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Evaluation of Traffic Speed Deflectometer for Pavement Structural Evaluation in Louisiana

Zia Uddin Ahmed Zihan
Louisiana State University and Agricultural and Mechanical College, zuahmed07@gmail.com

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EVALUATION OF TRAFFIC SPEED DEFLECTOMETER FOR PAVEMENT STRUCTURAL EVALUATION IN LOUISIANA

A Thesis

Submitted to the Graduate Faculty of the Louisiana State University and Agricultural and Mechanical College in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering

in

The Department of Civil and Environmental Engineering

by

Zia Uddin Ahmed Zihan
B.Sc., Islamic University of Technology, 2015
August 2019
To my father for everything...
ACKNOWLEDGMENTS

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Many state agencies have recognized the importance of incorporating pavement structural conditions in the selection of maintenance and rehabilitation (M&R) strategies along with functional indices. To measure in-service pavement structural capacity, surface deflection under a defined load has been typically used. The Rolling Wheel Deflectometer (RWD) and Traffic Speed Deflectometer (TSD) have emerged as continuous pavement deflection-measuring devices as they operate at traffic speed and reduces lane closure and user delays.

The research objective of this study was to assess the feasibility of using TSD measurements at the network-level for pavement conditions structural evaluation in Louisiana. To achieve the objectives of the study, TSD and Falling Weight Deflectometer (FWD) measurements were collected in District 05 of Louisiana and data were available from experimental programs conducted at the MnROAD research test facility and in Idaho. TSD measurements were compared with FWD deflection measurements to evaluate the level of agreement and difference between the two devices. Based on this evaluation, an SN predictive model was developed and validated to assess the structural conditions of in-service pavements based on TSD measurements. The model was then used to identify structurally sound and structurally deficient in-service pavements. This study also assessed whether the use of surface indices only or the declining rates of these indices to identify structurally damaged sections is feasible instead of relying on RWD and TSD estimated pavement structural indices.

Based on the results of the analysis, it is concluded that the deflection reported by both FWD and TSD for the same locations are statistically different, which was expected given the differences in loading characteristics and load type between the two devices. It is also concluded that surface roughness has a notable effect on the TSD field measured deflections.
The present study successfully developed and validated a model to predict in-service SN based on TSD deflections at 0.01-mile intervals of a road section. Core samples showed that the sections that were predicted to be structurally deficient from the model suffered from asphalt stripping and debonding problems. Yet, some of these sections were in very good conditions according to their functional indices.

Findings suggest that structural deficiency, rates of deterioration, and surface indices were correlated to a certain extent. Yet, surface indices cannot be used as a reliable predictor of structural capacity. For RWD tested sections, the most accurate surface index, which was the alligator cracking surface index, erroneously identified 35% of structurally sound sections as structurally deficient and 51.5% of structurally deficient sections as structurally sound. Similar results were also obtained for the TSD tested sections. The cost implication associated with misinterpreted sections from functional indices was investigated. The incorporation of structural indices is expected to provide significant savings to state agencies.
CHAPTER 1. INTRODUCTION

In-service pavement conditions are typically described by a number of functional factors such as surface distresses, roughness, and rutting, which do not necessarily describe the structural conditions of a pavement. As roads are being subjected to loads higher than the design traffic loads and to extreme weather events, the increasing rate of deterioration necessitates the incorporation of a structural capacity indicator in Pavement Management System (PMS) for effective rehabilitation and maintenance decision-making. Structural capacity is a valuable input in the design of Asphalt Concrete (AC) overlays and for identifying structurally deficient pavements. Assessing pavement structural capacity is important in selecting treatment methods and in making cost-effective decisions (Zofka et al. 2014, Elseifi and Elbagalati 2017, Zofka et al. 2014).

Pavement deflection under a given static or moving load is a fast and reliable method to evaluate pavement structural capacity. Deflection is also an important measurement that is used in numerous pavement deterioration models (Katicha et al. 2014). Pavement deflection is typically measured by applying a defined load using the Falling Weight Deflectometer (FWD). In FWD testing, an impact circular load is applied to the pavement surface at a predefined frequency. This stationary device uses multiple sensors located at different distances from the load to measure pavement surface deflections. Pavement layer moduli can be backcalculated from the deflection basin obtained from FWD testing (Irwin et al. 2009, Xu et al. 2002). While FWD allows measuring deflections with acceptable accuracy, it requires lane closures causing traffic delays and safety concerns. This has limited the use of FWD to project level applications and has led to the introduction of Traffic-Speed Deflection Devices (TSDD) including the Traffic Speed Deflectometer (TSD), Rolling Wheel Deflectometer (RWD), and the new Rapid Pavement...
Tester (RAPTOR) (Flintsch et al. 2012). A recent Strategic Highway Research Program 2 (SHRP2) study identified the TSD and the RWD as the most promising continuous deflection measurement devices (Flintsch et al. 2013).

Since 2008, the Louisiana Transportation Research Center (LTRC) has researched TSDD in pavement evaluation and management. Repeatability of RWD measurements, the effect of truck speeds, and the relationship between RWD and FWD deflection measurements and pavement conditions were evaluated (Elseifi et al. 2012). Non-linear regression models were developed to predict in-service structural capacity based on RWD (Elbagalati et al. 2016, Zihan et al. 2018). Cost-efficiency of RWD testing was evaluated in light of the added economic benefits (Elbagalati et al. 2016). In addition, a framework was developed to incorporate RWD measurements in the Louisiana Pavement Management System (PMS) at the network level and in Asphalt Concrete (AC) overlay design at the project level (Elbagalati et al. 2017, Zhang et al. 2016).

In the present study, the traffic speed deflectometer was evaluated in assessing in-service pavement structural conditions in Louisiana. The TSD can measure pavement deflection at traffic speeds, which enable large spatial coverage and provide continuous deflection profiles rather than measuring deflection at discrete points (Chai et al. 2016). Another advantage of TSD is, unlike RWD, it allows complete measurement of the deflection bowls.

The present study focused on evaluating TSD based on deflection measurements obtained from three field-testing programs conducted in District 05 of Louisiana, at the MnROAD test facility in Minnesota, and in Idaho. Based on these measurements, the study evaluated the feasibility and effectiveness of TSD for structural pavement evaluation at the network-level. In addition, the study developed and validated a model to predict in-service pavement Structural
Number (SN) based on TSD measurements. In-service pavement structural number may be used to identify structurally deficient pavement sections, which are in need of structural repair, and is also an important input in overlay pavement design. Finally, the present study also evaluated if it is sufficient to describe pavement conditions solely based on PMS surface indices that quantify cracking, rutting, and roughness.

1.1. Problem Statement

State agencies ought to realize the importance of incorporating pavement structural conditions in the selection of maintenance and rehabilitation (M&R) strategies along with functional indices. To measure in-service pavement structural capacity, surface deflection under a defined load is typically used. The traffic speed deflectometer has emerged as a continuous pavement deflection-measuring device as it operates at traffic speed and reduces lane closure and user delays. Yet, conventional algorithms for pavement evaluation are mostly based on FWD measurements. Considering the differences in deflection measuring mechanism between TSD and FWD, measurements from TSD needs to be evaluated and compared to FWD measured deflections. There are notable differences between TSD and FWD deflection measurements mechanisms including loading characteristics, load speed, and material responses to differing loading types.

Despite the significant advancements of TSDD devices (i.e., RWD and TSD), state agencies in Louisiana and throughout the US are inclined to believe that surface indices that quantify cracking, rutting, and roughness are generally sufficient to describe in-service pavement conditions (functional and structural conditions) and to make sound and cost-effective
maintenance and rehabilitation decisions. Therefore, there is a critical need to address the following research challenges:

- Feasibility of using TSD testing in assessing in-service pavements;
- Identify factors that may influence TSD measurements;
- Evaluation of TSD measurements as compared to FWD measured deflections;
- Pavement structural evaluation based on TSD measurements;
- The efficiency level of TSD to identify structurally deficient pavements; and
- Whether surface-measured indices are a true representation of overall pavement condition.

1.2. RESEARCH OBJECTIVES

The ultimate goal of this study is to assess the feasibility of using TSD measurements at the network-level for pavement structural evaluation in Louisiana. The following objectives were achieved:

a. To evaluate TSD measurements as compared to FWD measurements.

b. To identify factors influencing TSD measurements.

c. To develop a structural capacity indicator model based on TSD measurements.

d. To assess whether the use of surface indices only to identify structurally deficient sections is feasible.

1.3. RESEARCH APPROACH

Figure 1.1 presents the general layout of the adopted methodology to achieve the objectives of this study.
Figure 1.1. The layout of the research approach
The research approach adopted in this study consists of completing the following main tasks:

1.3.1. Literature Review (Task 1)

The literature was comprehensively reviewed as related to the following topics:

a) The loading mechanism of TSD as compared to conventional deflection measuring devices;

b) Research studies comparing TSD and FWD measurements;

c) TSD measurements variation with testing conditions and other factors;

d) Studies conducted for pavement structural evaluation based on TSDD;

e) Louisiana PMS surface condition data collection and pavement assessment systems.

1.3.2. Field Testing Program (Task 2)

Traffic speed deflectometer and falling weight deflectometer measurements were conducted in Louisiana in May 20 to 21, 2016. FWD and TSD measurements were conducted successfully with no significant problems to report. Due to the size limitations of the data collected in Louisiana, data were also obtained from FHWA for two recently completed testing programs conducted at the MnROAD test facility in Minnesota and in Idaho.

1.3.3. TSD Measurement Evaluation as Compared to FWD (Task 3)

After processing the raw data from the Louisiana experimental testing program, TSD measured deflections will be compared to FWD measured deflections. Two statistical methods will be used to demonstrate and compare TSD and FWD deflections i.e., significance test considering 95% confidence level and the Limit of Agreement Method. Selected test sites will be compared
individually (FWD vs. TSD) and global comparison of all test sites will be conducted for the two measurement methods. Furthermore, potential factors that could influence TSD field measurements such as pavement roughness and testing speed will be evaluated.

1.3.4. Develop a Structural Capacity Prediction Model (Task 4)

A structural capacity indicator model to predict in-service SN will be developed based on TSD deflections at 0.01-mile intervals of the road sections. A regression model will be developed and validated with the field measurements obtained from two regions with different climatic conditions considering SN as the dependent variable. Furthermore, the importance of incorporating structural capacity along with the functional indices in PMS decision-making will be evaluated using functional and structural indices.

1.3.5. Model’s Efficiency in Identifying Structurally Deficient Sections (Task 5)

The efficiency of the proposed SN model developed in Task 4 will be evaluated. The proposed model’s ability in identifying structurally deficient sections will be assessed as compared to structural deterioration identified from extracted cores and from FWD. Since cores were only available for the Louisiana data set, this analysis will be exclusively conducted for the Louisiana road sections.

1.3.6. Correlate Surface-Measured Indices and Structural Conditions (Task 6)

In this task, the relationships between surface indices and in-service pavement structural conditions predicted from RWD and TSD measurements will be comprehensively analyzed. For this task, RWD testing data in Louisiana and developed models as an indicator of the structural
condition were used. The level of accuracy expected when relying only on surface indices to predict structural deficiency will also be quantified.

1.4. Scope

To achieve the objectives of the study, TSD and FWD measurements were collected in District 05 of Louisiana and data were available from experimental programs conducted at the MnROAD research test facility and in Idaho. TSD measurements were compared with FWD deflection measurements to evaluate the level of agreement and difference between the two devices. Based on this evaluation, an SN predictive model was developed and validated to assess the structural conditions of in-service pavements. The model was then used to identify structurally sound and structurally deficient in-service pavements. Furthermore, the level of accuracy expected when relying only on surface indices to predict structural deficiency was quantified based on RWD and TSD estimated pavement structural indices using a statistical approach.

1.5. Organization of the Thesis

This thesis consists of five chapters. The first chapter INTRODUCTION presents an overall depiction of the thesis where the background and objective of this work and required tasks to achieve the study objectives were briefly discussed. The second chapter LITERATURE REVIEW presents a review of the existing literature on Traffic Speed Deflection Devices, related studies concerning pavement structural evaluation and pavement management systems in Louisiana. The third chapter METHODOLOGY discusses the field-testing programs and research approach briefly for each of the objectives that were mentioned in the first chapter. The fourth chapter ANALYSIS AND RESULTS represents the analyses and findings with
interpretations. The last chapter: SUMMARY AND CONCLUSIONS summarizes the study outcomes and draws a conclusion based on them. The thesis ends with recommendations that may further enhance the research goal.
CHAPTER 2. LITERATURE REVIEW

Pavement-conditions data collection by DOTD evolved from windshield surveys in the 1970s to videotaping the pavement surface in 1992, and then to automatic distress data collection in 1995. At present, distress data are collected and analyzed every two years for the road network in Louisiana. DOTD PMS data collection protocol includes a collection of roughness, rutting, cracking, patching, and faulting data from all the nine districts of Louisiana. Each control section is divided into 1/10th of a mile and distress data are collected and are reported at 0.1-mile interval along a control section. An index scale that ranges from zero to 100 is then used to report and describe pavement surface conditions where a value of zero represents very poor conditions and a value of 100 indicates excellent conditions (Elseifi and Elbagalati 2017).

The need for considering pavement structural conditions along with functional conditions has been recognized in the past decade by various state agencies, which supported the incorporation of a structural condition index in PMS to assist in decision-making processes. The traffic speed deflection devices (TSDD), continuous deflection measurement device, has emerged as a promising method to measure vertical surface deflection velocity continuously along a road section. The TSD consists of an articulated truck that uses a rear axle of 22,000 lbs. to load the pavement structure. The operational speed of the device is up to 60 mph; the TSD concept is based on the measurement of the deflection velocity rather than the absolute deflection at the road surface (Chai et al. 2016, Elbagalati et al. 2017).
2.1. **Traffic Speed Deflection Devices**

2.1.1. **Rolling Wheel Deflectometer (RWD)**

The RWD was developed by Applied Research Associates (ARA, Inc.) in collaboration with the FHWA Office of Asset Management. It consists of a 53-ft. long semitrailer applying a standard 18,000-lb. load on the pavement structure by means of a regular dual-tire assembly over the rear single axle (Briggs et al. 2000). The trailer is specifically designed to be long enough to separate the deflection basin, due to the 18-kip rear axle load, from the effect of the front axle load. The original setup of RWD used laser sensors housed in a thermal chamber to measure surface deflection due to the rear axle (Elseifi and Elbagalati 2017). The beam laser has four laser sensors that are used concurrently to measure the pavement surface deflection due to the rear axle based on optical trigonometry. However, a new deflection measurements protocol based on digital image analysis of the pavement surface was recently introduced in 2017. The analysis presented in this study is based on the original laser deflection system, which provides a deflection accuracy of 0.25 mils (Elbagalati 2017).

