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Application of an improved transition probability matrix based crack rating prediction methodology in Forida's highway network

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University of South Florida

Application of an Improved Transition Probability Matrix Based Crack Rating Prediction

Methodology in Florida's Highway Network

by

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A thesis submitted in partial fulfillment
of the requirements for the degree of
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Dedication

To my parents, Siavosh and Mahshid Nasseri, and my brothers, Shahin and

Sepehr Nasseri, who have been supporting me throughout my education with

unconditional help and love. They have been my inspiration and support. To all my

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ABSTRACT

With the growing need to maintain roadway systems for provision of safety and comfort for travelers, network level decision-making becomes more vital than ever. In order to keep pace with this fast evolving trend, highway authorities must maintain extremely effective databases to keep track of their highway maintenance needs. Florida Department of Transportation (FDOT), as a leader in transportation innovations in the U.S., maintains Pavement Condition Survey (PCS) database of cracking, rutting, and ride information that are updated annually.

Crack rating is an important parameter used by FDOT for making maintenance decisions and budget appropriation. By establishing a crack rating threshold below which traveler comfort is not assured, authorities can screen the pavement sections which are in need of Maintenance and Rehabilitation (M&R). Hence, accurate and reliable prediction of crack thresholds is essential to optimize the rehabilitation budget and manpower. Transition Probability Matrices (TPM) can be utilized to accurately predict the deterioration of crack ratings leading to the threshold. Such TPMs are usually developed by historical data or expert or experienced maintenance engineers' opinion. When historical data are used to develop TPMs, deterioration trends have been used

indiscriminately, i.e. with no discrimination made between pavements that degrade at different rates. However, a more discriminatory method is used in this thesis to develop TPMs based on classifying pavements first into two groups. They are pavements with relatively high traffic and, pavements with a history of excessive degradation due to delayed rehabilitation.

The new approach uses a multiple non-linear regression process to separately optimize TPMs for the two groups selected by prior screening of the database. The developed TPMs are shown to have minimal prediction errors with respect to crack ratings in the database that were not used in the TPM formation. It is concluded that the above two groups are statistically different from each other with respect to the rate of cracking. The observed significant differences in the deterioration trends would provide a valuable tool for the authorities in making critical network-level decisions. The same methodology can be applied in other transportation agencies based on the corresponding databases.

Introduction

FDOT Pavement Condition Database (PCS)

For any highway agency to manage their roadway systems successfully, it is necessary to maintain a roadway condition inventory with constant annual updates. To fulfill this crucial need, Florida Department of Transportation (FDOT) has produced a Pavement Condition Survey (PCS) database. The PCS database was standardized in 1986 to incorporate pavement condition data from all 7 districts in Florida. Before 1986, separate districts would perform their individual survey by hiring either their own personnel or contractors. In the first 3 years of data collection after standardization, BM&R was the sole contractor performing the pavement condition survey. In 1989, the data collection was assigned to the State Materials Office personnel in Gainesville, FL (Appendix A). Since the introduction of the PCS database, FDOT has expended a major effort in the maintenance and improvement of this database. Since the initiation of the database, over 3000 rated miles (from 15,566 to 18,693) and 2500 sections (from 5,812 to 8,469) have been added to the database. Currently, all the maintenance and rehabilitation work associated with the FDOT pavement management systems is based on the PCS

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database. FDOT has achieved an elite status and recognition in the nation for its comprehensive database.

At present, the database contains about 9000 sections from all 7 districts of Florida. For each section, an identification number is used to distinguish that section. In the PCS database, for each section of the roadway, there are some fixed characteristics such as roadway ID, roadway direction (left or right), county and district allocation, and US or statewide roadway ID number (i.e. SR45, US41). There are also some characteristics that would change if any rehabilitation and maintenance is performed on that section, such as the begin mile post and the end mile post, surface asphalt type, asphalt thickness, number of duty cycles, and total lane mileage. Finally, there are characteristics that change annually such as age, Equivalent Single Axle Load (ESAL), and average daily traffic.

The other parts of the database are for the input of condition ratings based on annual survey of the roadway. The updated condition data include the Cracking Rating (CRK) which is of interest in this paper, Rutting Index (RUT), Ride Quality (RIDE), and Pavement Condition Rating (PCR). An extract from the Florida PCS database is shown in Figure 1.

Each section is then divided into sub-sections based on characteristics of the section so that each sub-section becomes more or less homogenous with respect to roadway geometry, traffic and condition features. Typical characteristics that are included in the database are: geographic location, pavement type (flexible, rigid), pavement surface type (open graded, dense graded), traffic level (A, B, C, D, and E), construction cycle, and extent of deterioration. Hence, it is obvious that every time any rehabilitation

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or maintenance is performed on a sub-section, a new sub-section(s) would emerge and the database is updated since the characteristics of the rehabilitated sub-section changes invariably. This implies that as time goes on, there would be more sections added to the database while the lengths of sub-sections would become smaller.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
1	TRAF	ADLEN	RDWY	BMP	EMP	RDWYID	AEFFDAT	CONTYD(	GEODIST	NEWLNA(	EXLNIMP(	VENDNA(	SURFTYF	ASTYPE
2	M	0.491	L	0	0.491	01010000	3/30/2007	01	01	0	4	BETTER I	FC5	
3	M	5.427	R	0	7.777	01010000	3/30/2007	01	01	0	4	BETTER I	FC5	
4	M		L	0.491	4.98	01010000		01	01				CONC	
5	M	2.302	L	4.98	7.777	01010000	3/30/2007	01	01	0	4	APAC-SO	FC5	
6	M	4.054	L	7.777	11.857	01010000	3/30/2007	01	01	0	4	AJAX PAV	FC5	SP
7	M	4.054	R	7.777	11.857	01010000	3/30/2007	01	01	0	4	AJAX PAV	FC5	SP
8	M	1.417	L	11.857	13.274	01010000	3/30/2007	01	01	0	2	BETTER I	FC125	SP
9	M	1.417	R	11.857	13.274	01010000	3/30/2007	01	01	0	2	BETTER I	FC125	SP
10	M	0.195	C	13.274	13.469	01010000	3/30/2007	01	01	0	2	BETTER I	FC125	SP
11	M	0.696	C	13.469	14.165	01010000	3/30/2007	01	01	0	2	BETTER I	FC125	SP
12	M	0.963	C	14.165	15.128	01010000	3/30/2007	01	01	0	0			
13	M		C	15.128	15.235	01010000		01	01					
14	M	4.121	L	15.235	21.021	01010000	3/30/2007	01	01	2	4			
15	M	4.121	R	15.235	21.021	01010000	3/30/2007	01	01	2	4			
16	M		L	21.021	22.145	01010000		01	01					
17	M		R	21.021	22.145	01010000		01	01					
18	M	3.801	L	22.145	25.946	01010000	3/30/2007	01	01	0	4	BETTER ROADS INC		
19	M	3.801	R	22.145	25.946	01010000	3/30/2007	01	01	0	4	BETTER ROADS INC		
20	M		C	0	0.118	01010101		01	01					
21	M		C	0.118	1.326	01010101		01	01					
22	M		C	1.326	2.042	01010101		01	01					
23	M	18.337	C	0	18.337	01030000	3/30/2007	01	01	0	2	WHITE C(	FC4	S
24	M	0.882	C	0	0.887	01040000	3/30/2007	01	01	0	3	AKDM DE	FC4	S
25	M	0.583	C	0.887	1.47	01040000	3/30/2007	01	01	0	3	BETTER I	FC2	S
26	M		L	1.47	2.678	01040000		01	01					
27	M		R	1.47	2.679	01040000		01	01					

Figure 1. Extract from Florida's PCS database

Pavement Evaluation

The data for the PCS database is gathered using FDOT's customized vehicle called the Multi-Purpose Survey Vehicle (MPSV) shown in Figure 2. MPSV is equipped with sophisticated on-board instrumentation and associated computer systems. All the relevant data that is collected by the MPSV is entered in PCS on an annual basis.

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Figure 2. Multi Purpose Survey Vehicle (MPSV)

Pavement Cracking

Of the different types of distresses, rutting and cracking are the two major distress types that are dominant in Florida's flexible pavements. Cracking is also a dominant type of distress in Florida's relatively small percentage of rigid pavements. A crack is a discontinuity in the pavement surface with minimum dimensions of 1 mm (1/25 in) width and 25 mm (1 in) length (AASHTO-PP44-01). There are different types of cracks which may include longitudinal cracks, transverse cracks, block cracks, edge cracks, and alligator cracks for flexible pavements and longitudinal cracks, transverse cracks, and corner cracks for rigid pavement. In general, cracks are divided into 3 levels of severity and intensity (AASHTO-PP44-01).

- Severity Level 1: Cracks ≤ 3 mm (1/8 in)
- Severity Level 2: Cracks with dimension > 3 mm (1/8 in) and ≤ 6 mm (1/4in)
- Severity Level 3: Cracks with dimensions > 6 mm (1/4 in)

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In asphalt pavements, cracks develop and propagate with time due to many causes such as age-induced fatigue that results in reduced tensile strength required to overcome wheel induced pavement flexural stresses, a condition which eventually leads to failure under repeated loading; age-induced hardening of the binder causing inadequate tensile strength to meet the stresses induced by daily temperature cycling; excessive tensile stresses induced by the swelling/shrinkage of roadbed (subgrade) soils when pavements are constructed in expansive soils; improper lane-joint and lane-shoulder joint construction causing edge and longitudinal cracks; and low temperature induced hardening of the binder which results in inadequate tensile strength to overcome even normal vehicle-induced strains (low temperature cracking in asphalt). Of the above, obviously only the first four types are relevant to asphalt pavements in Florida due to its temperate climate. On the other hand, cracks in concrete pavements of Florida occur primarily due to temperature induced curling stresses (Kumara et al, 2003). The major focus of this thesis is on load induced damage and the delayed maintenance and rehabilitation damage caused by poor roadbed (subgrade) of Florida's flexible pavement network.

Crack Rating

Crack Rating (CR) is a unique distress index of each section which can be used in network level decision making and budget appropriation since it is a appropriate measure of roadway safety and comfort. A shortcoming of this rating is the subjectivity involved in it. Although the raters are trained for rating consistency, human errors are inevitable and are also evident in the database. To address this issue, many softwares and

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instrumentations have been developed for automation of the crack rating. At present, this is a newly focused study area in pavement management. When the automation is widely established, the practicality and applicability of the methodology advanced in this thesis will be more evident since the CR ratings would follow an expected pattern as compared to the current random and less predictable pattern.

CR is a manually assigned rating to a pavement section in the range of 0-10 with 10 indicating an excellent pavement condition with respect to cracking, while 0 indicates a heavily deteriorated pavement. CR is assigned based on a windshield survey which is performed by a trained rater as the Multi Purpose Surveying Vehicle (MPSV) traverses a particular section. Then, the extent of each type of crack seen on the road is recorded in the relevant charts. Based on the severity and density of the dominant crack type in inside and outside wheel paths of each section, a deduct value is extracted from the FDOT's Flexible Pavement Manual Survey Handbook (FDOT, 2003) and CR is calculated by subtracting the deduct value from a perfect 10 CR rating as shown in Equation 1.

$$CR = 10 - (CO + CW) \tag{1}$$

Where

   CO = amount of crack outside wheel path

   CW = amount of crack inside wheel path

The CR rating is then recorded in the appropriate column of the PCS database each year. It should be noted that for newly rehabilitated sections, CR would be 10. Therefore, by locating the sudden rise of CR from a low CR value to 10, the starting year of the new duty cycle of that section can be determined. This concept is widely used in

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the analysis of the database especially in determining and sorting the individual cycles of a section.

Condition Prediction

Predicting the future condition of a pavement provides pavement engineers with a valuable tool to prioritize the pavement sections for M&R activities with better accuracy and efficiency. Therefore, reliable performance prediction models are becoming a necessity in today's pavement management systems (Gendreau et al, 1994).

Some researchers have developed analytical expressions to predict the future condition of a pavement (Kong et al, 2002). However, such equations are only applicable in specific locations because there is a multitude of variables involved with cracking such that one expression cannot incorporate such a vast number of variables and be universally representative. For instance, Equation 2 has been developed for Brevard County of Florida (Kong et al, 2002).

$$ycc = 0.979 \times ym1c - 0.08561 \times s \times (\frac{a}{1000})$$

(2)

Where

ycc = Last year crack rating,

ym1c = Year before last year crack rating,

s = the slope of rating deterioration,

a = annual average daily traffic.

It can be seen clearly that the methods available in the literature have been generated using data as random variables and that they lack the relevant technical input.

Although researchers have incorporated complex concepts such as neural network (Yang et al, 2005) and nonlinear regression models (Ortiz et al, 2006) into the pavement condition prediction, there is still much more research that have to be performed. Hence, an enhanced method of integration of engineering knowledge into the development of a more accurate prediction model is presented in this paper.

Condition Prediction Based on Markov Models

According to the Markov Chain based method of crack condition-prediction, which has been described in detail by Butt (1991), a pavement condition measuring scale can be divided into discrete intervals called '*condition states*'. In the case of the crack rating the scale can be divided into 10 condition states each 1 unit wide. In order to accurately predict the likely future behavior of pavements which are currently at a given condition state, in terms of probabilities, the '*transition probability matrix*' can be used (Shahin et al, 2003). In general, a Transition Probability Matrix (TPM) is used when the condition of a facility is transiting from one state (i) to the next lower state (j) in a single step as shown in Equation 3.

$$p_{ij} = P\{X_n = j | X_{n-1} = i\} \tag{3}$$

Where the transition probability matrix [P] consists of the one-step transition probabilities, p_{ij}. The most basic, yet time consuming, method to determine probability of the TPM elements is to solely use the historic data. In order to find the transition probability matrix [P] P_{ii} is defined as the probability of a pavement section remaining in the same condition state in the following year and P_{ij} is the probability that the pavement condition state degrades from '*i*' to '*j*'. as stated before, it is assumed that '*i*' and '*j*'

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cannot differ by more than one [1] state. Using historic data, one can find the number of sections that remained in the same condition state (i) in each year (N_{ii}) and also the number of sections that degraded into the lower condition state (N_{ij}). Then Equation 4 can be used to find the probability P_{ij}.

$$P_{ij} = \frac{N_{ij}}{N_i} \tag{4}$$

Where:

N_i is the number of sections that started the year in condition state 'i'

A shortcoming of this method is that the proportions are likely to vary from year to year thereby acquiring an average to be used to ensure accuracy. Also, the application of this method can be problematic in many agencies due to the insufficiency of reliable historic data. Since a simple averaging process might not be significantly accurate to be used in high-level analysis, in this thesis, a more sophisticated and reliable mathematical method is applied.

