PHASE I OF M-E PDG PROGRAM IMPLEMENTATION IN FLORIDA

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16. Abstract

National Cooperative Highway Research Program Project 1-37A delivered the *Mechanistic-Empirical Pavement Design Guide* (M-E PDG) and its companion software (Version 0.7) in 2004. The M-E PDG represents a major change in the way pavement design is performed. The design method considers site conditions (e.g., traffic, climate, subgrade, existing pavement condition for rehabilitation) and construction conditions in proposing a trial design for new pavement construction or rehabilitation. The trial design is then evaluated for adequacy through the prediction of key pavement performance indicators and comparisons of these predictions with performance criteria set by the engineer. Since its initial release, the Florida Department of Transportation (FDOT) has embarked on efforts to implement a pavement design method based on the M-E PDG program. The initial stage of this implementation project focused on establishing and testing in-service pavement sections across Florida to develop a database for calibrating the existing M-E PDG pavement performance models. Additionally, a conceptual framework was established for developing an M-E PDG-based pavement design method that is tailored to current FDOT practice. This report documents the efforts conducted in the first phase of the implementation project.

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CHAPTER I. INTRODUCTION

National Cooperative Highway Research Program (NCHRP) Project 1-37A delivered the *Mechanistic-Empirical Pavement Design Guide* (M-E PDG) and its companion software (Version 0.7) in 2004. The M-E PDG represents a major change in the way pavement design is performed. The design method considers site conditions (traffic, climate, subgrade, existing pavement condition for rehabilitation) and construction conditions in proposing a trial design for new pavement construction or rehabilitation. The trial design is then evaluated for adequacy through the prediction of key pavement performance indicators and comparisons of these predictions with performance criteria set by the engineer. Since its initial release, state departments of transportation (DOTs) have embarked on efforts to implement the M-E PDG program. Research projects have also been initiated to evaluate the original program and develop performance models for top-down cracking and reflection cracking. Through reviews conducted on research projects, implementation efforts within state DOTs, and from pavement practitioners, the design program has seen a number of updates to address inadequacies identified from trial program applications and sensitivity analyses. At the time of this report, Version 0.91 is the most current version of the design program with 1.0 expected to be released in early 2007.

The Florida Department of Transportation (FDOT) sponsored a research project with the Texas Transportation Institute (TTI) to implement the M-E PDG program in the state. The Department presently uses a design method based on the 1993 pavement design guide approved by the American Association of State Highway and Transportation Officials (AASHTO). Implementation of the M-E PDG program will entail a significant change in current design practice within the Florida DOT. Since the new program requires comprehensive input data for a given analysis, considerable efforts are needed to characterize traffic and material inputs with field and laboratory testing. For an engineer to design a specific pavement, it is necessary to assume an initial structure and run the program repetitively until a pavement design is identified that satisfies the performance criteria for the given problem. From this perspective, many practitioners have remarked that the program is not a pavement design program per se but an analytical tool for predicting pavement performance given the design parameters. In this respect, it is unlike the current Florida pavement design method. Implementing the M-E PDG program as is will mean a significant change in current practice within state DOTs.

The performance models in the new program were also developed and calibrated based on a national database of field pavement performance data. For states seeking to implement the M-E PDG, the developers of the design guide have recommended that the models be calibrated to local conditions. For this purpose, the authors of the new guide included an option in the program that permits users to input calibration factors to tailor the performance models to local conditions.

Given the above considerations, and recognizing that further changes to the design guide will come about from on-going national research projects, a staged implementation of the M-E PDG in Florida will need to be conducted. In the current project, this implementation proceeded with establishing and testing in-service pavement sections across Florida to develop a database for calibrating the existing M-E PDG performance models. Researchers also framed a conceptual pavement design guide that represents a proposed adaptation of the M-E PDG to suit current DOT practice. This report documents the initial steps taken to implement an M-E PDG-based design procedure within the Florida DOT.

RESEARCH OBJECTIVES

The primary objectives of this project are to:

- provide a database for verifying and calibrating, as necessary, the performance models in the existing M-E PDG program; and
- establish a conceptual framework for developing a new Florida pavement design method based on the M-E PDG.

To accomplish these objectives, the following tasks were conducted:

- examination of Florida's pavement condition survey (PCS) database to identify in-service pavement sections for model calibrations;
- sensitivity analyses to identify critical factors affecting predicted pavement performance from the M-E PDG program;
- characterization of climatic-soil variations across Florida;
- field and laboratory tests to characterize material properties of in-service pavement sections for model calibration;

- compilation of database for model calibration; and
- development of conceptual framework for establishing an M-E PDG-based pavement design guide for the Florida DOT.

SCOPE OF RESEARCH REPORT

This report documents the initial steps taken to implement an M-E PDG-based design procedure in Florida. The report is organized into the following chapters:

- Chapter I provides the impetus for this project and states its objectives.
- Chapter II describes the material characterizations required for the mechanistic-empirical design approach used in the new guide. This background material is provided for the purpose of establishing the test plan to characterize in-service pavement sections for model calibration.
- Chapter III presents the determination of candidate sections for model calibration.
- Chapter IV documents the efforts made by researchers to characterize the climatic and soil variations across the state for the purpose of establishing a conceptual framework for an M-E PDG-based design method tailored to Florida conditions.
- Chapter V describes the field and laboratory tests conducted in this project to characterize in-service pavement sections established for model calibration.
- Chapter VI presents the conceptual framework for developing a pavement design guide based on the M-E PDG program.
- Chapter VII summarizes the findings from this project and presents recommendations for continuing the efforts initiated in this project to implement the M-E PDG in Florida.

The appendices provide supporting material for the tasks conducted in this project that are documented in the different chapters of this report.

CHAPTER II. REVIEW OF M-E PDG INPUT REQUIREMENTS

This chapter describes the material characterizations required for the mechanistic-empirical design approach used in the guide. It provides background material on the input requirements of the M-E design guide system for the purpose of establishing a test plan to characterize the pavement sections proposed for model calibration. To provide a common basis for understanding the material requirements, the following M-E-based sub-categories have been developed (NCHRP 1-37A, 2004):

- In the first category are material properties required to predict the states of stress, strain, and displacement within pavement structures when subjected to wheel loads. These properties include elastic modulus and Poisson's ratio of each pavement material.
- In the second category are the materials-related inputs that are used directly in the distress and smoothness models incorporated into the M-E PDG. For each of these distresses, the critical structural response under a given wheel and climatic loading condition is affected by modulus and Poisson's ratio. In addition, parameters such as strength, expansion-contraction characteristics, friction between slab and base, erodibility of underlying layers, layer drainage characteristics, plasticity, gradation, and other materials-related parameters are needed in predicting the development of pavement distresses.
- Finally, in the third category are materials-related inputs that are used with the climatic module in the design guide program to determine the temperature and moisture profiles through the pavement cross section. These include engineering index properties, gradation parameters, and thermal properties.

Table 2.1 is a tabular summary of the material inputs required for mechanistic-empirical design arranged by the major material categories. Researchers used the information from this review to identify the laboratory and field tests needed to characterize material properties for calibrating the performance models in the design guide program.

Table 2.1. Material Input Requirements by Material Group (Olidis and Hein, 2004).

Material	Required Information
Hot Mix Asphalt	Dynamic modulus Poisson's ratio Tensile strength Coefficient of thermal expansion Thermal Conductivity Asphalt binder stiffness Aggregate properties
Portland Cement Concrete	Modulus of elasticity Poisson's ratio Unit weight Coefficient of thermal expansion Mix properties Aggregate type Thermal Conductivity Heat capacity
Chemically Stabilized Material	Elastic modulus Poisson's ratio Unit weight Modulus of rupture Thermal Conductivity Heat capacity
Unbound Material	Resilient modulus Poisson's ratio Unit weight Gradation Optimum moisture content Plasticity index
Recycled Asphalt Pavement	Resilient modulus Poisson's ratio Unit weight Gradation Hydraulic conductivity Optimum moisture content Plasticity index
Bedrock	Elastic modulus Poisson's ratio Unit weight

MULTI-LEVEL INPUT APPROACH

The M-E PDG program provides three levels for selecting or determining traffic, material, and environment-related inputs for pavement performance predictions. Since users may employ combinations of these three main levels for a particular design problem, the new guide provides flexibility in characterizing program inputs for analysis. Level 1 involves comprehensive laboratory and field testing to characterize design inputs. Cases in which a Level 1 design would be appropriate are on projects where a high confidence level is required, such as a principal arterial with high traffic volumes or test sections used for pavement research. In contrast, Level 3 requires the designer to estimate the most appropriate design input value of a material property based on experience with little or no testing. This level has the least accuracy and would typically be used for lower volume roadways. Inputs at Level 2 are estimated through correlations with other material properties that are measured from laboratory or field tests.

As indicated previously, it is possible for a designer to mix and match the levels of input for a specific project or region. For example, a user may select Level 2 to specify subgrade properties because the subgrade in a particular region is well characterized by other tests such as CBR, and Level 1 to specify traffic inputs determined from weigh-in-motion data collected at a comparable facility.

CHARACTERIZATION OF ASPHALT CONCRETE MATERIALS

The primary stiffness property for asphalt materials in the M-E PDG program is the time-temperature dependent dynamic modulus (E*). Dynamic modulus testing (as proposed by NCHRP 1-28, "Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design"), and asphalt binder complex shear modulus and phase angle testing (AASHTO T315) are used to develop a master curve that represents the time-temperature behavior of the asphalt concrete (AC) mix based on the following equation:

$$\log E^* = \delta + \frac{\alpha}{1 + e^{\beta + \gamma \log t_r}} \tag{2.1}$$

where t_r is the time of loading at the reference temperature, and the model coefficients δ, α, β, and γ are defined as follows:

$$\delta = 3.750 + 0.029\,p_{200} - 0.002\left(p_{200}\right)^2 - 0.003\,p_4 - 0.058V_a - 0.802\left(\frac{V_{beff}}{V_{beff} + V_a}\right) \tag{2.2}$$

$$\alpha = 3.872 - 0.002\,p_4 + 0.004\,p_{38} - 0.000017\,p_{38}{}^2 + 0.005\,p_{34} \tag{2.3}$$

$$\beta = -0.603 - 0.394\log \eta_T \tag{2.4}$$

$$\gamma = 0.313 \tag{2.5}$$

It is observed that δ and α are functions of volumetric mixture properties, specifically:

- air voids content V_a (%)
- effective bitumen content by volume V_{beff} (%),
- cumulative percent retained on ¾-inch sieve, p_{34},
- cumulative percent retained on 3/8-inch sieve, p_{38},
- cumulative percent retained on No. 4 sieve, p_4, and
- cumulative percent retained on No. 200 sieve.

The loading time t_r in Eq. (2.1) is related to the loading time t at the temperature of interest through the equation:

$$\log \frac{t_r}{t} = -1.256 \log \frac{\eta}{\eta_T} \tag{2.6}$$

where η and η_T are the bitumen viscosities at the corresponding temperatures.

The asphalt concrete materials characterization procedure used in the design guide accounts for short term binder aging during asphalt mixing and placement at initial construction, and age hardening on the basis of the global aging system. For a Level 1 rehabilitation project, the master curve for the asphalt concrete is developed using test data collected from the falling weight deflectometer (FWD) and from laboratory testing of extracted cores. To obtain the relationship between the backcalculated modulus and temperature, the test temperature needs to be recorded during the FWD data collection. Cores extracted from the pavement are subjected to standard asphalt concrete testing (air voids, asphalt cement content and gradation) to develop an undamaged master curve for the aged asphalt concrete. A damage transfer function then combines the results of the backcalculation and laboratory testing to develop a field master curve. The Level 2 design uses some additional resilient modulus testing while the Level 3 analysis employs a typical asphalt concrete master curve and the visual survey data to determine the field master curve.

The asphalt concrete master curves are used as inputs to the distress prediction equations to determine the amount of cracking and rutting. The following additional tests are conducted (as applicable for the given local conditions) to predict thermal cracking:

- Tensile strength and creep compliance (AASHTO T322); and
- Thermal conductivity and heat capacity (ASTM E 1592 and ASTM D2766).

For the first set of properties, the indirect tensile test (AASHTO T322) is performed on hot-mix asphaltic concrete (HMAC) specimens. Tensile strength and creep compliance are significant predictors of thermal cracking in the M-E PDG program. In addition, if the NCHRP 1-28 test protocol is employed to determine HMAC resilient modulus, the tensile strength of the material needs to be known to establish loading levels in accordance with the SHRP P07 protocol.

Based on the review of the design guide, researchers suggested that FWD testing and coring be performed to characterize material properties of pavement sections proposed for model calibrations. In addition, the individual AC lifts need to be identified from the cores to establish the requirements for characterizing the asphalt-bound material for predicting the performance of the calibration sections using the M-E PDG program. Researchers also recommended that pavement test temperatures be recorded during FWD testing, with infrared surface temperatures taken as a minimum to predict pavement temperatures at the time of testing using an existing method like the BELLS equation (Stubstad et al., 1998).

With respect to the asphalt concrete modulus, extracted cores may be tested in accordance with NCHRP 1-28 or AASHTO T315. If a core specimen has more than one layer, the layers should be separated at the layer interfaces by sawing. Layers containing more than one lift of the same material as placed under contract specification may be tested as a single specimen. If the NCHRP 1-28 protocol is selected, the resilient modulus is measured using the repeated load indirect tensile test. This allows characterization of the moduli of different asphalt lifts instead of a composite modulus that is estimated from the backcalculation. Alternatively, cores can be tested to establish mix volumetric parameters (air voids, asphalt volume, gradation, and asphalt viscosity) to estimate dynamic modulus using the equation by Witczak and Fonseca (1996) that is incorporated into the M-E pavement design guide.

To consider aging of materials, the binder viscosity can be estimated using conventional asphalt test data such as ring and ball softening point, absolute and kinematic viscosities, or using the Brookfield viscometer. If the binder viscosity-temperature relationship cannot be determined from binder testing, consideration may be given to using typical A-VTS coefficients provided in the design guide software based on performance grade, viscosity, or penetration grade of the binder.

Finally, the indirect tensile test (AASHTO T322) may be performed to measure tensile strength and creep compliance properties needed to predict crack propagation. For a Level 2 analysis, this test may be conducted only for the intermediate temperature of 14 °F. Alternatively, the following regression equations may be used in the absence of indirect tensile test data:

$$TS(psi) = 7417 - 114V_a - 0.304\,V_a^2 - 123VFA + 0.704VFA^2 + 406\,Log(Pen_{77}) - 2039Log(A) \tag{2.7}$$

$$D(t) = D_1\,t^m \tag{2.8}$$

$$\log D_1 = -8.524 + 0.013Temp + 0.796\log V_a + 2.010\log VFA - 1.923\log A \tag{2.9}$$

$$m = 1.163 - 0.002Temp - 0.046\,V_a - 0.011\,VFA + 0.002\,Pen_{77} + 0.002Temp \times Pen_{77}{}^{0.4905} \tag{2.10}$$

where,

TS = indirect tensile strength (psi) at 14 °F,

V_a = as-constructed air voids (percent),

VFA = voids filled with asphalt (percent),

Pen_{77} = binder penetration (mm/10) at 77 °F,

A = intercept of binder viscosity- temperature relationship,

$D(t)$ = creep compliance as a function of loading time *t*, and

$Temp$ = temperature (°F) at which creep compliance is measured.

Dynamic modulus curves in the form of a sigmoidal function can be developed by shifting laboratory test data. The resulting shift factors can be expressed as a function of binder viscosity. This step allows the consideration of binder aging using the global aging system (Mirza and Witczak, 1995), which includes four models:

- original to mix laydown model,
- surface aging model,
- air void adjustment, and
- viscosity-depth model.

To consider the effect of aging in the performance prediction, it is necessary to estimate the initial properties, i.e., viscosity and air void content. The M-E design guide program goes step by step through each of the above aging models beginning with the known initial properties to establish the undamaged master curve for the mix. However, for the model calibrations, tests would be run on existing pavements and samples taken from the calibration sections. Thus, the test data would represent aged conditions. For calibration, it would be necessary to backcalculate the initial properties using the M-E design guide aging models with the aged properties determined from testing. With this perspective, researchers established a procedure by which the initial viscosity and air void content may be backcalculated. This procedure uses the aging models in reversed order, i.e., from the viscosity-depth model to the original to mix/lay-down model. The steps in this procedure are given in the following:

- Determine aged volumetric and bitumen properties from cored samples.
- Estimate the mix laydown viscosity $\eta_{t=0}$ given the aged mix properties from tests on cores using the viscosity-depth model and the surface aging model. From the former model:

$$\eta_{t,z} = \frac{\eta_t (4 + E) - E(\eta_{t=0})(1 - 4z)}{4(1 + Ez)} \tag{2.11}$$

$$E = 23.83 e^{-0.0308\,Maat} \tag{2.12}$$

where,

$\eta_{t,z}$ = known aged viscosity at a specific time t (months) and depth z (inches),

η_t = aged surface viscosity,

$\eta_{t=0}$ = mix laydown viscosity, and

$Maat$ = mean annual air temperature, °F.

From the surface aging model:

$$\log\log(\eta_t) = \alpha = \frac{\log\log(\eta_{t=0}) + At}{1 + Bt} \tag{2.13}$$

$$\eta_t = 10^{10^{\alpha}} \tag{2.14}$$

In Eq. (2.13), A is a function of the binder temperature T_R in °R, mean annual air temperature, and mix laydown viscosity, while B is a function of T_R. By substituting Equation 2.13 into Equation 2.11, the mix laydown viscosity $\eta_{t=0}$ can be determined.

- Estimate the original bitumen viscosity η_{orig} using the calculated value of $\eta_{t=0}$ in the original to mix laydown model:

$$\log\log(\eta_{t=0}) = a_0 + a_1 \log\log(\eta_{orig}) \tag{2.15}$$

$$a_0 = 0.054 + 0.004\,code \tag{2.16}$$

$$a_1 = 0.972 + 0.011\, code \tag{2.17}$$

where code is the hardening ratio with a recommended value of zero for average hardening resistance.

- Estimate the original air voids VA_{orig} using the air void adjustment model with the measured air voids content VA from core samples and the other known parameters. The air void adjustment model is given by:

$$VA = \frac{VA_{orig} + 0.011(t) - 2}{1 + 4.24 \times 10^{-4}(t)(maat) + 1.169 \times 10^{-3}\left(\frac{t}{\eta_{orig}}\right)} + 2 \tag{2.18}$$

Once the initial bitumen viscosity and air voids content are determined, the M-E design guide program is executed using these as inputs to the aging model to predict the pavement performance for a given calibration section.

CHARACTERIZATION OF PORTLAND CEMENT CONCRETE MATERIALS

The Portland cement concrete (PCC) modulus of elasticity is used as an input to characterize the performance of PCC pavements. For Level 1, the PCC modulus of elasticity and Poisson's ratio are determined through laboratory testing using ASTM C469, "Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression." PCC elastic modulus values for the proposed mixture are required at 7, 14, 28, and 90 days. However, in practice, it might be difficult to perform these tests due to time constraints. For these cases, the modulus can also be backcalculated from FWD data. In this regard, the modulus values obtained from laboratory tests have been reported to be less than the corresponding moduli determined from nondestructive tests conducted with the FWD or with seismic methods. The ratio is approximately 0.8.

For Level 2, AASHTO T22, "Compressive Strength of Cylindrical Concrete Specimens," is performed at 7, 14, 28, and 90 days. Once the compressive strength data are input to the design program, PCC elastic modulus is internally determined using the following equation:

$$E_C = 33\rho^{3/2}(f_c^{'})^{1/2} \tag{2.19}$$

where,

E_C = PCC elastic modulus (psi),

ρ = unit weight of concrete (lb/ft^3), and

f'_c = compressive strength (psi).

Level 3 uses a correlation equation and the specified 28-day strength of the concrete mix to estimate the elastic modulus.

The flexural strength (MR) is defined as the maximum tensile stress at rupture at the bottom of a simply supported concrete beam during a flexural test with third point loading. For Level 1 design, the flexural strength is determined in the laboratory by testing beams under three point loading (AASHTO T97, "Flexural Strength of Concrete") for specimens aged at 7, 14, 28 and 90 days. Level 2 uses the compressive strength measured on cores taken at various ages (7, 14, 28 and 90 days) and a correlation equation to estimate MR. Level 3 uses a correlation equation and the specified 28-day strength of the concrete mix.

Poisson's ratio is a required input to the structural response computation models, although its effect on computed pavement response is not great. As a result, this parameter is rarely measured and is often assumed. Poisson's ratio for normal concrete typically ranges between 0.11 and 0.21, and values between 0.15 and 0.18 are typically assumed for PCC pavement design.

For Level 1 analysis, the unit weight of PCC materials may be determined by testing in accordance with AASHTO T121 "Mass per Cubic Meter, Yield, and Air Content of Concrete" and AASHTO T271 "Density of Plastic and Hardened Concrete in-Place by Nuclear Method" for new and rehabilitation projects, respectively. There is no specific correlation with other parameters that may be used for a Level 2 analysis. Typical values for normal weight concrete range from 140 to 160 lb/ft^3. A unit weight within this range may be assumed for a Level 3 design.

Other important parameters of PCC materials considered by the M-E design guide include:

- coefficient of thermal expansion (AASHTO TP 60, "Coefficient of Thermal Expansion of Hydraulic Cement Concrete"),
- PCC shrinkage (AASHTO T160, "Length Change of Hardened Hydraulic Cement Mortar and Concrete"),
- thermal conductivity (ASTM E 1592), and
- heat capacity (ASTM D 2766, "Standard Test Method for Specific Heat of Liquids and Solids").

These properties are important in modeling the effects of temperature and moisture variations on the properties of the PCC slabs. Shrinkage and thermal expansion can cause significant curling and warping in PCC slabs, resulting in pavement cracking. In particular, determining the thermal expansion coefficient through direct testing under Level 1 is recommended since this parameter is extremely significant.

CHRACTERIZATION OF UNBOUND GRANULAR AND SUBGRADE MATERIALS

The material parameters required for unbound granular materials, subgrade, and bedrock may be classified in one of three major groups:

1. pavement response model material inputs (resilient modulus and Poisson's ratio),
2. enhance integrated climatic model (EICM) material inputs (gradation, Atterberg limits, and hydraulic conductivity), and
3. other material properties (for example, the coefficient of lateral pressure).

For Level 1 analysis, resilient modulus (M_r) values for unbound granular materials, subgrade, and bedrock are determined from cyclic triaxial tests on prepared representative samples. The recommended standard methods for modulus testing are:

1. NCHRP 1-28A, "Harmonized Test Methods for Laboratory Determination of Resilient Modulus for Flexible Pavement Design," and
2. AASHTO T307, "Determining the Resilient Modulus of Soil and Aggregate Materials."

The Level 1 procedure is applicable to new design, reconstruction, and rehabilitation projects. For reconstruction and rehabilitation purposes, material samples can be obtained through destructive testing (i.e., coring, trenching). Furthermore, for rehabilitation and reconstruction of the existing pavement layer, the FWD may be used to backcalculate layer moduli.

At Level 2, relationships between soil index properties, strength properties, and resilient modulus may be used to estimate M_r. For Level 3 designs, the resilient modulus of unbound granular materials is selected based on the unbound material classification (AASHTO or Unified Soil Classification system). The design guide provides a general range of typical modulus values based on averages from data collected on long term pavement performance (LTPP) sections. These typical values are provided for each unbound granular material classification at their optimum moisture contents.

Other important parameters of unbound materials considered by the design guide include:

- Atterberg limits (AASHTO T 89-90),
- grain size distribution (AASHTO T 27), and
- moisture-density relationship (AASHTO T99).

It is highly recommended that samples be taken to identify properties mentioned above because these properties are critical in the assessment of moisture variation and seasonal modulus values that significantly affect predicted pavement performance.

CHARACTERIZATION OF CHEMICALLY STABILIZED MATERIALS

Chemically stabilized materials covered in the design guide include lean concrete, cement-stabilized, cement-treated open-graded drainage layers, soil-cement, lime, cement and fly ash-treated materials. The elastic modulus of the layer is the primary input parameter for chemically stabilized materials. For lean concrete and cement-treated materials in new pavements, the elastic modulus is determined using ASTM C 469. For lime-stabilized materials, AASHTO T 307 protocols apply. For each of the stabilized materials, relationships between the elastic modulus and compressive strength have been developed. For rehabilitation projects, the elastic modulus of the stabilized layer is determined through FWD backcalculation, or through dynamic cone penetrometer (DCP) testing in conjunction with a correlation equation.

The flexural strength of a stabilized layer is an important input parameter for flexible pavements only. Level 1 test procedures for chemically stabilized materials include:

- AASHTO T97 for testing lean concrete, cement-treated aggregate layers, and lime, cement and fly ash-treated layers, and
- ASTM D1635 for testing soil cement.

Level 2 test procedures use correlations to estimate the flexural strength for stabilized materials. Alternatively, the DCP can be used to obtain estimates of stiffness. The DCP provides a log of resistance to penetration under an impact load and has been correlated to CBR which is correlated with in situ modulus. Other important parameters of stabilized materials considered by the design guide include:

- thermal conductivity (ASTM E 1952), and
- heat capacity (ASTM D 2766).

RECYCLED CONCRETE MATERIALS

Recycled concrete materials are treated similarly as unbound materials. The recycled concrete is tested to determine its resilient modulus by laboratory testing (if broken to aggregate-sized pieces) or through FWD testing if broken in the field into fractured slabs.

RECYCLED HOT-MIX ASPHALT MATERIALS

Recycled hot mix asphalt is treated similarly as new asphalt concrete materials with inputs required to determine the modulus for each temperature, and shift factors obtained by data shifting from the master curve.

BEDROCK MATERAILS

The M-E design guide also requires input of the bedrock modulus for predicting the performance of a given pavement section. As actual resilient modulus testing of bedrock for pavement design is rare, the design guide provides the following default resilient modulus values for bedrock:

1. uniform, solid bedrock – 1000 ksi, and
2. highly fractured, weathered bedrock – 500 ksi.

The Poisson's ratio for bedrock is selected as 0.15 for uniform, solid bedrock and 0.30 for highly fractured and weathered bedrock.

REQUIRED TESTS TO DETERMINE M-E PDG INPUTS FOR MODEL CALIBRATIONS

Based on the review of M-E design guide inputs, researchers summarized the required tests to characterize the materials found on PCS pavement sections to calibrate the M-E design guide performance models to Florida conditions. To establish the scope of tests, researchers reviewed published reports on the sensitivity of the performance predictions to the design guide inputs and conducted sensitivity analyses to evaluate the effects of design guide input parameters, assuming pavement sections and environmental conditions representative of those found in Florida. For this purpose, researchers used conventional flexible and rigid pavement cross-sections in the runs made of the M-E PDG program (Version 0.8). The flexible pavement structure comprised a four-layer pavement system with an asphalt concrete layer (4 inches thick), a limerock base (10 inches thick), stabilized subgrade (12 inches thick), and sand subgrade. The rigid pavement structure consisted of six-layers with an 8-inch jointed plain concrete pavement (JPCP) slab, existing AC layers consisting of 4 inches of asphalt permeable base over a 2-inch dense graded mix, a 10-inch limerock base, 12-inch stabilized subgrade, and sand subgrade. Both pavements were assumed to receive an average annual daily truck traffic (AADTT) of 7000, which translates to approximately 40 million cumulative heavy trucks during the assumed 20-year design period. Climatic conditions representative of Orlando were input into the EICM module of the M-E pavement design guide system.

Table 2.2 identifies those variables that are deemed important on the basis of the sensitivity of the IRI and cracking predictions to changes in these input parameters and presents proposed tests to characterize these variables on the basis of these results. The sensitivity of the predicted pavement performance to the different input variables is indicated by the arrow in the second column of Table 2.2. An upward arrow indicates high sensitivity of the predicted performance to the given variable, and vice versa.

The sensitivity of AC modulus on the performance predictions based on cracking and IRI was performed at two different levels. For Level 1, researchers varied dynamic modulus values at different temperatures and frequencies by ± 30 percent. For Level 2, researchers specified different levels of gradation, air voids, and effective binder content to determine low, medium, and high AC modulus values considered representative of properties found in practice. The low AC modulus mixture contains a larger amount of percent passing the No. 200 sieve, and smaller amounts of percent retained on the ¾-inch, 3/8-inch, and No. 4 sieve sizes. Researchers varied the effective binder content from 10 to 12 percent by volume, and the air voids contents from 9 to 7 percent to determine, respectively, the low and high levels of AC modulus. Researchers observed that the pavement with higher AC modulus performed well with respect to both longitudinal cracking and IRI.

For PCC pavements, the variation of AC modulus under the slab did not show a significant influence on the predicted PCC pavement performance. For these pavements, the dominant factors are the concrete thermal coefficient of expansion and compressive strength. Also, variations of the dowel bar diameter and joint spacing affected the predicted pavement performance considerably.

The resilient moduli of base and subgrade materials were found to have a significant effect on the predicted performance of flexible pavements. For this reason, researchers recommend resilient modulus tests to characterize the base, subgrade, and embankment

Table 2.2. Proposed Test Plan for Characterizing Material Properties on Model Calibration Sections.

Parameter	Sensitivity	Level	Applicable Test(s)	Recommendation
AC modulus	↑	1	NCHRP (1-28A) : 4”×6”	A Level 2 approach is recommended. Since most AC layers are composed of several lifts, we need to identify each layer property. Most lift thicknesses are also expected to be thin and not appropriate for testing dynamic modulus. In addition, air voids, gradation, and asphalt content are general input parameters to the M-E design guide program and must be input even for a Level I analysis.
		2	AASHTO T 315	
Layer thickness	↑	1	GPR, DCP and/or coring	GPR and coring are recommended.
Tensile strength & creep compliance	↑	1	AASHTO T 322	A Level 1 approach is recommended. (4~6" diameter and 2~3" height) (Test temperature : -4, 14, and 32 °F for creep compliance and 14 °F for tensile strength)
		2	AASHTO T 322 (Test only at 14 °F for tensile strength and creep compliance)	
Base modulus	↑	1	FWD (ASTM D 4694)	The minimum recommended number of test points is 30 or at 0.1-mile intervals, whichever is less. Three drops should be performed at each test point, with the first two drops applied to seat the load. A target load of 9 kips is recommended for the third drop (usually achieved under drop height two of the FWD). Pavement temperature measurements are required.
Subgrade modulus	↑	1	FWD (ASTM D 4694)	
Soil suction	↑	1	Suction test, AASHTO T27, AASHTO T90, FM 1-T 180	A Level 1 approach is recommended. In addition, soil properties (PI, gradation, optimum moisture content, and maximum dry unit weight) need to be measured. Estimates of the required material quantities are: Suction test (50 lbs), FM 1-T180 (50 lbs), and AASHTO T27 (35 lbs).
		2	AASHTO T27, AASHTO T90, FM 1-T 180	
Bedrock modulus	↓	N/A	Not typically tested.	Assume typical value for analysis.
Thermal coefficient of expansion	↑	1	AASHTO TP 60	If a catalog of thermal coefficients from tests done on typical concrete mixtures is available, then use this catalog to get appropriate value. Otherwise, run test.
Joint spacing	↑	N/A	Field survey or historical data	Measure joint spacing.
Dowel bar spacing	↓	N/A	Historical data or plan sheets	No recommendation. Assume typical value for analysis.
Dowel bar diameter	↑	N/A	Coring or plan sheets	Check plan sheets or ask pavement design engineer. If not available, take a core at the joint.
PCC compressive strength	↑	2	AASHTO T 22 (7,14,28, and 90 days)	A Level 3 approach is recommended. Run test to get compressive strength at 28 days on 4-inch diameter, 8-inch high concrete samples.
		3	AASHTO T 22	
Vehicle speed	↓	N/A	Traffic data collection	Use posted highway speed.

materials on the selected HMAC calibration sections. In addition, FWD tests should be performed to verify the correlation between laboratory resilient modulus and the corresponding modulus based on FWD backcalculations. Table 2.2 gives the recommendations for FWD testing on this project.

Researchers also found that the parameters of the soil-water characteristic curve significantly influenced the predicted pavement performance, especially for flexible pavements. For the Level 1 analysis, researchers varied the soil suction parameters to represent soils with high moisture content (low suction), and low moisture content (high suction). At this level, the optimum moisture content, maximum dry unit weight, percent passing the No. 200 sieve, and the plasticity index (PI) are required to compute the specific gravity and saturated volumetric water content. For the Level 2 analysis, researchers varied the soil physical properties (optimum moisture content, maximum dry unit weight, percent passing the No. 200 sieve, and PI) to determine soil suction parameters using correlation equations between soil properties and suction parameters. Researchers found that the change of suction parameters was significantly related to the development of predicted distresses. In practice, the soil-water characteristic curve for a given material can be characterized using filter paper or the psychrometer. Based on current test practice, researchers provided estimates of the sample quantities required to obtain suction parameters and soil properties. Table 2.2 shows these estimates.

CHAPTER III. DETERMINATION OF CANDIDATE SECTIONS FOR MODEL CALIBRATION

This chapter describes how researchers identified candidate in-service pavement sections for verifying and calibrating the performance models in the existing M-E PDG program. The factors considered in selecting candidate sections were established based on consultation with the Department's project manager. These factors are the observed performance history and the availability of traffic data from existing weigh-in-motion (WIM) sites located on highways with the same functional classification as a given candidate section. Researchers used FDOT's PCS database to identify candidate calibration sections so that the performance history reported in the database can be used for the planned calibration.

INITIAL SELECTION STAGE

From the PCS data, researchers initially identified segments where the pavement condition ratings (PCRs) have reached the critical value or have become deficient within the recent three years (2001 ~ 2004). Specifically, asphalt concrete (AC) pavements with PCRs of 6.6 or less and Portland cement concrete pavements with PCRs of 7.0 or less were first considered. Because the number of PCC segments is much less than the number of AC pavements, researchers used a higher critical PCR for PCC pavements to identify more candidate segments for this pavement type. In addition, researchers identified segments with no indications of unreported maintenance or rehabilitation treatments and/or erroneous rating values by examining the trends in the ratings provided in the PCS data base for a given segment. Since the M-E PDG program has different models for predicting the progression of cracking, rutting, and ride quality through a pavement's life cycle, a list of candidate segments was prepared for each of the three pavement condition indicators of cracking, rutting, and ride that FDOT reports in its PCS data base. The following criteria were used in identifying candidate segments for a given distress type:

- The distress rating has reached the critical value (6.6 for AC and 7.0 for PCC pavements).
- The length of the current cycle corresponding to the last rating reported in the data base is greater than 5 years.
- The minimum rating value for the current cycle is greater than 5 for observations before the current year.

- The range of the rating values for each distress type is greater than or equal to 2 for the current cycle.
- The rating values within the current cycle show a decreasing trend. Specifically, if the following nonlinear curve is fitted to the current cycle data, the estimates of β_1 should be greater than 0.01:

$$Distress\,rating \quad = \quad 20\left(1-\frac{1}{1+e^{-(\beta_0+\beta_1\,Age)}}\right)+error \qquad (3.1)$$

- The average residual sum of squares from fitting Eq. (3.1) to the condition survey data for a given PCS segment is less than 0.15.

Since each distress type has a different list of candidate segments, a summary of the candidate segments identified for each distress is given in Table 3.1. The two right most columns in the table indicate the cumulative length of candidate segments (in centerline miles) and the total number of segments without duplication for each subsystem and pavement type (note that a PCS segment can be used for calibrating more than one distress model).

Table 3.1. Summary of Candidate Sections from the Initial Stage.

Type	Subsystem	PCR		Cracks		Ride		Rutting		Unique Total	
		Length	#seg.	Length	#seg.	Length	#seg.	Length	#seg.	Length	#seg.
AC	Arterial 1	39.56	12	4.02	4	6.60	4	22.85	6	60.33	21
	Arterial 2	76.00	20	3.20	3	11.82	6	1.20	2	77.49	23
	Arterial 3	13.47	10	8.99	6	7.70	6	3.06	3	25.84	18
	Arterial 4	25.21	8	5.73	4	10.37	6	7.76	3	35.52	15
	Arterial 5	72.69	19	24.94	8	6.60	4	41.09	9	80.71	24
	Arterial 6	12.85	9	1.99	1	9.32	6	4.38	3	20.54	13
	Arterial 7	15.83	8	8.52	1	4.89	4	12.68	5	27.63	12
	Interstate	12.26	5	40.09	5	0.00	0	17.51	3	55.11	10
	Turnpike	16.06	6	0.00	0	0.00	0	6.40	2	22.47	8
	Subtotal	283.92	97	97.47	32	57.30	36	116.92	36	405.64	144
PCC	Arterial 2	4.17	2	0.00	0	5.13	3	0.00	0	5.13	3
	Arterial 5	1.86	1	0.00	0	1.86	1	0.00	0	1.86	1
	Arterial 6	1.28	1	2.55	2	0.00	0	0.00	0	2.55	2
	Arterial 7	0.00	0	0.25	1	0.00	0	0.00	0	0.25	1
	Interstate	2.14	1	2.06	3	8.01	3	0.00	0	12.22	7
	Turnpike	0	0	0	0	0	0	0	0	0	0
	Subtotal	9.44	5	4.86	6	15.00	7	0.00	0	22.01	14
Total		293.36	102	102.33	38	72.30	43	116.92	36	427.65	158

Since the M-E PDG program requires the axle load distribution for performance predictions, the proximity of a PCS segment to a WIM station was one criterion used by researchers to establish candidate segments for model calibration. The weigh-in-motion data can be used to determine the axle load distribution required by the program. Hence, researchers further screened the candidate segments to identify those that are close to a WIM station. Figure 3.1 shows the locations of WIM stations and candidate PCC segments. In this figure, the WIM station is identified by the WIM site number assigned by FDOT while the candidate PCC segment is identified by its roadway ID from the PCS database. The large printed numbers (1 to 7) denote the Florida Districts.

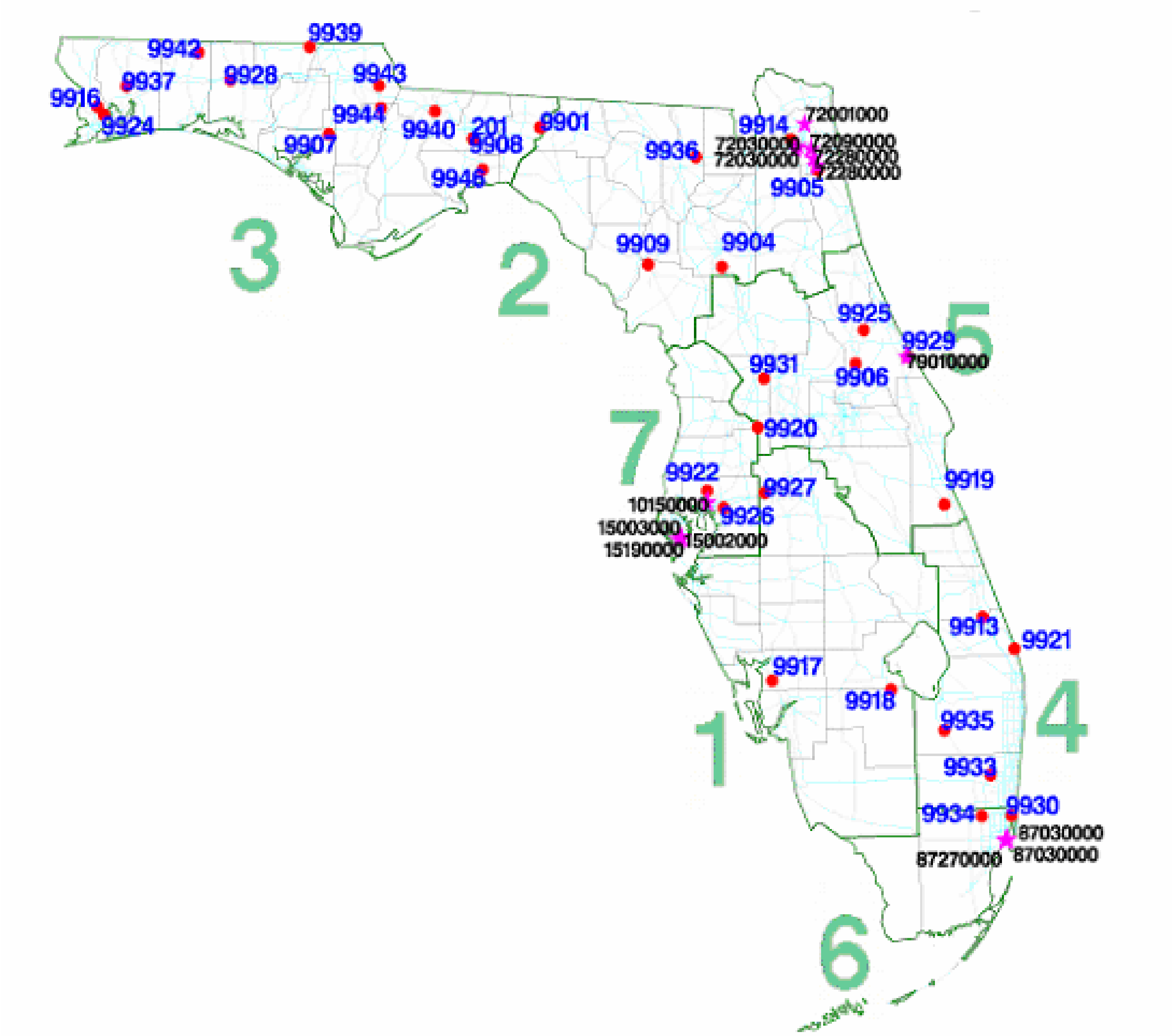


Figure 3.1. WIM Stations and Candidate PCC Segments (dots indicate WIM stations while stars denote mid-points of each candidate PCC segment).

For mapping purposes, researchers established the latitude-longitude (lat-long) coordinates utilizing the SAS road map data file (Roadnet2.SD2) provided by the FDOT project manager. Initially, the mid-mile post limits (MMP) of PCS segments were defined as follows:

$$\text{mid-mile post limit} = \tfrac{1}{2}\left(\text{ending mile post limit} + \text{beginning mile post limit}\right) \tag{3.2}$$

For each roadway ID, the lat-long coordinates of PCS segments were estimated to be proportional to those of the road map data. Because the road map data file has lat-long coordinates by mile post limits (MPs), the latitude and longitude coordinates of PCS segments can be interpolated. Consider a segment with MMP between MP_1 and MP_2, where MP_1 and MP_2 are MPs in the road map data file. Let Lat_1 and Lat_2 be the latitudes of MP_1 and MP_2, respectively. Then, the latitude and longitude coordinates of the mid-point of a PCS segment can be estimated as follows:

$$\begin{aligned} \text{Latitude}(^\circ) &= \text{Lat}_1 + \tfrac{\text{Lat}_2 - \text{Lat}_1}{\text{MP}_2 - \text{MP}_1}(\text{MMP-MP}_1) \\ \text{Longitude}(^\circ) &= \text{Long}_1 + \tfrac{\text{Long}_2 - \text{Long}_1}{\text{MP}_2 - \text{MP}_1}(\text{MMP-MP}_1) \end{aligned} \tag{3.3}$$

For the segment after the last MP in each roadway ID, researchers estimated the coordinates of the mid-mile post limit of the given PCS segment by extending the trend of the last two MPs.

To calculate the straight line distance between two locations based on lat-long coordinates, the coordinates were first converted from degrees to radians using the following equations:

$$\begin{aligned} \text{Latitude(rad)} &= \frac{\tan^{-1}(1)}{45}\text{Latitude}(^\circ) \\ \text{Longitude(rad)} &= \frac{\tan^{-1}(1)}{45}\text{Longitude}(^\circ) \end{aligned} \tag{3.4}$$

Then, if X_1 and Y_1 are the longitude and latitude, respectively, of a PCS segment in radians, and X_2 and Y_2 are the corresponding coordinates for a given WIM station, the *Great Circle Distance Formula* given by Eq. (3.5) can be used to calculate the distance in miles between two pairs of latitude/longitude values specified in radians:

$$D = 3949.99\cos^{-1}\left\{\sin Y_1 \sin Y_2 + \cos Y_1 \cos Y_2 \cos\left(X_1 - X_2\right)\right\} \tag{3.5}$$

Researchers obtained the above formula from the following web link: SAS technical support. For a candidate segment, researchers calculated all possible distances to the 36 WIM stations set up in Florida. For each candidate PCS segment, researchers then identified the WIM station closest to that segment based on the distances calculated from the lat-long coordinates.

Tables A1 and A2 in the appendix list all candidate segments for AC and PCC pavements, respectively. In these tables, the *SITE* and the *Distance of WIM* columns give the identification number and the distance of the nearest WIM station to the given candidate segment. The column *Road Condition* indicates the values of the pavement condition rating, cracks, ride, and rut (PCR, CR, RI, and RU) at the end of the life-cycle of a given segment. The mean residual sum of squares from fitting the non-linear model given by Eq. (3.1) to the PCS data for each candidate segment is given under the *Mean RSS* column. The column *Application* indicates the distress model(s) on which the given candidate segment can be used for model calibration. Note that a segment can be used for calibrating more than one distress model. The applicable distress models are identified by an *x* in the corresponding cells.

When the roadway ID of a candidate segment cannot be found in the road map data, the nearest WIM station and the distance to that station cannot be calculated. In this instance, the corresponding cells are filled in with *NAs* (not available). Researchers excluded these segments in the selection of PCS segments for model calibration. In addition, candidate segments that are more than 40 miles away from the nearest WIM station are screened out from this selection. These segments are highlighted in Tables A1 and A2.

Tables 3.2 and 3.3 summarize the candidate segments that remained after filtering out PCS segments (in the second round) where the WIM distance cannot be calculated, or where the nearest WIM station is more than 40 miles away from the given segment. Because few candidate PCC segments were found (relative to the number of AC segments), all candidate PCC pavements were selected in the pared down list of candidate segments established after considering the proximity to a WIM location. These PCC segments are also within 40 miles of a WIM station. In addition, candidate segments in the Interstate and Turnpike subsystems are within 40 miles of the corresponding WIM sites. Hence, all of these segments were selected. Figures 3.2, 3.3 and 3.4 identify the selected AC candidate segments in the Arterial, Interstate, and Turnpike subsystems, respectively. Since no PCC segments were screened out, the selected PCC candidate segments after the second round are as shown in Figure 3.1.

Table 3.2. Summary of Candidate AC Segments Considering Proximity to WIM Sites.

Subsystem	PCR		Cracks		Ride		Rutting		Overall	
	Length	#seg	Length	#seg	Length	#seg	Length	#seg	Length	#seg
Arterial 1	33.97	9	1.56	2	5.48	3	19.28	4	50.95	16
Arterial 2	75.56	19	3.2	3	11.39	5	1.2	2	77.05	22
Arterial 3	13.47	10	8.99	6	7.7	6	3.06	3	25.84	18
Arterial 4	25.21	8	5.73	4	10.37	6	7.76	3	35.52	15
Arterial 5	72.08	18	24.94	8	6.6	4	40.48	8	80.1	23
Arterial 6	8.52	7	1.99	1	9.32	6	4.38	3	16.22	11
Arterial 7	15.83	8	8.52	1	4.89	4	12.2	4	27.15	11
Interstate	12.26	5	40.09	5	0	0	17.51	3	55.11	10
Turnpike	16.06	6	0	0	0	0	6.4	2	22.47	8
Total	272.96	90	95.01	30	55.74	34	112.26	32	390.38	134

Table 3.3. Summary of PCC Candidate Segments Considering Proximity to WIM Sites.

Subsystem	PCR		Cracks		Ride		Overall	
	Length	#seg	Length	#seg	Length	#seg	Length	#seg
Arterial 2	4.17	2			5.13	3	5.13	3
Arterial 5	1.86	1			1.86	1	1.86	1
Arterial 6	1.28	1	2.55	2			2.55	2
Arterial 7			0.25	1			0.25	1
Interstate	2.14	1	2.06	3	8.01	3	12.22	7
Total	9.444	5	4.863	6	14.997	7	22.002	14

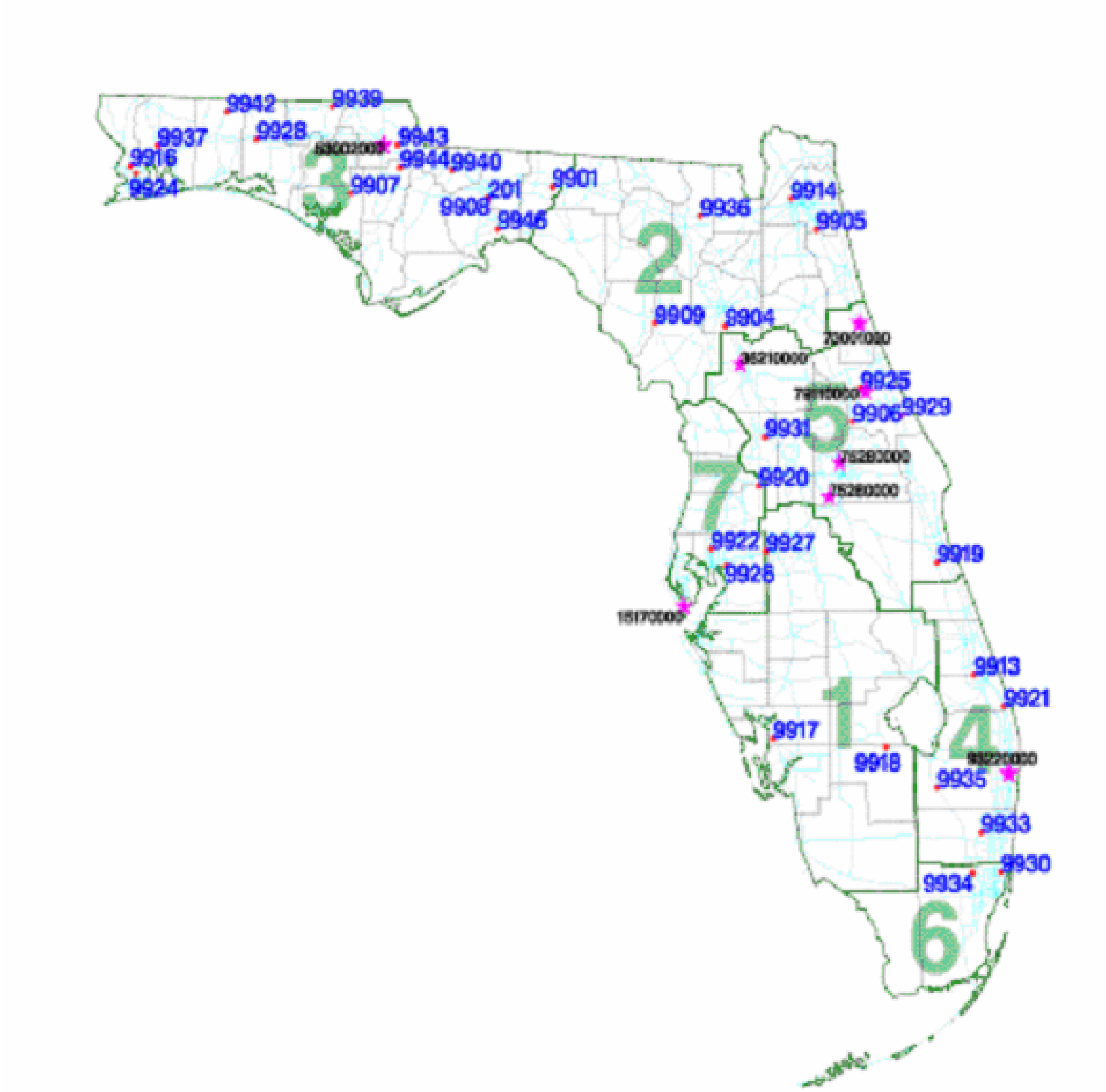


Figure 3.2. Candidate AC Segments in the Arterial Subsystems Considering Proximity of PCS Segments to WIM Stations (dots indicate WIM stations while stars denote candidate segments).

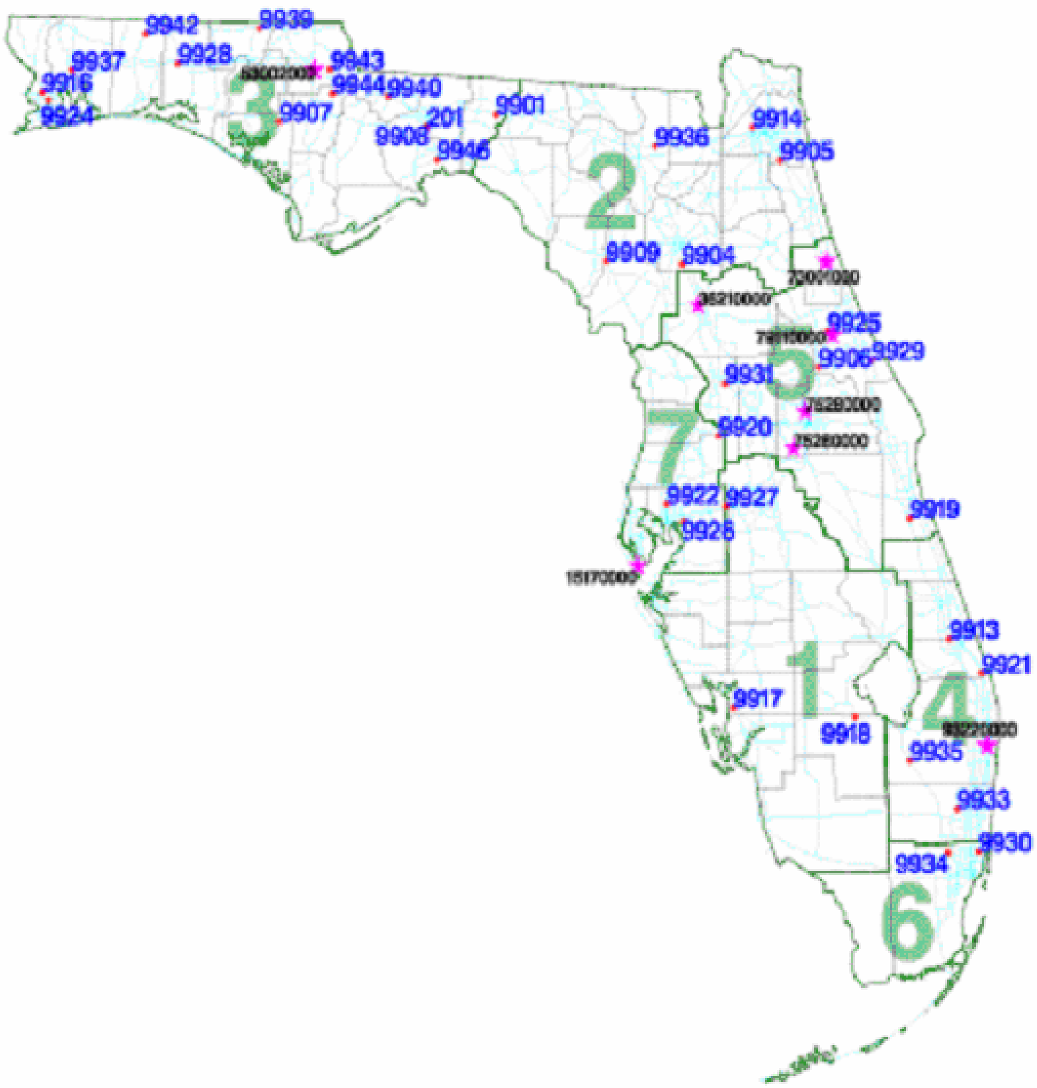


Figure 3.3. Candidate AC Segments in the Interstate Subsystem Considering Proximity of PCS Segments to WIM Stations (dots indicate WIM stations while stars denote candidate segments).

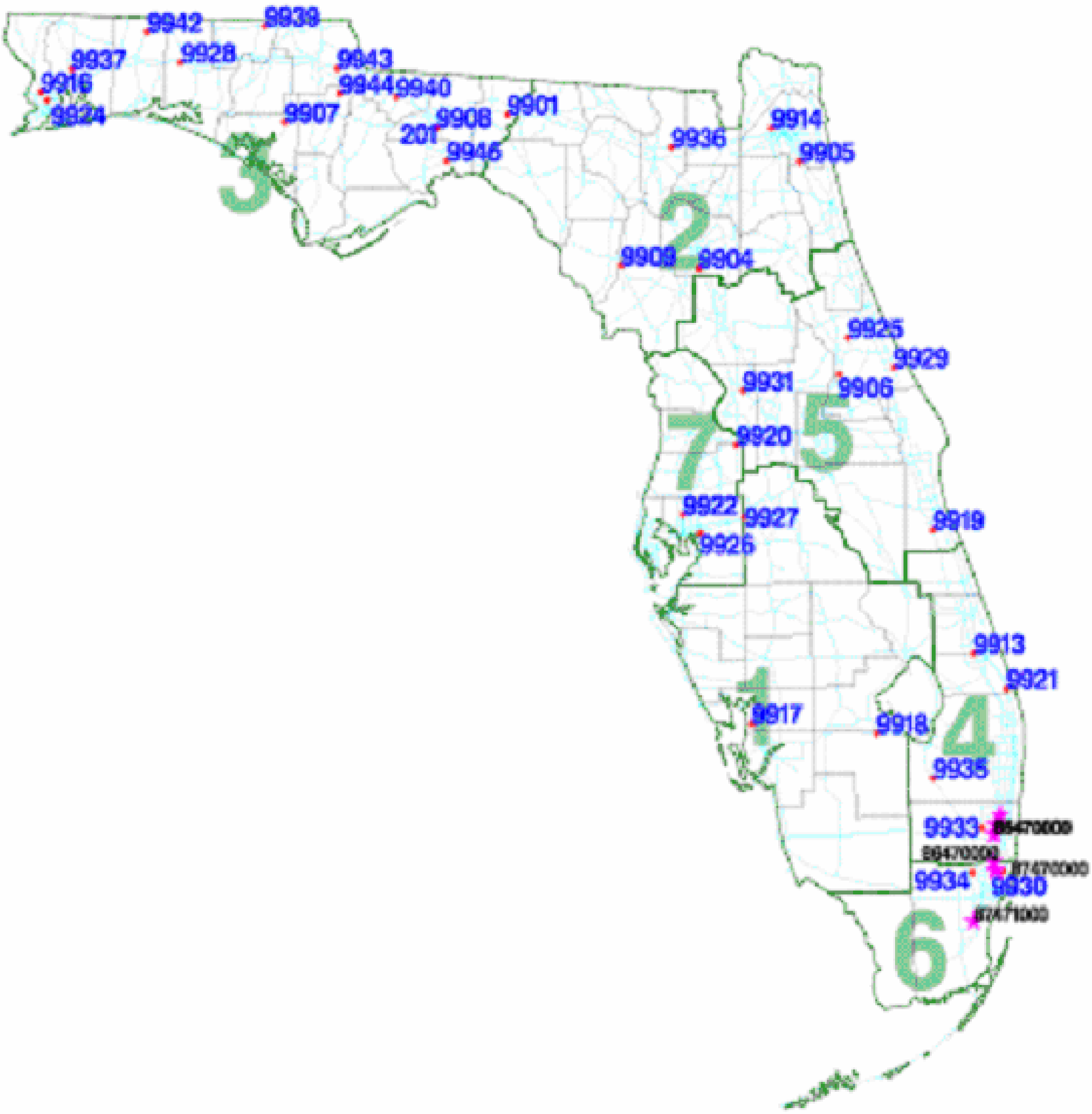


Figure 3.4. Candidate AC Segments in the Turnpike Subsystem Considering Proximity of PCS Segments to WIM Stations (dots indicate WIM stations while stars denote candidate segments).

