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16. Abstract

The goal of this project was to identify common features of good and poorly performing asphalt pavements. The types of asphalt pavements included in the study were grouped into four categories: new construction, crush & shape with HMA surface, mill & resurface, and HMA overlay. Deterioration relationships were used to identify roadway segments with good and poor performance based on three performance indicators: distress index (composed of multiple distresses), rut depth, and smoothness. In general, rut depth and smoothness were not the causes of premature pavement rehabilitation – the distress index was found to be the primary reason for poor performance. Longitudinal and transverse cracking were found to be the primary cause of poor performance. As such the study recommended four mitigation strategies to reduce the occurrence of these distresses – implementation of a longitudinal construction joint specification, biased sampling and testing based on the use of infrared camera during construction, revisions to the mixture design procedure (volumetric properties, number of gyrations, & fundamental performance test), and expanded use of higher quality wearing surfaces.

17. Key Words: Alligator Cracks; Crush & Shape; Demonstration Project; Deterioration Coefficients; Distress Index; Infrared Camera; IRI; Longitudinal Cracks; Mill & Resurface; Mitigation; Pavement Life; Pavement Performance; Pavement Preservation; Performance Indicators; Preventive Maintenance; Rut Depth; Smoothness; Temperatures; Transverse Cracks.	**18. Distribution Statement** No restrictions. This document is available to the public through the Michigan Department of Transportation.

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DISCLAIMER

The opinions and conclusions expressed or implied in the report are those of the research agency. They are not necessarily those of the Michigan Department of Transportation or Federal Highway Administration.

EXECUTIVE SUMMARY

Premature aging or accelerated distress of asphalt pavements costs agencies millions of dollars in maintenance and repair (M&R) costs each year to keep these pavements serviceable at a reasonable level. Identifying the causes of premature distress and taking corrective actions can save taxpayers millions of dollars, as well as reduce the number of roadway closures needed for M&R activities. Likewise, identifying pavements that exhibit good or exceptional performance and the features that contribute to this exceptional performance can increase the average service life of asphalt pavements, and thus, reduce life cycle costs (LCC).

The Michigan Department of Transportation (MDOT) has been tracking the performance and condition of all roadways for decades to understand their pavements performance characteristics, and have periodically evaluated their design, construction, and materials specifications to improve performance. To improve pavement performance and reduce life cycle costs (LCC), MDOT is using their pavement performance database to answer two basic but important questions:

1. Why do certain pavements fail to meet their specific design life?
2. Why do certain pavements exceed their specific design life?

The goal of this research project was to identify the common features of good and poorly performing asphalt pavements and HMA overlays. MDOT can then focus their efforts on specific features to improve pavement performance and reduce the number of roadway segments exhibiting premature distress.

Three performance indicators were used to categorize pavement performance: distress index (DI), rut depth, and International Roughness Index (IRI). The performance characteristics were defined by deterioration relationships for each performance indicator. The coefficients (regression constants) of the deterioration relationships were derived for each roadway segment using linear regression techniques to minimize the error between the predicted and measured performance indicator. These coefficients were determined for each roadway segment prior to and after the application of any preventive maintenance activity placed on that segment, as well as after preventive maintenance was applied to the pavement surface. The deterioration coefficients were then used to predict the time (age) to a value for each performance indicator. The following threshold values were used:

- Distress Index of 50.
- Rut depth of 0.40 in.
- IRI value of 120 in./mi.

The roadway segments were grouped by region (climate), pavement structure, roadway type, soil type, and traffic volume. The deterioration coefficients and estimated service life were used to categorize the performance of all segments included in MDOT’s performance database with sufficient data into those exhibiting good and poor (premature distress) performance. The detailed distress data included in MDOT's performance database were also used to determine the magnitude and severity of the individual distresses for those roadway segments categorized into poor and good performance. The detailed distress data were used to determine if construction and material parameters, not recorded in the MDOT pavement performance database, were the probable cause for the distress or poor performance. The following summarizes the findings and conclusions from the study.

- Most preventive maintenance strategies used in Michigan have provided enhanced performance for HMA pavements, as well as HMA overlays and other rehabilitation strategies. This management policy should be continued, because the preservation dollars provide a benefit to the Michigan taxpayers. The preservation strategies providing enhanced service lives, on the average, are: the cold-mill and resurface (7 years), thin and ultra thin HMA overlays (6 years), and micro-surfacing (5 years). Chip seals were found to provide only minimal added service life (3 years). Thus, the preventive maintenance policies and strategies that have been used by MDOT should be continued. It was recommended that MDOT restrict the use of chip seals to specific low volume roads with adequate structural support, and sponsor a materials research study for improving their performance.

- The maintenance activities of crack fill, overband crack fill, and crack treatment were found to have little to no effect on reducing or slowing the progression of the performance indicators after their application.

- Rutting was found to be very low and insignificant, with the exception of a few roadway segments. Department policies that have been implemented for the past 10 to 15 years have significantly mitigated the issue of rutting.

- IRI is considered low for many of the roadway segments along the freeways. On the average, the non-freeway segments were found to have about 20 percent higher IRI values than for the freeway segments.

- The distress index was found to be the predominate reason for maintenance and/or rehabilitation using the threshold values listed above. The detailed distress data was used to determine the individual distresses that were commonly recorded on roadway segments falling in the category of poor performance. Roadway segments falling in the poor performance category were found to exhibit excessive longitudinal centerline cracks, longitudinal center lane cracks, longitudinal wheel path cracks, edge cracks, alligator cracks, block cracks, and/or transverse cracks and tears.

The following lists the mitigation strategies recommended for implementation from this study.

- MDOT's preventive maintenance policy and strategies should be continued.

- A longitudinal construction joint specification should be implemented and used during construction to reduce the deterioration along longitudinal construction joints.
- Revise the mixture design procedure and material requirements. This includes lowering the number of N-design gyrations for both high and low volume roadways to ensure adequate mixture strength and durability, and using fewer gap-graded mixtures that are not polymer modified. The reduction in number of gyrations should be determined through a pilot study. Another mixture related strategy is to use higher quality wearing surfaces for high volume roadways; like stone matrix asphalt (SMA) and polymer modified asphalt (PMA) mixtures. The purpose of this strategy is to increase the effective asphalt content by volume in the mixture, improving on the durability of the mixture, and to use more PMA or SMA mixtures, especially for higher volume roadways.
- Increased inspection and biased sampling and testing requirements at the beginning of a project to confirm adequate densities near the center and other locations of the paver. Infrared cameras for biased sampling and testing during construction should be implemented, at least during the start of HMA paving operations, to reduce the amount of center lane longitudinal and edge cracking.
- Wearing surfaces with enhanced mixture properties should be used on high volume roadways to reduce surface deterioration in the form of transverse cracks and tears, alligator cracks, and longitudinal cracks in the wheel path. These surface mixtures include stone matrix asphalt and polymer modified asphalt.
- The other more long term mitigation strategy related to mixture design is to implement a fundamental test to be used during mixture design. This strategy is to include a fundamental test or torture test to confirm the HMA volumetric mixture design. The above mitigation strategy was recommended in parallel – revision to the HMA mixture design procedure by reducing the number of gyrations to select the target asphalt content. These mitigation strategies are more of a long term recommendation.

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EXTENDING THE LIFE OF ASPHALT PAVEMENTS

Michigan DOT Project # OR09086A

PART I

Final Report

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Extending the Life of Asphalt Pavements

CHAPTER 1 INTRODUCTION

1.1 Background

Premature aging or accelerated distress of asphalt pavements costs agencies millions of dollars in maintenance and repair (M&R) costs each year to keep these pavements serviceable at a reasonable level. Identifying the causes of premature distress and taking corrective actions can save taxpayers millions of dollars, as well as reduce the number of roadway closures needed for M&R activities. Likewise, identifying pavements that exhibit exceptional performance and the features that contribute to this exceptional performance can increase the average service life of asphalt pavements, and thus, reduce life cycle costs (LCC). The Michigan Department of Transportation (MDOT) recognizes the potential benefits and wants to increase the average service life of their roadways, thereby reducing LCC and making the limited tax dollars go further in maintaining and managing their roadway network.

MDOT has been tracking the performance and condition of all roadways for decades to understand their pavements performance characteristics, and have periodically evaluated their design, construction, and materials specifications to improve performance. To improve pavement performance and reduce life cycle costs (LCC), MDOT is using the pavement performance database to answer two basic but important questions:

1. Why do certain pavements fail to meet their specific design life?
2. Why do certain pavements exceed their specific design life?

The goal of this research project was to identify the common features of good and poorly performing asphalt pavements and HMA overlays. MDOT can then focus their efforts on specific features to improve pavement performance and reduce the number of roadway segments exhibiting premature distress.

1.2 Project Objective

The objective of this research project is to provide MDOT with recommendations to reduce the number of roadway segments exhibiting premature aging/distress and increase the average service life of asphalt pavements. To meet that objective, four research activities or tasks were accomplished:

1. Determine factors contributing to premature aging or extended life for asphalt pavements.
2. Identify the most common and severe trends in premature aging.
3. Propose mitigation strategies to combat deterioration.
4. Develop recommendations for implementing beneficial strategies and design a testing program for other potentially beneficial strategies.

This report documents the work completed to accomplish the first two tasks or answer the question – why do certain pavements fail to meet or exceed their design life; while the Implementation Plan accomplishes the last two tasks.

1.3 Scope of Report

This research report documents the work completed to determine the factors contributing to premature aging and extended life of asphalt pavements, and identifies the common and severe trends in premature aging. The report is grouped into six chapters; including the Introduction to the project, defined as Chapter 1. The other five chapters to the research report are listed and defined below.

- Chapter 2 is a summary of the performance indicators that were used in the study to determine the performance characteristics and trends of asphalt pavements. The performance indicators are ones monitored by MDOT in managing their roadway network.

- Chapter 3 provides a summary of the data analyses completed to determine the average service life of asphalt pavements, with and without preventive maintenance strategies that are used in Michigan to extend service life.

- Chapter 4 presents the relationships used to determine the deterioration rates for defining pavements with good or exceptional and poor or inferior performance.

- Chapter 5 includes a review and analysis of the detailed distress data used to determine the distress index and identify the causes for premature aging and distress. It also includes strategies to mitigate the occurrence of premature distress.

- Chapter 6 is a summary of the conclusions and recommendations from this project, including the mitigation strategies that MDOT can quickly implement to increase the average service life and reduce premature distress.

CHAPTER 2 PERFORMANCE INDICATORS

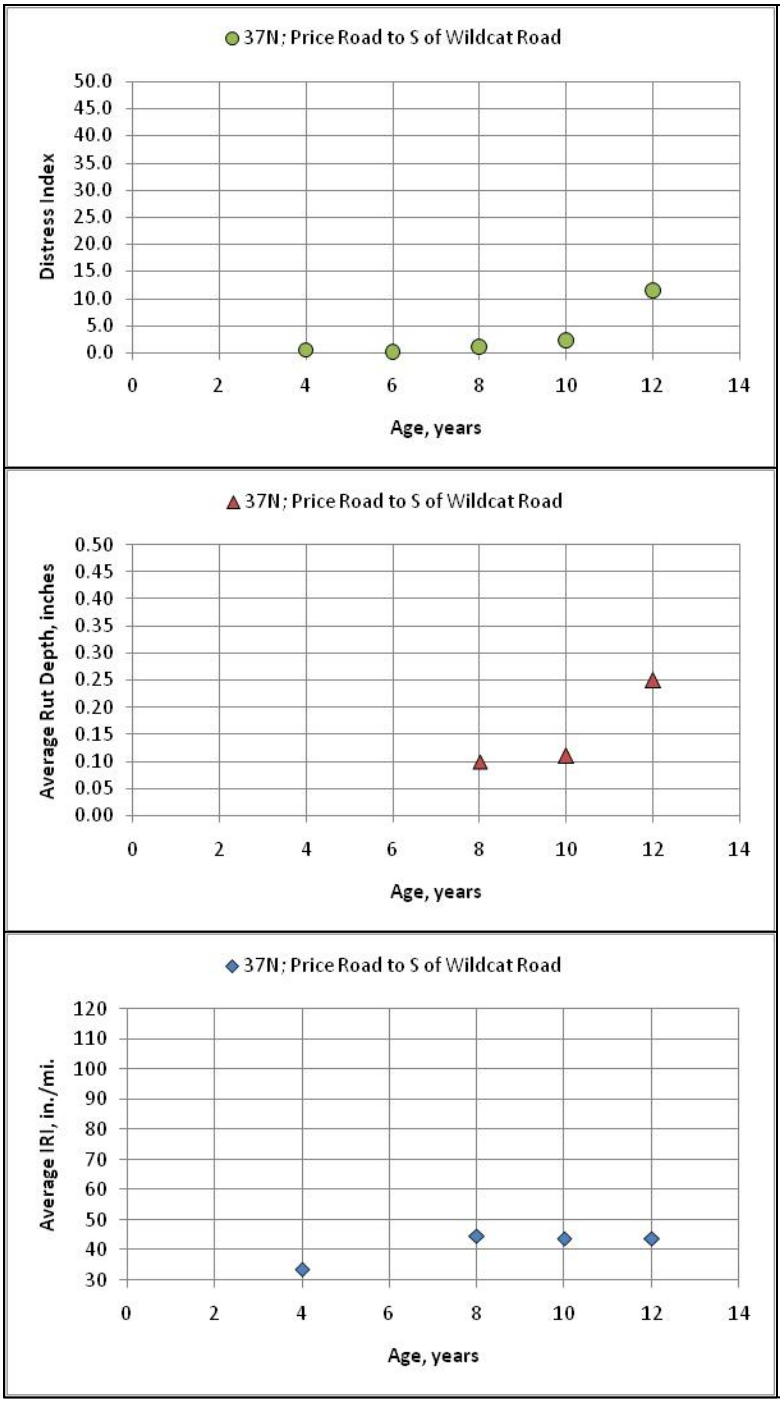
Three performance indicators are monitored by the MDOT to evaluate the performance and timing for rehabilitation of flexible pavements and hot mix asphalt (HMA) overlays. These include; distress index (DI), rut depth, and smoothness (as measured by the International Roughness Index [IRI]). MDOT provided the average values measured over time for each performance indicator along the roadway segments, as well as the detailed data measured within each roadway segment.

The roadway segments in MDOT's performance database are defined by a control section (CS) number and job number (JN) for each project. The job numbers can vary within a control section when preventive maintenance activities are applied to different areas along the same segment of roadway. The length and limits (defined by mile points) of different repair activities within each section along the roadway are also provided. For the remainder of this report a control section or a continuous segment of roadway for which an average performance indicator value is reported is referred to as a pavement management (PM) segment.

Figure 1 shows an example of the change in the average value for each performance indicator over time for one of the roadway segments. These performance indicators were used within this study to determine the expected service life and pavement deterioration parameters of separate data sets within MDOT's database.

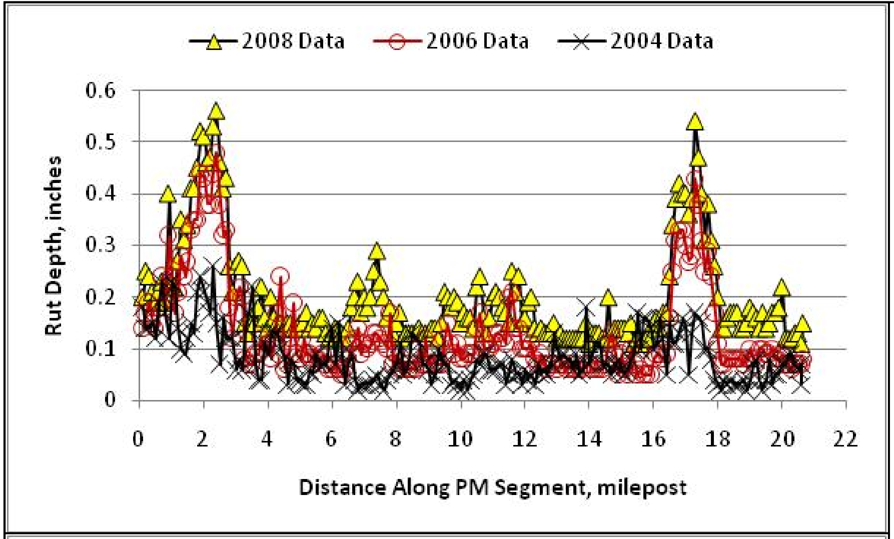
Detailed data are also stored by MDOT and grouped by region, pavement structure, and highway classification. Figures 2 and 3 include examples of the detailed data measured over time along selected control sections. As shown, the locations with the higher IRI values and rut depths are fairly consistent from year to year within the same control section. The actual values measured within the section, however, can be highly variable or abruptly change within the section. In addition, areas with the higher rut depths do not necessarily exhibit higher IRI values or rougher pavements. The detailed data were used to identify reasons or explain abrupt changes in the average values over time, and to identify those sections with high levels of deterioration in localized areas.

MDOT focuses on the use of the DI values for determining when to apply preventative maintenance or preservation activities. Table 1 summarizes the DI values and age that were extracted from the Michigan DOT *Pavement Design and Selection Manual* (March 2005). An analysis was initially completed to determine if there was correspondence between the different performance indicators for the control sections. In other words, do the IRI values consistently increase with increasing DI and rut depth values, or do the average rut depths decrease with lower DI values? Figures 4 through 7 are scatter plots that compare the performance indicators for different data sets using the pavement structural categories established by MDOT. As shown, there is no correspondence between the performance indicators, so each performance indicator is considered independent to the other values in the analysis.

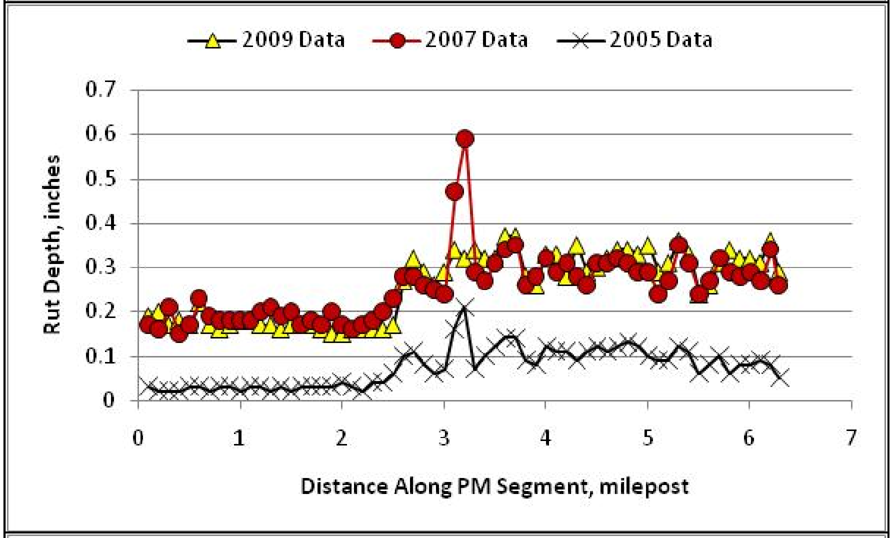


PM Segment #37N, Control Section 19033; US 127, Northbound Lane; Flexible Pavement; New Construction or Reconstruction; Freeways/Divided Highway; Overband Crack Fill (OCF) placed along roadway at age year 7.

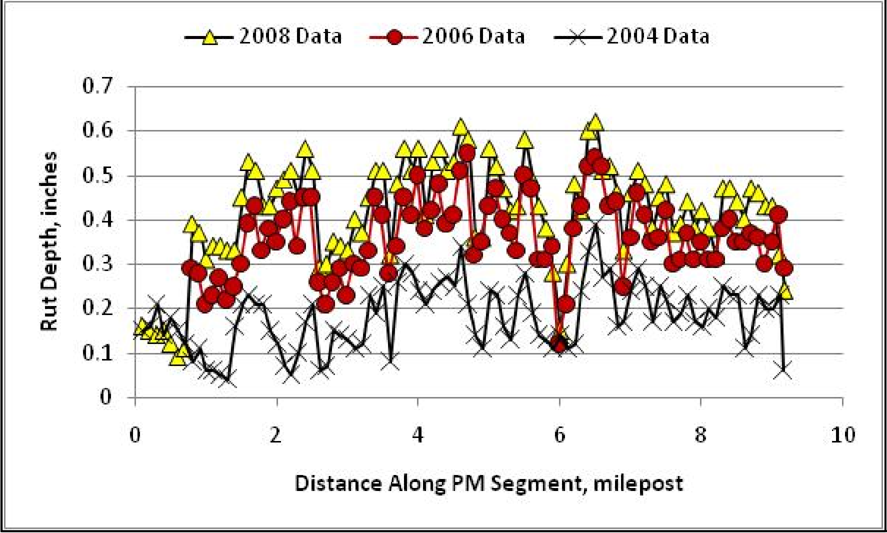
Figure 1. Performance Indicators Measured Over Time for One PM Segment



PM Segment #181, Superior Region; Control Section 7012; US 41, Flexible Pavement; New Construction or Reconstruction; Non-Freeways



Southwest Region; Control Section 13081; I-94, Mill & Resurface Flexible Pavement; Freeways



PM Segment #77, Grand Region; Control Section 34033; M-66, Crush & Shape with Hot Mix Asphalt (HMA) Surface; Non-Freeways

Figure 2. Detailed Rut Depth Data for Three PM Segments

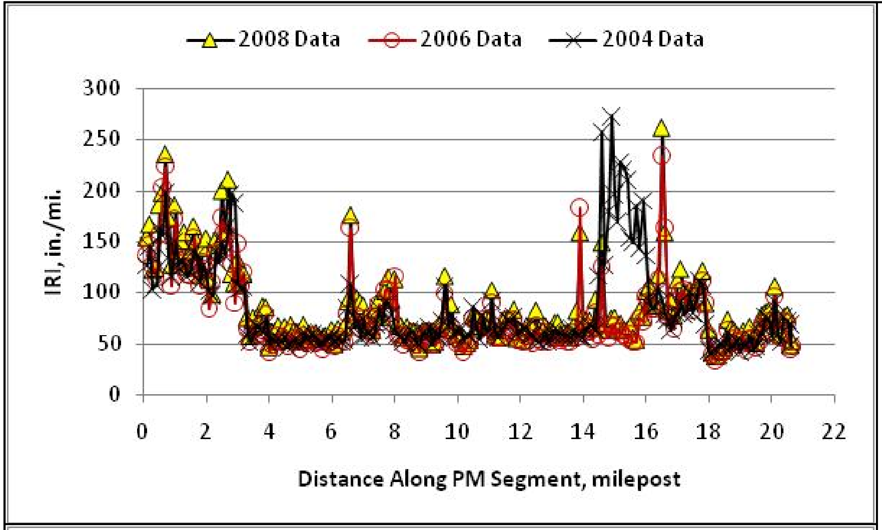
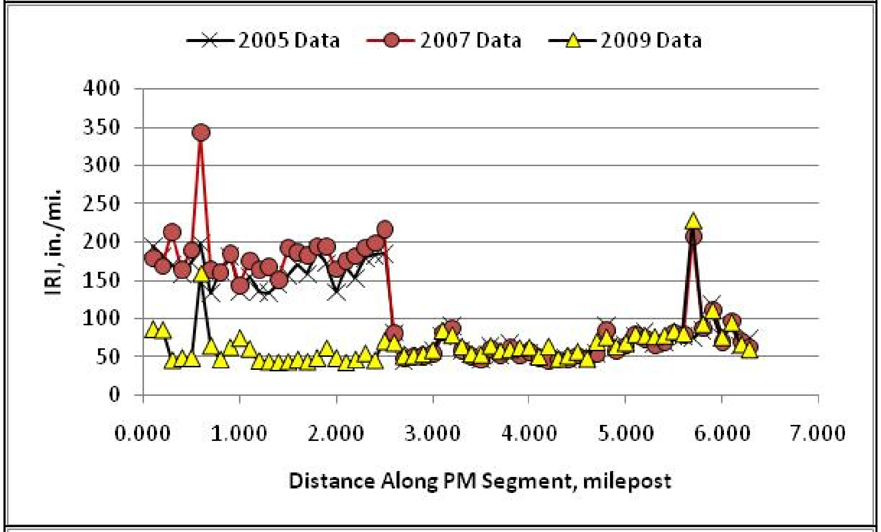
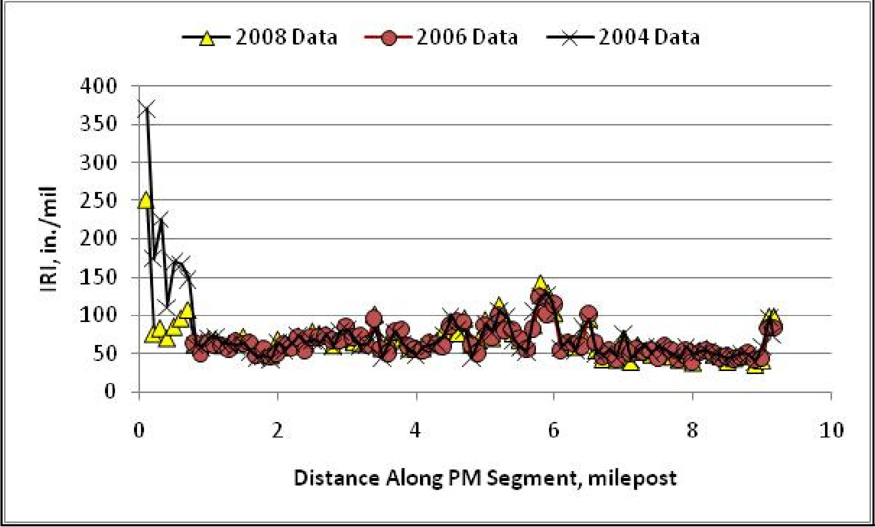
	PM Segment #181, Superior Region; Control Section 7012; US 41, Flexible Pavement; New Construction or Reconstruction; Non-Freeways
	Southwest Region; Control Section 13081; I-94, Mill & Resurface Flexible Pavement; Freeways
	PM Segment #77, Grand Region; Control Section 34033; M-66, Crush & Shape with HMA Surface; Non-Freeways

Figure 3. Detailed IRI Data for Three PM Segments

Table 1. Summary of Average Distress Index Values When Pavement Preventative Maintenance and Preservation Activities Occur (Extracted from MDOT *Pavement Design and Selection Manual* [March 2005])

Structural Classification	Highway Classification	Activity	DI Prior to Activity	Age at Application, yrs.
New Construction/Reconstruction	Freeway	1st Activity	29	10
		2nd Activity	18	13
		Reconstruction	50	26
	Low Volume & Non-Freeway	1st Activity	27	11
		2nd Activity	20	15
		Reconstruction	50	30
HMA Over Rubblized PCC pavement	Freeway	1st Activity	17	6
		2nd Activity	23	8
		3rd Activity	7	12
		Reconstruction	50	20
	Low Volume & Non-Freeway	1st Activity	10	6
		2nd Activity	20	9
		Reconstruction	50	20

Trend lines and statistical parameters of the trend lines were not provided for the correspondence between the different performance indicators, because of the scatter in the data (refer to Figures 4 through 7). Some trend lines are included in a few of the scatter plots (refer to Figures 5 and 7). It is expected, however, that there are confounding factors for which limited data appear to exhibit trends or correspondence between some of the performance indicators. Overall, there is no reasonable correspondence between the different performance indicators.

Observation: The performance indicators of distress index, rut depth, and IRI are independent. In other words, there is no correspondence between the performance indicators measured and monitored by MDOT in managing their roadway network.

This observation contradicts the finding from an analysis of the Federal Highway Administration’s (FHWA) Long Term Pavement Performance (LTPP) data for which IRI was found to be statistically related to different types and amounts of cracks and rut depths in flexible pavements and hot mix asphalt (HMA) overlays. The regression equation developed from the LTPP data is included in the new Mechanistic-Empirical Pavement Design Guide (MEPDG [AASHTO, 2008]). This contradiction was expected and has been found from other studies using network level data because of the measurement error (Smith, et al., 1998 [Ministry of Transportation of Ontario], 2005 [Arizona DOT], and 2006 [Wisconsin DOT]). One of the earlier studies of the LTPP data also reached a similar conclusion (Rauhut, et al., 1999). Thus, each performance indicator was considered separately in the analysis – they are independent of one another.

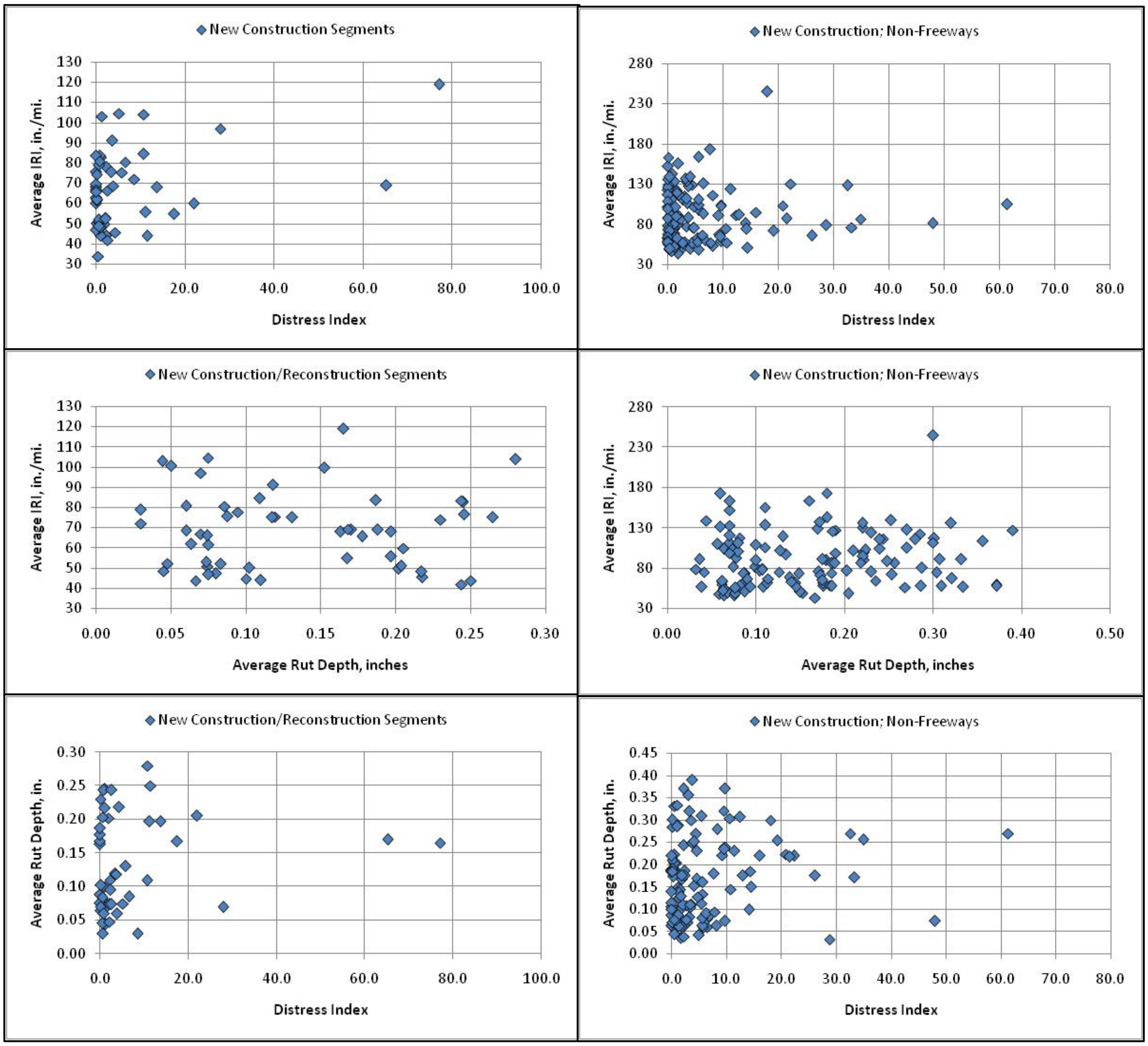


Figure 4. Comparison Between DI, Rut Depth and IRI for the PM Segments in the New Construction and Reconstructed Category

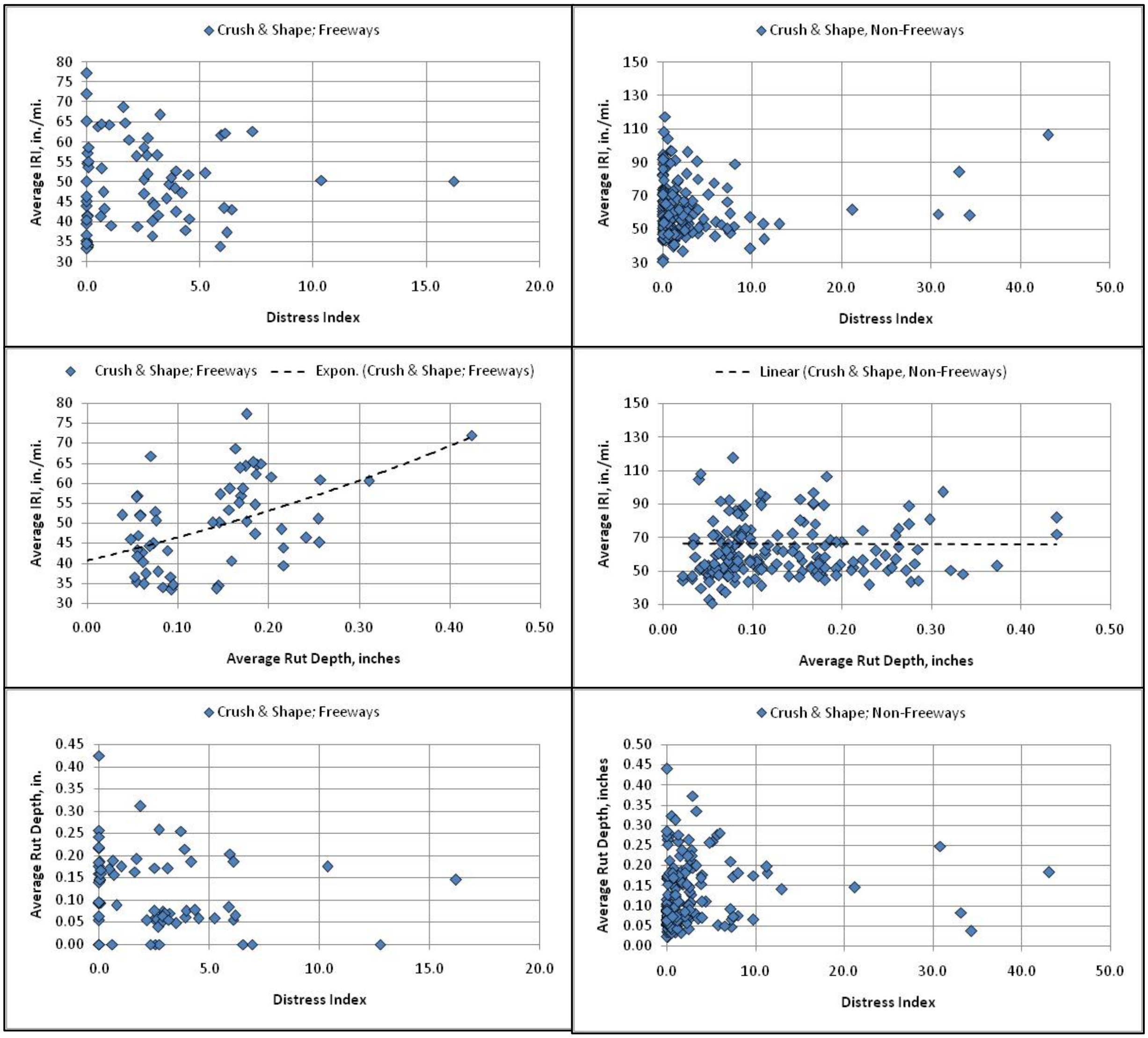


Figure 5. Comparison Between DI, Rut Depth and IRI for the PM Segments in the Crush and Shape with HMA Surface Category

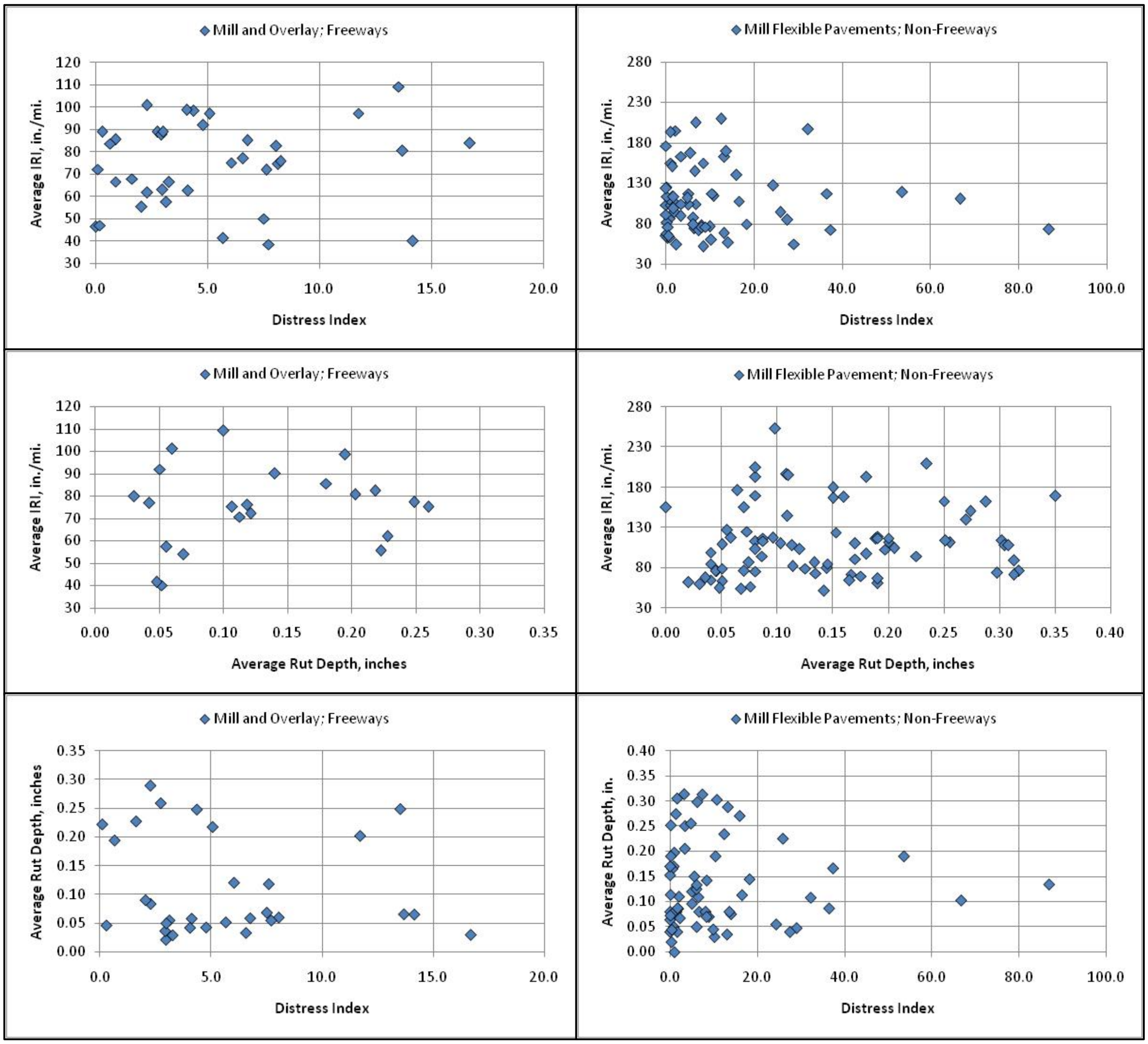


Figure 6. Comparison Between DI, Rut Depth and IRI for the PM Segments in the Mill and Resurface Category

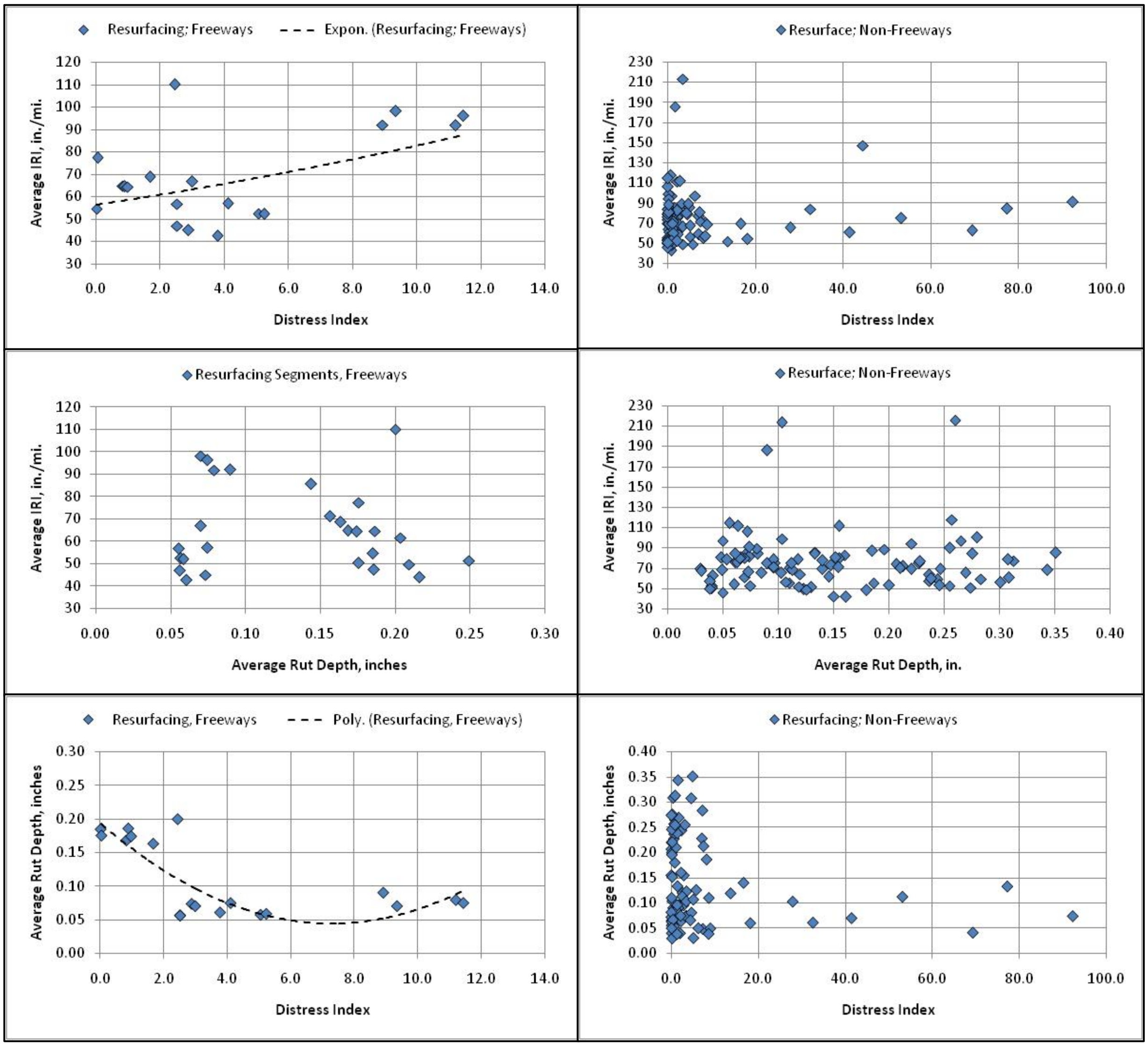


Figure 7. Comparison Between DI, Rut Depth and IRI for the PM Segments in the Resurface Category

CHAPTER 3 PAVEMENT SERVICE LIFE ANALYSES

Data stored in MDOT's pavement performance database was used to evaluate performance and determine the expected service life of asphalt pavements. The service life of individual roadway segments was used to categorize the performance of PM segments with sufficient data into those exhibiting good or exceptional (delayed distress) and poor or inferior (premature distress) performance. The PM segments were initially segregated by four factors that are listed below:

1. Pavement structure: MDOT groups all HMA surfaced roadways into multiple pavement structural categories. The four categories that were included in the scope of work for this project are listed below:
    a. New or reconstructed flexible pavements.
    b. Crush and Shape with HMA surface pavements.
    c. Mill and resurface flexible pavements.
    d. Resurface flexible pavements.
2. Roadway type: MDOT groups all PM segments into two types; freeway/divided highways and non-freeway/divided or undivided highways. This same classification was used within this project.
3. Soil type: Soil type and an estimate of the resilient modulus were considered by identifying the PM segment and the approximate type of soils along the roadway. The soil maps prepared by Michigan State University and the resilient modulus values recommended for specific soils in planning to calibrate the MEPDG were used in identifying to group the PM segments by soil type (Baladi, et al., 2009).
4. Region: MDOT Regions were used to group the PM segments. Seven regions have been established by MDOT: Metro, University and Southwest in the southern part of the state; Bay and Grand in the central area; North in the upper central area; and Superior in the northern part of the state. It was assumed for this study that climate effects would be represented by these regions.

The assumptions used in the analysis of pavement service life are listed below.

- The design period for all new or reconstructed flexible pavements (20 years) and HMA overlays (12 to 15 years) is the same for all PM segments.
- The procedure used to design new and reconstructed flexible pavements and HMA overlays is the same for all PM segments. For the mill and resurface category, a mill depth and HMA overlay thickness is selected – no design analyses are performed. Thus, it was assumed in this study the surface condition of the existing pavement is similar for the pavements included in this category.
- The flexible pavements and HMA overlays were built in accordance with MDOT's specifications and the specifications were properly enforced. Any project that did not meet the project specifications is assumed to have been removed and replaced or the deficiency corrected.
- Roadway segments with inferior material properties and obvious deficiencies (segregation, poor compaction, insufficient thickness etc.) are assumed to have been rejected during construction – layer removed and replaced.

Figure 8 shows the cumulative frequency of pavement age for the segments included in the analysis, while Table 2 lists the number of PM segments for each pavement structural category or data set. Nearly 500 PM segments were used in the service life analysis for each performance indicator. It should be noted that not all of the PM segments were used – many of the newer segments had too few data points or magnitudes to accurately determine the coefficients for an individual PM segment.

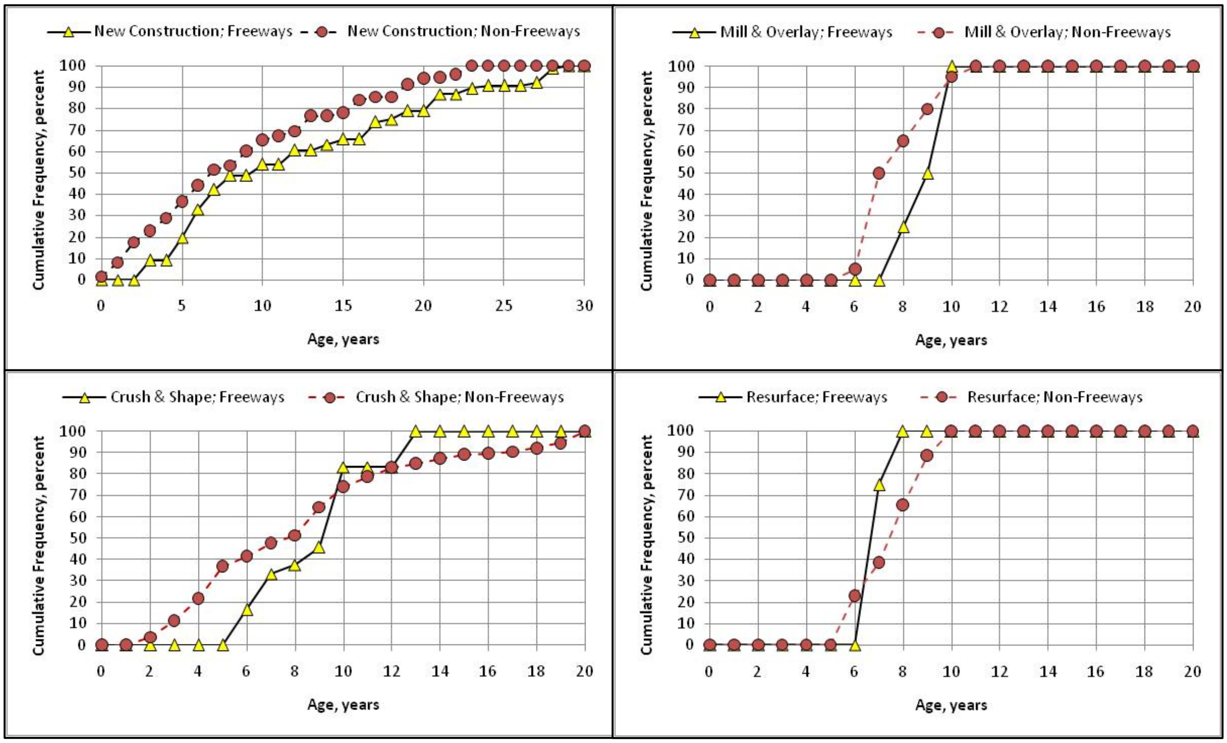


Figure 8. Cumulative Frequency Histogram of Pavement Age

Table 2. Number of PM Segments Included in the Data Analyses to Determine the Coefficients of the Deterioration Relationship for Each Performance Indicator

Highway Class	Pavement Structure Group	Number of PM Segments
Freeways/Divided Highways	New or reconstructed flexible pavements	76
	Crush & shape with HMA overlay	26
	Mill and resurface flexible pavements	16
	Resurface flexible pavements	8
Non-Freeways/Divided and Non-Divided Highways	New or reconstructed flexible pavements	152
	Crush & shape with HMA overlay	167
	Mill and resurface flexible pavements	20
	Resurface flexible pavements	26
	Total Number of PM Segments	**491**

As shown, there is a wide distribution in age for the PM segments included in the reconstruction and crush and shape with HMA surface. Conversely, there is a narrow distribution in age of the mill and resurface and resurface of flexible pavement categories. The older pavements provide a more accurate determination of the deterioration trends and coefficients, because of the higher distress values over a longer period of time and multiple distress or performance indicator measurements are included in the database. Small increases in distress magnitude early in the pavement's service life of newer pavements with only one or two values recorded in the database (without long-term observations) can distort or bias the deterioration relationships (refer to Chapter 4), especially if the increases are a result of measurement error.

3.1 Preventive Maintenance Effects on Service Life

Different pavement preservations methods or treatments have been placed within many of the PM segments over time, especially those in the freeway data sets. Figures 9 and 10 show the performance histories of two PM segments that have received multiple preservation or maintenance treatments. [Refer to Table 5 in Chapter 4 for a listing of the rehabilitation and preservation/maintenance treatments that are commonly used by MDOT and recorded in the PM database.] Most of these treatments affect the performance indicators and can include confounding factors in determining the service life and coefficients of the deterioration relationships, especially when the preventive maintenance activity was applied to a small portion of the initial PM segment or control section.

Figure 11 shows the cumulative frequency distribution of the age of the pavement when a pavement preservation activity was applied to the HMA surface. As shown, the range in age when the first preservation activity was applied to the pavement surface is 1 to 10 years with a median age of about 6 years for the resurfacing and crush and shape with HMA surface pavement structure categories. For the new construction/reconstruction category the range in age is 2 to 20 years with a median age of about 10 years.

In summary, a preservation activity was placed on 38 percent of the PM segments, while 62 percent have yet to receive any preservation activity. Many of the PM segments that have yet to receive any preservation activity are less than 4 years in age. There were roadway segments where the performance indicators abruptly changed or decreased, but no preservation or maintenance activity was recorded in the PM database. Figure 12 is an example of this observation for the distress index. These segments flagged for further analyses. Some of the flagged PM segments were used in the analysis, while others were excluded. As an example, the M-72 segment in Figure 12 was used, while the M-35 segment was excluded. The decision to include or exclude the segment was somewhat subjective but based on the number of values that abruptly changed over time without any explanation for the change.

Figure 13 shows the cumulative frequency of the service life of the pavement preservation methods (age when a second preservation activity was applied to the pavement surface) that were used on a sufficient number of PM segments. As shown, the cold mill-resurface category was found to have a longer service life than for the other activities. The cold mill-resurface category can increase structural capacity, while the other methods do not increase structural strength of the pavement structure. This could be one reason the cold mill-resurface category showed increased service life. Overall, the median service life for the different treatment methods is listed below.

- 3 years for chip seals.
- 5 years for micro-surfacing.
- 6 years for thin and ultra thin HMA overlays.
- 7 years for cold mill and resurface.

