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MECHANISTIC EVALUATION OF RUBBLIZED PCC PAVEMENTS

Yigong Ji

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Mechanistic Evaluation of Rubblized PCC Pavements

Introduction

HMA overlays rubblized PCC is considered to be one of the most common types of PCC rehabilitation. The objective of rubblization is to eliminate reflection cracking in HMA overlay by destroying the integrity of the existing slab action. Currently, the Indiana Department of Transportation is performing pavement rehabilitation based on the 1993 AASHTO Pavement Design Guide. Two arbitrary factors including the Layer Coefficient (LC) and the Structural Number (SN) are utilized to quantify the strength of the total pavement structure. The layer coefficients represent the relative strength of the construction materials and convert the actual thickness into a structural number.

While the AASHTO guide provides the LC and SN, this guide did not develop any model to predict the layer coefficients. Currently, INDOT is using 0.34 for HMA, 0.20 for rubblization layer, and 0.14 for base layer, respectively. The INDOT Pavement Design Engineers use both the LC and the CBR value for calculating the thickness of the flexible (HMA) overlay. If the overlay is being placed for structural improvements, the required overlay thickness depends on both the structural capacity for meeting future traffic demands and the structural capacity of existing pavement. Lower LCs result in over estimating the required HMA thickness. Consequently, cost of the pavement construction projects goes up and most importantly the user cost goes up. Therefore, there is a need for reconsidering the design parameter of LCs of flexible and rubblized concrete pavements on the basis of the 1993 AASHTO Design Guide.

The objective of the research was to explore a mechanistic calculation procedure for LCs of HMA overlay rubblized composite pavement and to investigate the changes of LCs, MR, and SN under the influence of traffic immediately after reopening of rehabilitated pavement. This project is intended to assist INDOT in developing tools for assessing LCs for the asphalt surface layer and rubblization base layer. Consideration was given to parameters such as SN, MR, and variation over several years.

Findings

This report begins with a literature review from other researchers. As many as 12 newly constructed pavement sections were selected for further investigation. During the course of this study, Non-Destructive Tests (NDT) were conducted on the pavement surface using the Falling Weight Deflectometer (FWD) testing equipment during a period of four years. The multi-layer system was examined and adopted as a model to calculate the rubblization pavement responses. A computer program was developed by the author to simulate the layer coefficients for different layers and overall pavement SN from the back-calculated moduli. A summary of the major findings are presented below.

FWD successfully determines the LC of the rubblized composite pavement. The mechanistic method (M-M) provides LCs similar to those recommended by the values from the 1993 AASHTO Pavement Design Guide for both HMA and rubblization layers.

The empirical method (E-M) provides HMA LCs different from the typical values from the 1993 AASHTO Pavement Design Guide and provides rubblization LCs similar to the typical values from the 1993 AASHTO.

There is a high correlation between the SNs obtained from in-situ FWD deflections and

layer elastic simulation deflections. This indicates that there is no significant difference between these two different procedures to evaluate the structural capacity. Therefore, the elastic layer system theory could provide good simulation for the in-situ FWD testing.

The back-calculated moduli fluctuated from approximately 6 ksi to 19 ksi. In addition, the resilient modulus (M_R) has significant differences during the testing period. Therefore, the assessment of how the material properties change over time and location is difficult. Also, seasonal moisture variation should be taken into account when designing HMA overlays on the rubblized PCC pavement.

The LCs for HMA obtained from empirical method (E-M) are relative higher than the values provided in the 1993 AASHTO Pavement Design Guide. An appropriate adjustment factor is recommended to estimate the LCs. These results could occur because the back-calculated modulus is always higher than the modulus from a lab testing.

LC is a function of many parameters, such as the modulus, the position of the layer in the pavement, and other parameters. Average LCs of 0.42 for HMA and 0.22 for rubblization layer are recommended for designing HMA overlays on the rubblized PCC pavement.

Implementation

This project demonstrates the in-situ layer coefficients mechanistically evaluated in a newly constructed rubblization pavement. It is recommended that the LCs developed in this study would be considered as a standard method for INDOT pavement design and analysis.

Based on research, it is recommended that INDOT use LCs for a pavement design to enable successful thickness design of rubblized pavements. Furthermore, the layer coefficients recommended are applicable for rubblized pavements and for asphalt pavements.

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MECHANISTIC EVALUATION OF RUBBLIZED PCC PAVEMENTS

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The contents of this report reflect the views of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration and the Indiana Department of Transportation. This report does not constitute a standard, specification or regulation.

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16. Abstract

This report presents a mechanistic approach and procedures for determining layer coefficients to characterize the in-situ behaviors of rubblized pavements. This procedure was developed based on the 1993 AASHTO Pavement Design Guide utilizing Falling Weight Deflectometer (FWD) testing and in-place simulation using back-calculation layer modulus. In order to evaluate the rubblized pavement systematically, twelve constructed pavement sections were extensively tested by FWD annually after construction. The FWD data was processed using the proposed mechanistic procedure. In addition, statistical analysis was conducted to compare the pavement structure parameters each year, including layer coefficient and in-situ resilient modulus using Analysis of Variance (ANOVA) and Z-tests.

In the ANOVA, the comparison of the structure capacities derived from the field FWD tests with those from the mechanistic procedure supports the null hypothesis, which concluded there is no difference between the structural numbers from these two methods. Also, statistically, the layer coefficients in each year are significantly different. The in-situ resilient moduli for each year varied significantly. The results from Z-test show that for Hot Mixed Asphalt (HMA) layers, the layer coefficient confidence interval could be as high as 0.70 and also as low as 0.38. For the base layers, the layer coefficient confidence interval could be as high as 0.25 and also as low as 0.16. A layer coefficient of 0.42 is recommended for HMA and 0.22 for rubblized layer for the design of HMA overlay on rubblized PCC pavement.

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Chapter 1 INTRODUCTION

1.1 Introduction

There are various alternatives for Portland Cement Concrete (PCC) rehabilitation techniques. Some of these techniques are commonly used in Indiana. These include the following methods: overlay, crack-and-seat with overlay, break-and-seat with overlay, and total reconstruction. The selection of alternatives primarily depends on the pavement type and its existing condition. Among these alternatives, HMA overlay rubblized PCC is considered to be the most common type of PCC rehabilitation. In this technique, the concrete PCC slab is reduced in-place to approximately aggregate base material size.

The objective of rubblization is to eliminate reflection cracking in HMA overlay by destroying the integrity of the existing slab action. Rubblization is applicable when there is little potential of retaining slab integrity and structural capacity of the original concrete pavements. Typically, the slab is reduced into pieces ranging from 9 to 12 inches. Subsequently, this layer becomes a high quality, free draining aggregate base layer.

Currently, the Indiana Department of Transportation (INDOT) is performing pavement rehabilitation based on the 1993 AASHTO Pavement Design Guide. In this procedure, the FWD has been commonly accepted by the pavement rehabilitation as an effective NDT device. The structural adequacy then could be evaluated by conducting an overlay design using the FWD data measured on the top of the HMA. There are two concerns regarding the overlay design: one is the layer coefficient (LC) concept; the other one is the structural number (SN). SN is deployed to quantify the strength of the total pavement

structure. The LCs are needed to characterize the component materials of the pavement structure. In short, this situation required the development of reliable and efficient approaches to assess the structural state of the existing pavement and recommendation of the rehabilitation and maintenance strategies.

1.2 Problem Statement

The most popular existing pavement in the state of Indiana is composite pavement. Much of this pavement has been rehabilitated to composite pavement through crack seat, break seat, and rubbilization after old PCC pavement gradually deteriorates with the development of different types of distress due to traffic and environmental factors. Rubblizing has been used extensively in the last 20 years because of the advantages of preventing reflecting cracks. In 1993, the AASHTO guide explained how to design the rubblized pavement. However, the guide did not develop any model to predict the layer coefficients. Currently, INDOT is using 0.34 for HMA, 0.20 for rubblization layer, and 0.14 for base, respectively. Based on the results of the SPR–2064 project entitled "Verification of Design Parameters for Overlaid Rubblized PCC Pavement," INDOT is implementing a layer coefficient of 0.22 for rubblization layer. The INDOT Pavement Design Engineers use both the layer coefficient and the California Bearing Capacity (CBR) value for calculating the thickness of the flexible (HMA) overlay. If the overlay is being placed for structural improvements, the required overlay thickness depends on both the structural capacity for meeting future traffic demands and the structural capacity of existing pavement. Lower layer coefficients result in over estimating the required HMA thickness. Consequently, cost of the pavement construction project increases.

With all of the considerations stated above, there is a need for reconsidering the design parameter for layer coefficients of flexible and rubblized concrete pavements on the basis of the 1993 AASHTO Design Guide.

1.3 Research Objective

The primary objective of the research was to explore a mechanistic calculation procedure for layer coefficients of HMA overlay rubblized composite pavement.

The second objective of this research was to investigate the changes of layer coefficients, M_R, and SN under the influences of traffic during the 4-year period of test time.

The basic objectives of this research can be summarized as follows:

1. Calculate in-situ layer coefficient for rubblized pavement mechanistically;
2. Monitor the variation of layer coefficients after pavement construction;
3. Recommend the layer coefficients for designing HMA overlay rubblization pavement;
4. Establish comparisons between a mechanistic procedure and an empirical procedure.

1.4 Report Layout

This report is organized as follows:

Chapter 2 presents the historical background of composite pavement, current overlay design methods using rubblized, crack/seat, and break/seat composite pavement. Also this chapter introduces nondestructive testing using FWD and backcalculation, layer coefficients in the AASHTO design, and modeling for an elastic layer pavement.

Chapter 3 develops the elastic model of a composite pavement and examines the robustness of the CHEVRONX computer program and sensitivity analysis from the mechanistic model under the FWD loading. Further, this chapter describes the mechanistic development of the rubblized pavement layer coefficients.

Chapter 4 describes numerical algorithms to evaluate the pavements.

Chapter 5 presents field verification. Twelve pavement sections were tested and analyzed extensively. In light of these results, two statistical methods were employed to process these findings. The comparison between empirical and mechanistic methods is discussed in detail.

Chapter 6 provides summary and conclusions.

Chapter 2 LITERATURE REVIEW

2.1 Introduction

Rehabilitation of existing PCC pavement is an important issue facing transportation agencies. Overlay on deteriorated PCC is an accepted option for restoring rideability, improving functional pavement performance and increasing structural deficiencies of existing PCC. Previous studies showed that the reflection cracking was a major distress in HMA overlays of existing PCC pavements. Therefore rubbilizing, crack and seat, and break and seat techniques are used to reduce the size of PCC slabs to minimize the differential movements at existing cracks and joints. This process minimizes the occurrence and severity of reflection cracks.

Crack and seat is performed on JPCP to reduce the effective slab length and reduce slab movement. This process involves cracking the slab into pieces typically one to three feet in size. Field testing of several crack and seat JPCP projects showed a wide range in backcalculated modulus values among different projects, from a few hundred thousand psi to a few million psi.

Break and seat is conducted on JRCP to shorten the slab length and to reduce slab movement. This process includes the requirement to rupture the reinforcing steel across each crack or to break its bond with the concrete. If the reinforcement is not ruptured or its bond is not broken, the differential movements at working joints and cracks will not be reduced and reflection cracks will occur.

Rubbilizing can be used on all types of PCC pavements. It destroys the slab action of rigid pavement. Fracturing the slab into pieces less than 12 inches reduces the slab to a high-strength granular base. Deflection testing of several rubbilized projects has shown a wide range in backcalculated modulus values among different projects. This range could be from less than 100,000 psi to 1,000,000 psi, and within project coefficients of variation could be as much as 40 percent (Galal, 1999).

Rubblizing has been used extensively by agencies in the past 20 years (Decker, 2006). In general, field performance of HMA overlays on rubblized PCC slab has been found to be good to excellent. Therefore, the rehabilitation approach itself no longer needs to be considered as research (Decker, 2006).

This chapter will present a brief literature review on the pavement design, nondestructive testing, backcalculation, and mechanistic analysis of pavement.

2.2 1993 AASHTO Design for AC over Fracture PCC

Empirical research from AASHTO provides overlay thickness design procedures on the basis of limited AASHTO road tests and limited pavement cross section. The AASHTO guide recommends using the following formula for determining the resilient modulus value of the subgrade soil based on the deflection measurements:

$$M_R = \frac{0.24P}{d_r r} \tag{2.1}$$

Where:

M_R = subgrade resilient modulus (psi)

P = applied load (pounds)

d_r = deflection at a distance r from the center of the load (inches)

r = distance from center of load (inches)

The minimum distance from the loading plate is determined with the following formula:

$$r \geq 0.7a_e \tag{2.2}$$

Where:

r = distance from center of load (inches)

a_e = radius of the stress bulb at the subgrade-pavement interface (inches)

The value of a_e is determined from the following formula:

$$a_e = \sqrt{a^2 + \left(D\sqrt[3]{\frac{E_p}{M_R}}\right)^2} \tag{2.3}$$

Where:

a_e = radius of the stress bulb at the subgrade-pavement interface (inches)

a = NDT load plate radius (5.91-in.)

D = total thickness of pavement layers above the subgrade (inches)

E_p = effective modulus of all pavement layers above the subgrade (psi)

AASHTO defines a two-layer pavement structure as a constructed material layer and subgrade layer. The value of constructed material layer, E_p is determined from the following formula:

$$d_0 = 1.5pa\left[\frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}} + \frac{1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}}{E_p}\right] \quad (2.4)$$

Where:

d_o = deflection measured at the center of the load plate (inches)

p = NDT load plate pressure (psi); a = NDT load plate radius (5.91-in.)

D = total thickness of pavement layers above the subgrade (inches); M_R = subgrade resilient modulus (psi);

E_p = effective modulus of all pavement layers above the subgrade (psi)

Eq. 2.1 through Eq. 2.4 from the 1993 AASHTO procedure were developed to estimate the subgrade modulus from the FWD test data. These values were utilized to estimate the CBR values using the correlation equation:

$$CBR = \frac{M_R}{1500} \quad (2.5)$$

The effective structural number (SN_{eff}) for fractured PCC pavements, based on non-destructive deflection measurements, is often obtained using a Falling Weight Deflectometer (FWD). The NDT method for determining SN_{eff} follows the assumption that the structural capacity of the pavement is a function of its total thickness and overall

stiffness. The relationship between SN_{eff}, thickness, and stiffness is determined by the following formula:

$$SN_{eff} = 0.0045D\sqrt[3]{E_p} \tag{2.6}$$

Where:

SN_{eff} = effective structural number

D = total thickness of all pavement layers above the subgrade (inches)

E_p = effective modulus of pavement layers above the subgrade (psi).

The SN_f value is determined by using several pieces of information. These items include the following: the effective design subgrade resilient modulus, design Present Serviceability Index (PSI) loss, overlay design reliability (R), and the overall Standard Deviation (S_o) for flexible pavement.

The overlay thickness (D_{ol}) is determined by taking the difference between the SN_f and SN_{eff} values and dividing this quantity by the layer coefficient for new asphalt pavement.

$$D_{ol} = \frac{SN_f - SN_{eff}}{a_{ol}} \tag{2.6}$$

2.3 Non-destructive Pavement Testing and Backcalculation

NDT is the most popular method in the pavement evaluation and overlay because of its advantages of low operational cost, short test duration and full scale model testing. Falling Weight Deflectometers (FWD) are widely used to evaluate the structural

properties of pavements nondestructively. Backcalculation of pavement properties from FWD data is usually carried out by matching the measured deflections under a known load with theoretical deflections generated by an analytical model of the pavement by varying the elastic moduli. Such procedures usually use error minimization techniques to minimize either the absolute or the squared error, with or without weighing factors.

For decades, numerous backcalculation computer programs have determined layer moduli. Most of these programs are 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. A number of computer programs, such as BISDEF (Bush, 1985), BOUSDEF (Roesset, 1995), CHEVDEF (Bush, 1980), and COMCOMP (Irwin, 1994), have been developed for back-calculation analysis using this method.

Ali and Khosla (1987) concluded that ELMOD and VESYS model are more appropriate for backcaluation purpose after they compare four backcalculation models including VESYS, ELMOD, and MODCOMP2. Nevertheless, the Strategic Highway Research Program (SHRP) indicated that the MODCOMP3, MODULUS, and WESDEF model are more appropriate for backcalculation purpose (PCS/Law, 1993) after testing six programs: ELCON, ILLI-BACK, ISSEM4, CODCOMP3, MODULUS, and WESDEF. In general, all of these programs use an elastic layer static program; the only difference in the processes is in the way the programs adjust the theoretical basin to the deflection basin. The 1993 AASHTO overlay design procedures treat pavement as a two layer system, in which M_R and equivalent moduli forpavement layer are backcalculated. Even though these computer programs provide pavement engineers with a quick method of

obtaining layer moduli, the following problems associated with back calculation procedures must be consideed (Uddin, 1984):

1. The non-uniqueness of the resilient modulus back calculated from the measured deflection basin.
2. Errors due to possible variation in thickness of pavement layers.
3. Errors involved in assuming a semi-infinite subgrade.
4. Time involved in the iterative process.
5. Errors in back calculated moduli because of the nonlinear behavior of granular layers and subgrade.
6. Errors involved in using input values out of the range for which the model was calibrated.

2.4 Temperature Correction

Seasonal climate variations and temperature changes affect the response of pavement. Therefore, the AASHTO Design Guide requires that seasonal and temperature corrections should be considered when FWD field testing is conducted. Previous FWD research results have shown that pavement deflections measured using the FWD at the same location and backcalculation results varied with the time of day when data were taken. In many cases, the variations can be the influence of temperature and moisture on pavement material properties due to seasonal variations. Currently, AASHTO temperature correction equations on flexible pavement are used in the project level testing in the state of Indiana. Therefore, for the FWD test applications, it is necessary for

engineers to adjust the results obtained from the FWD or correct them to reference or standard conditions of temperature, moisture, and loading magnitude.

Several research studies on deflection correction due to environmental factors have been conducted in the past decade. The NCHRP Report 327 (Lytton et al., 1990) stated that the temperature and moisture is important. Strategic Highway Research Program (SHRP) researchers established the procedure based on a multi-layer analysis to correct measured maximum surface deflection to a standard temperature. The Asphalt Institute (1982) proposed a relationship between temperature at a given depth below the surface and the mean monthly air temperature. The MODULUS program also has a temperature correction procedure based on the U.S. Army Cops of Engineer's equations (Bush, 1987).

2.5 Layer Coefficients

The layer coefficient concept is derived from the AASHTO Road Test in which the structural capacity of the flexible pavement is represented by a single parameter, the structural number (SN). The structural number is determined by SN = $a_1t_1 + a_2t_2 + a_3t_3$ and assuming a drainage coefficient of one for each layer. The layer coefficient expresses the empirical relationship between SN and thickness and is a measure of the relative ability of the material to function as a structural component of the pavement (1993 AASHTO).

In the Odemark hypothesis (Odemark, 1949), the equivalent thickness is computed below:

$$h_e = h_1 \sqrt[3]{\frac{E_1}{E_{ref}}} + h_2 \sqrt[3]{\frac{E_2}{E_{ref}}} + h_3 \sqrt[3]{\frac{E_3}{E_{ref}}} \tag{2.7}$$

where,

h_e = the equivalent thickness,

h_1, h_2, h_3 = the thickness of layer 1, 2, and 3 respectively,

E_{ref} = the reference modulus (75,644 MPa), and

E_1, E_2, E_3 = the elastic modulus of 1st, 2nd and 3rd layers respectively.

Accordingly, the layer coefficient of the second layer, a_2, can be evaluated as

$$a_2 = \sqrt[3]{\frac{E_1}{E_{ref}}} = 0.0045D\sqrt[3]{E_2} \tag{2.8}$$

Where, E_{ref} = 75,664 MPa.

