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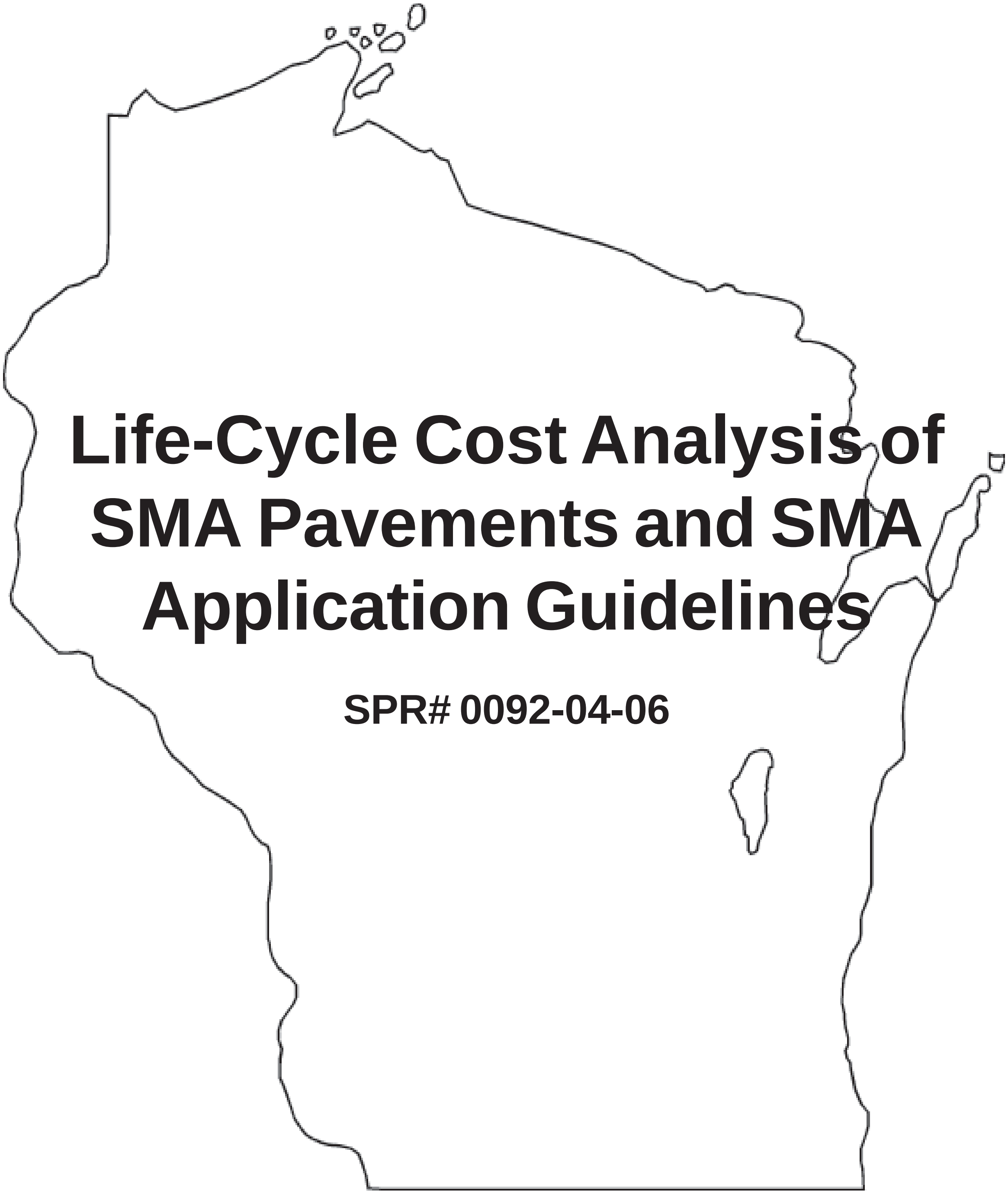

Life-Cycle Cost Analysis of SMA Pavements and SMA Application Guidelines

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Applied Research Associates, Inc.

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LIFE-CYCLE COST ANALYSIS OF SMA PAVEMENTS AND SMA APPLICATION GUIDELINES

FINAL REPORT

By

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16. Abstract

The objective of this research project was to provide a comparative cost analysis of pavements constructed using stone matrix asphalt (SMA) mixtures versus those built with WisDOT's conventional hot-mix asphalt (HMA) mixtures, based on parallel life-cycles (inclusive of any required maintenance) and resultant performance. The research entailed a thorough evaluation of SMA and conventional HMA mixture performance on Wisconsin highways, collection and review of SMA and conventional HMA unit costs, and full-scale life-cycle costing to determine the cost-effectiveness of SMA pavements.

Performance analysis, consisting of Pavement Distress Index (PDI) and International Roughness Index (IRI) threshold-based life projections combined with survival analysis techniques, resulted in SMA and HMA overlay service life estimates when placed on three groups of like pavements: (1) low-volume asphalt pavements on U.S./State routes, (2) high-volume jointed reinforced concrete (JRC) pavements on Interstate/U.S. routes, and (3) moderate-volume JRC pavements on U.S./State routes.

Based on the results of the performance analysis, pavement life-cycle models were developed for SMA and HMA overlays corresponding to each group. Using the life-cycle models and historical-based best estimates of pay item unit costs, deterministic and probabilistic life-cycle cost analyses (LCCAs) were conducted. For group 1 overlay applications, SMA was found to be more cost-effective than conventional HMA, leading to the recommendation that SMA mixture use under this scenario be considered on a case-by-case basis. For group 2 and 3 overlay applications, the reverse was observed, with HMA found to be more cost-effective. However, a similar recommendation for SMA use under these scenarios was given, based on indications that the SMA overlays examined in the study were placed on rougher (possibly more deteriorated) pavements.

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EXECUTIVE SUMMARY

PROJECT SUMMARY

The objective of this research project was to provide a comparative cost analysis of pavements constructed using stone matrix asphalt (SMA) mixtures versus those built with WisDOT's conventional hot-mix asphalt (HMA) mixtures, based on parallel life-cycles (inclusive of any required maintenance) and resultant performance. Based on the results of cost analysis, recommendations and guidelines were to be developed concerning future use of SMA mixtures on Wisconsin highways.

BACKGROUND

The Wisconsin Department of Transportation (WisDOT) began investigating the use of SMA mixtures in 1991, installing several test sections on state highways throughout the early and mid 1990s. Since that time, the Department has placed several additional SMA projects as part of the normal construction function. Overall, approximately 25 different SMA projects have been constructed in Wisconsin, all in the form of resurfacing.

Results of various performance studies have shown that SMA offers initial pavement performance benefits for the higher initial costs incurred. In a national review of SMA performance undertaken in 1995 (86 SMA projects in 19 States, including Wisconsin) and again in 2001 (11 SMA projects in five States, including Wisconsin), it was concluded that SMA mixtures are rut-resistant, are more resistant to thermal and reflective cracking than conventional hot-mix asphalt (HMA), and are generally meeting or exceeding agency's performance expectations (Watson, 2002).

Life-cycle cost analyses (LCCAs) have been performed in regional areas to determine if SMA mixtures are cost effective. However, a full and comprehensive LCCA has not been performed to determine the cost-effectiveness of SMA pavements in Wisconsin, as compared to WisDOT's conventional HMA mixtures (i.e., the Marshall-based "V" mixes used prior to 2001 and the Superpave-based "E" mixes used since then).

With the adoption of SMA in 2000 as a WisDOT standard product for pavement design and with the shrinking of revenues for highway construction and rehabilitation, a detailed economic-based analysis of SMA pavements is warranted. In addition, identification of the conditions under which SMA pavements are cost effective is highly desired.

PROCESS

This study involved multiple tasks and subtasks. To begin with, a national literature search and review was conducted pertaining to the performance, costs, and cost-effectiveness of SMA mixtures compared to traditional HMA mixtures. This effort was followed by an effort to identify all completed SMA construction projects in Wisconsin, as well as conventional HMA projects with similar profiles (i.e., similar locations and time of construction, similar structure and traffic loading), which could serve as companions for evaluating the cost-effectiveness of SMA.

All pertinent data (e.g., cross-section, traffic, M&R history, time-series condition/distress and ride quality) for the identified SMA and companion HMA projects were obtained from WisDOT and then carefully reviewed and compiled into an analysis database. Performance analysis, involving the projection of Pavement Distress Index (PDI) and International Roughness Index (IRI) trends to established threshold levels followed by statistical survival analysis, was done to estimate the service life of each mixture type when applied as an overlay on existing pavements grouped as follows:

- Group 1: Low-volume asphalt pavements on U.S./State routes.
- Group 2: High-volume jointed reinforced concrete (JRC) pavements on Interstate/U.S. routes.
- Group 3: Moderate-volume JRC pavements on U.S./State routes.

Contract unit prices for each mixture type for the years 2001 through 2004 were analyzed, from which best estimates were developed for use in the LCCA.

Based on the results of the performance analysis, pavement life-cycle models were developed for SMA and HMA overlays corresponding to each group. Using the established life-cycle models, the best estimates of pay item unit costs, a 45-year analysis period, and a discount rate of 5 percent, deterministic and probabilistic life-cycle cost analyses (LCCAs) were conducted to assess the cost-effectiveness of SMA compared with conventional HMA.

For overlays of pavements in the group 1 category, SMA was found to be more cost-effective than conventional HMA. For overlays of pavements in the group 2 and 3 categories, conventional HMA was found to be more cost-effective than SMA.

Because in many instances, the initial smoothness of SMA overlays was found to be significantly lower than the companion HMA overlays (implying that SMA was placed on rougher [possibly more deteriorated] pavements than HMA), it was surmised that SMA performance and life-cycle cost was negatively impacted. Thus, it was recommended that SMA mixtures be considered for overlay use on a case-by-case basis for both existing asphalt and concrete pavements, particularly those in relatively good condition.

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT.)

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1. INTRODUCTION

1.1 BACKGROUND AND PROBLEM STATEMENT

Stone Matrix Asphalt (SMA) has been successful at providing a rut-resistant, durable asphalt surface mixture in Europe since its development in Germany over 25 years ago. The primary use then was to resist the wear of studded tires. However, the use of SMA continued in Germany after studded tires were banned in 1975 because initial performance trends had shown it to perform better than the conventional asphalt concrete (AC) wearing course in resisting distresses such as rutting.

SMA pavement construction began in the U.S. in the early 1990's with the placement of SMA mixtures in five States—Wisconsin, Michigan, Georgia, Texas, and Maryland. This initial construction was followed by the construction of more than 250 SMA projects in at least 25 States in the past decade. The reason for this widespread use is the outstanding performance of the initial projects in the U.S.

The Wisconsin Department of Transportation (WisDOT) began investigating the use of SMA mixtures in 1991, installing several test sections on state highways throughout the early and mid 1990s. Since that time, the Department has placed several additional SMA projects as part of the normal construction function. Overall, approximately 25 different SMA projects have been constructed in Wisconsin, all in the form of resurfacing.

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With the adoption of SMA in 2000 as a WisDOT standard product for pavement design and with the shrinking of revenues for highway construction and rehabilitation, a detailed economic-based analysis of SMA pavements is warranted. In addition, identification of the conditions under which SMA pavements are cost effective is highly desired.

1.2 RESEARCH OBJECTIVES

The primary objective of this study was to provide a comparative LCCA of SMA and conventional HMA pavements in Wisconsin, based on parallel life cycles, inclusive of any required maintenance, and resultant performance. More specifically, this study was intended to generate answers to the following basic questions:

- What is the expected service life of SMA pavements and what factors have a significant effect on that service life?
- What is the significant initial, maintenance, and rehabilitation costs associated with SMA and WisDOT's conventional HMA mixture pavements?
- What are the life-cycle costs (LCCs) of SMA and conventional mixture pavements?
- How do the costs associated with SMA pavements compare to those of other conventional mixture pavements?
- Are SMA pavements cost-effective, when compared to conventional mixture pavements?

1.3 SCOPE OF REPORT

This report contains seven chapters, including this introductory chapter. Chapter 2 presents a summary of the SMA literature collected and reviewed in the study. Chapter 3 describes the data collection and database assembly effort, while chapters 4 and 5 discuss the analysis of pavement unit costs and pavement performance, respectively. The development of life-cycle models is covered in chapter 6 and the results of the LCCA are presented in chapter 7. Chapter 8 provides a summary of the key findings of the study, as well as recommendations concerning the future applications and use of SMA in Wisconsin.

CHAPTER 2. LITERATURE REVIEW

2.1 LITERATURE SEARCH

A comprehensive literature search and review was performed at the outset of this study. Both consultant library searches and Internet searches were made, resulting in the identification of over 50 reports, papers, and articles on SMA experimentation or use in the U.S. Each document was carefully reviewed for pertinence to this study. Brief summaries of the literature deemed most pertinent to this study are presented below.

2.1.1 Wisconsin Research

<u>Stone Matrix Asphalt: The Wisconsin Experience (1991-1996) (Schmiedlin and Bischoff, 2002)</u>

The first trial installation of SMA in Wisconsin was constructed in 1991. Based on the success of the trial installation, a thorough evaluation of SMA was conducted. Six projects at various locations around the State were constructed and evaluated. Each of the six projects had various test sections with different fiber and polymer-modified SMA mixes. The impact of aggregate size and hardness on the effectiveness of SMA mixes was also studied. These SMA projects were constructed between 1992 and 1994 and evaluated for ease of construction on a subjective basis and for performance after 5 years.

Over the short time period of the study (5 years after construction), the SMA test sections were found to be performing better than conventional HMA mixtures with respect to cracking and other distresses. The improved cracking performance of SMA at an early evaluation stage was by a factor of two in most instances. In other words, the SMA projects exhibited less than 50 percent of the cracking that occurred on conventional overlay projects. The SMA mixes with aggregate most resistant to abrasion and impact were more effective at retarding cracks. SMA mixes placed over another HMA pavement seemed to crack at the same rate at those placed over PCC pavements. For both types of overlays, SMA mixes seemed to provide better crack resistance than the conventional HMA mixes. The SMA mixes also provided consistently and significantly better frictional characteristics. Both the SMA mixes and the conventional HMA mixes had very low rutting.

Typical problems reported with the placement of SMA included being unable to maintain a tight paving train and difficulty in maintaining a proper mix temperature for adequate compaction, as a result of long haul or low ambient temperature. No evaluation on the overall cost effectiveness relative to conventional HMA was performed. Most of these problems can be solved by improving the construction operation.

SMA research in Wisconsin conducted following the first trial installation in 1991 indicated that over a short time period (5 years), the SMA test sections performed better than conventional HMA mixtures with respect to cracking and other distresses and with respect to frictional characteristics. However, no long-term analysis or cost analysis were performed.

Validation of Wisconsin Pavement Service Lives (2004-2005) (Titus-Glover et al., 2006)

The objective of this recently completed study was to validate current WisDOT pavement service life estimates by analyzing the performance of selected Wisconsin highway pavement types subjected to levels of traffic and climate conditions. The results of the research project are the development of service life estimates for use in the Department's pavement selection process.

The analyses were done using conventional statistical procedures, including survival analysis, with actual service life of new construction/reconstruction and rehabilitation events as input. Service life estimates were developed for different pavement categories, which included, among others, new HMA over flexible base, HMA overlay on existing flexible pavement, and HMA overlay on existing rigid pavements (i.e., non-doweled jointed plain concrete [JPC-ND], jointed reinforced concrete [JRC], and continuously reinforced concrete [CRC]).

Results of the analysis led to the following recommendations for mean service lives (in both years and cumulative heavy trucks) of conventional HMA overlays:

Interstates/Expressways

- HMA overlay (mean thickness = 3.9 in) on existing HMA: 19 years, 13.4 million trucks.
- HMA overlay (mean thickness = 3.9 in) on existing JRC: 14 years, 15.6 million trucks.

Primary/Secondary Routes

- HMA overlay (mean thickness = 3.9 in) on existing HMA: 30 years, 2.0 million trucks.
- HMA overlay (mean thickness = 4.3 in) on existing JRC: 19 years, 0.7 million trucks.

2.1.2 Other Research

Evaluation of Stone Mastic Asphalt Used in Michigan in 1991 (1991-1992) (Brown, 1993)

This report documents the results from a laboratory study on varying mixture components; no field performance data were included in the study. Michigan's first SMA pavement was constructed in 1991. The materials used in the construction of this pavement were evaluated to study the sensitivity of SMA mixture properties to changes in proportions of various mixture components. SMA samples for each of 17 mixture variations representing various combinations of percent passing No. 4 sieve, percent passing No. 200 sieve, asphalt content, and fiber content, were tested and compared with a dense-graded HMA using the same aggregate. This was a limited study using only one aggregate, one asphalt cement, and one additive to produce SMA and a dense-graded conventional mixture.

The results of the study indicated that the performance of SMA mixtures in the laboratory was significantly affected by the aggregate gradation, suggesting that very close control of the aggregate gradation and shape during construction is required. The conventional HMA performed better in many laboratory tests than some of the SMA blends that deviated from the job mix formula and other requirements. The confined creep test and the gyratory shear stress test, which are indicators of rutting resistance, were used to evaluate relative quality of SMA mixtures. The SMA mixture properties measured in the laboratory were minimally affected by varying asphalt contents.

Stone Mastic Asphalt Trials in Ontario (1990-1991) (Emery and Schenk, 1993)

Two trial SMA sections designed at an asphalt cement content of about 3 percent air voids were constructed in December 1990 in Toronto, Canada and tested by the Ministry of Transportation of Ontario (MTO). The first section was an SMA surface course with nominal maximum aggregate size of 13 mm, and the second section was a binder course with a nominal maximum aggregate size of 19 mm. Laboratory rutting tests were performed on slabs removed from the test sections. The measured rutting was considerably lower for both SMA sections than the control or conventional HMA mixture. Both SMA mixes met the MTO criteria after accounting for the initial seating within the laboratory rutting test, which was not the case for the conventional HMA mixtures.

Several other trial sections were constructed in Ontario in 1991 to evaluate various other features of SMA mixtures. There were initial problems with low mixture temperatures for some of these trial sections. The reported problems were rectified and subsequent mixing, placement, and compaction of the SMA mixtures proceeded satisfactorily. All test sections were monitored along with conventional control mixes. However, there were no reports available on pavement performance comparisons between these projects.

Performance of Stone Matrix Asphalt Mixtures in the United States (1994-1996) (Brown and Mallick, 1997)

A summary of mix design and performance data obtained from 86 SMA projects throughout the U.S. was compiled for this research study. The inspections of the pavements were conducted between 1994 and 1996 and included projects from Alaska, Arkansas, California, Colorado, Georgia, Illinois, Indiana, Kansas, Maryland, Michigan, Missouri, Nebraska, New Jersey, North Carolina, Ohio, Texas, Virginia, Wisconsin, and Wyoming. A majority of the pavements were constructed between 1992 and 1994.

The major conclusions of this research study can be grouped into three areas: materials, construction, and performance. The conclusions within each group are listed below.

Material Conclusions

- The recommended Los Angeles abrasion requirement of 30 was met 85 percent of the time.
- 90 percent of SMA mixtures had 25 to 35 percent of material passing 4.75 mm sieve.
- 80 percent of the SMA mixtures had 7 to 11 percent of the material passing the 0.075 mm sieve.
- 60 percent of the projects had greater than 6.0 percent asphalt content during production.

Construction Conclusions

- 30 percent of the projects had average air voids during construction less than 3 percent.
- Construction of good longitudinal joints were a problem on earlier construction but improved with contractor experience.

Performance Conclusions

- Over 90 percent of the SMA projects had rutting measurements less than 4 mm, 70 percent had less than 2 mm rutting, and over 25 percent had no measurable rutting. Six projects had rutting rates greater than 6 mm, in which the rutting could be attributed to the SMA mixture.
- No significant thermal cracking, reflective cracking, and raveling were observed.
- Fat spots caused by segregation, draindown, high asphalt content, or improper type or amount of stabilizer, were the biggest performance problems associated with SMA.

An Updated Review of SMA and Superpave Projects (2001) (Watson, 2002)

This study was conducted as a follow-up to evaluate the performance of SMA (constructed between 1991 and 1995 and first surveyed in 1995) and Superpave projects (majority constructed between 1994 and 1998 and first surveyed in 1998). As part of the follow-up, a total of 11 SMA projects and 18 Superpave projects located in 5 states (Colorado, Indiana, Maryland, Virginia, and Wisconsin) were reviewed in 2001.

Projects were evaluated for distresses such as fatigue cracking, block cracking, transverse cracking, reflective cracking, thermal cracking, raveling, segregation, patches, and rutting. Rut depths and crack widths were estimated visually. The results of the follow-up study suggest that both SMA and Superpave mixtures can be rut-resistant even on high traffic volume pavements. While several of the Superpave and SMA projects were still in excellent condition after being in service for 5 and 9 years, respectively, SMA mixtures can be expected to last longer than Superpave mixes before reaching the same condition level.

End-of-load segregation, paver streaks, thin overlays, and poor longitudinal joint construction were primarily responsible for much of the observed distress. The evaluation of pavement distresses supported European experience that SMA mixes can be expected to last up to 25 percent longer than conventional HMA mixtures.

Stone Mastic Asphalt in Colorado (1994-2000) (Harmelink, 2001)

In Colorado, the first SMA trial mixture was placed in 1994 on SH 119 and contained 3 SMA mixes, two polymer-stabilized mixes and one fiber mix. The second project included placing an SMA mix on a bridge deck. This project used a polymer stabilized mixture.

The overall performance of SMA exceeded CDOT's performance expectations. The SMA projects performed exceptionally well with virtually no rutting and no detrimental effects due to moisture. The performance of SMA as an overlay for bridge decks also exceeded CDOT's expectations with no evidence of cracking. Limited problems with flushing were observed but could be attributed to drain-down. It was reported by the authors that this problem could be mitigated with efficient delivery methods to the lay down machine. The SMA smoothness was comparable to that of conventional HMA mixtures and obtaining appropriate smoothness was not an issue. The skid numbers measured immediately after construction were comparable to those measured 6 years after construction, with no reduction in skid resistance.

Although costs of the SMA on both projects were substantially higher than the cost for conventional HMA, the authors hypothesized that with experience and removal of the risk of uncertainty, SMA costs can be expected to be competitive to conventional HMA. CDOT's experiences with these two projects are documented in this report. Based on these projects and experiences with other projects, CDOT currently uses SMA as a wearing surface on any high profile, high-volume roadway where a skid resistant, durable surface is required. CDOT has successfully placed overlays or wearing courses using nominal maximum aggregates size of 19.0 mm, 12.5 mm, and 9.5 mm. CDOT has also developed specifications for using SMA for bridge overlays.

Summary of Georgia's Experience with Stone Matrix Asphalt Mixes (1991-1996) (Georgia DOT, 2002)

The Georgia Department of Transportation (GDOT) became interested in SMA mixtures on the Georgia road system after the European Asphalt Study Tour in 1990. GDOT conducted two research projects in 1991 and 1992, to evaluate the performance of SMA versus that of conventional HMA mixes as (1) an intermediate and wearing course under heavy truck loads, and (2) an overlay for Portland cement concrete (PCC) pavements.

In 1991, various combinations of SMA and standard mixes were placed in a 2.5-mile, high traffic volume test section on I-85 in northeast Georgia (35,000 ADT, 40% trucks, 2 million ESALs/yr). In 1992, a test section was placed on I-75, south of Atlanta (47,000 ADT, 21% trucks), to determine if the coarseness of the SMA mixes might deter rutting and other distresses normally associated with HMA overlays.

These studies indicated that production cost savings could be realized if the aggregate quality requirements for SMA mixes were relaxed, assuming that the performance of these mixes would not be significantly reduced. GDOT, implemented use of aggregates which have less than 45% abrasion loss and less than 20% flat and elongated particles when measured at the 3:1 ratio. The GDOT experience supported the European experience and shows the following intrinsic benefits of SMA:

- 30-40% less rutting than standard Georgia HMA mixtures.
- 3 to 5 times greater fatigue life in laboratory experiments.
- Lower annualized cost.

Based on these studies and their overall experience with SMA mixes, GDOT has expanded the use of SMA as a dense-graded surface mix for Georgia interstate pavements.

Potential of Using Stone Matrix Asphalt (SMA) for Thin Overlays (Cooley and Brown, 2003)

This document included laboratory tests for comparing different size SMA mixtures. No field performance studies were included in the study. SMA mixtures typically have nominal maximum aggregate size (NMAS) of 12.5 or 19.0 mm. Of the 144 pavement sections evaluated by the National Center for Asphalt Technology (NCAT) in 1997 for a national study to evaluate the performance of SMA, only 6 had NMAS that differed from 12.5 or 19.0 mm. 5 of these 6 sections were placed on one project in Wisconsin with a NMAS of 9.5 mm.

The potential advantages of using a "fine" SMA with a NMAS of 4.75 or 9.5 mm is that it can be placed using a thinner lift thickness (less than 19 mm and 32 mm, respectively), thus making it useful within a preventive maintenance program. Using a thinner lift thickness also allows for more projects to be covered with the same tonnage of mix. Other potential benefits include reduction in permeability, smoother surface, and improved workability.

As part of this study, several SMA mixtures with of 4.75 and 9.5 mm NMAS were compared to more conventional SMA mixes with larger aggregate particles. Samples were tested for rut susceptibility and permeability. The results showed that the fine SMA mixtures could have stone-on-stone contact and can be utilized as rut resistant overlays. The permeability tests indicate that the fine SMA mixtures should be more durable because they have less potential for permeability than conventional SMA mixtures.

Stone Matrix Asphalt – VDOT's Initiative for Longer Lasting Roads (Mergenmeier, 2004)

Over a dozen projects have been constructed in Virginia using SMA, predominantly on Interstates. SMA costs on average $14 more per ton due to limited production quantities to date and few suppliers equipped to bid work. SMA mixtures have been found to provide superior performance on roadways where they have been used, but have yet to be adopted as a standard mixture for wearing surfaces. Reasons for this reluctance to adopt them as a standard mixture include higher initial cost, focus on Superpave over the last 5 to 8 years, and concerns with higher asphalt contents. Other potential concerns reported with SMA mixtures in Virginia include placement problems such as raveling, joints, and fat spots and more chemicals required for snow and ice removal. However, even with the above concerns, SMA is the preferred mixture for surface wearing courses on high-volume, high-ESAL routes with greater than 20,000 AADT and greater than 10 million cumulative ESALs projected over the 20-year period.

Construction problems encountered and reported with SMA in Virginia include fat/slick spots, crushed or fractured aggregate due to improper use of rollers, compaction problems resulting from contractor's lack of attention to detail, and not using proper quality control (QC) procedures. Lessons learned from SMA in Virginia include: (1) Good QC is a must; (2) Quality aggregates are essential for SMA, and (3) Voids in Coarse Aggregate (VCA) is required to ensure selected mix gradation has stone-on-stone contact.

The Challenge of SMA (Brown, 2005)

The use of SMA in the U.S. has increased due to its improved performance as compared to conventional dense-graded HMA mixtures, particularly in high-traffic applications, in more than 28 states. The primary challenge of using SMA is meeting the aggregate specifications. SMA relies on stone-on-stone contact between very hard, cubical aggregates to obtain structural strength. A high percentage of the aggregates are coarse and a low percentage of the aggregates are intermediate-sized particles. The shape of the aggregate is very critical because cubical aggregates are required to provide the necessary strength to the SMA mixture. Typical specifications call for a maximum of 20 percent of flat and elongated particles of a 3:1 ratio and a maximum of 5 percent of flat and elongated particles of a 5:1 ratio. The 20 percent maximum of flat and elongated particles of a 3:1 ratio can be difficult to achieve for many aggregate sources.

In Virginia, the state is building a case to support its long-term commitment to SMA so that aggregate producers can make the investment in crushers needed to make aggregates for SMA. Luck Stone in Virginia meets the specifications by adjusting a compression crusher by either tightening the crusher down on the closed side, or opening up the crusher to increase circulation load. The consequence, however, is a drop in production by 40 to 60 percent, and consequently increase in operating costs. In Maryland, Arundel meets the specifications by cutting down the reduction ratio of stone size input to stone size output using a supplementary crusher that takes No. 5 and No. 6 stones as feed-stocks and crushes them into No. 7 and No. 8. This output is blended back into the main plant's No. 7 and No. 8 stones. The practicality of using 100 percent stone from the supplementary plant is not economical. In South Dakota, the mixture design was adjusted and the aggregates hardness was used to compensate for more flat and elongated particles. Most aggregate suppliers have difficulty in meeting the limit of 20 percent 3:1 flat and elongated particles. The LA abrasion test of the aggregates used was 23 percent, which was far below the national specification of 45 percent.

The Virginia Department of Transportation (VDOT) is experimenting with mixtures made with four separated sizes of fractionated aggregates. The Indiana Department of Transportation (INDOT) uses steel slag, which has no problem meeting the 20 percent maximum 3:1 flat and elongated specification and has an LA abrasion of 20 percent. The 25 to 30 projects constructed with steel slag since the late 1990s are exhibiting excellent performance. However, steel slag SMA costs about 25 percent more than conventional surface mixtures and transportation costs can push costs upward.

2.2 REVIEW SUMMARY

As noted in chapter 1, SMA mixtures have been used in the U.S. since 1990. The majority of SMA use has been on rehabilitation projects of heavily traveled roadways. Research studies have shown that the use of SMA mixtures within a rehabilitation strategy or the surfacing layer for new construction have exceeded the performance of conventional mixtures.

The general consensus seen in the literature based on both European experience and that of various States in the U.S. is that SMA pavements can be expected to last up to 25 percent longer than conventional HMA pavements, before reaching the same condition level. However, there are several challenges facing large-scale use of SMA in the U.S., including meeting aggregate specifications, construction problems/contractor experience, and initial construction costs.

CHAPTER 3. DATA COLLECTION AND DATABASE ASSEMBLY

3.1 INTRODUCTION

This chapter describes in detail the data collection and database development effort undertaken in this study. The effort involved obtaining the most recent highway pavement databases and hardcopy records from WisDOT, manually and electronically uploading the data into a project database, reviewing the assembled data for accuracy and completeness, and developing a final project database for data analyses.

3.2 DATA COLLECTION

The data/information required for analysis to satisfy the project objectives included the following:

- Project location information—Highway number, beginning and ending reference points, direction, lanes, county, WisDOT District, setting (urban, rural), climatic region (north, central, south). This information and data were easily accessible from the WisDOT files.
- Pre-SMA/HMA overlay pavement information—Original construction year, original cross-section (i.e., layer thicknesses and material types), subsequent maintenance and rehabilitation (M&R) applications (years, material types and thicknesses).
- SMA/HMA overlay information—Thicknesses and mixture details of binder and surface course applications, milling depth (if applicable).
- Traffic information—Historic estimates of average daily traffic (ADT) or annual average daily traffic (AADT), percentage of trucks (i.e., FHWA vehicle class 4 through 13), equivalent single axle loads (ESAL), and traffic growth rates.
- Performance history data—Historical condition/performance data in terms of Pavement Distress Index (PDI), key distresses, and the International Roughness Index (IRI); both prior to and after overlay placement.

Several sources of data were obtained from WisDOT for use in developing the project database. The data were available in both electronic and hardcopy formats. Descriptions of the data sources along with the data they contained are provided in the sections below. Information relevant to this study was retrieved for each source and assembled as datasets (with a common electronic format). The assembled datasets were then merged to obtain the project database to be used in analysis.

3.2.1 HMA Mix Design Database

The HMA Mix Design Database contained mix design and test log information for all HMA projects performed between 1992 and 2003. Key data fields included the following:

- Project number, location (highway number, county, start and end points), and year.
- Contractor.

- Mix type and design number.
- Aggregate source.
- Asphalt binder source and type.
- Design test properties, including aggregate gradation, asphalt binder content, percent reclaimed asphalt pavement, voids in mineral aggregate (VMA), aggregate bulk and effective specific gravities (G_{sb} and G_{se}), mixture bulk and Rice maximum specific gravities (G_{mb} and G_{mm}), stability, flow, and tensile strength ratio (TSR).

3.2.2 Materials Tracking System

This online materials database contained detailed laboratory test information for mixtures used in asphalt construction/rehabilitation projects performed since 2000. Key data fields included those listed above, as well as other material properties.

3.2.3 AC Office All and PCC Office All Databases

The AC Office All database included nearly 1,200 asphalt construction/rehabilitation projects undertaken on Wisconsin highways between 1989 and 2003, while the PCC Office All database included nearly 300 concrete construction/rehabilitation projects for the same time period. Key data fields included the following:

- Project number and location (highway number, reference points, county, WisDOT District).
- Year of construction/rehabilitation activity.
- Type and thickness of construction/rehabilitation activity (including milling).
- AC design mix type.
- Pavement base type.
- Existing pavement type, if an overlay.

3.2.4 Layer Report

The Layer Report Database included nearly 29,300 construction/rehabilitation projects undertaken on Wisconsin highways between 1920 and 2004. Key data fields included the following:

- WisDOT District.
- Pavement sequence number identifying a section of highway defined by beginning and ending reference points (RPs) (e.g., highway intersections, bridge structures).
- Year of construction/rehabilitation activity.
- Pavement type and thickness of construction/rehabilitation activity.

3.2.5 New Construction Reports

The New Construction Reports included annual reports detailing the roughness measurements collected on new and rehabilitated pavements for years 1954 to 2003. The reports were used to supplement and verify the other sources of construction history data. Key data fields included the following:

- Project number, location (highway number, begin and end points, county, WisDOT District), and year.
- Length.
- Contractor.
- Existing pavement type.
- New pavement type and thickness.
- Ride quality (PSR before 1993, IRI after 1993) of new pavement.

3.2.6 Meta-Manager Database

The Meta-Manager Database Roadway Spreadsheet was the source of traffic data used for analysis. The Meta-Manager included 13 files with six separate years from 1998 to 2004 having multiple files for some years. The 13 files included over 244,000 records describing section geometric features and traffic counts. Key data fields included the following:

- Pavement sequence number.
- Divided/undivided/1-way highway section designation (D/U/1).
- Highway number and traffic direction.
- Functional class.
- Current annual average daily traffic (AADT) (directional split when roadway is divided 50/50).
- Projected AADT for 2010 (directional split when roadway is divided 50/50).
- Percent of current AADT that is truck traffic.
- Number of lanes (directional split when roadway is divided 50/50).

3.2.7 Traffic Books

Traffic books for years 1994, 1997, 1998, 2000, 2001, 2002, and 2003 containing estimates of traffic counts and AADT.

3.2.4 Pavement Information Files (PIF) Database

The Pavement Information Files (PIF) Database included the DESC, IRI, PDI, and other tables. The DESC table provided a description of 13,795 roadway sections by sequence number. The IRI and PDI tables provided performance data used for analysis. The IRI table included over 129,000 sets of ride quality data collected on Wisconsin highways between 1980 and 2004 (PSI data for entire time period, IRI data from 1990 to 2004). The PDI table included nearly 112,000 sets of PDI data collected on Wisconsin highways between 1985 and 2002.

Key data fields in the PIF database included the following:

DESC Table

- Pavement sequence number.
- Highway number, direction, and functional class.
- County and WisDOT District.
- Beginning physical feature.
- Divided highway section designation (Y/N).

IRI Table

- Pavement sequence number.
- Test year, month, and day.
- Surface year.
- Surface type.
- IRI section average.

PDI Table

- Pavement sequence number.
- Survey year, month, and day.
- Surface year.
- Individual distress types and values.
- PDI for pavement section/sequence number.

