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Network Level Pavement Structural Testing With the Traffic Speed Deflectometer

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Abstract:

This report describes research conducted to incorporate pavement structural condition information obtained from the Traffic Speed Deflectometer (TSD) into the Virginia Department of Transportation (VDOT) pavement management system decision making process for bituminous pavement sections. Testing was conducted on a 4,000-mile (1,500 interstate miles and 2,500 primary roads miles) subset of the VDOT network. The report showed that the pavement structural condition, as measured by the TSD, has an impact on the rate of deterioration of the pavement surface. In addition, for the set of collected data, the consistency between the TSD and Falling Weight Deflectometer (FWD) in identifying the same weak sections was found to be higher than the consistency between repeated sets of FWD measurements performed in the Bristol district. The consistency was defined as the percentage of structurally weak sections identified by both devices as a proportion of the number of weak sections. Also, the distribution of the effective structural number (SN_{eff}) calculated from the TSD measurements on interstate roads was found to be similar to that obtained from the FWD measurements. The relatively good consistency between the TSD and FWD SN_{eff} and the similarities between the SN_{eff} distributions suggest that the structural information derived from the TSD can be successfully used as an alternative to similar data derived from the FWD for VDOT network level pavement management applications.

The resilient modulus (M_R) based on FWD testing is a metric currently used by VDOT to characterize the subgrade strength. A number of TSD-based indices have been proposed in the literature to replace the FWD-based M_R. In this study, all indices investigated that could be used to replace the FWD-based M_R were also found to be highly correlated to the overall TSD-based structural properties of the pavement and not very highly correlated to the FWD based M_R. Thus, adding a TSD-based measure of the subgrade strength was not recommended at this time. Although the reasons for this lack of correlation between TSD-based and FWD-based subgrade strength measurements are not clear, they likely include unquantified differences in subgrade moisture conditions between measurements from the two devices and also possible limitations of the TSD technology in capturing very small deflections far away from the load application.

An augmented structural condition matrix was used to investigate the effects of incorporating the TSD-based structural condition on the annual mix of pavement rehabilitation treatments recommended and on the resulting average maintenance cost per mile on interstate roads. The approach did not account for the traffic level and pavement age as currently used by VDOT. The treatment categories considered by VDOT are Do Nothing, Preventive Maintenance, Corrective Maintenance, Restorative Maintenance, and Reconstruction. The augmented matrix modifies these treatments based on whether the structural condition is Strong, Fair, or Weak. In general, applying the augmented matrix on the tested interstate network reduced the percentage of the network recommended for Corrective Maintenance and increased the recommended percentages of the other treatments, mainly Preventive Maintenance and Restorative Maintenance, and to a lesser extent the percentages recommended for Do Nothing or Reconstruction.

FINAL REPORT

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

ABSTRACT

This report describes research conducted to incorporate pavement structural condition information obtained from the Traffic Speed Deflectometer (TSD) into the Virginia Department of Transportation (VDOT) pavement management system decision making process for bituminous pavement sections. Testing was conducted on a 4,000-mile (1,500 interstate miles and 2,500 primary roads miles) subset of the VDOT network. The report showed that the pavement structural condition, as measured by the TSD, has an impact on the rate of deterioration of the pavement surface. In addition, for the set of collected data, the consistency between the TSD and Falling Weight Deflectometer (FWD) in identifying the same weak sections was found to be higher than the consistency between repeated sets of FWD measurements performed in the Bristol district. The consistency was defined as the percentage of structurally weak sections identified by both devices as a proportion of the number of weak sections. Also, the distribution of the effective structural number (SN_{eff}) calculated from the TSD measurements on interstate roads was found to be similar to that obtained from the FWD measurements. The relatively good consistency between the TSD and FWD SN_{eff} and the similarities between the SN_{eff} distributions suggest that the structural information derived from the TSD can be successfully used as an alternative to similar data derived from the FWD for VDOT network level pavement management applications.

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INTRODUCTION

Every year, the Virginia Department of Transportation (VDOT), Maintenance Division publishes the *State of the Pavement* report, which summarizes the surface condition of the interstate, primary, and secondary VDOT roadway network, consisting of more than 128,000 lane miles (VDOT, 2018). These condition data are at the core of the following four primary pavement management activities:

1. **Pavement Needs Analysis:** Maintenance and rehabilitation needs are determined from the collected data and used for the development of the biennial maintenance budget and as a maintenance strategy guide 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 asset conditions and asset management practices of state highways.
4. **Federal Highway Performance Monitoring System Reporting:** The Highway Performance Monitoring System report submitted by VDOT to the Federal Highway Administration is the basis for the federal 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 alone is not enough to determine the appropriate pavement section treatment (VDOT, 2018).

Pavement structural condition is another important aspect needed to make better treatment decisions. While VDOT collected network level structural condition data on most of its interstate system between 2006 and 2008, such an effort has not been undertaken since then. This is because the Falling Weight Deflectomer (FWD), which was used for data collection and is the predominant device used for structural evaluation of pavements, is a stationary device that requires traffic control which can lead to issues in areas of high traffic volumes. The long time required to conduct the testing and the potential for risk are the main reasons that VDOT has not systematically updated FWD data on its interstate system. The emergence of new structural evaluation devices that operate at or near the roadway traffic speed has made network level pavement structural evaluation an achievable objective. This report documents the collection of network level structural condition data with the Traffic Speed Deflectometer (TSD) on 1,500 directional miles of interstate highways and 2,500 directional miles of primary roadways.

PURPOSE AND SCOPE

The objective of this project was to determine whether the structural condition information collected with the TSD on bituminous pavement sections was appropriate to use in the VDOT pavement management system (PMS). The TSD-based data was used to augment pavement treatment selection in a manner similar to the current VDOT procedure using FWD-based structural condition data on interstate roads. The potential reasons for VDOT to begin using TSD-based data include: 1) updating the structural information currently used on 1,500 miles of interstate roads, 2) providing structural information for 2,500 miles of primary roads so that an augmented decision making process similar to that used on interstate roads can be implemented, and 3) providing the appropriate information for VDOT to decide whether to fully implement the TSD-based pavement structural condition as part of the PMS decision making process.

Data used in this project was collected on 4,020 (directional) miles of Virginia's interstate and primary network. The data consists of pavement structural condition data, pavement layer thicknesses, and pavement surface condition data. The analysis presented in this report is limited to flexible pavement sections and excludes rigid or composite pavement sections.

METHODS

The following describes the methods used to complete the work. These include identifying the structural indicators that can be used based on continuous deflection measurements, collecting the required data from selected portions of VDOTs pavement network, processing the collected data, analyzing and interpreting the data, comparing newly collected structural data with existing structural data, and identifying ways the new data can be incorporated into VDOTs decision making processes.

Structural Indicators from Continuous Deflection Measurements

VDOT currently uses the effective structural number (SN_{eff}) and the subgrade resilient modulus (M_R) as structural condition indicators for the interstate network in the PMS. The TSD is a relatively new device that is advocated as an alternative to the FWD for network level pavement structural evaluation. Although the two devices broadly give the same indication of the pavement structural health (Flintsch et al., 2013, Katicha et al., 2014, Muller 2015, Chai et al., 2016, Březina et al., 2017, Katicha et al., 2017c), there are fundamental differences in terms of loading (impulse for FWD and rolling for TSD) and response measurements (peak response for FWD and instantaneous response for TSD) (see Jansen, 2017). These differences make it difficult to seamlessly transition from the FWD to the TSD without providing guidance on how to interpret TSD measurements based on sound engineering principles. Providing such guidance was one of the objectives of a Federal Highway Administration sponsored research projects (Rada et al., 2016). In that project, the relationship of 77 indices that can be calculated from TSD measurements was evaluated with the tensile strain at the bottom of the asphalt layer and the compressive strain at the top of the subgrade. These indices can be calculated using the following equations:

$$R1_r = \frac{r^2}{2D_0\left(1 - {}^{D_r}/_{D_0}\right)} \tag{1}$$

$$R2_r = \frac{r^2}{2D_0\left({}^{D_0}/_{D_r} - 1\right)} \tag{2}$$

$$F1 = \frac{D_0 - D_{24}}{D_{12}} \tag{3}$$

$$F2 = \frac{D_{12} - D_{36}}{D_{24}} \tag{4}$$

$$SCI_r = D_0 - D_r \tag{5}$$

$$SCIm_r = D_{\max} - D_r \tag{6}$$

$$DSI_{s-r} = D_s - D_r \tag{7}$$

$$SD_r = \frac{\tan^{-1}(D_0 - D_r)}{r} \tag{8}$$

$$AUPP = \frac{5D_0 - 2D_{12} - 2D_{24} + D_{36}}{2} \tag{9}$$

$$TS_r = \frac{dD}{dr} \tag{10}$$

Where
r, s = distance from applied load in inches ($s > r$)
D_x = deflection at distance x from the load
d = differential operator.

Some of the indices given in the equations above are well known. For example, the surface curvature index (SCI) is most often defined as the difference between the deflection under the load and the deflection 12 inches (or 300 mm) from the applied load. This corresponds to SCI_{12} and is often denoted by SCI300, the notation that will be used in this report. Another well-known indicator is the base damage index defined as the deflection 12 inches from the applied load minus the deflection 24 inches from the applied load. This would correspond to $DSI_{12\text{-}24}$ (as defined in Equation 7, the DSI is the deflection slope index). From the 77 indicators investigated (generated from the above equations by changing r and s) Rada et al. (2016) identified the most appropriate indices for the maximum horizontal tensile strain at the bottom of the asphalt layer (Table 1) and maximum vertical strain at the top of the subgrade layer (Table 2).

The R^2 values for the indices were obtained by modeling a wide range of simulated pavement cross-sections with ranging material properties. In both tables, R^2 values of the listed indices are relatively close and therefore any of the listed indices would seem to be appropriate for network level structural evaluation. Therefore, the SCI300 was selected from the first list of indicators related to the maximum horizontal strain at the bottom of the asphalt layer for evaluation because of its wide use (Rada et al., 2016). For the vertical compressive strain at the top of the subgrade layer, all possible DSI indices that could be calculated from the reported TSD deflections (e.g., D_0, D_8, D_{12}, D_{18}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72}) were evaluated. These include all DSI values listed in Table 2.

Table 1. Most Appropriate Indices Using Traffic Speed Deflectometer Data Related to Maximum Horizontal Strain at Bottom of Asphalt Layer (from Rada et al., 2016)

Best Indices with TSD Loading	Index	R^2
R1[a]	$R1_{12}$	0.94
	$R1_{18}$	0.92
R2[b]	$R2_{18}$	0.92
	$R2_{24}$	0.94
	$R2_{36}$	0.90
SCI[c]	SCI_{12} = SCI300	0.94
	SCI_{18}	0.92
	$SCIm_{12}$	0.92
	$SCIm_{18}$	0.91
DSI[d]	$DSI_{4–8}$	0.90
	$DSI_{4–12}$	0.91
	$DSI_{4–18}$	0.90
SD[e]	SD_{12}	0.93
	SD_{18}	0.92
TS[f]	TS_8	0.93
	TS_{24}	0.91
AUPP[g]		0.90

a: radius of curvature 1; b: radius of curvature 2; c: surface curvature index; d: deflection slope index; e: slope of deflection; f: tangent slope; g: area under pavement profile

Table 2. Most Appropriate Indices Using the Traffic Speed Deflectometer Data Related to Maximum Vertical Strain at Top of Subgrade (from Rada et al., 2016)

Best Indices with TSD Loading	Index	R^2
R2[a]	$R2_{60}$	0.92
DSI[b]	DSI_{4-48}	0.90
	DSI_{4-60}	0.90
	DSI_{8-23}	0.92
	DSI_{8-48}	0.93
	DSI_{8-60}	0.93
	DSI_{12-18}	0.90
	DSI_{12-24}	0.94
	DSI_{12-36}	0.95
	DSI_{12-48}	0.95
	DSI_{12-60}	0.95
	DSI_{18-24}	0.97
	DSI_{18-36}	0.97
	DSI_{18-48}	0.97
	DSI_{18-60}	0.97
	DSI_{24-36}	0.97
	DSI_{24-48}	0.97
	DSI_{24-60}	0.97
TS[c]	TS_{12}	0.90
	TS_{18}	0.92
	TS_{36}	0.95
F2[d]	F_2	0.91

a: radius of curvature 2; b: deflection slope index; c: tangent slope; d: shape factor 2

The work of Rada et al. (2016) anticipates the move of state highway agencies to mechanistic methods for the design and rehabilitation of pavements. This is reflected by the choice of mechanistic strain criteria for the evaluation of the indices. However, highway agencies including VDOT still use the SN_{eff} to characterize the structural condition of their pavements at the network level. Therefore, this study also compared the SN_{eff} calculated from TSD measurements to the SN_{eff} VDOT currently uses which was calculated from FWD measurements. For TSD-based SN_{eff} calculations, we used the method recommended by Nasimifar et al. (2019), which is based on Rohde's approach to calculate SN_{eff} from FWD measurements (Rohde, 1994) but modified to account for differences between the TSD and FWD. One of the most important differences is how the two devices record the pavement response. Rohde's equation to calculate SN_{eff} is given as follows:

$$SN_{eff} = k_1 SIP^{k_2} H_p^{k_3} \tag{11a}$$

Where
H_P = total pavement thickness (mm)

SIP = structural index of pavement, calculated as follows
$SIP = d_0 - d_{1.5Hp}$.

