IMPLEMENTATION AND IMPROVEMENT OF PAVEMENT & ASSET MANAGEMENT SYSTEMS IN TENNESSEE

Final Report
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16. Abstract

This project aims to assist in implementation of PMS including data collection, condition report and strategy analysis and help developing the asset management plan for the state. The research team accomplished several proposed tasks including: defining the most recent highway information and pavement maintenance treatments in PMS; examining "untreated" and "poor" pavement segments; updating pavement related data; generating annual pavement condition report; producing the MAP21 pavement condition report for the FHWA; conducting pavement maintenance strategy analysis. In addition to those routine work, the research team also worked on several tasks for pavement maintenance and management, including:

1. All the poor pavement sections in Tennessee were examined by investigating the pavement condition data in the PMS and visual inspection using the TRIMs. The causes of those poor sections were analyzed. By setting the roughness, rutting and distress thresholds, poor pavement segments in Tennessee were identified. Related traffic and maintenance information of those pavement segments were also collected.
2. The optimized sampling method for pavement inspection for Tennessee were analyzed based on TDOT's inspection protocol and historical passing percent.
3. Failure probability of typical pavement preventive maintenance treatments were analyzed using survival analysis. The pre-treatment performance level was identified as the most significant factor influencing treatment performance, followed by treatment type, traffic level and environmental factors.
4. To calibrate the weights of single pavement distress indicator and the overall pavement distress index (PDI), the structural equation modeling method was utilized to find the correlations between different pavement condition indices.
5. The effects of construction considerations on performance maintenance treatments were analyzed by data mining technology. To overcome the difficulties in those factor analysis, the decision tree method was adopted to identify and quantify the influence of those factors.

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EXECUTIVE SUMMARY

After the initialization of software systems, additional work is usually required to effectively utilize the Pavement Management System (PMS) and Asset Management System (AMS) for maintenance strategy analysis. This project aims to assist in implementation of PMS including data collection, condition report and strategy analysis and help developing the asset management plan for the state. The research team accomplished several proposed tasks including: defining the most recent highway information and pavement maintenance treatments in PMS; examining “untreated” and “poor” pavement segments; updating pavement related data; generating annual pavement condition report; producing the MAP21 pavement condition report for the FHWA; conducting pavement maintenance strategy analysis.

In addition to routine work for PMS and AMS, the research team also worked on several tasks for pavement maintenance and management. The following summarized the main works:

1. All the poor pavement sections in Tennessee were examined by investigating the pavement condition data in the PMS and visual inspection using the TRIMs. The causes of those poor sections were analyzed. By setting the roughness, rutting and distress thresholds, poor pavement segments in Tennessee were identified. Related traffic and maintenance information of those pavement segments were also collected.
2. The optimized sampling method for pavement inspection for Tennessee were analyzed based on TDOT’s inspection protocol and historical passing percent.
3. Failure probability of typical pavement preventive maintenance treatments were analyzed using survival analysis. The pre-treatment performance level was identified as the most significant factor influencing treatment performance, followed by treatment type, traffic level and environmental factors.
4. To calibrate the weights of single pavement distress indicator and the overall pavement distress index (PDI), the structural equation modeling method was utilized to find the correlations between different pavement condition indices.
5. The effects of construction considerations on performance maintenance treatments were analyzed by data mining technology. To overcome the difficulties in those factor analysis, the decision tree method was adopted to identify and quantify the influence of those factors.

1 INTRODUCTION

1.1 Background

The Pavement Management System (PMS) has been proven to an effective tool for optimizing pavement condition cost-effectively. The successful implementation of PMS involves the collection of high quality pavement condition data, producing accurate pavement condition report and conducting cost-effective maintenance strategy analysis under different funding and pavement condition levels. TDOT has been actively committed in promoting PMS program which can help preserving the highway system in a cost-effective manner. From 2007 to 2011, TDOT conducted a two-phase study on optimizing pavement preventative maintenance by evaluating the cost-effectiveness of different maintenance treatments and calibrating the treatment performance models in the PMS. From 2013, TDOT initiate a new project to investigate and improve the data quality in current PMS. To effectively utilize the optimization function of PMS, it is of great importance to manage the various components of PMS in terms of data collection, pavement condition evaluation, and strategy analysis.

In addition to pavement maintenance, the evaluation and preservation of other existing highway assets are also important for the public transportation. Wisconsin and Georgia DOTs have conducted compressive studies on performance measurement, management and benefit-cost analysis for transportation asset management since 2008. Recently, TDOT is developing a new Asset Management System (AMs) and integrating it with traffic and geographic systems. It is of great importance to set up a practical asset management plan to assist engineers selecting proper preservation measures. The asset management plan will involve developing inventory data, defining reliable performance index and maintenance criteria, and cost-benefit maintenance analysis of highway assets based on the developed asset management system.

With the implementation of AMS software, a Maintenance Management System (MMS) is developed to help TDOT track its assets and apply the maintenance funds correctly and effectively for all assets. Since pavements account for 80% of the value of total assets, the integration of PMS and MMS is critical for effective decision making in the MMS. To insure that maintenance activities are properly budgeted, scheduled, and accomplished, it is important to correlate the maintenance requirement information (MRI) data from MMS to PMS.

1.2 Objective

The main objective of this project is to produce an implementation guide of PMS including data collection, condition report and strategy analysis, to set up the AMS; and to integrate MMS with the two systems. This objective will be accomplished by a comprehensive investigation of the current assets systems.

1.3 Scope

The scope of the research work can be summarized as follows:

1. To maintain and utilize PMS for pavement maintenance strategy analysis
2. To set up asset management system (AMS).
3. To integrate PMS and AMS with MMS.

The proposed timing of milestones is presented in Table 1.

Table 1 Proposed Timing of Milestones

Milestones		Date	Budget (Cumulative)
1	Collection of data from Manley consultant	April 1, 2013	10% (10%)
2	Production of annual pavement condition report	May 31, 2013	10% (20%)
3	Pavement maintenance strategy analysis	June 30, 2013	10% (30%)
4	Work as a liaison for the UT research project	July 31, 2014	10% (40%)
5	Correlate MRI data from MMS to PMS	October 31, 2014	15% (55%)
6	Help develop a new guideline for the data collection	December 31, 2014	15% (70%)
7	Help develop an asset management plan	February 28, 2014	15% (85%)
8	Development of an asset management report	March 20, 2014	10% (100%)

The proposed period for this research project will be 12 months after receiving the fully executed contract or a letter authorizing the starting date. Table 2 presents the listed tasks as described in the above paragraph.

Table 2 Schedule and Proposed Timing (months) of Tasks

Tasks		1	2	3	4	5	6	7	8	9	10	11	12
1	Collection of data from Manley consultant												
2	Production of annual pavement condition report												
3	Pavement maintenance strategy analysis												
4	Work as a liaison for the UT research project												
5	Correlate MRI data from MMS to PMS												
6	Help develop a guideline for the data collection												
7	Help develop an asset management plan												
8	Development of an asset management report												

2 QUALITY CONTROL OF PAVEMENT PROFILE DATA

Accurate pavement condition data are critical to improve the effectiveness and efficiency of the PMS system. As shown in Figure 1, the National Cooperative Highway Research Program (NCHRP) Synthesis 401 recommended a complete quality management system to improve the quality of pavement condition data. One of the main methods for pavement data quality management is the calibration and verification of equipment before the data collection. (Flintsch and McGhee, 2009)

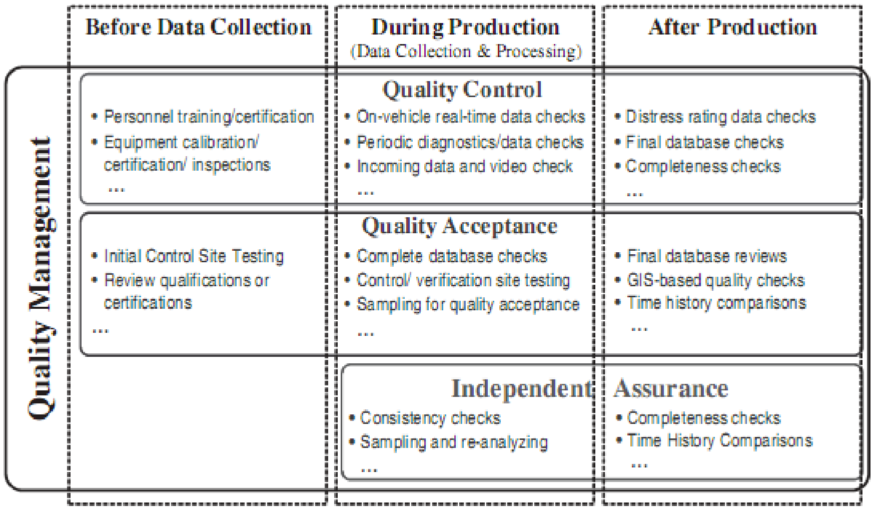


Figure 1 Framework of quality management for pavement data collection

To check the pavement profile data collected by the contracted data collector Mandli, TDOT pavement engineers used its own pavement profiler to collect pavement roughness data from randomly selected road sections on Tennessee highways. The "ProVAL" software was then used to compare the data collected from the two pavement profilers as validation.

2.1 Data Collection

TDOT pavement management engineer randomly selected 13 road sections located in Region 3 and 4, and collected the pavement roughness data using TDOT's profiler. The data collected by Mandli were extracted from the PMS system. Table 3 lists all the 13 test sections, the log-mile location of the two dataset were trimmed to match with each other.

Table 3 Summary of paired data samples

Sample no.	County name	County no.	Route no.	Begin log mile	End log mile
02SR080	Benford	2	80	0	11
09SR114	Carroll	9	114	0	20
11SR249	Cheatham	11	249	2.78	16.52
28SR273	Giles	28	273	0	10
42SR231	Houston	42	231	0	6
50SR242	Lawrence	50	242	22.9	32.9
52SR129	Lincoln	52	129	0	7

59SR050	Marshall	59	50	6.46	19.46
63SR238	Montgomery	63	238	0	8
74SR025	Robertson	74	25	0	17
75SR010	Rutherford	75	10	0	9
80SR025	Smith	80	25	0	10
83SR025	Summer	83	25	0	12

2.2 Comparison and Analysis

ProVAL is an engineering software specifically developed for viewing and analyzing pavement profile data. The "profile synchronization" function of ProVAL was used to compare the two pavement profile dataset and the cross-correlation (CC) value was used to measure the correlations. A CC value of 100% indicates the two data are perfectly correlated; 0 indicates no correlation; and -100% indicates perfectly but "negatively" correlated. Since the starting points of the two dataset may not match, the Profile Synchronization function can find the optimal offset of the profiles to match the starting point of the basis profile. The CC value represents repeatability for two measurement of the same device, reproducibility for two measurements of different device and accuracy when one device is to be corrected. (Karamihas, 2004)

Figure 2 shows the original left profile data of sample 83SR025 (state route 25 at Summer County). Figure 3 shows the correlation values for different offsets of sample 83SR025. It can be seen that the correlation is very low. This is mainly caused by wheel wondering and noises in the data. All the 13 test sections were examined and were found to have low correlations, less than 10%.

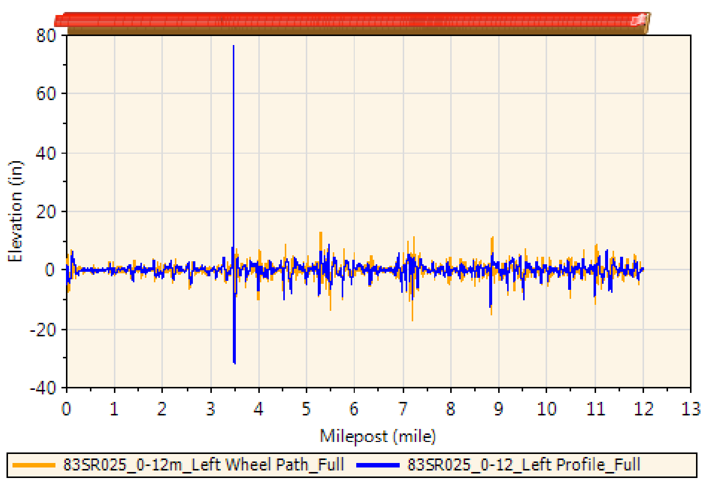


Figure 2 Left profile data of sample 83SR025 (0-12 mile)

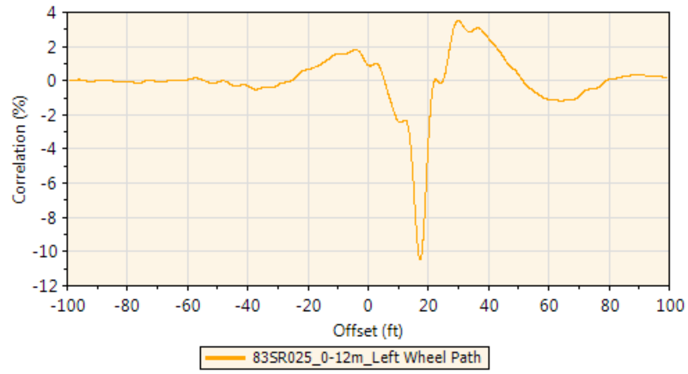


Figure 3 Correlations of the left profile data of sample 83SR025 (0-12 mile)

Because longer profiles tend to produce lower correlation due to wheel wandering, the data from 0-1 mile section was extracted for the correlation analysis. Figure 4 shows the trimmed 0-I mile left profile data of sample 83SR025. Generally, the two profiles followed the same trend. It can be seen that when setting 20 ft offset, the correlation is as high as 50%, which is greatly improved compared with the whole 12 mile profile data.

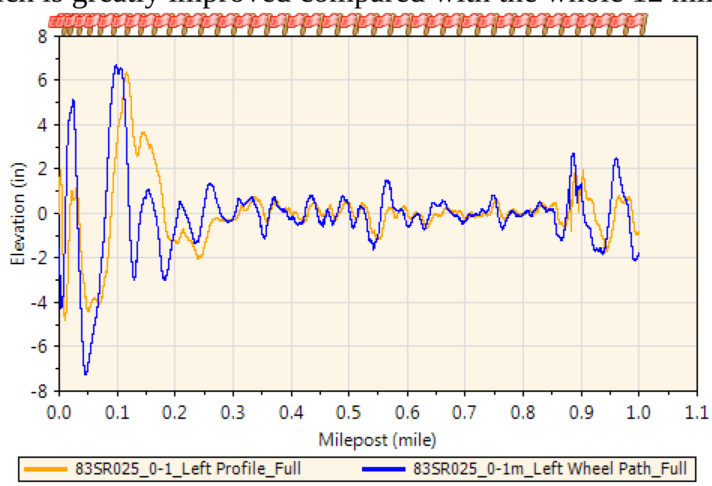


Figure 4 Left profile data of sample 83SR025 (0-1 mile)

Figure 5 Correlations of the left profile data of sample 83SR025 (0-1 mile)

Figure 4 shows that the measured elevation is extremely high within the first 0.1 mile. This is probably caused by the joint of bridge deck. Again, the 0.3 to 0.8 mile section was extracted for comparison. Figure 6 and Figure 7 show the left profile data and the correlation. It can be seen the highest correlation was achieved at 5 ft offset.

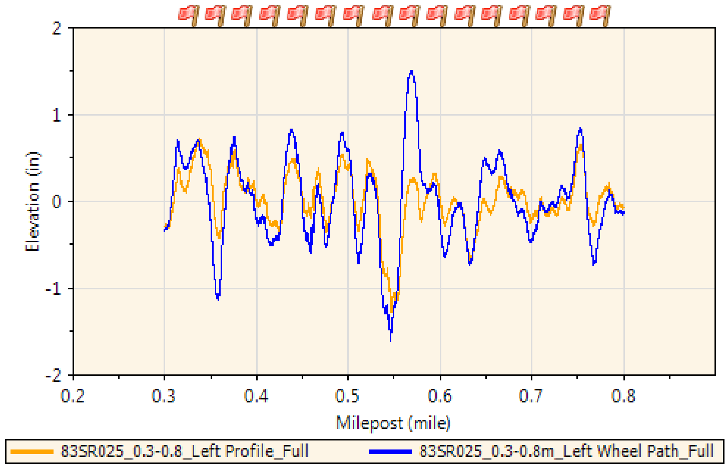


Figure 6 Left profile data of sample 83SR025 (0.3-1.8 mile)

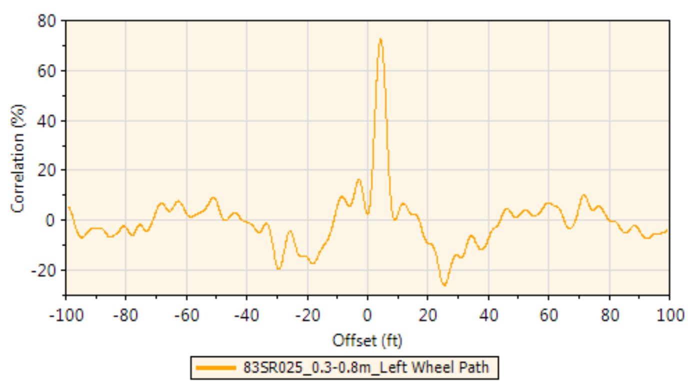


Figure 7 Correlations of the left profile data of sample 83SR025 (0.3-0.8 mile)

2.3 Summary

TDOT randomly selected 13 road sections in Region 3 and 4 and validated the pavement profile data collected from Mandli by comparing them with the data collected by TDOT's own pavement profiler. The "profile synchronization" function in the ProVAL software was utilized to analyze the correlations of the two dataset. Generally, short section length has higher correlation due to reduced noise and wheel wandering. Optimized section length needs to be determined for effective validation in the following research.

3 UTILIZING PMS DATA AS MRI

3.1 Definition of maintenance treatment methods

In the previous research project titled “Optimizing Pavement Preventive Maintenance Treatment Applications in Tennessee (Phase II)”, conducted by University of Tennessee, a series of asphalt resurfacing treatment methods were summarized by investigating historical maintenance projects in Tennessee (Table 4). The treatment performance models were also developed and calibrated for those 9 treatments at different levels of traffic, pre-treatment pavement condition. The identified resurfacing treatments methods were classified based on overlay thickness and milling depth. Table 5 lists the default pavement maintenance methods in HPMA, which also include 9 asphalt resurfacing treatment with various amount of milling and overlays. To utilize the developed model for future maintenance strategy analysis, it is necessary to match up the two asphalt resurfacing treatment.

Table 4 Classified asphalt resurfacing treatment methods

Milling depth (in.)		Total overlay thickness (in.)		Treatment method	Unit Cost ($/yard^2)	Sample no.
Value	Level	Value	Level			
0	0	1.25	1~2	O1	$9.4	125
0	0	2.5	2~3	O2	$18.1	188
0	0	3.75	>3	O3	$26.8	29
1.25	1~2	1.25	1~2	M1O1	$25.5	88
1.25	1~2	2.5	2~3	M1O2	$34.2	15
1.25	1~2	3.75	>3	M1O3	$42.9	101
2.5	>2	1.25	1~2	M2O1	$41.6	10
2.5	>2	2.5	2~3	M2O2	$50.3	14
2.5	>2	3.75	>3	M2O3	$59.0	55

Table 5 Default treatment methods in HPMA

Code	ID	Activity	Thickness	Typical structure in Tennessee
1	M1_2	Mill&Replace1"-2"	Mill&Replace1"-2"	MR1.25"
2	M2_4	Mill&Replace2"-4"	Mill&Replace2"-4"	MR3.25"
3	MO2200	MR1-2"+ Overlay <200PSY	MR1-2"+ 1.5” overlay	MR1.25"+1.25”D
4	MO4200	MR2-4"+ Overlay 200-400PSY	MR2-4"+1.5” overlay	MR3.25"+2”BM2+1.25”D
5	MO2400	MR1-2"+ Overlay <400PSY	MR1-2"+1.5-3” overlay	MR1.25"+2”BM2+1.25”D
6	MO4400	MR2-4"+ Overlay 200-400PSY	MR2-4"+1.5-3” overlay	MR3.25"+2”BM2+1.25”D
7	O200	Overlay<200PSY	Overlay < 1.5”	1.25”D
8	O400	Overlay200-400PSY	Overlay 1.5-3”	2”BM2+1.25”D
9	O>400	Overlay>400PSY	Overlay >3”	2.5”BM2+1.25”D
10	RECON	Reconstruction		
11	RO800	RubblizeOL900PSY	Rubberized overlay 9”	I75 Anderson county
12	GR	General Rehabilitation		
13	GRO400	GR3,4,5,0PSY		
14	O400-C	Overlay400PSY(C)	3.25” Overlay over concrete	
15	OC-BIT	Original Bitumen Construction		
16	OC-CON	Original Concrete Construction		
17	RECON2	Reconstruction		
18	CS650	Crack&Seat+OL900+PSY	Concrete crack&seat + 3.5”ACRL+2.5”BM2+1.25”D	
20	Seal	SurfaceTreatment		

By assuming the specific gravity of asphalt mixture as 2750 kg/m^3 = 4636.5 lb/yd^3, the thicknesses of the default treatments defined in HPMA can be calculated as shown in Table 5. Based on the thicknesses, the relationships between identified treatments and the defaults in HPMA can be summarized in Table 6. Then, both the treatment performance models developed in the previous preventive maintenance project and the default models in HPMA can be utilized in the network level strategy analysis.

Table 6 Relationships between identified treatments and the defaults in HPMA

Milling depth (in.)		Overlay thickness (in.)		Treatment		Description
Value	Level	Value	Level	Defined	Default	
1.5	1-2	1.5	1.5	M1O1	MO2200	MR1-2"+ 1.5” overlay
1.5	1-2	2.25	1.5-3	M1O2	MO2400	MR1-2"+1.5-3” overlay
1.5	1-2	3.75	>3	M1O3	MO2200+ O400	MR1-2"+ Overlay >3”
2.5	2-4	1.5	1.5	M2O1	MO4200	MR2-4"+1.5” overlay
2.5	2-4	2.25	1.5-3	M2O2	MO4400	MR2-4"+1.5-3” overlay
2.5	2-4	3.75	>3	M2O3	MO4200+ O400	MR1-2"+ Overlay >3”
0	0	1.5	1.5	O1	O200	Overlay < 1.5”
0	0	2.25	1.5-3	O2	O400	Overlay 1.5-3”
0	0	3.5	>3	O3	O>400	Overlay >3”

3.2 Identification of unsurfaced pavement sections

A VBA (Visual Basic for Applications) program was programmed to identify all the untreated pavement sections in Tennessee. A total of 482 untreated sections were identified by screening 6000 historical maintenance records. With the updated maintenance records in the future, the program can be used repeatedly to find the update gaps. Table 7 shows an example of the maintenance records and identified pavement section.

Table 7 Example of identified untreated sections within each county

	Name	Length	Start log mile	End log mile
Maintenance records	10SR0361	27.62	0	2.67
	10SR0911	20.97	0	2.65
	10SR0911	20.97	12.82	17.05
	10SR0911	20.97	17	20.96
Identified untreated sections	10SR0361	27.62	2.67	27.62
	10SR0911	20.97	2.65	12.82

The maintenance records sheet needs to be pre-processed. Firstly, a section name consisting of county number, route type, route number and sequence number was defined by merging certain columns. Then, the total length for each section could be identified by using the merging function. After sorted by sorted by end log mile, start log mile, year and the section name, the old maintenance records sheet is ready for the process. The VBA application firstly eliminated all the incomplete rows, which have empty log mile or length information. Then, it searched all the gaps between treated sections. The following codes are the VBA program for identifying untreated pavement.

```
Public Sub gap2()

    Dim i, j, n1, n2, m, n As Integer, k As String
    i = 2
    m = ActiveSheet.UsedRange.Rows.Count + 1                    'end of cycle

    Do While i < m
        If (Range("O" & i) = "" Or Range("P" & i) = "" Or Range("B" & i) = "") Then   'clean up empty rows
            Rows(i).Delete
            i = i - 1
            m = ActiveSheet.UsedRange.Rows.Count
        End If
        i = i + 1
    Loop

    i = 2                                                'cycle of the whole
    n1 = 2                                               'cycle of each section
    m = ActiveSheet.UsedRange.Rows.Count + 1                    'end of cycle
    n = m                                                'start of identified gaps

    Do While i < m
        If (Range("A" & i + 1) <> Range("A" & i)) Then
            j = n1
            n2 = i                                                        'end of section
            k = Range("P" & n1)                                           'the first end log mile

            If (Range("O" & n1) > 0.2) Then                                  'find gap at the beginning
                Rows(n1).Copy
                Range("A" & n + 1).Select
                ActiveSheet.Paste
                Range("O" & n + 1) = 0
                Range("P" & n + 1) = Range("O" & n1)
                n = n + 1
            End If

            Do While j < n2                                       'find gap between maintenance sections
                If (Range("O" & j + 1) > k + 0.2) Then
                    Rows(j).Copy
                    Range("A" & n + 1).Select
                    ActiveSheet.Paste
                    Range("O" & n + 1) = Range("P" & j)
                    Range("P" & n + 1) = Range("O" & j + 1)
                    n = n + 1
                End If
                j = j + 1
                k = Application.WorksheetFunction.Max(Range("P" & n1 & ":" & "P" & j))    'highest maintenance length
            Loop

            If (k + 0.2 < Range("B" & n2)) Then                           'find gap at the end
                Rows(n2).Copy
                Range("A" & n + 1).Select
                ActiveSheet.Paste
                Range("O" & n + 1) = k
                Range("P" & n + 1) = Range("B" & n2)
                n = n + 1
            End If

            n1 = i + 1
        End If
        i = i + 1
    Loop
End Sub
```

3.3 Utilizing PMS Data as MRI

In Tennessee, the pavement performance data are regularly collected every one or two years. Two types of pavement performance data are collected: the road profiler data including international roughness index and rutting depth; and the various pavement distresses identified from the pavement surface images captured by the survey vehicles. Generally, the interstates were collected every year and on both directions; whereas the state routes were collected every two years and only on one direction, from south to north or west to east. TDOT divides the whole state into 4 regions. Region 1 and 2 pavement performance are surveyed on odd years while Region 3 and 4 are surveyed on even years.

Each year, the maintenance quality assurance program of TDOT selected around 8000 pavement 0.1 mile segments from the whole state for maintenance quality survey, accounting for 5.8% of the total pavement segment population in the state. To improve the precision or confidence interval of maintenance quality rating, it is expected to select as many samples as possible. However, the quality inspection process consumes considerable amount of time and budget. Therefore, utilizing the PMS pavement distress data as part of the maintenance assurance program could potentially save the cost and improve precision. The research team took deep investigation into the pavement distress data in the PMS and the inspected maintenance elements for the maintenance quality assurance program. The objective is to identify correlations between the two data sets and try to incorporate PMS data into the maintenance quality assurance program. Table 8 shows the list of pavement distresses in PMS and the corresponding threshold value of different severity levels.

Table 8 Thresholds for different severity levels of distress

Pavement type		Code	Description	Measure	Low	Medium	High
1	Bituminous	1	Fatigue cracking	%Area			
		2	Longitudinal WPC	%Area	0.125		0.5
		3	Patching/pothole	%Area	0.125		0.5
		4	Block cracking	%Area			
		5	Rutting	Depth (in.)	0.25		0.75
		6	Transverse cracking	%Area			
		7	Longitudinal non-WPC	%Area			
		8	Long lane joint	% Length	0.125		0.5
2	Concrete	1	Faulting	%Slabs			
		2	Transverse cracking	%Slabs	0.125		0.5
		3	Longitudinal WPC	%Slabs	0.125		0.5
		4	Joint spalling	%Slabs			
		5	Patching	%Slabs			
3	Bituminous Surface Treated	1	Fatigue cracking	%Area			
		2	Block cracking	%Area	0.125		0.5
		3	Longitudinal WPC	%Area	0.125		0.5
		4	Transverse cracking	%Area	0.125		0.5
		5	Rutting	Depth (in.)			
		6	Long lane joint	%Length	0.25		0.75
		7	Patching/pothole	%Area			
		8	Raveling	%Area			
		9	Longitudinal non-WPC	%Area	0.125		0.5
4	Composite (Bituminous /Concrete)	1	Fatigue cracking	%Area			
		2	Block cracking	%Area	0.125		0.5
		3	Longitudinal WPC	%Area	0.125		0.5

		4	Transverse cracking	%Area	0.125	0.5
		5	Rutting	Depth (in.)		
		6	Long lane joint	%Length	0.25	0.75
		7	Reflective cracking	%Area		
		8	Patching/pothole	%Area		
		9	Raveling	%Area		
		10	Longitudinal non-WPC	%Area	0.125	0.5

Currently, TDOT use pass/fail method to inspect 49 maintenance elements. As shown in Table 2, the first 12 elements are pavement distresses. The current inspection pass/fail rule is that it fails as long as this type of distress appears. For example, the first one fails as long as one asphalt crack appears within the 0.1 inspected pavement segment. Table 2 also summarizes the corresponding PMS data that can be used to inspect the maintenance elements. However, it can be seen that the two "edge dropoff" elements and the "asphalt flush/heave" still need further field survey. In addition, the rutting needs to be differentiated as mainline or intersection based on the actual location of that pavement section. In all, although the current PMS pavement distress data cannot cover all the pavement maintenance elements in the MRI, it could be used to help reduce the inspection efforts of the maintenance quality assurance program.

Table 9 Thresholds for different severity levels of distress

Maintenance Elements	PMS data code
01 - ASHP CRACKING	All cracking related items in PMS
02 - ASPH POTHOLES	1-7, 3-7, 4-8
03 - ASPH ALLIGATOR CRACKING	1-1, 3-1, 4-1
04 - ASPH FLUSH/HEAVE/RAVEL	1.8, 3.8
05 - ASPH EDGE DROPOFF	
06 - MAINLINE RUTTING	1-5, 3-5, 4-5
07 - INTERSECTION RUTTING	1-5, 3-5, 4-5
08 - CONC JOINTS	2.4
09 - CONC CRACKING	2.2, 2.3
10 - CONC POTHOLES	2.5
11 - CONC EDGE DROPOFF	
12 - CONC SLAB FAULTING	2.1

3.4 Summary

By calculating the pavement thickness, the typical treatments identified in the previous pavement maintenance project by UT were matched up with the default treatments defined in HPMA. Both the default and developed performance models can be utilized in the future maintenance strategy analysis.

4 DIAGNOSIS OF HIGHWAYS IN TENNESSEE

4.1 Data Collection

To identify and diagnose poor pavement sections on Tennessee highways, the most recent pavement condition data exported from the HPMA were analyzed. TDOT collects the interstates pavement condition data every year and state route every two years. Both the roughness and distress data of 2011 and 2012 were collected. The roughness data include the left and right wheel international roughness index (IRI), the rutting depth and the pavement serviceability index (PSI) calculated by equation 1. The distress data include the extend of different severity levels of distress as listed in Table 10 and the calculated pavement distress index (PDI) calculated by equation 2 to 6.

$$PSI = 5\exp(-0.0055IRI) \tag{1}$$

Table 10 Pavement distress types in HPMA

	Bituminous pavement			Jointed concrete pavement		
No.	Name	Description	Unit	Name	Description	Unit
1	Fatg	Fatigue cracking	%Area	Falt	Faulting	%Slabs
2	LgWP	Longitudinal wheel patch cracking	%Area	Tran	Transverse crack	%Slabs
3	Pach	Patching/Pothole	%Area	Long	Longitudinal crack	%Slabs
4	Blok	Block cracking	%Area	Spal	Joint spalling	%Slabs
5	Ruts	Rutting		Ptch	Patching	%Slabs
6	Tran	Transverse cracking	%Area			
7	LgNW	Longitudinal non-wheel patch cracking	%Area			
8	LgLJ	Longitudinal lane joint	%Length			

In HPMA, PDI is calculated by the following procedure:

1. The DVs (Deduct Values) which provide the weighting for the relative importance of the distresses/severity levels in terms of the pavement performance.

$$DV = 10 \wedge (a + b * \log_{10}(PDA)) \tag{2}$$

Where, DV = calculated in the ADV_TDV model;
PDA = percent distressed area;
a , b = coefficients which define the shape of each distress at each severity level as listed in

2. The Total Distress Value (TDV) is then calculated as the sum of the individual distress values:

$$TDV = \sum DV_i \tag{3}$$

3. The Number of Equivalent Distresses (NED) is calculated as the sum of the ratios of each distress value to the maximum distress value (DV_{max}). The DV_{max} is the largest DV observed for the data). This can be expressed as:

$$NED = \sum(DV_i / DV_{max}) \tag{4}$$

Where, DV_i = distress value for distress/severity level;
DV_{max} = highest distress value observed.

4. The Adjusted Distress Value (ADV) is then calculated from the TDV based on the NED present.

$$ADV = 10\text{^}(0.0014 - 0.396*\log_{10}(NED) + 0.9565*\log_{10}(TDV)) \tag{5}$$

5. The PDI then can be calculated as the function of ADV.

$$PDI = 5 - ADV \tag{6}$$

Table 11 Parameter a and b in the DV model

Type	No.	Distress	Low severity		Moderate severity		High severity	
			a	b	a	b	a	b
Bituminous pavement	1	Fatg	-0.98	0.586	-0.893	0.649	-0.497	0.53
	2	Blok	-0.947	0.532	-0.763	0.533	-0.448	0.465
	3	LgWP	-0.98	0.586	-0.946	0.625	-0.448	0.465
	4	Tran	-0.98	0.586	-0.942	0.625	-0.394	0.402
	5	Ruts	1.1	2.5	0	0	0	0
	6	LgLj	-1.425	0.666	-1.194	0.725	-0.946	0.625
	7	Pach	-0.842	0.42	-0.763	0.533	-0.724	0.642
	8	Ravl	-0.915	0.516	-0.774	0.56	-0.448	0.465
	9	LgNW	-0.842	0.42	-0.942	0.625	-0.818	0.583
Jointed Concrete pavement	1	Falt	-1.35	0.761	-0.95	0.673	-0.58	0.592
	2	Tran	-1.52	0.761	-1.045	0.673	-0.705	0.592
	3	Long	-1.6	0.761	-1.08	0.673	-0.73	0.592
	4	Spal	-1.8	0.761	-1.25	0.673	-0.78	0.592
	5	Ptch	-1.35	0.761	-0.95	0.673	-0.64	0.592

HPMA uses pavement quality index (PQI) as the overall performance index. PQI can be calculated by using the following equation when both PDI and PSI are available.

$$PQI = PDI\text{^}0.7*PSI\text{^}0.3 \tag{7}$$

4.2 Criteria for Selecting Poor Sections

The most recent pavement condition data including both pavement profiler data and pavement distress data were exported from HPMA. A total of 138,511 pavement segments (0.1 mile per segment) containing all of the pavement condition indices were collected as shown in Figure 1.

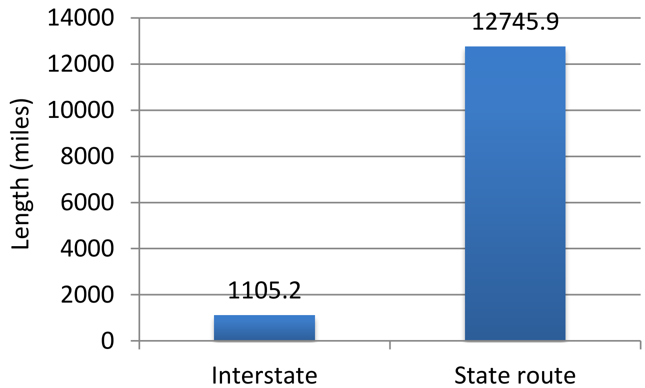


Figure 8 Total mileage of highways in Tennessee

To screen out the “poor” sections in highway, is it necessary to define appropriate criteria based on various pavement condition indices. Table 12 shows the default pavement condition category criteria defined in HPMA.

Table 12 Pavement condition category defined in HPMA

Highway type	Condition category	≤PSI	≤PDI	≤SAI	≤PQI	≥IRI
State route	Very poor	1.6	0.3	0	0.5	212
	Poor	**2.2**	**1.6**	**0**	**1.75**	**150**
	Fair	2.6	3.6	0	3.25	118
	Good	3.3	4.75	0	4.25	75
	Very good	5	5	0	5	0
Interstate	Very poor	2	0.5	0	0.75	170
	Poor	**2.6**	**1.45**	**0**	**1.75**	**120**
	Fair	3	3.35	0	3.25	95
	Good	3.6	4.9	0	4.5	60
	Very good	5	5	0	5	0

For each 0.1 mile road section, available pavement condition data includes IRI, PRI, rutting depth, PDI and the percent area of a specific distress. A pavement sections that met any of the following criteria was identified as a poor section.