For model calibrations, it is also important to consider PCS segments that exhibit long service lives. Calibrations that include these pavements can identify factors that are important in terms of designing longer-lasting pavements or generating cost-effective pavement designs. In addition, the presence of outliers in the condition survey ratings can influence any model calibration in the short term, because of the small number of observations. However, in the long term, the effects of outliers can be offset by many other normal observations. Thus, researchers identified PCS segments that showed service lives of more than 20 years before reaching the critical PCR values of 6.6 for AC and 7.0 for PCC pavements in the recent 3 years. Table 3.4 and Figure 3.5 summarize the PCS segments identified based on this criterion. In addition, Tables A3 and A4 in the appendix list AC and PCC segments, respectively, that exhibited service lives of more than 20 years.

Discussions with the project manager raised an additional criterion that researchers considered. Specifically, using data from a WIM site located along a roadway with the same functional classification as a given calibration section would be an appropriate approach for estimating the axle load distribution on the section. Thus, researchers applied this criterion to the list of candidate PCS segments previously identified. Table 3.5 shows the revised list of candidate segments that resulted from this screening. Three of the candidate sections (highlighted in the table) overlap with test sections used in a previous FDOT project conducted by Ping, Wang and Yang (2000) of Florida State University (FSU). For each segment, Table 3.5 identifies the closest WIM station located along a roadway having the same functional class as the candidate segment. The straight line distance between the segment and the WIM site is also given in Table 3.5.

Table 3.5 identifies the specific distress for which a given segment may be used for calibration. In establishing this preliminary list of candidate segments, researchers gave higher consideration to PCS segments that became deficient by cracking inasmuch as this is the predominant distress observed on Florida pavements. From this list, 21 test sections (15 for AC and 6 for PCC) were selected as shown in the last column.

Table 3.4. Summary of Candidate PCS Segments with Service Lives > 20 Years.

Subsystem	AC		PCC		Total	
	length	#seg	length	#seg	length	#seg
Arterial 1	53.56	14	8.1	5	61.64	19
Arterial 2	50.05	20	11.1	6	61.12	26
Arterial 3	3.44	4	1.8	1	5.27	5
Arterial 4	24.66	17			24.66	17
Arterial 5	46.72	17	8.7	5	55.45	22
Arterial 6	12.60	8	0.5	1	13.09	9
Arterial 7	2.60	4	0.3	1	2.85	5
Interstate			9.0	13	9.02	13
Turnpike	0.32	1			0.32	1
Total	193.94	85	39.5	32	233.41	117

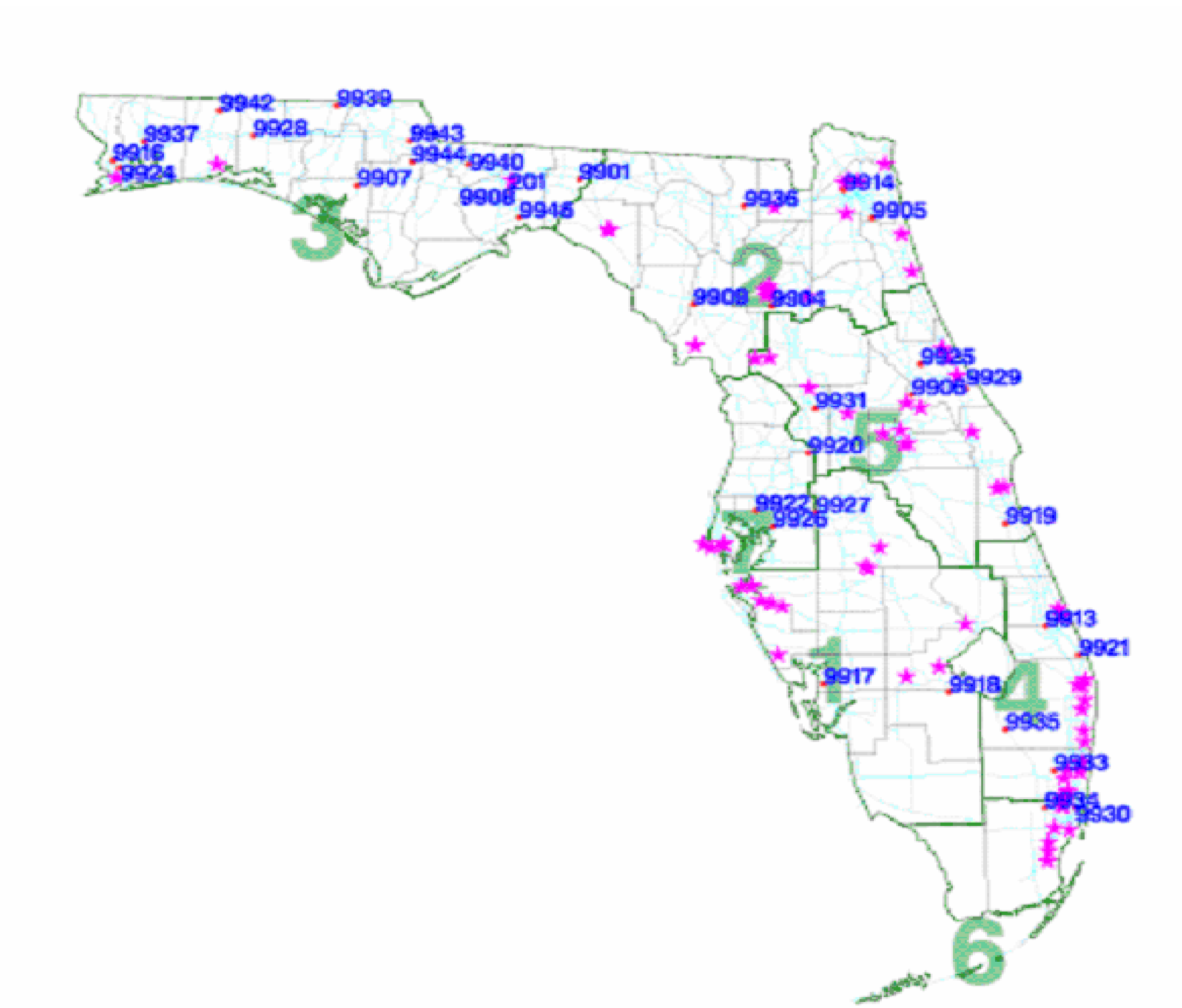


Figure 3.5. Candidate Segments with Service Lives > 20 Years (WIM stations identified by dots and corresponding WIM site numbers while stars denote long-life segments).

Table 3.5. Candidate Test Sections for Model Calibration after the Initial Stage.

RDWYID	Beginning Milepost	Ending Milepost	RDWYSIDE	System	District	New Year	Year	Age	Functional Class	Average Daily Traffic	ESALS per day	WIM Site	Straight Line Distance from WIM site	Pavement Type	Calibration for				Selected Calib. Sections
															PCR	Cracks	Ride	Rutting	
87060000	14.05	14.87	L	1	6	1997	2004	8	NON-FREEWAY URBAN	33500	535	9930	2.37	AC	x		x		
87090000	0.79	3.69	R	1	6	1986	2004	19	NON-FREEWAY RURAL	17100	408	9934	2.74	AC			x		
50010000	16.48	18.57	R	1	3	1996	2004	9	NON-FREEWAY RURAL	10933	297	9940	3.34	AC	x	x			✓
87080000	0.69	2.68	R	1	6	1992	2004	13	NON-FREEWAY URBAN	37500	611	9930	4.44	AC		x	x		✓
87080001	2.29	3.08	C	1	6	1997	2004	8	NON-FREEWAY URBAN	16500	622	9930	5.66	AC	x		x		
48003000	6.12	6.75	R	1	3	1987	2004	18	NON-FREEWAY URBAN	23500	546	9916	6.46	AC	x		x		
86100000	1.53	2.73	R	1	4	1981	2004	24	NON-FREEWAY URBAN	48000	983	9930	7.02	AC	x		x		
86190000	2.33	3.67	R	1	4	1999	2004	6	NON-FREEWAY URBAN	30750	238	9930	10.01	AC		x			✓
61010000	13.33	16.33	C	1	3	1986	2001	16	NON-FREEWAY RURAL	6200	300	9939	12.96	AC		x			✓
58010000	9.32	10.75	L	1	3	1989	2004	16	NON-FREEWAY URBAN	33000		9916	13.54	AC	x	x			✓
58010000	11.05	11.68	L	1	3	1989	2004	16	NON-FREEWAY URBAN	21500	509	9916	14.72	AC	x	x	x		✓
58060000	20.72	21.80	C	1	3	1997	2004	8	NON-FREEWAY RURAL	2400	81	9937	18.96	AC	x	x			✓
79270000	1.62	2.39	L	1	5	1997	2004	8	NON-FREEWAY URBAN	23500	367	9929	21.67	AC		x			✓
86020000	7.91	9.00	R	1	4	1983	2004	22	NON-FREEWAY URBAN	45250	257	9930	21.93	AC		x		x	✓
48010000	2.15	2.90	R	1	3	1987	2002	16	NON-FREEWAY RURAL	3621	190	9937	23.45	AC	x	x	x		✓
26060000	20.53	25.34	R	1	2	1995	2004	10	NON-FREEWAY RURAL	8600	1059	9909	47.55	AC		x			4
89010000	18.04	19.58	R	1	4	1987	2004	18	NON-FREEWAY URBAN	48750	222	9930	88.30	AC			x		2A
71020000	1.78	6.56	R	1	2	1996	2004	9	NON-FREEWAY URBAN	20968	454	9929	92.04	AC		x			5
87170000	3.70	5.23	L	1	6	1987	2004	18	NON-FREEWAY URBAN	39250	471	9930	1.34	AC	x				
18010000	19.48	21.60	C	1	5	1986	2002	17	NON-FREEWAY RURAL	7600	295	9931	3.07	AC	x			x	
87030000	24.96	25.38	R	1	6	1989	2004	16	NON-FREEWAY URBAN	37000		9930	4.10	AC	x				
48020000	7.87	9.65	L	1	3	1987	2004	18	NON-FREEWAY URBAN	27750	766	9916	5.88	AC			x		
86470000	8.51	15.16	L	5	4	1988	2001	14	FREEWAY	67207	1557	9933	6.10	AC	x				
86470000	8.51	15.16	R	5	4	1988	2001	14	FREEWAY	67207	1557	9933	6.10	AC	x				
86470000	15.16	16.95	L	5	4	1990	2001	12	FREEWAY	58199	1140	9933	6.26	AC	x				
48003000	6.12	6.75	L	1	3	1988	2004	17	NON-FREEWAY URBAN	23500	546	9916	6.46	AC	x				
79070000	26.97	29.29	R	1	5	1981	2004	24	NON-FREEWAY URBAN	18200	730	9929	7.54	AC	x		x		
93200000	0.00	1.37	C	1	4	1994	2004	11	NON-FREEWAY RURAL	20700	712	9935	28.36	AC		x			
77030000	5.09	6.04	R	1	5	1994	2004	11	NON-FREEWAY URBAN	14750	1223	9929	28.56	AC	x	x			✓
10060000	8.91	17.43	R	1	7	1994	2004	11	NON-FREEWAY URBAN	104838	3816	9927	30.11	AC	x	x			✓

RDWYID	Beginning Milepost	Ending Milepost	RDWYSIDE	System	District	New Year	Year	Age	Functional Class	Average Daily Traffic	ESALS per day	WIM Site	Straight Line Distance from WIM site	Pavement Type	Calibration for				Selected Calib. Sections
															PCR	Cracks	Ride	Rutting	
75280000	1.00	8.84	L	4	5	1992	2001	10	FREEWAY	52000	584	9920	35.02	AC	x	x			✓
75003000	2.05	5.00	R	1	5	1995	2004	10	NON-FREEWAY URBAN	52000	584	9929	39.24	AC	x	x			✓
75003000	2.05	4.92	L	1	5	1995	2004	10	NON-FREEWAY URBAN	82333	1334	9929	39.27	AC	x	x			✓
72280000	6.08	7.48	L	4	2	1988	2004	17	FREEWAY	80167	2852	9905	2.24	PCC			x		
72280000	7.48	13.10	L	4	2	1988	2004	17	FREEWAY	28250	1957	9905	5.62	PCC			x		
72001000	34.52	35.51	R	4	2	1983	2004	22	FREEWAY	33000	1622	9914	9.65	PCC			x		
72090000	8.23	11.14	R	1	2	1983	2004	22	FREEWAY	19100	719	9905	10.19	PCC	x		x		
87030000	8.83	10.10	L	1	6	1996	2004	9	NON-FREEWAY URBAN	19100	719	9930	11.01	PCC	x	x			✓
87030000	8.83	10.10	R	1	6	1996	2004	9	NON-FREEWAY URBAN	10501	95	9930	11.01	PCC		x			✓
79010000	9.60	11.46	L	1	5	1990	2004	15	NON-FREEWAY RURAL	20900		9925	23.76	PCC	x		x		
15002000	0.00	0.16	R	4	7	1983	2001	19	FREEWAY	19967	105	9926	23.93	PCC		x			✓
15003000	0.37	1.29	R	4	7	1983	2004	22	FREEWAY	61400	689	9926	24.03	PCC		x			✓
15190000	2.29	4.43	R	4	7	1983	2004	22	FREEWAY	39250	435	9926	25.37	PCC	x				
10150000	12.59	12.84	L	1	7	1983	2004	22	NON-FREEWAY URBAN	65500	2915	9927	27.56	PCC		x			✓
87270000	0.00	0.98	R	4	6	1983	2001	19	FREEWAY	50000	294	9933	30.13	PCC		x			✓
72030000	7.46	8.71	R	1	2	1983	2004	22	NON-FREEWAY URBAN	49000	662	9929	107.54	PCC	x		x		
72030000	9.43	10.39	R	1	2	1983	2004	22	NON-FREEWAY URBAN			9929	108.22	PCC			x		

FINAL SELECTION STAGE

After identifying candidate test sections, researchers selected calibration sections from the list generated during the initial selection stage based on discussions with FDOT engineers and considering the available funds on this project. Table 3.6 shows the list of calibration sections on which field tests to characterize material properties were conducted during the project. Researchers note that as the field tests progressed, some sections were added and others replaced due to the timing of resurfacing projects. A total of 31 calibration sections, consisting of 15 flexible and 16 rigid pavement sections were established.

Table 3.6. Final List of Calibration Sections.

County	District	Roadway ID	Highway	Type	Section Limits (mile)	Age (years)
Charlotte	1	10100000	US 41 / SR45	PCC	4.98 ~ 0.49	20
Polk	1	16250000	SR 37	HMAC	4.616 ~ 7.38	19
Polk	1	16003001	SR 563	HMAC	8.484 ~ 10.0	14
Polk	1	16100000	US 92	PCC	0.46~1.74	20
Alachua	2	26005000	SR 222	HMAC	10.691 ~ 7.954	16
Bradford	2	28040000	SR 18	HMAC	0.0 ~ 5.509	14
St. Johns	2	78020000	US 1	PCC	0.7 ~ 0.02	19
Gadsden	3	50010000	US 90	HMAC	16.48 ~ 18.57	10
Gadsden	3	123811 (LTPP section ID)	I 10 west	PCC	Beg. MP 187.1 (500-ft section)	31
Santa Rosa	3	58060000	SR 89	HMAC	21.80 ~ 20.72	17
Broward	4	86190000	SR 823	HMAC	2.33 ~ 3.67	7
Palm Beach	4	93100000	SR25/US27	HMAC	11.904 ~ 12.617	11
Palm Beach	4	93310000	SR 710	HMAC	17.796 ~ 12.215	18
Lake	5	11020000	SR 33	PCC	14.03 ~ 14.12	16
Seminole	5	77040000	SR 46	HMAC	5.808 ~ 11.046	13
Volusia	5	79270000	SR 483	HMAC	2.39 ~ 1.62	9
Monroe	6	90060000	US 1	HMAC	13.032 ~ 16.384	17
Dade	6	87060000	A1A	HMAC	2.715 ~ 0.872	12
Dade	6	87030000	SR 5	PCC	8.83 ~ 10.0	10
Dade	6	87030000	SR 5	PCC	10.0 ~ 8.83	10
Dade	6	87270000	9/9A	PCC	0.0 ~ 0.98	20
Dade	6	87061000	SR 886	PCC	0.21 ~ 0.0	14
Hillsborough	7	10060000	US 41	HMAC	8.91 ~ 17.43	12
Hillsborough	7	10160000	SR 597	HMAC	0.0 ~ 6.773	16
Hillsborough	7	10250001	SR 585 22nd St	PCC	0.0 ~ 1.102	5
Hillsborough	7	10075000	I 75	PCC	19.0 ~ 20.4	19
Hillsborough	7	10075000	I 75	PCC	23.4 ~ 24.69	20
Pinellas	7	15003000	I 175	PCC	0.37 ~ 1.29	23
Pinellas	7	15003000	I 175	PCC	1.29 ~ 0.37	23
Pinellas	7	15190000	I 275	PCC	2.287 ~ 4.44	23
Pinellas	7	15190000	I 275	PCC	4.44 ~ 2.287	23

CHAPTER IV. CHARACTERIZATION OF CLIMATIC AND SOIL VARIATIONS ACROSS FLORIDA

Climatic factors and the properties of soils on which pavements are built affect pavement design because of the influence of these variables on predicted performance. For the purpose of calibrating the M-E pavement design guide models, it is necessary (in the researchers' opinion) to characterize the variation of climatic and soil conditions in Florida. This task was accomplished by reviewing and collecting available data from weather stations in Florida, and from county soil survey reports published by the Natural Resources Conservation Service of the U.S. Department of Agriculture.

Researchers assembled a database of climatic and soil variables and used cluster analysis to group Florida counties into several regions. Climatic data collected covered air temperatures, precipitation, relative humidity and Thornthwaite moisture index (TMI). The Thornthwaite moisture index is calculated by a water balance procedure which involves:

- determination of monthly potential evapotranspiration,
- allocation of available water to storage, deficit, and runoff on a monthly basis, and
- summation of monthly runoff moisture depth, deficit moisture depth, and evapotranspiration to obtain annual values.

The index was proposed by Thornthwaite as a method for classifying climatic conditions. In this index, potential evapotranspiration (PET) represents the water quantity that soil would lose due to surface evaporation and to plant transpiration in an environment where continuous soil water storage exists. When PET is exactly balanced by precipitation over the year and water is available, there is neither a deficit nor surplus of water and the Thornthwaite moisture index is zero. When precipitations are lower than the PET, the index is negative and the climate is dry. When precipitations are higher than the PET, the index is positive and the climate gets humid. The TMI is given by (Lytton et al., 2004):

$$TMI = \frac{100R - 60DEF}{E_p} \tag{4.1}$$

where,

R	=	runoff moisture depth,
DEF	=	deficit moisture depth, and
E_p	=	evapotranspiration.

Previous research has found that in arid climates, the water table will dominate the moisture conditions in the subgrade if it exists within a depth of 30 ft of the pavement surface. Where the water table is below 30 ft, moisture movement will largely be controlled by unsaturated flow theory. For these conditions, the TMI may be used to predict the equilibrium soil suction value at the bottom node of the pavement for the purpose of estimating the initial soil suction profile that is an input to the enhanced integrated climatic model (EICM) included with the M-E pavement design guide system. The initial suction profile affects the predicted moisture variations in the pavement layers from EICM. The following equation may be used to predict the equilibrium soil suction value under unsaturated conditions (Lytton et al., 2004):

$$U_e \quad = \quad 3.5633 e^{-0.0051 TMI} \tag{4.2}$$

where U_e is the equilibrium boundary suction at the bottom node.

Researchers also reviewed published county soil surveys and identified the predominant soil types (based on volume) in the various Florida counties. Researchers used this information to establish a representative soil-water characteristic curve for each county. The soil-water characteristic curve defines the relationship between soil suction and soil moisture content as expressed in the following equation proposed by Fredlund and Xing:

$$\theta_w = C(h) \times \left[\frac{\theta_{sat}}{\left[\ln \left[\exp(1) + \left(\frac{h}{a_f} \right)^{b_f} \right] \right]^{c_f}} \right] \tag{4.3}$$

where,

$$C(h) = \left[1 - \frac{\ln(1 + \frac{h}{h_r})}{\ln\left(1 + \frac{1.45 \times 10^5}{h_r}\right)} \right] \tag{4.4}$$

θ_w	=	volumetric water content,
θ_{sat}	=	volumetric saturated water content,
h	=	soil suction in psi, and
a_f, b_f, c_f, & h_r	=	model parameters.

The soil-water characteristic curve is an input to the enhanced integrated climatic effects model that is part of the M-E pavement design guide program. This relationship is necessary to predict the mechanical behavior of soils, i.e., volume change and shear strength behavior.

CLIMATIC REGIONS

To establish the variation of climatic conditions, researchers used the following factors to characterize the climate of each county:

- mean air temperature,
- mean precipitation,
- daily temperature drop,
- mean relative humidity, and
- Thornthwaite moisture index.

Thirty-year averages for these variables were determined for each county. The authors note that some counties did not have weather station data to compute these averages. For these counties, they estimated the missing values by interpolation using corresponding data from neighboring counties. The cluster analysis was then performed based on the thirty-year averages of the climatic variables identified in the preceding. Figures 4.1 to 4.5 summarize these averages for each of the five climatic variables considered.

Two counties with similar values of the five climatic variables can be classified into the same climatic region or alternatively, into two distinct regions if the counties are dissimilar. The cluster analysis performed by researchers may be explained by defining the dissimilarity between two counties as:

$$D_{ij} = \sum_{k=1}^{5} \left(X_{ik} - X_{jk} \right)^2 , \quad i \neq j \tag{4.4}$$

where X_{ik} and X_{jk} are values of the climatic variable k for counties i and j, respectively. The clustering may be performed beginning with one cluster (representing the entire state of Florida) and progressing to n clusters, which in the limit will equal the number of Florida counties (note that researchers characterized the climatic conditions by county thus establishing it as the basic unit for the cluster analysis). This approach is called the top-down method. Alternatively, the clustering may begin with n clusters that are systematically reduced to fewer clusters by grouping similar counties (bottom-up method). To establish Florida climatic regions, researchers implemented this latter method. Hence, at the first step,

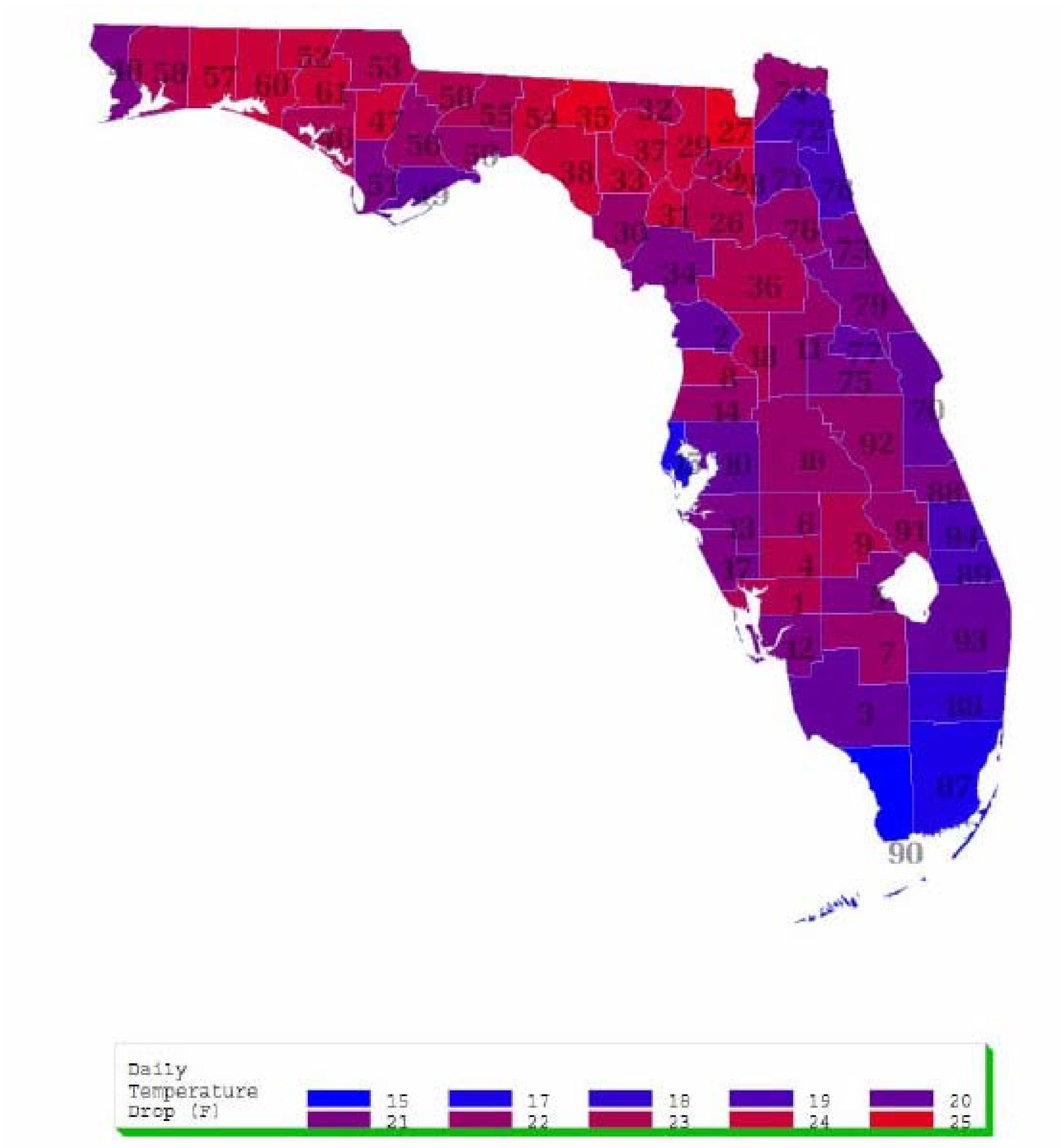


Figure 4.1. Variation in Daily Temperature Drops (°F) Across Florida Counties.

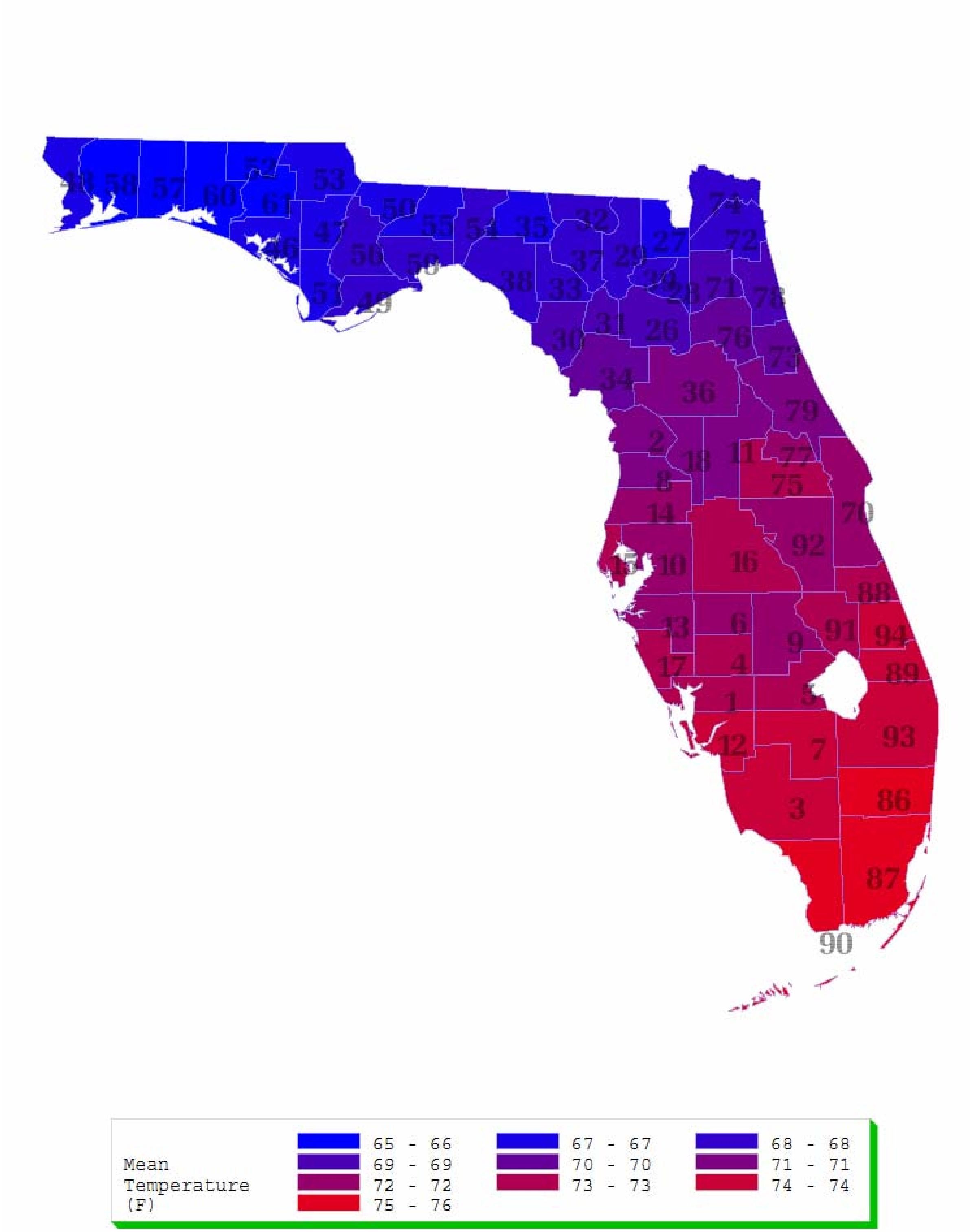


Figure 4.2. Variation in Mean Air Temperatures (°F) Across Florida Counties.

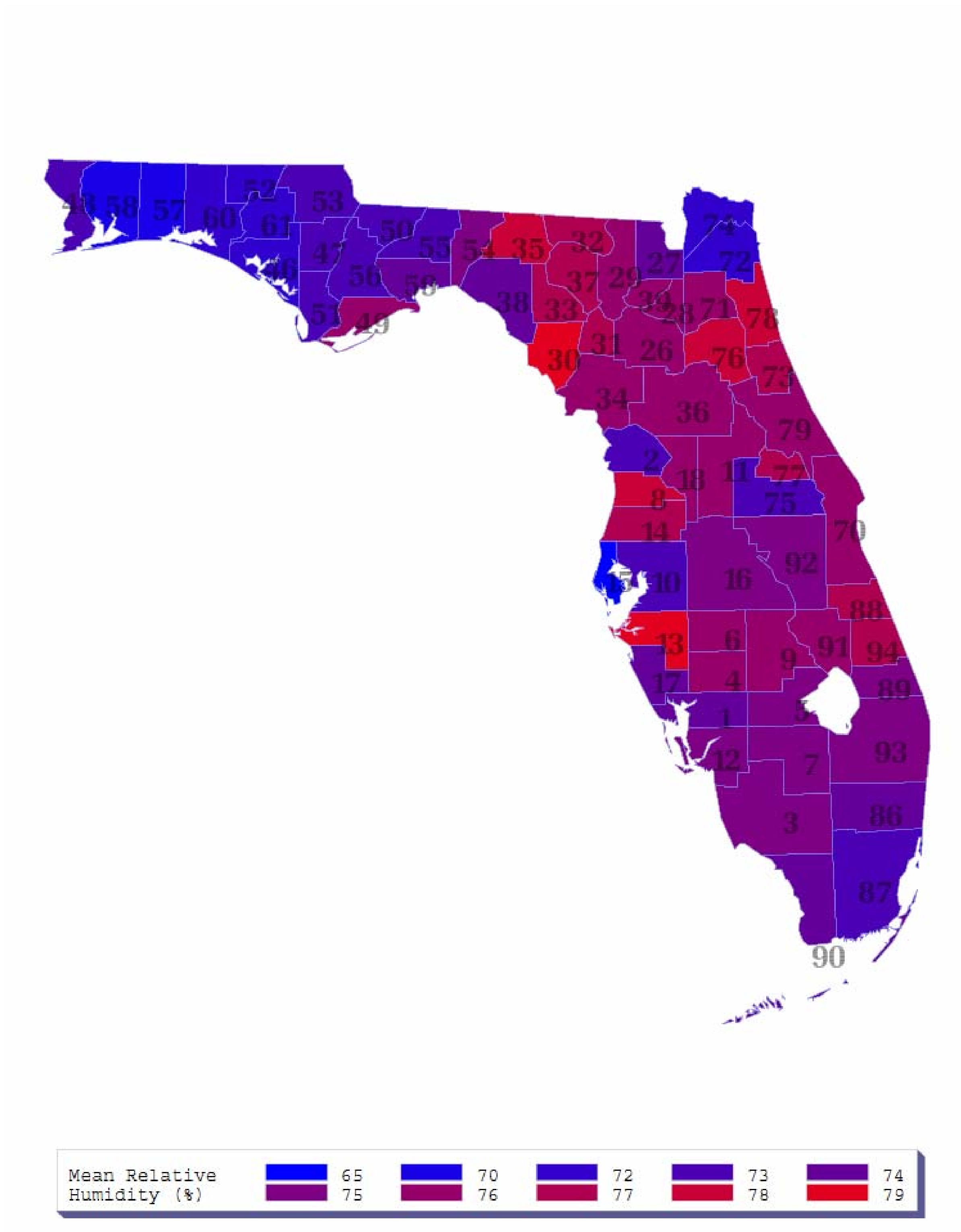


Figure 4.3. Variation in Mean Relative Humidities Across Florida Counties.

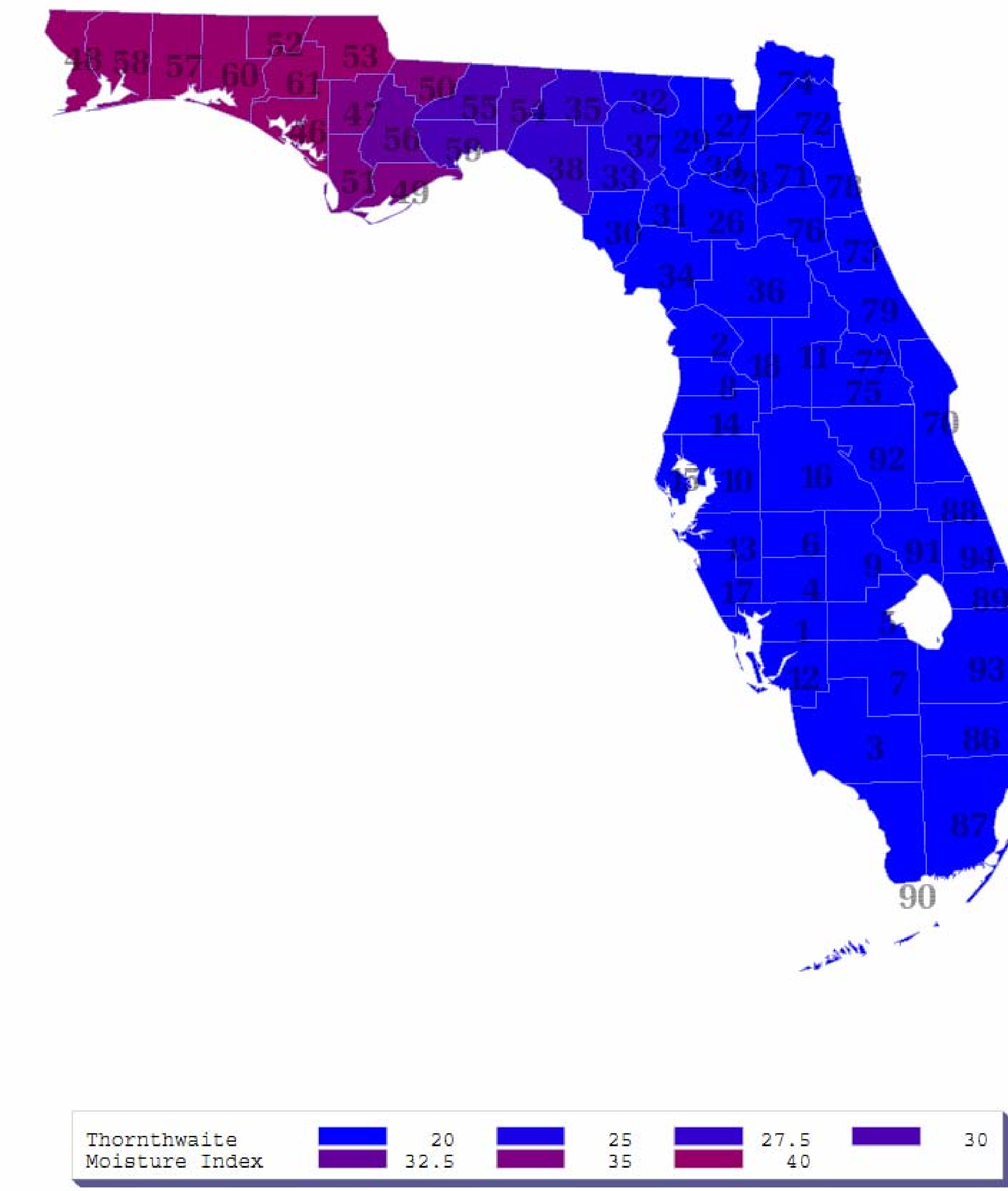


Figure 4.4. Variation in Thornthwaite Moisture Indices Across Florida Counties.

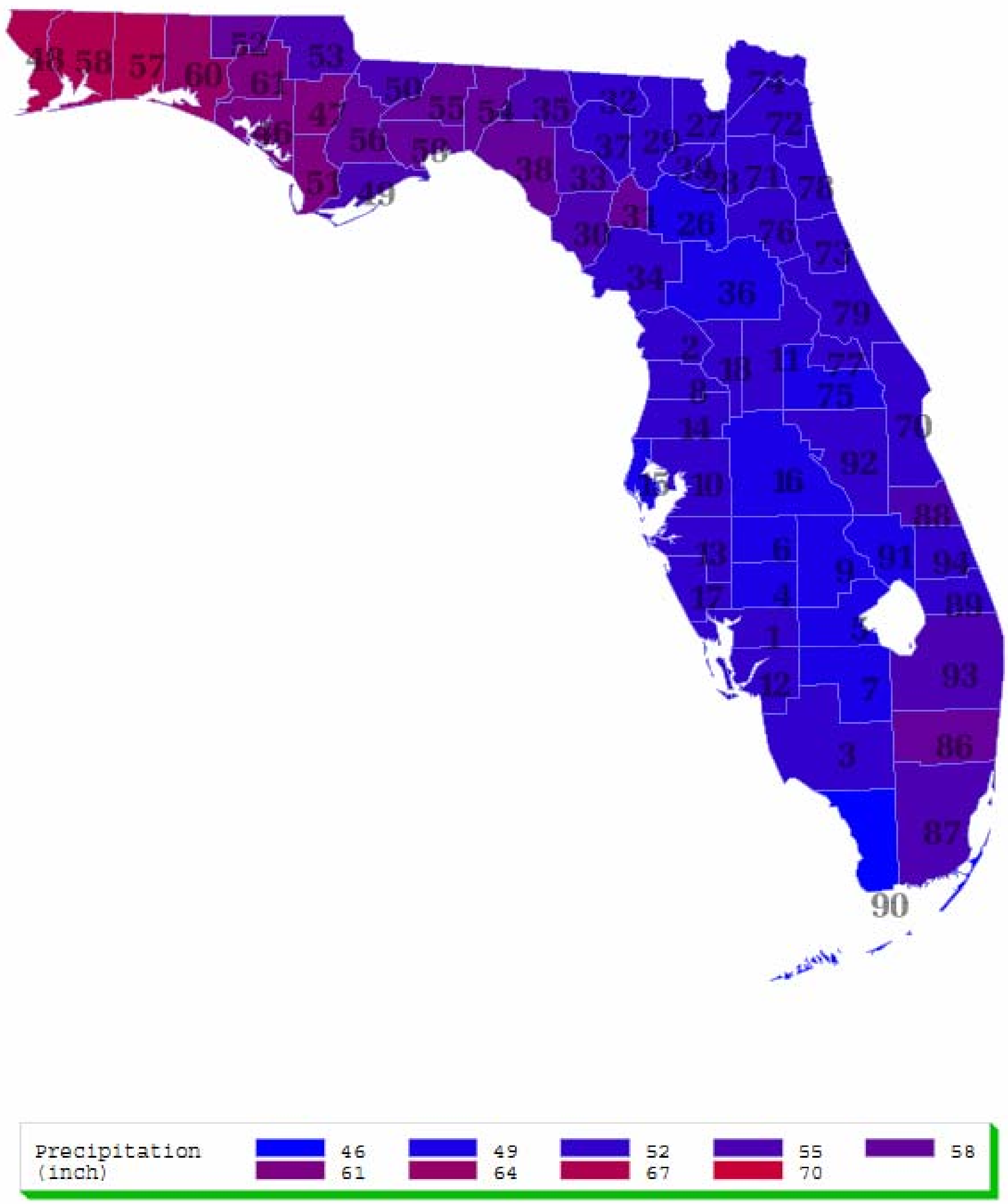


Figure 4.5. Variation in Mean Precipitations Across Florida Counties.

two closest counties that have the smallest dissimilarity among all possible pairs of counties are assigned in the same group. At the next step, the dissimilarity between a group and other counties is defined by the average of dissimilarities between each county of the group and the other county. The next group will then consist of the two closest counties, or one county and one group. After some steps, the dissimilarity between groups is calculated by the average of dissimilarities of all possible pairs of counties which are from different groups. Hence, two counties, one county and one group, or two groups are classified in a group. Figure 4.6 displays a diagram explaining the procedure. This procedure is called the average method of clustering. The average method tends to minimize the variance of climatic variables in each group.

To decide on the appropriate number of climatic regions for characterizing climatic conditions in Florida, let X_{ijk} be the *i*th climatic variable for the *k*th county classified in the *j*th cluster. Then, for a given number of clusters J_c, and denoting K_j as the number of counties in the *j*th cluster, the total number of counties in Florida is $\sum_{j=1}^{J_c} K_j = 67$. Hence, the mean square error (MSE) indicating the variability of the climatic conditions within a given cluster is determined as follows:

$$MSE = \sum_{i=1}^{5} \sum_{j=1}^{J_c} \sum_{k=1}^{K_j} \left(X_{ijk} - \bar{X}_{ij.} \right)^2 \Big/ 5\left(67 - J_c\right) \tag{4.5}$$

Researchers examined the MSE as the number of clusters is increased, where climatic variables are standardized by corresponding means and standard deviations. Starting with one cluster (J_c =1), researchers made a series of statistical hypothesis tests (partial F-tests) to decide whether the number of clusters needs to be increased. The partial F-test resulted in four clusters at a 5% level of significance. Figure 4.7 shows the variation in the mean square errors from the cluster analysis. This chart is referred to as the ”scree” plot (the plot looks like the side of a mountain, and “scree” refers to the debris fallen from a mountain and lying at its base). When four clusters are considered, increasing the number of clusters to five does not contribute significantly to the reduction of the MSE. Hence, the graphical method shows the same result.

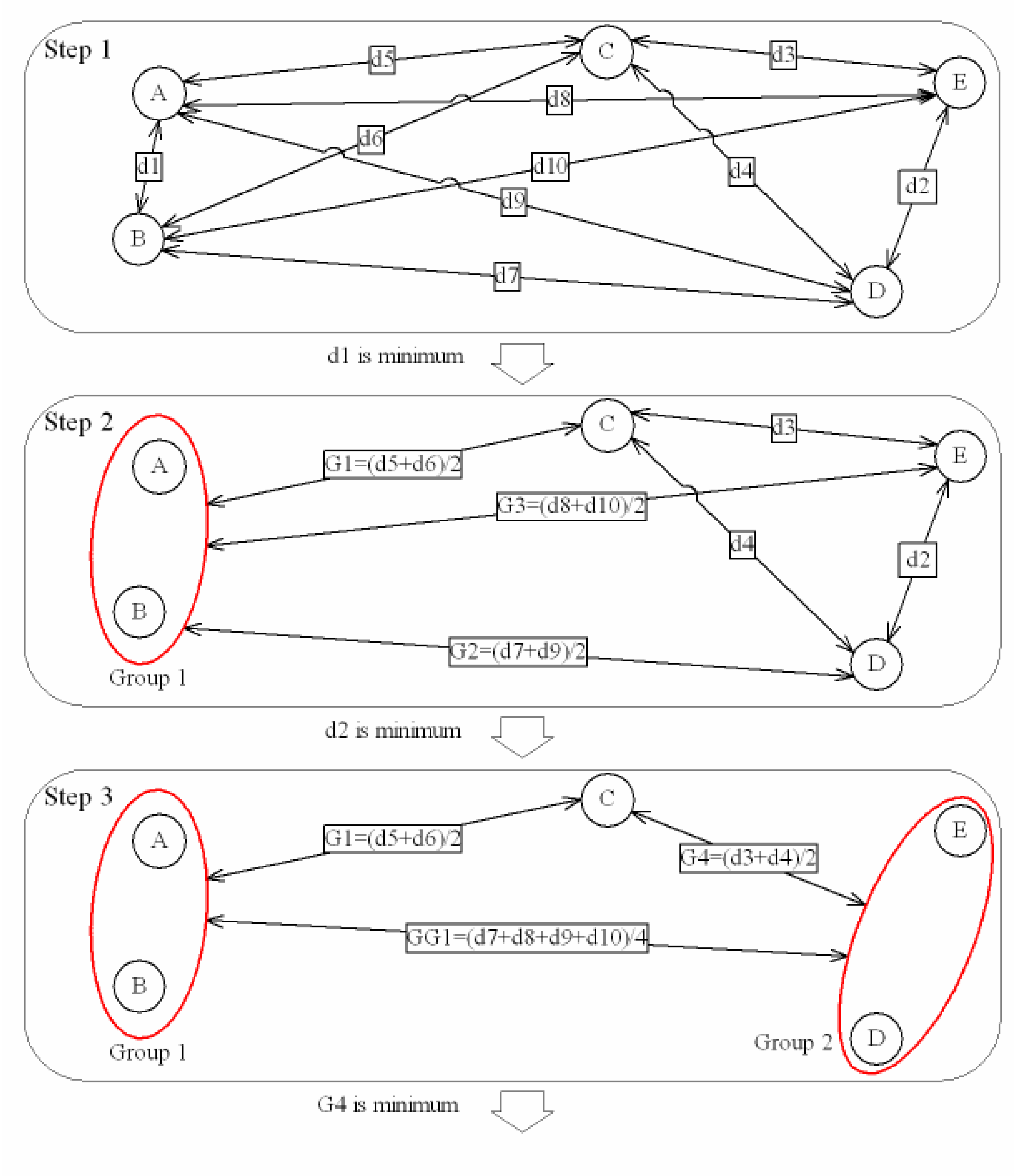


Figure 4.6. Diagram Illustrating the Cluster Analysis.

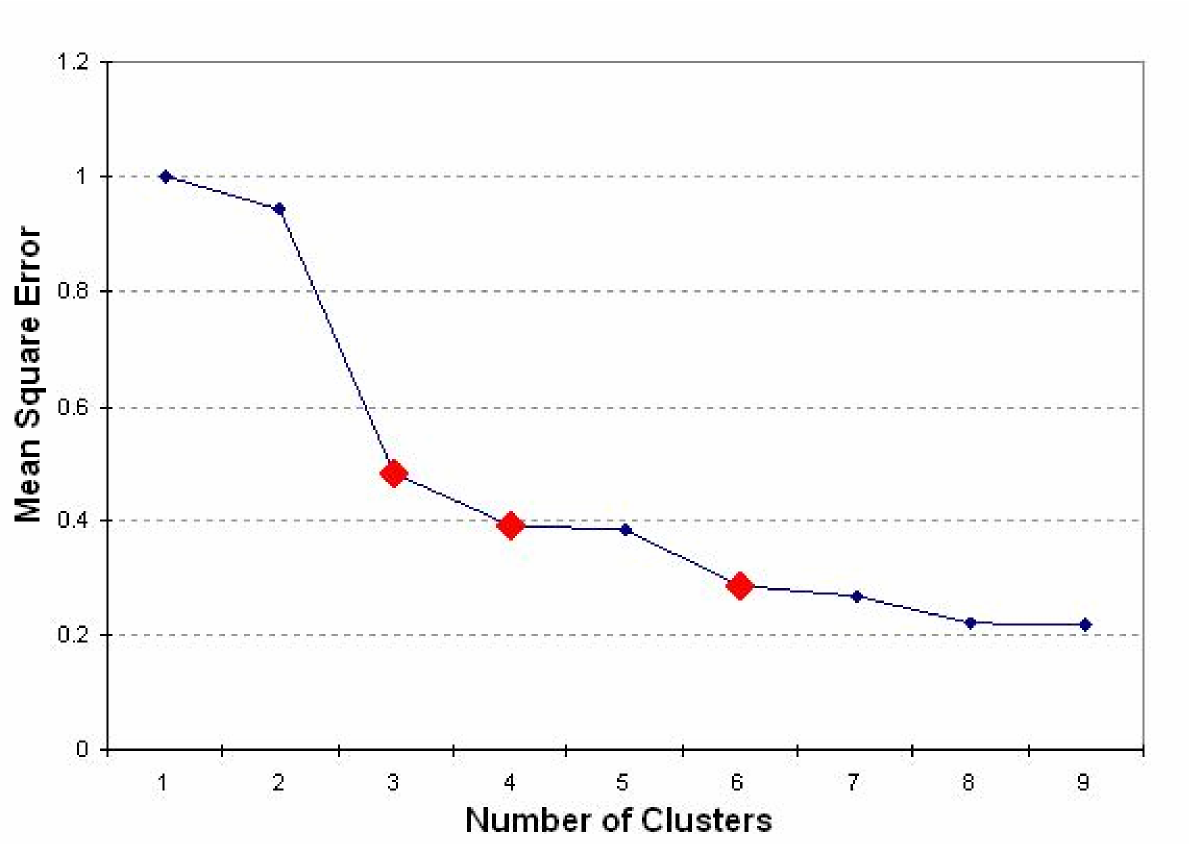


Figure 4.7. Scree Plot to Determine Appropriate Number of Climatic Regions.

Researchers also looked at other alternatives, specifically, three clusters and six clusters. Figures 4.8 to 4.10 show how the state would be subdivided into climatic regions when three, four, and six clusters are used, respectively. It is observed that the subdivision of the state into three regions reflects the effect of the Thornthwaite Moisture Index (Figure 4.4). Also, when six clusters are used, two are formed with only one county per cluster (Figure 4.10). Researchers are of the opinion that four clusters adequately capture the variation in climatic conditions across Florida and represent a good compromise between reducing the MSE and keeping the number of clusters small for simplicity.

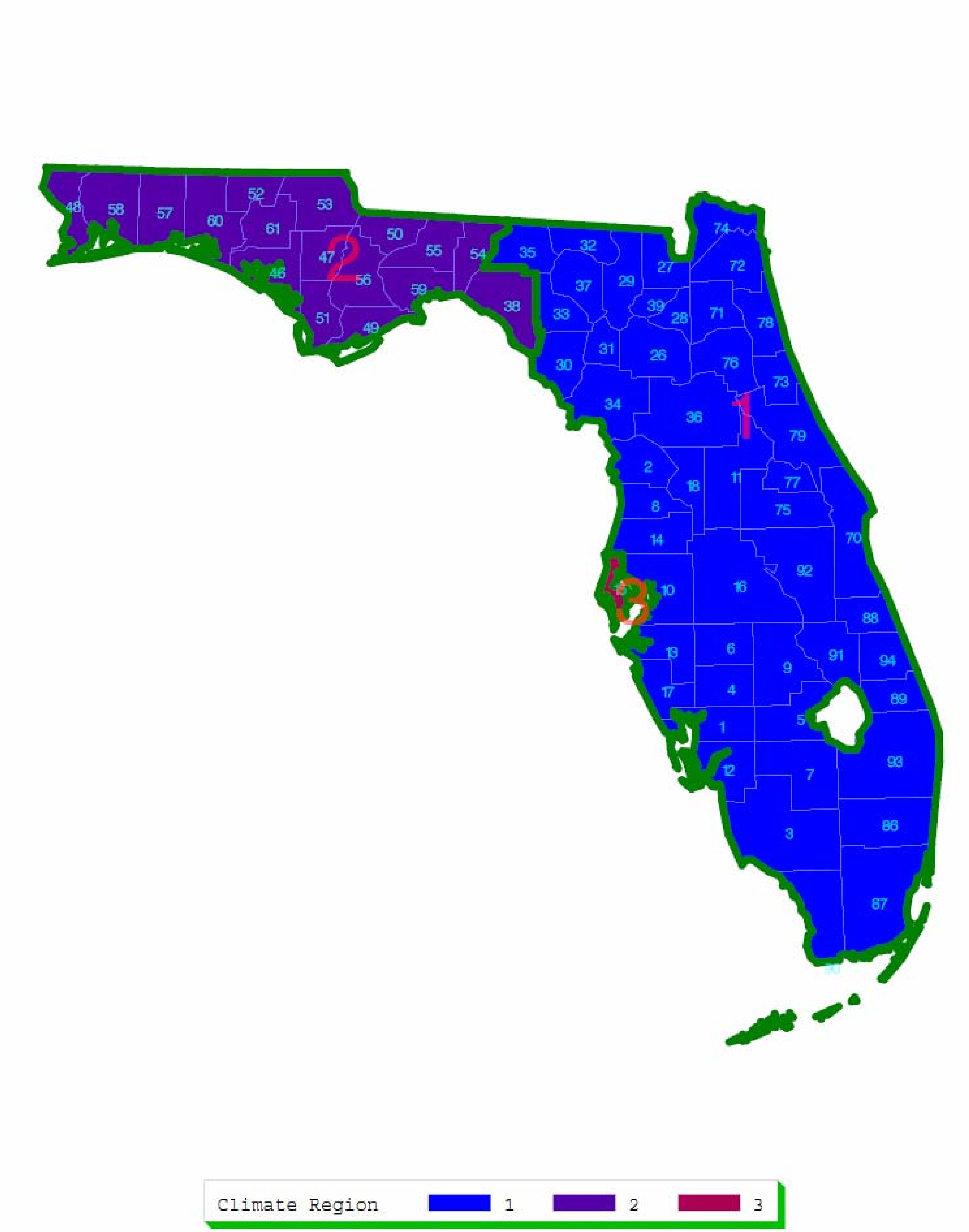


Figure 4.8. Subdivision of Florida into Three Climatic Regions.