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 these coefficients be adjusted to 0.4369, -0.4768, and 0.8182 for measurements obtained with the TSD.

The method used by VDOT to calculate the subgrade modulus is given as the following:

$$M_R = E_{subgrade} = \frac{(1-\mu^2)P}{\pi r D_{36}} \quad (11b)$$

Where
P = applied load
D_{36} = deflection 36 inches from the applied load
μ = Poisson's ratio
r = distance from applied load (36 inches) where deflection is measured.

Collected Data

TSD data was collected using the Australian Road Research Board **i**ntelligent **P**avement **A**ssessment **Ve**hicle (iPAVe). The data was collected at 10 m intervals in August and September 2017. A verification road was tested at the beginning and end of the data collection to verify that the device's measurements were repeatable. The results of this verification are shown in Figure 1. 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 were mounted at a small angle from the vertical. This allows horizontal speed (i.e., traveling speed of the vehicle) and vertical speed (i.e., pavement deflection speed) measurements. 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 was calculated mathematically by integration. The Doppler laser measurements were obtained at distances of 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. The axle load was 20,000 lbs and a strain gauge mounted on the rear axle that measured the bending moment to determine the load applied on each side of the axle (because of dynamic effects, the load will not be perfectly distributed to each side).

Pavement layer thicknesses in this report were obtained in two ways. For the interstate roads, the layer thicknesses were obtained from existing data in the VDOT PMS. For the primary roads, ground penetrating radar (GPR) data collected by Infrasense, Inc. using a GSSI SIR-30 GPR system and a Model 4108 1.0 GHz horn antenna was used. The GPR data was verified and calibrated using physical measurements from 260 cores distributed along the tested primary roads (see Figure 2). In instances where the difference between the asphalt core thickness and asphalt GPR thickness was greater than 1.5 inches, a detailed review of the core data and the GPR data was performed to reconcile the difference. This review revealed whether the issue was with the core data (e.g., a broken core with portion remaining in the hole) or whether the issue was with the GPR data needing recalibration. Figure 3 shows a good correlation between the calibrated GPR thickness and the measured core thickness.

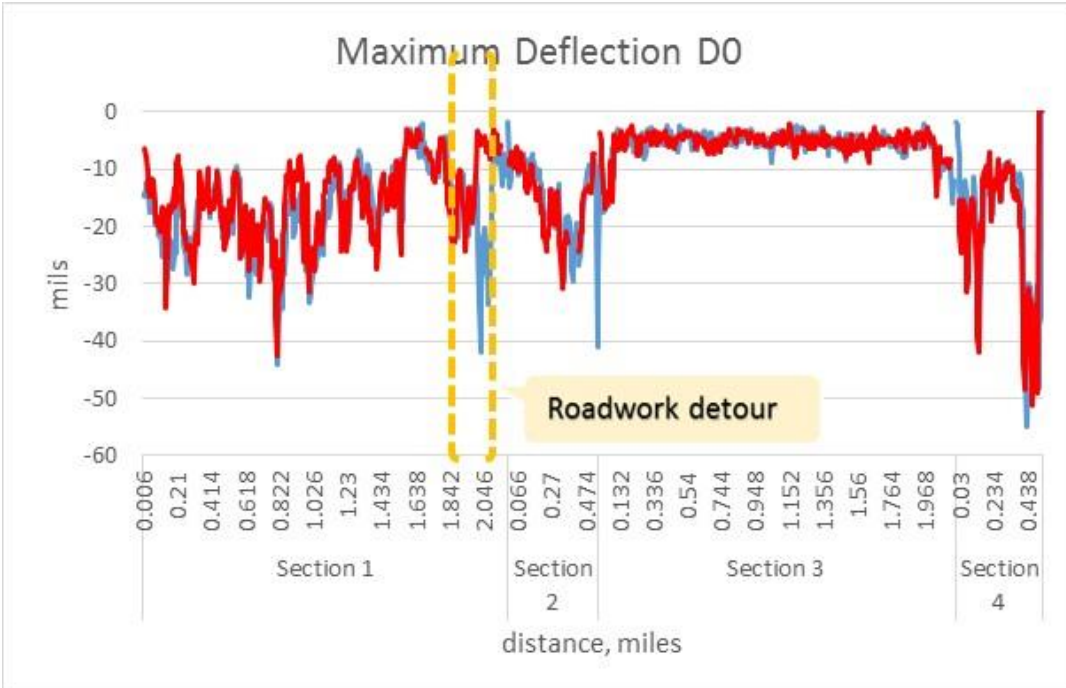


Figure 1. Pre- and Post-Survey Testing on Validation Loop

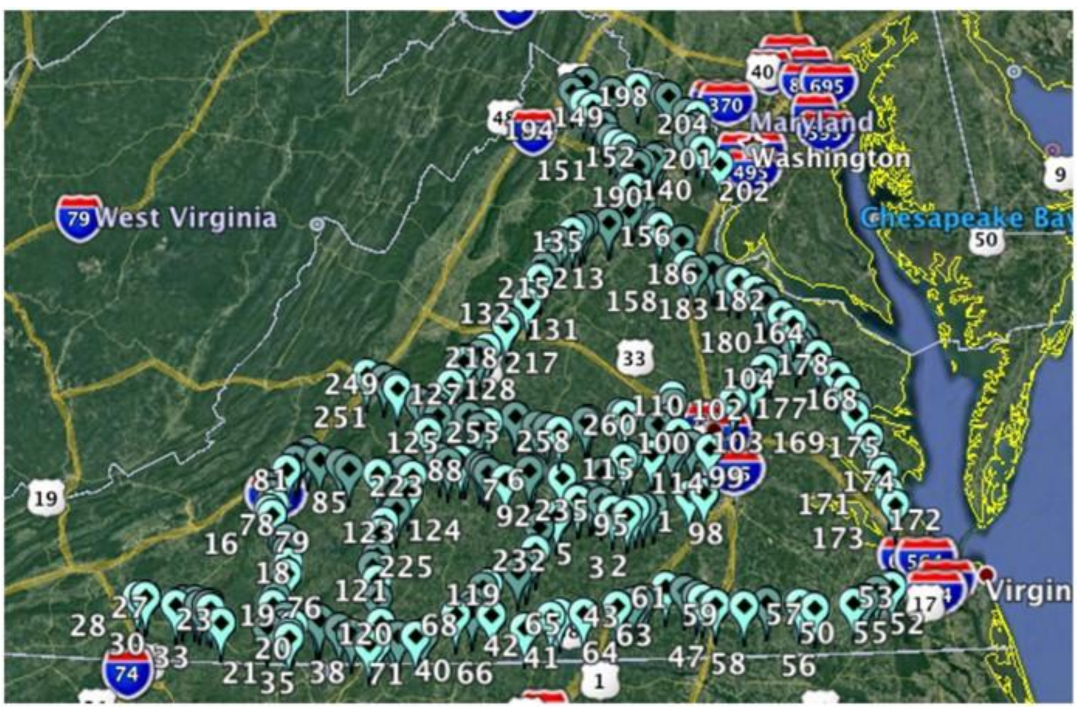


Figure 2. Location of 260 Cores on Tested Primary Roads

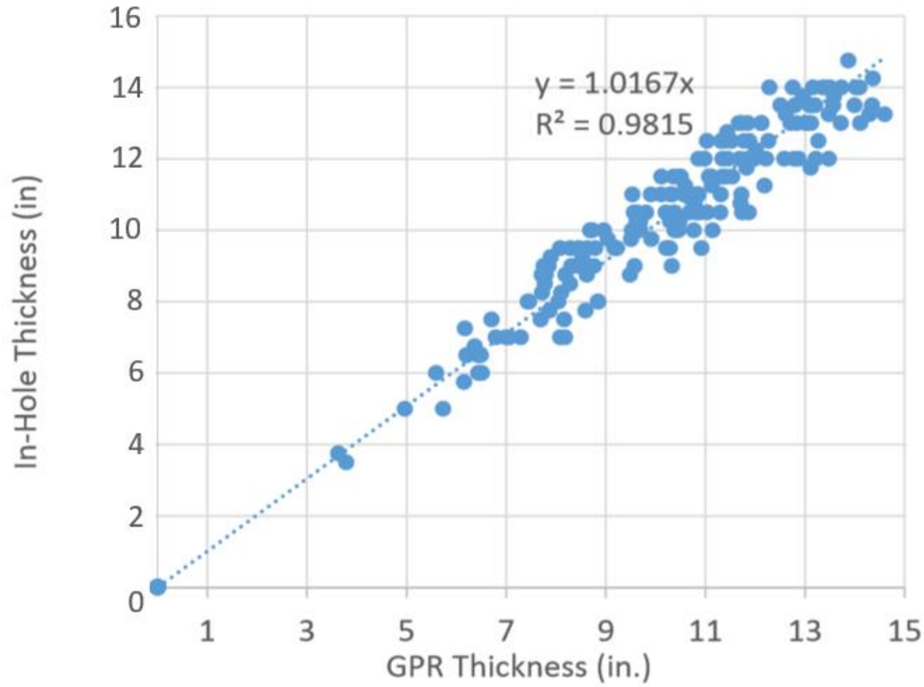


Figure 3. Correlation Between Cores and GPR Data

The pavement surface condition data was obtained from the iPAVe and from 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 obtain the pavement surface condition at a 0.1 miles resolution. This data was used to evaluate the effect the pavement structural condition has on the pavement deterioration and to augment VDOT's current treatment selection process using structural condition data. The VDOT PMS data was used instead of the iPAVe data for the following two reasons:

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

All collected data (TSD, GPR, and surface condition) were synchronized using GPS coordinates. The tested roads are shown in Figure 4.

The Australian Road Research Board provided comprehensive deflection testing data, including raw measurements, calculated deflections, and calculated structural indices such as SCI300. Additional processing was needed to perform temperature correction and to calculate the effective structural number (SN_{eff}).

(a)

(b)

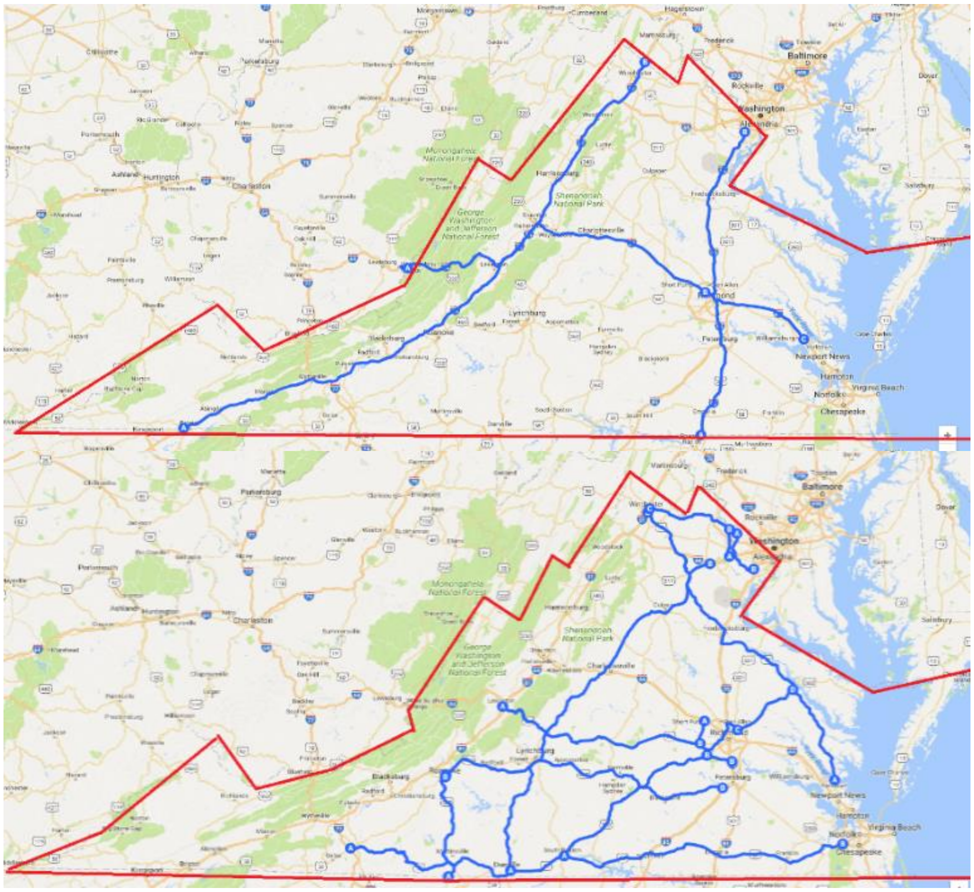


Figure 4. Routes Assessed with Traffic Speed Deflectometer: (a) Interstate; (b) Primary

Data Processing

Temperature correction was performed for SCI300 and the deflection under the applied load, D_0. The temperature correction for SCI300 was performed using the approach developed by Nasimifar et al. (2018). The approach takes into account the loading specific to a moving device and the viscoelastic effects, and is inspired from the Lukanen et al. (2000) method for FWD measurements. The temperature correction factor is calculated as follows:

$$\lambda = \frac{SCI_{Ref}}{SCI_T} = \frac{10^{-0.05014T_{Ref}+0.019049T_{Ref}\log(h_{AC})\log(\varphi)}}{10^{-0.05014T+0.019049T\log(h_{AC})\log(\varphi)}} \tag{12}$$

Where
λ = Temperature Adjustment Factor
SCI_{Ref} = Adjusted SCI300 at reference temperature
T_{Ref} = Reference temperature in °C
h_{AC} = Asphalt layer thickness, mm
T = Mid-depth asphalt concrete layer temperature at time of measurement in °C
φ = Latitude of location of measurement (within 30 to 50 degrees).