Based on the Odemark equivalent thickness theory, Lukennan (1987) conducted a research study and concluded that the calculated structural layer coefficient for rubblized materials was 0.28 using a typical backcalculated modulus of 3,500 MPa for rubblized materials.

The 1993 AASHTO Pavement Design Guide provides recommended layer coefficients for rubblized PCC pavement, which vary from 0.14 to 0.30. However, the guide does not provide any procedure to select those layer coefficients. A previous study conducted in Indiana (Galal, 1999) recommended a layer coefficient of 0.22 for the rubblized PCC pavement layer.

Ullidtz (1987) pointed out that the layer coefficient not only reflects the stress distribution ability of the material, but also reflects the strength of the material and location of the material in the pavement structure. The following empirical equations

related to layer coefficients are listed. In addition, the corresponding figures are illustrated in Figure 2.1 through Figure 2.3.

Asphalt concrete

$$a_1 = \left(0.40 * \log\left(E / 435\,\text{ksi}\right) + 0.44\right) \tag{2.9}$$

Granular base

$$a_2 = \left(0.25 * \log\left(E / 23\,\text{ksi}\right) + 0.11\right) \tag{2.10}$$

Granular subbase

$$a_3 = \left(0.23 * \log\left(E / 23\,\text{ksi}\right) + 0.15\right) \tag{2.11}$$

Broken and seated Portland cement concrete

$$a_2 = \left(0.27 * \log\left(E / 435\,\text{ksi}\right) + 0.35\right) \tag{2.12}$$

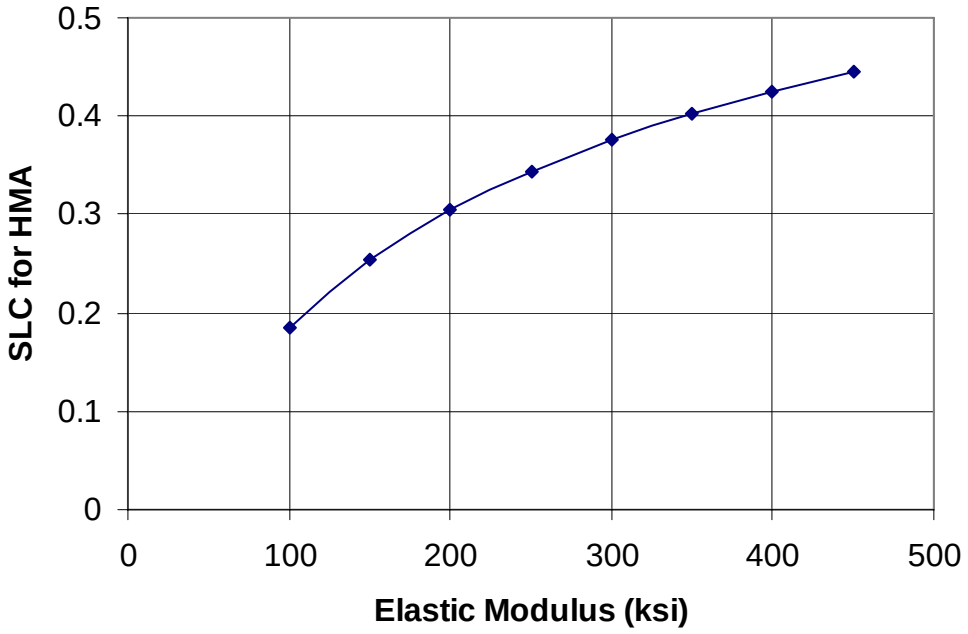


Figure 2.1 Suggested layer coefficients for existing AC pavement layer

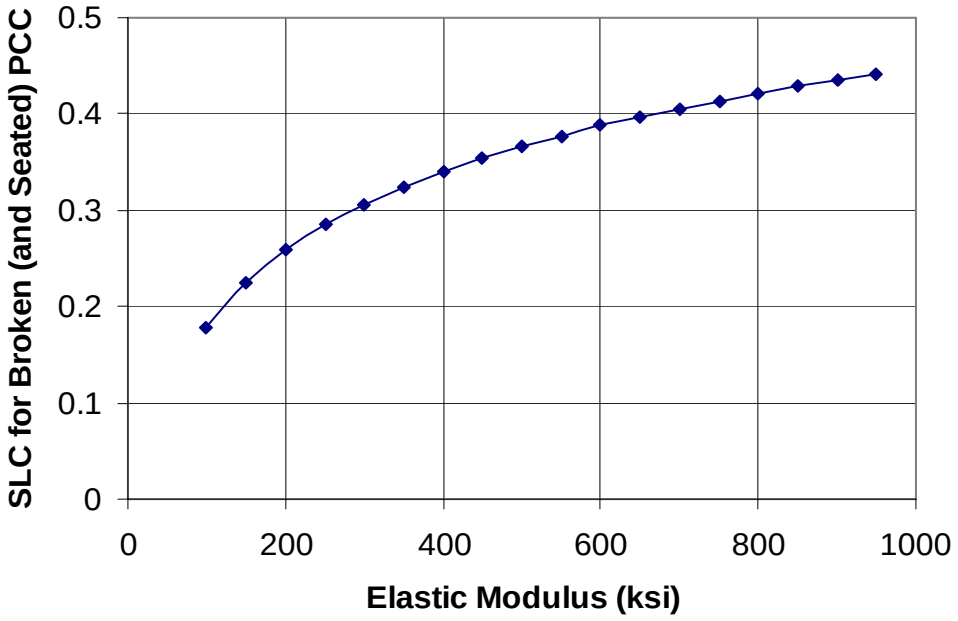


Figure 2.2 Suggested layer coefficients for broken (and seated) Portland cement concrete

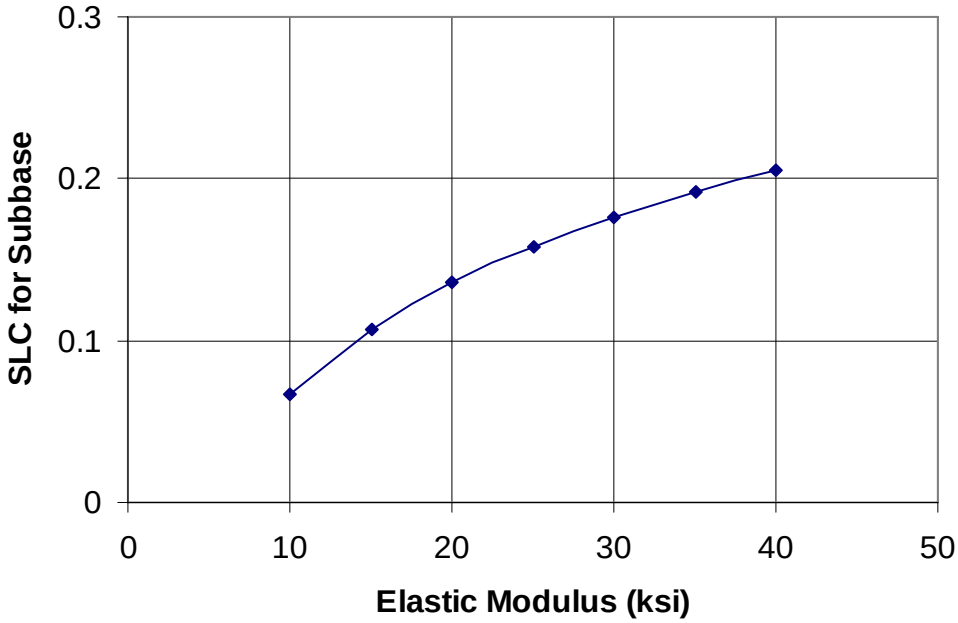


Figure 2.3 Suggested layer coefficients for subgrade existing AC pavement layer

2.6 Mechanistic Analysis of Pavement

2.6.1 Elastic Layer System

The simplest way to characterize the behavior of flexible pavements is based on Boussinesq's solution that models a flexible pavement as a homogeneous, isotropic, and elastic half-space. Later, Burmister (1943) presented a method for determining stress, strain and displacement in a two layer system. Based on Burmister's method, Acum and Fox (1951) presented the solution for a three-layered pavement system. Since then, a large number of computer programs have been developed for calculating the analytical response of multi-layered flexible pavements to different load and layer interface conditions, including CHEVRON (Warren and Dieckmann, 1963), BISAR (Dejong et al., 1973), ELSYM5 (Kopperman, 1985), and KENLAYER (Huang, 1993). Finite element

analysis is another method that can model a layered elastic system in which the layered pavement is divided into many small “elements”. The stress state in each element is calculated using the theory of elasticity. Programs such as MICH-PAVE (Yeh, 1989) and ILLI-PAVE (Raad and Figueroa, 1980) have been developed using the finite element method. Other approaches, such as the equivalent thickness method based on the equivalent layer theory, were introduced by Odemark (1949) and Ullidiz (1987).

2.6.2 Nonlinear Elastic Model

It is well known that granular materials and subgrade soils are nonlinear with their elastic modulus varying with the level of stress. Various constitutive equations have been developed to describe the behavior of nonlinear elastic materials. Computer programs that can handle nonlinear behavior within the layered elastic theory include KENLAYER (Huang, 1993) and NELAPAVE (Irwin, 1994). The finite element computer programs MICHPAVE and ILLIPAVE can model non-linear material behavior more accurately.

Chapter 3 MODELING OF RUBBLIZED COMPOSITE PAVEMENT

3.1 Introduction

Although HMA overlay rubblized pavement is neither a “true” rigid pavement nor a “true” flexible pavement, an M-E design study conducted by the Iowa Department of Transportation shows that HMA overlay rubblized PCC pavement could be considered a multilayer system (Ceylan, 2005). Therefore, the pavement system could be treated as a system of layers that are infinite in the horizontal direction and underlain by an elastic half-space. The materials are assumed to be isotropic and linearly elastic. Full interface bonding is assumed at the layer interfaces. The elastic moduli change with depth, from layer to layer, but are assumed to be constant within each layer. For the present application, the top layer represents the asphalt surface, which is supported by the rubblized base, subbase and subgrade.

3.2 Stucture of Rubblization

There are three main objectives of rubblizing concrete pavements:

1. Destroying the integrity of the concrete pavement joints as to eliminate reflective cracks.
2. Destroying the integrity of the concrete slab by debonding the temperature steel to avoid reflective cracks and by producing full depth rubblized material.
3. Changing the concrete slab into a particulate media whose maximum size is less than 6 in. This process would allow the rubblized concrete slab to behave like a base layer.

The rubblizing alternative changes the pavement cross-section from rigid to flexible pavement as shown schematically in Figure 3.1. If the above three objectives are achieved, the rubblized pavement will have flexible-like behavior and can be better explained, modeled and analyzed using the multi-layer elastic system. On the other hand, if one or more of the above objectives are only partially achieved, then the rubblized pavement may have flexible-, flexible-composite-, or composite-like behavior.

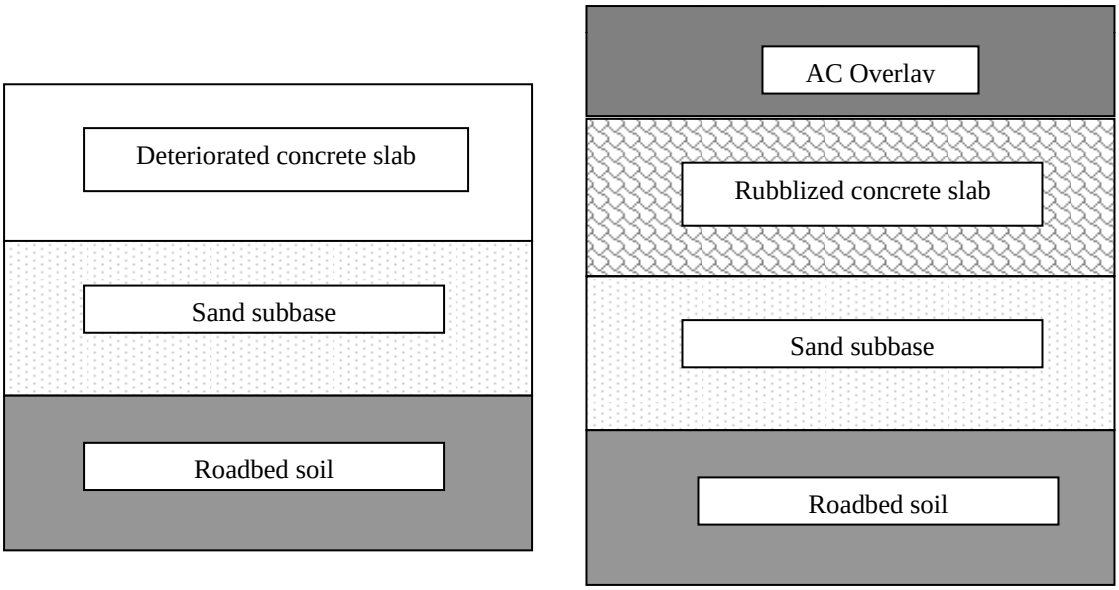


Figure 3.1 Schematic of concrete and rubblized pavement cross-sections

The modulus of a fractured PCC slab is important parameters for the successful performance of HMA overlays on the rubblization process. The greater the degree of slab fracturing and/or steel-concrete debonding achieved in the construction process, the lower the effective slab modulus will be (Decker, 2006). Therefore, the rubblized layer modulus has a significant impact on structural requirements of HMA overlay to eliminate distress caused by inadequate structure capacity of the rubblized layer. As the rubblized layer modulus decreases, the likelihood of having reflected crack problems in the HMA

overlay is significantly reduced, while the likelihood of having insufficient structural capacity increases (Decker, 2006).

3.3 Modeling of Composite Pavement of HMA on Fractured PCC

After a pavement is rubblized, the PCC slabs are broken into pieces less than 12 inches in size, and thus it becomes a high-strength granular base. Rubblized pavement has flexible-like behavior and can be better explained, modeled, and analyzed using a multi-layer elastic system. This analysis assumes that pavements are loaded only statically. Similar to the mechanistic characteristics of flexible pavement, all layers carry parts of loading.

3.3.1 Robustness of the CHEVRON Program

The CHEVRONX was used to analyze rubblized pavements. The validity and then the application of CHEVRONX to pavements were examined. Validation was accomplished by different software. The assumptions and limitations used in the analyses of each of the three programs are listed in Table 3.1.

To assess the robustness of the CHEVRONX and to compare the results of the three computer programs, several pavement cross-sections consisting of 4 pavement layers with rubblized layer were analyzed. Various mult-layer programs such as CHEVRONX, KANLAYER (Huang, 1993), and WESLEA (The U.S. Army Engineer Waterways Experiment Station, 1989), were used to calculate the responses of FWD 9,000-lb single wheel load with pressure of 82.02 psi. Figure 3.2 depicts the analyses model. Figure 3.3 illustrates numerical examples of analyses models using the underlined values listed in the Table 3.2. The results indicate that despite the different programs used in the analysis,

the load-induced radial tensile stresses at the top and at the bottom of the AC layer calculated using the three programs are practically the same.

Table 3.1 Assumptions used in the analyses and limitations of the CHEVRONX, KANLAYER and EVERSTRESS programs

Items	Assumptions		
	CHEVRONX	KENLAYER	WESLEA
Layer Interface	Full continuity (no slippage between the interfaces are allowed)		
Symmetry	Axisymmetry around the z-axis		
Material properties	Linear elastic isotropic and homogeneous	Linear elastic isotropic and homogeneous, nonlinear elastic, stress dependent	Linear elastic isotropic and homogeneous
Model dimensions	Infinite extent in horizontal direction Infinite thickness for the bottom layer		
Number of layers	5	6	6
Analyses types	Closed-form solution	Closed-form solution	Closed-form solution

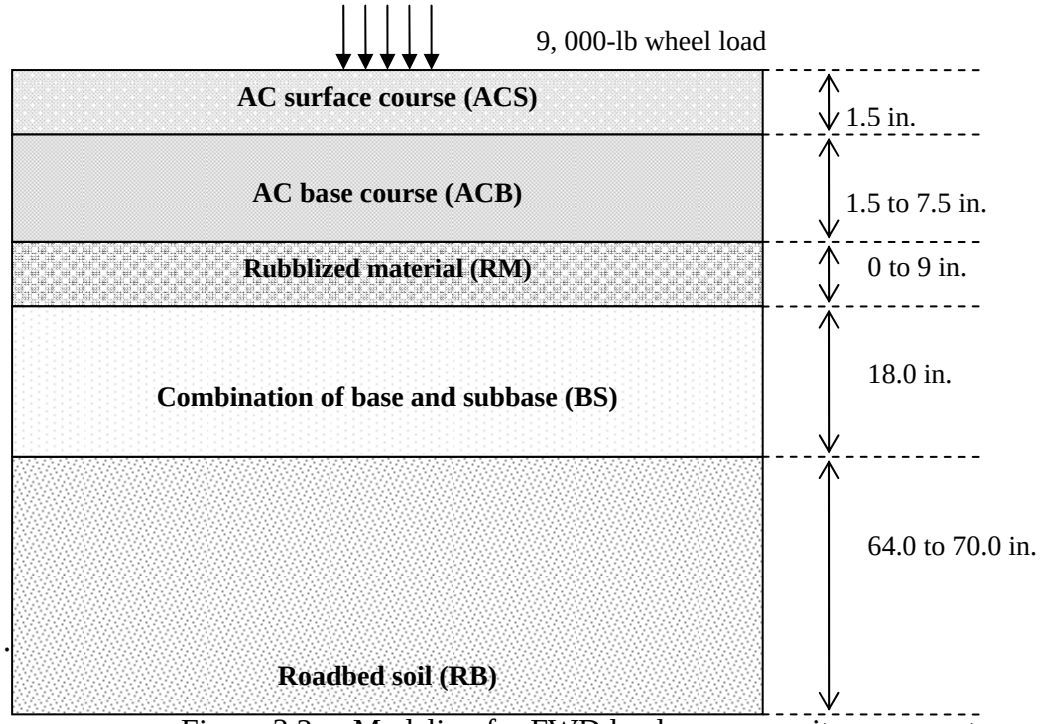


Figure 3.2 Modeling for FWD load on composite pavement

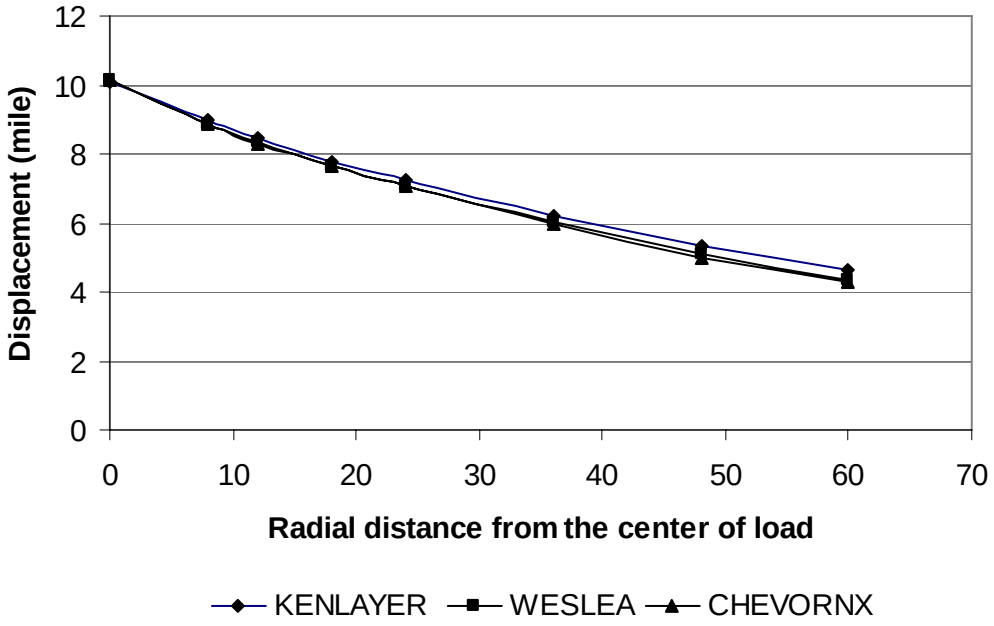


Figure 3.3 Comparison of the surface deflection obtained from the KENLAYER, CHEVRONX, and WESLEA computer programs

3.3.2 Sensitivity of Parameters

The WESLEA program, developed by Waterways Experiment Station, was used in the 2-D analyses to assess the impacts of the pavement layers and roadbed soil properties on the stresses induced in the AC layer. Table 3.2 provides the thicknesses and material properties of each layer used in the analyses. The range of the values of each variable listed in the tables represents the properties of rubblized, flexible and composite pavement sections. The calculation results were listed in Figure 3.4 through Figure 3.7

Table 3.2 The range of pavement cross-sections and material properties used in the sensitivity analyses

AC courses/ Layers	Thickness (in.)	Modulus (ksi)	Poisson's ratio
HMA	4, **8**, 12	100, 300, **500**, 1000	**0.35**
Rubblized material	6, **8**, 10	30, 50, 100, **300**	**0.3**
Combination of base and subbase	6**, 9,** 12	10, 20, **30**, 50	**0.40**
Roadbed soil	Infinite	3, 5, **10**, 20	**0.45**

Note:
1. The bolded and underlined figures represent the datum value of each variable.

