Wisconsin Highway Research Program

Development of Rational Overlay Design Procedures for Flexible Pavements

SPR# 0092-00-05

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September 2005

WHRP 05-12

WISCONSIN HIGHWAY RESEARCH PROGRAM #0092-00-05

DEVELOPMENT OF RATIONAL OVERLAY DESIGN PROCEDURES FOR FLEXIBLE PAVEMENTS

FINAL REPORT

by

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Submitted to:

WISCONSIN DEPARTMENT OF TRANSPORTATION

September 2005

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ACKNOWLEDGEMENTS

The authors greatly appreciate the support, advice, and technical contributions received from members of the WHRP Flexible Pavement Technical Oversight Committee during the conduct of this study.

Technical Report Documentation Page

1. Report No.	2. Government Accession No	3. Recipient's Catalog No
WHRP 05-12		

4. Title and Subtitle	5. Report Date
Development of Rational Overlay Design Procedures for Flexible Pavements	September 2005
	6. Performing Organization Code
7. Authors	8. Performing Organization Report No.
James A. Crovetti	
9. Performing Organization Name and Address	10. Work Unit No. (TRAIS)
Marquette University P.O. Box 1881 Milwaukee, WI 53201-1881	11. Contract or Grant No. WHRP Project 0092-00-05
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
Wisconsin Department of Transportation Division of Transportation Infrastructure Development Research Coordination Section 4802 Sheboygan Avenue Madison, WI 53707	Final Report, Jan 2000 – Sep 2005 14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

This report presents the findings of a research study conducted to develop procedures for the design of structural HMA overlays over existing flexible pavements in Wisconsin. The recommended procedures are presented in a hierarchal approach to allow the user the flexibility of estimating the effective structural number of an in-place HMA pavement based on visual and/or nondestructive deflection testing data and to develop overlay thickness requirements based on the structural deficiency approach.

Techniques for estimating the effective structural number of an existing pavement based on surface deflection are presented. The equations presented in the 1993 AASHTO Design Guide were modified to enhance their applicability. Alternate deflection-based techniques were also developed to allow for the estimation of effective structural number without pavement thickness information.

The overlay design procedures were developed to maintain consistency with the current WisDOT practice of new flexible pavement design based on the 1972 AASHO design equation.

17. Key Words	18. Distribution Statement
HMA Pavements, Falling Weight Deflectometer, Effective Structural Number, Overlay Design	No restriction. This document is available to the public through the National Technical Information Service,5285 Port Royal Road, Springfield VA 22161

19. Security Classif.(of this report)	19. Security Classif. (of this page)	20. No. of Pages	21. Price
Unclassified	Unclassified	59	

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

EXECUTIVE SUMMARY

Project Summary

This research study consists of developing rational overlay design procedures for flexible pavements that are consistent with current procedures utilized by the Wisconsin Department of Transportation for the design of new Hot Mix Asphalt (HMA) pavements. The recommended procedures are presented in a hierarchal approach to allow the user the flexibility of estimating the effective structural number of an in-place HMA pavement based on visual and/or nondestructive deflection testing data and to develop overlay thickness requirements based on the structural deficiency approach. The procedures are recommended for the design of structural HMA overlays on existing flexible pavement systems.

Project Background

The current WisDOT practice for the design of structural asphalt concrete overlays on existing flexible pavements pavement is largely empirical. Little guidance is provided in Procedure 14-10-30 of the WisDOT Facilities Development Manual (FDM) for quantifying the structural integrity of existing flexible pavement systems. This results in overlay thicknesses which can vary from project to project even when other pavement design parameters are the same, potentially resulting in rehabilitated pavement sections that do not perform as desired or a less than optimum use of valuable resources. This research was conducted to provide a consistent, objective methodology for determining the required thickness of structural HMA overlays to prolong the service life of existing flexible pavement systems.

Process

Literature was reviewed from various national sources detailing the best practices for design structural HMA overlays of existing flexible pavements. Overlay design methodologies utilized by surrounding states were also investigated. Literature relating to the conduct of visual and nondestructive deflection testing surveys was also reviewed to develop protocol beneficial to the study objectives. After considering all factors, it was deemed appropriate to develop overlay design methodologies that would be consistent with the current WisDOT practice for the design of new flexible pavements based on the structural number concept. A significant effort was expended on the development and analysis of deflection data generated by computer modeling of a factorial of flexible pavement structures. Statistical analyses of all generated data were conducted to develop predictive equations for estimating the effective structural number of existing HMA pavements.

Findings

The analyses conducted as part of this research resulted in the following findings:

(1) The design of structural HMA overlays of existing flexible pavements can be integrated within current WisDOT procedure for the design of new flexible pavements by utilizing the structural deficiency approach. This process establishes the required overlay thickness based on the difference between the effective structural number, SN_{eff}, of the existing pavement existing and the structural number required for a new flexible pavement design.

(2) The SN_{eff} of existing flexible pavements can be established based on deflections, distress, or ride quality. The use of deflection data is considered appropriate for pavements with

design traffic loadings in excess of 1 million ESALs. For lightly trafficked pavements the SN_{eff} may be developed without the use of deflection data. The accuracy of SN_{eff} estimations can be improved by including pavement layer thickness data obtained through selective coring; however, all analysis techniques have associated errors.

(3) Modified deflection-based SN_{eff} analysis procedures were developed based techniques presented in the 1993 AASHTO Guide for the Design of Pavement Structures. These procedures provided the best correlations between SN_{eff} and input SN using deflection data generated during computer modeling of a large pavement factorial. These procedures are somewhat cumbersome to apply and are best suited for analysis when pavement layer thicknesses are known. Based on the results presented, these procedures were shown to provide overlay thickness recommendations which were within ½ inch of "truth", as represented by exact component analysis of the pavement structures investigated during computer modeling, for 90% of the structures investigated.

(4) Alternative deflection-based analysis techniques developed as part of this research were also shown to provide reasonable correlations between SN_{eff} and input SN using deflection data generated during computer modeling of the large pavement factorial. These procedures are easier to apply and do not require knowledge of the in-place pavement layer thicknesses. Based on the results presented, these procedures were shown to provide overlay thickness recommendations which were within ½ inch of "truth" for 40% of the pavement structures investigated and within 1 inch of truth for 84% of the structures. These values were shown to be comparable to the modified AASHTO approach if the assumed pavement thickness is in error by 10%.

Recommendations

Based on the findings from this research, it is recommended that the structural deficiency approach be implemented for the design of structural HMA overlay thickness requirements for existing flexible pavements. The procedures presented in this report are considered appropriate for establishing thickness requirement for structural HMA overlays. Thickness requirements resulting from the application of these methods are not intended to supersede minimum/maximum HMA layer thickness guidelines as detailed in the WisDOT Standard Specifications, Section 460.3.2.

The structural deficiency approach utilizes both the effective structural number, SN_{eff}, of the existing pavement and the structural number required for new design. It is recommended that the deflection-based analysis procedures presented in Section 2.4.5 of this report be promoted to estimate the effective structural number, SN_{eff}, of the existing flexible pavement that are projected to carry at least 1 million ESALs after overlay. During initial implementations, both the modified AASHTO and revised AUPP-Eri should be utilized to establish SN_{eff} and asses the impacts of analyses with and without available coring data.

For lightly trafficked pavements with less than 1 million design ESALs, it is recommended that the SN_{eff} be established based on the deflection based-analysis techniques or a component analysis based on layer thickness and existing pavement distress. The guidelines presented by AASHTO for the selection of structural layer coefficients based on existing distress are recommended for use when deflection data is unavailable and the component analysis is selected.

The recommended overlay thickness design procedures are compatible with the current WisDOT procedures for the design of new flexible pavements, as published within Procedure 14-10-

5 of the Facilities Development Manual (FDM). When deflection data are utilized, the field subgrade modulus is determined directly from deflections. This value may require seasonal adjustments depending on the time of deflection testing as well as conversion to a representative soil support value following standard WisDOT procedures.

The overlay design procedures presented in this report may be utilized to develop thickness requirements for any user-supplied design life. The practical limitation for these procedures is a 20-year design life which is consistent with the maximum design life currently assumed for the design of traditional HMA pavements in Wisconsin following FDM Procedure 14-10-5. Shorter design lives can be considered by developing new pavement SN requirements using projected traffic levels within the 1972 AASHTO equation currently used by WisDOT.

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INTRODUCTION

1.1 Background and Problem Statement

The current WisDOT practice for the design of structural asphalt concrete overlays on existing flexible pavements pavement is largely empirical. Little guidance is provided in Procedure 14-10-30 of the WisDOT Facilities Development Manual for quantifying the structural integrity of existing flexible pavement systems. This results in overlay thicknesses which can vary from project to project even when other pavement design parameters are the same, potentially resulting in rehabilitated pavement sections that do not perform as desired or a less than optimum use of valuable resources.

The primary objectives of this research are to (1) develop a rational procedure for quantifying the effective structural capacity of existing flexible pavements, (2) recommend guidelines for the collection and use of data to determine the effective structural capacity of existing flexible pavements, (3) recommend procedures for designing structural asphalt concrete overlays on existing flexible pavement systems, and (4) recommend guidelines for implementing these procedures throughout the State of Wisconsin. These products will provide a consistent, objective methodology for determining the required thickness of asphalt concrete overlays to increase the structural capacity of existing flexible pavement systems.

This report presents the findings of a literature review of published flexible pavement overlay design procedures as well as the results of a survey of overlay design procedures used in States surrounding Wisconsin. Based on these findings, recommendations for key data elements to be included in the WisDOT overlay design procedures are presented.

1.2 Overlay Design Methodologies

HMA overlays are predominantly used to improve the structural capacity and/or functional requirements (i.e., skid resistance or ride quality) of existing pavements. Overlays may be required due to excessive deterioration of the existing pavement or because current or revised traffic projections indicate the existing pavement is deficient in structural capacity to provide adequate performance. Overlay thicknesses may be specified based on simple engineering judgment or policy decisions or designed based on structural deficiency, limiting deflection, or limiting fatigue damage approaches.

The most commonly used overlay design approach is the structural deficiency approach, whereby the overlay must satisfy a deficiency between the required traffic capacity of an existing pavement over some future time period and the actual traffic capacity of that pavement over the same time period. The current AASHTO overlay design procedures (*1*) are based on the structural deficiency concept, as are other overlay design procedures developed by agencies such as the Corps of Engineers (*2*) and the Asphalt Institute (*3*). Structural deficiency approaches have dominated overlay design to date because widely accepted performance models (such as the AASHTO models) are available for asphalt pavements but generally acceptable performance models are not available for overlaid pavements.

The second most commonly used overlay design method is based on the maximum deflection approach developed by the Asphalt Institute (*3*). In this method, total pavement deflection is related to the pavement's service life, expressed in terms of allowable 18-kip ESALs. Overlay thickness designs are developed to reduce total pavement deflections to tolerable levels based on the projected ESALs over the design analysis period.

A third overlay design method which is gaining wide acceptance is based on the limiting fatigue damage concept using mechanistic principles. In this approach, a stress-strain analysis of the existing pavement structure is conducted and the remaining service life, in terms of fatigue cracking and/or subgrade rutting, is estimated based on empirical transfer functions. Overlay thickness designs are developed to limit fatigue cracking and/or rutting of the overlaid pavement to tolerable levels based on the projected traffic over the design analysis period.

The focus of this research is the development of objective procedures for designing structural HMA overlays on existing flexible pavements which are 1) compatible with current WisDOT pavement design methods, and 2) utilize pavement performance data (i.e., IRI PSI, PDI, and deflections) commonly collected in Wisconsin. A framework for these procedures, based on published design methods, is presented in the following sections.

1.2.1 Structural Deficiency Approach

The basic concept of the structural deficiency approach is that the HMA overlay represents the difference between the structure required for a new pavement and the existing pavement structure. Inherent in this approach is the establishment of the in situ pavement's structural capacity, commonly termed the effective structural capacity of the existing pavement. This effective structural capacity must be established within the context of the design method used for determining the required new pavement structure. In other words, if the new pavement design is expressed in terms of a full-depth HMA layer thickness, then the effective structural capacity must be converted to an equivalent HMA layer thickness. On the other hand, if a new pavement structure is expressed in terms of a required structural number (SN), the in situ structural capacity must be converted to an effective SN.

The 1993 AASHTO Design Guide (*1*) provides three approaches for estimating the effective structural capacity of in situ flexible pavements, provided in terms of the effective structural number, SN_{eff}. These methods are based on visual assessment, nondestructive deflection testing, and/or remaining life analyses. Each analysis method is compatible with the SN pavement design concept promoted by AASHTO and used by WisDOT. Deflection testing is strongly recommended for this analysis.

The Asphalt Institute (*3*) provides two methods for estimating the effective structural capacity of in situ flexible pavements, provided in terms of the effective HMA thickness of the in situ pavement. These methods are based on the Present Serviceability Index (PSI) of the existing pavement or a component analysis based on visual distress.

The U.S. Army Corps of Engineers (*2*) utilizes the existing HMA pavement thickness without alteration for condition assessments when determining overlay thickness requirements.

1.2.2 Visual Pavement Assessments

A visual pavement assessment requires a detailed condition survey of pavement distress to identify the type, amount, severity, and location of key distress types. Subdrainage surveys and materials coring and testing are also recommended as part of this assessment. The results of the condition survey are used by AASHTO to conduct a component analysis of the existing pavement using the structural number equation:

$$SN_{eff} = a_i \, D_i \, m_i \qquad \text{Eq. 1.1}$$

where:
D_i = thickness of in situ pavement layer i
a_i = corresponding structural coefficient of layer i
m_i, = drainage coefficient for layer i

The Asphalt Institute utilizes condition information to compute the effective thickness of the in situ pavement using the equation:

$$h_e = \Sigma\, h_i\, C_i \qquad \text{Eq. 1.2}$$

where: h_e = total effective HMA thickness of existing pavement
h_i = thickness of pavement layer i
C_i = HMA conversion factor for pavement layer i

Limited guidance is provided for selecting appropriate structural layer coefficients or layer conversion factors. **Tables 1.2.1 and 1.2.2** provide appropriate layer coefficients suggested by AI *(3)* and AASHTO *(1)*.

Procedures for determining conversion factors for full-depth HMA pavements, based on the PSI of the existing pavement, are also provided by AI. For conservative analysis, this conversion factor can be computed for existing PSI values between 1.5 and 3.9 using the equation:

$$CF = 0.166 + 0.213\ PSI \qquad \text{Eq. 1.3}$$

where: CF = full-depth HMA conversion factor
PSI = existing PSI

Table 1.2.1: AI Conversion Factors for Determining Effective Thickness (*3*)

Material Description	Conversion Factor[a]
Well graded granular subbase or base with CBR > 20	0.1 - 0.2
Cement or lime-fly ash stabilized subbases and bases	0.2 - 0.3
Emulsified or cutback asphalt surfaces and bases that show extensive cracking, considerable raveling or aggregate degradation, appreciable deformation in the wheelpaths, and lack of stability	0.3 - 0.5
Emulsified or cutback asphalt surfaces and bases that exhibit some fine cracking, some raveling or aggregate degradation, and slight deformation in the wheelpaths but remain stable	0.5 - 0.7
Emulsified or cutback asphalt surfaces and bases that are stable, generally uncracked, show no bleeding, and exhibit little deformation in the wheelpaths	0.7 - 0.9
Asphalt concrete surface and base that exhibit appreciable cracking and crack patterns	0.5 - 0.7
Asphalt concrete surface and base that exhibit some fine cracking, have small intermittent cracking patterns and slight deformations in the wheelpaths but remain stable	0.7 - 0.9
Asphalt concrete, including asphalt concrete base, generally uncracked and with little deformation in the wheelpaths	0.9 - 1.0

[a]Originally meeting minimum specified strength and compaction requirements

Table 1.2.2: Suggested AASHTO Layer Coefficients (*1*)

Material	Surface Condition	Coefficient
AC Surface	Little or no alligator cracking and/or only low severity transverse cracking	0.35 - 0.40
	<10% low severity alligator cracking and/or <5% medium and high severity transverse cracking	0.25 - 0.35
	>10% low severity alligator cracking and/or <10% medium and high severity alligator cracking and/or >5-10% medium and high severity transverse cracking	0.20 - 0.30
	>10% medium severity alligator cracking and/or <10% high severity alligator cracking and/or >10% medium and high severity transverse cracking	0.14 - 0.20
	>10% high severity alligator cracking and/or >10% high severity transverse cracking	0.08 - 0.15
Stabilized Base	Little or no alligator cracking and/or only low severity transverse cracking	0.20 - 0.35
	<10% low severity alligator cracking and/or <5% medium and high severity transverse cracking	0.15 - 0.25
	>10% low severity alligator cracking and/or <10% medium and high severity alligator cracking and/or >5-10% medium and high severity transverse cracking	0.15 - 0.20
	>10% medium severity alligator cracking and/or <10% high severity alligator cracking and/or >10% medium and high severity transverse cracking	0.10 - 0.20
	>10% high severity alligator cracking and/or >10% high severity transverse cracking	0.08 - 0.15
Granular Base or Subbase	No evidence of pumping, degradation, or contamination by fines	0.10 - 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.0 - 0.10

1.2.3 Nondestructive Deflection Testing Assessments

Nondestructive deflection testing is recommended by AASHTO to provide data necessary to estimate the subgrade resilient modulus, M_R, and in situ effective structural number, SN_{eff}. Detailed deflection analysis procedures, using data provided by a heavy-load deflection device such as the falling weight deflectometer (FWD) are provided by AASHTO (*1*) and summarized here.

