A Pavement Structural Capacity Index for Use in Network-level Evaluation of Asphalt Pavements

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ABSTRACT

The objective of this research was to develop a structural index for use in network-level pavement evaluation, which facilitates the inclusion of the pavements structural condition in many pavement management applications. The primary goal of network-level pavement management is to maintain an acceptable condition of the pavements within the network using available, and often limited, resources. Pavement condition is described in terms of functional and structural condition, and the current widespread practice is to only consider the functional condition during network-level evaluation. This practice results in treatments that are often under-designed or over-designed when considered in more detail at the project-level. The disagreement may be reduced by considering the structural capacity of the pavements as part of the network-level decision process.

This research was conducted by identifying various structural indices, choosing an appropriate index, and then applying data from the state of Virginia to modify the index and show example application for the index. It was concluded that the Modified Structural Index best met the research objectives. Project-level and network level data were used to conduct a sensitivity analysis on the index, and example applications were presented. The results indicated that the inclusion of the Modified Structural Index into the network-level decision process minimized the errors between network-level and project-level decisions, when compared to the current network-level decision making process. Furthermore, the Modified Structural Index could be used in various pavement management applications, such as network-level structural screening, and developing structural performance measures.

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This thesis is dedicated to my wife Rebecca, who supported me unconditionally through this work, and my son Caleb Isaac who will be born after the defense of this work, but provided me with motivation nonetheless. Thank you for your encouragement.

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LIST OF ABBREVIATIONS

AASHTO…………………………..American Association of State Highway and Transportation Officials
BDI…………………………………………………………………………………...Base Damage Index
CCI……………………………………………………………….. Critical Condition Index (Used by VDOT)
CM…………………………………………………………………………………Corrective Maintenance
DN……………………………………………………………………………………………Do Nothing
FWD………………………………………………………………………Falling Weight Deflectometer
IRI……………………………………………………………………….International Roughness Index
LDR…………………………………………………….. Load-related Distress Rating (Used by VDOT)
M_R………………………………………………………………………...Resilient Modulus (Subgrade)
MSI………………………………………………………………………….Modified Structural Index
NCHRP……………………………………………National Cooperative Highway Research Program
NDR……………………………………………… ...Non-load related Distress Rating (Used by VDOT)
PM……………………………………………………………………………..Preventative Maintenance
PSI……………………………………………………………………………..Present Serviceability Index
PSI_t……………………………………………………..Terminal Value for Present Serviceability Index
RC…………………………………………………………………………Rehabilitation/Reconstruction
RM…………………………………………………………………………Restorative Maintenance
SCI……………………………………………………………………………..Structural Capacity Index
SCI_{300}……………………………………………………………………..Surface Curvature Index
SN…………………………………………………………………………………...Structural Number
SSI………………………………………………………………………..Structural Strength Index
VDOT…..Virginia Department of Transportation

CHAPTER 1. INTRODUCTION

1.1. Background

The primary goal of network-level pavement management is to evaluate and maintain an acceptable level of service of the pavements within the network using available, and often limited, resources. Resource allocation and preliminary maintenance strategy selection are typically based on the pavement condition that is determined at the network-level. Pavement condition can be described in terms of functional condition and structural condition, and the current widespread practice is to only consider the functional condition during network-level evaluation. One benefit of using only the functional condition is that the functional condition measurements are non-contact-based and the evaluation is usually performed at the prevailing traffic speed. This removes the need for traffic control that has the potential to be disruptive to the traveling public. However, this practice results in treatments that are often significantly under-designed or over-designed when they are considered in more detail at the project-level, where structural condition is often considered (Flora, 2009). This creates problems because the scope of the planned projects often changes substantially between network-level testing and project implementation, leading to significantly different costs than was originally budgeted for. This disagreement may be reduced by considering the structural capacity of the existing pavements as part of the network-level decision process.

Pavement deflection testing is the most widely used method for the nondestructive evaluation of the structural capacity of a pavement. Deflection measurements are most often gathered by using the falling weight deflectometer (FWD), a device that measures several deflections at discrete points along the pavement that result from an applied load. Results from FWD testing can be interpreted to analyze the structural capacity of all layers of the pavement, including the strength of the subgrade. Additionally, many states have begun to realize the benefits of adding network-level deflection testing into their pavement management system as reported by NCHRP Synthesis 39-01 (Flintsch and McGhee 2009). A possible use of network-level deflection testing is to develop structural health measures of a pavement network. Another possible application is the implementation of structural capacity measures into pavement warranty contracting, which may help reduce the risk taken on by the agency that owns the pavement over the life of the pavement.

1.2. Problem Statement

It has generally been believed that the functional properties of a pavement, such as the International Roughness Index (IRI), Pavement Condition Rating (PCR), and the Rut Depth of the pavement, provide adequate information about the overall in-situ condition of the pavement. Thus, structural indicators have generally been left for project specific evaluations and designs. However, recent research has shown that the structural condition of the pavement should also be considered in a pavement condition assessment for a more thorough assessment of the pavement. Although poor pavement conditions can be a result of poor structure, it is also the case that a pavement in poor functional condition is not an indicator of poor structural condition. Research performed in the state of Indiana indicated that at the 95% confidence level, there is very little statistical correlation between the center deflection of the FWD and the IRI, PCR, or the Rut Depth of a pavement (Flora, 2009). Furthermore, a study from New Jersey reported that network-level decisions that were made based only on functional condition were significantly influenced by the result of including deflection testing into the decision process (Zaghloul et al., 1998). These results show that, in many cases the functional indicators may be independent of the underlying structural condition of the pavement. Therefore, including structural capacity into network-level decision making may help close the gap between budget allocations and project needs, and result in using the most cost effective pavement rehabilitation option.

Another application for a structural index may be its use as a warranty contract specification by using it as an additional pavement screening indicator. Many states, such as New Mexico and Wisconsin, have reported considerable success from implementing warranties in their contracting processes (Cui et. al, 2008: Shober et. al., 1996). Some of the key benefits cited as a consequence of the implementation of pavement warranties are the overall improvement in quality, accelerated construction, reduced lifecycle costs (with increased initial costs typically), and the potential to increase contractor innovation (Cui et. al., 2010). The increased contractor innovation is thought to be due to the competition created over the bid, and is expected to increase with increased design inputs given by the contractor as opposed to the owning agency. Contractors will examine the use of innovative materials, methods, and construction and maintenance practices in an attempt to develop low cost, high performing products.

1.3. **Research Objectives**

The objective of this thesis was to develop a method for determining a structural index for use in network-level pavement evaluation, which also facilitates the inclusion of structural condition in network-level pavement management applications. This was done by: (1) researching and recommending adequate structural indices used in current practice, as well as other based on appropriate structural models, (2) developing a methodology for using a structural index in network-level pavement evaluation, and (3) identifying and recommending appropriate pavement management applications and situations for the use of structural measures. The index was calibrated for use in the state of Virginia: however the methodology should be applicable to other agencies that conduct network-level deflection testing.

1.4. **Thesis Outline**

Chapter 2 provides an introduction to: structural capacity measuring equipment, network-level decision making, data collection and network-level decision making at the Virginia Department of Transportation (VDOT), and previously proposed structural capacity measures. Chapter 3 discusses candidate structural indices, the correlation between functional and structural condition, and provides a comparison of the indices based on actual network-level and project-level data. Chapter 4 discusses the proposed structural index, the appropriate interpretation of the index including thresholds and other indicative values, and provides a parametric analysis to identify the important contributing parameters to the indices. Chapter 5 provides example applications using the structural index, and discusses deterioration modeling using the structural condition. Chapter 6 summarizes the main findings and conclusions, as well as provides recommendations for future research.

Chapter 2. Literature Review

The objective of this chapter is to present a basis for this thesis by discussing past research reported in the literature. Deflection testing using the Falling Weight Deflectometer (FWD) is discussed, along with some assumptions that are typically employed when interpreting pavement deflection response. Factors that affect the FWD response are presented in order to provide a background about deflection response model variations. Data collection and network-level decision making specific to VDOT is presented. Previously proposed structural indices are presented, and then the correlation between structural and functional parameters is discussed.

2.1. Introduction

The condition of a pavement is generally described using two condition categories: functional and structural. The functional condition describes the ability of the pavement to provide a certain level of serviceability (Park et al., 2007). An example of a functional measure of the pavement is the roughness of the road as perceived by the driving public. If the roughness increases past a certain value, the pavement is no longer considered to adequately serve its intended use of providing a smooth and safe riding surface. Thus, the pavement is said to have failed due to its functional condition. On the other hand, the structural condition of the pavement, measured by its structural capacity, describes the ability of the pavement to carry its design load over its intended life (Park et al., 2007). As the structural condition of the pavement deteriorates the rates of propagation of particular distresses, such as fatigue cracking and rutting, are expected to increase.

The structural capacity of a pavement is a function of properties of the many layers of the pavement. The individual layers of the pavement are designed such that stronger materials at the surface distribute the applied forces to the weaker layers by spreading the load and consequently reducing the stresses on the weaker layers. For asphalt pavements, the behavior of the pavement to an applied load has been traditionally characterized by a linear-elastic half space (Huang, 2004). Although it is true that the behavior of asphalt is visco-elastic, the treatment of its deflection response as elastic for short duration loads is generally seen as acceptable. Thus, the behavior of the pavement can be simplified and adequately modeled using mathematical techniques given the geometry, the applied load, and the material properties (e.g. Poisson’s Ratio and the Elastic Modulus).

2.2. Measuring Structural Capacity

Deflection testing is currently the most widely used method for the nondestructive evaluation of the structural capacity of a pavement. Pavement deflection measurements are important inputs for many pavement condition assessment tools, including structural capacity indicators and tools to calculate the remaining service life of pavements (Gedafa et al., 2010a). Early methods for measuring pavement deflections most often included the Benkelman Beam. The Benkelman Beam is a deflection-measuring device that is a narrow beam with a probe foot that is inserted between the dual tires of a truck and rests on the pavement approximately two feet in front of the truck axle (Kruse, 1983). The truck applies the load while moving at a creep speed. The measuring arm pivots around a fulcrum point that is attached to a reference beam resting well behind the deflection basin. The probe arm moves in respect to the resting point, and the movement is measured by a dial. The total pavement deflection between the dual tires can be read from the measuring dial.

The FWD is currently the most prevalent device for measuring pavement deflections (Hadidi & Gucunski, 2010). The FWD generates a 25 - 30 millisecond duration pulse load, intended to simulate the load from a fast moving truck, by dropping a weight and transferring a load through either a 150 mm (~6 in) or 300 mm (~12 in) load plate (MACTEC, 2006). The pavement response is then measured through a set of geophones spanned across the pavement radially from the load. The most important environmental factor affecting the response of flexible pavements is the pavement temperature, thus the pavement temperature is measured (or estimated) and deflections are corrected to a normalized temperature for each measurement (Gedafa et al., 2010b). Moisture also has a considerable effect on the response of the pavement by affecting the behavior of the soil layers in particular, but moisture variations are typically not corrected for during analysis. After the pavement response is collected, a set of analyses can be performed to determine the in-situ mechanical properties of the pavement. The input typically needed to perform this analysis is the load, temperature corrected pavement response, seed moduli and layer thicknesses.

While this research focuses on interpreting results from FWD testing, it is important to discuss modern updates in deflection testing. Flintsch et al. (2010) recently reported on a set of continuous deflection measuring devices that measures the deflection of pavements while traveling along the pavement. These devices apply a load to the pavement through a set of rear wheels of a tractor trailer and deflections are measured by a non-contact method (generally laser-based sensors).

Research is currently being conducted to evaluate continuous deflection measuring devices as a part of the Strategic Highway Research Program (SHRP2) (Flintsch et al., 2010). Gedafa et al. (2008) reported on one particular continuous deflection measuring device, the Rolling Wheel Deflectometer (RWD), and concluded that the RWD reading under the wheel and the FWD center deflection value were statistically similar based on a significant difference test statistic. However, Diefenderfer et al. (2010) reported on RWD testing conducted in Virginia and statistical testing of RWD repeatability using a non-paired t-test assuming equal variances showed that for only eight of 15 trials conducted on interstate highways and all non-interstate test sections, the RWD data were repeatable. A low linear correlation was found between the RWD and FWD measurements (adjusted R^2 values less than 0.2). It is important to note that the FWD measurements were taken several months after the RWD measurements and only on the interstate sections that had relatively low and uniform deflections. Though these devices are relatively new to pavement management, their growing popularity may dictate the need for the development of appropriate models to interpret the structural capacity of pavements using their measurements.

2.2.1. **Factors that Influence the FWD Response**

FWD testing is conducted on in-service pavements that sometimes exhibit distresses due to traffic and environmental loads. Some distresses, such as cracking, may have an effect on the measured deflection response during FWD testing. In addition to the effect of distresses, some assumptions are made in the deflection response models that also effect the interpretation of the pavement response. Many of the assumptions that are typically built into the pavement analysis algorithms are used either for computational efficiency, or because detailed information about the condition is not known (i.e., stress dependency model of base layers or moisture in subgrade). The assumptions that are made are generally accepted as good engineering practice, but may have a significant effect on the assumed behavior of the pavement structure. For instance, the moisture level in the subgrade may have a significant effect on the wave propagation effects from the dynamic nature of the FWD load. Assuming a static model will ignore the reflected stress waves from the lower layers. The reflected waves will not necessarily have an effect on the center load, but because they travel faster than the surface waves, if the pavement is relatively thin, the reflected waves could significantly affect the response of the outer FWD sensors.

For network-level analysis, which requires less detail than project-level analysis, many simplifying assumptions are made in order to accommodate a large number of computations over a short period of time. However, some simplifying assumptions can introduce significant errors in the modeled behavior. Therefore, it is important for the engineer to understand the assumptions built into the models, as well as when they may or may not be appropriate assumptions. Some factors and assumptions are presented in the following sections.

Temperature

Asphalt concrete is a non-linear visco-elastic composite material. This means that the behavior of asphalt concrete depends on the material temperature and loading rate along with the loading magnitude. In terms of interpreting FWD data, temperature correction of the deflection values has been employed in practice for a considerable amount of time. However, these temperature corrections mainly focus on the material deformation characteristics of the asphalt layer (Johnson and Baus, 1992). Another effect that temperature has on the behavior is through visco-elastic wave dispersion. As the temperature of the pavement is increased, the asphalt layer begins to exhibit behavior more indicative of a visco-elastic fluid than an elastic solid. Therefore, the wave energy from the dynamic FWD load will be dissipated more rapidly in a pavement at higher temperatures, causing less deformation further from the applied load.

Static Load Models

Elastostatic models are the most widely used models for back-calculation of the pavement moduli due to their simplicity and low computational time requirements (Westover and Guzina, 2007). The concept behind elastostatic models is that the dynamic load produced by the FWD (Figure 1) can be represented relatively accurately by an equivalent static load, and the pavement can be treated as a layered linear elastic half space. The static load model is typically considered adequate as long as the pavement is not significantly deteriorated, and wave and inertial effects are ignored. This is because the load acts over such a short period of time, the only expected response in the asphalt layer is due to the elastic deformation. This simplifies the model into a distributed load over a circular area onto an elastic system that behaves like a spring response, which is a common engineering model.

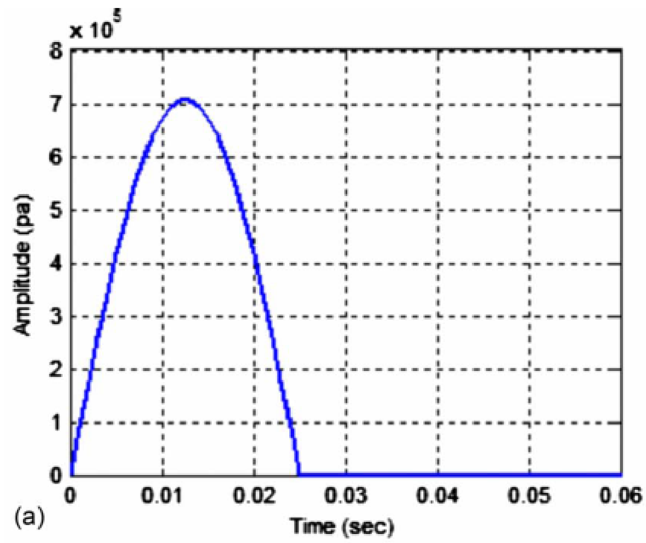

Figure 1. Time History Load for FWD Loading; Hadidi, R., & Gucunski, N. (2010). Comparative Study of Static and Dynamic Falling Weight Deflectometer Back-Calculations Using Probabilistic Approach. Journal of Transportation Engineering, 196-204. Used with permission of ASCE, 2012

The transformation of the dynamic load into a static load makes many generalizations. For instance, the generalized equation of motion for an applied load on a single degree of freedom is written as a second order differential equation as:

$$m\ddot{z} + c\dot{z} + kz = F(t) \quad (1)$$

Where m is the mass, c is the damping coefficient, k is the elastic spring constant, z is the displacement, and F(t) is the time dependent applied load.

The response of the system is time dependent for both the mass and damping terms, and thus is dependent on the time rate of application of the loads. This behavior can be seen in Figure 2 which shows the results from research conducted by Stolle et al. (1991). The research modeled the deflection response of the dynamic multi-layered pavement response, and the time dependency can be seen in the lag between the applied load and the center deflection response. However, in most cases the load is said to be static such that the model follows the following generalization:

$$kz = F \quad (2)$$

Where k is the elastic spring constant, z is the displacement, and F is the equivalent static load.

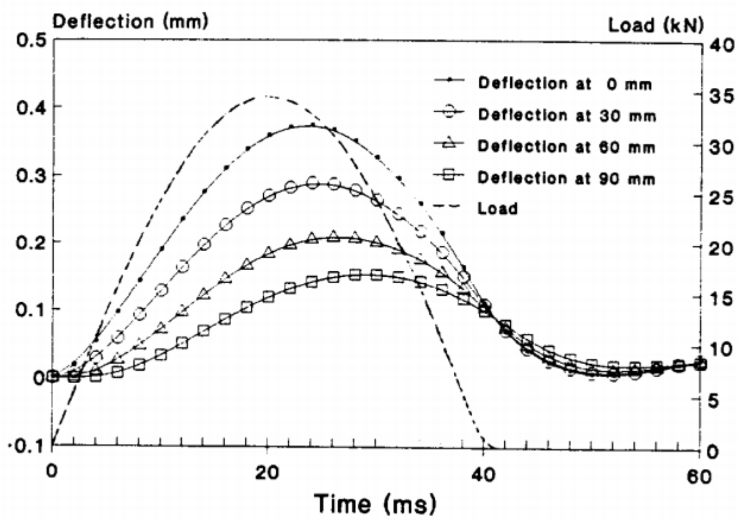

Figure 2. Dynamic Response for a Two Layer Pavement System[1]

A consequence of using the generalized static load model is that the wave propagation and inertial effects on the pavement are ignored. The FWD applies the peak load to the pavement over a very short period of time, which creates a companion set of compression waves (P-Waves) and shear waves (S-Waves). The compression waves are transmitted vertically through the pavement structure, and as an interface is broached, a reflective wave is sent back to the surface. This reflected wave causes a slight expansion in the layer that it is being reflected through. Therefore, the peak deflection in the lower layers of the

[1] Reprinted from Computers and Geotechnics, Vol. 11, D.F.E. Stolle, Modeling of dynamic response of pavements to impact loading, 83-94, 1992, with permission from Elsevier.

pavement occur as the upper layers of the pavement are rebounding, meaning that the peak measured deflection is not the same as the peak deflection of the pavement under a static load.

Past research has shown that treating the load as a static load underestimates the layer modulus when compared to treating the load as a dynamic load (Stolle, 1996). This is an effect of ignoring the inertial term in the response equation. However, the inclusion of dynamic load models is still seen as a computationally complex problem, and the static load simplification is needed to bring the problem to a point where it can be solved efficiently and quickly (Hadidi and Gucunski, 2010).

Pavement Cracking

Early research performed on simplified rational back-calculation models indicated that the treatment of the FWD load as a static load is not valid in the case of significantly cracked pavements (Stolle and Jung, 1992). Some of the differences attributed to the use of static loading models on significantly cracked pavement are due to the wave propagation effects. The shear waves created by the dynamic load contain a significant amount of energy. However, surface discontinuities, such as cracks, have a significant damping effect on the shear waves. This research was backed up by further research that analyzed the effect of pavement cracks on the response, and found that a considerably higher deflection was expected when a pavement was cracked than when not cracked (Uddin et al., 1994).

Further studies relating FWD response to cracked pavements was conducted at the Transport Research Laboratory in the United Kingdom (Nesnas et al., 2008). This research showed that pseudo wave velocity data from the geophones could be used to predict the presence of a crack. The pseudo velocity can be described by the lag witnessed in the peak deflections measured by the FWD geophones. The research stated that as long as the crack did not extend full depth, little to no effect was seen on the deflection response. However, as soon as the crack opened to full depth, a noticeable difference was seen in the deflection responses.

2.3. Pavement Management

Pavement management, a subset of infrastructure management, is the subject that seeks to use finite and often limited resources in order to evaluate and maintain the pavements within a network. Hudson et al. (1997) described infrastructure management in terms of the following:

> *Infrastructure management includes the systematic, coordinated planning and programming of investments or expenditures, design, construction, maintenance, operation, and in-service evaluation of physical facilities. It is a broad process, covering those activities*

involved in providing and maintaining infrastructure at a level of service acceptable to the public or owners.

Pavements are a service provided to the public in order to connect society and the economy to the natural environment. As the United States pavement network has expanded, the national economy has expanded as a consequence of greater connectivity between social nodes. Therefore, it is critical to assure that pavements are maintained at a level of good serviceability so that they continue to facilitate the growth and interconnectivity of society. Pavement management seeks to ensure that pavements can facilitate the needs of a connected society (mobility and access) while optimizing the cost benefit ratio of the activities required to maintain the pavements.

2.3.1. **Network-Level Analysis**

Pavement management decisions occur at two fundamental levels, network or system level, and project-level. Network-level pavement management takes a macro-level view of the assets, and is concerned with the entire system as opposed to smaller subsets within the system. Due to the broad nature of network-level pavement management, the detail of information required is less than the detail required for project-level management. However, because network-level pavement management occurs for the entire system, the data sets that are presented are large. This thesis focuses on network-level pavement management for the methods and data sets.

2.3.2. **Project-Level Analysis**

Network-level pavement management results in the identification of specific locations within the network that are in need of maintenance and rehabilitation. These locations are further investigated in more detail at what is known as project-level analysis. The goal of project-level analysis is to assess the extent of the work that is required for the pavement section, and develop a work plan for the pavement section. The information required for project-level analysis is considerably more detailed than for network-level, though the amount of sections considered is smaller.

2.4. **VDOT Data Collection**

The Virginia Department of Transportation (VDOT) manages more than 125,000 lane-miles of roads throughout the state (Heltzel, 2010). In order to more efficiently manage the condition of the pavements along these roads, VDOT maintains a database of historical condition and construction history, among other information. In 2006, VDOT began collecting distress data using digital images, and evaluating these images using automated systems (Chowdhury, 2010). The distress data collection has been contracted to Fugro-Roadware Inc., who uses a vehicle with continuous digital imaging, automated crack detection, and is equipped with sensors that measure roughness and rutting data (Heltzel, 2010). Each year, the entire Interstate and Primary road networks are evaluated, along with approximately 20% of the Secondary road network.

VDOT calculates two different condition indices from the distress information that is collected, and then combines them into a single condition value. The two indices are the Load Related Distress Rating (LDR), and the Non-Load Related Distress rating (NDR). The LDR is calculated from load related distresses, such as longitudinal cracking in the wheel path (VDOT, 2006). The NDR is calculated from non-load related distresses, such as construction deficiencies (VDOT, 2006). The CCI is then calculated as the lower of the NDR and LDR. The CCI values range from 1 to 100 and the roads are given an overall rating using the ranges can be seen in Table 1.

Table 1. Pavement Condition Definitions (Adapted from VDOT, 2006)

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
49and Below	Very Poor	Very Likely

In addition to collecting distress data, VDOT has also collected deflection data on its more than 5,400 lane-miles of interstate pavements. Testing was performed using a Dynatest Model 8000 FWD in the travel (right-hand) lane of the roadway in both directions. The sensors during the testing were located 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. Deflection testing was conducted at 0.2 mile (320 m) intervals and at three load levels: 9, 12 and 16 kips (40, 53 and 71 kN) (Diefenderfer, 2008). Original FWD data collection occurred at 10 points per mile, but Alam et al. (2006) determined that this number can be reduced without statistically compromising the data.

2.5. **Virginia Department of Transportation Decision Matrix**

VDOT utilizes a decision matrix with distresses as inputs and treatment activities as outputs. The matrices are separated based on the following administration classifications : Interstates, Primary Routes, Secondary Routes, and Unpaved Roads, in addition to the following pavement types: bituminous-surfaced (BIT), bituminous-surfaced composite pavements (with jointed concrete pavement below the surface) (BOJ), bituminous-surfaced composite pavements (with continuously reinforced concrete pavement below the surface) (BOC), continuously reinforced concrete (CRC), and jointed concrete pavements

(JCP). Additionally, updated cost estimates per mile for each treatment are available for each road category. The decision process is a two phase approach (Figure 3). In 2008, the procedure was modified to include structural condition and traffic, and the enhanced decision tree was integrated into the process.

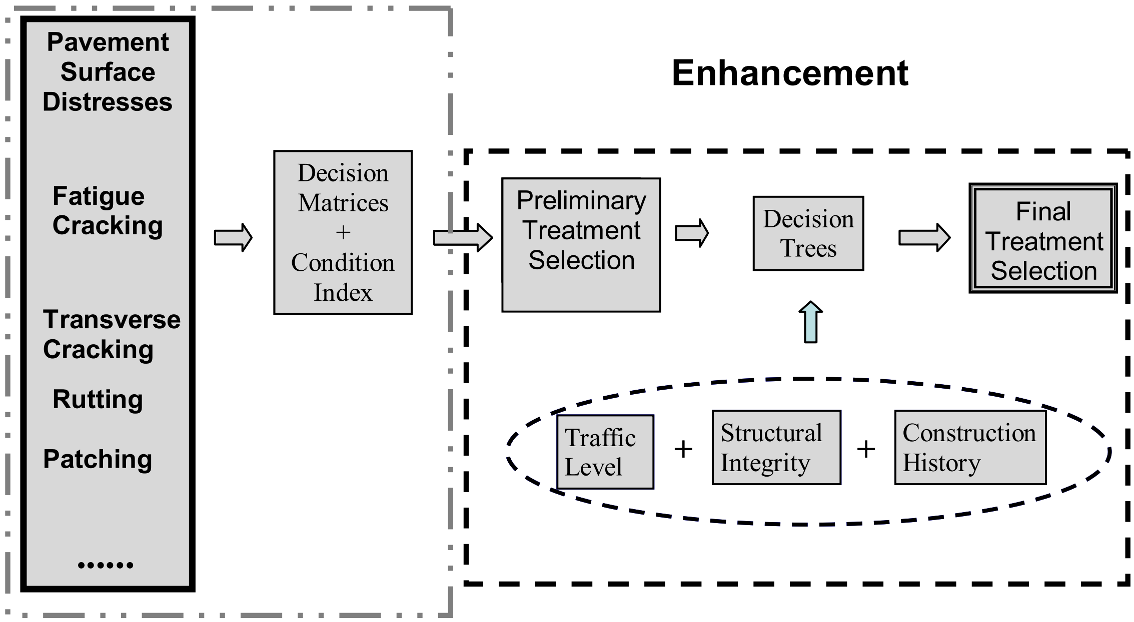

Figure 3. VDOT Two Phase Decision Process; Chowdhury, T. (2008). Supporting Document for the Devleopment and Enhancement of the Pavement Maintenance Decision Matrices Used in the Needs Based Analysis. Richmond, VA: Virginia Department of Transportation Maintenance Division. Used with permission of Chowdhury, Tanveer, 2012

The decision matrices guide the engineer to the best choice for a maintenance activity given certain distresses, and the severity of each distress. The decision matrices accommodate either single or multiple distresses as inputs. In the case where multiple distresses are present on a pavement, and yield the same maintenance activity required, one step higher of a maintenance activity is chosen. Otherwise, the highest level maintenance activity is chosen for the distresses present. After the preliminary maintenance activity is chosen, it is then directed through a Critical Condition Index (CCI) filter.

The decision matrix distress inputs are different for flexible and rigid pavements. The input distresses for flexible pavements are: Alligator Cracking, Transverse Cracks, Patching, and rutting. The input distresses for rigid pavements are: Concrete Distress Rating (CDR) and Concrete Punchout Rating (CPR) for CRC pavements, and Slab Distress Rating (SDR) for Jointed Concrete. For asphalt pavements, both the severity and frequency of distress is required as an input. For rigid pavements, only the CDR, CPR, and SDR are required for inputs into the decision matrix. An example of the decision matrix for the case of alligator cracking and rutting can be seen in Table 2. The maintenance activity categories are described in further detail in Table 3.

