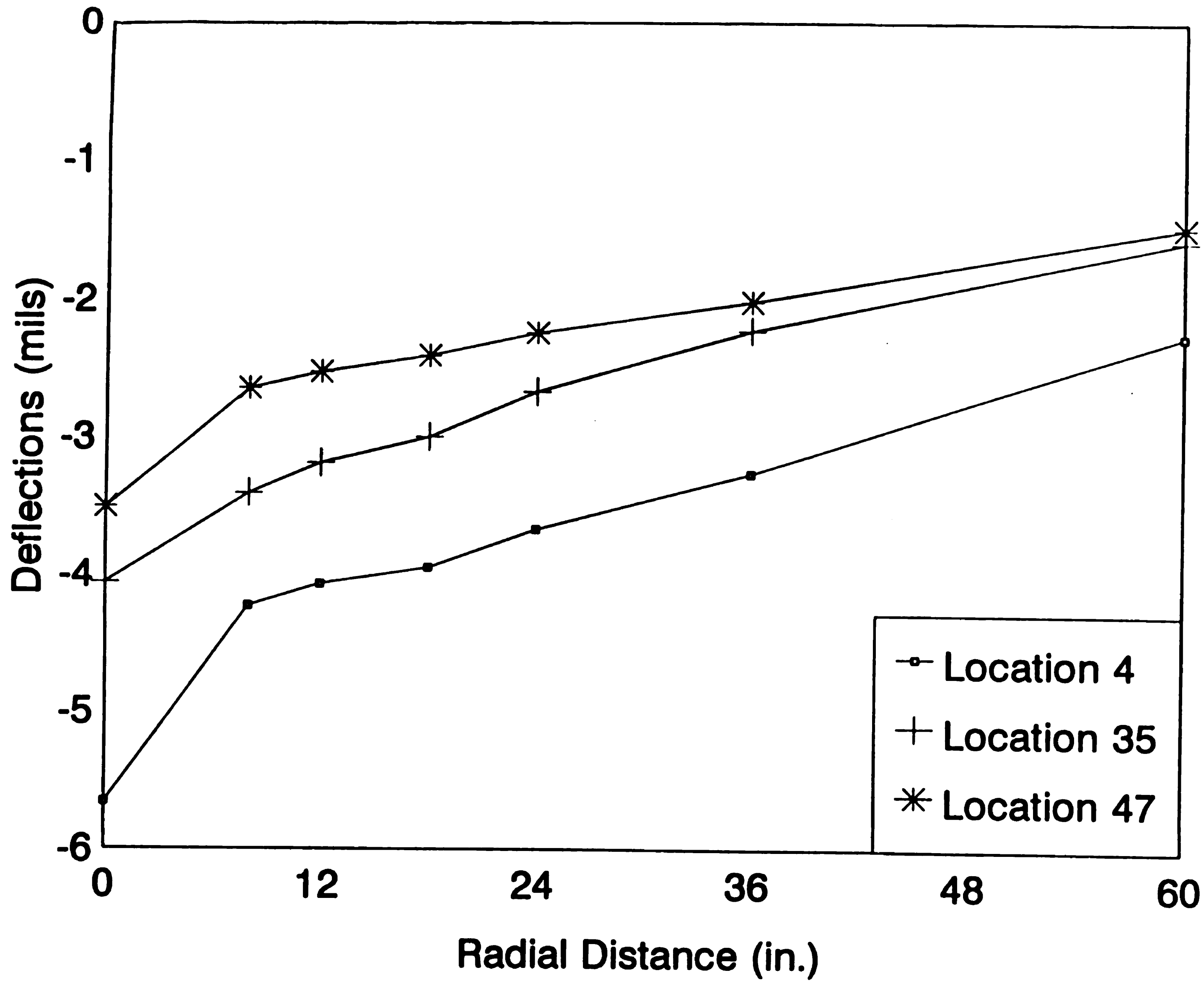

Figure 6.17. Typical deflection basins at test locations 4, 35, and 37 for pavement section MSU01C.

Table 6.14. Backcalculated moduli for pavement section MSU01C.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
01284611	4	337073	5406975	16271	2.28
01272411	12	319451	2636877	18653	2.36
01261611	17	282635	4694486	23958	1.01
01264221	20	195181	1520576	22012	0.92
01251611	25	302239	3191544	18924	1.39
01240411	31	762348	5040840	18107	0.95
01248931	35	1012610	524600	25662	1.12
01232611	42	725415	3293690	22761	1.33
01221711	47	610169	4900569	35123	1.57
01214111	56	523795	1206742	19998	1.16

variations of these values along the pavement section are consistent with those of the measured deflection profiles. For example, the measured deflections at station location number 47 are relatively lower than those at station 17. Consequently, the backcalculated layer moduli are higher at the former station than at the latter one.

6.4.10 Composite Pavement Section MSU05C - Variable Deflections

There are six test sites in this section located in two groups of three sites each which are approximately 1.0 mile apart. Lane-shoulder separation and medium to high severity transverse cracks (reflective cracking) were observed along the entire section. The transverse cracks are equally spaced and are occasionally connected by longitudinal cracks.

Figure 6.18 displays the extremely high variations in the measured deflection profile along the road. The equally spaced high deflection blips in the figure correspond to the equally spaced high severity transverse reflective cracking.

Table 6.15 provides a list of the values of the backcalculated layer moduli. Variations in the moduli values correspond to the variations in the measured deflection basins. For example, the measured deflections at location number 28 are much higher than those at locations 8 and 17. Correspondingly, the values of the backcalculated moduli of the first station are lower than those of the latter two stations. Further, the MICHBACK computer program did not converge on any set of moduli values for station number 45. The reason for this is the irregular shape of the deflection basin. Figure 6.19 displays the deflection basins measured at four location numbers 45, 46 (located only three feet away from station 45), 43, and 28. It can be seen that the shape of the deflection basins vary substantially from one station to another. Such variations in the shape of the deflection basins and in the values of the peak deflections are the consequences of the state of distress of the PCC slab. Therefore, for any distressed pavements, the backcalculated layer moduli (even with

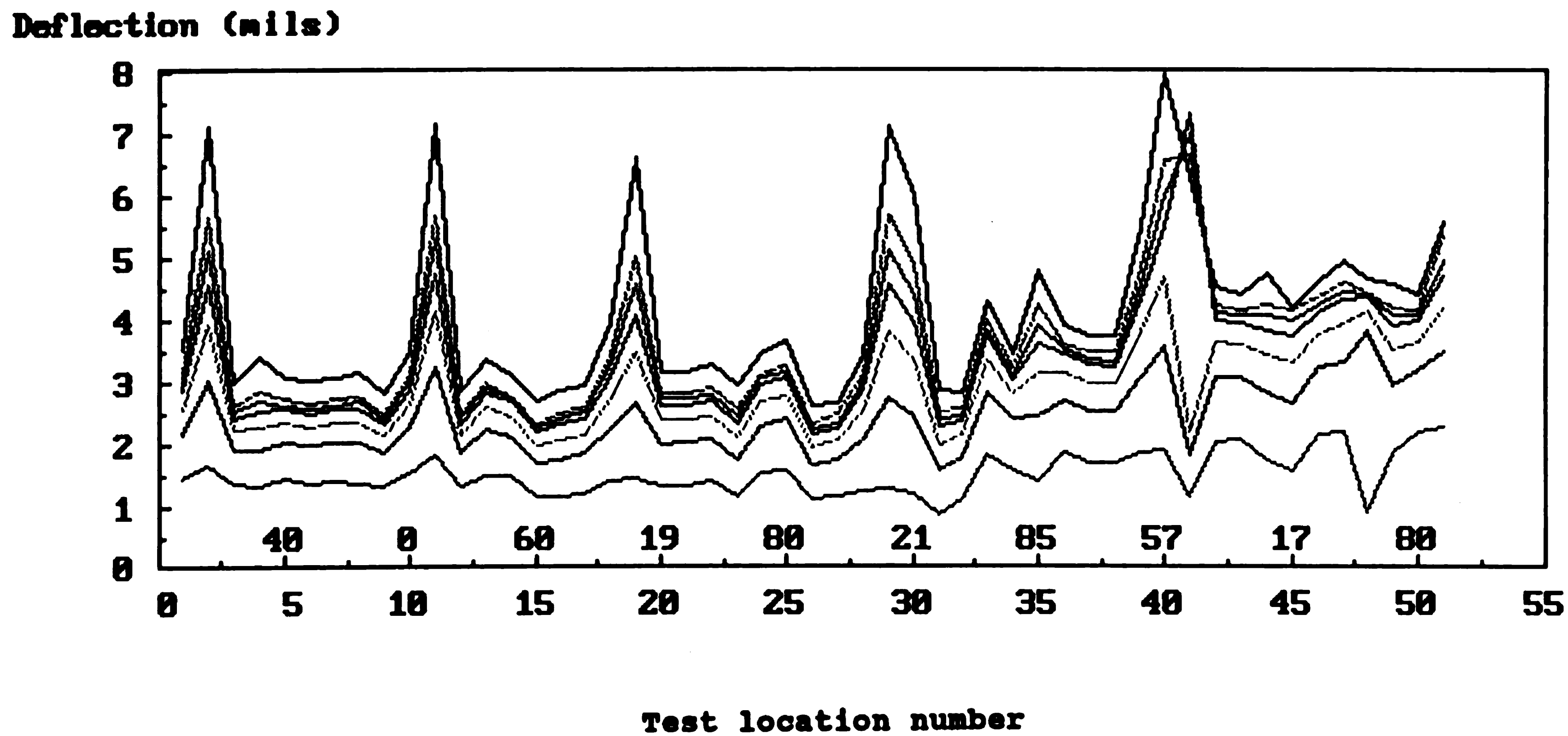

Figure 6.18. Deflection profile for pavement section MSU05C.

Table 6.15. Backcalculated moduli for pavement section MSU05C.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
05218611	8	1134797	8519459	30247	1.24
05229211	17	1292937	7494653	34832	1.11
05241411	28	1662644	4757409	33519	2.38
05258311	43	912615	7436017	20367	2.28
05261712	45	No Convergence			
05262012	46	1056789	7104805	19389	2.88

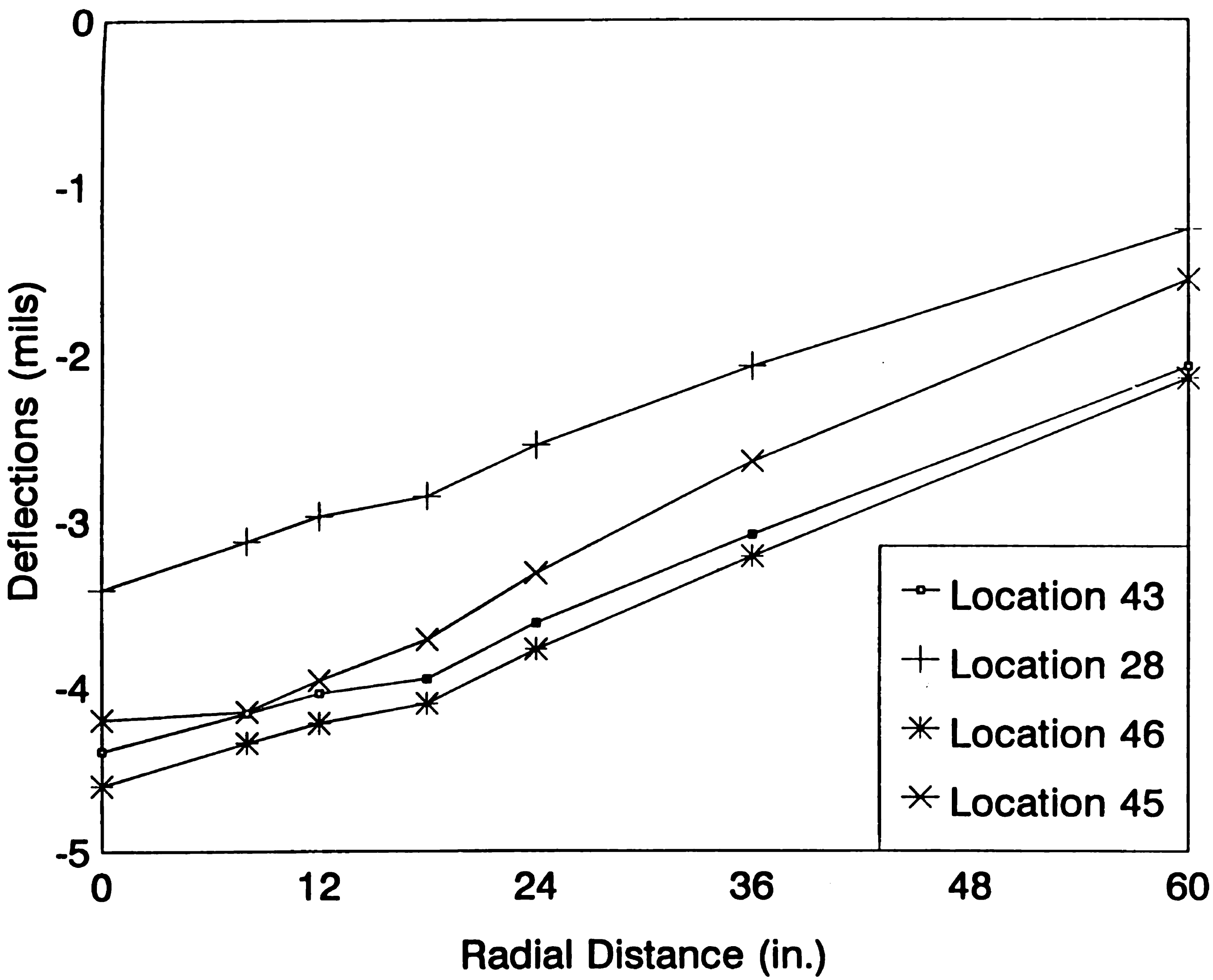

Figure 6.19. Typical deflection basins at test locations 28, 43, 45, and 46 for pavement section MSU05C.

small root-mean-square error) must be viewed cautiously.

6.4.11 Composite Pavement Section MSU08C - Variable Deflections

In this section, there are two groups separated by a distance of 2 miles. Each group consists of three adjacent test sites. There are 5 to 7 transverse (reflective) cracks in each site. The pavement is constructed on a 4-feet high embankment.

Figure 6.20 depicts the deflection profiles measured along the pavement section. It can be seen that the profile is characterized by fluctuation caused by the presence of reflective cracks. These variations in the measured deflections cause the similar variations in the backcalculated layer moduli provided in Table 6.16. Figure 6.21 depicts the irregularities of the deflection basins measured at various stations along the road. As for the previous composite pavement sections, these irregularities raise questions about whether the backcalculated moduli are reasonable.

6.4.12 Composite Pavement Section MSU04C - Variable Deflections

The three adjacent test sites of this section are apparently in good condition apart from the observed medium severity transverse (reflective) cracks. The section is located on a 4 to 5-feet high embankment. The thickness of the AC overlay varies along the pavement from 2.3 to 2.9-inch. During coring, a buried AC pavement was found underneath what was thought to be the roadbed soil. Because of the limitations of the MDOT coring equipment, the thicknesses of the various layers of the buried pavement structure could not be determined beyond the upper 3-inch.

Figure 6.22 shows the unique and moderate variations in the deflection profile measured along the road. It can be seen that the four lines representing the first four deflection sensors intersect and criss-cross each other frequently. Further, the prominent high deflection values in Figure 6.22 correspond to the presence of transverse cracks.

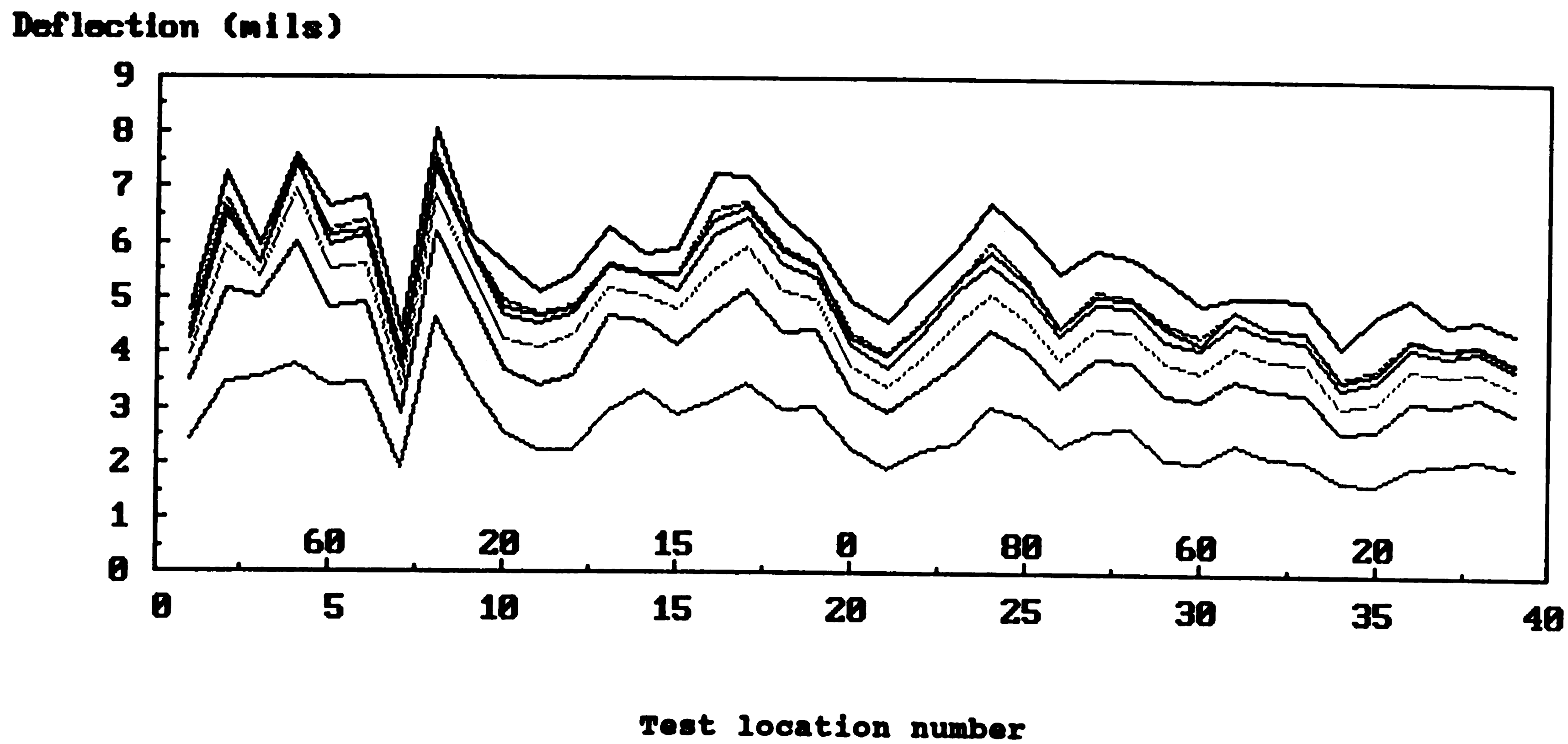

Figure 6.20. Deflection profile for pavement section MSU08C.

Table 6.16. Backcalculated moduli for pavement section MSU08C.