The AASHTO deflection analysis initially utilizes deflections recorded away from the applied load to provide an estimate of the field subgrade modulus, M_R., based on an integration of the Boussinesq point load equation. Assuming a Poisson's ratio of 0.5 for the subgrade, the equation for the field subgrade modulus is:

$$\text{Field } M_R = 0.24\ P / (r\ Dr) \qquad \text{Eq. 1.4}$$

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

Equation 1.4 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 author has 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 Eq. 1.4 can be used to calculate subgrade MR 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 1993 AASHTO Guide also presents the following equation for D_0, the deflection measured at the center of the FWD load plate:

$$D_0 = 1.5\ pa\ \{[1/(M_R\ ((1+((Tp/a)(Ep/M_R)^{1/3})^2))^{½})] + [(1-(1/((1 + (Tp/a)^2\ )^{½}\ )))/Ep\ ]\} \qquad \text{Eq. 1.5}$$

Where:	D_0	=	maximum deflection (at center of load plate) (mils)
	p	=	FWD plate pressure (psi)
	a	=	FWD plate radius (in)
	M_R	=	in-place subgrade modulus (psi)
	Tp	=	total thickness of pavement structure above subgrade (in)
	Ep	=	effective elastic modulus of the pavement structure (psi)

With the maximum measured deflection, D_0, the FWD plate pressure and radius, and the pavement thickness known, Eq. 1.5 can be used to solve for the effective pavement modulus Ep. Thus, combined use of Eqs. 1.4 and 1.5 can provide the solution to the backcalculation of elastic moduli for a two-layer system, using two deflection measurements (D_0 and Dr) to solve for two unknowns, namely M_R and Ep. Such a solution can be implemented in a computer spreadsheet program; however, Eq. 1.5 cannot be rearranged to solve for Ep directly. Ep can, however, be determined by iteration, that is, varying Ep until the calculated D_0 matches the measured D_0. This can also be accomplished using goal seeking functions available within spreadsheet applications.

The effective pavement modulus can then be used to estimate the effective structural number of the in-place pavement, SNeff, using the equation:

$$SN_{eff} = 0.0045\ Ep^{1/3}Tp \qquad \text{Eq. 1.6}$$

where: SN_{eff} = effective structural number of existing pavement
Tp = total pavement thickness, inches
E_p = effective pavement modulus, psi

Once SN_{eff} is established, the required SN for the overlay, and hence the required overlay thickness is simply computed as:

$$SN_{OL} = a_{OL} * D_{OL} = SN_f - SN_{eff} \qquad \text{Eq. 1. 7}$$

where: SN_{OL} = structural number required for new pavement
a_{OL} = structural coefficient for the HMA overlay
D_{OL} = HMA overlay thickness
SN_f = structural number required for new pavement
SN_{eff} = effective structural number of in situ pavement

1.2.4 Remaining Life Assessments

Techniques for estimating the effective structural capacity of an existing pavement based on remaining life estimates are provided by AASHTO and AI. The AASHTO procedures (*1,4*) utilize a past traffic analysis or an existing PSI analysis for this purpose while the AI procedures (*3*) utilize only deflection testing.

The AASHTO remaining life assessment procedures based on past traffic are most appropriate for estimating the remaining life of an original flexible pavement, i.e., no overlay has been applied. Where available, historic traffic data is used to compute ESAL applications to date. The designer must also determine the total ESALs to failure (PSI = 1.5) for the original pavement. Together, these two ESAL values are used to compute the percent remaining life using the equation (*1*):

$$RL = 100 \, [\, 1 - N_p / N_{1.5} \,] \qquad \text{Eq. 1.8}$$

where: RL = percent remaining life
N_P = total ESALs to date
$N_{1.5}$ = total ESALs to PSI=1.5

The effective structural number, SN_{eff}, of the existing pavement is computed based on the original pavement structural number, SN_{O}, and a condition factor, CF, using the equation:

$$SN_{eff} = CF * SN_O \qquad \text{Eq. 1.9}$$

The condition factor may be computed using the equation:

.
$$CF = 0.5 + 0.09 \, \text{Log RL} + 0.08 \, (\text{Log RL})^2 \qquad \text{Eq. 1.10}$$

The previous edition of the AASHTO Design Guide (*4*) also provided a method for estimating the remaining life of a pavement based on the Present Serviceability Index (PSI) and initial Structural Number, SN_o, of the pavement, as shown in **Figure 1.2.1**.

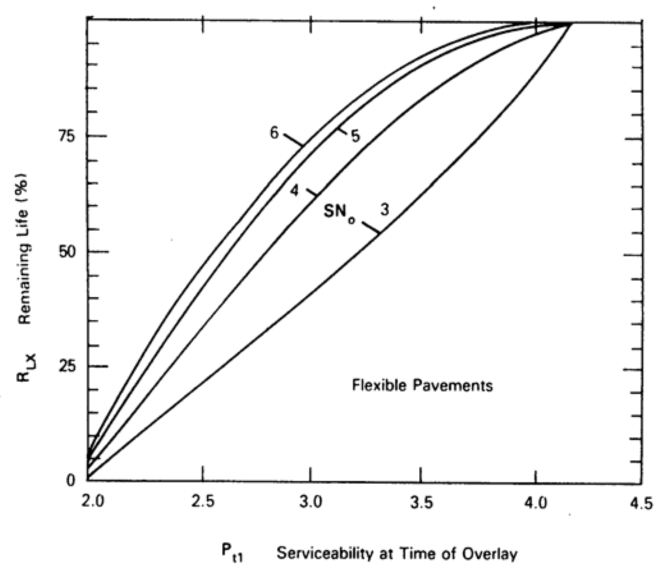


Figure 1.2.1: Remaining Life Based on PSI and SN_o (*4*)

Surface deflections are also used within the AI design method to provide an estimate of the remaining life of a pavement. The AI method was originally developed for use with the Benkelman beam using a rebound deflection test procedure. In this process, at least 10 deflection tests are required within a given design section and the design rebound deflection is calculated as the average plus two standard deviations, corrected for temperature and seasonal effects. The remaining life of the pavement is then computed by the equation:

$$ESAL_r = (1.0363 / \delta_{rd})^{4.1017} \quad \text{Eq. 1.11}$$

where: $ESAL_r$ = remaining life ESALs
δ_{rd} = design rebound deflection, inches

1.2.5 Maximum Deflection Approach

The maximum deflection approach, which is promoted by the Asphalt Institute (AI), is based on the concept that structural overlays are required to strengthen weak pavement systems to reduce pavement deflections to tolerable limits based on projected future traffic applications. This method was originally developed for use with the Benkelman beam using the rebound deflection test procedure discussed previously. The design rebound deflection is first used to estimate an equivalent pavement modulus for the in-place pavement system using the equation (*3*):

$$E_p = 1.5\ pa / \delta_{rd} \qquad \text{Eq 1.12}$$

where: E_p = equivalent pavement modulus, psi
p = contact pressure of load, psi
a = radius of load, inches
δ_{rd} = design rebound deflection, inches

Equation 1.12 is based on the assumption that the in-place pavement system can be modeled as an equivalent homogeneous half-space with a Poisson's ratio of 0.5. Based on the projected ESALs after overlay, the allowable pavement deflection is computed using the alternate form of Equation 1.11 (*3*):

$$\delta_{all} = 1.0363\ (ESAL_{OL})^{-0.2438} \qquad \text{Eq. 1.13}$$

where: δ_{all} = allowable pavement deflection, inches
$ESAL_{OL}$ = projected ESALs after overlay

Based on an assumed modulus and thickness for the HMA overlay, the expected deflection after overlay can be calculated using the previously determined E_p value using the equation (*3*):

$$\delta_{OL} = (1.5pa/E_p)*[\{1-(1+.8(h_{OL}/a)^2)^{-0.5}\}(E_p/E_{OL}) + \{1+(.8(h_{OL}/a)(E_{OL}/E_p)^{1/3})^2\}^{-0.5}] \qquad \text{Eq. 1.14}$$

where: δ_{OL} = expected deflection after overlay, inches
E_p = equivalent modulus of in-place pavement, psi
h_{OL} = assumed overlay thickness, inches
a = radius of applied load, inches
E_{OL} = assumed modulus of HMA overlay, psi

Equation 1.14 can be solved iteratively to determine the required overlay thickness, h_{OL}, to limit the expected post-overlay deflection, δ_{OL}, to the allowable deflection, δ_{all}, computed by Equation 1.13.

1.3 Overlay Design Methods Used in Surrounding States

A survey was conducted during the early phase of this project to identify key design engineers in States surrounding Wisconsin and to determine their current procedures utilized for overlay design. Prior to the initiation of this survey, a questionnaire was prepared to catalogue overlay design procedures utilized and to identify key data elements used for the characterizing the existing pavement structure. This questionnaire was discussed during initial phone contacts with the key design engineers and responses entered by the Marquette research staff. The completed questionnaires were then forwarded to the respective design engineers for verification and revision, as required. Revised questionnaires were returned by all design engineers contacted. This section presents a summary of the responses received.

1.3.1 Illinois Department of Transportation

Information relevant to the Illinois Department of Transportation (IDOT) procedures was provided by Mr. David Lippert. IDOT promotes the use of the Asphalt Institute's (AI) maximum deflection approach for establishing structural overlay thickness requirements. Based on projected traffic levels, overlay thicknesses are selected to reduce maximum surface deflections to tolerable levels using standard AI nomographs.

Surface deflections are obtained using a falling weight deflectometer. Deflection tests are conducted within the outer wheelpath of each travel lane using an applied load of approximately 9,000 lbs. A minimum of 30 tests per direction of travel are obtained, with a maximum test interval of 0.1 miles. Recorded maximum deflections are normalized to a 9,000 lb load and adjusted to represent critical season Benkelman beam rebound deflections at a standard pavement temperature of 70 ^oF using the formula:

$$BBD = 1.6\ FWD \times TAF \times CSAF \qquad \text{Eq. 1.15}$$

where: BBD = Benkelman beam rebound deflection, inches
FWD = normalized maximum FWD deflection, inches @ 9,000 lb load
TAF = temperature adjustment factor
CSAF = critical season adjustment factor

The deflection adjustment factors are established following standard AI procedures. Temperature adjustment factors are established based on the pavement thickness, the air and pavement surface temperatures recorded during testing, and the previous 5-day mean air temperature. Critical season adjustment factors are established based on soil type, pavement location, and time of testing using IDOT correlations.

The adjusted Benkelman beam deflections are utilized to compute the average deflection and standard deviation within the design section. These values are used to compute the representative rebound deflection, RRD, following standard AI procedures.

1.3.2 Indiana Department of Transportation

Information relevant to the Indiana Department of Transportation (InDOT) procedures was provided by Mr. Kumar Dave. InDOT promotes the use of the 1993 AASHTO procedures for determining overlay thickness requirements. However, for most applications, a visual assessment of distress data is used to indicate the need for an overlay and overlay thicknesses are specified based on engineering judgment. Surface deflections obtained with a falling weight deflectometer are utilized on a limited basis to establish both the design subgrade resilient modulus and the effective structural number of the existing pavement following standard AASHTO procedures. Visual observations of pavement condition are also used to validate the calculated effective structural number.

The required structural number after overlay is established based on a 20-year design scenario. The remaining life factor for the existing pavement, F_{rl}, is computed following AASHTO procedures. A standard HMA layer coefficient of 0.38 is used for overlay thickness calculations.

1.3.3 Iowa Department of Transportation

Information relevant to the Iowa Department of Transportation (IaDOT) procedures was provided by Mr. Chris Brakke, Pavement Design Specialist. IaDOT promotes the use of the 1993 AASHTO structural deficiency approach for establishing structural overlay requirements. Surface deflections obtained with a Road Rater are utilized to compute the design subgrade resilient modulus and the effective structural number of the existing pavement using an internal IaDOT method developed in the mid 1980s. A minimum of 10 deflection tests per project are obtained in the outer wheelpath at approximately 0.1 mile intervals.

The required structural number after overlay is established based on a 20-year design scenario. The remaining life factor for the existing pavement, F_{rl}, is set to 1.0 and a standard HMA layer coefficient of 0.44 is used for overlay thickness calculations

1.3.4 Michigan Department of Transportation

Information relevant to the Michigan Department of Transportation (MiDOT) procedures was provided by Mr. Steve Bower, Pavement Design Engineer. MiDOT promotes the use of the 1993 AASHTO procedures establishing structural overlay requirements. However, policy decisions are primarily used to establish structural overlay thickness designs. In most cases preventive maintenance, including a maximum 1-1/2 inch HMA overlay, is applied prior to significant structural deterioration.

A visual assessment of surface condition is used to indicate the need for an overlay. When required, a policy overlay thickness of 3 to 4 inches is applied based on regional decisions including an analysis of soil type, anticipated traffic, and existing pavement condition. Policy overlays have typically prolonged the pavement's service life in the range of 8 - 12 years. Policy overlays are used only once during the service life of a pavement. Subsequent improvements will typically include cold-in-place recycling or complete reconstruction.

1.3.5 Minnesota Department of Transportation

Information relevant to the Minnesota Department of Transportation (MnDOT) procedures was provided by Mr. Duane Young, Pavement Design Engineer. MnDOT utilizes a maximum deflection approach for establishing structural overlay thickness requirements. MnDOT uses an internal computer program (TONN) to determine the overlay thickness required to increase the load carrying capacity of the pavement to a desired level.

Surface deflections are obtained using a falling weight deflectometer. Deflection tests are conducted within the outer wheelpath of each travel lane using an applied load of approximately 9,000 lbs with a test interval of approximately 0.1 miles. Recorded maximum deflections are normalized to a 9,000 lb load and adjusted to represent critical season deflections at a standard pavement temperature of 70 ^{o}F. The representative deflection for a given design section is computed as the average plus two standard deviations.

Based on the computed representative deflection, the TONN program computes the Springtime single axle load carrying capacity of the existing pavement. Overlay thickness requirements necessary to increase the Springtime capacity to 9 or 10 tons (single axle loading) are also computed. Final overlay thickness recommendations are based on budget constraints.

1.3.6 Summary of Surrounding States

The results of the surrounding State survey are summarized in Table 3.1. Shown are the key data elements utilized for characterizing the existing pavement and for establishing structural overlay thickness requirements.

Table 1.3.1 - Summary of Overlay Design Procedures Used in Surrounding States

State	Promoted Overlay Design Procedures	Key Data Used to Characterize Existing Pavement	Methods Used to Determine Overlay Thickness Requirements
Illinois	Asphalt Institute	Maximum surface deflection obtained with a falling weight deflectometer	Asphalt Institute nomograph of overlay thickness vs representative rebound deflection.
Indiana	1993 AASHTO	Visual observations and surface deflections obtained with a falling weight deflectometer	1993 AASHTO structural deficiency approach
Iowa	1993 AASHTO	Surface deflections obtained with a Road Rater	Structural deficiency approach using internal IaDOT method for computing effective structural number of existing pavement
Michigan	Policy Decisions	Visual assessment of surface condition	Policy overlay thicknesses used for first structural overlay. Subsequent improvements utilize cold-in-place recycling or reconstruction.
Minnesota	Internal Methods	Surface deflections obtained with a falling weight deflectometer	Internal TONN program used to compute overlay thickness required to increase single axle load carrying capacity to desired level.