2.1.2. **Traffic Speed Deflectometer (TSD)**

In the early 2000s, the traffic speed deflectometer was introduced as a continuous deflection measuring device by Greenwood Engineering, which showed promising potential in assessing pavement structural conditions. The TSD is a continuous laser-based deflection measurement device that loads the pavement and measures vertical deflection velocity using Doppler lasers at four or six points (Chai et al. 2016, Elbagalati et al. 2017). At these discrete points, when the preliminary vertical surface deflection velocity collected by the Doppler lasers is divided by the instantaneous horizontal TSD vehicle speed, the deflection slope is obtained (Ramussen et al.
2008). The deflection slope is then converted to actual pavement deflection by curve fitting or numerical integration (Muller and Roberts 2013). Figure 2.1 illustrates the TSD vehicle used in the experimental program described in this study.

![Figure 2.1. TSD vehicle used in the experimental program in Louisiana](image)

As shown in Figure 2.1, the TSD consists of an articulated truck applying 22,000 lbs. on the rear axle; pavement response to the rear axle is measured as the vertical deflection velocity by fixed Doppler lasers mounted on a servo-hydraulic beam. The servo-hydraulic beam can move with the movement of the trailer, which allows the Doppler lasers to maintain a fixed height from the surface of the pavement. To address thermal fluctuations during testing, a constant 68°F (20°C) temperature is maintained in the servo-hydraulic beam. The TSD can collect one measurement every 0.00001-mile (0.787 in.) of road section at a rate of 1000 Hz while traveling at a traffic speed of up to 60 mph (Katicha et al. 2016). The maximum temporal resolution of the TSD is 1-millisecond and typical spatial resolution after processing is 0.0006-mile (Jenkins 2009). In the United Kingdom, TSD data are commonly reported at 0.006-mile and are stored at 0.0006-mile (39.37 in.) averages (Katicha et al. 2016).
The operation of the TSD is based on the vertical deflection velocity measurements rather than the actual surface deflections (Flintsch et al. 2013, Rada et al. 2011). The measured deflection velocity depends on the speed of the TSD; this dependency can be eliminated by dividing the vertical deflection velocity by the instantaneous horizontal TSD speed, which allows obtaining the deflection slope at each location of TSD measurement. The unit for measuring the deflection velocity and vehicle speed are millimeter per second and meters per second, respectively (Ferne et al. 2009). A good correlation has been reported by Simonin et al. between the center deflection and the calculated deflection slope (Simonin et al. 2005).

Along a pavement section, weak and sound locations can be identified through the deflection slopes but the estimation of the extent of weakness or soundness, which could assist in selecting maintenance and rehabilitation treatment methods, requires the pavement surface deflection. Pavement surface deflection can be calculated at any point from the center deflection (deflection under load) up to a radial distance by integrating the deflection slopes. Structural condition indicators such as the Base Damage Index (BDI) and Surface Curvature Index (SCI) can also be calculated using the calculated surface deflection (Katicha et al. 2013).

2.1.3. Deflection Measuring Techniques

The deflection measuring techniques for FWD and TSD are quite different. Even if both devices apply the same load magnitude, the measured deflection is conceptually different. The stationary FWD device applies an impact load to the surface of the pavement and measures the deflection at the center of the applied load and at multiple locations with varying distances from the center of the load. The FWD uses a circular plate to load the pavement as shown in Figure 2.2(a). In contrast, the TSD operates at a traffic speed up to 60 mph and loads the pavement through its rear axle. Over the right wheel, Doppler lasers are mounted to measure the deflection velocity
between the dual tires. Doppler lasers measure the deflection velocity at the midpoint between
the tires as shown in Figure 2.2(b).

![FWD testing using a circular plate (Elseifi et al. 2011)](image1)

![TSD measuring deflection velocity between the dual tires (Nasimifar et al. 2017)](image2)

**Figure 2.2. Deflection measuring technique of FWD and TSD**

While FWD applies a circular loading with uniform contact pressure, TSD applies an elliptical-
shape loading using regular tires with non-uniform contact pressure. Hence, pavement responses
are expected to be different due to the different loading mechanisms for TSD and FWD
(Nasismifar et al. 2017). It is also noted that a dynamic load of a five-axle truck-semi trailer can
vary by almost 33% of the load of that truck when measured on a static scale (Rabe et al. 2013).
As previously noted, TSD measurements are reported as deflection slopes (calculated by
dividing the vertical deflection velocity by the horizontal velocity of TSD), whereas FWD
measures the actual vertical deflection.

### 2.1.4. TSD and FWD Comparison

As previously noted, there is a fundamental difference between the TSD and FWD loading
mechanisms, which could lead to notable differences in the measured deflection values obtained
from these two devices. With respect to loading operations, TSD operates with a moving load at
traffic speeds, whereas, FWD load is stationary. Furthermore, TSD measured deflections could be highly influenced by the irregularities in the surface such as roughness and other pavement distresses (Flintsch et al. 2013, Rada and Nazarian 2011). Previous studies compared the SCI and BDI derived from TSD slope measurements and FWD deflection measurements (Katicha et. al. 2014). The study found a significant bias between these two devices and recommended using the Limit of Agreement (LOA) method to compare the measurements from the two devices measurements (Katicha et. al. 2014). In Australia and New Zealand, a research study found a strong correlation between TSD and FWD deflection measurements (Roberts et al. 2014). Another study compared the TSD and FWD measured deflections in Virginia (Katicha et al. 2017). The comparison indicated a similar trend in deflections between the two devices. The study suggested that the structural conditions along the tested road were successfully reflected in the measurements of the two devices; see Figure 2.3.

Figure 2.3. Comparison of TSD and FWD D0 in Virginia (Katicha et al. 2017)
2.1.5. TSD Measurement Dependency on Speed

In a previous study, the variation of TSD measurements with its operating speed was investigated (Rada et al. 2016). TSD testing was conducted at two different traffic speeds of 30 and 45 mph on low volume roads (LVR) and at 45 and 60 mph on the Mainline. Results indicated that the measured deflection is sensitive to the speed of loading. It was found that the coefficients of variation of the deflection slopes were about 24% less at 30 mph than at 45 mph along the LVR and were around 38% greater at 60 mph than the COVs at 45 mph on the Mainline. The developed graphs for the COVs in the LVR and Mainline are shown in Figure 2.4 and Figure 2.5. However, another research study concluded that TSD measures “real” pavement response, even at low speed (<20 mph) (Kannemeyer et al. 2014).

Figure 2.4. Comparison of deflection slope COVs in LVR (Rada et al. 2016)

Figure 2.5. Comparison of deflection slope COVs in the Mainline (Rada et al. 2016)
2.1.6. TSD Measurement Dependency on Pavement Structure

The correlation of TSD slope measurements to pavement stiffness and surface roughness was investigated in a previous study (Rada et al. 2016). TSD slope measurements were collected for different pavement sections and the COVs of the deflection slopes were calculated for each section. The average FWD central deflection was also measured for these sections. Pavement stiffness was represented by the FWD central deflection in the analysis; the greater the FWD central deflection, the lower the pavement stiffness. The authors reported that the COVs from the first four sensors decreased with the increase in FWD central deflection for the flexible pavement sections; see Figure 2.6. For the rigid pavement sections, the COVs of the deflection slopes were found to be relatively higher than for the far sensor locations; see Figure 2.7. However, in our opinion that the reported trends were not strongly evident, possibly due to the variation in the pavement structure concurrently with the variation in surface roughness. Pavement surface roughness was also correlated to the COVs of the TSD measurements. However, no strong correlation was observed, as shown in Figure 2.8 and Figure 2.9.

Figure 2.6. TSD measurement variation with stiffness on flexible pavement (Rada et al. 2016)
Figure 2.7. TSD measurement variation with stiffness on rigid pavement (Rada et al. 2016)

Figure 2.8. TSD measurement variation with roughness of flexible pavement (Rada et al. 2016)

Figure 2.9. TSD measurement variation with roughness of rigid pavement (Rada et al. 2016)
As previously noted, axle load can be dynamically amplified due to vehicle suspension type, traveling speed, tire contact pressure, tire thread pattern, axle and wheel configuration, and pavement stiffness. It was found that a rough pavement surface could cause a 50% increase in the static axle load, which explains the accelerated deterioration of rough pavements. A number of studies used a Dynamic Load Coefficient (DLC) to represent the dynamic amplification of static axle load. Statistically, DLC can be defined as one standard deviation from the mean static axle load. The typical value for DLC has been reported as 0.05 to 0.4 from previous studies (Zofka et al. 2014). For different vehicle suspension types and tire configurations, the DLC was correlated to different parameters as follows:

\[
\text{DLC}^* = \frac{\kappa \cdot R \cdot \text{IRI}}{2}
\]  

(2.1)

where,

\(\text{DLC}^*\) = DLC value for the normal distribution of the axle load;

\(\kappa\) = coefficient related mostly to the suspension type (assumed \(\kappa = 0.0016\));

\(R\) = truck speed \([\text{km/h}]\); and

\(\text{IRI}\) = International Roughness Index \([\text{m/km}]\).

From Equation (2.1), it can be noticed that with the increase in IRI and traffic speed, the DLC also increases, which causes dynamic amplification of the load. A probabilistic approach using 10,000 Monte Carlo simulation trials has been conducted to account for the random effects of pavement roughness. A normal distribution of the dynamic axle load for TSD vehicle was developed using Equation (2.1); see Figure 2.10. \(F_{\text{stat}}\) is the average static axle load and \(F_{\text{left}}\) and \(F_{\text{right}}\) account for the left and right side of the vehicle. From this distribution, it is observed that the static axle load increases by around \(\pm 20\%\) due to surface roughness (Zofka et al. 2014).
The need for considering structural conditions along with functional conditions in pavement management has been recognized in the past decade by various state agencies. The FWD allows practitioners to assess the structural conditions of in-service pavements (Zofka et al. 2014). Research studies have also developed methodologies to evaluate the structural conditions of in-service pavement and its structural number based on surface deflections measured using FWD and RWD. A recent pooled funded study has also developed a methodology for predicting the Effective Structural Number (SN$_{eff}$) from TSD measurements using Rohde’s (1994) method, which includes estimating the Structural Index of Pavement (SIP) using Equation (2.2):

$$SIP = D_0 - D_{1.5Hp}$$

(2.2)

where,

$D_0 =$ peak deflection under the 9,000-lbs. load;
D_{1.5Hp} = \text{deflection at lateral distance 1.5 times the pavement depth}; and
Hp = \text{pavement depth (thickness of all layers above the subgrade)}.

Afterward, SN_{eff} is predicted from the following Equation (2.3):

\[ SN_{eff} = K_1 \cdot \text{SIP} \cdot H_p^{k_2} \cdot K_3 \]  \hspace{1cm} (2.3)

where,

For asphalt pavements, \( k_1 = 0.4728 \), \( k_2 = -0.4810 \), and \( k_3 = 0.7581 \).

In the developed methodology, \( D_0 \) was corrected to a reference temperature of 68°F (20°C) using the procedure described by Lukanen et al. (2000). The pooled funded study also developed thresholds for assessing pavement structural conditions based on the derived parameters, SCI_{300} and Deflection Slope Index (DSI), from TSD measured deflections. The thresholds were used to classify pavement conditions as good, fair and poor. SCI_{300} and DSI were derived from TSD deflections based on Equations (2.4) and (2.5):

\[ SCI_{300} = D_0 - D_{300} \]  \hspace{1cm} (2.4)
\[ DSI = D_{100} - D_{300} \]  \hspace{1cm} (2.5)

where,

\( D_0 = \text{deflection at the point of load application (mid-point between the dual tires)} \);
\( D_{100} = \text{deflections at 100 mm (3.93 in.) from the center of the applied load} \); and
\( D_{300} = \text{deflections at 300 mm (11.81 in.) from the center of the applied load} \).
SCI\textsubscript{300} and DSI were corrected to a reference temperature of 70°F according to the methodology developed by Rada et al. (Katicha et al. 2017). The suggested thresholds for pavement conditions evaluation based on these parameters are shown in Table 2.1.

Table 2.1. Thresholds for SCI\textsubscript{300} and DSI from TSD measurements

<table>
<thead>
<tr>
<th>Road Category</th>
<th>AC layer thickness, in.</th>
<th>Threshold for Poor</th>
<th>Threshold for Fair</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>SCI\textsubscript{300} (mil)</td>
<td>DSI (mil)</td>
</tr>
<tr>
<td>Interstate</td>
<td>&gt;9</td>
<td>3.7</td>
<td>3.0</td>
</tr>
<tr>
<td>Primary</td>
<td>6-9</td>
<td>6.2</td>
<td>5.2</td>
</tr>
<tr>
<td>Secondary</td>
<td>3-6</td>
<td>9.7</td>
<td>7.7</td>
</tr>
</tbody>
</table>

The pool-funded study also compared the SNeff estimated from TSD deflections with the PMS SNeff from Pennsylvania and found a significant discrepancy between TSD SNeff and PMS SNeff, see Figure 2.11. Pennsylvania PMS SNeff is calculated according to the AASHTO 1993 design method with a reduction of layer coefficients with pavement age. As suggested by the authors, this may indicate that PMS SNeff does not accurately predict the effective pavement SN, since a good agreement was found between FWD and TSD deflections (Katicha et al. 2017).

Figure 2.11. Comparison between TSD SNeff and Pennsylvania PMS SNeff
The AASHTO equation for estimating the SN requires trial and error and numerical methods, which makes it complicated to use (Gedafa et al. 2014). An SN model was developed by Gedafa et al. based on FWD center deflection measurements along with other performance indices data from PMS in Kansas. The road network was divided into 23 categories and different regression models were developed for each of the road categories using the center deflection, pavement depth, and surface condition indices. Afterward, an overall SN model was proposed with a coefficient of determination ($R^2$) of 0.77. It was suggested that either RWD or FWD center deflection measurements could be used in the overall model:

$$SN= 6.3763-0.3364d_0+0.0062d_0^2-0.0805D+0.01D^2-0.0008(d_0*D)-0.4115 \log{(EAL)}
+0.1438(\log{(EAL)})^2 + 0.0836ETCR-0.0091 EFCR+0.0004 EFCR^2 -0.4061 \text{Rut}$$

(2.6)

where,

SN= pavement structural number;

$d_0$= center deflection (mils);

D= pavement depth (in.);

EAL = Equivalent standard daily traffic;

ETCR=EFCR=equivalent fatigue/transverse cracking; and

Rut=rut depth (in).

An SN-predictive model was developed by Elbagalati et al. based on the RWD average center deflection, deflection standard deviation along the pavement length at 0.1-mile interval, asphalt layer thickness, and traffic volume. The model accuracy was evaluated and indicated a Root-Mean-Square Error (RMSE) of 0.8 and a coefficient of determination ($R^2$) of 0.8. The model was then used to identify structurally deficient pavement sections assuming a 50% loss in
structural capacity (Elbagalati et al. 2016). Equation (2.7) presents the developed SN model based on RWD deflection measurements:

\[
SN_{RWD0.1} = -14.72 + 27.55 \cdot \left( \frac{A_{cth}}{D_0} \right)^{0.04695} - 2.426 \cdot \ln SD + 0.29 \cdot \ln ADTPLN
\]  

(2.7)

where,

\(A_{cth}\) = Asphalt layer(s) thickness of the pavement structure (in.);
\(D_0\) = Avg. RWD deflection measured each 0.1-mile (mils);
\(SD\) = Standard deviation of the RWD deflection each 0.1-mile;
\(ADTPLN\) = Average Annual Daily traffic per lane (vehicle/day);

\(SN_{RWD0.1}\) = Pavement SN based on RWD measurements defined each 0.16 km (0.1 mi.).