3.3 PROJECT IDENTIFICATION

3.3.1 SMA Projects

A list of SMA projects built in Wisconsin (primarily Districts 2, 3, and 6) was provided by WisDOT personnel at the outset of the study. This list was based on mix design records. The SMA mix design parameters were reviewed on the www.atwoodsystems.com/materials web site for validation of project status. Some of the approximately 25 SMA projects were not evaluated because they were relatively new projects that lacked sufficient historical performance data for analysis.

3.3.2 Companion Conventional HMA Mix Projects

Companion sections to the SMA projects were based on the similarity of the criteria defined in table 1. Section information in the PIF database were reviewed to best match criteria associated with each SMA project to select the most appropriate conventional HMA mix project for comparison.

Table 1. Companion selection criteria.

Criteria	Comment
Location—Same route, county, or district	Companion sections for Comparison 1, 2, 3, 5, 6, & 9 are the control sections for the WisDOT Experimental SMA site.
Construction/ rehabilitation date—Within ± 5 years	
Climate—Same climate zone (North, Central, South)	South – Districts 1 and 2 Central – District 3, 4, 5, and 6 North – District 7 and 8
Pavement structure—Similar pavement structure	Overlay thickness within ±0.5 in. Similar underlying structure type and thicknesses

3.4 DATABASE ASSEMBLY

Data assembly consisted of the following key steps:

1. Establishing reference identification numbers (i.e., WisDOT pavement management section sequence numbers) for each SMA and conventional mix project selected for analysis.
2. Converting data from all sources into a common electronic format. For this project, the common electronic format was Microsoft® Excel.
3. Estimating traffic for each SMA and conventional mix project with useable data available in the project database.
4. Assembling the final project database for use in analysis.

Detailed descriptions of the steps above are provided in the sections below.

3.4.1 Establish Reference Identification Number

Reference identification numbers (i.e., sequence numbers) were established for each SMA and conventional HMA mixture project from the PIF Database using the physical descriptions defined for each project. The sequence numbers were used to extract information in the Layer Report, Meta-Manager Database, and PIF Database.

Project numbers provided by WisDOT personnel for each SMA and conventional mix were also used to extract information from the HMA Mix Design Database, Materials Tracking System, AC and PCC Office All Databases, New Construction Reports, and traffic books.

3.4.2 Convert Data from All Sources into a Common Electronic Format

Data for each SMA and conventional mix project were obtained in different formats (paper hardcopies, text files, Microsoft® Excel files, Microsoft® Access files, etc.). To facilitate data assembly, relevant data from all the different sources were converted into a common electronic format (i.e., Microsoft® Excel formats). Conversion was done electronically for the data received in electronic format, while data received as paper hardcopies were converted by manually entering the relevant information into Microsoft® Excel spreadsheets.

3.4.3 Estimate Traffic

Traffic data from the traffic books (hardcopies) and Meta-Manager files were reviewed as part of data assembly. The goal was to determine their suitability for use in estimating cumulative traffic applied for a given construction event for a given pavement management section. Suitable data were those that enabled the project team to estimate AADT for the years of 1993 through 2004, as follows:

1. Create a historical database of AADT for each SMA and conventional HMA mixture project using traffic data from WisDOT traffic books and meta-manager files.
2. Plot the traffic data in Microsoft® Excel to evaluate the relationship between the data extracted from the Meta-Manager files and the traffic books.
3. Plot the best-fit linear line between the most appropriate data to calculate the initial traffic and traffic growth.
4. Determine average percentage of truck traffic for each SMA and conventional mixture project, using yearly (1998 through 2004) estimates contained in the Meta-Manager files.
5. Calculate the age of each SMA and conventional mixture project using the initial construction date and rehabilitation date or current year of 2004.
6. Use the linear prediction model shown in equation 1 to estimate missing AADT values for each SMA and conventional mix project.
7. Estimate cumulative AADT for each construction/rehabilitation event based on their start and end dates.

$$TRAF = TRAF_{INI} + \alpha Age \qquad \text{Eq. 1}$$

where: TRAF = Estimate of AADT at a given age, veh/day.
$TRAF_{INI}$ = Initial AADT (age = 0), veh/day.
α = AADT growth rate.
AGE = Time since initial construction or rehabilitation, years.

3.4.4 Assemble Final Dataset

The final database containing as complete as possible all information related to location and site information, construction and design information, traffic data, M&R information, and performance data (PDI, distress, and IRI data), for each SMA and conventional mix project was assembled into a project database. The information collected in the project database was used to perform the comparison analysis between the SMA and conventional mix projects. A summary of the SMA and companion conventional mix projects selected for analysis is presented in table 2.

Table 2. Projects included in pavement performance analysis.

Comparison No.	Site ID	Highway (Type & No.)	Direction	District	County(s)	Begin (Begin RP)	End (End RP)	No. Lanes	Setting	1-Way AADT$_0$, veh/day (Growth, %)	% Trucks	Climatic Zone	Const. Year
1	SMA-1a (Test SMAs---WisDOT Exp. Project 5)	IH-43	NB	2	Waukesha	CTH "U" (Guthrie Rd) (55G+0.00)	CTH "Y" (57K+0.00)	2	urban/ rural	16,000 (2.25)	13.5	south	1992
	SMA-1b (Test SMAs---WisDOT Exp. Project 5)	IH-43	SB	2	Waukesha	CTH "U" (Guthrie Rd) (55G+0.00)	CTH "Y" (57K+0.00)	2					1992
	Comp1 SMA-1a (Control---WisDOT Exp. Project 5)	IH-43	NB	2	Waukesha	STH-164 (54D+0.00)	CTH "U" (Guthrie Rd) (55G+0.00)	2	urban /rural	16,000 (2.25)	13.5	south	1992
	Comp1 SMA-1b (Control---WisDOT Exp. Project 5)	IH-43	SB	2	Waukesha	STH-164 (54D+0.00)	CTH "U" (Guthrie Rd) (55G+0.00)	2					1992
2	SMA-2a (Test SMAs---WisDOT Exp. Project 6)	IH-43	NB	2	Walworth	USH-12 (31K+0.00)	Bowers Rd (37T+0.00)	2, 3	rural	6,500 (2.75)	17.5	south	1993
	SMA-2b (Test SMAs---WisDOT Exp. Project 6)	IH-43	SB	2	Walworth	USH-12 (31K+0.00)	Bowers Rd (37T+0.00)	2, 3					1993
	Comp1 SMA-2a (Control---WisDOT Exp. Project 6)	IH-43	NB	2	Walworth	Bowers Rd (37T+0.00)	Townline Rd. (39G+0.00)	2, 3	rural	6,500 (2.75)	17.5	south	1993
	Comp1 SMA-2b (Control---WisDOT Exp. Project 6)	IH-43	SB	2	Walworth	Bowers Rd (37T+0.00)	Townline Rd. (39G+0.00)	2, 3					1993
3	SMA-3 (Test SMAs---WisDOT Exp. Project 2)	USH-63	NB&SB	8	Washburn & Sawyer	Brickman Lake Rd. (147A+0.00)	Washburn/Sawyer Co. Line (153+1.55)	2	rural	1,500 (3.75)	9.4	north	1993
	Comp1 SMA-3 (Control---WisDOT Exp. Project 2)	USH-63	NB&SB	8	Washburn & Sawyer	Stress Rd (157+0.00)	Nursery Rd. (158+0.00)	2		2,050 (3.5)	9.3		1993
4	SMA-4a	STH-100 (W. Ryan Road)	NB	2	Milwaukee	STH 241 (27th St.) (4K+0.79)	CTH “V” (3M+0.00)	2, 3	urban	6,500 (4.25)	6.6	south	1993
	SMA-4b	STH-100 (W. Ryan Road)	SB	2	Milwaukee	STH 241 (27th St.) (4K+0.79)	CTH “V” (3M+0.00)	2, 3					1993
	Comp1 SMA-4a	USH-145 (Fond Du Lac Frwy)	NB	2	Milwaukee	STH 181 OH (4+0.00)	North 107th St OH (6+0.00)	3	urban	9,500 (2.5)	7.3	south	1993
	Comp1 SMA-4b	USH-145 (Fond Du Lac Frwy)	SB	2	Milwaukee	STH 181 OH (4+0.00)	North 107th St OH (6+0.00)	3					1993

RP: Reference Point
AADT$_0$: Initial Annual Average Daily Traffic
Growth: Annual AADT growth rate

Table 2. Projects included in pavement performance analysis (continued).

Comparison No.	Site ID	Highway (Type & No.)	Direction	District	County(s)	Begin (Begin RP)	End (End RP)	No. Lanes	Setting	1-Way $AADT_0$, veh/day (Growth, %)	% Trucks	Climatic Zone	Const. Year
5	SMA-5 (Test SMAs---WisDOT Exp. Project 3)	USH-151	NB&SB	1	Grant & Lafayette	Grant/Lafayette Co. Line (23M+1.51)	Belmont Rd. (Belmont) (31+0.00)	2	rural	3,600 (3.25)	9.3	south	1993
	Comp1 SMA-5 (Control---WisDOT Exp. Project 3)	USH-151	NB&SB	1	Grant & Lafayette	Eastside Rd. (Platteville) (23M+0.00)	Grant/Lafayette Co. Line (23M+1.51)	2		3,850 (3.75)	9.3		1993
6	SMA-6 (Test SMAs---WisDOT Exp. Project 1)	USH-45	NB&SB	7	Vilas & Oneida	Brown Rd (320+0.0)	Eagle River (Pine Lake Rd/STH 70) (327K+0.00)	2	rural	2,050 (3.75)	9.3	north	1993
	Comp1 SMA-6 (Control---WisDOT Exp. Project 1)	USH-45	NB&SB	7	Vilas & Oneida	Rice Lake Rd. (317+0.00)	Brown Rd (320+0.0)	2		1,750 (4.0)	9.3		1993
7	SMA-7a	IH-894	EB	2	Milwaukee	W. Lincoln Ave (Milwaukee) (1R+0.00)	Hale Interchange (Jct I-43, Milwaukee) (4M+0.00)	3	urban	63,000 (1.75)	7.8	south	1994
	SMA-7b	IH-894	WB	2	Milwaukee	W. Lincoln Ave (Milwaukee) (1R+0.00)	Hale Interchange (Jct I-43, Milwaukee) (4M+0.00)	3					1994
	Comp1 SMA-7a	IH-94	EB	2	Milwaukee	STH-100 (108th St.) (304K+0.00)	70th St. Structure (306T+0.00)	2, 3	urban	75,000 (1.75)	6.3	south	1998
	Comp1 SMA-7b	IH-94	WB	2	Milwaukee	STH-181 (84th St) (305T+0.00)	70th St. Structure (306T+0.00)	2, 3					1997
8	SMA-8a	IH-43	NB	2	Milwaukee	Hale Interchange (Jct I-43 & I-894) (Milwaukee) (between 63T+0.00 & 65G+0.00)	Mitchell Apt Interchange (Milwaukee) (between 68K+0.00 & 69K+0.00)	3	urban	61,000 (1.5)	9.2	south	1994
	SMA-8b	IH-43	SB	2	Milwaukee	Hale Interchange (Jct I-43 & I-894) (Milwaukee) (between 63T+0.00 & 65G+0.00)	Mitchell Apt Interchange (Milwaukee) (between 68K+0.00 & 69K+0.00)	3					1994
	Comp1 SMA-8a	IH-94	EB	2	Milwaukee	76th Street (Milwaukee) (between 306T+0.00 & 305T+0.00)	13th Street (Milwaukee) (between 310K+0.00 & 315K+0.00)	3, 4	urban	77.500 (1.3)	6.4	south	1998
	Comp1 SMA-8b	IH-94	WB	2	Milwaukee	76th Street (Milwaukee) (between 306T+0.00 & 305T+0.00)	13th Street (Milwaukee) (between 310K+0.00 & 315K+0.00)	3					1997
9	SMA-9 (Test SMAs---WisDOT Exp. Project 4)	STH-21	EB&WB	4	Juneau	Juneau/Monroe Co. Line (43+0.99)	CTH "M" (Cutler Dr.) (50+0.00)	2	rural	1,500 (5.0)	12.3	central	1994
	Comp1 SMA-9 (Control---WisDOT Exp. Project 4)	STH-21	EB&WB	4	Juneau	CTH "M" (Cutler Dr.) (50+0.00)	9th Ave. (57+0.00)	2		1,450 (5.0)	12.3		1994

RP: Reference Point
$AADT_0$: Initial Annual Average Daily Traffic
Growth: Annual AADT growth rate

Table 2. Projects included in pavement performance analysis (continued).

Comparison No.	Site ID	Highway (Type & No.)	Direction	District	County(s)	Begin (Begin RP)	End (End RP)	No. Lanes	Setting	1-Way AADT$_0$, veh/day (Growth, %)	% Trucks	Climatic Zone	Const. Year
10	SMA-12a	USH-41	NB	3	Winnebago	Lake Butte De Mort Structure (102+0.00)	STH 76 Structure (107M+0.00)	2	rural	22,500 (4.5)	11.3	central	1996
	SMA-12b	USH-41	SB	3	Winnebago	Lake Butte De Mort Structure (102+0.00)	STH 76 Structure (107M+0.00)	2					1996
	Comp1 SMA-12a	USH-41	NB	3	Outgamie	French Rd Overpass (Appleton) (130M+0.00)	Holland Rd Overpass (Appleton) (132M+0.00)	2	urban	13,600 (5.75)	10.9	central	1990
	Comp1 SMA-12b	USH-41	SB	3	Outgamie	French Rd Overpass (Appleton) (130M+0.00)	Holland Rd Overpass (Appleton) (132M+0.00)	2					1990
11	SMA-13a	I-94	EB	6	Eau Claire	Otter Creek Structure (between US 53 & CTH "I") (71G+0.0)	Mallard Rd (Clear Creek) (77D+1.41)	2	rural	12,000 (3.0)	21.3	central	1997
	SMA-13b	I-94	WB	6	Eau Claire	Otter Creek Structure (between US 53 & CTH "I") (71G+0.0)	Mallard Rd (Clear Creek) (77D+1.41)	2					1997
	Comp1 SMA-13a	I-94	EB	6	Eau Claire	STH 37 Structure (65D+0.0)	USH 53 OH Structure (70M+0.0)	2	rural	10,000 (4.0)	22.2	central	1996
	Comp1 SMA-13b	I-94	WB	6	Eau Claire	STH 37 Structure (65D+0.0)	USH 53 OH Structure (70M+0.0)	2					1996
12	SMA-15	STH-29	EB&WB	6	Pierce	STH 65 @ River Falls (15K+0.00)	770th St (west of CTH "W") (21+0.00)	2	rural	2,350 (4.0)	9.6	central	2000
	Comp1 SMA-15	STH-37	NB&SB	6	Eau Claire	Eau Claire/Buffalo Co. Line (33K+0.41)	CTH "ZZ" (39G+0.00)	2		2,800 (5.5)	7.3		1999/ 2000

RP: Reference Point
AADT$_0$: Initial Annual Average Daily Traffic
Growth: Annual AADT growth rate

CHAPTER 4. ASSESSMENT OF PAY ITEM UNIT COSTS

4.1 INTRODUCTION

In comparing LCCs of SMA to that of conventional HMA mixtures, it is important to establish the initial construction and future M&R costs for use in the LCCA. According to a 2003 *Better Roads* magazine article (Kuennen, 2003), costs associated with constructing SMA mixtures are generally 10 to 30 percent and up to 50 percent higher than that of conventional HMA mixtures. An Asphalt Pavement Alliance (APA) brochure suggests that SMA costs 20 to 40 percent more and quotes Georgia DOT engineer Peter Wu, “I would say SMA costs 30 to 40 percent more, because of the polymer modifiers, the added mineral filler, fiber additive and the plant modifications needed.”

In a comprehensive investigation of pavement performance and life-cycle costing performed for the Ministry of Transportation of Ontario (MTO), Hein et al. (2000) found the costs associated with producing and placing SMA were typically 15 to 30 percent higher than traditional surface course mixtures, as reported by various States, Canadian agencies, and other countries (see table 3). Depending on the SMA design and the construction requirements of the project, however, the reported costs could be up to 30 to 40 percent higher.

Table 3. Typical reported increases in initial construction costs associated with use of SMA (Hein et al., 2000).

Agency (or Country)	Range in Initial Cost Increases
Sweden[a]	10 – 12%
Germany[a]	20 – 30%
AASHTO[a]	Up to 30%
State DOTs[a]	15 – 30%
Toronto Transportation[b]	15 – 30%
Ministere des Transports du Quebec	20 – 30%

[a] FHWA Report 92-008.

4.2 DEVELOPMENT OF BEST ESTIMATES OF PAY ITEM UNIT COSTS

To achieve the objectives of the study, the unit costs of all major pay items/activities associated with SMA- and HMA-overlaid pavements over a life-cycle had to be determined. These included the material, labor, and equipment costs of individual pavement layers placed during initial construction/ reconstruction, as well as the same costs of individual layers/treatments applied as part of M&R.

4.2.1 Sources of Data

Three sources of unit cost data were tapped for the cost analysis. These sources were as follows:

- WisDOT cost database (WISPRICE).
- Midwest regional pavement studies (i.e., Minnesota, Pennsylvania).
- National pavement studies.

Since the sources of unit costs data were available in different formats and units of measurement, the data were processed as follows:

- Assemble the raw data into a Microsoft® Excel spreadsheet.
- Transform raw data into estimates based on common units of measurements (e.g., costs in USD per ton, yd^2, etc.).
- Determine quantities for which estimates are based and eliminate outliers (cost estimates based on excessively large or very small material quantities).
- Determine average unit costs of pay items of relevance to this study.
- Adjust the average unit cost estimates for inflation (base year = 2005).

In general, cost estimates based on Wisconsin prices of materials and services were used when available. Default regional and national estimates were used to fill gaps as needed.

4.2.2 Unit Cost Estimates

Several factors affect the unit costs of construction and M&R pay items. Among the more notable factors are the number of projects, project size, raw material price fluctuations, special conditions, and geography. The process for determining costs for the relevant pay items required for LCCA are presented below.

SMA and HMA Unit Costs

Table 4 presents Wisconsin average contract unit prices for SMA (asphaltic binder and mix combined) and conventional HMA (Types E-0.3 through E-30) (mix only) for fiscal years 2001 through 2004. The table shows fluctuations in costs over time and the corresponding price differences between the mixes (Note: HMA E mix prices are for mix only, while SMA prices are for binder and mix). The historic trends of the SMA and conventional HMA average prices are further illustrated in figure 1.

Table 4. Historic average contract unit prices for Wisconsin SMA and HMA mixtures.

Item	Year				Average	Group Average
	2001	2002	2003	2004		
HMA Type E-0.3, $/ton (mix only)	17.45	20.35	23.28[a]	21.57	20.66	19.64
HMA Type E-1, $/ton (mix only)	16.81	16.26	20.21	21.30	18.65	
HMA Type E-3, $/ton (mix only)	18.06	18.95	19.94	21.48	19.61	
HMA Type E-10, $/ton (mix only)	19.34	24.48	22.59	25.80	23.05	23.05
HMA Type E-30, $/ton (mix only)	28.17	28.73	27.19[a]	23.61[a]	26.93	26.93
SMA, $/ton (asphaltic binder and mix)	37.42[a]	33.95	29.66	35.45	34.12	34.12

[a] Outliers not used in chart below.

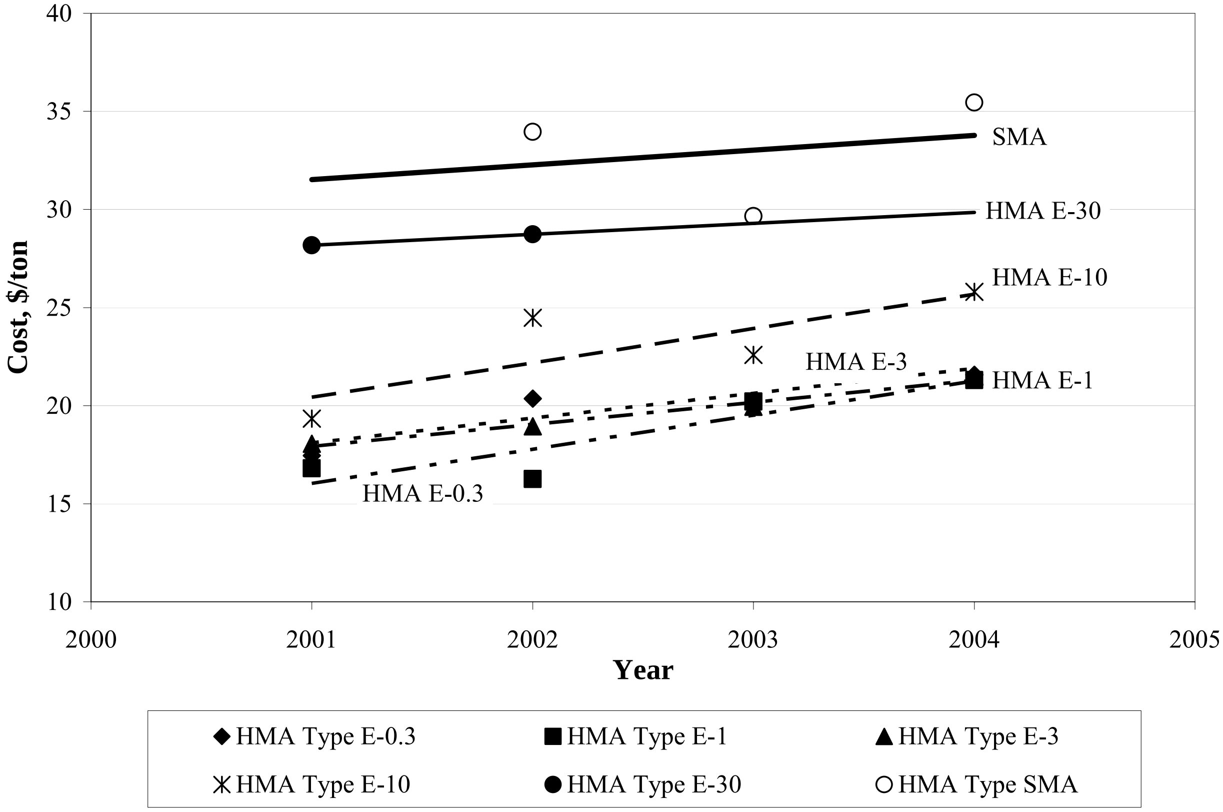


Figure 1. Historic trend of average contract unit prices for Wisconsin HMA (mix only) and SMA (binder and mix) mixtures.

As seen in table 4, the average contract price for HMA Types E-0.3, E-1, and E-3 (mix only) for the years 2001 through 2004 was $19.64/ton. These three mixes were grouped together because they had comparable prices and the price variance within a mix type was greater than the price variance between the three mix types. The average contract prices for HMA Types E-10 and E-30 (mix only) for these years were $23.05 and 26.93/ton, respectively, and the average contract price for SMA (binder and mix combined) was $34.12/ton.

The 4-year average shown in table 4 represents potential outliers. The trend lines in figure 1 were developed by ignoring these potential outliers. Using the historic trend of average contract unit prices shown in figure 1, the 2005 unit price for combined HMA Pavement Types E-0.3, E-1, and E-3 (mix only) was estimated as $23.01/ton. The 2005 unit prices for HMA Type E-10 and E-30 (mix only) were estimated as $27.43 and $30.41/ton, respectively. And, the 2005 unit price for SMA (binder and mix combined) was estimated as $34.55/ton.

Unit Costs of Binder

Figure 2 shows Wisconsin average contract unit prices for asphaltic material for fiscal years 1999 through 2005. The figure shows fluctuations in costs over time from about $130/ton to $190/ton. No significant trend can be seen between 1999 through 2005. However, with significant increase in the price of crude oil and asphalt over the last few years, the binder costs can be expected to be at or above the higher end of this range. A binder cost of $180/ton is recommended for use in the LCCA. For conventional HMA, which typically has 5.5 percent binder, this translates into an additional $9.90 per ton of conventional HMA. Although SMA uses a slightly higher percentage of binder (typically 6.0 percent), the binder cost is already included in the price of SMA, as mentioned previously.

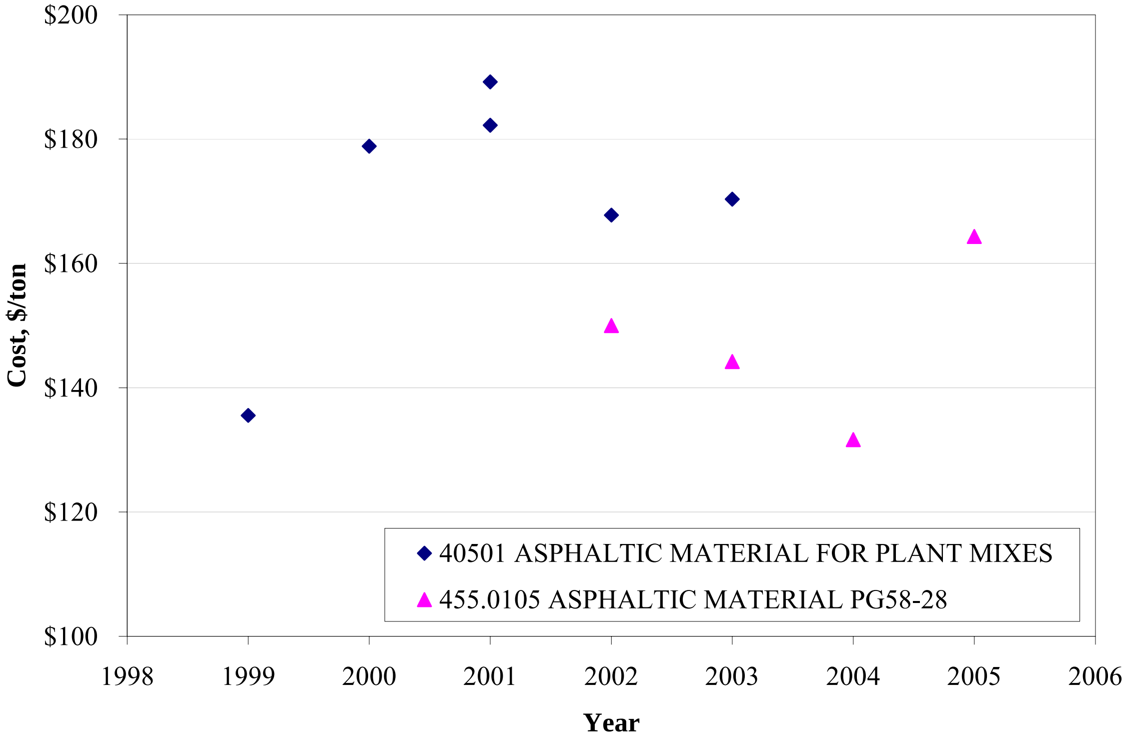


Figure 2. Historic trend of average contract unit prices for asphaltic material.

Unit Costs of Other Pay Items

Other unit cost information required for conducting the LCCA included the following:

- Tack coat: $114.25/hr, $1.29/gal.
- Prime coat: $222.00/ton.
- Reinforced concrete: $22.46/yd^2.
- Crushed aggregate base course: $8.24/ton.
- Granular subbase course: $5.74/yd^3.
- Seal coat (maintenance): $1.98/yd^2.

- Crack sealing (maintenance): \$0.28/lin. Ft.
- Mill and replace HMA: \$1.29/yd^2.
- Pavement removal: \$2.92/yd^2.

Estimates for these items were obtained from WisPrice and regional/national cost data sources.

Non-Pavement Costs

Non-pavement cost items relating to Wisconsin conditions and used in the LCCA were as follows:

- Traffic control costs—Average daily cost of traffic control, including Traffic Control Labor (4 people, 10 hr days), Sequential Arrow Sign, and Traffic Control Supervisor. A daily cost of \$1,080 was used.
- Mobilization—An average mobilization cost of 5 percent of the project total cost was used.
- Sales Tax—An average sales tax of 7.6 percent of the project total cost was used.
- Engineering and Contingencies—An average engineering and contingency cost of 15 percent of the project total cost was used.
- Preliminary engineering costs—An average preliminary engineering cost of 10 percent of the total construction cost was used.

Unit Cost Variability

The mean unit cost and standard deviations of all pay items are required for probabilistic LCCA. For this study, unit cost variability was characterized using coefficient of variation (COV), defined as the ratio of standard deviation to mean.

Historical cost data in WisPrice and the regional and national cost data sources were used to compute COV for the various cost items. Table 5 shows the historical COV for Wisconsin SMA and HMA mixtures. Outliers (e.g., cost estimates from projects that required less than 2,000 tons of HMA) were not included in the computations. Where historical data were not available in sufficient quantities, a COV of 10 percent was assumed.

Table 5. Historic coefficient of variation in contract unit prices for Wisconsin SMA and HMA mixtures.

Item	Year			Composite	Group Composite
	2000	2001	2002		
HMA Type E-0.3, \$/ton	0.26	0.31	0.25	0.28	0.28
HMA Type E-1, \$/ton	0.23	0.29	0.33	0.30	
HMA Type E-3, \$/ton	0.29	0.26	0.24	0.26	
HMA Type E-10, \$/ton	0.13	0.32	0.21	0.26	0.26
HMA Type E-30, \$/ton	-	0.35	0.29	0.31	0.31
Asphaltic binder (for HMA E Mixes), \$/ton	0.11	0.14	0.24	0.16	0.16
SMA, \$/ton	-	0.09	0.11	0.10	0.10

4.2.3 Adjustment for Inflation

The mean costs of the relevant activities were computed for each year with data using estimates from contractors' bids for all projects with pavement-related pay items. The mean unit costs were adjusted for inflation to 2005 prices using the following formula and a 2 percent inflation rate:

$$\$F = \$P * (1 + i)^{n} \qquad \text{Eq. 2}$$

where: $\$F$ = Current year (1999) cost adjusted for inflation, \$.
$\$P$ = Past year cost, \$.
i = Inflation rate, decimal (0.05).
n = Number of years between 2005 and base year (1996 to 2002), (i.e., 3 to 9 years).

The average and standard deviation of the inflation-adjusted unit costs for each pay item were then calculated to establish the unit cost inputs for the LCCA.

4.3 SUMMARY

Using historical Wisconsin, regional, and national price information and inflation-adjustment techniques, the means and COV of various pay item unit costs were determined for use in the LCCA. These values are summarized in table 6.

Table 6. Mean and standard deviation unit costs for various pay items used in the LCCA.

Pay Item	Unit of Measurement	Mean, \$	Coefficient of Variation	Standard Deviation, \$
HMA Pavement Type E-0.3 (mix only)	ton	24.52	0.28	6.44
HMA Pavement Type E-1 (mix only)	ton	23.73		
HMA Pavement Type E-3 (mix only)	ton	24.79		
HMA Pavement Type E-10 (mix only)	ton	27.29	0.26	7.13
HMA Pavement Type E-30 (mix only)	ton	31.45	0.31	9.43
Asphaltic binder for HMA E Mixes	ton of mix	9.90	0.16	1.58
HMA Pavement Type SMA (asphalt binder and mix)	ton	39.80	0.10	3.46
Tack Coat	gal	\$1.29	0.10	0.13
Tack coat application	hr	\$114.25	0.10	11.42
Prime Coat (PC)	ton	\$222.00	0.10	22.20
Reinforced concrete	yd^2	\$22.46	0.10	2.24
Crushed aggregate base course	ton	\$8.24	0.10	0.82
Granular subbase course	yd^3	\$5.74	0.10	0.57
Seal coat (maintenance)	yd^2	\$1.98	0.10	0.19
Crack Sealing	linear ft	\$0.28	0.10	0.03
Mill and replace HMA	yd^2	\$1.29	0.10	0.13
Pavement removal	yd^2	\$2.92	0.10	0.29

CHAPTER 5. PAVEMENT PERFORMANCE ANALYSIS

5.1 INTRODUCTION

As stated in chapter 1, the objective of this study was to provide a comparative cost analysis of SMA versus conventional HMA mixtures in Wisconsin, based on parallel life cycles, inclusive of any required M&R, and resultant performance. A major component of such a cost analysis is the pavement performance of the comparative sections of roadway. This chapter discusses the pavement performance of 12 SMA projects and selected companion conventional HMA mixture projects. The selected projects are shown previously in table 2.

The pavement performance analysis consisted of the following steps:

1. Review records for initial construction and subsequent M&R events of selected projects.
2. Collect pavement performance data from WisDOT databases.
3. Conduct performance analysis.

These steps are described in detail below.

5.2 REVIEW INITIAL CONSTRUCTION AND SUBSEQUENT M&R RECORDS (STEP 1)

The WisDOT databases listed and described in table 7 were reviewed to determine the pavement type and initial construction dates of the projects selected for the study.

Table 7. Summary of data provided by WisDOT.

Medium	File/Record Format	General Description
ACOfficeAll.xls	Microsoft® Excel	Historical construction/rehabilitation records
LayerReport.xls	Microsoft® Excel	Historical construction/rehabilitation records
Meta-Manager files	Microsoft® Excel	Roadway Worksheet (traffic data)
PCCOfficeAll.xls	Microsoft® Excel	Historical construction/rehabilitation records
Pavement Information Files (PIF) PIF_CD.mdb	Microsoft® Access	Various data tables. Includes the IRI and PDI test data (values) for each year of testing and the "Desc" table containing a description of pavement type for each sequence number
SectDescrip_proj.xls	Microsoft® Excel	Historical construction records including contract numbers for each year
Six-Year Highway Improvement Plan	Adobe® Acrobat	Reports listing future construction/rehabilitation activities.

5.3 COLLECT PAVEMENT PERFORMANCE DATA (STEP 2)

As described in chapter 3, a total of 12 SMA and companion HMA projects representing overlays on flexible and composite pavements were identified. Typically, each project was comprised of several WisDOT pavement management system (PMS) sections (i.e., sequence numbers). A summary of SMA and conventional HMA projects along with the PMS sections within the given projects is provided in table 8. Also, presented in this table is other descriptive information, such as construction date, traffic, etc.

For each of the identified PMS sections, performance data were collected and assembled for use in determining service life. Although, all available performance data (i.e., distress, composite indices, and IRI) were assembled, only the WisDOT composite index PDI and IRI were used in performance analysis. The reason for using both PDI and IRI was that they are the performance indicators that WisDOT uses in making decisions on when to rehabilitate or repair a segment of roadway. Also, both indices, when used in combination, represent the key distresses that are ultimately used to characterize pavement structural and functional condition.

Table 8. Summary of WisDOT projects and PMS sections within each given project.

Comparison No.	Site ID	Surface Type	Sequence No.	1-Way Init. AADT	1-Way Init. AADTT	Traffic Growth Rate	Percent Trucks	Construction Date
1	CSMA-1a	Conv	56300	16,000	2,160	2.3	13.5	1992
	CSMA-1b	Conv	57860	16,000	2,160	2.3	13.5	1992
	SMA-1a	SMA	56310	16,000	2,160	2.3	13.5	1992
	SMA-1a	SMA	56320	16,000	2,160	2.3	13.5	1992
	SMA-1b	SMA	57870	16,000	2,160	2.3	13.5	1992
	SMA-1b	SMA	57880	16,000	2,160	2.3	13.5	1992
2	CSMA-2a	Conv	56180	6,500	1,138	2.8	17.5	1993
	CSMA-2b	Conv	57740	6,500	1,138	2.8	17.5	1993
	SMA-2a	SMA	56130	6,500	1,138	2.8	17.5	1993
	SMA-2a	SMA	56140	6,500	1,138	2.8	17.5	1993
	SMA-2a	SMA	56150	6,500	1,138	2.8	17.5	1993
	SMA-2a	SMA	56160	6,500	1,138	2.8	17.5	1993
	SMA-2a	SMA	56170	6,500	1,138	2.8	17.5	1993
	SMA-2b	SMA	57700	6,500	1,138	2.8	17.5	1993
	SMA-2b	SMA	57710	6,500	1,138	2.8	17.5	1993
	SMA-2b	SMA	57720	6,500	1,138	2.8	17.5	1993
	SMA-2b	SMA	57730	6,500	1,138	2.8	17.5	1993
3	CSMA-3	Conv	85300	2,050	191	3.5	9.3	1993
	SMA-3	SMA	85240	1,500	141	3.8	9.4	1993
	SMA-3	SMA	85250	1,500	141	3.8	9.4	1993
	SMA-3	SMA	85260	1,500	141	3.8	9.4	1993
	SMA-3	SMA	85270	1,500	141	3.8	9.4	1993
	SMA-3	SMA	85280	1,500	141	3.8	9.4	1993

Table 8. Summary of WisDOT projects and PMS sections within each given project (continued).