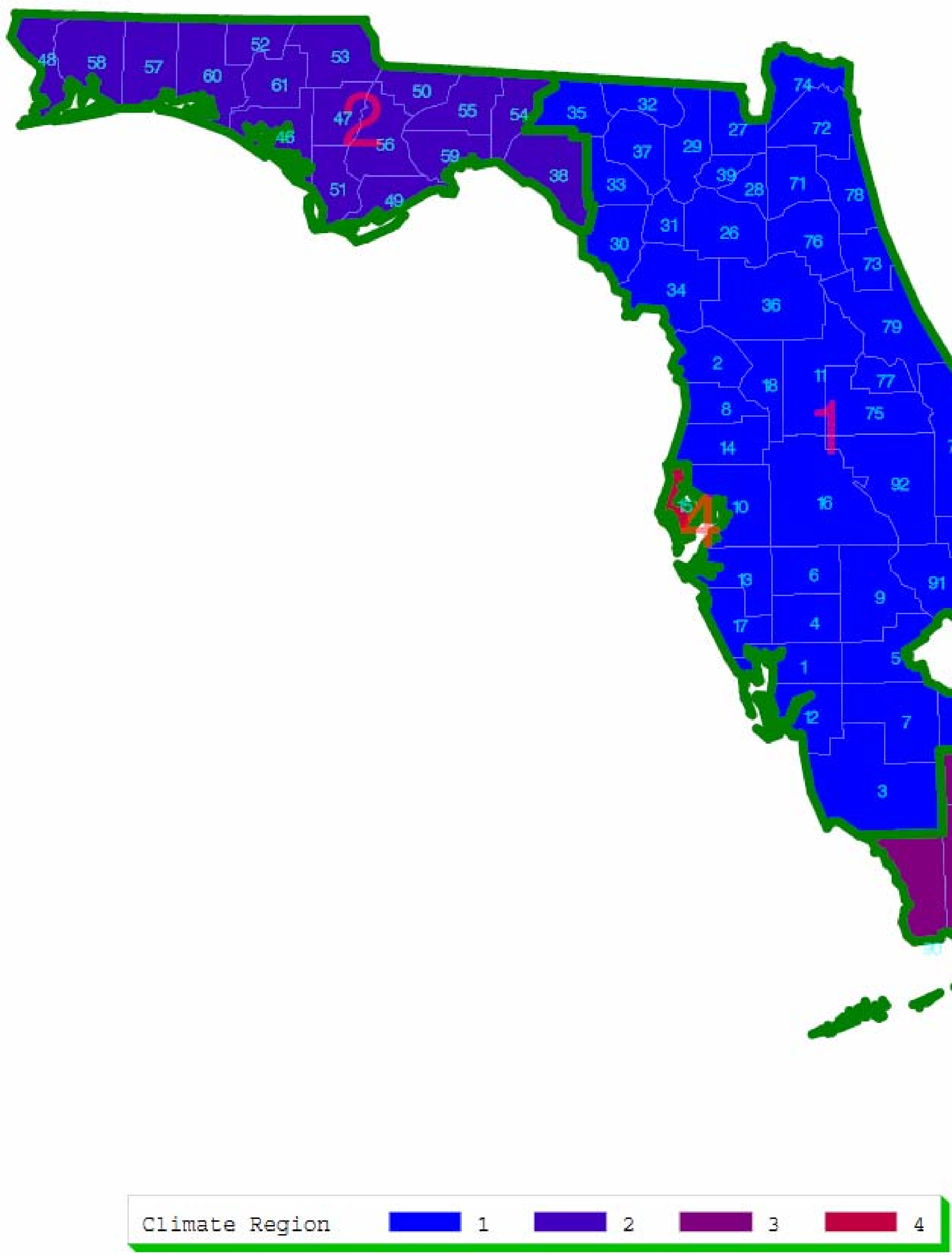


Figure 4.9. Subdivision of Florida into Four Climatic Regions.

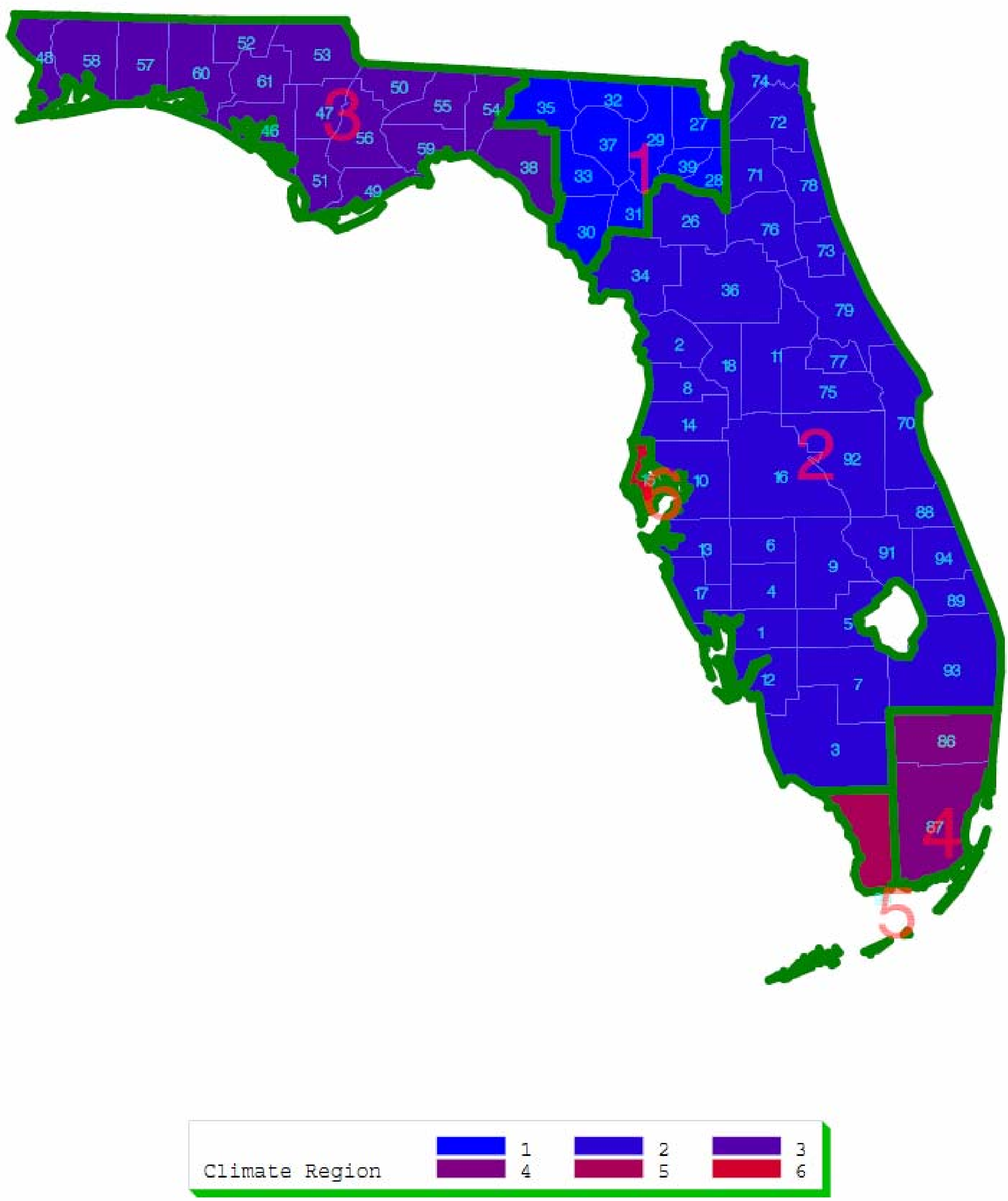


Figure 4.10. Subdivision of Florida into Six Climatic Regions.

SOIL CHARACTERIZATION

Researchers reviewed Florida county soil survey reports to identify the soil types found in the different counties. From this review, the predominant soils in each county were identified based on the volume occupied for a given soil as determined from the county soil survey report. Figure 4.11 shows the predominant soils found in each county. From this information, researchers established a representative soil suction curve per county using published information on soil-water characteristic curves (Mason et al., 1986 and Lytton et al., 1990). This representative soil suction curve is the weighted average (based on soil volume) of the soil-water characteristic curves for the predominant soils found in a given county. Researchers determined the soil volume from information on coverage (area and depth) reported for a given soil in the applicable county soil survey. As evident from the map given in Figure 4.11, most of the state consists of sandy soil.

Having established representative soil suction curves for all counties, researchers tabulated the estimated water contents corresponding to suction values ranging from 2.0 to 4.8 pF at 0.4 increments as presented in Table 4.1. Note that 1 pF is equal to the logarithm (base 10) of the absolute value of soil suction in cm of water. Researchers plan to use the data presented in Table 4.1 to establish soil-water characteristic curves by county for the purpose of developing the initial M-E PDG-based pavement design guide for Florida.

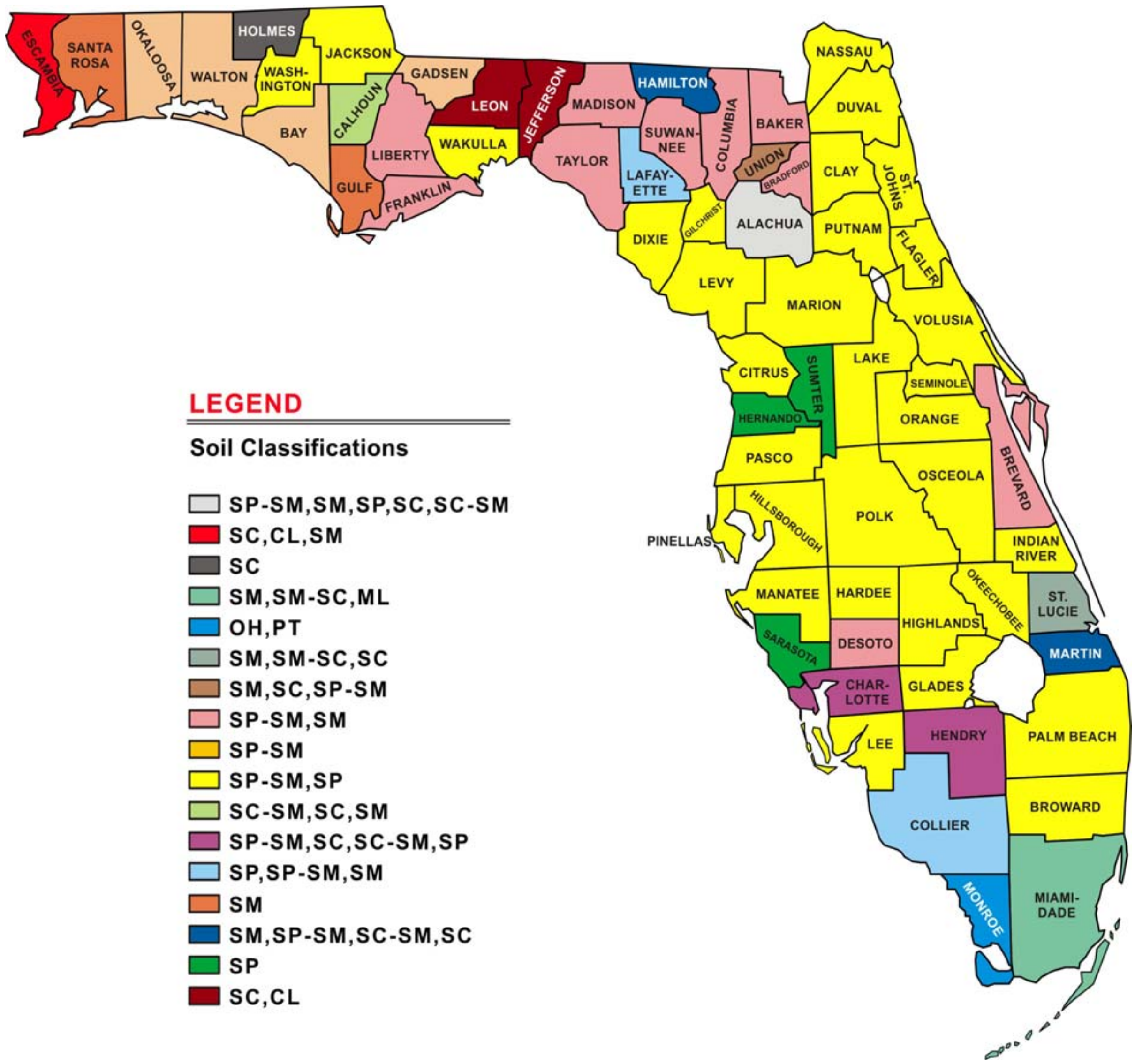


Figure 4.11. Predominant Soils Identified from County Soil Survey Reports.

Table 4.1. Estimated Volumetric Water Contents for 2 to 4.8 pF Suction Values.

County Name	Volumetric Moisture Content							
	2.0	**2.4**	**2.8**	**3.2**	**3.6**	**4.0**	**4.4**	**4.8**
Alachua	0.2368	0.1753	0.1257	0.0886	0.0619	0.0430	0.0297	0.0204
Baker	0.1580	0.1013	0.0613	0.0367	0.0221	0.0134	0.0080	0.0048
Bay	0.1369	0.0833	0.0464	0.0244	0.0124	0.0062	0.0031	0.0015
Bradford	0.2438	0.1900	0.1472	0.1153	0.0909	0.0715	0.0558	0.0430
Brevard	0.1814	0.1278	0.0882	0.0616	0.0437	0.0314	0.0227	0.0164
Broward	0.1312	0.0749	0.0394	0.0198	0.0097	0.0047	0.0023	0.0011
Calhoun	0.3726	0.3113	0.2482	0.1905	0.1426	0.1048	0.0761	0.0548
Charlotte	0.2673	0.2026	0.1476	0.1049	0.0739	0.0519	0.0364	0.0255
Citrus	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Clay	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Collier	0.1434	0.0854	0.0478	0.0263	0.0147	0.0085	0.0050	0.0031
Columbia	0.1672	0.1063	0.0637	0.0381	0.0231	0.0141	0.0086	0.0052
Dade	0.3023	0.2532	0.2051	0.1630	0.1286	0.1009	0.0787	0.0609
De Soto	0.1531	0.0892	0.0463	0.0226	0.0107	0.0051	0.0024	0.0011
Dixie	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Duval	0.1315	0.0752	0.0397	0.0138	0.0098	0.0047	0.0023	0.0011
Escambia	0.4082	0.3563	0.3050	0.2571	0.2140	0.1764	0.1439	0.1159
Flagler	0.1307	0.0740	0.0387	0.0193	0.0094	0.0045	0.0022	0.0010
Franklin	0.1590	0.0913	0.0462	0.0219	0.0101	0.0047	0.0022	0.0010
Gadsden	0.1369	0.0833	0.0464	0.0244	0.0124	0.0062	0.0031	0.0015
Gilchrist	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Glades	0.1309	0.0743	0.0389	0.0195	0.0095	0.0046	0.0022	0.0011
Gulf	0.2533	0.1998	0.1557	0.1216	0.0943	0.0721	0.0543	0.0404
Hamilton	0.2615	0.2004	0.1475	0.1060	0.0750	0.0524	0.0362	0.0249
Hardee	0.1308	0.0742	0.0388	0.0194	0.0095	0.0046	0.0022	0.0011
Hendry	0.2427	0.1792	0.1278	0.0895	0.0622	0.0434	0.0303	0.0212
Hernando	0.1240	0.0641	0.0304	0.0138	0.0061	0.0027	0.0012	0.0005
Highlands	0.1325	0.0767	0.0409	0.0208	0.0103	0.0050	0.0024	0.0012
Hillsborough	0.1309	0.0743	0.0389	0.0195	0.0095	0.0046	0.0022	0.0011
Holmes	0.5422	0.4611	0.3795	0.3027	0.2344	0.1771	0.1310	0.0954
Indian River	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Jackson	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Jefferson	0.4880	0.4382	0.3859	0.3335	0.2833	0.2366	0.1944	0.1571
Lafayette	0.1462	0.0860	0.0470	0.0252	0.0137	0.0075	0.0042	0.0024
Lake	0.1308	0.0742	0.0388	0.0194	0.0095	0.0046	0.0022	0.0011
Lee	0.1320	0.0760	0.0403	0.0204	0.0101	0.0049	0.0024	0.0011
Leon	0.4229	0.3761	0.3274	0.2793	0.2338	0.1923	0.1555	0.1236
Levy	0.1306	0.0739	0.0386	0.0192	0.0094	0.0045	0.0022	0.0010
Liberty	0.1590	0.0913	0.0462	0.0219	0.0101	0.0047	0.0022	0.0010
Madison	0.2063	0.1489	0.1064	0.0778	0.0585	0.0447	0.0342	0.0261
Manatee	0.1308	0.0742	0.0389	0.0194	0.0095	0.0046	0.0022	0.0011
Marion	0.1325	0.0768	0.0410	0.0208	0.0103	0.0050	0.0024	0.0012
Martin	0.2382	0.1766	0.1271	0.0903	0.0641	0.0454	0.0323	0.0229
Monroe	0.2228	0.1693	0.1264	0.0947	0.0717	0.0547	0.0419	0.0320
Nassau	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Okaloosa	0.1369	0.0833	0.0464	0.0244	0.0124	0.0062	0.0031	0.0015

Table 4.1. Estimated Volumetric Water Contents for 2 to 4.8 pF Suction Values (continued).

County Name	Volumetric Moisture Content							
	2.0	**2.4**	**2.8**	**3.2**	**3.6**	**4.0**	**4.4**	**4.8**
Okeechobee	0.1313	0.0750	0.0395	0.0198	0.0097	0.0047	0.0023	0.0011
Orange	0.1309	0.0743	0.0389	0.0195	0.0095	0.0046	0.0022	0.0011
Osceola	0.1322	0.0763	0.0406	0.0206	0.0102	0.0050	0.0024	0.0012
Palm Beach	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Pasco	0.1312	0.0748	0.0393	0.0197	0.0097	0.0047	0.0023	0.0011
Pinellas	0.1310	0.0746	0.0392	0.0196	0.0096	0.0046	0.0022	0.0011
Polk	0.1313	0.0750	0.0395	0.0198	0.0097	0.0047	0.0023	0.0011
Putnam	0.1320	0.0760	0.0403	0.0204	0.0100	0.0049	0.0024	0.0011
St. Johns	0.1319	0.0759	0.0402	0.0203	0.0100	0.0049	0.0024	0.0011
St. Lucie	0.3460	0.2795	0.2158	0.1616	0.1188	0.0864	0.0624	0.0448
Santa Rosa	0.2533	0.1998	0.1557	0.1216	0.0943	0.0721	0.0543	0.0404
Sarasota	0.1240	0.0641	0.0304	0.0138	0.0061	0.0027	0.0012	0.0005
Seminole	0.1312	0.0748	0.0393	0.0197	0.0097	0.0047	0.0023	0.0011
Sumter	0.1240	0.0641	0.0304	0.0138	0.0061	0.0027	0.0012	0.0005
Suwannee	0.1951	0.1415	0.1010	0.0730	0.0534	0.0392	0.0287	0.0209
Taylor	0.1725	0.1137	0.0716	0.0452	0.0288	0.0182	0.0114	0.0070
Union	0.3345	0.2724	0.2178	0.1722	0.1345	0.1036	0.0786	0.0589
Volusia	0.1393	0.0895	0.0541	0.0314	0.0178	0.0100	0.0056	0.0031
Wakulla	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010
Walton	0.1369	0.0833	0.0464	0.0244	0.0124	0.0062	0.0031	0.0015
Washington	0.1304	0.0737	0.0384	0.0191	0.0093	0.0045	0.0021	0.0010

CHAPTER V. FIELD AND LABORATORY TESTING

Researchers discussed the proposed test plan presented in Chapter II in several meetings with Florida DOT engineers to identify testing needs, request the Department's assistance in conducting the proposed field and laboratory tests, and coordinate the field activities at the different calibration sites. To determine the scope of coring and trenching activities, researchers analyzed the FWD data collected on the test sections, reviewed available data in FDOT's coring database, conducted additional sensitivity analyses of the M-E PDG program, and viewed video logs of the calibration sections maintained by the Department. This chapter describes how researchers established the coring and trenching locations, and the field tests conducted on the calibration sections. Data from tests completed in this project are also presented.

DETERMINATION OF CORING AND TRENCH LOCATIONS

With FDOT's assistance, material samples were collected from each calibration section by coring and trenching, for the purpose of running tests in the laboratory to characterize material properties needed for calibrating the performance models in the M-E PDG program. Researchers reviewed PCS and coring data, and viewed video logs of the calibration sections found on FDOT's intranet (http://webapp01.dot.state.fl.us/videolog/default.asp) to determine appropriate coring and trench locations. In addition, researchers examined the deflection profile along each section as determined from FWD testing to establish the uniformity of the section on the basis of the measured deflections and identify areas that show relatively high deflections. Appendix B shows FWD data and PCS data collected on the different calibration sites.

From examination of the FWD deflection data and review of available coring data and video logs, researchers selected locations where material samples would be taken by coring and trenching. Locations close to intersections and within residential areas were avoided to the extent possible. Compared to coring, trenching requires more time and personnel resources due to the associated operational requirements. Therefore, researchers tried to minimize the number of trench locations based on results from additional sensitivity analyses conducted to establish the scope of coring and trenching operations.

To verify the sensitivity of the performance predictions to base and subgrade resilient moduli, researchers considered two pavement sections. One section is the HMAC pavement located in Santa Rosa County with roadway ID 58010000. According to information obtained from FDOT's coring data base, this section consists of 4.5 inches of asphalt mix, overlying a 20-inch sand/clay base, on top of a silty sand subgrade. The other section is a typical Portland cement concrete (PCC) pavement consisting of a 10-inch concrete slab, 2-inch asphalt permeable base, 10-inch limerock base, and 12-inch stabilized subgrade on top of sandy soil subgrade.

For the sensitivity analyses of the HMAC and PCC sections, the base modulus was varied from 40 to 60 ksi in 10 ksi increments, while the subgrade modulus was varied from 15 to 35 ksi, also in 10 ksi increments. In addition, researchers examined the sensitivity of the PCC performance predictions to the modulus of subgrade reaction *k* by varying this variable from 100 to 400 pci in the analysis. Climatic, soils and traffic data considered representative of the conditions at the sites selected were input to the M-E PDG program. The results from the sensitivity analyses are shown in Figures 5.1 to 5.5. Figures 5.1 and 5.2 indicate that the predictions from the flexible pavement top-down cracking model in the MEPDG program are sensitive to the base and subgrade resilient moduli. It is interesting to note that higher subgrade modulus results in larger amounts of top-down cracks predicted for the HMAC section, as shown in Figure 5.2. A similar trend is also reported in Appendix II of the MEPDG program supplemental documents. In contrast, the base and subgrade moduli do not appear to significantly influence the predicted level of cracking on the PCC pavement section analyzed, as indicated in Figures 5.3 and 5.4. The change of *k*-value affected cracking in a somewhat different manner as shown in Figure 5.5. Researchers note that most of the predicted performance curves are close together except the case where the *k*-value is 100 pci, which is equivalent to a subgrade modulus of 5 ksi based on the M-E PDG program output for the pavement section analyzed.

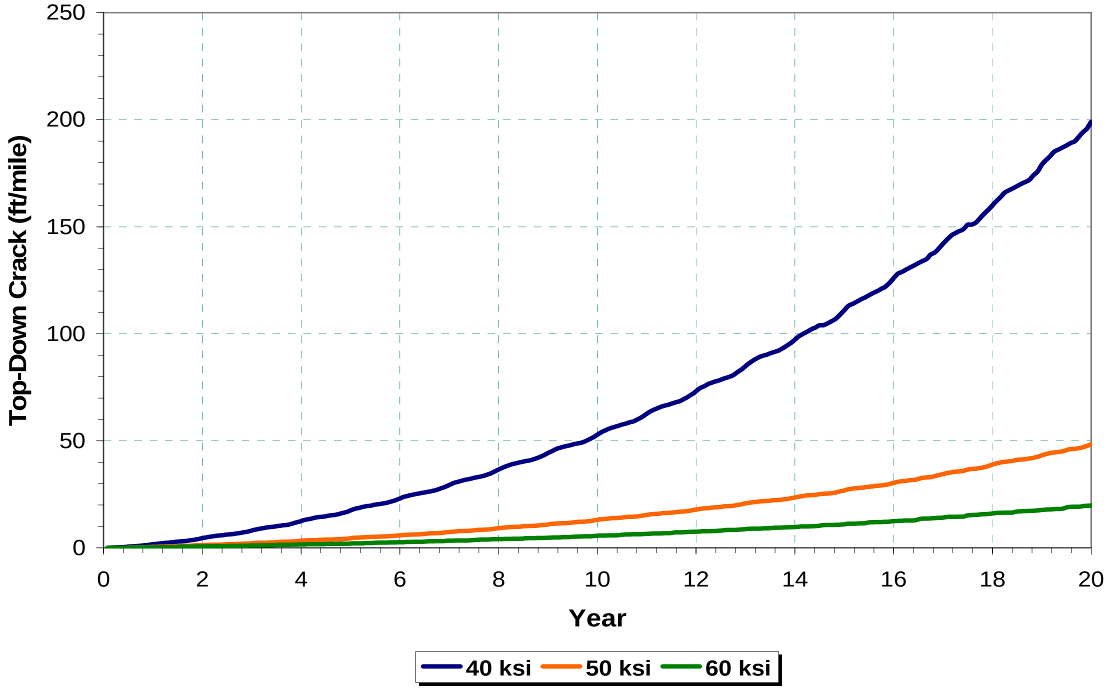


Figure 5.1. Sensitivity of Top-Down Cracking Performance Predictions to Base Modulus for HMAC Pavement Section.

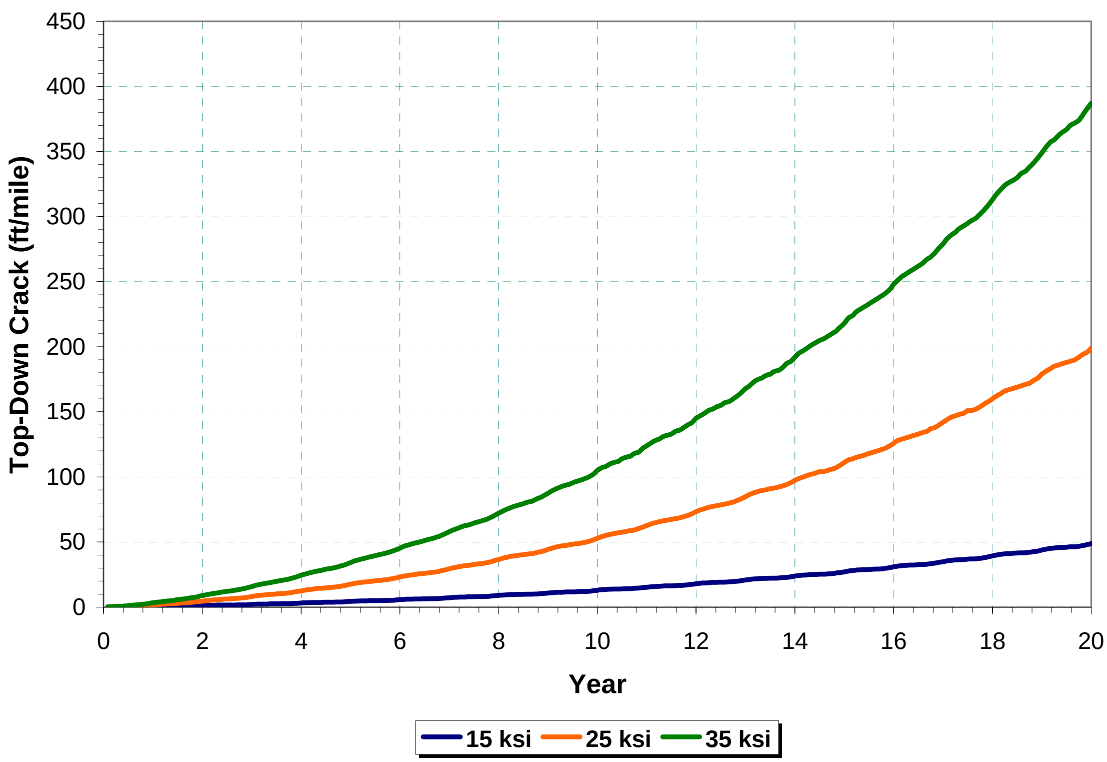


Figure 5.2. Sensitivity of Top-Down Cracking Performance Predictions to Subgrade Modulus for HMAC Pavement Section.

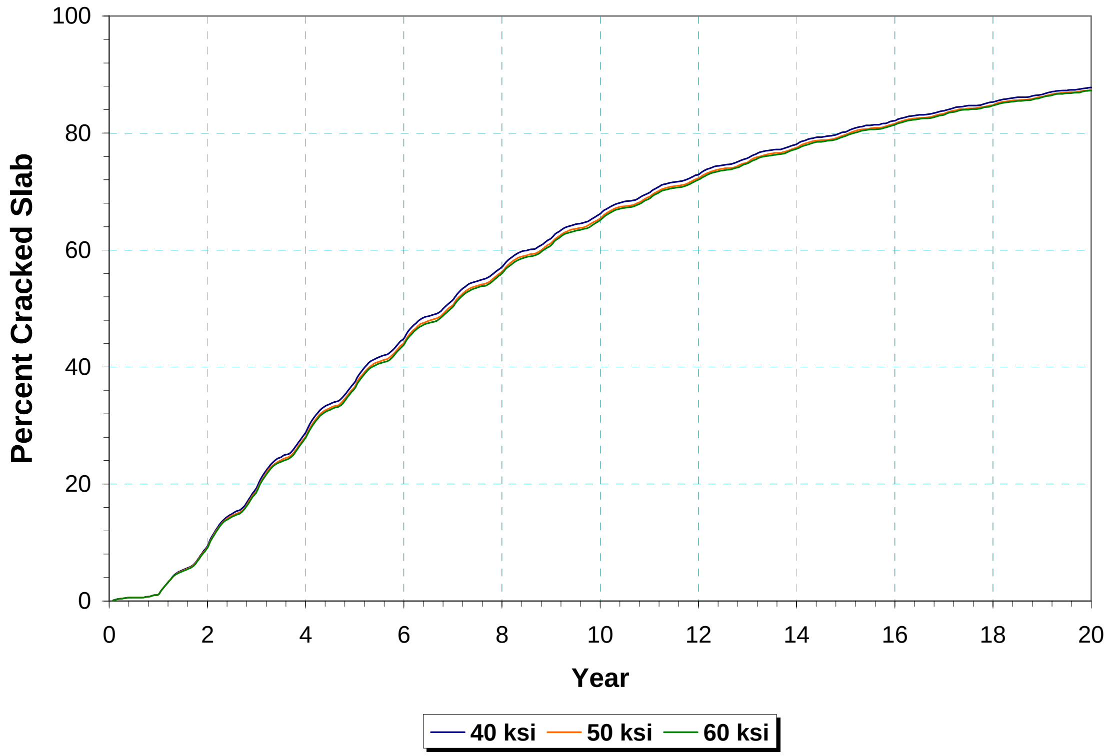


Figure 5.3. Sensitivity of PCC Crack Predictions to Base Modulus.

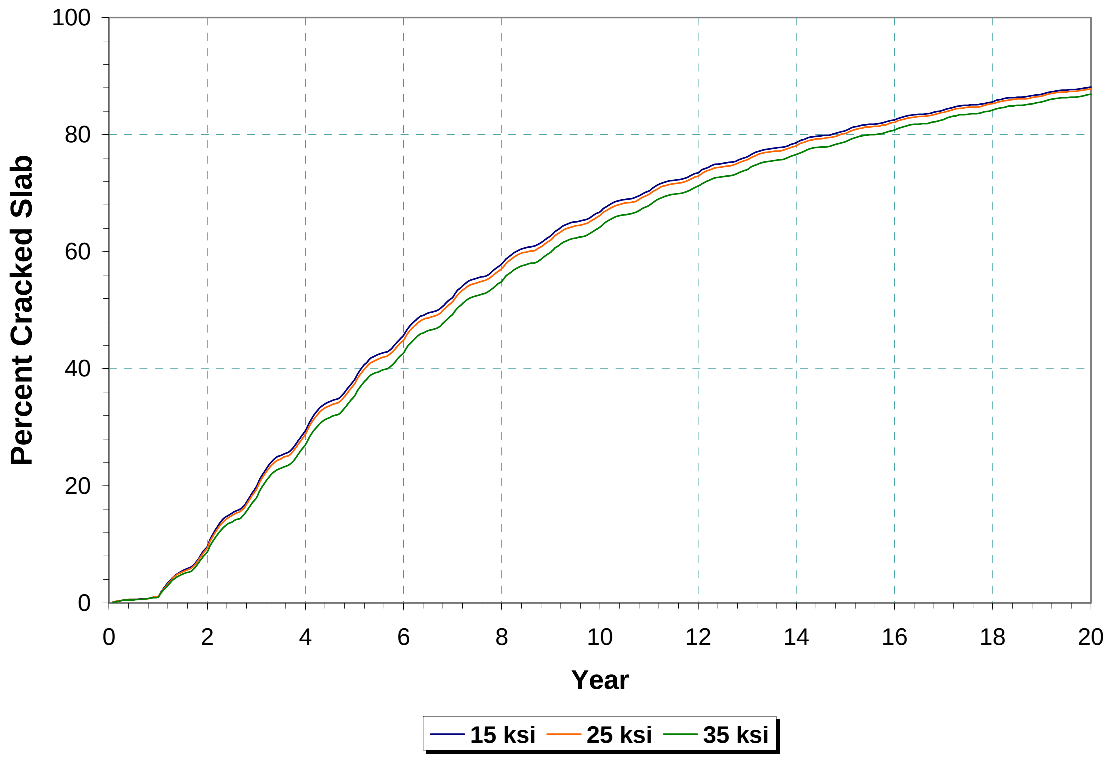


Figure 5.4. Sensitivity of PCC Crack Predictions to Subgrade Modulus.

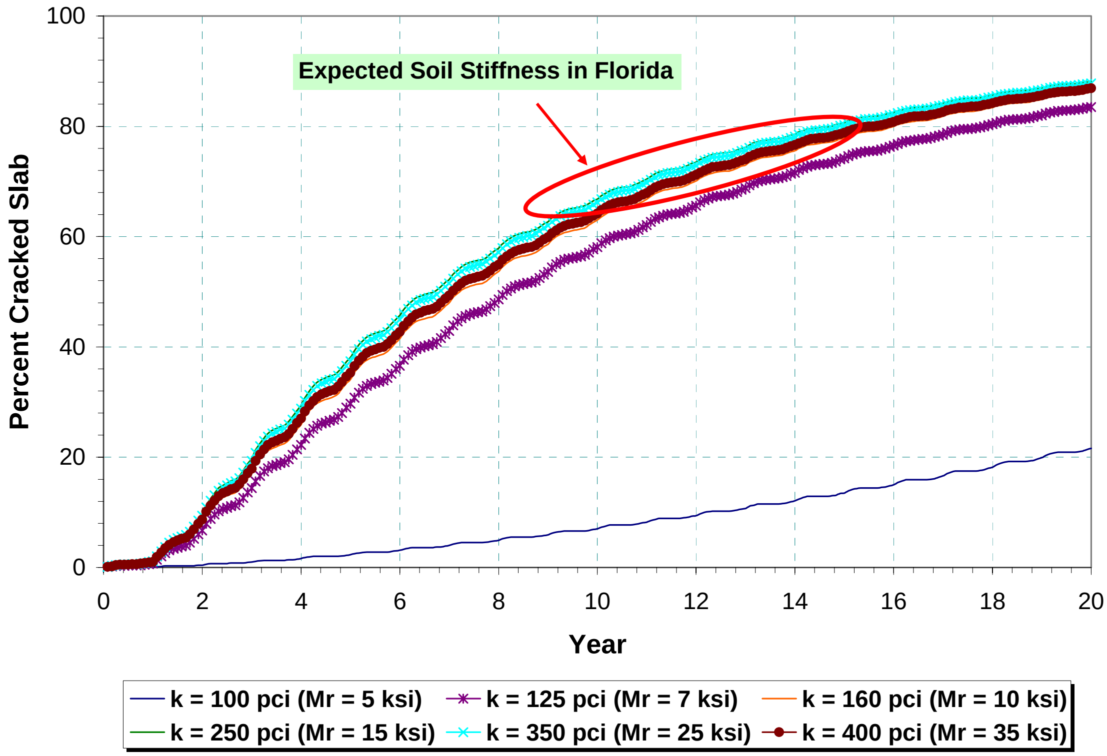


Figure 5.5. Sensitivity of PCC Crack Predictions to Subgrade Modulus of Reaction.

In view of the results obtained from the sensitivity analyses, one trench location was established for each flexible pavement calibration section. This location was generally within an area where the FWD deflections are high (relative to the range of deflections measured within the section) or where visual distresses were observed from the video logs. In the case of rigid pavements, the results from the sensitivity analyses suggested that trenching was not as important for the PCC calibration sections as it was for the HMAC sections. In view of these results and after consulting with the project manager, the decision was made to forego trenching in rigid pavement sections. For these sections, FWD test data will be used to characterize the underlying materials for the PCC model calibrations.

Table 5.1 identifies the specific coring and trench stations established by researchers. In this table, the mileposts printed in red identify locations where researchers proposed to core through cracks to verify the type of cracking (top-down or bottom-up) observed on the sections from the photo logs viewed from FDOT’s intranet site.

Table 5.1. Proposed Core and Trench Locations.

District	County	Roadway ID	Type	Section Limits	Core Stations (milepost)	Trench Station (milepost)
1	Charlotte	10100000	PCC	4.98 ~ 0.49	4.517, 3.092, 2.327, 0.687	-
	Polk	16250000	AC	4.616 ~ 7.38	4.652, 4.881, 5.109, 5.432, 5.789, 5.895, 6.143, 6.287, 6.536, 6.859, 7.143	6.143
	Polk	16003001	AC	8.484 ~ 10.0	8.606, 8.736, 8.80, 8.996, 9.20, 9.6, 9.866	8.736
	Polk	16100000	PCC	0.46~1.74	0.59, 0.75, 0.95, 0.967, 1.074, 1.538, 1.59	-
2	Alachua	26005000	AC	10.691 ~ 7.954	10.637, 10.40, 10.10, 9.925, 9.60, 9.30, 9.10, 8.642, 8.426, 8.142, 8.071	9.60
	Bradford	28040000	AC	0.0 ~ 5.509	0.1, 0.288, 0.7, 0.967, 1.40, 1.7, 1.967, 2.357, 2.536, 2.7, 3.288, 3.754, 4.4, 4.7, 5.1, 5.323	2.536
	St. Johns	78020000	PCC	0.7 ~ 0.02	0.5, 0.323, 0.14	-
3	Gadsden	50010000	AC	16.48 ~ 18.57	16.699, 16.977, 17.196, 17.275, 17.412, 17.765, 18.04, 18.393	17.412
	Gadsden	123811	PCC			-
	Santa Rosa	58060000	AC	21.80 ~ 20.72	21.78,21.693, 21.617, 21.53, 21.4, 21.336, 21.227, 20.971, 20.798	21.227
4	Broward	86190000	AC	2.33 ~ 3.67	2.401, 2.686, 2.837, 3.07, 3.20, 3.53	3.07
	Palm Beach	93100000	AC	11.904 ~ 12.617	12.047, 12.157, 12.305, 12.40, 12.547	12.305
	Palm Beach	93310000	AC	17.796 ~ 12.215	17.52, 17.2, 17.02, 16.444, 15.195, 14.666, 13.695, 13.196, 12.675	17.52
5	Lake	11020000	PCC	14.03 ~ 14.12	14.06, 14.12	-
	Seminole	77040000	AC	5.808 ~ 11.046	6.073, 6.222, 6.352, 6.682, 7.143, 7.462, 7.643, 8.214 8.643, 9.323, 9.672, 10.282, 10.69, 10.922	7.462
	Volusia	79270000	AC	2.39 ~ 1.62	2.34, 2.327, 2.134, 2.0, 1.927, 1.713, 1.642	1.713
6	Monroe	90060000	AC	13.032 ~ 16.384	13.3, 13.534, 13.76, 14.08, 14.155, 14.328, 14.73, 15.067, 15.337, 15.528, 15.917, 16.2	15.528
	Dade	87060000	AC	2.715 ~ 0.872	2.595, 2.266, 2.149, 2.009, 1.792, 1.629, 1.493, 1.429, 1.289, 0.996	2.595
	Dade	87030000	PCC	8.83 ~ 10.0	8.96, 9.13, 9.69, 9.50	-
	Dade	87030000	PCC	10.0 ~ 8.83	9.888, 9.517, 9.288, 8.962	-
	Dade	87270000	PCC	0.0 ~ 0.98	0.229, 0.56	-
	Dade	87061000	PCC	0.21 ~ 0.0	0.195, 0.126	-

Table 5.1. Proposed Core and Trench Locations (continued).

District	County	Roadway ID	Type	Section Limits	Core Stations (milepost)	Trench Station (milepost)
7	Hillsborough	10060000	AC	8.91 ~ 17.43	9.37, 9.84, 10.281, 10.5, 10.642, 11.377, 11.785, 12.40, 13.085, 13.661, 14.05, 14.238, 15.079, 15.491, 16.634, 17.222	10.50
	Hillsborough	10160000	AC	0.0 ~ 6.773	0.11, 0.282, 0.84, 1.436, 2.17, 2.51, 2.98, 3.45, 3.843, 4.214, 4.851, 5.157, 6.0, 6.296, 6.588	2.17
	Hillsborough	10250001	PCC	0.0 ~ 1.102		-
	Hillsborough	10075000	PCC	19.0 ~ 20.4	19.071, 19.57, 20.003	-
	Hillsborough	10075000	PCC	23.4 ~ 24.69	23.56, 23.74, 23.99, 24.5	-
	Pinellas	15003000	PCC	0.37 ~ 1.29	0.63, 0.80, 1.0	-
	Pinellas	15003000	PCC	1.29 ~ 0.37	1.17, 0.889, 0.859	-
	Pinellas	15190000	PCC	2.287 ~ 4.44	2.65, 2.92, 3.60, 4.10	-
	Pinellas	15190000	PCC	4.44 ~ 2.287	4.285, 3.555, 3.165, 2.445, 2.395	-

FIELD TESTING

With the assistance of the District Material Offices, researchers obtained samples of hot mix asphalt concrete (HMAC), Portland cement concrete (PCC), base, subgrade and embankment materials from the calibration sections. For hot-mix asphalt concrete (HMAC) sections, 6-inch diameter cores were sampled for the proposed laboratory tests to get asphalt volumetric properties, viscosity (AASHTO T 315), and tensile strength and creep compliance based on FDOT procedure that measures tensile strength and creep compliance of asphalt concrete mixtures for FDOT's top-down cracking model at the standard test temperature of 10 °C. In addition, as a side study, TTI's overlay tester was used to perform a limited investigation on whether the test results can rank pavement performance and whether the rankings may be correlated with PCS performance data.

For PCC sections, cores were sampled at least 4 inches in diameter and 8 inches high for the thermal coefficient of expansion test (AASHTO TP 60) and for the compressive strength test (AASHTO T 22). Two cores per station were extracted for the AASHTO TP 60 test to permit comparative testing between FDOT and TTI laboratories as discussed in a project meeting with FDOT engineers.

With respect to trenching, base, stabilized subgrade, and embankment materials were sampled for resilient modulus testing, sieve analysis, plastic limit, moisture-density, and soil suction tests. Three hundred fifty pounds of each material was collected. Prior to coring and trenching, District engineers marked test locations based on mileposts presented in Table 5.1. Field sampling was conducted under traffic control at the outside lane. All cores were taken between the wheel paths except cracked cores, which were taken at selected crack locations. The ground water table depth at each site was determined by boring adjacent to the trench location. In addition, field densities and moisture contents were measured using the nuclear density gage. The automated dynamic cone penetrometer (ADCP) test was performed from the top of the base layer to obtain correlations between test results and resilient modulus in an in-house study by the Florida DOT. ADCP tests were not conducted on test sections located within Districts 4 and 6 due to machine down time.

Sampled materials were marked, then packed cautiously to protect from damage during shipping. Figure 5.6 illustrates the activities performed during the field tests. In PCC sections, joint spacing and dowel diameter were measured when it was feasible to core at the joint as shown in Figure 5.7. Whenever cores were taken, the layer composition and

a. Core at trench location

b. Record core data and wrap samples

c. Trench

d. Nuclear density testing

e. ADCP testing at trench location

f. Boring close to trench location

Figure 5.6. Illustration of Field Work on HMAC Calibration Sections.

Figure 5.7. Measurement of Joint Spacing and Dowel Diameter.

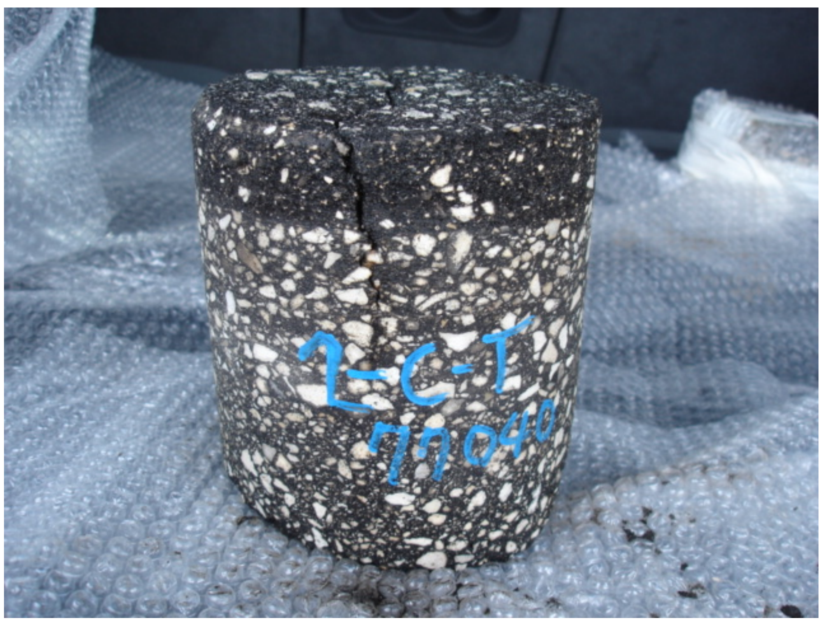


Figure 5.8. Top-Down Cracking Observed on AC Core.

thickness of each lift were determined and recorded on the data sheets as presented in Appendix C. Additionally, comments were made whenever any notable findings were detected during field sampling.

During coring, researchers also made an attempt to characterize the crack type by coring through selected cracks. The predominant distress on Florida pavements is top-down cracking. Figure 5.8 shows an example of this type of cracking observed on a core taken from the calibration section located in Seminole County.

LABORATORY TESTING

Tables 5.2 to 5.4 present the proposed laboratory tests to characterize material properties of cores and underlying materials taken from the calibration sections. The State Materials Office of the Florida DOT conducted resilient modulus tests on base, subgrade, and embankment materials as well as extractions on AC cores. TTI conducted indirect tensile strength tests, soil suction tests, and a limited amount of overlay tests on AC specimens. A significant amount of the proposed laboratory tests were completed on this project, with the remaining tests to be completed in a supplemental project.

Characterization of Asphalt Concrete Properties

Extractions and dynamic shear rheometer (DSR) tests were conducted on selected AC lifts of cores sampled from the calibration sections. Extractions (Figure 5.9) provided mix volumetric and gradation properties that are needed to predict dynamic modulus of asphalt mixtures in the M-E PDG program. DSR tests were also performed on the extracted binder to characterize the viscosity-temperature relationship for dynamic modulus prediction.

Table 5.5 presents data from extractions and DSR tests conducted on AC samples taken on section 16003001 in Polk County. The properties shown were determined using the equations given in Table 5.2.

Researchers note that the indirect tensile test method currently implemented by the Florida DOT to obtain resilient modulus, creep compliance, and tensile strength properties of HMAC samples will not provide properties directly compatible with the thermal cracking prediction model in the M-E PDG program. In a project meeting held at the State Materials Office, FDOT engineers noted that thermal cracking is not a problem in Florida and need to be considered only on a very limited basis in the model calibrations. Recognizing that a national project (NCHRP 1-42A) is underway to develop a top-down cracking model, the recommendation was made to support its future implementation in Florida by characterizing properties that can be used for future calibrations of the top-down cracking model that will come out of NCHRP Project 1-42A. Since this model is not expected to be available within the time frame of the supplemental project to develop the initial M-E PDG-based pavement design guide, researchers recommend that the supplemental project focus on calibrating the load-associated crack models in the latest version of the M-E PDG program. Implementation of the top-down cracking model that will come out of NCHRP 1-42A can be addressed in a future FDOT project. In addition, researchers recommend that a limited analysis be

Table 5.2. Tests on Asphalt Concrete Cores.

Property	Test Protocol	
	AASHTO / ASTM	**Florida Standard Test Method (FSTM)**
Binder content, % by weight	ASTM D 2172-01	FM 5-544
Gradation in terms of cumulative % retained on 3/4", 3/8", and #4 sieves, and % passing #200	AASHTO T30	FM 1-T 030
Effective bitumen content, % by volume (V_{be})*	Following tests are needed to be done to calculate V_{beff}. (a) Measure the bulk specific gravity (G_{sb}) of the coarse aggregate (AASHTO T 85 or ASTM C 127) and of the fine aggregate (AASHTO T 84 or ASTM C128). (b) Measure the specific gravity (G_b) of the asphalt cement (AASHTO T 228 or ASTM D 70). (c) Measure the bulk specific gravity (G_{mb}) of compacted paving mixture sample (FM 1-T 166 or AASHTO T 166-00 or ASTM D 2726). Measure the maximum specific gravity (G_{mm}) of paving mixture (FM 1-T 209 or AASHTO T 209-99 or ASTM D 2041).	
Air void (%)**	AASHTO T269	-
Temperature-viscosity relationship***	AASHTO T315	-
Resilient modulus, tensile strength, and creep compliance****	AASHTO T 322	Florida DOT indirect test method
Overlay*****		TTI test method

*Effective bitumen content by volume is calculated using below equations;

$$V_{be} = G_{mb}\left[\frac{P_b}{G_b} - (100 - P_b)\frac{(G_{se} - G_{sb})}{G_{se} * G_{sb}}\right] \quad \text{and} \quad G_{se} = \frac{100 - P_b}{\dfrac{100}{G_{mm}} - \dfrac{P_b}{G_b}}$$

where P_b is binder content by weight which is obtained from extraction and G_{se} is effective specific gravity of the aggregate.

**Air void is calculated using the following equation:

$$V_a = 100 * \frac{G_{mm} - G_{mb}}{G_{mm}}$$

***From this test, the temperature-viscosity relationship is obtained using following equations:

$$\eta = \frac{G*}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628} \quad \text{and} \quad \log\log\eta = A + VTS\log T_R$$

where $G*$ is binder complex shear modulus (Pa), δ is binder phase angle (°), η is viscosity (cP), T_R is temperature in degrees Rankine, and A, and VTS are regression parameters. At the very least, the output data set needs to include $G*$, δ and test temperature.

**** TTI will set up to run the indirect tensile test according to the current Florida test method.

***** Limited overlay tests conducted by TTI.

Table 5.3. Tests on Portland Cement Concrete Cores.

Property	Test Protocol	
	AASHTO / ASTM	**Florida Standard Test Method (FSTM)**
Coefficient of thermal expansion	AASHTO TP 60	-
Compressive strength	AASHTO T22	FM 1-T 022

Table 5.4. Tests on Base, Stabilized Subgrade, and Embankment Materials.

Property	Test Protocol	
	AASHTO / ASTM	**Florida Standard Test Method (FSTM)**
Gradation	AASHTO T27	FM 1-T 027
Atterberg limits	AASHTO T90	FM 1-T 090
Moisture-density curve	AASHTO T99 or AASHTO T180	FM 1-T 180
Resilient modulus*	AASHTO T307	
Soil water characteristic or soil suction curve**	Filter paper method	

*Resilient modulus performed at optimum moisture content. The output data set needs to include the confining pressure, deviatoric stress, and resilient modulus.

**Soil suction tests conducted by TTI using 50 lbs of each material type sampled from the trenches.

Figure 5.9. Illustration of Extractions Done on Asphalt Cores.

Table 5.5. Properties Determined from Extractions on Core Samples from Section 16003001 in Polk County.

Project No. **16003001**

Sample No.	G_{mb}	G_{mm}	P_b	G_{se}	G_{sb}	V_{be}	% AV
1-1	2.07	2.39	6.45	2.63	2.57	11.20	13.35
1-2	2.21	2.37	5.30	2.56	2.48	8.92	6.86
1-3	2.19	2.39	5.13	2.57	2.48	7.79	8.26
2-1	2.05	2.39	6.45	2.63	2.57	11.09	14.19
2-2	2.23	2.37	5.30	2.56	2.48	8.99	6.18
2-3	2.22	2.39	5.13	2.57	2.48	7.89	7.17
3-1	2.11	2.39	6.45	2.63	2.57	11.39	11.86
3-2	2.27	2.37	5.30	2.56	2.48	9.16	4.40
3-3	2.16	2.39	5.13	2.57	2.48	7.70	9.35
4-1	2.12	2.39	6.45	2.63	2.57	11.45	11.43
4-2	2.29	2.37	5.30	2.56	2.48	9.23	3.58
4-3	2.16	2.39	5.13	2.57	2.48	7.69	9.52
5-1	2.09	2.39	6.45	2.63	2.57	11.27	12.82
5-2	2.24	2.37	5.30	2.56	2.48	9.05	5.46
5-3	2.17	2.39	5.13	2.57	2.48	7.71	9.21
6-1	2.03	2.39	6.45	2.63	2.57	10.98	15.06
6-2	2.20	2.37	5.30	2.56	2.48	8.89	7.20
6-3	2.16	2.39	5.13	2.57	2.48	7.69	9.50
7-1	2.07	2.39	6.45	2.63	2.57	11.20	13.35
7-2	2.25	2.37	5.30	2.56	2.48	9.07	5.27
7-3	2.17	2.39	5.13	2.57	2.48	7.71	9.25

	% AV	$\mathbf{V_{be}}$
Layer 1	13.50	11.22
Layer 2	5.50	9.04
Layer 3	9.00	7.73

	Layer 1		Layer 2		Layer 3	
Temp.	Complex Modulus G* (Pa)	Phase Angle (δ°)	Complex Modulus G* (Pa)	Phase Angle (δ°)	Complex Modulus G* (Pa)	Phase Angle (δ°)
52ºC	284000	53.93	184667	71.03	114333	61.59
58ºC	135333	58.08	76667	67.33	50467	65.55
64ºC	62267	62.42	34600	69.09	22800	69.64
70ºC	29233	66.79	16100	71.76	11000	73.51
76ºC	13667	71.26	7693	75.13	5183	77.44
82ºC	6267	75.62	3817	78.15	2330	80.67
88ºC	3290	78.47	1800	81.41	1170	83.28
	A	10.317	**A**	10.89	**A**	11.262
	VTS	-3.4338	**VTS**	-3.6464	**VTS**	-3.7854

conducted using the latest version of the M-E PDG program to verify the importance (or lack thereof) of thermal cracking and as necessary, to perform a limited calibration of the thermal cracking model in the supplemental project.

Characterization of Portland Cement Concrete

Cored PCC samples were tested to obtain the concrete thermal expansion coefficient and compressive strength as shown in Figure 5.10. For the coefficient of thermal expansion tests, samples were submerged in a water tank for about a month prior to testing. The available test data shown in Table 5.6 that the thermal coefficients of expansion of the sampled concrete cores are typically below 5.6 με/°F, which is considered a representative value in the M-E pavement design guide. Considering the effect of this coefficient on the predicted performance of PCC pavements, a smaller coefficient is expected to yield better performance. Researchers note that the compressive strength of section 10100000 in Charlotte County is quite lower compared to the values obtained on the other sections. The cores from this section consist of 4 inches of concrete with 8.5 inches of econocrete base. The two lifts could not be cut for testing due to height limitations. Researchers plan to model the first layer of this section as a composite layer for the model calibrations.

(a)

(b)

Figure 5.10. Test Set Up for (a) Compressive Strength and (b) Concrete Coefficient of Thermal Expansion.

Table 5.6. Properties Determined from PCC Core Samples.

District	County	Roadway ID	Section limit	Location	CTE (10^-6 in/in F)	Compressive Strength (psi)
1	Charlotte	10100000	4.98 ~ 0.49	US 41	5.6	2604
1	Polk	16100000	0.46~1.74	US 92	4.74	6049
2	St. Johns	78020000	0.7 ~ 0.02	US 1	4.32	6965
3	Gadsden	123811	200 and 400 ft	I 10	5.11	5299
5	Lake	11020000	14.03 ~ 14.12	SR 50	4.56	4508
6	Dade	87030000	8.83 ~ 10.0	SR 5	5.56	6014
6	Dade	87030000	10.0 ~ 8.83	SR 5	5.42	5669
6	Dade	87270000	0.0 ~ 0.98	9/9A	5.23	5728
6	Dade	87061000	0.21 ~ 0.0	SR 886	4.72	5226
7	Hillsborough	10250001	0.0 ~ 1.102	SR 60	4.72	5485
7	Hillsborough	10075000	19.0 ~ 20.4	I 75	5.04	5866
7	Hillsborough	10075000	23.4 ~ 24.69	I 75	5.13	4978 (1854)*
7	Pinellas	15003000	0.37 ~ 1.29	I 175	4.89	4952 (1516)*
7	Pinellas	15003000	1.29 ~ 0.37	I 175	5.05	5177 (1600)*
7	Pinellas	15190000	2.287 ~ 4.44	I 275	4.92	4728 (1769)*
7	Pinellas	15190000	4.44 ~ 2.287	I 275	4.69	5743 (1542)*

*****Values inside parentheses indicate compressive strength of econocrete base.

Characterization of Underlying Materials

Underlying materials sampled from trenching were tested to obtain physical soil properties, gradation, moisture-density relationship, resilient modulus, and soil suction parameters. Resilient modulus was tested at optimum moisture content and field moisture content based on nuclear density gage measurements. Table 5.7 summarizes soil index properties determined from laboratory tests. Most of the materials obtained from the trenches cut on the calibration sections are non-plastic and classify as A-2-4 and A-3.

Soil suction tests were made using the filter paper method described by Bulut, Lytton, and Wray (2001). Figures from 5.11 to 5.13 show comparisons of soil-water characteristic curves for the embankment soils at sections 26005000, 28040000, and 58060000. In each figure, the solid line represents the soil suction curve based on data compiled from the comprehensive review of soil survey reports described in Chapter IV, while the plotted points represent the data from the soil suction tests conducted in the laboratory. The results shown in the figures appear encouraging as the soil suction curves established from published data provides a reasonable fit to the laboratory test data. However, the correlation needs to be further verified with additional test data.

Table 5.7. Summary of Soil Index Properties from Laboratory Tests on Underlying Materials.

