

**Network Level Decision-Making Using Pavement Structural Condition Information
from the Traffic Speed Deflectometer**

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ABSTRACT

Pavement structural condition plays a critical role in the rate of pavement deterioration, yet most state highway agencies' network-level decision-making processes are primarily based on surface distresses. Despite the limitations of the traditional structural condition measuring devices, some states have experimented with stationary deflection devices for network-level applications. Over the past decade, continuous deflection devices have become capable of measuring the network-level pavement structural condition information at traffic speeds. However, since the traffic speed deflection devices use newer technology, there is a need for guidelines on how the state agencies could make use of this information for pavement management decision-making. This dissertation developed processes and enhanced tools to incorporate the pavement structural condition from the TSD into Virginia's network-level pavement management process.

This first part of the study developed pavement deterioration models for a subset of road networks in Virginia, to show that the pavement structural condition as measured by the TSD has an impact on the rate of deterioration of the surface condition. A structural condition matrix was then developed to augment the treatment selection process currently used by VDOT. Application of the augmented matrix on the tested Interstate network resulted in reducing the percentage of the network requiring CM and increasing the percentage requiring PM and RM.

The second part of the study investigated the possibility of using pavement deflection measurements obtained from the TSD for network-level structural evaluation of pavements in Virginia. The study reported that the structural condition obtained with the TSD can replace the structural condition obtained from the FWD that is currently used in the VDOT PMS. The effective structural number (S_{Neff}) calculated from the TSD and FWD had

similar distribution, and the calculated consistency between the TSD S_{Neff} and FWD S_{Neff} was higher than the consistency between the S_{Neff} from two repeated sets of FWD measurements.

The third part of the study simulated the network level decision-making approaches based on both the structural condition parameter and the surface condition parameter, considering cases with and without the pavement treatment interval. The study reported that network-level decisions based on the pavement surface condition alone can result in significantly different treatment selection, compared to decisions based on the pavement structural condition. The study reported savings of 9% and 11% for cases with and without considering the pavement treatment intervals, using decision-making based on the structural condition.

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GENERAL AUDIENCE ABSTRACT

Pavement structural condition plays a critical role in the rate of pavement deterioration, yet most state highway agencies' network-level decision-making processes are primarily based on surface distresses. Despite the limitations of the traditional structural condition measuring devices, some states have experimented with stationary deflection devices for network-level applications. Over the past decade, continuous deflection measuring devices have become capable of measuring the structural condition of the pavements at traffic speeds. However, since the traffic speed deflection devices use newer technology, there is a need for guidelines on how the state agencies could make use of this information for pavement management decision-making. This dissertation developed processes and enhanced tools to incorporate the pavement structural condition from the Traffic Speed Deflectometer (TSD) into Virginia's network-level pavement management process.

This first part of the study developed pavement deterioration models for a subset of road networks in Virginia, to show that the pavement structural condition as measured by the TSD has an impact on the rate of deterioration of the surface condition. A structural condition matrix was then developed to augment the treatment selection process currently used by Virginia Department of Transportation (VDOT). The second part of the study investigated the possibility of using pavement deflection measurements obtained from the TSD for network-level structural evaluation of pavements in Virginia. The study reported that the structural condition obtained with the TSD can replace the structural condition obtained from the FWD that is currently used in the VDOT's pavement management system. The third part of the study simulated the network level decision-making approaches based on both the structural condition parameter and the surface condition parameter, considering cases with and without the pavement treatment interval. The study reported

savings of 9% and 11% for cases with and without considering the pavement treatment intervals, using decision-making based on the structural condition.

Dedication

I would like to dedicate this dissertation to the memory of my uncle and aunt,
Krishna Lal Shrestha (1942-2006) and Bina Shrestha (1947-2017).

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Chapter 1. Introduction

1.1 Background

Pavement networks are some of the highest value transportation assets in the country, covering over 4,000,000 miles of public roads and over 164,000 miles of highways in the National Highway System (Office of Highway Policy Information 2010). State Highway Agencies (SHAs) spend billions of dollars to maintain the existing road network. Despite the investments, there is a significant funding gap between the projected revenues and the surface transportation funding need. Thus, state highway agencies have to use the available funding effectively to manage and maintain the existing transportation infrastructure.

One of the tools available to enable the effective use of limited available funding are transportation asset management systems. For example, state agencies use Pavement Management System (PMS) in order to maintain the existing pavement assets. In simple terms, pavement management can be defined as “the process of maintaining the infrastructure cost-effectively” (Wolters et al. 2011a). It supports consistent and rational decision-making, which helps state agencies allocate maintenance and rehabilitation (M&R) activities to the road network effectively and optimize the use of available funds. Thus, the PMS lies at the center of pavement investment decision-making for State Highway Agencies.

The pavement management decision-making process can be categorized into three levels: Strategic, Network, and Project level (Wolters et al. 2011b). The strategic level is the broadest in terms of scope and less detailed than the network and project level. A few examples of strategic level decisions include establishing performance targets, determining, and allocating funding to meet the targets. The network level is more detailed compared to the strategic level and involves analyzing the entire pavement network, determining the funding needs for different pavement sections, and selecting cost-effective treatment actions and timing. The project level involves specific details of the projects being carried out. It involves detailed field investigation and identifying what exact treatments are needed, materials to be used, and designing pavements, etc. An effective PMS must integrate all three levels of information together for decision-making: strategic, network, and project level (Wolters and Zimmerman 2008).

One of the factors that play a crucial role in PMS decision-making is the pavement condition. In general, the pavement condition can be classified into two categories: functional condition and structural condition. Most state highway agencies use smoothness and surface distresses to summarize the overall pavement condition and make decisions based on it. The surface distresses though representative of the overall pavement condition, do not necessarily identify the cause of pavement deterioration. In general, the surface distresses are only the symptoms of the underlying issues and not the source of problems. Thus, the pavement structural condition should be incorporated into the pavement management decision-making process.

Because network-level structural condition surveys have been difficult to undertake, only a limited number of state agencies have performed such surveys (Flintsch et al. 2013). Traditionally used devices that measure the structural condition, such as the Falling Weight Deflectometer (FWD) and the Deflectograph are either stationary or travel at speeds significantly lower than the traffic speed. This limits the locations where the data can be collected at the network level and requires some kind of traffic control. Stationary deflection measuring devices also impact the safety of the operators as they have to work besides the traffic lane making it challenging to collect structural condition data at the network level. Due to these reasons, pavement structural evaluation has been mostly collected only at the project level.

With the advent of new technologies, in the early 2000s, traffic speed deflection measuring devices have been developed. The Traffic Speed Deflectometer (TSD) and the Rolling Wheel Deflectometer (RWD) have been identified as two devices that have the potential of network-level structural condition evaluation (Flintsch et al. 2013; Rada et al. 2016). Various studies have presented case studies and examples of applications of continuous deflection devices (both TSD and RWD) for network-level pavement management (Elseifi et al. 2012; Elseifi et al. 2011; Elseifi and Zihan 2018; Katicha et al. 2017).

1.2 Problem Statement

Although the pavement structural condition has a considerable effect on the pavement deterioration rate and required treatment, the use of the structural condition for network-level pavement management decision-making has been limited. This is mainly due to the limitations of the traditional structural condition measuring devices such as the FWD, which is stationary during

testing. The TSD, being a continuous deflection-measuring device, overcomes some of the limitations of the FWD in terms of network level testing capabilities. However, since TSD uses newer technology, there is a need for guidelines on how the state agencies could make use of this information for pavement management decision-making. Some of the important questions to be answered being: (1) what kind of information can the TSD provide? (2) how do the TSD measurements compare with those obtained with the FWD? and (3) how can the TSD measurements be effectively used for pavement management decision-making? This research attempts to address these questions and present an example on how the TSD measurements can be used for pavement management decision-making.

1.3 Research Objectives

The objective of this research was to develop processes and enhanced tools to incorporate the pavement structural condition into network-level pavement management. Specific tasks have been proposed to achieve the objective of the study.

1. Ascertain that pavement structural condition has a significant impact on the pavement deterioration rate and thus, should be included in network-level pavement management;
2. Demonstrate the practicality of a pavement treatment selection process, that incorporates the structural condition information from the TSD; and
3. Show that network-level decision-making that incorporates pavement structural condition along with pavement surface condition can lead to more efficient decision-making compared to the decision-making based solely on the surface condition.

1.4 Significance

This research focuses on the incorporation of TSD measurements into pavement management to enhance the decision-making process. The pavement structural condition plays a critical role in the rate of pavement deterioration. Various studies have documented the benefits of using the structural and functional condition together for pavement management decision-making (Lister et al. 1982; Steele et al. 2015). However, most state highway agencies have not incorporated the pavement structural condition into their network-level pavement management decision-making

process. Even when they do, the limitations of the traditionally used devices pose challenges to collect the data at the network level. Using the TSD instead of FWD for network-level pavement evaluation overcomes most of the challenges that arise from the FWD being a stationary deflection measuring device.

By incorporating the structural condition information into the network level decision-making, the mismatch between the information used in the network and the project level decisions can be minimized along with more accurate budget allocation and more efficient decision-making can be performed at the network level.

1.5 Justification

The US has a \$786 billion funding backlog for meeting the highway and bridge needs (ASCE 2021). The majority of the backlog (\$435 billion of the \$786 billion) is for repairing the existing highways. This has resulted in a ‘D’ grade for the current condition of the highways on the ASCE infrastructure report card (ASCE 2021). An efficient pavement management practice will actively work to meet the US highway system needs, eventually leading to significant improvement in the condition of the road infrastructure.

Over the past decade, various continuous deflection measuring technologies such as the TSD and the RWD have become capable of measuring the pavement structural condition at the network level. Various studies have reported the benefits of using traffic speed deflection measurements for network-level evaluation. The use of structural condition reduces the mismatch between the network level and the project level decisions and allows for a more accurate budget allocation at the network level. However, there has been limited research on the possibility of incorporating indices from these deflection devices into a state agency's existing pavement management system.

1.6 Dissertation Organization

The dissertation follows a manuscript format that includes a collection of 6 papers, 3 of which are manuscripts (i.e. chapters 3, 4, and 5). Each manuscript is included as an individual chapter of the dissertation. They represent the research work in which the author was involved at Virginia Tech during the duration of his doctoral studies.

Chapter 1 provides an introduction, objective and general background, problem statement, significance, and methodology.

Chapter 2 consists of a literature review that covers topics such as (i) pavement management systems, (ii) pavement condition indices (both structural and functional), (iii) application of pavement structural condition into pavement management decision-making, (iv) continuous deflection devices, and their application. This chapter summarizes the recent advances in the field of continuous deflection measurement and its application in pavement management.

Chapter 3 is the first manuscript. This chapter develops a pavement deterioration models using the structural condition measurements from the TSD to show that the structural condition affects the pavement deterioration rate. This chapter also presents an augmented decision-making approach to incorporate the pavement structural condition into the network-level decision-making process.

Chapter 4 is the second manuscript. This chapter investigates the possibility of using pavement deflection measurements obtained from the TSD for network-level structural evaluation on bituminous pavements in Virginia. To do so, the study compares the distribution of effective structural number (S_{Neff}) calculated from TSD measurements with S_{Neff} calculated from FWD measurements. and the consistency of TSD with FWD in identifying the same weak sections.

Chapter 5 is the third manuscript. This chapter shows that incorporating the pavement structural condition into the pavement management decision-making process results in more cost-efficient treatment selection. The study simulated the network-level decision-making approaches over a period of time, based on both the structural condition and the surface condition.

Chapter 6 presents the summary, findings, and conclusion of the dissertation and recommendations for future research.

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Chapter 2. Literature review

This Chapter presents a literature review on some of the relevant topics relating to pavement management system, pavement condition indices (both surface and structural) and continuous deflection measuring devices.

2.1 Pavement Management System

The first pavement management system (PMS) was developed around 1960s in the United States and the first comprehensive pavement management guide was published in Canada in 1977 by the Road and Transport Association of Canada (Haas et al. 2015). Over the years, though many agencies have interpreted the role of a PMS differently, the original concept of the PMS still remains quite valid, that is: “it incorporates all the activities in an organized and systematic way that go in providing and operating pavements” (Haas et al. 2015). Few well known definitions of PMS are as follows:

American Association of State Highway and Transportation Officials (AASHTO) pavement management system describes pavement management as “... *the effective and efficient directing of the various activities involved in providing and sustaining pavements in a condition acceptable to the traveling public at the least life cycle cost*”. (AASHTO 1985)

According to the American Public Works Association (APWA) “Pavement management is a systematic method for routinely collecting, storing, and retrieving the kind of decision-making information needed to make maximum use of limited maintenance (and construction) dollars” (Wolters et al. 2011).

2.1.1 Levels of management:

An effective PMS must integrate three levels of information together for decision-making: strategic, network, and project level. An effective PMS connects these three levels of management together and enables effective communication between these levels providing sufficient pavement management information at each of these levels for decision-making.

Table 2-1 Literatures reviewed

No.	Topic	Sub-topic	Studies
1	Pavement Management Systems	History of PMS	Haas et al. 2015
2		Levels of decision-making	Wolters & Zimmerman 2008
3		Components of PMS	Wolters et al. 2011
4	Pavement Surface Condition Indices	Pavement Condition Index (PCI)	ASTM 2011
5		Critical Condition Index (CCI) - VDOT	McGhee 2002
6		Overall Pavement Index (OPI) -	Gharaibeh et al. 2011
7		Pavement Quality Index (PQI)	Janisch 2015
8		Pavement Condition Rating (PCR)	Saraf 1998
9	Understanding Pavement Deflections	Deflection bowl parameters	Horak 2008
10	Pavement Structural Condition Indices based on FWDs	Structural Adequacy Index (SAI)	Haas et al. 2001
11		Structural Strength Index (SSI)	Scullion 1998
12		Structural Number (SN)	Rohde 1994
13		Structural Condition Index (SCI*)	Zhang et al. 2003b
14		Modified Structural Index (MSI)	Bryce et al. 2012
15	Continuous deflection devices	Traffic Speed Deflectometer (TSD)	Katicha et al. 2017
16			Shrestha 2017
17		Rolling Wheel Deflectometer (RWD)	Gedafa et al. 2010
18			Harr and Ng-A-Qui 1977
19			Steele et al. 2020
20		Rapid Pavement Tester (RAPTOR)	Madsen and Pedersen 2019
21			Athanasiadis and Zoulis 2019
22	Application of continuous deflection devices	Traffic Speed Deflectometer (TSD)	Elseifi and Zihan 2018
23			Ferne et al. 2013
24			Flintsch et al. 2013
25			Lee et al. 2016
26			Nasimifar et al. 2017
27			Nasimifar et al. 2018
28			Nasimifar et al. 2019a
29			Nasimifar et al. 2019b
30			Rada and Nazarian 2011
31			Rada et al. 2016
32			Shrestha et al. 2018
33			Zihan et al. 2018
34			Zihan et al. 2019
35		Rolling Wheel Deflectometer (RWD)	Elbagalati et al. 2016
36			Elbagalati et al. 2017
37			Elseifi et al. 2012
38			Nam et al. 2016
39			Steele et al. 2015
40			Zhang et al. 2003a

Strategic Level:

At a strategic level, the PMS helps the public agency administration, managers and the directors in making decisions. Few decisions made at the strategic level are: establishing performance measures and targets, determining funding needs to meet the targets, allocating funds for different programs, determining strategic approaches to maintain and rehabilitate the existing pavement sections etc. The M&R plans developed at the network level and the project level would have to support and help achieve the strategic goals of the agency. The strategic decisions directly impact the network level and the project level M&R plans. Thus, though the individuals at the strategic level do not use the PMS on day-to-day basis, they play an important part on guiding the whole of pavement management decisions. At a strategic level the agency could decide to implement preserve-first strategy, meaning the pavements in the worst condition first would be given higher priority for repair compared to the rest of the pavement sections. Thus, at the network level and project level more detailed planning is done in order to carry out such a strategy (Wolters and Zimmerman 2008).

Network Level:

The decision makers at network level use higher degree of detailed information from the PMS compared to the strategic level. The current and the future condition of the road network are considered for decision-making at this level. The decision-making also considers various budget scenarios and their effects in the short- and long-term, helping decision makers at this level to select the most cost-effective treatments. The recommendations made at this level provides general guideline for where, when and the type of M&R activities to be carried out. The recommendations made from network level analysis are considered to be initial recommendations and detailed tasks for the final work is planned at the project level based on the recommendations received from the network level decision-making (Wolters and Zimmerman 2008).

Project Level:

At this level, the decisions made are very specific. The decisions could require specific details on the existing pavement condition, the amount of traffic, any specialized testing required at the site, selection of materials, specifics on the pavement design, selection of M&R activities for specific

projects etc. The final M&R activity is planned at this level based on the engineering and economic factors as per the recommendations from the network level (Wolters and Zimmerman 2008).

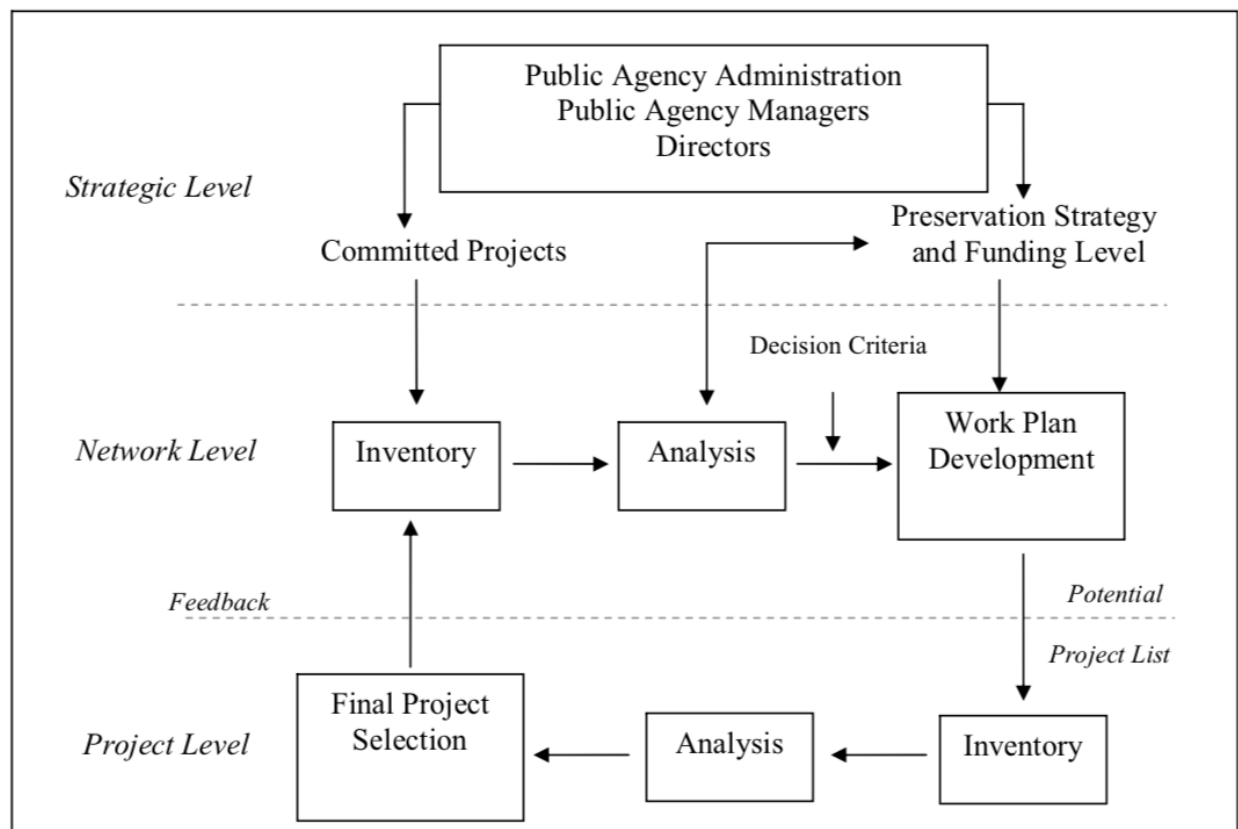


Figure 2-1 Pavement management decision process (Wolters and Zimmerman 2008)

2.1.2 Components of a PMS

In general, designing a pavement management process can be divided into seven-step process. It is important that the agency form a steering committee that works as a group to establish a process that meets the needs of the agency before the implementation. The six-step process is discussed as follows (Wolters et al. 2011):

1 - Defining the Roadway Network and Collecting Inventory Data

Defining the network and understanding the needs of the agency is the first step. Defining roadway network includes parameters such as: pavement surface type, thickness, type of materials used, roadway geometry, pavement condition, geographic boundaries etc. Using these factors, the

network is divided into meaningful segments to meet the agency's M&R needs. The type of data to be collected depends on what the agency needs to meet the objectives and support its decision-making process. Therefore, the agency defines the extent of the needed inventory data for their PMS. Once the agency has defined the road network and estimated the inventory data to be collected, the data collection is carried out.

2 - Collecting Condition data

The pavement condition data includes different distresses observed on the surface of the pavement based on the pavement type. There are a total of thirty-nine recognized distresses for pavements: twenty for asphalt and nineteen for concrete pavements (ASTM D6433-11 2011). Most of these agencies use a combination of different conditions to summarize the overall pavement condition using an index. The current condition data is used to identify the required M&R treatment and predict the future condition of the pavement. There is a tradeoff between the collection of all the condition data required for effective decision-making and enough data for good decision-making, which is governed by the agency needs and the resources associated with it (Wolters et al. 2011).

There are two considerations made during condition data collection: Data quality and Data quantity. The condition data can be collected using either manual or automated data collection methods.

3 - Predicting the Condition

The pavement condition can be predicted by using the average rates of deterioration or by using statistical analysis to develop prediction models. Prediction models are used by the agencies to estimate the future pavement condition and develop multiyear M&R programs based on them. The prediction models can also be used to identify the cost-effective treatment strategy and timing, demonstrate the effects of different investment strategies and also estimate funding requirements based on the pavement condition.

4 - Selecting treatments

Selecting appropriate treatments depends on the agency's M&R strategy. Pavement age, pavement surface condition, traffic, funding availability etc. are some factors that are taken into consideration

while assigning appropriate treatments to the pavement sections. The treatment selection can be either based on cyclical or a set of treatment rules. Cyclical treatment selection refers to the method when the agency applies certain treatment to the pavement section based on pavement age. This method might not result to selection of the most cost-effective treatments as it does not consider the needs of the pavement sections individually.

Treatment selection based on a set of rules uses a set of decision matrix or decision trees based on their trigger values. These decision matrices and decision trees utilize many factors and assign different treatments to the pavement sections based on trigger value for each of these factors. After the treatments are assigned to pavement sections project ranking and cost benefit analysis is performed based on the available funding.

5 - Reporting results

Reporting the results is the fifth step of a pavement management process. Visual aids such as tables, charts and maps can be used to effectively convey the results obtain from the PMS to the decision makers. The results of different investment strategies and what if scenarios can also be presented in the reports enabling decision makers to understand various scenarios and their results.

6 - Selecting a pavement management tool

Pavement management tool provides a platform to store the pavement management database and the ability to perform various analysis with the data. Selection of a pavement management tool depends on the agency's needs. Public domain software, private domain software and spreadsheets with/or GIS are the three different pavement management tools currently available. Spreadsheet is useful for agencies with smaller networks, public domain software are useful for agencies that want a standard rating system and private domain software are useful for agencies that require a customized rating system.

7 - Keeping the process current

The pavement management system is to be updated regularly with the list of completed projects, work history, latest pavement condition data, agency's goals and priorities, management strategies, available funding etc. The accuracy and effectiveness of the PMS depends on the accuracy of the

inputs provided to it. Thus, updating and reviewing steps 1 to 5 is crucial for accurate and effective decision-making. The decision makers can also provide feedback to the PMS which would make the PMS more effective.

2.1.3 Pavement Surface Condition Indices

Most of the state highway agencies utilize the pavement surface condition data for network level decision-making and the structural condition information for project level evaluations. State highway agencies use a condition index that combines multiple distresses to summarize the overall condition of the pavement. These condition indices mostly range from either a score of 0-100 or 0-5. Few examples some condition indices used by state highway agencies are Pavement Condition Index (PCI), Critical Condition Index (CCI), Overall Pavement Index (OPI), Pavement Quality Index (PQI) etc.

Pavement Condition Index (PCI)

The PCI was developed by U.S. Army Corps of Engineers. The PCI rates the pavement surface condition into 10 different condition categories. The PCI scale ranges from 0 to 100, 100 being the pavement section without any visible distress and 0 being a pavement that is completely damaged. A total of thirty nine pavement distresses have been documented in by ASTM D6433, *Standard Test Method for Roads and Parking Lots Pavement Condition Index Survey* (ASTM D6433-11 2011). Out of the total thirty nine distresses, twenty types of pavement distresses are identified for asphalt pavements and nineteen distresses types are identified for concrete pavements. Each of the distresses has a deduct value assigned to them based on their severity and extent. Distresses that severely affect the pavement condition have higher deduct values assigned to them. The deduct values are then summed up and deducted from a full score of 100 to get the PCI. “The pavement condition index though indicative of the pavement structural integrity cannot necessarily measure the pavement structural condition” (ASTM 2011).

Critical Condition Index (CCI)

The critical condition index is an overall pavement condition indicator used by Virginia Department of Transportation (VDOT). The CCI is defined as the lower of the Load related distresses (LDR) and the Non-load related distresses (NDR). LDR is comprised of distresses that

are caused due to vehicular load to the pavement. Some examples of the LDR are: alligator cracking, patching, potholes, rutting, delamination etc. NDR is comprised of distresses that are not caused due to the vehicular load and are caused by the effects of temperature, rain, defects of materials used etc. Some examples of NDR are: block cracking, reflection cracking, bleeding etc.

The LDR and NDR are based on deduct values. Deduct values assigned to each of the distresses are then summed up and subtracted from a total score of 100 to calculate the LDR and the NDR. The minimum of the LDR and NDR is assigned as the CCI as shown in equation 2-1 (McGhee 2002).

$$CCI = \text{Minimum of } LDR, NDR \quad (2-1)$$

Load related index(LDR):

$$LDR = 100 - RUT_{DED} - A_CR1_{DED} - A_CR2_{DED} - PAT_WP_{DED} - OTHRLD_{DED} \quad (2-2)$$

Where, RUT_{DED} = Rutting deduct value; A_CR1_{DED} & A_CR2_{DED} = Alligator cracking deduct values; PAT_WP_{DED} = Patching deduct values; $OTHRLD_{DED}$ = Other load related distress deduct values

Non loaded related index(NDR):

$$NDR = 100 - LIN1_{DED} - LIN2_{DED} - RF1_{DED} - RF2_{DED} - RF3_{DED} - PAT_{NWP_{DED}} - OTHRND_{DED} \quad (2-3)$$

Where, $LIN1_{DED}$ & $LIN2_{DED}$ = Linear cracking deduct values; $RF1_{DED}$, $RF2_{DED}$ and $RF3_{DED}$ = Reflective cracking deduct values; $PAT_{NWP_{DED}}$ = Patching deduct values; $OTHRND_{DED}$ = Other non-load distress deduct value.

Further details on the calculation of the CCI, LDR and NDR indices has been discussed in the paper McGhee (2002).

Overall Pavement Index (OPI)

Pennsylvania uses OPI to summarize the condition of their pavements. OPI is an index ranging from 0-100 that considers different pavement distresses along with the International Roughness Index (IRI). For asphalt pavements, the distresses considered during the calculation of OPI are:

alligator cracking, transverse cracking, miscellaneous cracking, edge deterioration, patching, raveling and rutting. The calculation of OPI is also based on the deduct values assigned to each distress based on their extent (Gharaibeh et al. 2011).