Table 2. Decision Matrix for the Combination of Alligator Cracking and Rutting

			Alligator Cracking								
Frequency			Rare			Occasional			Frequent		
Severity			Not Severe	Severe	Very Severe	Not Severe	Severe	Very Severe	Not Severe	Severe	Very Severe
Rutting	< 10%	None	DN	DN	CM	DN	PM	CM	PM	CM	RM
		< .5"	DN	DN	CM	DN	PM	CM	PM	CM	RM
		> .5"	CM	CM	CM	CM	CM	RM	CM	RM	RM
	>10%	None	DN	DN	CM	DN	PM	CM	PM	CM	RM
		< .5"	CM	CM	CM	CM	CM	CM	CM	RM	RM
		> .5"	RM	RM	RM	RM	RM	RM	RM	RC	RC

Where: DN = Do Nothing, PM = Preventative Maintenance, CM = Corrective Maintenance, RM = Restorative Maintenance, RC = Rehabilitation/Reconstruction.

2.5.1. Maintenance Activity Categories

The activity categories differ for interstates and primary roads from those of secondary roads (paved and unpaved). They are also different for asphalt and concrete pavements. For unpaved secondary roads, the maintenance activities are scheduled by time, instead of by measured distress (e.g., a particular treatment occurs at particular time intervals). As an example, Table 3 presents the maintenance categories for asphalt-surfaced (BIT) roads.

Table 3. Maintenance Categories for Asphalt-Surfaced (BIT) Interstate and Primary Roads (Chowdhury, 2008)

Activity Category	**Expected Life (Years)**	Activities
Do Nothing (DN)	N/A	N/A
Preventive Maintenance (PM)	2 – 5	1. Minor Patching (<5% of Pavement Area: Surface Patching: Depth 2”)
		2. Crack Sealing
		3. Surface Treatment (Chip Seal, Slurry Seal, Latex, ‘Macro Texture’, ‘Novachip’ etc.)*
Corrective Maintenance (CM)	7 – 10	1. Moderate Patching (<10% of pavement area: Partial Depth Patching: Depth 6”)
		2. Partial Depth Patching (<10% of Pavement Area: Depth 4”-6”) and Surface Treatment
		3. Partial Depth Patching (<10% of Pavement Area: Depth 4”-6”) and Thin (≤ 2”) AC Overlay
		4. ≤ 2” Milling and ≤ 2” AC Overlay
Restorative Maintenance (RM)	8 – 12	1. Heavy Patching (<20% of Pavement Area: Full Depth Patching: Depth 12”)
		2. ≤4” Milling and Replace with ≤4” AC Overlay
		3. Full Depth Patching (<20% of Pavement Area: Full Depth Patching: Depth 9”-12”) and 4” AC Overlay
Rehabilitation /Reconstruction (RC)	15+	1. Mill, Break and Seat and 9”-12” AC Overlay
		2. Reconstruction

2.5.2. **Critical Condition Index Filter**

The step after choosing an initial decision is to compare the decision against a set of minimum (or maximum) required treatments based on the CCI of the pavement. This set of bounding values based on the CCI is known as the CCI filter. The CCI filter is as follows:

Interstate:

- For CCI values above 89 the treatment category is always DN.
- For CCI values above 84 the treatment category is always DN or PM.
- For CCI values below 60 the treatment category is at least CM, i.e., CM, RM or RC.
- For CCI values below 49 the treatment category is at least RM, i.e., RM or RC
- For CCI values below 37 the treatment category is always RC.

Primary:

- For CCI values above 89 the treatment category is always DN.
- For CCI values above 79 the treatment category is always DN or PM.
- For CCI values below 60 the treatment category is at least CM, i.e., CM, RM or RC.
- For CCI values below 41 the treatment category is at least RM, i.e., RM or RC
- For CCI values below 26 the treatment category is always RC.

2.5.3. **Enhanced Decision Trees**

The final step in the decision process is to include age, structural information and traffic data by using an enhanced decision tree. The enhanced decision tree varies based on the preliminary treatment chosen. The first step is to determine the relative age of the pavement in terms of new or old. The second step is to determine the structural capacity in terms of strong or weak. For Interstate bituminous surfaced pavements (BIT), the back calculated structural number and resilient modulus are used: for interstate bituminous-surfaced composite pavements (BOC/BOJ), basin area and static K-values are used: and for interstate continuously reinforced concrete (CRC) and jointed concrete pavements (JCP), deflection under the center of the loading plate and basin area are used (Chowdhury, 2008). The final step is to determine the traffic in terms of the average annual daily truck traffic. An example of an enhanced decision tree can be seen in Figure 4 and Table 4. The decision tree in Figure 4 is to be used when the initial decision is chosen to be do nothing (DN), and the pavement is either bituminous (BIT), bituminous over jointed concrete (BOJ) or bituminous over continuously reinforced concrete (BOC).

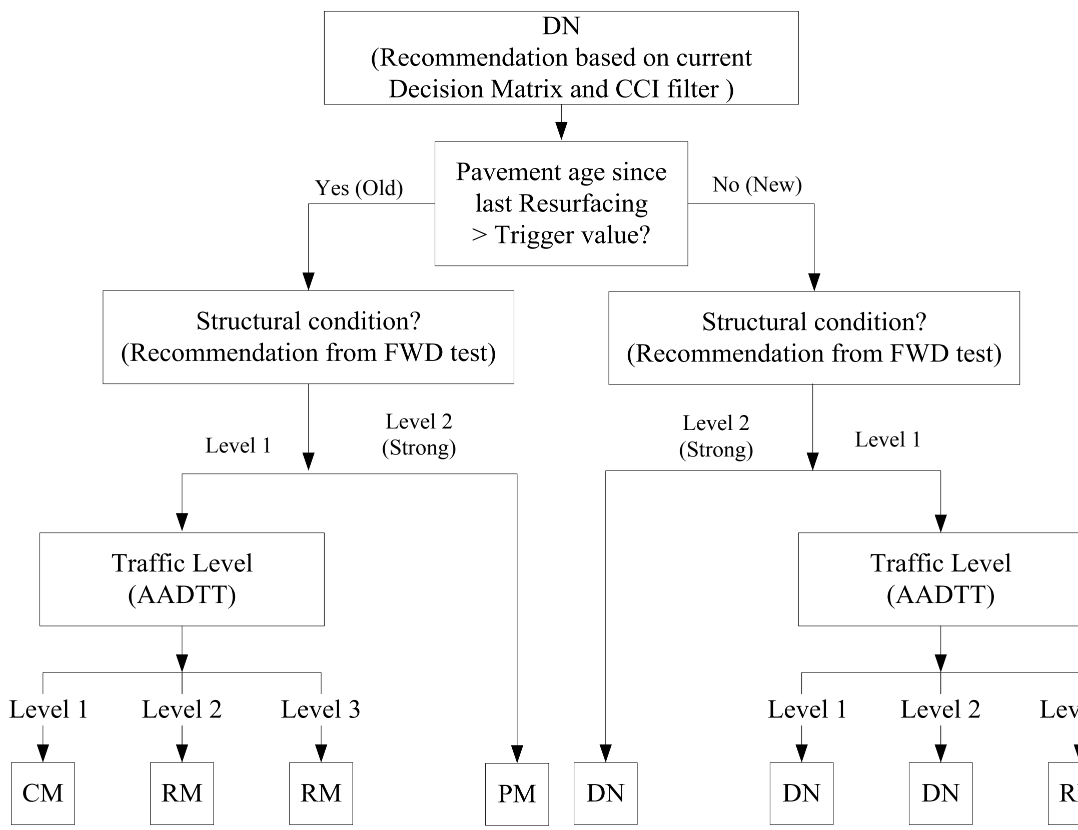

Figure 4. Augmented Decision Tree for IS (BIT/BOC/BOJ) with "Do Nothing" (Chowdhury, 2008)

Table 4. Trigger Values to use with the Enhanced Decision Tree (Chowdhury, 2008)

	Trigger Values		
Age (years)	New		Old
	≤ 6		*> 6*
FWD (BIT: SN & M_R BOC/BOJ: AREA & k)	Level 2 (Strong)		Level 1
	$SN \geq 6$ & $M_R \geq 10{,}000$ psi or AREA ≥ 32 in. & k ≥ 175 pci		*Otherwise*
Traffic (AADTT)	Level 1	Level 2	Level 3
	< 1500	[*1500, 5000*]	*> 5000*

2.6. Structural Indices

Many states have investigated the possibility of implementing structural capacity indicators into their network-level pavement management systems (Zhang et al., 2003: Flora, 2009). The structural capacity measures are derived from pavement deflections, and attempt to describe the overall in-situ state of the pavement. Many interpretation models have been developed in order to create an index that describes the in-situ structural state of the pavement. Some of the indices are based on center deflections from the FWD, while others attempt to describe the remaining life of the pavement using multiple deflection points. The following sections describe some indices that have been developed for implementation on a network wide scale.

2.6.1. Structural Adequacy Index

The structural adequacy index (SAI) was an early index developed by Hass (1994) for use with the Benkelman Beam, but can be used for any measure of deflection. The SAI can be defined on a closed scale, such as from 1 to 10. Then a maximum tolerable deflection (MTD) can be established based on the pavement and number of expected ESAL's (Hass et al., 2001). Deflections that match the maximum tolerable deflection would be considered a 5 (on the 1-10 scale), the worst deflection would be considered a 0, and the minimum deflection would be considered a 10. The 1-10 scale is arbitrary, and can be modified to meet the agency's needs.

The method for calculating the SAI is first to calculate the ESAL's for the pavement. Then the MTD can be calculated by the following equation (Hass et al., 1994):

$$MTD_B = 10^{(0.40824-0.30103*\log_{10} ESAL)} \tag{3}$$

$$MTD_B = 0.1\ for\ ESAL\ \leq 47651 \tag{4}$$

$$MTD_B = 0.02\ for\ ESAL > \ 10^7 \tag{5}$$

The MTD is given in terms of Benkelman Beam deflections, but can be converted to equivalent FWD deflections using the appropriate conversion equations. A density value is found from a table using the difference between the MTD and the design deflection (the mean deflection plus two standard deviations), the percentage of deflections exceeding the MTD, and the relative traffic levels. The density value is subtracted from the adequate score of 5 to give the SAI.

2.6.2. **Structural Strength Index**

The structural strength index (StSI) was developed by the Texas Department of Transportation in order to implement structural information into their pavement management information system. The StSI is based on the surface curvature index, and the deflection at 72 inches produced by the FWD at a 9,000 pound drop level (Zhang, 2003). It is then calculated based on values from two different tables, one for thin asphalt sections, and one for intermediate and thicker asphalt pavements. The final structural strength index is then corrected for rainfall and traffic by the following function (Scullion, 1988):

$$SSIF = 100(StSI)^{\frac{1}{RF*TF}} \tag{6}$$

Where RF is a rainfall factor and TF is a traffic factor

Scullion (1988) reported that the StSI, which is a statistically based index, produced superior results when compared to a mechanistically based index. The mechanistic methods were based on the Shell rutting and cracking models. The main drawback to the mechanistic methods was cited to be the unreliability of the mechanistic models for thicker pavement sections, as well as the complexity of trying to implement the mechanistic models into a pavement management system.

2.6.3. **Structural Capacity Index**

The Texas Department of Transportation developed a methodology for converting asphalt pavement deflections into a network-level index that is based on the effective structural number of the pavement. The development of the Structural Condition Index (SCI) was contingent on only having information from the FWD testing and the total thickness of the pavement (Zhang, 2003). The basis for the SCI is that it is possible to estimate the deflection originating solely in the pavement structure knowing that 95 percent of the deflections measured on the surface of a pavement originate below a line deviating 34 degrees from the horizontal (Irwin, 1983). The steps for determining the SCI are as follows:

1. The FWD measurements should be normalized to 9,000 lb. load deflections.
2. The deflections at an offset of 1.5 times the pavement depth are a good estimation for the deflections originating solely in the subgrade (Rohde, 1994). These can be found using the following interpolation function:

$$D_{1.5Hp} = \frac{(\mathrm{x-B})*(\mathrm{x-C})}{(\mathrm{A-B})*(\mathrm{A-C})} * (D_A) + \frac{(\mathrm{x-A})*(\mathrm{x-C})}{(\mathrm{B-A})*(\mathrm{B-C})} * (D_B) + \frac{(\mathrm{x-A})*(\mathrm{x-B})}{(\mathrm{C-A})*(\mathrm{C-B})} * (D_C) \tag{7}$$

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 & D_C are the deflections at points A, B and C respectively.

3. Determine the structural index of the pavement by the following:

$$SIP = D_0 - D_{1.5Hp} \tag{8}$$

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

4. Determine the existing pavement structural number as:

$$SN_{eff} = k1 * SIP^{k2} * H_p^{k3} \tag{9}$$

For Asphalt Pavements, k1 = 0.4728, k2 = -0.4810, k3 = 0.7581 (Rohde, 1994)

5. Estimate the design resilient modulus from the FWD measurements:

$$Mr = \frac{C*P*0.24}{D_r*r} \tag{10}$$

Where AASHTO Recommends a C = 0.33 (AASHTO, 1993), P is the load applied in pounds, and D_r is the deflection at distance r from the center deflection

6. For the calculated values of the resilient modulus and 20 year accumulated traffic volumes, the required SN can be found from Table 5.

7. Using the SN_{eff} and SN_{req}, a structural condition index can be calculated as follows:

$$SCI = \frac{SN_{eff}}{SN_{req}} \tag{11}$$

Table 5. Required SN for Mr and Required Traffic (Zhang, 2003)

<table>
<tr><td colspan="3" rowspan="3"></td><td colspan="6">20 Year Accumulated Traffic in ESAL's</td></tr>
<tr><td>Category</td><td>Very Low</td><td>Low</td><td>Medium</td><td>High</td><td>Very High</td></tr>
<tr><td>Range</td><td>50,000-945,000</td><td>945,000-1,687,000</td><td>1,687,000-2,430,000</td><td>2,430,000-3,172,000</td><td>3,172,000-50,000,000</td></tr>
<tr><td rowspan="4">Resilient Modulus, Mr (psi)</td><td>Category</td><td>Range</td><td>Average</td><td>498,000</td><td>1,316,000</td><td>2,059,000</td><td>2,801,000</td><td>26,586,000</td></tr>
<tr><td>Low</td><td>1,000-5,400</td><td>3,200</td><td>4.3</td><td>5.1</td><td>5.3</td><td>5.6</td><td>7.1</td></tr>
<tr><td>Medium</td><td>5,400-7,500</td><td>6,400</td><td>3.5</td><td>3.9</td><td>4.2</td><td>4.3</td><td>6.0</td></tr>
<tr><td>High</td><td>7,500-40,000</td><td>24,000</td><td>2.3</td><td>2.6</td><td>2.8</td><td>2.8</td><td>3.9</td></tr>
</table>

2.6.4. **Structural Strength Indicator**

The Structural Strength Indicator (SSI) was proposed in 2009 as a comparative index that is bounded and uses deflection values from the FWD. The SSI utilizes the center deflections from FWD testing over a pavement family in order to develop a function based on the cumulative distribution of the deflections. The SSI function is developed on the basis of Equation 12 and is in the form of Equation 13 (Flora, 2009).

$$SSI = 100 * \left[1 - F\left[\left(\delta_{ijk}\right)_1\right]\right] \tag{12}$$

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

$$SSI_{jk} = 100\left(1 - \alpha e^{-\frac{\beta}{(\delta_1)^\gamma}}\right) \tag{13}$$

Where the subscripts j and k denote the pavement family, and α, β and γ are found for each pavement family through minimizing the errors between Equation 12 and Equation 13

The basis for the SSI as developed by Flora is to determine the probability that a pavement in a given family will have a deflection larger than the measured deflection in a given highway section (Flora, 2009). Thus, the method compares a deflection measurement for a given pavement family to the overall deflection values for that particular family of pavements within the network. The index is on a scale of 0 to 100, with 0 being a poor SSI, and 100 being a perfect SSI. In order to utilize the values from the SSI, a set of thresholds would need to be developed for acceptable center deflections. Table 6 shows threshold values suggested by Flora (2009).

Table 6. SSI Thresholds Developed for Indiana Pavements (After Flora, 2009)

Pavement	System	Measure	Excellent	Good	Fair	Poor
Flexible	Interstate	Deflection (mil)	1.7	2.4	3.1	3.8
		SSI	99.5	74.8	40.2	20.8
	Non-Interstate National Highway System (NHS)	Deflection (mil)	2.7	3.8	5.0	6.1
		SSI	95.3	65.1	36.1	21.3
	Non-NHS	Deflection (mil)	3.7	5.8	8.0	10.1
		SSI	96.7	65.2	36.5	21.5
Rigid	Interstate	Deflection (mil)	1.8	2.3	2.9	3.5
		SSI	90.4	66.3	42.9	24.2
	Non-Interstate NHS	Deflection (mil)	2.6	3.1	3.7	4.3
		SSI	94.2	69.5	50.8	15.1
	Non-NHS	Deflection (mil)	3.2	4.9	6.5	8.2
		SSI	89.3	59.9	38.8	24.6

2.6.5. **Remaining Service Life**

The Kansas Department of Transportation and researchers from Kansas State University have developed a set of regression equations to estimate the remaining service life (RSL) of a pavement from the center deflection under a 9,000 lb. FWD load (Gedafa et al, 2010a). The Remaining Service Life (RSL) is the anticipated number of years left in a pavements functional or structural service life. The RSL employs sigmoidal performance models, and the center deflection of the FWD to predict a pavements remaining life. The RSL equations were calibrated based on information from non-interstate routes and showed good correlation to the remaining life predictions based on serviceability. Further work would need to be performed in order to calibrate the sigmoidal models for the road categories in other states.

The sigmoidal models developed by Gedafa (2010a) are in the form:

$$\mathrm{RSL} = \delta + \frac{\alpha}{1+e^{\beta-\gamma d_0}} \tag{14}$$

Where the variables are defined as,

$$\delta = \delta_0 + \delta_1 D + \delta_2 EAL + \delta_3 ETCR + \delta_4 EFCR + \delta_5 Rut + \delta_6 SN_{eff} \tag{15}$$

$$\alpha = \alpha_0 + \alpha_1 D + \alpha_2 EAL + \alpha_3 ETCR + \alpha_4 EFCR + \alpha_5 Rut + \alpha_6 SN_{eff} \tag{16}$$

$$\beta = \beta_0 + \beta_1 D + \beta_2 EAL + \beta_3 ETCR + \beta_4 EFCR + \beta_5 Rut + \beta_6 SN_{eff} \tag{17}$$

$$\gamma = \gamma_0 + \gamma_1 D + \gamma_2 EAL + \gamma_3 ETCR + \gamma_4 EFCR + \gamma_5 Rut + \gamma_6 SN_{eff} \tag{18}$$

The factors δ_n, α_n, β_n and γ_n (for n=1, 2, 3, 4, 5 & 6) are constants derived from regression models for different pavement types. D is the pavement depth, EAL is the Equivalent Axle Load per day, ETCR is the Equivalent Transverse Cracks, EFCR is the Equivalent Fatigue Cracking, Rut is the rut depth in inches and SN_{eff} is the effective structural number of the pavement.

2.6.6. **Index Based on Weighted Differences of Deflections**

An additional index to evaluate network-level deflections was tested in this thesis. The index was designed to reflect the critical differences in deflection measurements, defined by deflection basin parameters, and weight each difference according to a relative cost of rehabilitation or repair associated with each set of differences. Then, to account for traffic, the index was multiplied by a ratio of the actual traffic to the maximum feasible road capacity. The deflection basin parameters that were used to simulate differences were the Surface Curvature Index and Base Damage Index. In addition to the two difference parameters, an indicator of the subgrade condition, and the temperature corrected center deflection was added to the index.

According to research performed by Xu et al. (2002), the Surface Curvature Index (SCI_{300}) was the parameter that was most sensitive to the condition of the top asphalt layer of the pavement as measured by the modulus of the asphalt. Similarly, the Base Damage Index was the most sensitive parameter to the stiffness of the base layer, and an index labeled AI_4, which was developed by the researchers, was the most sensitive parameter to the condition of the subgrade (Xu, et al. 2002). An increase in value for each of these parameters is indicative of a reduction in their stiffness, and consequently a decrease in their contribution to the overall structure of the pavement. Thus, an equation in the form of the following equation can be developed:

$$\frac{ESAL}{ESAL_{MAX}}(SCI_{300} * A + BDI * B + AI_4 * C + D_0) \tag{19}$$

Where the SCI_{300} in this case is the Surface Curvature Index (the center deflection minus the deflection at 300 mm (12 in)), the BDI is the Base damage index (the deflection at 300 mm (12 in) minus the deflection at 600 mm (24 in)), the AI_4 is the index developed by Xu (2002), D_0 is the temperature corrected center deflection, and the factors A, B and C are indicative of the weighted cost associated to each layer index.

2.7. Summary of Literature Review

The structural capacity of a flexible pavement, as obtained from the results of deflection testing, can be used to supplement the pavement management decision process at the network-level. VDOT has implemented some structural information into their network-level decision process based on the results of network-level pavement deflection testing. Pavement deflection testing is most often conducted using the FWD. Although there are many factors that influence the response of the FWD, most current algorithms that are used to interpret pavement response into structural capacity address these factors.

There are several structural indices that have been used for supporting network-level pavement management applications. These include: the Structural Adequacy Index (Haas et al. 1994), the Structural Strength Index (Scullion, 1988), the Structural Capacity Index (Zhang, 2003), the Structural Strength Indicator (Flora, 2009), and methods to interpret Remaining Service Life (Gedafa et al., 2010).

CHAPTER 3. CANDIDATE INDICES

This chapter selects and compares three indices that are the most promising for implementation. The comparison is made against the baseline of the current VDOT methodology for implementing structural data into network-level decisions. First, the index functions are modified to match the stated goals of this research. Then, they are set to match Virginia thresholds from the VDOT decision process. In order to compare the indices with each other, and with the current VDOT methodology, the results from three pavement rehabilitation projects were chosen for analysis. The work done at the project-level was compared to the work that was predicted from the network-level data.

3.1. Preliminary Screening of the Structural Indices

An internal study performed by the Texas DOT found that the StSI was not sensitive enough to distinguish significantly different pavements in terms of distresses. A report by Zhang (2003) cited the following, "US-79 was in very good condition as it was reconstructed: whereas, US-77 had substantial amounts of distress such as alligator cracking, pumping, and rutting. In other words, the conditions of the two highways were significantly different. However, the results from the study indicated that the calculated S(t)SI values at an 85 percent confidence interval for the two highways were not very different: 90 for US-79 and 79 for US-77." Based on this, as well as the fact that the method was calibrated for Texas pavements, the StSI was not chosen to be researched further in this study.

The RSL models were calibrated specifically for Kansas non-interstate routes. Although the methodology employed by the researchers to develop the RSL models is repeatable, the development and implementation of RSL models would require additional field tests to calibrate the linear sub models. Thus, RSL will not be considered further in this thesis. However, the Kansas Department of Transportation reported good correlations between center deflections and remaining life, as well as the possibility to replace the center deflection from FWD with the deflection reported by a continuous deflection device (Gedafa et al, 2010). Thus, this methodology should be considered in future research.

The SAI was developed by an approximate fatigue analysis model, and is a bounded index. However, in order to use the SAI, the pavement must be sectioned before the analysis. Each section is then analyzed using the SAI method, as opposed to each deflection value being analyzed individually. The index is based on maximum tolerable deflections for a pavement section given certain traffic conditions. This is a similar concept to the SCI, which finds the minimum tolerable structure required for a pavement section given certain parameters, with the obvious difference being that the SCI is not bounded. Based on the relative age of the relationships, the requirement that the pavement be sectioned before analysis, and its similarity to the SCI, the SAI was not studied further during this thesis.

3.2. Assessing the Correlation of Deflection Measurements to Pavement Condition Rating

Based on the deflection data and pavement condition data collected on Virginia Interstate I-81 Southbound, a study was undertaken to determine the level of correlation between structural condition and the condition rating based on functional parameters. This section compares center deflection, the SSI, SCI and the weighted differences index to total alligator cracking, IRI, rut depth, and the condition indices used by VDOT: CCI, NDR and LDR. The deflection information was obtained in 2007 from FWD testing at 0.2 mile intervals, and the condition ratings were obtained from 2007 and were taken to represent 0.1 mile sections. Therefore, the condition rating at the locations of structural testing was averaged over 0.2 miles around the areas of deflection testing in order to compare similar sections.

The Spearman Rank Correlation was calculated in order to assess the relationship between the structural and functional parameters. This method was chosen because it does not require a specific distribution of the data, and is not limited to a linear relationship between the data. Instead, the only requirement is that the data only increases or decreases in relation to each other. The Spearman Rank Correlation is a non-parametric statistic that measures ranks in the differences in the data. The correlation coefficient is defined as (Zimmerman et al., 2003):

$$\rho_{xy} = 1 - \frac{6 * \sum_{i=1}^{n} d_i^2}{n(n^2-1)} \; for\ sufficiently\ large\ n\ values\ (n \sim > 30) \qquad (20)$$

Where: n is the number of samples and d is the difference as defined by:

$$d_i^2 = (X_i - Y_i)^2, X_i and\ Y_i\ are\ the\ samples \qquad (21)$$

To test whether there is any correlation between the measures, the following hypotheses were tested and the results are tabulated in Table 7:

H_0: There is no correlation, $\rho = 0$

H_1: $\rho \neq 0$

As can be seen in Table 7, there is a correlation between the structural parameters and the following functional parameters: LDR, NDR, CCI, and Total Alligator Cracking. Also, there is a correlation between IRI and center deflection of the FWD as well as a correlation between the IRI and SSI. The level of correlation is very weak, with the absolute value of all of the coefficients less than 0.14. The low levels of correlation can be observed graphically. Figure 5 shows the SCI versus the LDR for I-81 Southbound. It can be clearly seen that for LDR values below about 55, the SCI values do not exceed 2. However, as the LDR values increase to near perfect (100), there is a much wider range of SCI values. This can also be seen in Figure 6 where (except for 7 locations) pavements with an LDR less than about 70 have a center deflection less than 11 mils. The pavements represented in Figure 5 and Figure 6 are flexible.