County	District	Roadway ID	Location	Material type	Maximum Density	Optimum w%	LL	PI	Soil class	# 200 Passing
Polk	1	16250000	SR 37	Embankment	114.1	10.8%	NP	NP	A-2-4	12.7%
Polk	1	16003001	SR 563	Base	121.1	10.3%	NP	NP	NA	NA
				Subgrade	123.1	8.5%	NP	NP	A-2-4	15.1%
				Embankment	114.8	10.8%	NP	NP	A-2-4	12.2%
Alachua	2	26005000	SR 222	Base	117.8	10.6%	NP	NP	NA	NA
				Subgrade	120.7	8.5%	NP	NP	A-2-4	12.0%
				Embankment	125.7	9.6%	NP	NP	A-2-4	18.0%
Bradford	2	28040000	SR 18	Subbase	119.0	8.1%	NP	NP	A-2-4	12.0%
				Embankment	117.1	9.4%	NP	NP	A-3	9.0%
Gadsden	3	50010000	US 90	Subbase	130.9	8.8%	21	7.1	A-2-4	17.0%
				Embankment	119.0	12.8%	30.5	16.7	A-6	41.0%
Santa Rosa	3	58060000	SR 89	Base	133.6	7.9%	21	7.4	A-2-4	25.8%
				Subgrade	133.9	7.9%	19.3	4.8	A-2-4	34.1%
				Embankment	126.4	9.7%	21	6.3	A-2-4	34.1%
Broward	4	86190000	SR 823	Base	130.7	7.9%	NP	NP	NA	NA
				Subgrade	129.8	7.0%	NP	NP	A-2-4	17.0%
				Embankment	118.5	9.9%	NP	NP	A-3	8.8%
Palm Beach	4	93100000	SR25/US27	Base	122.0	11.0%	NP	NP	NA	NA
				Subgrade	123.9	11.1%	NP	NP	A-1-b	22.7%
				Embankment	113.5	14.0%	NP	NP	A-2-4	20.5%
Palm Beach	4	93310000	SR 710	Base	129.0	8.0%	NP	NP	NA	NA
				Subgrade	113.2	10.3%	NP	NP	A-3	6.2%
				Embankment	112.7	11.0%	NP	NP	A-3	6.0%
Seminole	5	77040000	SR 46	Subbase	101.3	11.4%	NP	NP	A-3	5.0%
				Embankment	103.7	12.2%	NP	NP	A-3	6.0%

Table 5.7. Summary of Soil Index Properties from Laboratory Tests on Underlying Materials (continued).

County	District	Roadway ID	Location	Material type	Maximum Density	Optimum w%	LL	PI	Soil class	# 200 Passing
Volusia	5	79270000	SR 483	Base	120.9	9.9%	NP	NP	NA	NA
				Subgrade	121.4	7.9%	NP	NP	A-2-4	13.6%
				Embankment	103.1	13.8%	NP	NP	A-3	3.7%
Monroe	6	90060000	US 1	Subbase	120.8	9.4%	NP	NP	NA	NA
Dade	6	87060000	A1A	Base	129.4	7.0%	NP	NP	NA	NA
				Embankment	124.9	7.9%	NP	NP	A-2-4	19.2%
Hillsborough	7	10060000	US 41	Base	120.1	10.0%	NP	NP	NA	NA
				Subgrade	112.6	11.2%	NP	NP	A-3	8.2%
				Embankment	116.8	12.4%	NP	NP	A-2-4	16.9%
Hillsborough	7	10160000	SR 597	Base	117.6	11.6%	NP	NP	NA	NA
				Subgrade	105.1	6.6%	NP	NP	A-3	9.0%
				Embankment	104.4	12.5%	NP	NP	A-3	6.6%

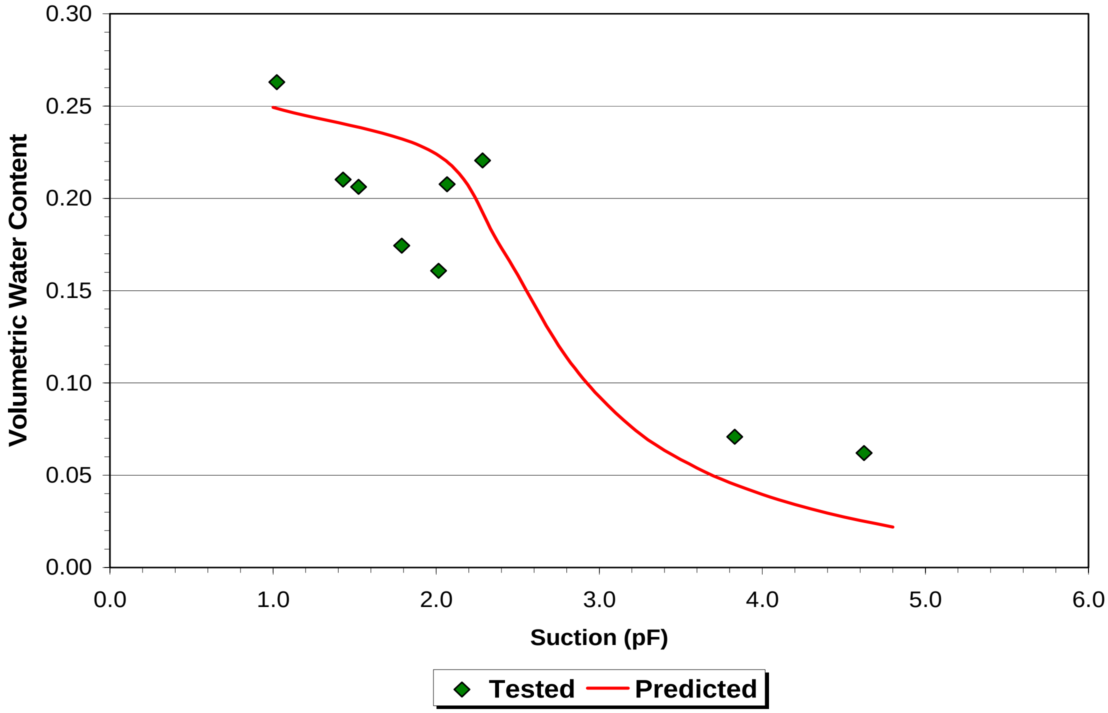


Figure 5.11. Soil Suction Data on Embankment Soil for Section 26005000.

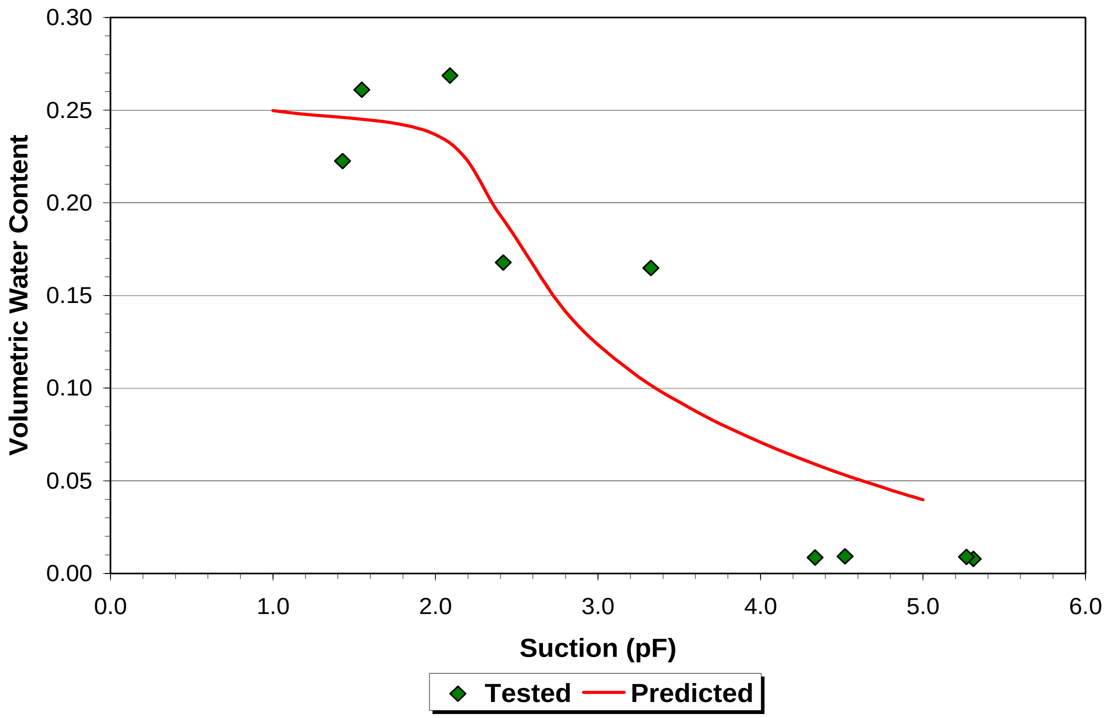


Figure 5.12. Soil Suction Data on Embankment Soil for Section 28040000.

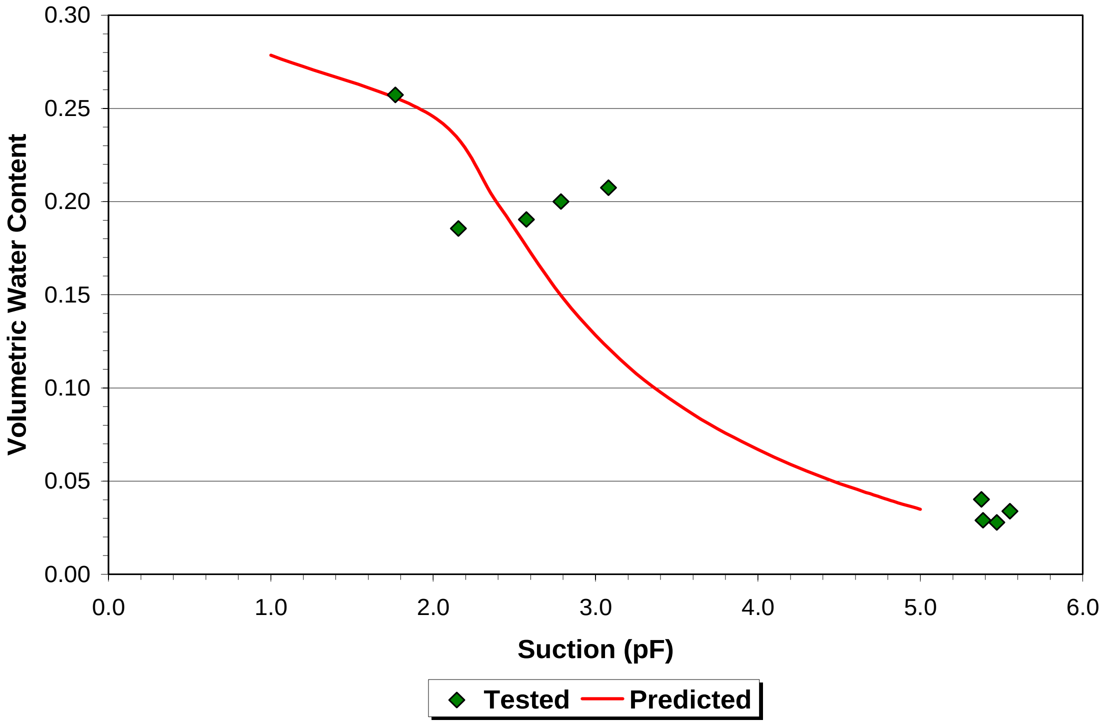


Figure 5.13. Soil Suction Data on Embankment Soil for Section 58060000.

CHAPTER VI. CONCEPTUAL PAVEMENT DESIGN GUIDE

This project established a conceptual framework for a new pavement design method based on the M-E PDG analysis program. The new pavement design guide based on this conceptual framework will be developed using Version 1.0 of the M-E PDG program that is expected to be released within the timeframe of the supplement to the current project. This development will require calibration of pavement performance models using the observed performance data on the calibration sections along with the traffic, materials, and environmental inputs characterizing these sections. This chapter discusses the proposed calibration of the M-E PDG performance models and presents the conceptual framework for an M-E PDG-based pavement design guide.

CALIBRATION OF PERFORMANCE MODELS

For the purpose of calibrating the M-E PDG performance models using PCS data, it is necessary to have a method for converting the design guide performance predictions to equivalent pavement condition survey scores based on current FDOT practice for rating pavements. This section describes the proposed method for converting the M-E PDG performance predictions to flexible and rigid pavement distress scores for rutting, cracking, surface roughness, and faulting.

Conversion Methods for Flexible Pavements

Rutting

The deduct points for rutting are based on measured rut depths from the pavement condition survey. Since the M-E rutting model predicts the progression of rut depth with time or with increasing axle load applications, the conversion of M-E rutting predictions to FDOT rut scores is straightforward. Table 6.1 (taken from FDOT's 2003 flexible pavement survey handbook) shows how this conversion can be accomplished.

Smoothness

In Florida, the ride rating (RR) is calculated using the following procedure:

- The Ride Number (RN) is computed from profile data taken on each wheel path.
- The average of the RNs for the left and right wheel paths is determined.

Table 6.1. Defect Points Corresponding to Measured Rut Depths.

Rut Depth (in)	Rut Depth (mm)	Range (in)	Range (mm)	Defect Points
0	0	0.00 – 0.06	0.00 – 1.59	00
1/8	3.18	0.07 – 0.19	1.60 – 4.76	01
¼	6.35	0.20 – 0.31	4.77 – 7.94	02
3/8	9.53	0.32 – 0.44	7.95 – 11.11	03
½	12.70	0.45 – 0.56	11.12 – 14.29	04
5/8	15.88	0.57 – 0.69	14.30 – 17.46	05
¾	19.05	0.70 – 0.81	17.47 – 20.64	06
7/8	22.23	0.82 – 0.94	20.65 – 23.81	07
1	25.40	0.95 – 1.06	23.82 – 26.99	08
1 1/8	28.58	1.07 – 1.19	27.00 – 30.16	09
1 ¼ +	31.75	1.20 +	30.17 +	10

- The ride rating for the PCS segment is obtained by multiplying the average RN by two.

The ride rating reported in the PCS data base ranges from 0 to 10 where 10 indicates a pavement that is perfectly smooth. In the M-E design guide program, the progression of pavement roughness over the life of the pavement is predicted in terms of the International Roughness Index (IRI). Thus, researchers determined a relationship between IRI and RN for converting the M-E design guide roughness predictions to the ride ratings reported in FDOT's PCS data base.

Figure 6.1 illustrates the relationships determined by researchers using profile data collected in ride surveys conducted for the Texas Department of Transportation (TxDOT) in 1999 and 2000 by Walker and Fernando (2002). Two equations are shown. The top equation is based on smoothness statistics computed from the individual wheel path profiles, while the bottom equation is based on the average of the wheel path IRIs and the ride number computed from both wheel path profiles following ASTM E1489-98. It is observed that the equations are nearly identical and provide an acceptable fit to the smoothness statistics computed from the measured profiles. The average of the absolute differences between the predicted and computed RNs is about 0.2.

For the purpose of converting the M-E design guide IRI predictions to equivalent FDOT ride scores, one approach that may be followed is to use the bottom equation given in Figure 6.1 to estimate an equivalent ride number from the IRI predicted at a given time. An

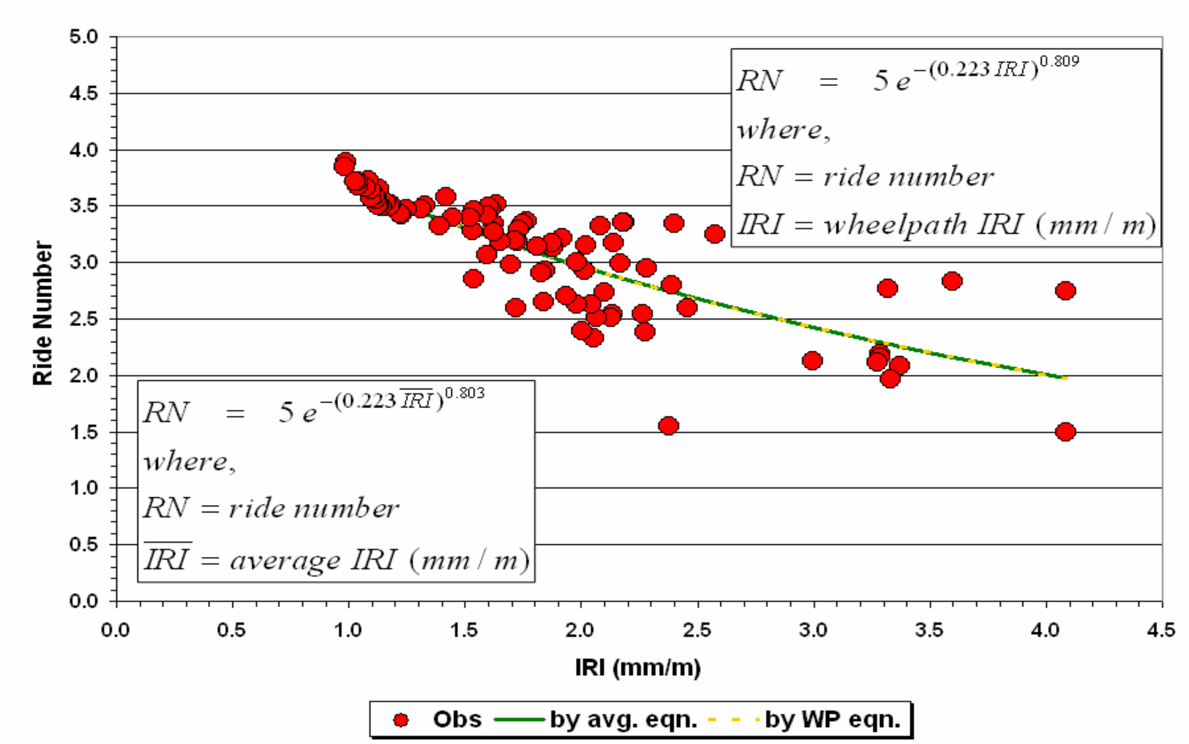


Figure 6.1. Relationships between IRI and Ride Number.

estimate of the equivalent FDOT ride score can then be obtained by multiplying the ride number by two. The researchers propose to take this approach. Alternatively, the IRIs determined for the calibration sections may be used but the available data are limited compared to the ride scores, which represent the standard method for reporting pavement smoothness in FDOT's PCS database.

Cracking

Current practice within FDOT considers the amount of cracks and the crack severity in rating the condition of flexible pavement sections. According to the pavement condition survey handbook (FDOT, 2003), three different types of cracking are considered based on crack severity:

- Class 1B are hairline cracks that are less than or equal to 1/8-inch wide in either the longitudinal or transverse direction.
- Class II cracks between 1/8-inch to ¼-inch wide in either the longitudinal or transverse direction. These cracks may exhibit moderate spalling or severe branching, and include alligator cracks where the crack width is below ¼-inch.

- Class III cracks are greater than ¼-inch wide that extend in a longitudinal or transverse direction and include cracks that have opened to the base or underlying material. Raveling and patching are also classified under Class III cracking.

Illustrations of the different types of cracking are given in the flexible pavement condition survey handbook. Only significant cracking is considered. Isolated areas of cracking are not measured if these are not considered representative of the section to be rated, based on the surveyor's engineering judgment.

Table 6.2 gives the deduct values for determining the crack score based on the area affected by cracking and the crack severity. The Florida DOT weighs cracks confined to the wheel paths more compared to cracks outside of the wheel paths as may be inferred from the higher deduct values assigned to wheel path cracking. An example is given below the table on how a crack defect rating is computed based on the measured cracked areas and severity. Note that Class 1B cracks are assumed to affect an area 1-ft wide. Class II and Class III cracks are considered rectangular, and the total square feet of pavement affected by these cracks is accumulated with the equivalent area of Class 1B cracking to determine the percent of surface area cracked. To get the deduct value corresponding to the percent of area cracked, only the predominant class of cracking is considered as shown in the example provided below Table 6.2.

Having reviewed the current practice of rating pavements based on cracking, the authors consider next the crack predictions from the M-E design guide program. In this program, three types of cracking are predicted for flexible pavements. These are longitudinal (top-down cracking), alligator (bottom-up cracking), and transverse (thermal cracking). Longitudinal and transverse cracking are expressed in units of ft/mile while alligator cracking is expressed in terms of percent of area cracked. Assuming that the area affected by linear cracks covers a width of 1 ft, the equivalent area covered by longitudinal and transverse cracks may be determined, which can then be combined with the area affected by alligator cracking to come up with an equivalent total area of cracked surface. However, in order to determine the applicable deduct value from Table 6.2, the crack severity must be known. This requirement presents a dilemma since the M-E design guide program does not predict crack severity. Thus, the predominant class of cracking cannot be identified. To come up with a method for converting the crack predictions from the M-E PDG program to equivalent pavement crack scores, researchers initially investigated whether a relationship exists

Table 6.2. Numerical Deductions for Determining Crack Score.

% of Pavement Area Affected by Cracking	CONFINED TO WHEEL PATHS (CW) PREDOMINANT CRACKING CLASS					
	IB CRACKING		II CRACKING		III CRACKING	
	CODE	DEDUCT	CODE	DEDUCT	CODE	DEDUCT
0 ~ 5	A	0.0	E	0.5	I	1.0
6 ~ 25	B	1.0	F	2.0	J	2.5
26 ~ 50	C	2.0	G	3.0	K	4.5
51~	D	3.5	H	5.0	L	7.0

% of Pavement Area Affected by Cracking	OUTSIDE OF WHEEL PATHS (CO) PREDOMINATE CRACKING CLASS					
	IB CRACKING		II CRACKING		III CRACKING	
	CODE	DEDUCT	CODE	DEDUCT	CODE	DEDUCT
0 ~ 5	A	0.0	E	0.0	I	0.0
6 ~ 25	B	0.5	F	1.0	J	1.0
26 ~ 50	C	1.0	G	1.5	K	2.0
51~	D	1.5	H	2.0	L	3.0

Notes: Percentages for CW and CO are estimated separately, each representing 100% of its respective area.

- Cracking classes cannot be combined. Only the predominant type of cracking is coded.
- The deduct value is assigned based on the total amount of cracking (accumulated area exhibiting Class IB, Class II and Class III cracks) and the predominant crack category
- Example: Given 1B = 10%, II = 6%, III= 6% Total = 22%
  Predominant class is 1B with the cracked area in the 6 ~ 25% range (code B)
  Crack Defect Rating = 10 – (CW + CO) = 10 – (1.0 + 0.5) = 8.5

between the area of cracking and the crack severity. For example, it might be realistic to assume that as more cracking develops, cracks progress from Class IB to Class II and finally to Class III. To investigate this relationship, the Florida DOT provided researchers with detailed crack data on a number of PCS segments. However, this investigation showed that no direct relationship exists between the crack severity and area of cracking, at least for the PCS data analyzed. In view of this finding, researchers propose that the calibration of the load-associated crack models be based on the combined area of longitudinal (top-down) and alligator (bottom-up) cracking. For this calibration, the detailed PCS crack data on each section need to be compiled in order to estimate the combined area of cracking reported for all three crack severity levels. The objective of the calibration will then be to achieve the best agreement between the predicted area of cracking from the M-E PDG program and the corresponding cracked area from the PCS database.

Conversion Methods for Rigid Pavements

This section describes the method to convert performance predictions for jointed plain concrete pavements (JPCPs) to equivalent pavement condition survey scores based on current FDOT practice for rating rigid pavements. The M-E design guide program predicts the following distress types for JPCPs:

- Defect (transverse cracking and faulting)
- IRI

Florida pavement condition survey rates pavements with respect to ride and defect which includes individual cracks, faulting, pumping, patching etc.

Transverse Crack Rating

For JPCP cracking, both bottom-up and top-down modes of cracking are considered because any given slab may crack either from the bottom-up or top-down mode (NCHRP 1-37, Part 3, 2004). The percentage of slabs with transverse cracks in a given traffic lane is used as the measure of transverse cracking and is predicted using the following model for both modes of cracking:

$$CRK = \frac{1}{1 + FD^{-1.68}} \qquad (6.1)$$

where,

CRK = predicted amount of bottom-up or top-down cracking (fraction), and

FD = fatigue damage.

The FDOT pavement management office provided detailed rigid pavement condition survey data on a number of calibration sections proposed in this project. The data give the number of transverse cracks corresponding to the light, medium, and severe categories along a given section. Knowing the section length and joint spacing, the number of slabs can be calculated. Since the M-E PDG program predicts the percent of cracked slabs, the corresponding quantity need to be estimated from FDOT's detailed PCC crack survey data. Since the number of cracked slabs is not reported from the survey, the project manager suggested assuming that each slab has one crack. Knowing the number of cracks and the number of slabs on a given PCC calibration section, the percent of cracked slabs can be determined. Herein, shattered slabs will be included in determining this distress quantity. Researchers plan to follow this conversion method for calibrating the PCC crack model in the M-E PDG program. However, at the suggestion of the project manager, researchers plan to view the video logs available on FDOT's intranet to verify the proposed methodology for estimating the percent of cracked slabs from the PCS data.

Faulting

The pavement survey data include a Fault Index (FI) that represents the average faulting for a rated section in thirty-seconds of an inch. For example, an FI of 3 indicates 3/32-inch of faulting, which corresponds to a 3 point negative defect. Since the M-E PDG program predicts faulting in inches, the conversion is straightforward.

Ride Rating

The options presented earlier for converting ride ratings on flexible pavements will also be considered in converting the ride ratings on the PCC calibration sections.

Performance Data to Use for Calibration

Using the proposed conversion methods with the input parameters from field and laboratory tests, researchers plan to initially run the M-E PDG program to predict the performance of the calibration sections using the national calibration factors built into the program. Depending on the results, the existing calibration factors will be revised as necessary to get the best agreement with the performance data on the calibration sections. One factor that needs to be considered in this task is the number of years of survey data to use for the calibration. Considering the current practice for programming projects within the Florida DOT, researchers are of the opinion that the necessary objective in the calibration is to accurately predict pavement service life as opposed to matching the observed performance history on a given section. Thus, researchers plan to put more weight on the last two to three years of a section's pavement life cycle to perform the model calibration in the supplemental project.

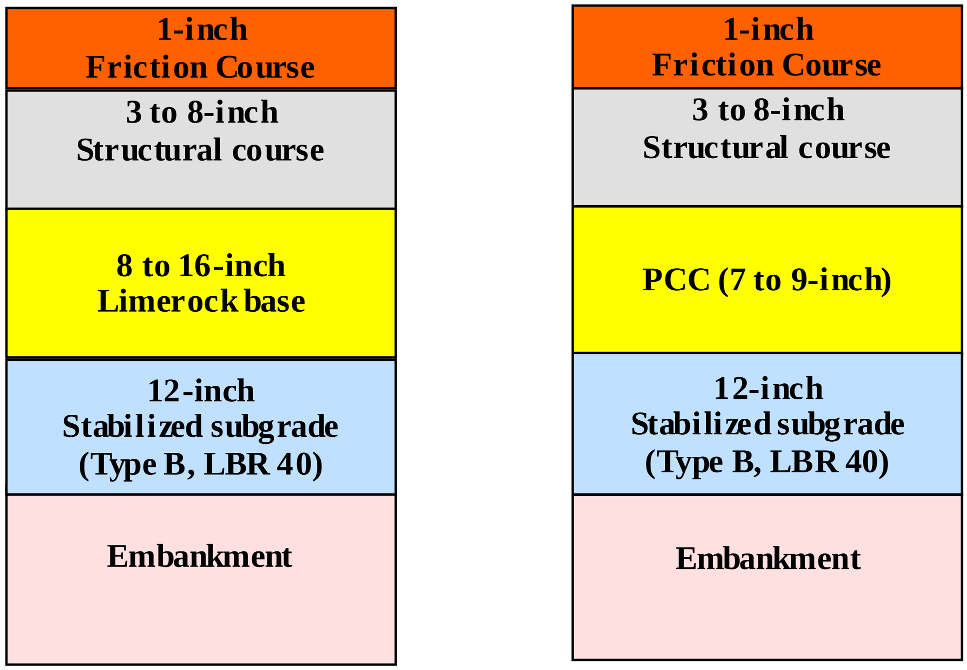
CONCEPTUAL FRAMEWORK FOR PAVEMENT DESIGN GUIDE

This section presents the proposed conceptual framework for developing the initial pavement design guide based on the M-E PDG. In establishing this framework, researchers reviewed the current design procedures for flexible and rigid pavements implemented by the Florida DOT. From this review, researchers identified flexible and rigid pavement cross-sections for developing a Florida DOT M-E PDG-based pavement design guide. In view of the large number of design variables that are input to the M-E PDG program, the new design guide may need to be packaged into a computer program for implementation. However, following the recommendation of the Project Manager, researchers plan to provide manuals of design tables and/or charts derived from runs of the M-E PDG program to implement the new guide in the same format as Florida's existing design methods. In view of the large number of design variables, and to keep the amount of M-E PDG program runs to a feasible number, researchers are considering the option of generating a database of pavement performance predictions from multiple runs of the M-E PDG program. These runs shall follow a factorial experiment that covers flexible and rigid pavement sections considered representative of Florida conditions and practice. An evaluation will then be made of relationships between predicted pavement life and pavement design variables using the database generated from the M-E PDG runs. These relationships would provide a viable alternative to developing the design tables and/or charts for the new Florida DOT pavement design guide, particularly for flexible pavements where there are more design factors that significantly influence the M-E PDG performance predictions (compared to rigid pavements).

Flexible Pavements

Figure 6.2 shows the general flexible pavement cross-sections researchers plan to analyze using the M-E PDG program. Structure 1 covers both new flexible pavements and asphalt concrete overlays of existing flexible pavements. Structure 2 covers AC overlays of existing JPCP sections. Multiple pavements classifying into the two pavement cross-sections illustrated in Figure 6.2 will be defined by various combinations of the design variables included in the factorial experiment for developing the new design guide. Table 6.3 identifies the design variables researchers plan to include in establishing the factorial experiment for the M-E PDG runs to be made.

In practice, the structural course layer usually comprises several lifts. To model each lift, users need to input the dynamic modulus per lift, or alternatively, the gradation, air void content, effective binder content, and the asphalt temperature-viscosity relationship to permit the program to calculate the dynamic modulus using Equation 2.1. Researchers realize that modeling each lift might considerably increase the number of M-E PDG runs that need to be made to develop the design tables. The increase will depend on the number of realistic combinations of the different lifts that are found in practice.



<Structure 1> <Structure 2>

Figure 6.2. General Flexible Pavement Cross-Sections.

Table 6.3. Proposed Design Factors for Generating Database of M-E PDG Pavement Performance Predictions.

Design Variable	New Asphalt Pavement	Asphalt Overlay	Jointed Plain Concrete Pavement
Climatic condition	✓	✓	✓
Traffic	✓	✓	✓
Reliability	✓	✓	✓
Surface modulus	✓	✓	✓
Surface thickness	✓	✓	✓
Base modulus	✓	✓	
Base thickness	✓	✓	
Subgrade modulus	✓	✓	
Embankment modulus	✓	✓	
Existing pavement condition		✓	
Milling depth		✓	
Coefficient of thermal expansion			✓
Compressive strength			✓

For overlay design, the milling depth and existing pavement condition are critical factors in determining the required overlay thickness. The current FDOT flexible pavement design guide characterizes the existing asphalt layer as good, fair, and poor depending on the pavement condition as evaluated from PCS survey data. The M-E PDG program has a similar option wherein the existing pavement is classified as excellent, good, fair, poor, and very poor. Additionally, it allows users to input milling depths from 0 to 4 inches. These features of the program can directly be used to develop the design procedure for asphalt concrete overlays.

The base thickness significantly affects the predicted performance of flexible pavements from the M-E PDG program. As shown in Figure 6.2, researchers plan to vary the base thickness from 8 to 16 inches at 2-inch increments. The moduli of the materials underlying the asphalt layer will be established so as to cover the range of resilient moduli from laboratory tests conducted on underlying materials commonly found in Florida flexible pavements. Consistent with the recommendation made by the project manager on characterizing the resilient modulus from laboratory tests for model calibration, the M-E PDG runs will cover the range of laboratory resilient modulus data on Florida base, subgrade, and embankment materials. Researchers are considering three levels of resilient modulus for each unbound material.

Besides material properties, the climatic condition, traffic characteristics, and design reliability are required inputs to the M-E PDG program. From the characterization of climatic and soil variations across Florida (Chapter IV), researchers identified four climatic regions and established representative soil-water characteristic curves for all counties comprising the state. Considering the geographic locations of the different Florida Districts, Table 6.4 identifies the climatic region in which each District belongs. This table also shows representative ground water table depths based on information obtained from soil borings and the proposed representative weather station for each District. Thus, given the location of a project, researchers envision that the new guide will provide a pavement design based, among other factors, on the climatic condition, ground water table depth, and soil-water characteristic curve considered representative of the specified project location.

Table 6.4. Grouping of Districts by Climatic Regions.

District	Climatic region	Weather station	Ground water table depth (ft)
1	1	Fort Myers	8
2	1	Gainesville and Jacksonville	10
3	2	Pensacola and Tallahassee	30
4	3	Palm Beach	8
5	1	Orlando	10
6	3	Miami and Key west	5
7	4	Tampa and St.Petersberg	5

The M-E pavement design guide also incorporates reliability in the design process just like the existing Florida pavement design method for flexible (2005) and rigid pavements (2006). However, reliability is defined differently between the two methods. In the existing Florida design method, reliability is the probability that the actual number of ESALs to a specified terminal serviceability level would be less than the predicted number. On the other hand, M-E PDG defines reliability as the probability that a particular distress type will be less than a specified critical level over the given design period. Since the existing procedure already incorporates design reliability, researchers expect that adapting the M-E PDG definition in the new pavement design method for Florida would be relatively straightforward.

With respect to traffic, the M-E PDG program uses the axle load distribution as input in lieu of the cumulative 18-kip equivalent single axle loads (ESALs) used in the current FDOT pavement design procedures. However, the authors note that M-E PDG also calculates the cumulative 18-kip ESALs given the annual average daily traffic (AADT), percent trucks, and traffic growth rate during the design period. For the model calibrations to be conducted in the supplemental project, estimates of axle load distributions on the calibration sections are expected to be provided from another study funded by FDOT.

Figure 6.3 illustrates conceptually the process for determining design thicknesses for new flexible pavements and asphalt concrete overlays in the M-E PDG-based pavement design guide expected to be developed in the supplement to this project.

Rigid Pavements

Similar to the framework for developing the flexible pavement design guide, researchers propose to use the M-E PDG program to predict the performance of the general rigid pavement cross-sections depicted in Figure 6.4. Since the predicted performance of

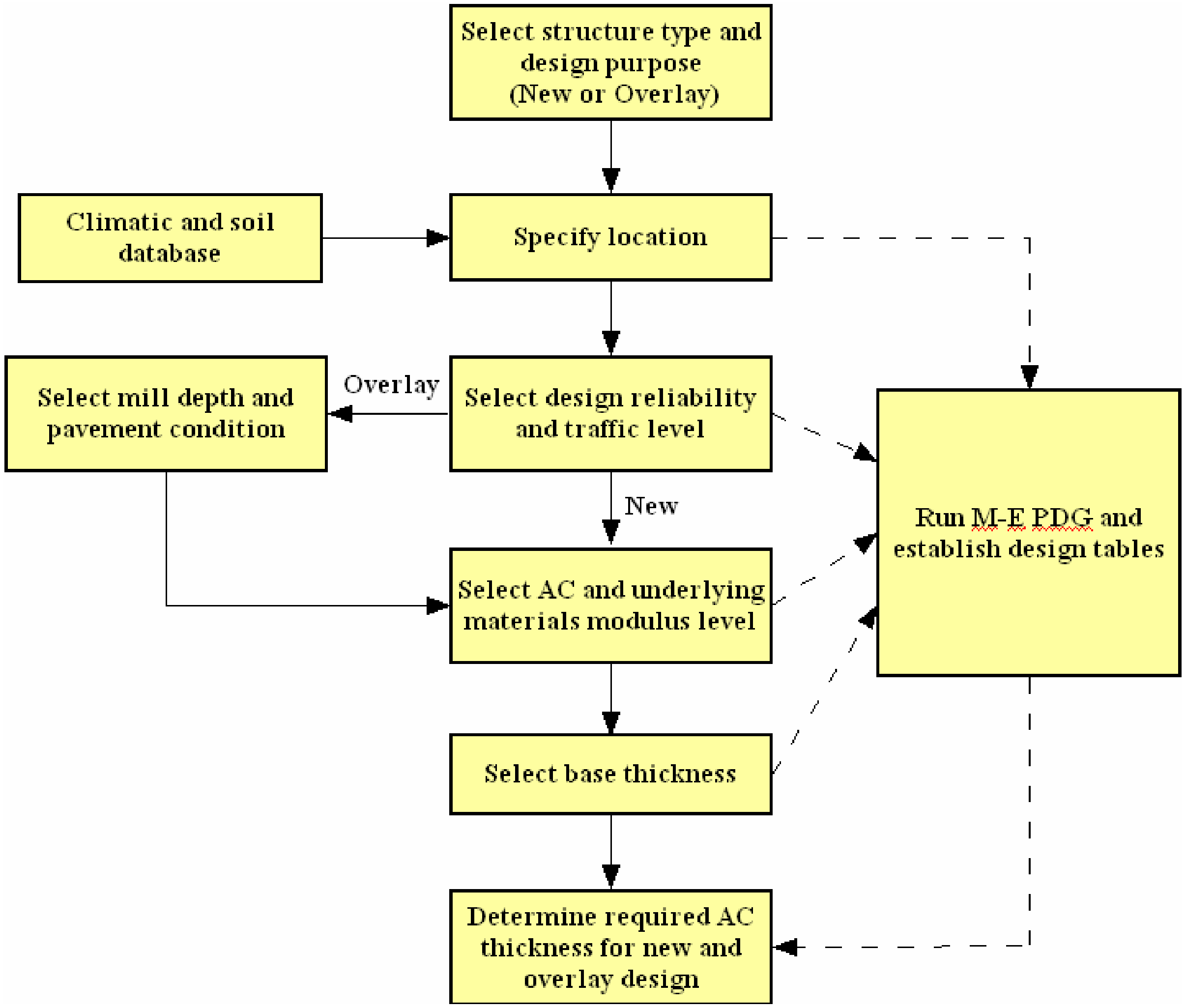


Figure 6.3. Conceptual Schematic of the Proposed Flexible Pavement Design Method.

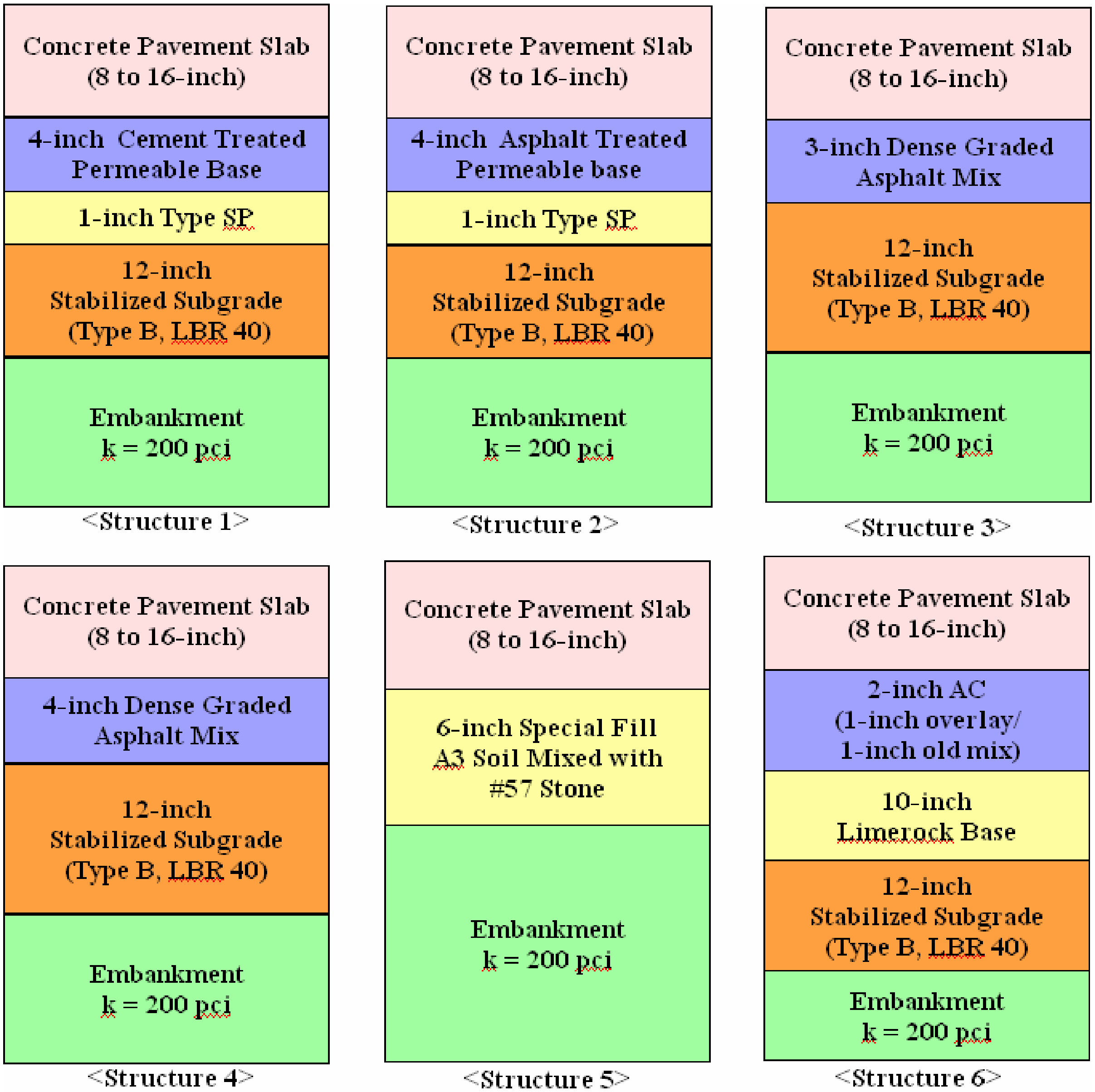


Figure 6.4. General Rigid Pavement Cross-Sections.

rigid pavements from the M-E PDG program is not significantly affected by the thickness and modulus of underlying materials unlike flexible pavements, researchers plan to consider only the most typical thickness and the most representative modulus for the base, subgrade, and embankment layers underneath the PCC slab. Note from Figure 6.4 that researchers plan to use a value of 200 pci for the modulus of subgrade reaction *k*, which is the recommended value for design of PCC pavements in current FDOT practice. Researchers are of the opinion that using only the typical *k*-value to develop the PCC damage equations is appropriate based on the findings from the sensitivity analyses that showed the subgrade modulus of reaction as having an insignificant effect on the predicted pavement performance from the M-E PDG program for the range of *k*-values representative of Florida embankment materials.

The most critical factors controlling predicted rigid pavement performance are concrete material properties, principally the thermal coefficient of expansion and compressive strength. For generating the database of performance predictions to develop the PCC pavement design guide, researchers plan to vary these properties over the range of values found in practice, based on laboratory test results and related material specifications implemented by the Florida DOT.

In addition to layer composition, the sensitivity analyses identified joint spacing and dowel diameter as significant predictors of PCC pavement performance. Table 6.5 shows recommended joint spacings and dowel diameters for various levels of slab thickness based on the Florida DOT rigid pavement design manual (2006). Researchers plan to use the values shown as inputs to generate the database of M-E PDG performance predictions.

For rigid pavements, design tables corresponding to reliability levels of 75, 80, 85, 90, 92, 94, and 95 percent are provided in the FDOT design manual for a specified design period of 20 years. Researchers plan to consider reliability levels covering this range in developing the new PCC pavement design guide based on the M-E PDG program.

In terms of traffic data, the axle load distributions are also specified for rigid pavement design. Likewise, the same two levels of user input (Level 1 and Level 3) are available for characterizing the design traffic in the M-E PDG program.

Figure 6.5 illustrates conceptually the process for determining design thicknesses for PCC pavements in the M-E PDG-based pavement design guide expected to be developed from the supplement to this project.

Table 6.5. Recommended Joint Spacings and Dowel Diameters for Varying Slab Thicknesses.

Slab thickness (in)	Joint spacing (ft)	Slab thickness (in)	Dowel diameter (in)
6	12	6	0.75
6.5	13	7 ~ 8.5	1.00
7	14	9 ~ 10.5	1.25
≥7.5	15	≥11	1.50

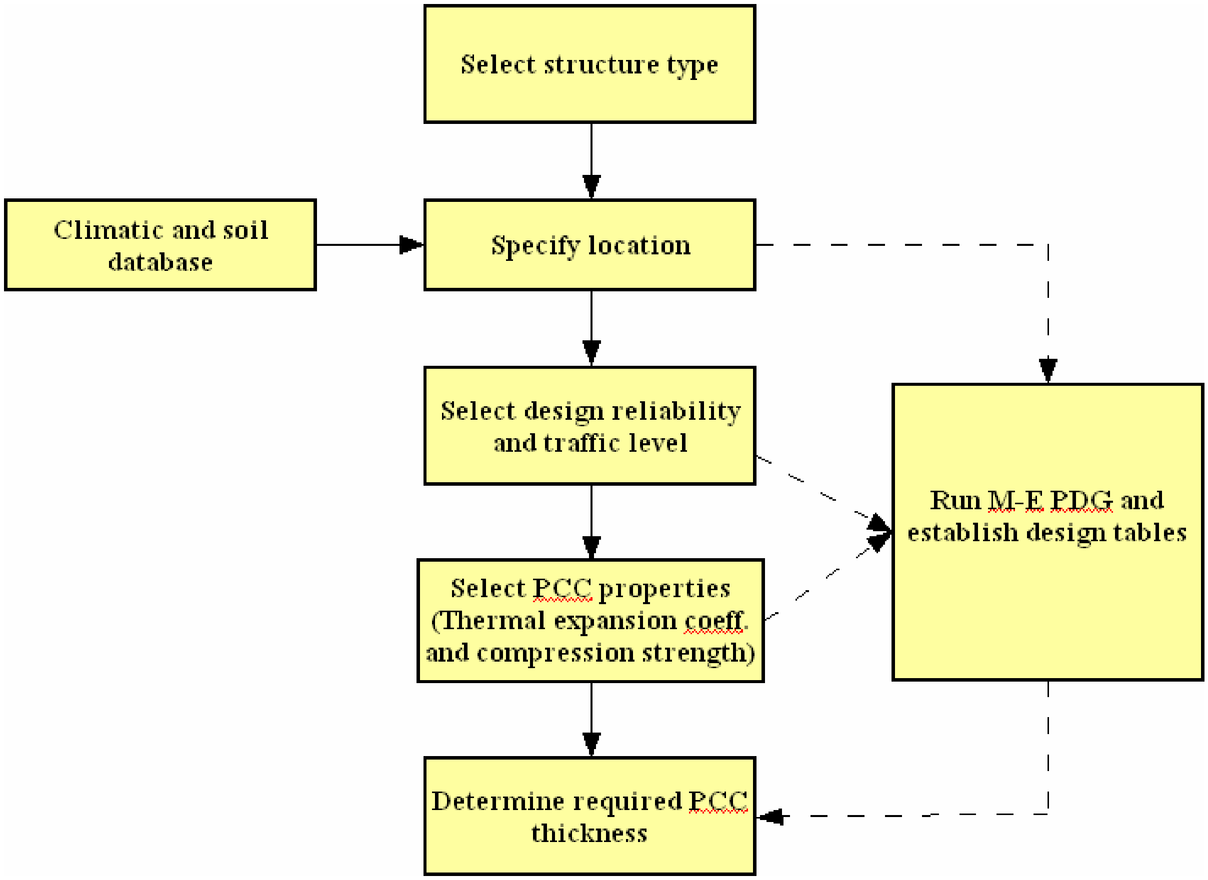


Figure 6.5. Conceptual Schematic of the Proposed Rigid Pavement Design Method.

CHAPTER VII. SUMMARY OF FINDINGS AND RECOMMENDATIONS

The primary objectives of this project are to provide a database for verifying and calibrating, as necessary, the performance models in the existing M-E PDG program and to establish a conceptual framework for a new Florida pavement design method based on the M-E PDG. To accomplish these tasks, researchers executed a comprehensive work plan that included the following:

- examination of Florida's pavement condition survey (PCS) database to identify in-service pavement sections for model calibrations,
- sensitivity analyses to identify critical factors affecting predicted pavement performance from the M-E PDG program,
- characterization of climatic-soil variations across Florida,
- field and laboratory tests to characterize material properties of in-service pavement sections for model calibration,
- compilation of database for model calibration; and
- development of conceptual M-E PDG-based pavement design guide framework.

Based on the research conducted, the following finds are noted:

- From the review of M-E PDG input requirements and sensitivity analyses, researchers identified the laboratory and field tests needed to characterize material properties for calibrating the performance models in the design guide program. The test plan is based on characterizing design factors that were found to significantly affect predicted pavement performance from the M-E PDG program. These factors include mixture properties that determine the dynamic modulus of the asphalt concrete material, specifically, gradation, air voids content, effective binder content, and the asphalt viscosity-temperature relationship. The sensitivity analyses also identified properties of the underlying unbound layers in flexible pavements that significantly affect predicted pavement performance. These properties include the resilient moduli of the base, subgrade, and embankment materials, and the soil-water characteristic curve, that may be characterized from soil suction tests or estimated using prediction equations that relate soil suction parameters to gradation, soil moisture-density relationship, and Atterberg limits.

- For jointed plain concrete pavements, the sensitivity analyses identified the concrete thermal coefficient of expansion, and compressive strength as significant predictors of PCC pavement performance. On this project, these properties were characterized from laboratory tests done on concrete cores taken from in-service pavement sections established for model calibrations. In addition, joint spacing and dowel diameter were found to significantly affect the performance predictions from the M-E PDG program. However, the moduli of the underlying unbound materials as well as the modulus of subgrade reaction were found to have minimal effect on the PCC performance predictions.
- From the cluster analysis of climatic variables, researchers identified four climatic regions into which Florida may be subdivided. In addition, researchers established representative soil-water characteristic curves for the different Florida counties based on a comprehensive review of soil survey reports and published soil suction data. Verification of curves determined from this review found a reasonable agreement between the curves and data from soil suction tests conducted in this project. However, the correlation needs to be further verified with additional test data.
- M-E PDG runs made to evaluate the effect of ground water table depth showed that the effect of this factor on the performance predictions diminishes with depths greater than 20 ft. Thus, for the field tests done on the calibration sections, borings to determine the depth of the water table were made to a depth of 20 ft or until the water table was reached, whichever came earlier.

Given the above findings, researchers offer the following recommendations with respect to developing the initial M-E PDG-based pavement design method for the Florida DOT:

- Researchers recommend that the new pavement design guide based on the conceptual framework presented in Chapter VI be developed using Version 1.0 of the M-E PDG program that is expected to be released within the timeframe of the supplement to this project.
- In view of the large number of design variables that are input to the M-E PDG program, researchers identified an alternative approach for developing the new Florida DOT pavement design guide that uses damage equations based on the M-E PDG program. Researchers recommend that this option be considered in developing the M-E PDG-based pavement design guide, particularly if the task of generating the

design tables would entail far too many runs of the computer program so as to render it infeasible under the time frame of the supplemental project. Alternatively, the damage equations provide the option of implementing the new guide in the form of a pavement design program, an option that might have to be considered and adopted should the design manuals become too voluminous to implement in practice.

- For the purpose of model calibration, researchers recommend that detailed crack survey data be provided by FDOT so that the progression of cracking on the calibration sections can be expressed in terms of quantities that can be compared to the performance predictions from the M-E PDG program. The performance indicators needed for calibrating the load-associated crack models are the cracked area and percent of cracked slabs for flexible and rigid pavements, respectively.
- Since the top-down cracking model being developed in NCHRP Project 1-42A is not expected to be available within the time frame of the supplement to this project, researchers recommend that the supplemental project focus on calibrating the load-associated crack model in Version 1.0 of the M-E PDG program. Implementation of the top-down cracking model that will come out of NCHRP 1-42A can be addressed in a future FDOT project.
- Considering the current practice for programming projects within the Florida DOT, researchers are of the opinion that the model calibrations should primarily aim to achieve the best prediction of pavement service life in lieu of matching the observed performance history on a given section. Thus, researchers recommend that more weight be given to the last two to three years of a section’s pavement life cycle, even to the extent of using only the last two to three data points for the model calibrations.

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APPENDIX A

CANDIDATE PCS SEGMENTS FOR MODEL CALIBRATIONS

Table A1. Candidate AC Segments.

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (mi)	P	C	I	U
Arterial District 1																	
12020000	4.35	5.13	L	6.5	6.5	7.2	9.0	0.20	0.13		0.20	9917	14.20		x		
12020000	4.35	5.13	R	6.5	6.5	6.8	9.0	0.25	0.14	0.27	0.12	9917	14.20		x		
16110000	15.39	15.74	R	6.5	7.0	6.5	8.0	0.16	0.49	0.06		9927	15.46			x	
16030000	29.06	29.69	L	6.5	7.0	6.5	7.0	0.29	0.63	0.23	0.13	9927	16.66				x
16180000	15.54	16.49	L	6.5	6.5	8.0	8.0	0.43	0.46		0.09	9927	22.64				x
16250000	4.62	7.38	C	6.5	6.5	7.1	8.0	0.22	0.58	0.10		9927	22.75			x	
04010000	6.19	13.16	C	6.5	6.5	7.8	8.0	0.10	0.37		0.12	9917	24.80	x			x
04040000	11.80	12.11	C	6.5	6.5		9.0	0.12	0.21	0.23		9917	27.76	x			
13020000	4.55	6.67	L	6.5	6.5	6.9	8.0	0.12	0.78	0.08		9926	30.66	x			
13140000	0.00	10.74	C	6.5	6.5	7.7	8.0	0.28	0.91		0.10	9926	34.07				x
91070000	0.00	8.11	C	6.5	6.5	7.9	7.0	0.13	0.87		0.15	9918	34.76	x			
13160000	6.27	9.73	C	6.5	6.5	8.0	8.0	0.13	0.55		0.42	9926	36.77	x			
13160000	9.73	15.57	C	6.5	6.5	8.0	8.0	0.13	0.59		0.24	9926	37.59	x			
09060000	0.36	3.53	C	6.5	6.5	7.7	8.0	0.15	0.48	0.09	0.17	9917	37.61	x			
13010000	2.96	5.33	L	6.6	9.0	6.6	9.0	0.10	0.08	0.10		9926	38.51	x		x	
06010000	11.91	13.54	C	6.5	6.5	7.9	9.0	0.13	0.27	0.09	0.20	9927	38.67	x			
03010000	5.91	8.36	L	6.5	6.5	7.5	9.0	0.27	0.53		0.11	9917	43.60				x
17030000	1.20	2.54	L	6.6	8.0	6.6	9.0	0.16	0.09	0.18		9926	46.41		x		
03001000	3.14	4.70	R	6.5	6.5	7.2	9.0	0.09	0.24			9917	49.90	x			
03080000	0.00	2.91	C	6.5	6.5	7.1	8.0	0.11	0.39	0.08		9935	57.93	x			
03030000	0.00	1.12	L	6.5	7.5	6.5	8.0	0.14	0.13	0.14	0.13	9917	62.63	x	x	x	x
Arterial District 2																	
26010000	0.00	11.64	L	6.5	6.5	7.4	9.0	0.08	0.27	0.15	0.25	9904	0.56	x			
72017000	1.49	2.10	R	6.6	7.5	6.6	9.0	0.06	0.27	0.08	0.28	9914	5.96	x		x	
72170000	5.64	6.32	L	6.5	6.5	7.4	9.0	0.13	0.14	0.07		9914	6.11		x		

Table A1. Candidate AC Segments (continued).

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (mi)	P	C	I	U
Arterial District 2 (continued)																	
72170000	4.11	5.64	R	6.5	6.5	6.9	9.0	0.13	0.28	0.23		9914	6.98	x			
26070068	0.61	1.51	C	6.5	6.5	6.6	7.0	0.13	0.53		0.15	9904	8.53	x			
26010000	13.44	16.63	R	6.6	7.0	6.6	9.0	0.11	0.48	0.11		9904	9.08	x		x	
34110000	12.61	14.68	C	6.5	6.5	7.3	9.0	0.14	0.28	0.05	0.13	9909	9.35	x			
26005000	7.95	10.69	L	6.5	6.5	7.2	9.0	0.09	0.29	0.06		9904	10.68	x			
34040000	12.74	19.64	C	6.5	6.5	7.7	9.0	0.14	0.22	0.07		9904	10.91	x			
26005000	0.35	3.50	L	6.5	6.5	7.4	9.0	0.11	0.20	0.08		9904	12.22	x			
27010000	10.08	11.90	C	6.5	6.5	7.4	9.0	0.04	0.08	0.05		9936	13.86	x	x		
76050000	2.03	8.20	C	6.5	6.5	7.1	8.0	0.11	0.28	0.07		9904	21.31	x		x	
71050000	10.45	16.12	C	6.5	6.5	7.1	9.0	0.09	0.16	0.05		9905	21.61	x			
76080000	0.36	5.70	C	6.5	6.5	7.4	9.0	0.07	0.40	0.04	0.17	9904	22.94	x			
71110000	4.39	5.86	C	6.5	6.5	6.9	9.0	0.15	0.40	0.10		9904	23.53	x			
34050000	9.83	24.03	R	6.5	6.5	8.1	9.0	0.15	0.19		0.13	9909	24.61	x			
28040000	0.00	5.51	C	6.5	6.5	7.6	9.0	0.14	0.33	0.05		9936	25.02	x			
37040000	0.00	0.34	C	6.6	10.0	6.6	9.0	0.07		0.07	0.08	9909	28.03	x		x	
76010000	5.17	6.41	C	6.5	6.5	7.7	9.0	0.04	0.30	0.04	0.11	9925	29.84	x			
28030000	0.00	0.69	C	6.5	8.0	6.5	9.0	0.18	0.12	0.18		9904	31.55		x		
78090000	14.37	14.49	C	6.5	6.5		8.0		0.52		0.10	9905	31.83				x
76010000	29.28	30.37	R	6.5	6.5	6.7	7.0	0.09	0.56	0.09	0.11	9905	35.36	x		x	x
32010000	30.75	31.18	C	6.5	7.0	6.5	9.0	0.07	0.40	0.10		9901	40.30	x		x	
Arterial District 3																	
48003000	6.12	6.75	L	6.5	6.5	6.8	10.0	0.09	0.17	0.06		9924	2.69	x			
48003000	6.12	6.75	R	6.5	6.5	6.7	10.0	0.15	0.54	0.09		9924	2.69	x		x	
50010000	16.48	18.57	R	6.5	6.5	8.1	9.0	0.05	0.11	0.01	0.18	9940	3.34	x	x		
48012000	7.38	7.70	C	6.5	8.5	6.5	10.0	0.09	0.10	0.08		9924	3.69			x	
58010000	11.05	11.68	L	6.5	6.5	7.3	10.0	0.14	0.15	0.08		9937	4.48	x	x		
48020000	7.87	9.65	L	6.5	6.5	6.8	7.0	0.25	0.76	0.12	0.25	9924	4.70			x	

Table A1. Candidate AC Segments (continued).

