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Developing a Network-Level Structural Capacity Index for Structural Evaluation of Pavements

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Abstract:

The objective of this project was to develop a structural index for use in network-level pavement evaluation to facilitate the inclusion of the pavement's structural condition in pavement management applications. The primary goal of network-level pavement management is to provide the best service to the users for the available, often limited, resources. Pavement condition can be described in terms of functional and structural condition. The current widespread practice of network-level pavement evaluation is to consider only the functional pavement condition. This practice results in suggested treatments that are often under-designed or over-designed when considered in more detail at the project level. The disagreement can be reduced by considering the structural capacity of the pavements as part of a network-level decision process. This study developed a flexible pavement structural index to use for network-level pavement applications. Available pavement condition data were used to conduct a sensitivity analysis of the index, and example applications were tested.

The results indicated that including the structural index developed, named the Modified Structural Index (MSI), into the network-level decision process minimized the discrepancy between network-level predictions and project-level decisions when compared to the current network-level decision-making process. A pilot implementation of the MSI showed that it can be used to support various pavement management decision processes, such as network-level structural screening, deterioration modeling, and development of structural performance measures. The pilot test also indicated that the impact of the structural condition of the pavement on the performance of a maintenance treatment and its impact on life-cycle costs can be quantified.

FINAL REPORT

DEVELOPING A NETWORK-LEVEL STRUCTURAL CAPACITY INDEX FOR STRUCTURAL EVALUATION OF PAVEMENTS

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ABSTRACT

The objective of this project was to develop a structural index for use in network-level pavement evaluation to facilitate the inclusion of the pavement's structural condition in pavement management applications. The primary goal of network-level pavement management is to provide the best service to the users for the available, often limited, resources. Pavement condition can be described in terms of functional and structural condition. The current widespread practice of network-level pavement evaluation is to consider only the functional pavement condition. This practice results in suggested treatments that are often under-designed or over-designed when considered in more detail at the project level. The disagreement can be reduced by considering the structural capacity of the pavements as part of a network-level decision process. This study developed a flexible pavement structural index to use for network-level pavement applications. Available pavement condition data were used to conduct a sensitivity analysis of the index, and example applications were tested.

The results indicated that including the structural index developed, named the Modified Structural Index (MSI), into the network-level decision process minimized the discrepancy between network-level predictions and project-level decisions when compared to the current network-level decision-making process. A pilot implementation of the MSI showed that it can be used to support various pavement management decision processes, such as network-level structural screening, deterioration modeling, and development of structural performance measures. The pilot test also indicated that the impact of the structural condition of the pavement on the performance of a maintenance treatment and its impact on life-cycle costs can be quantified.

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INTRODUCTION

It has generally been assumed that the functional properties of a pavement, such as the International Roughness Index (IRI) or the Pavement Condition Rating (PCR), 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 for a more thorough assessment of the pavement network. Although poor pavement condition can be a result of a poor structure, it is also the case that a pavement in poor functional condition is not necessarily 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 Falling Weight Deflectometer (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 made based only on functional condition were significantly different from the result obtained when deflection testing is included into the decision process (Zaghloul et al., 1998). These results show that, in many cases, the functional indicators are independent from 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 lead to more cost-effective pavement rehabilitation options.

Deflection testing is currently the most widely used method for nondestructive evaluation of the structural capacity of a pavement. Pavement deflection measurements are important inputs to many pavement condition assessment tools, including structural capacity indicator tools and tools to calculate the remaining service life of pavements (Gedafa et al., 2010a). The FWD is currently the most prevalent device used to measure pavement deflections (Hadidi and 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) diameter load plate (MACTEC, 2006). The pavement deflection response is then measured through a set of geophones spanned across the pavement radially from the load. Pavement temperature is the most important environmental factor that affects the response of flexible pavements. It is therefore measured (or estimated) and used to correct deflections to a reference temperature (Gedafa et al., 2010b). Moisture levels can significantly affect the strength of the subgrade and subsequently the overall pavement deflection response to the FWD test; however, because subgrade moisture levels are difficult to measure, moisture correction of measured deflections is typically not performed. After the pavement response is obtained, it is analyzed to determine the in-situ mechanical properties of the pavement. The input typically needed to perform this analysis is the load applied by the FWD, temperature corrected pavement response, seed moduli and layer thicknesses.

PURPOSE AND SCOPE

The purpose of this project was to develop a structural-based condition index for use in network-level pavement evaluation and management applications. This effort was divided into the following three tasks: (1) develop a structural-based condition index for network-level pavement management applications; (2) develop a methodology to use the structural-based condition index in network-level pavement evaluation; and (3) identify and recommend appropriate pavement management applications and situations to use structural-based condition indices.

The first task was completed with a review of the existing literature focused on methods to implement pavement structural information into network-level pavement management applications. The current Virginia Department of Transportation (VDOT) methodology used to incorporate structural information into the network-level decision process was also thoroughly evaluated. The different methods found in the literature were then evaluated by comparing actual project-level work done to network-level predicted work using data from the VDOT pavement management system, as well as work orders from completed projects. As a result of this evaluation, the Texas Department of Transportation Structural Capacity Index (SCI) was selected to be further modified and implemented within the VDOT pavement management system.

The first step in the second task was to perform a sensitivity analysis of the chosen index to identify important input parameters. The chosen index was then implemented in a pilot application as part of a process that builds on methods currently used by VDOT to incorporate structural capacity measures into network-level decisions. The main difference between the current VDOT methodology and the newly developed one is that treatment selection is made a function of the proposed index as opposed to a number of previously defined VDOT parameters.

The third task demonstrated the use of the developed index to enhance pavement management applications. The investigated applications were (1) project scoping, (2) structural screening of pavement sections, (3) overlay thickness estimation, and (4) pavement deterioration modeling. Finally, a life-cycle cost analysis was conducted to demonstrate the potential capability of the proposed index to differentiate between the expected cost of preserving a strong and weak pavement.

CURRENT VDOT PRACTICE

Data Collection

VDOT manages more than 125,000 lane-miles of roads throughout the state (VDOT, 2010). 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 an outside contractor, who uses a vehicle with continuous digital imaging, automated crack detection, and is equipped with sensors that measure roughness and rutting data (VDOT, 2010). Each year, the entire Interstate and Primary road networks are evaluated, along with approximately 20% of the Secondary road network.

Asphalt Pavements

For asphalt pavements, 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 fatigue cracking in the wheel path (VDOT, 2006). The NDR is calculated from non-load-related distresses, such as construction deficiencies, bleeding, etc. (VDOT, 2006). The Critical Condition Index (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 shown in Table 1.

Table 1. Pavement Condition Definitions (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 approximately 2,300 directional miles (3,700 directional km) 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. (2007) determined that this number can be reduced without statistically compromising the data.

Rigid Pavements

The distress indices used for continuous reinforced concrete (CRC) pavements are Concrete Distress Rating (CDR) and Concrete Punchout Rating (CPR) (VDOT, 2008). For jointed concrete pavements (JCP), the distress index used by VDOT is the Slab Distress Rating (SDR). Tables 2 and 3 show the index ranges for different work categories. It is important to note that preventive maintenance is typically not conducted on CRC pavements because they are designed to have low severity transverse cracks not closely spaced.

Table 2. Maintenance Activities for CRC Pavements and Index Ranges (VDOT, 2008)

Maintenance Action	CDR/CPR Range
Do Nothing (DN)	≥87
Corrective Maintenance (CM)	76 -86
Restorative Maintenance (RM)	58 -75
Major Rehabilitation / Reconstruction (RC)	<58

Table 3. Maintenance Activities for JCP and Index Ranges (VDOT, 2008)

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

VDOT Pavement Management Decision Matrix

VDOT utilizes a set of pavement management decision matrices with distresses as inputs and treatment activities as outputs. The matrices are separated based on the following roadway 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 1). In 2008, the procedure was modified to include structural condition and truck traffic volumes, and the enhanced decision tree was integrated into the process.

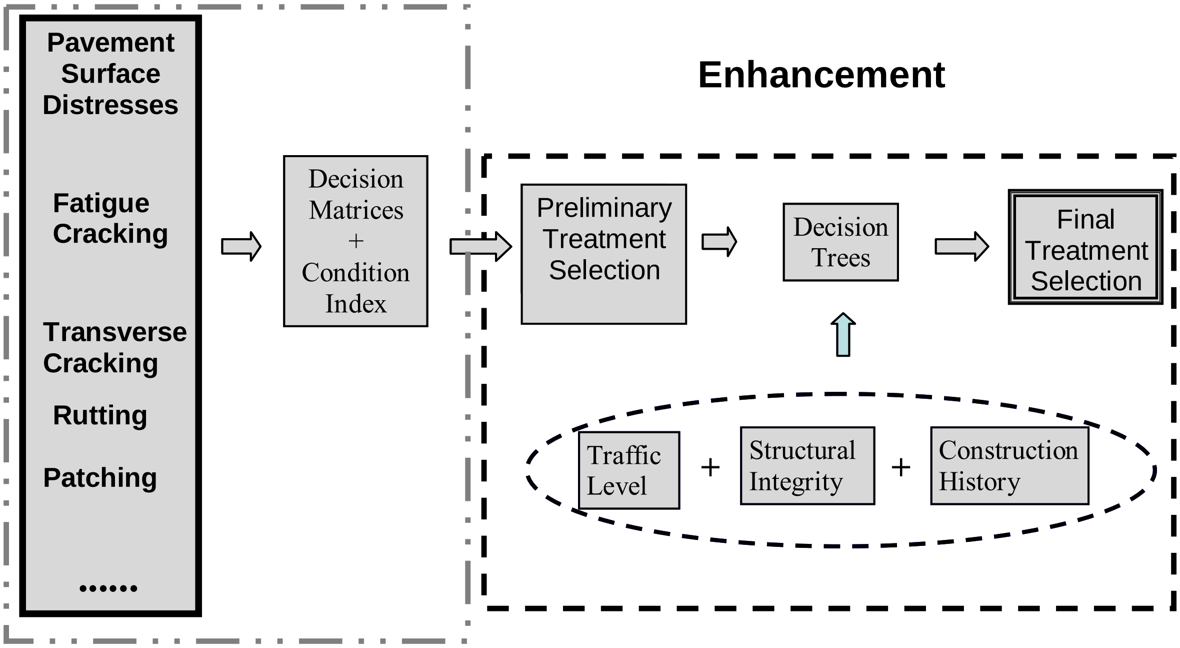

Figure 1. VDOT 2-Phase Decision Process (VDOT, 2008)

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 which will be presented later.

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 4.

Table 4. 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 in	DN	DN	CM	DN	PM	CM	PM	CM	RM
		> .5 in	CM	CM	CM	CM	CM	RM	CM	RM	RM
	>10%	**None**	DN	DN	CM	DN	PM	CM	PM	CM	RM
		< .5 in	CM	CM	CM	CM	CM	CM	CM	RM	RM
		> .5 in	RM	RM	RM	RM	RM	RM	RM	RC	RC

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

The maintenance activity categories for asphalt-surfaced roads are described in further detail in Table 5. 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).

Critical Condition Index Filter for Asphalt and Composite Pavements

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 defines a required level of work based on the CCI of the pavement. For example, given a CCI above 84, the work will be always either preventative maintenance or do nothing, regardless of the output from the decision matrices. However, a CCI value between 60 and 84 requires the work to be the output from the decision matrix. 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.

Table 5. Maintenance Categories for Asphalt-Surfaced Interstate and Primary Roads (VDOT, 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 in)
		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 in)
		2. Partial Depth Patching (<10% of Pavement Area: Depth 4-6 in) and Surface Treatment
		3. Partial Depth Patching (<10% of Pavement Area: Depth 4-6 in) and Thin (≤ 2 in) AC Overlay
		4. ≤ 2 in Milling and ≤ 2 in AC Overlay
Restorative Maintenance (RM)	8-12	1. Heavy Patching (<20% of Pavement Area: Full Depth Patching: Depth 12 in)
		2. ≤4 in Milling and Replace with ≤4 in AC Overlay
		3. Full Depth Patching (<20% of Pavement Area: Full Depth Patching: Depth 9-12 in) and 4 in AC Overlay
Rehabilitation /Reconstruction (RC)	15+	1. Mill, Break and Seat and 9-12 in AC Overlay
		2. Reconstruction

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.

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. This part of the decision process is where this research is expected to have the greatest impact. The enhanced decision trees were introduced in 2008 and, during this research, were evaluated along with other structural methods to determine their adequacy for network-level decision making. The current implementation of the enhanced decision trees for asphalt pavement treats the structural number and resilient modulus of the pavement as a proxy for the strength of the pavement, and pavement class and traffic as a proxy for the required strength. It is expected that this type of structural evaluation can be improved upon using a more detailed evaluation of existing and required structure.

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 using the following structural parameters (VDOT, 2008):

- *For Interstate bituminous surfaced pavements (BIT):* the back-calculated structural number and resilient modulus.

- *For Interstate bituminous-surfaced composite pavements (BOC/BOJ):* basin area (AREA) and static K-values. AREA refers to the deflected area of the pavement resulting from an applied load, and is estimated using a trapezoidal technique. The K-value is a measure of the subgrade strength below the pavement.

- *For Interstate continuously reinforced concrete (CRC) and jointed concrete pavements (JCP):* deflection under the center of the loading plate and AREA.

The final step is to determine the traffic in terms of the average annual daily truck traffic (AADTT). An example of an enhanced decision tree for asphalt surfaced pavements can be seen in Figure 2 and Table 6. The decision tree in Figure 2 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).

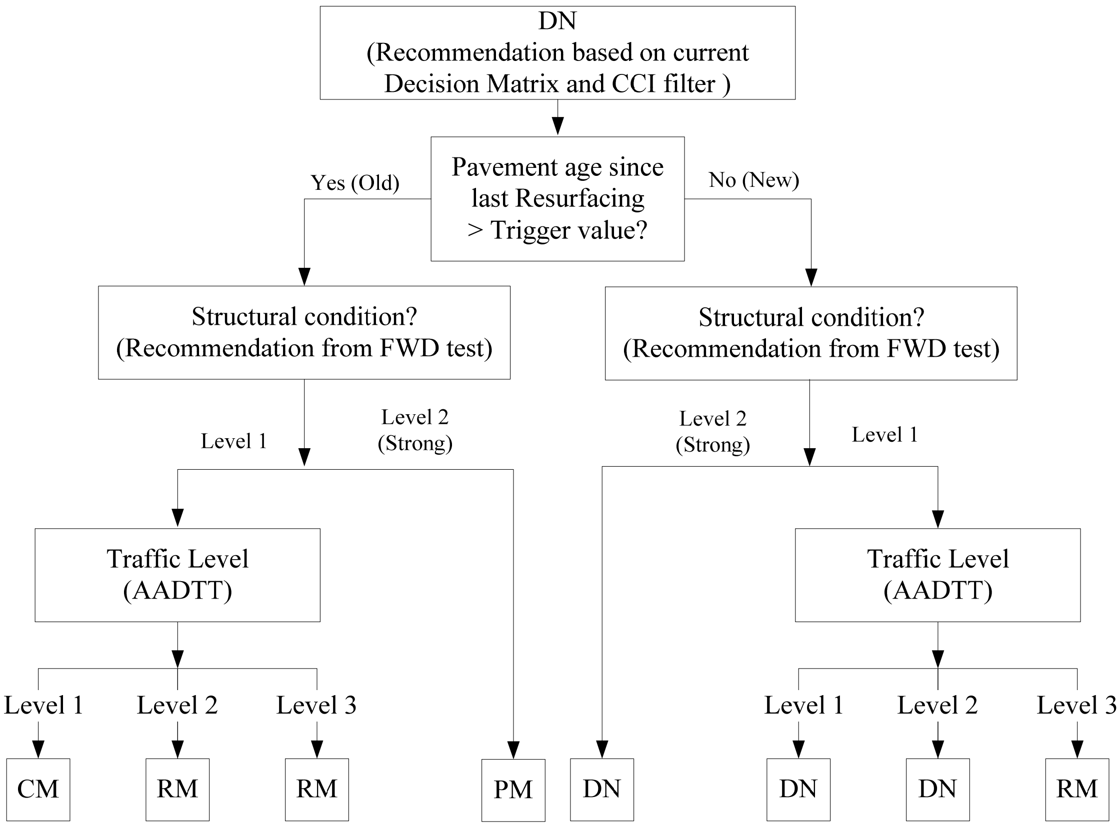

Figure 2. Augmented Decision Tree for IS (BIT/BOC/BOJ) With "Do Nothing" (VDOT, 2008)

Table 6. Trigger Values to Use with the Enhanced Decision Tree (VDOT, 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* $AREA \geq 32$ *in* & $k \geq 175$ *pci*		*Otherwise*
Traffic (AADTT)	Level 1	Level 2	Level 3
	< 1500	[*1500, 5000*]	*> 5000*

The enhanced decision trees for rigid interstate pavements do not include pavement age, only pavement strength and traffic. Furthermore, the only times that the strength of a rigid pavement impacts the maintenance decisions are for the following cases on interstates: (1) a CRC pavement with initial decision of CM and an area under the deflection basin (AREA) less than 32 in (813 mm) or center deflection (D_0) greater than 5 mils (127 microns), (2) a JRC or JCP pavement with initial decision of PM and an AREA less than 32 in (813 mm) or D_0 greater than 5 mils (127 microns), and with traffic greater than 5,000 AADTT, and (3) a JRC or JCP pavement with initial decision of CM and an AREA less than 32 in (813 mm) or D_0 greater than 5 mils (127 microns). In each of the cases, the selected maintenance treatment is increased by one category.

The current enhanced decision trees were found to have some potential shortcomings. First of all, it was determined that the strength of the pavement should not be classified as either the effective structural number or the resilient modulus. The main reason is that an inadequate strength of subgrade is often compensated for during the pavement design process by increasing the structural number of the pavement. Secondly, the traffic should not be discretized into bins, but should be used to directly calculate the required structure of the pavement. Finally, it was recognized that a continuous structural index, as opposed to discrete pavement strength classifications, could be advantageous when developing pavement management applications.

METHODS

Development of a Structural Index for Flexible Pavements

Several network-level structural indices for flexible pavements were identified in the literature review. 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 found in the literature review that have been developed for implementation on a network wide scale.

Structural Adequacy Index

The structural adequacy index (SAI) was an early index developed by Haas (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 Equivalent 18-kip (8,100 kg) Single Axle Loads (ESALs) (Haas 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 1, 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.