Schnoor et al. assessed flexible pavement structural conditions using a simple SN model, which was developed based on derived parameters from FWD deflection measurements; i.e., area under the pavement profile and the base layer index; see Equation (2.8) (Schnoor et al. 2012):

\[
SN = e^{5.12} A_{UPP}^{-0.78} BLI^{0.31}
\]

(2.8)

where,

\(A_{UPP}\) = Area under pavement profile; and

\(BLI\) = Base layer index.

Other noteworthy SN models were also developed based on FWD and RWD deflection measurements and are presented in Table 2.2 (Schnoor et al. 2012).
Table 2.2. Developed SN models based on FWD and RWD measurements

<table>
<thead>
<tr>
<th>Method</th>
<th>SN models</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>FWD center deflection-based SN model</td>
<td>SN = 6.3763 − 0.3364 d0 + 0.0062 × d0^2 − 0.0805 × D + 0.01 × D^2 − 0.0008 × (d0 × D) − 0.4115 × log (EAL) + 0.1438 × (log(EAL))^2 + 0.0836 × ETCR − 0.0091 × EFCR + 0.0004 × EFCR^2 − 0.4061 × Rut</td>
<td>SN= pavement structural number; d0= center deflection (mils); D= pavement depth (in.); EAL = equivalent standard daily traffic; EFCR/ETCR= equivalent fatigue/transverse cracking; and Rut=Rut depth (in.).</td>
</tr>
<tr>
<td>Backcalculated Moduli</td>
<td>SN = ( \sum_{i=1}^{n} h_i \alpha_g \left( \frac{E_i}{E_g} \right)^{1/3} )</td>
<td>( \alpha_g = ) layer coefficient of standard materials; ( E_i = ) layer resilient modulus (MPa); ( E_g = ) layer resilient modulus of standard materials (MPa); and ( h_i = ) layer thickness (in.).</td>
</tr>
<tr>
<td>AASHTO NDT Method</td>
<td>( D_0 = \frac{1.5P}{\pi l r} \left( \frac{0.045 HP}{SN^3} - \frac{1}{1 + (HP/\pi l r)^{1/2}} \right) ) + ( \frac{1}{ESG(1 + 40000SN^2/\pi^2l^2E_{SG}^2)^{1/2}} )</td>
<td>( D_0 = ) the peak FWD deflection (in.); ( P = ) FWD load (lbs.); ( H_p = ) layer thickness (in.); ( l_r = ) load radius (in.); ( E_{SG} = ) subgrade modulus (psi).</td>
</tr>
<tr>
<td>AASHTO Method II</td>
<td>SN = 1.69 + ( 842.8 \left( \frac{D_0 - D_{1500}}{D_{900}} \right) + \frac{42.94}{D_{900}} )</td>
<td>( D_0 = ) the peak deflection (microns); ( D_{900} = ) Deflection at 900 mm from loading (microns); ( D_{1500} = ) Deflection at 1500 mm from loading (microns).</td>
</tr>
<tr>
<td>Jameson’s formula</td>
<td>SNC = a0 ( (D_0)^{a1} )</td>
<td>SNC = modified structural number; ( a0, a1 = ) Asgari coefficients; ( D_0 = ) Peak deflection (mm).</td>
</tr>
<tr>
<td>Asgari’s formula</td>
<td>SNC = a0 ( (D_0)^{a1} )</td>
<td>SNC = modified structural number; ( a0, a1 = ) Asgari coefficients; ( D_0 = ) Peak deflection (mm).</td>
</tr>
<tr>
<td>The Wimsatt formula</td>
<td>SN_{eff} = 0.0045 ( (D)E_p^{0.333} )</td>
<td>( D = ) Total layer thickness; ( E_p = ) Existing pavement modulus of the layers above subgrade.</td>
</tr>
</tbody>
</table>

As the use of FWD is limited at the network level and with the acceptance of continuous deflection measurement devices in many countries, a model is needed to predict SN from continuous devices such as TSD. The present study developed an SN-prediction model based on TSD deflections; the model can also be used to identify structurally deficient pavements. Such
information would benefit state agencies at the network-level in decision-making processes and
in avoiding inaccurate selection of Maintenance and Rehabilitation (M&R) activities.

2.3. LIMIT OF AGREEMENT METHOD

Limit of Agreement (LOA) is a statistical method introduced by Bland and Altman, which is
widely used to evaluate the difference between two sets of measurements by two independent
devices (Bland and Altman 1986). The error between each set of measured data by the two
devices can be calculated based on Equations (2.9) and (2.10) as follows:

\[ y_{1i} - y_{2i} = (s_{1i} - s_{2i}) + (e_{1i} - e_{2i}) \]  \hspace{1cm} (2.9)

where,

\[ y_{1i} = \text{Measurement at location } i \text{ obtained from device 1}; \]
\[ y_{2i} = \text{Measurement at location } i \text{ obtained from device 2}; \]
\[ s_{1i} = \text{Actual value at location } i \text{ obtained from device 1}; \]
\[ s_{2i} = \text{Actual value at location } i \text{ obtained from device 2}; \]
\[ e_{1i} = \text{Error in measurement at location } i \text{ for device 1}; \]
\[ e_{2i} = \text{Error in measurement at location } i \text{ for device 2}. \]

\[ D_i = B_i + E_i \]  \hspace{1cm} (2.10)

where,

\[ D_i = \text{Difference in measurements between two devices}; \]
\[ B_i = \text{Difference of systematic error of the two devices}; \]
\[ E_i = \text{Difference of the random error of the two devices}. \]
Bi can be considered constant for simple cases and if e_{i1}, e_{i2} are normally distributed as N (0, σ_1) and N (0, σ_2) respectively, then E_i can also be assumed normally distributed, N (0, σ). Therefore, in such cases, D_i will also be normally distributed as N (B, σ) where B is the constant difference of systematic error of the two devices and σ is the standard deviation. The standard deviation can be calculated as follows:

\[ B = \frac{\sum_{i=1}^{N} B_i}{N} \]  \hspace{1cm} (2.11)

\[ \sigma^2 = \frac{\sum_{i=1}^{N} (B-B_i)^2}{N-1} \]  \hspace{1cm} (2.12)

The plot of the difference between the measurements is evaluated by D_i versus the average measurements of the two devices. This type of plot is very useful in identifying the lack of agreement between two device measurements and the relationship between true measurements and the error in device measurements. However, if the true value is unknown, the mean of the two devices can be assumed as the mean value. For example, for a set peak expiratory flow rate (PEFR) data measured by two flow-measuring meters, the plot in Figure 2.12 presents the concept of the Limit of Agreement method. According to this figure, one may conclude that the two meters show a considerable lack of agreement up to a difference of 800 l/min. If there is no relationship between the measurement difference and the mean, the lack of agreement can also be summarized using the calculated bias from the two data sets. The difference between the two data sets are expected to be within the confidence limits constructed for the data set; typically, within \( d-2s \) and \( d+2s \) (Figure 2.12) or \( d-1.96s \) and \( d+1.96s \) for normally distributed differences, where \( d \) is the mean difference and \( s \) is the standard deviation of the differences (Bland and Altman 1986).
The Louisiana Department of Transportation and Development (LaDOTD) PMS maintains an extensive database that contains pavement distresses and performance data for each state highway. Pavement performance data are available in the LaDOTD pavement management system for the period ranging from 1995 to the present time. The PMS data are based on pavement condition measurements that are collected biennially using the Automatic Road Analyzer (ARAN®) system that provides a continuous assessment of the road network. Conditions of the pavement are assessed using cracking, rutting, roughness, and patching. In addition, video crack surveys are collected once every two years and are available for each state highway in Louisiana. Collected data are reported every 1/10th of a mile and are analyzed to calculate the Pavement Condition Index (PCI) on a scale from zero to 100. A number of

Figure 2.12. Example of Limit of Agreement (LOA) method (Bland and Altman 1986).

2.4. LOUISIANA PMS SURFACE INDICES

The Louisiana Department of Transportation and Development (LaDOTD) PMS maintains an extensive database that contains pavement distresses and performance data for each state highway. Pavement performance data are available in the LaDOTD pavement management system for the period ranging from 1995 to the present time. The PMS data are based on pavement condition measurements that are collected biennially using the Automatic Road Analyzer (ARAN®) system that provides a continuous assessment of the road network. Conditions of the pavement are assessed using cracking, rutting, roughness, and patching. In addition, video crack surveys are collected once every two years and are available for each state highway in Louisiana. Collected data are reported every 1/10th of a mile and are analyzed to calculate the Pavement Condition Index (PCI) on a scale from zero to 100. A number of
Threshold values are also used to trigger a specific course of maintenance and rehabilitation (M&R) actions based on surface indices (Briggs et al.).

For flexible pavements, the random cracking index (RNDM) encompasses all random cracks, which include thermal, reflective, longitudinal, block, and cement-treated reflective cracks. The equations used to calculate the alligator cracking index (ALCR), the random cracking index (RNDM), the roughness index (ROUGH), and the rutting index (RUTT) are as follows (Elbagalati 2017):

\[
ALCR = \text{MIN} (100, \text{MAX} (0, 100 - \text{ALGCRK\_L DEDUCT} - \text{ALGCRK\_M DEDUCT} - \text{ALGCRK\_H DEDUCT}))
\]  

(2.13)

where,

\(ALCR\) = Alligator cracking index

\(\text{ALGCRK\_L DEDUCT}, \text{ALGCRK\_M DEDUCT}, \text{and ALGCRK\_H DEDUCT}\) = deduct point due to alligator cracks for low, medium, and high severity of the cracks, respectively.

\[
RNDM = \text{MIN} (100, \text{Max} (0.100 - \text{DPL} - \text{DPM} - \text{DPH}))
\]  

(2.14)

where,

\(DP\) = deduct point due to random cracks; and

Subscripts L, M, and H refer to the low, medium, and high severity of the cracks, respectively.

\[
ROUGH = \text{MIN} (100, 100 - ((\text{Avg\_IRI} * (1/5)) - 10))
\]  

(2.15)

where,
ROUGH = Roughness Index

Avg. IRI = Avg. International Roughness Index (inches/mile)

\[
RUTT = \text{MIN} \left( 100, 100 - \left( R_{\text{Avg}} \times \left( \frac{10}{0.125} \right) - 10 \right) \right)
\]

(2.16)

where,

RUTT = Rutting Index

R_{\text{Avg.}} = \text{Average Rutting (inch.)}
CHAPTER 3, METHODOLOGY

Nondestructive testing of in-service pavements was conducted using both TSD and FWD in District 05 of Louisiana. TSD and FWD measurements were also obtained from FHWA for recently conducted testing programs at the MnROAD test facility and in Idaho. The soundness of TSD measurements was evaluated and data were processed and filtered to calculate the surface deflections. After processing and filtering the TSD raw measurements, the deflection data were compared to the FWD deflection measurements to evaluate whether the two sets of measurements are statistically equivalent or different. TSD deflection data were also used to develop an SN-predicting model and the model’s efficiency in identifying structural deficient pavement locations was evaluated by comparing the model prediction to the conditions of extracted cores from the pavement sections. To this end, surface indices data collected over pavement service life were compared and evaluated based on in-service structural condition estimated from TSD measurements (current study) and RWD measurements (earlier study) in Louisiana. The level of accuracy expected when relying only on surface indices to predict structural deficiency was quantified demonstrating risk analysis and associated cost implication was also assessed.
3.1. Field Testing Program

Traffic speed deflectometer and falling weight deflectometer measurements were conducted in Louisiana from May 20 to 21, 2016. FWD and TSD measurements were conducted successfully with no significant problems to report. Due to the size limitations of the data collected in Louisiana, data were also obtained from FHWA for two recently completed testing programs conducted at the MnROAD test facility in Minnesota and in Idaho. Earlier in 2009, a comprehensive RWD testing was conducted in Louisiana. The Louisiana Transportation Research Center (LTRC) has researched TSDD in pavement evaluation and management. Repeatability of RWD measurements, the effect of truck speeds, and the relationship between RWD and FWD deflection measurements and pavement conditions were evaluated (Elseifi et al. 2012).

3.1.1. TSD Testing Program in Louisiana

In 2016, a TSD device operated by the Australian Road Research Board (ARRB), known as iPAVe, was used to measure vertical deflection velocity, horizontal speed of the vehicle, air temperature, and pavement surface temperature in six parishes of District 05 in Louisiana. Measurements were collected for 13 control sections at 0.01-mile interval. FWD measurements were also collected for the same control sections at 0.1-mile interval for the evaluation and comparison with TSD measurements. The 13 selected sites in District 05 are presented in Figure 3.1.
Figure 3.1. Locations of the TSD road segments in Louisiana (District 05)

The pavement surface and air temperature were recorded to an accuracy of +/-1°F and were reported within the TSD dataset. These measurements were made with a calibrated air temperature probe situated beneath the trailer chassis, above the ballast weight for the ambient air, and a calibrated infrared temperature sensor that measures pavement surface temperature in the outer wheel path location. The load is ‘static’ and is comprised of the base trailer mass itself, plus the mass of the main ballast weight of 7220 lbs. located under the belly of the trailer, and a small ballast weight of 475 lbs. situated underneath the rear of the trailer. These weights are balanced to provide a suitable center of gravity for the trailer road handling, as well as the nominal equal load over each wheel set. Figure 3.2 shows a typical arrangement of loading in the TSD device. It is to be noted that for the testing in Louisiana, the rear ballast weight (475 lbs.) was removed to comply with axle weight regulations, which resulted in a reduced load of
10,000 lbs. on each wheel set. Strain gauges were mounted on the rear axle to measure the bending moment on the loaded axle on both the left and right side. The load data were collected continuously and were averaged over the selected report interval, and were converted into a mass measurement, for both left and right-side axles. The mass measure was derived from a load vs. signal equation derived from the strain gauge outputs and was not a direct load cell weight or force measurement. The tolerance between actual and measured strain (weight) in a static setting is ±440 lbs., which is acceptable considering the weight of the trailer, air pressure, and suspension balancing valving, and engineering tolerance in the iPAVe chassis/suspension construction.

Figure 3.2. Typical loading configurations in the TSD device

The nominal load was set at 20,000 lbs. on the axle and was distributed on the left and right sides depending on the movement of the trailer Center of Gravity (CoG), the cross fall, and the grade of the road. Therefore, it varied dynamically within a range of a few percentages as the TSD traveled down the road. High-resolution horizontal velocity measurements (i.e., the travel speed of the iPAVe) are critical to deflection slope calculations. Distance and velocity are measured using a specialized odometer wheel assembly. Having a dedicated Distance Measuring
Instrument (DMI) increases accuracy and limits error induced due to physical factors, such as tire loading, tracking, tire pressure, and thermal expansion. The overall accuracy of the DMI is defined with an error of less than +/-0.1% and subsequent bias of less than 0.1%. The same odometer pulse count is used for all distance measurements within the iPave system. Measurements were reported for the 13 control sections at 0.01-mile intervals. FWD measurements were also collected for the same control sections at 0.1-mile intervals for the evaluation and comparison with TSD measurements.