1. PSI < 2 or IRI > 167 in./mile;
2. Rutting depth > 0.5 in.;
3. PDI < 1.5;
4. PQI <1.75.

4.3 Identification of Poor Pavement Sections

Figure 9 shows the number of identified “poor” pavement sections based on the pavement condition thresholds above. The “All” represents sections with any of the four indices lower than the threshold. 6% of the highways in Tennessee fell into the poor category. It is noted that there are only 300 sections with both poor PSI and rutting, or with poor PQI which is a combination of PSI and PDI.

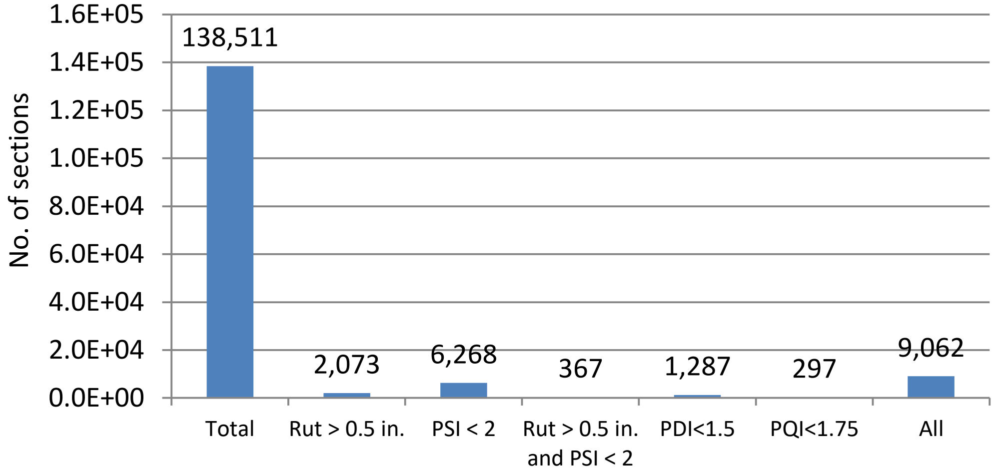


Figure 9 Identified "poor" pavement sections

To investigate the details of those identified poor sections, it is necessary to examine the field images of those poor sections by the photo log data in the Tennessee Roadway Information Management System (TRIMS) as shown in Figure 10.

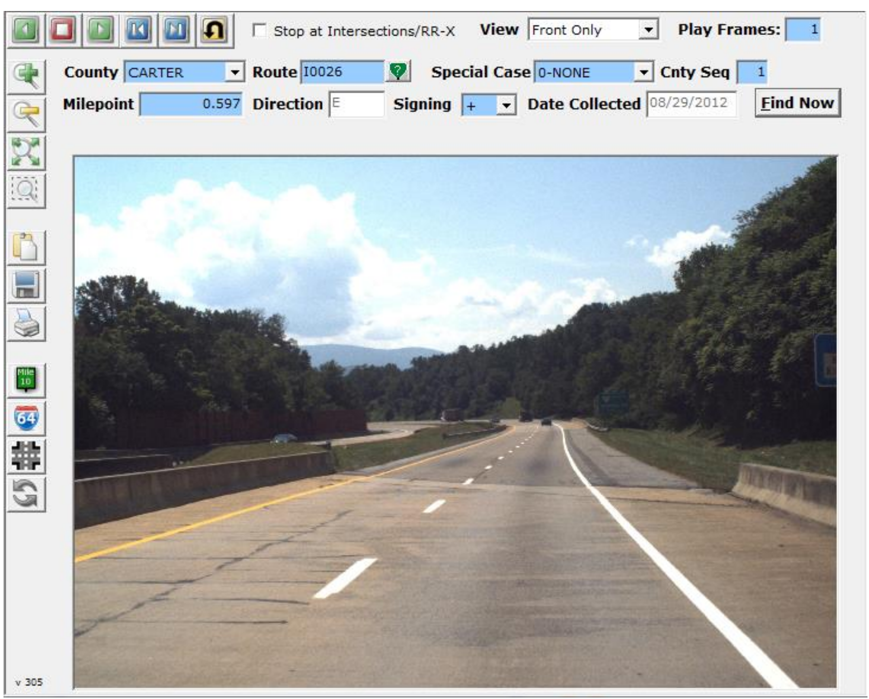


Figure 10 Interface of TRIMS

4.4 Investigation of Poor Pavement Sections

Firstly, the photo logs of all the 1287 pavement segments with PDI less than 1.5 were checked. It is noted that most of the 1287 segments are adjacent. There are a total of 168 pavement sections. The age of surface layer was calculated by checking the latest resurfacing treatment. Traffic levels were also collected. The average surface age is 15.4±5.5 years. Figure 11 shows the typical pavement segments with poor PDI. Most of the low PDI is caused by lack of maintenance.

Figure 11 Typical “poor” pavement sections with low PDI

Then, the photo logs of 367 pavement segments with both poor PSI and high rutting depth were checked. The 367 segments constitute 164 pavement sections, indicating that most of the poor PSI and rutting segment are separate. The average surface age is 15.9±6.8 years. Figure 12 shows the typical pavement segments. It is noted lack of maintenance is not the only reason cause those poor PSI and rutting sections. There are a number of poor sections located in the intersections where vehicle braking is severe.

Figure 12 Typical “poor” pavement sections with both low PSI and high rutting depth

Figure 13 summarized the percentage of pavement segments in the intersections or of high grade or curvature. Those factors potentially accelerate the deterioration of pavement, especially rutting. It can be seen that more than 20% of those poor section having at least one of the three factors.

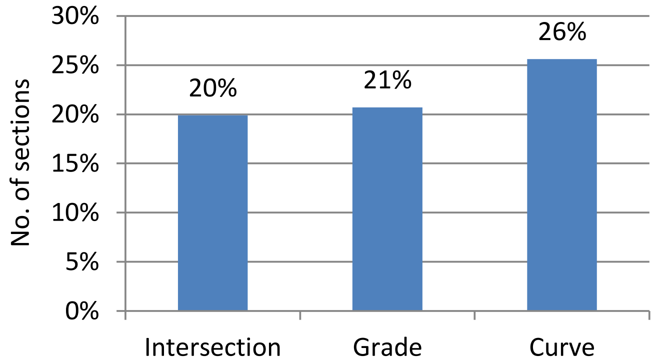
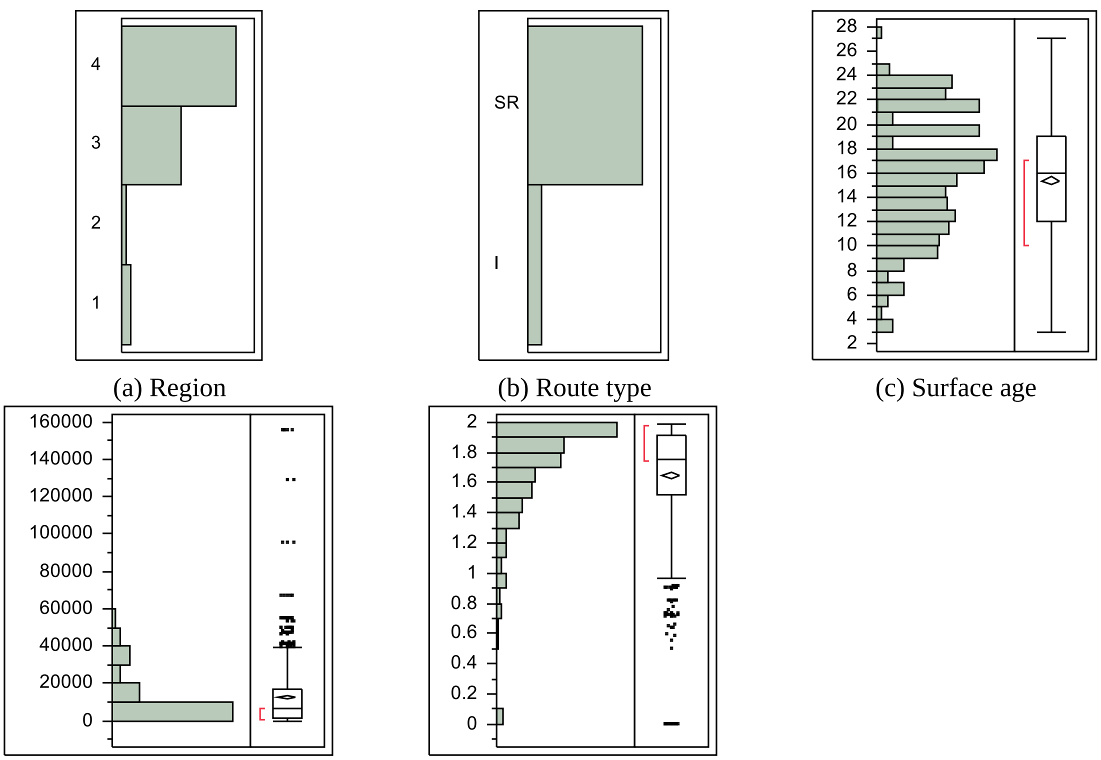


Figure 13 Identified “poor” pavement sections

4.5 Influence of Traffic and Pavement Age

Figure 15 shows the distribution of regions, route type, surface age, traffic level (AADT) and PDI of collected pavement segments with PDI value lower than 2. It can be seen that the Region 4 seems have a large number of “poor” pavement segments, which is mainly caused by the old surface age.



(a) Region (b) Route type (c) Surface age

(d) AADT (e) PDI

Figure 14 Distribution of collected data

The multiple linear regression method (Equation 8) was used to investigate the influence of traffic level and pavement surface age on the distress conditions of those poor segments.

$$Y_j = \beta_0 + \beta_1 X_1 + \cdots + \beta_i X_i + \cdots + \beta_k X_k + \varepsilon \tag{8}$$

Where, Y_j = PDI.

X_i = Traffic level and surface age.

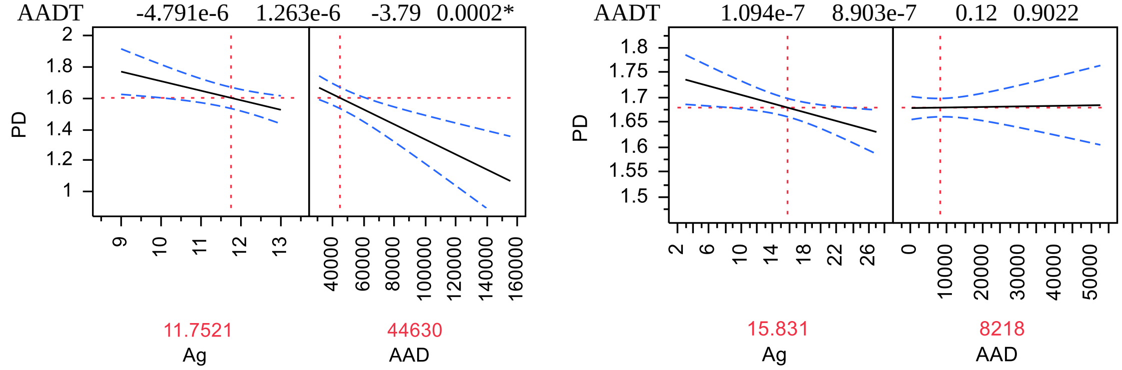
β_i = Parameters estimate of factor X_i, is the magnitude and direction change in response with each one-unit increase in predictor X_i.

ε = random error term.

The ordinary least square method was used to estimate the parameters. **Error! Reference source not found.** shows the p-value of each factor as shown in the column of "**Prob>|t|**" and the profiler of the influence of each predictor on the response. Factors with p-values lower than 0.05 are usually regarded as significant; indicating the probability of getting this result by chance is less than 5%. The middle lines within the profiler plots show how the response changes when changing the current value of an individual factor. The dotted curve surrounding the prediction trace show the 95% confidence interval for the predicted values. It can be clearly seen that surface age is a significant factors for both interstate and state route. Pavement segments with old surface tend to have severe distress. High traffic level significantly increase pavement distress of interstate but seems not a significant factor for state route. This is probably because the traffic levels of the state route clustered together. As indicated in Figure 15(d). The traffic levels of more than 95% the pavement segments are lower than 20,000 AADT.

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	2.5318596	0.297844	8.50	<.0001*
Age	-0.061014	0.023604	-2.58	0.0110*
AADT	-4.791e-6	1.263e-6	-3.79	0.0002*

Term	Estimate	Std Error	t Ratio	Prob>|t|
Intercept	1.7480518	0.031319	55.81	<.0001*
Age	-0.004389	0.001839	-2.39	0.0172*
AADT	1.094e-7	8.903e-7	0.12	0.9022



(a) Interstate (b) State route

Figure 15 Influence of different factors

4.6 Summary

By setting the roughness, rutting and distress thresholds, poor pavement segments in Tennessee were identified. Related traffic and maintenance information of those pavement segments were also collected. The average surface age of both low PDI and PSI sections is 15 years. Most of the poor sections are caused by lack of maintenance. Intersections, high grade or curvature are potential factors accelerating the deterioration of pavement, especially rutting. Statistical analyses results show that surface age significantly reduce pavement condition. Traffic level potentially accelerates the deterioration of pavement.

5 2014 ANNUAL REPORT OF PAVEMENT CONDITION

5.1 Pavement Condition Data collection

Based on the updated pavement condition data, the pavement condition were exported from the PMS first and then the average condition indices in Region 3 and 4 were calculated as shown in Table 13 and Table 14. Those average PSI, PDI and PQI values were the weighted average values calculated by the equation below.

$$Avg.Pavement\ condition = \frac{\sum Length * Pavement\ condition\ index}{\sum Length} \qquad (9)$$

Table 13 Summary of the 2014 pavement condition of Region 1

2014 Region 1 by District		Interstate				State Route			
District	County	Length (Mile)	Ave. PSI	Ave. PDI	Ave. PQI	Length (Mile)	Ave. PSI	Ave. PDI	Ave. PQI
17	10 CARTER	5.38	3.762	4.944	4.556	118.02	3.376	4.466	4.094
17	30 GREENE	64.32	3.997	4.531	4.357	189.2	3.165	3.759	3.541
17	32 HAMBLEN	19.88	3.916	4.729	4.451	85.35	3.164	3.829	3.589
17	34 HANCOCK					69.21	2.874	3.543	3.295
17	37 HAWKINS					162.36	3.173	3.893	3.632
17	46 JOHNSON					94.8	3.149	4.134	3.799
17	82 SULLIVAN	63.9	3.936	4.670	4.434	195.841	3.115	3.902	3.621
17	86 UNICOI	53.46	3.674	4.335	4.113	43.91	3.204	4.715	4.189
17	90 WASHINGTON	38.16	3.477	4.132	3.916	141.31	3.260	4.128	3.813
Average			3.818	4.487	4.267		3.174	3.990	3.699
18	1 ANDERSON	24.24	3.833	4.272	4.131	126.94	3.107	4.096	3.749
18	15 COCKE	43.84	3.567	3.315	3.375	118.44	2.873	3.858	3.502
18	29 GRAINGER					102.14	3.413	3.935	3.743
18	45 JEFFERSON	55.3	3.766	4.023	3.932	120.79	3.253	3.914	3.672
18	47 KNOX	137.5	3.684	4.178	4.013	225.98	3.123	3.999	3.692
18	78 SEVIER	9.52	4.098	4.837	4.601	168.39	3.078	4.217	3.820
18	87 UNION					71.8	3.094	4.220	3.834
Average			3.710	4.038	3.924		3.127	4.033	3.713
19	5 BLOUNT	4.5	3.497	3.239	3.307	166.74	3.053	4.053	3.692
19	7 CAMPBELL	63.28	3.988	4.565	4.378	83.93	3.324	3.675	3.550
19	13 CLAIBORNE					98.05	3.325	3.990	3.745
19	53 LOUDON	47.94	4.013	4.824	4.563	92.27	3.218	4.206	3.865
19	62 MONROE	12.98	3.565	4.325	4.080	157.84	3.110	4.250	3.856
19	65 MORGAN					108.02	3.351	4.144	3.867
19	73 ROANE	45.96	3.621	4.409	4.152	127.44	3.401	4.257	3.964
19	76 SCOTT					70	3.371	3.232	3.207
Average			3.854	4.543	4.320		3.244	4.037	3.753

Table 14 Summary of the 2014 pavement condition of Region 2

2014 Region 1 by District		Interstate				State Route			
District	County	Length (Mile)	Ave. PSI	Ave. PDI	Ave. PQI	Length (Mile)	Ave. PSI	Ave. PDI	Ave. PQI
27	14 CLAY					68.82	3.342	4.198	3.907
27	18 CUMBERLAND	72.18	4.121	4.154	4.140	154.36	3.451	3.994	3.802

27	21 DEKALB					105.38	3.310	4.526	4.112
27	25 FENTRESS					113.07	3.298	3.900	3.689
27	44 JACKSON					126.14	3.161	4.516	4.046
27	67 OVERTON					140.88	3.333	4.286	3.958
27	69 PICKETT					45.09	3.185	4.248	3.881
27	71 PUTNAM	74.18	4.207	4.577	4.461	140.52	3.249	4.272	3.924
27	93 WHITE					97.74	3.459	4.418	4.093
Average			4.165	4.368	4.303		3.317	4.255	3.932
28	4 BLEDSOE					68.09	3.290	3.375	3.292
28	8 CANNON					79.9	3.379	4.230	3.936
28	16 COFFEE	60.32	4.178	4.556	4.435	110.47	3.623	4.384	4.135
28	26 FRANKLIN					178.37	3.481	4.296	4.024
28	31 GRUNDY	14.62	3.755	4.122	4.002	99.2	3.451	4.005	3.811
28	58 MARION	64.26	4.063	4.781	4.550	154.21	3.289	4.094	3.816
28	77 SEQUATCHIE					67.19	3.492	3.666	3.579
28	88 VAN BUREN					78.34	3.289	4.267	3.929
28	89 WARREN					165.95	3.341	4.109	3.834
Average			4.080	4.614	4.443		3.405	4.103	3.858
29	6 BRADLEY	38.62	4.097	4.447	4.337	110.6	3.397	4.350	4.028
29	33 HAMILTON	65.2	3.547	4.555	4.215	223.78	3.146	4.204	3.836
29	54 MCMINN	50	4.091	4.448	4.334	135.41	3.377	4.078	3.830
29	61 MEIGS					78.4	3.398	4.091	3.860
29	70 POLK					108.22	3.250	4.472	4.054
29	72 RHEA					96.81	3.297	3.986	3.732
Average			3.862	4.480	4.295		3.285	4.196	3.894

With the calculated average PQI values, the pavement condition levels of Region 1 and 2 were calculated and summarized in Table 15 to Table 16 and Figure 16 to Figure 17.

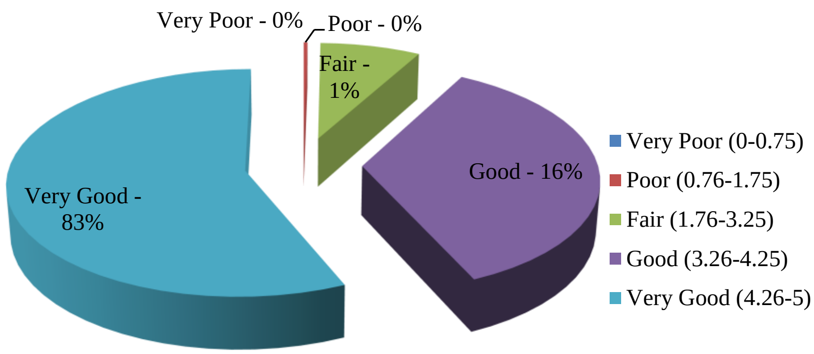
Table 15 Pavement PQI levels in Region 1

PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
Very poor (0-0.75)	2.22	0%	0	0%
Poor (0.76-1.75)	42.05	1%	2	0%
Fair (1.76-3.25)	596.01	20%	53.62	8%
Good (3.26-4.25)	1626.651	55%	243.93	35%
Very good (4.26-5)	671.84	23%	390.61	57%
Total mileage	2938.771		690.16	

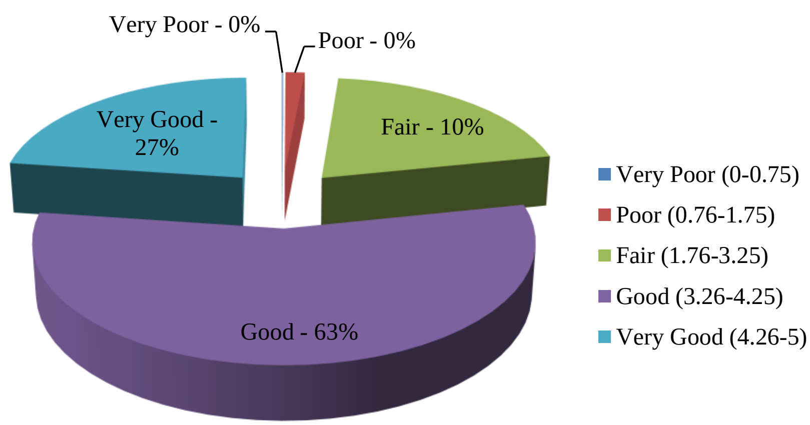
Table 16 Pavement PQI levels in Region 2

PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
Very poor (0-0.75)	0.15	0%	0	0%
Poor (0.76-1.75)	28.4	1%	0	0%
Fair (1.76-3.25)	360.5	13%	9.15	2%

Good (3.26-4.25)	1464.28	53%	123.82	28%
Very good (4.26-5)	893.61	33%	306.41	70%
Total mileage	2746.94		439.38	

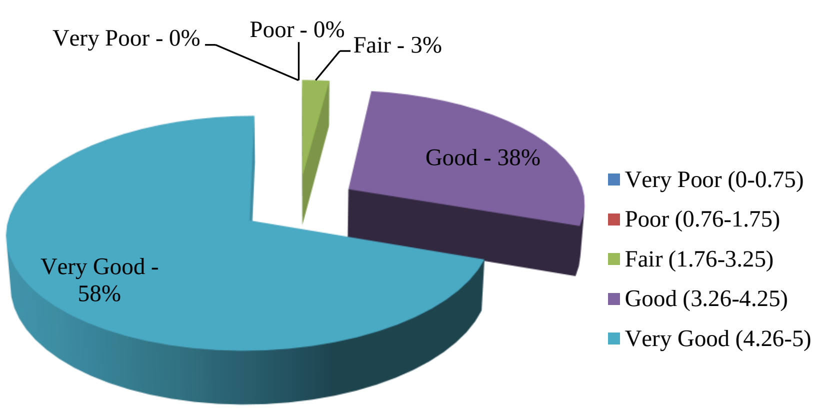


(a) Interstates

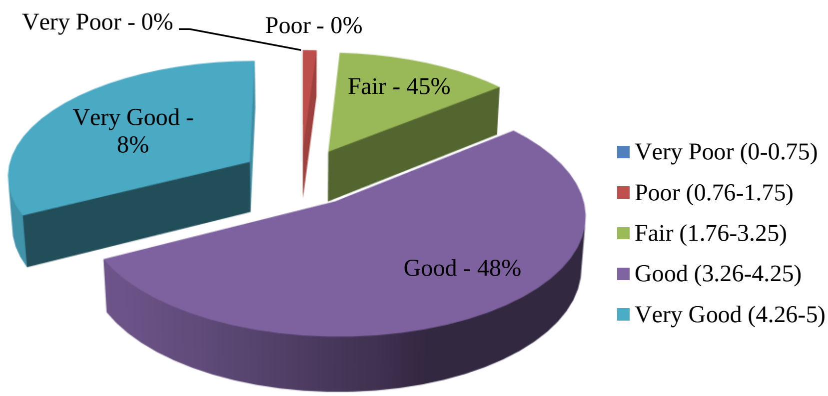


(b) State Routes

Figure 16 Summary of PQI values in Region 1



(a) Interstates



(b) State Routes

Figure 17 Summary of PQI values in Region 2

5.2 MAP 21 Pavement Condition Report

The Moving Ahead for Progress in the 21st Century Act (MAP-21) is a funding and authorization bill to govern United States federal surface transportation spending. This new law intends to help the Federal Motor Carrier Safety Administration (FMCSA) to reduce crashes, injuries and fatalities involving large trucks and buses.

In accordance with section 1106 of the MAP-21, national performance management measures and standards were developed by the FHWA to help the States to meet the national transportation goals identified in section 1203 of MAP-21. Section 1203 of the MAP-21 stipulates USDOT to promulgate performance measures in the areas of the National Highway Performance Program (NHPP), Highway Safety Improvement Program (HSIP), the Congestion Mitigation and Air Quality Improvement Program (CMAQ), and the National Freight Movement (Freight) within 18 months after the date of enactment of the MAP-21. The established performance measures aim to help State DOTs to carry out the National Highway Performance Program (NHPP) and to assess: condition of pavements on the National Highways System (NHS) (excluding the Interstate System), condition of pavements on the Interstate System, and condition of bridges on the NHS.

In § 490.313, FHWA proposes the method for calculating the pavement measures using the pavement metrics and data elements. In § 490.313(a), FHWA proposes how the pavement measures would be used by FHWA, State DOTs, and MPOs. In § 490.313(b), FHWA proposes the method to calculate condition ratings that would use a Good, Fair, and Poor rating approach for each of the four pavement metrics discussed in § 490.311. The proposed thresholds are based on documented research. As an example, the proposed pavement rutting thresholds have been correlated to threshold levels that minimize the risk of vehicle hydroplaning. These proposed criteria are also based on the levels used by FHWA to report ride quality conditions for the IRI metric and the default design criteria thresholds established for the Mechanistic Empirical Pavement Design Guide (MEPDG). Table 17 summarizes proposed criteria to determine Good, Fair, and Poor ratings.

Table 17 Proposed Pavement Condition Rating Thresholds

Surface type	Metric	Metric range	Rating
All pavements	IRI (in./mile)	<95	Good.
		95-170: Areas with a population less than 1million	Fair.
		95-220: Urbanized areas with a population of at least 1million	
		>170: Areas with a population less than 1million	Poor.
		>220: Urbanized areas with a population of at least 1million	
Asphalt Pavement and Jointed Concrete Pavement	Crack (%slabs)	<5% 5-10% >10%	Good. Fair. Poor.
Asphalt Pavement	Rutting (in.)	<0.2 0.2-0.4 >0.4	Good. Fair. Poor.
Jointed Concrete Pavement	Faulting (in.)	<0.05 0.05-0.15 >0.15	Good. Fair. Poor.
CRCP	Slab (%)	<5% 5-10% >10%	Good. Fair. Poor.

According to the MAP-21 requirement, asphalt pavement performance indices include roughness, rutting and percent of fatigue cracking. Concrete pavement performance indices include roughness, faulting and percent of number of slabs with cracking. Each index can be rated as Good, Fair or Poor based on the criteria in Table 17. A 0.1 mile pavement segment is rated as Good when all three indices are rated as Good, or Poor when more than two of the indices are rated as Poor. Otherwise, it is all rated as Fair. Firstly, we assigned different values for different ratings, 0, 2 and 3 as Good, Fair and Poor. Then, if the sum of the three ratings is less than 3, it is rated as Poor. If the sum of the three ratings is equal to 9, it is rated as good. Any values between 3 and 9 are rated as Fair. The performance rating of Tennessee highways of the last 4 years are shown in Table 18 and Figure 18. Table 19 and Figure 19 summarize the performance rating for different performance indices.

For concrete pavement, MAP-21 rate the pavement condition based on percentage of slabs with cracking and the average faulting height. To get an approximate estimation of the two indices, the number of transverse cracking was utilized to estimate the number of slabs with cracking, which was then used to calculate the percentage of cracking slabs. Table 8 to 9 and Figure 5 to 6 summarize the performance rating for different highway types.

Table 18 Summary of the Performance Rating of last four years

Year	Category	Roads	Bridge	Construction	Railway	% without construction	% Construction as poor

2014	Good	94087	1911	392	4	65.3%	65.1%
	Fair	48906	4365	106	254	34.1%	34.0%
	Poor	722	83	2	14	0.5%	0.9%
	Total	143715	6359	500	272	100%	100%
2013	Good	89578	1914	716	5	62.6%	62.2%
	Fair	52611	4412	135	260	36.9%	36.7%
	Poor	729	70	5	9	0.5%	1.1%
	Total	142918	6396	856	274	100%	100%
2012	Good	93750	1912	305	6	65.2%	65.0%
	Fair	49312	4052	84	248	34.5%	34.4%
	Poor	425	42	2	8	0.3%	0.6%
	Total	143487	6006	391	262	100%	100%
2011	Good	92208	1560	243	7	63.9%	63.8%
	Fair	51302	4289	83	268	35.8%	35.7%
	Poor	434	42		8	0.3%	0.5%
	Total	143944	5891	326	283	100%	100%

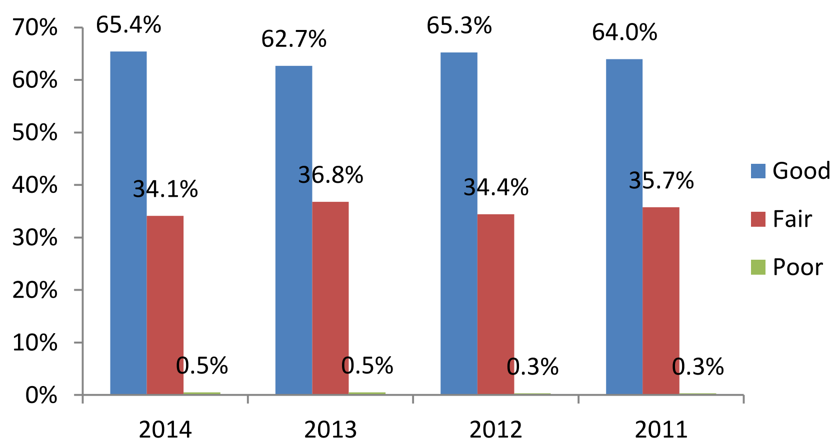
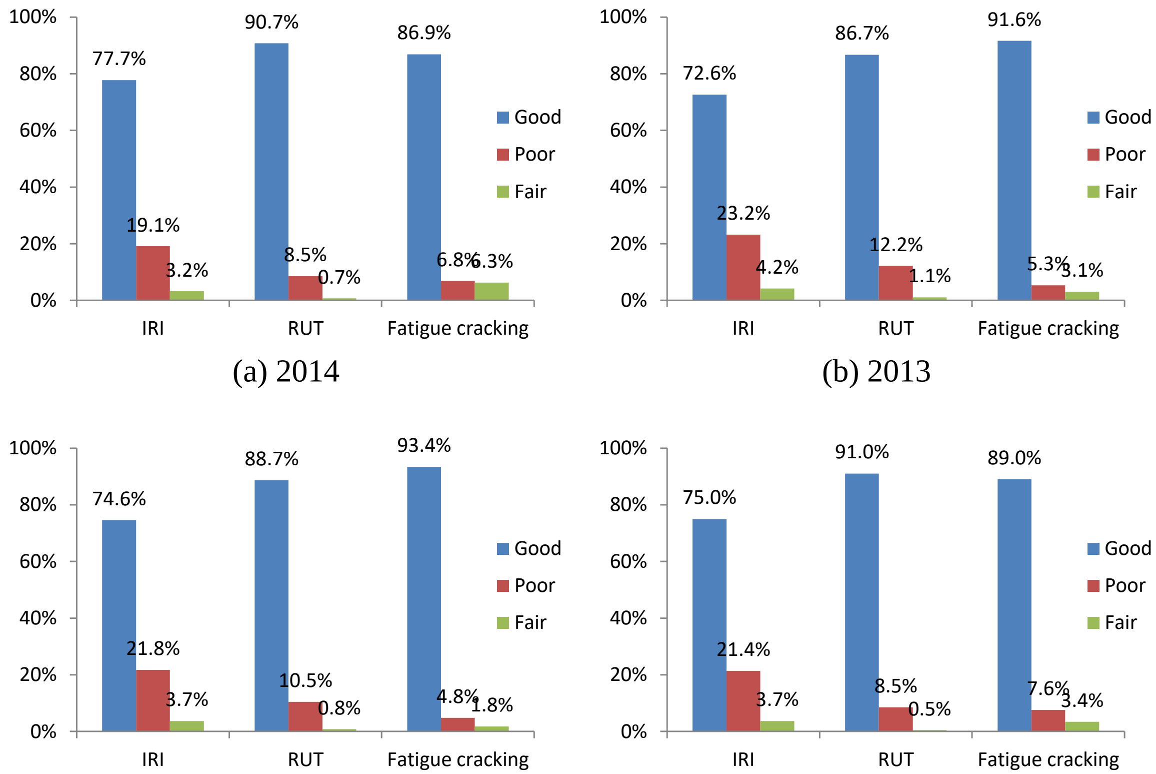


Figure 18 Pavement performance rating for last four years

Table 19 Pavement performance rating for each index

Year	Index	IRI	RUT	Fatigue cracking
2014	Good	77.7%	90.7%	86.9%
	Poor	19.1%	8.5%	6.8%
	Fair	3.2%	0.7%	6.3%
2013	Good	72.6%	86.7%	91.6%
	Poor	23.2%	12.2%	5.3%
	Fair	4.2%	1.1%	3.1%
2012	Good	74.6%	88.7%	93.4%
	Poor	21.8%	10.5%	4.8%
	Fair	3.7%	0.8%	1.8%
2011	Good	75.0%	91.0%	89.0%
	Poor	21.4%	8.5%	7.6%
	Fair	3.7%	0.5%	3.4%



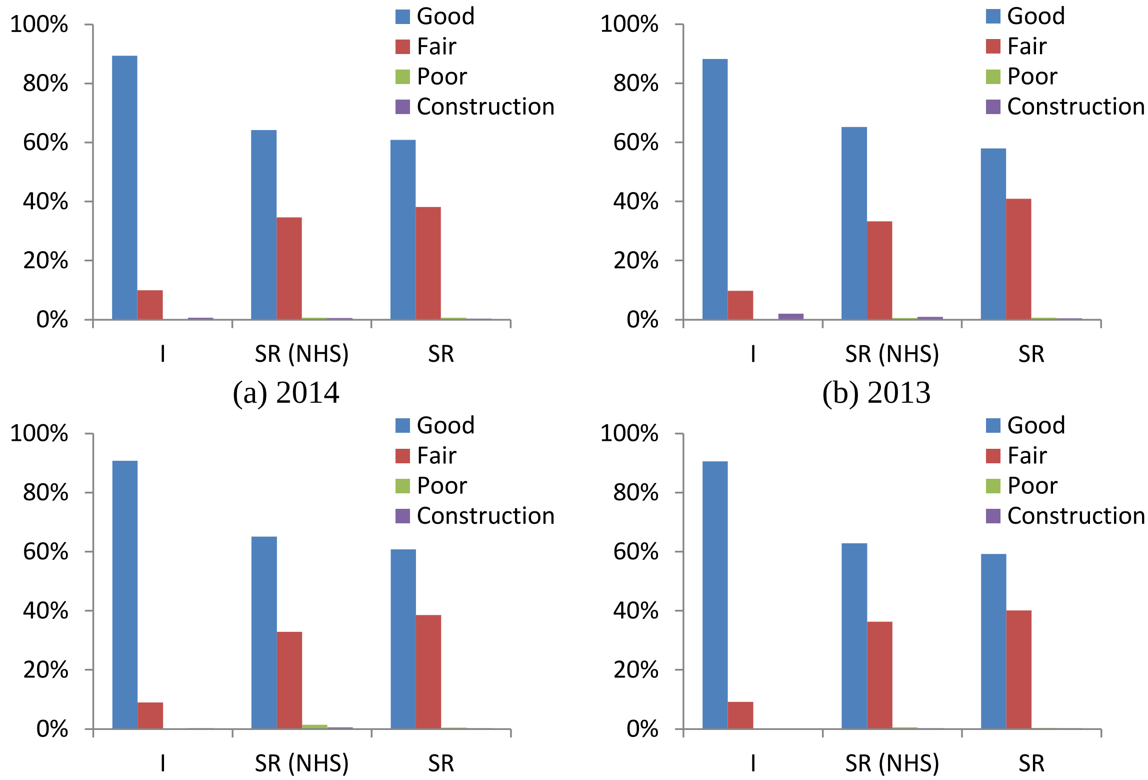
(a) 2014 (b) 2013

(c) 2012 (d) 2011

Figure 19 Performance rating for each performance index of the last 4 years

Table 20 Performance rating for different highway types (asphalt pavement)