FWD test code	Test location No.	Backcalculated moduli (psi)			RMS Error (%)
		AC	Base	Roadbed	
08211211	2	1088087	2998202	11126	2.54
08213022	7	1707418	2689131	20807	1.67
08221011	9	705630	6592483	10862	3.58
08231511	15	1516300	2865745	13158	1.86
08241011	21	890504	3811759	20863	1.31
08251811	27	487404	4267114	14644	2.08
08260911	33	1350763	2221784	18871	1.31
08261622	34	801686	4523869	22724	2.14

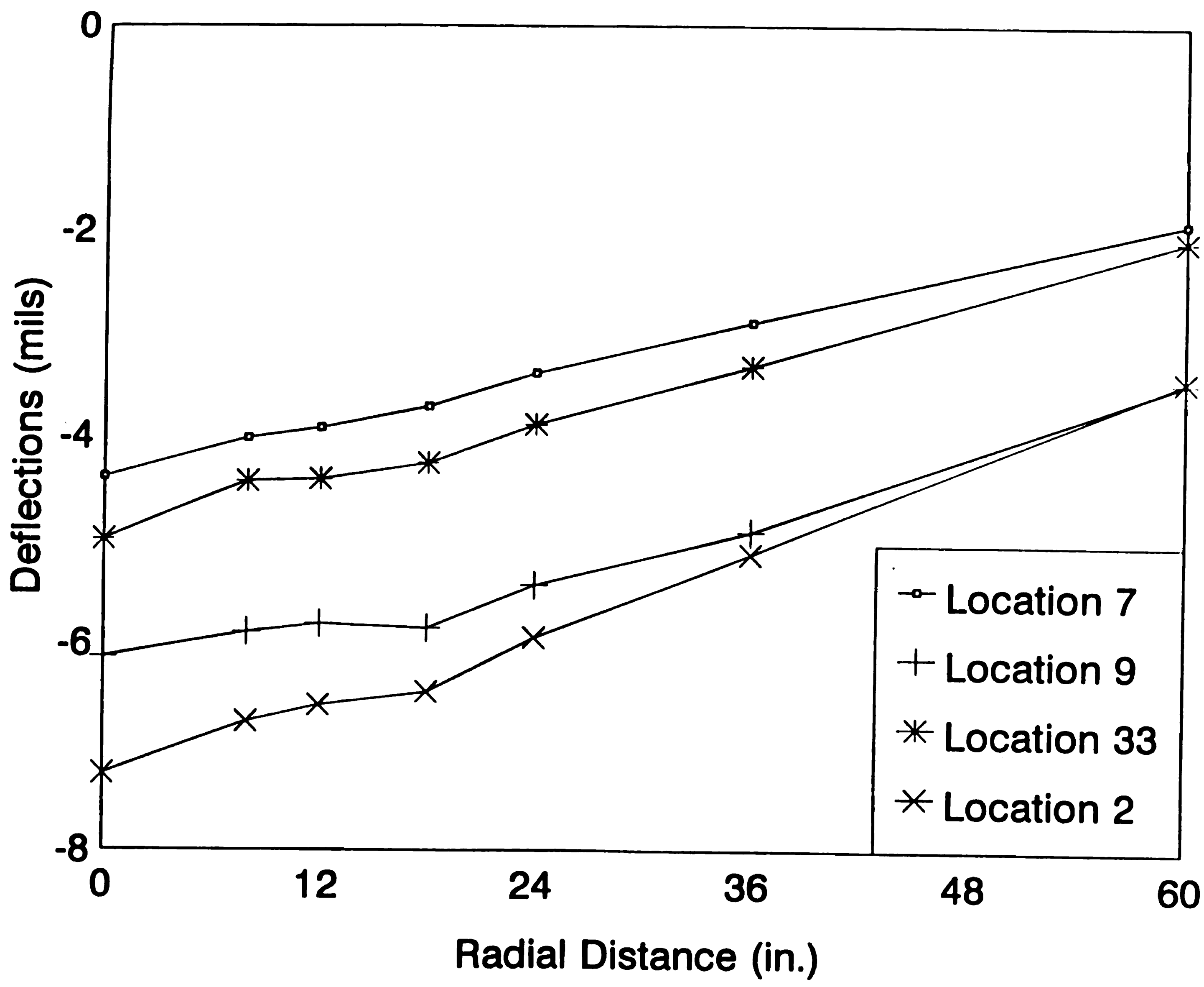

Figure 6.21. Typical deflection basins at test locations 2, 7, 9, and 33 for pavement section MSU08C.

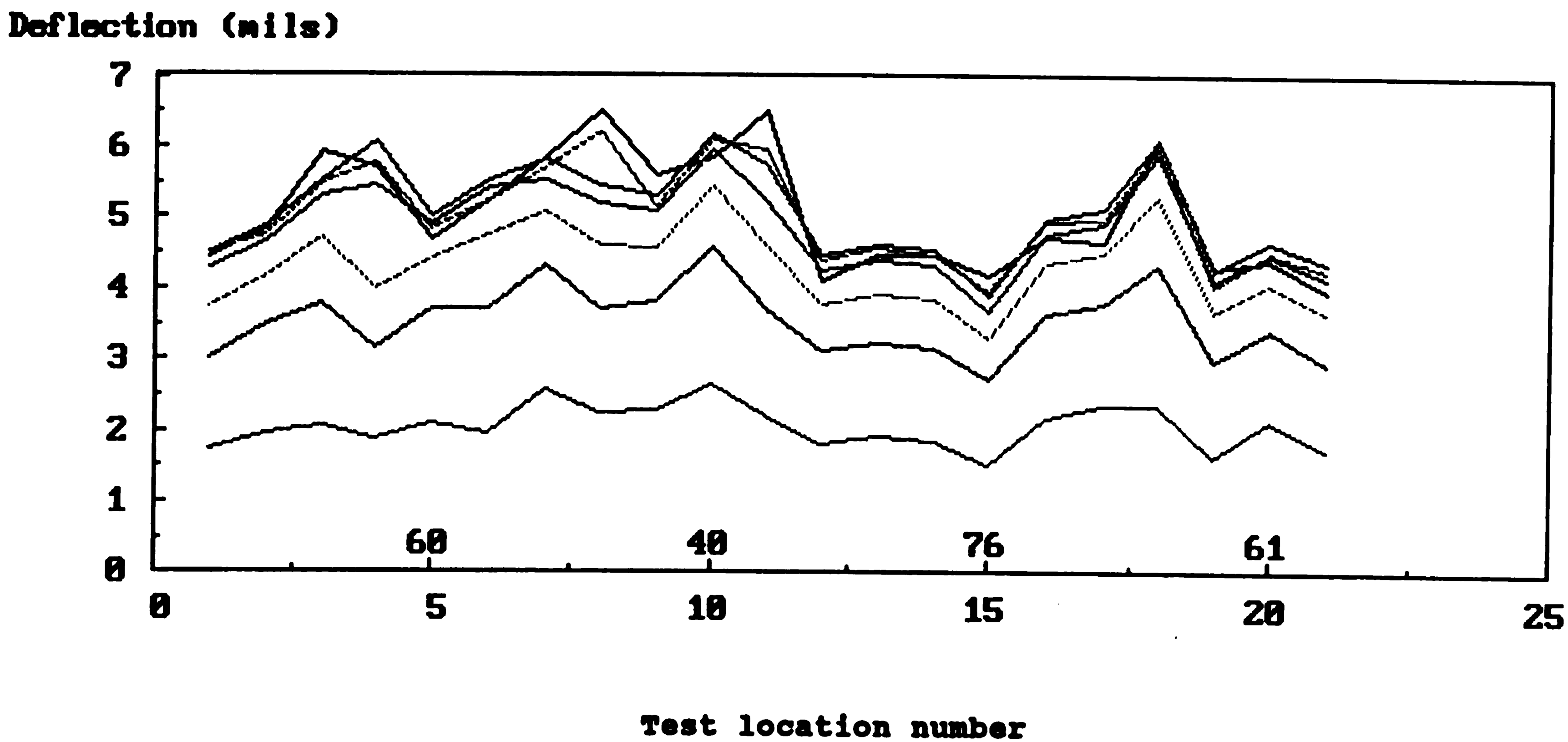

Figure 6.22. Deflection profile for pavement section MSU04C.

For this section, the MICHBACK computer program did not converge on any set of layer moduli for any of the test locations. The reason for this is the extremely irregular shapes of the measured deflection basins. Figure 6.23 depicts the deflection basins measured at three different locations. Beside the presence of the buried pavement structure, no other explanation can be offered at this time regarding the irregularities of the deflection basins.

6.5 BACKCALCULATION OF LAYER MODULI AT DIFFERENT LOAD LEVELS

During the summer of 1992, the FWD testing was expanded to include different load levels. All the non-cored pavement sections were tested at the standard target load of 9000 lbs. At each test location of the cored pavement sections, the NDT procedure was modified to include seven drops as follows:

1. The first drop 4500 lb (pavement seating).
2. Second drop 4500 lb (the deflection data was used for backcalculation).
3. Next three drops 9000 lb (the average deflection and the average load were used for backcalculation)
4. Seventh drop 16000 lb (the deflection data was used for backcalculation).

To avoid unnecessary repetitions, only the results of backcalculating the layer moduli for a few typical test locations along two flexible and one composite pavements are presented and discussed in this section. Table 6.17 provides a list of the types of the roadbed soils (the information is obtained from the 1970 MDOT Field Manual of Soil Engineering) encountered in the four pavement sections.

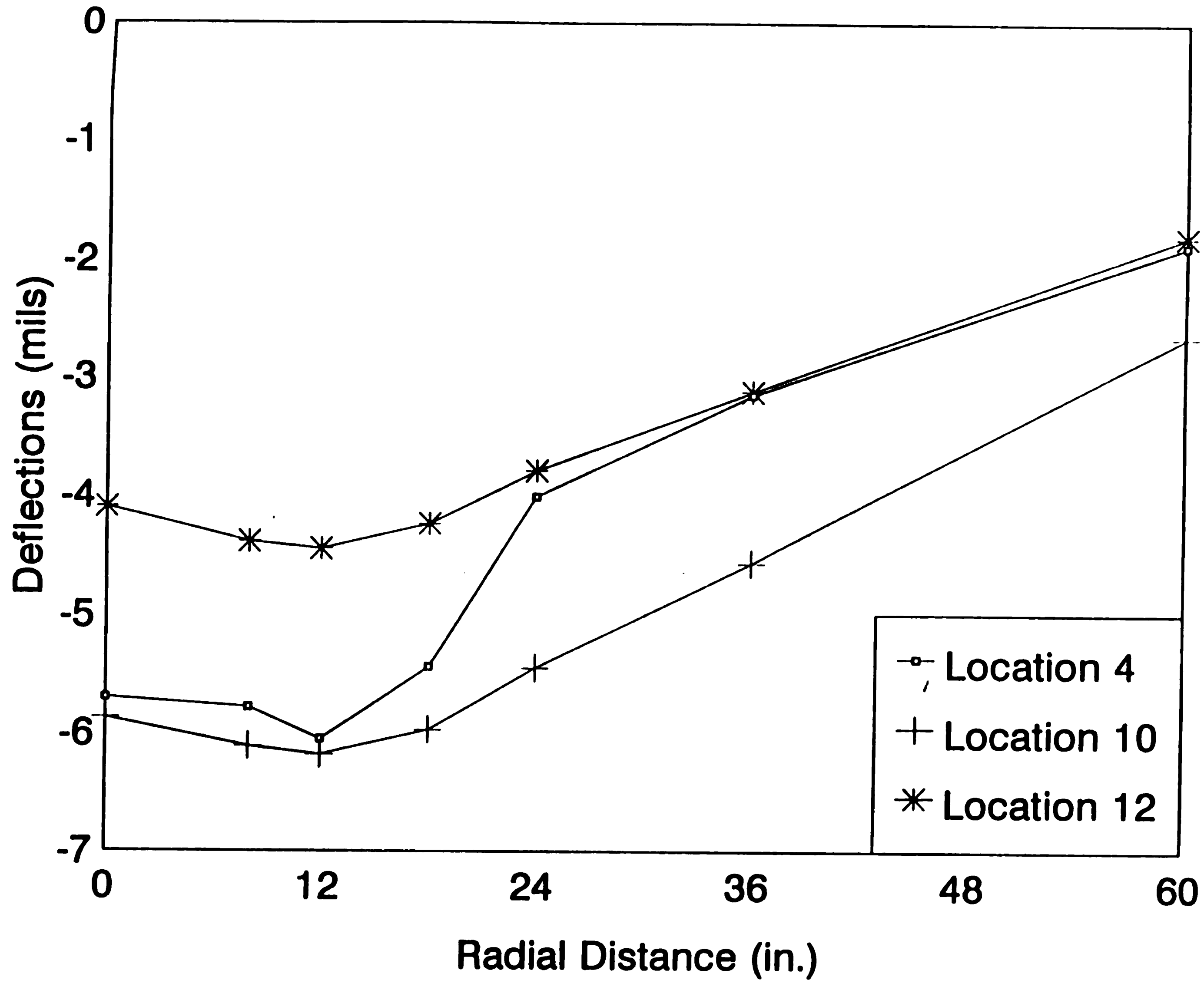

Figure 6.23. Typical deflection basins at test locations 4, 10, 11, and 12 for pavement section MSU04C.

Table 6.17. Roadbed characteristics of three pavement sections.

Pavement Section	Location County	Drainage	Roadbed soil type (percentage by weight)				
			Sand			Silt	Clay
			Fine	Medium	Coarse		
MSU19F	Otsego	Good	23	55	19	2	1
MSU35F	Cheboygan	Good	23	55	19	2	1
MSU01C	Lenawee	Poor	2.4	0.2	0.1	63.0	34.3

6.5.1 Flexible Pavement Section MSU19F - Variable Load Level

A brief description of this pavement is included in Section 6.4.5. The test section is located in Otsego County and the roadbed soil is granular in nature. The backcalculated layer moduli for the three load levels are listed in Table 6.18. The layer moduli backcalculated at the standard load of 9000 pounds are referred to in the remaining parts of this discussion as the reference values. The percent differences between the reference moduli and those calculated at the other two load levels were computed and are listed in Table 6.19.

Examination of the layer moduli calculated at the various load levels and the percent differences between those calculated at the standard load level and those at the other two load levels indicates that:

1. Relative to the reference moduli of the AC and base layers, the modulus values calculated at the low load level are consistently low, whereas those calculated at the higher load level are consistently high.
2. The variations in the backcalculated roadbed soil moduli are within a tolerable range.

First, the variations in the AC and base moduli backcalculated at the different load levels were investigated in relation to the measured deflection basins. Figures 6.24 through 6.27 depict four typical deflection basins measured at three load levels at test location numbers 19113011, 19118331, 19121611, and 19128221, respectively. The dotted lines in the figures represent the linearly extrapolated response of the pavement structure at the low (4648-pounds) and high (16080-pounds) load levels assuming that the pavement is a linear elastic system. That is, if the deflection basin measured at the standard target load level of 9000 pounds is used as a reference basin, and if the pavement response is purely linear with the load, then the measured basins at the low and high load levels should be exactly the same as those shown by the dotted lines. In reference to Figure 6.24, it can be seen that:

Table 6.18. Backcalculated moduli at various loads for pavement section MSU19F.

FWD test code	Load level (lb)	Backcalculated moduli (psi)						RMS Error (%)
		AC		Base		Roadbed		
		Modulus	Difference (%)	Modulus	Difference (%)	Modulus	Difference (%)	
19113011	4648	219734	-22.6	47231	-11.6	47879	1.82	4.07
	9277	283712		53437		47023		3.09
	16080	311783	9.9	67332	26.0	46359	-1.4	2.54
19118331	4634	217445	-12.9	40276	-18.8	51994	9.6	3.45
	9229	249738		49618		47452		3.64
	16004	281930	12.9	62106	25.2	46655	-1.7	2.76
19121611	4563	170882	-20.3	42985	-0.9	45234	1.5	3.11
	9208	214454		43385		44578		11.9
	16041	265320	23.7	59972	38.3	43719	-1.9	5.02
19128221	4630	207595	-19.4	51535	-14.5	51744	7.6	4.03
	9226	257593		60242		48097		3.70
	16032	319212	10.7	71365	18.5	47324	-1.6	3.69

Table 6.19. Differences between measured and linearly extrapolated deflection basins for pavement section MSU19F.

FWD test code	Load level (lb)	Difference in deflections at sensor locations (%)						
		1	2	3	4	5	6	7
19113011	4648	14.45	9.73	8.29	1.93	0.42	-2.53	-0.62
	9277							
	16080	-12.26	-11.88	-11.23	-8.09	-4.05	1.07	1.98
19118331	4634	13.55	10.56	7.81	4.84	1.42	-9.05	-13.23
	9229							
	16004	-12.67	-12.49	-10.94	-6.98	-3.35	1.71	1.22
19121611	4563	15.52	-18.71	7.18	2.48	0.02	-1.63	-7.53
	9208							
	16041	-13.44	-31.09	-9.87	-6.17	-3.82	0.91	2.51
19128221	4630	13.44	9.13	8.69	-2.25	-3.87	-1.41	-11.44
	9226							
	16032	-12.73	-11.97	-4.39	-4.45	-2.26	0.33	1.60

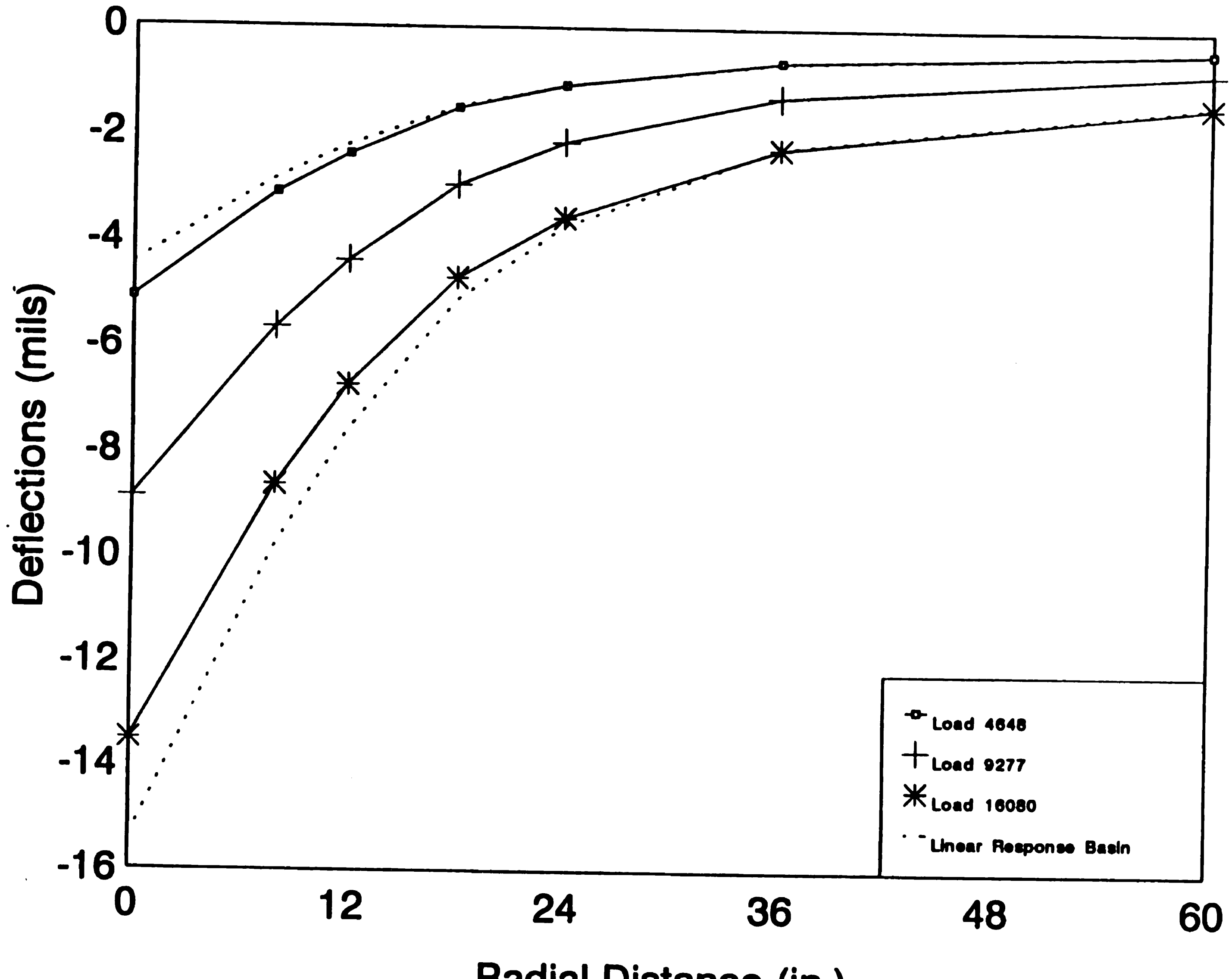

Figure 6.24. Comparison of measured and expected linear response deflection basins for pavement section MSU19F (FWD test code 19113011).

