RESEARCH & DEVELOPMENT

Backcalculation of Dynamic Modulus from Falling Weight Deflectometer Data

Y. Richard Kim, Ph.D., P.E., F. ASCE
Zhe Zeng
Kangjin Lee, Ph.D.
Dept. of Civil, Construction, & Environmental Engineering
North Carolina State University

NCDOT Project 2017-03

FHWA/NC/2017-03

January 2021

Backcalculation of Dynamic Modulus from Falling Weight Deflectometer Data

FINAL REPORT
Research Project No. HWY-2017-03

Submitted to:

North Carolina Department of Transportation
Office of Research

Submitted by:

Y. Richard Kim, Ph.D., P.E., F.ASCE
Jimmy D. Clark Distinguished University Professor
Campus Box 7908
Department of Civil, Construction, & Environmental Engineering
North Carolina State University
Raleigh, NC 27695-7908
Tel: 919-515-7758, Fax: 919-515-7908
E-mail: kim@ncsu.edu

Zhe Zeng
E-mail: zzeng5@ncsu.edu

Kangjin Lee, Ph.D.
E-mail: klee18@ncsu.edu

Department of Civil, Construction, & Environmental Engineering
North Carolina State University
Raleigh, NC

January 2021

North Carolina Department of Transportation
Research and Development

Technical Report Documentation Page

1. Report No. FHWA/NC/2017-03	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Backcalculation of Dynamic Modulus from Falling Weight Deflectometer Data		5. Report Date January 12, 2021
		6. Performing Organization Code
7. Author(s) Y. Richard Kim, Zhe Zeng, and Kangjin Lee		8. Performing Organization Report No.
9. Performing Organization Name and Address Campus Box 7908, Dept. of Civil, Construction, & Environmental Engrg. NCSU, Raleigh, NC 27695-7908		10. Work Unit No. (TRAIS)
		11. Contract or Grant No.
12. Sponsoring Agency Name and Address NC Department of Transportation Research and Analysis Group 1 South Wilmington Street Raleigh, NC 27601		13. Type of Report and Period Covered Final Report August 2016 – July 2020
		14. Sponsoring Agency Code RP2017-03
15. Supplementary Notes		

16. Abstract
The Mechanistic-Empirical Pavement Design Guide (Pavement ME Guide) and its related software, AASHTOWare Pavement ME Design, have been widely used in the asphalt pavement community. The Guide proposes three analysis levels for highway agencies to use for rehabilitation design. However, as these methods were developed in the early 2000s, certain aspects were not considered sufficiently in terms of their accuracy and efficiency. Over time, researchers have offered various suggestions to modify the theories and guidelines in the Pavement ME Guide, and from a practical point of view, some parts of the Guide still need to be improved, such as how best to perform backcalculations and characterize multilayered existing asphalt concrete (AC) pavement. To this end, this study focused specifically on improving methods to characterize damaged existing AC layers.

Based on North Carolina (NC) 96 highway data, North Carolina State University researchers evaluated and compared the accuracy of the three levels in the Pavement ME Guide. Through tests and analysis of field cores, this report provides specific guidelines for the North Carolina Department of Transportation to select representative layers in multilayered AC pavements. The research team compared predicted damaged dynamic modulus mastercurves to laboratory-measured values. The results show that Level 1 should always be the first choice for highway agencies. Nonetheless, if Level 2 or 3 needs to be applied, then the transfer function that relates the damage factor to the percentage of bottom-up cracking needs to be calibrated first to ensure that the results are consistent with those of Level 1. Although sometimes the *in situ* modulus of the granular layer is measured during the backcalculation process, the modulus value is not recommended to be fixed in the program in order to give the algorithm enough freedom to satisfy the root mean square error criterion. Regarding the selection of the representative layer, the total core, not the thickest layer, should be used for laboratory evaluation. The NCDOT can thus save time and effort without sacrificing the accuracy of rehabilitated pavement performance predictions.

17. Key Words Asphalt concrete, rehabilitation design, dynamic modulus, Witczak's predictive equation, falling weight deflectometer (FWD), backcalculation	18. Distribution Statement		
19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages 66	22. Price

Form DOT F 1700.7 (8-72) **Reproduction of completed page authorized**

DISCLAIMER

The contents of this report reflect the views of the authors and are not necessarily the views of North Carolina State University. The authors are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the North Carolina Department of Transportation at the time of publication. This report does not constitute a standard, specification, or regulation.

ACKNOWLEDGEMENTS

This research was sponsored by the North Carolina Department of Transportation. The Steering and Implementation Committee was comprised of Clark Morrison, Ph.D., P.E. (Chair), Josh Holland, P.E., Lamar Sylvester, P.E., Scott Capps, P.E., Christopher A. Peoples, P.E., James B. Phillips, P.E., and Mustan Kadibhai, P.E. (PM). These advisors have given invaluable direction and support to the research team throughout the project. In addition, Prof. Murthy Guddati led the theoretical development of the forward modeling and back-calculation algorithms in this project, while Amin Sahafipourfard enhanced the formulations and implemented the algorithms.

EXECUTIVE SUMMARY

Research Objectives

The overall objectives of this research are to:

1. Evaluate the reliability of the three analysis levels that are recommended in the Mechanistic-Empirical Pavement Design Guide (Pavement ME Guide) to estimate the dynamic modulus values of asphalt layers in existing pavements.

2. Recommend an effective way to determine the *in situ* dynamic modulus mastercurve by comparing the predicted dynamic modulus values used in Witczak's predictive equation against the dynamic modulus values measured from laboratory tests of field cores.

3. Investigate the effect of the inputs used in the backcalculation process on the backcalculated modulus values.

4. Develop recommendations for pertinent falling weight deflectometer (FWD) test protocols and reliable methods to determine the dynamic modulus values of existing asphalt layers for the North Carolina Department of Transportation (NCDOT)'s routine use of the Pavement ME Guide's rehabilitation design.

Research Methodology

To achieve the research goals, the North Carolina State University (NCSU) research team applied various means to investigate efficient ways to characterize the dynamic modulus of existing asphalt concrete (AC) pavements. North Carolina (NC) 96 was selected for this study because it is a full-depth pavement that consists of multiple AC layers that have experienced damage since construction, thus making this pavement a good candidate for the research purposes. Seven pavement sections, each 1000 feet long, were chosen for the field tests, i.e., falling weight deflectometer (FWD) and dynamic cone penetration (DCP) tests. The NCSU team recorded the air and pavement surface temperatures during testing and conducted a condition survey to obtain the required information for Pavement ME Design.

The NCSU team extracted field cores from the same locations the field tests were conducted and used various laboratory tests to determine the dynamic modulus mastercurves for these field cores. The research team conducted two types of laboratory tests, i.e., tests required by Witczak's predictive equation and dynamic modulus tests. The former provides an indirect way to determine the dynamic modulus value and the latter provides direct measurements. Also, because field cores do not necessarily have a standard geometry or provide sufficient materials for testing, the NCSU research team devised different solutions to ensure that the tests of the field cores were as close as possible to typical laboratory tests without sacrificing measurement accuracy.

In addition to testing, the research team evaluated the three levels of analysis that are recommended in the Pavement ME Guide using AASHTOWare Pavement ME Design software. Level 1 analysis, which is the most accurate approach, not only involves the backcalculation of FWD-tested deflections, but also requires measured data to satisfy Witczak's predictive equation. The research team also investigated ways to backcalculate the elastic modulus using the Pavement ME backcalculation tool by testing three sets of subgrade inputs based on experimental measurements, empirical predictions, and direct backcalculation, respectively. The research team then determined the best way to use the Pavement ME backcalculation tool by comparing the root mean square errors (RMSEs) of the inputs. In addition, due to the limited amount of material that each field core could provide, the team carefully considered the order that the tests were conducted to obtain maximum operational efficiency and data accuracy.

Instead of the FWD and backcalculation needed for Level 1 analysis, Level 2 analysis requires a field condition survey of alligator cracking. Using the precalibrated transfer function given in the Pavement ME Guide, the research team determined damage factors for the NC 96 selected test sections. Then, the team developed the NC 96 *in situ* damaged dynamic modulus mastercurve in combination with the dynamic modulus mastercurve predicted by Witczak's equation. Note that, in this report, 'undamaged' and 'damaged' mastercurves refer to the dynamic modulus mastercurves of undamaged and damaged AC pavements, respectively. For Level 3 analysis, instead of using a specific number to quantify the NC 96 field conditions, the research team used ratings as inputs into AASHTOWare, coupled with typical Witczak's predictive equation inputs for North Carolina. In this way, the research team obtained the field dynamic modulus mastercurve. Based on comparisons of the three analysis levels in terms of their accuracy and practicability, the NCSU research team is able to provide final recommendations to the NCDOT to characterize the dynamic modulus of an existing AC pavement.

Another important problem to be solved in this project was determining the best way to select the representative layer of an existing AC pavement, especially when the pavement consists of multiple layers. The NC 96 pavement has five layers (or even more in reality) and its thickest layer is the top layer (3 in.). The research team categorized the field core testing into three cases: top (top layer only), bottom (remaining layers after excluding the top layer), and the total core. Considering each of these cases separately as the representative layer, the research team tested the characterization method given in the Pavement ME Guide and predicted the rehabilitated pavement performance using AASHTOWare. By comparing the differences in predicted performance and considering the complexity of the test procedures, the research team is able to provide suggestions to the NCDOT for selecting the representative layer of an existing AC pavement.

Finally, the NCSU research team tried one more step to backcalculate the *in situ* dynamic modulus mastercurve directly in an attempt to remove the burden of conducting the tests required by the Pavement ME Guide. To achieve this goal, the dynamic modulus mastercurve had to be obtained solely from FWD-measured deflections. Therefore, the team evaluated the conditions that are required to determine the mastercurve and compared the mastercurve against the information obtained by the FWD.

Conclusions

The main conclusions drawn from this research are:

- The undamaged mastercurve that is predicted using Witczak's predictive equation has a different shape and magnitude than the mastercurve that is constructed based on a field core. Because the shape of the curve is linked to viscoelasticity and the damaged mastercurve is merely the vertical shift of the undamaged mastercurve, the current protocol in Pavement ME based on Witczak's predictive equation could result in erroneous damaged mastercurves.

- The results from Level 1, 2, and 3 analyses differ significantly in terms of damage factor estimations and cracking performance predictions. Level 1 is recommended as the first choice for agencies to use in practice. However, if Level 2 or 3 needs to be applied, then the transfer function that relates the damage factor to the percentage of bottom-up cracking needs to be calibrated first to ensure that the results are consistent with those of Level 1.

- When the tests specified by Levels 1 and 2 need to be conducted for a multilayered AC pavement, then the total core should be used for laboratory evaluation, not the thickest layer. Dividing the existing pavement into multiple layers is possible only for Level 1, but even in that case is not recommended because this approach requires multilayer backcalculations and considerable time and resources to characterize the individual layer materials.

- The dynamic modulus mastercurve cannot be backcalculated solely from FWD data due to the limited frequency range that the FWD can capture. Therefore, to determine the damaged mastercurve, some other means, such as FWD testing at multiple times of a day or multiple seasons or laboratory measurements, need to be employed to obtain more information outside the FWD frequency range. Because the Pavement ME Guide already provides a way for agencies to determine a damaged mastercurve and its software is convenient for the NCDOT to access, instead of developing a completely new protocol, the NCSU research team suggests that the NCDOT use the Pavement ME method that has been modified in this project based on NC 96 data.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

1. INTRODUCTION

The dynamic modulus ($|E^*|$) is an important material property that describes the time- and temperature-dependent behavior of viscoelastic material, such as asphalt concrete (AC). Dynamic modulus values measured at multiple loading frequencies and temperatures can be used to develop a dynamic modulus mastercurve, which allows for the prediction of AC's responses to any loading history and temperature within the linear viscoelastic range. Therefore, the dynamic modulus is widely accepted by the asphalt pavement industry as a fundamental stiffness property of AC. For example, the Mechanistic-Empirical Pavement Design Guide (referred to as Pavement ME Guide) and its related software, AASHTOWare Pavement ME Design, use the dynamic modulus as the stiffness property of asphalt mixtures in the design of new pavements as well as for rehabilitation design. In either new pavement design or rehabilitation design, the user is required to input the dynamic modulus values of the existing asphalt layers using one of three analysis approaches (Level 1 to Level 3). Of these three approaches, Level 1 is the most accurate approach, but it requires significant work and testing. Levels 2 and 3 are simplifications of Level 1 and are easier to implement but lack the same degree of accuracy.

When an overlay needs to be designed, the first step in all three analysis levels is to establish the 'undamaged' modulus mastercurve using Witczak's predictive equation. Levels 1 and 2 require laboratory tests of field cores to obtain the equation's inputs whereas Level 3 uses typical values. If Level 1 analysis is chosen, a falling weight deflectometer (FWD) is employed to detect deflections in the existing pavement. Then, the *in situ* elastic modulus value is backcalculated to represent the field conditions. By assuming the FWD loading frequency (typically from 5 Hz to 30 Hz) (Ayyala et al. 2018), the damage factor (d_{AC}) is calculated at the assumed frequency. Next, this damage factor is used to shift the entire undamaged mastercurve downwards to determine the so-called 'damaged' mastercurve, which reflects the material's stiffness under field damage. If Level 2 is chosen, instead of using a FWD, resilient modulus testing or a cracking condition survey is used to calculate the damage factor. Level 3 is a simplification of Level 1 and Level 2 where, instead of conducting the tests and analyses, the user merely rates the pavement's alligator cracking as excellent, good, fair, poor, or very poor. These ratings are related to the extent of damage in the existing pavement, which eventually determines the position of the damaged mastercurve. Note that, in this report, 'undamaged' and 'damaged' mastercurves refer to the dynamic modulus mastercurves of undamaged and damaged AC pavements, respectively.

Since the development of the Pavement ME Guide in the early 2000s, technical limitations have led researchers not to recommend taking direct dynamic modulus measurements of field cores in the laboratory (ARA 2004). Instead, researchers use Witczak's predictive equation to predict the dynamic modulus mastercurve using a series of methods, such as gradation analysis and binder viscosity measurements. For new pavement design, the determination of the dynamic modulus is not difficult or time-consuming because the amount of available material is sufficient for specimen fabrication and the mixture has not yet undergone field aging. However, these advantages are not the case for rehabilitation design. Not only are tests of field cores (that provide only limited materials) required, but also the *in situ* materials have been negatively affected by aging and various types of distress while in service. These factors hinder the determination of accurate field modulus values. Currently, well-established AASHTO specifications (i.e., AASHTO TP 79-15/PP 61-13, T 342-11, and R 62) allow the dynamic modulus values of asphalt mixtures to be determined accurately by testing laboratory-fabricated specimens using a Superpave gyratory compactor. However, these test specifications require 100-mm diameter, 150-mm tall specimens, so they are not necessarily applicable for field cores. In order to address this shortcoming, Kim et al. developed viscoelastic solutions that allow the determination of dynamic modulus values from indirect tensile tests (Kim et al. 2004). Pape et al. developed small geometry test specimens that also solved problems associated with dynamic modulus tests of field cores (Pape et al. 2018). Other researchers also have measured field cores directly using cylindrical specimens (Loulizi et al. 2007, Habbouche et al. 2018). These test protocols have provided new technical means to determine field dynamic modulus values and, compared to Witczak's predictive equation, these protocols may be even easier for highway agencies to implement in practice.

In addition, because pavement construction projects occur over different years, the existing pavement always consists of multiple AC layers. Such multilayer pavement structures increase the difficulty of using the Pavement ME method because Witczak's predictive equation is based on single-layer material. When this equation was developed, gradation parameters, binder viscosity, and other properties were all based on a single mix design process. Hence, whether or not the prediction accuracy would decrease when applying the equation to multilayered existing asphalt pavement was unknown. In addition, the backcalculation of FWD-measured data is sometimes tricky, especially when multiple AC layers are involved. In this scenario, whether or not the existing AC layers should be regarded as a single layer becomes a question, because multilayer backcalculation may present technical issues and compromise accuracy. Also, when backcalculation is conducted, whether the base and/or subgrade modulus values should be determined via backcalculation or via field tests needs to be investigated. The direct measurement of the base/subgrade modulus may be more reliable, but setting this value in the backcalculation algorithm can cause the program not to converge. Therefore, the best way to use the backcalculation program correctly and efficiently should be evaluated as well.

Based on the background information, the overall objectives of this research are to:

1. Evaluate the reliability of the three analysis methods (Levels 1, 2, and 3) that are recommended in the Pavement ME Guide to estimate the dynamic modulus values of asphalt layers in existing pavements.

2. Recommend an effective way to determine the *in situ* dynamic modulus mastercurve by comparing the dynamic modulus values predicted by Witczak's predictive equation to the dynamic modulus values measured from laboratory tests of field cores.

3. Investigate the effect of inputs on the backcalculated modulus values in the backcalculation process.

4. Develop recommendations for pertinent FWD testing protocols and reliable methods to determine the dynamic modulus values of existing asphalt layers for the North Carolina Department of Transportation's (NCDOT's) routine use of Pavement ME rehabilitation design.

2. PAVEMENT ME REHABILITATION DESIGN METHOD

The Pavement ME Design Guide indicates that the following information can be obtained from FWD backcalculation:

- Time- and temperature-dependent properties of hot mix asphalt layers
- Resilient modulus values for unbound base/sub-base and subgrade materials
- Elastic modulus values of bedrock, if present

The determination of the asphalt layer dynamic modulus for rehabilitation design follows the same general concepts as for new or reconstruction design, with the exceptions noted in the following subsections that describe the procedural steps for each of three input levels recommended in the Pavement ME Design Guide. Note that two types of rehabilitation design methods are described in the National Cooperative Highway Research Program (NCHRP) 1-37A report with regard to damage factor calculation and implementation of the different analysis levels (ARA 2004). The first method is documented in Chapter 2 and the second method is documented in Chapter 6. For the first method, the applications of the three levels are presented in the following subsections.

2.1. Input Level 1: Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (First Method)

1. Conduct nondestructive testing in the outer wheel path using the FWD for the project to be rehabilitated and compute the mean backcalculated asphalt bound modulus value, E_i, for the project. Be sure to include cracked as well as uncracked areas. The corresponding asphalt pavement temperature at the time of testing also should be recorded. Perform coring to establish the layer thickness throughout the project pavement. Layer thickness also can be determined using ground penetrating radar (GPR). Backcalculate the asphalt bound modulus value by combining layers with similar properties at each FWD test point in the project (with known pavement temperature).