Year	Category	I	SR (NHS)	SR	I	SR (NHS)	SR
2014	Good	4442.6	6339.5	18867.4	89.4%	64.2%	60.9%
	Fair	492.2	3415.2	11823.6	9.9%	34.6%	38.2%
	Poor	1.9	63.9	192.8	0.0%	0.6%	0.6%
	Construction	32.0	54.0	107.3	0.6%	0.5%	0.3%
2013	Good	4365.6	6413.7	17879.4	88.2%	65.3%	57.9%
	Fair	482.2	3270.0	12628.5	9.7%	33.3%	40.9%
	Poor	0.0	52.0	203.7	0.0%	0.5%	0.7%
	Construction	99.3	92.7	146.3	2.0%	0.9%	0.5%
2012	Good	4553.9	6434.8	18721.4	90.8%	65.1%	60.8%
	Fair	450.0	3250.7	11869.6	9.0%	32.9%	38.5%
	Poor	0.2	141.3	125.5	0.0%	1.4%	0.4%
	Construction	13.0	57.1	87.2	0.3%	0.6%	0.3%
2011	Good	4556.5	6181.4	18348.0	90.6%	62.9%	59.3%
	Fair	461.9	3570.7	12408.4	9.2%	36.3%	40.1%
	Poor	0.6	55.8	119.6	0.0%	0.6%	0.4%
	Construction	9.5	24.3	78.6	0.2%	0.2%	0.3%



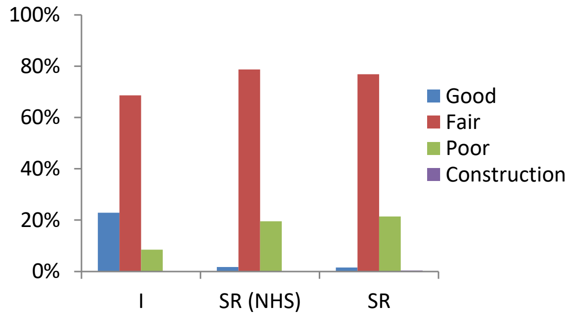
(a) 2014 (b) 2013

(c) 2012 (d) 2011

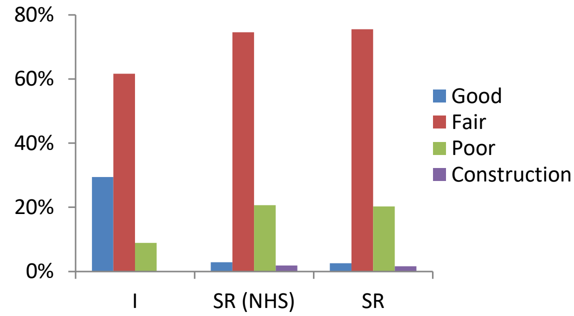
Figure 20 Performance rating for different highway types (asphalt pavement)

Table 21 Performance rating for different highway types (concrete pavement)

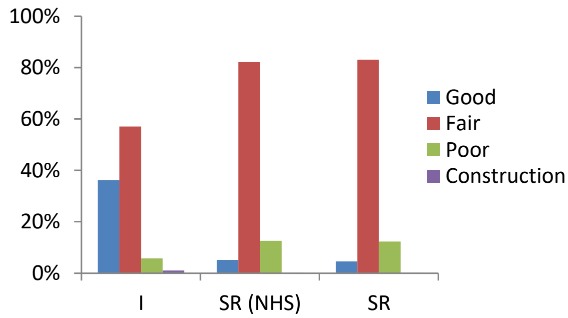
Year	Category	I	SR (NHS)	SR	I	SR (NHS)	SR
2014	Good	57.0	1.1	1.1	22.9%	1.7%	1.5%
	Fair	170.9	50.8	55.2	68.6%	78.8%	76.8%
	Poor	21.2	12.6	15.4	8.5%	19.5%	21.4%
	Construction			0.2	0.0%	0.0%	0.3%
2013	Good	73.7	1.9	1.9	29.4%	2.9%	2.5%
	Fair	154.5	48.9	56.4	61.7%	74.6%	75.6%
	Poor	22.4	13.5	15.2	8.9%	20.7%	20.3%
	Construction		1.2	1.2	0.0%	1.8%	1.6%
2012	Good	88.0	4.0	4.0	36.2%	5.1%	4.5%
	Fair	138.4	64.3	73.3	57.0%	82.2%	83.0%
	Poor	13.9	9.9	10.9	5.7%	12.6%	12.3%
	Construction	2.4	0.1	0.1	1.0%	0.1%	0.1%
2011	Good	92.2	2.2	2.2	38.0%	2.9%	2.6%
	Fair	139.1	60.8	67.0	57.4%	80.1%	78.7%
	Poor	11.0	12.9	15.9	4.5%	17.0%	18.7%
	Construction				0.0%	0.0%	0.0%



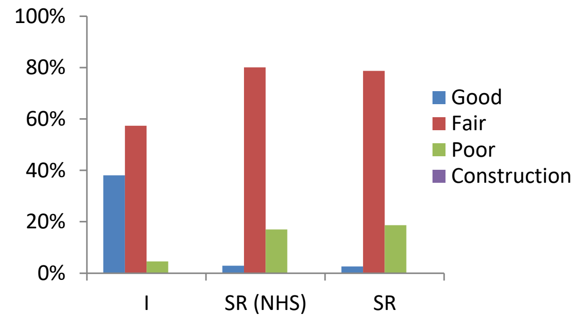
(a) 2014



(b) 2013



(c) 2012



(d) 2011

Figure 21 Performance rating for different highway types (concrete pavement)

6 2015 ANNUAL REPORT OF PAVEMENT CONDITION

6.1 Pavement Condition Data collection

Based on the updated pavement condition data, the pavement condition were exported from the PMS first and then the average condition indices in Region 3 and 4 were calculated as shown in Table 13 and Table 14. Those average PSI, PDI and PQI values were the weighted average values calculated by the equation below.

$$Avg.Pavement\ condition = \frac{\sum Length * Pavement\ condition\ index}{\sum Length} \tag{10}$$

Table 22 Summary of the 2015 pavement condition of Region 3

2015 Region 3 by District		Interstate			State Route		
District	**County**	**Average PSI**	**Average PDI**	**Average PQI**	**Average PSI**	**Average PDI**	**Average PQI**
37	11 Cheatham	3.483	4.364	4.065	3.006	4.013	3.665
37	19 Davidson				3.157	3.912	3.656
37	56 Macon	4.067	4.720	4.512	3.166	4.472	4.023
37	80 Smith	3.695	4.259	4.078	3.261	3.479	3.388
37	83 Summer				3.306	4.135	3.856
37	85 Trousdale	3.806	4.019	3.938	3.182	4.069	3.762
37	94 Williamson	3.959	4.768	4.507	3.098	4.145	3.786
37	95 Wilson	3.679	4.413	4.167	3.143	3.978	3.688
	Average	**3.926**	**4.389**	**4.241**	**3.448**	**4.146**	**3.914**
38	22 Dickson	4.144	4.821	4.605	3.634	4.196	4.006
38	41 Hickman	4.051	4.784	4.550	3.540	4.258	4.020
38	42 Houston				3.457	4.269	3.993
38	43 Humphreys	3.816	4.911	4.550	3.706	4.295	4.102
38	60 Maury	3.906	4.698	4.443	3.347	4.237	3.940
38	63 Montgomery	4.118	4.877	4.635	3.395	4.148	3.893
38	74 Robertson	3.840	4.035	3.970	3.443	3.722	3.622
38	81 Stewart				3.304	4.268	3.938
	Average	**3.965**	**4.593**	**4.390**	**3.463**	**4.156**	**3.922**
39	2 Bedford	3.785	4.615	4.350	3.332	4.190	3.901
39	28 Giles	3.854	4.782	4.480	3.289	3.842	3.655
39	50 Lawrence				3.359	3.901	3.712
39	51 Lewis				3.466	4.136	3.908
39	52 Lincoln				3.352	4.182	3.901
39	59 Marshall	4.174	4.874	4.653	3.296	4.358	4.000
39	64 Moore				3.284	4.478	4.075
39	68 Perry				3.177	3.602	3.457
39	75 Rutherford	3.823	4.519	4.295	3.260	4.069	3.792

39	91 Wayne				3.126	3.736	3.515
	Average	**3.898**	**4.671**	**4.422**	**3.290**	**4.032**	**3.778**

Table 23 Summary of the 2015 pavement condition of Region 4

2015 Region 4 by District		Interstate			State Route		
District	**County**	**Average PSI**	**Average PDI**	**Average PQI**	**Average PSI**	**Average PDI**	**Average PQI**
47	3 BENTON	3.629	4.368	4.123	3.089	3.038	2.998
47	9 CARROLL	4.120	4.210	4.180	2.785	3.226	3.057
47	23 DYER	3.924	4.074	4.016	2.798	2.652	2.647
47	27 GIBSON				2.850	3.116	3.005
47	40 HENRY				3.030	2.865	2.868
47	48 LAKE				2.395	2.855	2.697
47	66 OBION				2.728	2.879	2.805
47	92 WEAKLEY				2.768	2.886	2.811
	Average	**3.827**	**4.179**	**4.057**	**2.827**	**2.963**	**2.884**
48	12 CHESTER				3.011	3.177	3.088
48	17 CROCKETT				2.960	3.232	3.134
48	20 DECATUR	4.080	4.053	4.057	3.159	3.726	3.512
48	35 HARDEMAN				3.064	3.281	3.183
48	36 HARDIN				3.046	2.792	2.836
48	38 HAYWOOD	3.790	3.994	3.918	2.748	3.375	3.155
48	39 HENDERSON	3.780	4.225	4.075	2.811	2.769	2.716
48	55 MCNAIRY				2.886	2.931	2.874
48	57 MADISON	3.762	3.520	3.557	2.968	3.492	3.301
	Average	**3.797**	**3.906**	**3.852**	**2.951**	**3.166**	**3.063**
49	24 FAYETTE	3.812	3.642	3.675	3.075	3.577	3.393
49	49 LAUDERDALE				2.699	2.801	2.723
49	79 SHELBY	3.465	4.170	3.927	2.593	3.390	3.098
49	84 TIPTON				2.809	3.022	2.924
	Average	**3.536**	**4.061**	**3.875**	**2.769**	**3.273**	**3.078**

With the calculated average PQI values, the pavement condition levels of Region 3 and 4 were calculated and summarized in Table 15 to Table 16 and Figure 16 to Figure 17.

Table 24 Pavement PQI levels in Region 3

PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
Very poor (0-0.75)	7.92	0%	0.91	0%
Poor (0.76-1.75)	46.21	1%	6.57	1%
Fair (1.76-3.25)	639.996	17%	31.32	4%
Good (3.26-4.25)	2179.44	59%	235.93	33%

Very good (4.26-5)	851.96	23%	433	61%
Total mileage	3725.526		707.73	

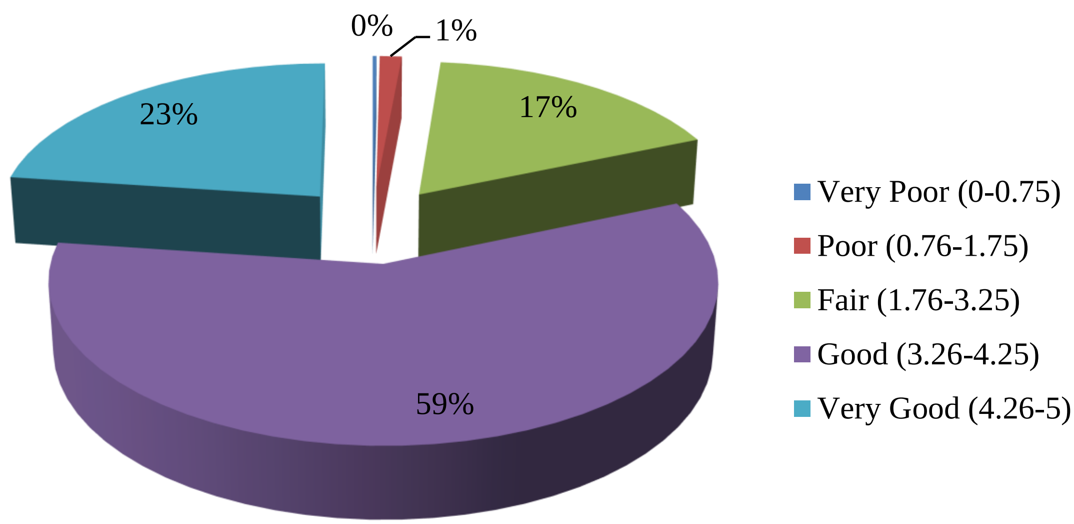
PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
0-0.5	3.465	0%	0.22	0%
0.51-1	11.355	0%	2.28	0%
1.01-1.5	19.93	1%	1.84	0%
1.51-2	40.79	1%	4.43	1%
2.01-2.5	88.466	2%	3.37	0%
2.51-3	260.76	7%	14.31	2%
3.01-3.5	705.03	19%	21.35	3%
3.51-4	1131.06	30%	112.93	16%
4.01-4.5	1124.67	30%	287	41%
4.51-5	340	9%	260	37%
	3725.526		707.73	

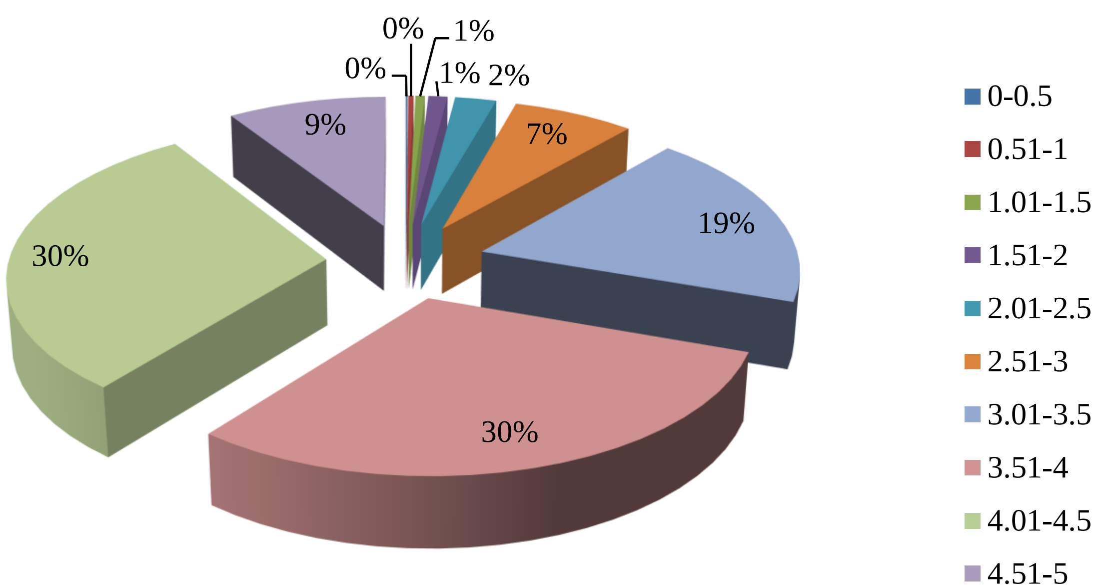
Table 25 Pavement PQI levels in Region 4

PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
Very poor (0-0.75)	15.39	0%	0.2	0%
Poor (0.76-1.75)	292.19	9%	2.42	1%
Fair (1.76-3.25)	1831.62	54%	73.8	20%
Good (3.26-4.25)	1005.94	30%	168.44	45%
Very good (4.26-5)	226.97	7%	127	34%
Total mileage	3372.11		371.86	

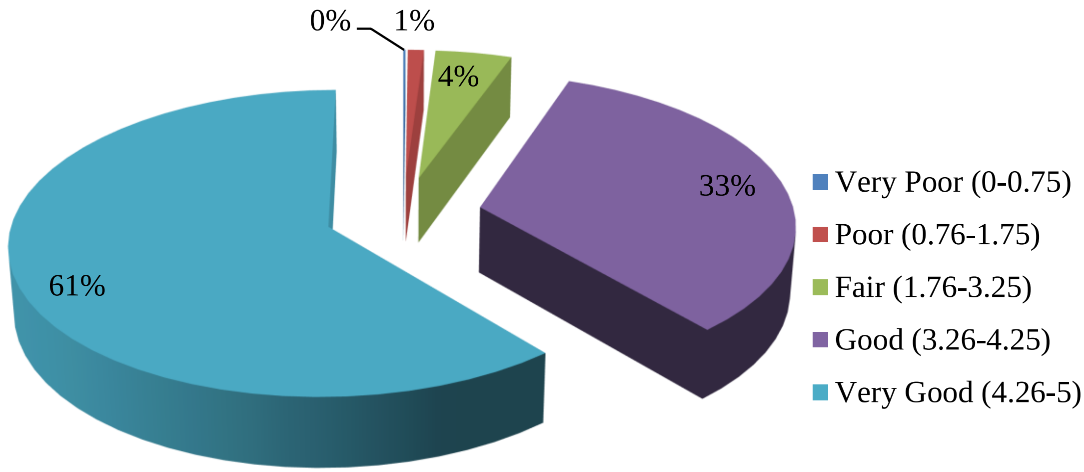
PQI condition levels	State Route		Interstate	
	Length (mile)	Percentage	Length (mile)	Percentage
0-0.5	6.31	0%	0.2	0%
0.51-1	22.39	1%	0	0%
1.01-1.5	144.46	4%	1.42	0%
1.51-2	323.26	10%	1	0%
2.01-2.5	494.01	15%	14.94	4%
2.51-3	728.37	22%	24.14	6%
3.01-3.5	782.93	23%	61.44	17%
3.51-4	461.52	14%	85.94	23%
4.01-4.5	313.86	9%	118.78	32%
4.51-5	95	3%	64	17%

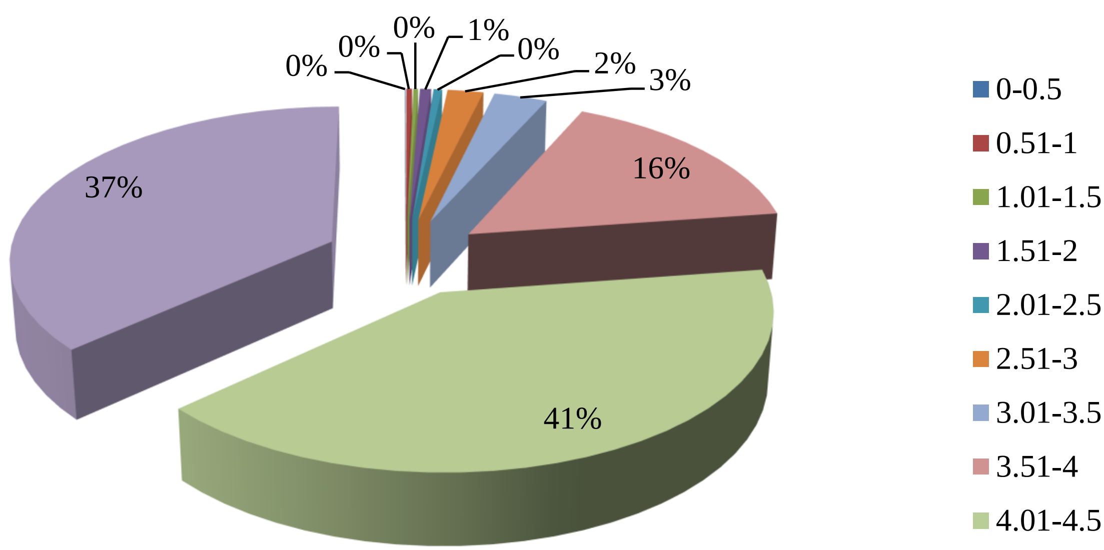
	3372.11		371.86	





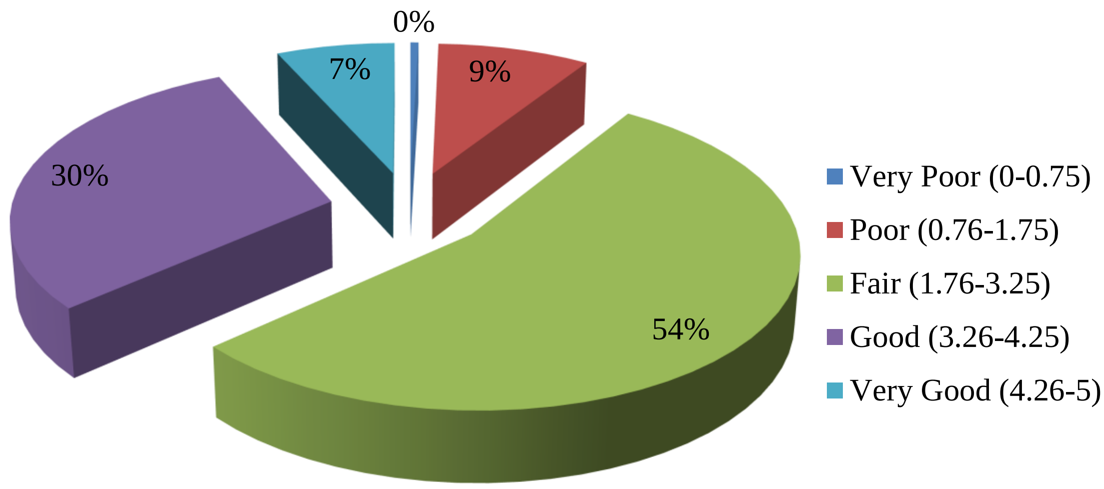
(a) State Routes

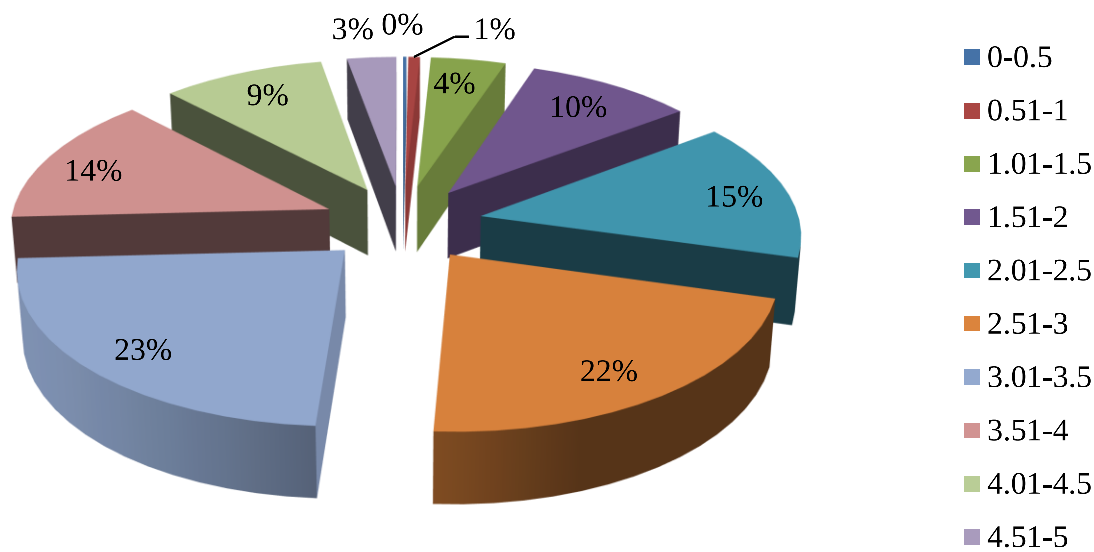




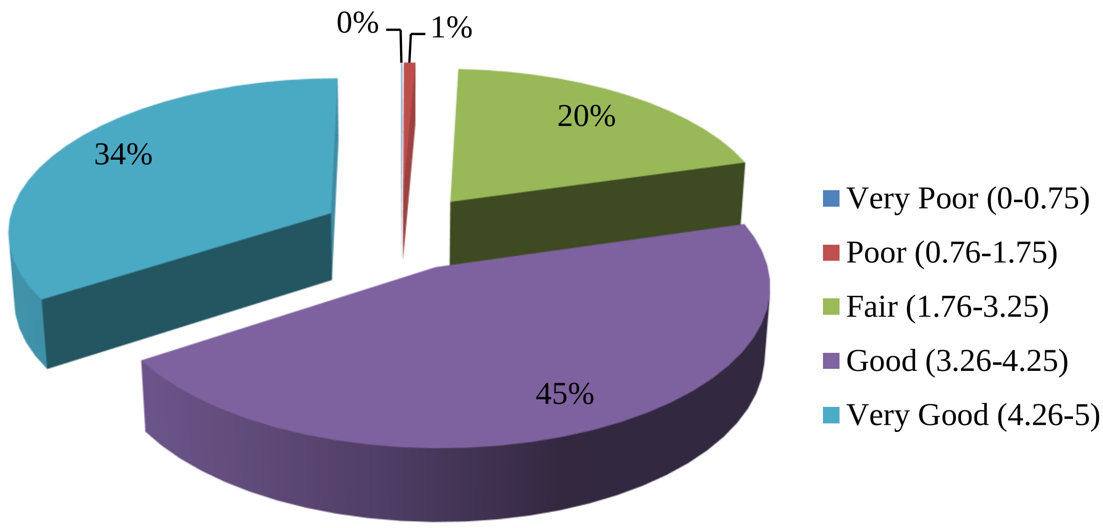
(b) Interstates

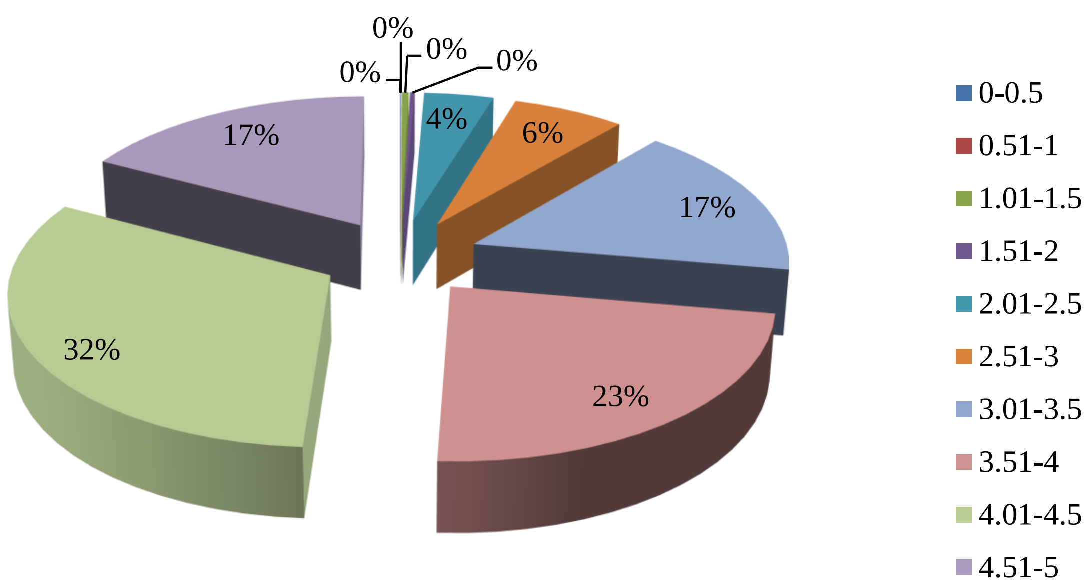
Figure 22 Summary of PQI values in Region 3





(a) State Routes





(b) Interstates

Figure 23 Summary of PQI values in Region 4

6.2 MAP 21 Pavement Condition Report

The Moving Ahead for Progress in the 21st Century Act (MAP-21) is a funding and authorization bill to govern United States federal surface transportation spending. This new law intends to help the Federal Motor Carrier Safety Administration (FMCSA) to reduce crashes, injuries and fatalities involving large trucks and buses.

In accordance with section 1106 of the MAP-21, national performance management measures and standards were developed by the FHWA to help the States to meet the national transportation goals identified in section 1203 of MAP-21. Section 1203 of the MAP-21 stipulates USDOT to promulgate performance measures in the areas of the National Highway Performance Program (NHPP), Highway Safety Improvement Program (HSIP), the Congestion Mitigation and Air Quality Improvement Program (CMAQ), and the National Freight Movement (Freight) within 18 months after the date of enactment of the MAP-21. The established performance measures aim to help State DOTs to carry out the National Highway Performance Program (NHPP) and to assess: condition of pavements on the National Highways System (NHS) (excluding the Interstate System), condition of pavements on the Interstate System, and condition of bridges on the NHS.

In § 490.313, FHWA proposes the method for calculating the pavement measures using the pavement metrics and data elements. In § 490.313(a), FHWA proposes how the pavement measures would be used by FHWA, State DOTs, and MPOs. In § 490.313(b), FHWA proposes the method to calculate condition ratings that would use a Good, Fair, and Poor rating approach for each of the four pavement metrics discussed in § 490.311. The proposed thresholds are based on documented research. As an example, the proposed pavement rutting thresholds have been correlated to threshold levels that minimize the risk of vehicle hydroplaning. These proposed criteria are also based on the levels used by FHWA to report ride quality conditions for the IRI metric and the default design criteria thresholds established for the Mechanistic Empirical Pavement Design Guide (MEPDG). Table 17 summarizes proposed criteria to determine Good, Fair, and Poor ratings.

Table 26 Proposed Pavement Condition Rating Thresholds

Surface type	Metric	Metric range	Rating
All pavements	IRI (in./mile)	<95	Good.
		95-170: Areas with a population less than 1million	Fair.
		95-220: Urbanized areas with a population of at least 1million	
		>170: Areas with a population less than 1million	Poor.
		>220: Urbanized areas with a population of at least 1million	
Asphalt Pavement and Jointed Concrete Pavement	Crack (%slabs)	<5% 5-10% >10%	Good. Fair. Poor.
Asphalt Pavement	Rutting (in.)	<0.2 0.2-0.4 >0.4	Good. Fair. Poor.
Jointed Concrete Pavement	Faulting (in.)	<0.05 0.05-0.15 >0.15	Good. Fair. Poor.
CRCP	Slab	<5%	Good.

	(%)	5-10% >10%	Fair. Poor.

According to the MAP-21 requirement, asphalt pavement performance indices include roughness, rutting and percent of fatigue cracking. Concrete pavement performance indices include roughness, faulting and percent of number of slabs with cracking. Each index can be rated as Good, Fair or Poor based on the criteria in Table 17. A 0.1 mile pavement segment is rated as Good when all three indices are rated as Good, or Poor when more than two of the indices are rated as Poor. Otherwise, it is all rated as Fair. Firstly, we assigned different values for different ratings, 0, 2 and 3 as Good, Fair and Poor. Then, if the sum of the three ratings is less than 3, it is rated as Poor. If the sum of the three ratings is equal to 9, it is rated as good. Any values between 3 and 9 are rated as Fair. The performance rating of Tennessee highways of the last 4 years are shown in Table 18 and Figure 18. Table 19 and Figure 19 summarize the performance rating for different performance indices.

For concrete pavement, MAP-21 rates the pavement condition based on percentage of slabs with cracking and the average faulting height. Table 8 to 9 and Figure 5 to 6 summarize the performance rating for different highway types.

Table 27 Summary of the Performance Rating of last four years (centerline miles)

Year	Category	Roads	Bridge	Construction	Railway	% without construction	% Construction as poor
2015 Plus	Good	7754.7	178.5	50.7	0.9	58.7%	58.4%
	Fair	5231.5	424.6	16.4	22.4	39.7%	39.5%
	Poor	206.2	27.4	1.6	2.1	1.6%	2.1%
	Total	13192.4	630.5	68.7	25.4	100.0%	100.0%
2015	Good	8618.4	230.2	64.3	0.9	60.7%	60.3%
	Fair	5360.4	469.4	16.9	22.4	37.9%	37.7%
	Poor	206.7	27.9	1.6	2.1	1.5%	2.0%
	Total	14185.5	727.6	82.8	25.4	100.0%	100.0%
2014	Good	9408.7	191.1	39.2	0.4	65.3%	65.1%
	Fair	4890.6	436.5	10.6	25.4	34.1%	34.0%
	Poor	72.2	8.3	0.2	1.4	0.5%	0.9%
	Total	14371.5	635.9	50	27.2	100%	100%
2013	Good	8957.8	191.4	71.6	0.5	62.6%	62.2%
	Fair	5261.1	441.2	13.5	26	36.9%	36.7%
	Poor	72.9	7	0.5	0.9	0.5%	1.1%
	Total	14291.8	639.6	85.6	27.4	100%	100%
2012	Good	9375	191.2	30.5	0.6	65.2%	65.0%
	Fair	4931.2	405.2	8.4	24.8	34.5%	34.4%
	Poor	42.5	4.2	0.2	0.8	0.3%	0.6%
	Total	14348.7	600.6	39.1	26.2	100%	100%
2011	Good	9220.8	156	24.3	0.7	63.9%	63.8%
	Fair	5130.2	428.9	8.3	26.8	35.8%	35.7%
	Poor	43.4	4.2	0	0.8	0.3%	0.5%
	Total	14394.4	589.1	32.6	28.3	100%	100%

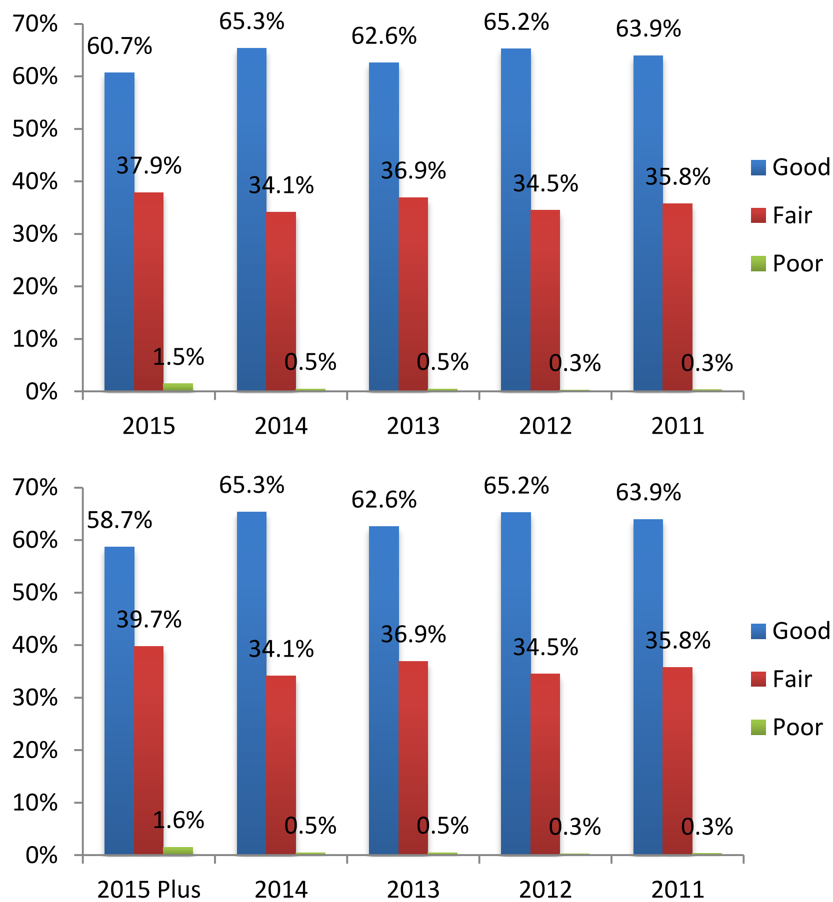


Figure 24 Pavement performance rating (no construction)

Table 28 Pavement performance rating for each index

Year	Index	IRI	RUT	Fatigue cracking
2015 plus	Good	69.5%	93.4%	78.4%
	Fair	25.6%	6.2%	8.4%
	Poor	4.9%	0.4%	13.3%
2015	Good	71.2%	93.7%	79.3%
	Fair	24.2%	6.0%	8.0%
	Poor	4.6%	0.4%	12.7%
2014	Good	77.7%	90.7%	86.9%
	Poor	19.1%	8.5%	6.8%
	Fair	3.2%	0.7%	6.3%
2013	Good	72.6%	86.7%	91.6%
	Poor	23.2%	12.2%	5.3%
	Fair	4.2%	1.1%	3.1%
2012	Good	74.6%	88.7%	93.4%
	Poor	21.8%	10.5%	4.8%
	Fair	3.7%	0.8%	1.8%
2011	Good	75.0%	91.0%	89.0%
	Poor	21.4%	8.5%	7.6%