Table 7. Correlation between Structural and Functional Parameter

	Distress	Sample Size	Spearman Rank Coefficient	Reject H_0 at 95% (p-value)
Center Deflection	LDR	1672	-0.1444	Yes (0)
	NDR	1672	-0.1285	Yes (0)
	CCI	1672	-0.1254	Yes (0)
	IRI	1672	0.0597	Yes (0)
	Rut Depth	1657	0.0319	No (0.19)
	Total Alligator Cracking	1672	0.1382	Yes (0)
SCI	LDR	1672	0.0895	Yes (0)
	NDR	1672	0.0754	Yes (0.002)
	CCI	1672	0.0662	Yes (0.007)
	IRI	1672	-0.0205	No (0.40)
	Rut Depth	1657	0.0246	No (0.31)
	Total Alligator Cracking	1672	-0.095	Yes (0)
SSI	LDR	1672	0.1467	Yes (0)
	NDR	1672	0.1284	Yes (0)
	CCI	1672	0.1275	Yes (0)
	IRI	1672	-0.0532	Yes (0.03)
	Rut Depth	1657	-0.0311	No (0.20)
	Total Alligator Cracking	1672	-0.1407	Yes (0)
Weighted Differences of Deflections Method	LDR	1672	0.0124	No (0.61)
	NDR	1672	0.0708	Yes (0.004)
	CCI	1672	0.0736	Yes (0.003)
	IRI	1672	-0.0368	No (0.13)
	Rut Depth	1657	-0.1296	Yes (0)
	Total Alligator Cracking	1672	-0.0186	No (0.45)

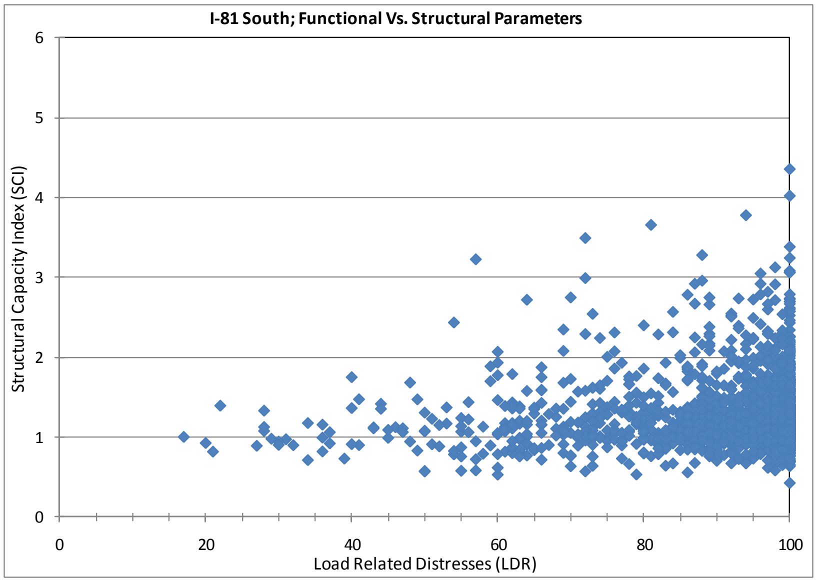

Figure 5. SCI vs. LDR on I-81 Southbound

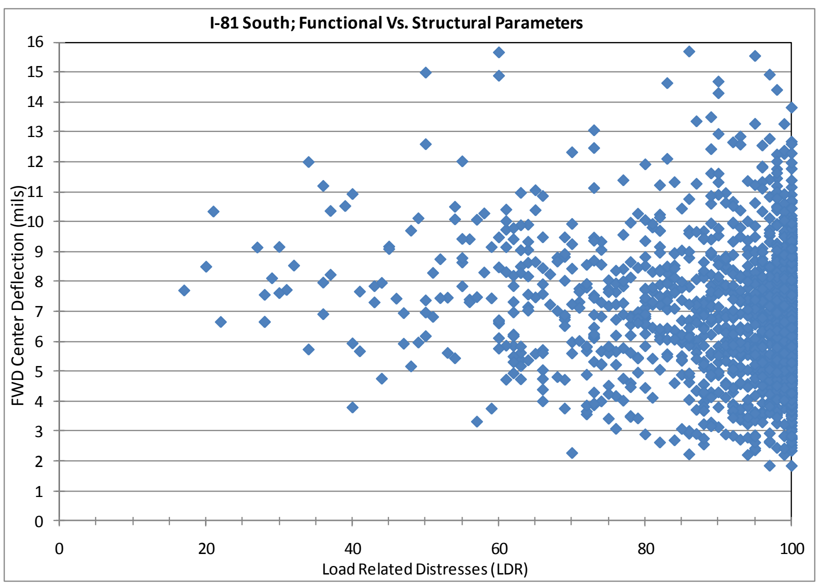

Figure 6. FWD Center Deflection vs. LDR on I-81 Southbound

The previous figures seem to indicate that for pavements in very poor functional condition (i.e., poor LDR or poor IRI), functional characteristics may be indicative of poor structure. However, pavements that exhibit good functional performance may be in poor structural condition. This can be explained by the fact that highway agencies attempt to keep their pavement networks at a certain level of performance. Thus, as a pavement's functional characteristics deteriorate, maintenance may be performed to bring the functional characteristics back to minimum standards, while structural deficiencies are not addressed.

3.3. **Methodologies for Implementing the Indices**

The three indices that were chosen for further evaluation, the Weighted Difference Index, SCI and SSI, were further analyzed, and modified in order to match project data that was available. The indices were also revised in order to be implemented easily into spreadsheet format so that large amounts of data could be analyzed efficiently.

3.3.1. **Weighted Differences of Deflections Methodology**

The first step in developing the weighted differences methodology is to determine the equations related to each of the parameters. The Surface Curvature Index (SCI_{300}) is defined as the center deflection minus the deflection at 300 mm (12 in) (D_0-D_{300}). The Base Damage Index (BDI) is defined as the deflection at 300 mm (12 in) minus the deflection at 600 mm (24 in) (D_{300}-D_{600}). Each of the deflections that define the SCI_{300} and BDI are significantly influenced by pavement temperature (Chen, et al. 2000). In order to correct the deflections to a normalized temperature, the delta basin shape factors that were developed as part of the Long Term Pavement Performance project (LTPP) were used. These particular basin shape factors were chosen because the input information was commensurate with the information that was collected during the pavement deflection testing. The following factors were used to determine the SCI_{300} and BDI as defined by the LTPP results (FHWA 1998):

For the SCI_{300}, the Delta12 factor can be used directly.

$$\text{Log}(\text{Delta}_{12})=3.45-1.59*\text{Log}(ac)+0.489*\text{Log}(theta)+0.449*\text{Log}(D_{36})-0.0275*T+0.012*T*\text{Log}(ac)*\text{Log}(theta) \quad (22)$$

Where ac is the total thickness of the HMA in mm, theta is the latitude of the pavement section, D_{36} is the deflection (load-normalized to 40.5 kN (9 kip)) at 915 mm (36 in.) from the center of the load plate in µm, and T is the temperature at mid-depth of the HMA in degrees Celsius.

The temperature at the mid depth of the asphalt can be estimated using the BELLS2 equation as:

$$T_d=2.78+0.912*IR+\{\log(d)-1.25\}\{-0.428*IR+0.553*(1\text{-day})+2.63*\sin(hr18-15.5)\}+0.027*IR*\sin(hr18-13.5) \quad (23)$$

Where T_d is the pavement temperature at depth d in degrees Celsius, IR is the pavement surface temperature in degrees Celsius, log is the base 10 logarithm, d is the depth at which material temperature is to be predicted in mm, 1-day is the average air temperature the day before testing in degrees Celsius, sin is the sine function on an 18-hr clock system with 2π radians equal to one

18-hr cycle, and hr_{18} is the time of day, in a 24-hr clock system but calculated using an 18-hr asphalt concrete (AC) temperature rise-and-fall time cycle.

For the BDI, the $Delta_{24}$ factor minus the $Delta_{12}$ factor can be used. The $Delta_{12}$ factor is calculated as was shown above, and the $Delta_{24}$ factor calculated as.

$$\text{Log(Delta24)}=3.30-1.32*\text{Log(ac)}+0.514*\text{Log(theta)}*\text{Log(defl36)}-0.00622*T*\text{Log(theta)}*\text{Log}(D_{36})+0.00838*T*\text{Log(ac)}*\text{Log(theta)} \qquad (24)$$

Where the factors for calculating $Delta_{24}$ are defined the same as in calculating $Delta_{12}$, and the temperature at the mid-depth is calculated using the BELLS2 equation.

The AI_4 factor was developed as an area index by Xu (2002), and is calculated as follows:

$$AI_4 = \frac{D_{900}+D_{1200}}{2*D_0} \qquad (25)$$

Where D_{900} and D_{1200} are the deflections at 900 mm (36 in) and 1200 mm (48 in) respectively, and D_0 is the temperature corrected center deflection.

In order to estimate the factors A, B, and C a cost associated with the degradation of each layer was developed. As noted, the condition of the top asphalt layer was indicated mostly by the SCI_{300}. Thus, the A factor associated with the SCI_{300} should be a cost associated with repairing the upper layer of asphalt. Similarly, the B factor should be a cost associated with more extensive maintenance of the base layer, and the C factor should be a cost associated with maintenance that extends to the subgrade. The Virginia Department of Transportation has developed costs associated with particular maintenance activities, as can be seen in Table 8. As preliminary values for the factors A, B and C, corrective maintenance was chosen to represent the upper layer, restorative maintenance was chosen to represent the base layer, and Rehabilitation /Reconstruction was chosen to represent the subgrade. This resulted in normalized factors of A=1, B=2.52 and C=7.07.

Table 8. Costs for Different Activity Category for Interstate Asphalt (BIT) Pavements (VDOT, 2006)

Activity Category	**Expected Life (Years)**	**Activity Cost ($/Lane Mile)**
		Most Likely
Do Nothing (DN)	N/A	N/A
Preventive Maintenance (PM)	2 – 5	$6,975.90
Corrective Maintenance (CM)	7 – 10	$71,817.50
Restorative Maintenance (RM)	8 – 15	$180,631.65
Rehabilitation /Reconstruction (RC)	15+	$506,885.50 *(BOJ and BOC Pavements)*
		$507,958.45 *(BIT Pavements)*

3.3.2. **Structural Strength Indicator Methodology**

The initial step in determining the SSI was to determine the standard normal cumulative distribution (Figure 7). From the cumulative distribution results, the function for the SSI was developed by minimizing the sum of square of the errors between one minus the cumulative distribution function and equation 13. The minimized errors yielded the following function for the SSI along the bituminous interstate for I-81:

$$SSI = 100\left(1 - 1.0069 * e^{-\frac{1071.8}{(\delta_1)^{3.9622}}}\right) \quad (26)$$

Where δ_1 is the FWD center deflection

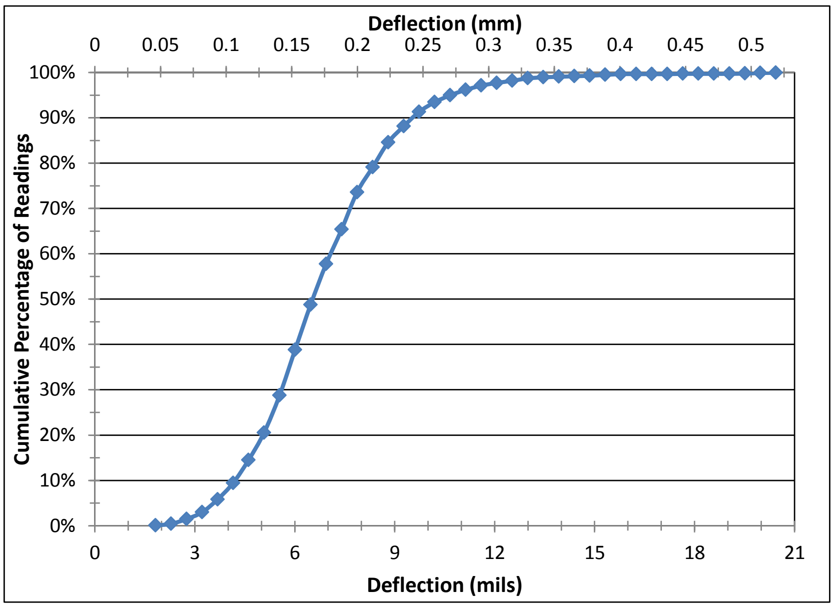

Figure 7. Cumulative Distribution of Deflections along I-81 South

SSI Thresholds in Indiana

The SSI only provides an index of deflections relative to other deflections along the pavement, and not whether the pavement deflection value is acceptable. Therefore a set of deflection thresholds, similar to the thresholds developed in Indiana, would need to be developed for each pavement type and road classification. The thresholds developed in Indiana for flexible interstate sections are presented in Table 9. Based on the deflections presented in Table 9 and the SSI equation for I-81, a new set of SSI thresholds was calculated and included in the last row of Table 9.

Table 9. SSI Thresholds for Indiana Interstates with Flexible Pavement Construction

Pavement	System	Measure	Excellent	Good	Fair	Poor
Flexible	Interstate	Deflection (mil)	1.7	2.4	3.1	3.8
		SSI From Indiana	99.5	74.8	40.2	20.8
		SSI Calculated from I-81 Data	100	100	99.9	99.5

The SSI values that were calculated based on the deflections illustrate the need to develop a set of thresholds specific to the characteristics of Virginia's Interstate system. Noureldin et al. (2005) developed a set of deflection thresholds based on cumulative ESAL's, and ranked corresponding deflections using a set subjective ratings (i.e. good or poor) specific to Indiana's pavement network.

Integration of SSI with Other Condition Indices

A major benefit of using the SSI methodology is that it is on the same scale as the main condition index used by VDOT (CCI). Therefore, the SSI could feasibly be weighted and combined with the CCI. Another benefit is that the same index can be used for flexible and rigid pavements with no adjustments made other than the calculation of a new SSI curve. A major drawback is that the index does not incorporate traffic directly. Therefore, the index could possibly identify two pavement sections in poor structural health, but only one of the pavement sections experiences significant traffic. Therefore, equal weight is given to pavements that may experience significant more loadings than their counterparts.

3.3.3. **Structural Capacity Index Methodology**

The methodology for calculating the SCI was presented for flexible pavements in Section 2.4.3 of this thesis. It is possible that a similar methodology can be developed for rigid pavements using back-calculation techniques. However, the prevalence of flexible pavements in many highway networks has resulted in more research available for techniques for evaluating flexible pavements. The SCI methodology is modified in the next section, thus it will be distinguished by using the term modified.

Calculating the Effective SN

The calculation of the effective structural number (SN_{eff}) for the pavement in the SCI methodology presented in Chapter 2 of this thesis utilizes an empirical relationship. Finding the SN_{eff} through this relationship differs from the method presented in the AASHTO design guide, which uses an open form equation to determine the effective structural number. Furthermore, Diefenderfer (2008) presented results for the network-level estimation of the effective structural number of Virginia Interstates using the AASHTO method, which was programmed into an analysis tool. Therefore, to compare the results obtained from each method, several thousand data points tested with the FWD were analyzed using both methods. The results are shown in Figure 8.

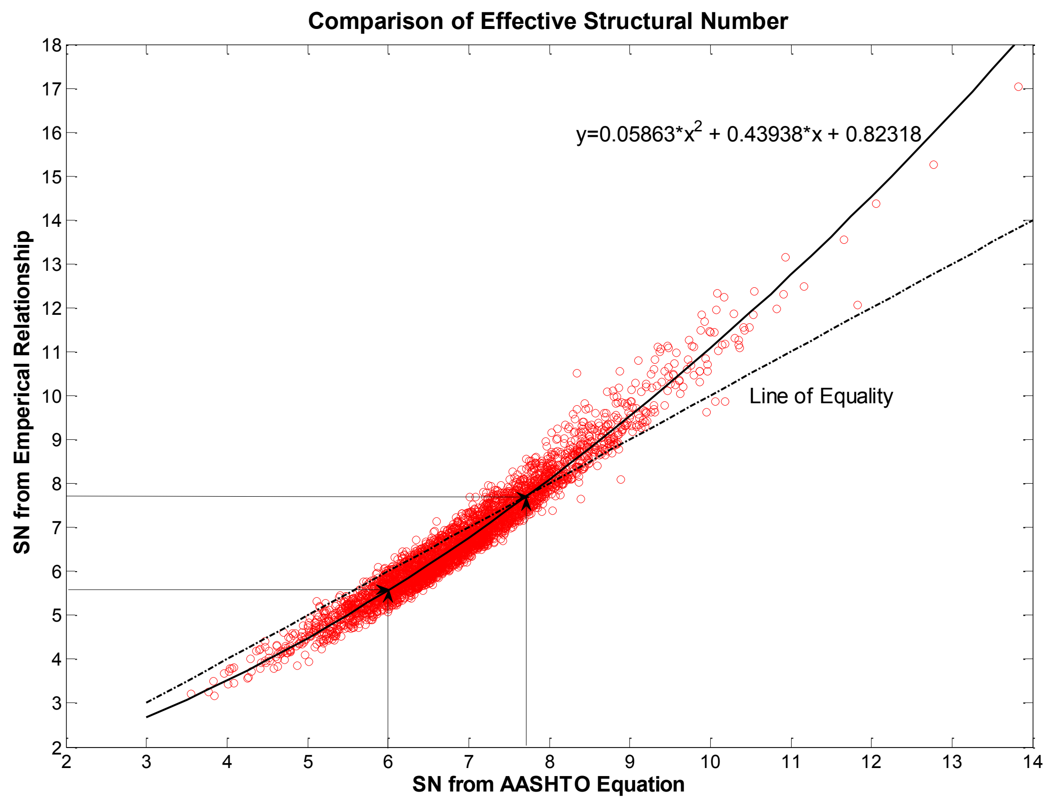

Figure 8. Comparison of Effective SN from AASHTO Equation and Rhode (1994) Method

The relationship shown in Figure 8 was calculated using orthogonal regression given that error existed in both sets of data. It can be seen that even though the relationship is not along the line of equality, the data follows a trend. For lower values of SN, the empirical relationship tends to underestimate the SN when compared to the AASHTO method, and that trend reverses for SN values above approximately 7.7.

Calculating the Required SN

In order to calculate the SCI, the method requires the input of an SN required (SN_{Req}) (step 6, Section 2.4.3). The Texas Department of Transportation uses a set of tabulated values of SN_{req} based on the resilient modulus and traffic (Table 5). These discrete points are useful in the case that many different levels of traffic and resilient modulus are not encountered over the pavement being studied. However, upon investigating the case for I-81 southbound in Virginia, it was found that the resilient modulus varied from 4,500 psi to more than 30,000 psi and the 20 year accumulated ESAL's (from 2009 values) varied from $2.5*10^7$ to $8*10^7$. One major reason in the large variation of traffic is that Interstate 64 runs along the same pavement as I-81 for a length near central Virginia. Based on the large variation of traffic and resilient modulus, as well as the fact that a continuous function would be useful for programming purposes, it was decided to develop a closed form function for the AASHTO SN equation in order to calculate SN_{req}. The AASHTO SN equation is an open form equation as given by equation 27:

$$\log(W_{18}) = (Z_R * S_0) + 9.36 * \log(SN + 1) - 0.2 + \left(\frac{\log\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40+\frac{1094}{(SN+1)^{5.19}}}\right) + 2.32 * \log_{10} M_R - 8.07 \quad (27)$$

Where W_{18} is the equivalent single axle loads (traffic), Z_R is the Z statistic from the standard normal distribution, S_0 is the standard deviation for the material, SN is the structural number of the pavement, ΔPSI is the PSI of the constructed pavement minus the PSI_T, and M_R is the resilient modulus of the subgrade in psi.

The first step in developing a closed form solution to the AASHTO SN equation was to fix a number of variables. VDOT has a design guide set up to guide engineers, that presets the following parameters for interstates: PSI should be set at 4.2, the PSI_T should be set to 3, the reliability should be 95% and the standard deviation should be set at 0.49 (VDOT, 1993). After fixing these values, a range of traffic and resilient modulus values were randomly generated. The generated values represented the range of traffic and resilient modulus values found along I-81 in Virginia. The values were then input into the AASHTO equations, and an SN was solved for each case using Microsoft Excel solver. The summary statistics for the traffic and resilient modulus values that were used, as well as the SN values that were obtained, are shown in Table 10. The distribution for the SN values that were obtained from the traffic and resilient modulus are presented in Figure 9.

Table 10. Summary Statistics for Chosen ESAL's and Resilient Modulus

ESAL's		*Resilient Modulus*		*SN*	
Mean	9.61E+07	Mean	9,623	Mean	6.57
Standard Error	9.38E+06	Standard Error	375	Standard Error	0.20
Median	1.10E+07	Median	9,400	Median	6.43
Mode	3.00E+06	Mode	14,300	Mode	7.48
Standard Deviation	1.26E+08	Standard Deviation	5,047	Standard Deviation	2.65
Sample Variance	1.59E+16	Sample Variance	2.55E+07	Sample Variance	7.02
Kurtosis	-0.062	Kurtosis	-1.254	Kurtosis	-0.38
Skewness	1.12	Skewness	-0.029	Skewness	0.62
Range	4.36E+08	Range	16,900	Range	11.52
Minimum	5.00E+04	Minimum	1,100	Minimum	2.50
Maximum	4.36E+08	Maximum	18,000	Maximum	14.02
Sum	1.74E+10	Sum	1,741,700	Sum	1,189
Count	181	Count	181	Count	181

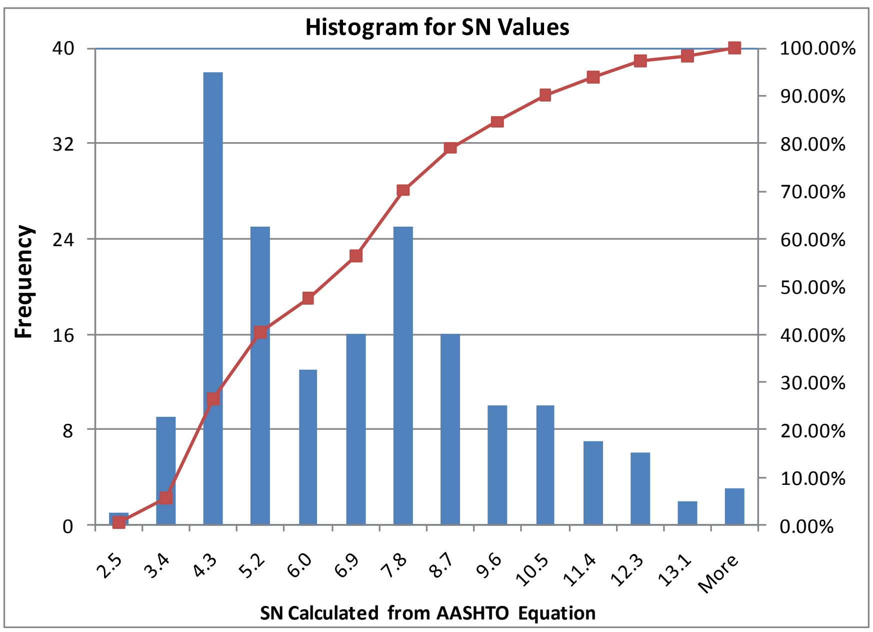

Figure 9. SN Values from Generated Traffic and Resilient Modulus

After fixing the parameters found in the VDOT design guide and developing a set of SN values from the AASHTO equation, the following equation was formed by rearranging the original AASHTO equation:

$$\log(ESAL) - 2.32 * \log(M_R) + 9.07605 = 9.36 * \text{LOG}(\text{SN}+1) - \left(\frac{0.352183}{0.4+\left(\frac{1094}{(\text{SN}+1)^{5.19}}\right)}\right) \quad (28)$$

The left side of the equation is the closed form equation that was sought. Based on this, a plot of the errors, defined as the solution of the left hand side of the equation minus the AASHTO solution, is presented in Figure 10. It is clearly seen in Figure 10 that the behavior of the error follows an exponential functional form. Thus, a solution in the form of an exponential function that included the left hand side of Equation 28 was sought.

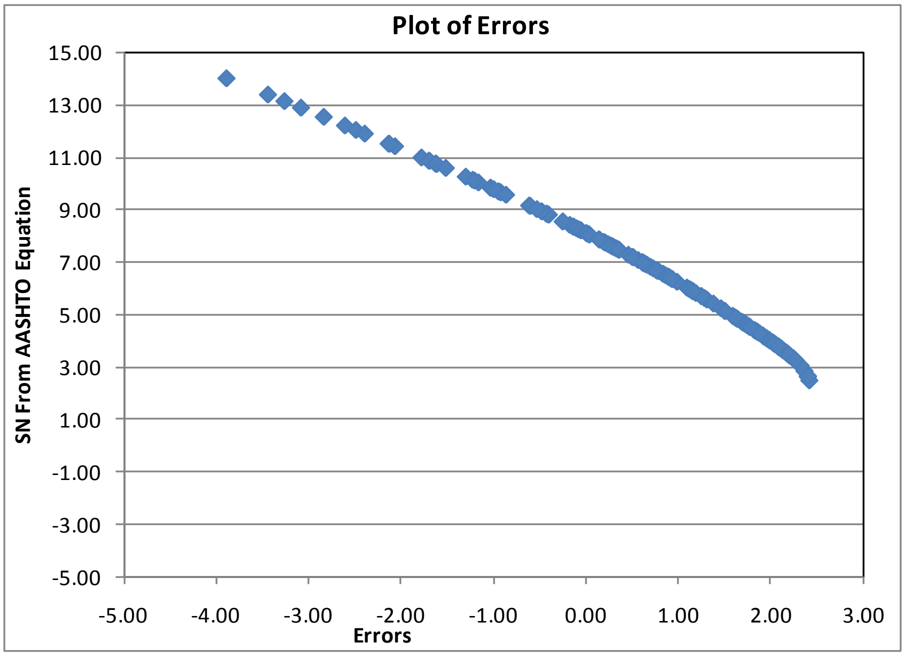

Figure 10. Plot of Errors: Closed Form Equation minus AASHTO Equation

Microsoft Excel was used in order to minimize the errors between the assumed form of the equation and the AASHTO solution, where the assumed form was:

$$\alpha(log(ESAL) - 2.32 * log(Mr) + 9.076056)^{\gamma} \quad (29)$$

Based on the results, the final closed form solution was found to be optimal with α= 0.05716 and γ= 2.36777. The plot of the solutions recalculated from the closed form solution and the AASHTO solution can be seen in Figure 11. The relationship follows the line of equality with an R^2 of 0.9997. Thus, using the closed form equation for SN, as well as the equations defined in the previous methodology, the Modified SCI can be calculated as:

$$Modified\ SCI = \frac{0.4728*\left(D_0 - D_{1.5Hp}\right)^{-0.4810} * Hp^{0.7581}}{(0.05716*(\log(ESAL) - 2.32*\log(\text{Mr}) + 9.07605)^{2.36777})} \quad (30)$$

Where D_0 is the FWD center deflection for an equivalent 9000 pound load, $D_{1.5Hp}$ is the deflection at 1.5 time the pavement depth, Hp is the pavement depth, ESAL is the calculated traffic, and Mr is calculated as ((0.33*9,000*0.24))/(D_{60}*60) with D_{60} as the deflection at 60 inches away from the center of the load.

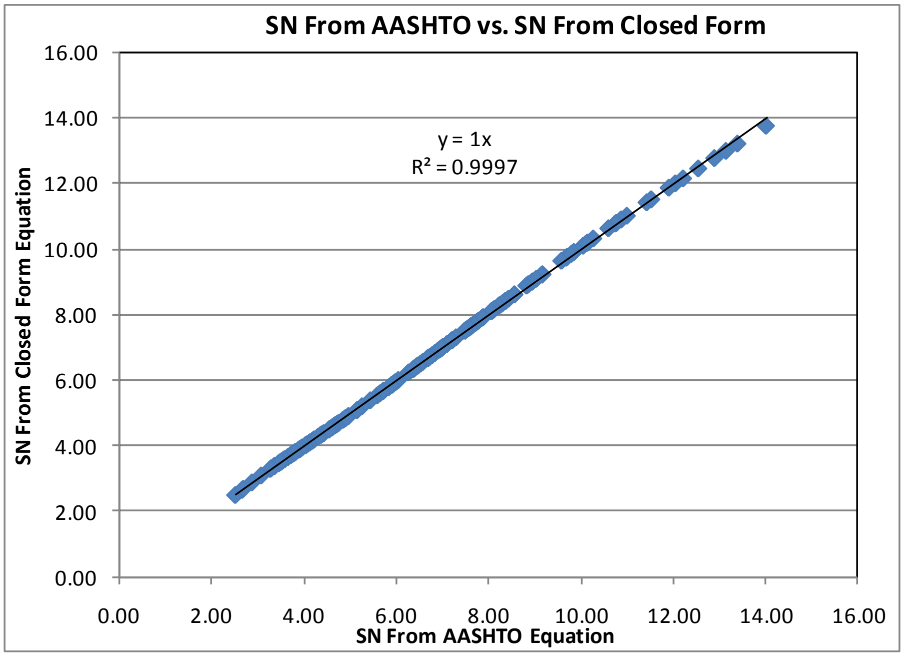

Figure 11. Comparison of AASHTO and Closed Form Equations

Integration of Modified SCI with Other Condition Indices

The Modified SCI is an unbounded index, which consequently means that the index will provide absolute condition, as opposed to the condition relative to other locations in the pavement system. Thus, the Modified SCI cannot be scaled to match other index scales like the pavement condition ratings used by many departments of transportation. However, the development of thresholds may prove to facilitate a way to integrate the Modified SCI with bounded condition indices by providing ranges in which a certain structural condition may be defined.

3.4. **Comparing the Indices Using VDOT Methods and Data**

A set of analyses were conducted using VDOT data and applying the structural indices as additional indicators. Three projects that were completed in the two years following deflection testing were evaluated using the project data, as well as the network-level condition and deflection data. The pavement condition was supplied by VDOT as data collected and aggregated into 0.1 mile sections on average over the Interstate 81 corridor throughout Virginia. The condition data was supplied for the years 2007 through 2010.

The analysis in the following sections was performed in order to compare the indices with each other, as well as the current VDOT process. Therefore the thresholds and critical values are based on VDOT recommended values. After the comparison is made, and an index is chosen as optimal, a sensitivity analysis will be performed in order to determine the actual critical values that should be used.

3.4.1. **Modified SCI Thresholds**

In order to analyze the impact of the structural indices on the decision process, a set of thresholds was developed to simulate trigger values for treatments. The thresholds for the Modified SCI were developed directly from applying the trigger values used by VDOT from the enhanced decision trees. This was accomplished because the main inputs for the Modified SCI methodology are the effective structural number, traffic data and the resilient modulus of the subgrade. For bituminous (BIT) interstate pavements, the following methodology was used to develop the thresholds:

$$SCI = \frac{SN_{eff}}{SN_{req}} \quad (31)$$

Where SN_{eff} is determined by deflection testing, and the SN_{req} is determined from traffic and resilient modulus data.