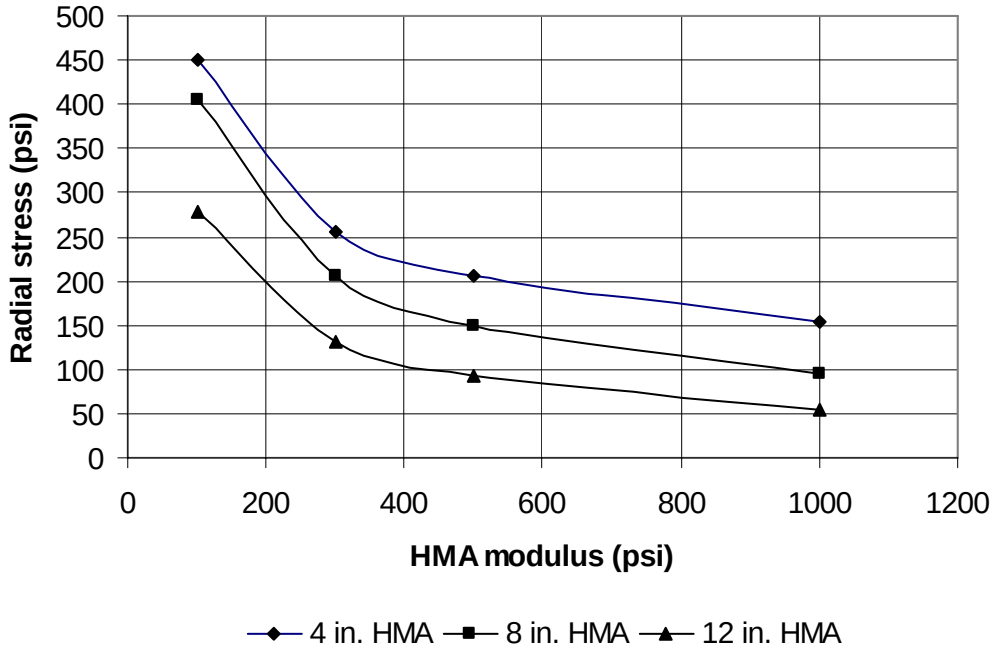


Figure 3.4 Radial tensile stresses at the AC bottom under FWD versus the HMA moduli for various AC thicknesses

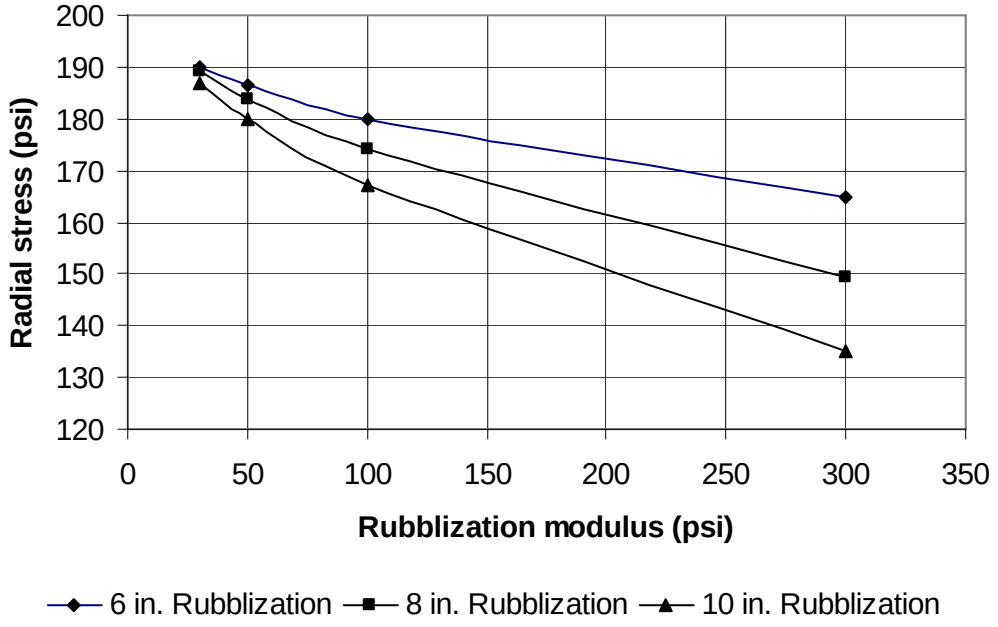


Figure 3.5 Radial tensile stresses at the AC bottom under FWD versus the rubblization moduli for various AC thicknesses

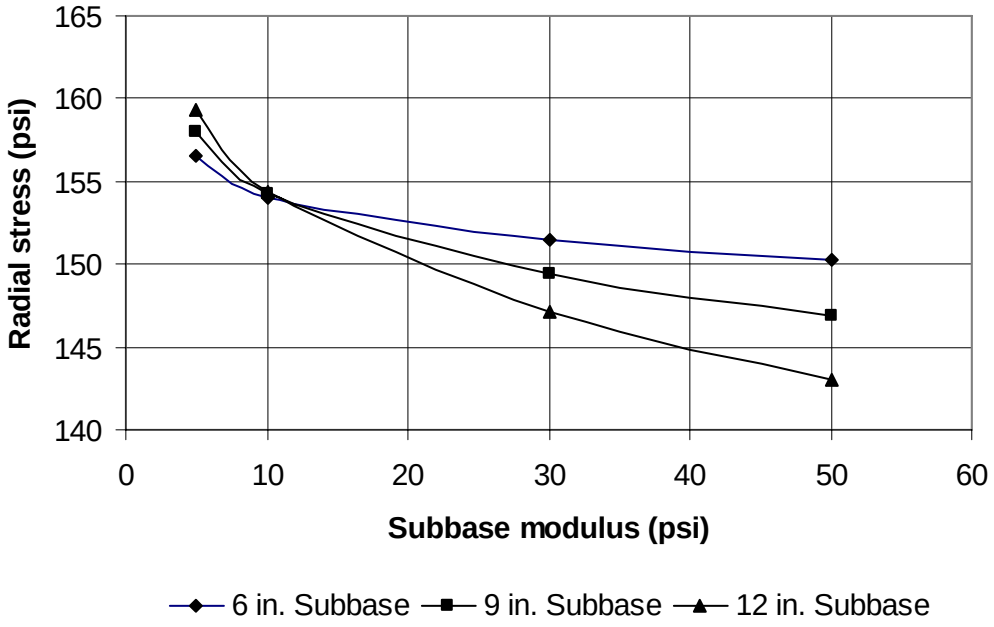


Figure 3.6 Radial tensile stresses at the AC bottom under FWD versus the subbase moduli for various AC thicknesses

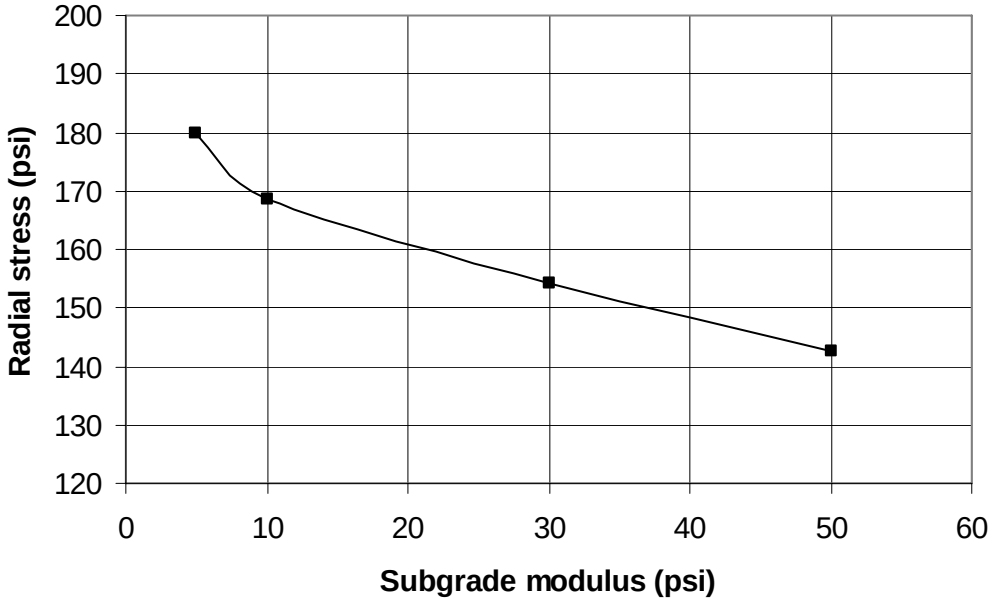


Figure 3.7 Radial tensile stresses at the AC bottom under FWD versus the subgrade moduli

3.4 In-situ FWD Deflection and Backcalculation

During the course of this study, Non-Destructive Tests (NDT) were conducted on the pavement surface using the FWD testing equipment. The FWD testing is commonly used for pavement evaluation and rehabilitation purposes. This testing applies an impact load to the pavement surface and measures the induced surface deflections. Based on the formed deflection basin due to loading, the applied load, layer thicknesses, and layer moduli can be calculated using a FWD back-calculation technique. A typical example of the back-calculation technique was a computer program, the ELMOD 5.0 software developed by Dynatest. The program simulates and iterates the deflection data to achieve a “convergence” solution between the measured deflection basin and the calculated deflection basin to estimate the layer moduli.

The configuration and spacing of the 9 deflection sensors of the FWD are shown in Table 3.3 and Figure 3.8. At the rubblized pavement project, a test section was selected and the section was divided into one test site separated by 100-meters of pavement.

Table 3.3 FWD sensors arrangement for underseal

Sensor number	#1	#2	#3	#4	#5	#6	#7	#8	#9
Distance from plate (inches)	0	-12	8	12	18	24	36	48	60

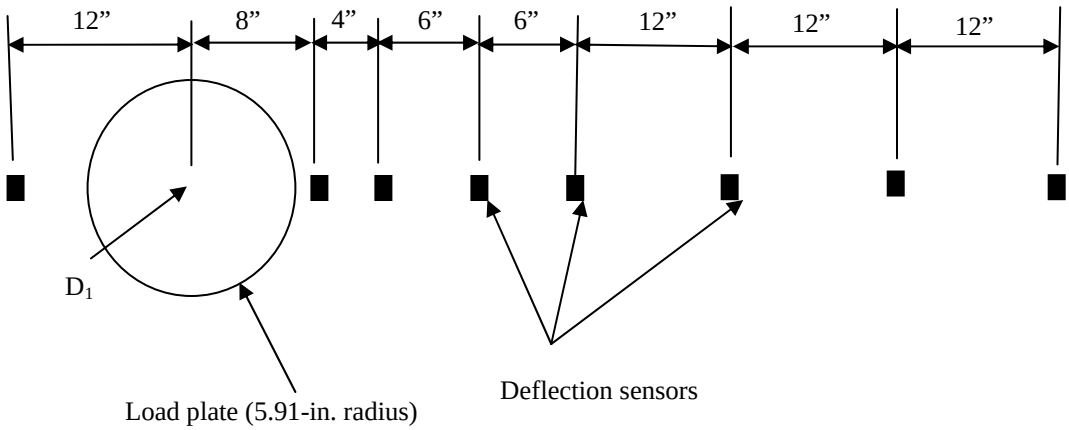


Figure 3.8 FWD sensor layout

3.5 Elastic Layer Simulation and Layer Coefficients

A mechanistic approach considers the fundamental characteristics of pavement materials and characterizes the response of pavement to a traffic load. With the advent of computers, a number of programs were developed to calculate stress, strain and deflection of layered elastic systems subjected to traffic loads. CHEVRONX is one of the most commonly used programs to treat a flexible pavement as a layer elastic system subjected to a circular load. The program is then used to determine deflections of each layer under FWD loading.

Layer coefficient is a function of material thickness, underlying material, and stress. Appendix L of the 1993 AASHTO Guide provided procedures for determining the in-place SN of a pavement structure using FWD deflection data. Neverthless, Sebally (1992) pointed out that the use of FWD to directly evaluate the in-situ properties of the rubblized PCC pavement encountered serious limitations due to the practical difficulties with the placement of the geophones close to cracks. Simulation to determine the layer

coefficients is needed to obtain the deflections on the surface of PCC rubblization. A computer program was therefore developed by the author to simulate the layer coefficients for different layers and overall pavement SN from the back-calculated moduli. Figure 3.9 provides a graphical illustration of the simulation procedures. The detailed steps are summarized below:

1. Determine the structural number for the total layer (HMA, rubblized PCC, and subbase). The relationship between effective modulus (Ep_1) and effective structural number (SN_{eff}) is as follows (1993 AASHTO):

$$SN_{eff} = 0.0045D\sqrt[3]{E_p} \tag{3.1}$$

   where, SN_{eff} = effective structural number; D = total thickness of all pavement layers above the subgrade (inches); and E_p = effective modulus of pavement layers above the subgrade (psi).

2. Calculate the deflection on the rubblized PCC surface using CHEVRON in conjunction with the back-calculated moduli;
3. Determine Ep_2 for the rubblized PCC and subbase, and the structure number of the base course (SN_2) using Eq. 3.1;
4. Determine the layer coefficient (a_1) for HMA by dividing the difference between SN_1 from SN_2 by the thickness of HMA as follows:

$$a_1 = \frac{SN_1 - SN_2}{D_{HMA}} \tag{3.2}$$

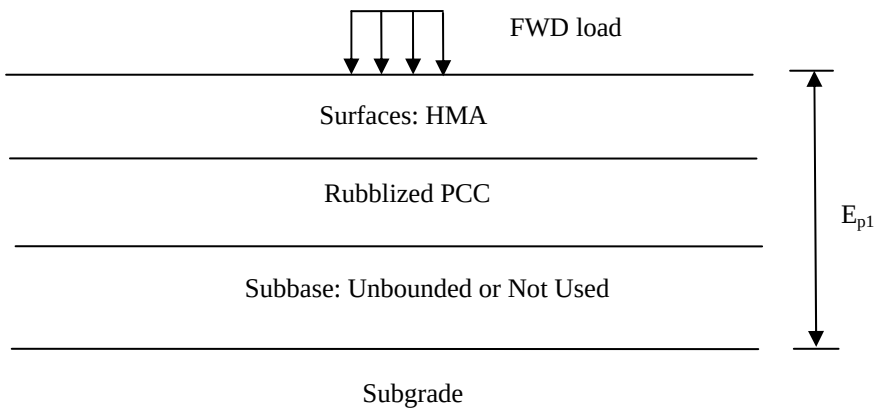
5. Compute the subbase surface deflections using CHEVRONX program if any;
6. Determine Ep_3 for the subbase;

7. Determine the layer coefficients (a_2) for the base course by dividing the difference between SN_2 and SN_3 by the thickness of base:

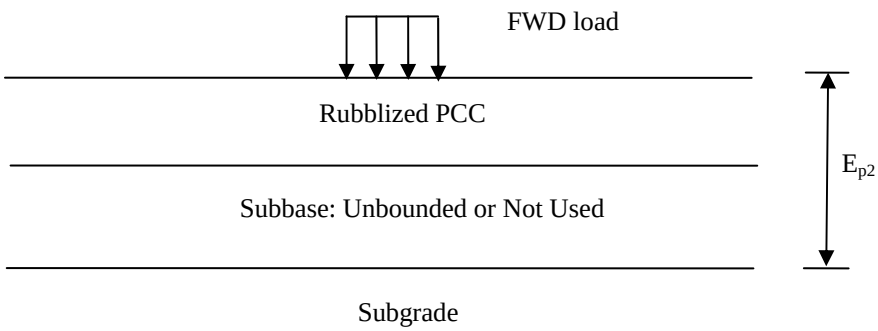
$$a_2 = \frac{SN_2 - SN_3}{D_{rubblized\ \ PCC}} \tag{3.3}$$

8. The layer coefficients (a_3) for the subbase is obtained by dividing SN_3 by the thickness of subbase:

$$a_3 = \frac{SN_3}{D_{subbase}} \tag{3.4}$$



(a)



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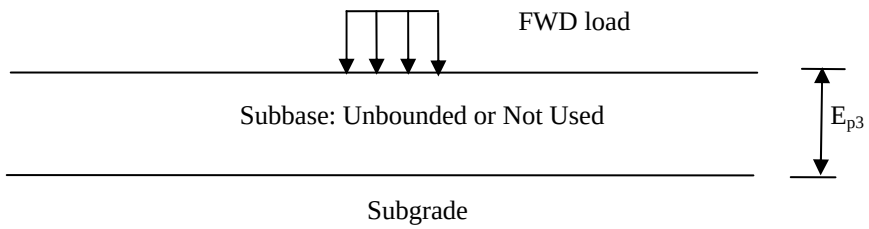

(c)

Figure 3.9 Elastic layer simulation of HMA overlay rubblized pavement

Chapter 4 PROGRAM STRUCTURE AND FEATURES

4.1 Introduction

A computer program was developed by the author to simulate the layer coefficients for different layers and overall pavement SN from the back-calculated moduli. These source files were written using the Microsoft FORTRAN complier version 4.0. The program can read the outputs of the backcalculation program, ELMOD5.0. In this chapter, the general structure and the features of the program are described.

4.2 Data Input

Deflection data, backcalculated modulus and temperature can be read and processed by the program automatically. Pavement parameters including number of layers, Poisson's ratios, the thicknesses of the layers, and the number of sampled points, must be entered using the keyboard. Modulus values were backcalculated employing the ELMOD 5.0 program with three layer idealization. Appendix A provides the input data file.

4.3 Program Structure

The main flow chart of the program is presented in Figure 4.1 and Figure 4.2. The program first reads the inventory data (layer thicknesses, assumed material properties, sensor configuration, etc.) and the load, temperature, and deflections. Once the input data are entered, the program allows the user to select the pavement structure information: i.e., Poisson's ratio, layer numbers. During this process, the center deflection is first corrected to the 68° F using the 1993 AASHTO Design Guide. The program calculates M_R of

subgrade, SN_{eff} for the three-layer pavement system. The program then calculates the surface deflections at each layer including HMA, rubblization, subbase, respectively. Furthermore, each layer's SN and layer coefficients is obtained.

4.4 Output

The program will provide the following information:

1. The effective structure number for each layer (SN_{eff1} and SN_{eff2});
2. The field pavement layer coefficient (a_1,a_2, and a_3);
3. Field subgrade resilient modulus value (M_R).