2. Perform field coring and establish the mix volumetric parameters (air void content, asphalt volume, gradation, and asphalt viscosity parameters to define *A-VTS* values that represent two coefficients of the linear function between viscosity and temperature) that are required for computing the dynamic modulus value using Witczak's predictive equation, i.e., Equation (1). Equation (1) is based on the classical sigmoidal function form where each parameter can be expressed as a function of the asphalt mixture properties.

$$\log\left(|E^*|\right) = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log t_r)}} \tag{1}$$

where

$|E^*|$ = dynamic modulus,

t_r = time of loading at the reference temperature,

α, δ = fitting parameters; for a given set of data, δ represents the minimum value of the dynamic modulus and $\alpha + \delta$ represents the maximum value of the dynamic modulus, and

β, γ = parameters that describe the shape of the sigmoidal function.

Detailed descriptions of the parameters in Equation (1) are given below:

$$\delta = 3.750063 + 0.02932\rho_{200} - 0.001767(\rho_{200})^2 - 0.002841\rho_4 - 0.058097V_a - 0.802208\left[\frac{Vb_{eff}}{Vb_{eff} + Va}\right]$$

$$\alpha = 3.871977 - 0.0021\rho_4 + 0.003958\rho_{38} - 0.000017\rho_{38}{}^2 + 0.005470\rho_{34}$$

$$\beta = -0.603313 - 0.393532\log\left(\eta_{T_r}\right)$$

$$\log(t_r) = \log(t) - c\left(\log(\eta) - \log\left(\eta_{T_r}\right)\right)$$

$$\gamma = 0.313351$$

$$c = 1.255882$$

where:

ρ_{200} = percentage passing the No. 200 sieve,

ρ_{34} = cumulative percentage retained on the 3/4 sieve,

ρ_{38} = cumulative percentage retained on the 3/8 sieve,

ρ_4 = cumulative percentage retained on the No. 4 sieve,

Va = air void content,

Vb_{eff} = effective binder content in terms of volume,

t = time, s,

η = viscosity, 10^6 Poise, and

η_{Tr} = viscosity at reference temperature, 10^6 Poise.

3. Develop an undamaged dynamic modulus mastercurve from the data collected in Step 2 using Equation (1).

4. Estimate AC damage, d_{AC}, expressed as Equation (2).

$$d_{\text{AC}} = 1 - \frac{E_i}{|E^*|} \tag{2}$$

where

E_i = the backcalculated modulus value at a given reference temperature recorded in the field, and

$|E^*|$ = the predicted modulus value at the same temperature as the temperature recorded in the field.

5. Determine α' as shown in Equation (3).

$$\alpha' = \left(1 - d_j\right)\alpha \tag{3}$$

6. Determine the field-damaged mastercurve using α' instead of α used in Equation (1).

The following tests and procedures are required to determine the dynamic modulus mastercurve based on the asphalt layers of the existing pavement using the Level 1 approach:

- FWD testing
- Coring
- Binder extraction and recovery
- Viscosity tests of extracted binder
- Mixture bulk specific gravity (G_{mb}) and maximum specific gravity (G_{mm}) measurements of field cores
- Sieve analysis
- Specific gravity tests of aggregate

2.2. Input Level 2: Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (First Method)

1. Perform field coring and establish the mix volumetric parameters (air void content, asphalt volume, gradation) and asphalt viscosity parameters to define the *A-VTS* values.

2. Develop an undamaged dynamic modulus mastercurve using Equation (1).

3. Conduct indirect resilient modulus (M_{ri}) laboratory tests of field cores using the revised protocol developed at the University of Maryland for NCHRP 1-28A (Witczak 2004). Use two to three temperatures below 70°F.

4. Estimate damage, d_{AC}, at similar temperatures and loading frequency conditions using Equation (4).

$$d_{\mathrm{AC}} = 1 - \frac{M_{ri}}{|E^*|} \tag{4}$$

where

M_{ri} = laboratory-estimated resilient modulus value at a given reference temperature, and

$|E^*|$ = dynamic modulus value predicted at the same temperature as the given reference temperature.

5. Determine α' using Equation (3).

6. Determine the field-damaged mastercurve using α' instead of α that is used in Equation (1).

The following tests and procedures are required to determine the dynamic modulus mastercurve from the asphalt layers of an existing pavement using the Level 2 approach:

- Coring
- Indirect tensile resilient modulus testing
- Binder extraction and recovery
- Viscosity tests of extracted binder
- G_{mb} and G_{mm} measurements of field cores
- Sieve analysis
- Specific gravity tests of aggregate

2.3. Input Level 3: Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (First Method)

1. Use typical estimates of the mix parameters (mix volumetrics, gradation, and binder type) to develop an undamaged mastercurve with aging for the *in situ* pavement layers using Equation (5).

$$\log \log \eta = A + VTS \log T_R \tag{5}$$

where

η	=	viscosity, cP,
T_R	=	temperature, Rankine,
A	=	regression intercept, and
VTS	=	regression slope of viscosity temperature susceptibility.

2. Use the results of the distress/condition survey to obtain estimates of the pavement rating: excellent, good, fair, poor, very poor.
3. Use the pavement rating to estimate the (asphalt bound) pavement layer damage factor, d_{AC}.
4. Determine α' using Equation (3).
5. Develop the field-damaged mastercurve using α' rather than α in Equation (1).

Only the distress survey data are needed to determine the dynamic modulus mastercurve from the asphalt layer of the existing pavement using the Level 3 approach.

For the second method in the Pavement ME Guide, the applications of the three levels are presented in the following subsections.

2.4. Input Level 1 – Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (Second Method)

This Level 1 approach is the same as the method presented in Section 2.1 except that the damage parameter, d_{AC}, is estimated by Equation (6) instead of Equation (2). Then, the determined d_{AC} is used in Equation (6) to obtain the entire damaged mastercurve from the undamaged mastercurve.

$$|E^*|_{\mathrm{dam}} = 10^{\delta} + \frac{|E^*| - 10^{\delta}}{1 + e^{-0.3 + 5\log(d_{AC})}} \tag{6}$$

where

$|E^*|_{dam}$ = damaged modulus.

2.5. Input Level 2 – Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (Second Method)

1. Perform field coring and establish the mix volumetric parameters (air void content, asphalt volume, gradation) and asphalt viscosity parameters to define the *A-VTS* values.

2. Develop an undamaged dynamic modulus mastercurve using Equation (1).

3. Use the results of the distress/condition survey to obtain the amount of alligator cracking that initiated at the bottom of the asphalt layers and was measured at the pavement surface, as expressed in Equation (7).

$$FC_{\text{bottom}} = \left(\frac{6000}{1 + e^{C_1 C_1' + C_2 C_2' \log_{10}(d_{AC} 100)}} \right) \frac{1}{60} \tag{7}$$

where

FC_{Bottom} = area of alligator cracking that initiates at the bottom of hot mix asphalt layers, % of total lane area,

C_1 = 1.31,

C_2 = 5 for asphalt layer thickness (h_{ac}) less than 5 in.; 3.9666 for h_{ac} greater than 12 in.; otherwise, equal to $0.867+0.2583\times h_{ac}$,

C_1' = $-2C_2'$, and

C_2' = $-2.40874-39.748\ (1+h_{ac})\ -2.856$.

4. Obtain the entire damaged mastercurve from the undamaged mastercurve using Equation (6).

The following tests and procedures are required to determine the dynamic modulus mastercurve from the asphalt layers of existing pavement using the Level 2 approach:

- Coring
- Condition survey
- Binder extraction and recovery

- Viscosity tests of extracted binder
- G_{mb} and G_{mm} measurements of field cores
- Sieve analysis
- Specific gravity tests of aggregate

2.6. Input Level 3: Approach to Determine Field-Damaged Dynamic Modulus Mastercurve (Second Method)

The Level 3 approach is the same as that described in Section 2.3; Table 1 presents the criteria used to evaluate the pavement conditions (AASHTO 2015). After the condition rating is determined, an approach similar to that for Level 2 is used to determine the damage factor and damaged mastercurve.

Table 1. Flexible Pavement: Overall Estimate of Surface Cracking

Category	Structural Condition
Excellent	< 5% area cracked
Good	5%–15% area cracked
Fair	15%–35% area cracked
Poor	35%–50% area cracked
Very Poor	> 50% area cracked

2.7. Problems Associated with the Pavement ME Guide

Note that the current AASHTOWare Pavement ME Design software adopts the second method in its algorithm for asphalt pavement rehabilitation design. Therefore, in the latter part of this report, only the second method is used and evaluated. Based on the procedures outlined in Sections 2.1 through 2.6, the Pavement ME Guide method does not require direct dynamic modulus measurements from field cores. Instead, its approach follows three steps:

1. Depending on the analysis level selected, either use typical values or collect field cores and conduct laboratory tests to obtain the gradation, air void content, effective asphalt content, and binder viscosity of the existing AC layers. Next, input the measured properties into Witczak's predictive equation to obtain the undamaged modulus mastercurve.

2. If Level 1 is chosen, conduct FWD tests in the field and backcalculate the field-damaged modulus values at a single temperature and frequency using a linear elastic program (ARA 2004). By comparing the undamaged modulus and damaged modulus at the FWD test frequency, the damage factor can be estimated. If Level 2 or Level 3 is chosen, then the damage factor is evaluated using a transfer function as defined in Pavement ME guideline to match the amount of alligator cracking that initiated at the bottom of the pavement.

3. Using this damage factor to shift the undamaged mastercurve, the new curve is regarded as the damaged mastercurve that represents the field conditions.

Although this protocol is widely used by highway agencies and contractors, the possibility of significant errors is present for all three approaches. First, the accuracy of the dynamic modulus values obtained using Witczak's predictive equation (i.e., Equation (1)) varies from case to case. Applying a damage factor that is estimated from the predicted dynamic modulus value to the entire dynamic modulus mastercurve could amplify the effects of a dynamic modulus prediction error. Importantly, because Level 1 and Level 2 both require coring, dynamic modulus tests of the cores at different temperatures and frequencies can be conducted to obtain the dynamic modulus mastercurve. The amount of laboratory testing effort that this direct approach requires may be less than that required for the various tests needed to adopt the Level 1 approach, which include binder extraction and recovery testing, sieve analysis, specific gravity tests of the aggregate, and viscosity tests of the extracted binder. Even though the Level 2 approach does not require FWD testing, it requires a field condition survey in addition to the laboratory tests required in the Level 1 approach. Hence, the Level 2 approach, as it is described in the Pavement ME Guide, does not dramatically reduce the required effort compared to Level 1.

The reason the direct dynamic modulus testing approach is not recommended in the Pavement ME Guide is simple. At the time the Pavement ME Guide was being developed, no laboratory test method was available that allowed the dynamic modulus mastercurve to be determined from thin field cores. However, this situation has since changed. Kim et al. (2004) developed the viscoelastic solutions for the indirect tensile test and thus allowed the dynamic modulus testing of thin disks that can be obtained from field cores. Park and Kim (2004) and Park et al. (2014) used the 38 mm diameter cores obtained from field cores by horizontal coring to perform the dynamic modulus testing. A more refined method of fabricating the 38 mm diameter specimens was suggested by Pape et al. (2018) and verified against the 100 mm diameter specimens by Lee et al. (2017).

Some researchers found that the measured modulus mastercurve is very close to the undamaged mastercurve predicted by Witczak's equation (Loulizi et al. 2007). Other researchers showed that Witczak's equation results in huge errors compared with laboratory-measured modulus values (Habbouche et al. 2018). Because the reported accuracy varies, implementing the Pavement ME Guide approach for dynamic modulus testing relates closely to the field conditions and *in situ* material properties, which should be evaluated based on local resources.

With regard to implementing the different analysis levels, several studies have pointed out that the determined damaged mastercurves do not overlap when using the three levels in the Pavement ME Guide (Loulizi et al.2007, Harsini et al. 2013, Habbouche et al. 2018), indicating that analysis level selection may also result in significant variation. In an attempt to resolve this reported problem, Loulizi et al. suggested that Level 1 should be applied to all projects if the inputs are available (Loulizi et al. 2007). Harsini et al. recommended relating the pavement condition ratings to the modulus characterized in Level 1 in order to make the performance predictions consistent (Harsini et al. 2013). Habbouche et al. concluded that the use of Witczak's predictive equation (Level 1) leads to damage overestimation and proposed a hybrid method to improve accuracy; their method includes both Level 1 and Level 2 as well as laboratory dynamic modulus tests of field cores (Habbouche et al. 2018). Also, Ayyala et al. investigated three modified approaches to determine the damage factor using Long-Term Pavement Performance (LTPP) program data: (1) calibrate Equation (7) so that it results in the same damage factor calculated in Equation (6); (2) calibrate Equation (6) to match the damage factor in Equation (7); and (3) change the equation expression so that the damaged mastercurve is obtained through horizontal shifting instead of vertical shifting. Even though all these concepts make sense logically, the researchers did not find a definitive relationship between the modulus-based damage factor and the *in situ* cracking condition survey results (Ayyala et al. 2017 and 2018).

Furthermore, actual pavements always consist of multiple AC layers due to the construction of either different lifts or mixes, which increases the uncertainty and complexity surrounding the implementation of Witczak's predictive equation because this equation is based on single mix properties. As of yet, no clear solution is available for multilayer pavement systems. The only guidance from Pavement ME is to conduct tests of a "representative lift" or combine different layers to determine the inputs of Witczak's equation (AASHTO Pavement ME Design Task Force 2013), which may cause a large variation in the predicted modulus values. Some of the aforementioned researchers encountered situations where field cores contained multiple AC layers, but they did not provide details or guidance about representative lift selection and/or testing. Therefore, this gap in the literature stills needs research to determine the possible effects when different layer selections/combinations are employed.

3. MOTIVATIONS

Although various researchers have provided good information about the determination of damaged mastercurves and have modified current methods in the right directions, research gaps and problems remain. Specifically:

- The reported accuracy of the three analysis levels used in Pavement ME varies. Even though a hybrid method (Habbouche et al. 2018) has been proposed to improve accuracy, the effort required to complete all the Level 1 and Level 2 tests plus the suite of dynamic modulus tests is difficult for agencies to rationalize.

- Not enough guidelines are provided for the dynamic modulus characterization of multilayered AC pavements, especially in terms of representative lift selection that could affect both experimental efficiency and performance predictions.

- No clear trend is evident between the damage factor obtained through FWD testing (Level 1) and observed field cracking percentages (Levels 2 and 3) using a large database such as the LTPP program database, suggesting that this relationship may relate specifically to local materials and *in situ* conditions. Clear guidance based on the agency's own database would benefit local agencies.

Given these limitations and the need for localized guidance, the objective of this study is to evaluate the three levels of Pavement ME for determining existing AC layer modulus values based on North Carolina materials and field conditions. The NCSU research team will provide guidance to the NCDOT regarding efficient and reliable ways to characterize existing AC layers. The following goals are targeted in this research:

- Recommend a practical way to implement Pavement ME analysis methods with regard to both accuracy and efficiency.

- Propose a method to determine damaged mastercurves for multilayer AC pavements that includes the selection of a representative pavement layer.

4. FIELD TESTS

Two full-depth pavement structures in NC 96 were selected as the test sections for this study. They are located in the section of NC 96 that crosses the county line between Wake County and Franklin County in North Carolina (near 35.98°N, 78.45°W). Table 2 provides the pavement structure information for these two pavement structures. A total of seven test sections are identified from the two pavement structures. The first 6.5 test sections have the same pavement structure (Structure 1 in Table 2) and the last 0.5 section has a different pavement structure (Structure 2 in Table 2). Each section was 1000 ft long with 250 ft of coring sections at both ends. The middle 500 ft is the condition monitoring section. In the condition monitoring sections, five FWD tests were conducted in the middle of the traffic lane at every 100 ft. In each of the coring sections, four FWD tests were conducted first and then field cores were extracted at the same locations. Each FWD testing location had two FWD drops. The pavement surface temperatures were measured during the FWD tests.

Table 2. Pavement Structure Information for Test Sections

Structure 1 (length: 1.57 mile)			Structure 2 (length: 0.27 mile)		
Construction Year	Material	Thickness (in.)	Construction Year	Material[b]	Thickness (in.)
2006	S 9.5B[a]	3	2006	S 9.5B	3
1994	I-2[b]	1	2000	I-2	1
1986	BCSC[c]	1	2000	HDS[e]	1.25
1970	BCSC	1	1992	I-2	1
1941	BST[d]	0.5	1981	I-2	1
—	Subgrade	—	1941	BST	0.5
—	—	—	—	Subgrade	—
Total	—	6.5	Total	—	7.75

[a]S 9.5B = surface mix with 9.5 mm nominal maximum aggregate size (NMAS) designed for traffic level B; [b] I-2 = light-duty Marshall surface mix; [c] BCSC = bituminous concrete surface course; [d] BST = bituminous surface treatment; and [e] HDS = heavy-duty Marshall surface mix.

Table 3 provides information on FWD testing, coring, and DCP testing. Note that, in the coring sections, DCP testing was done on every other coring location. Figure 1 and Figure 2 present schematics of the field core locations for the first four test sections and the last three test sections, respectively. After coring, dynamic cone penetrometer (DCP) tests were conducted to measure the penetration index of the subgrade at some locations; these values were converted to resilient modulus (*Mr*) values using the relationships expressed in Equations (8) and (9) (Christopher et al. 2010, ARA 2004). Figure 3 and Figure 4 show photographs of the FWD testing and DCP testing processes on NC 96, respectively. Figure 5 shows two field cores extracted from each of the two test pavement structures in NC 96; these photographs show that the core extracted from Structure 2 is longer than the core from Structure 1.

$$CBR = \frac{394}{\overline{x}^{1.066}} \tag{8}$$

$$M_r = 2555 \cdot CBR^{0.64} \tag{9}$$

where

CBR = California bearing ratio, %,

$\overline{x}$ = penetration index, mm/blow, and

M_r = resilient modulus, psi.

Table 3. Field Test Information

	Monitoring Sections (middle 500 ft)	Coring Sections (250 ft at both ends)		
	FWD test	FWD test	Coring	DCP test
Section 1	5	8 (4+4)	8 (4+4)	4 (2+2)
Section 2	5	8 (4+4)	8 (4+4)	4 (2+2)
Section 3	5	8 (4+4)	8 (4+4)	4 (2+2)
Section 4	5	8 (4+4)	8 (4+4)	4 (2+2)
Section 5	5	8 (4+4)	6 (3+3)	4 (2+2)
Section 6	5	8 (4+4)	6 (3+3)	4 (2+2)
Section 7	5	8 (4+4)	6 (3+3)	4 (2+2)

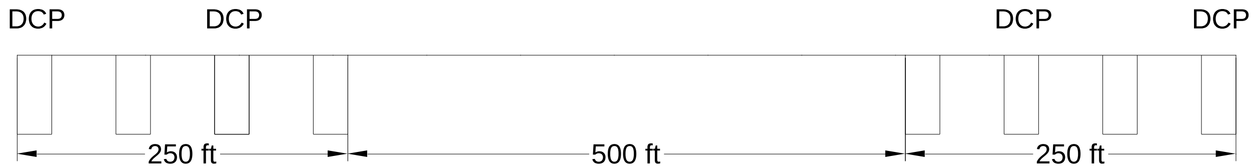


Figure 1. Locations of field cores in Section 1 - Section 4.