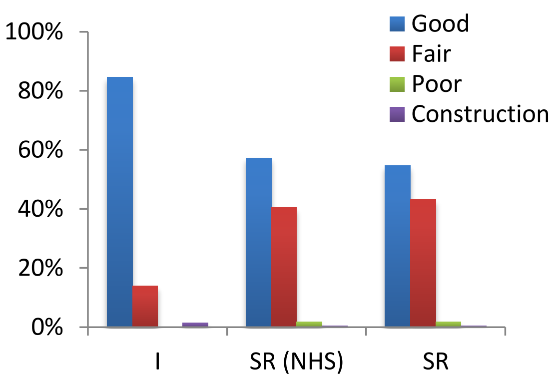
	Fair	3.7%	0.5%	3.4%

Figure 25 Performance rating for each performance index

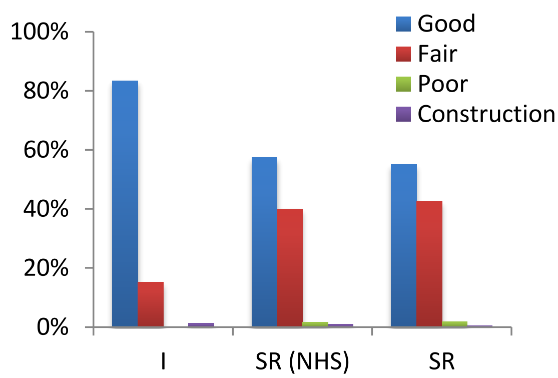
Table 29 Performance rating for different highway types (asphalt pavement)

Year	Category	I	SR (NHS)	SR	I	SR (NHS)	SR
2015	Good	936.0	2026.7	6998.1	84.6%	57.3%	54.6%

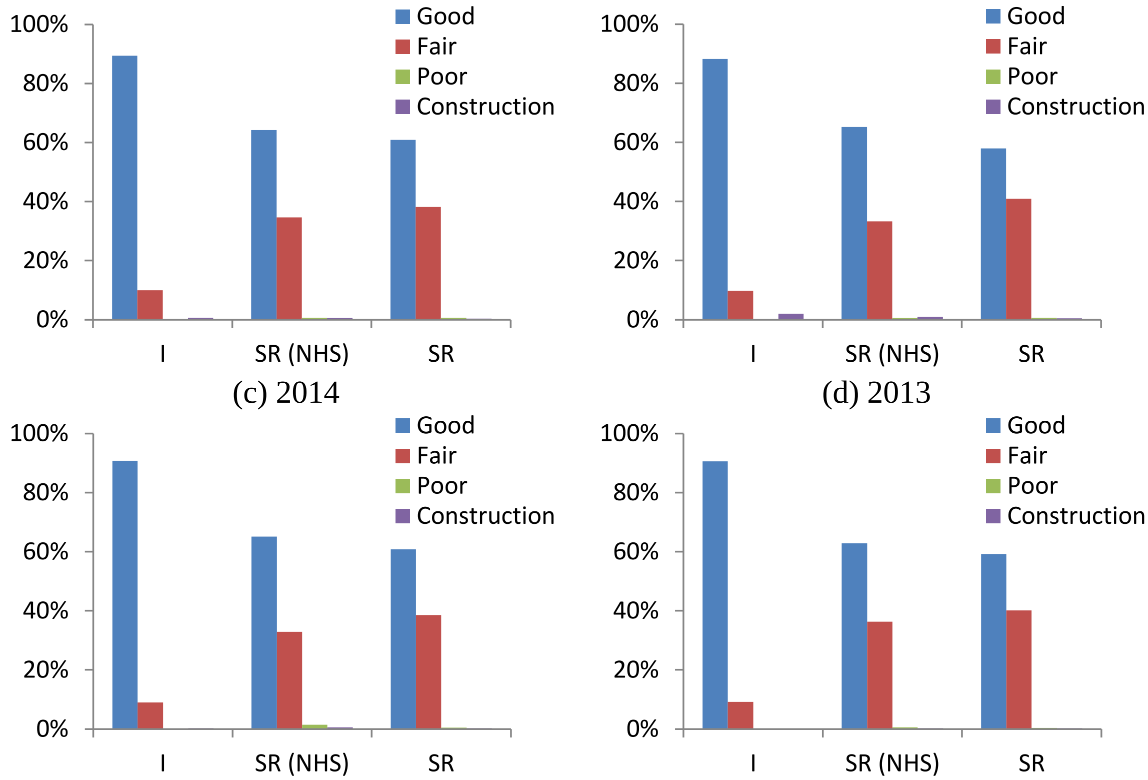
plus	Fair	154.2	1429.2	5524.2	13.9%	40.4%	43.1%
	Poor	0.5	62.9	235.3	0.0%	1.8%	1.8%
	Construction	15.3	17.3	53.4	1.4%	0.5%	0.4%
2015	Good	1851.5	2026.7	6998.1	83.8%	57.0%	54.6%
	Fair	328.0	1429.2	5524.2	14.8%	40.2%	43.1%
	Poor	1.5	62.9	235.3	0.1%	1.8%	1.8%
	Construction	29.4	36.1	53.4	1.3%	1.0%	0.4%
2014	Good	1914.6	2248.1	7649.0	86.8%	63.5%	59.9%
	Fair	279.7	1250.2	5011.2	12.7%	35.3%	39.2%
	Poor	1.5	21.1	77.1	0.1%	0.6%	0.6%
	Construction	11.1	18.7	38.6	0.5%	0.5%	0.3%
2013	Good	1898.3	2281.0	7217.1	85.9%	64.6%	56.7%
	Fair	279.1	1200.8	5386.6	12.6%	34.0%	42.3%
	Poor		16.5	78.3	0.0%	0.5%	0.6%
	Construction	31.8	34.5	53.2	1.4%	1.0%	0.4%
2012	Good	1964.1	2293.3	7577.0	88.5%	65.2%	59.6%
	Fair	249.6	1181.6	5046.6	11.3%	33.6%	39.7%
	Poor	0.2	21.2	45.5	0.0%	0.6%	0.4%
	Construction	4.2	23.2	34.4	0.2%	0.7%	0.3%
2011	Good	1941.9	2184.3	7407.1	87.9%	62.0%	58.1%
	Fair	263.3	1309.0	5255.7	11.9%	37.2%	41.3%
	Poor	0.3	17.7	47.0	0.0%	0.5%	0.4%
	Construction	3.5	9.3	28.8	0.2%	0.3%	0.2%



(a) 2015 plus



(b) 2015



(c) 2014 (d) 2013

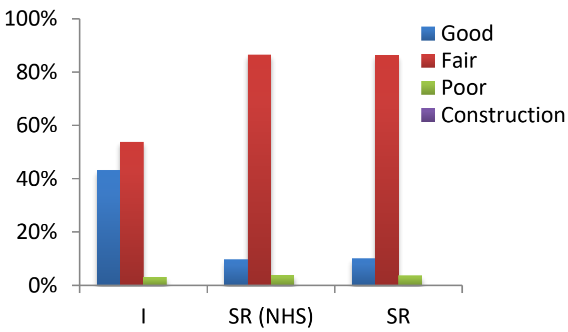
(e) 2012 (f) 2011

Figure 26 Performance rating for different highway types (asphalt pavement)

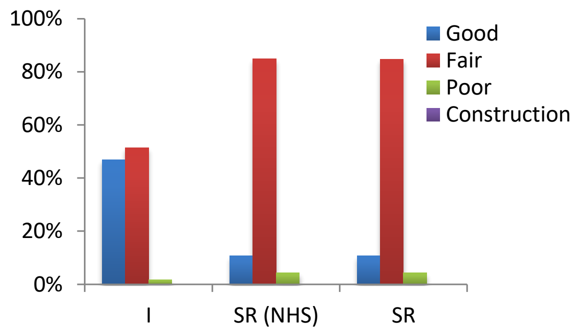
Table 30 Performance rating for different highway types (concrete pavement)

Year	Category	I	SR (NHS)	SR	I	SR (NHS)	SR
2015 plus	Good	17.4	2.1	2.3	43.2%	9.7%	10.1%
	Fair	21.7	18.4	19.9	53.8%	86.5%	86.3%
	Poor	1.2	0.8	0.8	3.0%	3.8%	3.7%
	Construction						
2015	Good	32.8	2.1	2.3	41.9%	9.7%	10.1%
	Fair	43.9	18.4	19.9	56.1%	86.5%	86.3%
	Poor	1.6	0.8	0.8	2.0%	3.8%	3.7%
	Construction						
2014	Good	17.7	0.3	0.3	20.7%	1.7%	1.4%
	Fair	59.2	14.8	16.6	69.5%	76.4%	74.7%
	Poor	8.4	4.2	5.2	9.8%	22.0%	23.4%
	Construction			0.1			0.4%
2013	Good	19.0	0.4	0.4	21.8%	2.2%	1.9%
	Fair	60.2	14.5	17.3	69.2%	75.5%	76.0%
	Poor	7.8	4.3	5.0	9.0%	22.4%	22.1%
	Construction						
2012	Good	26.4	1.2	1.2	31.5%	5.0%	4.3%
	Fair	50.4	19.4	23.0	60.1%	81.3%	82.3%

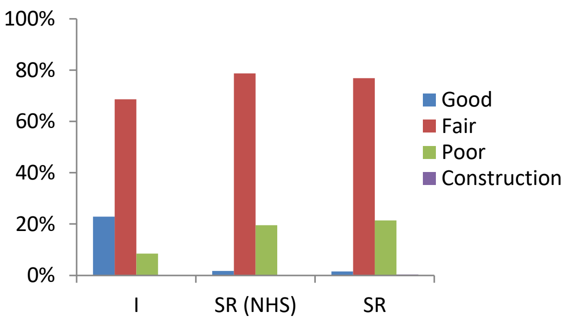
	Poor	5.8	3.2	3.7	6.9%	13.5%	13.2%
	Construction	1.2	0.1	0.1	1.4%	0.2%	0.2%
2011	Good	28.2	0.9	0.9	33.7%	3.8%	3.3%
	Fair	50.9	18.9	21.4	60.9%	80.2%	78.8%
	Poor	4.5	3.8	4.9	5.4%	15.9%	17.9%
	Construction						



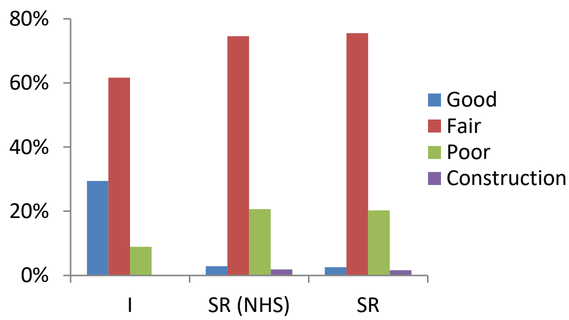
(a) 2015 plus



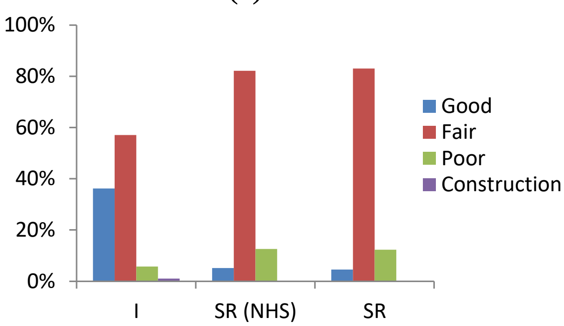
(b) 2015



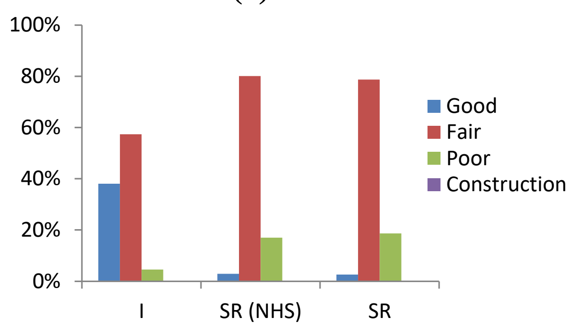
(c) 2014



(d) 2013



(e) 2012



(f) 2011

Figure 27 Performance rating for different highway types (concrete pavement)

7 PAVEMENT MAINTENANCE STRATEGY ANALYSIS

7.1 Updating Pavement Maintenance Records

The team firstly collected all the pavement construction and maintenance records in Tennessee from 1951 to 2015. There are a total of 8061 records, including the county name, route type, route number, special cases, county sequences, begin and end log miles and specific treatment type according to the definition in HPMA. Figure 28 shows the top layers of asphalt pavement used in Tennessee, which are also the typical pavement rehabilitations. Almost all the asphalt pavements in Tennessee use a 1.25 in. thick dense mixed asphalt mixture called “D-mix” as the surface layer. Around 40% of pavement resurfacing repairs in Tennessee are this 1.25 in. thick “D-mix” overlay. For pavements need major repair, especially when milling is involved, a 2 in. thick leveling or base layer called “BM2” is added underneath the “D-mix” surface layer. This 3.25 in. thick combined overlay accounts for around 35% of all the pavement rehabilitations. Further, when additional structural capacity is required for increasing traffic or to compensate the capacity loss due to deep milling, one or multiple 3 in. thick “A” asphalt base layer will be added underneath the “BM2” leveling layer.

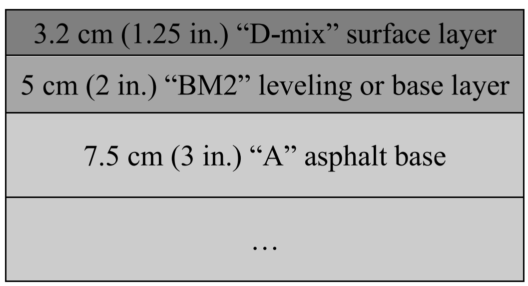
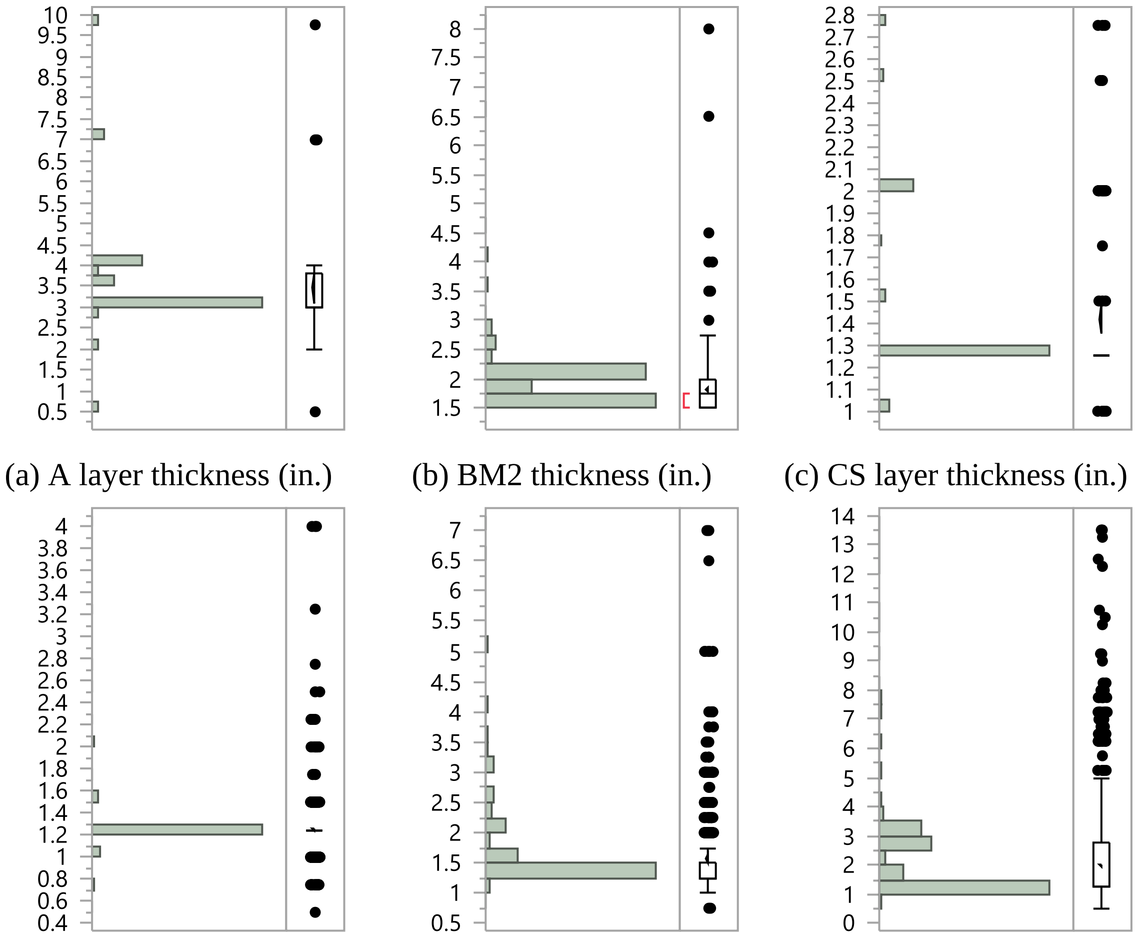


Figure 28 Typical asphalt pavement rehabilitations in Tennessee

The actual thickness of those layers varies based on the materials and other field factors. To determine the treatment methods, the team firstly calculated the total thickness of those maintenance records. Figure 16 shows the thickness of different asphalt layers in maintenance records. A total of 2722 projects include the details of treatment methods. Among them, 626 projects include milling with varying depth. 2495 of them have D mix surface. 122 projects have CS layer. 754 projects have BM2 layer and 50 of them have A layer.



(a) A layer thickness (in.) (b) BM2 thickness (in.) (c) CS layer thickness (in.)

(d) D layer thickness (in.) (e) Milling depth (in.) (f) Overlay thickness (in.)

Figure 29 Summary layer thickness of those maintenance treatments

After calculate the thickness of those treatments, the corresponding activity type was identified according the definition in HPMA. Table 31 summarizes the activity definitions in HPMA. Figure 30 shows the identified maintenance treatment methods.

Table 31 Maintenance activity definitions in HPMA

Code	ID	Activity
1	M1_2	Mill&Replace1"-2"
2	M2_4	Mill&Replace2"-4"
3	MO2200	MR1-2"+OL
4	MO4200	MR2-4"+OL
5	MO2400	MR1-2"+OL
6	MO4400	MR2-4"+OL
7	O200	Overlay<200PSY
8	O400	Overlay200-400PSY
9	O>400	Overlay>400PSY

10	RECON	Reconstruction
11	RO800	RubblizeOL900PSY
12	GR	GeneralRehab
13	GRO400	GR3,4,5,0PSY
14	O400-C	Overlay400PSY(C)
15	OC-BIT	Orig.BITConstr
16	OC-CON	Orig.CONConstr
17	RECON2	Reconstruction
18	CS650	Crack&Seat+OL900+PSY
20	Seal	SurfaceTreatment

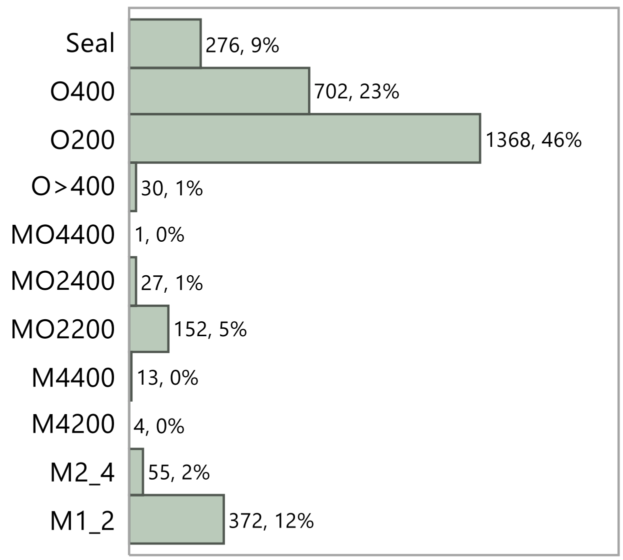


Figure 30 Identified maintenance treatment methods

A total of 2654 new records were added to the HPMA. 303 records were replaced with updated information. With the new updated records, it will be easier to determine pavement segments for maintenance and more accurate maintenance strategy analysis can be conducted.

7.2 Pavement Maintenance Strategy Analysis

The "M&R Optimization Analysis" function in the HPMA was utilized to help maintenance decision making. The process of decision making at a project level in HPMA are shown in Figure 31. The optimal timing decision making for a specific road section can be divided into 2 steps:

1. The decision tree selects some treatment candidates based on current pavement condition and the pre-defined rehabilitation trigger values
2. The historical pavement performance data and defined treatment performance models will be used as do-nothing performance curve and post-treatment performance curve respectively to calculate the effectiveness and cost-

effectiveness. The effectiveness is indicated as PQI areas. Since the cost-effectiveness of different treatment candidates applied at different years are different, the scenario that achieves the highest cost-effectiveness will be selected as the optimized treatment and application time.

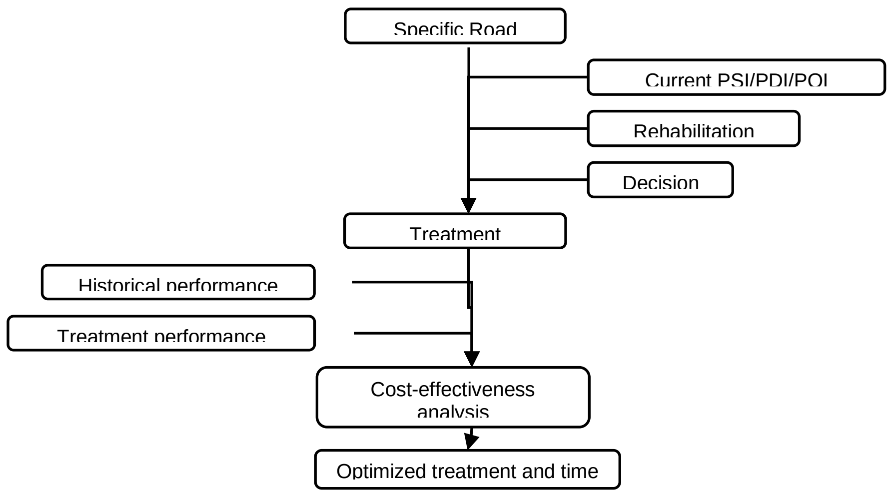


Figure 31 Methodology of project level maintenance decision making in HPMA

Firstly, a section data view (SDV) including all the highways in Tennessee was built, the project segment length was set as 1 mile. The "Break on project limits" is activated to allow section breaks to occur at each project segment location. The, five subsets including Interstates and State routes of the four Regions were defined for the analyses. The "maintenance and rehabilitation strategy analysis" was conducted to get the maintenance strategy at project level. Figure 32 shows the interface of "M&R analysis Set". The analysis period is 10 years. For Interstates, the "Less Severe Interstate Decision Trees" was employed. For State routes, the "State Route with micro" decision tree was employed. After conducting the M&R analysis for each subset, the "Network Optimization" function was utilized to determine the strategy at specific budget constraints.

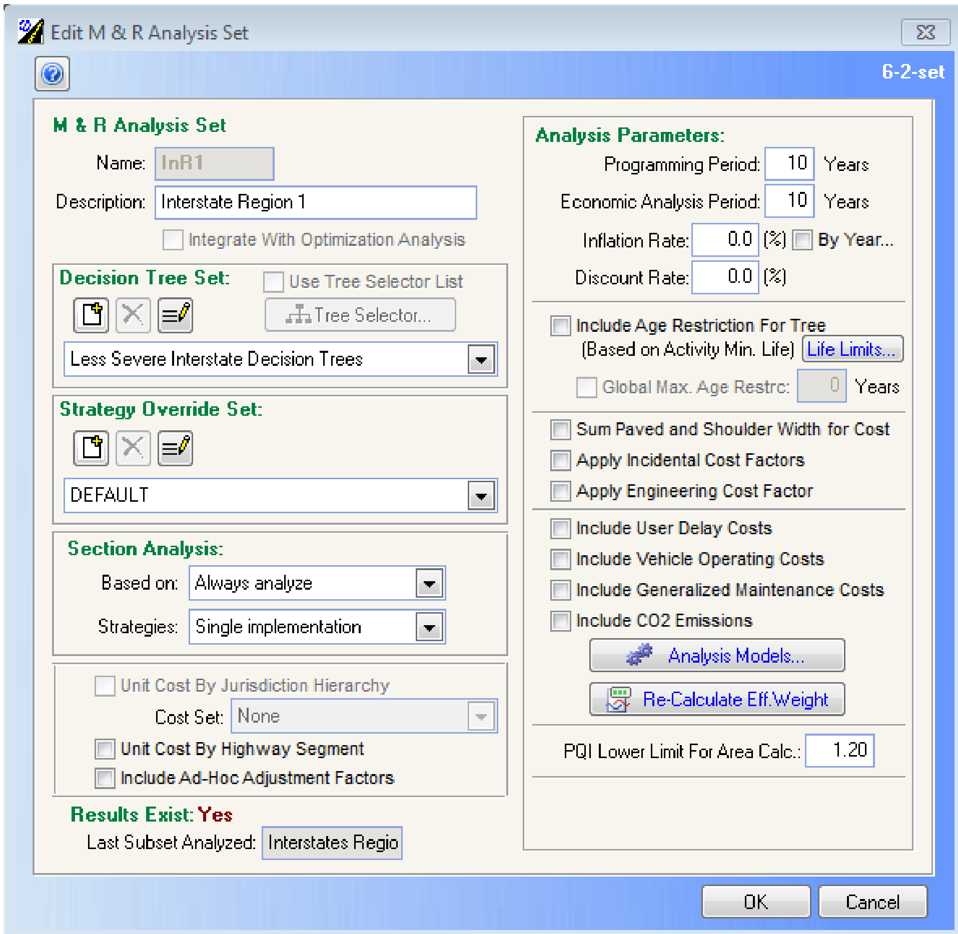
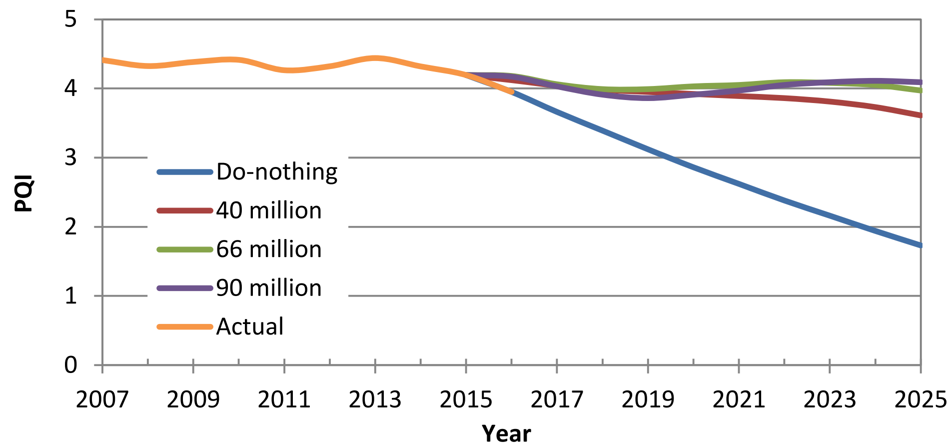


Figure 32 M&R analysis set

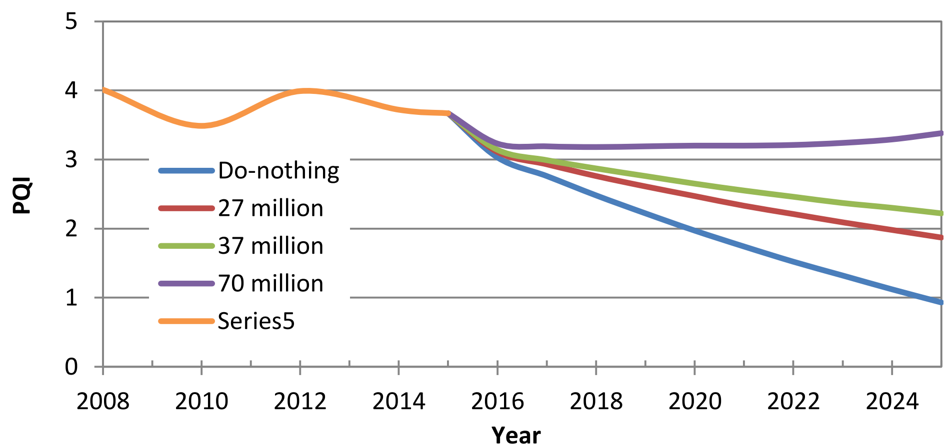
Table 32 shows the budget levels for each subset. Figure 33 shows the predicted average pavement performance PQI at different annual budget levels. It can be seen that to maintain the pavement serviceability at current level, Interstates need 66 $million/year. State routes need 70, 60, 80 and 60 $million/year for Region 1, 2, 3 and 4, respectively. The following table shows the predicted condition range from 2016 to 2025.

Table 32 Budget levels for each subset

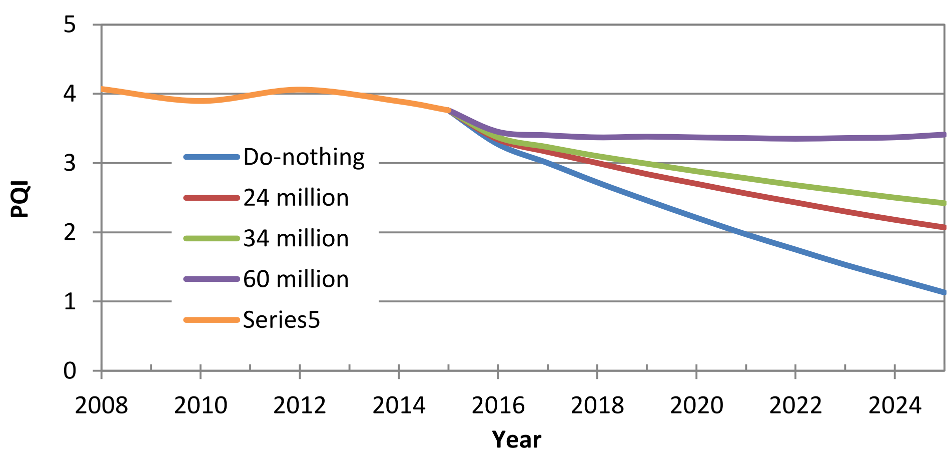
Subsets	Budgets ($million/year)
Region 1	37,476,501
Region 2	33,559,965
Region 3	46,387,601
Region 4	43,575,934
Interstates	6,000,000



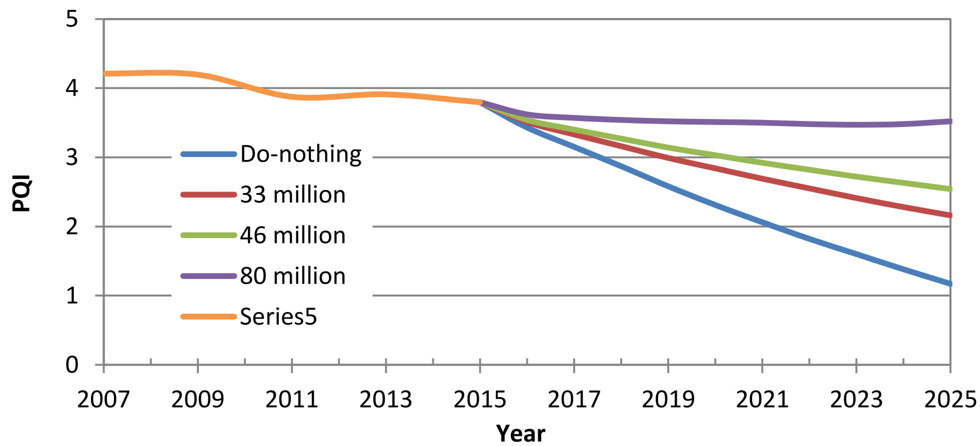
(a) Interstates



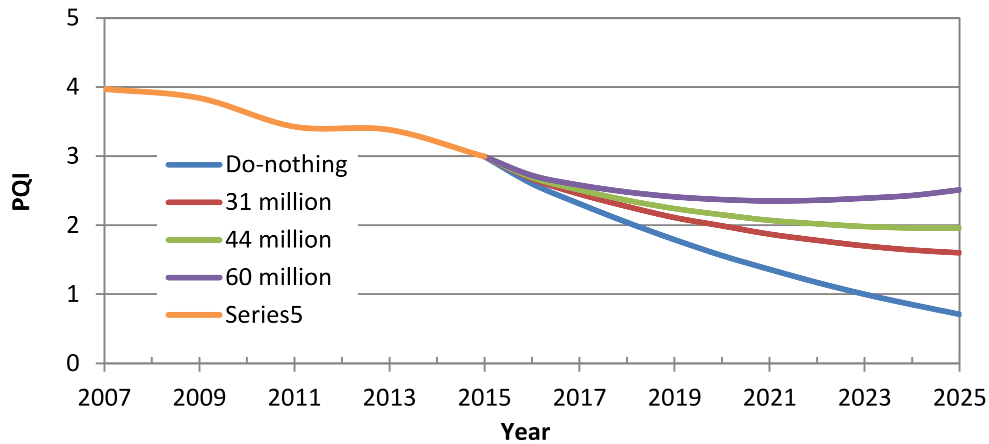
(b) State routes at Region 1



(c) State routes at Region 2

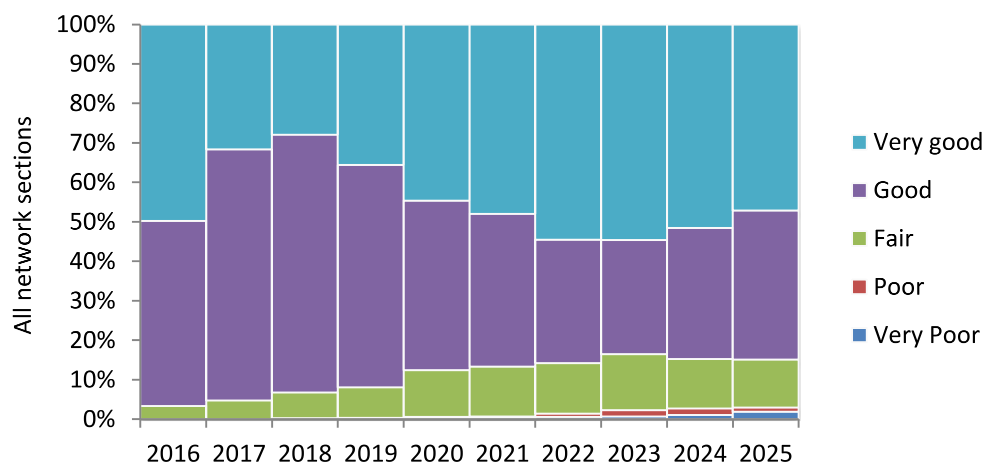


(d) State routes at Region 3

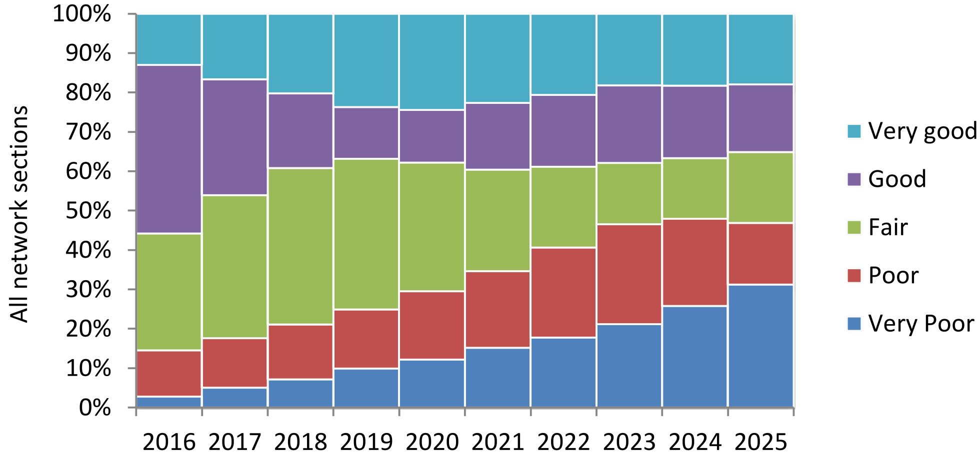


(e) State routes at Region 4

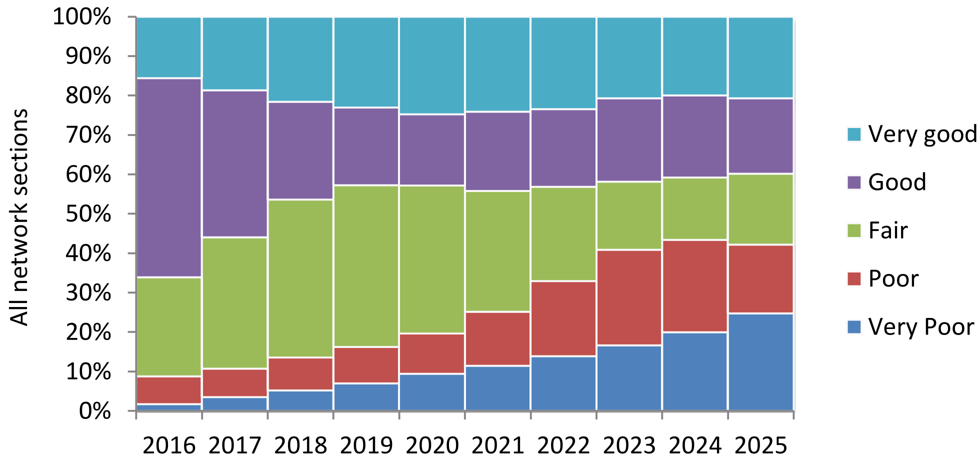
Figure 33 Predicted pavement average performance



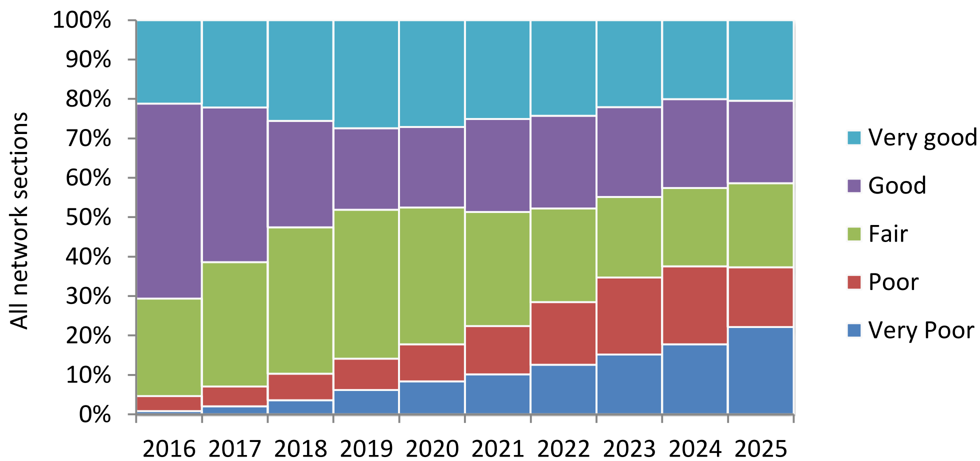
(a) Interstates (66 $million/year)



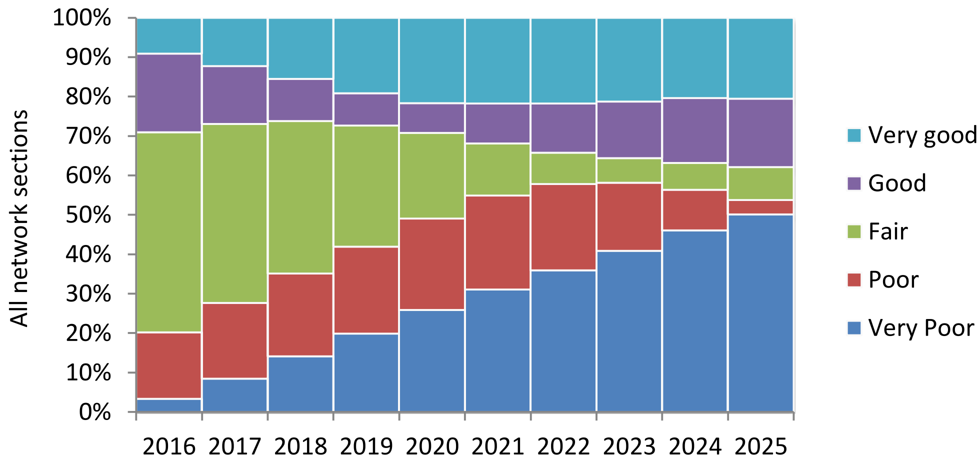
(b) State routes at Region 1 (37 $million/year)



(c) State routes at Region 2 (34 $million/year)



(d) State routes at Region 3 (46 $million/year)



(e) State routes at Region 4 (44 $million/year)

Figure 34 Condition ranges of each subset (Very good: 4.26-5, Good: 3.26-4.25, Fair: 1.76-3.25, Poor: 0.76-1.75, Very Poor: 0-0.75)

8 SAMPLE SIZE FOR PAVEMENT INSPECTION IN TENNESSEE

A paper titled "Sample Size and Precision for Pavement Inspection in the Maintenance Quality Assurance Program" was prepared for an international pavement conference "International Symposium on Frontiers of Road and Airport Engineering" in 2015. The conference paper is prepared based on a previous study on the optimized sample size of MRI pavement inspections for the TDOT maintenance division. Below is the manuscript of the paper.