Substituting the SN_{req} obtained through the regression analysis, the equation becomes:

$$Modified\ SCI = \frac{SN_{eff}}{(0.05716*(\log(ESAL)-2.32*\log(\mathrm{Mr})+9.07605)^{2.36777})} \quad (32)$$

The trigger values for bituminous interstate pavements are: an effective structural number of 6 or a resilient modulus value of 10,000 psi. Three truck traffic levels of 0 to 1,500 AADTT, 1,500 to 5,000 AADTT, and greater than 5,000 AADTT represent low, intermediate and high traffic levels (VDOT 2008). In order to convert the AADTT into ESALs, the following AASHTO equation was used:

$$ESAL = (AADTT)(T_f)(G)(D)(L)(365)(Y) \quad (33)$$

Where 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 assumption of a 20 year design life with 3% growth was used, leading to a composite growth factor (G*Y) of 26.87. The truck factor was defined from previous VDOT research for the Virginia Interstate network (Diefenderfer, et al. 2009), and was ultimately taken to be 0.96 for the combination of trucks on Interstate 81. The directionality factor was taken as 0.5, and the lane factor was taken as 0.9 (Huang 2004). Substituting the VDOT trigger values and traffic into the Modified SCI equation, the following range of thresholds were developed for Modified SCI: greater than 1.08, between 1.08 and 0.91, and less than 0.91, corresponding to the low, intermediate, and high traffic values respectively. These Modified SCI thresholds were compared to the traffic range over the Interstate 81 corridor through Virginia, and can be seen in Figure 12. The traffic ranges for Interstate 81 all fall within the high and medium traffic categories.

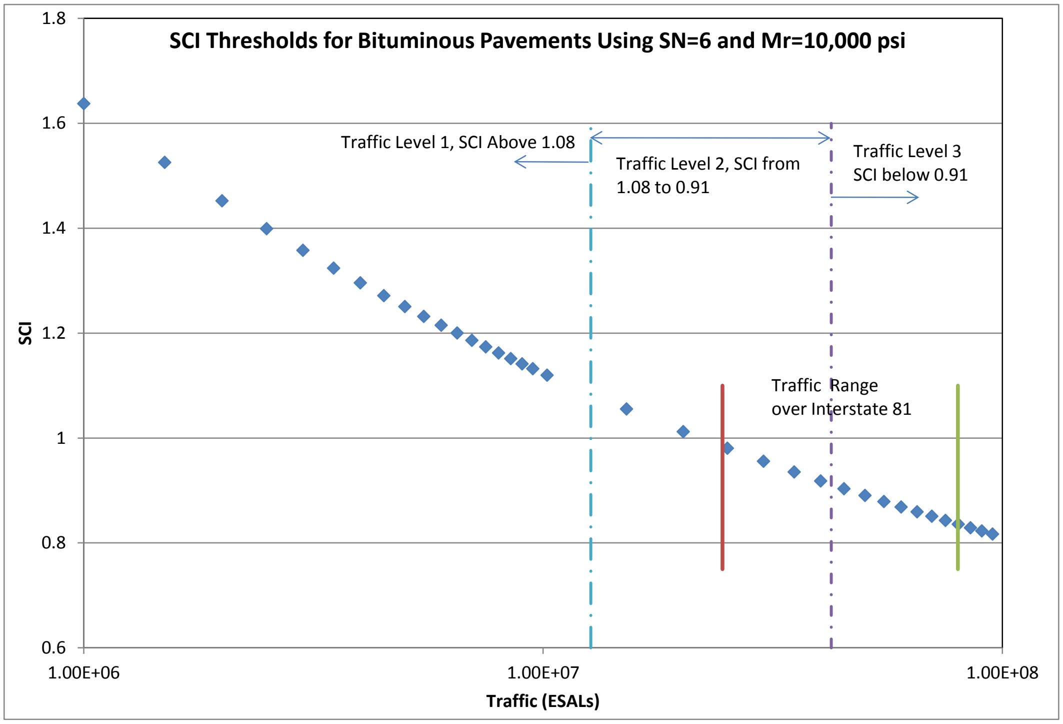

Figure 12. Modified SCI Threshold Values for Bituminous Interstate Pavements Using VDOT Trigger Values

3.4.2. SSI Thresholds (Center Deflections)

Given that the SSI is solely a function of the center deflection values from the FWD, the thresholds and applications of the SSI to projects will be based on the center deflection values. Also, the SSI does not account for differing traffic levels, so a single threshold will be developed to account for the strength of the pavement. The threshold for the center deflection was based on the trigger value for the effective structural number of 6. The effective structural number as a function of the center deflection can be seen in Figure 13.

The pavements with an effective structural number of 6 have an average center deflection of 7.7 mils, and a standard deviation of deflections of 0.8 mils. Thus, a threshold center deflection value of 6.5 mils, representative of approximately the lower 95th percentile of deflection values for an SN of 6, was chosen for bituminous interstate pavements.

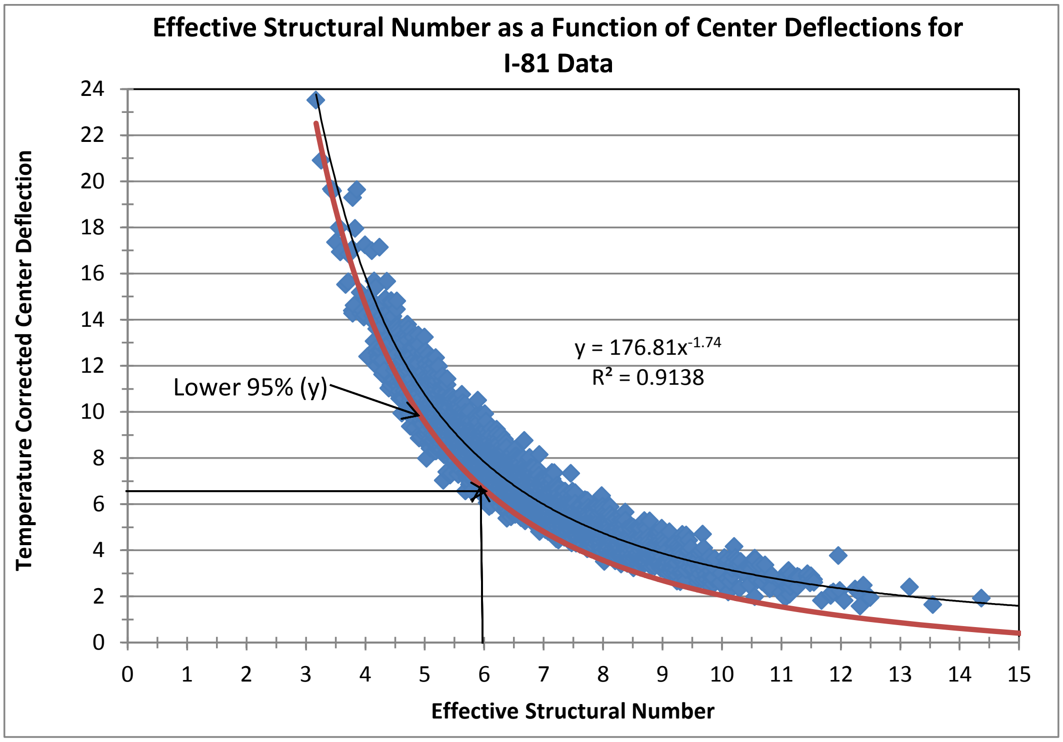

Figure 13. Trend of SN_{eff} as a Function of D_0

3.4.3. Thresholds for the Weighted Differences Method

Developing thresholds for the weighted differences method was problematic because of the lack of a clear relationship between the index and the resilient modulus or the effective structural number (Figure 14). This is because the weighted difference index includes the center deflection in three of the four deflection basin parameters, including the parameter that represents the condition of the subgrade. Also, the resilient modulus parameter is an inverse function of the center deflection. As the center deflection increases, the resilient modulus parameter decreases. However, the index may increase or decrease with increasing center deflections depending on the range of the center deflections, and the overall shape of the deflection basin. Therefore, the thresholds for the weighted differences method will be explored further in specific case studies.

Figure 14. Weighted Differences Index as compared to SN_{eff} and M_R

Interstate 81 Northbound in Pulaski County

The first section that was analyzed was 3 lane-miles of pavement along Interstate 81 Northbound in Pulaski County, Virginia. The work order obtained for this pavement section was put out on November 12, 2008. The condition data used was the 2008 data with the condition testing conducted on December 6, 2007. The deflection testing was conducted on this section on March 6, 2007. This section of pavement received a 1½-inches mill and placement of stone matrix asphalt (SMA-12.5(76-22)) along its entirety, along with an additional 0.86 lane-miles of 6 inches mill and replacement with base mix asphalt (BM-25.0A). According to the VDOT decision process, the 1½-inches mill and overlay would be considered corrective maintenance (CM), and the 6-inch mill and overlay would either be considered restorative maintenance (RM) or rehabilitation/reconstruction (RC) depending on the application. For the purposes of this example, the 6-inches mill and overlay will be considered RM/RC. It is also important to note that specific locations of the work types are not given, thus only the total length of each maintenance action can be compared.

The treatment was conducted between county relative mileposts 10.31 and 11.81. The critical condition index trend along the pavement index can be seen in Figure 15. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 16. The pavement section is defined as structurally weak according to the VDOT criteria ($SN<6$ or $M_R<10{,}000$) for approximately 0.8 lane miles.

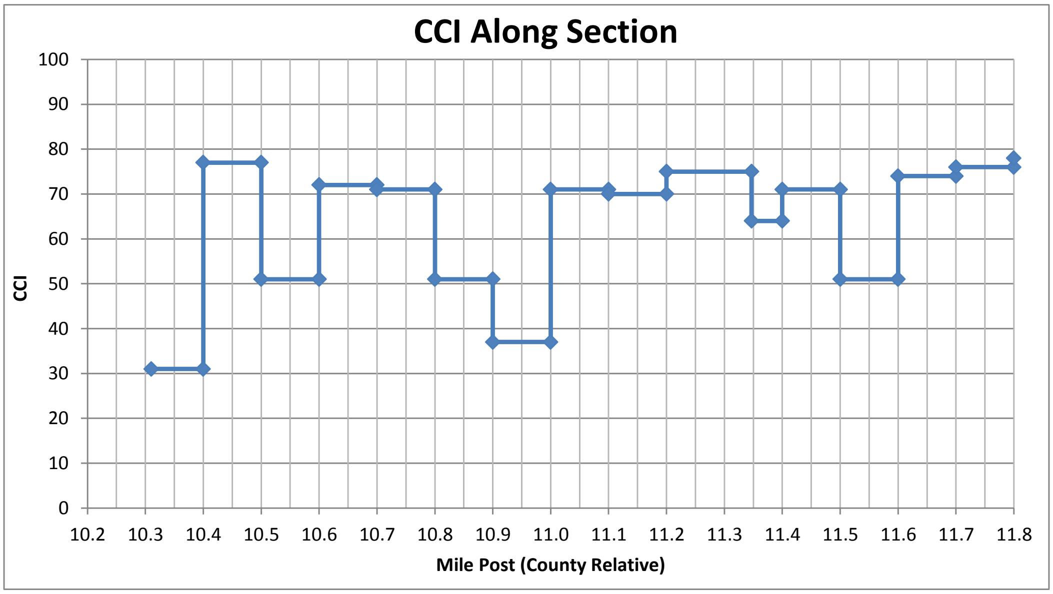

Figure 15. CCI along Pavement Section: Interstate 81 Northbound in Pulaski County

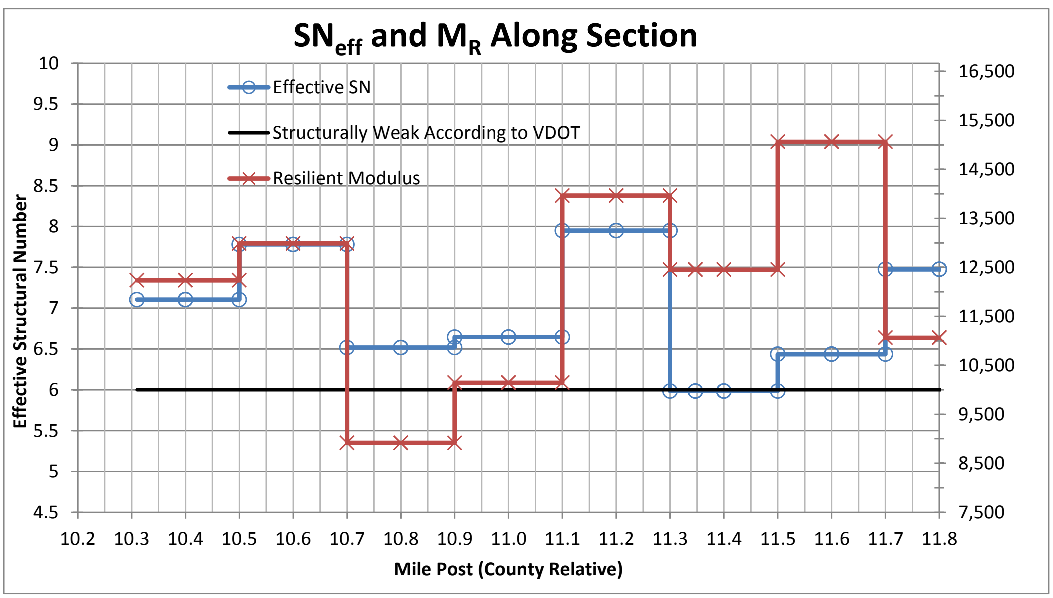

Figure 16. Structure along Pavement Section: Interstate 81 Northbound in Pulaski County

The first step in analyzing this section was to get the distresses in terms of the measures used in the VDOT decision matrices. Each level of distress was then analyzed differently, and the worst case was chosen. For instance, if the pavement section had 150 non-severe transverse cracks per mile and 50 severe transverse cracks per mile, both cases were analyzed and the treatment to fix the worst case was chosen. The method for translating each distress into values that can be read from the matrices is as follows:

Rutting

Rutting was analyzed directly using the values given from the distress data. The frequency of the rutting was assumed to be greater than 10% in each case. This assumption was based on the fact that each pavement section had significant recorded rutting consistently before and after each site.

Alligator Cracking

Alligator cracking was given in total square footage of cracked area, and was required to be translated into the percent of alligator cracking in the wheel path for each severity. The assumption was made that the alligator cracking that was reported was in the wheel path. A 2½ft wheel path was assumed per the LTPP Distress Identification Manual (Federal Highway Administration 2003). Occasional Alligator Cracking is considered less than 10% of area, where greater than 10% is defined as frequent. The following formula was used to get the alligator cracking into percent area:

$$\%AREA\_Alligatior = \frac{AlligCrack\ (in\ square\ feet)}{5'*segment\ length\ in\ feet} \tag{34}$$

Patching Area

Patching area was given in both the patching area in the wheel path, and the patching area not in the wheel path. The decision matrix has inputs of percentage of area patched such that: P0 is no patching, P1 is up to 10%, and P2 is beyond 10% patched area. In order to categorize the patching, the following equation was used:

$$\%Area\_Patched = Max\left\{\frac{Area\ Patch\_Wheel\ Path}{5'*segment\ length\ (ft)}, \frac{Area\ Patch\ Non\ Wheel\ Path}{(LaneWidth-5')*segment\ length\ (ft)}\right\} \tag{35}$$

Transverse Cracking

The transverse cracking is reported in linear feet, and has an input into the decision matrix as linear cracks per mile. It was assumed the majority of the transverse cracks are the full width of the lane. It is stated in the VDOT distress manual that the cracks should be reported in half width and full width, however, the distress data was only given in total crack length for each crack level. To convert the given measure to cracks per mile, the following was used:

$$Transverse\ Cracks\ Per\ Mile = \frac{Transverse\ Crack\ Length}{Lane\ Width\ (ft)\quad *segment\ length\ (miles)} \tag{36}$$

Decision Process

After converting the distress data into inputs for the decision matrices, the decision process was followed. The input data can be seen in Table A.1 in appendix A. The sections from mileposts 10.31 to 10.4, and 10.9 to 11.0 are recommended to be at least reconstructed due to a very low CCI. The sections from mileposts 10.5 to 10.6, 10.8 to 10.9, and 11.5 to 11.6 are recommended to be at least corrective maintenance due to their low CCI values. The decisions made based on the condition data are given in Table A.2 in Appendix A.

The final decision column in Table A.2 in Appendix A is the result of processing the decision from the matrix through the enhanced decision tree. A similar process was followed using the initial decisions, and applying the Modified SCI, center deflection (SSI), and the weighted differences method. The thresholds were applied similarly to the enhanced decision trees, and the results can be seen in Table A.3 in Appendix A. For the SCI index, the level one, level two and level three corresponded to a Modified SCI of less than 0.91, between 0.91 and 1.08, and greater than 1.08 respectively. For the center deflection, a weak structure was considered a deflection greater than 6.5, and then the traffic levels were analyzed to determine the level in the decision tree. A decision based on the weighted differences index was not made based on the lack of thresholds. The weighted index method was analyzed based on the comparison of the values to the decisions from the other index.

One item to note is that the Weighted Difference Index does not differentiate between the sections from 10.31 to 10.50 and 11.50 to 11.70. The structural number and resilient modulus for each of these sections are 7.10 and 12,235, and 6.43 and 15,063 for the sections beginning at 10.31 and 11.50, respectively. This seems to indicate that the weighted difference index is not sensitive enough to differentiate in sections with weaker structure versus sections with weaker subgrade. The inability of the weighted differences index to differentiate the strength of the subgrade and the strength of the pavement is because the method incorporates a linear relationship between the two such that a strong subgrade is equivalent to a strong structure. Where it is true in the design process that a weaker subgrade requires a pavement with a higher structural number, a pavement that has been in service will exhibit deterioration in terms of the structural number, with little to no change in resilient modulus due to trafficking. Thus, it is no longer adequate to say that the strength of the subgrade and the structural number of a pavement can be independently evaluated, but instead the relationship between the two must be considered. Based on this, the weighted differences index will no longer be considered in this research.

The final decisions that were made based on the VDOT decision process, Modified SCI and center deflections (SSI) are shown in Table A.4 in appendix A. The VDOT decision process and the Decision Based on the FWD center deflection yielded 1.82 lane-miles of CM, 0.8 lane-miles of RM and 0.38 lane-miles of RC. The decision based on Modified SCI yielded 2.02 lane-miles of CM, 0.6 lane-miles of RM and 0.38 lane-miles of RC. Recall that the actual project work included 2.14 lane-miles of CM and 0.86 lane miles of work considered RM or RC. The Modified SCI method most closely predicted the actual work completed.

Interstate 81 Southbound in Botetourt County

A similar process was undertaken for a 3.44 lane-mile pavement section along Interstate 81 southbound in Botetourt County, Virginia as was completed for the pavement section in Pulaski County. The work order obtained for this pavement section was put out on November 12, 2008. The condition data used was the 2008 dataset with condition testing conducted on December 4, 2007. The deflection testing was conducted on this section on May 15, 2007. This section of pavement received a 1½-inch mill and placement of SMA-12.5(76-22) along its entirety, along with an additional 0.58 lane-miles of 6-inch mill and placement of BM-25.0A. According to the VDOT decision process, the 1½-inch mill and overlay would be considered corrective maintenance (CM), and the 6-inch mill and overlay would either be considered restorative maintenance (RM) or rehabilitation/reconstruction (RC) depending on the application. For the purposes of this example, the 6-inch mill and overlay will be considered RM/RC.

The treatment was conducted between county relative mileposts 16.32 and 14.60. The critical condition index trend along the pavement index can be seen in Figure 17. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 18. The pavement section is defined as structurally weak according to the VDOT criteria ($SN<6$ or $M_R<10{,}000$) for approximately 1.24 lane-miles.

Decision Process

After converting the distress data into inputs for the decision matrices, the decision process was followed. The input data can be seen in Table B.1 in Appendix B. The section from mileposts 16.0 to 16.1 is suggested to be at least corrective maintenance due to the low CCI values. The sections from mileposts 15.4 to 15.6 and 16.2 to 16.32 should be at most preventative maintenance due to the high CCI values. The decisions made based on the condition data are given in Table B.2 in Appendix B.

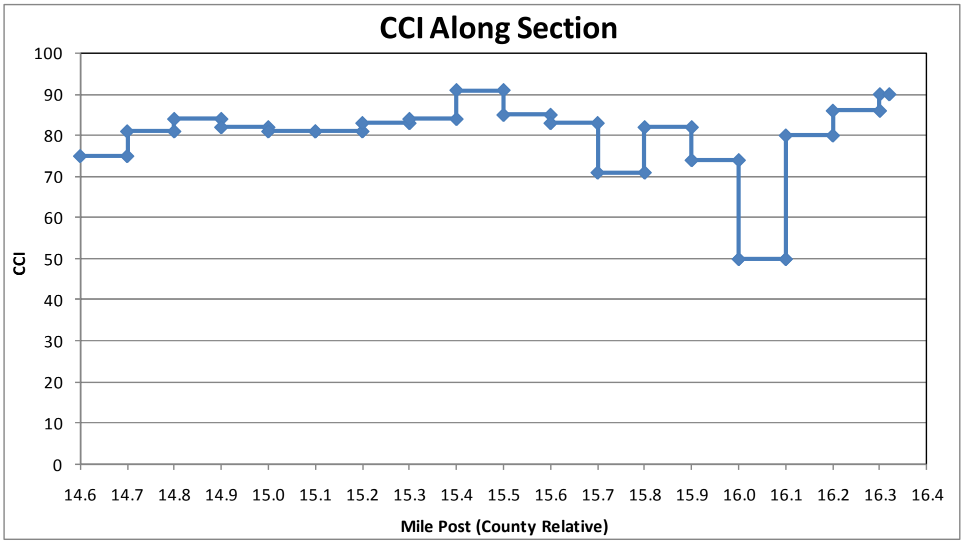

Figure 17. CCI along Pavement Section: Interstate 81 Southbound in Botetourt County

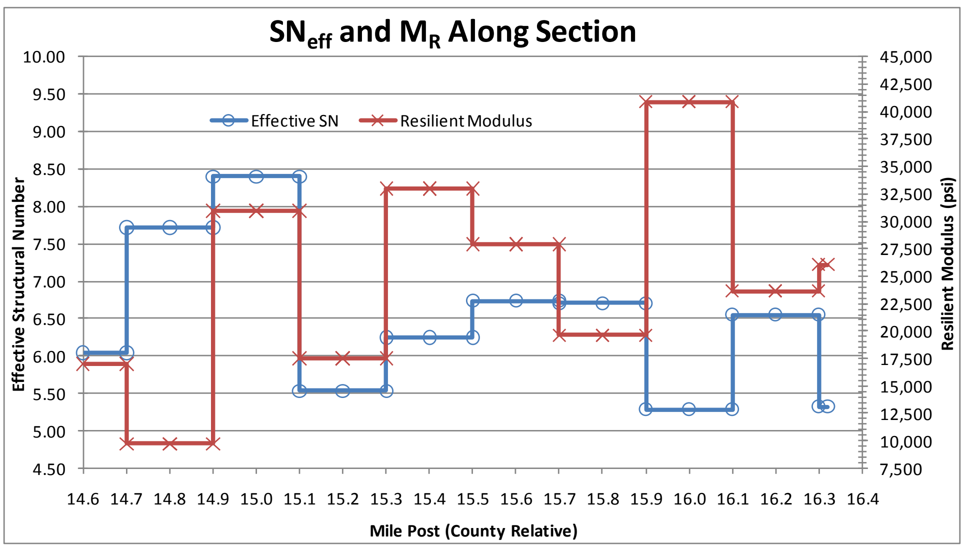

Figure 18. Structure along Pavement Section: Interstate 81 Southbound in Botetourt County

The final decision column in Table B.2 in Appendix B was the result of processing the decision from the matrix through the enhanced decision tree. The process was followed using the initial decisions, and applying the Modified SCI, center deflection (SSI) indices (Table B.3 in Appendix B). The final decisions that were made based on the VDOT decision process, Modified SCI and center deflections (SSI) are shown in Table B.4 in Appendix B. The VDOT decision process yielded 1.8 lane-miles of CM, 0.4 lane-miles of PM and 1.24 lane-miles of RM. The decision based on the FWD center deflection yielded 2.0 lane-miles of CM, 0.4 lane-miles of PM or DN and 1.04 lane-miles of RM. The decision based on Modified SCI yielded 2.4 lane-miles of CM, 0.4 lane-miles of PM or DN and 0.64 lane-miles of

RM. Recall that the actual project work included 2.86 lane-miles of CM and 0.58 lane-miles of work considered RM or RC. The Modified SCI method most closely predicted the actual work completed.

Interstate 81 Southbound in Montgomery County

The process was repeated for a 9.18 lane-mile pavement section along Interstate 81 southbound in Montgomery County Virginia as was completed for the pavement sections in Pulaski and Botetourt Counties. The work order obtained for this pavement section was put out on March 5, 2008. The condition data used was the 2007 dataset due to the fact that the work order was put out early in 2008 with condition testing conducted on January 3, 2007. The deflection testing was conducted on this section on March 28, 2007. This section of pavement received a 2-inch mill and placement of SMA-12.5(76-22) along its entirety, along with an additional 5.46 lane-miles of 6-inch mill and placement of BM-25.0A. According to the VDOT decision process, the 2-inch mill and overlay would be considered corrective maintenance (CM), and the 6-inch mill and overlay would either be considered restorative maintenance (RM) or rehabilitation/reconstruction (RC) depending on the application. For the purposes of this example, the 6-inch mill and overlay will be considered RM/RC.

The treatment was conducted between county relative mileposts 5.07 and 9.66. The critical condition index trend along the pavement index can be seen in Figure 19. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 20. The pavement section is structurally weak according to the VDOT criteria (SN<6 or M_R<10,000) for approximately 3.18 lane-miles.

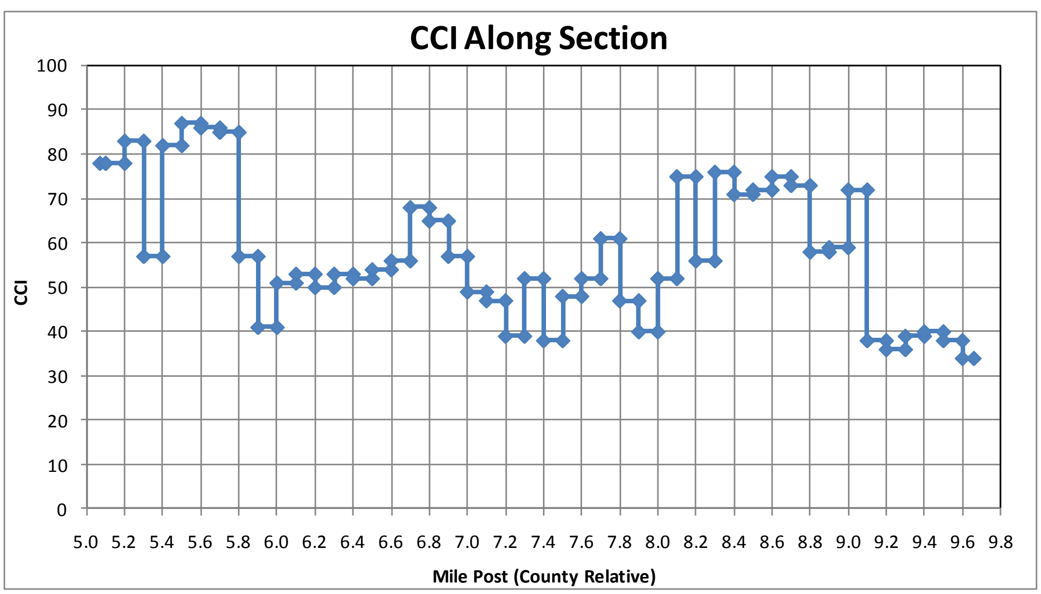

Figure 19. CCI along Pavement Section: Interstate 81 Southbound in Montgomery County

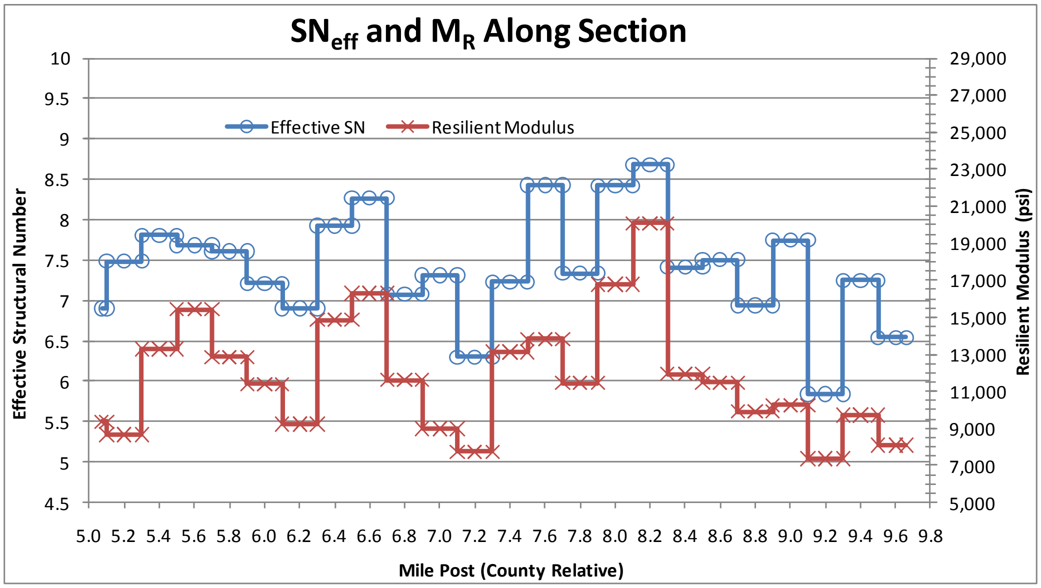

Figure 20. Structure along Pavement Section: Interstate 81 Southbound in Montgomery County

Decision Process

After converting the distress data into inputs for the decision matrices, the decision process was followed. The input data can be seen in Table C.1 in Appendix C. The decisions made based on the condition data are given in Table 22. The final decision column in Table C.2 in Appendix C was the result of processing the decision from the matrix through the enhanced decision tree. The process was followed using the initial decisions, and applying the Modified SCI, center deflection (SSI) indices (Table C.3, Appendix C).