3.2 Changes in Performance Indicators over Time

3.2.1 Distress Index

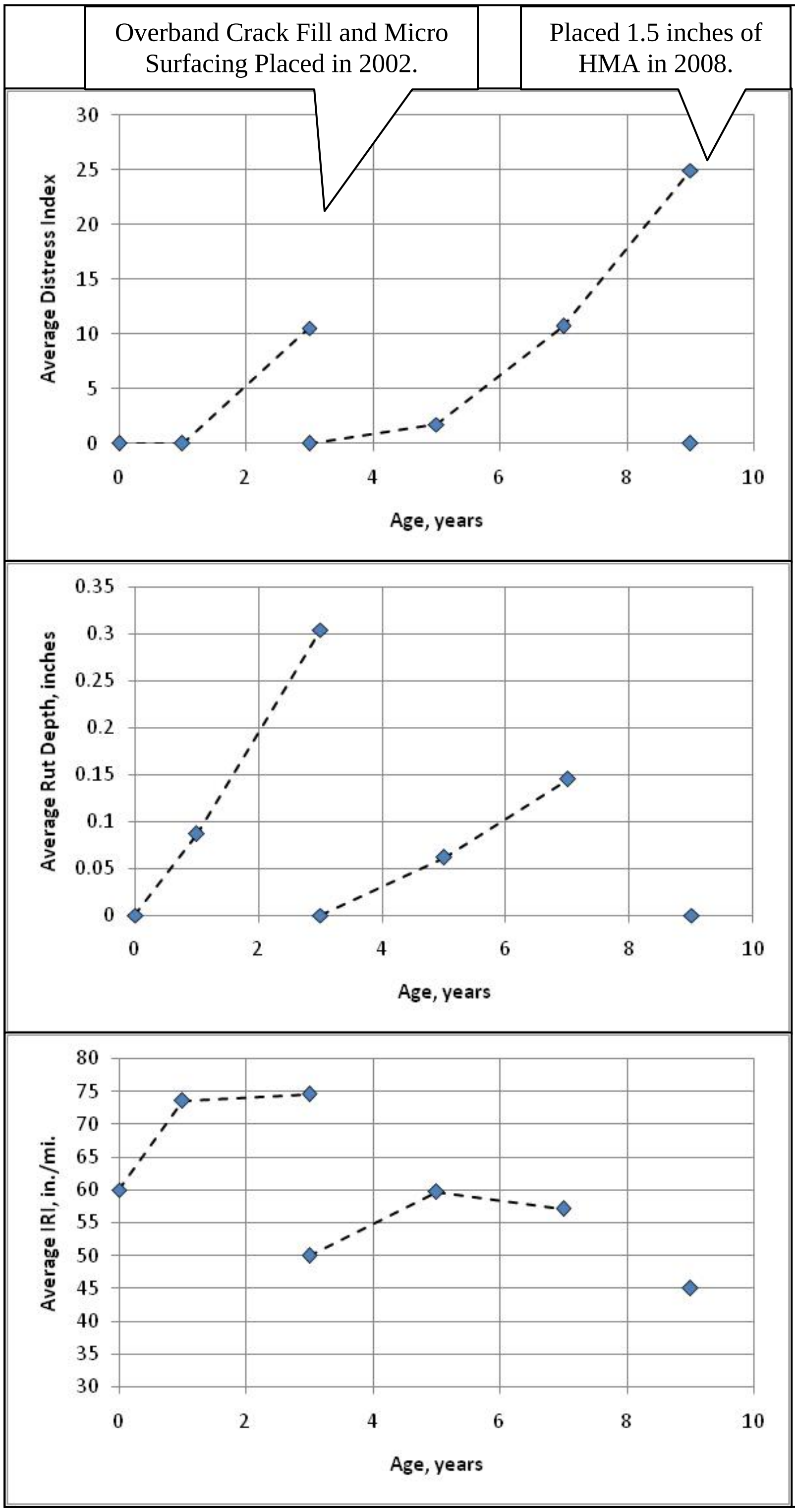
MDOT uses a composite Distress Index (DI) that is determined from different surface distresses. Other agencies also use similar composite distress terms within their PM database. Figure 14 shows a histogram of DI values stored in the PM database and provided for this study, while Figure 15 shows the change in the network-wide DI values over time within each pavement structure category. The interval of DI values included in Figure 14 was set based on the values included in Table 1 and the range of data included in MDOT's database. The network-wide DI values for each year included in Figure 15 are the average DI values recorded for the PM segments included in the analysis. As shown, the average network DI values significantly decreased over time for most of the different data sets.

It is expected that changes in operational policies and specifications during the 1980's and 1990's and/or implementation of different pavement preservation methods being used by MDOT have had a positive and beneficial impact on performance. Some of these changes include the use of polymer modified asphalt (PMA) mixtures, discontinued use of the C-type HMA mixture designations that were susceptible to cracking, adoption of the Superpave binder specification, use of the gyratory compactor for HMA mixture design, revisions to the quality assurance program, more extended application and use of pavement preservation methods, etc.

Observation: The operational policies and specifications implemented by MDOT in the 1990's, including an aggressive preventive maintenance program, have had a positive impact on performance.

It would be beneficial and informative to determine the effect of the different policy and specification changes made over time. The operational policies and specification changes, however, were made at different times. Thus, it is almost impossible to quantify the impact of these changes in policy and specifications using network level data, especially when the changes are implemented over multiple construction seasons.

Another observation from this data review is that the DI values are less than the average values previously reported by MDOT (refer to Table 1; a DI value of 50 is used for reconstruction). As shown, most of the DI values are significantly less than 20.

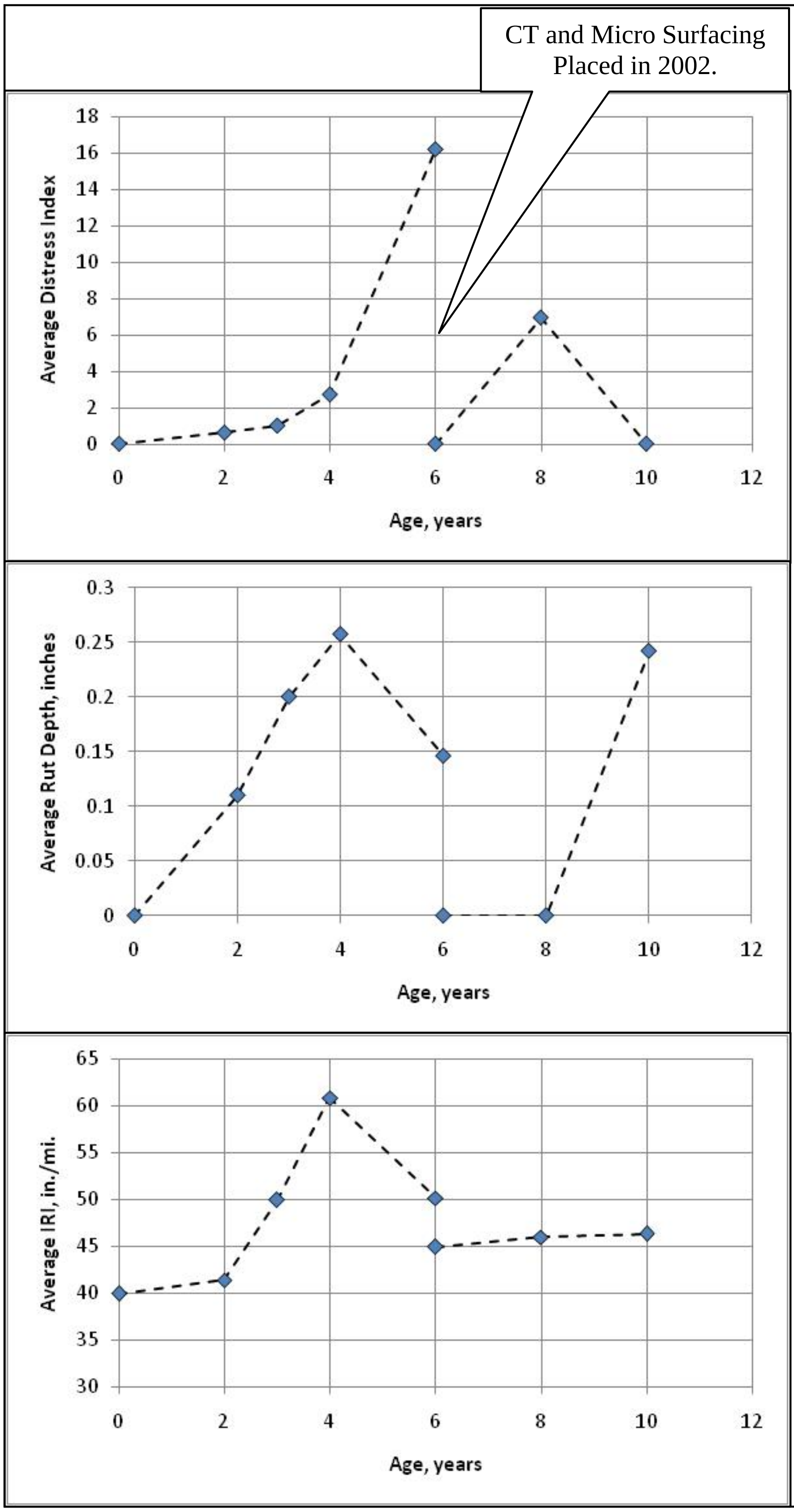

Overband Crack Fill and Micro Surfacing Placed in 2002.

Placed 1.5 inches of HMA in 2008.

Note: When two values of a performance indicator are shown for the same year or pavement age, the value after a preventive method was placed is an assumed value that is dependent on the type of method placed.

PM Segment ID#147; M-32
East of Townline Road to Murner Road
BMP – 8.974; EMP – 9.8
CS – 69021
JN – 32331

Figure 9. Performance History of PM Segment with Different Preservation Methods, New Construction, Non-Freeway



CT and Micro Surfacing Placed in 2002.

Note: When two values of a performance indicator are shown for the same year or pavement age, the value after a preventive method was placed is an assumed value that is dependent on the type of method placed.

PM Segment ID#13N; I-75, NB
US-127 merge to So. of M-72
BMP – 0.076; EMP – 4.232
CS – 20014
JN – 44827

Figure 10. Performance History of PM Segment with Different Preservation Methods, Crush & Shape with HMA Surface, Freeway

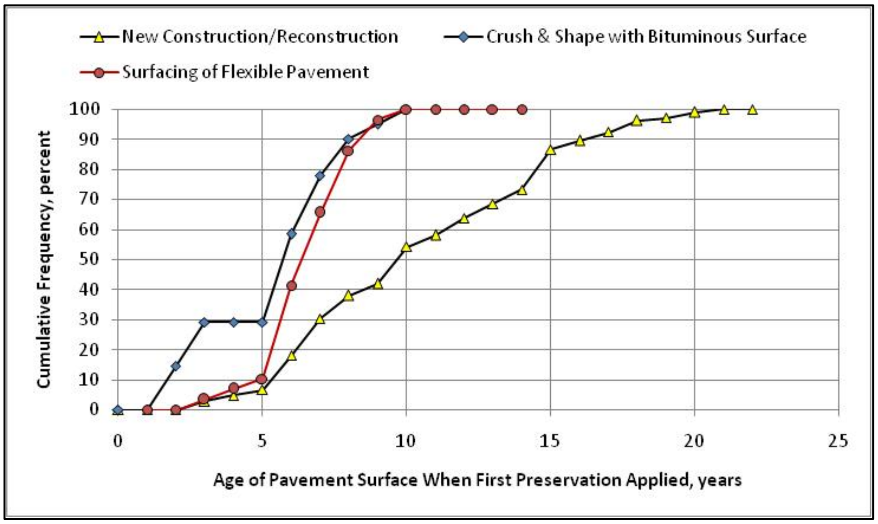
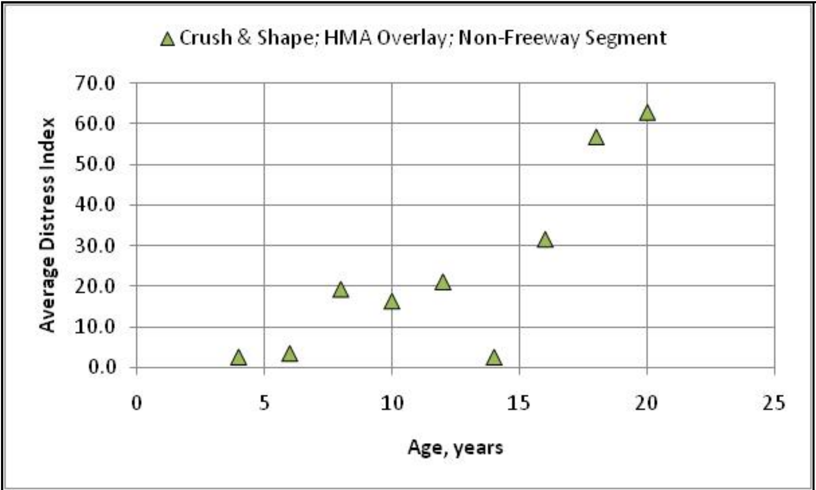
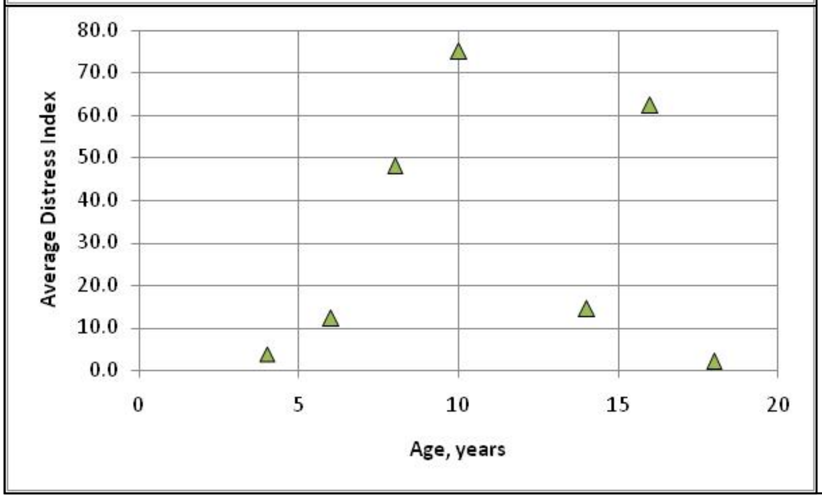


Figure 11. Cumulative Frequency of Pavement Age at Time of Preservation Placement



PM Segment ID#1; M-72
East of M-33 to the
Oscoda/Alcona County Line
BMP – 0.102; EMP – 8.239
CS – 68042
JN – 25538

The average distress index increases over time, as expected; but is near zero at year 2002 (14 years old). The section was milled & resurfaced in 2000, which was not recorded in database.



PM Segment ID#17; M-35
Anderson Rd to South of
County RD 480
BMP – 19.234; EMP – 24.648
CS – 52032
JN – 26628

The average distress index increases over time, as expected; but decreases to near zero at year 2000 (12 years old) and 2006 (18 years old). No preservation or rehabilitation activity recorded in database.

Figure 12. PM Segments with Significant Decrease in Performance Indicators, But No Pavement Preservation or Maintenance Activity Recorded in Database

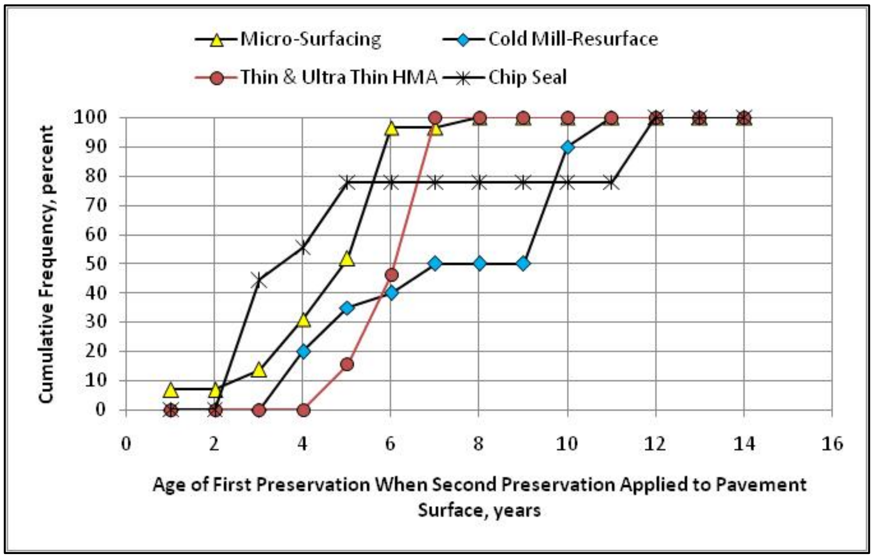


Figure 13. Cumulative Frequency of the Service Life of Different Pavement Preservation Methods Used in Michigan

The detailed distress data included in MDOT's performance database were also used to determine the magnitude and severity of the individual distresses for those roadway segments categorized into poor and good performance. The detailed distress data were used to identify construction and material parameters not recorded in the MDOT pavement performance database. Chapter 5 summarizes the analysis of the detailed distress data for the roadway segments with poor and good performance.

3.2.2 Rut Depth

MDOT monitors rut depths in the PM segments. The detailed rut depth data were provided by MDOT for each PM segment and reviewed to determine the range of values measured within the PM segments.

Figure 16 shows a histogram of the rut depths reported in the PM database for individual control sections or PM segments and provided for use in this study, while Figure 17 shows the change in the network rut depths over time within each pavement structure category. The interval of rut depths included in Figure 16 were set based on the range of data included in MDOT's database and values typically used by other agencies to trigger some type of rehabilitation. As shown, the average network rut depths significantly decreased for two monitoring periods and then increased.

Possible explanations for the trend shown in Figure 17 include; abnormally cool summers over a couple of years, a change in the method or equipment used to measure rut depth, and/or implementation of specifications that result in stiffer HMA mixtures followed by abnormal hot summers over a couple of years. The other observation from this initial data review is that most rut depths are significantly less than the threshold or trigger values used by many agencies (0.35 to 0.50 inches).

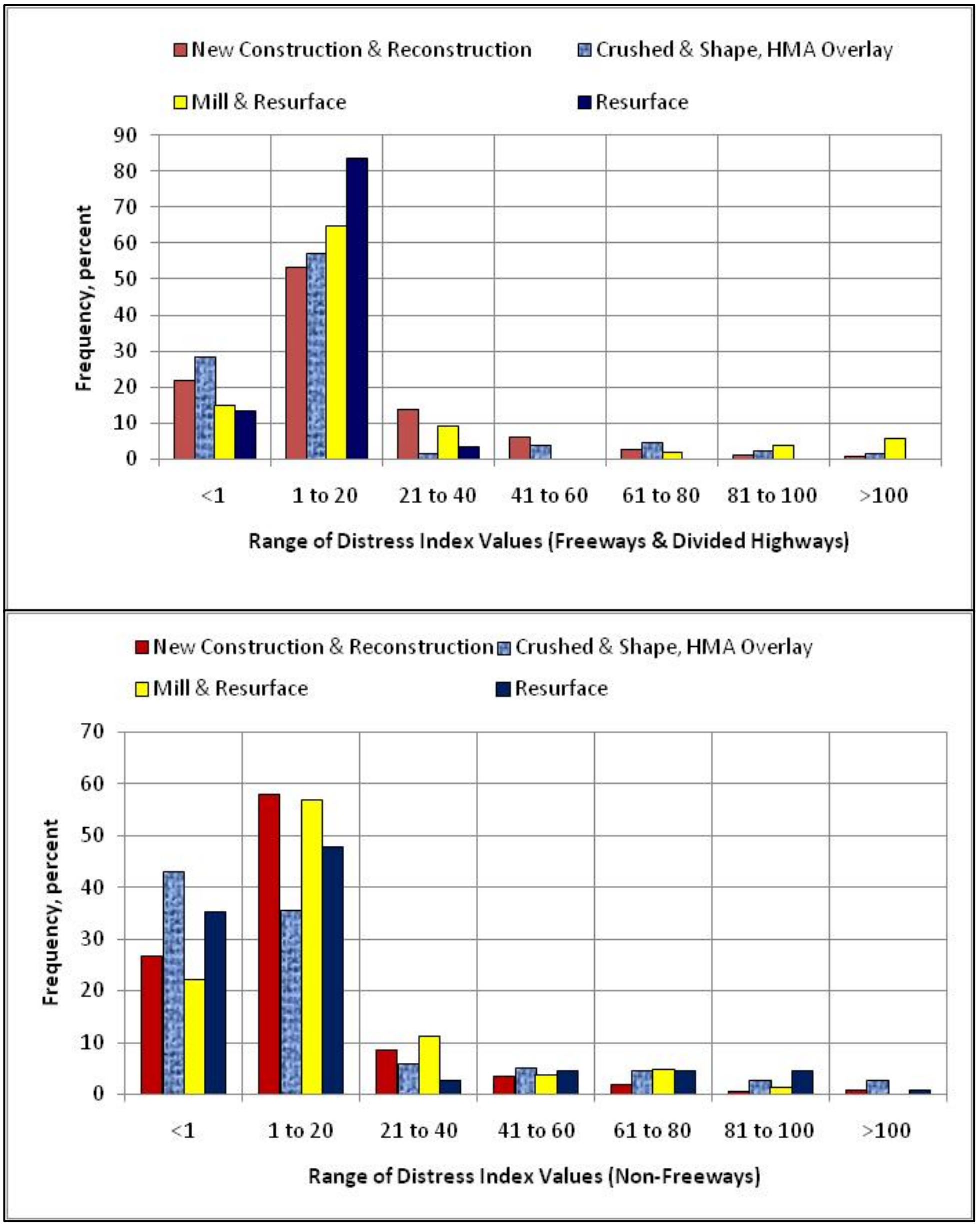


Figure 14. Histogram of DI Values Used in Data Analyses

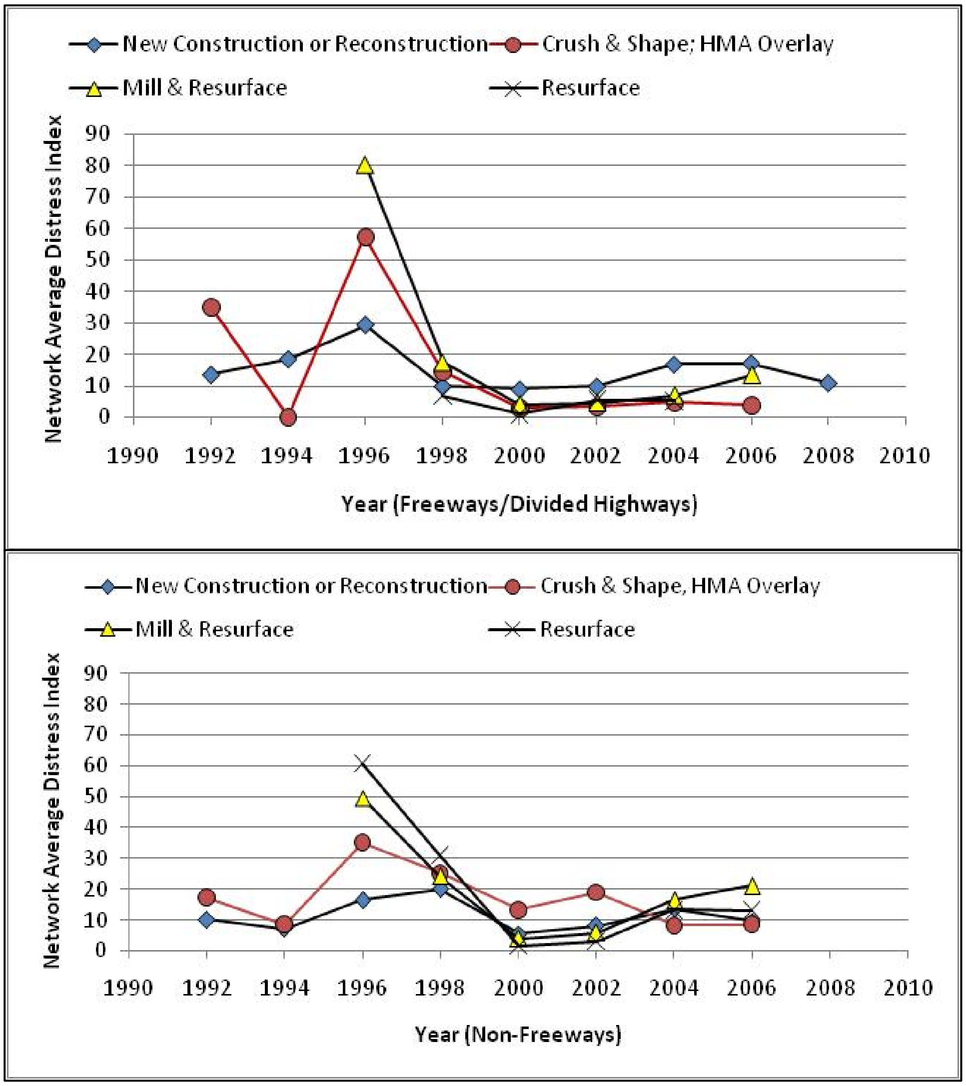


Figure 15. Change in Average DI Values Over Time for Different Pavement Structures

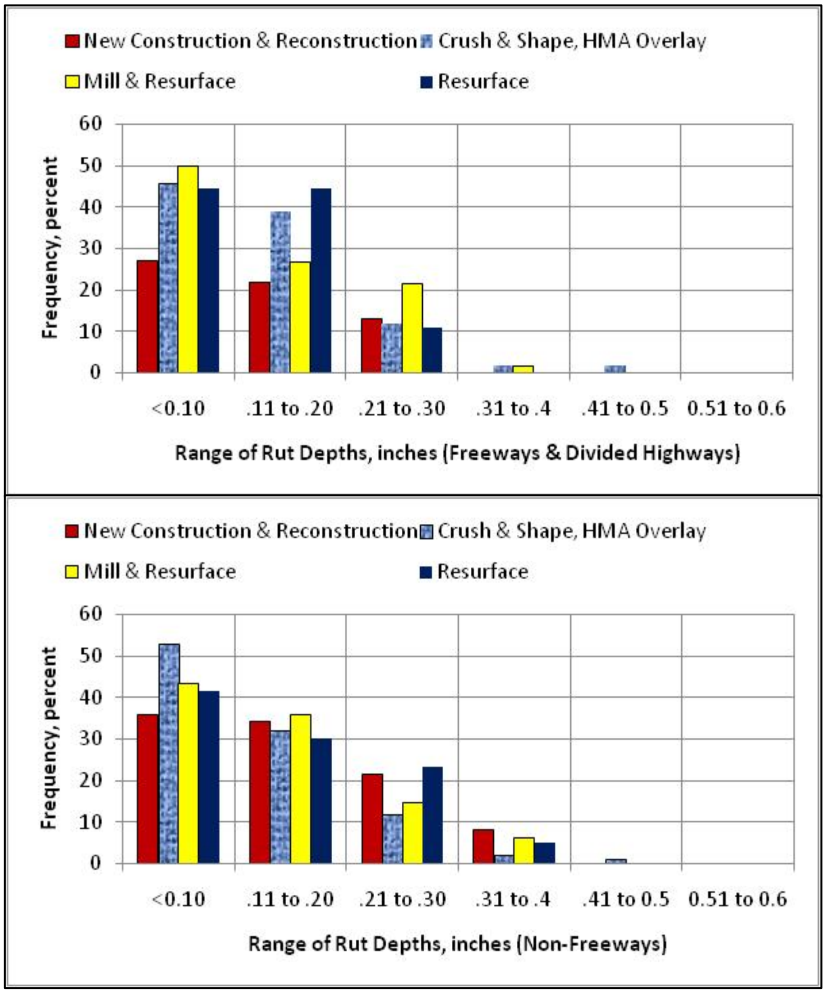


Figure 16. Histogram of Rut Depths Used in the Data Analysis

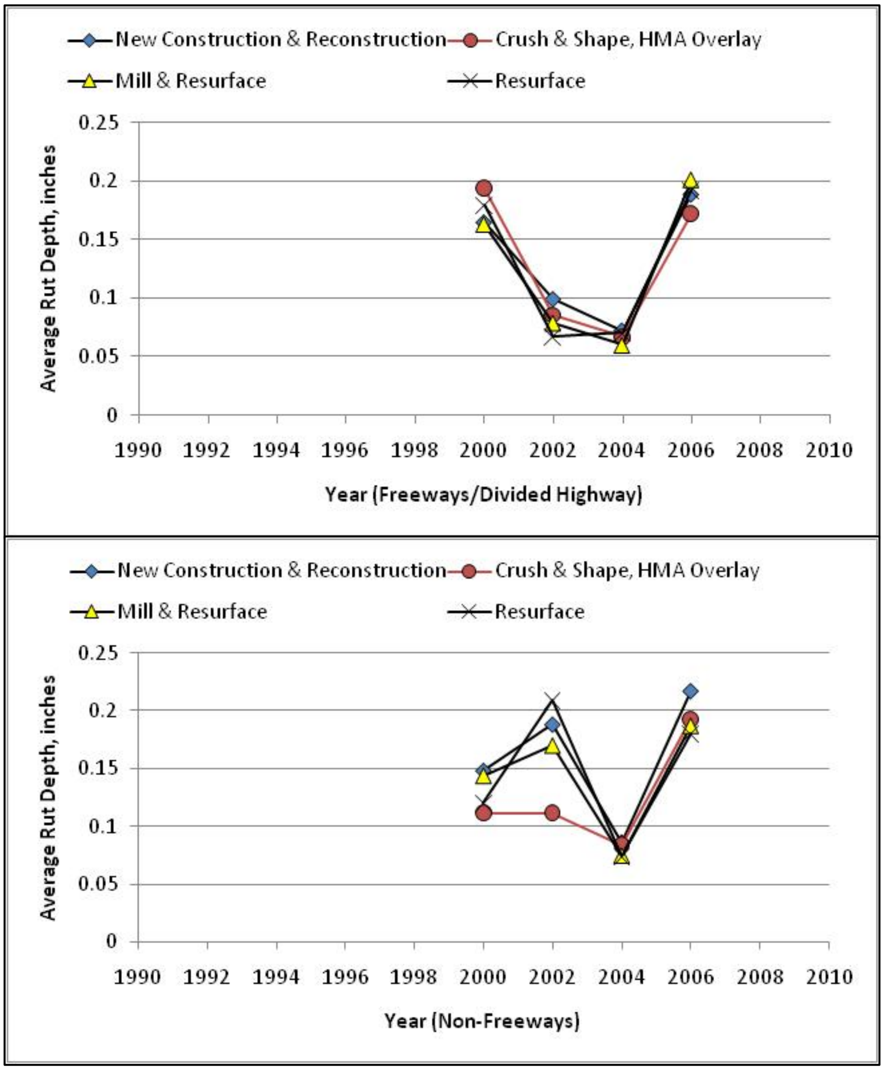


Figure 17. Change in Average Rut Depths Over Time for Different Pavement Structures

3.2.3 Smoothness

IRI is also monitored by MDOT as an indicator of pavement smoothness. Detailed IRI data were provided for each PM segment and reviewed to determine the range of values measured within the PM segments. Figure 18 shows a histogram of the IRI values reported in the PM database for the individual control sections or PM segments and provided for use in this study, while Figure 19 shows the change in the network IRI values over time within each pavement structure category. The interval of the IRI values included in Figure 18 were set based on the range of data included in MDOT's database and values typically used by other agencies to trigger some type of rehabilitation. As shown, the average IRI values have remained about the same over time within each pavement structure category.

The other observation from this initial data review is that most values are significantly less than the threshold or trigger values used by many agencies for interstate or primary arterials (less than

120 in./mi.). The IRI values measured along the lower volume, non-freeway highways have an appreciable number of PM segments that are significantly higher – exceeding 100 in./mi.

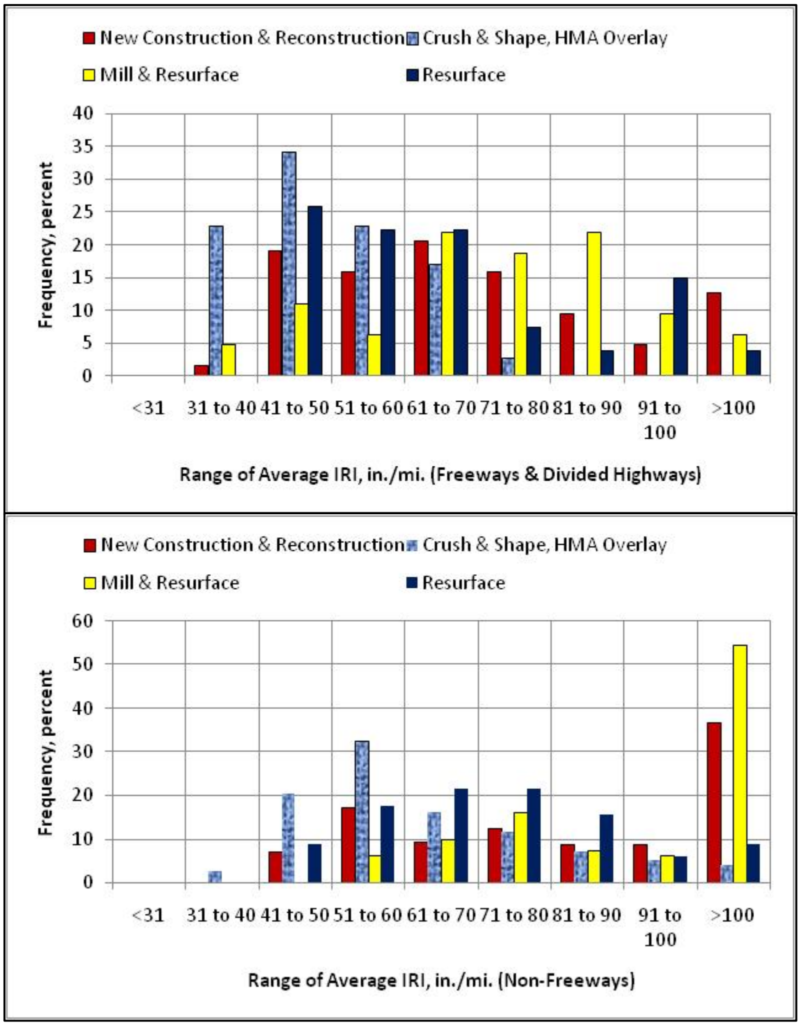


Figure 18. Histogram of IRI Values Used in the Data Analysis

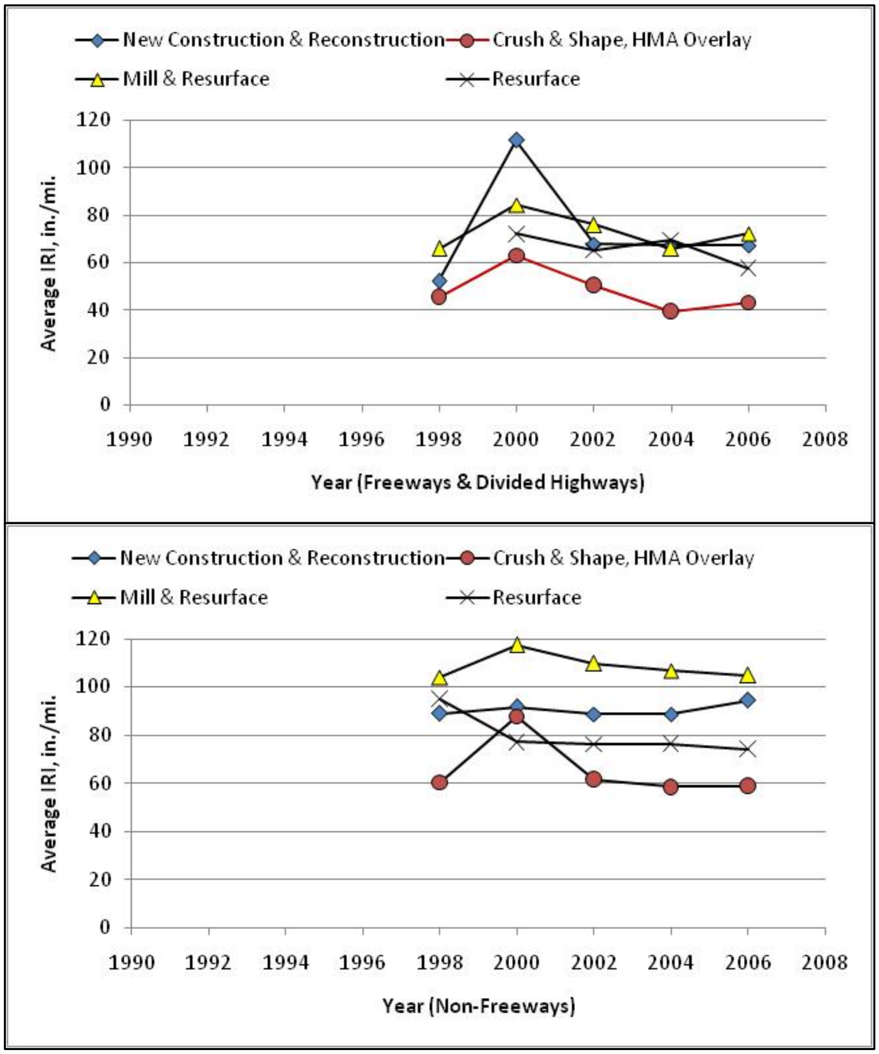


Figure 19. Change in Average IRI Values Over Time for Different Pavement Structures

3.3 Data Analysis of Service Life

The roadway segments included in the MDOT database with sufficient data were used to determine the average service life. The average service life was used to group the roadway segments with poor and good performance. Two approaches were used to determine if specific design and site features were significantly different between the two groups of roadway segments; those exhibiting poor and good performance, which are listed below.

- The Student's t-test approach was used in comparing good and poorly performing pavements for those parameters with continuous numerical values, such as for traffic. The t-test approach compares the mean of each variable in the good group to its mean in the poor group. The hypothesis that the two means are indifferent is rejected if the t-value is significantly large or the p-value is significantly small.

- For those parameters without continuous numerical values (subgrade type, highway type or climate), categorical analyses were used to decide whether trends existed in each of these variables that distinguished good and poor performance. In other words, the number of good and poor performance segments was determined for each variable within individual groups. Chi-square statistical tests are then used to compare the numbers with each other across all levels of the variable to determine whether there is a statistical difference.

These two approaches were used by Rauhut, et al. for comparing the properties, design features, and/or site conditions of good and poorly performing asphalt pavements in the LTPP program (Rauhut, et al., 1999). Results from this LTPP project did not identify any significant pavement structural or material property, design feature, or site condition factor that would explain the difference between good and poorly performing asphalt pavements. The study concluded that many of the parameters evaluated are interrelated and separating individual properties without considering the effects of other design features and properties can lead to improper conclusions. Once some of the parameters were blocked by specific features, many of the results concurred with previous pavement engineering experience.

3.3.1 Survivability Analysis to Define Good and Poor Performance

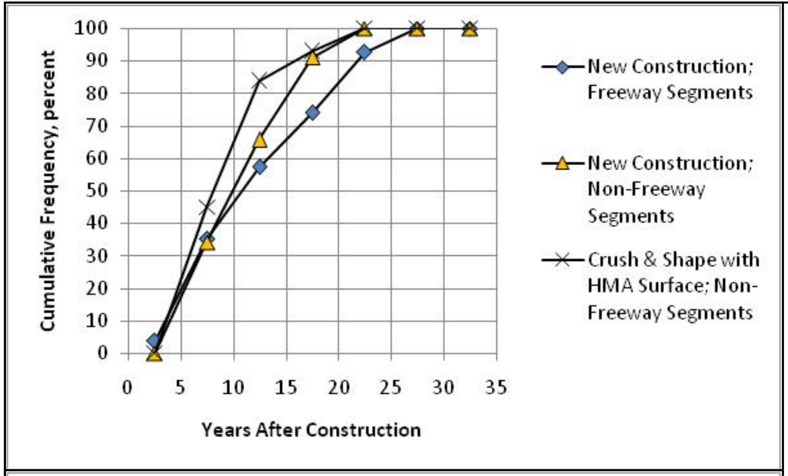
A survivability analysis was completed on age of the roadway segments with sufficient time series data (projects paved prior to 2001, or about 6 to 8 years of performance data). The survivability analysis was completed using those segments in the new construction or reconstruction and crush and shape with HMA surface categories. The purpose of the survivability analysis was to determine the pavement age that can segregate good (delayed distress) and poor (premature distress) performance.

Figure 8 included a cumulative frequency diagram of pavement age for all roadway segments, while Figure 20 is a cumulative frequency diagram for those segments with multiple measurements of the performance indicators. Figure 20.a shows the cumulative frequency of age for individual segments built prior to 2001, while Figure 20.b shows the cumulative frequency of age when the criteria triggering reconstruction was exceeded (a distress index value greater than 50). Table 3 lists the average age of asphalt pavements that reached the threshold value requiring reconstruction (refer to Figure 20.b).

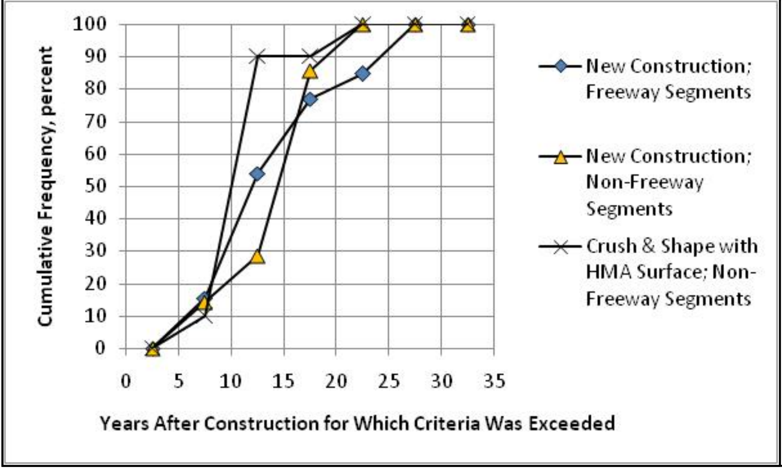
Table 3. Age Used to Identify Pavements with Good and Poor Performance

Pavement Structure	Performance Definition Using Age, years		
	Average	Poor	Good
New Construction; Freeway Segments	13	<9	>17
New Construction; Non-Freeway Segments	15	<10	>20
Crush & Shape with HMA Surface	10	<6	>14

The reason that the mill and resurface and resurface categories are not included in Figure 20 is they have a narrow distribution in age, in comparison to the new construction and crush and shape categories (refer to Figure 8). Thus, poor and good performance can be segregated by the magnitudes of the performance indicators – time is not a factor; while it is a factor for the other pavement groups. Table 3 lists the pavement age for defining poor and good performance; the age at which the threshold value is exceeded for the new construction and crush and shape pavement groups.



20.a Cumulative frequency of age of all roadway segments built prior to 2001.



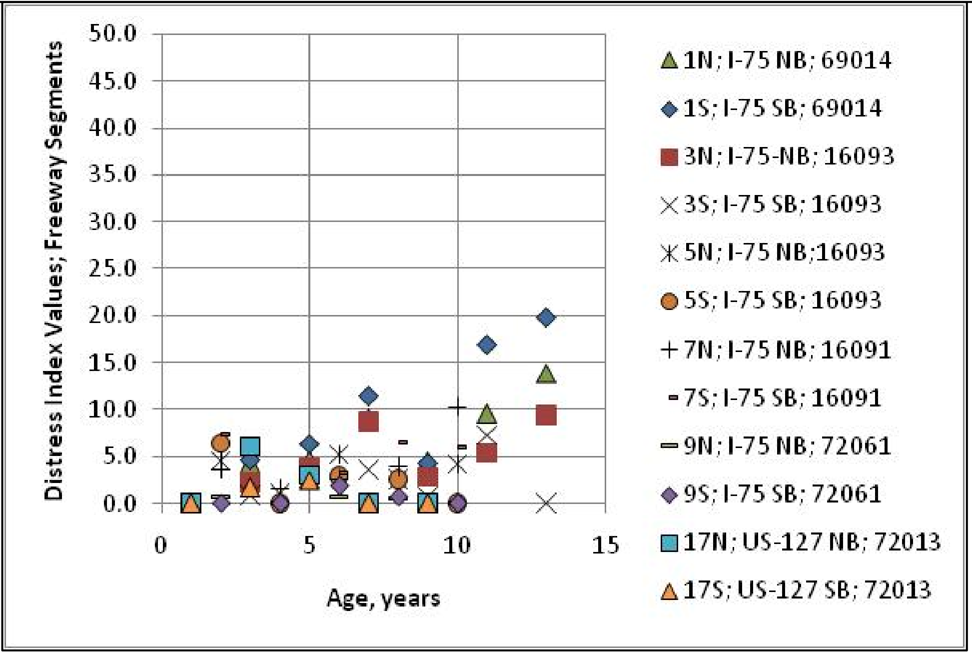
20.b Cumulative frequency of age for roadway segments at the time when the criteria for reconstruction was exceeded.

Figure 20. Cumulative Frequency of Age for Roadway Segments Built Prior to 2001

A finding from the survivability analysis was that the crush and shape with HMA surface category has exhibited better performance than some of the other pavement structural categories. This observation contradicted the experience from some MDOT staff. Thus, the database was used to determine the number of sites for which the crush and shape with HMA surface category have exhibited lower levels of distress for extrapolating the service live based the computed distress index values (discussed in Chapter 4). The following paragraphs summarize the findings from PM segments with sufficient data to extrapolate the distress index values for this pavement structure.

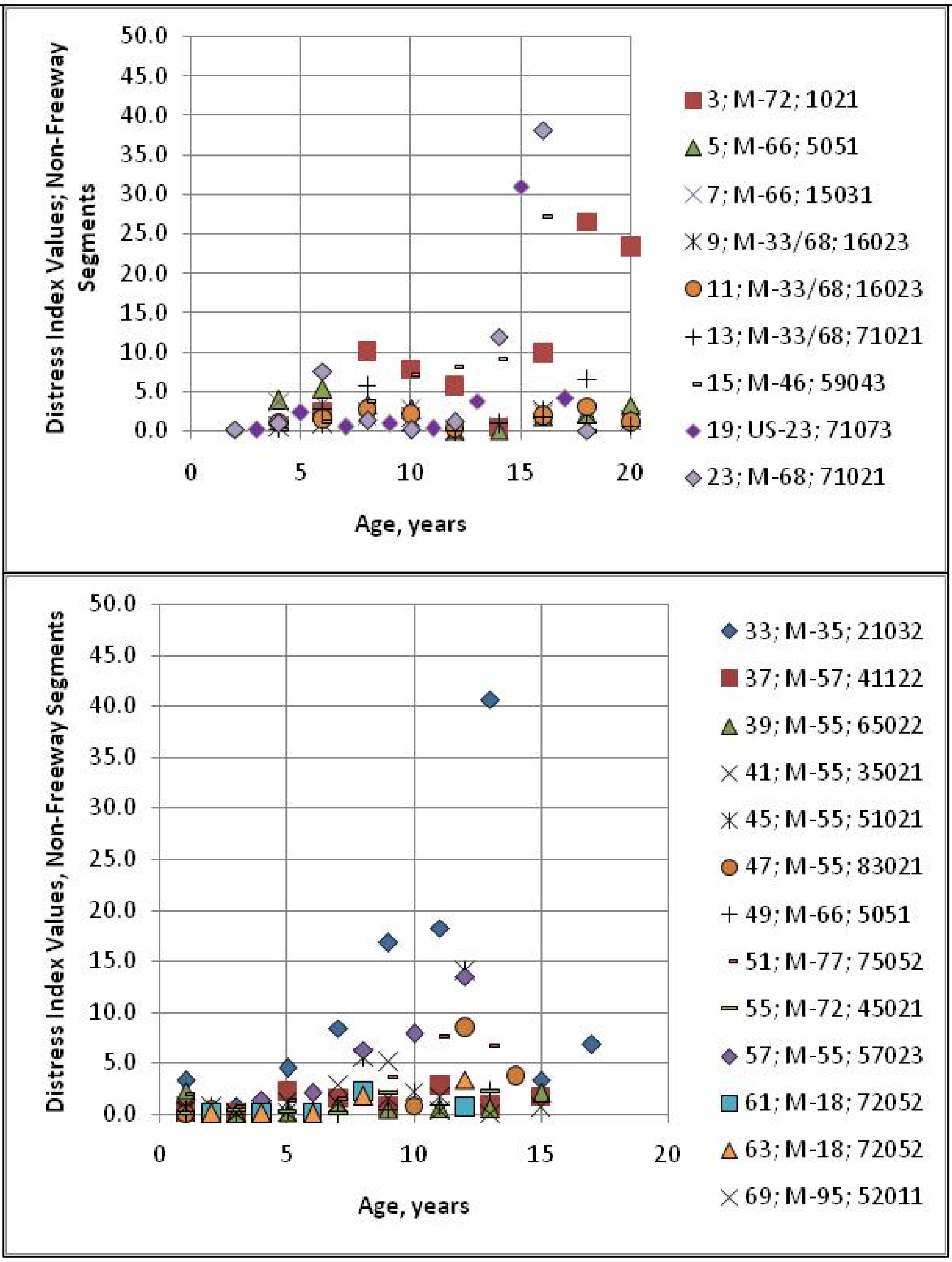
- Figure 21 shows the segments located along freeways. Many of these were along I-75 and have exhibited nearly 15 years of service with relatively low distress index values. Some type of preventive maintenance or pavement preservation activity was applied to the pavement surface on just about all crush and shape with HMA surface structures. The pavement preservation activity was generally applied between 5 to 10 years after construction. There are other crush and shape with HMA surface freeway segments, but they do have higher levels of the distress index. Those included in Figure 21 include those with exceptional or good performance based on the distress index.

- Figure 22 shows the segments located along non-freeways. As shown, many of the roadway segments with the crush and shape with HMA surface category have exhibited good performance with relatively low distress index values, even as long as 20 years. In addition, a preventive maintenance or pavement preservation activity is not recorded in the database for some of these non-freeway segments, even some approaching 20 years in age. As for the freeway segments, there are other crush and shape with HMA surface non-freeway segments, but they have higher levels of the distress index (values approaching or over 50) within 10 years after construction.



Actual Distress Index Values Extracted from the Michigan Database for Freeway Segments

Figure 21. Performance of Crush and Shape with HMA Surface, Freeway Roadway Segments Based on the Distress Index



Actual Distress Index Values Extracted from the Michigan Database for Non-Freeway Segments

Figure 22. Performance of Crush and Shape with HMA Surface, Non-Freeway Roadway Segments Based on the Distress Index

In summary, there are at least 6 crush and shape with HMA surface segments along the freeway category that have exhibited nearly 15 years of service without excessive distress. In addition, there are at least 12 crush and shape with HMA surface segments along the non-freeway category that have exhibited nearly 15 years of service without excessive distress and 5 segments that have nearly 20 years of service without excessive distress. From the data, it is concluded that there are a sufficient number of roadway segments to predict the distress index value for ages approaching 20 years. In other words, the deterioration relationships (discussed in Chapter 4) are not extrapolating beyond a reasonable time frame based on the data included in MDOT's database.

3.3.2 Analysis of Parameter Differences Between Good and Poor Performance

The MDOT roadway segments were grouped by region (climate), pavement structure, roadway type, soil type, and traffic volume. The estimated service life was used to categorize the performance of all segments included in MDOT's performance database with sufficient data into those exhibiting good and poor (premature distress) performance. Table 4 lists the number of roadway segments with poor and good performance by HMA mixture type, climate (region), highway classification, and pavement structure. The hypothesis that the means of the two groups was indifferent was accepted. In other words, no significant or consistent difference was identified between the two groups for any of the parameters included in the database. The following summarizes the findings from grouping the roadway segments into different performance categories.

- Traffic (CAAT) for the segments with poor performance varies from 275 to 2110, while traffic varies from 41 to 5434 for segments with good performance. Thus, traffic does not explain the higher levels of distress (premature versus delayed distress).
- The MDOT database does not designate type of HMA mixture for many of the older segments with poor and good performance. For the segments where HMA mixture type has been reported, none of the different mixture designations used by MDOT over time have significantly more segments with poor or good performance. Thus, mixture type does not explain the higher levels of distress.
- Although there are regions with more roadway segments with poor performance, those same regions generally have the greater number of segments with good performance. Thus, climate/region does not explain reasons for the premature distress.
- Resilient modulus of the subgrade soil and/or type of soil varies across all ranges for the roadway segments with poor and good performance, similar to traffic.
- The majority of the roadway segments with poor performance fall in the new construction category, but there are more overall segments included in this category.