3.1.2. TSD Loading Conditions for Louisiana

Traveling at normal traffic speed, TSD loads the pavement using its rear axle tires. The articulated Doppler lasers over the right wheel of the rear axles measure the deflection velocity along the midline between these dual tires. The applied load for these tires was reported through strain gauge measurements. TSD loading variation under static and dynamic conditions is discussed in this section.

TSD Load and Tire Pressure

The applied load by the TSD, loaded area of pavement surface, and tire contact pressure at static condition were measured. As shown in Figure 3.3, TSD applied a load of 20,360 lbs. on its rear axle and distributed this load evenly over its left and right dual tires producing a load of 9,800 lbs. and 10,560 lbs. on the left and right sides, respectively. The contact tire pressure was reported at 115 psi in static conditions. It is to be noted that the ARRB TSD used in the testing program was intentionally slightly biased towards the right dual tire with a greater load to increase the deflection since it measures the deflection along the midline between the right dual tires.
Assuming the load on the right and left dual tire configurations is evenly distributed over each tire, the load on each tire shown in Table 3.1 can be calculated.

<table>
<thead>
<tr>
<th>Tire location</th>
<th>Loads (lbs.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outer Left</td>
<td>4,900 lbs.</td>
</tr>
<tr>
<td>Inner Left</td>
<td>4,900 lbs.</td>
</tr>
<tr>
<td>Outer Right</td>
<td>5,280 lbs.</td>
</tr>
</tbody>
</table>

The loaded area and tire dimensions were calculated by measuring the footprint from the outside the tire as shown in Figure 3.4. Tire longitudinal dimension (travel direction) was measured at 7.48 in. (190 mm) and at 9.45 in. (240 mm) in the transverse direction; see Figure 3.4. The spacing between the two tires was measured at 4.33 in. (110 mm).
As previously noted, TSD loading, tire pressure, and loaded area vary significantly in dynamic conditions at the time of deflection velocity measurements. The loading profile for each processed data point was obtained through the strain gauges measurements.
3.1.3. FWD Testing Program in Louisiana

FWD testing was conducted in Louisiana within 24 hours of the TSD measurements to maintain the consistency in pavement and environmental conditions. FWD measurements were reported for the 13 test sites at an interval of 0.1-mile. Two loading drops were conducted for FWD at all test locations. The two drops varied within a load range of 9,700 to 10,200 lbs. and 24,200 to 24,700 lbs., respectively. The obtained deflections due to the drop with a load of 9,700 to 10,200 lbs. were used in this study and were normalized to a load of 9,000 lbs. Along with the deflection, FWD also measured the surface temperature during testing. Table 3.2 presents the details of the 13 test sections evaluated in the Louisiana testing program.
Table 3.2. General descriptions of the 13 test sites in Louisiana

<table>
<thead>
<tr>
<th>Site ID</th>
<th>Control Section</th>
<th>Route</th>
<th>Parish</th>
<th>Pavement Type</th>
<th>Type of Treatment</th>
<th>Last Treatment Year</th>
<th>TSD Test Site Log-miles</th>
<th>Traffic</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>067-08</td>
<td>LA 34-1</td>
<td>Ouachita</td>
<td>Asphalt</td>
<td>A7- Asph Surf Treat</td>
<td>2003</td>
<td>5.55 - 6.95</td>
<td>2,400</td>
</tr>
<tr>
<td>2</td>
<td>067-09</td>
<td>LA 34-2</td>
<td>Ouachita</td>
<td>Asphalt</td>
<td>A3- Asph Ovly Pvmnt</td>
<td>2001</td>
<td>3.35 - 4.75</td>
<td>11,714</td>
</tr>
<tr>
<td>3</td>
<td>451-05</td>
<td>I-20 eb</td>
<td>Lincoln</td>
<td>Composite</td>
<td>A3- Asph Ovly Pvmnt</td>
<td>2005</td>
<td>22.25 - 23.95</td>
<td>35,528</td>
</tr>
<tr>
<td>4</td>
<td>326-01</td>
<td>LA 594-2</td>
<td>Ouachita</td>
<td>Asphalt</td>
<td>Z1- RCND AGGR SURF</td>
<td>2003</td>
<td>5.05 - 6.45</td>
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</tr>
<tr>
<td>5</td>
<td>324-02</td>
<td>LA 616</td>
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<td>Asphalt</td>
<td>A1-Asphalt New Pvmnt</td>
<td>1995</td>
<td>3.55 - 4.95</td>
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<td>US 425</td>
<td>Richland</td>
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<td>A5- AC Ovly/In-place Base</td>
<td>2008</td>
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<td>LA 582</td>
<td>E Carroll</td>
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<td>ZA- Asphalt Pavement Rehab</td>
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<td>LA 594-1</td>
<td>Ouachita</td>
<td>Asphalt</td>
<td>A3- Asph Ovly Pvmnt</td>
<td>2006</td>
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<td>3,800</td>
</tr>
<tr>
<td>13</td>
<td>451-08</td>
<td>I-20 wb</td>
<td>Madison</td>
<td>Composite</td>
<td>A6- AC Ovly Rubblized Pvmnt</td>
<td>2013</td>
<td>29.3-30.8</td>
<td>25,600</td>
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</table>

Note: All sites were tested in the Primary direction except Site ID 2 and 13.

(Table cont’d.)
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<tr>
<th>ID</th>
<th>Surface Type</th>
<th>Base Type</th>
<th>Layer Thicknesses (in.)</th>
<th>Pavement Group</th>
<th>Core Conditions</th>
<th>IRI [2015] in./mile</th>
<th>PCI [2015]</th>
<th>Condition (PCI)</th>
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<td>111.6</td>
<td>77.9</td>
<td>Fair</td>
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<tr>
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<td>Stripping</td>
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<td>89.3</td>
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<td>97.6</td>
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<td>92.7</td>
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<td>Separation</td>
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<td>Good</td>
</tr>
<tr>
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</tr>
<tr>
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<td>Thin</td>
<td>---</td>
<td>95.0</td>
<td>91.0</td>
<td>Good</td>
</tr>
<tr>
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<td>Stabilized Granular</td>
<td>8.5 8 19.5</td>
<td>Thick</td>
<td>---</td>
<td>67.5</td>
<td>90.2</td>
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<tr>
<td>13</td>
<td>Asphalt</td>
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<td>5.25 18 0</td>
<td>Medium</td>
<td>Stripping</td>
<td>49.8</td>
<td>95.8</td>
<td>Very good</td>
</tr>
</tbody>
</table>
3.1.4. Idaho Testing Program

In September 2015, TSD measurements were conducted in Idaho by Greenwood Engineering of Denmark under FHWA pooled funded project TPF 5(282) (Maser et al. 2017). TSD and FWD measurements were conducted for one road segment that was 13.4 mile in length. TSD measurements were reported at 0.006-mile intervals and FWD measurements were reported at 100 ft. (0.02-mile) interval. Data were reported in one direction and six Doppler lasers were located at a distance of 3.9, 7.9, 11.8, 23.6, and 59 in. ahead of the rear axle of the vehicle to measure deflection velocity at different offsets. Another sensor acting as a reference laser was placed at a distance of 138.0 in. from the rear axle, which is beyond the deflection basin distance. Lasers were positioned on a beam that moved up and down in the opposite direction of the trailer movement to maintain a constant height from the road surface. A constant trailer temperature of 68°F (20°C) was maintained by a temperature control system in order to prevent thermal distortion of the steel beam.

The objective of the FWD testing program was to compare the measured deflections to TSD measurements. FWD measurements were collected within a month of the TSD survey using a Dynatest truck-mounted deflectometer. FWD testing was conducted on a 2-mile long road segment resulting in more than 100 FWD data points with an interval of 100 ft. Five load drops were conducted at each test location of the selected road segment. Out of the five drops, three were conducted at 12,000 lbs. and the remaining two were conducted at 9,000 lbs. Vertical deformation of the pavement surface due to the FWD drops was measured by seven sensors located at 0.0, 8.0, 12.0, 18.0, 24.0, 36.0, and 60.0 in. from the center of the load. Temperature of the pavement surface and air temperature, and GPS data were also collected to assist in the analysis.
3.1.5. MnROAD Testing Program

FWD and TSD measurements were collected at the MnROAD facility in Minnesota (Elseifi and Elbagalati 2017). The surveyed road network consisted of a 3.5-mile mainline roadway (ML) with 45 sections and with “live traffic” as part of Interstate 94 near Albertville, Minnesota. In addition, a 2.5-mile closed-loop low volume roadway (LVR) consisting of 28 sections was also surveyed; the section lengths were typically about 500 ft. In addition to the test sections along the mainline and low volume road of the MnROAD, an 18-mile segment in Wright County was also tested. The segment is located about 20 miles from the MnROAD facility and was divided into nine sections.

Testing was conducted using the TSD, RWD, and the Euro-consult Curvimeter. FWD was also conducted and was used as a reference for comparison and evaluation purposes. Tested sections varied between flexible pavements, rigid pavements, and composite pavement sections. Yet, the present study focused on the use of the TSD measurements in conducting backcalculation analysis of flexible pavements layer moduli, therefore, only TSD and FWD data collected on flexible pavements were considered. The flexible pavement test segments at which both FWD and TSD measurements were conducted consisted of 16 sections; six in the main line and 10 in the low volume roadway. The TSD and FWD deflection data for MnROAD were reported as an average over the 16 sections while the other testing program measurements were reported at a log-mile interval; hence, were analyzed separately.
3.1.6. TSD Raw Measurements Processing

The TSD measures the velocity of the surface deflection under load using Doppler lasers rather than measuring the displacement directly. It collects vertical velocity ($V_v$) and horizontal velocity ($V_h$) continuously at a 0.001-mile interval as shown in Figure 3.6. The deflection slope was calculated at each measurement point by dividing the vertical deflection velocity by the horizontal velocity. Horizontal velocity is equivalent to the measured speed of the TSD.

Figure 3.5. MnROAD road facility in Minnesota

![Figure 3.5. MnROAD road facility in Minnesota](image)

Figure 3.6. Schematic of Doppler lasers mechanism

![Figure 3.6. Schematic of Doppler lasers mechanism](image)
Collected raw measurements (vertical deflection velocity and actual horizontal speed) of the TSD device were used to calculate the deflection basin at each milepost according to the methodology known as “Area under the Curve (AUTC)” proposed by Muller and Roberts (2013). According to this method, the vertical deflection velocity is divided by the actual speed of the vehicle to get the deflection slope; slopes are then plotted against TSD sensor locations; see Figure 3.7. Afterward, the plotted curve is numerically integrated assuming the deflection slope is zero at locations 0 and 137.8 in. (3500 mm) from the load as shown in Figure 3.8(a). The slope value was then calculated at the selected locations with adequate curve fitting using the Piecewise Cubic Hermite function as suggested by the AUTC method. The deflections were then calculated at nine locations (i.e., 0, 8.0, 12.0, 18.0, 24.0, 36.0, 48.0, 60.0, 72.0 in. from the center of the load). An example of deflection basin computation is shown in Figure 3.8(b).

![Slope numerically integrated over the offset distances](image)

Figure 3.7. Slope numerically integrated over the offset distances
Temperature Correction for FWD and TSD Measurements

FWD and TSD deflections were corrected to a reference temperature of 20°C. The Bells equation was used to calculate the pavement temperature at asphalt mid-depth (Lukanen et al.)
Pavement surface deflections at radial offsets were then corrected using the methodology described in Equations (3.1) to (3.3) based on the approach proposed by Kim and Park (Kim et al. 2002).

\begin{equation}
\lambda_w = \frac{w_{T0}}{w_T}
\end{equation} \hspace{1cm} (3.1)

where,

\begin{itemize}
  \item $w_{T0}$ = the deflection corrected to temperature $T_0$;
  \item $w_T$ = the deflection at temperature $T$; and
  \item $\lambda_w$ = the deflection correction factor calculated as follows:
\end{itemize}

\begin{equation}
\lambda_w = 10^{-C(H_{ac})(T-T_0)}
\end{equation} \hspace{1cm} (3.2)

where,

\begin{itemize}
  \item $H_{ac}$ = Asphalt layer thickness; and
  \item $C$ = Regression constant calculated as follows:
\end{itemize}

\begin{equation}
C = -Ar + C_0
\end{equation} \hspace{1cm} (3.3)

where,

\begin{itemize}
  \item $r$ = the radial distance from the center of the load; and
  \item $A$ = - 5.26x10^{-8} for U.S. Central Region; and
  \item $C_0$ = 5.80x10^{-5} for U.S. Central Region.
\end{itemize}

**3.1.7. RWD Testing Program in Louisiana**

The Louisiana Transportation Research Center (LTRC) has researched TSDD in pavement evaluation and management. Repeatability of RWD measurements, the effect of truck speeds,
and the relationship between RWD and FWD deflection measurements and pavement conditions were evaluated (Elseifi et al. 2012). The RWD testing program consisted of two phases; the first phase consisted of testing about 1,000 miles of asphalt roads in District 05 of Louisiana and in the second phase, for a comprehensive evaluation of RWD technology, 16 road sections of 1.5-mile each were tested. Research milestone achieved from RWD testing:

- Repeatability of measurements and comparison with FWD measurements
- A model to predict in-service structural number (SN) based on RWD measurements
- A model to predict subgrade resilient modulus based on RWD measurements
- A structural health monitoring model based on RWD measurements
- A framework for implementation in Louisiana PMS and overlay design
- Cost-efficiency of RWD testing

3.2. EVALUATION OF TSD MEASUREMENTS

Collected TSD and FWD measurements from the Louisiana experimental testing program were processed and filtered as described in the previous section for precise comparison and thorough evaluation. TSD measures deflections at 0.01-mile interval along a pavement section while FWD measured deflections were reported at an interval of 0.1-mile. Hence, to match the data points where FWD deflection measurements were available, TSD deflections were also processed at 0.1-mile intervals at the exact same locations of FWD testing. Furthermore, FWD deflections were measured at a distance of 0.0, 8.0, 12.0, 18.0, 24.0, 36.0, 48.0, 60.0 and 72.0 in. from the center of the plate load. Therefore, TSD deflections were processed from the deflection slopes at the same offset distances from the center of the load, which involves numerical integration of the slopes and subsequently, area under the curve computations. Two separate
statistical methods were used to demonstrate and to evaluate the comparison of TSD and FWD deflections (i.e., significance test considering 95% confidence level and the Limit of Agreement Method (Schnoor and Horak 2012). Furthermore, the potential factors that could influence the TSD field measurements such as pavement roughness were evaluated by calculating the coefficient of variation (COV) within each section.

3.3, DEVELOPMENT OF TSD-BASED STRUCTURAL CAPACITY MODEL

This study developed a non-linear regression model for the prediction of in-service pavement SN and structural-deficiency at 0.01-mile intervals. The proposed model was developed based on TSD surface deflection measurements calculated from the deflection slope by the AUTC method.

