Development of Pavement Structural Capacity Requirements for Innovative Pavement Decision-Making and Contracting: Phase II

http://www.virginiadot.org/vtrc/main/online_reports/pdf/16-r20.pdf

JAMES M. BRYCE
Graduate Research Assistant, Center for Sustainable Transportation Infrastructure, Virginia Tech Transportation Institute

GERARDO W. FLINTSCH, Ph.D., P.E.
Director, Center for Sustainable Transportation Infrastructure, Virginia Tech Transportation Institute, and Professor, Charles E. Via, Jr. Department of Civil and Environmental Engineering, Virginia Tech

SAMER W. KATICHA, Ph.D.
Senior Research Associate, Center for Sustainable Transportation Infrastructure, Virginia Tech Transportation Institute

BRIAN K. DIEFENDERFER, Ph.D., P.E.
Associate Principal Research Scientist, Virginia Transportation Research Council

ALEX SARANT
Undergraduate Research Assistant, Center for Sustainable Transportation Infrastructure, Virginia Tech Transportation Institute

Final Report VTRC 16-R20

VIRGINIA TRANSPORTATION RESEARCH COUNCIL
530 Edgemont Road, Charlottesville, VA 22903-2454

www.VTRC.net

Standard Title Page - Report on Federally Funded Project

1. Report No.: FHWA/VTRC 16-R20	2. Government Accession No.:	3. Recipient's Catalog No.:
4. Title and Subtitle: Development of Pavement Structural Capacity Requirements for Innovative Pavement Decision-Making and Contracting: Phase II		5. Report Date: June 2016 6. Performing Organization Code:
7. Author(s): James M. Bryce, Gerardo W. Flintsch, Ph.D., P.E., Samer W. Katicha, Ph.D., Brian K. Diefenderfer, Ph.D., P.E., and Alex Sarant		8. Performing Organization Report No.: VTRC 16-R20
9. Performing Organization and Address: Virginia Tech Transportation Institute 3500 Transportation Research Plaza (0536) Blacksburg, VA 24061		10. Work Unit No. (TRAIS): 11. Contract or Grant No.: 103933
12. Sponsoring Agencies' Name and Address: Virginia Department of Transportation 1401 E. Broad Street Richmond, VA 23219	Federal Highway Administration 400 North 8th Street, Room 750 Richmond, VA 23219-4825	13. Type of Report and Period Covered: Final Contract 2012-2014 14. Sponsoring Agency Code:
15. Supplementary Notes:		

16. Abstract:

The structural capacity of a pavement has been shown to be a good indicator of the required maintenance and expected performance of pavements. However, results in the literature provide few indices for rigid and composite pavements.

This study developed an index that produced promising results for use with composite pavements. The index relates deflection measurements to the performance of the pavement. Limited conclusions could be drawn from the results of rigid pavement testing due to the limited sample size and scope of the available data. The report presents steps for including the structural indices in pavement evaluation and management processes and a tool that can be used by engineers to calculate the structural indices for flexible and composite pavements based on defined input parameters.

A number of recommendations for implementation are made based on the findings of this study.

17 Key Words: Deflection Testing, Non-Destructive Evaluation, Network-Level Decision Making, Structural Capacity Index		18. Distribution Statement: No restrictions. This document is available to the public through NTIS, Springfield, VA 22161.	
19. Security Classif. (of this report): Unclassified	20. Security Classif. (of this page): Unclassified	21. No. of Pages: 45	22. Price:

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

FINAL REPORT

DEVELOPMENT OF PAVEMENT STRUCTURAL CAPACITY REQUIREMENTS FOR INNOVATIVE PAVEMENT DECISION-MAKING AND CONTRACTING: PHASE II

James M. Bryce
Graduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Gerardo W. Flintsch, Ph.D., P.E.
Director
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute
and
Professor
Charles E. Via, Jr. Department of Civil and Environmental Engineering
Virginia Tech

Samer W. Katicha, Ph.D.
Senior Research Associate
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Brian K. Diefenderfer, Ph.D., P.E.
Associate Principal Research Scientist
Virginia Transportation Research Council

Alex Sarant
Undergraduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

VTRC Project Manager
Brian K. Diefenderfer, Ph.D., P.E., Virginia Transportation Research Council

In Cooperation with the U.S. Department of Transportation
Federal Highway Administration

Virginia Transportation Research Council
(A partnership of the Virginia Department of Transportation
and the University of Virginia since 1948)

Charlottesville, Virginia

June 2016
VTRC 16-20

DISCLAIMER

The project that is the subject of this report was conducted under contract for the Virginia Department of Transportation, the Virginia Transportation Research Council. The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Virginia Department of Transportation, the Commonwealth Transportation Board, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation. Any inclusion of manufacturer names, trade names, or trademarks is for identification purposes only and is not to be considered an endorsement.

Each contract report is peer reviewed and accepted for publication by staff of the Virginia Transportation Research Council with expertise in related technical areas. Final editing and proofreading of the report are performed by the contractor.

ABSTRACT

The structural capacity of a pavement has been shown to be a good indicator of the required maintenance and expected performance of pavements. However, results in the literature provide few indices for rigid and composite pavements.

This study developed an index that produced promising results for use with composite pavements. The index relates deflection measurements to the performance of the pavement. Limited conclusions could be drawn from the results of rigid pavement testing due to the limited sample size and scope of the available data. The report presents steps for including the structural indices in pavement evaluation and management processes and a tool that can be used by engineers to calculate the structural indices for flexible and composite pavements based on defined input parameters.

A number of recommendations for implementation are made based on the findings of this study.

FINAL REPORT

DEVELOPMENT OF PAVEMENT STRUCTURAL CAPACITY REQUIREMENTS FOR INNOVATIVE PAVEMENT DECISION-MAKING AND CONTRACTING: PHASE II

James M. Bryce
Graduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Gerardo W. Flintsch, Ph.D., P.E.
Director
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute
and
Professor
Charles E. Via, Jr. Department of Civil and Environmental Engineering
Virginia Tech

Samer W. Katicha, Ph.D.
Senior Research Associate
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

Brian K. Diefenderfer, Ph.D., P.E.
Associate Principal Research Scientist
Virginia Transportation Research Council

Alex Sarant
Undergraduate Research Assistant
Center for Sustainable Transportation Infrastructure
Virginia Tech Transportation Institute

INTRODUCTION

Pavement condition is classified into two main categories: the structural condition and the functional condition (Park et al., 2007). The failure of a pavement (i.e., the inability of the pavement to carry traffic at a specified level of service) is generally measured in terms of functional condition. For example, a pavement may be said to have failed if the roughness of the pavement exceeds certain thresholds. This differs from the structural condition of a pavement, which is generally not perceived directly by the user. The structural condition of the pavement describes the ability of the pavement to carry its design load without appreciable permanent deformation or damage (Park et al., 2007). The structural condition of the pavement has a direct impact on the functional performance of the pavement (Bryce et al., 2013). As the structural condition of the pavement deteriorates, the rate of change of the functional condition increases. Therefore, a pavement with poor structural condition will fail more rapidly than a pavement with adequate structural condition, as measured in terms of functional condition.

Research has shown that a needs analysis for the pavement network can be improved if the structural capacity of pavements is included in network-level pavement management (Zaghloul et al., 1998; Zhang et al., 2003; Bryce et al., 2013). However, literature documenting the development of structural capacity indices for the network-level evaluation has generally been limited to flexible pavements (Ali and Tayabji, 1998; Zhang et al., 2003; Chakroborty et al., 2007; Bryce et al., 2013). It is generally accepted that deflection tests conducted at the network level on rigid pavements provide little to no useful information regarding expected performance (Zhang et al., 2003), or no generally accepted model has been developed, as in the case of composite pavements.

This project continues and expands upon an earlier Virginia Department of Transportation (VDOT) study that focused on the structural evaluation of pavements at the network level to enhance the quality of decision-making within the VDOT pavement management system (PMS). The first phase of this project, titled "Developing a Network-Level Structural Capacity Index for Structural Evaluation of Pavements," developed and proposed a structural capacity index for asphalt pavements (Bryce et al., 2013) and presented data from a limited number of sections on VDOT's interstate network. The developed Modified Structural Index (MSI) uses the effective structural number (SN_{eff}) of the pavement and a measure of the required structural number of the pavement (SN_{Req}) to create an index that could then be used to determine the structural adequacy of the pavement section. Two important findings from the first phase were that (1) traffic greatly influenced the structural adequacy of a pavement (as measured by the MSI) and (2) the MSI could be used to predict the performance (i.e., the rate of deterioration of the functional condition over time) of a pavement segment more accurately than empirical estimates based on historic functional condition deterioration rates of similar pavements.

PURPOSE AND SCOPE

The scope of this project encompassed two tasks:

1. *Calculate MSI values for the entire interstate pavement network for flexible pavements, and develop plots of the data for use by VDOT district pavement engineers*. This effort expanded on the pilot study of Bryce et al. (2013) and obtained the MSI of all roadway sections where network-level falling weight deflectometer (FWD) data are available. These MSI values were summarized by counties.

2. *Determine if a network-level structural-based condition index for concrete-surfaced and composite pavements can be developed and implemented in the VDOT PMS.*

For the first task, all the deflection data for flexible interstate pavements were compiled into a single database, along with the traffic and pavement depth information obtained from the VDOT PMS. The MSI procedure described in Bryce et al. (2013) was then used to evaluate the flexible pavement network. For the second task, the existing literature that focused on methods to implement pavement structural condition information into network-level pavement management applications was reviewed. The current VDOT methodology used to incorporate structural information into the network-level decision process was also thoroughly evaluated. The different methods found in the literature were then evaluated by comparing the predicted performance (using the models found in literature and developed in this report) to the actual performance of the pavements (taken from the VDOT data). Finally, improved indices for composite and rigid pavements were developed based on VDOT data.

BACKGROUND

VDOT Structural Capacity Tests

From 2005 to 2007, VDOT conducted structural capacity tests using the FWD along its entire interstate network at a spacing of 0.2 mi (Diefenderfer, 2008). The effectiveness of the testing protocol and frequency was evaluated and reported in Alam et al. (2007) and Galal et al. (2007). The FWD is a static device that imparts a pulse load at the pavement surface and measures the resulting pavement deflection. The response from the pulse load is measured by a set of geophones spanned radially across the pavement surface. The FWD is the most widespread tool used to non-destructively assess the structural capacity of pavements (Hadidi and Gucunski, 2010). Additional details of the FWD can be found in MACTEC (2006).

Current VDOT Practice

Data Collection

Data collection and management practices are critical to assuring that VDOT manages its more than 125,000 lane-miles of roads throughout the state as efficiently as possible. Data collection practices include evaluating the functional condition of the entire interstate and primary road networks every year, and the secondary road network every 2 to 3 years. For flexible and composite pavement sections, the distress data collection process includes evaluating captured digital images using automated systems (Chowdhury, 2010). The collection of the pavement distress data has been contracted to an outside contractor since 2006. The contractor uses a vehicle equipped with continuous digital imaging, automated crack detection, and sensors that measure roughness and rutting data (VDOT, 2010). Rigid pavement sections are manually rated. The final data set is stored as part of the VDOT PMS, along with databases of historical pavement condition and construction history, among other information.

Rigid Pavements

The collected distress data are combined into a set of indices that provide a measure of the functional condition of the pavement. The indices developed are different for the two types of rigid pavements: continuous reinforced concrete (CRC) pavement and jointed concrete pavement (JCP). The distress indices used for CRC are known as the Concrete Distress Rating (CDR) and the Concrete Punch-out Rating (CPR) (VDOT, 2011). The index used for JCP is the Slab Distress Rating (SDR). All indices have a discrete scale that ranges from 0 to 100. Table 1 and Table 2 show the index ranges that are expected to trigger the different maintenance categories during a network-level evaluation. Table 1 shows that preventive maintenance is not a maintenance category used for CRC pavements.

Table 1. Maintenance Activities and Index Ranges for CRC Pavements (VDOT, 2011)

Maintenance Action	CDR/CPR Range
Do Nothing (DN)	≥80
Corrective Maintenance (CM)	65-79
Restorative Maintenance (RM)	50-64
Major Rehabilitation/Reconstruction (RC)	<50

Table 2. Maintenance Activities and Index Ranges for JCP (VDOT, 2011)

Maintenance Action	SDR Range
Do Nothing (DN)	≥ 80
Preventive Maintenance (PM)	70 – 79
Corrective Maintenance (CM)	60 – 69
Restorative Maintenance (RM)	50 – 59
Major Rehabilitation/Reconstruction (RC)	< 50

Composite Pavements

The indices calculated for composite pavements are the same as the ones used for flexible pavements. That is, two different indices (i.e., the Load-related Distress Rating [LDR] and the Non-load-related Distress Rating [NDR]) are calculated from the data collected during the distress survey; the minimum value of the two is defined as the Critical Condition Index (CCI). The LDR is calculated from load-related distresses, such as fatigue cracking in the wheel path (VDOT, 2010). The NDR is calculated from non-load-related distresses, such as construction deficiencies, bleeding, etc. (VDOT, 2010). CCI values range from 0to 100; the index ranges that are expected to trigger maintenance actions during a network-level evaluation are shown in Table 3.

Table 3. Pavement Condition Definitions (VDOT, 2010)

Index Scale (CCI)	Pavement Condition	Likelihood of Corrective Action
90 and Above	Excellent	Very Unlikely
70-89	Good	Unlikely
60-69	Fair	Possibly
50-59	Poor	Likely
49 and Below	Very Poor	Very Likely

Deflection Data Collection

Network-level deflection data were collected between 2005 and 2007 on approximately 2,300 directional miles (3,700 directional km) of interstate pavements. Additional details about the collection of the deflection data can be found in Diefenderfer (2008). The deflection testing was conducted using a Dynatest Model 8000 FWD placed in the wheel path of the travel (right-hand) lane of the roadway in both directions. The placements of the sensors during the testing were at 0, 8, 12, 18, 24, 36, 48, 60, and 72 in. (0, 200, 305, 457, 610, 915, 1220, 1524, and 1830 mm) from the center of a load plate. The testing interval was 0.2 mi (320 m). The testing was conducted at three load levels: 9, 12, and 16 kips (40, 53 and 71 kN; Diefenderfer, 2008).

METHODS

This section describes the methodology used to expand the MSI to the entire interstate network and the methods identified to produce candidate structural indices for rigid and composite pavements.

MSI for Flexible Pavement Segments

A brief description of the MSI follows in this section; additional details about the development of the MSI can be found in Bryce et al. (2013). The MSI is based on the Structural Capacity Index (SCI) (Equation 1) proposed by the Texas Department of Transportation (Zhang et al., 2003). However, to facilitate the calculation of large data sets in a reasonable timeframe, it was decided in Bryce et al. (2013) that the MSI should be a closed-form equation. The calculation of SN_{eff} in the SCI equation can be made using many methods, but the method developed by Rohde (1994) was chosen (Equation 2). A closed-form estimator for the American Association of State Highway and Transportation Officials (AASHTO) structural number (SN) equation (AASHTO, 1993) was developed to calculate the SN_{Req}. The estimator specifically used for flexible pavements on the interstate network is given in the denominator of Equation 4 of the MSI (Bryce et al., 2013).

$$SCI = \frac{SN_{Eff}}{SN_{Req}} \qquad \text{(Eq. 1)}$$

where SN_{eff} is the effective structural number of the pavement and SN_{Req} is the required structural number of the pavement.

$$SN_{eff} = k1 * SIP^{k2} * H_p^{k3} \qquad \text{(Eq. 2)}$$

where, for asphalt pavements, k1 = 0.4728, k2 = -0.4810, k3 = 0.7581, SIP is calculated using Equation 3, and H_P is the depth of the pavement structure (all layers above the subgrade).

$$SIP = D_0 - D_{1.5Hp} \qquad \text{(Eq. 3)}$$

where D_0 is the peak deflection under the 9,000-lb. load, and $D_{1.5Hp}$ is the deflection at 1.5 times the pavement depth (found through a Lagrangian interpolation function).