Observation: Using the service life defined as the age at which the threshold value is exceeded (refer to Table 3); climate/region, pavement structure, roadway type, soils type and resilient modulus, and traffic volume do not explain the difference between roadway segments with good (delayed distress) and poor (premature distress) performance.

It is difficult to determine the factors or design-site features that are different between poor and good performing pavements because of periodic changes to their design, construction, and/or material specifications (discussed in Section 3.2 of this chapter). These changes made over time increases the challenge to pinpoint the reasons for the difference in performance by only considering time to rehabilitation or expected service life.

Another difficulty is that pavements with the same cross section (materials and layer thickness) and site condition features do not exhibit the same performance. Pavements with “identical” features and design periods will exhibit higher to lower amounts of distress. This difference typically is referred to as the pure error or variance. The typical standard deviation of this pure error has been found to vary between 3 to 6 years (Smith, et al., 1998 and 2005; Von Quintus, et al., 2003). The difficulty is to separate the pure error in average service life from pavements that exhibit shortened and longer design lives because of some systematic difference in cross section, physical properties between layers, construction defects, and/or operational-management policies. Ignoring this pure error can introduce confounding factors between perceived groups of good and poor performing pavements that actually are indifferent. The next chapter uses deterioration relationships for quantifying poor and good performance and accounts for the measurement error by using average deterioration rates or trends rather than the peak magnitude of a performance indicator at a specific point in time.

Observation: (a) The average DI values for about 75 percent of the PM segments are less than 20.
(b) The average rut depths for over 90 percent of the PM segments are less than 0.30 inches.
(c) The average IRI values for over 85 percent of the PM segments along freeways is less than 100 in./mi., while only about 50 percent of the non-freeway segments are less than 100 in./mi.

Table 4. Number of Roadway Segments with Poor (Premature Distress) and Good Performance Based on Magnitude of the Performance Indicators

Data Category		Number of Segments with:	
		Poor Performance	Good Performance
Pavement Structure Type	New Construction	24	23
	Crush & Shape with HMA Surface	10	24
	Mill & Resurface & Resurface	6	26
Roadway Type	Freeway	13	25
	Non-Freeway	27	48
HMA Mixture Type	Unknown or Not Designated in Database	29	22
	Type A	1	7
	Type B	3	8
	Type C	3	17
	E-1	0	3
	E-3	0	8
	E-10	4	8
Climate/Region	Bay	1	3
	Grand	8	10
	Metro	1	1
	North	8	30
	Southwest	9	4
	Superior	11	19
	University	2	6
Total Number of Roadway Segments in Each Group		40	73

CHAPTER 4 DETERIORATION RELATIONSHIPS AND ANALYSES

This chapter presents the deterioration relationships used for defining pavement segments with poor and good performance. The deterioration relationships explain the increase in distress magnitude (DI, rut depth, and IRI) over time. The coefficients of the deterioration relationships are used to categorize the performance of PM segments into those exhibiting good and poor performance.

As noted earlier in this report, nearly 500 PM segments were available to determine the deterioration coefficients for each performance indicator. However, not all PM segments were used – many of the newer segments had too few data points or magnitudes to accurately determine the coefficients for an individual PM segment. The older pavements provide a more accurate determination of the deterioration trends and coefficients, because of the higher distress values over a longer period of time and multiple data values are included in the database. Small increases in distress magnitude early in the pavement's service life of newer pavements with only one or two values recorded in the database (without long-term observations) can distort or bias the deterioration relationships, especially if the increases are a result of measurement error.

Deterioration coefficients for each performance indicator were also determined for the different preventive maintenance methods or strategies that were found to have a significant reduction in distress or performance indicator (refer to Table 5). Too few treatment methods for the crush & shape (including those in the structural data set) and hot in place recycling and resurface methods were recorded in the PM database to determine the deterioration coefficients for these activities separately, so they were combined with other preservation methods. The crush & shape, hot in place recycling, and resurface were all combined with the cold mill and resurface category.

Most of the preventive maintenance methods affect the performance indicators and can include confounding factors in determining the coefficients of the deterioration relationships. These confounding factors were not identified and, thus excluded from the performance analysis. In addition, there were roadway segments where the performance indicators abruptly changed or decreased, but no preservation or maintenance activity was recorded in the PM database. Figure 12 was an example of this observation for the distress index. The deterioration coefficients were determined and flagged for these PM segments. Some of the flagged regressed values were used in the analysis, while others were excluded. As an example, the M-72 segment in Figure 12 was used, while the M-35 segment was excluded. As noted for the service life analysis, the decision to include or exclude the segment was somewhat subjective but based on the number of values that abruptly changed over time without any explanation for the change.

The average deterioration coefficients for the different data groups were used to predict the time (age) to a level requiring rehabilitation for each performance indicator, using the following threshold values:

- Distress Index of 50.
- Rut depth of 0.40 in.
- IRI value of 120 in./mi.

Table 5. Listing and Definition of the Rehabilitation and Pavement Preservation Methods Included in the PM Database

Rehabilitation and Preservation/Maintenance Method ID		Description; As Used In the Analysis Plan
	Resurface or Overlay	Major activity; age of PM segment adjusted back to "0" when method was applied to pavement surface.
CMR	Cold Mill and Resurface	
HIPR&R	Hot In Place Recycling & Resurface	
C&S	Crush & Shape	
Micro	Micro-surface Method	Affects all types of cracking; less of an effect on rut depths and IRI; age of PM segment adjusted back to "0" when method was applied to pavement surface.
	Chip Seal	
OCF	Overband Crack Fill	Affects some types of cracking, but DI values not significantly reduced after application; less of an effect on rut depths but can increase IRI; age of PM segment not adjusted back to "0" when method was applied to pavement surface.
CT	Crack Treatment	No significant effect on the three performance indicators; age of PM segment not adjusted back to "0" when methods were applied to pavement surface.
CF	Crack Fill	

4.1 Distress Index

The average and range of DI values for different data sets are listed in Table 6 for when preventive maintenance (preservation activity) was applied to the pavement surface, while Figure 23 shows the cumulative frequency of those DI values. As shown, there is a significant difference in the DI values between the pavement structural categories when preventive maintenance is applied to the pavement; the crush and shape with HMA surface pavements have lower DI values.

The DI values were found to be highly variable and relatively low across all pavement structure categories. In fact, many of the DI values reported are considered low at the time the preservation method was applied to the pavement. It is expected that one of the other performance indicators (rut depth or IRI) was the reason for applying the preservation activity to the pavement surface, or the preservation method is applied on an age or subjective basis not related to surface condition. The next two sections of this chapter focus on rut depth and IRI. [It is expected that preventive maintenance methods are placed on an age basis, rather than surface condition.]

An empirical relationship was used to estimate the rate of deterioration of HMA pavements and overlays. This deterioration relationship is shown as equation 1 and has been used to predict the distress indices of flexible pavements and HMA overlays for use in life cycle cost analyses for the Ontario Ministry of Transportation and other agencies (Smith, et al., 1998). The deterioration coefficients (*a* and *b* regression constants) were used to identify PM segments with similar performance (good versus poor performance).

$$DI = 100\left(1 - e^{-a\left(\frac{t}{t_{design}}\right)^{b}}\right) \tag{1}$$

Where:

t = Time in years.

t_{design} = Design life or period in years.

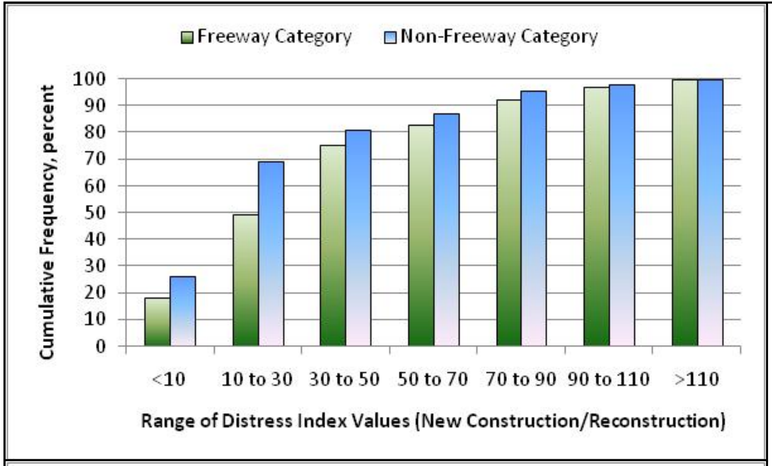
a, b = Regression constants referred to in this report as deterioration coefficients derived using linear regression techniques to minimize the error between the predicted and measured DI values for individual PM segments.

Table 6. Average and Range of DI Values When Preservation Activity was Placed on Pavement Surface

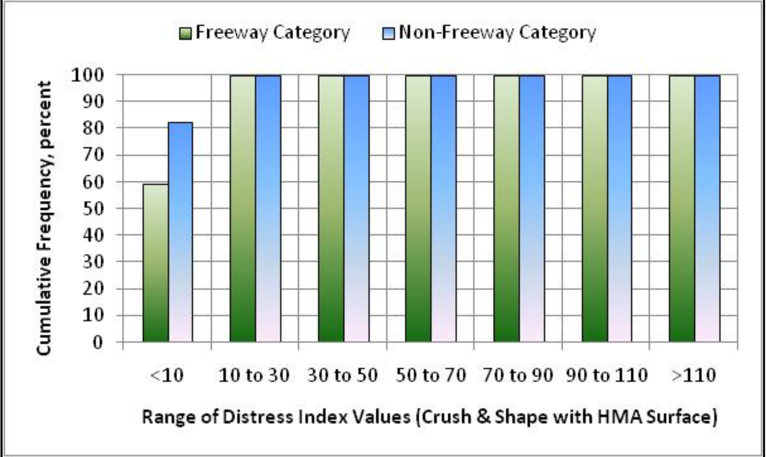
Pavement Structure Category	Highway Category	DI Values	
		Average	Range
New Construction	Freeway	36.9	2 to 134
	Non-Freeway	27.5	0 to 123
Crush & Shape with HMA Surface	Freeway	8.7	1 to 19
	Non-Freeway	5.3	0 to 19
Resurfacing with and without Milling	Freeway	12.6	6 to 26
	Non-Freeway	26.5	1 to 102

The DI deterioration coefficients (refer to equation 1) were determined through linear regression for each PM segment with and without preventive maintenance. The design life of flexible pavements was assumed to be 20 years for all PM segments for new flexible pavements and 15 years for HMA overlays. For these analyses, the underlying assumption was that all of the flexible pavements were designed using the same procedure and criteria. The assumptions listed near the beginning of Chapter 3 also apply to the analyses completed using the deterioration relationships.

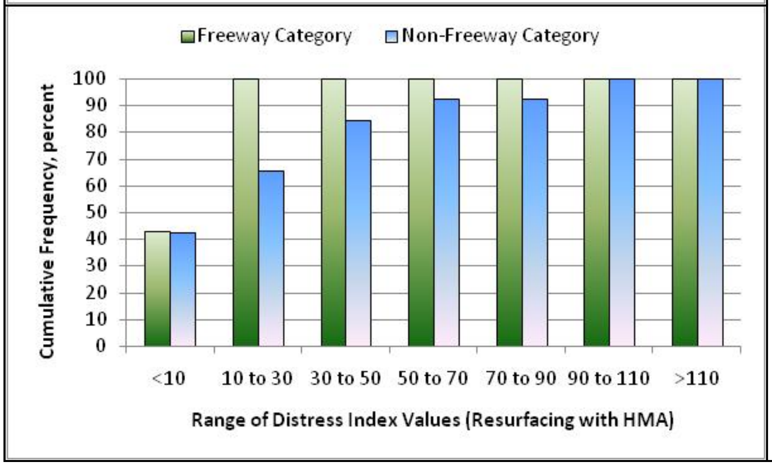
Figures 24 and 25 compare the measured and predicted DI values for selected PM segments for which different preservation methods were placed at different times. As shown, equation 1 does a reasonable simulation of predicting the increase in DI values over time. Conversely, there are some PM segments for which equation 1 does not accurately simulate the change or increase in DI values measured over time. This difference between predicted and measured values is probably related to the measurement error, other maintenance activities not recorded in the PM database, and/or equation 1.



DI values for when preservation activity applied to pavement surface for the New Construction/Reconstruction data group.

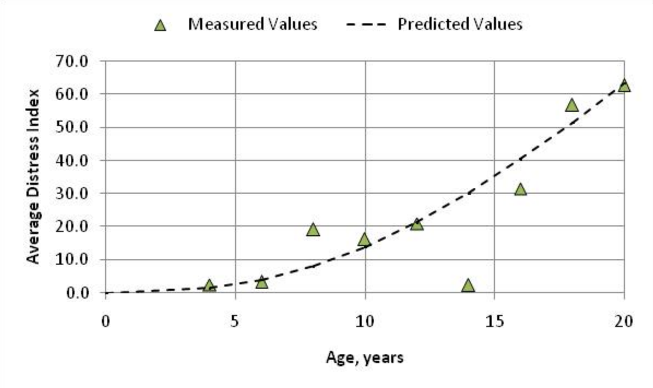


DI values for when preservation activity applied to pavement surface for the Crush and Shape with HMA Surface data group.

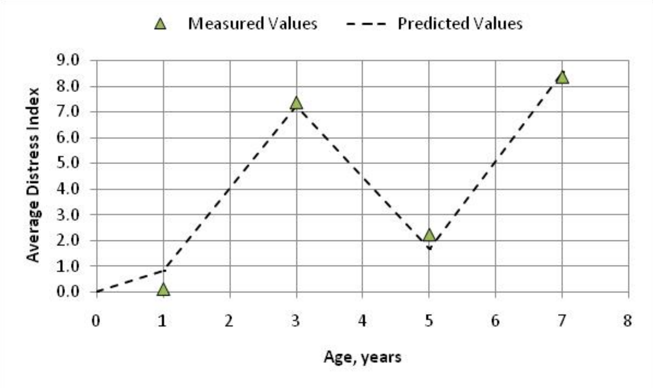


DI values for when preservation activity applied to pavement surface for the Resurfacing and Mill and Resurface data groups.

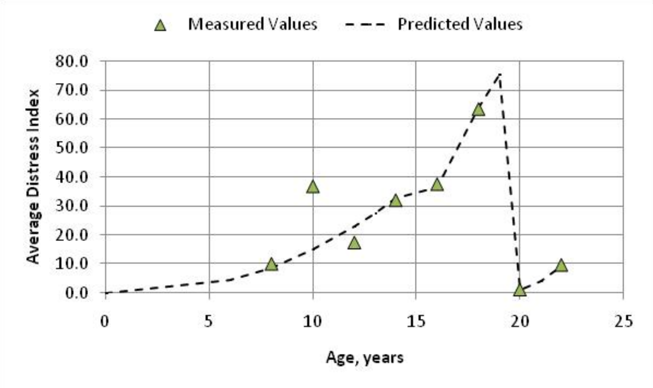
Figure 23. Cumulative Frequency of the DI Values When Preservation Method Was Placed on Pavement Surface



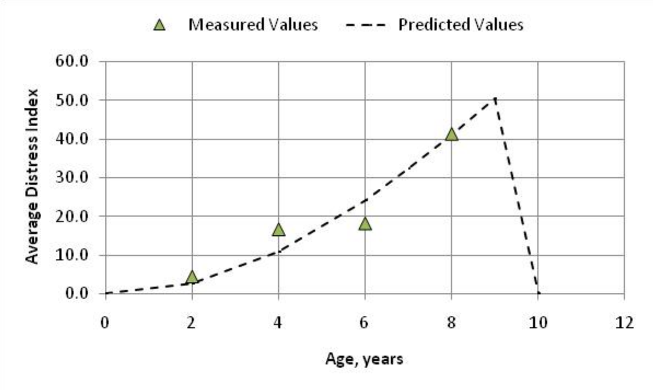
Crush & Shape with HMA Surface; Non-Freeway
PM Segment, ID #1; M-72
East of M-33 to the Oscoda/Alcona County Line
BMP – 0.102; EMP – 8.239
CS – 68042
JN – 25538
No preservation/maintenance activity recorded in the PM database but section was milled & resurfaced in 2000 (12 years old).



Mill and Resurface; Non-Freeway
PM Segment, ID #73; M-32
West of Hallock Road to Murner Road
BMP – 4.140; EMP – 9.760
CS – 69021
JN – 32331
OCF & Micro-Surfacing in 2002 (year 3)

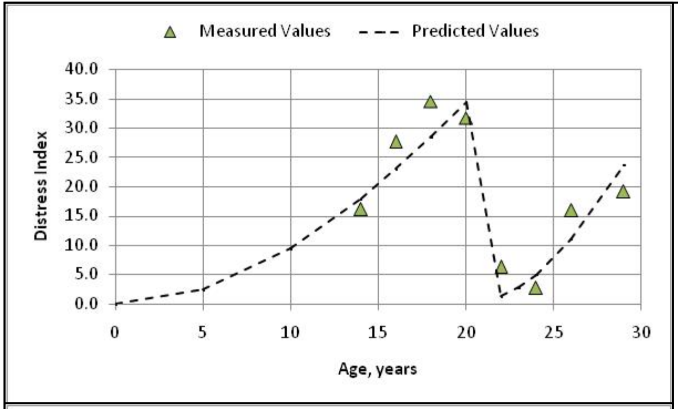


New Construction/Reconstruction; Non-Freeway
PM Segment, ID #15; M-54
Grand Blanc Road to Gibson Road
BMP – 0.869; EMP – 2.032
CS – 25074
JN – 00337
Novachip in 2000 (year 14)
Cold Mill-Resurface in 2005 (year 19)

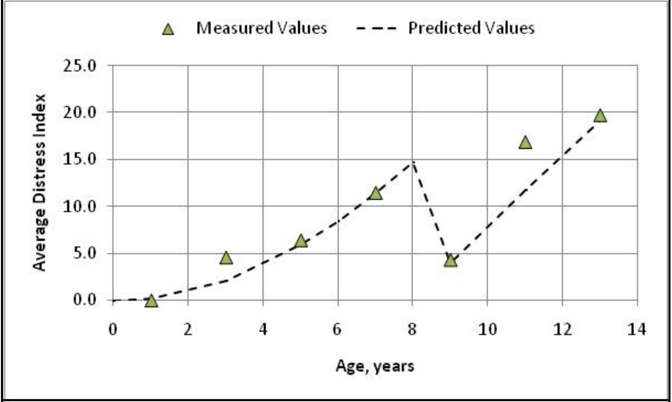


Resurface Flexible Pavement; Non-Freeway
PM Segment, ID #201; US-131
North of Lake Street to South of Bear River Road
BMP – 12.982; EMP – 13.251
CS – 15091
JN – 32322
Cold Mill & Resurface in 2006 (year 9)

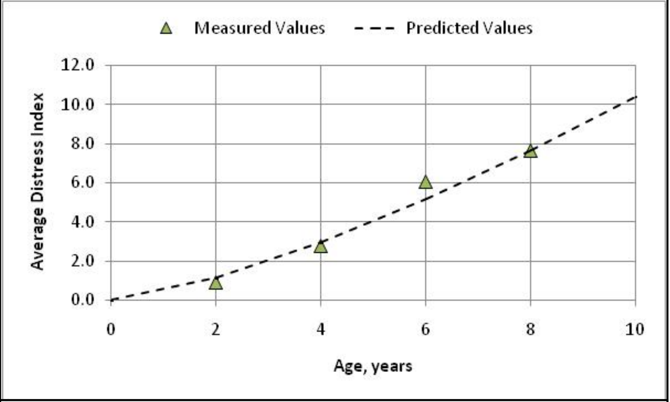
Figure 24. Predicted and Measured DI Values for Selected Non-Freeway PM Segments



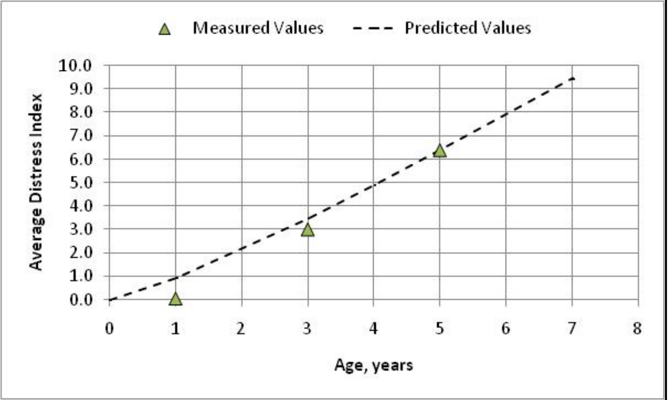
New Construction/Reconstruction;
Freeway-Divided Highway
PM Segment, ID #9N; M-99-NB
Victor Avenue to Moores River Road
BMP – 4.260; EMP – 5.178
CS – 33011
JN – 00434
Cold-Mill & Resurface in 1998 (year 20)



Crush & Shape with HMA Surface;
Freeway-Divided Highway
PM Segment, ID #1S; I-75 SB
Sturgeon Valley Road to
Otsego/Cheboygan County Line
BMP – 8.235; EMP – 13.105
CS – 69014
JN – 30727
OCF & Micro-Surfacing in 2002 (year 8)
OCF & Micro-Surfacing in 2007 (year 13)



Mill & Resurface; Freeway-Divided
Highway
PM Segment, ID #27; I-196 SB
I-94 to Berrien/Van Buren County Line
BMP – 0.000; EMP – 7.346
CS – 11111
JN – 44788
No preservation/maintenance activity



Resurface Flexible Pavement; Freeway-
Divided Highway
PM Segment, ID #25S; I-75 SB
I-94 to Berrien/Van Buren County Line
BMP – 0.000; EMP – 2.100
CS – 16091
JN – 53353
No preservation/maintenance activity

Figure 25. Predicted and Measured DI Values for Selected Freeway, Divided Highway PM Segments

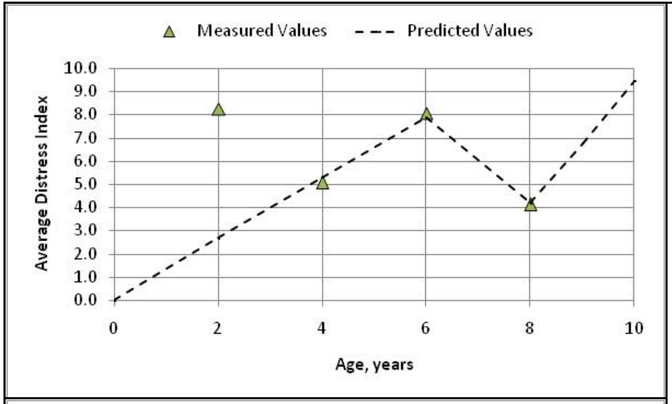
Figure 26 provides an example of some discrepancies for a few PM segments. The individual DI measurements for determining the deterioration coefficients of each PM segment were analyzed for outliers when sufficient time series data was available. Some of the data shown in Figure 26 would obviously be considered as outliers. When identified as an outlier, the individual DI measurement was excluded from determining the deterioration coefficients of equation 1. As an example, the DI values at year 2 for control segments I-96 EB and US-23 in Figure 26 are considered outliers and were excluded from determining the deterioration coefficients. Conversely, the DI values at years 6 and 8 for control segments M-32 and M-66, respectively, could also be identified as potential outliers in terms of the data. These data points, however, were not considered outliers because anomalies can occur resulting in accelerated increases in cracking.

Figure 27 includes an overall comparison of the predicted and measured DI values for each of the major data sets. As shown, equation 1 did a reasonable simulation of the DI values measured over time. The greater dispersion in the data was for the non-freeway, crush and shape with HMA surface structural category or data set – suggesting some confounding factor not adequately captured by equation 1.

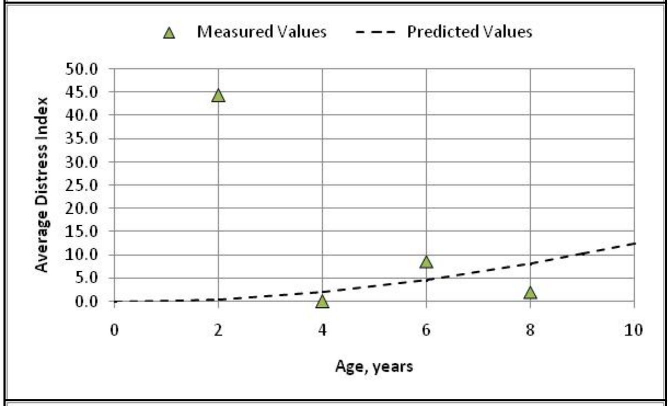
Figure 28 provides a comparison (or scatter plot) of the deterioration coefficients (*a* and *b* regression constants in equation 1) derived from each of the PM segments without any preservation method applied to the surface during the monitoring period. As shown, there is a lot of variability, which was expected. Correspondence between the deterioration coefficients and magnitude of the values were evaluated and compared between the different Regions, highway type, pavement structure, preservation strategy, and soil type. No significant or statistical correspondence was identified between the deterioration coefficients for the different data sets. This observation suggests the DI deterioration coefficients are probably site or project specific and/or affected by parameters not included in the MDOT performance database.

Trend lines are included in Figure 28 for the different pavement structural categories to illustrate there is little difference in the deterioration coefficients between the different structural categories. The "b" coefficient is slightly lower for the mill and resurface category, but it is insignificant considering the amount of variability in the data. The same is true for the other DI data categories noted above.

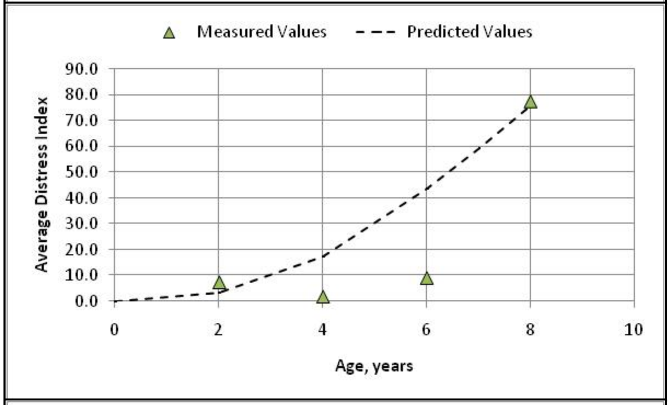
The deterioration coefficients were also analyzed to determine if the values were related to the DI values of the existing pavement prior to overlay placement or the application of preventive maintenance, and if the coefficients systematically change with the application of preventive maintenance. Figure 29 provides a comparison of the deterioration coefficients derived from the PM segments after preventive maintenance had been applied to the pavement surface. As shown, the "*a*" coefficient after preventive maintenance was applied is generally in the same range for pavements with no preventive maintenance. Conversely, the "*b*" coefficient is consistently smaller after preventive maintenance was placed. Smaller values of "b" mean that distresses are being delayed – the predicted DI value is smaller.



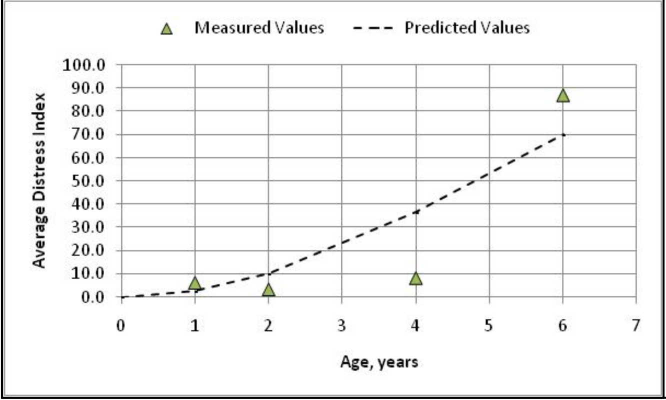
Mill & Resurface; Freeway-Divided Highway
PM Segment, ID #25E; I-96 EB
M-104 to 88th Avenue
BMP – 0.000; EMP – 3.867
CS – 70063
JN – 44155
OCF & Multiple Course Micro-Surface in 2003 (year 6).



Resurface Flexible Pavement; Non-Freeway
PM Segment, ID #199; US-23
East of Nearman to East of Court Street
BMP – 7.349; EMP – 9.430
CS – 06072
JN – 32357
No preservation/maintenance activity.



Resurface Flexible Pavement; Non-Freeway
PM Segment, ID #211; M-66
S Drive South to L Drive
BMP – 2.222; EMP – 6.337
CS – 13031
JN – 34497
No preservation/maintenance activity.



Mill & Resurface; Non-Freeway
PM Segment, ID #77; M-32
Baker Road to West Street
BMP – 12.435; EMP – 14.441
CS – 60021
JN – 51248
No preservation/maintenance activity.

Figure 26. Examples of PM Segments for Which Equation 1 Does Not Simulate the DI Changes over Time

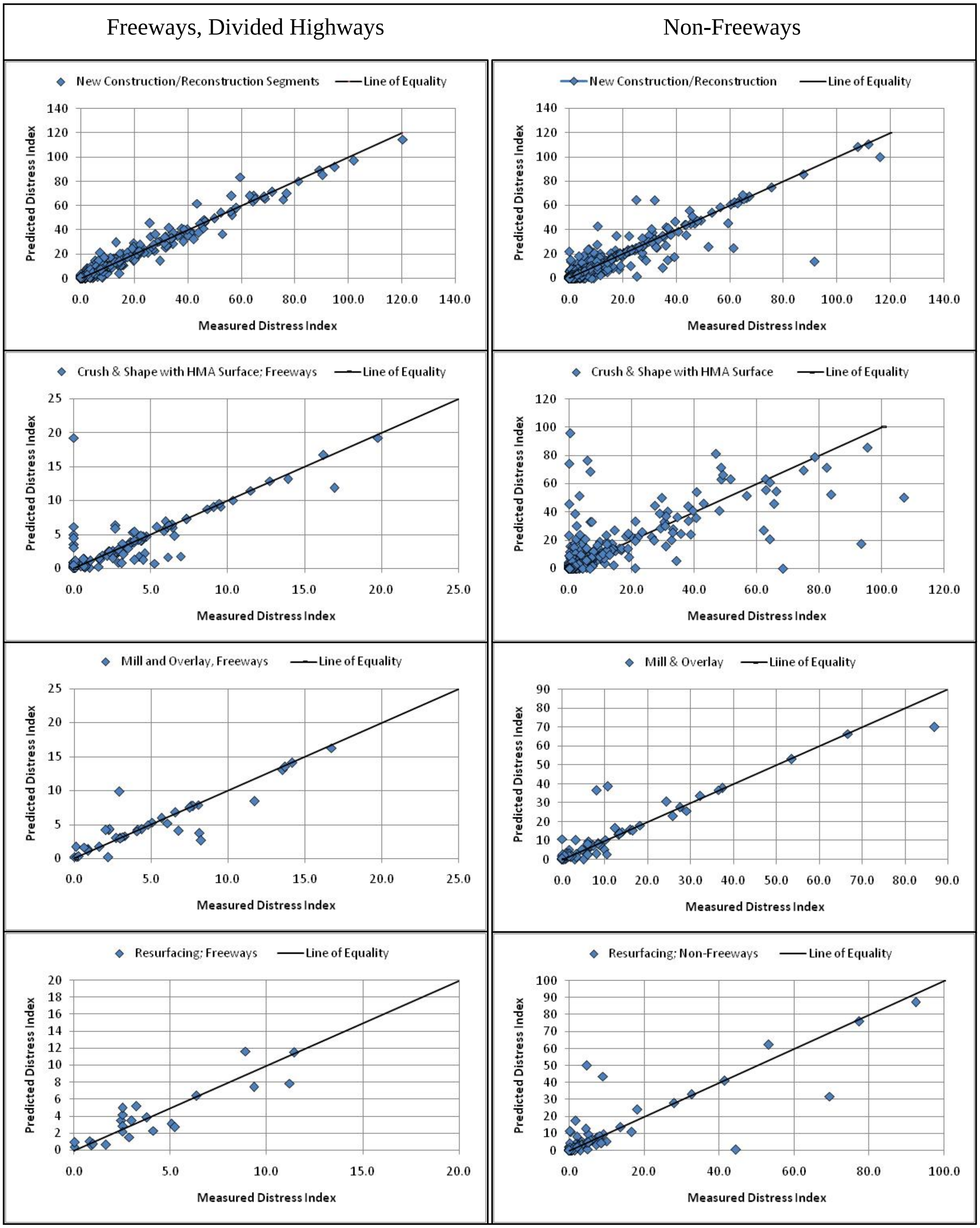


Figure 27. Comparison of Measured and Predicted DI Values

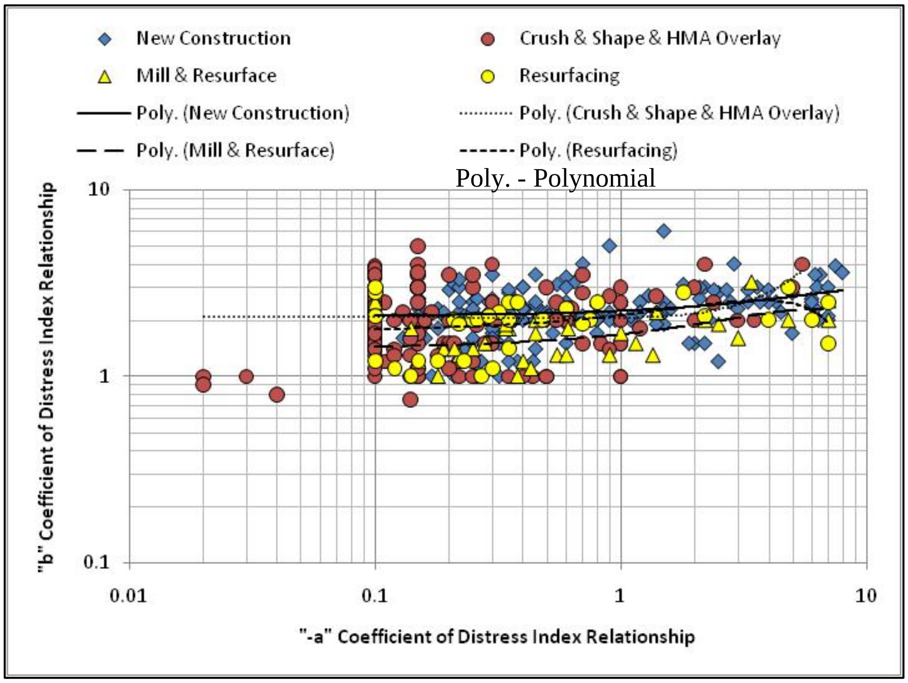


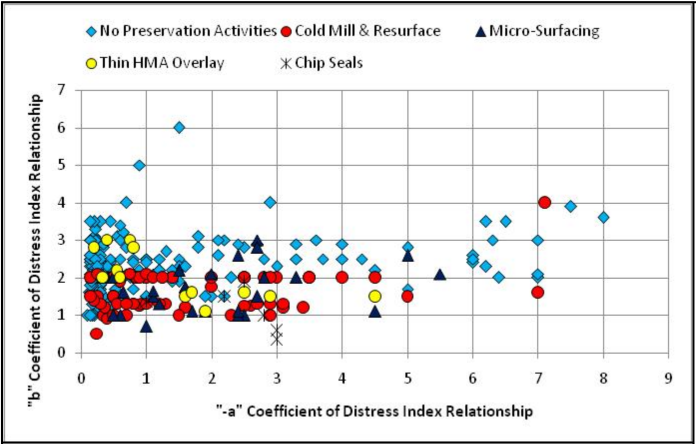
Figure 28. Comparison of DI Deterioration Coefficients for Pavement Structures Without any Preservation Activity

Greater variation or a larger range in the “b” deterioration coefficient (refer to equation 1) exists for the new construction and crush and shape with HMA surface categories, as compared to the mill & resurface and resurfacing categories. The reason for this observation in the data is unknown. The greater variation for both deterioration coefficients usually is found for HMA overlays, because the condition of the existing pavement has an effect or influence on overlay performance (Rauhut, et al., 1999; Von Quintus, et al., 2000).

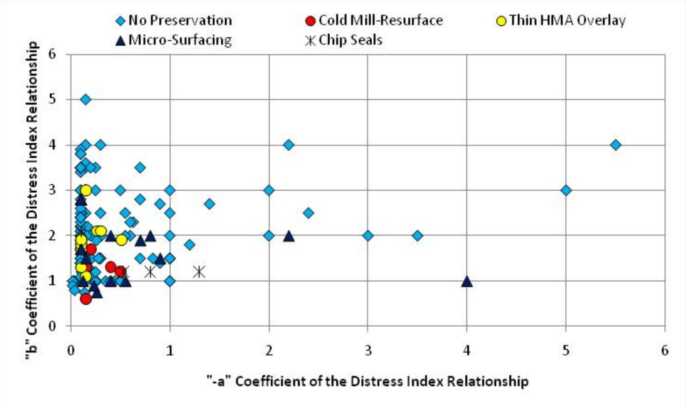
Table 7 lists the median DI deterioration coefficients for the different data sets prior to the application of any method, while Table 8 lists the median values for the different preservation methods commonly used in Michigan. Good performance, related to the DI values, can be defined by the lower “b” values in combination with higher “a” values (smaller negative value), while poor performance is defined by higher “b” values and lower “a” values. For example, the following quantifies good and poor performance in terms of the deterioration coefficients regressed from MDOT’s database.

Performance Category, Distress Index	Range of DI Deterioration Coefficients	
	A	b
Good Performance	> -0.2	< 1.5
Poor Performance	< -2.8	> 2.7

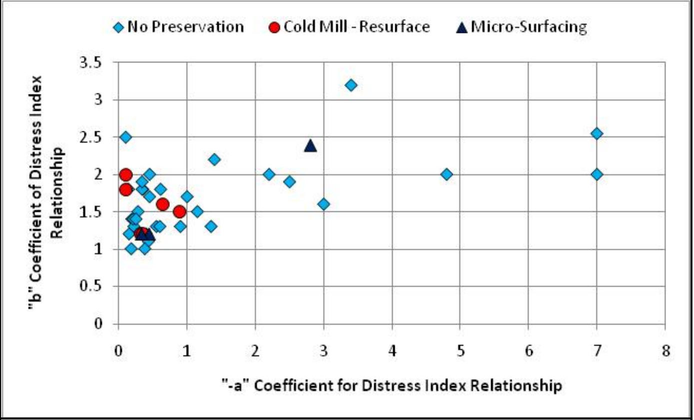
The above values were based on the range of deterioration coefficients determined for the PM roadway segments exhibiting good and poor performance (refer to Appendix A).



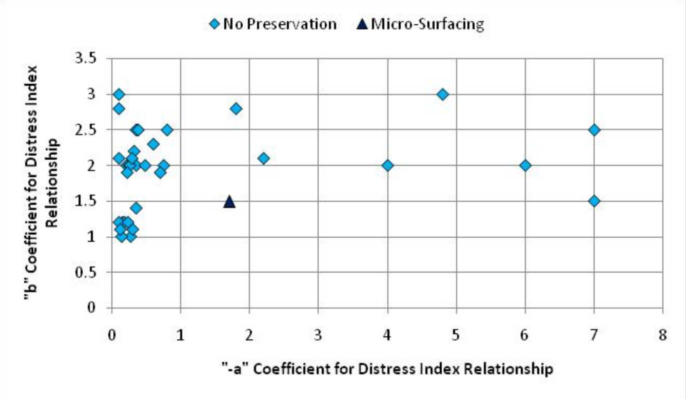
Preservation Methods Placed on Surface of Pavement Structural Category New Construction-Reconstruction



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Crush and Shape with HMA Surface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Mill and Resurface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Resurface Flexible Pavement

Figure 29. Comparison of DI Deterioration Coefficients for Different Preservation Methods

Table 7. Median DI Deterioration Coefficients for PM Roadway SegmentsWithout any Preservation Method Placed on the Pavement Surface

Structure Category	Preservation Treatment	DI Deterioration Coefficients	
		a	b
New Construction/Reconstruction	None	-0.5	2.2
Crush & Shape with HMA Surface	None	-0.15	2.0
Mill and Resurface with HMA	None	-0.45	1.7
Resurface with HMA (No Milling)	None	-0.32	2.0

Table 8. Median DI Deterioration Coefficients for PM Roadway SegmentsWith Different Preservation Methods Placed on the Pavement Surface

Structure Category	Preservation Treatment	DI Deterioration Coefficients	
		a	b
New Construction/Reconstruction	Cold-Mill & Resurface	-1.2	1.5
	HMA Overlay	-0.80	2.0
	Micro-Surface	-1.65	1.75
	Chip Seal	-2.65	1.2
Crush & Shape with HMA Surface	Cold-Mill & Resurface	-0.17	1.3
	HMA Overlay	-0.10	1.8
	Micro-Surface	-0.26	1.7
	Chip Seal	-0.60	1.2
Mill and Resurface with HMA	Cold-Mill & Resurface	-0.32	1.6
	HMA Overlay	---	---
	Micro-Surface	-0.45	1.2
	Chip Seal	---	---
Resurface with HMA (No Milling)	Cold-Mill & Resurface	---	---
	HMA Overlay	---	---
	Micro-Surface	---	---
	Chip Seal	---	---
The cells without a numerical value had an insufficient number of PM roadway segments and/or insufficient DI values to determine the deterioration coefficients for that category.			

Some of the PM segments in MDOT's database have received as many as three preservation methods within the monitoring period. The question becomes: how many preservation methods can be applied to the pavement and still extend the service life before reconstruction is needed? There is insufficient data within the PM database to answer this question. More PM segments, however, have received at least two preservation methods and are still in service with DI values below 50. Thus, two preventive maintenance applications were used in evaluating the extended service life or delaying surface distress from these preventive maintenance activities.

Figure 30 shows the predicted DI values for two conditions using equation 1: (1) not using preventive maintenance (letting the pavement deteriorate to a DI value of 50); and (2) using multiple preservation methods based on the DI values included in Table 1. Different preservation methods were used for the examples included in Figure 30 to be consistent with MDOT actual practice recorded in the performance database. The DI deterioration coefficients (equation 1) used in the examples represent the median values (refer to Tables 7 and 8). As shown, MDOT policy of using pavement preservation to increase pavement service life appears to be very beneficial from a DI standpoint. Of the different pavement preservation methods, the HMA overlay was found to have the better performance, while chip seals were found to have poorer performance.

The other observation is that the crush and shape with HMA surface was found to exhibit more resistance to cracking or have better performance in comparison to the other structural categories (refer to Figure 30, and Tables 5 and 6). The reason for this better performance is unknown, but could be related to the fact that preventive maintenance is applied to these pavements when they are in a much better condition (lower DI values). Regressing the deterioration coefficients (a and b; refer to equation 1) from low DI values (less than 10) can result in inaccurate deterioration coefficients and predictions using exponential relationships (or power laws) to predict much higher DI values (near 50) – extrapolating the age to a DI value of 50.

There are PM segments that deviate significantly from the median values (refer to Table 7). Figure 31 illustrates the range in performance based on the DI values from PM segments exhibiting good (delayed distress) and poor (accelerated distress) performance. As shown, the performance between the two groups (refer to Appendix A) are significantly different.

The deterioration coefficients used to predict the DI values for good and poor performance are included in Figure 31. Eliminating only a few poor performers can extend the average service life of asphalt pavements. The following provides a general definition for delayed and accelerated distress.

- Delayed Distress or Good Performance is defined as new pavement and rehabilitation projects that have an average DI value less than 15 for 10+ years, or an average DI value less than 50 for 30+ years.
- Accelerated Distress or Poor Performance is defined as new pavement and rehabilitation projects that have an average DI value greater than 25 in less than 10 years, or an average DI value greater than 50 in less than 15 years.

The reason(s) for the range in performance of DI deterioration coefficients listed above was not found to be related to parameters recorded in the PM database. It is expected that the deterioration coefficients are site or project specific, and heavily influenced by materials and construction methods which are not documented in the MDOT performance database. Chapter 5 includes an analysis of the detailed distress data, rather than the DI composite value, to determine the probable causes for poor performance.

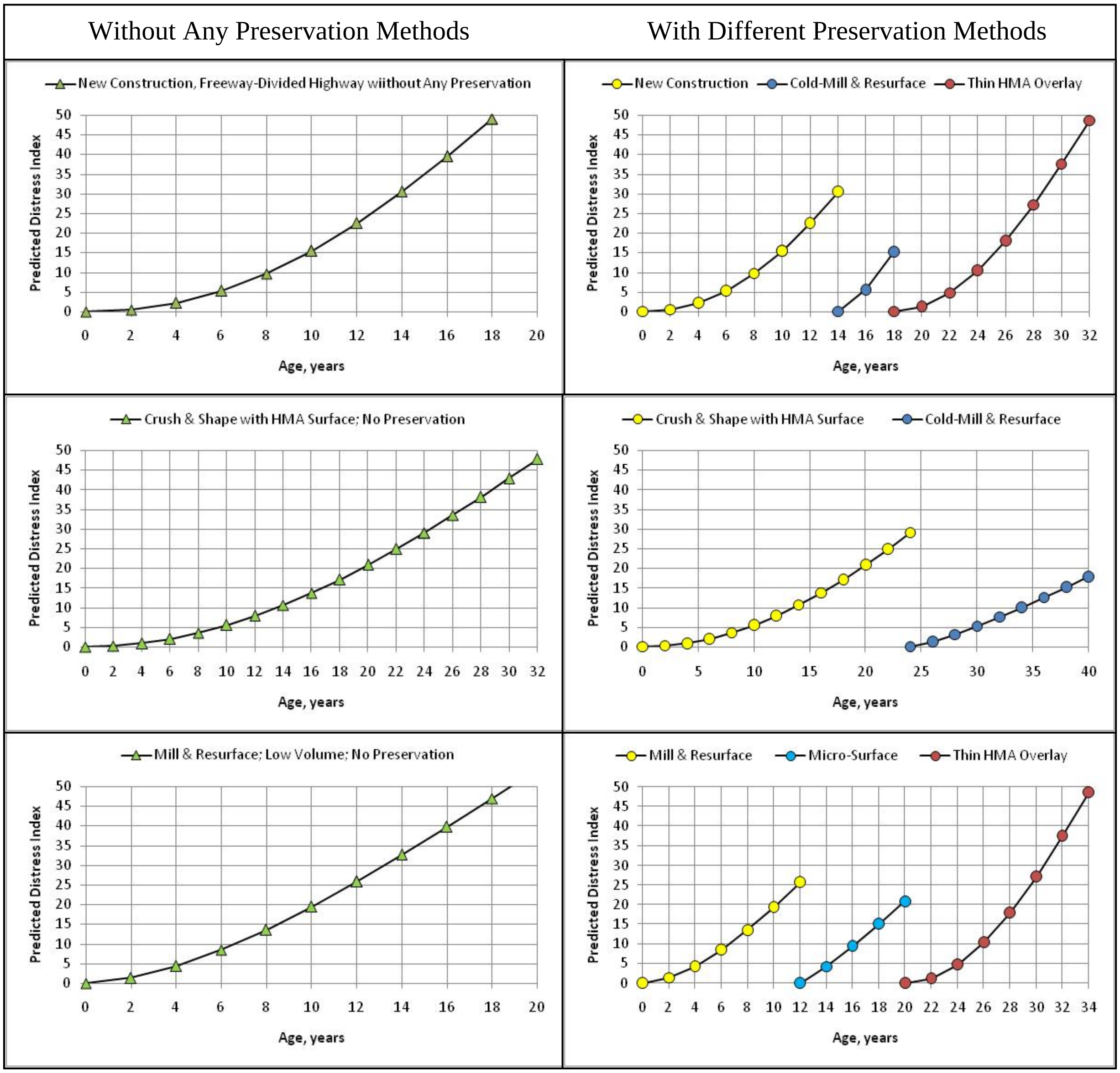


Figure 30. Predicted Service Life Based on DI Values from Equation 1 with and without Using Preservation Methods

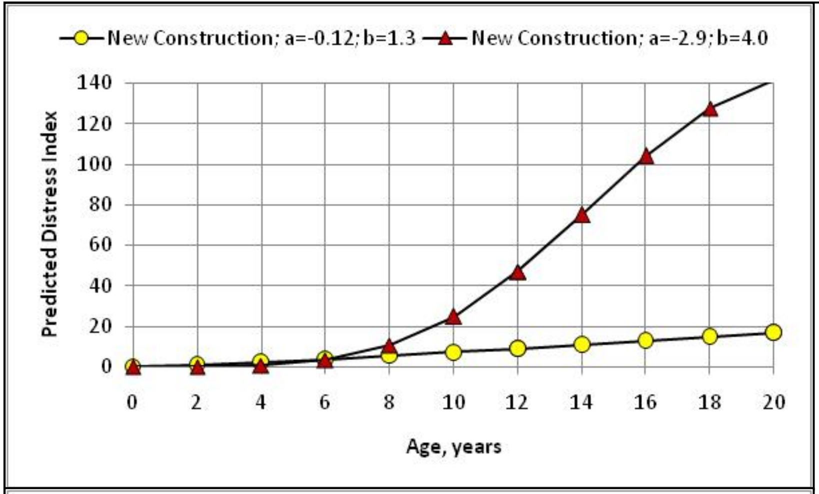
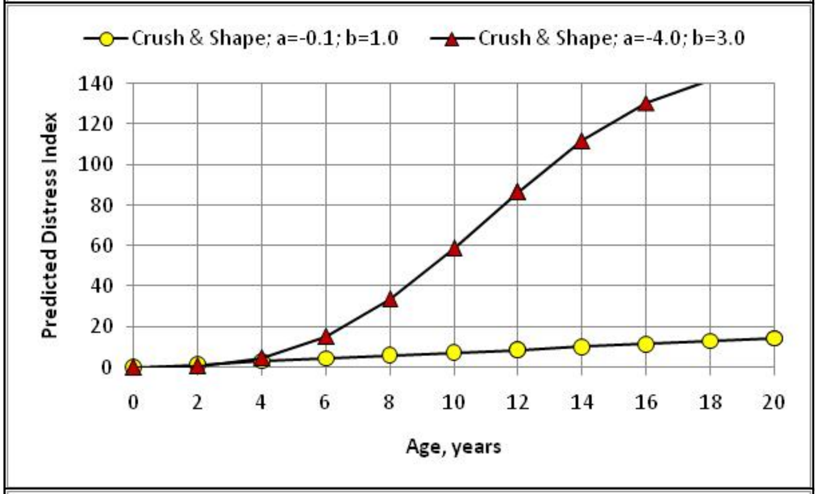
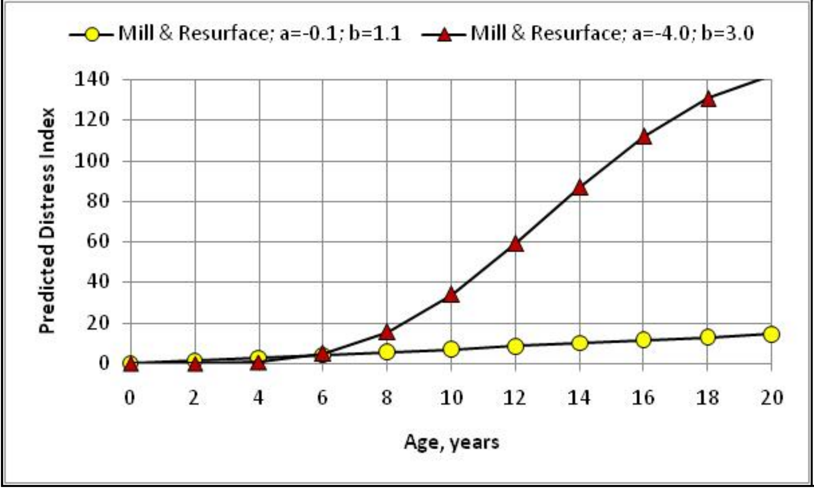
	New Construction or Reconstruction Category showing good & poor performance based on distress index.
	Crush & Shape with HMA Surface Category showing good & poor performance based on distress index.
	Mill & Resurface Category showing good & poor performance based on distress index.

Figure 31. Examples of Good and Poor Performance Based on Extreme Values for the DI Deterioration Coefficients for Different Structural Categories

4.2 Rut Depth

The average and range of rut depths for different data sets are listed in Table 9 for when the preservation activity was applied to the pavement surface, while Figure 32 shows the cumulative frequency of those rut depths. As shown, the new construction or reconstruction category was found to have more PM segments with the higher rut depths prior to placing any preservation method. The average rut depth reported for the other structural categories were found to be low. It is expected that the preservation method was placed for some other reason and not excessive rutting.