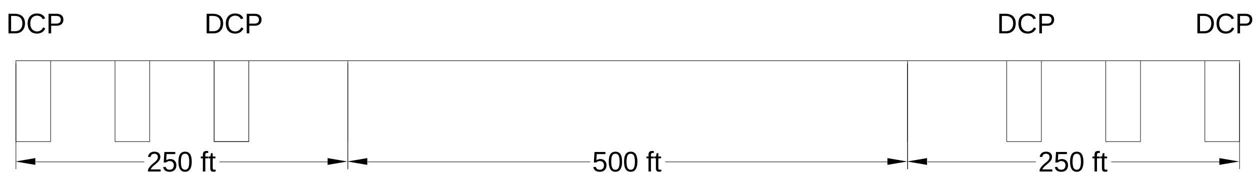


Figure 2. Locations of field cores in Section 5 - Section 7.

Figure 3. Falling weight deflectometer testing on NC 96.

Figure 4. Dynamic cone penetrometer testing on NC 96.

(a) (b)

Figure 5. Field cores obtained from two NC 96 test structures: (a) field core extracted from pavement Structure 1 and (b) field core extracted from pavement Structure 2.

Because the FWD tests and DCP tests were conducted along the NC 96 pavement at multiple locations, the variation in the test results along the project length could be investigated in this research. The maximum FWD-measured deflections range from 4.4 mils to 19.6 mils, indicating that the deflection basin varies significantly when the test location changes. All the measured maximum deflections were plotted against the pavement surface temperature in Figure 6. No clear trend between the maximum deflection and pavement surface temperature, except that Section 6 shows the increase in the deflection as the temperature increases. This figure shows that the variation in the deflections among the locations in the same pavement structure is larger than the effect of temperature change in a day on deflections. Figure 7 shows the plot of deflection versus pavement thickness. Only the locations that were cored are plotted because the thickness was determined by measuring the core heights. In general, Structure 2 shows lower deflections values than Structure 1, but no clear trend is evident regarding the deflections in Structure 1 as a function of pavement thickness.

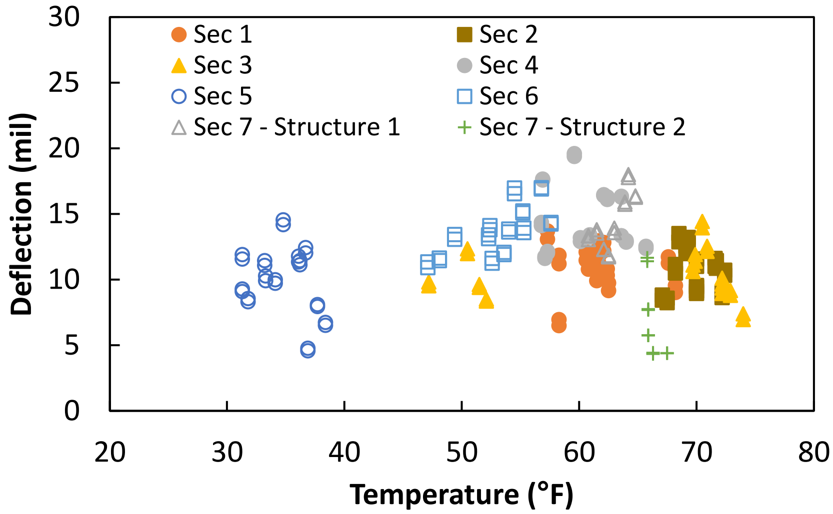


Figure 6. Deflections versus pavement surface temperature.

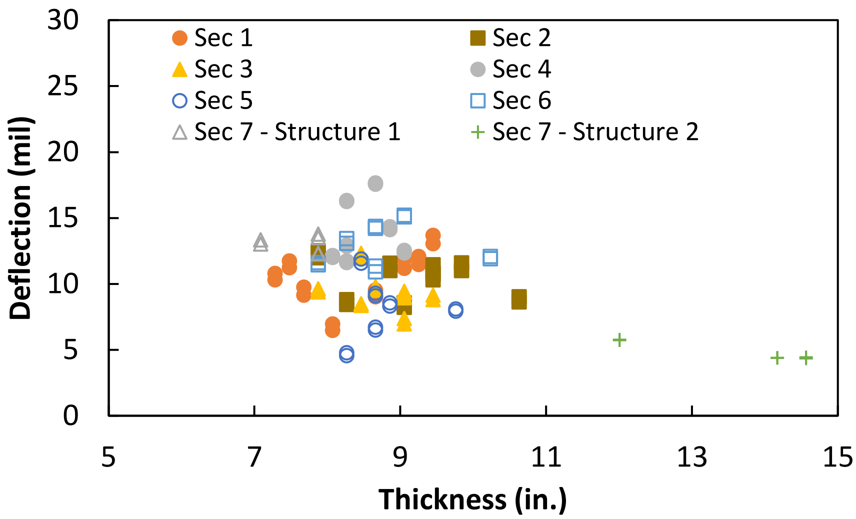
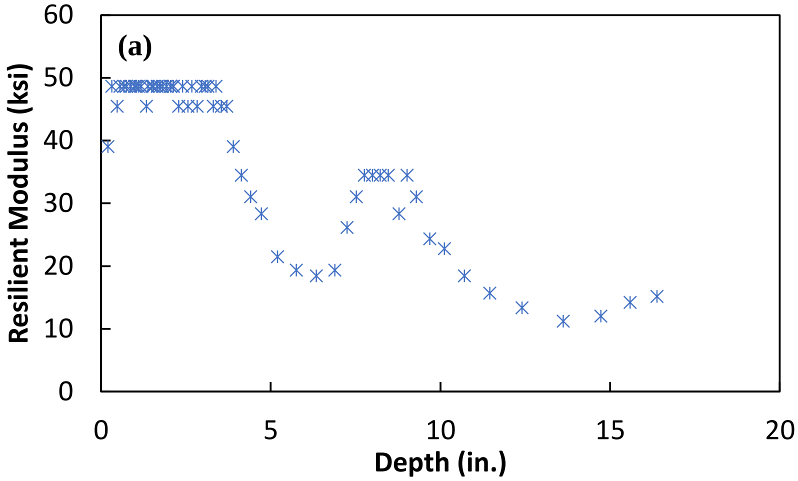
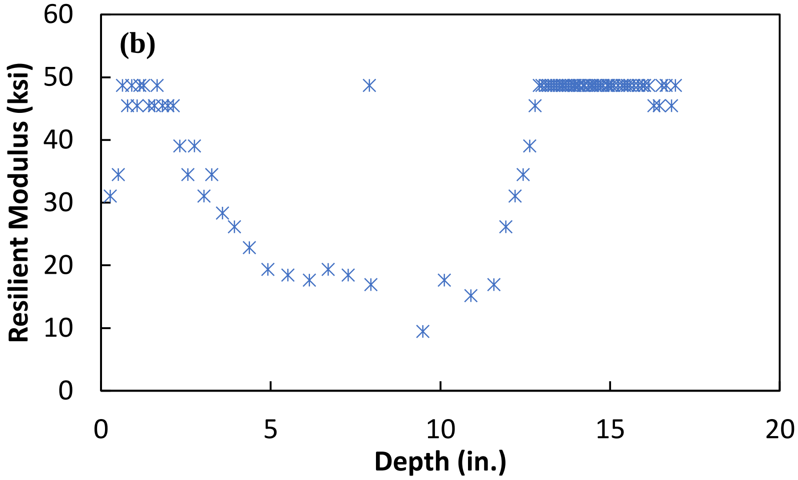
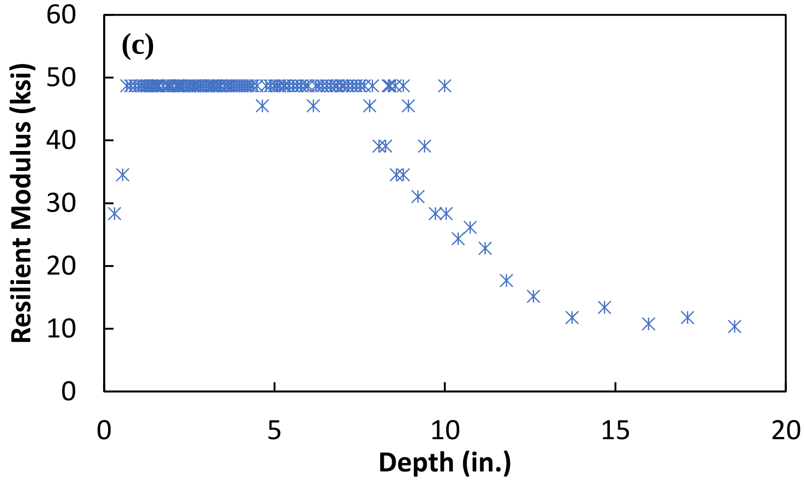


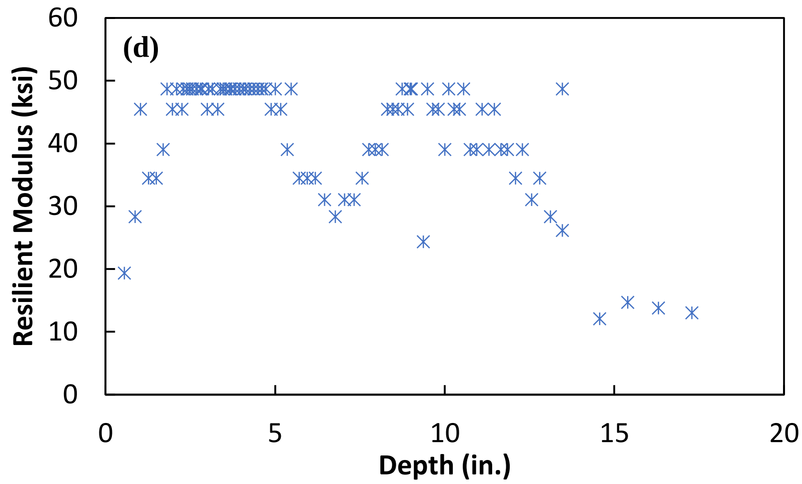
Figure 7. Deflections versus pavement thickness.

Figure 8 shows five different patterns in the DCP-measured resilient modulus results. Each pattern presents a unique relationship between the measured resilient modulus and penetration depth. Note that, even when the test locations are close, the test results show significantly different patterns. This observation indicates that the field variability is large at the different pavement locations.


(a)
Resilient Modulus (ksi)
60
50
40
30
20
10
0
0
5
10
15
20
Depth (in.)


(b)
Resilient Modulus (ksi)
60
50
40
30
20
10
0
0
5
10
15
20
Depth (in.)


(c)
Resilient Modulus (ksi)
Depth (in.)
0
10
20
30
40
50
60
5
15


(d)
Resilient Modulus (ksi)
Depth (in.)
0
10
20
30
40
50
60
5
15

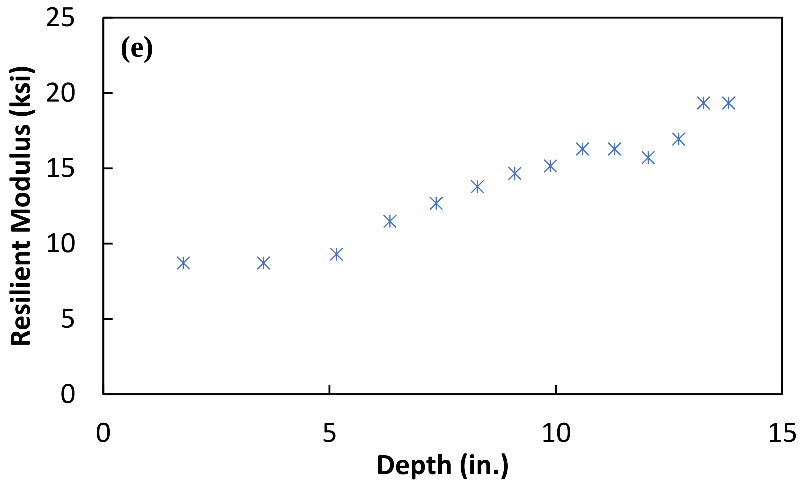


Figure 8. Different patterns of converted resilient modulus values using DCP data: (a) pattern 1, (b) pattern 2, (c) pattern 3, (d) pattern 4, and (e) pattern 5.

Table 4 and Table 5 present the height of each field core measured in the laboratory and the distress type of each field core, respectively. Note that the thicknesses of the field cores are always greater than the total AC thickness shown in Table 2, and the height difference between the longest core and shortest core from the same structure is as much as 3.5 inches, indicating significant construction variability in the field.

To investigate the variation in AC stiffness throughout the different pavement sections, two field cores were selected from each test section for dynamic modulus testing, as shown in Figure 9. The selection of the cores was based mainly on the integrity of the samples. If the core did not have debonding or severe cracking that would affect the dynamic modulus test operation, then it could be selected as a test specimen. In addition, three different locations were selected for the tests required by the Pavement ME method. Only three locations were chosen because the Pavement ME test method requires large amounts of materials (details are provided later in this report). These three locations are in Section 3, Section 6, and Section 7, respectively, and are referred to hereafter as Locations A, B, and C. Locations A and B represent Structure 1 and Location C represents Structure 2. In terms of the distress type at the selected sections, Location A did not show severe debonding (based on visual observation) whereas Location B had serious debonding issues. The debonded cores are marked as ‘D’ in Table 5 and had broken into parts when extracted from the field. Similarly, if the core surface showed cracks or stripping, the core is labeled as ‘C or ‘S’ in Table 5, respectively.

Table 4. Height of Each Field Core (in.)

Section	Number of the field core							
	1	2	3	4	5	6	7	8
1	9.4	9.1	8.1	9.3	7.9	7.3	8.7	7.5
2	9.1	9.1	8.3	7.9	9.4	9.8	10.6	9.4
3	9.4	9.1	9.4	9.1	7.9	8.5	8.7	8.5
4	7.9	8.9	8.7	8.3	7.9	8.3	8.3	9.1
5	8.5	8.7	8.7			8.3	9.1	8.7
6	8.7	8.3	7.9			10.2	9.1	8.7
7	7.9	7.1	7.9			12.0	14.6	14.2

Table 5. Distress of Each Field Core

Section	Number of the field core							
	1	2	3	4	5	6	7	8
1	D & C	None	None	D	None	None	None	None
2	None	D	None	None	None	C	None	C
3	None	None	D	C	C	C	C	None
4	None	C	D & C	None	D & S	C	C	D
5	None	None	None	None	None	None	None	None
6	None	D	D	None	None	D	C	D
7	S	None	S	None	None	None	None	None

Note: C, D, and S stand for cracking, debonding, and stripping, respectively.

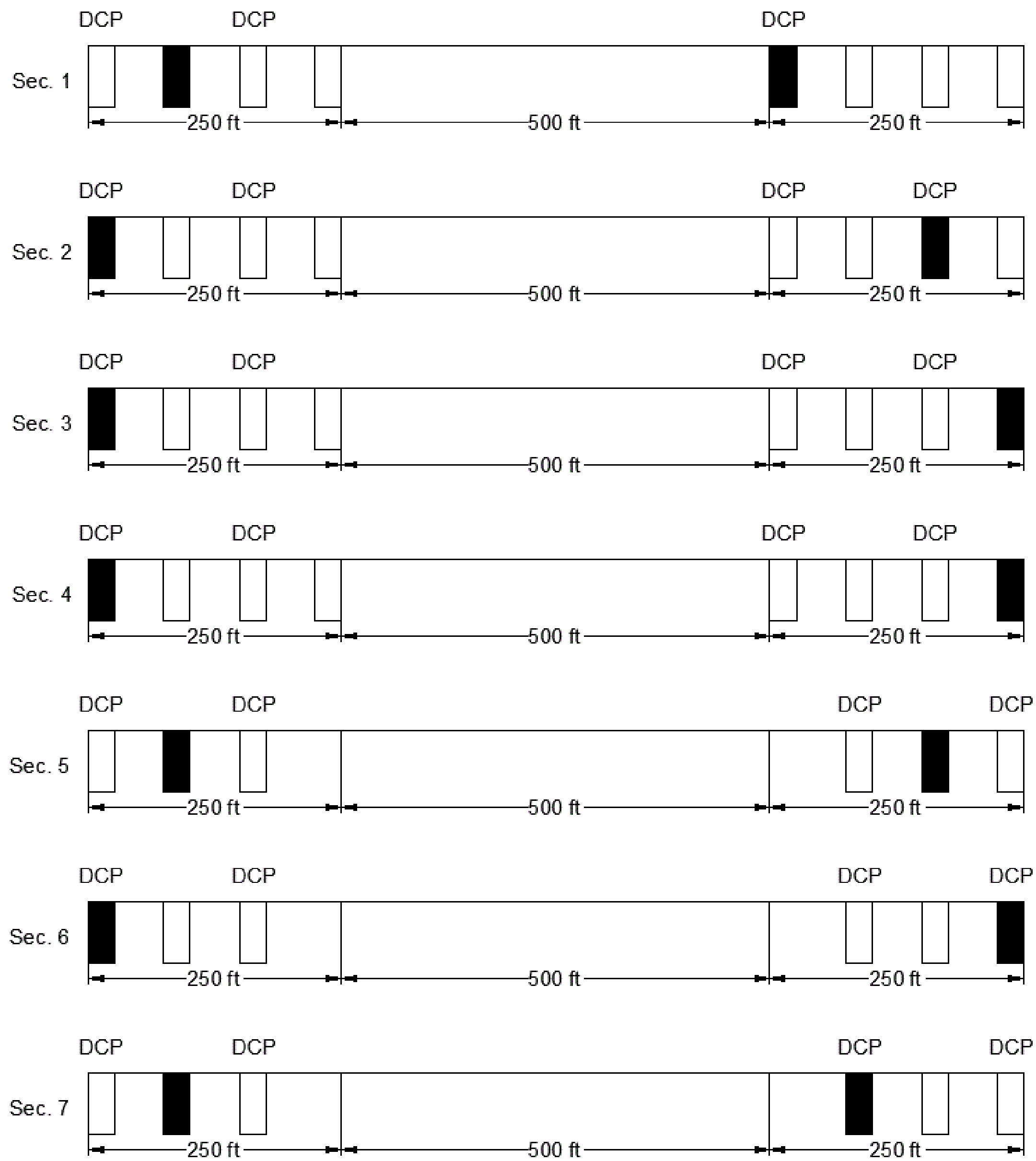


Figure 9. Field cores selected in Sections 1 through 7 for dynamic modulus testing.