The VDOT decision process yielded 4.4 lane-miles of CM, 0.6 lane-miles of PM, 3.06 lane-miles of RM and 1.12 lane-miles of RC. The decision based on the FWD center deflection yielded 5.2 lane-miles of CM, 0.6 lane-miles of PM or DN, 2.66 lane-miles of RM and 0.72 lane-miles of RC. The decision based on Modified SCI yielded 5.2 lane-miles of CM, 0.6 lane-miles of PM or DN, 2.26 lane-miles of RM and 1.12 lane-miles of RC. Recall that the actual project work included 6.45 lane-miles of CM and 2.73 lane-miles of work considered RM or RC. The methods based on FWD center deflection and Modified SCI both closely predicted the actual work completed.

3.4.4. **Interpretation of the Results**

In two of the three cases presented, the methodology implementing the Modified SCI at the network-level proved to most closely predict the project-level work. In the third case, the threshold based on the center deflection matched the Modified SCI to most closely represent the project-level work. In no case did the predicted work at the network-level exactly match the work done at the project-level. Overall, the Modified SCI procedure most closely predicted the work done (Table 11).

Given the results from the previous case studies, as well as the ease of developing thresholds for different traffic levels, the Modified SCI methodology is the index that should be implemented to interpret network-level deflection data for flexible pavements. The SSI methodology does not provide adequate discrimination between traffic levels for accurate evaluation. Furthermore, the Modified SCI methodology can be easily programmed into spreadsheet format to provide a quick and relatively simple method for obtaining an index.

Table 11. Comparison of Predicted Work for all Sites (Lane-Miles)

<table>
<tr><th></th><th>Project-Level Work Done</th><th>VDOT Enhanced Decision Tree</th><th>Center Deflection</th><th>Modified SCI</th></tr>
<tr><td>DN</td><td>-</td><td>-</td><td>0.2</td><td>0.2</td></tr>
<tr><td>PM</td><td>-</td><td>1</td><td>1</td><td>1</td></tr>
<tr><td>CM</td><td>11.45</td><td>8.02</td><td>9.02</td><td>9.62</td></tr>
<tr><td>RM</td><td rowspan="2">4.17</td><td>5.1</td><td>4.5</td><td>3.5</td></tr>
<tr><td>RC</td><td>1.5</td><td>1.1</td><td>1.5</td></tr>
</table>

Chapter 4. Analysis of the Chosen Structural Index

This chapter presents the results from a detailed study of the structural index, including a sensitivity analysis to define the most critical inputs to the function, threshold analysis and variants of the function for use with primary and secondary facilities. The term Modified SCI, as the structural index was called in Chapter 3 of this thesis, was renamed due to its similarity to another term in deflection testing, the Surface Curvature Index (SCI_{300}). Thus, from this point on, the structural index will be called the Modified Structural Index (MSI).

4.1. Sensitivity Analysis of the MSI Function

The first analysis that was run on the MSI was to assess the sensitivity of the function to the various input parameters. Each of the inputs was varied to simulate any error that may arise during testing, and the behavior of the function was monitored. Two initial conditions were chosen for the sensitivity analysis, a case that yields a MSI of 1.02, and a case that yields a MSI of 1.51 using typical values found along Interstate 81 southbound in Virginia. Each of the deflection values was varied by 10% to represent error in the geophone measurements, the pavement thickness was varied by 10% to simulate uncertainty in the known thickness of the pavement, and the traffic was varied by 50% to simulate the range of traffic expected over similar routes. The results for the case of the initial MSI of 1.02 are presented in Figure 21.

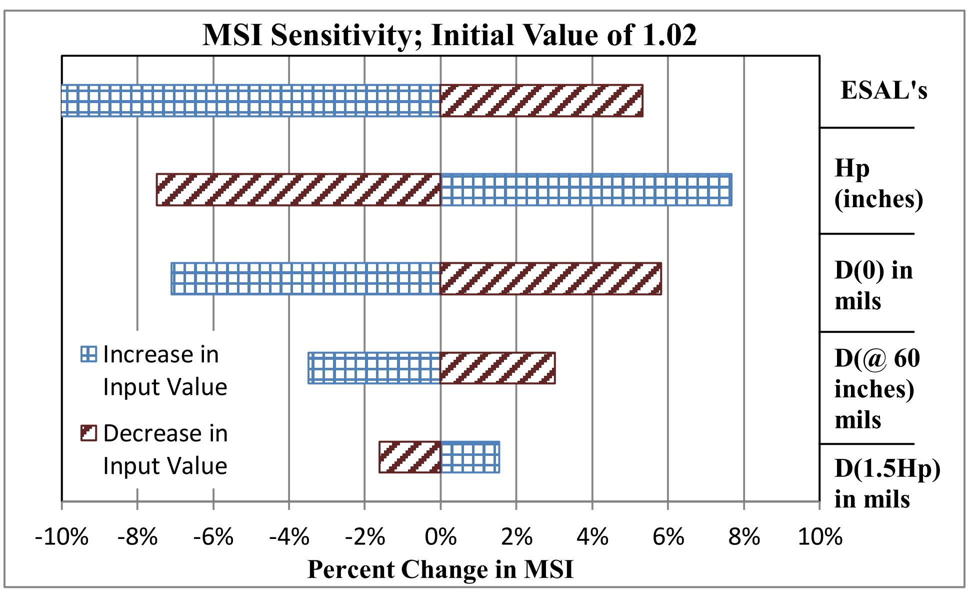

Figure 21. Sensitivity Analysis of the MSI function for Interstates

One important aspect to note is the influence of the traffic level on the MSI value. This is important because it infers that splitting traffic into discrete bins, as was done with the SCI function that is used by the Texas Department of Transportation, does not account for the wide range of structural requirements that could be present in each bin. Furthermore, the traffic is a part of the load that is placed on the pavement, and is the only input value that is not a physical property of the pavement. Therefore, a manager should take into account the possible variations in traffic. Furthermore, variation in estimated traffic can fit into a model of reliability engineering where stochastic modeling can be used to determine uncertainties in the structural needs.

A second finding was that the percent change in MSI value for each case is the same. Thus, varying the input parameters for a pavement with a lower initial MSI has less of an effect than varying the input parameters for a pavement with a higher initial MSI. This means that as the MSI value becomes more critical, small errors have less of an effect on the reading. It can be inferred from the equation of the MSI that the deflection at 1.5 times the pavement depth has as much influence on the value of the MSI as the center deflection, but since the deflection at 1.5 times the pavement depth is much smaller than the center deflection, a 10% change in its value does not have as great an effect on the MSI as seen in Figure 21.

4.1.1. **Sensitivity of the Threshold Values**

When developing the thresholds to compare the project data to the network data in Chapter 3 of this thesis, the thresholds were chosen to represent values that approximately matched the VDOT trigger values from the enhanced decision tree. It was assumed that the trigger values presented by VDOT were the most appropriate values at the network-level. However, the definition of the general form of the MSI as the effective structural number divided by the required structural number implies that the threshold for the MSI should be 1. To investigate this further, a sensitivity analysis was run to assess the validity of the trigger values that were obtained using the VDOT values, and then a second sensitivity analysis was run to determine if a threshold could be determined independently of the VDOT trigger values.

Sensitivity Analysis of Threshold Using VDOT Trigger Values

Project-level work was chosen for the I-81 pavement sections from the three projects introduced in Chapter 3 of this thesis by using both the VDOT methodology and the MSI method. Initially, the threshold for the MSI was based on the same trigger values for the VDOT enhanced decision tree and any differences were expected only because the MSI combines the resilient modulus and structural number into one index as opposed to being treated separately by the decision tree. Two thresholds were found to parallel the three different discrete traffic classifications used in the VDOT decision tree. The thresholds were analyzed by varying their values, and comparing the differences between the total project lengths of certain treatments (i.e., CM or RM) obtained from VDOT decision process and those obtained via the MSI. The results can be seen in Figure 22.

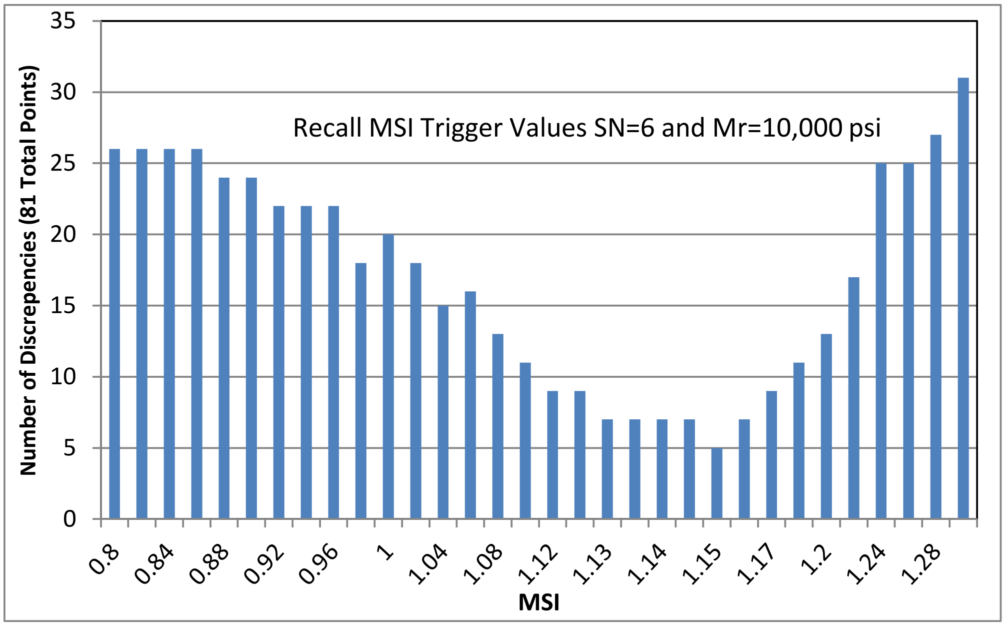

Figure 22. Sensitivity Analysis of Threshold Using VDOT Trigger Values

One important note is that the optimal trigger value for the MSI occurs at a higher level than is indicated when using the VDOT trigger values (1.15 versus 1.08). This may be a result of the fact that the structural number of the pavement and the resilient modulus of the pavement are not independent of each other. In pavement design, a weak resilient modulus requires a higher structural number than a stronger resilient modulus. For instance, one pavement location has a resilient modulus value of 9,712 psi, a structural number of 7.72 and an MSI of 1.12. The resilient modulus value forces it into the weak category, but the MSI value is well above the threshold that would be indicated by using the VDOT trigger values (1.08). This discrepancy can be explained because the structural number of the pavement is high enough to compensate for the weaker subgrade, thus the pavement should not be classified as weak.

Sensitivity Analysis of Threshold Using Only Project Data

An analysis was also conducted to determine the appropriate threshold independent of using the VDOT trigger values. The initial decisions were obtained through using the VDOT decision matrix, and the decision tree was not used for the structural aspect of the pavement sections. Then the threshold for the MSI was varied and the discrepancy between the actual project lengths that fell within each category and the decision that would be reached using the MSI was minimized. The results can be seen in Figure 23.

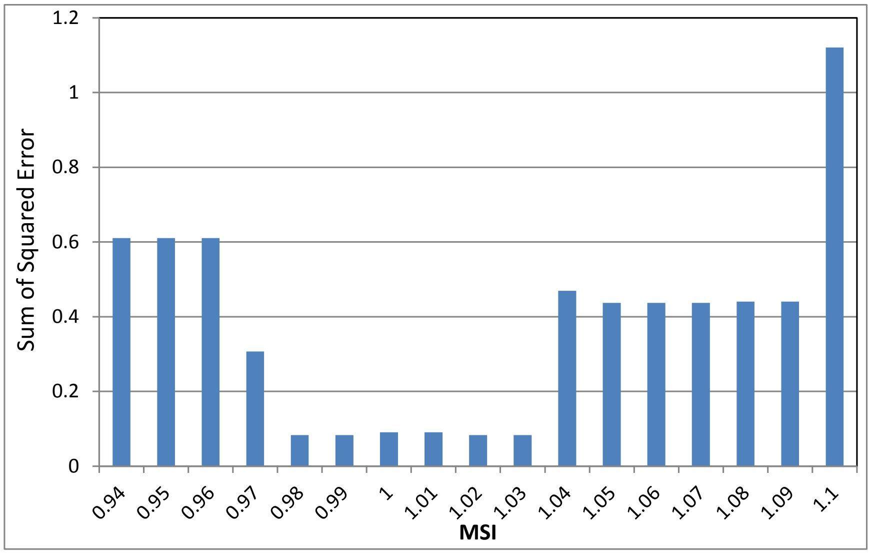

Figure 23. Sum of Squared Error between Total Category Length Predicted by the MSI and the Total Category Length from Project Data – 20 Year Design Period

It can be seen in Figure 23 that the minimum error between the predicted work and actual work falls between an MSI value of 0.98 and 1.03. The errors between the MSI values of 0.98 and 1.03 transitions from positive to negative with similar absolute values. Figure 24 shows the sum of the errors at a closer interval. The total error is minimized for MSI values between 1 and 1.015. It should be concluded then that the optimum trigger value (or threshold) for the MSI should be taken as 1. This confirms the earlier statement that given the general form of the MSI, the effective structural number divided by the required structural number, the implication should be a threshold value of 1.

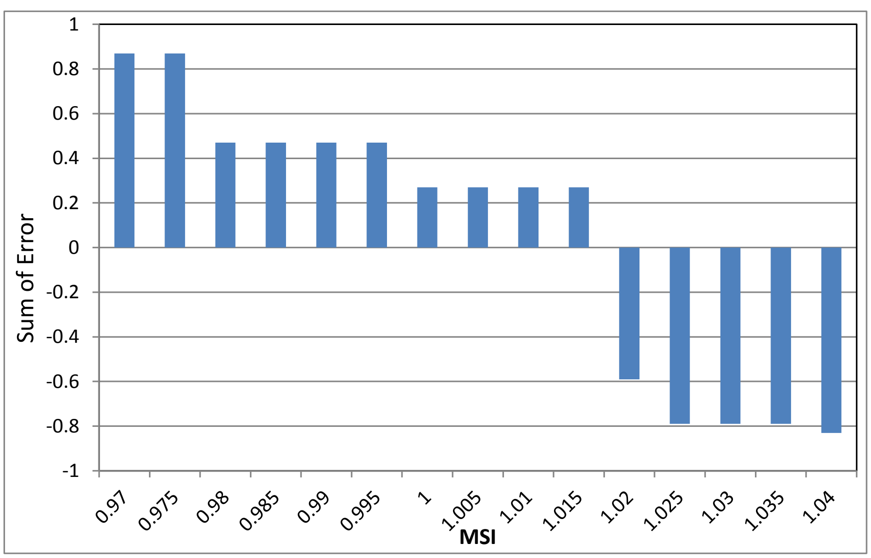

Figure 24. Sum of Errors between Total Category Length Predicted by the MSI and the Total Category Length from Project Data – 20 Year Design Period

Sensitivity of Thresholds to ESAL Analysis Period

Given that the MSI is dependent on accumulated traffic, the values for the MSI will change with a change in the analysis period. For example, if a pavement has an MSI of 1 when the 20 year traffic is analyzed, the MSI should be much greater than 1 for a 5 year analysis period due to the less traffic loading (a smaller required structural number is needed for lower traffic levels with all other factors the same). Thus, the optimal threshold value that was calculated for the 20 year analysis period in the previous section will not be the optimal threshold for periods other than 20 years.

In order to explore the dependency of the threshold to the analysis period, the function was also evaluated for 5 and 10 years of accumulated traffic. The analysis for the 10 year design period yielded an optimal threshold value of between 1.15 and 1.16 (Figure 25). The increase in the MSI value was on average 0.12 times the MSI value for the 10 year design period. Similarly, the analysis for the 5 year design period yielded an optimal threshold value of between 1.28 and 1.29 (Figure 26). The increase in the MSI value for the 5 year design period was on average 0.22 times the MSI value for the 5 year design period.

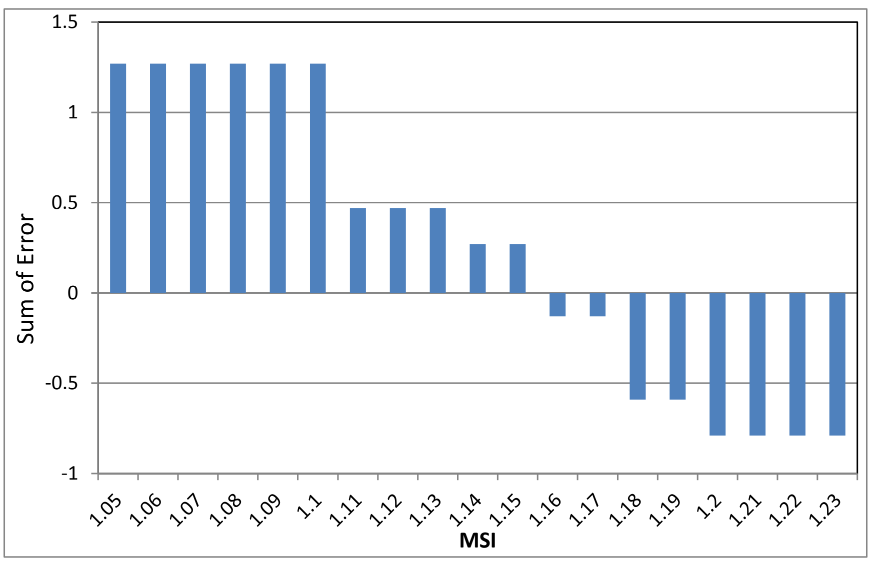

Figure 25. Sum of Errors between Total Category Length Predicted by the MSI and the Total Category Length from Project Data – 10 Year Design Period

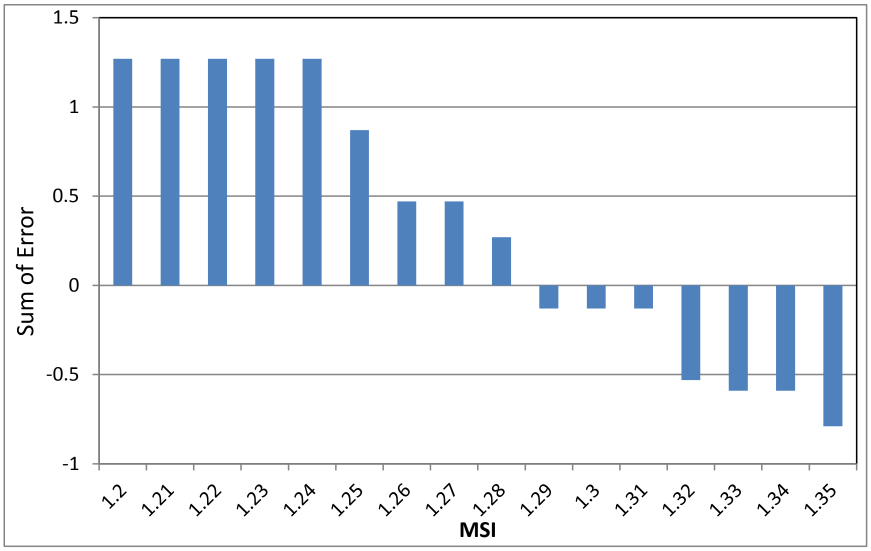

Figure 26. Sum of Errors between Total Category Length Predicted by the MSI and the Total Category Length from Project Data – 5 Year Design Period

4.2. MSI for Different Road Categories

The MSI presented up to this point was based on interstates for the reason that network-level deflection testing in Virginia is only performed on the interstates. However, the concept can be applied to primary or secondary roads with modifications to the denominator of the MSI formula. The main difference will be the assumptions taken when constructing the equation. Recall that to develop the MSI, the following parameters were held steady: PSI of 4.2, the PSI_T of 3, the reliability of 95% and the standard deviation of 0.49. These parameters will change based on recommended values from VDOT.

For most flexible pavement designs VDOT sets the initial value of the serviceability index (PSI) at 4.2, and the standard deviation at 0.49. Farm to market secondary roads and residential roads have a PSI value of 4.0, but this thesis is intended to focus on higher priority routes. Therefore the only changes will be reflected in the reliability and the PSI_t. Table 12 presents the VDOT design values for each case.

Table 12. Pavement Design Values (VDOT, 2003)

	Reliability (%)		Terminal Serviceability Index (PSI_t)
	Urban	Rural	
Interstate	95	95	3.0
Divided Primary Route	90	90	2.9
Un-Divided Primary Route	90	85	2.8
High Volume Secondary Route	90	85	2.8

Recall Equation 28 from Chapter 3 of this thesis, the value of 9.07605 on the left hand side of the equation was obtained by combining the reliability and standard deviation with the other constants from the AASHTO design equation (Equation 27). Therefore, this is the value in the MSI equation (Equation 30) that will be effected by changing the reliability. The serviceability index is on the right hand side of Equation 29, thus the changes in serviceability indices will be reflected in the alpha and gamma factors shown in Equation 29. The procedure to determine the values for alpha and gamma in Equation 29 for alternate routes was to develop a database of SN values from randomly generated traffic and resilient modulus values that are representative of the typical values from Interstate 81 in Virginia. The summary statistics for this database can be found in Appendix D. The equations for the MSI are presented below. The equation that was developed in Chapter 3 of this thesis for use in interstates is also presented for completeness of the list.

MSI for Interstates:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{(0.05716 * (\log(ESAL) - 2.32 * \log(\mathrm{Mr}) + 9.07605)^{2.36777})} \tag{37}$$

MSI for Divided Primary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{(0.060488 * (\log(ESAL) - 2.32 * \log(\mathrm{Mr}) + 8.89818)^{2.32752})} \tag{38}$$

MSI for Un-Divided Primary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{(0.05943 * (\log(ESAL) - 2.32 * \log(\mathrm{Mr}) + 8.89818)^{2.32506})} \tag{39}$$

MSI for High Volume Secondary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{(0.05919 * (\log(ESAL) - 2.32 * \log(\mathrm{Mr}) + 8.77764)^{2.32729})} \tag{40}$$

4.2.1. Comparison of the MSI Equations

In order to assess the extent to which Equations 37 through 40 differ from each other, a database of inputs was developed to simulate conditions that would produce a range of MSI's. The MSI values that were produced were evaluated for a high traffic case (5.00×10^7 ESAL's), and a relatively low traffic case (5.00×10^5 ESAL's). The results can be seen in Figure 27 and Figure 28.

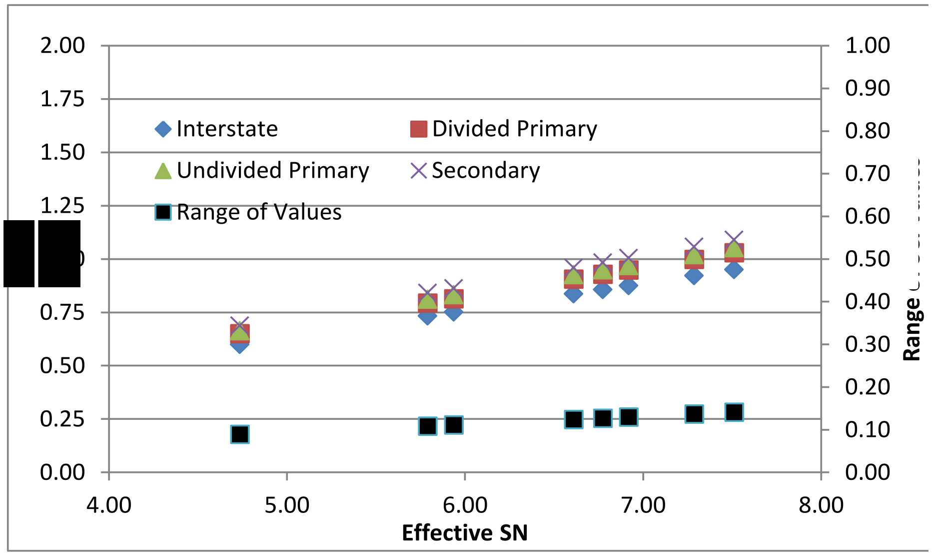

Figure 27. MSI Equation Comparison for $5*10^7$ ESAL and Resilient Modulus of 6,000 psi

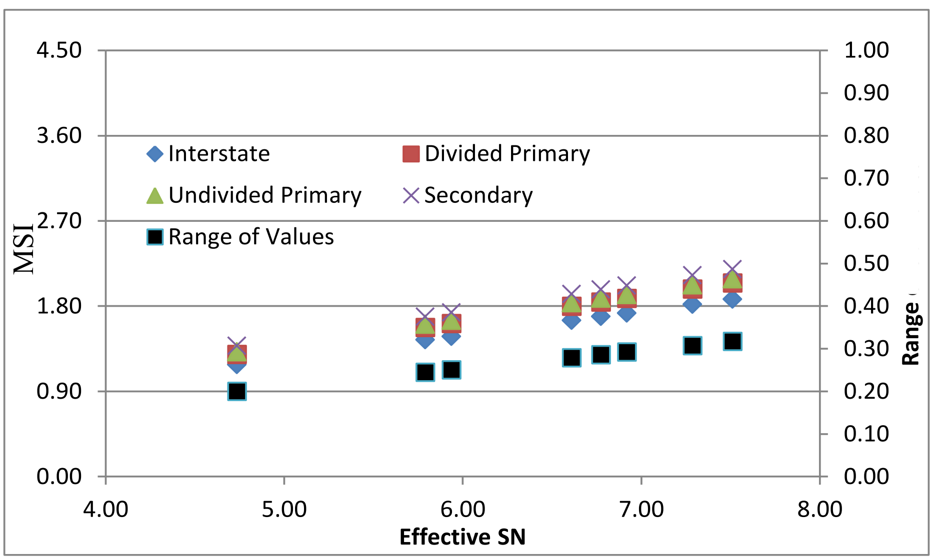

Figure 28. Equation Comparison for $5*10^5$ ESAL and Resilient Modulus of 6,000 psi

The first item to note is that in both cases (low traffic and high traffic) the range of values produced by comparing the four equations increases as the effective structural number increases. However, the range of values is much more pronounced in the low traffic case. This is because the required structural number for the lower traffic case is less than that for the higher traffic case. Thus, the equations are more sensitive to changes in the effective structural number for low traffic cases.