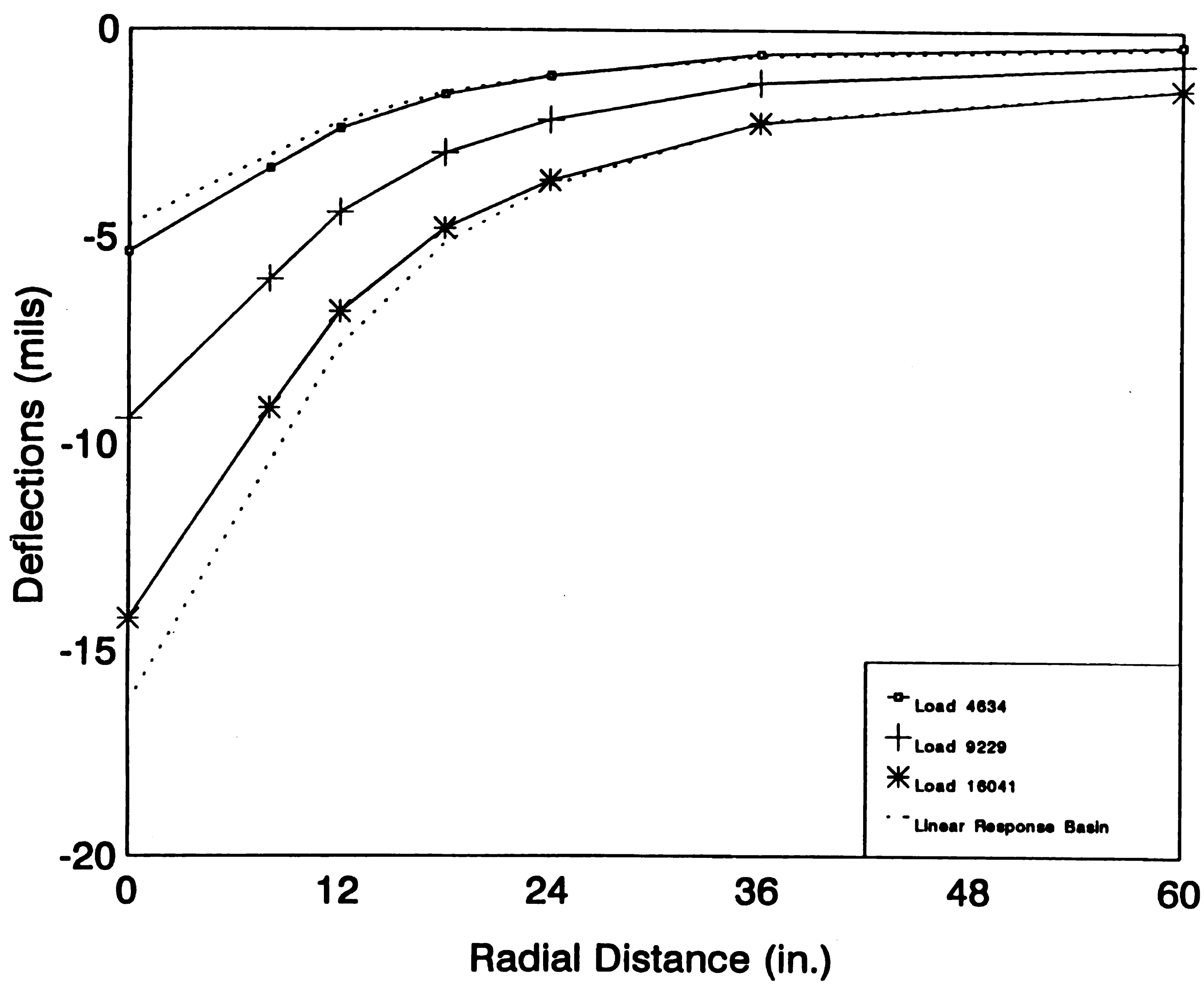

Figure 6.25. Comparison of measured and expected linear response deflection basins for pavement section MSU19F (FWD test code 19118311).

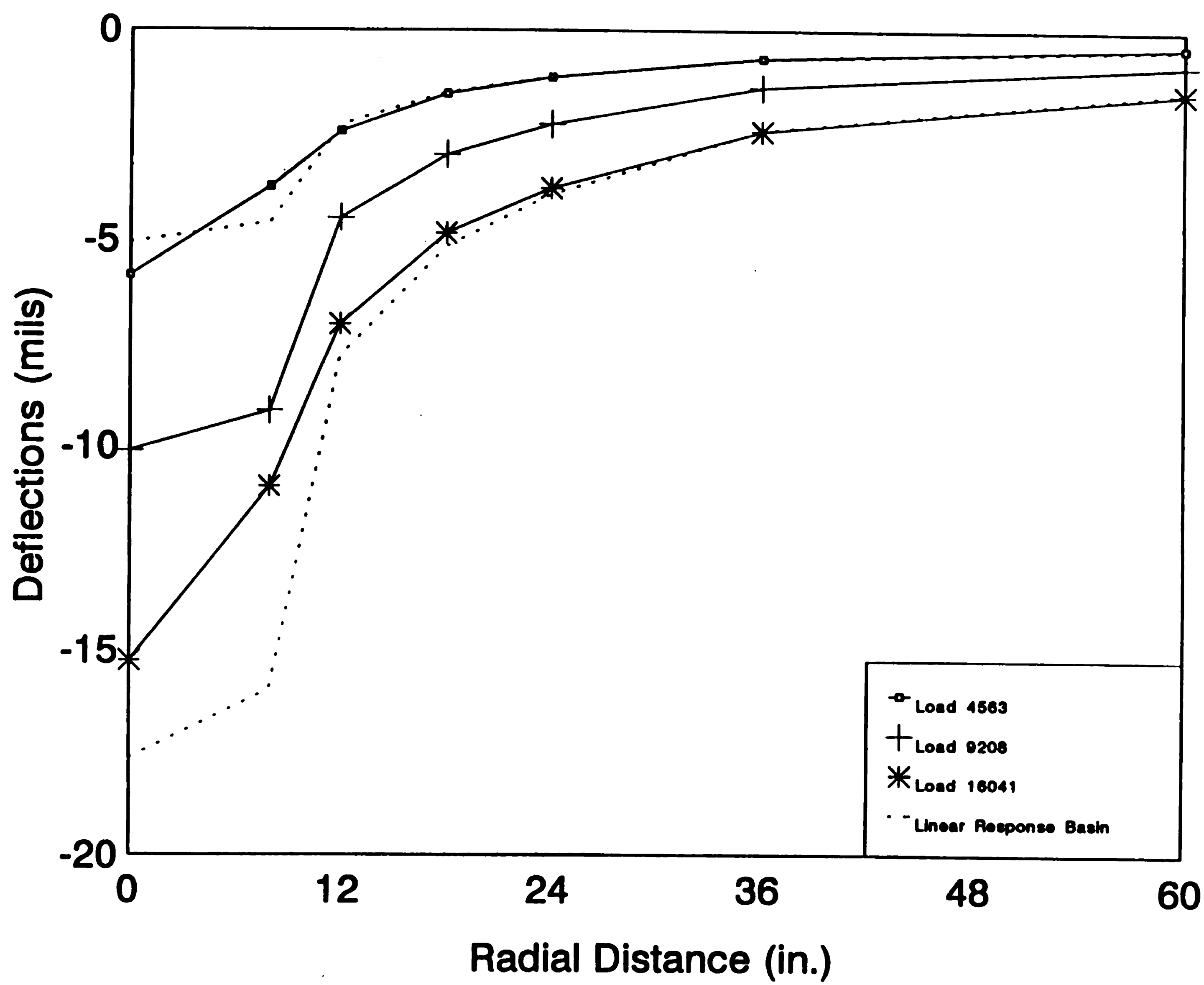

Figure 6.26. Comparison of measured and expected linear response deflection basins for pavement section MSU19F (FWD test code 19121611).

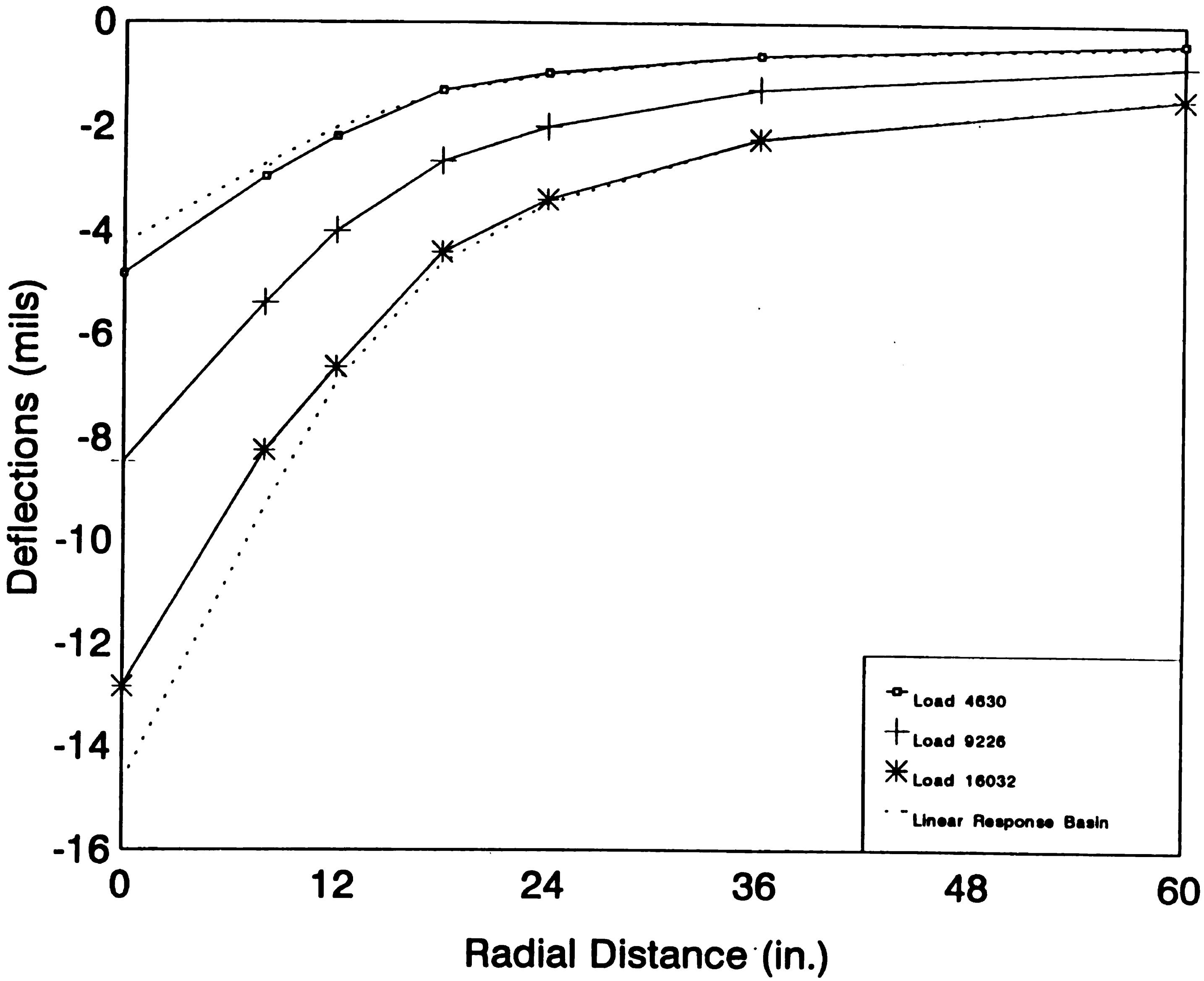

Figure 6.27. Comparison of measured and expected linear response deflection basins for pavement section MSU19F (FWD test code 19128221).

1. For the lower load level, the measured deflections at the first five sensors are higher than the linearly extrapolated responses, whereas, the measured deflections at the outer two sensors are almost the same as those of the linearly extrapolated responses.
2. For the higher load level, the measured deflections at the first five sensors are lower than the linear responses, whereas, the measured deflections at the outer two sensors are almost the same as those of the linearly extrapolated responses.

The above two observations imply that the increase in the pavement response is not linearly proportional to the load level. Discussion of this apparent non-linearity in the pavement response is presented in subsection 6.5.4. In the remaining part of this section, the trend between the backcalculated layer moduli and the measured deflection basins at the three load levels is discussed.

Table 6.19 provides a list of the differences between the deflection responses measured at the three load levels and those calculated by using a linear analysis. Figure 6.28 and 6.29 show the percent differences between the deflection measured at sensor locations 1, 2, and 7 due to the low and high load levels, respectively, and the corresponding linearly extrapolated deflections. Further, the percent differences between the layer moduli backcalculated at the same load levels and the reference moduli (obtained from Table 6.19) are also shown in the figures. Examination of the data listed in Tables 6.18 and 6.19 and shown in Figures 6.28 and 6.29 indicates that (for ease of discussion, consider test location number 19113011):

1. The measured deflection basins at the 4648 pounds load level are higher than the corresponding linearly extrapolated responses (Table 6.19). Consequently, the backcalculated AC and base moduli are lower than the reference values computed at the 9277 pounds load level (Table 6.18).


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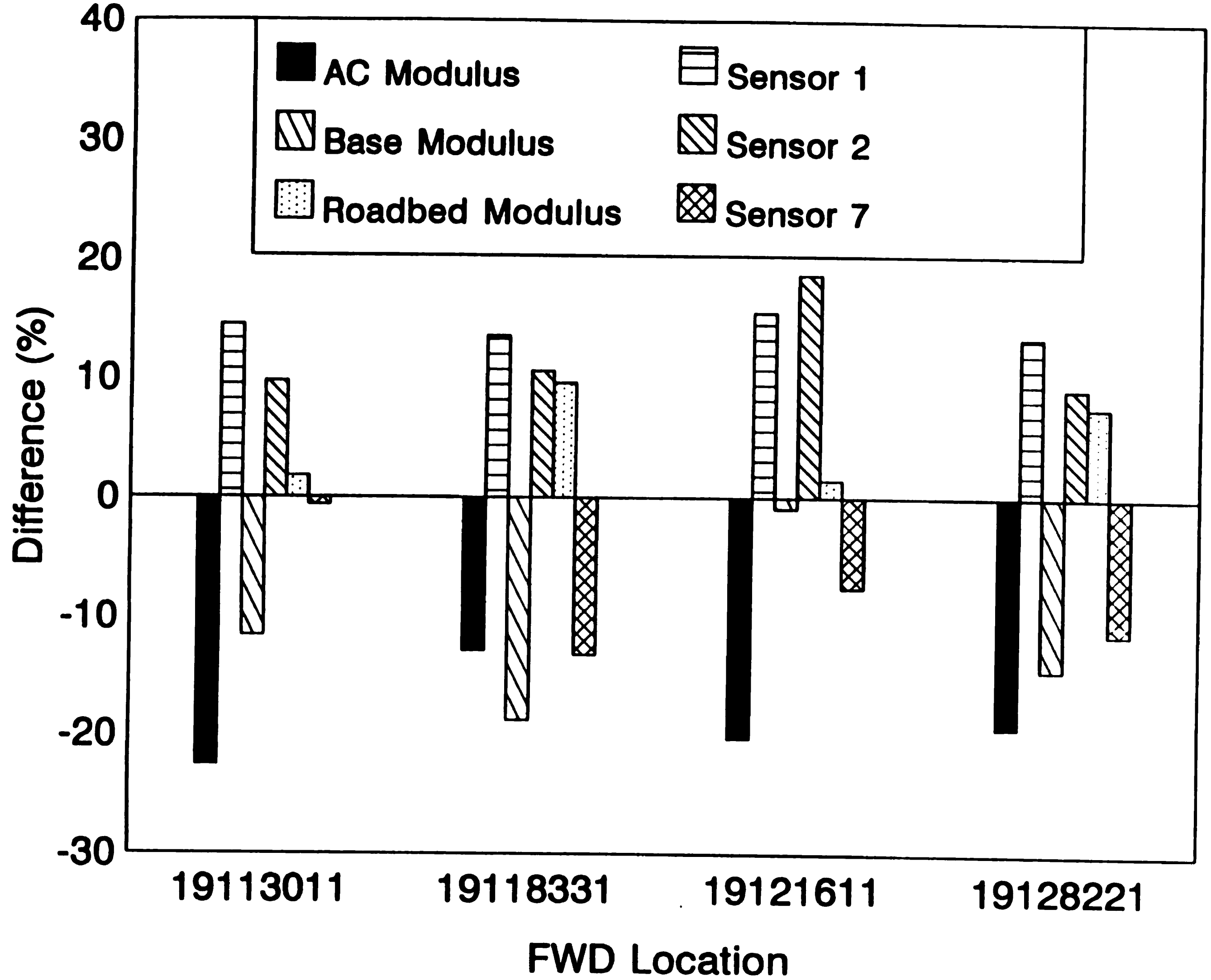

Figure 6.28. Variation of backcalculated layer moduli and the deflection of sensor 1, 2, and 7 along pavement section MSU19F at low load level.

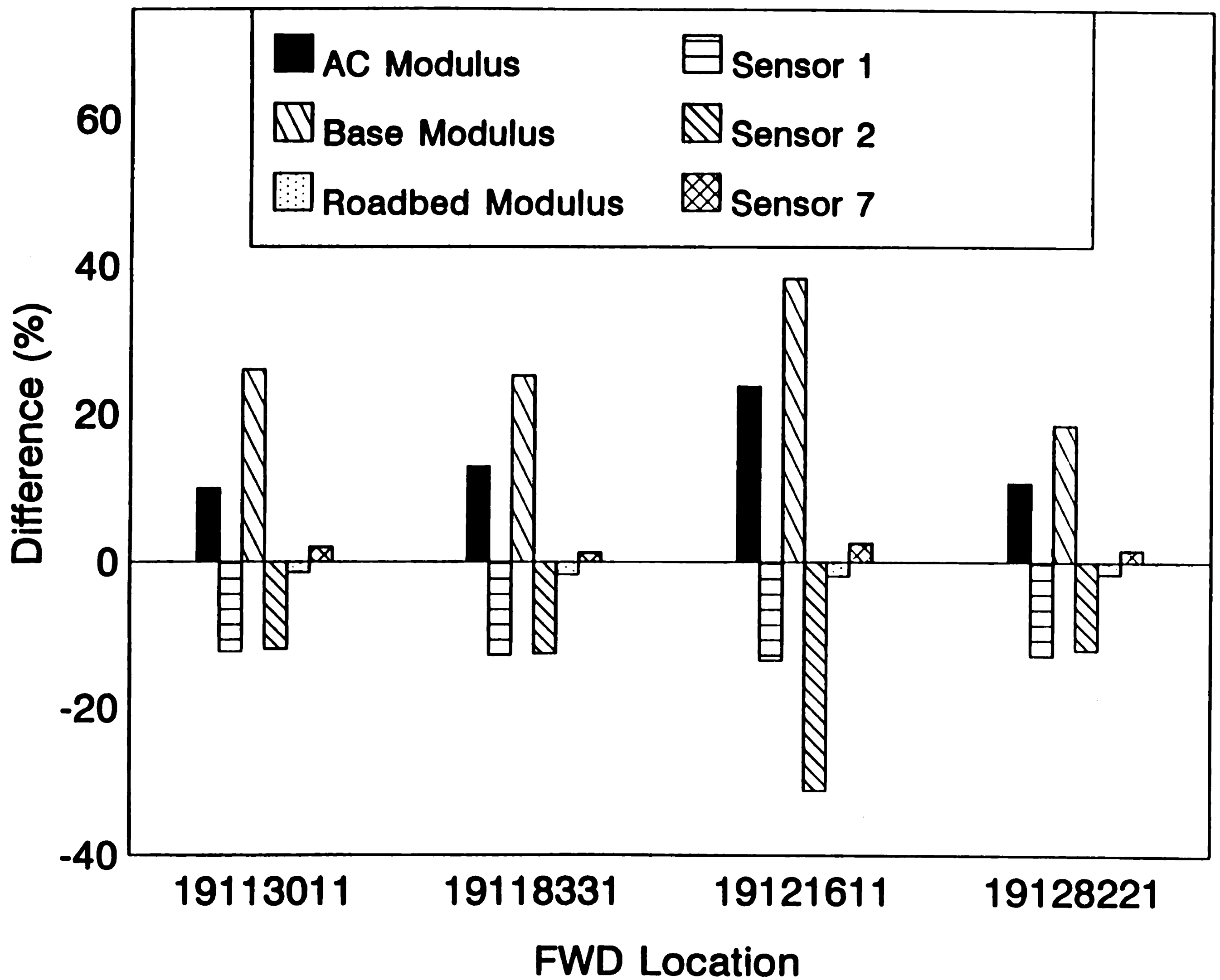

Figure 6.29. Variation of backcalculated layer moduli and the deflection of sensor 1, 2, and 7 along pavement section MSU19F at high load level.

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2. The measured deflection basins at the 16080 pounds load level are lower than the corresponding calculated linear responses (Table 6.19). Consequently, the backcalculated AC and base moduli are higher than the reference values computed at the 9277-pounds load level (Table 6.18).
3. For the outer-most sensors, the variations between the measured values and those calculated by linear extrapolation are insignificant (Table 6.19). Since the outer sensor measurements strongly influence the backcalculated roadbed modulus, the values estimated at the various load levels are almost constant (they vary within a very reasonable range). See Table 6.18.