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 lb load level (Zhang et al., 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.

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.

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 et al., 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 equation:

$$D_{1.5Hp} = \frac{(x-B)*(x-C)}{(A-B)*(A-C)}*(D_A) + \frac{(x-A)*(x-C)}{(B-A)*(B-C)}*(D_B) + \frac{(x-A)*(x-B)}{(C-A)*(C-B)}*(D_C) \quad \text{(Eq. 1)}$$

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:

$$SIP = D_0 - D_{1.5Hp} \quad \text{(Eq. 2)}$$

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:

$$SN_{eff} = k1 * SIP^{k2} * H_p^{k3} \quad \text{(Eq. 3)}$$

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

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

$$M_R = \frac{C * P * 0.24}{D_r * r} \quad \text{(Eq. 4)}$$

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 7.

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

$$SCI = \frac{SN_{Eff}}{SN_{Req}} \quad \text{(Eq. 5)}$$

Table 7. Required SN for M_R and Required Traffic (Zhang et al., 2003)

			20 Year Accumulated Traffic in ESALs					
			Category	**Very Low**	**Low**	**Medium**	**High**	**Very High**
			Range	**50,000-945,000**	**945,000-1,687,000**	**1,687,000-2,430,000**	**2,430,000-3,172,000**	**3,172,000-50,000,000**
Resilient Modulus, M_R (psi)	**Category**	**Range**	Average	498,000	1,316,000	2,059,000	2,801,000	26,586,000
	Low	**1,000-5,400**	3,200	4.3	5.1	5.3	5.6	7.1
	Medium	**5,400-7,500**	6,400	3.5	3.9	4.2	4.3	6.0
	High	**7,500-40,000**	24,000	2.3	2.6	2.8	2.8	3.9

Structural Strength Indicator

The Structural Strength Indicator (SSI) was proposed in 2009 as a comparative index that is bounded and uses center 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 6 and is in the form of Equation 7 (Flora, 2009).

$$SSI = 100 * \left[1 - F\left[\left(\delta_{i\,jk}\right)_1\right]\right] \quad \text{(Eq. 6)}$$

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

$$SSI_{jk} = 100\left(1 - \alpha e^{\frac{-\beta}{(d_1)^{\gamma}}}\right) \quad \text{(Eq. 7)}$$

where δ is the center deflection, 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 Equations 6 and 7.

The basis for the SSI as developed by Flora (2009) 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 8 shows threshold values suggested by Flora (2009).

Table 8. 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

Remaining Service Life

The Kansas Department of Transportation and researchers from Kansas State University 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 RSL is the anticipated number of years left in a pavement's functional service life. The RSL employs sigmoidal performance models, and the center deflection of the FWD to predict a pavement's 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:

$$RSL = \delta + \frac{\alpha}{1 + e^{\beta - \gamma * d_0}} \qquad \text{(Eq. 8)}$$

where d_0 is the temperature corrected center deflection from the FWD, the variables α, β, γ and δ are best-fit linear regression models for different pavement types considering the following independent variables: pavement depth, Equivalent Axle Load per day, Equivalent Transverse Cracks (a specific coded measure of transverse cracks used by Kansas DOT), Equivalent Fatigue Cracking (a specific coded measure of fatigue cracking used by the Kansas DOT), rut depth and SN_{eff} (effective structural number of the pavement).

RESULTS

Preliminary Screening of the Flexible Pavement Structural Indices

An internal study performed by the Texas Department of Transportation found that the StSI was not sensitive enough to distinguish significantly different pavements in terms of distresses. A report by Zhang et al. (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 StSI 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 can be reproduced, 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 research. 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., 2010a). Thus, this methodology could 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 research.

Assessing the Correlation of Deflection Measurements to Pavement Condition Rating

Using deflection data and pavement condition data collected on a section of 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 and the SCI 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 correlation coefficient is defined as (Zimmerman et al., 2003):

$$\rho_{xy} = 1 - \frac{6 * \sum_{i=1}^{n} d_i^{\,2}}{n(n^2 - 1)}, \text{ For sufficiently large n values (n\sim>30)} \qquad \text{(Eq. 9)}$$

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

$$d_i^{\,2} = (X_i - Y_i)^2, \ X_i \text{ and } Y_i \text{ are the samples} \qquad \text{(Eq. 10)}$$

To test whether there is any correlation between the measures, the following hypotheses were tested and the results are tabulated in Table 9:

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

H_1: $\rho \neq 0$

As can be seen in Table 9, there is a significant 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 between the IRI and SSI. However, 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 3 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. On the other hand, 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 4 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 Figures 3 and 4 are flexible.

The figures seem to indicate that for pavements in very poor functional condition (i.e., poor LDR), 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.

Table 9. Correlation Between Structural and Functional Parameter

	Distress	Sample Size	Spearman Rank Coefficient	Reject H_0 at 95%[1] (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)

[1]Rejecting H_0 indicates that it is not possible to say within the statistical constraints that there is not a correlation between functional distresses and structural condition indicators.

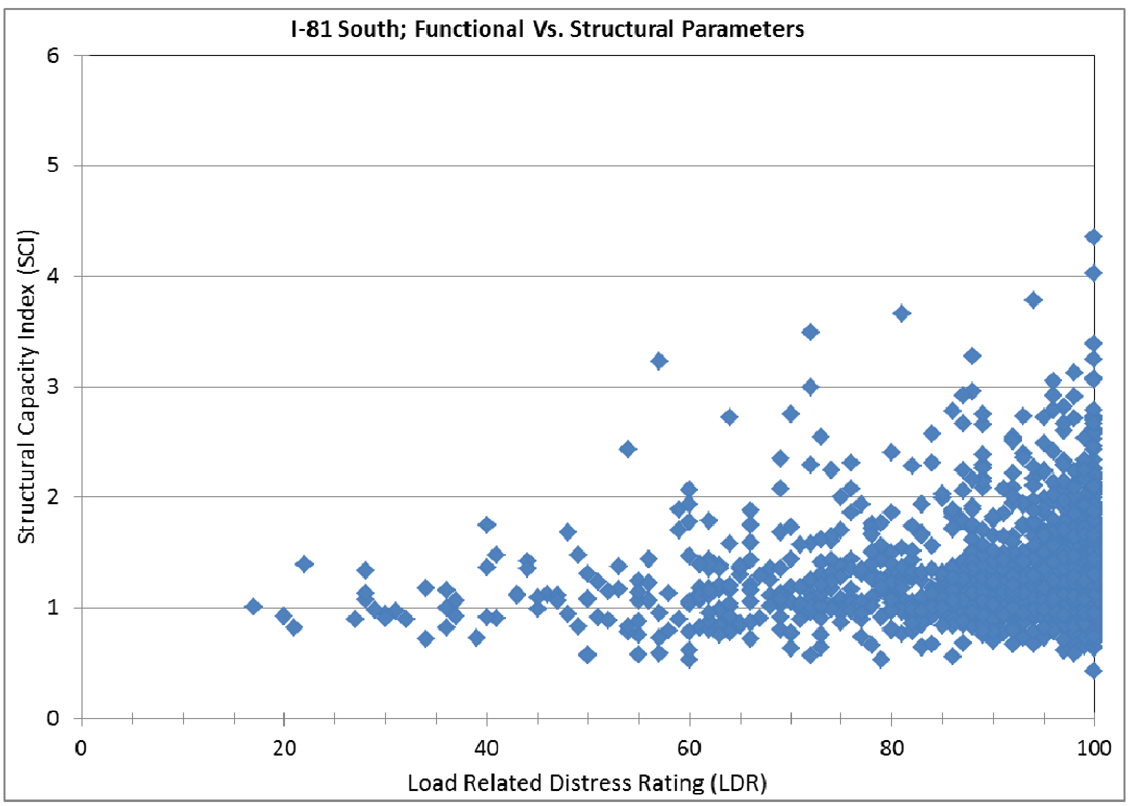

Figure 3. SCI vs. LDR on I-81 Southbound

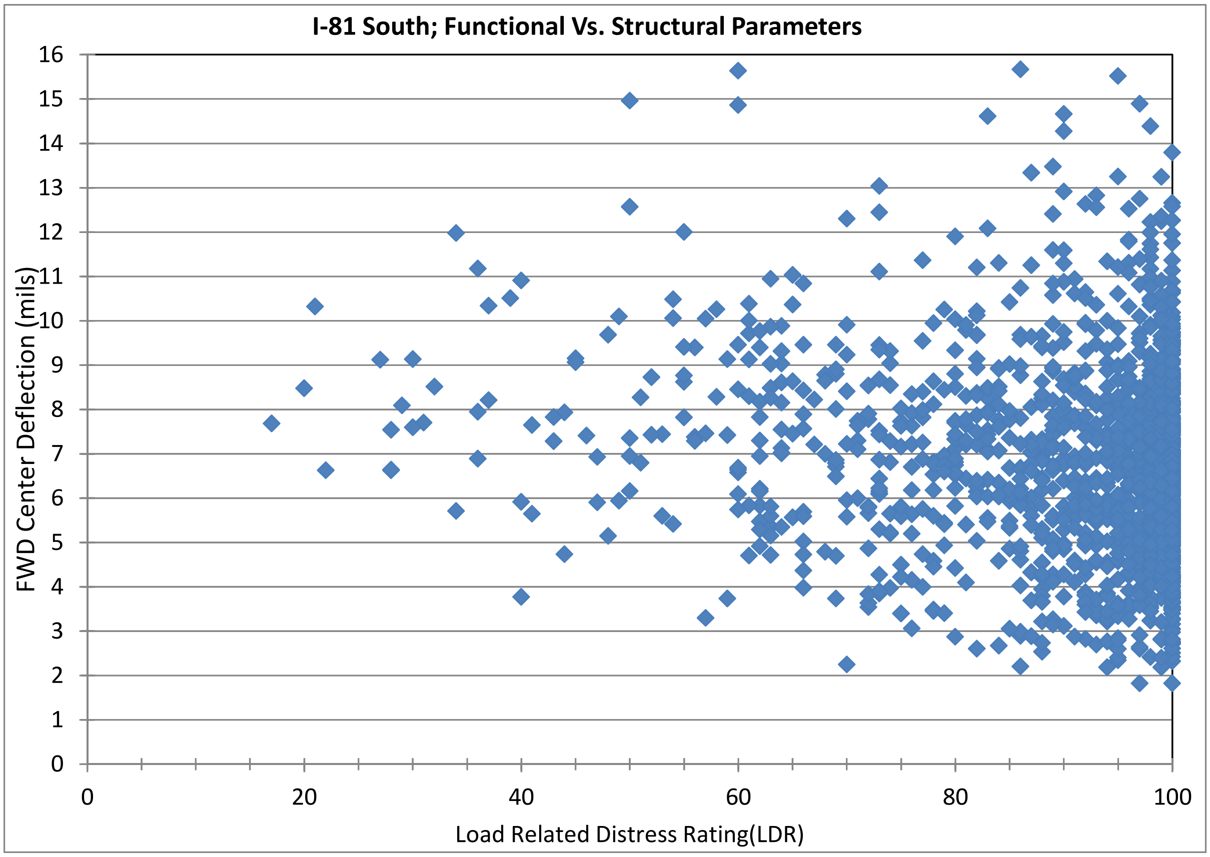

Figure 4. FWD Center Deflection vs. LDR on I-81 Southbound

Methodologies for Implementing the Flexible Pavement Indices

The two indices that were chosen for further evaluation, the SCI and SSI, were further analyzed, and modified in order to match project data that were 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.

Structural Strength Indicator Methodology

The initial step in determining the SSI was to determine the standard normal cumulative distribution (Figure 5). 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 7. The minimized errors yielded the following function for the SSI along the bituminous interstate for I-81 Southbound:

$$SSI = 100\left(1 - 1.0069e^{\frac{-1071.8}{(d_1)^{3.9622}}}\right) \qquad \text{(Eq. 11)}$$

where δ_1 is the FWD center deflection.

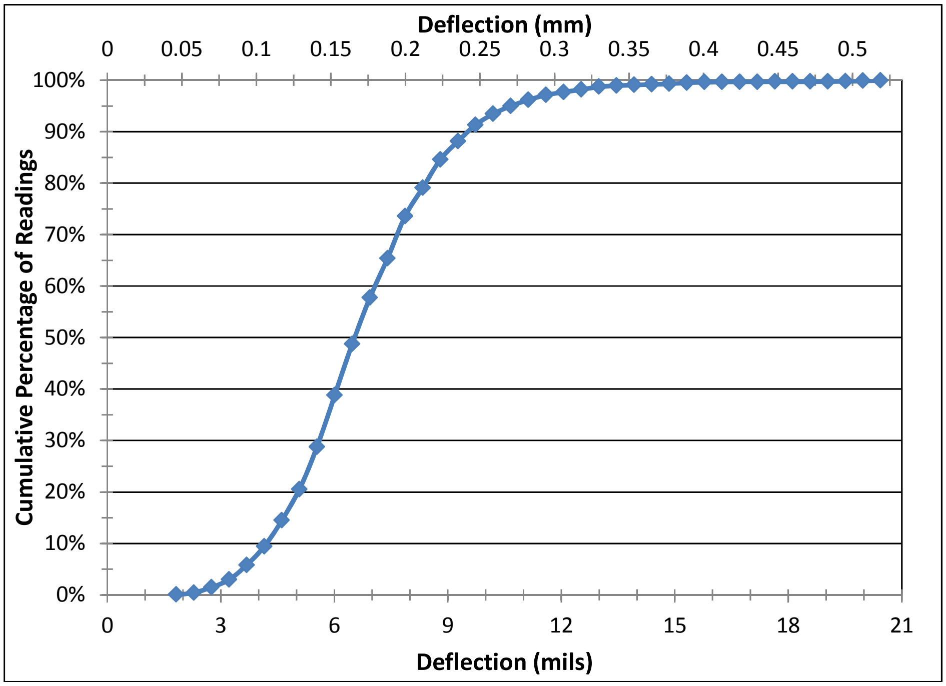

Figure 5. Cumulative Distribution of Deflections Along I-81 Southbound

SSI Thresholds

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 10 and the SSI equation for I-81, a new set of SSI thresholds was calculated and included in the last row of Table 10.

Table 10. 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 ESALs, and ranked corresponding deflections using a set of 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.

Structural Capacity Index Methodology

The methodology for calculating the SCI was presented for flexible pavements in a previous section of this report. It is possible that a similar methodology can be developed for rigid and composite pavements. 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 was modified for further analysis in this research, thus it will be distinguished by using the term Modified Structural Index (MSI). This will also help to eliminate confusion of the SCI with another common term in deflection testing, the Surface Curvature Index (SCI_{300}).

The modification of the SCI included developing a closed form method for estimating the required structural number, and combining it with Equations 3 and 5 to form a continuous equation. The details of the modification can be seen in Appendix A. The final form of the MSI is shown in Equation 12.

$$\mathrm{MSI} = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05716 * (\log(ESAL) - 2.32 * \log(M_R) + 9.07605)^{2.36777}} \qquad \text{(Eq. 12)}$$

Integration of MSI with Other Condition Indices

The MSI 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 MSI 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 MSI with bounded condition indices by providing ranges in which a certain structural condition may be defined.

Comparing the Flexible Pavement 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 automated data collected and aggregated into 0.1 mile sections on average over the I-81 corridor throughout Virginia. The condition data were 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 is performed in order to determine the actual critical values that should be used.

MSI 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 MSI were developed directly from applying the trigger values used by VDOT from the enhanced decision trees. This methodology was selected because the main inputs for the MSI 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:

$$\mathrm{SCI} = \frac{\mathrm{SN_{Eff}}}{\mathrm{SN_{Req}}} \qquad \text{(Eq. 13)}$$

where SN_{eff} is determined by deflection testing, and the SN_{req} is determined from traffic and resilient modulus data. Substituting the SN_{req} for Equation A3 developed in Appendix A, Equation 13 becomes:

$$\mathrm{MSI} = \frac{\mathrm{SN_{Eff}}}{0.05716 * (\log(\mathrm{ESAL}) - 2.32 * \log(\mathrm{M_R}) + 9.07605)^{2.36777}} \qquad \text{(Eq. 14)}$$

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 (in terms of AADTT) of 0 to 1,500, 1,500 to 5,000, and greater than 5,000 represent low, intermediate and high traffic levels, respectively (VDOT 2008). In order to convert the AADTT into ESALs, the following equation was used:

$$ESAL = (AADTT)(T_f)(G)(D)(L)(365)(Y) \qquad \text{(Eq. 15)}$$

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 (Smith and Diefenderfer, 2009), and was ultimately taken to be 0.96 for the combination of trucks on I-81. The directionality factor was taken as 0.5, and the lane factor was taken as 0.9 (VDOT, 2003). Substituting the VDOT trigger values and traffic into the MSI equation, the following thresholds were developed for MSI: 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 MSI thresholds were compared to the traffic range over the

I-81 corridor through Virginia and can be seen in Figure 6. The traffic ranges for I-81 all fall within the high and medium traffic categories.