The Markov chain is said to be time homogeneous if the transition probabilities from one state to another (p_{ij}) are independent of the time. The 'm'-step transition probability is the probability of transitioning from state 'i' to state 'j' in 'm' steps as shown in Equation 5.

$$P_{ij}^{(m)} = P\{X_{n+m} = j | X_n = i\} \tag{5}$$

Therefore, by applying the Markov Chain rule, the state vector at time 'm' [P(m)] can also be found in terms of the transition probability matrix [P] and the initial state vector, *P(0)*.

$$P(m) = P(0) \cdot \left[P \right]^m \tag{6}$$

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By applying the above formulation to the pavement crack rating data recorded in the PCS database, the future crack condition of a pavement section can be predicted. If this process is applied to all the sections present in the roadway network database of the state, network level rehabilitation decisions can be made effectively based on the established tolerance levels. This development would certainly enhance the planning process of a Pavement Management System (PMS).

Updated TPM Development Methods

One of the most common and time efficient methods to develop TPMs is by observation of deterioration trends. Identification of specific historic data and analysis of trends will ultimately lead to the development of TPM's. Although this method is the most convenient approach of developing TPMs, it requires historic data. In the absence of historic data an alternative empirical method that can be used to estimate the TPM is to use expert opinion from a panel of experienced engineers. However, in recent years, research has been done to develop more scientific methods to obtain TPMs. Among them is a method that involves the use of a recurrent or a dynamic Markov chain for modeling the pavement crack performance with time in which the transition probabilities are determined based on a logistic model (Yang et al., 2005). In this method, a dynamic Markov chain process is presumed to work for the pavement condition survey database available for the entire roadway network of Florida. The limitations of such a methodology roots back to the limitations of Markovian models in general.

In research reported in Ortiz-Garcia (2006) three alternative methods are proposed to improve the efficiency of developing TPMs. The first method assumes that the raw

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data (i.e. CR) used in the regression analysis of the deterministic model are readily available. If the condition of a site 'j' at time't' is denoted by 'c_{jt}', the objective function Z can be given by:

$$Z = \min \sum_t \sum_j \left[c_{jt} - \overline{y}(t) \right]^2 \tag{7}$$

Where:

$\overline{y}(t)$ is the average pavement condition at time t

The objective function aims, therefore, at minimizing the sum of the squared differences between each of the data points and the average condition calculated from the distribution of a condition, a_t.

The second method also uses the raw data, but after a regression equation has been obtained to describe the progression. If $y(t)$ denotes the regression equation, the objective function, Z, employed to obtain the transition probabilities is as follows:

$$Z = \min \sum_t \left[y(t) - \overline{y}(t) \right]^2 \tag{8}$$

The objective function aims, therefore, at minimizing the difference between the average of a_t and the ordinates of the regression equation. This minimizes the distance between the regression curve and the transition matrix fitted curve.

In the third method the raw data are aggregated into bands of condition and presented in the form of distributions. Using the same nomenclature as above, if $a_t(i)$ denotes the ith element of the TPM predicted distribution at time t, and $a'_t(i)$ is the ith element of the original data distributions at time t, the objective function Z takes the form:

$$Z = \min \sum_t \sum_i \left[a_t(i) - a'_t(i) \right]^2 \tag{9}$$

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It must be noted that using Equation 5, a_t can be obtained as $a_t = a_0 P^t$. The objective function aims, therefore, at minimizing the difference between the distributions of condition obtained from the raw data and the distributions predicted by the transition probabilities. It may be observed from the definitions of the three different objective functions, as in Equations 7-9, that the iterated values in the optimization process are the element probabilities, p_{ij}, of the transition matrix. In the nonlinear optimization algorithm used by Ortiz-Garcia (2006) a search is made for the optimum p_{ij} from initial p_{ij} values. It assesses the gradient of the objective function on the current region and changes the p_{ij} along the path of greatest gradient. The search continues until the objective function cannot be minimized further. After analysis, the author determined that the third method would yield the most optimized and practical transition matrix to be used in the present methodology.

Problem Statement

One shortcoming of the current practice is that TPMs are developed based on observation of trends in historical data or by using expert opinion. Additionally, all of the studies and researches have been carried out with no differentiation among different deterioration trends based on the respective causes of deterioration. Such shortcomings of application of TPMs on the network database may have forced Florida Department of Transportation (FDOT) to disregard the prediction method in their decision-making and utilize simple crack thresholds in their rehabilitation decisions.

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Proposed Research

Development of an improved and more practical TPM could enhance the current prediction process significantly. Therefore, optimizing the historical data-based TPM by using mathematical techniques to improve the accuracy of prediction models is one objective of this thesis. Categorizing the database into two independent groups of excessively trafficked sections and structurally deficient sections due to postponed rehabilitation and the development of specific TPMs for each group is another objective of this thesis.

Thesis Organization

This thesis is divided into four chapters. The first chapter is the introduction. Chapter Two consists of a detailed methodology and procedures that are used to obtain the results. Chapter Three is the results and the appropriate analysis. Finally, the conclusions and limitations are discussed in the fourth chapter.

Chapter Two

Experimental Methodology

Data Filtering

There are geometric and pavement condition data on approximately 9000 pavement sections in the Florida's 2007 PCS database. To facilitate the handling of such a vast amount of data for the analytical needs of this project, the entire database was divided into 7 parts which correspond to the seven (7) administrative districts of Florida. This process provides manageable sub-databases which are easier to handle. In addition, the subdivision has the advantage that if the geographical effects were to be considered, the database would already be divided into desired geographical boundaries.

Since the Crack Rating (CR) is a subjective rating by its very nature and a substantial degree of human error is involved in it, data must be first filtered to eliminate abnormalities. The filtering process will ensure that the sections that have unusual trends are eliminated and will not be allowed to affect the results. Unusual trend can be defined as a sudden CR drop (more than 2 states per year) or a sudden CR increase due to erroneous rating recorded with no obvious sign of rehabilitation or cycle change. What would remain in the database is a series CR records in declining order within each construction cycle for each section.

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Impact Grouping

After the filtering process was complete, another sub-division was needed to further clarify the filtered sections into smaller and more specifically oriented batches. Based on previous research (Yang, et al, 2002) construction duty cycle have been seen to have a critical impact on cracking and deterioration of the pavement. Therefore, the duty cycle can be identified as a major categorizing criterion. In this respect, the most recent cycle of a section would determine the group it belongs to (i.e. cycle 1, 2, 3). After completion of this process, it was observed that most sections in Florida's PCS database were in their 2nd or 3rd duty cycle. Hence, operating in either cycle 2 or 3 was chosen to be one criterion for categorization.

Next step was to identify other significant attributes that lead cracking to approach CR based threshold conditions. For the purpose of this thesis, two such effects were chosen, (1) heavy traffic impact and (2) loss of structural integrity due to delayed maintenance and rehabilitation. In order to understand the effect of heavy traffic on the deterioration of a pavement, sections that are currently operating under traffic levels of C or worse were chosen for the traffic impact study. On the other hand, sections with low pre-rehabilitation CR values (equal or less than FDOT's 6.4 threshold value) were grouped for the low structural integrity impact study. Amongst the sections in the heavy traffic impact set, the sections that had low CR values before rehabilitation (for the considered construction cycle) and a traffic volume that was close to the boundary of traffic levels C and B, were excluded and added to the structural integrity impact group. Similarly, the sections that had pre-rehabilitation CR value close to the threshold value and relatively high traffic volumes were removed from the structural integrity set and

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transferred to the traffic impact study group. Then the relevant data of all the sections were transferred to a database where pre-rehabilitation CR value, Average Daily Traffic (ADT), Equivalent Single Axle Load (ESAL), and the crack ratings for the desired construction cycle were recorded. After the completion of this meticulous sorting procedure, the database was ready for statistical analysis.

Table 1. Statewide comparison of grouping outcome

Districts	Traffic				Low Structural Integrity			
	Ave. ESAL	Ave. AADT	Ave. CR	Ave. CR @ pre M&R year	Ave. ESAL	Ave. AADT	Ave. CR	Ave. CR @ pre M&R year
1	11,921,631	16.4	7.1	6.5	7,317,625	13.5	7.7	4.5
2	10,635,814	14.1	6.7	7.0	3,878,968	12.4	6.8	4.9
3	7,597,771	8.2	8.3	6.6	4,378,110	10.2	6.9	4.0
4	14,975,981	10.2	7.7	8.1	4,545,181	10.1	6.6	5.5
5	12,301,748	10.9	7.8	7.5	8,164,607	9.8	7.2	4.9
6	9,989,384	8.2	8.3	7.7	4,712,325	5.3	7.4	5.9
7	18,380,200	12.6	8	6.9	5,511,969	8.4	7.0	4.7
Total	12,257,504	12	8	7	5,501,255	10	7	5

TPM Development

When the grouping was completed as mentioned above, each group contained two (2) subdivisions (cycle 2 and 3) for each of the seven (7) districts of Florida. For the purpose of generating the Transition Probability Matrices (TPM) for the entire state of Florida, all the data belonging to each of the subdivisions were placed in different databases based on the initial grouping and duty cycle discrepancies. Then, the TPM generation process was performed separately to produce a specific TPMs representative

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of each sub-division. The outcome was four different TPMs which correspond to the specific criteria used to develop them (i.e. two groups with two cycle each).

In order to develop the TPM for each subdivision, a percentage of the sections were randomly taken out of the specific batch, by using a random number generating function built into *Microsoft Excel*, and placed in a different database. This small group is then used to test the accuracy of the TPMs developed based on the larger group. To determine the percentage of sections to be used for testing, three different percentages, 5%, 10%, and 20%, were tried out. For each of the remaining larger groups (95%, 90%, and 80%) a mean CR value was calculated for each year.

To be consistent with other engineering ratings and indices assigned to pavements and the resulting TPMs, the TPM for this thesis has been set to have 10 states of length 1, in the CR scale of 0-10, as can be seen in Equation 10. Equation 10 is an expansion of Equation 3 in which p_i corresponds to p_{ii} and q_i corresponds to p_{ij}.

$$
\text{State}
\begin{matrix}
1 \\ 2 \\ 3 \\ 4 \\ 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10
\end{matrix}
\begin{pmatrix}
p_1 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
q_1 & p_2 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & q_2 & p_3 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & q_3 & p_4 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & q_4 & p_5 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & q_5 & p_6 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & q_6 & p_7 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & q_7 & p_8 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & q_8 & p_9 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & q_9 & 1
\end{pmatrix}
\tag{10}
$$

The most convenient method to obtain p_1 through p_9 is by observation of the trend of the mean CR values (p_{10} is always 1 since the pavement cannot deteriorate any further from the 10th state and the equivalent of $q_{10} = 0$). This trend generally produces an S-curve indicating that CR must be stable at high ratings (CR>8). Then CR degradation must be sharp for intermediate rating values ($5 < CR < 8$) and finally follow a more gradual degradation trend for lower ratings (CR < 5) since deterioration rate slows down after CR surpasses a threshold state. According to this established trend, a preliminary overall TPM was developed to encompass all four groups solely based on observation of the general deterioration trend in the mean CR values in the PCS database. Then by using a multiple nonlinear regression function built in *Microsoft Excel* an optimum TPM was obtained for each group. The process to obtain these optimized TPMs is described in the following section. Explanatory

TPM Optimization

The first step of optimization is to use the preliminary overall TPM and predict future CR values by post-multiplying the TPM by the current perfect CR vector shown in Equation 11.

$$[C] = \text{Perfect Initial Condition CR Vector} = \begin{Bmatrix} 1 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{Bmatrix} \quad (11)$$

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Since the Markov chain rule is applied, the condition vector of each year can be post-multiplied by the TPM to obtain the condition vector of the following year only. The length of the analysis was set to 15 years of age since most pavement sections are rehabilitated before reaching this age and sections older than 15 years of age are found in the database only occasionally. In the next step, the expected value of the following year's CR can be determined by multiplying the previously obtained CR vector by the state average CR vector in Equation 12.

$$[A] = \text{State Average CR Vector} = [9.5 \quad 8.5 \quad 7.5 \quad 6.5 \quad 5.5 \quad 4.5 \quad 3.5 \quad 2.5 \quad 1.5 \quad 0.5] \tag{12}$$

To better illustrate the mentioned matrix operation, the Equations 13 and 14 are used to determine the condition of the pavement section after one year and after 'm' years respectively.

$$[CR]_{1(10x1)} = [P]_{(10x10)}.[C]_{(10x1)} \tag{13}$$

$$[CR]_{m(10x1)} = [P]^m_{(10x10)}.[C]_{(10x1)} \tag{14}$$

Where $[CR]_1$ and $[CR]_m$ are the crack rating vectors after one and m years of rehabilitation respectively, and $[P]$ is the developed TPM.