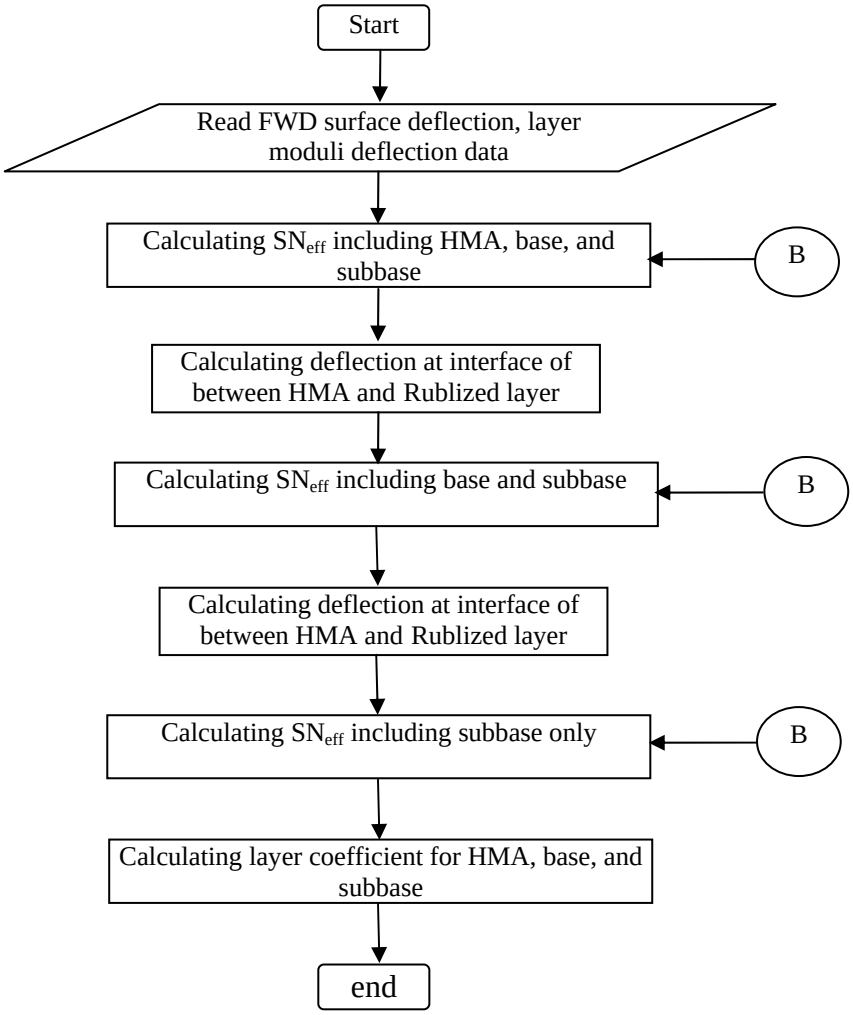

Figure 4.1 Procedure to determine layer coefficients, SN, and M_R

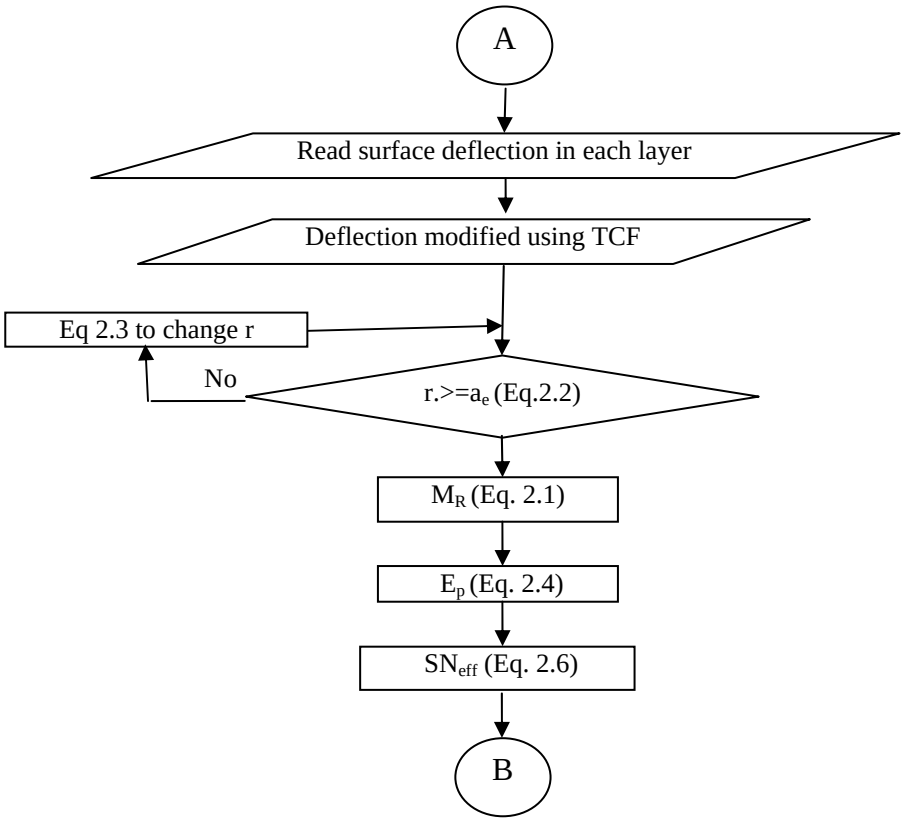

Figure 4.2 1993 AASHTO procedure to determine SN_{eff}

Chapter 5 MECHANISTIC EVALUATION

5.1 Background

Pavement engineers continuously look for ways to evaluate pavement service life and performance. Historically, pavement evaluation procedures were empirical. During the last decade, however, traditional pavement evaluation procedures have been changed to incorporate elastic and/or viscoelastic theories as well as experiences and various empirical tests. The mechanistic evaluation allows engineers to examine the stresses, strains, and deflections in the pavement structure. The empirical evaluation, on the other hand, tries to establish a relationship between these mechanistic responses and the performance of the pavement structure.

For the past ten years, INDOT has been utilizing the 1993 AASHTO Guide for Design of Pavement Structures to design its new or rehabilitated pavement structures. As INDOT moves to implement the proposed Mechanistic-Empirical Pavement Design Guide (MEPDG), characterizing existing pavement conditions is necessary to ensure optimum designs.

This study evaluated the in-situ layer conditions, the in-situ structural number, and the in-situ subgrade resilient modulus from FWD deflection data to evaluate layer coefficients. This information can be used by pavement designers for overlay thicknesses.

5.2 Data Collection

The selection of pavement test sections was accomplished after consulting the study advisory members from the Indiana Department of Transportation (INDOT). The main criteria used in the pavement section selection are as follows: the pavement sections should be representative of typical INDOT pavement cross-sections, environmental conditions, and new construction. Figure 5.1 shows the pavement test sections on interstate highways I-65, US Highways US-41, US-52, and State Routes SR-9, SR-39 SR-46.

FWD tests were conducted in 2001, 2002, and 2004 respectively, immediately following the completion of pavement construction. Tests were conducted in the driving lanes in both directions at a 100-meter interval. Based on previous INDOT studies and experiences, a minimum of 16 testing locations per mile is adequate to provide statistically sound analysis at project level analysis. Three drop load levels consisting of 9 kip, 11 kip and 13 kip were used in testing protocols.

Figure 5.1 FWD tests sites around Indiana

5.3 Statistical Analysis

Several variables were available for statistical analysis after processing FWD testing data followed by Chapter 4. To obtain the suitable layer coefficients for design, two types of methods were used for overall statistical analysis in this chapter. One is ANOVA testing, which makes a comparison between the mean values and variances in each testing year. The other type of method is based on the Z test, which provides confidence intervals that provide designers with the more reliable design parameters as well as mean and standard deviations.

5.3.1 ANOVA (Analysis of Variance)

The ANOVA process is a method that allows for better understanding of the differences between two population means and the ratio of two population Standard Deviations. Statistical significance of a factor (p-value) indicates if there exists a significant mean difference between two analysis methods. First, before performing F test, researchers must make assumptions, i.e., the data are normally distributed. Second, researchers develop a hypothesis that is based on the two methods being equal, and testers try to test and reject this hypnosis. If the p-value is greater than the critical alpha (i.e. 0.05), for example, the probability of making the least favorable type of error, then the researchers have enough evidence to accept the given hypothesis.

5.3.2 Z test for Confidence Interval

Collected samples are usually limited; therefore, the true value of the sample is difficult to obtain. Researchers usually would not know the true value of mean from sampling; rather, researchers would select a single random sample and construct the associate

certain (i.e. 95%) confidence interval in which the true value of the population parameters can be contained. Therefore, engineers can pick the safer parameter for the design purpose at cost effectiveness with confidence in the statistical procedure. Z-tests allow the endpoints of interval to be computed based on sample information. Usually larger samples generally provide more information about the target population than do smaller samples. Therefore, the more samples that are collected, the greater the confidence is obtained in findings and results. Also it is note that for a given sample size, the width of the confidence interval for a parameter increases as the confidence coefficient increases. On the basis of this theory, the more FWD data that is collected on the road, the more confidence engineers have about the evaluation of pavement structure.

5.4 State Routes and US Roads

Two sites were analyzed in this part as an example. One rubblization project was on the portion of SR-39 in Clinton/Boone county, Crawfordsville district, Indiana. This site is approximately 11.2 km in length. After rubblization pavement was open to traffic, FWD data were collected in July of 2001, August of 2002, and July of 2004, respectively. The other rubblization project was on the portion of US-52 in Benton county, Crawfordsville district, Indiana. Both sections used an asphalt overlay of over approximately 14.4 km section of rubblized PCC pavement. Deflections were obtained in October of 2001, November of 2002, and August of 2004, respectively.

5.4.1 Comparative Analysis of Layer Coefficient

<u>SR-39 from RP 72 to RP 79</u>

Figure 5.2 illustrates the center deflections at the northbound section after temperature correction, which measured average values of 4.2 mils for 2001, 4.4 mils for 2002, and 3.8 mils for 2004, respectively. The average deflection at 2002 was approximately 95% as large as the value at 2001. Similarly, the average value of deflection at 2004 was approximately 95% as large as the value at 2001. Figure 5.3 and Figure 5.6 depict the HMA layer coefficient at each direction. Figure 5.4 and Figure 5.7 depict the rubblization layer coefficient at both directions. The changes of average center deflection from 2001 to 2004 were ploted in Figure 5.5. Notable are deflections of 2002: the average value decreased by 15% and decreased by 30% at 2004.

At the northbound (NB) section, Table 5.1 shows layer coefficients for HMA with 95% confidence level (CL) was the range from 0.37 to 0.40 in 2001, from 0.33 to 0.38 in 2002, and from 0.38 to 0.41 in 2004. These results show that layer coefficients for rubblization with 95% confidence level (CL) were the range from 0.24 to 0.26 in 2001, from 0.28 to 0.31 in 2002, and from 0.24 to 0.26 in 2004. The average values were almost at the middle point of these ranges. The standard deviations for HMA were 0.09 in 2001, 0.11 in 2002, and 0.06 in 2004. The standard deviations for rubblization were 0.06 in 2001, 0.08 in 2002, and 0.06 in 2004. The P value was 0 for both HMA and rubblized PCC layer, which means the value was significantly different statistically.

At the southbound (SB) section, Table 5.2 shows that layer coefficients for HMA with 95% confidence level (CL) ranged from 0.31 to 0.35 in 2001, from 0.23 to 0.27 in 2002, and from 0.38 to 0.41 in 2004. It shows that layer coefficients for rubblization with 95% confidence level (CL) was the range from 0.25 to 0.27 in 2001; it ranged from 0.30 to 0.37 in 2002; and it ranged from 0.24 to 0.26 in 2004. The standard deviations for HMA

were 0.09 in 2001, 0.10 in 2002, and 0.07 in 2004. The standard deviation for rublization was 0.06 in 2001, 0.09 in 2002, and 0.06 in 2004. The average values were almost at the middle point of these ranges. The P-value is 0.0. These values indicate that the null hypothesis should be rejected, that is, there is a difference between each year.

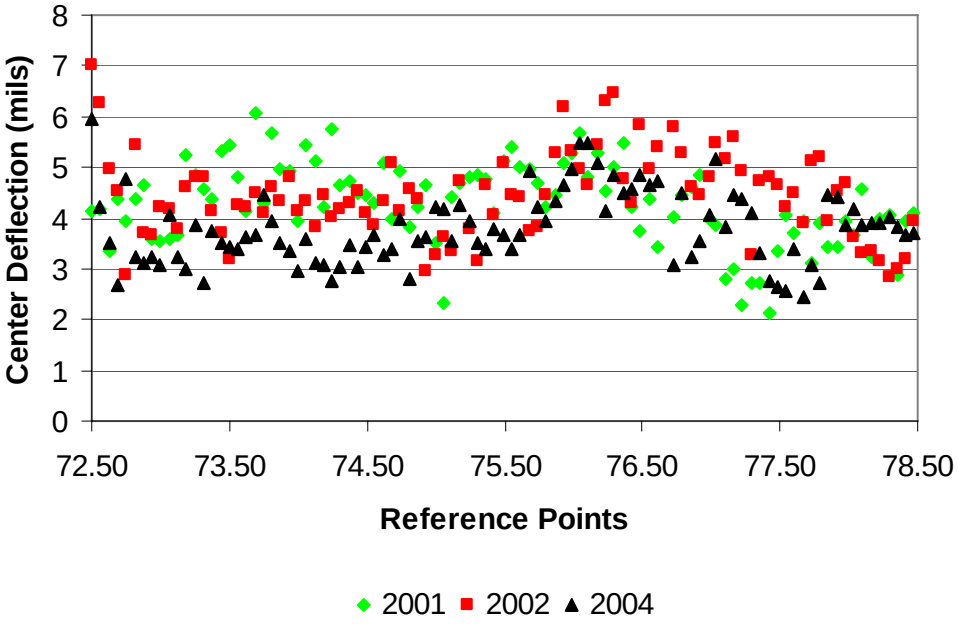

Figure 5.2 Comparison for center deflection at SR-39 NB (after temperature correction)

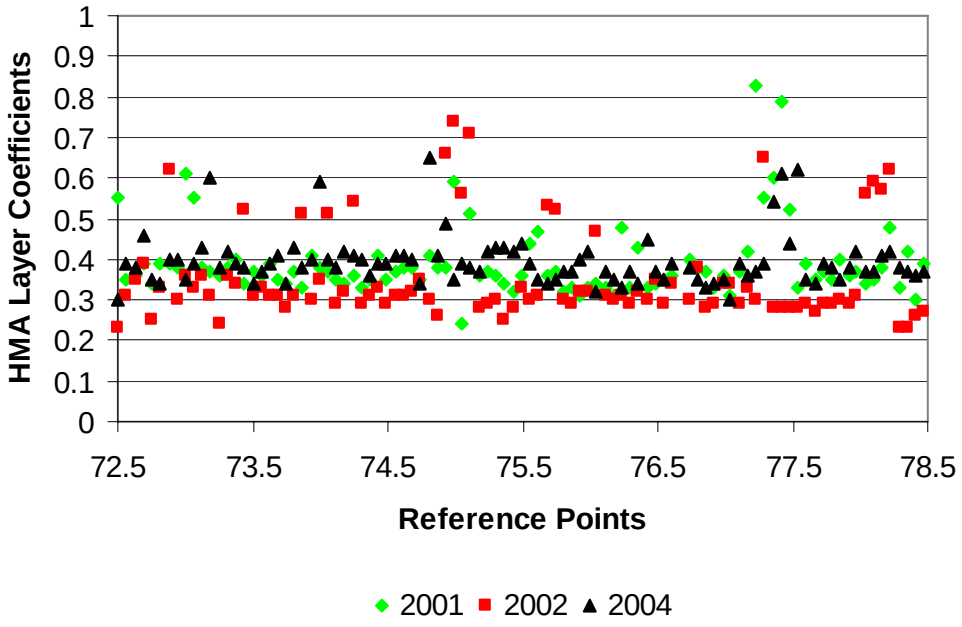

Figure 5.3 Comparison for HMA layer coefficients at SR-39 NB

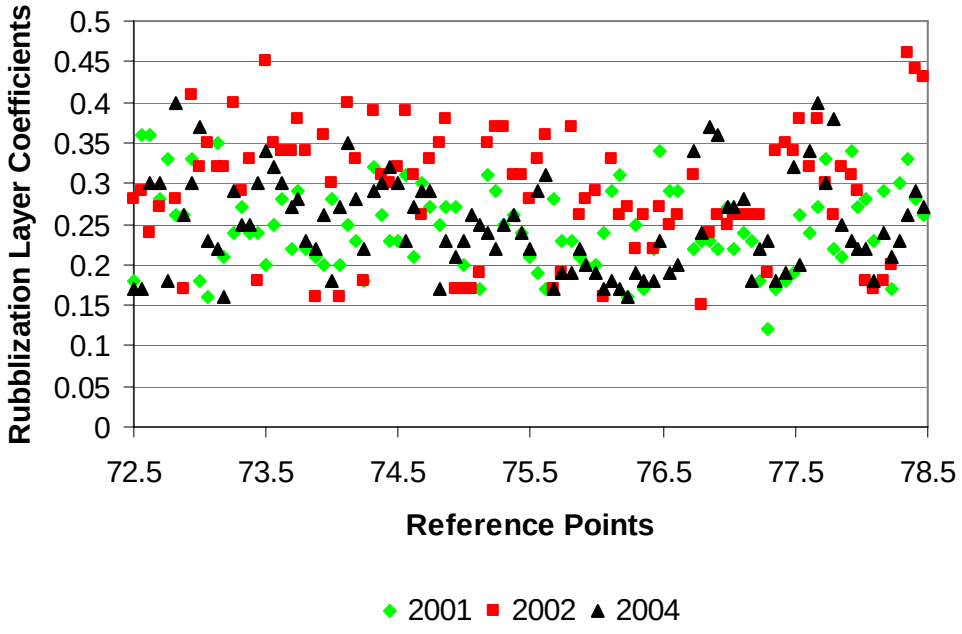

Figure 5.4 Comparison for rubblization layer coefficients at SR-39 NB

Table 5.1 Summary statistic of layer coefficient at SR-39 NB

	HMA 2001	*HMA 2002*	*HMA 2004*	*RB 2001*	*RB 2002*	*RB 2004*
Mean	0.39	0.35	0.40	0.24	0.30	0.25
Standard Deviation	0.09	0.11	0.06	0.06	0.08	0.06
Interval Lower Limit with CI 95%	0.37	0.33	0.38	0.24	0.28	0.24
Interval Upper Limit with CI 95%	0.40	0.38	0.41	0.26	0.31	0.26
P(Z<=z) two-tail	0.00			0.00		