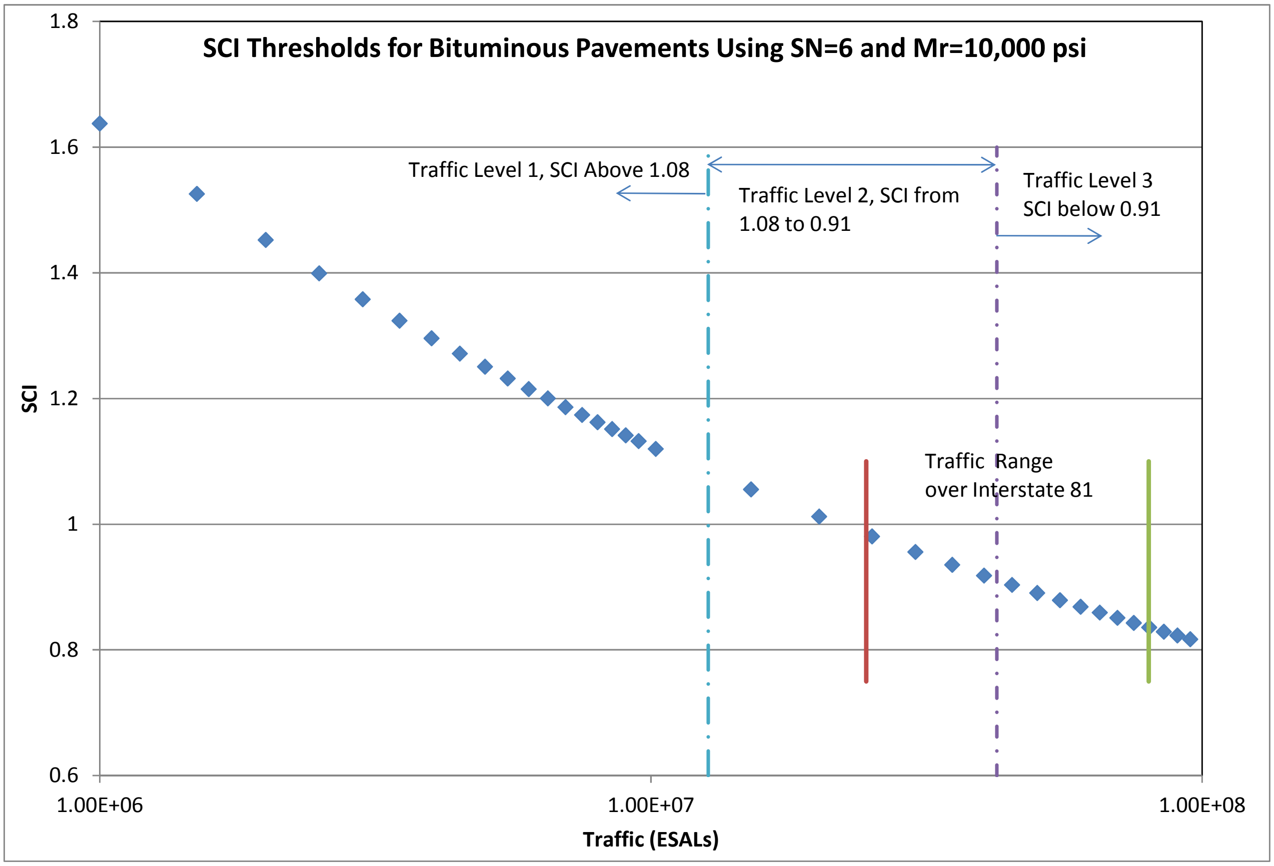

Figure 6. MSI Threshold Values for Bituminous Interstate Pavements Using VDOT Trigger Values

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 7.

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.

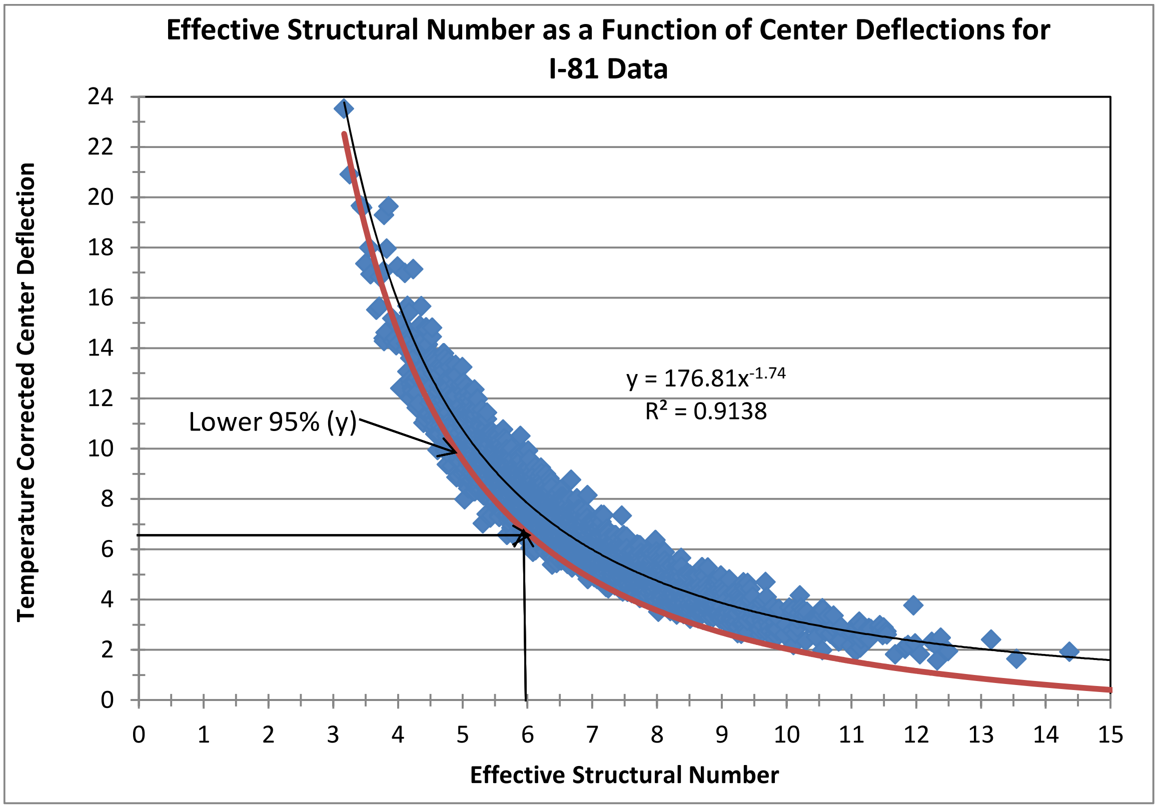

Figure 7. Trend of SN_{eff} As a Function of D_0

Selection of the Most Appropriate Index

The selection of the most promising structural index was done by comparing network-level predictions incorporating each of the structural indices to actual project-level work. The project-level work was obtained from district level work orders for specific project contracts that were awarded in 2007. The network-level data were collected from the VDOT pavement management system. The details of the work can be seen in Appendix B.

In two of the three cases presented in Appendix B, the methodology implementing the MSI at the network-level proved to most closely predict the project-level work. In the third case, the method based on the center deflection matched the MSI 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 MSI 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 MSI methodology was selected for implementation to interpret network-level deflection data for flexible pavements. Furthermore, the MSI methodology can be easily programmed into spreadsheet format to provide a quick and relatively simple method for obtaining an index. Also, it was determined that the SSI methodology does not provide adequate discrimination between traffic levels for accurate evaluation.

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

	Project-Level Work Done	VDOT Enhanced Decision Tree	Center Deflection	MSI
DN	-	-	0.2	0.2
PM	-	1	1	1
CM	11.45	8.02	9.02	9.62
RM	4.17	5.1	4.5	3.5
RC		1.5	1.1	1.5

Sensitivity Analysis of the MSI

The sensitivity of the MSI to the various input parameters was evaluated. Each of the inputs was varied, and the change in the calculated MSI was obtained. Two baseline values were chosen for the sensitivity analysis based on MSI values of sections of pavement with different known structural conditions, a case that yields a MSI of 1.02, and a case that yields a MSI of 1.51. The two baseline values were also indicative of typical values found along I-81 Southbound in Virginia. Each of the deflection values was varied by 10%, the pavement thickness was varied by 10%, and the traffic was varied by 50%. The results are presented in Figure 8. The percent change in the MSI was the same regardless of the baseline value (1.02 and 1.51 in this case).

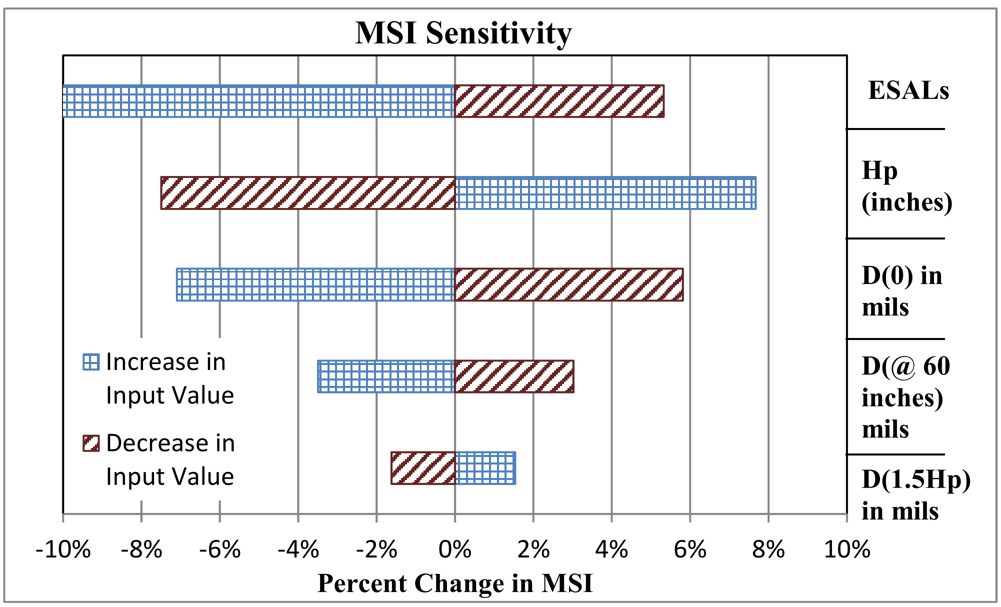

Figure 8. Sensitivity Analysis of the MSI Function for Interstates

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 8.

Sensitivity of the Threshold Values

The MSI threshold values were obtained based on the enhanced decision tree trigger values for the structural number (SN) and subgrade resilient modulus (M_R). However, this results in an interstate MSI threshold for maintenance application greater than one. A value greater than one is conflicting with the principle behind the MSI; the MSI is the ratio of effective structural number over the required structural number. If this ratio is greater than one, then it is expected that no structural rehabilitation is required as the effective structural number is greater than the required structural number. To evaluate the effect of threshold on MSI, two sensitivity analyses were performed. In the first sensitivity analysis, the MSI threshold was varied to match the predicted work obtained from the VDOT enhanced decision tree methodology. In the second sensitivity analysis, the MSI threshold was varied to match the actual performed work detailed in the project work order.

Sensitivity Analysis of Threshold Using VDOT Trigger Values

The MSI thresholds were analyzed by varying the values of the thresholds, 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 9.

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 reflects the fact that although the MSI threshold was set based on trigger values from the enhanced decision matrix, the two procedures are not identical but give similar results. It is also thought that this discrepancy may be due to treating the resilient modulus and structural number as independent indicators of strength like the VDOT method currently treats them. The MSI method incorporates the resilient modulus and structural number by using the AASHTO relationship between them.

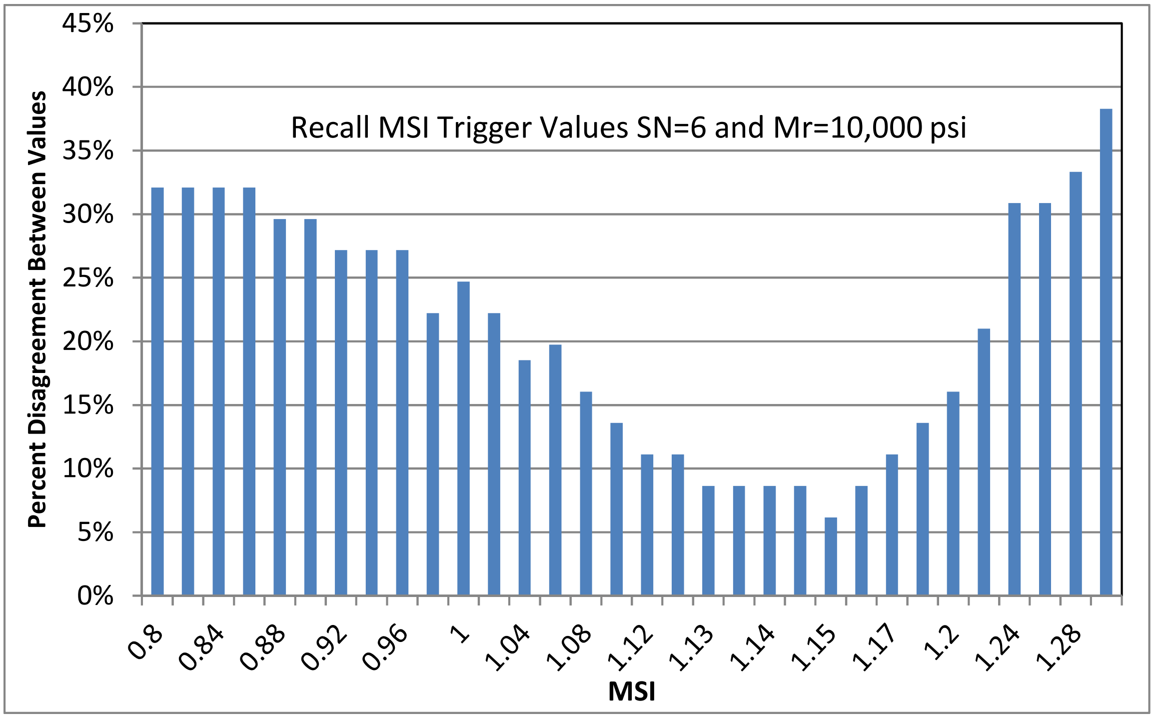

Figure 9. Sensitivity Analysis of Threshold Using VDOT Trigger Values

Sensitivity Analysis of Threshold Using Only Project Data

An analysis was also conducted to determine the threshold that results in treatments that best match actual project-level treatments. The MSI was varied and the difference between the total treatment category (in terms of length of applied treatment) predicted from the MSI and actual total treatment category was evaluated. The results can be seen in Figure 10.

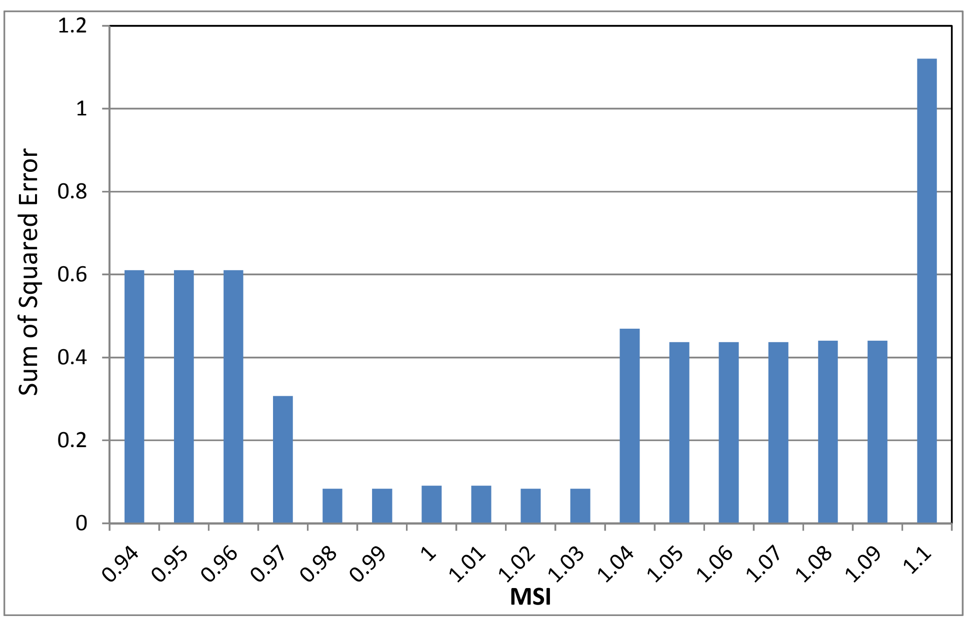

Figure 10. Sum of Squared Error Between Total Category Length Predicted by the MSI and the Total Category Length From Project Data – 20 Year Design Period

It can be seen in Figure 10 that the minimum difference between the predicted work and actual work falls between an MSI value of 0.98 and 1.03. Figure 11 shows the sum of the errors over a narrower interval. The total error is minimized for MSI values between 1 and 1.015. This suggests that a trigger value of 1 for the MSI could be selected. Note that the actual work performed at the project-level takes into account many other parameters that are not considered at the network level. Therefore, network-level decisions will not always match project-level decisions; however a good network-level decision methodology should, on average, give results that reasonably match project-level decisions. The limited cases analyzed suggest that the MSI is capable of producing such network-level decisions.

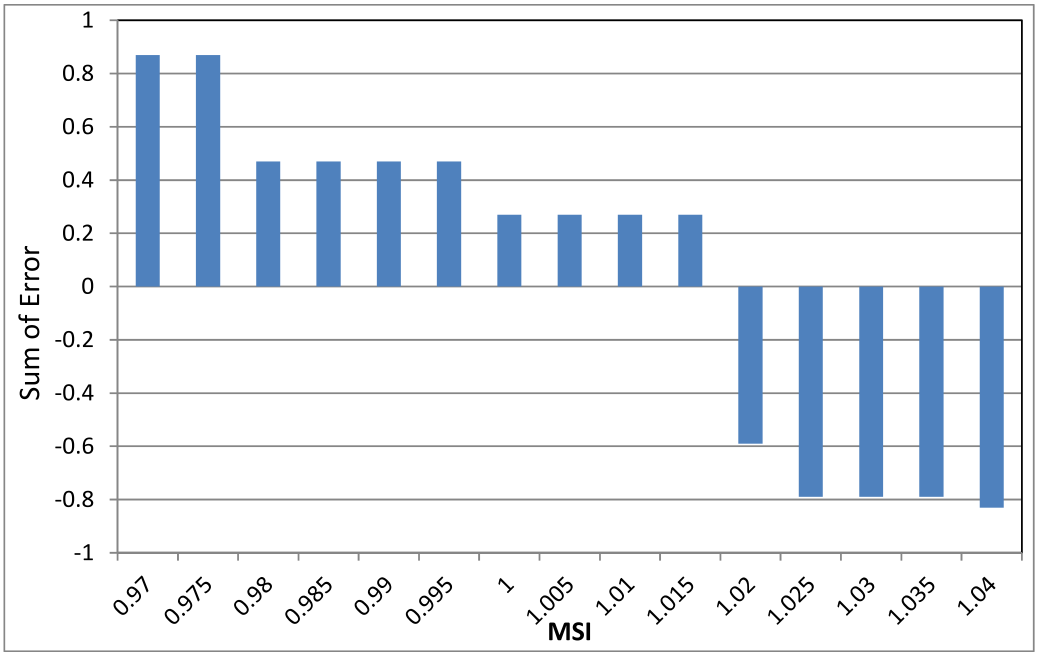

Figure 11. Sum of Errors Between Total Category Length Predicted by the MSI and the Total Category Length From Project Data – 20 Year Design Period

Example Applications

A number of example applications and case studies using the MSI as a network-level tool were developed to demonstrate the utility of the index developed. It is important to recognize that these applications, as is the case throughout this report, have been developed using a 20-year design period for the ESAL calculations. Use of the MSI in network-level analysis, as well as its integration into decision making is presented in the following sections. The applications are specific to the pavement management process in Virginia, but can feasibly be applied to other state pavement management programs that incorporate network-level deflection testing with minor modifications.

