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DEVELOPMENT OF MODULUS-TO-TEMPERATURE RELATIONS FOR HMA MIXTURES USED IN WISCONSIN

FINAL REPORT

by

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16. Abstract

This report presents the findings of a research study conducted to develop modulus-to-temperature relations for HMA mixtures used in Wisconsin. Surface deflection data gathered from in-place HMA pavements was used to estimate the resilient modulus of the HMA layer at the various test temperatures. Laboratory resilient modulus testing was also conducted on recovered HMA cores to establish trends of HMA resilient modulus as a function of test temperature and load frequency. Prediction equations for estimating modulus-to-temperature trends were developed from laboratory testing to account for mixture-specific parameters, including fines content, air voids and binder content.

Estimations of HMA modulus-to-temperature trends based on solely on nondestructive deflection data were found to correlate with laboratory trends for some, but not all of the projects tested. Observed estimation errors were deemed due to variations in the thickness and/or stiffness of lower pavement layers which were not measured during field testing.

A simple process for developing site-specific variations in HMA resilient modulus in response to monthly air and pavement temperature changes is presented. These monthly variations represent valuable inputs for mechanistic-empirical performance analysis.

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EXECUTIVE SUMMARY

Project Summary

This research study consists of developing modulus-to-temperature relations for Hot Mix asphalt (HMA) mixtures currently utilized in Wisconsin. Surface deflection data gathered from in-place HMA pavements was utilized to estimate the resilient modulus of the HMA layer at various test temperatures. Laboratory resilient modulus testing was also completed to establish trends of HMA resilient modulus as a function of test temperature and load frequency. Correlations between resilient modulus values estimated from field data and measured from laboratory testing were developed. Correlations between HMA mixture properties and resilient modulus were also developed based on the results of laboratory testing.

Project Background

The Wisconsin Department of Transportation currently utilizes HMA pavement thickness design procedures which are based on the 1972 Interim Guide for the Design of Pavement Structures. The Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavements Structures, recently produced under NCHRP Project 1-37A, represents the next-generation in pavement design procedures, incorporating mechanistic-empirical (M-E) pavement models to predict pavement damage as a function of specific traffic, materials and environmental inputs. It is envisioned that the new M-E design guide will soon be released as a provisional or interim future edition of the AASHTO Design Guide and will be adopted for use by the Wisconsin Department of Transportation.

In order to take full advantage of these new design procedures, detailed information on the material properties of component pavement layers will be required. In the context of HMA pavement design, the in-place modulus of the HMA pavement layer, and its seasonal variation due to environmental effects, represents one of the most critical input parameters. The standard HMA layer coefficients used within the current Structural Number design concept, will be replaced with specific assignable material properties based on inputs of HMA mix type, aggregate structure, binder specifications, compacted mixture volumetrics, and HMA mix temperature. Predictive equations have been developed to estimate the dynamic modulus of the HMA layer as a function of these inputs, but these models will need to be validated for the specific mixture types used in Wisconsin. At the time of this study, laboratory test equipment and procedures for determining the dynamic modulus of HMA mixtures were still in the development stage. As such, it was deemed reasonable to utilize standardized equipment and procedures for measuring the resilient modulus of HMA mixtures as a surrogate for the dynamic modulus. Developed resilient modulus-to-temperature relations would then be available as comparative measures for their dynamic modulus counterparts which are currently being developed under a separate WHRP Project conducted by Michigan Technological University.

Process

Literature was reviewed from various national sources detailing the best practices for conducting nondestructive testing on HMA pavements and utilizing this data to estimate the resilient modulus of component HMA layers. Literature relating to the conduct of laboratory resilient modulus testing was also reviewed to develop test protocol beneficial to the study objectives. Field testing was conducted on selected HMA pavement in Wisconsin to develop a database of nondestructive test data and to recover cores of in-place HMA materials for laboratory testing. Statistical analyses of all collected data were conducted to develop predictive equations for estimating the modulus-to-temperature trends for the sampled HMA mixtures based on key mixture properties.

Findings

The analyses of collected field and laboratory data produced the following study findings:

(1) The developed regression equations are appropriate for predicting HMA moduli versus temperature variations which closely match values obtained from laboratory testing at load frequencies of 1 Hz and 2 Hz. Better agreement is generally obtained using the refined models as compared to the general models.

(2) For the mixtures analyzed, modulus-to-temperature relations were developed using inputs for key mixture properties including fines content (P_{200}), air voids (V_v) and binder content (P_b). In general, the HMA resilient modulus increases at any temperature as the P_{200} increases, the percentage of binder decreases, and/or the percent voids decreases.

(3) Estimation of HMA resilient modulus from deflection data can provide reasonable estimates of laboratory measurements; however, variations in the thickness and/or stiffness of lower pavement layers may significantly affect analysis results. Care must be exercised when using deflection data as the sole source of information for any given project.

(4) Monthly HMA moduli can be readily predicted based on prevailing air temperatures and

HMA layer thickness using the developed regression equations which are based on readily obtainable mixture data. These monthly moduli values represent valuable inputs for mechanistic performance analyses.

Recommendations

For the purposes of mechanistic performance analyses using HMA resilient modulus values, it is recommended that mixture-specific prediction equations which account for mixture type (E-type) and nominal maximum aggregate size (NMAS) be utilized to develop modulus-to-temperature relations. These equations provide better agreement to laboratory test values for all mixtures investigated during this project and are based on readily obtainable project data for fines content (P_{200}), air voids (V_v), and binder content (P_b). The projects included in the model development were weighted towards mixtures using PG 58-28 binders (116 of 143 samples), and thus it is recommended that more study be conducted on mixtures using other PG binder grades to confirm the appropriateness of the general model to predict the modulus-to-temperature trends for these mixtures. It is also recommended that site or regional weather data be used during project analysis to better define the monthly modulus-to-temperature variations.

The resilient modulus relations developed during this project can serve as surrogate values prior to the development of dynamic modulus relations which are required for input to the proposed AASHTO mechanistic-empirical design procedures.

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CHAPTER ONE
INTRODUCTION

1.1 Background and Problem Statement

The Wisconsin Department of Transportation (WisDOT) currently utilizes hot mix asphalt (HMA) pavement thickness design procedures which are based on the 1972 Interim Guide for the Design of Pavement Structures. A number of revisions have been made to the interim guide, with subsequent publications of the 1986 and 1993 AASHTO Guides for Design of Pavement Structures. While these updated guides contained refinements on design reliability and material input parameters, both were still fundamentally based on empirical equations derived from the original AASHO Road Tests. The Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavements Structures, recently produced under NCHRP Project 1-37A, represents the next-generation in pavement design procedures, incorporating mechanistic-empirical (M-E) pavement models to predict pavement damage as a function of specific traffic, materials and environmental inputs. While these new procedures and companion software are still being evaluated under NCHRP Project 1-40, it is envisioned that the new M-E design guide will ultimately be released as a provisional or interim future edition of the AASHTO Design Guide.

In order to take full advantage of these new design procedures, detailed information on the material properties of component pavement layers will be required. In the context of HMA pavement design, the in-place modulus of the HMA pavement layer, and its seasonal variation due to environmental effects, represents one of the most critical input parameters. The standard HMA layer coefficients used within the current Structural Number design concept, will be replaced with specific assignable material properties based on inputs of HMA mix type, aggregate structure, binder specifications, compacted mixture volumetrics, and HMA mix temperature. Predictive equations have been developed to estimate the dynamic modulus of the HMA layer as a function of these inputs, but these models will need to be validated for the specific mixture types used in Wisconsin. It is further anticipated that model adjustment factors will need to be developed to allow for adequate characterization of these mixtures throughout the pavement's service life.

The primary objective of this research is the development of specific modulus-to-temperature relations for HMA mixtures currently used in Wisconsin. At the time of this study, laboratory test equipment and procedures for determining the dynamic modulus of HMA mixtures were still in the development stage. As such, it was deemed reasonable to utilize standardized equipment and procedures for measuring the resilient modulus of HMA mixtures as a surrogate for the dynamic modulus. Developed resilient modulus-to-temperature relations would then be available as comparative measures for their dynamic modulus counterparts which are currently being developed under a separate WHRP Project conducted by Michigan Technological University.

1.2 Literature Review

This section provides a review of published literature related to the new or revised techniques for estimating/backcalculating the resilient modulus of in-place HMA layers based on collected nondestructive test data. Particular emphasis has been placed on test methods, equipment, data and/or analysis techniques which have shown good correlations to the traditionally reported resilient modulus developed from laboratory testing per ASTM D4123 guidelines. This literature review provides an overview of the following topics related to asphalt pavement testing and backcalculation:

- Nondestructive deflection testing devices;
- Practices for conducting deflection testing on asphalt highway pavements;
- Materials sampling and testing;

- Backcalculation of asphalt concrete moduli from deflection data; and
- Other nondestructive testing methods.

1.2.1 Nondestructive Deflection Testing Devices

Nondestructive deflection testing is routinely conducted on asphalt pavements for the purposes of backcalculating the stiffnesses of the subgrade and pavement layers, assessing the remaining life of the pavement, and/or determining the overlay thickness required to satisfy a structural deficiency. Nondestructive deflection testing (NDT) devices are classified based on the type of loading method used; namely static, vibratory, or impulse. Falling weight deflectometers (FWDs), which are impulse loading devices, represent the most common device type in current use. Overviews of the different devices available for nondestructive deflection testing are provided in NCHRP Report *Rehabilitation Strategies for Highway Pavements* by Hall, et al (1), *Pavement Management for Airports, Roads, and Parking Lots* by Shahin (2), the FHWA report *Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements* (3), the FHWA report *Evaluation of Pavement Deflection Measurement Equipment* (4), and the National Highway Institute's *Techniques for Pavement Rehabilitation Manual* (5)

FWDs are available in a variety of configurations which allow for test loads ranging from 1,500 lbs (7 kN) to 53,000 lbs (240 kN). Lighter load devices, producing maximum loads of approximately 22,000 lbs (100 kN) are best suited for testing HMA highway pavements. Testing on thicker rigid and composite pavements requires heavyweight deflectometers, which are capable of applying loads up to 53,000 lbs.

General guidelines for deflection testing are given in ASTM D4695, *Standard Guide for General Pavement Deflection Measurements,* ASTM D4694, *Standard Test Method for Deflections* *with a Falling-Weight-Type Impulse Load Device,* 1993 AASHTO Guide (6), the *Techniques for Pavement Rehabilitation Manual* (5) and by Shahin (2) and Hall (7).

1.2.2 Deflection Testing Practices on Asphalt Highway Pavements

Deflection testing on multi-lane highway pavements is usually limited to the outer traffic lane to provide data where the majority of truck traffic and load associated distress typically occurs. Outer lanes closures, required for deflection testing, are also considered safer than inner lane closures due to the perception that users are more accustomed to, and better able to respond to closures in the outer lane. Testing of HMA highway pavements may be conducted in the outer wheel path for the purpose of attempting to assess the extent of fatigue damage and in the middle of the outer traffic lane, between the two wheel paths, for the purpose of estimating the pavement layer and foundation moduli while minimizing the effect of traffic loadings.

Asphalt mix temperature measurements should be obtained during HMA pavement testing to better account for changes in HMA stiffness due to temperature variations. Mid-depth HMA temperatures are preferred over surface temperatures. If it is not possible to obtain mix temperature measurements, the mix temperature may be estimated from air and surface temperatures, using procedures developed by Southgate (8), Shell (9), the Asphalt Institute (10), Hoffman and Thompson (11), or Lukanen, et al (12).

It is recommended that at least two, and often three target load levels be used during deflection testing to allow for analysis of stress-dependent base and foundation materials. Based on the authors' experience, a target load of sufficient magnitude to produce a mean maximum deflection of 6 mils (1 mil = 0,001 inch) is needed to obtain deflection basins of sufficient curvature to lend themselves to successful backcalculation. For highway pavements, at least one of the target load levels should be 9000 pounds, to facilitate an analysis of the pavement's structural capacity using the 1993 AASHTO Guide method (6). Typical target load levels for highway pavements are 6000, 9000, and 12000 pounds.

Different sensor configurations yield different backcalculation results. Specifically, two configuration issues that significantly influence the magnitudes of the modulus values obtained from backcalculation are the outer radius to which the deflection basin is measured, and whether or not the maximum deflection (d0 measured at the center of the load plate) is used in the backcalculation. Most FWDs have at least six deflection transducers in addition to the transducer in the middle of the load plate. All seven deflection transducers should be used, and positioned to include at least one measurement beyond 36 inches. For highway pavements, a sensor configuration of 0, 8, 12, 18, 24, 36, and 60 inches is commonly used (1).

1.2.3 Materials Sampling and Testing

The material types and layer thicknesses should be determined from inventory records before deflection testing is conducted. Coring to check layer thicknesses may be done before deflection testing, but it is preferable to do coring after deflection testing, so that any unanticipated changes in the deflection magnitudes can be investigated by coring. It is not usually feasible to conduct coring and routine deflection testing simultaneously as the coring operation may not be able to keep pace with deflection testing. Coring during deflection testing may, however, be beneficial when considering traffic control requirements.

Precise layer thicknesses are important when using deflection data to backcalculate layer moduli. If there are no plans to perform laboratory testing on the pavement layer materials and only the layer thicknesses are needed, a small-diameter (e.g., half-inch) drill bit may be used to determine asphalt surface and stabilized base layer thicknesses. It is also useful to note from observations of cores whether or not the layers are bonded together. However, layers that come out unbonded in the core may not have been unbonded in place; sometimes pavement layers are separated by torsion during the coring operation. Examination of the interface may indicate whether the layer samples were separated during coring or had been unbonded for some time.

Diametral resilient modulus testing, following ASTM D4123 *Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures,* may be conducted on cores from asphalt concrete and asphalt-treated base layers. Typically, tests are conducted at three temperatures (e.g., 41, 77, and 104 ± 2°F), each at one or more loading frequencies (e.g., 0.33, 1, 2, and 10 Hz). Core diameters of 4 or 6 inches may be used based on the maximum aggregate size. The average resilient modulus is typically reported for each temperature, load duration, and load frequency used.

The dynamic modulus of the asphalt concrete mix may be estimated using Equation 1.1 (13). This equation was developed by Witczak and is a refinement of work originally done for the Asphalt Institute by Kallas and Shook (10). Equation 1.1 is considered reliable for new, unaged dense-graded asphalt concrete mixes with gravel or crushed stone aggregates. For aged in-service mixes, it is recommended that the results of diametral resilient modulus testing on cores be established at two or more temperatures and used to calibrate Equation 1 for a particular mix being evaluated.

$$\begin{aligned}\log E_{ac} = {} & 5.553833 + 0.028829\,(P_{200} / F^{0.17033}) - 0.03476\,V_v \\ & + 0.070377\,\eta_{70°F,10^6} + 0.000005\,t_p^{(1.3 + 0.49825 \log F)}\,P_{ac}^{0.5} \\ & - (0.00189 / F^{1.1})\,t_p^{(1.3 + 0.49825 \log F)}\,P_{ac}^{0.5} \\ & + 0.931757\,(1 / F^{0.02774})\end{aligned} \qquad \text{Eqn 1.1}$$

where:

Eac	=	dynamic modulus of the asphalt concrete, psi
P_{200}	=	percent of aggregate passing the No. 200 sieve
F	=	loading frequency, Hz
Vv	=	air voids, percent
$\eta_{70°F,10^6}$	=	absolute viscosity at 70°F, 10^6 poise
P_{ac}	=	asphalt content, percent by weight of mix
t_p	=	asphalt concrete mix temperature, °F

FWD testing is typically conducted with a load pulse duration of 25-30 msec. In order to draw valid comparisons between resilient moduli backcalculated from FWD test data with results of laboratory resilient modulus testing, adjustments for loading frequency must be made. A laboratory resilient modulus testing load frequency of F = 1.0 Hz represents a load pulse duration of 100 msec followed by a 900 msec rest period. It may therefore be concluded that an FWD load pulse of 25 msec duration, which is four times faster than the aforementioned load duration, would be comparable to a laboratory loading frequency of 4 Hz.

Eqn 1.1 may also be used to calculate the expected modular ratio of backcalculated to laboratory values based on the applicable loading frequencies of each test and known or assumed asphalt concrete mix parameters (P_{200}, Vv, η, P_{ac}, t_p). A sensitivity analysis of modular ratios computed with Eqn 1.1 indicates that for any assumed HMA mix temperature, computed modular ratios are independent of air voids and viscosity and dependent on frequency ratio, binder content, and fines content. Figures 1.2.1 and 1.2.2 illustrate computed modular ratios for a range of HMA mix temperatures using binder contents of 5.0% and 6.0%, fines contents of 2%, 5%, and 10%, and frequency ratios (FWD/Lab) of 4Hz/2Hz and 40Hz/2Hz. These values were selected to represent the range of typical values expected for HMA mixtures in Wisconsin. As shown in these figures, for either selected frequency ratio, computed modular ratios increase as fines content decrease and/or binder content increases. It can also be seen that the selected frequency ratio significantly affects the magnitude of computed modular ratios, especially for elevated HMA mix temperatures. It is also important to note that the specific FWD and Lab frequencies used in the analysis, rather than the numerical ratio of these values, results in unique computed modular ratios, In other words, the computed modular ratios for FWD/Lab frequencies of to 40Hz/2Hz are not the same as those computed for a frequency of 20Hz/1Hz even though the numerical ration of each is 20.

1.2.4 Backcalculation of Asphalt Concrete Moduli From Deflection Data

Backcalculation of pavement layer moduli from deflection data is typically accomplished with the aid of computerized pavement analysis programs which are based on multi-layer elastic theory. The in situ elastic modulus of pavement and foundation layers are established as those which would produce the closest match between measured (i.e., FWD) and theoretical surface deflections. A few backcalculation programs exist that use the equivalent thickness concept, i.e., reduction of a multilayer elastic system to an equivalent system of fewer layers for which a solution is more easily obtainable.