For flexible pavements, the OPI is calculated as follows:

$$OPI_{ACP} = (0.25 \times RUF) + (0.15 \times FCI) + (0.125 \times TCI) + (0.10 \times MCI) + (0.10 \times EDI) + (0.05 \times BPI) + (0.05 \times RWI) + (0.175 \times RUT) \quad (2-4)$$

Where, $RUF = 100 - ((0.27 \times IRI) - 11)$, FCI–Fatigue Cracking Index, TCI–Transverse Cracking Index, MCI–Miscellaneous Cracking Index, EDI–Edge Deterioration Index, BPI–Bituminous Patching Index, RWI–Raveling / Weathering Index and RUT–Rut Depth Index. The detailed calculation of each individual distress index has been explained by Gharaibeh et al. (2011).

Pavement Quality Index (PQI)

Minnesota Department of Transportation (MnDOT) uses PQI as their pavement condition index. The PQI is composed of the Ride Quality Index (RQI) and the Surface Rating (SR) and ranges from a score of 0-4.5. The PQI is calculated using the following equation (Janisch 2015):

$$PQI = \sqrt{RQI \times SR} \quad (2-5)$$

Where, $RQI = 5.697 - (0.264)(\sqrt{IRI})$; IRI is International Roughness Index in inches/mile; $SR = e^{(1.386 - (0.045)(TWD))}$; TWD stands for Total Weighted Distress.

Further details on the calculation of TWD has been explained by Janisch (2015).

Pavement Condition Rating (PCR)

Ohio Department of Transportation (ODOT) uses PCR as a rating to summarize the condition of their roads based on visual inspection of the pavement distresses. A full PCR score of 100 means the pavement with no observable distress and a score of 0 represents pavement with all distress present at “High” levels of severity and “Extensive” levels of extent. The calculation of PCR is based on the summation of deduct points for each type of distress observed. The deduct values depend on the types of the distress and the level of their extent and severity. For flexible pavements 13 types of distresses, 3 levels of distress severity (Low, Medium, High), and 3 levels of distress

extent (Occasional, Frequent and Extensive) are considered. Based on the PCR values the pavement condition can be categorized into six different levels: very good, good, fair, fair to poor, poor, and very poor. The PCR is calculated using the following equation:

$$PCR = 100 - \sum_{i=1}^n Deduct_i \quad (2-6)$$

Where, n= number of observable distresses; Deduct = (Weight for distress type) (Weight for severity) (Weight for Extent)

The deduct value is calculated by multiplying the weight for distress type, weight for severity, and weight for distress extent. The weight for distress type is the maximum number of deductible points for each type of distress observed (Saraf 1998).

2.2 Understanding Pavement Deflections

2.2.1 Deflection bowl parameters

Horak (2006), presented a benchmarking technique using deflection bowl parameters from the FWD in conjunction with the standardized visual survey methodology to give an insight on the individual layer strengths and the cause of structural distress. The visual condition survey and structural condition were categorized in a three-tiered condition rating as sound, warning, or severe. The study utilized existing procedures to classify pavement visual condition into different categories. Further details on the visual and structural classification procedures can be found in the paper. Using the same rating system, the structural strength of different layers can be linked to the pavement visual condition rating. A demonstration example of the benchmarking methodology was also presented. The example analyzed the pavement structural condition from the bottom to the top based on different structural condition parameters presented in Table 2-2. The visual condition indicated mostly warning to severe sections owing to cracking, potholes and rutting. Using the benchmarking procedure, the origination of the distress was linked to the presence of a weaker sub-base layer showing lack of support. Thus, the cause of the distress was identified without detailed knowledge of layer thickness and limited built information (Horak 2006).

Table 2-2 Summary of deflection bowl parameters (Horak 2006).

Parameter	Formula	Zone correlated to	Structural indicator
Maximum Deflection	D0	1,2,3	all structural layers with about 70% contribution by the subgrade
Base Layer Index (BLI)	$BLI = D0 - D300$	1	base layer
Middle Layer Index (MLI)	$MLI = D300 - D600$	2	subbase layer
Lower Layer Index (LLI)	$LLI = D600 - D900$	3	subgrade layer
<i>where 1 ,2, and 3 refer to Zone of Positive Curvature, Zone of Curvature Inflection, and the Zone of Reverse Curvature respectively</i>			

2.3 Structural condition indices for network-level pavement management

Some of indices based on the structural condition and developed for use at the network level using stationary deflection devices are as follows:

2.3.1 Structural Adequacy Index (SAI)

The structural adequacy index was originally developed in 1994 by Hass to evaluate the structural status of the pavement section or an entire network as a whole. It utilized the deflection data from the Benkelman beam originally but later on was also used with the FWD data. The deflection data is transformed into SAI on a scale of 0 to 10 or 0 to 5 by establishing a maximum tolerable deflection (MTD) for the expected number of equivalent single axle loads (ESALs) and then comparing the maximum deflection to the MTD. If the maximum deflection matched the MTD, the SAI of the pavement would be 5 (on a scale of 0 to 10). Deduct values are then assigned to higher and lower deflections measurements to calculate the SAI's <5, representing a structurally inadequate pavement or SAI's >5 representing structurally adequate problem (Haas et al. 2001).

2.3.2 Structural Strength Index (SSI)

The Structural Strength Index (SSI) was developed by the Texas Transportation Institute (TTI) for use in Texas Department of Transportation's (TxDOT) pavement management system. The SSI was based on i) Surface curvature index and ii) FWD deflection sensor 7 (W_7) which is 72 inches from the applied load. The FWD deflections are measured under a 9,000 lb load with seven sensors that are spaced 12 inches apart from each other. Each combination of surface curvature index and

W_7 was assigned a structural strength value from 0.1 to 1.0. Traffic level and rainfall were then taken into consideration to calculate the final SSI (Scullion et al. 1988).

The surface curvature index is calculated using the following equations:

$$\text{Surface Curvature Index} = W_1 - W_2 \quad (2-7)$$

where, W_1 = first sensor of the FWD; W_2 = second sensor of the FWD

The final structural strength index (SSIF) was calculated using the following equations:

$$SSIF = 100 \times (SSI)^{1/(RF \times TF)} \quad (2-8)$$

where, RF= Rainfall Factor; TF= Traffic Factor.

2.3.3 Structural Number (SN)

Rohde developed a method to calculate the structural number using the total pavement thickness and the shape of the deflection bowl from the FWD test. This approach of calculating the SN was verified on 62 pavement structures. Rhode's equation for calculation of structural number is as follows (Rohde 1994):

$$SIP = D_0 - D_{1.5Hp} \quad (2-9)$$

where, D_0 = Peak deflection measured under a standard 9000 lb FWD load; $D_{1.5Hp}$ = deflection measured at an offset of 1.5times H_p under standard 9000 lb FWD load; H_p = total pavement thickness (mm).

$$SN = k_1 SIP^{k_2} H_p^{k_3} \quad (2-10)$$

where, SN= Structural number; SIP = Structural Index of pavement (μm); H_p = total pavement thickness (mm), k_1 , k_2 and k_3 = coefficients for different surface types.

2.3.4 Structural Condition Index (SCI*)

Zhang et al. proposed Structural Condition Index (SCI*) as a structural index for TxDOT to be used in the selection of M&R projects. The SCI* is defined as the ratio of the existing structural number to the required structural number of the pavement. The required structural number is

calculated using estimated ESALs for the next 20 years. A SCI* value of one or higher represents pavement that is structurally sound whereas, SCI value of less than one represents the need for structural improvement (Zhang et al. 2003b).

The SCI is represented mathematically as follows:

$$SCI^* = \frac{SN_{eff}}{SN_{req}} \quad (2-11)$$

where, SCI* = Structural Condition Index; SNeff = existing Structural Number; SNreq = required Structural Number.

2.3.5 Modified Structural Index (MSI)

Bryce et al. developed a structural index that incorporated the traffic, pavement thickness, and deflections obtained from the FWD into one index called the Modified Structural Index (MSI). The MSI can be used as a screening tool at a network level to identify sections that might be structurally deficient. Low MSI values represent pavement in poor structural condition whereas high MSI values represent pavements in strong structural condition (Bryce et al. 2013).

$$MSI = \frac{K_1 \times (D_0 - D_{1.5Hp})^{K_2} \times H_p^{K_3}}{\alpha \times (\log(ESAL) - 2.32 \times \log(M_R) + \beta)^\gamma} \quad (2-12)$$

where, D_0 = FWD center deflection under 9000lbs load; $D_{1.5Hp}$ = Deflection at 1.5 times the pavement thickness; H_p = Pavement thickness; ESAL = Calculated traffic; M_R = Resilient modulus = $((0.33 \times 9,000 \times 0.24)) / (D_{60} \times 60)$; D_{60} = Deflection at 60 inches away from the center of the load; K_1 , K_2 , K_3 , α , β , and γ are constants that depend on the different types of roads: Interstates, Primary and Secondary.

2.4 Continuous deflection measurement devices

Flintsch et al. (2013) defined continuous deflection measurement devices as “deflection measuring device constantly moving that can collect data at intervals of approximately 300 mm (1 ft) or smaller using load levels typical of truck loading (i.e. 40-50 kN (9-11 kips) per wheel or load assembly” (Flintsch et al. 2013).

Currently three continuous deflection measurement devices are available that can operate at traffic speeds: 1) Traffic Speed Deflectometer 2) Rolling Wheel Deflectometer 3) Rapid Pavement Tester.

2.4.1 Traffic Speed Deflectometer (TSD)

Traffic Speed Deflectometer is an articulated truck that is equipped with a series of Doppler lasers that measure the pavement deflection velocity while traveling at a speed of up to 60 mph (100 km/h). TSDs that have been operated in the United States have been equipped with six Doppler positioned at 100mm, 200mm, 300mm, 600mm, 900 mm, and 1,500 mm (i.e. 4 in., 8 in., 12 in. 24 in., and 60 in.) in front of the loading axle to get the deflection bowl. A seventh sensor is positioned 3,500 mm (i.e. 137 in.) in front of the rear axle, largely outside the deflection bowl, to act as a reference laser. The lasers are mounted on a servo-hydraulic beam that measures the lasers at a constant height from the pavement's surface. The truck trailer is also equipped with a climate control system that maintains the trailer temperature at 68°F (20°C) to prevent thermal distortion of the steel measurement beam that houses the laser measurement system (Katicha et al. 2017; Shrestha 2017).

The Doppler lasers are mounted at a small angle to the vertical to measure the vertical pavement deflection velocity together with components of the horizontal vehicle speed. The pavement deflection velocity is divided by the instantaneous vehicle speed to obtain the deflection slope as shown in equation 2-13 (Katicha et al. 2017).

$$S = \frac{V_v}{V_h} \quad (2-13)$$

Where, S = deflection slope, V_v = vertical pavement deflection velocity, and V_h = vehicle horizontal velocity.

The deflection measurements are obtained by integrating the thus obtained deflection slope measurements as shown in equation 2-14.

$$d(x) = \int_x^\infty s(y)dy \quad (2-14)$$

Where, $s(y)$ = the slope at distance y measured from the applied load, and $d(x)$ = the deflection at distance x measured from the applied load.

The latest generation of the TSDs is being equipped with newer versions of the Doppler lasers. The number of doppler lasers present on the TSD may vary from device to device.

2.4.2 Rolling Wheel Deflectometer (RWD)

The first generation of the Rolling Wheel Deflectometer was launched in 2003 for demonstration projects at numerous field tests throughout the United States. The RWD was 2.60 m wide, 3.66 m high, and 22.88 m long. It consisted of a single double-wheeled axle trailer 16 meters in length and housed a set of four triangulation lasers attached to an aluminum beam that was mounted beneath the trailer. Three lasers were placed in front of the loaded rear axle to define the unloaded pavement surface profile, whereas the fourth laser was placed between the dual tires to measure the deflected pavement surface. Due to the length of the trailer, the RWD was able to isolate the deflection basin produced by the RWD trailer from deflections produced by the RWD tractor. The deflection was calculated by comparing the undeflected pavement surface with the deflected pavement profile at the same location. Further details on this technique can be found in Harr and Ng-A-Qui (1977). (Harr and Ng-A-Qui 1977; Gedafa et al. 2010).

The second generation of the RWD uses an imaged-based system to measure the pavement deflection beneath the dual tires that impart load to the pavement. It collects pairs of spatially coincident images of the pavement surfaces: first undeflected image of the pavement surface and second the deflected image after the truck moves forward. These images are then processed by a customized image processing algorithm that determines the corresponding vertical deflection. The RWD typically collects images every 25 ft and averages deflection basins every 500 ft for use in network-level management (Steele et al. 2020).

2.4.3 Rapid Pavement Tester (RAPTOR)

Rapid Pavement Tester (RAPTOR) is a continuous deflection measurement device developed by Dynatest. It is a truck trailer that consists of 12 line lasers that are mounted on a steel beam that spans from 1.5 m behind the wheel to 3.6 m in front of the wheel. The RAPTOR applies a load of 50 kN load on each rear wheel. As the RAPTOR drives over the pavement, it generates surface

deflection which is measured by the line lasers. The line of the laser is also referred to as the “detection window” and contains over 1000 individually measured points in a width of 200mm. The detection window serves to ensure that all the points seen by the first lasers are also seen by all the other lasers. Using 12 line lasers with a measurement rate of 4000Hz the RAPTOR produces a 3D scan of the pavement. Image correction technique is then used to measure the distance of the pavement at the same point. A set of gyroscopes and accelerometers are also mounted on the support beam to measure the changes in its horizontal and vertical alignments (Athanasiadis and Zoulis 2019; Madsen and Pedersen 2019).

Details on the measurement principle of the RAPTOR can be found in Athanasiadis and Zoulis (2019).

2.5 Application of continuous deflection measuring devices.

2.5.1 Use of continuous deflection devices at Network Level

Ferne et al. (2013) summarized the methodology for assessing the structural strength of the road network using a TSD for the UK Highways Agency. TSD and Deflectograph surveys were carried out on 107 - 100m sites. For short-term implementation, the study developed a relationship between the deflection slope measured from the TSD and the deflection measurements from UK Deflectograph. Each of the 1m TSD slopes was converted to an estimated peak Deflectograph value. Using construction history and traffic information along with the estimated peak deflectograph value, one of four levels of the Network Structural Condition (NSC) category (as shown in Table 2-3) was assigned to each 100m test segments. The methodology is now an established part of the Highways Agency’s pavement assessment strategy.

Table 2-3 UK Network Structural Condition Categories

Category	Description
1	Flexible pavements without any need for structural maintenance
2	Flexible pavements unlikely to need structural maintenance
3	Flexible pavements likely to need structural maintenance
4	Flexible pavements very likely to need structural maintenance

In 2013, Federal Highway Administration (FHWA) initiated a pooled fund project to perform a field demonstration of the TSD for interested state departments of transportation (DOTs). The TSD showed good long-term and short-term repeatability. The study concluded that the TSD was capable of differentiating between relatively structurally strong and weak pavement sections and that it provided a more detailed assessment when used in conjunction with the PMS information. The study also presented a framework and an example application, based on VDOT's two-step decision-making approach, to incorporate the TSD measured structural condition index into a SHA's PMS as shown in Figure 2-2. The example modified the original recommended treatment from the distresses using the pavement structural condition obtained from the TSD (Katicha et al. 2017).

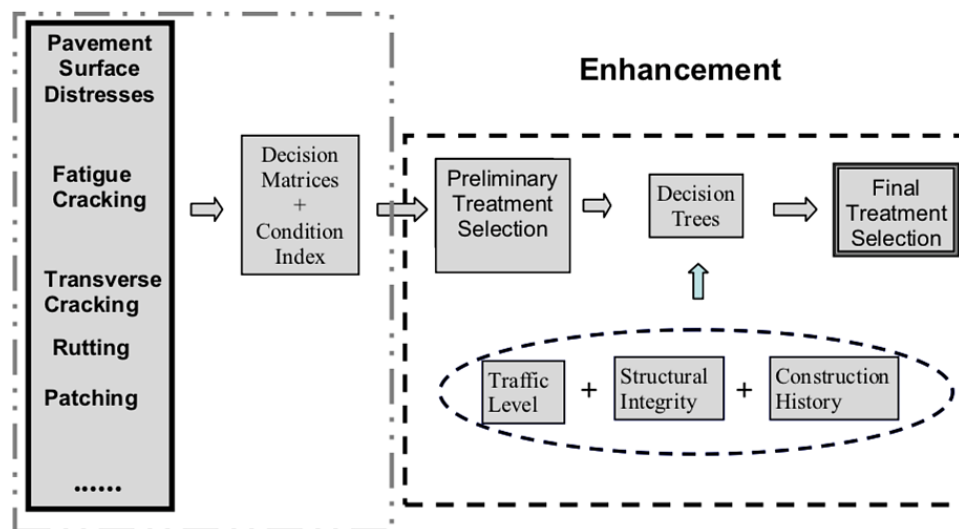


Figure 2-2 VDOT two-phase decision process (Katicha et al. 2017)

Steele et al. (2015) investigated the potential benefit of integrating the RWD data into the Oklahoma Department of Transportation's (ODOT) pavement management process. The study was performed on 1000 miles of ODOT's highway with the RWD data matched to each of the pavement management sections. The study utilized ODOT's FWD deflection ratios to categorize the pavement structural condition into good, fair, and poor sections based on the RWD data. The study concluded that incorporating the RWD-based structural index resulted in significant cost savings due to the use of more cost-effective pavement preservation techniques on pavements with

stronger pavement structural conditions. Also, delaying treatment on roads with a poor structural response until pavement rehabilitation was a cost-effective option helped to make better use of the funding.

Rada et al. (2016) assessed, evaluated, and validated the capability of traffic speed deflection devices for network-level pavement structural evaluation. The study confirmed that the two traffic speed deflection measuring devices: TSD and RWD met a set of minimum specifications related to network-level pavement structural evaluation including accuracy and precision requirements. 3D move software was used to explore the relationships between the load-induced structural-related response and the corresponding deflection basin indices. The study concluded that the deflection index SCI_{12} based on deflections at 0 and 12 inches, (also known as SCI_{300} i.e. 0 and 304.8mm) best reflects the upper portion of the pavement system which was significantly influenced by the AC thickness and modulus. DSI_{4-18} and SCI_{12} were recommended as the most appropriate indices regardless of the AC thickness. An approach to enhance the decisions based on traditional condition metrics by implementing the TSD based indices was also presented as shown in Figure 2-3.

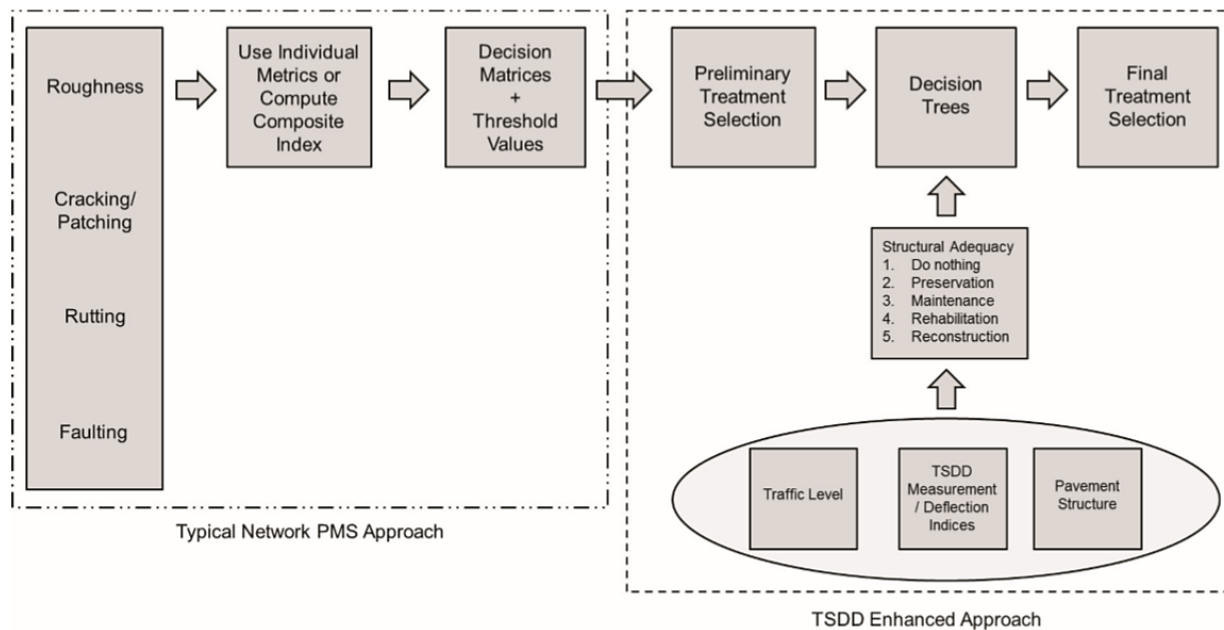


Figure 2-3 Flowchart of an idealized PMS containing TSDD structural evaluation component (Rada et al. 2016)

Elbagalati et al. (2017) introduced a framework to incorporate the pavement structural condition obtained from the RWD into Louisiana's pavement management system decision matrix. Pavement structural number was computed from deflection measurements from the RWD using the relationship shown in equation 2-15. The relationship was developed by Elbagalati et al. (2016) using SAS9.4 software. Structural condition index (SCI), which is the ratio of the existing structural number to the required structural number (Zhang et al. 2003a), was calculated from the SN obtained from the RWD. The pavement structural condition based on the SCI values was categorized into 4 intervals from very low to high. The study proposed an enhanced decision tree that incorporated the SCI in the decision-making process as shown in Figure 2-4. Implementation of the enhanced decision tree showed that incorporation of the pavement structural capacity at the network level decision-making process would save state agencies significant funds on roads with high traffic levels. Applying the enhanced decision tree on local and minor roads did not result in cost savings. This might be because the SCI concept for low-volume roads results in overestimated treatment selection and the cost of RWD testing is comparatively higher for local roads (Elseifi et al. 2012; Nam et al. 2016).

$$SN_{RWD0.16} = -14.72 + 27.55 * \left(\frac{AC_{th}}{D_0} \right)^{0.04695} - 2.426 * \ln SD + 0.29 * \ln ADTPLN \quad (2-15)$$

Where, AC_{th} = asphalt concrete layer's thickness of the pavement structure (in.); D_0 = average RWD deflection measured each 0.16 km (mils) ; SD = standard deviation of the RWD deflection each 0.16 km (0.1 mi); $AADTPLN$ = average annual daily traffic per lane (vehicle per day); $SN_{RWD0.16}$ = pavement structural number based on RWD measurements defined each 0.16 km (0.1 mi).

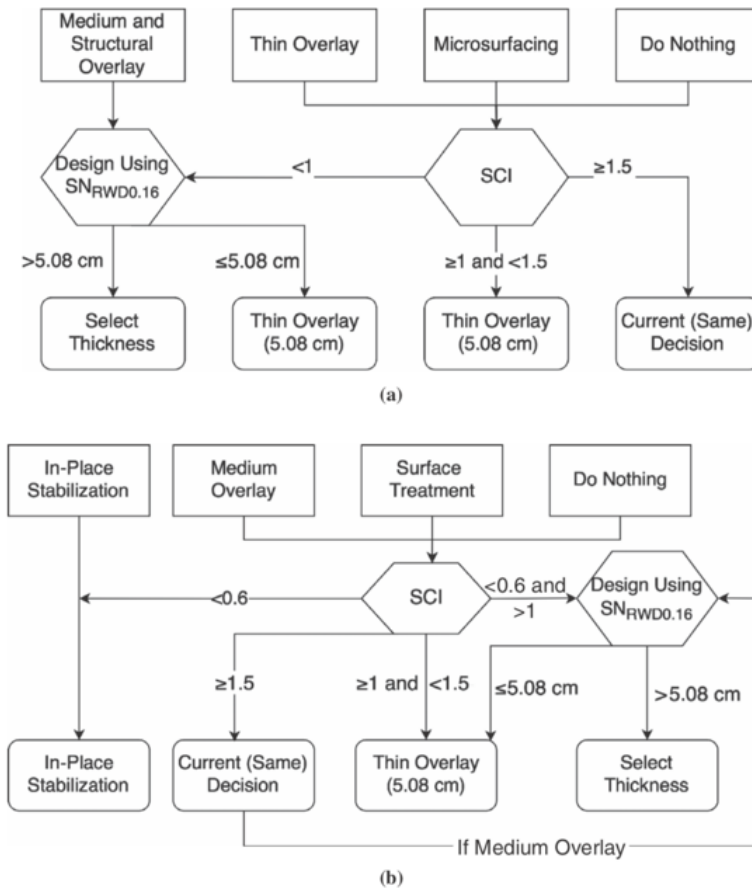


Figure 2-4 Enhanced decision trees proposed for (a) arterials and (b) collectors (Elbagalati et al. 2017)

Shrestha et al. (2018) evaluated the TSD and presented an approach to implement the TSD measurements in a Pavement Management System (PMS). The study presented an overview of the TSD testing, data processing and results of the evaluation carried out during the Federal Highway Administration (FHWA) initiated pooled fund study with nine participating state highway agencies (SHAs). The study utilized D0, SCI300, and Rohde's SN_{eff} to represent the pavement structural condition. Temperature adjustment developed by Rada et al was carried out in the study. The study reported good short-term and long-term repeatability of the TSD with the measurements following similar trends. A comparison of the TSD with FWD on a section of I81 showed that both the devices followed similar trends. Rehabilitation work on a section of I81 was picked up by both the devices, as shown in Figure 2-5, showcasing the TSD's ability to identify pavement sections with varying structural condition. An example using the pavement structural condition to supplement

VDOT's existing decision-making process was presented. The pavement structural condition was used to modify the treatment category based on the pavement surface condition was presented, as shown in Figure 2-6.