Integration of the MSI into the Project Scoping Decision Process

The current VDOT decision process incorporates deflection information into an enhanced decision tree after an initial decision is made based on surface distresses. The enhanced decision tree incorporates additional pavement related information which is: the age of the pavement, the traffic levels, the structural number of the pavement, and the subgrade resilient modulus. Given that the MSI incorporates traffic levels, the structural number of the pavement, and the resilient modulus of the pavement, the enhanced decision tree can be modified to incorporate the MSI. This modified decision tree is shown in Figure 12.

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 possible treatments categories based on MSI and its threshold represent three potential levels of structural deficiency. The first treatment category is used for MSI values that exceed the selected threshold for the pavement, in which case it is structurally adequate. The second and third treatment categories are used for MSI values lower than the selected threshold. The parameter α that is used to differentiate between the second and third treatment categories reflects whether a pavement section is deficient (second treatment category) or severely deficient (third treatment category).

To determine an appropriate value for α, the MSI of flexible pavement sections along I-81 was calculated using a 20 years design period. A plot of the calculated MSI for 325 miles of I-81 Southbound can be seen in Figure 13. A particular pavement location between mileposts 213 and milepost 217 on I-81 Southbound was recently reconstructed and was considered in very poor structural condition. The MSI along this pavement section prior to reconstruction was determined to also be the worst case MSI along the 325 mile section of the interstate. The distresses along the pavement included cracking that extended through the full pavement depth, extensive rutting, and extensive patching (VDOT, 2011b). Thus it was determined that a full reconstruction was needed.

The MSI values averaged over the weakest two sections (the area of the reconstruction project) 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). The percentage of MSI values less than 0.9 for the flexible pavement section is 15%. However, when the MSI is averaged over structurally homogenous pavement sections, defined by circular binary segmentation, only 2% of the pavement is less than 0.9. Based on these results, a value of $\alpha = 0.9$ could be recommended to characterize structurally deficient sections. Using the above information, as well as the current VDOT enhanced decision trees, a decision matrix can be developed to account for the MSI in decision making. The decision matrix is shown in Table 12.

MSI As a Structural Screening Tool

Another potential application of the MSI is as a network-level structural screening tool. The structural condition of I-81 Northbound in Virginia is shown in Figure 14. Seven potentially critical locations in terms of the MSI were identified and labeled in the figure for further investigation. The seven locations are ones whose MSI falls below 1 for a considerable length or falls below 0.9 for any length. 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 Figure 14.

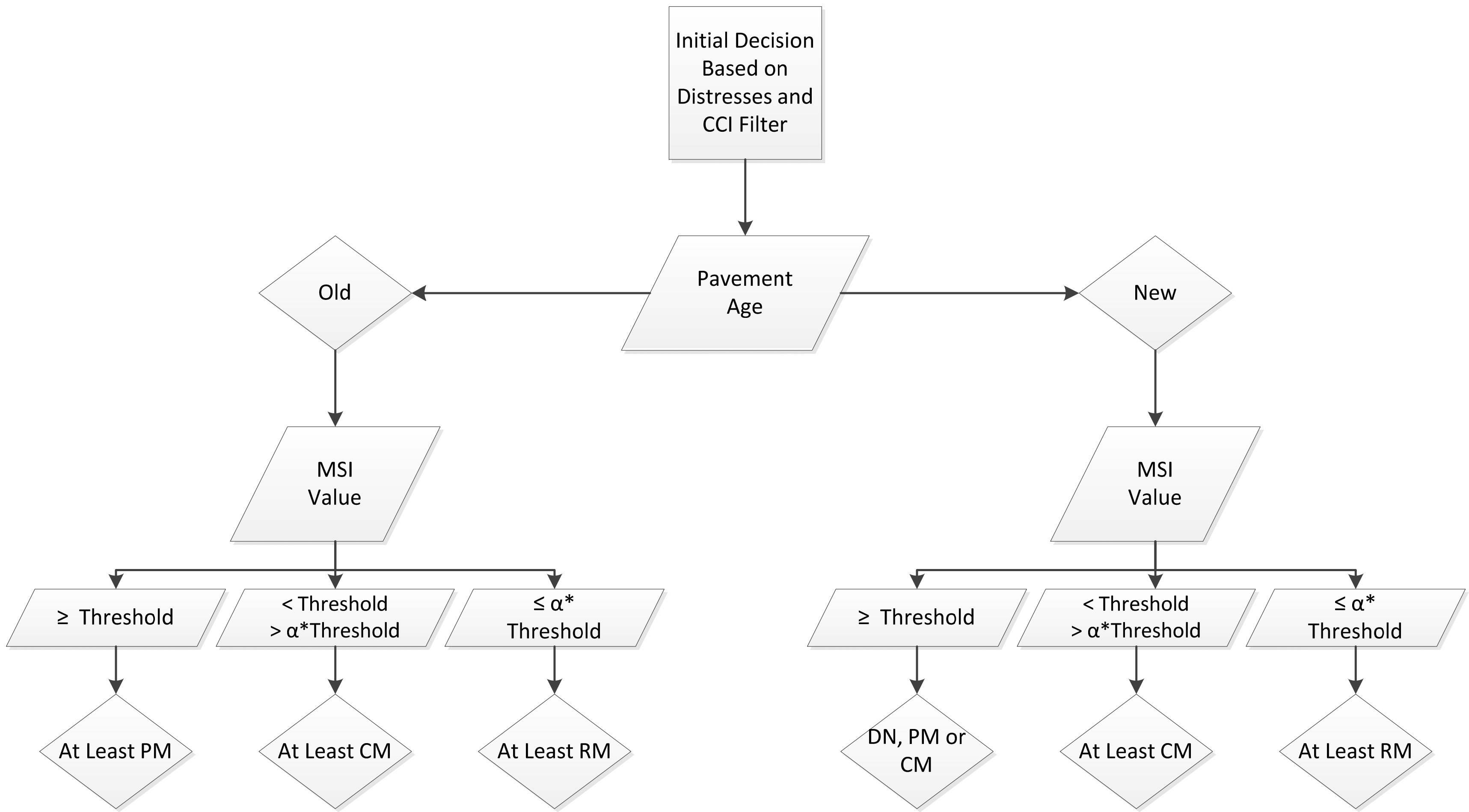

Figure 12. Decision Process Based on MSI for Bituminous Pavements

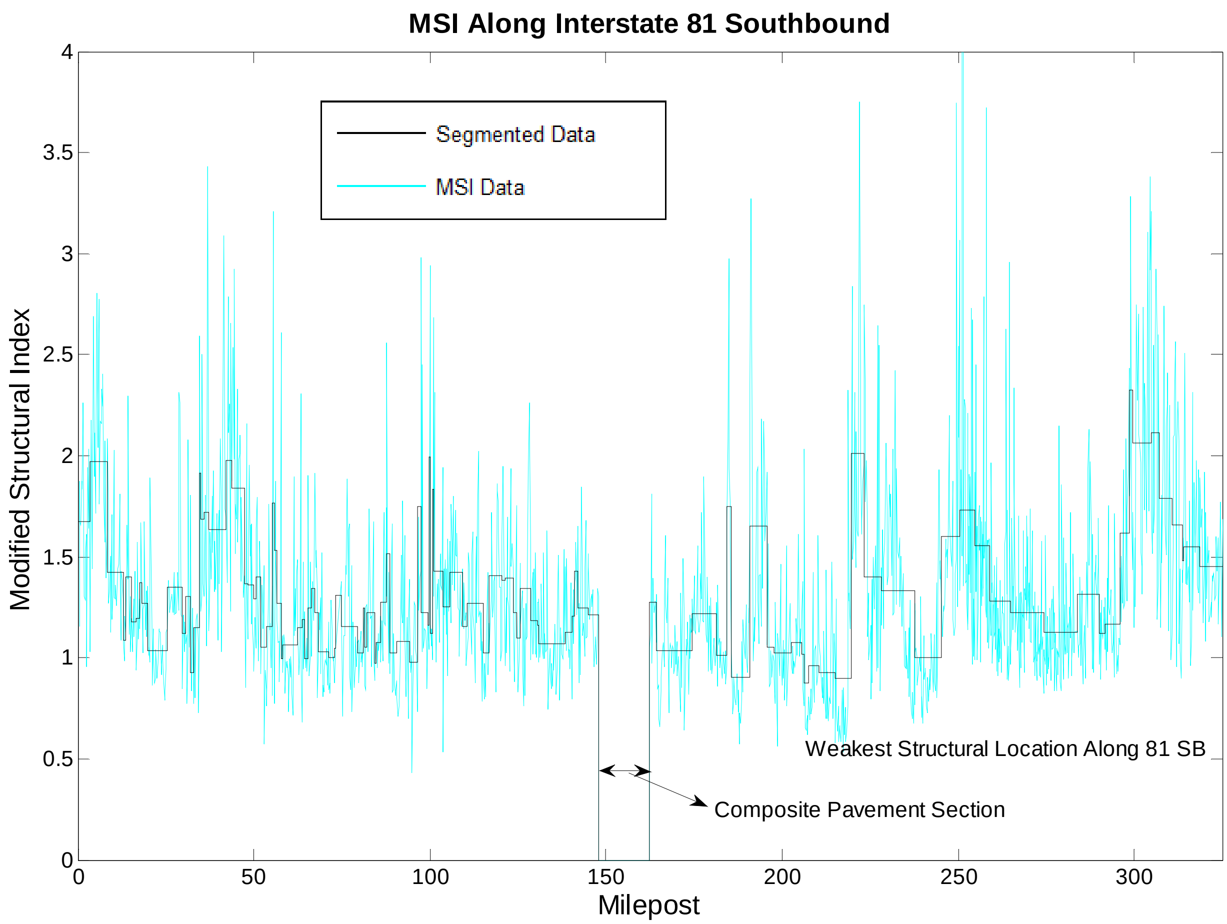

Figure 13. MSI for I-81 Southbound at 0.2 Mile Intervals and Segmented Using Binary Segmentation

Table 12. 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**
MSI	**≥ 1**	DN	PM	PM	PM	CM	CM	RM	RM	RC	RM
	< 1 and ≥ 0.9	CM	RM	CM	RM	RM	RM	RC	RC	RC	RC
	< 0.9	RM	RM	RM	RM	RC	RC	RC	RC	RC	RC

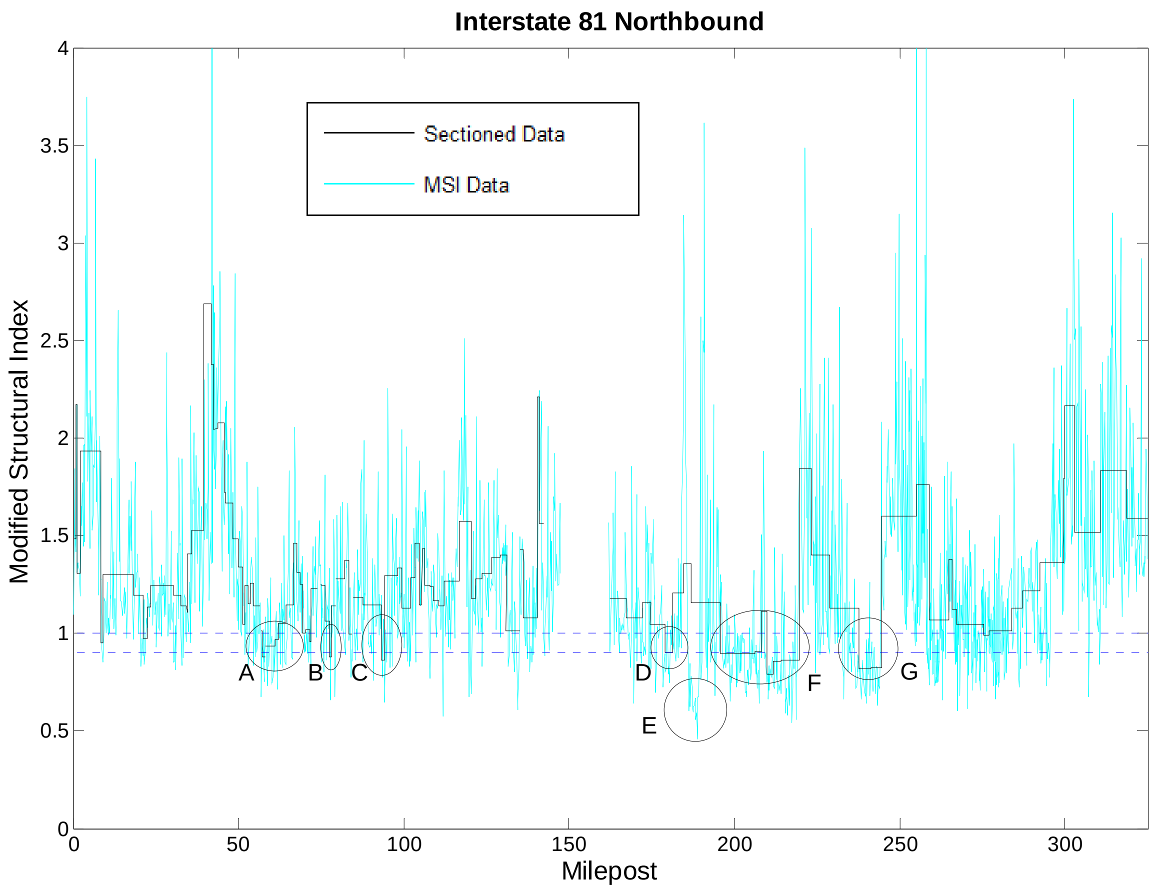

Figure 14. MSI Along I-81 Northbound in Virginia

The first section, labeled A in Figure 14, 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 15. It is clear from the figure (i.e., at mile marker 58.7 and 59.9) 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 13. 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 13 that accounts for structural information, the decision should have been at least restorative maintenance. The fact that corrective maintenance was applied when structural information in the form of the MSI suggests 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 13).

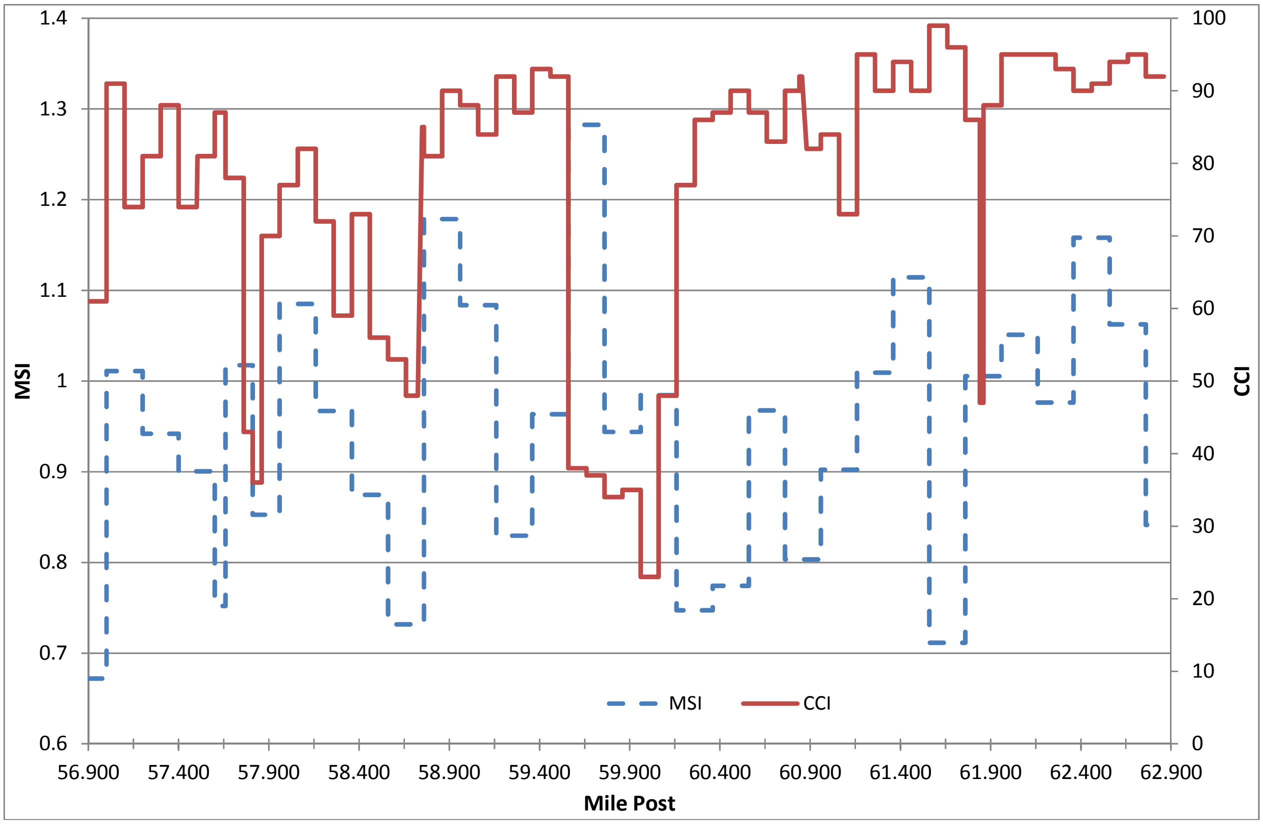

Figure 15. MSI and CCI Along a Section of I-81 Northbound

Table 13. 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. (Sev. 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 14 are the sections labeled B and F, which coincide with pavement sections where I-81 overlaps with Interstate 77 and Interstate 64, respectively. It is expected that 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.

Example 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 16 can be employed. Equation 16 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:

$$d = \frac{SN_{\mathrm{Req}} - SN_{Eff}}{0.44 * (1 - c)} \qquad \text{(Eq. 16)}$$

where d is the required overly in inches, 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 was calculated for I-81 Northbound. 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 16. It can be seen in Figure 16 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 previous sections of this report.

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).