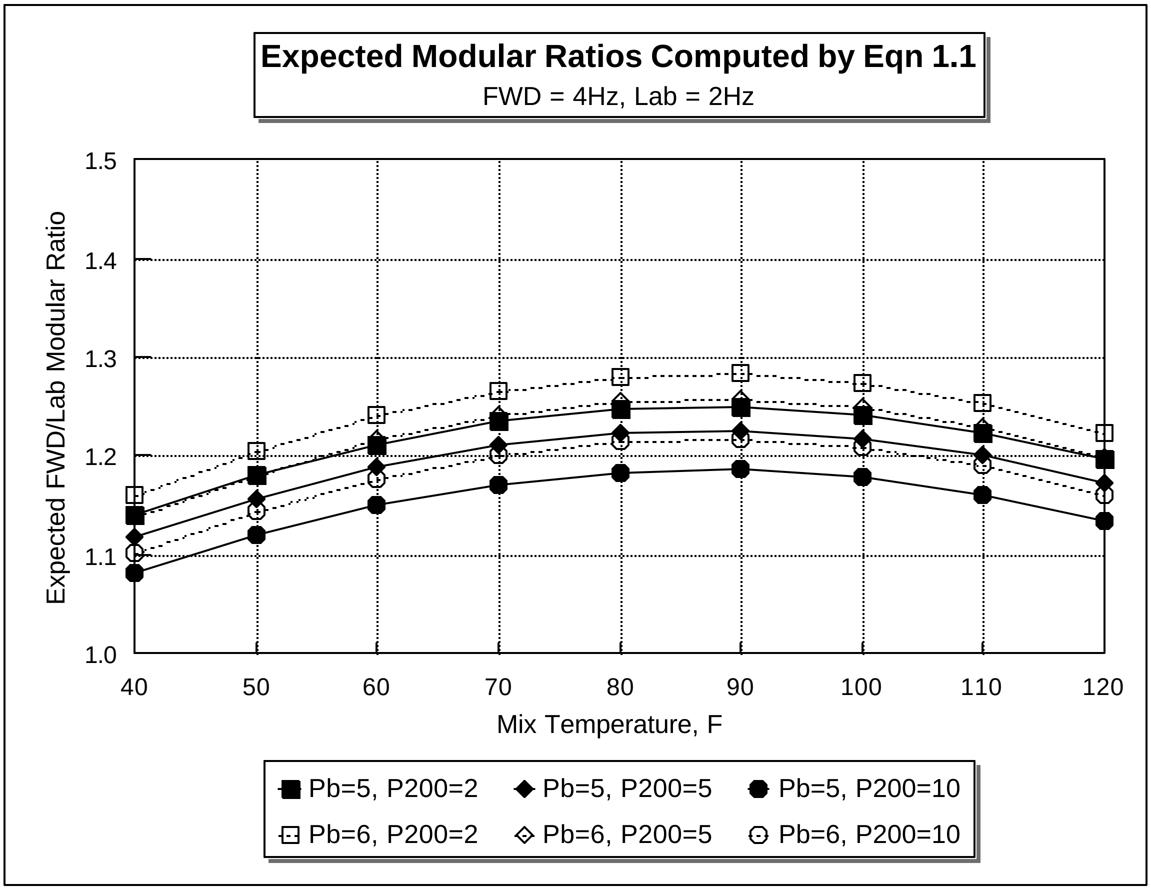

Figure 1.2.1: Computed Modular Ratios vs HMA Mix Temperature

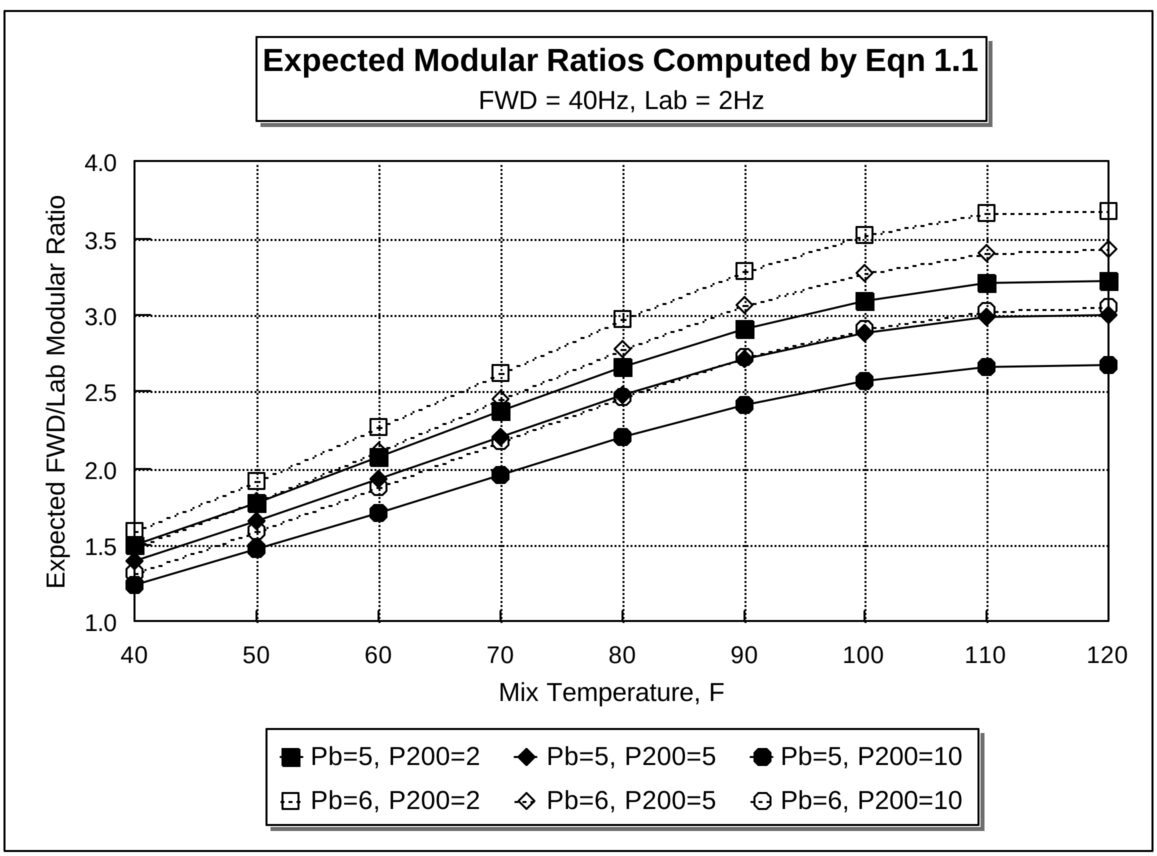

Figure 1.2.2: Computed Modular Ratios vs HMA Mix Temperature

Backcalculation programs based on multi-layer elastic theory utilize iterative numerical integration or database search alogorithms to arrive at a convergence solution. Backcalculation by the equivalent thickness method may also be done by iteration or by database search. A number of algorithms have also been developed to estimate asphalt layer moduli based on computed deflection terms. These algorithms were developed using both finite element and multi-layer pavement analysis programs and may be used in spreadsheet format to analyze large data sets.

BISDEF (14), CHEVDEF (15), MODCOMP (16), and WESDEF (17), are examples of iterative backcalculation programs, which make repeated calls to an elastic layer analysis subroutine (BISAR (18) for BISDEF, CHEVRON (19) for CHEVDEF and MODCOMP, and WESLEA (20) for WESDEF) to generate theoretical surface deflections which are matched to measured deflections. The iteration process stops when the measured and theoretical deflections match within user-defined tolerances. A detailed description of the solution algorithm used in this class of programs is given by Anderson (21). The robustness of the solution is dependent upon user-entered starting values ("seed moduli") and allowable ranges for the pavement layer and foundation moduli. There is no unique solution to the set of moduli that will produce a given deflection basin. Rather, there are in theory as many solutions as there are layers in the pavement structure, although not all of them may be realistic (22, 23, 24). Success with iterative backcalculation programs requires not only a good knowledge of pavements and materials but also experience in the backcalculation for the specific pavement type in question. Some researchers have suggested that iterative elastic layer backcalculation can never be truly automated except in conjunction with an expert system to guide decisions such as selection of seed moduli (22, 25).

In general, iterative elastic layer backcalculation programs produce the most reasonable results when the surface layer is the stiffest and the elastic modulus of each underlying layer is significantly less than that of the layer above. Poor results may be expected when the base layer is stiffer than the surface layer (e.g., asphalt concrete over cement-treated base) and/or the AC surface layer is fairly thin (e.g., less than 3 inches). In such situations, iterative programs tend to greatly under predict the modulus of the AC surface and consequently over predict the modulus of the layer(s) below. Obtaining reasonable results from iterative backcalculation programs when analyzing thin AC layers over stiffer base layers usually requires restricting the AC modulus to a very narrow range based on the AC mix temperature at the time of testing (1).

BOUSDEF (26) is an iterative backcalculation program based on the equivalent thickness concept. Deflections for trial layer moduli combinations are computed by an equivalent thickness subroutine which dramatically reduces execution time. However, the accuracy of BOUSDEF's solutions may be limited by some assumptions which are inherent to the equivalent thickness concept. MODULUS (27) is a widely used database backcalculation program which utilizes the CHEVRON program to generate surface deflection based on user-defined variables. MODULUS first generates a deflection basin database from a defined factorial of elastic layer modulus values and layer thickness combinations. This database is then searched to identify the deflection basin that most closely matches the field-measured deflection basin. Database backcalculation programs run much more quickly than iterative programs, but require more computer storage. MODULUS was originally developed for analysis of flexible pavements, but has since been shown to be suitable for analysis of other pavement types as well. Another database backcalculation program, COMDEF (28), developed by Anderson, is applicable only to asphalt-overlaid concrete pavements.

The backcalculation programs described herein are all used in a static, linear elastic analysis of peak pavement deflections. A static, stress-dependent, analysis of pavement deflections may also be completed using peak surface deflections from applied loads of varying magnitudes, such as can be produced by the FWD (29). Such an analysis is best suited to a finite element model, however recent advancements in the KENLAYER computer program (30) does allow for the analysis of stress dependent base and subgrade materials. The use of artificial neural networks to address the computational complexity of dynamic deflection analysis has been explored to a limited extent (31,32).

Dynamic analysis of FWD data, assuming either linear or nonlinear response, is substantially more computation-intensive than static analysis, and thus remains at present largely confined to the realm of research. In a 1994 paper, Uzan provided an overview of the concepts and techniques associated with linear and nonlinear dynamic deflection analysis (33). In addition to the computational challenges, there may be a practical reason for many practitioners' lack of motivation to pursue dynamic deflection analysis: such techniques involve characterizing soils and unbound granular materials by stress-strain models that are more sophisticated than the simple elastic modulus-Poisson's-ratio characterization of a linearly elastic material. However, the applications in which backcalculated material properties are routinely used, i.e., pavement and overlay thickness and design procedures, are still based almost exclusively on performance models based on linear elastic modulus characterizations of unbound materials.

In 1990, the Strategic Highway Research Program (SHRP) conducted a review of the features and capabilities of most of the then-available backcalculation programs, for the purpose of recommending which program or programs should be used in analysis of deflection data collected in the Long-Term Pavement Performance (LTPP) studies (34). The programs reviewed and some of their key features are summarized in Table 1.1. On the basis of this review, SHRP recommended the MODULUS program for LTPP deflection data analysis (35). However, subsequent efforts to obtain a modified version of MODULUS suitable for analysis of large volumes of data were unsuccessful, and LTPP later endorsed MODCOMP as its backcalculation program of choice. More than ten years later, MODULUS, MODCOMP, and EVERCALC appear to be the most widely used publicly available backcalculation programs for asphalt pavement deflection data analysis. Notwithstanding the computational efficiency offered by the database search approach, use of MODULUS remains hampered by the antiquated DOS interface of the public-domain version and its difficulty of use with large data sets. BISDEF and WESDEF are also widely used with small data sets, but use of both for large data sets is hampered by the excess of text in the output files, and use of BISDEF is further hampered by the proprietary nature of the elastic layer analysis subroutine BISAR.

Thompson developed various deflection-based algorithms for flexible pavement analysis utilizing the Area Under the Pavement Profile (AUPP) concept. One algorithm, which is easily applied in spreadsheet format, relates the overall flexural rigidity of the HMA layer (ET^3) to AUPP. When HMA layer thickness is known, the elastic modulus of the HMA layer can be estimated. To test the viability of algorithms of this type, a factorial analysis of conventional asphalt pavement structures was conducted using the KENLAYER computer program. The results of this analysis and applications to deflection data collected during this research are described in Chapter 2.

Table 1.1 Comparison of Backcalculation Programs (from SHRP, 1990 (37))

Program	Developer	Forward Calculation*	Forward Subroutine	Back-Calculation	Nonlinear Analysis	Rigid Layer Analysis
BISDEF	Bush	ELT	BISAR	Iterative	no	fixed at 20ft
BOUSDEF	Zhou	MET	MET	Iterative	yes	yes
CHEVDEF	Bush	ELT	CHEVRON	Iterative	no	fixed at 20ft
COMDEF	Anderson	ELT	DELTA	Database	no	no
DBCONPAS	Tia	FEM	FEACONS III	Database	yes (?)	Unknown
ELMOD	Ullidtz	MET	MET	Iterative	yes	yes
ELSDEF	Texas A&M	ELT	ELSY5	Iterative	no	fixed at 20ft
EMOD	PCS/LAW	ELT	CHEVRON	Iterative	yes	no
EVERCALC	Mahoney	ELT	CHEVRON	Iterative	yes	yes
FPDE01	Uddin	ELT	BASINF7	Iterative	yes	yes
ISSEM4	Ullidtz	ELT	ELSYM5	Iterative	yes	no
MODCOMP	Irwin	ELT	CHEVRON	Iterative	yes	yes
MODULUS	Uzan	ELT	WESLEA	Database	yes	yes
PADAL	Brown	FEM	Unknown	Iterative	yes	Unknown
RPEDD1	Uddin	ELT	BASINR7	Iterative	yes	yes
WESDEF	USACE	ELT	WESLEA	Iterative	no	yes

* ELT = multi-layer elastic theory, MET = method of equivalent thickness, FEM = finite element method

1.2.5 Other Nondestructive Testing Methods

Ground-penetrating radar (GPR) may be used to estimate pavement layer thicknesses, joint deterioration, moisture contents in base layers, and stripping in asphalt concrete layers. Vehicles equipped with ground-penetrating radar equipment operate at speeds from 3 to 70 mph, with travel speed dependent on the antenna type (air launched or ground coupled) and the frequency at which data are required. For routine project-level pavement surveys data can be collected at posted highway speeds. Ground-penetrating radar testing is described in ASTM D4748, *Standard Test Method for Determining the Thickness of Bound Pavement Layers Using Short-Pulse Radar*.

The major advantages of ground-penetrating radar testing are its speed and accuracy. It continues to be the only technology which can provide meaningful subsurface information at close to highway speed. Its disadvantages include the complexity of the radar output and the lack of good software to convert the signals into information meaningful to pavement engineers. Current data analysis methods are labor intensive and require considerable expertise for interpretation of the raw data. Some coring is required with ground-penetrating radar for calibration purposes (36). Nonetheless, information obtained from ground-penetrating radar on layer thickness variability can be used in deflection analysis to obtain better pavement layer modulus estimates than might be obtained with the limited thickness variability information that can be obtained from a few cores (37).

Infrared thermography is used to locate reinforcing steel and detect concrete delaminations in reinforced concrete pavements. Infrared thermography has also been used to detect debonding at asphalt/concrete interfaces, and to measure temperature differentials in newly placed asphalt overlays. Various types of infrared scanners have been used to detect both delamination and debonding. Often, real-image video recording equipment is mounted together with the scanner to record surface defects such as potholes and patches which may otherwise be interpreted incorrectly when viewed on the infrared output. The infrared scanning equipment can be van mounted and operated at speeds of 15 mph.

The major advantages of infrared thermography are its speed and accuracy, relative to destructive methods such as coring, for subsurface data collection. Disadvantages of infrared thermography include its sensitivity to non-pavement-related conditions such as time of day and recent weather conditions. Also, the two-dimensional output cannot indicate the depth of the distressed area. Perhaps the greatest practical disadvantage of infrared, however, is the complexity of the infrared outputs and video images.

Wave propagation is a technique for monitoring the dispersion (change in velocity with frequency or wavelength) of surface waves in a pavement, to predict pavement condition. In a layered system, the dispersion of surface waves is indicative of the relative stiffnesses of distinct layers. The technique for characterizing layer stiffnesses and thicknesses from surface wave data is known as spectral analysis of surface waves (SASW).

Surface waves may be produced using vibratory devices, strike hammers, or drop weight devices, including the FWD (38). The SASW technique involves using a series of progressively larger blows to produce waves of increasing wavelength, which tend to propagate through the deeper layers of a pavement. The waves generated in the pavement, and their dispersions, are monitored by two transducers acting as receivers. The data are collected by a spectral signal analyzer and passed to a computer for processing. The wave velocities can be transformed into representations of modulus versus depth.

Spectral analysis of surface waves has been applied to asphalt, concrete, and asphalt-overlaid concrete pavements, over both fine-grained and coarse-grained subgrades. SASW analysis results have been shown to compare well with backcalculation results from deflection analysis. An advantage of SASW over deflection-based backcalculation is that it is capable of predicting pavement layer moduli without advance knowledge of layer thicknesses or material types (39). Both layer moduli and thicknesses can in theory be determined from SASW output, although in practice the accuracy of the modulus estimates are improved when thickness data from boring logs are used. SASW can be used to determine the depth to bedrock, information that can improve the accuracy of backcalculation from deflection measurements. A disadvantage of SASW is the difficulty and time involved in data collection and interpretation. Some work has been done in developing artificial neural networks to aid in the interpretation of SASW data (40, 41). More automated data acquisition and processing methods are needed to make this testing technique more practical (36).

1.3 Research Objectives

The objectives of this research are summarized as follows:

1. Determine the influence of load level and mix temperature on the deflection response for selected HMA pavements constructed in Wisconsin according to the Superpave criteria.
2. Evaluate the collected field deflections to estimate backcalculated HMA layer moduli as a function of mix temperature.
3. Evaluate the effects of temperature and load frequency on the resilient modulus determined from laboratory testing.
4. Develop modulus-to-temperature relations for examined HMA mixtures that may be suitable for use in a mechanistic analysis of pavement performance.
5. Develop key prediction variables that may be used during the mix design process to estimate HMA layer moduli.

1.4 Research Plan

The research plan was divided into six major work tasks which are described as follows:

Task 1: Literature Review

A literature review was conducted to document the available techniques for estimating/backcalculating the resilient modulus of in-place HMA layers based on collected nondestructive test data. The results of this task are summarized in Section 1.2 of this report. Based on this literature review, the most promising techniques were selected for implementation during the deflection data analysis completed during Task 5.

Task 2: Development of Research Factorial and Data Collection Plan

In this task the research team, in collaboration with Wisconsin DOT, established a main research factorial which included six primary target cells representing all possible combinations of two targeted Superpave mixture types (E1 and E3) and three levels of coarse aggregate type/location (poor quality sedimentary deposits of western Wisconsin, good quality sedimentary and glacial deposits of southeastern Wisconsin, and good quality igneous deposits of northern Wisconsin). The field data collection plan was developed to include nondestructive deflection testing, utilizing the KUAB 2-mass falling weight deflectometer (2m-FWD) owned and operated by the WisDOT, and pavement surface coring completed by the Geotechnical Unit of WisDOT.

Task 3: Identify Projects for Field Testing

In this task, projects were selected for inclusion into this study based on a review of candidate project lists provided by District personnel. The projects include Superpave mixtures with different nominal maximum aggregate size, gradations, aggregate sources, and ESAL design levels. Additional tests projects, which fall outside of the targeted E1 and E3 mix types, were added to extend the range of applicability of the research results.

Task 4: Conduct Field and Laboratory Studies

In the field, deflection testing was conducted for an extended period on a selected test day to obtain representative results over a range of pavement temperatures. Testing days were selected based on considerations of both equipment availability and environmental conditions to provide for a wide a range of pavement test temperatures. HMA layer cores were transported to Michigan Technological University (MTU) for resilient modulus testing over a range of temperatures and load frequencies. The relationship between mix temperature and asphalt layer modulus (backcalculated from deflections and measured in the lab) were determined from the results obtained in this task.

Task 5: Analyze Data and Prepare Guidelines

Collected deflection data were used to backcalculate representative in-place HMA layer moduli over the range of mix temperatures tested. These results were used in conjunction with laboratory test results to establish modulus-to-temperature relations for each mix type tested. Guidelines for the use of these relationships in pavement design were developed.

Task 6: Prepare and Submit Final Report

This final report was written to include work conducted in Tasks 1 to 5 of this research study. It also includes guidelines for incorporating representative asphalt layer moduli into the mechanistic pavement design process.

1.5 Summary

This report is organized into five chapters. Chapter 1 includes the background, problem statement, literature review, research objectives and research plan. Chapter 2 includes a review of the field study and the developed project matrix. Chapter 3 provides analysis results of the collected field deflection and temperature data. The effects of different field test variables on the resulting backcalculated HMA layer moduli are provided. Chapter 4 includes the results of the laboratory study. Relationships between laboratory test variables and measured HMA resilient moduli values are provided. Chapter 5 includes a summary of the findings, conclusions from this study, and recommended guidelines for inclusion of these results into mechanistic pavement designs.

CHAPTER 2
FIELD STUDY

2.1 Introduction

The field study was designed to collect pavement deflection data and core specimens from in-service HMA pavements which are composed on typical E-type mixtures used in Wisconsin. Deflection testing was targeted to obtain data over as wide a range of mix temperatures as possible. Deflection data gathering was limited to a single day of testing for each selected project. Full-depth HMA cores were also obtained at three locations within each test project. The main project variables included Superpave mixture type and predominant coarse aggregate source. Project specific variables included nominal maximum aggregate size (NMAS), aggregate gradation, and layer position.

Table 2.1.1 provides a listing of project variables for each pavement section incorporated into this study. This information was obtained from mix design reports provided by WisDOT personnel as well as from verified test reports downloaded from the WisDOT Materials Tracking System website maintained by Atwood Systems. NMAS and gradation classifications indicated in Table 2.1.1 were determined from the reported job mix formula gradations. Aggregate gradations were classified as "fine" or "coarse" based on the percentage of material passing the 2.36 mm sieve (P_8) as follows:

25 mm NMAS - Fine = $P_8 > 25\%$, Coarse = $P_8 \leq 25\%$

19 mm NMAS - Fine = $P_8 > 30\%$, Coarse = $P_8 \leq 30\%$

12.5 mm NMAS - Fine = $P_8 > 35\%$, Coarse = $P_8 \leq 35\%$

9.5 mm NMAS - Fine = $P_8 > 40\%$, Coarse = $P_8 \leq 40\%$

Table 2.1.1 - Project Variables

Project ID	Site Location	Mix Type	Layer Position	Binder Type	NMAS	Coarse Aggregate	Gradation
2748-01-70	STH 164 - Slinger	E-1 12.5mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	9.5 mm 12.5 mm	Gravel Gravel	Fine Fine
1420-09-70	South Frontage Road - Waupun	E-1 12.5mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine
2370-07-70	STH 164 - Waukesha	E-10 12.5mm E-10 19mm	Upper Lower	PG 70-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine
1032-08-70	IH 90/94 - Kenosha	E-30x 12.5mm E-30x 25mm	Upper Lower	PG 76-28 PG 70-22	12.5 mm 25 mm	Limestone Limestone	Fine Coarse
5111-06-71	STH 131 - Ontario	E-1 12.5mm	Upper	PG 58-28	12.5 mm	Limestone	Fine
5181-07-71	STH 60 - Boscobel	E-1 12.5 mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine
1593-05-70	USH 8 - Rhinelander	E-3 12.5mm E-3 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Gravel Gravel	Fine Fine

2.2 Equipment and Methods

Field data collection for each project site included the following major activities: (1) coordination of traffic control, FWD equipment/personnel and coring equipment/personnel, (2) repeated deflection testing, (3) temperature measurements throughout the depth of the HMA pavement layers, and (4) pavement coring. Field data collection was completed during a single day of testing for all projects except the South Frontage Road in Waupun. Field testing on this project was completed on two separate days (10/20/2003 and 07/26/2004) due to seasonal paving of the lower and upper HMA layers.

2.2.1 Project Coordination

Projects were selected from all candidate sections provided by WisDOT District personnel. Only candidate projects where pavement coring would be allowed were considered. Prior to field testing, site visits were conducted by Marquette research staff to validate the project selection and to identify testing limits which would be permissible based on traffic control limitations. Coordination of testing schedules for traffic control, FWD testing and pavement coring was completed by Marquette staff.