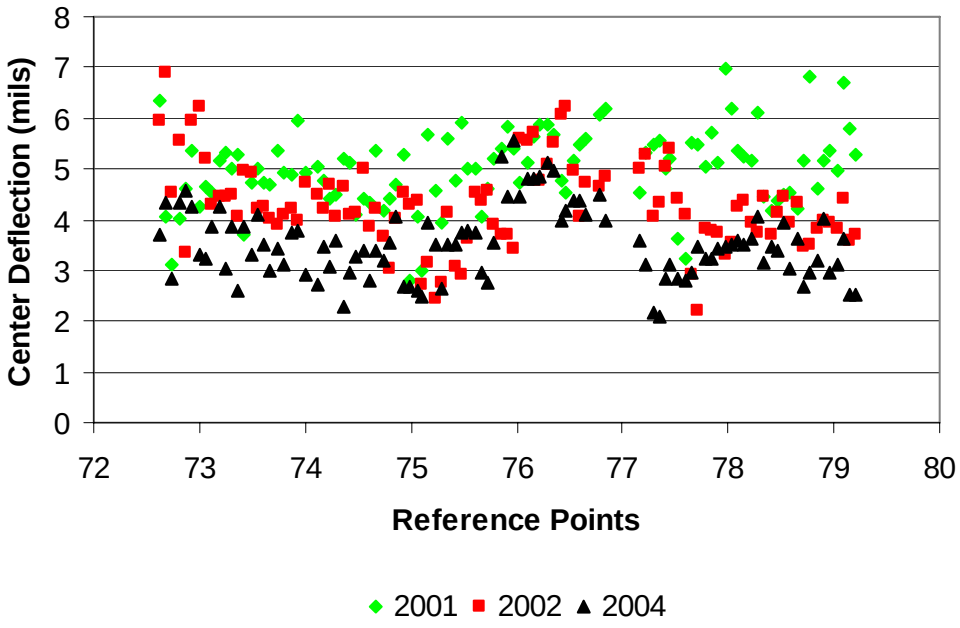

Figure 5.5 Comparison for Center deflection at SR-39 SB

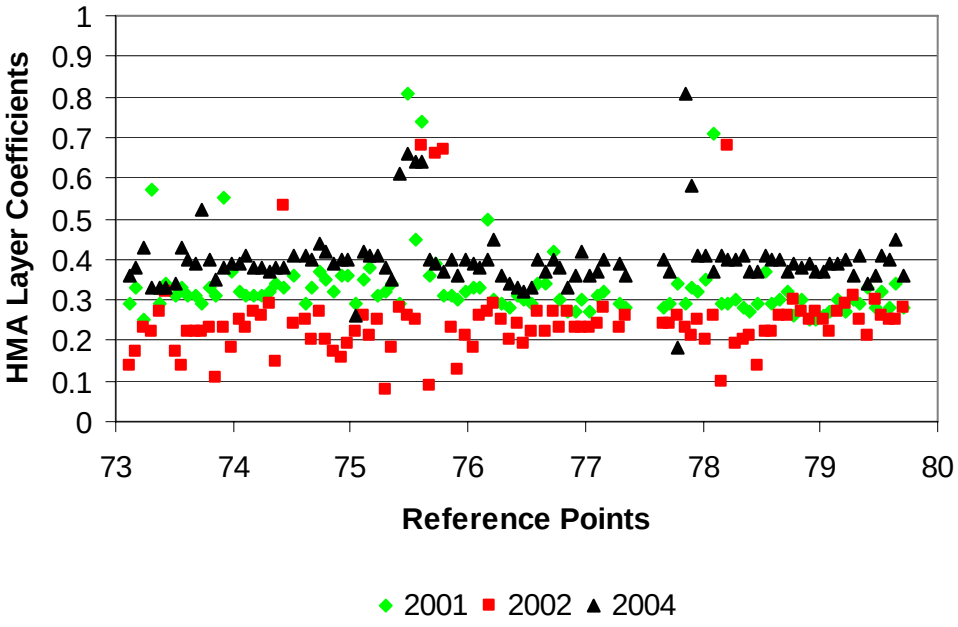

Figure 5.6 Comparison for HMA Layer coefficients at SR-39 SB

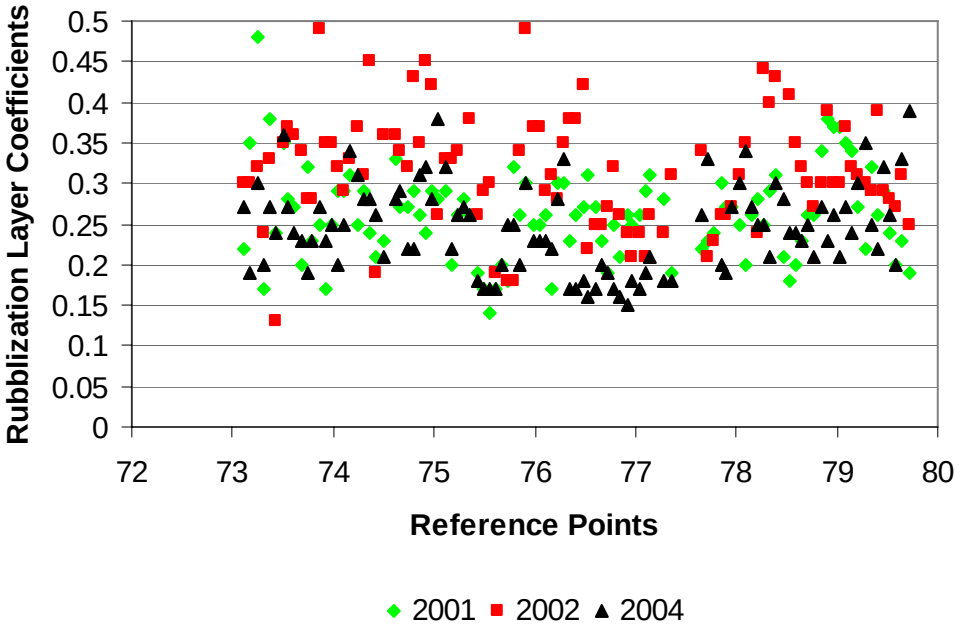

Figure 5.7 Comparison for rubblized layer coefficients at SR-39 SB

Table 5.2 Summary statistic of layer coefficient at SR-39 SB

	HMA 2001	*HMA 2002*	*HMA 2004*	*RB 2001*	*RB 2002*	*RB 2004*
Mean	0.33	0.24	0.40	0.26	0.32	0.25
Standard Deviation	0.09	0.10	0.07	0.06	0.09	0.06
Interval Lower Limit with CI 95%	0.31	0.23	0.38	0.25	0.30	0.24
Interval Upper Limit with CI 95%	0.35	0.27	0.41	0.27	0.37	0.26
P(Z<=z) two-tail	0.00			0.00		

US-52 at from RP 14 to RP 25

The center deflection in Figure 5.8 consists of testing points in the eastbound section with an average of 5.3 mils in 2001, 7.3 mils in 2002, and 7.8 mils in 2004. With traffic loading between the interval of testing, the average deflection increased by 38% in 2002 and 48% in 2004 compared to the deflection in 2001. Figure 5.11 shows the center deflections measured in 2001, 2002, and 2004 in the westbound lane. The average deflections were 7.8 mils, 5.1 mils, and 7.8 mils, respectively. The average deflection in 2002 was 35% lower than that of in 2001, and the average deflection in 2004 was almost the same as that of in 2001.

Figure 5.9 and Figure 5.12 show HMA layer coefficient versus the reference points. Figure 5.10 and Figure 5.13 show the rubblization layer coefficients versus the reference points. Table 5.3 and Table 5.4 provide summarized statistical results. With a 95% Confidence Interval (CI) level, it was found in the eastbound section that the HMA Layer Coefficient (LC) values were from 0.42 to 0.45, with an average value of 0.43 in 2001. In 2002, the LC values were from 0.39 to 0.41 with an average value of 0.40. The LC values were from 0.27 to 0.29 with an average value of 0.40 in 2004. The rubblized layers remained the same value of 0.15 in 2001. On the other hand, the rubblized layers ranged

from 0.16 to 0.18 with an average value of 0.17 in 2002, and from 0.24 to 0.26 with an average value of 0.25 in 2004. Based on the P-values, the HMA LC values were all significantly different by years.

In the westbound lane, with a 95% Confidence Interval (CI) level, the HMA LC values were from 0.40 to 0.44 with an average value of 0.42 in 2001, from 0.46 to 0.48 with an average value of 0.47 in 2002, and from 0.39 to 0.44 with an average value of 0.42 in 2004. The LC values for the rubblization layer were from 0.16 to 0.18 with an average value of 0.17 in 2001. However it remained the same value of 0.15 in 2002. On the other hand, this value ranged from 0.17 to 0.18 with an average value of 0.17 in 2004. The P-values show that the rubblization layer was significantly different by years.

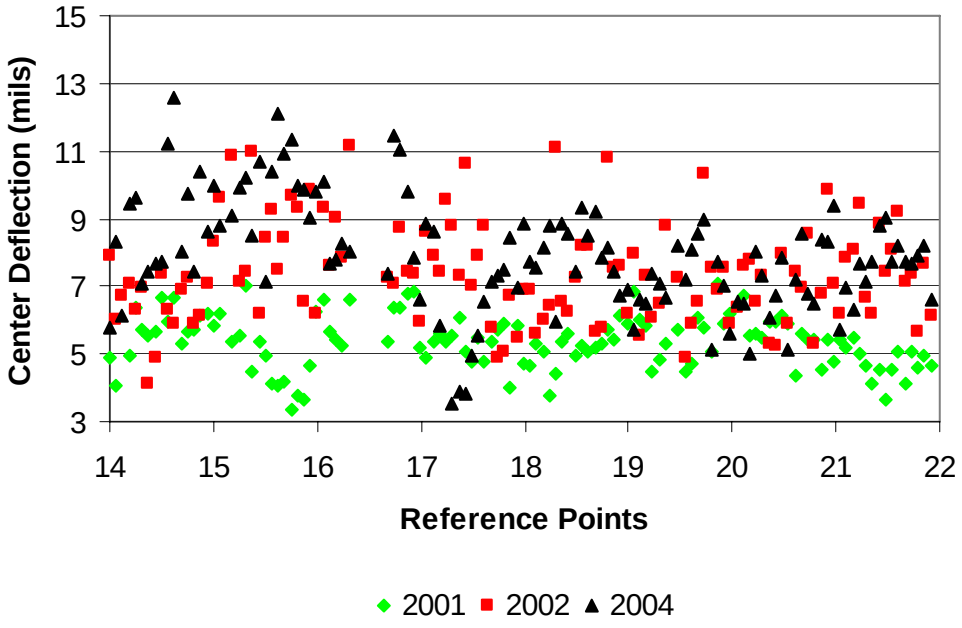

Figure 5.8 Comparison for Center deflection at US-52 EB

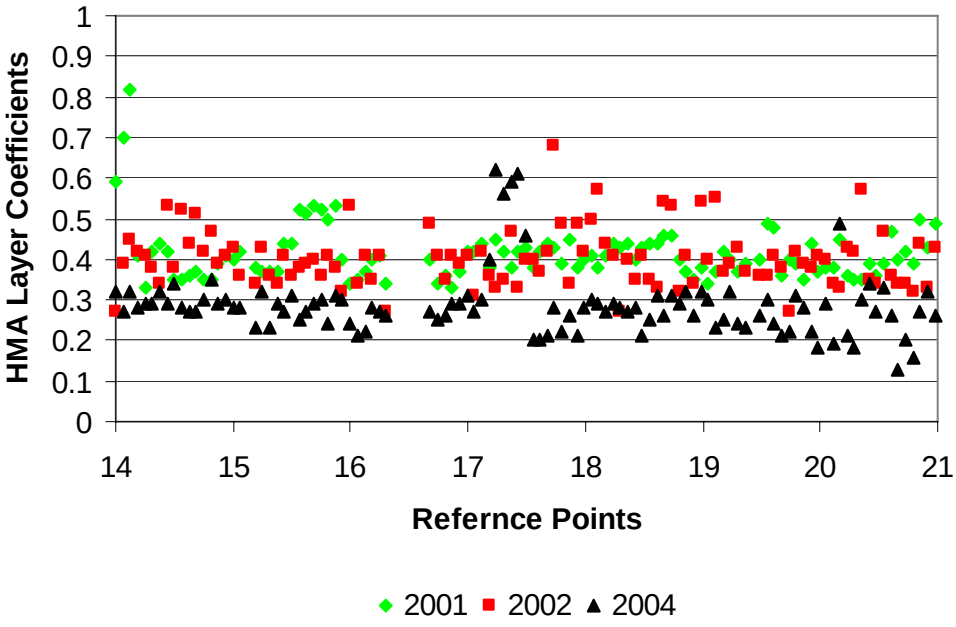

Figure 5.9 Comparison for HMA layer coefficients at US-52 EB

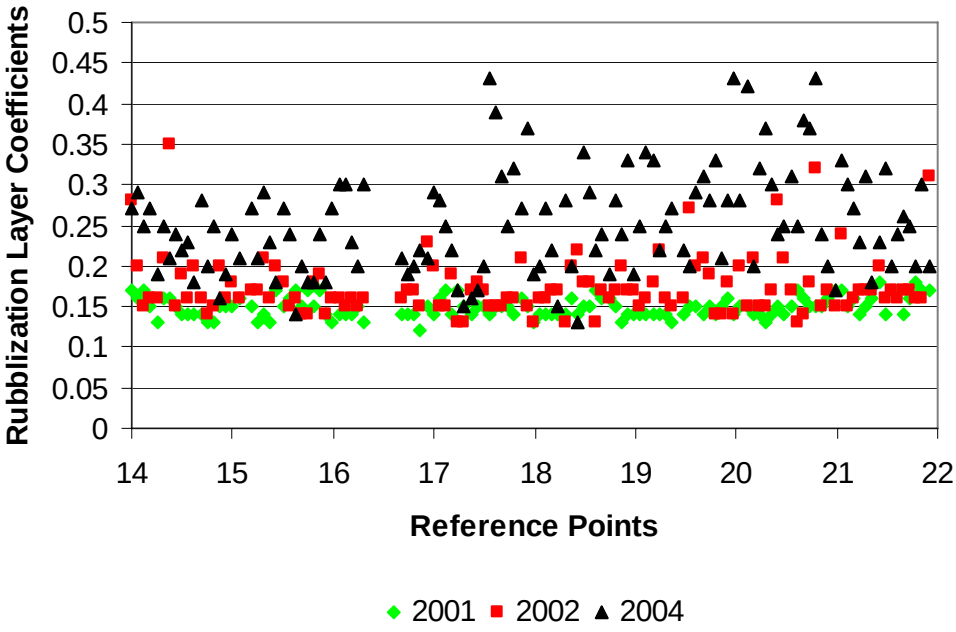

Figure 5.10 Comparison for rubblization layer coefficients at US-52 EB

Table 5.3 Summary statistic of layer coefficient at US-52 EB

	HMA 2001	*HMA 2002*	*HMA 2004*	*RB 2001*	*RB 2002*	*RB 2004*
Mean	0.43	0.40	0.28	0.15	0.17	0.25
Standard Deviation	0.09	0.07	0.08	0.01	0.04	0.06
Interval Lower Limit with CI 95%	0.42	0.39	0.27	0.15	0.16	0.24
Interval Upper Limit with CI 95%	0.45	0.41	0.29	0.15	0.18	0.26
P(Z<=z) two-tail	0.00			0.00		

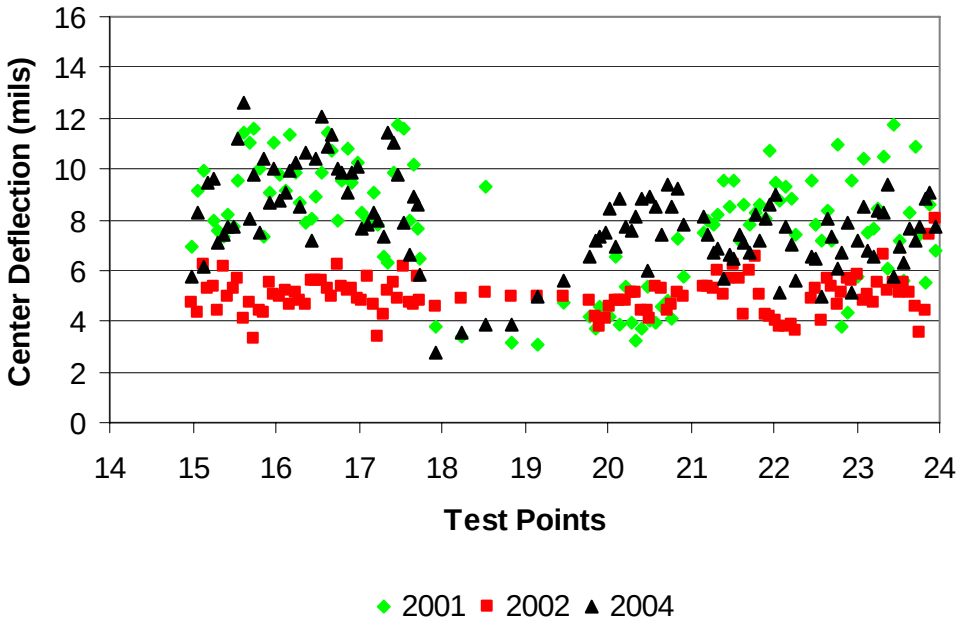

Figure 5.11 Comparison for center deflection at US-52 WB

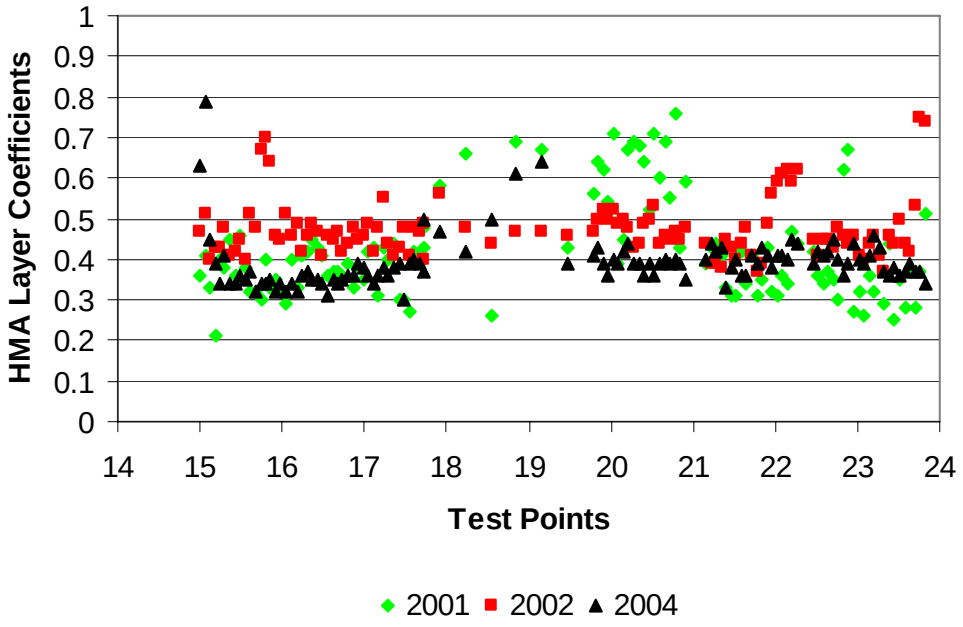

Figure 5.12 Comparison for HMA layer coefficients at US-52 WB

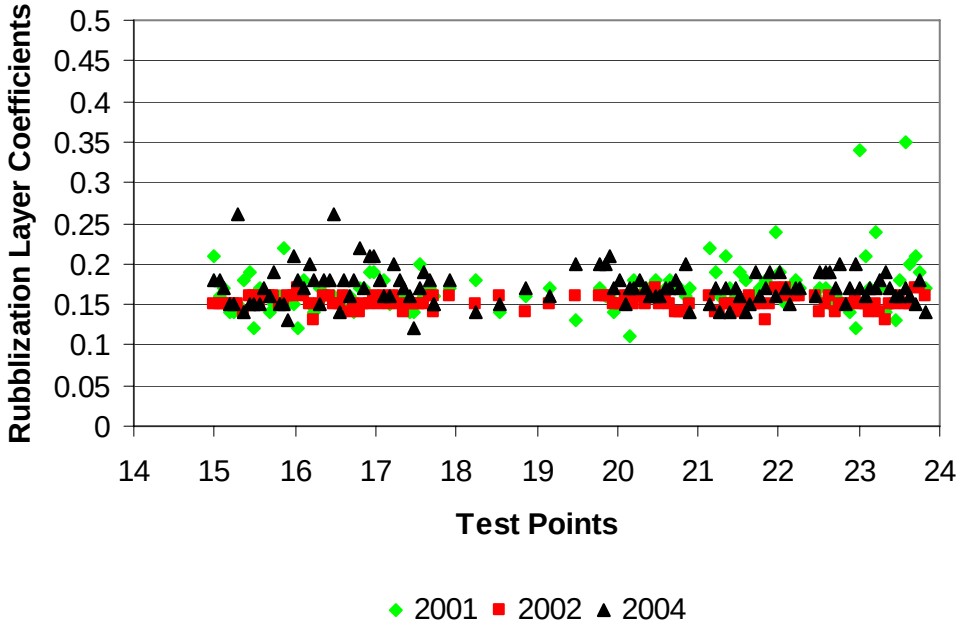

Figure 5.13 Comparison for rubblization layer coefficients at US-52 WB

Table 5.4 Summary statistic of layer coefficient at US-52 WB

	HMA 2001	*HMA 2002*	*HMA 2004*	*RB 2001*	*RB 2002*	*RB 2004*
Mean	0.42	0.47	0.42	0.17	0.15	0.17
Standard Deviation	0.12	0.07	0.12	0.03	0.01	0.02
Interval Lower Limit with CI 95%	0.40	0.46	0.39	0.16	0.15	0.17
Interval Upper Limit with CI 95%	0.44	0.48	0.44	0.18	0.15	0.18
P(Z<=z) two-tail	0.00			0.00		

5.4.2 Comparative Analysis of In-situ M_R

SR-39 from RP 72.51 to RP 79.82

Figure 5.14 illustrates the subgrade moduli varied with years in the northbound (NB) section. As shown in Table 5.5, the statistical summary for M_R was listed to compare the differences. The average value was 8.27 ksi in 2001, 7.97 in 2002, and 11.41 ksi in 2004. As expected, the value changes with the season. The P-value shows that it was less than 0.05; therefore, there was statistical difference during three years. In addition, the standard deviation was 1.83 in 2001, 1.70 in 2002, and 2.45 in 2004, respectively.