Table 9. Average and Range of Rut Depths When Preservation Activity was Placed on to Pavement Surface

Pavement Structure Category	Highway Category	Rut Depths	
		Average	Range
New Construction	Freeway	0.29	0.26 to 0.32
	Non-Freeway	0.32	0.20 to 0.44
Crush & Shape with HMA Surface	Freeway	0.19	0.12 to 0.45
	Non-Freeway	0.17	0.09 to 0.35
Resurfacing with and without Milling	Freeway	0.16	0.07 to 0.26
	Non-Freeway	0.26	0.15 to 0.36

The formulation or accumulation of rutting in the MDOT database was described by the following empirical relationship.[1]

$$RD = 0.05 + k_1 \left(Age\right)^{k_2} \qquad (2)$$

Where:

Age = Time after HMA placement in years.

k_1, k_2 = Regression constants referred to in this report as deterioration coefficients derived using linear regression techniques to minimize the error between the predicted and measured rut depths for individual PM segments.

Most empirical and mechanistic-empirical relationships use 18-kip equivalent single axle loads (ESALs). Age, however, has been used when the same mixture design procedure and specifications have been used to design and accept the materials (refer to assumptions included near the beginning of Chapter 3).

The average rut depth deterioration coefficients (refer to equation 2) were determined through linear regression for each PM segment. For this analysis, the underlying assumption was that all of the HMA mixtures and pavement structures were designed using the same procedures. Figure 33 compares the measured and predicted rut depths for all PM segments and suggests that

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[1] Equation 2 is similar to the standard rut depth power law used to predict rut depth based on the number of load applications – typically 18-kip ESALs (Von Quintus, et al., 1991). The number of load applications can be replaced by age in evaluating network rut depth data, which was used in this study to segregate pavements with good and poor performance (Rauhut, et al., 1999).

equation 2 is not a good simulation of the increase in rut depth over time, because of the amount of scatter around the line of equality.

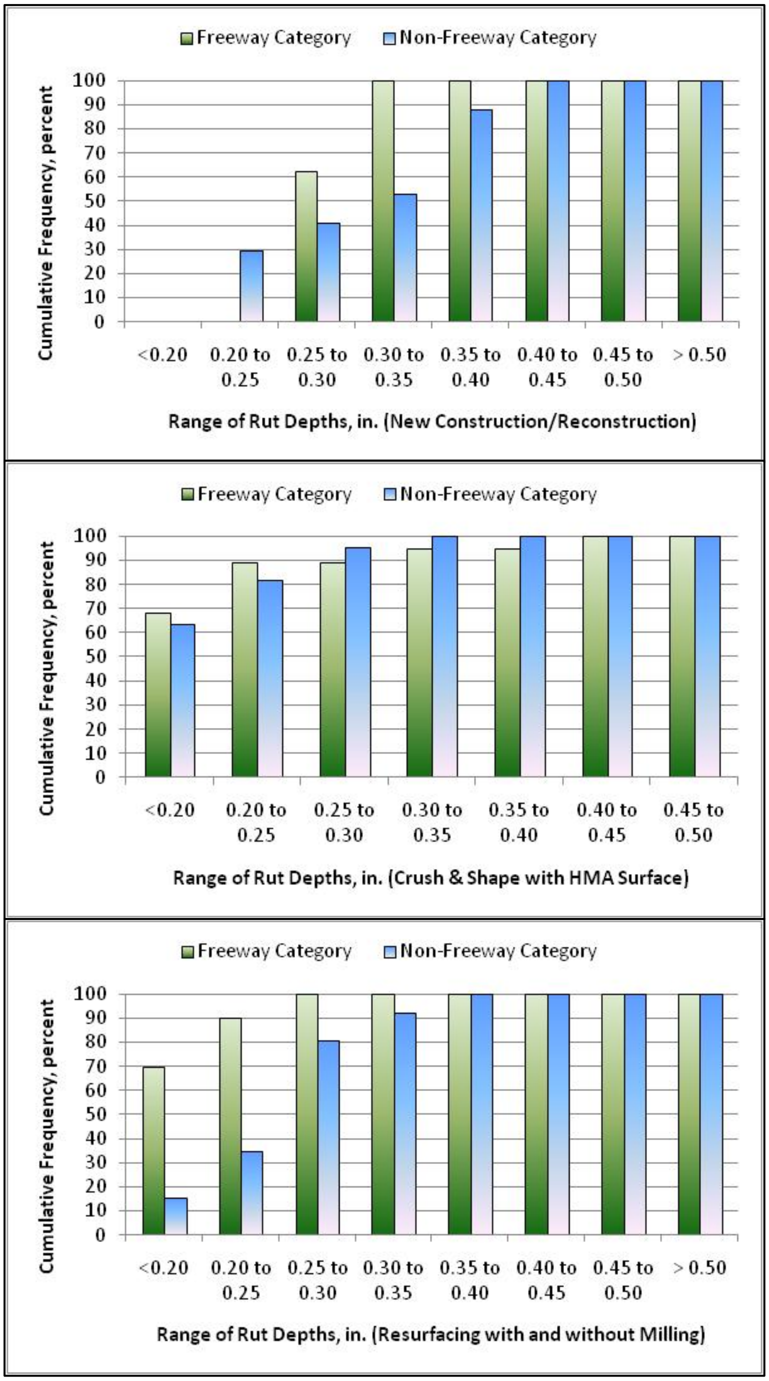


Figure 32. Cumulative Frequency of the Rut Depths When Preservation Method Was Placed on Pavement Surface

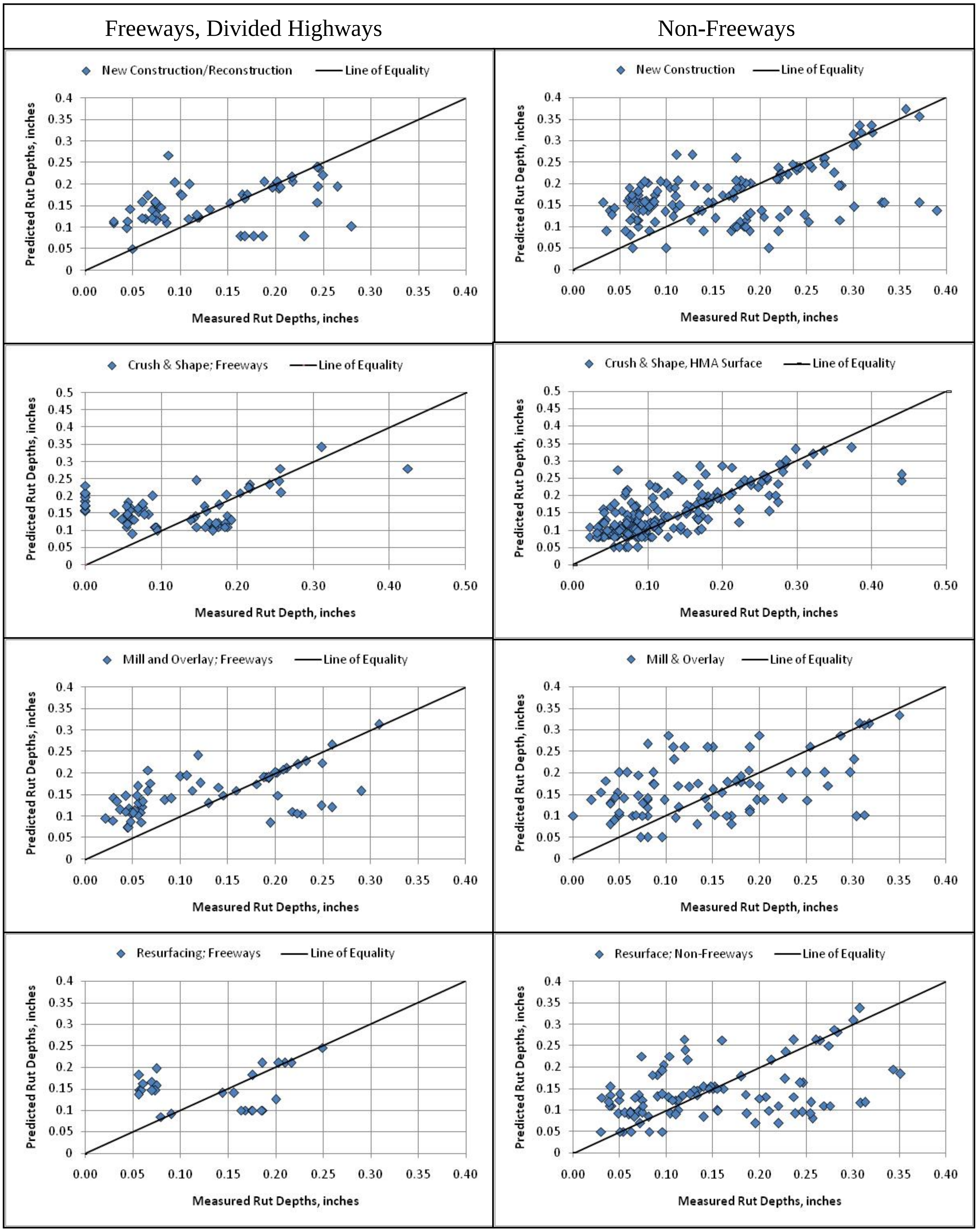


Figure 33. Comparison of Measured and Predicted Rut Depths for the PM Segments

Figure 34 is a comparison of the measured and predicted rut depths for selected PM segments. Most of the variability or difference between the measured and predicted rut depths is believed to be associated with measurement error and/or a change in the method and equipment used to measure rut depth. For example, many of these PM segments show an increase in measured rut depths, followed by a decrease and then increase in the values. This amount of variation in the average measured values over time, however, is common for network data and even common in the LTPP database where the same equipment and precise procedures were used to measure rutting over time.

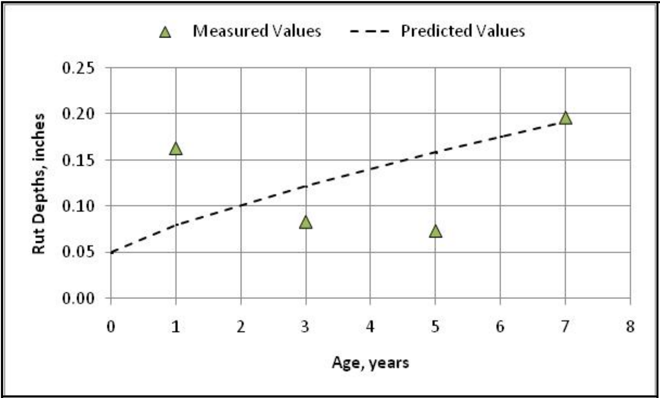
Tables 10 and 11 list the median average rut depth deterioration coefficients (equation 2) for the different data sets, while Figure 35 shows a comparison (or scatter plots) of the rutting deterioration coefficients. In summary, the rut depth deterioration coefficients were found to be similar for most of the PM segments, which exhibit good resistance to rutting.

There are a few PM segments, however, that exhibit significantly higher rut depths. Correspondence between the rut depth deterioration coefficients and magnitude of the values were compared between different regions, highway type, pavement structure, preservation strategy, and soil type. No significant difference or correspondence between the rutting deterioration coefficients was identified for the different data sets. As such, the PM segments with the higher rut depths rutting were identified. The following quantifies good and poor performance in terms of the rut depth deterioration coefficients regressed from MDOT's database, which were based on the range of deterioration coefficients determined for the PM segments that exhibited lower and higher rut depths (refer to Table 9 for the range of measured values).

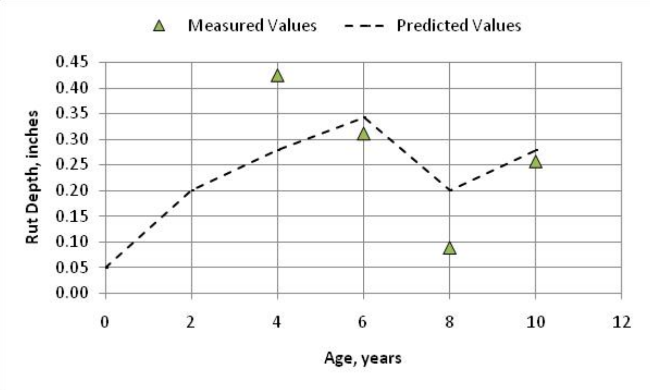
Performance Category, Rut Depth	Range of Rut Depth Deterioration Coefficients	
	k_1	k_2
Good Performance	< 0.05	< 0.60
Poor Performance	> 0.08	> 0.70

Figure 36 illustrates predicted values using the median rutting deterioration coefficients for different combinations of structure and preservation methods. Few of the PM segments will exceed an average rut depth of 0.35 inches; suggesting that rutting is not a critical parameter causing the application of pavement preservation methods or reconstruction. On the average, the PM segments within the new construction or reconstruction category did exhibit higher rut depths (refer to Table 9). The reason for this poorer rutting performance is unknown, but it could be related to accumulation of rutting in the unbound layers and subgrade. Rutting in the unbound layers and subgrade will lead to greater overall rutting, as compared to when rutting is confined to the HMA layers.

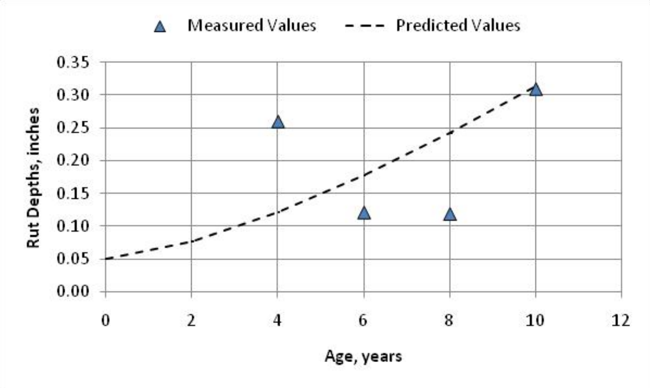
Different preventive maintenance methods were used for the examples included in Figure 36 to be consistent with MDOT actual practice recorded in the performance database. Of the different pavement preservation methods, the cold-mill and resurface and thin HMA overlay placed over the mill and resurface category were found to have the poorer rutting performance (refer to Figure 36).



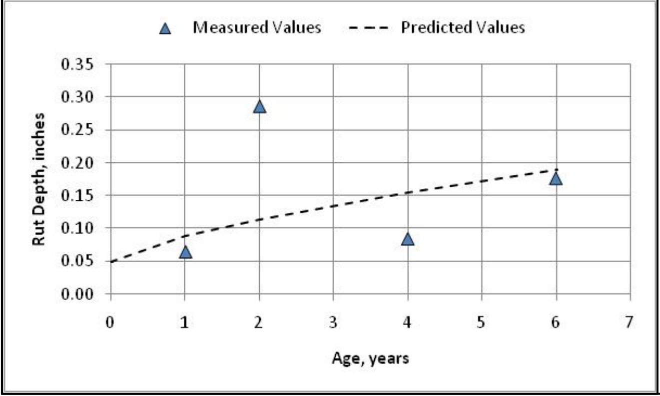
New Construction or Reconstruction;
Freeway-Divided Highway
PM Segment, ID #54N; US-127 NB
Roosevelt Road to North of Roosevelt Road
BMP – 3.030; EMP – 3.451
CS – 29011
JN – 43893
No preservation/maintenance activity.



Crush & Shape with HMA Surface;
Freeway-Divided Highway
PM Segment, ID #9S; I-75 SB
Nine Mile Hill Road to North of M-18
BMP – 13.066; EMP – 19.208
CS – 72061
JN – 34066
Micro-Surface placed in 2003 (year 6).

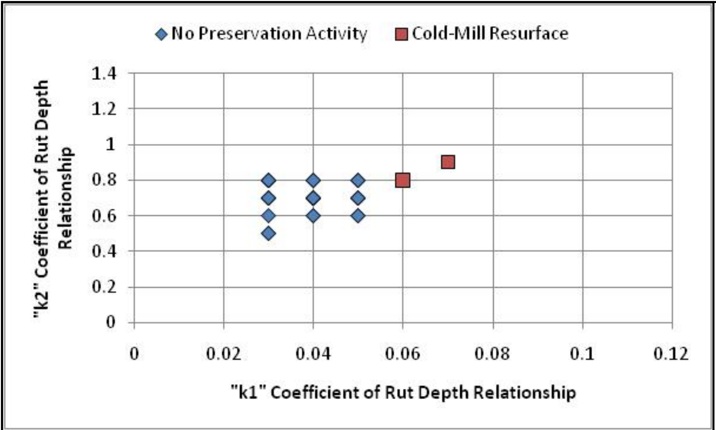


Mill & Resurface Flexible Pavement;
Freeway-Divided Highway
PM Segment, ID #27S; I-196 SB
I-94 to Berrien/Van Buren County Line
BMP – 0.000; EMP – 7.346
CS – 11111
JN – 44788
No preservation/maintenance activity.

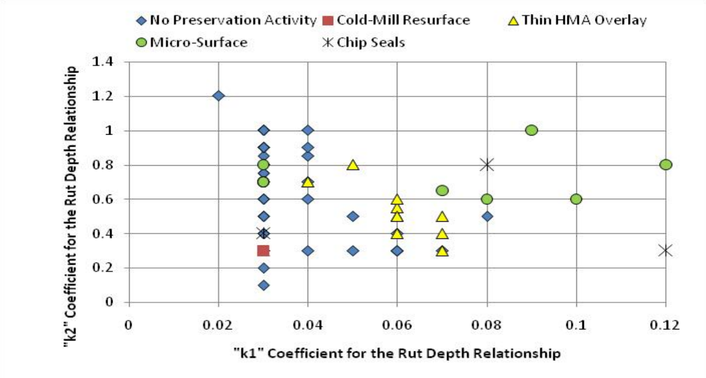


New Construction or Reconstruction;
Non-Freeway
PM Segment, ID #153; M-75
West of Moll Drive to East of Air Industrial Drive
BMP – 3.600; EMP – 4.109
CS – 15071
JN – 45002
No preservation/maintenance activity.

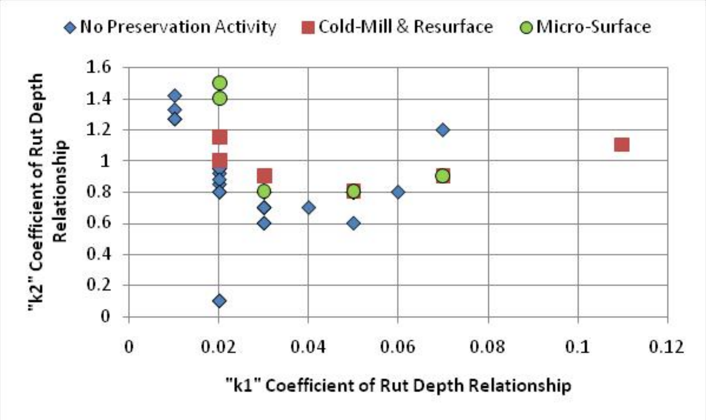
Figure 34. Comparison of Measured and Predicted Rut Depths for PM Segments Illustrating the Extensive Variability in the Measured Values



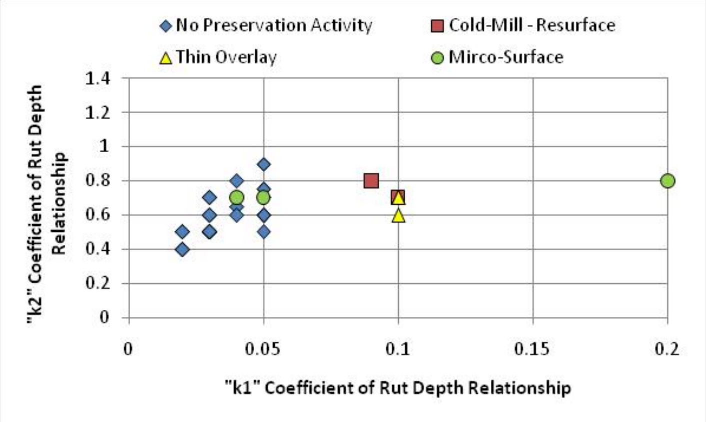
Preservation Methods Placed on Pavement Surface of Pavement Structural Category New Construction-Reconstruction



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Crush and Shape with HMA Surface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Mill and Resurface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Resurface Flexible Pavement

Figure 35. Comparison of Rut Depth Deterioration Coefficients for Different Methods

Table 10. Median Rut Depth Deterioration Coefficients for PM Roadway Segments Without any Preservation Method Placed on Pavement Surface

Structure Category	Preservation Treatment	RD Deterioration Coefficients	
		k_1	k_2
New Construction/Reconstruction	None	0.04	0.70
Crush & Shape with HMA Surface	None	0.03	0.60
Mill and Resurface with HMA	None	0.03	0.80
Resurface with HMA (No Milling)	None	0.03	0.60

Table 11. Median Rut Depth Deterioration Coefficients for PM Roadway Segments With Different Preservation Methods Placed on Pavement Surface

Structure Category	Preservation Treatment	RD Deterioration Coefficients	
		k_1	k_2
New Construction/Reconstruction	Cold-Mill Resurface	0.065	0.75
	HMA Overlay	---	---
	Micro-Surface	---	---
	Chip Seal	---	---
Crush & Shape with HMA Surface	Cold-Mill Resurface	0.03	0.30
	HMA Overlay	0.06	0.50
	Micro-Surface	0.03	0.70
	Chip Seal	0.08	0.40
Mill and Resurface with HMA	Cold-Mill Resurface	0.05	0.90
	HMA Overlay	---	---
	Micro-Surface	0.03	0.90
	Chip Seal	---	---
Resurface with HMA (No Milling)	Cold-Mill Resurface	0.09	0.80
	HMA Overlay	0.10	0.70
	Micro-Surface	0.04	0.70
	Chip Seal	---	---
The cells without a numerical value had an insufficient number of PM roadway segments and/or insufficient rut depth measuremetns to determine the deterioration coefficients for that category.			

There are PM segments that deviate from the median values. Figure 37 illustrates predicted rut depths from PM segments exhibiting good (delayed rutting) and poor performance (accelerated rutting). The deterioration coefficients used to predict the rut depths for good and poor performance are included in Figure 37. Refer to Figure 35 and Table 10 for a relative comparison of the deterioration coefficients used for the different structural categories (Figure 37) and those derived for individual PM segments. The following provides a general definition for delayed and accelerated rutting:

- Delayed Rutting or Good Performance is defined as new pavement and rehabilitation projects that have an average rut depth less than 0.25 inches for 10+ years, or an average rut depth less than 0.40 for 30+ years.

- Accelerated Rutting or Poor Performance is defined as new pavement and rehabilitation projects that have an average rut depth greater than 0.4 in less than 10 years.

The reason(s) for this range in performance or rut depth deterioration coefficients was not found to be related to the parameters recorded in the PM database. It is expected that the rut depth deterioration coefficients are project and material specific, and heavily influenced by compaction or construction methods that are not documented in the MDOT performance database.

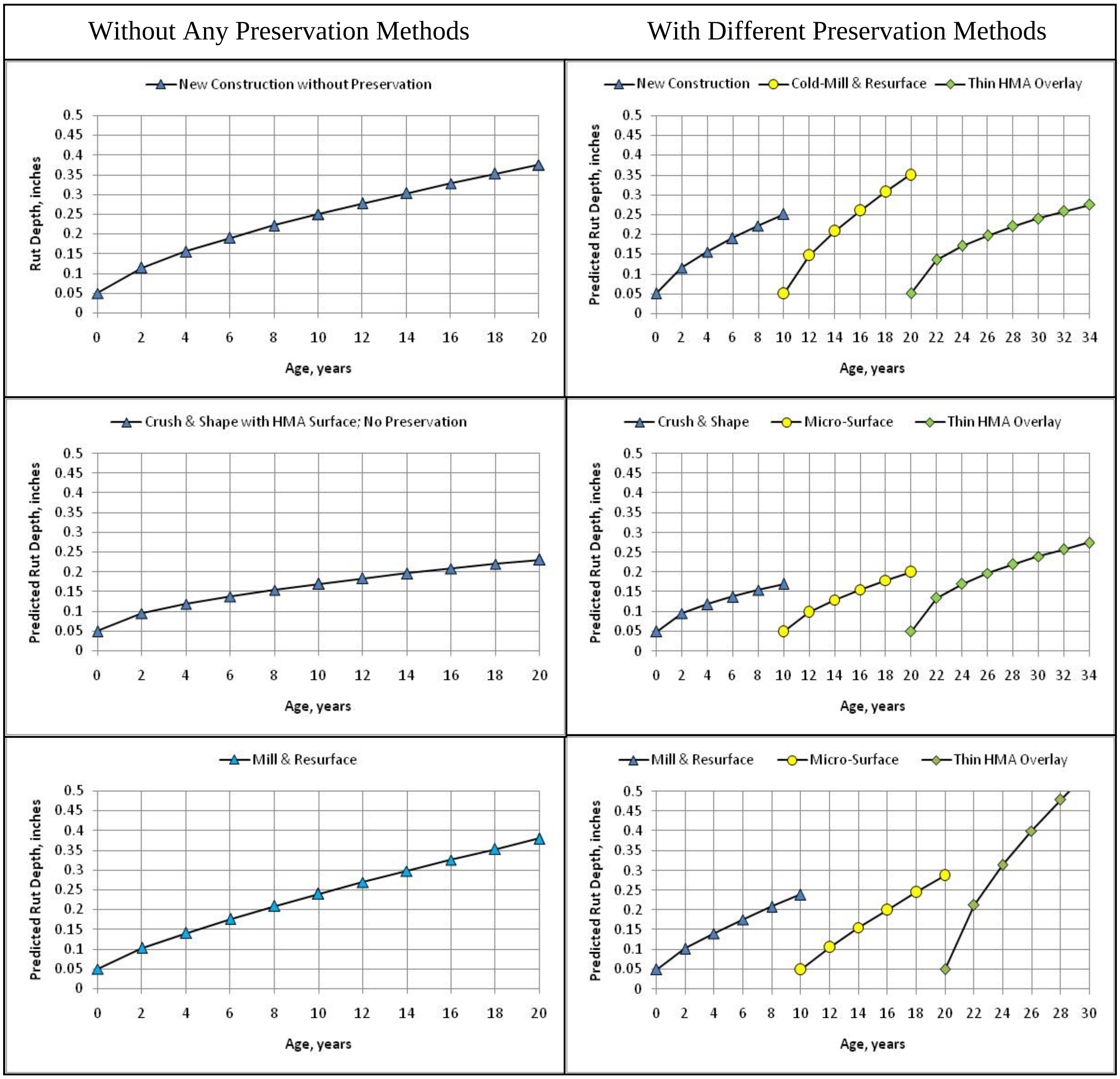


Figure 36. Predicted Service Life Based on Rut Depths from Equation 2 with and without Preservation Methods

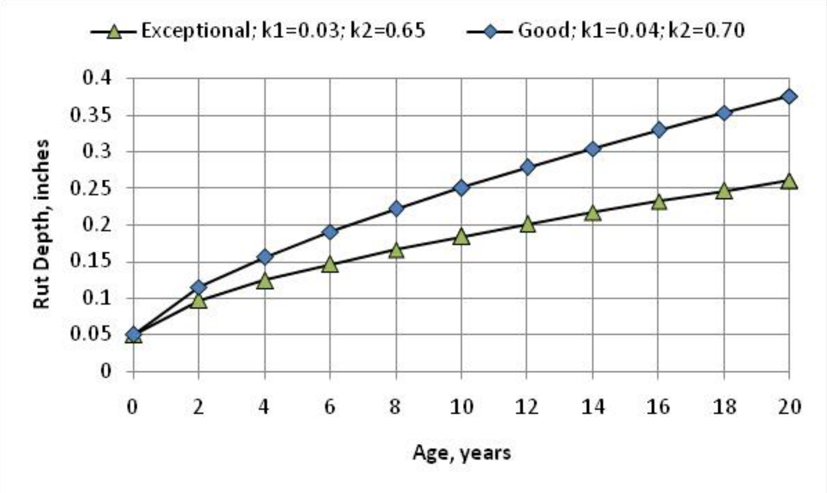
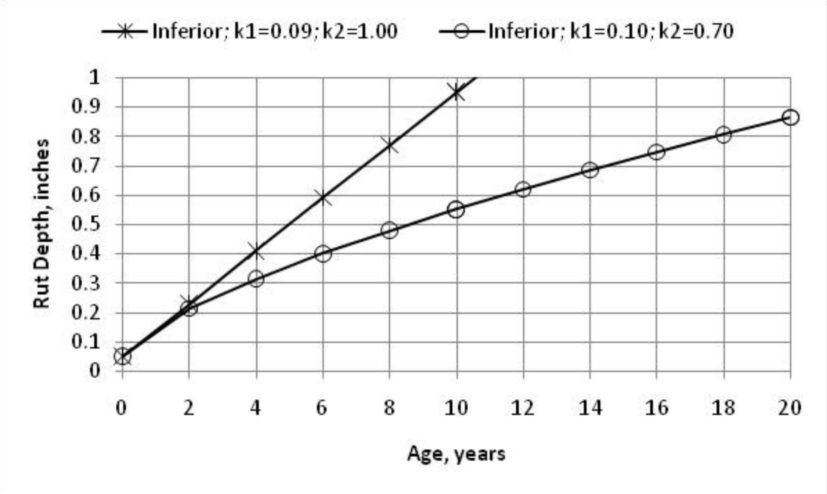
	Pavement structures and HMA mixtures with exceptional or good rutting resistance.
	Pavement structures and HMA mixtures with inferior or poor rutting resistance.

Figure 37. Examples of Good and Poor Performance Based on Extreme Values for the Rut Depth Deterioration Coefficients

4.3 Smoothness or Roughness

The average and range of IRI values for different data sets are listed in Table 12 for when the preservation method was placed on the pavement surface, while Figure 38 shows the cumulative frequency of those IRI values. On the average, the IRI values along the non-freeway highways are about 20 percent higher than for the freeway highways. In addition, there is a difference in the IRI values between the pavement groups when preventive maintenance is applied to the pavement; the crush and shape with HMA surface pavements are smoother.

As discussed in Chapter 2 of this report (*Performance Indicators*), no correspondence or consistent trend was found between the DI, rut depth, and IRI values (refer to Figures 4 through 7). Thus, a more simplistic empirical relationship was used to estimate IRI over time and is shown as equation 3. This relationship is similar to the empirical function that was developed and used by Perera, et al. in analyzing the test sections included in the LTPP program (Perera, et al., 1998).

$$IRI = IRI_0 (e)^{g_1 \left(\frac{t}{20}\right)^{g_2}} \tag{3}$$

Where:

IRI_0 = Initial IRI value after construction. This parameter was unavailable for the PM segments, so it was estimated based on the values recorded in the MDOT database shortly after construction for the newer flexible pavements and HMA overlays. At present MDOT has a threshold of 75 in./mi. in their smoothness specification for new flexible pavements.

t = Time in years.

g_1, g_2 = Regression constants referred to in this report as deterioration coefficients derived using linear regression techniques to minimize the error between the predicted and measured IRI values for individual PM segments.

Table 12. Average and Range of IRI Values When Any Preservation Activity was Placed on Pavement Surface

Pavement Structure Category	Highway Category	IRI Values	
		Average	Range
New Construction	Freeway	70	50 to 84
	Non-Freeway	86	51 to 171
Crush & Shape with HMA Surface	Freeway	52	44 to 67
	Non-Freeway	64	44 to 102
Resurfacing with and without Milling	Freeway	87	45 to 110
	Non-Freeway	99	55 to 220

The average IRI deterioration coefficients (refer to equation 3) were determined through linear regression for each PM segment. For this analysis, the underlying assumption is that all pavements were designed and constructed in accordance with the same procedures. Figure 39 compares the measured and predicted IRI values and suggests that equation 3 is a reasonable simulation of the increase in IRI over time. Figure 40 shows a comparison of the measured and predicted IRI values for selected PM segments. The IRI values reported along these segments illustrate the large change in IRI, as well as the decrease in IRI over time (the segment becoming smoother, rather than rougher). Both of these conditions can account for the higher variability in comparing the measured and predicted IRI values.

Figure 41 compares the IRI deterioration coefficients that are segregated between freeway and non-freeway categories. The non-freeway segments consistently have a lower g_2 coefficient and higher g_1 coefficient as compared to the freeway segments. This implies that the freeway segments have been consistently constructed to a higher standard – a lower loss of smoothness over time. In addition, the IRI deterioration coefficients are interrelated; the g_2 coefficient is inversely related to the g_1 coefficient. This observation or finding for the IRI deterioration coefficients is different than found for the DI and rut depth deterioration coefficients – the DI and rut depth deterioration coefficients (equations 1 and 2) are independent of one another. Thus, to establish the IRI deterioration coefficients for good and poor performance, g_2 will be dependent on g_1.

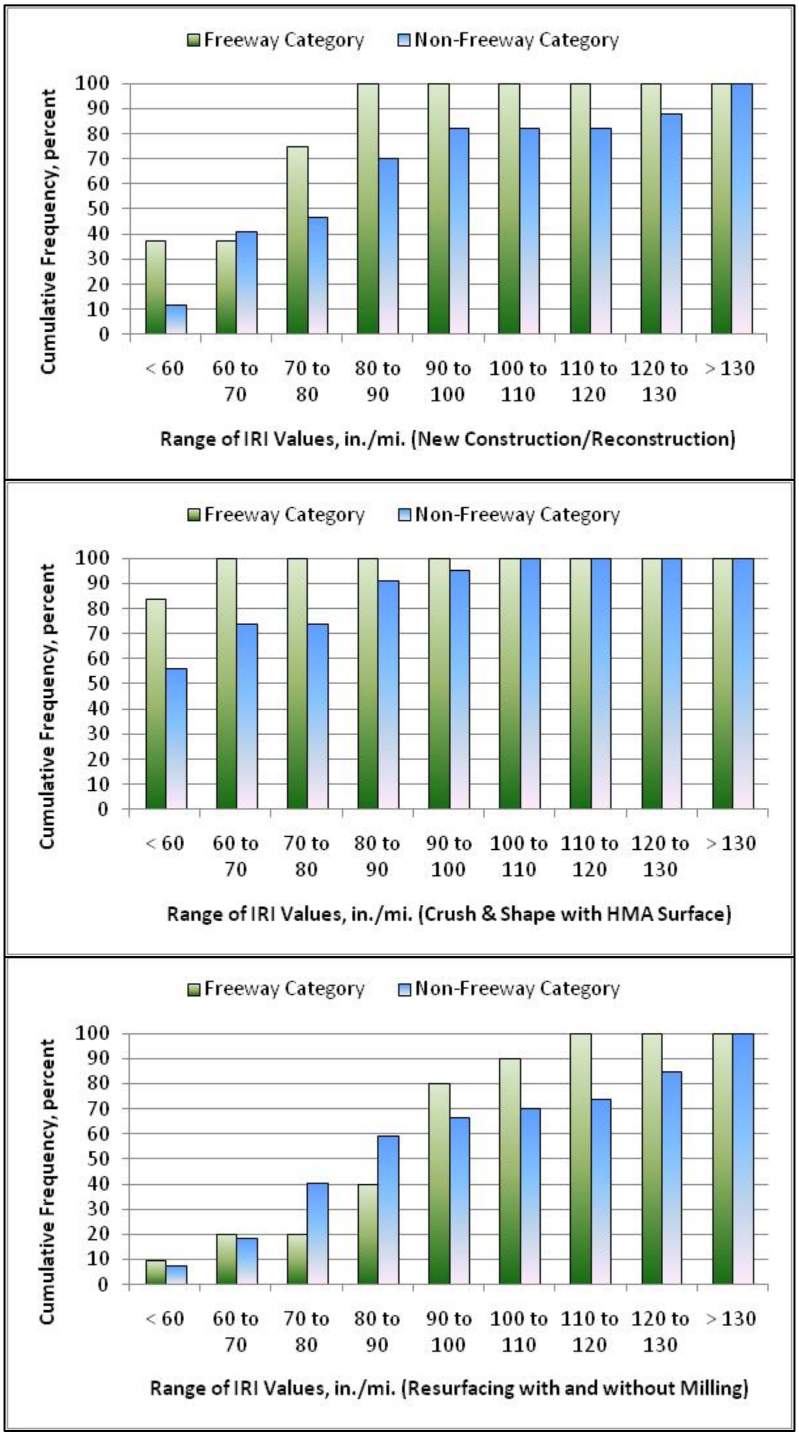


Figure 38. Cumulative Frequency of the IRI Values When Preservation Method Was Placed on Pavement Surface

Freeways, Divided Highways	Non-Freeways

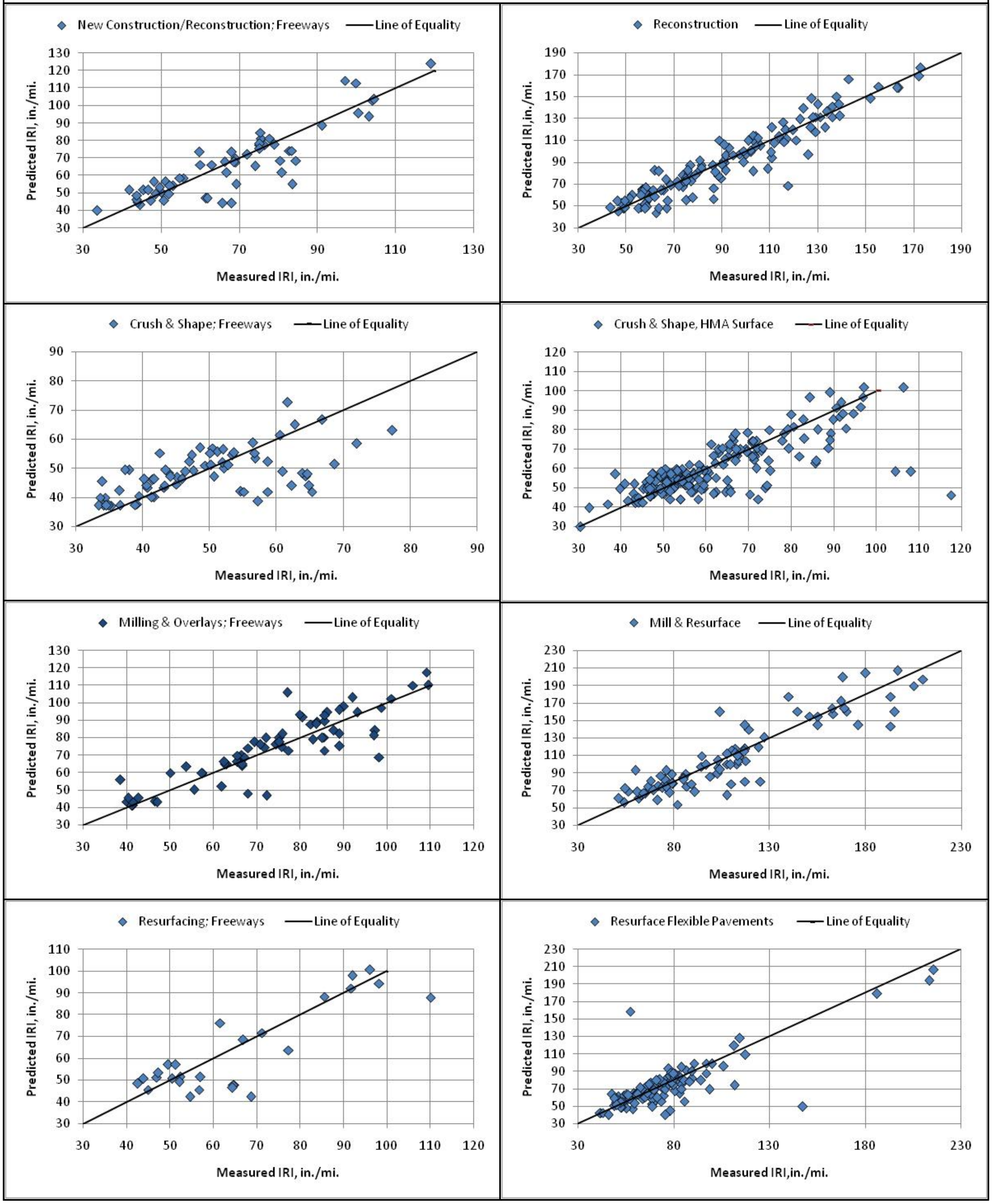
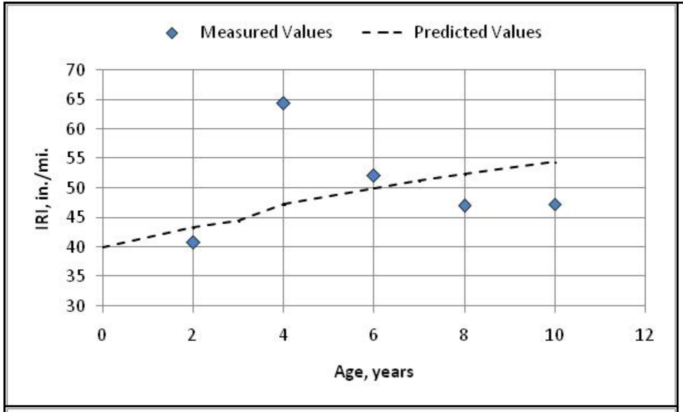
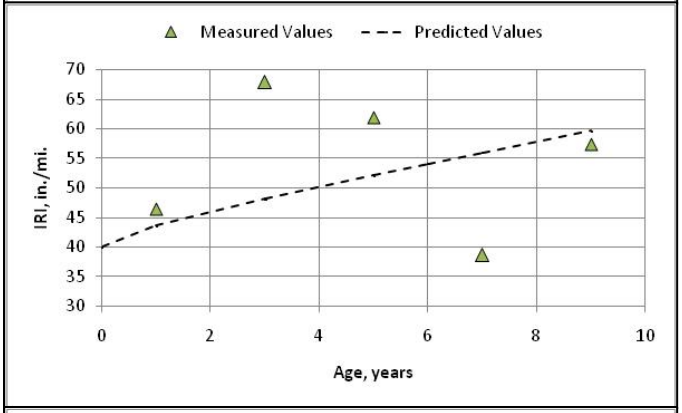


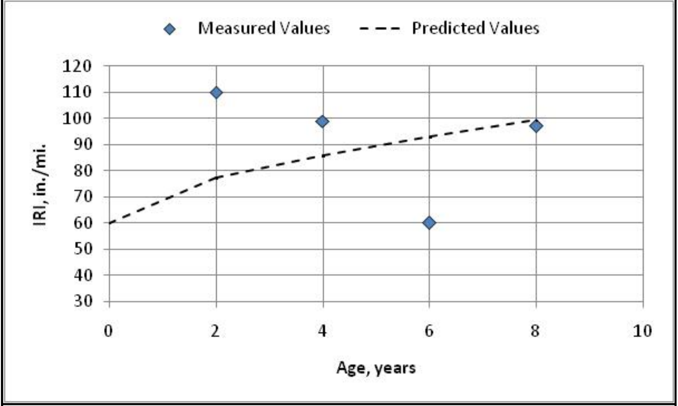
Figure 39. Comparison of Measured and Predicted IRI Values for the PM Segments



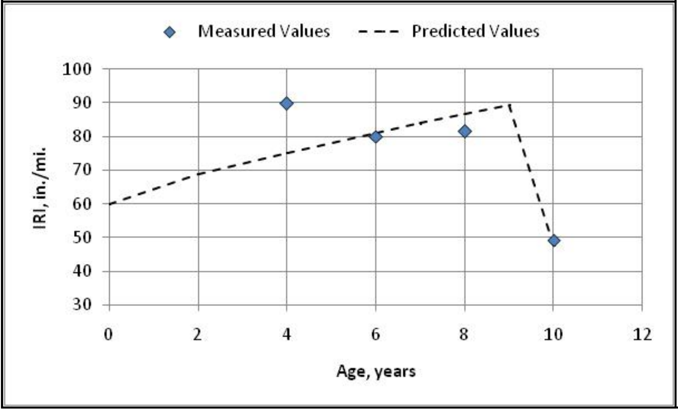
Crush & Shape with HMA Surface;
Freeway-Divided Highway
PM Segment, ID #5N; I-75 NB
North of Afton Road to M-68
BMP – 6.653; EMP – 15.092
CS – 16093
JN – 32510
Thin HMA overlay placed in 2000 (year 3).



Mill & Fill; Freeway-Divided Highway
PM Segment, ID #29N; US-31 NB
North of M-46 to South of Russell Road
BMP – 0.440; EMP – 6.019
CS – 61075
JN – 44771
No preservation/maintenance activity.



Mill & Resurface Flexible Pavement;
Non-Freeway Highway
PM Segment, ID #67; M-239
South of Wilson Road to South of Holiday Drive
BMP – 0.635; EMP – 1.340
CS – 11019
JN – 45461
No preservation/maintenance activity.



New Construction or Reconstruction;
Non-Freeway
PM Segment, ID #115; US-131
North of Springvale Road to North of County Club Road
BMP – 11.251; EMP – 12.675
CS – 15091
JN – 32322
Cold-Mill & Resurface in 2006 (year 9).

Figure 40. Comparison of Measured and Predicted IRI Values for Selected PM Segments

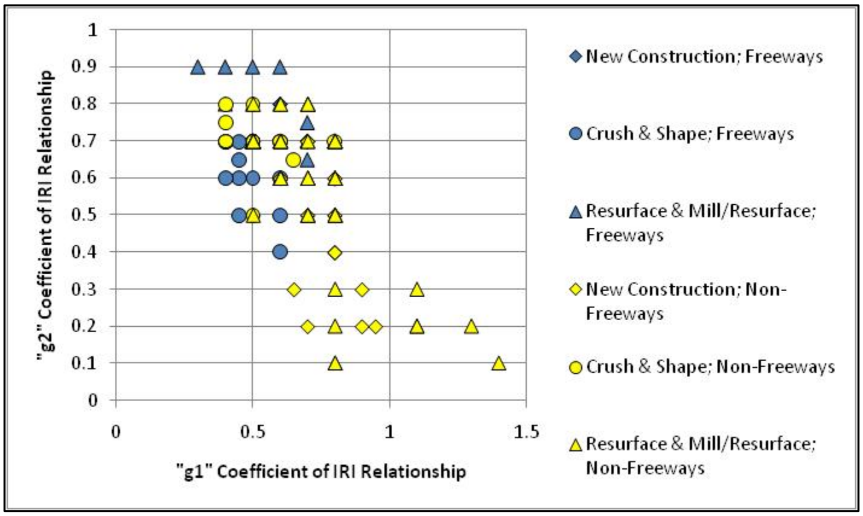


Figure 41. Comparison of IRI Deterioration Coefficients for Freeways and Non-Freeways

Tables 13 and 14 summarize the IRI deterioration coefficients for the different data sets, while Figure 42 is a comparison of the two deterioration coefficients for the individual PM segments that are grouped by pavement structure and preservation method. As shown, the IRI deterioration coefficients (refer to equation 3) were found to be similar for most of the PM segments, with and without pavement preservation. The median value for the g_2 coefficient was found to be 0.70 for many of the data sets. In addition, the g_1 coefficient is related to the g_2 coefficient for many of the data sets (as the g_1 value increases, the g_2 value decreases [refer to Figures 41 and 42]). This finding or observation makes it easier to establish the IRI deterioration coefficients for defining good and poor performance.

Table 13. Median IRI Deterioration Coefficients for PM Roadway Segments Without any Preservation Method Placed on Pavement Surface

Structure Category	Preservation Treatment	IRI Deterioration Coefficients	
		g_1	g_2
New Construction/Reconstruction	None	0.80	0.70
Crush & Shape with HMA Surface	None	0.50	0.70
Mill and Resurface with HMA	None	0.70	0.70
Resurface with HMA (No Milling)	None	0.60	0.70

Table 14. Median IRI Deterioration Coefficients for PM Roadway Segments With Different Preservation Methods Placed on Pavement Surface

Structure Category	Preservation Treatment	IRI Deterioration Coefficients	
		g_1	g_2
New Construction/Reconstruction	Cold-Mill & Resurface	0.80	0.60
	HMA Overlay	---	---
	Micro-Surface	---	---
	Chip Seal	---	---
Crush & Shape with HMA Surface	Cold-Mill & Resurface	0.50	0.70
	HMA Overlay	0.50	0.70
	Micro-Surface	0.50	0.70
	Chip Seal	0.50	0.70
Mill and Resurface with HMA	Cold-Mill & Resurface	0.80	0.70
	HMA Overlay	---	---
	Micro-Surface	0.60	0.70
	Chip Seal	---	---
Resurface with HMA (No Milling)	Cold-Mill & Resurface	0.60	0.70
	HMA Overlay	0.65	0.65
	Micro-Surface	0.65	0.65
	Chip Seal	---	---
The cells without a numerical value had an insufficient number of PM roadway segments and/or insufficient IRI values to determine the deterioration coefficients for that category.			

The only parameter that was found to consistently segregate the IRI data was highway type (freeway versus non-freeway PM segments). Table 15 summarizes the median IRI deterioration coefficient for the different pavement structural categories between the freeways and non-freeway data groups. As shown, the g_2 value for the non-freeways is slightly lower than for the freeway data set. There are PM segments, however, that exhibit significantly rougher pavements. The following quantifies good and poor performance in terms of the IRI deterioration coefficients, which were based on the range of deterioration coefficients determined for the PM segments that exhibited lower and higher IRI values (refer to Table 12 for the range of measured values).

Performance Category, IRI	Range of IRI Deterioration Coefficients	
	g_1	g_2
Good Performance	< 0.50	< 0.50
Poor Performance	> 0.90	> 0.80

The deterioration coefficients (equation 3) were evaluated and compared between different regions, highway type, pavement structure, pavement preservation strategy, and soil type. Other than highway type, no significant or consistent difference between the deterioration coefficients was identified or found for the different data sets. The reason(s) for the consistently smoother PM segments in the freeway group is probably related to construction and paving techniques. The following provides a general definition for delayed and accelerated roughness.

- Delayed Roughness or Good Performance is defined as new pavement and rehabilitation projects that have an average IRI values less than 80 in./mi. for 10+ years, or an average IRI less than 120 in./mi. for 30+ years. As noted above, MDOT has a threshold values of 75 in./mi. in their smoothness specification for new flexible pavements. Most of the IRI values recorded in the MDOT performance database over time for the new construction and crush and shape categories are less than that initial threshold value (refer to Figure 38). The resurfacing with and without milling pavement category has a higher percentage of IRI values greater than 75 in./mi. over time.
- Accelerated Roughness or Poor Performance is defined as new pavement and rehabilitation projects that have an average IRI greater than 120 in./mi. in less than 10 years.

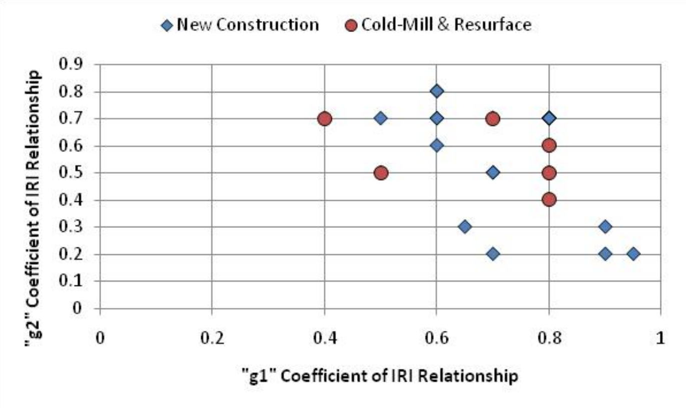
Table 15. Median IRI Deterioration Coefficients Between the Freeway and Non-Freeway Data Sets

Pavement Structure Category	Freeway Data Set		Non-Freeway Data Set	
	g_1	g_2	g_1	g_2
New Construction	0.8	0.7	0.7	0.5
Crush & Shape with HMA Surface	0.5	0.6	0.5	0.7
Mill & Resurface	0.6	0.7	0.8	0.5
Resurface	0.5	0.7	0.7	0.6

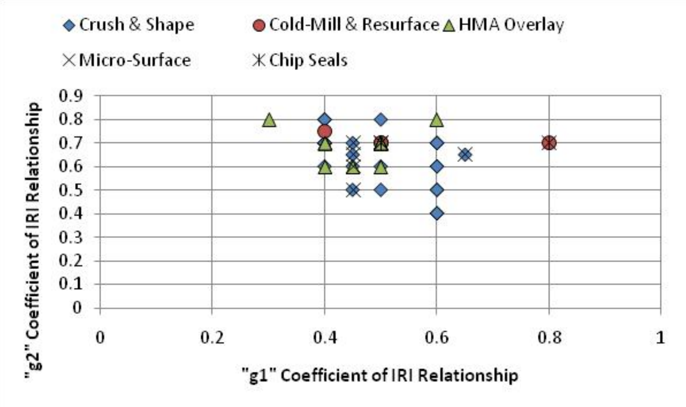
4.4 Summary of Deterioration Relationships

Tables 16 to 18 summarize the statistical data and information for each performance indicator. As shown, the DI and IRI values have better correlations, while the rut depth relationship is considered very poor. The major reason why the rut depth regression equation is a poor simulation of rutting is that the measured values decrease with time on many of the roadway segments. In addition, the rut depths measured after the first couple of years remain relatively the same for many other roadway segments. All existing rut depth transfer functions or regression equations predict increasing rut depth, but at a decreasing rate. Thus, none of the other rut depth relationships reported in the literature would accurately simulate the measured values.