The three observations indicate that the results produced by the MICHBACK computer program are compatible with the measured deflection basins. That is, for all test location numbers, the values of the layer moduli backcalculated for any load level are higher or lower than the reference values if the measured deflection basins are, respectively, shallower or deeper than the linearly extrapolated responses. This tend to support the contention that the variations in the results obtained by the MICHBACK computer program are accurate and reflect the variations of the measured deflection basins.

6.5.2 Flexible Pavement Sections MSU35F - Variable Load Level

This test section is located in Cheboygan County and the roadbed soil is similar to that for section 19F (Table 6.17). The trends of the deflection basins measured at the three load levels and the corresponding linearly extrapolated basins for three test locations are shown in Figures 6.30 through 6.32. Table 6.20 provides a list of the layer moduli computed at the three load levels and the percent differences between the reference modulus values (calculated at the standard target load of 9000 pounds) and those determined at the other two load levels. For the three load levels, Table 6.21 provides a list of the differences between the measured deflection basins

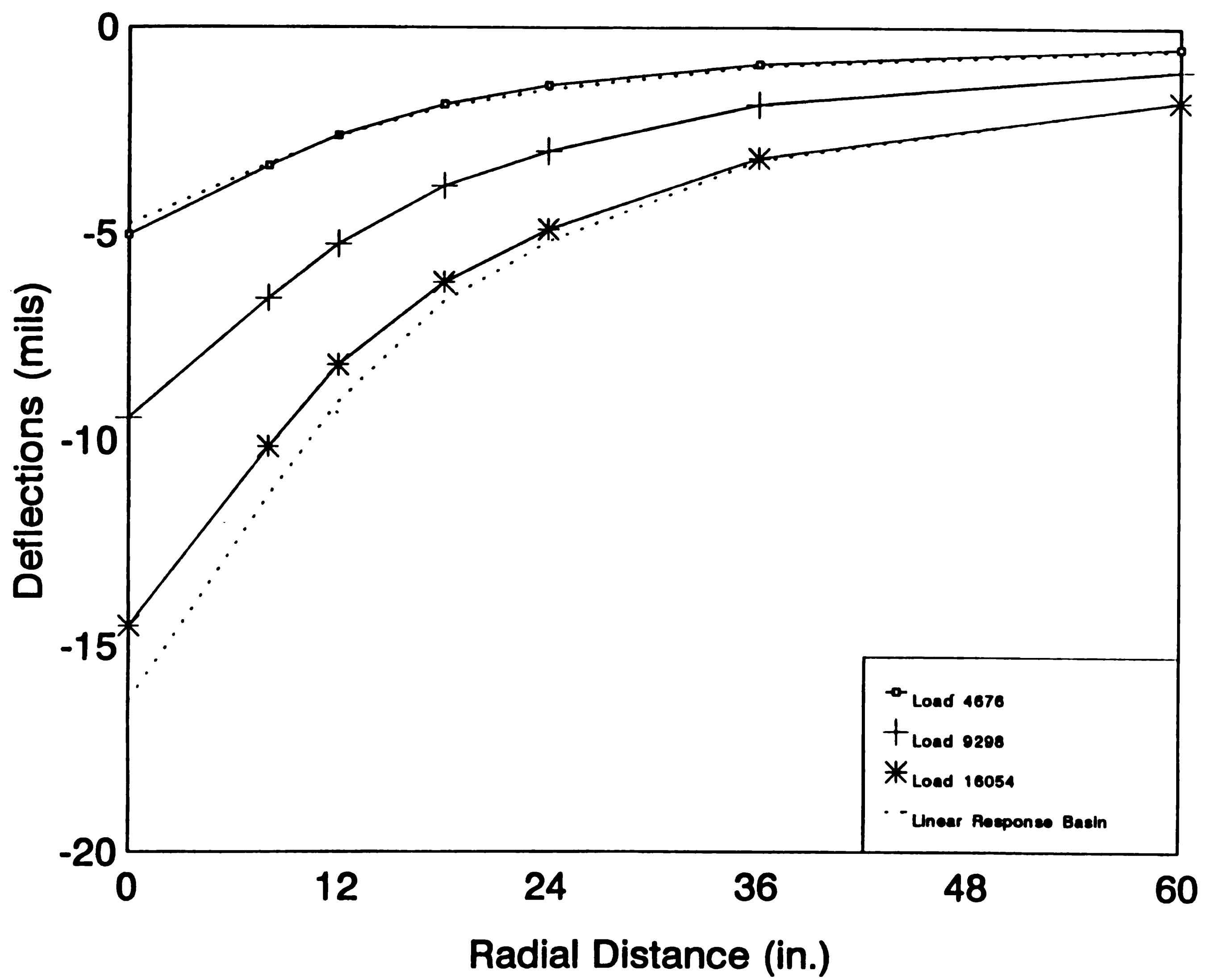

Figure 6.30. Comparison of measured and expected linear response deflection basins for pavement section MSU35F (FWD test code 35118931).

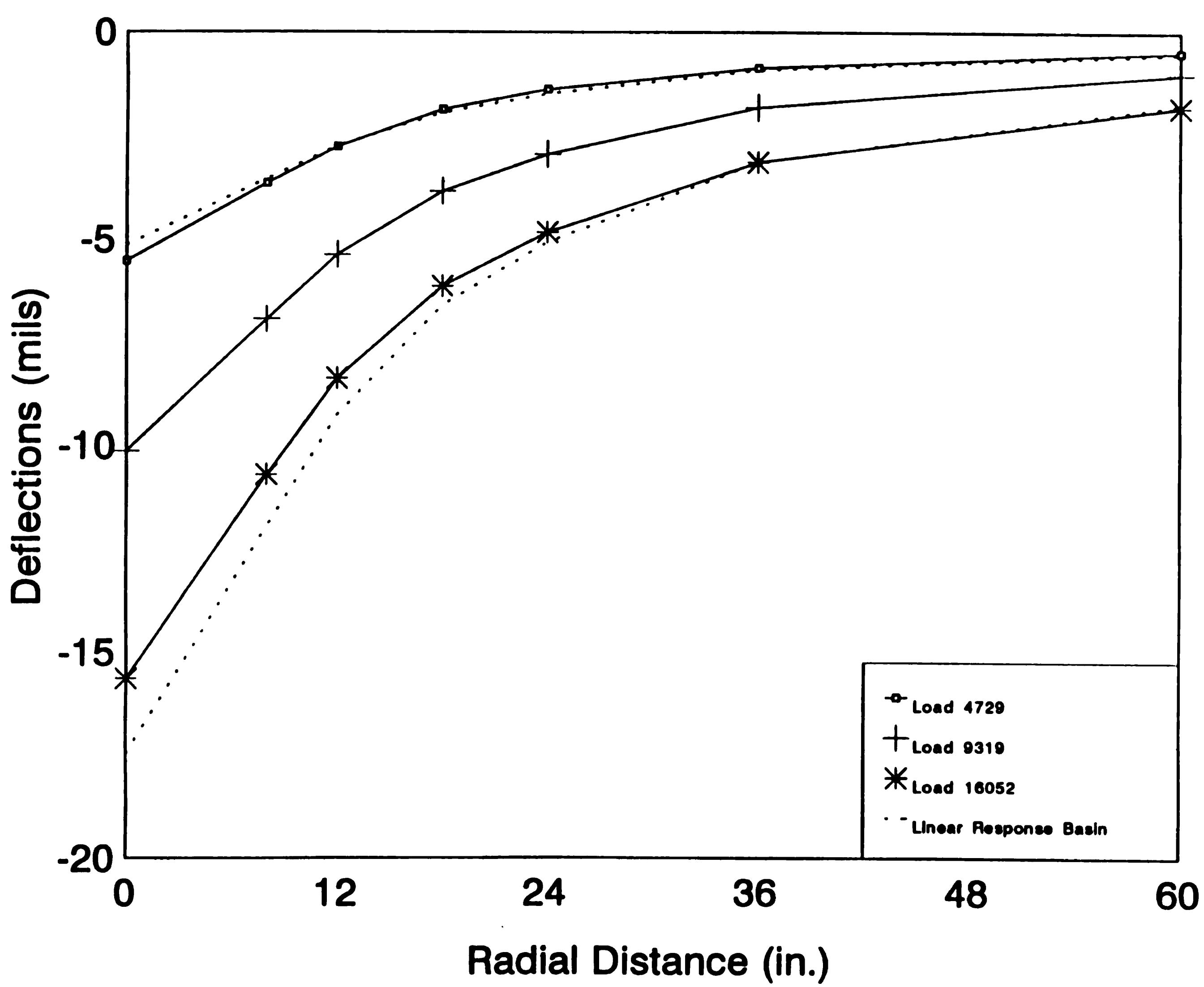

Figure 6.31. Comparison of measured and expected linear response deflection basins for pavement section MSU35F (FWD test code 35114222).

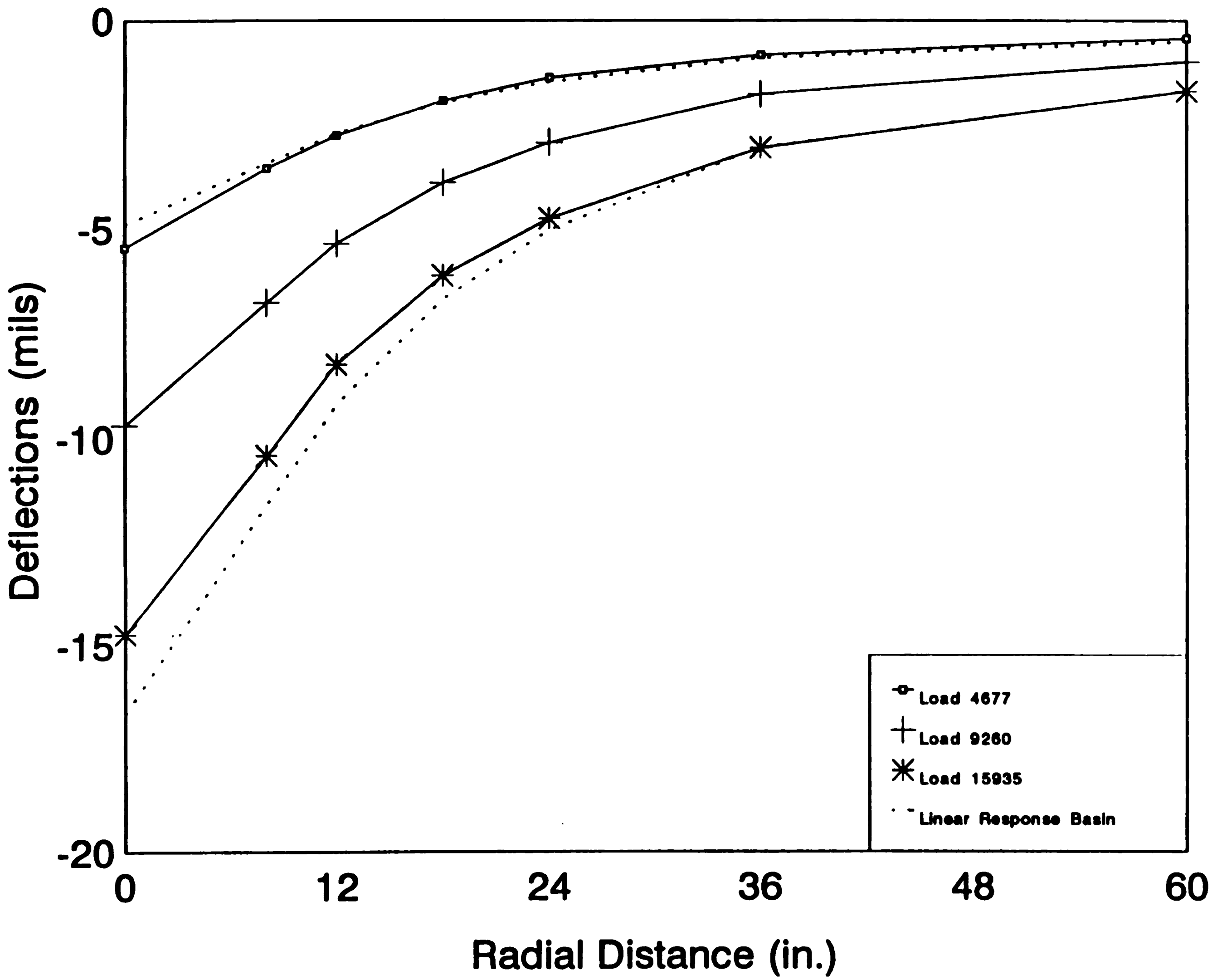

Figure 6.32. Comparison of measured and expected linear response deflection basins for pavement section MSU35F (FWD test code 35123311).

Table 6.20. Backcalculated moduli at various loads for pavement section MSU35F.

FWD test code	Load level (lb)	Backcalculated moduli (psi)						RMS Error (%)
		AC		Base		Roadbed		
		Modulus	Difference (%)	Modulus	Difference (%)	Modulus	Difference (%)	
35118931	4676	343035	-19.8	53522	-4.4	28433	8.6	1.28
	9298	427748		55995		26171		2.09
	16054	470903	10.1	68885	23.0	25516	-2.5	1.04
35114222	4729	258317	-19.2	42936	-6.8	35917	12.0	0.49
	9319	319879		46056		32079		1.17
	16052	338934	6.0	57353	24.5	31344	-2.3	1.13
35123311	4677	247295	-30.1	42698	-1.5	32167	-4.0	1.31
	9260	353628		43329		33519		1.34
	15935	390088	10.3	54555	25.91	31764	-5.2	0.81%

Table 6.21. *Differences between measured and linearly extrapolated deflection basins for pavement section* MSU35F.

Table 6.21. Differences between measured and linearly extrapolated deflection basins for pavement section MSU35F.

FWD Test Code	Load Level (lb)	Difference in Deflections at Sensor Locations (%)						
		1	2	3	4	5	6	7
35118931	4676	5.69	1.69	-0.96	-3.42	-6.96	-5.53	-9.56
	9298							
	16054	-11.26	-10.44	-9.82	-6.88	-5.63	-1.69	0.33
35114222	4729	7.43	3.16	1.09	-4.12	-7.04	-5.97	-8.60
	9319							
	16052	-10.43	-10.50	-9.46	-7.10	-4.64	-0.02	3.12
35123311	4677	11.40	3.59	1.83	-1.94	-7.11	-7.93	-16.6
	9260							
	15935	-11.63	-10.34	-10.44	-8.41	-5.47	0.51	0.90

and those calculated for a linear elastic system. It can be seen that the trends of the data in the figures and tables are the same as those for pavement section MSU19F presented in the previous section.

6.5.3 Composite Pavement Section MSU01C - Variable Load Level

This pavement section is located in Lenawee county in the State of Michigan. The pavement exhibited medium to high severity transverse reflective cracks. The roadbed soil is cohesive in nature (see Table 6.17).

The percent differences between the layer moduli backcalculated at the three load levels are listed in Table 6.22. Table 6.23 provides a list of the percent differences between the measured deflection basins and those computed by assuming a linear elastic system. Figures 6.33 through 6.35 depict the measured deflection basins and the linearly extrapolated responses (dotted lines) at the low and high load levels for test location numbers 01284611, 01272411, and 01261611, respectively.

Besides the difficulties associated with the analysis of a composite pavements with deteriorated concrete slabs, the trends of the backcalculated results are similar to those presented above.

6.5.4 Discussion

In this section, the discrepancies between the deflection basins measured at the three load levels and those calculated by assuming a linear elastic system (i.e, the apparent non-linearity in the pavement response) is discussed. These discrepancies can be attributed to several factors including:

1. Poisson's Ratio - During the analyses of the deflection basins, Poisson's ratios of the AC, base and roadbed soil were kept at constant values of 0.35, 0.4, and 0.45 respectively. Although, these are typical values of Poisson's ratios, the actual ones may vary. For example, Baladi (1988) stated that Poisson's

Table 6.22. Backcalculated moduli at various loads for pavement section MSU01C.

FWD test code	Load level (lb)	Backcalculated moduli (psi)						RMS Error (%)
		AC		Base		Roadbed		
		Modulus	Difference (%)	Modulus	Difference (%)	Modulus	Difference (%)	
01284611	4662	645675	-0.5	3833530	2.6	13227	8.05	1.96
	9303	642730		3499738		12242		0.48
	16073	662688	1.4	3366683	3.8	11880	-2.96	0.37
01272411	4672	218838	-15.4	5525738	-4.4	24675	16.6	3.37
	9345	258598		5781405		21163		1.88
	16190	319608	23.6	4652639	-19.5	19957	-5.7	5.22
01261611	4621	224915	-14.2	5315136	7.0	26860	7.1	3.20
	9295	262194		4968147		25079		0.85
	16084	300558	14.6	4697131	-5.5	24695	-1.5	0.64

Table 6.23. **Differences between measured and linearly extrapolated deflection basins for pavement section MSU01C.**

FWD Test Code	Load Level (lb)	Difference in Deflections at Sensor Locations (%)						
		1	2	3	4	5	6	7
01284611	4662	-5.24	-7.53	-3.40	-5.97	-4.91	-4.22	-9.84
	9303							
	16073	1.09	1.75	1.63	1.37	3.14	2.49	1.73
01272411	4672	1.62	-7.12	-6.35	-9.90	-7.43	-7.92	-14.91
	9345							
	16190	-2.25	3.64	3.45	18.81	3.97	4.07	3.96
01261611	4621	2.90	-3.19	-3.48	-3.56	-9.86	-6.04	-3.55
	9295							
	16084	-4.05	0.09	-0.09	0.74	1.67	0.56	1.25

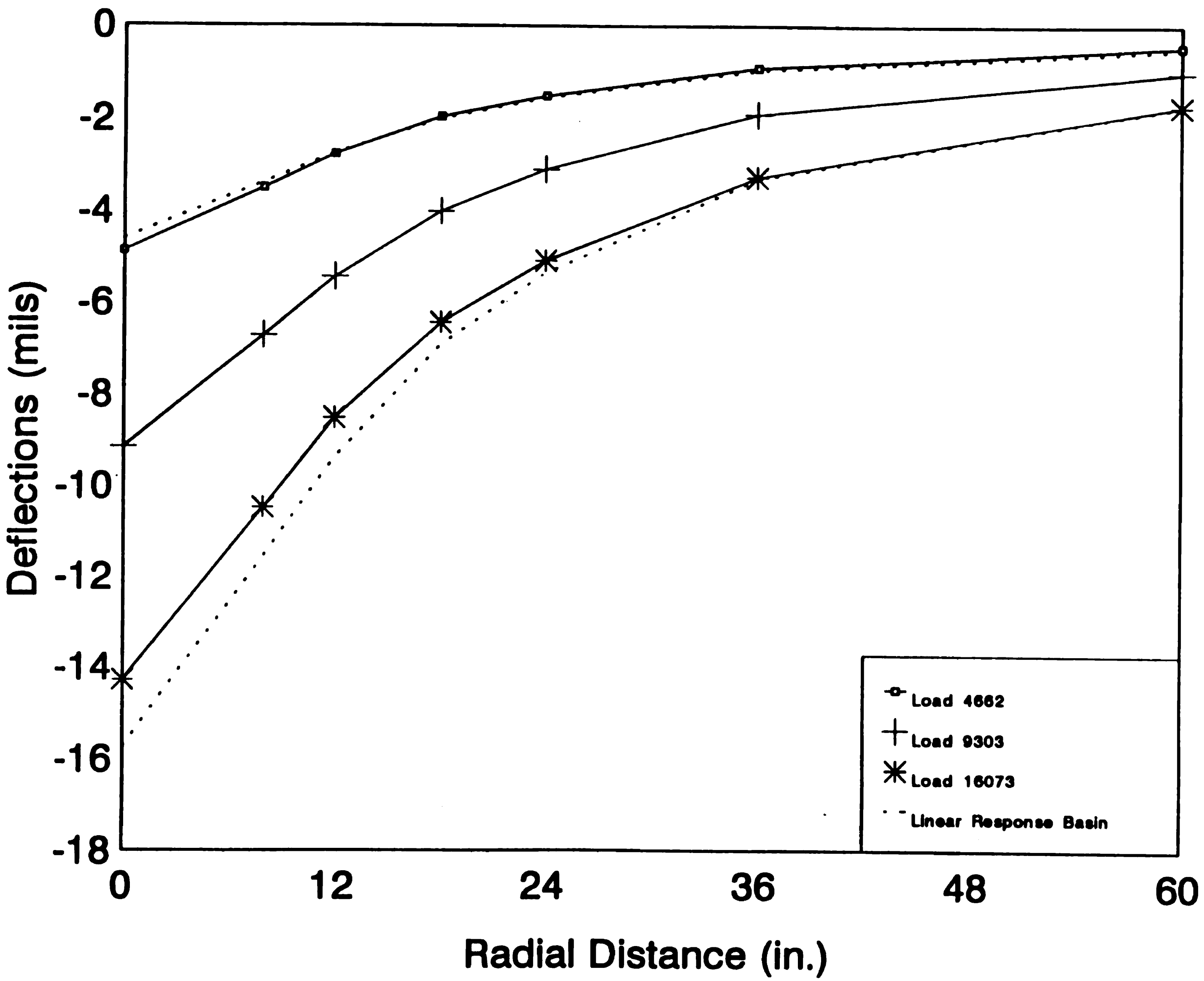

Figure 6.33. Comparison of measured and expected linear response deflection basins for pavement section MSU01C (FWD test code 01284611).