5. LABORATORY TESTS

As described, 14 field cores (two from each of the seven test sections) were selected for dynamic modulus testing. Analysis of the Pavement ME method was conducted based on the three selected core locations (i.e., Locations A, B, and C). Because the NC 96 pavement consists of multiple AC layers and most of them are less than 1.5 inches in thickness, the individual layers could not be evaluated through laboratory testing. The top layer (S 9.5B) is relatively thick (3 in.) and could be evaluated easily based on visual observation of the core surface. To investigate ways to select a representative layer, each core was divided into a top part (the S 9.5B layer) and bottom part (the rest of the AC layers). Accordingly, tests of the top part, bottom part, and total core were conducted separately to mimic scenarios in which each case (top, bottom, and total core) was selected as the representative lift to determine the damaged mastercurve. To maximize testing efficiency, the procedures and tests were conducted in a certain order, as shown in Figure 10. Details of each test are given in the following sections.

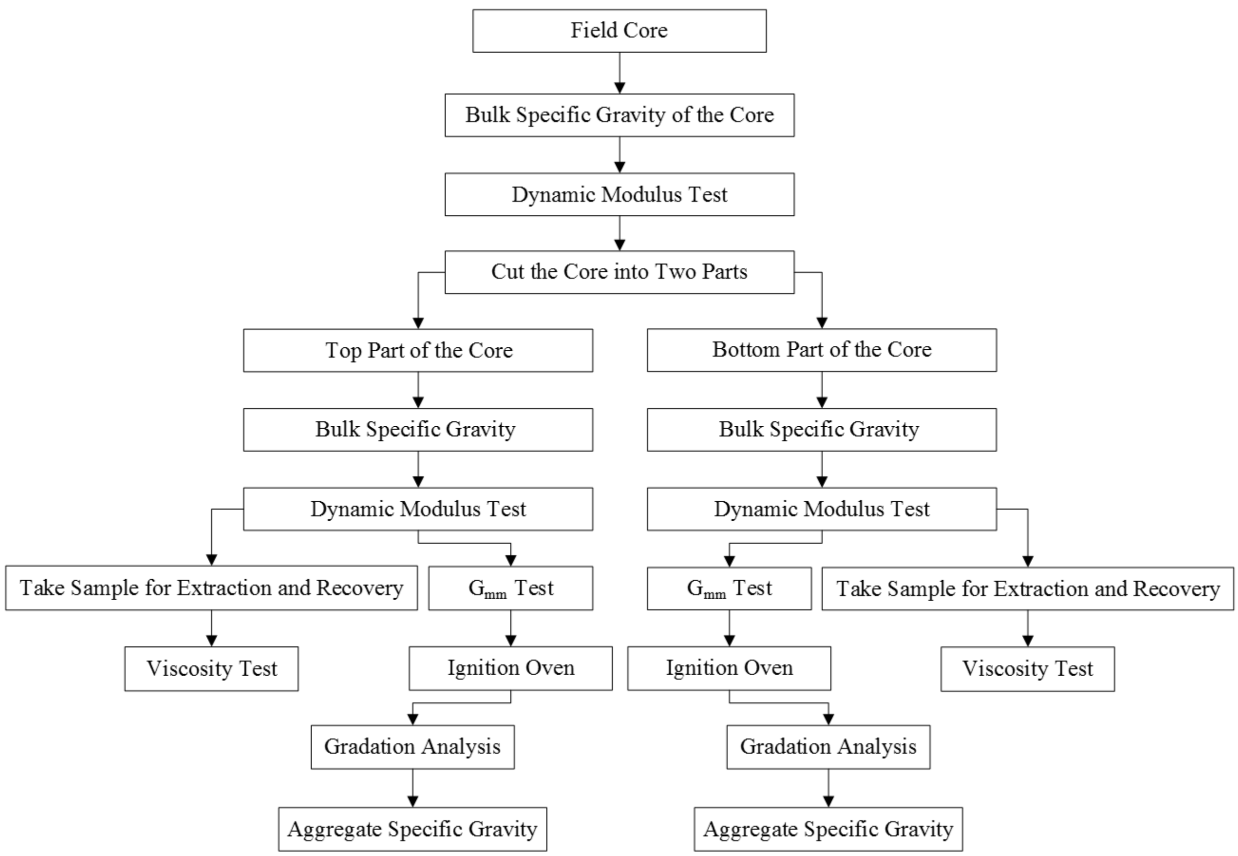


Figure 10. Flowchart of test plan for dynamic modulus testing.

5.1. Testing Required to Determine Input Properties for Witczak's Predictive Equation

5.1.1. Volumetric Property Tests and Gradation Analysis

The bulk specific gravity (G_{mb}) of each core was measured before breaking down the specimen. After this property was obtained, the core was heated to 230°F. Edge particles were removed because these particles are affected by cutting and do not hold their original aggregate shape. The maximum specific gravity (G_{mm}) was measured from the loose mix following AASHTO T 209.

Next, ignition oven tests were performed to obtain the binder content. The left-over aggregate from the ignition oven tests was used for gradation analysis that provided the percentage of the aggregate that was retained on each sieve. Finally, the specific gravity values of the coarse aggregate and fine aggregate were measured to help determine the effective binder content/volume.

Each test had two replicates. In order to mitigate field variation, every effort was made to ensure that the materials used in each test came from the same location. Moreover, volumetric measurements were taken only of the top part and bottom part of each core; then, the same data were used to calculate the corresponding property for the total core. Despite using this procedure, a single core was unable to provide enough material to satisfy the minimum mass requirement for gradation analysis and specific gravity testing. Therefore, in such cases, nearby cores were used to complete the tests.

5.1.2. Extraction, Recovery, and Binder Viscosity Measurements

For the selected specimens, vertical cores that contained all the targeted layers were used for binder extraction and recovery. To make sure any solvent was removed completely following binder recovery, a degassing oven and Fourier transform infrared spectroscopy were used to check the amount of solvent that remained in the binder specimens. Once no solvent could be detected, the extracted binder was put into a dynamic shear rheometer (DSR) to measure the shear modulus values at 1.59 Hz and multiple temperatures (40°F, 55°F, 70°F, 85°F, 100°F, 115°F, and 130°F), as specified in the Pavement ME Guide (ARA 2004). Equation (10) was employed to convert the shear modulus to viscosity.

$$\eta = \frac{|G^*|}{10}\left(\frac{1}{\sin\delta}\right)^{4.8628} \cdot 1000 \tag{10}$$

where

η = viscosity, cP,

$|G^*|$ = shear modulus, Pa, and

δ = phase angle, °.

Extraction and recovery were performed only for the top part and bottom part of each core. As for the total core, instead of taking another core sample to run extraction and recovery, the viscosity values of the top and bottom parts were used to estimate the viscosity of the total core based on the assumption shown in Equation (11). This decision was made to maintain consistency in the results because any variation due to sample selection or test operations could influence the interpretation of the results dramatically. In addition, even though the total core was extracted, homogenizing it for DSR testing would be difficult due to its small specimen geometry (8 mm in diameter and 2 mm in height), which could cause errors in the sampling process when the amount of extracted binder is large.

$$\eta_{\text{total}} = \frac{\eta_{\text{top}} w_{\text{top}} + \eta_{\text{bottom}} w_{\text{bottom}}}{w_{\text{top}} + w_{\text{bottom}}} \tag{11}$$

where

η_{total} = viscosity of total core,

w_{top} = top part weight,

η_{top} = viscosity of top part,

η_{bottom} = viscosity of bottom part, and

w_{bottom} = bottom part weight.

5.2. Dynamic Modulus Testing

The selected cores were trimmed and smoothed before testing to ensure that they would make good contact with the loading platens to ensure a uniform stress condition. Dynamic modulus measurements of the total vertical core were taken first of specimens 150 mm (6 in.) in diameter. The reason for not using 100-mm (4-in.) specimens is that the cores could debond easily at the layer interface during the additional coring process. A Material Test System (MTS) was used to conduct the dynamic modulus tests. The test temperatures used for the dynamic modulus tests were 39.2°F, 68°F, and 104°F and the test frequencies were 0.1 Hz, 0.5 Hz, 1 Hz, 5 Hz, 10 Hz, and 25 Hz. Figure 11 presents the test configuration. The reason to use this test configuration is to include all the layers in the testing. Before the measurements were taken, a dummy specimen of the same size as the test specimens was used to determine the conditioning time. Two thermocouples were installed on the dummy specimen by drilling and sealing: one on the surface and the other at the center of the specimen. By placing the dummy specimen into the same environment as that used for dynamic modulus testing, the temperatures could be recorded and evaluated to determine the conditioning time. As an example, Figure 12 presents the 104°F data. Based on this plot and data analysis, the conditioning time at this temperature was determined to be five hours. Following the same procedure, the conditioning time of five hours also was confirmed as a reasonable time at the other test temperatures.

Figure 11. Dynamic modulus test configuration.

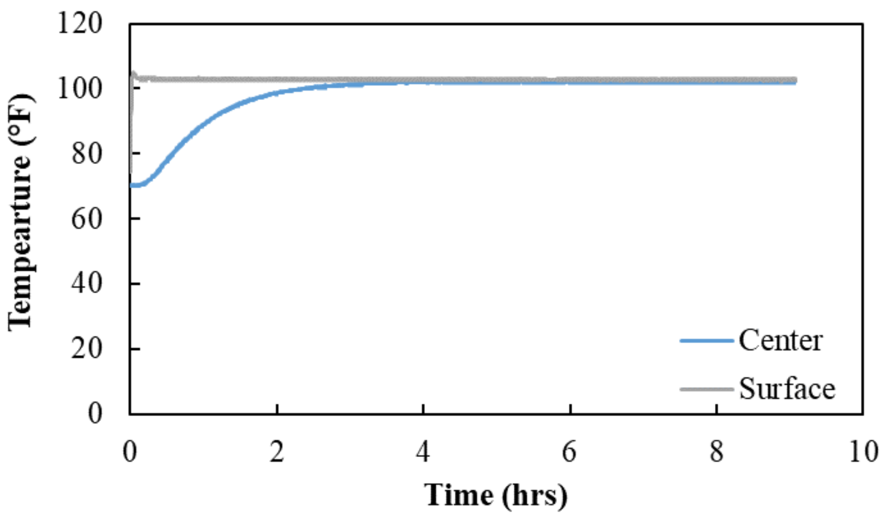


Figure 12. Example of conditioning time determination (104°F).

For the cores selected for Pavement ME analysis, after measurements were taken of the whole field core, the core was cut into the top and bottom parts. Two cores (4.3 in. × 1.5 in. or 110 mm × 38 mm) were obtained from the top part by coring the field core horizontally, but the bottom part was still tested vertically. The reason for using horizontal cores for the top part is based on the vertical thickness limitation (Castorena et al. 2017, Pape et al. 2018). The test temperatures and frequencies used to test these two specimen geometries were the same as the conditions used for the total core.

6. DYNAMIC MODULUS TEST RESULTS

Figure 13, Figure 14, Figure 15 and Figure 16 present the dynamic modulus test results for the selected 14 cores and Figure 17, Figure 18, Figure 19, and Figure 20 present the phase angle results after shifting the measured modulus values and phase angles horizontally using the time-temperature superposition principle. In the graphs, the specimen name is formatted as X-Y, where X represents the section number and Y represents the location of the core. The measured results clearly indicate that the modulus obeys the time-temperature superposition principle very well. Most of the phase angle results also have smooth curves after horizontal shifting. The measured stiffness values that the specimens display in the reduced frequency domain are important because these values indicate the modulus variance throughout the NC 96 test sections. In addition, the measured dynamic modulus values can be used in comparison with the backcalculated modulus values when the Pavement ME backcalculation tool is applied.

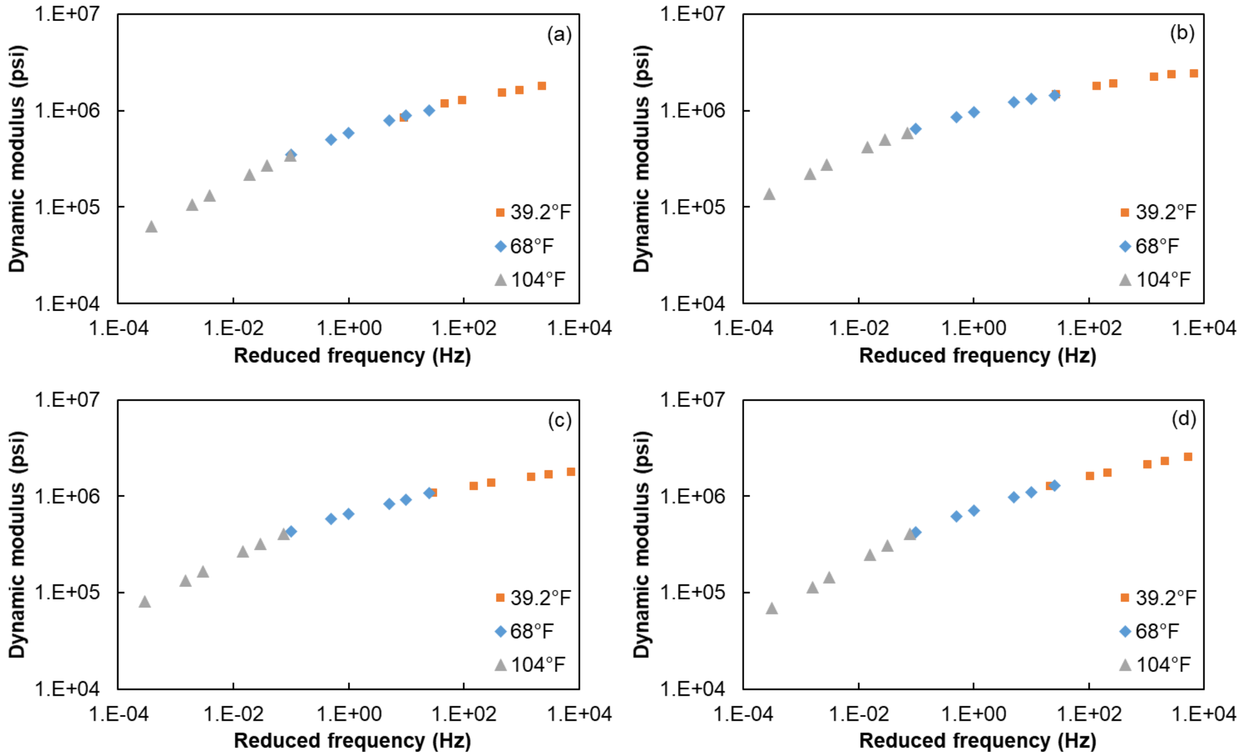


Figure 13. Dynamic modulus results: (a) 1-2, (b) 1-10, (c) 2-1, and (d) 2-12.

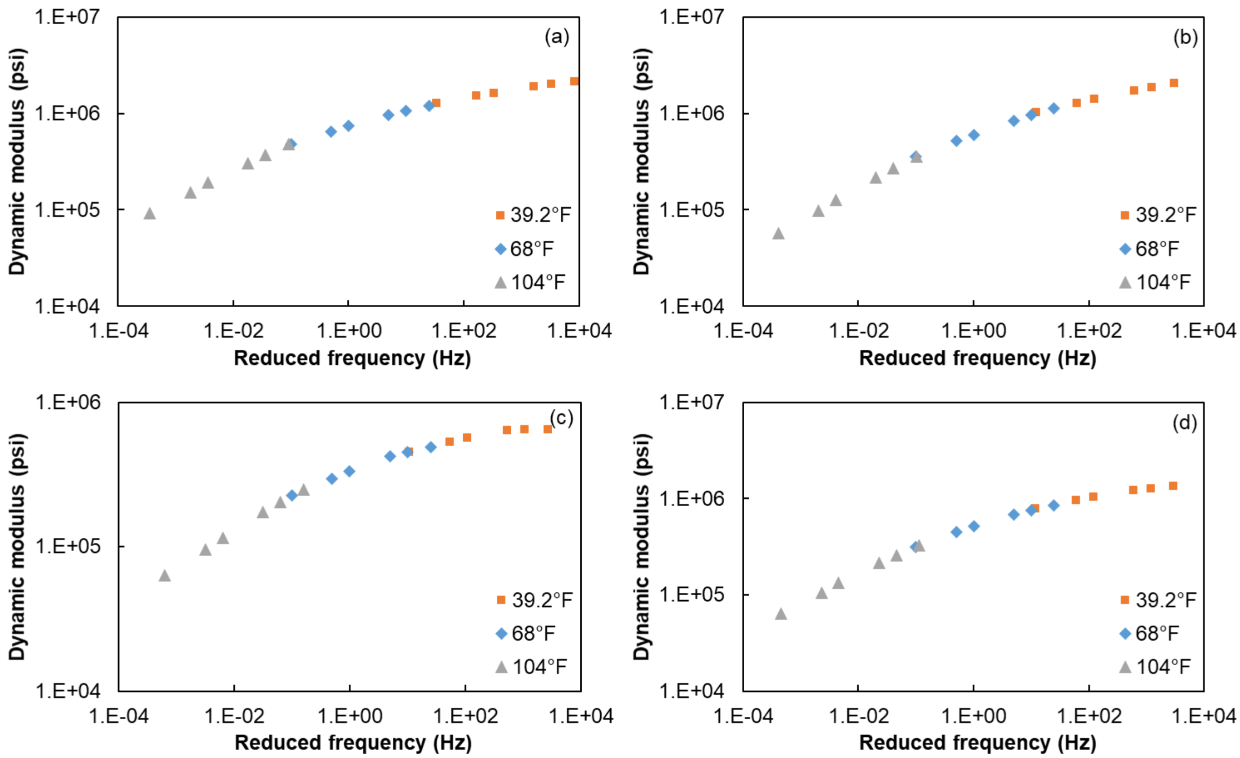


Figure 14. Dynamic modulus results: (a) 3-1, (b) 3-13, (c) 4-1, and (d) 4-13.