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (ml)	P	C	I	U
Arterial District 3 (continued)																	
48020000	13.45	16.15	L	6.6	7.5	6.6	8.0	0.22	0.67	0.07	0.14	9924	4.94			x	
58010000	9.32	10.75	L	6.5	6.5	8.1	9.0	0.14	0.15	0.03		9937	5.43	x	x		
48010000	2.15	2.90	R	6.5	6.5	6.7	9.0	0.10	0.11	0.09		9916	7.59	x	x	x	
61002000	2.86	3.25	C	6.5	6.5	6.7	8.0	0.20	0.55	0.17	0.12	9939	11.46				x
50040000	0.81	3.05	R	6.5	6.5	7.4	9.0	0.05	0.38	0.02		9940	11.89	x			
61010000	13.33	16.33	C	6.5	6.5	7.5	8.0	0.16	0.13		0.15	9939	12.96		x		
61010000	17.55	19.08	L	6.5	6.5	6.6	9.0	0.17	0.58	0.02	0.08	9939	13.35			x	
46060000	9.10	11.96	C	6.5	6.5	7.8	8.0	0.11	0.61	0.03	0.08	9907	14.29	x			
58060000	20.72	21.80	C	6.5	6.5	7.8	9.0	0.09	0.12	0.03		9937	18.96	x	x		
46040000	0.00	1.12	C	6.5	6.5	6.7	8.0	0.09	0.26	0.07	0.07	9907	20.80	x			
57130000	0.00	1.33	L	6.5	6.5	8.4	8.0	0.62	0.73		0.08	9937	29.31				x
57130000	0.00	1.33	R	6.5	6.5	8.0	8.0	0.62	0.73		0.10	9937	29.31				x
Arterial District 4																	
86190000	2.33	3.67	R	6.5	6.5	7.3	9.0	0.08	0.08	0.05		9934	5.37		x		
86100000	1.53	2.73	R	6.5	6.5	6.7	8.0	0.11	0.41	0.06		9930	7.02	x		x	
86230000	0.00	2.61	R	6.6	7.5	6.6	9.0	0.06	0.26	0.06		9930	8.85	x		x	
94050000	0.00	2.78	C	6.5	6.5	6.9	8.0	0.17	0.51	0.10		9913	9.04			x	
86090000	6.00	8.64	R	6.5	6.5	6.5	8.0	0.08	0.54	0.05		9933	10.40	x		x	
94050000	2.78	8.75	C	6.5	6.5	7.1	9.0	0.10	0.72	0.09		9913	10.62	x			
93001000	3.88	5.81	L	6.5	8.0	6.5	9.0	0.20	0.08	0.20		9921	11.06		x		
86010001	0.00	2.55	L	6.5	6.5	6.7	9.0	0.10	0.32	0.08		9930	11.34	x			
93310000	7.99	12.22	L	6.5	6.5	8.0	8.0	0.14	0.41		0.31	9921	12.56	x			
93310000	12.22	17.80	L	6.5	6.5	7.7	8.0	0.08	0.23		0.12	9921	12.75	x			x
86012000	0.00	0.45	R	6.5	6.5	6.5	8.0	0.11	0.55	0.11		9933	13.00	x		x	
86020000	7.91	9.00	L	6.6	9.0	6.6	8.0	0.04	0.07	0.04	0.09	9933	13.02				x
86020000	7.91	9.00	R	6.6	7.5	6.6	8.0	0.04	0.14	0.04	0.10	9933	13.02		x		x
93100000	11.90	12.62	L	6.5	6.5	6.8	9.0	0.16	0.40	0.14		9935	13.56			x	

Table A1. Candidate AC Segments (continued).

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (ml)	P	C	I	U
Arterial District 4 (continued)																	
93200000	0.00	1.37	C	6.5	6.5	6.9	9.0	0.09	0.13	0.09		9933	24.96		x		
Arterial District 5																	
18010000	19.48	21.60	C	6.5	6.5	7.9	8.0	0.11	0.60		0.14	9931	3.07	x			x
79181000	0.38	2.86	R	6.5	6.5	8.1	8.0	0.46	0.88		0.11	9906	5.61				x
77030000	5.09	6.04	R	6.5	6.5	7.5	9.0	0.09	0.14	0.06	0.22	9906	6.06	x	x		
70060000	11.99	14.29	C	6.5	6.5	7.8	9.0	0.28	0.12			9919	6.90		x		
70060000	0.00	11.07	C	6.5	6.5	7.8	9.0	0.14	0.14			9919	7.47	x	x		
79070000	26.97	29.29	R	6.6	7.0	6.6	9.0	0.09	0.34	0.08		9929	7.54	x		x	
11010047	0.00	2.09	R	6.5	6.5	7.0	9.0	0.16	0.39	0.07		9931	7.58			x	
77010000	5.94	11.65	L	6.5	6.5	7.6	9.0	0.10	0.22	0.02	0.13	9906	9.59	x			x
11010000	6.04	9.70	R	6.5	6.5	7.8	8.0	0.15	0.12		0.15	9931	10.28	x	x		
79270000	1.62	2.39	L	6.5	8.0	6.5	10.0	0.10	0.09	0.10	0.14	9925	11.32		x		
77040000	5.81	11.05	C	6.5	6.5	7.4	7.0	0.11	0.33	0.03	0.08	9906	11.93	x			x
77010000	1.85	2.53	L	6.5	6.5	6.5	7.0	0.13	0.49	0.15	0.27	9906	15.67	x		x	
79080000	5.13	6.79	L	6.5	6.5	6.6	9.0	0.08	0.36	0.07	0.08	9925	16.16	x			
77040000	11.05	16.10	C	6.5	6.5	8.1	8.0	0.12	0.24	0.02	0.06	9906	16.84	x			
36009000	3.96	5.46	C	6.6	8.5	6.6	9.0	0.09	0.14	0.09	0.13	9931	22.88	x		x	
36110000	6.60	22.51	C	6.5	6.5	7.4	8.0	0.12	0.22	0.08	0.09	9904	25.02	x			x
36060000	4.79	11.93	C	6.5	6.5	8.3	8.0	0.14	0.37		0.10	9904	26.51	x			x
75003000	2.05	5.00	R	6.5	6.5	7.8	10.0	0.10	0.12	0.03		9906	26.63	x	x		
75003000	2.05	4.92	L	6.5	6.5	7.7	10.0	0.12	0.12	0.05		9906	26.67	x	x		
36080000	5.17	6.53	L	6.5	6.5	7.4	8.0	0.09	0.56	0.10	0.08	9904	26.82	x			
36080000	2.36	3.31	L	6.5	6.5	7.8	7.0	0.09	0.43	0.08	0.11	9904	27.04	x			x
36080000	2.36	3.31	R	6.5	6.5	7.4	7.0	0.13	0.60	0.08	0.12	9904	27.04	x			x
70070002	0.00	0.38	L	6.5	6.5	6.6	7.0	0.05	0.10	0.09		9919	32.32		x		
92030000	0.00	0.61	R	6.5	6.5	7.6	7.0	0.14	0.24	0.08	0.10	9927	40.68	x			X

Table A1. Candidate AC Segments (continued).

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (mi)	P	C	I	U
Arterial District 6																	
87170000	3.70	5.23	L	6.5	6.5	7.3	9.0	0.11	0.46			9930	1.34	x			
87060000	14.05	14.87	L	6.6	9.0	6.6	9.0	0.12	0.08	0.12	0.20	9930	2.37	x		x	
87090000	0.79	3.69	R	6.6	7.0	6.6	7.0	0.06	0.26	0.07		9934	2.74			x	
87030000	24.96	25.38	R	6.5	6.5	7.4	8.0	0.10	0.52		0.29	9930	4.10	x			
87038000	0.00	0.96	R	6.5	6.5	6.8	8.0	0.07	0.44	0.13	0.10	9934	4.21				x
87080000	0.69	2.68	R	6.5	6.5	6.8	9.0	0.15	0.11	0.14		9930	4.44		x	x	
87080001	2.29	3.08	C	6.6	10.0	6.6	9.0	0.05		0.05	0.08	9930	5.66	x		x	
87002000	7.55	9.70	L	6.5	6.5	7.5	9.0	0.12	0.22			9934	6.14	x			
87060000	0.87	2.72	L	6.5	6.5	7.5	8.0	0.20	0.43	0.06	0.10	9930	9.38				x
87220000	2.29	3.86	L	6.6	7.5	6.6	8.0	0.14	0.30	0.15	0.12	9934	9.74	x		x	x
87281000	2.62	3.86	L	6.5	6.5	6.5	9.0	0.15	0.18	0.15		9930	12.15	x		x	
90060000	13.03	16.38	C	6.5	6.5	7.4	9.0	0.08	0.18			9934	65.33	x			
90030000	6.13	7.10	C	6.5	6.5	7.5	10.0	0.10	0.19			9934	101.94	x			
Arterial District 7																	
10160000	0.00	6.77	R	6.5	6.5	7.4	8.0	0.21	0.32	0.04	0.11	9922	3.62				x
10340000	1.28	2.67	C	6.5	9.0	6.5	8.0	0.09		0.09	0.13	9922	6.32				x
10110000	0.00	0.44	R	6.6	8.5	6.6	9.0	0.07	0.15	0.07		9926	6.37	x		x	
14150000	0.00	1.79	C	6.5	6.5	7.3	8.0	0.07	0.17	0.05	0.11	9920	7.14	x			
10070000	4.28	4.58	L	6.5	6.5	7.5	9.0	0.05	0.58	0.01		9927	7.92	x			
14130000	0.48	0.80	C	6.5	6.5	7.5	9.0	0.11	0.76	0.02	0.08	9920	11.39	x			
10060000	8.91	17.43	R	6.5	6.5	7.9	9.0	0.15	0.09		0.11	9926	13.09	x	x		
15009000	1.81	2.32	C	6.5	6.5	6.9	9.0	0.11	0.26	0.13	0.25	9922	17.88	x		x	
15007000	0.00	3.06	L	6.6	8.0	6.6	9.0	0.05	0.33	0.05	0.10	9922	22.31	x		x	
15010000	1.34	4.50	R	6.6	7.0	6.6	8.0	0.03	0.50	0.03	0.12	9922	24.82				x
02030000	14.65	15.53	L	6.5	6.5	6.5	9.0	0.08	0.27	0.10	0.12	9931	36.43	x		x	x
15070001	0.00	0.48	L	6.6	10.0	6.6	8.0	0.06		0.06	0.11	N/A	N/A				x

Table A1. Candidate AC Segments (continued).

Road Identification				Road Condition				Mean RSS				WIM		Application			
RDWYID	BEG	END	S	PCR	CR	RI	RU	PCR	CR	RI	RU	SITE	Dist. (mi)	P	C	I	U
Interstate																	
79110000	14.67	25.27	L	6.5	6.5	8.3	8.0	0.29	0.83		0.11	9925	2.84				x
53002000	13.61	19.50	L	6.5	6.5	8.4	9.0	0.26	0.14		0.08	9943	7.44		x		
53002000	13.61	19.50	R	6.5	6.5	8.2	8.0	0.30	0.14		0.10	9943	7.44		x		x
36210000	22.50	23.51	L	6.5	6.5	7.1	7.0	0.07	0.41	0.02	0.06	9904	18.95	x			x
75280000	22.37	24.67	R	6.5	6.5	8.2	8.0	0.11	0.20	0.06		9906	19.36	x			
15170000	5.94	7.66	R	6.5	6.5	8.1	9.0	0.06	0.13		0.16	9926	28.01		x		
73001000	0.00	18.73	L	6.5	6.5	7.8	9.0	0.30	0.11		0.22	9925	29.56		x		
93220000	16.45	17.00	L	6.5	6.5	7.8	8.0	0.09	0.30			9933	29.76	x			
93220000	16.45	17.00	R	6.5	6.5	7.9	9.0	0.12	0.30			9933	29.76	x			
75280000	1.00	8.84	L	6.5	6.5	8.5	9.0	0.09	0.08			9920	35.02	x	x		
Turnpike																	
87470000	0.00	0.17	R	6.5	6.5	6.6	10.0	0.13	0.42	0.11		9930	3.61	x			
87470000	0.00	0.24	L	6.5	6.5	7.0	9.0	0.13	0.24	0.13		9930	3.64	x			
86470000	8.51	15.16	L	6.5	6.5	8.4	9.0	0.04	0.52			9933	6.10	x			
86470000	8.51	15.16	R	6.5	6.5	8.4	8.0	0.07	0.45			9933	6.10	x			
86470000	15.16	16.95	L	6.5	6.5	8.4	9.0	0.05	0.41			9933	6.26	x			
86470000	0.00	0.55	R	6.5	6.5	7.9	9.0	0.06	0.28			9930	6.30	x			
86470000	18.89	22.30	R	6.5	6.5	8.0	8.0	0.43	0.57		0.12	9933	9.88				x
87471000	14.37	17.37	L	6.5	6.5	7.5	9.0	0.16	0.24		0.10	9934	19.67				x

Table A2. Candidate PCC Segments.

Subsystem	Road Identification				Road Condition			Mean RSS			WIM		Application		
	RDWYID	BEG	END	S	PCR	CR	RI	PCR	CR	RI	SITE	Dist. (mi)	P	C	I
Arterial 2	72030000	9.43	10.39	R	6.4	7.3	6.4	0.12		0.12	9914	4.42			x
	72030000	7.46	8.71	R	6.8	8.4	6.8	0.11		0.13	9914	4.80	x		x
	72090000	8.23	11.14	R	6.1	8.7	6.1	0.13		0.13	9905	10.19	x		x
Arterial 5	79010000	9.60	11.46	L	6.6	9.4	6.6	0.07		0.07	9929	0.08	x		x
Arterial 6	87030000	8.83	10.10	L	6.5	6.5	6.7	0.03	0.05	0.02	9930	11.01	x	x	
	87030000	8.83	10.10	R	6.1	6.1	6.5	0.38	0.06	0.42	9930	11.01		x	
Arterial 7	10150000	12.59	12.84	L	6.3	8.0	6.3	0.56	0.08	0.56	9922	5.30		x	
Interstate	72280000	6.08	7.48	L	6.3	6.3	7.2	1.52	1.52	0.10	9905	2.24			x
	72280000	7.48	13.10	L	6.5	6.5	7.7	1.63	1.67	0.06	9905	5.62			x
	72001000	34.52	35.51	R	6.7	9.0	6.7	0.11		0.13	9914	9.65			x
	87270000	0.00	0.98	R	6.9	6.9	7.6		0.11		9930	11.47		x	
	15002000	0.00	0.16	R	6.3	7.0	6.3	0.60	0.04	0.60	9926	23.93		x	
	15003000	0.37	1.29	R	6.5	6.9	6.5	0.05	0.10	0.05	9926	24.03		x	
	15190000	2.29	4.43	R	6.3	6.3	7.2	0.08	0.36		9926	25.37	x		

Table A3. Candidate AC Segments with Long Service Lives.

Subsystem	RDWYID	BEG	END	S	PCR	CR	RI	RU	SITE	Dist. (ml)	Age
Arterial 1	03080000	0.00	2.91	C	6.5	6.5	7.1	8.0	9935	57.93	29
	05020000	6.38	8.29	C	6.5	6.5	7.5	8.0	9918	13.44	22
	05090000	2.66	12.44	C	6.5	6.5	7.1	9.0	9918	20.53	28
	13020000	4.55	6.67	L	6.5	6.5	6.9	8.0	9926	30.66	21
	13030000	0.51	1.91	R	6.5	6.5	6.7	8.0	9926	32.63	22
	13050000	22.39	27.12	C	6.5	6.5	7.5	7.0	9926	39.61	28
	13160000	6.27	9.73	C	6.5	6.5	8.0	8.0	9926	36.77	29
	13160000	9.73	15.57	C	6.5	6.5	8.0	8.0	9926	37.59	27
	16040000	11.27	13.59	C	6.5	6.5	7.2	9.0	9927	33.98	26
	16040000	13.99	15.06	C	6.5	6.5	7.0	9.0	9927	35.92	26
	16090000	6.26	12.37	C	6.5	6.5	7.0	8.0	9927	33.30	29
	17010000	4.30	7.85	R	6.5	6.5	7.4	8.0	9917	25.19	29
	17030000	2.54	2.80	L	6.5	6.5	6.7	8.0	9926	46.51	28
	91070000	0.00	8.11	C	6.5	6.5	7.9	7.0	9918	34.76	24
Arterial 2	26010000	12.54	13.18	R	6.5	6.5	7.3	10.0	9904	6.93	23
	26010000	13.44	16.63	R	6.6	7.0	6.6	9.0	9904	9.08	23
	26020000	3.25	6.07	C	6.5	9.5	6.5	9.0	9904	10.03	29
	26050000	0.62	3.91	L	6.5	6.5	7.2	8.0	9904	7.18	29
	26070068	0.61	1.51	C	6.5	6.5	6.6	7.0	9904	8.53	24
	26090000	15.77	15.96	L	6.5	6.5		9.0	9904	7.56	29
	26220000	9.19	11.10	L	6.5	6.5	6.7	9.0	9904	5.96	28
	27010000	10.08	11.90	C	6.5	6.5	7.4	9.0	9936	13.86	21
	32010000	30.75	31.18	C	6.5	7.0	6.5	9.0	9901	40.30	23
	34070000	9.30	16.54	C	6.5	6.5	7.6	9.0	9909	20.18	29
	38030000	2.34	3.98	C	6.5	6.5	7.3	9.0	9901	27.64	28
	38030000	3.98	4.64	C	6.5	6.5	7.1	8.0	9901	28.64	28
	72018000	1.99	2.84	L	6.5	6.5	7.1	10.0	9914	5.85	23

Table A3. Candidate AC Segments with Long Service Lives (continued).

Subsystem	RDWYID	BEG	END	S	PCR	CR	RI	RU	SITE	Dist. (ml)	Age
Arterial 2	72050000	13.95	15.74	L	6.5	6.5	6.5	9.0	9914	7.81	29
	72080000	7.83	8.44	R	6.5	6.5	8.3	10.0	9914	3.70	26
	72170000	0.00	0.70	L	6.5	6.5	6.8	9.0	9914	11.38	29
	74130000	0.81	3.09	C	6.5	6.5	6.9	9.0	9914	22.61	27
	76050000	0.00	2.03	C	6.5	6.5	7.4	9.0	9904	17.29	23
	78030000	1.95	18.89	C	6.5	6.5	6.7	7.0	9905	15.55	28
	78090000	14.37	14.49	C	6.5	6.5		8.0	9905	31.83	26
Arterial 3	48020000	12.08	13.03	C	6.5	6.5	6.6	8.0	9924	5.00	29
	55020000	0.51	1.13	L	6.5	7.0	6.5	9.0	9908	4.93	26
	55020000	0.51	1.13	L	6.5	7.0	6.5	9.0	201	4.93	26
	57050000	0.00	1.25	L	6.5	6.5	6.5	8.0	9928	21.41	27
Arterial 4	86006000	0.00	2.14	R	6.6	9.0	6.6	8.0	9933	5.98	29
	86018000	0.00	1.21	R	6.6	9.0	6.6	8.0	9930	7.98	27
	86020000	3.45	4.16	R	6.5	6.5	6.7	9.0	9933	11.86	29
	86020000	7.91	9.00	L	6.6	9.0	6.6	8.0	9933	13.02	22
	86020000	7.91	9.00	R	6.6	7.5	6.6	8.0	9933	13.02	22
	86100000	1.53	2.73	R	6.5	6.5	6.7	8.0	9930	7.02	24
	86100000	13.84	14.78	R	6.5	6.5	7.0	9.0	9933	6.38	23
	86230000	3.44	3.64	R	6.5	10.0	6.5	9.0	9930	8.85	27
	93006000	1.28	2.08	L	6.5	7.5	6.5	8.0	9921	26.64	29
	93010000	2.87	5.10	L	6.6	8.0	6.6	9.0	9933	19.80	26
	93012000	0.00	0.37	R	6.6	9.0	6.6	9.0	9921	14.78	26
	93012000	1.96	2.86	L	6.6	7.0	6.6	8.0	9921	14.99	26
	93030000	7.16	8.27	L	6.5	6.5	6.7	9.0	9933	23.89	26
	93040000	0.84	2.19	L	6.6	7.0	6.6	9.0	9921	12.49	29
	93120000	19.75	20.33	L	6.5	6.5	6.5	9.0	9921	22.21	29
	94050000	0.00	2.78	C	6.5	6.5	6.9	8.0	9913	9.04	26
	94050000	2.78	8.75	C	6.5	6.5	7.1	9.0	9913	10.62	24

Table A3. Candidate AC Segments with Long Service Lives (continued).

Subsystem	RDWYID	BEG	END	S	PCR	CR	RI	RU	SITE	Dist.	Age
Arterial 5	11080000	1.52	4.82	C	6.5	6.5	7.4	8.0	9931	14.94	28
	18010000	28.74	30.26	C	6.5	6.5	7.6	8.0	9931	10.41	22
	36060000	4.79	11.93	C	6.5	6.5	8.3	8.0	9904	26.51	22
	36110000	6.60	22.51	C	6.5	6.5	7.4	8.0	9904	25.02	24
	70002000	0.24	2.09	R	6.6	7.0	6.6	9.0	9929	21.34	26
	70004000	2.86	4.59	L	6.5	6.5	6.7	8.0	9919	17.98	28
	70020000	9.45	9.82	R	6.6	7.0	6.6	8.0	9919	17.64	24
	75003000	5.00	7.30	R	6.5	6.5	6.6	8.0	9906	24.02	29
	75011000	0.66	2.49	L	6.5	6.5	6.9	8.0	9906	18.21	26
	75080000	17.42	17.66	C	6.5	9.0	6.5	9.0	9906	24.48	29
	75250000	4.81	7.27	L	6.5	8.0	6.5	8.0	9906	22.94	24
	77010000	16.53	16.93	R	6.5	9.5	6.5	9.0	9906	4.57	23
	77161000	0.00	0.90	C	6.6	9.0	6.6	9.0	9906	7.64	22
	79070000	26.97	29.29	R	6.6	7.0	6.6	9.0	9929	7.54	24
	79190000	8.21	9.62	L	6.5	6.5	6.5	7.0	9925	12.47	26
	79190000	8.21	9.62	R	6.5	6.5	6.9	8.0	9925	12.47	26
	79230000	2.38	4.00	R	6.5	7.5	6.5	8.0	9925	13.03	26
Arterial 6	87015000	0.35	1.93	C	6.6	8.5	6.6	8.0	9934	26.27	26
	87019000	0.00	2.91	L	6.5	6.5	7.0	8.0	9930	5.50	29
	87030000	8.23	8.83	L	6.5	6.5	6.5	10.0	9930	11.65	29
	87046000	0.00	1.29	R	6.6	10.0	6.6	9.0	9934	16.89	22
	87047000	8.54	9.51	R	6.5	9.0	6.5	9.0	9934	10.58	24
	87091000	7.09	8.06	L	6.6	10.0	6.6	9.0	9934	21.37	24
	87260000	18.96	22.27	R	6.5	6.5	8.3	7.0	9930	6.71	25
	90030000	6.13	7.10	C	6.5	6.5	7.5	10.0	9934	101.9	24
Arterial 7	15010000	8.98	10.42	L	6.5	6.5	6.7	8.0	9922	26.72	24
	15060000	5.42	5.62	L	6.5	6.5	6.5	7.0	9922	21.55	29
	15060000	5.42	5.62	R	6.5	6.5	6.5	7.0	9922	21.55	29
	15140000	3.30	4.06	R	6.5	9.0	6.5	10.0	9922	28.40	24
Turnpike	86471000	4.21	6.54	R	6.5	6.5	7.8	8.0	9930	7.61	24

Table A4. Candidate PCC Segments with Long Service Lives.

Subsystem	RDWYID	BEG	END	S	PCR	CR	RI	SITE	Dist. (ml)	Age
Arterial 1	01010000	0.49	4.98	L	6.7	6.7	7.0	9917	1.85	22
	16100000	2.22	2.89	L	6.3	7.8	6.3	9927	1.75	22
	16100000	2.22	2.89	R	6.8	6.9	6.8	9927	1.75	22
	16100000	3.93	5.05	L	6.2	6.2	7.0	9927	3.68	22
	16100000	3.93	5.05	R	6.1	6.1	7.2	9927	3.68	22
Arterial 2	72030000	7.46	8.71	R	6.8	8.4	6.8	9914	4.80	22
	72030000	9.43	10.39	R	6.4	7.3	6.4	9914	4.42	22
	72090000	8.23	11.14	L	6.3	8.9	6.3	9905	10.19	22
	72090000	8.23	11.14	R	6.1	8.7	6.1	9905	10.19	22
	72090445	0.50	0.91	R	6.7	8.2	6.7	9914	7.76	22
	78010000	16.12	18.73	R	6.1	8.2	6.1	9905	22.39	22
Arterial 3	48080060	0.00	1.83	L	6.5	8.8	6.5	9924	6.08	24
Arterial 5	75008000	5.78	7.88	R	6.2	8.6	6.2	9906	24.54	28
	75008000	8.20	9.67	L	6.7	9.2	6.7	9906	24.18	28
	75008000	8.20	9.67	R	6.9	8.5	6.9	9906	24.18	28
	79010000	26.87	28.71	L	6.6	8.6	6.6	9925	13.57	24
	79010000	26.87	28.71	R	6.8	8.8	6.8	9925	13.57	23
Arterial 6	87004000	0.00	0.49	R	6.2	7.8	6.2	9930	7.64	22
Arterial 7	10150000	12.59	12.84	L	6.3	8.0	6.3	9922	5.30	22
Interstate	72001000	34.52	35.51	L	6.2	9.0	6.2	9914	9.65	22
	72001000	34.52	35.51	R	6.7	9.0	6.7	9914	9.65	22
	72270000	17.05	21.00	L	6.4	7.9	6.4	9914	3.45	29
	72270000	17.16	21.00	R	6.3	7.5	6.3	9914	3.46	29
	75280000	14.73	18.78	L	6.5	8.9	6.5	9906	25.41	29
	87004000	0.49	0.79	R	6.2	9.1	6.2	9930	7.49	22
	87004000	0.79	1.28	L	6.3	8.6	6.3	9930	7.36	22
	87270000	0.00	0.98	L	6.3	8.9	6.3	9930	11.47	22
	10320000	0.42	4.72	L	6.3	8.4	6.3	9922	5.00	22

Table A4. Candidate PCC Segments with Long Service Lives (continued).

Subsystem	RDWYID	BEG	END	S	PCR	CR	RI	SITE	Dist. (ml)	Age
Interstate	15002000	0.60	0.99	L	6.2	7.6	6.2	9926	23.33	22
	15003000	0.37	1.29	R	6.5	6.9	6.5	9926	24.03	22
	15190000	2.29	4.43	R	6.3	6.3	7.2	9926	25.37	22
	15190000	5.29	10.96	L	6.2	8.1	6.2	9922	21.86	22

APPENDIX B

SUMMARY OF FWD AND PCS DATA

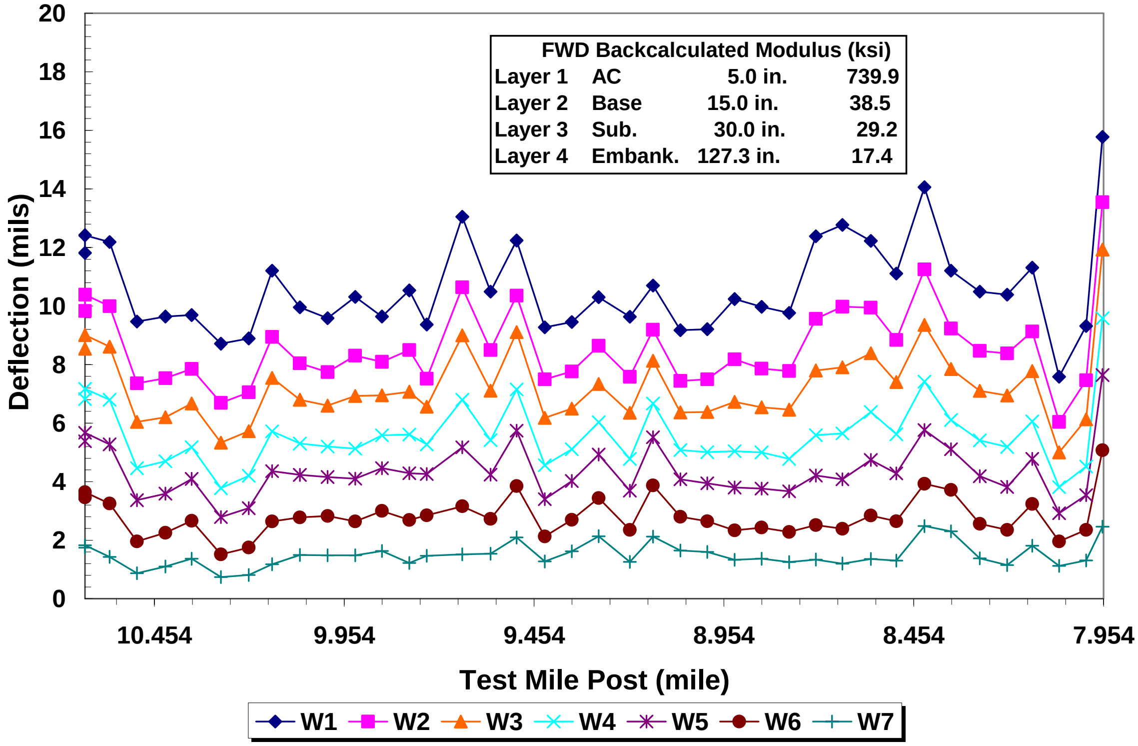

Figure B1. **FWD Data on Section 260050000 at Alachua County.**

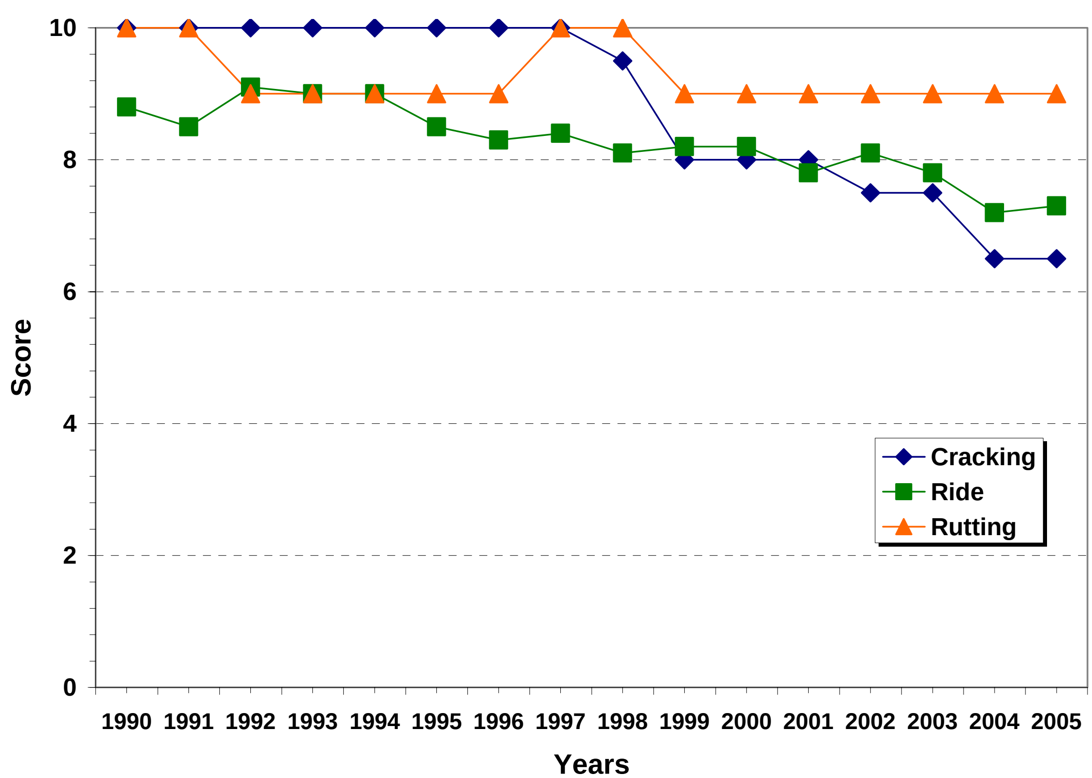

Figure B2. PCS Data on Section 26005000 at Alachua County.

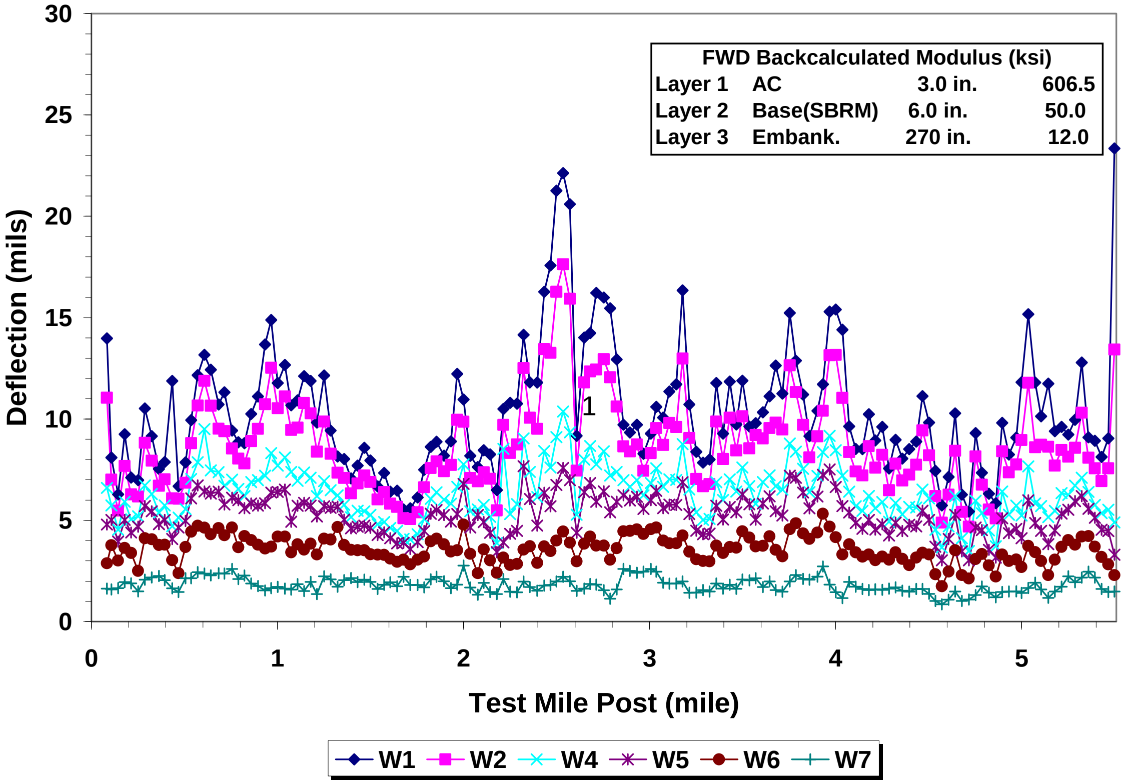

Figure B3. FWD Data on Section 28040000 at Bradford County.

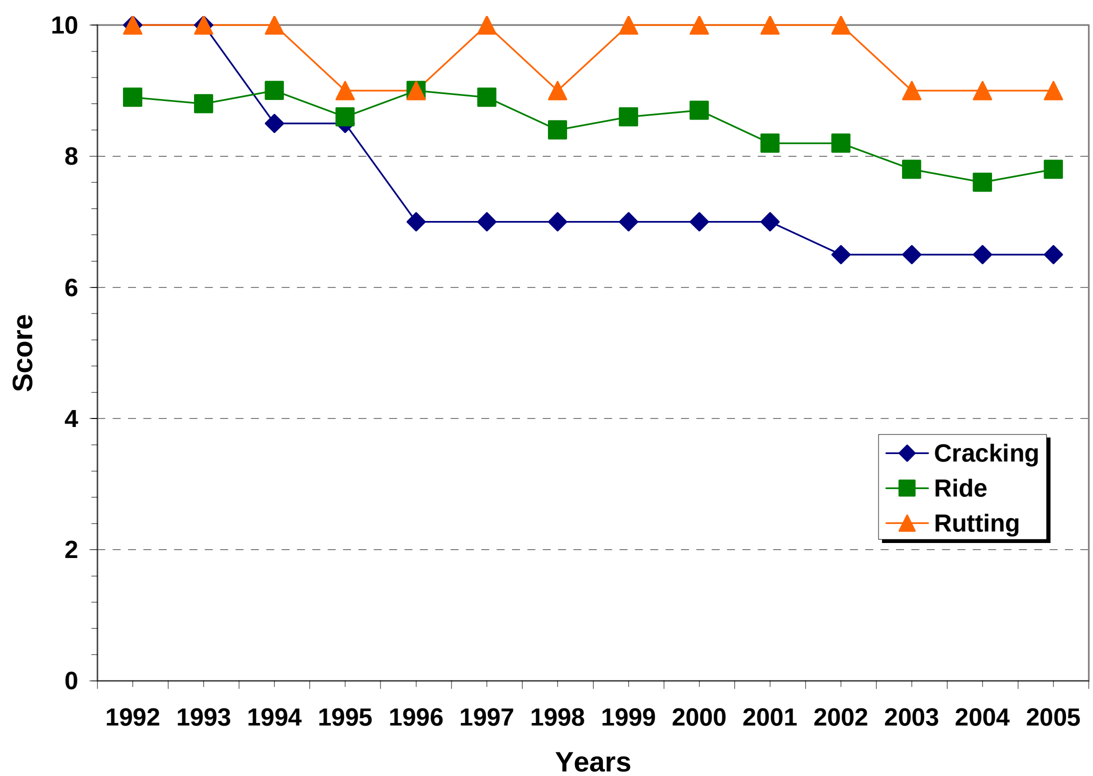

Figure B4. PCS Data on Section 28040000 at Bradford County.

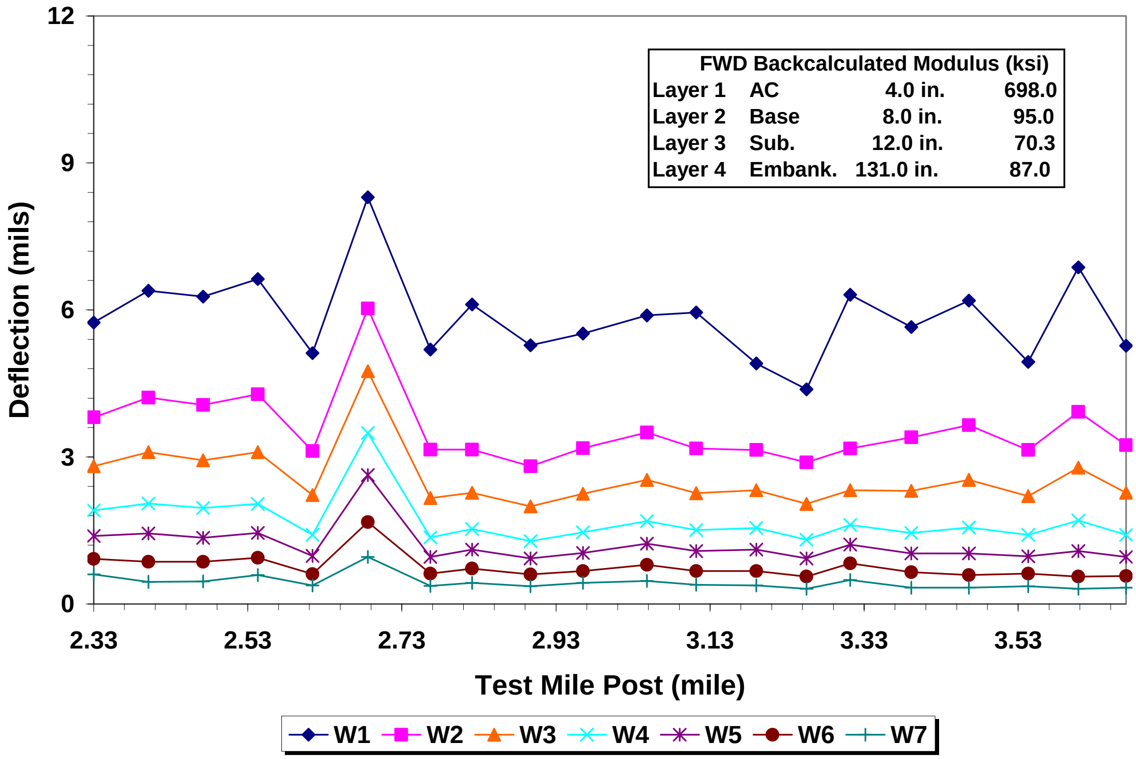

Figure B5. FWD Data on Section 86190000 at Broward County.

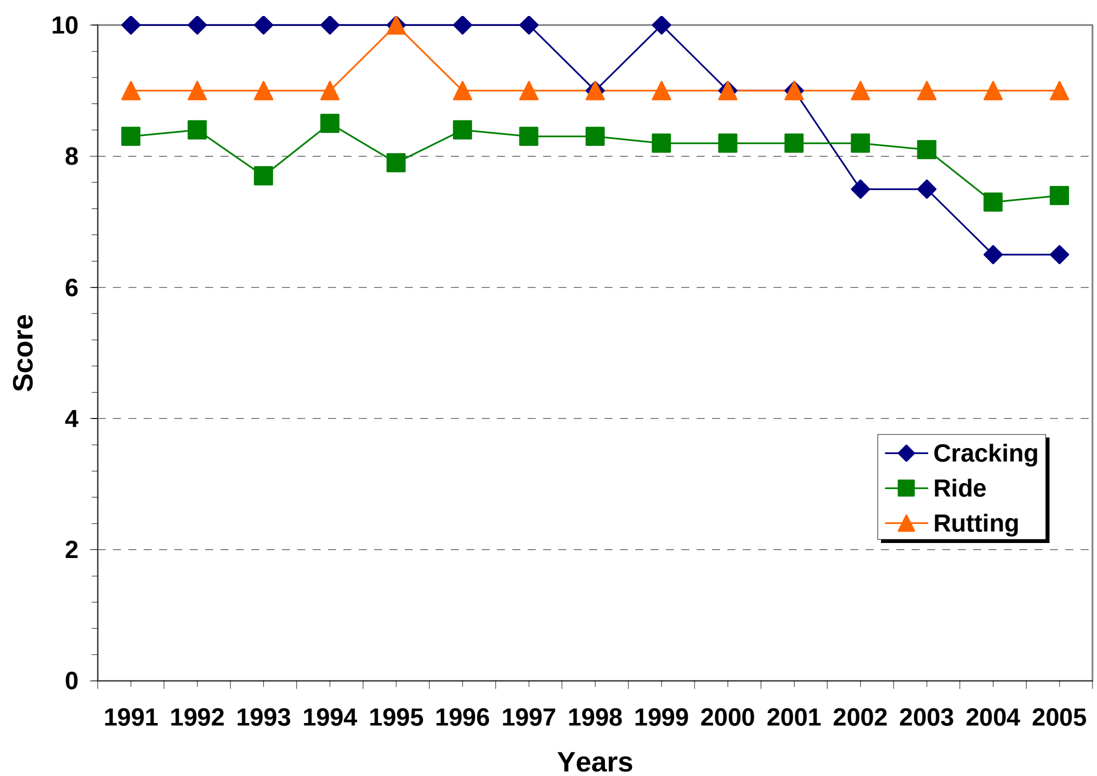

Figure B6. PCS Data on Section 86190000 at Broward County.

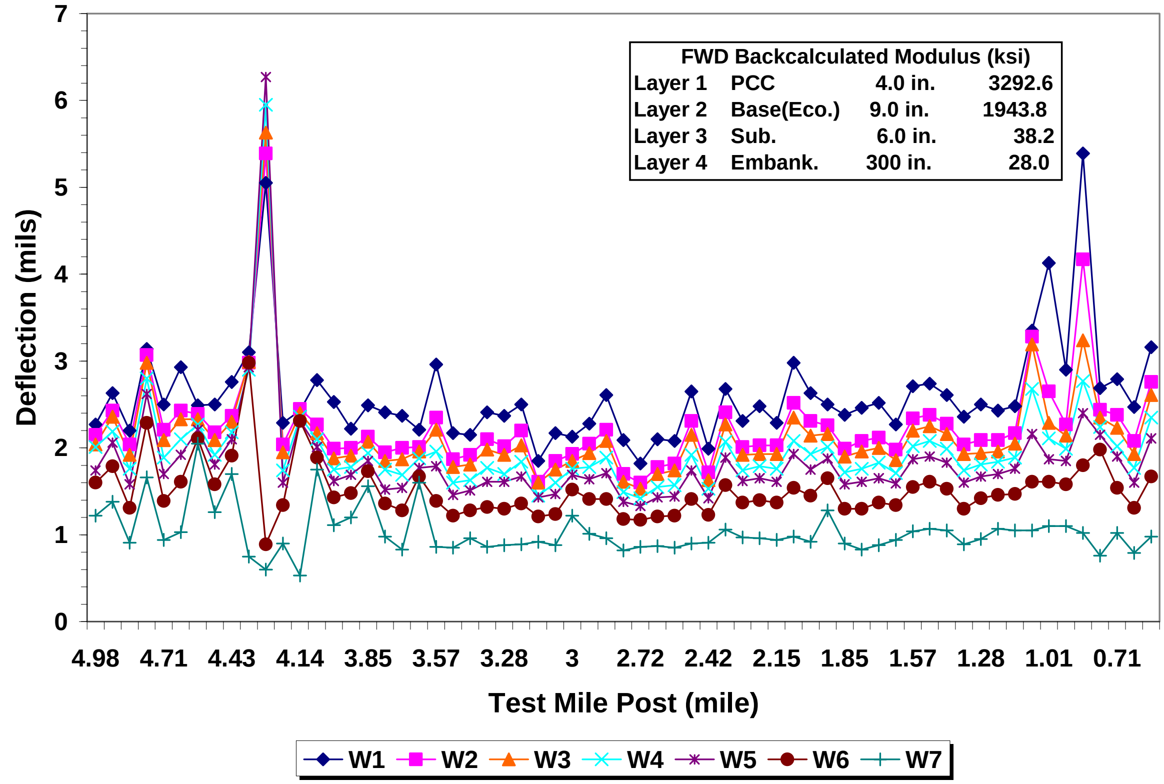

Figure B7. **FWD Data on Section 01010000 at Charlotte County.**

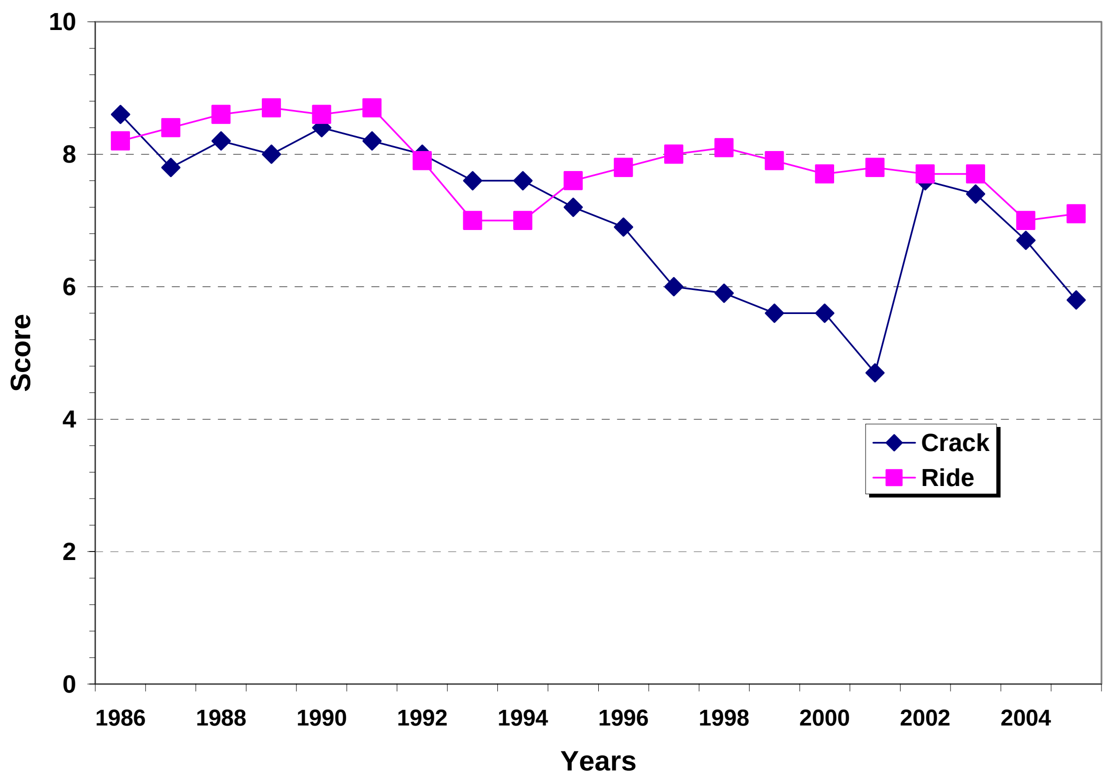

Figure B8. PCS Data on Section 01010000 at Charlotte County.

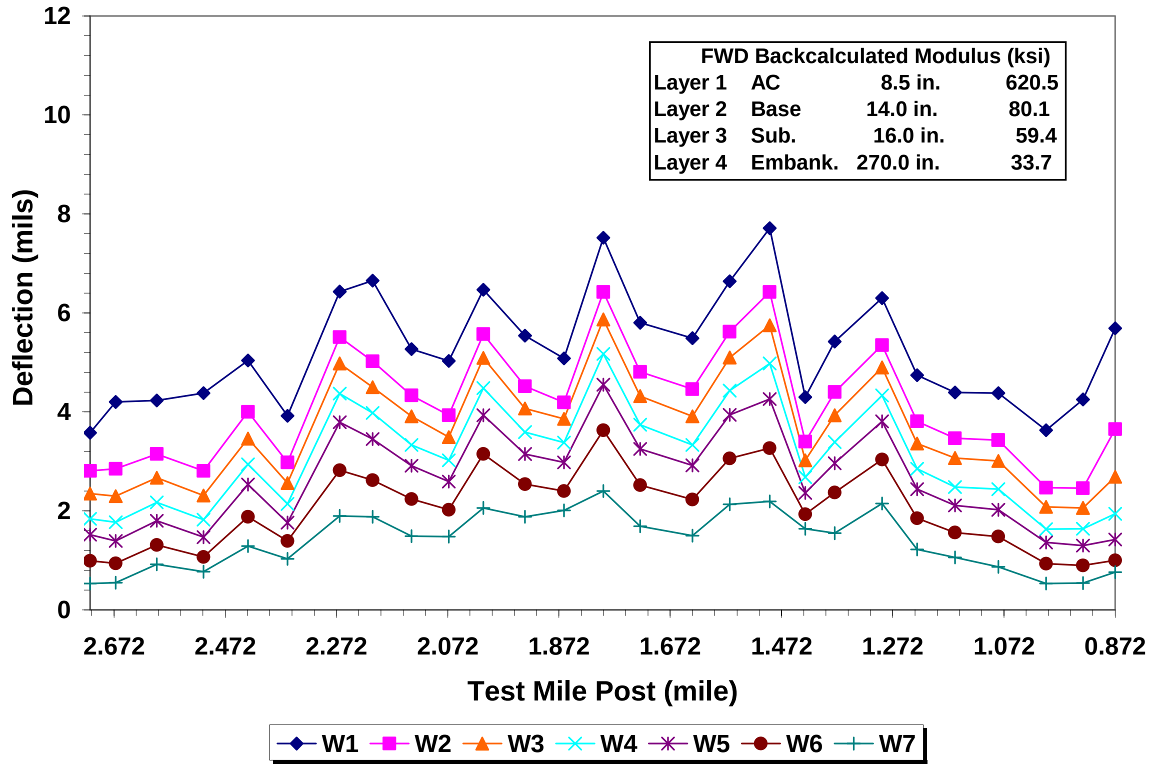

Figure B9. FWD Data on Section 87060000 at Dade County.

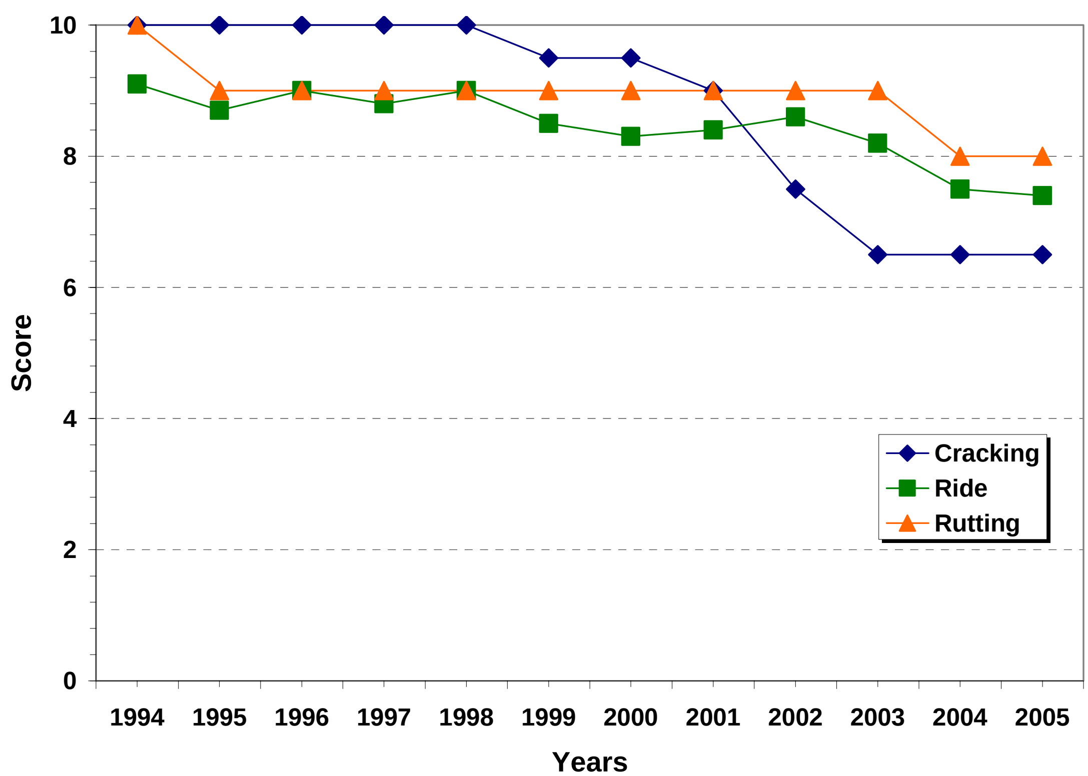

Figure B10. PCS Data on Section 87060000 at Dade County.