Comparison No.	Site ID	Surface Type	Sequence No.	1-Way Init. AADT	1-Way Init. AADTT	Traffic Growth Rate	Percent Trucks	Construction Date
4	CSMA-4a	Conv	123470	9,500	694	2.5	7.3	1993
	CSMA-4a	Conv	123480	9,500	694	2.5	7.3	1993
	CSMA-4a	Conv	123490	9,500	694	2.5	7.3	1993
	CSMA-4b	Conv	123710	9,500	694	2.5	7.3	1993
	CSMA-4b	Conv	123720	9,500	694	2.5	7.3	1993
	CSMA-4b	Conv	123730	9,500	694	2.5	7.3	1993
	SMA-4a	SMA	113890	6,500	429	4.3	6.6	1993
	SMA-4b	SMA	114220	6,500	429	4.3	6.6	1993
5	CSMA-5	Conv	124580	3,850	358	3.8	9.3	1993
	SMA-5	SMA	124590	3,600	335	3.3	9.3	1993
	SMA-5	SMA	124600	3,600	335	3.3	9.3	1993
	SMA-5	SMA	124610	3,600	335	3.3	9.3	1993
	SMA-5	SMA	124620	3,600	335	3.3	9.3	1993
	SMA-5	SMA	124630	3,600	335	3.3	9.3	1993
6	CSMA-6	Conv	62050	1,750	163	4.0	9.3	1993
	SMA-6	SMA	62060	2,050	191	3.8	9.3	1993
	SMA-6	SMA	62070	2,050	191	3.8	9.3	1993
	SMA-6	SMA	62080	2,050	191	3.8	9.3	1993
	SMA-6	SMA	62090	2,050	191	3.8	9.3	1993
7	CSMA-7a	Conv	109660	75,000	4,725	1.8	6.3	1998
	CSMA-7a	Conv	109670	75,000	4,725	1.8	6.3	1998
	CSMA-7b	Conv	111760	75,000	4,725	1.8	6.3	1998
	SMA-7a	SMA	135310	63,000	4,914	1.8	7.8	1994
	SMA-7a	SMA	135320	63,000	4,914	1.8	7.8	1994
	SMA-7a	SMA	135330	63,000	4,914	1.8	7.8	1994
	SMA-7b	SMA	135400	63,000	4,914	1.8	7.8	1994
	SMA-7b	SMA	135410	63,000	4,914	1.8	7.8	1994
	SMA-7b	SMA	135420	63,000	4,914	1.8	7.8	1994
8	CSMA-8a	Conv	109680	77,500	4,960	1.3	6.4	1998
	CSMA-8a	Conv	109690	77,500	4,960	1.3	6.4	1998
	CSMA-8a	Conv	109700	77,500	4,960	1.3	6.4	1998
	CSMA-8a	Conv	109710	77,500	4,960	1.3	6.4	1998
	CSMA-8b	Conv	111770	77,500	4,960	1.3	6.4	1998
	CSMA-8b	Conv	111780	77,500	4,960	1.3	6.4	1998
	CSMA-8b	Conv	111790	77,500	4,960	1.3	6.4	1998
	CSMA-8b	Conv	111800	77,500	4,960	1.3	6.4	1998
	SMA-8a	SMA	56400	61,000	5,612	1.5	9.2	1994
	SMA-8a	SMA	56410	61,000	5,612	1.5	9.2	1994
	SMA-8a	SMA	56420	61,000	5,612	1.5	9.2	1994
	SMA-8a	SMA	56430	61,000	5,612	1.5	9.2	1994
	SMA-8b	SMA	57970	61,000	5,612	1.5	9.2	1994
	SMA-8b	SMA	57980	61,000	5,612	1.5	9.2	1994
	SMA-8b	SMA	57990	61,000	5,612	1.5	9.2	1994
	SMA-8b	SMA	58000	61,000	5,612	1.5	9.2	1994

Table 8. Summary of WisDOT projects and PMS sections within each given project (continued).

Comparison No.	Site ID	Surface Type	Sequence No.	1-Way Init. AADT	1-Way Init. AADTT	Traffic Growth Rate	Percent Trucks	Construction Date
9	CSMA-9	Conv	24130	1,450	178	5.0	12.3	1994
	CSMA-9	Conv	24140	1,450	178	5.0	12.3	1994
	CSMA-9	Conv	24150	1,450	178	5.0	12.3	1994
	CSMA-9	Conv	24160	1,450	178	5.0	12.3	1994
	SMA-9	SMA	24100	1,500	185	5.0	12.3	1994
	SMA-9	SMA	24110	1,500	185	5.0	12.3	1994
	SMA-9	SMA	24120	1,500	185	5.0	12.3	1994
10	CSMA-12a	Conv	52040	13,600	1,482	5.8	10.9	1990
	CSMA-12b	Conv	53810	13,600	1,482	5.8	10.9	1990
	SMA-12a	SMA	51850	22,500	2,543	4.5	11.3	1996
	SMA-12a	SMA	51860	22,500	2,543	4.5	11.3	1996
	SMA-12a	SMA	51870	22,500	2,543	4.5	11.3	1996
	SMA-12b	SMA	53620	22,500	2,543	4.5	11.3	1996
	SMA-12b	SMA	53630	22,500	2,543	4.5	11.3	1996
	SMA-12b	SMA	53640	22,500	2,543	4.5	11.3	1996
	SMA-12b	SMA	53650	22,500	2,543	4.5	11.3	1996
	SMA-12b	SMA	53660	22,500	2,543	4.5	11.3	1996
11	CSMA-13a	Conv	108450	10,000	2,220	4.0	22.2	1996
	CSMA-13a	Conv	108460	10,000	2,220	4.0	22.2	1996
	CSMA-13a	Conv	108470	10,000	2,220	4.0	22.2	1996
	CSMA-13a	Conv	108480	10,000	2,220	4.0	22.2	1996
	CSMA-13b	Conv	110560	10,000	2,220	4.0	22.2	1996
	CSMA-13b	Conv	110570	10,000	2,220	4.0	22.2	1996
	CSMA-13b	Conv	110580	10,000	2,220	4.0	22.2	1996
	CSMA-13b	Conv	110590	10,000	2,220	4.0	22.2	1996
	SMA-13a	SMA	108500	12,000	2,556	3.0	21.3	1997
	SMA-13a	SMA	108510	12,000	2,556	3.0	21.3	1997
	SMA-13a	SMA	108520	12,000	2,556	3.0	21.3	1997
	SMA-13a	SMA	108530	12,000	2,556	3.0	21.3	1997
	SMA-13a	SMA	108540	12,000	2,556	3.0	21.3	1997
	SMA-13a	SMA	108550	12,000	2,556	3.0	21.3	1997
	SMA-13b	SMA	110610	12,000	2,556	3.0	21.3	1997
	SMA-13b	SMA	110620	12,000	2,556	3.0	21.3	1997
	SMA-13b	SMA	110630	12,000	2,556	3.0	21.3	1997
	SMA-13b	SMA	110640	12,000	2,556	3.0	21.3	1997
	SMA-13b	SMA	110650	12,000	2,556	3.0	21.3	1997
12	CSMA-15	Conv	47430	2,800	204	5.5	7.3	1999
	CSMA-15	Conv	47440	2,800	204	5.5	7.3	1999
	CSMA-15	Conv	47450	2,800	204	5.5	7.3	1999
	CSMA-15	Conv	47460	2,800	204	5.5	7.3	1999
	CSMA-15	Conv	47470	2,800	204	5.5	7.3	1999
	CSMA-15	Conv	47480	2,800	204	5.5	7.3	1999
	SMA-15	SMA	34110	2,350	226	4.0	9.6	2000
	SMA-15	SMA	34120	2,350	226	4.0	9.6	2000
	SMA-15	SMA	34130	2,350	226	4.0	9.6	2000
	SMA-15	SMA	34140	2,350	226	4.0	9.6	2000

5.4 PERFORMANCE ANALYSIS (STEP 3)

Performance analysis consisted of the following steps:

- a. Group PMS sections according to pavement type, asphalt surfacing type (SMA versus HMA), functional class, traffic levels, layer thickness, etc.
- b. Determine PDI and IRI thresholds for each group.
- c. Create time-series plots of pavement performance and develop performance models.
- d. Use the performance models/trends and PDI- and IRI-based pavement performance thresholds to determine the service life of each PMS section.
- e. Conduct survival analysis for each group of projects and determine service lives.
- f. Compare survival life for the different asphalt surfacing types within each group.

5.4.1 Group PMS Sections for Analysis (Step 3a)

Pavement sections with similar designs (layer thicknesses and material types), traffic, and material characteristics were grouped for analysis. The groupings were done in order to identify pavements that were likely to have identical performance. Based on the parameters identified, three distinct groups of pavements were identified, which are listed in table 9.

Table 9. Description of pavement groups established for analysis.

Group	Pavement Type	Functional Class	Asphalt Surface Type	Baseline Trucks per Day
1	Flexible (Asphalt Overlay on Existing Asphalt Pavement)	U.S. and State Routes	HMA	220
			SMA	
2	Composite (Asphalt Overlay on Existing JRC Pavement)	Interstate and U.S. Routes	HMA	4,658
			SMA	
3	Composite (Asphalt Overlay on Existing JRC Pavement)	U.S. and State Routes	HMA	1,905
			SMA	

5.4.2 Determine PDI and IRI Thresholds for Each Group (Step 3b)

PDI and IRI thresholds were determined for all three groups, based on WisDOT guidelines and practices (i.e., functional class). These threshold values are listed in table 10. PDI is an overall pavement condition indicator that takes into account a variety of distresses, including fatigue cracking, rutting, longitudinal cracking, transverse cracking, and edge raveling. Dual consideration of PDI and IRI was considered to be more than adequate for analyzing performance in this study.

Table 10. WisDOT threshold PDI and IRI values.

Functional Class	Performance Indicator Threshold Values	
	Distress (PDI)	(Functional) IRI, in/mi
Interstate and U.S. highways	85	155
State routes	90	220

5.4.3 Create Time-Series Plots of Pavement Performance and Develop Performance Models (Step 3c)

Plots of PDI and IRI versus pavement age were developed for each section. These plots were used to (1) select appropriate model forms for curve fitting and (2) determine model coefficients for the model forms selected to relate PDI and IRI to age. Different model types were considered for this study, but a simple linear model was found to be as accurate as the more complicated power law models. The Asphalt Institute in some of their studies also used a linear model in extrapolating pavement condition index data to determine the service lives of asphalt pavements. The linear models used and their coefficients are listed below for IRI and PDI:

$$IRI = \alpha Age + \beta \qquad \text{Eq. 3}$$

$$PDI = \gamma Age + \eta \qquad \text{Eq. 4}$$

where: IRI = International Roughness Index, in/mi.
PDI = Pavement distress index.
Age = Pavement age, years.
α, β, γ, η = Regression constants

Examples of the plots and corresponding models are presented in figures 3 and 4 for WisDOT sequence number 56420 (SMA surface). Table 11 lists the model coefficients for the IRI and PDI models developed for each PMS section. Finally, figures 5 and 6 are plots of predicted versus measured IRI and PDI, respectively, based on the models given in table 11.

5.4.4 Determine Service Life (Step 3d)

The service life of each PMS section was determined as follows:

- For pavement sections where the PDI or IRI threshold values were exceeded, the service life was determined as the age when the terminal PDI or IRI value was reached. Linear interpolation was used to determine the specific age. Where both threshold values were reached, the lower of the two ages was used as the service life.

- For pavement sections where the PDI or IRI threshold values were not reached, the service life was determined by using the linear models (equations 3 and 4) to forecast or extrapolate future performance. The predicted performance was used to estimate the age at which the terminal PDI or IRI value is exceeded. When both threshold values were exceeded, the lower of the two ages was used as the service life.

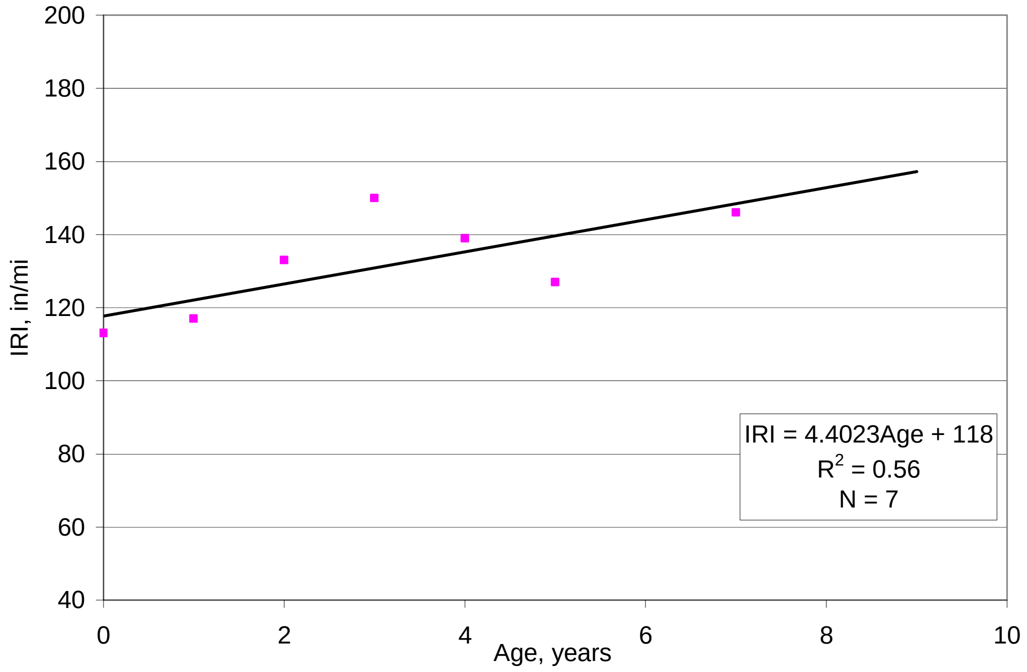


Figure 3. Plot of IRI versus age for WisDOT PMS section 56420.

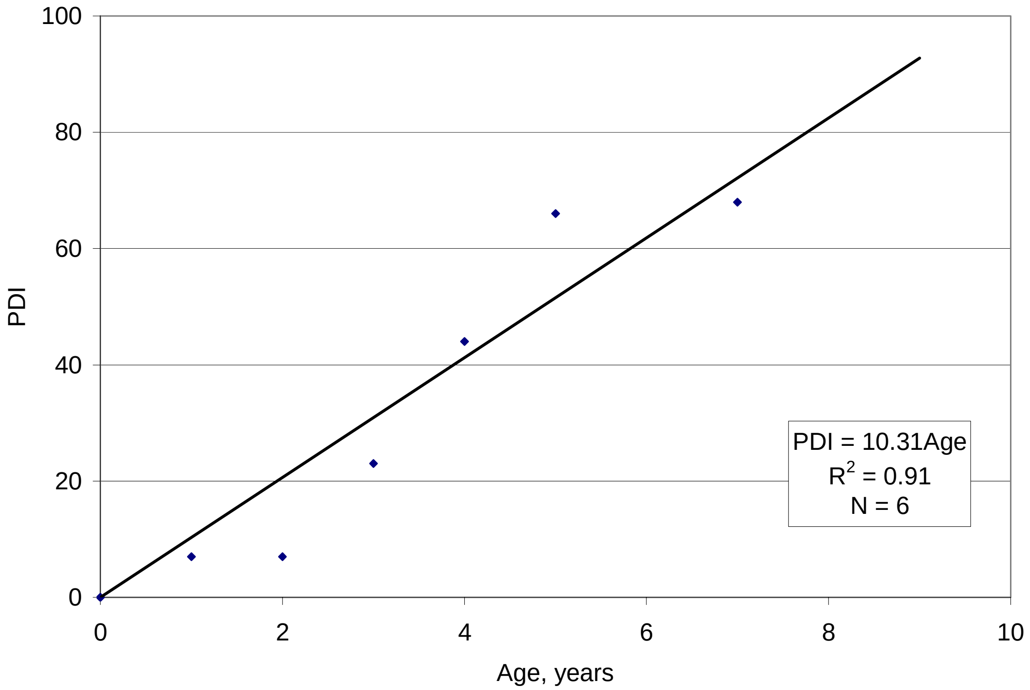


Figure 4. Plot of PDI versus age for WisDOT PMS section 56420.

Table 11. PDI and IRI linear model coefficients.

Comp.	Site ID	Surface Type	Seq. No.	PDI				IRI			
				Intercept (η)	Slope (γ)	N	R^2	Intercept (β)	Slope (α)	N	R^2
1	CSMA-1a	Conv	56300	0.0	3.4	10	0.43	54.8	3.5	8	0.90
1	SMA-1a	SMA	56310	0.0	4.8	11	0.62	70.4	3.7	9	0.91
1	SMA-1a	SMA	56320	0.0	4.2	11	0.48	56.1	2.6	9	0.68
1	CSMA-1b	Conv	57860	0.0	2.3	10	0.35	62.0	1.9	9	0.58
1	SMA-1b	SMA	57870	0.0	5.0	10	0.61	69.9	0.9	8	0.40
1	SMA-1b	SMA	57880	0.0	4.0	11	0.55	53.7	2.8	9	0.84
2	SMA-2a	SMA	56130	0.0	5.7	10	0.90	64.5	1.9	8	0.44
2	SMA-2a	SMA	56140	0.0	1.9	10	0.28	75.0	2.1	8	0.61
2	SMA-2a	SMA	56150	0.0	2.6	10	0.32	65.7	0.7	8	0.05
2	SMA-2a	SMA	56160	0.0	4.1	10	0.62	53.0	2.0	8	0.67
2	SMA-2a	SMA	56170	0.0	4.8	10	0.75	59.8	2.1	8	0.47
2	CSMA-2a	Conv	56180	0.0	4.2	9	0.74	40.7	3.3	8	0.67
2	SMA-2b	SMA	57700	0.0	3.7	10	0.66	70.1	2.3	8	0.79
2	SMA-2b	SMA	57710	0.0	4.0	10	0.64	73.0	1.6	8	0.40
2	SMA-2b	SMA	57720	0.0	6.1	9	0.77	50.3	2.5	8	0.46
2	SMA-2b	SMA	57730	0.0	3.7	9	0.63	45.4	2.8	8	0.73
2	CSMA-2b	Conv	57740	0.0	6.0	8	0.87	48.2	4.5	8	0.78
3	SMA-3	SMA	85240	0.0	6.3	5	0.99	67.9	0.3	3	0.03
3	SMA-3	SMA	85250	0.0	3.9	6	0.66	57.0	1.5	3	0.93
3	SMA-3	SMA	85260	0.0	5.0	6	0.63	76.4	-0.2	3	0.09
3	SMA-3	SMA	85270	0.0	1.4	5	0.60	72.1	-0.4	3	0.66
3	SMA-3	SMA	85280	0.0	2.5	6	0.63	50.1	0.7	3	0.29
3	CSMA-3	Conv	85300	0.0	2.2	6	0.50	45.9	1.6	3	0.94
4	SMA-4a	SMA	113890	0.0	3.1	7	0.92	93.3	2.8	3	0.62
4	SMA-4b	SMA	114220	0.0	2.8	7	0.87	85.3	3.8	3	0.75
4	CSMA-4a	Conv	123470	0.0	2.2	7	0.94	33.3	4.0	3	0.98
4	CSMA-4a	Conv	123480	0.0	2.3	7	0.93	43.1	7.3	3	0.99
4	CSMA-4a	Conv	123490	0.0	2.3	7	0.94	24.8	6.5	3	0.81
4	CSMA-4b	Conv	123710	0.0	2.4	6	0.49	82.7	1.0	3	0.04
4	CSMA-4b	Conv	123720	0.0	2.4	6	0.60	81.8	5.5	3	0.96
4	CSMA-4b	Conv	123730	0.0	2.3	6	0.90	82.6	1.8	3	0.22
5	CSMA-5	Conv	124580	0.0	7.2	5	0.84	49.7	5.3	2	1.00
5	SMA-5	SMA	124590	0.0	6.7	6	0.93	76.7	4.7	3	0.99
5	SMA-5	SMA	124600	0.0	3.1	6	0.94	88.0	2.8	3	1.00
5	SMA-5	SMA	124610	0.0	3.7	6	0.79	71.5	5.1	3	1.00
5	SMA-5	SMA	124620	0.0	2.6	6	0.96	74.0	5.2	3	0.98
5	SMA-5	SMA	124630	0.0	5.8	6	0.85	73.6	3.9	3	0.99
6	CSMA-6	Conv	62050	0.0	1.6	6	0.58	52.5	3.5	2	1.00
6	SMA-6	SMA	62060	0.0	1.3	7	0.47	86.9	-0.3	3	0.00
6	SMA-6	SMA	62070	0.0	1.6	6	0.61	NA	NA	NA	NA
6	SMA-6	SMA	62080	0.0	2.0	5	0.52	NA	NA	NA	NA
6	SMA-6	SMA	62090	0.0	1.6	6	0.49	NA	NA	NA	NA

NA = not available.

Table 11. PDI and IRI linear model coefficients (continued).

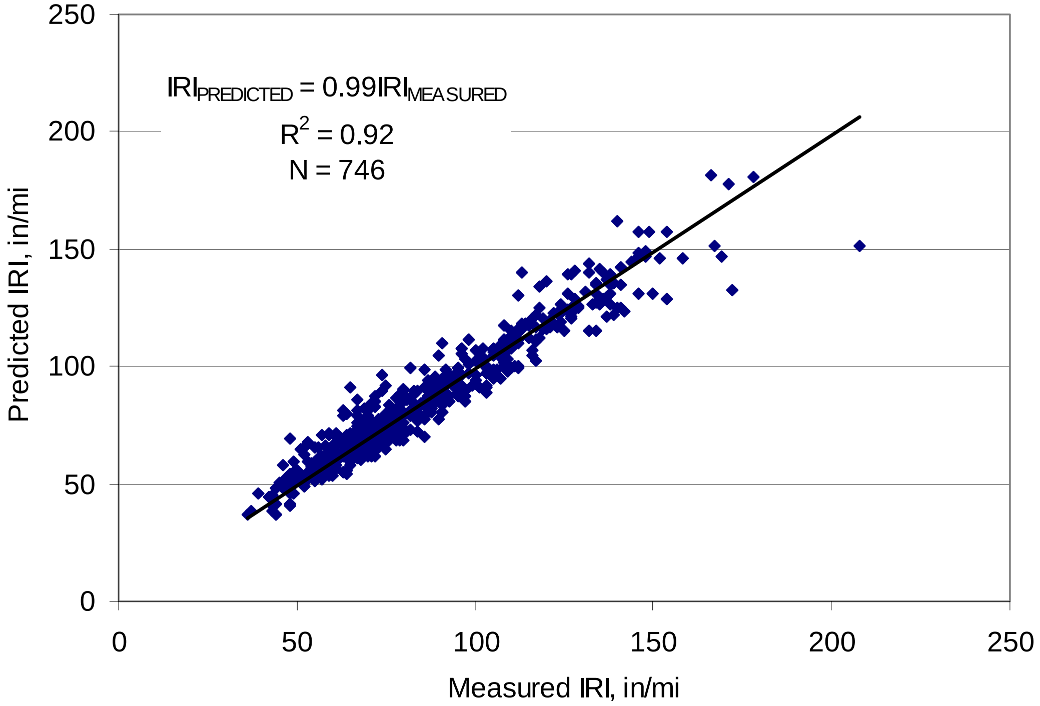
Comp.	Site ID	Surface Type	Seq. No.	PDI				IRI			
				Intercept (η)	Slope (γ)	N	R^2	Intercept (β)	Slope (α)	N	R^2
7	CSMA-7a	Conv	109660	0.0	1.9	4	0.06	69.7	2.6	4	0.37
7	CSMA-7a	Conv	109670	0.0	3.9	4	0.31	77.0	2.4	3	0.54
7	CSMA-7b	Conv	111760	0.0	9.1	6	0.84	68.1	3.0	6	0.59
7	SMA-7a	SMA	135310	0.0	10.3	8	0.89	117.3	5.8	8	0.92
7	SMA-7a	SMA	135320	0.0	10.1	8	0.91	105.2	4.9	8	0.59
7	SMA-7a	SMA	135330	0.0	9.8	8	0.91	101.9	4.7	7	0.88
7	SMA-7b	SMA	135400	0.0	7.9	7	0.77	106.8	9.6	6	0.98
7	SMA-7b	SMA	135410	0.0	8.4	8	0.85	107.4	4.8	8	0.51
7	SMA-7b	SMA	135420	0.0	7.6	8	0.79	106.0	4.8	8	0.57
8	SMA-8a	SMA	56400	0.0	9.7	7	0.80	99.7	11.6	7	0.95
8	SMA-8a	SMA	56410	0.0	9.0	8	0.88	105.9	10.3	8	0.85
8	SMA-8a	SMA	56420	0.0	10.3	6	0.91	117.6	4.4	7	0.56
8	SMA-8a	SMA	56430	0.0	8.3	8	0.91	111.3	5.7	8	0.42
8	SMA-8b	SMA	57970	0.0	9.7	8	0.91	111.5	10.1	8	0.49
8	SMA-8b	SMA	57980	0.0	9.4	8	0.90	117.2	3.8	8	0.23
8	SMA-8b	SMA	57990	0.0	9.9	8	0.90	118.1	4.4	8	0.80
8	SMA-8b	SMA	58000	0.0	3.2	8	0.93	108.3	0.7	7	0.22
8	CSMA-8a	Conv	109680	0.0	6.9	4	0.36	93.0	2.5	3	0.48
8	CSMA-8a	Conv	109690	0.0	5.0	4	0.60	101.5	1.2	3	0.21
8	CSMA-8a	Conv	109700	0.0	7.6	4	0.85	78.3	4.2	4	0.61
8	CSMA-8a	Conv	109710	0.0	8.6	3	0.57	148.0	-0.3	3	0.25
8	CSMA-8b	Conv	111770	0.0	4.4	5	0.57	70.9	5.3	6	0.81
8	CSMA-8b	Conv	111780	0.0	8.8	6	0.63	97.3	1.6	6	0.24
8	CSMA-8b	Conv	111790	0.0	5.6	6	0.56	88.8	2.6	6	0.66
8	CSMA-8b	Conv	111800	0.0	9.4	6	0.71	84.2	3.8	6	0.58
9	SMA-9	SMA	24100	0.0	6.0	6	0.95	66.7	0.8	5	0.37
9	SMA-9	SMA	24110	0.0	6.0	6	0.89	54.8	3.5	5	0.64
9	SMA-9	SMA	24120	0.0	6.5	6	0.93	64.0	1.6	5	0.68
9	CSMA-9	Conv	24130	0.0	5.3	6	0.80	49.0	6.0	5	0.92
9	CSMA-9	Conv	24140	0.0	1.6	6	0.62	47.3	5.3	6	0.80
9	CSMA-9	Conv	24150	0.0	2.5	6	0.50	48.9	5.1	6	0.87
9	CSMA-9	Conv	24160	0.0	1.8	6	0.29	42.4	5.7	6	0.82
10	SMA-12a	SMA	51850	0.0	0.3	4	0.07	89.5	1.2	4	0.31
10	SMA-12a	SMA	51860	0.0	3.8	4	0.60	56.5	2.6	4	0.36
10	SMA-12a	SMA	51870	0.0	5.8	4	0.72	59.5	4.4	4	0.42
10	CSMA-12a	Conv	52040	0.0	3.3	8	0.62	59.7	2.0	3	0.99
10	SMA-12b	SMA	53620	0.0	1.8	3	0.85	65.5	7.5	4	0.60
10	SMA-12b	SMA	53630	0.0	6.9	4	0.60	79.5	4.7	4	0.75
10	SMA-12b	SMA	53640	0.0	1.5	3	0.63	53.5	4.7	4	0.56
10	SMA-12b	SMA	53650	0.0	0.0	4	0.01	88.5	0.7	4	0.10
10	SMA-12b	SMA	53660	0.0	0.1	4	0.00	78.0	0.0	4	0.00
10	CSMA-12b	Conv	53810	0.0	5.0	9	0.79	67.1	2.7	4	0.65

NA = not available.

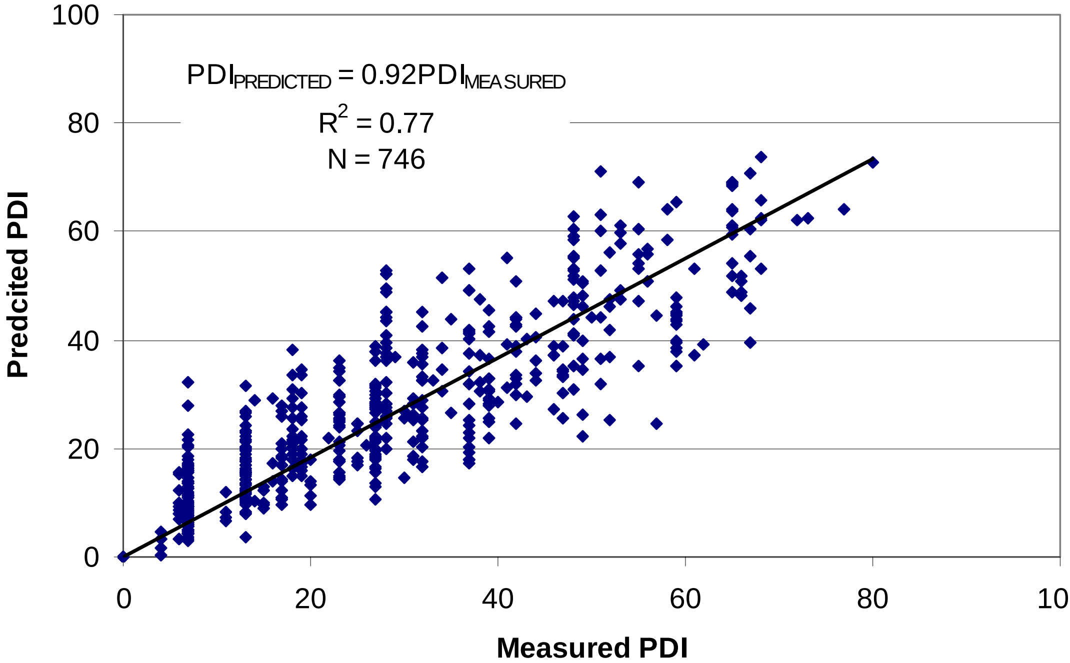
Table 11. PDI and IRI linear model coefficients (continued).

Comp.	Site ID	Surface Type	Seq. No.	PDI				IRI			
				Intercept (η)	Slope (γ)	N	R^2	Intercept (β)	Slope (α)	N	R^2
11	CSMA-13a	Conv	108450	0.0	7.5	7	0.73	58.8	6.9	7	0.86
11	CSMA-13a	Conv	108460	0.0	4.3	7	0.60	47.5	7.6	7	0.82
11	CSMA-13a	Conv	108470	0.0	9.4	7	0.90	51.4	4.8	7	0.96
11	CSMA-13a	Conv	108480	0.0	6.4	7	0.60	64.2	5.4	7	0.90
11	SMA-13a	SMA	108500	0.0	3.5	6	0.19	52.3	13.1	6	0.79
11	SMA-13a	SMA	108510	0.0	6.1	6	0.57	42.2	13.6	6	0.76
11	SMA-13a	SMA	108520	0.0	3.4	6	0.43	43.4	11.9	6	0.68
11	SMA-13a	SMA	108530	0.0	2.4	6	0.22	42.0	7.1	6	0.80
11	SMA-13a	SMA	108540	0.0	2.9	5	0.85	49.2	3.8	6	0.77
11	SMA-13a	SMA	108550	0.0	1.3	6	0.57	57.1	0.6	6	0.03
11	CSMA-13b	Conv	110560	0.0	6.9	6	0.96	46.1	7.4	6	0.78
11	CSMA-13b	Conv	110570	0.0	5.3	6	0.74	30.4	10.2	6	0.79
11	CSMA-13b	Conv	110580	0.0	8.4	6	0.90	39.9	6.0	6	0.86
11	CSMA-13b	Conv	110590	0.0	5.1	6	0.65	49.7	5.9	6	0.64
11	SMA-13b	SMA	110610	0.0	5.5	6	0.57	56.6	4.0	6	0.67
11	SMA-13b	SMA	110620	0.0	0.2	6	0.01	28.5	9.8	6	0.83
11	SMA-13b	SMA	110630	0.0	1.5	6	0.69	26.9	9.7	6	0.77
11	SMA-13b	SMA	110640	0.0	6.0	6	0.84	34.0	7.8	6	0.81
11	SMA-13b	SMA	110650	0.0	3.6	6	0.47	31.6	9.5	6	0.68
12	SMA-15	SMA	34110	NA	NA	NA	NA	87.0	9.0	2	1.00
12	SMA-15	SMA	34120	NA	NA	NA	NA	75.0	9.5	2	1.00
12	SMA-15	SMA	34130	0.0	3.5	2	1.00	80.0	6.5	2	1.00
12	SMA-15	SMA	34140	0.0	3.5	2	1.00	88.0	3.0	2	1.00
12	CSMA-15	Conv	47430	0.0	8.0	3	0.90	38.2	3.8	3	0.93
12	CSMA-15	Conv	47440	0.0	8.0	3	0.86	49.0	2.5	3	0.68
12	CSMA-15	Conv	47450	NA	NA	NA	NA	51.3	5.0	3	0.86
12	CSMA-15	Conv	47460	0.0	6.8	3	0.75	44.7	4.5	3	0.79
12	CSMA-15	Conv	47470	0.0	3.3	3	1.00	37.2	1.8	3	0.75
12	CSMA-15	Conv	47480	0.0	11.5	3	0.99	45.3	3.5	3	0.75

NA = not available.



Figures 5. Plot of predicted versus measured IRI (for all WisDOT pavement sections analyzed).



Figures 6. Plot of predicted versus measured PDI (for all WisDOT pavement sections analyzed).

Table 12 presents the pavement service lives determined for each PMS section and the performance indicator threshold that governed the service life estimates. Note that there were many sections where the service live estimate exceeded 20 years. Because extrapolations past 20 years were considered unreliable, the value of 20 years was assigned to those sections.

Table 12. Estimates of pavement service life for each PMS section.

Group	Comparison	Seq. No.	Age, years	Comment
1	3	85240	14.1	PDI
	3	85250	19.3	PDI
	3	85260	15.6	PDI
	3	85270	20*	Censored
	3	85280	20*	Censored
	3	85300	20*	Censored
	5	124580	12.7	PDI
	5	124590	14.4	PDI
	5	124600	20*	Censored
	5	124610	20*	Censored
	5	124620	20*	Censored
	5	124630	15.9	PDI
	6	62050	20*	Censored
	6	62060	20*	Censored
	6	62070	20*	Censored
	6	62080	20*	Censored
	6	62090	20*	Censored
	9	24100	15.7	PDI
	9	24110	16.6	PDI
	9	24120	14.6	PDI
	9	24130	17.2	PDI
	9	24140	20*	Censored
	9	24150	20*	Censored
	9	24160	20*	Censored
	12	34110	14.8	IRI
	12	34120	15.3	IRI
	12	34130	20*	Censored
	12	34140	20*	Censored
	12	47430	11.6	PDI
	12	47440	10.8	PDI
	12	47450	20*	Censored
	12	47460	12.7	PDI
	12	47470	20*	Censored
	12	47480	7.7	PDI

Table 12. Estimates of pavement service life for each PMS section (continued).