2.2.2 Deflection Testing Program

Repeated deflection testing was conducted on each project to obtain representative deflection data over as broad a range of HMA mix temperatures as possible during the available testing window. Test locations were selected at approximately 100 foot intervals along the outer driving lane or pavement shoulder, depending on project constraints, to provide a minimum of 10 test locations per project. Test loads of approximately 5,000, 9,000 and 12,000 lbs were applied at each selected location. After completion of all tests along a given test lane, the FWD was re-positioned at the start of testing and the full test sequence was repeated. This process continued throughout the day until it was deemed that no additional significant variations in mix temperature would occur or until the test program was terminated due to traffic control restrictions.

2.2.3 Temperature Measurements

Pavement temperature measurements were obtained during the deflection testing program by on-site Marquette research staff. These measurements were obtained using a six-inch temperature probe which was inserted into a 3/8 inch hole pre-drilled trough the HMA layer(s). The temperature probe is equipped with three thermisters, individually placed at the mid-depth of the probe and at ½ inch offsets from each probe end. The temperature probe was connected to a digital display for manual recordation. Temperature data was obtained at approximately 15-30 minute intervals throughout the deflection testing program. Additional air and pavement surface temperatures were recorded by the FWD during testing at each test location.

2.2.4 Pavement Coring

HMA surface cores were collected to provide specific layer thickness information and to provide material samples for laboratory testing of resilient modulus. Three core locations were selected within each project at approximately 300 ft intervals to provide sufficient materials for comparative analysis. Core locations were typically offset by approximately 5 ft from adjacent deflection test locations to allow for both operations to be completed simultaneously.

After the core was cut and removed from the pavement, it was marked and layer thicknesses were recorded. Any layer separation that occurred during coring or removal was noted. The cores were then transferred to Michigan Technological University for laboratory testing. Care was taken to protect each core during shipment to minimize/eliminate core disturbance.

CHAPTER 3
ANALYSIS OF FIELD DEFLECTION AND TEMPERATURE DATA

3.1 Introduction

This chapter provides analysis results of collected deflection data using a variety of analysis techniques. This section presents a comparison of results from several different backcalculation methods, using data from one of the tested sites, and presents the backcalculation results for the six additional sites tested.

3.2 Trial Two-Layer AASHTO Backcalculation Approach Using Data from STH 164-Slinger

The 1993 AASHTO Guide for Design of Pavement Structures suggests the use of Boussinesq's point load equation to solve for the field subgrade modulus (M_R). Assuming a Poisson's ratio of 0.5 for the subgrade, the equation for the field subgrade modulus is:

$$\text{Field } M_R = 0.24\, P / (r\, Dr) \qquad \text{Eqn 3.2.1}$$

where:	M_R	=	backcalculated subgrade modulus (psi)
	P	=	load (pounds)
	r	=	distance (inches) from center of load plate
	Dr	=	deflection (mils) at distance r

Eqn 3.2.1 yields an estimate of the in-place modulus of the subgrade, independent of the thickness and stiffness of the overlying pavement structure, so long as at least one deflection sensor is located at a sufficient distance from the center of the load plate. What distance is sufficient is often difficult to determine a priori. In practice, the authors have found that a reasonable estimate of this distance is approximately twice the HMA layer thickness plus the base layer(s) thickness. This is only a general relation which can be used to select target outer sensor positions based on pavement structures being tested. It has also been observed that Eqn 3.2.1 can be used to calculate

subgrade M_R values for all sensor distances greater than zero. If at least one of the included sensors was positioned at a sufficient distance to isolate the subgrade M_R, the resulting plot of computed M_R vs sensor position is typically concave upwards, and the minimum value of M_R can be determined by inspection and used as a reasonable estimate of the Field M_R. When the outer sensor yields the minimum calculated M_R value, it may be assumed that this sensor was not positioned sufficiently far from the load plate to isolate the subgrade M_R and analysis results should then be viewed with caution.

It should also be noted that the concave upwards trend of most subgade M_R vs sensor location plots indicates the subgrade materials are stress-dependent, which is typically expected for fine-grained, stress-softening subgrade materials. For this reason, computed M_R values for sensor placements greater than that required to isolate the subgrade M_R are higher due to lower stress states at deeper levels in the subgrade.

The data from the SH164-Slinger site consists of measurements from eleven passes in northbound direction and eleven passes in the southbound direction. For this data, plots of the subgrade M_R versus sensor position were consistently concave upwards and the average sensor distance corresponding to the minimum M_R was 20.2 inches. For 64 percent of the deflection basins, the minimum M_R was obtained from the deflection measured at a distance of 18 inches from the load (d18), and for another 29 percent the minimum M_R was obtained from the deflection measured at a distance of 24 inches from the load (d24), both values being well below the outermost sensor distance.

The 1993 AASHTO Guide also presents the following equation for D0, the deflection measured at the center of the FWD load plate:

$$D0 = 1.5\, p\, a \left\{ \left[1/\left(MR \left((1+((D/a)(Ep/MR)^{1/3})^2)\right)^{1/2}\right)\right] + \left[(1-(1/((1 + (D/a)^2)^{1/2})))/Ep\right] \right\} \quad \text{Eqn 3.2.2}$$

where:

	D0	=	maximum deflection (at center of load plate) (mils)
	p	=	FWD plate pressure (psi)
	a	=	FWD plate radius (in)
	MR	=	in-place subgrade modulus (psi)
	D	=	total thickness of pavement structure above subgrade (in)
	Ep	=	effective elastic modulus of the pavement structure (psi)

With the maximum deflection D0 measured, the FWD plate pressure and radius known, and the pavement thickness known, Eqn 3.2.2 can be used to solve for the effective pavement modulus Ep. Thus, combined use of Eqns 3.2.1 and 3.2.2 can provide the solution to backcalculation of elastic moduli for a two-layer system, using two deflection measurements (D0 and Dr) to solve for two unknowns, namely M_R and Ep. Such a solution can be implemented in a computer spreadsheet program; however, Eqn 3.2.2 cannot be rearranged to solve for Ep directly. Ep can, however, be determined by iteration, that is, varying Ep until the calculated D0 matches the measured D0.

The determination of Ep as indicated above yields an effective pavement modulus which includes contributions from both the HMA and base layers. For pavements with thin and/or weak base layers, backcalculated Ep values determined using Eqn 3.2.2 with D = HMA thickness can provide reasonable estimates of the in situ modulus of the HMA layer. However, when the base layer(s) are thick and/or stiff, backcalculated Ep values following this approach would be expected to be significantly higher then the in situ HMA modulus.

Figure 3.2.1 illustrates the backcalculated subgrade resilient modulus, $M_{R,}$ computed by Eqn 3.2.1 for all collected data. As shown, there is little sensitivity to mix temperature and a slight indication of stress sensitivity for the load range used. The overall average subgrade modulus computed from all results is approximately 26,200 psi with a coefficient of variation of 20% (n=748;

μ=26,163psi; σ=5,223psi; COV=19.9%). Based on the applied load levels, the average subgrade moduli were 27,396, 25,734 and 25,442 psi for the 5, 9 and 12 kip load levels, respectively.

The measured thickness of the asphalt pavement at the SH164-Slinger site was 7 inches at each of the three core locations. Using this value as the assumed thickness at all test locations, the backcalculated effective pavement modulus, Ep, was computed for all test locations by Eqn. 3.2.2 and is provided in Figure 3.2.2. As shown, the backcalculated Ep values exhibit a notable sensitivity to mix temperature, as one would expect, but there is a large scatter in the data at any selected mix temperature. This scatter is most likely due to thickness variations in the component pavement layers at locations were no cores were recovered. There was only a 1% reduction in backcalculated Ep values for tests conducted at the 5 kip load level compared to the 12 kip load level (at the same temperature) indicating essentially no stress sensitivity for the parameter for the range of testing.

Figures 3.2.3 and 3.2.4 illustrate the AASHTO backcalculation results applied only to those test locations were pavement cores were extracted. As shown, data scatter is significantly reduced for these test locations where HMA thickness is known. The overall average subgrade resilient modulus computed for these locations is approximately 27,400 psi with a coefficient of variation of 6%. Based on data provided in Figure 3.2.4, the best-fit relationship between mix temperature and effective modulus for the E1 mixture used on SH164-Slinger can be described as follows:

$$Ep = 1{,}724{,}666 - 17{,}745\ tp \qquad R^2 = 0.9546 \qquad SEE=30{,}776\text{ psi} \qquad \text{Eqn 3.2.3}$$

where: Ep = Effective pavement modulus, psi
tp = Mid-depth mix temperature, F

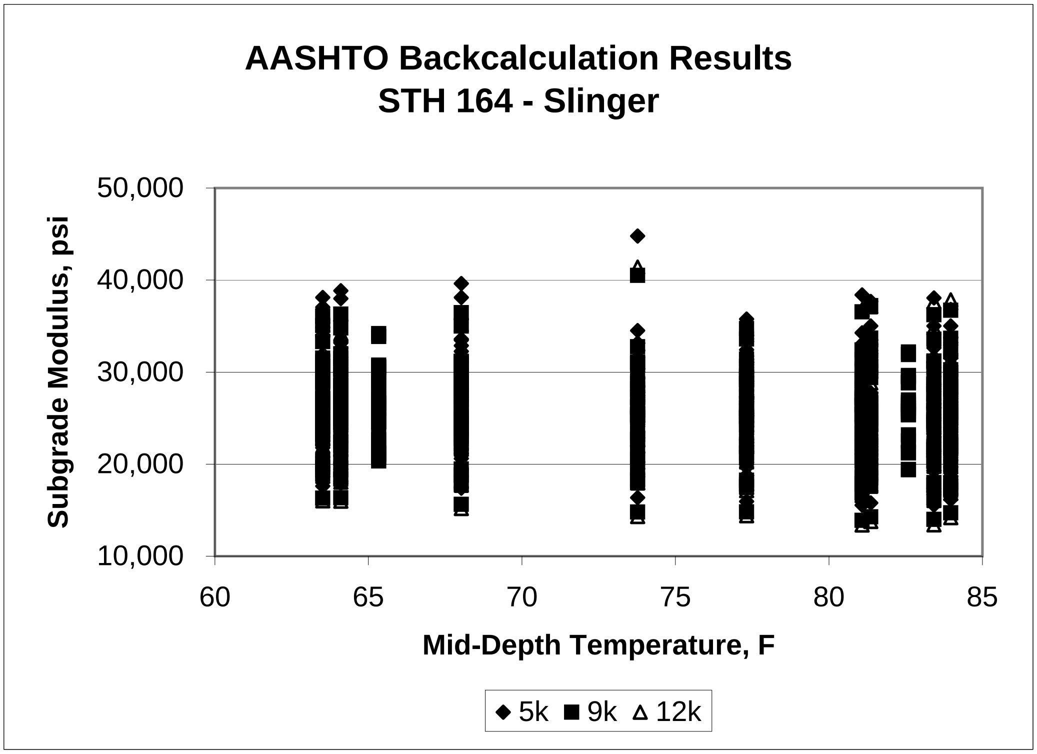

Figure 3.2.1 Subgrade Modulus vs Mid-Depth Temperature for all STH164-Slinger Data

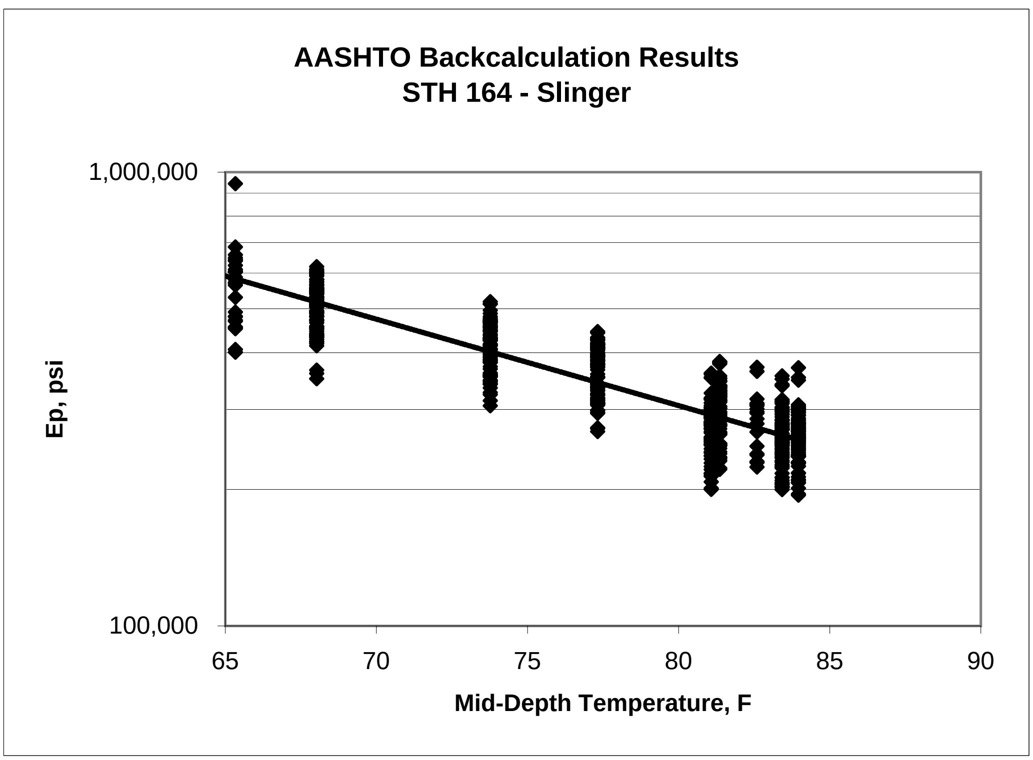

Figure 3.2.2 Pavement Modulus vs Mid-Depth Temperature for all STH164-Slinger Data

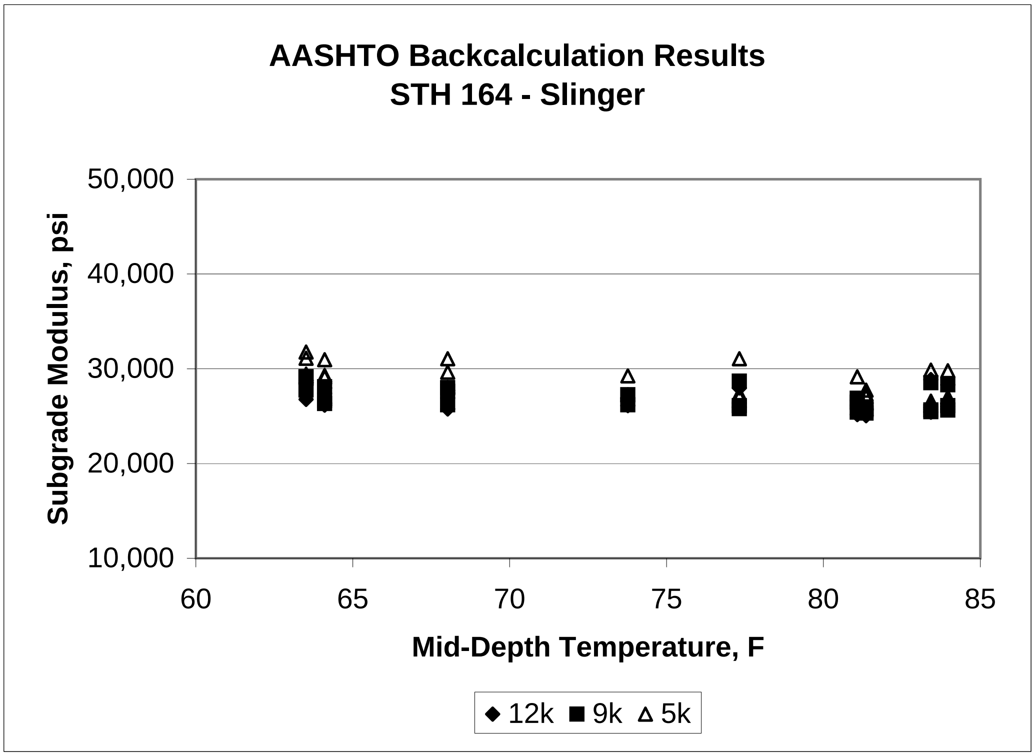

Figure 3.2.3 Subgrade Modulus vs Mid-Depth Temperature for STH164-Slinger Coreholes

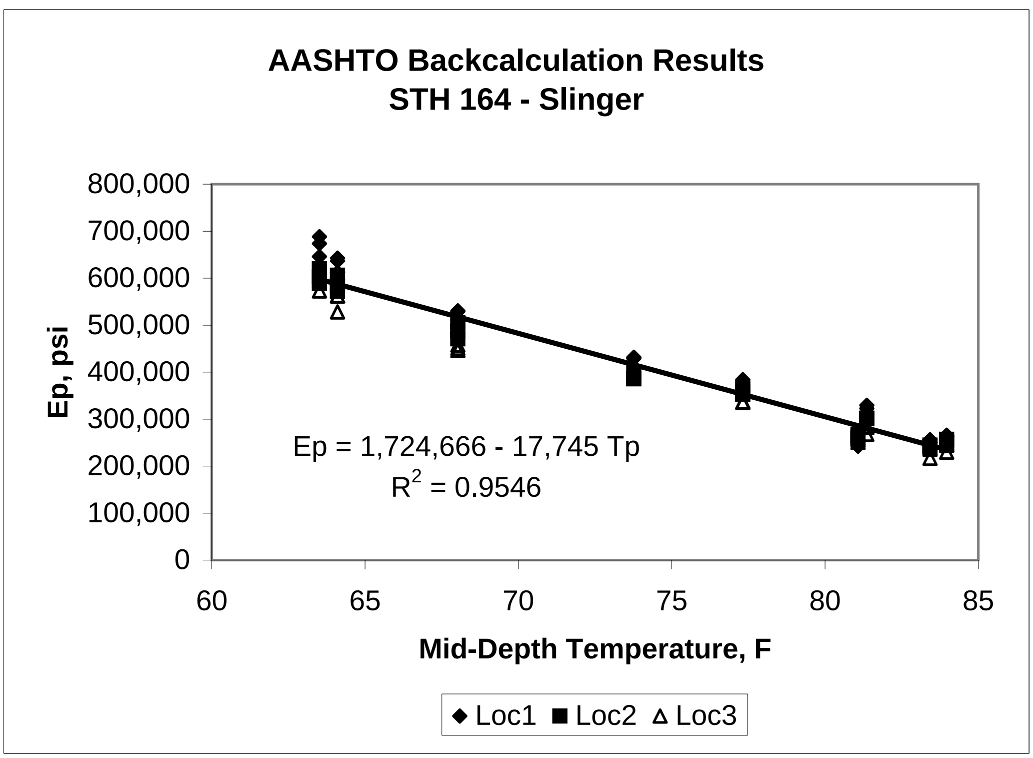

Figure 3.2.4 Pavement Modulus vs Mid-Depth Temperature for STH164-Slinger Coreholes

3.3 AASHTO Two-Layer Backcalculation Approach Applied to Other Data

The deflection data collected from other Wisconsin sites included in this study were analyzed using Eqns 3.2.1 and 3.2.2 to provide estimates of the in-place elastic modulus of the HMA layer asphalt concrete at these different sites. Figures 3.3.1 through 3.3.6 provide summary plots of average backcalculated HMA modulus for all other included projects where deflection data was obtained at corehole locations. Due to the observed non-stress sensitivity of the backcalculated HMA modulus, each data point represents the average backcalculated modulus using all applied load levels from each test series. As shown, results from each project site indicate temperature dependency, as expected. Results from four project sites (Kenosha, Waukesha, Boscobel, Ontario) also indicate location specific variations in the HMA modulus. These results will be further examined during the comparison between field and laboratory measures in Section 4.5.

3.4 Deflection-Based Algorithm for Estimating HMA Modulus

Research was conducted with the aim of developing a simple algorithm for estimating HMA layer moduli from surface deflections using only HMA layer thickness as an additional needed input. In the course of this research, the KENLAYER computer program was executed over a comprehensive pavement factorial which included stress-dependent base and subgrade layers. Table 3.4.1 provides the range of pavement structures investigated. A circular surface loading of 9,000 lb at 82.14 psi (radius = 5.9055 in) was applied in all cases to represent a standard FWD loading. Surface deflections were calculated at offset locations similar to those used during FWD testing.