Measured deflections at nine offset distances referred as D0, D8, D12, D18, D24, D36, D48, D60, and D72 were initially used as independent variables along with the corresponding pavement total thickness ($T_{th}$), and the Average Daily Traffic (ADT). The SN calculated from the AASHTO 1993 method based on FWD and TSD deflections was used as the dependent variable in the development of the model. To ensure accuracy, several statistical analyses were conducted; i.e., pairwise correlation, significance testing using regression analysis, and multicollinearity testing among all the independent variables. The model was successfully validated based on data points obtained from TSD and FWD measurements in Louisiana and Idaho. The validation and performance evaluation of the model was conducted by comparing its prediction with the SN calculated from FWD deflection measurements. Furthermore, model validation was conducted by evaluating SN prediction accuracy and residual plots. Extracted cores and
functional indices data collected from the DOTD PMS were also used for evaluation of the model’s ability in identifying structurally deficient locations.

3.4. Correlate Surface-Measured Indices and Structural Conditions

In addition to the previously described PMS data, RWD and TSD continuous deflection data measured in Louisiana were used in the analysis. The RWD testing program consisted of two phases; the first phase consisted of testing about 1,000 miles of asphalt roads in District 05 of Louisiana and in the second phase, for a comprehensive evaluation of RWD technology, 16 road sections of 1.5-mile each were tested. The model presented in Equation (2.7) was used to predict the in-service structural number (SN) at an interval of 0.1-mile for the tested road network.

In 2016, a TSD device operated by the Australian Road Research Board (ARRB) was used to measure vertical deflection velocity, horizontal speed of the vehicle, air temperature, and pavement surface temperature in six Parishes of District 05 in Louisiana. Measurements were conducted for 13 control sections at a 0.01-mile interval. FWD measurements were also collected for the same control sections at a 0.1-mile interval for the evaluation and comparison with TSD measurements. Collected raw measurements (vertical deflection velocity and actual horizontal speed) of the TSD device were used to calculate the deflection basin at each milepost according to the methodology known as “Area under the Curve (AUTC)” proposed by Muller and Roberts (2013). The model presented in Equation (4.3) was used to predict the in-service structural number (SN) at an interval of 0.1-mile for the tested road sections.

The number of locations in the testing program amounted to about 11,000 data points for RWD and TSD, see Table 3.3. The in-service SN predicted from RWD and TSD measurements
was compared to the initial AASHTO design SN at each test location. The loss in SN was then calculated at every 0.1-mile interval using Equation (3.4):

\[
\text{Loss in SN(\%)} = \frac{\text{Design SN} - \text{SN}_{\text{RWD/TSD}}}{\text{Design SN}} \times 100
\]

(3.4)

The percentage loss in SN is a logical indicator of structural deficiency. In Louisiana and for AC overlay design, a 50% loss in in-service SN is assumed as the threshold to identify structurally deficient locations.

Table 3.3. General description of the RWD and TSD datasets

<table>
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<tr>
<th>Pavement Category</th>
<th>RWD tested segments</th>
<th>TSD tested segments</th>
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</thead>
<tbody>
<tr>
<td>Thick</td>
<td>3023</td>
<td>97</td>
</tr>
<tr>
<td>Medium</td>
<td>5880</td>
<td>40</td>
</tr>
<tr>
<td>Thin</td>
<td>1869</td>
<td>13</td>
</tr>
<tr>
<td>Total</td>
<td>10,772</td>
<td>150</td>
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</table>

Four relevant surface indices were extracted from the PMS and were considered in the analysis: Alligator Cracking Index (ALCR), Random Cracking Index (RNDM), Rutting Index (RUTT) and Roughness Index (ROUGH). These indices range from zero to 100 with higher values indicating better conditions of the pavement. These indices were extracted for the RWD and TSD 11,000 test locations from the PMS database. Indices were matched to each data point and were collected for the survey cycles from 2005 to 2015. Correlation and relationship were established between the PMS indices, their rate of deterioration over the monitored years, and the percentage loss in in-service pavement structural capacity.

To calculate the rate of deterioration of surface indices over the analysis period (i.e., from 2005 to 2015), the slope was calculated for the data collected from the PMS distress surveys. For RWD testing, the slope was calculated from 2005 to 2009 to represent the period just before
RWD testing whereas, for TSD testing, the slope was calculated from 2011 to 2015 to represent the period just before TSD testing. Figure 3.9 shows an example of deterioration slope calculation where the slope of each straight line was calculated by fitting a straight line to the data points.

Figure 3.9. Slope calculation from PMS surface indices data
CHAPTER 4. ANALYSIS AND RESULTS

4.1. ASSESSMENT OF TSD MEASUREMENTS

TSD measurements were compared to FWD measurements conducted at the same test locations. Two analysis methods were conducted to identify if measurements from both devices are statistically equivalent (i.e., ANOVA and Limit of Agreement).

4.1.1. FWD and TSD Comparisons Using ANOVA

To compare FWD and TSD measured deflections, an Analysis of Variance (ANOVA) was conducted. TSD and FWD deflections were compared at the same locations within a control section at an interval of 0.1-mile. Before comparing the deflection measurements from FWD and TSD, both data sets were corrected to a reference temperature of 20°C (68°F). Afterward, ANOVA was conducted using the SAS 9.4 software package. A 95% confidence level was assumed to identify significant differences; therefore, a P-value less than 0.05 would indicate a significant difference between the measurements of the two devices. The results from the ANOVA are presented in Table 4.1 with their corresponding P-values. Significant differences were referred to as ‘S’ whereas, non-significant differences were referred to as ‘NS’ in Table 4.1. Measurements were compared within each section and results indicated that significant differences exist between the measured deflections of TSD and FWD in most of the sections and at the different sensor locations. Yet, some of the comparisons showed non-significant differences between FWD and TSD measurements. Therefore, results should be compared concurrently with the findings of the Limit of Agreement, which is presented in the following section.
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<th>D0</th>
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<th>D18</th>
<th>D24</th>
<th>D36</th>
<th>D48</th>
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<td>$S$</td>
<td>$S$</td>
<td>$S$</td>
<td>$S$</td>
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<td>$NS$</td>
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<td>($&lt;.0001$)</td>
<td>($&lt;.0001$)</td>
<td>($&lt;.0001$)</td>
<td>($&lt;.0001$)</td>
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<td>($0.3126$)</td>
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<td>$NS$</td>
<td>$S$</td>
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<td>($0.2944$)</td>
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<td>$S$</td>
<td>$S$</td>
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Note: Non-significant relationships are marked in Italic with $P$-value greater than 0.05.

### 4.1.2. Limit of Agreement Method

The Limit of Agreement Method is suitable to identify statistical differences between two device measurements at the same locations as suggested in the literature. As shown in the previous section, using typical statistical analysis, results showed significant differences at some locations while being statistically equivalent at other locations. Therefore, the Limit of Agreement (LOA) method was conducted to compare FWD and TSD measurements. According to the LOA method, the difference between FWD and TSD measurements were plotted against the mean of two measurements at each location. Since no true deflection value for those locations is known, the mean of the measurements was used in the developed plots. A consistent bias was used to summarize the agreement between these two devices. The linear bias was calculated by taking
the average of all differences in the measurement for the two devices. The upper and lower confidence limit was constructed using a 95% confidence level. The upper and lower confidence limits were calculated using Equations (4.1) and (4.2):

95% Lower Confidence Limit: Lower CL = B − 1.96 * σ  
95% Upper Confidence Limit: Upper CL = B + 1.96 * σ  

where,

B= Bias, σ = Standard Deviation.

Plots were constructed combining all the data points at each offset distance from the applied load. The results shown in Figure 4.1 indicate statistical differences between the measurements by FWD and TSD. A significant number of data points deviated from the linear bias line and some data points exceeded the constructed upper and lower confidence limits. Hence, it can be concluded that the deflection reported by both FWD and TSD for the same locations are statically different, which is reasonable given the differences in loading characteristics and load type between the two devices.

![Figure 4.1. Comparison of FWD and TSD measurements using LOA method](image-url)
(b) Comparison of FWD D8 and TSD D8

(c) Comparison of FWD D12 and TSD D12

(d) Comparison of FWD D18 and TSD D18
(e) Comparison of FWD D_{24} and TSD D_{24}

(f) Comparison of FWD D_{36} and TSD D_{36}

(g) Comparison of FWD D_{48} and TSD D_{48}
4.1.3. FWD and TSD Comparisons for Different Functional Conditions

FWD and TSD measured deflections were compared for different road functional conditions. Roads were divided into four road categories (Poor, Fair, Good, Very Good) based on PCI; see Table 3.2. Figure 4.2 shows that TSD and FWD measurements correlated well with more uniform measurements for roads in good functional conditions and more scatterings for roads in
poor functional conditions. Similar findings were reached in a previous RWD study in Louisiana (Elseifi et al. 2011).

Figure 4.2. TSD and FWD comparison plots at different road conditions
(Figure cont’d)
4.1.4. Effect of Pavement Roughness in TSD Field Measurements

According to the literature and previous studies on the topic, the effect of pavement roughness is debatable. Studies showed considerable effect of surface roughness in moving load amplification, which would influence the deflection measurements reported by TSD. Yet, studies also found no significant correlation between TSD measurements variation with International Roughness Index (IRI) (Zofka et al. 2014, Rada et al. 2016).

In the present study, surface roughness was obtained for the Louisiana sections in terms of IRI and at 0.1-mile intervals. To analyze the variation in TSD measurements with IRI, the coefficient of variation (%) for TSD deflection measurements was calculated for each test section. Since FHWA categorizes the pavement section based on IRI as Good if IRI is less than 95 and acceptable if IRI is less than 170, the analysis was conducted by categorizing the control section based on FHWA IRI specifications. Figure 4.3 indicates that there is a noticeable
difference in COV (%) for the two roughness categories. As shown in this figure, the COV (%) was relatively greater for the sections with IRI<170 than the sections with IRI<95. The difference is COV (%) was found to be the largest for the deflections under load (D₀) and the lowest for the far distance deflections (D₆₀ and D₇₂). Therefore, it can be reasonably concluded that surface roughness has a notable effect on the TSD field measured deflections.

![Figure 4.3. COV (%) comparison for TSD deflections for two roughness categories](image)

Figure 4.3. COV (%) comparison for TSD deflections for two roughness categories

Figure 4.4 presents the variation in COV (%) of loading for each section against the average IRI. As shown in Figure 4.4, the two variables appear to be correlated with an R² of 0.62. It is also noted that the COV (%) in load variation was relatively small with a maximum COV of 4.5%, which can be attributed to the technology advancements in TSD in the last few years. Recent upgrades have introduced new fast-acting responsive dynamic servo systems, climate control systems, beam temperature, and gyroscopic compensation, significantly improved horizontal velocity measurement, and advancements in laser calibration processes, as well as improved software.
4.1.5. Effect of TSD Speed on Measured TSD Deflections

In a previous study, it was found that the variation of TSD measurements (i.e., COV) were higher at higher TSD speeds (Rada et al. 2016). In the present study, the experimental program was conducted at only one speed for every section; therefore, the effect of TSD speed variation could not be assessed with field measurements. Yet, the effect of speed on surface deflections was evaluated using 3D Move simulation. 3D-Move software was selected as it has been shown effective in simulating deflections due to a moving load while considering the vehicle speed and viscoelastic material properties. Deflection variation with speeds was studied at a single location. TSD loading condition and dynamic modulus for AC layer were incorporated as inputs in 3D Move to calculate the corresponding surface deflection at different radial offsets.

Simulated deflections were in good agreement with the field-measured deflection at a speed of 25.1 mph. The simulation was conducted at five different speeds. From the simulated results, the increase in vehicle speed caused a decrease in the majority of the deflections, as shown in
Table 4.2 and Figure 4.5. For a comprehensive evaluation of TSD measurements variation with speed, additional field-testing is recommended.

Table 4.2. 3D Move simulation results with different speed

<table>
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<tr>
<th></th>
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<td>15</td>
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<td>9.38</td>
<td>8.39</td>
<td>6.99</td>
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<td>3.72</td>
<td>2.30</td>
<td>1.34</td>
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<td>8.15</td>
<td>6.77</td>
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<td>60</td>
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<td>2.31</td>
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<td>0.68</td>
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</table>

Asterisk mark (*) represents the actual testing speed in this section.

Figure 4.5. Deflection basin obtained from 3D Move simulation at different speeds

4.2. Structural Capacity Prediction Model Development

The SN prediction model development along with its validation procedure is discussed in this section. Model development was followed by several statistical analyses to ensure selection of
appropriate independent variables and to evaluate their effectiveness in the model. After
development, model validation was conducted based on an independent data set.

4.2.1. Pairwise Correlation

The pairwise correlation was conducted among the independent variables to avoid using
collinear or multi-collinear independent variables in the model, which may increase the variance
of the estimated regression coefficients. All the possible independent variables that may have an
influence on the prediction of SN were subjected to pairwise correlation analysis. The
correlation coefficient is an indication of the level of collinearity among the independent
variables (Miller and Freund 2004, Freund et al. 2006). The coefficient is called Pearson’s
correlation coefficient, which ranges from -1 to +1. A large absolute value indicates high
collinearity between those variables. The positive or negative sign represents the positive or
negative relationship between the variables. The results of the pairwise correlation analysis are
presented in Table 4.3. Pearson’s coefficient that are greater than 0.6 was considered highly
collinear in this analysis. As shown in this table, most of the deflection measurements were
correlated, which was expected as surface deflections tend to increase or to decrease
concurrently with the exception of far distance deflections (i.e., D_{60} and D_{72}), which may indicate
weakness in the underlying layers and the subgrade. It is worth noting that the final model did
not include D_{60} or D_{72} because the use of these variables would limit the application of the model
since some TSD surveys do not measure far distance deflections from the load. Moreover, the
prediction accuracy was satisfactory with the use of D_{48}, which was deemed more reasonable
than the use of D_{60} or D_{72}, even though they showed a better correlation with SN_{FWD}. Based on
the results of the analysis, collinear variables were not used. Different combinations of non-
collinear variables were considered in the regression analysis.
Table 4.3. Pearson correlation coefficients

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<th>D0</th>
<th>D8</th>
<th>D12</th>
<th>D18</th>
<th>D24</th>
<th>D36</th>
<th>D48</th>
<th>D60</th>
<th>D72</th>
<th>Tth</th>
<th>ADT</th>
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<td>0.36</td>
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<td>0.42</td>
<td>0.46</td>
<td>0.80*</td>
<td>0.76*</td>
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<td>D0</td>
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<td>0.95*</td>
<td>0.88*</td>
<td>0.74*</td>
<td>0.58</td>
<td>0.38</td>
<td>0.29</td>
<td>0.21</td>
<td>0.10</td>
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<td>-0.42</td>
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<tr>
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<td>0.98*</td>
<td>0.90*</td>
<td>0.78*</td>
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<td>0.44</td>
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<tr>
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<td>0.98*</td>
<td>1.00</td>
<td>0.97*</td>
<td>0.89*</td>
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<td>0.67*</td>
<td>0.58</td>
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<tr>
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<td>0.74*</td>
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<td>1.00</td>
<td>0.97*</td>
<td>0.89*</td>
<td>0.83*</td>
<td>0.75*</td>
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<td>0.33</td>
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<td>-0.24</td>
<td>-0.10</td>
<td>0.08</td>
<td>0.24</td>
<td>0.39</td>
<td>0.43</td>
<td>0.45</td>
<td>0.50</td>
<td>0.52</td>
<td>1.00</td>
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</table>

Note: Highly collinear relationships are marked as **Italic with asterisk mark.**

4.2.2. Regression Analysis and Variance Inflation factor (VIF)

To assess the significance of the independent variables (D₀, D₈, D₁₂, D₁₈, D₂₄, D₃₆, D₄₈, D₆₀, D₇₂, T₅th, and ADT) on the prediction of the dependent variable (SN), regression analysis was conducted. Independent variables with no significance on the dependent variable were removed from the model to avoid overfitting of the dependent variable. When overfitting occurs, the regression model becomes tailored to fit the random noise in the data set rather than reflecting the actual trends in the measurements. A regression analysis was conducted on several combinations of independent variables using SAS 9.4 software. With 95% confidence level, a P-value less than 0.05 would represent a significant effect. The combination of independent variables that had significant effect on the dependent variable is presented in Table 4.4.
Table 4.4. Results of regression analysis and multi-collinearity test

| Variable | Pr > |t|  | Interpretation | Variance Inflation (VIF) |
|----------|------|----------------|------------------|-------------------------|
| Intercept | <.0001 | Significant | 0 |
| D₀        | <.0001 | Significant | 1.86 |
| D₄₈       | <.0001 | Significant | 1.94 |
| Tₚₜ       | <.0001 | Significant | 2.22 |
| ADT       | <.0001 | Significant | 1.43 |

To further filter out the multi-collinear independent variables, a second statistical factor known as the ‘Variance Inflation Factor (VIF)’ was used. Even after pairwise correlation analysis between two variables, there is a possibility of multi-collinearity resulting from the combination of one variable with more than one variable. To address this issue, the most used statistical factor is the variance inflation factor. Because of multi-collinearity, an inflation can occur in the standard error, which is measured by VIF. A maximum VIF value of 5 to 10 is recommended in the literature (Hair et al. 1995). In this study, the VIF values for the selected independent variables were within the acceptable range (Table 4.4), which indicates that no multi-collinear independent variables have been used in the model.