The temperature at the mid-depth of the asphalt concrete layer was estimated using BELLS3 equation shown in Equation 2 (Lukanen et al., 2000) as follows:

$$T_d = 0.95 + 0.892IR + \{\log(d) - 1.25\}\{-0.448IR + 0.621T_p + 1.83\sin(hr_{18} - 15.5) + 0.042IR\sin(hr_{18} - 13.5)\} \quad (13)$$

Where
T_d = Pavement temperature at depth d, ºC
IR = Pavement surface temperature, ºC
log = Base 10 logarithm
d = Depth at which temperature is to be predicted, mm
T_p = Average air temperature the day before testing, ºC
sin = sine function on an 18-hr clock system, with 2π radians equal to one 18-hr cycle
hr_{18}= Time of the day in a 24-hr clock system but calculated using an 18-hr asphalt concrete temperature rise and fall time cycle.

Using National Oceanic and Atmospheric land-based weather station data, the average temperature on the day before testing was obtained. The temperature adjustment factor for D_0 was calculated based on the American Association of State Highway and Transportation Officials temperature adjustment charts (1993).

Data Analysis and Interpretation

The correlation between SCI300 and cracking and SCI300 and rutting was calculated for all tested roads to show that the structural condition influenced the rate of deterioration of the pavement surface. 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 the PMS surface condition data could not be obtained). The deterioration models were developed using quasi-Poisson regression, as that method was found to adequately represent the VDOT PMS data's statistical characteristics (see Katicha et al., 2017a; Katicha et al., 2017b; Pantuso et al., 2019).

Pavement Condition Data

The tested roads' pavement condition data was obtained from the VDOT PMS. VDOT summarizes the condition of the pavement in the Critical Condition Index (CCI), an index ranging from 0 to 100, with a rating of 100 representing no distresses (new surface) and a rating of below 60 considered deficient. The approach VDOT follows to calculate the CCI is to first consider the load-related distress rating (LDR) (which is influenced by alligator cracking, wheel path patching, and rutting) and the non-LDR (NDR; which is influenced by longitudinal cracking, transverse cracking, non-wheel path patching, and bleeding) as two separate indices. These two indices also range from 0 to 100, and are calculated as follows:

$$LDR = 100 - D_{AC} - D_{WP} - D_R \quad (14)$$

$$NDR = 100 - D_{LC} - D_{TC} - D_B - D_{NP} \quad (15)$$

$$CCI = \min\{NDR, LDR\} \tag{16}$$

Where
D_{AC} = Deduct value for alligator cracking
D_{WP} = Deduct value for wheel path patching
D_R = Deduct value for rutting
D_{LC} = Deduct value for linear cracking
D_{TC} = Deduct value for transverse (reflective) cracking
D_B = Deduct value for bleeding
D_{NP} = Deduct value for non-wheel path patching.

Deduct values are based on the extent of each distress and can be obtained from McGhee (2002). The CCI is determined from the LDR and NDR by taking the lower of the two values. The CCI, LDR, and NDR were obtained from the VDOT PMS for 0.1-mile sections. The date of last treatment for each pavement section was also obtained to determine the appropriate age of the pavement surface.

Regression Model for Pavement Deterioration

The effect of structural condition on pavement condition indices was evaluated by fitting a regression model using data at 0.1-mile section resolution (10 m structural evaluation data was averaged over the 0.1-mile length). Deterioration equations were developed for the LDR, NDR, and CCI, (shown for LDR and SCI300) as follows:

$$LDR = 100 - \exp\{\beta_0 + \beta_1 \log(Age) + \beta_2 \log(Age) \times SCI300\} \tag{17a}$$

$$LDR = 100 - \exp\{\beta_0 + \beta_1(1 + \beta_3 SCI300)\log(Age)\} \tag{17b}$$

Where
LDR = load related distress
Age = pavement age calculated as the difference between the year at which the LDR is observed minus the year of the last applied treatment recorded in the PMS *SCI*300 = surface curvature index β_0, β_1, β_2, and β_3 = regression coefficients with $\beta_3 = \beta_2 / \beta_1$.

The representation given in Equation 17a illustrates how the model behaves; the pavement deterioration is a function of age with the rate of deterioration depending on the structural condition as determined by $\beta_1(1 + \beta_3 SCI300)$.

To fit the model, the variable 100 – *LDR* was used to obtain the parameters of the exponential function (see 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 (as 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 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, making it harder to fit the model, as no standard fitting procedure is available in this case.

Comparison Between FWD and TSD Data on Interstate Roads

VDOT has implemented an enhanced decision process that considers interstate roads' pavement structural condition using FWD data with SN_{eff} and M_R as the two structural indicators. The FWD data was collected between 2006 and 2008 (making it 12 to 14 years old at the time of this writing). With the exception of limited sections, this data has not been updated, mainly because of the difficulties in using the FWD for network level structural evaluation (Flintsch et al., 2013). Since the TSD appears to be well suited to physically collect network level structural evaluation data that could be used as an alternative to the FWD for network level pavement management applications, it is important to evaluate to what extent the TSD data agrees with the FWD data. To accomplish this, a detailed comparison between the SN_{eff} calculated from the TSD and the SN_{eff} calculated from the FWD was performed. Furthermore, the correlation between indices calculated with the TSD data and the M_R calculated from the FWD data was evaluated. The comparison between TSD and FWD SN_{eff} was performed as follows:

1. Compare a scatter plot of TSD and FWD SN_{eff}
2. Compare the distribution of TSD and FWD SN_{eff}. Distributions of TSD and FWD SCI300 and D_0 were also compared.
3. Compare quantitatively if the SN_{eff} from the TSD and SN_{eff} from the FWD identify the same weak sections using a consistency test.

Because the FWD data was collected at 0.2-mile intervals, the comparison was performed at this aggregation level. The third evaluation, the consistency test, arose from the following question related to the TSD measurements: *Does the TSD identify the same weak sections as those identified by the FWD?*

This is the main concern in substituting the TSD for the FWD as a network level structural evaluation device. Note that although the FWD is often considered as a benchmark device, it is not a perfect device, and the same question can also be asked of the FWD: *Do successive measurements using the FWD identify the same weak sections?*

This second question is important because the answer is a benchmark to which the first question's answer can be compared. If the FWD does not perfectly identify the same weak sections from two sets of measurements, then it should be reasonable for the TSD to also not perfectly identify the same weak sections as those identified by the FWD. This should be true at least to the extent that the FWD does not identify the same sections from repeated measurements.

The two posed questions are qualitative; however, the consistency test can also be used to provide a quantitative answer. The underlying assumption of the consistency test is that the structurally weakest sections identified by one set of measurements should also be identified by another set of measurements (whether via the same device or another device). 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 SN_{eff} at section i) and by D_i (e.g., the FWD SN_{eff} 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:

$$I_1 = \{i: C_i \leq C_\alpha\} \tag{18}$$

$$I_2 = \{i: D_i \leq D_\alpha\} \tag{19}$$

Where C_α is the structural condition of the $\lfloor \alpha N \rfloor$ weakest section in the set C_i, and D_α is the structural condition of the $\lfloor \alpha N \rfloor$ weakest section in the set D_i (here $\lfloor \alpha N \rfloor$ 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.

$$T = I_i \cap I_2 = \{i: i \in I_1 \text{ and } i \in I_2\} \tag{20}$$

The consistency CT_α is then calculated as follows:

$$CT_\alpha = 100 \frac{\text{card}(T)}{\lfloor \alpha N \rfloor} \tag{21}$$

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_α = 100%.

The consistency test was performed between the TSD and FWD for α ranging from 1% to 100% at 1% increments. The results were compared with the consistency test performed by Katicha et al. (2017c) between two sets of the FWD collected in 2006 and 2011 on the interstate sections in the Bristol district (α of 5%, 10%, 15%, 20%, 25%, and 33%).

Incorporating Structural Condition Information into the PMS Decision Process

VDOT uses a set of pavement management decision matrices with distresses as inputs and recommended treatment activities as outputs. A CCI-based filter 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 (Figure 5). 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 rather provided an additional step that can be used when pavement structural condition is available. If structural information is unavailable, the decision process can revert to the core process already in place. VDOT currently uses the following five recommended treatment categories (shown here in order from lesser to heavier treatments): Do Nothing (DN), Preventive Maintenance (PM), Corrective Maintenance (CM), Restorative Maintenance (RM), and Reconstruction (RC). At the preliminary treatment stage, one of these five categories is recommended based on the condition index and the decision matrices. In the enhanced decision process, based on the structural condition (and traffic level and construction history), the selected preliminary treatment can be either retained or modified to a heavier or lesser treatment. A similar approach was investigated with the TSD data using SCI300 but without considering the traffic level and construction history. The results section shows the implications of the proposed approach on the selected treatment at the network level. The reason for not including the traffic level and the construction history is so that the results of the TSD and the FWD can be compared without the compounding effect of these additional two variables.

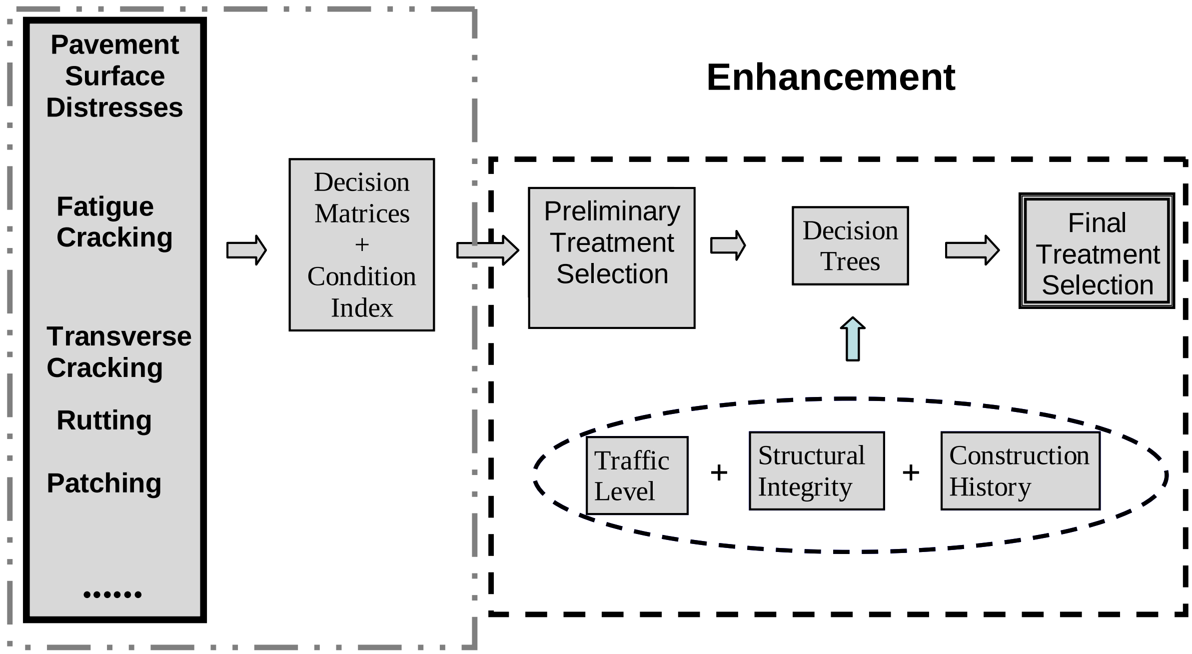


Figure 5. Enhanced Decision Process Framework Used by Virginia Department of Transportation when Adequate Structural Information from Falling Weight Deflectometer is Available

Sensitivity of PMS Enhanced Decision to the Selection of Structural Parameter and Measurement Accuracy

Rada et al. (2016) discussed several indices that can be used to evaluate TSD-based measurements. The study explicitly lists SCI300 and SN_{eff} as two possible structural condition indices to use for network level PMS applications. Either of the two indices would seem appropriate to use however, it is important to understand how the choice of index impacts the identification of weak sections. If the identification of weak sections is not significantly affected by the chosen index, then the choice of which index to use is not very critical. However, if the identification of weak sections is significantly affected by the choice of index (i.e., different indices mostly identify different weak sections), the choice of which index to use will have a significant impact on the PMS decision making process. For this purpose, the consistency in identifying the same weak sections, defined in Equation 21, and the Spearman rank correlation between TSD SCI300 and TSD SN_{eff} was evaluated. The Spearman rank correlation between SCI300 *and* SN_{eff} is the correlation calculated using the ranking of the measurements rather than the actual measurements. It is more appropriate than the regular (Pearson) correlation because SCI300 is linearly related to the measured deflections while the SN_{eff} is nonlinearly related to the measured deflections. This suggests that the relationship between SCI300 and SN_{eff} is nonlinear.