Group	Comparison	Seq. No.	Age, years	Comment
2	7	109660	20*	Censored
	7	109670	17.5	PDI
	7	111760	9.2	PDI
	7	135310	6.6	IRI
	7	135320	8.6	PDI
	7	135330	9.0	PDI
	7	135400	5.1	IRI
	7	135410	10.0	IRI
	7	135420	10.2	IRI
	8	56400	4.8	IRI
	8	56410	4.8	IRI
	8	56420	8.0	PDI
	8	56430	7.6	IRI
	8	57970	4.3	IRI
	8	57980	9.6	PDI
	8	57990	8.4	IRI
	8	58000	20*	Censored
	8	109680	10.6	PDI
	8	109690	14.6	PDI
	8	109700	10.2	PDI
	8	109710	9.1	PDI
	8	111770	15.9	IRI
	8	111780	8.8	PDI
	8	111790	14.1	PDI
	8	111800	8.3	PDI
	10	51850	20*	Censored
	10	51860	20*	Censored
	10	51870	15.6	PDI
	10	52040	20*	Censored
	10	53620	12.0	IRI
	10	53630	13.8	PDI
	10	53640	20*	Censored
	10	53650	20*	Censored
	10	53660	20*	Censored
	10	53810	16.8	PDI

Table 12. Estimates of pavement service life for each PMS section (continued).

Group	Comparison	Seq. No.	Age, years	Comment
3	1	56300	18.7	PDI
	1	56310	15.4	PDI
	1	56320	15.8	PDI
	1	57860	20*	Censored
	1	57870	14.5	PDI
	1	57880	18.1	PDI
	4	113890	20*	Censored
	4	114220	18.6	IRI
	4	123470	20*	Censored
	4	123480	15.4	IRI
	4	123490	20*	Censored
	4	123710	20*	Censored
	4	123720	13.3	IRI
	4	123730	20*	Censored
	11	108450	10.4	PDI
	11	108460	14.2	IRI
	11	108470	9.1	PDI
	11	108480	12.3	PDI
	11	108500	7.9	IRI
	11	108510	8.3	IRI
	11	108520	9.4	IRI
	11	108530	15.9	IRI
	11	108540	20*	Censored
	11	108550	20*	Censored
	11	110560	10.8	PDI
	11	110570	12.2	IRI
	11	110580	9.5	PDI
	11	110590	14.4	PDI
	11	110610	13.4	PDI
	11	110620	12.9	IRI
	11	110630	13.2	IRI
	11	110640	15.2	PDI
	11	110650	13.0	IRI

5.4.5 Conduct Survival Analysis (Step 3e)

Survival analysis has been used extensively in pavement evaluations to study the effect of site conditions, design features, construction techniques, maintenance treatments, and rehabilitation activities on pavement service life. In survival analysis, data associated with the time (measured in terms of pavement age or truck traffic applications) to a major cost event (overlay or reconstruction) or to a terminal/threshold performance indicator level is used in a non-parametric or parametric procedure to estimate a survival distribution function, conventionally denoted by S, and defined as follows:

$$S(t) = Pr(T>t) \qquad \text{Eq. 5}$$

where: t = Time or age of pavement (or cumulative number of truck loadings).
T = Time or age of pavement at failure (or cumulative number of truck loadings at failure).
Pr = Probability.

Survival functions have the following characteristics:

1. It assumes that $S(0) = 1$ (although it could be less than 1 if there is the possibility of immediate pavement failure due to construction error).
2. Survival probability decreases with increasing life (i.e., $S(u) < S(t)$ if $u > t$). This expresses the notion that survival is only less probable as the pavement ages or as more trucks are applied to the pavement.
3. Survival probability is usually assumed to approach zero as pavement age or traffic applications increases without bound (i.e., $S[t] \rightarrow 0$ as t [measured as pavement age of the number of truck applications] $\rightarrow \infty$).

For this study, the parametric (Kaplan-Meier) and non-parametric (actuarial) approaches were used to develop survival functions using the SAS© statistical package and the estimates of service life presented in table 12. For service life estimates greater than 20 years (refer to table 12), the given pavement section was assumed to still be in service after 20 years, thereby causing it to be censored.

Figures 7 through 12 present the plots of the survival functions developed in this study (one each for the combination of groups and surface types) for determining service life in years. Similar plots corresponding to cumulative truck traffic are provided in figures 13 through 18. All results are summarized in tables 13 and 14 for age and traffic, respectively.

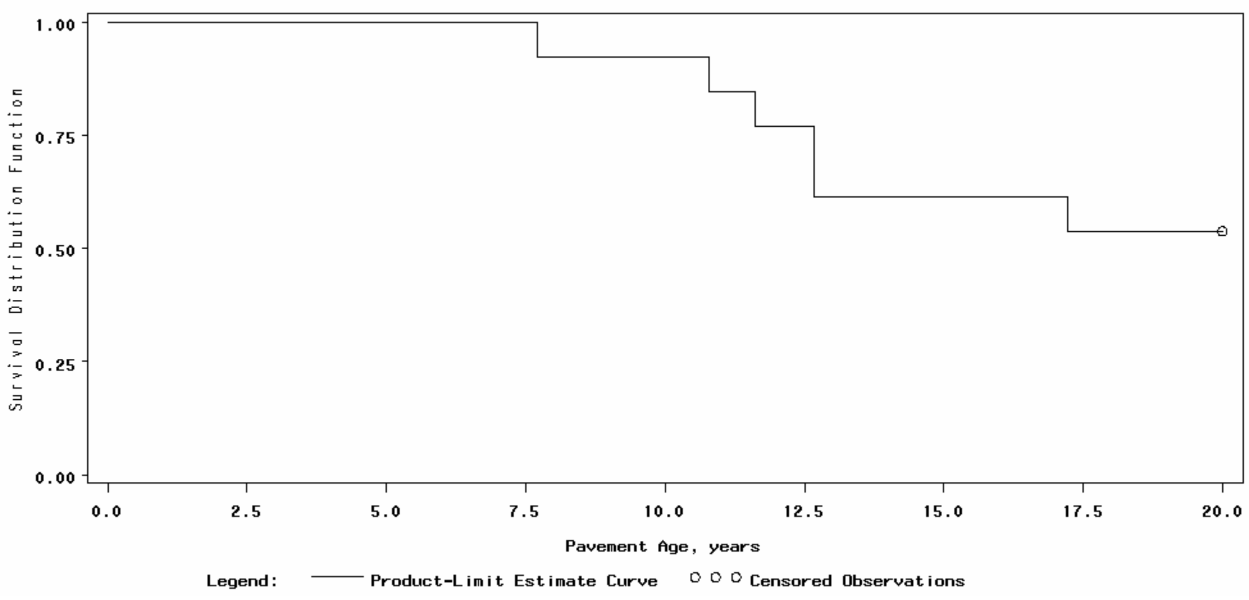


Figure 7. Age-based survival distribution function for group 1 (HMA surfacing).

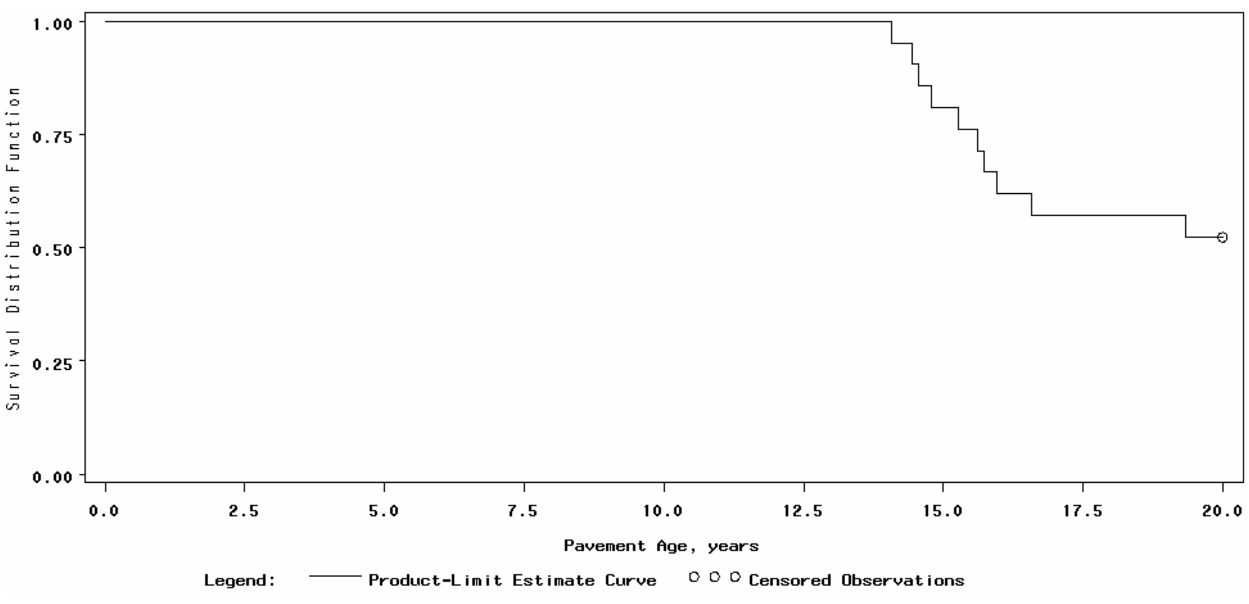


Figure 8. Age-based survival distribution function for group 1 (SMA surfacing).

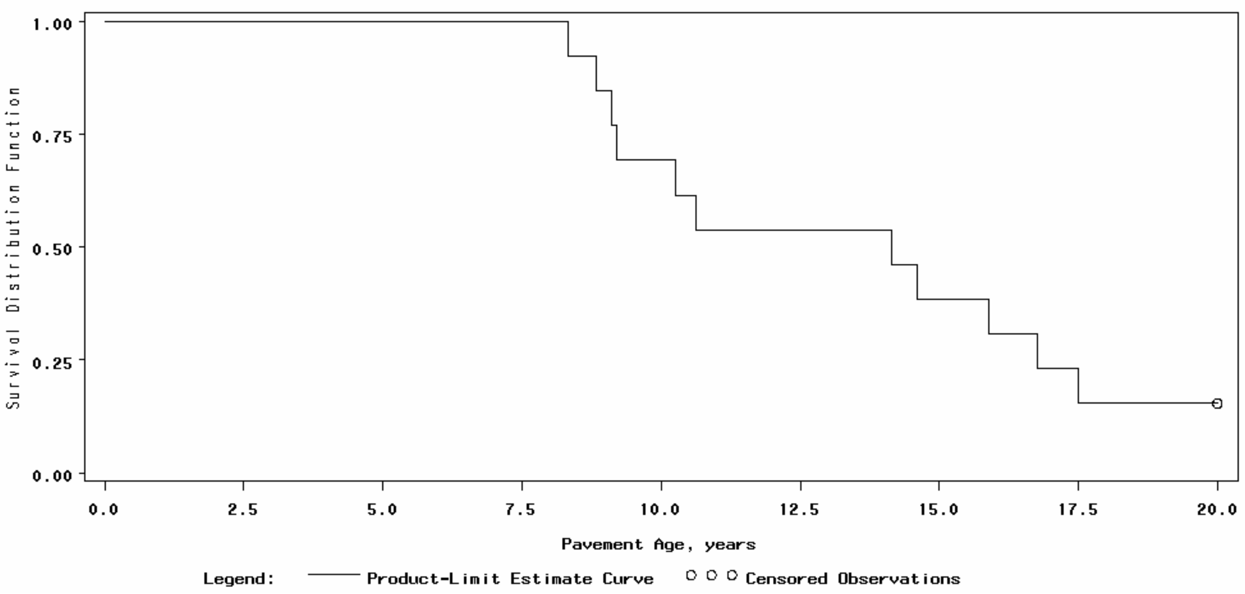


Figure 9. Age-based survival distribution function for group 2 (HMA surfacing).

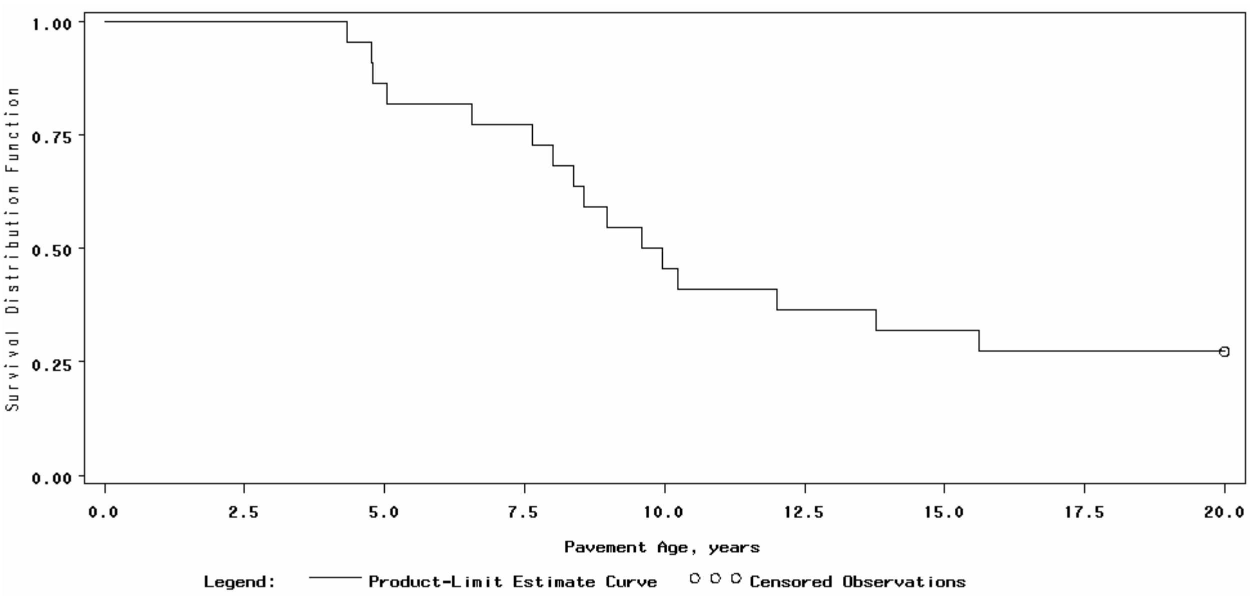


Figure 10. Age-based survival distribution function for group 2 (SMA surfacing).

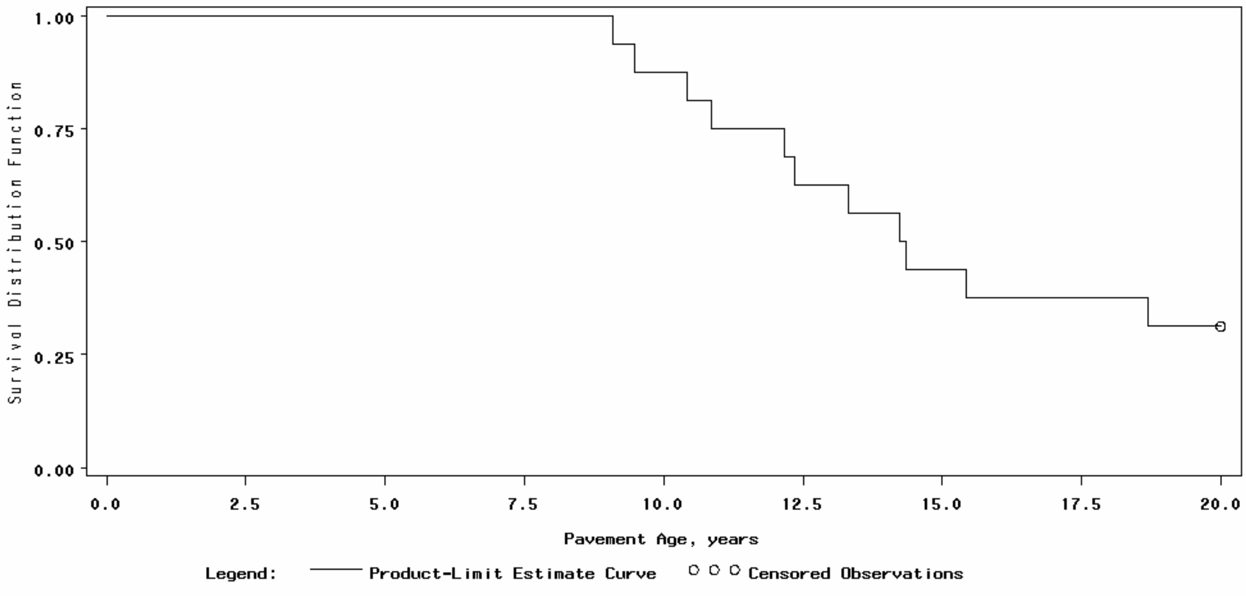


Figure 11. Age-based survival distribution function for group 3 (HMA surfacing).

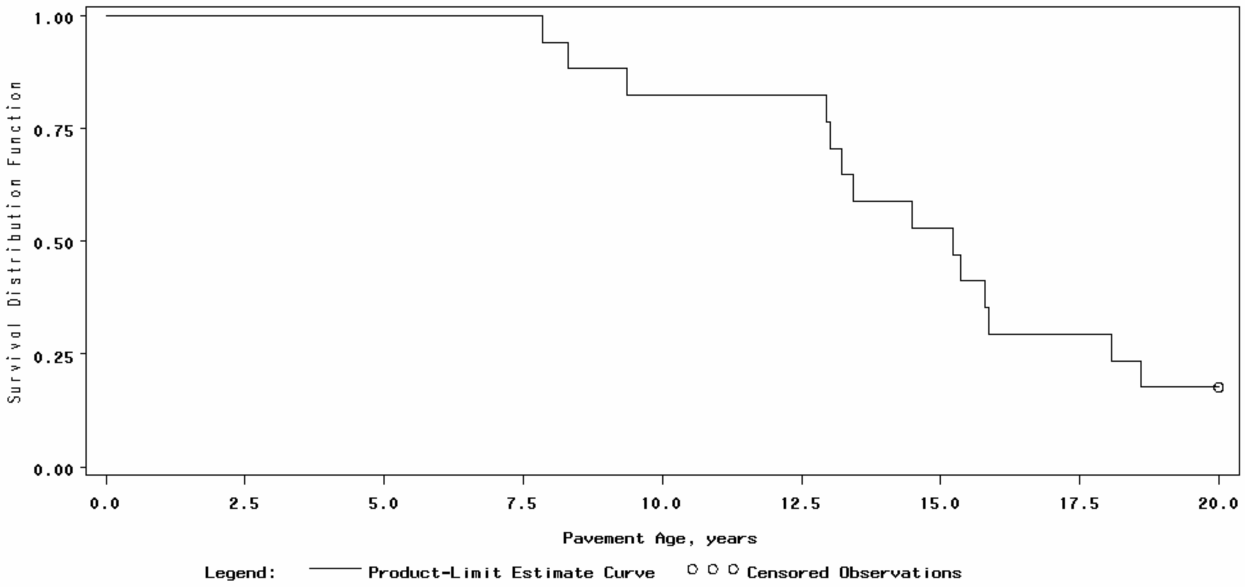


Figure 12. Age-based survival distribution function for group 3 (SMA surfacing).

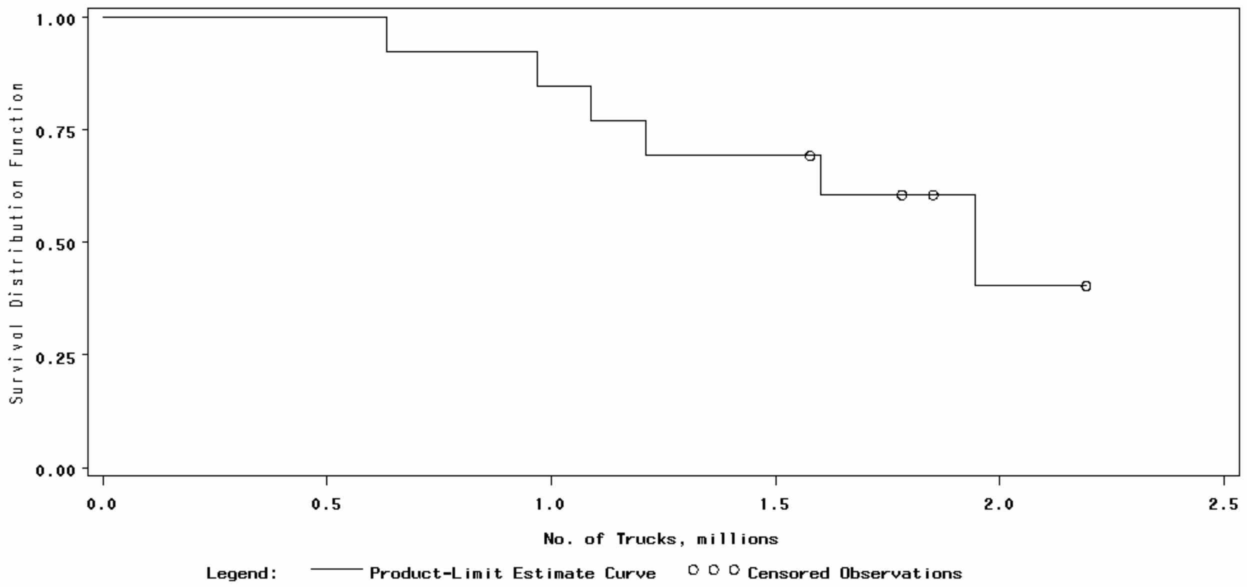


Figure 13. Traffic-based survival distribution function for group 1 (HMA surfacing).

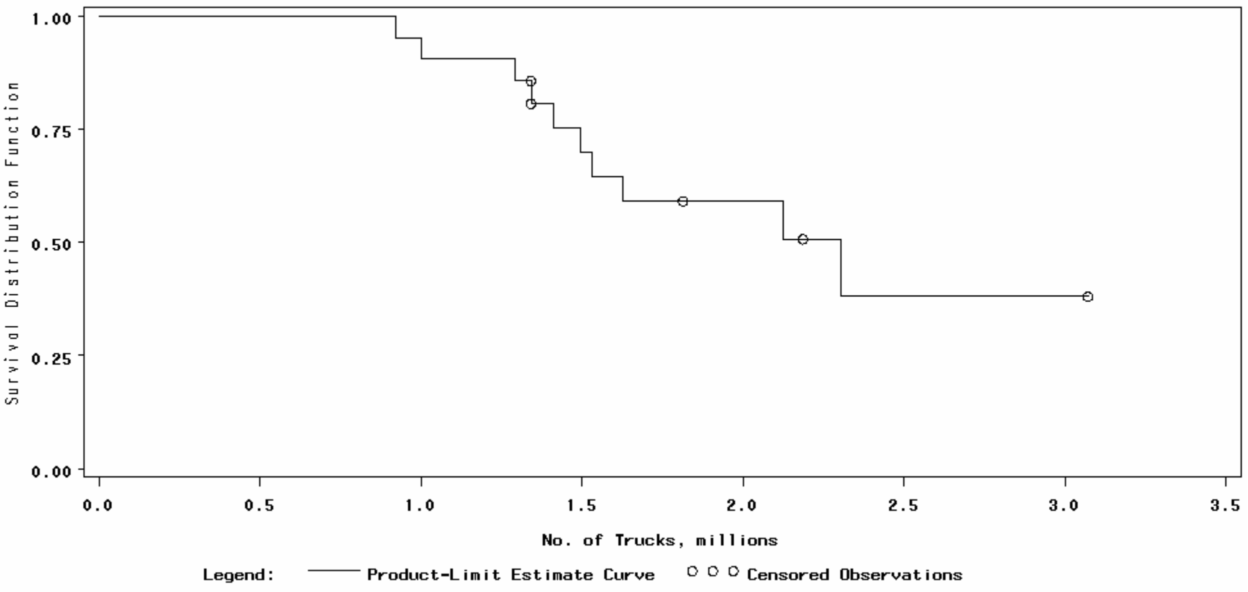


Figure 14. Traffic-based survival distribution function for group 1 (SMA surfacing).

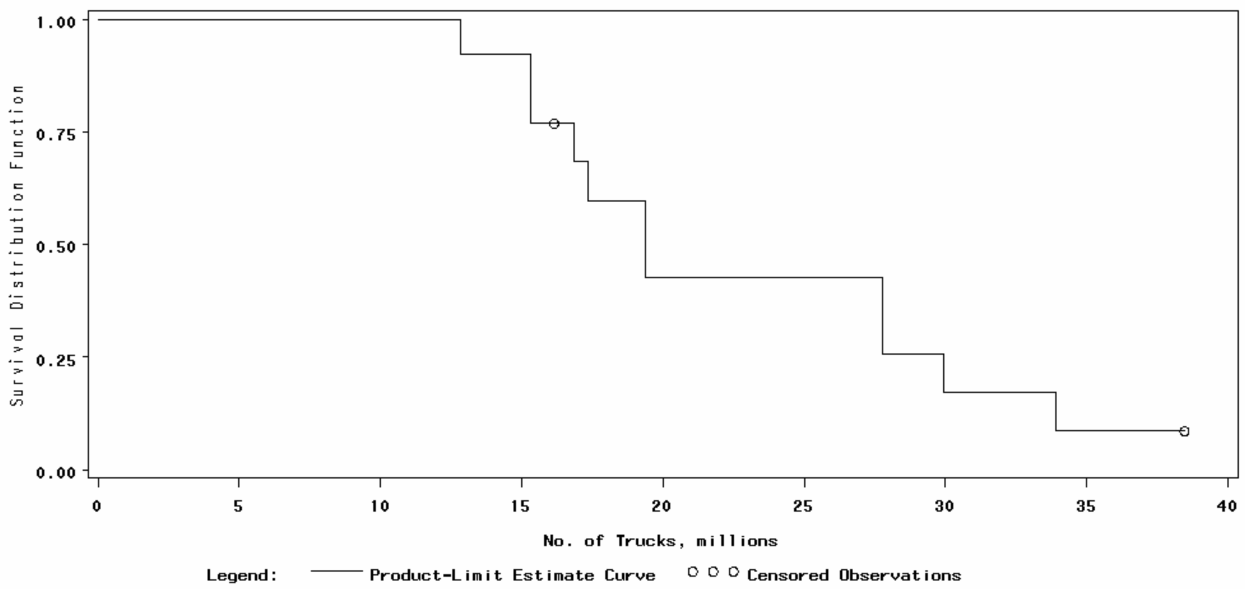


Figure 15. Traffic-based survival distribution function for group 2 (HMA surfacing).

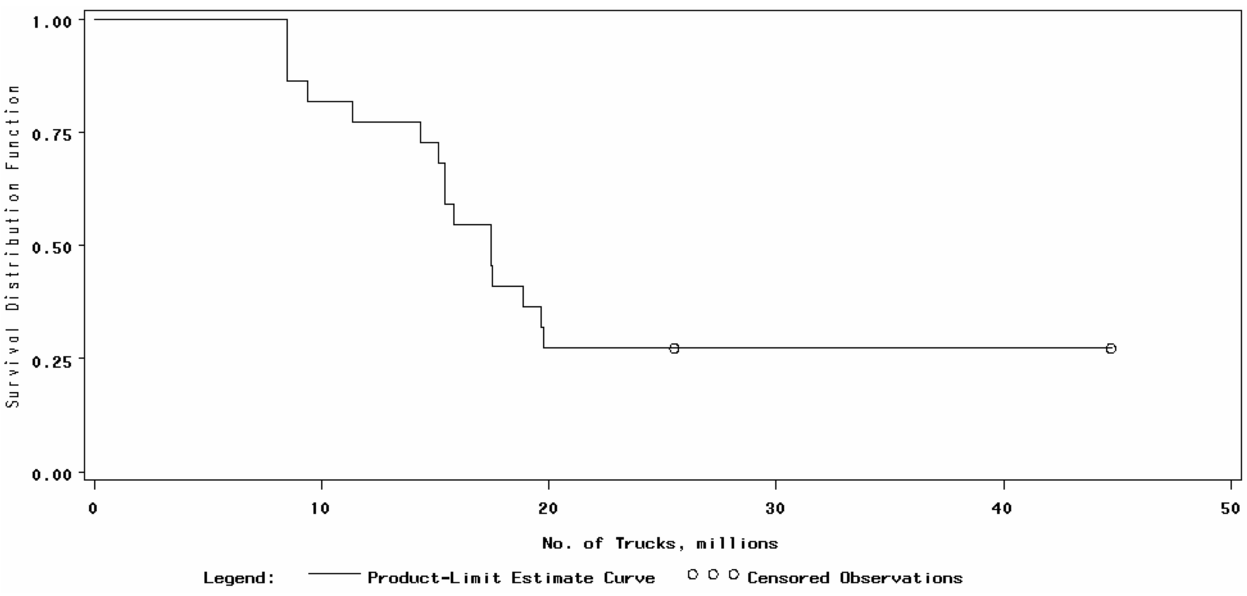


Figure 16. Traffic-based survival distribution function for group 2 (SMA surfacing).

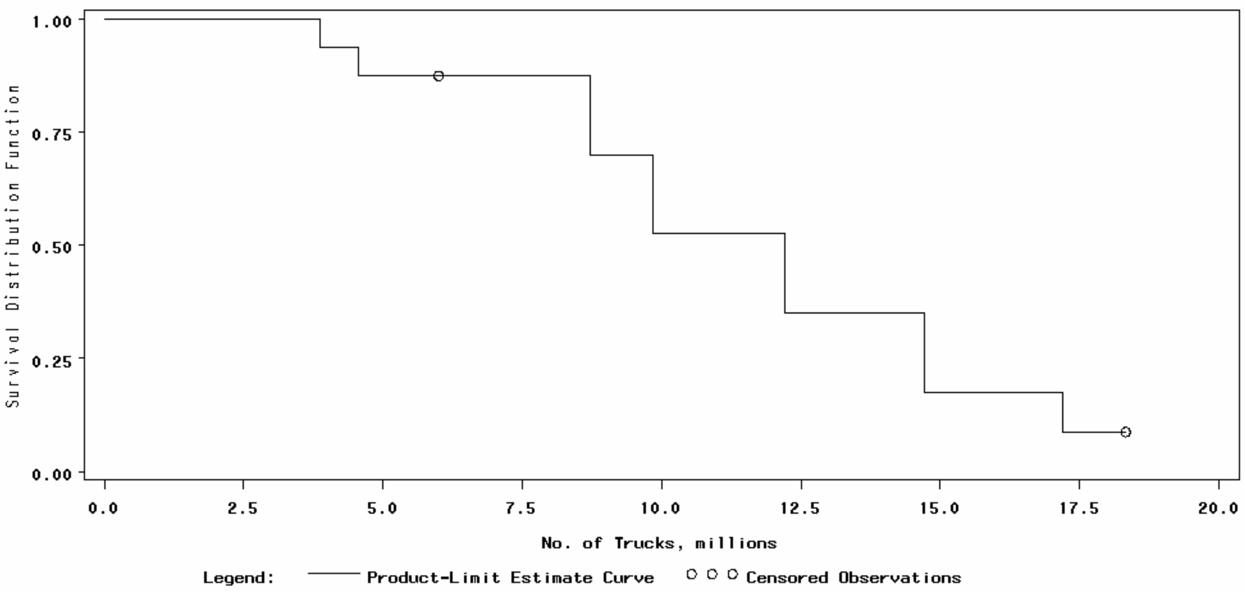


Figure 17. Traffic-based survival distribution function for group 3 (HMA surfacing).

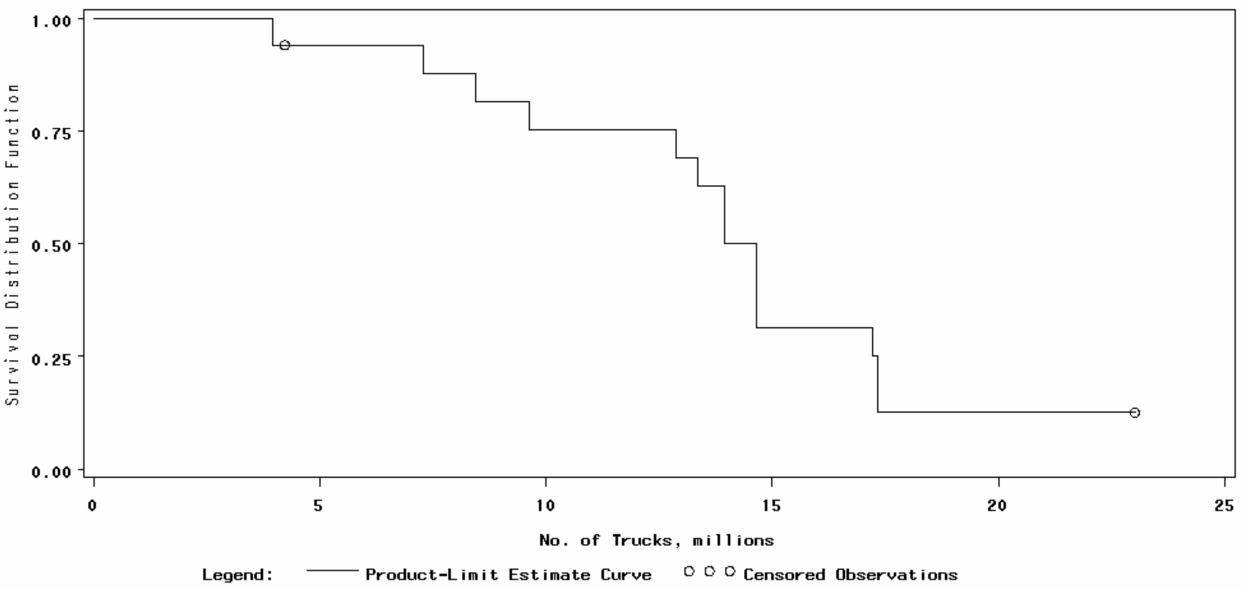


Figure 18. Traffic-based survival distribution function for group 3 (SMA surfacing).

Table 13. Age-based survival analysis results (service life estimates).

Group	Pavement Type	Baseline Trucks per Day	Asphalt Surface Type	
			HMA	SMA
1	Flexible (Asphalt Overlay on Existing Asphalt Pavement)	220	12.7 – NA - NA 14.7 - 18.6 - 22.4 N = 13	15.6 – NA - NA 16.5 - 20.4 - 24.2 N = 21
2	Composite (Asphalt Overlay on Existing JRC Pavement)	4,658	9.2 - 14.1 - 16.8 10.1 - 14.0 - 17.8 N = 13	7.6 - 9.8 - NA 8.6 - 12.5 - 16.3 N = 22
3	Composite (Asphalt Overlay on Existing JRC Pavement)	1,905	11.5 - 14.3 - NA 12.2 - 16.1 - 19.9 N = 16	13.0 - 15.2 - 18.1 11.5 - 15.4 - 19.2 N = 17

NA: Not available.

Table 14. Traffic-based survival analysis results (service life estimates).