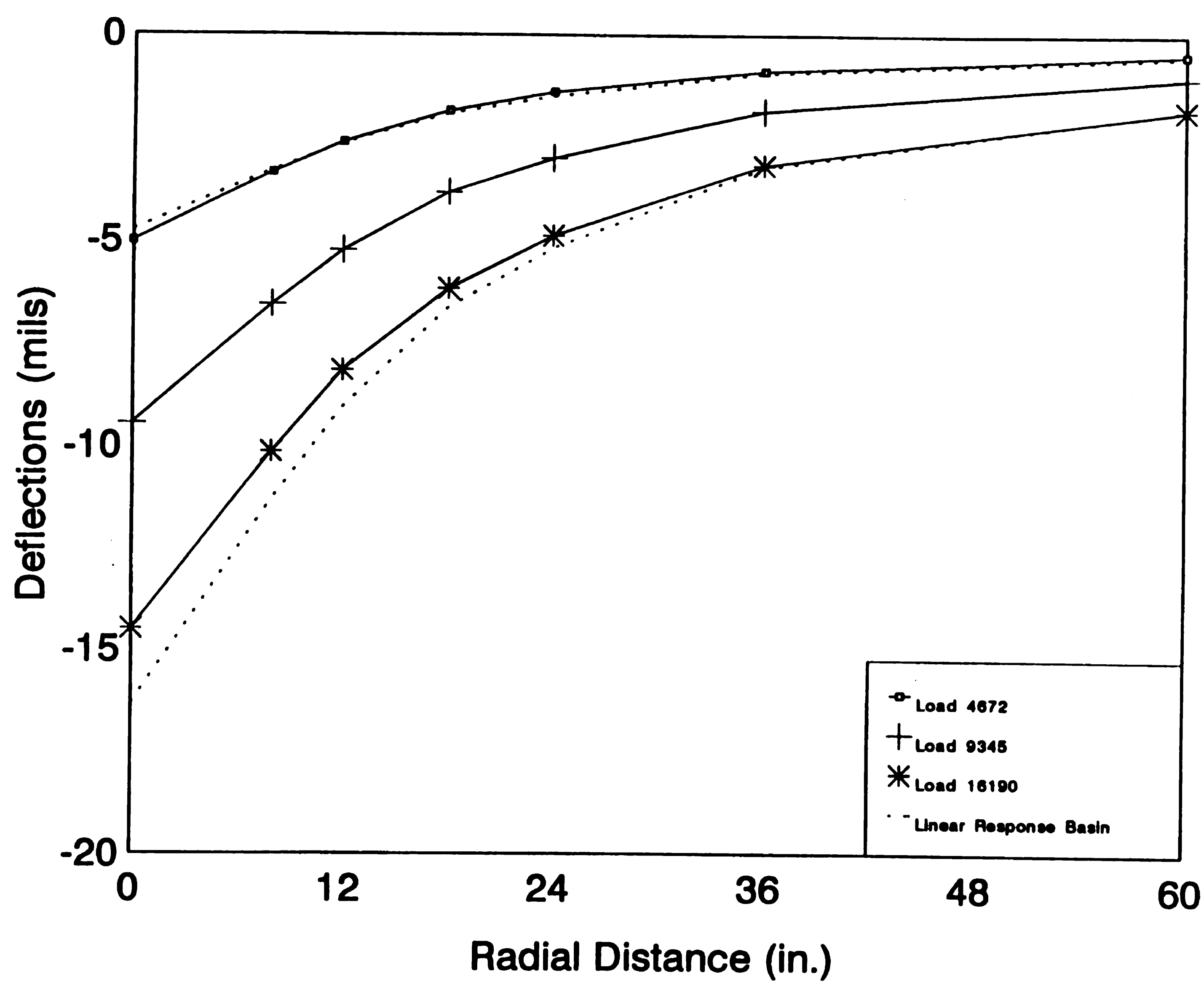

Figure 6.34. Comparison of measured and expected linear response deflection basins for pavement section MSU01C (FWD test code 01272411).

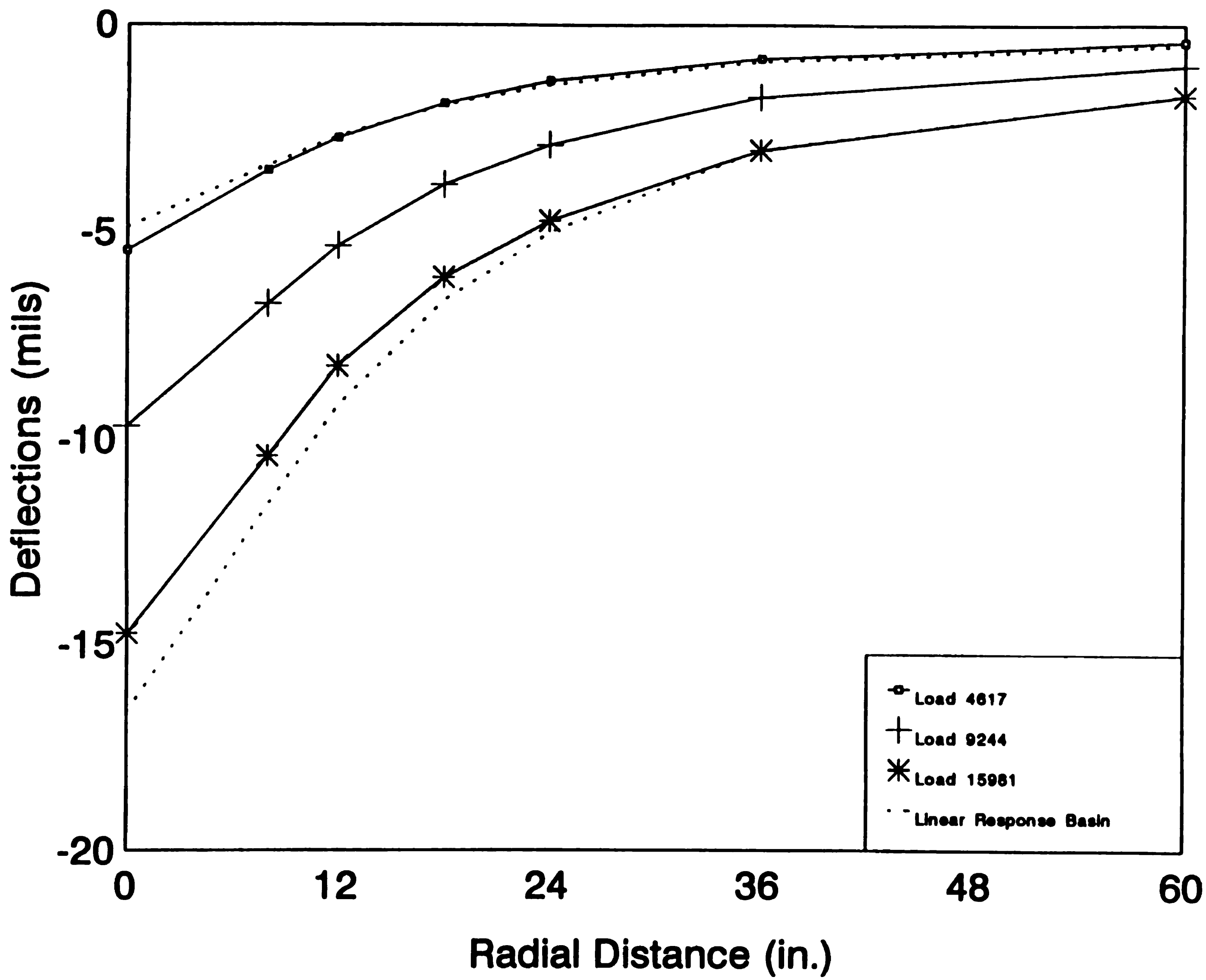

Figure 6.35. Comparison of measured and expected linear response deflection basins for pavement section MSU01C (FWD test code 01261611).

ratio of the AC mix increases with increasing load level. He reported a range of laboratory measured Poisson's ratio from 0.1 to 0.45. Likewise, in the AAMAS study, Vanquintus (1989) reported a range of the Poisson's ratio of the AC from negative values to values of over 0.5. Bouldin, et al. (1993) reported a range of Poisson's ratio from 0.1 to 7. Therefor, it appears that the Poisson's ratio of the AC mix is a function of the magnitude of the applied load and the stiffness of the AC mix. In order to assess the impacts of Poisson's ratio of the AC on the backcalculated layer moduli, two typical pavement sections (19113011 and 35114222) were analyzed by using Poisson's ratios of 0.25, 0.35, and 0.45 for the target load levels of 4500, 9000, and 16,000 pounds, respectively. Tables 6.24 and 6.25 provide lists of the values of the layer moduli backcalculated at the three load levels with a constant Poisson's ratio of the AC of 0.35, and the backcalculated values for the other two Poisson's ratios. The percent differences between the reference moduli (calculated at the target load level of 9000 pounds using a Poisson's ratio of 0.35) and those calculated for the other two load levels are also provided in the table. In reference to Table 6.24., it can be seen that a significant part of the apparent non-linearity at the higher load level is eliminated. For example, The percent difference between the reference modulus of the AC and that at the higher load level dropped from 9.9 percent to only 3.00 percent. Less significant improvements were found at the low load levels and in the modulus of the base layer. Since, the exact values of Poisson's ratios are not known, part of the apparent non-linearity in the values of the backcalculated layer moduli can be attributed to variations in Poisson's ratios.

2. Shape Factor - Throughout this study, and because of the configuration of the steel plate of the FWD, a circular loaded area with uniform contact pressure

Table 6.24. Backcalculation of layer moduli at different Poisson's ratios for pavement section MSU19F (test No. 19113011).

Poisson's ratio	Backcalculated layer moduli in psi at the indicated load level (pounds)								
	4648			9277			16080		
	AC	Base	Roadbed	AC	Base	Roadbed	AC	Base	Roadbed
0.35	219734	47231	47679	283712	53437	47023	311783	67332	46359
0.25	226833	48948	47431						
0.45							292237	65660	44281
Percent error at the indicated Poisson's ratio									
0.35	-22.60	-11.60	1.82	0.00	0.00	0.00	9.90	26.0	-1.4
0.25	-20.05	-8.40	0.87						
0.45							3.00	22.90	-5.8

Table 6.25. Backcalculation of layer moduli at different Poisson's ratios for pavement section MSU35F (test No. 35114211).

Poisson's Ratio	Backcalculated layer moduli in psi at the indicated load level (pounds)								
	4729			9319			16052		
	AC	Base	Roadbed	AC	Base	Roadbed	AC	Base	Roadbed
0.35	258137	42936	35917	319879	46056	32079	338934	57353	31344
0.25	273755	43844	35751						
0.45							312821	54800	33834
Percent error at the indicated Poisson's ratio									
0.35	-19.20	-6.80	12.0	0.00	0.00	0.00	6.0	24.5	-2.3
0.25	-14.42	-4.80	11.5						
0.45							2.21	18.98	5.47

was assumed. In reality, the pressure distribution is a function of the relative rigidity of the steel plate and the AC layer, the macro-texture of the pavement surface, the magnitude of the applied load and the interface conditions between the plate and the pavement surface. According to Saint-Venant's principle (Timoshenko and Goodier, 1987) the effect of changes in the pressure distribution is more significant in the zone near the loaded area while the response further away is sensitive to the total applied load. This is similar to the observations of the measured deflections data in which the deviation from the linear response due to varying load level is more pronounced for the first few deflection locations than the far ones. Hence, part of the apparent non-linear behavior may be attributable to the shape of the loaded area and the pressure distribution.

3. System Non-Linearity - An earlier study conducted by Nazarian and Chai (1992) studied the effect of load induced non-linearity. The test results indicate that the AC modulus increases and the base and roadbed moduli decrease with increasing load levels. This observation is confirmed by the results presented earlier. The base modulus was observed to decrease with increasing load levels, which is opposite to the trends noticed for base material for the Michigan pavements that were analyzed. But one discrepancy between the method of data analysis is that for the above quoted study the AC modulus was fixed at average values after initial backcalculation. In the initial backcalculation, however, it seems that all the moduli were varied simultaneously. Backcalculation of the base layer modulus was then performed at fixed values of the AC modulus. No explanation has been offered for deviation from the standard procedure of backcalculating all the layer moduli simultaneously. The same study also concluded that the load induced non-linearity is more pronounced for KUAB (used by MDOT) than for any other

FWD equipment, although, the problem with vibratory devices is certainly more severe.

4. Stress Dependency - Boker (1978) conducted study on Michigan cohesionless soils concluded that the resilient modulus of the soils tested is a function of the stress level, water content and dry density. In a similar study conducted by Goitom (1981), the above observation was confirmed for cohesive soils of Michigan as well. Chatti (1987) found that the resilient modulus of asphalt paving mixtures increase with an increase in the cyclic load. Tests conducted by Pronk (1989) with field data using the FWD showed that small changes in the roadbed modulus due to non-linear behavior can induce greater changes in the moduli of the other layers. However, he attributed confining pressure or the dead weight of the pavement as the main contributing factor towards the non-linear behavior rather than the changes in the applied loads. Hence, the observed trends can in part be attributed to the non-linearity induced by loading as well as material behavior. More comprehensive studies are certainly required to make more forceful claims.

5. Dynamic Behavior - Most existing backcalculation routines, including the MICHBACK computer program, are based on a linear layered elastic system subjected to static loading. The FWD delivers dynamic loads to the pavement structure. Hence, the magnitudes of the measured deflections at the various sensors are a function of the dynamic (not static) properties of the pavement and loading systems. These include, that part of the mass of the pavement structure affected by the load (higher loads affect a larger mass whose properties are not constant with depth), the velocities of the compression, shear, and surface waves, the magnitude of the induced vibration in the system (which causes some energy losses), and the damping (viscous properties) of the various pavement layers. Hence, a part of the apparent non-linearity of the

backcalculated layer moduli may also be attributed to the dynamic behavior of the system.

To this end, one may argue that the above enumerated factors should be included in any backcalculation routines. Although, from the academic viewpoint, this is a reasonable argument, in practice, it makes an insignificant impact on the accuracy of the backcalculated results. To illustrate, consider a pavement structure with AC, base and subbase thicknesses of 6-inch each, and a roadbed soil thickness of 60-inch (a total depth to the bedrock or to a stiff layer of 78-inch). These thicknesses are not accurately known. For example, the true thickness of the AC may vary from 5 to 6-inch at best. The depth to the stiff layer may vary by several inches or several feet. For most pavement structures, a variation of the AC thickness of 1-incyh from the mean may cause significant variations in the calculated moduli of the AC, base, and subbase. These variations, in some cases, are much larger than those due to the apparent non-linearity of the system. Besides, it is not possible to include all the factors affecting pavement responses in a backcalculation routine, since the number of system unknowns cannot exceed the number of deflection sensors (seven in the case of the MDOT KUAB FWD).

Therefore, it is more reasonable to use a linear elastic layered system whereby the thicknesses of some pavement layers are treated as unknowns (they possess a significant impact on the backcalculated results), than to include other factors which have lesser impact. Nevertheless, it is very important that accurate measurements of the layer thicknesses must be made before one can achieve accurate backcalculation results. Systems for making such measurements are currently under development and the most promising one is a system based on the radar technology.

The results of the analysis of a variety of pavement sections and the associated discussions presented in this chapter provides enough evidence that the backcalculated results of the MICHBACK program are consistent with the measured deflections and thickness variations observed in the field. The difficulty associated with the analysis of composite pavements has also been highlihgted.

CHAPTER 7

TEMPERATURE EFFECTS ON THE BACKCALCULATED LAYER MODULI

7.1 GENERAL

In this chapter, the effects of temperature variations on the deflections and backcalculated layer moduli for Michigan pavements are examined. As stated in Chapter 6, the pavement sections were tested during various seasons to observe the seasonal variation effects. Additional tests were designed to monitor the pavement response to temperature variations and they were performed on three pavement sections during the summer of 1993. Discussion and analyses of the test results along with recommendations are presented in this chapter.

7.2 TEMPERATURE EFFECTS

As noted earlier, nondestructive deflection tests (NDTs) are typically conducted to evaluate the structural capacity and the state of deterioration of the pavement structure with time. Unfortunately, the measured pavement deflections are influenced by seasonal variations in moisture and temperature. The moisture variation influences the stiffness of the base and subbase layers and the roadbed soil. Temperature variation, on the other hand, affects the stiffness of the AC layer. Further, any change in the stiffness of any pavement layer causes changes in the responses of the other layers and of the roadbed soil. This implies that the measured deflection data are influenced by variations in both the AC temperature and the moisture level in the other layers and in the roadbed soil. In order to conduct a proper engineering evaluation of the pavement structure and to assess its rate of deterioration with time, the deflection data or the backcalculated moduli of the pavement layers must be corrected to a standard temperature and a standard moisture level.

Two temperature correction methods can be found: the American Association of State Highway and Transportation Official (AASHTO) method which calls for the correction of the measured pavement peak deflection (the deflection under the center of the load), and the Asphalt Institute (AI) method which corrects the backcalculated AC modulus to a standard temperature of 77 °F. Existing backcalculation routines, however, use the temperature correction method. Most routines use the AI temperature correction equation or some modified form of the equation. The AASHTO method is mainly used in the deflection-based design of pavement overlays.

Presently, there is no standard practice regarding seasonal corrections. Some State Highway Agencies (SHA) have collected seasonal deflection data over several years. The data were examined and deflection correction factors were deduced. In some cases the correction factor is constant for all pavements, while in others the value of the factor is a function of the pavement cross-section and the material properties. Since the weather elements (freeze-thaw cycles, and amount of rainfall) are not constant from one year to another, the development of accurate seasonal correction factors is a long time process that requires deflection data to be collected on various pavement sections over a long-term period.

In this study, deflection data were collected over a two year period and three seasons. In addition, during the summer of 1993, NDTs were designed to assess the impact of the AC surface temperature on its backcalculated modulus values. Originally, eight pavement sections (6 flexible and 2 composite) were selected for this part of the study. Unfortunately, because of the weather (the summer was cooler than normal) and equipment limitation, only three flexible pavement sections were tested. For each section, the tests were commenced in the morning when the pavement temperature was around 60 °F, and continued throughout the day at half-hour time intervals. The tests were conducted at the same location which was previously marked by the MDOT FWD crew.

Two of the three pavement sections (MSU13F and MSU19F) have already been introduced in Chapter 6. The AC thicknesses for these two pavements are 4.8- and 6.2-inch, respectively. The third pavement was not included in the study plan but it was selected because of its near proximity to the MDOT testing facilities. This pavement has an AC thickness of 4.5-inch. The pavement is in a good condition with no apparent distresses and it will be referred to as MSU52F.

The test results of pavement section MSU52F are discussed in more detail, it was tested under better conditions (sunny day with the temperature rising consistently) and it has the least amount of distress.

7.2.1 Flexible Pavement Section MSU52F - Temperature Variation

Figure 7.1 depicts the relationship between the measured pavement surface temperature and the peak pavement deflection (deflection under the center of the load). The figure indicates that:

1. The peak pavement deflection increases almost linearly with increasing pavement temperatures.
2. For a constant temperature, the pavement deflection measured during the heating cycle is different from that measured during the cooling cycle.