CHAPTER 2
RECOMMENDED DATA ELEMENTS

2.1 Introduction

Based on the review of published overlay design procedures and the survey of surrounding State DOTs, it is recommended that the WisDOT flexible pavement overlay design procedures include measures of both pavement condition and surface deflections and allow for independent as well as integrated usage.

The WisDOT flexible pavement overlay design procedures must be compatible with current WisDOT design procedures for new pavements and must be flexible enough to be integrated into revised pavement design procedures which may include items such as the subgrade resilient modulus and design reliability. The overlay design procedures should be applicable to deteriorated pavements in need of repair as well as newer pavements which require structural improvements to handle increased traffic demands, such as detour routes. The following sections describe the framework for key data elements recommended for inclusion into the WisDOT flexible pavement overlay design procedures.

2.2 Effective Structural Number of Existing Pavement

The current WisDOT flexible pavement design procedures are based on the structural number (SN) concept developed as a result of the original AASHTO Road Test. At this time, the procedures are based on 1972 AASHO design equation. Discussions with WisDOT design engineers have indicated that the current design procedures may be updated within the next 3-5 years, depending on the applicability of the Mechanistic-Empirical AASHTO design procedures currently under review. In the interim, it is recommended that the overlay design procedures developed through this research be based on the existing SN concept, which requires the determination of the effective structural number, SN_{eff}, of an existing pavement.

It is highly recommended that deflection testing be required for establishing SN_{eff} for all but lightly trafficked routes. Recommended procedures for this analysis are provided in Section 2.4. For those cases where deflection data is unavailable, procedures are provided for establishing SN_{eff} based on pavement condition measures (ride quality, distress) and the original pavement structural number. Recommended procedures for this analysis are provided in Section 2.3.

It is further recommended that multi-level procedures that allow for the determination SN_{eff} using distress, ride quality, and/or deflection data based on design ESALs be considered. An example of a decision matrix for this purpose is provided in Table 2.2.1.

Table 2.2.1: Example Decision Matrix for Establishing SN_{eff}

Design ESALs (millions)	Data Recommended to Establish SN_{eff}			
	Deflections	Distress	Ride Quality	Original SN
< 0.3	2	1	2	2
0.3 to < 1.0	2	1	2	2
1.0 to < 3.0	1	2	2	2
3.0 to < 10	1	2	2	2
> 10	1	2	2	2

1. Strongly recommend for consideration in design
2. Recommended for consideration, if available

2.3 Pavement Condition Measures

WisDOT routinely collects flexible pavement distress data and ride quality measures on a system-wide basis. The Marquette University research staff has obtained historical flexible pavement distress and ride quality data from WisDOT. Historical distress data is available dating back to 1985 and ride quality data is available back to 1980.

Distress data is currently utilized by WisDOT to compute the overall Pavement Distress Index (PDI), a value which has been used to indicate the need for pavement rehabilitation. Figure 2.3.1 illustrates PDI data trends for a subset of the available WisDOT data. This subset was selected for illustrative purposes and represents the first 500 non-zero entries within the PDIFLEX database.

It is also recommended that a new distress index, such as a Structural Distress Index, (SDI) be considered for development which uses using only key structural distress data such as alligator cracking and rutting. The SDI could be computed in a manner similar to the existing PDI equation, with possible modifications to the distress factors currently used for PDI calculations. This concept could also be integrated with other condition measures such as the Pavement Condition Index (PCI). A numerical and graphical procedure, similar to the 1993 AASHTO procedure (*1*), could be developed to use the SDI to estimate of the remaining service life of the pavement and to select a condition factor for modifying the in situ pavement's effective structural number, SN_{eff}. This procedure should be developed based on historical distress data already available from WisDOT.

Figure 2.3.2 illustrates example SDI trends for the data subset illustrated in Figure 2.3.1. For this illustration, SDI was calculated only from rutting, alligator cracking, and transverse cracking distress data using standard WisDOT distress factors.

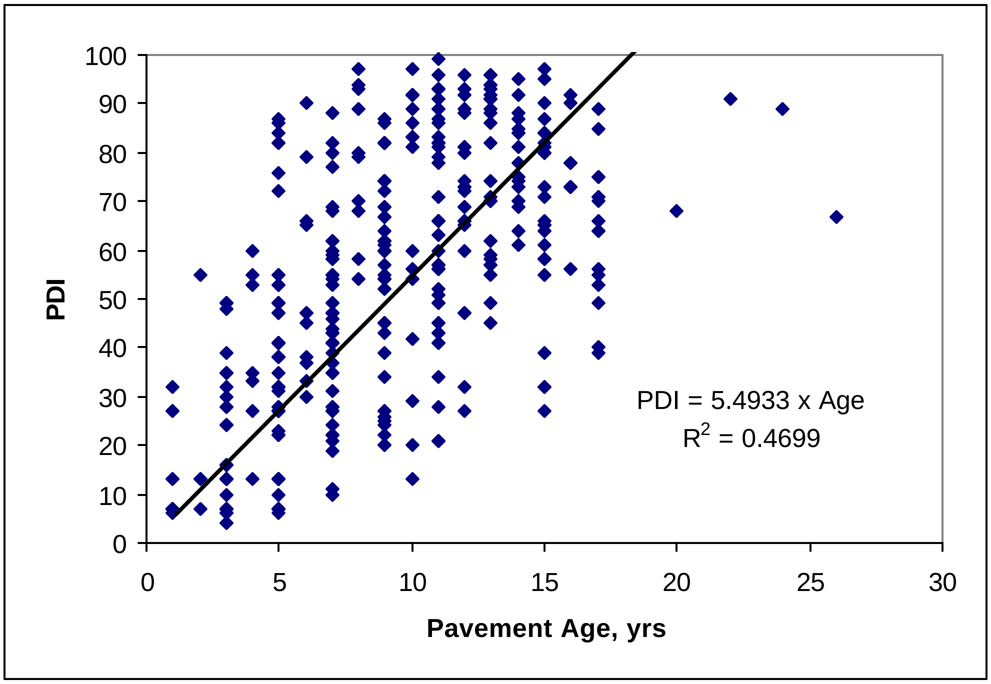


Figure 2.3.1: PDI versus Age for WisDOT Performance Data

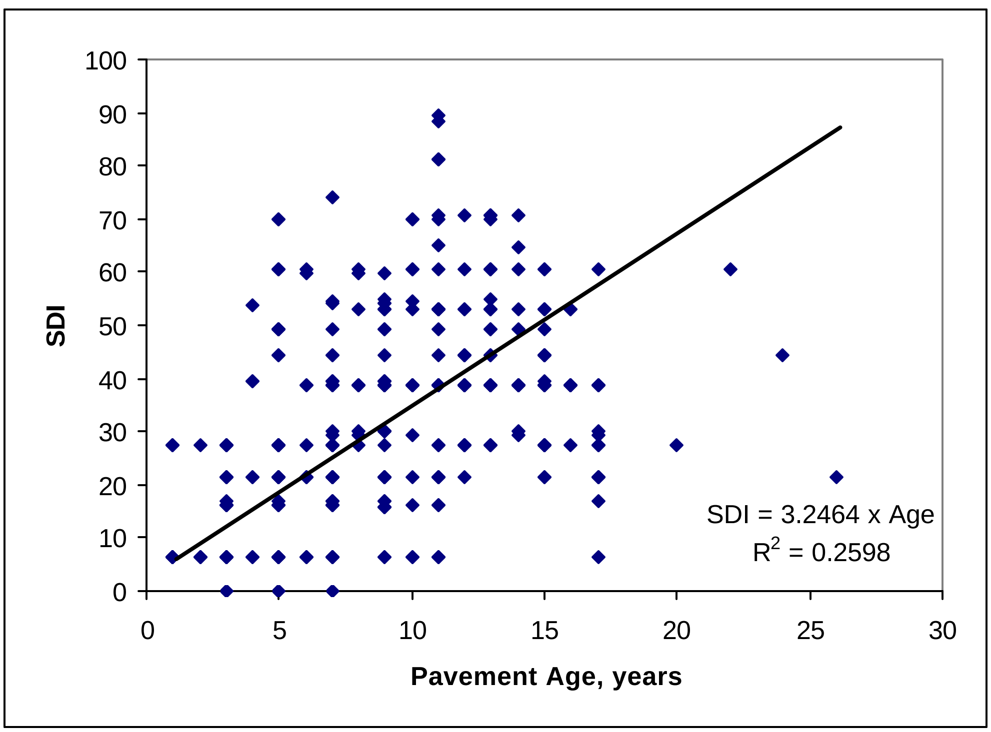


Figure 2.3.2: SDI versus Age for WisDOT Performance Data

Figure 2.3.3 illustrates a comparison of SDI versus PDI for this data. The poor correlations exhibited in Figures 2.3.2 and 2.3.3 indicates more analysis is required before these concepts could be utilized within the overlay design procedures.

Ride quality is currently calculated from profile data and reported in terms of the International Roughness Index (IRI). Figure 2.2.4 illustrates IRI trends for a similar data subset (i.e., first 500 non-zero entries) extracted from the PSIFLEX database. It is recommended that a procedure be developed to utilize IRI data for estimating the remaining life of a given pavement and to select a condition factor for modifying the in situ pavement's effective structural number, SN_{eff}. A numerical and graphical procedure, similar to the Asphalt Institute's procedure (*3*), should be developed based on historical IRI trends of flexible pavements in Wisconsin.

2.4 Pavement Deflection Measures

Pavement deflections obtained with heavy-load deflection devices provide a valuable assessment tool for estimating the structural capacity of in situ pavements. It is highly recommended that pavement deflection data be required for estimating both SN_{eff} and the subgrade resilient modulus, M_R, for all but lightly traffic roadways. WisDOT currently owns and operates a KUAB falling weight deflectometer (FWD) for collecting pavement deflection data FWD testing data can also be provided by a number of independent contractors. Various techniques for utilizing deflection data for analysis of in-place flexible pavements are provided in the following sections.

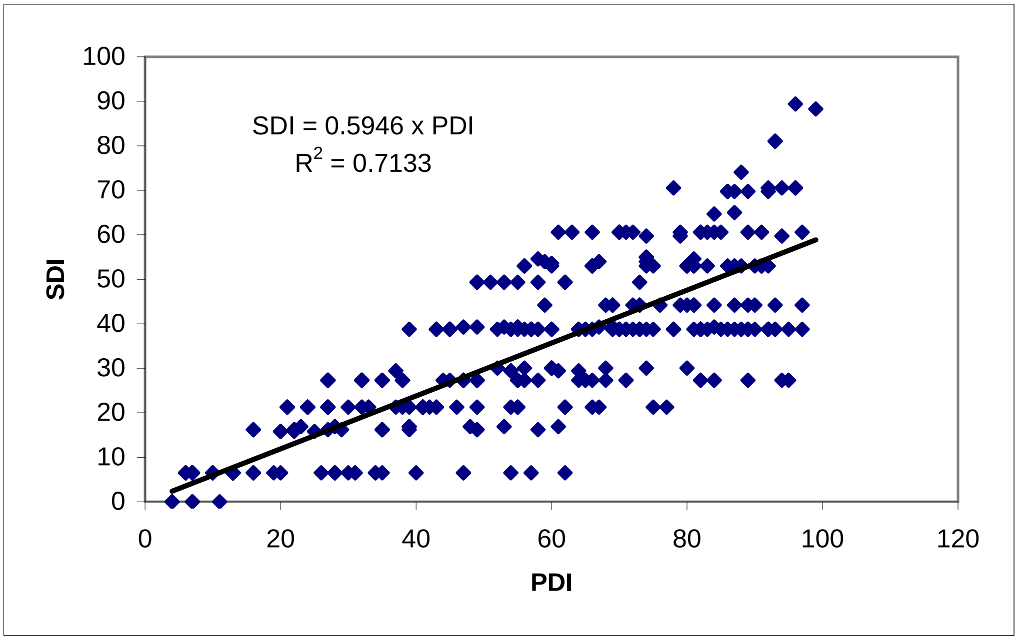


Figure 2.3.3: SDI versus PDI Values

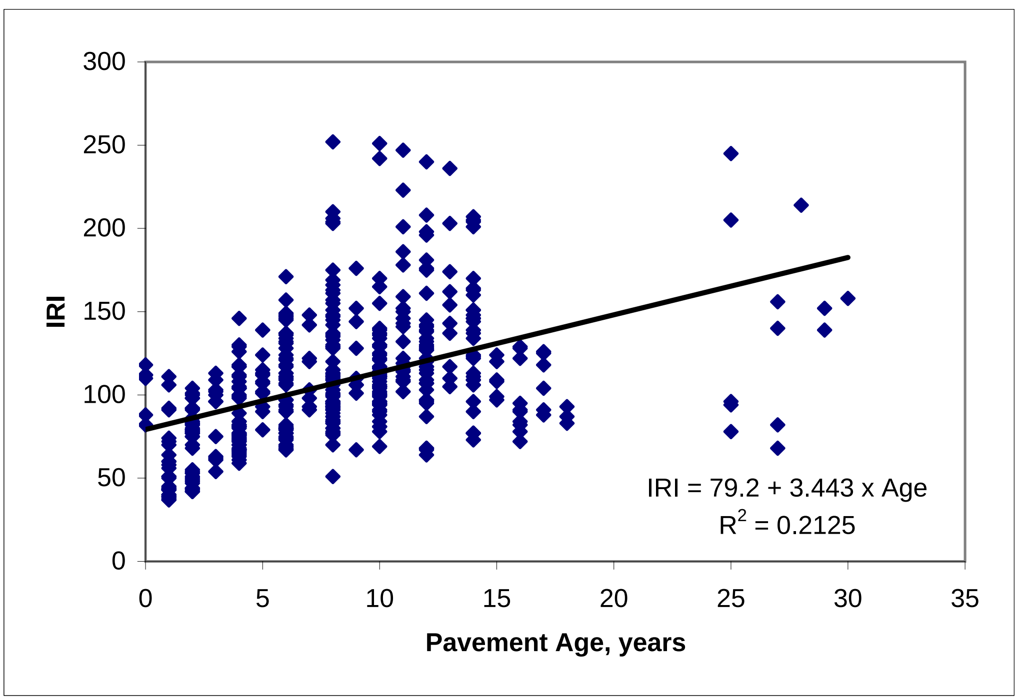


Figure 2.3.4: IRI versus Age for WisDOT Performance Data

A research factorial of pavement response data was generated to provide response data to test the validity of available analysis procedures as well as to develop new equations, where appropriate. The KENLAYER *(5)* computer program, which allows for stress-dependent base and subgrade layer analyses, was utilized for this effort. Table 2.4.1 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.

Table 2.4.1 KENLAYER Pavement Factorial

Layer	Thickness Range	Modulus Range
HMA (8x8)	2" to 9 " (1" inc.)	Eac = 250 - 950 ksi (100 ksi inc.)
Aggregate Base (8x3)	6" - 15" (1" inc.)	$M_R = 4000\ \square^{.6}$ $M_R = 5000\ \square^{.5}$ $M_R = 8000\ \square^{\square}$
Subgrade (1x4)	240"	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 (8x8x8x3x4) with base to HMA thickness ratios varying from 0.67 to 7.5. 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 1.8 to 3.25, which is more in line with pavement design practices in Wisconsin, resulting in a total of 2,592 separate pavement structures. The input SN of each pavement structure was computed based on the input thickness and modulus values for each layer. The computed SN values for the parsed factorial ranged from 2.09 to 6.73. These values, along with the surface deflections generated by the program were used to test the validity of available models and to developed improved equations, where warranted, to estimate key structural pavement parameters.

2.4.1 SN_{eff} Predictions Based on AASHTO Equations

The deflection-based AASHTO equations (Eqns. 1.4 - 1.6) presented in Section 1.2.3 were investigated to test their validity in predicting the input structural number used during the KENLAYER factorial analysis. The three-step AASHTO process for estimating the in situ SN_{eff} is summarized as:

1. Estimate subgrade modulus based on surface deflections at all offset distances greater than 0, and consider the minimum computed modulus as the estimated in-place modulus.