8.1 Introduction

Every year, transportation agencies spend large amount of funds maintaining existing transportation assets, including pavements, markings, barriers and etc. The maintenance quality assurance programs have been developed to inspect the quality of maintenance activities on asset items. The maintenance quality assurance programs aim to detect insufficient maintenance efforts and poor facility performance by rating the level of service of asset items. (Schmitt et al. 2006; NCHRP 2010) Although pavement roughness, rutting, cracking and other related performance indices can be quickly collected by pavement profilers, many other highway asset items, such as barriers, traffic signs, pipes, ditches and drainage systems, require field survey conducted by an inspection crew. This level of service inspection process consumes considerable amount of time and budget. It is also impossible and unnecessary to inspect the all of the asset items. Therefore, a reliable sampling method is needed to determine the optimized sample size that is not only large enough to reflect the true condition of the whole population but also practical for limited budget and time.

The most straightforward sampling method is to select the sample size for a specific population, for example pavement segments, based on specified percent of the population. Previous studies found that a sample size of 2% to 5% is adequate to determine average condition of the network of roads. A sample size of 10% to 15% is recommended to predict the distribution of a pavement network condition. A sample size of 30% to 35% is needed to predict the cost for repairing poor segments. (Schmitt et al. 2006; AASHTO 2001; Templeton and Lytton 1984).

When the distribution of the population is available, statistical sampling tools can be utilized to determine sample size based on a specified confidence level and desired precision. Two commonly used approaches for highway assets are pass/fail approach and quantitative approach. (Schmitt et al. 2006; NCHRP 2010) Both of them can be used to calculate the passing percent as the level of service. The pass/fail approach has the advantages of fast rating and has been widely used by many agencies. The quantitative approach takes more efforts but can show the actual extent of the deficiencies of each asset item. For a quantitative variable, i.e. the actual number of cracking, the sample size can be determined based on the mean and variance of the data by Equation 1. (Schmitt et al. 2006; Stivers et al. 1999) For a qualitative variable, i.e. the pass/fail rating of cracking, the sample size can be determined by Equation 2. (McCullouch and Sinha 2003)

$$n = \frac{z^2 s^2}{e^2} \tag{11}$$

$$n = \frac{p(1-p)}{\left(\frac{e}{z}\right)^2} \tag{12}$$

Where, n is required sample size; z is the confidence coefficient (z-statistic) at different confidence levels; s is the standard deviation of ratings from a pilot study or historical data; e is the confidence interval, or precision; and p is the passing percent of the population that meets a specified criteria.

The presented paper discussed the determination of sample size for the pass/fail pavement inspection approach. The historical pavement inspection data from Tennessee were collected and analyzed for demonstration. The influence of significant factors on sample size was investigated. Sampling precision at different management levels was also evaluated. Further, statistical paired t-tests were conducted to evaluate the necessity of stratified sampling for different management levels and highway types.

8.2 Pavement Inspection

Many transportation agencies have developed pavement inspection manual for maintenance quality assurance, which includes the performance-based rating criteria for each sample unit and maintenance element. (de la Garza 2008) A unit length of a pavement segment independent of directions is usually used as the sample unit. The maintenance elements are the performance details of asset items, such as rutting, cracking, ditches, drains and etc. For the widely used pass/fail method, acceptable performance levels are defined for all the maintenance elements. The passing percent of each maintenance element then can be determined as the level of service by inspecting a sufficient number of samples. According to the field inspection manual of Tennessee Department of Transportation (TDOT) maintenance rating program, each 0.1 mile pavement segment contains as many as 49 maintenance elements, and the pass/fail method is adopted for inspection. (TDOT 2010)

8.3 Determination of Sample Size

The objective of sampling is to randomly select a statistically significant number of samples from within the total population of 0.1 mile pavement segments. For the pass/fail method, the sample size can be calculated based on Equation (3), derived from Equation (2) by introducing the population size (N) into the equation. This equation has been used by several states including Alabama, Louisiana and Mississippi at a maintenance district level (Kardian and Woodward 1990; Schmitt et al. 2006; de la Garza et al. 2008; NCHRP, 2010).

$$n = \frac{z^2 p(1-p)}{e^2 + \frac{z^2 p(1-p)}{N}} \tag{13}$$

8.4 Confidence Level and Precision

The confidence level is expressed as a percent and represents how often the true percent of the population lies within the confidence interval. Most researchers use the 95% confidence level, meaning that we can be 95% certain. The confidence coefficient z for 90% and 95% confidence level is 1.645 and 1.96, respectively. The confidence interval, also named precision, is an observed interval that frequently includes the passing percent of the population in this case. How frequently the observed interval contains the true passing percent is determined by the confidence level. A value of 0.07 is suggested by National Cooperative Highway Research Program (NCHRP) report 677; using a value of 0.05 will increase the sample size by about 75%.(NCHRP 2010) 95% confidence level and 0.04 precision means that we are 95% sure that the true passing percent of the population is between the calculated passing percent plus or minus 4%.

8.5 Influence of Parameters on Sample Size

8.5.1 Passing Percent

The passing percent (p) is the most significant factor influencing sample size. As shown in FIGURE 35, a proportion close to 0 or 1 needs fewer samples to inference the population proportion while a 50% passing percent means the largest sample size. NCHRP-677 recommends a value of 0.8 for Interstate highways.

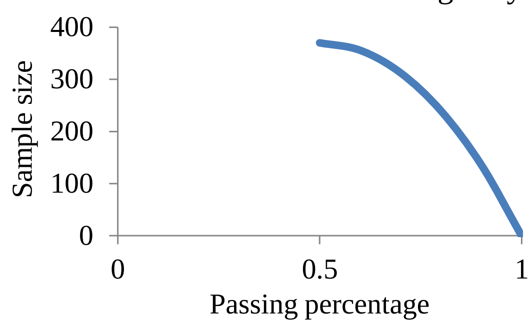


FIGURE 35 Influence of passing percent on sample number

8.5.2 Population Size

It can be seen from Equation (3) that population size (N) has much less influence on the sample size, especially when the population is large. As shown in FIGURE 36, a sample of 350 could be equally useful in examining the population of 10,000 as it would for 200,000.

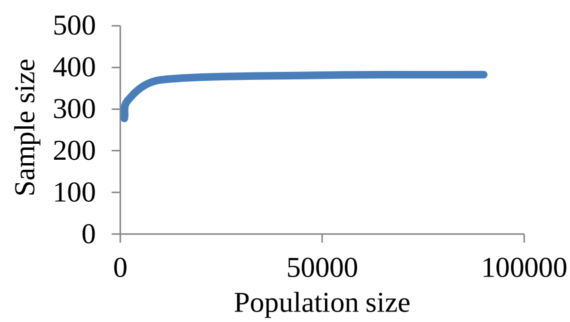


FIGURE 36 Influence of population size on sample number

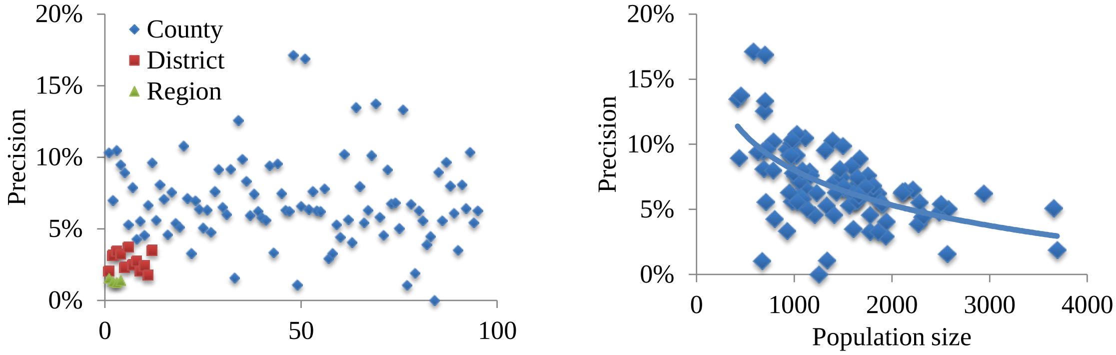
8.6 Calculated Sample Size

Based on the historical pavement inspection data of TDOT, the average passing percent of the 49 maintenance elements is 0.87. The total population size (0.1 mile segment) of the state is 138,960. For 95% confidence level and 4% precision, the calculated sample size at state level is 984, accounting for 0.7% of the population size. TDOT also divides the whole state into 4 regions and 12 districts for maintenance and management. The state has 95 counties. If the same confidence level and precision are expected at different management levels, the sample size at region, district and county maintenance levels are 3489, 8144 and 69546, respectively; accounting for 2.5%, 5.9% and 50% of the total population size. Generally, much more samples are needed if we want to maintain the high precision at smaller subgroups. Since the population size has little influence on the sample size, the main cause of the dramatic increase of samples size is the multiplication of subgroups.

8.7 Precisioni for a Given Sample Size

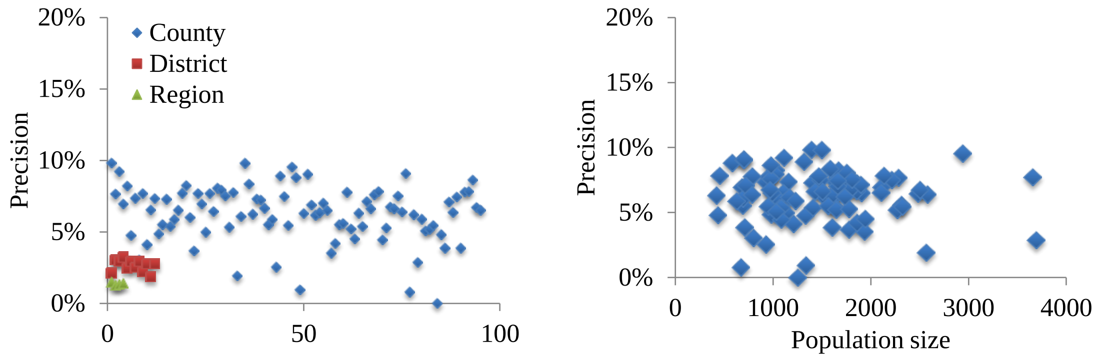
NCHRP-677 also reported that Florida requires the sample size is significant at the county and district levels; in Mississippi it is significant at maintenance district level; and in Washington they are significant at the regional level. (NCHRP 2010) Many highway maintenance departments are concerned about how significant or precise they can achieve at different management levels with their current efforts. TDOT inspects 7200 segments each year for the whole state. The precision at each region, district and county were calculated and shown in FIGURE 37. The confidence level here was 95%. It can be clearly seen that smaller subgroups have large confidence interval or low precision, due to the relatively insufficient sample size. The average precisions at the state, region, district and county levels are 0.7%, 1.4%, 2.7% and 6.9%, respectively. It can also be seen that several counties have very high precisions; this is because of their fairly high passing percent.

TDOT determines the sample size in each county according to the population size within that county. However, this causes insufficient samples for the counties with less road mileage, and then large confidence interval or low precision of the inspection as shown in FIGURE 37(b). Therefore, it is necessary to make sure a sufficient number of samples are inspected. FIGURE 38 shows the confidence intervals if sample numbers are evenly assigned disregarding the total road mileage within each subgroup. Compared to FIGURE 37, it clearly shows improved precision, especially at county level and for counties with less road mileage.



(a) Precisions at each management level (b) Precision vs. population size in counties

FIGURE 37 Calculated confidence intervals for different management levels



(a) Precisions at each management level (b) Precision vs. population size in counties

FIGURE 38 Calculated confidence intervals for evenly sampling

8.8 Stratified Sampling

Stratified sampling is to divide the population into homogeneous subgroups and sample each subgroup (stratum) independently. It can reflect the diversity of the population when there are significant differences between subgroups or between subgroups and the overall population. For example, the overall level of service for interstates may be different than that of state route. However, the analysis above shows that the sample size will increase dramatically if using stratified sampling. Therefore, it is of great interest to investigate the necessarily of stratified sampling.

Firstly, we investigate if there is significant difference between the levels of service of interstates and state routes. The passing percent of the 49 maintenance elements were calculated for both interstates and state routes. The paired t-test is conducted and the result shows that interstate was 0.1% higher than state route. The P-value (2-tailed) is 0.956, suggesting no significant difference between interstates and state routes. Therefore, there is no need to sample different highway types separately.

In addition to highway types, another potential stratum of stratified sampling is different geographical areas, i.e. regions or districts. Paired t-test is employed again to investigate the difference between the passing percent of the state and the passing percent of each region or district and the result is shown in FIGURE 39. The "Average difference" is the average passing percent at state level minus that at region or district level. Positive average difference means the passing percent at the state level is higher than that at the region or district level. P-value less than 0.05 suggests that there is significant difference between the two pairs, as indicated by the grey tail at the end of columns in FIGURE 39. It can be seen that 2 out of the 4 regions and 6 out of the 12 districts are significantly different than that at state level. Stratified sampling at those subgroups could potentially improve the precision for half of the subgroups. However, adding an additional stratum means multiply the sample size, it may not be cost-effective to use stratify sampling. In addition, the analysis above showed that the current practice of TDOT achieved an average of 2.7% precision at district level, which is fairly high as compared to 0.07 suggested by NCHRP.(NCHRP 2010)

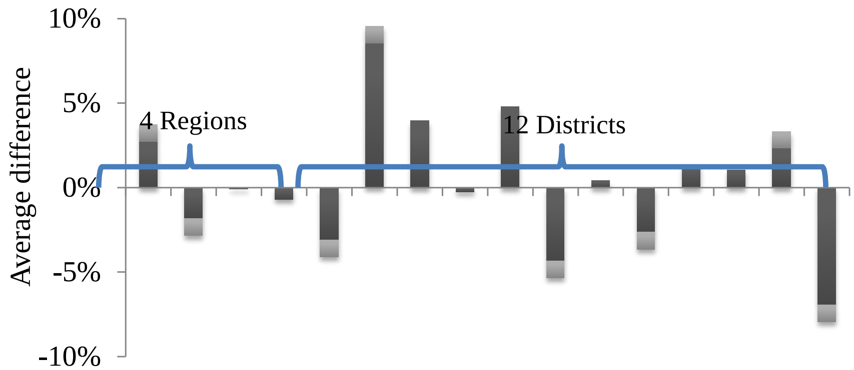


FIGURE 39 Comparison between state and different subgroups

8.9 Conclusions

It's found that passing percent is the most significant factor for sample size while population size has little influence, especially for large populations. The precision at different management levels (regions, districts and counties) were also analyzed. Based on the historical pavement inspection data of TDOT, we are 95% sure that, the inspected passing percent has a precision of 0.7% at state level, 1.4% at region level, 2.7% at district level and 6.9% at county level. An effective approach to improve the precision at lower management levels without increasing sample size is to evenly assign sample size among different subgroups disregarding the road mileage within that subgroup.

To investigate the necessity of stratified sampling, statistical paired t-tests are conducted to see if there are significant differences between subgroups and the overall population. It was found that there is no significant difference between the passing percent of interstates and state routes. Therefore, it is unnecessary to sample interstates or state routes separately. Sampling at district levels could potentially improve the significance level of half of the districts; however the sample size would be multiplied and is not cost-effective.

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9 FAILURE PROBABILITY OF PAVEMENT MAINTENANCE TREATMENTS

9.1 Background

Compared to pavement rehabilitation, pavement preventive maintenance is usually conducted before severe pavement deterioration occurs, aiming to extend pavement life and restore pavement functional performance. Preventive maintenance can effectively improve pavement serviceability and prevent further deterioration in an economical way(Peshkin, Hoerner and Zimmerman, 2004). Various pavement preventive maintenance treatments, including crack sealing, thin asphalt concrete overlay and different asphalt sealing coats, have been widely used for pavement maintenance.

The effects of pavement preventive maintenance include improving riding quality, retarding further deterioration, and repairing pavement distresses such as rutting, cracking, raveling, bleeding, asphalt aging and friction loss(Peshkin, Hoerner and Zimmerman, 2004). Different preventive treatments have different repairing functions. For example, crack sealing is mainly applied to prevent water infiltration into the underlying layers; whereas, asphalt sealing coats not only seal the pavement, but also enrich hardened asphalt and inhibit raveling. With the addition of aggregates, the asphalt sealing coats can effectively improve friction and repair minor permanent deformation. To select appropriate preventive treatments, it is necessary to investigate their field performance, especially their failure probability in terms of different pavement distresses.

9.1.1 Previous Studies

Many studies have been conducted to evaluate the effectiveness of pavement preventive treatments, mainly focusing on service life and surface treatment performance. Different measures of effectiveness included immediate improvement of pavement performance (Labi and Sinha, 2003, Dong and Huang, 2011, Dong, Huang, Richards and Yan, 2013), deterioration rate (Labi and Sinha, 2003, Dong and Huang, 2011, Dong, Huang, Richards and Yan, 2013), the long-term average pavement performance (Labi, Lamptey, Konduri and Sinha, 2005, Labi, Lamptey and Kong, 2007, Labi and Sinha, 2004, Wang and Wang, 2013, Chen, Lin and Luo, 2003), treatment service life(Labi, Lamptey, Konduri and Sinha, 2005, Irfan, Khurshid and Labi, 2009, Wang, Wang and Mastin, 2011), life cycle cost(Rajagopal and George, 1990) and the area between the performance curve and lower performance threshold(Dong and Huang, 2011, Dong, Huang, Richards and Yan, 2013, Labi, Lamptey and Kong, 2007). Although the average pavement performance and riding quality are more preferable for public satisfaction, service life is usually employed to quantify the effectiveness of preventive treatments for designing a maintenance strategy plan and conducting life-cycle cost analysis. In the Highway Pavement Management Application (HPMA), which is a Pavement Management System (PMS) software used by several state agencies, deterioration models are not usually included for preventive treatments because they do not improve pavement structural function significantly. Instead, those preventive treatments are treated as extensions of service life added to the original pavement in the maintenance strategy analysis in the HPMA.

In previous studies, most frequently used pavement performance indicators include Pavement Condition Rating (PCR), Pavement Serviceability Index (PSI), International Roughness Index (IRI), rutting, and overall distress score. Pavement cracking (Wang, Mahboub and Hancher, 2005, Wang, 2012, Haider and Chatti, 2009) and friction (Wang and Wang, 2013) were also investigated. Employed mathematical methods include direct ranking, statistical difference tests, time series performance regression and multiple linear regressions. Innovative statistical methods including Logistic regression (Wang, 2012, Haider and Chatti, 2009) and parametric survival models (Wang, Mahboub and Hancher, 2005, Dong and Huang, 2012, Chen, Williams and Mervyn, 2014) have also been employed to investigate cracking development on treatment surface.

To evaluate the effectiveness of pavement treatments, previous studies classified them into different traffic levels, climatic conditions and pavement functional classes (Labi, Lamptey, Konduri and Sinha, 2005, Labi, Lamptey and Kong, 2007). The pre-treatment pavement performance, which reflects the combined effects of environment, traffic, pavement materials and structural capacity of that specific road section, is a significant factor for treatment performance (Dong and Huang, 2011, Dong, Huang, Richards and Yan, 2013, Rajagopal and George, 1990, Eltahan, Daleiden and Simpson, 1999). Therefore, to effectively characterize the performance of different preventive treatments, potential significant factors including climatic, traffic, pavement structural and pre-treatment deterioration conditions should be considered.

9.1.2 LTPP Experiments

The Long Term Pavement Performance (LTPP) program is managed by Federal Highway Administration (FHWA) and has been monitoring in-service pavement performance since the 1990s. It includes several experiment sections to address the effectiveness of pavement maintenance and rehabilitations.(Hanna, Tayabji and Miller, 1994) The LTPP database has been utilized by many researchers to evaluate the performance of preventive treatments. In addition, several FHWA studies have been conducted to evaluate the effectiveness of pavement treatments in LTPP test roads by statistical significance tests and developed time-history performance curves (Eltahan, Daleiden and Simpson, 1999, Hall, Correa and Simpson, 2003, Perera and Kohn, 1999). It is noted that one study compared the survival times of different preventive treatments in the LTPP SPS-3 experiment(Eltahan, Daleiden and Simpson, 1999). The results showed the failure probability of sections that are in poor condition before treatment is 2 to 4 times higher than those sections in better conditions. The authors recommended that parametric methods could be employed to predict survival times at any given failure probability, which will be one of the focuses of this presented study.

9.1.3 Right Censored Observation

During the observation of a pavement treatment performance, a treatment failure is recorded when the pavement performance drops below a lower threshold value. However, a frequently encountered scenario is that no clear failure is observed during the survey because most pavements are repaired before severe deterioration occurs. In this case, we only know that the service time is longer than a certain time t, which is a typical right censored observation. Although the right censored data do not show the exact failure time it still provides useful information that the treatment survives till time t.

To incorporate those unobserved failure events, a statistical regression method called survival analysis has been developed and applied in many fields, including biostatistics, medical treatments, economics and engineering to evaluate various duration problems (Washington, Karlaftis and Mannering, 2011). In pavement engineering, previous studies utilized survival analysis to analyze pavement failure time, initiation of pavement cracking, or nominal pavement failure indicated as the rapid increase of fatigue cracking (Wang, Mahboub and Hancher, 2005, Dong and Huang, 2012, Chen, Williams and Mervyn, 2014, Prozzi and Madanat, 2000, Loizos and Karlaftis, 2005). Those studies found that traffic level, pavement layer thickness, intensity of precipitation and freeze-thaw cycles are significant factors for the initiation of pavement cracking.

9.2 Preparation of Data

9.2.1 Preventive Maintenance Treatments

In the LTPP database, the "MNT_IMP" Table contains the basic information of all pavement maintenance activities conducted during the monitoring of the LTPP program. Only asphalt seal coat preventive maintenance treatments were collected and used in this study. It is noted that most of the asphalt preventive maintenance was included in the Specific Pavement Studies-3 (SPS-3) experiment, which is designed to evaluate the effectiveness of preventive maintenance treatments for flexible pavements (Hanna, Tayabji and Miller, 1994). Generally, asphalt seal coats aim to seal the pavement, repair light distress and improve surface friction; however they add little structural capacity. Four main asphalt seal coat treatments collected from the LTPP database include thin asphalt overlay, chip seal, slurry seal and fog seal. The characteristics of these preventive treatments are summarized below (Peshkin, Hoerner and Zimmerman, 2004):

- Thin Asphalt Concrete (AC) overlays are plant mixed asphalt mixtures applied on pavement at a thickness of 3 cm (1.25 in.). They can be used on most traffic and climatic conditions and repair most types of pavement distress, including cracking, bleeding, and raveling.
- Chip seal consists of asphalt (usually emulsion) followed by aggregate chips, which are then rolled to facilitate embedment of the chips into the asphalt at a depth range of 50 to 70 %. It aims to repair cracking, raveling and improve friction loss.
- Slurry seal is the application of a mixture of well-graded fine aggregate, mineral filler and asphalt emulsion. It can effectively seal pavement, improve skid resistance and repair cracking, raveling and asphalt aging.
- Fog seal is the application of a diluted asphalt emulsion directly on pavement with no aggregate. It aims to seal the pavement, prevent moisture infiltration, enrich hardened asphalt and inhibit raveling.

9.2.2 Pavement Performance Indicators

The LTPP database records four types of pavement performance data, roughness, distress, rutting and friction, all of which are critical for riding quality, safety and the integrity of pavement structure. For example, pavement cracking permits water infiltration into the underlying layers and may cause structural failure. The roughness, fatigue cracking, longitudinal cracking, thermal cracking and rutting have been included in the Mechanical Empirical Pavement Design Guide (MEPDG) as performance criteria to design new or rehabilitated flexible pavement. The MEPDG recommends three levels of criteria depending on road class, such as interstate, primary or secondary(NCHRP, 2002). As listed in Table 8, the criteria for primary roads were used as the pavement failure threshold value in this study. Although MEPDG does not include friction criteria, a commonly used maintenance trigger value of friction number was adopted (Peshkin, Hoerner and Zimmerman, 2004). The six pavement performance indicators are summarized as follows:

- Roughness is the longitudinal irregularity along pavement surface and is the most widely used pavement performance indicator. It directly affects riding quality and vehicle operation costs and has been used as the terminal performance indicator in many pavement design methods (Huang, 1993).
- Alligator cracks are bottom-up fatigue cracks caused by the repeated bending of asphalt layer under traffic. A large area of alligator cracking indicates severe fatigue failure of pavement surface.
- Wheel path longitudinal cracks are the typical surface-down fatigue cracks at the edge of wheel patch. They are usually caused by large tensile stress on pavement surface and the shearing of surface mixture.
- Thermal cracks typically appear as transverse cracks on the pavement surface and are usually caused by rapid temperature drops. It reduces riding quality and normally indicates the aging and hardening of asphalt (NCHRP, 2002).
- Rutting is usually the consolidation and/or plastic flow of asphalt mixture under wheel loads. It is a severe safety concern for the public. Generally, rutting in excess of 6 mm is considered to be a hydroplaning safety hazard by state transportation agencies (Jackson and Baldwin, 2000).
- Friction (skid resistance) is directly related to public safety. The loss of friction is mainly caused by surface abrasion, bleeding and/or aggregate loss. Effective treatments are expected to provide sufficient friction over pavement service lives. It is noted that the criterion for friction number is the lower threshold; whereas, the other five threshold values are higher threshold values.

Table 33 MEPDG Recommended Performance Thresholds for Pavement Design

Pavement performance indicator	Failure criteria for primary roads
Roughness	3.14 m/km (200 in./mile)
Alligator cracking	20% of the lane area
Longitudinal cracking	132.6 m/km (700 ft/mile)
Transverse cracking	132.6 m/km (700 ft/mile)
Rutting depth	12.5 mm (0.5 in.)
Friction Number	35

9.2.3 Data Collection

The pavement performance data and related traffic, climatic and pavement structure data of the preventive treatments were exported from different Tables and then merged together according to their state code, SHRP ID and construction number. Table 34 lists all the data and the source Table in the LTPP database. The sum of different severity levels of cracking was used. Table "TRF_ESAL_INPUTS_SUMMARY" is a summary of inputs in the annual ESAL estimation for a given section. It includes average freeze index, average annual precipitation, climatic region, and structural number for asphalt pavement. The freeze index is the negative of the sum of all average daily temperatures below 0°C within a year. High freeze index values indicate a severe freezing condition. The LTPP climate datasets contain several types of climatic information such as average annual temperature, snow falls and etc. Those variables have strong correlations with each other and with the climatic regions defined by LTPP and thus could not be all included in one model in order to avoid incorrect parameter estimates. Only two representative variables, freeze index and precipitation, were used in the multiple parametric survival models as shown in Table 34. The structural number is an abstract number indicating the relative structural strength of a pavement and can be converted to layer thicknesses using a layer coefficient which represents the relative strength of the materials in that layer.(Huang, 1993) Therefore, comparing to the total pavement or subgrade support used by previous researchers (Dong and Huang, 2011, Chen, Lin and Luo, 2003, Rajagopal and George, 1990, Dong and Huang, 2012), the structural number is a better indicator for pavement structural capacity. Figure 40 displays the scatter plot of predictors, showing no significant correlations.

Table 34 Variables Collected from the LTPP Database

Variables		Name	Source Table in LTPP Database
Targets	Pavement Condition Indicators	International Roughness Index (m/km)	MON_PROFILE_MASTER
		Alligator cracking (m^2)	MON_DIS_AC_REV
		Longitudinal cracking (m)	
		Transverse cracking (m)	
		Rutting depth (mm)	MON_RUT_DEPTH_POINT
		Friction Number	MON_FRICTION
Predictors	Traffic	Annual kilo equivalent single axle load (kESAL)	TRF_ESAL_COMPUTED
	Climate	Annual average freeze index (°C-days)	TRF_ESAL_INPUTS_SUMMARY
		Annual precipitation (cm)	
	Structure	Structural number	

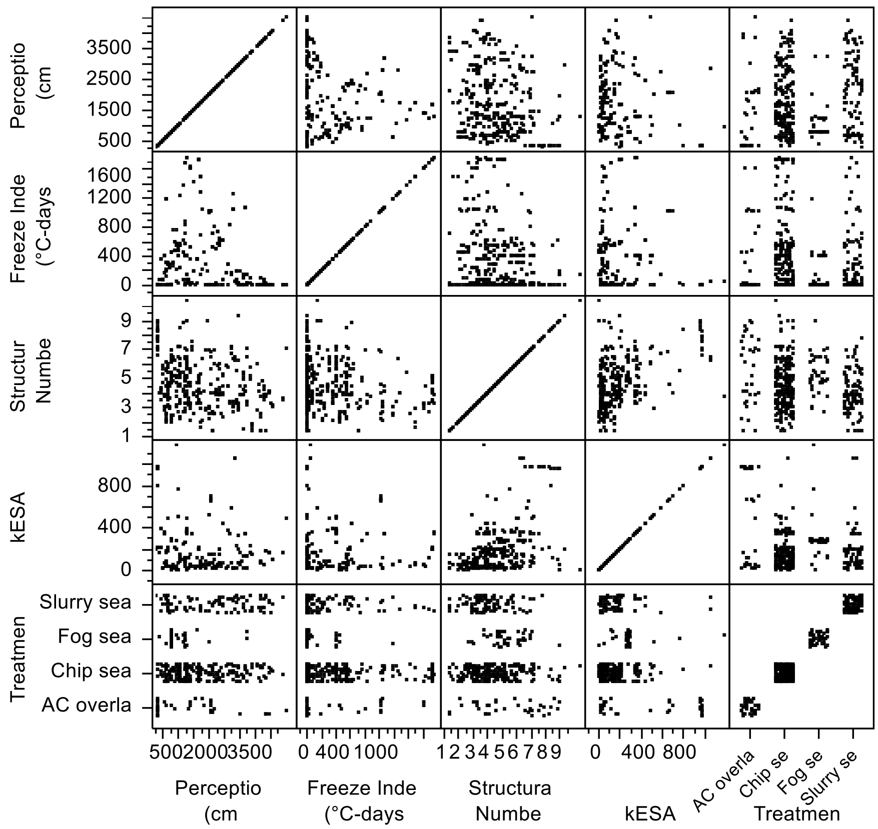


Figure 40 Scatter plot of predictors

Each LTPP test section is 152 m long and typically 7.3 m wide (2 lanes). This area was used to calculate the extents of cracking per unit area (Elkins, Schmalzer, Thompson and Simpson, 2003). During the survey, if a performance indicator increased above or dropped below certain thresholds, a treatment failure was observed and the censorship was denoted as 0. If a treatment did not fail in the survey, it was right censored and denoted as 1. In the model of each pavement performance indicator, the pre-treatment factor is the most recent corresponding pavement performance before treatment. For example, the pre-treatment performance for the alligator cracking model is the extent of alligator cracking before the treatment. It was found that only a few pre-treatment rutting and friction number data were available. In addition, pavement receiving preventive treatments usually have little rutting problem since preventive treatments generally do not address a rutting problem (Peshkin, Hoerner and Zimmerman, 2004). In addition, the post-treatment surface friction is mainly determined by the characteristics of a new treatment. Therefore, the rutting and friction survival models did not include the pre-treatment factor. Table 35 summarizes the sample numbers and the percentage of observed failure (uncensored) data. It can be seen that the failure percentage was low for most of the performance indicators.

Table 35 Sample Numbers and Failure Percentage for Each Survival Model

Performance Indices	Asphalt overlay	Chip seal	Fog seal	Slurry seal	Total	Failure (%)

Roughness	34	218	43	96	354	6%
Alligator cracking						6%
Longitudinal cracking	25	136	52	45	258	21%
Transverse cracking						35%
Rutting	10	78	5	62	144	14%
Friction loss	16	118	26	69	204	16%

9.3 Survival Analysis

9.3.1 Survival Function

If we define the service life of a pavement treatment as the survival time, T, the duration function, F(t), gives the probability that the treatment will not survive until time t (Equation 1) (Washington, Karlaftis and Mannering, 2011).

$$F(t) = P(T \leq t) = \int_0^t f(u)du \qquad (14)$$

Where, f(u) is the density function of the treatment failure.