Another parameter that affects the PMS decision making process is the repeatability of TSD measurements. This study considered the measurements collected on the validation loop (shown in Figure 1) to evaluate the repeatability of TSD SCI300 measurements as follows:

$$R = \sqrt{\sum_{i}^{n} \frac{\left(SCI300_i^1 - SCI300_i^2\right)^2}{n}} \quad (22)$$

Where

R = repeatability
n = number of measurements
$SCI300_i^1$ = first set of measurements at location i
$SCI300_i^2$ = second set of measurements at location i.

The repeatability R is used to randomly simulate variations in the measured TSD SCI300 and calculate how this random variation affects the consistency of TSD SCI300.

RESULTS AND DISCUSSION

Correlation Between Structural Condition and Surface Condition

The correlation analysis between the structural and functional conditions was performed in two stages. At the first stage, the correlation between SCI300 and cracking, SCI300 and rutting, and cracking and rutting were calculated for each tested road to see whether there was a strong correlation between the structural condition and the surface condition. At the second stage, the average SCI300, average cracking, and average rutting were calculated for each tested road. From these average values, the correlation was then calculated for all roads.

Table 3 shows the pairwise correlations for each road. In general, within a tested road, the correlation between SCI300 and cracking was very weak. The average correlation for all roads was 0.06. In a few cases there was moderate correlation. For example, Route 28 south had the strongest correlation at 0.39. For the primary roads, most of the correlations between SCI300 were positive (16 out of 19), though weak. For the interstate roads, the correlation was even weaker, with three positive and three negative values. This suggests that the amount of cracking observed on a road is not a good indicator of that road's structural condition. The correlation between SCI300 and rutting was even weaker, with 13 positive and 12 negative values. The correlation between cracking and rutting was higher at an average of 0.20, with all calculated correlations being positive. This shows a definitive, albeit weak, link between cracking and rutting. This link is likely due to the fact that both rutting and cracking are positively correlated to the age of the pavement surface (i.e., older surfaces will tend to show higher rutting and cracking).

Table 3. Pairwise Correlation Between SCI300, Cracking, and Rutting

Road	Direction	SCI300 and Cracking	SCI300 and Rutting	Cracking and Rutting	Number of Data Points
Rt 7	East	0.21	0.14	0.19	6590
	West	0.09	-0.03	0.19	6624
Rt 17	North	0.08	-0.09	0.05	28694
	South	0.20	0.06	0.16	27953
Rt 28	North	0.01	0.11	0.41	2225
	South	0.39	0.49	0.60	1488
Rt 29	North	0.17	-0.03	0.01	29509
	South	0.14	-0.08	0.04	26966
Rt 58	East	-0.02	-0.02	0.34	29851
	West	0.00	-0.08	0.22	41402
Rt 60	East	0.14	0.09	0.26	18114
	West	0.02	0.03	0.27	18004
Rt 220	North	0.05	0.08	0.12	8196
	South	-0.12	-0.05	0.25	7921
Rt 288	North	-0.03	0.12	0.16	7840
	South	0.00	0.09	0.37	6894
Rt 360	East	-0.09	-0.04	0.15	18208
	West	0.04	-0.02	0.18	20321
Rt 460	East	0.22	0.11	0.14	23510
	West	0.14	0.08	0.26	24800
I-64	East	0.01	0.04	0.20	42163
	West	-0.03	-0.17	0.17	39164
I-81	North	-0.04	-0.02	0.21	39538
	South	0.03	-0.01	0.16	44680
I-95	North	-0.11	0.06	0.05	20086
	South	0.06	0.23	0.14	18876
Average Non-Interstate		0.09	0.05	0.22	
Average Interstate		-0.01	0.02	0.15	
Average All		0.06	0.04	0.20	

Figure 6 shows 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 does not fit well into the overall trend either, although the difference is not as clear as for Route 60. The correlation between average cracking and average SCI300 increases to 0.86 with Route 60 data removed, and increases to 0.89 if Route 28 data was also removed. Note that the high correlation found when data was averaged and all roads are considered is in contrast to the relatively low correlation within a road. The results obtained with the averages are potentially affected much more by other confounding factors than the results obtained within individual roads. For example, roads that are considered important (i.e., that carry more traffic) are often designed to be structurally strong. Furthermore, the surfaces of these roads are likely kept in better condition than roads carrying relatively less traffic. Therefore, the relative importance of the road is a confounder that will result in increasing the correlation between structural condition and observed surface condition. This confounder has minimal effect when examining the data within a specific road, as the importance is likely the same throughout. The correlation 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 is very low in general.

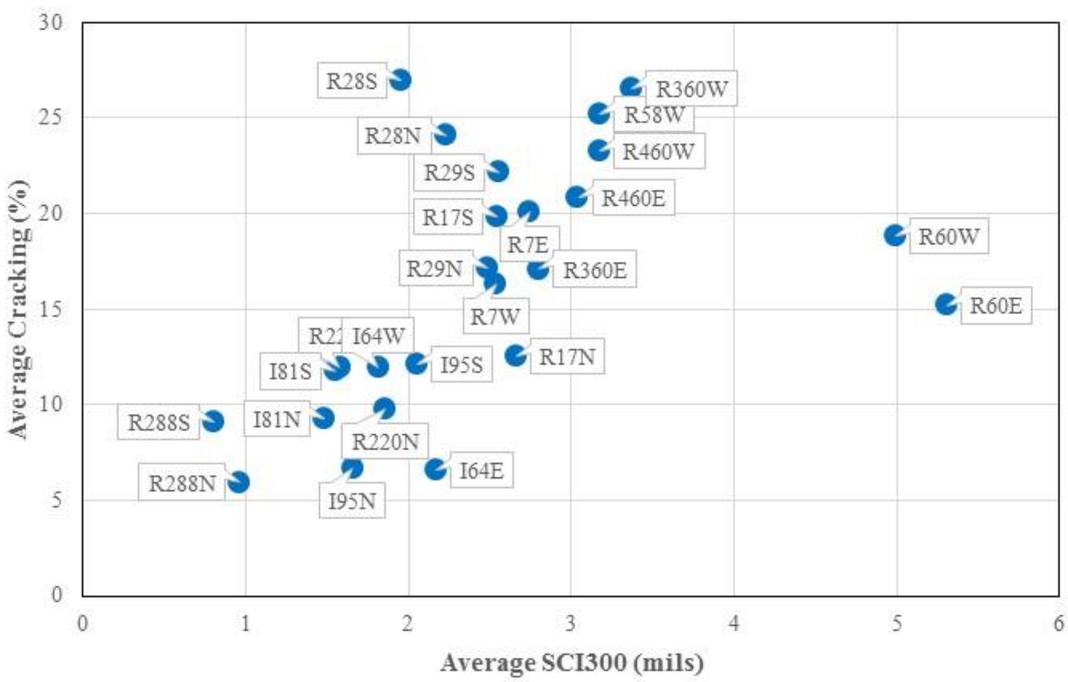


Figure 6. Cracking vs SCI300 for All Tested Roads

Figure 7 shows the average condition index (CCI, LDR, and NDR) as a function of the time since the last treatment for the structurally strongest 25th percentile of sections and the structurally weakest 25th percentile of 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 improved significantly. This was likely caused by a rapid decrease in the number of sections considered, such that the sections included beyond years 9 and 10 are really outliers in terms of performance. On average, the interstate sections had a time span of 6.5 years between the last applied maintenance treatment and the time when the TSD data was collected. However, sections that are performing well are treated for longer periods. Therefore, the average condition at higher ages for the structurally weak sections is biased, as only the sections that have performed exceptionally well are represented in that group. Figure 7 suggests that this biasing effect becomes very significant at about 9 or 10 years. Accordingly, only the data up to year 8 was used to develop the pavement deterioration models.

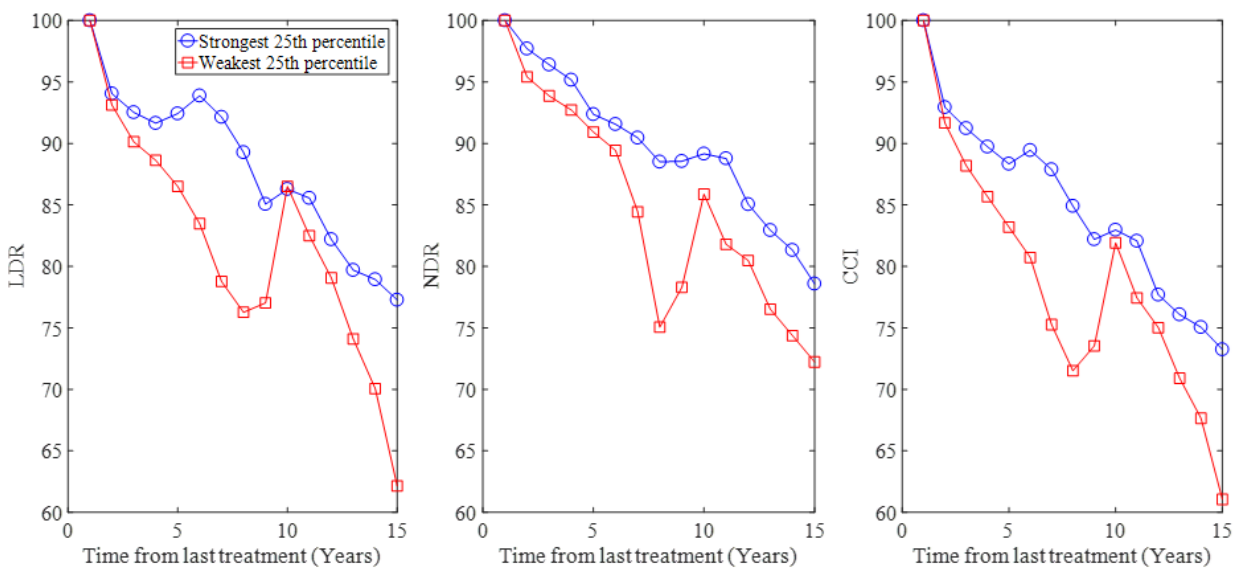


Figure 7. Average Condition for Tested Interstate Roads (LDR, NDR, and CCI) of Structurally Strongest 25th Percentile and Structurally Weakest 25th Percentile of Sections as a Function of Time from Last Treatment. LDR = Load Related Deterioration Rating, NDR = Non-Load Related Deterioration Rating, and CCI = Critical Condition Index.

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. Quasi-Poisson modeling is appropriate for discrete data that has a linear relationship between the mean and the variance. The condition data is discrete and the relationship between the mean and the variance for I-81NB is shown in Figure 8. The data in the figure was obtained by calculating the mean deterioration (e.g., 100 – LDR) and the standard deviation of the deterioration for each year on I-81NB. A linear relationship 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 trend was also shown by Katicha et al. (2017b) and Pantuso et al. (2019).