2. Estimate Ep by iterative process based on total pavement thickness and estimated subgrade modulus.

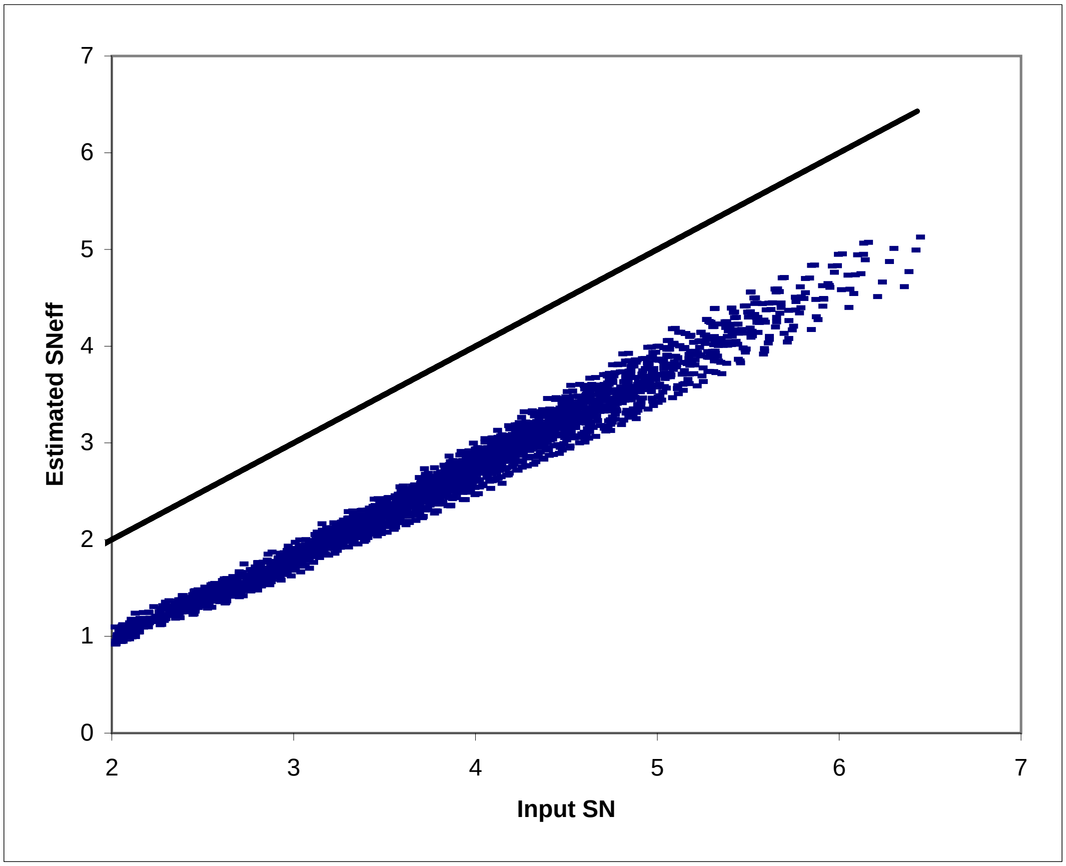
3. Estimate SN_{eff} based on the estimated Ep and total pavement thickness using the published AASHTO equation $SN_{eff} = 0.0045\ Ep^{1/3}T$.

The above analysis process was applied to all pavement structures included in the parsed KENLAYER output file. Figure 2.4.1 illustrates a comparison of estimated SN_{eff} versus input SN values based on this standard AASHTO process. As shown, SN values are consistently under-predicted. When used in the context of an overlay design procedure based on structural deficiencies, this under-estimation of SN_{eff} would result in an increased overlay thickness requirement. Based on standard HMA layer coefficients, the increased overlay thickness can be directly computed as:

$$HMA_{OL\text{-}inc} = (\text{Input SN} - SN_{eff}) / 0.44 \qquad \text{Eq. 2.1}$$

where: $HMA_{OL\text{-}inc}$ = increased HMA overlay thickness requirement, inches

Based on the data provided in Figure 2.4.1, the median increased overlay thickness is 2.7 inches (maximum = 3.9 inches).



Figuire 2.4.1: Effective SN versus Input SN Based on Current AASHTO Equation

Upon further investigation, it was determined that the under-predicted SN values were due in part to an inconsistency in the AASHTO equations. Within the 1993 AASHTO guide *(1)*, the relation between HMA structural coefficients and elastic modulus is provided in nomographic format. Using data generated from this nomograph, the following relation was determined:

$$a_{HMA} = 0.0057\ (E_{HMA})^{1/3} \qquad \text{Eq. 2.2}$$

Where: a_{HMA} = structural coefficient of the HMA layer
E_{HMA} = elastic modulus of the HMA layer, psi

When applied to a full-depth HMA pavement with a singular layer modulus, the use of Eq. 2.2 would result in a computed SN value as:

$$SN_{F\text{-}D} = a_{HMA}\ T_{HMA} = 0.0057\ (E_{HMA})^{1/3}\ T_{HMA} \qquad \text{Eq. 2.3}$$

Where: $SN_{F\text{-}D}$ = SN of full-depth HMA pavement
T_{HMA} = HMA layer thickness, inches

Equation 2.3 indicates an SN under-prediction bias of 21% [(.0057-.0045)/.0057] results from direct application of the published AASHTO equation (Eqn. 1.6). Furthermore, when computing SN for a conventional HMA pavement (HMA + aggregate base) by this process, it is reasonable to compute structural layer coefficients for all layers by Eq. 2.3, resulting in:

$$SN = \Sigma\ a_i\ T_i\ m_i = \Sigma\ 0.0057\ E_i^{1/3}\ T_i\ m_i \qquad \text{Eq. 2.4}$$

Where: a_i = structural coefficient of layer i
T_i = thickness of layer i, inches
m_i = drainage coefficient of layer i
E_i = elastic modulus of layer i, psi

It also follows that estimations of SN_{eff} should be computed by:

$$SN_{eff} = 0.0057\ E_p^{1/3}\ T_p \qquad \text{Eq 2.5}$$

where: SN_{eff} = effective structural number of the in-place pavement
Ep = equivalent modulus of the in-place pavement
Tp = total pavement thickness above subgrade, inches

Effective structural numbers for each pavement structure were computed using these revised equations. Figure 2.4.2 provides predicted (Eq. 2.5) versus actual (Eq. 2.4) SN values for this same data set. As shown, SN values are still consistently under-predicted, resulting in a median increased overlay thickness requirement of 1.6 inches (maximum = 2.4 inches).

Based on a regression analysis of the parsed KENLAYER data, better agreement is achieved using a modified form of the SN_{eff} equation as:

$$SN_{eff} = 1.055 + 0.0051\ E_p^{1/3}\ T \qquad R^2 = 0.9805 \qquad \text{Eq. 2.6}$$

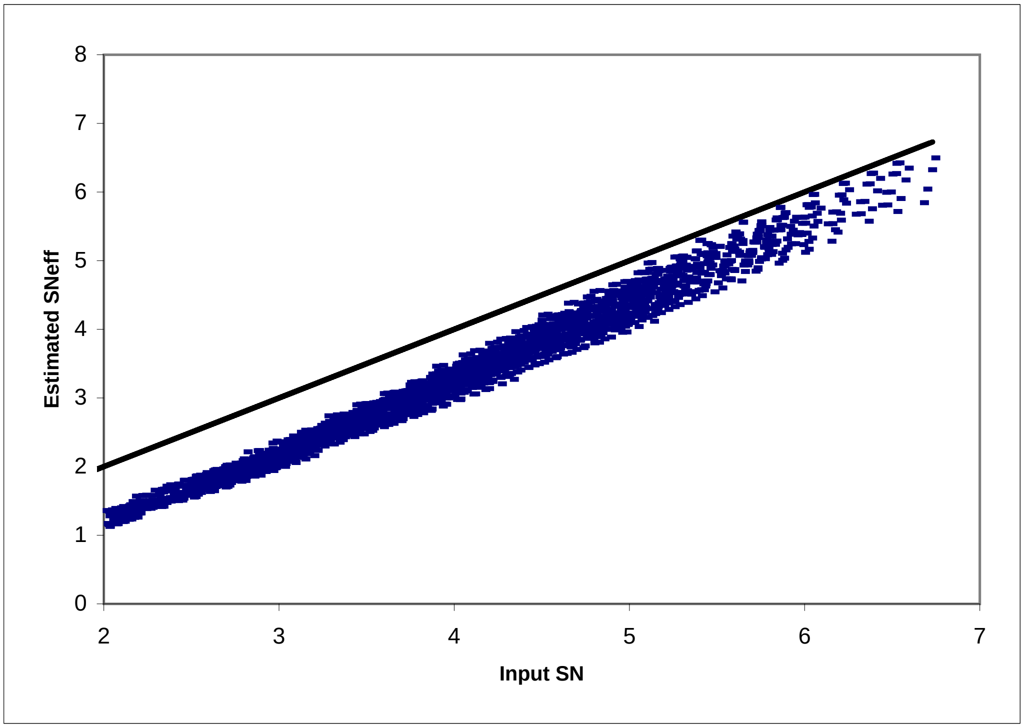
Figure 2.4.3 provides predicted (Eq. 2.6) versus actual (Eq. 2.4) SN values. As shown, the SN under-prediction bias is eliminated and the maximum increased overlay thickness requirement is reduced to +/- 0.9 inches, with 89% of the errors less than +/- 0.5 inches. While this represents a marked reduction in overlay thickness errors resulting from SN_{eff} predictions, further attempts were made to find alternate deflection-based strategies which may further reduce the associated overlay thickness error.

2.4.2 SN_{eff} Predictions Based on Deflection Algorithms

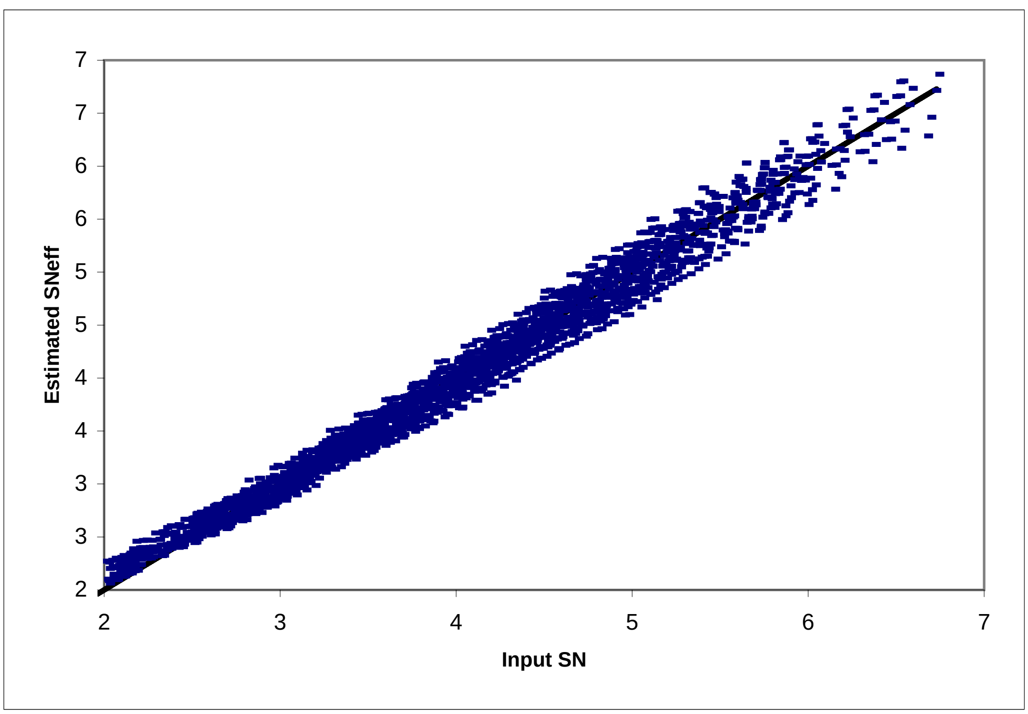
Previous research (*5,6*) has shown that the subgrade modulus and pavement flexural rigidity can be directly back-calculated from deflection data. Thompson *(5)* provides the following equation for estimating the breakpoint resilient modulus of the subgrade:

$$E_{ri} = 26.45 - 5.12\ D_{36} + 0.2586\ D_{36}^2 \qquad \text{Eq. 2.7}$$

where: E_{ri} = subgrade breakpoint resilient modulus, ksi
D_{36} = surface deflection at 36 inches from load, mils @ 9,000 lb



Figuire 2.4.2: Effective SN versus Input SN Based on Revised AASHTO



Figuire 2.4.3: Effective SN versus Input SN Based on Modified AASHTO

The backcalculated E_{ri} values determined by Eq., 2.7 are reported to provide reasonable estimates of the design M_R value which is required for new pavement design within the AASHTO process.

Thompson *(5)* also introduced an additional deflection term known as the Area Under the Pavement Profile, AUPP. AUPP is simply calculated from multi-sensor deflection data commonly obtained during FWD testing using the equation:

$$AUPP = \tfrac{1}{2} \left(5\,D_0 - 2\,D_{12} - 2\,D_{24} - D_{36} \right) \qquad \text{Eq. 2.8}$$

where: AUPP = Area Under the Pavement Profile
D_i = surface deflection at i inches from the center of loading, mils @ 9,000 lb

Preliminary models for estimating ET^3 and SN_{eff} from AUPP (Eq. 2.8) and E_{ri} (Eq. 2.7) were developed by Maguire (*6*):

$$\text{Log } ET^3 = 6.21 - 0.49 \text{ Log } AUPP + 0.0023 \text{ Log } E_{ri} \qquad \text{Eq. 2.9}$$

$$SN_{eff} = 0.1477\,(ET^3)^{1/3} - 0.014\,E_{ri} - 6.43 \qquad \text{Eq. 2.10}$$

where: ET^3 = flexural rigidity of entire pavement system, kip-inches

Equations 2.9 and 2.10 were developed based on a limited factorial analysis of flexible pavement response using stress-dependent elastic layer computer modeling. It is important to note that pavement layer thicknesses, commonly obtained by coring, are not required for the deflection analysis using these equations.

Figure 2.4.4 illustrates a comparison of predicted versus actual SN values using the preliminary models applied to the parsed output results from the KENLAYER factorial analysis. As shown, the preliminary equations result in a consistent under-prediction of the SN_{eff} values, which correlates to a median increased overlay thickness requirement of 2.0 inches (maximum = 4.1 inches).