If the future crack rating of a pavement section after 'm' years is to be predicted, the following equation can be used to calculate the expected crack rating:

$$CR_{predicted} = [A]_{(1x10)}.[CR]_{m(10x1)} \tag{15}$$

Now, there are two sets of CR ratings for the analysis period of 15 years: one that is calculated by using the preliminary TPM and Equation 15 and the other is the mean CR value of the specific sections ($CR_{database}$) which can be obtained from the database. To

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optimize the TPM for each group, the calculated CR value from Equation 15 is set equal to the mean CR value.

$$CR_{predicted} = CR_{database} \quad \text{for } i=1,2,\ldots,15 \tag{16}$$

The multiple nonlinear optimization function then iterates the TPM elements to implement the equality with such defined constrains as the 'p' and 'q' values are between 0 and 1 in Equation 10, and other elements of the TPM are zero. This equating process should be performed for each year so that when it is completed, the manipulated TPM would be optimized. Then the Mean Square Error (MSE) was calculated to check the difference between the average CR values and the TPM predicted values in each year.

$$MSE = \frac{\sum_{i-1}^{n}(CR_{predicted} - CR_{database})^2}{n} \tag{17}$$

Verification of TPM

Next step is test the developed TPM on the small set of validation sections that was set aside originally. To do so, the mean CR values of the validation group are calculated for each year ($CR_{smallaverage}$) and compared to the TPM prediction. Again the MSE is calculated to observe the differences.

$$MSE = \frac{\sum_{i-1}^{15}(CR_{predicted} - CR_{smallaverage})^2}{15} \tag{18}$$

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Chapter Three

Analysis of Results

Results

Network/Project Level Decision-Making

In order to perform a systematic pavement management process, the following

essential steps can be executed at the network and project levels respectively.

- Inventory preparation and maintenance

- Pavement condition survey

- Condition assessment   Network Level

- Condition prediction

- Condition analysis

- Work planning   Project Level

The importance of the database is clearly seen in pavement management

especially at the network level. A systematic approach to pavement management would

start with network level projects and lead to more in-depth project level tasks. This will

ensure optimum budget prioritization and efficient labor deployment. On the other hand,

an ad hoc approach to pavement management could lead to accumulation of unfunded

major M&R requirements (Shahin, 2003).

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The FDOT PCS database is designed to contain pavement condition survey data from the entire roadway network of Florida. Any analysis and decision-making based on this data becomes an input for network level rehabilitation decision-making. For instance, finding a threshold for differentiating well-performing sections from deteriorated sections, based on the crack ratings available in the database, is considered a major network level project that will lead to screening of pavement sections for rehabilitation. When sections in the database are screened and the critical ones are set aside for more specific analysis and rehabilitation, further consideration of them is a project level activity. To exemplify this point consider a section determined to be at the crack threshold level and hence is earmarked for more detailed analysis (i.e. manual survey), it is considered to be a project level task.

The significance of the PCS database on network or project level activities and decision-making is now evident. Therefore, the ensuing section is dedicated to the analysis of results obtained based on the application of the improved TPM development methodology on the PCS database described in Chapter Two. First, results of each step of the study in the methodology section is presented and analyzed in sequence. Finally, application of the overall methodology is presented.

Applicability of Grouping

After the generation of TPMs, a major part of the analysis performed in this thesis was to verify the accuracy and applicability of the grouping process explained in the Experimental Methodology (Chapter Two). The two major groups are the excessively trafficked group and structural deficient group. To illustrate that the two groups are

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distinct and have different characteristics, specific statistical methods were used. Since a large number of sections exists in each category (sometimes up to 600 sections), based on the Central Limit Theorem (CLT), normal distribution approximation was used to represent the distribution of the CR values at each age. Because of this approximation, the normal distribution table and other characteristics of the normal distribution can be applied to the data. Table 1 and 2 show the information on the all the filtered sections that are currently operating in their second and third construction duty cycles, respectively.

Table 2. Characteristics of the sections operating in their 2nd cycle

Age	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
x_1	9.97	9.93	9.79	9.57	9.28	8.91	8.44	7.83	7.33	6.74	6.23	5.83	5.6	5.29	5.25
x_2	9.98	9.81	9.7	9.57	9.35	9.16	8.84	8.5	8.04	7.63	7.25	6.99	6.63	6.23	5.99
n_1	631	631	631	630	629	622	614	594	567	520	470	386	278	188	118
n_2	232	232	232	232	231	230	227	224	220	210	195	159	139	124	105
σ_1	0.26	0.33	0.6	0.85	1.07	1.22	1.38	1.56	1.69	1.86	2.04	2.1	2.21	2.17	2.2
σ_2	0.15	0.54	0.88	0.95	1.1	1.24	1.32	1.53	1.72	1.85	1.95	1.83	1.95	2.07	2.11

Where

x_1 and x_2 are the sample mean CR values of structural integrity deficient and excessive traffic groups respectively

n_1 and n_2 are number of sections in structural integrity deficient and excessive traffic groups respectively

σ_1 and σ_2 are standard deviations of structural integrity deficient and excessive traffic groups respectively

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Table 3. Characteristics of the sections operating in their 3rd cycle

Age	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
x_1	9.97	9.87	9.58	9.29	8.95	8.54	7.92	7.29	6.64	6.17	5.6	5.69	5.42	5.33	5.27
x_2	9.99	9.92	9.84	9.7	9.46	9.05	8.69	8.28	7.75	7.1	6.55	6.53	6.48	6.31	6.11
n_1	310	310	310	310	306	300	295	285	271	235	207	143	107	77	57
n_2	189	189	189	187	189	189	188	185	180	165	155	116	101	85	59
σ_1	0.16	0.43	0.92	1.14	1.34	1.5	1.61	1.88	1.94	2.08	2.27	2.19	2.38	2.56	2.57
σ_2	0.07	0.3	0.48	0.71	0.9	1.22	1.3	1.55	1.68	2.03	2.18	1.96	1.75	1.88	1.99

Depending on the desired confidence level, the following expression can be used to calculate an interval in which the mean differences, μ_1-μ_2, would fall at each age. Equation 19 would yield a lower and an upper limit for the μ_1-μ_2 interval.

$$(X_1 - X_2) \pm Z_{\alpha/2} \sqrt{\left(\frac{\sigma_1^2}{n_1} + \frac{\sigma_2^2}{n_2} \right)} \tag{19}$$

Where

$Z_{\alpha/2}$ is the two tail normal variate corresponding to the confidence interval of (1-α)

In the resultant μ_1-μ_2 interval, μ_1 and μ_2 are the population mean CR values of the structural integrity deficiency and excessively trafficked groups respectively

To be consistent with other engineering confidence interval applications, a confidence interval of 95% was chosen for this analysis; thus, α =0.025 and $Z_{\alpha/2}$ =1.96. The lower limit (L) and upper limit (U) of this 95% confidence interval at each age are presented in Table 3 and 4.

Table 4. Difference in mean CR values for the two groups operating in their 2nd cycle

Age \ Limit	1	2	3	4	5	6	7	8	9	10
U	0.021	0.189	0.210	0.138	0.085	-0.066	-0.196	-0.429	-0.441	-0.593
L	-0.035	0.040	-0.035	-0.140	-0.245	-0.440	-0.602	-0.901	-0.973	-1.187

Table 4. (Continued)

Age \ Limit	11	12	13	14	15
U	-0.694	-0.810	-0.617	-0.461	-0.172
L	-1.353	-1.516	-1.448	-1.418	-1.303

Table 5. Difference in mean CR values for the two groups operating in their 3rd cycle

Age \ Limit	1	2	3	4	5	6	7	8	9	10
U	0.000	0.020	-0.132	-0.249	-0.320	-0.267	-0.516	-0.680	-0.775	-0.526
L	-0.041	-0.109	-0.378	-0.574	-0.714	-0.752	-1.038	-1.305	-1.450	-1.342

Table 5. (Continued)

Age \ Limit	11	12	13	14	15
U	-0.482	-0.332	-0.498	-0.279	0.000
L	-1.406	-1.345	-1.629	-1.674	-1.678

If the sign of the upper and lower limits are the same (positive or negative), it means that one of the means is dominant at that age. In the results shown in Table 3, in the first five years, depending on the intensity of each impact source (extremely high traffic loading or structural inadequacies) either one can be the dominant cause of deterioration. However, after the age of 6, both signs become negative. This means that after 6 year of age, at a 95% confidence, the pavement sections that are operating in their 2nd cycle tend to have lower mean CR value if they belong to the structurally deficient group as compared to the excessive traffic group. The above observation holds true for the sections of the 3rd cycle after the age of 3. However, for practical applications, not all the differences would be considered significant. Therefore, a threshold should be set as the minimum required difference in the CR readings for that difference to be significant. After further studies, a difference of one (1) in the CR is determined to be significant. This means that if the average CR values of two different groups differ by more than 1 unit, that difference can be considered significant. In the case of Florida's PCS database, for sections operating in their 2nd construction duty cycle, after the age of eight (8) years, the structural deficient pavement sections behave differently from the traffic loading impacted sections. The same conclusion holds true for sections operating at their 3rd duty cycles after the age of seven (7). The significance of this finding is that it shows that the sections that have delayed M&R deteriorate faster than the sections that have higher traffic loadings. It is a critical managerial decision making criterion which will be explained in more detail in the application section (Figures 3 and 4).
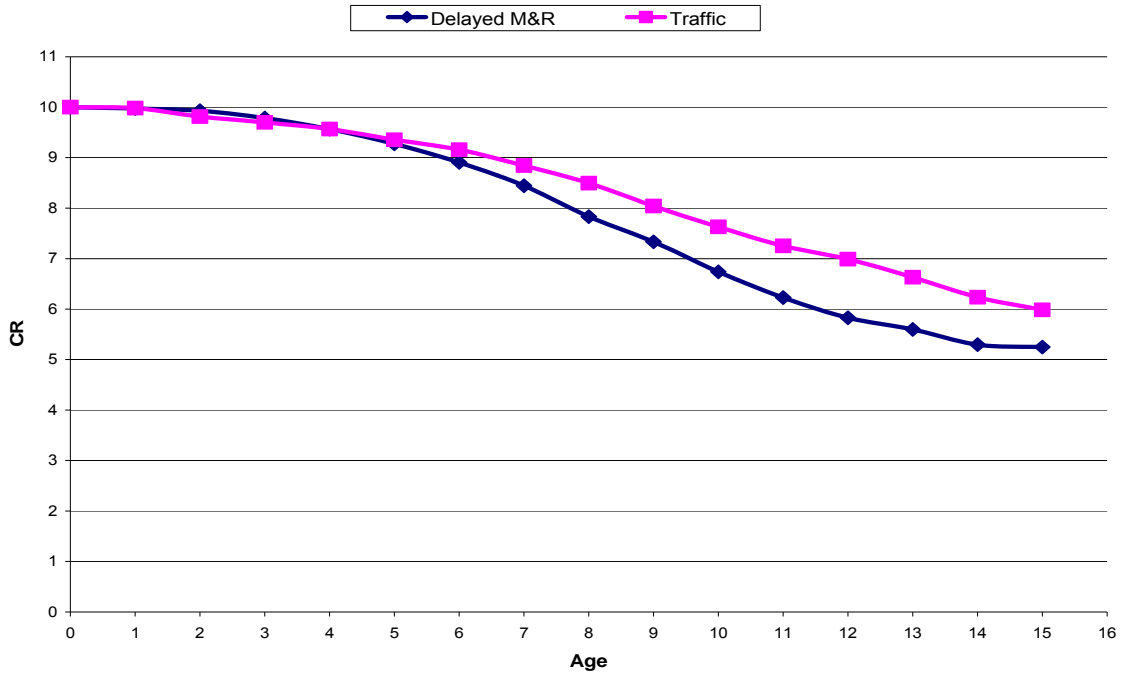
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Figure 3. Comparison of degradation between structural deficient and excessive traffic impact (construction duty cycle =2)
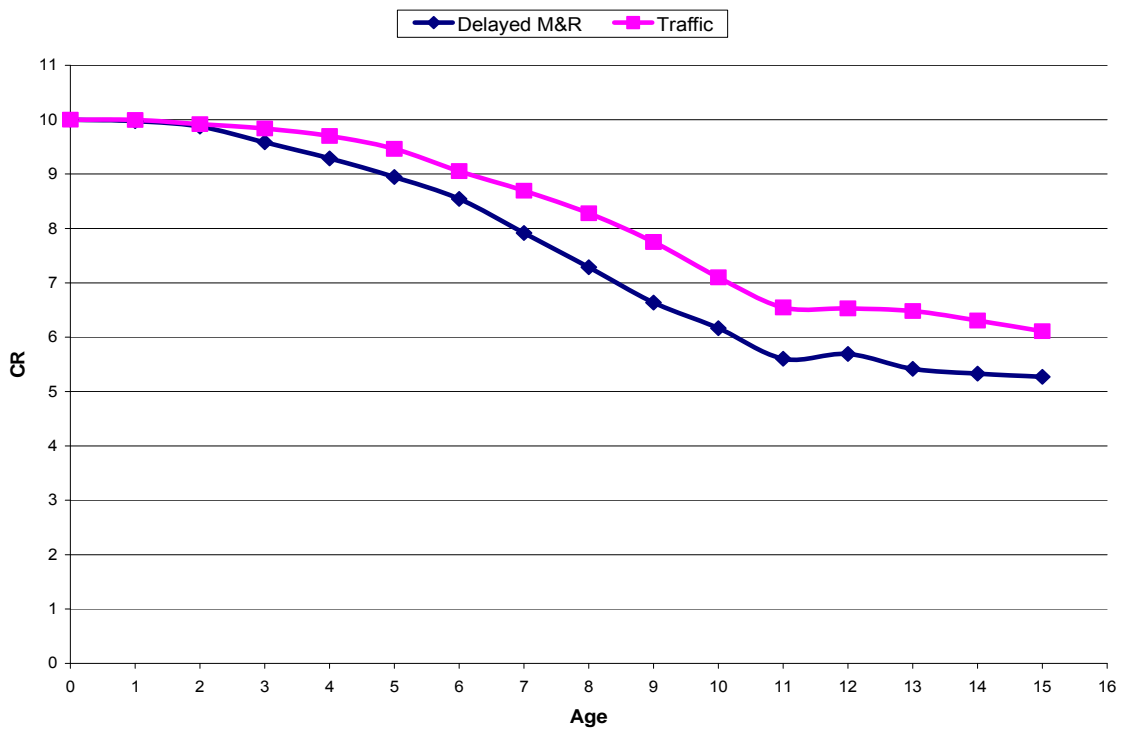