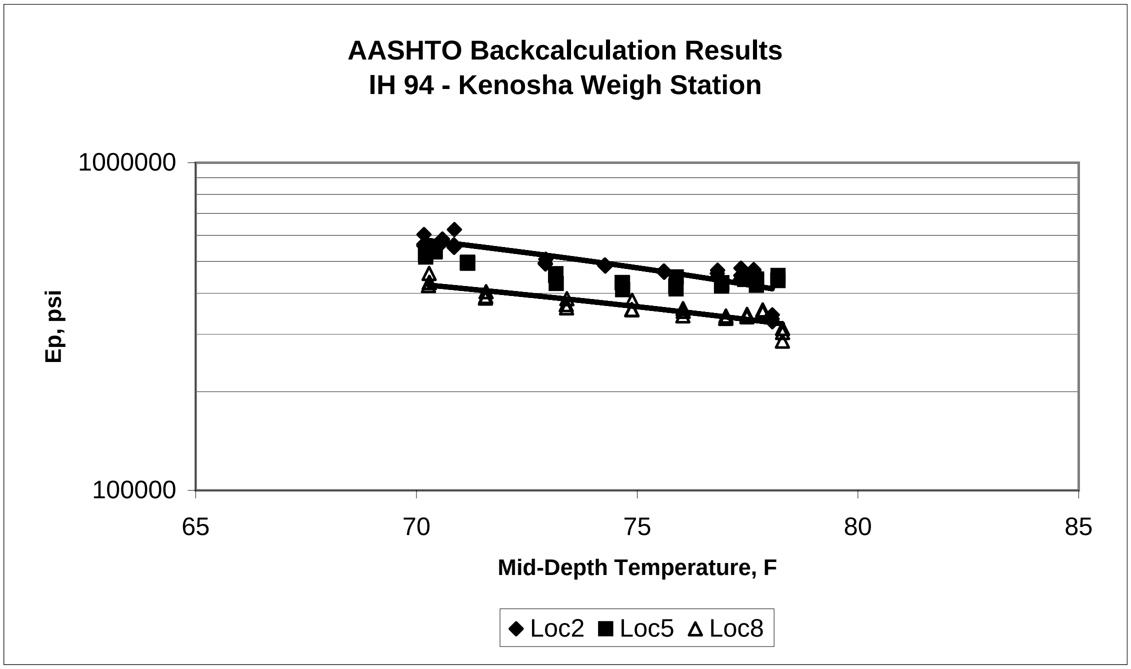

Figure 3.3.1 Pavement Modulus vs Mid-Depth Temperature for IH94-Kenosha Corehole Locations

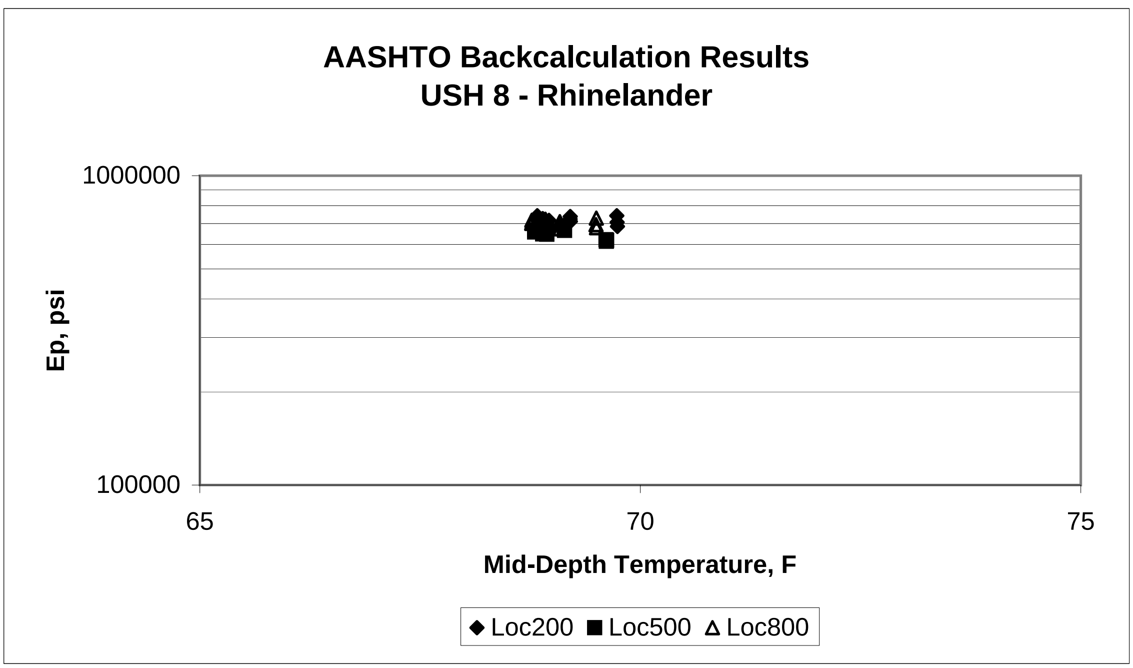

Figure 3.3.2 Pavement Modulus vs Mid-Depth Temperature for USH8-Rhinelander Corehole Locations

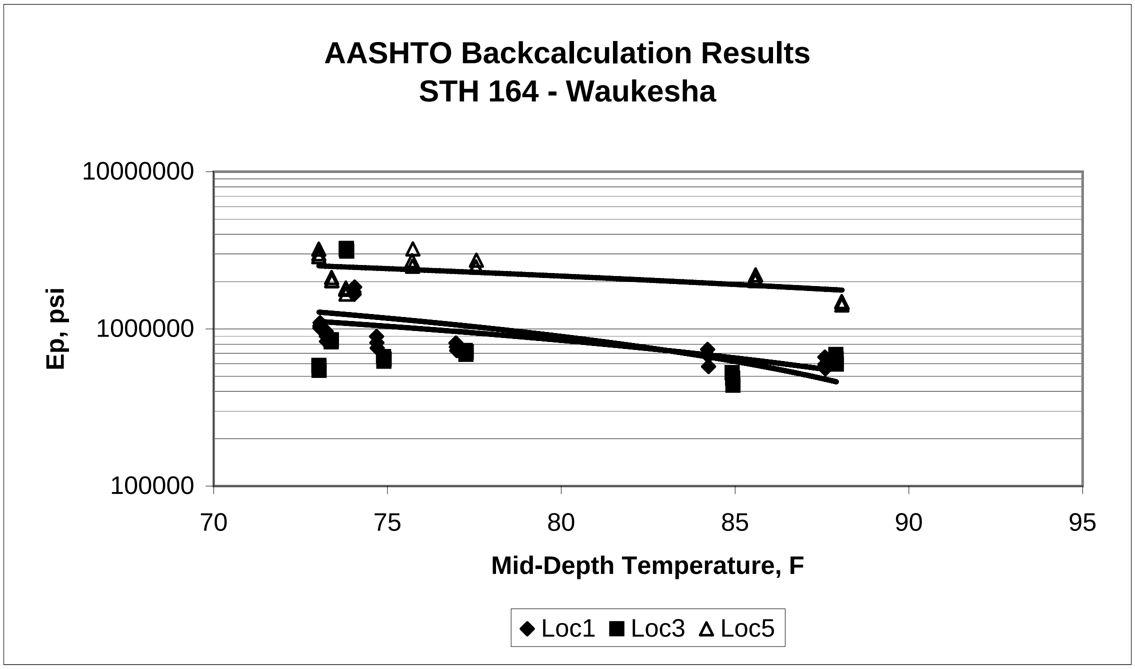

Figure 3.3.3 Pavement Modulus vs Mid-Depth Temperature for STH164-Waukesha Corehole Locations

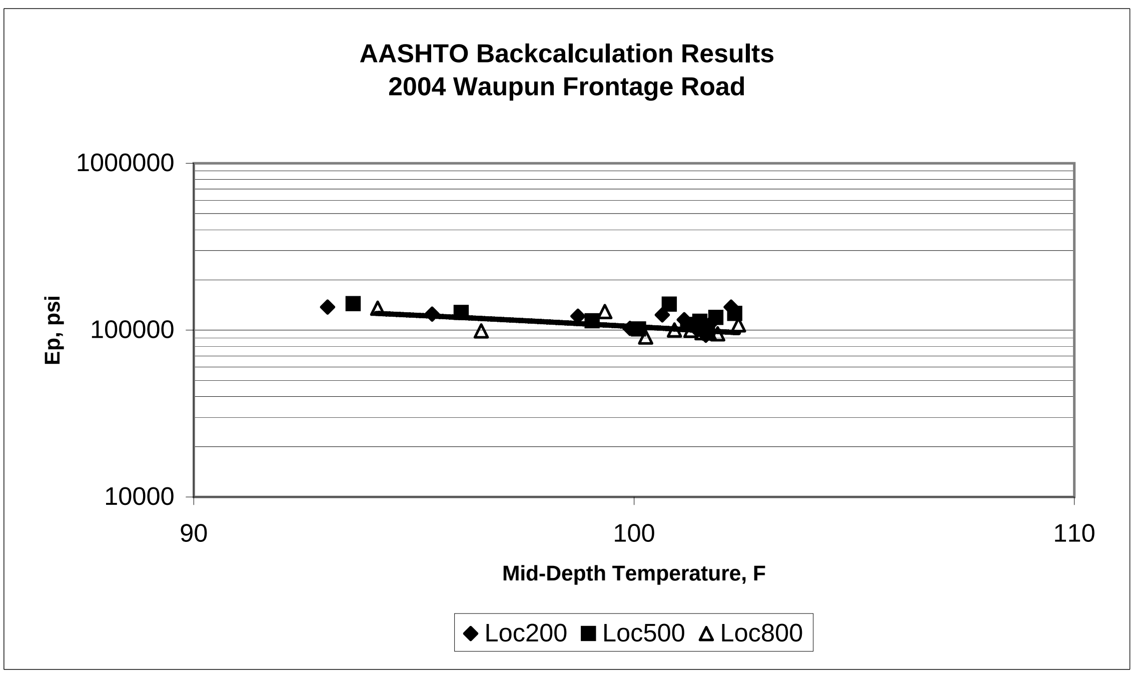

Figure 3.3.4 Pavement Modulus vs Mid-Depth Temperature for Waupun Frontage Road Corehole Locations

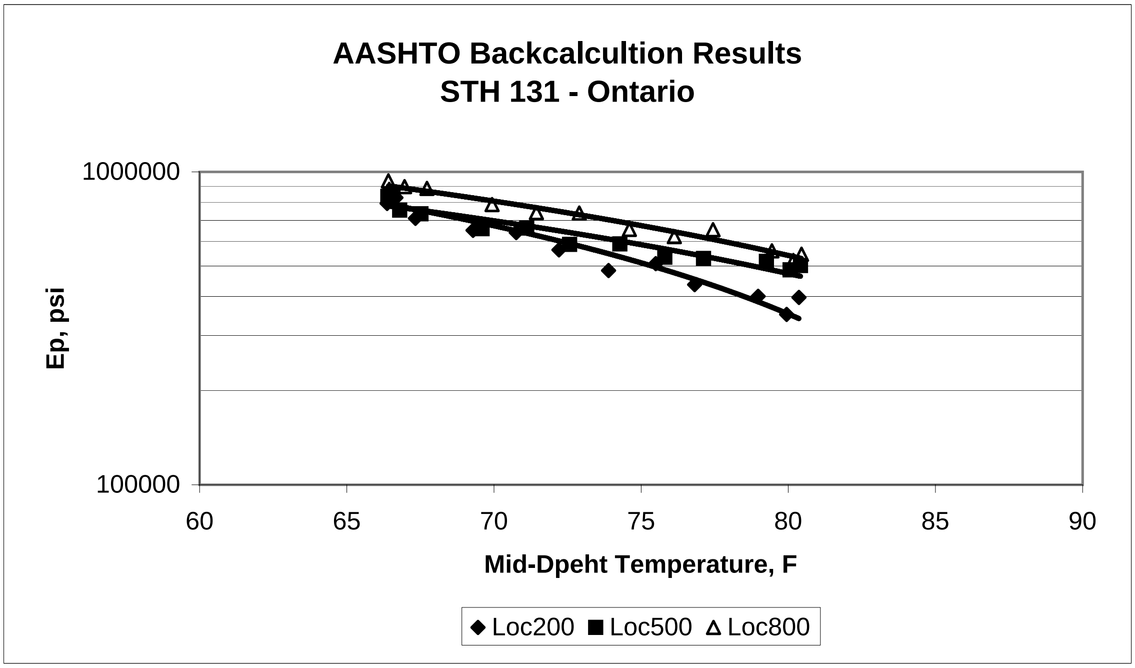

Figure 3.3.5 Pavement Modulus vs Mid-Depth Temperature for STH131-Ontario Corehole Locations

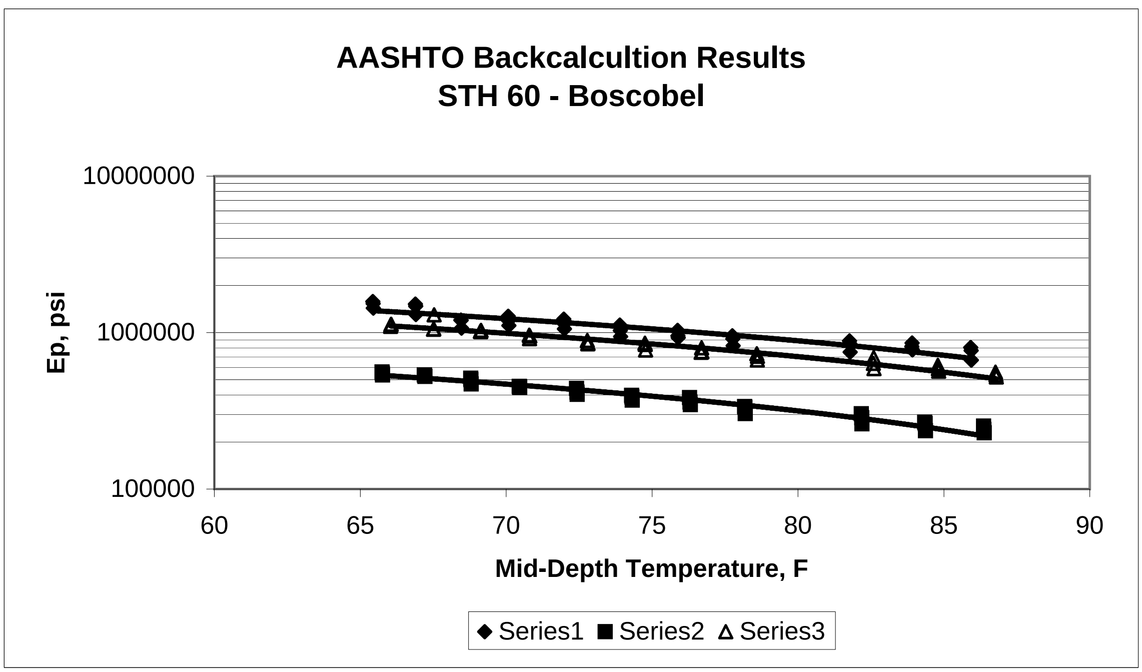

Figure 3.3.6 Pavement Modulus vs Mid-Depth Temperature for STH60-Boscobel Corehole Locations

Table 3.4.1 KENLAYER Pavement Factorial

Layer	Thickness Range	Modular Range
HMA	2 - 9 inches	Eac = 250 - 950 ksi
Aggregate Base	6 - 15 inches	$M_R = 4000\ \theta^{.6}$ $M_R = 5000\ \theta^{.5}$ $M_R = 8000\ \theta^{.4}$
Subgrade	240 inches	E_{RI} = 1 - 12.34 ksi
Bedrock	semi-infinite	E = 4,000 ksi

The complete factorial of KENLAYER runs included 7,680 separate pavement structures. The output results were parsed to include only those pavement structures where the ratio of base to HMA layer thickness was in the range of 2.0 to 3.0, which is consistent with pavement design practices in Wisconsin. The surface deflections contained in this parsed data set were analyzed to develop specific algorithms relating HMA properties to various deflection-based terms. The conducted analysis indicated the best agreement was achieved between the surface flexural rigidity and the surface curvature index, described as:

$$ET^3 = 1786039 * SCI^{-1.5893} \qquad \text{Eqn 3.4.1}$$

where: ET^3 = HMA flexural rigidity, ksi-in^3
E = HMA modulus, ksi
T = HMA thickness, in
SCI = Surface Curvature Index, mils (D0 - D12)
D0 = Maximum surface deflection normalized to 9,000 lbs, mils
D12 = Surface deflection at 12 inches from load plate normalized to 9,000 lb, mils

Figure 3.4.1 illustrates the comparison of ET^3 vs SCI that was used during equation development. It is important to note that the use of Eqn 3.4.1 does not require a backcalculation of the subgrade resilient modulus. Furthermore, the determination of ET^3 is direct and does not require iterative solutions. Both of these traits make this algorithm easily transferable to spreadsheet analysis and much more efficient than the previously described AASHTO 2-layer backcalculation approach.

3.4.1 Application of Algorithm to Wisconsin Projects

Deflection data collected from all projects were analyzed using Eqn 3.4.1. Figure 3.4.2 provides an illustration of the SCI versus mix temperature for all STH164-Slinger deflection data. As expected, Figure 3.4.2 indicates a definite trend of increased SCI with increasing HMA mix temperatures. Furthermore, there is no apparent bias based on the applied load level. This again is as expected as the HMA layer is not expected to be noticeably stress-dependent for the range of applied loadings. The increasing SCI versus temperature trend is predominately associated with the temperature dependent increases in maximum deflection, D0, as shown in Figure 3.4.3.

Figure 3.4.4 illustrates the results of HMA layer moduli backcalculation by Eqn 3.4.1 as applied to the STH 164-Slinger data set, based on an assumed HMA thickness of 7.0 inches at all locations. As shown, there is an expected trend of decreasing HMA modulus with increasing mix temperature. There is, however, significant scatter in the data for any given mix temperature. This scatter is due to both equation prediction errors as well as likely variations in HMA layer thickness. Because a main scope of this project is the comparison of backcalculated and measured HMA resilient moduli, more emphasis was placed on analysis results of deflection data collected at the three corehole locations on each project where HMA layer thickness are known.