Using regression analysis on the larger parsed KENLAYER output file, revised predictive equations were developed during this research. These equations, which should be applied sequentially, are as follows:

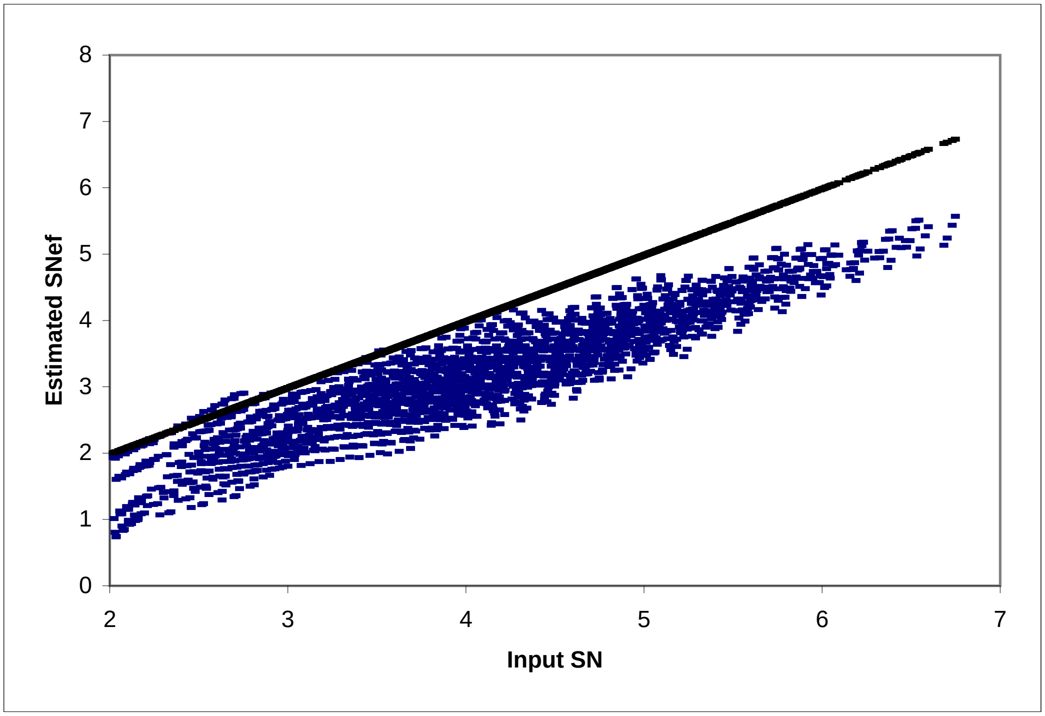
$$eE_{ri} = 22.04 - 3.645\ D_{36} + 0.158\ D_{36}^2 \quad R^2 = 0.9188 \qquad \text{Eq. 2.11}$$

$$\text{Log } eE^{1/3}T = 3.574 - 0.437 \text{ Log AUPP} - 0.066 \text{ Log } eE_{ri} \quad R^2{=}0.9045 \qquad \text{Eq. 2.12}$$

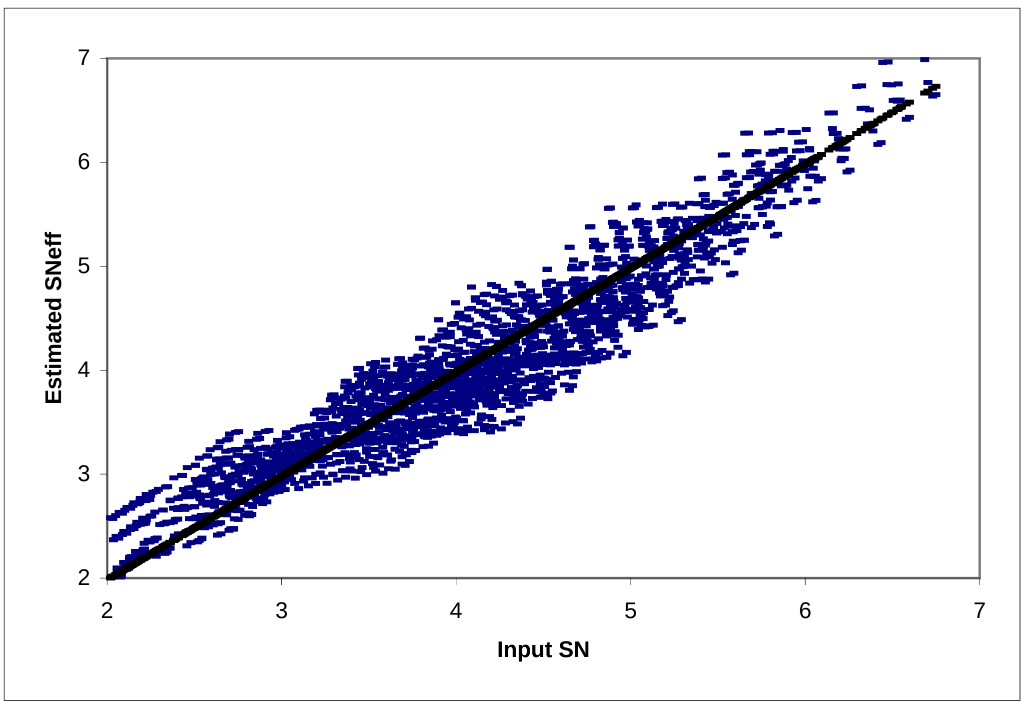
$$SN_{eff} = 0.0055\ eE^{1/3}T - 0.0012\ eE_{ri} + 0.144 \quad R^2{=}0.9058 \qquad \text{Eq. 2.13}$$

Where: eE_{ri} = estimated breakpoint subgrade resilient modulus, ksi
D_{36} = surface deflection at 36 inches from the center of loading, mils@9k
$eE^{1/3}T$ = estimated overall pavement flexural rigidity term, lb-in$^{1/3}$

Figure 2.4.5 illustrates predicted versus input SN values resulting from the application of the revised equations. As shown, the predicted SN values are unbiased and clustered along the line of equality. However, the scatter in the predicted SN values correlates to an overall range in the overlay thickness estimation error of approximately +/- 1.9 inches, with 84% of the values within +/- 1.0 inches and 49% within +/-0.5 inches. While equations such as these offer the benefit of not having to obtain in situ pavement layer thicknesses from coring, the associated overlay estimation errors may render them impractical to apply.



Figuire 2.4.4: Effective SN versus Input SN Based on Preliminary Eri-AUPP



Figuire 2.4.5: Effective SN versus Input SN Based on Modified Eri-AUPP

2.4.3 SN_{eff} Predictions Based on Asphalt Institute Procedures

The Asphalt Institute (AI) deflection-based analysis procedures presented in Section 1.2.6 were further investigated to determine if applicable analysis strategies could be developed for predicting SN_{eff} without knowledge of in-place pavement thickness. As developed, the AI procedures estimate allowable ESALs based only on maximum surface deflection. Knowledge of the subgrade modulus, which significantly contributes to overall pavement deflection, is necessary to provide estimates of the SN_{eff}.

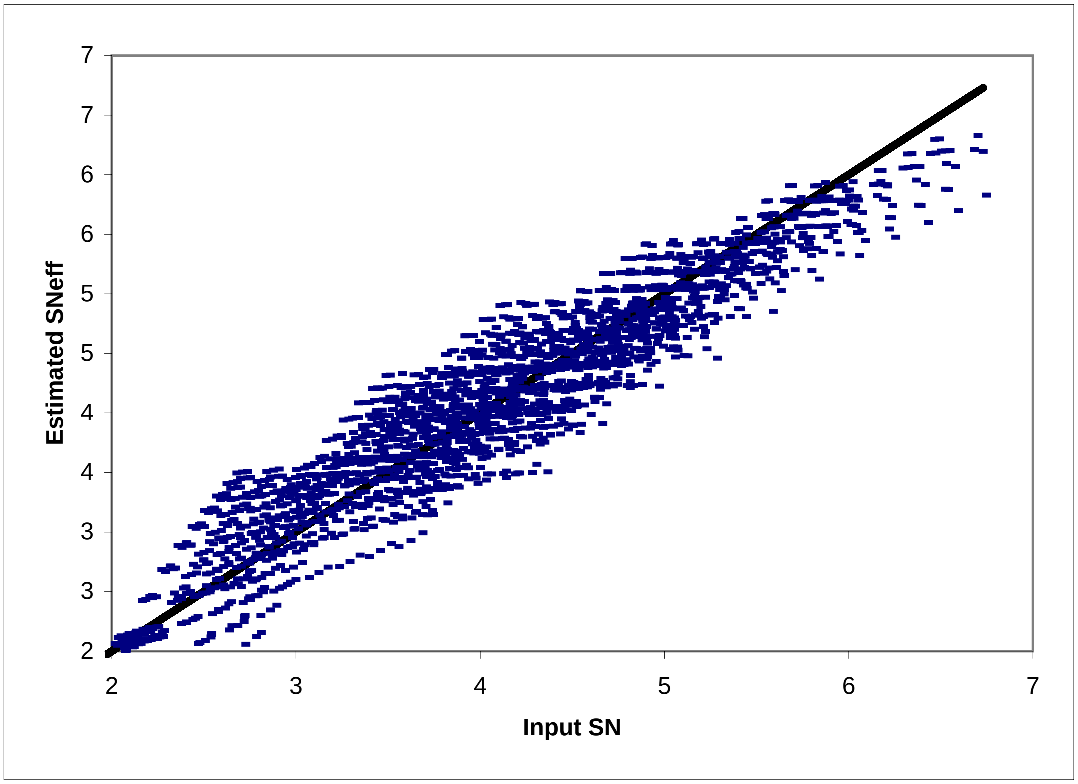
Based on a regression analysis of the parsed KENLAYER data, the best models for estimating SN_{eff} from only maximum deflection and subgrade modulus are:

$$T_{HMA} > 2 \text{ inches: } SN_{eff} = 17.4 - 0.263\ Esg - 7.56 \text{ Log } D0 \quad R^2 = 0.881 \qquad \text{Eq. 2.14}$$

$$T_{HMA} = 2 \text{ inches: } SN_{eff} = 5.2 - 0.074\ Esg - 1.44 \text{ Log } D0 \quad R^2 = 0.547 \qquad \text{Eq. 2.15}$$

Where: Esg = Minimum subgrade modulus computed by AASHTO (Eq. 1.4), ksi
D0 = Maximum deflection, mils at 9,000 lb

Figure 2.4.6 provides an illustration of the predicted versus actual SN values determined by the above equations. As shown, the data are clustered along the line of equality but the range of errors for the required overlay thickness is approximately +/- 2 inches, with 81% of the values less than +/- 1 inch and 49% less than +/- 0.5 inches. These errors are very similar to those associated with the E_{ri}-AUPP approach and again may be considered excessive for practical applications.

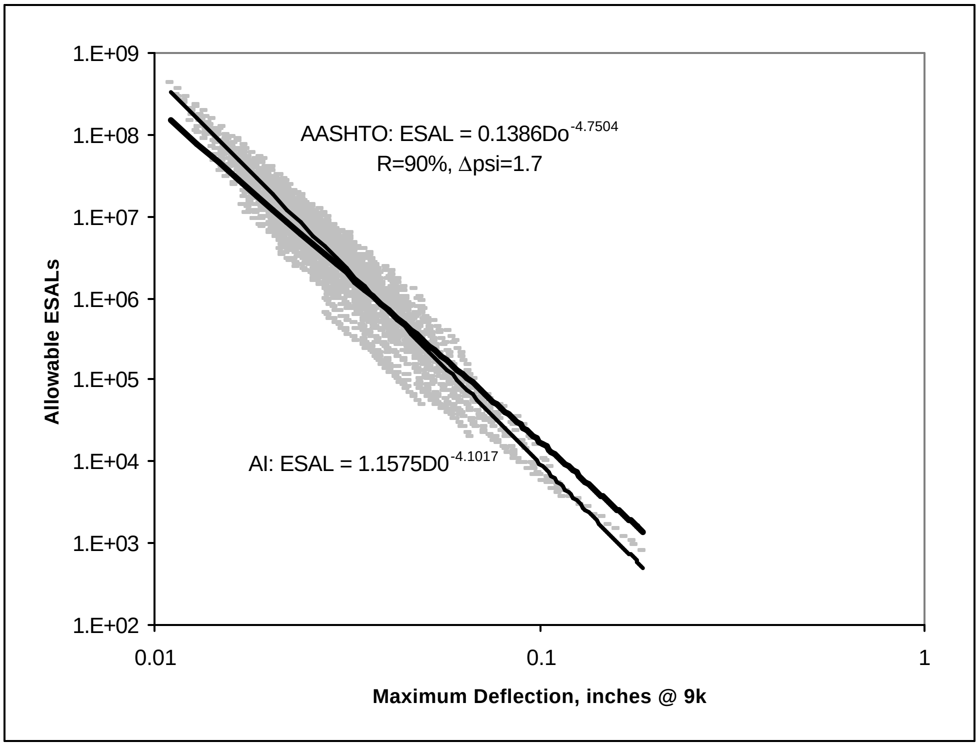


Figuire 2.4.6: Effective SN versus Input SN Based on AI

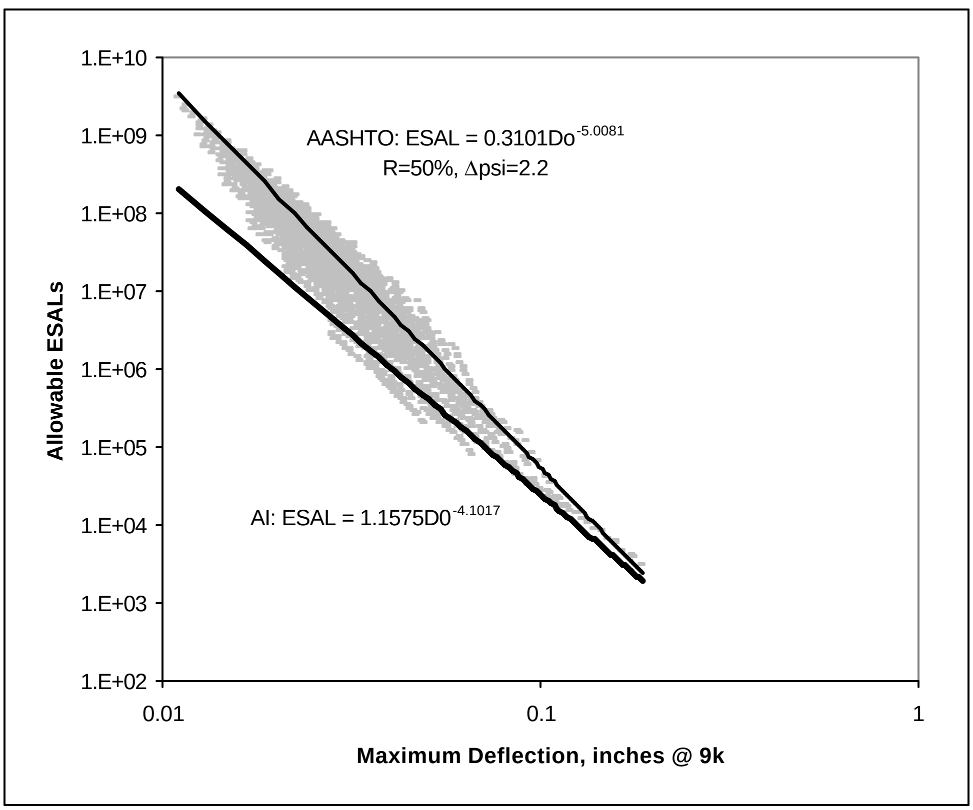
2.4.4 Combined Overlay Design Approach

An overlay design approach which combines aspects of both the AASHTO and AI analysis procedures was investigated to determine if practical guidelines could be established. Using the data from the parsed KENLAYER analysis, the allowable ESALs for each pavement system were computed following current AASHTO and AI procedures. While the AI procedure computes allowable ESALs based solely on maximum deflection, the current AASHTO procedures require inputs of subgrade modulus, pavement structural number, terminal serviceability, and design reliability. It is recognized that the current WisDOT design procedures do not require inputs of design reliability; however, it was deemed appropriate to include this factor to expand the applicability of the results.

During the AASHTO analysis, deflection data contained in the parsed KENLAYER results were first used to estimate the field subgrade modulus using AASHTO procedures (Eq. 1.4). Allowable ESALs were then computed based on the input SN and arbitrary selections of design reliability and terminal serviceability. The calculated allowable ESALs were then plotted against maximum surface deflection to examine the appropriateness of the current AI relation. Figure 2.4.7 illustrates an example plot based on AASHTO allowable ESALs computed using a design reliability of 90% and a terminal serviceability of 2.5 ($\Delta_{PSI} = 1.7$). Also shown are data trend lines based on the AASHTO data and the AI equation, which can be seen to be in general agreement for this data set. However, changes to inputs values of design reliability and/or terminal serviceability can result in significant discrepancies between the AASHTO and AI results, as shown in Figure 2.4.8 which was developed based on a design reliability of 50% and a terminal serviceability of 2.0 ($\Delta_{PSI} = 2.2$).



Figuire 2.4.7: Allowable ESALs versus Maximum Deflection



Figuire 2.4.8: Allowable ESALs versus Maximum Deflection

Based on an analysis of AASHTO allowable ESALs computed for a range of design reliability and terminal serviceability values, the following general model form was consistently applicable for estimating allowable ESALs from maximum deflection, D_0:

$$\text{Allowable ESALs} = A\, D_0^{B} \qquad \text{Eq 2.16}$$

Table 2.4.2 provides specific coefficients and exponents for varying levels of design reliability and terminal serviceability. The use of Equation 2.16 with appropriate terms selected from Table 2.4.2 allows for a direct analysis of the allowable ESALs based on maximum deflection only.