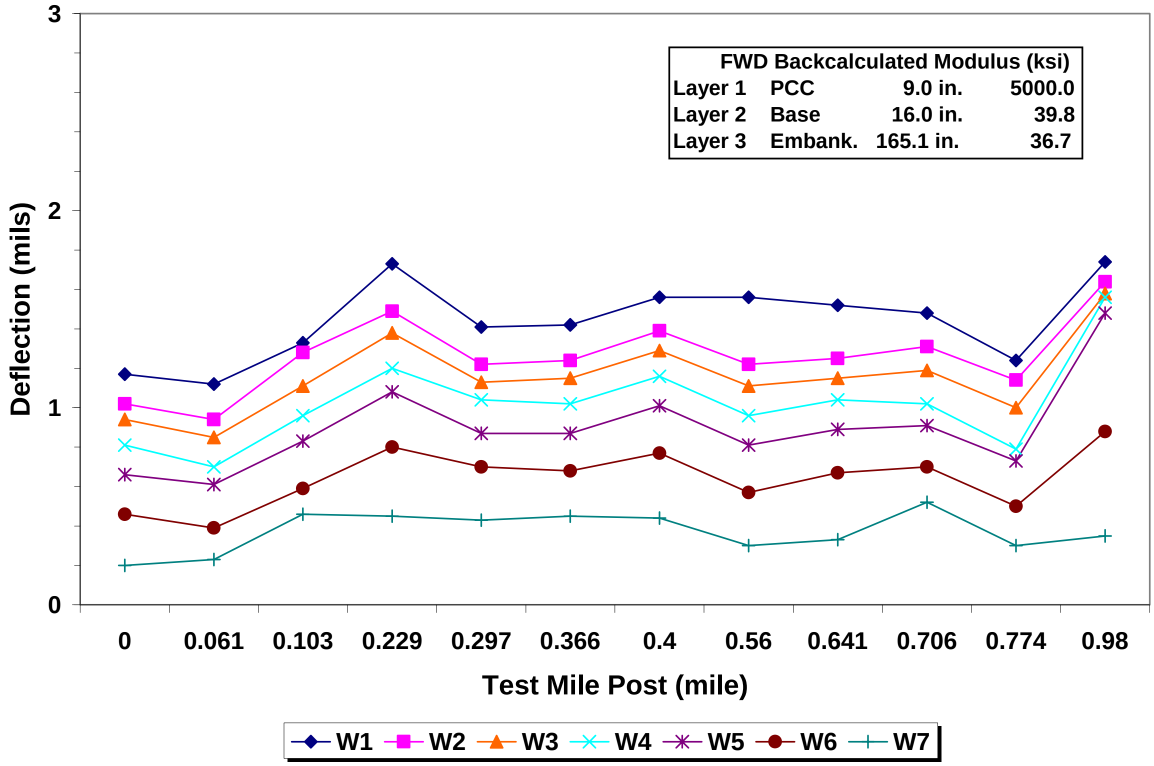

Figure B11. **FWD Data on Section 87270000 at Dade County.**

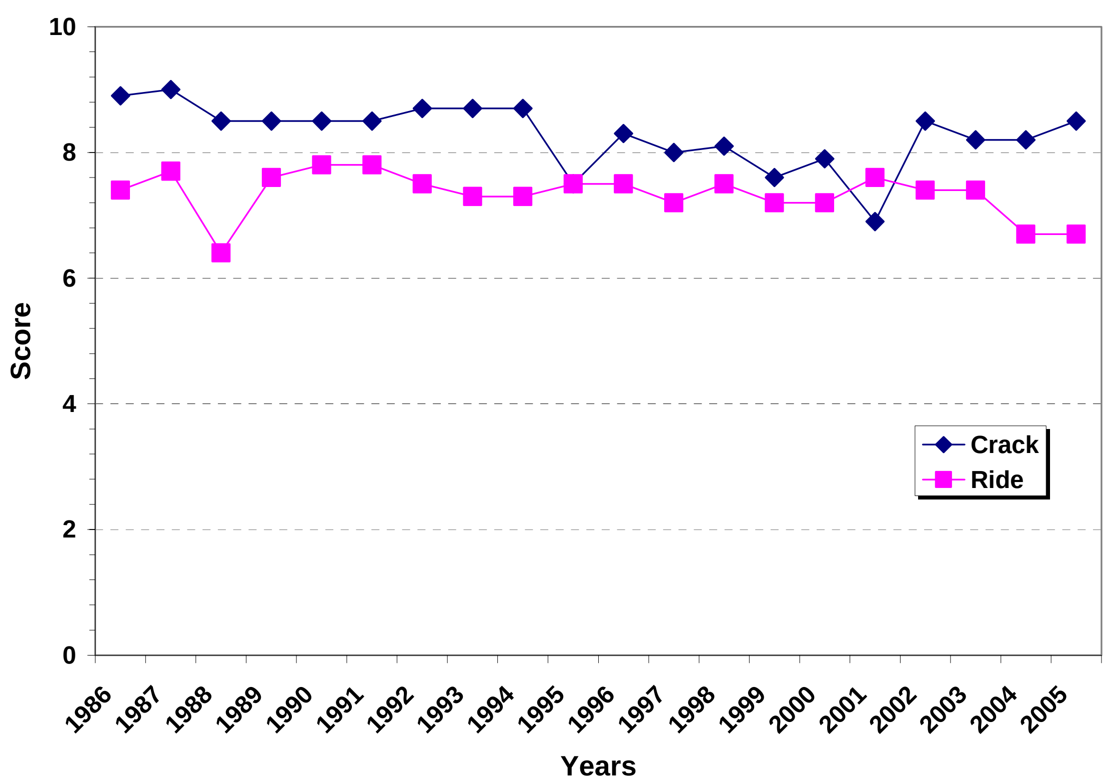

Figure B12. PCS Data on Section 87270000 at Dade County.

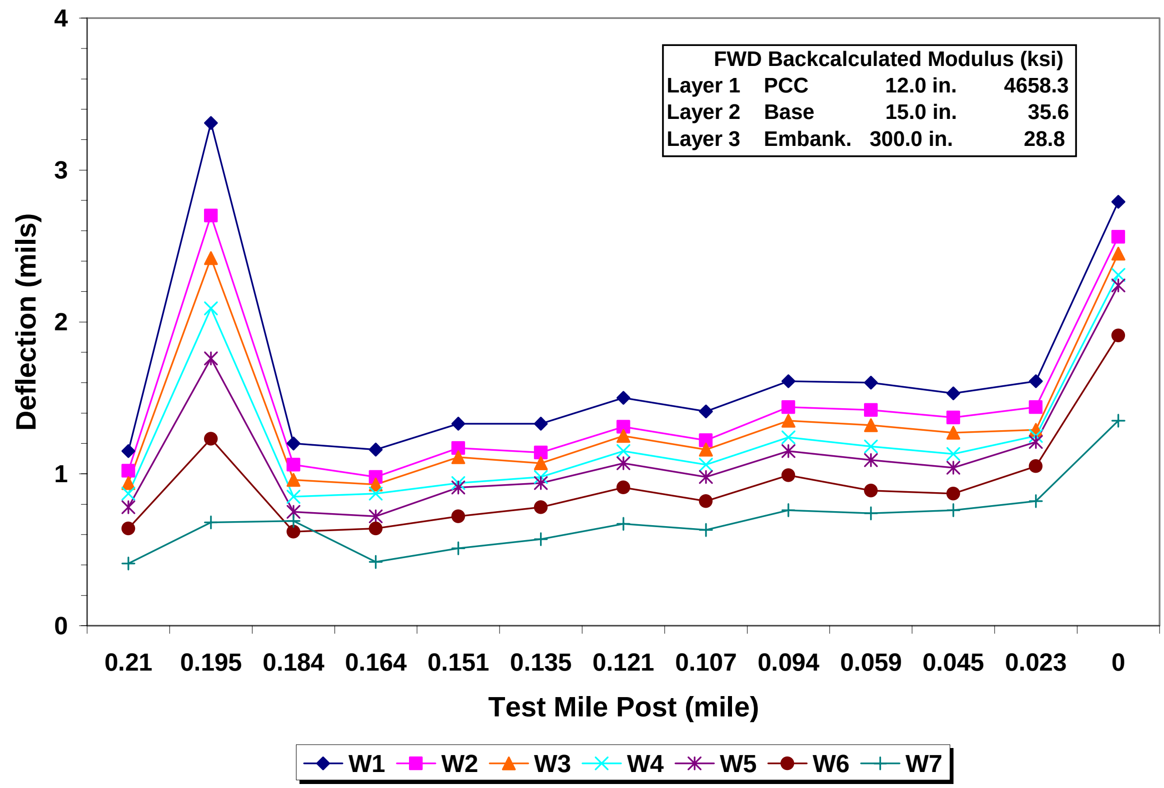

Figure B13. FWD Data on Section 87061000 at Dade County.

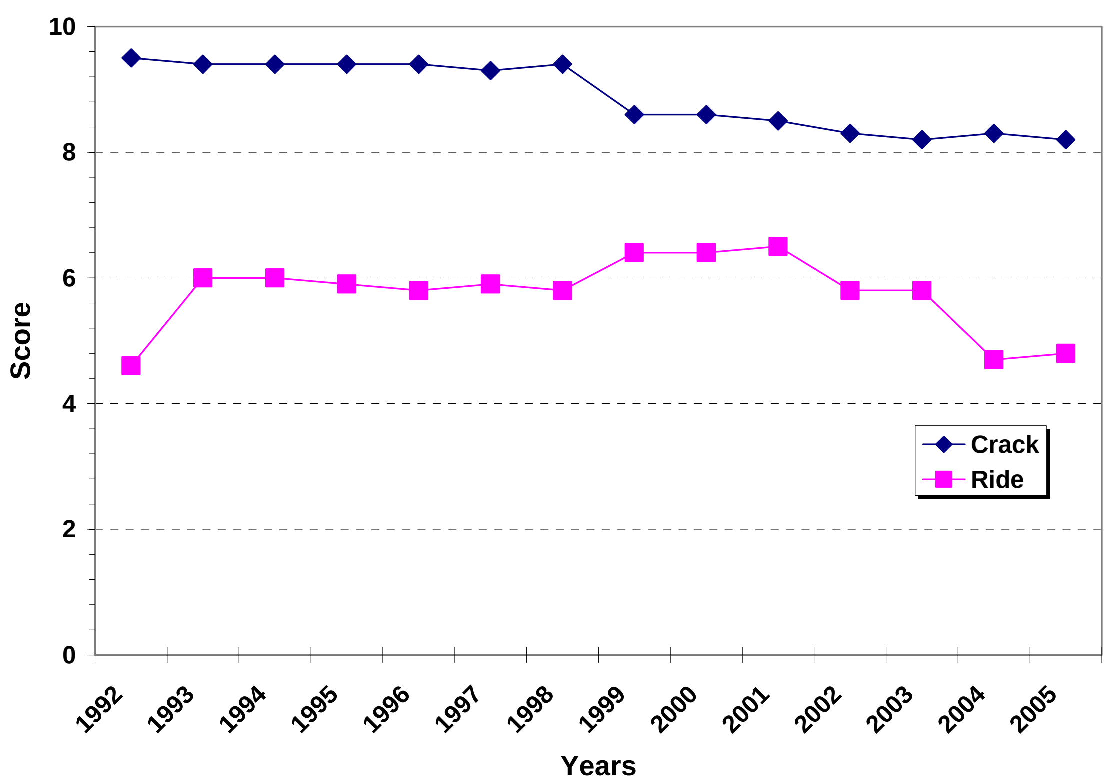

Figure B14. PCS Data on Section 87061000 at Dade County.

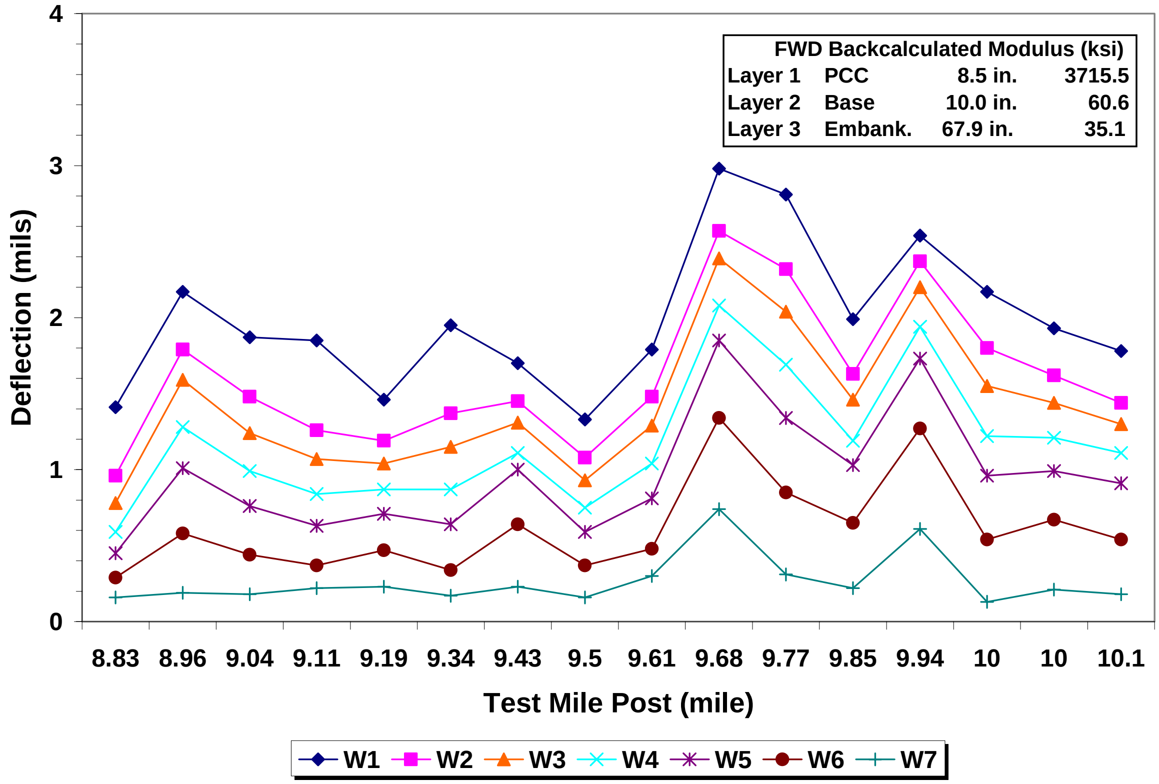

Figure B15. **FWD Data on Section 87030000R at Dade County.**

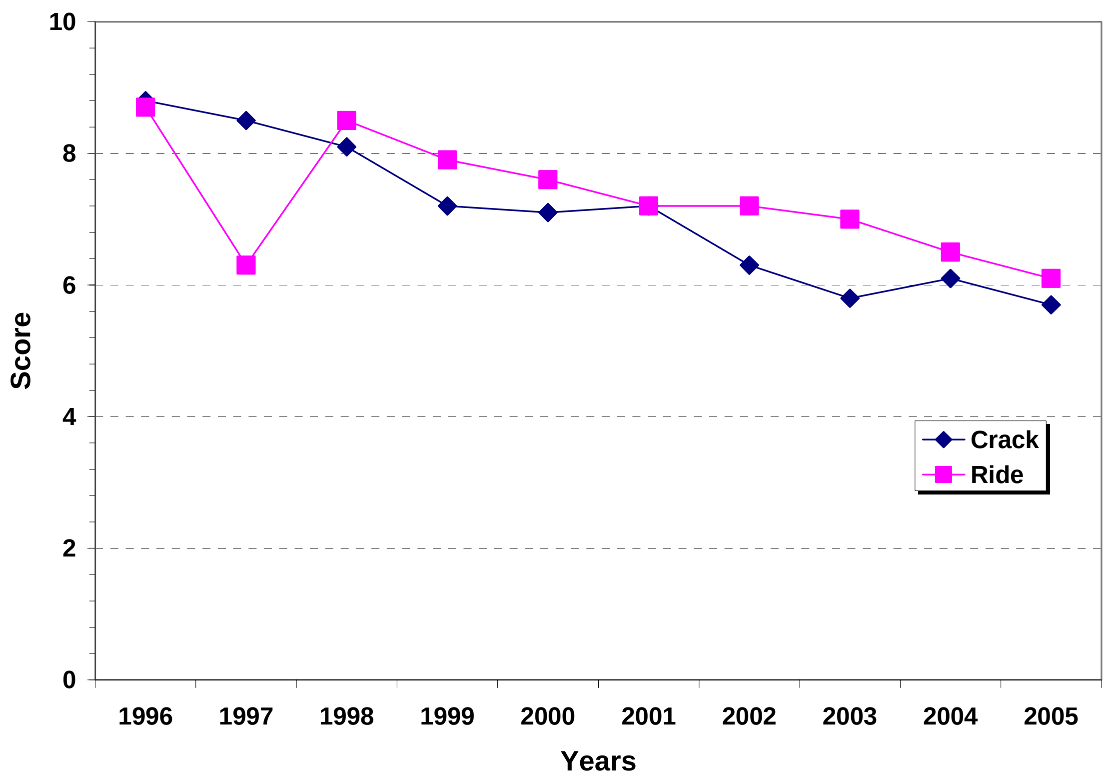

Figure B16. PCS Data on Section 87030000R at Dade County.

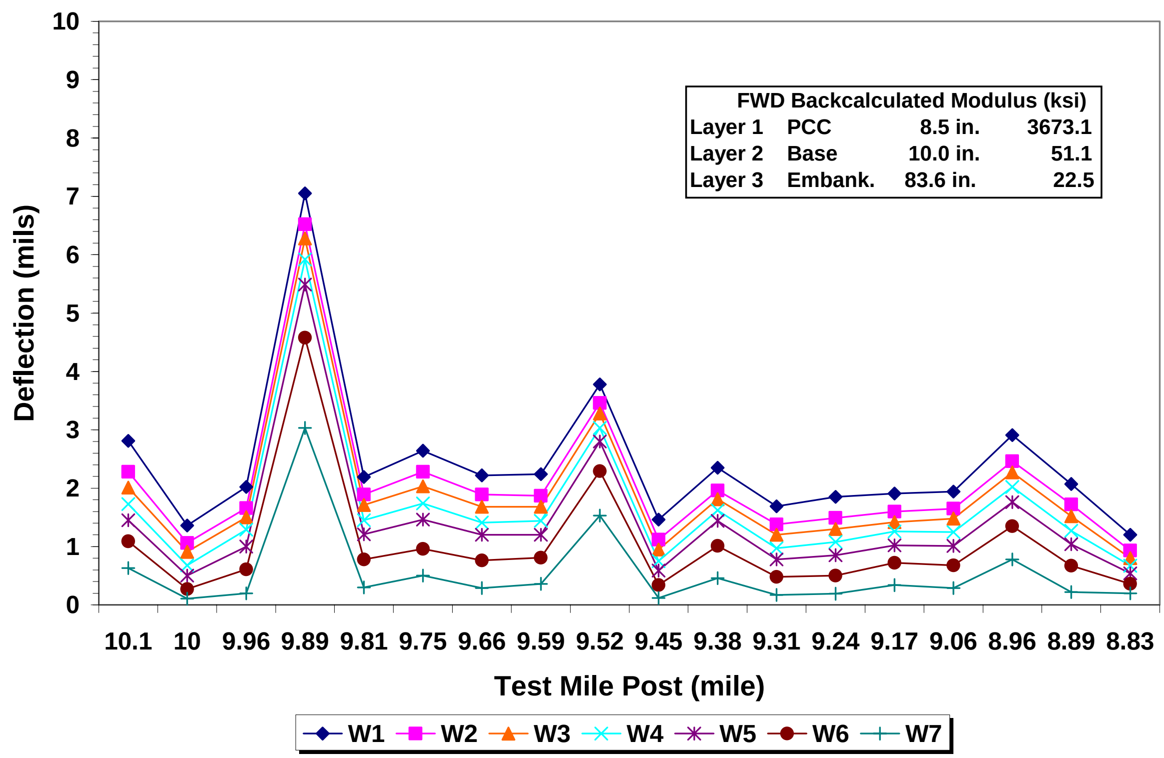

Figure B17. FWD Data on Section 87030000L at Dade County.

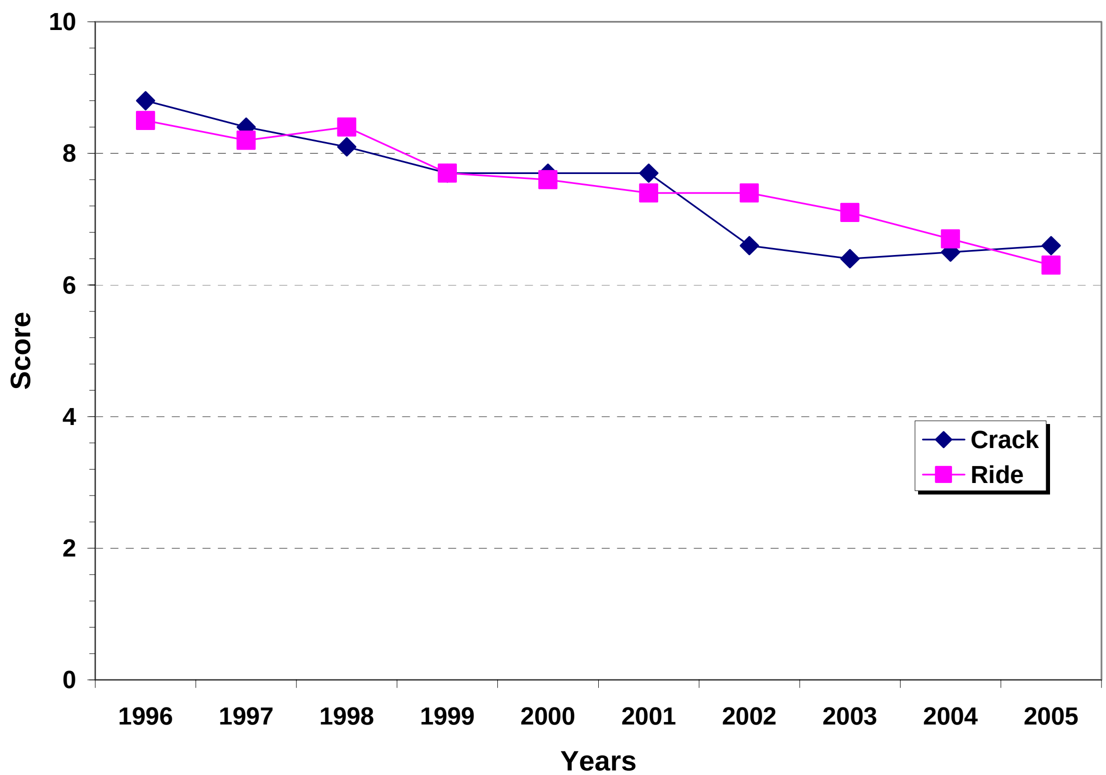

Figure B18. PCS Data on Section 87030000L at Dade County.

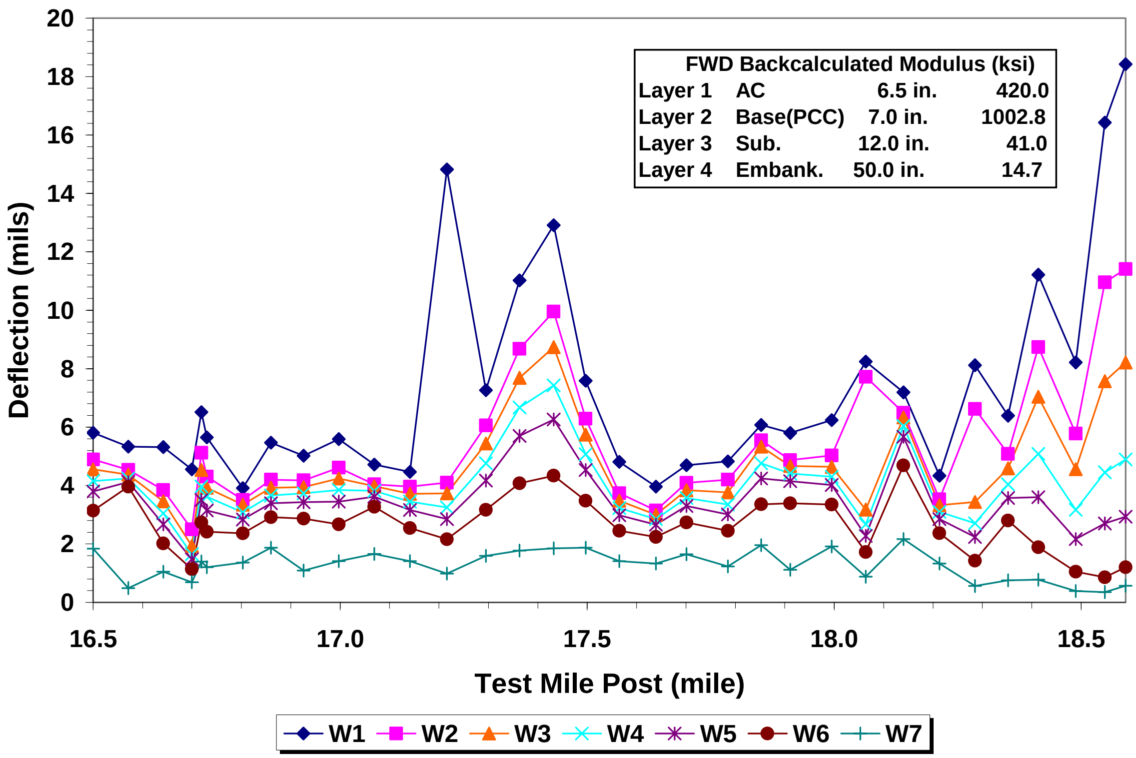

Figure B19. FWD Data on Section 50010000 at Gadsden County.

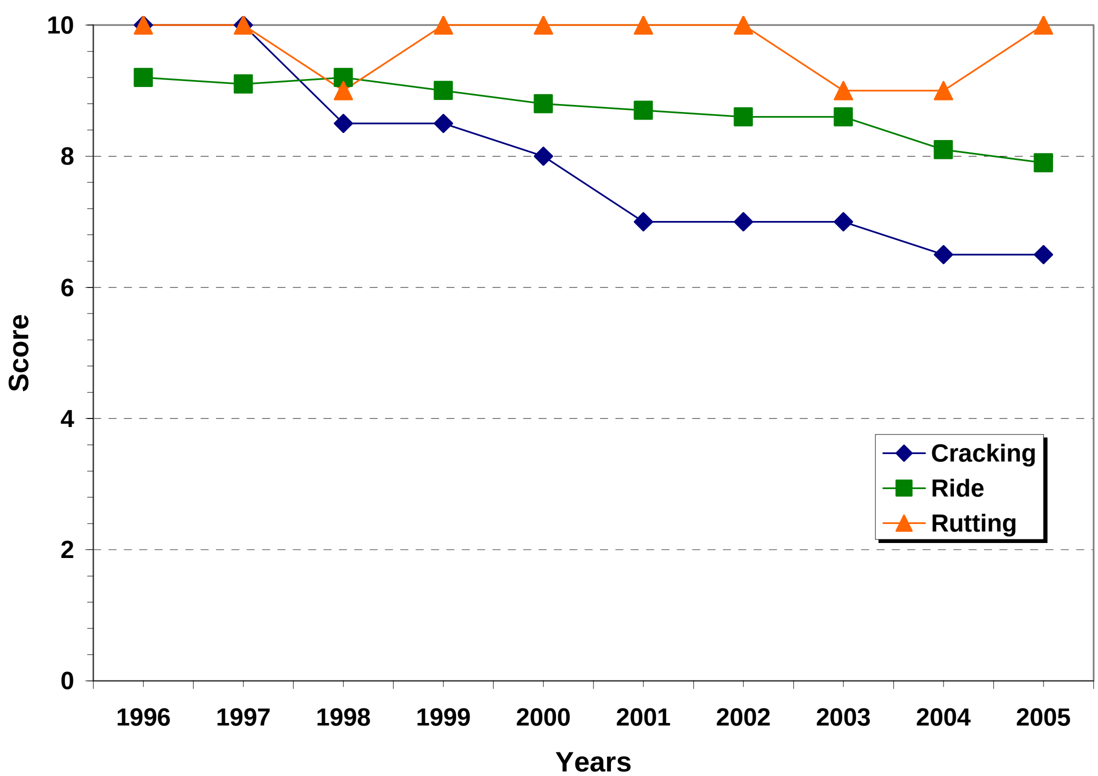

Figure B20. PCS Data on Section 50010000 at Gadsden County.

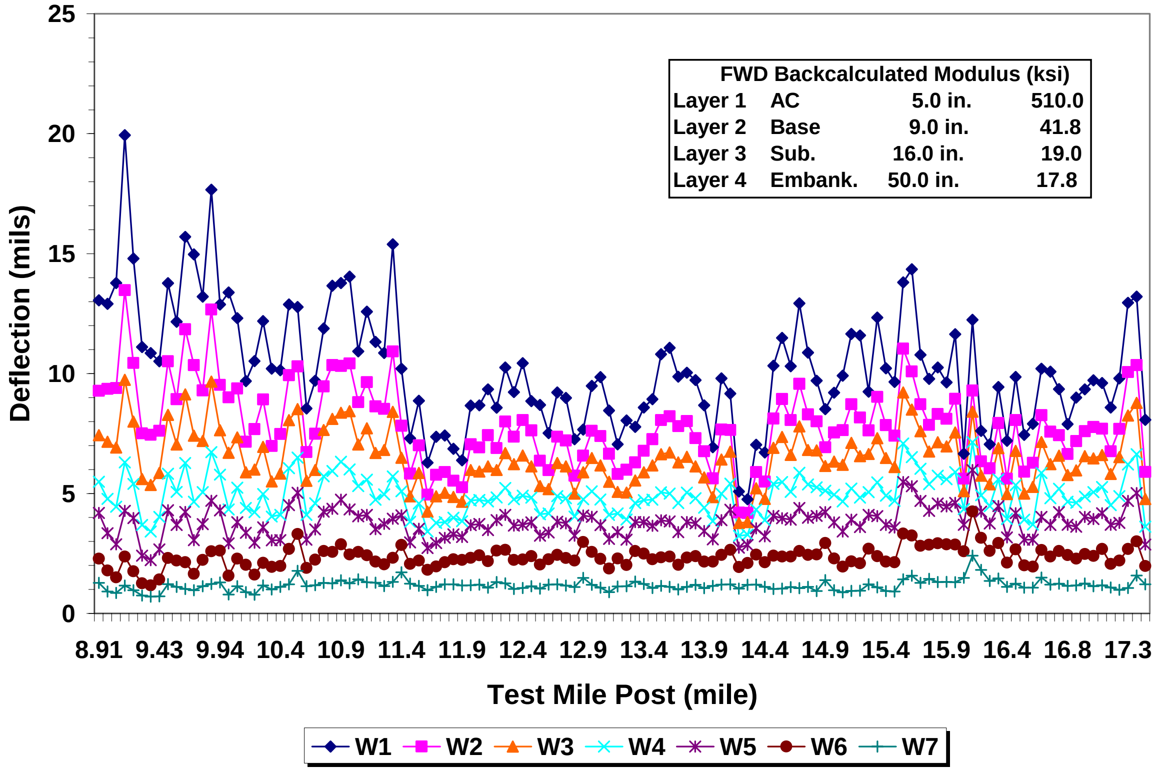

Figure B21. FWD Data on Section 10060000 at Hillsborough County.

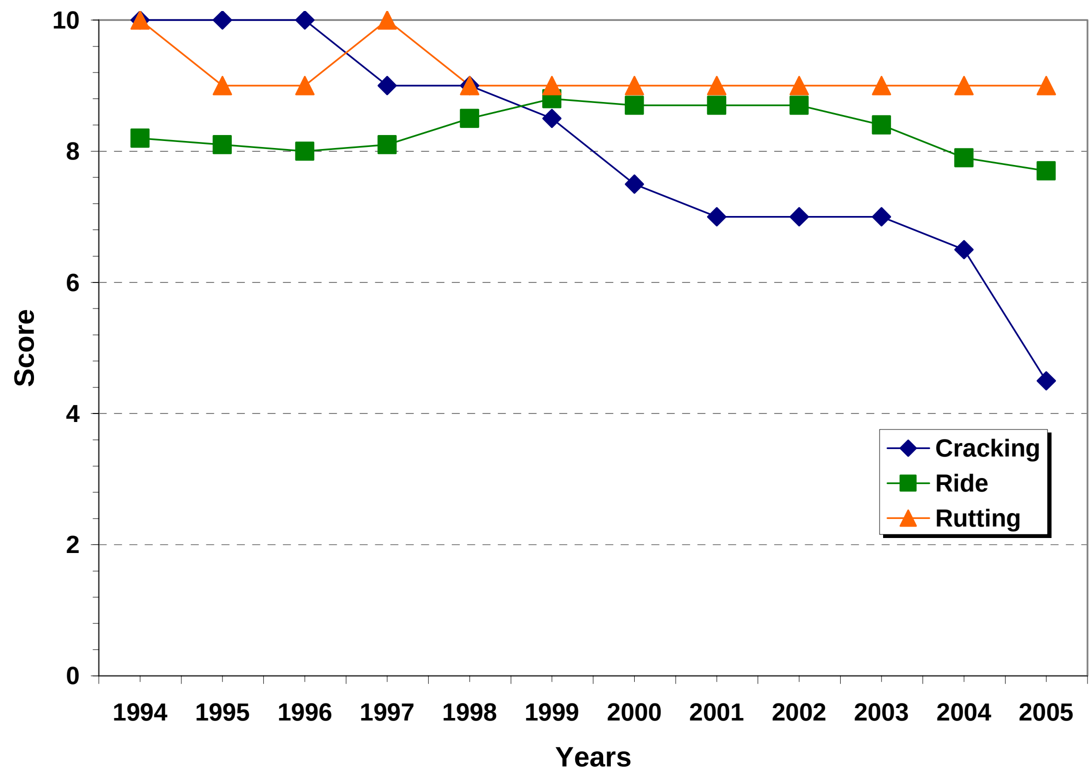

Figure B22. PCS Data on Section 10060000 at Hillsborough County.

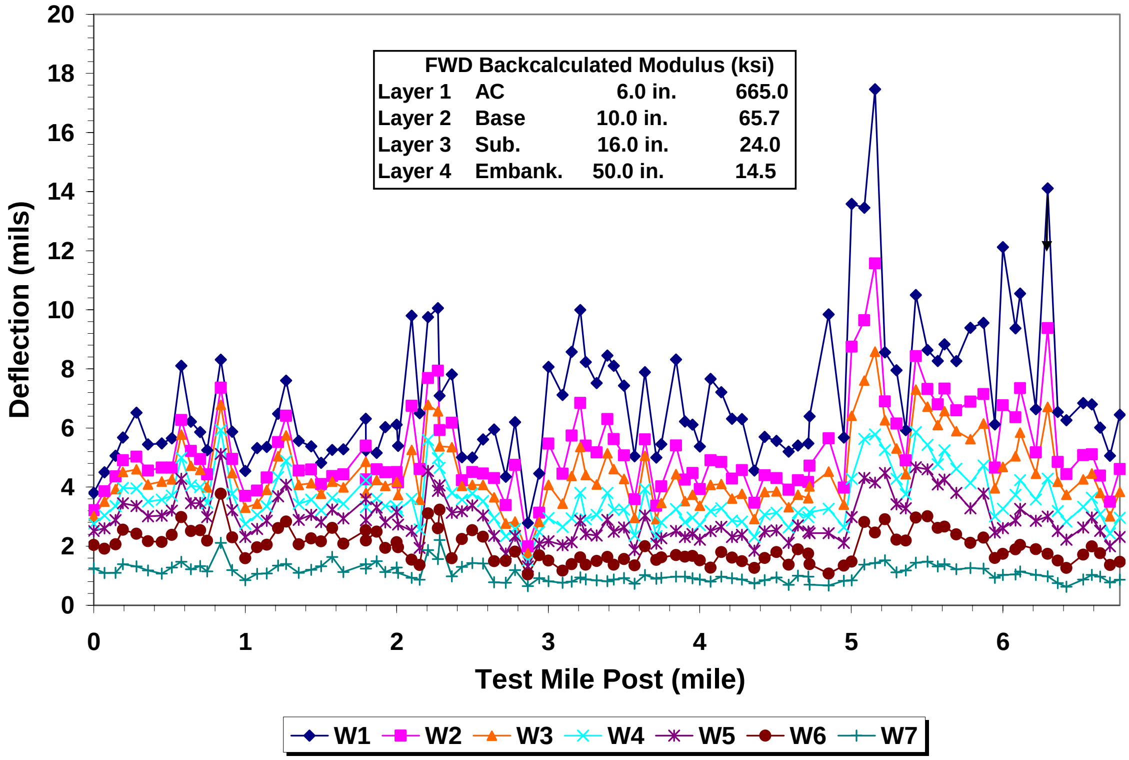

Figure B23. FWD Data on Section 10160000 at Hillsborough County.

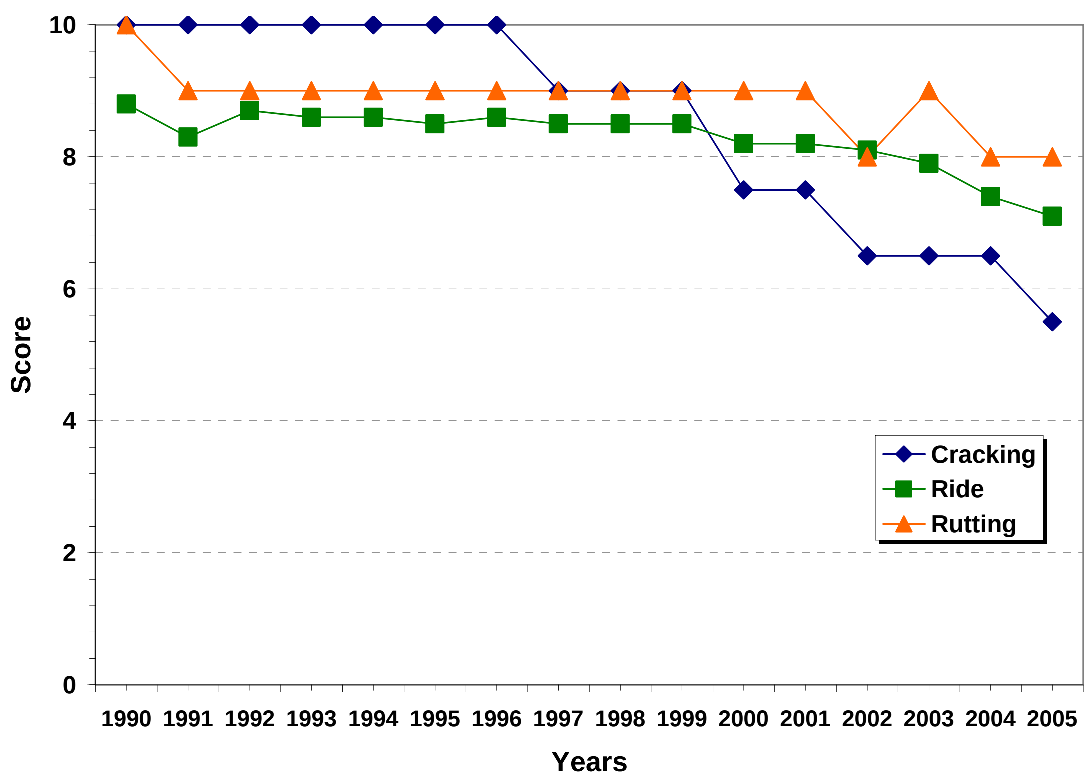

Figure B24. PCS Data on Section 10160000 at Hillsborough County.

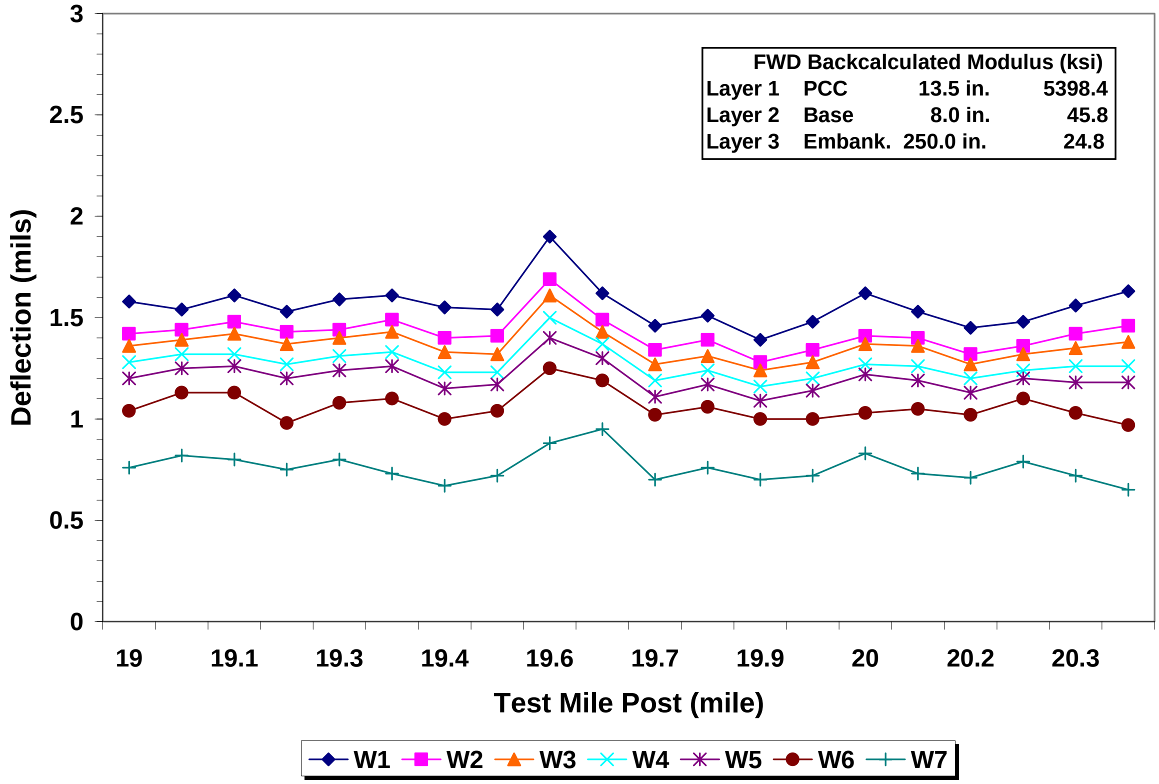

Figure B25. **FWD Data on Section 10075000A at Hillsborough County.**

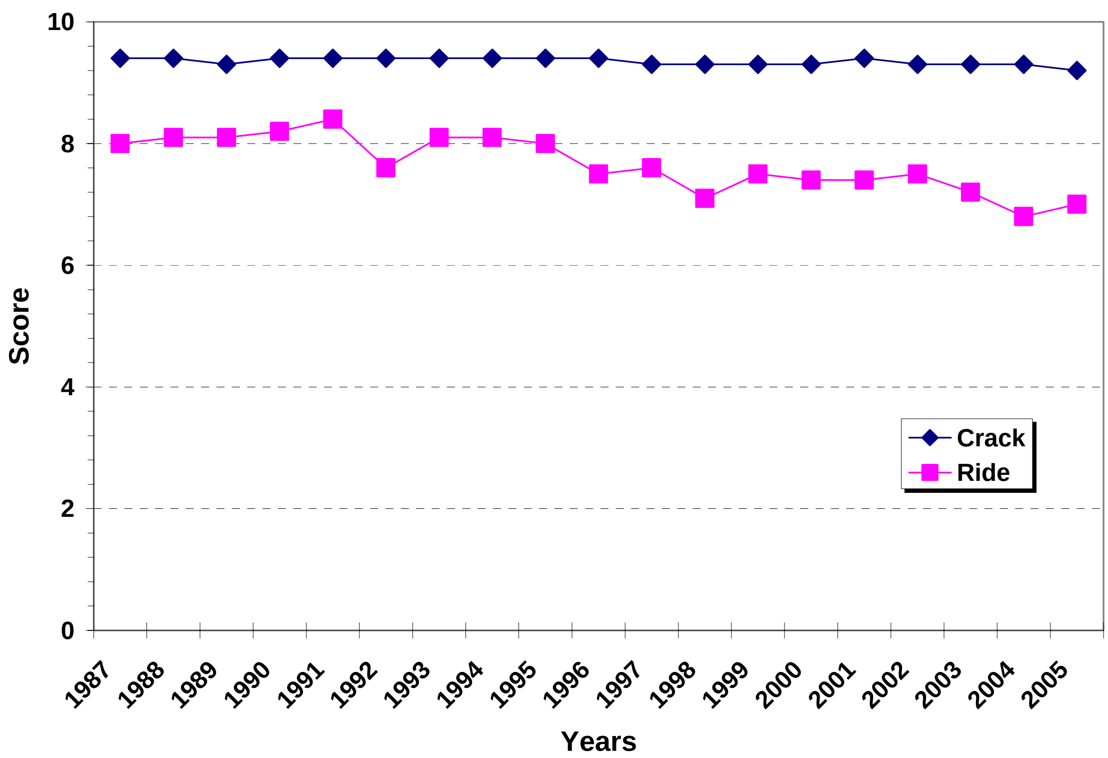

Figure B26. PCS Data on Section 10075000A at Hillsborough County.

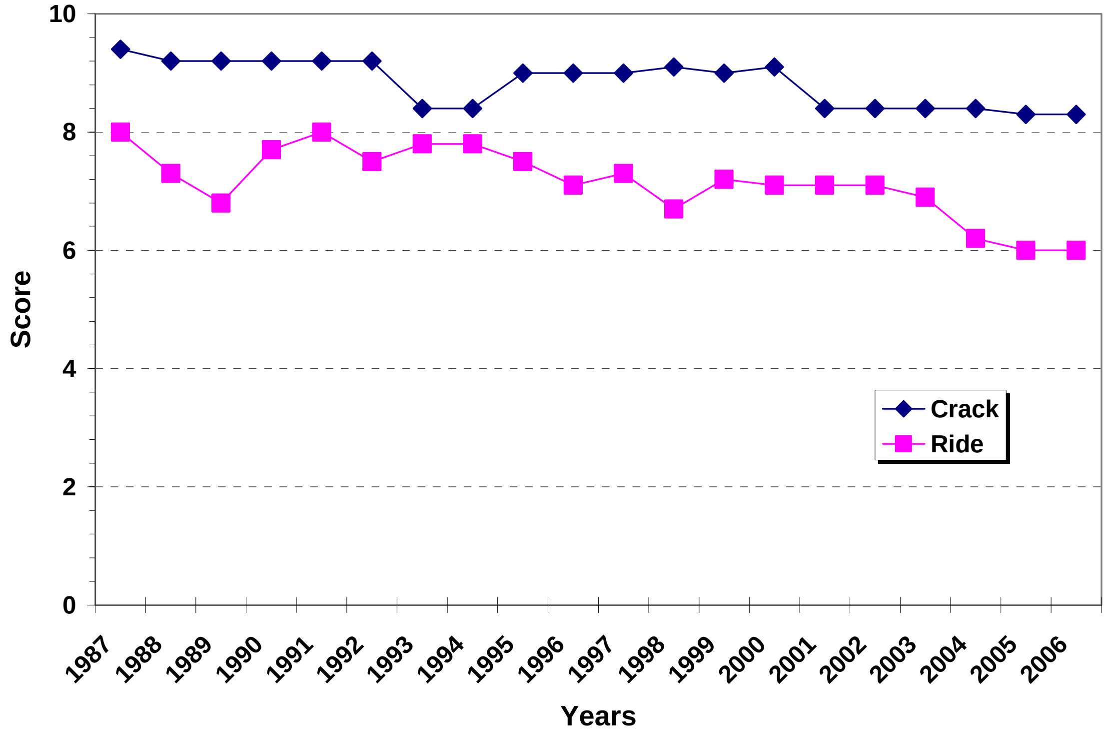

Figure B27. PCS Data on Section 10075000B at Hillsborough County.

Note: This section was added later during the project so FWD tests were not performed.

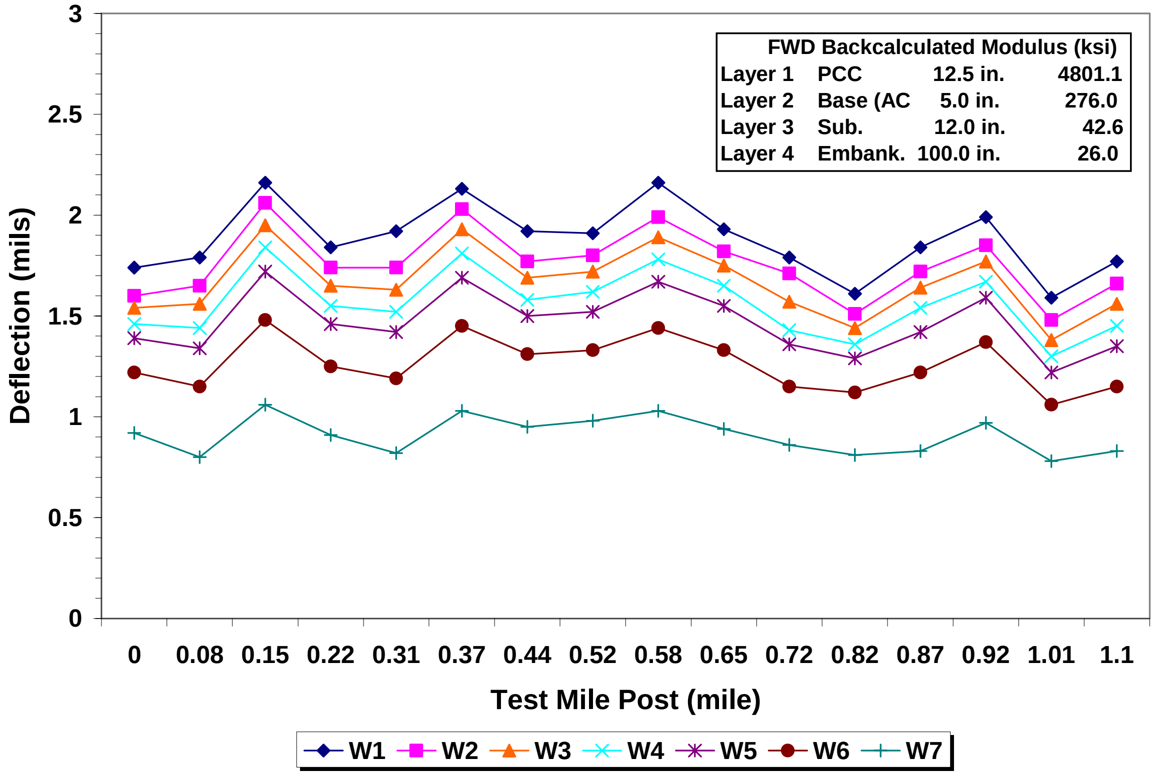

Figure B28. **FWD Data on Section 10250001 at Hillsborough County.**

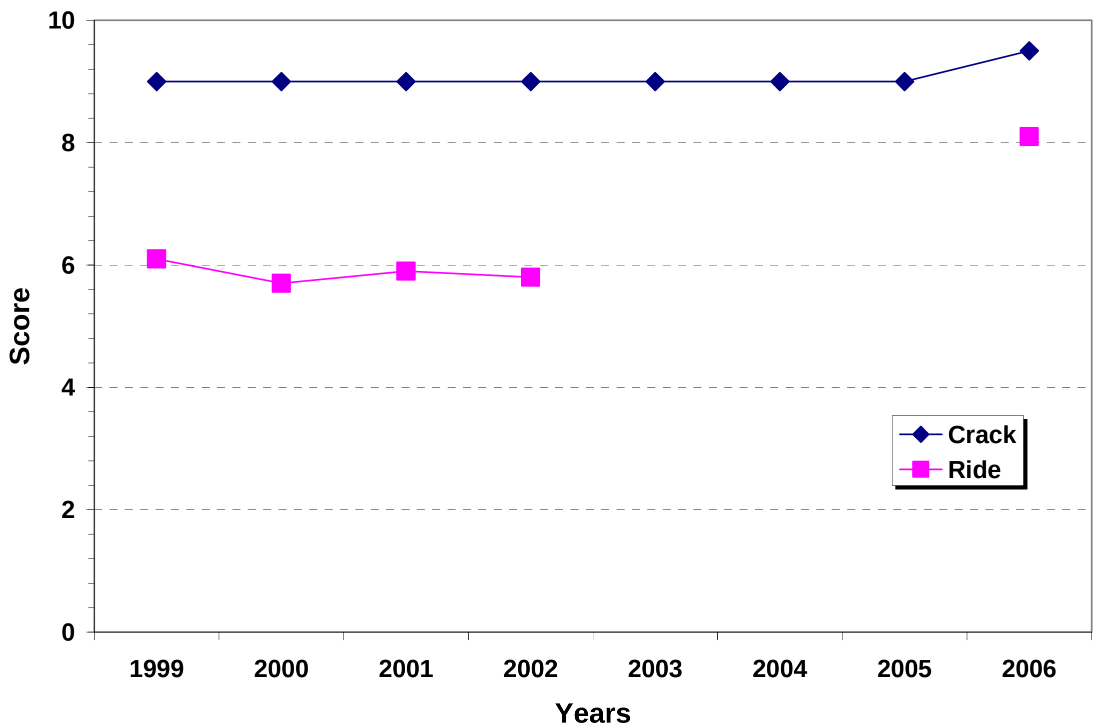

Figure B29. PCS Data on Section 10250001 at Hillsborough County.

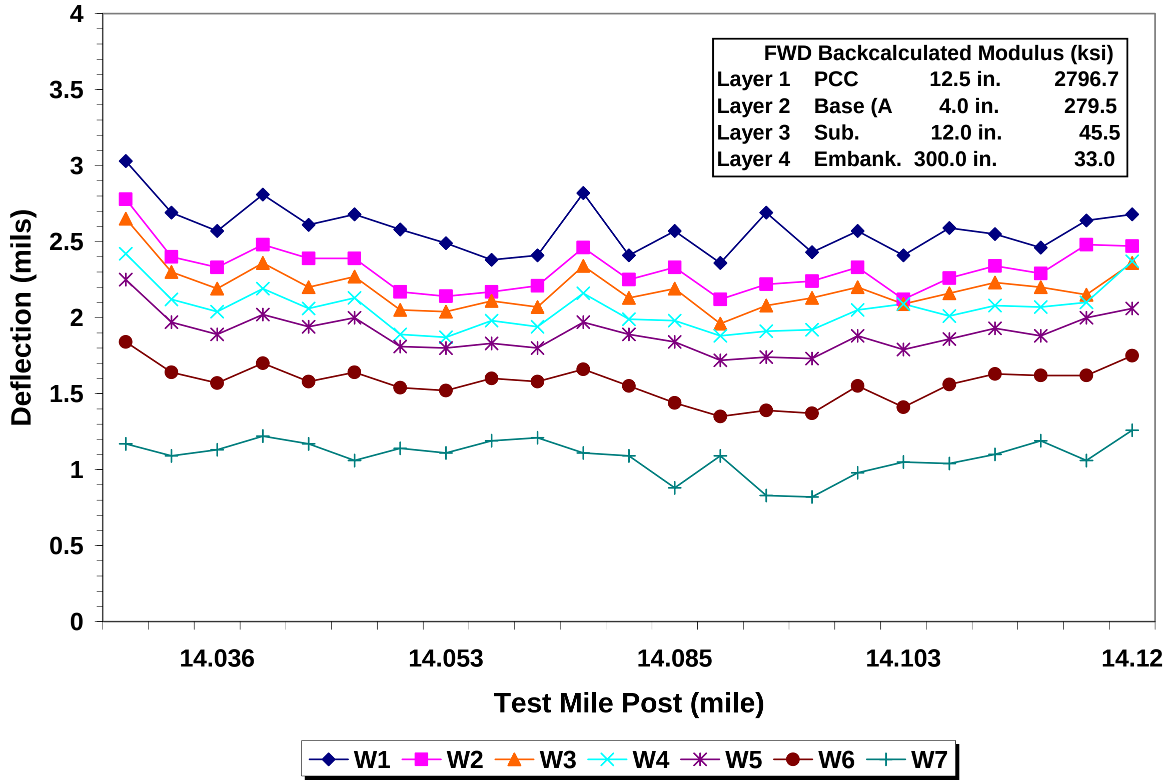

Figure B30. FWD Data on Section 11020000 at Lake County.

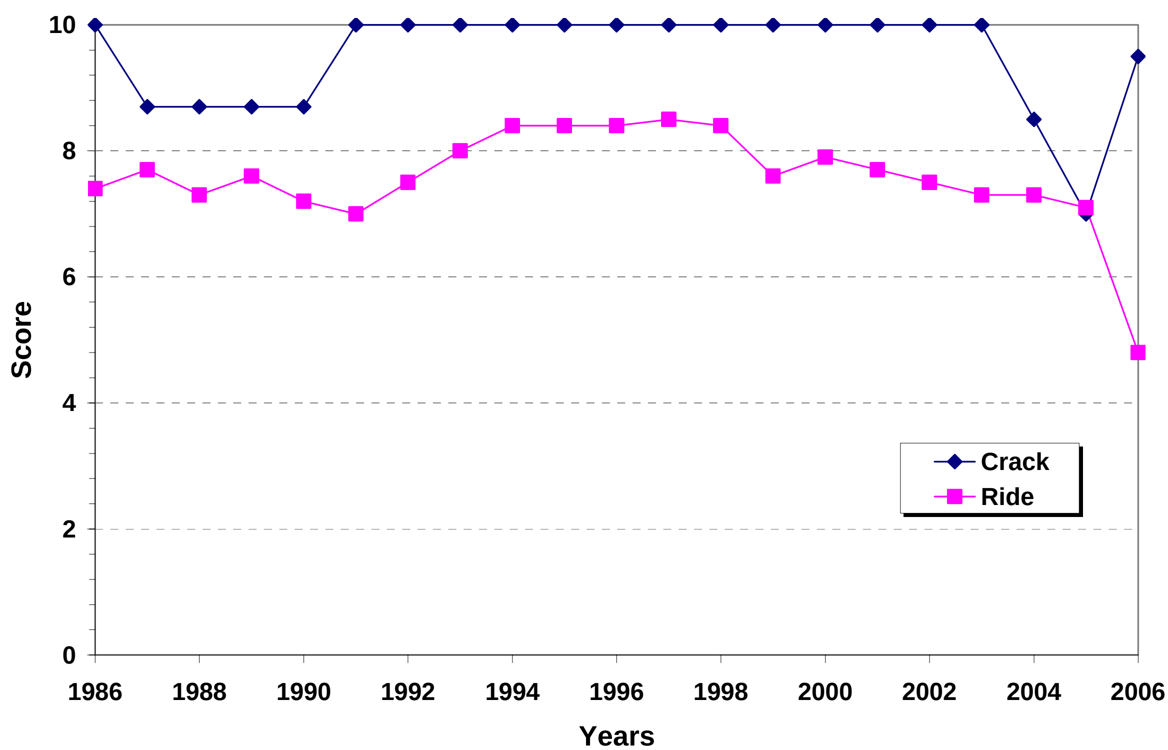

Figure B31. PCS Data on Section 11020000 at Lake County.