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05716 * (\log(ESAL) - 2.32 * \log(M_R) + 9.07605)^{2.36777}} \quad \text{(Eq. 4)}$$

where ESAL is the equivalent single axle load (VDOT, 2003) and M_R is the subgrade resilient modulus (Equation 5).

$$M_R = \frac{C * P * 0.24}{D_r * r} \quad \text{(Eq. 5)}$$

where AASHTO recommends C = 0.33 (VDOT, 2003), P is the load applied (in pounds), and D_r is the deflection at distance r from the center deflection. The distance r should be chosen so that it is sufficiently far enough from the load to measure deflections originating only in the subgrade. Typically, the value for r is taken as the distance to the first sensor that is greater than the distance of 1.5 times the pavement depth.

MSI Evaluation for VDOT Interstates

Equation 4 was used in this project to calculate the MSI for all of the interstate flexible pavement sections on which deflection testing was performed (at a spacing of 0.2 mile). Thickness information was provided by VDOT. The first step was to estimate the pavement deflection at 1.5 times the pavement height ($D_{1.5Hp}$). To estimate the pavement deflection at $D_{1.5Hp}$, the three deflection points closest to $D_{1.5Hp}$ were identified and entered into Equation 6. The height of the pavement (or an estimate in some cases) was provided by VDOT for each testing location.

$$D_{1.5Hp} = \frac{(x-B)*(x-C)}{(A-B)*(A-C)} * (D_A) + \frac{(x-A)*(x-C)}{(B-A)*(B-C)} * (D_B) + \frac{(x-A)*(x-B)}{(C-A)*(C-B)} * (D_C) \quad \text{(Eq. 6)}$$

where x is 1.5 times the depth of the pavement (Hp); A, B, and C are points closest to x where the deflection is known; and D_A, D_B, and D_C are the deflections at points A, B, and C, respectively.

The M_R was calculated using the deflection information 60 in away from the applied load because it was assumed that this deflection value was far enough from the load to originate solely from the subgrade (Rohde, 1994). The final step for calculating the MSI was to calculate the traffic in terms of ESALs (Equation 7). The annual average daily traffic (AADT) was obtained from the 2012 traffic data available on the VDOT website (VDOT, 2013). The percent of trucks was also available with the traffic data.

$$ESAL = (AADT)(T)(T_f)(G)(D)(L)(365)(Y) \quad \text{(Eq. 7)}$$

where T is the percent of trucks, T_f is the truck factor, G is a growth factor, D is a directionality factor, L is a lane factor, and Y is the number of years in the design period.

The truck factor (T_f) was set at 0.46 for Single Unit Trucks and Buses and 1.05 for Combination Trucks (Smith and Diefenderfer, 2009). A 3% growth rate was assumed across a 20-year design period. The 20-year design period and the lane distribution factor (see Table 4) were determined using VDOT standard design assumptions (VDOT, 2003) and verified in Bryce et al. (2013).

Table 4. Lane Distribution Factors

Number of Lanes	Lane Distribution Factor
2	0.9
3	0.7
4 or more	0.6

Development of a Structural Index for Rigid Pavements

Unlike the case for flexible pavements, only a relatively small number of structural indices have been proposed for use with rigid pavements. This is because many past projects have not identified a link between parameters derived from pavement deflection values and the pavement performance, which is a critical step in developing a structural index for rigid pavements (Carvalho et al., 2012). This section describes a few methodologies proposed for evaluating the structural condition of rigid pavements based on surface deflections.

Alaska Department of Transportation

As described in Carvalho et al. (2012), the Alaska Department of Transportation (AkDOT) recently implemented a methodology for including FWD results in its PMS. However, the methodology includes back-calculations to obtain pavement properties, which may preclude use at the network level where large data sets are encountered. Secondly, a validation of the AkDOT technique has yet to be made and will require data across a certain number of years.

Structural Strength Indicator

Flora (2009) proposed a statistical indicator for flexible and rigid pavements based on the center deflections from the FWD. The indicator proposed, the Structural Strength Indicator (SSI), is a function based on the cumulative distribution of the center deflections across a given pavement family. The SSI function is developed on the basis of Equation 8 and is in the form of Equation 9 (Flora, 2009).

$$SSI = 100 * \left[1 - F\left[\left(\delta_{i\,jk}\right)_1\right]\right] \quad \text{(Eq. 8)}$$

where $F[(\delta_{ijk})_1]$ is the Cumulative Empirical Probability Distribution of $(\delta_{ijk})_1$.

$$SSI_{jk} = 100\left(1 - \alpha e^{\frac{-\beta}{(d_1)^{\gamma}}}\right) \quad \text{(Eq. 9)}$$

where δ in Equation 8 is the center deflection; the subscripts j and k denote the pavement family; and α, β, and γ are found for each pavement family by minimizing the errors between Equations 8 and 9.

The SSI is based on determining the probability that a pavement in a given family will have a deflection larger than the measured deflection in a given highway section. The index is on a scale of 0 to 100, with 0 being a poor SSI and 100 being a perfect SSI. To use the values from the SSI in this project, a set of thresholds would need to be developed for acceptable center deflections for VDOT rigid pavements, and the relationship between structural adequacy and center deflections would need to be verified.

VDOT Enhanced Decision Trees

VDOT incorporates the structural adequacy of rigid pavements into the network-level decision process through a set of enhanced decision trees. The enhanced decision trees were introduced in 2008 as a method to complement decisions based solely on pavement surface distresses (VDOT, 2011). The enhanced decision trees lead to a recommended treatment level for rigid pavements as a function of the treatment level chosen from pavement distresses, the average annual daily truck traffic (AADTT), the FWD center deflection, and the AREA (a measure of the deflected area of the pavement resulting from an applied load). According to the enhanced decision trees, a structurally adequate rigid pavement is considered a pavement with an AREA value greater than or equal to 32 in (81 cm) and an FWD center deflection less than or equal to 5 mils (0.13 mm).

FHWA Simplified Method

Carvalho et al. (2012) proposed a method based on predicting acceptable probabilities of pavement deterioration via certain predictor variables. Using an analysis from the Mechanistic-Empirical Pavement Design Guide (MEPDG), the following distress thresholds were set for rigid pavements: a pavement International Roughness Index (IRI) of 172 in/mi (2.7 m/km), joint faulting of 0.12 in (3 mm), and transverse cracking less than or equal to 15% of slabs. The analysis was conducted for a 20-year pavement life. Thus, instead of using deflections to back-calculate the elastic moduli of pavement layers, mechanistic validation was completed by simulating different pavement structures and material properties using a multilayer elastic theory.

Based on the analysis conducted, Carvalho et al. (2012) proposed models to predict if pavements would be acceptable throughout their design life. Deflections, stresses, and strains were used to compute deflection parameters for each model and to estimate the damage and performance of the pavement. A generalized logistic model was proposed as the method to calculate the probability of performance of a given pavement based on pavement deflections. The generalized logistic model is given in Equation 10.

$$P(acceptable) = \frac{1}{1+e^{-b}} \qquad \text{(Eq. 10)}$$

where *P(acceptable)* is the probability that the pavement will perform at an acceptable level of service across a 20-year life, *e* is the base of the natural logarithm, and *b* is given by Equation 11.

$$b = c + \sum a_i m_i \qquad \text{(Eq. 11)}$$

where the values for c, a_i, and m_i are given in Table 5 for roughness performance and Table 6 for transverse slab cracking performance.

Table 5. Model Based on Roughness Performance

Coefficient	Variables
m_1	CI_5 (Equation 12)
m_2	Average daily Class 9 truck volume
a_1	-0.078
a_2	-0.0003877
c	2.057

Table 6. Model Based on Transverse Slab Cracking Performance

Coefficient	Variables
m_1	CI_4 (Equation 13)
m_2	Average daily Class 9 truck volume
a_1	-0.254
a_2	-0.001012
c	3.630

The model for joint faulting was not included in this project given that it required the pavement subgrade to be classified into specific types. In Table 5, CI_5 is calculated as follows:

$$CI_5 = d_{24} - d_{36} \qquad \text{(Eq. 12)}$$

Where d_{24} is the deflection (in micrometers) 24 in (61 cm) away from the center of the load plate and d_{36} is the deflection (in micrometers) 36 in (91 cm) away from the center of the load plate. CI_4 is calculated as follows:

$$CI_4 = d_{18} - d_{24} \qquad \text{(Eq. 13)}$$

Where d_{18} is the deflection (in micrometers) 18 in (46 cm) away from the center of the load plate and d_{24} is the deflection (in micrometers) 24 in (61 cm) away from the center of the load plate.

Pavement Performance

One important aspect of developing a structural index for rigid pavements is the concept of linking the pavement deflection parameters to pavement performance, as opposed to linking them to pavement condition. Pavement performance is generally defined as the area under the condition curve, as a function of time (Zimmerman, 1995). The linkage between expected pavement performance and the MSI for flexible pavements was discussed in Bryce et al. (2013) when the MSI was proposed as a potential variable to augment the deterioration curves used by VDOT. This project expands this concept and attempts to develop structural indices for rigid and composite pavements by studying the link between pavement performance and structural indicators.

Development of a Structural Index for Composite Pavements

Composite pavements within the state of Virginia are most often the result of the rehabilitation of rigid pavements and generally comprise hot-mix asphalt (HMA) over CRC or JCP. A study of composite pavements conducted by Hein et al. (2002) concluded that the pavement condition ratings based only on the HMA surface do not accurately reflect the condition of the overall pavement structure and/or concrete base (e.g., faulting and spalling may be effectively hidden from view). Furthermore, the study noted that there was often early (within three to five years) deterioration due to reflective cracking in the HMA from the discontinuities of the underlying rigid layers (Hein et al., 2002).

VDOT currently evaluates the structural adequacy of composite pavements based on the AREA and k-value of the pavement. The AREA of the pavement is a measure estimating the area of the deflection basin generated by the FWD, and the k-value is a measure of the modulus of the subgrade reaction (Huang, 2004). According to the VDOT (2011), a pavement with an AREA greater than 32 in and a k-value greater than 175 pci can be considered strong.

The literature review did not reveal any agencies outside of VDOT that currently use a structural index to evaluate the condition or expected performance of composite pavements at the network level. However, some methodologies have been proposed for the evaluation of the condition of composite pavements based on deflection basin parameters (Kim and Park, 2002; Horak, 2008). Flintsch et al. (2008) concluded in a study about composite pavement systems that composite pavements can mitigate certain distresses typical to flexible pavements, particularly if an asphalt overlay is placed on a concrete layer in good condition. Furthermore, the study showed that surface deflection of a composite pavement was approximately one-third that of deep strength asphalt pavement (asphalt layers placed over a granular base; Flintsch et al., 2008).

RESULTS

MSI of VDOT Interstate Pavement Network

The MSI at each testing point and the MSI averaged across each pavement section contained in the PMS were calculated for each interstate and each VDOT maintenance county. Summary data for each interstate are shown in the Appendix, and detailed plots for each county can be obtained by contacting the authors of this report. The average MSI for flexible pavements for the entire network was 1.25, and 38% of the measurements yielded MSI values less than 1. An MSI value of 1 means that the pavement section has enough structural capacity to carry the expected traffic for the next 20 years (see Bryce et al., 2013). The distribution of the MSI values across the interstates follows a lognormal distribution with a mean value of 1.24, standard deviation of 0.50, and mode of 0.99 (Figure 1). The average MSI values for each interstate are given in Table 7, and the average MSI values across all interstates within each county maintenance jurisdiction are given in Table 8. The distribution of the MSI values for each interstate is given in the Appendix.

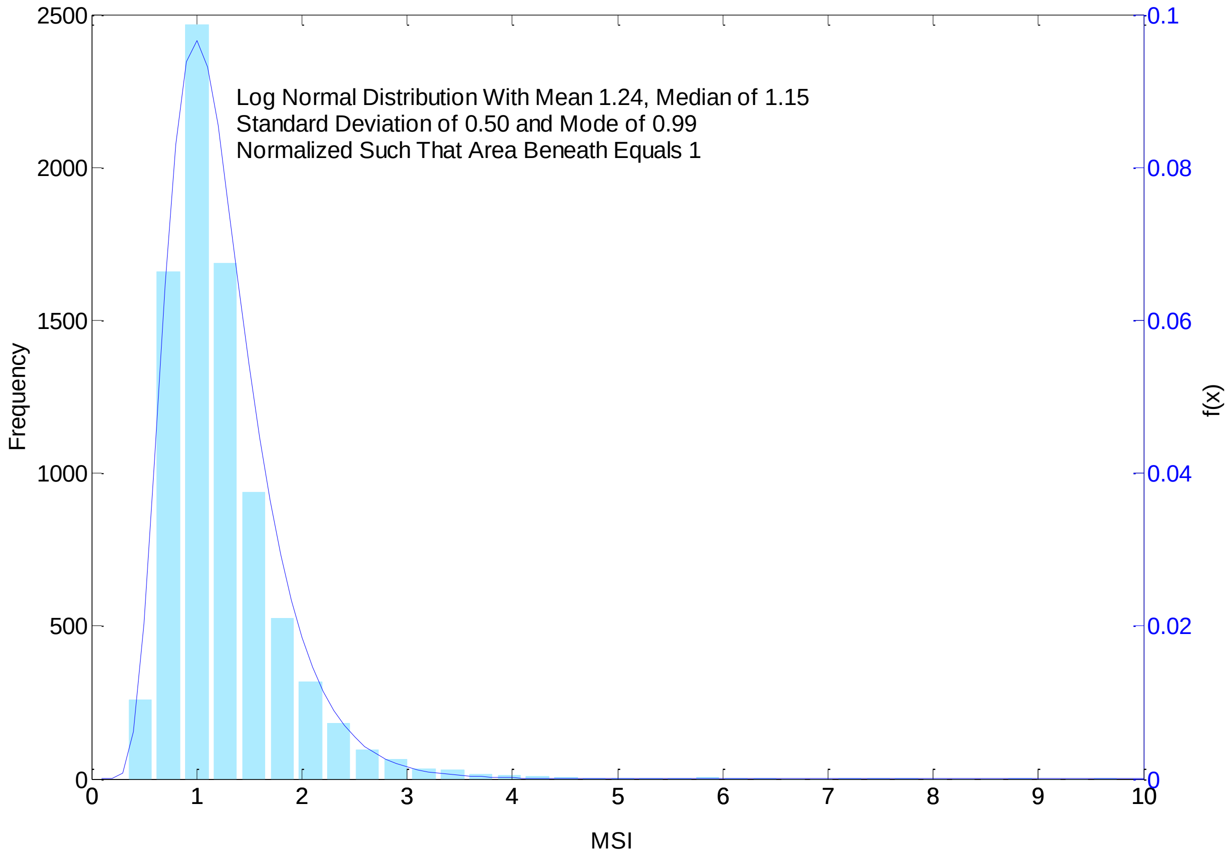


Figure 1. Distribution of All MSI Values for the Interstate Network

Table 7. Average MSI Values for Each Interstate*

Route	Average MSI Value	Number of Data Points	Percent of Data Points with MSI<1
64	1.52	1,826	24.3%
66	1.46	601	30.0%
77	1.24	1,122	34.9%
81	1.17	3,394	38.7%
85	1.06	399	43.6%
95	0.88	736	74.1%
264	1.19	76	35.5%
381	1.86	30	0%
464	0.97	44	59.1%
581	1.26	62	11.3%
664	0.81	28	85.7%

* MSI is calculated only for flexible pavements.