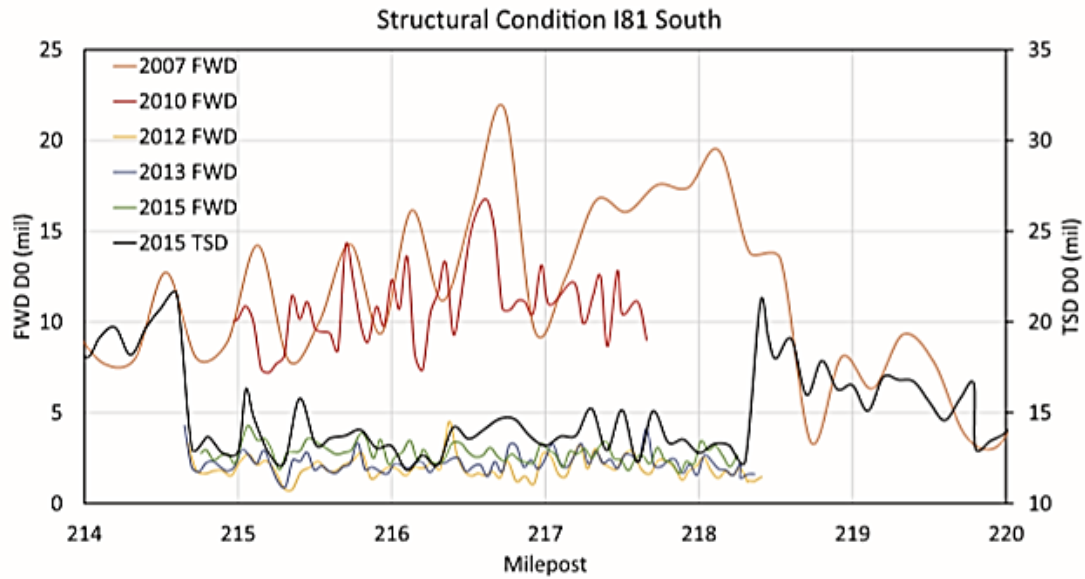


Figure 2-5 Comparison of D0 from TSD and FWD on a rehabilitated section on I-81 South
(Shrestha et al. 2018)

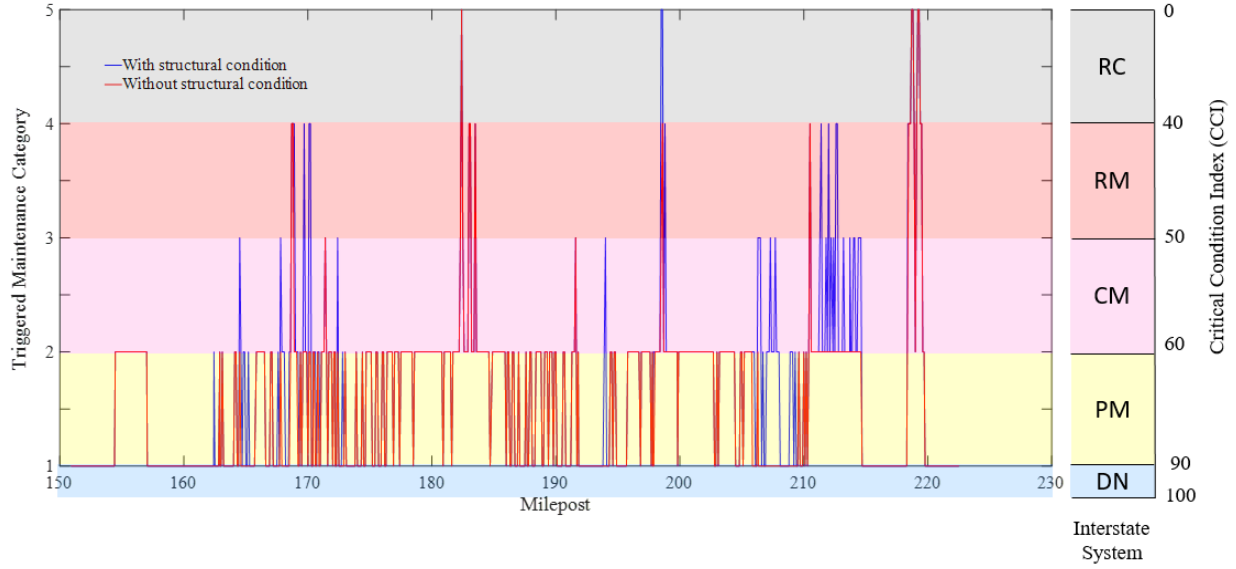


Figure 2-6 Maintenance category before and after considering the pavement structural condition (Shrestha et al. 2018)

Elseifi and Zihan (2018) assessed the feasibility of using the TSD measurements for network-level pavement structural evaluation and backcalculation analysis. A SN predictive model was developed and used to identify if the pavement were structurally sound or structurally deficient. The nonlinear regression model, developed using SAS 0.4, was based on statistically significant TSD deflections (D_0 and D_{48}), ADT, and total pavement thickness (T_{th}) as shown in equation 2-16. During the model validation, the coefficient of determination (R^2) was 0.88 with a RMSE of 1.06.

$$SN_{TSD} = 18.67 \times e^{(-0.013 \cdot D_0)} + 8.65 \times (D_{48})^{0.11} + 0.18 \times (T_{th}) + 0.31 \times \ln(ADT) - 24.28 \quad (2-16)$$

Where, SN_{TSD} = SN based on TSD measurements; D_0 = Deflection under the load, i.e. Center Deflection (mils); D_{48} = Deflection at 48 in. distance from Center Deflection (mils); T_{th} = Total layer thickness of pavement (in.); ADT = Average Daily Traffic (veh/day).

The estimated percentage loss in pavement structural capacity was calculated and was found to agree with the percentage loss calculated from the FWD. The study developed an Artificial Neural network (ANN) model to predict the corresponding FWD deflections (TSD*) from TSD deflection measurements. The backcalculated moduli from the FWD measurements and the TSD*

measurements showed good agreement as shown in Figure 2-7. The model was also successfully validated by comparing the critical pavement responses, number of cycles to fatigue failure, and Structural health Index (SHI) calculated from the FWD and TSD* measurements (Elseifi and Zihan 2018).

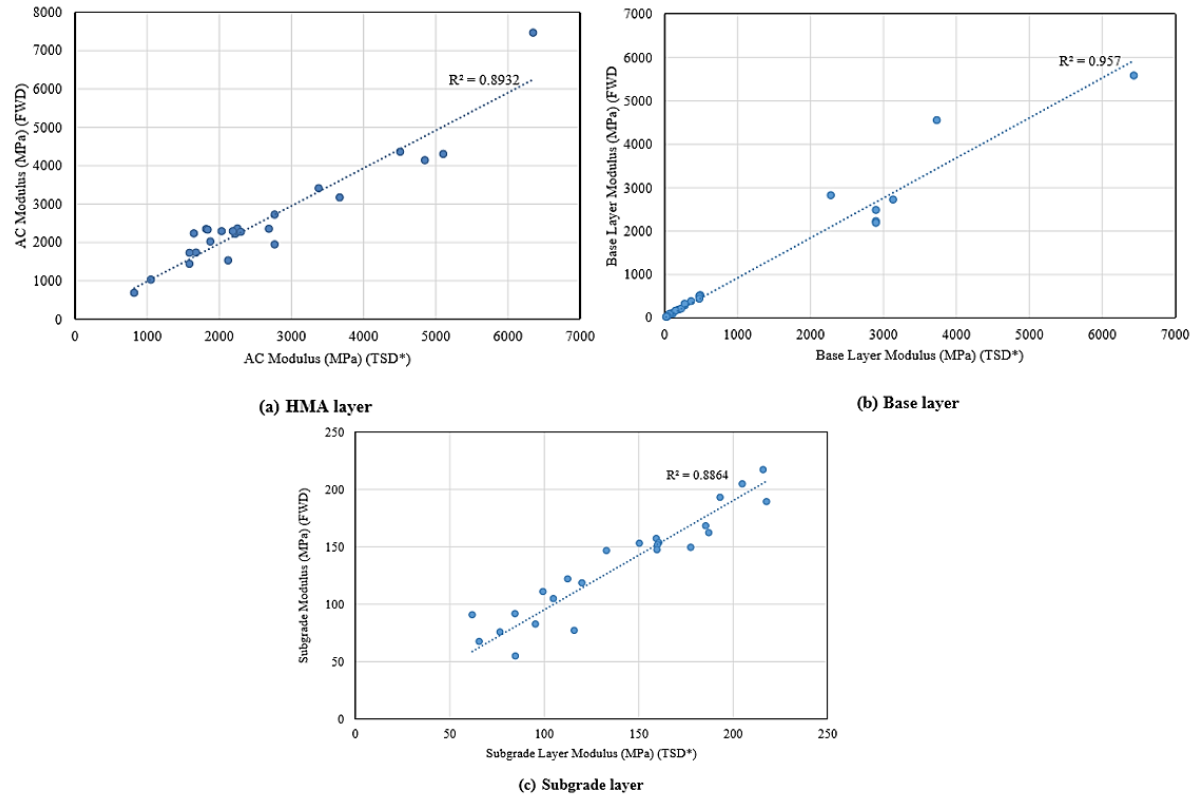


Figure 2-7 Correlation between backcalculated layer moduli Using FWD measurements and TSD* (Elseifi and Zihan 2018)

Nasimifar et al. (2019b) developed an approach to compute and utilize the SN using the TSD deflection measurements for network-level PMS applications. The SN proposed in the paper recalibrated Rohde's SN_{eff} model for TSD loading configuration, 3D-Move was used to generate a database of 426 pavement structures with pavement thickness and layer modulus information. The pavement structures in Viscoelastic (VE) analysis were evaluated with linear elastic analysis (LEA) under circular loading of 40kN. The new calibration coefficients were derived by minimizing the difference between the estimated values from equation 2-17 using VE analysis of TSD and AASHTO NDT approach (shown in equation 2-18) using LE analysis results.

Proposed Model:

$$SN_{eff} = C_1 SIP^{C_2} H_p^{C_3} \quad (2-17)$$

Where, SIP = structural index of pavement; H_p = total pavement thickness (mm); C_1 , C_2 and C_3 = calibration coefficients $C_1 = 0.4369$, $C_2 = -0.4768$ and $C_3 = 0.8182$.

AASHTO NDT method:

$$SN_{eff} = 0.0045 H_p^3 \sqrt[3]{E_p} \quad (2-18)$$

Where, H_p = total thickness of all pavement layers above subgrade (inches); E_p = effective modulus of pavement layers above subgrade (psi).

The study also presented an example application on utilizing the SN_{eff} calculated from the proposed model. Structural Number Ratio (SNR), which is the ratio of SN_{eff} to SN_{req} was used to evaluate the structural adequacy of the pavement section. Pavement sections with $SNR < 1$ indicated sections that needed structural intervention and required overlay thickness to meet the structural needs. Remaining structural life was also calculated and assigned for the sections. The RSL was estimated by comparing the remaining ESALs with the expected ESALs for the sections (Nasimifar et al. 2019b).

$$SNR = \frac{SN_{eff}}{SN_{req}} \quad (2-19)$$

$$a H_o = SN_{req} - SN_{eff} \quad (2-20)$$

Where, A = overlay structural coefficient (i.e. 0.44 for asphalt); H_o = required overlay thickness; SN_{req} = required SN based on traffic for design period and subgrade modulus; SN_{eff} = effective SN of the existing pavement.

Zihan et al. (2019) assessed if the use of the surface indices and its rate of deterioration only could be used to identify structurally deficient pavement sections instead of relying on TSD and RWD measurements. The structural deficiency of the pavement sections was calculated using loss in SN (%) as shown in equation 2-21. The SN was calculated from RWD and TSD using equation 2-15 and equation 2-16 respectively. The results showed that although some correlation was present between the structural condition and the rate of surface deterioration, the pavement surface

condition cannot be used to accurately predict the pavement structural condition. For RWD, alligator cracking, the most accurate surface index, erroneously identified 35% of the structurally sound sections as structurally deficient and 51.5% of the structurally deficient sections as structurally sound sections. For the TSD random cracking index was the most accurate surface index identifying 16.7% of structurally sound sections as structurally deficient and 51% of structurally deficient sections a structurally sound. The study concluded that incorporating the pavement structural condition into the PMS decision-making process can assist in cost-effective decision-making. (Zihan et al. 2019).

$$Loss\ in\ SN(\%) = \frac{Design\ SN - SN_{RWD/TSD}}{Design\ SN} \times 100 \quad (2-21)$$

Nasimifar et al. (2019a) summarized the efforts on developing analysis methodologies and procedures for using the TSD in network-level pavement management activities. The study summarized previous methodologies developed by the authors for 1) numerical simulation of TSD using 3D-Move program (Rada et al. 2016), 2) backcalculation of pavement layer moduli from TSD measurements (Nasimifar et al. 2017), 3) prediction of critical pavement responses through Deflection Indices from TSD (Rada et al. 2016), 4) temperature adjustment of SCI300 index from TSD (Nasimifar et al. 2018) and 5) estimation of SN from TSD measurements (Nasimifar et al. 2019b). The study presented an example of implementing the developed methods on a 180km road section in Route 60 in Virginia. An Excel-based tool was developed and used for data extraction and processing the structural data (TDEPS). The tool estimated parameters such as SCI_{TSD} , temperature corrected SCI_{TSD} , and SN from the TSD measurements. The pavement structural condition information was then used to either confirm or adjust the decisions based on the traditional pavement condition metrics. This was done in two stages: 1) Bumping up the pavement treatment categories depending on the pavement structural condition 2) Estimating AASHTO's overlay thickness based on the SN. Based on the Critical Condition Index (CCI) 53% of all the section was identified as Do Nothing (DN) section, incorporating the pavement structural condition suggested that 26% of these sections might require higher treatment. The example presented showed that pavement condition indices based on visible surface distresses (CCI in the example) do not always agree with the pavement structural condition, confirming the need for

pavement structural evaluation in pavement management decision-making (Nasimifar et al. 2019a).

2.6 Applications at the project level

Lee et al. (2016) presented an exploratory study on using the TSD for project-level pavement evaluation based on data collected in 2014 and 2015 in Queensland, Australia. A comparison between the FWD and TSD was performed. The FWD was found to have a higher maximum deflection compared to the TSD. The study utilized CIRCLY to compute theoretical deflection basins for the TSD and used EFROMD3 backcalculation software to estimate the pavement layer moduli. CIRCLY is the principal software used for pavement analysis and design in Australia. It conducts response to static load calculations based on a linear elastic model. The study used EFROMD3 for backcalculation as it relies on CIRCLY as the mechanistic computation engine. The study showed that backcalculation analysis can be performed using TSD deflection measurements to estimate the pavement layer moduli.

2.7 Summary of the Literature Review

This chapter presented a literature review on some of the relevant topics on the pavement management system, different indices used in the PMS, recently developed continuous deflection devices, and their application for pavement management. The Pavement Management System plays a significant role in assisting various state agencies to manage and rehabilitate their pavements. Both the pavement management system and the decision-making approaches have been evolving since they were first introduced over a few decades ago. This chapter summarized various surface condition and structural condition indices used by different state agencies in their PMS. Most of the structural condition indices were based on FWD measurements. With the advent of new technologies in the last decade, continuous deflection devices have evolved to make it easier to collect pavement structural condition information at the network level. The literature review summarized the potential continuous deflection devices and their applications for network-level pavement management.

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Chapter 3. Pavement Deterioration Modeling and Network-Level Pavement Management Using Continuous Deflection Measurements ¹

3.1 Abstract

This paper shows that the pavement structural condition as measured by the traffic speed deflectometer (TSD) has an impact on the rate of deterioration of the surface condition on a 6,469-km (4,020-mi) subset of the Virginia Department of Transportation (VDOT) network. Because of this finding, a structural condition matrix was developed to augment the treatment selection process currently used by VDOT. The treatment categories considered by VDOT are do nothing (DN), preventive maintenance (PM), corrective maintenance (CM), restorative maintenance (RM), and reconstruction (RC). The augmented matrix modifies these treatments based on whether the structural condition is strong, fair, or weak. Applying the augmented matrix on the tested interstate network resulted in reducing the percentage of the network requiring CM and increasing the percentage requiring PM and RM. The percentage of the network requiring DN or RC remained practically the same. Initially, D_0 (deflection measured under the applied load), $SCI300$ (surface curvature index), and SN_{eff} (effective structural number) were investigated as possible structural condition parameters to use. Only the results obtained with $SCI300$ are shown because it was the parameter that best explained the change in the rate of deterioration as a function of structural condition.

3.2 Introduction

State agencies rely on pavement management systems (PMS) to keep track of the pavement condition and maintain them. Pavement M&R are some of the most costly transportation infrastructure activities carried out (Wang et al. 2003, Gao et al. 2012). Thus, PMSs have evolved as the primary tool to support and plan pavement M&R activities. An important step in this process is to analyze the current pavement condition and predict the future pavement condition.

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Over the years, various studies have looked into predicting the pavement future condition and modeling the pavement deterioration behavior. Li et al. (1996), Kleiner (2001), Yang et al. (2005), Osorio-Lird et al. (2018), have used Markovian models to estimate the pavement deterioration. Pozzie et al. (2017) and Špačková and Straub (2017) have used the Markov process to model the impact of climate change on infrastructure. Gong et al. (2018) used random forest regression to predict the roughness of asphalt pavements whereas, Gong et al. (2019) developed a gradient boosting model to enhance fatigue cracking prediction in Mechanistic-Empirical Pavement Design Guide. These are some examples of the few studies that have contributed to this field. Though many research studies are carried out to address various concurrent problems in this field, they are not perfect and have some limitations to their approach. For example, Markovian models are criticized for their disregard of the pavement's maintenance and deterioration history and inability to explicitly represent attributes such as climate or traffic. (Piroynesi and El-Diraby 2018, 2020).

Although many studies have worked on developing different types of deterioration models, they rely on a fixed dataset (generally limited to project level) for the analysis. This is because most of the state agencies perform detailed data collection (such as Ground Penetrating Radar and deflection testing) only at the project level. In the current context, network-level data collection is generally limited to surface distresses and ride quality. As most of the studies use project-level datasets for different types of modeling and analysis, the application of most of these techniques has not been verified at the network level. This, coupled with various other existing problems in the PMS dataset such as missing data (El-Diraby et al. 2017; Piroynesi and El-Diraby 2020) and inaccurate data (Abdelaty et al. 2018), has made it more challenging to implement the latest practices. Thus, implementing the state of the art practices from the body of knowledge is still a practical challenge to the state agencies. Hence, most state agencies still rely on deterioration curves to predict the road condition (El-Diraby et al. 2017; Piroynesi and El-Diraby 2020; Wu 2015).

A common limitation of most of the sources in the literature is that they do not present a clear guideline on how agencies can transition from the current state of the practice (using surface condition data only) to the proposed method that incorporates the structural condition from the Traffic Speed Deflection Devices (TSDD) or how the agencies can make decisions in the absence

of the TSDD data. This is a major challenge that, if properly addressed, will allow the agencies to smoothly transition from the current state of practice to the methods proposed in the literature. At the network level, very few states utilize pavement structural condition information for the decision-making process (Flintsch et al. 2013). Although some sources in the literature have discussed approaches for incorporating the FWD at the network level, approaches for incorporating the TSD are very limited, as it is relatively new. This study presents a practical and scientifically sound method to incorporate the TSD into VDOT's pavement management decision-making process based on VDOT's current method of practice. This will assist VDOT in transitioning from FWD-based pavement management decision-making to the TSD-based modified decision-making approach. The approach presented in this study is based on expert opinions from VDOT on how to use and interpret the TSD pavement structural condition and the pavement deterioration information. Such methods to modify the decision-making process based on VDOT's current practice have not been proposed in previous sources in the literature. This study also reinforces that the structural condition from the TSD affects the rate of pavement deterioration, which has only been studied and supported previously with the FWD data.

3.3 Virginia Department of Transportation (VDOT)

Every year, the Virginia Department of Transportation (VDOT), Maintenance Division publishes the *State of the Pavement* report summarizing the surface condition of the interstate, primary, and secondary VDOT roadway network, which consists of more than 205,996 lane km (i.e. 128,000 lane miles) (VDOT 2018). These condition data are at the core of the department's four primary pavement management activities:

1. Pavement Needs Analysis: Maintenance and rehabilitation needs are determined based on the collected data and used for the development of the biennial maintenance budget and as a guide to maintenance strategy for the districts.
2. Planning for Preventive Maintenance and Resurfacing: Decision trees based on the measured distresses obtained from the collected data are used to determine which sections are more suitable for preventive maintenance and which sections are more suitable for resurfacing.

3. Pavement Performance Reporting: The data play a major role in two legislatively mandated reports about highway asset conditions and management practices.
4. Federal Reporting: The Highway Performance Monitoring System report submitted by VDOT to the Federal Highway Administration is the basis for the apportionment of Virginia's share of federal funds. This report relies on the collected pavement condition data.

Overall, the collection of good quality pavement surface condition data has allowed VDOT to make better investment decisions that maximize pavement life and optimize the use of scarce resources. However, VDOT's Maintenance Division recognizes that the surface condition data are not enough to determine what should be done to a pavement section (VDOT 2018). The structural condition of the pavement is another important aspect needed to make better decisions. The structural condition of the pavement is typically not evaluated at the network level. This is because the Falling Weight Deflectometer (FWD), which is the predominant device used for structural evaluation of pavements, must be stationary while testing. Therefore, the process of collecting structural condition is slow, requires traffic control, and can be relatively dangerous in areas of high traffic volume. However, the emergence of new structural evaluation devices that operate at or near roadway traffic speed has made network-level pavement structural evaluation an achievable objective.

One device capable of continuously measuring pavement deflection velocity is the TSD. The TSD is a continuous deflection measuring device that travels at speeds up to 96 kmph (i.e. 60 mph). A trailer housing Doppler lasers is maintained at a constant temperature of 20 °C to prevent the thermal distortion of the steel beams on which the lasers are mounted. The TSD uses Doppler lasers to measure the instantaneous pavement deflection velocity as the load is applied to the pavement via the rolling trailer tires on the rear axle. The Doppler lasers are mounted at a small angle from the vertical. This allows measurement of the horizontal speed (this is the traveling speed of the vehicle) and the vertical speed (this is the pavement deflection speed). The ratio of the vertical to the horizontal speed gives the deflection slope, which is the slope of the deflection basin. From the deflection slope, the pavement deflection is calculated mathematically by using integration. The Doppler laser measurements are obtained at 110 mm (4 inches), 210 mm (8

inches), 310 mm (12 inches), 610 mm (24 inches), 910 mm (36 inches), and 1,510 mm (60 inches) from the center of the wheel load as shown in Figure 3-1. The axle loading is set at 20,000 lbs, and a strain gauge mounted on the rear axle to measure the bending moment is used to determine the applied load on each side of the axle (because of dynamic effects, the load will not be perfectly distributed to each side). (See Shrestha et al. 2018 for more details on the device and data measurement method).

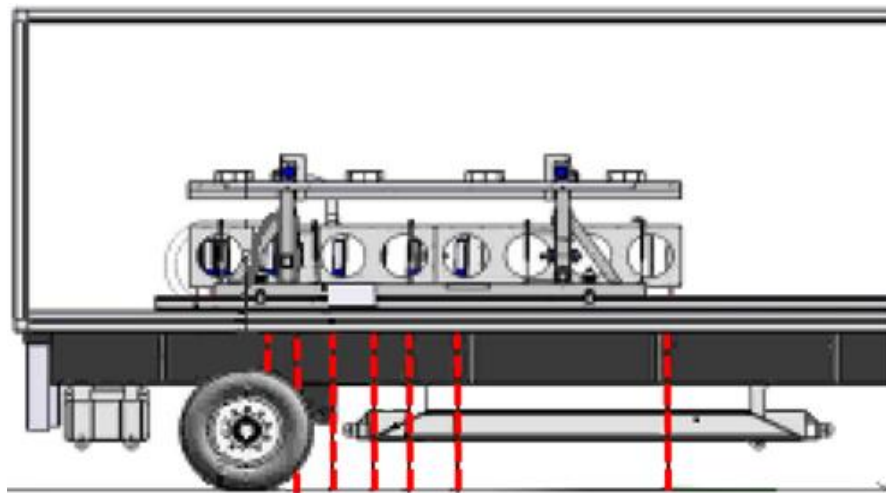


Figure 3-1 Schematic of the TSD

3.4 Objective

The objective of this paper is to develop and justify the use of a treatment selection matrix that takes into account pavement structural condition and augments treatment selection based solely on observed surface condition. To justify the use of the structural condition, pavement deterioration models were developed that show that the pavement deterioration rate is affected by the structural condition.

The collected structural condition data were used to develop the pavement deterioration models. The structural condition data were also used to augment the treatment selection process

within the pavement management system (PMS) decision process. Treatments selected based on the pavement surface condition were modified according to the structural condition which was classified as Strong, Fair, or Weak. The structural condition was divided into 3 categories similar to what VDOT currently uses (i.e., 2 structural condition categories: Strong and Weak). The effects of the modification on the overall percentage of each treatment category on the tested interstate roads were also evaluated. The analysis presented here is limited to flexible pavement sections.

3.5 Methodology

The discussion of the methodology is divided into four sections: (1) Data Collection, (2) Data Processing, (3) Data Analysis and Development of Deterioration Models, and (4) Incorporating Structural Condition Information into the PMS Decision Process. Section 3 and Section 4 involve the analysis, development, and implementation phases of the study. Section 3 justifies why the pavement structural condition is an important factor in pavement deterioration, as represented by the developed deterioration model. The deterioration model shows how the pavement structural condition affects the rate of pavement deterioration. It is not to be used for estimating pavement performance, as various factors such as pavement thickness, traffic, etc., should be considered for this purpose. The deterioration model can be used for efficiently estimating and allocating budgets at a network level for various pavement treatment activities on different routes based on their structural condition. Section 4 presents an implementation tool that integrates the TSD structural condition into the traditional decision-making process.

3.5.1 Data Collection

Data were collected on 6,469 km (4,020 miles) of Virginia's interstate and primary road network (approximately 2,414 km of interstate and 4,055 km of primary roads). The data consist of pavement structural condition, pavement layer thicknesses, and pavement surface condition. For the structural condition data, the Australian Road Research Board (ARRB) collected TSD data at 10-m intervals using the intelligent Pavement Assessment Vehicle (iPAVe). The data were collected in August and September 2017. A verification road was tested at the beginning and end

of the data collection to verify that the device is repeatable. The results of this verification are shown in Figure 3-2.

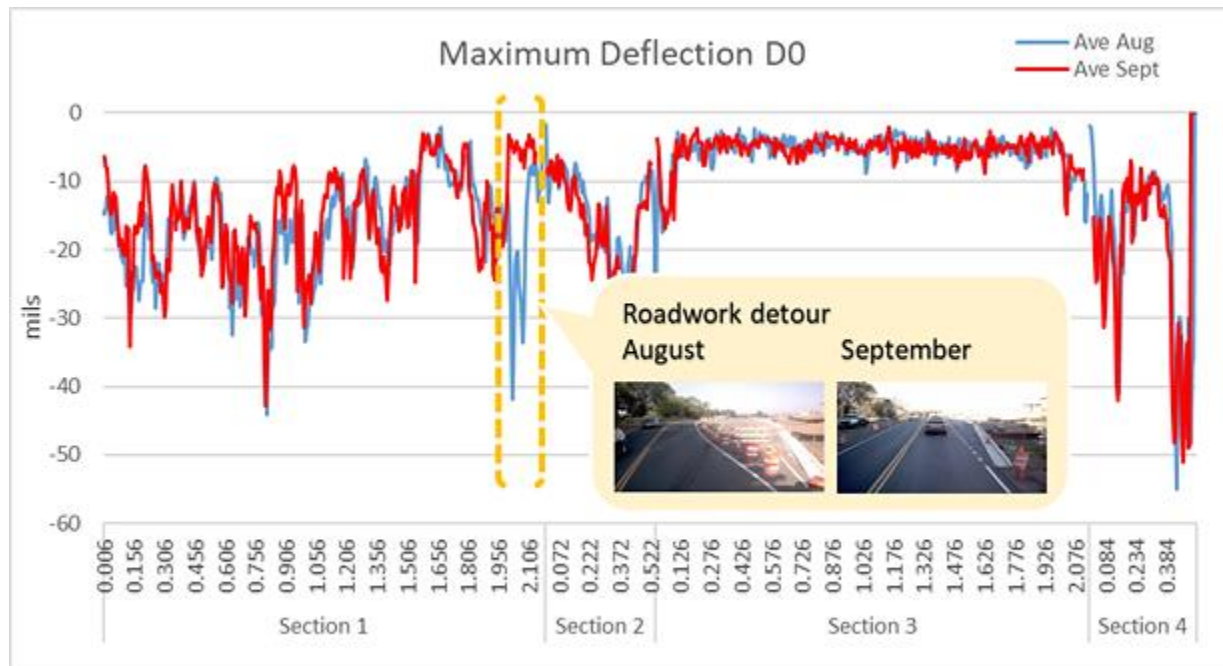


Figure 3-2 Pre- and post-survey testing on validation loop

Pavement layer thicknesses were obtained in two ways. For the interstate roads, the layer thicknesses were obtained from the VDOT PMS. For the primary roads, Ground Penetrating Radar (GPR) data was collected using a GSSI SIR-30 GPR system and a Model 4108 1.0 GHz horn antenna. The GPR data was verified and calibrated using data from 260 cores. The pavement surface condition data were obtained from the iPAVe and the VDOT PMS. The iPAVe collects cracking, rutting, roughness, macrotexture, and geometric data at 10-m intervals. The cracking and rutting data were used to evaluate the correlation between the measured structural condition and the surface distresses. The VDOT PMS data was used to obtain the pavement surface condition at a 0.1-mile resolution to evaluate the effect that pavement structural condition has on pavement deterioration, develop deterioration models, and augment the treatment selection process currently used by VDOT with structural condition data. The PMS data is collected and processed by a contractor that uses continuous digital imaging and automated crack detection technology to

measure the pavement distresses. The reason that PMS data was used instead of the iPAVe data are as follows:

1. To develop pavement deterioration models: The iPAVe data is only available for 2017. To develop deterioration models, multiple years are needed. Therefore data from 2014 to 2018 were obtained from the PMS.
2. To augment the treatment selection process: The surface condition data reported by the iPAVe is not the same as the way it is reported in the VDOT PMS. Therefore, VDOT PMS data was used to develop an approach that could potentially be used by VDOT.