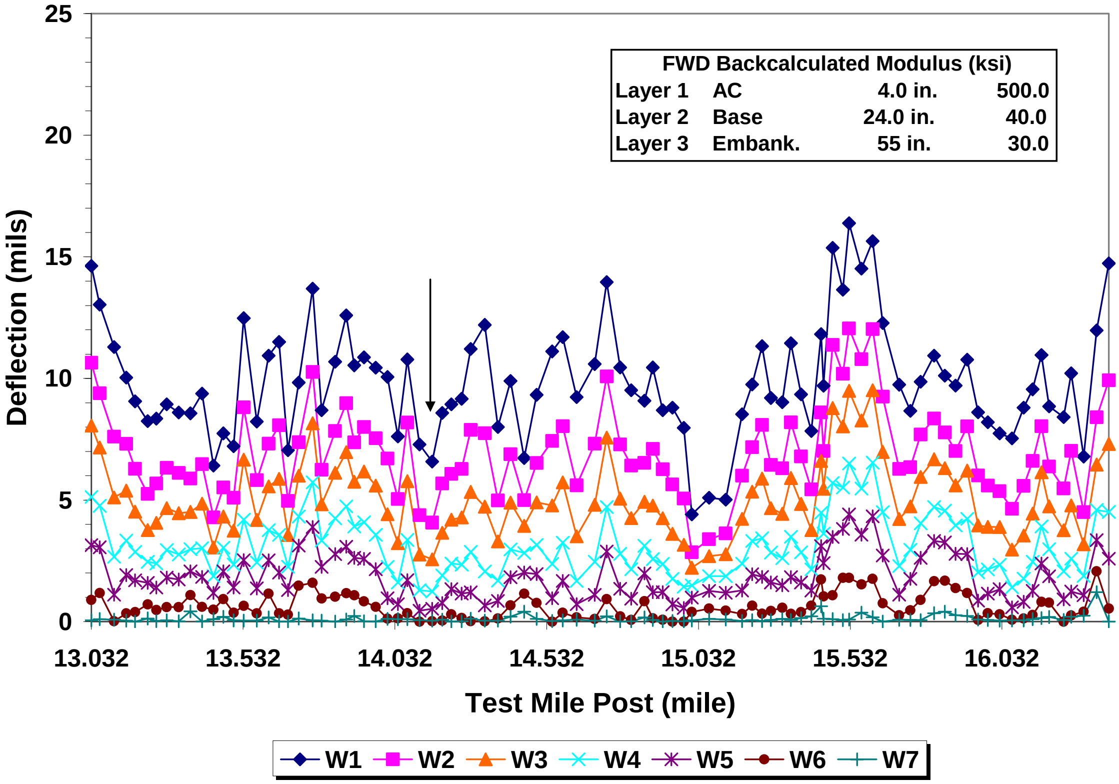

Figure B32. FWD Data on Section 90060000 at Monroe County.

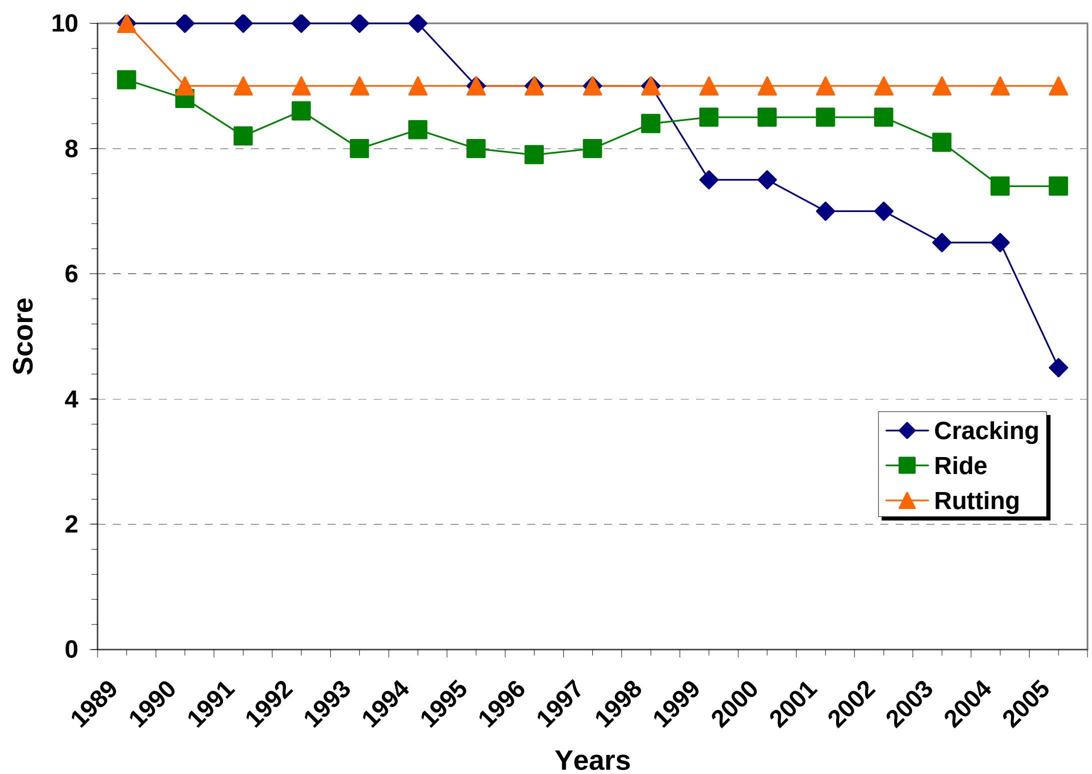

Figure B33. PCS Data on Section 90060000 at Monroe County.

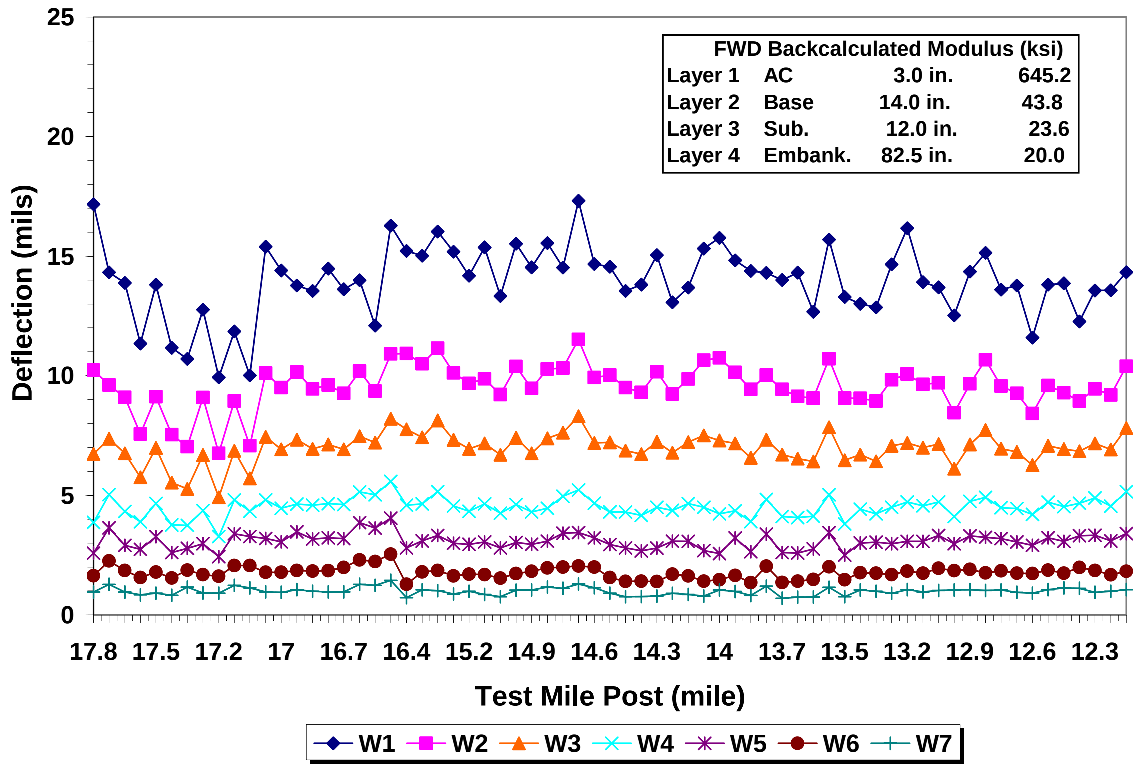

Figure B34. FWD Data on Section 93310000 at Palm Beach County.

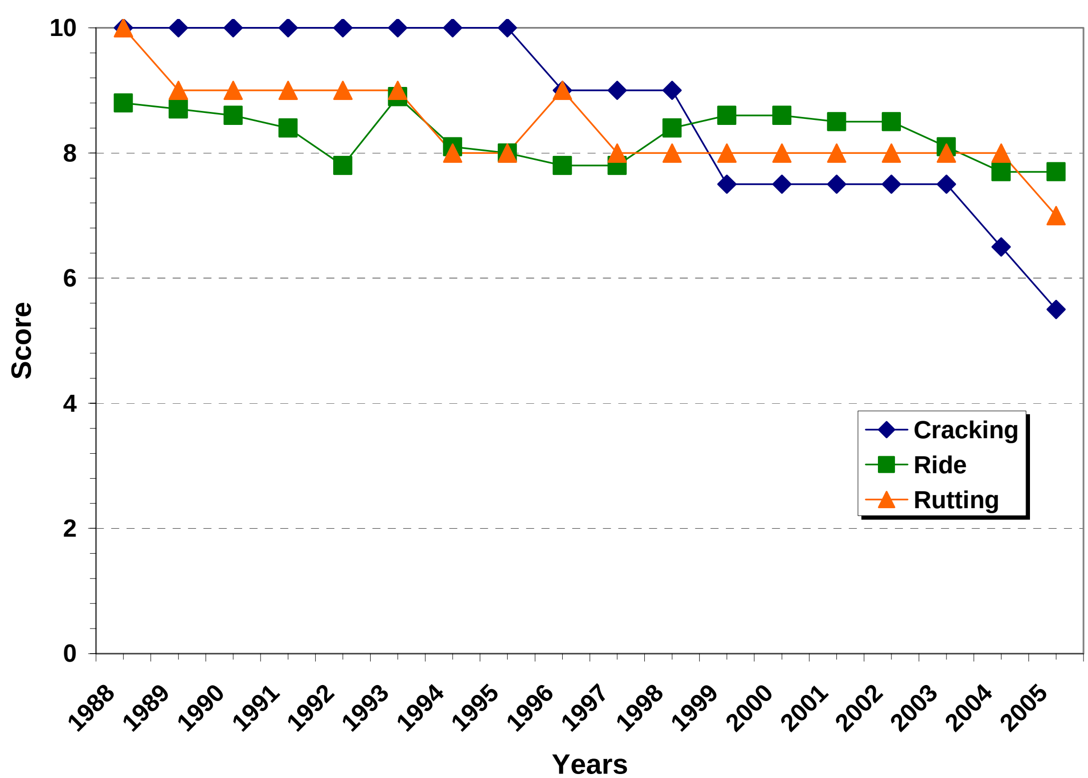

Figure B35. PCS Data on Section 93310000 at Palm Beach County.

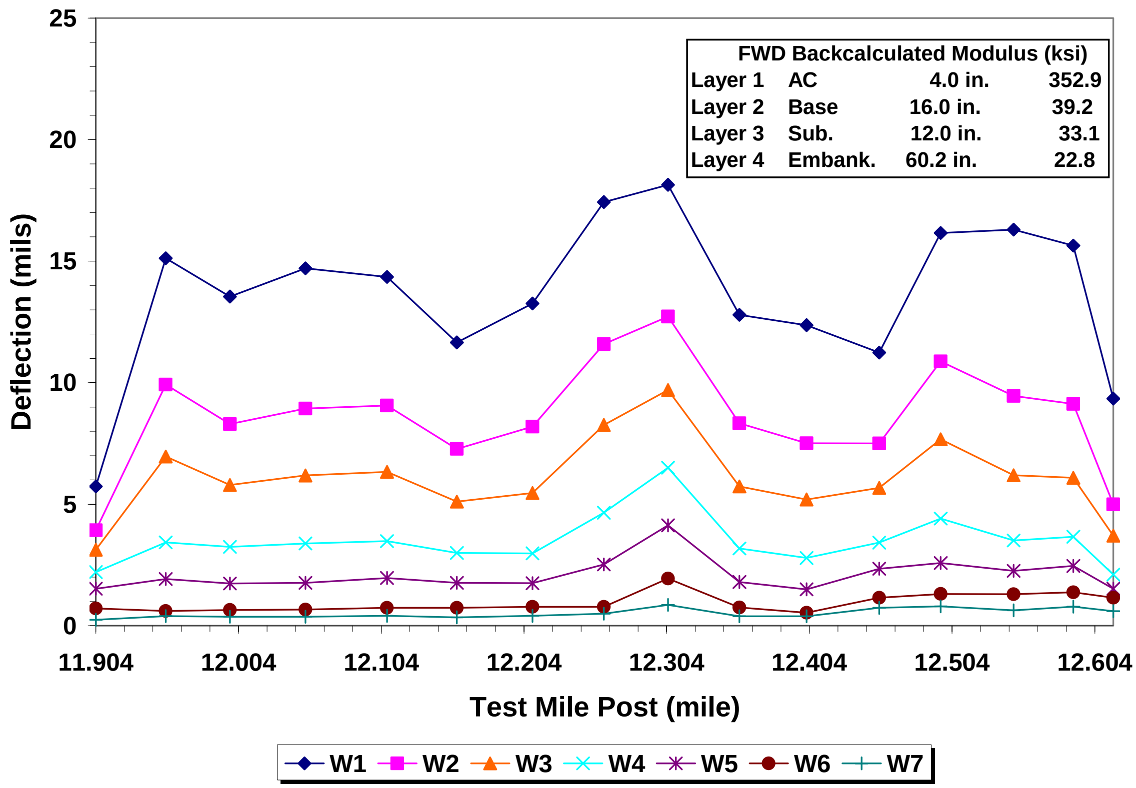

Figure B36. FWD Data on Section 93100000 at Palm Beach County.

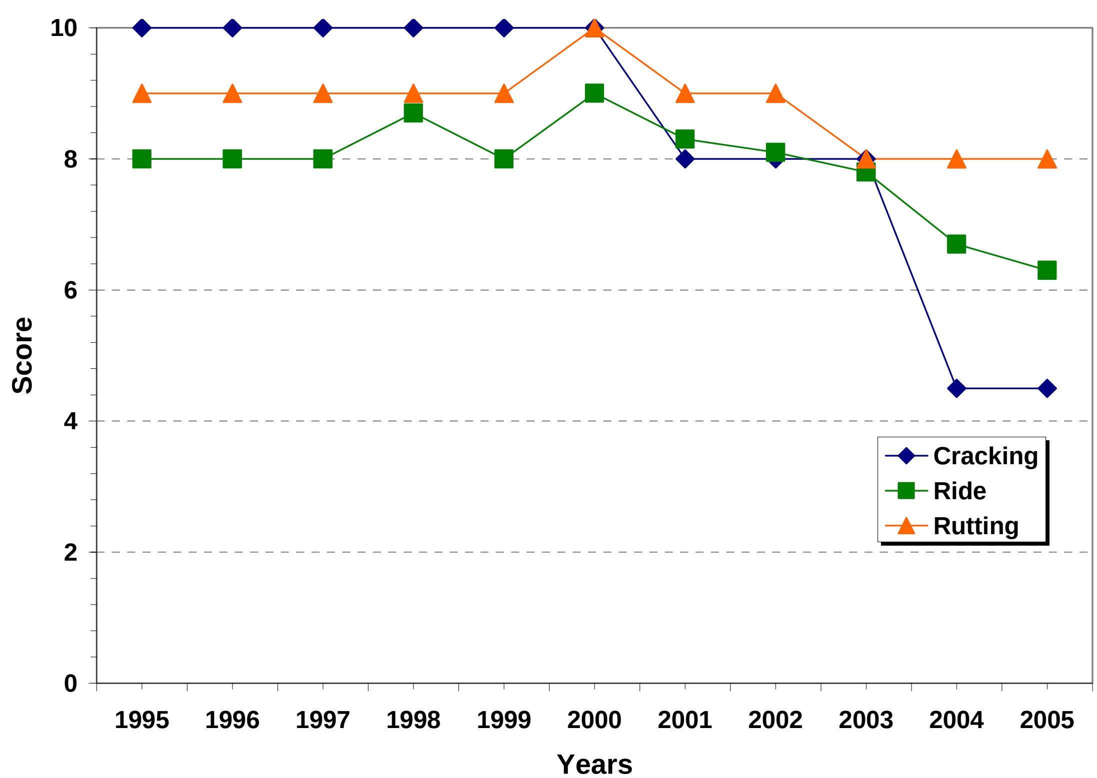

Figure B37. PCS Data on Section 93100000 at Palm Beach County.

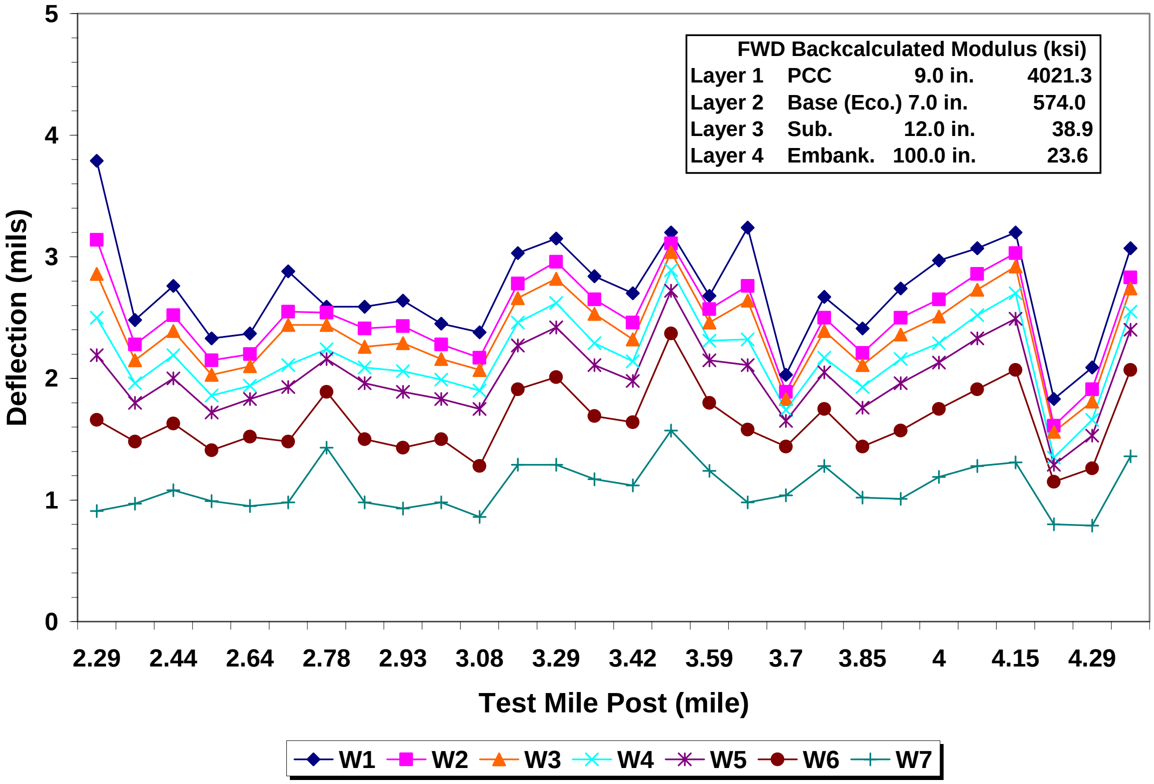

Figure B38. FWD Data on Section 15190000R at Pinellas County.

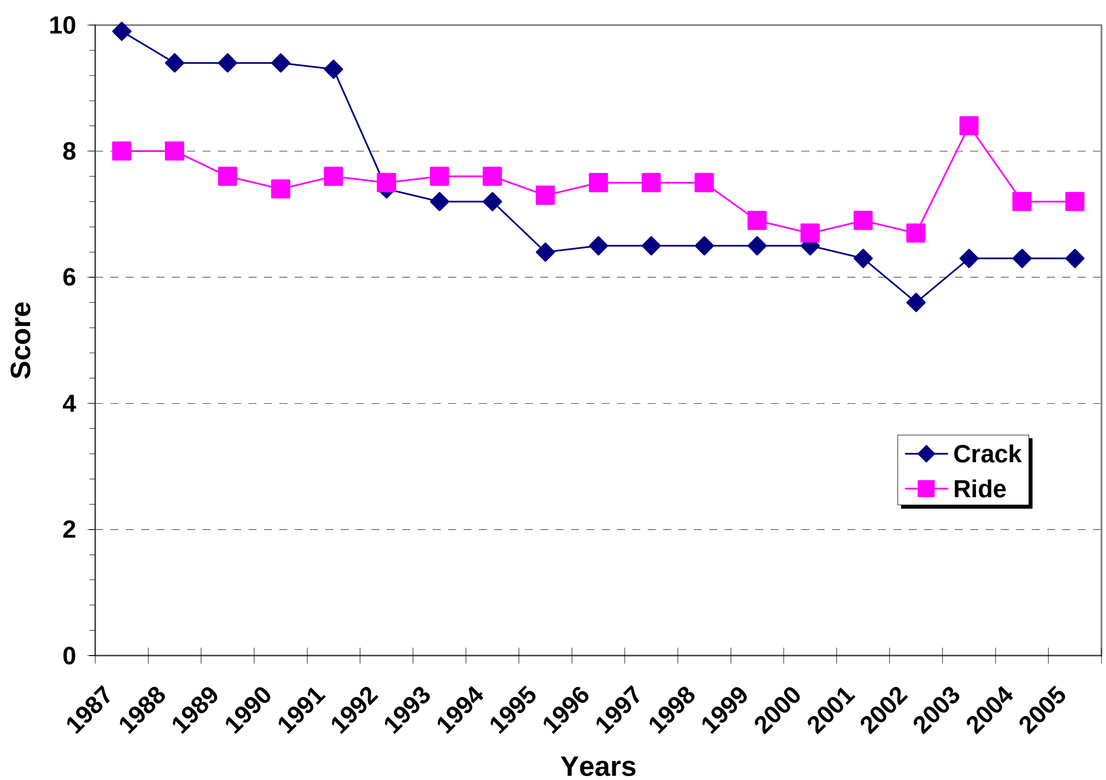

Figure B39. PCS Data on Section 15190000R at Pinellas County.

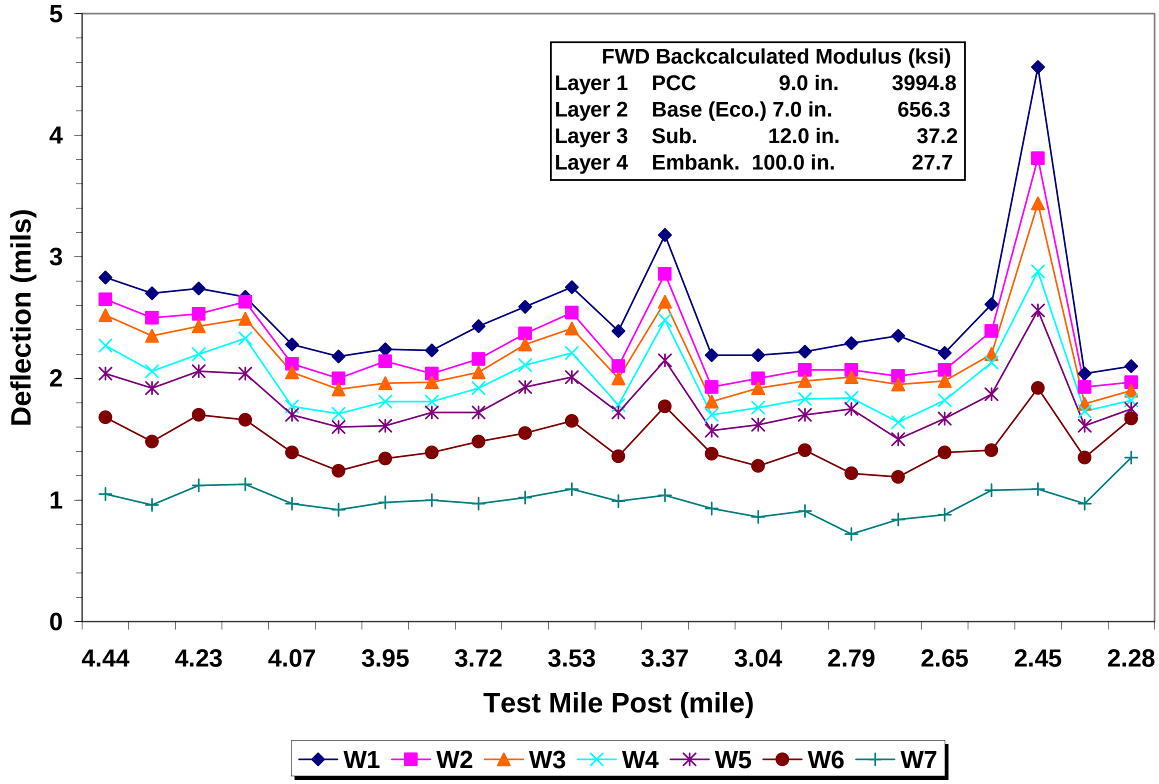

Figure B40. **FWD Data on Section 15190000L at Pinellas County.**

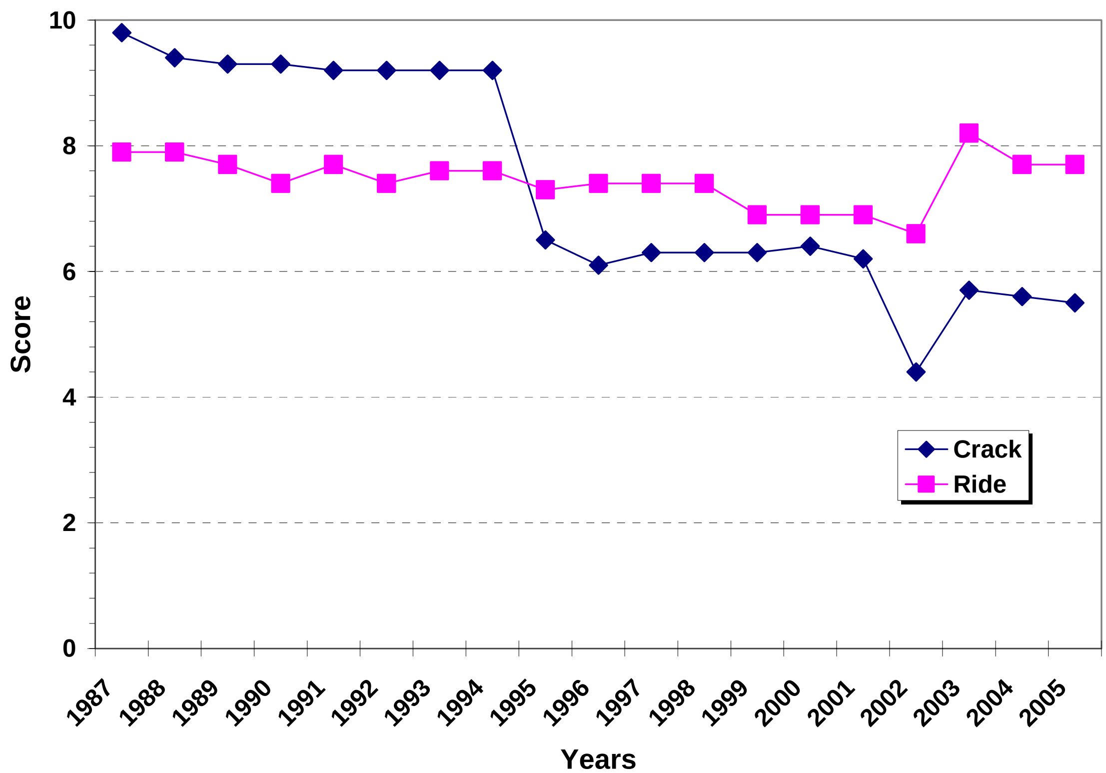

Figure B41. **PCS Data on Section 15190000L at Pinellas County.**

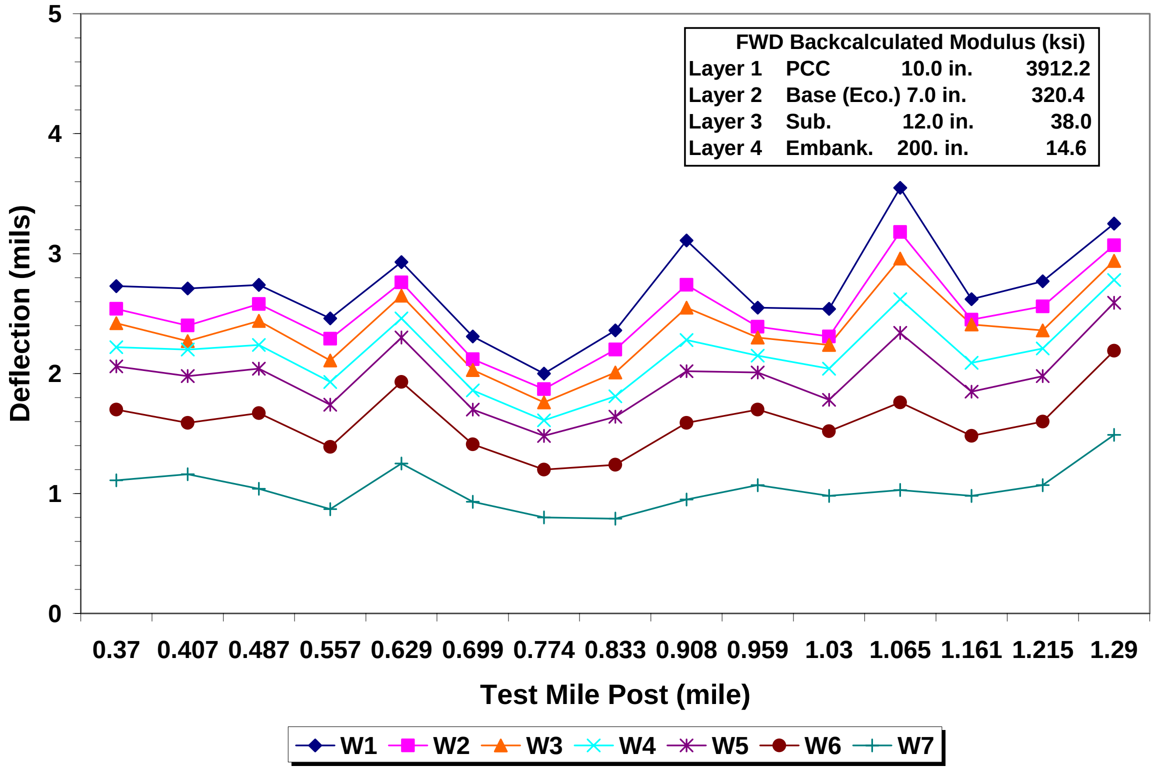

Figure B42. FWD Data on Section 15003000R at Pinellas County.

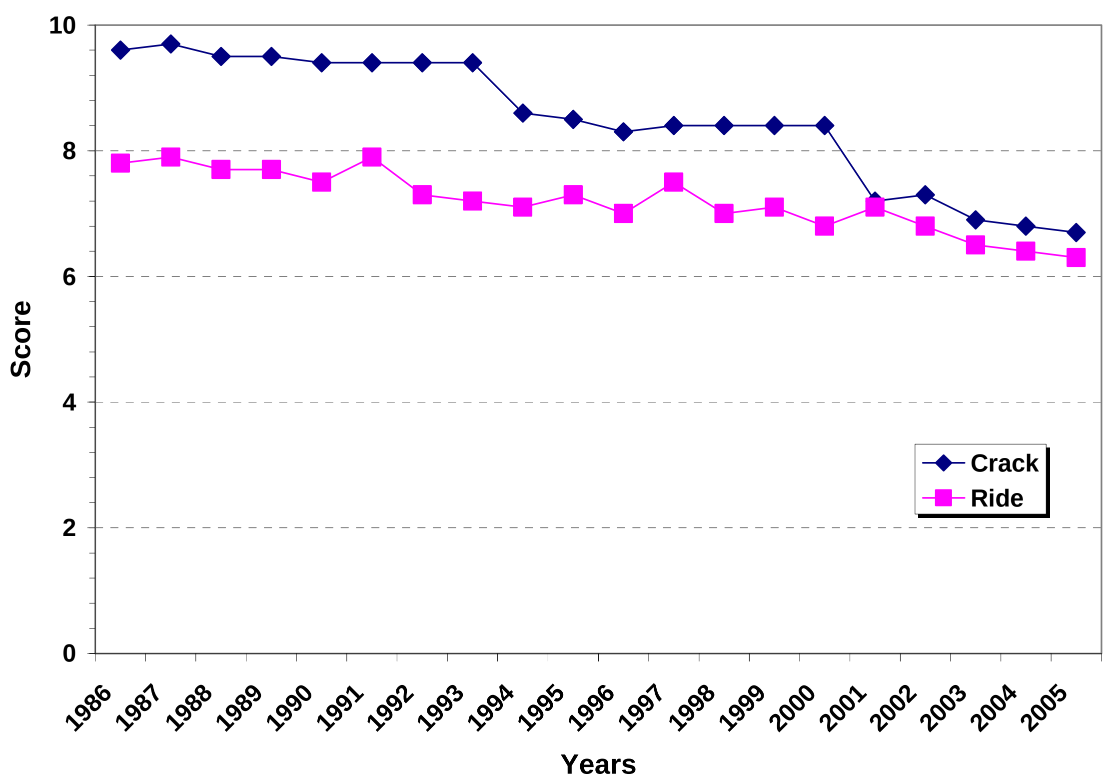

Figure B43. PCS Data on Section 15003000R at Pinellas County.

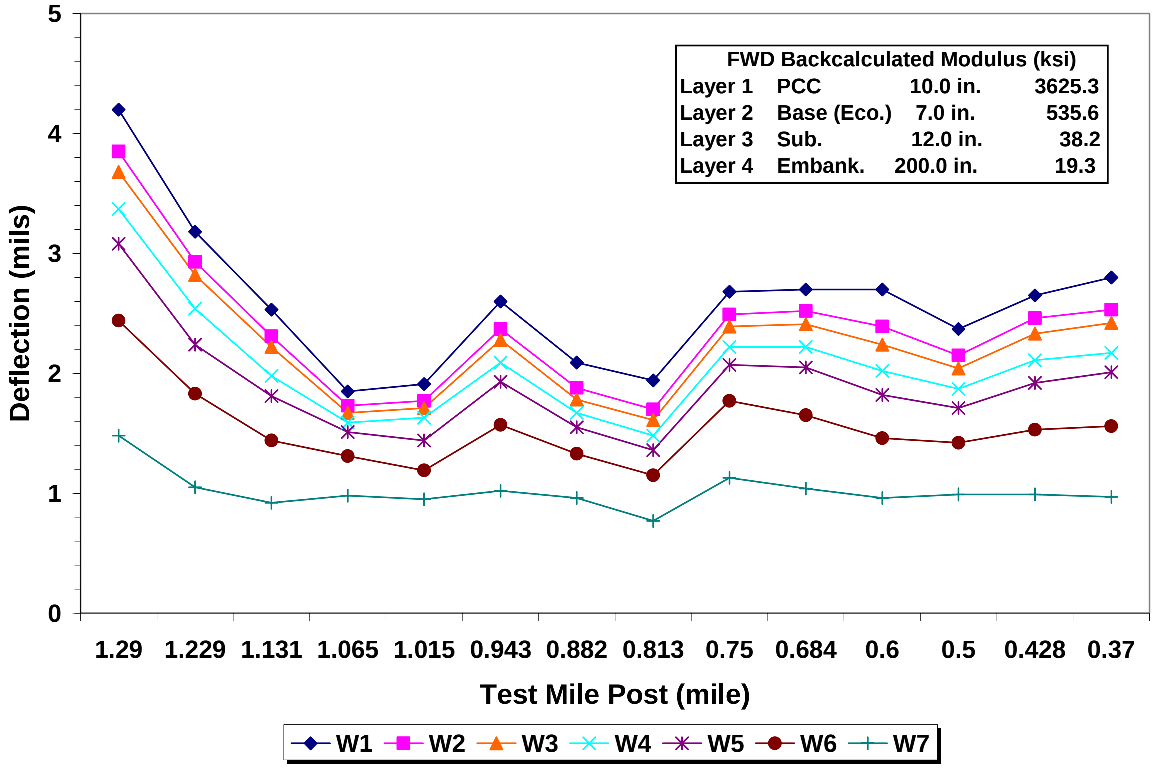

Figure B44. FWD Data on Section 15003000L at Pinellas County.

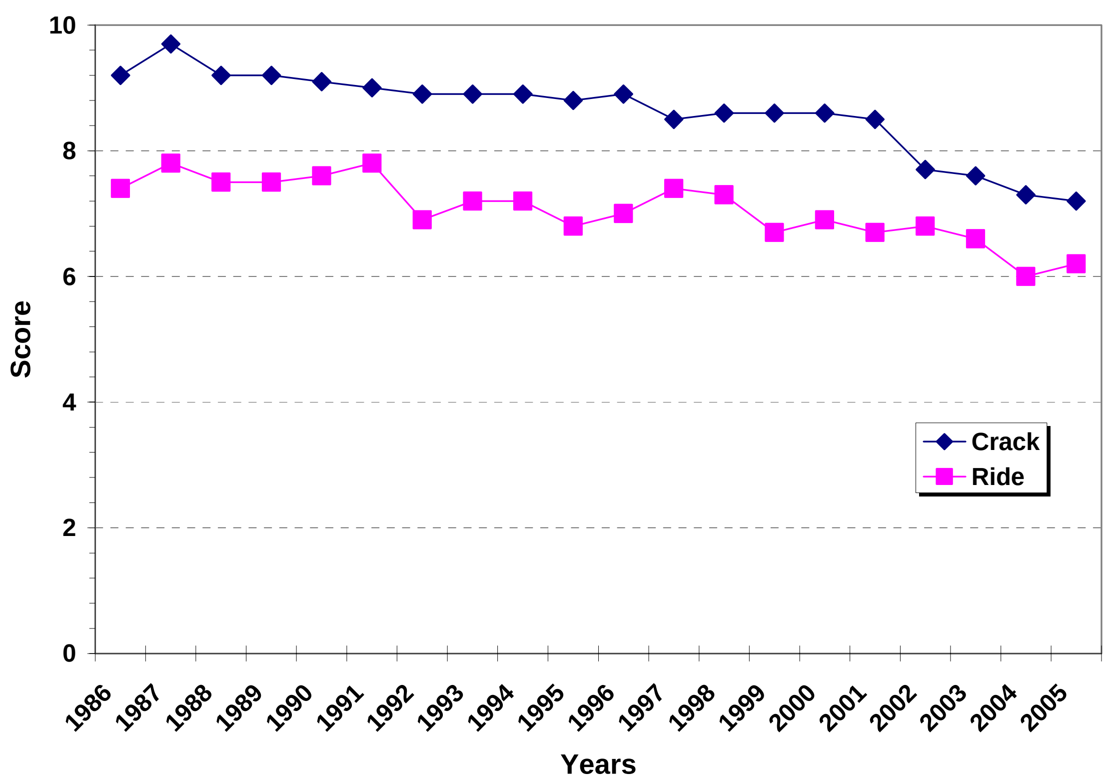

Figure B45. PCS Data on Section 15003000L at Pinellas County.

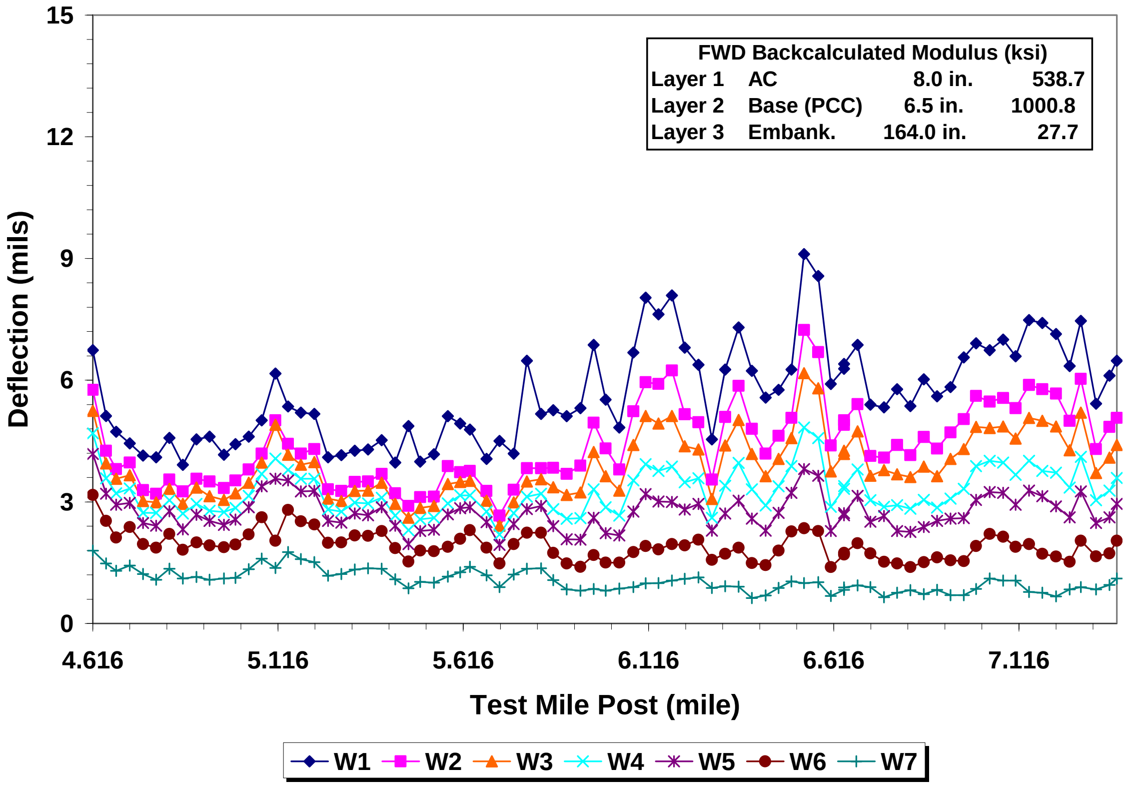

Figure B46. FWD Data on Section 16250000 at Polk County.

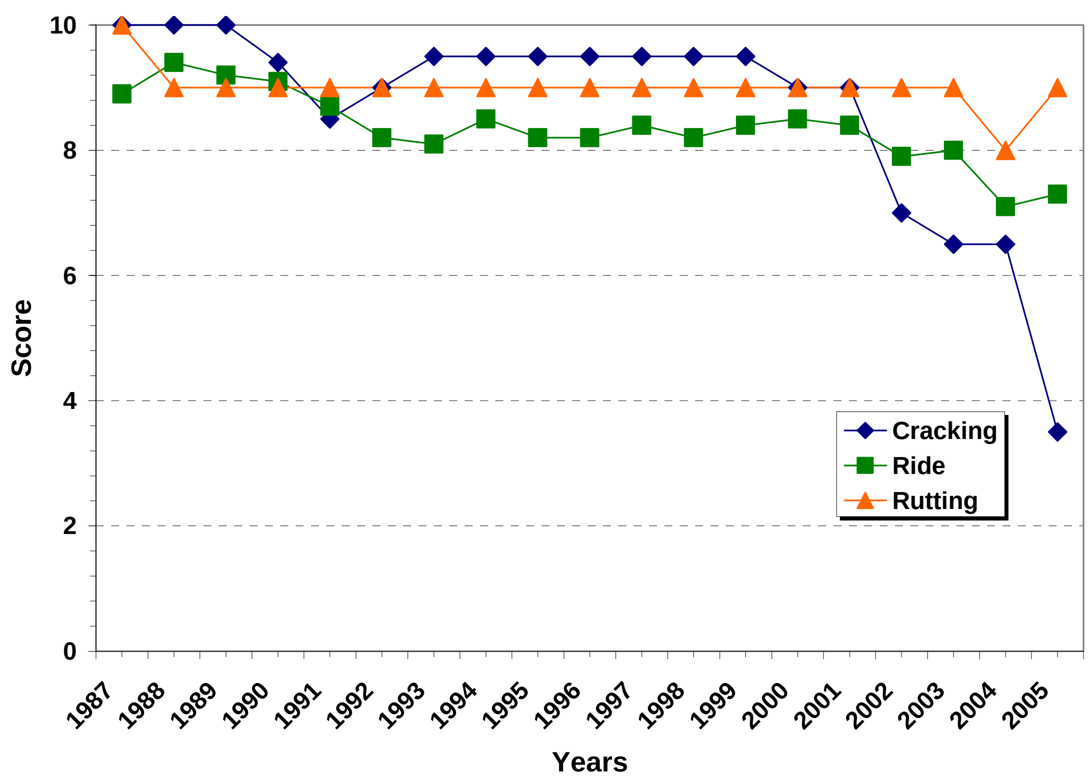

Figure B47. PCS Data on Section 16250000 at Polk County.

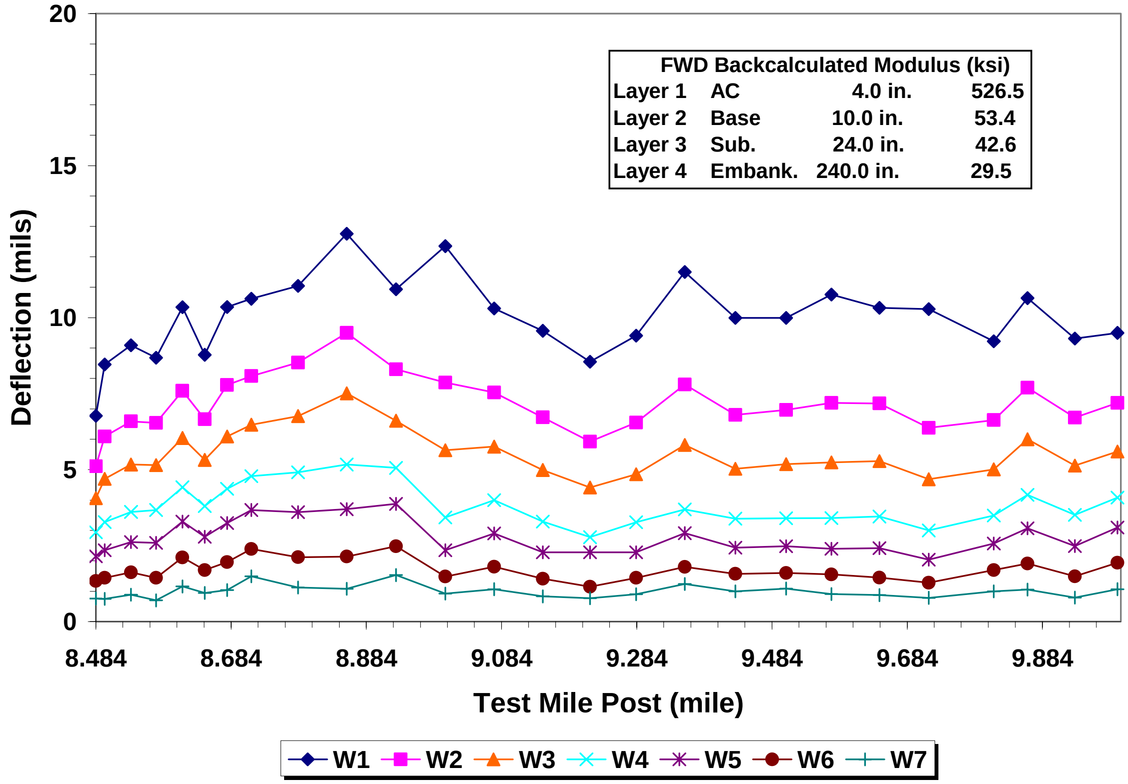

Figure B48. FWD Data on Section 16003001 at Polk County.

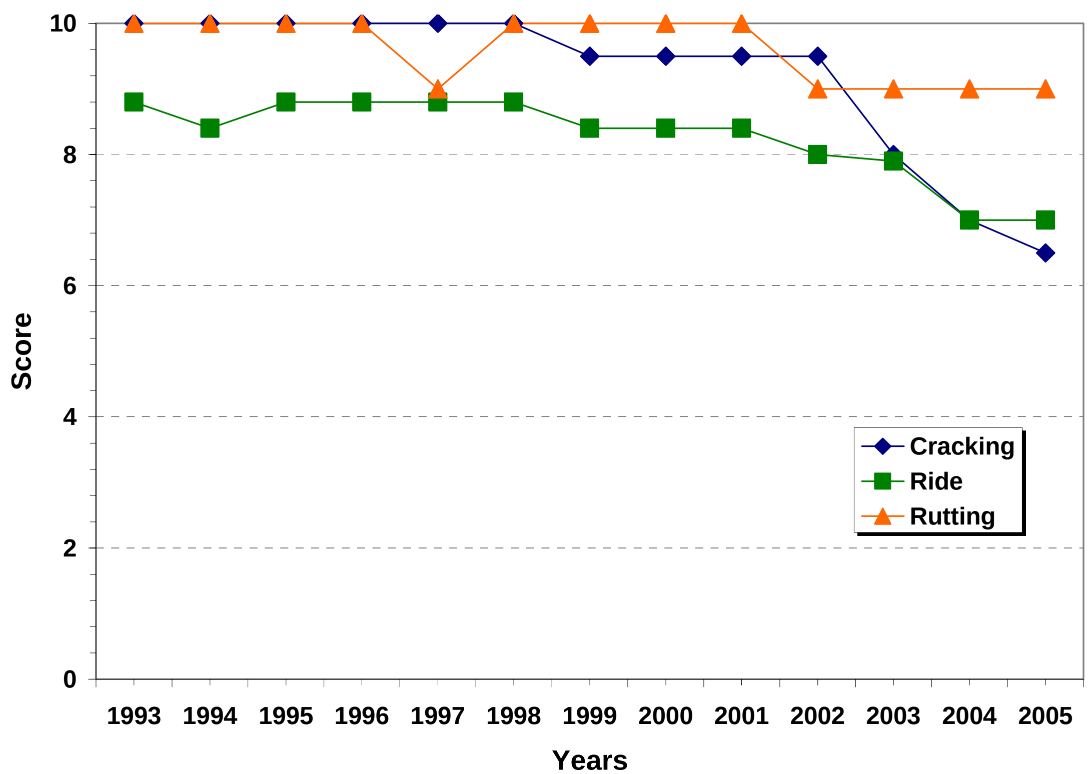

Figure B49. PCS Data on Section 16003001 at Polk County.

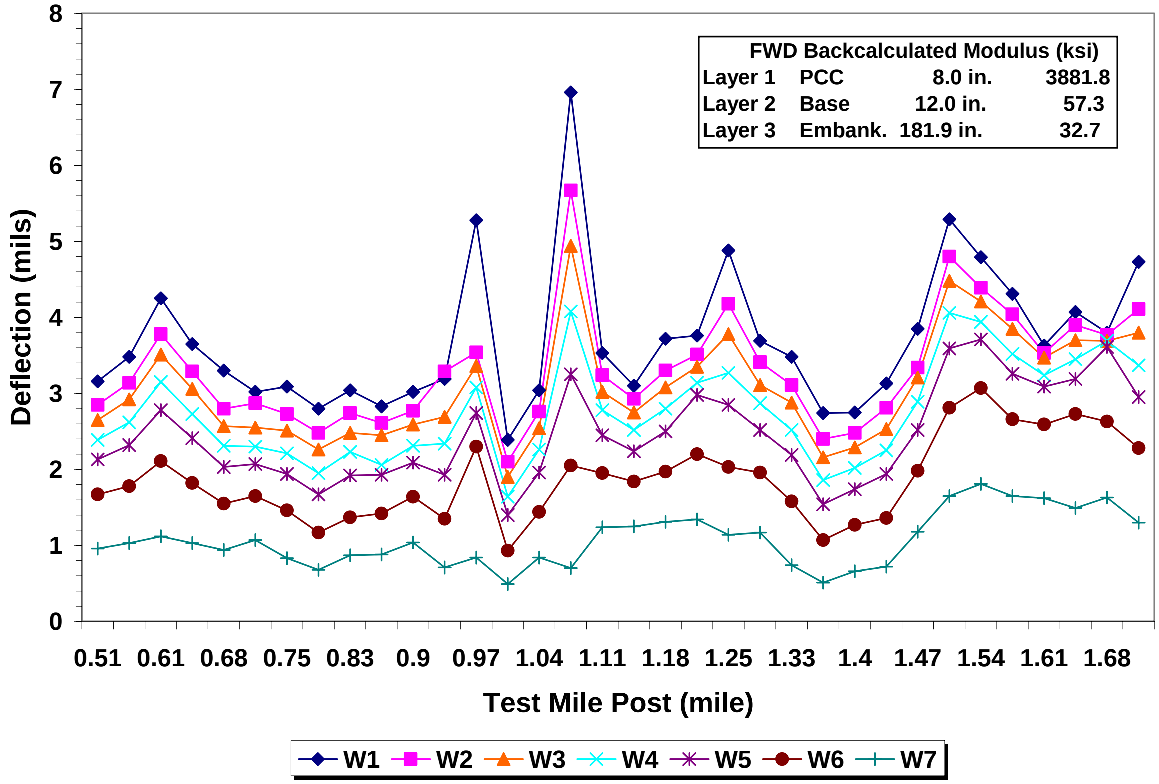

Figure B50. FWD Data on Section 16100000 at Polk County.

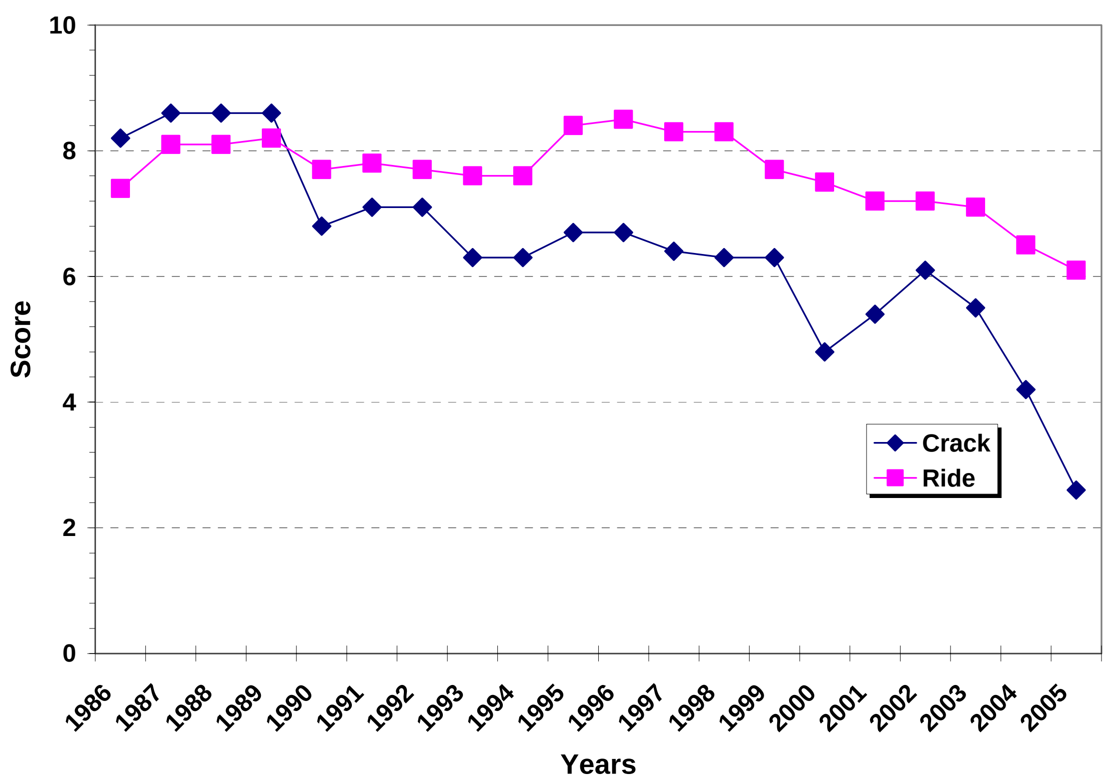

Figure B51. PCS Data on Section 16100000 at Polk County.

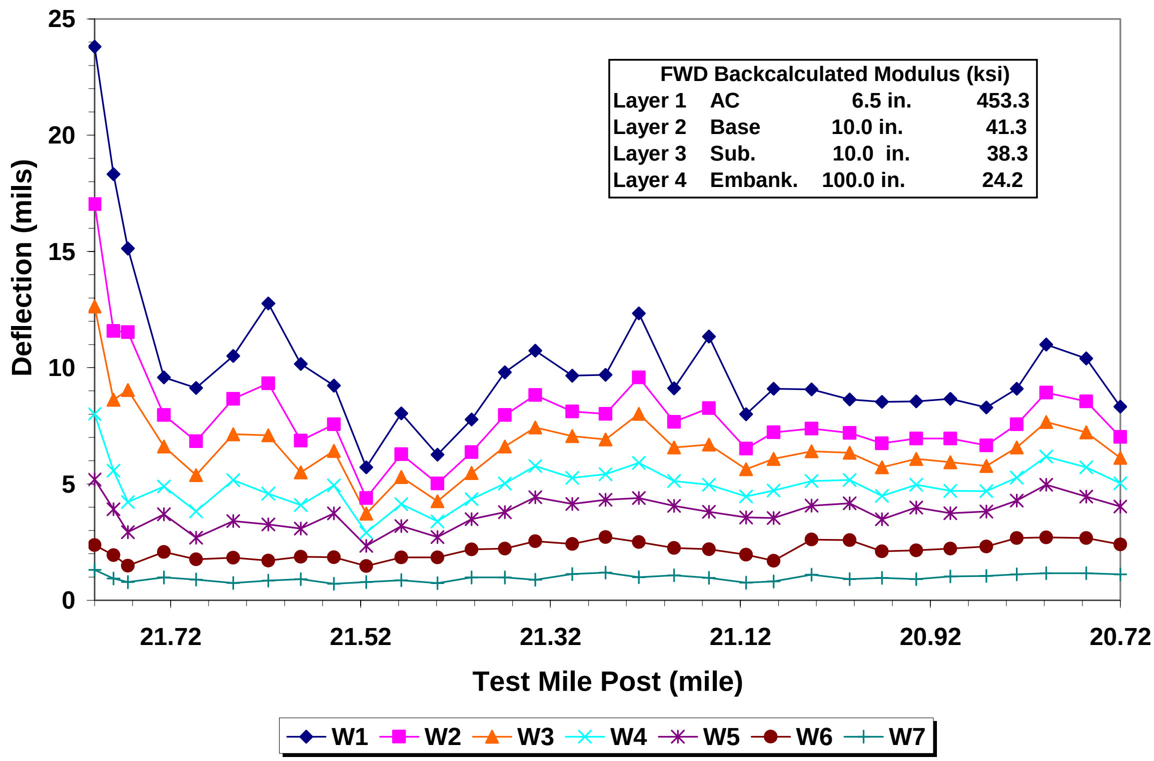

Figure B52. FWD Data on Section 58060000 at Santa Rosa County.