Group	Pavement Type	Baseline Trucks per Day	Asphalt Surface Type	
			HMA	SMA
1	Flexible (Asphalt Overlay on Existing Asphalt Pavement)	220	1.2 - 1.9 - NA 0.7 - 5.1 - 9.5 N = 13	1.5 - 2.3 - NA 0.9 – 5.2 - 9.6 N = 21
2	Composite (Asphalt Overlay on Existing JRC Pavement)	4,658	16.8 - 19.4 - 22.9 18.8 - 23.1 - 27.5 N = 13	14.4 - 17.5 - NA 14.9 - 19.3 - 23.6 N = 22
3	Composite (Asphalt Overlay on Existing JRC Pavement)	1,905	8.7 - 12.2 - 14.7 7.8 - 12.2 - 16.5 N = 16	12.9 - 14.6 - 17.3 10.1 - 14.5 - 18.9 N = 17

NA: Not available.

5.4.6 Compare Survival Life Estimates (Step 3f)

Flexible pavements with an SMA surfacing were found to have a longer service life than those with a conventional HMA surfacing. However, this difference was not as profound as was expected. Conversely, a longer service life was found for HMA surfacing used in composite situations, as compared to SMA surfacing. A comparison of traffic-based estimates of service life showed no significant differences in the traffic carried within each group, regardless of surfacing type.

Other agencies have reported a larger difference in performance when SMA mixtures are used as the wearing surface on high-volume roadways. Georgia and Michigan are two that have found substantial differences in the service lives of flexible pavements with SMA and HMA wearing surfaces. The increased service life for SMA wearing surfaces has been related to a reduction in longitudinal cracking and rutting. The difference in the service lives for SMA and HMA mixtures was found to be somewhat less in Wisconsin. The reason for this difference is unknown at this time. It is expected that, with time, the difference in the service life will become more-well defined between the two types of wearing surfaces.

CHAPTER 6. DEVELOPMENT OF LIFE-CYCLE MODELS

6.1 INTRODUCTION

Life-cycle models reflect the types and sequence of M&R activities that can be expected to occur for a particular original pavement structure over the chosen analysis period. To evaluate the life-cycle costs of SMA- and conventional HMA-overlaid asphalt and concrete pavements, a variety of scenarios were identified for which costs could be calculated. Based on the groups of pavements presented in chapter 5 (table 9), typical Wisconsin pavement structures were developed.

For each combination of asphalt surfacing type (SMA or HMA) and pavement group, a continuous preservation life-cycle model was established that largely represents WisDOT M&R strategies. Information on the service lives of SMA- and HMA-overlaid pavements from this study, combined with pavement performance findings from a recently completed study on Wisconsin pavement service lives, were used to develop specific continuous preservation life-cycle models for each combination of asphalt surface type and pavement group.

Figure 19 illustrates a typical life-cycle model reflecting WisDOT design and M&R strategies. For this study, the initial event was the SMA or conventional HMA overlay placed on the existing asphalt or concrete pavement structure. Discussions of the development of life-cycle models and a presentation of the final models are provided in the sections below, corresponding to the three pavement groupings.

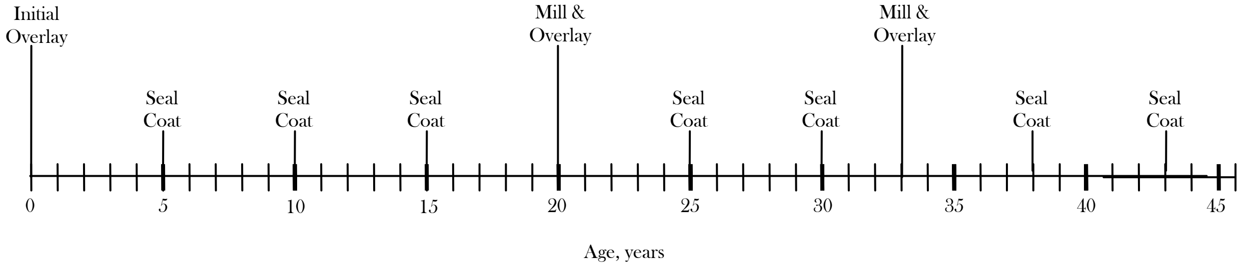


Figure 19. Conceptual illustration of initial design, continuous preservation, and reconstruction design strategies.

6.2 MODELS FOR GROUP 1 OVERLAY APPLICATION

Table 15 shows the alternative life-cycle models developed for group 1 overlay applications (flexible pavement, U.S./State routes). The representative pavement section is a 4.5-in mill-and-fill asphalt surface layer (SMA or HMA) constructed over existing 7-in HMA, 8-in granular base, and a prepared subgrade. Full-depth asphalt shoulders have been assumed to accompany the mainline structure. Based on the characteristics of the flexible pavements analyzed, the pavement is a 4-lane rural interstate, with 12-ft travel lanes and 8-ft outside and 4-ft inside shoulders. The section had a 2005 AADT (1-way) of approximately 2,290 vehicles/day, and the 9.6 percent trucks on this facility yields an estimated 220 trucks per day (1-way, design lane).

Survival analysis results indicated a median life of 20.4 and 18.6 years for an SMA and HMA resurfacing, respectively. A COV of 10 percent was assumed. To account for traffic growth and deterioration of the underlying pavement layers over time, the life of each treatment was adjusted downward by 2 years for each rehabilitation cycle, starting with the second cycle. Also, for probabilistic LCCA purposes, typical M&R treatment was repeated, as necessary, beyond the 45-year analysis period. Cost estimates were determined for each activity based on unit costs assembled as part of this study.

6.3 MODELS FOR GROUP 2 OVERLAY APPLICATION

Table 16 shows the alternative life-cycle models developed for group 2 overlay applications (composite pavement, interstate/U.S. routes). The representative pavement section is a 3-in asphalt overlay (SMA or HMA) constructed over an existing 9-in JRC over a 6-in crushed aggregate base and prepared subgrade. Full-depth asphalt shoulders have been assumed to accompany the mainline structure. Based on the characteristics of the composite pavement in group 2 analyzed, the pavement is a 4-lane rural interstate, with 12-ft travel lanes and 8-ft outside and 4-ft inside shoulders. The section had a 2005 AADT (1-way) of approximately 54,805 vehicles/day, and the 8.5 percent trucks on this facility yields an estimated 4,658 trucks per day (1-way, design lane).

Survival analysis results indicated a median life of 11.2 and 14.0 years for an SMA and HMA resurfacing, respectively. A COV of 10 percent was assumed. To account for traffic growth and deterioration of the underlying pavement layers over time, the life of each treatment was adjusted downward by 2 years for each rehabilitation cycle, starting with the second cycle. Also, for probabilistic LCCA purposes, typical M&R treatment was repeated, as necessary, beyond the 45-year analysis period. Cost estimates were determined for each activity based on unit costs assembled as part of this study.

Table 15. Life-cycle models developed for group 1 (flexible pavement, U.S./State routes).

Year	Activity Sequence for Reconstruction Alternatives					
	HMA			SMA		
	Activity	Structure	Cost, $1000	Activity	Structure	Cost, $1000
1	Initial construction	4.5-in HMA Overlay	299.09 (35.9)	Initial construction	4.5-in SMA Overlay	305.08 (36.6)
2						
3						
4						
5	Maintenance	Single layer seal coat	59.9 (7.2)			
6						
7				Maintenance	Single layer seal coat	59.9 (7.2)
8						
9						
10	Maintenance	Single layer seal coat	59.9 (7.2)			
11						
12						
13						
14				Maintenance	Single layer seal coat	59.9 (7.2)
15	Maintenance	Single layer seal coat	59.9 (7.2)			
16						
17						
18						
19	Rehabilitation	4.5-in mill-and-fill	337.43 (40.5)			
20						
21				Rehabilitation	4.5-in mill-and-fill	343.42 (41.2)
22						
23						
24	Maintenance	Single layer seal coat	59.9 (7.2)			
25						
26						
27						
28				Maintenance	Single layer seal coat	59.9 (7.2)
29	Maintenance	Single layer seal coat	59.9 (7.2)			
30						
31						
32	Rehabilitation	4.5-in mill-and-fill	337.43 (40.5)			
33						
34						
35				Maintenance	Single layer seal coat	59.9 (5.9)
36						
37	Maintenance	Single layer seal coat	59.9 (7.2)			
38						
39				Rehabilitation	4.5-in mill-and-fill	343.42 (41.2)
40						
41						
42	Maintenance	Single layer seal coat	59.9 (7.2)			
43						
44						
45				Maintenance	Single layer seal coat	59.9 (7.2)

Table 16. Life-cycle models developed for group 2 (composite pavement, interstate/U.S. routes).

Year	Activity Sequence for Reconstruction Alternatives					
	HMA Overlay			**SMA Overlay**		
	Activity	**Structure**	**Cost, $1000**	**Activity**	**Structure**	**Cost, $1000**
1	Initial construction	3.0-in HMA Overlay	194.96 (23.4)	Initial construction	3.0-in SMA Overlay	223.79 (26.8)
2						
3						
4						
5	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
6						
7						
8						
9						
10	Maintenance	Crack sealing	5.2 (0.6)			
11				Rehabilitation	3.0-in mill and fill	265.83 (31.9)
12						
13						
14	Rehabilitation	3.0-in mill and fill	237.01 (28.4)			
15						
16				Maintenance	Crack sealing	5.2 (0.6)
17						
18						
19	Maintenance	Crack sealing	5.2 (0.6)			
20				Rehabilitation	3.0-in mill and fill	265.83 (31.9)
21						
22						
23						
24						
25				Maintenance	Crack sealing	5.2 (0.6)
26	Rehabilitation	3.0-in mill and fill	237.01 (28.4)			
27				Reconstruction	9-in JRC	1,606.1 (192.7)
28						
29						
30						
31	Maintenance	Crack sealing	5.2 (0.6)			
32						
33						
34						
35						
36	Reconstruction	9-in JRC	1,606.1 (192.7)			
37						
38						
39						
40						
41						
42						
43						
44						
45						

6.4 MODELS FOR GROUP 3 OVERLAY APPLICATION

Table 17 shows the alternative life-cycle models developed for group 3 overlay applications (composite pavement, U.S/State routes). The representative pavement section is a 3-in asphalt overlay (SMA or HMA) constructed over an existing 9-in JRC over a 6-in crushed aggregate base and prepared subgrade. Full-depth asphalt shoulders have been assumed to accompany the mainline structure. Based on the characteristics of the composite pavement in group 3 analyzed, the pavement is a 4-lane rural interstate, with 12-ft travel lanes and 8-ft outside and 4-ft inside shoulders. The section had a 2005 AADT (2-way) of approximately 11,454 vehicles/day, and the 16.6 percent trucks on this facility yields an estimated 1,905 trucks per day (1-way, design lane).

Survival analysis results indicated a median life of 15.3 and 15.2 years for an SMA and HMA resurfacing, respectively. A COV of 10 percent was assumed. To account for traffic growth and deterioration of the underlying pavement layers over time, the life of each treatment was adjusted downward by 2 years for each rehabilitation cycle, starting with the second cycle. Also, for probabilistic LCCA purposes, typical M&R treatment was repeated, as necessary, beyond the 45-year analysis period. Cost estimates were determined for each activity based on unit costs assembled as part of this study.

Table 17. Life-cycle models developed for group 3 (composite pavement, U.S./State routes).

Year	**Activity Sequence for Reconstruction Alternatives**					
	HMA Overlay			**SMA Overlay**		
	Activity	**Structure**	**Cost, $1000**	**Activity**	**Structure**	**Cost, $1000**
1	Initial construction (flex/flex)	3.0-in HMA Overlay	194.96 (23.4)	Initial construction (flex/flex)	3.0-in SMA Overlay	223.79 (26.8)
2						
3						
4						
5	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
6						
7						
8						
9						
10	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
11						
12						
13						
14						
15	Rehabilitation	3.0-in mill and fill	237.01 (28.4)	Rehabilitation	3.0-in mill and fill	265.83 (31.9)
16						
17						
18						
19						
20	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
21						
22						
23						
24						
25	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
26						
27						
28	Rehabilitation	3.0-in mill and fill	237.01 (28.4)	Rehabilitation	3.0-in mill and fill	265.83 (31.9)
29						
30						
31						
32	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
33						
34						
35						
36						
37	Maintenance	Crack sealing	5.2 (0.6)	Maintenance	Crack sealing	5.2 (0.6)
38						
39	Reconstruction	9-in JRC	1,606.1 (192.7)	Reconstruction	9-in JRC	1,606.1 (192.7)
40						
41						
42						
43						
44						
45						

CHAPTER 7. LIFE-CYCLE COST ANALYSIS

7.1 LCCA PROGRAM SELECTION

For this study, LCCA was performed using both deterministic and probabilistic approaches. The deterministic approach reflects current WisDOT procedure for performing LCCA, while the probabilistic approach accounts for real-world variability and/or uncertainty associated with the various input parameters (e.g., costs, performance) that are used to compute life-cycle costs. The program used in analysis was the FHWA's LCCA spreadsheet program, *RealCost* Version 2.1 (FHWA, 2004).

RealCost applies a simulation technique that entails defining individual input parameters by either a frequency (or probability) distribution or by a discrete value, as is done typically in deterministic LCCA and computing an array of life-cycle costs (using an iterative sampling of the pre-defined frequency distributions of each input variable for the probabilistic approach). The resulting life-cycle costs (or unique probability distribution of costs), can then be examined and compared with the cost or cost distribution of a competing design alternatives. The simulation technique utilized by *RealCost* for probability simulation is the Monte Carlo simulation and is illustrated in figure 20 (Walls and Smith, 1998).

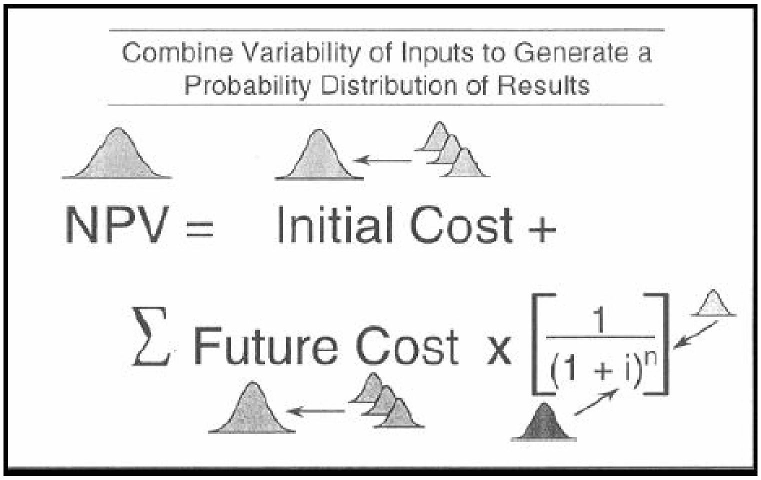


Figure 20. NPV distribution generation (Smith & Walls, 1998).

RealCost computes life-cycle costs in the form of net present value (NPV), which is defined as follows:

$$NPV = Initial\ Cost + \sum Future\ Cost * \left[\frac{1}{(1+i)^n} \right] \qquad \text{Eq. 6}$$

where: NPV = Net present value, $.
i = Discount rate, percent.
N = Time of future cost, years.

7.2 LCCA INPUTS

Figure 21 shows the overall program layout for *RealCost*. The layout shows the various function and inputs of the program and inputs required for analysis. A summary of the functions and inputs are as follows:

- Project-level inputs—Inputs common to both alternatives under consideration such as traffic, project details, analysis options, and so on. (Note that for this study user costs were not considered in LCCA).
- Alternative-level inputs—Inputs specific to a given design alternative (i.e., HMA and SMA design alternatives). The information required includes initial construction and subsequent M&R activity and reconstruction cost and service lives and anticipated maintenance costs and frequency.
- Activity work zone inputs.
- Input warning—Warnings trigger by possible errors in input data.
- Simulations and outputs—Inputs for actual running of the LCCA such the number of simulations and so on and reported results.
- Administrative functions.

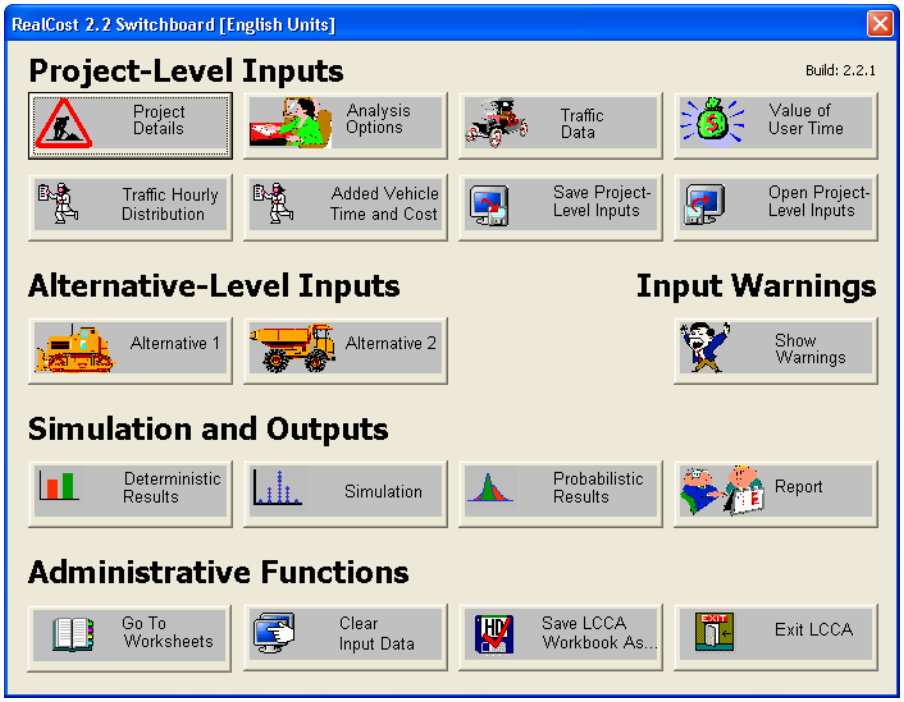
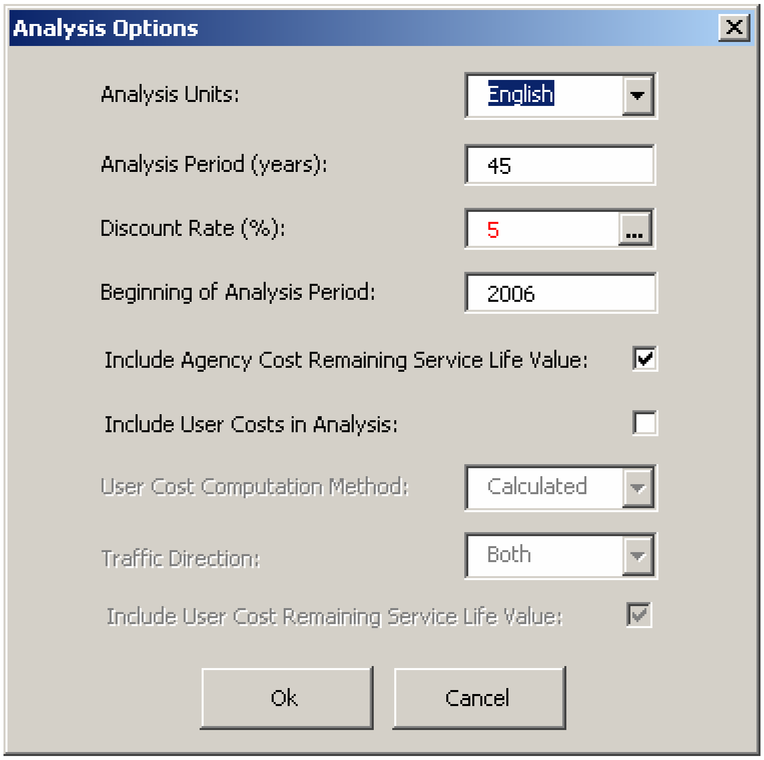


Figure 21. Overall program layout for *RealCost*.

The project details (inventory type information), analysis assumptions applied for this analysis (e.g., analysis period of 45 years, discount rate of 5 percent over the analysis period, and the inclusion of agency cost remaining life value in the analysis). Note that user costs were not considered in this analysis, as they are not required by WisDOT.

Traffic volumes expected on the highways being analyzed along with default roadway speed, traffic capacity values, etc., obtained from the *Realcost* user guide were assumed (FHWA, 2004). Figures 22 through 24 present examples of default inputs such as analysis period, traffic capacity, and traffic hourly distribution (obtained from *RealCost* representing hourly traffic distribution for urban environments) used for analysis.



Analysis Options

Analysis Units: English

Analysis Period (years): 45

Discount Rate (%): 5

Beginning of Analysis Period: 2006

Include Agency Cost Remaining Service Life Value: ☑

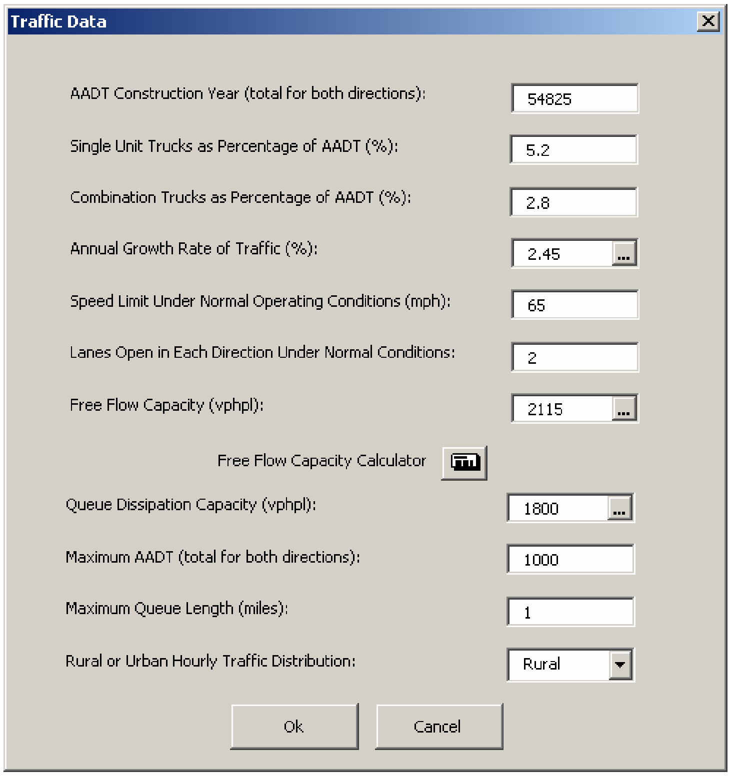
Include User Costs in Analysis: ☐

User Cost Computation Method: Calculated

Traffic Direction: Both

Include User Cost Remaining Service Life Value: ☑

Figure 22. *RealCost* analysis options applied and used for computing life-cycle costs.



AADT Construction Year (total for both directions): 54825

Single Unit Trucks as Percentage of AADT (%): 5.2

Combination Trucks as Percentage of AADT (%): 2.8

Annual Growth Rate of Traffic (%): 2.45

Speed Limit Under Normal Operating Conditions (mph): 65

Lanes Open in Each Direction Under Normal Conditions: 2

Free Flow Capacity (vphpl): 2115

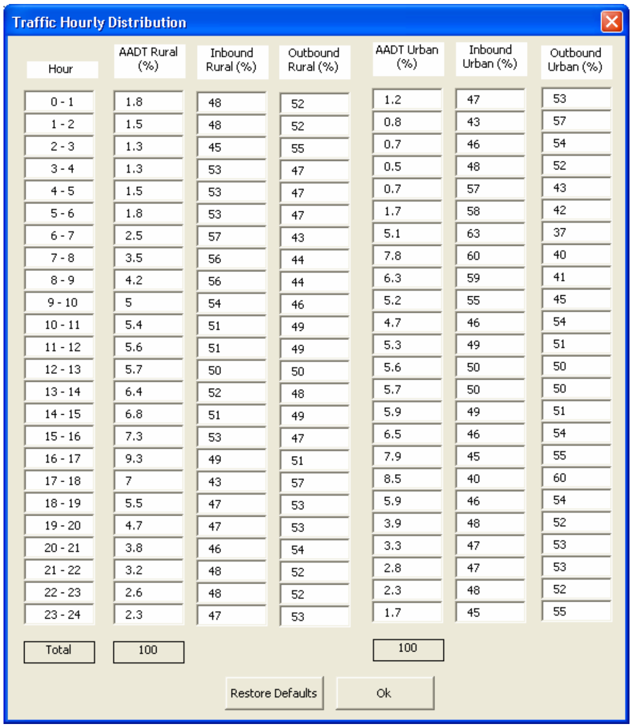
Queue Dissipation Capacity (vphpl): 1800

Maximum AADT (total for both directions): 1000

Maximum Queue Length (miles): 1

Rural or Urban Hourly Traffic Distribution: Rural

Figure 23. Traffic data used for analysis.



Hour	AADT Rural (%)	Inbound Rural (%)	Outbound Rural (%)	AADT Urban (%)	Inbound Urban (%)	Outbound Urban (%)
0 - 1	1.8	48	52	1.2	47	53
1 - 2	1.5	48	52	0.8	43	57
2 - 3	1.3	45	55	0.7	46	54
3 - 4	1.3	53	47	0.5	48	52
4 - 5	1.5	53	47	0.7	57	43
5 - 6	1.8	53	47	1.7	58	42
6 - 7	2.5	57	43	5.1	63	37
7 - 8	3.5	56	44	7.8	60	40
8 - 9	4.2	56	44	6.3	59	41
9 - 10	5	54	46	5.2	55	45
10 - 11	5.4	51	49	4.7	46	54
11 - 12	5.6	51	49	5.3	49	51
12 - 13	5.7	50	50	5.6	50	50
13 - 14	6.4	52	48	5.7	50	50
14 - 15	6.8	51	49	5.9	49	51
15 - 16	7.3	53	47	6.5	46	54
16 - 17	9.3	49	51	7.9	45	55
17 - 18	7	43	57	8.5	40	60
18 - 19	5.5	47	53	5.9	46	54
19 - 20	4.7	47	53	3.9	48	52
20 - 21	3.8	46	54	3.3	47	53
21 - 22	3.2	48	52	2.8	47	53
22 - 23	2.6	48	52	2.3	48	52
23 - 24	2.3	47	53	1.7	45	55
Total	100			100		

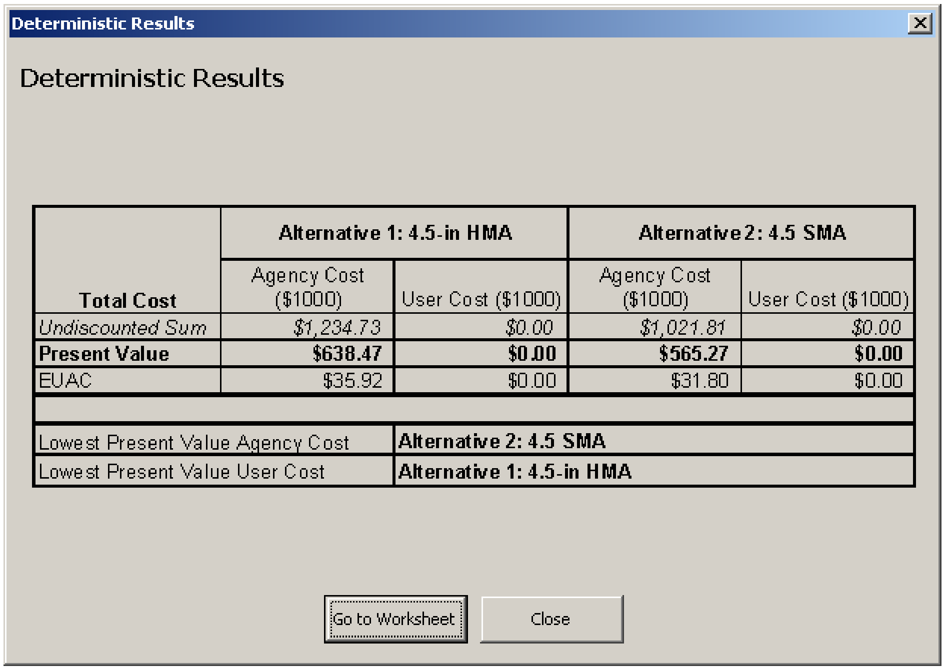
Figure 24. Hourly traffic distribution (default) used for analysis.

7.3 LCCA RESULTS

For this study, a normal probability distribution was assumed for all probabilistic inputs. A total of 50 simulations were performed for the LCCA. The resulting life-cycle cost estimates for each combination of asphalt surfacing type and pavement group are presented in figures 25 through 27. Both the deterministic results and the probabilistic (mean and standard deviation) results based on 50 simulation runs are given.

Life-cycle costs computed for SMA and conventional HMA overlays for each group were compared to determine if there was any significant difference between the estimates for HMA and SMA designs. This comparison was done by performing a t-test (statistical analysis). The t-test was used to determine if the difference in computed life-cycle costs is large enough to warrant a rejection of the null hypothesis, that in fact such differences are due solely to "chance" (i.e., life cycle costs for the HMA and SMA are not significantly different). The result of the t-test is presented in table 18.

The results show a "p" value (two-tail) of less than 0.0001 for the overlays on asphalt pavements (group 1) and less than 0.0001 and 0.0048 for the overlays on JRC pavements (groups 2 and 3, respectively). This implies that, for group 1 pavements, the null hypothesis is valid at the 95 percent significance level ($\alpha = 0.05$) and there is a significant difference in life-cycle costs between the two designs, with SMA being more cost effective. For the overlays on JRC pavements (groups 2 and 3), the t-test results show a significant difference in life-cycle costs at both significance levels, with the conventional HMA surfacing being more cost-effective.

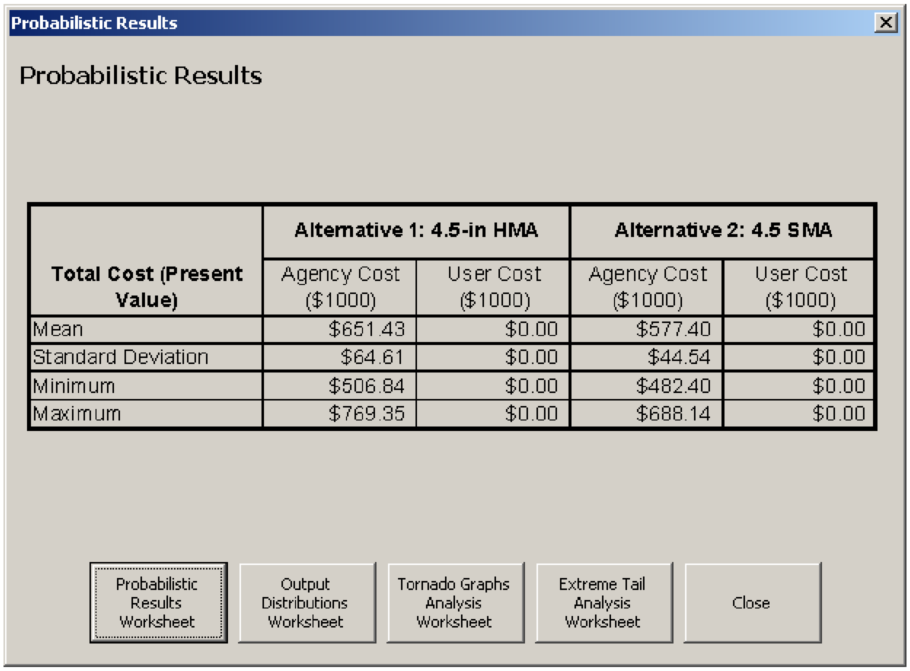


Deterministic Results

Total Cost	Alternative 1: 4.5-in HMA		Alternative 2: 4.5 SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Undiscounted Sum	*$1,234.73*	*$0.00*	*$1,021.81*	*$0.00*
Present Value	**$638.47**	**$0.00**	**$565.27**	**$0.00**
EUAC	$35.92	$0.00	$31.80	$0.00

Lowest Present Value Agency Cost	**Alternative 2: 4.5 SMA**
Lowest Present Value User Cost	**Alternative 1: 4.5-in HMA**

(a)
Output screen showing computed deterministic life cycle costs.

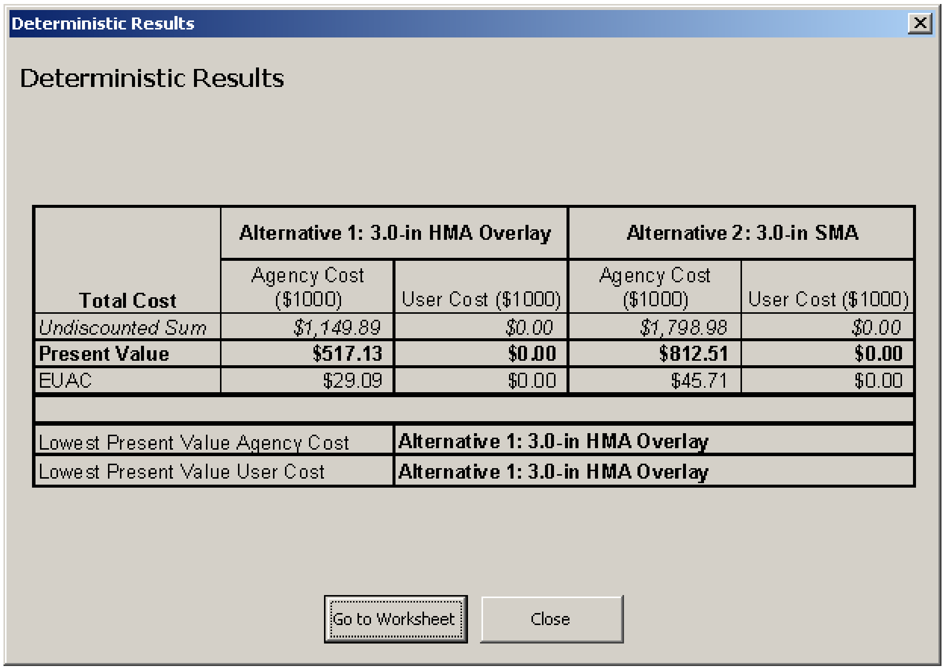


Probabilistic Results

Total Cost (Present Value)	Alternative 1: 4.5-in HMA		Alternative 2: 4.5 SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$651.43	$0.00	$577.40	$0.00
Standard Deviation	$64.61	$0.00	$44.54	$0.00
Minimum	$506.84	$0.00	$482.40	$0.00
Maximum	$769.35	$0.00	$688.14	$0.00

(b)
Output screen showing computed probabilistic life cycle costs.

Figure 25. Computed life-cycle cost statistics for group 1 overlay applications.