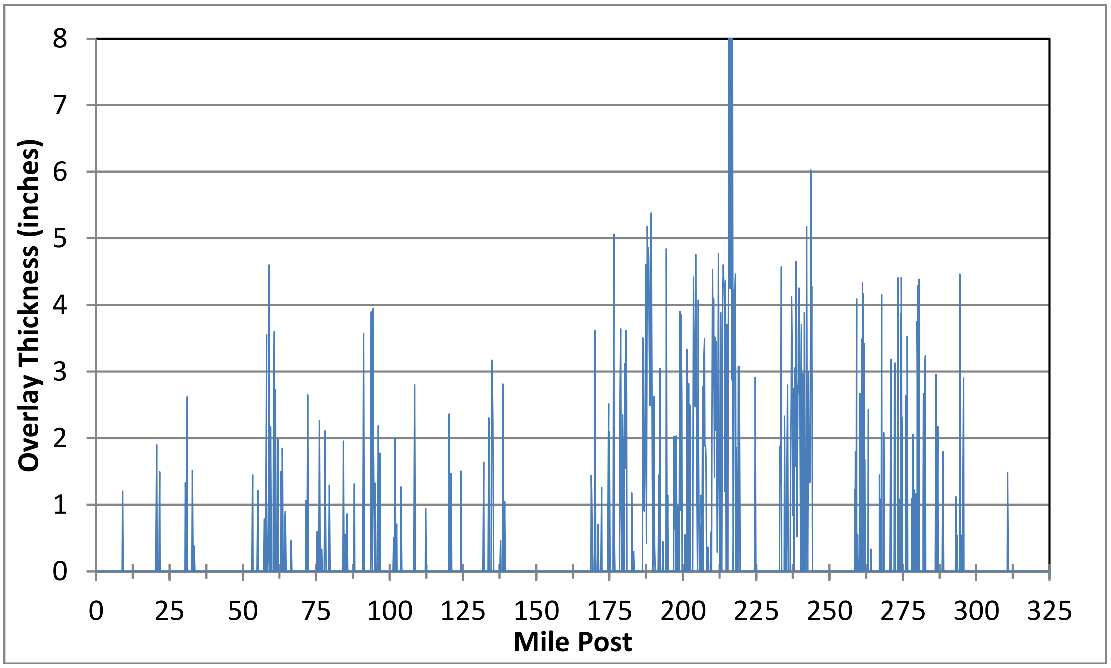

Figure 16. Estimated Overlay Thickness Along I-81 Northbound in Virginia

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.
- *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.

I-81 Performance

In order to demonstrate the use of the MSI as a performance indicator, the MSI was compared against the condition of I-81 in terms of the CCI. The 2007 condition data were 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 17). Thus, the performance indicator for condition would be 87.9.

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

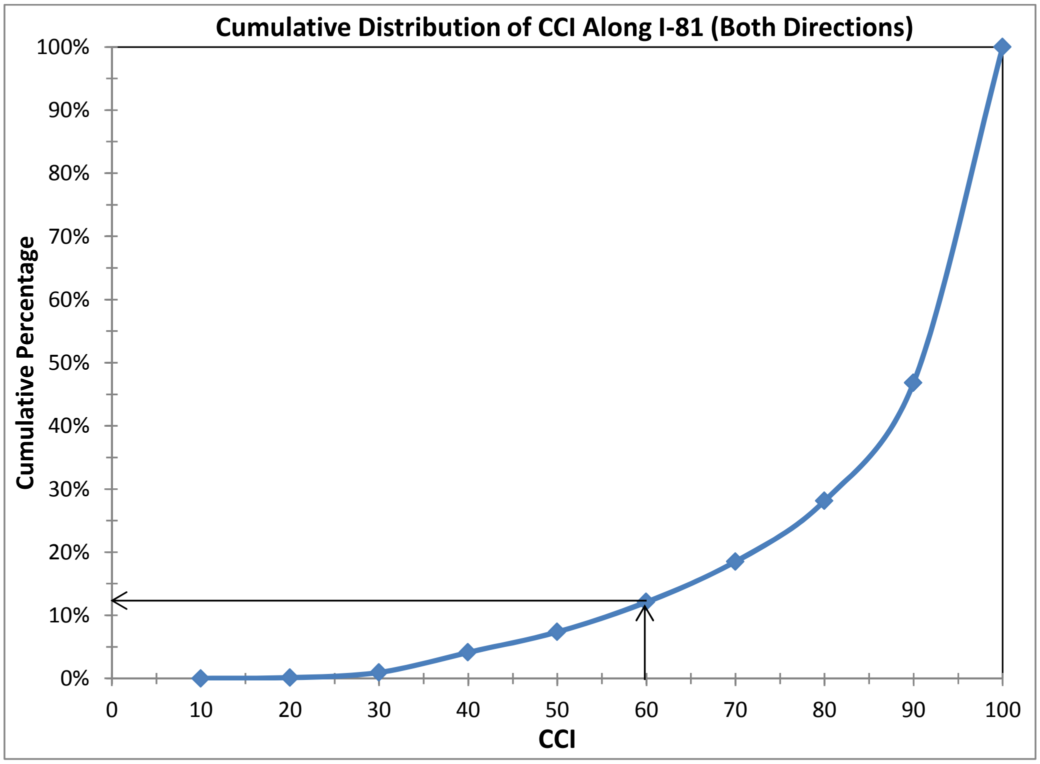

Figure 17. Condition of I-81

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. For a 20 year design period and an MSI threshold of 1 as is shown in Figure 18, the performance indicator is shown to be 100% - 27.3% = 72.7% of pavements in structurally adequate condition.

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. This can be seen in the CCI trend in Figure 19 where two pavement sections with similar CCI values but different MSI values were treated in 2008. The two sections of pavement are located in the same maintenance district in Virginia on I-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.

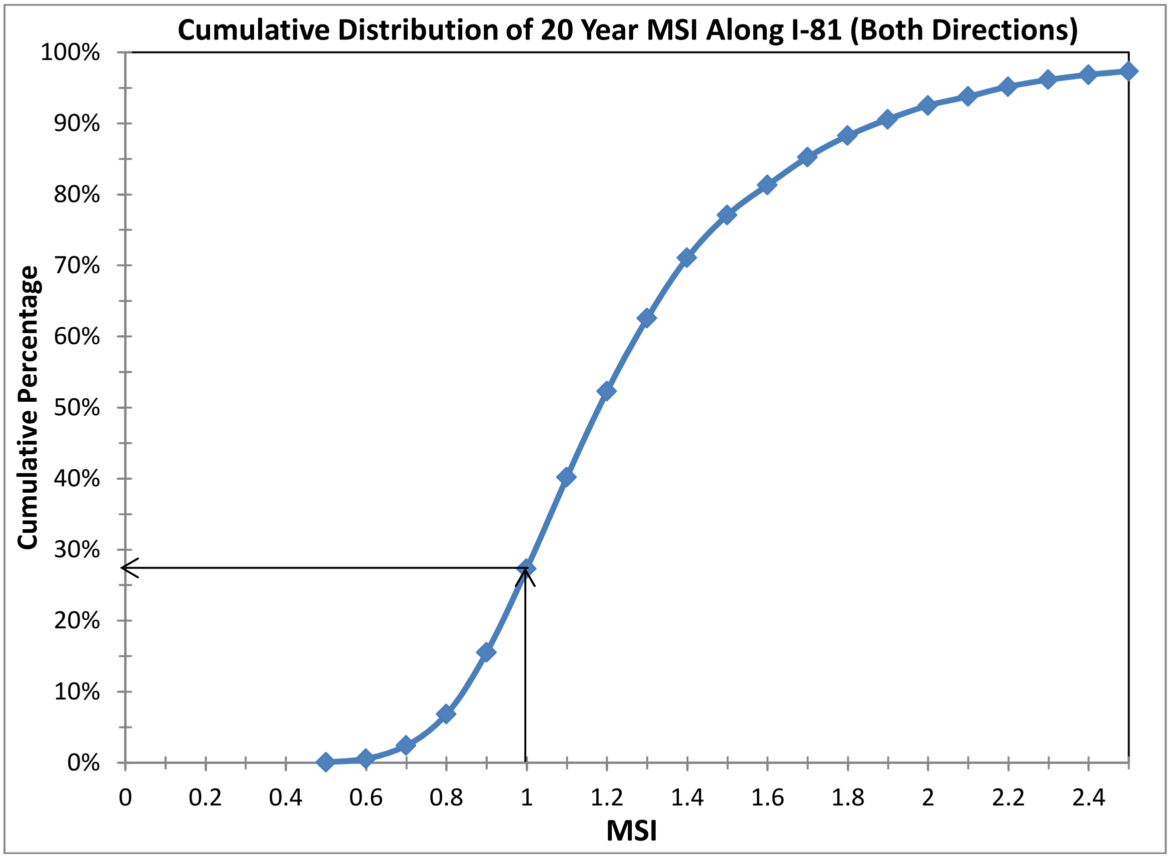

Figure 18. Cumulative Distribution of MSI

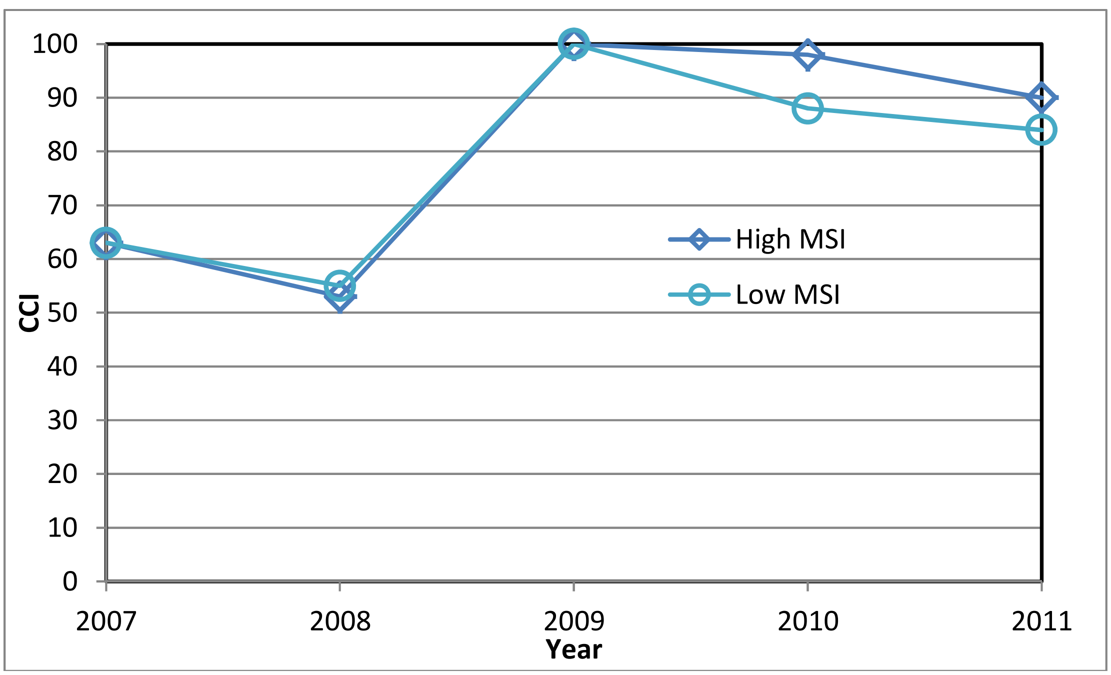

Figure 19. CCI for 2 Pavement Sections With Different MSI Values

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, the deterioration of a set of pavement sections which received corrective maintenance treatment was evaluated. Work that was completed in the year prior to the network-level deflection testing was gathered from the construction history in the VDOT database. The data from the year prior to deflection testing were important because, in these cases, the structural condition was measured post-treatment. Twelve sections along Interstates 81 and 95 were investigated, 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 were also available for up to five years following the treatment for the majority of these sites.

VDOT has developed a set of deterioration curves for particular treatments. Equation 17 shows the deterioration curve used for corrective maintenance performed on bituminous pavements.

$$CCI(t) = 100 - e^{a-b*c^{LN(\frac{1}{T})}} \qquad \text{(Eq. 17)}$$

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 is applied.

For relatively short time periods, (e.g. 1 year or less), variable *a* controls the behavior of Equation 17. However, as time increases, Equation 17 becomes much more sensitive to changes in variables *b* and *c*. Figure 20 shows the actual deterioration of two sections with different average MSI values along with the predicted deterioration using Equation 17 (dashed line). Although the deterioration predicted using Equation 17 follows the general deterioration trend of both sites, a better deterioration trend can be obtained by allowing the parameter *c* in Equation 17 to depend on the MSI as shown in Figure 20.

The change in *c* as a function of the MSI was investigated further by sorting the sites into bins of similar MSI values 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 21.

An interesting note from Figure 21 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.

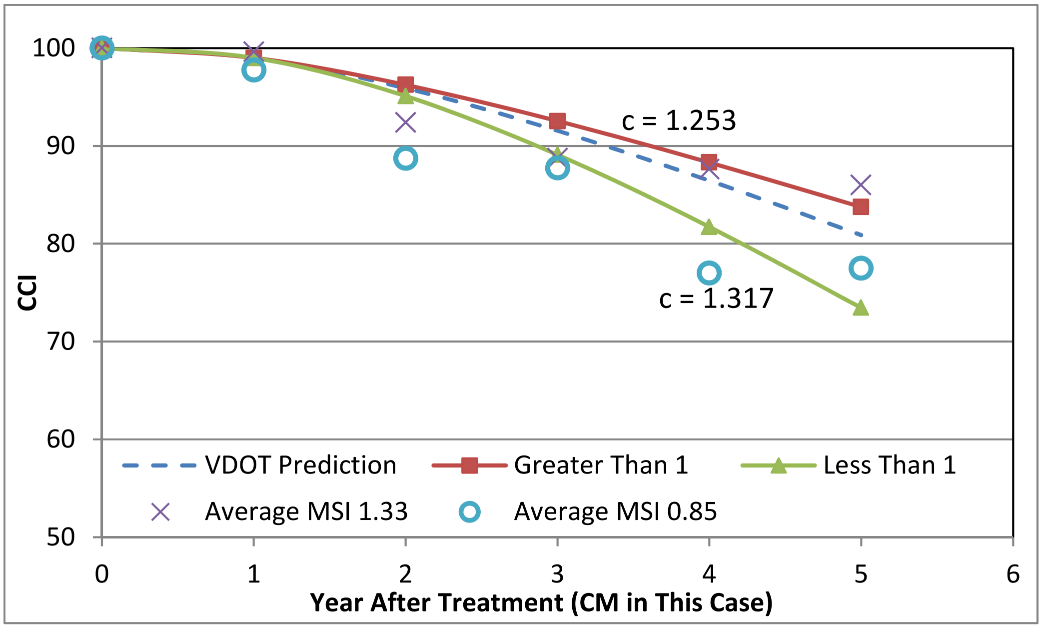

Figure 20. CCI Prediction Curves for Varying MSI Values

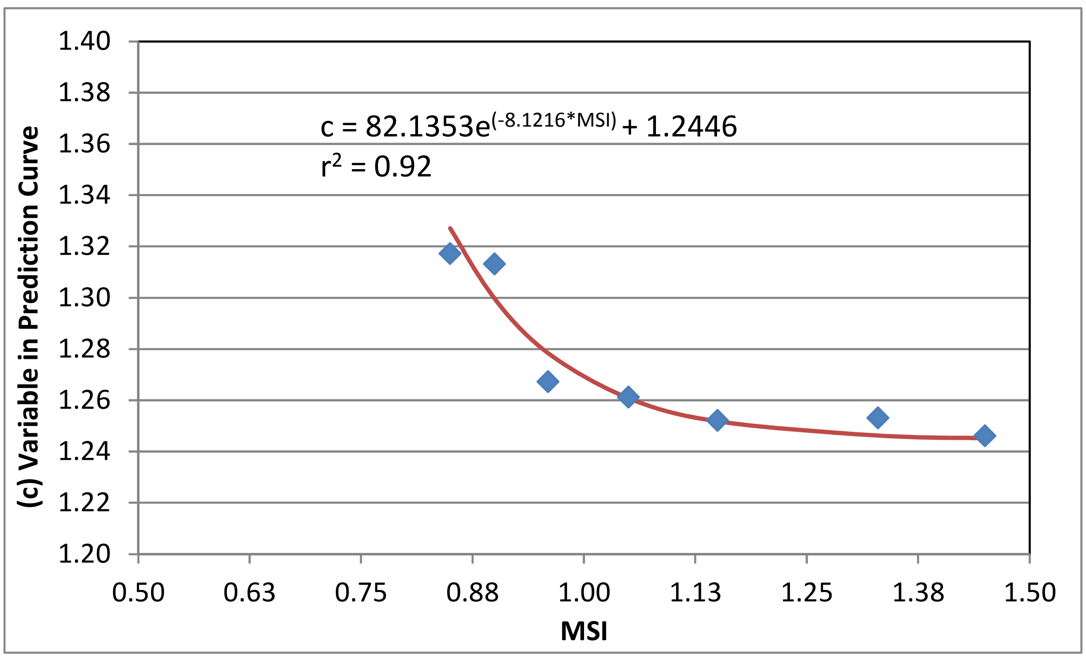

Figure 21. Prediction Curve (*c*) Variable As a Function of MSI

Cost Analysis of Structurally Weak Pavements

As was shown in the previous section, the expected performance of a maintenance treatment can be related to its structural condition. Thus, it follows that a structurally weak pavement will have a higher life-cycle cost than that of a structurally adequate pavement. This was investigated further by analyzing the cost of performing corrective maintenance treatments over a section of pavements with varying MSI values.

The first step was to determine the expected life of a corrective maintenance treatment as a function of the MSI. For the purposes of this exercise, a CCI value of 60 was chosen as the trigger for a treatment, thus the service life of the treatment was said to be the time it takes for the pavement to reach a CCI of 60 beginning from a CCI of 100. The relationship shown in Figure 21 was used to determine a *c* value for a range of MSI values. This *c* value was then input into Equation 17 to develop the relationship between MSI and expected life. The variables *a* and *b* in Equation 17 were input as 9.176 and 9.18 (respectively). The expected life of a CM treatment as a function of the MSI can be seen in Figure 22.