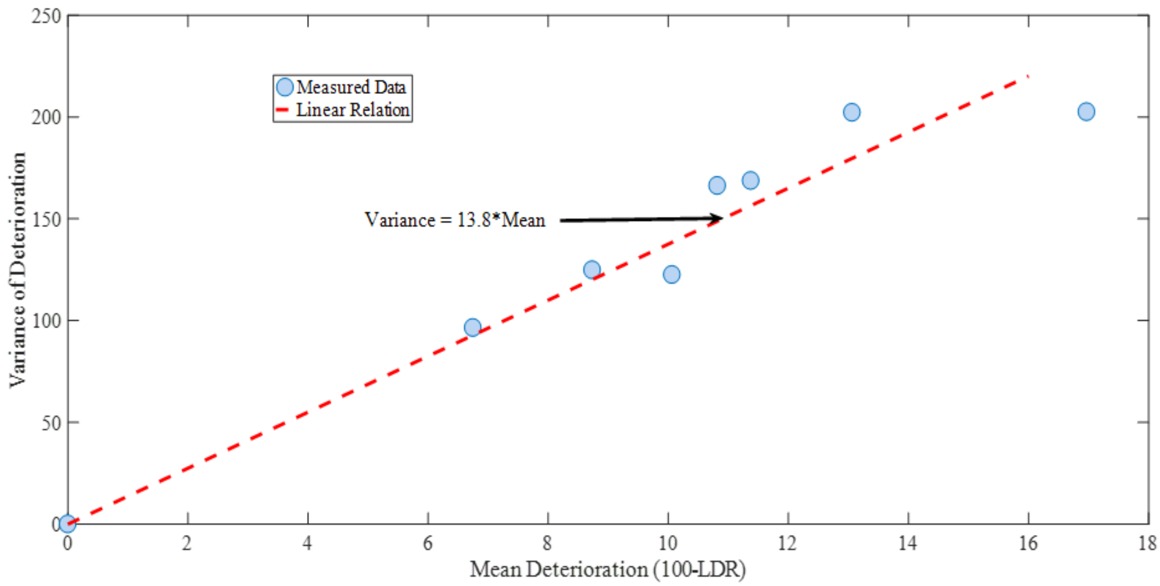
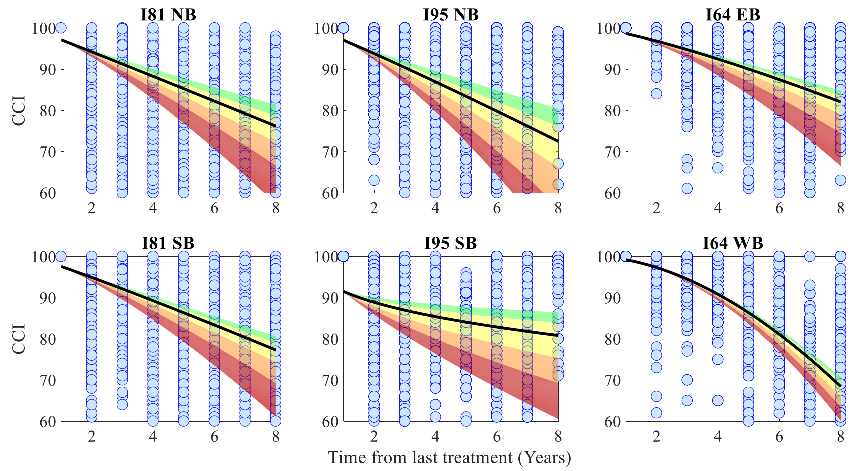


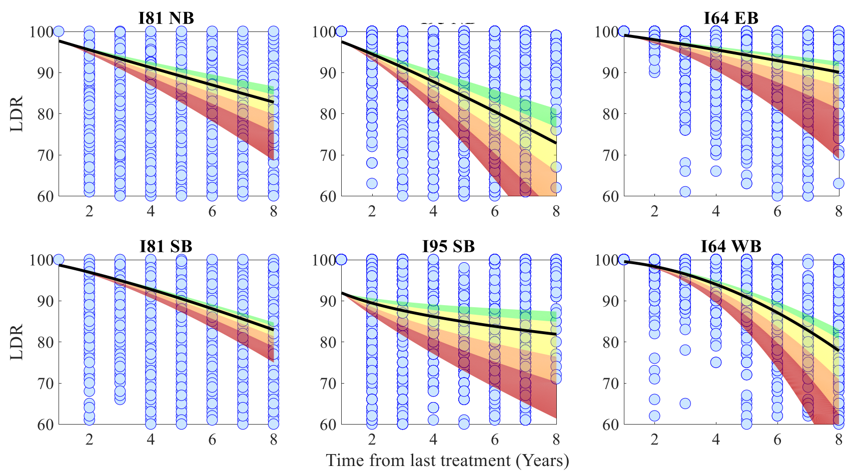
Figure 8. Variance of 100- Load Related Deterioration Rating as a Function of Mean Deterioration Defined as 100- Load Related Deterioration Rating. The Dependence of the Variance on the Mean Is Best Approximated by a Linear Relationship.

Figure 9a shows the deterioration model for CCI, with SCI300 used as the parameter representing the structural condition. The shaded colors show how the structural condition affects the rate of the deterioration, with green representing very strong sections and red representing very weak sections. The figure shows CCI values between 60 and 100, as VDOT considers pavement below 60 to be deficient. For I-81NB, I-81SB, I-95NB, I-95SB, and I-64EB, the structural condition had a significant effect on the CCI rate of deterioration. For I-64WB, the structural condition had a very small effect on the CCI rate of deterioration. A closer look at LDR and NDR is shown in Figure 9b and Figure 9c.

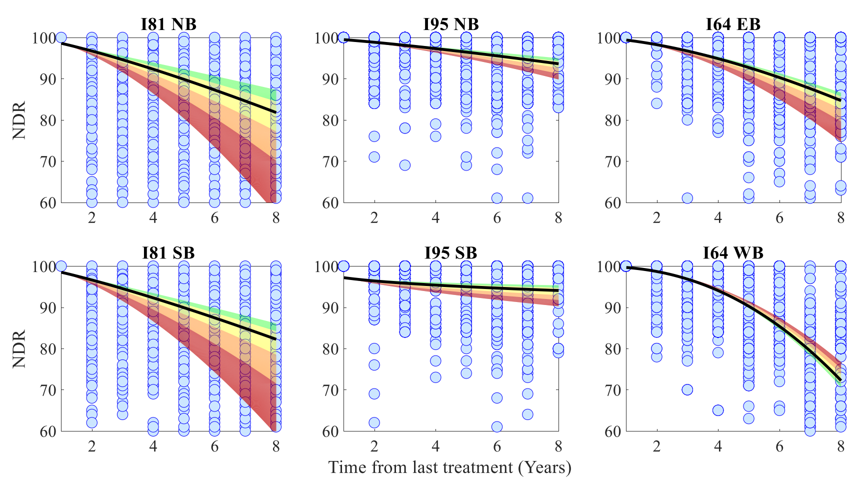
For LDR, Figure 9 shows that the structural condition affects the rate of deterioration on all roads. The most interesting case is I-64WB, which shows that structural condition has a large effect on the LDR as opposed to the CCI discussed earlier. For NDR, we see that the structural condition has practically 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 greater than the LDR, while for I-64, the two indices are much closer, with the NDR generally being less. The fact that the CCI is the minimum of LDR and NDR may explain why the structural condition did not have a significant effect on the rate of deterioration for I-64WB; the CCI on I-64WB is mostly the NDR value since the NDR is generally less than the LDR. For I-95, the CCI and LDR are practically the same since the NDR is usually much greater. For I-81 the structural condition seems to have a larger effect on the NDR than the LDR. This seems counterintuitive, as the NDR values are not supposed to be caused by loading and therefore should not be significantly affected by structural condition. However, LDR and NDR classifications are based on the predominant mechanism of the specific distress and even NDR could be affected by loading to a degree. I-81 carries the highest loading in Virginia in terms of truck volume, which could also contribute to the most observed distresses being affected by loading and therefore structural condition (I-95 is second but has more lanes overall than I-81.)



(a)



(b)



(c)

Figure 9. Deterioration Models: a) Critical Condition Index; b) Load Related Deterioration Rating; and c) Non-load-Related Distress rating with Respect to Structural Condition. Green Shading Green Indicates Very Strong Sections, Yellow Indicates Strong Sections, Orange Indicates Weak Sections, and Red Indicates Very Weak Sections.

The deterioration model parameters for interstate roads are shown in Table 4, while those for primary roads are shown in Table 5. 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 sections). There are a few cases (13 out of the 72 models; 3 out of the 24 models for LDR) where higher SCI300 result in lower deterioration rates. However, in most of these cases (7 out of the 13 cases; 3 out of the 3 cases for LDR) the parameter was not statistically significant at the 0.05 level. Deterioration models using d_0 and SN_{eff} (results not shown here) were also developed; however, the SCI300 was found the best parameter to use based on the number of models that show a consistent increase in deterioration rate for structurally weaker sections.

Table 4. Deterioration Model parameters Estimate and Significance for Interstate Roads (Values in Bold Not Significant at the 5% Level; Values in Italic Have Negative Sign)

Road	Parameter	CCI		LDR		NDR	
		Estimate	p-value	Estimate	p-value	Estimate	p-value
I-81N	β_1	0.875	< 0.01	0.818	< 0.01	1.028	< 0.01
	β_2	7.145	< 0.01	7.362	< 0.01	10.489	< 0.01
I-81S	β_1	0.987	< 0.01	1.175	< 0.01	1.060	< 0.01
	β_2	4.550	< 0.01	3.227	< 0.01	7.064	< 0.01
I-95N	β_1	0.873	< 0.01	0.937	< 0.01	1.115	< 0.01
	β_2	5.271	< 0.01	5.674	< 0.01	3.749	< 0.01
I-95S	β_1	0.194	< 0.01	0.182	< 0.01	0.227	< 0.01
	β_2	5.529	< 0.01	5.794	< 0.01	3.845	< 0.01
I-64E	β_1	1.138	< 0.01	0.948	< 0.01	1.492	< 0.01
	β_2	3.86	< 0.01	7.042	< 0.01	3.124	< 0.01
I-64W	β_1	1.729	< 0.01	1.688	< 0.01	2.272	< 0.01
	β_2	1.886	< 0.01	6.734	< 0.01	***-1.426***	**0.08**

Comparison Between FWD and TSD Data

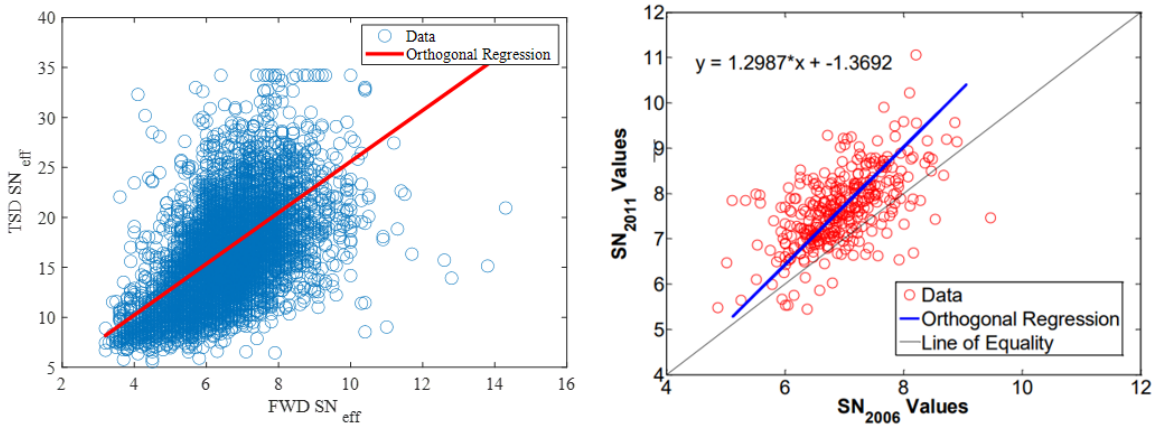
Comparison of SN_{eff}

A comparison between the TSD and FWD SN_{eff} on the interstate roads was performed. No comparison was made for primary roads since there are no network level FWD data available. The repeated FWD SN_{eff} are from Bristol district measurements collected in 2006 and subsequently in 2011 as part of a study to analyze repeated network level FWD testing (Bryce et al., 2017). The scatter between the TSD and FWD is similar to the scatter between repeated measures of the FWD as shown in Figure 10. To evaluate replacing FWD measurements with TSD measurements, the following comparisons were performed for data collected on interstate roads:

1. Compare the cumulative distribution of TSD and FWD SN_{eff}.
2. Calculate the consistency in identifying the same weak sections between TSD and FWD SN_{eff}.