Figure 4. Comparison of degradation between structural deficient and excessive traffic impact (construction duty cycle =3)

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Another interesting finding of the mean comparison is that as the age increases, so does the interval length between the upper and lower limits. This phenomenon can be due to the randomness involved with the data and the fact that as the sections age, more randomness is introduced to the data points (Figures 5 and 6). Additionally, as the interval length increases, it becomes harder and more challenging to predict the ratings in the future. This is why a more scientific and mathematically involved procedure is needed to develop the TPM rather than pure observation.
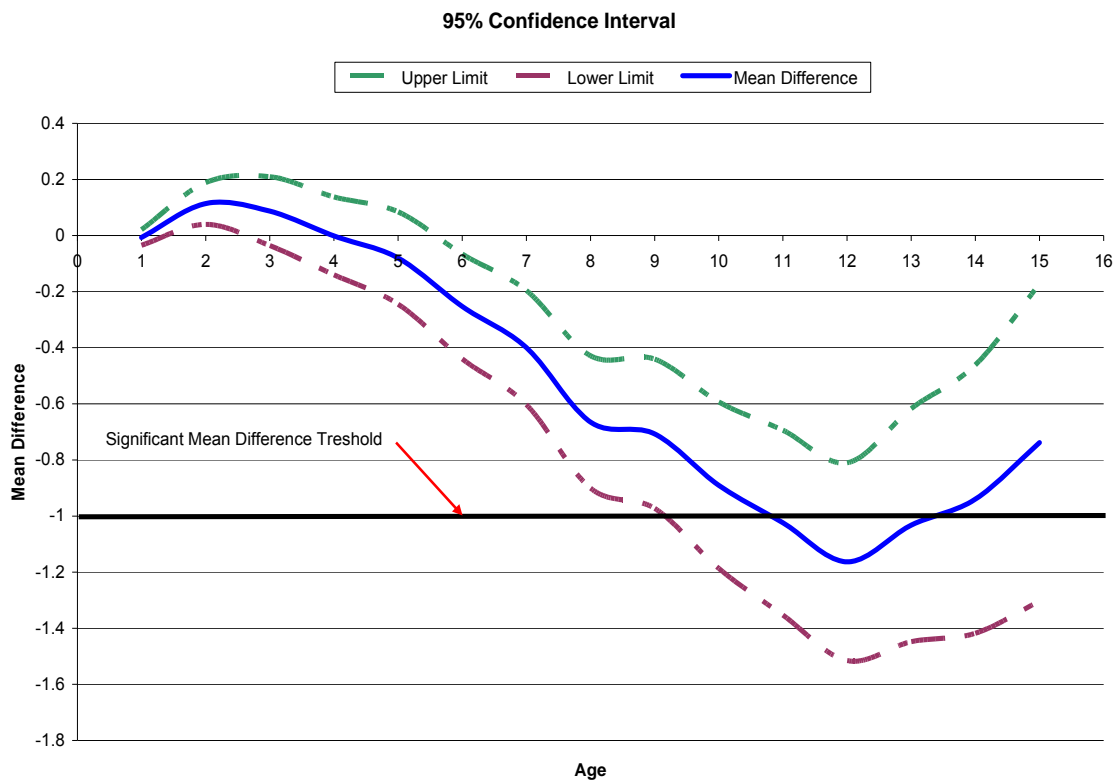

95% Confidence Interval

Figure 5. Mean difference between structural deficient and traffic groups (construction duty cycle =2)
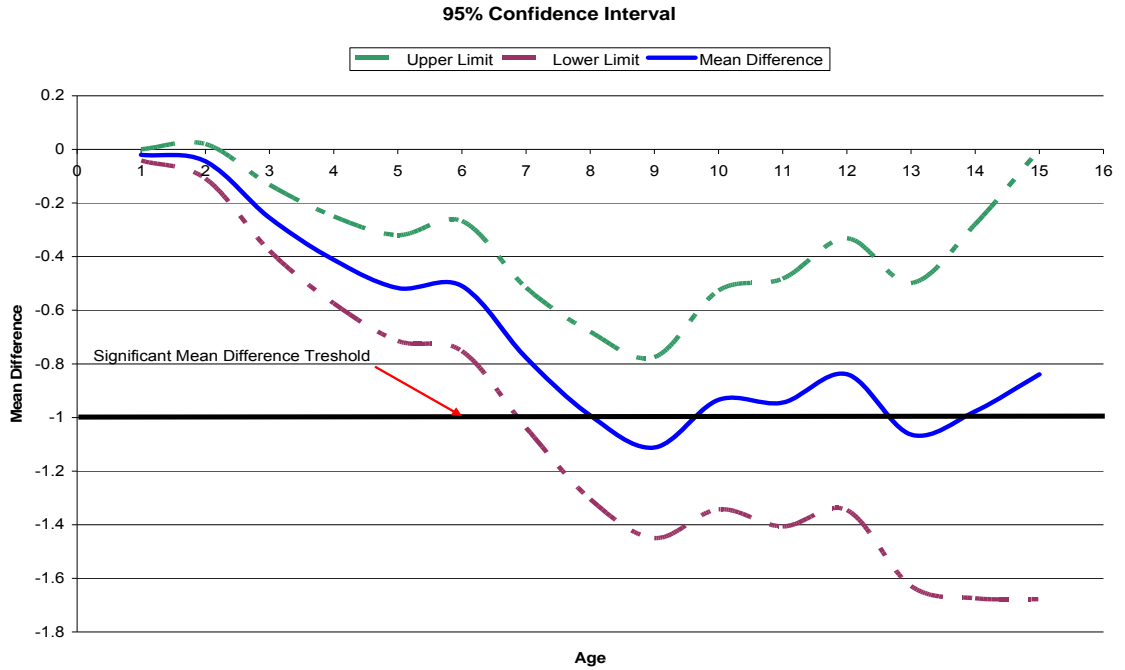
28

95% Confidence Interval

**Figure 6. Mean difference between structural deficient and traffic groups
(construction duty cycle =3)**

Developed TPMs

Now that it is proven there is a difference between the deterioration rates of
sections depending on the cause of deterioration, different TPMs can be developed to
represent each category. The matrices represented in Equations 20 through 23 are the
TPMs developed based on the methodology explained in prior sections. Equations 20 and
21 are TPMs for structurally deficient sections operating at their 2nd and 3rd cycle
respectively, and Equations 22 and 23 are TPMs for excessively trafficked sections
operating at their 2nd and 3rd cycle respectively.

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$$
\begin{pmatrix}
0.89188 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0.09947 & 0.68699 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0.28838 & 0.59811 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0.39825 & 0.44981 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.54977 & 0.40018 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0.60010 & 0.35021 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0.65014 & 0.30014 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0.70009 & 0.25007 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.75004 & 0.15002 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.85001 & 1
\end{pmatrix}
\tag{20}
$$

$$
\begin{pmatrix}
0.89371 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0.09796 & 0.69009 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0.29086 & 0.59963 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0.39942 & 0.45054 & 0 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0.55033 & 0.40060 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0.60041 & 0.35043 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0.65029 & 0.30024 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0.70015 & 0.25010 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.75006 & 0.15003 & 0 \\
0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.85002 & 1
\end{pmatrix}
\tag{21}
$$

$$\begin{pmatrix} 0.90143 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0.10496 & 0.69269 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.29350 & 0.59911 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.39918 & 0.44992 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.54991 & 0.40009 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.60005 & 0.35010 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.65006 & 0.30006 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0.70004 & 0.25003 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.75002 & 0.15001 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.85001 & 1 \end{pmatrix} \tag{22}$$

$$\begin{pmatrix} 0.90399 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0.10708 & 0.69050 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.29128 & 0.59953 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0.39937 & 0.45042 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0.55026 & 0.40051 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0.60035 & 0.35036 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0.65024 & 0.30020 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0.70013 & 0.25009 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.75005 & 0.15003 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0.85001 & 1 \end{pmatrix} \tag{23}$$

In all cases, it can be seen that the CR values decrease sharply after they pass the beginning condition and flatten out once they reach the degradation threshold where the pavement cannot deteriorate significantly any further. This behavior agrees with the S-shape curve that is used to represent the degradation of pavement condition.

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Applicability of Developed TPMs

Before applying the developed TPMs on the data, some statistical analysis must be performed to verify the accuracy and applicability of the TPMs. In order to do so, the TPM predicted CR values are plotted against the average CR values of the small group of verification sections (Figures 7 through 10). Then, the mean square error is used to verify the accuracy of the developed TPMs. As it can be seen in the following figures, the errors were insignificant and negligible in term of crack rating; therefore, it can be concluded that the developed TPMs are representative of the actual trends that exist in the database.
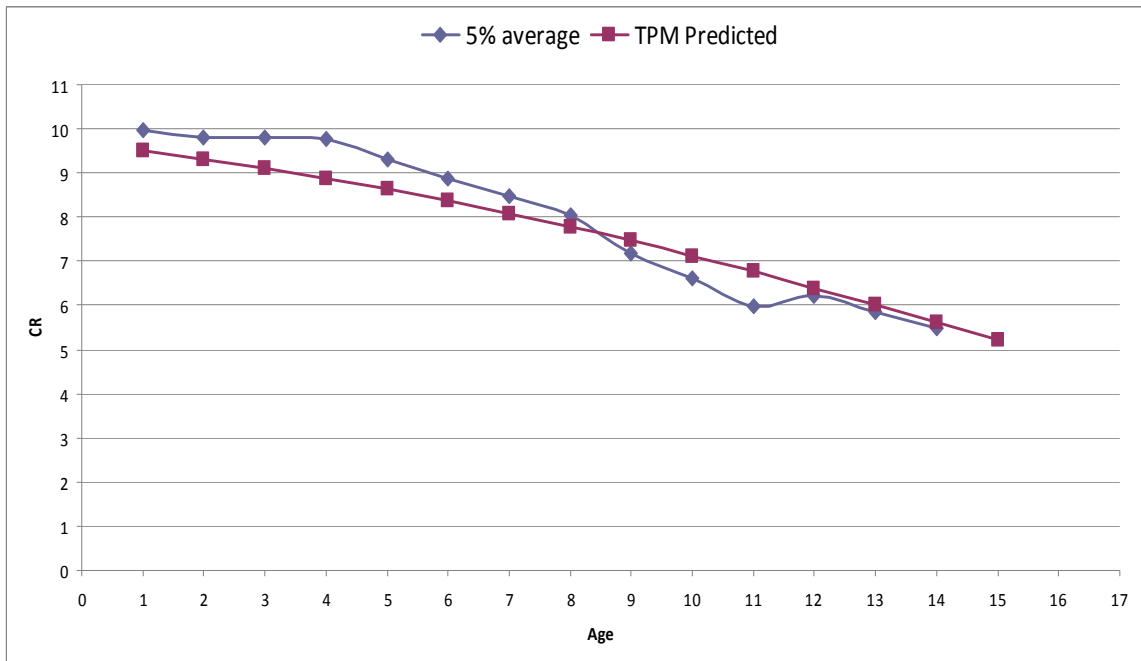

Figure 7-A. Comparison with the 5% verification set at their 2nd duty cycle (structural integrity deficient sections)

Figure 7-B. Comparison with the 10% verification set at their 2nd duty cycle (structural integrity deficient sections)

Figure 7-C. Comparison with the 20% verification set at their 2nd duty cycle (structural integrity deficient sections)