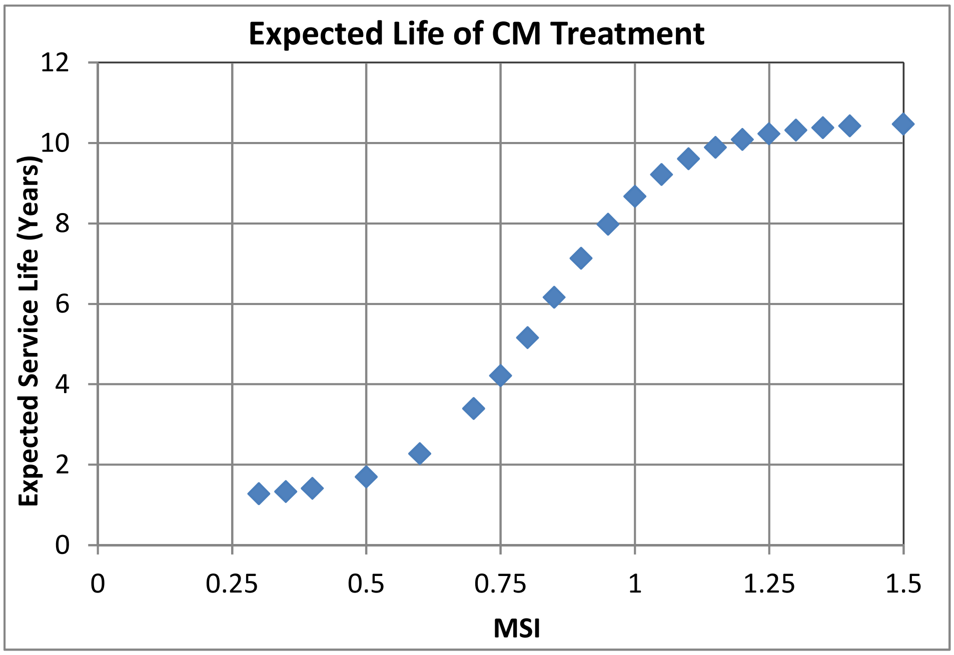

Figure 22. Expected Life As a Function of MSI

It can be seen in Figure 22 that a pavement with an MSI value of 1 would take just less than 9 years to go from a perfect CCI to a CCI of approximately 60. The function seems to follow a sigmoidal shape where MSI values below 0.5 and above 1.2 have very little impact on the expected life. Furthermore, the lowest MSI values from VDOT testing were found to be approximately 0.7. Thus, the values on the lower end of Figure 22 are not expected to be encountered often.

After the expected life function was developed, the number of expected treatments to be applied over a 25 year analysis period was calculated. This was done by simply dividing 25 years by the expected service life value from Figure 22. The number of expected treatments was then multiplied by the expected corrective maintenance cost per mile found in VDOT (2008) ($71,818) to determine the expected cost for performing corrective maintenance treatment over a 25 year period. The results can be seen in Figure 23. It was also recognized that a partial treatment would not be performed, thus the number of expected treatments and consequently the costs were rounded to reflect this, as can be seen in Figure 23.

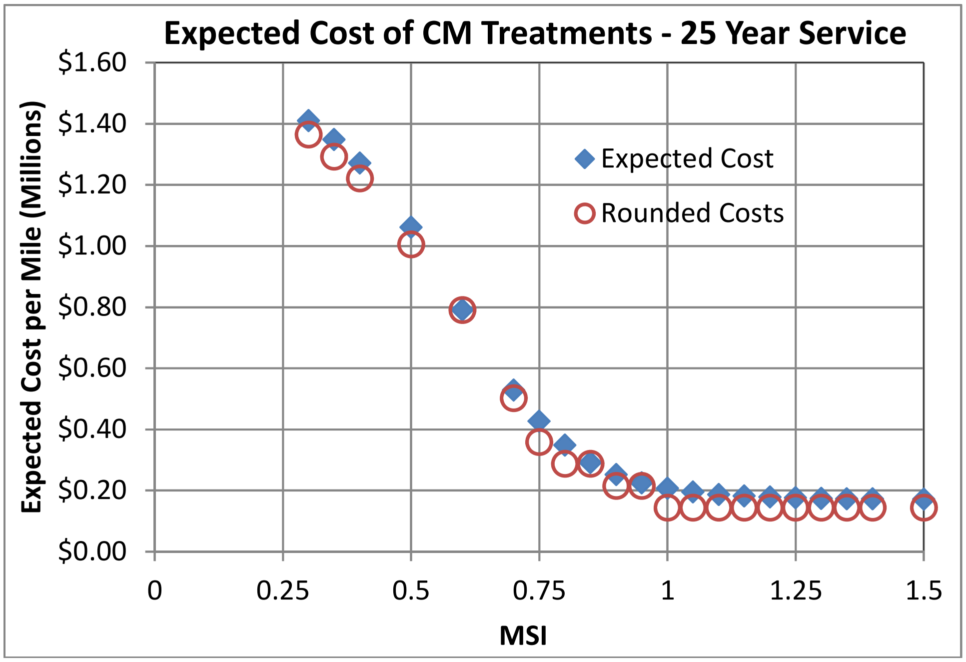

Figure 23. Expected Cost of CM Treatment As a Function of MSI

An interesting note from Figure 23 is the fact that not less than 2 corrective maintenance treatments can be expected to be performed during this analysis period. Once the MSI value reaches 1, the minimum costs for maintaining this pavement has been minimized for the 25 year case. When the MSI reaches 0.8, the costs to maintain this pavement has doubled over the 25 year analysis.

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 was 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 made to develop the MSI equation. When developing the MSI for interstates, the following parameters were held steady: a Present Serviceability Index (PSI) of 4.2, the terminal value of the Serviceability Index (PSI_T) of 3, the reliability of 95% and the material 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. Low volume secondary roads and residential roads have a PSI value of 4.0, but this research is intended to focus on higher priority routes. Therefore the only changes will be reflected in the reliability and the PSI_t. Table 14 presents the VDOT design values for each case.

Table 14. 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 A2 from Appendix A, 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 A1). Therefore, this is the value in the MSI equation (Equation 12) that will be effected by changing the reliability. The serviceability index is on the right hand side of Equation A2, thus the changes in serviceability indices will be reflected in the alpha and gamma factors shown in Equation A3. The procedure to determine the values for alpha and gamma in Equation A3 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 I-81 in Virginia. A similar process was followed for the other routes to determine Equations 18 through 21. The equation that was developed earlier in this report for use in interstates is also presented for completeness of the list.

MSI for Interstates:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05716 * (\log(ESAL) - 2.32 * \log(M_R) + 9.07605)^{2.36777}} \quad \text{(Eq. 18)}$$

MSI for Divided Primary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.060488 * (\log(ESAL) - 2.32 * \log(M_R) + 8.89818)^{2.32752}} \quad \text{(Eq. 19)}$$

MSI for Un-Divided Primary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05943 * (\log(ESAL) - 2.32 * \log(M_R) + 8.89818)^{2.32506}} \quad \text{(Eq. 20)}$$

MSI for High Volume Secondary Routes:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05919 * (\log(ESAL) - 2.32 * \log(M_R) + 8.77764)^{2.32729}} \quad \text{(Eq. 21)}$$

Comparison of the MSI Equations of Different Road Categories

In order to assess the extent to which Equations 18 through 21 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 scenario ($5.00x10^7$ ESALs), and a relatively low traffic scenario ($5.00x10^5$ ESALs). The results can be seen in Figure 24 and Figure 25.

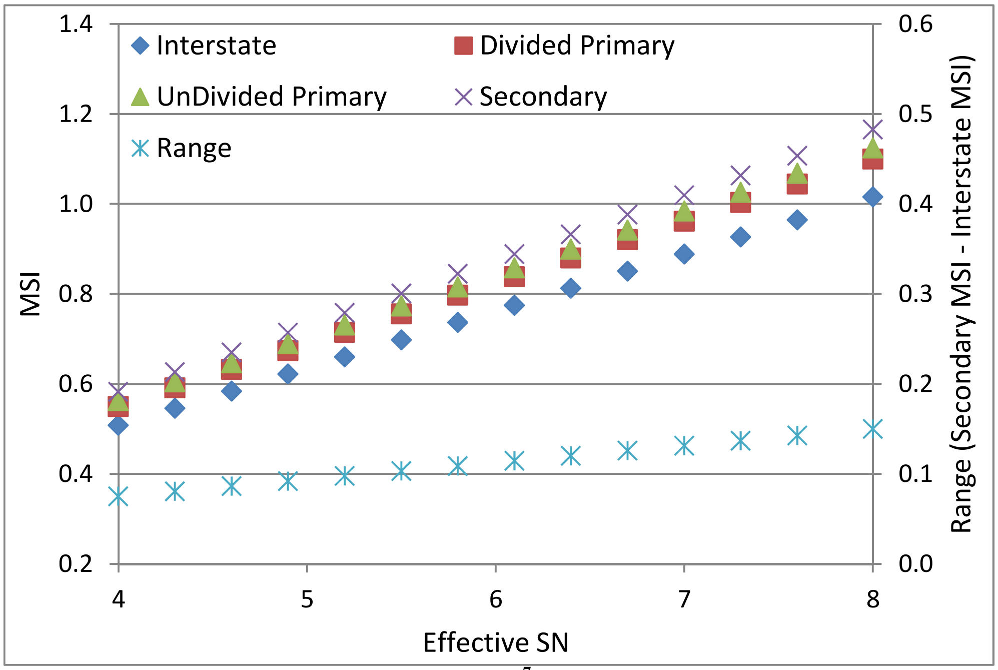

Figure 24. MSI Equation Comparison for $5*10^7$ ESAL and Resilient Modulus of 6,000 psi

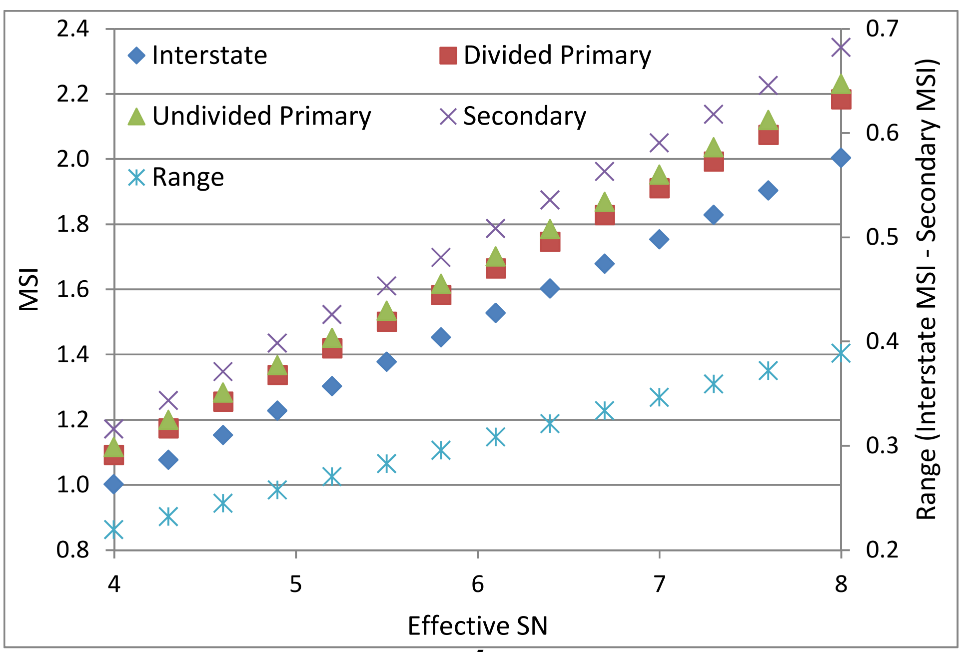

Figure 25. Equation Comparison for $5*10^5$ ESAL and Resilient Modulus of 6,000 psi

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 24 and 25 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 26. 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:

$$MSI = \frac{0.4728 * (D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.0600 * (\log(ESAL) - 2.32 * \log(M_R) + 8.89818)^{2.32629}} \qquad \text{(Eq. 22)}$$

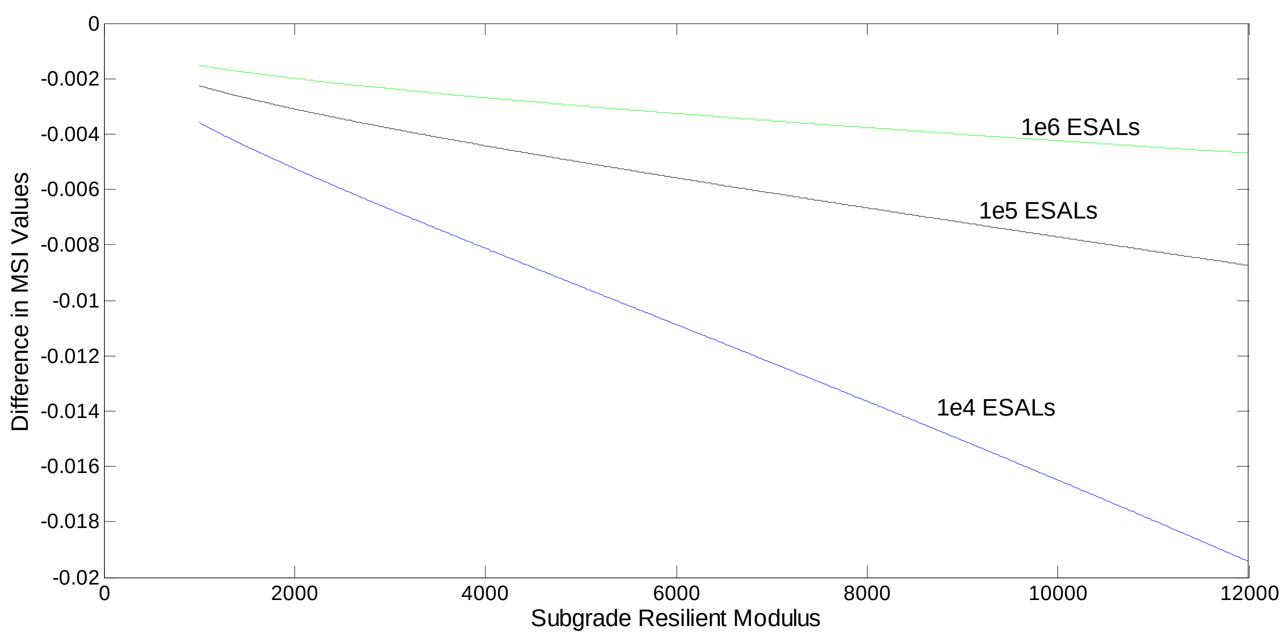

Figure 26. Comparison of Divided and Undivided Primary Routes

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 27.

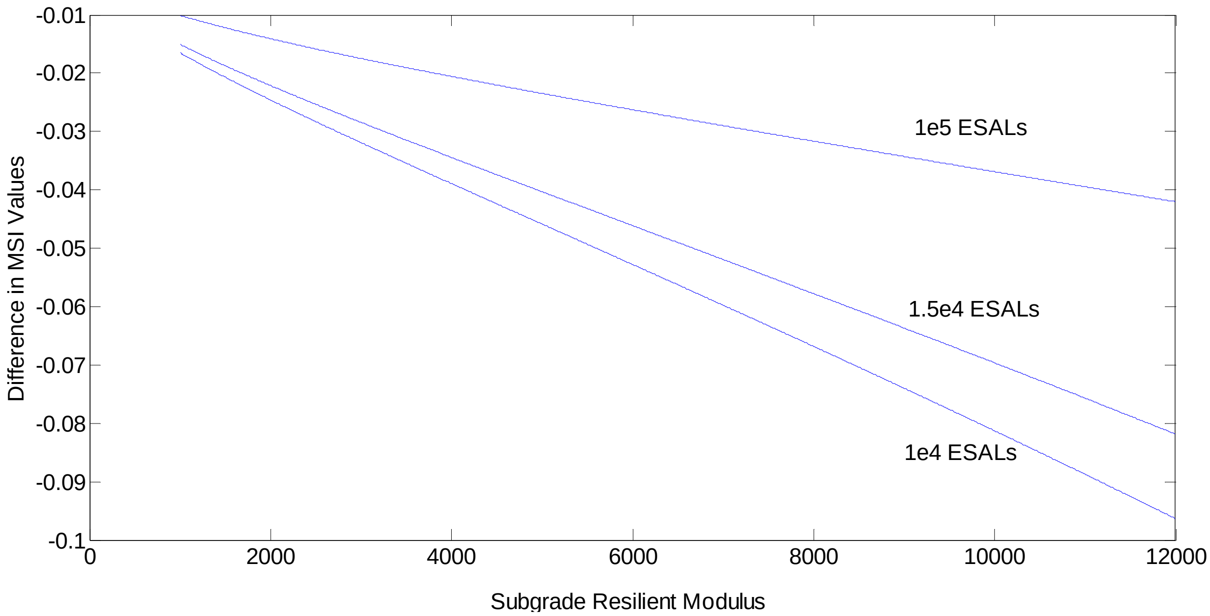

Figure 27. Comparison of Primary and Secondary Routes

It can be seen in Figure 27 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 $1.5*10^5$ ESALs. The deflection data are presented in Table 15, 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 15. Deflection Data for a Primary Route

Structural Parameters							MSI	
D_0 (mils)	$D_{1.5Hp}$ (mils)	Pavement Depth (in)	ESAL	D_{60} (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 was 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. The difference in MSI values is due to the differences in the MSI equations. This case demonstrates when a pavement would be considered weak if categorized as primary, but adequate when categorized as secondary.

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, it was decided that these equations should not be combined.

Final Form of the MSI Equations

The MSI equations were generalized into a single form (Equation 23), and the constants were tabulated (Table 16).

$$MSI = \frac{K1 * (D_0 - D_{1.5Hp})^{K2} * Hp^{K3}}{\alpha * (\log(ESAL) - 2.32 * \log(M_R) + \beta)^{\gamma}} \qquad \text{(Eq. 23)}$$

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 M_R 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 16. 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

SUMMARY OF FINDINGS

- *A weak level of correlation was found between the various distress-based condition indicators and the structural measures of the pavement.* 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 surface condition of the pavement while adding no structural capacity.

- *Subgrade strength and pavement strength should not be independently used to make pavement maintenance decisions.* The two parameters should be combined into an overall structural index for the pavement section. This becomes clear when one considers that the initial design of the pavement depends on the subgrade strength. Therefore, the in-service structural condition of the pavement should be evaluated as a function of the contribution of the subgrade strength and the pavement structure strength.

- *The currently used traffic classification based on pavement classification does not appropriately take into account the effect of traffic level on pavement structural performance.* This is because (1) structural performance is sensitive to changes in traffic levels that are much smaller than the ranges set by the different traffic classification and (2) actual traffic levels do not always correspond to the appropriate traffic classification based on pavement classification (i.e., the actual carried traffic can be significantly higher than suggested by the pavement classification).