Table 8. MSI Values in Each Maintenance County*

Maintenance Jurisdiction		Average MSI for all Interstates
0	Arlington County	2.22
2	Albemarle County	1.12
3	Alleghany County	1.55
7	Augusta County	1.37
10	Bland County	1.55
11	Botetourt County	1.00
12	Brunswick County	1.05
16	Caroline County	0.67
17	Carroll County	0.97
20	Chesterfield County	0.71
26	Dinwiddie County	0.84
29	Fairfax County	1.85
30	Fauquier County	0.98
34	Frederick County	1.49
37	Goochland County	0.80
42	Hanover County	0.95
43	Henrico County	0.82
47	James City County	1.09
54	Louisa County	1.63
58	Mecklenburg County	1.11
60	Montgomery County	1.15
64	Norfolk	1.10
74	Prince George County	0.99
76	Prince William County	1.24
77	Pulaski County	1.09
80	Roanoke County	1.13
81	Rockbridge County	1.55
82	Rockingham County	1.22
85	Shenandoah County	1.06
86	Smyth County	1.38
91	Sussex County	0.90
93	Warren County	1.94
95	Washington County	1.33
98	Wythe County	1.19
99	York County	1.17

* MSI is calculated only for flexible pavements. Counties that do not include interstate flexible pavement sections are therefore not listed in the table.

MSI with Respect to PMS Sections

VDOT can view the conditions on its pavement network either at 0.1-mile intervals or along sections defined as having the same surface type (also known as PMS sections). The MSI values were averaged across the pavement sections defined in the VDOT PMS using 2012 segment data. MSI data were available for, and averaged across, 579 flexible pavement management sections. The distribution of the MSI values averaged across the PMS sections is shown in Figure 2. Of the sections, 213 (approximately 37%) had an MSI value less than 1. The greatest number of PMS sections that had an average MSI value less than 1 occurred along Interstate 95, where approximately 77% of the bituminous PMS sections with available deflection data had an average MSI value less than 1.

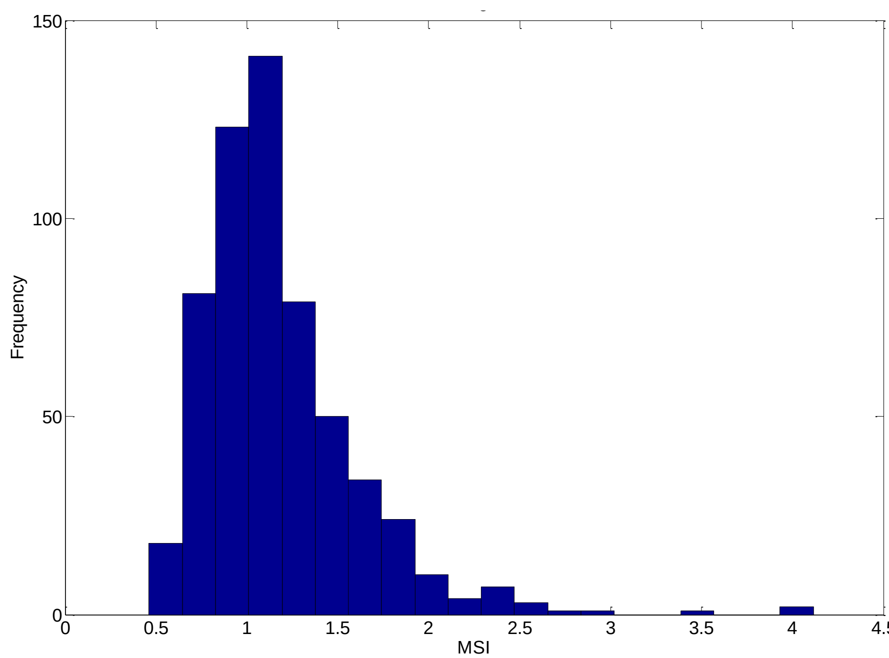


Figure 2. Distribution of MSI Averaged Across PMS Sections

MSI and Pavement Condition

Bryce et al. (2013) investigated the relationship between the functional and structural condition of pavements for given locations on the pavement network. It was determined that no relationship can be formed to infer the structural condition of the pavement based on its functional condition, without accounting for the pavement age and expected deterioration. However, if the condition of a group of pavements with similar ages are compared, it can be expected that the pavements with a lower MSI will have lower functional condition values (i.e., the lower MSI can be linked to a higher rate of deterioration over the same time period).

To investigate the relationship between the CCI and MSI of a group of pavements, the average CCI was plotted along with the average MSI for each VDOT district (Figure 3). It can be seen that districts with the lowest average MSI values tended to also have the lowest CCI, as measured by the 2012 condition data. Given that Bryce et al. (2013) demonstrated that there is a relationship between structural condition and the rate of deterioration of the functional condition, the districts with the lowest average MSI will be expected to have lower condition values. Spearman's rank correlation was conducted to determine the relationship between MSI values and the average CCI for each of the nine VDOT districts. The rank correlation value (Spearman's Rho) was determined as 0.83 with a p-value of 0.0154, indicating a statistically significant relationship with a relatively ideal fit between the average MSI and average CCI for each VDOT district. The data were also split into individual counties, and Spearman's Rho was lower (0.48) with a p-value of 0.013, indicating a statistically significant relationship with moderate correlation (Figure 4).

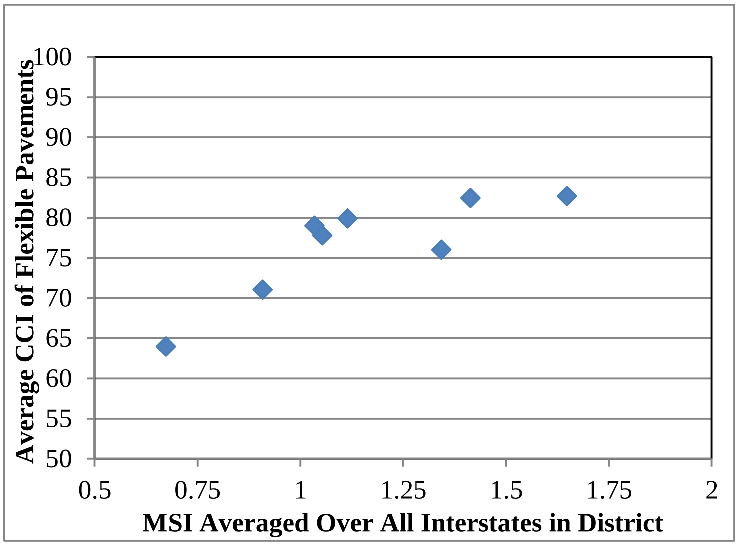


Figure 3. Average CCI and Average MSI With Respect to VDOT District

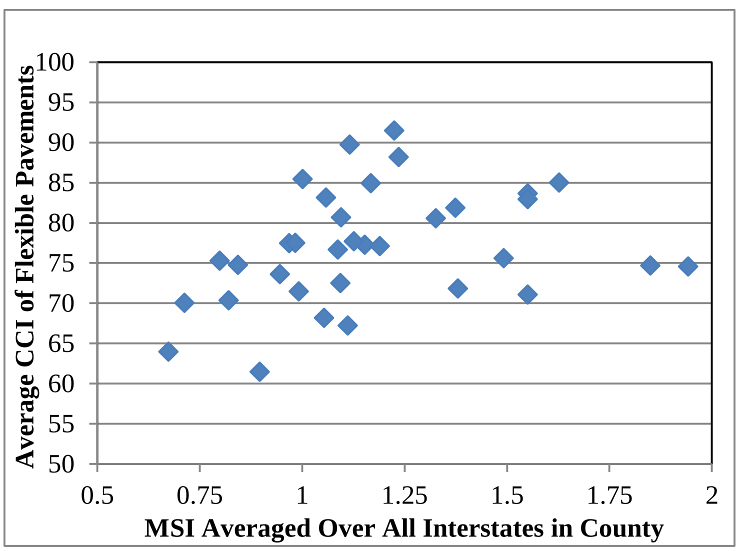


Figure 4. Average CCI and Average MSI With Respect to VDOT County

Developing an Index for Composite Pavements

The MSI developed in Bryce et al. (2013) is not directly applicable to composite pavements, given that the denominator of Equation 4 was developed specifically for flexible pavement design, using the VDOT design assumptions (VDOT, 2003). Therefore, it was decided that new structural models would be explored for composite pavements. Two steps were completed for testing models to determine the structural adequacy of composite pavements. First, the current VDOT practice of using the AREA and k-value to determine the structural adequacy of a pavement was evaluated. Then, the results were compared to a model developed as part of this project. Given that the literature review for composite pavements did not yield a model outside of VDOT that was applicable for the network level, a model was developed using VDOT PMS data. The model follows literature recommending that the difference in two deflection measurements, often referred to as deflection basin parameters, is the most appropriate indicator for the structural evaluation of composite pavements (Kim and Park, 2002; Horak, 2008). Traffic data were also evaluated as potential inputs for the composite models that were developed.

Performance of Composite Pavements

Following the concepts discussed in Carvalho et al. (2012), the analysis of the current VDOT method for evaluating the structural adequacy of composite pavements was conducted by relating the performance of the pavement to the deflection parameters. In other words, the performance of pavement sections was tracked, and the structural condition of the pavement section could be defined as weak if it performed worse than expected and adequate otherwise. Given that different treatments are expected to have different performances, a performance ratio was designed to be used during the analysis. The performance ratio is defined as Equation 14.

$$PR = \frac{Performance_{Actual}}{Performance_{Expected}} \qquad \text{(Eq. 14)}$$

where PR is the performance ratio, $Performance_{Actual}$ is the actual performance of the pavement section given by fitting a curve in the form of Equation 15 to the condition data as a function of the pavement age, and $Performance_{Expected}$ is the expected performance of the pavement given the deterioration models detailed in Stantec and Lochner (2007).

$$100 - e^{a - bc^{\ln(\frac{1}{T})}} \qquad \text{(Eq. 15)}$$

where the parameters a and b were taken from Stantec and Lochner (2007) for the specific type of maintenance performed, T is taken as the age of the pavement following the maintenance, and the variable c was changed to minimize the sum of the square errors between the model defined by Equation 15 and the actual condition data.

The concept of the performance ratio is demonstrated in Figure 5. The shaded area in Figure 5(a) corresponds to the expected deterioration of the pavement that is currently implemented in the VDOT PMS. The shaded area in Figure 5(b) corresponds to the actual fitted deterioration for the specific pavement section. The performance ratio is defined as the ratio of the shaded area in Figure 5(b) to the shaded area in Figure 5(a). In this example, the shaded are in Figure 5(b) is larger than the shaded are in Figure 5(a) and the performance ratio is larger than one indicating that the pavement is performing better than expected. The performance ratio can be used to evaluate whether a structural condition index based on FWD testing can classify pavement into strong pavements (performance ratio > 1) and weak pavement (performance ratio < 1). The best structural index would be the index that results in the lowest classification error where an error occurs if a pavement section with a performance ratio greater than 1 is classified as weak or a section with a performance ratio less than 1 is classified as strong.

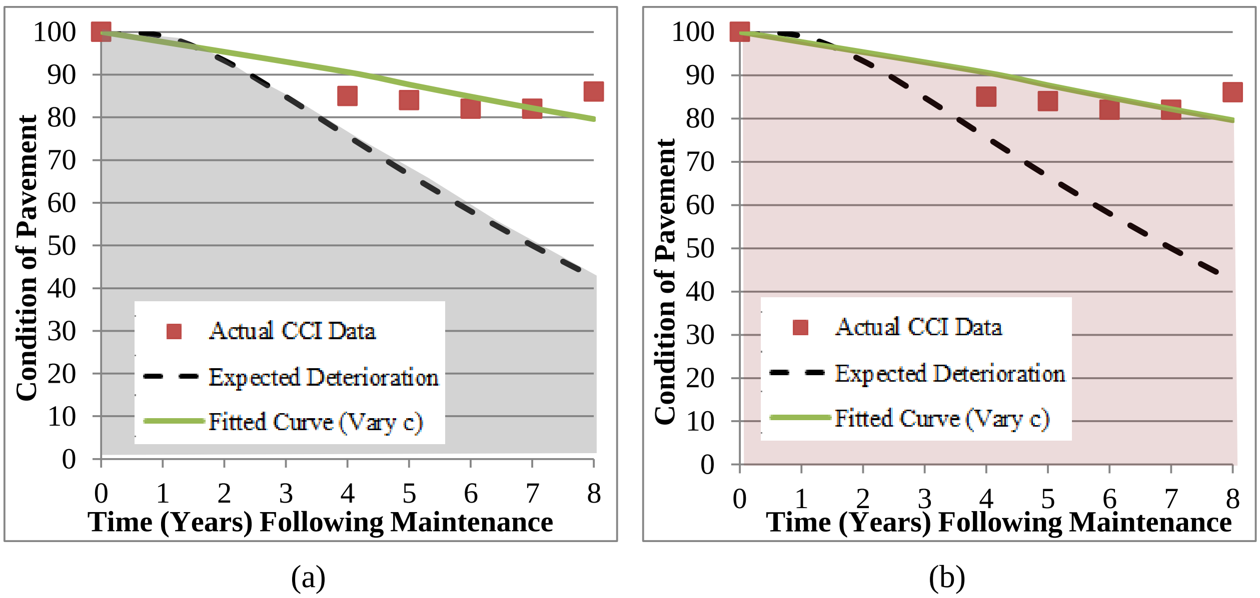


Figure 5. (a) Expected Performance of Maintenance Action and (b) Actual Performance of Maintenance Action

Data Used for Composite Pavements

The data used for the comparison of the composite pavement methods were gathered from the VDOT PMS. Pavement segments were identified on which maintenance was performed within five years prior to the deflection testing. After these pavement segments were identified, their condition was investigated and any pavement segment that had a considerable increase (10 points or higher) in LDR or CCI was removed from the analysis as it was assumed that these segments received treatment that was not recorded in the PMS. Finally, only the pavement segments on which the work could be identified in the VDOT construction history database were used. This is because the different types of maintenance (i.e., preventive, corrective, restorative, or major rehabilitation/reconstruction) are expected to have different treatment lives and, thus, different performance.

After reviewing all composite pavement segments and applying the criteria stated in the previous paragraph, only six pavement segments (totaling approximately 11.5 directional miles) were available for the analysis. The pavement segments used in the model development are given in Table 9. Finally, the condition data were obtained in 0.1-mile averaged increments, and the condition 0.1 mile on either side of each deflection measurement was treated as the condition associated with each deflection measurement. By splitting each pavement segment into 0.2-mile sections, with the center point of each section being the location of a deflection measurement, 81 different sections were identified and used during the analysis.

Table 9. Composite Pavement Segments Used in Analysis

County	Route	Direction	Begin County MP	End County MP	Year Work Performed
Botetourt	81	North	13	14.6	2005/2011
Caroline	95	North	13.68	15.54	2002/2011
Hanover	95	North	0	1.75	2004
Hanover	95	North	1.75	2.39	2005
Louisa	64	West	0	5.18	2005
Spotsylvania	95	South	0	0.53	2002/2011

The performance ratio was calculated for the CCI, LDR, and NDR. The histogram of the CCI performance ratio is shown in Figure 6. It can be seen in Figure 6 that slightly more than 50% of the sections performed better than expected, with the rest performing worse than expected.