The survival function, S(t), which is the probability that the treatment will provide service at least up to time t can be expressed as Equation 2.

$$S(t) = P(T \geq t) = 1 - F(t) = 1 - \int_0^t f(u)du \qquad (15)$$

Equation 3 presents the hazard (rate) function, h(t), which is the conditional probability that the treatment will not survive between time t and t+dt, given that the pavement has survived up to time t. The hazard function is the ratio of the probability density function f(t) to the survival function S(t).

$$h(t) = \lim_{\Delta t \to 0} \left(\frac{P(t \leq T \leq t + \Delta t)}{\Delta t}\right) = \frac{f(t)}{S(t)} \qquad (16)$$

Usually, the Kaplan-Meier curves are generated first to show the shapes of survival functions. The Kaplan-Meier curve is the most popular nonparametric survival model, which is a maximum likelihood estimator of the survival function using product-limit method (Washington, Karlaftis and Mannering, 2011). The Kaplan-Meier curve does not rely on any specific statistical distribution assumption, but it illustrates the general survival probability of the population.

9.3.2 Parametric Survival Analysis

To investigate the influence of predictors on the hazard rate or survival time, the predictors are incorporated into the hazard function. Then, the survival model can be estimated by the Maximum Likelihood Estimation (MLE) method (Washington, Karlaftis and Mannering, 2011). The likelihood ratio test can be used to test the significance of each predictor by comparing the log-likelihood from the fitted model to one that removes each term from the model individually.

The hazard function indicates the underlying mechanism of the failure probability. Three frequently used hazard functions are exponential, Weibull, lognormal and log-logistic. The exponential function suggests that the hazard rate is a constant along time. The Weibull function indicates that the hazard rate increases or decreases monotonically. The lognormal function allows the hazard rate to increase to an inflection point and decrease thereafter (Wang, Mahboub and Hancher, 2005, Dong and Huang, 2012). Previous studies on pavement failure usually employed the Weibull hazard function to model the serviceability of pavement since the failure probability of asphalt pavement increases as the service time increases (Wang, Mahboub and Hancher, 2005, Dong and Huang, 2012).

A proper hazard function is critical for the high accuracy of a parametric survival model. There have been several diagnostic methods developed to evaluate the relative quality of a statistical model for a given set of data. One statistical test that can be used to select a proper hazard function is the corrected Akaike's Information Criterion (AICc)(Proust, 2013). It estimates the relative distance between the unknown true-likelihood function of the data and the likelihood function of the fitted model. The model with smaller AICc value is considered to be closer to the true distribution.

The statistic software JMP 10.0 was employed to conduct the diagnoses, as well as the following nonparametric and parametric survival analyses. Table 36 summarizes the AICc values of the investigated hazard models. Although the AICc values were very close for several performance indicators, the more preferable model could still be identified based on the smaller AICc value rule:

- The Weibull model was best-suited to describe the failure probability of roughness. The diagnosis also showed that the estimated shape parameter of the Weibull model was higher than 1, indicating a monotonically increasing hazard rate.
- The Lognormal model was suitable for the three cracking distresses, indicating that the failure probability of pavement cracking rose to a peak and then decreased.
- The Exponential model was suitable for rutting and friction failure, implying that the failure rate of rutting and friction varied little along time.

Table 36 Model Comparisons by the AICc values

Pavement Performance Indicators	Exponential	Weibull	Lognormal
Roughness	254.9	**253.6**	253.7
Alligator cracking	169.7	170.4	**169.2**
Longitudinal cracking	478.7	476.4	**469.0**
Transverse cracking	746.5	743.3	**729.1**
Rutting	**327.0**	329.1	328.3
Friction number	**189.7**	191.0	190.7

9.4 Discussion of Result

9.4.1 Survival Probabilities for Different Performance Indicators

Figure 41 presents the Kaplan-Meier curves, or the survival probabilities, of the six pavement performance indicators. It can be seen that the survival probabilities decreased with the increase of pavement age. Transverse cracking had the lowest survival probability followed by longitudinal cracking, friction loss, rutting, alligator cracking and roughness. Generally, severe critical fatigue cracking (alligator cracking) and pavement serviceability loss (roughness) were not observed for the preventive treatments presented in this paper; whereas transverse cracking turned out to be more severe. As also indicated in Table 35, the failure percentages for roughness and alligator cracking were low.

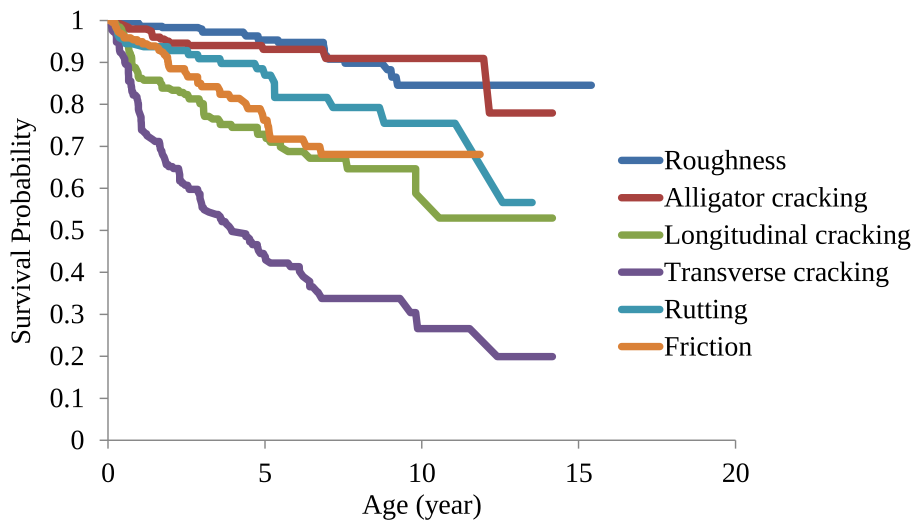


Figure 41 Survival probabilities for different pavement performance indicators

9.4.2 Effectiveness of Preventive Treatments

Table 37 shows the p-values of the likelihood tests. A significance level of 0.05 is usually used, indicating that the probability of the result by chance is less than 5%. Figure 42 shows the influence of different factors on the treatment failure probability. Observations from Figure 42 are discussed below.

Roughness

It can be seen from Figure 42(a) that all four preventive treatments had low failure probabilities for roughness. Only 5.6% of the samples failed the roughness criterion before the end of the survey or before a new treatments was installed, which also explains why some factors seemed to have not taken effect. However, the pre-treatment roughness level still showed significant influence on the treatment failure probability. Poor pre-treatment performance greatly increased the failure probability and therefore preventive maintenance treatments should not be applied on severely deteriorated pavements.

Alligator cracking

Similar to the situation of roughness, only 6.5% of investigated samples failed the alligator cracking criterion. It can be seen from Figure 42(b) that the failure probabilities were generally low except for the fog seal. In addition to the extent of pre-treatment alligator cracking, structural number was also a significant factor. Stronger pavement structural capacity effectively reduced the bending of the asphalt layer and the risk of alligator cracking.

Longitudinal cracking
Around 21% of the samples failed the longitudinal cracking criterion. Pre-treatment longitudinal cracking condition was the only significant factor. Generally, asphalt overlays and chip seals performed better than slurry seals and fog seals.

Transverse cracking
It can be seen from Table 37 and Figure 42(c) that most of the factors are significant or marginally significant for the failure probability of the transverse cracking criterion except the traffic level. A large extent of pre-treatment cracking and/or a severe freeze condition increased the risk of transverse cracking. High annual precipitation reduced the risk of transverse cracking in that it is less likely to have a severe temperature drop in high precipitation regions. Stronger pavement capacity also tended to reduce transverse cracking. Generally, chip seal had the lowest transverse cracking failure, followed by asphalt overlay, slurry seal and fog seal.

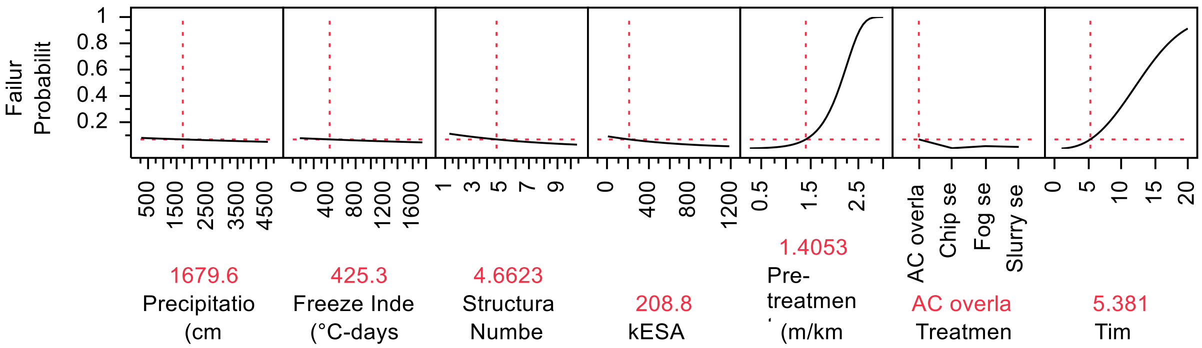
Rutting
As shown in and Figure 42(d), it is interesting to note that the precipitation was the only significant factor for rutting failure. It is probably because high precipitation is usually accompanied with high temperature which aggravates asphalt pavement rutting problem. Among the four treatments, fog seal had the lowest resistance to rutting because it only has a thin layer of asphalt without aggregate.

Friction Number
The significant factors for friction failure include traffic level, precipitation and freeze index. Heavy traffic aggravated the abrasion of surface texture. High precipitation or low freeze index usually correlate with high temperature, which may cause asphalt soften and/or bleeding and therefore the friction loss. Slurry seals provided the best friction while chip seal tended to lose friction. A potential reason for the friction loss of chip seals is that the imbedded aggregate might be easily pulled out or worn out by moving wheels.

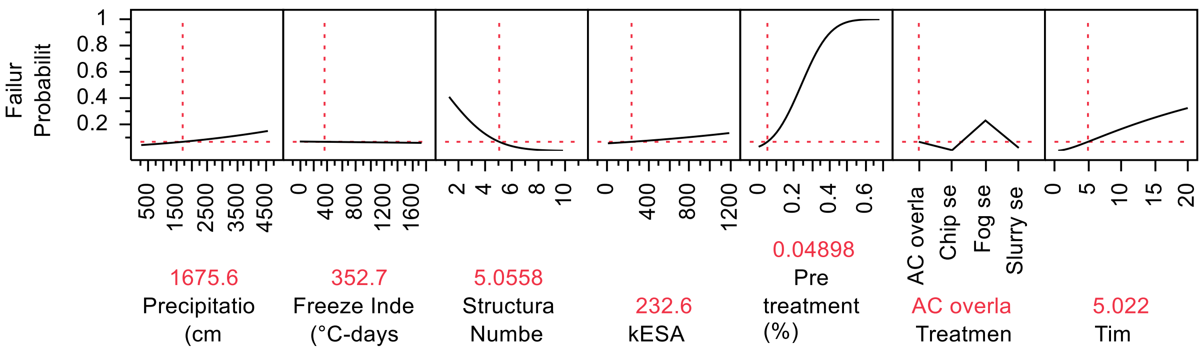
Table 37 Significance Test Results (P-values) for Predictors

Predictors	Roughness	Alligator cracking	Longitudinal cracking	Transverse cracking	Rutting	Friction number
Precipitation	0.75	0.48	0.82	**0.06***	**0.04**	**0.007**
Freeze Index	0.61	0.91	0.28	**<.0001**	0.59	**0.04**
Structural Number	0.45	**0.017**	0.15	**0.009**	0.12	0.24
kESAL	0.2	0.63	0.14	0.25	0.42	**0.0004**
Pre-condition	**<.0001**	**<.0001**	**<.0001**	**<.0001**	NA	NA
Treatment	0.44	**0.007**	**<.0002**	**0.001**	**0.04**	**0.008**

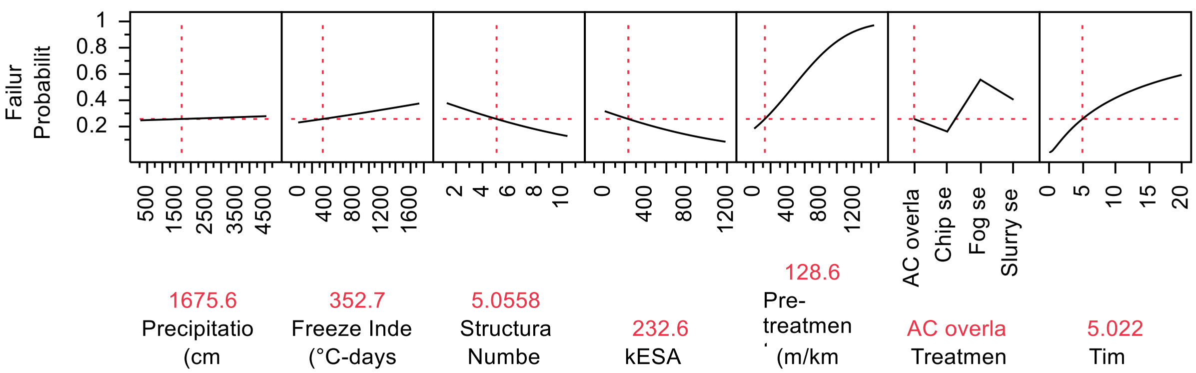
Note:*0.06 is close to 0.05 and can be regarded as marginally significant.



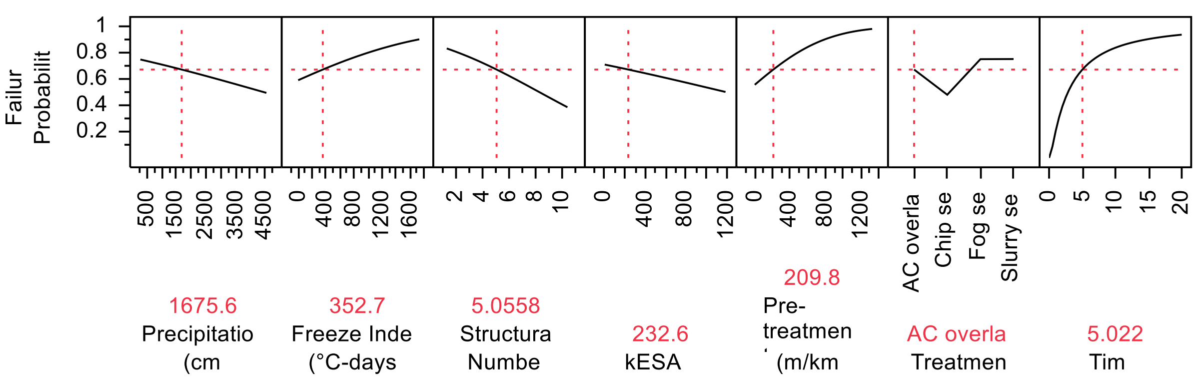
(a) Roughness



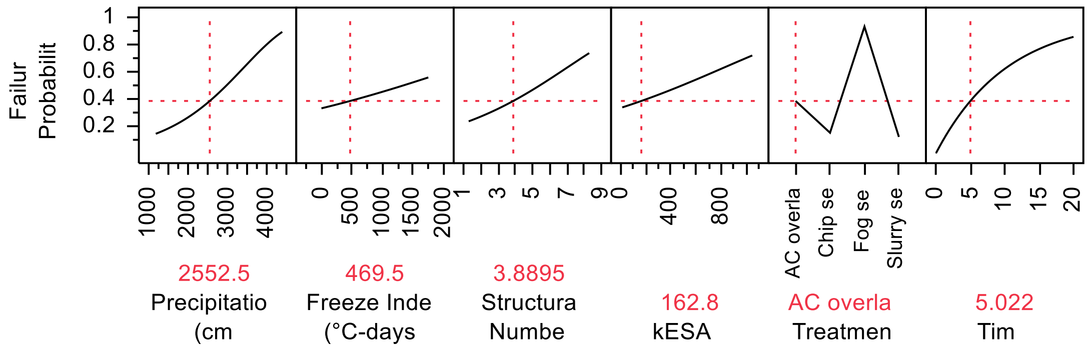
(b) Alligator cracking



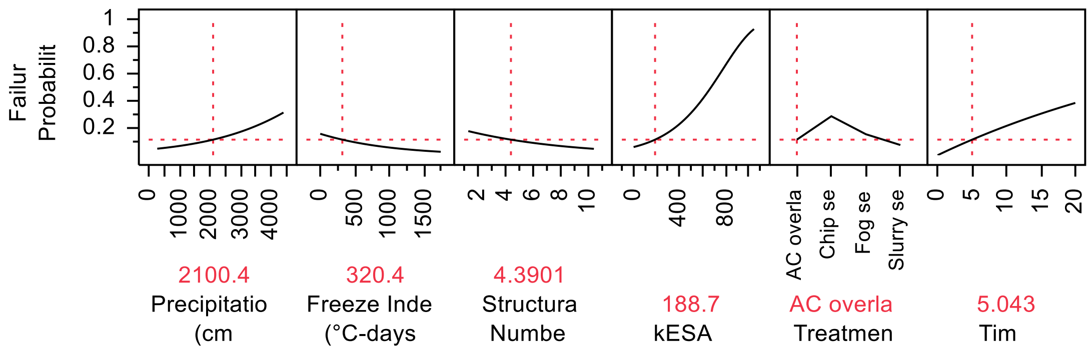
(c) Longitudinal cracking



(d) Transverse cracking



(e) Rutting



(f) Friction number

Figure 42 Influence of different predictors on the failure probability of treatment

TABLE 44 summarizes the influence of different factors on the failure probability of treatments and presents the performance ranking of treatments. Generally, pre-treatment pavement performance was the most significant factor for post treatment performance. Pavement structural capacity, climate and traffic level were all significant factors for treatment deterioration. Chip seal performed best, followed by asphalt overlay, slurry and fog seal. The rankings of treatments generally agreed with the finding from previous studies (Wang and Wang, 2013, Wang, Mahboub and Hancher, 2005, Wang, 2012).

Table 38 Influence of Significant Factors and Performance Rankings Treatments

Predictors		Roughness	Alligator cracking	Longitudinal cracking	Transverse cracking	Rutting	Friction
High precipitation						↑[a]	↑
Severe freeze condition					↑		↓[b]
Low structural capacity			↑		↑		
High traffic level							↑
Poor Pre-condition		↑	↑	↑	↑		
Treatment ranking	1	Chip seal	Chip seal	Chip seal	Chip seal	Slurry seal	Slurry seal
	2	Slurry seal	AC overlay	AC overlay	AC overlay	Chip seal	AC overlay
	3	Fog seal	Slurry seal	Slurry seal	Slurry seal	AC overlay	Fog seal
	4	AC overlay	Fog seal	Fog seal	Fog seal	Fog seal	Chip seal

Note: [a]"↑" means high precipitation increased failure probability in terms of rutting; [b]"↓" means severe freeze condition decreased the failure probability of friction loss.

9.5 Summary

Statistical diagnosis showed that the failure rate of pavement roughness increased monotonically as the increase of pavement age. The failure probability of pavement cracking rose to a peak and then decrease; whereas, rutting and friction failure rate varied little along time.

Pre-treatment pavement performance was the most significant factor for the performance of post preventive treatments. Poor pre-treatment performance significantly increased the failure probability of treatments. Therefore, it is not recommended to apply preventive treatments on poor pavements. Pavement structural capacity, climate and traffic levels significantly affected post treatment performance. Stronger pavement structural capacity greatly reduced fatigue alligator cracking on treatment surface because the bending of the asphalt layers could be reduced with a strong bases and subgrade support. Climatic factors influenced the failure of transverse cracking, rutting and friction loss. Generally, warm climate increased the risk of rutting and friction loss.

Generally, chip seals provided the best overall effectiveness followed by asphalt overlay, slurry seal and fog seal. Chip seals performed best in terms of roughness and cracking, whereas they were vulnerable to friction loss, indicating that the aggregate chips could be easily pulled out due to the loss of asphalt adhesion. Fog seal performed the worst, because it is simply an asphalt sealing coat without aggregate. The roughness or distress performance of fog seal is mainly determined by the original pavement.

9.6 Reference

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10 CORRELATIONS OF PAVEMENT CONDITION INDICES IN TENNESSEE

10.1 Background

With the rapid development of Pavement Management System (PMS) software and automatic pavement survey methods, large quantities of pavement distress data become available. Based on the measured condition data, the intuitive and straightforward approach to evaluate the pavement condition is through one type of distress which can reflect the specific aspect of pavement condition, such as roughness, rutting, cracking and patching (Carey and Irick, 1960). However, the evaluation of the same road section based on different types of condition data can be inconsistent and sometimes even contradictory. For instance, there is no cracking on the pavement section of Interstates 81 in Sullivan County in the State of Tennessee, and its IRI value is quite low as 1.32 m/km, while the rutting depth is as great as 12.7 mm indicating that this section needs to be treated immediately.

Thus, a comprehensive pavement condition evaluation which can incorporate various types of condition data is of great necessity. The existing pavement condition indices such as Present Serviceability Rating (PSR), Present Serviceability Index (PSI) and Pavement Condition Index (PCI) are widely used and acknowledged indices which can incorporate various condition measurements into one index. In the 1950s, the pavement serviceability condition which was rated from 0 to 5 points was investigated, and the PSR for each road section was established based on experts' and laymen's judgement (Carey and Irick, 1960, Nakamura, 1962). PCI is calculated by subtracting the deducted values of each distress at each severity level from 100 which is the best possible condition value (ASTM D6433-99, 1999). Besides the above pavement condition evaluation indices that are currently used throughout the United States, many state Departments of Transportation (DOTs) also have developed their own condition indices such as Texas DOT's condition score, South Dakota DOT's surface condition index, Ohio DOT's pavement condition rating (Gharaibeh, Zou and Saliminejad, 2009) and Tennessee DOT's Pavement Distress Index (PDI). In addition, many studies have been done to modify the existing pavement condition indices or propose new ones through different methods such as expert survey, fuzzy logic method (Gharaibeh, Zou and Saliminejad, 2009, Ben-Akiva and Ramaswamy, 1993, Muthén and Muthén, 2012, Shafizadeh and Mannering, 2003) and multiple regression. For instance, Eldin and Senouci (1995) determined the coefficients of cracking and rutting of pavement condition index by the judgement and experience of experts. In the fuzzy logic method, the relative importance of pavement distresses was generally determined by empirical knowledge.

For pavement with different surface types or functions, the relative importance of different distresses may vary. For example, the newly constructed Open Graded Friction Course (OGFC) generally has higher macro texture and roughness level which are comparable to an old traditional dense mixed asphalt mixture surface. However, the comprehensive condition of OGFC is higher than that of the old dense mixed asphalt surface. Another example is that, for the same amount of cracking, patching or raveling distress, a rural collector road of low traffic volume and speed limit can still provide sufficient service level while a principal arterial Interstates road with high traffic volume and speed limit may need maintenance immediately. Therefore, the relative importance of the same distress varies for roads with different functional classes. As summarized from above, it is of great interests to investigate the relationship between condition measurements and the comprehensive pavement condition and the relative importance of condition measurements on the comprehensive pavement condition for different roads.

10.1.1 Confirmatory Factor analysis (CFA)

CFA is the basis and measurement model of structural equation modeling (SEM) which deals with the relationship between the observed variables and latent variables objectively and rationally (Brown, 2014). Ben-Akiva, Humplick, Madanat and Ramaswamy (1993) estimated the weight coefficients of latent PSI from the observed pavement distresses, pavement characteristics, traffic and environmental variables through SEM analysis. In order to conduct SEM analysis, the fundamental step is to investigate the relationship between the observed and latent variables and to ensure the good fitness of CFA. CFA is a widely used statistical method in the area of psychology, education, economics, and sociology. In those fields, the observed variables include behavioral ratings, test scores and questionnaire results and the latent variables can be personality, students' study ability, etc. CFA was used to obtain the estimates of the relationship between the observed variables and latent variables and the relationship among latent variables and verify the measurement model which is driven by prior theory or research results (Brown, 2014, Ben-Akiva, Humplick, Madanat and Ramaswamy, 1993, Ramaswamy, 1989, Wang and Wang, 2012, Chen, Dong, Zhu and Huang, 2016).

In the pavement engineering field, the comprehensive pavement condition can be regarded as the latent variable which is not easily or directly measured, while various condition measurements can be treated as the observed variables aiming to reflect the condition level (Brown, 2014, Ben-Akiva, Humplick, Madanat and Ramaswamy, 1993, Ramaswamy, 1989, Wang and Wang, 2012, Chen, Dong, Zhu and Huang, 2016, Chu and Durango-Cohen, 2008). Compared with other data mining methods such as classification and regression trees, the advantages of CFA include that the CFA model is specified in advance (Brown, 2014), the unobserved variables are treated as latent variables, the correlations among the variables can be taken into consideration, etc. Therefore, CFA method could potentially be used to investigate the correlations between comprehensive pavement condition and collected pavement condition measurements in an objective and rational rather than subjective way.

10.1.2 Objectives and Scope

The CFA method was utilized to evaluate the correlations between latent comprehensive pavement condition and condition measurements and the relative importance of condition measurements on the comprehensive pavement condition for different roads. Pavement condition data collected by Tennessee DOT (TDOT) was used in this study. Two types of road functional class, Interstates and state routes, were considered. Two CFA models were established. The single factor model included one overall condition index and the multiple factors model included three condition indices representing ride quality, early age cracking, and severe pavement deterioration, respectively. EFA was conducted to determine the number of multiple latent factors. The relative importance of condition measurements for Interstates and state routes was determined and discussed.

10.2 Statistical Methodology

10.2.1 CFA Model

In the pavement engineering field, the comprehensive condition can illustrate the pavement condition better than the single distress indicator. Generally, the comprehensive condition cannot be measured or observed directly. Thus, the comprehensive condition is treated as the latent variable. The measurable pavement roughness or distresses are the observed indicators of the latent condition. For instance, latent PSI was calculated through the slope variance, cracking, patching and rutting measurements (Ben-Akiva, Humplick, Madanat and Ramaswamy, 1993, Chu and Durango-Cohen, 2008). Among those latent approaches, SEM analyses could be used to estimate the latent pavement performance from observed pavement damage and the weight coefficients of overall pavement condition index (Ben-Akiva and Ramaswamy, 1993, Ramaswamy, 1989, Chen, Dong, Zhu and Huang, 2016) and to evaluate the pavement maintenance effectiveness (Chu and Durango-Cohen, 2008). SEM consists of two models, the structural model and the measurement model. The structural model evaluates the relationship among the latent variables. While the measurement model analyzes the relationship between the observed variables (or indicators) and latent variables (or factors). Moreover, the measurement model is of critical importance and it should be established before the further SEM analysis. If the SEM model is poor fitted, it is more likely that the measurement model is misspecified than that the structural model is not established well (Brown, 2014). Therefore, CFA models of pavement comprehensive condition index were proposed and investigated in this study. If CFA models fit well and more observed variables such as traffic, environment and pavement capacity are available in the State of Tennessee, SEM model based on CFA can be utilized to estimate the latent pavement condition further in the future.

CFA is the measurement model of SEM. It analyzes the relationship between the observed variables (e.g., the pavement distresses) and latent variables (e.g., latent PSI). Through CFA analysis, the factor loadings of observed variables on the latent variables can be estimated to determine the statistically significant indicators and determine the strong indicators of latent variables. Meanwhile, the relationships among latent variables can also be estimated. CFA is driven by prior knowledge so that it can verify whether the model is viable. The basic function of CFA is shown as Equation (1), in which variable Y is the observed measurement, η is the latent variable, ε is the error term and Λ_y is the factor loading of η on Y. Equation (2) and FIGURE 1 illustrates an example of a typical two factors CFA model.

$$Y = \Lambda_y \eta + \varepsilon \tag{17}$$

$$\begin{bmatrix} y_1 & 0 \\ y_2 & 0 \\ y_3 & 0 \\ y_4 & 0 \\ 0 & y_5 \\ 0 & y_6 \\ 0 & y_7 \\ 0 & y_8 \end{bmatrix} = \begin{bmatrix} \lambda_{y11} & 0 \\ \lambda_{y21} & 0 \\ \lambda_{y31} & 0 \\ \lambda_{y41} & 0 \\ 0 & \lambda_{y52} \\ 0 & \lambda_{y62} \\ 0 & \lambda_{y72} \\ 0 & \lambda_{y82} \end{bmatrix} \begin{bmatrix} \eta_1 & 0 \\ 0 & \eta_2 \end{bmatrix} + \begin{bmatrix} \varepsilon_1 & 0 \\ \varepsilon_2 & 0 \\ \varepsilon_3 & 0 \\ \varepsilon_4 & 0 \\ 0 & \varepsilon_5 \\ 0 & \varepsilon_6 \\ 0 & \varepsilon_7 \\ 0 & \varepsilon_8 \end{bmatrix} \tag{18}$$

Where η_1 and η_2 are the latent variables; $y_1, y_2, \cdots, y_8$ are the observed variables; y_1, y_2, y_3, y_4 are the indictors of η_1 and y_5, y_6, y_7, y_8 are the indictors of η_2; λ_{yij} is the factor loading of indicator y_i on latent variable η_j; $\varepsilon_1, \varepsilon_2, \cdots, \varepsilon_8$ are the error terms. In FIGURE 1, the ellipses represent the latent variables and rectangles represent the observed variables. The single-headed arrows indicate the factor loadings of the observed variables on the latent variables. The double-headed arrows mean that the two variable are correlated with each other and the parameters on the arrows are the correlation coefficients. In this study, the observed variables Y included IRI, rutting, transverse cracking, fatigue cracking, non-wheel path longitudinal cracking, longitudinal lane joint, longitudinal cracking in wheel path, block cracking, raveling and patching. The latent variable η was one overall condition index in the single factor model and the three condition indices representing ride quality, early age cracking, and severe pavement deterioration in the multiple factors model.

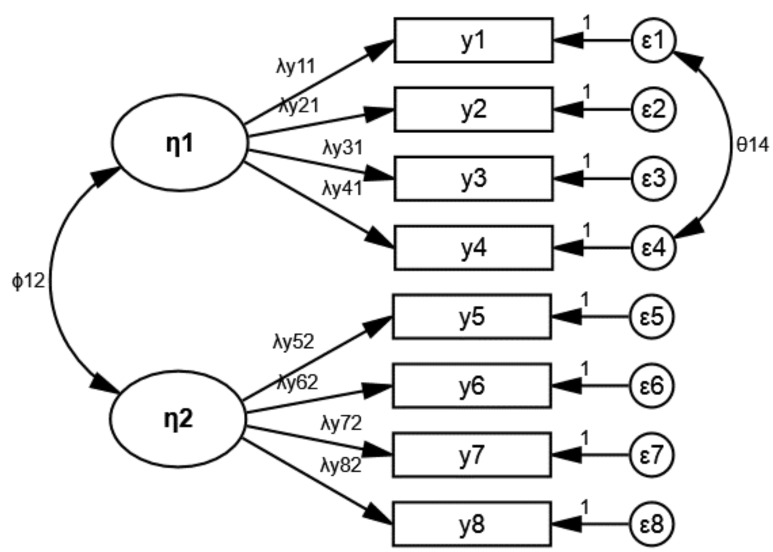


FIGURE 43 Illustration of CFA model

CFA generally includes the process of model specification, model identification, model estimation, model evaluation and model modification. Model specification is driven by previous theory or research or empirical evidences. For instance, research shows that person’s anxiety, hostility, depression and self-consciousness can indicate the degree of neuroticism. Thus, the four measured variables are the indicators of neuroticism (Brown, 2014). In order to make sure that the model can be over identified or at least just identified, the degrees of freedom (DF) should be greater than or equal to zero (that is $DF \geq 0$).

Unlike the general multiple regression analysis, the parameter estimation of SEM or CFA is to minimize the discrepancy between the sample variance/covariance and the variance/covariance estimated by the model. The common used estimation methods include maximum likelihood (ML), generalized least squares (GLS), asymptotically distribution free (ADF), robust maximum likelihood (MLR). One of the assumptions of ML and GLS method is that the distribution of the observed variables is multivariate normal, while for ADF and MLR, the data is not constrained to be normally distributed. When the data sample size is large, ADF can be applied for parameter estimation. MLR is suitable for estimation when the sample is small or medium-sized. Several statistics software packages have been developed for SEM/CFA analysis, including Amos (Analysis of moment structure) (Arbuckle, 2013), Mplus (Muthén and Muthén, 2012) and LISREL (Linear structural relationships) (Jöreskog and Sörbom, 1996), etc. Among those software, Amos provides a more user-friendly interface by allowing users to create and edit the graphical model. As can be seen from TABLE 39 and TABLE 40, the skewness and kurtosis of the observed variables are very high indicating that they are not normally distributed. On the other hand, the sample size in this study was great with the number of 68488. Therefore, Amos was utilized to establish the CFA model and ADF method in Amos was employed to estimate the parameters.

Amos outputs many model fit indicators to evaluate the fit goodness. The root mean square error of approximation (RMSEA), 90% confidence interval of RMSEA, goodness of fit index (GFI), comparative fit index (CFI) and Tucker-Lewis index (TLI) were chosen to evaluate the model fit goodness. Generally, the smaller RMSEA, the better the model fits; the greater GFI, CFI and TLI, the better the model fits. If RMSEA ≤ 0.05, it indicates that the model is closely fitted. If $0.05 <$ RMSEA ≤ 0.08, it indicates that the model is fairly fitted. If RMSEA > 0.08, it indicates that the model is not fitted very well. If the values of GFI, CFI and TLI are greater than 0.9, it indicates that the model is fitted well.

If the model is not well-fitted, it should be modified according to the modification index (MI). The modification method includes freeing the fixed or constrained parameters. For instance, the error terms of two observed variables can be set as correlated to improve fit goodness. The one with greatest MI should be modified first. One prerequisite of model respecification is that the modification is based on prior theory or research (Brown, 2014).

10.2.2 Exploratory Factor Analysis (EFA)

In applied research, EFA is often used in the previous phase of CFA to determine the appropriate number of common factors and to find out the reasonable indicators of the latent variables (Brown, 2014). EFA was initially developed to describe variability among observed variables in terms of lower number of unobserved factors. For example, only two factors may account for the common variance of 8 observed variables. Equation (3) shows a typical EFA model, where X is a matrix of the common factors.

$$Y = \beta X + \varepsilon \tag{19}$$

Similar with CFA, EFA also analyzes the variance/covariance of the indicators. However, EFA is driven by the data and no specifications are made for EFA such as the initial number of latent variables and the relationship between the factors and indicators. The general processes of EFA include factor extraction, factor selection, factor rotation and factor interpretation and evaluation. Large communality shared by the observed variables indicates that they are influenced by the same latent variable. Many statistics software packages can be used for EFA analysis such as SPSS, SAS and Mplus. JMP Pro 12 was employed in this study. Based on the prior knowledge and the factors have been explored and refined by EFA, CFA can then be used to estimate the correlations between the latent variable and observed variables.

10.2.3 Interpretation of Parameter Estimation

The factor loading explains the correlation between the observed variables and latent variables. The observed variables with factor loadings equal to or greater than 0.3 or 0.4 are often treated as salient or strong indicators (Brown, 2014, Wang and Wang, 2012). Also, the P-value of factor loading is lower than the significance level α (usually α=0.05) indicating that the observed variable is significantly related to its corresponding latent variable at the significance level and it cannot be eliminated from the CFA model. The squared factor loading (R^2) indicates how much the observed variable can be explained by the latent variable. Thus, the higher the factor loading, the more the indicators can be explained by the factor.

10.3 Data Collection

The observed data used in this study were exported from the PMS of TDOT, including IRI, rutting depth, transverse cracking, fatigue cracking, non-wheel path longitudinal cracking, longitudinal lane joint cracking, longitudinal cracking in wheel path, block cracking, raveling and patching. All of the condition measurements indicate pavement deterioration from different aspects. They are mainly caused by repeated traffic loading, wheel abrasion, environmental effects, asphalt aging, hardening or adhesion loss, etc. Brief descriptions of these data are shown as below.

1) IRI is the most commonly used pavement roughness index. It is the accumulated irregularity obtained from longitudinal road profiler and usually considered to be directly related to ride quality.
2) Rutting is the longitudinal depression in the wheel path caused by deformation of middle and lower layer, compaction or shear failure of asphalt mixture, etc. (Miller and Bellinger, 2014).
3) Transverse cracking is the cracking perpendicular to the pavement centerline and is usually considered as thermal cracking caused by acute temperature change.