Another issue that arises is whether the different functions can be combined, especially the functions for Divided Primary routes and Un-Divided Primary routes. It can be seen from Figure 27 and Figure 28 that the difference between these two functions is very small. The difference in the MSI equations for Divided Primary routes and Un-Divided Primary routes was investigated further, and the results are presented in Figure 29. It can be seen that the largest difference in the equations occurs for low traffic and high subgrade resilient modulus values. However, the largest difference in the equations for the cases investigated was approximately 0.02. This is equivalent to having an error in the estimate of the subgrade resilient modulus of 12,000 psi ± 400 psi, or 12,000 psi ± 3.3%. When using the FWD with a standard 9,000 lb load and the deflection at 60 inches from the center of the load to calculate the subgrade resilient modulus, 12,000 psi ± 400 psi would equate to a measured deflection of 0.99 mils ± 0.03 mils (or ± 3%). Given that a 3% total error is reasonably small, it was decided that the MSI equations for Divided Primary routes and Un-Divided Primary routes could be combined. Therefore the equation for primary routes becomes:

$$MSI = \frac{0.4728*\left(D_0 - D_{1.5Hp}\right)^{-0.4810} * Hp^{0.7581}}{(0.0600*(\log(ESAL) - 2.32*\log(\mathrm{Mr}) + 8.89818)^{2.32629})} \tag{41}$$

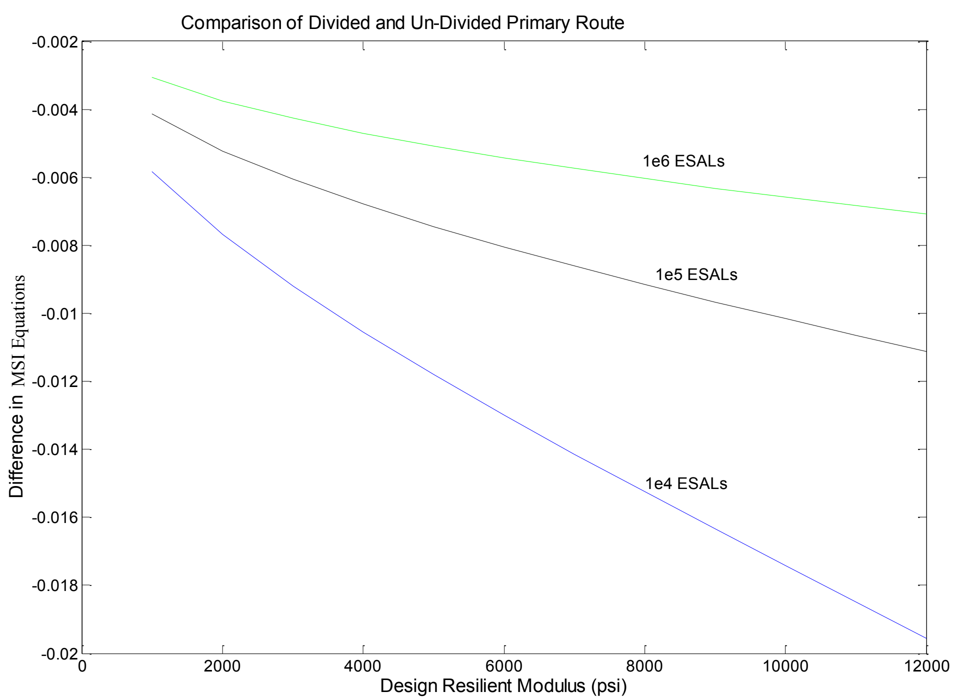

Figure 29. Comparison of Equation 38 and Equation 39

Similar to comparing the MSI functions for the primary routes, a comparison was undertaken to determine whether the MSI equations for primary routes and secondary routes could be combined. The difference in the MSI functions for these route types are shown for a range of traffic values and subgrade resilient moduli in Figure 30.

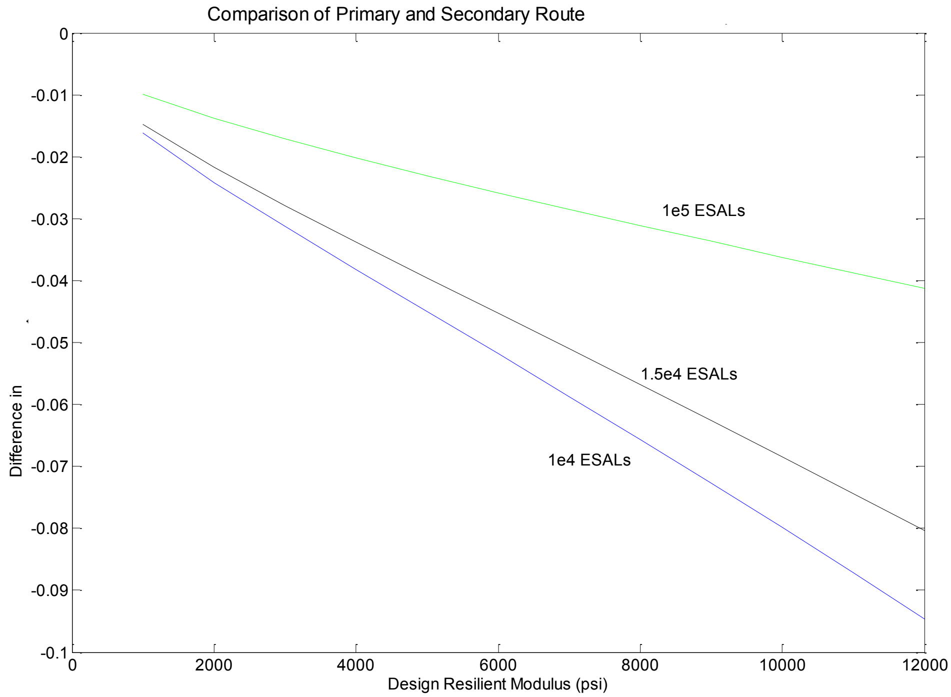

Figure 30. Comparison of Equation 40 and Equation 41

It can be seen in Figure 30 that the largest difference in the equations occurs for low traffic and high subgrade resilient modulus values, and the largest difference in the equations for the cases investigated was approximately 0.1. This is equivalent to having an error in the estimate of the subgrade resilient modulus of 12,000 psi ± 1900 psi, or 12,000 psi ± 16%. When using the FWD with a standard 9,000 lb load and the deflection at 60 inches from the center of the load to calculate the subgrade resilient modulus, 12,000 psi ± 1900 psi would equate to a measured deflection of 0.99 mils, 1.18 mils and 0.85 mils for 12,000 psi, 10,100 psi and 13,900 psi respectively (or + 18% and – 14%).

To further compare the two functions, a number of scenarios were developed to simulate conditions that would trigger a weak MSI value. The MSI function for primary routes will result in lower values than that of the function for secondary routes due to the difference in terminal serviceability indices for each. The first case setup to compare the two functions is a 16 inch deep flexible pavement subject to $1.5*10^5$ ESAL's. The deflection data is presented in Table 13, and was developed to simulate a pavement with a structural number of approximately 3.5 and a MSI value that would trigger a structurally deficient value (MSI<1).

Table 13. Deflection Data for a Primary Route

							MSI	
D0 (mils)	D1.5Hp (mils)	Pavement Depth (in)	ESAL	D60 (mils)	SN_{Eff}	Resilient Modulus (psi)	Primary Routes	Secondary Routes
13.10	5.00	16.00	1.50E+05	2.95	3.47	4,027	0.99	1.07

Each of the inputs were varied for the primary routes to determine the required error in the measurements for the MSI for the primary routes to equal 1.07, as well as the required error in the measurements for MSI for the secondary routes to equal 0.99. Using a standard deviation due to errors of ± 0.08 mils for an appropriately calibrated FWD (Law Engineering, 1993), a 95% confidence interval was applied to the measurements. The deflection measurements were then varied within the confidence interval to determine the range of MSI values that would result from errors within the deflection measurements. The ranges of values for the MSI equations were 0.960 to 1.030 and 1.031 to 1.100 for primary and secondary routes respectively. Given that these ranges do not intersect at the 95% confidence level, these equations should not be combined.

4.3. Final Form of the MSI Equations

The MSI equations were generalized into a single form (Equation 42), and the constants were tabulated (Table 14).

$$MSI = \frac{K1*(D_0-D_{1.5Hp})^{K2}*Hp^{K3}}{(\alpha*(\log(ESAL)-2.32*\log(\mathrm{Mr})+\beta)^{\gamma})} \tag{42}$$

Where D_0 is the FWD center deflection for an equivalent 9000 pound load, $D_{1.5Hp}$ is the deflection at 1.5 times the pavement thickness, Hp is the pavement thickness, ESAL is the calculated traffic, and Mr is calculated as $((0.33*9{,}000*0.24))/(D_{60}*60)$ with D_{60} as the deflection (inches) at 60 inches away from the center of the load.

Table 14. Constants for the MSI Equations

	α	β	γ	K1	K2	K3
Interstates	0.05716	9.07605	2.36777	0.4728	-0.4810	0.7581
Primary	0.06000	8.89818	2.32752	0.4728	-0.4810	0.7581
Secondary	0.05919	8.77764	2.32729	0.4728	-0.4810	0.7581

CHAPTER 5. EXAMPLE APPLICATIONS AND CASE STUDIES USING THE MODIFIED STRUCTURAL INDEX

This chapter presents example applications and case studies using the MSI as a network-level tool. Its use in network-level analysis, as well as its integration into decision making will be presented. The applications will be specific to the pavement management process in Virginia, but can be applied to other state pavement management programs with minor modifications.

5.1. Integration of the MSI into the Project Scoping Decision Process

The current VDOT decision process incorporates deflection information into an enhanced decision tree after a decision is made based on surface distresses. The enhanced decision tree is based on the age of the pavement, the traffic levels, the structural number of the pavement, and the subgrade resilient modulus. Given that the MSI function incorporates traffic levels, the structural number of the pavement, and the resilient modulus of the pavement, a simplified version of the enhanced decision tress can be developed. This decision tree is shown in Figure 31.

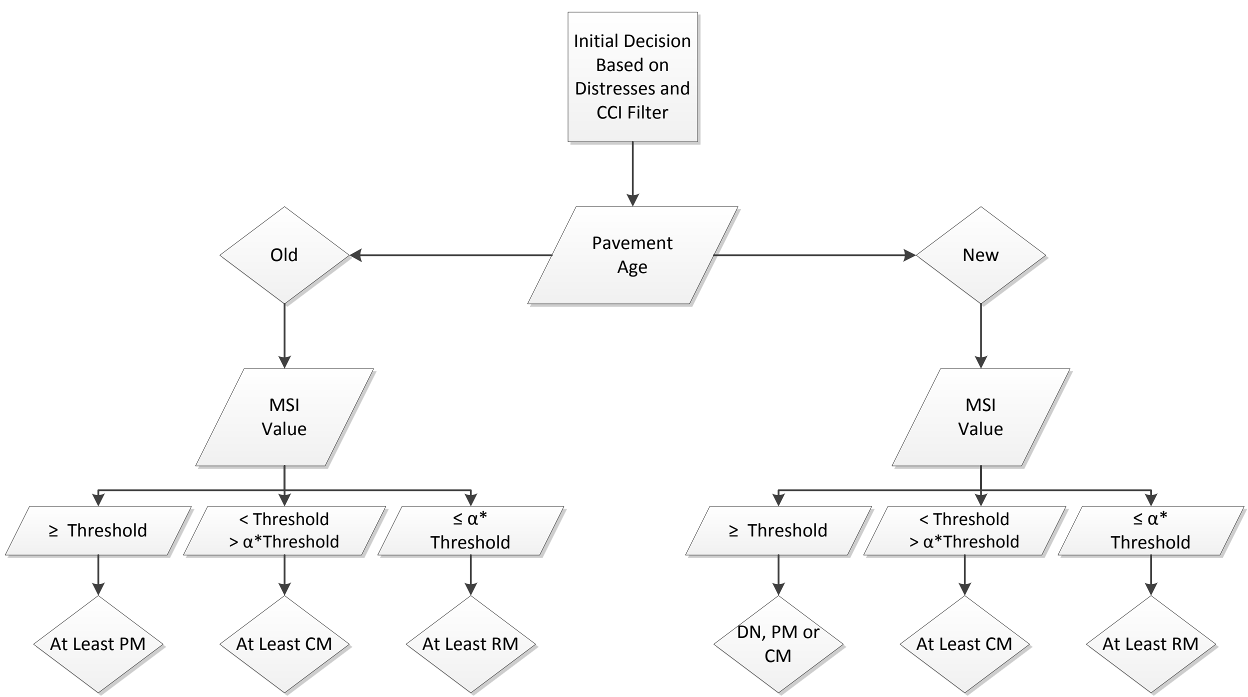

Figure 31. Decision Process Based on MSI for Bituminous Pavements

The pavement age is seen as an important factor to keep in the decision process because older pavements are more likely to require at least some level of preventative or corrective maintenance. The three decisions based on MSI represent three potential levels of structural deficiency. The first case is that the

MSI exceeds the threshold for the pavement, in which case it is structurally adequate. The second case is that the MSI does not meet the threshold, but is in a range of values approaching the threshold. The third case is that the MSI is a considerably less than the threshold (by a factor of α), and is considered structurally deficient. The reason for splitting the MSI decision into two categories for the case that the value is less than the threshold is to account for severely deficient locations. These are locations that will most likely require heavy restorative maintenance or reconstruction due to their structural state.

To determine the value for α, a study was undertaken to determine the MSI values of pavement locations that are known to be structurally deficient. A plot of 325 miles of Interstate 81 southbound can be seen in Figure 32. A particular pavement location between mileposts 213 and milepost 217 on Interstate 81 southbound was recently reconstructed and was considered in very poor structural condition. The MSI along this pavement section was determined to also be the worst case MSI along the 325 mile section of pavements. The distresses along the pavement included cracking that extended through the full pavement depth, extensive rutting, and extensive patching (VDOT, 2011). Thus it was determined that a full reconstruction was needed.

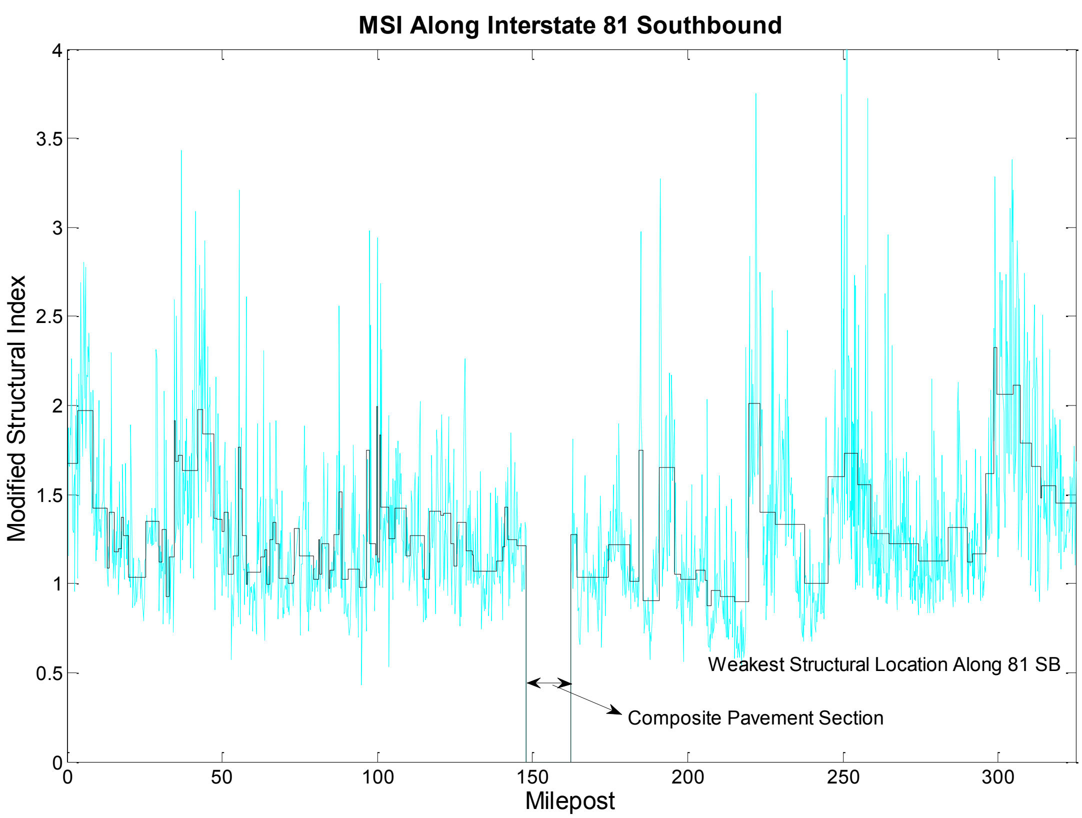

Figure 32. MSI for Interstate 81 SB

The MSI values were calculated using a 20 year analysis period so the threshold value would be 1. The MSI values averaged over the weakest two sections are approximately 0.89. This is the only location along the 325 mile pavement length that has MSI values less than 0.9 (comparing VDOT sectioned pavements from the PMS). Neglecting the 14.5 mile stretch of composite pavement, the percentage of MSI values less than 0.9 for the pavement section is 15%. However, when the MSI is averaged over the VDOT pavement sections, only 2% of the pavement is less than 0.9. Furthermore, the percentage of MSI values less than 1 for the pavement section is 26% when neglecting the 14.5 mile stretch of composite pavement and 10% of the VDOT pavement sections have an average MSI value less than 1. Based on the mean MSI values for the pavement sections, the α value could be chosen as 0.9.

Using the above information, as well as the current VDOT enhanced decision tress, a decision matrix can be developed to account for the MSI in decision making. The decision matrix can be seen in Table 15. The MSI value is normalized against its threshold in order to allow for many different analysis periods to use the same matrix. The normalized MSI is calculated as:

$$Normalized\ MSI = \frac{MSI_{Calculated}}{Threshold} \tag{43}$$

Table 15. Decision Matrix Incorporating the MSI

Initial Decision		DN		PM		CM		RM		RC	
Pavement Surface Age (Years)		≤ 6	> 6	≤ 6	> 6	≤ 6	> 6	≤ 6	> 6	≤ 6	> 6
Normalized MSI	≥ 1	DN	PM	PM	PM	CM	CM	RM	RM	RC	RM
	< 1 and ≥ 0.9	CM	RM	CM	RM	RM	RM	RC	RM	RC	RC
	< 0.9	RM	RM	RM	RM	RC	RC	RC	RC	RC	RC

5.2. MSI as a Structural Screening Tool

A potential application of the MSI is as a network-level structural screening tool. The structural condition of Interstate 81 Northbound in Virginia is shown in Figure 33. Seven potentially critical locations in terms of the MSI were identified and labeled in the figure for further investigation. The dark line in the figure represents the MSI averaged over the VDOT homogenous sections as identified in the VDOT inventory. Some of the VDOT sections did not coincide with deflection testing, and thus do not show up in the Figure 33.

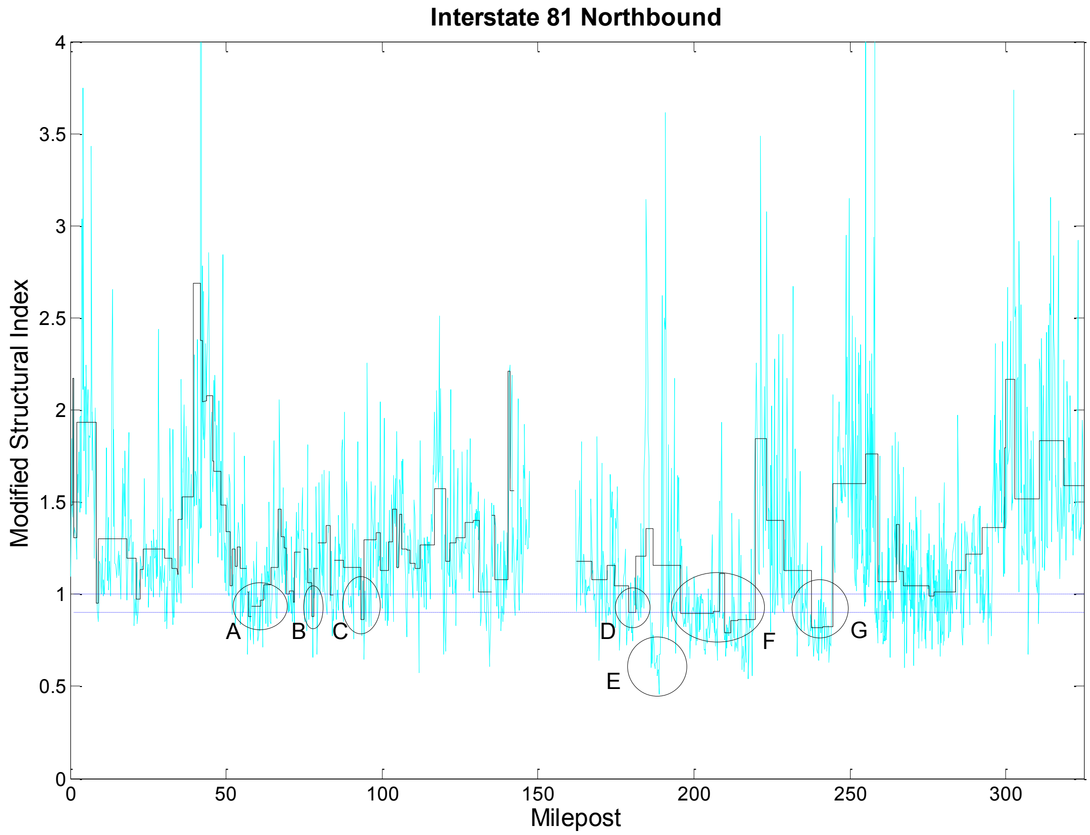

Figure 33. MSI along I-81 Northbound in Virginia.

The first section, labeled A in Figure 33, is between state mile marker 57 and 62, which is a stretch of pavement that spans across Smyth and Wythe counties. The plot of the MSI and 2007 condition data in terms of the CCI can be seen in Figure 34. It is clear from the figure that many of these sections should be candidates for reconstruction because of the very low CCI coinciding with the very low values of MSI. According to the VDOT inventory data, many of these sections received a 1.5 inch overlay in 2008.

To further illustrate the usefulness of the MSI as a structural screening tool to enhance the decision process, the section from milepost 57.66 to milepost 60.84 (the first 3.18 miles into Wythe County) was further evaluated. The inputs into the decision matrix can be seen in Table 16. Using the decision matrix without the enhanced decision tree, or any structural information, the section was a candidate for corrective maintenance. The 1.5 in overlay falls within the category of corrective maintenance. Using the additional process from Table 16 that accounts for structural information, the decision should have been at least restorative maintenance (a 20 year traffic analysis was used). The fact that corrective maintenance was applied when restorative maintenance should have been utilized may explain the relatively rapid deterioration of the condition of the pavement after the treatment was applied (see CCI in Table 16).

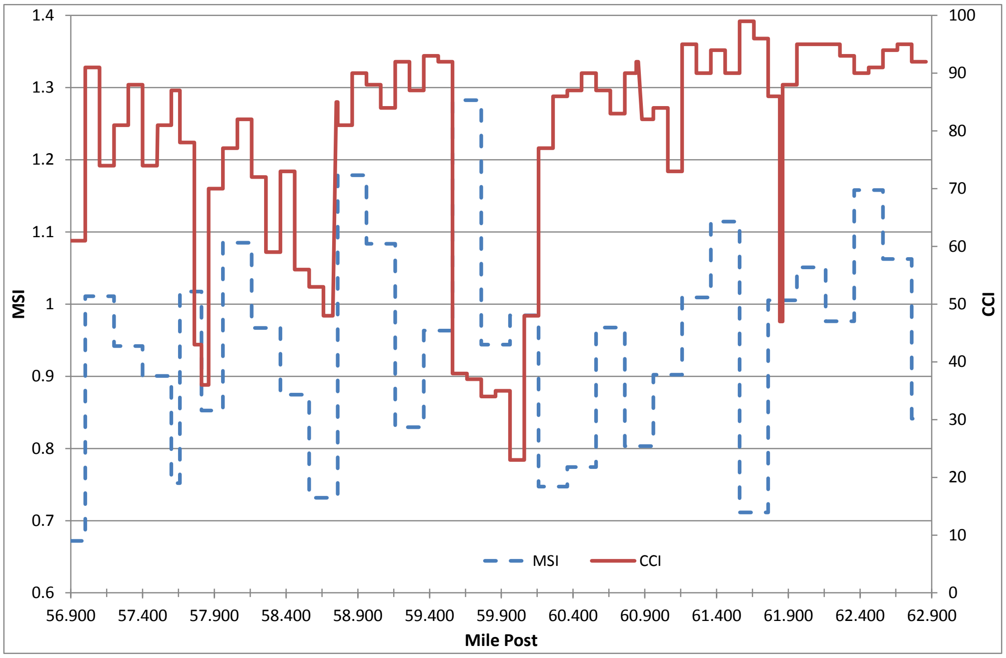

Figure 34. MSI and CCI along a Section of I-81 Northbound

Table 16. Decision Matrix Inputs for the Section of Interstate

		2007	2008	2009	2010	2011
	CCI	63	55	100	88	84
	Traffic	Level 2	Level 2	Level 2	Level 2	Level 2
Alligator Crack Frequency	NS	-	-	0	1	1
	Severe	2	1	-	-	-
	VS	-	-	-	-	-
Rutting Freq (Severity 0)		2	2	2	2	2
Transverse Cracking	NS	0 - 50 / mile	51 - 74 / mile	0 - 50 / mile	0 - 50 / mile	0 - 50 / mile
	Severe	N/A	N/A	N/A	N/A	N/A
	VS	N/A	N/A	N/A	N/A	N/A
	Patching	P1 (Sev)	P1 (Sev)	0	0	P1 (NS)
	Surface Age	Old	Old	New	New	New

Additional sections that should be noted from Figure 33 are the sections labeled B and F, which coincide with pavement sections where Interstate 81 overlaps with Interstate 77 and Interstate 64, respectively. One of the factors that contribute to a low MSI at these locations is increased traffic loading due to the parallel corridors running on the same pavement. Also, note that section E has several MSI values considerably lower than the average for the section. The fact that the MSI identified this section indicates that it may be used to better segment structurally homogenous sections.

5.3. Estimated Overlay Thickness

Given the form of the MSI as the effective structural number divided by the required structural number, it is possible to estimate the required overlay thickness to bring the MSI above a specified value for a given time period. The benefit of this will be to estimate a required cost to maintain a network in adequate structural condition, or to better estimate project costs at the network-level. To estimate the required overlay thickness, Equation 44 can be employed. Equation 44 is based on the equivalent thickness approach used by both the Asphalt Institute, and AASHTO (Huang, 2004). The assumptions built into the equation are that the asphalt has a structural coefficient of 0.44 per inch, the thickness of the overlay must match the thickness of the milled asphalt, and that the milled asphalt results in the removal of a certain amount of structural capacity. Thus, the required overlay thickness is defined as:

$$d = \frac{Threshold * SN_{Req} - SN_{Effec}}{0.44 * (1 - C)} \tag{44}$$

Where d is the required overly in inches, Threshold is the specified value of the MSI to be exceeded (1 should be used for a 20 year analysis), SN_{Req} and SN_{Eff} are the required structural number and effective structural number (respectively), and C is a factor based on the condition of the pavement (Huang 2004).

The C factor represents the percent of contributing structure that remains in the removed layer of asphalt. For example, 0.5 to 0.7 should be used for asphalt concrete pavement that exhibits appreciable cracking (Huang, 2004). It is feasible that the C factor can be derived from the condition surveys of the pavement.

To demonstrate this concept, the required overlay thickness for a 10 year design was calculated for Interstate 81 southbound using the threshold of 1. Recall from Chapter 4 of this thesis that for a 10 year analysis, it was suggested that a threshold value of 1.15 be used for the decision making process. However, the equivalent thickness approach is based on maintaining a structural number only equivalent to the required structural number (thus 1 was used). The analysis resulted in approximately 50 miles of pavement being identified as requiring an overlay, with an average overlay thickness of approximately 2½ inches. The results can be seen in Figure 35. It can be seen in Figure 35 that a large concentration of required overlays occurs between mileposts 200 and 225. Recall that this is the section of the interstate where I-81 and I-64 run along the same pavement as discussed in Section 5.2 of this thesis.

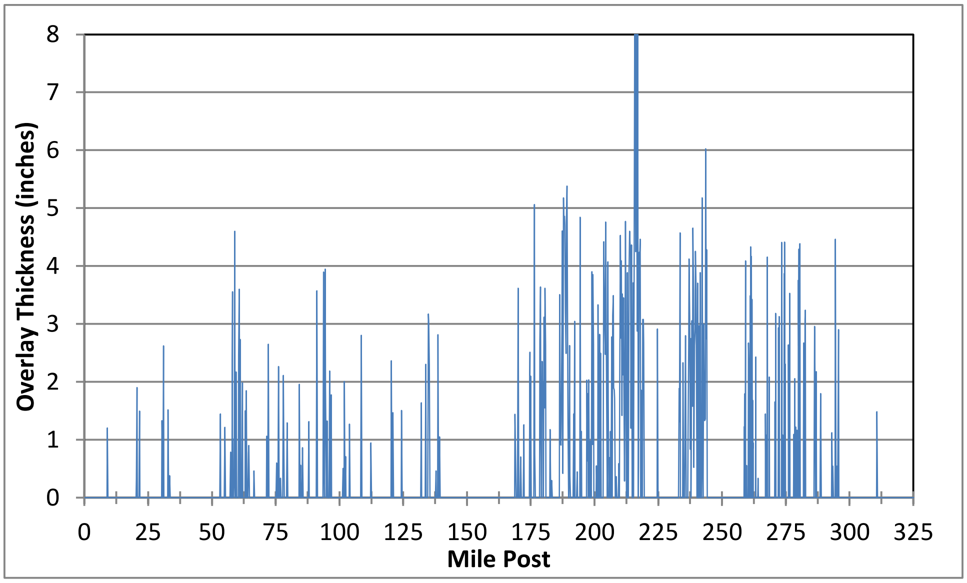

Figure 35. Estimated Overlay Thickness along I-81 Northbound in Virginia

5.4. MSI as a Performance Indicator

Performance indicators are measurements used to gage the condition of an asset in order to compare the current condition to the stated goals for the asset. The FHWA recently published a state of the practice research of performance indicators used throughout Australia, British Columbia, the United Kingdom, and the United States. The definition used for a performance indicator was given as, "…specific milestones in or components of performance measures that serve as precursors to indicate progress toward the eventual achievement of the desired performance measures" (Garvin et al., 2011).