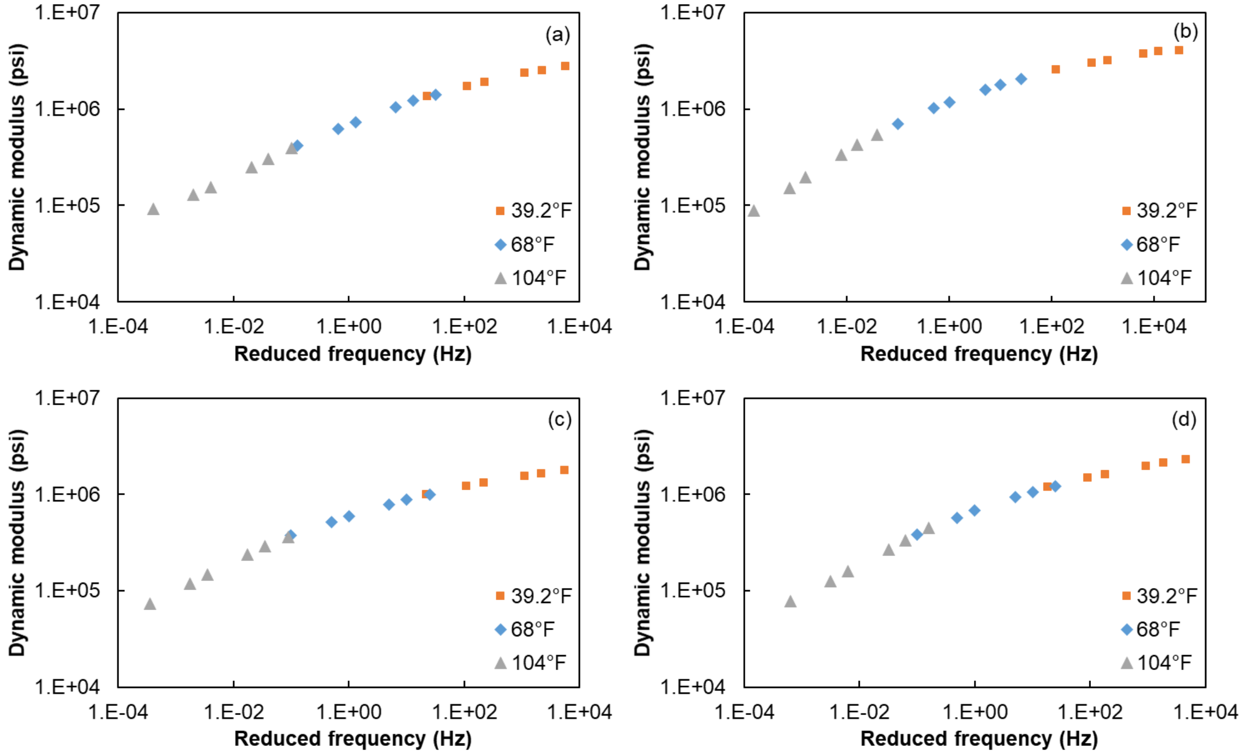


Figure 15. Dynamic modulus results: (a) 5-3, (b) 5-12, (c) 6-1, and (d) 6-13.

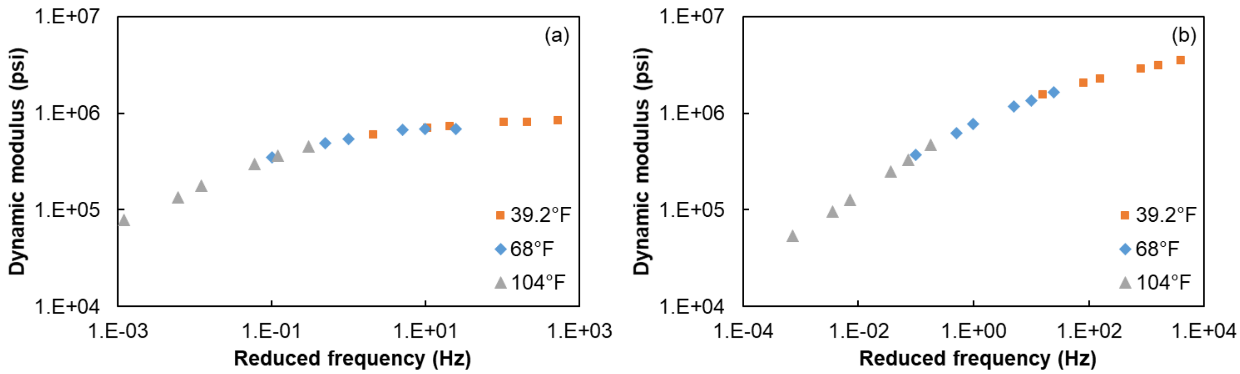


Figure 16. Dynamic modulus results: (a) 7-2, (b) 7-11.

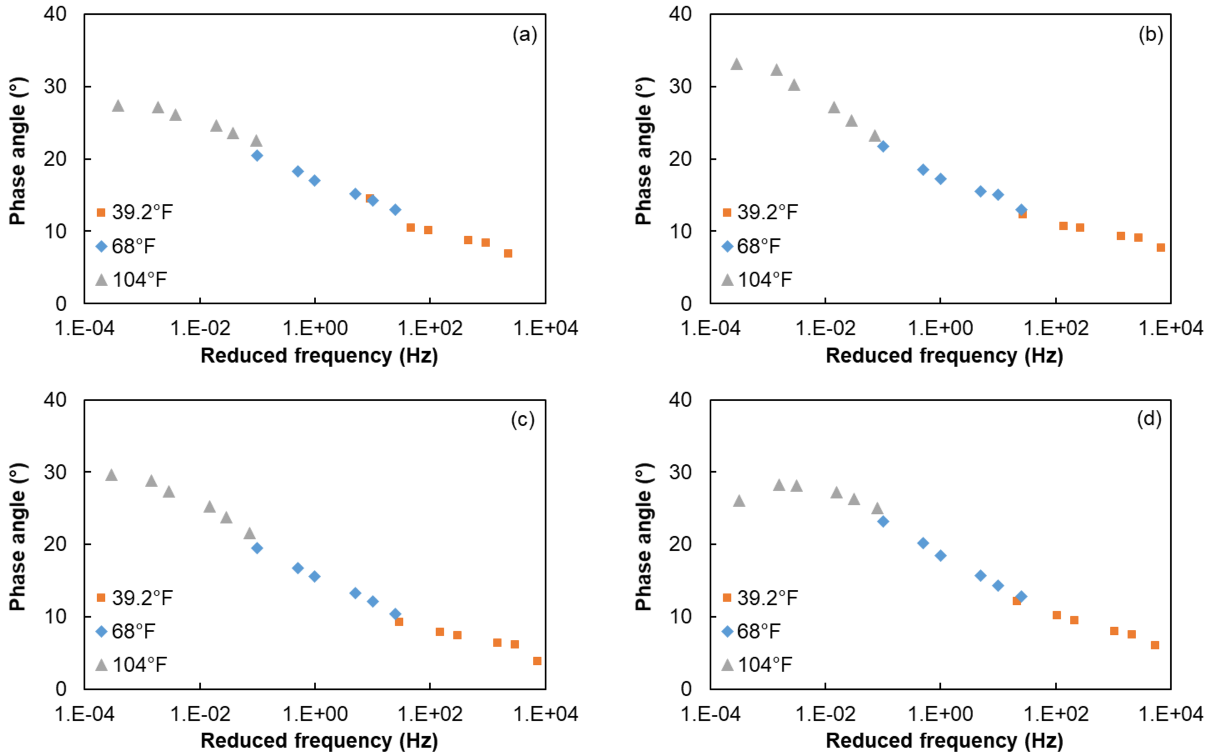


Figure 17. Phase angle results: (a) 1-2, (b) 1-10, (c) 2-1, and (d) 2-12.

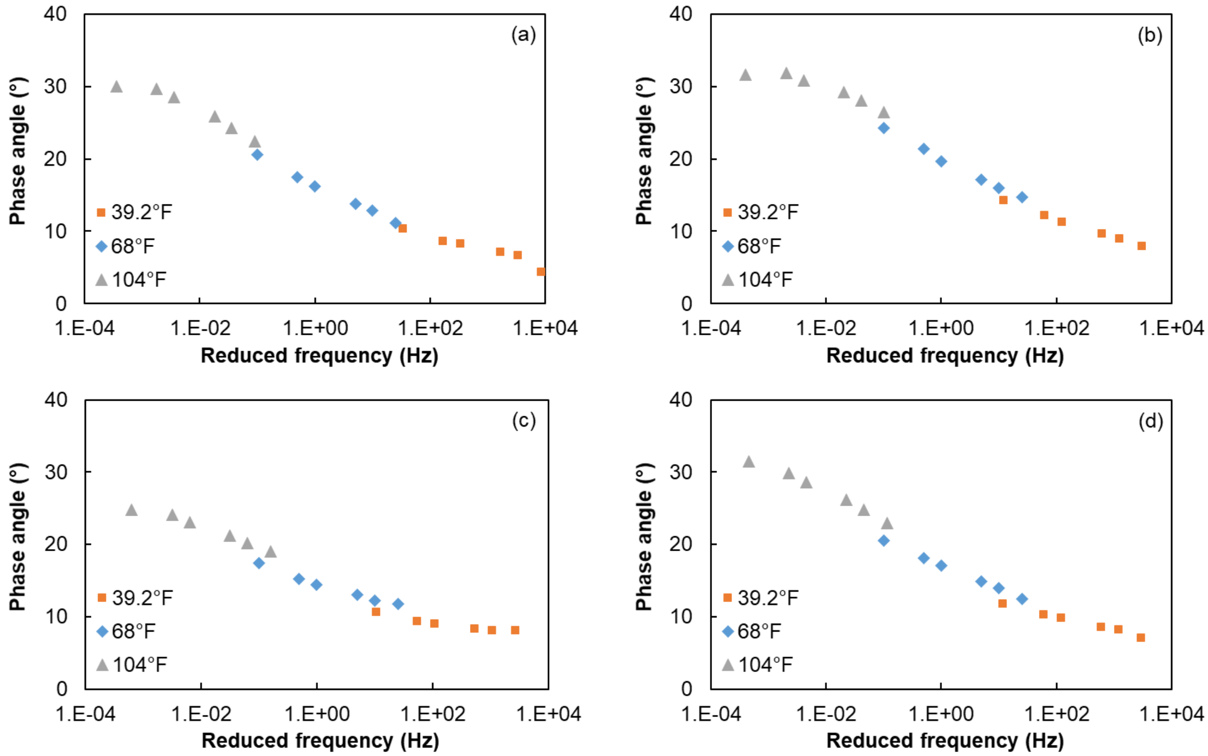


Figure 18. Phase angle results: (a) 3-1, (b) 3-13, (c) 4-1, and (d) 4-13.

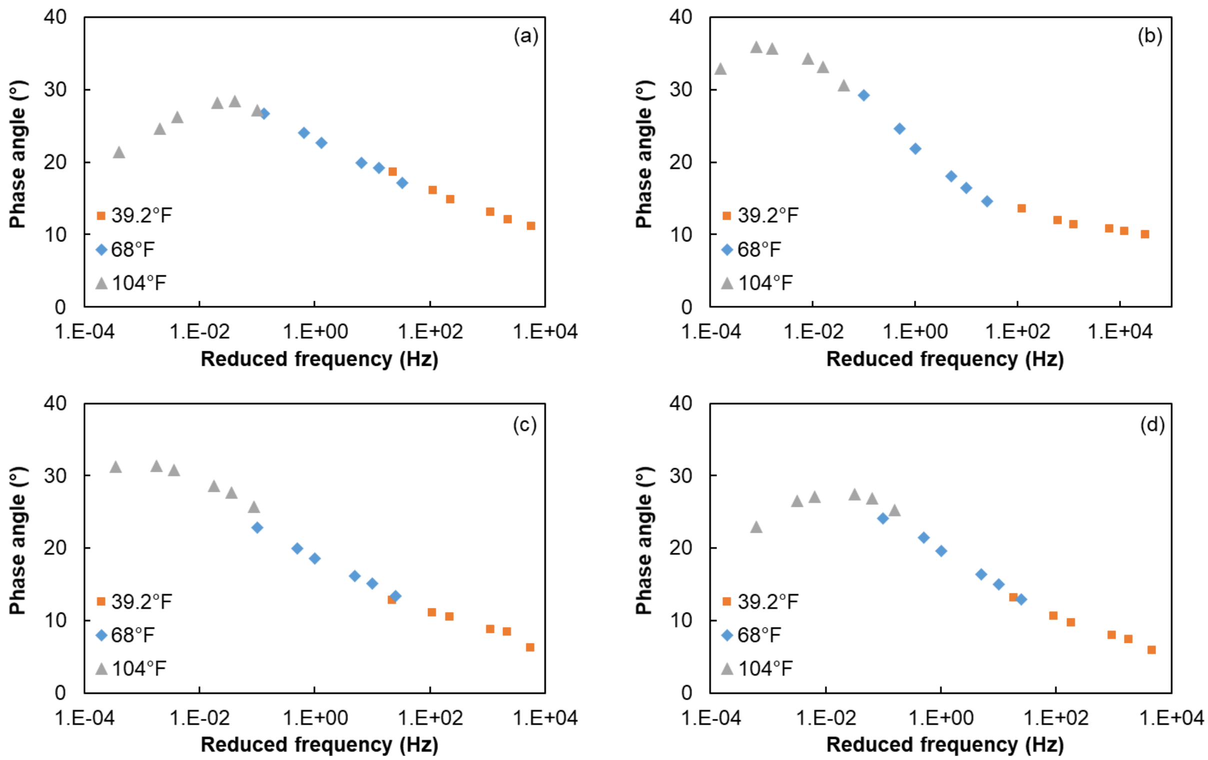


Figure 19. Phase angle results: (a) 5-3, (b) 5-12, (c) 6-1, and (d) 6-13.

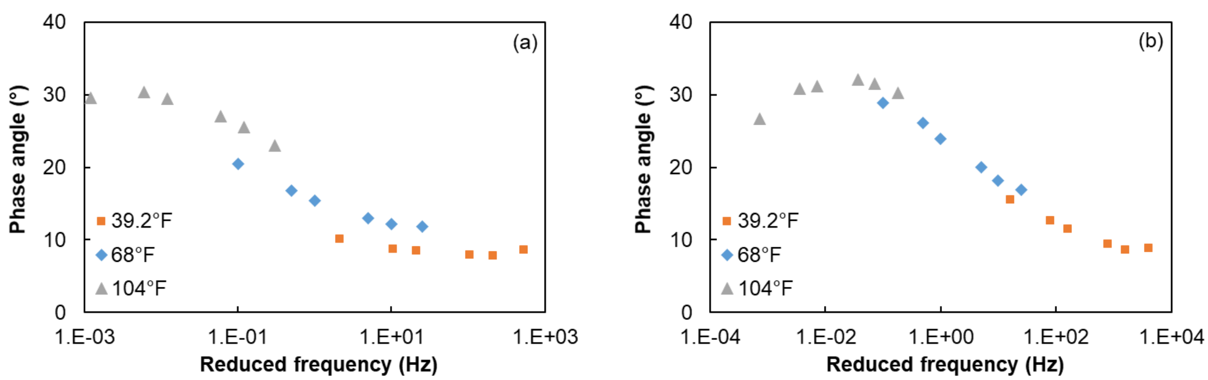


Figure 20. Phase angle results: (a) 7-2, (b) 7-11.

A sigmoidal function was used to fit the dynamic modulus data to achieve a single smooth mastercurve, as shown in Equation (12). Equation (13) shows the relationship between reduced frequency and frequency. A second polynomial function was used to obtain the shift factor, as shown in Equation (14). By fitting the mastercurves presented in Figures 13 through 18 using these three equations, the coefficients of the model, shown in Table 6, could be determined. Table 6 and the mastercurves for each core show that the mastercurve magnitude varies according to pavement location, which is indicated by the sigmoidal coefficients and the modulus values in the figures. However, the shapes of the mastercurves are all similar. In the next step, these coefficients are used to estimate the elastic modulus value of each field core for comparison against the backcalculated values.

$$\log E\left(f_R\right) = c_1 + \frac{c_2}{1+e^{-c_3-c_4 \log f_R}} \tag{12}$$

$$f_R = f \cdot \alpha_T \tag{13}$$

$$\log \alpha_T\left(T\right) = aT^2 + bT + c \tag{14}$$

where

f = frequency, Hz,

α_T = shift factor,

f_R = reduced frequency, Hz,

T = temperature, °C, and

a, b, c = fitting coefficients.

Table 6. Coefficients of Mastercurves

Name	a	b	c	c_1	c_2	c_3	c_4
1-2	-0.0002	-0.1036	2.1487	1.7530	2.5314	1.0094	0.4782
1-5	0.0005	-0.1524	2.8427	1.7516	2.6181	1.3210	0.4132
2-1	0.0009	-0.1741	3.1205	0.9318	3.2882	1.5687	0.3998
2-12	0.0004	-0.1552	2.9504	0.2346	4.2917	1.4242	0.3137
3-1	0.0010	-0.1773	3.1569	0.4487	3.8958	1.6360	0.3570
3-13	0.0001	-0.1196	2.3498	1.3130	3.0264	1.1796	0.4553
4-1	0.0005	-0.1384	2.5769	1.5036	2.2355	1.6042	0.4941
4-13	0.0004	-0.1386	2.6266	0.5525	3.5895	1.6241	0.3883
5-3	0.0004	-0.1498	2.8314	2.0491	2.4165	0.7302	0.4570
5-12	0.0011	-0.1995	3.5553	1.5256	3.0600	1.2600	0.4319
6-1	0.0006	-0.1587	2.9220	0.4325	3.8777	1.5187	0.3435
6-13	0.0009	-0.1628	2.9026	0.9860	3.4043	1.3096	0.3980
7-2	-0.0004	-0.0741	1.6276	2.0631	1.7126	2.0348	0.8492
7-11	0.0008	-0.1598	2.8595	0.8505	3.7548	1.1791	0.4226

Note: The specimen name is formatted as X-Y, where X represents the section number and Y represents the location of the core.

7. COMPARISON OF MEASURED DYNAMIC MODULUS VALUES VERSUS BACKCALCULATED MODULUS VALUES

To compare the measured modulus values with the backcalculated values, the first step is to convert the dynamic modulus to the elastic modulus. In previous studies, the FWD frequency was normally assumed to be 5~30 Hz and 30 Hz was frequently used in literature (AASHTO 2017, Lee et al. 2017, Ayyala et al. 2017, 2018). For this study, the dynamic modulus at 30 Hz is assumed to be approximately equal to the elastic modulus, as shown in Equation (15).

$$|E^*|_{f=30\text{Hz}} \approx E_{Elastic} \tag{15}$$

where

$|E^*|_{f=30Hz}$= dynamic modulus value when the frequency is 30 Hz, and

$E_{Elastic}$ = elastic modulus.

Initially, only one AC layer and one subgrade layer were considered for backcalculation. Nevertheless, the backcalculated results were found to be dependent on the way the material properties and structural information were input. Specifically, using only a one-layer subgrade in the backcalculation sometimes resulted in a large root mean square error (RMSE). Based on this observation, scenarios with two subgrade layers (20 in. + infinity) also were tried. Meanwhile, the subgrade modulus values were determined using three methods: DCP measurements, Equation (16) predictions (AASHTO 1993), and direct backcalculation.

$$E_{\text{sg}} = \frac{\left(1-\mu^2\right)P}{\pi d_r r} \tag{16}$$

where

d_r = deflection at distance r from the center of the load,

E_{sg} = subgrade resilient modulus,

μ = Poisson's ratio,

P = applied load, and

r = distance from center of load.