Figure 5.15 illustrates the subgrade moduli varied with years in the southbound (SB) section. Statistic summaries for M_R are shown in Table 5.6. The average value ranged 8.37 ksi and 9.16 ksi with the average of 8.27 ksi in 2001; in 2002 it ranged 7.16 ksi and 7.77 ksi with the average of 7.97 ksi in 2002. And in 2004, it ranged 10.97 ksi and 12.20 ksi with the average of 11.58 ksi in 2004. The relative difference between 2001 and 2002 was 14%, while the relative difference between 2001 and 2004 was 32%. As for the standard deviation, it was 2.01 in 2001, 1.55 in 2002, and 3.18 in 2004. The P-value was less than 0.05; therefore, there was statistical difference during three years.

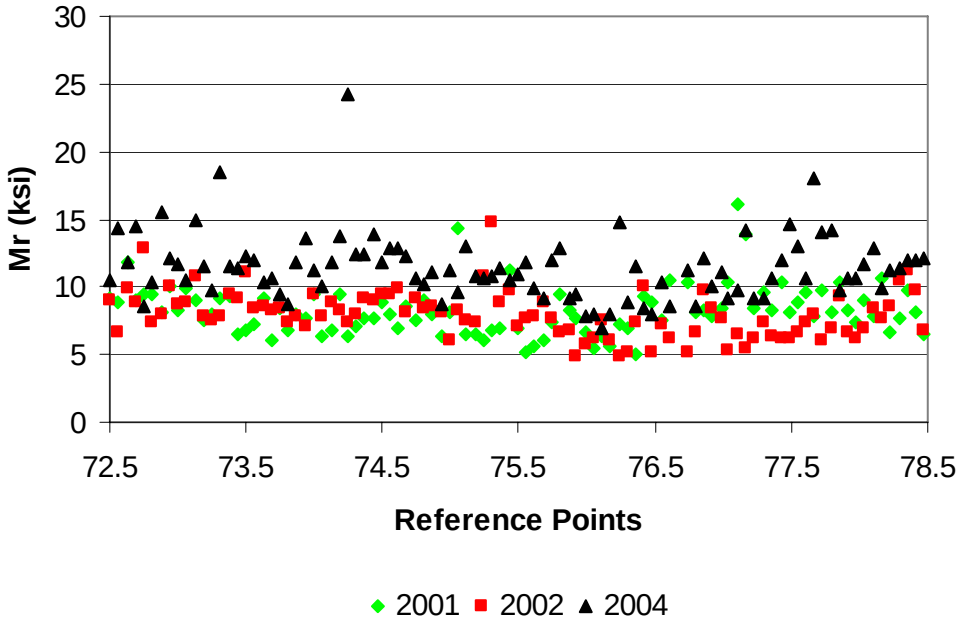

Figure 5.14 Comparison for M_R at SR-39 NB

Table 5.5 Summary statistic of M_R at SR-39 NB

	2001	*2002*	*2004*
Mean (ksi)	8.27	7.97	11.41
Standard Deviation (ksi)	1.83	1.70	2.45
Interval Lower Limit with CI 95% (ksi)	7.90	7.63	10.93
Interval Upper Limit with CI 95% (ksi)	8.63	8.30	11.90
P(Z<=z) two-tail	0.00		

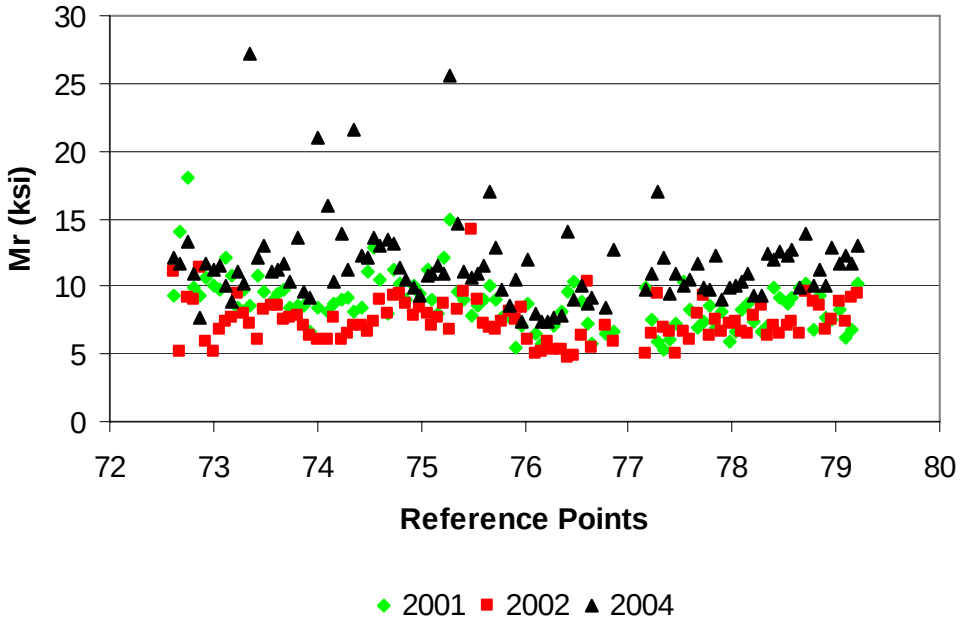

Figure 5.15 Comparison for M_R at SR-39 SB

Table 5.6 Statistic summaries of M_R at SR-39 SB

	2001	*2002*	*2004*
Mean (ksi)	8.76	7.47	11.58
Standard Deviation (ksi)	2.01	1.55	3.18
Interval Lower Limit with CI 95% (ksi)	8.37	7.16	10.97
Interval Upper Limit with CI 95% (ksi)	9.16	7.77	12.20
P(Z<=z) two-tail	0.00		

US-52 at from RP 14 to 25

Figure 5.16 and Figure 5.17 illustrate the range for M_R varied with years at both sections. Table 5.7 and Table 5.8 summarize the variation of M_R within the three years. In the eastbound section, the relative absolute difference between 2001 and 2002 was 13%, while the difference between 2001 and 2004 was 34%. In the westbound section, the relative absolute difference between 2001 and 2002 was 78%, while the difference

between 2001 and 2004 was 60%. The P-value shows that there were significant differences during test seasons in the statistical analysis.

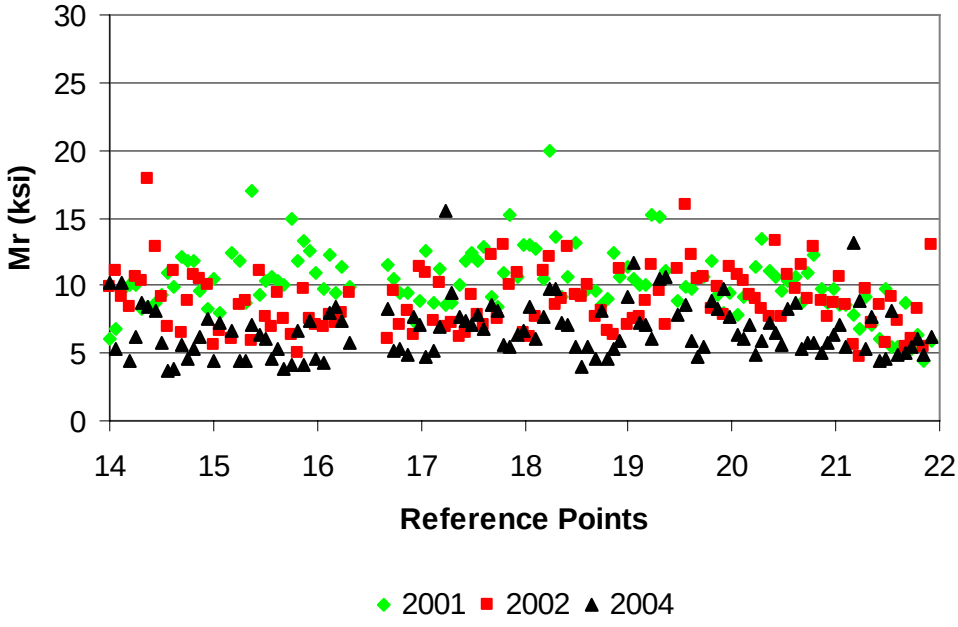

Figure 5.16 Comparison for M_R at US-52 EB

Table 5.7 Summary statistic of M_R at US-52 EB

	2001	*2002*	*2004*
Mean (ksi)	10.20	8.90	6.70
Standard Deviation (ksi)	2.35	2.27	1.99
Interval Lower Limit with CI 95% (ksi)	9.78	8.49	6.34
Interval Upper Limit with CI 95% (ksi)	10.62	9.31	7.05
P(Z<=z) two-tail	0.00		

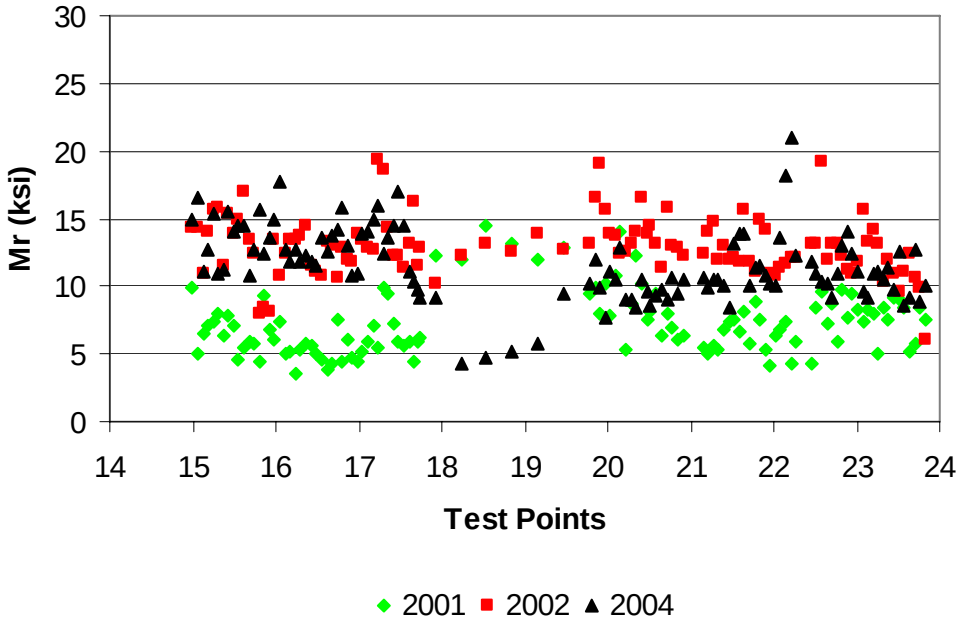

Figure 5.17 Comparison for M_R at US-52 WB

Table 5.8 Summary statistic of M_R at US-52 WB

	2001	*2002*	*2004*
Mean	7.26	12.95	11.63
Standard Deviation	2.29	2.17	2.68
Interval Lower Limit with CI 95%	6.84	12.55	11.13
Interval Upper Limit with CI 95%	7.68	13.34	12.12
P(Z<=z) two-tail	0.00		

5.5 Interstate Highway

One site was analyzed in this part as an example. This rubblization project was on the portion of I-65 in Tippecanoe/White county, Crawfordsville district, Indiana. This site is approximately 25.6 km in length. Both northbound (NB) and southbound (SB) sections

were rubblized. FWD data were obtained in November of 2001, October of 2002, and June of 2004, respectively.

5.5.1 Comparative Analysis of Layer Coefficient

I-65 at from RP 181 to RP 197

The ceneter deflection values in the northbound and southbound are shown graphically in Figure 5.18 and Figure 5.21, respectively. In the northbound section, the average value in 2002 was approximately 18% lower than those measured in 2001, and the average deflctions in 2004 was approximately 26% higher than those measured in 2001. In the southbound lane, deflections have more than approximately 19% of the decrease in 2002 testing, and deflections increase by approximately 29% in 2004 testing. Figure 5.19 and Figure 5.22 show that HMA layer coefficients fluctuate with the test sites during three years. Figure 5.20 and Figure 5.23 show that rubblization layer coefficients varied with the test sites for the four-year testing period. A comparision of the statistical analysis for layer coefficients in the northbound and southbound lanes is given in Table 5.9 and Table 5.10, respectively.

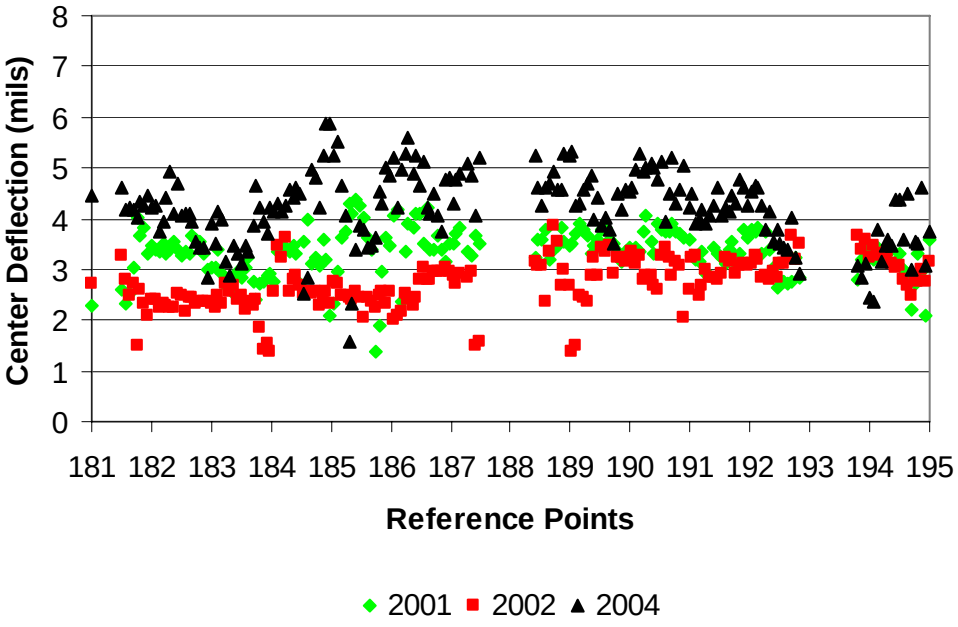

Figure 5.18 Comparison for center deflection at I-65 NB

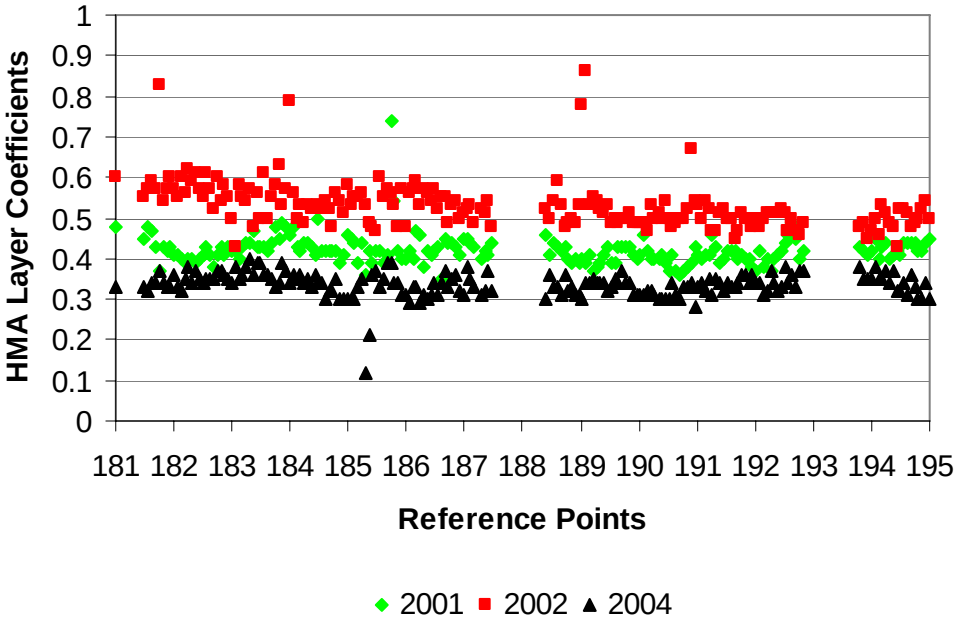

Figure 5.19 Comparison for HMA layer coefficients at I-65 NB

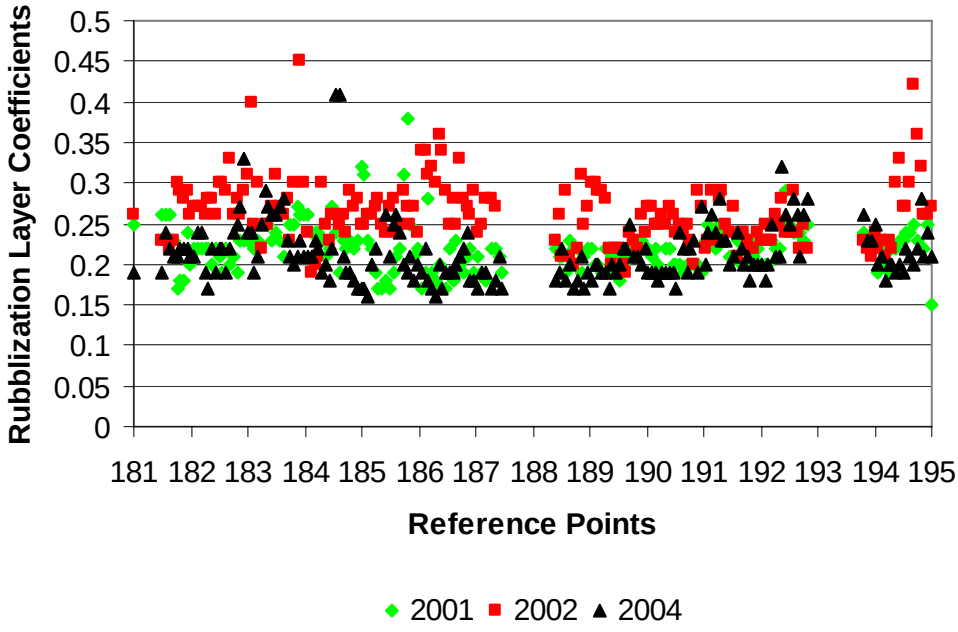

Figure 5.20 Comparison for Rubblization layer coefficients at I-65 NB

Table 5.9 Summary statistic of layer coefficient at I-65 NB

	HMA 2001	*HMA 2002*	*HMA 2004*	*SB 2001*	*SB 2002*	*SB 2004*
Mean	0.42	0.53	0.34	0.22	0.27	0.22
Standard Deviation	0.04	0.06	0.03	0.03	0.05	0.06
Interval Lower Limit with CI 95%	0.42	0.52	0.33	0.22	0.26	0.21
Interval Upper Limit with CI 95%	0.43	0.53	0.35	0.22	0.27	0.23
P(Z<=z) two-tail	0.00			0.00		