Table 2.4.2 Coefficients and Exponents for Equation 2.16

Reliability %	Terminal Serviceability	A	B	Terminal Serviceability	A	B
50	2.5	0.5232	-4.7504	2.0	0.3101	-5.0081
75	2.5	0.2602	-4.7504	2.0	0.1542	-5.0081
85	2.5	0.1786	-4.7504	2.0	0.1059	-5.0081
90	2.5	0.1386	-4.7504	2.0	0.0821	-5.0081
95	2.5	0.0951	-4.7504	2.0	0.0564	-5.0081
99	2.5	0.0469	-4.7504	2.0	0.0278	-5.0081

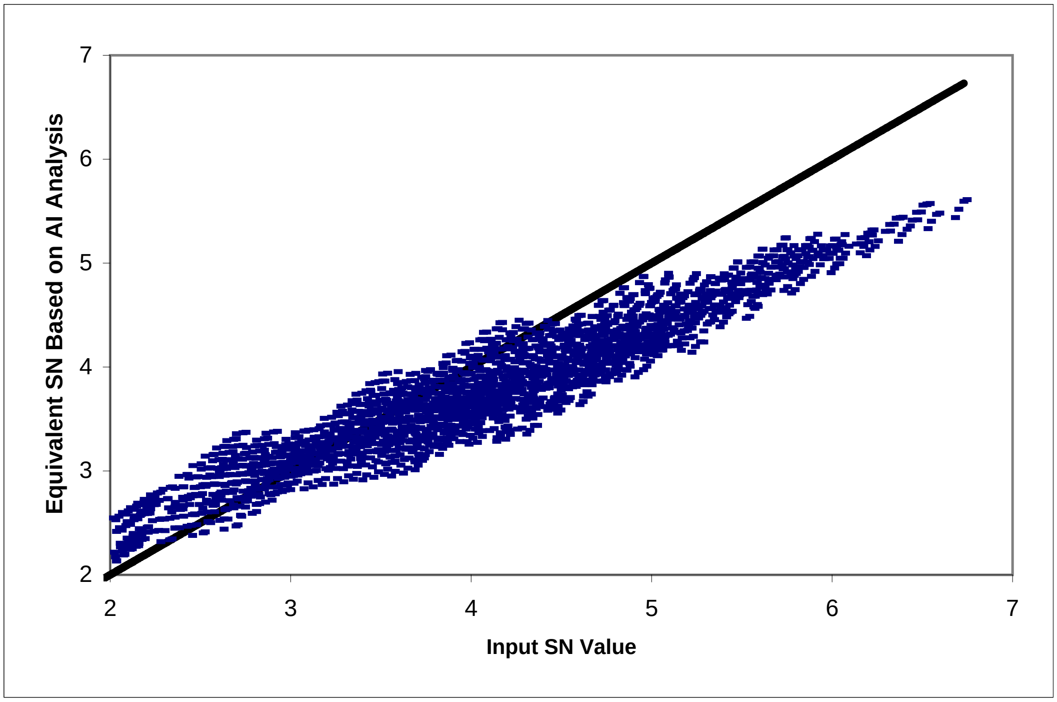
A secondary analysis was completed which utilized the allowable ESALs computed by the AI equation to compute a related SN_{eff} value based on AASHTO criteria. During this analysis, the in-place subgrade modulus was estimated based on the AASHTO equation and inputs for design reliability and terminal serviceability were set to 90% and 2.5, respectively. The SN_{eff} of the pavement was varied until agreement was reached between calculated AI and AASHTO ESALs.

Figure 2.4.9 provides a comparison equivalent AI SN values versus input SN values. As shown, the equivalent AI SN values tends to under-estimate the SN values as input SN values increase.

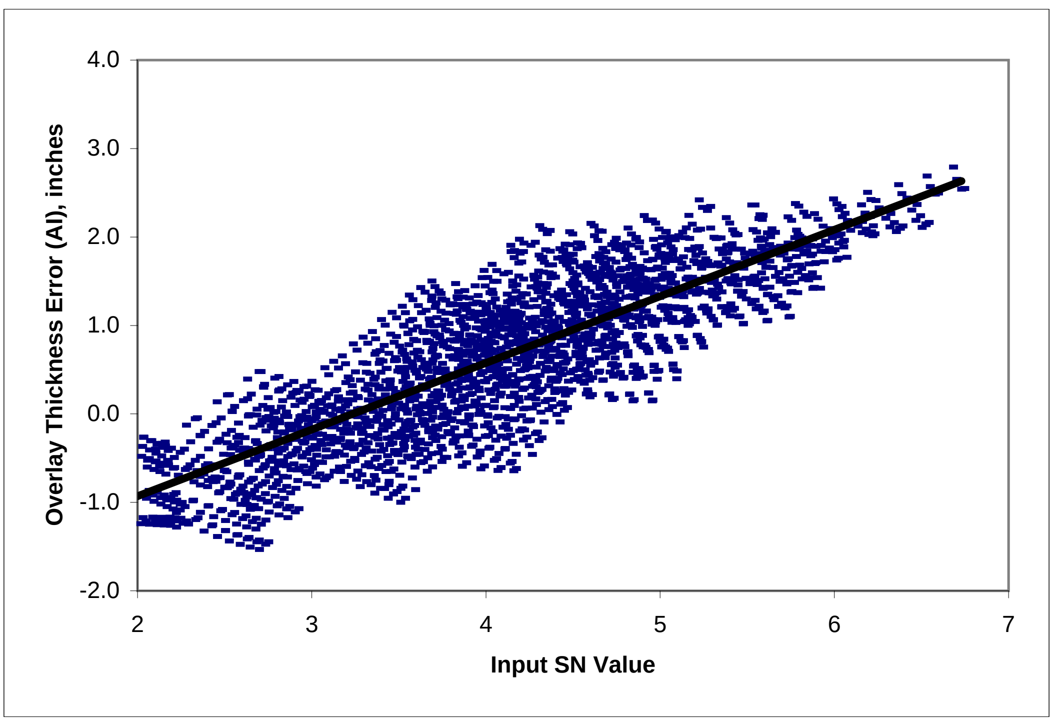
The equivalent AI SN values were also compared to input SN values to determine the impact on overlay thickness requirements. Figure 2.4.10 provides a plot of the associated overlay thickness estimation error versus the input pavement structural number, with positive values indicating a thicker overlay would be required due to an under-prediction of the input SN ($SN_{eff} < SN_{input}$), and vice versa. As shown, the overlay thickness estimation error tends to increase as the input structural number increases. Furthermore, the overlay thickness estimation error ranges from -1.5 inches to +2.8 inches, with only 57% of the values within +/- 1.0 inches. making this method impractical.

2.4.5 Preferred Deflection-Based Methods

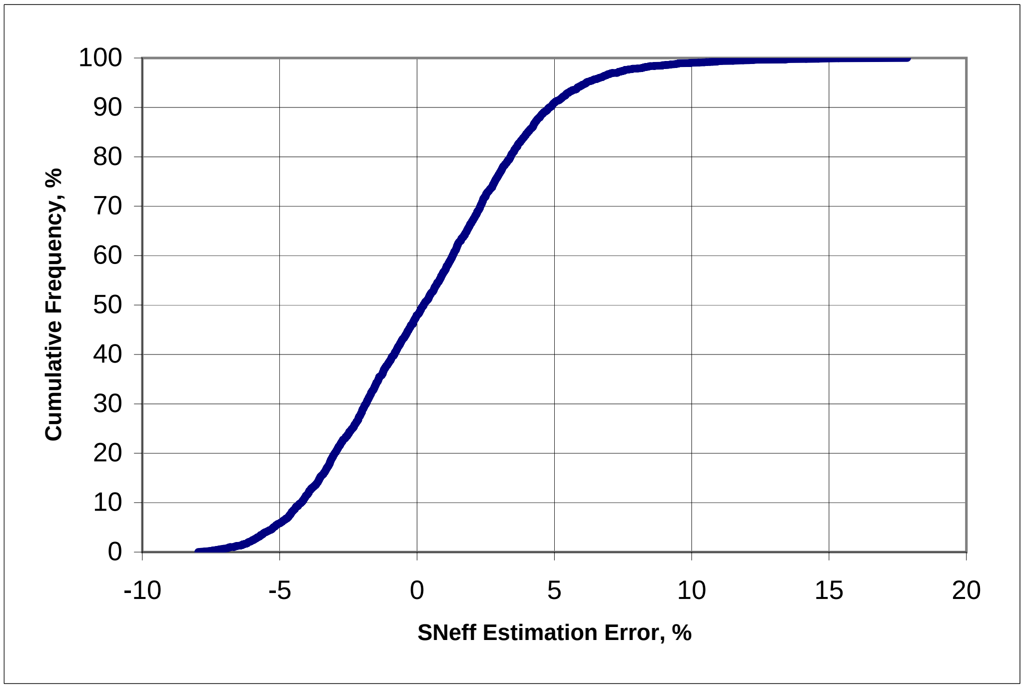
To maintain consistency with the current WisDOT flexible pavement design procedures which are based on the SN concept, the analysis results presented in the previous sections indicate the preferred deflection-based method which provides the best estimate of SN_{eff} for in-place HMA pavements is the modified AASHTO method. Based on the results of the factorial analysis, this method provided estimations of SN_{eff} values which were within 5% of the input values for 85% of the pavement structures investigated, as illustrated in Figures 2.4.11 and 2.4.12. Furthermore, application of this method is projected to provide overlay thickness requirements which are within +/- 0.9 inches of those required based on perfect assessment of SN_{eff}, with 90% of the values being within +/- 0.5 inches, as illustrated in Figures 2.4.13 and 2.4.14. This appears to be the practical limit of deflection-based approaches for developing overlay thickness requirements based on the structural deficiency approach.



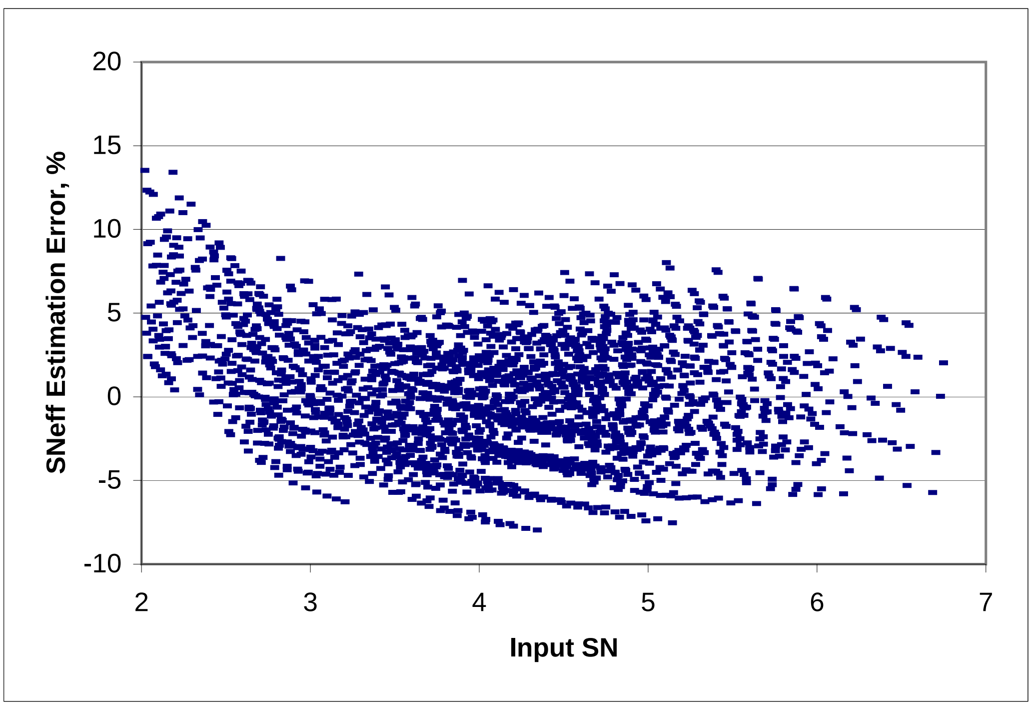
Figuire 2.4.9: Effective SN Based on AI Analysis versus Input SN



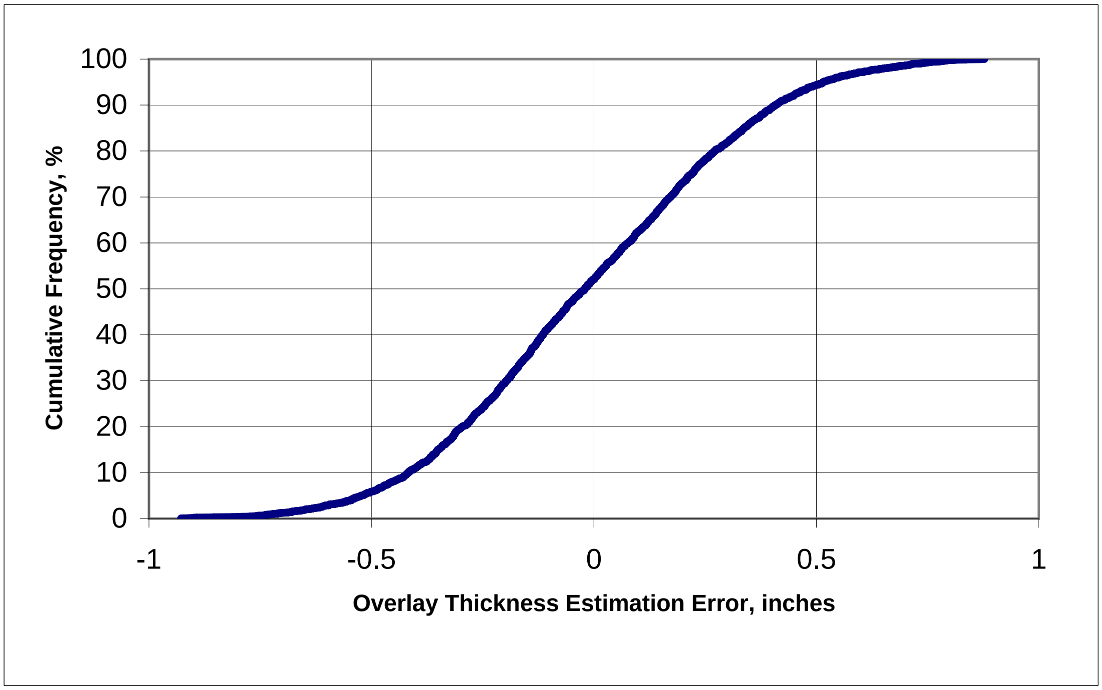
Figuire 2.4.10: Overlay Thickness Estimation Error Based on AI Analysis versus Input SN



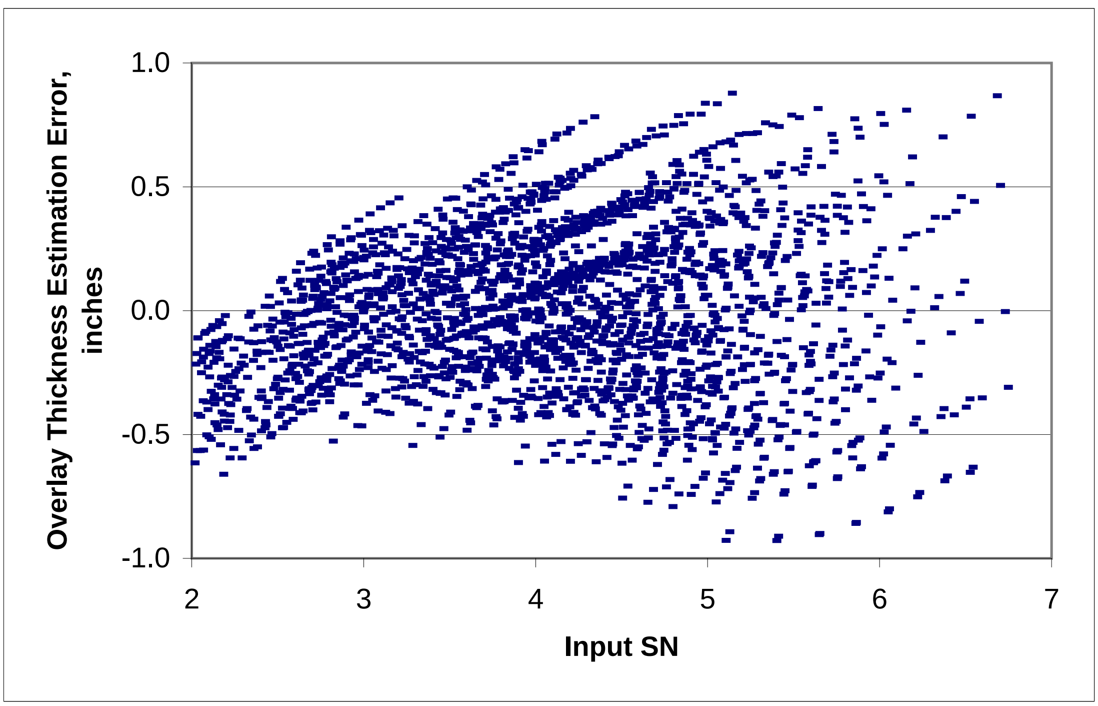
Figuire 2.4.11: SNeff Estimation Errors Based on Modified AASHTO