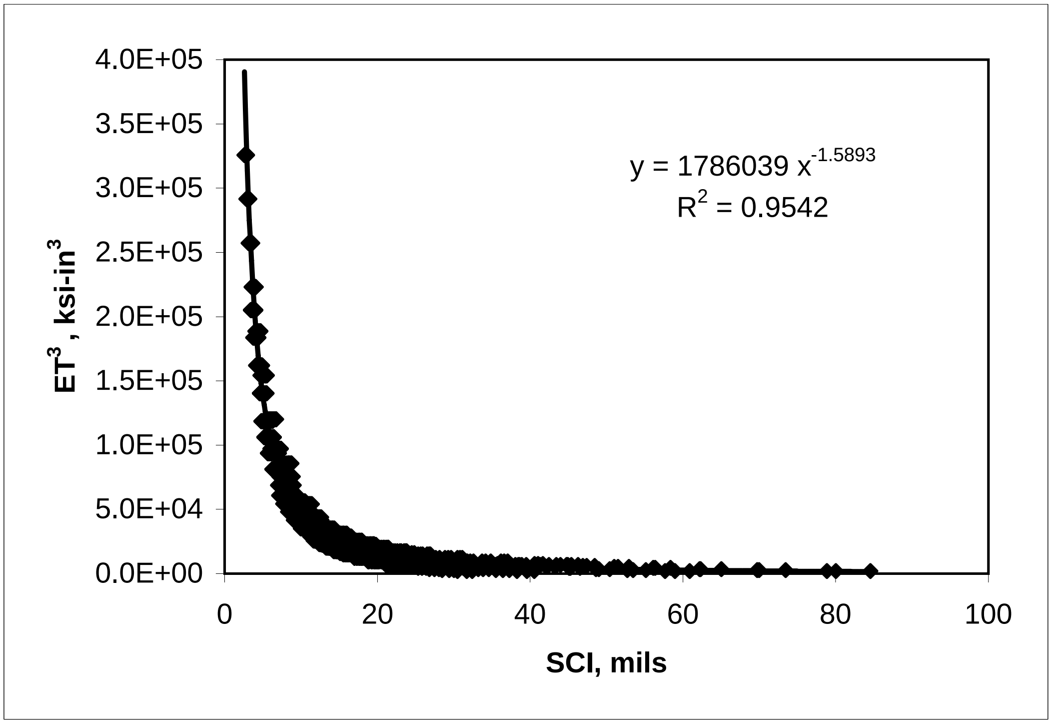

Figure 3.4.1: ET^3 vs SCI Used for Model Development

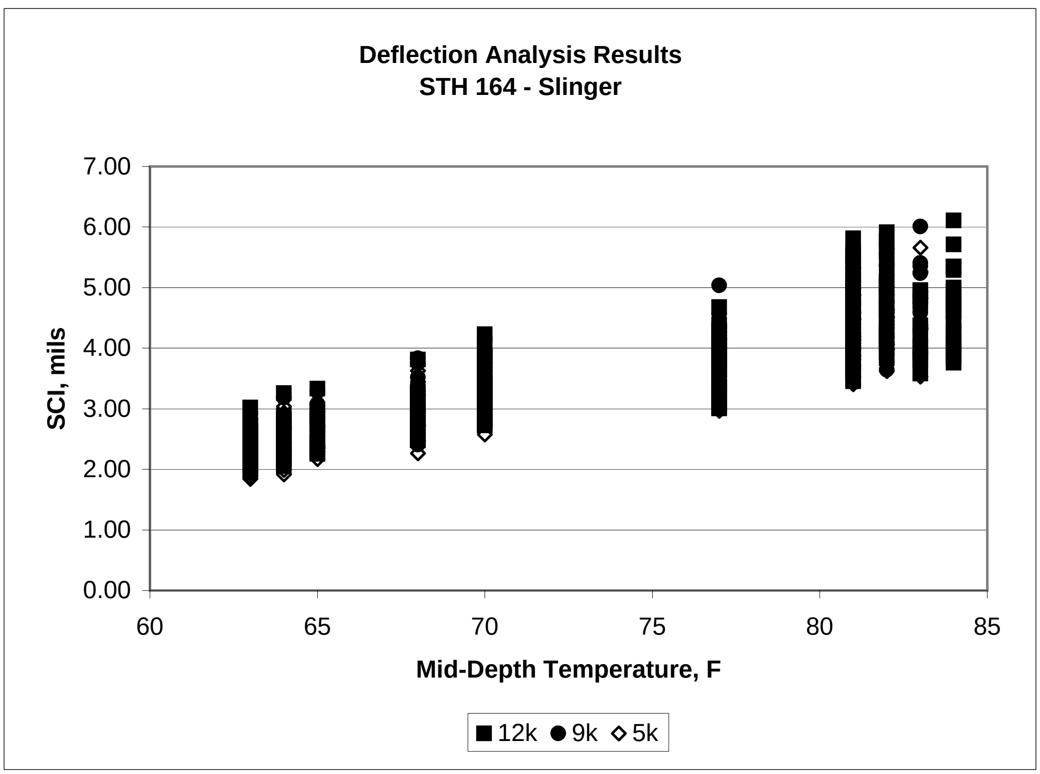

Figure 3.4.2: SCI vs Mid-Depth Temperature and Load Level for STH164-Slinger

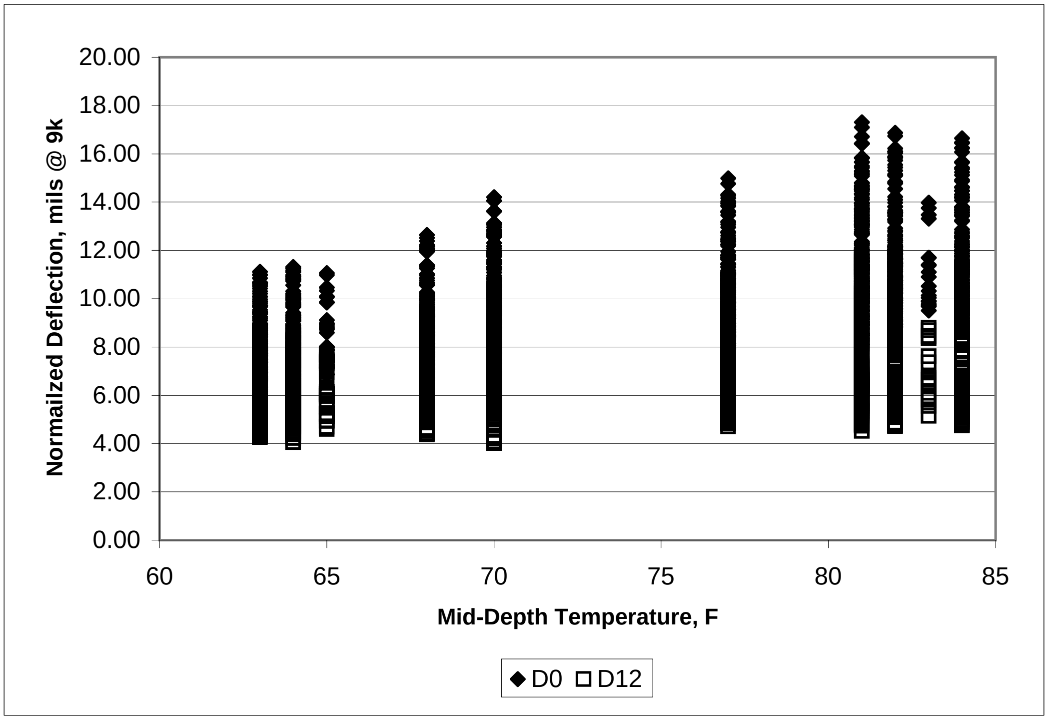

Figure 3.4.3: Deflections vs Mid-Depth Temperature for STH 164 - Slinger

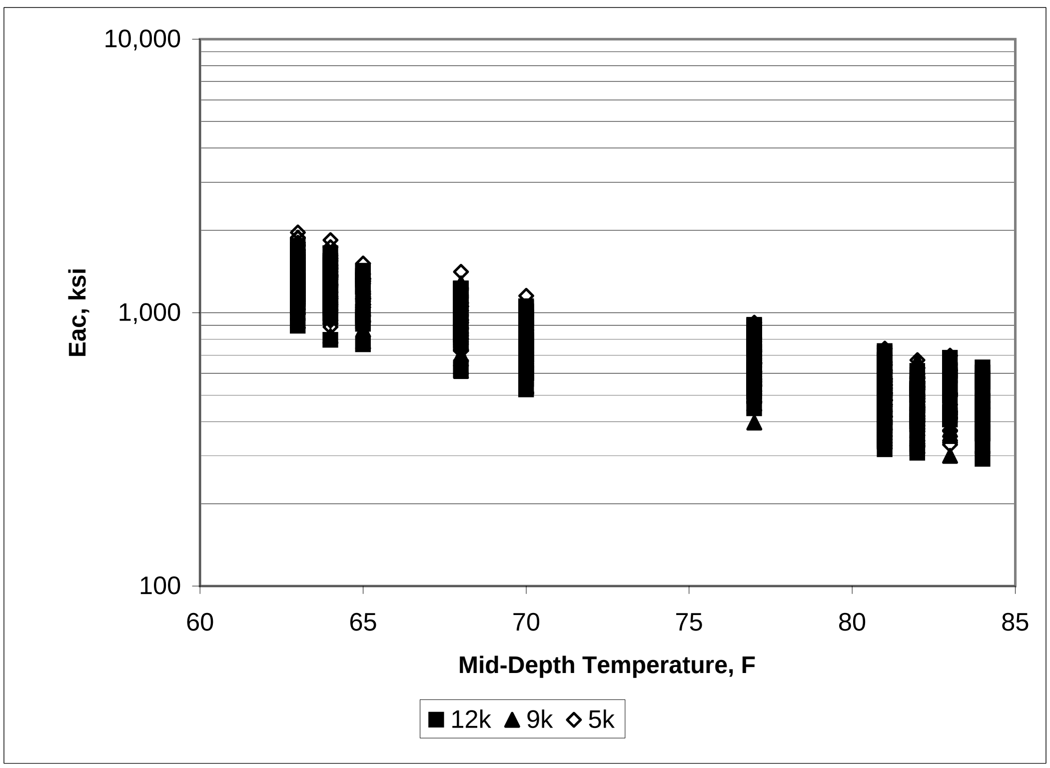

Figure 3.4.4: Eac vs Mid-Depth Temperature for STH 164 - Slinger

Figure 3.4.5 illustrates the analysis results for this scoped data set from STH164-Slinger. As shown, data scatter is significantly reduced when using only deflection data where HMA layer thickness is known (compare to Figure 3.4.4). Furthermore, no appreciable bias is noted for HMA moduli backcalculated at different FWD load levels.

Figures 3.4.6 through 3.4.12 provide summary plots of average backcalculated HMA modulus for all other included projects where deflection data was obtained at corehole locations. Each data point represents the average backcalculated modulus using all applied load levels from each test series. A comparison of these backcalculated values to laboratory obtained values is provided in Chapter 4.

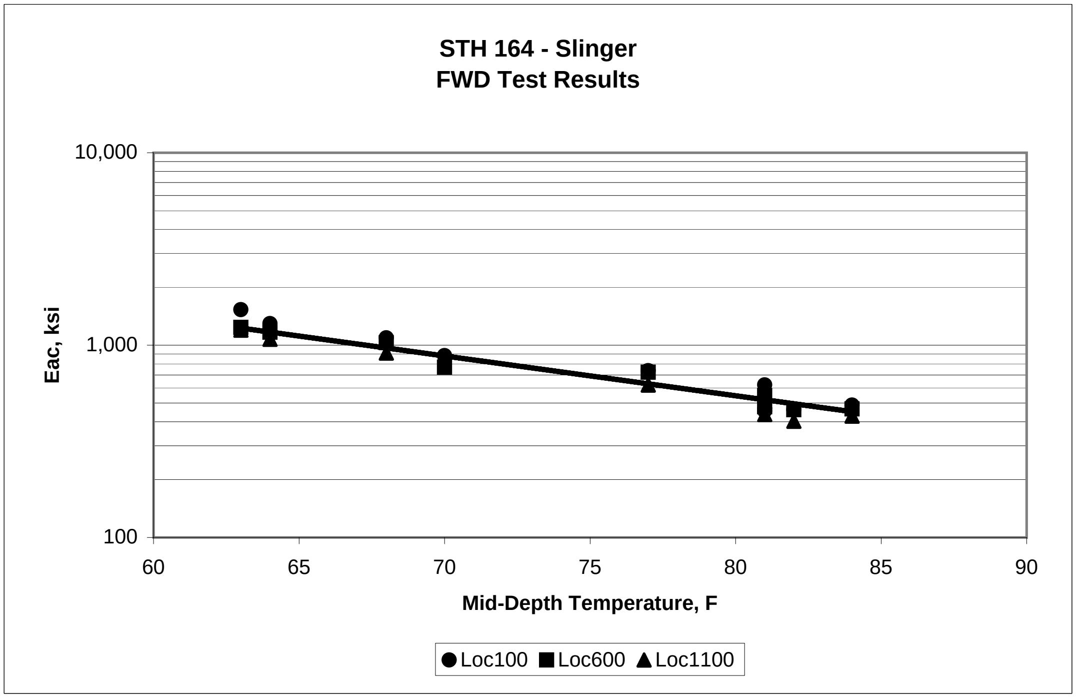

Figure 3.4.5 Eac vs Mid-Depth Temperature for STH164-Slinger Corehole Locations

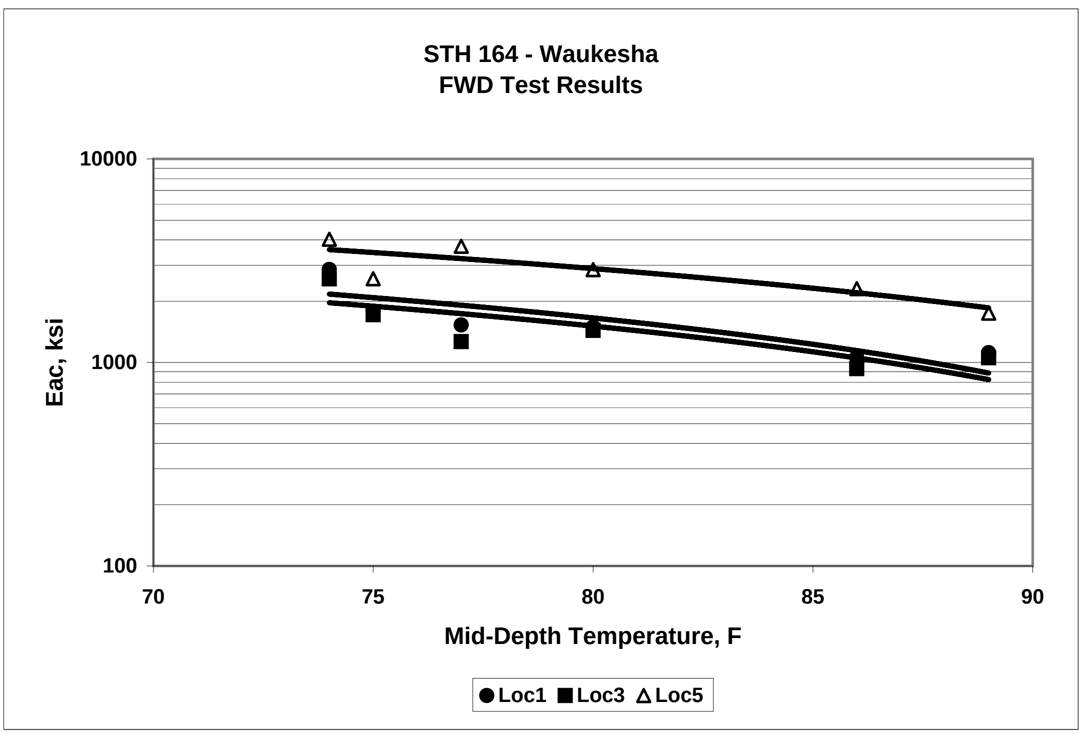

Figure 3.4.6 Eac vs Mid-Depth Temperature for STH164-Waukesha Corehole Locations

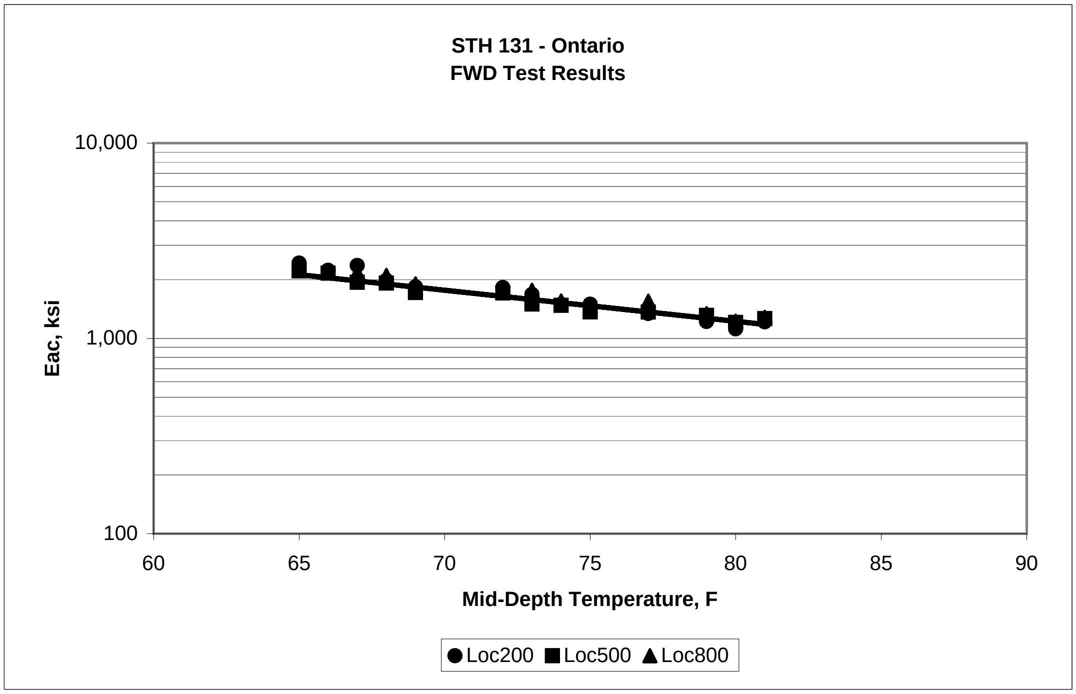

Figure 3.4.7 Eac vs Mid-Depth Temperature for STH131-Ontario Corehole Locations

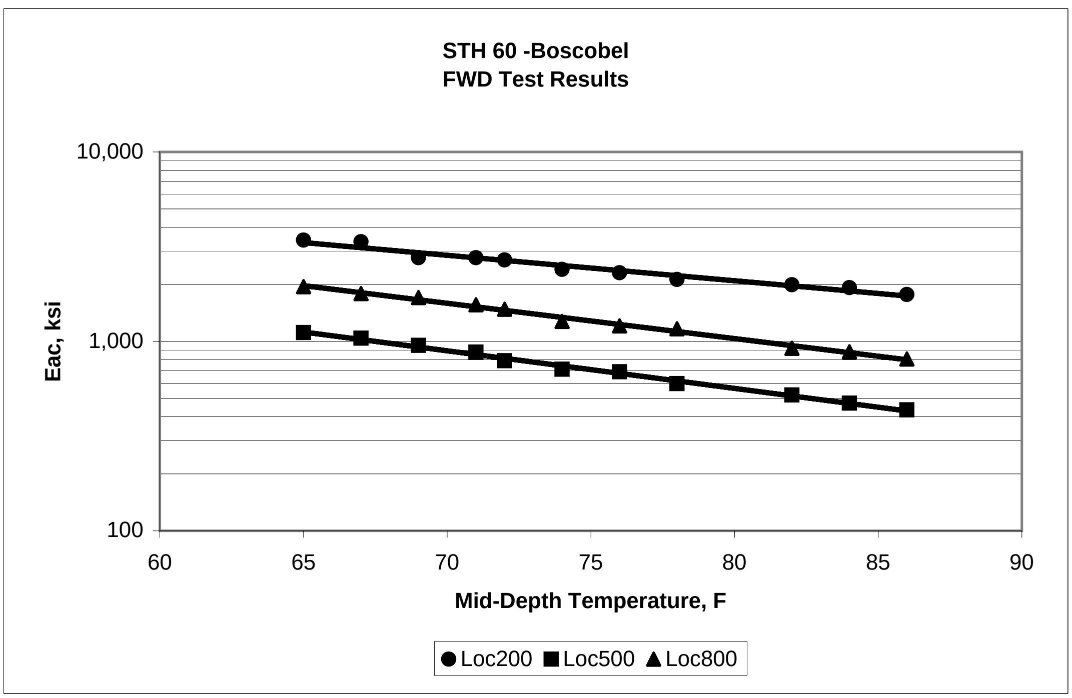

Figure 3.4.8 Eac vs Mid-Depth Temperature for STH60-Boscobel Corehole Locations

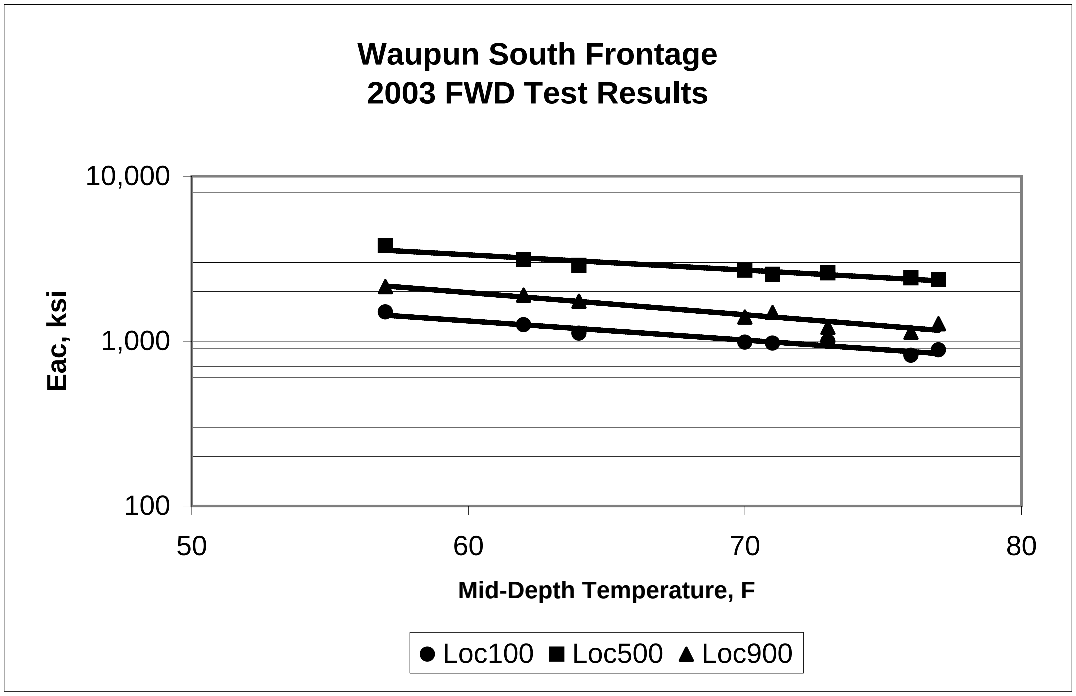

Figure 3.4.9 Eac vs Mid-Depth Temperature for 2003 Waupun Frontage Road Corehole Locations

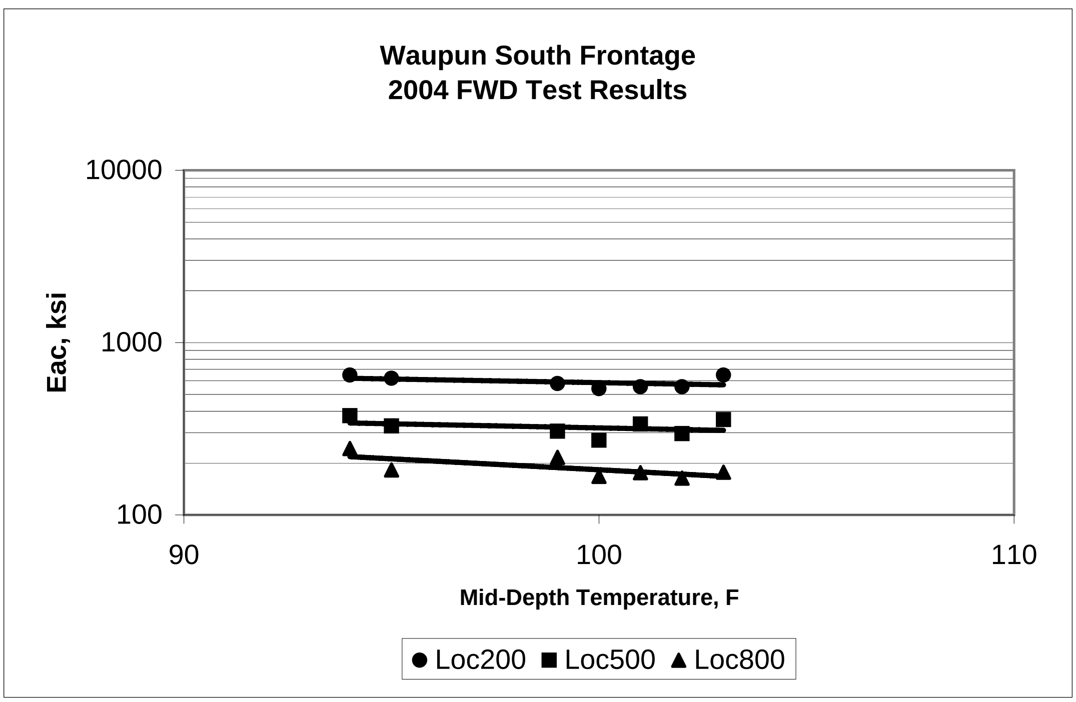

Figure 3.4.10 Eac vs Mid-Depth Temperature for 2004 Waupun Frontage Road Corehole Locations