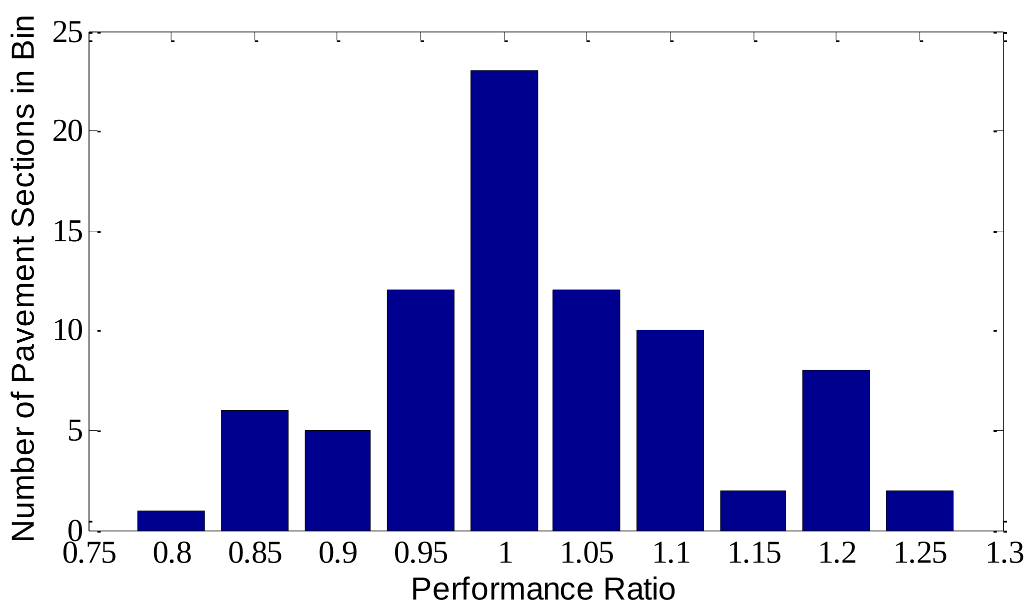


Figure 6. CCI Performance Ratio on Composite Sections Defined From Segments in Table 9

Evaluation of VDOT Current Practice

To evaluate current VDOT practice, pavement sections with an AREA of less than or equal to 32 in and a k-value greater than 175 pci were identified as strong; the remaining sections were defined as weak. The performance ratios for each of the sections were then calculated. The performance ratios for the strong pavements were then compared to those of the weak pavement sections. It should be expected that pavements categorized as strong have a higher performance ratio than those classified as weak. The results are shown in Figure 7. A nonparametric hypothesis test, the Mann–Whitney–Wilcoxon test, was performed to determine whether the two data sets in Figure 7 are statistically different. The Mann–Whitney–Wilcoxon test indicated that the results are significantly different, with a resulting p-value of 0.024 and a z-value of -2.265. Therefore, it can be said that the pavements identified as weak using the existing VDOT criteria have statistically lower performance ratios than the pavements identified as strong.

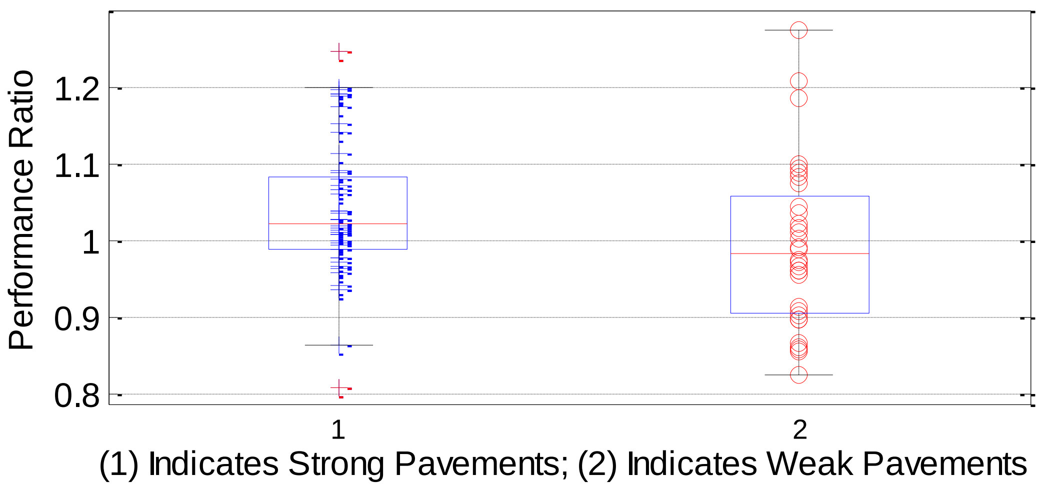


Figure 7. Comparison of "Weak" and "Strong" Pavements Using the Existing VDOT Criteria

Model Development for Composite Pavements

A second model was developed in this project based on the concept of using deflection basin parameters to identify the structural adequacy of pavements, as discussed in Kim and Park (2002) and Horak (2008). In terms of 20-year ESALs (Equation 7), traffic was also included as a parameter in the model development, although it was determined to be an insignificant variable in the final model. Several forms were explored for the model, and the function shown in Equation 16 was determined to be the best fit for the data.

$$PR = a * \left(\frac{D_0 - D_8}{D_0 - D_{18}} \right)^b + c \qquad \text{(Eq. 16)}$$

where *a*, *b*, and *c* are model parameters that were used to fit the function to the data; D_0 is the temperature-corrected center deflection from the FWD; D_8 is the measured deflection 8 in (203 mm) away from the center deflection; and D_{18} is the measured deflection 18 in (457 mm) away from the center deflection.

The variables *a*, *b*, and *c* were found using nonlinear (orthogonal) regression. The model was compared against the performance ratio calculated using the LDR (see Figure 5 for how the performance ratio is calculated using the LDR), given that it is expected that the LDR will relate most closely to structural inadequacies. The final values for coefficients *a*, *b*, and *c* were 0.4118, 1.793, and 0.8402, respectively. The model and a plot of the differences in the model and data calculated using condition data are shown in Figure 8.

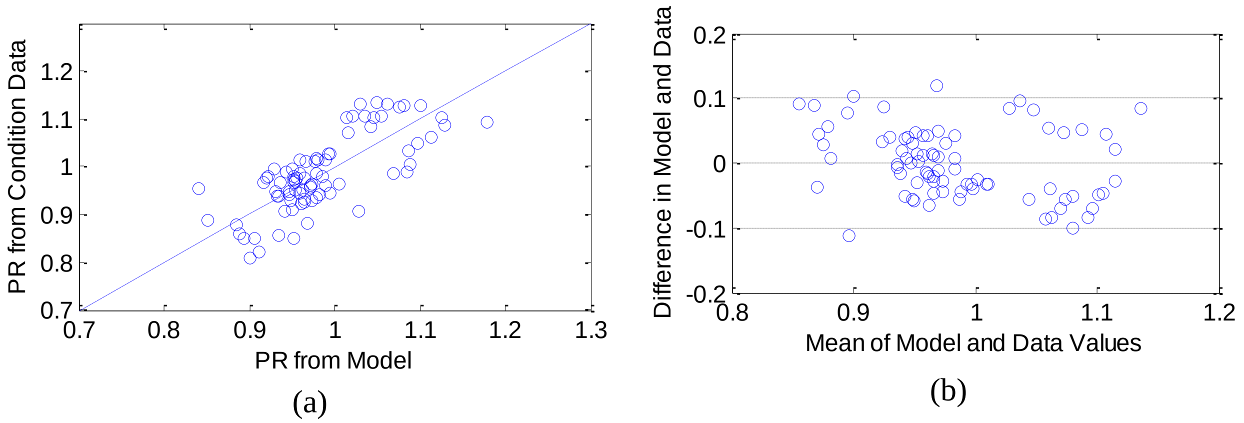


Figure 8. (a) Model Results Compared to Results Obtained From LDR Condition Data and (b) Differences in Model Results and Results From LDR Data as a Function of the Mean

Finally, to compare the results obtained from the model to the actual performance ratio, pavements that yielded a performance ratio less than 1 using Equation 16 were identified as weak. The pavements classified as weak or strong were then compared to the actual performance ratio using VDOT condition data. The results are shown in Figure 9.

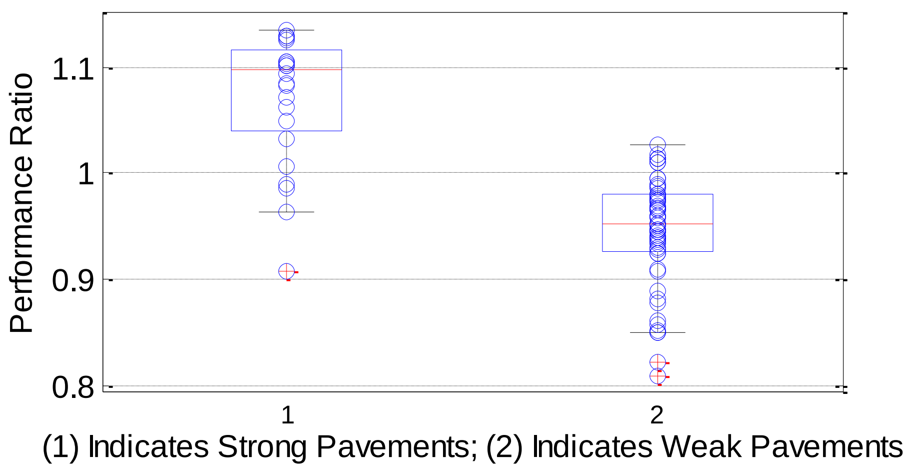


Figure 9. Comparison of "Weak" and "Strong" Pavements Using Equation 16

Analysis of Composite Pavement Models

Given the analysis of the methods for composite pavements, it can be concluded that implementing a model similar to Equation 16 into the PMS will increase the accuracy of identifying structurally weak pavements from their counterparts. It can be seen from Figure 9 that pavements classified as weak (i.e., PR of less than 1 when calculated using Equation 16) typically had performance ratios less than 1, as based on LDR calculations. Furthermore, the relationship in Equation 16 tends to predict actual pavement performance better than the existing VDOT criteria for identifying weak pavements (Figure 7). However, the number of pavement sections available for the analysis was limited, and more data may be required for a calibration of Equation 16.

Model Development for Rigid Pavements

Data Used for Rigid Pavements

The data used for the comparison of the rigid pavement methods were gathered from the VDOT PMS and included the years from 2007 to 2013. Selection of the rigid pavements used in the analysis was performed similarly to the selection of the composite pavements. That is, pavement segments on which maintenance was performed within five years prior to the deflection testing were identified based on the construction history data. The condition of each pavement segment was investigated, and any pavement segment that had a considerable increase (10 points or higher) in the CCI was removed from the analysis. One difference between the composite pavements and rigid pavements was that some segments of the latter were found to have CCI values significantly lower than 100 during the year following maintenance. Thus, these segments were removed from the analysis, as it was assumed that a type of maintenance (probably patching) was performed and indicated in the construction history database.

After reviewing the remaining pavement segments, it was found that only two met the criteria. Therefore, two pavement segments on which work was performed during the year following deflection testing were included in this analysis, and it was assumed that the work did not significantly impact the structural capacity of the pavements. Finally, the specific category of work involved in the maintenance (i.e., preventive, corrective, restorative, or major rehabilitation/reconstruction) was not indicated in the construction history of rigid pavements. Therefore, only the performance was evaluated for rigid pavements as opposed to the performance ratio, which was the case in composite pavements.

After removing all segments based on the previous criteria, only four pavement segments (totaling approximately 37 directional miles) were available for the analysis. The pavement segments that were used in the model development are given in Table 10. The condition data were obtained in 0.1-mile averaged increments, and the condition 0.1 mile on either side of each deflection measurement was treated as the condition associated with each deflection measurement. By splitting each pavement segment into 0.2-mile sections, with the center point of each section as the location of a deflection measurement, 119 different sections (19 continuously reinforced concrete pavements (CRC) and 100 jointed reinforced concrete pavements (JCP)) were identified and used during the analysis.

Table 10. Rigid Pavement Segments Used in Analysis

County	Route	Direction	Begin County MP	End County MP	Year Work Performed	JCP or CRC
York	64	East	5.17	20.7	2008	JCP
York	64	East	28.81	31.25	2006	CRC
York	64	West	4.18	20.7	2008	JCP
York	64	West	28.96	31.5	2006	CRC

Evaluation of Current VDOT Practice

To evaluate current VDOT practice, pavement sections with an AREA less than or equal to 32 in and a center deflection less than 5 mils were identified as strong; the remaining pavement sections were defined as weak. The performance for each of the sections was then calculated for a five-year period. The performance values for the strong pavements were then compared to those of the weak pavement sections. The performance was calculated by integrating Equation 15 between zero and five years. The values for *a* and *b* were 4.244 and 4.22, respectively, for JCP and -9.88 and -9.887, respectively, for CRC. The value of *c* was found for each section by minimizing the sum of the square errors between Equation 15 and the actual performance data from the PMS. Finally, JCP and CRC were initially separated for the analysis. However, given the relatively small sample size and the fact that the indicators and thresholds used by VDOT are the same for each, it was decided to analyze the two pavement types together. It should be noted that these two pavement types are expected to behave differently and should be analyzed separately in future analysis, given larger sample sizes.

It is expected that pavements categorized as strong should perform better than those classified as weak. However, using the VDOT criteria, only two of the pavement sections (one JCP and one CRC) would be considered strong. However, 17 of the 119 pavement sections had a CCI of at least 90 during the five years following maintenance, and 43 sections had a CCI of at least 85. The histogram of CCI values during the five years following maintenance is shown in Figure 10. Based on the results in Figure 10, it can be seen that significantly more than two pavement sections perform adequately.

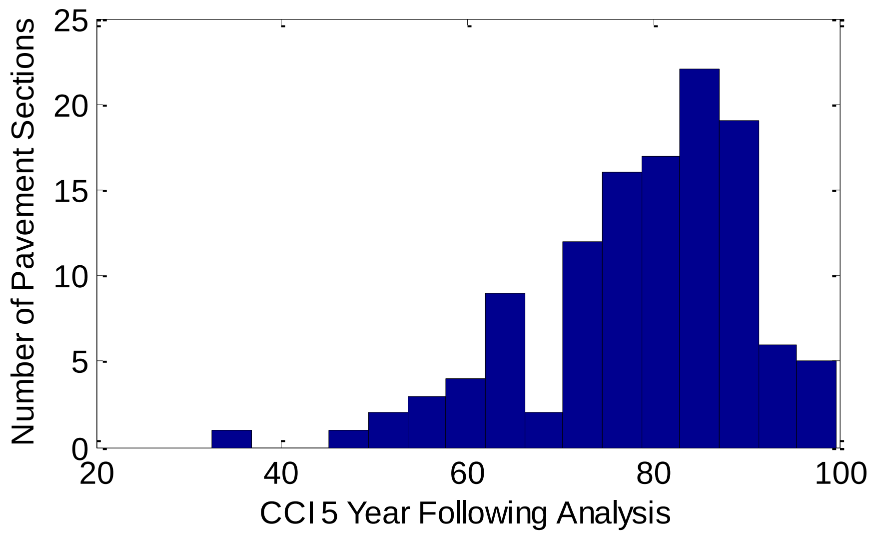


Figure 10. Histogram of CCI Five Years Following Maintenance for Rigid Sections

To test whether the center deflection or AREA value can be used as an indicator of structural adequacy for rigid pavements, the performance of the pavement segments was plotted against each criterion. It can be seen in Figure 11 that practically no relationship exists between the performance of the pavements and the AREA. Figure 12a seems to indicate a decreasing level of performance with increasing center deflection, which becomes more obvious when the six pavement sections with center deflection values exceeding 15 mils are removed from the analysis (Figure 12b).