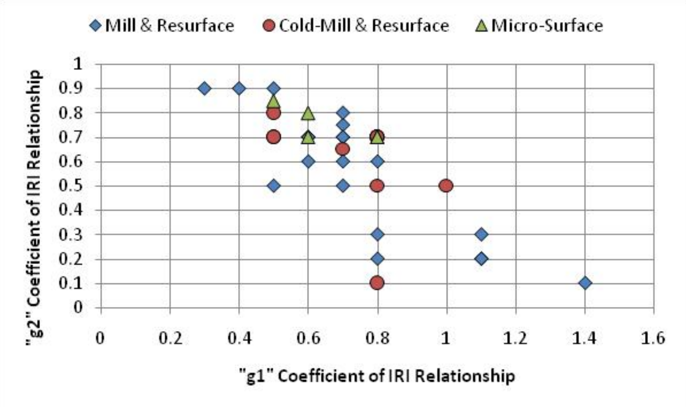
The other observation from this analysis is that the crush and shape pavement category had the poorer correlation between the measured and predicted DI and IRI values. It is unclear why this group of pavements consistently has the poorer correlation than for the other pavement structural groups or categories. However, preventive maintenance was applied to the crush and shape with HMA surface pavements that have lower performance indicators than for the other pavement groups (refer to Tables 6, 9, and 12). In other words, these pavements are in a better condition when preventive maintenance is applied to the surface. Another potential reason could be the amount of variability in the base layer, which is not recorded in the performance database.



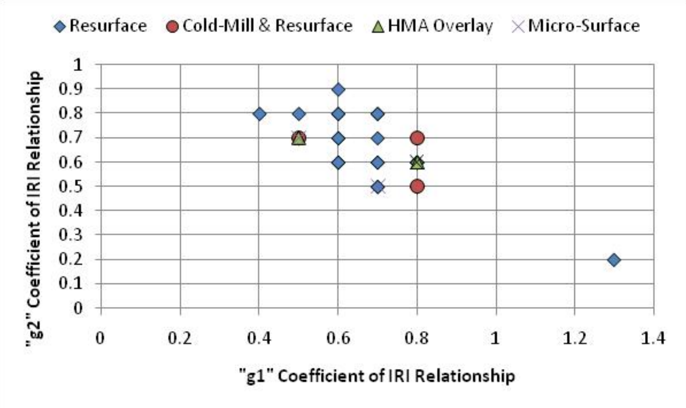
Preservation Methods Placed on Pavement Surface of Pavement Structural Category New Construction-Reconstruction



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Crush and Shape with HMA Surface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Mill and Resurface



Preservation Methods Placed on Pavement Surface of Pavement Structural Category Resurface Flexible Pavement

Figure 42. Comparison of IRI Deterioration Coefficients for Different Preservation Methods

Overall, the regression equations selected for defining the roadway segments with good and poor performance are considered reasonable based on the statistical values summarized in Tables 16 and 17. The DI and IRI relationships and deterioration coefficients were primarily used to define poor and good performance. The rut depth relationship was also used, but few of the roadway segments were found to have poor performance based solely on using a rut depth threshold value typically used by other agencies (Rauhut, et al., 1999). In fact, rut depth has all but been eliminated as a cause for rehabilitating asphalt pavements and overlays. Table 19 quantifies and summarizes the deterioration coefficients that define good and poor for each performance indicator based on an analysis of the data included in MDOT's database, while Table 20 provides a summary of the definitions for delayed and accelerated distress.

Observation: The DI and IRI deterioration relationships used to identify good and poor performing pavements are considered a reasonable simulation of the measured values, and can be used to predict these parameters on an individual PM segment basis.

The shorter service life and/or higher value of the performance indicators (refer to Table 20) estimated from the deterioration coefficients was used to categorize the performance of all segments included in MDOT's performance database with sufficient data into those exhibiting good and poor (premature distress) performance. As presented in Chapter 3, the MDOT roadway segments were grouped by region (climate), pavement structure, roadway type, soil type, and traffic volume. The hypothesis that the means of the two groups was indifferent was accepted. In other words, no significant or consistent difference was identified between the two groups for any of the parameters included in the database.

In summary, results from the analysis completed on the service life determined from the peak performance indicator (refer to Chapter 3) and from the estimated age at which a threshold value of the performance indicator is exceeded did not identify a consistent parameter (design feature or site condition factor) that would explain the difference between good and poor performance. This finding suggests that the cause of poor or good performance is not directly recorded in MDOT's performance database. Other studies have concluded that construction activities and HMA mixture properties are the more important factors. As such, a detailed analysis of the DI data was completed to identify specific distresses that are common to the asphalt pavements with poor performance – this analysis is presented in Chapter 5.

Table 16. Statistical Values from the Comparison of the Predicted and Measured Distress Indices (refer to Figure 27)

Pavement Structural Group	Type Roadway	R^2 Term	Standard Error	Slope of Relationship	Relative Error
New/Reconstructed Flexible Pavements	Freeways	0.862	4.226	0.9427	0.2792
	Non-Freeways	0.854	5.626	0.905	0.6124
Crush & Shape with HMA Surface	Freeways	0.655	2.285	0.821	0.7115
	Non-Freeways	0.579	9.052	0.775	1.6245
Mill and Resurface	Freeways	0.796	1.835	0.831	0.3553
	Non-Freeways	0.866	5.613	0.908	0.5451
Resurface	Freeways	0.790	1.515	0.857	0.3802
	Non-Freeways	0.711	8.446	0.818	1.1889

Table 17. Statistical Values from the Comparison of the Predicted and Measured IRI Values (refer to Figure 39)

Pavement Structural Group	Type Roadway	R^2 Term	Standard Error	Slope of Relationship	Relative Error
New/Reconstructed Flexible Pavements	Freeways	0.786	9.135	0.911	0.1343
	Non-Freeways	0.775	15.531	0.840	0.1696
Crush & Shape with HMA Surface	Freeways	0.360	6.314	0.439	0.1282
	Non-Freeways	0.070	13.026	0.101	0.1978
Mill and Resurface	Freeways	0.777	---	0.923	---
	Non-Freeways	0.809	18.025	0.850	0.1617
Resurface	Freeways	0.722	10.232	0.853	0.1567
	Non-Freeways	0.657	16.607	0.789	0.2176

Table 18. Statistical Values from the Comparison of the Predicted and Measured Rut Depths (refer to Figure 33)

Pavement Structural Group	Type Roadway	R^2 Term	Standard Error	Slope of Relationship	Relative Error
New/Reconstructed Flexible Pavements	Freeways	0.149	0.0433	0.250	0.3215
	Non-Freeways	0.261	0.0545	0.358	0.3326
Crush & Shape with HMA Surface	Freeways	0.124	0.0476	0.206	0.4003
	Non-Freeways	0.570	0.0431	0.610	0.3388
Mill and Resurface	Freeways	0.305	0.043	0.343	0.3360
	Non-Freeways	0.211	0.0600	0.355	0.4184
Resurface	Freeways	0.045	0.045	0.150	0.3338
	Non-Freeways	0.192	0.057	0.325	0.3905

Table 19. Summary of the Deterioration Coefficients that Define Good and Poor Performance Based on the Data Included in MDOT Database

Performance Indicator	Deterioration Coefficients	Good Performance	Poor Performance
Distress Index (equation 1)	a	> -0.2	< -2.8
	b	< 1.5	> 2.7
Rut Depth (equation 2)	k_1	< 0.05	> 0.08
	k_2	< 0.60	> 0.70
IRI (equation 3)	g_1	< 0.50	> 0.90
	g_2	> 0.50	> 0.80

Table 20. Summary of the Deterioration Coefficients that Define Good and Poor Performance Based on the Data Included in MDOT Database

Performance Indicator	Delayed Distress or Good Performance		Accelerated Distress or Poor Performance	
	Age, yrs.	Value	Age, yrs.	Value
Distress Index	10	< 15	10	> 25
	30	< 50	15	> 50
Rut Depth, in.	10	< 0.25	10	> 0.40
	30	< 0.40	---	---
IRI, in./mi.	10	< 80	10	> 120
	30	< 120	30	> 180

CHAPTER 5 ANALYSIS OF DETAILED DISTRESS DATA

The detailed distress data included in MDOT’s performance database were used to determine the magnitude and severity of the individual distresses to identify construction and/or material parameters that could explain why some segments exhibited premature distress, while others exhibited a significant delay in the distress. The purpose of this chapter is to present the results from an analysis of the data used to calculate DI and logic used to identify construction related parameters, as to their impact on the pavements exhibiting poor performance.

5.1 Distresses Contributing to the Distress Index Value

The amount of detail in the distress data is good and MDOT should be commended for taking an aggressive approach in collecting this data to manage their roadway system. This detail in the distress data can be used to determine if the increase in DI values are related to construction defects, HMA mixture properties, and/or site features.

Many of the distresses collected and recorded in MDOT’s database can have a significant impact on a particular project, but have been reported on a limited number of roadway segments or projects. Other distresses occur more frequently on MDOT’s roadway network. The more frequently occurring distresses are the important ones for identifying mitigation strategies that will have the greater impact across Michigan to enhance pavement performance and extend service life. Common distress types and magnitudes were determined for both groups.

5.1.1 Distresses Recorded for Pavements Exhibiting Poor Performance

Detailed distress data were extracted for the roadway segments identified as exhibiting poor performance (refer to Appendix A) based on the deterioration coefficients determined from Chapter 4. Appendix B includes the distress magnitudes that were recorded on some of the roadway segments included in the poor performance group. Selected PM segments were randomly selected from this group for taking a detailed look at the type and magnitude of surface distresses recorded in the database.

Table 21 lists the frequency of occurrence of distresses recorded for these projects. Longitudinal cracking and transverse defects have occurred on all of the roadway segments with poor performance. Alligator cracking was recorded on well over 50 percent of the projects with poor performance, while block cracking was recorded on over 50 percent of the projects. Few projects with poor performance had large amounts of both alligator and block cracking – it was either one or the other. These cracking distresses account for the majority of the distress index value for the roadway segments exhibiting poor performance. Other distress types were also found to be excessive, but for specific pavement structural groups.

In summary, longitudinal cracks, transverse cracks and tears have occurred on 100 percent of the projects, alligator or block cracking have occurred on well over 50 percent of the projects, and patches or surface treatments have been placed on over 25 percent of the projects. Shattered area, raveling, and flushing were found to be less frequent on projects with poor performance.

Figures 43 through 45 compare the average magnitude of the values recorded in the database for those frequently occurring distresses on pavements with poor performance. As shown, most of the cracking distresses were recorded for more than 10 percent of the length of the project and the number of occurrences of transverse cracks and tears exceed 100 per mile in a short period of time, relative to the design life.

Table 21. Frequency of Occurrence of Distresses for Roadway Segments with Poor Performance

Frequency of Occurrence for Roadway Segments with Poor Performance, %				
Distress Type	Pavement Structural Group			
	Reconstruction & New Construction	Crush and Shape	Mill and Resurface	Resurface
Transverse Cracking; Straight & Irregular	100	100	100	100
Transverse Tears	100	100	100	100
Longitudinal Centerline Cracking	100	100	100	100
Longitudinal Center Lane Cracking	96	100	100	100
Longitudinal Edge Cracking	100	100	100	100
Longitudinal Wheel Path Cracking	100	100	100	100
Alligator Cracking	48	67	50	100
Block Cracking	64	56	50	83
Patches or Surface Treatments	32	33	50	17
Flushing	4	11	0	0
Raveling	4	0	13	0
Shattered Areas	16	33	25	0

5.1.2 Distresses Recorded for Pavements Exhibiting Good Performance

Detailed distress data were also extracted for the roadway segments identified as exhibiting good to exceptional performance (refer to Appendix A) based on the deterioration coefficients determined from Chapter 4. Appendix B includes the distresses that were recorded on some of the roadway segments included in the good performance group. Selected PM segments were randomly selected from this group for taking a detailed look at the type and magnitude of surface distresses recorded in the database.

Table 22 lists the frequency of occurrence for the distresses recorded for these projects. Longitudinal centerline cracking and transverse defects have also occurred on all of the roadway segments with good performance. Figures 46 through 48 compare the average magnitudes of the values for pavements with good performance. The average values for the individual distresses recorded for segments with good performance are significantly less over a longer period of time than the segments with poor performance, with the exception of longitudinal centerline cracks recorded for the mill and resurface pavement category. The other important observation is that block and alligator cracking were recorded on 6 and 25 percent of the projects with good performance, respectively, while these distresses were recorded on well over 50 percent of the projects with poor performance. The percent lane length with block and alligator cracking is close to 0 for pavements with good performance and between 10 to 20 percent for pavements with poor performance.

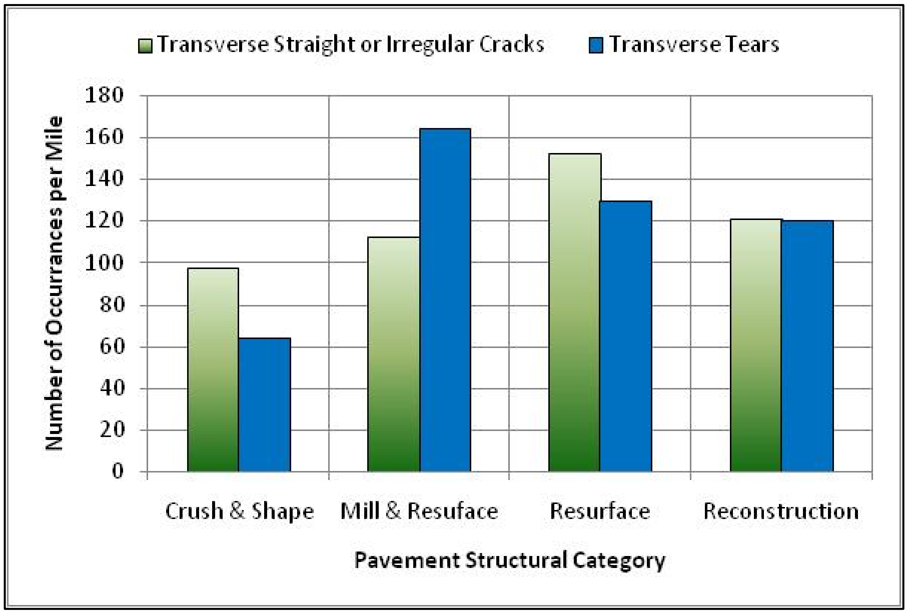


Figure 43. Overall Average Number of Occurrences of Transverse Cracks and Tears for Roadway Segments Exhibiting Poor Performance

5.2 Expected Cause of Common Distresses

5.2.1 Longitudinal Centerline Cracking

Longitudinal centerline cracking was recorded on 100 percent of the projects exhibiting poor and good performance (refer to Tables 21 and 22). Figure 49 is an example of excessive longitudinal cracking and deterioration along the centerline joint, which is directly related to the construction of the centerline joint. Figure 44 shows the amount of centerline cracking for projects with poor performance, while Figure 47 shows the amount of cracking for projects with good performance. The magnitude and severity of the centerline cracks are lower over an extended period of time for the asphalt pavements and overlays with good to exceptional performance with the exception of the mill and resurface pavement category.

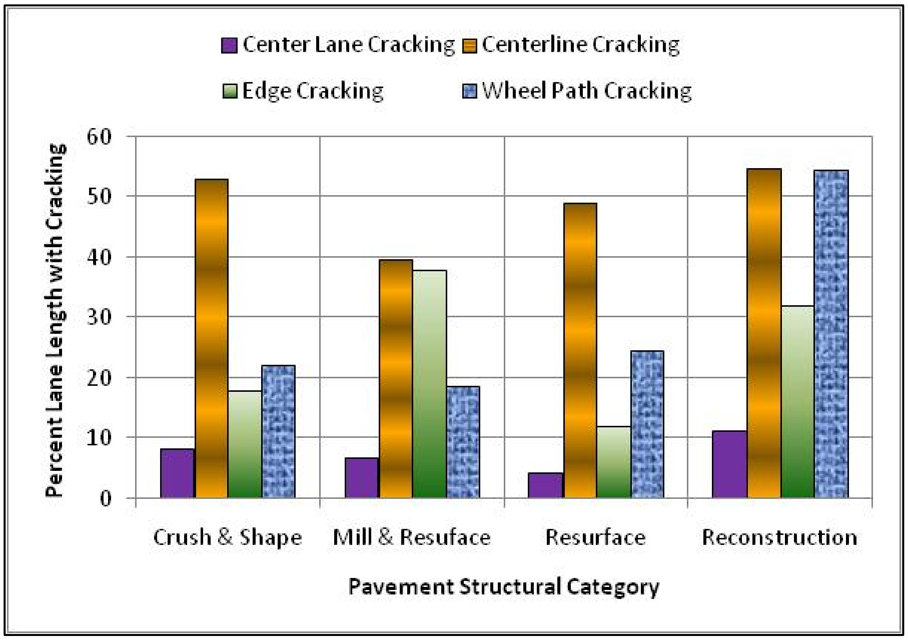


Figure 44. Overall Average Percentage of Roadway Length with Longitudinal Cracking for Segments Exhibiting Poor Performance

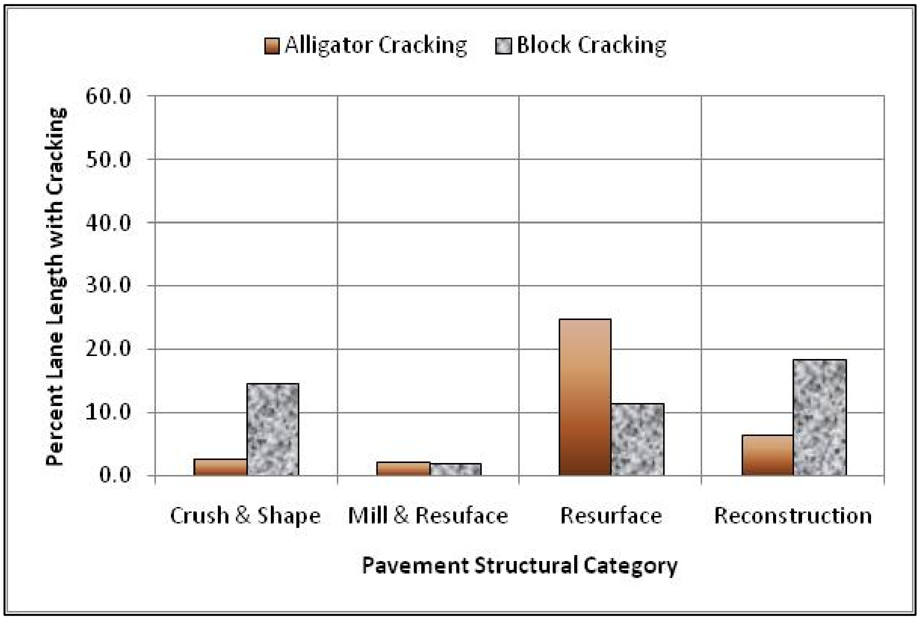


Figure 45. Overall Average Percentage of Roadway Length with Alligator and Block Cracking for Segments Exhibiting Poor Performance

Table 22. Frequency of Occurrence of Distresses for Roadway Segments with Good Performance

Distress Type		Frequency of Occurrence, %
Transverse Defects	Straight and Irregular Cracks	100
	Tears	100
Longitudinal	Centerline Cracking	100
	Center Lane Cracking	81
	Edge Cracking	87
	Wheel Path Cracking	75
Area	Alligator Cracking	25
Block Cracking		6
Patches or Surface Treatments		12
Flushing		0
Raveling		6
Shattered Areas		0

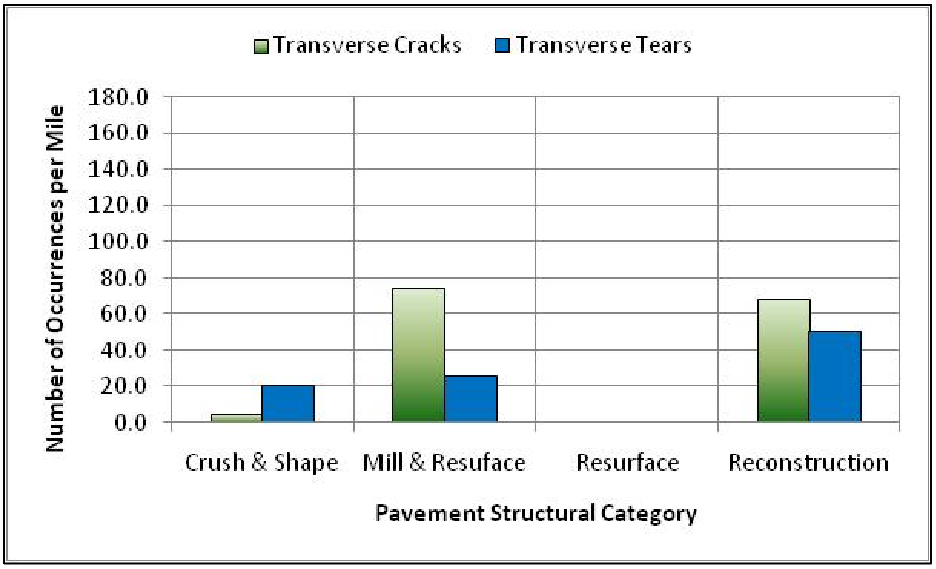


Figure 46. Overall Average Number of Occurrences of Transverse Cracks and Tears for Segments Exhibiting Good Performance

Whether longitudinal centerline cracks can be eliminated from all projects is questionable, but the magnitude and severity can be reduced over a longer period of time through the use of improved rolling patterns and increased HMA density along the joint. Based on the experience of other agencies, an effective method to reduce this cracking and its severity (lowering the DI value on many projects) is to implement a longitudinal construction joint specification.

5.2.2 Longitudinal Center Lane Cracking

Although center lane cracking has occurred on all projects with poor performance and over 80 percent of the projects with good performance, the overall average length is relatively low in comparison to the other forms of longitudinal cracking. Figure 50 shows an example of longitudinal center lane cracking. Figure 44 shows the amount of center lane cracking for projects with poor performance, while Figure 47 shows the amount of cracking for projects with good performance. The magnitude and severity of the center lane cracks are lower over an extended period of time for the asphalt pavements and overlays with good to exceptional performance.

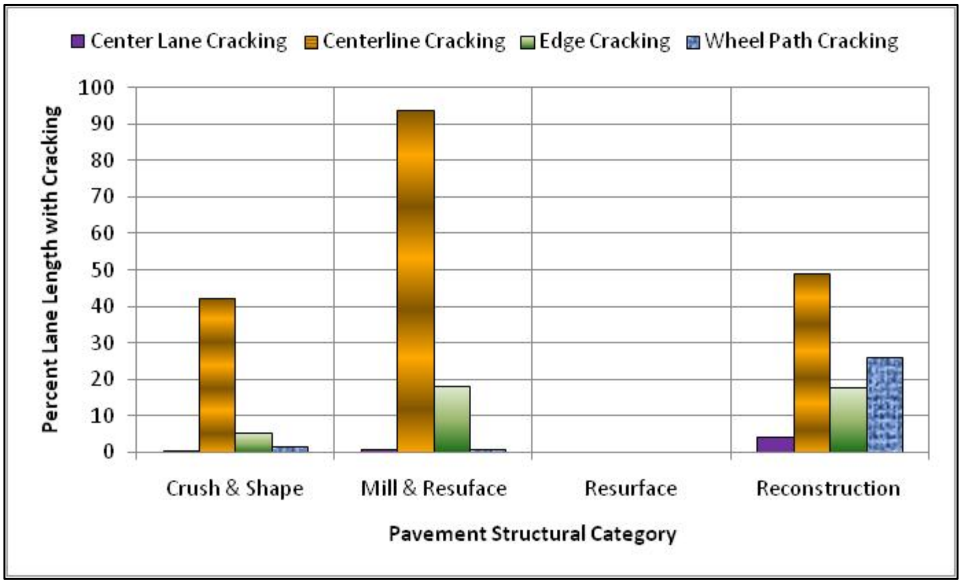


Figure 47. Overall Average Percentage of Roadway Length with Longitudinal Cracking for Segments Exhibiting Good Performance

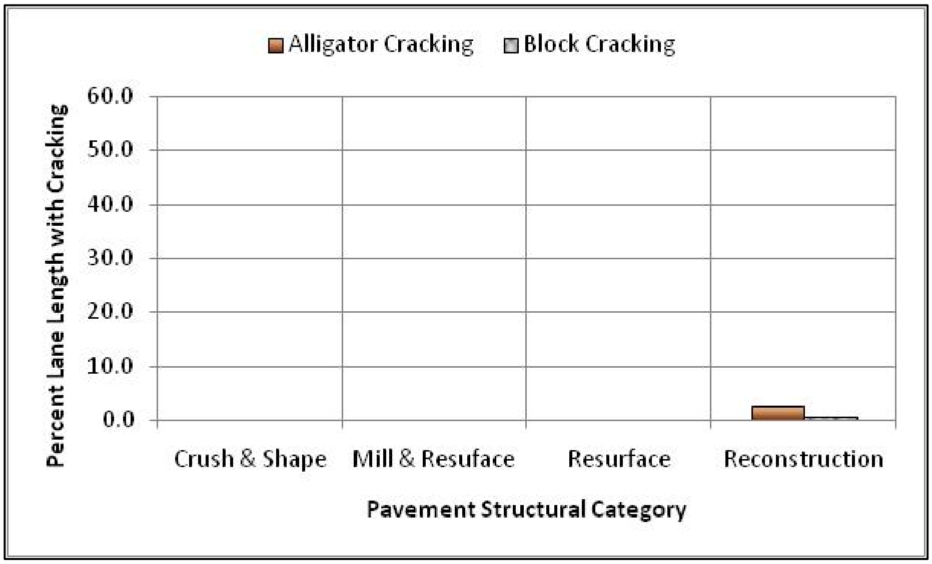


Figure 48. Overall Average Percentage of Roadway Length with Alligator and Block Cracking for Segments Exhibiting Good Performance

Figure 49. Cracking and Accelerated Deterioration Along Longitudinal Centerline Joints that were Inadequately Constructed

Figure 50. Cracking and Deterioration Along the Center of the Lane where the HMA was Improperly Placed

Center lane cracking has been reported to be a result from the center lane segregation (refer to Figure 50), inadequate material being pushed under the gear box of the paver, the flow gates being set too low, and/or the lead crown of the screed being too low relative to the tail lead crown. In summary, most causes of longitudinal center lane cracking are related to the paver and/or its operation. When center lane cracking is caused by center lane segregation or worn out kick back flights, this cracking is usually more predominant along the entire project. The average length of center lane cracking recorded for those projects with inferior performance are generally less than 10 percent of the lane length. Its occurrence along the project, however, is dependent on the contractor's ability to achieve adequate HMA density in the center of the lane regardless of the specific cause. To identify localized areas with inadequate density during construction requires the use of biased sampling and testing methods.

5.2.3 Longitudinal Edge Cracking

Longitudinal edge cracking has occurred on all projects with poor performance and on nearly 90 percent of projects with good performance (refer to Tables 21 and 22). Figure 44 shows the length of edge cracking for projects with poor performance, while Figure 47 shows the length of cracking for projects with good performance. The magnitude and severity of the edge cracks are lower over an extended period of time for the asphalt pavements and overlays with good to exceptional performance (refer to Figures 44 and 47). Based on the experience of the authors, some longitudinal cracking along the edge of the outside wheel path have been recorded as edge cracks. It is assumed, however, that is not the case for the projects included in the two groups of segments (poor and good performance).

Based on previous experience, longitudinal edge cracking is related to the frost susceptibility of the soils and other site features, and/or improper rolling of an unconfined edge in combination with deficient mixture properties. Soil type and mixture type, however, were not found to be factors that explain the higher lengths of edge cracking for segments with poor performance.

5.2.4 Longitudinal Wheel Path Cracking

Longitudinal wheel path cracking has occurred on all projects with poor performance and on about 75 percent of projects with good performance (refer to Tables 21 and 22). Figure 44 shows the length of longitudinal wheel path cracking for projects with poor performance (varying from 20 to over 50 percent of the project length), while Figure 47 shows the length of cracking for projects with good performance (varying from 0 to about 25 percent of the project length). The magnitude and severity of the longitudinal wheel path cracks are much lower over an extended period of time for the asphalt pavements and overlays with good to exceptional performance (refer to Figures 44 and 47).

Longitudinal wheel path cracking is a common distress type reported along many roadways. Most agencies combine longitudinal and area or alligator cracking in the wheel path area. It is believed that MDOT took the correct path in recording these cracks as separate distresses. Longitudinal wheel path cracking is believed to be initiated at the surface of the pavement and propagates downward when rutting is not present, while it has been reported to initiate at the bottom of the HMA when subgrade rutting occurs. The magnitude of rutting is very low for all of the pavements categorized as having poor performance, so it is expected that subgrade rutting is a nonissue.

Longitudinal cracking within and along the outside edges of the wheel path can be a result of a significant stiffness or modulus gradient through the HMA layers; stiff or high modulus, brittle wearing surface over a lower modulus layer. Lower amounts of longitudinal wheel path cracking were reported for the crush and shape, mill and resurface, and resurface categories. Greater lengths have been recorded for the new construction/reconstruction category. The design of coarse and/or gap-graded mixtures using high laboratory compaction efforts (N_{design} gyrations) can result in lower target asphalt contents and brittle mixtures that are susceptible to accelerated aging and cracking. To reduce the occurrence and length of longitudinal wheel path cracks requires the design and production of more strain tolerant or less brittle mixtures.

5.2.5 Transverse Cracks and Tears

Transverse defects (cracks and tears) have occurred on all of the projects with poor and good performance. Some transverse cracking, however, is expected in Michigan's climate. The difference is in the time it takes for the transverse cracks and tears to reach a specific magnitude. Figure 43 summarizes the overall average occurrences of transverse cracks and tears recorded for the segments with poor performance (varying from over 60 to more than 160 occurrences per mile of the project), while Figure 46 summarizes the average occurrence for segments with good performance (nearly 0 to over 70 occurrences per mile). As shown, the segments with good performance have less than half the number of occurrences over a much longer time period (refer to Figure 31 in Chapter 4).

Transverse cracking is heavily dependent on the climate, asphalt grade, and volumetric properties (Von Quintus, et al., 1998 and 1999). The segments with poor and good performance, however, are not restricted to a specific climate. Air void level, asphalt content, and gradation are the more important properties related to the occurrence of transverse cracks, but these mixture properties are not included in the MDOT database. Whether transverse cracks can be eliminated from all projects is questionable for Michigan's climate, but the magnitude can be reduced over a longer period of time through the use of different wearing surfaces and mixture design modifications.

5.2.6 Block Cracking

Block cracking has occurred on well over 50 percent of the projects with poor performance and on about 6 percent of projects with good performance (refer to Tables 21 and 22). Figure 45 shows the average length of block cracking for projects with poor performance (varying from nearly 5 to 20 percent of the project length), while Figure 48 shows the average length of block cracking for projects with good performance (varying from 0 to less than 1 percent of the project length). The magnitude and severity of the block cracks are much lower over an extended period of time for the asphalt pavements and overlays with good performance.

Block cracking is dependent on the volumetric properties of the mixture, especially air voids and effective asphalt content by volume. Other agencies have reported premature distress, in terms of non-load related cracking, for projects where the HMA mixture was designed using a high number of N_{design} gyrations, originally recommended for use in the Superpave mixture design procedure (Colorado DOT). Some agencies have reduced the number of gyrations because of premature cracking and deterioration.

5.2.7 Alligator Cracking

Alligator cracking has occurred on over 60 percent of the projects with poor performance and on about 25 percent of projects with good performance (refer to Tables 21 and 22). Figure 45 shows the average length of alligator cracking for projects with poor performance (varying from about 2 to nearly 25 percent of the project length), while Figure 48 shows the average length of alligator cracking for projects with good performance (varying from 0 to less than 2 percent of the project length). The magnitude and severity of the alligator cracks are much lower over an extended period of time for the asphalt pavements and overlays with good performance.

Alligator cracks are dependent on traffic level, mixture properties, and structural features of the pavement. The greater amounts of alligator cracking consistently occur on roadway segments with poor performance in the resurface category (refer to Figure 45). The other pavement structural categories have relatively short lengths of alligator cracking, on the average. Alligator cracking has been observed when debonding occurs between the existing HMA surface and HMA overlay. Debonding has a lower probability of occurrence for milled surfaces and higher probability of occurrence on unmilled surfaces (Von Quintus, et al., 2000). The amount of alligator cracking is less frequent within the reconstruction/new construction and crush and shape pavement structural categories but was still recorded on many project with poor performance. As noted above for some of the other cracking distresses, designing mixtures that are more tolerant to tensile strain increases fatigue strength or the resistance to fracture (Von Quintus, et al., 1991).

5.3 Recommended Strategies to Reduce Occurrence of Premature Distress

Based on the review and analysis of the detailed distress data for the roadway segments exhibiting good and poor performance, the cause of premature distress or aging can be attributed to two factors: construction related causes and mixture related causes. The following lists those mitigation strategies that will have a significant impact on pavement performance; reducing premature distress and/or extending the service life of HMA pavements and overlays. The mitigation strategies are discussed in much more detail in the Implementation Plan, which was submitted as Part II of the research report. These mitigation strategies are listed in order of importance or impact on future performance (1 being the most important or having the greatest impact).

1. Implement a longitudinal construction joint specification. It is believed that this item will have the greatest benefit to MDOT. Most agencies that have implemented a longitudinal construction joint specification have reported longer service lives prior to rehabilitation and lower amounts of maintenance activities.

2. Revise the mixture design procedure and material requirements. This includes lowering the number of N-design gyrations for both high and low volume roadways to ensure adequate mixture strength and durability, and using fewer gap-graded mixtures that are not polymer modified. Another mixture related strategy is to use higher quality wearing surfaces for high volume roadways; like stone matrix asphalt (SMA) and polymer modified asphalt (PMA) mixtures. MDOT and/or the local contractors have historically used gap-graded HMA mixtures, which can result in mixtures with lower asphalt contents and higher permeability. The purpose of this strategy is to increase the effective asphalt content by volume in the mixture, improving on the durability of the mixture, and to use more PMA or SMA mixtures, especially for higher volume roadways.

3. Increased inspection and biased sampling and testing requirements at the beginning of a project to confirm adequate densities near the center and other locations of the paver. In addition, measuring the density under each roller pass to ensure that mixture checking does not become an issue along the project. Although checking is not recorded in the pavement performance database and can only be detected during construction, it has been observed by the authors on projects in Michigan and abroad where the roller was operated within the temperature sensitive zone of the mixture. The authors have noted this as being a significant issue during construction, especially for gap-graded HMA mixtures.

4. The other more long term mitigation strategy related to mixture design is to implement a fundamental test to be used during mixture design. This strategy is to include a fundamental test or torture test to confirm the HMA volumetric mixture design. Some state agencies use a laboratory loaded wheel tester (for example, the Hamburg or Asphalt Pavement Analyzer (APA) devices) to confirm the mixture design. As an example, the Texas and Colorado DOTs use the Hamburg device, while the Georgia and Mississippi DOTs use the APA device. These devices, however, only confirm the rutting resistance of the mixture and not the fracture resistance. Rutting was found not to be an issue in Michigan at this time, so a fracture test is recommended for use. This recommendation is provided in the implementation plan provided as Part II.

These mitigation strategies were based on the analysis of pavement performance data and the distresses and their magnitudes that have occurred on the roadway segments with poor and good performance. These mitigation strategies are included in the Implementation Plan (Mitigation Strategies and Pilot Projects; refer to Part II).

Table 23 summarizes the mitigation strategies recommended for enhancing flexible pavement performance. The first three are considered high priority mitigation strategies that can have a significant impact on improving flexible pavement performance without increasing construction costs.

5.3.1 Longitudinal Construction Joint Specification

Echelon paving is the best strategy to eliminate longitudinal construction joints, but echelon paving is impractical for routine paving of multi-lane roadways; especially for rehabilitation projects for which existing traffic flow must be maintained.

The amount and severity of centerline cracking can be reduced by improving on the construction and rolling of the centerline joint and joint between adjacent lanes in the same direction. Many agencies have already developed and implemented a longitudinal construction joint specification because of the joint's impact on pavement maintenance and performance. It is understood that MDOT drafted a longitudinal construction joint specification in 2009, but that specification has yet to be implemented or included in any pilot study.

Table 23. Mitigation Strategies to Reduce Premature Distress and Increase Pavement Service Life

Mitigation Strategy	Objective or Purpose	Importance	Important Feature	Impact on Construction Cost	Time for Implementation
Develop, Enforce Longitudinal Construction Specification	Reduce length & severity of centerline cracks & deterioration.	High; impact should be immediate.	None, immediate implementation	None.	2012 construction season
Reduce Gyrations to Estimate Target Asphalt Content & Job Mix Formula	Reduce length & severity of transverse cracks, longitudinal cracks in wheel path & along the edge.	High; impact will take a couple of years	Laboratory experiment is required for implementation	Minor increase in cost.	2012 for the lab experiment & initial pilot project; 2012 construction season for evaluating performance.
Biased Inspection & Testing of HMA	Reduce length & severity of longitudinal center lane cracks.	High; impact should be immediate	Purchase infrared cameras	None.	2012 construction season
Use Wearing Surface with Enhanced Properties; PMA & SMA	Reduce severity of transverse cracks & tears; longitudinal cracks in wheel path & alligator cracks	Moderate; impact will be immediate on higher volume roadways	None, immediate implementation	Increase in cost.	2012 construction season to implement; performance based tests need to confirm reduction in distress.
Use Fundamental Performance Test for Design	Reduces all distresses.	Moderate; impact will take time.	Long term strategy after others are completed	Increase in cost.	Future development & work.

Implementation of a longitudinal construction joint specification is considered a high importance mitigation strategy to MDOT and industry in terms of increasing pavement life and reducing life cycle costs of flexible pavements. This mitigation strategy can reduce the length and severity of longitudinal centerline cracks without increasing construction costs. Proper rolling patterns for compacting a confined and unconfined longitudinal construction joint are available in various HMA construction courses and documents (NHI Course #132032, Hot Mix Asphalt Construction [Seeds, et al., 2002]; various NAPA, Asphalt Institute, and FHWA courses). There are different opinions within industry, however, regarding the most effective rolling pattern to achieve higher densities along the centerline joint. The objective of this implementation strategy is two-fold:

1. Provide evidence to MDOT and contractors that compacting longitudinal construction joints and enforcing the specification will not result in significant penalties.
2. Provide data for confirming the values included in a Percent Within Limits (PWL) type of specification, as well as a contractors quality control plan.

5.3.2 Revise Mixture Design Criteria

Extensive lengths of transverse cracks, alligator cracks, longitudinal edge and wheel path cracks, block cracking, and raveling were recorded on just about all roadway segments exhibiting poor performance. Conversely, segments with good performance exhibited significantly less amounts of transverse cracks and tears, and minor lengths of longitudinal wheel path cracks, alligator cracks, block cracking, and raveling.

The roadway segments with excessive cracking were not restricted to colder climates or MDOT regions, soil type/strength, or traffic level so it was concluded that these cracks are more of a materials issue rather than a climate, traffic, or structural issue. Excessive alligator cracks, longitudinal cracks in the wheel path and along the edge, and transverse cracks are characteristic of high stiffness, low strength HMA mixtures relative to the supporting layers. Higher laboratory compactive efforts (higher N_{design} values) will result in lower effective asphalt contents by volume. Reducing the number of gyrations during mixture design will increase the effective asphalt content by volume, which has an effect on mixture durability and its resistance to cracking, especially for lower volume roadways that are thinner or pavements built over weak soils – both of which have higher deflections.

The hypothesis is that some HMA mixtures are susceptible to fracture because of lower asphalt contents. Lower asphalt contents can reduce the tensile strength of HMA and result in brittle mixtures. Higher laboratory compaction efforts can result in lower effective asphalt contents by volume. More importantly, MDOT and industry have designed and placed gap-graded, neat or unmodified HMA mixtures on numerous projects, especially for the wearing surface. Gap-graded and/or uniform-graded on the coarse side, unmodified HMA mixtures can exhibit higher permeability because of higher portions of larger (coarser) aggregate in the aggregate blend. Low asphalt content mixtures with high permeability are more susceptible to accelerated aging and moisture infiltration, which increases surface deterioration and reduces the mixture's resistance to cracking. Revising the mixture design guidelines and laboratory compaction criteria should improve on the mixture's resistance to cracking for both low and high volume roadways (Von Quintus, et al., 1998 and 1991). The objective of this implementation strategy and pilot project is to:

- Reduce the number of gyrations for mixture design, and revise the HMA mixture design criteria and aggregate blends for both higher and lower volume roadways to increase mixture strength and durability; and make the mixture more tolerant to tensile strains.

Multiple agencies have already lowered the number of gyrations for selecting the target asphalt content and job mix formula. Some of these agencies observed that cracking and deterioration of wearing surfaces occurred on a higher percentage on HMA mixtures designed using high levels of N_{design} gyrations.

A pilot project is needed before making any revisions to the current HMA mixture design procedure. This pilot project, discussed in detail in the Implementation Plan, will provide data to determine the effect of lowering the number of gyrations on the volumetric properties that are used for acceptance and payment. The pilot project will also provide data to compare the fundamental properties between different aggregate blends (gap-graded versus coarse and fine-graded mixtures). Simply lowering the number of gyrations without checking the fundamental properties is not recommended because of the potential impact on rutting and other distresses.

More importantly, the aggregate blend or gradation can be altered to offset any increase in the target asphalt content through lowering the number of gyrations, especially for gap-graded and uniform-graded aggregate blends. Thus, implementation of this mitigation strategy should be completed in parallel with the adoption and use of a fundamental performance test for confirming the volumetric based mixture design (refer to mitigation strategy #5).

Implementation of revised mixture design criteria is considered a high importance strategy to MDOT and industry because it will reduce the number of premature failures and extend the service life of flexible pavements. The strategy may increase construction costs because of higher asphalt contents and potential effects on the aggregate blend or gradation. However, the increase is construction cost is considered minimal.

5.3.3 Biased Sampling and Testing to Identify Construction Defects

Nearly all projects with poor performance exhibited center lane longitudinal cracking. Longitudinal cracking in the center of the lane is not related to the HMA mixture itself or structural properties. These cracks are related to the paving equipment and construction practice, and a result of an inadequate amount of mixture being pushed under the paver gear or drive box; sometimes referred to as center lane segregation. This condition can be easily identified through visual observations and density tests conducted in a specific area – rather than at random locations.

Identifying specific areas with insufficient mixture or segregation and taking corrective action can totally eliminate these longitudinal center lane cracks. An effective method to reduce the occurrence of these longitudinal cracks is to conduct density tests and visual inspection at the center of the paver during the first couple of days of paving and then on an as needed basis, as directed by the project engineer (Von Quintus, et al., 1995 and 1999).

The infrared camera is a device that can be easily used to identify areas with construction defects that cause center lane longitudinal cracks and deterioration (Von Quintus, et al., 2009). As such, biased sampling and testing with the use of an infrared camera is recommended to identify factors causing center lane cracking during the first day of paving so corrective actions can be taken, if needed. Multiple agencies have purchased infrared cameras to assist in identifying and locating these types of construction defects, and some Michigan contractors have already purchased these cameras as part of their quality control programs. The objective of this implementation strategy is two-fold:

1. Prepare a set of guidelines that can be used by MDOT staff to locate problem areas at the beginning of paving so that corrective actions can be taken by the contractor.
2. Demonstrate use of infrared camera to identify construction defects near the center of the auger chamber and in other areas of the mat.

A few agencies (for example; Washington DOT) already use biased testing to identify areas with temperature differences (sometimes referred to as temperature segregation). An infrared camera or sensors can be used to identify areas with a significant loss of temperature during paving. Figures 51 and 52 are examples of cold spots that were identified with the infrared camera. Figure 53 is an example showing uniform surface temperatures across the paving lane. Implementation of this mitigation strategy does require the purchase and use of infrared cameras.

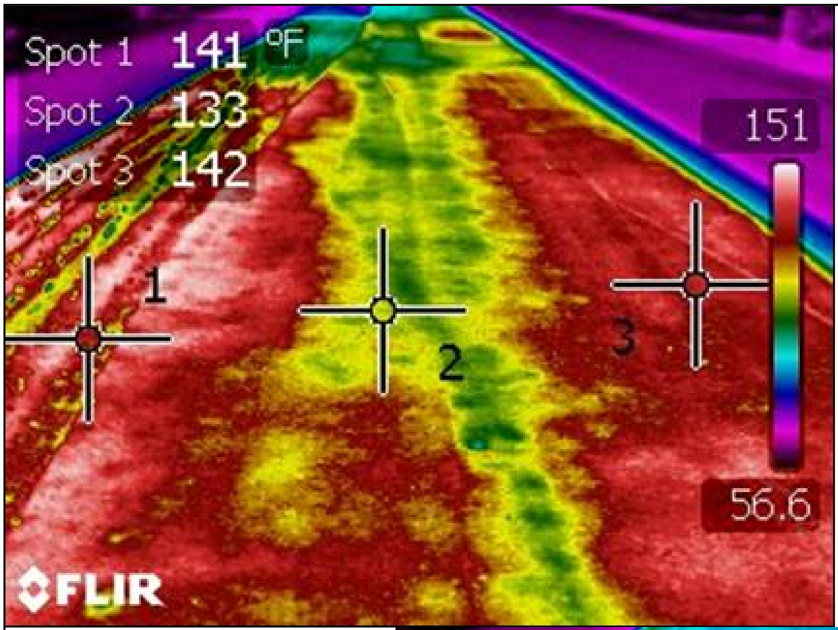
A demonstration project is suggested to illustrate the biased inspection and testing and use of the infrared cameras, which is discussed in the Implementation Plan. More importantly, implementation of biased inspection and testing activities should have no impact on construction costs, but should extend the service life of flexible pavements by eliminating the center lane longitudinal cracks and deterioration.

5.3.4 HMA Mixtures with Enhanced Performance Properties

All projects with inferior performance were found to exhibit transverse cracks and tears, alligator cracks, and longitudinal cracks in the wheel path. In addition, surface deterioration (raveling) was recorded on over 50 percent of these projects. The amount and severity of these cracks and raveling can be reduced by using higher quality wearing surfaces; such as SMA and PMA mixtures.

Discussions with contractors, review of field reports, and observations of surface distress suggest that the Type C mixtures specified and placed in the 1980's were susceptible to premature cracking. This condition has changed with some of the revisions made to the HMA specifications in the latter 1990's and early 2000's. However, there are still many projects where excessive cracking has occurred. It is hypothesized that a cause for this premature cracking is a result of the gap-graded and/or uniform-graded, unmodified HMA mixtures that have been used in Michigan, especially for higher volume roadways. Use of wearing courses with enhanced mixture and asphalt properties is expected to reduce the amount of transverse, block cracking, and longitudinal cracking in the wheel path.

MDOT has allowed the use of gap-graded, neat HMA mixtures for the wearing surface. Gap-graded, neat or unmodified HMA mixtures can exhibit high permeability because of the higher portions of larger aggregate in the aggregate blend. Higher permeability mixtures are more susceptible to accelerated aging and moisture infiltration, which increase surface deterioration of the mixture and reduce its resistance to cracking. The intent of this strategy and pilot project is to reduce the amount and severity of various types of cracking (block, alligator, transverse cracks and tears, and longitudinal cracks in the wheel path) and surface deterioration by using HMA mixtures with enhanced properties (PMA and SMA).



Surface temperatures are measured with the infrared cameras. The infrared images define areas with different surface temperatures at a point in time. If areas with a significant drop in temperature are identified with the infrared camera, other tests or inspection techniques are required to determine the cause of the difference.

These two examples or illustrations from an infrared camera show cold spots along the center of the paver (center lane cold spots). Multiple photos or illustrations can be taken to monitor the change in temperature after paving and/or compaction.

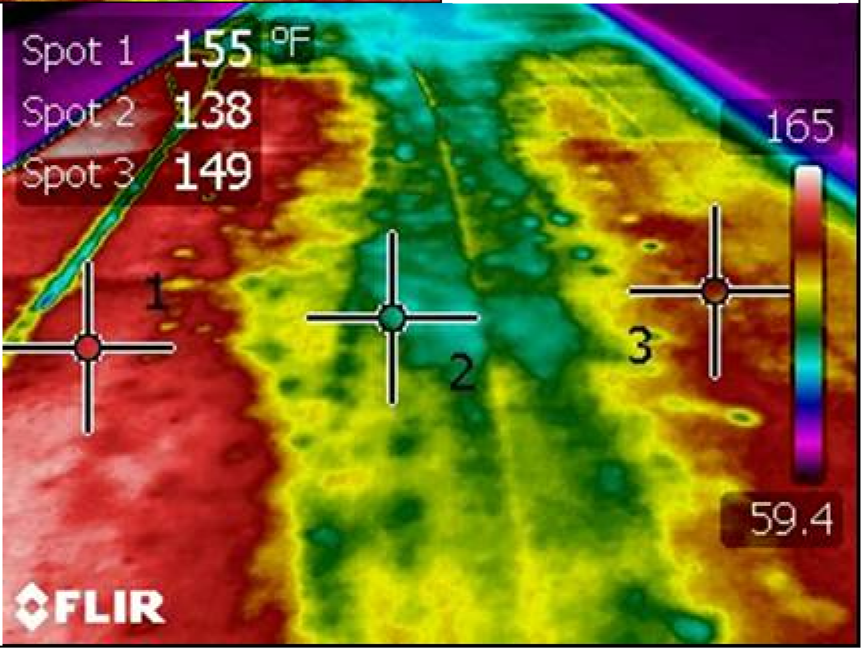
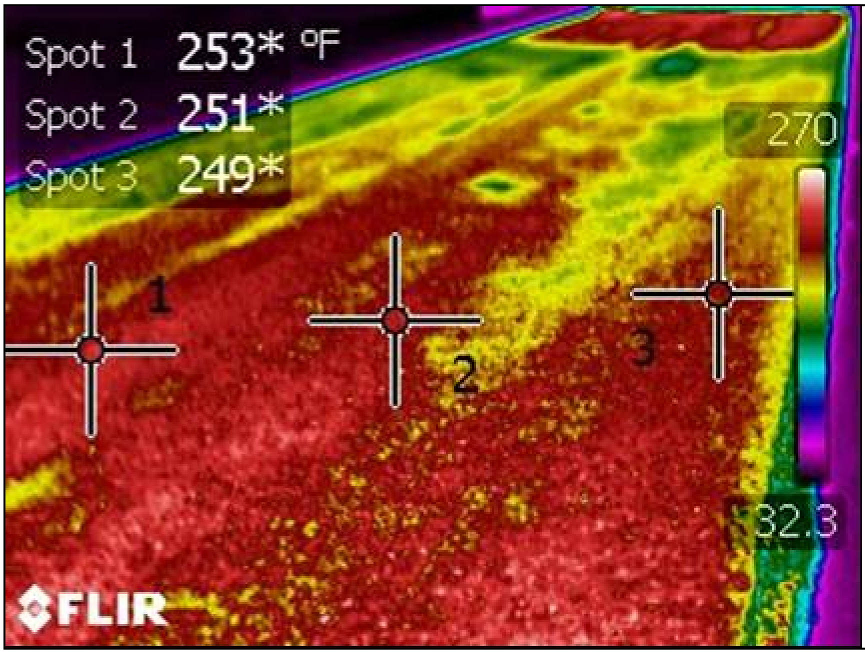
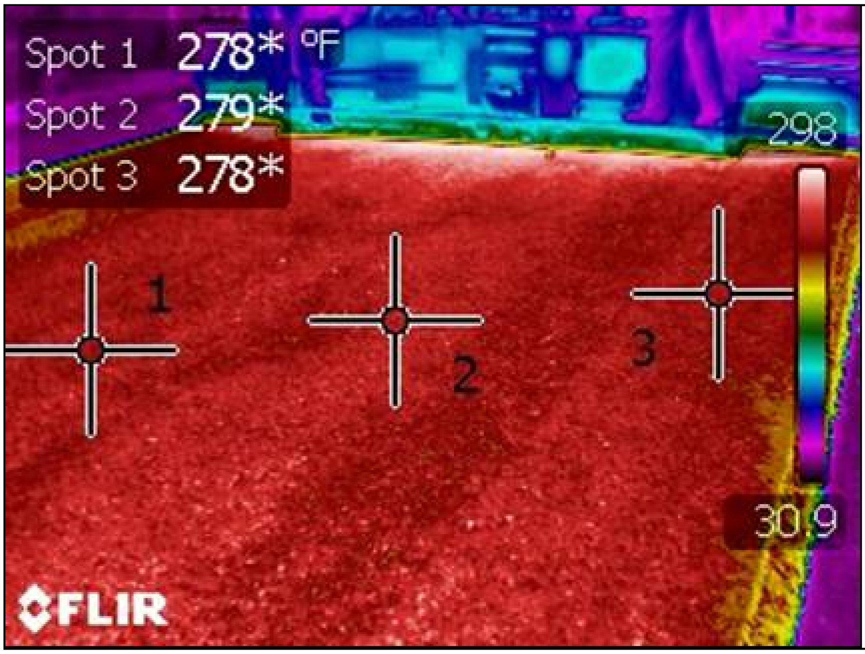


Figure 51. Use of Infrared Camera to Locate Cold Spots or Areas with Low Density; Near Center of Paver (sometimes referred to as temperature segregation)



Infrared photo taken behind the paver prior to compaction and shows truck to truck temperature difference caused by a delay in trucks arriving at the site; the paver had to stop and wait for the next truck.