4) Fatigue cracking is the bottom-up cracking that occurs in the areas of repeated traffic loadings (wheel paths).
5) Non-wheel path longitudinal cracking is the longitudinal cracking that is not located in the wheel path.
6) Longitudinal lane joint refers to the cracking or raveling failure occurred at the interface between two adjacent and parallel asphalt mixture lanes and is mainly caused by poor construction.
7) Longitudinal cracking in wheel path is the longitudinal cracking within the wheel path. It is usually regarded as an early-age fatigue cracking and could evolve into fatigue cracking.
8) Block cracking is the cracking that divides the pavement into approximately rectangular pieces. Block cracking is generally not load associated. It is caused by the significant asphalt hardening. Larger blocks are generally classified separately as longitudinal and transverse cracking.
9) Raveling is the wearing away of pavement surface which is caused by dislodging of aggregate particles, loss of asphalt binder and abrasion of wheels (Miller and Bellinger, 2014).
10) Patching is the surface area where the pavement has been removed and replaced with a new material, which is an indication of fatigue cracking or potholes. In fact, a pothole forms when interconnected fatigue cracks cause the pavement surface to break into small pieces which are pulled up by traveling wheels (Dong, Huang and Zhao, 2014).

The pavement condition survey frequency is once a year for Interstates and biennial for state routes. The data are recorded every 0.1 mile. Among the data listed above, IRI and rutting are obtained from a laser profiler while the other items are obtained through pavement surface digital image processing. In the PMS, IRI and rutting data are their actual value, while other distresses are recorded as the percentages whose value range from 0 to 100. For transverse cracking, each transverse crack counts 1%. If the total number of transverse cracks is equal to or greater than 100, the value of transverse cracking is recorded as 100%. For other distresses, their values are the percentages of total length, area, or affected area to the total pavement segment length or area. What to be noted is that the total area of fatigue cracking is the two -wheel path area instead of the lane area. The data analyzed in this study is from the year of 2014. The data with IRI greater than 5 m/km (317 inch/mile) or rutting depth higher than 12.7 mm (0.5 inch) were considered outliers (Shafizadeh and Mannering, 2003). The sample size was 68,488 after deleting the outliers, including 11,397 Interstates samples and 57,091 state routes samples. The descriptive statistics of the observed data are summarized in TABLE 39 and TABLE 40.

TABLE 39 Descriptive Statistics of Observed Variables for State routes

Variable	Description	Min.	Max.	Mean	Std Dev	Skewness	Kurtosis
IRI	International roughness index (m/km)	0.35	5.00	1.29	0.62	1.90	5.25
RUT	Rutting depth (mm)	0	12.65	2.46	1.24	1.61	4.27
FATG	Fatigue cracking (%)	0	100	6.87	16.54	3.64	14.52

TRAN	Transverse cracking (Count)	0	87	3.11	5.88	2.90	11.66
LWP	Longitudinal cracking in wheel path (%)	0	100	4.28	7.37	4.96	41.80
LNWP	Non-wheel path longitudinal cracking (%)	0	100	6.14	9.35	3.73	21.89
BLOCK	Block cracking (%)	0	100	2.29	8.85	5.20	31.07
LLJ	Longitudinal lane joint cracking (%)	0	100	2.60	11.18	5.29	30.35
RAVEL	Raveling (%)	0	100	0.68	5.09	11.45	161.98
PATCH	Patching (%)	0	81	0.24	2.23	13.71	230.02

TABLE 40 Descriptive Statistics of Observed Variables for Interstates

Variable	Description	Min.	Max.	Mean	Std Dev	Skewness	Kurtosis
IRI	International roughness index (m/km)	0.30	4.35	0.76	0.45	2.67	9.41
RUT	Rutting depth (mm)	0.71	12.7	2.58	1.11	1.67	5.53
FATG	Fatigue cracking (%)	0	100	1.83	8.31	7.78	73.05
TRAN	Transverse cracking (Count)	0	36	0.55	2.22	6.07	44.66
LWP	Longitudinal cracking in wheel path (%)	0	99	1.64	4.04	4.81	48.68
LNWP	Non-wheel path longitudinal cracking (%)	0	95	6.00	8.70	2.23	6.64
BLOCK	Block cracking (%)	0	83	1.03	5.44	8.00	77.30
LLJ	Longitudinal lane joint cracking (%)	0	99	1.83	9.42	6.33	44.11
RAVEL	Raveling (%)	0	100	0.23	3.69	21.83	535.15
PATCH	Patching (%)	0	47	0.11	1.40	20.06	481.37

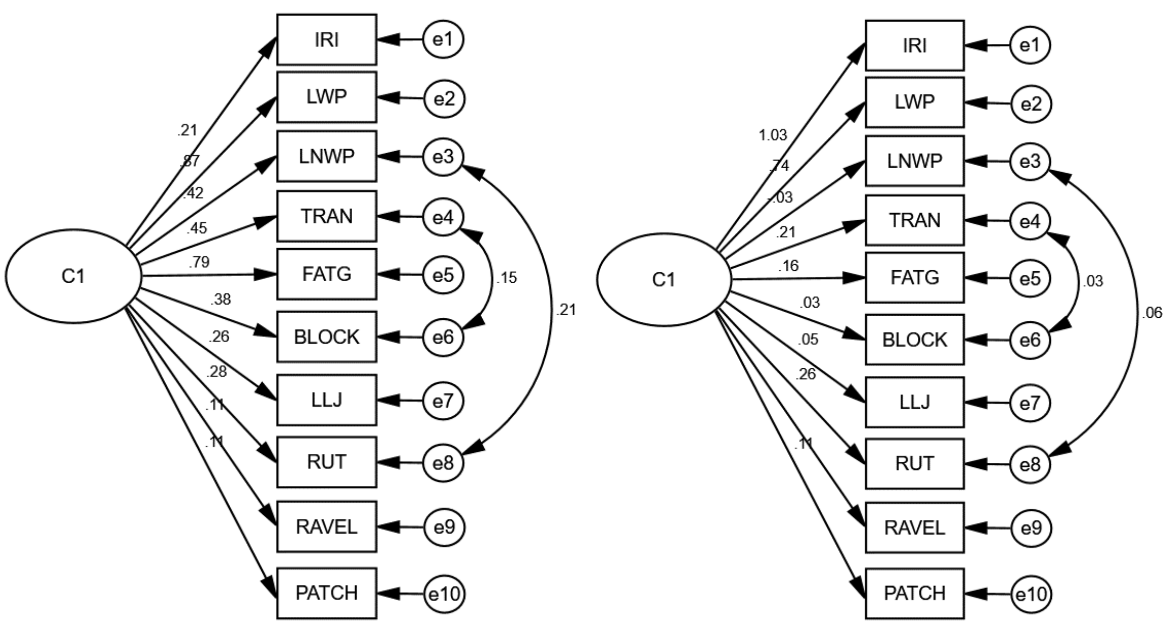
10.4 Discussion of Results

Two CFA analyses, the single factor and multiple factors model respectively, were conducted. The single factor CFA model used only one latent condition factor, whereas the multiple factor CFA model used multiple latent condition indices, aiming to represent different aspects of condition. An EFA was carried out to provide prior theory basis on determining number of multiple latent factors before conducting the multiple factor CFA analysis.

10.4.1 Single Factor Model

All the measured variables listed in TABLE 39 and TABLE 40 including IRI, rutting depth, transverse cracking, fatigue cracking, non-wheel path longitudinal cracking, longitudinal lane joint, longitudinal cracking in wheel path, block cracking, raveling and patching were considered as the indicators of the single and comprehensive condition factor for state routes and Interstates as shown in Fig. 2 (a) and (b), respectively. Based on prior knowledge and modification indices, the error terms e3 and e8 and e4 and e6 were set as correlated as shown in Fig. 2. The standardized results are shown in Fig. 2 and TABLE 2. The RMSEA for the single factor model of state routes is 0.042 with the 90% confidence interval (0.041, 0.044) which is smaller than 0.05 and GFI=0.888 <0.90. While for the Interstates, the RMSEA is 0.043 with the 90% confidence interval (0.040, 0.045) which is also less than 0.05 and GFI=0.841 <0.9. The model fit indicator RMSEA indicates that the model fits very well while GFI shows that the model does not fit very well.

It is noted that although Amos cannot calculate the significance test for standardized parameter estimates, their values are very close to the significance test results of unstandardized parameter estimates (Brown, 2014). Therefore, the P-values of unstandardized parameter estimates are presented in TABLE 2. As can be seen from TABLE 2 that all the variables for the state routes are significant indicators at the significance level α=0.05. The strong indicators were variable LWP, LNWP, TRAN, FATG and BLOCK with the factor loadings of 0.874, 0.417, 0.452, 0.787 and 0.385, respectively. The squared factor loading is the portion of the observed variable that can be explained by the latent variable. For instance, 76% (0.874^2) of longitudinal cracking in wheel paths can be explained by latent variable C1. However, for the model of Interstates, variable RAVEL is not significant with the P-value greater than the significance level α=0.05. Besides, the factor loading of variable LNWP is -0.029 indicating that with the longitudinal cracking not in wheel path increasing, the pavement condition would get better. It can be seen that this result is not reasonable. In order to build a more reasonable and appropriate mode and improve the model fit goodness, the multiple factors CFA model utilized and discussed as following.



(a) State routes (b) Interstates

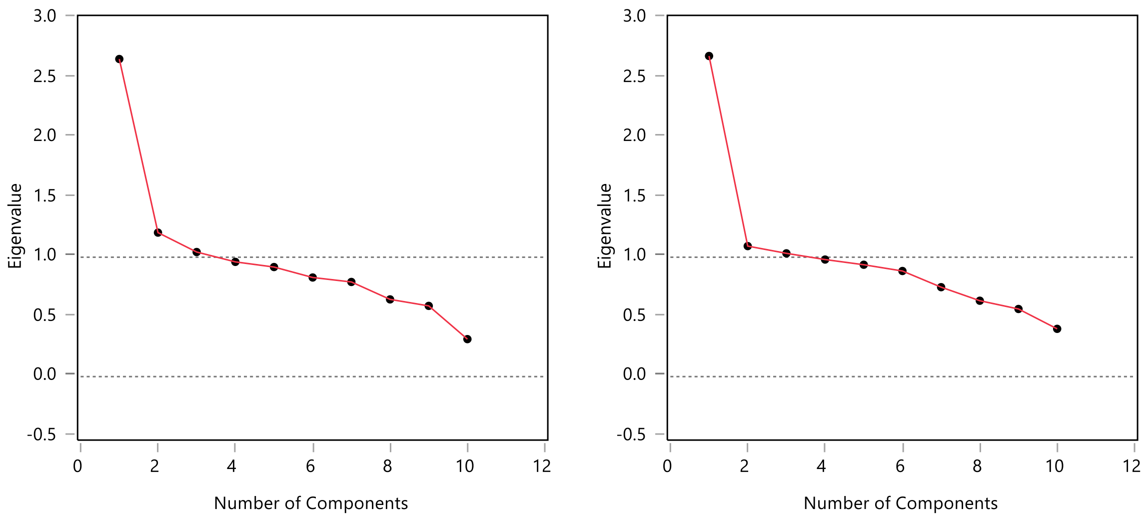
FIGURE 44 CFA model of single factor

TABLE 41 Parameter Estimation of the CFA model of single factor

Variables	State routes		Interstates	
	Estimate	P-value	Estimate	P-value
IRI	0.213	-	1.035	-
LWP	0.874	<0.001	0.735	<0.001
LNWP	0.417	<0.001	-0.029	0.010
TRAN	0.452	<0.001	0.208	<0.001
FATG	0.787	<0.001	0.163	<0.001
BLOCK	0.385	<0.001	0.034	0.080
LLJ	0.259	<0.001	0.049	0.030
RUT	0.285	<0.001	0.257	<0.001
RAVEL	0.112	<0.001	0.000	0.433
PATCH	0.111	<0.001	0.106	0.001

10.4.2 Multiple Factors Model

Before CFA was employed to analyze the relationship among variables, EFA was used to determine the number of latent variables. Fig. 3 is the scree plot of EFA for both state routes and Interstates. In Fig. 3, three eigenvalues exceed 1.0 indicating that three factors are necessary for both the models of state routes and Interstates. For each variable, the rotated factor loading regarding to the three factors was calculated and shown in TABLE 4. The greater the rotated factor loading regarding to some factor indicates the higher probability that this variable belongs to the factor. Among those variables, IRI is the only available roughness indicator that reflects pavement smoothness and ride quality. On the other hand, although IRI is not essentially a direct measurement like cracking length, the assumptions and algorithm to calculate IRI have been optimized and standardized (Sayers, 1995, Sayers and Karamihas, 1996). Thus, IRI was treated as the independent factor different with other two factors as shown in TABLE 4 and Fig. 4. Other pavement distress variables such as variable LWP, LNWP and TRAN were the indicators of factor C2. Variable FATG and BLOCK were the indicators of factor C3. Although variable LLJ, RUT and RAVEL were not highly correlated with factor C3 (the loadings were smaller than 0.3), they were more correlated with C3 than C2. PATCH was also the indicator of C3 in this study because C3 was the latent variable representing severe pavement deterioration. The results obtained by using four and even more factors are no better than the result of three factors. Therefore, LLJ, RUT, RAVEL and PATCH were also considered as the indicators of C3 as shown in Fig.4. Generally, latent factor C1 was IRI, a direct indicator of ride quality. Latent factor C2 were related to the longitudinal cracking in wheel path, non-wheel path longitudinal cracking and transverse cracking which indicated the aging and hardening of asphalt or weather effects. The three distresses of C2 can occur in the early life of pavement surface. Latent factor C3 related to severe asphalt pavement cracking, material deterioration due to compaction, shear or adhesion loss, the causes of which involved repeated traffic loading and wheel abrasion.

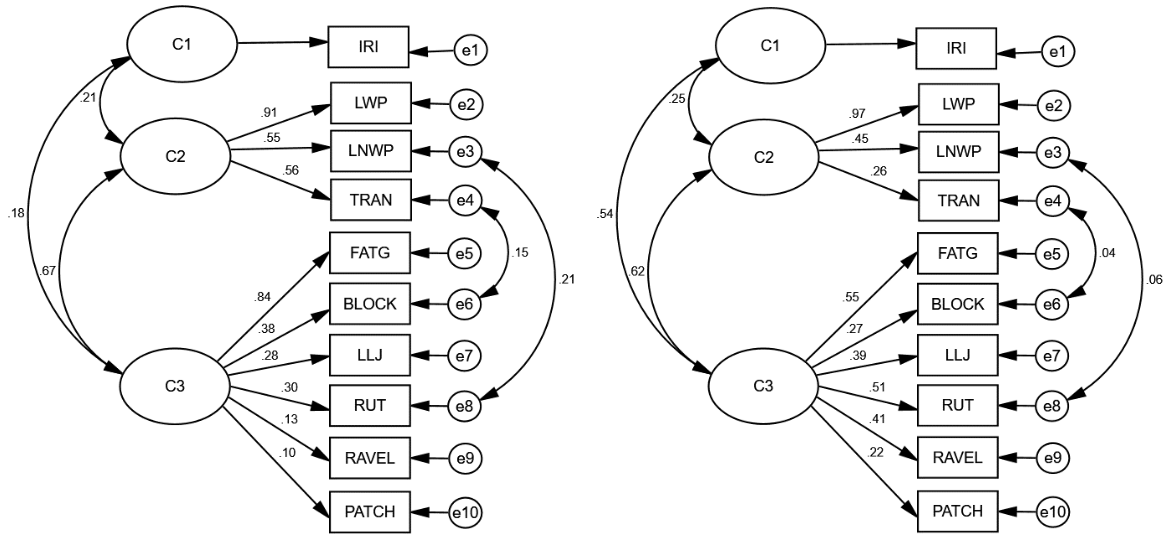


(a) State routes (b) Interstates

FIGURE 45 Scree plot of exploratory factor analysis

TABLE 42 Rotated Factor Loading of Exploratory Factor Analysis

Variable	State routes			Interstates		
	Factor 1	Factor 2	Factor 3	Factor 1	Factor 2	Factor 3
IRI	**0.994**	0.056	0.090	**0.989**	0.070	0.127
LWP	0.091	**0.980**	0.175	0.167	**0.970**	0.178
LNWP	0.008	**0.581**	0.201	0.036	**0.415**	0.175
TRAN	0.109	**0.413**	0.299	0.219	**0.327**	0.133
FATG	0.074	**0.379**	**0.548**	0.180	**0.380**	**0.481**
BLOCK	0.007	0.120	**0.471**	0.052	0.150	**0.491**
LLJ	0.040	0.073	**0.324**	0.108	0.109	**0.320**
RUT	0.139	0.097	**0.314**	0.287	0.158	0.278
RAVEL	0.101	0.064	0.098	0.046	0.204	0.130
PATCH	0.194	0.007	0.024	0.169	0.044	0.039



(a) State routes (b) Interstates

FIGURE 46 CFA model of multiple factors

TABLE 43 Parameter Estimation of the CFA model of multiple factors

Factor	Variables	State routes		Interstates	
		Estimate	P-value	Estimate	P-value
C1	IRI	**1.000**	-	**1.000**	-
C2	LWP	**0.906**	-	**0.967**	-
	LNWP	**0.551**	<0.001	**0.449**	<0.001
	TRAN	**0.556**	<0.001	0.257	<0.001
C3	FATG	**0.839**	-	**0.548**	-
	BLOCK	**0.376**	<0.001	0.273	<0.001
	LLJ	0.277	<0.001	**0.388**	<0.001
	RUT	0.296	<0.001	**0.505**	<0.001
	RAVEL	0.127	<0.001	**0.410**	<0.001
	PATCH	0.103	<0.001	0.223	0.004

Since IRI was the only and perfect indicator of C1, the variance of its error term ε_1 was set as zero so that the model can be identified. The standardized parameter estimation result is shown in FIGURE 46 and TABLE 43. For the model of state routes, the fit indicator RSMEA= 0.037 < 0.05, its 90% confidence interval was (0.036, 0.039) and GFI=0.917 indicating that the model fitted very well. For the model of Interstates, the fit indicator RSMEA= 0.043 < 0.05, its 90% confidence interval was (0.040, 0.046) and GFI=0.847. Compared with the fitness indicators of the single factor model described above, it can be seen that the RSMEA of multiple factors model was less and the GFI was greater for the state routes. FIGURE 46 and TABLE 43 show that all the variables are significant and the factor loadings are positive which indicates that the multiple factors model for Interstates is also better than the single factor model. Therefore, it can be concluded that the multiple factors models were better than the single factor model. The correlations between the latent factors are shown in FIGURE 46. For instance, the correlation between C1 and C2, C1 and C3, C2 and C3 for the state routes were 0.21, 0.18 and 0.67, respectively. All the indicators were statistically significant when the significance level equaled 0.05. The indicators with the factor loadings greater than 0.3 are the strong indicators. Therefore, the strong indicators of state routes were IRI, LWP, LNWP, TRAN, FATG and BLOCK. The strong indicators of Interstates were variable IRI, LWP, LNWP, FATG, LLJ, RUT and RAVEL. The indicators with higher factor loadings indicated that they had more influence on pavement condition and their deducted values or weight coefficients should be greater during the pavement condition evaluation procedure. For instance, the factor loadings of longitudinal cracking in wheel path and fatigue cracking were greater than that of raveling as can be seen from TABLE 43. It meant that the deducted values or weight coefficients of longitudinal cracking in wheel path and fatigue cracking were greater than that of raveling. The results were consistent with common practice and previous research that the load-related distresses weigh more than surface distresses while evaluating the overall pavement condition.

10.4.3 Comparison Between the Models of State routes and Interstates

FIGURE 47(a) shows the average performance or distresses values for state routes and Interstates. It can be seen that LNWP and RUT are comparable for state routes and Interstates. While for the other condition measurements, the values of state routes are greater than those of Interstates, indicating that the overall condition of state routes is worse than Interstates. Among those condition measurements, IRI is the indicator of C1 which represents the pavement ride quality; LWP and TRAN are the indicators of C2 which is an early age cracking factor; FATG, BLOCK, LLJ, RAVEL and PATCH are the indicators of C3 which is a factor indicating severe pavement deterioration.

FIGURE 47(b) shows the comparison between the factor loadings of state routes and Interstates. The factor loadings of IRI are identical since they were set as 1.0 during the establishment of the multiple factor models. Although the LWP of state routes is more severe than that of Interstates, the factor loading of LWP for state routes is lower. The factor loadings of LNWP, TRAN, FATG and BLOCK for state routes are greater than those of Interstates, while for the other indicators including LLJ, RUT, RAVEL and PATCH, the factor loading for state routes are lower.

The difference of the factor loadings between state routes and Interstates is due to the different condition levels of the two types of roads. State routes generally have lower average condition and may have already encountered certain amount of distress. Therefore, compared with LLJ, RUT, RAVEL and PATCH, more severe or late age distresses such as LNWP, TRAN, FATG and BLOCK have higher impacts on the condition level of state routes. Those cracks are associated with severe fatigue failure or asphalt aging. While for Interstates with a higher condition level requirement, only few distress have been encountered and no severe distress is allowed. Those less severe or early age distresses such as LLJ, RUT, RAVEL and PATCH are more important to the condition. The CFA results prove that relative importance of condition measurements vary for different types of roads, as discussed in the beginning of this paper. This principle is valid in other field. For example, a healthy person may only need to pay attention to his/her body weight to monitor the health condition while a severely sick person need to exam the proper functioning of major organs.

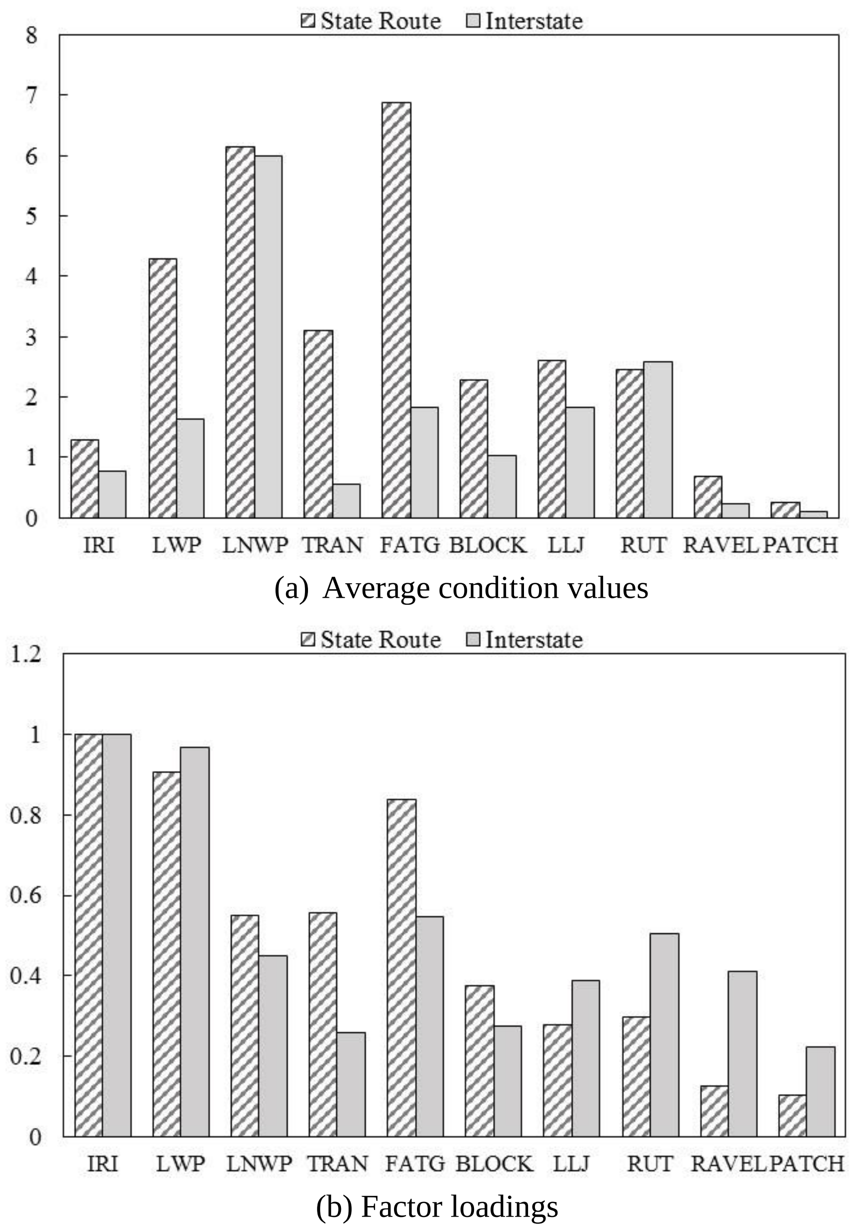


FIGURE 47 Comparison between state routes and Interstates

10.5 Concluions

In this study, CFA was utilized to analyze the contributions of condition measurements on the latent comprehensive pavement condition and the relative importance of the condition measurements. The pavement condition was considered as the latent variable (factor), while the condition measurements were the observed variables. Both single and multiple factors modeling were conducted. For the multiple factors analysis, EFA was employed to determine the number of factors and their corresponding indicators.

Analysis of results revealed that the multiple factor model provided better model fit goodness than the single factor model. All the observed variables were statistically significant. Among those indicators, IRI, longitudinal cracking in wheel path, non-wheel path longitudinal cracking, transverse cracking, fatigue cracking and block cracking were strong indicators with factor loadings higher than 0.3. The multiple factors model suggested three latent factors, a ride quality related factor, an early age cracking factor and a factor indicating severe pavement deterioration. The correlations among the three factors were 0.225, 0.222 and 0.684, respectively.

Comparison between the factor loadings of state routes and Interstates indicated that the more severe distresses such as fatigue and transverse cracking have higher contributions to the latent pavement condition of state routes while less severe distresses such as raveling and patch have higher contributions to Interstates. Since state routes have relatively lower condition level and may have already encountered certain amount of distress, the severe distresses have higher impacts on their condition. Whereas Interstates are maintained at higher condition level, the less severe distresses could impact its overall condition. Therefore, it is rational to consider the relative importance of condition measurements when evaluating different types of roads.

The intercorrelations between pavement condition and its measurements evaluated in this study provide guidelines to determine the deducted values and weights for calculation of the overall pavement condition index in the future. When the traffic, environment and pavement characteristics are available for building the structural model, SEM combined with multiple regression method can be utilized to estimate the weight coefficients of the overall pavement condition index in the further study. Furthermore, it will be interesting to investigate the possibility of using different weights for roads according to their functional classes or using dynamic weights to calculate pavement condition index.

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11 Effects of Construction Factors on Slurry Seal Performance

11.1 Background

Due to limited budget and increasing traffic volume, transportation agencies are under the pressure to seek more cost-effective ways to maintain existing pavements. Pavement preventive maintenance such as thin asphalt overlay, slurry seals, micro surfacing, chip seals and etc. has been widely used to maintain sufficient pavement serviceability at a relatively lower cost. The increasing interest in cost-effective preventive maintenance has heightened the need to evaluate performance of those treatments to optimize their application.

Researchers usually investigate the change of pavement condition indicated as roughness, distresses, or friction caused by the application of treatments. The long-term effectiveness, such as the average pavement performance (Labi, Lamptey and Kong, 2007, Labi and Sinha, 2004, Wang and Wang, 2013, Chen, Lin and Luo, 2003, Labi, Lamptey, Konduri and Sinha, 2005) and the area between the performance curve and lower performance threshold (Dong, Huang, Richards and Yan, 2013, Labi, Lamptey and Kong, 2007, Labi and Sinha, 2004, Shu, Huang, Wu, Dong and Burdette, 2011, Chen, Zhu, Dong and Huang, 2016), are more preferred since they consider both performance improvement and the deterioration of new pavement surface. Previous studies also found that the performance of preventive maintenance is highly dependent on many factors, such as the original pavement condition, traffic, environment, and materials (Rajagopal and George, 1990, Wang, 2012, Hanna, Tayabji and Miller, 1994, Dong, Dong and Huang, 2015, Dong and Huang, 2012, Dong and Huang, 2015, Dong, Huang and Jia, 2014, Dong, Jiang, Huang and Richards, 2013, Gong, Dong, Huang and Jia, 2015, Gransberg, 2010). Although considerable research has been devoted to the performance evaluation of pavement preventive maintenance, rather less attention has been paid to the influence of materials or construction practices. Therefore, it is of great interest to investigate the effects of those construction factors to help optimizing the application of pavement preventive maintenance.

11.1.1 Slurry Seal

Originated in 1930s in Germany, slurry seal is a mixture of well-graded fine aggregate, mineral filler and asphalt emulsion(Gransberg, 2010, ISSA, 2010). Slurry seal is applied in a mono-layer, namely one stone thickness. Polymer or other special purpose additive can be added to enhance the bond between the asphalt and the aggregate, and to improve durability and toughness of the seal. It is noted that micro surfacing, also called polymer modified slurry seal, is a very similar asphalt seal coat, but can be placed in layers thicker than one stone deep and always contains polymers and cures rapidly through chemical reaction(Gransberg, 2010). Micro surfacing uses coarse and better aggregate and can be applied for roads with higher traffic levels.

Slurry seal can effectively seal low-severity surface cracks and oxidized pavement surface, repair low level rutting, cracking and raveling, and improve skid resistance at speeds below 64 km/h (30 mph). However, it is not recommended on pavement with structural failure such as significant fatigue cracking or with high level thermal cracks(Peshkin, Hoerner and Zimmerman, 2004). Due to its mono-layer nature, slurry seals cannot address deep rutting problems. It follows the old pavement surface profile and thus does not significantly change the pavement's roughness(Gransberg, 2010).

11.1.2 Previous Studies

Many State Departments of Transportation (DOTs) have developed application guides on slurry seals, including specific requirements on the materials, projects section, and construction practices. For example, residual asphalt and workability are considered to select proper asphalt rate. Gradation, specific gravity and surface texture of aggregates need to be considered to determine aggregate rates. Mineral fillers or additives are added to improve consistency and promote cohesion by forming a mortar with residual asphalt. It also provides control of the set time because fillers can absorb water from the emulsion to make it break faster and some fillers such as cement can counteract the emulsifier ions, resulting in a fast-break mix(Gransberg, 2010).

NCHRP synthesis 411 conducted a survey and an extensive literature review on the construction practices of the polymer modified slurry seal (micro surfacing), focusing on surface preparations and weather related factors(Gransberg, 2010). Many specifications require completing patching and crack sealing before treatment and sweep pavement surface clean (Gransberg, 2010, NDOR, 2002, NMDOT, 2009). Pre-wet process which involves spraying water to dampen the surface is also recommended. Another common practice is to spray a tack coat, a "bond coat" or a "paint binder" to improve the bond between slurry seal and old pavement surface(Gransberg, 2010, NMDOT, 2009). Air temperature, relative humidity, wind velocity, and precipitation all will impact the constructability and performance of micro surfacing or slurry seal(ISSA, 2010). The ideal weather conditions are those with low humidity, a slight breeze, and with sustained high temperatures into the forthcoming days(Hicks, Seeds and Peshkin, 2000, GDOT, 2001). High humidity retards the breaking of the emulsion in slurry seal(AI, 1988). Some specifications recommend that micro surfacing only be placed if the humidity is no more than 50-80%(Gransberg, 2010, ISSA, 2010). In addition, since emulsions can retain some water for several weeks, freezing temperature should be avoided. Slurry seals can open to traffic shortly after the placement. However, in areas such as intersections where stop and go traffic is prevalent, additional curing time is needed, especially during hot or cold weather(Gransberg, 2010).

Many studies have been conducted to evaluate the service time and performance of pavement preventive maintenance including slurry seals at different traffic levels and weather conditions (Labi, Lamptey and Kong, 2007, Labi and Sinha, 2004, Chen, Lin and Luo, 2003, Wang, Wang and Mastin, 2011, Eltahan, Daleiden and Simpson, 1999, Dong and Huang, 2015, Gong, Dong, Huang and Jia, 2015, Hicks, Seeds and Peshkin, 2000) to help selecting optimized preventive treatments. There have also been established specifications for emulsified asphalt mixtures and laboratory studies on the properties of slurry seals from the aspects of material selection and mixture design (ASTM, 2013, Khan and Wahhab, 1998). For example, the application of emulsified asphalt seal coats is not recommended in winter season. But how will air temperature or relative humidity influence the field performance of emulsified asphalt seal coats is not quantified. To further optimize materials selection and construction techniques, it is of great interest to investigate how materials and construction practices influence the long term field performance of slurry seals.

11.1.3 Research Objectives and Scope

This study aims to evaluate the influence of materials and construction practices on the performance of slurry seals. An equivalent effectiveness based on the "area" under performance curves is developed to measure the long term effectiveness of in-service slurry seal projects in terms of different pavement condition indicators. A statistical data mining tool, the Classification and Regression Tree (CART) method, is employed to identify significant factors and classify slurry seal projects according to their performances. A total of 89 slurry seal projects were collected from the Long Term Pavement Performance (LTPP) program database for the analyses.

11.2 Data Preparation

11.2.1 LTPP Experiments

Initiated in the late 1980s by Federal Highway Administration (FHWA), the Long Term Pavement Performance (LTPP) program aims to provide a large scale pavement experiment to help engineers design, construct and manage pavements. It has collected pavement structure, materials, construction, performance, traffic, and climatic data on more than 2500 test sections in North America and has been extensively explored by many researchers for pavement performance studies (Wang and Wang, 2013, Chen, Lin and Luo, 2003, Wang, Wang and Mastin, 2011, Haider and Chatti, 2009, Hanna, Tayabji and Miller, 1994, Hall, Correa and Simpson, 2003, Chen, Dong, Zhu and Huang, 2016, Shu, Huang, Wu, Dong and Burdette, 2011, Dong and Huang, 2012, Dong and Huang, 2015, Dong, Jiang, Huang and Richards, 2013, Gong, Dong, Huang and Jia, 2015, Haider, Chatti, Buch, Lyles, Pulipaka and Gilliland, 2007). The Specific Pavement Studies-3 (SPS-3) experiment in the LTPP is designed to evaluate the effectiveness of preventive maintenance treatments for flexible pavements (Elkins, Schmalzer, Thompson and Simpson, 2003). Specifically, it contains the details of construction practices for a total of 89 slurry seal projects in the "SPS3_SLURRY" table.

The materials, construction practices, traffic, weather, structure and pavement condition data of the slurry seal projects were exported and summarized in TABLE 44. Material variables include asphalt, aggregate and mineral filler types and application rates, aggregate moisture condition, and additional water ratio added to adjust slurry workability. Construction practice variables include temperatures, humidity, surface condition and time for curing. Pavement condition variables include pre-treatment roughness and structural number. Weather and traffic variables during treatment service life include freeze index, precipitation and kESALs.

TABLE 44 Variables Used for Analysis

Variable	Description	Min.	Max.	Mean	Std Dev
Materials					
AC	Type of Asphalt	CQS, CQS-H, CRS-1, CSS-1h, Ralumac, Ralumaclatex			
AGG	Type of Aggregate	Granite, limestone, sandstone, slag			
MINE	Type of Mineral filler	Aluminum sulfate, cement, latex, or lime.			
ACR	Application rate of asphalt (L/m^2)	0.5	1.8	1.4	0.3
AGGR	Application rate of aggregate (kg/m^2)	7.7	17.1	11.3	2
MINER	Application rate of mineral fillers (kg/m^2)	0	1	0.4	0.5
SLUR	Actual application rate of the slurry mixture (kg/m^2)	9.2	18.8	13	2.7
AGGMO	The moisture content of aggregate (%)	0.2	6.7	2.8	1.7
WAT	Water ratio (gal water/gal emuls)	0.5	1.4	0.7	0.2
Temperature and humidity					
TEMSL	Temperature of slurry material at discharge (°C)	10	33.3	27.7	4.3
TEMPA	Temperature of pavement at time of sealing (°C)	16.1	51.7	37.4	7.8
TEMAI	Air temperature (°C)	16.7	42.2	29.8	5.2
HUMID	Relative humidity (°C)	12	100	50.2	14.6
Curing					
MAXSP	Maximum speed in initial curing period (mph)	20	55	34.6	6.7
TIMBO	Time open to reduced speed traffic (h)	1	4.5	2.4	0.9
TIMBF	Time open to full speed traffic (h)	1	10.5	3.4	1.3
Surface condition					
SURPR	The method of surface preparation	Sweep, none, blower or already clean			
PAVCO	Pavement surface condition	Clean or mostly clean			
SURMO	Pavement surface moisture condition	dry or mostly dry			
CRALE	Crack severity level on old pavement	High, moderate or low			
CRATY	Primary type of cracks on old pavement	Alligator, Block, Edge, Longitudinal, Raveling, Transverse			
Pavement condition and structure					
IRIP	Roughness level before treatment (m/km)	0.6	3.6	1.3	0.6
SN	Structural Number	1.3	8.3	4.1	1.4
Weather and traffic level during service					
PERP	Annual average perception (cm)	397.9	4394.2	2084.8	1082.5
FI	Annual freeze Index (°C-days)	0	1746.2	389.3	447.4
kESAL	Kilo equivalent single axle load	5.2	1048.7	135.6	172.2
Equivalent long term effectiveness (Targets)					
IRI	Effectiveness of international roughness index	0	30	9.5	4.4
FN	Effectiveness of friction number	112	441.6	309.9	59
RUT	Effectiveness of rutting depth	9.5	158	52.9	36.8

AL	Effectiveness of alligator cracking	0.6	2520.4	290.5	481.8
LW	Effectiveness of wheel path longitudinal crack	0	428.8	88.2	107.8
LNW	Effectiveness of non-wheel path longitudinal crack	0	1600	347.6	454
TL	Effectiveness of transverse crack	0	1093.6	249.8	257.3

11.2.2 Pavement Condition Indicators

As a preventive maintenance, slurry seals can seal low-severity cracks and oxidized pavement surface, repair low level rutting, cracking and raveling, and improve skid resistance. The pavement condition indicators investigated in this study include roughness, friction resistance, rutting depth, fatigue cracking, wheel path longitudinal cracking, non-wheel path longitudinal cracking, and transverse cracking. It is noted that high friction number indicates good friction resistance, whereas for the other six pavement condition indicators, lower values mean better pavement condition. Since no bleeding, shoving, polishing or raveling distress was observed and very few patches and potholes were encountered on those slurry seal projects, those distresses were not considered in this study.