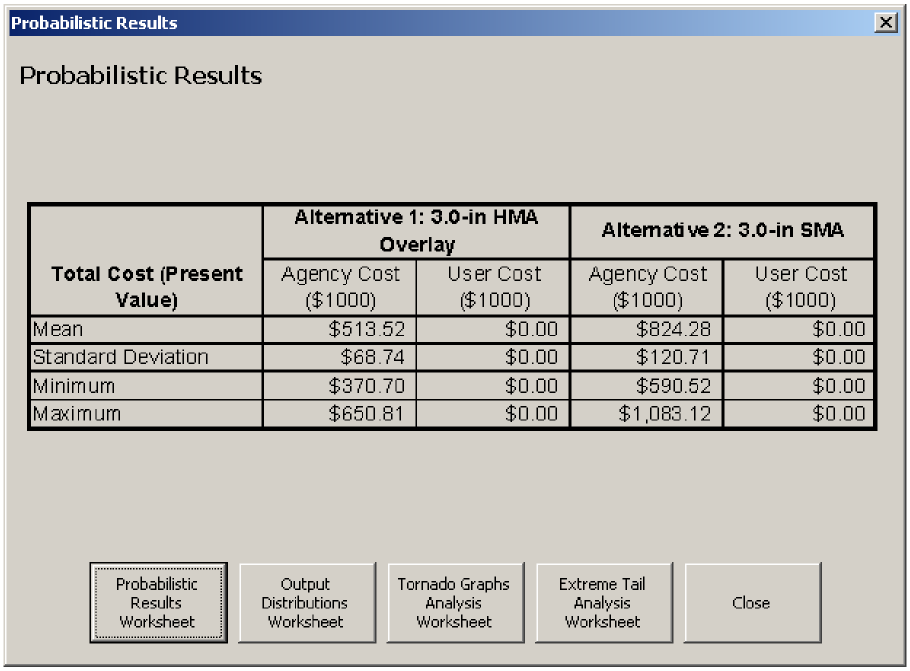


Deterministic Results

Total Cost	Alternative 1: 3.0-in HMA Overlay		Alternative 2: 3.0-in SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Undiscounted Sum	*$1,149.89*	*$0.00*	*$1,798.98*	*$0.00*
Present Value	**$517.13**	**$0.00**	**$812.51**	**$0.00**
EUAC	$29.09	$0.00	$45.71	$0.00
Lowest Present Value Agency Cost	**Alternative 1: 3.0-in HMA Overlay**			
Lowest Present Value User Cost	**Alternative 1: 3.0-in HMA Overlay**			

(a)

Output screen showing computed deterministic life cycle costs.



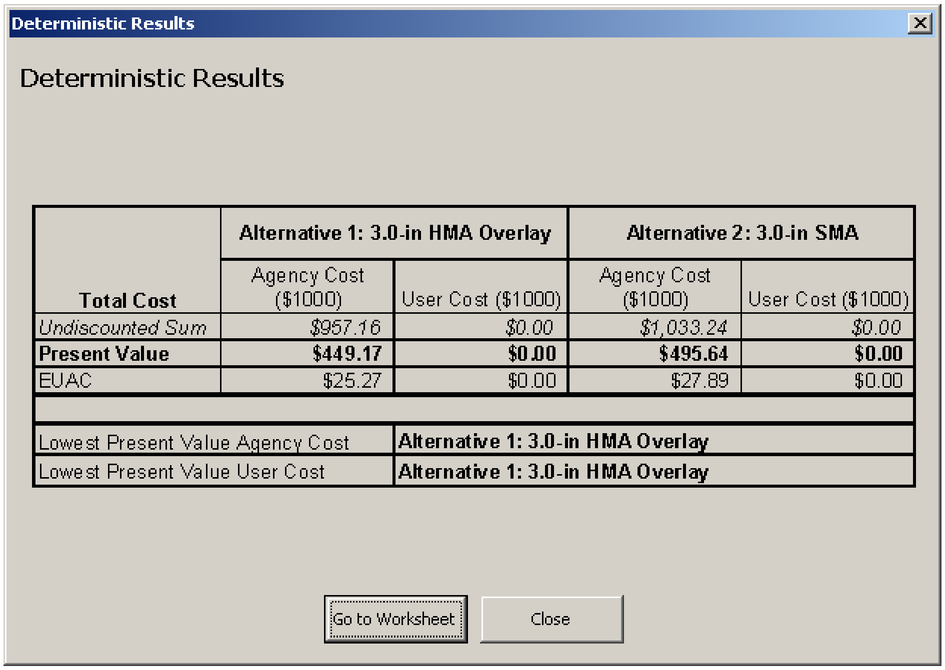
Probabilistic Results

Total Cost (Present Value)	Alternative 1: 3.0-in HMA Overlay		Alternative 2: 3.0-in SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$513.52	$0.00	$824.28	$0.00
Standard Deviation	$68.74	$0.00	$120.71	$0.00
Minimum	$370.70	$0.00	$590.52	$0.00
Maximum	$650.81	$0.00	$1,083.12	$0.00

(b)

Output screen showing computed probabilistic life cycle costs

Figure 26. Computed life-cycle cost statistics for group 2 overlay applications.



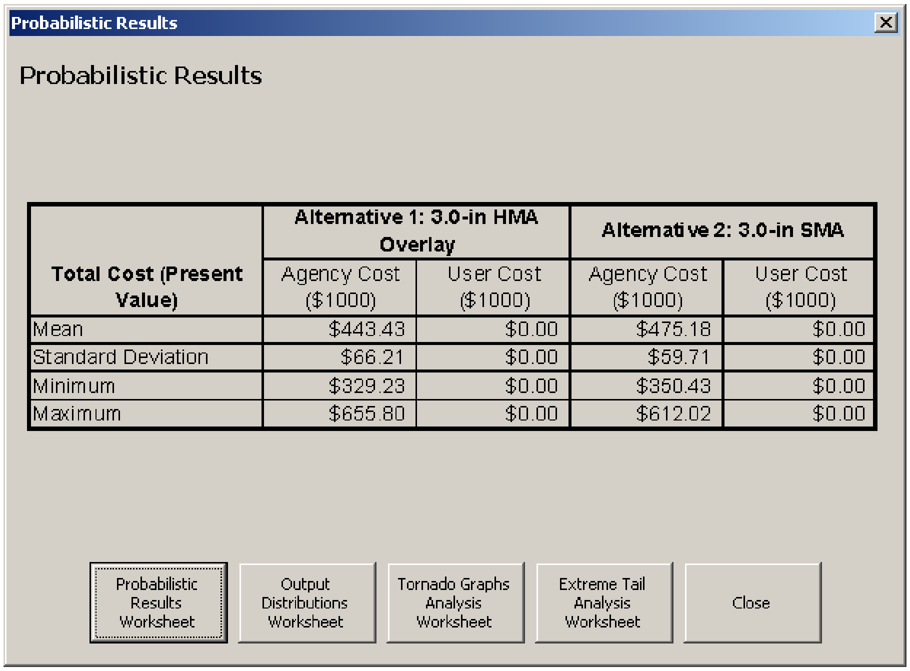
Deterministic Results

Total Cost	Alternative 1: 3.0-in HMA Overlay		Alternative 2: 3.0-in SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Undiscounted Sum	*$957.16*	*$0.00*	*$1,033.24*	*$0.00*
Present Value	**$449.17**	**$0.00**	**$495.64**	**$0.00**
EUAC	$25.27	$0.00	$27.89	$0.00

Lowest Present Value Agency Cost	**Alternative 1: 3.0-in HMA Overlay**
Lowest Present Value User Cost	**Alternative 1: 3.0-in HMA Overlay**

(a)

Output screen showing computed deterministic life cycle costs.



Probabilistic Results

Total Cost (Present Value)	Alternative 1: 3.0-in HMA Overlay		Alternative 2: 3.0-in SMA	
	Agency Cost ($1000)	User Cost ($1000)	Agency Cost ($1000)	User Cost ($1000)
Mean	$443.43	$0.00	$475.18	$0.00
Standard Deviation	$66.21	$0.00	$59.71	$0.00
Minimum	$329.23	$0.00	$350.43	$0.00
Maximum	$655.80	$0.00	$612.02	$0.00

(b)

Output screen showing computed probabilistic life cycle costs.

Figure 27. Computed life-cycle cost statistics for group 3 overlay applications.

Table 18. Summary of t-test results.

Group	Statistics	HMA Overlay	SMA Overlay
1	Mean Life-Cycle Cost ($1000 per 2 lane-mi)	651.4	577.4
	Life-Cycle Cost Variance ($1000 per 2 lane-mi)	4174.4	1984.0
	Number of Cost Iterations	50	50
	Hypothesized mean difference	0	
	Degrees of freedom	49	
	t –test statistic	7.77	
	P(T<=t) one-tail	2.15E-10	
	t Critical one-tail	1.68	
	P(T<=t) two-tail	4.31E-10	
	t Critical two-tail	2.01	
2	Mean Life-Cycle Cost ($1000 per 2 lane-mi)	513.5	824.3
	Life-Cycle Cost Variance ($1000 per 2 lane-mi)	4724.9	14571.7
	Number of Cost Iterations	50	50
	Hypothesized mean difference	0	
	Degrees of freedom	49	
	t –test statistic	-17.8	
	P(T<=t) one-tail	2.34E-23	
	t Critical one-tail	1.68	
	P(T<=t) two-tail	4.69E-23	
	t Critical two-tail	2.01	
3	Mean Life-Cycle Cost ($1000 per 2 lane-mi)	443.4	475.2
	Life-Cycle Cost Variance ($1000 per 2 lane-mi)	4383.2	3565.8
	Number of Cost Iterations	50	50
	Hypothesized mean difference	0	
	Degrees of freedom	49	
	t –test statistic	-2.95	
	P(T<=t) one-tail	0.0024	
	t Critical one-tail	1.68	
	P(T<=t) two-tail	0.0048	
	t Critical two-tail	2.01	

CHAPTER 8. FINDINGS AND RECOMMENDATIONS

This study examined the performance and costs of SMA- and conventional HMA-overlaid asphalt and concrete pavements in Wisconsin. Both the initial pavement structure type and the series of M&R treatments applied to each design alternative over a 45-year period were evaluated to compare the cost-effectiveness of the SMA mixture to conventional HMA.

Initial overlay life and subsequent M&R treatment performance for both overlay design alternatives were evaluated using survival analysis techniques. Additionally, estimates of pavement performance from a recently completed WisDOT study on pavement service lives were used as needed in developing life-cycle models. Unit costs of initial construction and M&R pay items were analyzed to develop best estimates for use in the LCCA process.

Using the assembled LCCA inputs, a comprehensive comparative LCCA was performed using life-cycle models developed for each overlay design alternative. The established models, along with the best estimates of unit costs, were then entered into the FHWA probabilistic LCCA spreadsheet program, *RealCost*, whereupon life-cycle costs were computed using a 45-year analysis period and 5 percent discount rate.

8.1 SUMMARY OF FINDINGS

8.1.1 Initial Pavement and M&R Service Life

Although the difference in pavement life measured in terms of age/years was not substantial, SMA overlays on existing low-volume asphalt pavements were found to provide a longer service life than conventional HMA overlays on similar pavements. The opposite was true for overlays on moderate- to high-volume existing JRC pavements, whereby the average estimated life for SMA was about the same or less than the life of conventional HMA. A comparison of traffic-based estimates of service life showed insignificant differences in the traffic carried within each group, regardless of surfacing type.

Although an evaluation of the pavement conditions prior to application of the SMA and HMA overlays was not performed in this study, significant differences in the average initial smoothness of the SMA and companion HMA overlays were observed for all but two of the 12 comparison sets. The IRI differences ranged from 8 in/mi to 40 in/mi, with the HMA overlays being the smoother of the two.

Based on this observation, it is quite likely that the SMA overlays were placed on rougher (possibly more deteriorated) pavements than their HMA counterparts. Assuming similar levels of roughness progression over time (i.e., similar slope [α] values in the IRI model discussed earlier) for SMA and HMA, it can be fathomed that SMA will reach the terminal IRI threshold sooner than HMA, thereby resulting in a shorter service life. It is believed that this was the case for at least some of the projects examined in this study.

8.1.2 Construction and M&R Pay Item Unit Costs

- Sufficient HMA and SMA cost data were available in the WisDOT cost databases to develop best estimates of unit costs for use in the LCCAs conducted in this study. All original cost data were inflated to 2005 values and, where appropriate, were reprocessed to filter out the effects of projects having large quantities.
- For the other pay items with limited or no cost data, best estimates were developed using all available cost information and engineering judgment.

8.1.3 Life-Cycle Costs

A detailed summary of the LCCA results is presented in table 19 for both asphalt surfacing types (SMA and conventional HMA) for each of the three pavement groups. This table shows the NPV for each alternative at both the mean (50 percent) and 90 percent probability levels. Based on the probabilistic modeling completed for this project, the 50 percent probability means that 5 times out of 10, the pavement costs will be higher than that indicated in table 19. The 90 percent probability means that 9 times out of 10, the cost of the pavement will be less than that shown in table 19.

Table 19. Detailed summary of the results of the LCCA for both asphalt surface types.

Group	Cost Variables	Surfacing Type		Percent Difference in Overlay Alternatives
		HMA	SMA	
1	Estimated Agency Cost @ 50 percent Probability ($1000 per 2 lane-mi)	651.4	577.4	-11.4
	Std. Dev., ($1000)	64.6	44.5	
	Estimated Agency Cost @ 90 percent Probability, ($1000 per 2 lane-mi)	757.4	650.5	-14.1
2	Estimated Agency Cost @ 50 percent Probability ($1000 per 2 lane-mi)	513.5	824.3	60.5
	Std. Dev., ($1000)	68.7	120.7	
	Estimated Agency Cost @ 90 percent Probability, ($1000 per 2 lane-mi)	626.3	1022.3	63.2
3	Estimated Agency Cost @ 50 percent Probability ($1000 per 2 lane-mi)	443.4	475.2	7.2
	Std. Dev., ($1000)	66.2	59.7	
	Estimated Agency Cost @ 90 percent Probability, ($1000 per 2 lane-mi)	552.0	573.1	3.8

The results show that, at the 50 and 90 percent probability level, SMA overlays are more cost-effective than conventional HMA overlays, when applied to existing low-volume asphalt pavements on U.S./State routes. The difference in costs between the two surfacing types in this scenario ranges from -14.1 to -11.4 percent.

For overlays on existing moderate- to high-volume JRC pavements, the difference in costs between the two surfacing types was between 3.8 and 63.2 percent, with conventional HMA being the more cost-effective option. Hence, based on the pavement sections selected for evaluation, the higher initial surfacing cost associated with SMA does not provide the added life required to offset the additional up-front cost.

8.2 RECOMMENDATIONS

The analysis results showed that the life-cycle costs of SMA overlays placed on low-volume asphalt pavements are lower (statistically lower at 90% significance level) than the costs of conventional HMA overlays. For overlay applications on moderate- to high-volume JRC pavements, however, the SMA showed higher (statistically higher at both 90 and 95% significance level) life-cycle costs than conventional HMA.

Based on these results, it appears that there is a cost advantage to applying SMA overlays to existing asphalt pavements, particularly those on low-volume routes. Because the difference in life-cycle costs between the two mixture types was not substantial (say, greater than 5 or 10 percent), it is recommended that the decision as to whether to apply an SMA or conventional HMA overlay on flexible pavements be made on a case-by-case basis. Such an evaluation must consider the following items:

- Initial costs.
- Future M&R frequency.
- Sources of roadway construction and maintenance funding.
- Availability of resources (personnel and financial) for performing frequent M&R activities.
- Disruptions due to lane/road closures.

The selection of one alternative over another will likely be influenced by the sources and availability of funding and city resources (e.g., levies, State funding, developer funding, etc.).

For overlay applications of JRC pavements on moderate- to high-volume interstate and U.S./State routes, the conventional HMA mixture appears to be more cost-effective than SMA. Based on the sections evaluated in this study, the added cost of applying SMA does not result in the substantive increases in pavement life needed to offset the additional up-front cost.

However, as noted previously, the higher life-cycle costs experienced by SMA may be attributed to the likelihood that the SMA overlays evaluated in the study were placed on rougher (possibly more deteriorated) pavements than their HMA counterparts, thereby resulting in shortened lives. In light of this observation, it is recommended that the decision as to whether to apply an SMA or conventional HMA overlay on concrete pavements also be made on a case-by-case basis, particularly when the condition of the existing pavement is in relatively good condition.

8.3 FUTURE WORK

While the results of the analysis completed in this study are considered to be representative of typical Wisconsin conditions, they are based on a limited sampling of pavement performance data. Also, the pavements analyzed are relatively young with very few failures. Consideration should be given to expanding the database of performance and cost information. It is also recommended that an updated study be initiated in about 3 years to re-evaluate the service lives of SMA overlays placed on asphalt and concrete pavements. In addition, the project team recommends that future analyses include both linear and non-linear IRI and PDI modeling (only linear modeling was undertaken in this study), and at least a preliminary assessment of user costs.

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APPENDIX A. PAVEMENT PERFORMANCE CURVES

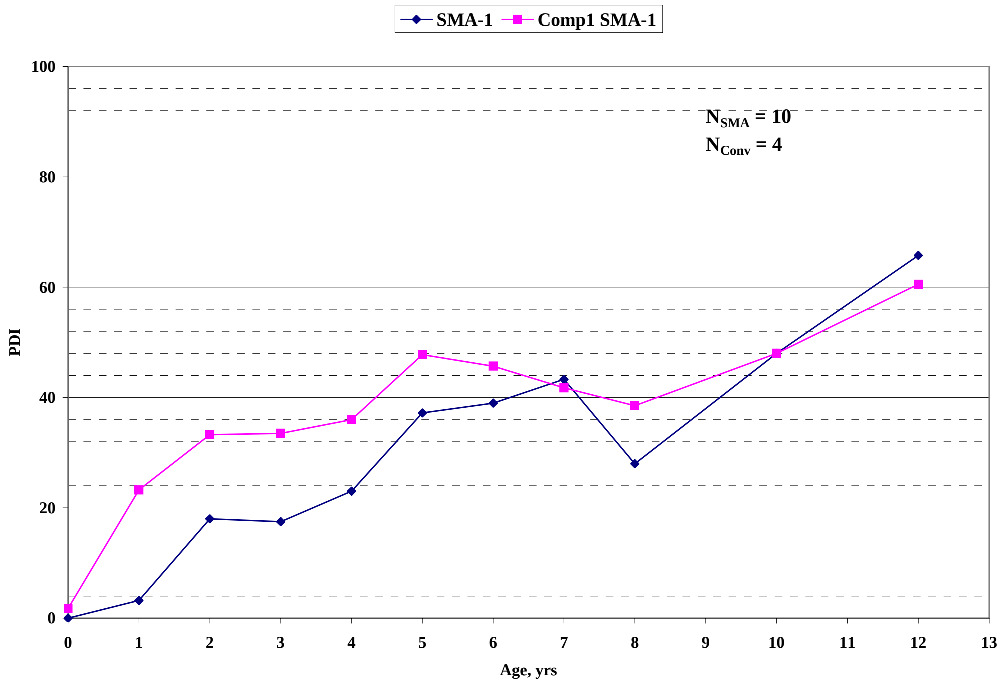


Figure A-1. PDI plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

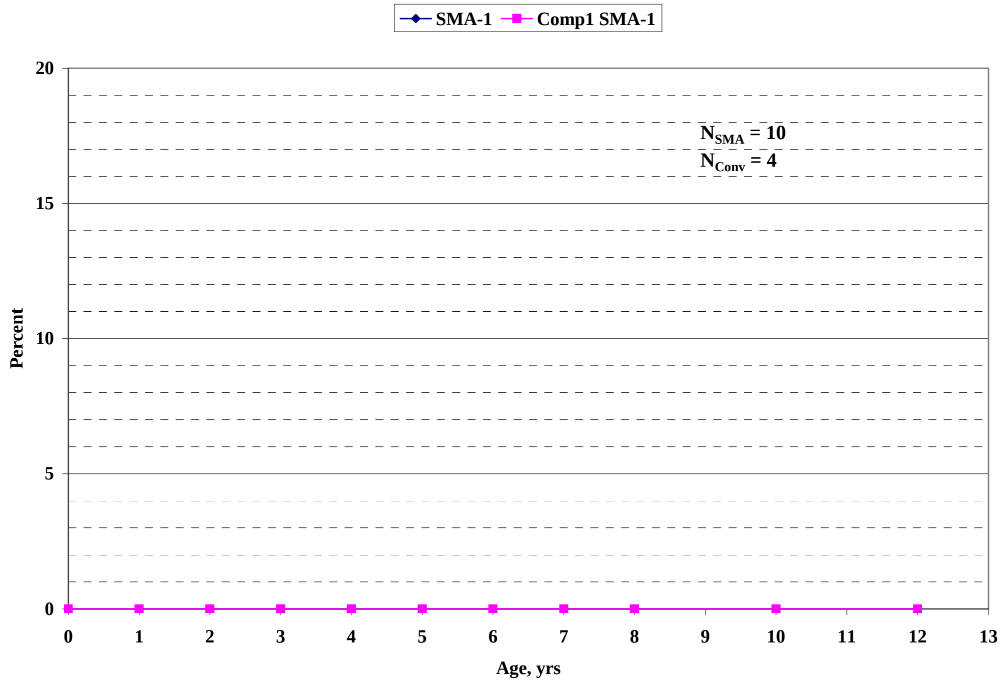


Figure A-2. Alligator cracking plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

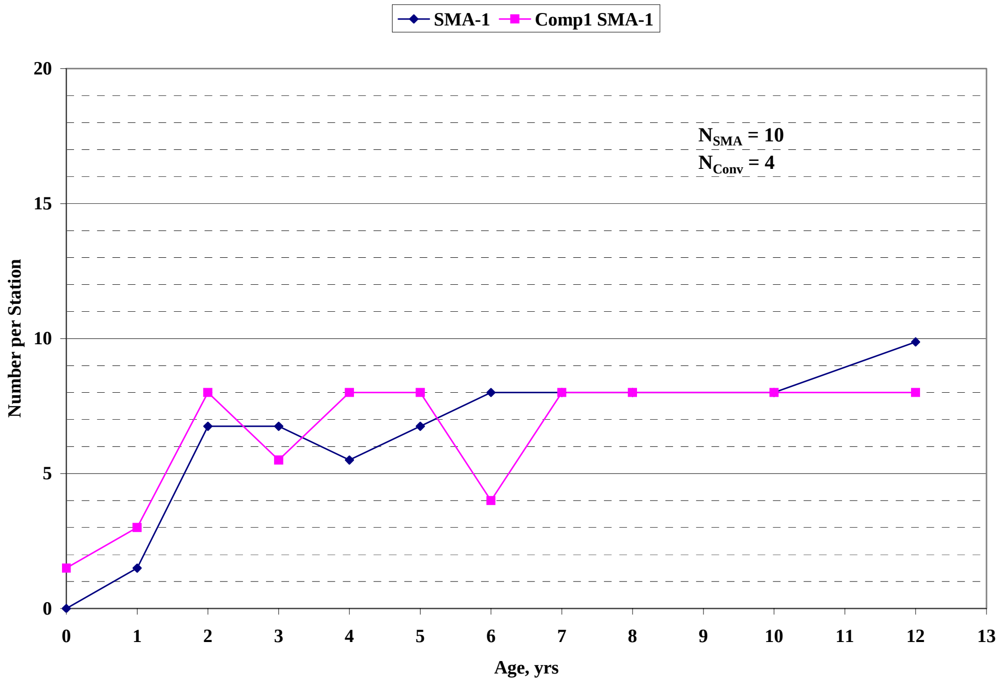


Figure A-3. Transverse cracking plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

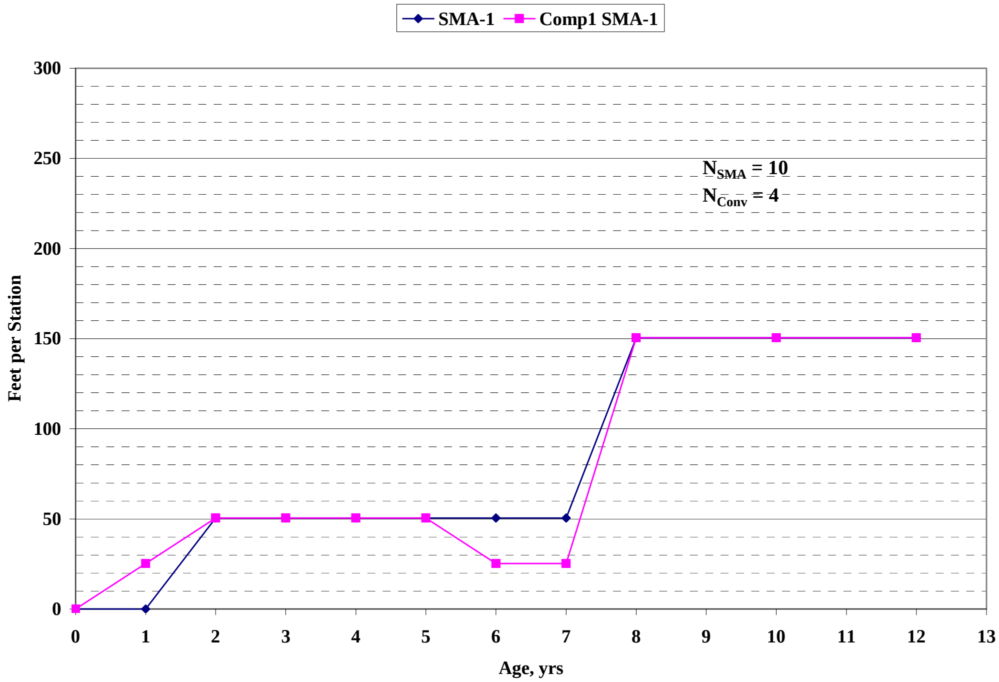


Figure A-4. Longitudinal cracking plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

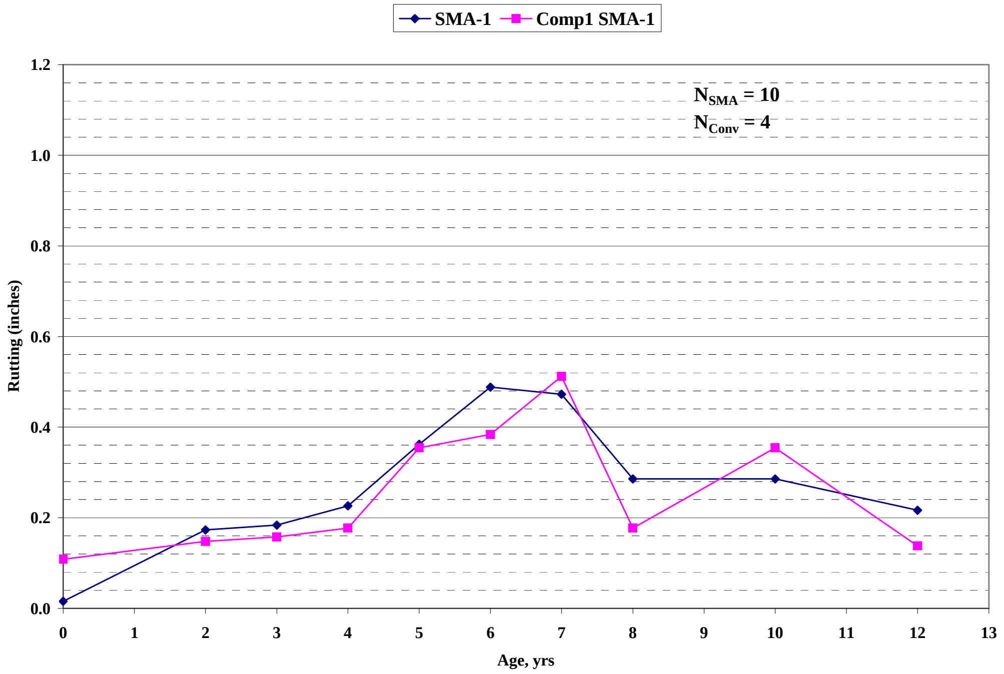


Figure A-5. Rutting plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

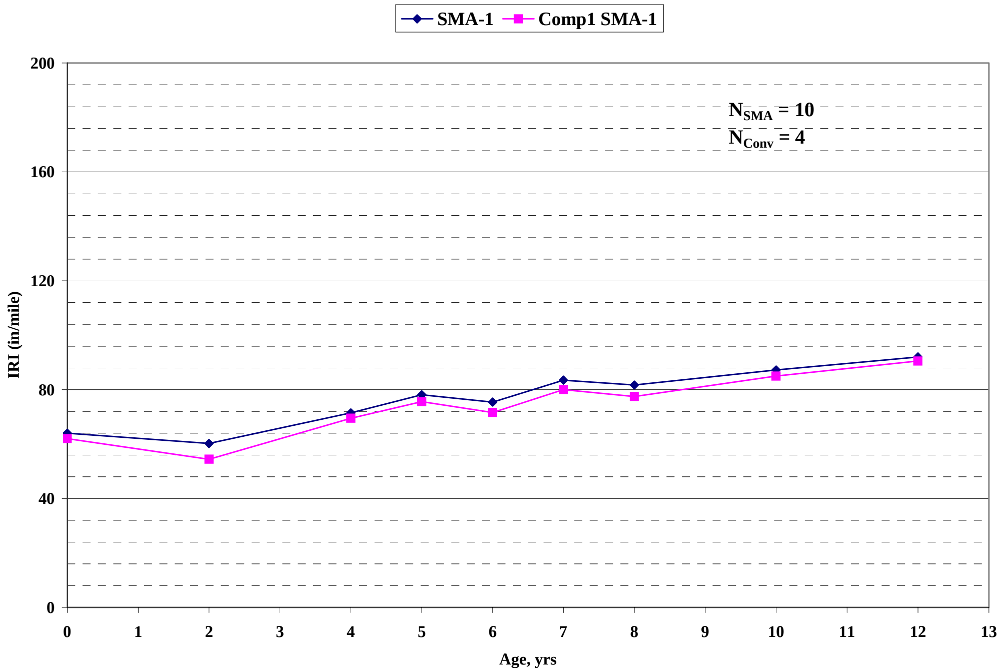


Figure A-6. IRI plot for comparison 1 (I-43 NB/SB, Waukesha County and I-43 NB/SB, Waukesha County).

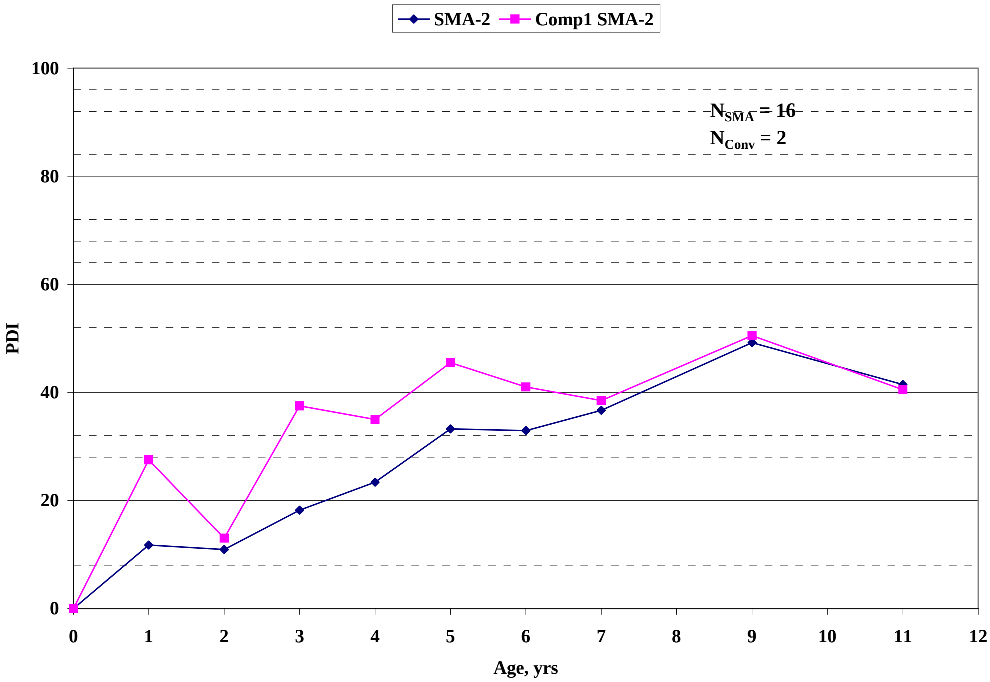


Figure A-7. PDI plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

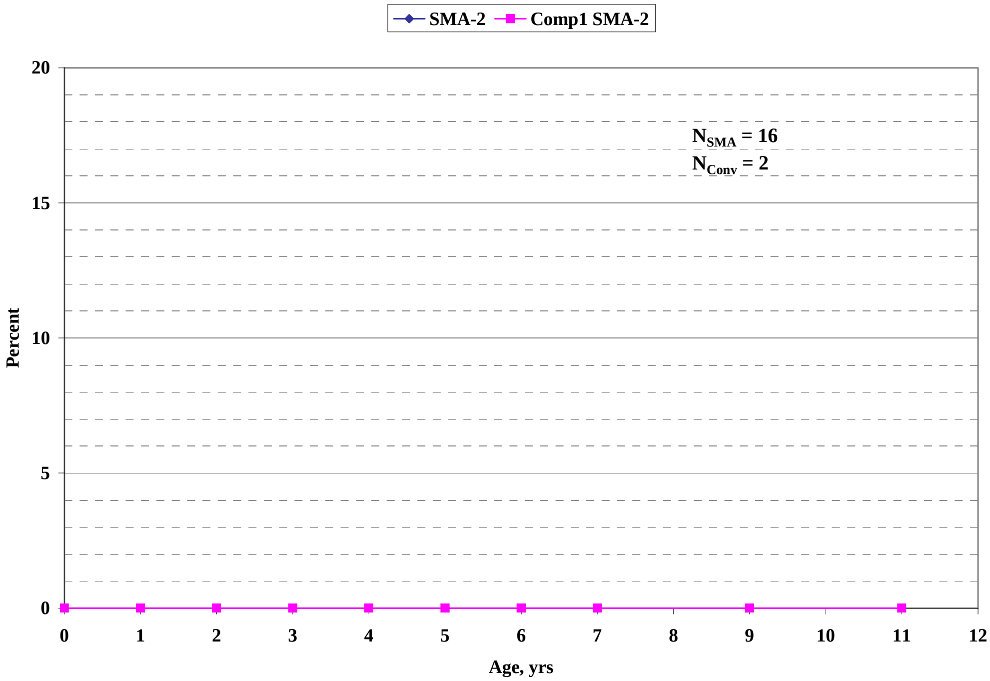


Figure A-8. Alligator cracking plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

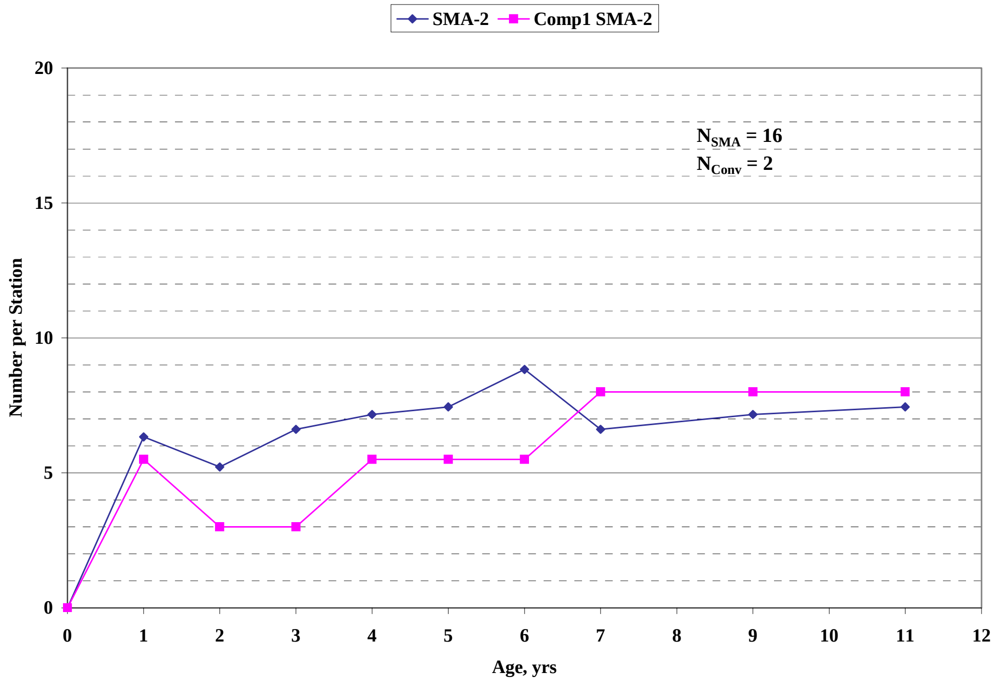


Figure A-9. Transverse cracking plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