Figure 52. Use of Infrared Camera to Locate Cold Spots or Areas with Low Density; Delay in Delivery of Mix Where Paver is Sitting for an Extended Period of Time



Infrared photo taken behind the paver showing uniform surface temperatures across the paving lane.

Figure 53. Use of Infrared Camera to Check for Temperature Differences Behind the Paver

The MDOT database does not identify those projects where PMA or SMA type engineered mixtures were placed as the wearing surface. It is recommended that MDOT start recording and documenting the projects where these mixtures with enhanced properties have been used to establish performance characteristics that can be quantified and compared to conventional, unmodified or neat HMA mixtures for the site features, materials, and other conditions encountered in Michigan. In the interim, however, there is a lot of support that documents the benefit and reduction in surface distress with the use of PMA and/or SMA mixtures to be used as the wearing surface (Von Quintus, et al., 2003). In addition, the Asphalt Institute and other agencies (for example; Colorado and Wisconsin DOT) have sponsored studies related to the use of PMA and SMA mixtures to enhance pavement performance and reduce pavement distress. Thus, the objective of this strategy is:

- Documentation and evidence to MDOT and contractors for quantifying the magnitude of the extended service life or reduction in pavement distress with the use of engineered mixture with enhanced properties (PMA and SMA mixtures) by reducing the length of transverse cracks, block cracking, longitudinal cracks in the wheel path and surface deterioration, or to minimize the use of gap-graded aggregate blends.

The data from the demonstration project can be used to confirm the expected increase in service life of 3 to 5 years that has been documented and reported by other agencies (Asphalt Institute, Colorado DOT, etc.). It is recommended that MDOT start recording and documenting the projects where these mixtures with enhanced properties have been used to establish performance characteristics that can be quantified and compared to conventional, neat HMA mixtures for the site features, materials, and other conditions encountered in Michigan.

5.3.5 Use of A HMA Performance Test to Confirm Mixture Design

The last strategy recommended to extend pavement life is to include a fundamental test within the mixture design or confirmation stage. It is expected that industry (contractors, as well as MDOT personnel) may object to this recommendation, and it will take longer to implement. In addition, the strategies previously discussed must first be implemented for this strategy to have any significant impact on extending service life.

It has been reported by multiple researchers that volumetric properties by themselves do not ensure an HMA mixture has the required performance properties to meet the design requirements (Von Quintus, et al., 1991 and 2009; Von Quintus and Leahy, 1994). A fundamental performance test is recommended to confirm the HMA properties used in structural design and support the volumetric mixture design procedure. This is a long term implementation mitigation strategy. Specifically, this mitigation strategy is compatible with and a confirmation of the mitigation strategy discussed under subsection 5.3.2. This strategy should be implemented after the first three mitigation strategies have been completed. It is also suggested that this strategy be implemented during the implementation and use of the new Mechanistic-Empirical Pavement Design (MEPDG) procedure.

A pilot project is recommended for this mitigation strategy because any changes in the mixture design procedure and/or criteria will take time to implement. This pilot project should be conducted after the other mitigation strategies have been implemented. The reason that the implementation of a fundamental performance test is included as a mitigation strategy is to start the planning process early. In addition, this mitigation strategy should be compatible with the use of the MEPDG for pavement structural design – integrating mixture design, structural design, and quality assurance or construction.

The objective of this implementation strategy is to select and use a fundamental performance test for confirming the volumetric properties used during the mixture design stage in selecting the target asphalt content and job mix formula, and to predict the behavior and performance of HMA mixtures. In other words, the objective is to integrate structural design, mixture design, and construction (quality assurance/acceptance), which currently does not occur.

MDOT has already sponsored a study for measuring the dynamic modulus and flow number on different HMA mixtures (You, et al., 2009). This laboratory study will be useful in moving forward with this mitigation strategy. However, MDOT is encouraged to consider and use a mixture's resistance to cracking because nearly all of the roadway segments with poor performance exhibited excessive cracking, rather than excessive rutting. The fundamental properties and test mentioned under mitigation strategy #2 should be considered in supporting the volumetric mixture design procedure.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

This section of the report summarizes key findings from the analyses and comparisons for identifying common trends of the pavement performance indicators and factors that contribute to accelerated deterioration and/or enhanced performance.

6.1 Findings

6.1.1 Preventive Maintenance

Most preventive maintenance strategies used in Michigan have provided enhanced performance for HMA pavements, as well as HMA overlays and other rehabilitation strategies. This management policy should be continued, because the preservation dollars provide a benefit to the Michigan taxpayers. The preservation strategies providing enhanced service lives, on the average, are: the cold-mill and resurface (7 years), thin and ultra thin HMA overlays (6 years), and micro-surfacing (5 years). Chip seals were found to provide only minimal added service life (3 years).

Chip seals have provided minimum increases in performance with the median service life of 3 years. Most agencies that routinely use this preservation strategy have seen 5+ years of service life. In general, the difference between Michigan and these other agencies with longer service live for chip seals is a harsher climate. The median service life for the other commonly used pavement preservation strategies in Michigan is similar to what other agencies have reported through their individual pavement management databases.

Pavement preservation or preventive maintenance activities affecting the performance indicators have been placed on 38 percent of the PM segments. The crush and shape, hot in place recycling, and resurface were combined with the cold mill and resurface category for evaluating pavement performance. There were too few data within a specific preservation category to evaluate the performance separately. These preservation methods were found to have a significant reduction in one or more of the performance indicators.

6.1.2 Analysis of Performance Indicators

The DI and IRI values were related to increasing age and/or traffic level. However, the DI values and rut depths were found to be independent of the study parameters included in the analysis and in MDOT's performance database (highway type, traffic, climate, HMA mix type, and subgrade). IRI was the only parameter found to be somewhat related to the highway type and traffic volume from a categorical analysis. This trend, however, did not explain the difference between pavements with poor and good performance. The following summarizes the important findings from the research study, as related to the performance indicators included in MDOT's database.

- The operational policies and specifications implemented by MDOT in the 1990's, including an aggressive preventive maintenance program, have had a positive impact on pavement performance.

  - Rutting was found to be very low, with the exception of a few roadway segments. Department policies that have been implemented for the past 10 to 15 years have all but eliminated the issue of rutting. The average rut depths for over 90 percent of the roadway PM segments are less than 0.30 inches.

  - IRI is considered low for many of the roadway segments along the freeways. On the average, the non-freeway segments were found to have about 20 percent higher IRI values than for the freeway segments. The average IRI values for over 85 percent of the roadway segments along freeways is less than 100 in./mi., while only about 50 percent of the non-freeway segments are less than 100 in./mi.

  - The distress index was found to be the predominate cause for maintenance and/or rehabilitation based on the deterioration coefficients determined from this study. The DI values for about 75 percent of the roadway segments are less than 20. More importantly, the average DI values determined from the PM roadway segments used in this study were found to be lower at the time when preventive maintenance and/or rehabilitation activities were performed than the values reported in MDOT’s *Pavement Design and Selection Manual* dated 2005.

- The median age of the pavement at the time of applying the first pavement preservation activity is similar to the value listed in the Michigan *Pavement Design and Selection Manual* (2005) for new construction or reconstruction. The average values determined from this study for the different pavement groups are listed below.

  - New construction/reconstruction – 10 years
  - Crush and shape with bituminous surface – 6 years
  - Mill and resurface– 6 years

  The distress indices, however, are lower. In addition, the DI and IRI values at the time of when a pavement preservation activity was applied to the surface are lower than what other agencies have used in managing their pavements. As noted above, the DI value at which some preventive maintenance activity is recorded in the database was found to be lower than MDOT’s average values included in the *Pavement Design and Selection Manual*. This finding does not imply that MDOT’s practices should be revised, but suggests that the values should be reduced or the average service life to a preventive maintenance activity increased from a life cycle cost standpoint. More in depth analyses are needed before making any revisions to MDOT’s Manual.

- The crush and shape with HMA surface structural category was found to have the lower DI values and better performance than for pavements in the new construction or reconstruction category. Most of the crush and shape structures, however, are located in the northern part of Michigan with lower traffic volumes. The analysis did not determine which factor was the more important one contributing to this finding.

- Preventive maintenance is applied sooner to the crush and shape with HMA surface pavements with lower performance indicators than for the other pavement structural groups (refer to Tables 6, 9, and 12).

- The coefficients for the DI and rut depth deterioration relationships (equations 1 and 2) were found to be independent of one another, while the coefficients for the IRI deterioration relationship were found to be related (g_2 is inversely proportional to g_1).

- PM segments were identified that exhibited good and poor performance. These roadway segments are listed in Appendix A, and were used in more detailed studies to try and explain or confirm the reasons for the more extreme performance differences. These PM segments were primarily identified based on the distress indices and IRI values. The majority of the PM segments have exhibited good rutting resistance – at least based on the average rut depths stored in the MDOT database. The detailed distress data for these segments was found to be useful in determining reasons for the poor performance. The reasons are provided in the next section of this chapter.

6.1.3 Factors Contributing to Good and Poorly Performing Pavements

Factors contributing to good and poor performance were not identified through analyses of MDOT’s data. Pavement structure, HMA mixture type, soil type, traffic volume, MDOT region, and climate were not found to be factors in discriminating between roadway segments exhibiting good and poor performance. In other words, these factor-variables do not explain the difference between the roadway segments with poor and good performance. This finding does not mean that these factors are unimportant to pavement performance, but it does suggest that MDOT design and management policies have adequately accounted for these factors. It also suggests that other factors are more important. The factors identified include construction and HMA mixture related factors.

The detailed distress data was used to determine the individual distresses that were commonly recorded on roadway segments falling in the category of poor performance. Roadway segments falling in the poor performance category were found to exhibit excessive longitudinal centerline cracks, longitudinal center lane cracks, longitudinal wheel path cracks, edge cracks, alligator cracks, block cracks, and/or transverse cracks and tears. Many of the segments with good performance also exhibited longitudinal centerline, center lane, edge and wheel path cracking, alligator cracking, and transverse cracking. The magnitudes of these cracks, however, were much lower and were recorded over a longer period of time for the segments with good performance.

A detailed forensic investigation, including field and laboratory tests, will be needed to determine the cause of the projects exhibiting poor performance on a project by project basis. Based on experience, historical information, discussions with MDOT and industry personnel, and an evaluation of the detailed distress data, the following construction and mixture factors are related to or resulting in poor performance; which is project specific and difficult to prove or confirm using network data.

a. Longitudinal construction joint defects, causing longitudinal centerline cracking.
b. Center lane defects, causing longitudinal center lane cracking.

c. Gap-graded, neat HMA mixtures placed as a wearing surface, causing longitudinal wheel path, alligator, block, and transverse cracks.
d. Mixture design process using high levels of Ndesign in the gyratory compactor to determine the target asphalt content of HMA mixtures, causing longitudinal wheel path, alligator, block, and transverse cracks.

6.2 General Recommendations to Enhance Pavement Performance

Preventive Maintenance

The preventive maintenance policies and strategies that have been used by MDOT should be continued. The only exception to this recommendation is the use of chip seals. The average service life of chips seals was found to be 3 years. It was recommended that MDOT restrict the use of chip seals to specific low volume roads with adequate structural support, and sponsor a materials research study for improving their performance.

Longitudinal Construction Joint Specification

Extensive longitudinal centerline cracking was observed on 100 percent of the projects falling in the group with poor performance. The amount and severity of centerline cracking can be reduced by improving on the construction and compaction of the longitudinal construction joint. Implementation of a longitudinal construction joint specification is considered a high importance mitigation strategy to MDOT and industry in terms of extending the service life and reducing LCCs of flexible pavements. It is recommended that this strategy be implemented immediately. Implementation of a longitudinal construction joint specification is included in the Implementation Plan.

Biased Sampling and Testing During Construction

Nearly all projects falling in the category with poor performance exhibited excessive center lane longitudinal cracking. These cracks are more related to the paving equipment and construction practice. Implementation and use of biased sampling and testing methods is considered a high importance mitigation strategy to MDOT and industry to reduce the number of projects with accelerated aging and deterioration. A draft set of guidelines for biased sampling and testing is included in the Implementation Plan, which includes the purchase of infrared cameras. It was also recommended that this mitigation strategy be implemented immediately.

Revision to HMA Mixture Design Procedure

Transverse, longitudinal (edge and wheel path), alligator, and block cracking were found to be common distresses recorded in the distress index database for roadway segments with poor performance. These cracks are characteristic of high stiffness, low strength HMA mixtures relative to the supporting layers. These cracks can be reduced by designing HMA mixtures that are more tolerant to tensile strains, rather than increasing the thickness of the HMA layers. Lowering the number of N_{design} gyrations for mixture design and revising the aggregate blend or gradation for dense-graded, neat HMA wearing surfaces is considered a high importance mitigation strategy to reduce the number of projects with accelerated aging and deterioration.

Wearing Surface with Enhanced Mixture Properties

Transverse cracks and tears, alligator cracks, and longitudinal cracks in the wheel path were also recorded for many projects falling in the category with poor performance, especially those with higher traffic volumes. Many of these projects also had excessive levels of raveling or surface deterioration. The length and severity of these cracks and surface deterioration can be reduced by using higher quality wearing surfaces, like SMA and PMA mixtures. Specifying the use of SMA and PMA mixtures with enhanced mixture properties on higher volume roadways is considered important to extend the service life of flexible pavements and HMA overlays.

Fundamental Performance Test

A long term recommendation is to include the use of a fundamental test in the HMA mixture design stage. The purpose of this mitigation strategy is to select and use a fundamental performance test for confirming the mixture design using volumetric properties to select the target asphalt content and job mix formula. It was also recommended that this strategy be implemented, but only after the other mitigation strategies have been completed.

6.3 Other Recommendations to Assist in Future Research Studies

- MDOT has been improving on the information included in the performance database for tracking the impact of different parameters on the performance of asphalt pavements and HMA overlays. To support the pilot projects that have been recommended, it is suggested that MDOT include an additional column in the performance database for the specific type of mixture being placed on the roadway. This mixture information will be needed to confirm the enhanced performance of SMA and PMA mixtures and aggregate blend or gradation.

- MDOT has used a standard power law (referred to as a logistic growth curve) for predicting the DI values with time. The power law is calibrated based on data collected in previous years. However, it is recommended that MDOT begin using the deterioration relationships that were used to predict the age at which the threshold or critical value is exceeded for the different performance indicators monitored by MDOT. It is also recommended that MDOT begin using IRI as an additional factor to establish and predict the service life of asphalt pavements and HMA overlays.

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APPENDIX A –PAVEMENT CONTROL SECTIONS EXHIBITING GOOD AND POOR PERFORMANCE

Appendix A includes a listing of the roadway segments with good and poor performance as determined from the performance indicators used in the analysis. Table A.1 is a listing of the roadway segments that have exhibited good or exceptional performance characteristics based on the pavement deterioration analysis that was completed on each segment included in the study with sufficient performance time series data. These segments have exhibited significantly delayed performance indicators (distress index, rut depth, and IRI).

Table A.2 is a listing of the roadway segments that have exhibited poor or inferior performance characteristics or premature distress based on the pavement deterioration analysis that was completed on each segment included in the study with sufficient performance time series data. These segments have exhibited accelerated distresses (based on the distress index, rut depth, and IRI values).

Table A.1. Roadway Segments with Good Performance

ID Number	Location	CS #	Mile Point		JN #
			Beginning	End	
Crush & Shape with HMA Surface					
7S	I-75 SB; M-68 to NYC RR; Thin HMA Overlay	16091	0.000	2.096	32510
9N	I-75 NB; Nine Mile Rd. to N. of M-18; Original Structure & Micro-Surface	72061	13.061	19.154	34066
15N, 15S	I-75 NB & SB; N. of M-32 to Sturgeon Valley Road; Thin HMA Overlay	69014	0.493	8.239	44972
17N; 17S	US-127 NB & SB; N. of Wexford Dr. to M-55; Original Structure & Thin Overlay	72013	3.002	12.176	34069
19S	I-75 SB; S. of I-75 BL to M-72; Original & Thin Overlay	20014	4.104	5.392	45845
27S	I-75 SB; N. of M-18 to Roscommon/Crawford Co. Line; Original Structure	72061	19.208	23.675	45080
5	M-66; Lilack Creek to Antrim/Charleviox Co. Line; Original Structure	5051	11.948	15.583	26646
7	M-66; Charleviox/Antrim Co. Line to N. of Goebel Rd.; Original Structure	15031	0.000	1.888	26646
9	M-33/68; E. of the W. Junction of M-33 to Clark St.; Original Structure	16023	0.153	6.932	26670
11	M-33/68; E. of Black River Ave. to Cheboygan/Presque Isle Co.; Original Structure	16023	7.243	9.668	26670
49	M-66; US-131 to N. Of Old State Rd.; Original Structure	5051	0.016	11.962	32310
111	M-65; Alcona/Iosco Co. Line to S. of Bamfield Rd.; Original Structure	1011	0.000	3.904	38089

Table A.1. Roadway Segments with Good Performance, continued

ID Number	Location	CS #	Mile Point		JN #
			Beginning	End	
Crush & Shape with HMA Surface					
113	M-37; N. of Swaney Rd. to N. of Eagle Rise Rd.; Original Structure & Chip Seal	28052	15.544	17.591	32326
141	M-115; W of 17 ½ Rd. to S. of 21 ½ Rd.; Original Structure	83052	2.806	6.590	37.868
153	M-553; M-35 to N. of County Rd NNA; Original Structure	52055	0.000	7.231	48407
165	M-94; N. of 5th St. to US-41; Original Structure	52022	3.646	10.789	50392
185	US-45; S. of Federal Forest Rd. 730 to S. of M-26; Original Structure	66032	7.182	13.811	45050
187	M-120; M-82 to S. of Sunset Blvd.; Original Structure	62021	0.000	6.435	45788
195	M-65; M-72 West to N. of M-72 East; Original Structure	1022	0.000	6.934	48554
197	US-2; Roosevelt St. to E. of Powderhorn/Puritan Rd.; Original Structure	27021	2.463	5.378	48343
199	M-22; S. of Novotny Rd. to M-201; Original Structure	45013	7.462	13.262	39869
233	US-2; W. of FFR 3920 to E. of Golden Lake Trail; Original Structure	36021	1.639	5.336	45115
Mill & Resurface					
31N	M-99 NB; Victor Ave. to N. of Moores River Dr.; Original Overlay	33011	4.233	5.238	44737
31S	M-99SB; Victor Ave. to N. of Moores River Dr.	33011	4.233	5.241	44737
25W	I-96 WB; M-104 to 88th Ave.	70063	0.000	3.528	44155
23E	I-96 EB; Ottawa/Muskegan Co. Line to M-104	70064	0.000	3.860	44155
21E	M-44 EB; I-96 to Eagle Crest Drive	41051	4.240	5.383	44157
47	US-10; East of Emily St. to Jackson Rd.	53021	0.534	1.130	40743
49	US-23; East of Sterling Rd. to West of Washington Cutoff	06072	5.389	5.834	32357
57	M-21; Jackson St. to E. of James St.; Cold-Mill Resurface	41043	7.043	15.077	90090
Resurface					
23S	I-75 SB; N. of Afton Rd. to M-68; Original Overlay	16093	6.714	15.170	53353
23N	I-75 NB; North of Afton Rd. to M-68	16093	6.714	15.170	53353
21N	I-75 NB; M-32 to South of Sturgeon Valley Rd.	69014	0.000	8.220	53353
197	M-86; M-66 to West of Lepley	78062	0.000	0.488	32381
207	M-64; Ontonagon C. Line to South of M-28	66011	0.000	0.488	32381
213	M-62; South of Redfield St. to Eltzroths Rd.	14031	0.000	2.066	38083

Table A.1. Roadway Segments with Good Performance, continued

ID Number	Location	CS #	Mile Point		JN #
			Beginning	End	
219	M-37; North of Eagle Rise Rd. to End of M-37	28052	17.457	18.041	32326
225	M-46; East of Maynard to East of Loree Rd.	74062	13.440	13.969	38023
239	M-203; North of Anthony St. to Cemetary Rd.	31031	1.189	1.814	44292
Reconstruction/New Construction					
32S	M-44 SB; Windcrest Court to S. of 3 Mile Rd.	41051	5.487	6.882	25745
34S	M-44 SB; South of 3 Mile Rd. to Plainfield Ave.	41051	7.855	10.055	25745
35W	I-96 WB; West of Williams Rd. to Eaton/Ingham Co. Line	23151	1.558	2.858	29581
35E	I-96 EB; West of Williams Rd. to Eaton/Ingham Co. Line	23151	1.621	2.842	29581
37N	US-127 NB; Price Rd. to South of Wildcat Rd.	19033	8.526	12.775	20046
1	US-2; West of Chippewa Ave. to M-94	75021	12.501	13.455	07906
3	US-2; M-94 to West of Range Street	75022	1.276	1.416	07906
11	M-54; I-75 to Grand Blanc Rd.	25074	0.086	0.869	00367
13	M-104; Lake Ave. to Fruitport Rd.	70081	1.679	2.048	21381
17	US-2; Boucha Rd. to Blake St.	49022	5.820	6.20	19434
25	US-2; County Rd. 557 South to East of County Rd. 557 North	55022	4.953	5.307	07901
31	M-24; End Divided (Goldengate) to Begin Divided (Elizabeth)	63112	6.577	7.683	11320
33	US-2; East of Worth Rd. to East of Wildwood Dr.	49023	4.316	8.561	17730
41	M-183; South of KK Rd. to North of Fayette State Park Entrance	21041	15.154	16.263	24572
43	M-183; North of KK Rd. to West of LL Rd.	21041	14.100	14.860	24572
53	US-2; East of US-41 to the Rapid River	21024	0.171	0.526	27836
57	US-41; M-203 to East of White St.	31052	0.943	1.919	26620
65	M-32; M-33 to Montmorency/Alpena Co. Line	60022	0.000	10.265	21218
69	M-68; North of Wilson Rd. to Barbara Ave.	16021	6.875	7.282	31055
83	M-55; Federal Ave. to M-18	72022	0.000	5.200	31009
93	M-95; Woodward Ave. to US-1/US-141	22011	1.859	2.792	34039

Table A.2. Roadway Segments with Poor Performance

PM Segment						Critical Perf. Indicator
ID Number	Location	CS #	Mile Point		JN #	
			Start	End		
Crush & Shape with HMA Surface						
3S	I-75 SB; Cheboygan/Otsego Co. Line to N. of Afton Rd.	16093	0.000	6.656	30728	DI
13N	I-75 NB; US-127 Merge to S. of M-72; Original Pavement	20014	0.076	4.232	44827	DI
9S	I-75 SB; Nine Mile Rd. to N. of M-18; Original Structure & Micro-Surface	72061	13.061	19.154	34066	RD
17	M-35; Anderson Rd. to S. of County Rd. 480; Original Structure	52032	19.234	24.648	26628	DI
25	M-38; Houghton/Ontonagon Co. Line to Houghton/Baraga Co. Line; Original Structure	31041	0.040	12.298	26624	DI
35	M-26; County Rd. EM26T to Kearsarge St./Chassell-Paireso; Original Structure	31011	7.228	19.674	32262	DI
53	M-43; 41st St. to W. of M-40; Original Structure	80042	6.584	9.951	31084	DI
87	M-140; M-62 to Napier Ave.; Original Structure	11071	0.0251	7.522	3.4089	DI
101	US-2; W. of County Rd. 525 to Old US-2; Original Structure	27023	8.191	24.386	35983	DI
103	M-115; S. of 28 Road to N. of 13th St.; Original Structure	83052	8.788	15.229	37903	DI
271	US-41; N. of Traunik Kiva Rd. to the W. Branch of the Whitefish; Original Structure	2011	9.289	9.715	50702	DI
Mill & Resurface						
21E	M-44EB; I-96 to Eagle Crest Dr.; Original Surface	41051	4.240	5.383	44157	IRI
35N	US-31 BR NB; Shoreline Dr. to Bayou Ave.; Original Overlay & Cold-Mill & Resurface	61153	0.986	1.714	45782	IRI
51	M-55; US-127 to Federal Ave.; Original Overlay	72031	0.000	3.582	44829	DI, IRI
55	M-21; W. of Valley Vista Dr. to W. of Smith St.; Original Overlay	41043	12.764	13.317	34074	DI, IRI
55	M-21; W. of Valley Vista Dr. to W. of Smith St.; Cold-Mill & Resurface	41043	7.187	15.077	59608	IRI
57	M-21; Jackson St. to E. of James St.; Original Overlay	41-43	13.994	14.311	34074	DI
59	M-34; US-127 to Maple Grove Ave.; Original Overlay	46041	0.000	0.690	38005	IRI
65	M-13; Remington St. to Janes Ave.; Original Overlay	73051	17.348	18.216	45441	DI

Table A.2. Roadway Segments with Poor Performance, continued

PM Segment						Critical Perf. Indicator
ID Number	**Location**	**CS #**	**Mile Point**		**JN #**	
			Start	**End**		
Mill and Resurface						
67	M-236; S. of Wilson Rd. to S. of Holiday Dr.; Original Overlay	11019	0.635	1.340	45461	DI
73	M-32; W. of Hallock Rd. to Murner Rd.; Original Overlay & Mirco-Surface	69021	4.140	9.760	32331	DI
77	M-32; Baker Rd. to West St.; Original Overlay	60021	12.435	14.441	51248	DI
85	US-223; E. of Treat Hwy. to Humphrey Hwy.; Original Overlay	46062	4.280	6.160	43498	DI
Resurface						
203	M-25; N. of Woods St. to N. of Heineman Rd.; Original Overlay	32092	0.498	7.348	32361	DI
209	M-28; E. of Sand River Rd. to Shelter Bay Rd.; Original Overlay	02041	0.000	8.177	44806	DI
211	M-66; S. Drive S to L Dr.; Original Overlay	13031	2.222	6.337	34497	DI
231	M-69; M-95 to Tower Rd.; Original Overlay	22042	0.000	9.631	50785	DI
243	M-179; 12th St. to Patterson Rd.; Original Overlay	03042	0.000	6.129	52083	DI
245	M-179; Patterson Rd. to M-43; Original Overlay	08033	0.000	10.709	52083	DI
New Construction – Flexible						
13S	US-131 SB; E. Branch of M-46 to Montcalm/Mecosta Co. Line; Cold-Mill Resurface	59012	9.650	13.080	46447	DI
14N	US-131 NB; Tamarack Rd. to Cutler Rd.; Cold-Mill Resurface	59012	9.650	13.080	46447	DI
15N	US-131 NB; Cutler Rd. to Montcalm/Mecosta Co. Line; Cold-Mill Resurface	59012	9.650	13.080	46447	DI
17W	M-59 WB; Oakland/Livingston Co. Line to Bogie Lake Rd.; Cold-Mill Resurface	63041	0.000	12.350	44344	DI
19N	US-131 NB; Lincoln Rd. to 13 Mile Rd.; Original Structure & CMR	54014	0.000	5.026	17765	DI
20N	US-131 NB; 13 Mile Rd. to N. of 19 Mile Rd.; Original Structure	50414	5.369	11.577	17765	DI
20S	US-131 SB; 14 Mile Rd. to N. of 19 Mile Rd.; Cold-Mill & Resurface	50414	0.000	11.660	53285	DI
21N	US-131 NB; N. of 19 Mile Rd. to Mecosta/Osceola Co. Line; Overlay & Cold-Mill & Resurface	50414	11.611	16.126	74790	DI

Table A.2. Roadway Segments with Poor Performance, continued

PM Segment						Critical Perf. Indicator
ID Number	Location	CS #	Mile Point		JN #	
			Start	End		
New Construction – Flexible						
13N	US-131 NB; E. Branch of M-46 to Tamarack Rd.; Cold-Mill & Resurface	59012	9.868	12.791	5.3285	DI
23S	US-131 SB; Osceola/Mecosta Co. Line to US-10; Micro-Surface & Cold Mill & Resurface	67016	0.010	5.750	39250	DI
25N	US-131 NB; US-10 to S. of 13 Mile Rd.; Micro-Surface	67017	0.000	7.573	47975	DI
25S	US-131 SB; US-10 to S. of 13 Mile Rd.; Original Structure & Micro-Surface	67017	0.000	7.573	18255 44208	DI
31N	M-44 NB; N. of I-96 to Windcrest Court; Original Structure	41051	4.287	5.155	25745	DI
31S	M-44 SB; N. of I-96 to Windcrest Court; Original Structure & CMR	41051	4.232	10.055	44157	DI
48S 48N	M-66 NB & SB; Begin Divided to Beckley Rd.; Original Structure	13031	13.077	14.094	79856	DI & IRI
56N 56S	US-127 NB & SB; M-57 to N. of Tuscola-Saginaw-Bay RR; Original Structure	29011	4.030	10.360	84176	DI
55E	M-6 EB; W. of Patterson Ave. to CSX Railroad (S. of I-96); Original Structure & Cold-Mill & Resurface	41064	11.618	16.309	53508	DI
5	US-12; Fairview Dr. to Crooked Creek Dr.; Original Structure, CMR & Mirco-Surface	78022	3.864	7.504	50856 13376	DI
7	M-37; M-82 to S. of 64th St.; Original Structure	62031	9.583	10.525	16655	DI
9	M-32; Jerome St. to Hall Rd.; Chip Seal	60021	14.700	18.080	20301	DI
15	M-54; Grand Blanc Rd. to Gibson Rd.; Chip Seal & Cold-Mill Resurface	25074	0.180	2.840	50805 79835	DI
19	US-2; Balsam Lane to Nomenco Rd.; Original Structure, Thin Overlay & Cold-Mill & Resurface	55022	0.000	9.583	07901 45116 47455	DI & IRI
21	US-2; E. of Nomenco Rd. to Daves Lane; Original Structure, Thin Overlay, & Cold-Mill & Resurface	55022	0.000	9.583	07901 45116 47455	DI & IRI
29	I-196 BL; Burlingame Ave. to Plaster Creek; Cold-Mill & Resurface	41042	2.102	3.138	79321	DI
55	US-10/US-31; E. of Brye Rd. to Reinberg Rd.; Micro-Surface	53032	1.890	6.170	60363	DI
57	US-41; M-203 to E. of White St.; Original Structure	31052	0.915	12.050	26620	DI
61	M-183; N. of Water St. to S. of Fayette Ave.; Micro-Surface	21042	0.000	16.420	76229	DI

Table A.2. Roadway Segments with Poor Performance, continued

PM Segment						Critical Perf. Indicator
ID Number	Location	CS #	Mile Point		JN #	
			Start	End		
New Construction – Flexible						
75	US-31; N. of Beyer Rd. to S. of the Big Sable River; Micro-Surface	53033	6.543	13.691	50625	DI
135	M-37; Moon Rd. to N. of Smith Rd.; Original Structure	61131	1.486	2.897	03036	DI
143	M-32; N. of Hallenius Rd. to N. of Greenview Dr.; Micro-Surface	69021	0.000	9.781	58168	DI
145	M-32; E. of Burdo Rd. to W. of Townline Rd.; Original Structure	69021	6.800	7.900	32331	DI
147	M-32; E. of Townline Rd. to Murner Rd.; Original Structure & Micro-Surface	69021	0.000	9.781	32331 58168	DI
165	Old M-14; Canton Center Rd. to Lilly Rd.; Original Structure	82101	3.361	4.729	45707	DI & IRI
169	US-2; E. of Karling Rd. to W. of Comet Rd.; Original Structure	27023	0.000	8.190	54079	DI
171	US-2; W. of Sampson Rd. to E. of Sampson Rd.; Original Structure	27023	0.000	8.190	54079	DI

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APPENDIX B – DISTRESS DATA SUMMARIES FOR INDIVIDUAL PROJECTS EXHIBITING POOR AND GOOD PERFORMANCE

B.1 Roadway Segments Exhibiting Inferior or Poor Performance

Detailed Distress Index Evaluation for the Crush and Shape with Bituminous Surface Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane length	Block Cracking; percent of lane length	Longitudinal Cracking; percent of lane length				Transverse Cracking; Occurrances per mile		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
I-75	16093	0.0	0.0	1.2	78.8	8.3	0.5	4.2	12.0	0.0	0.0	0.0	0.0
I-75	20014	2.1	0.0	0.1	100.8	44.1	21.3	77.2	15.9	0.0	0.0	0.0	0.0
US-2	27023	5.5	5.3	19.2	64.7	51.0	18.1	60.8	172.1	0.0	0.0	0.0	0.5
US-41	2011	0.0	0.0	4.6	62.4	13.2	48.0	136.0	50.0	0.0	0.0	0.0	0.0
M-35	52032	0.1	0.0	3.1	78.8	26.1	15.9	303.7	45.7	45.7	0.0	0.0	0.0
M-38	31041	0.0	0.4	0.6	63.4	0.8	10.3	220.3	67.0	0.5	0.0	0.0	0.0
M-26	31011	6.6	55.9	3.5	22.8	14.6	4.5	44.8	108.0	0.0	7.0	0.0	0.0
M-43	80042	1.5	69.3	11.4	3.8	0.3	3.2	21.5	74.2	0.0	0.0	0.0	0.0
M-140	11071	8.7	1.2	30.6	2.1	2.8	77.2	8.9	34.3	0.0	0.0	0.0	0.1
Overall Average Values		2.7	14.7	8.3	53.1	17.9	22.1	97.5	64.4	5.1	0.8	0.0	0.1

Detailed Distress Index Evaluation for the Mill and Resurface Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane length	Block Cracking; percent of lane length	Longitudinal Cracking; percent of lane length				Transverse Cracking; Occurrances per mile		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
US-223	46062	0.0	0.0	1.3	70.8	57.2	18.2	91.0	56.4	0.0	0.0	0.0	6.6
M-21	41043	0.0	0.0	0.9	28.7	40.5	27.1	130.9	152.7	0.2	0.0	0.0	13.3
M-21	41043(1)	0.0	0.0	0.9	15.6	50.6	31.9	340.6	143.8	2.2	0.0	0.0	0.0
M-13	73051	11.1	0.3	1.3	11.0	31.0	24.9	50.6	87.4	2.0	0.0	0.0	0.0
M-239	11019	0.0	0.0	2.7	38.3	28.1	2.3	32.6	61.0	0.0	0.0	0.0	0.0
M-32	69021	0.4	0.4	14.6	55.0	39.4	17.2	3.7	124.6	0.0	0.0	0.1	0.0
M-32	60021	0.4	12.2	27.0	46.0	30.3	12.9	44.8	526.4	0.1	0.0	0.0	0.0
M-52	73031	5.6	3.1	5.1	52.2	27.3	16.1	207.0	159.7	0.0	0.0	0.0	0.0
Overall Agerage Values		2.2	2.0	6.7	39.7	38.1	18.8	112.7	164.0	0.6	0.0	0.0	2.5

Detailed Distress Index Evaluation for the Resurface Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane length	Block Cracking; percent of lane length	Longitudinal Cracking; percent of lane length				Transverse Cracking; Occurrances per mile		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
M-25	32092	15.9	0.2	1.0	4.1	4.3	81.1	175.9	107.0	0.0	0.0	0.0	0.0
M-28	2041	3.6	0.0	9.3	70.2	25.3	11.6	64.8	119.7	0.0	0.0	0.0	0.0
M-66	13031	18.0	51.3	7.1	5.9	4.2	10.7	54.9	311.9	0.0	0.0	0.0	0.0
M-69	22042	2.0	0.9	3.4	89.9	21.6	5.2	329.2	27.8	0.0	0.0	0.0	0.0
M-179	3042	93.3	14.4	1.9	54.1	11.1	15.2	148.3	92.0	0.1	0.0	0.0	0.0
M-179	8033	16.4	2.4	3.2	69.9	5.6	23.5	139.7	119.1	0.0	0.0	0.0	0.0
Overall Average Values		24.9	11.5	4.3	49.0	12.0	24.6	152.1	129.6	0.0	0.0	0.0	0.0

Detailed Distress Index Evaluation for the New Construction/Reconstruction Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane length	Block Cracking; percent of lane length	Longitudinal Cracking; percent of lane length				Transverse Cracking; Occurrances per mile		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
US-2	55022	0.2	***22.0***	11.1	89.4	92.6	83.8	230.5	217.1	0.0	0.0	0.0	0.0
US-31	53032	26.3	7.2	9.6	68.7	30.8	46.0	115.3	152.6	0.0	0.0	0.0	0.0
US-41	31052	0.0	0.0	6.9	29.1	3.3	41.1	28.5	13.8	0.2	0.0	0.0	0.0
US-31	53033	32.2	1.9	5.4	55.0	6.0	23.3	12.5	23.6	0.0	0.0	0.0	0.0
US-2	27032	0.0	0.0	1.0	28.1	18.5	35.6	85.0	35.0	1.4	0.0	0.0	0.0
US-131	59012	0.0	***12.6***	15.0	90.7	10.6	122.1	189.0	212.6	1.6	0.0	0.0	0.0
US-131	59012(2)	0.0	***0.6***	2.0	89.3	19.4	98.2	161.1	79.4	31.2	0.0	0.0	0.0
US-131	54014	0.0	***11.7***	12.4	92.5	64.6	126.2	347.7	155.4	27.8	0.0	0.0	0.0
US-131	54014(2)	0.0	***8.4***	9.6	92.8	48.8	136.2	301.1	148.4	61.4	0.0	0.0	0.0
US-131	54014(3)	0.0	***7.9***	5.8	10.4	8.1	121.2	204.8	83.7	0.3	0.0	0.0	0.0
US-131	59013(3)	0.2	0.0	2.4	69.5	32.4	112.5	170.0	98.2	0.0	0.0	0.0	0.0
US-131	54014(4)	0.0	0.0	2.1	78.7	11.9	22.5	107.4	45.9	0.0	0.0	0.0	0.0
US-127	29011	19.7	0.7	0.0	84.8	21.6	33.1	53.5	86.6	0.0	0.0	0.0	0.0
US-12	78022	0.0	0.0	34.9	64.4	48.0	116.4	248.9	151.1	0.0	0.0	0.0	0.0
M-32	60021	0.1	87.1	0.0	9.0	0.0	0.8	25.9	24.1	0.0	0.0	0.0	0.0
M-54	25074	39.9	11.4	0.4	9.2	15.7	24.7	206.7	53.3	0.0	1.2	0.0	0.0
M-183	21041	5.0	14.3	1.8	18.0	6.5	14.3	120.0	122.5	0.0	0.0	0.0	0.0
M-37	61131	37.2	14.9	1.0	16.2	25.2	16.9	96.9	162.8	0.0	0.0	0.0	1.6
M-32	69021	0.9	0.0	22.5	68.1	52.7	7.8	5.0	72.9	0.0	0.0	0.0	0.0
M-32	69021(2)	0.5	4.6	18.1	41.5	34.4	24.9	5.8	214.7	0.0	0.0	0.1	0.0
M-44	41051	0.0	0.0	91.3	97.1	79.1	71.1	84.1	193.3	123.1	0.0	0.0	0.0
M-66	13031	0.0	0.0	10.7	57.7	44.7	34.7	60.0	476.7	0.0	0.0	0.0	33.0
M-66	13031(2)	0.0	0.0	9.0	48.7	46.7	13.7	63.3	120.0	0.0	0.0	0.0	46.3
M-6	41064	1.7	0.7	5.9	52.1	78.4	23.5	18.5	31.6	0.0	0.0	0.0	0.4
M-37	62031	0.0	***256.3***	0.5	7.3	1.0	10.8	91.3	25.0	0.0	0.0	0.0	0.0
Overall Average Values		6.6	18.5	11.2	54.7	32.0	54.5	121.3	120.0	9.9	0.0	0.0	3.3

NOTE: The values noted in bold and in italics for the block cracking column represent the number of occurrences per mile, rather than the percentage of lane length with block cracking. The values included in the MDOT database varied between mileage and number of occurrences within a specific length. In addition, the overall average value of block cracking for the new construction/reconstruction category is skewed because of the high number of occurrences for Segment number 62031 for roadway M-37. The original analysis completed on the distress data for block cracking did not recognize this difference in measurement values (miles versus occurrences), which skewed the results and more importance was placed on block cracking.

B.2 Roadway Segments Exhibiting Exceptional or Good Performance

Detailed Distress Index Evaluation for the Crush and Shape with Bituminous Surface Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane	Block Cracking; percent of lane	Longitudinal Cracking; percent of lane length				Transverse Cracking;		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
I-75	16091	0.0	0.0	0.2	94.8	4.2	0.0	1.0	5.7	0.0	0.0	0.0	0.0
I-75	72061	0.0	0.0	0.0	10.3	0.0	0.0	2.0	2.1	0.0	0.0	0.0	0.0
I-75	69014	0.0	0.0	0.2	98.9	0.1	0.5	0.4	2.3	0.0	0.0	0.0	0.0
US-127	72013	0.0	0.0	0.0	1.0	0.0	0.1	1.0	5.5	0.0	0.0	0.0	0.0
I-75	20014	0.0	0.0	0.0	77.1	36.9	0.8	19.4	0.8	0.0	0.0	0.0	0.0
M-66	5051	0.1	0.0	1.3	3.0	2.1	10.1	3.6	51.0	0.1	0.0	1.2	0.0
M-66	15031	0.4	0.0	1.0	44.6	1.2	0.0	5.8	58.7	0.0	0.0	0.0	0.0
M-68	16023	0.0	0.0	0.1	7.7	0.3	1.4	3.8	39.0	0.0	0.0	0.0	0.0
Overall Average Values		0.1	0.0	0.4	42.2	5.6	1.6	4.6	20.6	0.0	0.0	0.2	0.0

Detailed Distress Index Evaluation for the Mill and Resurface Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane	Block Cracking; percent of lane	Longitudinal Cracking; percent of lane length				Transverse Cracking;		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
I-75	16093	0.0	0.0	0.1	99.7	7.4	0.0	0.6	3.8	0.0	0.0	0.0	0.0
M-99	33011	0.0	0.0	1.1	88.2	28.7	1.7	148.0	48.0	0.5	0.0	0.0	0.0
Overall Agerage Values		0.0	0.0	0.6	94.0	18.1	0.9	74.3	25.9	0.3	0.0	0.0	0.0

Detailed Distress Index Evaluation for the New Construction/Reconstruction Pavement Structural Category

Roadway	Segment	Alligator Cracking; percent of lane	Block Cracking; percent of lane	Longitudinal Cracking; percent of lane length				Transverse Cracking;		Patch or Surface Treatment	Flushing	Raveling	Shattered Area
				Center Lane	Centerline	L Edge	Wheel Path	Irregular or Straight	Tears				
US-127	19033	0.0	***0.0***	5.9	97.2	41.1	31.7	54.4	14.4	0.0	0.0	0.0	0.0
I-75	24071	4.1	0.0	2.9	91.0	17.0	63.5	59.2	27.2	0.0	0.0	0.0	0.0
M-54	25074	0.0	0.0	3.3	37.4	9.8	42.4	121.4	32.1	0.0	0.0	0.0	0.0
US-2	49022	0.0	0.0	1.5	33.5	15.3	6.5	130.0	182.5	0.0	0.0	0.0	0.0
M-183	21041	8.3	0.0	0.7	23.3	19.5	1.2	6.3	4.5	0.0	0.0	0.0	0.0
M-68	16021	2.4	2.8	11.0	12.4	4.2	12.6	42.0	40.0	0.0	0.0	0.0	0.0
Overall Average Values		2.5	0.5	4.2	49.1	17.8	26.3	68.9	50.1	0.0	0.0	0.0	0.0

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APPENDIX C – RESEARCH PROGRAMS PROJECT SPLOTLIGHT

Project Annual Summary Report (Report #R4)
Michigan DOT Project #OR09086A

Extending the Life of Asphalt Pavements

Authors: Harold L. Von Quintus, PE; and Rohan Perera, PE

Introduction

Premature aging or accelerated distress of asphalt pavements costs agencies millions of dollars in maintenance and repair (M&R) costs each year to keep these pavements serviceable at a reasonable level. Identifying the causes of premature distress and taking corrective actions can save taxpayers millions of dollars, as well as reduce the number of roadway closures needed for M&R activities. Likewise, identifying pavements that exhibit exceptional performance and the features that contribute to this exceptional performance can increase the average service life of asphalt pavements, and thus, reduce life cycle costs (LCC).

The Michigan Department of Transportation (MDOT) has been tracking the performance and condition of all roadways for decades to understand their pavements performance characteristics, and have periodically evaluated their design, construction, and materials specifications to improve performance. To improve pavement performance and reduce life cycle costs (LCC), MDOT is using the pavement performance database to answer two basic but important questions:

- Why do certain pavements fail to meet their specific design life?
- Why do certain pavements exceed their specific design life?

The goal of this research project was to identify the common features of good and poorly performing asphalt pavements and HMA overlays. MDOT can then focus their efforts on specific features to improve pavement performance and reduce the number of roadway segments exhibiting premature distress.

What we did

Pavement Deterioration Study

Three performance indicators were used to categorize pavement performance: distress index (DI), rut depth, and International Roughness Index (IRI). The performance characteristics were defined by deterioration relationships for each performance indicator. The deterioration relationships used to explain the increase in distress magnitude (distress index, rut depth, and IRI – an estimate of smoothness) over time are listed below.

Distress Index Deterioration Relationship:

$$DI = 100\left(1 - e^{-a\left(\frac{t}{t_{design}}\right)^{b}}\right) \tag{1}$$

Where:

t = Time in years.

t_{design} = Design life or period in years.

a, b = Distress index deterioration coefficients.

Rut Depth Deterioration Relationship:

$$RD = 0.05 + k_1(t)^{k_2} \tag{2}$$

Where:

k_1,k_2 = Rut depth deterioration coefficients.

Smoothness Deterioration Relationship:

$$IRI = IRI_0(e)^{g_1\left(\frac{t^{g2}}{20}\right)} \tag{3}$$

Where:

IRI_0 = Initial IRI value after construction. This parameter was unavailable for the roadway segments, so it was estimated based on the values recorded in the MDOT database shortly after construction for the newer flexible pavements and HMA overlays.

g_1,g_2 = IRI deterioration coefficients.

The coefficients (regression constants) of the deterioration relationships were derived for each roadway segment using linear regression techniques to minimize the error between the predicted and measured performance indicator. These coefficients were determined for each roadway segment prior to and after the application of any preventive maintenance activity placed on that segment.

Determination of Pavement Service Life

The deterioration coefficients were then used to predict the time (age) to a level for each performance indicator. The following threshold values were used:

- Distress Index of 50.
- Rut depth of 0.40 in.
- IRI value of 120 in./mi.

Data Analyses

The roadway segments were grouped by region (climate), pavement structure, roadway type, soil type, and traffic volume. The deterioration coefficients and estimated service life were then used to categorize the performance of all segments included in MDOT's performance database with sufficient data into those exhibiting good and poor (premature distress) performance. Two approaches were used to determine if specific features or parameters were significantly different between the two groups of roadway segments; those exhibiting poor and good performance, which are listed below.

- The Student's t-test approach was used in comparing good and poorly performing pavements for those parameters with continuous numerical values, such as for traffic. The t-test approach compares the mean of each variable in the good group to its mean in the poor group. The hypothesis that the two means are indifferent is rejected if the t-value is significantly large or the p-value is significantly small.
- For those parameters without continuous numerical values (subgrade type or highway type), categorical analyses were employed to decide whether trends existed in each of these variables that distinguished good and poor performance. In other words, the number of good and poor performance segments was determined for each variable within individual groups. Chi-square statistical tests were then employed to compare the numbers with each other across all levels of the variable to determine whether there was a statistical difference.

The detailed distress data included in MDOT's performance database were also used to determine the magnitude and severity of the individual distresses for those roadway segments categorized into inferior and exceptional performance. The detailed distress data were used to determine if construction and material parameters, not recorded in the MDOT pavement performance database, were the probable cause for the distress or poor performance.

What we found

Preventive Maintenance Evaluation

Most preventive maintenance strategies used in Michigan have provided enhanced performance for HMA pavements, as well as HMA overlays and other rehabilitation strategies. This management policy should be continued, because the preservation dollars provide a benefit to the Michigan taxpayers. The preservation strategies providing enhanced service lives, on the average, are: the cold-mill and resurface (7 years), thin and ultra thin HMA overlays (6 years), and micro-surfacing (5 years). Chip seals were found to provide only minimal added service life (3 years).

Factors Contributing to Good and Poorly Performing Pavements

Factors contributing to good and poor performance were not identified through the analyses of MDOT's data. Pavement structure, HMA mixture type, soil type, traffic volume, MDOT region, and climate were not found to be factors in discriminating between roadway segments exhibiting exceptional and inferior performance. In other words, these factor-variables do not explain the difference between the roadway segments with poor and good performance. This finding does not mean that these factors are unimportant to pavement performance, but it does suggest that MDOT design and management policies have adequately accounted for these factors. It also suggests that other factors are more important.

Analysis of Performance Indicators

The DI values and rut depths were found to be independent of the study parameters included in the analysis and in MDOT's performance database (highway type, traffic, climate, HMA mix type, and subgrade). IRI was the only parameter found to be somewhat related to the highway type and traffic volume. The following summarizes the important findings from the research study, as related to the performance indicators included in MDOT's database.

- Rutting was found to be very low, with the exception of a few roadway segments. Department policies that have been implemented for the past 10 to 15 years have all but eliminated the issue of rutting.

- IRI is considered low for many of the roadway segments along the freeways. On the average, the non-freeway segments were found to have about 20 percent higher IRI values than for the freeway segments.

- The distress index was found to be the predominate reason for maintenance and/or rehabilitation using the threshold values listed above. The detailed distress data was used to determine the individual distresses that were commonly recorded on roadway segments falling in the category of poor performance. Roadway segments falling in the poor performance category were found to exhibit excessive longitudinal centerline cracks, longitudinal center lane cracks, longitudinal wheel path cracks, edge cracks, alligator cracks, block cracks, and/or transverse cracks and tears.

The age of the pavement at the time of applying the first pavement preservation activity is similar to the values recommended in the Michigan Pavement Design and Selection Manual. The distress indices, however, are lower. In addition, the distress index and IRI values at the time of when a pavement preservation activity was applied to the surface is lower than what other agencies have used in managing their pavements. More importantly, the DI value at which some preventive maintenance activity is recorded in the database is lower than MDOT's values reported in their Pavement Design and Selection Manual. This finding does not imply that MDOT's practices should be revised, but suggests that the values should be reduced or the average service life to a preventive maintenance activity increased.

The crush and shape with HMA surface structural category was found to have the lower DI values and better performance than for pavements in the new construction or reconstruction category. Most of the crush and shape structures, however, are located in the northern part of Michigan with lower traffic volumes. The analysis did not determine which factor was the more important one contributing to this finding.

What we recommended

Preventive Maintenance

The preventive maintenance policies and strategies that have been used by MDOT should be continued. The only exception to this recommendation is the use of chip seals. The average service life of chips seals was found to be 3 years. It was recommended that MDOT restrict the use of chip seals to specific low volume roads with adequate structural support, and sponsor a materials research study for improving their performance.