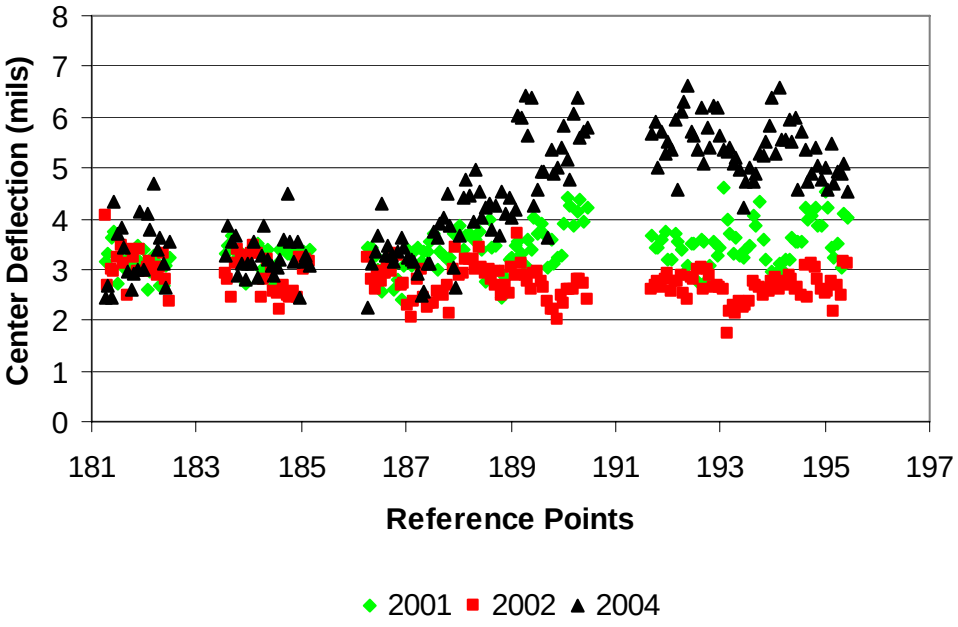

Figure 5.21 Comparison for center deflection at I-65 SB

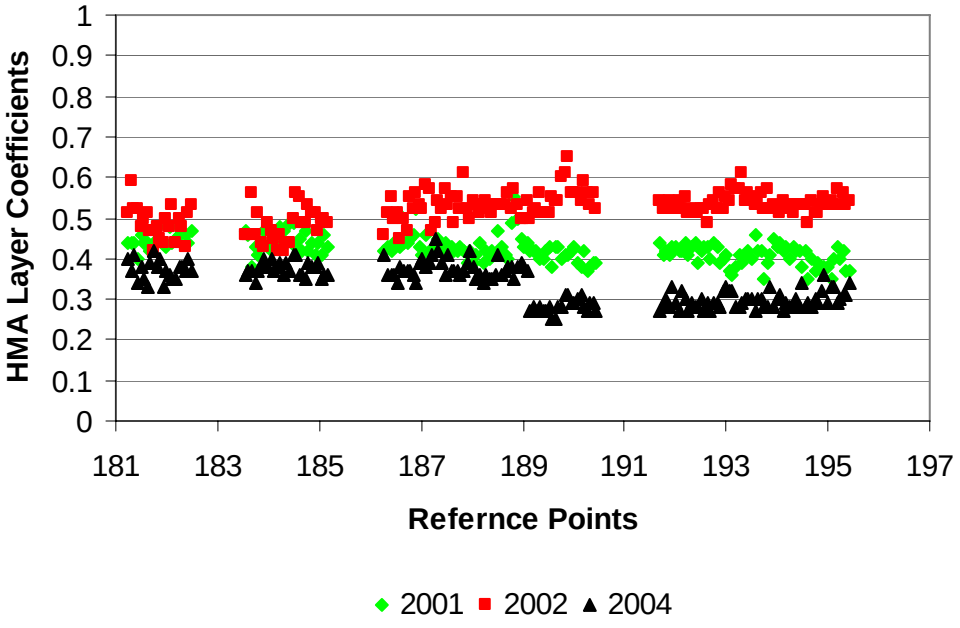

Figure 5.22 Comparison for HMA layer coefficients at I-65 SB

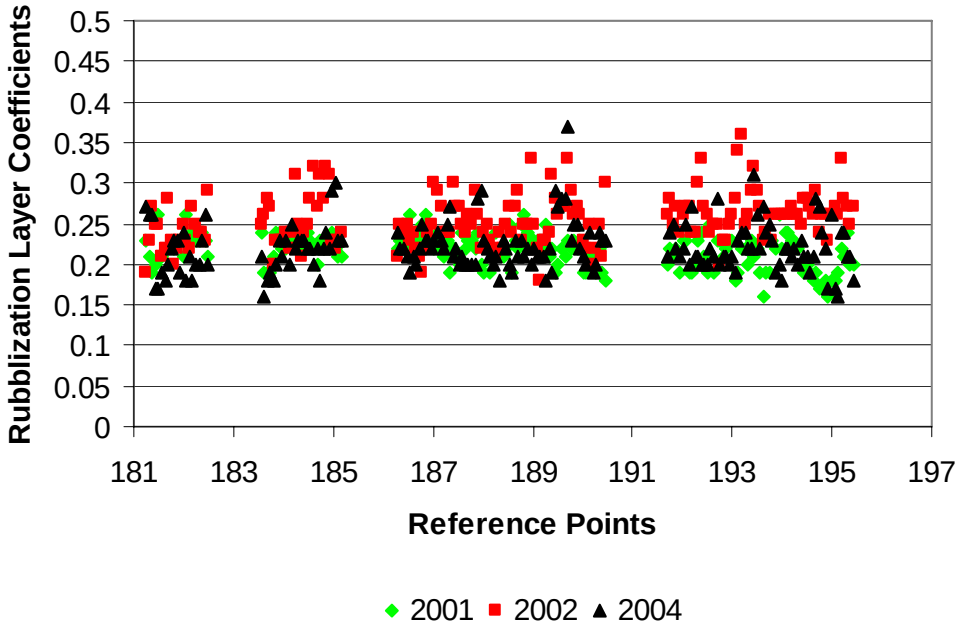

Figure 5.23 Comparison for Rubblization layer coefficients at I-65 SB

Table 5.10 Summary statistic of layer coefficient at I-65 SB

	HMA 2001	*HMA 2002*	*HMA 2004*	*SB 2001*	*SB 2002*	*SB 2004*
Mean	0.41	0.53	0.30	0.22	0.25	0.22
Standard Deviation	0.03	0.04	0.05	0.02	0.03	0.03
Interval Lower Limit with CI 95%	0.40	0.53	0.29	0.21	0.25	0.22
Interval Upper Limit with CI 95%	0.41	0.54	0.30	0.22	0.26	0.23
P(Z<=z) two-tail	0.00			0.00		

5.5.2 Comparative Analysis of In-situ M_R

<u>I-65 from at RP 181 to RP 197</u>

Figure 5.24 and Figure 5.25 illustrate the M_R varied with years at the northbound and southbound sections. Table 5.11 and Table 5.12 summarize for the statistical variation of M_R with years. The P-value is 0.00. This value indicates that the null hypothesis should be rejected. There were significant statistically different results found during test seasons in both northbound and southbound lanes. Results also show that the mean 2002 modulus is 87% as large as the mean 2001 value and the mean 2004 modulus is 95% as large as value in 2001.

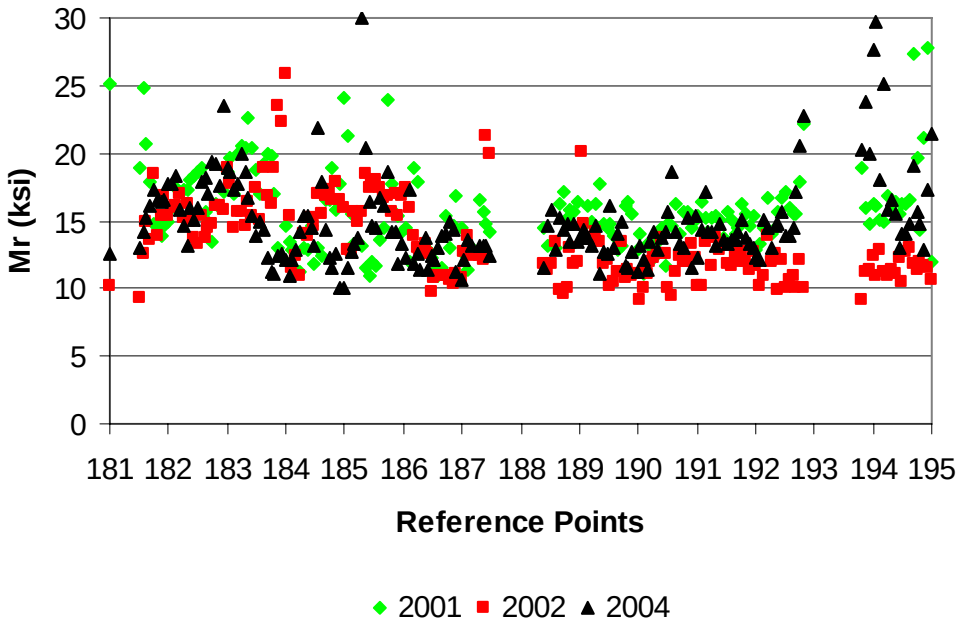

Figure 5.24 Comparison for M_R at I-65 NB

Table 5.11 Summary statistic of M_R at I-65 NB

	2001	*2002*	*2004*
Mean (ksi)	15.65	13.56	14.94
Standard Deviation (ksi)	2.95	2.89	3.24
Interval Lower Limit with CI 95% (ksi)	15.25	13.15	14.47
Interval Upper Limit with CI 95% (ksi)	16.10	13.97	15.40
P(Z<=z) two-tail	0.00		

In the northbound lane, relative difference between 2001 and 2002 was -13%, while difference between 2001 and 2004 was -5%. Standard deviation was 2.95 in 2001, 2.89 in 2002, and 3.24 in 2004. The P-value shows that there were significant differences during test seasons. In the southbound lane, the relative difference between 2001 and 2002 was -28%, while the difference between 2001 and 2004 was -22%.

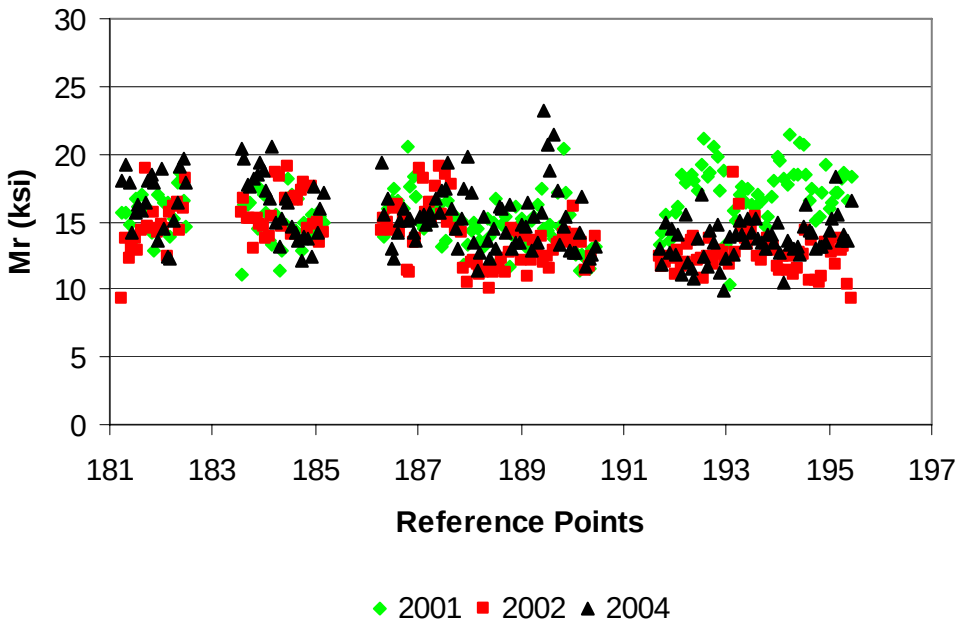

Figure 5.25 Comparison for M_R at I-65 SB

Table 5.12 Summary statistic of M_R at I-65 SB

	2001	*2002*	*2004*
Mean (ksi)	17.56	12.56	13.66
Standard Deviation (ksi)	2.16	2.11	2.37
Interval Lower Limit with CI 95% (ksi)	17.24	12.37	13.31
Interval Upper Limit with CI 95% (ksi)	17.88	12.99	14.00
P(Z<=z) two-tail	0.00		

5.6 Comparison of Empirical and Mechanistic procedure for Layer Coefficients

Two different techniques were used to evaluate the structural layer coefficients. Data were collected from the twelve pavement sections for a period of four years. The average value was used to present the typical value for each section. The first technique is the suggested empirical equations for AASHTO (Ullidtz, 1993), i.e., Equations 2.9 and 2.12. The moduli in these equations were determined from the back-calculation from ELMOD 5.0. The second technique was based on the concept of structural number. It first evaluates the required structural numbers over the rubblization layer and over the subgrade using the thickness of layers. Then, it calculates the corresponding structural coefficients with the procedures described earlier.

Figure 5.26 and Figure 5.27 depict the comparison of those twelve sections using the mechanistic method (M-M) and empirical method (E-M) from Equations 2.9 and 2.12. Results show that there is a good correlation in layer coefficients between the two methods. Results also provide strong evidence that the mechanistic method can be used to predict the layer coefficient accurately. The E-M layer coefficients for HMA are larger than those from the M-M. For HMA, the ratio of LC_{E-M} to LC_{M-M} varies from 0.97 to 2.5 with an average ratio of 1.7. The average layer coefficient for HMA is 0.57 from the in-situ data. The average layer coefficient for the rubblization layer is 0.43 from the simulation data. Similarly, for the rubblization layer, the ratio of LC_{E-M} to LC_{M-M} varies from 0.44 to 1.29 with an average ratio of 0.9. The P-value for the rubbilization layer is 0.60.

It is concluded that the two approaches have a significant difference for HMA layer coefficients, but not for the rubblization layer. The average layer coefficient for the rubblization layer is 0.22 from the in-situ FWD data and 0.22 from the simulation FWD data. The empirical equation developed using moduli back-calculation using ELMOD provides higher layer coefficient values for HMA than those suggested by the 1993 AASHTO Pavement Design Guide. This indicates that adjustment factors should be used to produce a reasonable HMA layer coefficient.

It is noted that the E-M calculation was conducted with the elastic moduli determined from back-calculation. On US highways and State Routes, the HMA moduli were consistent within the range of 500 ksi. For Interstate highways, the HMA moduli exceeded the upper limit of 500 ksi published by the 1993 AASHTO Pavement Design Guide. Due to this limitation in moduli back-calculation, using the empirical method frequently gives higher values than that of the mechanistic method. Another reason for this outcome is that the FWD measures in-situ behavior. The laboratory-supported 1993 AASHTO modulus coefficient relations may not accurately predict the in-situ situation. As expected, the backcalculation modulus is always higher than the lab testing modulus. Therefore, the strength and stiffness properties are very different from the properties witnessed in the laboratory.

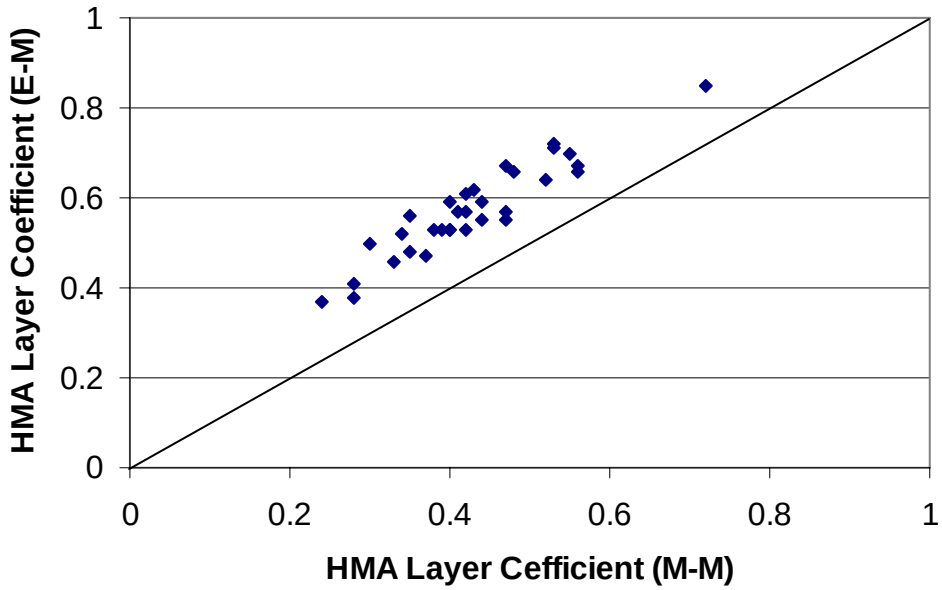

Figure 5.26 Comparison for HMA layer coefficients using mechanistic and empirical method

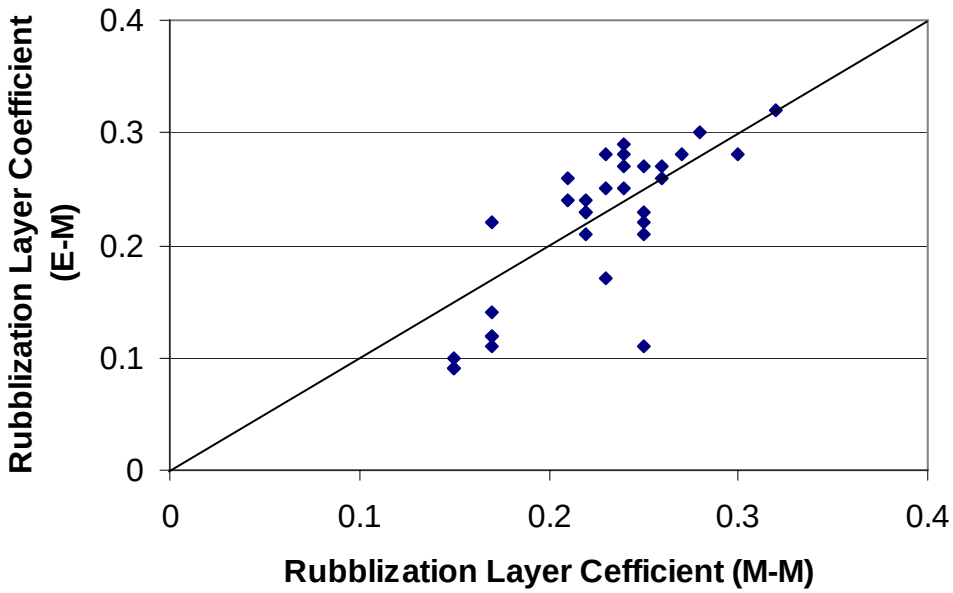

Figure 5.27 Comparison for rubblization layer coefficients using mechanistic and empirical method

5.7 Comparison of Empirical and Mechanistic procedure for Structure Number

The objective of this comparison was to analyze the structural adequacy between the in-situ data and data from elastic layer system theory. The first approach uses the in-situ FWD data to evaluate 1993 AASHTO structural numbers. The second approach uses the simulated FWD deflections from the back-calculated moduli and CHEVRONX program. Figure 5.28 shows the comparison of SN using deflections from the elastic layer simulation and FWD in-situ deflection data. The in-place SN is slightly larger than that of the elastic layer simulation. The ratio of SN_{E-M} to SN_{M-M} varies from 0.94 to 1.72 with an average ratio of 1.15. A statistical analysis was carried out using ANOVA, which assumed the differences between the two pairs of data were zero. The P-value is 0.06 that is greater than the α, i.e., 0.05. This implies that the differences are not significant. Thus, the elastic layer simulation is suitable for evaluating the capacity of pavement. The proposed approach can also be confidently used to calculate the SN number.