- *The analysis verified that the in situ structural condition of the pavement significantly influences the performance of maintenance activities.* As the existing structural condition of the pavement decreases, the performance of a given maintenance action decreases. This was expanded to show that a structurally weak pavement has a much higher life-cycle cost when compared to a structurally strong pavement.

- *A structural index in the same form as the one developed by the Texas Department of Transportation (the SCI) is the index that best fit the use as a network-level structural capacity index for flexible pavements for VDOT.*

CONCLUSIONS

- *The project 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 overall condition of the pavement. 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.

- *Based on the criterion of minimizing errors between network-level predictions and project-level work done, the SCI developed by the Texas Department of Transportation is best suited for modification and use as a network-level structural capacity index for flexible pavements by VDOT.* Because the pavement structural requirements depend on the pavement class, three different MSI equations were developed for the different pavement classes: interstate routes, primary routes, and secondary routes. The equations reflect the different required reliability and terminal value of the serviceability index for the different pavement classes.

- *The application in a series of pilot applications using limited condition data from I-81 showed that the resulting index for flexible pavements, the MSI, can enhance several pavement management decision processes, including the following:*

  — as a structural screening tool

  — as a tool in scoping projects at the network level and estimating overlay thicknesses (from a network-level perspective)

  — as a performance indicator

  — as a tool in developing enhanced deterioration curves that take into account structural capacity of the pavement (current deterioration curves do not take that into account).

- *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 index developed 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). However, the MSI developed for flexible pavements was found not to be appropriate to use for rigid or composite pavements.

RECOMMENDATION

1. *The Virginia Center for Transportation Innovation and Research and VDOT's Maintenance Division should consider additional research to do the following:*

   - *Develop a structural condition index (similar to the MSI) for composite and rigid pavements.*

   - *Expand the pilot study performed on I-81 to all roadway sections where network-level FWD data are available; refine the structural capacity thresholds; and recommend an enhanced decision tree for potential implementation in VDOT's pavement management system (PMS).*

   - *Develop guidelines to implement the structural capacity requirements (structural condition index) in terms of data collection, threshold values, and statistical analysis.*

   - *Expand on the concept of the structural index to applications such as improving the pavement deterioration curves currently used by pavement management personnel in VDOT's Maintenance Division.*

BENEFITS AND IMPLEMENTATION PROSPECTS

This research is expected to enhance the maintenance planning procedures for VDOT at the network level by providing structural condition information about the in-situ state of the pavement. The implementation of the MSI procedure into the network-level decision process is expected to minimize the difference between the need-based pavement budget planning at the network level and actual project needs that are determined with a more accurate assessment of the pavement at the project level. This will lead to improved planning capabilities that can identify structurally inadequate pavement sections before their functional condition deteriorates. Furthermore, the MSI can potentially be used to develop performance prediction curves that will have better prediction capabilities because the structural condition of the pavement can be incorporated into the prediction models.

The next steps should include formalizing a decision process that includes the MSI, along with the other relevant condition parameters. This decision process can potentially be a modification of the current decision process, with the MSI replacing the current structural parameters. The MSI evaluation should then be completed on the remainder of the VDOT network where deflection data are available so that a complete database of structural condition is available for network-level decision making.

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APPENDIX A

DEVELOPMENT OF THE MSI

Calculating the Effective SN

The calculation of the effective structural number (SN_{eff}) for the pavement in the SCI methodology presented earlier in this research 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 28.

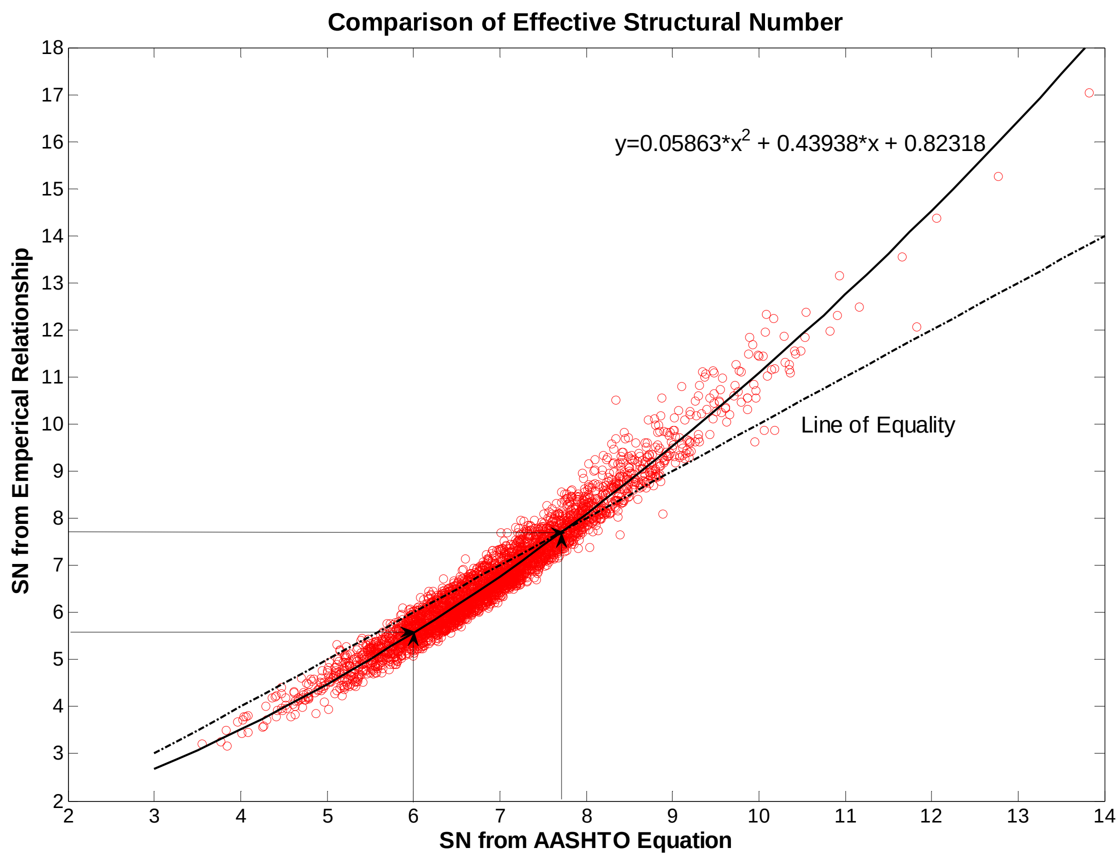

Figure 28. Comparison of Effective SN From AASHTO Equation and Rhode (1994) Method

The relationship shown in Figure 28 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 follow 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 ESALs (from 2009 values) varied from $2.5*10^7$ to $8*10^7$ (Diefenderfer, 2008). One major reason for the large variation of traffic is that Interstate 64 runs along the same pavement as I-81 for a length near central Virginia. In addition to large variations in traffic and resilient modulus, research reported at the 2012 meeting of the Transportation Research Board identified shortcomings in the tabulated SN_{req} used by the Texas Department of Transportation (Nam et al., 2012). 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 A1:

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

(Eq. A1)

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 Present Serviceability Index (PSI) of the constructed pavement minus the terminal value for the Serviceability Index (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, 2003). 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 17. The distribution for the SN values that were obtained from the traffic and resilient modulus are presented in Figure 29.

Table 17. Summary Statistics for Chosen ESALs and Resilient Modulus

ESALs		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

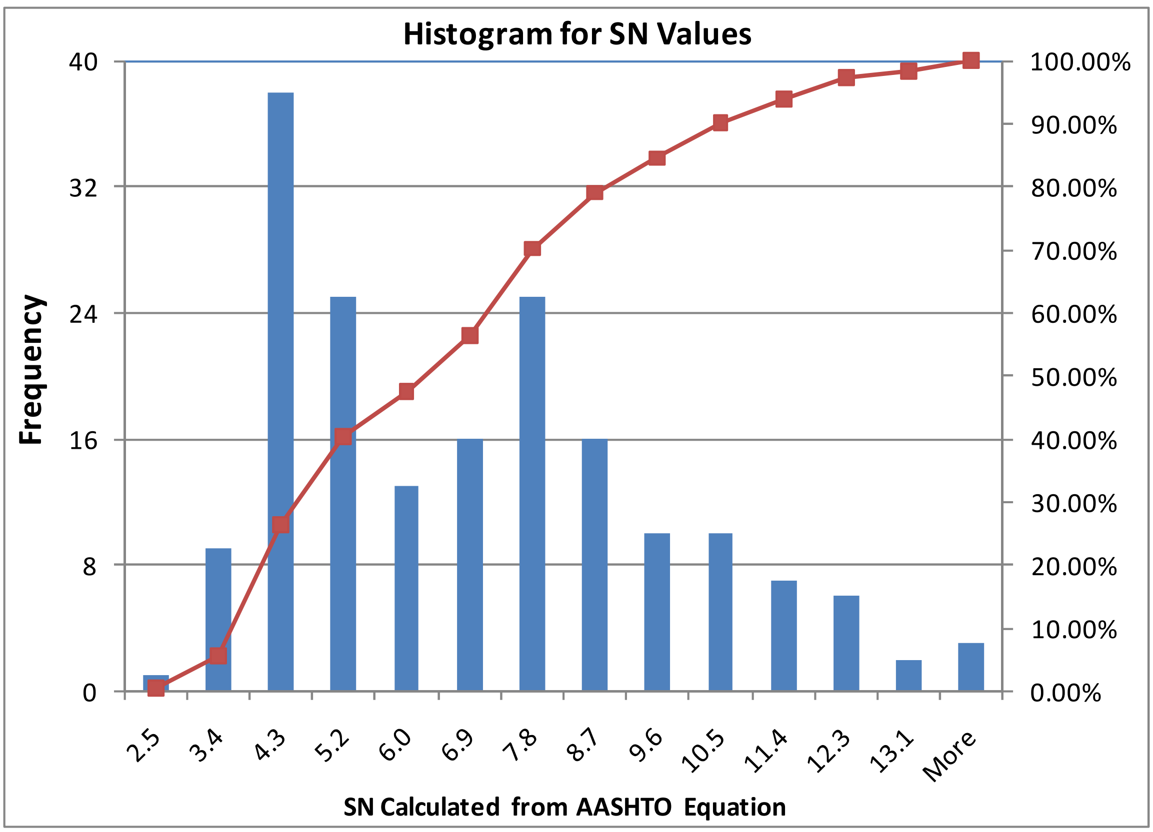

Figure 29. SN Values From Generated Traffic and Resilient Modulus

After fixing the parameters found in the VDOT design guide and developing a set of SN values from the AASHTO equation, Equation A2 was formed by rearranging the original AASHTO equation:

$$\log(\mathrm{ESAL}) - 2.32 * \log(\mathrm{M_R}) + 9.07605 = 9.36 * \log(\mathrm{SN}+1) - \left(\frac{0.352183}{\left(0.40 + \frac{1094}{(\mathrm{SN}+1)^{5.19}}\right)} \right) \quad \text{(Eq. A2)}$$

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 30. It is clearly seen in Figure 30 that the behavior of the error can be corrected using a power transform. Thus, a solution in the form of an exponential function that included the left hand side of Equation A2 was sought.

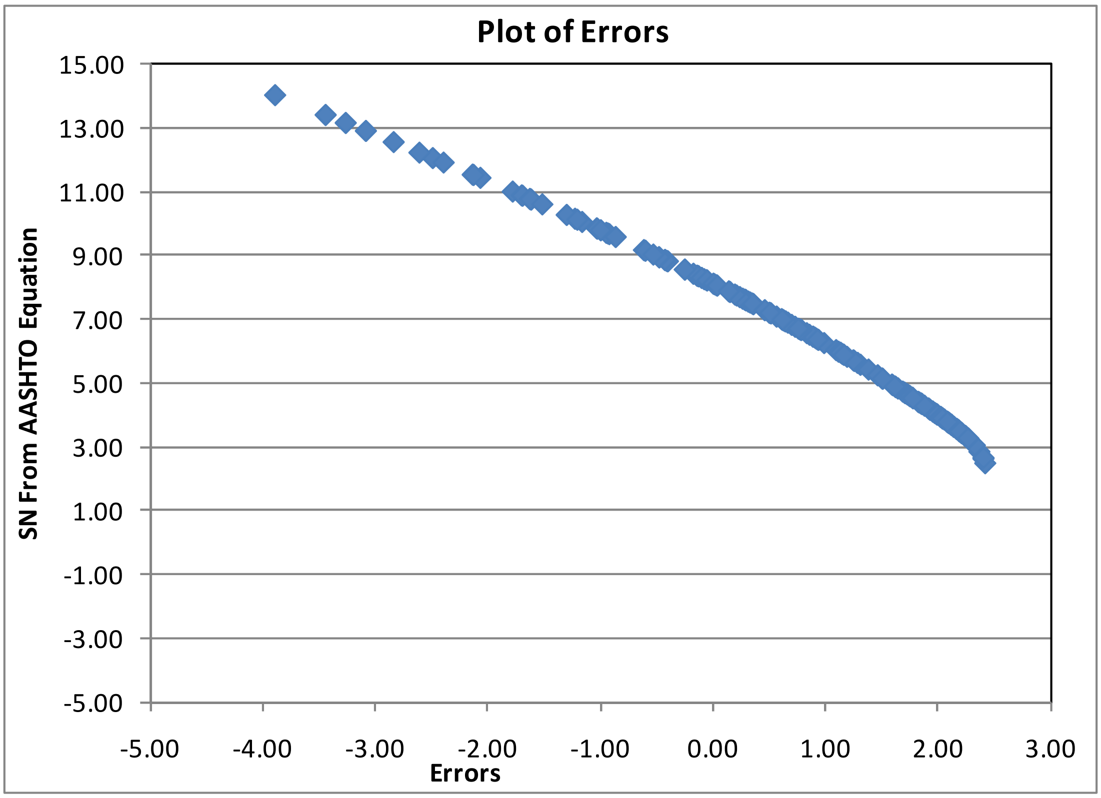

Figure 30. Plot of Errors: Closed Form Equation Minus AASHTO Equation

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 is shown in Equation A3.

$$\alpha\left(\log(ESAL) - 2.32 * \log(M_R) + 9.07656\right)^{\gamma} \quad \text{(Eq. A3)}$$

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 31. 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 Equation 3 in the main body of this report, the MSI can be calculated as:

$$MSI = \frac{0.4728*(D_0 - D_{1.5Hp})^{-0.4810} * Hp^{0.7581}}{0.05716*(\log(ESAL) - 2.32*\log(M_R) + 9.07605)^{2.36777}} \quad \text{(Eq. A4)}$$

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 M_R 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.

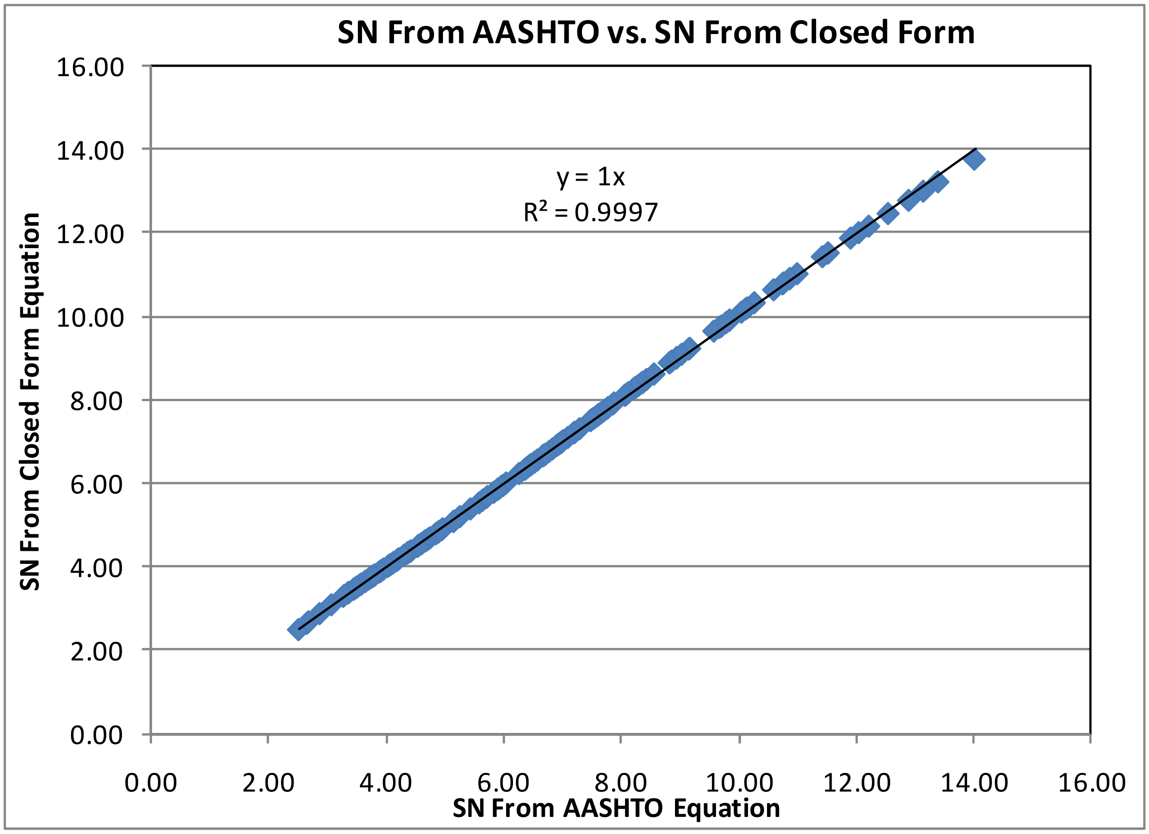

Figure 31. Comparison of AASHTO and Closed Form Equations

APPENDIX B

DETAILED COMPARISON OF INDICES

I-81 Northbound in Pulaski County

The first section that was analyzed was 3 lane-miles of pavement along I-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 were 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 32. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 33. 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.