All collected data (TSD, GPR, and surface condition) were synchronized using GPS coordinates.

3.5.2 Data Processing

ARRB provided comprehensive deflection testing data, including raw measurements, calculated deflections, and calculated structural indices such as SCI_{300} (see Flintsch et al. 2013 or Nasimifar et al. 2016 for definitions of structural indices). Additional data processing was needed to perform temperature correction and calculate the effective structural number (SN_{eff}).

$$SCI_{300} = D_0 - D_{300} \quad (3-1)$$

Where, D_0 is the deflection measured at 0 mm from the rear axle of the TSD and D_{300} is the deflection measured at 300 mm in front of the rear axle of the TSD.

Temperature correction was performed for SCI_{300} and the deflection under the applied load, D_0 . The temperature correction for SCI_{300} used the approach developed by Nasimifar et al. (2018), which takes into account the loading specific to a moving device, the viscoelastic effects, and is inspired from the Lukanen et al. (2000) method for FWD measurements. The temperature correction factor is calculated as follows:

$$\lambda = \frac{SCI_{Ref}}{SCI_T} = \frac{10^{-0.05014T_{Ref}+0.019049T_{Ref} \log(h_{AC}) \log(\varphi)}}{10^{-0.05014T+0.019049T \log(h_{AC}) \log(\varphi)}} \quad (3-2)$$

where, λ is the temperature adjustment factor; SCI_{Ref} is the adjusted SCI_{300} at reference temperature; T_{Ref} is the reference temperature in °C; h_{AC} is the asphalt layer thickness in mm, T is

the mid-depth asphalt concrete (AC) layer temperature in °C at time of measurement and φ is the latitude of the location of measurement (within 30 to 50 degrees).

The temperature at the mid-depth of the AC layer was estimated using the BELLS3 equation shown in equation 3-3 (Lukanen et al. 2000).

$$T_d = 0.95 + 0.892IR + \{\log(d) - 1.25\} \\ \{-0.448IR + 0.621(1 - day) + 1.83 \sin(hr_{18} - 15.5) + 0.042IR \sin(hr_{18} - 13.5)\} \quad (3-3)$$

where, T_d is the pavement temperature in °C at depth d ; IR is the pavement surface temperature in °C; $\log$ is the base 10 logarithm; d is the depth in mm at which temperature is to be predicted; (1-day) is the average air temperature in °C the day before testing; $\sin$ is the sine function on an 18-hour clock system, with 2π radians equal to one 18-hour cycle and hr_{18} is the time of the day in a 24-hour clock system but calculated using an 18-hour asphalt concrete (AC) temperature rise- and fall-time cycle.

The average temperature of the day before testing was obtained from the land-based weather station data available from the National Oceanic and Atmospheric database (NOAA). The temperature adjustment factor for D_0 was calculated based on the AASHTO temperature adjustment charts.

We used the method recommended by Nasimifar et al. (2019) to calculate SN_{eff} . The method is based on Rohde's approach to calculate SN_{eff} using FWD measurements (Rohde 1994). However, the Rohde approach cannot be directly used with TSD measurements because it uses the peak deflection measurements obtained from FWD testing. The TSD records instantaneous measurements which, because of the viscoelastic response of the pavement, are different from the peak measurements. In addition, the loading magnitude and configuration of the FWD are different from the TSD. Therefore, Nasimifar et al. (2019) modified the constant coefficients used in Rohde's equation to adjust for this discrepancy. The Rohde equation to calculate SN_{eff} is given below:

$$SN_{eff} = k_1 SIP^{k_2} H_p^{k_3} \quad (3-4)$$

$$SIP = D_0 - D_{1.5Hp} \quad (3-5)$$

where, H_P is the total pavement thickness (mm) and SIP is the structural index of the pavement. The constant coefficients K_1 , K_2 , and K_3 given by Rohde for an asphalt pavement are 0.4728, -0.4810 , and 0.7581 , respectively. Nasimifar et al. (2019) recommended that these coefficients be adjusted to 0.4369 , -0.4768 , and 0.8182 for measurements obtained with the TSD.

3.5.3 Data Analysis and Development of Deterioration Models

The correlations between $SCI300$ and cracking, and $SCI300$ and rutting were calculated for all tested roads to verify that the structural information differs from the surface condition information. It was then postulated that although the structural condition is not highly correlated with the surface condition, the former influences the rate of deterioration of the latter. Therefore, pavement deterioration models for VDOT PMS data as a function of age and structural condition were developed for all tested roads (except for one tested road for which we could not obtain surface condition data from the PMS). The deterioration models were developed using quasi-Poisson regression because it was found that it adequately represents the statistical characteristics of the VDOT PMS data used (see Katicha et al. 2017a, Katicha et al. 2017b, Pantuso et al. 2019).

Pavement Condition Data

The pavement condition data were obtained for the tested roads from the VDOT PMS. VDOT summarizes the condition of pavement with the Critical Condition Index (CCI), which ranges from 0 to 100, with 100 representing no distresses (new surface). Pavement with a CCI below 60 is considered deficient. The approach VDOT follows to calculate the CCI is to first consider the load-related distresses (alligator cracking, wheel-path patching, and rutting) and the non-load-related distresses (longitudinal cracking, transverse cracking, non-wheel-path patching, and bleeding) separately as two separate indices. Both the Load-related Distresses Rating (LDR) and Non-load-related Distresses Rating (NDR) range from 0 to 100 and are calculated as follows:

$$LDR = 100 - D_{AC} - D_{WP} - D_R \quad (3-6)$$

$$NDR = 100 - D_{LC} - D_{TC} - D_B - D_{NP} \quad (3-7)$$

where, D_{AC} is the deduct value for alligator cracking; D_{WP} is the deduct value for wheel-path patching; D_R is the deduct value for rutting; D_{LC} is the deduct value for linear cracking; D_{TC} is the deduct value for transverse (reflective) cracking; D_B is the deduct value for bleeding and D_{NP} is the deduct value for non-wheel-path patching. Deduct values are based on the extent of each distress and more details on the calculation of the deduct values can be found in McGhee (2002). The *CCI* differs from the Pavement Condition Index (*PCI*), a pavement condition index defined by the American Society for Testing and Materials, based on the different types of distress it considers. Details on the calculation of *PCI*, its deduct values and types of distress it considers can be found in ASTM D6433 (ASTM, 2018). The *CCI* is determined from the *LDR* and *NDR* by taking the lesser of the two values. The *CCI*, *LDR*, and *NDR* were obtained from the VDOT PMS for 0.1-mile sections. The date of the last treatment for each pavement section was obtained to determine the appropriate age of the pavement surface.

Regression Model for Pavement Deterioration

The effect of structural condition on the pavement condition index was evaluated by fitting a regression model using data at 0.16-meter (0.1-mile) sections (10-m structural evaluation data were averaged over the 0.16-m length). Deterioration equations were developed for the *LDR*, *NDR*, and *CCI*. The equations are of the following form (shown for *LDR* and *SCI300*)

$$\begin{aligned} LDR &= 100 - \exp(\beta_0 + \beta_1 \log(Age) + \beta_2 \log(Age) \times SCI300) \\ &= 100 - \exp[\beta_0 + \beta_1 (1 + \beta_3 SCI300) \log(Age)] \end{aligned} \quad (3-8)$$

where, *LDR* is the load-related distress; *Age* is the pavement age calculated as the difference between the year at which the *LDR* was observed minus the year of the last applied treatment recorded in the PMS; *SCI300* is the surface curvature index and β_0 , β_1 , β_2 , and β_3 are the regression coefficients with $\beta_3 = \beta_2/\beta_1$. Equation 3-8 illustrates how the model behaves: the pavement deterioration is a function of age with the rate of deterioration depending on the structural condition and determined by $\beta_1(1 + \beta_3 SCI300)$.

The model was fitted using Generalized Linear Model (GLM) approach and has an exponential form. This is similar to what VDOT uses in their PMS. To fit the model, the variable $100 - LDR$

was used to obtain the parameters of the exponential function (see also Katicha et al. 2017a, Katicha et al. 2017b, Pantuso et al. 2019). This variable takes on nonnegative discrete values like Poisson-distributed variables. Furthermore, it was found (shown in the results section) that the variance of $100 - LDR$ is proportional to the mean of $100 - LDR$ similar to a quasi-Poisson variable. Therefore, quasi-Poisson regression, which is a standard regression procedure, can be used to obtain the model parameters. Note that while the variable LDR is also discrete, its variance is not proportional to the mean, a fact that would make it harder to fit the model as no standard fitting procedure is available in this case. Some other parameters were also considered for the deterioration model but were not included in the final model. Traffic was one such parameter that was initially considered in the regression model but since it was found to not be a significant parameter in the regression, it was not considered in the final deterioration model. Other features related to the pavement maintenance history were not included because of their lack of accuracy and details at the network level. However, VDOT is getting better at recording these various treatments applied to the pavement sections and keeping track of them. Therefore, including the maintenance history in the future could be possible.

3.5.4 Incorporating Structural Condition Information into the PMS Decision Process

VDOT uses a set of pavement management decision matrices with distresses as inputs and treatment activities as outputs. A filter based on the *CCI* is then used to obtain the final recommended treatment based on the surface condition. In 2008, this two-phase approach was modified to include structural condition and truck traffic volumes, and the enhanced decision tree was integrated into the process when adequate structural information from FWD testing was available (as shown in Figure 3-3). One of the main features of the approach is that the addition of the pavement structural information did not alter the core of the decision process already in place but provided an additional step that can be used when pavement structural condition is available. If structural information becomes unavailable, the decision process can revert to the core process already in place. VDOT currently uses the following five treatment categories (listed in order from least to greatest severity): DN, PM, CM, RM, and RC. At the preliminary treatment stage, one of these five categories is selected based on the condition index and the decision matrices.

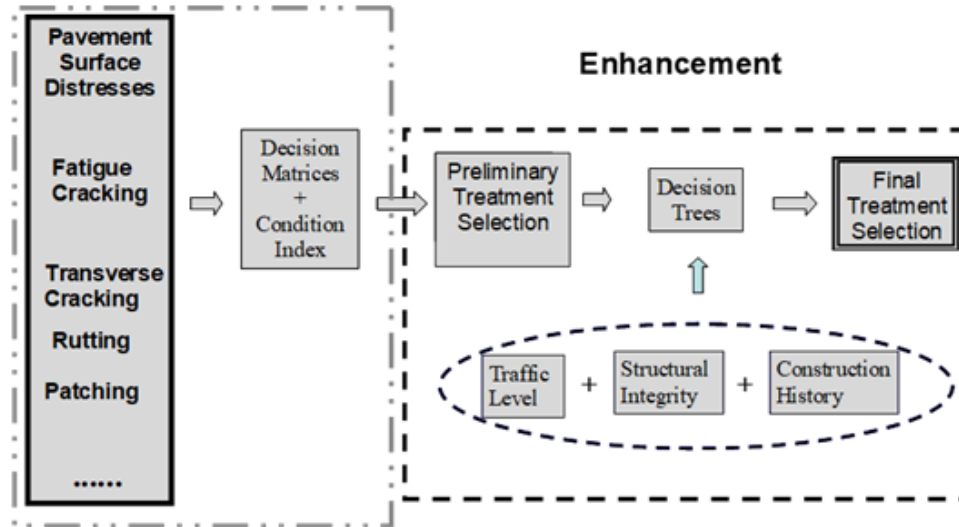


Figure 3-3 Enhanced decision process used by VDOT when adequate structural information from FWD is available

In the enhanced decision process, based on the structural condition (and traffic level in terms of annual average daily truck traffic and maintenance history), the selected preliminary treatment can be either retained or modified to more severe or less severe treatment. In this paper, a similar approach was implemented with the TSD data using *SCI300* but without considering the traffic level and maintenance history. The results section shows the implications of the proposed approach on the selected treatment at the network level. Chowdhury (2008) explains further details on the VDOT's two-phase approach.

3.6 Results

3.6.1 Correlation between Structural and Surface Condition

The correlation between the structural condition and the functional condition is performed at two levels. At the first level, the correlations between *SCI300* and cracking, *SCI300* and rutting, and cracking and rutting are calculated for each tested road to see whether there is a strong correlation between the structural condition and the surface condition. At the second level, the average *SCI300*, average cracking, and average rutting are calculated for each tested road. From these average values, the correlation is then calculated for all the roads.

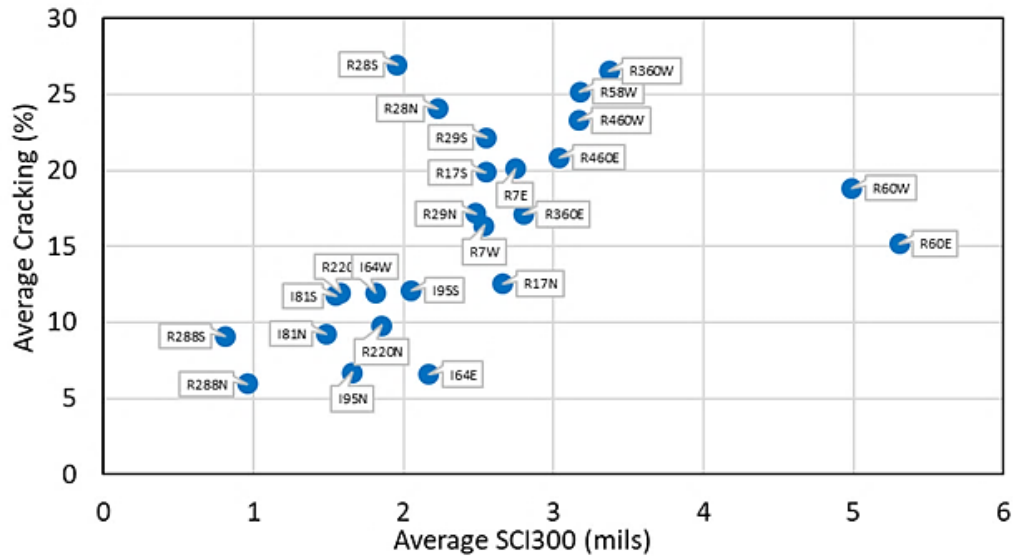


Figure 3-4 Cracking vs. SCI300 for all tested roads

Figure 3-4 shows a scatter plot of the average cracking versus the average *SCI300* for all tested roads. The overall trend is clear, and the correlation between these two values is 0.49. Route 60 seems to be an outlier, having large *SCI300* values and average cracking values. Route 28 also does not fit well into the overall trend, although not as clearly as Route 60. The correlation between cracking and *SCI300* with Route 60 removed increases to 0.86 (0.89 if Route 28 is also removed). Note that the high correlation found when the data were averaged and all roads are considered is in contrast with the relatively low correlation within a road. The results obtained with the averages were potentially much more affected by other confounding factors than the results obtained within individual roads. For example, roads are designed according to their functional classification system. Interstate roads generally carry more traffic and are designed to be structurally stronger compared to primary and secondary roads. Furthermore, the surfaces of these roads are likely to be kept in better condition than other roads. Therefore, the road classification is a confounder that will result in increasing the correlation between the structural condition and observed surface condition. This confounder has a minimal effect when looking at the data within a specific road (as the importance is practically the same throughout).

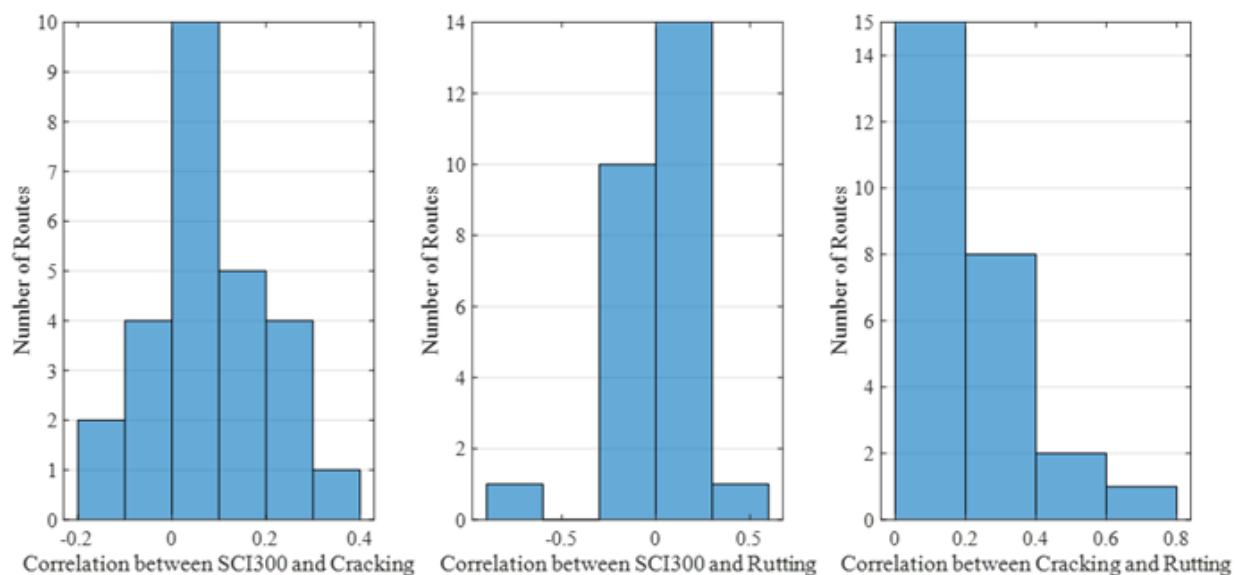


Figure 3-5 Pairwise correlation between SCI300 and cracking

Figure 3-5 shows the pairwise correlations for each road. In general, within a tested road, the correlation between *SCI300* and cracking is very small. The average correlation for all roads is 0.064, which is almost zero. In a few cases, there is a moderate correlation. For the primary roads, most of the correlations between *SCI300* and cracking are positive (16 out of 19) although very small. For the interstate roads, the correlation is even weaker, with three positive and three negative values. Theoretically, a good correlation between the pavement surface and the structural condition is expected, as the rate of pavement deterioration depends on the structural condition. But in practice, when pavements deteriorate agencies intervene and fix them. Most of the treatments are carried out fix the pavement surface but do not contribute to the structural strength. This artificial intervention removes most of the correlation that is supposed to exist between the pavement surface and structural condition. This suggests that, for the most part, the amount of cracking observed on a road is not a good indicator of the structural condition of that road.

The correlation between *SCI300* and rutting is even weaker with 13 positive and 12 negative values. The correlation between cracking and rutting is higher at an average of 0.20, with all calculated correlations being positive. This shows a definitive, although weak link, between cracking and rutting. This link is most probably because both rutting and cracking are positively correlated to the age of the pavement surface (older surfaces will show higher rutting and

cracking). The correlations between *SCI300* and rutting and cracking and rutting are -0.042 and 0.002 , respectively, showing no relationship between rutting and the other two variables (the measured rutting, in general, is very low).

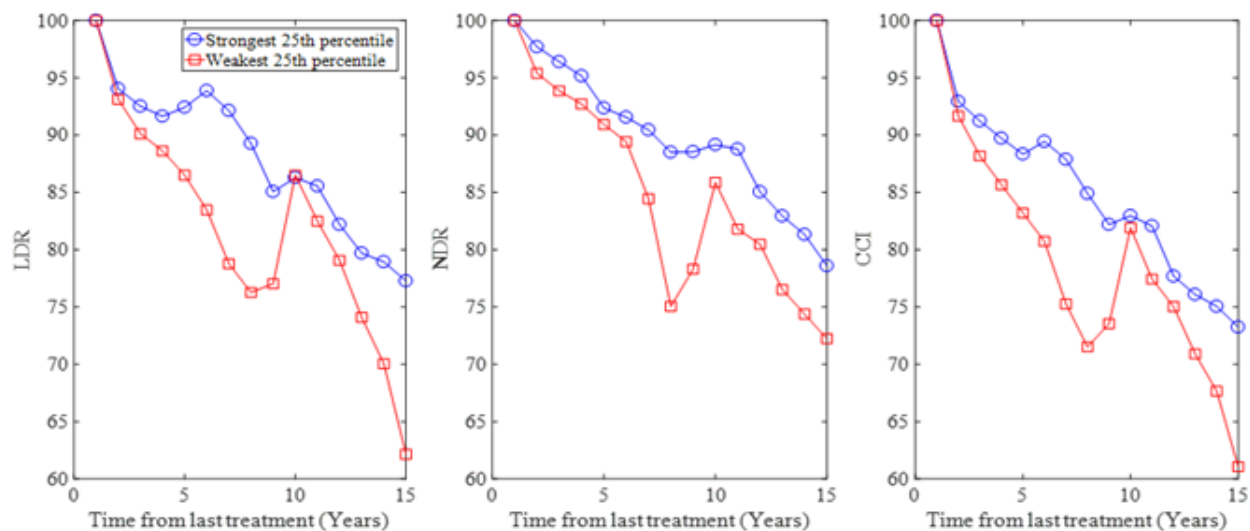


Figure 3-6 Average condition for tested interstate roads (*LDR*, *NDR*, and *CCI*) of structurally strong (top 25th percentile) and structurally weak sections (bottom 25th percentile) as a function of time from last treatment

Figure 3-6 shows the average condition index (*CCI*, *LDR*, and *NDR*) as a function of the time since the last treatment for the 25% structurally strongest sections and the 25% structurally weakest sections on interstate roads. The figure shows that, in general, the structurally weaker sections deteriorate faster than the structurally stronger sections. At years 9 and 10, the condition of the weak pavement sections seems to significantly improve almost to a level equal to that of the strong sections. What is causing this jump is most probably the treatment actions that are applied more frequently as the pavement surface ages. On average, the surface of interstate sections is replaced after 7 or 8 years of service. However, sections that are performing well are not resurfaced and remain for longer periods. Therefore, the average condition at greater ages for the structurally weak sections is biased because only the sections that have performed exceptionally well are represented in that group. Figure 3-6 suggests that this biasing effect becomes very significant at about 9 or 10 years and, therefore, only the data up to year 8 were used to develop the pavement deterioration models.

3.6.2 Pavement Deterioration Models with Structural Condition-Dependent Rates

Quasi-Poisson regression (a form of a generalized linear model) was used to develop the pavement deterioration models in this study. Quasi-Poisson modeling is appropriate for discrete data that have a linear relationship between the mean and the variance. A linear relation gives the best representation between the mean and the variance, showing that the quasi-Poisson model is the most appropriate for the VDOT condition data (this was also observed for the other roads). The condition data are discrete and the relationship between the mean and the variance is shown in Figure 3-7. The data in the figure were obtained by calculating the mean deterioration (e.g., $100 - LDR$) and the standard deviation of the deterioration for each year on the northbound direction of I-81.

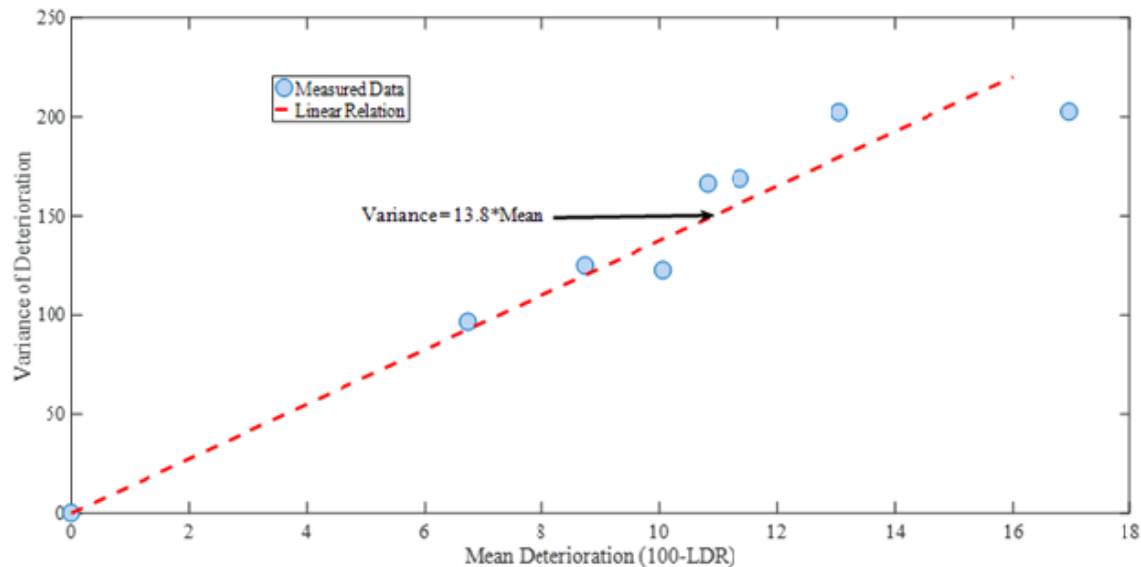
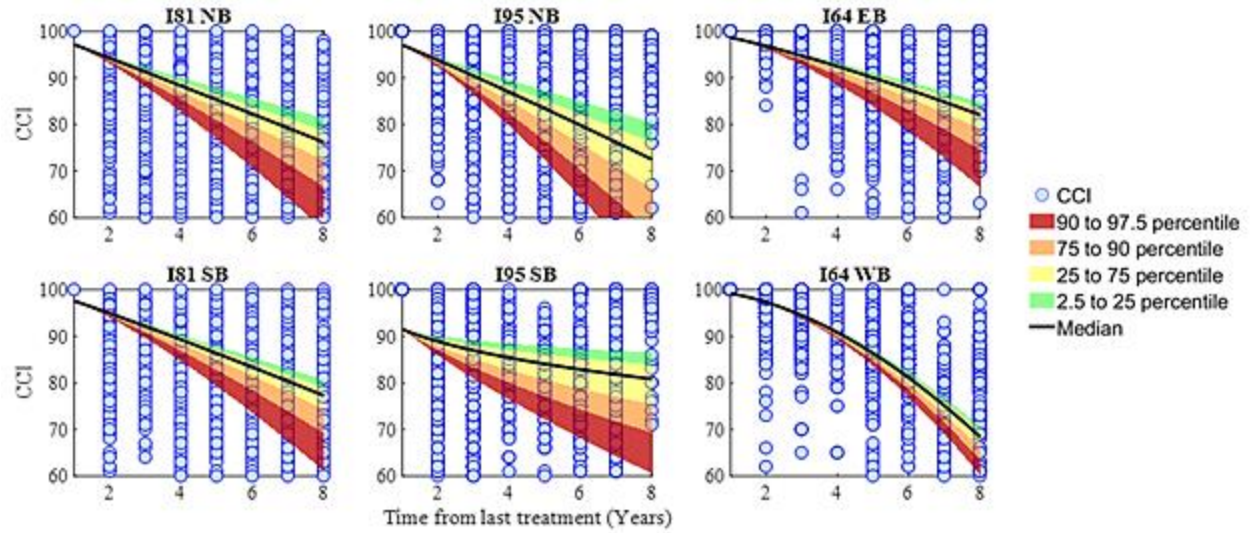


Figure 3-7 Variance of $100 - LDR$ as a function of mean deterioration defined as $100 - LDR$.