For instance, VDOT uses the following performance measures for highways (VDOT, 2011):

- Performance, in terms of congestion free travel,
- Safety, in terms of deaths accumulated since the beginning of the year,
- Condition, in terms of the quality of the road surface, and
- Finance, in terms of the planned versus actual expenditures.

The condition performance measure is in terms of the percentage of highways in fair or better condition. The MSI can add a dimension to the performance measure by distinguishing between a highway that is in fair or better condition, and a highway that is structurally deficient. The potential benefit to adding the MSI as a performance indicator is the ability to discern between pavements that are in poor structural condition, but are in fair or better functional condition because of recent surface improvements.

5.4.1. **Interstate 81 Performance**

In order to demonstrate the use of the MSI as a performance indicator, the MSI was compared against the condition of Interstate 81 in terms of the CCI. The 2007 condition data was used because it was the condition measured following the deflection testing. Recall that a CCI greater than or equal to 60 is in fair condition and any pavement with a CCI less than this is considered deficient. Using this measure, 12.1% of the pavement is in a deficient condition (Figure 36). Thus, the performance indicator for condition would be 87.9.

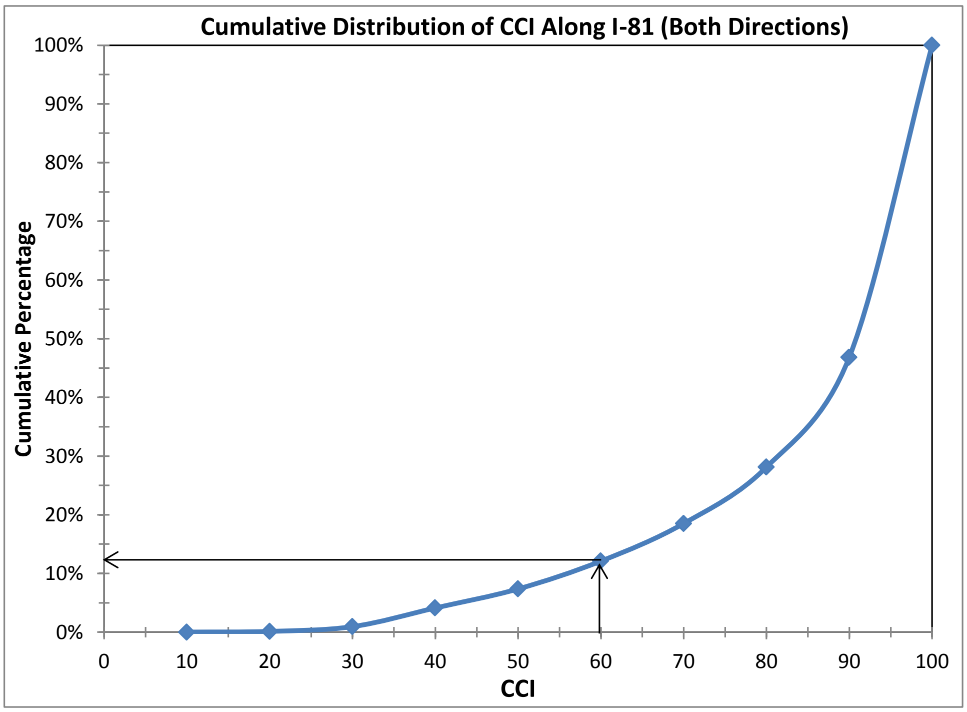

Figure 36. Condition of I-81

Recall that the CCI is a combination of surface distresses (cracking, rutting, etc.) that attempts to describe the overall in-situ condition of the pavement. However, in some cases the CCI for a pavement may be excellent while the structural condition of the pavement is deficient. This is a consequence of some sections receiving light treatment, such as thin overlays or crack sealing, when it is in need of structural rehabilitation. Thus, analyzing the MSI as a performance indicator will provide more information about the cases of structurally deficient pavements.

Depending on an agencies goals, various analysis periods can be chosen, or various thresholds can be chosen. Given that changing the analysis period changes the MSI by a factor in the denominator of the MSI equation, changing the threshold is inversely related to changing the analysis period. This is shown in Figures 37 through Figure 39. For a 20 year design period and an MSI threshold of 1, the performance indicator is shown to be 100% - 27.3% = 72.7% of pavements in structurally adequate condition. Similarly, maintaining a threshold of 1 for 10 year and 5 year analysis periods yields performance indicators of 85.9% and 93.7% respectively.

These can also be obtained by varying the thresholds instead of varying the analysis period. Recall from Chapter 4 of this thesis that for 10 year and 5 year analysis periods optimal threshold values were found to be 1.15 and 1.28 respectively. Thus, approaching Figure 37 using modified thresholds of 1/1.15 = 0.87 for a 10 year analysis and 1/1.28 = 0.78 for a 5 year analysis yields performance indicators values of 85.9% and 93.7% respectively.

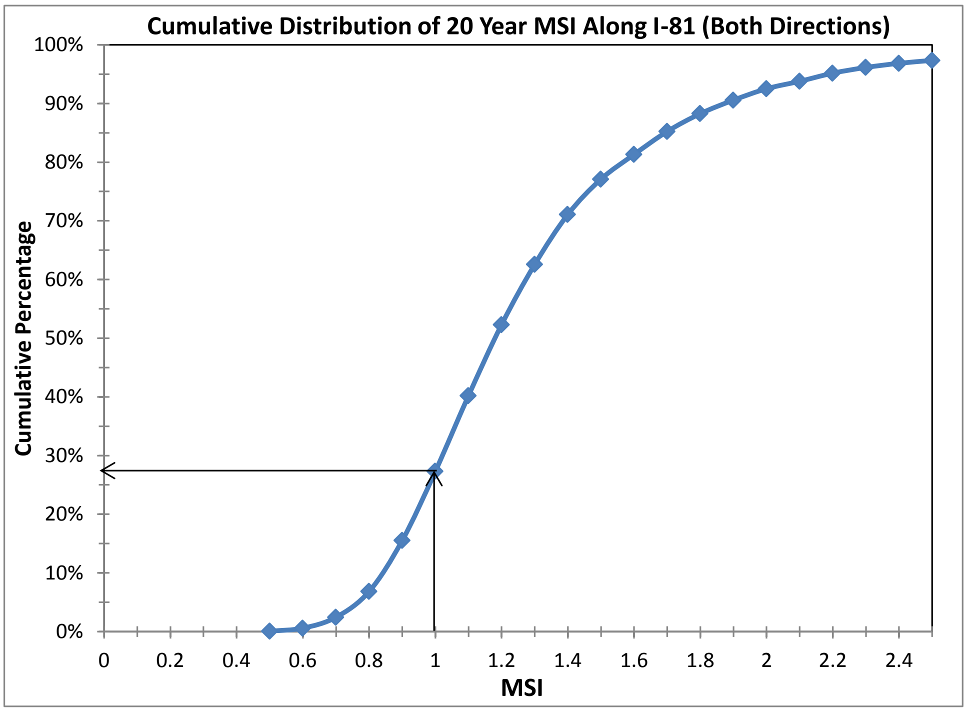

Figure 37. Cumulative Distribution of MSI Using a 20 Year Design Period

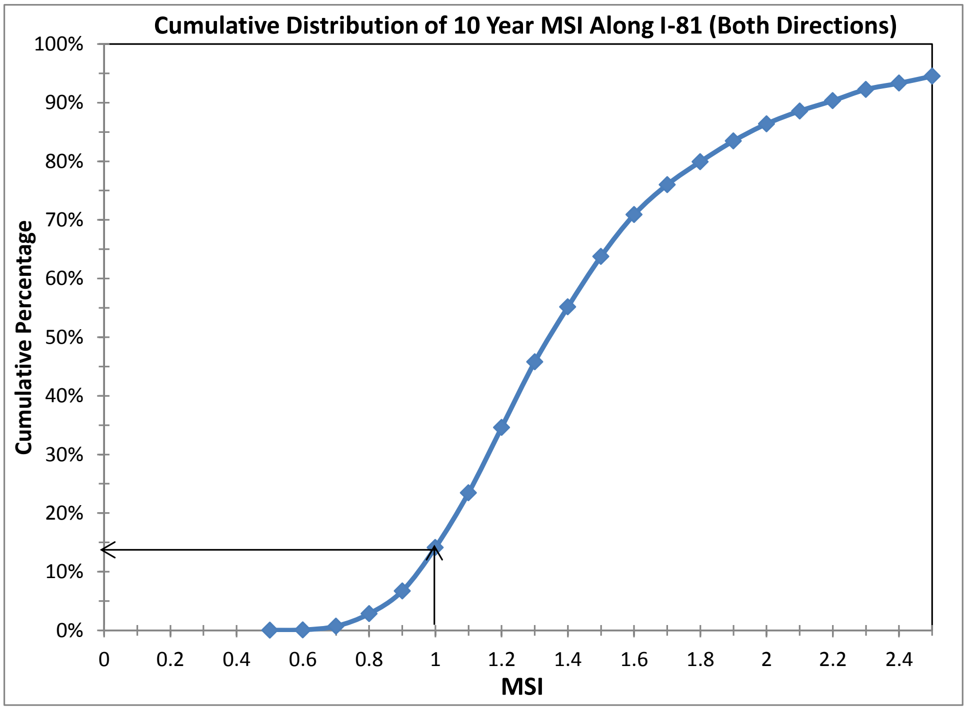

Figure 38. Cumulative Distribution of MSI Using a 10 Year Design Period

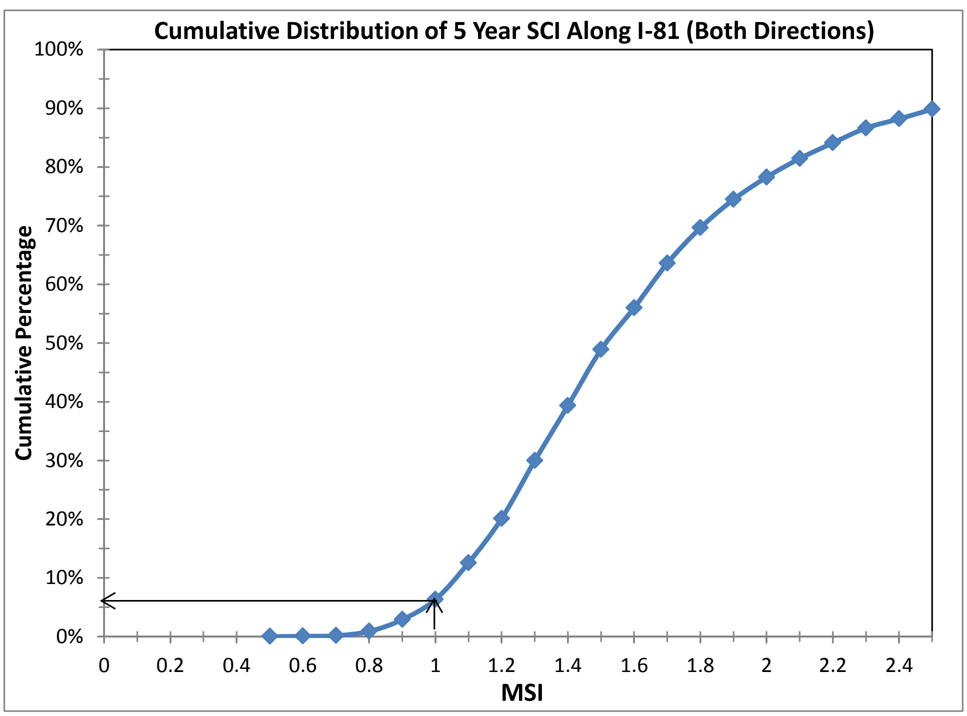

Figure 39. Cumulative Distribution of MSI Using a 5 Year Design Period

5.5. **Deterioration Modeling**

It has been shown that practically no correlation exists between the surface condition of the pavement (e.g., cracking, IRI, etc.), and the structural condition of the pavement. This is thought to be mainly due to the fact that maintenance practices tend to mask the poor functional parameters of the road, while the structural capacity of the pavement remains unchanged. However, the MSI value has an effect on the rate of deterioration of the pavement condition. This can be seen in the CCI trend in Figure 40. The two sections of pavement are located in the same maintenance district in Virginia on Interstate 81, and both pavement sections received a 1½-inch layer of surface mix asphalt during 2008. The line labeled High MSI had an average MSI value of 1.18 along the section, whereas the line labeled Low MSI had an average MSI value of 0.94. It can be seen in the figure that the pavement with a low MSI value exhibits more rapid deterioration of its functional condition than the high MSI pavement. In other words, the pavement that was in poor structural condition deteriorated more rapidly than the pavement that was in adequate structural condition.

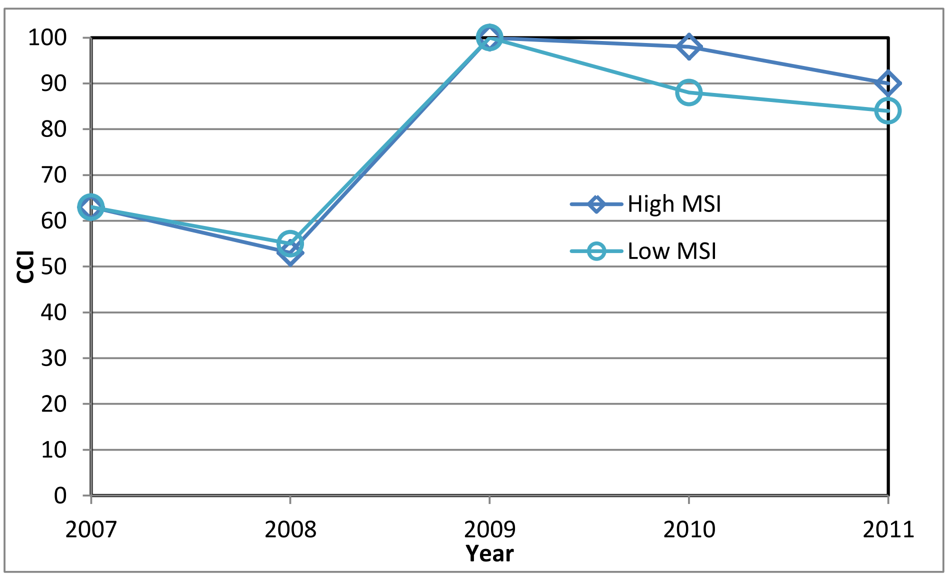

Figure 40. CCI for Two Pavement Sections with Different MSI values

5.5.1. **Impact of MSI on Service Life of Pavements**

In order to quantify the impact of low MSI values on the service life of asphalt pavements, a set of case studies were performed. The analysis was conducted for the case of corrective maintenance treatment being performed on interstates. Work that was completed in the year prior to deflection testing was gathered from the construction history in the VDOT database. Twelve locations were identified along Interstates 81 and 95 in Virginia, eight locations had an MSI value greater than 1, and four locations had an MSI value less than 1. The MSI values were calculated using the 2009 traffic data, a 20 year growth period, and then averaged over the maintenance sections. Windshield survey data was also available for up to five years following the treatment for the majority of these sites.

VDOT has developed a set of prediction curves for particular treatments. Equation 45 shows the prediction curve used for corrective maintenance performed on bituminous pavements.

$$CCI(t) = 100 - e^{a - b * c^{LN\left(\frac{1}{T}\right)}} \quad (45)$$

Where CCI(t) is the predicted CCI in year t for CM treatment, a is 9.176, b is 9.18, c is 1.27295, and t is the time (in years) after the treatment was applied.

For relatively short time periods, (e.g. 1 year or less), variable *a* controls the behavior of equation 45. However, as time increases, equation 45 becomes much more sensitive to changes in variables *b* and *c*. Given that the product of *b* and *c* is obtained for any given time, and the variable *c* is a function of time, *b* should be held steady and the variable *c* can be changed to determine the impact of the MSI on the behavior of the curves. The result of this can be seen in Figure 41.

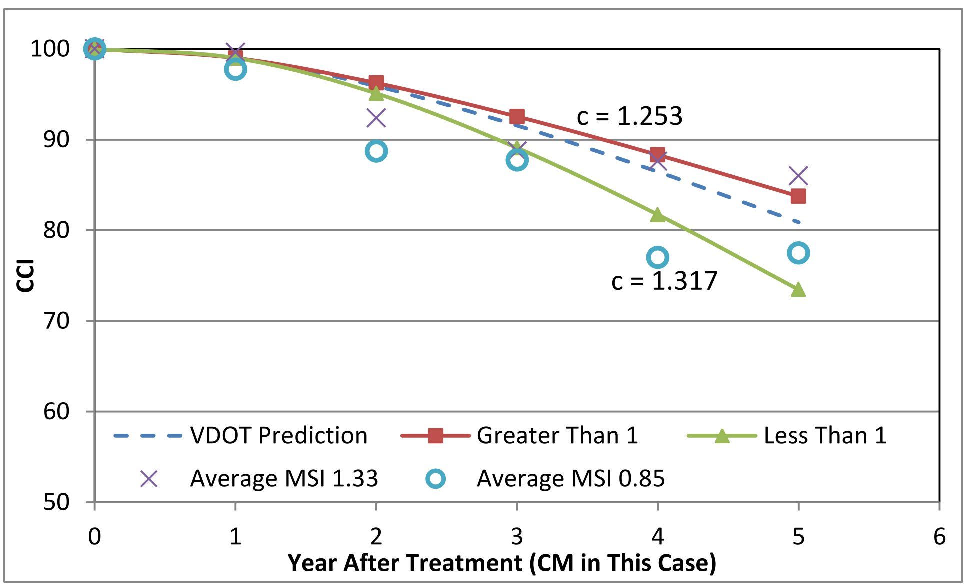

Figure 41. CCI Prediction Curves for Varying MSI Values

It is clear from Figure 41 that decreasing the MSI increases the value for the variable c, which in turn increases the rate of deterioration for the treatment. This behavior is expected because structurally inadequate pavements are expected to deteriorate in their functional condition much more rapidly than structurally adequate pavement. The change in the variable c as a function of the MSI was investigated further by sorting the sites into bins that produced different average MSI values. For each bin an average MSI value and its corresponding *c* value were calculated. The results can be seen in Figure 42.

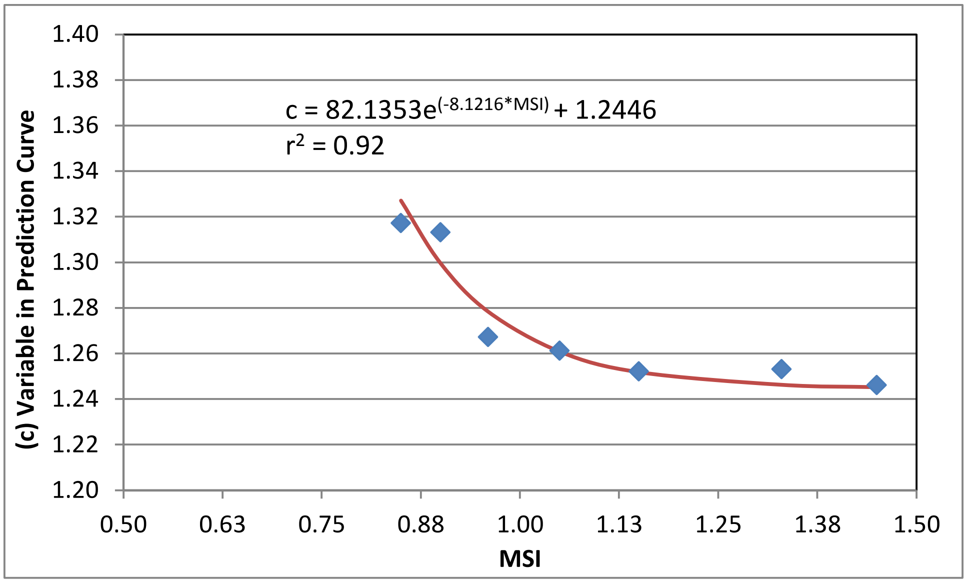

Figure 42. Prediction Curve (c) Variable as a Function of MSI

An interesting note from Figure 42 is the behavior of the plot around the value of one. The shape of the function seems to indicate that MSI values much greater than one do not affect the service life of a pavement as dramatically as values of MSI between 0.88 and one. Furthermore, once the MSI for a pavement decreases below about 0.9, the rate of change of the variable (c) as a function of the MSI increases rapidly.

5.6. **Example Applications Conclusion**

It was clearly shown that the structural response is required in order to fully characterize the condition of a pavement. In this case, the MSI was shown to be most effective way to quantify the structural response of a pavement while incorporating the traffic load on the pavement. Furthermore, a number of applications to pavement management were presented in this chapter.

It was shown that the integration of the MSI into the decision process can enhance decision making by incorporating structural response. In addition, the MSI can be used as an initial network-level screening tool to identify problematic locations, or locations where the pavement structure is inadequate to carry its intended load. The structural screening tool was taken a step further to estimate a required overlay thickness to increase the structure of the network to a particular level.

The MSI can add a dimension to performance measures by distinguishing highways that are structurally deficient. The potential benefit of adding the MSI as a performance indicator is the ability to discern between pavements that are in poor structural condition, but are in fair or better condition because of recent work. This will lead to a more thorough understanding of the condition of the pavements within a network.

Finally, the use of the MSI in pavement deterioration forecasting was discussed. It was shown that the condition of a pavement in poor structural condition (i.e., low MSI values) will deteriorate much faster than a pavement in adequate structural condition. Thus, the performance of a maintenance activity as a function of the structural condition can be quantified.

CHAPTER 6. CONCLUSIONS AND RECOMMENDATIONS

The findings and conclusions presented in this chapter are based on the analysis presented in the previous chapters of this thesis. The objective of the thesis was to develop a structural index for network-level pavement evaluation. Furthermore, several examples of pavement management applications using the structural index were presented.

6.1. Findings

The following are the main findings of the research:

1. There are several structural indices that have been used for supporting network-level pavement management applications. These include: the Structural Adequacy Index (Haas et al. 1994), the Structural Strength Index (Scullion, 1988), the Structural Capacity Index (Zhang, 2003), the Structural Strength Indicator (Flora, 2009), and methods to interpret Remaining Service Life (Gedafa et al., 2010). Of these indices the Structural Condition Index used by the Texas DOT was deemed to most appropriate for VDOT needs
2. A weak level of correlation was found between the various distress-based condition indicators and the structural measures of the pavement. Additionally, the correlation is weak enough to be deemed insignificant for practical purposes. The weak correlation is thought to be due to the fact that some maintenance practices address functional condition but do not correct structural deficiencies. For example, crack sealing or fog seals will increase the functional condition of the pavement while adding no structural capacity.
3. The strength of the subgrade and the strength of the pavement should not be used independently when making pavement maintenance decisions. This is because the subgrade strength is compensated for during the structural design of the pavement. Also, the strength of the subgrade deteriorates very little over time, whereas the strength of the pavement may change significantly as age and level of traffic increases. Therefore, the condition of the pavement should be evaluated as a function of the strength of the subgrade.
4. Traffic should be included when describing the structural condition of a pavement, and not discretized into bins based on pavement classifications. This is because the traffic can vary considerably along the same road classification, causing a large variation in the required structure for the pavement. The range of traffic was found to be especially wide for the case that two routes merge, and follow each other along the same roadway for some distance.
5. The analysis verified that the *in situ* structural condition of the pavement significantly influences the service life of a maintenance treatment. As the existing structural condition of the pavement decrease, the expected life of a maintenance action also decreases.

6.2. **Conclusions**

This thesis confirmed that network level pavement management decisions that incorporate a structural condition measure more closely match the decisions made during project-level assessment than those based only on functional condition and surface distress. The functional characteristics of a pavement alone do not seem adequate to describe the structural condition of the pavement independent from a structural evaluation. Therefore, the structural condition of the pavement, e.g., based on the results from deflection testing, should be considered when making network-level pavement management decisions.

The structural design of a pavement follows engineering principles that are ubiquitous in engineering practice. When designing a structure of any sort, it is imperative to take into account the load the structure receives, as well as the strength of the materials and design. Therefore, the structural evaluation of a pavement should take into account both the *in-situ* strength and the required strength of the pavement. Using these principles, the Modified Structural Index presented in this thesis accounts for both the required strength of the pavement (traffic and strength of resilient modulus), and the existing load carrying capacity of the pavement (effective structural number).

6.3. **Implementation**

The Virginia Department of Transportation has already begun conducting a second round of network-level deflection testing. Therefore, incorporating the MSI using the results from the deflection testing will require only the modification of the decision process as was shown in Chapter 5 of this thesis. Furthermore, the applications presented in Chapter 5 of this thesis can be expanded to all locations where deflection testing has been conducted on the Virginia Interstate network.

6.4. **Recommendations for Future Research**

This thesis focused on estimating the *in-situ* structural number of the pavement using one particular method developed by Rhode (1994). However, there are many methods to estimate the *in-situ* structural number of a pavement, each of which should be explored in relation to the MSI, and their impact on the MSI should be determined.

The index that was proposed in this research was based on the structural number concept from the AASHO road test. The structural number was an empirical relationship that incorporated driver perception, along with many other factors, to calculate the thickness required for combinations of particular materials used in road construction. Although the index was shown to be a useful tool for network-level pavement management, the introduction of the MEPDG/ DARWin-ME in recent years has begun to move the transportation profession away from the structural number concept, and toward mechanistic models. Thus, it is recommended that equivalent mechanistic models be developed that adequately models the effective and required structure of a pavement. Such models may be based on asphalt strains or pavement deflections that represent the mechanical characteristics of the asphalt.

Furthermore, the impact of implementing the MSI as part of a pavement warranty specification should be further investigated. For example, the risk should be quantified, and the costs associated with the additional risk should be determined. Also, the methods for determining allowable MSI values along a pavement section should be further researched and developed.

The proposed index was developed to incorporate deflections from the FWD, which is the most common deflection measuring device used on pavements. However, continuous deflection measuring devices are becoming more common in pavement management applications. Therefore, research should be conducted to determine an appropriate structural index that incorporates measurements from continuous deflection measuring devices.