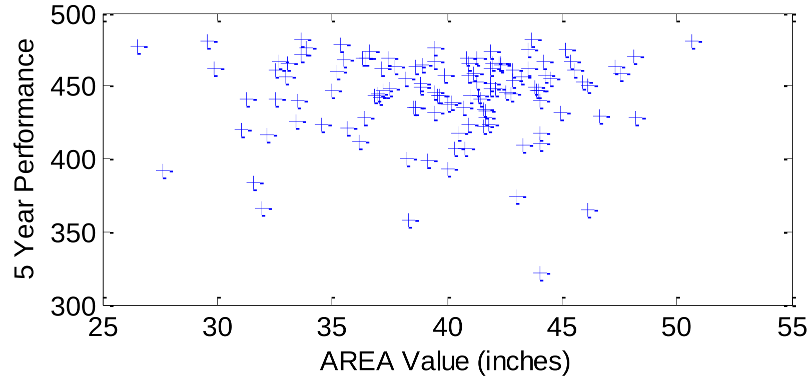


Figure 11. Pavement Performance as a Function of AREA

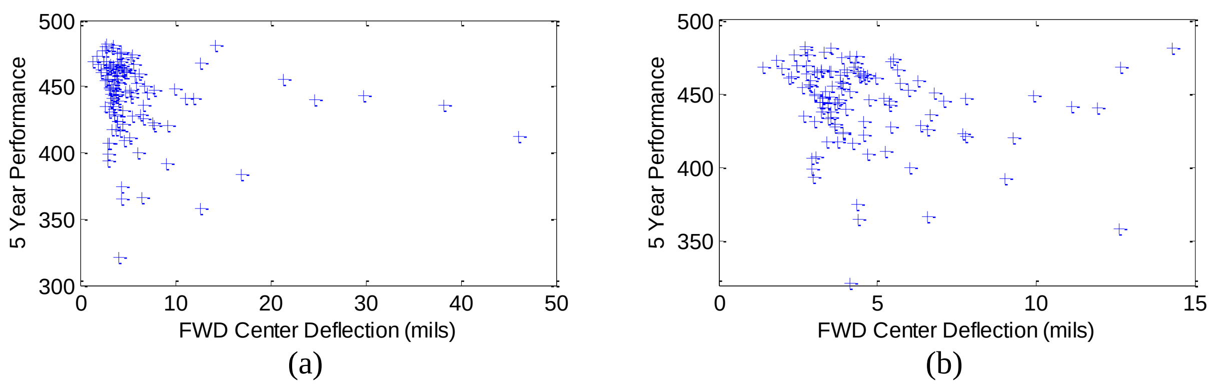


Figure 12. (a) Pavement Performance as a Function of FWD Center Deflection and (b) Pavement Performance as a Function of FWD Center Deflection Without Outliers

Figure 12 suggests a decreasing trend of performance with increasing center deflection; therefore, the center deflection of the FWD may be an indicator of the performance of rigid pavements. This is the basis of the SSI developed by Flora (2009). However, the figure also shows that the association is at best very weak. To further explore this concept, the value of the center deflection used as a threshold for defining weak versus strong rigid pavements was varied between 3 mils and 10 mils, and the difference in the mean performance for weak versus strong pavements was calculated. The results are shown in Figure 13. It can be seen that the greatest observed difference in performance between weak and strong rigid pavements occurs at 6 mils.

To test whether the 6-mil threshold for the center deflection may be useful as an indicator for the structural adequacy of rigid pavements, the 119 pavement sections were classified as strong or weak based on their center deflection values. The results are shown in Figure 14, and it can be seen that a statistically significant difference in performance values exists based on the pavement classification (weak or strong). This was verified by using the Mann–Whitney–

Wilcoxon test, which indicated a statistically significant difference and a resulting p-value of practically 0.

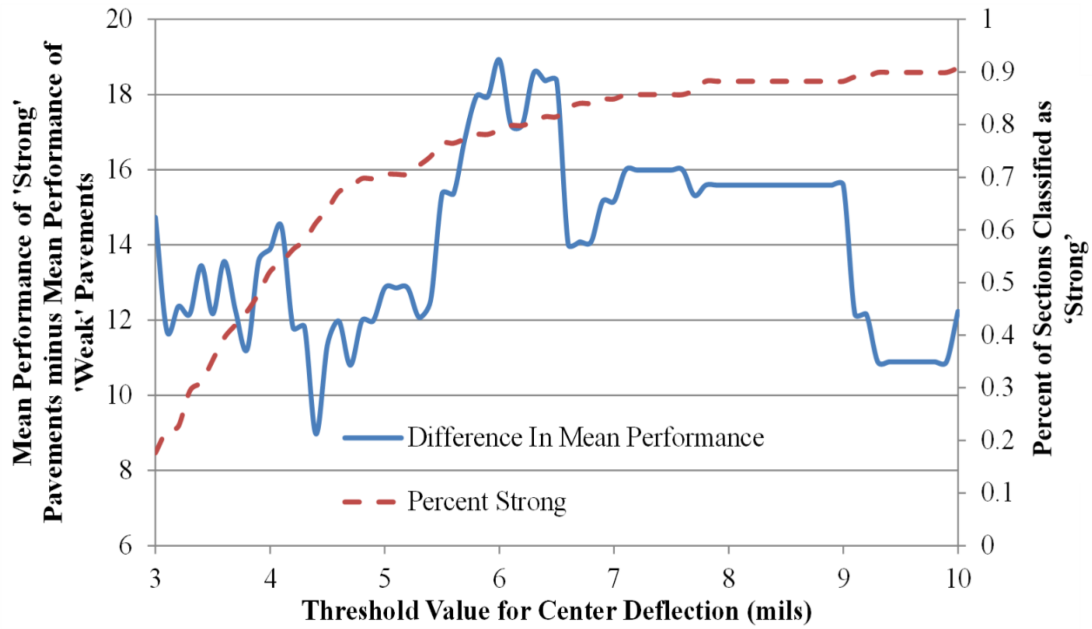


Figure 13. Difference in Performance Between "Weak" and "Strong" Rigid Pavements as a Function of the Center Deflection Value Chosen as the Threshold

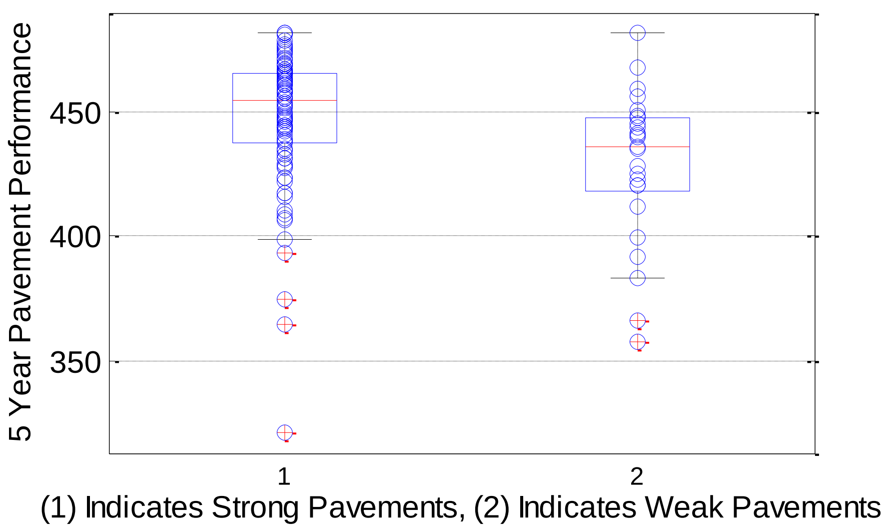
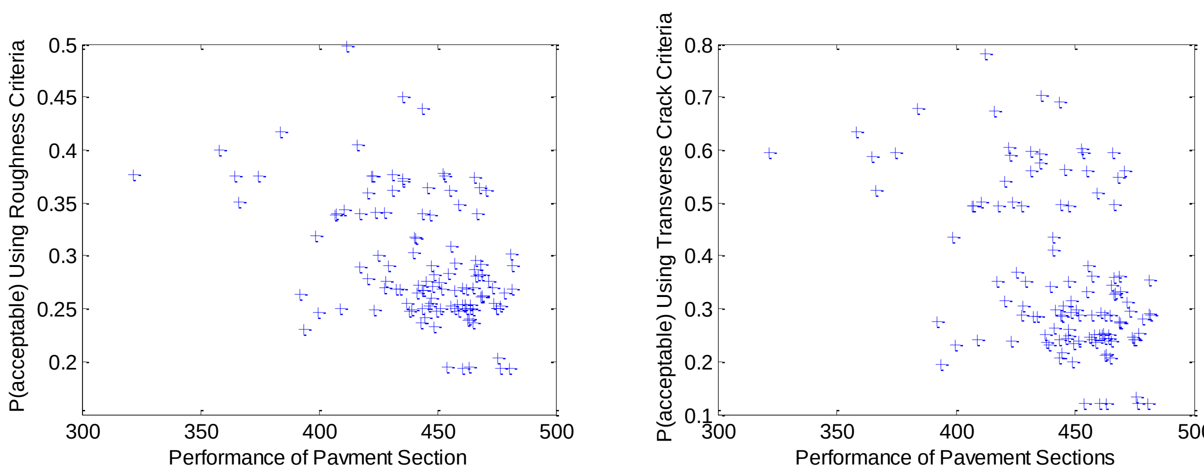


Figure 14. Comparison of "Weak" and "Strong" Rigid Pavements Using a Center Deflection Threshold of 6 mils

Analysis of the Federal Highway Administration (FHWA) Simplified Model

The FHWA simplified models are based on the concept that structurally inadequate pavements have a lower probability of performing adequately during their design life (Carvalho et al., 2012). However, the techniques presented by Carvalho et al. (2012) are based on exceeding a given level of roughness or transverse cracking. To test the methodology using the VDOT data, the pavement performance was set as the criterion to determine the adequacy of the pavement. The first analysis was to determine whether a relationship exists between the models presented by Carvalho et al. (2012) and the performance data for the concrete sections. The coefficients in Table 5 and Table 6 were used, and the AADTT was taken from the VDOT PMS. It is important to note that the AADTT reported by VDOT includes more truck classes than the Class 9 trucks used by Carvalho et al. (2012), but this is not expected to impact whether or not a relationship can be identified given that both estimations are a measure of the number of trucks on the road. The results of using VDOT data in the models developed by Carvalho et al. (2012) are shown in Figure 15. It can be seen that practically no relationship exists between the models and the performance of the pavement sections. This result could be expected given that the models in Carvalho et al. (2012) were developed to describe the performance of specific distresses and not the performance in terms of aggregated condition scores.



(a) (b)

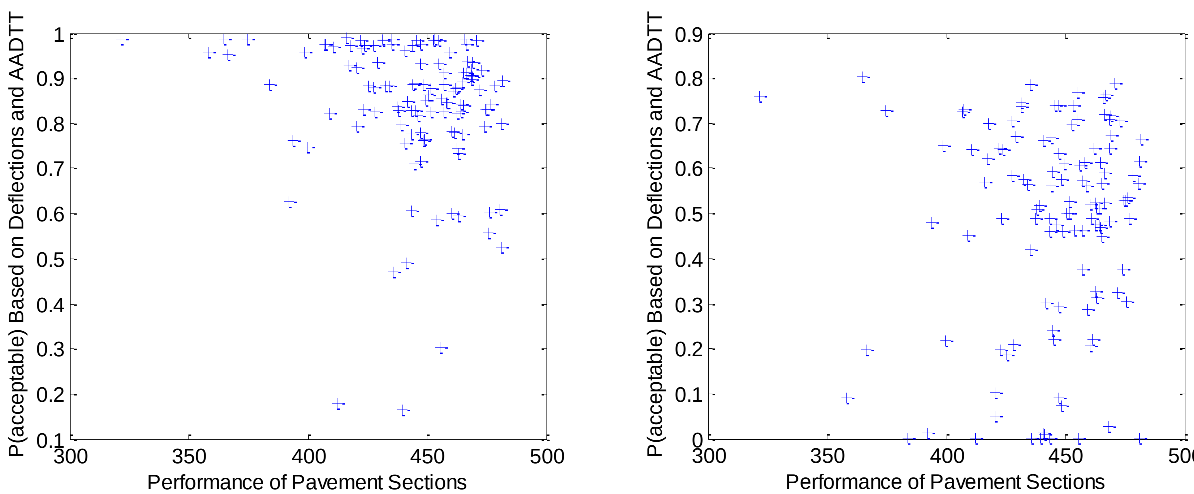
Figure 15. Evaluation of Pavement Adequacy Following Carvalho et al. (2012) Using (a) Roughness Criteria and b) Transverse Cracking Criteria

To determine whether the methodology in Carvalho et al. (2012) could be used by VDOT, Equation 11 was evaluated to determine whether the coefficients and parameters could be calibrated to the data for rigid pavements in this project. Equation 10 was then used to calculate the probability of the pavement performing adequately (using the value for *b* calculated from Equation 11). Three levels of performance were chosen as thresholds to determine whether a pavement is acceptable or not: (1) 430 CCI-yr, or approximately the 25th percentile of performance; (2) 440 CCI-yr, or approximately the 35th percentile of performance; and (3) 450 CCI-yr, or approximately the 50th percentile of performance.

A similar process to Carvalho et al. (2012) was followed in this project using VDOT data, with the exception that Equation 11 was modified to determine a new set of coefficients.

The variables used in Equation 11 for this analysis were: m_1 as the center deflection minus the deflection at 18 in (d_0-d_{18}), m_2 as the deflection at 18 in minus the deflection at 24 in (d_{18}-d_{24}), m_3 as the deflection at 24 in minus the deflection at 36 in (d_{24}-d_{36}), and m_4 as the average AADTT. To determine the coefficients (a_i) and the constant (c), a minimization algorithm was programmed into MATLAB. The algorithm was configured this way to meet the objective that pavements above the threshold have a *P(acceptable)* (from Equation 10) equal to 1, and pavements below the threshold have a *P(acceptable)* (from Equation 10) equal to 0.

The steps taken in the algorithm were twofold: (1) determine which pavement sections perform better than the threshold and set their corresponding values to 1,000 (with the pavements below the threshold set to 0), and (2) determine the values for a_i and c that minimize the sum of the square of the differences between *P(acceptable)* (Equations 10 and 11) and the value for the pavement section set in the first step. The genetic algorithm tool built into the MATLAB global optimization toolbox was used to perform the minimization. The results for calibrating to a performance threshold of 440 and 450 can be seen in Figure 16(a) and Figure 16(b), respectively. It can be seen that there is no relationship between the calibrated logistic models and the performance of the pavement sections.



(a) (b)

Figure 16. Models Based on Calibrating the Generalized Logistic Model Presented in Carvalho et al. (2012) for Performance Thresholds of (a) 440 and (b) 450

Conclusions From the Analysis of Rigid Pavement Models

The results from the previous analysis suggest that center deflection is a better indicator of pavement structural adequacy in terms of pavement performance than the AREA parameter. However, this analysis was performed using a very limited data set (i.e., four pavement segments within one VDOT district). Therefore, any interpretation of the previous models should be made with the caveat that more data will be required to determine if the results remain consistent as the sample size increases. It is evident from Figure 11 that the pavement AREA is not expected to be a good indicator of structural adequacy when used independently. Furthermore, the fact that only two pavement sections were identified as strong using the AREA and the FWD center deflection method indicates that AREA and center deflection taken together are not an accurate indicator of structural adequacy, particularly when considerably more pavement sections than identified had five-year CCI values of 90 or greater.

Implementing Structural Capacity Indices Into the Network-level Pavement Evaluation

Given the analysis performed in this report and the analysis in Bryce et al. (2013), the following sections detail methods that VDOT should follow to collect data for calculating the structural indices, as well as update the MSI based on work performed.

Data Collection Procedure

The data collection includes information about the spatial test spacing and the relevant parameters to collect during testing on the VDOT interstate network using the FWD. Information about the temporal frequency of testing, for all pavement types, has produced inconclusive results however, much of the literature (e.g., Noureldin et al., 2005; Carvalho et al., 2012) suggests a five-year cycle for collecting FWD measurements on bituminous pavements. In Carvalho et al. (2012), it is determined that the cycle for rigid pavement testing can be expanded to 10 years. Therefore, it is suggested that VDOT follow the five-year cycle for flexible and composite pavements.