Longitudinal Construction Joint Specification

Extensive longitudinal centerline cracking was observed on 100 percent of the projects falling in the group with poor performance. The amount and severity of centerline cracking can be reduced by improving on the construction and compaction of the longitudinal construction joint. Implementation of a longitudinal construction joint specification is considered a high importance mitigation strategy to MDOT and industry in terms of extending the service life and reducing LCCs of flexible pavements, and should be implemented immediately.

Biased Sampling and Testing During Construction

Nearly all projects falling in the category with poor performance exhibited excessive center lane longitudinal cracking. These cracks are more related to the paving equipment and construction practice. Implementation and use of biased sampling and testing methods is considered a high importance mitigation strategy to MDOT and industry to reduce the number of projects with accelerated aging and deterioration. A draft set of guidelines for biased sampling and testing was included in the implementation plan, which includes the purchase of infrared cameras. It was also recommended that this mitigation strategy be implemented immediately.

Wearing Surface with Enhanced Mixture Properties

Transverse cracks and tears, alligator cracks, and longitudinal cracks in the wheel path were also recorded for all projects falling in the category with poor performance, especially those with higher traffic volumes. Many of these projects also had excessive levels of raveling or surface deterioration. The length and severity of these cracks and surface deterioration can be reduced by using higher quality wearing surfaces, like SMA and PMA mixtures. Specifying the use of SMA and PMA mixtures with enhanced mixture properties on higher volume roadways is considered important to extend the service life of flexible pavements and HMA overlays.

Revision to HMA Mixture Design Procedure

Transverse, longitudinal (edge and wheel path), and block cracking were found to be common distresses recorded in the distress index database for roadway segments with poor performance. These cracks are characteristic of high stiffness, low strength HMA mixtures relative to the supporting layers. These cracks can be reduced economically by designing HMA mixtures that are more tolerant to tensile strains, rather than increasing the thickness of the HMA layers. Lowering the number of N_{design} gyrations for mixture design and revising the aggregate blend or gradation for dense-graded, neat HMA wearing surfaces is considered a high importance mitigation strategy to reduce the number of projects with accelerated aging and deterioration.

Fundamental Performance Test

A long term recommendation is to include the use of a fundamental test in the HMA mixture design stage. The purpose of this mitigation strategy is to select and use a fundamental performance test for confirming the mixture design using volumetric properties to select the target asphalt content and job mix formula. It was also recommended that this strategy be implemented, but only after the other mitigation strategies have been completed.

Other Recommendations

Other recommendations from the research study are listed below:

- Add an additional column in MDOT's performance database for the specific type of mixture being placed on the roadway. This mixture information will be needed to confirm the enhanced performance of SMA and PMA mixtures and aggregate blend or gradation.
- Implementation and use of the deterioration relationships that were included in the research study and used to predict the age at which the threshold or critical value is exceeded for the different performance indicators being monitored by MDOT.

EXTENDING THE LIFE OF ASPHALT PAVEMENTS

Michigan DOT Project # OR09086A

PART II

Implementation Plan:

Mitigation Strategies & Demonstration/Pilot Projects

Products P1 and P3

May 2011

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Implementation Plan:

Mitigation Strategies and Demonstration/Pilot Projects

Section 1: Introduction

This report presents the mitigation strategies and demonstration/pilot projects that are recommended to enhance performance and reduce the occurrence of pavements exhibiting accelerated aging or deterioration. The report is grouped into two parts, following the introduction; (1) mitigation strategies and (2) demonstration/pilot projects. The mitigation strategies are those items/features that can be implemented in a reasonable amount of time to extend pavement service life and/or reduce accelerated aging. The demonstration/pilot projects provide additional data to increase the understanding of the mitigation strategy and its effect on construction and performance prior to implementation. The pilot projects also demonstrate the value and effectiveness of the mitigation strategy, where appropriate.

The following summarizes the mitigation strategies and demonstration/pilot projects recommended for enhancing flexible pavement performance. The first three are considered high priority mitigation strategies that can have a significant impact on improving flexible pavement performance without significantly increasing construction costs. Table 1 summarizes some of the details about each mitigation strategy. [Table 1 is located at the end of the Introduction; page 8.] It is recommended that MDOT discuss and debate these mitigation strategies with industry to obtain their support.

1. ***Develop and Enforce a Longitudinal Construction Joint Specification***. All projects with poor performance exhibited excessive longitudinal centerline cracking. This mitigation strategy is being recommended to reduce the length and severity of longitudinal centerline cracking and deterioration (raveling) along the construction joint.

   Nearly all of the DOT individuals interviewed identified the centerline construction joint as being a major concern based on their experience. This helps confirm the observation from the distress data that longitudinal centerline cracking was very prominent on most of the projects with high distress index (DI) values.

   A demonstration project is recommended for this mitigation strategy to confirm the specification values and to obtain industry support.

2. ***Reduce Number of Gyrations to Determine Target Asphalt Content of HMA Mixtures***. All projects with poor performance were found to exhibit longitudinal cracking in the

wheel path and transverse cracking and tears. Excessive longitudinal cracking in or adjacent to the wheel paths and transverse cracks are characteristic of high stiffness-brittle, and/or low strength HMA mixtures, relative to the supporting layers. Reducing the number of gyrations during mixture design can increase the effective asphalt content by volume, which has an effect on mixture durability and resistance to cracking, especially for the lower volume roadways that are thinner and usually have higher deflections.

Multiple agencies have already lowered the number of gyrations for selecting the target asphalt content and job mix formula. Revising the mixture design criteria should improve the mixture's resistance to cracking for both low and high volume roadways; reducing the amount and severity of longitudinal cracks in the wheel path, edge cracks, and transverse cracks.

A pilot project is recommended for this mitigation strategy because any changes to the mixture design procedure and/or criteria will take time to implement. In addition, the pilot projects provide supporting data to confirm the effect on the HMA mixture's volumetric properties that are used for acceptance and payment.

3. ***Biased Inspection and Testing HMA***. Many projects with poor performance exhibited excessive center lane longitudinal cracking. Longitudinal cracking in the center of the lane is not related to the HMA mixture itself or the pavement structure. These cracks are more related to the paving equipment and construction practice.

   It is expected that this cracking is a result of an inadequate amount of mixture being pushed under the paver gear or drive box; sometimes referred to as center lane segregation. An economic and effective method to reduce the occurrence of these longitudinal cracks is to conduct density tests and visual inspection at the center of the paver during the first couple of days of paving and then on an as needed basis as directed by the project engineer. As such, biased inspection and testing is recommended to reduce the length and severity of longitudinal cracks in the center of the lane.

   A few agencies (for example; Washington DOT) already use biased testing to identify areas with temperature differences (sometimes referred to as temperature segregation), while more agencies are considering biased sampling and testing on a routine basis. An infrared camera or sensors can be used to identify areas with a significant loss of temperature during paving.

   A demonstration project is recommended for this mitigation strategy, but only to illustrate use of these procedures for improving construction and performance of HMA pavements.

4. ***Use of Wearing Courses or Surfaces with Enhanced Mixture Properties***. All projects with poor performance were found to exhibit transverse cracks and tears and other forms of cracking and surface deterioration. The Asphalt Institute and other agencies (for example; Colorado and Wisconsin DOT) have sponsored studies related to the use of polymer modified asphalt (PMA) and stone matrix asphalt (SMA) to enhance pavement performance and reduce pavement distress. The MDOT database does not identify those projects where PMA or SMA type engineered mixtures were placed.

   MDOT has allowed the use of gap-graded, neat or unmodified HMA mixtures for the wearing surface. Gap-graded HMA mixtures can exhibit high permeability because of the higher portion of larger aggregate in the aggregate blend. Higher permeability mixtures are more susceptible to accelerated aging and moisture infiltration, which increase surface deterioration of the mixture and reduce its resistance to cracking. The intent of this mitigation strategy is to reduce the amount and severity of various types of cracking (block, fatigue, transverse cracks and tears, etc.) and surface deterioration (raveling).

   No pilot project is suggested for this mitigation strategy because there is a lot of field and laboratory data that document the benefit and reduction in surface distress with the use of PMA and/or SMA mixtures. However, it is recommended that MDOT identify projects with PMA and/or SMA mixtures so that the DI, rut depth, and IRI can be monitored over time in comparison to those with conventional neat HMA mixtures to confirm the increase in service life for life cycle costs analysis.

5. ***Use of a Fundamental HMA Mixture Test.*** It has been reported by multiple researchers that volumetric properties by themselves do not ensure an HMA mixture has the properties required to meet the design requirements (service life). Insufficient data were available to estimate the benefit of using a performance test to identify inferior mixtures and to confirm the job mix formula and target asphalt content based on volumetric properties. The authors, however, recommend its use based on the results from other studies and projects.

   A pilot project is recommended for this mitigation strategy because any changes in the mixture design procedure and/or criteria will take time to implement. Additional data will be needed to confirm the HMA properties used in design and support the volumetric mixture design procedure. This mitigation strategy is a long term effort and a continuation of mitigation strategy #2 – Revised HMA Mixture Design Criteria. The fundamental test used for mixture performance testing can be selected or quantified in accordance with the work completed under mitigation strategy #2.

Table 1. Summary of the Mitigation Strategies

Mitigation Strategy	Objective or Purpose	Importance	Important Feature	Impact on Construction Cost	Time for Implementation
Develop, Enforce Longitudinal Construction Specification	Reduce length & severity of centerline cracks & deterioration.	High; impact should be immediate.	None, immediate implementation	None.	2012 construction season
Reduce Gyrations to Estimate Target Asphalt Content & Job Mix Formula	Reduce length & severity of transverse cracks, longitudinal cracks in wheel path & along the edge.	High; impact will take a couple of years	Laboratory experiment is required for implementation	Minor increase in cost.	2012 for the lab experiment & initial pilot project; 2013 construction season for evaluating performance.
Biased Inspection & Testing of HMA	Reduce length & severity of longitudinal center lane cracks.	High; impact should be immediate	Purchase infrared cameras	None.	2012 construction season
Use Wearing Surface with Enhanced Properties; PMA & SMA	Reduce severity of transverse cracks & tears; longitudinal cracks in wheel path & alligator cracks	Moderate; impact will be immediate on higher volume roadways	None, immediate implementation	Increase in cost.	2012 construction season to implement; performance based tests need to confirm reduction in distress.
Use Fundamental Performance Test for Design	Reduces all distresses.	Moderate; impact will take time.	Long term strategy after others are completed	Increase in cost.	Future development & work.

Section 2: Mitigation Strategies

Product P1

The mitigation strategies recommended are activities or features that can be implemented within one or two years to reduce accelerated aging and deterioration (premature failures) and extend the service life of flexible pavements and HMA overlays. Identification of these mitigation strategies was based on a review of the data included in MDOT's performance database and from discussions with MDOT and industry staff, as well as personnel knowledge of the authors related to flexible pavements materials and construction practices in Michigan.

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Mitigation Strategy #1:

Implementation of a Longitudinal Construction Joint Specification

Introduction

Longitudinal centerline cracking was recorded on 100 percent of the projects exhibiting poor performance. Figure 1 is an example of excessive longitudinal cracking and deterioration along the centerline joint. This cracking and deterioration is directly related to the construction of the centerline joint. Echelon paving is the best strategy to eliminate longitudinal construction joints, but echelon paving is impractical for routine paving of multi-lane roadways; especially for rehabilitation projects for which existing traffic flow must be maintained.

The amount and severity of centerline cracking can be reduced by improving on the construction and rolling of the centerline joint and joint between adjacent lanes in the same direction. Many agencies have already developed and implemented a longitudinal construction joint specification because of the joint's impact on pavement maintenance and performance. It is understood that MDOT drafted a longitudinal construction joint specification in 2009, but that specification has yet to be implemented or included in any pilot study.

Implementation of a longitudinal construction joint specification is considered a high importance mitigation strategy to MDOT and industry in terms of reducing life cycle costs of flexible pavements. This mitigation strategy can reduce the length and severity of longitudinal centerline cracks without increasing construction costs.

Purpose or Objective of Mitigation Strategy

Proper rolling patterns for compacting a confined and unconfined longitudinal construction joint are available in various HMA construction courses and documents (NHI Course #132032, Hot Mix Asphalt Construction; various NAPA, Asphalt Institute, and FHWA courses). There are different opinions within industry, however, regarding the most effective rolling pattern to achieve higher densities along the centerline joint. The objective of this implementation strategy is two-fold:

1. Provide evidence to MDOT and contractors that the longitudinal construction joint specification will not result in significant penalties.
2. Provide data for confirming the values included in a Percent Within Limits (PWL) type of specification, as well as a contractors quality control plan.

Figure 1. Photograph Showing an Example of Accelerated Deterioration Along a Longitudinal Centerline Joint that was Inadequately Constructed

A demonstration project is recommended to achieve the second objective prior to implementation. MDOT, however, can decide to proceed with the values (percent density level and associated penalty or bonus) originally included in the draft longitudinal construction joint specification. The data from this part of the study would simply be used to confirm those values. It is expected that sufficient data from the demonstration project can be obtained from the 2012 construction season. MDOT is encouraged to proceed with this mitigation strategy.

Suggested Changes to the Longitudinal Construction Joint Specification

As noted above, MDOT drafted a longitudinal construction joint specification in 2009 but that specification has yet to be implemented or included in any pilot study. The following are suggested changes to that draft specification.

- It is recommended that the specification be included in some demonstration projects during the first construction season of implementation to demonstrate and confirm the “Best Practices and Methods” for rolling longitudinal construction joints to achieve the maximum density along the joint relative to the density achieved near the center of the

HMA mat. In addition, the gradation of the HMA mixture can have a significant effect on the joint density along notched wedge type joints. The surface voids and/or surface texture of the notched wedge can result in low densities that are mix dependent and not a result of the contractor's standard care and workmanship. Demonstration project #1 under Section 3, Demonstration/Pilot Projects, provides details regarding the demonstration projects to be used.

- Longitudinal joints shall be compacted to a target density of 91 percent of the theoretical maximum specific gravity (G_{mm}) or 2 percent less than the density obtained within the center of the HMA mat. The theoretical maximum specific gravity used to determine the joint density will be the average of the daily theoretical maximum specific gravity for the material that was placed on either side of the joint. The target density of 91 percent of G_{mm} will be evaluated during the construction season for implementing the longitudinal joint specification, but may be increased to 92 percent in future construction seasons. The longitudinal joints of each lift shall be tested separately – the joints for each lift shall be tested.
- Any area or lot with an average joint density less than 88 percent of G_{mm} will be considered unacceptable.
- If a layer or lift of HMA has joints constructed on both sides of the lift, incentive and disincentive payment for each of those joints will apply to one half of the HMA lift between the joints.
- In areas that include intersections and other areas requiring phasing and traffic traveling over the longitudinal joint before the adjacent lift is placed, the Engineer can waive the requirement for joint density testing.
- When constructing joints in an echelon paving process, the longitudinal joints shall be marked to ensure consistent coring locations.
- Six inch diameter cores shall be taken at the locations designated and marked by the Engineer. The center of the core shall be within 1 inch of the visible joint line, which is marked by the Engineer in designating the core location along the longitudinal joint. The contractor can take additional cores at his own expense.
- A calibrated nuclear or non-nuclear density gauge can be used by the contractor to judge the density and compaction of the longitudinal joint after finish rolling – prior to receiving the test results within four calendar days after the Engineer has taken possession of the cores at the project site. If the test results are considered low, the contractor shall notify the Engineer to accelerate testing of the cores.

Performance Indicator to be Monitored

It is hypothesized that the length and severity of longitudinal centerline cracks can be reduced by including joint density in the construction specification. Reducing the length and severity of longitudinal centerline cracks will delay the occurrence of a distress index (DI) value requiring some type of rehabilitation and/or preventive maintenance. Thus, density needs to be monitored during construction, and the length of centerline cracks and DI values need to be monitored over time to achieve the objectives of this mitigation strategy. Implementing this mitigation strategy should have no impact on the IRI values and rut depths recorded in the MDOT database.

Criteria for Project Selection and Number of Projects

The criterion for selecting projects to demonstrate this mitigation strategy is not restrictive. Basically, all projects that have extensive lengths without intersections can be considered. It is suggested that areas with intersections be avoided because of cross over traffic over time. Projects with more than 6 days of paving are also recommended to ensure that lots with different rolling patterns of the longitudinal construction joint can be included in the demonstration.

The sampling matrix for projects included in this mitigation strategy consists of two major factors or tiers which are listed below and shown in Figure 2 – the recommended sampling matrix or experimental factorial.

1. HMA overlay and new construction or reconstruction projects. It is suggested that both rehabilitation (overlays) and new construction type projects be included within the sampling matrix. Type of pavement structure should be kept separate within the sampling matrix, even though pavement type should have no impact on the results from the demonstration projects.

2. Type of longitudinal construction joint. Two major types of longitudinal construction joints should be included in the sampling matrix, because of the need to open the highway to traffic during construction: butt and tapered joints.

    a. Butt joints are more common to HMA overlays with lift thicknesses of 2 inches or less. HMA lifts with thickness greater than 2 inches are usually tapered, because of safety issues in opening the roadway to traffic prior to placing the lift in the adjacent lane. Two types of butt joints have been used to evaluate the performance on longitudinal construction joints; a standard butt joint created by the paver's end plate and a sawed butt joint. The sawed butt joints increases construction costs because of the added equipment and time that is needed for sawing along the HMA mat's edge to remove the edge material. Sawed butt joints are more commonly used for airfield paving projects, where there is more time prior to opening the facility to traffic and heavier loads operate across the joints.

Sawed longitudinal construction joints are typically not used on roadway projects because of the need to open up the roadway to traffic prior to placing the adjacent mat. For this reason, it is recommended that only butt joints created by the paver be included in the demonstration project.

b. Tapered longitudinal construction joints are needed for safety purposes when the HMA lift thickness exceeds 2 inches and the contractor is required to open the lane to traffic prior to placing the adjacent lane. Two types of tapered joints are recommended for use in the sampling matrix or factorial. The first tapered joint is a standard taper, and the second is referred to as the notched edge or wedge joint. The notched wedge joint has a flatter taper (1:12 slope) than the standard tapered joint.

Between Project Parameters		Within Project Parameters			
Type of Construction	Type of Longitudinal Construction Joint	Type of Roller			
		Pneumatic Rubber-Tired Roller	Steel Wheel Roller; Rolling Patterns dependent on confined or unconfined lift placement.		
			1	2	3
New or Reconstructed Flexible Pavement, Includes Crush & Shape with HMA Surface	A-Butt				
	B-Standard Taper				
	C-Notched Wedge Joint				
HMA Overlay of Flexible Pavements [Joint type & lift thickness dependent on opening unconfined joint to traffic.]	A-Butt				
	B-Standard Joint				
	C-Notched Wedge Joint				

Figure 2. Suggested Sampling Matrix for Implementing a Longitudinal Construction Joint Specification and Confirming the Specification Values

Three to four projects for each joint type should provide sufficient data and information to confirm the specification values and provide confidence to MDOT and industry on the proper rolling pattern to maximize joint density. The butt joint is probably the more common joint used for new construction, while the notched wedge joint is more common for HMA overlays. Thus, 6 to 8 projects within the 2012 construction season should be sufficient. For the within project parameters, three lots per rolling pattern should be sufficient to evaluate the null hypothesis (rolling pattern has no impact on density of the longitudinal construction joint).

HMA lift thickness is a secondary parameter included in the sampling matrix. The purpose or reason for adding lift thickness to the sampling matrix is to confirm the effect of the tapered joint on the longitudinal construction joint density specification. Most agencies have found that lift thickness is not a factor in defining the density for a longitudinal construction joint.

Two or more lanes being paved in the same direction but at different times can be used to increase the amount of data collected within a particular project. Thus, projects within multiple lanes in the same direction can be given a higher priority to increase the amount of data collected on any one demonstration project. Other parameters that should be varied within a particular project are listed below.

- Type of roller used in the primary or breakdown position. Vibratory rollers and rubber tired pneumatic rollers. Most of the HMA construction courses for rolling an unconfined longitudinal joint are similar. The differences in rolling strategies are related to the use of the steel wheel rollers (static and vibratory modes). Thus, the sampling matrix is structured to determine the rolling pattern that will result in the highest density.

- Type of rolling pattern used for the longitudinal construction joints. Different rolling patterns are recommended for use by different organizations when steel wheel rollers are used in the breakdown or primary position. Three different rolling patterns should be evaluated within the sampling matrix or demonstration for rolling the longitudinal joint. The following are the common ones used and depend on whether the roller operator is compacting an unconfined or confined longitudinal joint.

    - Unconfined Joint: Two locations are recommended for use during the first pass of the steel wheel roller along the joint (static or vibratory modes).

        - The first and preferred location of the first roller pass along the joint – the edge of the steel drum is extended 4 to 6 inches over the edge of the lift.

        - The second location of the first roller pass along the joint – the edge of the steel drum is adjacent to the edge of the lift; in other words, no overhang of the roller over the edge of the lift.

    - Confined Joint: Three locations are recommended for use during the first pass of the steel wheel roller along the longitudinal construction joint (static or vibratory modes dependent on location of roller for the first pass).

- The first and preferred location of the first roller pass along the joint – the roller is operated on the hot side of the joint and overhangs the edge of the lift by 4 to 6 inches (static or vibratory modes).
- The second location of the first roller pass along the joint – the roller is operated on the cold side of the joint for the first pass; only about 6 inches of the roller is operated on the hot side of the mat. This is defined as the cold side pinch method (static mode only for the first pass).
- The third location of the first roller pass along the joint – the roller is operated on the hot side of the joint but the first pass is located about 4 to 6 inches from the longitudinal joint on the hot side. This is referred to as the hot side pinch method. The second pass of the roller is typically over the part not rolled during the first pass (static or vibratory modes for both passes).

Number of lifts placed to evaluate the effect of staggering longitudinal construction joints can be a secondary parameter of the sampling matrix. Recommended practice is to stagger or offset the longitudinal joints between the upper and lower lifts by 12 inches. Staggering longitudinal construction joints is done so that there is no weakened plane (a cold joint) from the top of the pavement to the bottom of the HMA layer. This secondary factor will be difficult to include in the sampling matrix because many of the rehabilitated projects are confined and restricted to existing lane widths for maintaining existing traffic flow. The effect of staggered longitudinal construction joints can be included more easily for new construction or new alignment type projects. Placing a longitudinal construction joint of any lift under or adjacent to the wheel paths of trucks, however, should always be avoided. For this reason, staggering longitudinal construction joints becomes difficult for roadways with confined widths, and thus, was excluded from the demonstration project.

Assessment of Construction and Pavement Performance: Tests and Data Interpretation

Construction Practices and Rolling Patterns

Two types of field tests are recommended for use in monitoring construction and assessing pavement performance or joint condition at the time of construction. These tests include measuring the density and stiffness of the in place mixture along the joint and within the interior of the HMA mat. The frequency and location of these tests are described in the second part of this document – the pilot projects.

1. Densities can be measured with the nuclear or non-nuclear density gauges, as long as they have been calibrated to cores recovered from the HMA during construction. For this

demonstration project, either of the devices can be used. Cores should also be taken along the edge of the pavement (both along unconfined and confined joints) to confirm the air voids and densities. Densities measured along and adjacent to the joints will be used to confirm the specification values for what can be achieved by the contractor using standard care and workmanship.

2. Stiffness measurements are made with the Portable Seismic Pavement Analyzer (PSPA) in accordance with the procedure outlined in NCHRP project 10-65; *NDT Technology for Quality Assurance of HMA Pavement Construction*.[1] Stiffness values are used as a secondary property for comparing the different features/rolling patterns used to compact the longitudinal construction joints.

Stiffness and density measurements should be taken along the joints and at the same location within the interior of the mat for comparing the measured values. The interior measurements provide the information and data to determine the allowable reduction in density along the unconfined and confined side of the longitudinal joint.

Data from the field stiffness and density tests, along with the cores, are analyzed to determine statistical differences between the different rolling patterns used to compact the longitudinal construction joint. The null hypothesis for this mitigation strategy is that the different rolling patterns for the unconfined and confined longitudinal joints do not have an effect on the density or stiffness measured along the edge of the mat. Sufficient tests should be taken to evaluate the null hypothesis and confirm the values included in the draft longitudinal joint construction specification prepared by MDOT in 2009. It is expected that the null hypothesis will be rejected, and a preferred rolling pattern identified for the longitudinal construction joints.

Performance of Longitudinal Construction Joints

Distress surveys should be completed at periodic intervals to monitor the condition of the joint (centerline cracking length and severity) over time. The project should be divided into lots for acceptance using the MDOT standard procedures and practice. The distress surveys can be completed in accordance with MDOT standard procedures. Each lot should be monitored to determine the impact of rolling pattern on long term performance, as well as type of longitudinal construction joint. These lots can be monitored over a period of at least 5 years to confirm the lower DI values and preferred rolling pattern identified during construction.

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[1] *NDT Technology for Quality Assurance of HMA Pavement Construction*, Report Number 626, National Cooperative Highway Research Program, Transportation Research Board of the National Academies, Washington, DC, January 2009 (Harold L. Von Quintus, Chetana Rao, Robert E. Minchin, Soheil Nazarian, Kenneth Maser, and Brian Prowell).

Mitigation Strategy #2:

Implementation of Revised HMA Mixture Design Criteria

Introduction

Transverse, longitudinal (edge and wheel path), and block cracking are common distresses recorded in the distress index database on those roadway segments with poor performance. In fact, extensive lengths of transverse cracks, longitudinal edge and wheel path cracks were recorded on just about all of the projects classified with poor performance. Conversely, the flexible pavements identified as having good to exceptional performance exhibited significantly less transverse cracks and tears, longitudinal cracks in the wheel path, and block cracking. Longitudinal cracks adjacent to the wheel path cracks and transverse cracks were also noted by some of the MDOT individuals contacted or interviewed as causing premature maintenance.

Pavements with excessive transverse and longitudinal cracking were not restricted to colder climates or MDOT regions, soil type/strength, or traffic level so it was concluded that these cracks are more of a materials issue rather than a climate, traffic, or structural issue. Excessive longitudinal cracks in the wheel path and along the edge and transverse cracks are characteristic of high stiffness, low strength HMA mixtures relative to the supporting layers. Reducing the number of gyrations during mixture design can increase the effective asphalt content by volume, which has a significant effect on mixture durability and its resistance to cracking, especially for lower volume roadways that are thinner and have higher deflections. A mitigation strategy is recommended to minimize the occurrence of material related transverse and longitudinal cracking and is:

- Reducing the number of gyrations for mixture design and revising the HMA mixture design criteria for both higher and lower volume roadways to increase mixture strength and durability; and make the mixture more tolerant to tensile strains.

Multiple agencies have already lowered the number of gyrations for selecting the target asphalt content and job mix formula. Some of these agencies observed that cracking and deterioration of wearing surfaces occurred on a higher percentage on HMA mixtures designed using high levels of N_{design} gyrations.

A pilot project is recommended for this mitigation strategy, because any change in the mixture design procedure and/or criteria will take time to implement. In addition, a pilot project is required to provide data to confirm the effect on the HMA mixture's volumetric properties that are used for acceptance and payment. Simply lowering the number of gyrations without checking the fundamental properties is not recommended because of the potential impact on rutting and other distresses (refer to Mitigation Strategy #5).

Implementation of revised mixture design criteria is considered a high importance strategy to MDOT and industry because it will reduce the number of premature failures and extend the service life of flexible pavements. The strategy may increase construction costs because of higher asphalt contents and potential effects on the aggregate blend or gradation.

Purpose or Objective of Mitigation Strategy

All projects with poor performance were found to exhibit various forms of longitudinal cracking (wheel path and edge) and transverse cracking. The intent of this mitigation strategy is to reduce the length and severity of longitudinal (wheel path and edge) and transverse cracks and tears.

The hypothesis of this mitigation strategy is that some HMA mixtures are susceptible to fracture because of lower asphalt contents. Lower asphalt contents can reduce the tensile strength of HMA and result in brittle mixtures. Higher laboratory compaction efforts can result in lower effective asphalt contents by volume. Revising the mixture design guidelines and laboratory compaction criteria should improve on the mixture's resistance to cracking for both low and high volume roadways.

Revised Mixture Design Criteria

MDOT requested specific implementable recommendations regarding the revised gyration levels and aggregate blends for the different conditions and HMA layers. This request goes beyond the scope of work for this project. The intent of this mitigation strategy is to balance a mixture's resistance to distortion, fracture, and surface disintegration. Based on the findings from this study, excessive rutting has all but been eliminated, but at the expense of making some mixtures more brittle and susceptible to fracture which is dependent on the site conditions. To balance between distortion and fracture properties for extending pavement life requires the use of fundamental performance-based properties (refer to Mitigation Strategy #5).

A pilot project is recommended for this mitigation strategy to confirm the effect of changing the mixture design criteria (refer to Pilot Project #3 and Figure 3 in this section). More importantly, revisions for this mitigation strategy are volumetric-based properties. Volumetric properties are important, but their overall effects on the fundamental properties of the HMA mixture can be altered by using different amounts and combination of materials (for example, mineral filler, sand, etc.). Fundamental performance-based property tests are needed to confirm the HMA mixture design will be resistance to fracture and distortion.

The following provides suggested changes to MDOT's current Superpave mixture design criteria or guidelines that can be used as a starting point for evaluating their effectiveness for extending pavement service life.

NOTE: IF MDOT DOES NOT PLAN TO USE A FUNDAMENTAL PERFORMANCE-BASED PROPERTY TEST TO CONFIRM THE DIFFERENT GYRATIONS AND HIGHER VMA VALUES, IT IS SUGGESTED THAT THE VALUES IN THE CURRENT MIXTURE DESIGN GUIDELINES CONTINUED TO BE USED.

Number of Gyrations for Selecting Target Asphalt Content:

Estimated Design Traffic (million ESALs)	Number of Gyrations, N_{Design} at 96% of G_{mm}	Comments
< 0.1	50	
0.1 to 0.3	50	Some agencies have increased this value to 75.
0.3 to 3.0	75	
3.0 to 10.0	75	For the heavier traffic volumes, some agencies have made this value dependent on climate. The cooler areas are 75, and the hotter areas can be increased to 100.
10.0 to 30.0	90	This value can also be climate dependent; varying from 90 to 100.
> 30.0	115	This value can also be climate dependent, varying up to 125 for hot climates.

Voids in Mineral Aggregate for Selecting or Determining Aggregate Gradation:

Nominal Maximum Aggregate Size (As Defined by Superpave)	Minimum Voids in Mineral Aggregate, %
1 inch (25 mm)	12.5
¾ inch (19 mm)	13.5
½ inch (12.5 mm)	14.5
3/8 inch (9.5 mm)	15.5

Performance Indicator to be Monitored

It is hypothesized that the length and severity of longitudinal cracks adjacent to the wheel path and transverse cracks can be reduced by making revisions to the mixture design procedure to increase the mixture's resistance to fracture. Reducing the length and severity of longitudinal and transverse cracks will delay the occurrence of a distress index value requiring some type of rehabilitation and/or preventive maintenance. Thus, the length of longitudinal cracks adjacent to the wheel path and along the edge, transverse cracks, and distress index values need to be monitored to achieve the objective.

Implementing this mitigation strategy may have an impact on the IRI values and rut depths recorded in the MDOT database. As such, other distresses, rut depth, and IRI should be monitored for at least 5 years to confirm the increase in service life (lower DI values, rut depths, and IRI).

Criteria for Project Selection and Number of Projects

The criterion for selecting projects included within this mitigation strategy is that the project needs to have a sufficient amount of HMA paving so that mixtures can be designed and placed using two different design criteria: the existing mixture design procedure defined as the standard sections and the revised mixture design criteria based on a fewer number of gyrations (lower laboratory compactive effort) defined as the companion sections. The sampling matrix for projects included in this mitigation strategy consists of three multiple factors or tiers which are listed below and summarized in Figure 3.

- Layer type: HMA base layer, intermediate layer, and wearing surface for new construction or reconstruction (including crush and shape with HMA surfaces) and HMA overlays. Layer type is the primary factor, while pavement structure is a secondary factor in the sampling matrix.
- Traffic level: High to low traffic volumes. This primary factor will be used to evaluate the use and impact of number of gyrations on the volumetric and fundamental properties of a particular aggregate blend and aggregate type.
- Aggregate type and blend: Coarse-graded, gap-graded and fine-graded mixtures, and/or small versus large aggregate blends. This factor can be included in the sampling matrix by including pavements with thicker HMA base layers to thinner wearing surfaces. Layer thickness should be compatible with aggregate size because of the minimum lift to nominal aggregate size ratio requirement.

It is recommended that the climate or regional effect on asphalt performance grade selection or determination be kept the same and not be included in the sampling matrix. However, projects should be selected to include different performance grade asphalts that are typically specified and used by MDOT.

For planning purposes, a minimum of 4 overlay projects and 6 new construction projects should be included within this mitigation strategy to determine the appropriate number of gyrations for maximizing performance.

Between Project Parameters		Within Project Parameters			
Traffic Level	Aggregate Blend	HMA Layer, if available	Number of Gyrations		
			Current Level	Revised Level 1	Revised Level 2
Low	Coarse-Graded	Base			
		Surface			
	Gap-Graded	Base	NOTE: The gyration levels selected and used should be based on preliminary studies; either conducted by MDOT or other agencies.		
		Surface			
	Fine-Graded	Base			
		Surface			
Moderate	Coarse-Graded	Base			
		Surface			
	Gap-Graded	Base			
		Surface			
	Fine-Graded	Base			
		Surface			
High	Coarse-Graded	Base			
		Surface			
	Gap-Graded	Base			
		Surface			
	Fine-Graded	Base			
		Surface			

Figure 3. Suggested Sampling Matrix for Implementing Revised HMA Mixture Design Criteria and Lowering the Number of Gyrations for Design

Assessment of Mixture Design Guidelines and Pavement Performance: Tests and Data Interpretation

Assessment of this mitigation strategy needs to be divided into two parts; one for the laboratory evaluation in determining the target asphalt content and job mix formula, while the second part is the performance evaluation to confirm the reduction in specific longitudinal and transverse cracks, as well as lower distress index values, while not increasing rut depth and IRI – extending the pavement service life.

Mixture Tests for Laboratory Evaluation

The laboratory evaluation is grouped into two subsets.

1. The first subset of test mixtures: all HMA mixtures included in the sampling matrix should be designed with the current mixture design procedure and criteria (N_{design} gyrations). After the target asphalt content and job mix formula have been determined using the existing (or standard) procedure, the fundamental properties should be measured on laboratory prepared specimens at the expected air void level based on the construction specification.

2. The second subset of test mixtures or specimens: the HMA mixture should be compacted using reduced levels of compaction or N_{design} levels. The target asphalt content and job mix formula is determined for the revised compaction levels. The fundamental properties are measured on laboratory prepared specimens at the same expected air void level specified during construction.

Two types of laboratory and field tests are recommended for use in monitoring construction and assessing pavement performance at the time of construction. These tests include the volumetric and fundamental properties of the HMA.

- Volumetric properties include the properties normally measured using the current mixture design process; density, air voids, Voids in Mineral Aggregate (VMA), and Voids Filled with Asphalt (VFA).

- Fundamental performance properties include dynamic modulus, tensile strength and tensile strain at failure using the indirect tensile test (or a measure of the strain energy required to fracture the specimens), and a repeated load permanent deformation test (flow number test).

The deformation tests should be performed on test specimens that have been short term aged, while the fracture tests should be performed on test specimens that have been long term aged. Short term aging is used to evaluate rutting, while long term aging is used to evaluate transverse and longitudinal cracking and other mixture disintegration type distresses. The fundamental tests are used to determine the effect of changing volumetric properties on the performance properties. The frequency and location of these tests are described in the second part of this document – the pilot projects.

MDOT has already sponsored the use of some fundamental tests to characterize HMA mixtures (You, et al., 2009).[2] The two tests included within that study was the dynamic modulus and flow number (or repeated load permanent deformation) tests. Flow number is an estimate of the mixture's resistance to rutting, while dynamic modulus provides some measure of the mixture's resistance to alligator cracking and rutting.

Rutting was not found to be a major issue in terms of premature failures; few roadway segments were found to have excessive rut depths. Longitudinal and transverse cracks were the more predominant distress for roadway segments with inferior performance. As such, MDOT is encouraged to use a practical fundamental test that measures a mixture's resistance to cracking.

The tensile strength and tensile strain at failure or the strain energy of the mixture can be measured using the indirect tensile test. MDOT is encouraged to use a fracture test for evaluating any change in the mixture design procedure (reducing the number of gyrations for design). Dynamic modulus and flow number (the raw data of plastic strain versus number of load cycles and not the flow number) are still beneficial, especially in determining the HMA mixture inputs to the new Mechanistic-Empirical Pavement Design Guide (MEPDG).

Performance of HMA Mixtures Designed Using Different Compaction Levels

Distress surveys should be completed at periodic intervals to monitor the condition of the flexible pavement or HMA overlay over time. The project should be divided into lots used for acceptance based on MDOT standard procedures and practice. Some of the lots of the project should be designed and placed using current mixture design practice (the standard sections), and the others designed and placed using the revised mixture design guidelines (the companion sections).

The distress surveys should be completed in accordance with MDOT standard procedures. Each lot should be monitored to confirm the impact of HMA design criteria on long term performance.

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[2] You, Zhanping, Shu Wei Goh, and Christopher Williams, *Development of Specifications for the Superpave Simple Performance*, Research Report Number RC-1532, Michigan Department of Transportation, Lansing, Michigan, May 2009.

Mitigation Strategy #3:

Implementation of Biased Inspection and Testing During Construction

Introduction

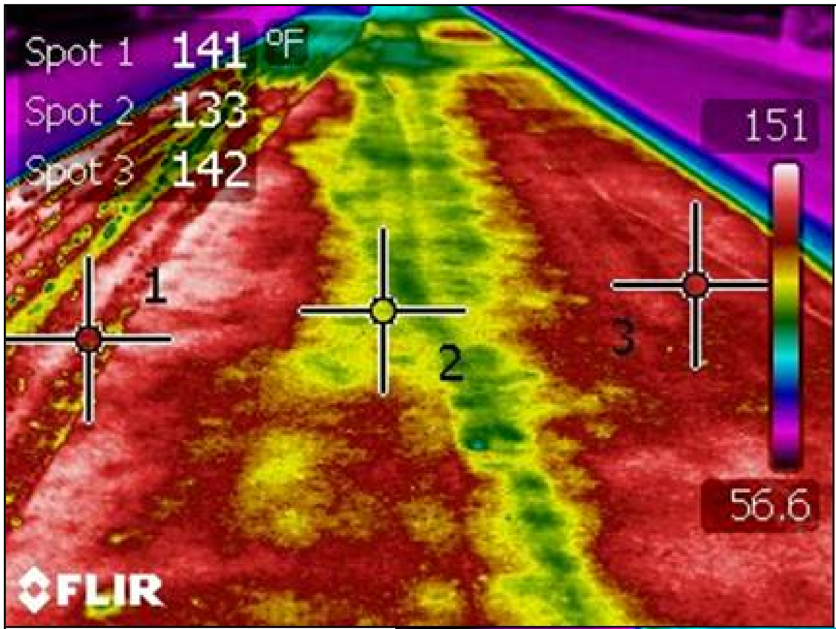
Nearly all projects classified with poor performance exhibited excessive center lane longitudinal cracking. This distress was not identified as a critical issue from the MDOT contacts and interviews, but raveling or mixture disintegration near the center of the lane was identified as an issue. This experience and knowledge helps confirm that a construction defect of segregation or insufficient material at the center of the auger chamber is probably an issue.

Longitudinal cracking in the center of the lane is not related to the HMA mixture itself or the structure. These cracks are more related to the paving equipment and construction practice. It is expected that this cracking is a result of an inadequate amount of mixture being pushed under the paver gear or drive box; sometimes referred to as center lane segregation.

An economic and effective method to reduce the occurrence of these longitudinal cracks is to conduct density tests and visual inspection at the center of the paver during the first couple of days of paving and then on an as needed basis as directed by the project engineer. Biased sampling and testing should identify factors causing center lane cracking during the first day of paving so corrective actions can be taken, if needed. As such, biased sampling and testing is recommended to reduce the length and severity of center lane longitudinal cracks.

A few agencies (for example; Washington DOT) already use biased testing to identify areas with temperature differences (sometimes referred to as temperature segregation). An infrared camera or sensors can be used to identify areas with a significant loss of temperature during paving. Figures 4 and 5 are examples of cold spots that were identified with the infrared camera. Figure 6 is an example showing uniform surface temperatures across the paving lane. Implementation of this mitigation strategy does require the purchase and use of infrared cameras.

No pilot project is recommended for this mitigation strategy to monitor performance. Demonstration construction projects, however, are suggested to illustrate the biased inspection and testing and use of the infrared camera. Implementation of this mitigation strategy should have no impact on construction costs but should extend the service life of flexible pavements. In addition, it should have no impact on the rut depths and IRI values measured by MDOT.



Surface temperatures are measured with the infrared cameras. The infrared images define areas with different surface temperatures at a point in time. If areas with a significant drop in temperature are identified with the infrared camera, other tests or inspection techniques are required to determine the cause of the difference.

These two examples or illustrations from an infrared camera show cold spots along the center of the paver (center lane cold spots). Multiple photos or illustrations can be taken to monitor the change in temperature after paving and/or compaction.

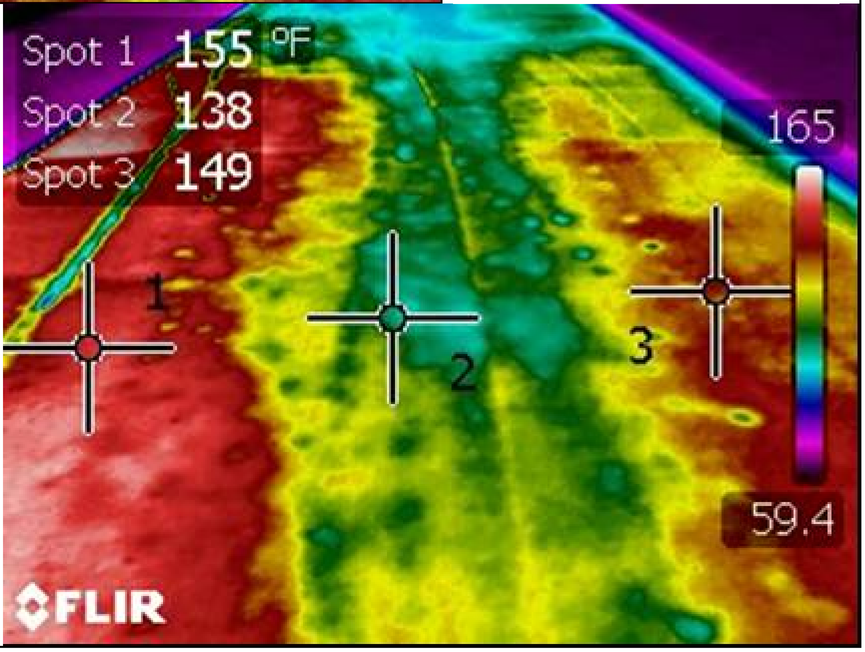
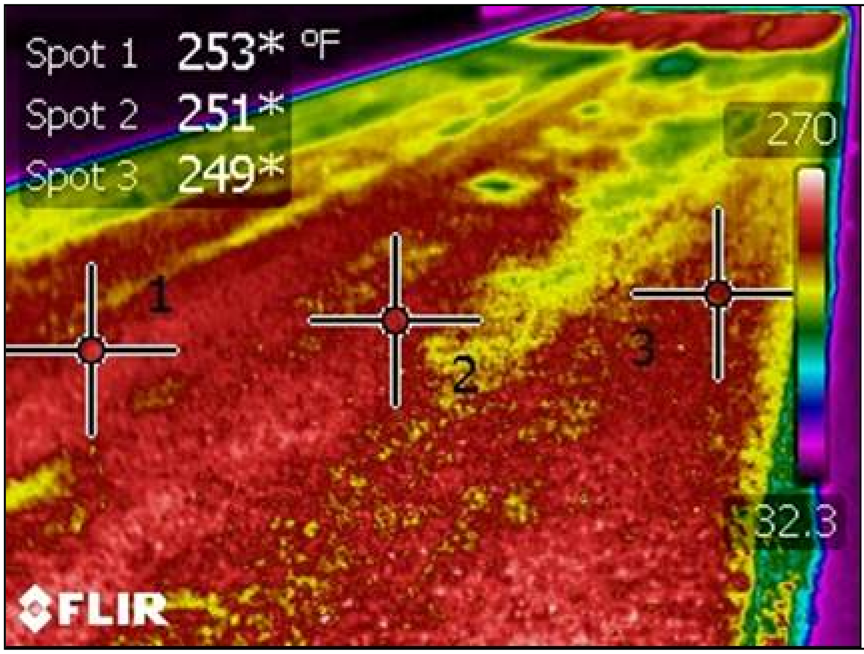
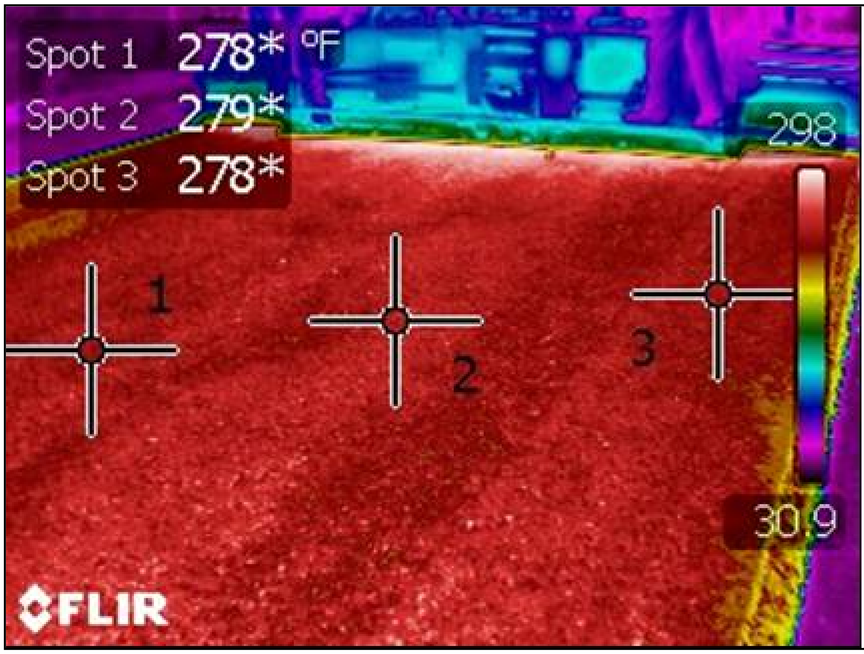


Figure 4. Use of Infrared Camera to Locate Cold Spots or Areas with Low Density; Near Center of Paver (sometimes referred to as temperature segregation)



Infrared photo taken behind the paver prior to compaction and shows truck to truck temperature difference caused by a delay in trucks arriving at the site; the paver had to stop and wait for the next truck.

Figure 5. Use of Infrared Camera to Locate Cold Spots or Areas with Low Density; Delay in Delivery of Mix Where Paver is Sitting for an Extended Period of Time



Infrared photo taken behind the paver showing uniform surface temperatures across the paving lane.

Figure 6. Use of Infrared Camera to Check for Temperature Differences Behind the Paver

Purpose or Objective of Mitigation Strategy

Longitudinal center lane cracking has been attributed to an insufficient amount of mixture in the center of the paver (directly under the paver's gear box) as a result of worn-out or improperly installed kick-back paddles or aggregate segregation. This condition can be easily identified through visual observations and density tests conducted in a specific area – rather than at random locations. Identifying specific areas with insufficient mixture or segregation and taking corrective action can totally eliminate these longitudinal center lane cracks.

Multiple agencies have purchased infrared cameras to assist in identifying and locating these types of construction defects. Some Michigan contractors have already purchased these cameras as part of their quality control programs.

The objective of this implementation strategy is two-fold:

1. Prepare a set of guidelines that can be used by MDOT staff to locate problem areas at the beginning of paving so that corrective actions can be taken by the contractor. [A draft set of guidelines is included at the end of this subsection.]
2. Demonstrate use of infrared camera to identify construction defects near the center of the auger chamber and in other areas of the mat (refer to Figures 4 and 5).

Infrared Camera Recommendations and Guidelines

A demonstration project has been recommended to achieve the second objective prior to implementation. The steps and activities recommended for the demonstration project are included in the next Section 3: Demonstration/Pilot Projects – Product P2. MDOT, however, can decide to proceed with implementing this mitigation strategy on a routine basis. It is recommended that MDOT purchase at least one infrared camera for use in the 2012 construction season to demonstrate the effectiveness of biased sampling and testing. In the future, at least one infrared camera per region is recommended.

Many different cameras are available, but FLIR Systems has the following hand-held models that are suitable for application during paving operations.

- T-300 Series Cameras: Models 300, 360, and 400. These cameras range in price from about $10K to $15K. These were the cameras initially used by Washington DOT in the late 1990's to identify cold spots during paving, which had a resolution of 320x256. In the latter 1990's this camera was priced at nearly $50K.
- P-Series Cameras: Models 620 and 640. These cameras range in price from $28K to $40K.

The primary difference between the T-Series and P-Series cameras is the image resolution. The T-Series have a 320x240 resolution, while the P-Series have a 640x480 resolution and higher. Both camera series can be used by an operator riding in a car or truck or walking behind the paver, or they all can be mounted to the back of the paver on a tripod. In addition, they can be hooked to an onboard device for reading the thermal images in real-time. The following lists some of the criteria that should be specified in purchasing the cameras.

- Accuracy and repeatability (+/- 2 percent or 2 degrees Centigrade [3.6 degrees F]).
- Detector resolution or quality of the image collected and stored in the camera for future use.
- Easy to replace battery or charge on an automobile.
- Outputs image in JPEG format (fully radiometric JPEG, which has temperature information).
- Lightweight and ergonomic (less than 2 pounds).
- Mega pixel visual camera with a built-in illuminator lamp (analogous to a flash in a camera).
- Laser pointer built-in.
- Image fusion capabilities.
- Temperature range suitable for HMA behind the paver (all cameras noted above will exceed the range on paving projects).
- Upgrade potential for the camera, including software upgrade potential.
- Post-sale technical support and warranty.

The following is the draft set of guidelines for biased sampling and testing.

> *During the first day of paving, the inspector shall monitor the paving operation and measure the density in specific areas. The infrared camera should be used to identify "cold spots," if present. Cold spots can be the result of longitudinal and truck to truck aggregate segregation, or an insufficient amount of mixture being placed in selected areas – center of the auger chamber (refer to Figures 4 and 5). One area or location to monitor is the mixture placed at the center of the auger chamber and along the outside edges of the slat conveyor (transferring mixture from the paver hopper to the auger chamber).*

A density reading with a calibrated nuclear or non-nuclear density gauge should be taken at the center of the paver at periodic intervals depending on the length of each sublot during the first day of paving. If the density readings are consistently low, relative to other areas of the mat, paving should be discontinued to determine the reason for the lower density values and corrective action taken.

If no defects or "cold spots" with low density readings are found, paving can continue. The infrared camera should be used over the course of the project to identify potential "cold spots" and/or cores taken to confirm that the material has been adequately compacted.

If conditions change during the course of the project, biased sampling and testing should be performed at the direction of the project engineer.

Performance Indicator to be Monitored

It is hypothesized that the length and severity of longitudinal center lane cracks can be reduced by continuously supplying an adequate amount of mix in the center of the auger chamber and that the HMA has been properly compacted in this area in conformance with the density specification. The location of the test is defined or located using biased techniques, rather than at random at the beginning of the project. Reducing the length and severity of longitudinal center lane cracks will delay the occurrence of a distress index value requiring some type of rehabilitation and/or preventive maintenance. Thus, the length of center lane cracks and distress index values need to be monitored to achieve the objective. Implementing this mitigation strategy should have no impact on the IRI values and rut depths recorded in the MDOT database.

Criteria for Demonstration Project Selection

The criterion for selecting projects included within this mitigation strategy demonstration is not restrictive. Basically, all projects can be considered. The number of projects depends on the number of available infrared cameras. The sampling matrix for selecting projects included in this mitigation strategy for the 2012 construction season consists of two major factors or tiers which are listed below.

1. Lift thickness: less than 2 inches and greater than 2 inches. Lift thickness has a significant impact on the loss of temperature or time available for compaction. Thin and thicker lifts should be included to demonstrate this strategy.