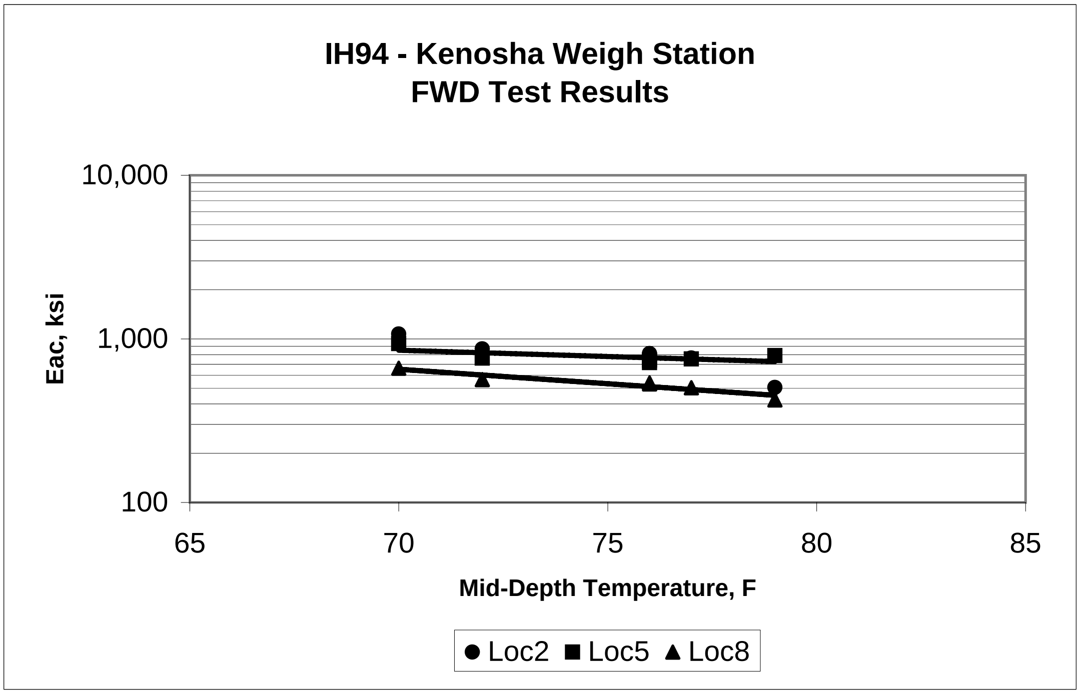

Figure 3.4.11 Eac vs Mid-Depth Temperature for IH 94-Kenosha Corehole Locations

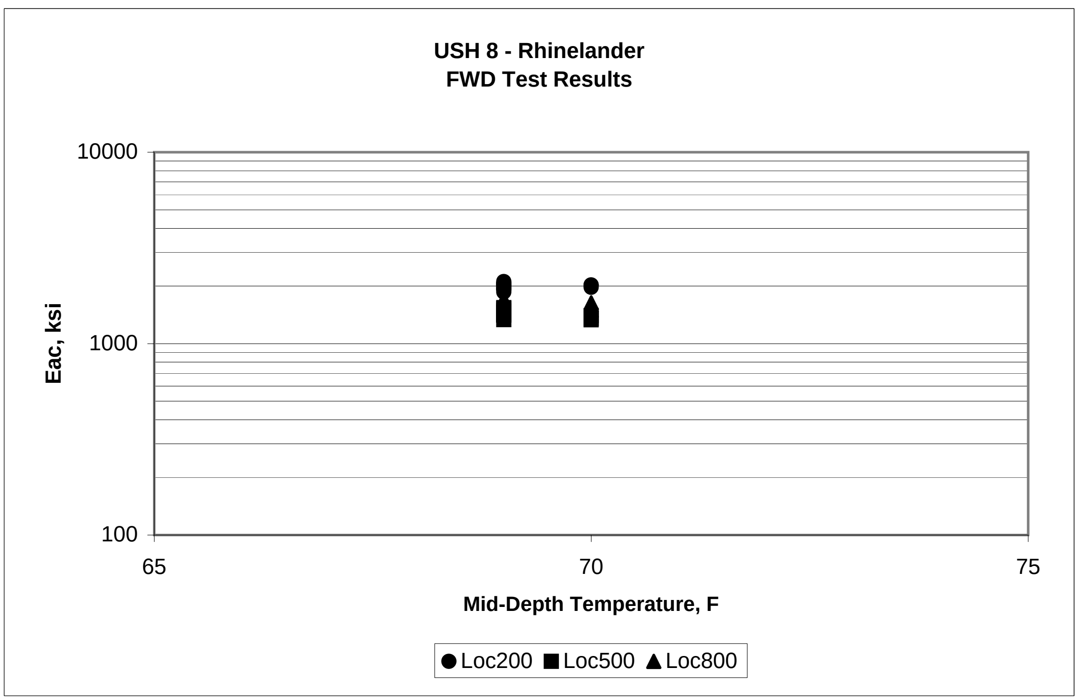

Figure 3.4.12 Eac vs Mid-Depth Temperature for USH 8-Rhinelander Corehole Locations

CHAPTER 4
LABORATORY TESTING

4.1 Introduction

This chapter provides details on the laboratory testing procedures used for this project, the results obtained, and an analysis of these results as they relate to the project objectives. All laboratory testing, including characterization of field cores and resilient modulus testing, was conducted at Michigan Technological University (MTU).

4.2 Characterization of Field Cores

Specimens were cored directly from the pavement using both 4-inch and 6-inch coring bits. The specimens from Waupun and Slinger were cored using a 4-inch coring bit, whereas specimens from Rhinelander, Ontario, Waukesha, Boscobel, and Kenosha were cored using a 6-inch coring bit. In order to maintain uniformity for the test specimens, it was decided that the 4-inch specimen would be the standard specimen size for resilient modulus testing. The 6-inch cores were subsequently cored to provide a 4-inch diameter test specimens using a specimen clamp designed by MTU.

Core specimens were, for the most part, procured with multiple lifts where the tack coat had sufficiently bonded the layers together. The locations of the individual lifts were visually identified and separated either through diamond saw cutting or prying apart. Special care was exercised during the separation process to eliminate/minimize damage to the specimens. The upper and lower surfaces of the specimens were rough and the lower layers were covered in a film of tack. To remove major surface irregularities, specimens were trimmed down once again using a diamond saw. To address minor surface irregularities and to ensure that the specimens had smooth and parallel surfaces, a lapping wheel was used to grind the specimens.

After the specimens were ground down, the bulk specific gravity of the specimens was determined using the procedure outlined in AASHTO T166. The two surfaces of the specimens were then marked with perpendicular lines, so as to record the specimen heights and diameters measurements at specific locations and to identify the correct orientation of the specimen during resilient modulus testing.

4.3 Resilient Modulus Testing

Resilient modulus testing was conducted at three test temperatures of 4°C, 25°C, and 40°C to provide data necessary for determining representative material properties over a broad temperature range. ASTM D4123: *Indirect Tension Test for Resilient Modulus of Bituminous Mixtures* suggests a low test temperature of 5°C while NCHRP Report 465: *Simple Performance Test for Superpave Mix Design* suggests a minimum test temperature of 4°C be used for dynamic modulus testing. As there is relatively little difference between these temperatures, the low temperature of 4°C was selected because it is the opinion of MTU researchers that it would be prudent to select a low temperature that would be applicable to both testing formats as there is a move at the national level to make dynamic modulus testing the standard test for pavement design. The intermediate test temperature of 25°C was selected because it is the base temperature for most tests and is recommended in ASTM D4123. The high test temperature of 40°C was selected based on the high range of the temperatures listed in ASTM D4123. NCHRP Report 465 suggests the highest temperature at which specimens should be tested is 60°C; however, the MTU research team felt that a maximum test temperature of 40°C would be more appropriate to eliminate creep that could easily occur at elevated temperatures. Testing went from the lowest to highest temperature so that if any unexpected creep did occur, there would at least be data from the lower temperature testing for comparative analysis.

The low temperature testing at 4°C was conducted on the UTM 100 servo-hydraulic system with the remaining test conducted on the UTM 5 servo-pneumatic system. The UTM 100 was selected for low temperature testing because it is believed that the servo-pneumatic system on the UTM 5 could have problems with condensed moisture in the line which could freeze and cause erratic results at this temperature. Testing at 25°C and 40°C was conducted on the UTM 5 to provide for better resolution at the lower loads. Figures 4.3.1 and 4.3.2 illustrate the UTM 100 and UTM 5 machines, respectively, and the test configurations used for resilient modulus testing.

Prior to resilient modulus testing, specimens were brought to test temperatures in a temperature controlled chamber. A dummy specimen with a thermocouple placed in its center was used to indicate when test temperatures were achieved. In many instances specimens were conditioned over night for a period of not less than 12 hours.

The load pulse durations that were selected for this project were 50 and 100msec, which correspond to test frequencies of 2.0 and 1.0Hz, respectively. In order to draw direct comparisons between the results of in situ FWD testing and laboratory resilient modulus testing, the load pulse duration during testing should be similar. Additionally, the rest period used during laboratory should be of sufficient length to allow for the specimen to recover sufficiently from the loading, especially with no confining medium. Typical Falling Weight Deflectometer (FWD) testing is conducted with a load pulse duration of 25-30 msec with a rest period between loadings of 10 seconds or more.

Based on the FWD load pulse duration, a laboratory test frequency of approximately 4 Hz would be required (25 msec load pulse + 225 msec rest period = 250 msec total test time per cycle). It is the opinion of the MTU research staff that this 225 msec recovery period would not be sufficient, and thus the maximum test frequency of 2 Hz was selected as appropriate. The second loading frequency of 1 Hz was utilized to provide data on the materials response to changes in load pulse durations which may be useful for developing appropriate shift factors for correlating laboratory resilient modulus values with those estimated from FWD deflection data. These shift factors could also be compared to those predicted by Equation 1.1, previously described in Chapter 1.

Test loads were selected based on the ten-percent of the maximum tensile strength rule as stipulated by ASTM D4123. The maximum tensile strength test is a destructive test and all of the core specimens that were procured from Marquette University were intended for resilient modulus testing. MTU had several pavement cores remaining from other field studies which were considered as representative of many of the core specimens obtained for this project. As it was felt that differences in tensile strength would be negligible, these additional specimens were tested to determine their maximum tensile strengths at the aforementioned resilient modulus test temperatures, resulting in selected resilient modulus test loads of 500, 450, and 200N for test temperatures of 4°C, 25°C, and 40°C, respectively.

Table 4.4.1 Laboratory Resilient Modulus Test Results

Project	Mix	Binder	Coarse	Gradation	Core						Lab Mr @ 2Hz , psi			Lab Mr @ 1Hz , psi		
Location	Type	Type	Aggregate	Class	ID	Gmb	Gmm	Vv	P200	%AC	39.2F	77F	104F	39.2F	77F	104F
Boscobel	E1-12.5	PG58-28	Limestone	Fine	1T	2.354	2.472	4.8	4.1	5.7	1,893,177	409,151	151,129	1,995,864	318,938	108,343
Boscobel	E1-12.5	PG58-28	Limestone	Fine	2T	2.370	2.472	4.1	4.1	5.7	2,189,779	562,021	102,252	2,028,497	338,663	82,962
Boscobel	E1-12.5	PG58-28	Limestone	Fine	3T	2.343	2.472	5.2	4.1	5.7	2,307,840	567,097	242,648	2,193,550	456,289	162,587
Boscobel	E1-19	PG58-28	Limestone	Fine	1M	2.311	2.490	7.2	4.4	6.0	1,555,094	279,633	83,107	1,387,576	216,541	57,725
Boscobel	E1-19	PG58-28	Limestone	Fine	2M	2.349	2.490	5.7	4.4	6.0	1,884,475	289,495	81,801	1,485,621	207,694	67,733
Boscobel	E1-19	PG58-28	Limestone	Fine	3M	2.379	2.490	4.5	4.4	6.0	1,842,414	320,678	90,358	1,701,002	241,633	61,061
Boscobel	E1-19	PG58-28	Limestone	Fine	1B	2.376	2.490	4.6	4.4	6.0	1,687,224	438,739	171,580	1,840,383	366,365	111,099
Boscobel	E1-19	PG58-28	Limestone	Fine	2B	2.356	2.490	5.4	4.4	6.0	1,539,285	247,724	78,320	1,428,041	192,320	57,145
Boscobel	E1-19	PG58-28	Limestone	Fine	3B	2.351	2.490	5.6	4.4	6.0	1,660,392	263,824	76,870	1,484,171	206,679	47,427
Kenosha	E30x-12.5	PG76-28	Limestone	Fine	1T	2.344	2.529	7.3	4.8	5.4	1,277,927	221,473	50,038	1,111,569	151,129	34,519
Kenosha	E30x-12.5	PG76-28	Limestone	Fine	2T	2.330	2.529	7.9	4.8	5.4	1,108,668	153,160	44,527	954,638	101,671	32,633
Kenosha	E30x-12.5	PG76-28	Limestone	Fine	3T	2.279	2.529	9.9	4.8	5.4	1,177,996	154,755	42,786	1,025,272	108,198	33,214
Kenosha	E30x-25	PG70-22	Limestone	Coarse	1B	2.426	2.554	5.0	3.8	4.5	2,664,488	794,807	265,564	2,716,411	638,456	194,351
Kenosha	E30x-25	PG70-22	Limestone	Coarse	2B	2.408	2.554	5.7	3.8	4.5	2,471,152	565,647	184,198	2,061,856	448,166	121,687
Kenosha	E30x-25	PG70-22	Limestone	Coarse	3B	2.412	2.554	5.6	3.8	4.5	2,001,375	471,808	93,259	1,805,429	333,877	61,496
Ontario	E1-12.5	PG58-28	Limestone	Fine	1T	2.354	2.515	6.4	4.1	5.4	1,772,506	388,411	119,366	1,601,651	311,251	94,710
Ontario	E1-12.5	PG58-28	Limestone	Fine	2T	2.345	2.515	6.8	4.1	5.4	1,732,040	396,968	92,679	1,718,552	312,121	72,519
Ontario	E1-12.5	PG58-28	Limestone	Fine	3T	2.357	2.515	6.3	4.1	5.4	1,636,605	423,510	124,152	1,502,736	309,365	81,801
Ontario	E1-12.5	PG58-28	Limestone	Fine	1B	2.390	2.515	5.0	4.1	5.4	1,724,208	440,044	110,374	1,556,400	305,014	76,145
Ontario	E1-12.5	PG58-28	Limestone	Fine	2B	2.402	2.515	4.5	4.1	5.4	2,253,596	469,632	113,855	2,028,642	349,106	77,160
Ontario	E1-12.5	PG58-28	Limestone	Fine	3B	2.380	2.515	5.4	4.1	5.4	1,791,506	451,647	119,221	1,801,803	355,487	77,595
Rhinelander	E3-12.5	PG58-28	Gravel	Fine	1T	2.356	2.494	5.5	3.3	5.1	1,796,727	315,312	81,076	1,728,849	241,488	53,374
Rhinelander	E3-12.5	PG58-28	Gravel	Fine	2T	2.354	2.494	5.6	3.3	5.1	1,705,643	328,946	112,984	1,582,506	252,366	65,557
Rhinelander	E3-12.5	PG58-28	Gravel	Fine	3T	2.369	2.494	5.0	3.3	5.1	1,402,370	313,862	83,542	1,450,812	237,282	56,855
Rhinelander	E3-19	PG58-28	Gravel	Fine	1B	2.342	2.500	6.3	3.3	4.9	1,450,087	310,961	93,259	1,318,103	236,702	55,259
Rhinelander	E3-19	PG58-28	Gravel	Fine	2B	2.371	2.500	5.2	3.3	4.9	1,252,256	302,114	57,435	1,195,691	221,037	40,466
Rhinelander	E3-19	PG58-28	Gravel	Fine	3B	2.335	2.500	6.6	3.3	4.9	850,356	149,824	48,733	579,571	110,954	35,534
Slinger	E1-12.5	PG58-28	Gravel	Fine	1AT	2.292	2.517	8.9	4.4	5.5	1,544,361	330,541	n.a.	1,608,178	357,373	n.a.
Slinger	E1-12.5	PG58-28	Gravel	Fine	2AT	2.347	2.517	6.8	4.4	5.5	1,931,467	366,510	94,565	1,811,811	454,548	58,595
Slinger	E1-12.5	PG58-28	Gravel	Fine	3AT	2.309	2.517	8.3	4.4	5.5	1,306,500	294,717	70,923	1,305,774	293,991	40,756
Slinger	E1-19	PG58-28	Gravel	Fine	1AM	2.326	2.567	9.4	4.4	4.8	1,565,102	389,281	92,244	1,519,705	344,610	56,710
Slinger	E1-19	PG58-28	Gravel	Fine	2AM	2.353	2.567	8.3	4.4	4.8	1,899,849	497,479	130,824	1,737,987	449,327	79,916
Slinger	E1-19	PG58-28	Gravel	Fine	3AM	2.299	2.567	10.4	4.4	4.8	1,382,789	592,189	96,885	1,313,461	489,502	60,046
Slinger	E1-19	PG58-28	Gravel	Fine	1AB	2.366	2.567	7.8	4.4	4.8	1,739,727	369,701	137,206	1,587,583	315,022	76,435
Slinger	E1-19	PG58-28	Gravel	Fine	2AB	2.337	2.567	9.0	4.4	4.8	1,567,858	350,266	130,244	1,373,652	310,526	75,565
Slinger	E1-19	PG58-28	Gravel	Fine	3AB	2.322	2.567	9.5	4.4	4.8	1,504,186	327,350	111,969	1,251,530	309,510	39,885
Waukesha	E10-12.5	PG70-28	Limestone	Fine	1T	2.416	2.565	5.8	5.0	5.5	1,605,277	376,373	80,931	1,467,346	270,640	56,130
Waukesha	E10-12.5	PG70-28	Limestone	Fine	2T	2.378	2.565	7.3	5.0	5.5	1,307,515	325,755	69,038	1,167,118	238,587	47,717
Waukesha	E10-12.5	PG70-28	Limestone	Fine	3T	2.417	2.565	5.8	5.0	5.5	1,480,980	392,327	88,908	1,350,301	284,274	61,061
Waukesha	E10-19	PG58-28	Limestone	Fine	1B	2.422	2.593	6.6	3.7	4.6	2,004,711	571,594	251,495	1,943,360	489,647	165,343
Waukesha	E10-19	PG58-28	Limestone	Fine	2B	2.387	2.593	7.9	3.7	4.6	1,647,918	493,273	154,755	1,520,140	399,869	85,717
Waukesha	E10-19	PG58-28	Limestone	Fine	3B	2.419	2.593	6.7	3.7	4.6	2,656,220	589,578	159,687	2,236,771	499,945	91,954
Waupun03	E1-19	PG58-28	Limestone	Fine	1	2.348	2.588	9.3	4.1	4.7	1,423,255	366,655	72,809	1,237,607	261,938	55,840
Waupun03	E1-19	PG58-28	Limestone	Fine	2	2.361	2.588	8.8	4.1	4.7	1,006,997	327,930	75,275	1,323,179	253,381	42,496
Waupun03	E1-19	PG58-28	Limestone	Fine	3	2.358	2.588	8.9	4.1	4.7	1,268,790	257,007	n.a.	1,054,569	394,648	n.a.
Waupun04	E1-12.5	PG58-28	Limestone	Fine	1T	2.364	2.571	8.1	4.5	5.1	2,107,978	335,327	74,839	1,995,429	256,282	49,313
Waupun04	E1-12.5	PG58-28	Limestone	Fine	2T	2.381	2.571	7.4	4.5	5.1	1,254,141	222,488	50,183	1,188,439	154,175	33,794
Waupun04	E1-12.5	PG58-28	Limestone	Fine	3T	2.357	2.571	8.3	4.5	5.1	1,214,111	194,060	40,756	1,181,187	131,839	26,832
Waupun04	E1-19	PG58-28	Limestone	Fine	1B	2.392	2.588	7.6	4.1	4.7	1,699,842	377,388	100,221	1,610,354	276,007	59,175
Waupun04	E1-19	PG58-28	Limestone	Fine	2B	2.392	2.588	7.6	4.1	4.7	1,947,276	301,968	63,527	1,825,880	215,091	39,015
Waupun04	E1-19	PG58-28	Limestone	Fine	3B	2.389	2.588	7.7	4.1	4.7	1,502,155	297,617	62,946	1,452,553	199,282	36,985

4.4 Density and Resilient Modulus Test Results

Table 4.4.1 provides a summary of the laboratory resilient modulus test results. Density results are provided as the AASHTO T166 bulk density and as a percentage of the theoretical maximum density values indicated on the associated verified test reports. Additional mix parameters are provided for comparative purposes.