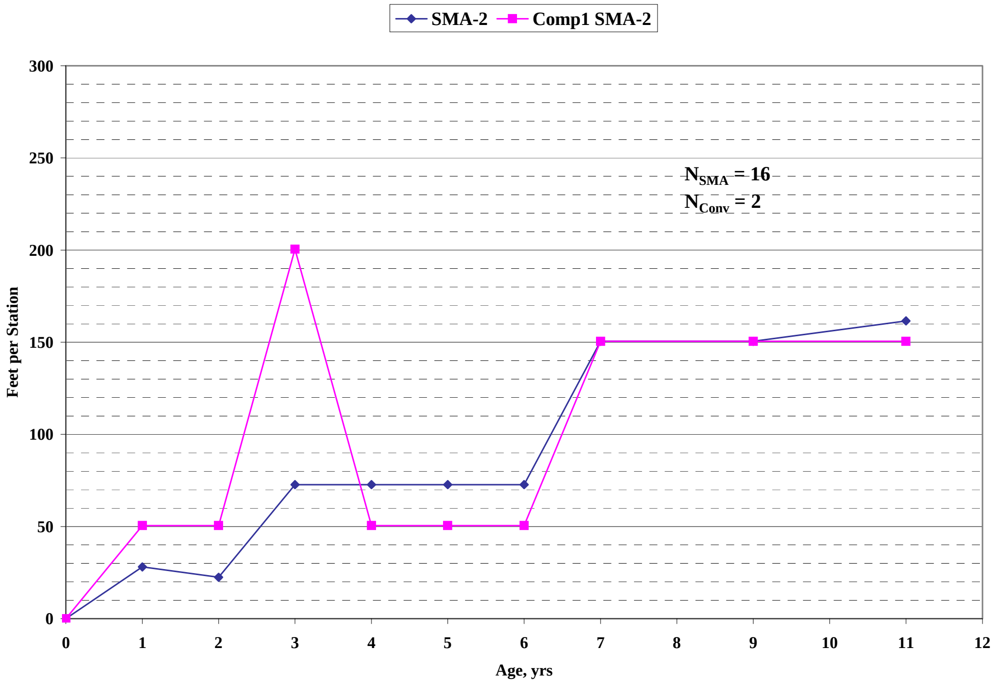


Figure A-10. Longitudinal cracking plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

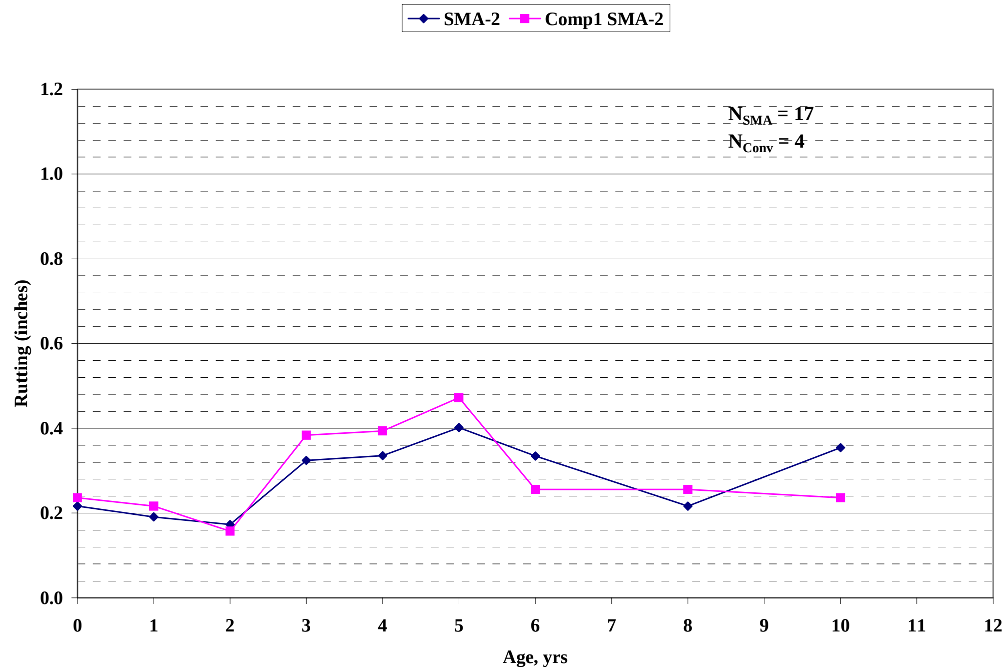


Figure A-11. Rutting plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

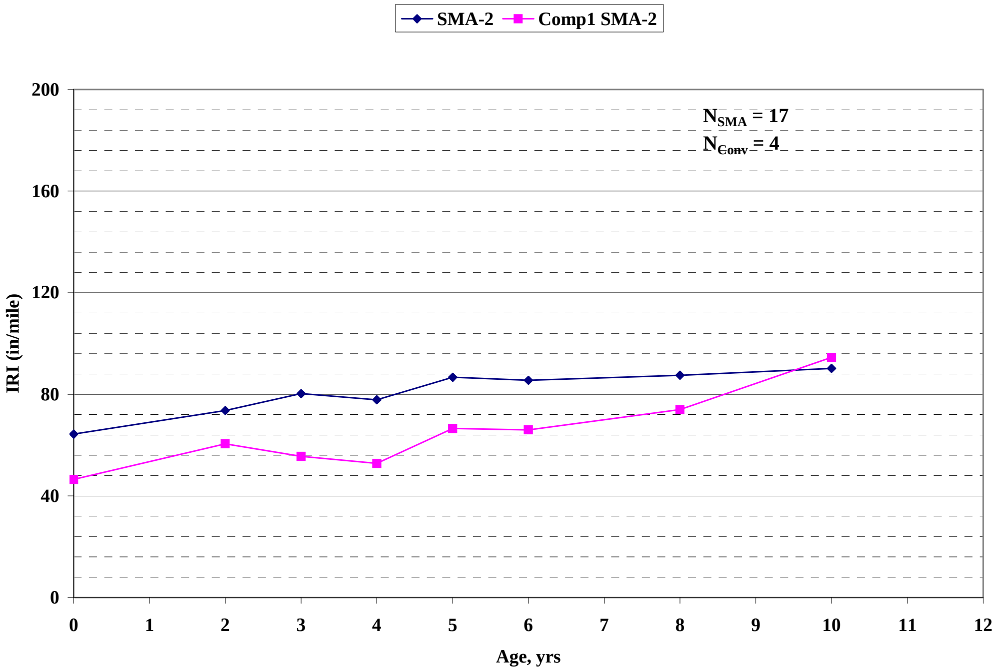


Figure A-12. IRI plot for comparison 2 (I-43 NB/SB, Walworth County and I-43 NB/SB, Walworth County).

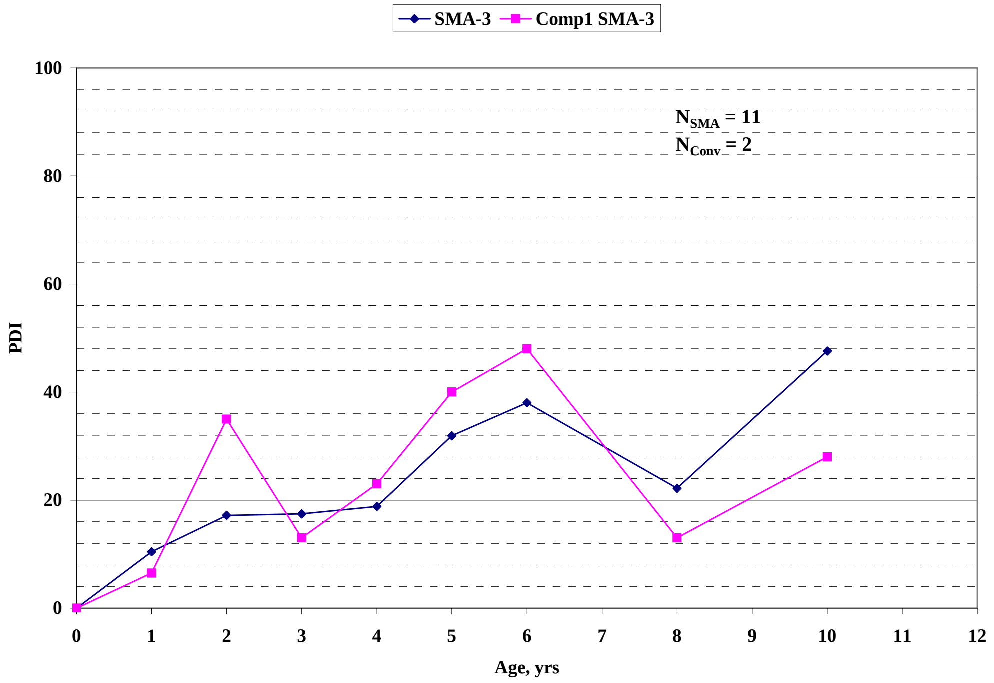


Figure A-13. PDI plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

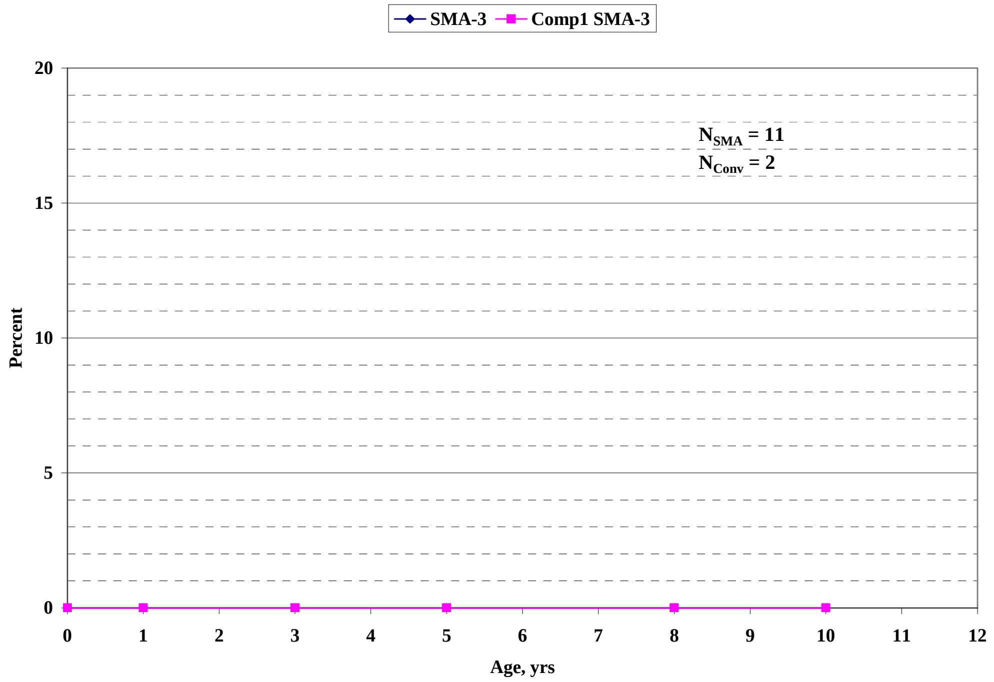


Figure A-14. Alligator cracking plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

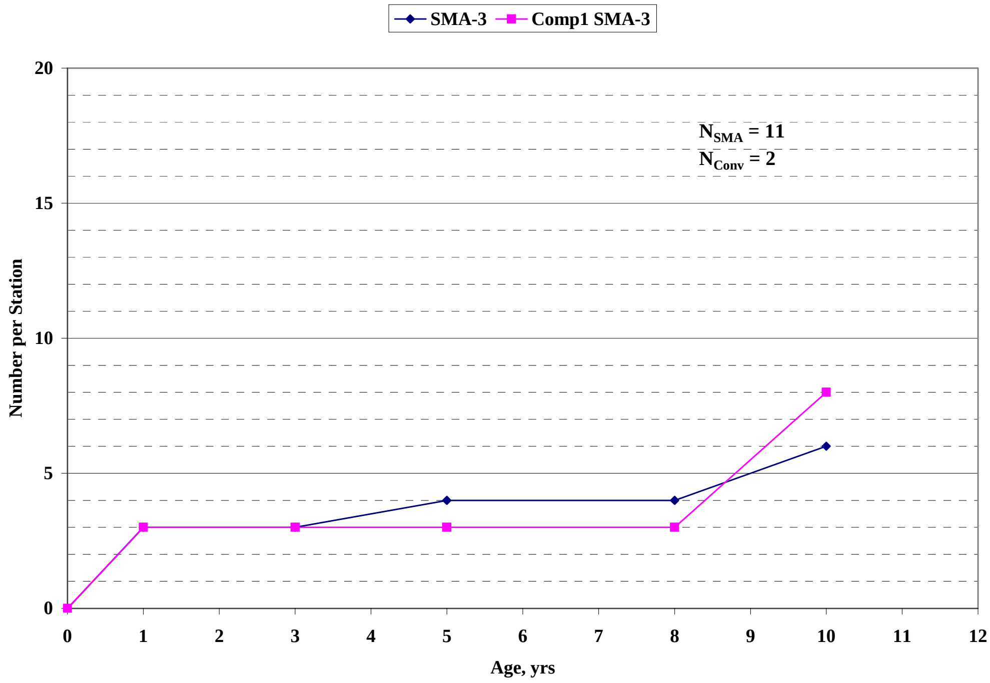


Figure A-15. Transverse cracking plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

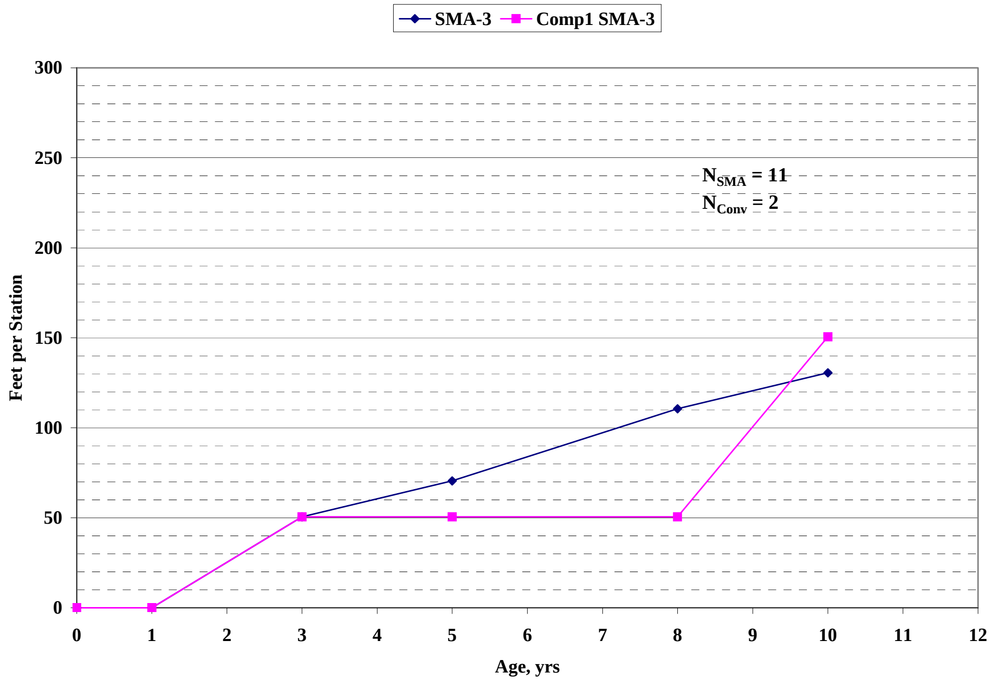


Figure A-16. Longitudinal cracking plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

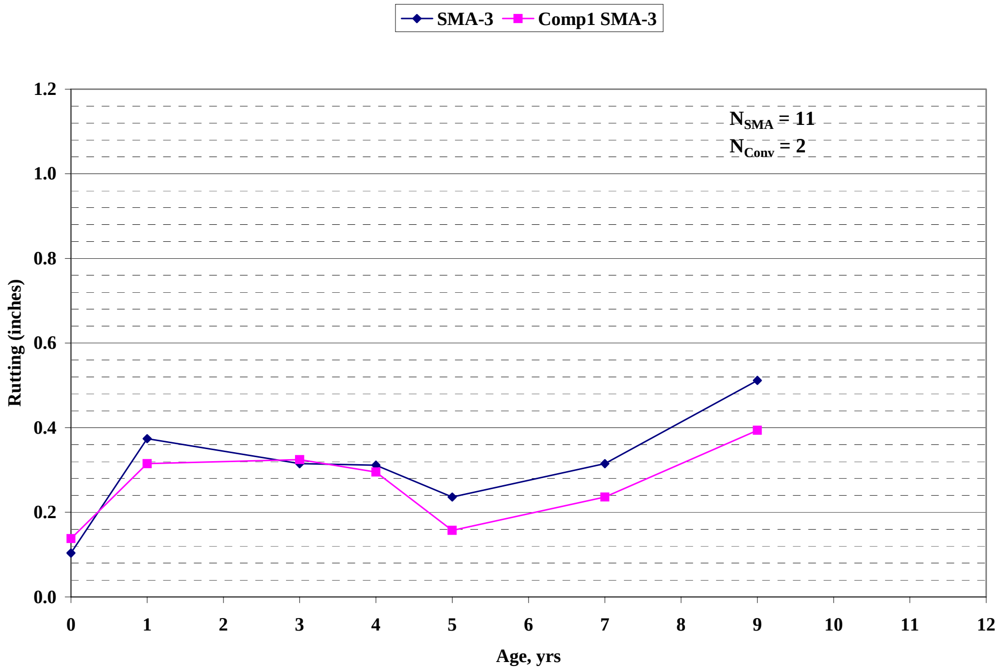


Figure A-17. Rutting plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

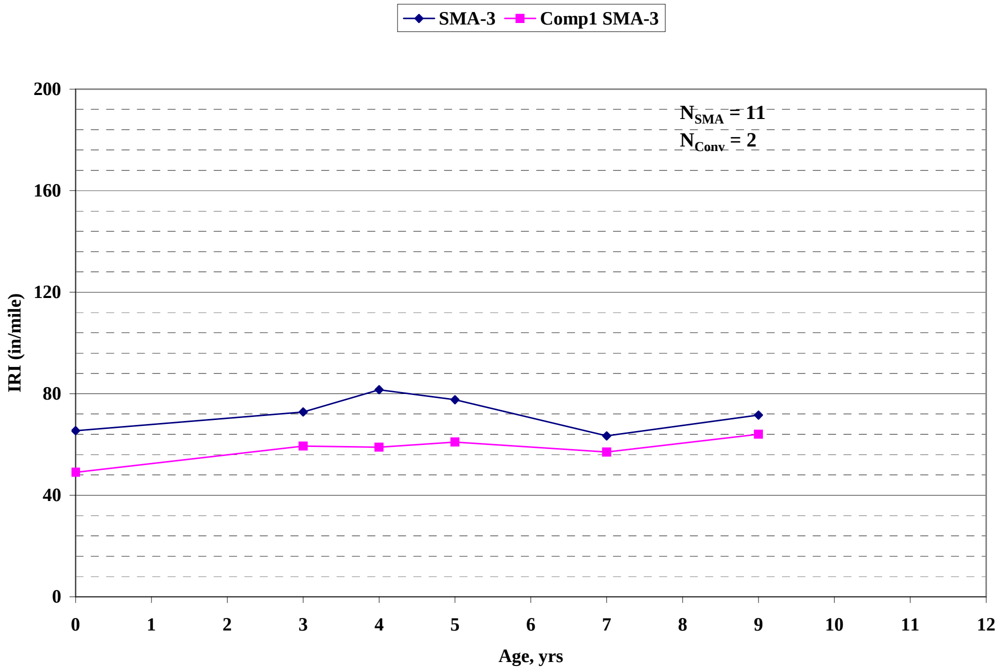


Figure A-18. IRI plot for comparison 3 (USH-63 NB/SB, Washburn/Sawyer Counties and USH-63 NB/SB, Washburn/Sawyer Counties).

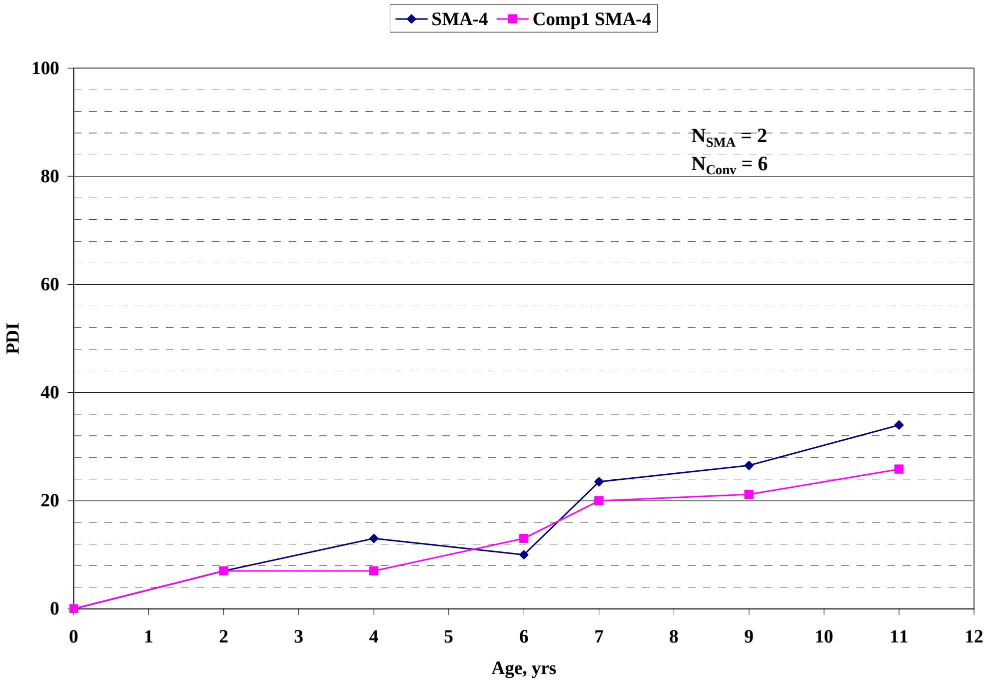


Figure A-19. PDI plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

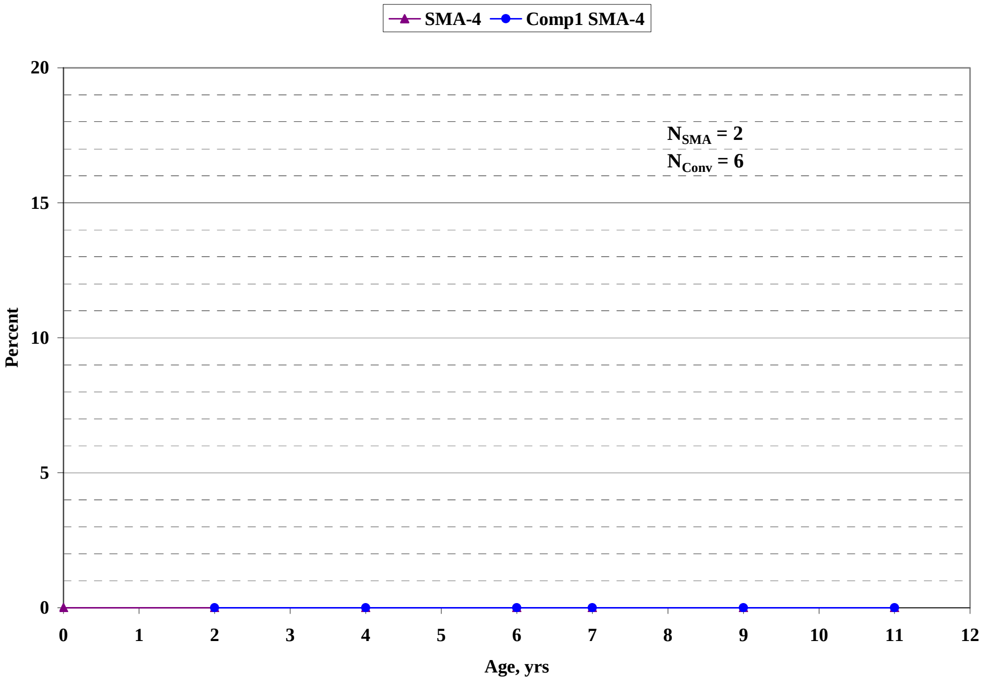


Figure A-20. Alligator cracking plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

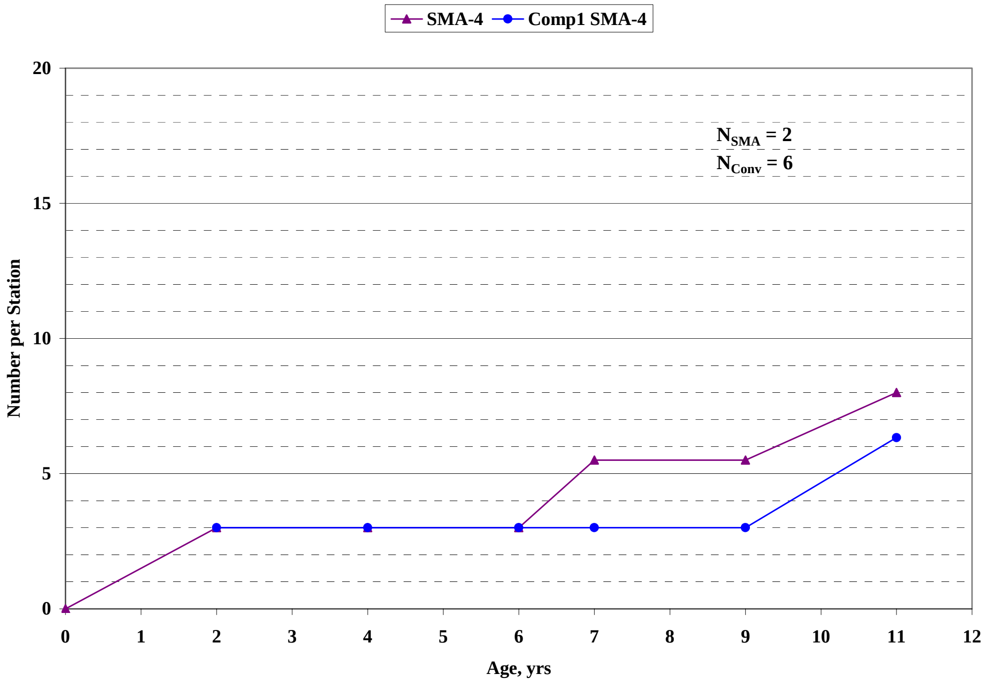


Figure A-21. Transverse cracking plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

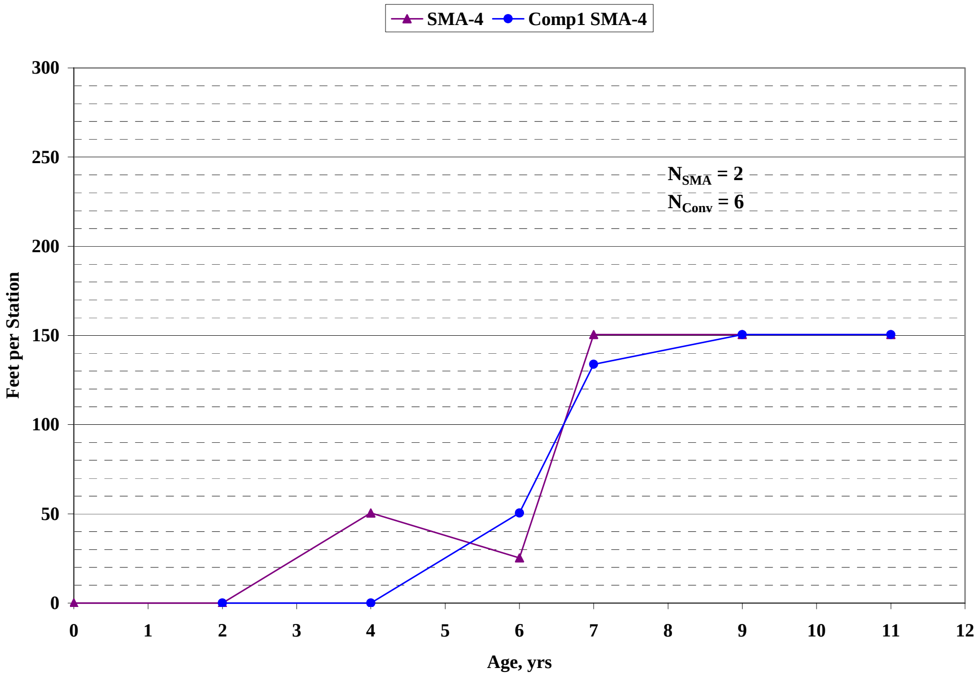


Figure A-22. Longitudinal cracking plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

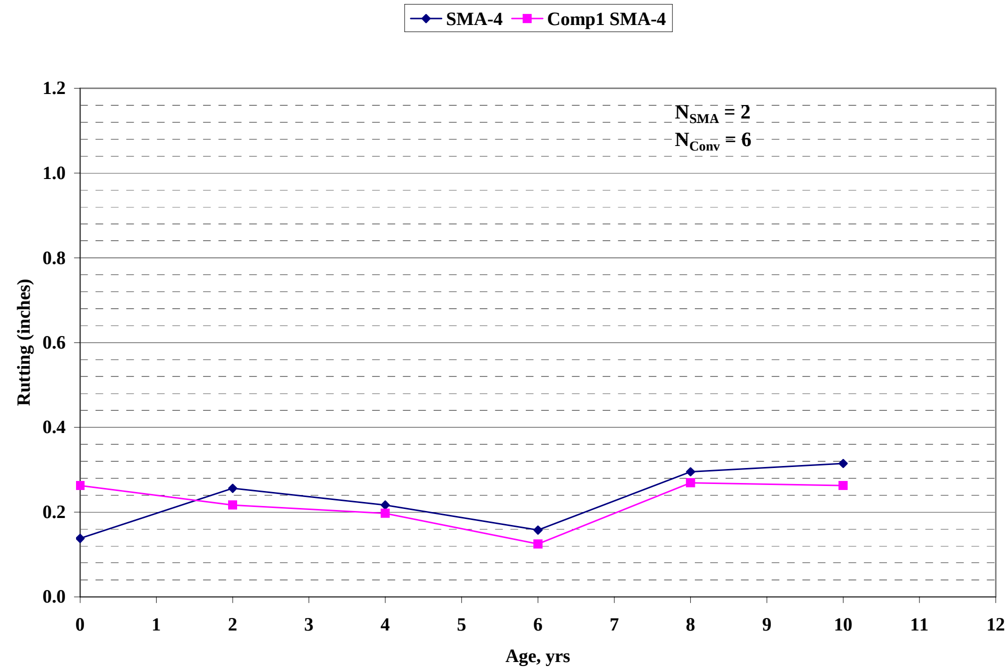


Figure A-23. Rutting plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

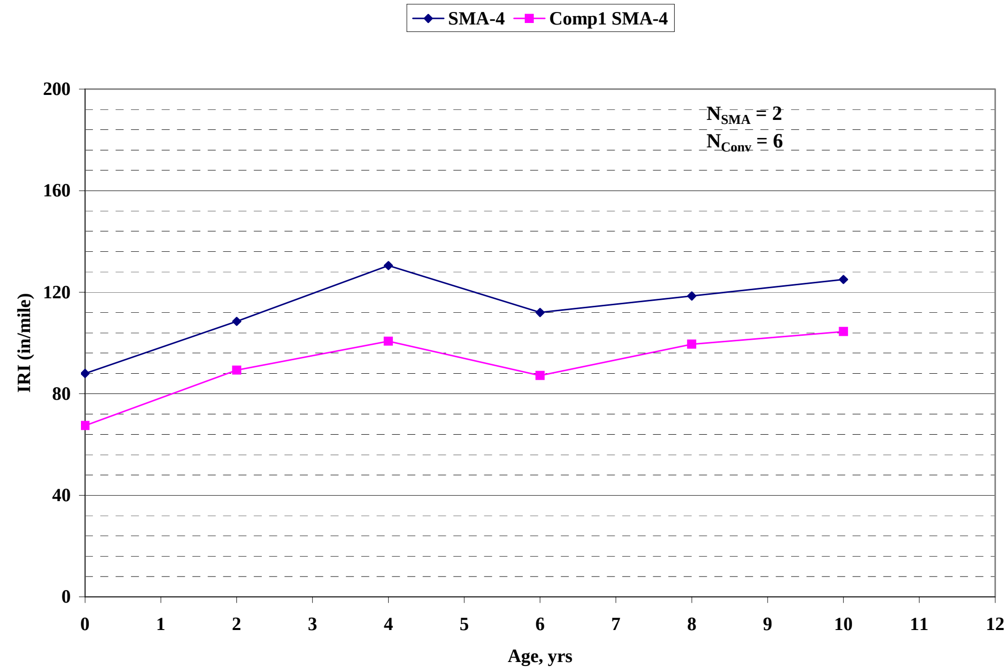


Figure A-24. IRI plot for comparison 4 (STH-100 NB/SB, Milwaukee County and USH-145 NB/SB, Milwaukee County).