Instead of showing the results for each test section, the preselected locations A, B, and C are chosen here for evaluation and comparison purposes. In total, seven scenarios were tried for backcalculating the dynamic modulus values, as shown in Table 7. Because the RMSE must be less than 5 to be considered for reasonable backcalculation results (AASHTO Pavement ME

Design Task Force 2017), the only scenario that satisfies this requirement is the last row in Table 7, which will be used for elastic modulus determination. Note that, although the results for the other locations are not shown, their circumstances are similar to those explained above in terms of modulus backcalculation.

Table 7. Backcalculated Modulus Values

Number of Subgrade Layers	Subgrade Modulus Determination Method	Determined Modulus Values[f] (ksi)		
		A	B	C
1	DCP Measured	206 + 39 (83.1)	108 + 35 (34.6)	866 + 26 (6.6)
1	Predicted	118 + 97 (44.7)	166 + 24 (10.8)	966 + 25 (9.4)
1	Backcalculated	151 + 72 (39.2)	178 + 22 (9.0)	728+ 28 (3.2)
2	DCP + Predicted	250 + 39 + 97 (37.0)	124 + 35 + 24 (16.5)	943 + 26 + 25 (9.2)
2	DCP + Backcalculated	245 + 39 + 73 (30.1)	128 + 35 + 22 (15.4)	746 + 26 + 28 (3.0)
2	Backcalculated + Predicted	796 + 10 + 97 (14.6)	289 + 13 + 24 (1.6)	634 + 48 + 25 (6.9)
2	Backcalculated + Backcalculated	498 + 12 + 74 (4.8)	292 + 13 + 24 (1.7)	848 + 16 + 30 (1.8)

[f]A and B represent two separate locations in Structure 1, C represents the location in Structure 2, and the backcalculated modulus values are formatted as X + Y (Q) or X + Y + Z (Q) where X = modulus of AC, Y and Z = moduli of subgrade, and Q = RMSE, %.

Another input that is required for the viscoelastic and elastic conversion is the mid-depth temperature of the pavement. This temperature can be predicted using the LTPP program's BELLS3 equation, shown here as Equation (17) (Lukanen et al. 2000).

$$\begin{aligned} T_d &= 0.95 + 0.892IR \\ &+ \left(\log(d) - 1.25\right)\left(-0.448IR + 0.621(1-day) + 1.83\sin\left(hr_{18} - 15.5\right)\right) \\ &+ 0.042IR\sin\left(hr_{18} - 13.5\right) \end{aligned} \tag{17}$$

where

T_d = temperature at pavement depth, °C,

IR = infrared pavement surface temperature, °C,

d = depth at which the temperature needs to be predicted, mm,

1-day = average air temperature the day before testing, °C, *sin* = sine function in an 18-hr clock system, and

hr_{18} = time of day calculated using an 18-hr AC temperature rise and fall time cycle.

Using the aforementioned method, the AC elastic modulus values at the 14 locations where the dynamic modulus mastercurves were determined were backcalculated. Figure 21 presents a comparison of the backcalculated modulus values versus the lab-measured modulus values. The results show that the lab-measured modulus values are much higher than the backcalculated modulus values for all locations. This outcome may be related to the fact that NC 96 is an old pavement and has aged for a long time in the field. When conducting the FWD tests, the technicians noticed that the pavement surface already had many cracks, as shown in Figure 22. Therefore, when the FWD loading plate was dropped onto the pavement surface, the load transfer throughout the pavement may have been affected by existing cracks. If debonding of the layers was present beneath the road surface, then the modulus values measured via the FWD would be affected accordingly. In comparison, the dynamic modulus specimens tested in the laboratory did not have debonding; thus, the modulus values obtained from those measurements should be higher than the field modulus values.

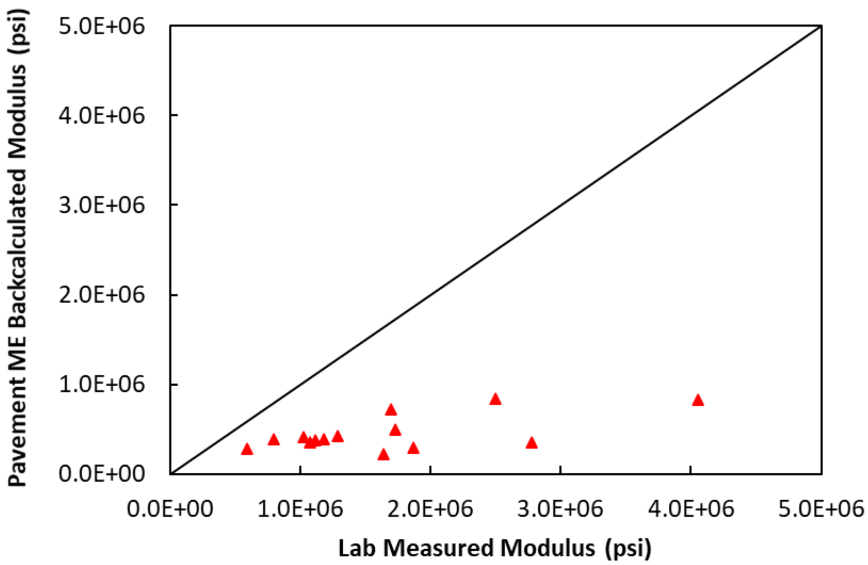


Figure 21. Comparison between backcalculated modulus values versus lab-measured modulus values.

Figure 22. Cracks on NC 96 pavement surface.

To evaluate the amount of cracking on NC 96, the cracking condition survey data obtained from the NCDOT were checked. Table 8 shows that the percentages of alligator cracking are high, revealing that the FWD test results could be affected by these cracks when the stress wave propagates throughout the pavement.

Table 8. Alligator Cracking Condition Survey Results

Name	Structure 1	Structure 2
Amount of Alligator Cracking (%)	33	28
Ratings of Transverse Cracking	Low	None

8. EVALUATION OF THE REHABILITATION ANALYSIS LEVELS IN THE PAVEMENT ME GUIDE

This part of the evaluation is based on the field cores that were extracted from the three preselected locations (i.e., locations A, B, and C). The NCSU research team determined dynamic modulus mastercurves for the top, bottom, and total core and conducted the laboratory tests required by Witczak's predictive equation following the procedures listed in Section 5.1. Following the Pavement ME Guide, the research team also performed the different levels of rehabilitation design analysis in this study. Sections 8.1, 8.2, and 8.3 provide details of the Levels 1, 2, and 3 analyses, respectively.

8.1. Level 1 Analysis

The test results for the total cores were used to determine the inputs for Witczak's predictive equation to satisfy the equation for Level 1 analysis. In addition, FWD tests and backcalculations were conducted to obtain the field modulus values following the protocol described in Section 7. To determine the damage factor (d_{AC}), the backcalculated AC modulus value was put into Equation (6) for comparison with the undamaged modulus value predicted by Witczak's equation at the FWD testing temperature and frequency. The equivalent frequency of the FWD test was assumed as 30 Hz. Table 9 lists the corresponding damage factor results.

8.2. Level 2 Analysis

The Pavement ME Guide requires a condition survey of alligator cracking for Level 2 analysis. The NCDOT uses four ratings to evaluate the amount of alligator cracking in the recorded automated distress data: none, 'alligator low', 'alligator moderate', and 'alligator high'. In earlier NCDOT research work, a conversion equation, shown here as Equation (18), was developed to convert the NCDOT condition survey data to the LTPP condition survey data (Corley-Lay et al. 2010). Based on this equation, the alligator cracking for the three locations is shown in Table 9.

$$\text{Total Alligator Cracking} = X_1(m^2)(Low) + X_2(m^2)(Mod) + X_3(m^2)(High) \tag{18}$$

where

X_1 = 5.9,

X_2 = 1.7,

X_3 = 13.9,

Low = amount of cracking that corresponds to the rating 'alligator low', m^2,

Mod = the amount of cracking that corresponds to the rating 'alligator moderate', m^2, and

High = the amount of cracking that corresponds to the rating 'alligator high', m^2.

The condition survey data for alligator cracking were used in Equation (7) to estimate the damage factor for Level 2 analysis. Table 9 also lists these calculated damage factors (d_{AC}). The d_{AC} values at Level 1 and Level 2 differ significantly for all three locations, revealing that these two levels can lead to very different results. Note that the Level 3 results are not shown in the table because Level 3 requires only condition survey ratings and the AASHTOWare Pavement ME Design software calculates the d_{AC} internally.

Table 9. Damage Factor Results

Name	Core Location	A	B	C
Level 1	AC Thickness (in.)	8.5	8.7	12.0
	Mid-depth Temperature of Pavement (°F)	51.0	50.8	54.7
	Undamaged $\|E^*\|$ at FWD Test Temperature and Frequency (psi)	2,736,286	2,703,510	2,525,988
	Backcalculated Damaged $\|E^*\|$ (psi)	498,100	291,600	848,400
	d_{AC}	2.15	3.04	1.57
Level 2	FC_{bottom} (%)	48.4	3.7	5.4
	d_{AC}	0.07	0.03	0.02

Note: $|E^*|$ is dynamic modulus, d_{AC} is damage factor, and FC_{bottom} is amount of alligator cracking that initiates at the bottom of hot mix asphalt layers, %.

8.3. Damaged Mastercurve Comparison

Figure 23 (a), (b), and (c) present the resulting damaged mastercurves for the three core locations (A, B, and C), respectively. Figure 23 shows that the damaged mastercurves of Level 1 are shifted downwards from the original undamaged mastercurves. The amount of downward shift represents the severity of the distress in the existing pavement. Figure 23 also shows that the downward shift at Location B is the greatest among the three locations and is the least at Location C. This observation matches the level of severity of the debonding distress observed from the cores. Most cores from Location B (except the cores that are described in Chapter 7) already had debonded when they were extracted, whereas cores from Location C showed no visual signs of distress.

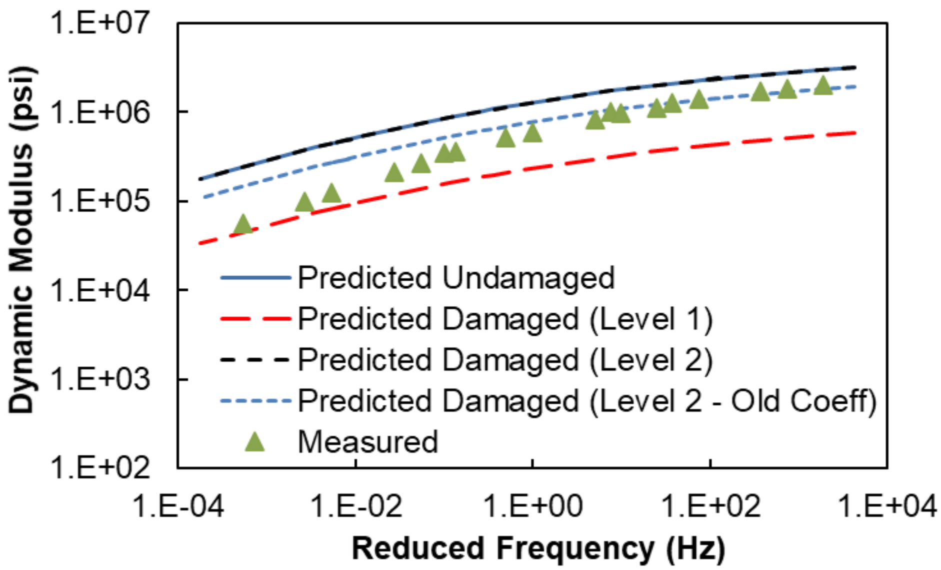
Another important observation from Figure 23 is that the damaged mastercurves generated by Level 2 analysis almost overlap with the undamaged mastercurves, indicating that the field modulus values are similar to the undamaged modulus values despite the fact that 48.4% of the alligator cracking that initiated at the bottom of the hot mix asphalt layers, FC_{bottom}, was detected at Location A. The reason for this overlap may be related to the current coefficients used in Equation (7) in AASHTOWare that were updated from the old coefficients. Originally, C_1 and C_2 were both 1.0 according to the Pavement ME Guide (AASHTO 2015). Hence, the research team tried the old coefficients in Equation (7) in order to determine the damaged mastercurve; the corresponding plots are shown in Figure 23 as well, labeled as 'Predicted Damaged (Level 2 – Old Coeff)'. As shown in Figure 23 (a), when the amount of alligator cracking is 48.4%, the predicted damaged mastercurve at Level 2 is lower than the mastercurve generated by Level 1.

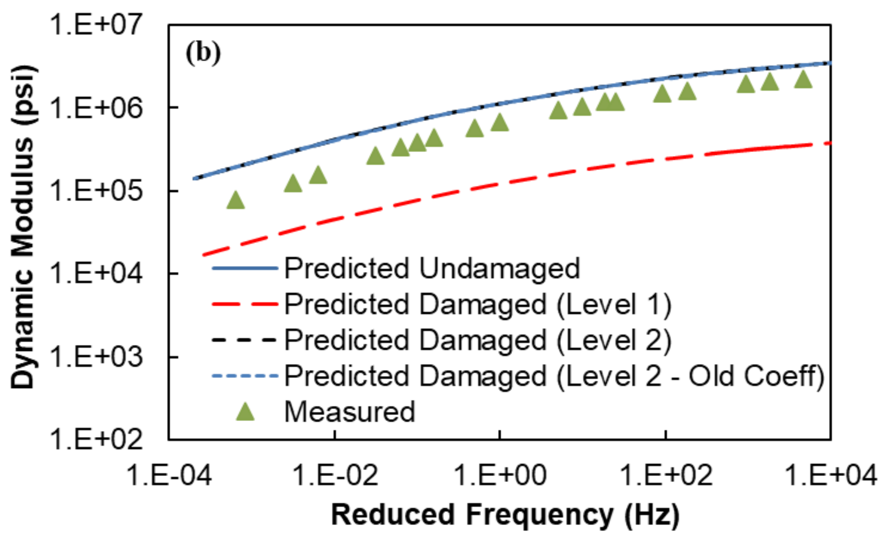
Another possible way to explain the unreasonable overlap is to calibrate Equation (7) to match the damage factor from Equation (6). However, this solution works only for Location A because the amount of alligator cracking (FC_{bottom}) at Locations B and C is extremely low. No matter whether calibrated coefficients are used or not, Equation (7) will not give the same damage factor (d_{AC}) values as Equation (6). In spite of the contradictory results obtained for Levels 1 and 2, Location A nonetheless can be used to develop a preliminary calibrated Equation (7) for North Carolina, where C_2 equals 1.11 and the other coefficients keep their original values. Different sets of Pavement ME coefficients are shown in Table 10, and NC 96 calibrated coefficients are recommended for NCDOT. In addition, the measured mastercurves for the cores lie lower than the undamaged mastercurve predicted from Witczak's predictive equation. This observation is reasonable for the field cores because Witczak's equation predicts the asphalt mixture's modulus value at the mixture's virgin state immediately after mixture fabrication. In contrast, the measured modulus values were obtained from field cores that had experienced field damage and had lost some integrity.

Table 10. Pavement ME Coefficients

Pavement ME Coefficients	C_1	C_2	C_1'	C_2'
Old Coefficients	1	1	$-2C_2$'	$-2.40874 - 39.748 \times (1+h_{AC})^{-2.856}$
Current Coefficients	1.31	5 for AC less than 5" 3.9666 for AC greater than 12" Otherwise, $C_2 = 0.867+0.2583\times h_{AC}$		
NC 96 Calibrated Coefficients	1.31	1.11		

Moreover, the measured mastercurve is higher than the Level 1 predicted damaged mastercurve at all three core locations, especially at high reduced frequencies that represent low-temperature behavior. Also, the measured mastercurve is steeper than the Level 1 predicted damaged mastercurve at all three core locations, indicating greater temperature and rate dependency. Three potential reasons for these observations are as follows. First, the method used to shift the undamaged mastercurve downward to construct the damaged mastercurve is questionable. That is, the shape of the mastercurve, which represents the temperature and rate dependence of the material, may be affected by damage. Second, although the properties of aged binder (extracted and recovered from field cores) were used in Witczak's equation to generate the undamaged mastercurve, the applicability of Witczak's equation, which was calibrated using dynamic modulus data of short-term aged mixtures, to aged mixtures has not been fully verified. Third, the downward shift used in Level 1 is based on FWD deflections that were affected by the presence of debonding in the pavement, whereas the measured mastercurve was developed based on cores without debonding or at least without enough debonding to separate the layers by manual force. Therefore, it makes sense that the Level 1 predicted damaged mastercurves lie below the measured mastercurves.


1.E+07
1.E+06
1.E+05
1.E+04
1.E+03
1.E+02
Dynamic Modulus (psi)
1.E-04
1.E-02
1.E+00
1.E+02
1.E+04
Reduced Frequency (Hz)
Predicted Undamaged
Predicted Damaged (Level 1)
Predicted Damaged (Level 2)
Predicted Damaged (Level 2 - Old Coeff)
Measured


(b)
1.E+07
1.E+06
1.E+05
1.E+04
1.E+03
1.E+02
Dynamic Modulus (psi)
1.E-04
1.E-02
1.E+00
1.E+02
1.E+04
Reduced Frequency (Hz)
Predicted Undamaged
Predicted Damaged (Level 1)
Predicted Damaged (Level 2)
Predicted Damaged (Level 2 - Old Coeff)
Measured

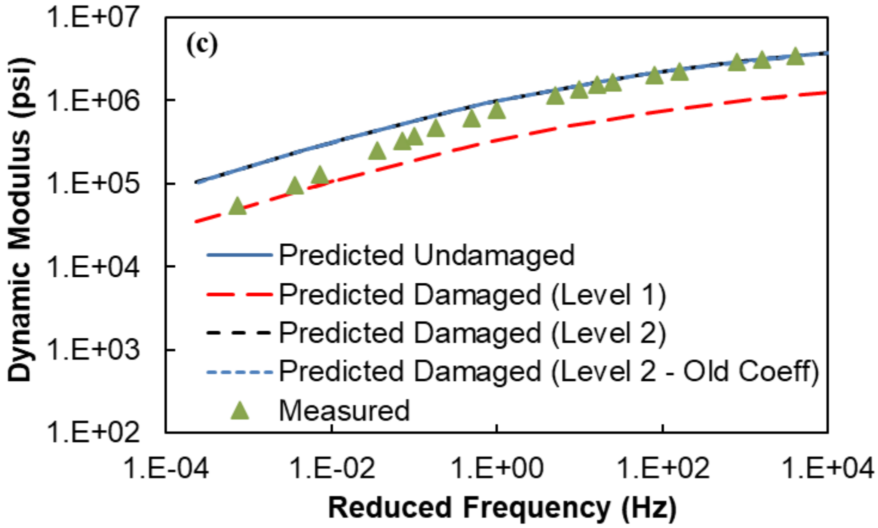


Figure 23. Undamaged and damaged predicted mastercurves and measured mastercurve: (a) at core Location A, (b) at core Location B, and (c) at core Location C.