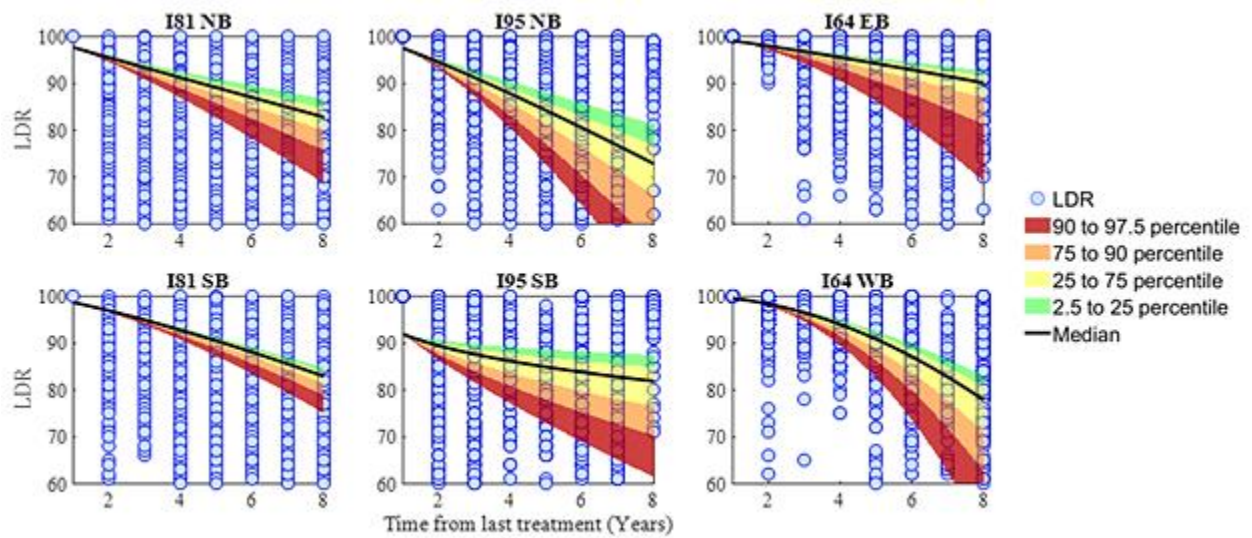
The dependence of the variance on the mean is best approximated by a linear relationship.

Figure 3-8 shows a scatter plot of existing pavement condition (in terms of *CCI*, *LDR*, and *NDR*) of sections used in the deterioration model up to 8 years in age. Blue markers in the figure represent the existing *CCI* condition. The shaded regions (in green, yellow, orange, and red as shown in the legends) represent different structural conditions based on percentile values for each route. The figure shows how the structural condition affects the rate of deterioration, with green representing very strong sections and red representing very weak sections. Figure 3-8a shows the deterioration

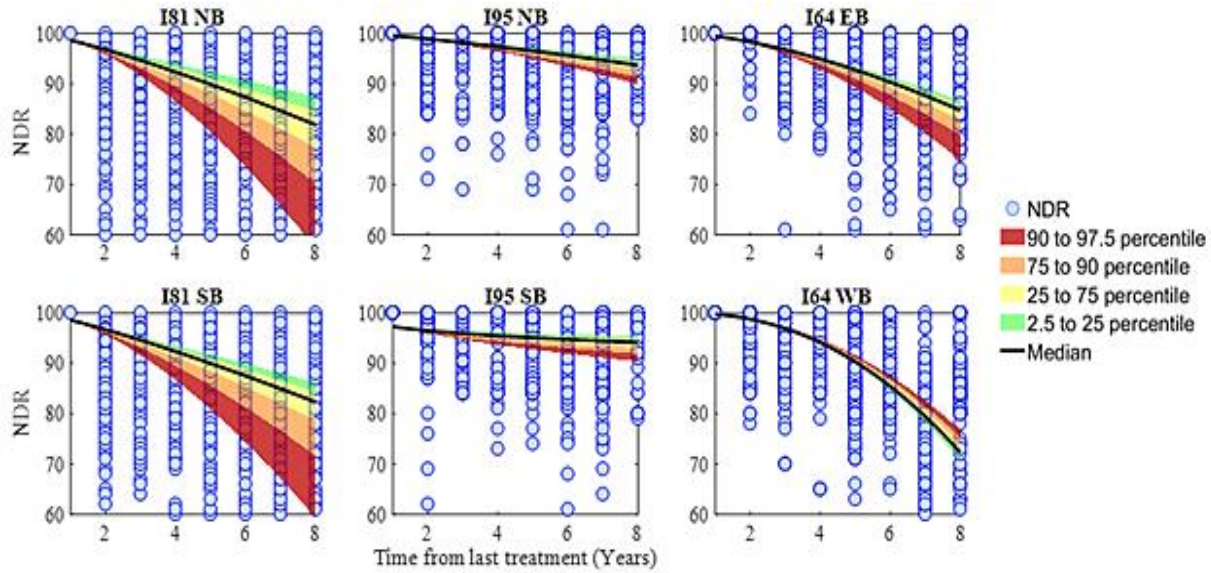
model for *CCI*, with *SCI300* used as the parameter that represents the structural condition. The figure shows *CCI* values between 60 and 100 because VDOT considers pavement below 60 to be deficient. For I-81 Northbound, I-81 Southbound, I-95 Northbound, I-95 Southbound, and I-64 Eastbound, the structural condition has a significant effect on how fast the *CCI* deteriorates. For I-64 Westbound, the structural condition has a very small effect on how fast the *CCI* deteriorates. Figure 3-8b and Figure 3-8c represent deterioration models for *LDR* and *NDR* respectively. For *LDR*, we can observe that the structural condition affects the rate of deterioration on all roads. The most interesting case is I-64 Westbound, which shows that the structural condition has a large effect on the *LDR* as opposed to the *CCI* discussed earlier. For the case of *NDR*, the structural condition has practically no effect on the rate of deterioration for I-95 and I-64 (both directions). Comparing the *LDR* to the *NDR* for these two roads, we see that for I-95, the *NDR* is much higher than the *LDR*. For I-64, the two indices are much closer, with the *NDR* generally smaller. The *CCI* is the minimum of *LDR* and *NDR*, this explains why the structural condition did not have a significant effect on the rate of deterioration for I-64 Westbound; the *CCI* on I-64 Westbound is mostly the *NDR* value since the *NDR* is generally smaller than the *LDR*. For I-95, the *CCI* and *LDR* are practically the same since the *NDR* is usually much higher. For I-81 we can observe that structural condition seems to have a bigger effect on the *NDR* than on the *LDR*. This seems counterintuitive since the *NDR* is not supposed to be caused by loading and therefore should not be significantly affected by the structural condition. However, *LDR* and *NDR* classifications are based on the predominant mechanism of the specific distress, and in case of heavy traffic conditions, even *NDR* could be to a certain extent affected by loading. I-81 carries the highest load in terms of truck tonnage in Virginia, which could also contribute to the fact that most observed distresses will be affected by loading. (I-95 comes second but also in general has more lanes than I-81.)



(a) Deterioration models for CCI for pavement in different structural conditions



(b) Deterioration models for LDR for pavement in different structural conditions



(c) Deterioration models for NDR for pavement in different structural conditions.

Figure 3-8 Deterioration models for pavements in Virginia

The parameters of the deterioration model for interstate roads using *SCI300* as the structural condition parameter are shown in Table 3-1. In general, the results show that sections with higher *SCI300* (i.e., weaker sections) have a higher rate of deterioration than those with a lower *SCI300* (i.e., stronger). The authors also developed 72 deterioration models in the study, i.e., 24 models (6 for Interstates and 18 for Primary roads) for each structural condition index *SCI300*, SN_{eff} , and D_0 . There are a few cases (13 out of the 72 models; 3 out of the 24 models for *LDR*) where a higher *SCI300* result in a lower rate of deterioration; however, in most of these cases (7 out of the 13 cases; 3 out of the 3 cases for *LDR*) the parameter is not statistically significant at the 0.05 level. However, it was found that *SCI300* is the best parameter to use based on the number of models that show a consistent increase in deterioration rate for structurally weaker sections.

Table 3-1 Deterioration model parameters estimate and significance for interstate roads

Route	Parameters	CCI		LDR		NDR	
		Estimate	p-value	Estimate	p-value	Estimate	p-value
81 NB	β_1	0.875	< 0.001	0.818	< 0.001	0.280	< 0.001
	β_2	7.145	< 0.001	7.362	< 0.001	10.489	< 0.001
81 SB	β_1	0.987	< 0.001	1.175	< 0.001	1.060	< 0.001
	β_2	4.550	< 0.001	3.227	< 0.001	7.064	< 0.001
95 NB	β_1	0.873	< 0.001	0.937	< 0.001	1.115	< 0.001
	β_2	5.271	< 0.001	5.674	< 0.001	3.749	< 0.001
95 SB	β_1	0.134	< 0.001	0.182	0.0011	0.227	0.3498
	β_2	5.529	< 0.001	5.794	< 0.001	3.845	0.0063
64 EB	β_1	1.139	< 0.001	0.948	< 0.001	1.492	< 0.001
	β_2	3.860	< 0.001	7.042	< 0.001	3.124	< 0.001
64 WB	β_1	1.729	< 0.001	1.688	< 0.001	2.272	< 0.001
	β_2	1.886	0.0017	6.734	< 0.001	-1.426	0.0832

3.6.3 Incorporating Structural Condition Information into the PMS Decision Process

The significance of the structural condition on the rate of deterioration of the surface condition suggests that the structural condition should also be considered during treatment selection. For example, if the treatment category based on surface condition is RC and the structural condition is very strong (similar to a new pavement), it would not make sense to keep RC as the selected treatment as it would not be cost-effective. Therefore, a treatment selection matrix that incorporates the pavement structural condition and modifies the initial treatment based on the surface condition was developed (presented in Table 3-2). The structural condition was classified into three categories: Strong, Fair, and Weak based on the 25th and 75th percentiles of *SCI300* values for each route, respectively. Thus, every route was classified into three different categories based on its structural condition. For pavements in Fair structural condition, no modification to the treatment was recommended. For pavements in Good structural condition, CM was modified to PM, RM to CM, and RC to RM (the recommended treatment was reduced by one severity category). DN and PM are not modified. The reason for not modifying PM to DN is because preventive maintenance treatments extend the life of the surface, which helps maintain the structural integrity of the pavement even if the distresses are not load-related. For example, crack

sealing is generally considered to fall under PM. Even if the cracks are not load-related, they allow moisture to infiltrate, which in the long term will contribute to loss of strength. Therefore, crack sealing should still be performed. For pavements in Weak structural condition, CM is modified to RM, RM to RC (recommended treatments of CM and RM were increased by one severity category), and PM to DN (reduced by one treatment category); DN and RC are not modified. The modifications of CM to RM and RM to RC are expected because the pavement is weak and a more severe treatment should be anticipated. The modification of PM to DN seems counterintuitive; however, a CM treatment is likely to be ineffective for these sections and it would be better to let these sections further deteriorate and apply a more severe treatment later.

Table 3-2 Modified treatment category based on structural condition

Initial Treatment Category	Modified Treatment Category with Structural Condition Category		
	Strong	Fair	Weak
DN	DN	DN	DN
PM	PM	PM	DN
CM	PM	CM	RM
RM	CM	RM	RC
RC	RM	RC	RC

The effect of the treatment modification can be easily estimated for any network and any initial treatment configuration. For example, in a network where the initial treatment categories are uniformly distributed for all structural conditions (i.e., 20% of each treatment category for structurally Weak, Fair, and Strong sections), the modification will result in 25% DN (an increase of 5%), 20% PM (no change), 15% CM (decrease of 5%), 20% RM (no change), and 20% RC (no change). Note that if we had recommended that PM be modified to CM for the structurally weak sections, then the percentages would have stayed the same (20%) for all treatment categories (although distributed differently among the sections). The increase of 5% in the DN category would seem to indicate some savings. This is not the case. This 5% set represents sections that need a more severe treatment such as RM but the surface condition is still good and could serve a couple of years before deteriorating to the point RM would be appropriate. For the actual interstate network tested, the percentages of the initial treatments and the modified treatments are shown in

Table 3-3. The changes are much different from the case of uniform initial treatments. On average, there is a reduction of 15% in the CM category, which is mostly evenly redistributed, as well as an increase of about 7% for the PM and RM categories. The percentages of the DN and RC categories are not significantly affected.

Table 3-3 Percentage of initial treatments categories obtained from VDOT PMS and the percentage of modified treatments after taking into consideration the structural condition

PMS Decision	I-81 NB		I-81 SB		I-95 NB		I-95 SB		I-64 EB		I-64 WB		Average	
	I	II	I	II	I	II	I	II	I	II	I	II	I	II
DN	24.4	27.2	28.0	29.4	45.7	47.1	34.1	35.8	11.5	14.3	37.8	40.3	30.3	32.4
PM	13.3	22.2	10.1	19.5	14.8	21.8	10.9	16.2	22.9	31.2	14.4	17.2	14.4	21.4
CM	47.6	29.5	52.2	34.4	32.4	19.6	47.4	29.9	51.9	34.0	32.0	22.2	43.9	28.3
RM	11.7	17.5	8.6	13.4	1.1	6.8	1.0	12.3	6.2	13.4	6.2	12.8	5.8	12.7
RC	3.0	3.7	1.0	3.3	5.9	4.7	6.5	5.7	7.5	7.2	9.5	7.4	5.6	5.3
Note: I represents treatment categories based on surface condition, whereas II represents modified treatment categories														

3.7 Conclusions

This paper presented an approach to incorporate the TSD-measured pavement structural condition into the PMS decision-making process. The TSD measurements were temperature corrected. Correlation analysis found little evidence that the surface condition provides a good indication of the structural condition. Analysis of the surface condition as a function of time showed that structurally weak sections tend to deteriorate at a faster rate than structurally strong sections. Three structural condition parameters, SCI_{300} , D_0 , and SN_{eff} were investigated, and it was found that SCI_{300} is the best parameter to use in a deterioration model. In general, except for a small number of the analyzed roads and pavement condition indices, the deterioration models behaved as expected, with the deterioration rate increasing for structurally weaker sections. This suggests that it would be beneficial to incorporate the structural condition into the decision process for treatment selection. This was done using three structural condition categories: Weak, Fair, and Strong. For the Fair category, the treatments are kept the same as recommended based on the surface condition. For the structurally Strong sections, CM, RM, and RC were modified to a less severe treatment

category. For the structurally Weak sections, CM and RM were modified to a more severe treatment category, while PM was modified to DN. For the interstate network tested, the approach resulted in a reduction of the percentage of sections with a recommended CM treatment from 44% to 28% and an increase of the percentage of PM from 14% to 21% and RM from 6% to 13%. Percentages for DN and RC remained practically the same.

3.8 Recommendations

As this study was a part of the implementation research project and had to be inclined with the current VDOT practices, pavement deterioration was modeled using a deterministic approach as implemented by VDOT in their pavement management system. Recommendations that could be considered for future research are as follows:

1. Additional factors such as pavement traffic and construction history that were not included in the deterioration model can be included for enhancing the modified treatment decisions using decision trees.
2. It would be interesting to explore various other alternative research methods such as: using probabilistic deterioration models (Markov's process), data clustering, etc. for application in the existing network-level decision-making practices.

3.9 Data Availability Statement

Some or all data, models, or codes used during the study were provided by a third party. Direct request for these materials may be made to the provider as indicated in the Acknowledgments.

3.10 Acknowledgments

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Chapter 4. Implementing Traffic Speed Deflection Measurements for Network-level Pavement Management ²

4.1 Abstract

This paper investigates the possibility of using pavement deflection measurements obtained from a traffic speed deflectometer (TSD) for network-level structural evaluation on bituminous pavements in Virginia. Virginia currently uses falling weight deflectometer (FWD) measurements for network-level pavement management decision-making for their interstate roads. Two factors were deemed important to determine if FWD data can be replaced with TSD data: (1) the distribution of effective structural number (S_{Neff}) calculated from TSD measurements compared with S_{Neff} calculated from FWD measurements, and (2) the consistency of TSD with FWD in identifying the same weak sections. The results show that the distributions of the S_{Neff} from the two devices were similar, and the calculated consistency in identifying weak sections between the TSD S_{Neff} and the FWD S_{Neff} was higher than the consistency between the S_{Neff} from two repeated sets of FWD measurements. This suggests that the structural condition obtained from the TSD can be used to replace the structural condition obtained from the FWD currently used in the VDOT pavement management system. The study also assessed whether the choice of the structural index calculated with TSD measurements could potentially have a significant impact on the network-level decision-making process. Very little practical difference was observed between using the S_{Neff} or SCI300 to identify structurally weak sections from TSD measurements. Similar analysis can be performed by other states to incorporate TSD structural condition into their pavement management decision-making process.

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4.2 Introduction

Pavement structural condition plays an important role in maintaining and preserving the overall pavement condition (Katicha et al. 2016, Shrestha et al. 2018). Various technologies have been developed to assess and quantify the pavement structural condition, with the falling weight deflectometer (FWD) being the most widespread. The FWD is a stationary device that applies an impulse load to the pavement surface and measures the pavement deflection response with sensors (geophones or force balance seismometers) placed at varying distances from the applied load. The measured deflections can be used to quantify the pavement's structural health, identify strong/weak sections, calculate layer properties and required overlay thickness, and estimate the remaining service life of the measured sections (Zaghloul et al. 1998, Livneh and Goldberg 2001, Zhang et al. 2003, Werkmeister and Alabaster 2007, Gedafa et al. 2010). Although pavement structural condition is an important pavement condition parameter, its measurement has mostly been limited to project-level applications (Flintsch et al. 2013) because of the limitations of the FWD for network-level data collection.

One of the biggest limitations of the FWD is that it must stop to measure the pavement deflection. Stopping along the road for testing (1) limits the number of measurements that can be collected per day, (2) requires the use of traffic control, and (3) poses a safety hazard to the personnel conducting the testing because they have to operate adjacent to active traffic lanes. Thus, using the FWD for network-level applications is both time- and resource-intensive, as well as potentially dangerous. More recently developed continuous deflection measuring devices, such as the traffic speed deflectometer (TSD), overcome these obstacles, making them good candidates for network-level testing (Flintsch et al. 2013, Rada et al. 2016).

The Virginia Department of Transportation (VDOT) currently uses the pavement structural condition, measured with the FWD between 2006 and 2008, for network-level pavement management treatment category selection on interstate roads. The selection of treatment category is performed in two phases, with the first phase based solely on the observed surface distresses and the second phase on modifying the selected treatment in the first phase based on the FWD structural information. VDOT initially planned to update the FWD data every 5 years, but because of the difficulties in collecting network-level FWD data, this update has not been performed, and

the FWD data collected between 2006 and 2008 is still being used in 2021. The more-than-12-year time period since the data was last collected has raised some concerns regarding its validity and use in network-level treatment category selection. Therefore, in 2017 VDOT decided to explore network-level structural evaluation with the TSD as an alternative to FWD data collection. The TSD can collect data more than 200 miles in a single day, at traffic speed, and without the need for traffic control, all of which are limitations of the FWD. For the investigation, TSD data was collected during a 5-week period in August and September 2017 on 1,500 lane miles of interstate roads and 2,050 lane miles of primary roads.

4.3 Objective

The objective of this paper is to compare the structural information obtained from TSD and FWD testing to support VDOT management in deciding whether to move forward with using the TSD instead of the FWD in the current VDOT network-level pavement management treatment selection procedure.

VDOT currently uses the effective structural number (S_{Neff}), calculated from FWD measurements, as the structural parameter in the treatment selection process. Therefore, one of the main tasks in this paper is to determine the relationship between the S_{Neff} obtained from the FWD and the S_{Neff} obtained from the TSD. This will ensure that the devices produce comparable results and that the sets of identified weak sections by each device have a large proportion of sections in common. Specifically, the study assessed (1) the impact of substituting FWD with TSD measurements for network-level pavement management, and (2) whether the choice between surface curvature index (SCI300) and S_{Neff} as the structural index calculated with TSD measurements could potentially have a significant impact on the network-level decision-making process.

4.4 Background

It is widely accepted in the pavement engineering community that the pavement structural condition is an important condition indicator of pavement network health that can help in selecting more cost-effective M&R treatments (Zaghrou et al. 1998, Bryce et al. 2013, Maser et al. 2017, Shrestha et al. 2018, Zhang et al. 2016, Elseifi et al. 2018, Thyagarajan et al. 2019). The FWD has

historically been the device of choice for the structural evaluation of pavements. However, the FWD is not well-suited for network-level testing, which has led to the development of traffic speed deflection devices (TSDDs) such as the rolling wheel deflectometer (RWD), the Rapid Pavement Tester (RAPTOR), and the TSD. The TSD is currently the only TSDD that is commercially available and operating in the United States.

4.4.1 Evaluation of TSDDs

TSDDs were developed in the late 1990s and early 2000s. By the late 2000s and early 2010s, the technology had matured enough to the point where it became clear that TSDDs were ready to be implemented and provide valuable network-level pavement structural information to highway agencies (Rada and Nazarian 2011, Flintsch et al. 2013). More TSDD-related research efforts (mostly focusing on the TSD and to a lesser extent the RWD) covering different evaluation and application areas followed. These included a detailed evaluation on instrumented pavements (Rada et al. 2016), demonstration of device capabilities and evaluation under production testing (Katicha et al. 2017), implementation efforts (Elseifi and Elbagalati 2017, Elseifi and Zihan 2018), and comparisons with the FWD (Katicha et al. 2014, Muller 2015, Elseifi and Zihan 2018). These studies (as well as many others) have further confirmed that TSDDs are ready for implementation.

Among all the TSDD evaluation efforts, comparisons with the FWD are of great interest to practitioners familiar with FWD testing and data interpretation. In general, TSDDs and FWD show similar trends in the variation of structural response along a tested road; weak or strong sections identified by one device are very often also identified by the other. Therefore, good empirical models can be developed between TSDD measurements and FWD measurements for network-level applications (e.g., Elbagalati et al. 2016). However, for more detailed project-level applications, TSDDs and FWD measurements are not equivalent (Flintsch et al. 2013, Nasimifar et al. 2017, Katicha et al. 2014, 2017, Shrestha et al. 2019). TSDDs record the pavement response under maximum load at a fixed time, whereas FWD measurements are recorded for the maximum deflection, which can occur at different times for the different sensors (Shrestha et al. 2018). Therefore, detailed project-level analysis methods that have been developed with FWD data will in general not be as accurate when used with TSDD data.

4.4.2 Structural Parameters and Deflection Indices

Structural parameters that can be calculated from FWD measurements can also be calculated from TSDD measurements. These include (1) deflection bowl indices (i.e., indices based solely on deflections), (2) S_{Neff}, and (3) calculation of layer moduli. Deflection indices are generally used in network-level applications, while calculation of layer moduli is generally used for project-level applications. The S_{Neff} can be used for both network-level and project-level applications.

Deflection bowl indices originated from structural evaluation with the FWD and date as far back as 1982 (Hoffman and Thompson 1982). These indices have naturally been adopted for TSDDs, although, as previously noted, the measurement principles of TSDDs and FWD are different. Nevertheless, both devices measure the structural capacity and produce indices that at least qualitatively agree in terms of the strong, fair, and weak sections identified. This general agreement makes these indices well-suited for network-level applications. Rada et al. (2016) evaluated 77 indices that can be calculated from TSDD measurements to find which ones best correlate with the tensile strain at the bottom of the asphalt layer and the compressive strain at the top of the subgrade. Because of a large number of indices investigated, many are highly correlated (practically equivalent). In general, indices calculated from deflections closer to the applied load had the highest correlation with the tensile strain at the bottom of the asphalt layer. Indices calculated from deflections farther from the applied load had the highest correlation with the compressive strain on top of the subgrade. The surface curvature index (SCI300), one of the indices that had a high correlation with the tensile strain at the bottom, is used in this paper and is presented in the “Methods” section.

The S_{Neff} is a structural parameter linked to the structural number (SN) used in the AASHTO 1993 design method. The SN is determined by the pavement layer thickness and layer coefficient. It can also be used in pavement design to determine the pavement layer thicknesses based on mainly truck traffic and design period (along with other parameters). The S_{Neff} is calculated from the measured deflections and the pavement thickness. Two main approaches have been proposed to calculate S_{Neff} from TSD measurements. The first is based on the Rohde equation (Rohde 1994) originally developed using FWD data. This method is used in this paper and presented in the “Methodology” section. The Rohde equation approach has been used by Flintsch et al. (2013),

Katicha et al. (2014, 2017), and Nasimifar et al. (2019). The first three references used the original equation developed for measured FWD, while Nasimifar et al. (2019) used the same equation but with constants calibrated to loading conditions based on the TSD. The second method is based on the AASHTO 1993 method for overlay design but because it is not used in this paper, it will not be presented here.

4.4.3 Current VDOT Decision-making Approach Using FWD Measurements

VDOT is one of the few state highway agencies that use network-level structural condition as part of the pavement management treatment selection process. Even then, the use of structural condition is limited only to interstate roads. For all roads managed by VDOT, the selected treatments are based on (1) decision matrices that take into account the measured surface distresses, and (2) an additional filter based on the Critical Condition Index (CCI). These two steps are used for primary and secondary roads. For interstate roads, a third step accounting for the structural condition, in terms of S_{Neff} for flexible pavement sections and traffic volume, was added in 2008 (Figure 4-1**Error! Reference source not found.**). The addition of this third step for interstate roads did not change the core of the current decision-making process (the first two steps) but rather provided an additional criterion that can be used when the structural condition information is available. In the absence of the pavement structural condition information, the state agencies can revert back to the core decision-making process.

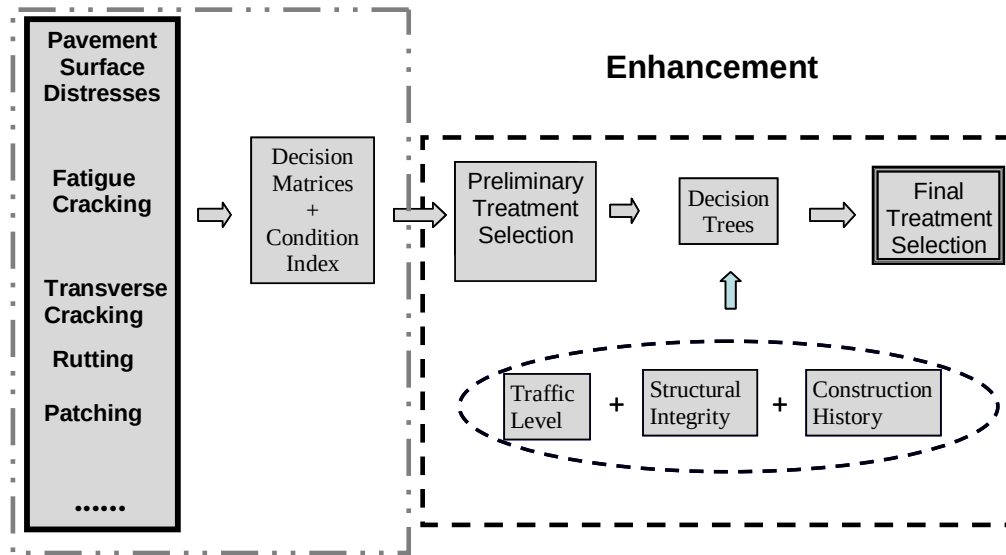


Figure 4-1 Enhanced decision process framework used by VDOT when adequate structural information from FWD is available (Chowdhury, 2016).

For the network-level treatment category selection, VDOT currently uses five treatment categories (listed here in order from lesser to heavier treatments): DN, PM, CM, RM, and RC. After running the first two steps (decision matrices and CCI filter), one of these five maintenance categories is selected. The third step incorporates structural condition, traffic level, and construction history, and either retains the selected treatment category or modifies it to a heavier or a lighter treatment. To characterize the structural condition of flexible pavements, VDOT uses a S_{Neff} threshold of 6 (based on the 30th percentile of S_{Neff} cumulative distribution) calculated from FWD deflections as a threshold to identify structurally weak sections on interstate routes (Chowdhury, 2016).