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Figure 8-A. Comparison with the 5% verification set at their 3rd duty cycle (structural integrity deficient sections)
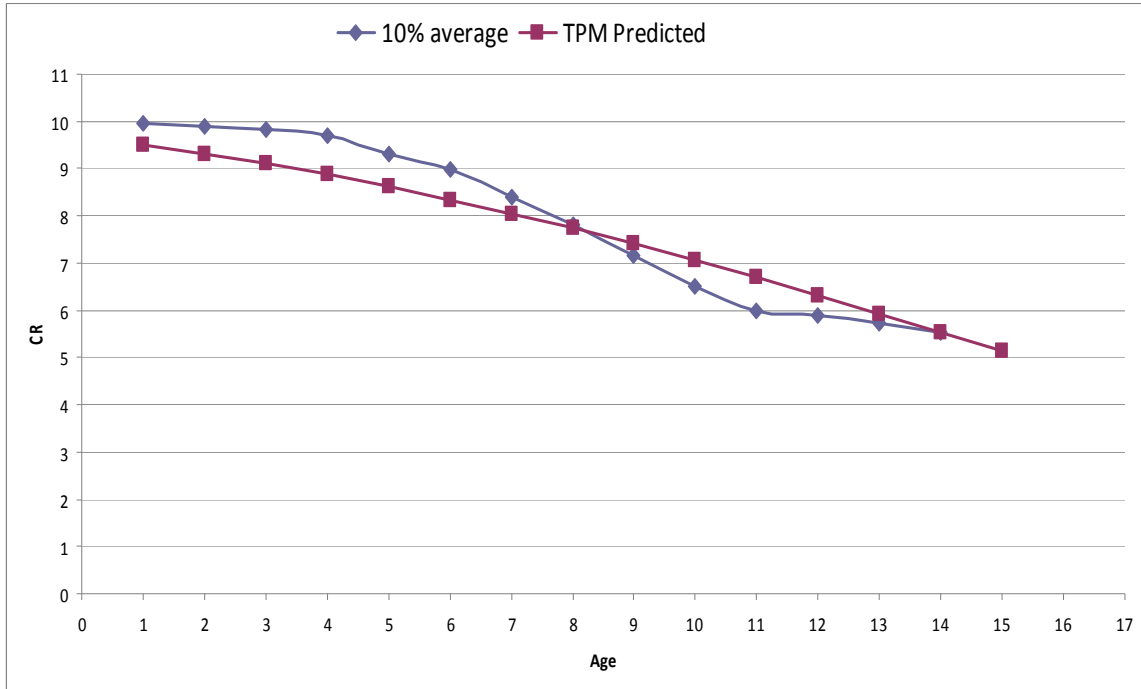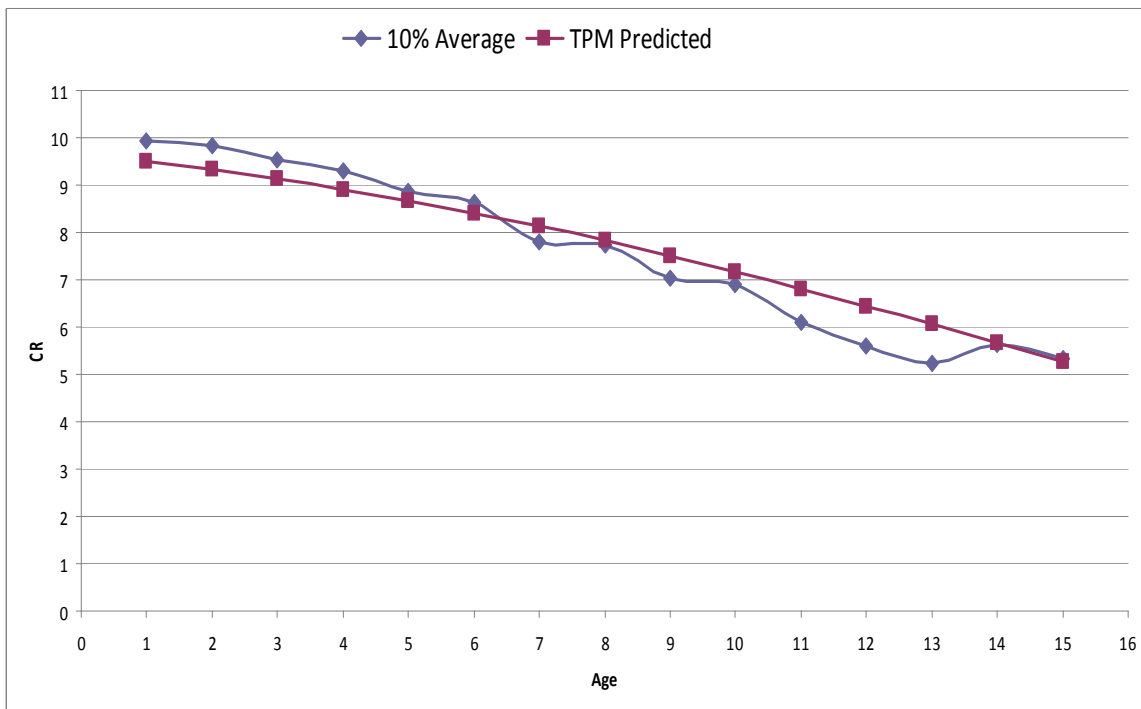

Figure 8-B. Comparison with the 10% verification set at their 3rd duty cycle (structural integrity deficient sections)
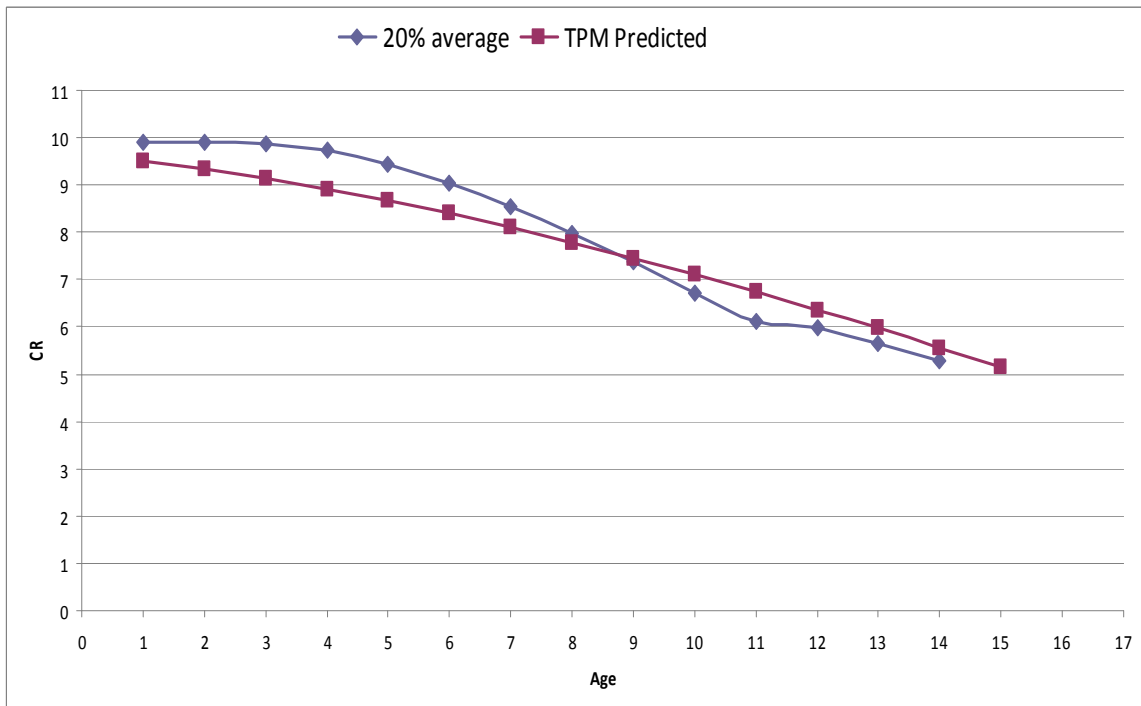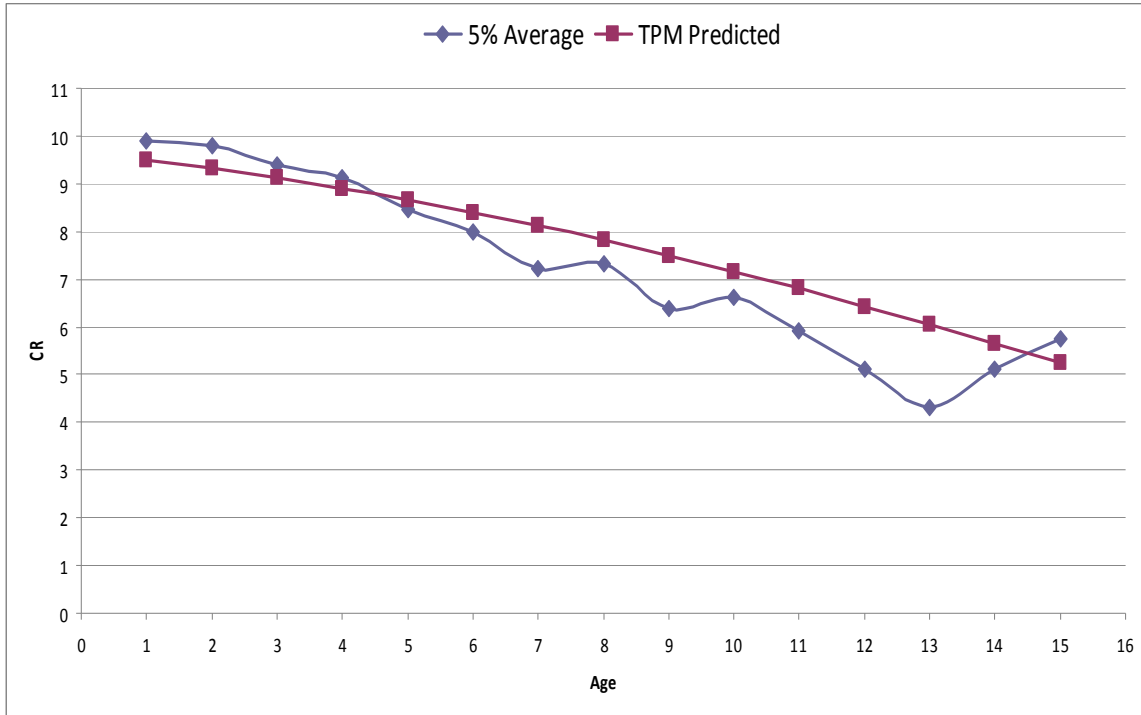
34
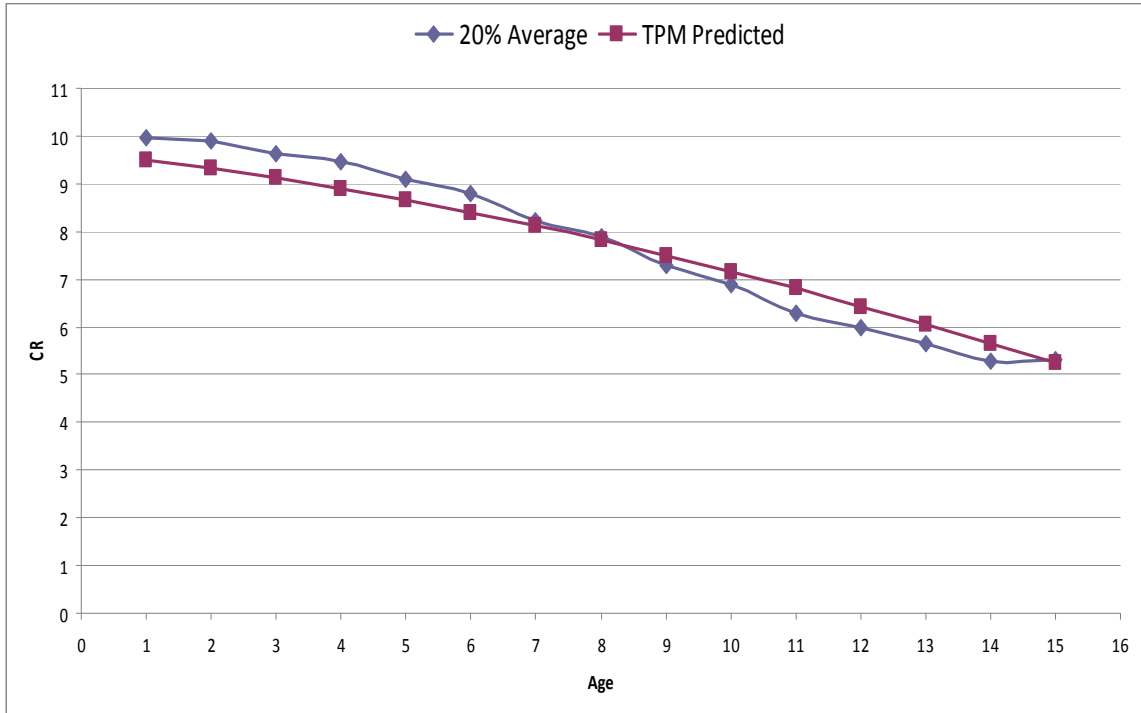
Figure 8-C. Comparison with the 20% verification set at their 3rd duty cycle (structural integrity deficient sections)

Figure 9-A. Comparison with the 5% verification set at their 2nd duty cycle (excessive traffic sections)

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Figure 9-B. Comparison with the 10% verification set at their 2nd duty cycle (excessive traffic sections)
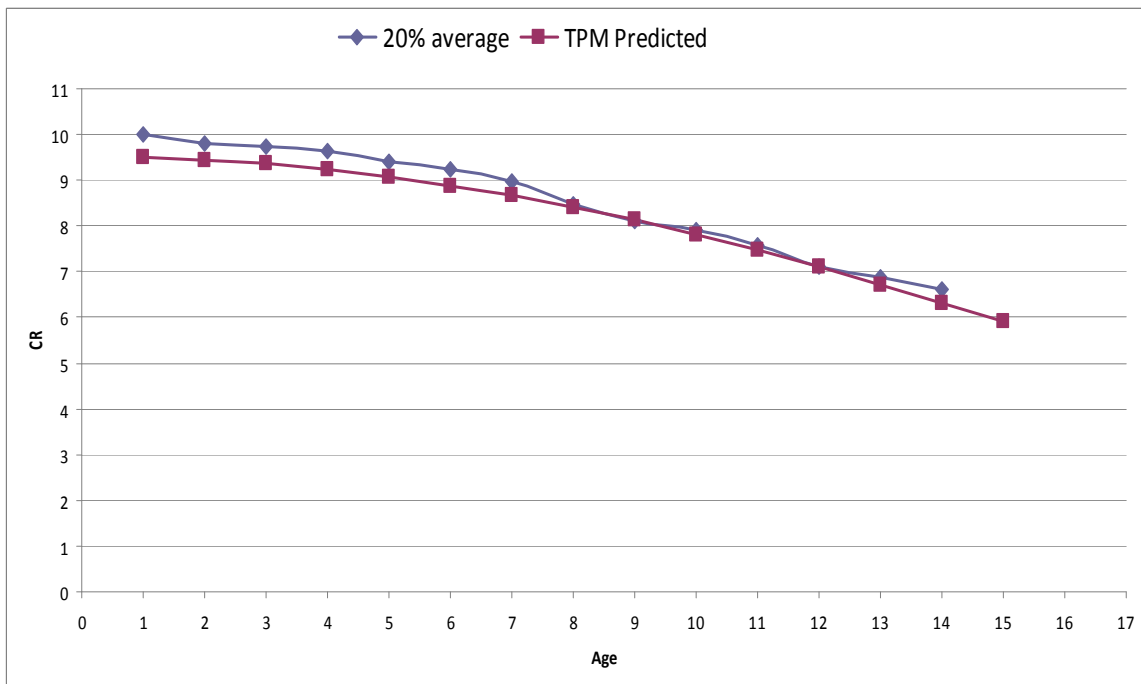
Figure 9-C. Comparison with the 20% verification set at their 2nd duty cycle (excessive traffic sections)