Note that the current AASHTOWare Pavement ME Design software does not provide an option for inputting measured modulus values with respect to the properties of existing layers. Based on this limitation and the observations described here, the NCSU research team recommends to the NCDOT that, for now, the Level 1 Pavement ME method should be used to determine the dynamic modulus mastercurves for existing asphalt layers.

9. EVALUATION OF REPRESENTATIVE LAYERS

9.1. Damaged Mastercurve Comparison

As noted in Chapter 8, the Level 1 Pavement ME method is recommended to be used by the NCDOT. This section addresses the question as to which layer(s) should be selected in a multilayered existing pavement to determine the input properties for Witczak's equation. As described earlier, the selected field cores were investigated as three separate cases in this study: the top and bottom parts, and the total core. Each case was considered as the representative layer for all the AC layers. That is, the measured properties of each case were assumed to represent the properties of all the existing AC layers.

Figure 24, Figure 25, and Figure 26 respectively show the mastercurve results obtained for Locations A, B, and C. For both the predicted and measured results, most data points for the total core are located in the middle of the curves for the top and bottom parts. This method of presentation not only shows that the total core's behavior tends to be the combined results of the top and bottom parts, but also implies that the subjective choice of representative layers has the potential to result in different mastercurves. To determine the damaged mastercurves, the Level 1 and Level 2 procedures were applied to the undamaged mastercurves that were predicted from

Witczak's equation and the properties that were determined from the top part, bottom part, and total core, respectively. The results are shown in the last two plots, i.e., (c) and (d), of Figure 24, Figure 25, and Figure 26. Although the magnitude and vertical locations of the curves vary among the core locations, the difference in modulus values among the three sets of data seems not to be significant in logarithmic space. However, visual judgement may not necessarily reflect the true scenario. Whether or not this amount of difference would result in a large performance variation is evaluated later in this report.

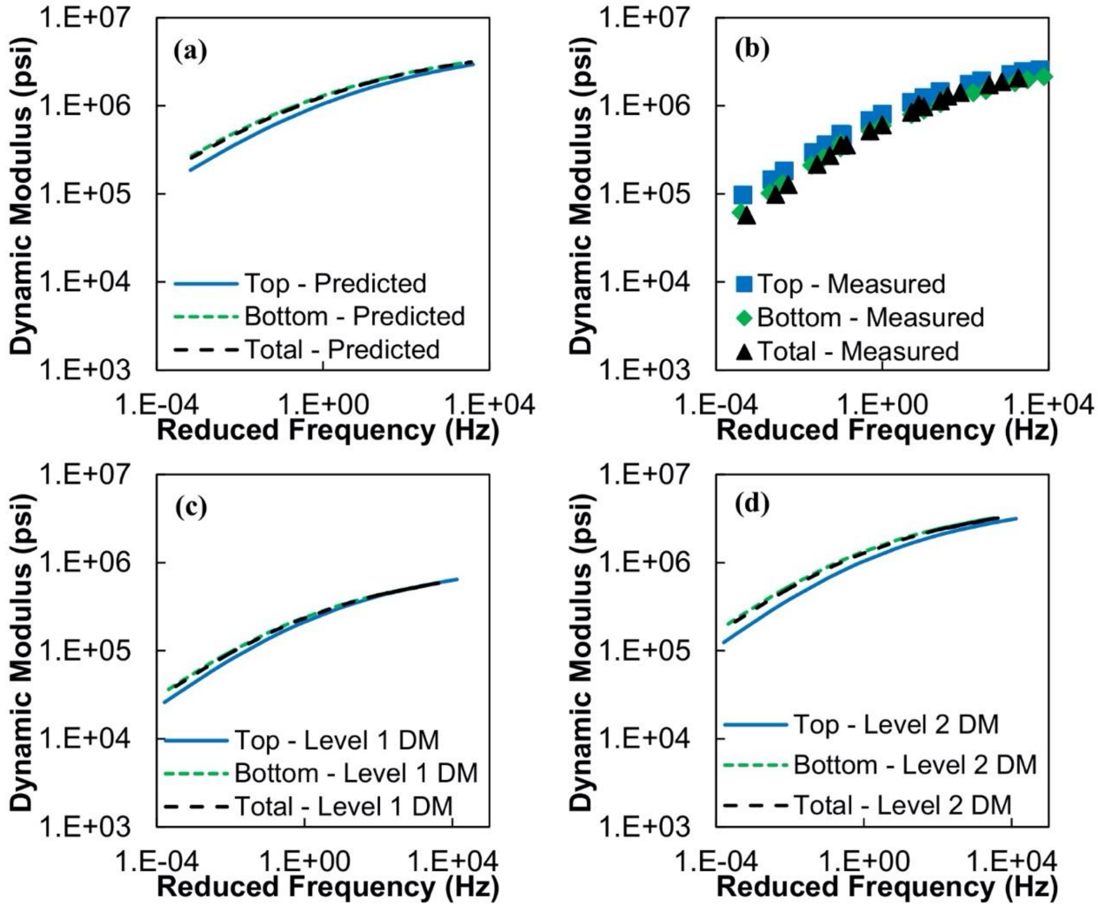


Figure 24. Mastercurves for core Location A: (a) predicted by Witczak's predictive equation, (b) measured, (c) damaged mastercurves at Level 1, and (d) damaged mastercurves at Level 2.

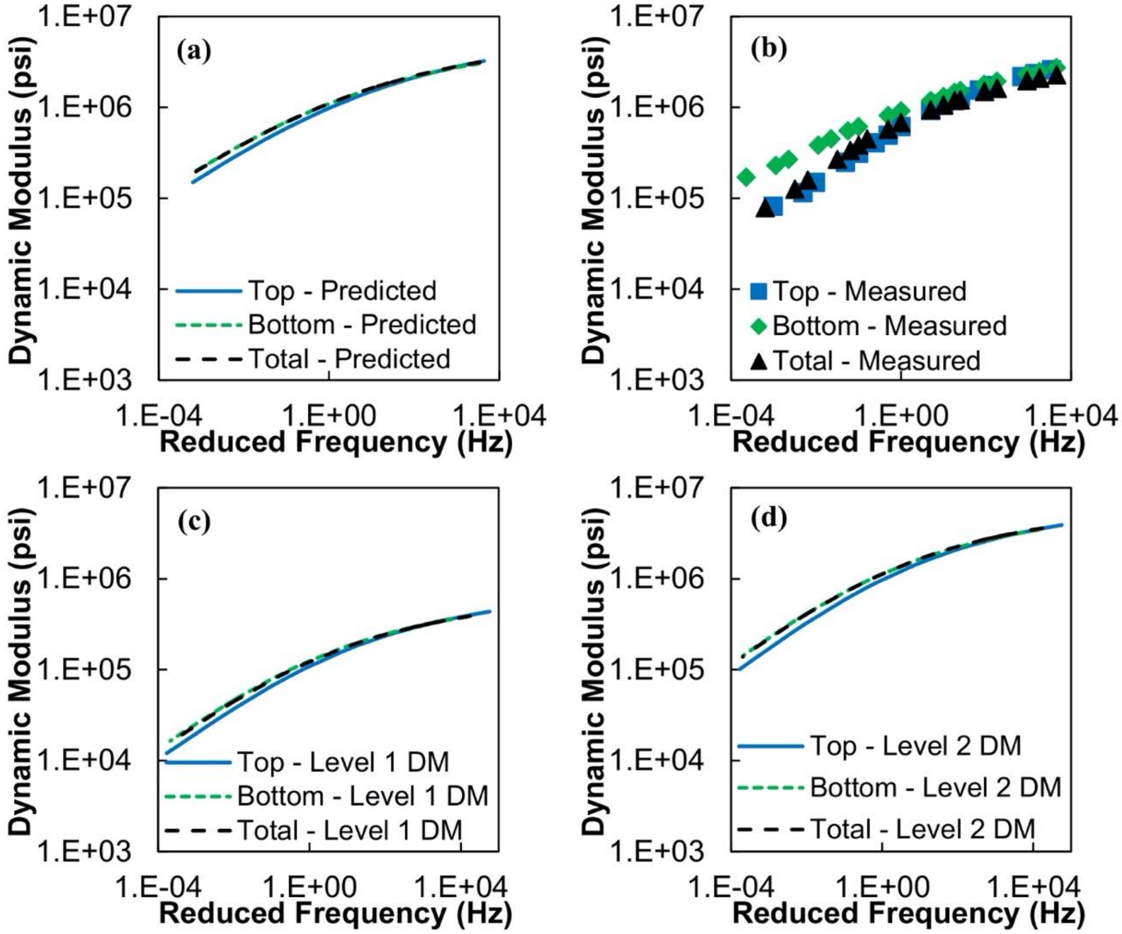


Figure 25. Mastercurves for core Location B: (a) predicted by Witczak's predictive equation, (b) measured, (c) damaged mastercurves at Level 1, and (d) damaged mastercurves at Level 2.

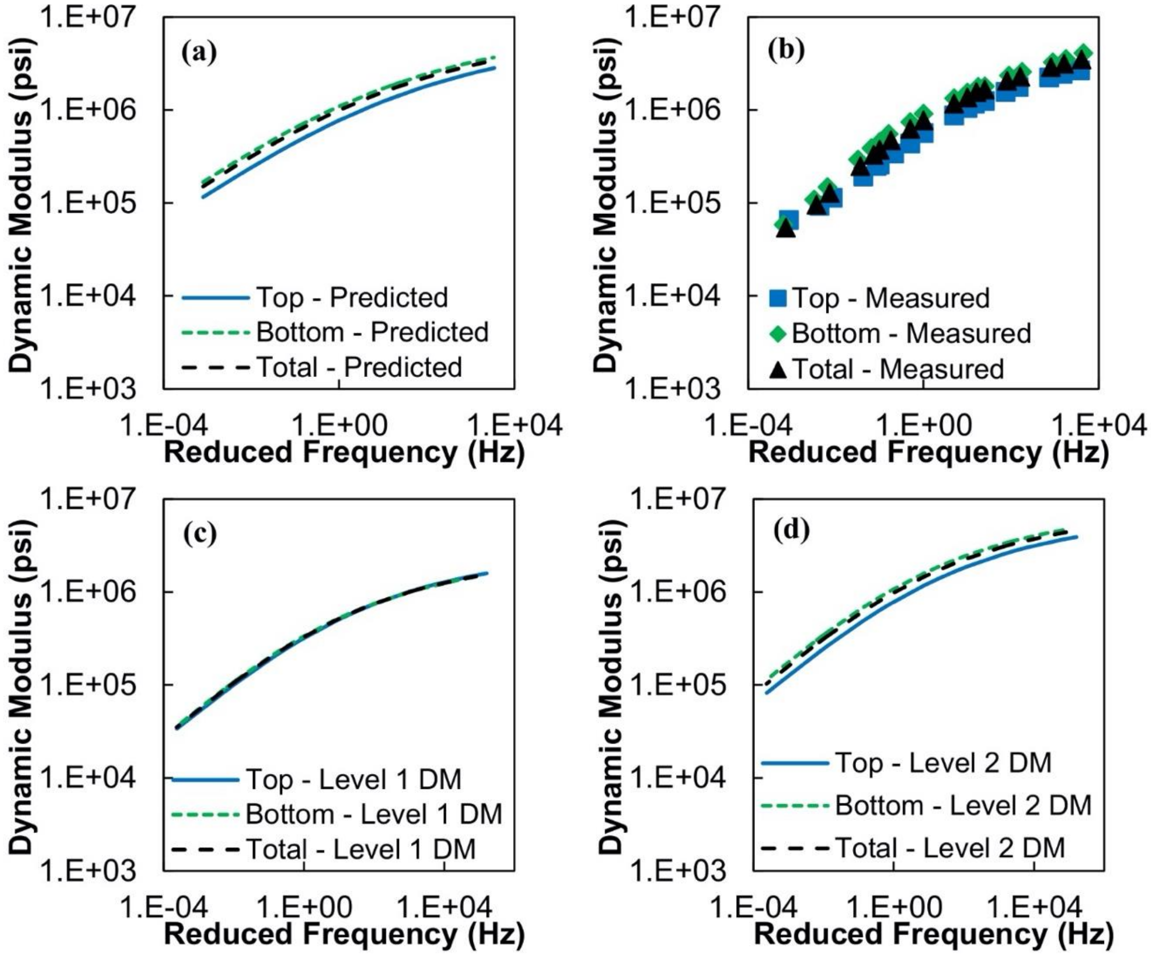


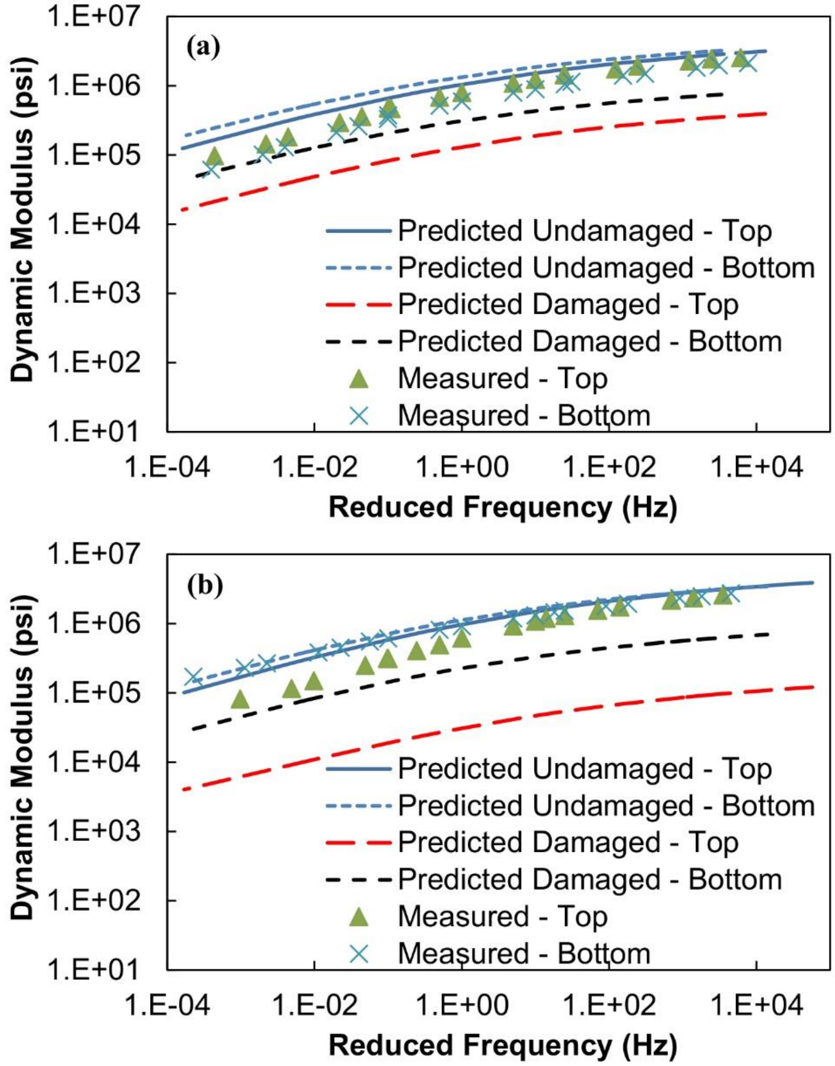
Figure 26. Mastercurves for core Location C: (a) predicted by Witczak's predictive equation, (b) measured, (c) damaged mastercurves at Level 1, and (d) damaged mastercurves at Level 2.

In all the cases presented in Figure 24, 23, and 24, the existing layers are regarded as one single layer and, therefore, only one *in situ* modulus value needs to be backcalculated for the AC layer. In fact, even though the accuracy may be compromised when the number of backcalculated layers is increased, the Pavement ME Guide does not prohibit the use of multiple AC layer backcalculations in the current pavement characterization protocol (AASHTO Pavement ME Design Task Force 2013). Hence, the research team tried another scenario in the analysis where the existing layers were divided into two sublayers (top and bottom) and backcalculations were conducted for each sublayer at Level 1. Table 11 presents the undamaged modulus values, damaged modulus values, and damage factor values at the FWD testing temperature and frequency. As this approach resulted in two sets of damaged mastercurves, a direct comparison with the previous scenarios was not feasible. However, upon close inspection, the backcalculated damaged modulus values indicate that the averaged values of the top layer and bottom layer for all three locations (A, B, and C) are 476,700 psi, 306,550 psi, and 885,200 psi, respectively, which are reasonably close to the values presented in Table 9 when the existing pavement is considered as a single layer.

Table 11. Damage Factors Resulting from Dividing Existing Layers into Two Sublayers

Core Location	A	B	C
Undamaged $\|E^*\|$ - Top Layer (psi)	2,444,663	2,613,167	2,083,527
Undamaged $\|E^*\|$ - Bottom Layer (psi)	2,793,981	2,645,637	2,730,193
Backcalculated Damaged $\|E^*\|$ - Top Layer (psi)	304,100	81,300	355,900
Backcalculated Damaged $\|E^*\|$ - Bottom Layer (psi)	649,300	531,800	1,414,500
d_{AC} - Top Layer	2.82	5.62	2.38
d_{AC} - Bottom Layer	1.99	2.17	1.11

Figure 27 (a), (b), and (c) present the results for the scenario in which the damage factors that correspond to the separate sublayers are used to develop the damaged mastercurves for Locations A, B, and C, respectively. Compared to the cases shown in Figure 24, 23, and 24, the magnitudes of the damaged mastercurves for the top and bottom layers change because the vertical location of the damaged mastercurves is dominated by the FWD backcalculated modulus values. Also, now the mastercurves for the laboratory measurements are not necessarily located between the undamaged curves and damaged curves. For example, Figure 27 (b) shows that the measured mastercurve for the bottom layer almost overlaps with the predicted undamaged curve at low reduced frequencies, and Figure 27 (c) shows that the measured curve is lower than the predicted damaged curve at low reduced frequencies. If Level 2 or 3 is used, then only one damage factor value can be determined using the cracking condition survey and Equation (7), which is the same value as that shown in Table 9. As such, using Level 2 or 3 will increase the uncertainties and errors of damage characterization because both the top and bottom sublayers need to use the same damage factor in order to calculate the damaged mastercurve. Regardless of other disadvantages, this feature should be considered carefully before dividing an existing pavement into sublayers.