Table 5. Deterioration Model Parameters Estimate and Significance for Primary Roads (Values in Bold Not Significant at the 5% Level; Values in Italics Are Negative)

Road	Parameter	CCI		LDR		NDR	
		Estimate	p-value	Estimate	p-value	Estimate	p-value
Rt 17N	β_1	1.539	< 0.01	2.062	< 0.01	1.627	< 0.01
	β_2	1.901	< 0.01	2.461	< 0.01	0.892	< 0.01
Rt 17S	β_1	0.980	< 0.01	0.922	< 0.01	1.114	< 0.01
	β_2	3.031	< 0.01	3.924	< 0.01	2.648	< 0.01
Rt 29N	β_1	1.287	< 0.01	1.311	< 0.01	2.031	< 0.01
	β_2	1.808	< 0.01	2.128	< 0.01	**1.457**	**0.06**
Rt 29S	β_1	1.322	< 0.01	1.573	< 0.01	1.227	< 0.01
	β_2	***-0.213***	**0.65**	***-0.742***	**0.20**	1.947	< 0.01
Rt 7E	β_1	0.836	< 0.01	0.836	< 0.01	0.956	< 0.01
	β_2	1.892	< 0.01	3.269	< 0.01	***-0.961***	**0.22**
Rt 7W	β_1	0.416	< 0.01	0.740	< 0.01	0.449	< 0.01
	β_2	5.409	< 0.01	6.093	< 0.01	3.744	< 0.01
Rt 28N	β_1	0.953	< 0.01	0.924	< 0.01	1.327	< 0.01
	β_2	3.218	< 0.01	4.016	< 0.01	2.478	< 0.01
Rt 28S	β_1	0.832	< 0.01	0.740	< 0.01	1.549	< 0.01
	β_2	2.772	< 0.01	3.617	< 0.01	2.308	< 0.01
Rt 58E	β_1	1.127	< 0.01	1.065	< 0.01	1.334	< 0.01
	β_2	4.925	< 0.01	6.611	< 0.01	3.599	< 0.01
Rt 58W	β_1	0.586	< 0.01	0.431	< 0.01	1.052	< 0.01
	β_2	**0.096**	**0.84**	1.186	< 0.01	*-2.141*	< 0.01
Rt 220N	β_1	1.015	< 0.01	1.401	< 0.01	1.050	< 0.01
	β_2	1.870	< 0.01	**0.642**	**0.52**	**1.183**	**0.22**
Rt 220S	β_1	0.967	< 0.01	1.448	< 0.01	0.994	< 0.01
	β_2	3.626	< 0.01	2.502	< 0.01	**1.948**	**0.13**
Rt 360E	β_1	1.539	< 0.01	1.434	< 0.01	2.050	< 0.01
	β_2	*-2.006*	< 0.01	***-1.596***	**0.06**	*-1.639*	0.02
Rt 360W	β_1	1.427	< 0.01	1.482	< 0.01	1.807	< 0.01
	β_2	3.152	< 0.01	4.811	< 0.01	1.306	< 0.01
Rt 460E	β_1	1.272	< 0.01	1.113	< 0.01	1.971	< 0.01
	β_2	*-0.3062*	< 0.01	***-1.961***	**0.07**	*-5.361*	< 0.01
Rt 460W	β_1	1.345	< 0.01	1.318	< 0.01	1.617	< 0.01
	β_2	**0.059**	**0.86**	0.622	< 0.01	***-0.405***	**0.27**
Rt 288N	β_1	*-0.450*	< 0.01	*-0.564*	< 0.01	**0.121**	**0.44**
	β_2	72.227	< 0.01	73.670	< 0.01	70.04	< 0.01
Rt 288S	β_1	**0.200**	**0.164**	**0.155**	**0.301**	**0.283**	**0.08**
	β_2	65.085	< 0.01	65.540	< 0.01	72.78	< 0.01



(a) (b)

Figure 10. Comparison Between Values: (a) Traffic Speed Deflectometer and Falling Weight Deflectometer *SN*$_{eff}$; (b) Repeated Falling Weight Deflectometer *SN*$_{eff}$

Figure 11 shows that the shape of the cumulative distributions for SN_{eff} from the TSD and FWD were similar. This suggests the procedure used to determine the threshold differentiating between structurally weak and structurally adequate sections based on the cumulative distribution of FWD measurements can also be used with the TSD. The SN_{eff} threshold of 6 was adopted by VDOT for the FWD. This threshold results in a proportion of 0.3 (30%) of the sections being identified as structurally weak sections and corresponds to a SN_{eff} of 14 calculated from TSD measurements. This threshold results in a proportion of 0.3 (30%) of the sections being identified as structurally weak sections (see Figure 11). The distributions of SCI300 and D_0 are shown in Figure 12 and Figure 13, respectively. Figure 14 shows the cumulative distribution of the TSD-based SN_{eff} on the primary roads. For the primary roads, network level FWD data is not available. The 0.3 proportion suggested for interstate roads corresponds to a TSD-based SN_{eff} of 7 on primary roads. This seems to be an appropriate threshold to use on primary roads since these roads generally carry less truck traffic than interstate roads.

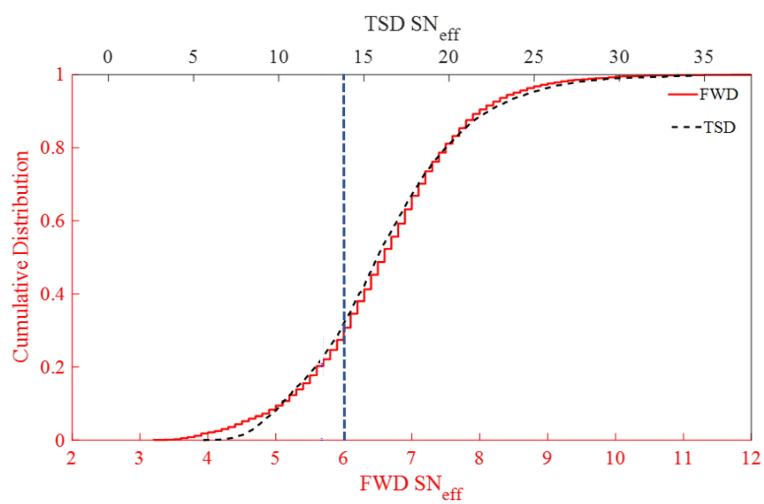


Figure 11. Cumulative Distribution of Traffic Speed Deflectometer and Falling Weight Deflectometer *SN*$_{eff}$ on Interstate Roads

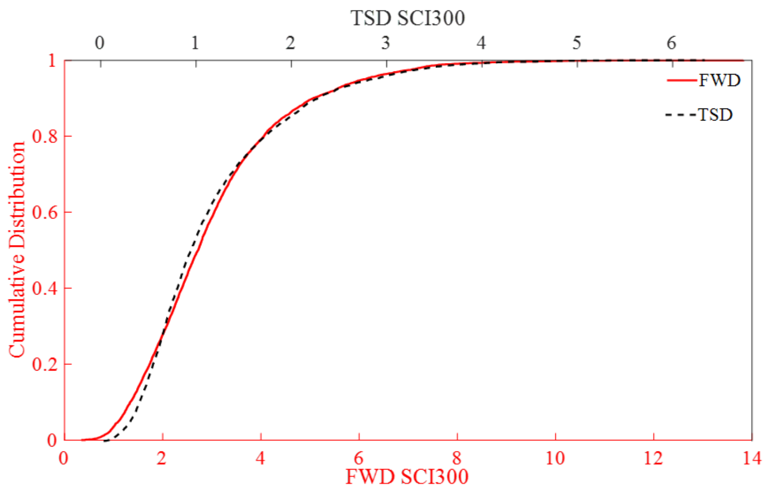


Figure 12. Cumulative Distribution of Traffic Speed Deflectometer and Falling Weight Deflectometer SCI300 on Interstate Roads

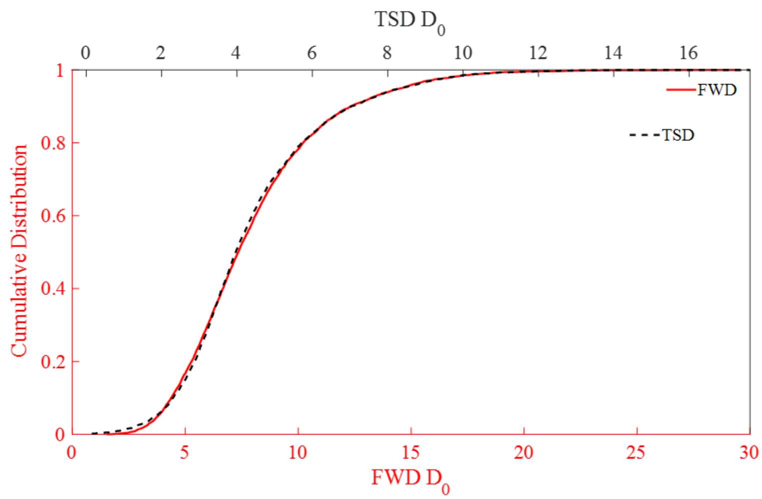


Figure 13. Cumulative Distribution of Traffic Speed Deflectometer and Falling Weight Deflectometer D_0 on Interstate Roads

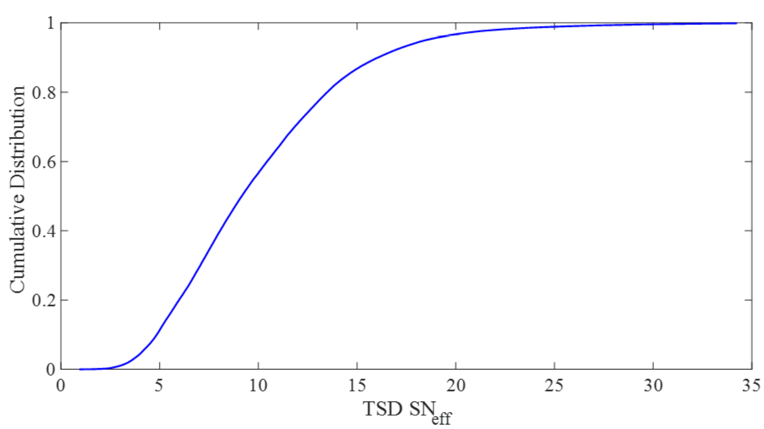


Figure 14. Cumulative Distribution of Traffic Speed Deflectometer SN_{eff} on Primary Roads

Figure 15 shows the results of the consistency test. For $\alpha = 1\%$, 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 FWD-based SN_{eff} of 6 and TSD-based SN_{eff} 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 $\alpha = 100\%$. Figure 15 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 2017. The consistency between the TSD and FWD is higher than the consistence between two sets of FWD measurements. The TSD data was collected at 10 m (3.3 ft) intervals and averaged over the 0.2-mile length used for FWD data collection. The averaging reduced the TSD measurement variability, contributing to increased consistency. For the FWD, only one measurement was collected at every 0.2 mile, 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 consistency results indicate that the measurements from each device are comparable. This suggests that replacing the current FWD SN_{eff} data with the new TSD SN_{eff} data is, from a statistical perspective, at least as good as updating the existing FWD SN_{eff} data with a new set of FWD SN_{eff} data.

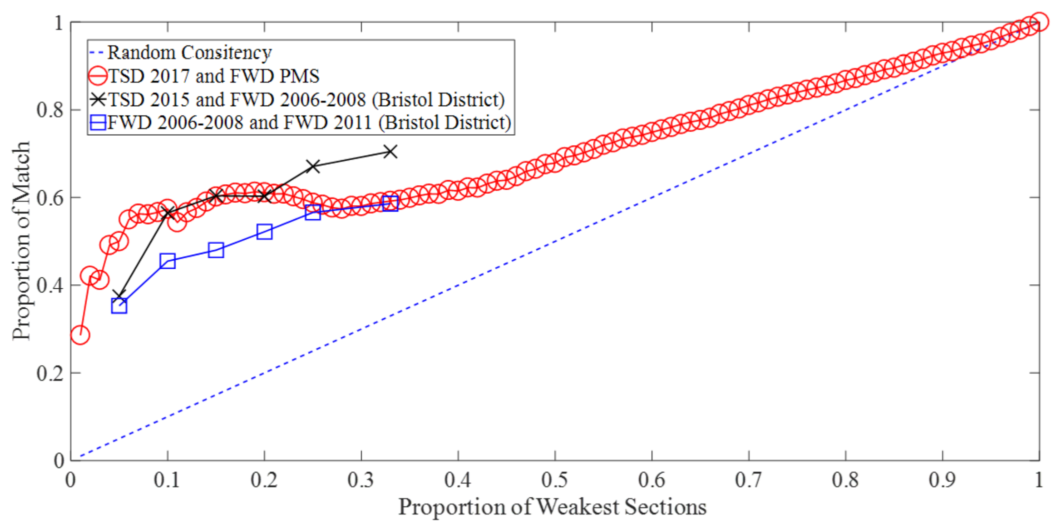


Figure 15. Results of the Consistency Test

Comparison of TSD Indices to FWD Resilient Modulus (M_R)

In the PMS enhanced decision process, VDOT uses the SN_{eff} and the subgrade resilient modulus (M_R) to characterize the structural condition of the pavement sections. This portion of the study was conducted to identify if any of the TSD-based indices could be used as a substitute for the FWD-derived M_R. The set of FWD and TSD deflections include D_0, D_8, D_{12}, D_{18}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72} where the subscript refers to the distance in inches from applied load.