4.2.3. Non-Linear Regression Model Development

A non-linear regression model was developed using SAS 9.4 to predict the SN of in-service pavement. The structural number, which is referred to as SNₜₚₜ, was predicted based on the statistically significant TSD deflections (D₀ and D₄₈), ADT, and total pavement thickness (Tₚₜ). About 70% of the data points from Louisiana and 30% of the data points from Idaho were used in the development phase to fit the model; the remaining data points from Louisiana and Idaho were used to validate the fitted model. The model demonstrated an acceptable accuracy with a
Coefficient of determination ($R^2$) of 0.92 in the development phase and with an RMSE of 0.88 as shown in Figure 4.6. The proposed model is illustrated in Equation (4.3):

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

where,

$SN_{TSD} =$ SN based on TSD measurements;

$D_0 =$ Deflection of pavement under loaded tire or Center Deflection (mils);

$D_{48} =$ Deflection at 48 in. distance from Center Deflection (mils);

$T_{th} =$ Total layer thickness of pavement (in.); and

$ADT =$ Average Daily Traffic (veh/day).

Figure 4.6. Model fitting in the development phase

### 4.2.4. Model Validation

An independent TSD and FWD data set was used to validate the model. The use of Idaho data points in the model validation demonstrated the model’s compatibility with different climatic
regions and construction practices. The model performed satisfactorily in the validation phase with an $R^2$ of 0.88 and with an RMSE of 1.06, as shown in Figure 4.7. A good agreement was also found when the average $SN_{TSD}$ for each road section was compared with the average $SN_{FWD}$; see Figure 4.8.

![Figure 4.7. Model fitting in the validation phase](image1)

![Figure 4.8. Average SN comparison between TSD and FWD for each section](image2)
Table 4.5 shows the calculated RMSE for each section. It should be noted that the concept of SN is not used for composite pavements in the AASHTO 93 pavement design method; therefore, the tested composite sections (Site ID 3 and 13) were not used in the development and the validation of the model. As shown in this table, RMSE obtained from the model’s output was satisfactory within each section.

Table 4.5. Comparison between predicted and measured SN for each section

<table>
<thead>
<tr>
<th>Control Section</th>
<th>Route</th>
<th>Log-mile</th>
<th>SN RMSE</th>
</tr>
</thead>
<tbody>
<tr>
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<td>LA 34-1</td>
<td>5.55 - 6.95</td>
<td>0.581</td>
</tr>
<tr>
<td>067-09</td>
<td>LA 34-2</td>
<td>3.35 – 4.75</td>
<td>0.168</td>
</tr>
<tr>
<td>326-01</td>
<td>LA 594-2</td>
<td>5.05 - 6.45</td>
<td>0.840</td>
</tr>
<tr>
<td>324-02</td>
<td>LA 616</td>
<td>3.55 - 4.95</td>
<td>0.607</td>
</tr>
<tr>
<td>831-05</td>
<td>LA 821</td>
<td>2.05 - 3.25</td>
<td>0.967</td>
</tr>
<tr>
<td>071-02</td>
<td>US 425</td>
<td>1.00 - 2.50</td>
<td>1.029</td>
</tr>
<tr>
<td>069-03</td>
<td>LA 33</td>
<td>3.05 - 4.45</td>
<td>1.189</td>
</tr>
<tr>
<td>315-02</td>
<td>LA 143</td>
<td>6.00 - 7.50</td>
<td>0.708</td>
</tr>
<tr>
<td>333-03</td>
<td>LA 582</td>
<td>3.00 - 4.50</td>
<td>0.827</td>
</tr>
<tr>
<td>862-14</td>
<td>LA 589</td>
<td>4.00 - 5.50</td>
<td>1.743</td>
</tr>
<tr>
<td>326-01</td>
<td>LA 594-1</td>
<td>2.00 - 3.50</td>
<td>1.268</td>
</tr>
<tr>
<td>IDAHO</td>
<td>ID-SH22</td>
<td>Seg 05</td>
<td>0.793</td>
</tr>
</tbody>
</table>

The residual plots are shown in Figure 4.9 with each independent variable used in the model. The residuals were calculated as the difference between the measured and predicted SN. The plots were drawn for both data sets used in the development and validation phases. As illustrated, the residuals were reasonably scattered and no clear trend is visible in the plots; therefore, the model’s estimation can be assumed random with the model inputs.
4.2.5. Longitudinal Profile Comparison

A continuous SN profile was developed from the proposed model at short intervals of 0.01-mile. Longitudinal profiles for SN$_{TSD}$ obtained from the model were compared to SN$_{FWD}$ for both the Louisiana and Idaho control sections. The longitudinal profiles for one Louisiana section and one Idaho section are shown in Figures 4.10 and 4.11, respectively. Higher variability was noted
in the Idaho section, possibly due to higher roughness and cracking at the surface as suggested by the lower SN predicted for this section. Past studies concluded that both FWD and RWD test methods resulted in a greater average deflection and scattering in sites in poor conditions (Elseifi et al. 2012).

Figure 4.10. Longitudinal comparison of SN_{TSD} and SN_{FWD} for Louisiana

Figure 4.11. Longitudinal comparison of SN_{TSD} and SN_{FWD} for Idaho
4.2.6. Sensitivity Analysis

A sensitivity analysis was conducted for the proposed model’s output as a function of the model’s inputs. Sensitivity of the dependent variable ($SN_{TSD}$) was tested for different input parameters varied within their maximum and minimum values. The average of each of the input parameters was used as the baseline in the sensitivity analysis. From the results of the sensitivity analysis, it was found that the predicted $SN_{TSD}$ was most sensitive to $D_0$ among all other parameters and the least sensitive to ADT. The change in the predicted $SN_{TSD}$ with the varying input parameters is shown in Figure 4.12.

![Figure 4.12. Sensitivity analysis for the $SN_{TSD}$ model](image)

4.3. Correlation of Structural Capacity with PMS Functional Indices

An Analysis of Variance was conducted with the measurements from the Louisiana control sections to evaluate the degree of influence of functional indices on in-service structural capacity at a confidence level of 95%. Functional indices considered in the comparison with $SN_{TSD}$ were
alligator cracking (ALCR), random cracking (RNDM), patching (PTCH), rutting index (RUT_IND), roughness (RUFF), and Pavement Condition Index (PCI). Road sections were divided into five categories based on asphalt layer(s) thickness and type of base layer (treated and untreated) as shown in Table 4.6 (Elbagalati et al. 2016). Thick sections were those that had an AC layer greater than 6 in.; medium sections were those with AC layers between 3 and 6 in.; and thin sections were those with AC layers less than 3 in. It is noted that there was no thin untreated section tested using TSD in the present study.

Table 4.6. Classification of the control sections

<table>
<thead>
<tr>
<th>Asphalt layer thickness</th>
<th>Base layer type</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 6 in.– Thick</td>
<td>If Stabilized – Treated</td>
</tr>
<tr>
<td>3 in. &lt; thickness &lt; 6 in. – Medium</td>
<td>If not stabilized- Untreated</td>
</tr>
<tr>
<td>&lt; 3 in. – Thin</td>
<td></td>
</tr>
</tbody>
</table>

The correlation between each of the aforementioned functional indices and structural indices (SN{TSD}) was evaluated using P-value obtained from the statistical analysis. Since 95% confidence limit was used, a P-value less than 0.05 would represent significant correlation between the condition indices and vice-versa. As shown by the results in Table 4.7, non-significant statistical relation was found between functional indices and SN in four of the five road categories. Therefore, one may assume that considering a structurally based index in PMS would allow for the identification of road segments that are in need of structural repair and that are not currently identified by the functional indices.
Table 4.7. Significance of functional indices on SN

| Road Category       | Functional Indices (Pr > |t|)     |
|---------------------|-------------------------|
|                     | ALCR  | RNDM  | PTCH  | RUT_IND | RUFF  | PCI     |
| Thick (Treated)     | 0.7457* | 0.0002 | 0.4741* | 0.5430* | 0.7276* | 0.2646* |
| Thick (Untreated)   | 0.0010  | 0.2563* | 0.9565* | 0.3777* | 0.7837* | 0.4982* |
| Medium (Treated)    | 0.2943* | 0.0729* | 0.0014  | 0.1078* | 0.8731* | 0.1786* |
| Medium (Untreated)  | 0.4792* | 0.6867* | NA     | 0.7065* | 0.7834* | 0.8272* |
| Thin (Treated)      | 0.2205* | 0.6867* | NA     | NA     | 0.1645* | 0.2347* |

Note: Non-significant relationships (P-value<0.05) are marked in Italic with asterisk mark. NA=Not Available.

4.4. PERFORMANCE EVALUATION OF THE DEVELOPED MODEL

The model’s precision and adequacy were evaluated by comparing the percentage loss in SN_{TSDeff} and SN_{FWDeff}. The proposed model’s ability in identifying structurally deficient sections was also evaluated as compared to structural deterioration identified from extracted cores and from FWD. Since cores were only available for the Louisiana data set, this analysis was exclusively conducted for the Louisiana road sections.

4.4.1. Calculation of Loss in In-Service SN

To determine the percentage loss in in-service SN, the AASHTO design SN during construction was calculated using equation (4.4):

\[
SN = a_1 * D_1 + a_2 * m_2 * D_2 + a_3 * m_3 * D_3
\]  

(4.4)

where,

\[ a_1 = \text{asphalt layer coefficient}, \quad a_2 = \text{base layer coefficient}, \quad a_3 = \text{subbase layer coefficient}; \]
D_1 = asphalt layer thickness (in.), D_2 = base layer thickness (in.), and D_3 = subbase layer thickness (in.); and

m_2 = base layer drainage coefficient and m_3 = subbase layer drainage coefficient.

The values of the layer coefficients were selected in accordance with LaDOTD design standards: a_1 = 0.42; a_2 = 0.28 for treated (cement stabilized) base and 0.07 for untreated base; a_3 = 0.11 for cement treated subbase and 0.04 for untreated subbase. The values of m_1 and m_2 were considered 1.0 in all cases.

After calculating the design SN of the sections during construction, SN_{TSD} was corrected according to the findings of a study conducted by Wu and Gaspard to account for design and construction practices in the State as follows (Wu and Gaspard 2009, Wu et al. 2013):

\[ SN_{eff} = 2.58 \ln (SN_{FWD}) - 0.77 \] (4.5)

Since the SN_{TSD} model was developed and validated based on SN calculated from FWD deflections, the predicted SN_{TSD} of the road sections were also adjusted through the same model using Equation (4.6):

\[ SN_{TSDeff} = 2.58 \ln (SN_{TSD}) - 0.77 \] (4.6)

The loss in SN_{TSDeff} was then calculated at every 0.1-mile interval using Equation (4.7):

\[ \text{Loss in SN(\%)} = \frac{\text{Design SN} - SN_{TSDeff}}{\text{Design SN}} \times 100 \] (4.7)

The percentage loss in SN_{TSDeff} was compared with SN_{FWDeff} at each extracted core location. It is noted that the only control sections considered in this comparison were the ones, which had the cores extracted at almost the same location to ensure precise evaluation of the SN model.
average SN over 1.5-mile was also calculated and compared. As shown in Figures 4.13 and 4.14, the estimated percentage loss from the model was in good agreement with the percentage loss predicted from SN\textsubscript{FWD}.

![Figure 4.13. Comparison of loss in in-service SN at core location](image)

![Figure 4.14. Comparison of loss in average in-service SN](image)
4.4.2. Analysis of the Extracted Cores

In this section, the model’s efficiency in identifying structural deficient sections was evaluated. Past studies by the authors used 50% loss of AASHTO SN as the threshold to identify structurally deficient locations (Elbagalati et al. 2016). In the present study, the developed model’s evaluation in identifying structurally deficient locations was also based on a 50% loss in structural capacity. The extracted cores were compared with the estimated loss in SN (%) along with the functional indices at the same locations. A detailed evaluation of four typical road section is presented in the following sections.