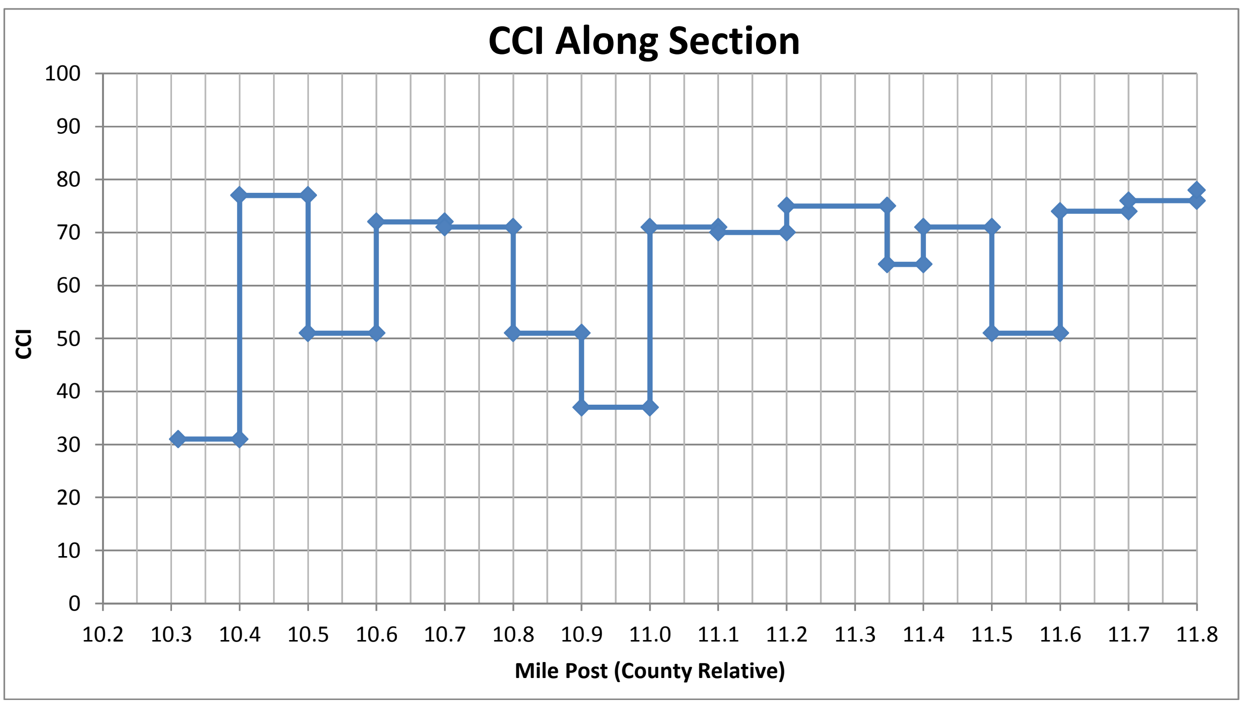

Figure 32. CCI Along Pavement Section: I-81 Northbound in Pulaski County

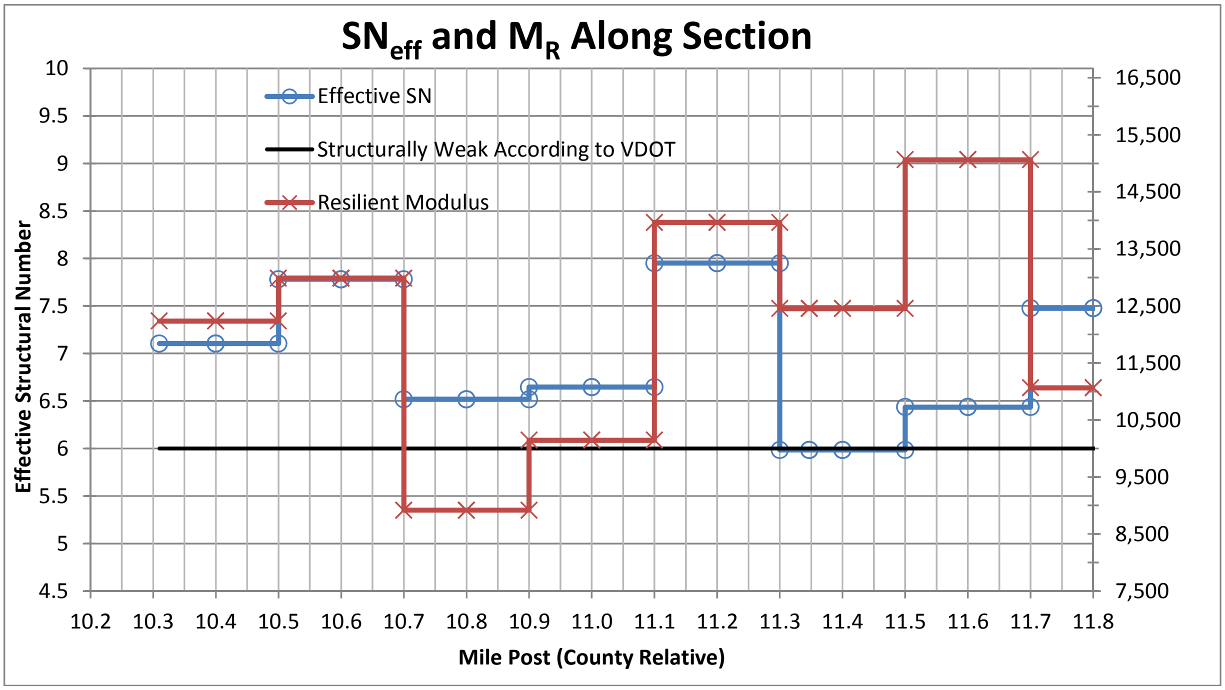

Figure 33. Structure Along Pavement Section: I-81 Northbound in Pulaski County

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, which is how the VDOT decision matrices are designed to work. 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 2½ ft wheel path was assumed per the LTPP Distress Identification Manual (Miller and Bellinger, 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:

$$\%AREA_{Alligatior} = \frac{\text{AlligCrack (in square feet)}}{5'*\text{segment length in feet}} \quad \text{(Eq. B1)}$$

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:

$$\%Area_{Patched} = Max\left\{\frac{\text{Area Patch}_{\text{Wheel Path}}}{5'*\text{segment length (ft)}}, \frac{\text{Area Patch}_{\text{Non Wheel Path}}}{(\text{LaneWidth - 5'}) * \text{segment length (ft)}}\right\} \quad \text{(Eq. B2)}$$

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 were only given in total crack length for each crack level. To convert the given measure to cracks per mile, the following was used:

$$\text{Transverse Cracks Per Mile} = \frac{\text{Transverse Crack Length}}{\text{Lane Width (ft)} * \text{segment length (miles)}} \quad \text{(Eq. B3)}$$

Decision Process

After converting the distress data into inputs for the decision matrices, the decision process was followed. 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 final decision 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 MSI and center deflection (SSI). The thresholds were applied similarly to the enhanced decision trees. For the MSI index, the level one, level two and level three corresponded to a MSI 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 mils, and then the traffic levels were analyzed to determine the level in the decision tree.

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 MSI 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 MSI method most closely predicted the actual work completed.

I-81 Southbound in Botetourt County

A similar process was undertaken for a 3.44 lane-mile pavement section along I-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 comprised 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 34. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 35. 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 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.

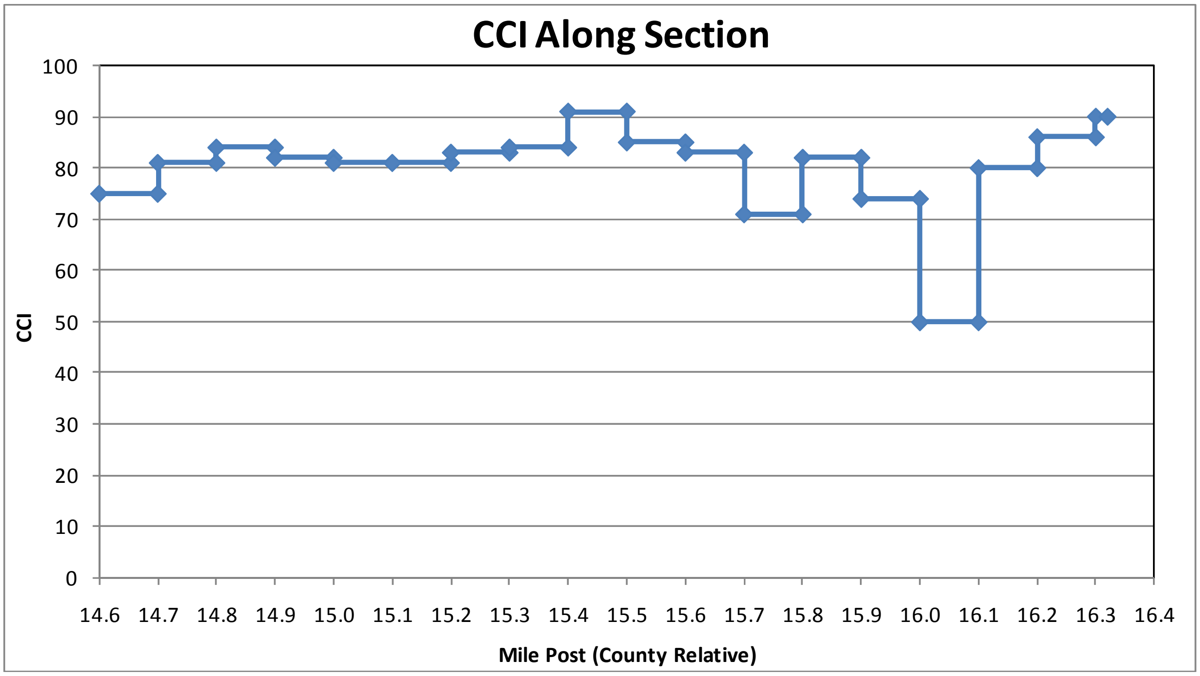

Figure 34. CCI Along Pavement Section: I-81 Southbound in Botetourt County

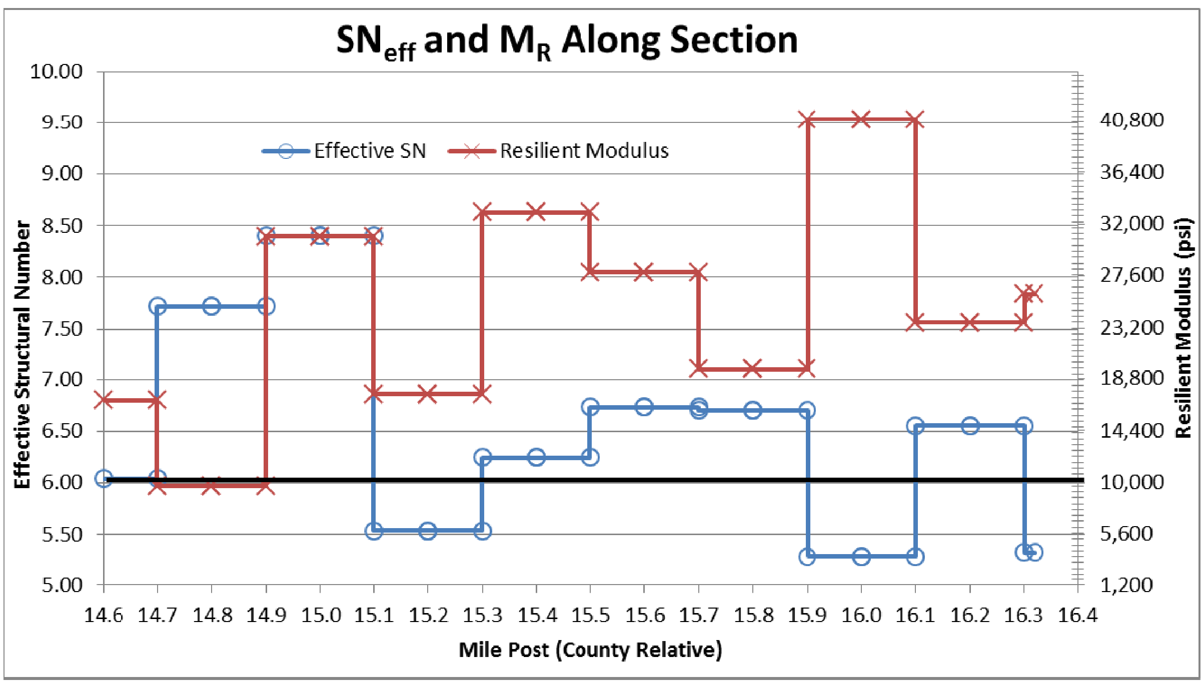

Figure 35. Structure Along Pavement Section: I-81 Southbound in Botetourt County

The decision process was followed using the initial decisions, and applying the MSI, center deflection (SSI) indices. 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 MSI 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 MSI method most closely predicted the actual work completed.

I-81 Southbound in Montgomery County

The process was repeated for a 9.18 lane-mile pavement section along I-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 comprised 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 36. The structure, in terms of the effective structural number and the resilient modulus of the subgrade, can be seen in Figure 37. The pavement section is structurally weak according to the VDOT criteria (SN < 6 or M_R < 10,000) for approximately 3.18 lane-miles.

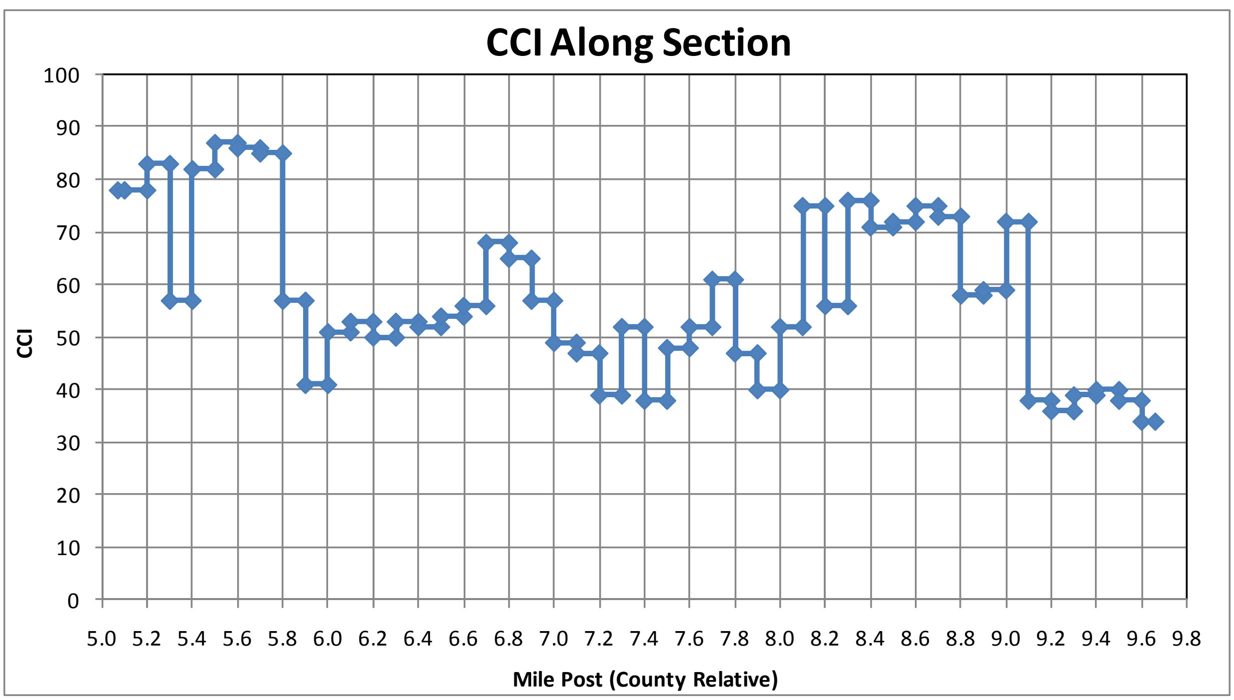

Figure 36. CCI Along Pavement Section: I-81 Southbound in Montgomery County

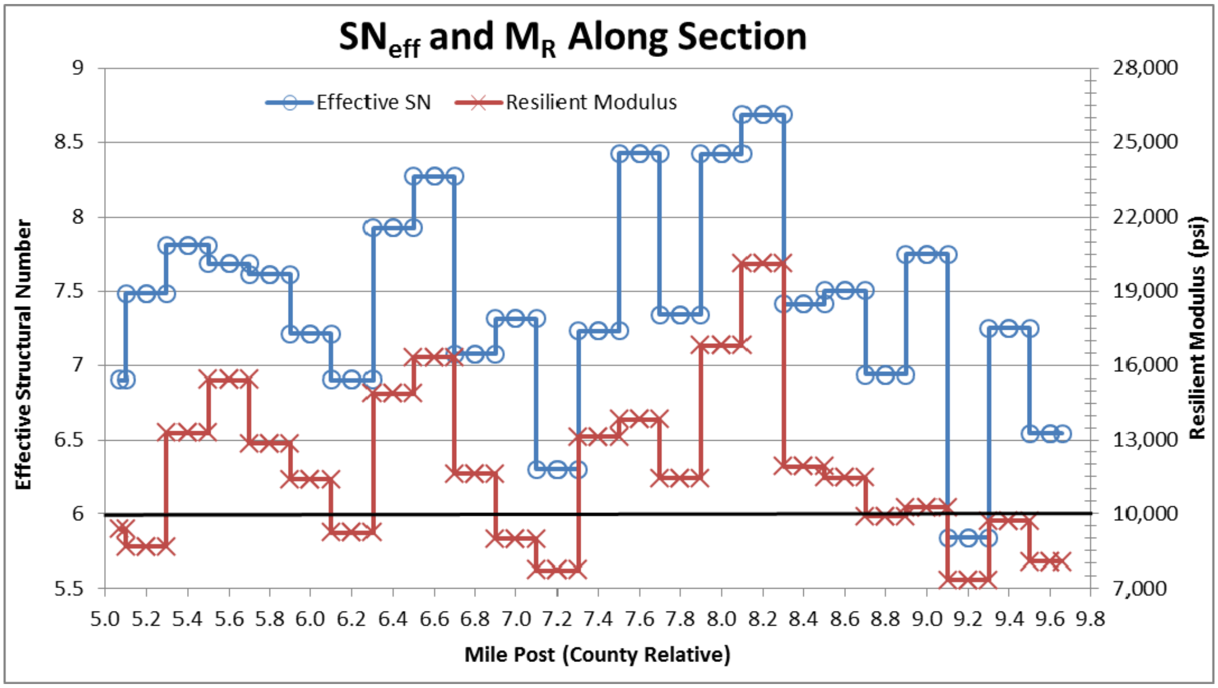

Figure 37. Structure Along Pavement Section: I-81 Southbound in Montgomery County

Decision Process

After converting the distress data into inputs for the decision matrices, the decision process was followed. The final decision 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 MSI, center deflection (SSI) indices.

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 MSI 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 MSI both closely predicted the actual work completed.