Figure 16 shows the relationship between FWD measured D_{36} and M_R. The M_R estimate in Equation 11b is obtained assuming the load is 9,000 lb and the variation between the data and the model is due to the fact that the load in the FWD test is not always equal to 9,000 lb. However, the variation is small so that the Spearman rank correlation between D_{36} and M_R is -

0.98 and not exactly -1. The Spearman correlation of these deflections with FWD M_R is given in Table 6 for the FWD and the TSD and for both cases of all pavement sections and all pavement sections that have not had a treatment applied between 2007 and 2017 (i.e., the period between FWD and TSD testing). The correlation between the TSD deflections and the M_R is very low whether all sections are used or only sections that were not treated are used. Furthermore, the sign of the correlation for D_{18} to D_{72} is positive which is opposite what should be expected (the sign should be negative so that higher deflections correspond to lower M_R).

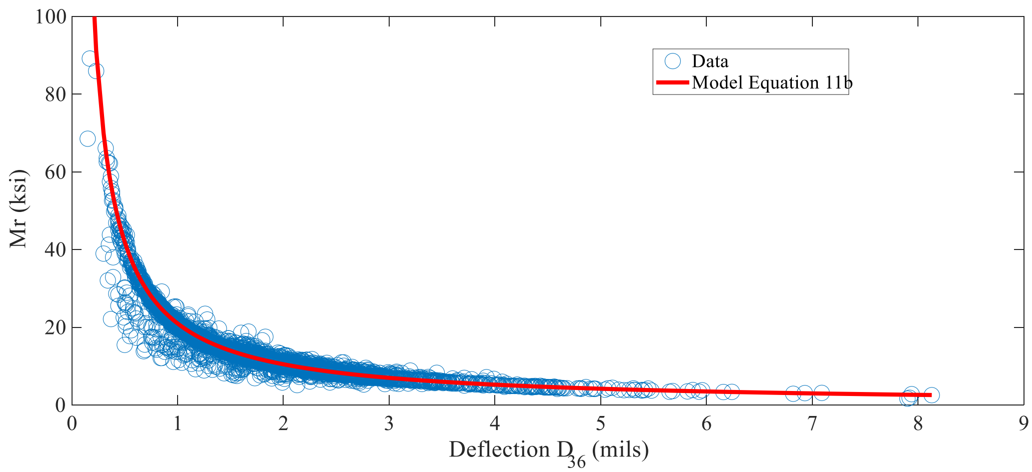


Figure 16. Resilient Modulus vs Falling Weight Deflectometer Deflection D_{36}

Rada et al. (2016) proposed indices based on the difference between measured deflections as indicators of strain at the top of the subgrade and therefore subgrade strength (see Table 2). Using the whole set of deflections, there are 36 possible differences between two deflections (or indices) that can be calculated. The Spearman correlations of the 36 possible differences calculated with the TSD data and the FWD M_R are given in Table 7. The deflection difference used to calculate the correlation in each cell is that obtained by taking the difference between the deflection listed in the left column and the deflection listed in the top row. For example, the correlation of -0.29 shown in the table is between D0 – D12 and M_R. In general, the correlations are low although in some cases higher than the ones in Table 6. The highest correlation in absolute value is between D0 – D12 and M_R which is not expected. D0 – D12 is SCI300 which is an indicator of the structural condition of the top layers (generally the asphalt layer), not the subgrade.

The correlations in Table 6 and Table 7 show there are no indices based on TSD deflections or their differences that result in good agreement with the M_R obtained from FWD measurements. This could be due to a number of factors including:

1. There is a difference between how the tests are performed for FWD and TSD and how the data is collected (peak deflections for FWD vs instantaneous deflections for TSD).
2. The TSD measures the deflection slope and the deflections are calculated from the deflection slopes using numerical integration. This involve making assumptions about the slope at each end of the deflection bowl that might introduce some error in the integration.

3. The network level FWD data was mostly collected between 2007 and 2008 while the TSD data was collected in 2017. This is a relatively long period during which the structural condition of the subgrade could have changed.
4. Bryce et al. (2016) reported that the Spearman correlation for M_R calculated from network level repeated FWD testing is 0.67 based on measurements obtained five years apart. This suggests that correlation between FWD M_R and TSD measurements will in general be lower than 0.67.
5. The small deflections calculated using the sensors furthest from the load may approach the measurement error of the sensors.

Failure to obtain a good correlation between the FWD measured M_R and TSD indices suggests the need for more detailed investigation of the two devices with measurements collected on the same sections and at the same time. Ideally, these sections would also be instrumented so that the response of both devices can be compared to that of the pavement. As such these sections could be used to establish a calibration site for the TSD. Some important desirable characteristics of the calibration site are:

1. Include pavement sections that are spatially close.
2. Include sections with a wide range of structural strength (relatively weak and strong sections).
3. Sections should preferably be relatively straight to reduce the effect of road horizontal curvature on the TSD axle load distribution.
4. Sections should be located where they can be easily accessed and were the traffic volume is not too high.
5. Section layer thicknesses and composition should be documented (coring and GPR).
6. Sections should be instrumented. Work by Rada et al. (2016) suggests that geophones embedded in the pavement work best.

Table 6. Spearman Rank Correlation Between Falling Weight Deflectometer Resilient Modulus and Traffic Speed Deflectometer and Falling Weight Deflectometer Measrured Deflections

	Untreated Sections		**All Sections**	
	TSD[a]	**FWD[b]**	**TSD**	**FWD**
D0	-0.13	-0.68	-0.20	-0.63
D8	-0.07	-0.76	-0.15	-0.72
D12	-0.03	-0.82	-0.11	-0.79
D18	0.03	-0.89	-0.05	-0.88
D24	0.07	-0.95	0.00	-0.94
D36	0.12	-0.99	0.09	-0.98
D48	0.16	-0.94	0.14	-0.92
D60	0.19	-0.88	0.18	-0.86
D72	0.21	-0.84	0.22	-0.82

a: Traffic Speed Deflectometer; b: Falling Weight Deflectometer

Table 7. Spearman Rank correlation Between Falling Weight Deflectometer Resilient Modulus and Traffic Speed Deflectometer Deflection Differences

	D8	D12	D18	D24	D36	D48	D60	D72
D0	-0.25	-0.29	-0.26	-0.21	-0.13	-0.04	0.04	0.11
D8		-0.27	-0.28	-0.25	-0.17	-0.09	0.00	0.08
D12			-0.27	-0.27	-0.21	-0.13	-0.06	0.03
D18				-0.27	-0.24	-0.18	-0.10	-0.03
D24					-0.25	-0.21	-0.15	-0.07
D36						-0.23	-0.18	-0.12
D48							-0.21	-0.16
D60								-0.19

Effects Incorporating Structural Condition Information into the PMS Decision Process

The effect of the structural condition on the surface condition deterioration rate suggests that the structural condition should also be considered for appropriate treatment selection of road sections. For example, if the treatment selection recommendation, based on surface condition only, is RC and the structural condition based on TSD or FWD testing is very strong, it may not make sense to keep RC as the selected treatment, as it is likely that the poor condition could be remedied by less invasive treatments. Conversely, applying a thinner treatment (such as PM or RM) on a pavement section that is structurally weak will not address the cause of the structural deficiency and may not represent the optimum use of maintenance funding, as it is likely the light treatment will not last long enough to justify the investment. Therefore, an augmented structural-condition-based treatment selection matrix was developed that takes into account structurally strong and weak sections and modifies the treatment selected based on the surface condition (see Table 8).

Three structural condition categories (strong, fair, and weak) are proposed based on the percentile in which the structural condition falls. The threshold that separates strong and fair sections is taken as the 25th percentile of SCI300 and the threshold that separates fair and weak is the 75th percentile of SCI300. For pavements noted as having a fair structural condition, no treatment modification is done. For pavements described as having a strong structural condition, the treatment category is generally reduced by one level. That is, CM is modified to PM, RM to CM, and RC to RM (but 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 infiltration, which will contribute to loss of strength in the long term, and therefore crack sealing should still be performed. For pavements in a weak structural condition, the treatment category is generally increased by one level of severity; that is, CM is modified to RM and RM is modified to RC. The categories of DN and RC are not modified. The modification of CM to RM and RM to RC are expected because the pavement is weak and a heavier treatment should be anticipated to address the cause of the deterioration. Suggestions of PM are modified to DN which seems counterintuitive; however, a PM treatment is likely to be ineffective for structurally weak sections and it would be more cost effective to let these sections further deteriorate and apply a heavier treatment later.

Table 8. Modified Treatment Category Based on Structural Condition

Initial Treatment Category	Modified Treatment Category with Structural Condition Category		
	Strong	**Fair**	**Weak**
DN[a]	DN	DN	DN
PM[b]	PM	PM	DN
CM[c]	PM	CM	RM
RM[d]	CM	RM	RC
RC[e]	RM	RC	RC

a: Do Nothing; b: Preventive Maintenance; c: Corrective Maintenance: d; Restorative Maintenance; e: Reconstruction

The effects of the treatment modification for the actual interstate network tested as part of this study are shown in Table 9. On average, there was a reduction of 15% in the CM category, which is mostly evenly redistributed to PM and RM categories. The percentages of the DN and RC categories were not significantly affected.

Table 9. Percentage of Initial Treatment Category Recommended by Virginia Department of Transportation Pavement Management System and the Percentage of Modified Treatments After Taking Into Consideration the Traffic Speed Deflectometer-Based Structural Condition

Road	Treatment Type	Percentage of PMS[a] Recommendation, %				
		DN[b]	**PM[c]**	**CM[d]**	**RM[e]**	**RC[f]**
I81N	Without Structural Condition	24.4	13.3	47.6	11.7	3.0
	With Structural Condition	27.2	22.2	29.5	17.5	3.7
I81S	Without Structural Condition	28.0	10.1	52.2	8.6	1.0
	With Structural Condition	29.4	19.5	34.4	13.4	3.3
I95N	Without Structural Condition	45.7	14.8	32.4	1.1	5.9
	With Structural Condition	47.1	21.8	19.6	6.8	4.7
I95S	Without Structural Condition	34.1	10.9	47.4	1.0	6.5
	With Structural Condition	35.8	16.2	29.9	12.3	5.7
I64E	Without Structural Condition	11.5	22.9	51.9	6.2	7.5
	With Structural Condition	14.3	31.2	34.0	13.4	7.2
I64W	Without Structural Condition	37.8	14.4	32.0	6.2	9.5
	With Structural Condition	40.3	17.2	22.2	12.8	7.4
Average	Without Structural Condition	30.3	14.4	43.9	5.8	5.6
	With Structural Condition	31.4	21.4	28.3	12.7	5.3

a: pavement management system; b: Do Nothing; c: Preventive Maintenance; d: Corrective Maintenance; e: Restorative Maintenance; f: Reconstruction

Figure 17 shows the average maintenance cost per mile as a function of the selected SCI300 threshold to define weak and strong sections. The thresholds of the weak and strong sections are defined as a function of the cumulative distribution function (CDF) of SCI300. For example, if the strong sections threshold is defined based on the 0.1 CDF value and the weak sections are defined based on the 0.8 CDF value, this means that the sections having an SCI300 value less than the 10th percentile SCI300 value are defined as strong sections while the sections having an SCI300 value greater than the 80th percentile SCI300 value are defined as weak sections. The figure illustrates how increasing the threshold defining the strong sections decreases the average maintenance cost while reducing the threshold defining the weak sections increases the average maintenance cost. The maintenance cost at the point (1, 0) is the cost based solely on the surface condition (i.e., not including the structural condition). The black line along the surface is the set of CDF values for defining weak and strong sections that results in a cost that is equal to the cost based solely on the surface condition criteria treatment selection. The part of the surface to the left of the black line corresponds to a combination of CDF values that results in increased cost while the part of the surface to the right of the black line corresponds to a combination of CDF values that results in decreased cost. Figure 17 case be used to help determine where the cut-off values can be applied if equal costs are desired.