Data That Should Be Collected

When FWD data are collected for flexible pavements, it is recommended that the following data also be noted: (1) depth of the pavement structure; (2) current air temperature; (3) average air temperature of the previous day; and (4) pavement temperature at time of testing. In addition, the analysis will require traffic parameters in terms of the design AADT, truck factor (or the assumed truck factor), the percent of trucks, and the number of lanes. For the case of composite pavements, it is expected that collection of only the deflection values is adequate. Given that the model for structural adequacy of rigid pavements was inconclusive, no guidance can be given about which data should be collected for calculation of a rigid pavement structural index.

Spacing Between FWD Tests

Extensive research has been conducted to determine the optimal testing interval for measuring network-level deflections using the FWD (Noureldin et al., 2005; Alam et al., 2007; Carvalho et al., 2012). Many of the results indicate that three to five test points per mile can be collected without too much loss of information. The current VDOT practice to collect five FWD measurements per directional mile on its interstate pavement network (Diefenderfer, 2008), which is within the recommended practice. The collection frequency for flexible and composite pavements is based on the concept that a statistical representation of the pavement can be gathered using random testing (i.e., the probability of identifying a weak section is not changed based on the location of testing). The research presented in this report and in Bryce et al. (2013), shows that for flexible pavements and composite pavements, five FWD tests per mile allows one to differentiate between strong and weak pavements.

The weak association between FWD deflections and performance on rigid pavements found in this report suggests that the five points per mile test procedure is not effective for rigid pavements. For jointed rigid pavements, failure tends to occur at joints or in the form of edge cracking, more so than randomly throughout the slab. This localized mode of failure cannot be effectively detected through uniform 0.2 miles intervals of FWD testing. On the other hand, testing of individual joints (or even say 20% of the joints) is currently not a feasible approach for network-level FWD testing.

Data Preparation

Data preparation for flexible pavements should include correction for the pavement temperature on the center deflections. This may be accomplished using the VDOT available program, MODTAG. Pavement temperature corrections may also be made via the MSI code provided as a deliverable of this project. The code delivered in this project corrects temperature using the BELLS2 equation as described by FHWA (2011). Additional preparation for flexible and composite pavements should include placing the data in Microsoft Excel spreadsheets to make it available for calculations. No recommendations can be made for the case of rigid pavements based on the results of this report.

Updating of the Structural Condition Index to Reflect Maintenance Treatments

Flexible Pavements

The MSI for flexible pavements is based on the ratio of the required structural number (SN_{Req}) to the effective structural number (SN_{eff}). Therefore, any significant maintenance actions are expected to affect the SN_{eff}. To update the MSI value, Equation 20 may be used when the depth of the maintenance does not exceed the depth of the asphalt layers. If the maintenance exceeds the depth of the asphalt layers, the numerator in Equation 17 should be replaced by the SN_{eff} from the pavement design documents.

$$\mathrm{MSI}_{\mathrm{Updated}} = \mathrm{MSI}_0 + \frac{0.44\left(d_{placed} - c * d_{milled}\right)}{0.05716 * (\log(\mathrm{ESAL}) - 2.32 * \log(\mathrm{M_R}) + 9.07605)^{2.36777}} \qquad \text{(Eq. 17)}$$

where $MSI_{Updated}$ is the updated value for the MSI, MSI_0 is the original value of the MSI, d_{placed} is the depth of the asphalt layer (inches) placed, d_{milled} is the depth of the milled asphalt layer (inches), and c is a factor based on the condition of the pavement (Huang, 2004). The ESAL and M_R values are the same as defined earlier in this report. The c factor represents the percent of contributing structure that remains in the removed layer of asphalt. Values of 0.5 to 0.7 should be used for c with asphalt concrete pavement that exhibits appreciable cracking.

Composite Pavements

The performance ratio (Equation 16) calculated for composite pavements is a function of the deflection in the top layers and the combined deflection in the top layer and underlying layers as illustrated in Figure 17. Typically, it is expected that the rigid layers contribute less to the overall deflection than the asphalt layers. Thus, only significant changes in the structure of the asphalt layer, or more extensive actions that extend to the rigid layers, are expected to contribute to increasing the performance ratio. Furthermore, updating the performance ratio to indicate maintenance actions would require linear elastic analysis techniques. Therefore, it is recommended that FWD testing be conducted on the pavement following significant maintenance actions to update the performance ratio.

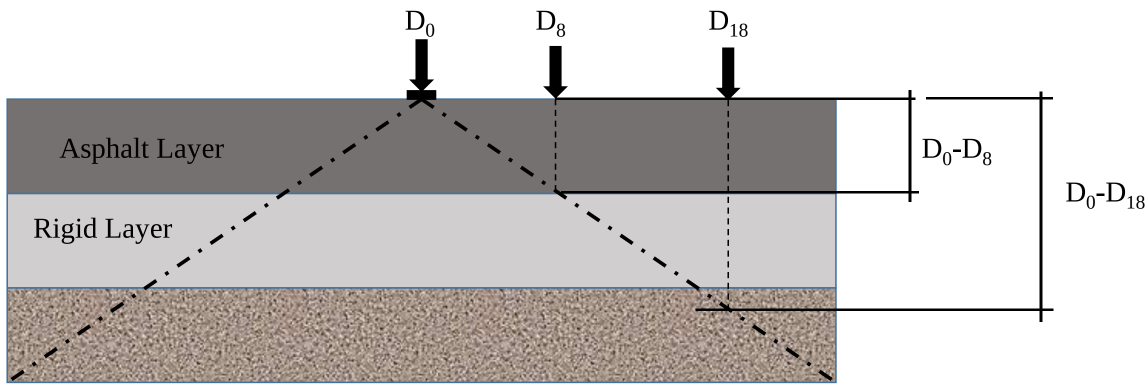


Figure 17. Representation of Deflections Using Equation 19

SUMMARY OF FINDINGS

- Whereas many states have begun implementing network-level structural capacity indices into their PMS's for flexible pavements, the use of indices to describe the structural adequacy of rigid and composite pavements is far less prevalent.

- No adequate network-level structural capacity model for composite pavements was found in the literature outside of the VDOT methodology.

- It was found that districts with a lower average MSI also have a lower average condition index (as measured by the CCI) than districts with higher average MSI values. This may be because each district has a similar distribution of pavement ages.

- It was found that the current method for determining the structural adequacy of composite pavements provides statistically significant results when discriminating between weak and strong sections. However, the model based on differences in deflection values can discriminate better than using AREA and center deflection. Note that the results are based on limited data.

- For the sections studied, the AREA value does not seem to provide meaningful information about the expected performance for rigid pavements when taken independently or when considered along with the FWD center deflection. It is important to note that the data available were very limited and this finding is based on the analysis of only four pavement segments in the same district.

- It was found that, on average, the MSI values of the interstate pavement network are greater than 1, and the mode of the distribution of MSI values is approximately 1.

- Sources in the literature suggested that successive rounds of network-level deflection testing should be conducted in five-year cycles for flexible and composite pavements and 10-year cycles for rigid pavements.

CONCLUSIONS

- *Deflection testing using the current VDOT protocol provides useful information for flexible and composite pavements. However, very few conclusions could be drawn from the results for rigid pavements.* Some sources in the literature suggest that deflection testing at the network level may not be useful for rigid pavements especially if testing is conducted on a set interval rather than at specific locations along the slabs. For example, a major concern for jointed concrete pavements is the load transfer efficiency at the joints. Information on the load transfer efficiency cannot be obtained at long testing intervals (0.2 miles).

- *This project confirmed that the MSI can be used to predict the structural adequacy of flexible pavements.* This conclusion was reached based on the evaluation of the entire network and the trend showing that districts with a lower MSI had pavements in poorer condition than districts with a higher MSI.

- *This research showed that a structural index based on deflection ratios can be used to identify composite pavements that are anticipated to not perform as well as expected.* Although the sample size for composite pavements was limited, the locations (in terms of counties and roads) of the sample were diverse enough that it is expected the results could be relatively consistent throughout the remaining composite network.

RECOMMENDATIONS

1. *VDOT's Maintenance Division should replace the current structural criteria for flexible pavements in the current enhanced decision tree with the MSI as outlined in this report and in Bryce et al. (2013).* The composite structural criteria should also be revised to include the performance ratio technique discussed in this report, as opposed to the current methodology. VDOT's Maintenance Division should further evaluate the new criteria on the network to determine its effect on suggested maintenance treatments, the cost of maintenance treatments, and sensitivity of the results to decide on the approach.

2. *From the literature review, VDOT's Maintenance* Division *and VDOT's Materials Division should consider conducting successive rounds of network-level deflection testing in 5-year cycles for flexible and composite pavements and 10-year cycles for rigid pavements.*

3. *VDOT's Materials Division should conduct future network-level deflection testing on jointed rigid pavements at approximately 0.2-mi intervals but at the transverse joint, as opposed to maintaining the 0.2-mi testing interval in the current protocol.* No changes to the 0.2-mi test interval for flexible and composite pavements are necessary.

4. *VDOT's Maintenance Division should use a D_0 threshold of 6 mils to define strong/weak pavement rather than the currently used approach.*

BENEFITS AND IMPLEMENTATION

By including structural information into a network-level decision program, the errors between the need-based (network-level) pavement budget and the actual project-level needs can be minimized by using a more accurate assessment. The performance of those pavements that will require rehabilitation can also be more accurately predicted using models that incorporate the structure condition of the pavement by identifying inadequate pavement sections before their functional condition deteriorates.

VDOT's Maintenance Division will work in cooperation with VDOT's Information Technology Division for the evaluation and implementation of the suggested method. The final product will be the developed MSI index for all interstate pavements incorporated into VDOT's Pavement Management System.

VDOT's Maintenance Division, along with VDOT's Materials Division, will revise current network-level testing protocols using the FWD on jointed rigid pavements to test at specific locations along the slab as outlined in this report. VDOT is also a participant in Transportation Pooled Fund TPF-5(282), *Demonstration of Network Level Pavement Structural Evaluation with Traffic Speed Deflectometer*. If future deflection testing at the network is conducted with this device, there may not be an opportunity to conduct testing at specific locations along a slab for network analysis.

Changes to the D_0 threshold used to define strong/weak pavement sections will also be incorporated into VDOT's Pavement Management System by VDOT's Maintenance Division. The suggested value from this study is 6 mils.

ACKNOWLEDGMENTS

The authors acknowledge the invaluable contributions of providing timely data and detailed feedback by Tanveer Chowdhury, Raja Shekharan, and Akyiaa Hosten, VDOT Maintenance Division; and Affan Habib, VDOT Materials Division. The members of the technical review panel included Steven Mullins, Tom Tate, Affan Habib, Raja Shekharan, and Tanveer Chowdhury, VDOT; and Stacey Diefenderfer, VTRC.

REFERENCES

Alam, J., K.A. Galal, and B. K. Diefenderfer. Network-Level Falling Weight Deflectometer Testing: Statistical Determination of Minimum Testing Intervals and Number of Drop Levels on Virginia's Interstate System. In *Transportation Research Record: Journal of* *the Transportation Research Board*, No. 1990. Transportation Research Board of the National Academies, Washington DC, 2007, pp. 111-118.

Ali, H.A., and S.D. Tayabji. Evaluation of Mechanistic-Empirical Performance Prediction Models for Flexible Pavements. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1629. Transportation Research Board of the National Academies, Washington, DC, 1998, pp. 169-180.

American Association of State Highway and Transportation Officials. *AASHTO Guide for Design of Pavement Structures.* Washington, DC, 1993.

Bryce, J. M., G.W. Flintsch, S.W. Katicha, and B.K. Diefenderfer. *Developing a Network-Level Structural Capacity Index for Structural Evaluation of Pavements*. VCTIR 13-R9. Virginia Center for Transportation Innovation and Research, Charlottesville, 2013.

Carroll, R.J., D. Ruppert, L.A. Stefanski, and C.M. Crainiceanu. *Measurement Error in Nonlinear Models*. Taylor & Francis, Boca Raton, 2006.

Carvalho, R., R. Stubstad, R. Briggs, O. Selezneva, E. Mustafa, and A. Ramachandran. *Simplified Techniques for Evaluation and Interpretation of Pavement Deflections for Network-Level Analysis.* FHWA-HRT-12-023. Federal Highway Administration, McLean, VA, 2012.

Chakroborty, P., P.K. Agarwal, and A. Das. *Simple Model to Predict the Structural Condition of Asphalt Concrete Pavements at the Network Level*. 86th Annual Meeting of the Transportation Research Board, Washington DC, 2007.

Chowdhury, T. *VDOT's Pavement Management Program*. Presentation. Virginia Department of Transportation, Richmond, March 10, 2010.

Diefenderfer, B.K. *Network-Level Pavement Evaluation of Virginia's Interstate System Using the Falling Weight Deflectometer*. VTRC 08-R18. Virginia Transportation Research Council, Charlottesville, 2008.

Federal Highway Administration. *LTPP Guide to Asphalt Temperature Prediction and Correction.* FHWA-RD-98-085. Federal Highway Administration, Washington, DC, August 5, 2011. http://www.fhwa.dot.gov/publications/research/infrastructure/ pavements/ltpp/fwdcd/ delta.cfm#delta36. Accessed March 15, 2013.

Flintsch, G.W., B.K. Diefenderfer, and O. Nunez. *Composite Pavement Systems: Synthesis of Design and Construction Practices.* VTRC 09-CR2. Virginia Transportation Research Council, Charlottesville, 2008.

Flora, W.F. *Development of a Structural Index for Pavement Management: An Exploratory Analysis*. Master's Thesis. Purdue University, West Lafayette, Indiana, 2009.

Galal, K., B.K. Diefenderfer, and J. Alam. *Determination by the Falling Weight Deflectometer of the In-Situ Subgrade Resilient Modulus and Effective Structural Number for I-77 in Virginia*. VTRC 07-R1. Virginia Transportation Research Council, Charlottesville, 2007.

Hadidi, R., and N. Gucunski. Comparative Study of Static and Dynamic Falling Weight Deflectometer Back-Calculations Using Probabilistic Approach. *Journal of Transportation Engineering*, Vol. 13, No. 3, 2010, pp. 196-204.

Hein, D.K., C. Olidis, E. Magni, and D. MacRae. *A Method for the Investigation and Validation of Composite Pavement Performance Including the Use of the Falling Weight Deflectometer*. Pavement Evaluation—A Joint Conference of the FWD and Road Profilers User Groups, Roanoke, VA, 2002.

Horak, E. Benchmarking the Structural Condition of Flexible Pavements with Deflection Bowl Parameters. *Journal of the South African Institution of Civil Engineering*, Vol. 50, No. 2, 2008, pp. 2–9.

Huang, Y.H. *Pavement Analysis and Design.* Prentice-Hall, Upper Saddle River, NY, 2004.

Kim, Y.R., and Park, H. *Use of FWD Multi-Load Data for Pavement Strength Estimation.* FHWA/NC/2002-006. North Carolina Department of Transportation, Raleigh, 2002.

MACTEC Engineering and Consulting, Inc. *Long-Term Pavement Performance Program Manual for Falling Weight Deflectometer Measurements, Version 4.1*. FHWA-HRT-06-132. Federal Highway Administration, Washington, DC, 2006.

Noureldin, S., K. Zhu, D.A. Harris, and S. Li. *Non-Destructive Estimation of Pavement Thickness, Structural Number and Subgrade Resilience Along INDOT Highways.* FHWA/IN/JTRP-2004/35. Purdue University, West Lafayette, IN, 2005.

Park, K., N. Thomas, and K.W. Lee. Applicability of the International Roughness Index as a Predictor of Asphalt Pavement Condition. *Journal of Transportation Engineering,* Vol. 133, No. 12, 2007, pp. 706-709.

Rohde, G.T. Determining Pavement Structural Number from FWD Testing. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1448. Transportation Research Board of the National Academies, Washington, DC, 1994, pp. 61-68.