The laboratory resilient modulus results determined at the 2 Hz loading frequency were analyzed to determine if meaningful equations could be develop predict M_R as a function of specific mix variables in a similar fashion to the Witczak equation (Eqn 1.1). Project mix data was available for all of the terms of Eqn 1.1 with the exception of absolute viscosity at 70F. For a fixed loading frequency of 2 Hz (F=2), Eqn 1.1 can be rewritten as:

$$\log Eac = 7.381857 + 0.025619\, P_{200} - 0.03476\, Vv + 0.070377\, \eta_{70°F,10\wedge6} - 0.000877\, t_p^{\ 1.45}\, P_{ac}^{\ 0.5} \qquad \text{Eqn 4.1}$$

Neglecting the viscosity term, Eqn 4.1 can be re-written in a more general form as:

$$\log M_R = A + B\, P_{200} + C\, Vv + D\, t_p^{\ 1.45}\, P_{ac}^{\ 0.5} \qquad \text{Eqn 4.2}$$

For HMA mixtures with binders of equal PG grading, the absolute viscosity value may be considered similar and thus the contribution of this term for predicting M_R would be captured in the regression constant A. Available laboratory data was regressed using the general model form of Eqn 4.2 to determine if meaningful predictive equations could be developed. Using all available data, the resulting general model is:

$$\log M_R = 6.834506 + 0.018151\, P_{200} - 0.043790\, Vv - 0.000856\, t_p^{\,1.45}\, P_{ac}^{\,0.5} \qquad \text{Eqn 4.3}$$

Equation 4.3 was developed based on 143 discrete temperature/resilient modulus data pairs and has a coefficient of determination, R^2 = 93.34% and SEE = 0.138.

Figure 4.4.1 provides a comparison of predicted versus measured resilient moduli values using the general model form for all included data. While the agreement is good, the general model does not take into account the variable binder grades used within these mixes which would be expected to influence the viscosity nor does the general model specifically address mix E-type, NMAS, gradation class or coarse aggregate type. The three data clusters represent the three mix temperatures used during laboratory testing.

To minimize prediction errors, separate regression equations were developed for meaningful subsets of the data. Because of the predominant use of PG 58-28 binders in Wisconsin, regression models were first explored for this subset of data, representing 116 of the 143 total data pairs. Based on all available data with PG 58-28 binders, the following general model was developed:

$$\text{All PG 58-28:}\ \log M_R = 6.448936 + 0.105301\, P_{200} - 0.039568\, Vv - 0.000847\, t_p^{\,1.45}\, P_{ac}^{\,0.5} \qquad \text{Eqn 4.4}$$

The coefficient of determination (R^2) for Eqn 4.4 is 93.90% and SEE = 0.130, which is slightly improved from the global model. Figure 4.4.2 illustrates the comparative plot of predicted versus measured resilient modulus. This data set was further investigated to determine if prediction errors could be minimized by considering the predominant coarse aggregate type, resulting in the following:

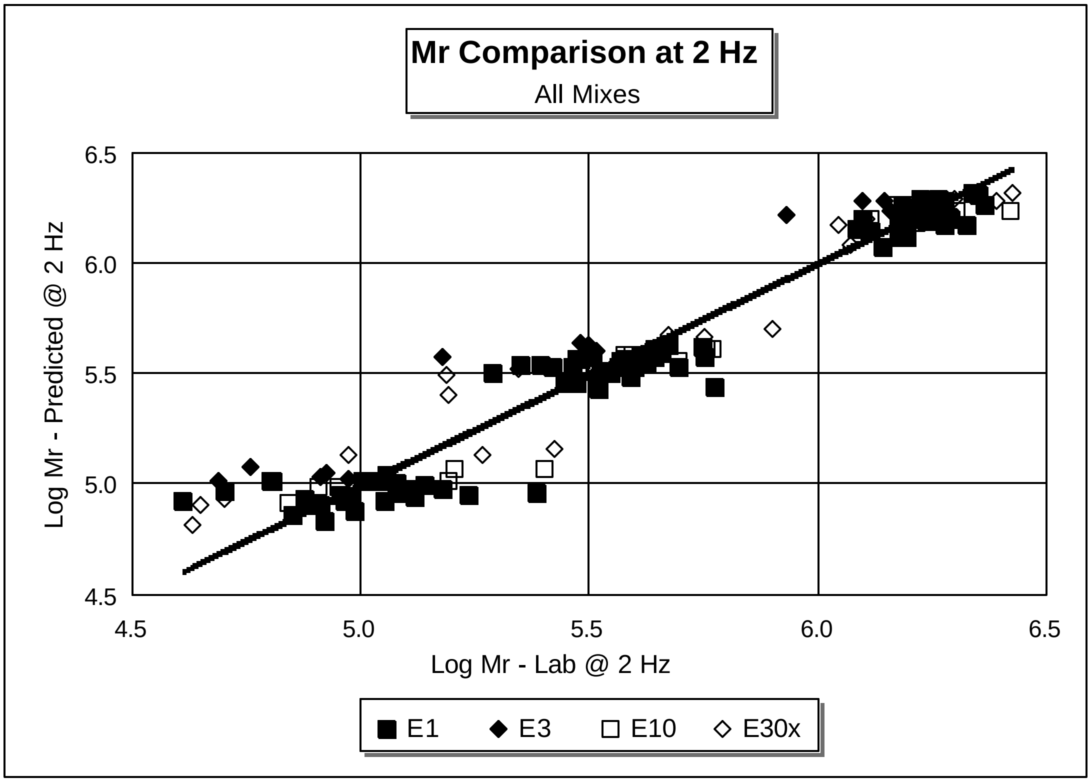

Figure 4.4.1 Comparison of Mr values using the Global Model

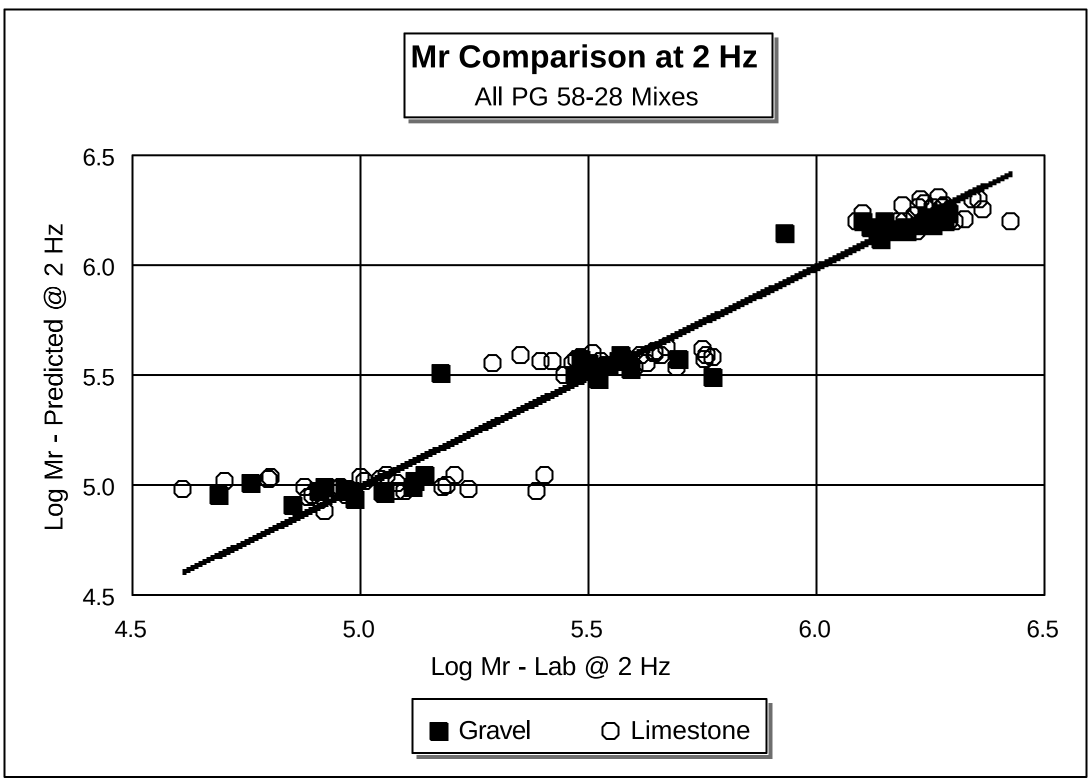

Figure 4.4.2 Comparison of Mr values using the General PG 58-28 Model

PG58-28, Gravel:

$$\log M_R = 6.035518 + 0.202680 P_{200} - 0.036173 Vv - 0.000853 t_p^{1.45} P_{ac}^{0.5} \quad \text{Eqn 4.5}$$

PG 58-28, Limestone:

$$\log M_R = 8.169317 - 0.262713 P_{200} - 0.069675 Vv - 0.000839 t_p^{1.45} P_{ac}^{0.5} \quad \text{Eqn 4.6}$$

The coefficients of determination (R^2) for Eqns 4.5, and 4.6 are 96.76%, and 96.28%, respectively and the SEE values are 0.095 and 0.104. Figure 4.4.3 illustrates a comparative plot of predicted versus measured resilient moduli using these aggregate specific equations and included data sets. As indicated by the regression statistics (R^2 and SEE), prediction errors are significantly reduced (compare to Figure 4.4.2).

Additional regression models were developed for other meaningful data subsets. All models are of the general form as Eqn 4.2. These additional models, as well as the above four models are provided in Tables 4.4.2 and 4.4.3 for comparative purposes between models and load frequency. These regression models represent powerful analysis tools for predicting resilient modulus over a range of mixture temperatures based on readily obtainable HMA mixture data. Care should be exercised when using models developed from limited data sets and/or when extrapolating beyond the test temperatures of 39.2F to 104F which were used during laboratory testing and subsequent model development.

Figure 4.4.4 provides a comparative plot of predicted versus measured resilient modulus using the regression equations specific to mix type and NMAS. As shown, prediction errors are significantly reduced over those displayed in Figure 4.4.1.

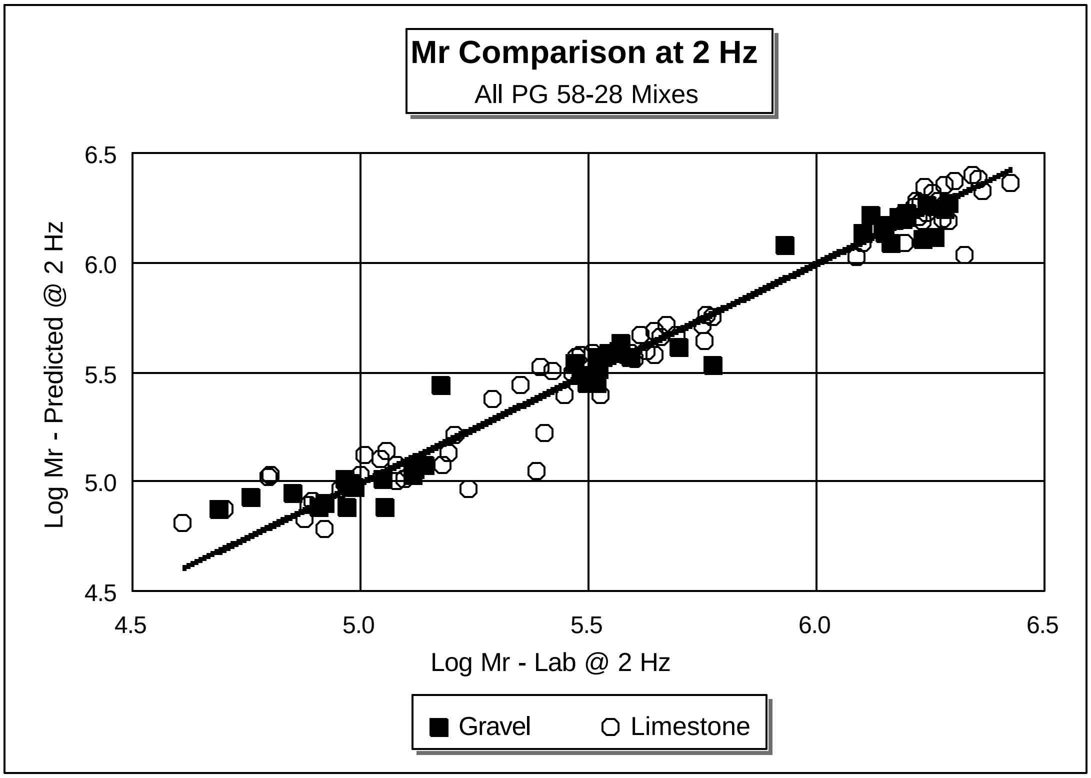

Figure 4.4.3 Comparison of Mr values using the PG 58-28 Aggregate Models

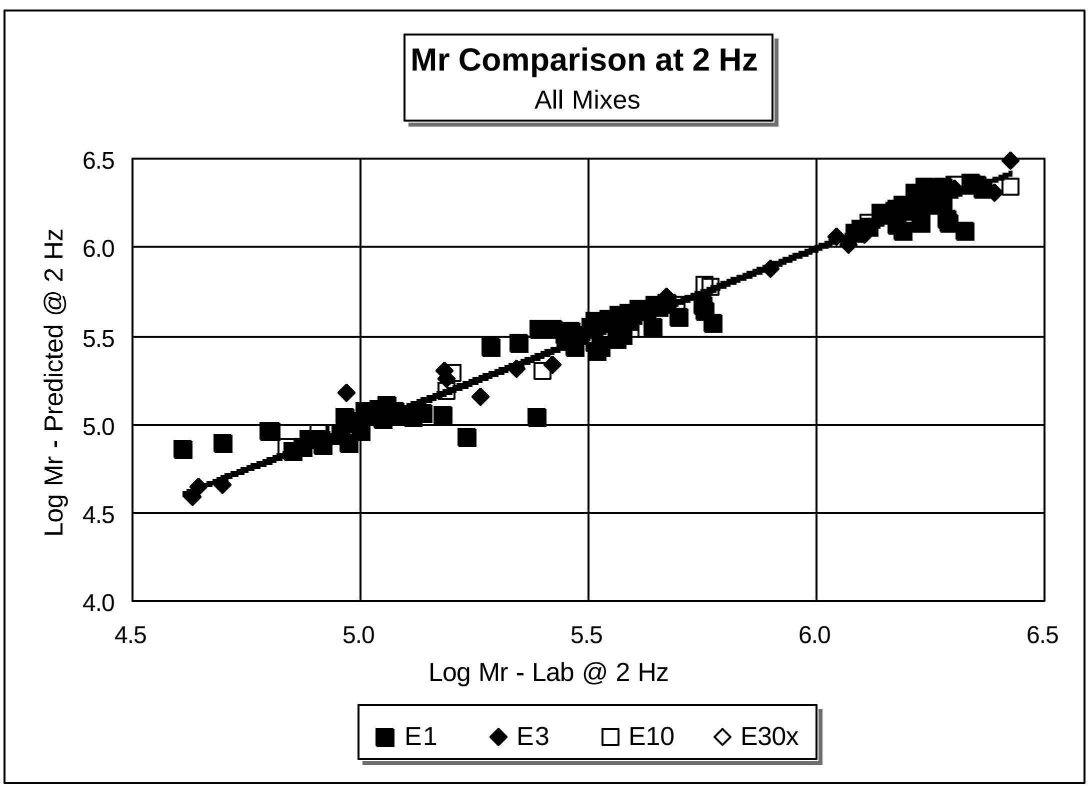

Figure 4.4.4 Comparison of Mr values using Specific Models

Table 4.4.2 Resilient Modulus Prediction Equations for 2Hz Loading

Model ID	Sample Size	A	B	C	D	R^2	SEE	Comment
R1	143	6.834506	0.018151	-0.043790	-0.000856	0.9334	0.138	All Mix Data
R2	116	6.448936	0.105301	-0.039568	-0.000847	0.9390	0.130	All PG 58-28 Mixtures
R3	44	6.035518	0.202680	-0.036173	-0.000853	0.9676	0.095	All PG 58-28 Mixtures w/ Gravel
R4	72	8.169317	-0.262713	-0.069675	-0.000839	0.9628	0.104	All PG 58-28 Mixtures w/ Limestone
R5	89	7.448738	-0.136400	-0.034885	-0.000845	0.9541	0.114	All E1 Mixtures (1)
R6	44	8.669426	-0.432215	-0.030246	-0.000846	0.9620	0.108	All E1 Mixtures w/ 12.5mm NMAS (1)
R7	45	5.032447	0.392769	-0.017537	-0.000847	0.9717	0.090	All E1 Mixtures w/ 19mm NMAS (1)
R8	18	7.161585	n.a. (2)	-0.110808	-0.000880	0.9585	0.118	All E3 Mixtures (1)
R9	9	6.011061	n.a. (2)	0.109790	-0.000871	0.9936	0.050	All E3 Mixtures w/ 12.5mm NMAS (1)
R10	9	6.892669	n.a. (2)	-0.071110	-0.000893	0.9523	0.141	All E3 Mixtures w/ 19mm NMAS (1)
R11	18	7.726693	-0.151496	-0.066780	-0.000811	0.9881	0.061	All E10 Mixtures (3)
R12	9	6.917428	n.a. (2)	-0.051730	-0.000846	0.9951	0.045	All E10 Mixtures w/ 12.5 mm NMAS (4)
R13	9	7.271528	n.a. (2)	-0.088349	-0.000770	0.9831	0.069	All E10 Mixtures w/ 19 mm NMAS (5)
R14	18	7.972338	-0.246051	-0.041293	-0.000911	0.9721	0.111	All E30x Mixtures (6)
R15	9	6.707624	n.a. (2)	-0.024280	-0.000959	0.9911	0.067	All E30x Mixtures w/ 12.5mm NMAS (7)
R16	9	8.168291	n.a. (2)	-0.261272	-0.000854	0.9612	0.118	All E30x Mixtures w/ 19mm NMAS (8)

See Table Notes in Table 4.4.4

Table 4.4.3 Resilient Modulus Prediction Equations for 1Hz Loading

Model ID	Sample Size	A	B	C	D	R^2	SEE	Comment
R1	143	6.823477	0.032199	-0.048537	-0.000959	0.9376	0.149	All Mix Data
R2	116	6.407551	0.133747	-0.047104	-0.000956	0.9419	0.141	All PG 58-28 Mixtures
R3	44	5.943412	0.278618	-0.058025	-0.000980	0.9630	0.113	All PG 58-28 Mixtures w/ Gravel
R4	72	8.137035	-0.246098	-0.072512	-0.000940	0.9610	0.115	All PG 58-28 Mixtures w/ Limestone
R5	89	7.208643	-0.057909	-0.045609	-0.000954	0.9530	0.130	All E1 Mixtures (1)
R6	44	9.289990	-0.605900	-0.008965	-0.000944	0.9611	0.122	All E1 Mixtures w/ 12.5mm NMAS (1)
R7	45	5.037300	0.410381	-0.023696	-0.000968	0.9681	0.108	All E1 Mixtures w/ 19mm NMAS (1)
R8	18	7.339340	n.a. (2)	-0.141962	-0.000979	0.9662	0.118	All E3 Mixtures (1)
R9	9	6.325058	n.a. (2)	0.059667	-0.000999	0.9957	0.041	All E3 Mixtures w/ 12.5mm NMAS (1)
R10	9	7.018833	n.a. (2)	-0.099547	-0.000961	0.9407	0.147	All E3 Mixtures w/ 19mm NMAS (1)
R11	18	7.768171	-0.147226	-0.074323	-0.000915	0.9890	0.059	All E10 Mixtures (3)
R12	9	6.906424	n.a. (2)	-0.052354	-0.000926	0.8312	0.028	All E10 Mixtures w/ 12.5 mm NMAS (4)
R13	9	7.431944	n.a. (2)	-0.105807	-0.000903	0.8113	0.080	All E10 Mixtures w/ 19 mm NMAS (5)
R14	18	7.986683	-0.260195	-0.038928	-0.000982	0.9603	0.129	All E30x Mixtures (6)
R15	9	6.581754	n.a. (2)	-0.016137	-0.001010	0.8104	0.099	All E30x Mixtures w/ 12.5mm NMAS (7)
R16	9	8.561152	n.a. (2)	-0.333631	-0.000948	0.7896	0.120	All E30x Mixtures w/ 19mm NMAS (8)

See Table Notes in Table 4.4.4

Table 4.4.4 Resilient Modulus Prediction Equations Table Notes

Table Note	Explanation
General	All predictive equations of the form: $\text{Log } M_R = A + B\ P_{200} + C\ Vv + D\ tp^{0.145}\ Pb^{.5}$
1	All mixtures used PG 58-28 binder
2	Model could not include term because there was no variation in the P_{200} for included mixtures
3	E10 mixtures include variable binder grading
4	All mixtures used PG 70-28 binder
5	All mixtures used PG 58-28 binder
6	E30x mixtures include variable binder grading
7	All mixtures used PG 76-28 binder
8	All mixtures used PG 70-22 binder

4.5 Comparison of Backcalculated and Laboratory Resilient Modulus Values

The use of pavement deflection data to estimate in-place HMA resilient modulus was presented in Chapter 3. This section explores the applicability of these estimates to provide meaningful comparisons to laboratory values. Where possible, resilient modulus estimation from deflection data offers a cost-effective method for analyzing in-service HMA pavements and providing necessary inputs into mechanistic performance analysis.

Backcalculated resilient moduli determined from the deflection algorithm developed during this project (Eqn 3.4.1) were compared to those measured during laboratory testing at the 2 Hz load frequency. The deflection algorithm was chosen for this comparison due to its ease of use for analyzing deflection data sets. Figure 4.5.1 provides a comparative plot of backcalculated and measured resilient moduli for the STH164-Slinger data sets. As shown, resilient moduli backcalculated from FWD data are consistently above the trend line of laboratory values, which is expected due to load frequency effects. A frequency adjustment factor of 1.93 was determined for this data set to result in an average prediction error of 1.000 (prediction error = MR_{FWD}/MR_{lab}). Figure 4.5.2 illustrates the comparison of adjusted FWD backcalculated resilient moduli versus laboratory values.