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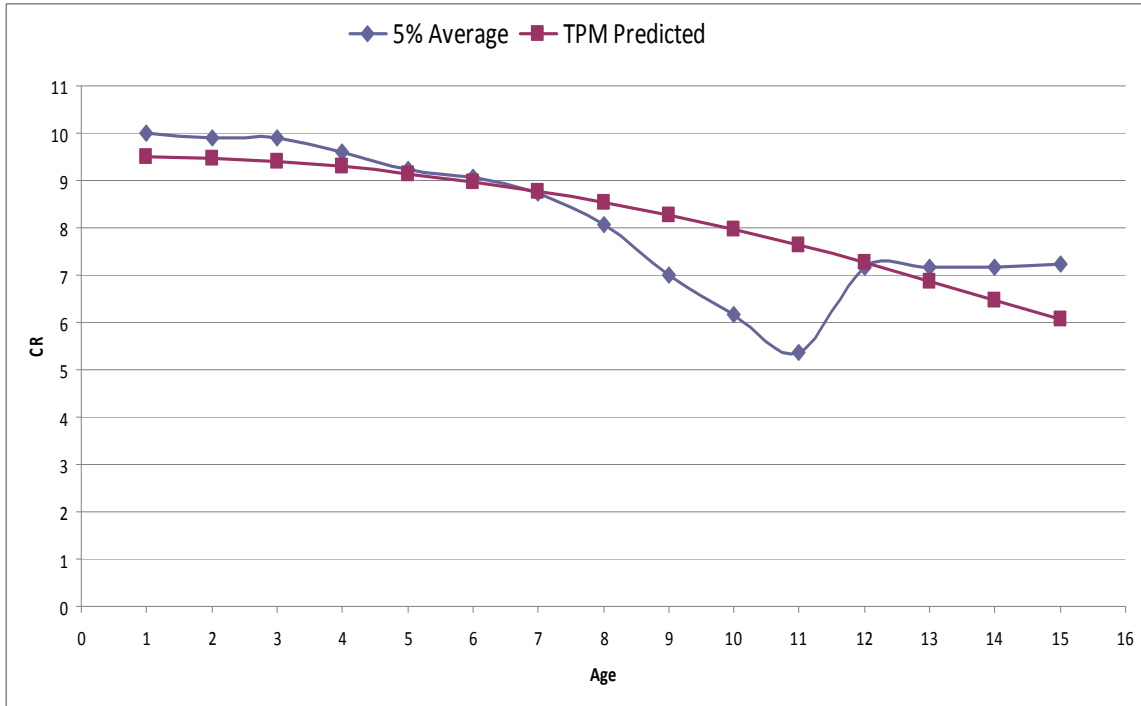
Figure 10-A. Comparison with the 5% verification set at their 3rd duty cycle (excessive traffic sections)
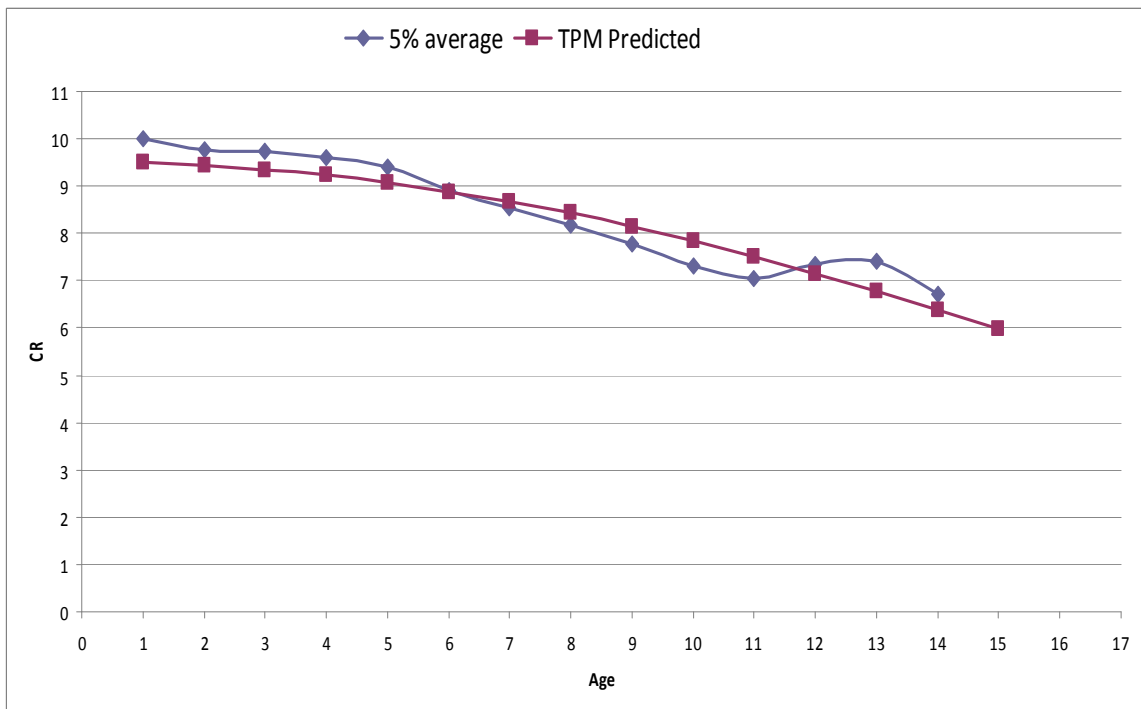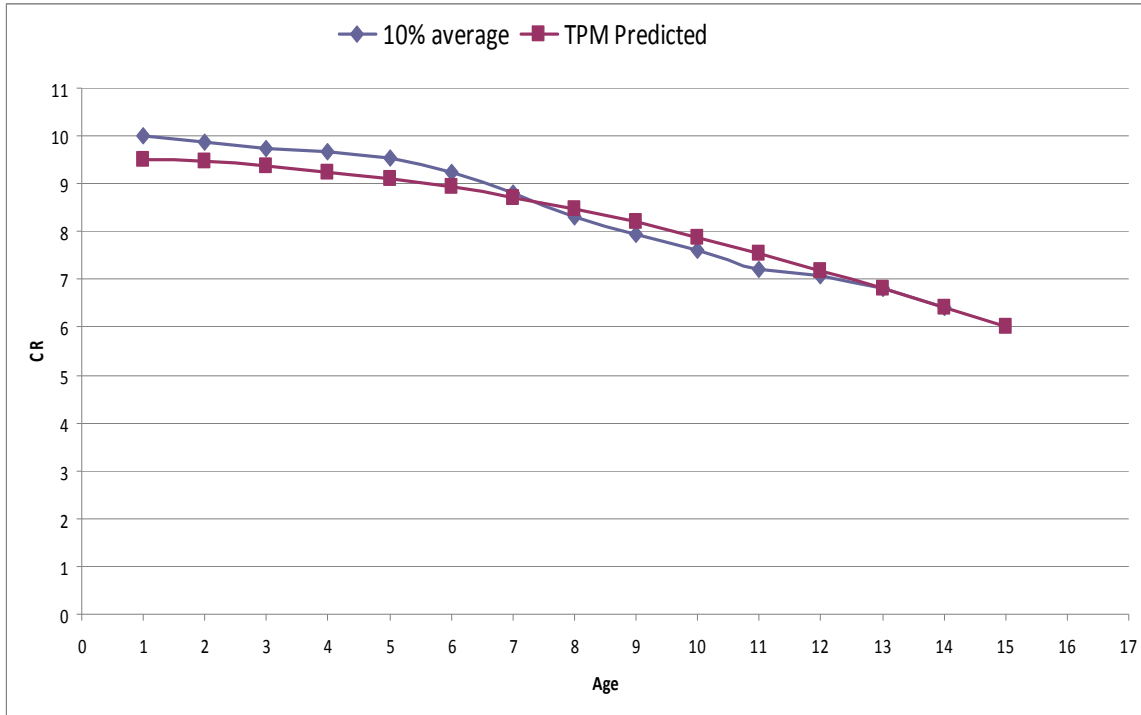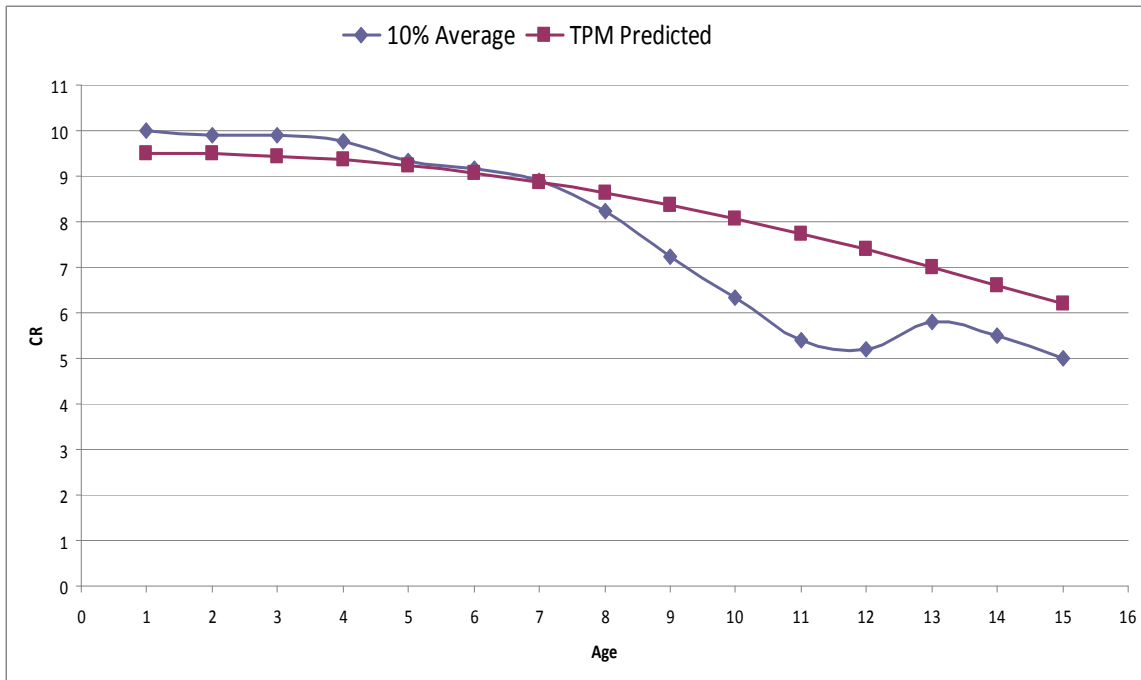

Figure 10-B. Comparison with the 10% verification set at their 3rd duty cycle (excessive traffic sections)

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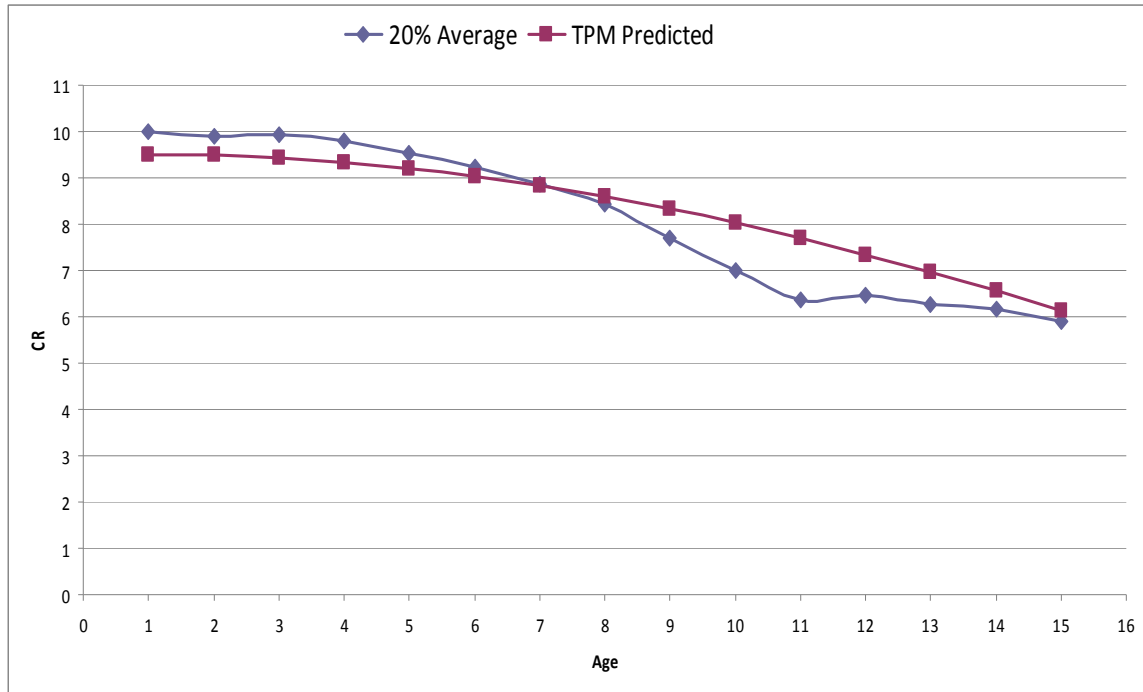
Figure 10-C. Comparison with the 20% verification set at their 3rd duty cycle (excessive traffic sections)

However, for each case an optimum verification group size should be determined to be applied in future studies. A mere comparison as presented in Table 6 cannot be deterministic because as it can be seen in the relative figures, some verification plots have undesired abnormalities. The abnormalities usually happen if the test batch is small (5% or 10%). This might be due to their small sample size, but once the sample size increases, the plots smoothen out.