Figuire 2.4.12: SNeff Estimation Errors Based on Modified AASHTO



Figuire 2.4.13: Overlay Thickness Estimation Errors Based on Modified AASHTO



Figuire 2.4.14: Overlay Thickness Estimation Errors Based on Modified AASHTO

The preferred deflection-based analysis method is summarized by the following steps:

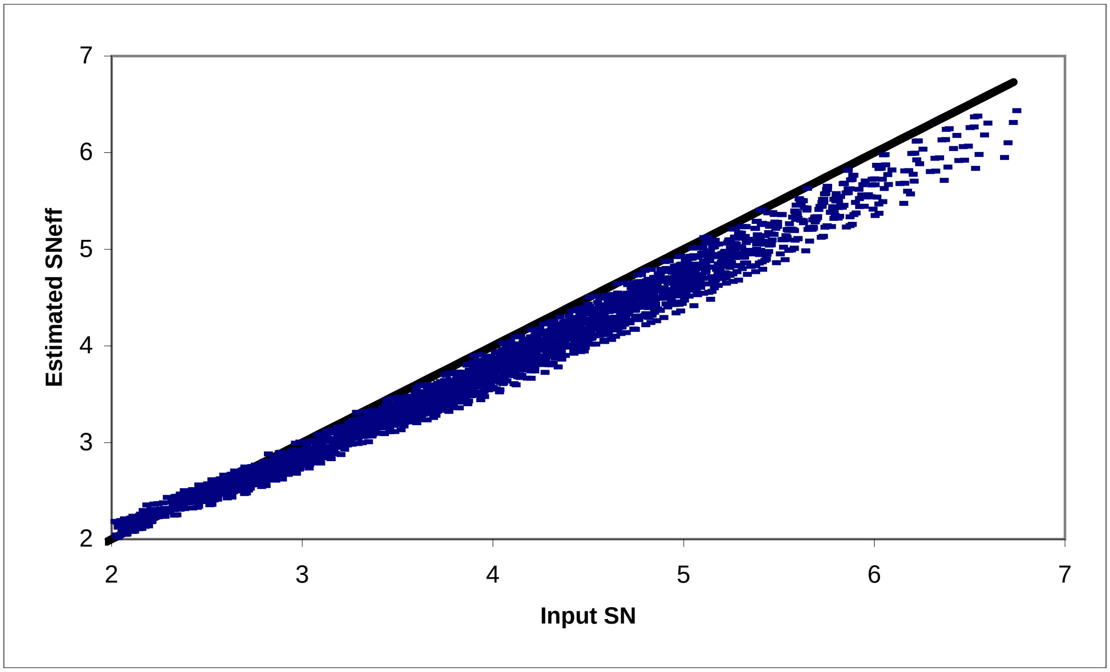
1. Surface deflections at all sensor locations outside the center of loading are used to estimate the field subgrade modulus by Eq. 1.4.

2. The effective pavement modulus is estimated by an iterative process using the previously estimated subgrade modulus (Step 1), measured load and maximum deflection, and known total pavement thickness by Eq 1.5.

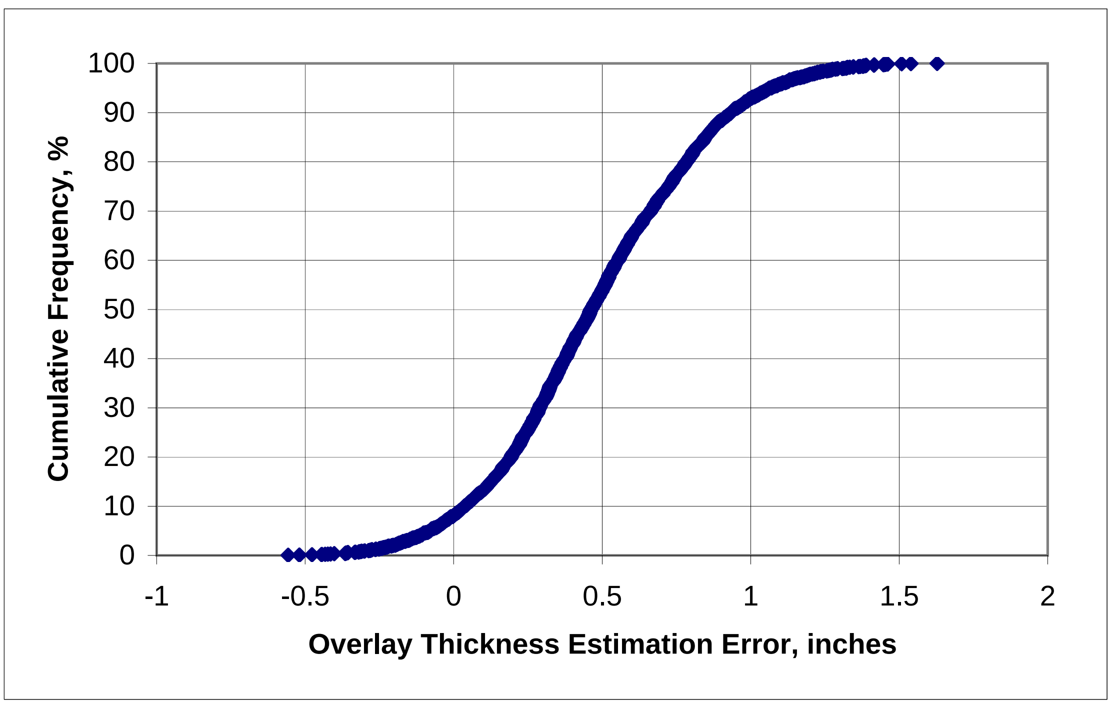
3. The effective SN of the pavement is estimated based on the previously estimated effective pavement modulus (Step 2) and known total pavement thickness by Eq 2.6.

This modified AASHTO deflection-based analysis method is somewhat cumbersome to apply and requires the use of iterative analysis in Step 2. This can be accomplished manually or with goal seeking functions available in current spreadsheet programs. Additionally, accurate measures of the total in-place pavement thickness are required to obtain reasonable estimates of the in situ SN_{eff}. While these measures can be readily obtained by selective pavement coring, variations in pavement layer thicknesses along a given project may invalidate many deflection test results where cores are not obtained and total pavement thickness must be estimated.

To illustrate the impacts of pavement thickness on the modified AASHTO approach, the parsed data set was re-analyzed using adjusted total pavement thicknesses equal to 90% of the actual input values. Figures 2.4.15 and 2.4.16 illustrate the impacts of associated pavement thickness errors on estimated SN_{eff} and overlay thickness requirements. As shown, SN_{eff} tends to be increasingly under-predicted as the input SN increases and the percentage of estimated overlay thickness errors less than +/- 0.5 inches drops to approximately 54%, which is essentially equal to results obtained with alternative methods which do not require pavement thickness as an input.



Figuire 2.4.15: SN Estimation Errors Based on Modified AASHTO with Decreased Thickness



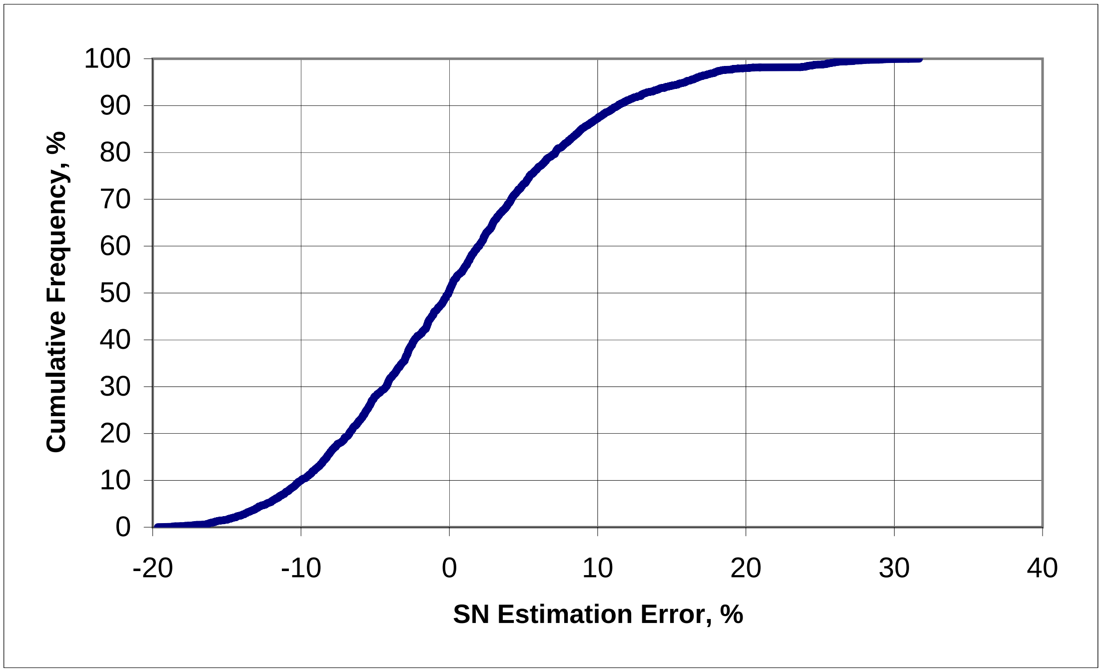
Figuire 2.4.16: Overlay Thickness Errors Based on Modified AASHTO with Decreased Thickness

Considering the implications of pavement thickness errors within the preferred AASHTO approach, it appears reasonable to consider the Eri-AUPP analysis method as a practical alternative to the modified AASHTO approach. While this method introduces greater error, the removal of pavement coring requirements may offset this deficiency. Based on the results of the factorial analysis, this method provided estimations of SN_{eff} values which were within 10% of the input values for 77% of the pavement structures investigated as illustrated in Figures 2.4.17 and 2.4.18. Furthermore, this method is projected to have associated overlay thickness errors of less than +/- 1.0 inches 84% of the time, with errors within +/- 0.5 inches occurring for 49% of the trials, as illustrated in Figures 2.4.19 and 2.4.20.

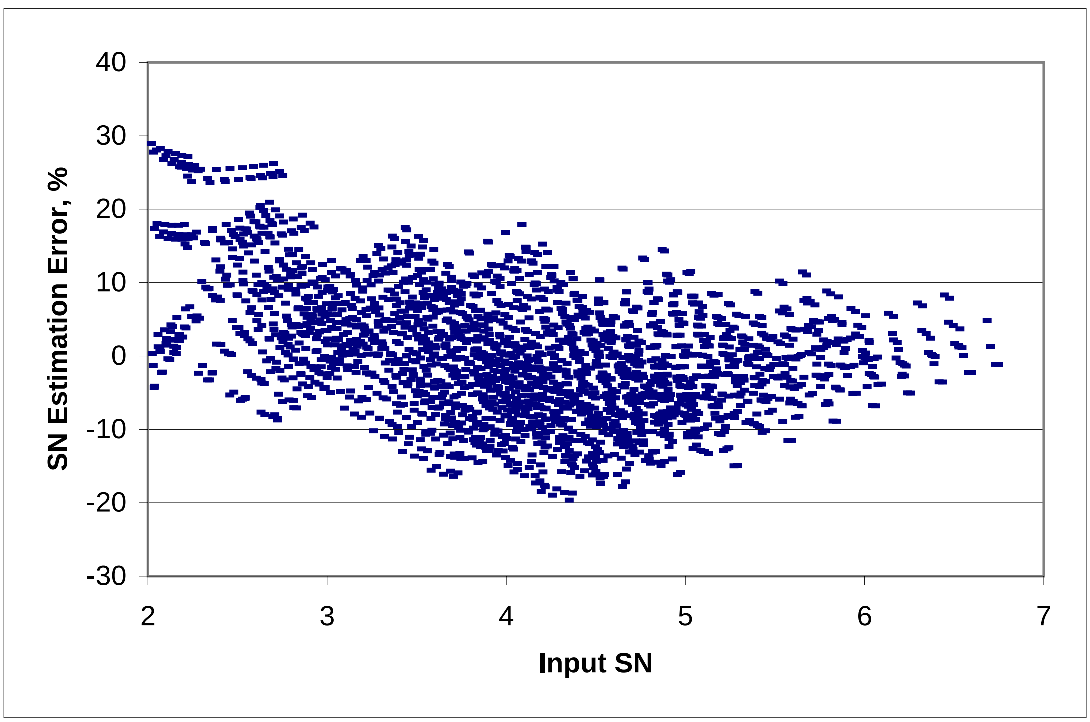
The Eri-AUPP analysis method is summarized as follows:

1. Surface deflections obtained at 36 inches from the center of loading are used to estimate the breakpoint subgrade resilient modulus, Eri by Eq 2.11.
2. Surface deflections measured at 0, 12, 24 and 36 inches from the center of loading are used to compute the Area Under the Pavement Profile, AUPP by Eq 2.8.
3. The pavement flexural rigidity term $E^{1/3}T$ is estimated based on the previously estimated Eri (Step 1) and calculated AUPP by Eq 2.12.
4. The estimated SN_{eff} is determined based on the estimated Eri and $E^{1/3}T$ values by Eq 2.13.

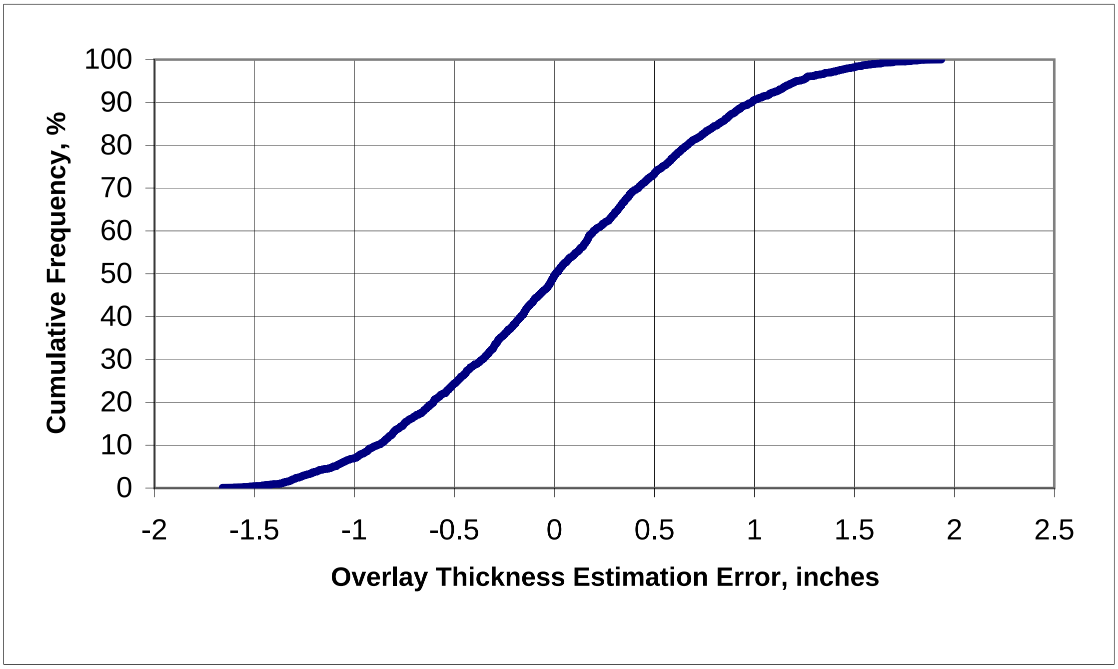
The Eri-AUPP analysis is relatively simple to apply and can be easily implemented in spreadsheet format. Because no pavement coring is required in this method, the process offers an attractive alternative to the modified AASHTO method.