(a)
Dynamic Modulus (psi)
Reduced Frequency (Hz)
Predicted Undamaged - Top
Predicted Undamaged - Bottom
Predicted Damaged - Top
Predicted Damaged - Bottom
Measured - Top
Measured - Bottom
(b)
Dynamic Modulus (psi)
Reduced Frequency (Hz)
Predicted Undamaged - Top
Predicted Undamaged - Bottom
Predicted Damaged - Top
Predicted Damaged - Bottom
Measured - Top
Measured - Bottom

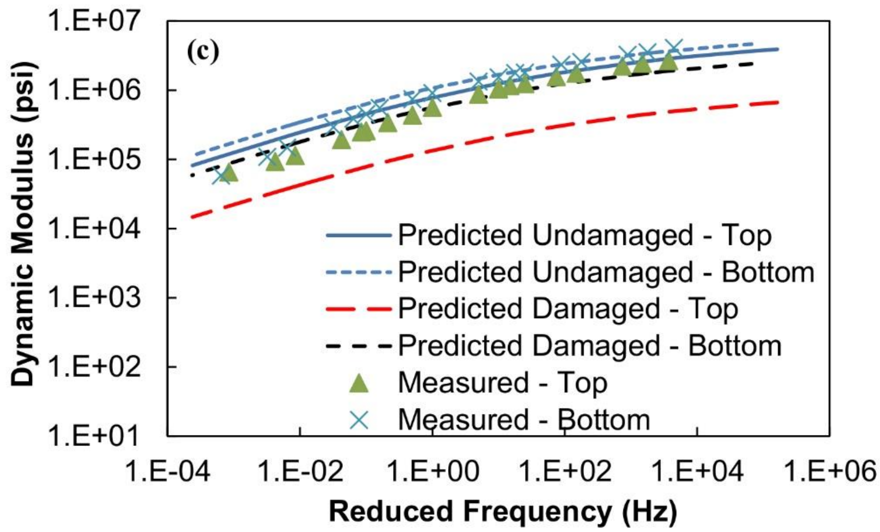


Figure 27. Mastercurves obtained by dividing existing layers into two sublayers: (a) at core Location A, (b) at core Location B, and (c) at core Location C.

9.2. Pavement Performance Predictions

This section presents four ways to determine the damaged mastercurve. Table 12 provides a summary of these four methods in terms of the layer(s) ('Case') used to determine the material properties for the undamaged mastercurve generation and the backcalculation schemes (single layer vs. two separate layers).

Table 12. Four Ways to Determine Damaged Mastercurve

Case	Layer Used for Undamaged Mastercurve	Backcalculation Scheme
Top Layer	Top Layer	All AC Layers in a Single Layer
Bottom Layer	Bottom Layer	All AC Layers in a Single Layer
Total Core	Total Core	All AC Layers in a Single Layer
Top + Bottom	Each of Top and Bottom Layers	Top and Bottom Layers Separated

In order to see the differences from another perspective other than modulus magnitude or curve shape and location, the NCSU research team decided to predict the pavement performance using AASHTOWare Pavement ME Design software. For this simulation, the annual average daily truck traffic (AADTT) was obtained from the NCDOT's historical records and the nearest station to NC 96 was selected as the climate station. Default values were used for other required information. Table 13 presents details of the Pavement ME software inputs. Note that Table 13 presents the critical performance results obtained by using different representative layers at Level 1. The results for Level 2 and Level 3 are not shown in the table because the predicted amount of distress is zero in Level 2 and mostly zero (except for low numbers) for Location A in Level 3.

The reason for the low distress prediction in Level 2 is that the damaged mastercurve is essentially the same as the undamaged mastercurve (see Figure 23). For Level 3, the condition ratings for Locations A, B, and C are poor, excellent, and good, respectively, according to the AASHTO Guide (1993). The resulting amount of distress derived from the Pavement ME simulations for the overlay design shown in Table 13 is nearly zero.

Table 13 presents three sets of Pavement ME outputs. Note that other performance outputs (e.g., permanent deformation, thermal cracking) are not shown here because they either indicated ‘good’ condition all the time or they are not the main focus of this study. The results from Locations A and C indicate that dividing the existing pavement into two sublayers (denoted as ‘Top + Bottom’ in Table 13) could lead to predictions of much worse cracking. As for the other three representative layer choices (top, bottom, and core), their cracking performance varies considerably at Location A but is relatively stable at Locations B and C. At Location B, 100% cracking is predicted for all four cases, which is probably due to the fact that the backcalculated *in situ* damaged modulus value at Location B is much lower than at Locations A and C, as shown in Table 9. Such a low modulus value makes the corresponding damaged mastercurve exhibit relatively low magnitude, which facilitates crack propagation. Thus, no matter which layer is selected as the representative layer, the reflective cracking resistance is predicted to be very poor. In terms of pavement life, the variation among the different representative layer choices is relatively small. In fact, reflective cracking dominates the pavement life in all cases. From a practical point of view, because pavement life does not vary much with respect to the choice of representative layer, using the total core to characterize the existing pavement is recommended. Based on the experience of the NCSU research team, the tests required for Witczak’s predictive equation consume a substantial amount of materials, particularly to determine the gradations and aggregate specific gravity. Therefore, using the total core to obtain the Witczak’s predictive equation coefficients would save time and effort.

Table 13. Inputs and Outputs for Pavement Performance Simulation

	Core Location		A	B	C
Input	Design Life (years)		20		
	Two-Way AADTT		1,560		
	Number of Lanes		2		
	Trucks in Design Direction (%)		50		
	Trucks in Design Lane (%)		100		
	Operational Speed (mph)		40		
	Traffic Growth Rate (%)		3		
	Traffic Growth Function		Compound		
	Climate Station		US.NC (139555)		
	Overlay Thickness (in.)		3		
	Overlay Dynamic Modulus (psi)		Default Values at Level 2		
	Milled Thickness (in.)		2		
	Measured Existing Layer Thickness[a] (in.)		8.46 = 2.08 + 6.38	8.66 = 2.11 + 6.55	12.01 = 3.07 + 8.94
	Measured Existing Dynamic Modulus (psi)		Measured Inputs for Levels 1 and 2 Default Values for Level 3		
	DCP Measured Subgrade Modulus (psi)		38,983	35,280	26,164
Critical Performance Output at Level 1 (%)	FC_{bottom}	Top Layer	48.6	100	0.16
		Bottom Layer	1.5	100	0.03
		Total Core	3.1	100	0.05
		Top + Bottom	95.2	100	99.1
	FC_{bottom} + FC_{ref}[b]	Top Layer	100	100	61.74
		Bottom Layer	76.1	100	65.03
		Total Core	78.7	100	63.99
		Top + Bottom	100	100	100
	Pavement Life	Top Layer	6.0	1.3	7.5
		Bottom Layer	5.8	1.8	6.9
		Total Core	5.9	1.4	7.0
		Top + Bottom	5.6	1.2	6.0

[a]The format of the thickness input is L = M + N. If the existing pavement is regarded as one layer, the thickness input is L. If the existing pavement is divided into two sublayers, the inputs are M and N for each of them.
[b]FC_{ref} is the amount of reflective cracking (% lane area).

10. DYNAMIC MODULUS MASTERCURVE BACKCALCULATION

The NCSU research team tried one more option to backcalculate dynamic modulus mastercurves directly from FWD test data. To achieve this goal, the team developed a highly efficient forward model by combining various traditional and recent computational techniques. Specifically, to obtain the surface response of the pavement under FWD loading, the research team used a modal expansion technique for the axisymmetric system of discretized layers in the frequency domain (unlike Hankel transform that is common in many existing models). Such modeling was facilitated with the help of the so-called 'complex-length finite element method' that reduced the number of elements required to discretize the mesh in the vertical direction. The computational cost was reduced further with the help of Padé interpolation across the frequency spectrum of the load. In the end, the accurate surface response of a pavement under typical FWD loading could be computed within less than 0.2 second on quad-core processors that are common in standard laptop and desktop computers.

Although the main efficiency of the inversion approach is inherited from the forward model, the number of forward solutions nonetheless was reduced while ensuring the robustness and convergence of the backcalculation. The approach involved combining gradient-based optimization with Monte Carlo-type global optimization techniques. By carefully comparing the various approaches with respect to cost and performance, the research team determined that a finite difference computation of the gradient was the most desirable approach and would work well in a Gauss-Newton optimization framework. To help with convergence towards the global minimum, the gradient iteration was embedded into a global multi-start optimization framework based on a Monte Carlo approach. This method, coupled with other techniques such as nonlinear transformation of the parameter space, led to the reduction of the number of function evaluations and eventually the overall computational cost. This inversion approach takes 1 to 25 minutes, most of the time at the lower end of the spectrum.

However, FWD tests involve low-frequency excitation, where the load is significant below 50 Hz to 100 Hz typically. Similarly, the response is also significant below this frequency. Given that the measurement time is limited to 0.06 seconds, the lower limit of the frequency typically is 5 Hz to 10 Hz. Therefore, for a given pavement structure and known excitation, the response can be reliably predicted only within this narrow frequency range. A natural consequence is that, even in the absence of non-uniqueness issues, the pavement modulus can be backcalculated reliably only for this frequency range. Because this range constitutes only a narrow portion of the entire mastercurve, the research team concluded that the mastercurve cannot be backcalculated solely from FWD data.

11. CONCLUSIONS

Based on the field tests and laboratory analysis of field cores extracted from NC 96, the NCSU research team evaluated the accuracy and efficiency of the three analysis levels used in the current Pavement ME Guide with regard to the determination of damaged mastercurves of existing AC layers for rehabilitation design. In addition to comparing the three analysis levels, the research team also investigated various ways that highway agencies conduct tests that involve multilayered pavement structure systems. The team investigated four cases in terms of undamaged mastercurve characterization and damaged mastercurve determination, and the corresponding pavement performance predictions.

With respect to the three levels of Pavement ME rehabilitation analysis, the following conclusions can be drawn from this study.

- For Level 1 applications, FWD measurements and backcalculations are needed to determine the *in situ* modulus values. However, the backcalculated modulus values are related to how the user inputs the structural information into the backcalculation program. Based on the evaluation results, fixing a certain layer modulus value during backcalculation is not recommended. Instead, the user should adjust the pavement structure (i.e., divide the subgrade into two sublayers) to give the program enough leeway to determine the field modulus values. The RMSE of the output should be lower than 5 percent.

- Comparing the backcalculated elastic modulus values with the laboratory-measured modulus values, and assuming that the elastic modulus equals the dynamic modulus at 30 Hz, the laboratory-determined modulus value is much higher than the backcalculated modulus value. The research team believes that this trend is due to the fact that the FWD measurements could have been influenced by debonding or cracking observed from the field cores, which was not reflected by the laboratory-measured dynamic modulus values.

- The undamaged mastercurve that is predicted using Witczak's predictive equation has a different shape and magnitude than the mastercurve that is based on measured field cores. Because the shape of the curve is linked to viscoelasticity and the damaged mastercurve is merely the vertical shift of the undamaged mastercurve, the current protocol in Pavement ME based on Witczak's predictive equation could result in erroneous damaged mastercurves.

- The results from Level 1, 2, and 3 analyses differ significantly in terms of the damage factor estimations and cracking predictions. Level 1 is recommended as the first choice for agencies to use in practice. However, if Level 2 or 3 needs to be applied, then a transfer function that relates the damage factor to the percentage of bottom-up cracking needs to be calibrated first to ensure that the results are consistent with those of Level 1. Based on the data used in this project, the calibrated new C_2 equals 1.11 but the other coefficients keep their original values for the transfer function (Equation (7)). However, more North Carolina field data are needed to confirm the reasonableness of the calibrated new C_2 value.

With respect to multilayered pavement characterization, the following conclusions can be drawn from this study.

- When the tests that are specified by Levels 1 and 2 need to be conducted using multilayered AC pavements, the total core should be used for laboratory evaluation, not the thickest layer. In this project, different representative layers (the top layer of the core, bottom layer(s) of the core, and total core) were selected for the Pavement ME method analysis. The results show that most dynamic modulus data points from the total core are located in the middle of the curves for the top and bottom parts of the core, indicating that the total core's behavior tends to be the combined results of the top and bottom parts.

- Dividing the existing pavement into multiple layers is possible only for Level 1 analysis. Even so, this approach is not recommended because it requires multilayer backcalculations and considerable time and resources to characterize the individual layer materials. However, in terms of pavement life, the variation among the different representative layer choices is relatively small. In fact, reflective cracking dominates the pavement life in all cases. Hence, using the total core is the most effective way to characterize existing AC layers.

With regard to dynamic modulus mastercurve backcalculation, the conclusion drawn by the research team is that the mastercurve cannot be backcalculated solely from FWD data due to the limited frequency range that the FWD can capture. Therefore, to determine the damaged mastercurve, some other means, such as FWD testing at multiple times of a day or multiple seasons or laboratory measurements, need to be employed to obtain more information outside the FWD frequency range. Because the NCDOT and other state highway agencies have already adopted the Pavement ME Guide, instead of developing a completely new protocol, the NCSU research team recommends that the NCDOT use the modified Pavement ME method based on NC 96 data.

12. REFERENCES

AASHTO. 1993. *Guide for Design of Pavement Structures*. American Association of State Highway and Transportation Officials (AASHTO).

AASHTO Pavement ME Design Task Force. 2013. *Material and Design Inputs for Pavement Rehabilitation with Asphalt Overlays*. Presented at AASHTO Pavement ME Design Webinar.

AASHTO. 2015. TP 79-15, *Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester*. American Association of State Highway and Transportation Officials.

AASHTO. 2015. PP 61-13, *Developing Dynamic Modulus Master Curves for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)*. American Association of State Highway and Transportation Officials.

AASHTO. 2015. T 342-11, *Determining Dynamic Modulus of Hot Mix Asphalt*. American Association of State Highway and Transportation Officials.

AASHTO. 2015. R 62-13, *Developing Dynamic Modulus Master Curves for Asphalt Mixtures*. American Association of State Highway and Transportation Officials.

AASHTO. 2015. *Mechanistic-Empirical Pavement Design Guide: A Manual of Practice*. American Association of State Highway and Transportation Officials.

AASHTO. 2016. T 209-12, *Theoretical Maximum Specific Gravity (G_{mm}) and Density of Hot Mix Asphalt (HMA)*. American Association of State Highway and Transportation Officials.

AASHTO Pavement ME Design Task Force. 2017. *FWD Deflection Data Analysis and Backcalculation Tool: BcT*. Presented at AASHTO Pavement ME Design Webinar.

AASHTO Pavement ME Design Task Force. 2017. *User Manual for Pavement M-E Deflection Data Analysis and Backcalculation Tool*. American Association of State Highway and Transportation Officials.

ARA, Inc., ERES Consultants Division. 2004. *Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. Part 2. Design Inputs*. Chapter 2. Material Characterization. National Cooperative Highway Research Program, 2004.

Ayyala, D., H. S. Lee, and H. L. Von Quintus. 2017. *Damage Characterization in Existing Asphalt Layer Concrete for ME Rehabilitation Design*. Presented at 96th Annual Meeting of the Transportation Research Board, Washington, D.C.

Ayyala, D., H. S. Lee, and H. L. Von Quintus. 2018. *Characterizing Existing Asphalt Concrete Layer Damage for Mechanistic Pavement Rehabilitation Design*. Publication FHWA-HRT-17-059. FHWA, U.S. Department of Transportation.

Castorena, C., Y. R. Kim, S. Pape, and K. Lee. 2017. *Development of Small Specimen Geometry for Asphalt Mixture Performance Testing*. National Cooperative Highway Research Program Innovations Deserving Exploratory Analysis Project 181.

Christopher, B.R., Schwartz, C.W. and Boudreau, R.L., 2010. *Geotechnical aspects of pavements: Reference manual*. US Department of Transportation, Federal Highway Administration.

Corley-Lay, J., F. M. Jadoun, J. N. Mastin, and Y. R. Kim. 2010. Comparison of flexible pavement distresses monitored by North Carolina Department of Transportation and Long-Term Pavement Performance program. *Transportation Research Record: Journal of the Transportation Research Board*, 2153(1): 91-96.

Habbouche, J., E. Y. Hajj, P. E. Sebaaly, and N. E. Morian. 2018. Damage assessment for ME rehabilitation design of modified asphalt pavements: Challenges and findings. *Transportation Research Record: Journal of the Transportation Research Board*, 2672(40): 228-241.

Harsini, I., W. C. Brink, S. W. Haider, K. Chatti, N. Buch, G. Y. Baladi, and E. Kutay. 2013. Sensitivity of input variables for flexible pavement rehabilitation strategies in the MEPDG. *Airfield and Highway Pavement 2013: Sustainable and Efficient Pavements*: 539-550.

Kim, Y. R., Y. Seo, M. King, and M. Momen. 2004. Dynamic modulus testing of asphalt concrete in indirect tension mode. *Transportation Research Record: Journal of the Transportation Research Board*, 1891(1): 163-173.

Lee, H.S., Ayyala, D. and Von Quintus, H., 2017. Dynamic backcalculation of viscoelastic asphalt properties and master curve construction. *Transportation Research Record*, *2641*(1), pp.29-38.

Lee, K., S. Pape, C. Castorena, and Y.R. Kim. 2017. Evaluation of Small Specimen Geometries for Asphalt Mixture Performance Testing and Pavement Performance Prediction. Journal of the Transportation Research Board, 2631: 74-82.

Loulizi, A., G. W. Flintsch, and K. McGhee. 2007. Determination of in-place hot-mix asphalt layer modulus for rehabilitation projects by a mechanistic–empirical procedure. *Transportation Research Record: Journal of the Transportation Research Board*, 2037(1): 53-62.

Lukanen, E. O., R. Stubstad, R. C. Briggs, and B. Intertec. 2000. *Temperature Predictions and Adjustment Factors for Asphalt Pavement*. Publication FHWA-RD-98-085. FHWA, U.S. Department of Transportation.

Pape, S., K. Lee, C. Castorena, and Y. R. Kim. 2018. Optimization of the laboratory fabrication of small specimens for asphalt mixture performance testing. *Transportation Research Record: Journal of the Transportation Research Board*, 2672(28): 438-450. https://doi.org/10.1177/0361198118790845.

Park, H.J. and Y.R. Kim. 2004. Primary Causes of Cracking of Asphalt Pavement in North Carolina: Field Study. International Journal of Pavement Engineering, 16(8): 684-698.

Park, H., M. Eslamania, and Y.R. Kim. 2014. Mechanistic Evaluation of Cracking in In-Service Asphalt Pavements. Materials and Structures, 47(8): 1339-1358.

Witczak, M.W. 2004. *Laboratory determination of resilient modulus for flexible pavement design* (No. 285). National Cooperative Highway Research Program Project No. 1-28A.