Smith, B., and B.K. Diefenderfer. *Development of Truck Equivalent Single-axle Load (ESAL) Factors Based on Weigh-in-motion Data for Pavement Design in Virginia*. VCTIR 09-R18. Virginia Center for Transportation Innovation and Research, Charlottesville, 2009.

Stantec Consulting Services Inc., and H.W. Lochner, Inc. *Development of Performance Prediction Models for Virginia Department of Transportation Pavement Management System*. Richmond, VA, 2007.

Virginia Department of Transportation. *Guidelines for the 1993 AASHTO Pavement Design Guide*. VDOT Materials Division, Richmond, 2003.

Virginia Department of Transportation. *State of the Pavement 2010*. VDOT Maintenance Division, Richmond, 2010.

Virginia Department of Transportation. *Supporting Document for the Development and Enhancement of the Pavement Maintenance Decision Matrices Used in the Needs Based Analysis*. VDOT Maintenance Division, Richmond, 2011.

Virginia Department of Transportation. *VDOT Traffic Data*. http://www.virginiadot.org/info/ct-trafficcounts.asp. Accessed August 15, 2013.

Zaghloul, S., Z. He, N. Vitillo, and J.B. Kerr. Project Scoping Using Falling Weight Deflectometer Testing: New Jersey Experience. In *Transportation Research Record: Journal of the Transportation Research Board*, No. 1643. Transportation Research Board of the National Academies, Washington, DC, 1998, pp. 34-43.

Zhang, Z.E., L. Manuel, I. Damnjanovic, and Z. Li. *Development of a New Methodology for Characterizing Pavement Structural Condition for Network-Level Applications*. 0-4322-1. Texas Department of Transportation, Austin, 2003.

Zimmerman, K.A. *Pavement Management Methodologies to Select Projects and Recommend Preservation Treatments*. NCHRP Synthesis of Highway Practice 222. Transportation Research Board of the National Academies, Washington, DC, 1995.