This adjustment process was completed for the remaining Wisconsin projects included in this research project. For some projects, a single adjustment factor was not sufficient due to variability of backcalculated and/or laboratory results from one location to another. In these cases, separate adjustment factors were developed to account for the location specific variability. Table 4.5.1 provides a summary of the FWD backcalculation adjustment factors developed for all projects. For the majority of cases, the frequency adjustment factor is in the range of 1.9 to 2.6, which is higher than predicted by the Witczak equation for an FWD load frequency of 4 Hz but in the range of values typically expected for analyses of this type. For some projects, adjustment factors are significantly higher indicating a very stiff base layer and/or shallow depth to bedrock, neither of which were measured in the field and thus cannot be accounted for.

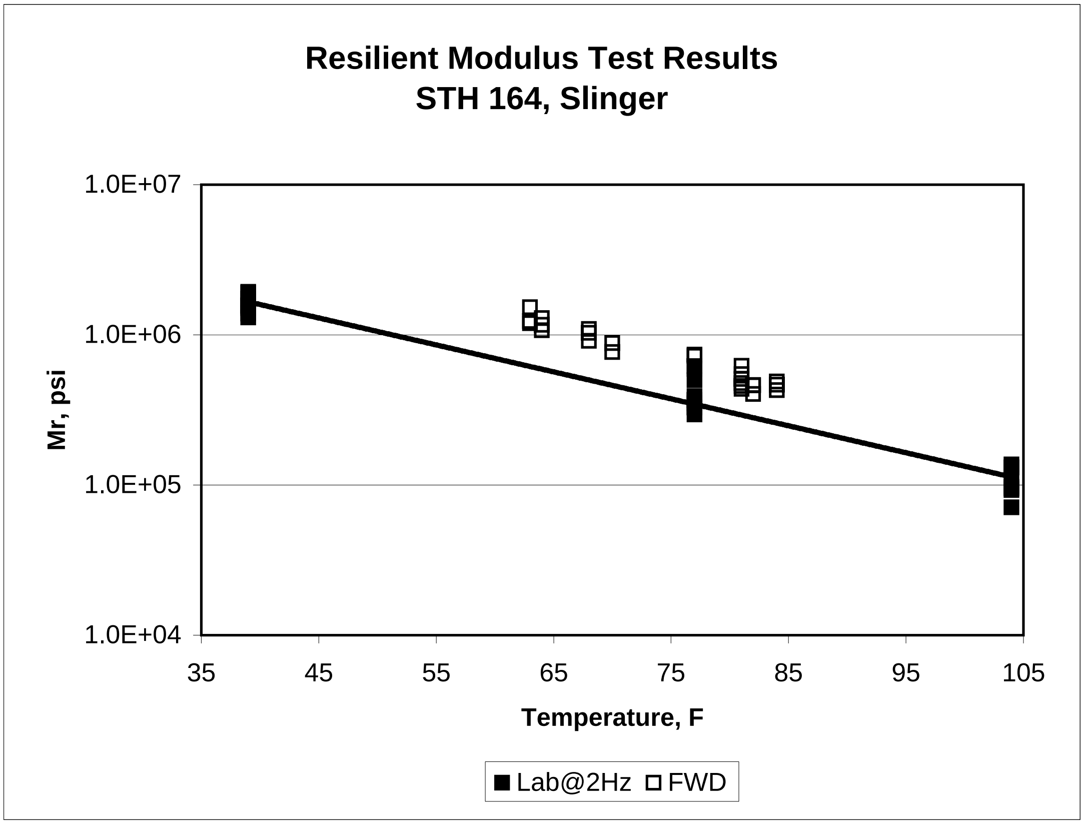

Figure 4.5.1: Comparison of Backcalculated and Laboratory Resilient Moduli

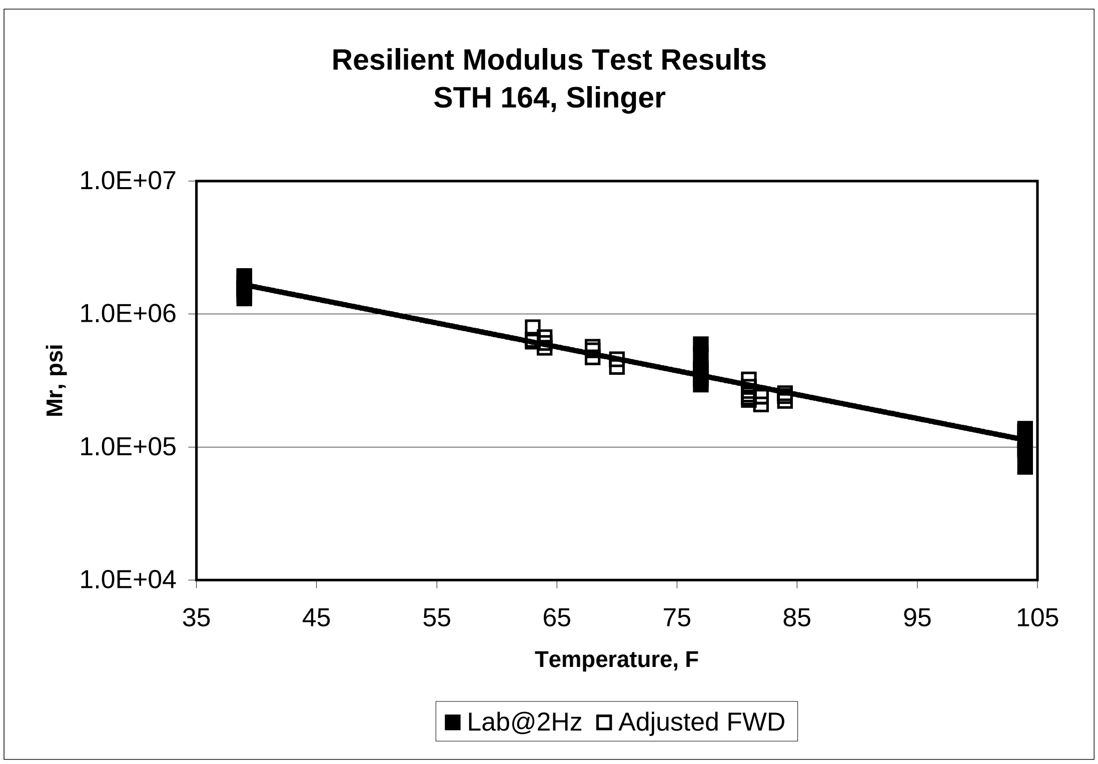

Figure 4.5.2: Comparison of Adjusted FWD and Laboratory Resilient Moduli

Table 4.5.1 - FWD Backcalculation Adjustment Factors

Site Location	Mix Type	Layer Position	Binder Type	NMAS	Coarse Aggregate	Gradation	Location	Adjustment Factor
STH 164 - Slinger	E-1 12.5mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	9.5 mm 12.5 mm	Gravel Gravel	Fine Fine	All	1.93
Waupun 03	E-1 19mm	Lower	PG 58-28	19 mm	Limestone	Fine	1 2 3	2.61 8.26 3.87
Waupun 04	E-1 12.5mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine	1 2 3	6.31 4.91 2.70
STH 164 - Waukesha	E-10 12.5mm E-10 19mm	Upper Lower	PG 70-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine	1 2 3	4.24 4.42 8.51
IH 90/94 - Kenosha	E-30x 12.5mm E-30x 25mm	Upper Lower	PG 76-28 PG 70-22	12.5 mm 25 mm	Limestone Limestone	Fine Coarse	1 2 3	1.87 2.48 2.08
STH 131 - Ontario	E-1 12.5mm	Upper	PG 58-28	12.5 mm	Limestone	Fine	All	2.22
STH 60 - Boscobel	E-1 12.5 mm E-1 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Limestone Limestone	Fine Fine	1 2 3	5.97 2.39 3.18
USH 8 - Rhinelander	E-3 12.5mm E-3 19mm	Upper Lower	PG 58-28 PG 58-28	12.5 mm 19 mm	Gravel Gravel	Fine Fine	All	6.75

4.6 Applications to Mechanistic Performance Analysis

During mechanistic analysis, the properties of the component pavement layers are adjusted throughout the year to account for the climatic effects of moisture and temperature. For HMA layers, mix temperature variation is the most important climatic factor to consider and the modulus-to-temperature relation of the mix is utilized to assign appropriate HMA material properties. Figure 4.6.1 illustrates the modulus-to-temperature trends developed from laboratory test data for mixtures containing PG 58-28 binders. Shown is the general trend, computed with the R2 model, and aggregate specific trends for gravel and limestone aggregates, computed by the R3 and R4 models, respectively, using baseline values of P_{200} = 4.2 %, Vv = 6.7 % and Pb = 5.2 %.

R2 $\quad \text{Log } M_R = 6.834506 + 0.018151\, P_{200} - 0.043790\, V_v - 0.000856\, t_p^{0.145}\, P_b^{0.5}$

R3 $\quad \text{Log } M_R = 6.035518 + 0.202680\, P_{200} - 0.036173\, V_v - 0.000853\, t_p^{0.145}\, P_b^{0.5}$

R4 $\quad \text{Log } M_R = 8.169317 - 0.262713\, P_{200} - 0.069675\, V_v - 0.000839\, t_p^{0.145}\, P_b^{0.5}$

The baseline values represent the averages of all the PG 58-28 mixtures tested. As shown in Figure 4.6.1, there is no noticeable variation in the different models using these baseline values.

Figures 4.6.2 and 4.6.3 were prepared for illustrative purposes to illustrate the effects of input values on the model trend lines. Figure 4.6.2 was prepared using P_{200} = 4.5 %, Vv = 10.4 % and Pb = 6.0 %, which were the maximum measured values for each parameter, while Figure 4.6.3 was prepared using P_{200} = 3.3 %, Vv = 4.1 % and Pb = 4.6 %, which were the minimum measured values for each parameter. As shown in these figures, there is significant separation between the various trend lines, which reinforces the need for mixture specific data (P_{200}, V_v, P_b) when developing modulus-to-temperature relations.

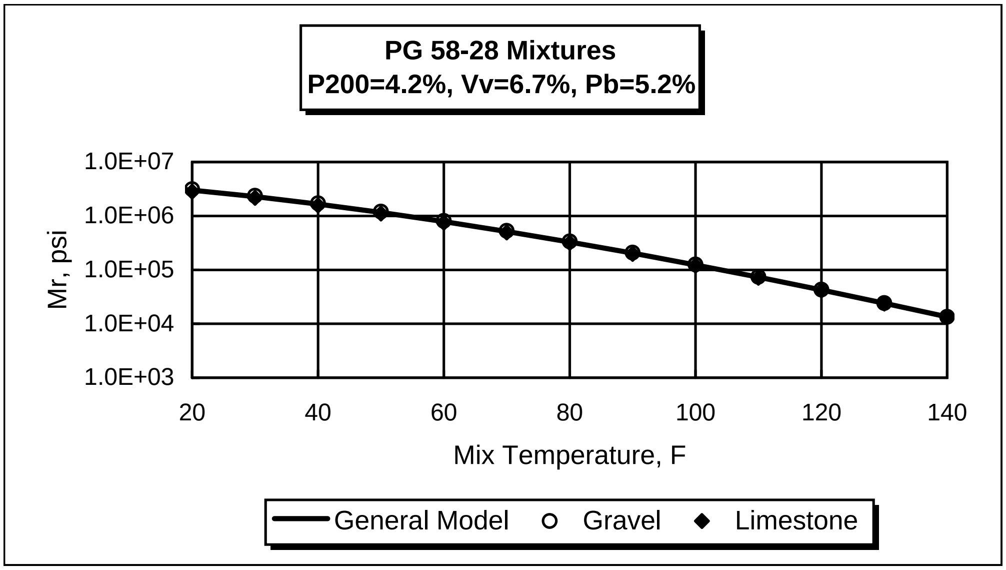

Figure 4.6.1 Mr vs Temperature Trends for the PG 58-28 Models

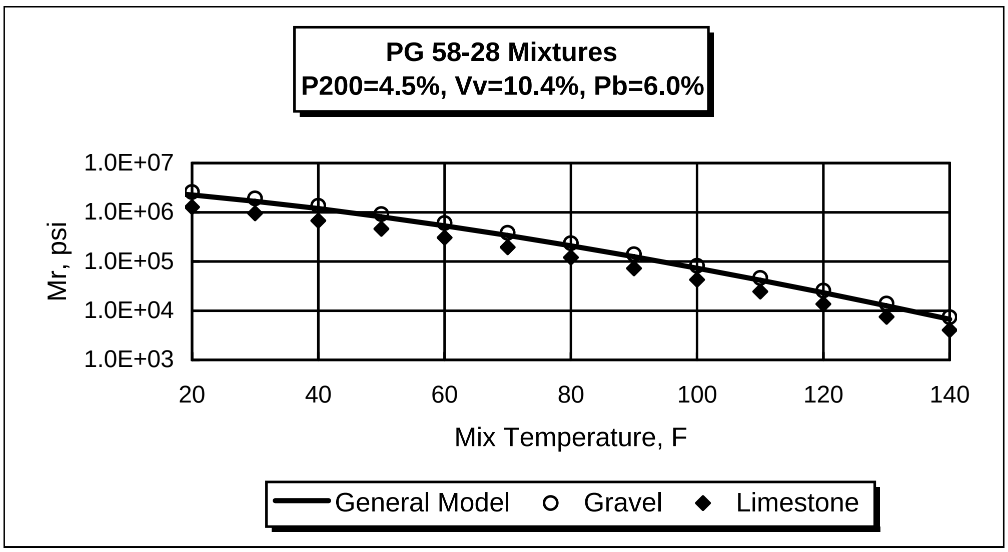

Figure 4.6.2 Mr vs Temperature Trends for the PG 58-28 Models

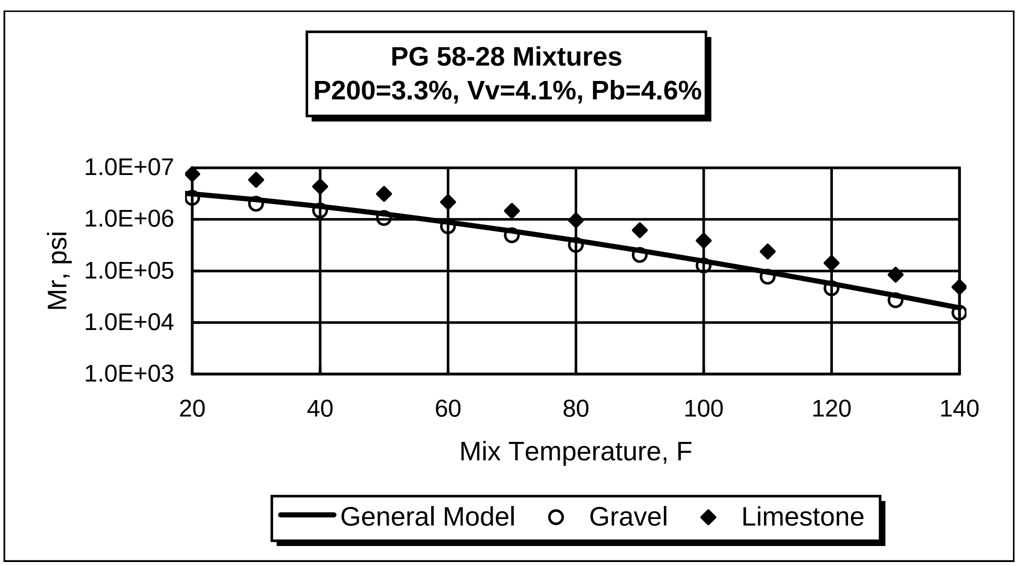

Figure 4.6.3 Mr vs Temperature Trends for the PG 58-28 Models

In additional to mixture specific data, knowledge of the expected mix temperature variations is required to allow for estimations of HMA resilient modulus. The Asphalt Institute developed a simple model to estimate mean monthly HMA mix temperatures as a function of the mean monthly air temperature. This model is of the form:

$$MMPT = MMAT\, [\, 1 + (\, 1 / \{\, z + 4\})] - [\, 34 / (z + 4)] + 6 \qquad \text{Eqn 4.6.1}$$

Where: MMPT = Mean monthly pavement temperature, ^{o}F

MMAT = Mean monthly air temperature, ^{o}F

z = Depth from pavement surface, inches

Eqn 4.6.1 can easily be applied to estimate mean monthly mid-depth mix temperatures for any location where mean monthly air temperature data is available. For illustrative purposes, hourly climatic data was obtained for five Wisconsin weather stations covering the period from 1976 - 1995. This data was analyzed to develop 20-year average MMAT values for each site and is provided in Table 4.6.1. Using mixture data, HMA layer thickness, MMAT and the appropriate modulus-to-temperature regression equation, monthly variations in HMA resilient modulus can be developed for use in mechanistic performance analysis. Table 4.6.2 illustrates the results of this application process for a 6-inch HMA layer constructed in the Madison area using the general PG 58-28 model.

Table 4.6.1 20-Year Mean Monthly Air Temperatures for Selected Wisconsin Cities

Month	Madison	Milwaukee	Eau Claire	La Crosse	Green Bay
January	17.4	20.6	12.6	15.7	16.0
February	22.6	25.0	18.7	21.5	20.5
March	34.7	35.4	31.9	34.5	31.6
April	47.0	45.6	46.2	48.4	44.4
May	59.2	56.8	59.5	60.8	57.1
June	67.9	66.3	67.5	69.4	65.9
July	72.4	72.2	71.9	73.5	70.7
August	69.3	70.3	68.9	70.7	67.9
September	60.4	62.4	59.4	61.6	59.1
October	48.0	50.0	46.3	48.6	46.8
November	34.6	37.4	31.1	33.9	33.6
December	22.7	25.7	17.7	21.0	21.2

Table 4.6.2 Example Monthly HMA Resilient Moduli for Madison, Wisconsin

Month	MMAT	MMPT[1]	M_R[2]
January	17.4	21.0	2,143,029
February	22.6	27.0	1,831,744
March	34.7	40.8	1,196,254
April	47.0	54.9	721,082
May	59.2	68.8	410,334
June	67.9	78.7	265,477
July	72.4	83.9	209,805
August	69.3	80.3	246,911
September	60.4	70.2	387,028
October	48.0	56.0	690,031
November	34.6	40.7	1,200,848
December	22.7	27.1	1,825,917

Notes: (1) Estimated mid-depth temperature for 6-inch HMA layer (z=3)
(2) M_R values computed using the general PG 58-28 model with P_{200}=3%, Vv=7%, Pb=5%

CHAPTER 5
SUMMARY, CONCLUSIONS & RECOMMENDATIONS

5.1 Summary

This report provides the results of research conducted to develop modulus-to-temperature relations for HMA mixtures used in Wisconsin. Field and laboratory data from seven selected HMA pavement projects were included in this study, representing a range of HMA mixture types and binder gradings. An analysis of collected deflection data and laboratory resilient modulus testing was presented. The results of the laboratory resilient modulus testing were used to develop a series of prediction equations that are applicable for estimating temperature-specific HMA resilient moduli trends based on fines content (P_{200}), void content (V_v), binder content (P_b) and test frequency. General prediction models were developed based on all available data for each load frequency used. Additional refined models were also developed to capture observed variations in binder grade, mixture type, coarse aggregate type, and nominal maximum aggregate size (NMAS).

The analysis of deflection data provided a simple means for utilizing surface deflections to estimate the in-place HMA resilient modulus using a backcalculation algorithm. General agreement was observed between backcalculated and laboratory values; however unknown base and bedrock information was shown to lead to significant backcalculation errors.

This report also provides a simple means for utilizing climatic data, in the form of mean monthly air temperatures, to develop monthly estimates of HMA resilient moduli based on readily obtainable mixture data. These monthly estimates are appropriate for use as inputs in a mechanistic performance analysis.

5.2 Conclusions

Based on the findings presented in this report, the following conclusions can be stated:

1) The developed regression equations are appropriate for predicting HMA moduli versus temperature variations which closely match values obtained from laboratory testing at load frequencies of 1 Hz and 2 Hz. Better agreement is generally obtained using the refined models as compared to the general models.

2) For the mixtures analyzed, modulus-to-temperature relations were developed using inputs for key mixture properties including fines content (P_{200}), air voids (V_v) and binder content (P_b). In general, the HMA resilient modulus increases at any temperature as the P_{200} increases, the percentage of binder decreases, and/or the percent voids decreases.

3) Estimation of HMA resilient modulus from deflection data can provide reasonable estimates of laboratory measurements; however, variations in the thickness and/or stiffness of lower pavement layers may significantly affect analysis results. Care must be exercised when using deflection data as the sole source of information for any given project.

4) Monthly HMA moduli can be readily predicted based on prevailing air temperatures and HMA layer thickness using the developed regression equations which are based on readily obtainable mixture data. These monthly moduli values represent valuable inputs for mechanistic performance analyses.

5.3 Recommendations

For the purposes of mechanistic performance analyses using HMA resilient modulus values, it is recommended that mixture-specific prediction equations (E-type, NMAS) be utilized to develop modulus-to-temperature relations. These equations provide better agreement to laboratory test values for all mixtures investigated during this project and are based on readily obtainable project data for fines content (P_{200}), air voids (V_v), and binder content (P_b). The projects included in the model development were weighted towards mixtures using PG 58-28 binders (116 of 143 samples), and thus it is recommended that more study be conducted on mixtures using other PG binder grades to confirm the appropriateness of the general model to predict the modulus-to-temperature trends for these mixtures. It is also recommended that site or regional weather data be used during project analysis to better define the monthly modulus-to-temperature variations.

The resilient modulus relations developed during this project can serve as surrogate values prior to the development of dynamic modulus relations which are required for input to the proposed AASHTO mechanistic-empirical design procedures.

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