4.5 Methodology

This section is divided into four subsections: (1) data collection, (2) data processing and analysis, (3) comparison between FWD and TSD data on interstate roads, and (4) the impact of the structural parameter selected on the identification of structurally weak sections.

Because the paper is concerned with network-level structural evaluation and applications, the focus of the comparison was to evaluate the percentage of structurally weak sections that were identified by both the TSD and FWD. It is important to stress that the purpose was not to compare measurements collected by the TSD to measurements collected by the FWD. The time span of 9

to 12 years between the TSD and FWD data collections makes such a comparison inadequate. The objective of the comparison was to evaluate the effect of replacing the FWD data with TSD data on identified structurally weak sections. This was one aspect, along with other considerations, that VDOT managers felt was important to consider in making the final decision on whether or not to replace FWD with TSD. In this case, the 9- to 12-year time span, although not ideal, is reasonable. For one, collecting new FWD data would take 2 to 3 years to complete (at a significant cost). Therefore, for network-level comparison purposes, it is not realistic to expect data from both devices to be collected on the same day (as expected for a detailed device comparison), month, and even year. Second, it is known in the pavement engineering community that structural deterioration occurs at a much slower rate than surface condition deterioration. This is an important reason why the recommended frequency of network-level structural evaluation is generally 3 to 5 years, or even higher (Noureldin et al. 2005, Stubstad et al. 2012). This is supported by results obtained by Bryce et al. (2016), who, based on network-level FWD data collected in 2006 in the Bristol district in Virginia and data collected 5 years later in 2011, found that the structural condition had not deteriorated over the 5-year period. In fact, contrary to what was expected and after discarding pavement sections that had undergone M&R treatment, the authors found that the measured deflections had on average slightly decreased, suggesting the structural condition had slightly improved.

4.5.1 Data Collection

FWD structural condition data was obtained from VDOT's pavement management system (PMS) along with the pavement layer thickness for the interstate routes. The TSD data was collected between August and September 2017 by the ARRB intelligent Pavement Assessment Vehicle (iPAVe) on 6,469 km (4,020 mi) of roads. This included 2,414 km (1,500 mi) of interstate roads and 4,055 km (2,520 mi) of primary roads. This study used only the structural condition data collected on the interstates, as shown in Figure 4-2, as this was where the FWD data was available. The analysis is limited to flexible pavement sections and excludes rigid or composite pavement sections.

where, H_p is the total pavement thickness (mm) and SIP is the structural index of the pavement computed in equation 4-3. The coefficients k_1 , k_2 , and k_3 were 0.4369, -0.4768 , and 0.8182 for TSD deflection measurements, as proposed by Nasimifar et al. (2019).

$$SIP = D_0 - D_{1.5H_p} \quad (4-3)$$

where $D_{1.5H_p}$ is the deflection at a lateral distance of 1.5 times the pavement depth, H_p .

Temperature Correction

Temperature correction was performed for the deflection under the applied load, D_0 , and SCI300. Temperature correction for SCI300 was performed using the approach presented by Nasimifar et al. (2018), as shown in equation 4-4. The approach was inspired by the Lukanen et al. (2000) method for FWD measurements and utilizes the BELLS3 equation. The temperature correction factor is calculated as follows:

$$\lambda = \frac{SCI_{Ref}}{SCI_T} = \frac{10^{-0.05014T_{Ref} + 0.019049T_{Ref}\log(h_{AC})\log(\varphi)}}{10^{-0.05014T + 0.019049T\log(h_{AC})\log(\varphi)}} \quad (4-4)$$

where λ is the temperature adjustment factor, SCI_{Ref} is the adjusted SCI300 at a reference temperature, T_{Ref} is the reference temperature in degrees Celsius, h_{AC} is the asphalt layer thickness in millimeters, T is the mid-depth asphalt concrete layer temperature at the time of measurement in degrees Celsius (calculated according to equation 4-5), and φ is the latitude of the location of measurement (within 30 to 50 degrees).

$$T_d = 0.95 + 0.892 IR + 0.042 IR \sin(hr_{18} - 13.5) + \{\log(d) - 1.25\} \{-0.448 IR + 0.621 T_p + 1.83 \sin(hr_{18} - 15.5)\} \quad (4-5)$$

where T_d is pavement temperature at depth (d) in degrees Celsius, IR is pavement surface temperature in degrees Celsius, $\log$ is the base 10 logarithm, d is the depth in millimeters at which temperature is to be predicted, T_p is the average air temperature the day before testing in degrees Celsius, $\sin$ is the sine function on an 18-hour clock system with 2π radians equal to one 18-hour cycle, and hr_{18} is the time of the day in a 24-hour clock system but calculated using an 18-hour asphalt concrete temperature rise and fall time cycle. The average temperature on the day before testing (T_p) was obtained from National Oceanic and Atmospheric Administration land-based

weather station data. The S_{Neff} was computed using the temperature-corrected D_0 . The temperature correction for D_0 was performed based on the American Association of State Highway Transportation Officials temperature adjustment charts (AASHTO 1993).

4.5.3 Comparison between FWD and TSD Data on Interstate Roads

In order to migrate to the TSD for network-level applications, it is necessary to understand to what extent the TSD data and the FWD data are similar despite the more-than-9-year span between the collections of the two datasets. To achieve this, a detailed comparison between the S_{Neff} calculated from the TSD and the S_{Neff} calculated from the FWD was performed as follows:

- 1) Compare scatter plot of TSD and FWD S_{Neff}.
- 2) Compare the cumulative distribution of TSD and FWD S_{Neff}, SCI300, and D_0 .
- 3) Determine if S_{Neff} from the TSD and S_{Neff} from the FWD identify the same weak sections using a consistency test.

The first two comparisons were done using structural parameters calculated from the TSD and FWD. The third comparison, the consistency test, is more relevant to network-level decision-making because it is based on the weak sections that are identified by each device. These weak sections are the ones that will most likely have the initial treatment category selected after the first two steps of treatment selection (the steps based on the surface condition) changed to a heavier treatment category. This is an important factor to investigate as it is one of the criteria that will determine whether the FWD data currently in the PMS should be replaced with the more recently collected TSD data. As will be explained in more detail, the consistency can have a maximum value of 1. This occurs when two sets of measurements identify the same weak sections. A value of 1 might seem to be a good benchmark to which the consistency between the FWD and the TSD should be compared (i.e., compared to perfect matching); however, this is not the case. This is because two different sets of measurements, even if they are both obtained from the FWD, will generally not have a consistency of 1. In the case where a new set of FWD measurements is obtained to replace an older set, however, the newer set is readily accepted without much consideration to the consistency (unless there are some issues with the validity of the measurements). Therefore, a better benchmark for the consistency between the FWD and the TSD

would be the consistency that can be achieved when comparing two sets of FWD measurements. Although there were no FWD measurements collected in 2017, in 2011 VDOT collected a set of FWD measurements in addition to the one collected in 2006 on interstate roads in the Bristol district. These two sets of measurements, collected 5 years apart, were used to estimate a consistency between two sets of measurements that could then be used as a benchmark for comparing TSD and FWD values

The underlying assumption of the consistency test was that the structurally weakest sections identified by one set of measurements should also be identified by another set of measurements. In the consistency test, the structurally weakest sections were defined as a percentage of the total number of tested sections (e.g., the 10% weakest sections). The number of sections used for the consistency test is labeled N , and the two sets of measurements being compared are labeled C_i (e.g., the TSD SNeff at section i) and D_i (e.g., the FWD SNeff at section i), where $i = 1, \dots, N$. To calculate the consistency, a percentile α is selected to define the set of weakest sections (e.g., $\alpha = 0.05$ for 5%), and the two sets, I_1 and I_2 , of the weakest sections from C_i and D_i are determined as follows:

$$I_1 = \{i: C_i \leq C_\alpha\} \quad (4-6)$$

$$I_2 = \{i: D_i \leq D_\alpha\} \quad (4-7)$$

Where, C_α is the structural condition of the $[\alpha N]$ weakest section in the set C_i , and D_α is the structural condition of the $[\alpha N]$ weakest section in the set D_i (here $[\alpha N]$ is the largest integer n such that $n \leq \alpha N$).

After obtaining I_1 and I_2 , the consistency set T can be determined as the intersection of I_1 and I_2 .

$$T = I_1 \cap I_2 = \{i: i \in I_1 \text{ and } i \in I_2\} \quad (4-8)$$

The consistency CT_α is then calculated as follows:

$$CT_\alpha = 100 \frac{card(T)}{[\alpha N]} \quad (4-9)$$

where $card(T)$ is the cardinality of the set T (the number of elements in T).

Note that if the two measurement sets identify the same sections at a level α , then $CT_\alpha = 100\%$.

The consistency test was performed between TSD and FWD measurements on sections that did not have a major structural rehabilitation between 2006 and 2017 for α ranging from 1% to 100% at 1% increments. The results were compared with the consistency test performed by Katicha et al. (2017) between two sets of FWD measurements collected in 2006 and 2011 on interstate sections in the Bristol district (α of 5%, 10%, 15%, 20%, 25%, and 33%).

4.5.4 Impact of the Selected Structural Parameter on Identified Weak Sections

Based on the study by Rada et al. (2016) and the current VDOT practice with FWD data, the SCI300 and S_{Neff} are two possible structural condition indices to use for network-level PMS applications. If the identification of weak sections is not significantly affected by the chosen index, then the choice of which index to use for network-level applications is not too critical. Therefore, the consistency in identifying the same weak sections and the Spearman rank correlation between TSD SCI300 and TSD S_{Neff} were evaluated. The Spearman rank correlation between SCI300 and S_{Neff} is the correlation calculated using the ranking of the measurements rather than the actual measurements. It is more applicable than the regular Pearson correlation for comparing SCI300 and S_{Neff} because SCI300 is linearly related to the measured deflections while S_{Neff} is nonlinearly related to the measured deflections. This suggests that the relationship between SCI300 and S_{Neff} is nonlinear.

4.6 Results

The results are presented in two subsections. The first subsection compares the TSD to the FWD, and the second subsection compares SCI300 to S_{Neff}, which are both calculated from the TSD measurements.

4.6.1 Comparison of FWD and TSD Measurements

Figure 4-3Error! Reference source not found.(a) compares the TSD S_{Neff} and FWD S_{Neff}. Figure 4-3Error! Reference source not found.(b) compares the two sets of FWD S_{Neff} measurements collected in the Bristol district in 2011 and 2006 (Bryce et al. 2016). The scatter between the TSD and FWD is more pronounced than the scatter between repeated measures of the FWD. The Spearman rank correlation for the TSD S_{Neff} and FWD S_{Neff} is 0.54, whereas the Spearman rank correlation for the repeated FWD S_{Neff} values between 2011 and 2006 is 0.71.

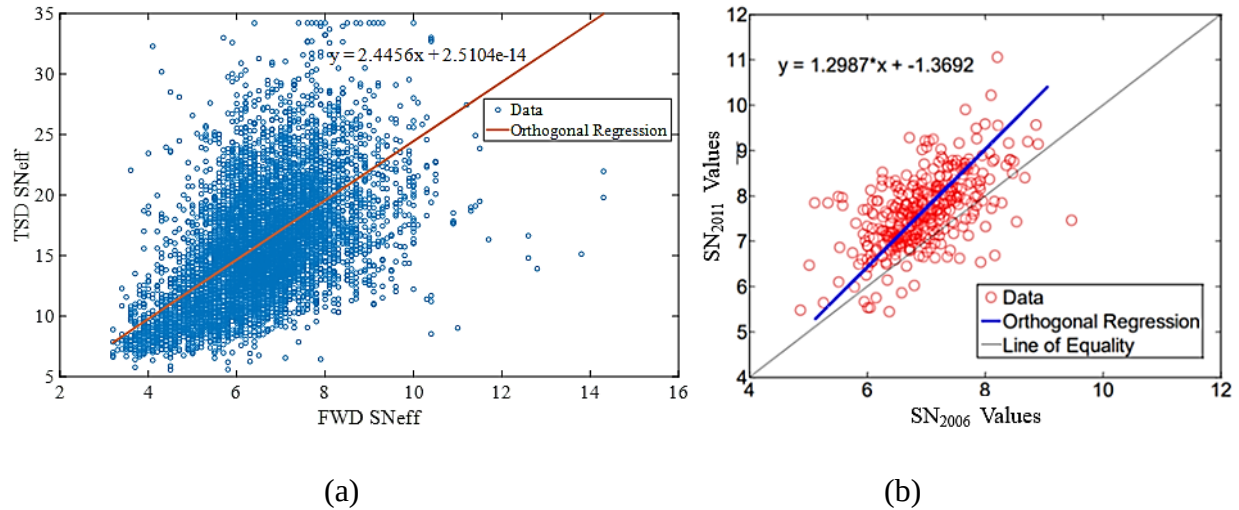
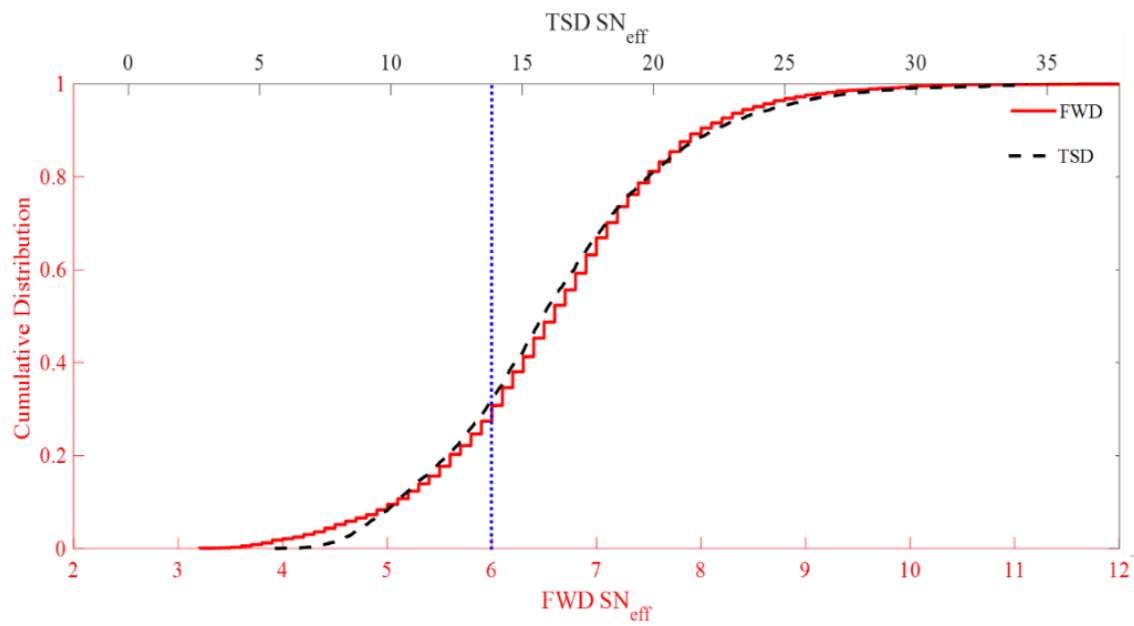
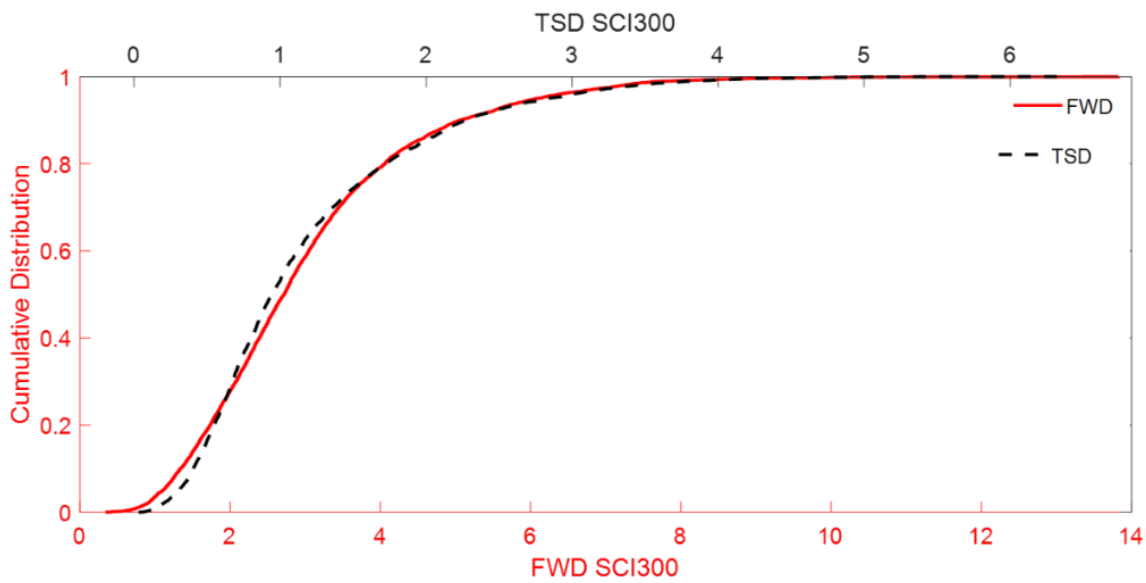


Figure 4-3 Comparison between values (a) TSD and FWD SNeff, (b) repeated FWD SNeff

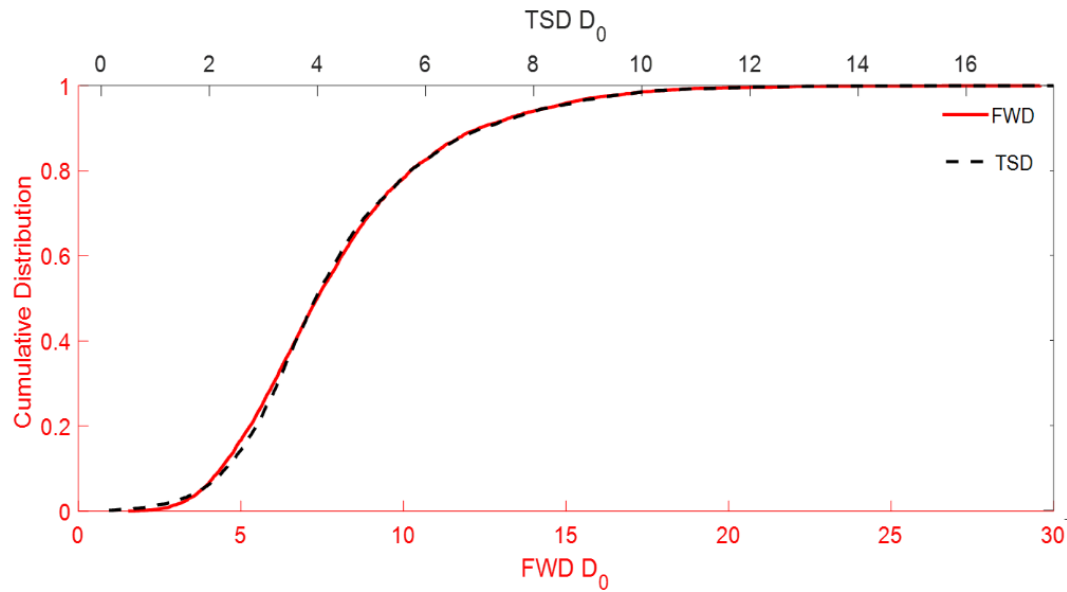
Figure 4-3 shows the comparisons between the cumulative distributions from the FWD measurements and the TSD measurements for SNeff, SCI300, and D₀. Figure 4-4(a) shows that the shapes of the SNeff cumulative distributions for the TSD and FWD are similar. This suggests that the procedure based on the cumulative distribution of FWD measurements used to determine the threshold between structurally weak sections can be also used for TSD measurements. The SNeff threshold of 6 was adopted by VDOT for the FWD. This threshold results in a proportion of 0.3 of the sections being identified as structurally weak and corresponds to a SNeff of 14 as calculated from TSD measurements. This means that using a TSD SNeff of 14 and an FWD SNeff of 6 identifies the same proportion of the network as weak sections. The distributions of SCI300 and D₀ are shown in Figure 4-4(b) and (c), respectively. These two figures also show similar distributions of TSD and FWD measurements, although the distributions of the TSD and FWD measurements have different means and standard deviations. A similar method to the one used to determine a weak section threshold from SNeff can be used to determine a weak section threshold for TSD SCI300 and D₀ measurements.



(a) TSD and FWD SN_{eff}



(b) TSD and FWD $SCI300$



(c) TSD and FWD D_0

Figure 4-4 Cumulative distributions on interstate roads.

Figure 4-5 shows the results of the consistency test. For a proportion $\alpha = 1\%$ of weak sections, the consistency is 0.28 (or 28%). As α increases, the consistency initially increases rapidly, reaching a value of 0.57 for $\alpha = 6\%$. Between $\alpha = 6\%$ and $\alpha = 30\%$ (corresponding to a FWD SNeff of 6 and a TSD SNeff of 14), the consistency varies within a narrow range of 0.57 to 0.62. For $\alpha > 30\%$, consistency increases at a slower rate until it reaches a value of 1 at $\alpha = 100\%$. The figure also shows the consistency of the FWD with the two datasets collected in Bristol and the consistency between the TSD and FWD with another set of TSD measurements collected in 2015. The consistency between the TSD and FWD is higher than the FWD consistency. The TSD data was collected at 10-m (33 ft) intervals and averaged over the 321.87 m (0.2 mi) length used for FWD data collection. Averaging the TSD measurement reduced the variability, increasing the consistency. For the FWD, only one measurement was collected every 321.87 m (0.2 mi), which could result in higher variability, especially if the two sets of measurements were not collected at the same spot, which is very likely since the two sets of measurements were collected 5 years apart. The benchmark consistency calculated for the FWD is smaller than the TSD-FWD consistency. This suggests that replacing the current FWD SNeff data with the new TSD SNeff data is, from a

statistical perspective, at least as good as updating the existing FWD SNeff data with a new set of FWD SNeff data. This finding supports adopting the TSD as a network-level structural evaluation device instead of the FWD.

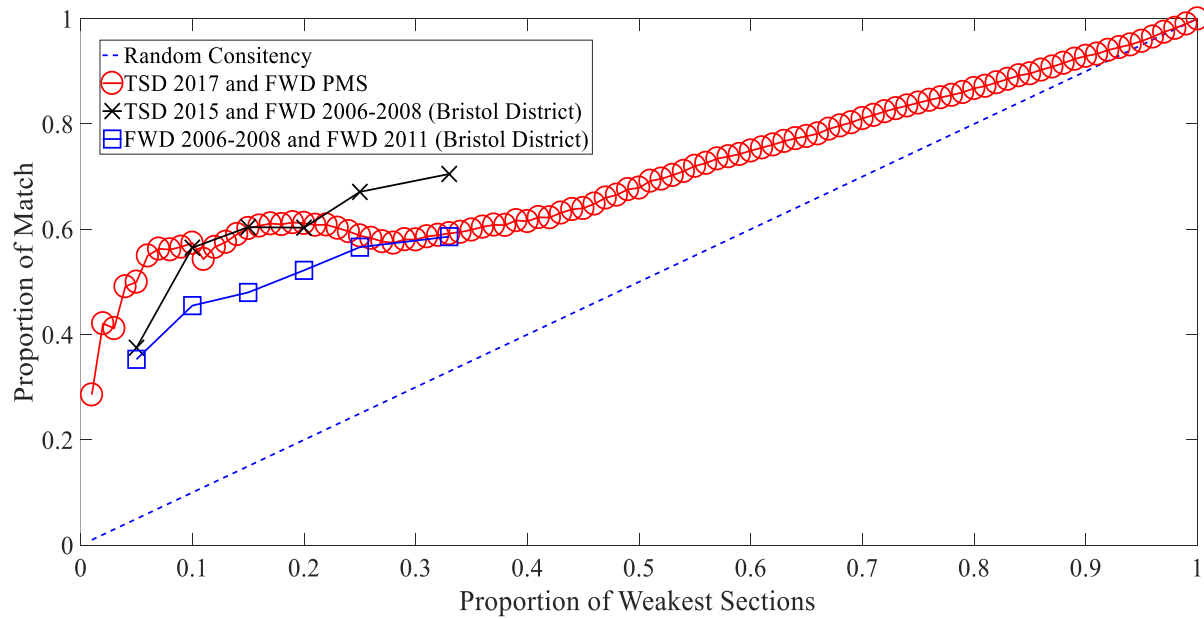


Figure 4-5 Results of the consistency test.

4.6.2 Impact of the Index Selection

This section evaluates the effect of choice between SCI300 and SNeff on the PMS-identified weak sections. The minimum consistency between SCI300 and TSD-based SNeff is 0.58 for a proportion of weak sections of 0.01, as shown in Figure 4-6. However, the consistency quickly increases to more than 0.8 when the proportion of weak sections considered is 0.15. The Spearman rank correlation between SCI300 and SNeff is -0.93 , which is relatively high (in absolute value), further confirming the consistency results. The results suggest that the effect of whether SCI300 or SNeff is used to characterize the structural condition does not significantly affect which sections are classified as weak or strong. The SCI300 was recommended by Rada et al. (2016) as a good parameter that relates to the tensile strain at the bottom of the asphalt layer. One of its advantages is that it (generally) does not require information about the layer thicknesses. On the other hand, SNeff is the index currently used by VDOT. Therefore, using TSD SNeff will provide a transition

from FWD- to TSD-collected data without the need to reinterpret the structural condition. This approach might be preferred from an implementation perspective.

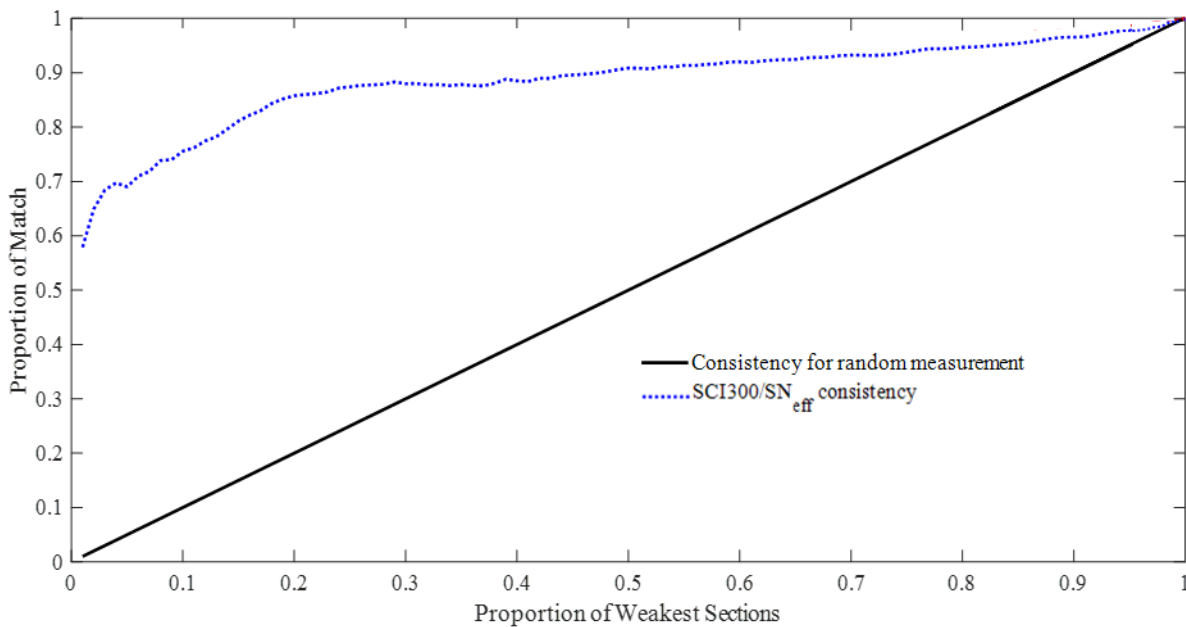


Figure 4-6 Consistency between SCI300 and SNeff.