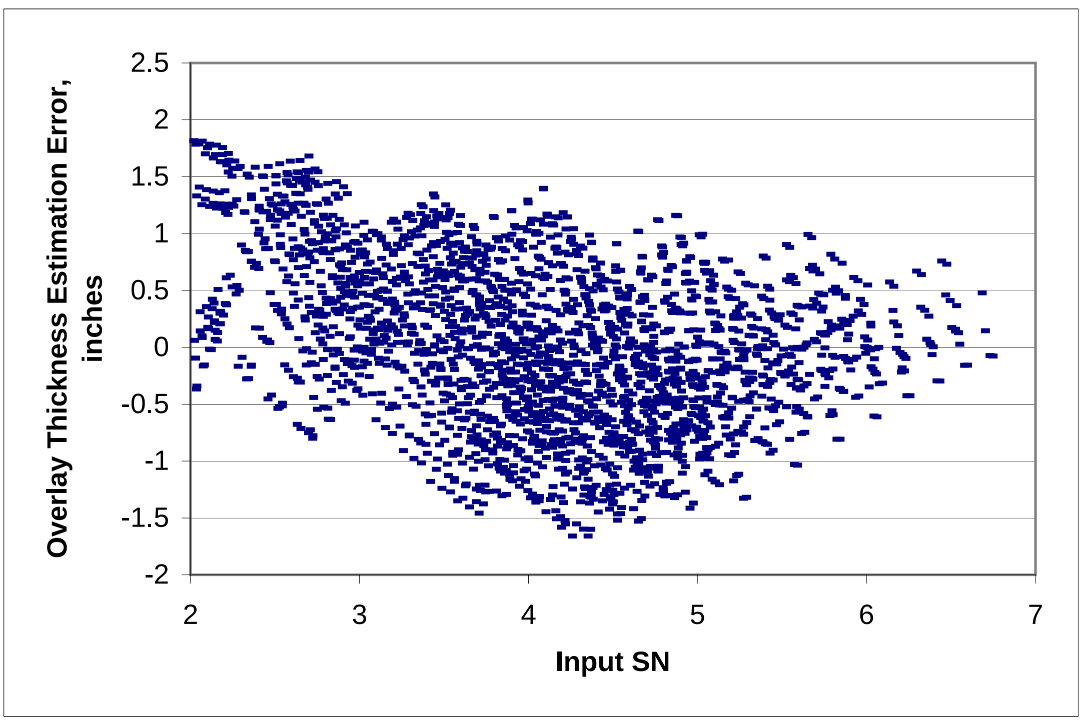
Figuire 2.4.17: SN Estimation Errors Based on Modified Eri-AUPP



Figuire 2.4.18: SN Estimation Errors Based on Modified Eri-AUPP



Figuire 2.4.19: Overlay Thickness Errors Based on Modified Eri-AUPP



Figuire 2.4.20: Overlay Thickness Errors Based on Modified Eri-AUPP

CHAPTER 3
OVERLAY DESIGN CONSIDERATIONS

3.1 Introduction

The current WisDOT flexible pavement design procedures published in Procedure 14-10-5 of the Facilities Development Manual (FDM) provide for the determination of the required SN for a new pavement based on provided soils and traffic data. FDM Procedure 14-10-30 provides little guidance for the design of a structural HMA overlay over an existing flexible pavement. Guidelines 44.15 and 44.20 of the Highway Maintenance Manual provide criteria for the selection of candidate projects for maintenance overlays. These overlays are limited to a maximum thickness of 2.5 inches and have an expected service life of 2 – 18 years depending on existing pavement condition and repair process.

The overlay design procedures developed during this research represent an extension to current maintenance policies, allowing the designer to target HMA overlay thickness to the structural capacity of the existing flexible pavement as defined by its effective structural number, SN_{eff}. Within this context, maximum overlay thickness requirements resulting from the application of these procedures are not constrained. Furthermore, overlays may be designed by these procedures to serve any desired service life; however a practical maximum of 20 years may be considered appropriate for the AASHTO based structural deficiency approach.

It is recognized that the design of structural overlays based on surface deflections and pavement coring may not be economically justifiable for all pavements. For lightly trafficked pavements with design ESAL values less than 1 million, structural overlay thickness requirements may be developed based on surface distress evident within the existing pavement. It is recommended that the guidelines provided by AASHTO and presented in Table 1.2.2 be utilized to estimate the SN_{eff} for these pavements.

For heavier trafficked roadways with design ESAL values greater than 1 million, it is recommended that the deflection-based procedures presented in Section 2.4.5 of this report be used to establish the SN_{eff} of the in-place pavement and ultimately the overlay thickness requirement based on the structural deficiency approach. This approach is summarized by the following:

$$T_{OL} = (SN_{new} - SN_{eff}) / 0.44 \qquad \text{Eq. 3.1}$$

Where:
- T_{OL} = overlay thickness, inches
- SN_{new} = Required SN for a new flexible pavement based on current WisDOT design procedures
- SN_{eff} = effective SN of the in situ pavement determined from procedures presented in Section 2.4.5

The SN_{eff} analysis procedures presented in Section 2.4.5 provide guidance on the analysis of collected deflection data with or without coring data. During initial implementations, it is recommended that comparative overlay thickness design be developed using both analysis paths (with and without coring data). The following provides further information relating to the conduct of the deflection testing program which is integral to each analysis method.

3.2 Deflection Testing Procedures

For use in the overlay design process, deflection testing should be conducted with a falling weight deflectometer (FWD). The FWD has been shown to provide data which closely simulates the effects of moving wheel loads and is considered the current state-of-practice for deflection testing. WisDOT currently owns and operates an FWD and numerous private agencies also own/operate equipment of this type and can provide data collection services as needed. The major drawback to the use of the FWD is its requirement for traffic control during testing which can significantly increase project costs. High-speed deflection test equipment is currently under development to eliminate this need for traffic control, but at the time of this report it is too soon to know if data collected by these devices will be directly transferable.

Test loads of approximately 9,000 lbs are recommended to provide data which simulates the action of a standard 18,000 single axle load. At least one 9,000 lb seating load should be applied at each test location prior to data collection. Deflection testing should be conducted along the outer wheel path to obtain representative data from this critical pavement area. Test spacings should be selected at 100 - 250 ft intervals to provide sufficient data to characterize the variations in pavement quality along the entire project length, with a minimum of 10 deflection tests collected from any given project. Deflection testing may conducted in sound (uncracked) and unsound locations. As a general rule, deflections should not be collected within six feet of isolated transverse cracks. Deflections may be obtained in areas of fatigue cracking; however these locations should be identified such that the effects on SN_{eff} may be considered during the overlay thickness analysis.

Deflection testing should be conducted during warmer periods when there is no chance of frost or frozen layers within the pavement structure. Pavement temperatures should be recorded during testing and used to adjust the maximum measured deflection to a reference pavement temperature of 68^{o}F. It is recommended that direct measurement of the mid-depth HMA pavement temperatures be made during testing. Procedures for correcting maximum deflection to the reference temperature of 68^{o}F (20^{o}C) are provided by FHWA (8). Figure 3.2.1 provides a figure extracted from this report which illustrates the temperature correction factor as a function of HMA thickness.

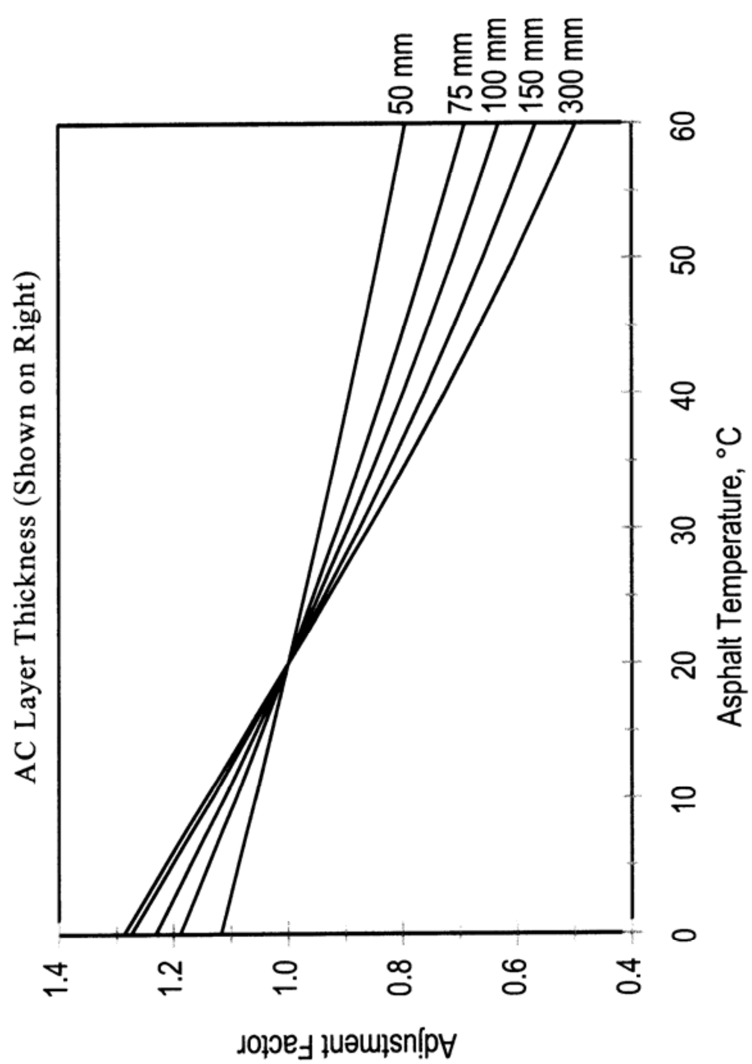


Figure 3.2.1 Maximum Deflection Temperature Adjustment Factor (from FHWA-RD-98-085)

3.3 Project Analysis

The deflection-based analysis procedures presented in section 2.4.5 should be applied to the data from each test location to establish a profile of structural overlay thickness requirements along the entire project limits. Examination of this profile may indicate that sub-sectioning is warranted, with variable overlay thickness requirements established within each subsection. Whether or not sub-sectioning is warranted, a statistical analysis of overlay thickness requirements should be conducted to establish the mean and standard deviation for each design section. These values may then be utilized to develop reliability-based overlay thickness designs.

It should be noted that any outliers in the data set, resulting from atypical overlay thickness requirements for unusually weak or strong pavement locations, should be considered for exclusion prior to the calculation of the section mean and standard deviation, especially for smaller data sets. Unusually strong pavement outliers will result in a reduced mean and an increased standard deviation, with the magnitude of the change dependent on sample size. Depending on the level of design reliability selected, these changes could result in a decreased or increased overlay thickness requirement. For example, consider a small data set of 10 test locations with location-specific overlay thickness requirements of 3.5, 3.7, 3.5, 4.0, 3.9, 4.2, 3.6, 4.0, 4.2, 2.0 inches. For this data set, one may consider the 2.0 inch overlay thickness as an outlier due to a strong pavement location. Including this outlier results in a mean overlay thickness requirement of 3.66 inches and a standard deviation of 0.640 inches while excluding the outlier results in a mean overlay thickness requirement of 3.84 inches and a standard deviation of 0.279 inches. For a design reliability of 50%, setting the overlay thickness requirement equal to the mean value obviously results in a reduced requirement if the strong outlier is included in the calculations (3.66 vs 3.84 inch design overlay). If, however, the design reliability is increased to 68% (mean + 1 standard deviation), the data set including the outlier would result in a design overlay thickness of 4.30 inches compared to 4.12 inches if the outlier was excluded. While this is a simplified example, it serves to highlight the unexpected impacts of outliers.

Unusually weak pavement locations should also be considered for exclusion during the selection of overlay thickness requirements if it is anticipated that these locations will be repaired by base patching prior to overlay. However, if no pre-overlay repairs are considered then it may be logical to include the outlier in the statistical analysis. It may also be warranted to conduct an economic analysis of overlay requirements and associated costs both with and without the pre-overlay repair of weakened sections to establish the most cost-effective overlay design scenario.

CHAPTER 4
FINDINGS AND RECOMMENDATIONS

This report presents the results of a review of published procedures for the design of HMA overlays of existing flexible pavements. Overlay design procedures utilized by surrounding States were also examined and summarized. Based on these reviews, a number of key data elements have been identified for consideration within the proposed WisDOT overlay design procedures for flexible pavements, including surface distress, ride quality, and pavement deflections.

The overlay design procedures developed as part of this research are appropriate for the design of structural HMA overlays which are intended to significantly increase the load-carrying capacity of existing flexible pavements. These procedures are intended to extend the current options available to designers as published within the WisDOT Facilities Development Manual (FDM) and the Highway Maintenance Manual (HMM).

4.1 Summary of Findings

The analyses conducted as part of this research resulted in the following findings:

(1) The design of structural HMA overlays of existing flexible pavements can be integrated within current WisDOT procedure for the design of new flexible pavements by utilizing the structural deficiency approach. This process establishes the required overlay thickness based on the difference between the effective structural number, SN_{eff}, of the existing pavement existing and the structural number required for a new flexible pavement design.

(2) The SN_{eff} of existing flexible pavements can be established based on deflections, distress, or ride quality. The use of deflection data is considered appropriate for pavements with

design traffic loadings in excess of 1 million ESALs. For lightly trafficked pavements the SN_{eff} may be developed without the use of deflection data. The accuracy of SN_{eff} estimations can be improved by including pavement layer thickness data obtained through selective coring; however, all analysis techniques have associated errors.

(3) Modified deflection-based SN_{eff} analysis procedures were developed based techniques presented in the 1993 AASHTO Guide for the Design of Pavement Structures. These procedures provided the best correlations between SN_{eff} and input SN using deflection data generated during computer modeling of a large pavement factorial. These procedures are somewhat cumbersome to apply and are best suited for analysis when pavement layer thicknesses are known. Based on the results presented, these procedures were shown to provide overlay thickness recommendations which were within ½ inch of "truth", as represented by exact component analysis of the pavement structures investigated during computer modeling, for 90% of the structures investigated.

(4) Alternative deflection-based analysis techniques developed as part of this research were also shown to provide reasonable correlations between SN_{eff} and input SN using deflection data generated during computer modeling of the large pavement factorial. These procedures are easier to apply and do not require knowledge of the in-place pavement layer thicknesses. Based on the results presented, these procedures were shown to provide overlay thickness recommendations which were within ½ inch of "truth" for 40% of the pavement structures investigated and within 1 inch of truth for 84% of the structures. These values were shown to be comparable to the modified AASHTO approach if the assumed pavement thickness is in error by 10%.

4.2 Recommendations

Based on the findings from this research, it is recommended that the structural deficiency approach be implemented for the design of structural HMA overlay thickness requirements for existing flexible pavements. The procedures presented in this report are considered appropriate for establishing thickness requirement for structural HMA overlays. Thickness requirements resulting from the application of these methods are not intended to supersede minimum/maximum HMA layer thickness guidelines as detailed in the WisDOT Standard Specifications, Section 460.3.2.

The structural deficiency approach utilizes both the effective structural number, SN_{eff}, of the existing pavement and the structural number required for new design. It is recommended that the deflection-based analysis procedures presented in Section 2.4.5 of this report be promoted to estimate the effective structural number, SN_{eff}, of the existing flexible pavement that are projected to carry at least 1 million ESALs after overlay. During initial implementations, both the modified AASHTO and revised AUPP-Eri should be utilized to establish SN_{eff} and asses the impacts of analyses with and without available coring data.

For lightly trafficked pavements with less than 1 million design ESALs, it is recommended that the SN_{eff} be established based on the deflection based-analysis techniques or a component analysis based on layer thickness and existing pavement distress. The guidelines presented by AASHTO for the selection of structural layer coefficients based on existing distress are recommended for use when deflection data is unavailable and the component analysis is selected.

The recommended overlay thickness design procedures are compatible with the current WisDOT procedures for the design of new flexible pavements, as published within Procedure 14-10-5 of the Facilities Development Manual (FDM). When deflection data are utilized, the field subgrade modulus is determined directly from deflections. This value may require seasonal adjustments depending on the time of deflection testing as well as conversion to a representative soil support value following standard WisDOT procedures.

The overlay design procedures presented in this report may be utilized to develop thickness requirements for any user-supplied design life. The practical limitation for these procedures is a 20-year design life which is consistent with the maximum design life currently assumed for the design of traditional HMA pavements in Wisconsin following FDM Procedure 14-10-5. Shorter design lives can be considered by developing new pavement SN requirements using projected traffic levels within the 1972 AASHTO equation currently used by WisDOT.

5.0 REFERENCES

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