Therefore, based on the second observation, the FWD test results measured during the cooling cycle of the pavement are ignored. Table 7.1 provides a list of the deflections recorded at different sensors and the corresponding pavement temperatures. The deflections measured at the lowest temperature (78 °F) were taken as reference values and the percent variation in the deflections at other temperatures were calculated and are listed in Table 7.2, and shown in Figure 7.2. Examination of Figure 7.2 indicates that:

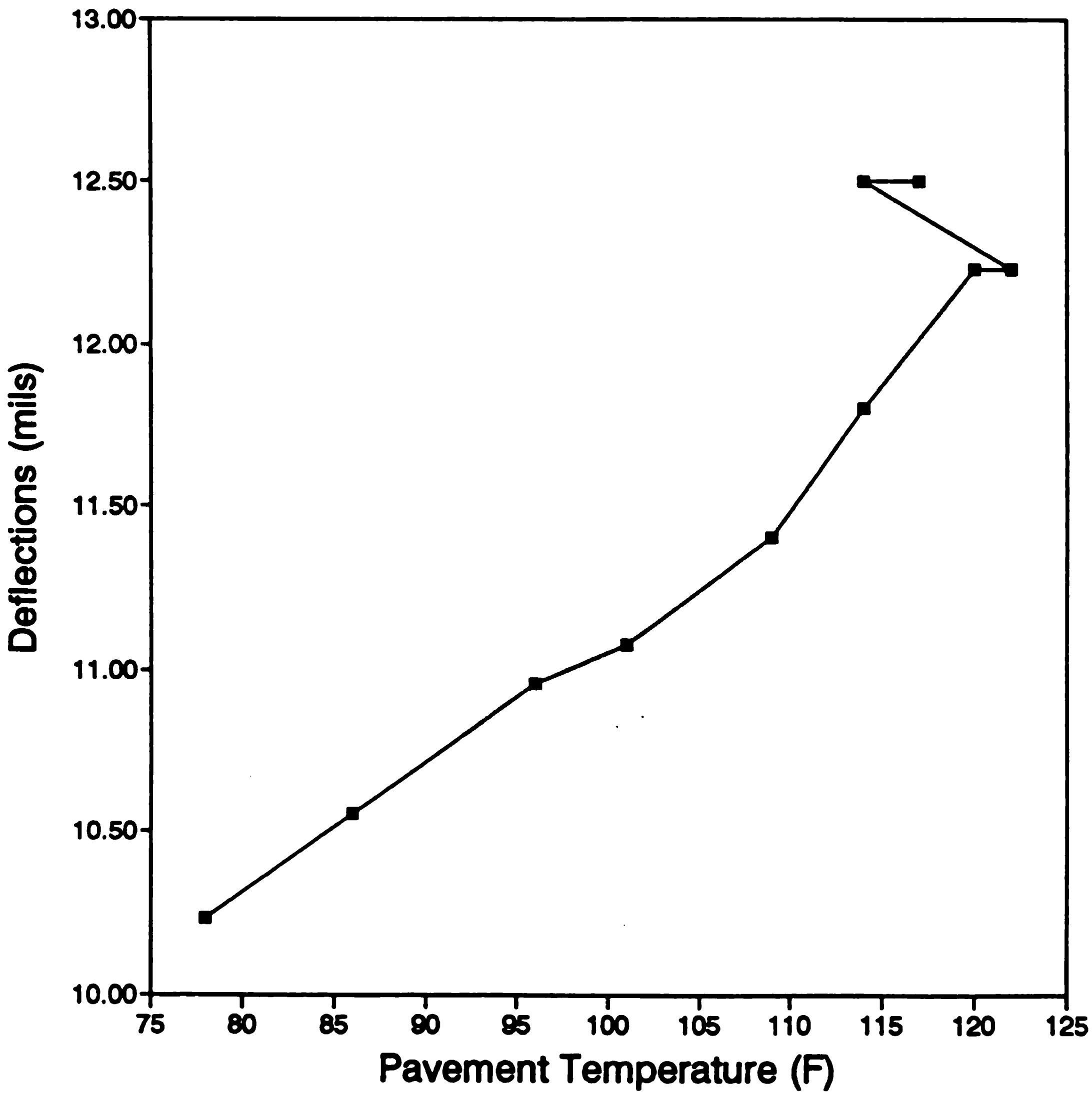

Figure 7.1. Variation of maximum deflection with temperature observed for pavement section MSU52F.

Table 7.1. Deflections variation with temperature for pavement section MSU52F.

Pavement Temp. (F)	Measured deflections at different sensor locations (mils)						
	1	2	3	4	5	6	7
78	10.23	8.11	6.51	4.80	3.62	2.11	1.11
86	10.56	8.24	6.55	4.73	3.53	2.09	1.10
96	10.96	8.42	6.53	4.59	3.42	2.04	1.09
101	11.08	8.27	6.40	4.46	3.29	2.00	1.08
109	11.40	8.32	6.39	4.35	3.22	1.97	1.09
114	11.80	8.43	6.37	4.31	3.19	1.95	1.08
120	12.23	8.39	6.29	4.25	3.14	1.95	1.10
122	12.23	8.12	6.14	4.15	3.09	1.96	1.08

Table 7.2. Percent variation in deflections from lowest temperature recorded for pavement section MSU52F.

Pavement temp. (F)	Variation at different sensor locations (%)						
	1	2	3	4	5	6	7
86	3.19	1.52	0.61	-1.60	-2.40	-0.63	-0.30
96	7.14	3.82	0.31	-4.44	-5.44	-3.32	-1.81
101	8.28	1.89	-1.64	-7.22	-8.94	-5.06	-2.11
109	11.47	2.51	-1.79	-9.51	-10.97	-6.65	-1.81
114	15.38	3.86	-2.10	-10.34	-11.71	-7.28	-2.41
120	19.58	3.37	-3.38	-11.59	-13.09	-7.44	-0.60
122	19.58	0.08	-5.69	-13.67	-14.65	-7.12	-2.11

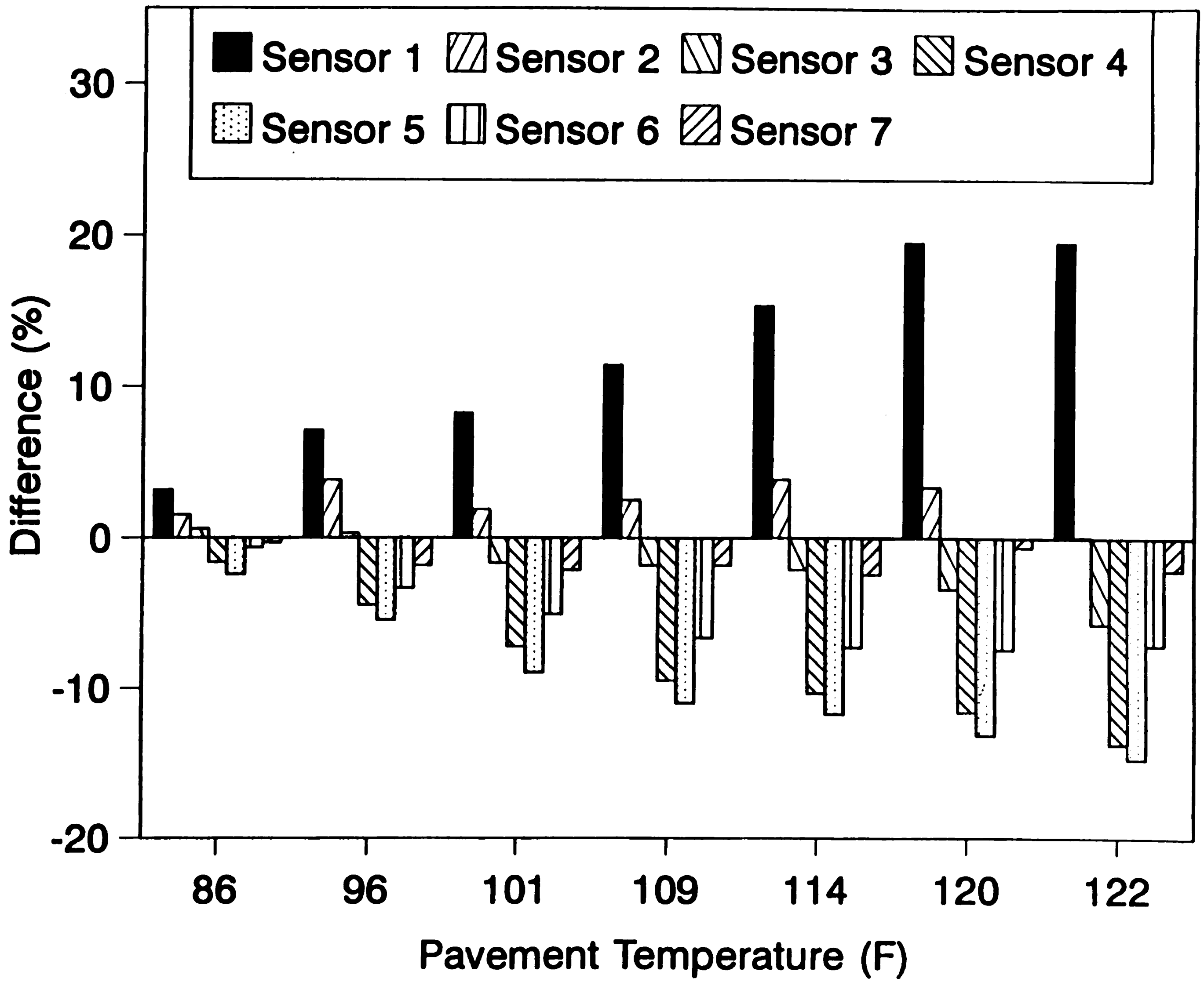

Figure 7.2. Percent variation in deflections with temperature for pavement section MSU52F.

1. The pavement deflections measured at all sensor locations are affected by the pavement temperature.
2. The deflections of sensor 1 and 2 increase with increasing temperature, while the deflections for sensor 3, 4, 5, 6, and 7 decrease with increasing temperatures.

These observations were expected, and are due to the reduction in the ability of the pavement to spread the applied load radially as the AC layer looses its stiffness at higher temperatures.

The backcalculated layer moduli are listed in Table 7.3 and shown in Figures 7.3 and 7.4. It can be seen that the AC modulus decreases almost linearly with increasing temperatures. The rate of change of the AC modulus with temperature is about 10900 psi per °F. The base modulus also increases with increasing temperatures as shown in Figure 7.4. The roadbed modulus, on the other hand, is not affected by the AC temperature. The above observations indicate that the results obtained with the MICHBACK computer program are consistent and compatible with the measured deflection basins.

One may argue that the modulus of the base layer (granular material) should not be affected by the AC temperature. However, there are two factors which may contribute towards the increase in base modulus:

1. The state of stress in the base layer changes with changes in the AC modulus and if the base is stress sensitive, then this could cause the base modulus to change.
2. The Poisson's ratios of the AC mix (which was assumed constant at all temperatures) increases as the stiffness of the AC decreases. This also affects the state of stress in the base layer (see Chapter 6).

Table 7.3. Backcalculated layer moduli for selected pavement sections.

Pavement section	Pavement temp. (F)	Backcalculated moduli (psi)		
		AC	Base	Roadbed
MSU19F	56	421429	68053	49339
	57	442388	64732	47933
	59	430099	64580	47642
	80	390032	67622	46814
	86	367670	69960	46359
	89	339443	70747	47287
	97	333389	68595	47038
	100	302871	69506	46253
MSU13F	56	878214	54197	30674
	57	871973	53457	30654
	58	867489	53876	31080
	64	857004	53156	30535
	75	745520	55459	30882
	87	617385	55191	30426
MSU52F	78	1210232	29806	30562
	86	1128475	30243	30834
	96	1004659	31229	31246
	101	916238	32908	31634
	109	833275	34108	31653
	114	807198	34108	31958
	120	751909	35564	31638
	122	730563	36920	31881

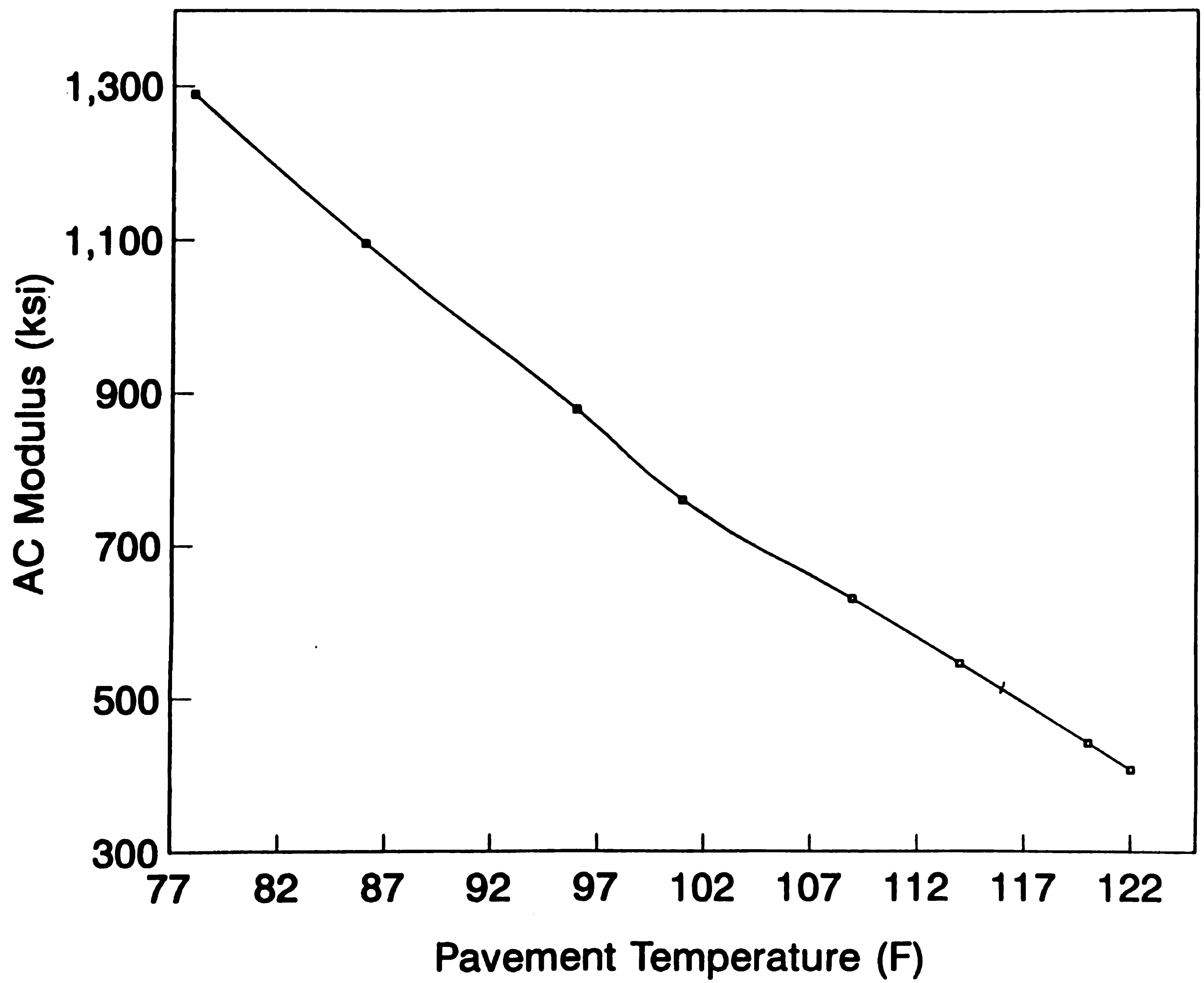

Figure 7.3. Variation of backcalculated AC modulus with temperature for pavement section MSU52F.

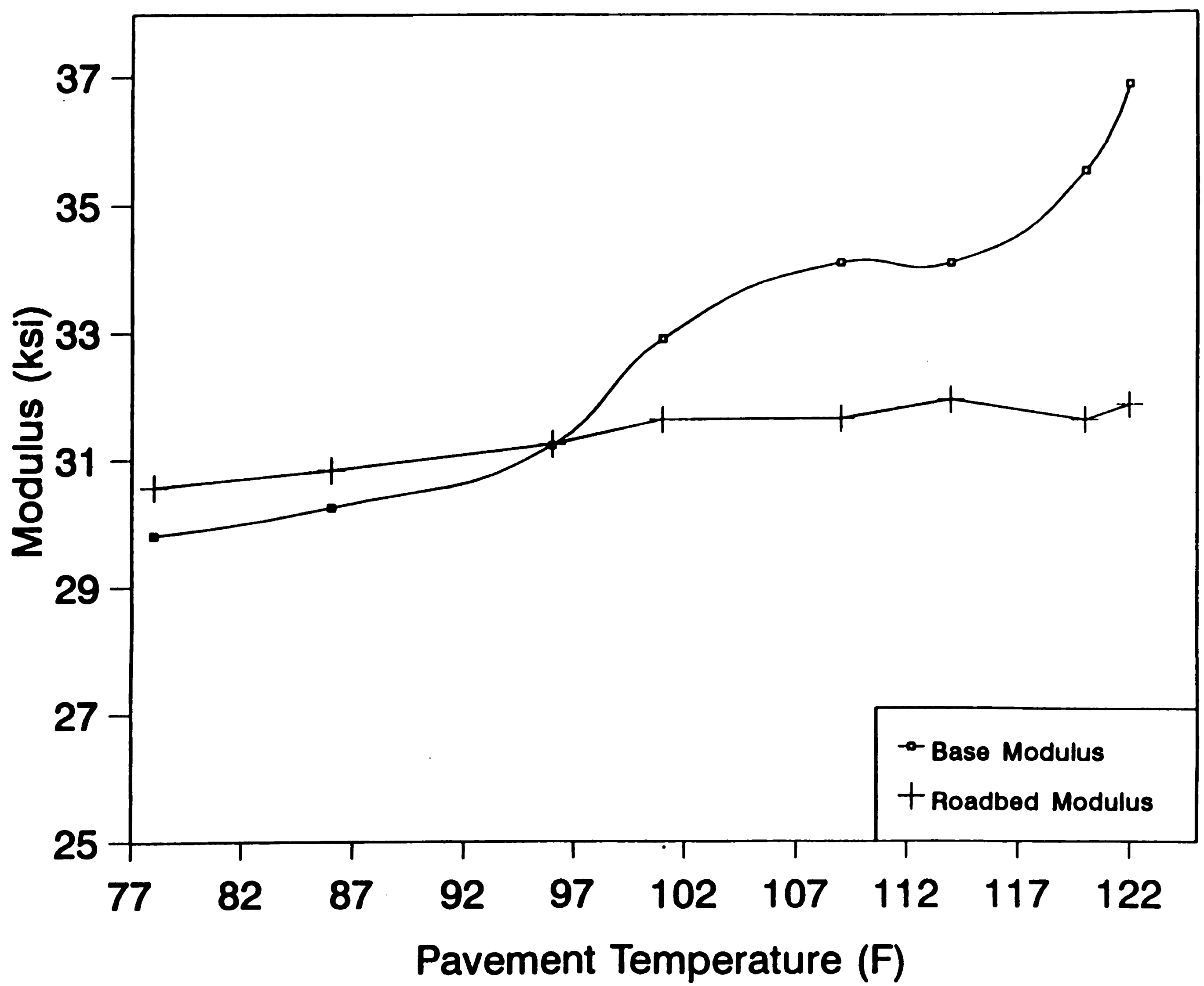

Figure 7.4. Variation of backcalculated base and roadbed moduli with temperature for pavement section MSU52F.

Therefore an increase in the AC temperature causes a decrease in its modulus and indirectly causes the base modulus to increase.

7.2.2 Flexible Pavement Section MSU13F - Temperature Variation

The NDTs were conducted on a cloudy day and under a light rainfall. The pavement deflections measured at different sensor locations are listed in Table 7.4. Once again the deflections measured at the lowest temperature (57 °F) were taken as reference values and the percent variation in the deflections at the other temperatures were calculated and are listed in Table 7.5 and shown in Figure 7.5. Examination of Figure 7.5 indicates that:

1. Sensor 1 deflection increases consistently with rising temperatures.
2. Sensor 6 consistently registered lower deflections with higher temperatures.
3. The deflections at all other sensors are erratic and do not display a consistent change with temperature.

Comparison of the response of the previous pavement (MSU52F) and this pavement indicates that the way in which the two pavements spread the load is different and depends on the AC stiffness and on the temperature range. The thicknesses of the two pavement sections are almost the same. However, MSU52F had a higher original stiffness, having been constructed to study the characteristics of Stone Mastic Asphalt (SMA) mixes. Hence, there is a difference in the basic AC mix properties of the two pavements.

The backcalculated layer moduli are presented in Table 7.3 along with the results of the other pavement sections and the variation in backcalculated AC modulus with temperature is shown in Fig 7.6. The rate of change of the AC modulus with temperature is about 8415 psi per °F. The base modulus increased by a modest

Table 7.4. Deflection variation with temperature for pavement section MSU13F.