4.7 Conclusions

This study evaluated the possibility of using TSD data for network-level pavement management decision-making in Virginia. To achieve this, the study compared the SNeff from the TSD and FWD, the cumulative distribution of the measurements (i.e., SNeff, SCI300, and D_0) from both the devices, and the consistency of identifying weak sections from the two devices. The study also evaluated the impact of selecting an index, either SCI300 or SNeff, from the TSD to evaluate the consistency of weak spot identification in Virginia. The conclusions of this study are as follows:

1. The structural condition obtained with the TSD can replace the structural condition obtained from the FWD that is currently used in the VDOT PMS. The distribution of the TSD SNeff was similar to the FWD SNeff, and the calculated consistency between the TSD SNeff and FWD SNeff was higher than the consistency between the SNeff from two repeated sets of FWD measurements.

2. The lower limit of TSD-based SNeff to identify structurally weak sections can be based on the 30th-percentile value. This was a similar process to the one followed with the network FWD data. The lower limit of TSD-based SNeff may be based on all collected data, or a separate SNeff may be developed for interstate, primary, and secondary routes.
3. There is very little practical difference between using the SNeff or SCI300 to identify structurally weak sections from TSD measurements based on the 30th-percentile value. The consistency between the two parameters is 0.88, and the Spearman rank correlation is -0.93. The SNeff has the advantage that it is the index currently used by VDOT. The SCI300 has the advantage that it does not require pavement thickness information and is mechanistically related to the tensile strain at the bottom of the asphalt layer.

Hence, the study recommends using the TSD measurements for network-level decision-making on bituminous pavements in Virginia. The TSD-based structural condition gives VDOT the pavement structural conditions at a much shorter sampling interval (0.1 mi) than the FWD-based structural condition (0.2 mi). Since the TSD is a continuous deflection measuring device, it does not pose interruptions to the traffic and has a higher production rate than the FWD. This gives TSD an advantage over FWD for network-level data collection and offers VDOT a realistic means of conducting network-level structural evaluation in its primary and secondary roads.

4.8 Recommendations for Implementation and Future Research

The current analysis was developed for VDOT, as it is based on the current VDOT FWD method for identifying structurally weak pavement sections. Similar analysis can be performed by other states based on their current methods of practice.

Virginia uses the structural condition to identify pavement sections that are structurally weak and assign heavier treatments to these sections. Similar procedures can be implemented for using the structural condition to identify structurally strong sections in the network and assign lighter treatment to them. This can help state agencies realize cost savings by reducing average maintenance costs per mile, eventually leading to more efficient pavement management decision-making.

4.9 Data Availability Statement

Some or all data, models, or codes used during the study were provided by a third party. Direct request for these materials may be made to the provider as indicated in the Acknowledgments.

4.10 Acknowledgments

We are grateful to ARRB Group Inc. for collecting the TSD data, Infrasense for collecting the ground penetrating radar data, and VDOT for help in obtaining the pavement condition data. The results reported in this paper are a part of a broader study conducted for VDOT.

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Chapter 5. Benefits of Network-Level Pavement Management Decision Making Based on Pavement Structural Condition

5.1 Abstract

Pavement surface distresses have been the major indicator to quantify the overall pavement condition. Historically, surface cracking was used as an indicator of the pavement structural condition. This was worked when state agencies were mostly doing the worst-first type of rehabilitation. However, with effective pavement preservation activities that intervene early to preserve and extend the life of pavements, the surface cracks can no longer be relied on as a reliable indicator of structural condition or “health” of the pavement structure. Therefore there is a need to directly incorporate the pavement structural condition into network-level pavement management decision-making. This paper compares network-level pavement management decision-making approaches based on pavement surface condition and pavement structural condition. A life-cycle analysis was performed on a pavement section considering cases with and without setting a minimum pavement treatment interval between successive treatments. The treatment interventions, average Critical Condition Index (CCI), average Condition Factor (CF), and cumulative cost over the life cycle were compared for the different decision-making approaches. The results show that decision-making using the pavement structural condition provides a more cost-effective M&R strategy over the pavement section’s life cycle. The study reported savings between 9% and 11% when the decision-making was based on the pavement structural condition compared to decision-making based on the surface condition.

5.2 Introduction

Pavement condition can be assessed using different criteria, including the surface and structural conditions. The pavement surface condition represents the distresses present at the pavement surface (such as rutting, cracking, patching, roughness, potholes, and delamination), whereas the pavement structural condition represents the overall pavement bearing capacity. It is important to

note that some of the distresses (e.g., cracking) are indication of the pavement structural condition. Evaluating both, the pavement surface and the structural conditions allows for a better assessment of the overall strength of the pavement (Bryce et al. 2012).

Pavement surface distresses have been evaluated using both manual and automated distress identification methods. Various semi-automated and automated tools have been used to identify, rate, and record the distress severity and frequency (Attoh-Okine and Adarkwa 2013). These have allowed accurate high-speed distress assessment. On the other hand, until recently, the evaluation of the pavement structural condition has been mostly limited to stationary or slow-moving deflection measuring devices. However, recent technological advancements have made possible the monitoring of pavement structural condition at traffic speeds. This makes collecting pavement structural condition information at the network level practical, which opens up the possibility of using this information for network-level pavement management decision-making.

In the United States, the falling weight deflectometer (FWD) has been the go-to device for measuring the pavement structural condition at the project level. Although they have been also used for network-level evaluation, their operation is inefficient because of the need to stop for testing, which requires traffic control, increases operation costs, limits the frequency of data collected, and creates a potential safety hazard for the operator (as the FWD is stationary and usually operates adjacent to a traffic lane). Hence, most states have relied only on the pavement surface condition for network-level pavement management decision-making (Flintsch et al 2013). The issue with this approach is that pavement surface distress is a “lagging indicator” of the overall pavement health and does not always reflect the current pavement structural condition (Rada et al. 2016). Most preservation treatments correct surface cracks but do not correct bottom-up fatigue cracking, instead conceals them, while the bottom-initiated cracks continue to develop. In addition, the prevalence of top-down cracking in thicker pavements also makes it difficult to distinguish bottom-up fatigue cracking which is the common indicator of structural deterioration. Therefore, the surface cracks can no longer be relied on as a reliable indicator of structural condition. Furthermore, weak correlations between the pavement surface condition and the structural condition have also been previously reported by several studies (Bryce et al. 2012; Flora 2009). Hence, pavement management decision-making should not be solely based on the pavement

surface condition, as the pavement surface condition is not a reliable predictor of the structural capacity (Zihan et al. 2019).

The use of only pavement surface (and sometimes functional) condition for network-level pavement management decision often result in an increasing gap between network-level and project-level decisions. This leads to less than optimal predictions and budget estimations. To mitigate this issue, the pavement structural condition from continuous deflection devices can be used for enhancing network-level decision-making (Shrestha et al. 2021). Therefore, it is necessary to understand the impact of using structural condition information on network-level decision-making. To do so, this study compares network-level decision-making based on the structural condition and surface condition index for a typical pavement section.

5.3 Objective

The main objective of this paper is to show that incorporating the pavement structural condition into the pavement management decision-making process results in more cost-efficient treatment selection. To achieve this, the study simulated various network-level decision-making scenarios based on two different approaches (1) based on a structural condition parameter, and (2) the surface condition parameter, considering both the cases with and without the pavement treatment intervals. The simulation was run on a “typical” pavement section using various network-level decision-making approaches. The treatment intervention schedule, average Critical Condition Index (CCI), average Condition Factor (CF), and the cumulative cost of the treatment are compared for the different scenarios considered.

5.4 Background

5.4.1 Pavement Management System

The concept of a pavement management system (PMS) was first developed in the late 1960s as a systematic way to manage the pavement. It integrated systems engineering, management principles, engineering analysis, and economic evaluation (Haas and Hudson 2015). The American Association of State Highway and Transportation Officials (AASHTO) describes pavement management as “...the effective and efficient directing of the various activities involved in

providing and sustaining pavements in a condition acceptable to the traveling public at the least life cycle cost.” (AASHTO 1985)

Generally, pavement management decisions can be classified into two different levels based on the level of analysis: network level and project level. At the network level, the primary goal of the PMS is to assess the overall pavement condition, examine time and budget constraints to produce a prioritized schedule, and estimate future funding needs. At the project level, more detailed analysis is carried out within scheduled projects, and decisions for various M&R activities are finalized.

5.4.2 Current pavement management decision-making practice

Most states currently rely primarily on pavement surface conditions to make network-level pavement management decisions. The pavement surface condition is usually represented by several parameters and these parameters vary from state to state. They combine various surface distresses, such as rutting, patching, bleeding, transverse cracking, fatigue cracking, and longitudinal cracking, into a pavement condition index, which usually ranges from 0 to 100. Sometimes, the index also include the pavement roughness, which is measured in terms of the International Roughness Index (IRI). To identify various distresses and quantify their frequency and severity, most states use an approach similar to that described in the *Long-Term Pavement Performance (LTPP) Distress Identification Manual for the Long-Term Pavement Performance Program* (Miller and Bellinger 2003). The pavement surface parameters are used in network-level decision-making process in various stages to assign appropriate pavement treatment activity to the pavement sections based on their overall condition.

For example, the Virginia Department of Transportation (VDOT) has developed a two-step network-level decision-making process. In the first step, Virginia relies on four individual surface distresses: patching, rutting, transverse cracking, and alligator cracking, as well as the combined index, CCI, to assign a treatment category to a pavement section. Decision matrices are used to assign the treatment category based on the combination of the distresses present, their severity, and frequency of occurrence. The treatment categories assigned are either DN, PM, CM, RM, or RC (Chowdhury 2016). Specific treatment activities within each of the treatment categories can be found in Chowdhury (2016).

The allocated treatment is then filtered using Virginia’s surface condition index, the CCI, as shown in Table 5-1 to determine the preliminary treatment category.

Table 5-1 CCI triggers for selecting for interstate roads in Virginia (Chowdhury 2016)

CCI	Treatment Category
above 89	DN
above 84	DN or PM
below 60	CM, RM, or PM
below 49	RM or RC
below 37	RC

For interstate roads, a second step considers three other parameters, pavement age, traffic, and pavement structural condition, to enhance the preliminary treatment category. For primary roads, the pavement age and the traffic are used, and for the secondary roads, only the traffic level is used to modify the preliminary treatment category. This is because the pavement structural condition information is not readily available for the primary and secondary roads.

Virginia is one of the very few states that include the pavement structural condition in their network-level decision-making. Most states just rely on the pavement surface condition and do not incorporate structural condition information into their network-level pavement management decision-making process (Flintsch et al. 2013; Flintsch and McGhee 2009; Shrestha et al. 2018; Zaghoul and Kerr 1999).

5.4.3 Relationship between Surface Condition and Structural Condition

Both the structural and the surface conditions play a significant role in pavement deterioration. Although most states solely use the pavement surface condition to quantify the pavement deterioration, surface condition alone does not always account for the actual pavement structure condition. It is a lagging indicator as it takes time for surface distresses to reflect a weak pavement structure on the pavement surface (Haas and Hudson 2015). Therefore, it is possible for a pavement with a good structural condition to have a poor surface condition and vice versa (Zhang et al. 2016). Therefore, the structural condition should also be considered for network-level decision-making and deterioration modeling.

The network-level decisions are significantly different when they are based on the surface condition compared to when the structural condition is taken into account (Katicha et al. 2020). Hence, the decision-making based solely on the pavement surface condition might not always yield the most-efficient decision-making (Elbagalati et al. 2017; Lister et al. 1982; Smart 2011; Steele et al. 2015).

5.5 Methodology

This paper compares network-level life cycle analysis based solely on the pavement surface condition versus decision-making solely based on the structural condition. The methodology discussion is divided in two main sections. The first section explains the surface and structural deterioration models used. The second section presents the simulation scenarios investigated. Various parameters used in this study, such as decision-making practices, treatments within the different treatment categories, cost of the treatment categories, pavement condition indicators used, etc., are inspired by Virginia's current practice. The data used in the study is based on the information available on Interstate 81 Southbound (I-81 SB) in Virginia.

5.5.1 Condition Assessment Parameters

The parameters used in this study to quantify the pavement surface condition and the pavement structural condition are the CCI and CF, respectively.

Surface condition parameter: CCI. As this study uses measurements collected in Virginia, the CCI was selected as the pavement surface condition parameter. VDOT aggregates various pavement distresses into two categories: Load-related Distress Rating (LDR) and Non-load-related Distress Rating (NDR). The LDR comprises load-related distresses that arise due to repeated traffic loading action on the pavement surface, including patching, rutting, and fatigue cracking. The NDR comprises distresses that are non-load related and caused by various other factors, such as construction deficiency and weathering of the pavement surface due to environmental factors. Distresses that come under NDR include longitudinal cracking, transverse cracking, and bleeding. The CCI is calculated as the lower of LDR and NDR. It ranges from 0 to 100, 100 representing a pavement with no visible distress, and 0 representing a failed pavement.

(McGhee 2002; Vlacich 2018). Further details on the calculation of the CCI can be found in McGhee (2002).

Structural condition parameter: CF. The CF was selected as the structural condition parameter to track the structural condition deterioration. AASHTO (1993) defines CF as the ratio of effective structural number (SN_{eff}) to the designed structural number (SN_{design}), as shown in equation 5-1. A CF of 1 represents a pavement structure as strong as it was originally designed (i.e., structurally sound for the designed future traffic), whereas a CF of 0.5 represents a case of structural failure.

$$CF = \frac{SN_{eff}}{SN_{design}} \quad (5-1)$$

A design structural number of 7 was assumed to calculate the pavement CF based on the 20-year future traffic prediction. To calculate the effective structural number (SN), the study used the pavement deflection measurements collected by the TSD on I-81 Southbound in Virginia in 2017. Rohde's approach was used to calculate the effective SN from the TSD measurements (Rohde 1994). For further details on the TSD data collection and calculation of the effective SN, please refer to Shrestha et al. (2021).

5.5.2 Deterioration Models

Two different deterioration models were used in this study: Surface Condition Deterioration Model, and Structural Condition Deterioration Model.

Surface Condition Deterioration Model: The pavement surface deterioration model originally developed by Shrestha et al. (2021) for I-81 Southbound was modified and used in this study. The original CCI deterioration model was based on a quasi-poisson regression and accounted for two factors: pavement age and surface curvature index (SCI300) (Shrestha et al. 2021). To track and quantify the pavement structural deterioration more accurately, the original deterioration model was modified and the SCI300 parameter was replaced with the CF. The equation for the modified CCI deterioration model used in this study is shown below:

$$CCI = 100 - \exp(\beta_0 + \beta_1 \times \log(Age) + \beta_2 \times \frac{\log(Age)}{CF^3}) \quad (5-2)$$

where, CCI = Critical Condition Index; Age = pavement age; CF = Condition Factor and β_0 , β_1 , and β_3 = regression coefficients.

Structural Condition Deterioration Model: The pavement structural condition model was developed based on the relationship between the CF and the remaining life (RL) of the pavement for flexible pavements as reported in AASHTO's 1993 *Guide for Design of Pavement Structures* (AASHTO 1993). The CF-RL relationship as shown in equation 5-3, which was first developed by Elliott for AASHTO's 1986 *Guide for Design of Pavement Structures*, was adjusted to avoid the issue that whenever RL is 0, CF is also 0, which was not considered to be realistic.

$$CF = RL^{0.165} \quad (5-3)$$

where CF = Condition Factor and RL = Remaining life of the pavement.

A deterioration model was developed to track the pavement structural deterioration. The structural deterioration model deteriorated from a CF of 1 to 0.5 over the design life span of 20 years, with the rate of deterioration increasing every 5 years.

Figure 5-1 presents the assumed pavement surface condition deterioration (i.e., CCI deterioration on the left y-axis in blue) and the structural condition deterioration (i.e., CF deterioration on the right y-axis in orange) over a period of time when no treatment is applied to the pavement. As previously stated, the surface fails sooner than the pavement structure if no treatment is applied to the pavement.

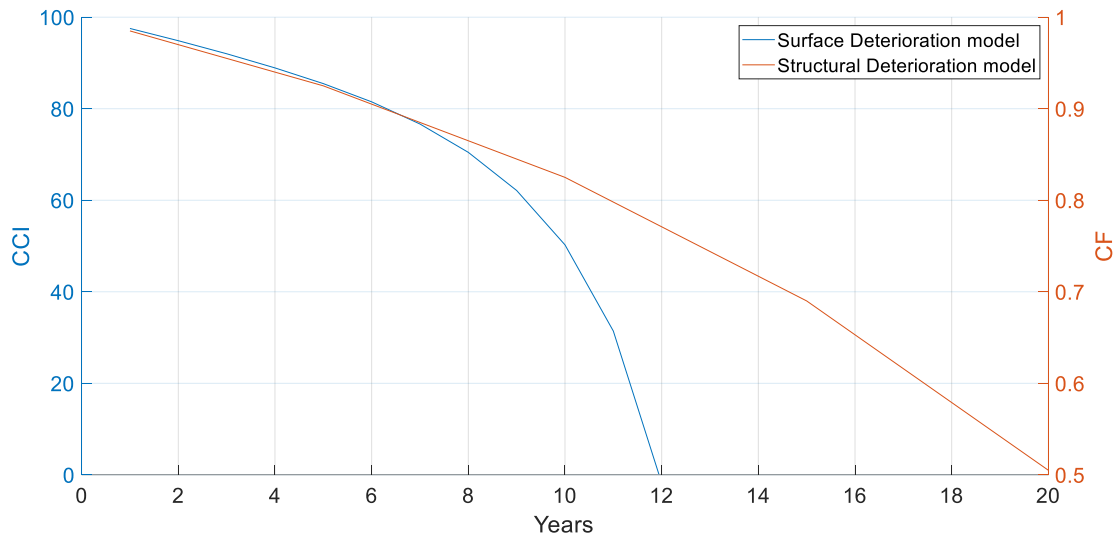


Figure 5-1 Pavement surface condition and structural condition deterioration over time

5.5.3 Network-level decision-making simulation

The network-level life cycle analysis was performed using simulation on a pavement section over a long period of time. An analysis period of 100 years was selected to capture at least one complete pavement lifecycle, starting with a new pavement to the end of pavement life (i.e., reconstruction). The simulated pavement M&R decisions included procedures based on the surface condition and on pavement structural condition. The scenarios aim to mimic current practices used by state agencies. While most current practices rely only on the surface condition indicators, future implementation scenarios are expected to use both the surface condition and structural condition information for network-level treatment selection decisions. This study takes the first step towards that goal.

Error! Reference source not found. shows a summary of simulation steps. It begins with a new pavement and a full CCI of 100 and a full CF of 1, as no distresses are present and it is constructed to the designed SN. The pavement deteriorates with time as shown in Step 1. In Step 2, the treatment category is selected based on the decision-making approach opted for (i.e., either based on the surface condition or based on the structural condition). Further details on Steps 2a and 2b are discussed in the following sections. In Step 3, the treatment is applied to the pavement, and the

CCI and CF of the pavement are modified according to the treatments applied. The steps are repeated for the year of the analysis period.

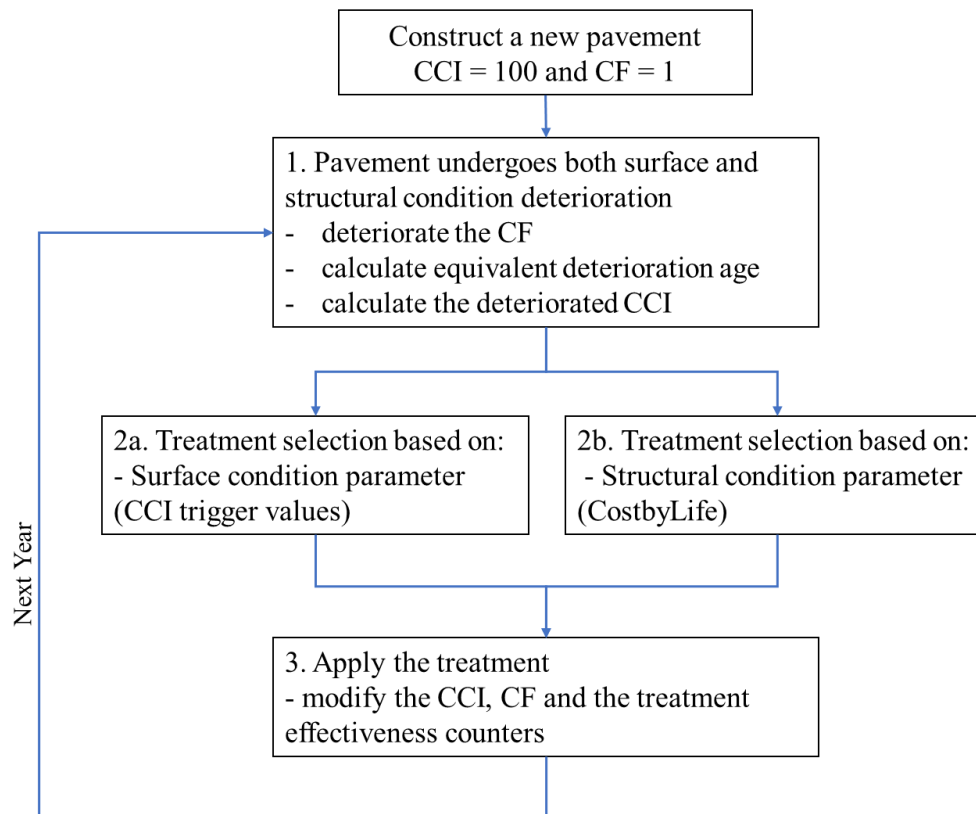


Figure 5-2 Summary of steps in network-level decision-making simulation

Treatment category selection

Two different treatment selection approaches were evaluated: (a) based on surface condition (i.e., Step 2a as shown in **Error! Reference source not found.**) and (b) based on the structural condition (i.e., Step 2b as shown in **Error! Reference source not found.**). The two different decision-making approaches are discussed below:

Treatment selection based on the pavement surface condition: The treatment categories were selected solely based on the CCI trigger values as shown in Table 5-2. These trigger values were inspired by Virginia’s current practice of using trigger values based on the CCI for their network-level treatment selection. VDOT uses the triggers based on the CCI as a secondary filter after evaluating the severity and frequency of four different distresses: rutting, patching, alligator

cracking, and transverse cracking. However, the process was simplified for the simulation and no individual distresses were analyzed for this part of the study.

Table 5-2 Trigger Values and Cost for treatments based on CCI

Treatment Category	Trigger Values	Cost (\$ per mile)
DN	$CCI \geq 87$	-
PM	$73 \leq CCI < 87$	25,000
CM	$55 \leq CCI < 73$	90,000
RM	$40 \leq CCI < 55$	170,000
RC	$CCI < 40$	500,000

It is to be noted that some state agencies also use other functional parameters, such as IRI and surface friction, in their network-level analyses, but these were not used in this study.

Treatment selection based on the pavement structural condition: A Cost-by-Life parameter was used to select the most effective treatment based on the effective structural capacity. The Cost-by-Life parameter is defined as the overall cumulative cost of the pavement treatment divided by the total life of the pavement. The Cost parameter used in Cost-by-Life includes the initial cost of construction of the pavement along with the costs of all the treatments applied on the pavement to date until the pavement undergoes reconstruction. The Life parameter used in Cost-by-Life includes the total life of the pavement until it reaches structural failure (i.e., CF of 0.5). The Cost-by-Life (C/L) parameter is calculated as follows:

$$C/L = \frac{\text{Cumulative Cost of treatment including the initial construction(Cost)}}{\text{Total life of the pavement (Life)}} \quad (5-4)$$

The costs associated with each treatment category were approximately based on Virginia's treatment category costs for interstate asphalt pavements (Chowdhury 2016). The costs adopted for all treatment categories are shown in Table 5-2.

Impact of treatments on the pavement surface and structural condition

The surface and structural condition improvements from the application of different treatment categories were closely based on VDOT's current practice. Almost all of the treatment categories except DN involve some form of pavement resurfacing (Chowdhury 2008; Izeppi et al. 2015). Hence, it was assumed that these treatments (PM, CM, RM, and RC) reset the CCI to a full score of 100. The treatments CM, RM, and RC also contribute to the improvement of the pavement structural condition. PM is comparatively a lighter treatment and does not make any significant contribution to the pavement structural condition. Hence the structural improvement from applying PM was dependent on the existing pavement structural condition. For CM, RM, and RC, it was assumed that applying these treatments added a certain structural capacity to the pavement (in terms of CF) independent of the existing structural condition. The structural capacity improvements for each treatment category is calculated using equation 5-5.

$$CF_{imp} = \Delta CF \times Timefactor \quad (5-5)$$

where CF_{imp} = final structural improvement from each treatment category; ΔCF = initial structural improvement; and Timefactor = time factor depending on the pavement effectiveness counter as presented in Table 5-3. Both parameters are based on the *effective age counter* (EAge) for each treatment category. The Timefactors reduce the treatment's effectiveness with time. This ensures that, as the pavement ages, the lighter treatments start to become less effective sooner compared to the heavier treatments, similar to what is observed in the real world.

Table 5-3 Initial structural contribution and Timefactor for each treatment category

Treatment Categories	ΔCF	Timefactors
PM	$0.08 \times CF_{current}^{1.5}$	$1 - (PM_EAge - 5) \times (1/20)$
CM	0.28	$1 - (CM_EAge - 17) \times (1/20)$
RM	0.5	$1 - (RM_EAge - 30) \times (1/20)$
RC	1	Timefactor
	$CF_{current}$ is the actual Condition Factor (CF) of the pavement for the year	PM_EAge, CM_EAge, RM_EAge are effective age counters.

The effective age counters increase every year and reset to 0 every time a heavier treatment is applied on the pavement. For example, the effective age counter for PM resets every time CM or

RM is applied on the pavement, and the effective age counter for CM resets every time RM is applied. This increases the effectiveness of the lighter treatments after every application of a heavier treatment.

5.6 Results

To reflect different practices among agencies, the simulation also included scenarios that establish minimum treatment intervals for each treatment. Minimum intervals of 4 years, 7 years, and 12 years were considered for PM, CM, and RM, respectively. For each scenario, the treatment time series, average CF (i.e., average of CF before and after treatment), average CCI (i.e., average of CCI before and after treatment), and the cumulative cost of the pavement are reported.