Control Section 831-05

The control section is located in Route LA 821 with a length of 8.18-mile located at Lincoln parish in Louisiana District 05. The total layer thickness of this control section from the extracted core was found to be 13 in. consisting of three AC layers of 5 in. and a granular base layer of 8 in. After assessment of the extracted core, deterioration (stripping) was detected in the third asphalt layer, which was 2 in. thick as shown in Figure 4.15. The percentage loss in SN_{TSDeff} at the core location and the average SN (%) loss over 1.5-mile was found 55.3% and 61.4%, respectively. The average Pavement Condition Index (PCI), a combined functional index, for this control section was 92.7 over 1.5 mile and the PCI at the core location was 90.3 indicating excellent functional conditions. Given the average SN loss (%) is greater than 50%, this control section was identified as structurally deficient even with a sound PCI rating.
The control section is located in Route LA 143 with a length of 9.26-mile located at Ouachita parish in Louisiana District 05. The total layer thickness of the control section from the extracted core was 23 in. consisting of two AC layers of 9.5 in. on top of a cement stabilized sand clay gravel base layer of 13.5 in. After assessment of the extracted core, debonding was detected between the bottom AC layer and the underlying base layer, as shown in Figure 4.16. It is to be noted that poor drainage conditions were also detected in this road section. The average percentage loss in SN$_{TSDeff}$ over 1.5-mile and at the core location was 33.4% and 35.1%, respectively. The average PCI for this control section was found to be 92.7 over 1.5 mile and 96.7 at the core location indicating excellent function conditions. Though the predicted structural capacity loss was less than 50%, the model predicted a loss in SN possibly related to the detected debonding between the AC and the base layers.
Figure 4.16. Control Section 315-02

Control Section 333-03

The control section is located in Route LA 582 with a length of 6.83-mile located at E Carroll parish in Louisiana District 05. The total layer thickness of this control section from the extracted core was 18 in. consisting of five AC layers of 9.5 in. and a granular base layer of 8.5 in. After assessment of the extracted core, deterioration (stripping) was found in the bottom AC layer that was 1.5 in. thick, as shown in Figure 4.17. The average percentage loss in SN_{TSD} over 1.5-mile was calculated as 36.6% and 17.8% at the core location. Upon further assessment of the control section, it was found that a new overlay was applied since the core extraction and functional survey explaining the adequate structural capacity of the control section. Hence, the model’s estimated loss in SN can reasonably be justified. The average PCI in 2015 for this control section was found to be 67.9 over 1.5 mile and the PCI at the core location was 73.6, which was prior to the new overlay.
4.5. **SUMMARY OF THE MODEL’S STRUCTURAL EFFICIENCY PREDICTION**

As discussed in the previous section, the proposed model can reasonably estimate the average loss in in-service SN (%) as compared with the extracted cores. While functionally sound, a number of control sections were identified as structurally deficient as summarized in Table 4.8.

### Table 4.8. Model’s performance evaluation based on extracted cores

<table>
<thead>
<tr>
<th>Control Section</th>
<th>Avg. PCI</th>
<th>Type of deterioration in Cores</th>
<th>Note</th>
<th>Avg. Loss in SN_{TsDef}(%)</th>
<th>Remarks on Model’s efficiency</th>
</tr>
</thead>
<tbody>
<tr>
<td>831-05</td>
<td>92.7</td>
<td>Stripping</td>
<td>N/A</td>
<td>61.4</td>
<td>Identified structurally deficient section</td>
</tr>
<tr>
<td>315-02</td>
<td>92.7</td>
<td>Separation</td>
<td>Cement stabilized base layer</td>
<td>33.4</td>
<td>Predicted loss in structural capacity due to debonding</td>
</tr>
<tr>
<td>333-03</td>
<td>67.9</td>
<td>Stripping</td>
<td>New overlay applied since core extraction</td>
<td>36.6</td>
<td>Reasonable estimation as new overlay was applied</td>
</tr>
</tbody>
</table>

*Note: N/A = not applicable*
4.6. **SURFACE INDICES EVALUATION FOR RWD AND TSD TESTED LOCATIONS**

4.6.1. **Statistical Relationship between Loss of Structural Capacity and Surface Indices**

Functional indices were statistically tested against structural-deficiency as identified from RWD and TSD testing and assuming a 50% loss in structural capacity as the threshold for structural deficiency. Both RWD and TSD tested sections were categorized into three groups based on AC layer(s) thickness (i.e., thin sections (AC layer < 3 in.), medium sections (AC layer between 3 to 6 in.), and thick sections (AC layer > 6 in.).

Regression analysis was performed at a 95% confidence level where a P-value less than 0.05 for the individual surface indices would indicate the presence of a statistical correlation between surface indices and structural deficiency as identified from RWD and TSD testing. As shown in Table 4.9, structural deficiency and surface indices were correlated to a certain extent for RWD and to a lesser extent for TSD. It is worth noting that all thin sections in the TSD dataset were structurally sound; therefore, no statistical correlation could be conducted in this case. The difference between RWD and TSD may be due to the difference in the size of the data sets used in the analysis, see Table 3.3. In addition, TSD tested sections had better surface conditions and less deterioration rate of indices over time than the RWD tested sections. Nevertheless, the statistical correlation between structural deficiency and surface indices was expected, as pavements that are structurally deteriorated will exhibit surface deficiencies over time. In addition, some of the indices adopted in Louisiana such as ALCR and RUTT describe some types of structural deficiencies but only after they appear at the surface. Yet, results do not necessarily mean that surface indices can serve as a reliable predictor of structural capacity.

Further evaluation of these findings was conducted as presented in the following sections.
Table 4.9. Statistical analysis of the significance of surface indices on in-service structural conditions

<table>
<thead>
<tr>
<th>Test type</th>
<th>Pavement Category</th>
<th>Surface Indices</th>
<th>Structural Deterioration</th>
<th>% loss in in-service SN</th>
</tr>
</thead>
<tbody>
<tr>
<td>RWD tested sections</td>
<td>Thick</td>
<td>ALCR</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thick</td>
<td>RNDM</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thick</td>
<td>RUTT</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thick</td>
<td>ROUGH</td>
<td>&lt;0.0001 (Significant)</td>
<td>0.0041 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Medium</td>
<td>ALCR</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Medium</td>
<td>RNDM</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Medium</td>
<td>RUTT</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Medium</td>
<td>ROUGH</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thin</td>
<td>ALCR</td>
<td>0.0893 (Non-significant)</td>
<td>0.0027 (Significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thin</td>
<td>RNDM</td>
<td>0.9880 (Non-significant)</td>
<td>0.1189 (Non-significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thin</td>
<td>RUTT</td>
<td>0.4412 (Non-significant)</td>
<td>0.1763 (Non-significant)</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>Thin</td>
<td>ROUGH</td>
<td>0.1048 (Non-significant)</td>
<td>0.0758 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Thick</td>
<td>ALCR</td>
<td>0.8529 (Non-significant)</td>
<td>0.5316 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Thick</td>
<td>RNDM</td>
<td>0.0622 (Non-significant)</td>
<td>0.0164 (Significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Thick</td>
<td>RUTT</td>
<td>0.8876 (Non-significant)</td>
<td>0.4339 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Thick</td>
<td>ROUGH</td>
<td>0.1225 (Non-significant)</td>
<td>0.5131 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Medium</td>
<td>ALCR</td>
<td>0.1424 (Non-significant)</td>
<td>0.2317 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Medium</td>
<td>RNDM</td>
<td>0.1495 (Non-significant)</td>
<td>0.2145 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Medium</td>
<td>RUTT</td>
<td>0.3804 (Non-significant)</td>
<td>0.0572 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Medium</td>
<td>ROUGH</td>
<td>0.2112 (Non-significant)</td>
<td>0.9647 (Non-significant)</td>
</tr>
</tbody>
</table>

4.6.2. Average Indices Comparison for Structurally Sound and Deficient Pavements

Surface indices were averaged for structurally sound and structurally deficient sections as identified from RWD and TSD testing. For the RWD sections shown in Figure 4.18(a), the indices of ALCR, RNDM, and ROUGH were somewhat greater for structurally sound sections than for structurally deficient sections. For the TSD sections shown in Figure 4.18(b), the indices of RNDM, RUTT, and ROUGH were also noticeably greater for structurally sound sections than for structurally deficient sections. The difference between the two groupings can be attributed to the fact that although surface indices only represent pavement surface conditions,
structurally deficient pavement tends also to deteriorate functionally over time especially if left without repair for an extended period of time.

![Figure 4.18. Functional indices comparison for (a) RWD tested sections (b) TSD tested sections](image)

Since the average values may not always depict the entire picture, statistical 95% confidence intervals were constructed for the functional indices as shown in Table 4.10. The confidence intervals are presented in Table 4.10 as lower bounds and upper bounds and were calculated using Equations (4.8) and (4.9):

95% Lower Confidence Limit: Lower bound = $\bar{X} - \frac{1.96 \cdot \sigma}{\sqrt{n}}$ \hspace{1cm} (4.8)

95% Upper Confidence Limit: Upper bound = $\bar{X} - \frac{1.96 \cdot \sigma}{\sqrt{n}}$ \hspace{1cm} (4.9)

where,

$\bar{X}$ = Sample mean;

$\sigma$ = Standard Deviation; and

n = Sample size.
For the RWD tested sections, it was found that the alligator cracking and roughness indices showed a noticeable difference in the confidence intervals between structurally sound and structurally deficient locations. For the TSD tested sections, the confidence intervals developed for most of the indices were found to be merging between sound and deficient segments. While indices were higher for structurally sound segments, it was difficult to conclude a noteworthy correlation. Thus, it appears difficult to identify structurally sound and structurally deficient pavements if only the surface indices were known.

Table 4.10. Developed 95% confidence interval of indices for in-service structural conditions

<table>
<thead>
<tr>
<th>Test type</th>
<th>Functional Indices</th>
<th>95% confidence interval for indices</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sound</td>
<td>Deteriorated</td>
<td></td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>ALCR</td>
<td>(89.7, 90.2)</td>
<td>(82.4, 83.4)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>(91.9, 92.4)</td>
<td>(86.6, 87.3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>(92.7, 93.1)</td>
<td>(92.3, 92.8)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>(81.8, 82.4)</td>
<td>(70.6, 71.5)</td>
<td></td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>ALCR</td>
<td>(94.6, 98.3)</td>
<td>(94.3, 98.3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>(97.2, 98.9)</td>
<td>(92.8, 96.0)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>(83.8, 93.3)</td>
<td>(78.2, 87.2)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>(92.6, 96.3)</td>
<td>(84.6, 92.2)</td>
<td></td>
</tr>
</tbody>
</table>

4.6.3. Functional Indices Evaluation Based on SN Loss

Any possible relation between surface indices and pavement structural conditions was further evaluated in this section. Each surface index was categorized into five groups for RWD tested sections to compare the SN loss for each of the groups. For example, the group between 90 and 100 for the alligator-cracking index (ALCR) represents segments with almost no alligator cracking. The 95% confidence intervals for the loss in SN were calculated for each of the five functional groups and their ranges are presented in Figure 4.19. As shown in Figure 4.19, all the
surface indices in general except RUTT exhibited an increase in SN loss as the index deteriorated. Yet, the confidence intervals for the loss in SN shown in the bars within parenthesis were found to be merging among the indices groups and do not indicate a definitive limit in SN loss over the range of the indices. For example, the confidence intervals for the loss in SN ranged from 42.9 to 58.0 for an ALCR index varying from 100 to 60, which is unlikely to be deemed as significant.

![Figure 4.19. Comparison of SN loss (%) for different ranges of functional indices](image)

TSD tested sections were divided into two functional index groups due to the smaller dataset as compared to the RWD analysis. A clear increase in SN loss (%) was noticed between the two index groups (i.e., RUTT and ROUGH) as shown in Figure 4.20; however, the confidence
intervals overlapped between ALCR groups even though there was an increase in SN loss with index deterioration. Hence, the identification of structural-deficiency based on functional indices is challenging. Even though this analysis presents a clear trend between surface distresses and SN loss (%) for both RWD and TSD tested sections, it was difficult to establish a definitive threshold between structurally sound and structurally deficient pavements based on surface indices.

![Figure 4.20. Comparison of functional indices groups for TSD sections](image)

### 4.6.4. Risk Analysis Based on a 95% Confidence Interval

While it was mentioned in the previous sections that surface indices could not identify structural-deficiency independently, it was not quantified. To assess the extent of accuracy in identifying structurally-damaged sections, the percentage of road sections, which would result in an inaccurate prediction of in-service structural conditions, was calculated based on the confidence intervals presented in Table 4.10. Results presented in Table 4.11 show that the percentage of misinterpreted sections is quite significant. For example, 34.9% of sound road segments had an ALCR index less than the lower bound of the 95% confidence interval presented in Table 4.10.
This means that even though the sections were structurally sound, these sections would be considered structurally deficient if only assessed by the ALCR index. Similarly, 51.5% of deteriorated road segments had ALCR index greater than the upper bound. This means that even though the sections were structurally deficient, these sections would be considered structurally sound if only assessed by the ALCR index.

Table 4.11. Results of risk analysis associated with considering only surface indices

<table>
<thead>
<tr>
<th>Test type</th>
<th>Surface Indices</th>
<th>Percentage of road segments</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sound but indices less than lower bound</td>
<td>Deteriorated but indices greater than upper bound</td>
<td></td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>ALCR</td>
<td>34.9 %</td>
<td>51.5 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>37.7 %</td>
<td>45.4 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>37.1 %</td>
<td>59.1 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>41.6 %</td>
<td>54.5 %</td>
<td></td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>ALCR</td>
<td>23.8 %</td>
<td>68.2 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>16.7 %</td>
<td>51.0 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>26.2 %</td>
<td>51.0 %</td>
<td></td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>51.4 %</td>
<td>60.8 %</td>
<td></td>
</tr>
</tbody>
</table>

4.7. EVALUATION OF THE RATES OF DETERIORATION OF INDICES OVER TIME

4.7.1. Comparison of Deterioration rates

Average indices deterioration rates from 2005 to 2009 and from 2011 to 2015 were compared separately for the RWD and TSD sections. In this analysis, only negative slopes of functional indices were considered since a declining rate of indices is the subject matter. Positive slopes may occur if maintenance activities (e.g., crack sealing) were conducted but they were not recorded in the PMS. This analysis was conducted to assess whether there are statistical correlations between the rates of deterioration of functional indices and the structural conditions
of the pavement. Figure 4.21 compares the rates of deterioration for structurally sound and structurally deficient sections for the four surface indices (i.e., ALCR, RNDM, RUTT, and ROUGH). For the RWD sections shown in Figure 4.21(a), the rates of deterioration of ALCR were noticeably higher for structurally deficient sections than for structurally sound sections.

For the TSD sections shown in Figure 4.21(b), the rates of deterioration of RNDM, RUTT, and ROUGH were also noticeably higher for structurally deficient sections than for structurally sound sections. Although pavement roughness is not typically correlated to structural deterioration, the roughness index was found in previous studies to be indicative of overall pavement quality, which may correlate to structural deterioration (Radović et al. 2016). Further assessment of these findings was conducted in the following sections.

4.7.2. Statistical Relationship between Structural Capacity Loss and Deterioration Rates

The rates of deterioration of surface indices were statistically tested against structural deficiency as identified from RWD and TSD testing and assuming a 50% loss in structural capacity as the threshold for structural deficiency. Results of this analysis are presented in Table 4.12. At a 95%
confidence level, a P-value less than 0.05 for the individual surface indices would indicate the presence of a statistical correlation between the rates of deterioration of surface indices and structural deficiency as identified from RWD and TSD testing. As shown in Table 4.12, structural deficiency and the rates of deterioration of surface indices were correlated to a certain extent but they mostly depicted non-significant statistical correlations. These results were expected, as pavements that are structurally deteriorated would also exhibit surface deficiency over time. Yet, results do not necessarily mean that the rates of deterioration of surface indices can serve as a reliable predictor of structural deficiency. Further evaluation of these findings was conducted as presented in the following sections.