Pavement temp. (F)	Deflections at different sensor locations (mils)						
	1	2	3	4	5	6	7
56	8.87	6.85	5.53	4.12	3.18	2.09	1.28
57	8.92	6.93	5.58	4.14	3.19	2.09	1.29
58	8.85	6.87	5.49	4.09	3.14	2.06	1.27
64	8.92	6.95	5.54	4.13	3.18	2.08	1.28
75	8.97	6.84	5.41	3.99	3.08	2.04	1.26
87	9.35	7.07	5.52	3.97	3.08	2.03	1.29

Table 7.5. Percent variation in deflections from lowest temperature recorded for pavement section MSU13F.

Pavement temp. (F)	Variation at different sensor locations (%)						
	1	2	3	4	5	6	7
57	0.56	1.12	1.03	0.57	0.31	-0.32	0.52
58	-0.30	0.34	-0.72	-0.65	-1.15	-1.59	-1.30
64	0.49	1.41	0.18	0.32	0.00	-0.64	0.00
75	1.09	-0.19	-2.17	-3.00	-3.14	-2.71	-1.56
87	5.37	3.26	-0.18	-3.56	-3.14	-2.87	0.78

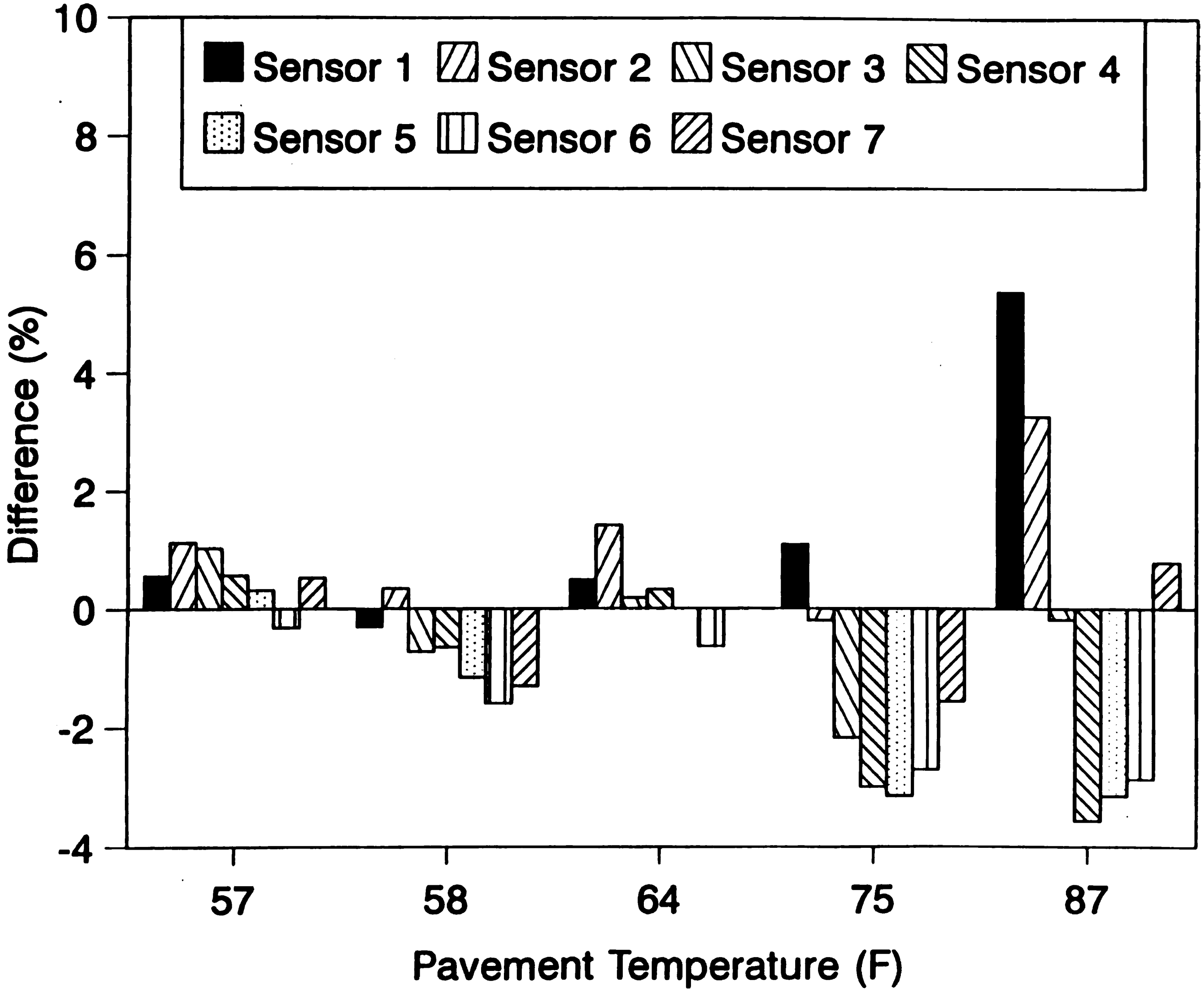

Figure 7.5. Percent variation in deflections with temperature for pavement section MSU13F.

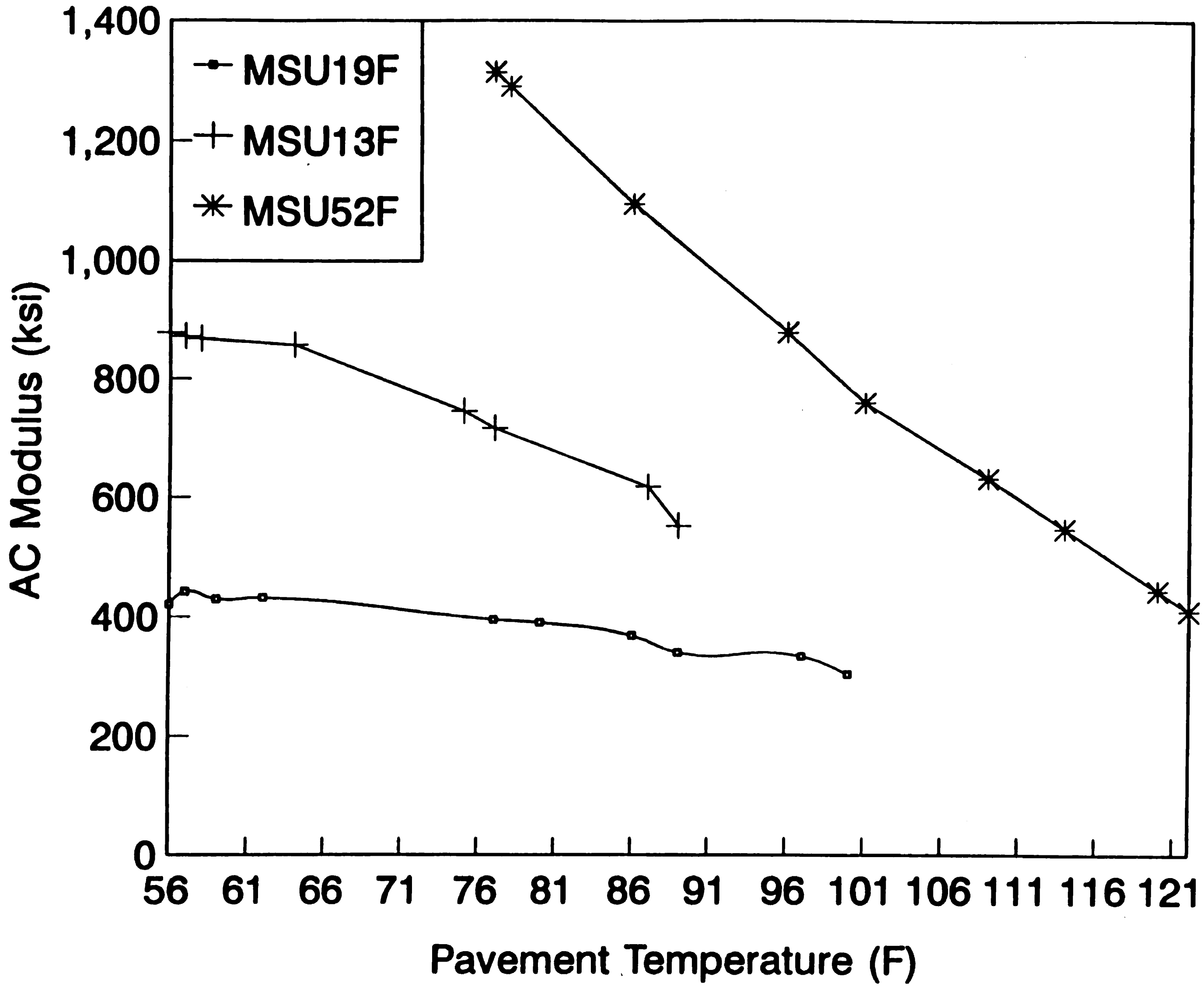

Figure 7.6. Variation of backcalculated AC modulus with temperature for selected pavement sections.

1.83% over the same temperature range. The roadbed modulus, on the other hand is not affected by the AC temperature. The backcalcualated results seem to be consistent with the observed deflections.

7.2.3 Flexible Pavement Section MSU19F - Temperature Variation

This section has a greater AC thickness than the previous two sections and has a lower distress than MSU13F. The range of temperature during which the FWD tests were conducted is 56 to 100 °F.

The deflections recorded at different sensor locations and the test temperatures are listed in Table 7.6. Once more, the deflections measured at the lowest temperature (56 °F) were taken as reference values and the percent variation in the deflections measured at other temperatures were calculated and are listed in Table 7.7 and shown in Figure 7.7. It can be seen that:

1. The deflections of sensors 1, 2, 3, 6, and 7 increased with increasing temperatures.
2. The deflections for sensors 4 and 5 initially increased with increasing temperatures and then decreased.

The backcalculated moduli are presented in Table 7.3. Variations in the AC modulus are illustrated in Figure 7.6 along with the results of the other two pavement sections. The rate of change of the AC modulus with temperature is about 2695 psi per °F. The base and roadbed moduli remain almost unaffected by the pavement temperature.

Temperature, deflections, and the backcalculated moduli for the three pavement sections have a wide variation of trends and ranges. Although the data is limited, some of the trends observed can be summarized as follows:

1. The effect of the AC temperature on its modulus increases with decreasing AC

Table 7.6. Deflection variation with temperature for pavement section MSU19F.

Pavement temp. (F)	Deflections at different sensor locations (mils)						
	1	2	3	4	5	6	7
56	7.14	5.05	3.85	2.72	2.03	1.28	0.80
57	7.21	5.14	3.99	2.82	2.08	1.32	0.82
59	7.24	5.23	3.98	2.81	2.10	1.31	0.83
80	7.33	5.19	3.95	2.77	2.08	1.31	0.85
86	7.34	5.13	3.89	2.73	2.06	1.30	0.86
89	7.42	5.05	3.82	2.66	2.01	1.28	0.84
97	7.54	5.22	3.88	2.69	2.03	1.30	0.84
100	7.74	5.17	3.89	2.69	2.02	1.31	0.85

Table 7.7. Percent variation in deflections from lowest recorded temperature for pavement section MSU19F.

Pavement Temp. (F)	Variation at different sensor locations (%)						
	1	2	3	4	5	6	7
57	1.027	1.7162	3.8126	3.672	2.6314	3.394	1.6606
59	1.4006	3.6303	3.3795	3.0602	3.7826	2.3499	2.9054
80	2.6144	2.7723	2.6862	1.7137	2.4671	2.8715	5.8096
86	2.7545	1.5842	1.1264	0.3672	1.6446	2.0882	7.4689
89	3.9216	0.0659	-0.693	-2.203	-0.823	0.2608	4.1502
97	5.5556	3.3004	0.8665	-1.346	0	1.8274	4.1502
100	8.4034	2.3762	1.213	-1.346	-0.165	2.8715	5.395

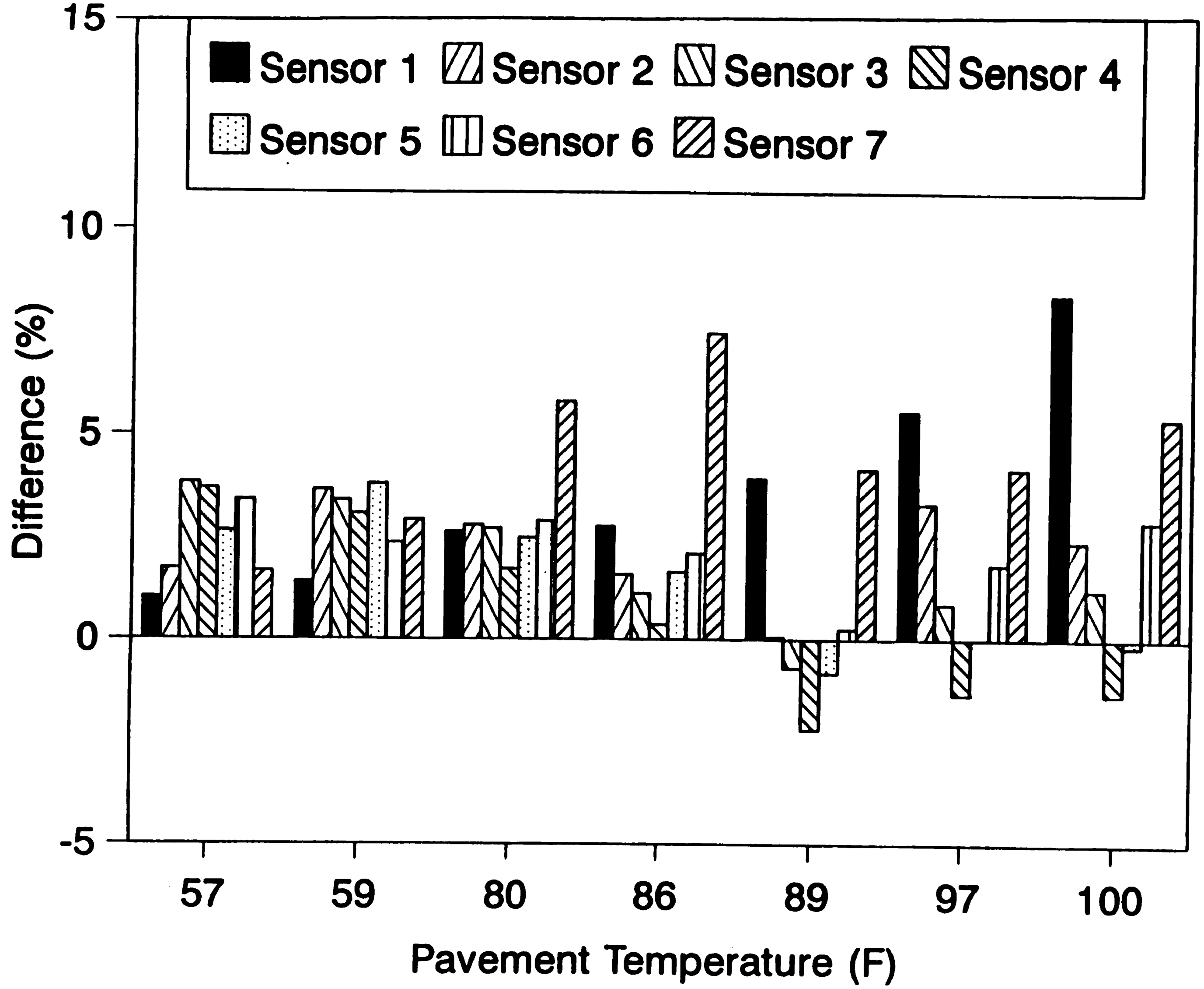

Figure 7.7. Percent variation in deflections with temperature for pavement section MSU19F.

thickness, and decreasing pavement distress.

2. For the same AC temperature, the pavement behavior under a cooling cycle is different than that under a warming cycle.

Unfortunately, the above observations cannot be stated with a good degree of confidence because of the limited data. Consequently, it is recommended that additional tests be conducted on various pavement sections. The results of these tests should enable MDOT to establish temperature and seasonal correction factors.

The applicability of the Asphalt Institute (AI) temperature correction procedure for the AC modulus and the AASHTO method to impart temperature correction to the peak deflection (D_0) to Michigan pavements are examined in the next subsection.

7.3 THE ASPHALT INSTITUTE TEMPERATURE CORRECTION PROCEDURE

Temperature correction to the AC modulus was performed using the AI Equation introduced in Chapter 2 and repeated here for convenience:

$$\log E_o = \log E + P_{200}\left[\frac{1}{(f_o)^l} - \frac{1}{(f)^l}\right] + 0.000005\sqrt{P_{ac}}\left[(t_o)^{r_o} - (t)^r\right]$$

$$-.00189\sqrt{P_{ac}}\left[\frac{(t_o)^{r_o}}{(f_o)^{1.1}} - \frac{(t)^r}{(t)^{1.1}}\right] + .9317\left[\frac{1}{(f_o)^n} - \frac{1}{(f)^n}\right]$$

where l = 0.17033;

n = 0.02774;

E = backcalculated uncorrected modulus;

t, t_o = test and reference temperatures in °F;

f, f_o = loading and reference frequency in Hz;

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P_{ac} = percentage of asphalt cement in the mix;

E_o = the corrected modulus;

r_o = $1.3 + 0.49825 \log (f_o)$;

r = $1.3 + 0.49825 \log (f)$; and

P_{200} = the percent aggregate passing the No. 200 sieve.

For all test sites and measured temperatures, the following data were used in calculating the AI correction factors:

1. P_{ac} = 7 percent (the average percent fine content in the AC mixes),
2. $f = f_o$ = 25Hz; and
3. t_o = 77 °F.

The calculated AI temperature correction factors are listed in Table 7.8.

Table 7.8 also provides a list of the backcalculated AC modulus and the ratios of the backcalculated value at the reference temperature 77°F to that at any other temperature. Figure 7.8 depicts these calculated ratios and the computed AI correction factors plotted against the temperature. It can be seen that:

1. The AC mixes encountered in the three pavement sections are less sensitive to temperature changes than predicted by the AI equation.
2. Thin pavement sections are more sensitive to the AC temperature than thick sections.

The first observation could be related to the asphalt grades used in Michigan pavements (softer asphalt grades are typically used in colder regions). The second observation was expected because the average temperature of a thin pavement is higher than that for a thick pavement. That is, for a thick pavement section, more time is required before the temperature at the bottom of the AC starts to increase due

Table 7.8. Observed and Asphalt Institute AC modulus temperature correction factor.