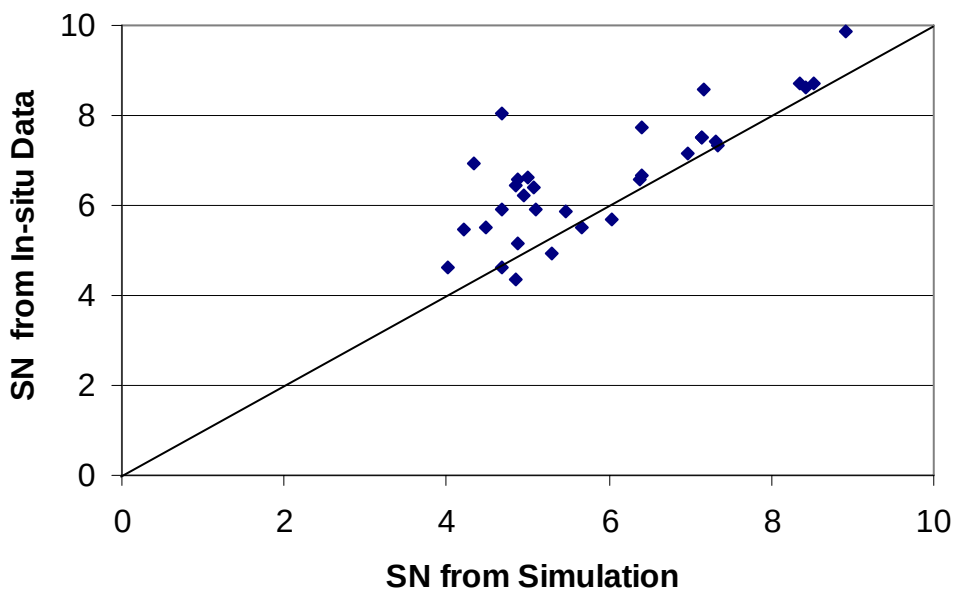

Figure 5.28 Comparison for SN using mechanistic and empirical method

5.8 Comparative Analysis of Structure Number with Years

Continueous monitoring of pavement structure numbers can provide better understanding of the deterioration of the overlaid rubblized pavement. Figure 5.29, Figure 5.30, and Figure 5.31 present comparisons of structural numbers between different pavement testing sections in 2001, 2002, and 2004. They show the Interstate pavements always have a higher structural number than that of the US Highways and State Routes. The Interstate pavements carry more traffic loading than the US Highways and State Routes. It can be concluded from these three figures that the SN does not always decrease over time. Therefore, more data from subsequent years are needed to monitor the pavement sections for future study. It also can be observed that the average values for both bound limits are almost the same. It can be concluded that the mean values may represent the pavement structural capacity.

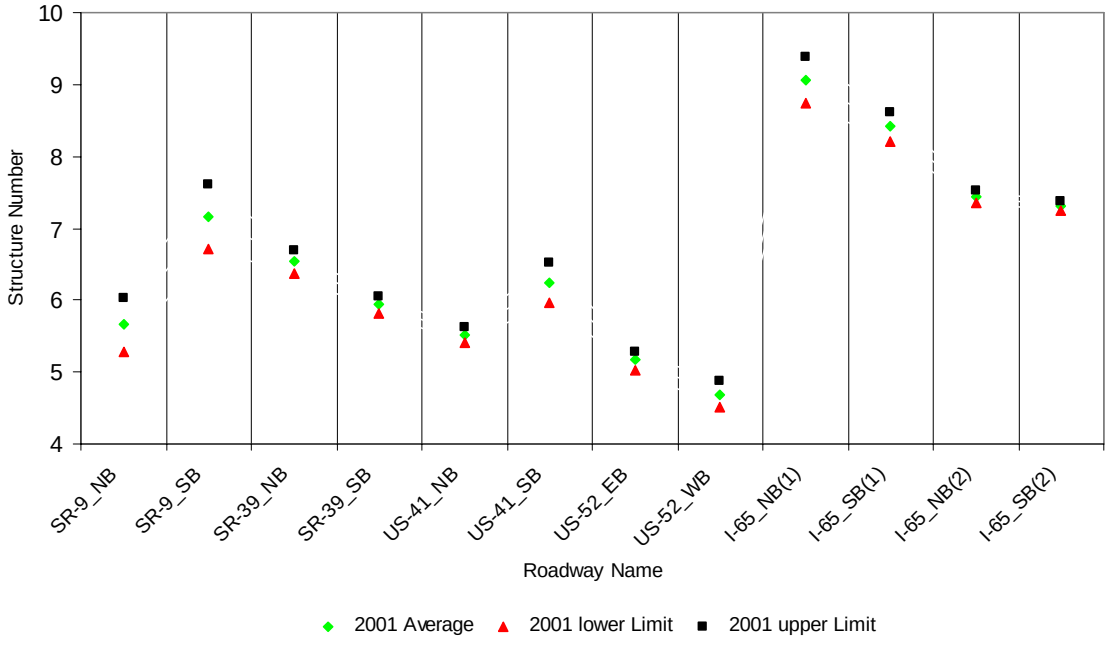

Figure 5.29 Summary statistic of Structure Number (SN) at 2001

Note: SR-9_NB: SR-9 NB from RP 47 to RP 49
SR-9_SB: SR-9 SB from RP 47 to RP 49
SR-39_ NB: SR-39 NB from RP 72 to RP 80
SR-39_ SB: SR-39 SB from RP 72 to RP 80
US-52_ EB: US-52 EB from RP 15 to RP 26
US-52_ WB: US-52 WB from RP 15 to RP 26
US-41_NB: US-41 NB from RP 91 to RP 99
US-41_SB: US-41 SB from RP 91 to RP 99
I-65_NB (1): I-65 NB from RP 117 to RP126
I-65_SB: (1) I-65 SB from RP 117 to RP 126
I-65_NB (2): I-65 NB from RP 181 to RP 198
I-65_SB: (2) I-65 SB from RP 181 to RP 198

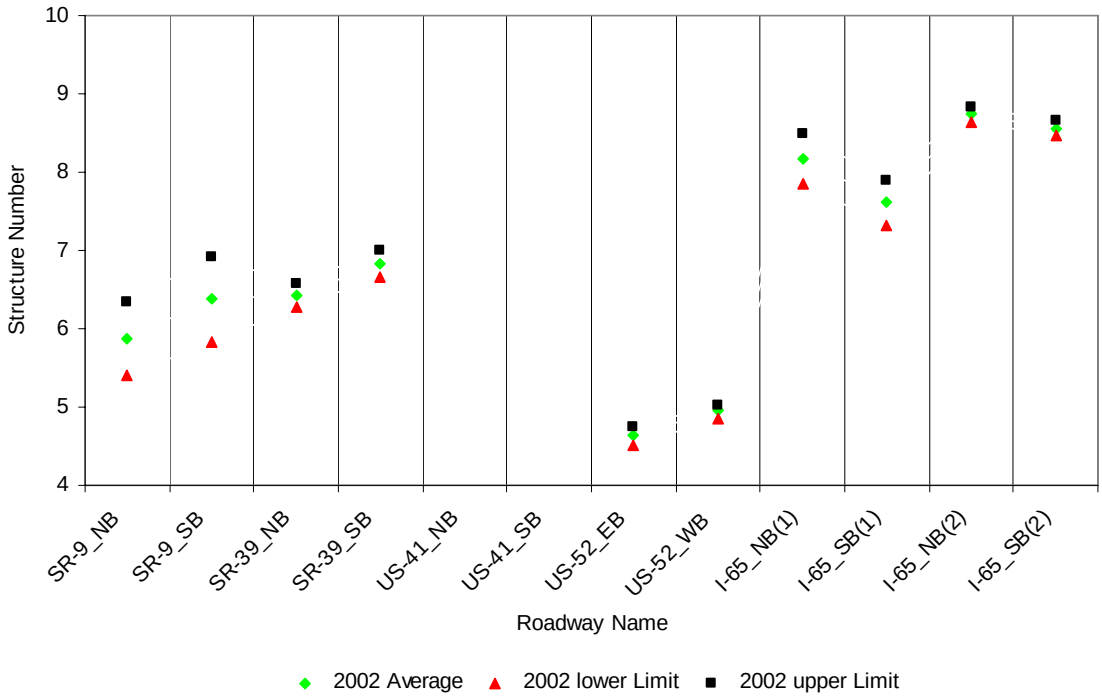

Figure 5.30 Summary statistic of Structure Number (SN) at 2002

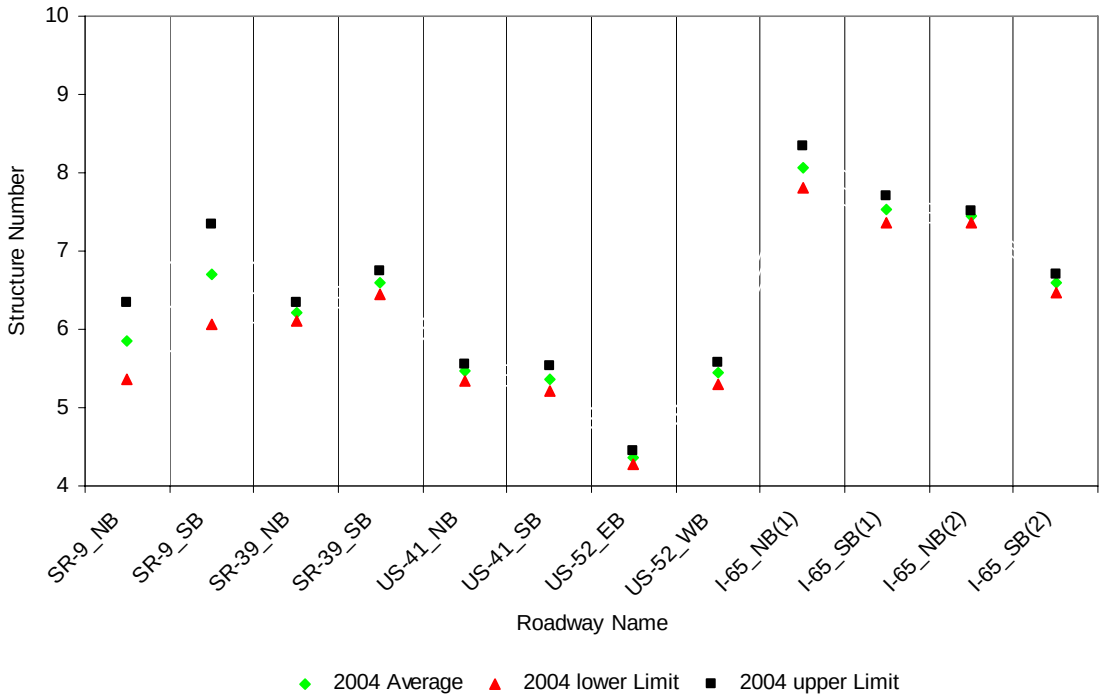

Figure 5.31 Summary statistic of Structure Number (SN) at 2004

5.9 Comparative Analysis of M_R with Years

As a part of this study, the results of M_R with data from three years (2001, 2002, and 2004) were compared. Figure 5.32 shows the estimated average subgrade resilient modulus. These values varied from year to year, since they were tested at different seasons in different years. Therefore, it can be concluded that the resilient modulus of soil varies seasonally. The relative difference between each year could be as high as 79% and as low as -28%. From the pavement design point of view, the design thickness will be changed greatly using different modulus. Seasonal modulus should be considered in the pavement thickness.

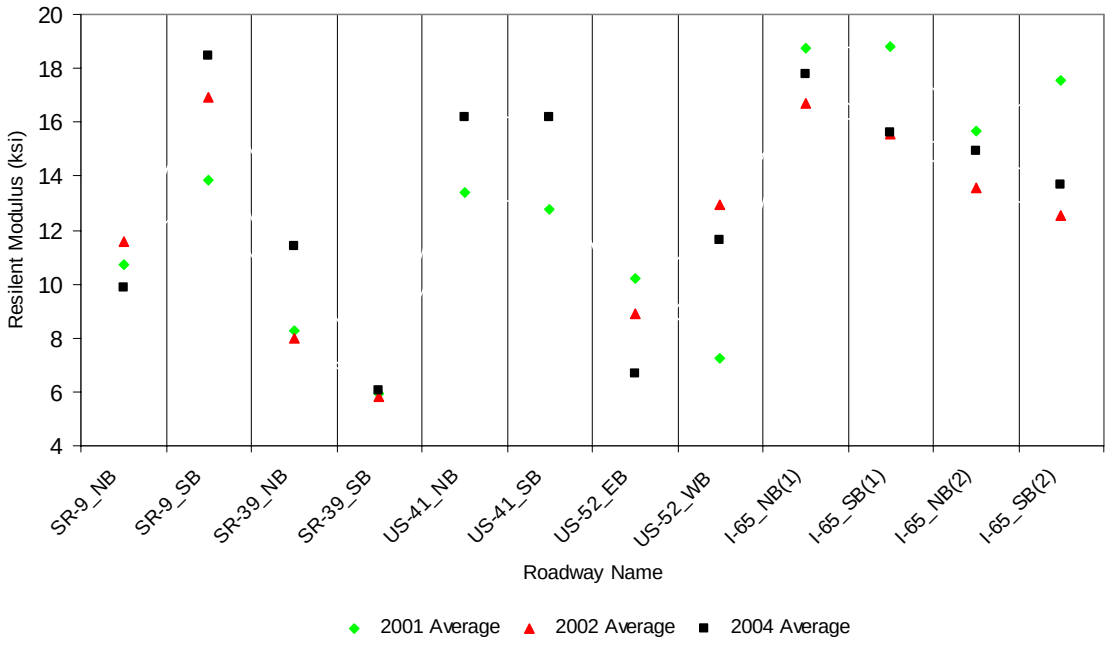

Figure 5.32 Summary statistic of M_R

Note: SR-9_NB: SR-9 NB from RP 47 to RP 49
SR-9_SB: SR-9 SB from RP 47 to RP 49
SR-39_ NB: SR-39 NB from RP 72 to RP 80
SR-39_ SB: SR-39 SB from RP 72 to RP 80
US-52_ EB: US-52 EB from RP 15 to RP 26
US-52_ WB: US-52 WB from RP 15 to RP 26
US-41_NB: US-41 NB from RP 91 to RP 99
US-41_SB: US-41 SB from RP 91 to RP 99
I-65_NB (1): I-65 NB from RP 117 to RP126
I-65_SB: (1) I-65 SB from RP 117 to RP 126
I-65_NB (2): I-65 NB from RP 181 to RP 198
I-65_SB: (2) I-65 SB from RP 181 to RP 198

Chapter 6 FINDINGS AND RECOMMENDATIONS

6.1 Summary

A computer program was developed by the author to simulate the layer coefficients for different layers and overall pavement SN from the back-calculated moduli. The results of in-situ FWD data then were carried out to explore trends of structure parameters variation such as subgrade resilient modulus, SN, and the layer coefficients. An analysis was presented using the 1993 AASHTO guide and mechanistic approach. Two different statistical analyses including ANOVA and Z-testing were then conducted to evaluate prepared pavement sections.

6.2 Findings

Based on the theoretical analysis conducted during this study, the following findings can be made:

1. FWD successfully determines the structural coefficient of the rubblized composite pavement. The mechanistic method (M-M) provides layer coefficients similar to those recommended by the values from the 1993 AASHTO Pavement Design Guide for both HMA and rubblization layers.

2. The empirical method (E-M) provides HMA layer coefficients different from the typical values from the 1993 AASHTO Pavement Design Guide and provides rubblization layer coefficient similar to the typical values from the 1993 AASHTO.

3. There is a high correlation between the structural numbers obtained from in-situ FWD deflections and layer elastic simulation deflections. This indicates that there is no significant difference between these two different procedures to evaluate the structural capacity. Therefore, the elastic layer system theory could provide good simulation for the in-situ FWD testing.

4. For 12 sections with a wide spectrum of soils, the modulus fluctuated from approximately 6 ksi to 19 ksi. In addition, M_R had significant difference during this testing period. Therefore, the assessment of how material properties change with time and location is difficult, and the seasonal moisture variation should be taken into account when designing HMA overlays PCC rubblized pavement.

6.3 Recommendations

Based on the theoretical analysis conducted during this study, the following recommendations can be made:

1. Layer coefficients are a function of many parameters, such as modulus, the stress level, and the position of layer, etc. The layer coefficients 0.42 for HMA and 0.22 for rubblization are recommended for the HMA overlay PCC pavement thickness design.

2. The accurate and precise rubblization layer thickness is critical to backcalculation, any error in the thickness assessment can have error in the backcalculation, and thus will affect the accuracy of evaluation.

3. The layer coefficients for HMA obtained from empirical equation are relatively higher than the range specified by the AASHTO Design Guide; the suitable adjustment factor is recommended to use to estimate the value of layer coefficients. The reason for using this factor could be that the backcalculation modulus is always higher than the value from lab testing.

4. No significant relationship was founded between the decrease of SN and ESAL from these test sections in the four-year testing period.

5. Finally, this mechanistic approach could be used to evaluate the layer coefficients for a flexible pavement.

APPENDIX – Program INPUT FILE

Data input files include LCC.dat and ELMOD_FWD.dat. The following shows an example of input data.

LCC.DAT

3	Pavement layer (including subgrade or stiff layer)
8.0 , 8.0	AC thickness, Rubblization thickness (unit: inch).
0.3 0.38 0.40	Poisson's ratio for HMA, Rubblization, and subgrade, respectively.
30	Number of testing points
9	Number of sensors
0 8 12 18 24 36 48 60 72	Sensor distance (Unit: inch)

ELMOD_FWD.DAT

DMI	D1	D2	D3	D4	D5	D6	D7	D8	D9	Load	DMI	E1	E2	E3	Air T	Sur T
0	2.35	1.87	1.74	1.42	1.14	0.94	0.76	0.61	0.52	87.21	0	1220	189	70	81.3	80.1
100	2.72	2.18	2.01	1.63	1.31	1.05	0.84	0.65	0.51	87.01	100	1044	189	47	81.6	80.2
211	2.24	1.72	1.61	1.37	1.19	0.99	0.81	0.73	0.57	87.31	211	1156	409	61	82.0	80.2
320	2.06	1.63	1.54	1.31	1.15	0.95	0.8	0.7	0.59	87.11	320	1410	381	64	81.7	80.6

Note: DMI unit is meter, Deflection unit is mils, E1 is elastic modulus of HMA (ksi), E2 is elastic modulus of rubblized PCC layer (ksi), E3 is elastic modulus of sugared (ksi), and temperature unit is F.

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