APPENDIX

MSI DISTRIBUTIONS FOR EACH VDOT INTERSTATE

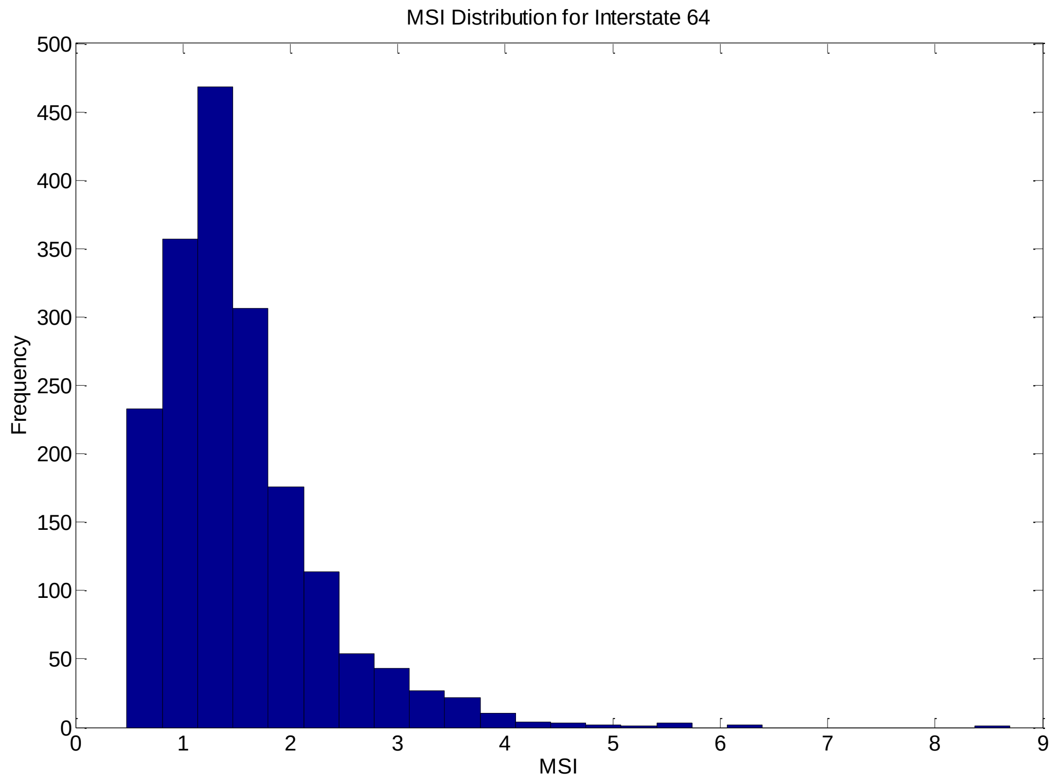


Figure A1. MSI Distribution for Interstate 64

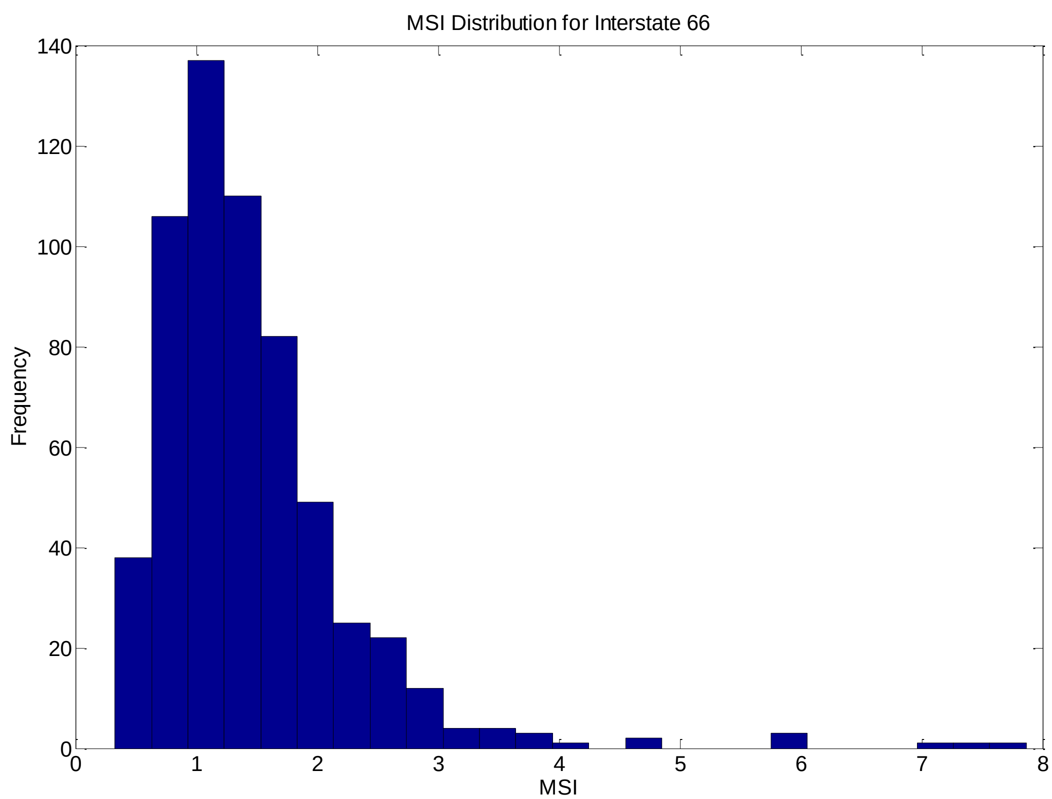


Figure A2. MSI Distribution for Interstate 66

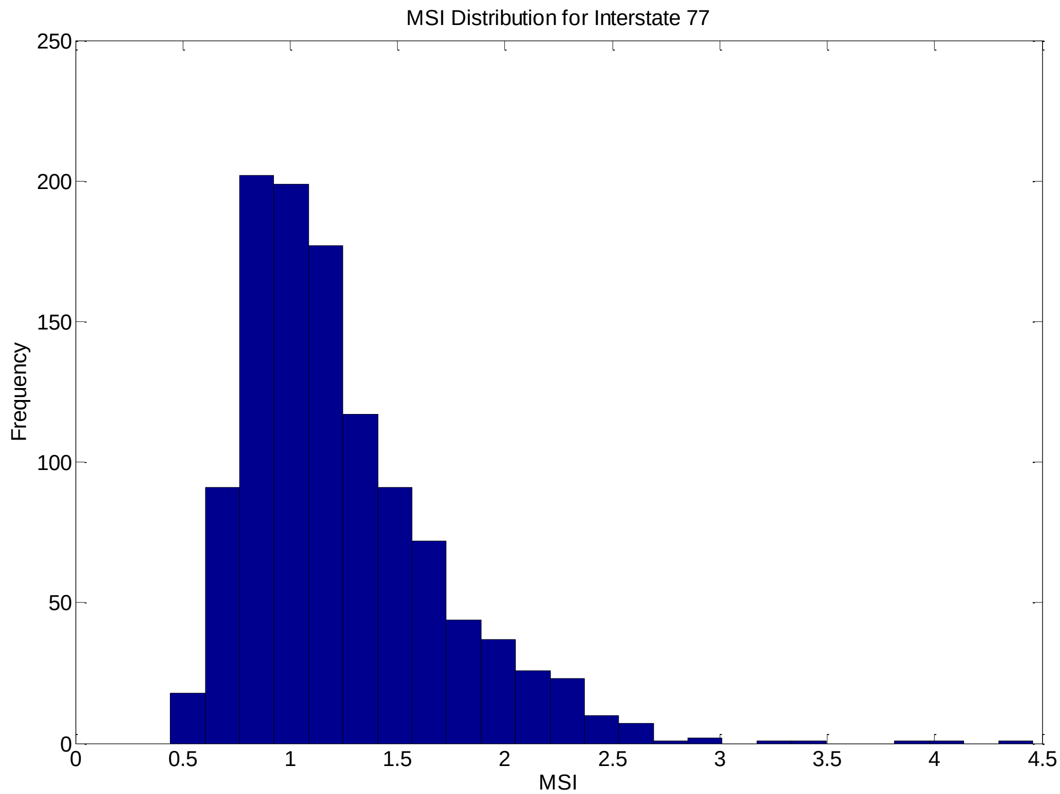


Figure A3. MSI Distribution for Interstate 77

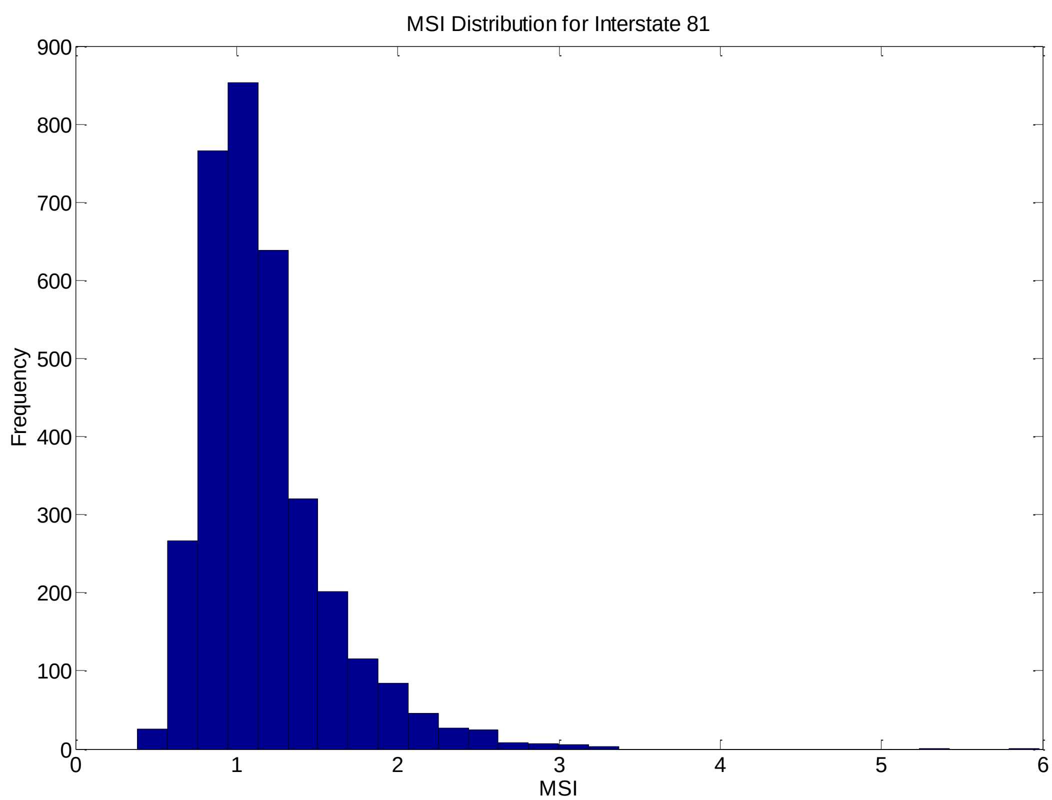


Figure A4. MSI Distribution for Interstate 81

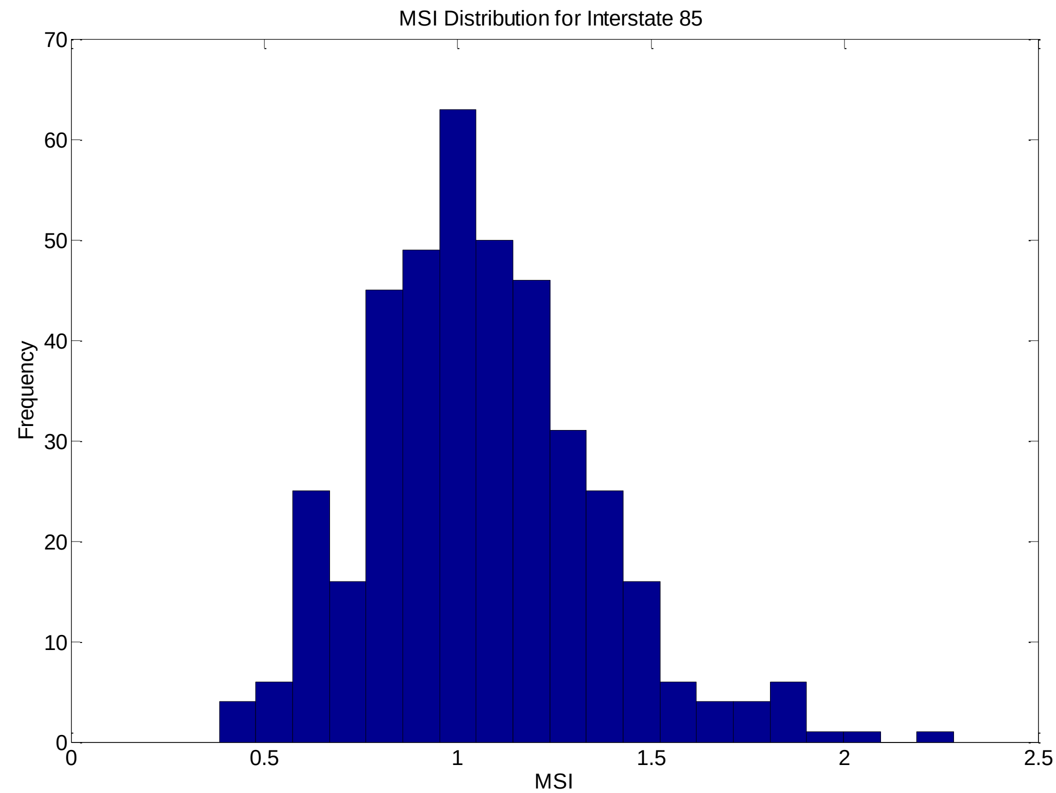


Figure A5. MSI Distribution for Interstate 85

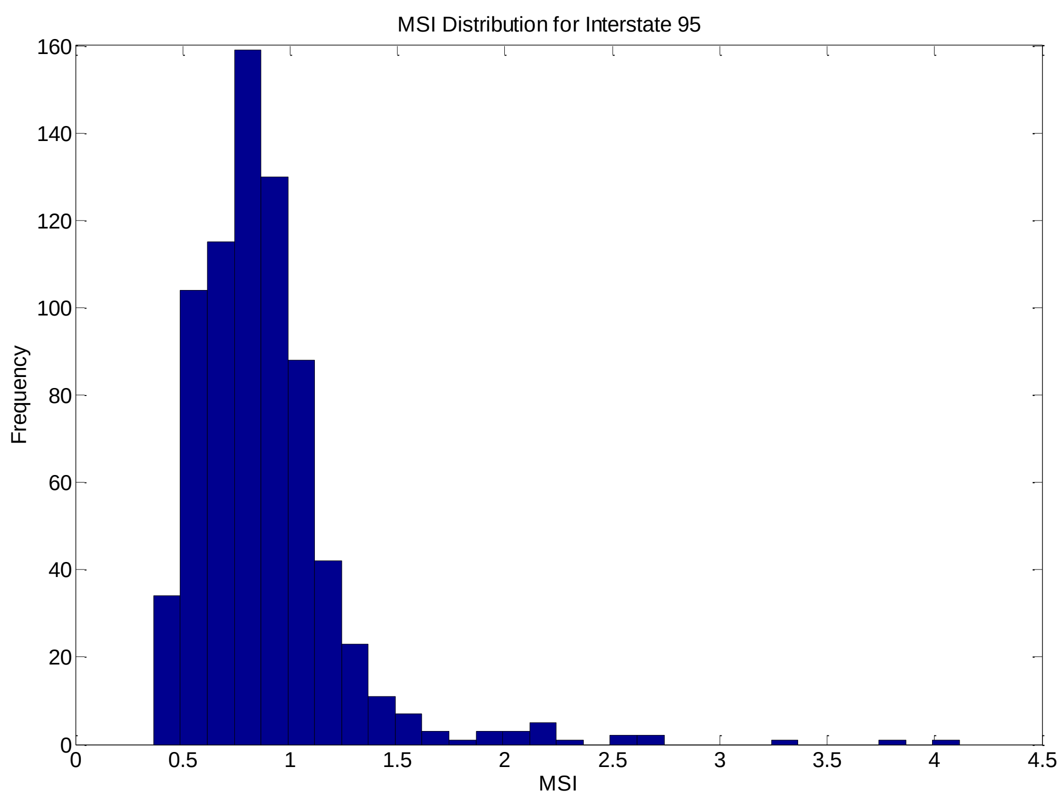


Figure A6. MSI Distribution for Interstate 95

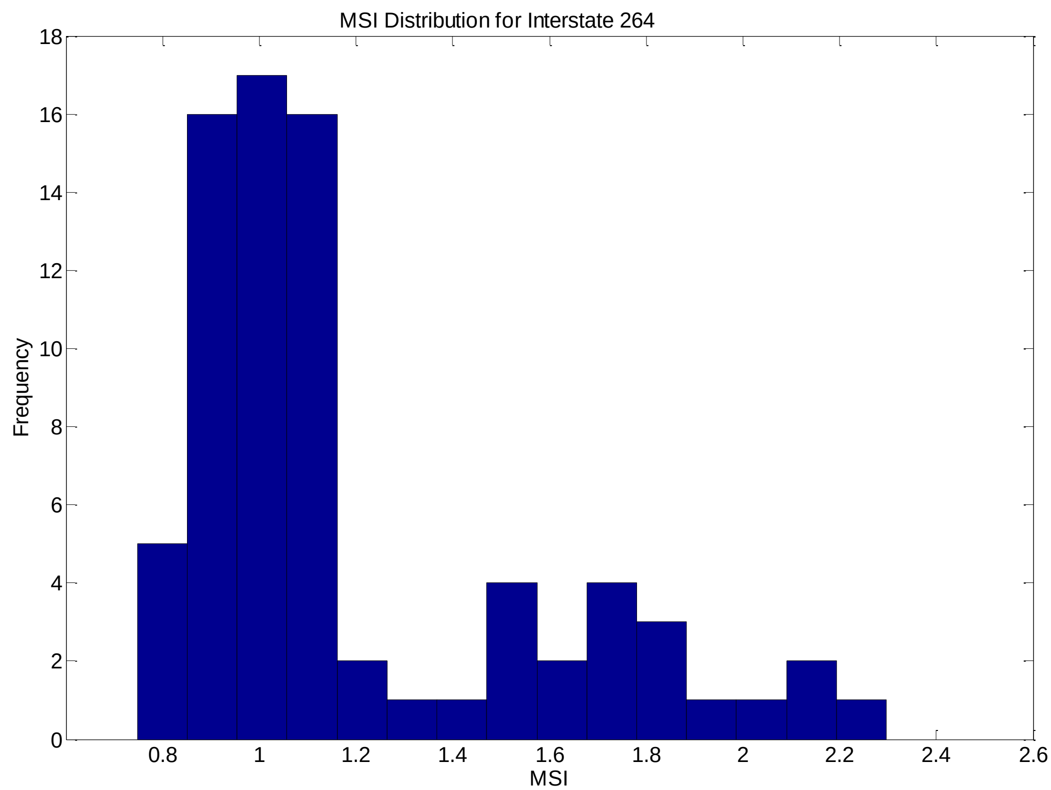


Figure A7. MSI Distribution for Interstate 264

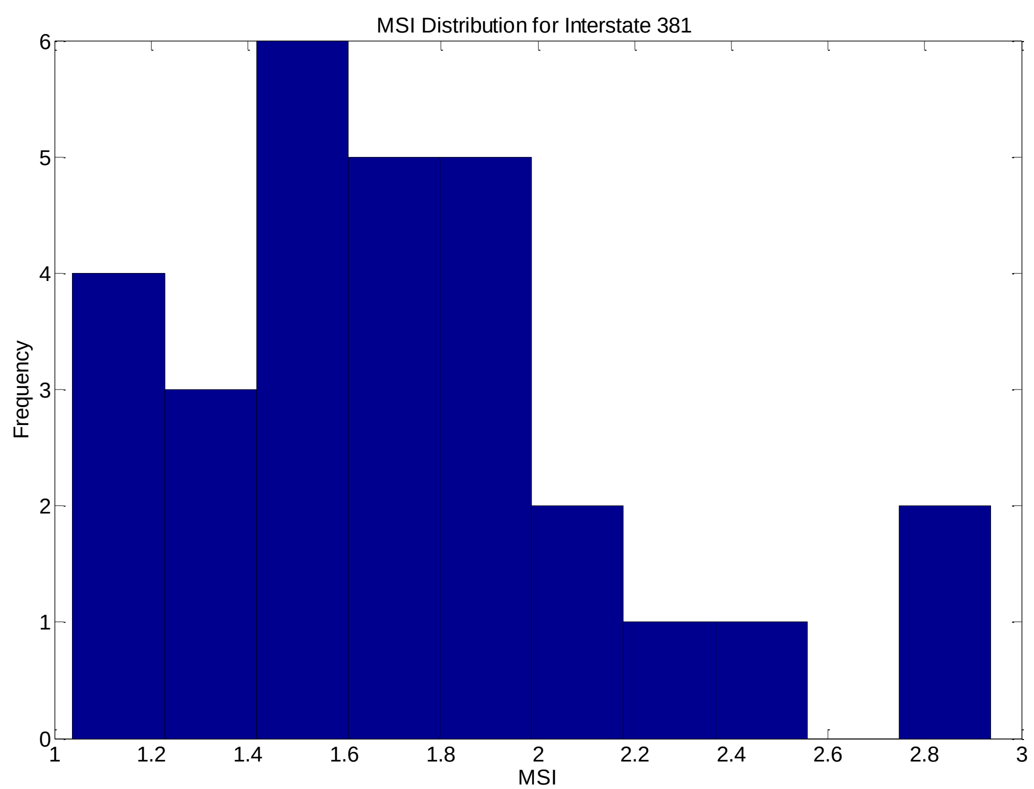


Figure A8. MSI Distribution for Interstate 381

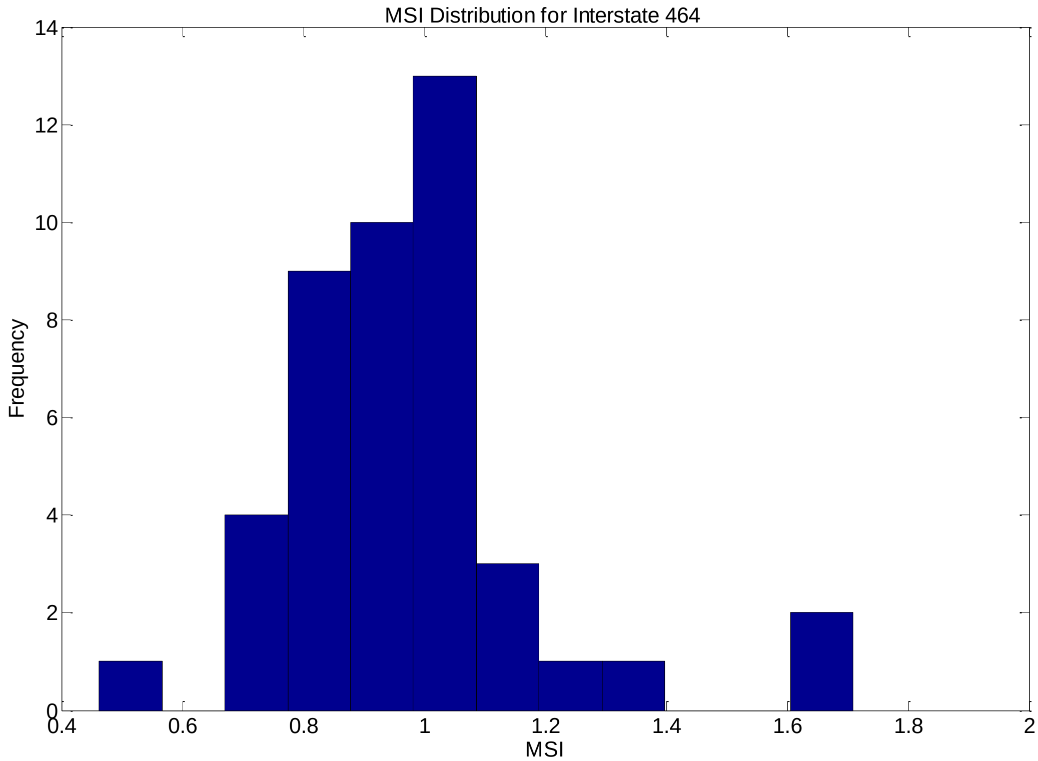


Figure A9. MSI Distribution for Interstate 464

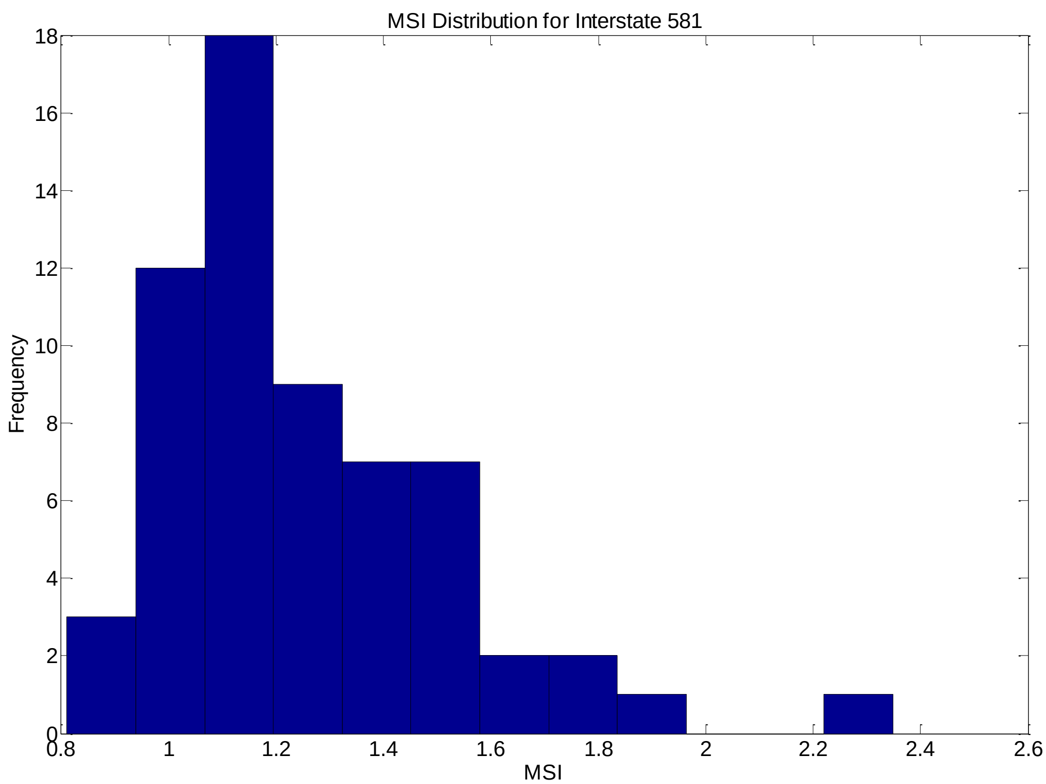


Figure A10. MSI Distribution for Interstate 581

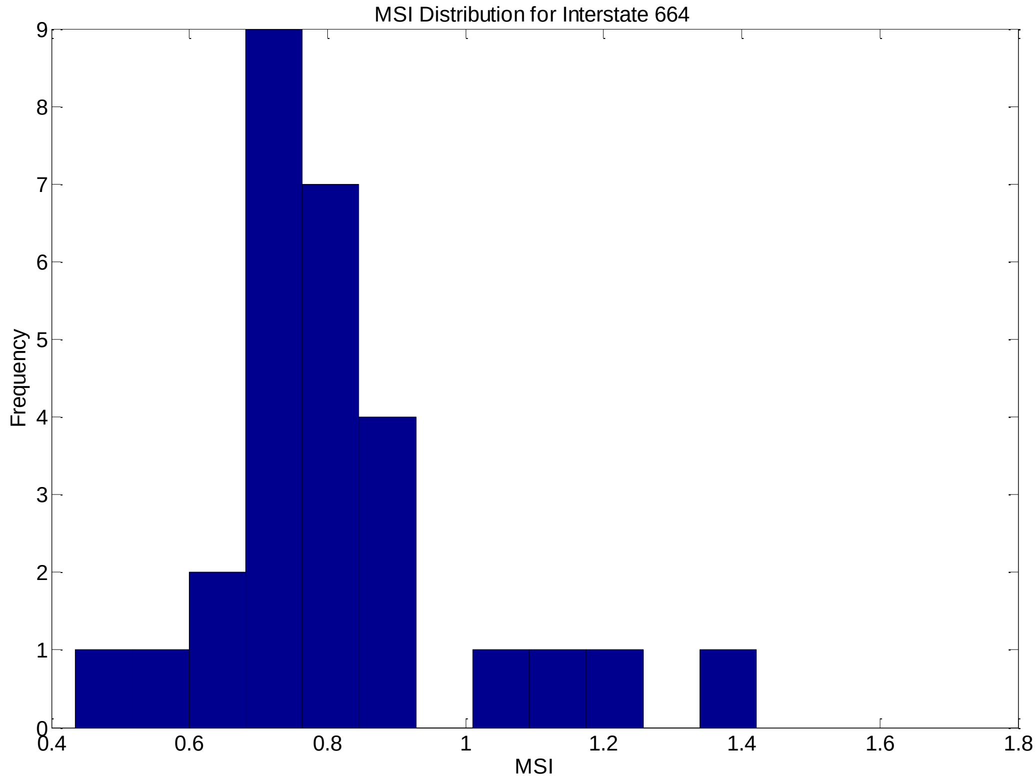


Figure A11. MSI Distribution for Interstate 664