Table 6. Comparison of Mean Square Error (MSE)

	Structural Integrity Deficient			Excessive Traffic		
	5%	10%	20%	5%	10%	20%
Cycle =2	0.267	0.314	0.272	0.143	0.088	0.082
Cycle =3	0.627	0.214	0.156	0.854	1.314	0.384

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Based on the review of the findings in Table 6 and Figures 7 through 10, the verification group size in Table 7 is recommended for application. The above recommendation was based on the logic that if the comparison study with a smaller test group would yield the same results as that with a larger group, the smaller test sample in fact would be adequate for verification, hence introducing less randomness to the study.

Table 7. Recommended verification group size

	Structural Integrity Deficient	Excessive Traffic
Cycle =2	10%	10%
Cycle =3	20%	20%

Application of Test Results

It can be seen from Figures 5 and 6 that the pavement sections currently performing in their 3rd construction duty cycles have a faster deterioration rate (in terms of cracking) compared to the pavement sections currently performing in their 2nd construction duty cycles. However, after reaching a threshold state (around a crack rating of 6) all the pavement sections, regardless of the cycle they are operating in, deteriorate with the same gradual rate. If further research can be performed on ensuing construction duty cycles (i.e. 4th and 5th cycle) and the similar results hold true (i.e. pavement sections performing in higher cycles deteriorate faster), then this observation can significantly impact the rehabilitation decisions. As the duty cycle of a pavement increases, its vulnerability to deterioration also increases.

Additionally, statistical analysis verified the applicability of the grouping and the development of individual TPMs. The study of two groups of structural integrity

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deficient and excessively trafficked pavements showed that not all the deterioration and crack propagation is due to traffic loading. Sometimes a pavement section with low traffic can reach a low crack rating state in a short time because of the poor sub-surface condition of that pavement at the time of overlay. The deteriorated condition of the sub-surface can induce bottom-up cracking since there are high stress areas at the distress locations underneath. The bottom-up cracking can propagate and intensify quicker than the top-bottom cracks since it feeds from two sources, loading on top and stresses at bottom. Next step is to demonstrate how this analysis can improve the engineering decision-making practice. The results show that the decision-making should not always focus on the highly trafficked sections (levels C, D, and E), but also consider the low trafficked sections that have experienced low crack ratings before rehabilitation. These sections have the potential to deteriorate faster at times; therefore, making them high priority candidates for maintenance and rehabilitation.

Knowing the cause of the low rating of a given pavement section is helpful when prioritizing the projects for rehabilitation. Based on the rating of a section and the probable cause (structural deficiency or excessive traffic), a pavement manager can use the prediction models based on the relevant transition probability matrices to determine and prioritize the severity of the sections in any desired time span. This would considerably help in budget optimization at the network level decision-making.

There are two important comparisons that can be made to check the applicability and accuracy of the developed TPMs with the actual data. FDOT has been using a linear regression computation to predict the CR for 5 years after the last CR entry of the database (currently at 2007). Four sections were chosen for comparison purposes each

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performing in a different cycle and belonging to different group as represented in Table 8 through 11.

Table 8. Prediction comparison of a structurally deficient section in its 2nd cycle

Section Characteristics	Roadway Direction	Roadway ID	Begin Milepost	End Milepost	CR 2007
	C	01040000	0	0.887	9
	TPM Predicted		FDOT Predicted		Condition State Difference
CR 2012	7.24		8.30		1

Table 9. Prediction comparison of a structurally deficient section in its 3rd cycle

Section Characteristics	Roadway Direction	Roadway ID	Begin Milepost	End Milepost	CR 2007
	C	01040000	0.887	1.47	8.5
	TPM Predicted		FDOT Predicted		Condition State Difference
CR 2012	6.27		7.16		1

Table 10. Prediction comparison of an excessively trafficked section in its 2nd cycle

Section Characteristics	Roadway Direction	Roadway ID	Begin Milepost	End Milepost	CR 2007
	L	70050000	9.956	15.158	9
	TPM Predicted		FDOT Predicted		Condition State Difference
CR 2012	7.58		8.28		1

Table 11. Prediction comparison of an excessively trafficked section in its 3rd cycle

Section Characteristics	Roadway Direction	Roadway ID	Begin Milepost	End Milepost	CR 2007
	R	36030000	0.652	2.606	6.5
	TPM Predicted		FDOT Predicted		Condition State Difference
CR 2012	3.48		5.68		2

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As it can be seen in Tables 8 through 11, the difference between the predictions is significant for pavement management purposes. As explained before, the condition state difference of one (1) is significant in pavement management to differentiate between the future condition of a sections. Moreover, in Table 11, this difference is two states. This can be due to the fact that the CR at this section is at a steep degradation stage of the S-curve deterioration trend. Therefore, depending on how the prediction method is developed, the forecasted CR can vary significantly from one prediction method to another. Although a conclusion cannot be drawn at this time, once the crack rating survey results are available, the accuracy of both methods can be checked.

Another important comparison can be made in the near future when the CR data for the year 2008 is recorded in the database. By comparing the actual data with the predicted values from the developed TPMs, the predictability of the developed TPMs would be verified. The 2008 predicted CR values for the same four sections selected above are shown in Table 12.

Table 12. Predicted CR values for 2008

Section ID	01040000	01040000	70050000	36030000
CR 2007	9	8.5	9	6.5
CR 2008	8.7	8	8.8	6

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<center>**Chapter Four**</center>

<center>**Conclusions and Limitations**</center>

Conclusions

As explained in the Chapter Two (Experimental Methodology), two distinct pavement groups have been identified in this work: structural integrity deficient group and excessively trafficked group. Also, for each group two different TPMs are developed based on the current construction duty cycle of the pavement sections (cycles 2 and 3). In this study, the two criteria used to group the data, the deterioration cause and current duty cycle, were proven to be statistically significant. By comparing the deterioration rates the two study groups of excessive trafficked sections and structural integrity deficient sections, it was proven statistically that the latter group, the pavement sections that have lower CR at the time of rehabilitation and low traffic volume (ESAL < 3,000,000), tend to deteriorate faster than the pavement sections that have a higher CR value at the time of rehabilitation and high traffic volume (ESAL > 3,000,000). This can be attributed to the degraded strength and support of the underlying pavement layers. The significance and applicability of this result is evident in pavement management decision-making where projects are prioritized for maintenance and rehabilitation. Based on the above findings, the sections that have low traffic but low crack ratings at the time of rehabilitation must

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also be considered as top priorities. To illustrate this point, two sections operating close to their crack threshold can be considered: one with excessive traffic and another with exposure to severe low crack levels in earlier cycles. By using the results from this thesis, it can be concluded that the latter section will deteriorate faster and its delayed rehabilitation might be much more costly and in extreme cases even impractical. This conclusion is somewhat contrary to the usual tendency of agencies to prioritize the rehabilitation of an excessively trafficked section. Also, by using the relevant TPM, the remaining life span of a section can be estimated more accurately until reaches its threshold. This would help the agencies to appropriate the budget based on the order in which the sections would reach their crack threshold level.

In general, it was observed that the pavement sections currently performing in their 3rd construction duty cycles have a faster deterioration rate (in terms of cracking) as compared to the pavement sections currently performing in their 2nd construction duty cycles. The reason for this phenomenon can be that after each rehabilitation and as the pavement ages, the pavement materials get fatigued which ultimately lead to a faster deterioration rate. This fact is also crucial when prioritizing the projects in that pavements in their 3rd duty cycle should be prioritized over the pavement sections currently in their 2nd duty cycle.

Although the developed TPMs for each group seem to be approximately equal to each other, once they are applied to a pavement's life span, the differences would accumulate and the predicted difference in behavior in the two groups would be evident. Also, the confidence interval for the predicted crack rating grows with the age. This phenomenon can be explained by considering the randomness involved in the rating and,

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the variation of the pavement condition with age. This suggests that predicting the future condition of pavement sections in the 3rd cycle is a more challenging task compared to that of pavement sections in the 2nd cycle.

Overall, it can be concluded that grouping the pavement sections based on the degradation cause and developing relevant individual TPMs is a more accurate mean of predicting pavement behavior. In this manner, instead of applying the same TPM to predict the future condition of all sections of a database, more specific and appropriate TPMs can be developed for enhanced condition prediction.

Limitations

Access to an up-to-date database is the key for successful grouping and prediction. If the available data are limited to certain areas or specific time periods, it would not result in accurate prediction. As an example, in order to extract and filter the sections suitable for the structurally deficient group from the PCS database, it is necessary to know the crack rating of the year the rehabilitation was performed. Same need holds true for the excessively trafficked group of pavements with the availability of all of the necessary traffic information.

Statistical analysis is based on acceptance of normality of the data set. This assumption is justified by the large number of samples used in this study under the applicability of the Central Limit Theorem to them. If a small sample is available for analysis, the normality should be checked or other appropriate approximation must be used.

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Appendices

Appendix A: History of Florida Pavement Condition Survey

(1973 –2006)

Revised 09/24/2006

1973   Complete Flexible Pavement Survey performed by the Districts.

1974   Complete Flexible Pavement Survey performed by the Districts.

1975   Complete Flexible Pavement Survey performed by the Districts.

1976   Complete Flexible and Rigid Pavement Surveys performed by the Districts 1. Rigid pavement survey was newly added.

1977   Complete Flexible and Rigid Pavement Surveys performed by the Districts.

1978   Complete Flexible and Rigid Pavement Surveys performed by the Districts.

1979   Complete Flexible and Rigid Pavement Survey by the Districts.

1980   No survey was performed due to change over in equipment – Mays Ride Meters originally mounted in survey vehicles were to mounted on Standard Trailers.

1981   Complete Flexible and Rigid Pavement Surveys performed by the Districts. 1) Flexible had ride values above 100 - no upper limit. 2) First survey using PCR's and trailers. 3) Number of Lanes was added to the survey data collection table.

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1982   Complete Flexible and Rigid Pavement Surveys performed by the Districts.

  1) Pavement Type 7 & 8 were added to the Flexible PCS - Districts 3 & 5 did not

  use these codes. 2) Started calculated Ride between 1 & 5 if section was too short

  to test to prevent basic ratings of 0.

1983   Complete Flexible and Rigid Pavement Surveys performed by the Districts.

  1) Pavement Type 7 & 8 were used by all Districts. 2) Procedure for calculating

  Ride was included in the manual. 3) Roadway 4 code was added for two- lane

  roads to give the direction surveyed.

1984   Complete Flexible and Rigid Pavement Surveys performed by the Districts.

  1) Defect on sections with a basic rating below 60 remaining section adjusted

  from 1983 survey. 2) Ride was not evaluated; Ride ratings were adjusted from

  the 1983 survey data.

1985   Flexible and Rigid Pavement Surveys included Ride only. 1) BM&R tested

  Districts 2, 3 and 5. 2) BM&R assisted with Districts 4 and 6. 3) District 1

  conducted the District survey. 4) District 3 rated I-10 rigid for defects. 5) BM&R

  collected Ride values on the rigid pavement of I-10. 6) Defect ratings were

  adjusted from the 1984 survey data.

1986   Complete Flexible and Rigid Pavement Surveys performed by BM&R personnel.

  1) 3 ruts per mile. 2) ADT was eliminated. 3) Adjusted ratings were eliminated.

  4) District 3 personnel rated own rigid pavements. 5) Survey was started in the

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second week of September 1985. 6) Survey was completed in the first week of
September 1986. 7) BM&R personnel rated one section of rigid pavement per
county in District 3 (Interstate) as a verification of the rigid survey. 8) Type 6
code was added to survey to reflect No Ride. Ride value will match defect. 9)
Added Crack Type to Flexible Survey: A = Alligator, B = Block, or C =
Combination. 10) Flexible Miles Rated 15,468.834, Rigid Miles Rated 96.923
Total Miles Rated 15,565.757. Flexible Miles Represented 32,937.004. Rigid
Miles Represented 277.744. Total Miles Represented 33,214.748. 11) Flexible
sections rated 5,765. Rigid sections rated 47. Total sections rated 5,812.

1987   Complete Flexible and Rigid Pavement Surveys performed by BM&R personnel.
1) Changes made to computer programs - flexible edit, flexible compare, and
flexible difference. 2) Survey was started in the third week of September 1986,
and was completed in the last week of June 1987. 3) Verification of rigid
pavement survey in the District 3 was performed on seven sections of Interstate
10. 4) Flexible Miles Rated 16,333.001. Rigid Miles Rated 937.385. Total Miles
Rated 17,270.386. 5) Flexible Miles Represented 33,010.922. Rigid Miles
Represented 2,078.848. Total Miles Represented 35,089.770. 6) Flexible sections
rated 6,196. Rigid sections rated 398. Total sections rated 6,594.

1988   Complete Flexible and Rigid Pavement Surveys performed by BM&R personnel.
1) Survey was started in the third week of August 1987, and was completed in 1st

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week of May 1988. 2) Flexible Miles Rated 16,423.565. Rigid Miles Rated 939.608. Total Miles Rated 17,363.173. 3) Flexible Miles Represented 33,334.466. Rigid Miles Represented 2,064.341. Total Miles Represented 35,398.807. 4) Flexible sections rated 6,347. Rigid sections rated 401. Total sections rated 6,748.