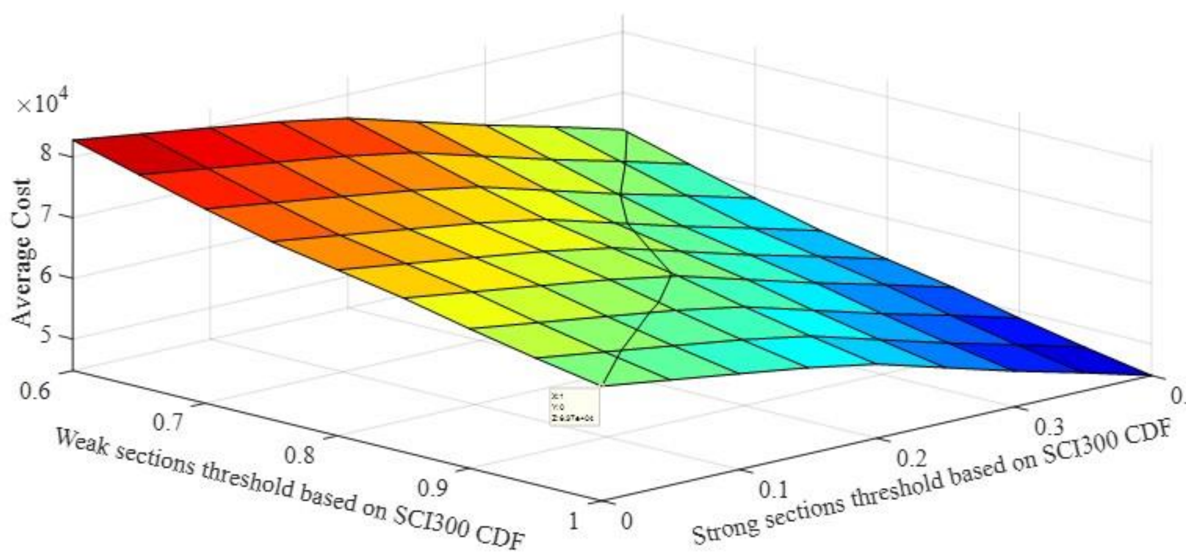


Figure 17. Average Maintenance Cost per Mile as a Function of Selecting a Threshold for Strong and Weak Sections Based on Cumulative Distribution of SCI300

Figure 18 shows how the proportion for each selected treatment changes for a 0.7 CDF threshold for weak sections and no threshold for strong sections (i.e., sections are defined as either Fair or Poor) based on TSD and FWD data. The reason for not including a threshold for strong sections is because VDOT currently only modifies treatment for weak sections. Treatment modification is performed based on Table 4. The proportion of sections and the percentage of sections having the same treatment based on FWD and TSD-based structural condition is 84%.

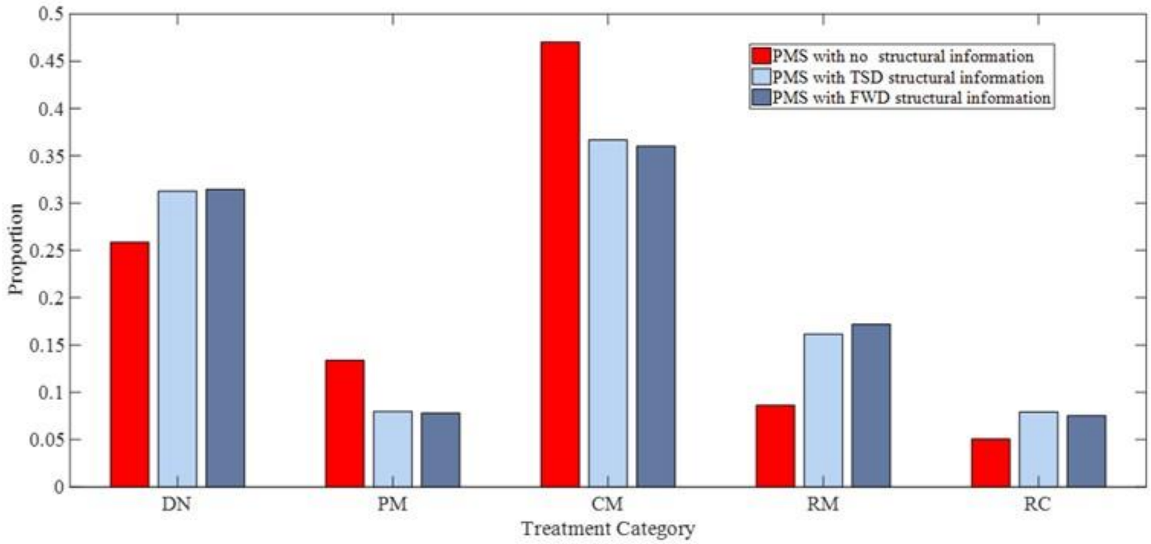


Figure 18. Proportion of Selected Treatment Category Based on Surface Condition, Structural Condition from TSD, and Structural Condition from Falling Weight Deflectometer

Sensitivity Analysis

The sensitivity analysis was performed to evaluate if using the SCI300 or the TSD-based SN_{eff} resulted in identifying the same structurally weak sections. The effect is measured by the consistency to identify the same weak sections. The minimum consistency between SCI300 and TSD-based SN_{eff} is 0.58 for a proportion of weak sections of 0.01 (i.e., 1%) however, the consistency quickly increases to more than 0.8 when the proportion of weak sections considered is 0.15 (see Figure 19).

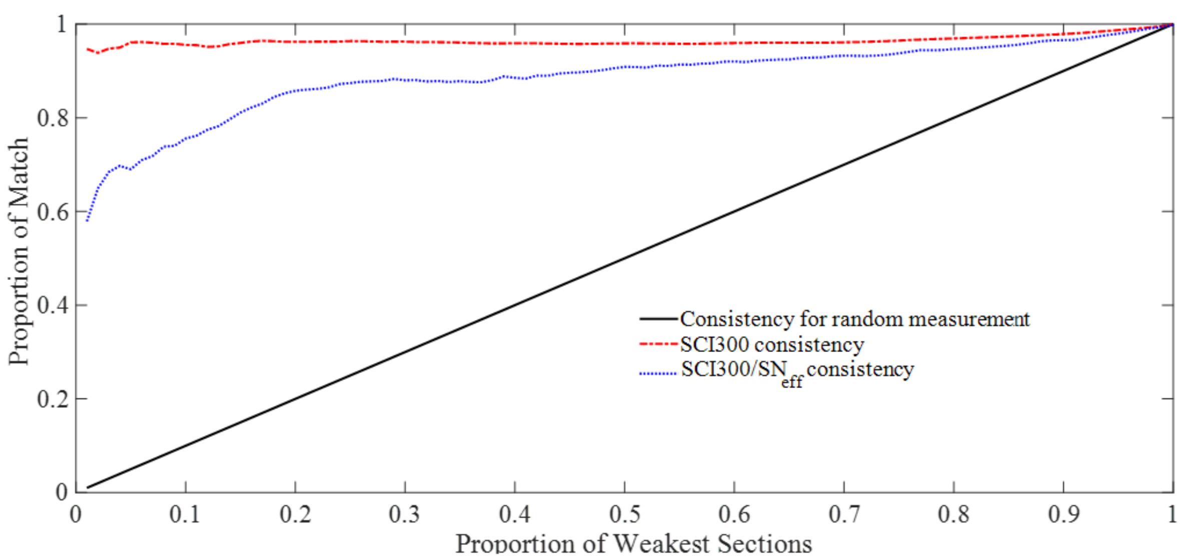


Figure 19. Consistency Between SCI300 and Traffic Speed Deflectometer-based *SN_{eff}* and Between Simulated Repeated Measurements of SCI300

For a weak section proportion of 0.3, the consistency is 0.88 suggesting that there is not a significant difference between identified weak sections whether SCI300or TSD-based SN_{eff} is used. That is, these sections will receive the same suggested treatment whether they are identified by using the SCI300 or TSD-based SN_{eff}. The Spearman rank correlation between SCI300 and TSD-based SN_{eff} is -0.93 which is relatively high (in absolute value) further confirming the consistency results.

The simulated consistency for repeated SCI300 measurements is 0.94 suggesting the repeatability of TSD measurements is very good in identifying the same weak sections. This is partly due to the fact that the TSD continuously measures the pavement response along a pavement section and the resulting SCI300 is the average of many measurements which reduces the variability.

The results of the sensitivity analysis suggest that 1) the effect of whether SCI300 or TSD-based SN_{eff} is used to characterize the structural condition does not significantly affect which sections are considered weak or strong and 2) the repeatability of TSD measurements is very good for identifying weak sections at the network level.

CONCLUSIONS

- *The structural condition of a pavement section influences the rate of deterioration of the surface condition.* Those sections found to have a weak structural condition were found to deteriorate faster than those sections found to have a strong structural condition. This further reinforces the need to account for structural condition in the treatment selection process.

- *The structural condition obtained using the TSD can replace the structural condition obtained from the FWD that is currently used in the VDOT PMS.* The distribution of the TSD-based SN_{eff} was similar to the FWD-based SN_{eff} and the calculated consistency between the TSD-based SN_{eff} and FWD-based SN_{eff} was higher than the consistency between the SN_{eff} from two repeated sets of FWD measurements.

- *The lower limit of TSD-based SN_{eff} to identify structurally weak sections can be based on the 30^{th} percentile value.* This was a similar process followed with the network FWD data. The lower limit of TSD-based SN_{eff} may be based on all collected data or a separate SN_{eff} may be developed for interstate, primary, and secondary routes.

- *There is very little practical difference between using the SN_{eff} or SCI300 to identify structurally weak sections from TSD measurements based on the 30^{th} percentile value.* The consistency between the two parameters is 0.88 and the Spearman rank correlation is -0.93. The SN_{eff} calculated from TSD measurements has the advantage that it is the index currently used by VDOT with FWD data. The SCI300 has the advantage that it does not require pavement thickness information and it is mechanistically related to the tensile strain at the bottom of the asphalt layer.

RECOMMENDATIONS

1. *VDOT's Maintenance Division should replace the currently used FWD-based SN_{eff} with the TSD-based SN_{eff} in the agency pavement management system.*

2. *VDOT's Maintenance Division should base the definition of structurally weak sections on the 30th percentile of SN_{eff}.*

3. *VDOT's Maintenance Division should study the impact of including a structurally strong designation to describe certain pavement sections that were identified to have a strong structural condition.* This could result in reduced average maintenance costs per mile. The influence on the overall network performance is unclear at this time.

4. *VTRC should continue to develop similar descriptors of structural capacity that can be used to describe the structural properties of composite and concrete surfaced pavements.*

IMPLEMENTATION AND BENEFITS

Implementation

Regarding Recommendations 1 and 2, VDOT's Maintenance Division will start an investigation to determine the network-wide impact of replacing the currently used structural data based on FWD testing with data updated based on TSD testing. Maintenance Division will use the definition of structurally weak sections on the 30th percentile of SN_{eff} from the TSD testing. The results of this work will identify the financial impact of such changes on the maintenance needs and budget distribution before it can be incorporated in the network-wide pavement management decision-making process. This work will be completed by July 2021.

Regarding Recommendation 3, VDOT's Maintenance Division will incorporate a structurally 'strong' designation using TSD data in the pavement maintenance recommendations for future years. This information will be made available to the districts through PMS so that they can use this to make maintenance and rehab decisions for the annual paving schedule development process. This analysis will be completed by July 2021.

Regarding Recommendation 4, VTRC will continue to support agency participation in National Pooled Fund Study 5(385) as a venue to coordinate additional TSD data collection efforts nationally and the TSD-based structural properties for composite and concrete surfaced pavements can be studied.

Benefits

The benefit of implementing Recommendation 1 is twofold. First, using TSD-based structural condition includes data at a much shorter sampling interval (0.1 mile) as compared to the FWD-based structural condition (0.2 mile). Second, since the TSD does not require interruptions to the traveling public and has a production rate many times that of the FWD, VDOT now has a realistic means of conducting structural testing on the primary and secondary networks. FWD-based structural testing was only attempted previously on the interstate system due to the large extent of the primary and secondary systems.

The benefit of implementing Recommendation 2 is that VDOT will maintain the same protocol as was used to determine structurally weak sections based on FWD testing.

The benefit of implementing Recommendation 3 is that VDOT might be able to realize cost savings from a reduced maintenance treatment severity on structurally strong sections.

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