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APPENDIX A. TABLES FOR INTERSTATE 81 NORTHBOUND IN PULASKI COUNTY

Table A. 1. Decision Matrix Inputs for Interstate 81 Northbound in Pulaski County

Begin Mile Post	End Mile Post	CCI	Transverse Cracks Not Severe	Transverse Cracks Severe	Patching	Rutting	Alligator Crack Not Severe	Alligator Crack Severe	CCI Filter
10.31	10.40	31	76	200	P1	>10%, <0.5"	Occa	Freq	RC
10.40	10.50	77	72	2	P1	>10%, <0.5"	Freq	Occa	
10.50	10.60	51	82	25	P1	>10%, <0.5"	Occa	Occa	At Least CM
10.60	10.70	72	105	3	P0	>10%, <0.5"	Occa	Occa	
10.70	10.80	71	154	3	P1	>10%, <0.5"	Occa	Occa	
10.80	10.90	51	110	25	P1	>10%, <0.5"	Freq	Occa	At Least CM
10.90	11.00	37	110	40	P1	>10%, <0.5"	Occa	Occa	RC
11.00	11.10	71	100	0	P1	>10%, <0.5"	Occa	Occa	
11.10	11.20	70	113	3	P1	>10%, <0.5"	Freq	Occa	
11.20	11.30	75	66	0	P1	>10%, <0.5"	Occa	Occa	
11.30	11.40	64	3	0	P0	>10%, <0.5"	Occa	None	
11.40	11.50	71	0	0	P0	>10%, <0.5"	Occa	None	
11.50	11.60	51	4	0	P0	>10%, <0.5"	Occa	None	At Least CM
11.60	11.70	74	5	2	P0	>10%, <0.5"	Freq	None	
11.70	11.80	76	3	0	P0	>10%, <0.5"	Freq	None	
11.80	11.81	78	0	0	P0	>10%, <0.5"	Freq	None	

Table A. 2. Decisions from VDOT Decision Process for Interstate 81 Northbound in Pulaski County

Begin Mile Post	End Mile Post	Initial Decision	Resilient Modulus	Structural Number	"Weak" or "Strong"	Traffic Level	Final Decision
10.31	10.40	RC	12,235	7.10	Strong	3	RC
10.40	10.50	CM	12,235	7.10	Strong	3	CM
10.50	10.60	CM	12,987	7.78	Strong	3	CM
10.60	10.70	CM	12,987	7.78	Strong	3	CM
10.70	10.80	CM	8,918	6.52	Weak	3	RM
10.80	10.90	CM	8,918	6.52	Weak	3	RM
10.90	11.00	RC	10,146	6.64	Strong	3	RC
11.00	11.10	CM	10,146	6.64	Strong	3	CM
11.10	11.20	CM	13,964	7.95	Strong	3	CM
11.20	11.30	CM	13,964	7.95	Strong	3	CM
11.30	11.40	CM	12,454	5.98	Weak	3	RM
11.40	11.50	CM	12,454	5.98	Weak	3	RM
11.50	11.60	CM	15,063	6.43	Strong	3	CM
11.60	11.70	CM	15,063	6.43	Strong	3	CM
11.70	11.80	CM	11,064	7.47	Strong	3	CM
11.80	11.81	CM	11,064	7.47	Strong	3	CM

Table A. 3. Decisions Based on Structural Indices for Interstate 81 Northbound in Pulaski County

Begin Mile Post	End Mile Post	Initial Decision (Pre-Decision Tree)	SCI	Decision from SCI	D0	Decision from D0	Weighted Difference Index
10.31	10.40	RC	1.24	RC	5.50	RC	3.73
10.40	10.50	CM	1.24	CM	5.50	CM	3.73
10.50	10.60	CM	1.28	CM	4.83	CM	3.57
10.60	10.70	CM	1.28	CM	4.83	CM	3.57
10.70	10.80	CM	0.98	RM	6.85	RM	4.56
10.80	10.90	CM	0.98	RM	6.85	RM	4.56
10.90	11.00	RC	1.00	RC	6.66	RC	4.48
11.00	11.10	CM	1.00	RM	6.66	RM	4.48
11.10	11.20	CM	1.38	CM	4.53	CM	3.34
11.20	11.30	CM	1.38	CM	4.53	CM	3.34
11.30	11.40	CM	1.15	CM	7.17	RM	4.26
11.40	11.50	CM	1.15	CM	7.17	RM	4.26
11.50	11.60	CM	1.20	CM	6.14	CM	3.73
11.60	11.70	CM	1.20	CM	6.14	CM	3.73
11.70	11.80	CM	1.18	CM	5.32	CM	3.85
11.80	11.81	CM	1.18	CM	5.32	CM	3.85

Table A. 4. Final Maintenance Decisions for Interstate 81 Northbound in Pulaski County

Begin Mile Post	End Mile Post	Decision Based on VDOT Decision Process	Decision Based on SCI	Decision Based on D_0
10.31	10.40	RC	RC	RC
10.40	10.50	CM	CM	CM
10.50	10.60	CM	CM	CM
10.60	10.70	CM	CM	CM
10.70	10.80	RM	RM	RM
10.80	10.90	RM	RM	RM
10.90	11.00	RC	RC	RC
11.00	11.10	CM	RM	RM
11.10	11.20	CM	CM	CM
11.20	11.30	CM	CM	CM
11.30	11.40	RM	CM	RM
11.40	11.50	RM	CM	RM
11.50	11.60	CM	CM	CM
11.60	11.70	CM	CM	CM
11.70	11.80	CM	CM	CM
11.80	11.81	CM	CM	CM

APPENDIX B. TABLES FOR INTERSTATE 81 SOUTHBOUND IN BOTETOURT COUNTY

Table B. 1. Decision Matrix Inputs for Interstate 81 Southbound in Botetourt County

Begin Mile Post	End Mile Post	CCI	Transverse Cracks Not Severe	Transverse Cracks Severe	Patching	Rutting	Alligator Crack Not Severe	Alligator Crack Severe	CCI Filter
14.60	14.70	75	21.2	10.4	P0	>10%, <0.5"	Occa	Occa	
14.70	14.80	81	28.6	3.6	P0	>10%, <0.5"	Occa	None	
14.80	14.90	84	8.8	0.0	P0	>10%, <0.5"	Occa	Occa	
14.90	15.00	82	14.0	0.0	P0	>10%, <0.5"	Occa	None	
15.00	15.10	81	12.5	2.0	P1	>10%, <0.5"	Occa	None	
15.10	15.20	81	7.6	0.0	P1	>10%, <0.5"	Occa	None	
15.20	15.30	83	8.4	0.0	P1	>10%, <0.5"	Occa	None	
15.30	15.40	84	4.4	0.4	P0	>10%, <0.5"	Occa	Occa	
15.40	15.50	91	1.6	0.0	P0	>10%, <0.5"	Occa	Occa	DN
15.50	15.60	85	7.6	0.0	P1	>10%, <0.5"	Occa	None	DN or PM
15.60	15.70	83	8.4	0.0	P0	>10%, <0.5"	Occa	None	
15.70	15.80	71	12.0	0.0	P1	>10%, <0.5"	Occa	Occa	
15.80	15.90	82	11.2	0.0	P1	>10%, <0.5"	Occa	None	
15.90	16.00	74	16.4	1.2	P1	>10%, <0.5"	Freq	Occa	
16.00	16.10	50	26.8	0.0	P1	>10%, <0.5"	Freq	Occa	CM, RM, RC
16.10	16.20	80	8.4	0.0	P0	>10%, <0.5"	Occa	None	
16.20	16.30	86	6.4	0.0	P0	>10%, <0.5"	None	None	DN or PM
16.30	16.32	90	0.0	0.0	P0	>10%, <0.5"	None	None	DN

Table B. 2. Decisions from VDOT Decision Process: Interstate 81 Southbound in Botetourt County

Begin Mile Post	End Mile Post	Initial Decision	Resilient Modulus	Structural Number	"Weak" or "Strong"	Traffic Level	Final Decision
14.60	14.70	CM	16,980	6.04	Strong	3	CM
14.70	14.80	CM	9,712	7.72	Weak	3	RM
14.80	14.90	CM	9,712	7.72	Weak	3	RM
14.90	15.00	CM	30,973	8.40	Strong	3	CM
15.00	15.10	CM	30,973	8.40	Strong	3	CM
15.10	15.20	CM	17,497	5.54	Weak	3	RM
15.20	15.30	CM	17,497	5.54	Weak	3	RM
15.30	15.40	CM	32,996	6.25	Strong	3	CM
15.40	15.50	DN	32,996	6.25	Strong	3	CM
15.50	15.60	PM	27,907	6.74	Strong	3	PM
15.60	15.70	CM	27,907	6.74	Strong	3	CM
15.70	15.80	CM	19,635	6.70	Strong	3	CM
15.80	15.90	CM	19,635	6.70	Strong	3	CM
15.90	16.00	CM	40,899	5.29	Weak	3	RM
16.00	16.10	CM	40,899	5.29	Weak	3	RM
16.10	16.20	CM	23,648	6.55	Strong	3	CM
16.20	16.30	PM	23,648	6.55	Strong	3	PM
16.30	16.32	DN	26,071	5.32	Weak	3	RM

Table B. 3. Decisions Based on Structural Indices for Interstate 81 Southbound in Botetourt County

Begin Mile Post	End Mile Post	Initial Decision (Pre-Decision Tree)	SCI	Decision from SCI	D0	Decision from D0
14.60	14.70	CM	1.05	RM	7.52	RM
14.70	14.80	CM	1.12	CM	5.52	CM
14.80	14.90	CM	1.12	CM	5.52	CM
14.90	15.00	CM	1.81	CM	3.78	CM
15.00	15.10	CM	1.81	CM	3.78	CM
15.10	15.20	CM	0.97	RM	8.93	RM
15.20	15.30	CM	0.97	RM	8.93	RM
15.30	15.40	CM	1.38	CM	6.40	CM
15.40	15.50	DN	1.38	DN	6.40	DN
15.50	15.60	PM	1.40	PM	5.51	PM
15.60	15.70	CM	1.40	CM	5.51	CM
15.70	15.80	CM	1.23	CM	6.09	CM
15.80	15.90	CM	1.23	CM	6.09	CM
15.90	16.00	CM	1.27	CM	8.78	RM
16.00	16.10	CM	1.27	CM	8.78	RM
16.10	16.20	CM	1.28	CM	6.33	CM
16.20	16.30	PM	1.28	CM	6.33	CM
16.30	16.32	DN	1.08	RM	8.13	RM

Table B. 4. Final Maintenance for Interstate 81 Southbound in Botetourt County

Begin Mile Post	End Mile Post	Decision Based on VDOT Decision Process	Decision Based on SCI	Decision Based on D_0
14.60	14.70	CM	RM	RM
14.70	14.80	RM	CM	CM
14.80	14.90	RM	CM	CM
14.90	15.00	CM	CM	CM
15.00	15.10	CM	CM	CM
15.10	15.20	RM	RM	RM
15.20	15.30	RM	RM	RM
15.30	15.40	CM	CM	CM
15.40	15.50	CM	DN	DN
15.50	15.60	PM	PM	PM
15.60	15.70	CM	CM	CM
15.70	15.80	CM	CM	CM
15.80	15.90	CM	CM	CM
15.90	16.00	RM	CM	RM
16.00	16.10	RM	CM	RM
16.10	16.20	CM	CM	CM
16.20	16.30	PM	CM	CM
16.30	16.32	RM	RM	RM

APPENDIX C. TABLES FOR INTERSTATE 81 SOUTHBOUND IN MONTGOMERY COUNTY

Table C. 1. Decision Matrix Inputs for Interstate 81 Southbound in Montgomery County

Begin Mile Post	End Mile Post	CCI	Transverse Cracks Not Severe	Transverse Cracks Severe	Patch	Rutting	Alligator Crack Not Severe	Alligator Crack Severe	CCI Filter
5.07	5.10	78	0.00	0.00	P0	>10%, <½"	None	None	
5.10	5.20	78	0.00	0.00	P0	>10%, <½"	Occa	Occa	
5.20	5.30	83	0.80	0.00	P0	>10%, <½"	Occa	None	
5.30	5.40	57	1.60	0.00	P0	>10%, <½"	Occa	Occa	CM/RM/RC
5.40	5.50	82	0.80	0.00	P0	>10%, <½"	Occa	Occa	
5.50	5.60	87	0.00	0.00	P0	>10%, <½"	Occa	Occa	DN or PM
5.60	5.70	86	0.40	0.00	P0	>10%, <½"	Occa	None	DN or PM
5.70	5.80	85	0.00	0.00	P0	>10%, <½"	Occa	None	DN or PM
5.80	5.90	57	1.60	0.00	P0	>10%, <½"	Occa	Occa	CM/RM/RC
5.90	6.00	41	10.80	0.00	P0	>10%, <½"	Occa	Occa	RM/RC
6.00	6.10	51	0.00	0.40	P0	>10%, <½"	None	Occa	CM/RM/RC
6.10	6.20	53	0.40	9.60	P0	>10%, <½"	Occa	Occa	CM/RM/RC
6.20	6.30	50	0.40	0.00	P0	>10%, <½"	Occa	Occa	CM/RM/RC
6.30	6.40	53	0.00	0.40	P0	>10%, <½"	None	Occa	CM/RM/RC
6.40	6.50	52	0.00	0.00	P0	>10%, <½"	Occa	Occa	CM/RM/RC
6.50	6.60	54	0.40	3.20	P0	>10%, <½"	None	Occa	CM/RM/RC
6.60	6.70	56	0.00	0.40	P0	>10%, <½"	None	Occa	CM/RM/RC
6.70	6.80	68	0.00	0.00	P0	>10%, <½"	None	Occa	
6.80	6.90	65	0.40	3.60	P0	>10%, <½"	Occa	Occa	
6.90	7.00	57	3.60	8.40	P0	>10%, <½"	Occa	Occa	CM/RM/RC
7.00	7.10	49	3.60	4.00	P0	>10%, <½"	Freq	Occa	CM/RM/RC
7.10	7.20	47	2.00	14.80	P0	>10%, <½"	Occa	Freq	RM/RC
7.20	7.30	39	5.60	12.40	P0	>10%, <½"	Freq	Occa	RM/RC
7.30	7.40	52	1.60	12.80	P0	>10%, <½"	Occa	Occa	CM/RM/RC
7.40	7.50	38	13.60	7.60	P0	>10%, <½"	Occa	Freq	RM/RC
7.50	7.60	48	5.60	0.80	P0	>10%, <½"	Freq	Occa	RM/RC
7.60	7.70	52	13.20	0.00	P0	>10%, <½"	Freq	Occa	CM/RM/RC
7.70	7.80	61	4.80	0.00	P0	>10%, <½"	Freq	Occa	
7.80	7.90	47	7.20	0.00	P0	>10%, <½"	Freq	Freq	RM/RC
7.90	8.00	40	8.80	14.40	P0	>10%, <½"	Freq	Freq	RM/RC
8.00	8.10	52	2.40	2.00	P0	>10%, <½"	Occa	Freq	CM/RM/RC
8.10	8.20	75	0.00	0.80	P0	>10%, <½"	None	Occa	
8.20	8.30	56	0.40	4.80	P0	>10%, <½"	Occa	Occa	CM/RM/RC
8.30	8.40	76	0.00	0.40	P0	>10%, <½"	Occa	Occa	

Table C.1. Decision Matrix Inputs for Interstate 81 Southbound in Montgomery County (Cont'd)

Begin Mile Post	End Mile Post	CCI	Transverse Cracks Not Severe	Transverse Cracks Severe	Patch	Rutting	Alligator Crack Not Severe	Alligator Crack Severe	CCI Filter
8.40	8.50	71	0.80	0.40	P0	>10%, <½"	Occa	Freq	
8.50	8.60	72	8.00	1.20	P0	>10%, <½"	Occa	Occa	
8.60	8.70	75	4.40	0.80	P0	>10%, <½"	Occa	Occa	
8.70	8.80	73	10.00	0.80	P0	>10%, <½"	Occa	Occa	
8.80	8.90	58	10.00	0.40	P0	>10%, <½"	Occa	Freq	CM/RM/RC
8.90	9.00	59	13.60	0.00	P0	>10%, <½"	Occa	Occa	CM/RM/RC
9.00	9.10	72	3.20	0.00	P0	>10%, <½"	Occa	Occa	
9.10	9.20	38	5.20	0.00	P0	>10%, <½"	Occa	Freq	RM/RC
9.20	9.30	36	41.20	2.40	P0	>10%, <½"	Occa	Freq	RC
9.30	9.40	39	15.20	0.00	P0	>10%, <½"	Occa	Freq	RM/RC
9.40	9.50	40	32.00	4.00	P0	>10%, <½"	Occa	Freq	RM/RC
9.50	9.60	38	84.80	2.40	P0	>10%, <½"	Occa	Occa	RM/RC
9.60	9.66	34	80.89	4.00	P0	>10%, <½"	Occa	Freq	RC

Table C. 2. Decisions from VDOT Process: Interstate 81 South in Montgomery County

Begin Mile Post	End Mile Post	Initial Decision	Resilient Modulus	Structural Number	"Weak" or "Strong"	Traffic Level	Final Decision
5.07	5.10	CM	9,372	6.90	Weak	3	RM
5.10	5.20	CM	8,658	7.48	Weak	3	RM
5.20	5.30	CM	8,658	7.48	Weak	3	RM
5.30	5.40	CM	13,295	7.81	Strong	3	CM
5.40	5.50	CM	13,295	7.81	Strong	3	CM
5.50	5.60	PM	15,418	7.69	Strong	3	PM
5.60	5.70	PM	15,418	7.69	Strong	3	PM
5.70	5.80	PM	12,855	7.61	Strong	3	PM
5.80	5.90	CM	12,855	7.61	Strong	3	CM
5.90	6.00	RM	11,388	7.21	Strong	3	RM
6.00	6.10	CM	11,388	7.21	Strong	3	CM
6.10	6.20	CM	9,236	6.90	Weak	3	RM
6.20	6.30	CM	9,236	6.90	Weak	3	RM
6.30	6.40	CM	14,865	7.93	Strong	3	CM
6.40	6.50	CM	14,865	7.93	Strong	3	CM
6.50	6.60	CM	16,307	8.27	Strong	3	CM

Table C.2. Decisions from VDOT Process: Interstate 81 South in Montgomery County (Cont'd)

Begin Mile Post	End Mile Post	Initial Decision	Resilient Modulus	Structural Number	"Weak" or "Strong"	Traffic Level	Final Decision
6.60	6.70	CM	16,307	8.27	Strong	3	CM
6.70	6.80	CM	11,617	7.07	Strong	3	CM
6.80	6.90	CM	11,617	7.07	Strong	3	CM
6.90	7.00	CM	8,991	7.31	Weak	3	RM
7.00	7.10	CM	8,991	7.31	Weak	3	RM
7.10	7.20	CM	7,741	6.30	Weak	3	RM
7.20	7.30	CM	7,741	6.30	Weak	3	RM
7.30	7.40	CM	13,115	7.23	Strong	3	CM
7.40	7.50	RM	13,115	7.23	Strong	3	RM
7.50	7.60	RM	13,826	8.43	Strong	3	RM
7.60	7.70	CM	13,826	8.43	Strong	3	CM
7.70	7.80	CM	11,452	7.34	Strong	3	CM
7.80	7.90	RM	11,452	7.34	Strong	3	RM
7.90	8.00	RM	16,794	8.42	Strong	3	RM
8.00	8.10	CM	16,794	8.42	Strong	3	CM
8.10	8.20	CM	20,096	8.69	Strong	3	CM
8.20	8.30	CM	20,096	8.69	Strong	3	CM
8.30	8.40	CM	11,929	7.41	Strong	3	CM
8.40	8.50	CM	11,929	7.41	Strong	3	CM
8.50	8.60	CM	11,462	7.50	Strong	3	CM
8.60	8.70	CM	11,462	7.50	Strong	3	CM
8.70	8.80	CM	9,900	6.94	Weak	3	RM
8.80	8.90	CM	9,900	6.94	Weak	3	RM
8.90	9.00	CM	10,251	7.75	Strong	3	CM
9.00	9.10	CM	10,251	7.75	Strong	3	CM
9.10	9.20	RM	7,354	5.84	Weak	3	RC
9.20	9.30	RC	7,354	5.84	Weak	3	RC
9.30	9.40	RM	9,722	7.25	Weak	3	RC
9.40	9.50	RM	9,722	7.25	Weak	3	RC
9.50	9.60	RM	8,081	6.54	Weak	3	RC
9.60	9.66	RC	8,081	6.54	Weak	3	RC

Table C. 3. Final Maintenance Decisions for Interstate 81 Southbound in Montgomery County

Begin Mile Post	End Mile Post	Initial Decision	VDOT Decision	SCI	SCI Decision	D_0	D_0 Decision
5.07	5.10	CM	RM	1.04	RM	6.74	RM
5.10	5.20	CM	RM	1.10	CM	5.93	CM
5.20	5.30	CM	RM	1.10	CM	5.93	CM
5.30	5.40	CM	CM	1.52	CM	4.73	CM
5.40	5.50	CM	CM	1.52	CM	4.73	CM
5.50	5.60	PM	PM	1.45	PM	4.65	PM
5.60	5.70	PM	PM	1.45	PM	4.65	PM
5.70	5.80	PM	PM	1.29	PM	4.99	PM
5.80	5.90	CM	CM	1.29	CM	4.99	CM
5.90	6.00	RM	RM	1.16	RM	5.82	RM
6.00	6.10	CM	CM	1.16	CM	5.82	CM
6.10	6.20	CM	RM	1.02	RM	6.78	RM
6.20	6.30	CM	RM	1.02	RM	6.78	RM
6.30	6.40	CM	CM	1.51	CM	4.42	CM
6.40	6.50	CM	CM	1.51	CM	4.42	CM
6.50	6.60	CM	CM	1.62	CM	3.94	CM
6.60	6.70	CM	CM	1.62	CM	3.94	CM
6.70	6.80	CM	CM	1.21	CM	6.00	CM
6.80	6.90	CM	CM	1.21	CM	6.00	CM
6.90	7.00	CM	RM	1.13	CM	6.10	CM
7.00	7.10	CM	RM	1.13	CM	6.10	CM
7.10	7.20	CM	RM	0.88	RM	8.62	RM
7.20	7.30	CM	RM	0.88	RM	8.62	RM
7.30	7.40	CM	CM	1.30	CM	5.56	CM
7.40	7.50	RM	RM	1.30	RM	5.56	RM
7.50	7.60	RM	RM	1.60	RM	3.98	RM
7.60	7.70	CM	CM	1.60	CM	3.98	CM
7.70	7.80	CM	CM	1.17	CM	5.59	CM
7.80	7.90	RM	RM	1.17	RM	5.59	RM
7.90	8.00	RM	RM	1.76	RM	3.77	RM
8.00	8.10	CM	CM	1.76	CM	3.77	CM
8.10	8.20	CM	CM	2.02	CM	3.39	CM
8.20	8.30	CM	CM	2.02	CM	3.39	CM
8.30	8.40	CM	CM	1.21	CM	5.49	CM
8.40	8.50	CM	CM	1.21	CM	5.49	CM

Table C.3. Final Maintenance Decisions: Interstate 81 Southbound in Montgomery County (Cont'd)

Begin Mile Post	End Mile Post	Initial Decision	VDOT Decision	SCI	SCI Decision	D_0	D_0 Decision
8.50	8.60	CM	CM	1.23	CM	5.30	CM
8.60	8.70	CM	CM	1.23	CM	5.30	CM
8.70	8.80	CM	RM	1.04	RM	6.64	RM
8.80	8.90	CM	RM	1.04	RM	6.64	RM
8.90	9.00	CM	CM	1.23	CM	5.30	CM
9.00	9.10	CM	CM	1.23	CM	5.30	CM
9.10	9.20	RM	RC	0.79	RC	10.48	RC
9.20	9.30	RC	RC	0.79	RC	10.48	RC
9.30	9.40	RM	RC	1.08	RC	6.15	RM
9.40	9.50	RM	RC	1.08	RC	6.15	RM
9.50	9.60	RM	RC	0.91	RC	8.01	RC
9.60	9.66	RC	RC	0.91	RC	8.01	RC

Appendix D. Summary Statistics for Model Development for the Modified SCI for Primary and Secondary Routes

Table D. 1. Summary Statistics for Divided Primary Route Formula Development

	ESAL's	*Resilient Modulus*	*SN*
Mean	8.66E+06	7882	4.60
Standard Error	1.57E+06	220	0.07
Median	1.93E+06	7822	4.26
Mode	3.85E+06	6700	N/A
Standard Deviation	2.21E+07	3108	1.04
Sample Variance	4.90E+14	9661654	1.09
Kurtosis	12.67	-0.16	1.35
Skewness	3.65	0.25	1.29
Range	1.15E+08	15800	5.15
Minimum	2.50E+05	1200	3.04
Maximum	1.15E+08	17000	8.20
Sum	1.72E+09	1568436	915
Count	199	199	199

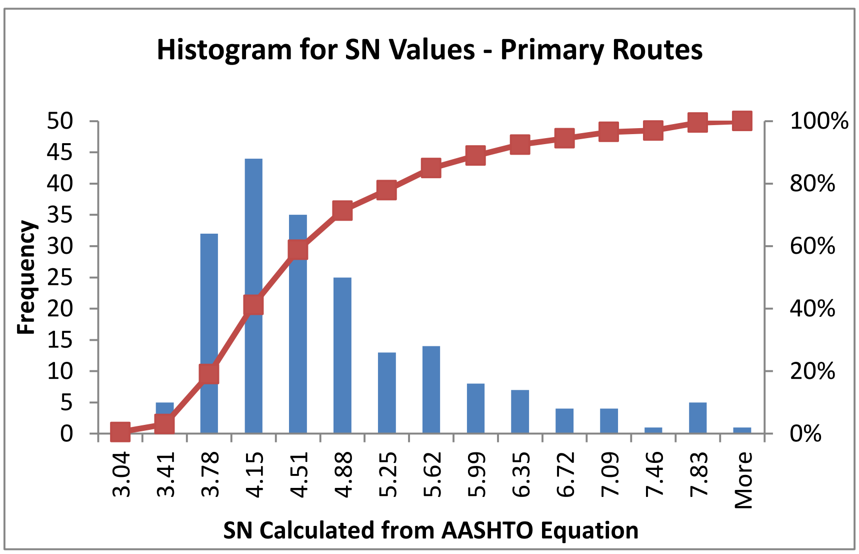

Figure D. 1. SN Values from Generated Traffic and Resilient Modulus for Divided Primary Routes

Table D. 2. Summary Statistics for Un-Divided Primary Route Formula Development

	ESAL's	*Resilient Modulus*	*SN*
Mean	1.33E+06	7614	3.91
Standard Error	8.28E+04	204	0.06
Median	1.05E+06	7708	3.75
Mode	3.85E+06	6700	N/A
Standard Deviation	1.09E+06	2681	0.78
Sample Variance	1.19E+12	7186890	0.61
Kurtosis	24.98	-0.46	1.50
Skewness	4.25	0.04	1.18
Range	9.75E+06	13207	4.50
Minimum	2.50E+05	1600	2.48
Maximum	1.00E+07	14807	6.98
Sum	230288432.1	1317136	676
Count	173	173	173

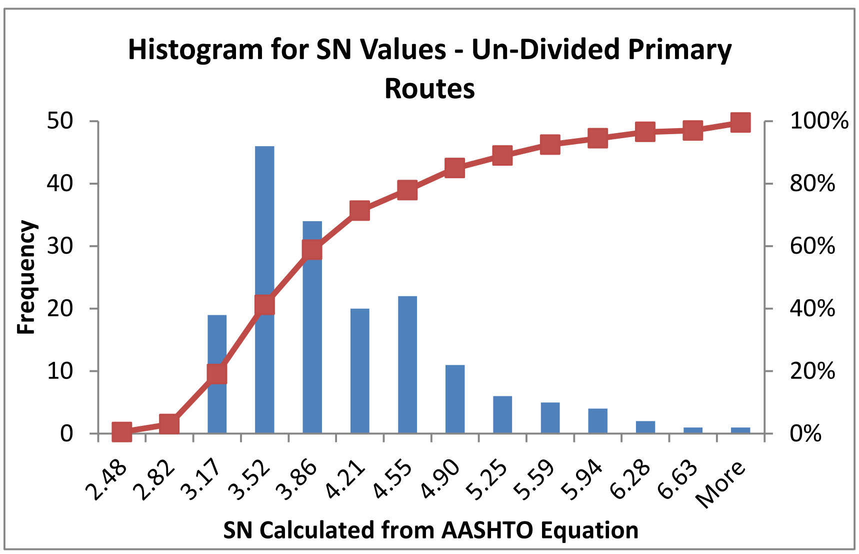

Figure D. 2. SN Values from Generated Traffic and Resilient Modulus for Un-Divided Primary Routes

Table D. 3 Summary Statistics for High Volume Secondary Route Formula Development

	ESAL's	*Resilient Modulus*	*SN*
Mean	1.24E+06	7639	3.68
Standard Error	6.72E+04	203	0.06
Median	1.01E+06	7742	3.50
Mode	3.85E+06	6700	N/A
Standard Deviation	8.82E+05	2667	0.72
Sample Variance	7.78E+11	7114344	0.52
Kurtosis	7.02	-0.44	1.04
Skewness	2.69	0.04	1.14
Range	4.70E+06	13207	3.55
Minimum	2.50E+05	1600	2.54
Maximum	4.95E+06	14807	6.09
Sum	2.13E+08	1313936	633
Count	172	172	172

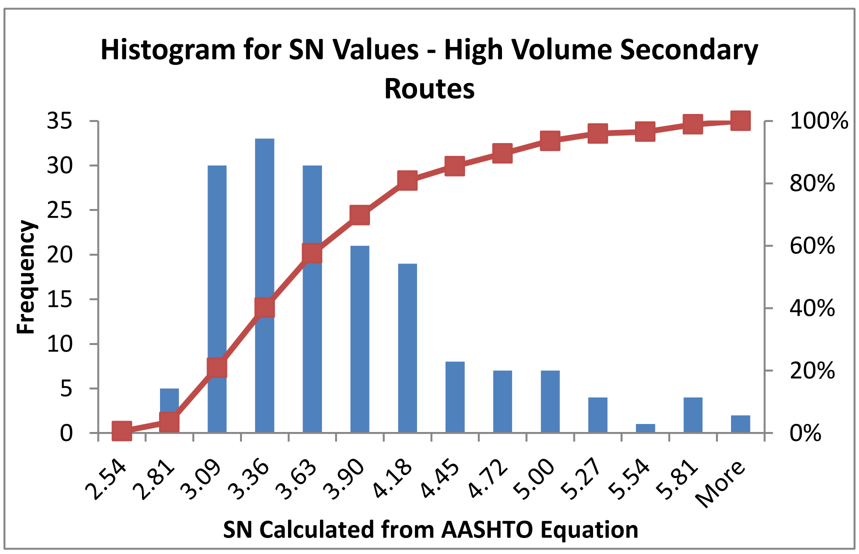

Figure D. 3. SN Values From Generated Traffic and Resilient Modulus for High Volume Secondary Routes