With Minimum Interval between treatment:

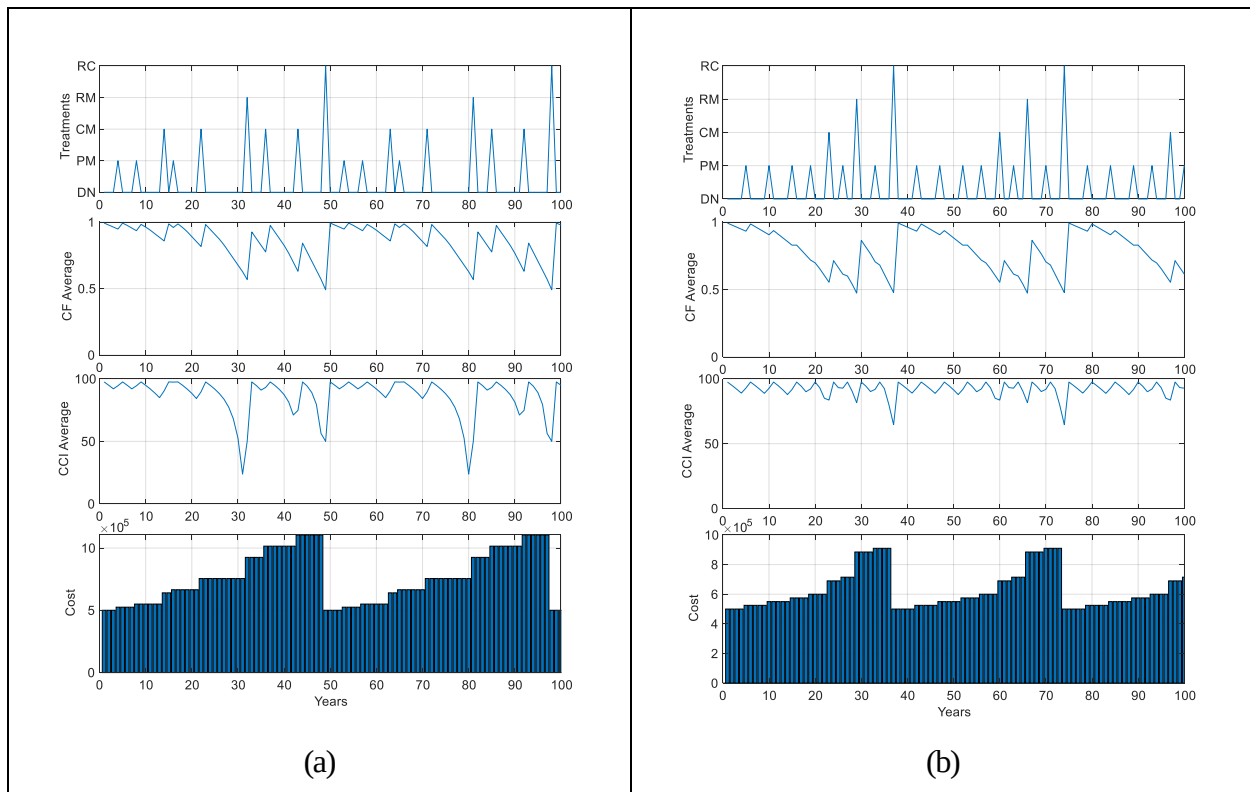
Figure 5-3(a) shows the results for the structural-condition-based treatment selection considering minimum intervals between treatments. The pavement lasts 48 years before it undergoes RC and has a cumulative cost of \$1,105,000 per mile. The first PM, CM, and RM are applied in years 4, 14, and 32, respectively. At year 31, the average CCI falls to 24, which would have called for a reconstruction based on the CCI trigger values, if the surface condition had been considered. Since the selection was based on the structural condition parameter (i.e., Cost-by-Life), RM was selected as it was the most cost-effective treatment. After the reconstruction, the pavement behaves as new pavement, and the intervention cycle and pavement deterioration repeat.

Figure 5-3(b) shows the results for surface-condition-based treatment selection when treatment intervals are considered. The pavement lasts 36 years before it undergoes RC and has a cumulative cost of \$910,000 per mile. The first PM, CM, and RM are applied in years 5, 22, and 29, respectively. At year 23, although the pavement has a good surface condition with an average CCI of 83, the structural condition is almost near failure with an average CF of 0.55. This is a typical example of why the pavement surface does not always necessarily represent the pavement structural condition.

Without Enforcing a Minimum Interval between treatment:

Figure 5-3(c) shows results for structural-condition-based decision-making when treatment intervals are not considered. The pavement lasts 49 years before it undergoes RC and has a cumulative cost of \$1,130,000 per mile. The first PM, CM, and RM are applied in years 4, 18, and 32, respectively. Similarly,

Figure 5-3(d) shows the results for surface-condition-based decision-making when treatment intervals are not considered. The pavement lasts 44 years before it undergoes RC and has a cumulative cost of \$1,140,000 per mile. The first PM, CM, and RM are applied in years 4, 25, and 39, respectively.



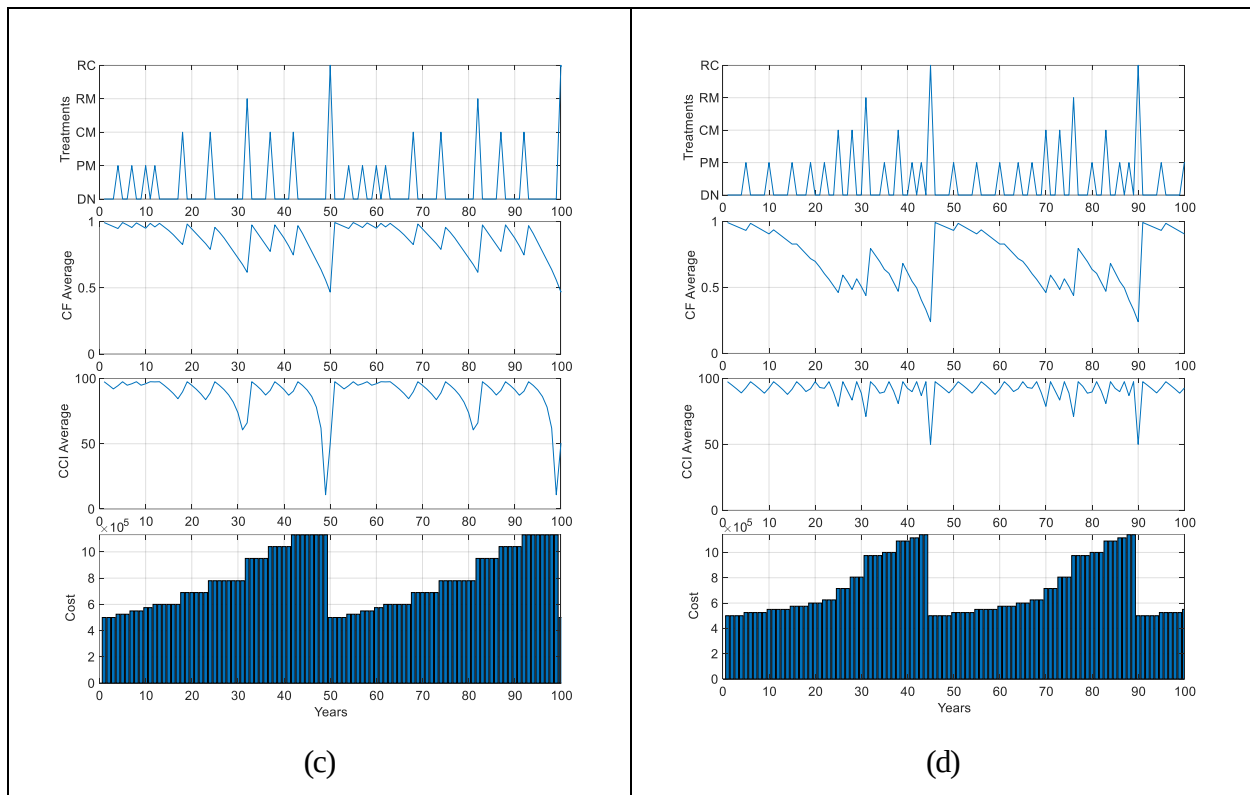


Figure 5-3 Results for Surface-Condition-based and Structural-Condition-based decision-making with and without interval.

Table 5-4 summarizes the simulation results for the four scenarios investigated. The comparison shows that the treatment selection based on the structural condition decision-making is more cost-effective (i.e., in terms of cost per year per mile) than the one based on surface condition, both with and without enforcing minimum pavement treatment intervals. The structural-condition-based treatment selection resulted in savings of 9% and 11% when compared to the cost for surface-condition-based selections.

Table 5-4 Cost-by-Life comparison between decision-making based on the surface condition and structural condition

Decisions based on	With Interval					Without Interval				
	Life (in yrs.)	Total Cost	Cost per year (Cost-by-Life)	Avg CF	Avg CCI	Life (in yrs.)	Total Cost	Cost per year (Cost-by-Life)	Avg CF	Avg CCI
Surface condition	36	\$910,000	\$25,278	0.78	92	44	\$1,140,000	\$25,909	0.71	92
Structural condition	48	\$1,105,000	\$23,021	0.85	86	49	\$1,130,000	\$23,061	0.87	89
Difference			\$2,257					\$2,848		

5.7 Conclusions

This study compared network-level pavement management decision-making based on pavement surface condition and structural condition. The results showed that using the structural condition resulted in a more cost-effective treatment selection. Other findings of the study are summarized below:

- 1) As expected, network-level decision-making based on pavement surface and structural condition can result in significantly different maintenance and rehabilitation interventions over the pavement life cycle.
- 2) Structural-condition-based treatment selections can be more cost-effective than decision based only on surface condition, as they account for the pavement structural life. In the case study investigated, making using the pavement structural condition resulted in 9 to 11 % savings in life-cycle costs.

Future research should explore decision-making scenarios based on both the structural and surface condition parameters together. These are expected to provide even more optimized life-cycle interventions.

5.8 Acknowledgments

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Chapter 6. Findings, Conclusions and Recommendations

This dissertation main objectives was to develop processes and enhance tools to incorporate the pavement structural condition into a network-level pavement management. The investigations showed that pavement structural condition plays an important role in the rate of pavement deterioration, that structural condition from TSD can be used to enhance network-level pavement management, and that the incorporation of structural condition into the pavement management decision-making can result to more cost-effective decisions.

Most of the state agencies rely only on pavement condition indices to summarize the overall pavement condition. The dissertation reviewed relevant literature on pavement management and condition indices (both surface and structural) used for pavement management decision-making. It provided a brief overview of continuous deflection measuring devices and their applications for network-level pavement management. The dissertation then investigated the impact on pavement structural condition as measured by the TSD on the rate of deterioration of the surface condition and how the TSD data can be incorporated the into the PMS decision-making process, based on VDOT's current pavement management decision-making practice. Finally, the study compared decision-making approaches based on the surface condition and structural condition.

6.1 Findings

This research has led to the following findings:

1. Pavement surface condition has a significant effect on the rate of pavement deterioration.
2. A weak correlation was observed between the pavement surface and structural conditions., for the tested sections in Virginia. The correlation between SCI300 and cracking was very small, and the correlation between SCI300 and rutting was reported to be even weaker.
3. From a statistical perspective, replacing the current FWD S_{Neff} data with the new TSD S_{Neff} data in Virginia for network-level decision-making is at least as good as updating the existing FWD S_{Neff} data with a new set of FWD S_{Neff} data.
4. In Virginia, applying an augmented matrix on the interstate network resulted in reducing the percentage of the network requiring CM and increasing the percentage requiring PM

and RM. One explanation for this is because the augmented matrix was able to target weak and strong pavement sections and allocate heavier and lighter treatments accordingly, instead of just applying CM to the sections.

5. There is very little practical difference between using the S_{Neff} or SCI300 to identify structurally weak sections from TSD measurements based on Virginia's current practice of identifying weak pavements (i.e based on the 30th-percentile value).
6. Network-level decision-making using structural condition parameters produce different life cycle maintenance and rehabilitation interventions than decisions based only on surface condition.

6.2 Conclusion

The main conclusion of the study are as follows:

1. Pavement structural condition plays a significant role in pavement deterioration, Structurally weak pavement sections tend to deteriorate at a faster rate than structurally strong sections.
2. Structural condition data obtained with the TSD can replace the structural condition obtained from the FWD that is currently used in the VDOT PMS. The distribution of the TSD S_{Neff} was similar to the FWD S_{Neff}, and the consistency between the TSD S_{Neff} and FWD S_{Neff} was higher than the consistency between the S_{Neff} from two repeated sets of FWD measurements.
3. Structural-condition-based treatment selections can be more cost-effective than decision based only on surface condition, as they account for the pavement structural life. In the case study investigated, making using the pavement structural condition resulted in 9 to 11 % savings in life-cycle costs.

6.3 Contributions

Network-level application of continuous deflection devices has gained a huge interest in the past decade. Using continuous deflection devices for network-level applications helps agencies to attain more accurate need-based budgets allocation, perform more cost-effective decision-making, and identify structurally deficient pavement sections at the network level. This dissertation has

documented one of the first efforts to incorporate the pavement structural condition based the TSD measurements into pavement management practice. The study proposed a treatment selection matrix that incorporates the pavement structural condition and modifies the initial treatment based on the surface condition. Since incorporating the structural condition is a supplementary step, it makes it easier for the state agencies to utilize and implement it in their current decision-making process. Furthermore, the dissertation showed that incorporating TSD measurements for network-level decision-making is possible in Virginia and can lead to more cost-effective treatment selection compared to decisions solely based on surface condition.

6.4 Recommendations for Future Work

1. Identify the most cost-effective approach for network-level decision-making, using pavement surface and structural condition parameters together.
2. Quantify the pavement remaining structural life using structural condition measurements collected by the Traffic Speed Deflectometer.
3. Develop structural condition thresholds, based on TSD deflection measurements, and computed indices, to categorize the pavements in Good, Fair, and Poor condition.

Appendix A. Code Used in Chapter 5: Benefits of Network-Level Pavement Management Decision Making Based on Pavement Structural Condition

Surface Condition Based Decision Making:

- *Main simulation code*

```
clear
%%% PLEASE READ EFFECTIVENESS COUNTER BEFORE RUNNING THE
SCRIPT TPMRESET 1 - NAMING A. TPM RESET2 - NAMING B%%%
S_Age=1;
St_Age=0;

%To get the structural condition deterioration
alpha=1.35;
[CF,r0,M,~,b] =
Deterioration_Curve(20,5,10,15,alpha,alpha^2,alpha^3);
cf_old1=1;
for t=1:300
    %      DR(t,1) = 0.025;
    DR_Struct(t,1) =
round(DeteriorationRate(r0,t,alpha),3);
    CF1(t,1)=cf_old1-DR_Struct(t,1) ;
    cf_old1=CF1(t,1);
end

%Simulation begins
DR=DR_Struct;
CCI=100;
cost=500000; %cost of constructing a new pavement
cf_old=1;
tDN=0;tPM=0;tCM=0;tRM=0;tRC=0;
CASE=4;
RC_finder=0;
treatment_surf=1;
p_point=0;
c_point=0;
r_point=0;
rc_point=0;
INTERVAL="NO"

N=150; %total time period
```

```

for i = 1:N
    if treatment_surf==5
        cost=500000;
    else cost=cost;
    end

    PM_point=max(p_point-1,0);
    CM_point=max(c_point-1,0);
    RM_point=max(r_point-1,0);
    RC_point=max(rc_point-1,0);
    ALL_POINT(i,:)=[PM_point,CM_point,RM_point,RC_point];

    St_Age=St_Age+1;
    %Structural Condition Deteriorates
    cf=cf_old-DR(round(St_Age));
    cf_avg=(cf+cf_old)/2;
    %Surface Deteriorates
    B0= 0.90065;
    B1= 0.88212;
    B2= 0.16766;

    CCI=max((100-
exp(B0+B1.*log(S_Age)+B2.*log(S_Age)./(cf_avg.^3))),0);
    S_Age =
getDeteriorated_Agenextyear_c2(B0,B1,B2,cf_avg,CCI);
    % end

    CCI1=CCI;
    %CCI based Preliminary decision making
    Points(i,:)=[PM_point,CM_point,RM_point,RC_point];

    [treatment_surf,PM_point1,CM_point1,RM_point1,RC_point1] =
Treatment_SurfaceBased(INTERVAL,CCI,PM_point,CM_point,RM_po
int,RC_point);
    %treatment improves life
    [tDN1,tPM1,tCM1,tRM1,tRC1,~] =
EffectvenessAgeCounter(i,treatment_surf,tDN,tPM,tCM,tRM,tRC
,0);

    [S_Age,St_Age,CCI,cf1,cost1]=Treatment_cf_improvement(cf,S_
Age,St_Age,cost,i,DR,tRM,tCM,tPM,CASE,treatment_surf,CCI);
    %Collect all necessary data

```

```

All_t_counters(i,:)=[tDN,tPM,tCM,tRM,tRC];
tDN=tDN1;
tPM=tPM1;
tCM=tCM1;
tRM=tRM1;
tRC=tRC1;
CCI_avg=(CCI1+CCI)/2;
All_CCI(i,:)=[CCI1,CCI,CCI_avg];
All_treatment(i,1)=treatment_surf;
cf_old=cf1;
All_cf(i,:)=[cf,cf1,cf_avg];
All_S_Age(i,:)=S_Age;
All_St_Age(i,1)=St_Age;
All_Cost(i,1)=cost1;
cost=cost1;

p_point=PM_point1;
c_point=CM_point1;
r_point=RM_point1;
rc_point=RC_point1;

```

end

- ***CCI based decision making code***

```

function
[treatment_surf,PM_point1,CM_point1,RM_point1,RC_point1] =
Treatment_SurfaceBased(INTERVAL,CCI,PM_point,CM_point,RM_point,RC_point)

%%%%%%%%%%Choose interval%%%%%%%%%%
if INTERVAL == "YES"
    PM_interval=4;
    CM_interval=7;
    RM_interval=12;
    RC_interval=20;
elseif INTERVAL == "NO"
    PM_interval=0;
    CM_interval=0;
    RM_interval=0;
    RC_interval=0;
end

```

```

%select treatments based on CCI value
if CCI<40
    if RC_point==0
        treatment_surf=5;
    else
        ERROR=1
        treatment_surf=5;
    end

elseif CCI<55 && CCI>=40
    if RM_point==0
        treatment_surf=4;
    elseif RC_point==0
        treatment_surf=5;
    else
        ERROR=1
        treatment_surf=1;
    end
elseif CCI<73 && CCI>=55
    if CM_point==0
        treatment_surf=3;
    elseif RM_point==0
        treatment_surf=4;
    elseif RC_point==0
        treatment_surf=5;
    else
        ERROR=1
        treatment_surf=1;
    end

elseif CCI<=87 && CCI>=73
    if PM_point==0
        treatment_surf=2;
    else ERROR=1
        treatment_surf=1;
    end
elseif CCI>87
    treatment_surf=1;
end

RC_point1=RC_point;

```

```

RM_point1=RM_point;
CM_point1=CM_point;
PM_point1=PM_point;

#####change treatment interval points after final
treatment is finalized#####
if treatment_surf==5
    RC_point1=RC_interval;
elseif treatment_surf==4
    RM_point1= RM_interval;
elseif treatment_surf==3
    CM_point1=CM_interval;
elseif treatment_surf==2
    PM_point1=PM_interval;
end

end

```

- ***Effectiveness Age Counter code***

```

function [tDN1,tPM1,tCM1,tRM1,tRC1,RC_finder] =
EffectivenessAgeCounter(i,treatment1,tDN,tPM,tCM,tRM,tRC,R
C_finder)
%EffectiveAgeCounters keeps on increasing every year unless
treatments are applied
tRC1=tRC+1;
tRM1=tRM+1;
tCM1=tCM+1;
tPM1=tPM+1;
tDN1=tDN+1;
if treatment1==5 %RC resets everything to 0
    RC_finder=i-1;
    tRC1=0; tRM1=0; tCM1=0; tPM1=0; tDN1=0;
elseif treatment1==4 %RM resets PM and CM to 2
    tCM1=2; tPM1=2; tDN1=0;
elseif treatment1==3 %CM resets PM to 2
    tPM1=2; tDN1=0;
end
end

```

- **Calculating equivalent age based on CCI code**

```
function [S_age_nextyear] =
getDeteriorated_Agenextyear_c2(B0,B1,B2,cf,CCI)

A=(exp((log(100-CCI)-B0)./(B1+(B2./(cf).^3)))));
%GET A=Age, add +1 for next years
S_age_nextyear=A+1
End
```

- **CF improvement based on treatments applied code**

```
function
[S_Age,St_Age,CCI,cf1,cost1,timefactor]=Treatment_cf_impr
ovement(cf,S_Age,St_Age,cost,i,DR,tRM,tCM,tPM,CASE,treatm
ent_surf,CCI)

EffectivenessAgePM=tPM;
EffectivenessAgeCM=tCM;
EffectivenessAgeRM=tRM;

%When treatment is DN
if treatment_surf ==1

[Life_remaining_DN,~,DN_cf,timefactor]=getLifel(cf,St_Age
,DR,1,EffectivenessAgePM,EffectivenessAgeCM,Effectiveness
AgeRM,CASE);
    LIFE(i,1)=Life_remaining_DN ;
    cost1 = cost ;
    St_Age=max(St_Age,0);
    cf1= DN_cf;
    CCI=CCI;
    S_Age=S_Age;
%When treatment is PM
elseif treatment_surf ==2

[Life_remaining_PM,~,PM_cf,timefactor]=getLifel(cf,St_Age
,DR,2,EffectivenessAgePM,EffectivenessAgeCM,Effectiveness
AgeRM,CASE);
    LIFE(i,1)= Life_remaining_PM;
    cost1 = cost + 25000;
    St_Age=max(St_Age,0);
```

```

        cf1= PM_cf;
        CCI=100;
        S_Age=1;

%When treatment is CM
elseif treatment_surf ==3

[Life_remaining_CM,~,CM_cf,timefactor]=getLifel(cf,St_Age
,DR,3,EffectivenessAgePM,EffectivenessAgeCM,Effectiveness
AgeRM,CASE);
    LIFE(i,1)= Life_remaining_CM;
    cost1 = cost + 90000;
    St_Age=max(St_Age-2,0);
    cf1= CM_cf;
    CCI=100;
    S_Age=1;

%When treatment is RM
elseif treatment_surf ==4

[Life_remaining_RM,~,RM_cf,timefactor]=getLifel(cf,St_Age
,DR,4,EffectivenessAgePM,EffectivenessAgeCM,Effectiveness
AgeRM,CASE);
    LIFE(i,1)= Life_remaining_RM;
    cost1 = cost + 170000;
    St_Age=max(St_Age-5,0);
    cf1= RM_cf;
    CCI=100;
    S_Age=1;

%When treatment is RC
elseif treatment_surf ==5

[Life_remaining_RC,~,RC_cf]=getLifel(cf,St_Age,DR,5,Effec
tivenessAgePM,EffectivenessAgeCM,EffectivenessAgeRM,CASE)
;
    LIFE(i,1)= Life_remaining_RC ;
    cost1 = 500000;
    St_Age=0;
    cf1= RC_cf;
    CCI=100;

```

```

        S_Age=1;

end

end

```

- **Structural condition deterioration code**

```

function [CF,r0] =
Deterioration_Curve(DesignLife,t1,t2,t3,alpha1,alpha2,alpha
3)

t = 0:20;

M = [1 0 0 t1-alpha1*t1;...
     -1 1 0 alpha1*t2-alpha2*t2;...
     0 -1 1 alpha2*t3-alpha3*t3;...
     0 0 1 -alpha3*DesignLife];

x = [1;0;0;0];

b = M\x;
b1 = b(1); b2 = b(2); b3 = b(3); r0 = b(4);

CF0 = 1-r0*t;
CF1 = b1-r0*alpha1*t;
CF2 = b2-r0*alpha2*t;
CF3 = b3-r0*alpha3*t;

CF = CF0;
CF(t>t1) = CF1(t>t1);
CF(t>t2) = CF2(t>t2);
CF(t>t3) = CF3(t>t3);

```

Structural Condition Based Decision Making:

- **Main simulation code**

```

clear
cf_old=1;

```

```

S_Age=1;
alpha=1.35;
[CF,r0,M,~,b] =
Deterioration_Curve(20,5,10,15,alpha,alpha^2,alpha^3);
N=150;
p_point=0;
c_point=0;
r_point=0;

CF1(1,1)=1;

for t=1:300
    %      DR(t,1) = 0.025;
    DR(t,1) = round(DeteriorationRate(r0,t,alpha),3);
    CF1(t,1)=cf_old-DR(t,1) ;
    cf_old=CF1(t,1);
end

all_S_Age(1,1)=S_Age;
cost=500000;

CF1_reduced_DN1=zeros(50,N);
CF1_reduced_PM1=zeros(50,N);
CF1_reduced_CM1=zeros(50,N);
CF1_reduced_RM1=zeros(50,N);
CF1_reduced_RC1=zeros(50,N);

cf_old=1;
all_treatment=[0];
treatment1=0;
RC_counter=0;RM_counter=0;CM_counter=0;PM_counter=0;
tRC=0;tRM=0;tCM=0;tPM=0;tDN=0;
RC_finder=0;
RCcount=0;
CASE=4;
S_age=1;
Surf_age=1;

B0= 0.90065;
B1= 0.88212;
B2= 0.16766;

INTERVAL="NO"

```

```

for i=1:N

    %%%%%%%%% Calculating DR per year %%%%%%%%%
    DR_peryear=DR(S_Age);
    %%%%%%%%% Calculating Cost %%%%%%%%%
    if treatment1==5
        cost = 500000;
    else
    end

    %%%%%%%%%
    %%%%%%%%%
    all_DR_peryear(i,1)=DR_peryear;
    cf=cf_old-DR_peryear;

    % for tracking CCI
    cfavg=(cf_old+cf)/2;
    CCI=CCItracker(cfavg, Surf_age);

Surf_age=getDeteriorated_Agenextyear_c2(B0,B1,B2,cfavg,CCI)
;

    % if cf>0.5
    PM_point=max(p_point-1,0);
    CM_point=max(c_point-1,0);
    RM_point=max(r_point-1,0);
    POINTS(i,:)=[PM_point,CM_point,RM_point];

    if i==1
        tRC1=0;tRM1=0;tCM1=0;tPM1=0;tDN1=0;
    else
        [tDN1,tPM1,tCM1,tRM1,tRC1,RC_finder] =
EffectvenessAgeCounter(i,treatment1,tDN,tPM,tCM,tRM,tRC,RC_
finder);
    end

    k(i,1)=RC_finder;
    %assign Treatment code

[CCI1,Surf_age,Cost_Coll,totalLife,All_S_Age,treatment1,Lif
e,Lifel,LbC,CbL,S_Age1,cf1,cost1,CF1_reduced_DN,CF1_reduced
_PM,CF1_reduced_CM,CF1_reduced_RM,CF1_reduced_RC,p_point,c_

```

```

point,r_point,timefactor_PM,timefactor_CM,timefactor_RM]=Ge
t_treatment_NonLinear_Deterioration_Repeating(INTERVAL,CCI,
cf,S_Age,cost,i,DR,PM_point,CM_point,RM_point,RC_finder,tRM
,tCM,tPM,CASE,Surf_age);

```

```

%Collect all data

```

```

    CCIavg=(CCI+CCI1)/2;
    all_CCI(i,:) = [CCI,CCI1,CCIavg];
    all_totalLife(i,:)=totalLife;
    all_cost(i,1)=cost1;
    all_cost_coll(i,:)=Cost_Coll;
    all_cf(i,:)=cf,cf1,cfavg];
    all_treatment(i,1)=treatment1;
    all_S_Age(i+1,1)=S_Age1;
    all_Life(i,:)=Life;
    all_Life1(i,:)=Life1;
    all_S_Age1(i,:)=All_S_Age;
    all_LbC(i,:)=LbC;
    all_CbL(i,:)=CbL;
    all_cfavgl(i,:)=cfavg;
    all_surf(i,:)=Surf_age;
    TimeFactor_PM(i,:)=timefactor_PM;
    TimeFactor_CM(i,:)=timefactor_CM;
    TimeFactor_RM(i,:)=timefactor_RM;
    t(i,1)=i;
    cf_old=cf1;
    S_Age=S_Age1;
    cost=cost1;
    l1=length(CF1_reduced_DN);
    l2=length(CF1_reduced_PM);
    l3=length(CF1_reduced_CM);
    l4=length(CF1_reduced_RM);
    l5=length(CF1_reduced_RC);
    CF1_reduced_DN1(1:l1,i)=CF1_reduced_DN;
    CF1_reduced_PM1(1:l2,i)=CF1_reduced_PM;
    CF1_reduced_CM1(1:l3,i)=CF1_reduced_CM;
    CF1_reduced_RM1(1:l4,i)=CF1_reduced_RM;
    CF1_reduced_RC1(1:l5,i)=CF1_reduced_RC;

    tDN=tDN1;
    tPM=tPM1;
    tCM=tCM1;

```

```
tRM=tRM1;  
tRC=tRC1;  
  
alltPM(i,:)=[tDN1,tPM1,tCM1,tRM1,tRC1];  
end
```