Table 4.12. Significance of average slope on structural deterioration and % SN loss

<table>
<thead>
<tr>
<th>Test type</th>
<th>Pavement Category</th>
<th>Deterioration rate of indices</th>
<th>P-value</th>
<th>% loss in in-service SN</th>
</tr>
</thead>
<tbody>
<tr>
<td>RWD tested sections</td>
<td>Thick</td>
<td>ALCR</td>
<td>0.0019 (Significant)</td>
<td>0.0096 (Significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RNDM</td>
<td>0.0004 (Significant)</td>
<td>0.0417 (Significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RUTT</td>
<td>0.3715 (Non-Significant)</td>
<td>0.0064 (Significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ROUGH</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>ALCR</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 ( Significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RNDM</td>
<td>&lt;0.0001 (Significant)</td>
<td>0.0281 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RUTT</td>
<td>0.2163 (Non-significant)</td>
<td>0.2020 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ROUGH</td>
<td>0.2067 (Non-significant)</td>
<td>0.2328 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td>Thin</td>
<td>ALCR</td>
<td>&lt;0.0001 (Significant)</td>
<td>&lt;0.0001 (Significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RNDM</td>
<td>0.6904 (Non-significant)</td>
<td>0.9611 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RUTT</td>
<td>0.5773 (Non-significant)</td>
<td>0.8099 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ROUGH</td>
<td>0.4850 (Non-significant)</td>
<td>0.8729 (Non-significant)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>Thick</td>
<td>ALCR</td>
<td>0.6501 (Non-significant)</td>
<td>0.5381 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RNDM</td>
<td>0.3781 (Non-significant)</td>
<td>0.0582 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RUTT</td>
<td>0.5961 (Non-significant)</td>
<td>0.1993 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ROUGH</td>
<td>&lt;0.0001 (Significant)</td>
<td>0.0754 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td>Medium</td>
<td>ALCR</td>
<td>0.9148 (Non-significant)</td>
<td>0.9911 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RNDM</td>
<td>0.6324 (Non-significant)</td>
<td>0.3177 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>RUTT</td>
<td>0.7393 (Non-significant)</td>
<td>0.2369 (Non-significant)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ROUGH</td>
<td>0.4877 (Non-significant)</td>
<td>0.0141 (Non-significant)</td>
</tr>
</tbody>
</table>
4.7.3. Percentage of Structurally-Deficient Sections with Declining Functional Indices

The percentage of structurally damaged sections, which had declining functional indices in the years prior to TSDD testing are shown in Figure 4.22. As shown in Figure 4.22, the ROUGH and ALCR indices were found to be declining in 78% and 61% of the structurally deficient sections, respectively. On the other hand, the RUTT index was declining in only 19% of the structurally deficient sections, which was expected since RUTT was found not significant in the previous analysis. A combination of two or more indices was also investigated; the ALCR and ROUGH indices were declining simultaneously in 48% of the structurally deficient sections. While the use of two more indices would allow more precision and accuracy in confirming structurally deficient sections, this approach would only be correct for a little less than half of the sections.

![Figure 4.22. Percentage of structurally deficient sections with declining surface indices](image)

(a) RWD tested sections

(Figure cont’d)
4.7.4. Risk Analysis Based on 95% Confidence Intervals

It was found in the previous sections that declining surface indices could not identify all structurally deficiency sections. The percentage of road sections, which would result in an inaccurate prediction of in-service structural conditions, was calculated based on 95% confidence intervals. Results presented in Table 4.13 shows that the percentage of misinterpreted sections is quite significant. For example, 40.2% of the sound road segments had ALCR deterioration rate less than the lower bound of the 95% confidence interval (i.e., -3.49). This means that even though these sections were structurally sound, these sections would be considered structurally deficient if only assessed by the ALCR index. Similarly, 50.4% of the deteriorated road segments had ALCR index greater than the upper bound (i.e., -4.30). This means that even though these sections were structurally deficient, these sections would be considered structurally sound if only assessed by the ALCR index.
Table 4.13. Results of risk analysis associated with considering only deterioration rates of indices

<table>
<thead>
<tr>
<th>Test type</th>
<th>Surface Indices</th>
<th>95% confidence interval</th>
<th>Percentage of road segments</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Sound</td>
<td>Deteriorated</td>
</tr>
<tr>
<td>RWD tested sections</td>
<td>ALCR</td>
<td>(-3.49, -3.24)</td>
<td>(-4.61, -4.30)</td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>(-1.91, -1.77)</td>
<td>(-1.92, -1.75)</td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>(-1.26, -1.06)</td>
<td>(-1.23, -0.99)</td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>(-1.18, -1.09)</td>
<td>(-1.32, -1.22)</td>
</tr>
<tr>
<td>TSD tested sections</td>
<td>ALCR</td>
<td>(-2.13, -1.31)</td>
<td>(-2.99, -1.49)</td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>(-2.17, -1.29)</td>
<td>(-2.38, -1.52)</td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>(-1.74, -1.22)</td>
<td>(-1.85, -1.09)</td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>(-0.81, -0.54)</td>
<td>(-2.16, -0.99)</td>
</tr>
</tbody>
</table>

4.8. Cost Implication of Misinterpreted Sections

As demonstrated in this study, surface indices cannot be used as a reliable predictor of structural capacity possibly leading to erroneous decision-making in treatments’ selection. If no structural condition data are collected or considered by the state to assist in the process of selecting a suitable treatment strategy, it may lead to two types of errors (Zhang et al. 2016): adding structure to a pavement that does not require it (Type I error – False Positive) and not adding structure to a pavement that requires it (Type II – False Negative). Examples of Type I errors include using treatments such as pavement reconstruction, medium overlays, and in some cases thin overlays on pavements that are not structurally deficient and that only necessitate functional repairs. Type II error examples include using functional treatments such as microsurfacing and surface treatment on pavements that are structurally deficient.

The cost implication of misinterpreted sections was evaluated for RWD since the cost of RWD testing was available (Elseifi et al. 2017). The cost-benefit of incorporating structural indices in
PMS decision-making was assessed based on the number of misinterpreted sections by only using surface indices or their deterioration rates in the decision-making process. Results presented in Table 4.11 and Table 4.13 illustrating the percentage of misinterpreted RWD sections were used in the cost analysis. Treatment costs and RWD testing cost assumed in the analysis are illustrated in Table 4.14. To quantify the cost of Type-I error, the cheapest structural treatment (i.e., thin AC overlay) was assumed with a cost of $9,200 per 0.1-mile segment/lane. Similarly, to quantify the cost of Type-II error, the cheapest functional treatment (i.e., chip seal) was assumed with a cost of $2,500 per 0.1-mile segment/lane. These treatment costs represent the lowest cost implication associated with Type-I and Type-II errors. RWD cost per lane-mile was found to range between $42 and $105 per lane-mile, see Table 4.14 (Elseifi et al. 2017).

Table 4.14. Typical cost associated with functional and structural treatments and RWD testing (Elbagalati et al. 2017)

<table>
<thead>
<tr>
<th>Cost category</th>
<th>Treatment Type</th>
<th>Construction cost/mile/2-lanes</th>
<th>Construction cost/0.1-mile/lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>Treatment Cost</td>
<td>Microsurfacing</td>
<td>$67,000</td>
<td>$3,350.0</td>
</tr>
<tr>
<td></td>
<td>Surface Treatment</td>
<td>$50,000</td>
<td>$2,500.0</td>
</tr>
<tr>
<td></td>
<td>Thin Overlay</td>
<td>$184,000</td>
<td>$9,200.0</td>
</tr>
<tr>
<td></td>
<td>Medium Overlay</td>
<td>$334,000</td>
<td>$16,700.0</td>
</tr>
<tr>
<td></td>
<td>Structural Overlay</td>
<td>$682,000</td>
<td>$34,100.0</td>
</tr>
<tr>
<td></td>
<td>In-Place Stabilization</td>
<td>$496,000</td>
<td>$24,800.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>RWD Testing Cost</th>
<th>Functional Class</th>
<th>Cost $ per lane-mile</th>
<th>Testing cost for 100 lane-mile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Interstate</td>
<td>$42</td>
<td>$4,200</td>
</tr>
<tr>
<td></td>
<td>Secondary roads</td>
<td>$70</td>
<td>$7,000</td>
</tr>
<tr>
<td></td>
<td>Local roads</td>
<td>$105</td>
<td>$10,500</td>
</tr>
</tbody>
</table>

As shown in Table 4.15, the cost implication of Type-I and Type-II errors was estimated to be about $508,600 in total for decisions based on surface indices and about $529,600 in total for
decisions based on the rates of deterioration per 100 misinterpreted segments. As expected, the total cost of Type I error, which is adding structure to a pavement section that does not require it, was more significant than the cost of Type II error, because AC overlays are nearly four times more expensive than surface treatments. While one may argue that adding structure to a road that does not require it is not a lost investment, it is definitely not cost-effective especially during times of reduced state budgets and the annual backlog of projects that are not funded due to budget limitations.

Table 4.15. Cost implication of misinterpreted sections

<table>
<thead>
<tr>
<th>Criterion</th>
<th>Type of indices</th>
<th>Misinterpreted as sound per 100 segments</th>
<th>Treatment cost (Type I error)</th>
<th>Misinterpreted as deteriorated per 100 segments</th>
<th>Treatment cost (Type II error)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Based on surface indices</td>
<td>ALCR</td>
<td>52</td>
<td>$478,400</td>
<td>35</td>
<td>$87,500</td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>46</td>
<td>$423,200</td>
<td>38</td>
<td>$95,000</td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>60</td>
<td>$552,000</td>
<td>38</td>
<td>$95,000</td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>55</td>
<td>$506,000</td>
<td>42</td>
<td>$105,000</td>
</tr>
<tr>
<td></td>
<td><strong>Savings</strong></td>
<td></td>
<td></td>
<td><strong>Savings</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$422,150 (per 100 segments)</td>
<td>$86,450 (per 100 segments)</td>
</tr>
<tr>
<td>Based on deterioration rates of indices</td>
<td>ALCR</td>
<td>51</td>
<td>$469,200</td>
<td>41</td>
<td>$102,500</td>
</tr>
<tr>
<td></td>
<td>RNDM</td>
<td>52</td>
<td>$478,400</td>
<td>42</td>
<td>$105,000</td>
</tr>
<tr>
<td></td>
<td>RUTT</td>
<td>62</td>
<td>$570,400</td>
<td>25</td>
<td>$62,500</td>
</tr>
<tr>
<td></td>
<td>ROUGH</td>
<td>61</td>
<td>$561,200</td>
<td>34</td>
<td>$85,000</td>
</tr>
<tr>
<td></td>
<td><strong>Savings</strong></td>
<td></td>
<td></td>
<td><strong>Savings</strong></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$468,150 (per 100 segments)</td>
<td>$61,450 (per 100 segments)</td>
</tr>
</tbody>
</table>
CHAPTER 5. SUMMARY AND CONCLUSIONS

The research objective of this study was to assess the feasibility of using TSD measurements at the network-level for pavement conditions structural evaluation in Louisiana. To achieve the objectives of the study, TSD and FWD measurements were collected in District 05 of Louisiana and data were available from experimental programs conducted at the MnROAD research test facility and in Idaho. TSD measurements were compared with FWD deflection measurements to evaluate the level of agreement and difference between the two devices. Based on this evaluation, an SN predictive model was developed and validated to assess the structural conditions of in-service pavements. The model was then used to identify structurally sound and structurally deficient in-service pavements. Furthermore, the study assessed whether the use of surface indices only to identify structurally deficient sections is feasible.

5.1. TSD MEASUREMENTS EVALUATION AND COMPARISON WITH FWD

- Based on ANOVA and Limit of Agreement plots, it can be concluded that the deflection reported by both FWD and TSD for the same locations are statically different, which is reasonable given the differences in loading characteristics and load type between the two devices.

- It is concluded that surface roughness had an effect on TSD loading variation and subsequent field measured deflections. From simulation results conducted using 3D Move, it is also concluded that the increase in vehicle speed caused a decrease in the deflections.
5.2. Development of a TSD-Based SN Prediction Model

- The present study successfully developed a model to predict in-service SN based on TSD deflections at 0.01-mile intervals of a road section. The non-linear regression model showed an acceptable prediction accuracy with a coefficient of determination of 0.92 and RMSE of 0.88 in the development phase and a coefficient of determination of 0.88 and an RMSE of 1.06 in the validation phase. The model was successfully developed and validated with SN calculated based on TSD and FWD deflection data obtained from two contrasting data sets from Louisiana and Idaho.

- The importance of considering structural indices along with functional indices was demonstrated based on ANOVA analysis and extracted cores.

- The estimated percentage loss in structural capacity from the model was in good agreement with the percentage loss calculated from FWD.

- Core samples showed that sections that were predicted to be structurally deficient suffered from asphalt stripping and debonding problems. Yet, some of these sections were in very good conditions according to their functional conditions.

5.3. Relationship between Surface Indices and Structural Conditions

- The main objective of this task was to assess whether the use of surface indices only or the declining rates of these indices to identify structurally damaged sections is feasible instead of relying on RWD and TSD estimated pavement structural indices. Results of the analysis showed that structural deficiency, rates of deterioration, and surface indices were correlated to a certain extent. These results were expected, as pavements that are
structurally deteriorated will exhibit surface deficiencies over time. Yet, surface indices cannot be used as a reliable predictor of structural capacity.

- For RWD, the most accurate surface index, which was the alligator cracking surface index, erroneously identified 35% of structurally sound sections as structurally deficient and 51.5% of structurally deficient sections as structurally sound. Similar results were obtained for the TSD; in this case, the most accurate surface index, which was the random cracking index, erroneously identified 16.7% of structurally sound sections as structurally deficient and 51.0% of structurally deficient sections as structurally sound.

- The cost implication associated with misinterpreted sections from functional indices was investigated. The incorporation of structural indices into PMS decision-making process is expected to provide significant savings to state agencies. The results presented in this study would allow practitioners to be aware of the risk associated with decision-making solely based on surface indices and the benefits expected from incorporating structural indices into PMS.
RECOMMENDATIONS

Based on the results and findings, the study recommends the following course of actions for future studies:

- Additional TSD and FWD comparison testing are recommended to be conducted throughout the state of Louisiana to validate and fine-tune the models and procedures presented in this study.
- Research should develop a methodology to incorporate TSD measurements in PMS decision-making processes and in pavement design.
- With the availability of additional measurements, the effects of surface roughness and vehicle speed should be further investigated.
- Cost-effectiveness of TSD measurements should be investigated in future studies.
REFERENCES


VITA

Zia Uddin Ahmed Zihan was born in Bangladesh to Mr. Hafez Ahmed and Samsunnahar Yeasmin on April 07, 1993. He finished his Bachelor’s in Civil Engineering from Islamic University of Technology, Bangladesh. Currently, he is anticipating the completion of Master’s in Civil Engineering (with a concentration in Transportation) at Louisiana State University, Baton Rouge. During his graduate studies, he worked as a Graduate Research Assistant for Dr. Mostafa Elseifi with a research focus on pavement evaluation and maintenance. He contributed to the research advancement of pavement maintenance by publishing peer-reviewed journals and technical reports for the projects funded by sponsors. He was appreciated and awarded for the quality of research at prestigious conferences.