1989   Complete Flexible and Rigid Pavement Surveys performed by the State Materials Office. 1) Type 5 (new construction) and Type 7 (new overlay) codes were added. 2) L (light), M(Moderate), and S(severe) codes were added in the Comments field to indicate the severity of up to 25% cracking.   3) Survey started 2nd week in June 1988, and was completed in 1st week of May 1989. 4) Flexible Miles Rated 16,715.302. Rigid Miles Rated 926.118. Total Miles Rated 17,641.420. 5) Flexible Miles Represented 33,875.971. Rigid Miles Represented 2,052.093. Total Miles Represented 35,928.064. 6) Flexible sections rated 6,476. Rigid sections rated 399. Total Sections Rated 6,875.

1990   Complete Flexible and Rigid Pavement Surveys by the State Materials Office. 1) Survey was started on 6/12/89 , and was completed on 05/02/1990. 2) Trailers were painted and reconditioned causing delay in survey schedule. 3) Added lanes to Type 9 (structures and/or exception) and Type 8 (under construction). 4) Flexible Miles Rated 17,087.904. Rigid Miles Rated 922.423. Total Miles Rated 18,010.327. 5) Flexible Miles Represented 34,684.121. Rigid Miles

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Represented 2,060.555. Total Miles Represented 36,744.676. 6) Flexible sections

rated 6,571. Rigid sections rated 407. Total Sections Rated 6,978.

1991   Complete Flexible and Rigid Pavement Surveys by State Materials Office.

1) Survey was started on 6/11/90, and was completed on 05/02/91.

2) Programming change to allow a menu driven data entry for Flexible Pavement

Survey. 3) A modified cracking method was added to Flexible Pavement Survey

for evaluation. 4) Added Type 0 to identify an exception, <u>not</u> state maintained, or

a duplicate roadway section evaluated under another county section number that

should be exceptions. 5) All verification reports completed on May 09, 1991. 6)

Survey on a 0 to 10 scale was introduced for Flexible and Rigid. 7) Flexible Miles

Rated 16,431.367. Rigid Miles Rated 912.414. Total Miles Rated 17,343.781.

8) Flexible Miles Represented 34,915.445. Rigid Miles Represented 2,009.968.

Total Miles Represented 36,925.413. 9) Flexible sections rated 6,456. Rigid

sections rated 397. Total Sections Rated 6,853.

1992   Complete Flexible and Rigid Pavement Surveys by State Materials Office.

1) Survey was started on 8/05/91, and was completed on 5/04/1992. 2) Ultrasonic

Profilers replaced Mays Ride Meters, Ride Rating (RR) = 99.7576 + (-0.1569 X

IRI) used until 1999 survey. 3) Rut Depth measured manually and with Ultrasonic

Profilers for comparison. 4) 0 to 10 scale implemented for Rut, Ride, and Defect

scale as new rating system selected by Pavement Management Committee. 5) Rut

scale changed to add 1 1/8" and 1¼" for 10 scale. 6) IRI reported for outside

wheel path only with no filtering .7) IRI converted to PSIsv (10 scale) through correlation to CHLOE Profilometer. Correlation combined all units at all speeds (30, 40 & 50 MPH) and for both wheel paths. 8) Number of lanes added to Type 9 code (State Maintained exception such as bridges, etc.). 9) Responsibility for HPMS sections added to Survey Personnel. 10) Rut depth (Ultrasonic) in 0.001 mile increments for interstate flexible system was added to mainframe database. 11) Ultrasonic Rut Depth was used for Rut rating (Flexible Pavement Survey). If Type 6 (No Ride) then Manual Rut Depth was used. 12) Cracking scale was adjusted from procedures manual to J=2.5 if confined to wheelpath (CW), and J=1.0 if outside of wheel path (CO). Adjustments made per Mr. Ken Morefield. 13) Flexible Miles Rated 16,504.153. Rigid Miles Rated 889.772. Total Miles Rated 17,392.183. 14) Flexible Miles Represented 35,402.349. Rigid Miles Represented 2,020.421. Total Miles Represented 37,422.770. 15) Flexible sections rated 6,726. Rigid rated section 394. Total sections rated 7,118.

1993   Completed Flexible and Rigid Pavement Surveys by State Materials Office personnel. 1) Survey started 7/06/92, and was completed on 4/22/1993. 2) Ultrasonic Rut Depth (Actual Values) were recorded in CC 44-47 in Team File and CC 60-63 in permanent file. 3) New instruction manuals flexible and rigid for the Pavement Condition Survey published April, 1993. 4) Released survey 5/28/1993. 5) Flexible Miles Rated 16,662.666.Rigid Miles Rated 861.677. Total Miles Rated 17,523.953. 6) Flexible Miles Represented 35,765.134. Rigid Miles

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Appendix A: (Continued)

Represented 1,959.640. Total Miles Represented 37,724.774. 7) Flexible sections rated 6,934. Rigid sections rated 389. Total sections rated 7,323.

1994 Completed Flexible Pavement by State Materials Office personnel 1) Survey started 6/07/93. 2) Instructions from Mr. Ken Morefield via Mr. L.L. Smith was to complete flexible survey by April 01, 1994. The rigid pavement will not be accomplished in 1994 in order to complete survey by April 01, 1994. 3) Completed survey field-work on February 3, 1994. 4) Released survey on February 21, 1994. 5) Flexible Miles Rated 16,766.683. Rigid Miles Rated 861.287. Total Miles Rated 17,627.970. 6) Flexible Miles Represented 36,065.275. Rigid Miles Represented 1,959.640. Total Miles Represented 38,024.915. 7) Flexible sections rated 7,026. Rigid rated section 387. Total sections rated 7,413.

1995 Completed Flexible and Rigid Pavement Survey by State Materials Office personnel. 1) Survey started 3/21/94. 2) Light moderate and severe raveling added to survey as separate identity. 3) Patching added to survey as separate identity. 4) Type 2 added to survey to reflect pavement improvements without complete overlay (Intersections overlays). 5) System coded under US number was changed to match system codes. 6) Completed survey field-work January 26, 1995. 7) Survey released on March 30, 1995. 8) HPMS - FHWA added primary and interstate system in one direction - Appendix J. 9) Produced PCS and HPMS Facts. 10) Flexible Miles Rated 16,879.704. Rigid Miles Rated 746.673. Total

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Miles Rated 17,626.377. 11) Flexible Miles Represented 36,390.738. Rigid Miles

Represented 1,738.909. Total Miles Represented 38,129.647. 12) Flexible

sections rated 7,078. Rigid rated section 347. Total sections rated 7,425.

1996   Completed Flexible and Rigid Pavement Survey by State Materials Office

personnel. 1) Survey started 3/27/95. 2) Survey field-work completed 1/17/96. 3)

Survey released 3/05/96. 4) Flexible Miles Rated 17,027.506. Rigid Miles Rated

718.910. Total Miles Rated 17,746.416. 5) Flexible Miles Represented

37,018.830. Rigid Miles Represented 1,694.010. Total Miles Represented

38,712.840. 6) Flexible sections rated 7,209. Rigid rated section 337. Total

sections rated 7,546.

1997   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) Survey started 3/22/96, and was completed on 1/16/97. 2) Survey released

3/05/97. 3) Flexible Miles Rated 17,121.634. Rigid Miles Rated 692.277. Total

Miles Rated 17,813.911. 4) Flexible Miles Represented 37,307.869. Rigid Miles

Represented 1,603.559. Total Miles Represented 38,911.428. 5) Flexible sections

rated 7,429. Rigid rated section 329. Total sections rated 7,758.

1998   Completed Flexible and Rigid Pavement Surveys by State Materials Office. 1)

Survey started 3/17/97, and was completed on 1/13/98. 2) Survey released

4/01/98. 3) Flexible Miles Rated 17,201.156. Rigid Miles Rated 681.677. Total

Miles Rated 17,882.833. 4) Flexible Miles Represented 37,572.317. Rigid Miles

Represented 1,592.399. Total Miles Represented 39,164.716. 5) Flexible sections

rated 7,524. Rigid rated section 330. Total sections rated 7,854.

1999   Completed Flexible and Rigid Pavement Surveys by State Materials Office. 1)

Survey was started on 03/30/98, and was completed on 01/12/99. 2) Survey was

released on 3/22/99. 3) Flexible Miles Rated 17,314.411. Rigid Miles Rated

622.325. Total Miles Rated 17,976.736. 4) Flexible Miles Represented

37,925.623. Rigid Miles Represented 1,566.420. Total Miles Represented

39,492.043. 5) Flexible sections rated 7,652 Rigid rated section 322 Total sections

rated 7,974. 6) Converted to laser profilers. 7) Used Ride Number (RN) times 20

for ride rating. Ride number was based on rate 4 filtered to 300 foot wavelength

from the outside wheel path. 8) Started using laser profiler for ride acceptance

Rate 2 Ride Number (RN) filtered to 300 foot. 9) Warranty specification

implemented this year.

2000   Completed Flexible and Rigid Pavement Surveys by State Materials Office. 1)

Survey was started on 03/22/99, and was completed on 1/12/2000. 2) Survey

released on 3/24/2000. 3) Flexible Miles Rated 17,486.318. Rigid Miles Rated

605.559. Total Miles Rated 18,091.877. 4) Flexible Miles Represented

38,535.787. Rigid Miles Represented 1,476.148. Total Miles Represented

40,011.935. 5) Flexible sections rated 7,770. Rigid rated section 307. Total

sections rated 8,077. 6) Tested Forest Roads per Federal Highway Administration

request. Total miles rated 530.190. Total number of roads 74. 7) Tested HPMS off- system sections for first time Total miles rated 357.4. Total sections rated 262.

2001   Completed Flexible and Rigid Pavement Surveys by State Materials Office. 1) Removed Code for Type and leading zero from State Road number and U.S. Road Number field. 2) Allowed laser measured rut depths to be used on Type 6 (no ride) in lieu of manual measurements. 3) Survey started 03/27/2000, and was completed on 01/10/2001. 4) Survey released on 3/12/2001. 5) Flexible Miles Rated 17,624.341 Rigid Miles Rated 546.806 Total Miles Rated 18,170.190. 6) Flexible Miles Represented 38,831.473. Rigid Miles Represented 1,331.175. Total Miles Represented 40,162.648. 7) Flexible sections rated 7,782. Rigid rated section 302. Total sections rated 8,084.

2002   Completed Flexible and Rigid Pavement Surveys by State Materials Office. 1) Survey started on 04/02/2001, and was completed on 01/17/2002. 2) Added Ride Number to Rut Depth in 0.01 intervals. 3) Added R to indicate profiler reruns under verification codes. 4) Survey released 03/15/2002. 5) Flexible Miles Rated 17,898.876. Rigid Miles Rated 397.640. Total Miles Rated 18,296.516. 6) Flexible Miles Represented 39,428.791. Rigid Miles Represented 1,034.599. Total Miles Represented 40,463.390. 7) Flexible sections rated 7,777. Rigid sections rated 275. Total sections rated 8,052.

Appendix A: (Continued)

2003   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) Added Code for Raters to CC 85 & CC 86 of the Flexible AREA file. 2) Added Code for Raters to CC 52 & CC 53 of the Flexible PERMANENT file. 3) Survey was started on 03/25/2002, and was completed 01/08/2003. 4) Survey released 3/27/03. 5) Flexible Miles Rated 17,916.53. Rigid Miles Rated 369.94. Total Miles Rated 18,286.47. 6) Flexible Miles Represented 39,800.39. Rigid Miles Represented 978.44. Total Miles Represented 40,778.82. 7) Flexible sections rated 7,871. Rigid sections rated 267. Total sections rated 8,138. 9) Added rater codes to the area data set in CC 85 & 86. Not included in permanent data set 10) Added to the handbook that all lanes could be considered for overall crack rating (reflective of overall condition).

2004   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) For the 2004 Survey, the profile data is collected using a sampling rate of 6 inch compared to a 12 inch sample interval in previous survey years. 2) Survey started 03/24/2003, and was completed on 01/14/04. 3) Survey released 03/23/04 4) Flexible Miles Rated 18071.48. Rigid Miles Rated 368.24. Total Miles Rated 18439.72. 5) Flexible Miles Represented 40039.01. Rigid Miles Represented 976.94. Total Miles Represented 41015.50. 6) Flexible sections rated 7,884. Rigid sections rated 269. Total sections rated 8,153.

2005   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) PCS Started 03/29/04, and was completed on 12/15/04. 2) Flexible Miles Rated

18061.64. Rigid Miles Rated 363.08. Total Miles Rated 18424.71. 3) Flexible

Miles Represented 40380.77. Rigid Miles Represented 975.7. Total Miles

Represented 41356.48. 4) Flexible sections rated 7966. Rigid sections rated 261.

Total sections rated 8227.

2006   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) Completed SIS survey June 15, 2005 Miles Rated 185.431. 2) Completed

SCRAP/SCOP survey July 31, 2005 Miles Rated 882.672. 3) PCS Started

03/14/05, and was completed on 12/ 14/05. 4) Flexible Miles Rated 18251.53.

Rigid Miles Rated 364.39. Total Miles Rated 18615.91. 5) Flexible Miles

Represented 40788.13. Rigid Miles Represented 993.21. Total Miles Represented

41781, 45. 6) Flexible sections rated 8013. Rigid sections rated 271. Total

sections rated 8284.

2007   Completed Flexible and Rigid Pavement Surveys by State Materials Office.

1) All four survey vehicles are using Windows XP operating systems. 2)

Completed SIS survey 02/28/07 Miles Rated 204.919. 3) Completed

SCRAP/SCOP (08/14/2006) Miles Rated 1103.66. 4) PCS Field Work Started

03/20/06, and was completed on 12/19/2006 5) Survey released 03/21/07. 6)

Flexible Miles Rated 18328.929. Rigid Miles Rated 363.891. Total Miles Rated

18692.820. 7) Flexible Miles Represented 41191.490. Rigid Miles Represented

88.434. Total Miles Represented 42179.924. 8) Flexible sections rated 8199.

Rigid sections rated 270. Total sections rated 8469.