Pavement section	Pavement temp. (F)	Backcalculated AC modulus and ratios		The AI correction factor
		Modulus (psi)	Ratio	
MSU19F	56	421429	0.937	0.463
	57	442388	0.893	0.503
	59	430099	0.918	0.524
	77	395000	1.000	1.000
	80	390032	1.013	1.137
	86	367670	1.074	1.443
	89	339443	1.164	1.558
	97	333389	1.185	2.386
	100	302871	1.304	2.586
MSU13F	56	878214	0.816	0.532
	57	871973	0.822	0.546
	58	867489	0.826	0.562
	64	857004	0.836	0.691
	75	745520	0.961	0.950
	77	716667	1.000	1.000
	87	617385	1.161	1.406
MSU52F	77	1313886	1.000	1.000
	78	1289605	1.019	1.028
	86	1095356	1.200	1.292
	96	879087	1.495	1.786
	101	759447	1.730	2.070
	109	631174	2.082	2.841
	114	546395	2.405	3.430
	120	443078	2.965	4.228
	122	408521	3.216	4.504

Note: Ratio = Modulus @ measured temp./ Modulus @ 77 F

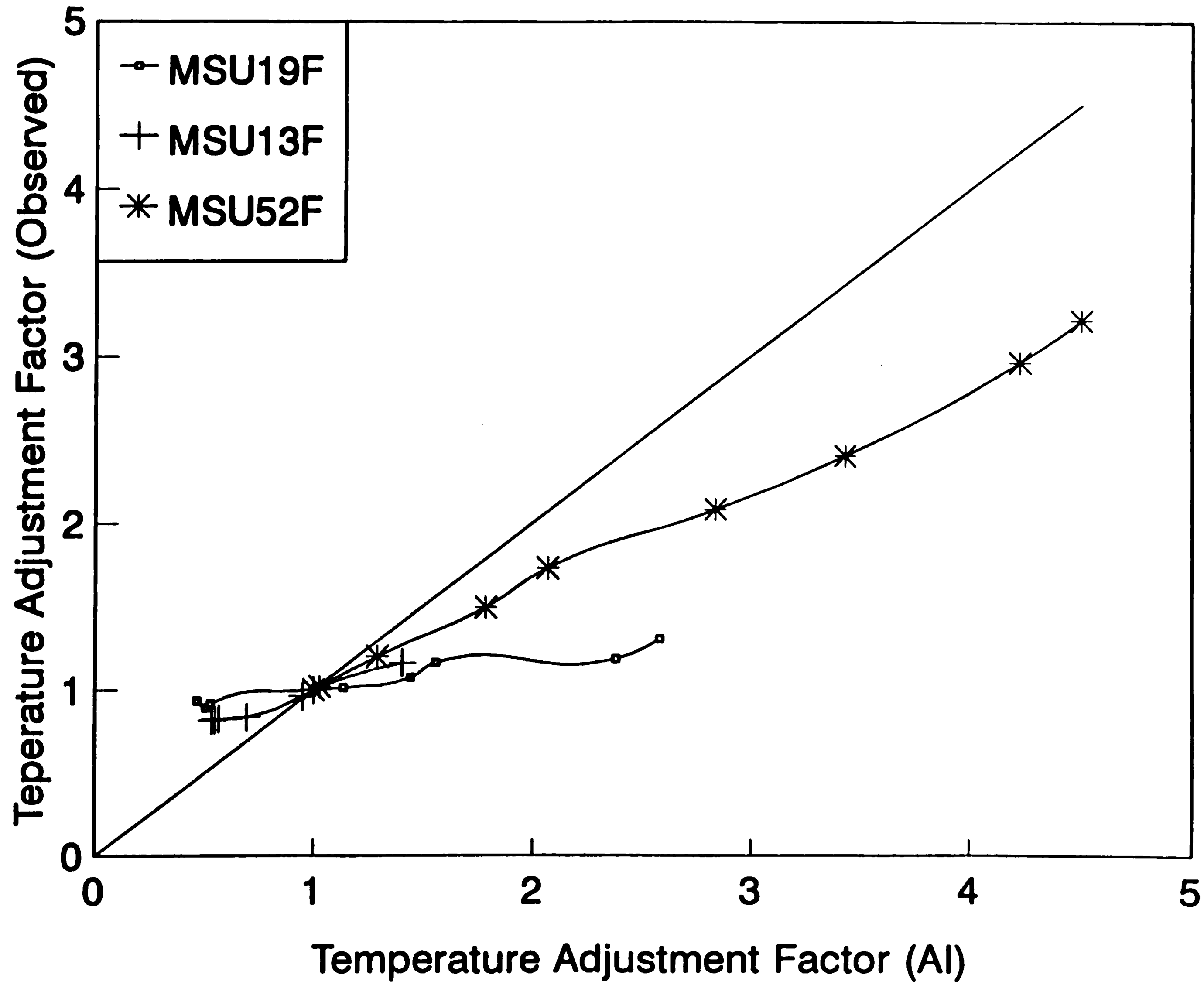

Figure 7.8. Comparison of observed and Asphalt Institute temperature conversion factors for AC modulus.

to increasing surface temperatures.

7.4 THE AASHTO TEMPERATURE CORRECTION PROCEDURE

The 1990 AASHTO Guide for the design of pavement structures introduces the concept of composite pavement stiffness (E_p) to be determined from NDT data. The Direct Structural Capacity Method (explained in Appendix L of the guide) requires the value of E_p to be determined along the length of the design project. The guide further recognizes the fact that the AC stiffness changes significantly with temperature. Since the peak deflections (D_o) are the only information from the NDT data used to calculate E_p, the guide calls for correcting the D_o values only. The reference temperature suggested in the AASHTO method is 68 °F, which is consistent with the one used in part II of the guide to determine the effective structural number (SN_{eff}) of the pavement. Further, the AASHTO Guide provides temperature correction charts. The charts were derived by using the Asphalt Institute equation along with an elastic layer analysis.

Table 7.9 provides a list of the peak deflection data measured at all 3 sites at the various test temperatures and Figure 7.9 shows a plot of these. The ratios of the peak deflection at 68°F (the reference temperature) to that at any other temperatures are also listed in the table along with the corresponding AASHTO temperature correction factors. Figure 7.10 shows a comparison of these ratios and the AASHTO factors. It can be seen that:

1. The trend in the ratios of the measured deflection at the reference temperature to that at any other temperature is similar for all three pavements.
2. The three pavement sections are less sensitive to temperature variations than predicted by the AASHTO method.

The above two observations are similar to those presented in the previous

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Table 7.9. Observed and AASHTO temperature correction factors for Do.

Pavement section	Pavement temp. (F)	Measure deflections and ratios		The AASHTO correction factor
		Deflections (mils)	Ratios	
MSU19F	56	7.14	1.020	1.094
	57	7.21	1.010	1.099
	59	7.24	1.006	1.074
	68	7.28	1.000	1.000
	80	7.33	0.993	0.906
	86	7.34	0.992	0.857
	89	7.42	0.981	0.846
	97	7.54	0.966	0.798
	100	7.74	0.941	0.797
MSU13F	56	8.87	1.008	1.081
	57	8.92	1.002	1.078
	58	8.85	1.010	1.059
	64	8.92	1.002	1.028
	68	8.94	1.000	1.000
	75	8.97	0.997	0.933
	87	9.35	0.956	0.962
MSU52F	68	9.82	1.000	1.000
	78	10.23	0.960	0.860
	86	10.56	0.930	0.813
	96	10.96	0.896	0.805
	101	11.08	0.886	0.784
	109	11.4	0.861	0.787
	114	11.8	0.832	0.762
	120	12.23	0.803	0.541
	122	12.23	0.803	0.541

Note: Ratio = Deflections @ measured temp. / Deflections @ 68 F

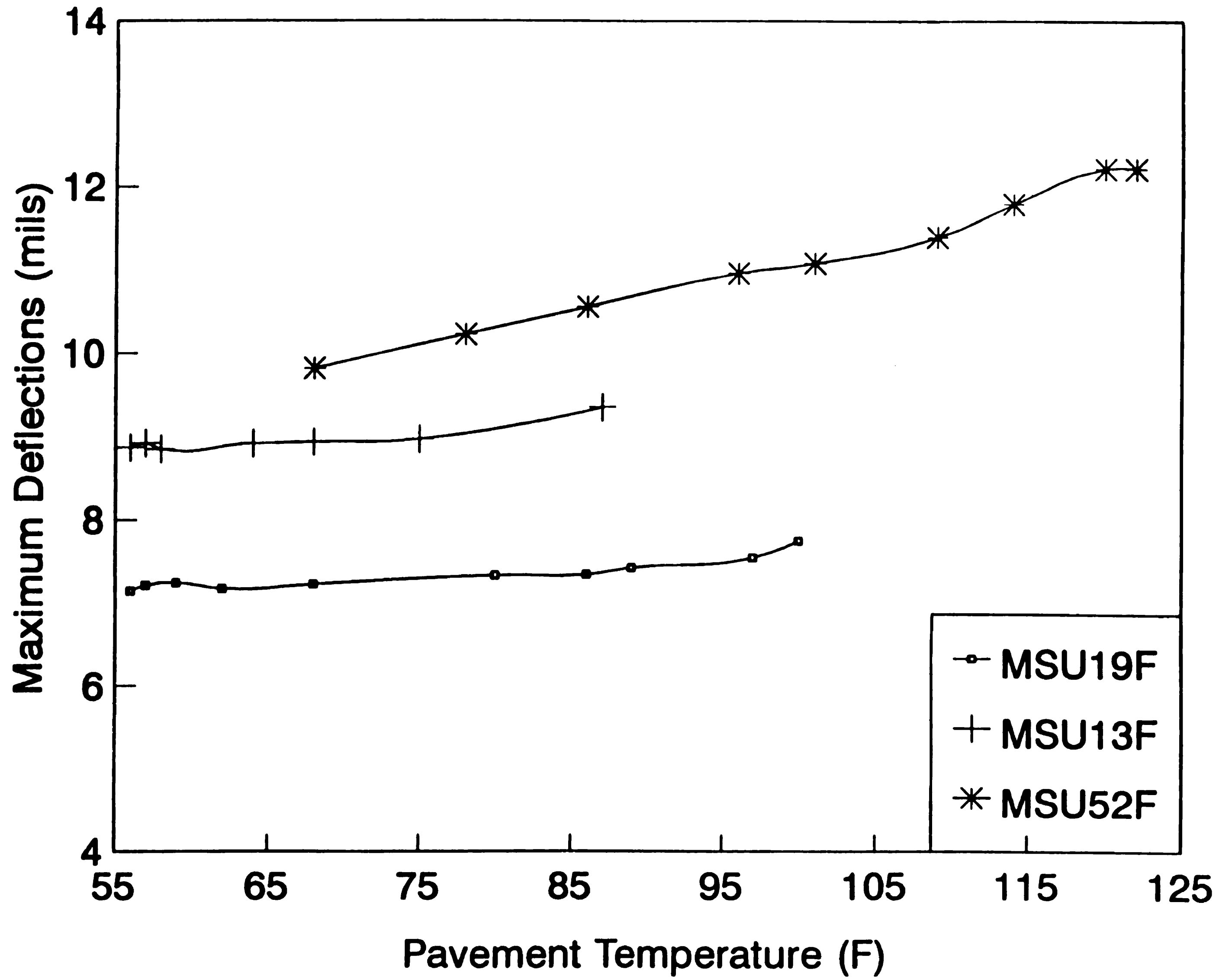

Figure 7.9. Variation of peak deflections with temperature for selected pavement sections.


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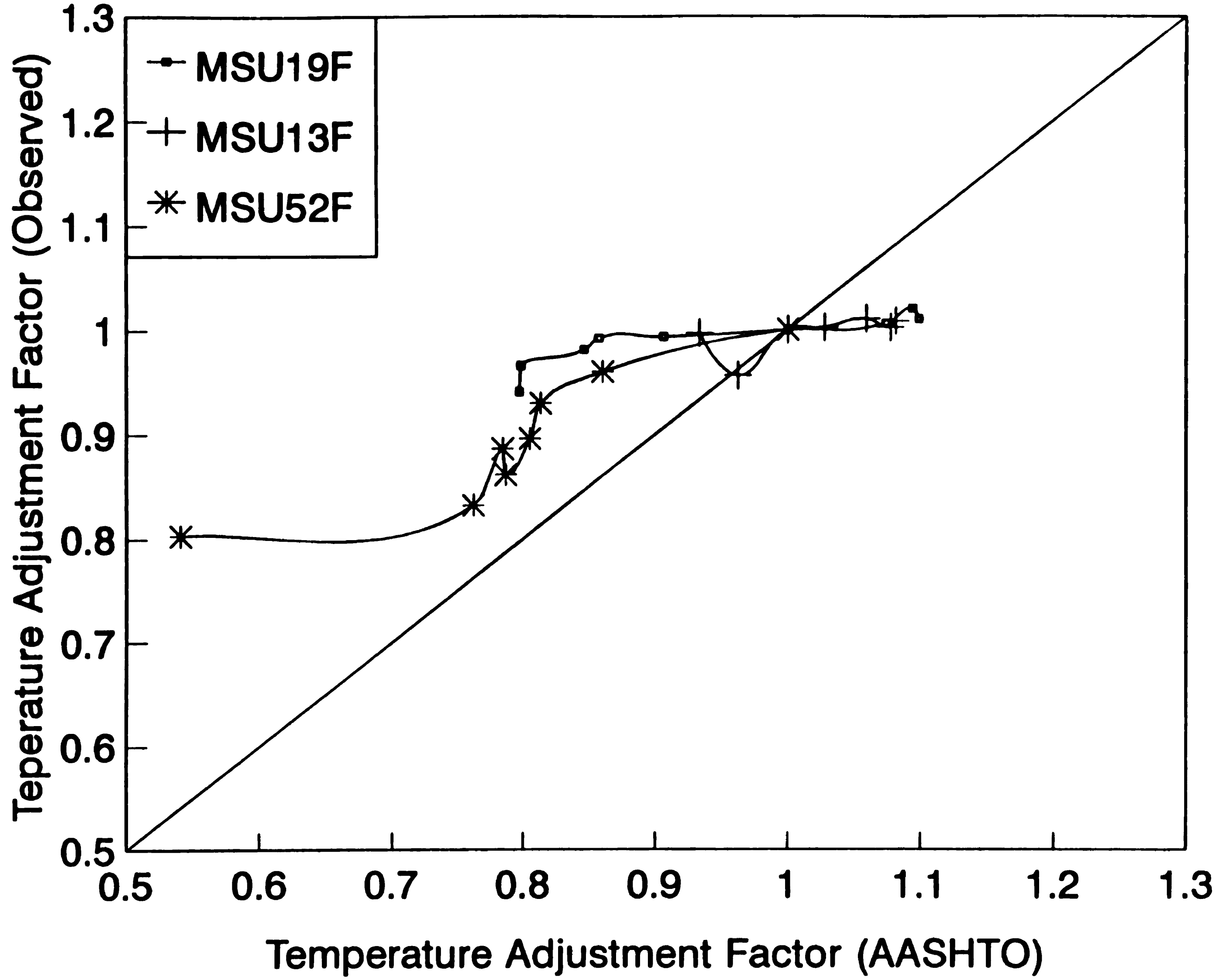

Figure 7.10. Comparison of observed and AASHTO temperature conversion factors for peak deflections.

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section. This suggests that the trend of the deflection data with temperature is similar to that of the backcalculated AC modulus. Hence the accuracy of the backcalculation is confirmed once again.

7.5 SUMMARY AND RECOMMENDATIONS

The data available to examine the validity of two suggested temperature correction methods was limited to three pavement sections only. Based on the analysis, the following conclusions are made:

1. The pavement behavior during the warming cycle is different than that during the cooling cycle. Hence, different temperature correction methods should be devised for the two cycles.
2. The three pavement sections included in this temperature correction study showed less sensitivity to temperature than that predicted by the AI and the AASHTO methods.
3. It appears that the temperature correction factors should include other pavement characteristics (such as AC thickness and stiffness, and asphalt grade) that are not part of the AI and the AASHTO methods.
4. The NDTs should be expanded to include more pavement sites than those included in this limited study.

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CHAPTER 8

SUMMARY, ACCOMPLISHMENTS, CONCLUSIONS AND RECOMMENDATIONS

8.1 SUMMARY

The continuous investment required to design, rebuild, rehabilitate, and maintain the nation's transportation infrastructure presents a major and complex task that challenges the ingenuity of every highway administrator and engineer. The infrastructure problems cannot be solved by simply studying pavement conditions and taking corrective actions. Accurate and comprehensive evaluation of the pavement structural capacity and their rates of deterioration allow the decision makers to direct the investment where needed and to properly select and schedule rehabilitation activities.

The accurate evaluation of the pavement structural capacity requires a balanced, accurate, robust, comprehensive, and mechanistically-based computer program for the analysis of the nondestructive deflection test (NDT) data. The problem associated with such analyses is that existing state-of-the-art computer programs are often not accurate and, for most programs, the accuracy of their solutions is dependent on the initial estimates specified by the user. The inaccuracy of the solution is mainly related to several important problems that affect the analyses of the NDT data. These problems include:

1. For most pavement structures, the deflection of the roadbed soil due to an applied load represents the bulk of the measured deflection data. Hence, an accurate estimate of the roadbed stiffness at the onset of the analysis can significantly increase the accuracy of the estimates of the stiffnesses of the other pavement layers.

2. The thickness of the roadbed soil (or the depth to a stiff layer) is typically not known. Erroneous estimates of this depth causes substantial errors in the solutions. Thus, an accurate estimate of the stiff layer depth based on the mechanical behavior of the pavement system should be obtained as a part of the overall analyses.
3. The pavement cross-section varies from one point to another. Variations in the thickness of the asphalt concrete layer causes significant errors in the solution. Consequently, a mechanistic routine whereby this thickness can be corrected should be a part of the analyses of the NDT data.

These and other problems were extensively examined during the course of this study and solutions were obtained and implemented in a computer program named MICHBACK. The accomplishments of this study are summarized in the next section.

8.2 ACCOMPLISHMENTS

Several major accomplishments have been achieved in this research study and they are implemented in the MICHBACK computer program. These include:

1. A new algorithm to accurately predict the roadbed modulus at the onset of the analysis (after only one call to a mechanistic analysis program) has been developed, tested, and implemented.
2. An algorithm using the modified Newton method to backcalculate layer moduli and layer thicknesses has been developed, tested, and implemented.
3. The Newton's method has been successfully extended to mechanistically calculate the stiff layer depth from deflection data. Hence, one of the major shortcomings of existing backcalculation programs has been eliminated.
4. The sensitivity of the backcalculated layer moduli of flexible pavements to the user's initial estimates (seed moduli) has been minimized.
5. User friendly-features have been developed and implemented in MICHBACK.

These features are designed not only to facilitate the use of the program, but also to encourage user interaction in the backcalculation process which is felt to be essential for obtaining meaningful results.

8.3 CONCLUSIONS

Based on the analysis of the theoretical and field measured deflection data, on an exhaustive testing of the MICHBACK program, and on a comprehensive comparison of the results of MICHBACK with those of other programs, the following conclusions were drawn:

1. The new algorithm that has been developed to predict the stiffness of the roadbed soil at the onset of the analysis by using only one call of the mechanistic analysis produces very accurate results.
2. The algorithm using the Newton's method that has been developed to mechanistically predict the stiff layer depth is accurate.
3. The MICHBACK computer program is capable of correcting an erroneous estimate of any one layer thickness and of accurately estimating the moduli of the pavement layers.
4. Poisson's ratios of the different pavement materials affect the accuracy of the backcalculated results. Hence, for each material, a range of Poisson's ratio must be known and used with caution to achieve good results.
5. The effect of inaccuracies in the deflections measured by those sensors located close to the loaded area on the backcalculated results is higher than those measured by the other sensors.
6. The original CHEVRON and ELSYM5 computer programs produce deflection basins that can be significantly different than those produced by the respected BISAR program (which is considered a very accurate computer program). This discrepancy increases with an increase in the stiffness of the pavement.

7. Although only limited deflection and temperature data were available in this study, preliminary results indicate that the Asphalt Institute (AI) and the AASHTO temperature correction methods have very limited applicability to the measured deflection basins. Further, results of the limited analysis of the NDT data obtained from those flexible pavements included in the study showed that the asphalt concrete layer may be less sensitive to temperature variations than those advocated by the AI and the AASHTO methods.
8. The measured deflection basins and the backcalculated results appear to be affected by the magnitude of the applied load. However, the exact cause of this effect could not be accurately determined or generalized for all pavement sections. Possible causes of system nonlinearities could be related to load magnitude, Poisson's ratios, and dynamic effects.
9. For any one pavement section, the variation of the backcalculated results obtained with MICHBACK from one station to another is compatible with the variation in the measured deflection basins.
10. The analysis of composite pavements is a difficult task compounded by the unknown state of distress of the PCC slabs.
11. Comparison of the backcalculated results obtained using MICHBACK with those obtained from other programs indicates:
    a) For pavements where a stiff layer is encountered, the results from MICHBACK are significantly better than those obtained with the other programs.
    b) For flexible pavements with a very deep stiff layer, the results from MICHBACK are similar to those obtained with the other programs. The increased accuracy of the results from MICHBACK becomes noticeable as the number of layers increase.
    c) For composite pavements, the results obtained with MICHBACK are

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significantly better than those of the other programs.

d) The performance of MICHBACK, measured in terms of the number of calls made to a mechanistic program, is slightly better than that of the EVERCALC 3.0 (which is known for rapid convergence).

8.4 RECOMMENDATIONS FOR FUTURE RESEARCH

Based on the results, accomplishments, and conclusions of this research study, the following recommendations are made:

1. The effect of Poisson's ratios on the backcalculated results should be examined in more detail. The feasibility of estimating some of the values of Poisson's ratios from the deflection data should be explored.
2. The effect of the asphalt concrete temperature on its modulus and on the measured deflection data should be examined in more detail to establish more dependable temperature correction functions.
3. The effects of seasonal variations on the backcalculated results and on the measured deflection basins should be undertaken as a part of a long term pavement performance (LTPP) study.
4. The MICHBACK computer program uses a linear elastic multilayer analysis program (CHEVRONX). The extent and causes of the apparent nonlinearity observed in the measured deflection data should be further investigated.
5. The user-friendly features of MICHBACK are similar to those of MICHPAVE (a linear and nonlinear finite element elastic layer program for the analysis and design of flexible pavements). The two programs enable the MDOT and other users to perform forward and backward analyses. However, the MDOT capability for the design of an overlay is very much limited to experience, a standard overlay thickness, and existing empirical procedures. It is strongly recommended mechanistic-based overlay design program be developed as an

integral part of MICHBACK. This development will have a major contribution to the capability of MDOT to perform mechanistic analysis, design, and evaluation of their pavement structures.

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