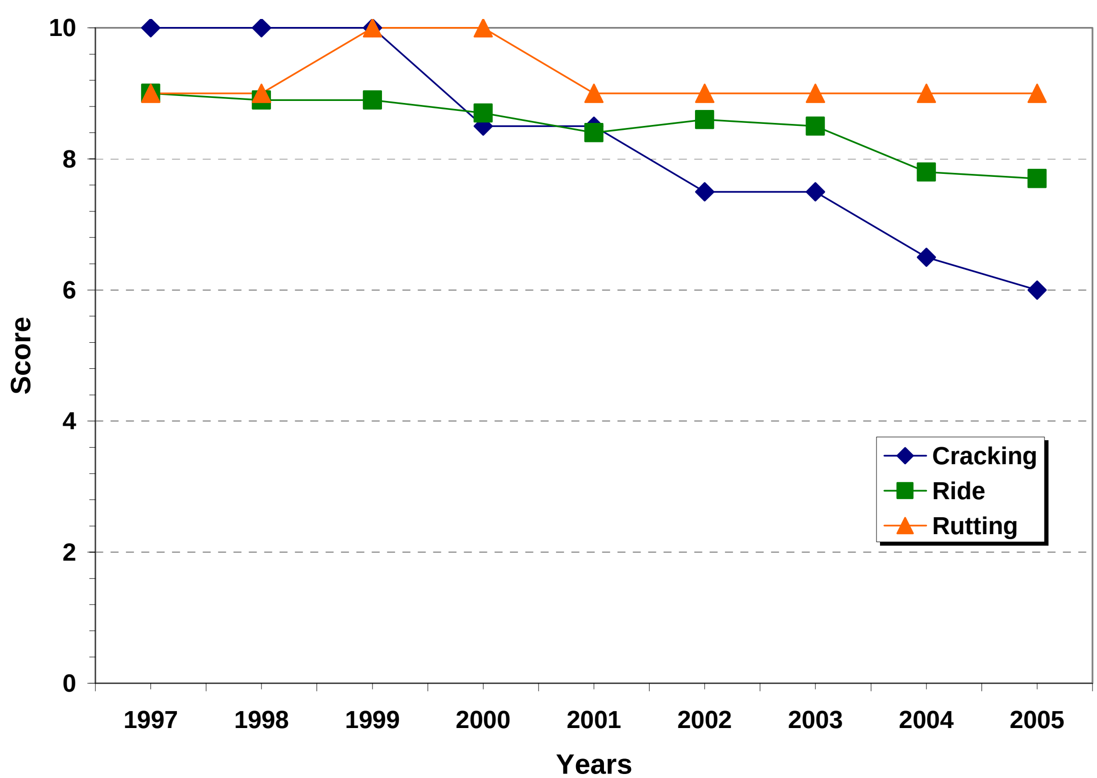

Figure B53. PCS Data on Section 58060000 at Santa Rosa County.

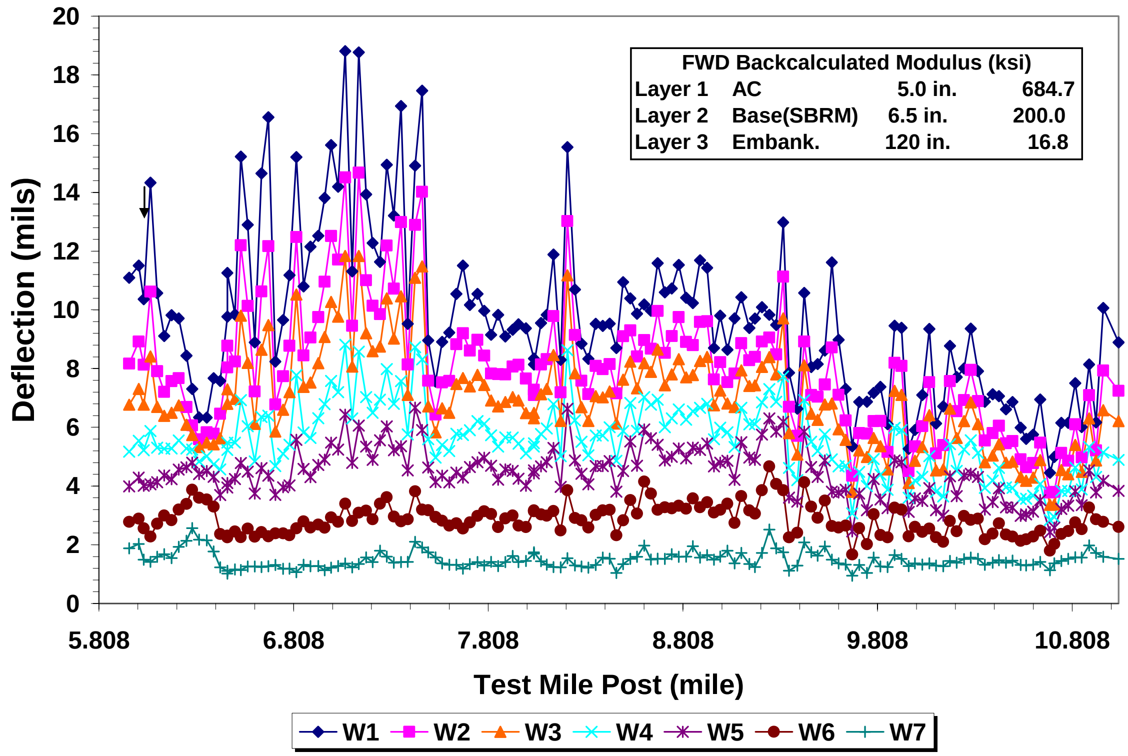

Figure B54. FWD Data on Section 77040000 at Seminole County.

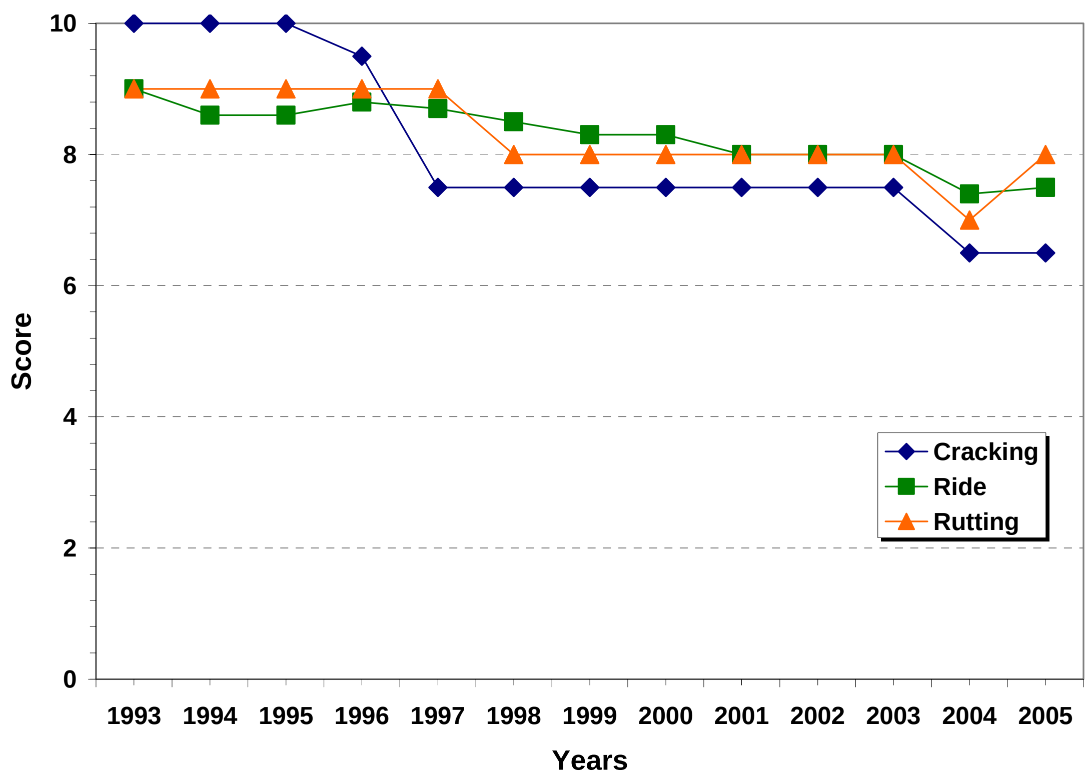

Figure B55. PCS Data on Section 77040000 at Seminole County.

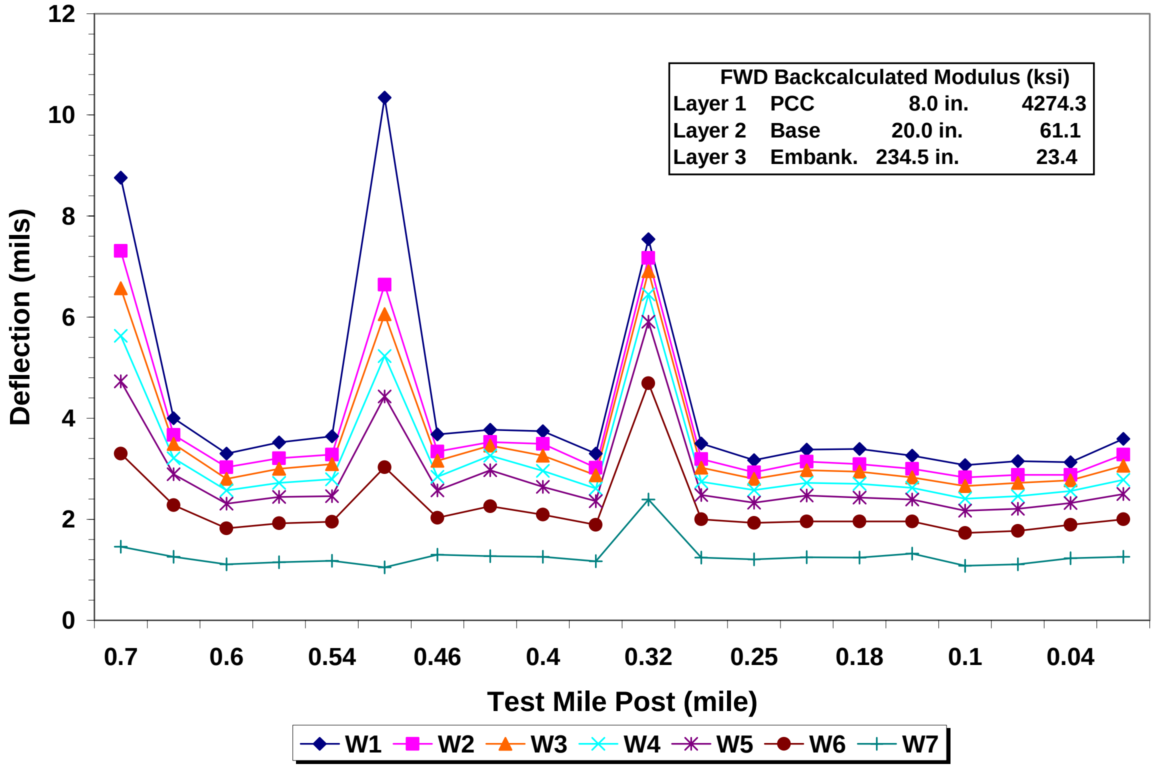

Figure B56. FWD Data on Section 78020000 at St. Johns County.

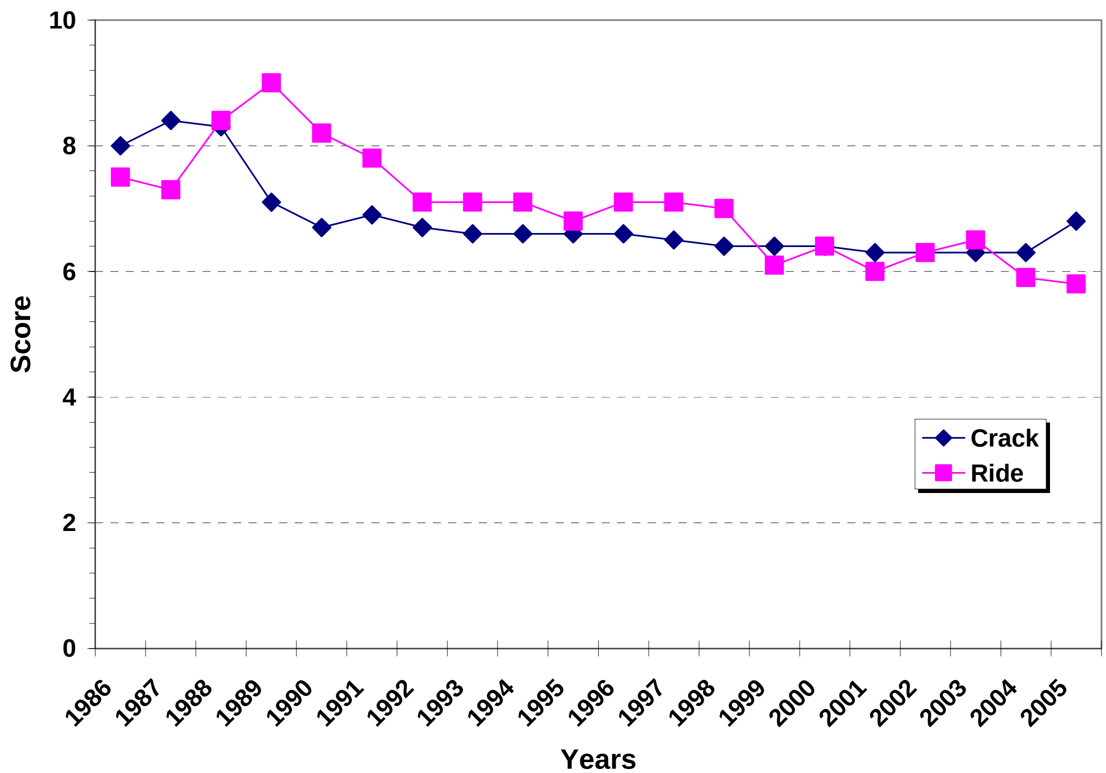

Figure B57. PCS Data on Section 78020000 at St. Johns County.

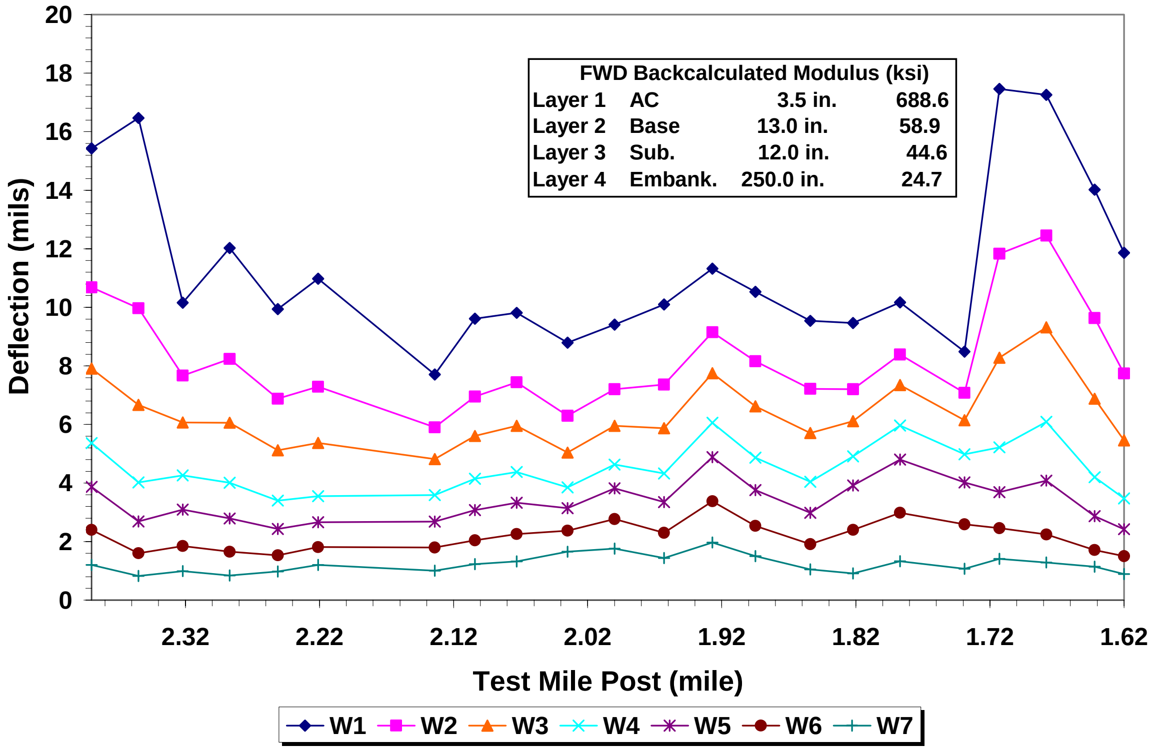

Figure B58. FWD Data on Section 79270000 at Volusia County.

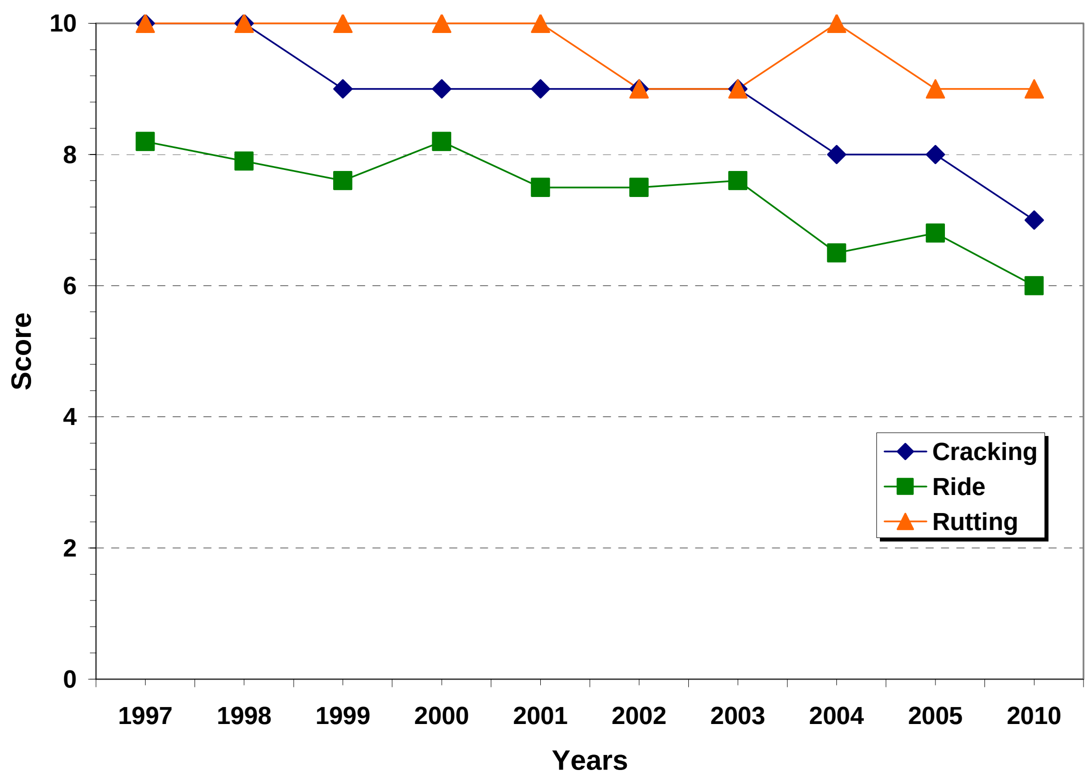

Figure B59. PCS Data on Section 79270000 at Volusia County.

APPENDIX C

SUMMARY OF CORE DATA

Table C1. Core Data on Section 58060000 at SR 89 in Santa Rosa County.

Core No.	Mile Post	FC3	S3	T2	S1	T1	HMAC Core Length (in)	Base		Comments
								SCLY	ST	
1-A-T	21.78	0.9	1	1	1.5	0.7	5.1			
1-B-F		0.9	1.2	1.1	1.4	0.6	5.2			
1-C-T		0.9	1.3	1.1	1.5	0.6	5.4	10	10	
2-A-T	21.693	0.8	1	1.1	2.1	0.6	5.6			
2-B-F		0.8	1	1.1	2.1	0.6	5.6			
3-A-T	21.617	1.1	0.7	2.1	2.2	0.6	6.7			
3-B-F		1	1	2	2.4	0.9	7.3			
3-C-T		1	1	2	2.3	0.7	7			
4-A-T	21.53	1.2	1	2	2.7	0.6	7.5			
4-B-F		1.1	1	2	2.8	0.6	7.5			
5-A-T	21.4	1.3	0.7	1.6	2.7	0.6	6.9			
5-B-F		1.3	0.7	1.6	2.7	0.6	6.9			
5-C-T		1.3	0.7	1.5	2.8	0.6	6.9			
5-D-T		1.2	0.9	1.75	2.5	0.6	6.95			Top Down
6-A-T	21.336	0.8	0.5	1.6	1.8	0.6	5.3			
6-B-F		1	0.6	1.7	1.6	0.6	5.5			
6-C-T		0.9	0.6	1.6	1.6	0.6	5.3			Top Down
7-A-T	21.227	0.7	0.8	2.3	2.1	0.5	6.4	10	9	Ground water table (GWT) = 30’
7-B-F		0.7	0.9	2.1	1.9	0.7	6.3			
7-C-T		0.8	0.8	2.1	2.1	0.7	6.5			
8-A-T	20.971	0.8	0.5	3	1.8	0.6	6.7			
8-B-F		0.9	0.4	2.9	1.7	0.6	6.5			
9-A-T	20.798	0.8	0.4	2.6	3.2	0.6	7.6			
9-B-F		0.7	0.4	2.5	3	0.6	7.2			
9-C-T		0.7	0.4	2.6	3.4	0.7	7.8			

Table C2. Core Data on Section 50010000 at US 90 and SR 10 in Gadsden County.

Core No.	Mile Post	FC3	S3	T2	S1	HMAC Core Length (in)	Base		Comments
							CONC	ST	
1-A-T	16.699	0.5	2.4	1.3	3.5	7.7			Cracks down through all the way
1-B-F		0.5	2.4	1.3	3.5	7.7			
1-C-T		0.5	2.1	1.7	3.4	7.7			
2-A-T	16.977	0.6	2	0.7	2.8	6.1			Cracks down through all the way
2-B-F		0.6	2	0.7	2.8	6.1			
2-C-T		0.6	2	0.7	2.8	6.1			
2-D-T		0.6	2	0.7	2.8	6.1			
3-A-T	17.196	0.5	1.8	0.9	2.6	5.8			Cracks down through all the way
3-B-F		0.5	1.8	0.9	2.6	5.8			
3-C-T		0.6	1.6	1	2.5	5.7			
4-A-T	17.275	0.5	2	1	2.2	5.7			
4-B-F		0.7	1.8	1	2.2	5.7			
4-C-T		0.7	2	1	2.3	6			
5-A-T	17.412	0.6	2	1	2.5	6.1	7	12	Trench Location Cracks down through all the way GWT = 30’
5-B-F		0.6	2	1	2.5	6.1			
5-C-T		0.6	1.9	1	2.5	6			
5-D-T		0.6	2	1	2.5	6.1			
6-A-T	17.765	0.6	2	1.4	2.3	6.3			
6-B-F		0.5	2	1.1	3	6.6			
7-A-T	18.04	0.5	1.6	1.3	2.3	5.7			
7-B-F		0.5	1.6	0.8	2.6	5.5			
8-A-T	18.393	0.5	1.6	1.5	2.4	6			
8-B-F		0.5	1.5	1.1	2.6	5.7			
8-C-T		0.5	1.6	1.6	2.1	5.8			

Table C3. Core Data on Section 123811 at I-10 in Gadsden County.

Core No.	Mile Post	PCC Core Length (in)	Base: Soil Cement	Comments
1-A-T	200 ft	9.8	7	GWT = 27’ Joint = 20’ AC Shoulder = 10’ Lane = 12’
1-B-F		9.5	6.5	
1-C-F		9.75	7	
2-A-T	400 ft	9	6.5	
2-B-F		9.2	7.6	
2-C-F		9.1	7	

Table C4. Core Data on Section 26005000 at SR 222 in Alachua County.

Core No.	Mile Post	FC3	S3	T2	S1	Core Length (in)	Base		Comments
							LR	ST	
1-A-T	10.61	1	3	1.75		5.75			
1-B-F		1	3	1.75		5.75			
2-A-T	10.4	0.9	1	1.6	1.5	5			
2-B-F		0.9	1	1.6	1.5	5			
2-C-T		0.9	1	1.6	1.5	5			
3-A-T	10.1	1.1	1.1	1.3	1.7	5.2			
3-B-F		1.1	1.1	1.3	1.7	5.2			
4-A-T	9.925	1	1.3	1.2	1.4	4.9			
4-B-F		1	1.3	1.2	1.4	4.9			
5-A-T	9.507	0.9	1.2	1.2	1.6	4.9			
5-B-F		0.9	1.2	1.2	1.6	4.9			
5-C-T		0.9	1.2	1.2	1.6	4.9			Top Down
6-A-T	9.3	0.9	1.3	1.4	1.3	4.9			
6-B-F	9.3	0.9	1.3	1.4	1.3	4.9	15	30	
6-C-T		0.9	1.3	1.4	1.3	4.9			
6-D-T		0.9	1.3	1.4	1.3	4.9			
7-A-T	9.1	0.8	1.5	1.3	1.6	5.2			

Table C4. Core Data on Section 26005000 at SR 222 in Alachua County (continued).

Core No.	Mile Post	FC3	S3	T2	S1	Core Length (in)	Base LR	Base ST	Comments
7-B-F		0.8	1.5	1.3	1.6	5.2			
7-C-T		0.8	1.5	1.3	1.6	5.2			
8-A-T	8.58	0.5	1.5	2.5		4.5	15	30	Trench 5.5' GWT Top Down
8-B-F		0.5	1.5	2.5		4.5			
8-C-T		0.5	1.5	2.5		4.5			
9-A-T	8.426	0.9	1.2	1.4	1.3	4.8			
9-B-F	8.426	0.9	1.2	1.4	1.3	4.8			
10-A-T	8.142	1.1	1.2	1.5	1.3	5.1			
10-B-F		1.1	1.2	1.5	1.3	5.1			
10-C-T		1.1	1.2	1.5	1.3	5.1			
10-D-T		1.1	1.2	1.5	1.3	5.1			
11-A-T	8.071	1.1	1.6	0.9	1.5	5.1			
11-B-F		1.1	1.6	0.9	1.5	5.1			
11-C-T		1.1	1.6	0.9	1.5	5.1			

Table C5. Core Data on Section 28040000 at SR 18 in Bradford County.

Core No.	Mile Post	FC3	S3	T2	Core Length (in)	Base SBRM	Comments
1-A-T	0.1	1	0.8	1.2	3		
1-B-F		1	0.8	1.2	3	7	
2-A-T	0.288	0.6	1	1.75	3.35		
2-B-F		0.6	1	1.75	3.35		
2-C-T		0.6	1	1.75	3.35	6	
3-A-T	0.7	0.5	1.25	1.5	3.25		
3-B-F		0.5	1.25	1.5	3.25	5.5	
4-A-T	0.967	0.7	0.9	1.5	3.1		
4-B-F		0.7	0.9	1.5	3.1	6	
5-A-T	1.4	0.8	0.7	1.5	3		
5-B-F		0.8	0.7	1.5	3		
5-C-T		0.8	0.7	1.5	3		Cracks all the way down
5-D-T		0.8	0.7	1.5	3	5.5	
6-A-T	1.7	0.8	1.2	1.3	3.3		
6-B-F		0.8	1.2	1.3	3.3	6.2	
7-A-T	1.967	0.7	0.9	1.5	3.1		
7-B-F		0.7	0.9	1.5	3.1	6.3	
8-A-T	2.357	0.9	0.9	1.5	3.3		
8-B-F		0.9	0.9	1.5	3.3	6	
9-A-T	2.536	0.7	0.6	2	3.3		Trench GWT = 4.5’
9-B-F		0.7	0.6	2	3.3		
9-C-T		0.7	0.6	2	3.3		
9-D-T		0.7	0.6	2	3.3	6	
10-A-T	2.7	0.7	0.7	1.4	2.8		
10-B-F		0.7	0.7	1.4	2.8	6	
11-A-T	3.288	0.6	0.9	1.6	3.1		
11-B-F		0.6	0.9	1.6	3.1	5.3	
12-A-T	3.754	0.8	0.7	1.2	2.7		
12-B-F		0.8	0.7	1.2	2.7		
12-C-T		0.8	0.7	1.2	2.7	5.8	
13-A-T	4.4	0.5	0.8	1.3	2.6		

Table C5. Core Data on Section 28040000 at SR 18 in Bradford County (continued).

Core No.	Mile Post	FC3	S3	T2	Core Length (in)	Base SBRM	Comments
13-B-F		0.5	0.8	1.3	2.6	6	
14-A-T	4.7	0.8	1.4	1.6	3.8		
14-B-T		0.8	1.4	1.6	3.8	6.3	
15-A-T	5.1	0.5	0.8	1.3	2.6		
15-B-F		0.5	0.8	1.3	2.6		
15-C-T		0.5	0.8	1.3	2.6	8.4	
16-A-T	5.323	0.6	0.8	1.3	2.7		
16-B-F		0.6	0.8	1.3	2.7		
16-C-T		0.6	0.8	1.3	2.7		
16-D-T		0.6	0.8	1.3	2.7	6.2	

Table C6. Core Data on Section 78020000 at SR 5/US 1 in St. Johns County.

Core No.	Mile Post	Core Length (in)	Base	Comments
1-A-T	0.502	8.25	Sand	
1-B-F		8.2		
1-C-F		8		
1-D-F		8		
2-A-T	0.323	7.5		Joint = 20', Dowel = 1.25" GWT = 4.5' Sand Embankment
2-B-F		7.6		
2-C-F		7.5		
2-D-F		7.7		
3-A-T	0.125	8		
3-B-F		7.9		
3-C-F		7.8		
3-D-F		8		

Table C7. Core Data on Section 77040000 at SR 46 in Seminole County.

Core No.	Mile Post	FC4	S3	S1	T1			Core Length (in)	Base SBRM	Comments
1-A-T	6.073	1	1	2	0.5			4.5	6.7	
1-B-F		1	1	2	0.5			4.5		
1-C-T		1	1	2	0.5			4.5		Crack all the way down
		FC4	**S3**	**S**	**S1**	**T**				
2-A-T		0.7	1.25	0.8	2.75	0.7		6.2	6	
2-B-F	6.222	0.7	1.25	0.8	2.75	0.7		6.2		
2-C-T		0.7	1.25	0.8	2.75	0.7		6.2		Bottom up
		FC4	**S3**	**S1**	**T**					
3-A-T	6.352	0.5	1.8	7.5	1.2			11	6	Widened section
3-B-F		0.8	3.5	5.5	1.5			11.3		Different core layer
4-A-T	6.682	0.8	0.8	1.8	0.9			4.3	6	

Table C7. Core Data on Section 77040000 at SR 46 in Seminole County (continued).

Core No.	Mile Post	FC4	S3	S1	T1			Core Length (in)	Base	Comments
									SBRM	
4-B-F		0.8	0.8	1.8	0.9			4.3		
5-A-T	7.143	0.6	1.1	2	1.1			4.8	6	
5-B-F		0.6	1	1.5	1			4.1		
6-A-T	7.466	0.5	0.8	2	1.4			4.7	6	Trench Location
6-B-F		0.5	0.8	2	1.4			4.7		Crack all the way down
6-C-T		0.5	0.8	2	1.4			4.7		GWT = 10’
7-A-T	7.643	0.5	1	1.7	0.8			4	6	
7-B-F		0.5	1	1.7	0.8			4		
7-C-T		0.5	1	1.7	0.8			4		
7-D-T		0.5	1	1.7	0.8			4		Top down
		FC4	**S3**	**S1**	**T2**	**T3**				
8-A-T	8.214	0.8	0.9	1.75	0.6	1.3		5.35	4.5	
8-B-F		0.8	0.9	1.75	0.6	1.3		5.35		
9-A-T	8.643	0.6	0.75	1.5	0.7	1.5		5.05	5.5	
9-B-F		0.6	0.75	1.5	0.7	1.5		5.05		
9-C-T		0.6	0.75	1.5	0.7	1.5		5.05		
10-A-T	9.323	0.5	0.7	1.5	1	1.1		4.8	6	
10-B-F		0.5	0.7	1.5	1	1.1		4.8		
		FC4	**S3**	**S1**	**T**					
11-A-T	9.672	0.9		2.5	1.2			4.6	6.25	
11-B-F		0.8		2.5	1.4			4.7		
11-C-T		0.9		2.5	1.2			4.6		
12-A-T	10.282	0.6	1	1.2	1.2			4	5.5	
12-B-F		0.7	1.1	1.5	1.1			4.4		
12-C-T		0.6	1	1.2	1.2			4		
13-A-T	10.69	0.8	1	1.7	1.4			4.9		
13-B-F		0.8	1	1.7	1.4			4.9	7	
		FC4	**FC3**	**S3**	**S1**	**T3**	**T2**			
14-A-T	10.922	1	0.7	1	1.5	0.6	0.6	4.8	5.5	Different layers due to
14-B-F		1	0.7	1	1.5	0.6	0.6	4.8		the location close to
14-C-T		1	0.7	1	1.5	0.6	0.6	4.8		intersection
14-D-T		1	0.7	1	1.5	0.6	0.6	4.8		centerline widened

Table C8. Core Data on Section 11020000 at SR 33 in Lake County.

Core No.	Mile Post	Core Length (in)	Base Permeable Base	Comments
1-A-T	14.06	12	4	
1-B-F		11.5		
1-C-F		12		
2-A-T	14.14	13	4	Joint = 18', Dowel = 1.25"
2-B-F		13		Lane width = 12'GWT = 15'
2-C-F		13		8.5' PCC shoulder

Table C9. Core Data on Section 79270000 at SR 483 in Volusia County.

Core No.	Mile Post		T3	T2	Core Length (in)	Base LR	Base STAB	Comments
1-A-T	2.34		1.25	1	2.25			
1-B-F			1.25	1	2.25			
		FC3	**S1**	**T2**				
2-A-T	2.327	0.75	1.3	1.75	3.8			
2-B-F		0.75	1.3	1.75	3.8			
2-C-T		0.75	1.3	1.75	3.8			
3-A-T	2.134	0.9	1.4	1.8	4.1			
3-B-F		0.9	1.4	1.8	4.1			
4-A-T	2	1.2	1.2	1.9	4.3			
4-B-F		1.2	1.2	1.9	4.3			
		FC3	**S3**	**S1**				
5-A-T	1.927	1.2	1	2	4.2			
5-B-F		1.2	1	2	4.2			
			FC3	**S1**				
6-A-T	1.713		0.8	8	8.8			Different layers
6-B-F			0.8	8	8.8			Widened section
6-C-T			0.8	8	8.8			Top down
7-A-T	1.655		0.7	1.75	2.45	13	12	Trench
7-B-F			0.7	1.75	2.45			Top down
7-C-T			0.7	1.75	2.45			GWT = 7.6'
7-D-T			0.7	1.75	2.45			

Table C10. Core Data on Section 16250000 at SR 37 in Polk County.

Core No.	Mile Post	FC4	S	T1	ST	Core Length (in)	Base CONC	Comments
1-A-T	4.652	0.6	2.2	1.7	2.7	7.2		
1-B-F		0.6	2.2	1.7	2.7	7.2		
2-A-T	4.881	0.8	3.5	3		7.3		
2-B-F		0.8	3.5	3		7.3		
3-A-T	5.247	0.8	3.3	2.5		6.6	6.5	Trench Bottom up crack
3-B-F		0.8	3.5	2.2		6.5		
3-C-T		0.5	3.5	10		14		
4-A-T	5.432	0.8	3.5	3.8		8.1		
4-B-F		0.8	3.5	3.8		8.1		
4-C-T		0.8	3.5	3.8		8.1		
5-A-T	5.789	0.8	3.6	3.5		7.9		Most cracks are along with widened lane GWT = TBD
5-B-F		0.7	4	3.2		7.9		
5-C-T		0.8	3.6	3.5		7.9		
5-D-T		0.6	4.2	3.5		8.3		
6-A-T	5.895	1	3.6	4		8.6		
6-B-F		1	3.6	4		8.6		
7-A-T	6.143	1	4.8	2.3	0.5	8.6		
7-B-F		1	4.8	2.3	0.5	8.6		
8-A-T	6.287	1	5.2	2.5	0.5	9.2		
8-B-F		1	5.2	2.5	0.5	9.2		
9-A-T	6.536	1	5	2.5	0.6	9.1		
9-B-F		1	5	2.5	0.6	9.1		
10-A-T	6.859	1	4	2.5	0.6	8.1		
10-B-F		1	4	2.5	0.6	8.1		
10-C-T		0.7	4	2.5	0.6	7.8		
11-A-T	7.143	0.7	4	2.5	0.6	7.8		
11-B-F		0.7	4	2.5	0.6	7.8		

Table C11. Core Data on Section 16003001 at SR 563 in Polk County.

Core No.	Mile Post	FC4	S	Core Length (in)	Base LR	Base ST	Comments
1-A-T	8.606	0.6	3	3.6			
1-B-F		0.6	3	3.6			
2-A-T	8.736	0.8	3.2	4	10	24	Trench
2-B-F		0.8	3.2	4			GWT =TBD
2-C-T		0.8	3.2	4			Top down crack
3-A-T	8.8	0.7	3.5	4.2			
3-B-F		0.7	3.5	4.2			
3-C-T		0.7	3.5	4.2			
4-A-T	8.996	0.7	3	3.7			
4-B-F		0.7	3	3.7			
4-C-T		0.7	3	3.7			Top down crack
5-A-T	9.2	0.7	2.8	3.5			
5-B-F		0.7	2.8	3.5			
6-A-T	9.6	0.7	3.5	4.2			
6-B-F		0.7	3.5	4.2			
6-C-T		0.7	3.5	4.2			
6-D-T		0.7	3.5	4.2			Top down crack
7-A-T	9.886	0.7	3.2	3.9			
7-B-F		0.7	3.2	3.9			

Table C12. Core Data on Section 16100000 at US 92/SR 546 in Polk County.

Core No.	Mile Post	Core Length (in)	Base LR	Comments
1-A-T	0.75	8		
1-B-F		8		
1-C-F		7.8		
2-A-T	0.95	7.8		
2-B-F		7.8	12	
2-C-F		7.8		
3-A-T	0.967	7.6		20' joint space
3-B-F		7.6		12' lane width
3-C-F		7.6		1" dowel bar
4-A-T	1.074	7.5		4' AC widened shoulder
4-B-F		7.5		
4-C-F		7.7		
5-A-T	1.538	6.5		
5-B-F		8	12	
5-C-F		8		
6-A-T	1.59	9.1		
6-B-T		10		
6-C-F		9		

Table C13. Core Data on Section 01010000 at US 41/SR 45 in Charlotte County.

Core No.	Mile Post	Core Length (in)	Base		Comments
			ECONC	ST	
1-A-T	3.092	4			
1-B-F		3.8	9		
1-C-F		5	9.2	6	
2-A-T	2.563	4	7.7		15' joint space
2-B-F		3.8	8.3		No dowel
2-C-F		3.8	8.2	6	4.5' AC widen, 12' lane

Table C14. Core Data on Section 10075000A at I-75 in Hillsborough County.

Core No.	Mile Post	Core Length (in)	Base #57 stone stab	Comments
1-A-T	19.071	13.5	8	
1-B-F		13.5		
1-C-F		13.5		
2-A-T	19.67	13.5	8	13.5' and 16.5' Joint 1.25" Dowel 9.5' PCC shoulder 14' Lane width
2-B-F		13.5		
2-C-F		13.5		
3-A-T	20.003	14.2	8	
3-B-F		14.2		
3-C-F		14.2		

Table C15. Core Data on Section 10075000B at I-75 in Hillsborough County.

Core No.	Mile Post	Core Length (in)	Base		Comments
			ECONC	Embank	
1-A-T	23.56	12.5			
1-B-F		12.8			
1-C-F		12.5			
2-A-T	23.74	12.6			13.5' and 16.5' Joint 1.25" Dowel 9.5' PCC shoulder
2-B-F		12.4			
2-C-F		12.6			
3-A-T	23.99	12.8	7.0		12' Lane width
3-B-F		12.7			
3-C-F		12.8			
4-A-T	24.5	12.5			
4-B-F		12.5			
4-C-F		12.5	6.5	sand	

Table C16. Core Data on Section 10250001 at 22nd St. at SR 585 in Hillsborough County.

Core No.	Mile Post	Core Length (in)	Base	Comments
			AC Permeable	
1-A-T	0.365	12.5	4	
1-B-F		12.5		
1-C-F		12.5		
2-A-T	0.565	12.5	5	15' Joint 12' Lane 1.375" Dowel 9.5' PCC shoulder
2-B-F		12.5		
2-C-F		12.5		
3-A-T	0.92	12	4	
3-B-F		14.2		
3-C-F		12.2		

Table C17. Core Data on Section 10060000 at US 41/SR 45 in Hillsborough County.

Core No.	Mile Post	FC	S	T1	ST	S	Core Length (in)	Base		Comments
								LR	STAB	
1-A-T	9.37	0.3	1.2	0.5	2.1		4.1			
1-B-F		0.3	1.2	0.5	2.1		4.1			
1-C-T		0.3	1.2	0.5	2.1		4.1			Top down crack
2-A-T	9.84	0.3	1.2	0.5	2		4			
2-B-F		0.3	1.2	0.5	2		4			
2-C-T		0.3	1.2	0.5	2		4			Top down crack
3-A-T	10.281	0.4	1.1	0.6	1.9		4			
3-B-F		0.4	1.1	0.6	1.9		4			
4-A-T	10.5	0.5	1.5	0.6	2		4.6	9	16	Trench
4-B-F		0.5	1.5	0.6	2		4.6			
4-C-T		0.5	1.5	0.6	2		4.6			
4-D-T		0.5	1.5	0.6	2		4.6			Top down crack
5-A-T	10.642	0.5	1.4	0.6	2.2		4.7			
5-B-F		0.5	1.4	0.6	2.2		4.7			
5-C-T		0.5	1.4	0.6	2.2		4.7			Top down crack

Table C17. Core Data on Section 10060000 at US 41/SR 45 in Hillsborough County (continued).

Core No.	Mile Post	FC	S	T1	ST	S	Core Length (in)	Base		Comments
								LR	STAB	
6-A-T	11.377	0.5	1.4	0.6	2.1		4.6			
6-B-F		0.5	1.3	0.6	2.2		4.6			
7-A-T	11,785		4.5	0.8	2.2		7.5			
7-B-F			4.5	0.8	2.2		7.5			
8-A-T	12.4		3.6	0.8	2		6.4			
8-B-F			3.6	0.8	2		6.4			
8-C-T			3.6	0.8	2		6.4			
9-A-T	13.085	1.9	1.9	1.7	2.5	3	11			
9-B-F		1.9	1.9	1.7	2.5	3	11			
10-A-T	13.661		2.9	2	1.5	3	9.4			
10-B-F			2.9	2	1.5	3	9.4	9	16	
10-C-T										Broken
11-A-T	14.05	4	0.5	1.3	2	3.2	11			
11-B-F		4	0.5	1.3	2	3.2	11			
12-A-T	14.238	3.5	1.5	1	2.2	3	11.2			Cores at section 13, 14, and 16 were withdrawn due to partially broken & enough cores to test
12-B-F		3.5	1.5	1	2.2	3	11.2			
15-A-T	16.634		3.5	1	2	3.3	9.8			
15-B-F			3.5	1	2	3.3	9.8			

Table 18. Core Data on Section 10160000 at SR 597 in Hillsborough County.

Core No.	Mile Post		FC4	S	T1	BIND		Core Length (in)	Base		Comments
									LR	ST	
1-A-T	0.282		1	2.8	4.5	1.7		10			.5 " TOP IS APART
1-B-F			1	3	4.5	1.8		10.3			
1-C-T			1	3	4.5	1.8		10.3			TOP DOWN
2-A-T	0.84		0.5	3.5	3	3.6		10.6			
2-B-F			0.5	3.5	3.5	3.5		11			
2-C-T			0.5	3.5	3.5	3.5		11			
3-A-T	1.436		0.5	3.5	3.5	3.5		11			
3-B-F			0.5	3.5	3.5	3.5		11			
4-A-T	2.17		0.7	1.7	1.1	1.9		5.4	10	16	TRENCH
4-B-F			0.7	1.75	1	2		5.45			
4-C-T			0.7	1.75	1	2		5.45			TOP DOWN
5-A-T	2.5		1	7.5	5.5			14			
5-B-F			1	7.5	5.5			14			
6-A-T	2.98		0.6	1.1	1.3	2.1		5.1			
6-B-F			0.6	1	1.2	2		4.8			
6-C-T			0.6	1	1.2	2		4.8			TOP DOWN
6-D-T			0.6	1	1.2	2	**BIND**	4.8			
7-A-T	3.45		0.8	0.8	1.2	2	2	6.8			
7-B-F			0.8	0.8	1.2	2	2	6.8			
7-C-T		**FC4**	0.8	0.8	1.2	2	2	6.8			ALLIGATOR
8-A-T	3.98	0.5	1.3	1.21	1.4	1.9	2	8.31			
8-B-F		0.5	1.3	1.21	1.4	1.9	2	8.31			
8-C-T		0.5	1.3	1.21	1.4	1.9	2	8.31			
9-A-T	4.214	0.75	1.4	1.2	1.3	1.3	2.3	8.25			
9-B-F		0.75	1.4	1.2	1.3	1.3	2.3	8.25			

Table C19. Core Data on Section 15003000 at I-175 in Pinellas County.

Core No.	Mile Post	Core Length (in)	Base	Comments
			ECONC	
1-A-T	0.7	10		
1-B-F		10		
1-C-F		10		
2-A-T	0.805	9.6		20' Joint
2-B-F		9.6	7	1.25" Dowel
2-C-F		9.6		6' PCC shoulder
3-A-T	0.935	9		12' Lane width
3-B-F		9		
3-C-F		9		

Core No.	Mile Post	Core Length (in)	Base	Comments
			ECONC	
1-A-T	1.123	9		
1-B-F		9		
1-C-F		9		
2-A-T	0.889	9.8		20' Joint
2-B-F		9.8	7	1.25" Dowel
2-C-F		9.8		6' PCC shoulder
3-A-T	0.817	9.5		12' Lane width
3-B-F		9.5		
3-C-F		9.5		

Table C20. Core Data on Section 15190000 at I-275 in Pinellas County.

Core No.	Mile Post	Core Length (in)	Base ECONC	Comments
1-A-T	2.5	9		
1-B-F		9	7	
1-C-F		9		
2-A-T	2.92	9		20' Joint
2-B-F		9		1.25" Dowel
2-C-F		9		8.3' PCC shoulder
3-A-T	3.5	8.8		12' Lane width
3-B-F		8.8		
3-C-F		8.8		
4-A-T	4.1	9.5		Frequent
4-B-F		9.5	7	transverse cracks
4-C-F		9.5	broken	
Core No.	**Mile Post**	**Core Length (in)**	**Base ECONC**	**Comments**
1-A-T	4.285	9.2		
1-B-F		9.2		
1-C-F		9.2		
2-A-T	3.5	9		20' Joint
2-B-F		9	7.5	1.25" Dowel
2-C-F		9		8.3' PCC shoulder
3-A-T	3.1	9.3		12' Lane width
3-B-F		9.3		
3-C-F		9.3		
4-A-T	2.45	9.4		
4-B-F		9.4	7	
4-C-F		9.4		
5-A-T	2.4	9.2		
5-B-F		9.2		
5-C-F		9.2		

Table C21. Core Data on Section 86190000 at SR 823 in Broward County.

Core No.	Mile Post	FC2	S1	S1	Core Length (in)	Base		Comments
						LR	ST	
1-A-T	2.401	0.7	1.3	2.1	4.1			
1-B-F		0.7	1.3	2.1	4.1			
2-A-T	2.686	0.7	1.4	2.1	4.2			
2-B-F		0.7	1.4	2.1	4.2			
3-A-T	2.837	0.5	1.3	2	3.8			
3-B-F		0.4	1.2	2	3.6			TOP DOWN
3-C-T		0.5	1.3	2	3.8			
3-D-T		0.5	1.3	2	3.8			
4-A-T	3.15	0.8	3.2		4	8	12	Trench
4-B-F		0.7	3.2		3.9			6' GWT
4-C-T		0.7	3.2		3.9			TOP DOWN
5-A-T	3.2	0.6	1.4	2	4			
5-B-F		0.5	1.5	2.1	4.1			
6-A-T	3.3	0.5	1.5	2	4			
6-B-F		0.5	1.5	2	4			
6-C-T		0.4	1.6	2.1	4.1			

Table C22. Core Data on Section 93310000 at SR 710 in Palm Beach County.

Core No.	Mile Post	FC2	S1	S2	Core Length (in)	Base LR	Base ST	Comments
1-A-T	17.52	0.6	1.6	0.8	3			7.8' GWT
1-B-F		0.7	1.5	1.6	3.8	14	12	TRENCH
1-C-T		0.7	1.5	0.75	2.95			
2-A-T	17.2	0.8	1	1.1	2.9			
2-B-F		0.8	1	1.1	2.9			
2-C-T		0.8	1	1.1	2.9			
3-A-T	16.444	0.5	1.1	1.2	2.8			
3-B-F		0.5	1.1	1.2	2.8			
3-C-T		0.5	1.1	1.2	2.8			
4-A-T	15.195	0.5	1	0.5	2			
4-B-F		0.3	1.1	0.55	1.95			
4-C-T		0.5	1	0.5	2			

Table C23. Core Data on Section 93100000 at SR 25/US 27 in Palm Beach County.

Core No.	Mile Post	FC2	S1	T1	ST	Core Length (in)	Base		Comments
							LR	ST	
1-A-T	11.904	0.7	1	1.5	0.9	4.1	16	12	Trench
1-B-F		0.7	1.1	1.5	1	4.3			9' GWT
1-C-T		0.7	1	1.5	0.9	4.1			Top down
2-A-T	12.157	0.8	1.1	1.4	1.1	4.4			
2-B-F		0.7	1.3	1.4	0.9	4.3			
2-C-T		0.8	1.1	1.4	1.1	4.4			
3-A-T	12.305	0.6	0.9	1.3	0.8	3.6	16	12	
3-B-F		0.6	0.9	1.3	0.8	3.6			
3-C-T		0.6	0.9	1.2	0.8	3.5			
3-D-T		0.6	0.9	1.2	0.8	3.5			Top down
4-A-T	12.4	0.7	1.2	1.8	0.8	4.5			
4-B-F		0.6	1.2	1.6	0.9	4.3			
4-C-T		0.7	1.2	1.8	0.8	4.5			Top down
5-A-T	12.547	0.8	0.9	1.3	0.9	3.9			
5-B-F		0.8	0.9	1.3	0.9	3.9			
5-C-T		0.8	0.9	1.3	0.9	3.9			

Table C24. Core Data on Section 87061000 at SR 886 in Dade County.

Core No.	Mile Post	Core Length (in)	Base LR	Comments
1-A-T	0.195	12.5		
1-B-F		12.5	15	
1-C-F		12.5		12' lane
2-A-T	0.126	12		18' Joint
2-B-F		12	15	We could not find dowel
2-C-F		12		No shoulder

Table C25. Core Data on Section 87270000 at SR 9A/I 95 in Dade County.

Core No.	Mile Post	Core Length (in)	Base LR	Comments
1-A-T	0.229	9		
1-B-F		8.8		5 ~ 8’ GWT
1-C-F		8.9		12' lane
2-A-T	0.46	9.1		19' Joint
2-B-F		9.1	16	We could not find dowel
2-C-F		9.1		Conc. shoulder (9')

Table C26. Core Data on Section 87030000 at SR 5/US 1 in Dade County.

Core No.	Mile Post	Core	Base	Comments
		Length (in)	LR	
1-A-T	8.96	8		
1-B-F		8.2	10	
1-C-F		8.3		
2-A-T	9.13	7.9		15.4' Joint
2-B-F		8.5		We could not find dowel
2-C-F		8.2		No shoulder
3-A-T	9.5	10.5		12' Lane width
3-B-F		8.2	10	Transverse cracks
3-C-F		8.8		Patched slabs

Core No.	Mile Post	Core	Base	Comments
		Length (in)	LR	
1-A-T	9.888	7.2		
1-B-F		7		
1-C-F		7.2		
2-A-T	9.517	8		15.4' Joint
2-B-F		8.2	8.5	We could not find dowel
2-C-F		8.2		No shoulder
3-A-T	9.288	9.3		12' Lane width
3-B-F		8.5		Transverse cracks
3-C-F		8.5		Patched slabs
4-A-T	8.962	8.5		
4-B-F		8	11	
4-C-F		8		

Table C27. Core Data on Section 87060000 at SR A1A in Dade County.

Core No.	Mile Post	FC2	S1	T	Core Length (in)	Base		Comments
						LR	STAB	
1-A-T	2.595	0.6	2.3	5.4	8.3			
1-B-F		0.6	2.4	5.5	8.5			
2-A-T	2.266	0.6	2.9	5.1	8.6			
2-B-F		0.6	2.9	5.1	8.6			
2-C-T		0.7	2.8	5.2	8.7			
3-A-T	2.149	0.5	2.7	5.1	8.3	14	16	Trench
3-B-F		0.5	2.7	5.1	8.3			GWT = 5.3'
3-C-T		0.5	2.7	5.1	8.3			
4-A-T	2.009	0.6	3.3	5.1	9			
4-B-F		0.6	3.3	5.1	9			
5-A-T	1.792	0.5	3	5.4	8.9			We trenched all the way
5-B-F		0.4	2.9	5.5	8.8			down up to 3' and still it
5-C-T		0.4	2.9	5.5	8.8			was LR.
5-D-T		0.3	3	5.5	8.8			Top down crack
6-A-T	1.629	0.5	3.3	4.8	8.6			
6-B-F		0.5	3.2	4.7	8.4			
7-A-T	1.493	0.4	2.9	4.5	7.8			
7-B-F		0.5	3	4.7	8.2			
7-C-T		0.5	3.1	4.8	8.4			
8-A-T	1.429	0.5	3.6	5.4	9.5			
8-B-F		0.5	3.6	5.4	9.5			
8-C-T		0.6	3.5	5.3	9.4			
9-A-T	1.289	0.5	3.1	5.4	9			
9-B-F		0.4	3	5.3	8.7			
9-C-T		0.4	3	5.3	8.7			
10-A-T	0.996	0.5	2.9	5.3	8.7			
10-B-F		0.5	2.9	5.3	8.7			

Table C28. Core Data on Section 90060000 at US 1 in Monroe County.

Core No.	Mile Post	FC2	S1	T1	ST	Core Length (in)	Base		Comments
							LR	Embank.	
1-A-T	13.3	0.4	1.8	0.9	0.4	3.5			
1-B-F		0.5	1.7	1	0.5	3.7			
2-A-T	13.534	0.6	1.3	1.1		3			
2-B-F		0.7	1.3	0.9	0.2	3.1			
3-A-T	13.76	0.5	1.5	0.5	1.5	4			
3-B-F		0.5	1.5	0.5	1.5	4			
3-C-T		0.5	1.5	0.5	1.5	4			
4-A-T	14.08	0.4	1	1.5	1	3.9			
4-B-F		0.4	1	1.5	1	3.9			
4-C-T		0.4	1	1.5	1	3.9			
5-A-T	14.155	0.4	1	2		3.4			
5-B-F		0.4	1	2		3.4			
5-C-T		0.5	1	2.5		4			
6-A-T	14.328	0.3	3.5	1		4.8			
6-B-F		0.3	3.5	1		4.8	24		Alligator crack
6-C-T		0.3	3.5	1		4.8			
6-D-T		0.3	3.5	1		4.8			
7-A-T	14.73	0.3	1	1.8		3.1			
7-B-F		0.3	1	1.8		3.1			
8-A-T	15.067	0.2	1	2.2	0.2	3.6			
8-B-F		0.2	1	2.2	0.2	3.6			
9-A-T	15.337	0.4	1.1	1	1.4	3.9			
9-B-F		0.4	1.1	1	1.4	3.9			
9-C-T		0.4	1.1	1	1.4	3.9			

Table C28. Core Data on Section 90060000 at US 1 in Monroe County (continued).

Core No.	Mile Post	FC2	S1	T1	ST	Core Length (in)	Base		Comments
							LR	Embank.	
10-A-T	15.528	0.5	0.9	1.8	0.9	4.1			Trench
10-B-F		0.5	0.9	1.8	0.9	4.1	24		Top Down
10-C-T		0.5	0.9	1.8	0.9	4.1			
11-A-T	15.917	0.4	1.1	2.5		4			GWT = 6.1'
11-B-F		0.4	1.1	2.5		4			
12-A-T	16.2	0.4	1.1	2.5		4			
12-B-F		0.4	1.1	2.5		4			
12-C-T		0.4	1.1	2.5		4			