2. Aggregate blend: gap-graded, coarse-graded, and fine-graded. Gap and coarse-graded mixtures are more susceptible to aggregate segregation and should be included in the demonstration project.

It is suggested that at least one infrared camera be purchased for the 2012 construction season. This camera can be used within a specific region or used on specific projects throughout Michigan.

Assessment of Construction: Tests and Data Interpretation

Two types of field tests are recommended for use in monitoring construction and assessing the condition of the HMA lift at the time of construction. These tests include density of the in place mixture measured with a nuclear or non-nuclear density gauge and surface temperature differences measured with the use of the infrared camera to locate cold spots. Cores should also be recovered to confirm and/or calibrate the density readings from the nuclear or non-nuclear density gauge.

Mitigation Strategy #4:

Implementation of Wearing Courses with Enhanced HMA Mixture Properties

Introduction

All projects with poor performance were found to exhibit transverse cracks and tears, alligator cracks and longitudinal cracks in the wheel path. Surface deterioration (raveling) was recorded on over 50 percent of these projects. The amount and severity of these cracks and surface deterioration can be reduced by using higher quality wearing surfaces like stone matrix asphalt (SMA) and polymer modified asphalt (PMA) mixtures. MDOT and local contractors have designed gap-graded or uniform-graded, neat (unmodified) HMA mixtures. These mixtures can have lower asphalt contents and high permeability resulting in durability issues; raveling, block cracking (longitudinal and transverse cracks), and alligator cracking with time.

Discussions with contractors, review of field reports, and observations of surface distress suggest that the Type C mixtures specified and placed in the 1980's were susceptible to premature cracking. This condition has changed with some of the revisions made to the HMA specifications in the latter 1990's and early 2000's. However, there are still many projects where excessive cracking has occurred. It is hypothesized that a cause for this premature cracking is a result of the gap-graded, unmodified HMA mixtures that have been specified and used in Michigan, especially for higher volume roadways. Thus, the intent of this strategy is:

- Use of wearing courses with enhanced mixture and asphalt properties to reduce the length of transverse cracks, block cracking, longitudinal cracks in the wheel path and surface deterioration, or to minimize the use of gap-graded aggregate blends (i.e.; mitigation strategy #2).

MDOT has allowed the use of gap-graded or uniform-graded dense HMA mixtures for the wearing surface. Gap-graded HMA mixtures can exhibit high permeability because of the higher portions of larger aggregate in the aggregate blend. Higher permeability mixtures are more susceptible to accelerated aging and moisture infiltration, which increase surface deterioration of the mixture and reduce its resistance to cracking. The intent of this mitigation strategy is to reduce the amount and severity of various types of cracking (block, alligator, transverse cracks and tears, and longitudinal cracks in the wheel path) and surface deterioration.

The Asphalt Institute and other agencies (for example; Colorado and Wisconsin DOT) have sponsored studies related to the use of PMA and SMA mixtures to enhance pavement performance and reduce pavement distress. The MDOT database does not identify those projects where PMA or SMA type engineered mixtures were placed as the wearing surface. No pilot project is suggested for this mitigation strategy because there are a lot of field and laboratory studies that document the benefit and reduction in surface distress with the use of PMA and/or SMA wearing surfaces. It is recommended, however, that MDOT start recording and documenting the projects where these mixtures with enhanced properties have been used to establish performance characteristics that can be quantified and compared to conventional, neat HMA mixtures for the site features, materials, and other conditions encountered in Michigan.

This mitigation strategy is compatible with mitigation strategy #2. In fact, the results from mitigation strategy #2 can be used to determine the fundamental properties for PMA and SMA mixtures, as compared to the existing HMA mixtures produced and placed under the current construction and material specifications. A fundamental performance test should eventually be used to measure the properties of any HMA mixture, but especially those on higher volume roadways (refer to mitigation strategy #5).

Purpose or Objective of Mitigation Strategy

The objective of this implementation strategy is to provide:

- Documentation and evidence to MDOT and contractors for quantifying the magnitude of the extended service life or reduction in pavement distress with the use of engineered mixtures with enhanced properties (PMA and SMA mixtures).

MDOT is encouraged to proceed with implementing this strategy. Insufficient data, however, exists for quantifying the increase in service life or reduction in distress for conditions encountered and materials used in Michigan. As such, a longer term demonstration project is recommended to achieve the objective during and after implementation of this mitigation strategy. The data from the demonstration project can be used to confirm the expected increase in service life of 3 to 5 years that has been documented and reported by other agencies (Asphalt Institute, Colorado DOT, etc.).

Performance Indicator to be Monitored

It is hypothesized that the amount and severity of alligator cracks, transverse cracks and tears, longitudinal cracks in the wheel path, and surface deterioration (raveling) can be reduced by specifying the use of PMA and SMA mixtures, especially for higher volume roadways. Reducing the amount and severity of these cracks will delay the occurrence of a distress index value requiring some type of rehabilitation and/or preventive maintenance. Thus, all distresses, rut depths, IRI, and the distress index values need to be monitored to achieve the objective. Implementing this mitigation strategy will have an impact on the IRI values and rut depths recorded in the MDOT database; they should stay the same or be lower.

Criteria for Project Selection and Number of Projects

The criterion for projects included within this mitigation strategy demonstration is generally restricted to higher volume roadways. No other site feature or factor should restrict the use of these mixtures or mitigation strategy. It is expected that 12 projects will be needed to estimate the reduction in distress and increase in service life, after the performance based tests are used and confirmed from implementation of mitigation strategy #2.

Assessment of Pavement Performance

Distress surveys should be completed at periodic intervals to monitor the condition of the flexible pavements over time. The distress surveys can be completed in accordance with MDOT standard procedures.

Mitigation Strategy #5:

Implementation of a Fundamental HMA Mixture Property Test to Confirm Performance

Introduction

The last strategy recommended to extend pavement life is to include a fundamental test within the mixture design or confirmation stage. It is expected that industry (contractors, as well as MDOT personnel) may object to this recommendation, and it will take longer to implement. In addition, the strategies previously discussed must first be implemented for this strategy to have any significant impact on extending service life.

It has been reported by multiple researchers that volumetric properties by themselves do not ensure an HMA mixture has the required performance properties to meet the design requirements (service life). A fundamental performance test is recommended to confirm the HMA properties used in structural design and support the volumetric mixture design procedure. This is a long term implementation mitigation strategy. Specifically, this mitigation strategy is compatible with and a confirmation of mitigation strategy #2. This strategy should be implemented after the first three mitigation strategies have been completed. It is also suggested that this strategy be implemented during the implementation and use of the new Mechanistic-Empirical Pavement Design (MEPDG) procedure.

A pilot project is recommended for this mitigation strategy because any changes in the mixture design procedure and/or criteria will take time to implement. This pilot project should be conducted after the other mitigation strategies have been implemented. The reason that the implementation of a fundamental performance test is included as a mitigation strategy is to start the planning process early. In addition, this mitigation strategy should be compatible with the use of the MEPDG for pavement structural design – integrating mixture design, structural design, and quality assurance or construction.

Purpose or Objective of Mitigation Strategy

The objective of this implementation strategy is to select and use a fundamental performance test for confirming the volumetric properties used during the mixture design stage in selecting the target asphalt content and job mix formula, and to predict the behavior and performance of HMA mixtures. In other words, the objective is to integrate structural design, mixture design, and construction (quality assurance/acceptance).

As noted under mitigation strategy #2, MDOT has already sponsored a study for measuring the dynamic modulus and flow number on different HMA mixtures. This laboratory study will be useful in moving forward with this mitigation strategy. However, MDOT is encouraged to consider and use a mixture's resistance to cracking because nearly all of the roadway segments with poor performance exhibited excessive cracking, rather than excessive rutting. The fundamental properties and test mentioned under mitigation strategy #2 should be considered in supporting the volumetric mixture design procedure.

Performance Indicator to be Monitored

All distresses, IRI, rut depths, and distress index values being measured and collected by MDOT for managing the roadway network should be monitored. It is recommended that this performance test be used to assist in calibrating the MEPDG to local conditions and materials.

Criteria for Project Selection

The criterion for selecting projects included within this mitigation strategy should be compatible with the sampling matrix developed for calibrating the MEPDG to local conditions, site features, and materials. This assumes, of course, that MDOT has future plans to adopt and use the AASHTO DARWin-ME version of the MEPDG software.

Assessment of Performance: Tests and Data Interpretation

Distress surveys should be completed at periodic intervals to monitor the condition of the flexible pavements and HMA mixtures included within this mitigation strategy. The mixture performance test and interpretation of the test data is dependent on whether this test or tests will be used in conjunction with the MEPDG. Thus, it is suggested that MDOT consider this mitigation strategy as it prepares plans to evaluate and use the MEPDG.

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Section 3: Demonstration/Pilot Projects

Product P2

This part of the implementation plan provides detailed information for the recommended demonstration and/or pilot and demonstration projects for selected mitigation strategies. Field investigations and testing plans have been prepared for two pilot projects and two demonstration projects. The pilot projects provide additional data to increase an understanding of the mitigation strategy and its impact on construction and performance prior to or during implementation. The demonstration projects illustrate the value and effectiveness of the mitigation strategy that can be immediately implemented. In other words, the demonstration projects provide data to assist in quantifying the benefit.

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Demonstration Project #1:

Longitudinal Construction Joint Specification

Introduction

Excessive lengths and severities of longitudinal centerline cracks are recorded in MDOT's performance database for flexible pavements and HMA overlays with poor performance. The magnitude and severity of the centerline cracks are lower for the pavements and overlays with good to exceptional performance. The implementation of a longitudinal construction joint specification would be beneficial to reduce the length and severity of centerline cracking and lower the distress index.

A draft longitudinal construction joint specification was prepared by MDOT in 2009, but has yet to be implemented.[3] The purpose of the specification is to delay the occurrence of longitudinal centerline cracks for longer periods of time by getting higher densities along the centerline and adjacent lane construction joint. This draft should be implemented immediately. A demonstration project, however, is recommended during implementation of the longitudinal construction joint specification in 2011.

Objective of Project

1. Provide documentation and evidence to MDOT and industry on rolling a longitudinal construction joint and enforcement of the specification will not result in excessive penalties using standard care and workmanship.
2. Provide data to establish (confirm) the testing guidelines for measuring the density along a longitudinal construction joint that can be used for acceptance. In other words, the testing guidelines should specify the locations of where the density tests/cores will be taken relative to the joint.
3. Provide data to confirm the values and limits included in the Percent Within Limits specification.

Experimental Hypotheses

- Null hypothesis related to objective #1: Rolling pattern and joint type have no impact or does not affect the density measured along the longitudinal construction joint. It is expected that the null hypothesis will be rejected from the experimental data.

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[3] Special Provision for the Acceptance of Longitudinal Joint Density in Hot Mix Asphalt (HMA), 2009.

- The longitudinal construction joint specification to ensure a minimum density level will improve performance and reduce the length and severity of longitudinal centerline cracks. To accept or reject this hypothesis requires that the demonstration projects and individual lots (or sublots) be monitored for at least 5 years. MDOT can decide to base the long term performance decision on the density level itself, because HMA density is one of the most important properties related to long term performance.

Experimental Factors

The following lists the primary experimental factors included in the sampling matrix (refer to Figure 2 under Mitigation Strategy #2). These factors are grouped into two types: those that are varied between the projects and those that can be varied within a particular project.

- *Type of Construction*: Projects should be selected to include both new construction and HMA overlays. Type construction should not be varied within a particular project, unless the project includes lane widening and rehabilitation.

- *Type of Joint*: Three types of joints should be included in the sampling matrix (refer to Figure 7); (1) butt joint created with the screed end plate, (2) a tapered joint, and (3) the notched wedge joint. Butt joints and the notched wedge joints are more commonly used in Michigan. Butt joints are used during new construction or for HMA lift thickness less than 2 inches. The notched wedge joint is used for safety reasons when the roadway must be opened to traffic and the HMA lift thickness is greater than 2 inches (refer to Figure 8). It is expected that the type of joint will be kept constant within a particular project, and only varied between projects. Type of joints is expected to have an effect on the final density of the joint.

- *Type of Roller in Breakdown Position*: Both steel wheel rollers and rubber tired rollers can be used in the breakdown position. Steel wheel rollers are the ones more commonly used in the breakdown position in Michigan. It is expected that few projects will be identified where the rubber tired pneumatic rollers are used in the breakdown position. Although the type of roller can be varied within a project, it is suggested that the type of roller used in the breakdown position be kept constant within a specific project.

- *Rolling Pattern*: Rolling pattern is dependent on the type of roller that is used in compacting the joint and whether it is an unconfined or confined. The rolling pattern can be varied along a specific project to reduce the number of projects that are required. It is recommended, however, that the same rolling pattern be used within a specific lot for the project so the roller operator is less likely to get confused about which pattern is needed in a particular lot of the project. MDOT should define the lot size for this experiment to reduce the number of projects and amount of HMA for any particular lot. Rolling pattern is expected to have an effect on the final density of the joint.

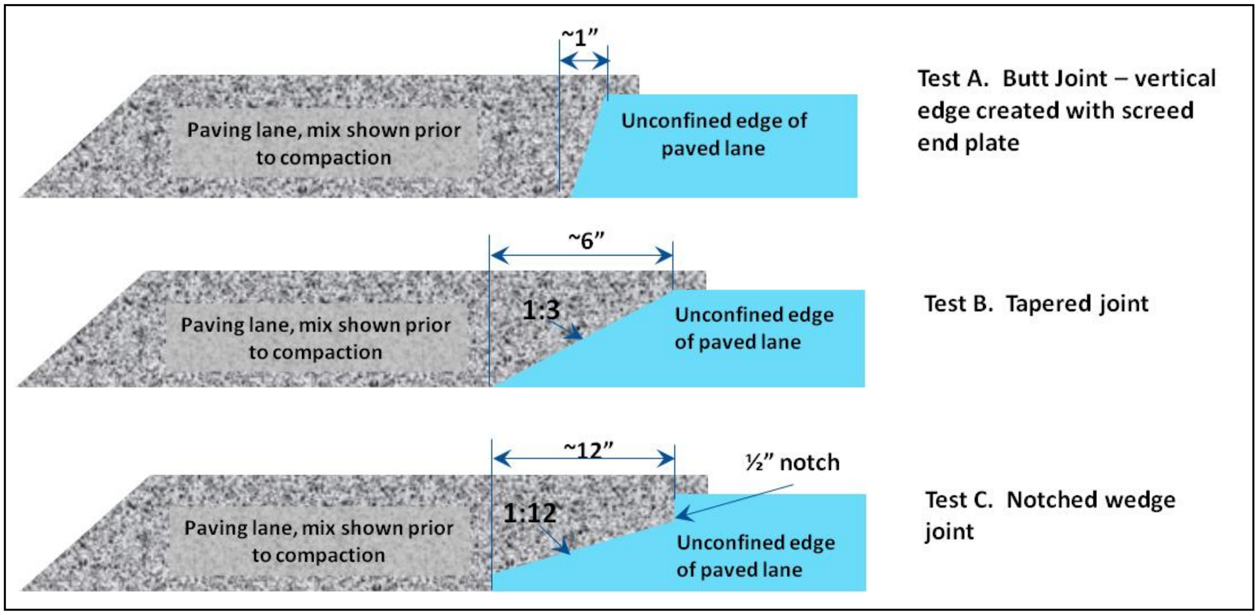
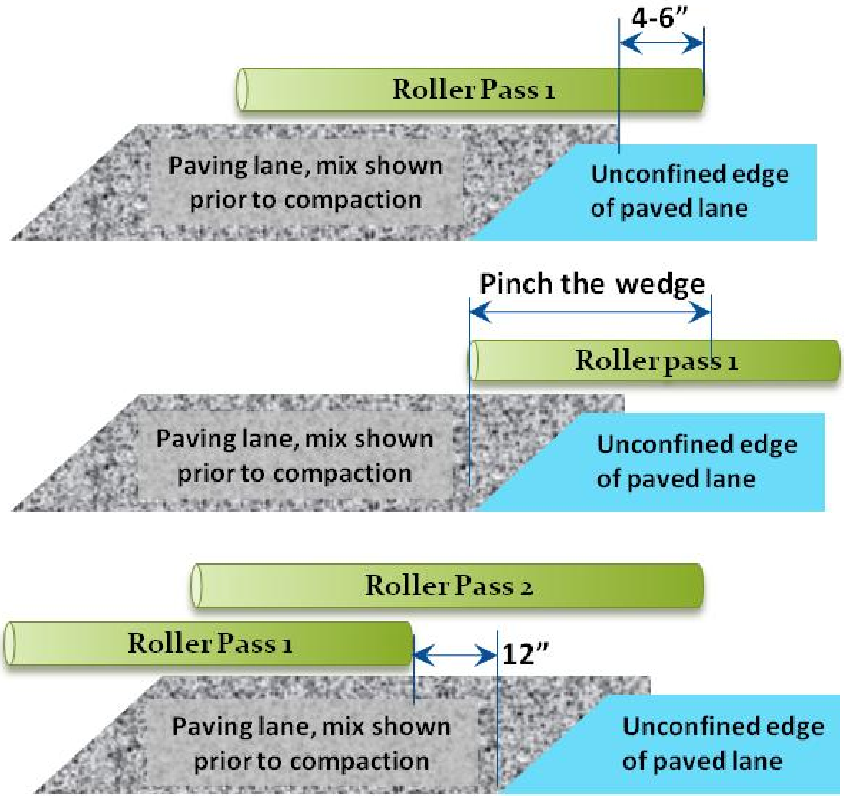


Figure 7. Type of Longitudinal Joints

Figure 8. Notched Wedge Joint (a small steel drum is attached to paver to roll the wedge behind the paver)

The following defines the rolling patterns included in the sampling matrix (refer to Figure 2) for a confined and unconfined joint.

- Steel wheel rollers (static and vibratory modes):
  - Unconfined Joint: Two locations are recommended for use during the first pass of the steel wheel roller along the joint (static or vibratory modes).
    1. The first and preferred location of the first roller pass along the joint – the edge of the steel drum is extended 4 to 6 inches over the edge of the lift.
    2. The second location of the first roller pass along the joint – the edge of the steel drum is adjacent to the edge of the lift; in other words, no overhang of the roller over the edge of the lift.
  - Confined Joint: Three locations are recommended for use during the first pass of the steel wheel roller along the longitudinal construction joint (static or vibratory modes dependent on location of roller for the first pass; refer to Figure 9).
    1. The first and preferred location of the first roller pass along the joint – the roller is operated on the hot side of the joint and overhangs the edge of the lift by 4 to 6 inches (static or vibratory modes).
    2. The second location of the first roller pass along the joint – the roller is operated on the cold side of the joint for the first pass; only about 6 inches of the roller is operated on the hot side of the mat. This is defined as the cold side pinch method (static mode only for the first pass).
    3. The third location of the first roller pass along the joint – the roller is operated on the hot side of the joint but the first pass is located about 4 to 6 inches from the longitudinal joint on the hot side. This is referred to as the hot side pinch method. The second pass of the roller is typically over the part not rolled during the first pass (static or vibratory modes for both passes).
- Rubber tired pneumatic rollers: For both the unconfined and confined joints, the edge of the tire should be located along the edge of the mat – no overhang of the roller. Rubber tired rollers are not commonly used in the breakdown or primary position in Michigan.



Pattern 1: Roll on the hot side of the joint with steel wheel roller extended over cold side by 4 to 6 in.

Pattern 2: Roll on the cold side of the joint first (static mode), pinching the hot side (referred to as the cold side pinch).

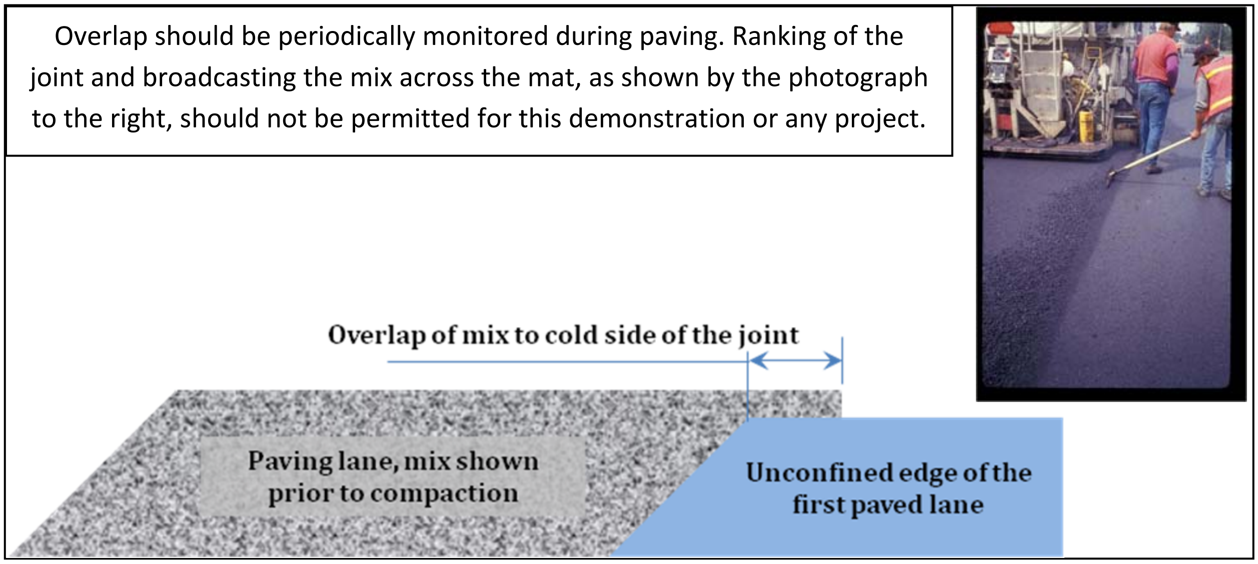
Pattern 3: Roll on the hot side of the joint first, but away from the edge; second pass is over the joint (referred to as the hot side pinch).

Figure 9. Type of Rolling Patterns for Longitudinal Construction Joint (Steel Wheel Roller)

- *Sealed and Unsealed Joints*: This experimental factor should not have an effect on the final density of the joint, but may have an effect on long term performance. MDOT can decide to exclude this factor from the sampling matrix, because a tack coat should be applied to all cold joints, especially if the joint was opened to traffic prior to placing the adjacent lane. It is recommended that the standard tack coat material specified by MDOT be used, unless MDOT wants to consider other more expensive materials that have been used as an adhesive for longitudinal construction joints. Sealed and unsealed (or glued and unglued) joints should be varied between the lots within the demonstration project. If this factor is included in the experiment, the distress surveys become mandatory to determine the benefit and effectiveness of sealing the joints in comparison to unsealed joints. Distress surveys and performance monitoring will require a minimum of 5 years to determine any systematic difference in centerline cracking and its severity between sealed and unsealed joints.

Other parameters or features that should be recorded during paving, but not included in the experimental matrix, are listed below.

- Overlap of HMA on the cold side of the joint (refer to Figure 10). Excessive overlap of the HMA onto the cold side of the joint can result in inadequate densities along the hot side of the joint because the amount of “roll down” is much less at the joint. No overlap of the HMA onto the cold side of the joint can result in an insufficient amount of mix along the joint. The proper amount of overlap should be 0.5 to 1 times the nominal aggregate diameter.
- Distance between the end of the auger and screed end plate (refer to Figure 11). Excessive distance between the end of the auger and screed end plate (24+ inches) can result in longitudinal segregation near the outside edge of the mat. Longitudinal segregation results in low densities along the joint.



Overlap should be periodically monitored during paving. Ranking of the joint and broadcasting the mix across the mat, as shown by the photograph to the right, should not be permitted for this demonstration or any project.

Figure 10. Overlap of Mixture on Cold Side of Joint

Segmentation of Demonstration Project

The layout of the individual test sections (lots) within each demonstration project is presented in Figure 12. The individual test sections or lots represent a different rolling pattern for the type of joint included in an individual project. The sampling matrix for this demonstration project was presented in Figure 2.

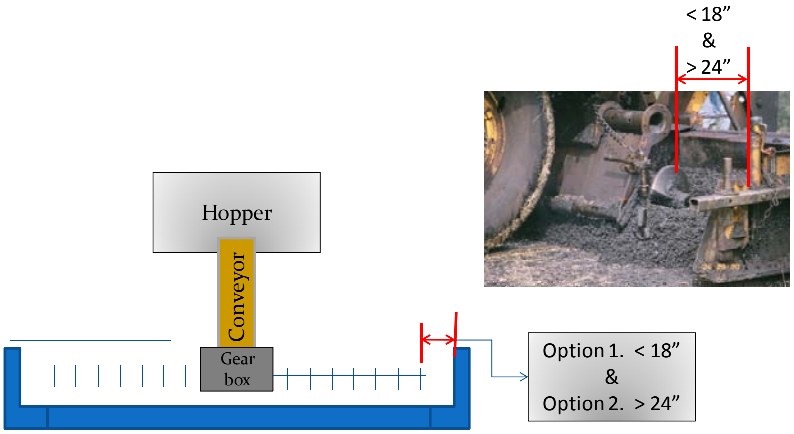
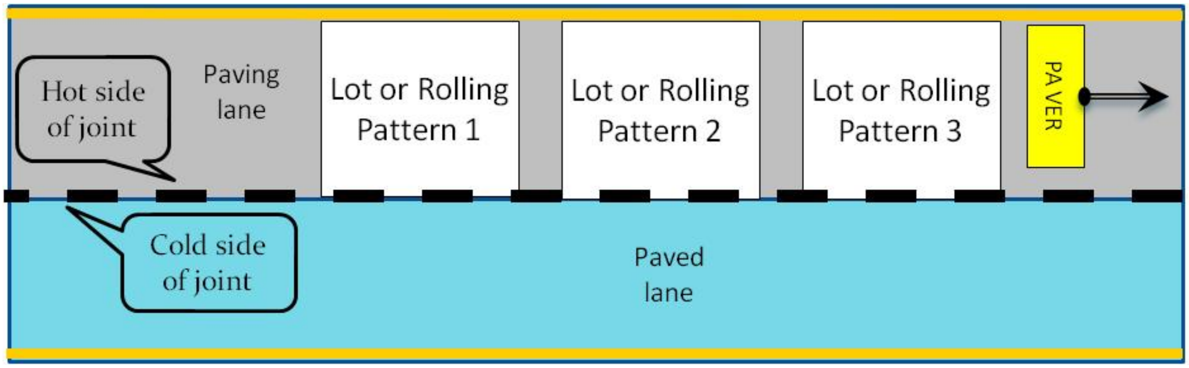


Figure 11. Distance Between End of Auger and Screed End Plate



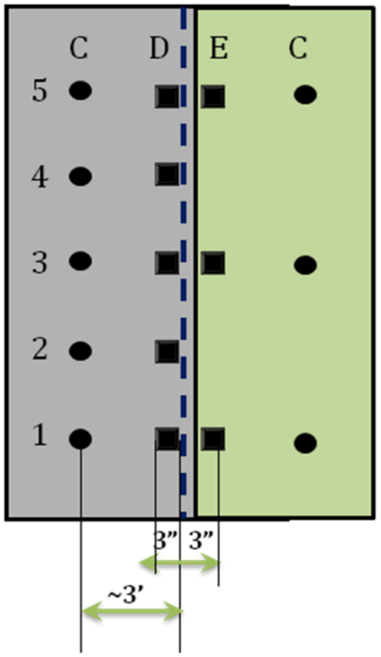
- Projects can be two lane roadways or multiple lanes in the same direction.
- The test sections represent lots within the project that have a specific rolling pattern. A typical lot is defined as a day’s paving, but MDOT can decide to define the lot on another basis for the demonstration projects to reduce the number of days of paving.
- The different rolling patterns used on the demonstration project should be varied along the project length, but be kept constant within a particular lot.
- For the set of rolling patterns, it is recommended that the construction joint be sealed or unsealed so that the experiment is not confounded by other factors.

Figure 12. Test Section Segments

Field Test Plan During Construction

Nondestructive Tests

Two field tests are recommended during construction to evaluate the condition of the joint to accept or reject experimental hypothesis #1: (1) stiffness, measured with the portable seismic pavement analyzer (PSPA); and (2) density, measured with the nuclear or non-nuclear density gauges. Stiffness is not included in the draft longitudinal joint specification, but is included in the field test plan to identify changes in other mixture properties rather than just density. Figure 13 shows the suggested layout of the test points for evaluating the condition of the joint after final rolling, while Table 2 is a summary of the field activities for this demonstration project.



Legend
● Test point in interior
■ Test point along joint

Notes
1. Points along line C are along the center of the roller.
2. Points D and E are on either side of the longitudinal joint
3. PSPA and density gauge tests are to be performed at all test points

PSPA and density readings with a nuclear or non-nuclear density gauge should be made at 5 locations on the hot side of the joint within the lot for each rolling pattern used to compact the longitudinal joint. Tests at 3 locations on the cold side of the joint should be sufficient. In addition, these tests should be made along the interior of the mat for comparison to the joint readings.

Figure 13. Test Point Location; Determining Specification Values for Joint Density for Each Lot

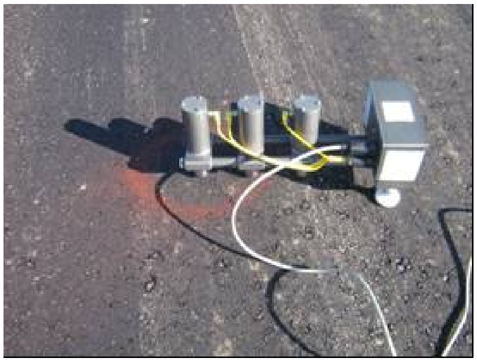
Table 2. Summary of Field Test Plan and Activities

Field Activity for One Type of Joint		Comment
1	Locate test sections for the different rolling patterns used during the paving operation. Multiple compaction zones are recommended within the same sublot.	Refer to Figure 13. Areas with the same rolling pattern should be marked for future distress surveys.
2	Monitor the material being placed along the longitudinal construction joint during placement of adjacent lanes.	The mix previously placed in the adjacent lane should have been monitored in the same manner.
3	After mix placement and finish rolling, mark the locations for the PSPA tests and density gauge readings along the joint and within the interior of the mat.	Refer to Figure 2.
4	Take PSPA and density gauge readings in accordance with procedure documented in NCHPR project 10-65.	Three density gauges readings and four PSPA tests should be made at each test point.
5	Mark locations for the three cores and drill cores. The bulk specific gravity should be measured on each core.	Two cores located along the joint and one within the interior of the mat (lowest and highest density).

The density gauge and PSPA device can take readings at a rapid rate and will not interrupt the contractor’s production rates. At each test location, cluster tests should be performed using both devices. Three readings with the density gauges and three readings with the PSPA should be taken at each test point. The following provides a summary of the tests and location of the devices relative to the longitudinal joint.

- <u>PSPA Test to Estimate the In Place HMA Stiffness:</u>

  1. Place the sensor bar on the pavement surface and parallel to the longitudinal joint and take the first reading.
  2. Rotate sensor bar to that it is perpendicular to the joint, but does not cross the joint.
  3. Move the sensor bar so that the joint is located between the loading point and first sensor on the sensor bar.



- <u>Gauge to Estimate the In Place HMA Density:</u>

  1. Place the density gauge on the pavement surface with the face parallel to the longitudinal joint (center of gauge is about 2 to 3 inches from the joint).
  2. Rotate the gauge so that its face is perpendicular to the joint, but not located over the joint.
  3. Move the gauge so that the middle of its base is located over the longitudinal joint.

The average measurement will be considered representative of the material property at each location, and used to evaluate the reasonableness of the values included in the draft longitudinal construction joint specification (objectives #1 and 3). The individual readings for both devices, however, should be recorded and consistently identified by their specific test location relative to the joint.

HMA Cores for Visual Observations and Density Measurements

A minimum of two cores should be taken within every section (lot) along the joint during construction. One core should be recovered from the interior of the mat. The cores should be located in areas with the highest and lowest density gauge readings. The cores are used to adjust the nuclear or non-nuclear density readings to the core densities. For the tapered or notched wedge joints, a 4 or 6-inch diameter core should be located so that its edge is on the hot side about 1 inch from the joint but material from the taper or wedge at the bottom of the layer is recovered. For butt joints, the edge of the core should be located less than 1 inch from the joint.

Post Construction Performance Data

Distress surveys are needed to evaluate experimental hypothesis #2. Distress surveys should be performed annually to measure the length and severity of longitudinal centerline cracks and any deterioration along the longitudinal construction joint. The distresses that should be monitored and quantified to confirm experimental hypothesis #2 include:

- Longitudinal cracking and deterioration along the longitudinal joint, grouped by low, medium, and high severity
- Potholes, grouped by number of potholes along joints
- Raveling, grouped by area adjacent to joints

Demonstration Project #2:

Biased Inspection and Testing During Construction

Introduction

Nearly all projects classified with poor performance exhibited excessive center lane longitudinal cracking. It is expected that this cracking is a result of an inadequate amount of mixture being pushed under the paver gear or drive box; sometimes referred to as center lane segregation. An economic and effective method to reduce the occurrence of these longitudinal cracks is to conduct density tests and visual inspection at the center of the paver during the first couple of days of paving and then on an as needed basis as directed by the project engineer.

The infrared camera is a device that can be easily used to identify areas with construction defects that cause center lane longitudinal cracks and deterioration. As such, biased sampling and testing with the use of an infrared camera is recommended to identify factors causing center lane cracking during the first day of paving so corrective actions can be taken, if needed.

A demonstration project is suggested to illustrate the biased inspection and testing and use of the infrared cameras. Implementation of biased inspection and testing activities should have no impact on construction costs but should extend the service life of flexible pavements by eliminating the center lane longitudinal cracks and deterioration.

Objective of Demonstration Project

1. Prepare/confirm a set of guidelines that can be used by MDOT staff to locate problem areas at the beginning of paving so that corrective actions can be taken by the contractor. The initial guidelines are included in the next section.
2. Demonstrate use of infrared cameras to identify construction defects near the center of the auger chamber and in other areas of the mat (refer to Figures 4 and 5 included in Mitigation Strategy #3).

Guidelines for Selecting Areas to be Sampled and Tested

The following is a draft set of guidelines that can be initially used for implementing biased inspection and testing activities.

> *During the first day of paving, the inspector shall monitor the paving operation and measure the density in specific areas that are identified as "cold spots." The infrared*

camera should be used to identify "cold spots," If present. Cold spots can be the result of longitudinal and truck to truck aggregate segregation, or an insufficient amount of mixture being placed in selected areas – center of the auger chamber. One area or location to monitor is the mixture placed at the center of the auger chamber and along the outside edges of the slat conveyor (transferring mixture from the paver hopper to the auger chamber).

A density reading with a calibrated nuclear (or non-nuclear) density gauge should be taken at the center of the paver at periodic intervals depending on the length of each sublot during the first day of paving. If the density readings are consistently low, relative to other areas of the mat, paving should be discontinued to determine the reason for the lower density values and corrective action taken.

If no defects or "cold spots" with low density readings are found, paving can continue. The infrared camera should be used over the course of the project to identify potential "cold spots" and/or cores taken to confirm that the material has been adequately compacted.

If conditions change during the course of the project, biased sampling and testing should be performed at the direction of the project engineer.

Construction/Project Features Included in Demonstration

A demonstration project is recommended to achieve the second objective prior to implementation. MDOT, however, can decide to proceed with implementing this strategy on a routine basis. 2012 paving projects selected for this demonstration project should include a range of HMA parameters or properties:

1. Lift thickness: Projects with lift thickness less than 2 inches and greater than 2 inches should be selected for the demonstration. Lift thickness has a significant impact on the loss of temperature or time available for compaction.

2. Aggregate blend: Projects with gap-graded, coarse-graded, and fine-graded aggregate blends should be selected for the demonstration. Gap and coarse-graded mixtures are more susceptible to aggregate segregation and more likely to exhibit greater temperature differences in localized areas for contractors not paying close attention to the paving operation.

Equipment and Field Test Plan During Construction

Two pieces of equipment are recommended for use in monitoring construction and assessing the condition of the HMA lift at the time of construction: nuclear or non-nuclear density gauges and an infrared camera. The density gauges are used to measure density of the in place mixture after final rolling in multiple locations. The infrared camera is used to locate cold spots behind the paver and after final rolling. Some cores will need to be taken to confirm the density readings.

As understood, MDOT does not have any infrared cameras for monitoring surface temperature differences during paving. It is recommended that at least one infrared camera be purchased for the 2012 construction season to demonstrate the effectiveness of biased sampling and testing. This camera can be used within a specific region or used on specific projects throughout Michigan. In the future, at least one infrared camera per region is recommended.

The following is a listing of the steps or activities suggested to achieve the project objectives.

- Take an infrared image of the HMA surface temperature behind the paver prior to rolling. The images can be saved within the camera for future reference. Images should be taken at different times during the rolling process to determine whether significant temperature differences occur. If the image illustrates uniform temperatures (refer to Figure 6), temperatures will usually stay uniform at a later time; except in areas that are shaded and adjacent to areas that have no shade.

- For projects where the surface temperature is uniform across and along the area paved (no cold spots; refer to Figure 6), the density gauge should be used to randomly measure the density along the center of the paver and outside the edges of the slat conveyor after final rolling. No bias or systematic difference should exist between the density values measured at the center of the paver and those measured in other interior areas of the mat.
    - If no systematic differences in densities are found, paving should continue.
    - If consistently lower densities are found at the center of the paver but those densities are above the specification value, paving can continue, but the inspector should continue to closely monitor the paving operation with the infrared camera and density gauge.
    - If consistently lower densities are found at the center of the paver and those densities are below the specification value, paving should be discontinued to determine the reason for the lower densities and corrective action taken.

- For projects where cold spots are located (refer to Figures 4 and 5), designate or mark the location of the image on the lift and mark the location of the cold spot. Multiple images should be taken as the paver travels down the roadway to confirm multiple locations of the cold spots. After final rolling, the density gauge should be used to take readings in the cold spots and in areas outside the cold spot.

  - If the densities are found to be consistently lower and outside the specification value, paving should be discontinued until the cause of the cold spots are determined and corrective action taken to eliminate the cold spots.
  - If the densities are found to be consistently lower in the cold spot, but exceed the specification value, paving can continue. The inspector should continue to closely monitor the paving operation with the infrared camera and density gauge.
- Cores should be taken in selected areas to adjust the nuclear or non-nuclear density readings. These cores are used to calibrate the density gauge.
- Once adequate density levels have been confirmed, the inspector should use the infrared camera periodically (or at random) to ensure that the surface temperatures of the lift are remaining uniform. If any cold spots are located during construction (longitudinal or truck-to-truck temperature differences; refer to Figures 4 and 5), densities should be taken within those areas to confirm that the density exceeds the specification value.

Pilot Project #3:

Revised HMA Mixture Design Criteria

Introduction

Extensive lengths of transverse cracks, alligator cracks, longitudinal edge and wheel path cracks, block cracking, and raveling were recorded on just about all of roadway segments exhibiting poor performance. Conversely, segments with exceptional performance exhibited significantly less transverse cracks and tears, and minor lengths of longitudinal cracks, alligator cracks, block cracking, and raveling.

The roadway segments with excessive cracking were not restricted to colder climates or MDOT regions, soil type/strength, or traffic level so it was concluded that these cracks are more of a materials issue rather than a climate, traffic, or structural issue. Excessive alligator cracks, longitudinal cracks in the wheel path and along the edge, and transverse cracks are characteristic of high stiffness, low strength HMA mixtures relative to the supporting layers. Higher laboratory compactive efforts (higher N_{design} values) will result in lower effective asphalt contents by volume. Reducing the number of gyrations during mixture design will increase the effective asphalt content by volume, which has an effect on mixture durability and its resistance to cracking, especially for lower volume roadways that are thinner or pavements built over weak soils – both of which have higher deflections.

The hypothesis is that some HMA mixtures are susceptible to fracture because of lower asphalt contents. Lower asphalt contents can reduce the tensile strength of HMA and result in brittle mixtures. Higher laboratory compaction efforts can result in lower effective asphalt contents by volume. More importantly, MDOT and industry have designed gap-graded for uniform-graded unmodified HMA mixtures on numerous projects, especially for the wearing surface. Gap-graded, unmodified HMA mixtures can exhibit higher permeability because of higher portions of larger (coarser) aggregate in the aggregate blend. Low asphalt content mixtures with high permeability are more susceptible to accelerated aging and moisture infiltration, which increases surface deterioration and reduces the mixture's resistance to cracking. Revising the mixture design guidelines and laboratory compaction criteria should improve on the mixture's resistance to cracking for both low and high volume roadways.

A pilot project is needed before making any revisions to the current HMA mixture design procedure. This pilot project will provide data to determine the effect of lowering the number of gyrations on the volumetric properties that are used for acceptance and payment. Simply lowering the number of gyrations is not recommended because of the potential impact on rutting and other distresses. The pilot project will also provide data to compare the fundamental properties between different aggregate blends (gap-graded versus coarse and fine-graded mixtures).

Objective of Pilot Project

1. Provide experimental data to determine whether lowering the number of gyrations for mixture design to determine the target asphalt content based on volumetric properties will increase the mixture's resistance to fracture, while maintaining its resistance to rutting.
2. Evaluate the fundamental properties (related to performance) of gap-graded, unmodified HMA mixtures, in comparison to coarse-graded and fine-graded neat mixtures and/or mixtures with enhanced fundamental properties. Mixtures with enhanced fundamental performance properties are included in Pilot Project #4.

Experimental Hypotheses

1. Reducing the number of gyrations for mixture design and increasing the minimum VMA will increase the effective asphalt content by volume, increasing the mixture's resistance to fracture and disintegration, and make the mixture more tolerant to tensile strains.

Experimental Factors

The following lists the experimental factors included in the sampling matrix (refer to Figure 3 under Mitigation Strategy #3).

- Layer type: HMA base layer and wearing surface for new construction or reconstruction (including crush and shape with bituminous surfaces) and HMA overlays. Layer type is the primary factor, while pavement structure is a secondary factor in the sampling matrix. Projects should be selected that include both new construction and HMA overlays.

- Traffic level: High to low traffic volumes. This experimental factor will be used to evaluate the use and impact of number of gyrations on the volumetric and fundamental properties of a particular aggregate blend and aggregate type related to durability versus load resistance properties. At present, N_{design} is dependent on traffic level. Other parameters that are related to mixture flexibility maybe as important. In other words, mixtures may need to be more flexible or more strain tolerant for pavements with higher deflections, independent of traffic level.

- Aggregate type and blend: Coarse-graded, gap-graded and fine-graded mixtures, and/or small versus large aggregate blends. Aggregate blend is the primary factor included in the sampling matrix, because of its effect on the asphalt content demand based purely on surface area, as well as on the mixture's resistance to cracking and rutting. Nominal aggregate size is a secondary parameter and is included in the sampling matrix through lift thickness; thicker HMA base layers to thinner wearing surfaces. Layer thickness should be compatible with aggregate size because of the minimum lift to nominal aggregate size ratio requirement.

- Number of Design Gyrations: The number of gyrations included in the Michigan mixture design procedure represents the baseline condition (Asphalt Institute SP-2 Mixture Design Manual). It is suggested that two other levels be used to determine the effect on the volumetric and fundamental properties of the mix at the target asphalt content. The gyration levels selected and used can be based on preliminary studies; either conducted by MDOT or other agencies that have already lowered N_{design}.

It is recommended that the climate or regional effect on asphalt performance grade selection be kept the same and not included in the sampling matrix. However, projects should be selected to include different performance grade asphalts that are typically specified and used by MDOT.

Laboratory Test Plan

The laboratory test plan represents a large testing effort, which is summarized in this section. A total of 8 test specimens are required for each gyratory level or 24 test specimens for three levels of gyration for each mixture. It is expected that the number of specimens can be reduced to optimize the sampling matrix for the different sets of test specimens. The following summarizes the testing plan and sampling matrix (refer to Figure 3).

The laboratory evaluation is grouped into two subsets.

1. The first subset of test specimens: all HMA mixtures included in the sampling matrix should be designed with the current mixture design procedure and criteria (N_{design} gyrations). After the target asphalt content and job mix formula have been determined using existing procedures, the fundamental properties should be measured on laboratory prepared specimens at the expected air void level based on the construction specification.
2. The second subset of test specimens: the HMA mixture should be compacted using reduced levels of compaction or N_{design} levels. The target asphalt content and job mix formula is determined for the revised compaction level. The fundamental properties are measured on laboratory prepared specimens at the same expected air void level specified during construction.

Two types of laboratory and field tests are recommended for use in monitoring construction and assessing pavement performance at the time of construction. These tests include volumetric and fundamental properties of the HMA.

- Volumetric properties include those properties normally measured using the current mixture design process; density, air voids, Voids in Mineral Aggregate (VMA), and Voids Filled with Asphalt (VFA). The volumetric properties are used to determine the target asphalt content in accordance with Michigan's existing procedures – current mix design methodology for selecting the target asphalt content.

- Fundamental performance properties include dynamic modulus, tensile strength and tensile strain at failure using the indirect tensile test (or a measure of the strain energy required to fracture the specimens), and a repeated load permanent deformation test. The fundamental properties are measured on laboratory compacted specimens to the expected in place air void level and compared to the number of gyrations used to determine the target asphalt content and job mix formula.

    - Dynamic modulus tests should be performed in accordance with AASHTO T 79 (*Determining the Dynamic Modulus and Flow Number for HMA using the Asphalt Mixture Performance Tester*) for preparing a master curve relationship (AASHTO PP 61 or PP 62). Replicate test specimens should be sufficient for each mixture.

    - Indirect tensile strength tests should be performed in accordance with AASHTO standards for determining the indirect tensile strength and tensile strain at failure or the strain energy. Triplicate test specimens are needed for this test because the strain measurements are variable. The test temperature is the equivalent temperature for fatigue.

    - Repeated load permanent deformation tests should be performed in accordance with AASHTO T 79 for determining the flow number, with the exception that confined tests are needed (the confining pressure is 10 psi and the applied deviator stress is 70 psi). The other difference is that the slope and intercept of the plastic strain versus number of load cycles need to be determined and reported in addition to flow number. The test temperature is the equivalent temperature for rutting. Triplicate test specimens are needed for this test because of the variability in the test results.

The deformation tests should be performed on test specimens that have been short term aged, while the fracture tests should be performed on test specimens that have been long term aged. Short term aging is used to evaluate rutting, while long term aging is used to evaluate transverse and longitudinal cracking and other mixture disintegration type distresses. The fundamental tests are used to determine the effect of changing volumetric properties on the performance properties.

As noted previously, MDOT has already sponsored the use of some fundamental tests to characterize HMA mixtures (You, et al., 2009). The two tests included within that study was the dynamic modulus and flow number (or repeated load permanent deformation) tests. Flow number is an estimate of the mixture's resistance to rutting, while dynamic modulus provides some measure of the mixture's resistance to alligator cracking and rutting.

Rutting was not found to be an issue in terms of premature failures; few roadway segments were found to have excessive rut depths. Longitudinal and transverse cracks were the more predominant distress for roadway segments with inferior performance. As such, MDOT is encouraged to use a practical fundamental test that measures a mixture's resistance to cracking.

The tensile strength and tensile strain at failure or the strain energy of the mixture can be measured using the indirect tensile test. MDOT is encouraged to use a fracture test for evaluating any change in the mixture design procedure (reducing the number of gyrations for design). Dynamic modulus and flow number (the raw data of plastic strain versus number of load cycles and not the flow number) are still beneficial, especially in determining the HMA mixture inputs to the new Mechanistic-Empirical Pavement Design Guide (MEPDG).

Performance Assessment of Revised HMA Mixture Design Guidelines

Distress surveys should be completed at periodic intervals to monitor the condition of the flexible pavement or HMA overlay over time. The project should be divided into lots used for acceptance based on MDOT standard procedures and practice. Some of the lots of the project should be designed and placed using current mixture design practice (the standard sections), and the others designed and placed using the revised mixture design guidelines (the companion sections).

The distress surveys should be completed in accordance with the standard procedures being used by MDOT. Each lot should be monitored to determine the impact of HMA mixtures on long term performance.

To maximize the benefit from this pilot project, it is recommended that these sections be identified and well documented for future use in calibrating the MEPDG to Michigan local conditions and materials.

Pilot Project #4:

Wearing Courses with Enhanced HMA Mixture Properties

Introduction

All projects with poor performance were found to exhibit transverse cracks and tears, alligator cracks, longitudinal cracks in the wheel path, and surface deterioration (raveling). The amount and severity of these cracks and raveling can be reduced by using higher quality wearing surfaces; such as stone matrix asphalt (SMA) and polymer modified asphalt (PMA). MDOT and local contractors have designed and used gap-graded, unmodified HMA mixtures. These mixtures can have lower asphalt contents and high permeability resulting in durability issues; raveling, block cracking (longitudinal and transverse cracks), and alligator cracking with time.

It is hypothesized that a cause for this premature cracking is a result of the gap-graded unmodified HMA mixtures that have been specified and used in Michigan, especially for higher volume roadways. Thus, use of wearing courses with enhanced mixture and asphalt properties is expected to reduce the amount of transverse, block cracking, and longitudinal cracking in the wheel path.

As noted previously, MDOT has allowed the use of gap-graded dense HMA mixtures for the wearing surface. Gap-graded HMA mixtures can exhibit high permeability because of the higher portions of larger aggregate in the aggregate blend. Higher permeability mixtures are more susceptible to accelerated aging and moisture infiltration, which increase surface deterioration of the mixture and reduce its resistance to cracking. The intent of this pilot project is to reduce the amount and severity of various types of cracking (block, alligator, transverse cracks and tears, and longitudinal cracks in the wheel path) and surface deterioration by using HMA mixtures with enhanced properties (PMA and SMA).

There is a lot of support that documents the benefit and reduction in surface distress with the use of PMA and/or SMA mixtures to be used as the wearing surface. The MDOT database, however, does not identify those projects were PMA or SMA type engineered mixtures were placed as the wearing surface. It is recommended that MDOT start recording and documenting the projects where these mixtures with enhanced properties have been used to establish performance characteristics that can be quantified and compared to conventional, neat HMA mixtures for the site features, materials, and other conditions encountered in Michigan.

It is recommended that MDOT proceed with the use of SMA and PMA wearing surfaces on the higher volume roadways, but only after the two demonstration projects have been completed.

This pilot project is compatible with pilot project #3 (Mitigation Strategy #2). In fact, the results from pilot project #3 can be used to determine the fundamental properties for PMA and SMA mixtures, as compared to the existing HMA mixtures produced and placed under the current construction and material specifications. A fundamental performance test should eventually be used to measure the properties of any HMA mixture, but especially those on higher volume roadways (refer to mitigation strategy #5).

Objective of Pilot Project

The objectives of this pilot project are to:

1. Collect performance data on roadway segments with PMA and SMA wearing surfaces for quantifying the magnitude of the extended service life or reduction is pavement distress.

2. Revise the MDOT performance database to designate and record the mixtures with enhanced surface properties.

It is recommended that MDOT proceed with implementing Mitigation Strategy #4, but there is insufficient data for quantifying the increase in service life or reduction in distress for conditions encountered and materials used in Michigan. This longer term pilot project has been recommended to achieve this objective. The data from the pilot project will be used to confirm the expected increase in service life of 3 to 5 years based on studies sponsored by other agencies (Asphalt Institute, Colorado DOT, etc.).

Performance Assessment of PMA and SMA Mixtures

Distress surveys should be completed at periodic intervals to monitor the condition of the flexible pavements over time. The distress surveys can be completed in accordance with MDOT standard procedures. It is recommended that the following distresses be monitored and quantified during the field distress surveys:

- Smoothness in terms of International Roughness Index (IRI)
- Rutting in the wheel path
- Alligator cracking grouped by low, medium, and high severity
- Block cracking grouped by low, medium, and high severity
- Longitudinal cracking in the interior of the lane, grouped by low, medium, and high severity
- Longitudinal cracking along the longitudinal joint, grouped by low, medium, and high severity
- Potholes, grouped by number of potholes in the interior and along joints
- Raveling, grouped by area in the interior and adjacent to joints

This pilot project will require a minimum of 10 years to complete to collect data within Michigan for confirming the increase in service life with the use of wearing surface with enhanced mixture properties. This increase in service life, however, can be estimated in a much shorter time period by measuring the fundamental performance properties of the mixtures used on selected project.

To decrease the amount of time for confirming the increase in service life, the procedure used by the Asphalt Institute in combination with the measured mixture properties under Pilot Project #3 is recommended.