1. Roughness is the longitudinal regulations along pavement surface, which directly affects riding quality.
2. Friction resistance is influenced by the aggregate, asphalt rate and even environmental conditions. The loss of friction is caused by raveling and bleeding for sealing coats.
3. Rutting is usually the consolidation or/and a plastic flow of asphalt mixture under wheel loads. For slurry seals, the deformation could be caused by the mixtures underneath the seals.
4. Alligator cracks are bottom-up fatigue cracks from bottom layer indicating severe fatigue failure of pavement surface.
5. Wheel path longitudinal cracks are the typical surface-down fatigue cracks at the edge of wheel path. It is usually caused by the large tensile stress on surface and the shearing of surface mixture.
6. Non-wheel path longitudinal cracks are parallel to pavement centerline but not on the wheel path. It could be caused by poor joint construction, thermal cracking, or aging of asphalt materials.
7. Transverse cracks are usually thermal cracks caused by rapid temperature drops. It also indicates the aging and hardening of asphalt.

11.2.3 Equivalent Effectiveness

To use the area bounded by the pre and post treatment performance curves requires large amount of pavement condition dataset both before and after the treatment which is hard to obtain for the SPS-3 test sections except for roughness. The area underneath the pavement performance curve was adopted in this study. A drawback with the average pavement performance or the normalized area under IRI over time curve (Labi, Lamptey and Kong, 2007, Labi and Sinha, 2004, Wang and Wang, 2013, Labi, Lamptey, Konduri and Sinha, 2005, Chen, Zhu, Dong and Huang, 2016) is that the calculated average pavement performance changes as the increase of service time. It is questionable to compare treatments with different service times or condition survey period. Therefore, an equivalent effectiveness which converts the effectiveness area to the same service time is developed to evaluate the long-term performance of slurry seal treatments.

As shown in FIGURE 48, the area underneath the pavement condition curve is firstly calculated as the original effectiveness for each slurry seal project. To make the projects with different service times or condition survey periods comparable, the equivalent effectiveness was then calculated by converting the effectiveness of different service times or survey periods to those of the same age. The average service life of the 89 slurry projects was 6.4±3.9 years and was used as the base age to calculate equivalent effectiveness. For pavement condition indices such as roughness or friction number whose initial value after treatment do not equal to zero, Equation (1) is used to calculate their original effectiveness and Equation (2) is utilized to calculate the equivalent effectiveness. For pavement condition indices such as cracks or rutting whose initial values after treatment equal to zero, Equation (3) and (4) are used to calculate their original effectiveness and equivalent effectiveness, respectively. TABLE 44 lists calculated equivalent effectiveness which will be used as dependent variables in the classification tress analyses. It is noted that due to the severe skewness of cracking area or length data, the effectiveness in terms of the four cracks showed high skewness. Values higher than 3 times of the mean were treated as outliers and dropped from the analyses. A total of 6 data points of the four types of cracks were dropped.

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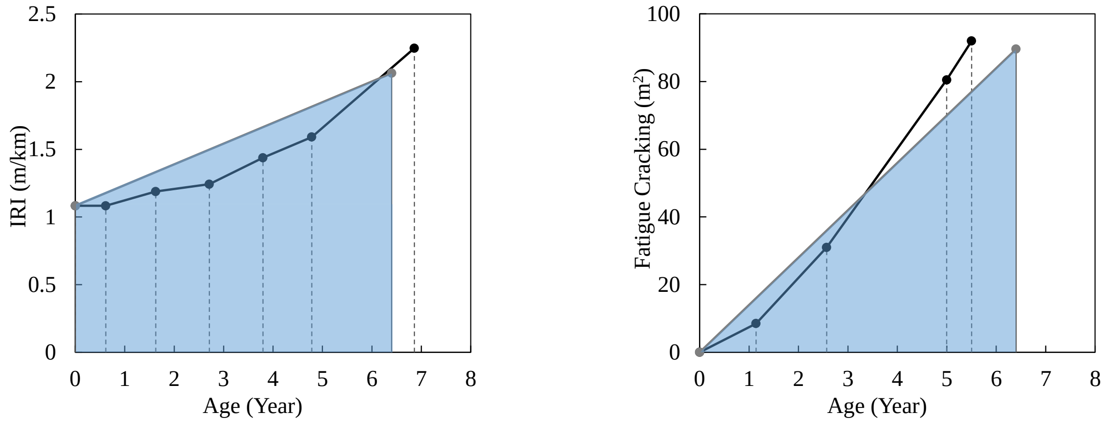
$$Eff_1 = Index_1 Age_1 + \sum_1^{n-1} \frac{(Age_{i+1} - Age_i)(Index_i + Index_{i+1})}{2} \quad (1)$$

$$Eff_{1eq} = \left(\frac{Eff_1 * 2}{Age_n} - Index_n\right) Age_a + \left(Index_n - \frac{Eff_1}{Age_n}\right) \frac{{Age_a}^2}{Age_n} \quad (2)$$

$$Eff_2 = \frac{Index_1 Age_1}{2} + \sum_1^{n-1} \frac{(Age_{i+1} - Age_i)(Index_i + Index_{i+1})}{2} \quad (3)$$

$$Eff_{2eq} = Eff_2 \left(\frac{Age_n}{Age_a}\right)^2 \quad (4)$$

Where, Eff_1 , Eff_2 = original long term effectiveness of pavement treatments; Eff_{1eq} , Eff_{2eq} = equivalent long term effectiveness of pavement treatments; Age_i = pavement treatment age at the ith condition survey; $Index_i$= value of pavement condition indicator at age i; and Age_a = average service life for a treatment used for comparison, which is 6.4 in this study.



(a) Roughness or friction number (b) Cracking or rutting

FIGURE 48 Calculation of effectiveness and the equivalent effectiveness

11.3 Classification and Regression Trees

Statistical methods including multiple linear regression, Poisson regression, logistic regression, and survival analysis, have been employed by previous researchers to explore the influence of factors on pavement performance and maintenance effectiveness (Wang, 2012, Chen, Williams and Mervyn, 2014, Dong and Huang, 2012, Dong and Huang, 2015, Loizos and Karlaftis, 2005). However, those methods require specific distribution of target variables or underlying relationships between the predictors and targets. If these assumptions are violated, the model could lead to erroneous parameter estimation. Decision tree is a widely used data mining tool to statistically split a data set hierarchically according to their association with the target attributes. Decision trees have many advantages in data exploring than traditional statistical regression methods. They do not require assumptions on the distribution of target variables or link function between the predictors and targets. They can handle large number of independent variables, especially categorical variables. They are not sensitive to outliers and tolerant of observations with missing values. Decision tree method has been extensively used for credit scoring, signal recognition, medical diagnosis and etc.(Baesens, Van Gestel, Viaene, Stepanova, Suykens and Vanthienen, 2003, Ravi Kumar and Ravi, 2007, Safavian and Landgrebe, 1991) In transportation and pavement engineering, decision trees have been used to investigate the influencing factors for traffic accident, to classify accidents with different injury level(Yan, Richards and Su, 2010, Clarke, Forsyth and Wright, 1998, Harb, Yan, Radwan and Su, 2009), to select pavement maintenance methods based on its current condition and to evaluate pavement performance *(Dong, Dong and Huang, 2015, Khan and Wahhab, 1998, Kang, Kim and Lee, 2010)*. In one study, the classification and regression tree method was adopted to explore the relationship between pavement roughness and design features(Wang, Montgomery and Owusu-Antwi, 2003). 238 pavement sections from the GPS-1 experiment of LTPP were selected for the study. Based on the analysis, the significance and priorities of the factors influencing pavement roughness were obtained.

Developed in 1984, Classification and Regression Tree (CART), or recursive partitioning method, is a strictly binary decision tree containing two branches for each decision node. The algorithm is to choose a split at each node so that the data in each subset (child node) is "purer" than the data in the parent node. CART measures the impurity of the data in the nodes of a split with an impurity measure. It uses the Gini Index, the Twoing Index or ordered Twoing Index to measure impurity for categorical target, while uses least-squared deviation (LSD) to measure impurity for continuous target *(Washington, Karlaftis and Mannering, 2011, Breiman, 1984, Larose, 2005, Kantardzic, 2011)*. In this study, the targets are the equivalent long term effectiveness and are continuous variables. As shown in Equation (5), the LSD criterion, which is the weighted within node variance for node t, will be used to measure the impurity.

$$R(t) = \frac{1}{N_w(t)} \sum_{i\in t}^{n} w_i f_i (y_i - \bar{y}(t))^2 \tag{5}$$

Where, $N_w(t)$ is the weighted number of records in node t; w_i is the value of the weighting variable for record i; f_i is the value of the frequency variable; y_i is the value of the response variable; and $\bar{y}(t)$ is the weighted mean for node t.

For a split *s* at node *t*, the improvement in the LSD is $I(s,t)$, as shown in Equation (6). Among the set of all possible candidates, the split gives the largest decrease in impurity, maximizing $I(s,t)$, will be chosen. It grows the tree by conducting an exhaustive search of all variables and all possible splitting values at each node until there is no difference within the subsets or some stop criteria is met, e.g. a minimal number of records in a subset(Gaudard, Ramsey and Stephens, 2006, Proust, 2012).

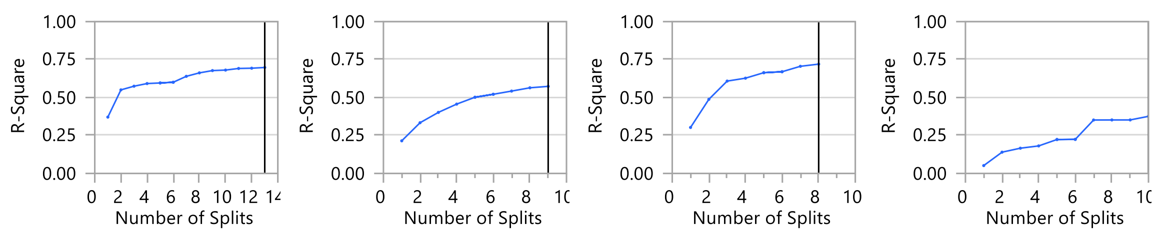
$$I(s,t) = R(t) - P_L R(t_L) - P_R R(t_R) \tag{6}$$

Where, t_L = left child node of node t; t_R = right child node of node t; $P_L = \frac{\text{number of records at } t_L}{\text{number of records in dataset}}$; and $P_R = \frac{\text{number of records at } t_R}{\text{number of records in data set}}$.

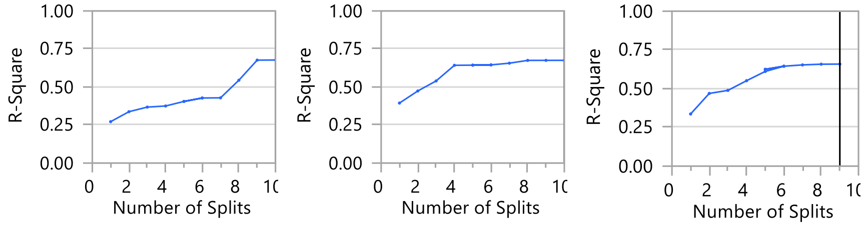
11.4 Discussion of Results

11.4.1 Development of Classification Trees

The Partition function of JMP 12.0 was employed to conduct the CART analysis to classify slurry seal projects into different material, environmental and construction practices groups based on their long term performance. In trial analyses, no stop criterion was set and thus the classification continued until all records were classified. FIGURE 49 shows the R squares of different number of splits for each performance indicator. It can be seen that additional splits do not significantly improve R squares. In addition, high number splits create biased estimations and subsets of few records. Therefore, 5 splits were used to develop the CART trees. FIGURE 50 shows the agreement between the actual and predicted data. It can be seen that the CART trees provide fairly good prediction for treatment effectiveness, especially for roughness, rutting and friction numbers.



(a) Roughness (b) Friction number (c) Rutting depth (d) Alligator cracking

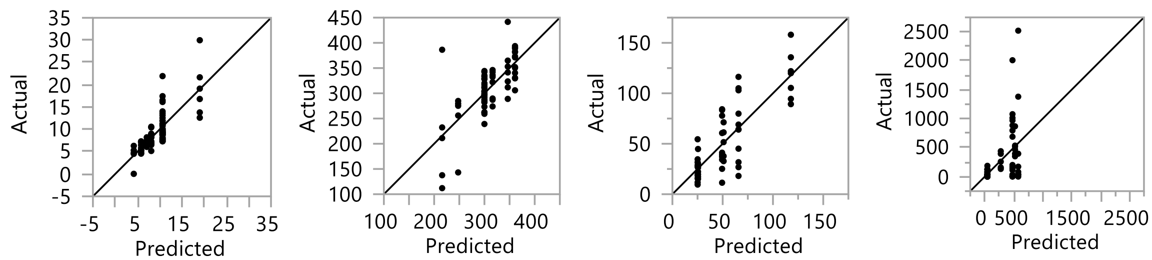


(e) Wheel path longitudinal cracking

(f) Non Wheel path longitudinal cracking

(g) Transverse cracking

FIGURE 49 R squares for full grown classification trees

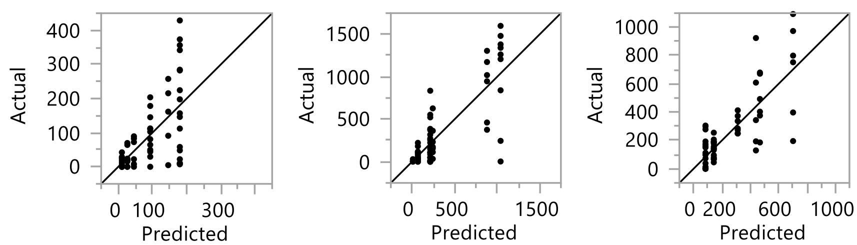


(a) Roughness

(b) Friction number

(c) Rutting depth

(d) Alligator cracking



(e) Wheel path longitudinal cracking

(f) Non Wheel path longitudinal cracking

(g) Transverse cracking

FIGURE 50 Actual vs. predicted of each classification tree

11.4.2 Influence of Factors

FIGURE 51 shows the developed CART tree for the equivalent effectiveness of the seven pavement performance indicators. It shows the hierarchy classification structure, the mean of the child nodes and the samples in the child nodes. The right child node is higher than the left child node. For example, in FIGURE 51(a), the first split is IRIP, resulting the left child node of "IRIP<2.06" and the right child node of "IRIP>=2.06". It means the 89 samples can be firstly classified into two groups based on their pre-treatment IRI values. The 6 samples with pre-treatment IRI higher than 2.06 are classified into the right group with higher roughness level, while the other 83 samples with pre-treatment IRI no more than 2.06 are classified into the left group with lower post-treatment roughness level. Then, the second split is still IRIP. For the 83 samples with pre-treatment IRI no more than 2.06, they are again classified into two groups according to their pre-treatment IRI. 38 samples with re-treatment IRI no less than 1.11 are classified into the group with higher roughness level while the rest 45 samples are classified into the group of lower roughness level. Details of each CART tree and the influence of factors on slurry seal effectiveness will be discussed as follows.

Roughness

It can be seen from FIGURE 51(a) that pre-treatment roughness level (IRIP) is selected twice as the split variable, accounting for more than 90% of the sum of squared deviations as shown in FIGURE 52(a). Rough pavements tend to have high roughness level after slurry seal treatment, confirming that slurry seal will not significantly change the roughness of old pavement(Gransberg, 2010). Other splits include slurry rate (SLUR), aggregate type (AGG), and pre-treatment crack type (CRATY). For the samples collected from LTPP database, slurry seals using sandstone, slag and granite tend to have higher roughness level than those of limestone. Pavements with more transverse cracking tend to have high roughness after slurry seal treatment.

Friction resistance

For friction number, high values indicate sufficient friction resistance. It can be seen from FIGURE 51(b) that the first split variable is maximum speed allowed during initial curing period (MAXSP). Projects with MAXSP lower than 45 mph have significantly higher average friction number during the 6.4 years' service life. The second and third splits are asphalt rate (ACR). Projects with asphalt rate between 0.97 and 1.4 (L/m^2) has higher friction resistance than those of either lower or higher asphalt rate. Insufficient asphalt causes losing aggregate while too much asphalt reduces the macro texture, which all will reduce friction resistance. The fourth and fifth splits are pavement surface temperature (TEMPA) and humidity (HUMID). High pavement surface temperature and low humidity (<50%) tend to have higher friction resistance. Several States' specifications also recommend sustained high temperature and low humidity (≤50-80%) to insure sufficient curing quality.(Gransberg, 2010, ISSA, 2010, NMDOT, 2009, GDOT, 2001)

Rutting

It can be seen from FIGURE 51(c) that traffic level (kESAL), followed by annual precipitation (PERP), slurry rate (SLUR), humidity and maximum speed in initial curing (MAXSP) are selected as splits for rutting severity. For projects with high traffic level, those in the area of high annual precipitation tend to have rutting problem. Low slurry rate (<9.66 kg/m^2) has high rutting potential. Again, high humidity (≥52%) may cause bad curing and cause rutting problem. It is interesting to note that lower maximum seed in initial curing tend to cause rutting problem. It is probably because low speed increases the compaction effect of traveling wheels due to the viscoelastic nature of asphalt materials. However, since the maximum speed is a less contributor for rutting as shown in FIGURE 51(c) and FIGURE 52(c), considering its effect on friction, it is recommended to limit speed limit at initial curing.

Alligator cracks

Alligator cracks are bottom-up type fatigue cracks, usually indicating a several structural fatigue failure. As shown in FIGURE 51(d), split variables for alligator cracks include pavement surface cleanness (PAVCO), freeze thaw condition (FI), rate of mineral fillers (MINER), annual precipitation (PERP) and water added to adjust slurry workability (WAT). Unclean pavement surface (PAVCO), severe freeze condition (FI≥522.6 °C-days), high precipitation (≥3680.4 cm), insufficient mineral filler content (≤0.05 kg/m^2)

and too much water added during construction (volume ratio of water to emulsion >=0.59) all aggravate alligator cracking. Therefore, it is of great importance to maintain pavement surface cleanness and should be cautious to add additional water during application. It is also interesting to note that increasing mineral fillers content could effectively improve slurry seals' resistance to fatigue failure.

Wheel path longitudinal cracks
As shown in FIGURE 51(e) that pavement temperature (TEMPA), asphalt rate (ACR), time before open to traffic (TIMBO), temperature of slurry seal (TEMSL) and traffic level (kESAL) are selected as split variables. Low pavement surface temperature (<34.44°C) and short time before traffic (<1.7 h) tend to induce wheel path longitudinal cracks. However, this CART tree shows that low traffic level, high asphalt rate and high slurry temperature are associated with bad wheel path longitudinal cracking, which is inconsistent with experience.

Non-wheel path longitudinal cracks
Non-wheel path longitudinal cracks are usually caused by loss of asphalt bonding or inner tension stress due to temperature drop. It can be seen from FIGURE 51(f) that freeze index (FI) is used as both the first and second split variable. Other split variables include traffic level (kESAL), air temperature (TEMAI) and amount of slurry materials (SLUR). Severe freeze condition and higher traffic level significantly aggravate this type of non-structural cracking. It seems high air temperature and large amount of slurry materials tend to induce the non-wheel path longitudinal cracks which is also inconsistent with the experience. The abnormal trend with the two longitudinal cracking is probably due to the high skewness and dispersion of cracking data. The results need to be further validated when more sample projects and more consistent cracking data are available in future research.

Transverse cracks
Transverse cracks are usually caused by reflective or thermal cracking. Similar with non-wheel path longitudinal cracks, freeze condition (FI) is the first split variable, followed by crack type on old pavement (CRATY), asphalt rate (ACR), and pavement temperature (TEMPA), as shown in FIGURE 51(g). Severe freeze-thaw conditions tend to induce the transverse cracks or thermal cracks. If the main types of cracks or distress on old pavement are transverse, alligator or raveling the slurry seals tend to have more transverse cracks than those with edge, block or longitudinal cracks. Further, pavement with alligator cracks and raveling are more likely to have transverse cracks than those with only transverse cracks.

<table>
<tr><td colspan="5">IRIP<2.06</td><td rowspan="5">IRIP>=2.06
Count: 6
Mean: 19</td></tr>
<tr><td colspan="4">IRIP<1.11</td><td rowspan="4">IRIP>=1.11
Count: 38
Mean: 10.6</td></tr>
<tr><td rowspan="3">AGG(Limestone)
Count: 5
Mean: 4.2</td><td colspan="3">AGG(Sandstone, Slag, Granite)</td></tr>
<tr><td colspan="2">SLUR>=12.75</td><td rowspan="2">SLUR<12.75
Count: 16
Mean: 8.1</td></tr>
<tr><td>CRATY(Block, Longitudinal,)
Count: 11
Mean: 5.8</td><td>CRATY(Transverse)
Count: 6
Mean: 7.1</td></tr>
</table>

(a) Roughness

<table>
<tr><td rowspan="4">MAXSP>=45
Count: 5
Mean: 215.9</td><td colspan="5">MAXSP<45</td></tr>
<tr><td rowspan="3">ACR<0.97
Count: 5
Mean: 247.7</td><td colspan="4">ACR>=0.97</td></tr>
<tr><td colspan="2">ACR>=1.4</td><td colspan="2">ACR<1.4</td></tr>
<tr><td>TEMPA<50
Count: 26
Mean: 300</td><td>TEMPA>=50
Count: 7
Mean: 346.4</td><td>HUMID>=52
Count: 8
Mean: 316.2</td><td>HUMID<52
Count: 13
Mean: 360.8</td></tr>
</table>

(b) Friction number

<table>
<tr><td colspan="4">kESAL<163.5</td><td colspan="2">kESAL>=163.5</td></tr>
<tr><td colspan="3">SLUR>=9.66</td><td rowspan="3">SLUR<9.66
Count: 11
Mean: 65.7</td><td rowspan="3">PERP<2611.98
Count: 5
Mean: 50.9</td><td rowspan="3">PERP>=2611.98
Count: 7
Mean: 117.8</td></tr>
<tr><td rowspan="2">MAXSP>=40
Count: 12
Mean: 25.1</td><td colspan="2">MAXSP<40</td></tr>
<tr><td>HUMID<52
Count: 8
Mean: 25.5</td><td>HUMID>=52
Count: 10
Mean: 49.4</td></tr>
</table>

(c) Rutting depth

<table>
<tr><td colspan="5">PAVCO(Clean)</td><td rowspan="4">PAVCO(, Mostly clean)
Count: 8
Mean: 578.3</td></tr>
<tr><td colspan="4">FI<522.8</td><td rowspan="3">FI>=522.8
Count: 17
Mean: 479.3</td></tr>
<tr><td colspan="2">MINER>=0.05</td><td colspan="2">MINER<0.05</td></tr>
<tr><td>PERP<3680.41
Count: 23
Mean: 45.7</td><td>PERP>=3680.41
Count: 5
Mean: 277</td><td>WAT<0.59
Count: 5
Mean: 47.7</td><td>WAT>=0.59
Count: 6
Mean: 523.2</td></tr>
</table>

(d) Alligator cracking

<table>
<tr><td colspan="5">TEMPA>=34.44</td><td rowspan="4">TEMPA<34.44
Count: 17
Mean: 180</td></tr>
<tr><td colspan="4">ACR<1.6</td><td rowspan="3">ACR>=1.6
Count: 5
Mean: 146.1</td></tr>
<tr><td colspan="2">TIMBO>=1.7</td><td colspan="2">TIMBO<1.7</td></tr>
<tr><td>TEMSL<31.67
Count: 15
Mean: 10.6</td><td>TEMSL>=31.67
Count: 6
Mean: 45.8</td><td>kESAL>=161.25
Count: 8
Mean: 26.6</td><td>kESAL<161.25
Count: 12
Mean: 93.5</td></tr>
</table>

(e) Wheel path longitudinal cracking

<table>
<tr><td colspan="5">FI<825.6</td><td rowspan="4">FI>=825.6
Count: 9
Mean: 1040.7</td></tr>
<tr><td colspan="2">FI<5</td><td colspan="3">FI>=5</td></tr>
<tr><td rowspan="2">TEMAI<31.67
Count: 5
Mean: 9.5</td><td rowspan="2">TEMAI>=31.67
Count: 15
Mean: 68</td><td rowspan="2">kESAL<82
Count: 21
Mean: 213.3</td><td colspan="2">kESAL>=82</td></tr>
<tr><td>SLUR<10.85
Count: 7
Mean: 243.1</td><td>SLUR>=10.85
Count: 6
Mean: 880.9</td></tr>
</table>

(f) Non Wheel path longitudinal cracking

<table>
<tr><td colspan="5">FI<1278</td><td rowspan="4">FI>=1278
Count: 6
Mean:
702.3</td></tr>
<tr><td colspan="2">CRATY(Edge, Block, ,
Longitudinal)</td><td colspan="3">CRATY(Transverse, Alligator, Raveling)</td></tr>
<tr><td rowspan="2">ACR<1.75
Count: 16
Mean: 45.2</td><td rowspan="2">ACR>=1.75
Count: 7
Mean: 173.5</td><td colspan="2">CRATY(Transverse)</td><td rowspan="2">CRATY(Alligator,
Raveling)
Count: 6
Mean: 468.5</td></tr>
<tr><td>TEMPA<45.5
6
Count: 22
Mean: 196.7</td><td>TEMPA>=45.5
6
Count: 5
Mean: 440.2</td></tr>
</table>

(g) Transverse cracking

FIGURE 51 Developed classification trees

FIGURE 52 shows the sum of squares explained by those variables used as splitting nodes in the CARTs. It can be seen from FIGURE 52(a) that the roughness level of slurry seal is mainly determined by old pavement roughness. Maximum allowed speed

(MAXSP) and asphalt rate (ACR) are critical for the friction of slurry seals. Traffic level (kESAL), weather (PERP) and thickness (SLUR) are the main factors for rutting problem on slurry seals. For non-wheel path longitudinal and transverse cracking, the freeze index is the first splits and accounts for majority of the sum of squares, indicating severe weather is the main cause of these cracks. Although freeze index is not the first splits of alligator cracking, it also plays a vital role in alligator cracking. Pavement surface cleanness, rate of mineral fillers and crack types on old pavement are all significant for the cracking on slurry seals.

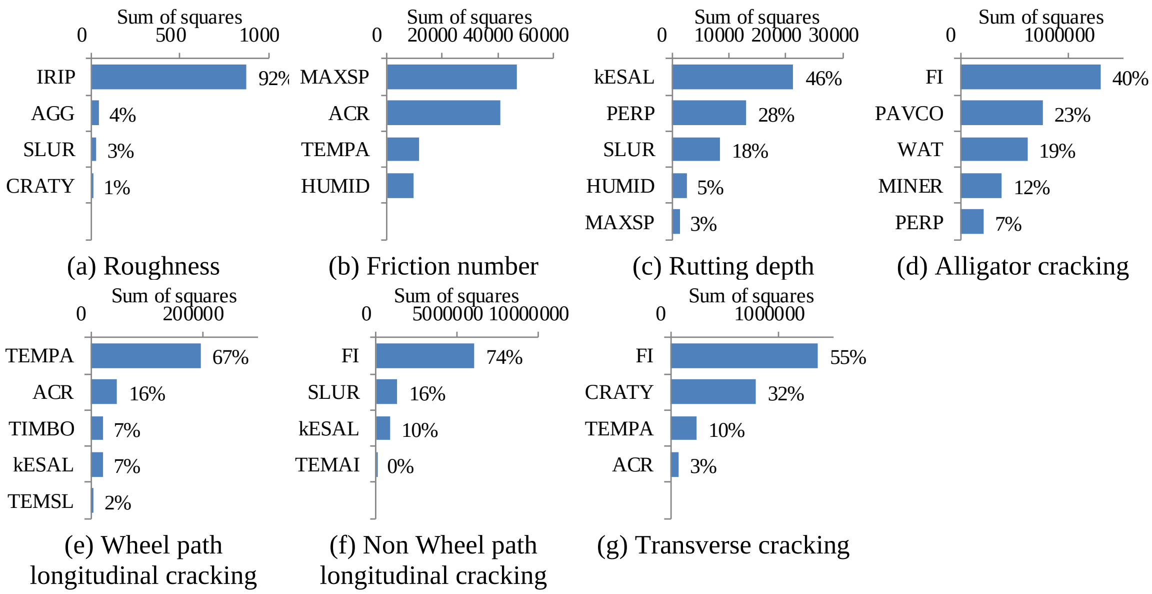


FIGURE 52 Rank of sum of squares for each classification tree

11.5 Conclusions

This study investigated the performance of 89 slurry seal projects collected from the SPS-3 experiment of the LTPP program and analyzed the influence of materials and construction practices on the long term performance of those slurry seals by CART classification tree data mining method. The equivalent effectiveness was developed to characterize long term performance of pavement treatments. It firstly calculated the area under the performance curves as treatment effectiveness and then converted the original effectiveness of different service times to those of the same age. The CART trees identified significant variables to classify slurry seal project into subsets regarding their performance and quantified the influence or contributions of material and construction factors.

Generally, the developed CART trees provided good fitness, especially for roughness, friction and rutting. In addition to the weather and traffic factors such as freeze index, precipitation, and kESAL, identified material and construction practices variables included asphalt rate, slurry rate, aggregate type, amount of additional water, maximum speed at initial curing, humidity, pavement temperature, air temperature, pavement surface cleanness and cracking type on old pavement. Based on the analyses above, several conclusions regarding the effects of factors on slurry seal performance can summarized as follows:

1. Pre-treatment roughness level was the most significant factor for the roughness of slurry seals which agreed with previous findings. Aggregate type and cracking type on old pavement were also significant factors for the roughness of slurry seals.
2. To ensure sufficient high friction resistance, the maximum allowed speed should be no more than 45 mph and asphalt rate should be between 0.97 and 1.4 L/m^2. High pavement surface temperature and low air humidity are also recommended to help curing.
3. Traffic level is the most significant factor for rutting of slurry seals, followed by precipitation, slurry rate, humidity and maximum speed at initial curing.
4. In addition to severe weather conditions, unclean pavement surface, insufficient mineral filler content (≤ 0.05 kg/m^2) and too much water added in the slurry also will aggravate alligator cracking.
5. For the 89 investigated slurry seal projects, those with relatively lower pavement surface temperature (<34.44°C) and short time before traffic (<1.7 h) tend to encounter wheel path longitudinal cracks.
6. Severe freeze-thaw conditions contribute the most to the formation of non-wheel path longitudinal cracks followed by slurry rate and traffic levels.
7. In addition to freeze thaw condition, old pavement (CRATY) condition, asphalt rate (ACR), and pavement temperature are also identified as contributing factors for transverse cracking. Pavements with transverse, alligator cracking or raveling are more likely to have transverse cracks on slurry seals than those with edge, block or longitudinal cracks.

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12 SUMMARY

This project aims to assist in implementation of PMS including data collection, condition report and strategy analysis and help developing the asset management plan for the state. The research team accomplished several proposed tasks including: defining the most recent highway information and pavement maintenance treatments in PMS; examining "untreated" and "poor" pavement segments; updating pavement related data; generating annual pavement condition report; producing the MAP21 pavement condition report for the FHWA; conducting pavement maintenance strategy analysis. In addition to routine work for PMS and AMS, the research team also worked on several tasks for pavement maintenance and management. Main finding are summarized as follows:

1. The IRI data collected by Mandli and TDOT's own pavement profiler were compared. The "profile synchronization" function in the ProVAL software was utilized to analyze the correlations of the two dataset. Generally, short section length has higher correlation due to reduced noise and wheel wandering. Optimized section length needs to be determined for effective validation in the following research.

2. By setting the roughness, rutting and distress thresholds, poor pavement segments in Tennessee were identified. Related traffic and maintenance information of those pavement segments were also collected. The average surface age of both low PDI and PSI sections is 15 years. Most of the poor sections are caused by lack of maintenance. Intersections, high grade or curvature are potential factors accelerating the deterioration of pavement, especially rutting. Statistical analyses results show that surface age significantly reduce pavement condition. Traffic level potentially accelerates the deterioration of pavement.

3. It's found that passing percent is the most significant factor for sample size while population size has little influence, especially for large populations. Based on the historical pavement inspection data of TDOT, we are 95% sure that, the inspected passing percent has a precision of 0.7% at state level, 1.4% at region level, 2.7% at district level and 6.9% at county level. An effective approach to improve the precision at lower management levels without increasing sample size is to evenly assign sample size among different subgroups disregarding the road mileage within that subgroup. It was found that there is no significant difference between the passing percent of interstates and state routes. Therefore, it is unnecessary to sample interstates or state routes separately. Sampling at district levels could potentially improve the significance level of half of the districts; however the sample size would be multiplied and is not cost-effective.

4. Pre-treatment pavement performance was the most significant factor for the performance of post preventive treatments. Poor pre-treatment performance significantly increased the failure probability of treatments. Therefore, it is not

recommended to apply preventive treatments on poor pavements. Pavement structural capacity, climate and traffic levels significantly affected post treatment performance. Stronger pavement structural capacity greatly reduced fatigue alligator cracking on treatment surface because the bending of the asphalt layers could be reduced with a strong bases and subgrade support. Climatic factors influenced the failure of transverse cracking, rutting and friction loss. Generally, warm climate increased the risk of rutting and friction loss. Generally, chip seals provided the best overall effectiveness followed by asphalt overlay, slurry seal and fog seal. Chip seals performed best in terms of roughness and cracking, whereas they were vulnerable to friction loss, indicating that the aggregate chips could be easily pulled out due to the loss of asphalt adhesion. Fog seal performed the worst, because it is simply an asphalt sealing coat without aggregate. The roughness or distress performance of fog seal is mainly determined by the original pavement.

5. CFA was utilized to analyze the contributions of condition measurements on the latent comprehensive pavement condition and the relative importance of the condition measurements. Analysis of results revealed that the multiple factor model provided better model fit goodness than the single factor model. All the observed variables were statistically significant. Among those indicators, IRI, longitudinal cracking in wheel path, non-wheel path longitudinal cracking, transverse cracking, fatigue cracking and block cracking were strong indicators with factor loadings higher than 0.3. Comparison between the factor loadings of state routes and Interstates indicated that the more severe distresses such as fatigue and transverse cracking have higher contributions to the latent pavement condition of state routes while less severe distresses such as raveling and patch have higher contributions to Interstates. Since state routes have relatively lower condition level and may have already encountered certain amount of distress, the severe distresses have higher impacts on their condition. Whereas Interstates are maintained at higher condition level, the less severe distresses could impact its overall condition. Therefore, it is rational to consider the relative importance of condition measurements when evaluating different types of roads.

6. Generally, the developed CART trees provided good fitness, especially for roughness, friction and rutting. In addition to the weather and traffic factors such as freeze index, precipitation, and kESAL, identified material and construction practices variables included asphalt rate, slurry rate, aggregate type, amount of additional water, maximum speed at initial curing, humidity, pavement temperature, air temperature, pavement surface cleanness and cracking type on old pavement.

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