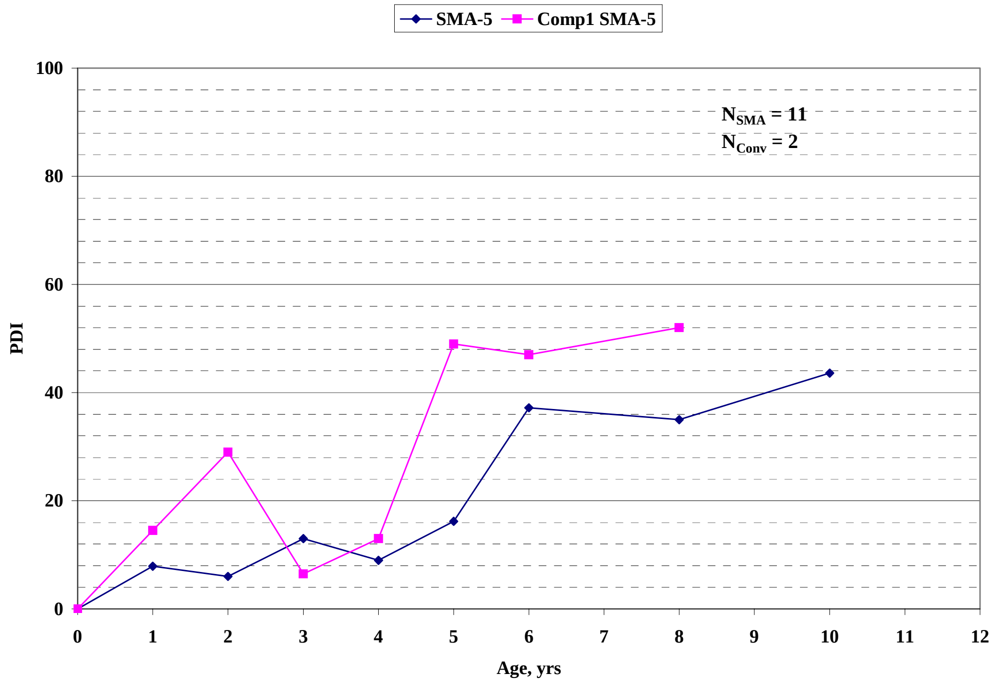


Figure A-25. PDI plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

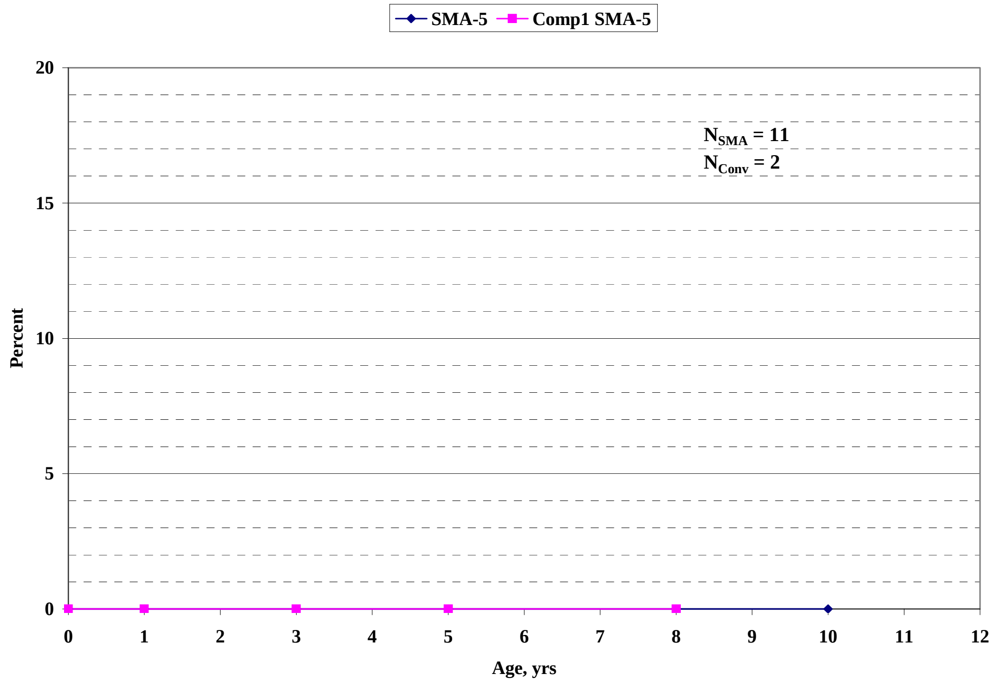


Figure A-26. Alligator cracking plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

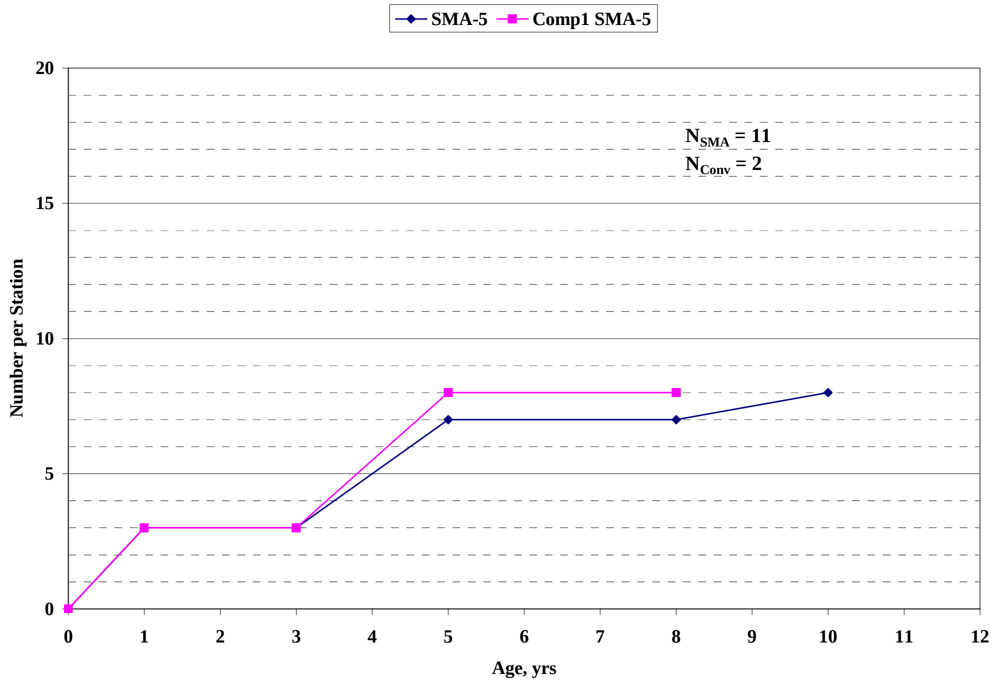


Figure A-27. Transverse cracking plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

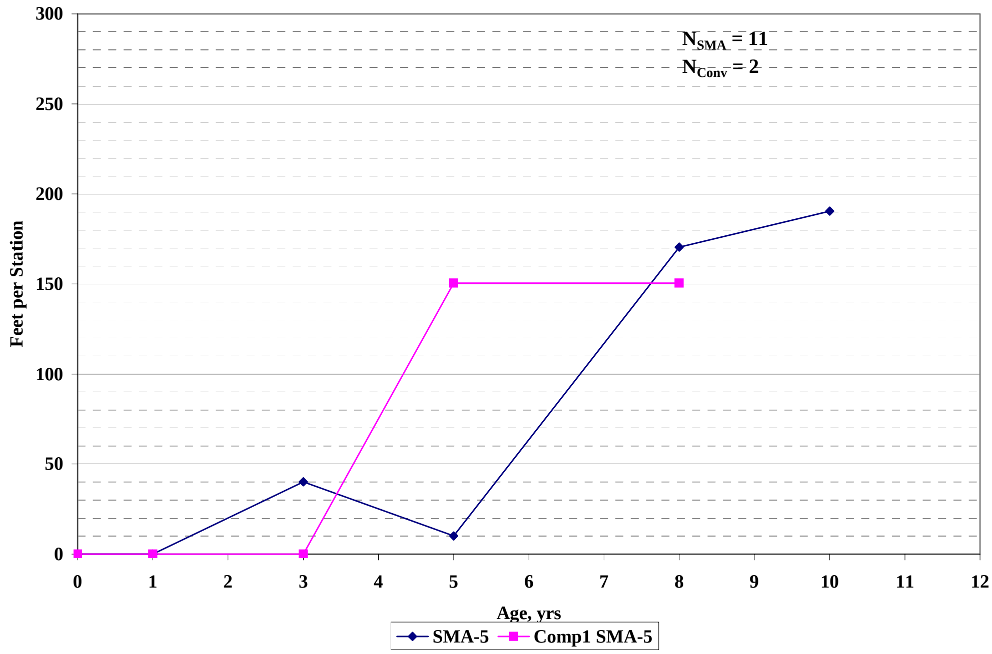


Figure A-28. Longitudinal cracking plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

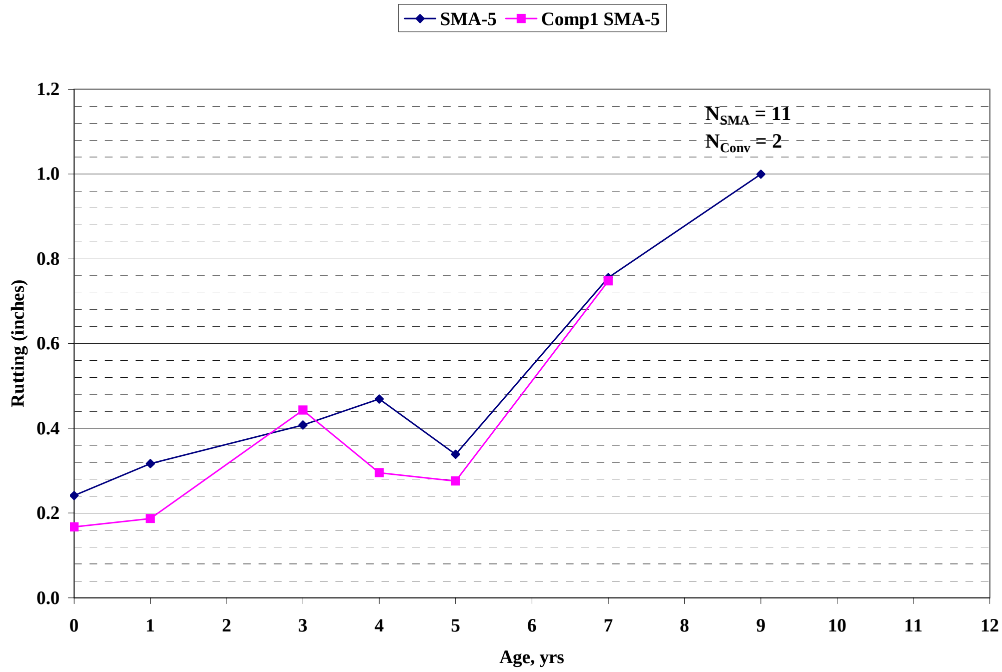


Figure A-29. Rutting plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

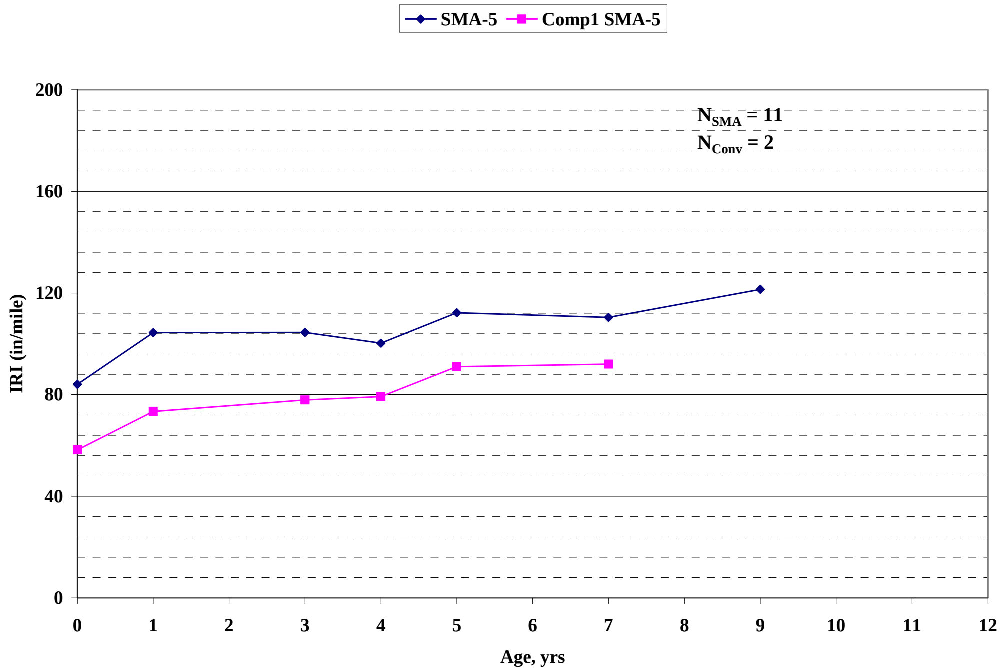


Figure A-30. IRI plot for comparison 5 (USH-151 NB/SB, Grant/Lafayette Counties and USH-151 NB/SB, Grant/Lafayette Counties).

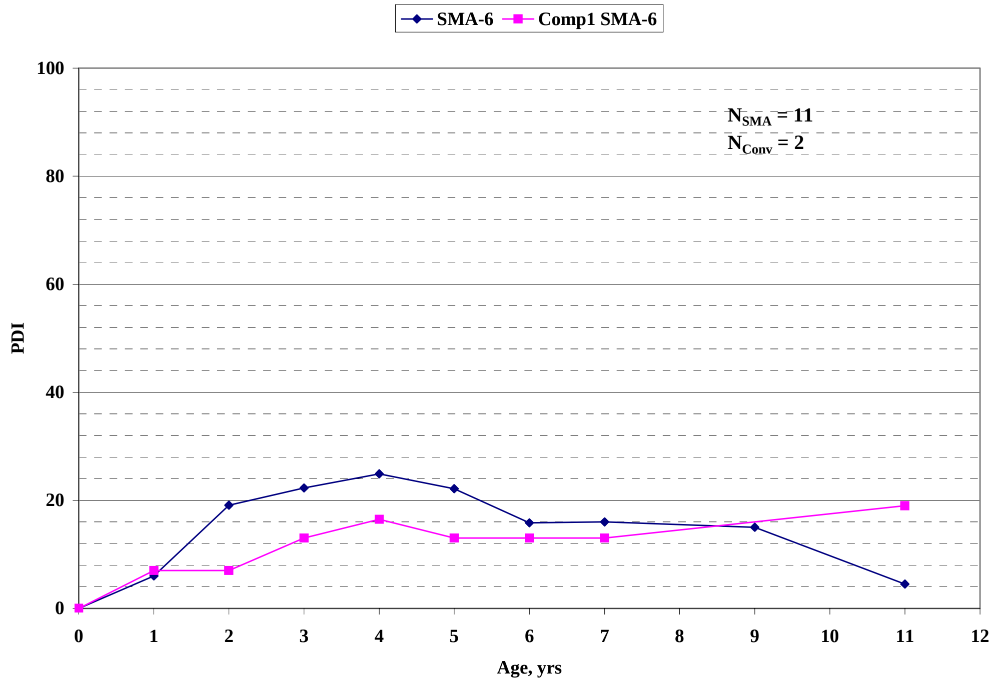


Figure A-31. PDI plot for comparison 6 (US 45 NB/SB Vilas/Oneida Counties and US 45 NB/SB Vilas/Oneida Counties).

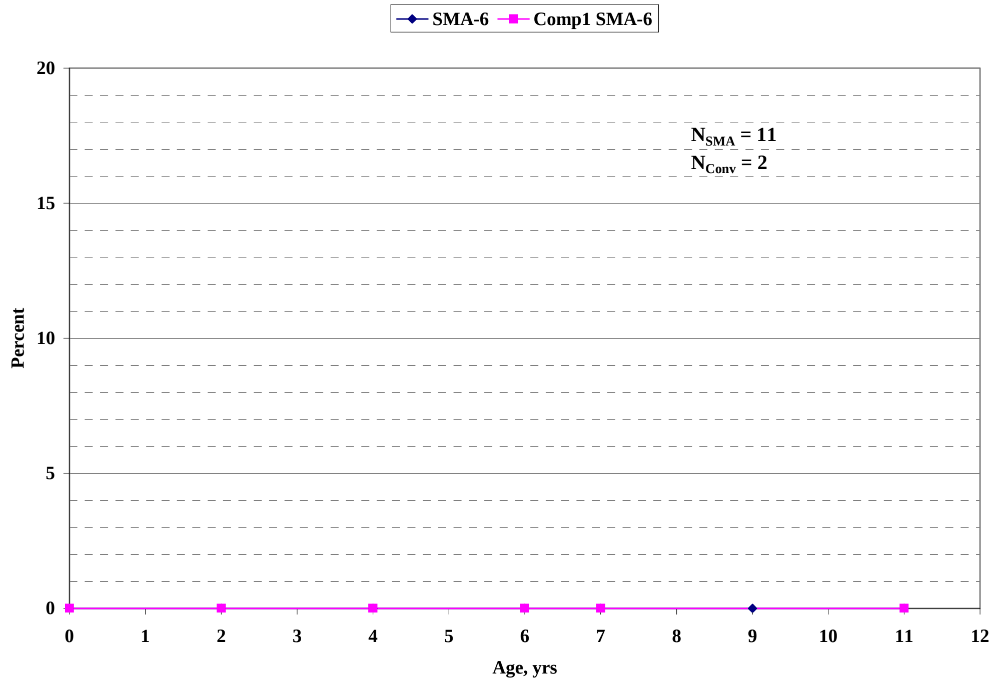
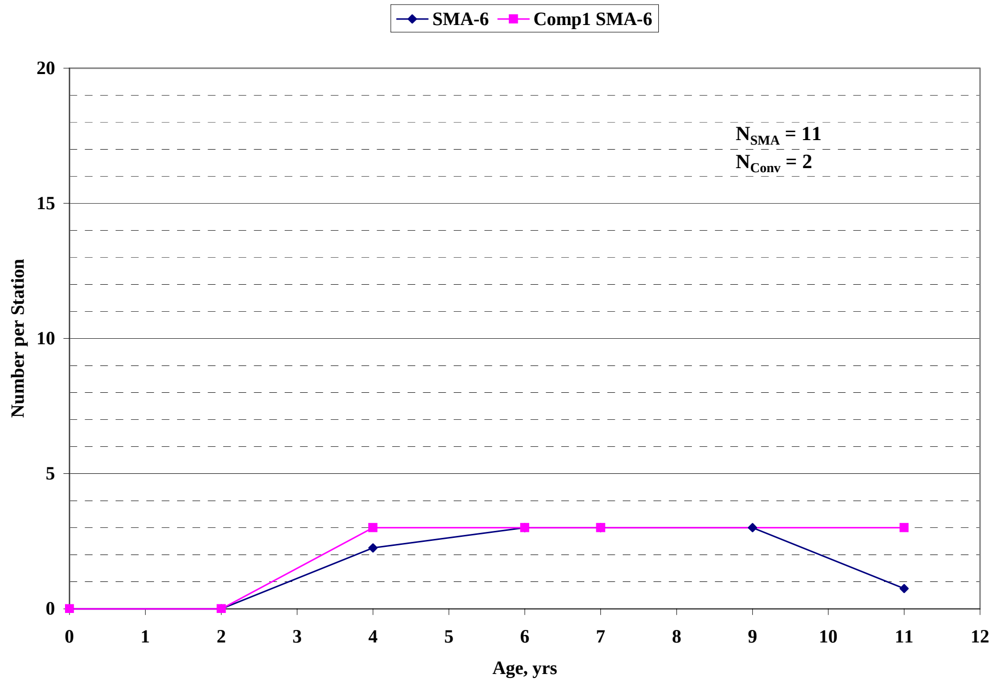


Figure A-32. Alligator cracking plot for comparison 6 (US 45 NB/SB Vilas/Oneida Counties and US 45 NB/SB Vilas/Oneida Counties).



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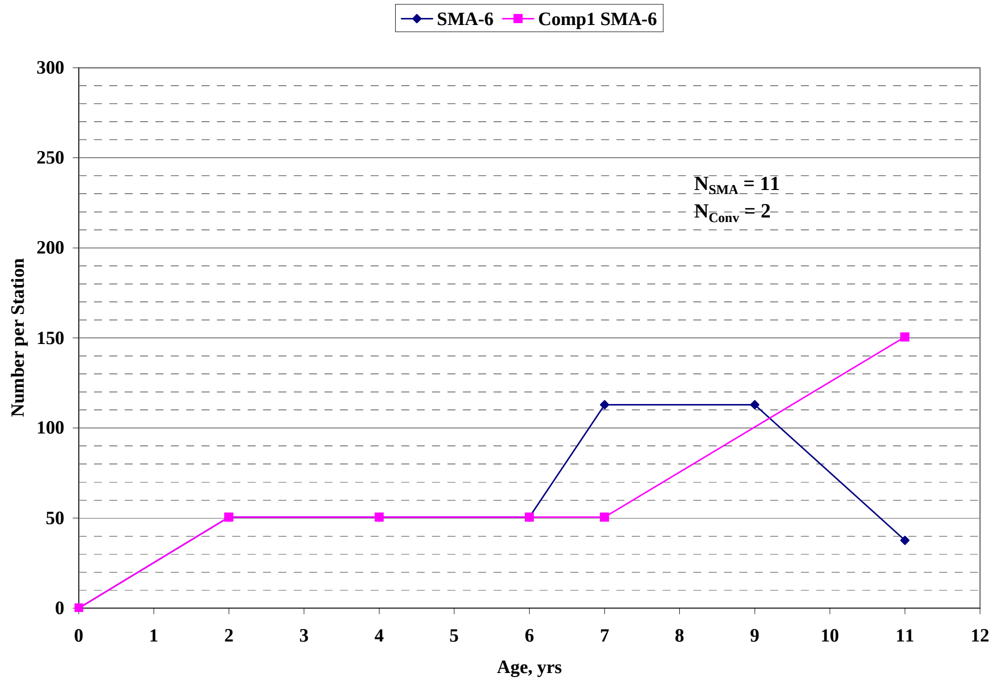

Figure A-34. Longitudinal cracking plot for comparison 6 (US 45 NB/SB Vilas/Oneida Counties and US 45 NB/SB Vilas/Oneida Counties).

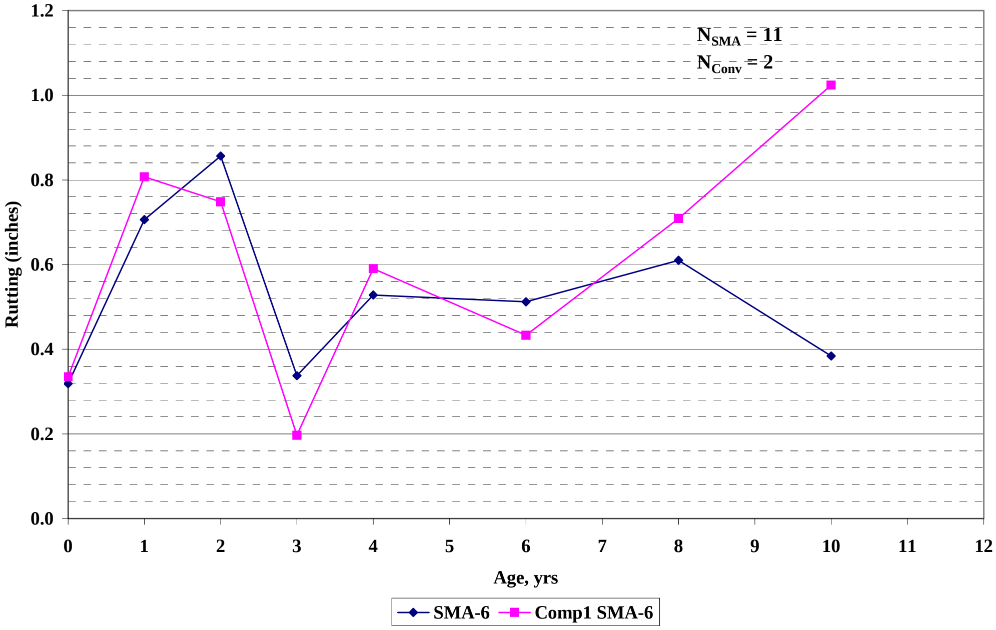

Figure A-35. Rutting plot for comparison 6 (US 45 NB/SB Vilas/Oneida Counties and US 45 NB/SB Vilas/Oneida Counties).

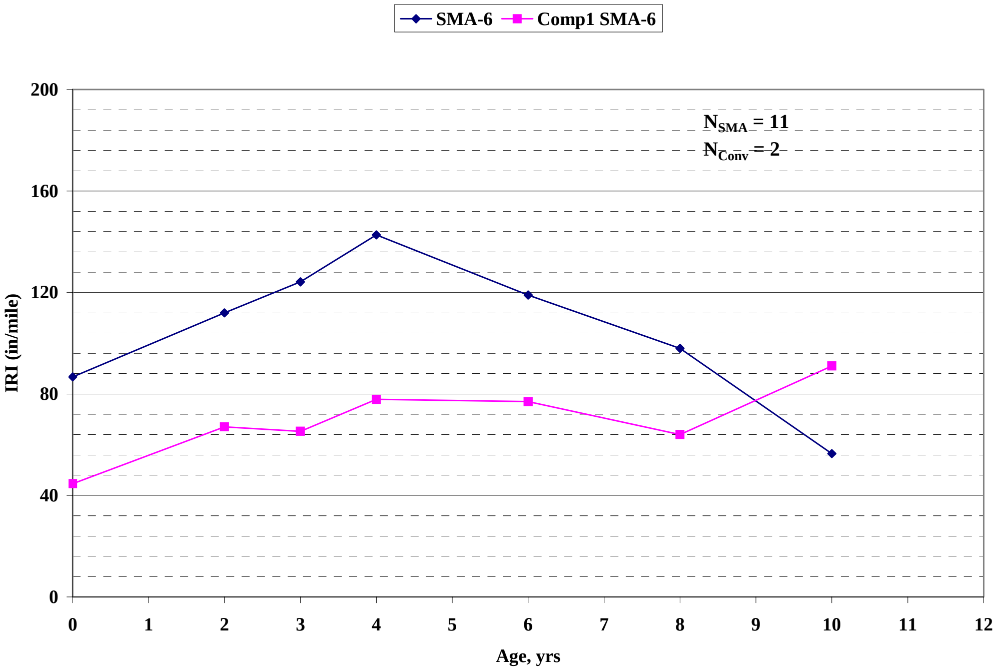

Figure A-36. IRI plot for comparison 6 (US 45 NB/SB Vilas/Oneida Counties and US 45 NB/SB Vilas/Oneida Counties).

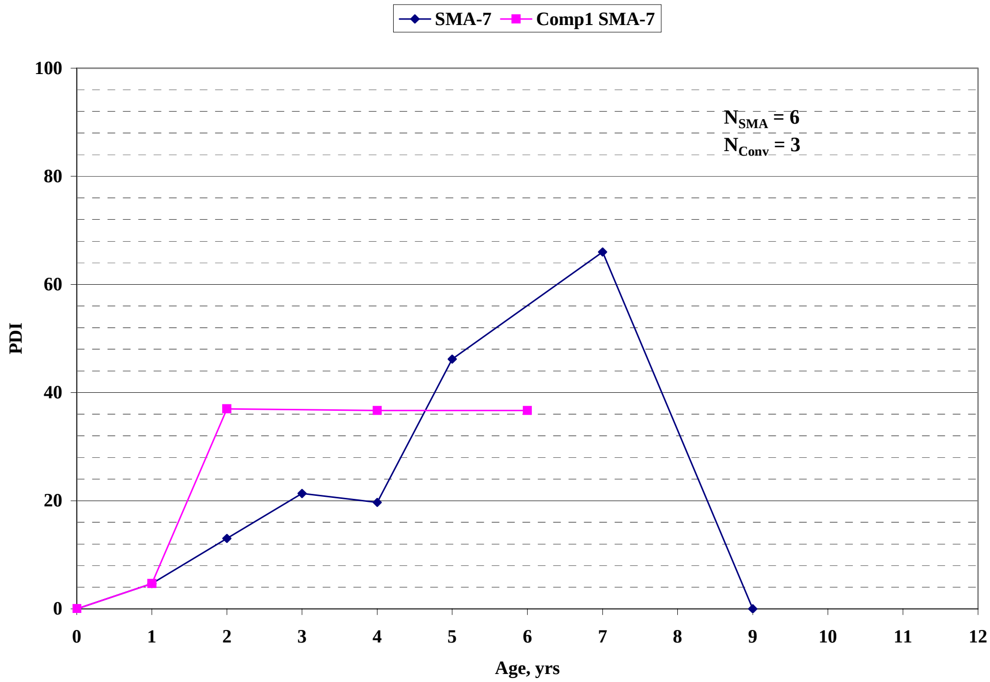

Figure A-37. PDI plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

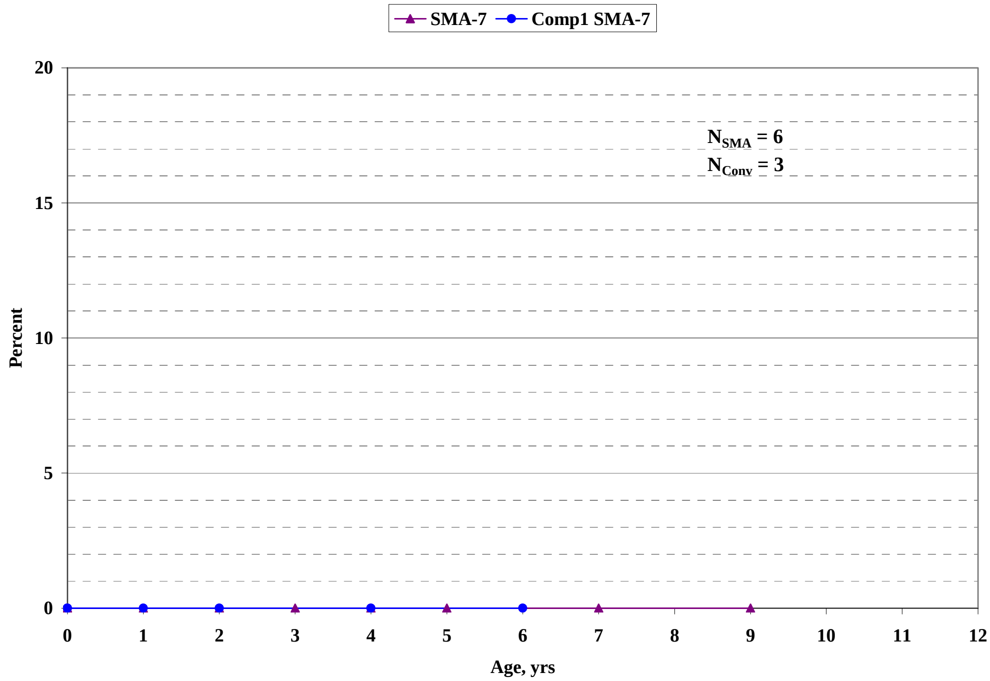

Figure A-38. Alligator cracking plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

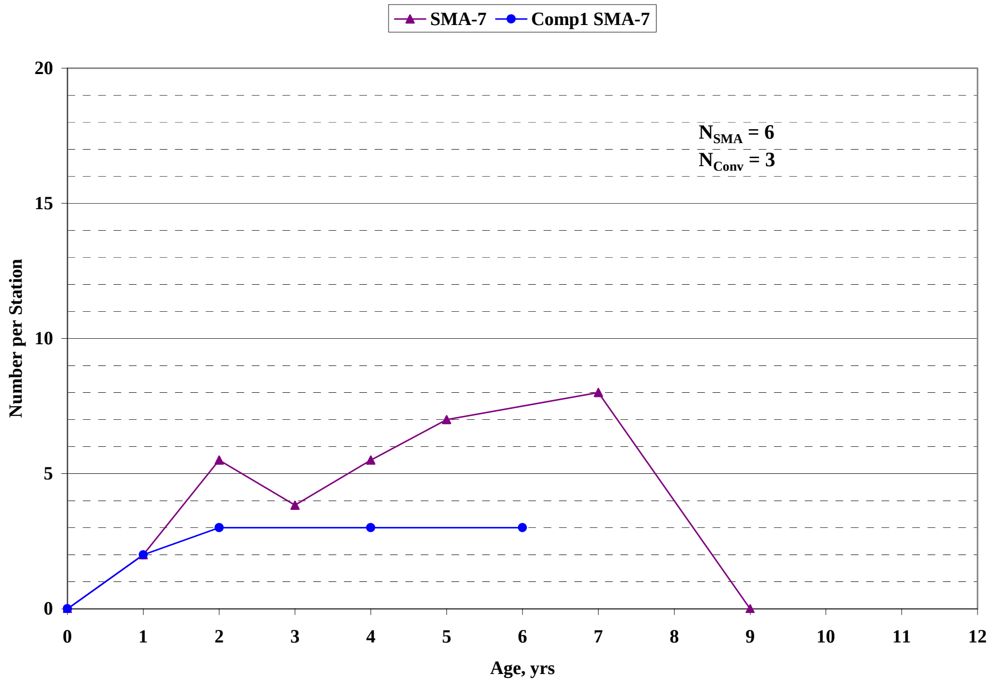

Figure A-39. Transverse cracking plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

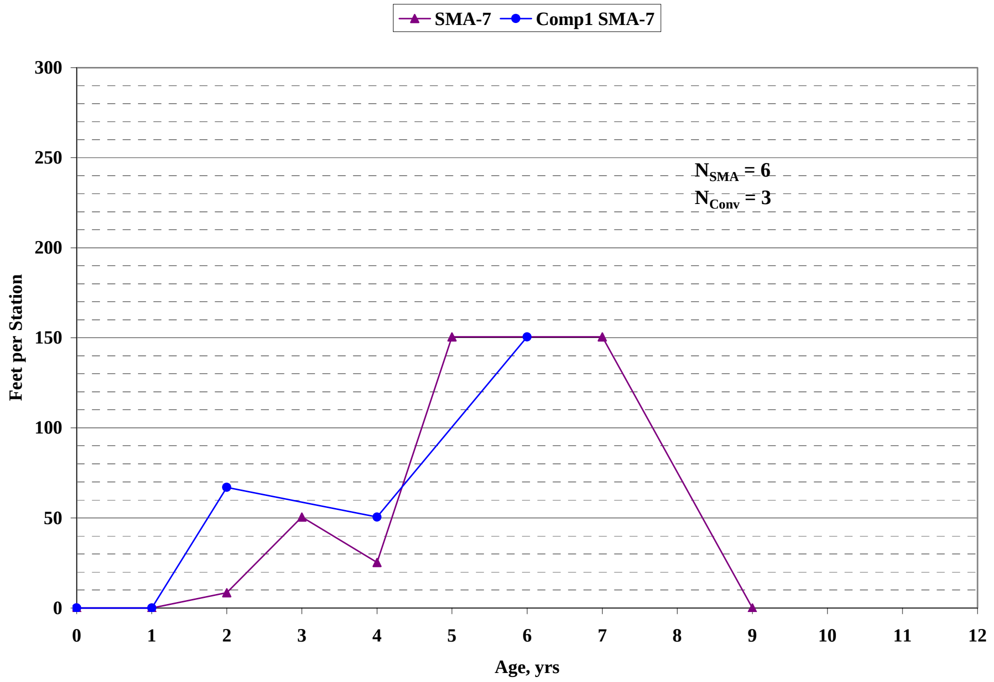

Figure A-40. Longitudinal cracking plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

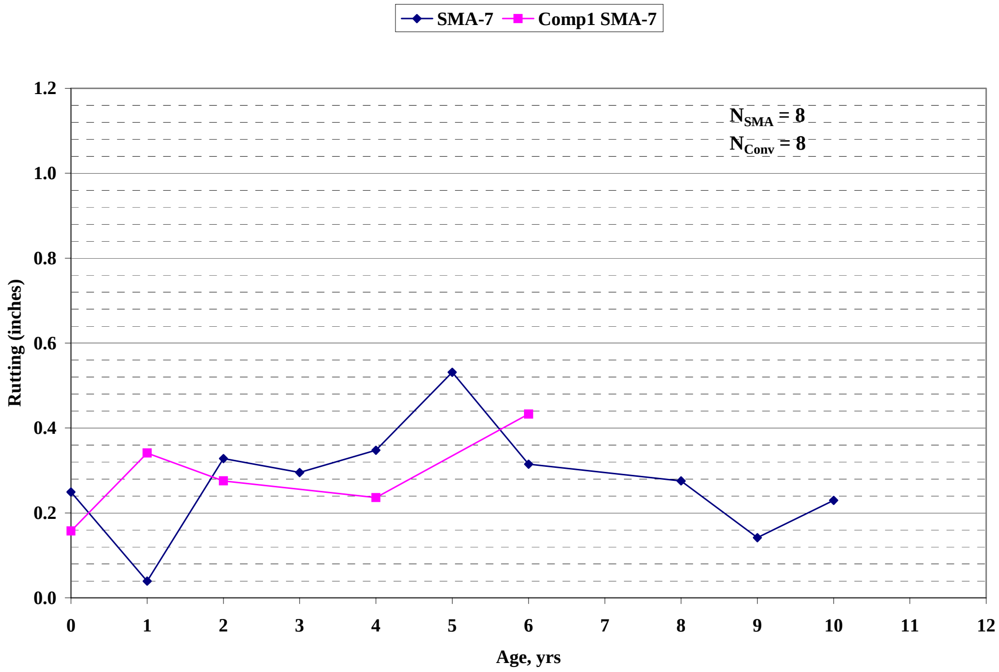

Figure A-41. Rutting plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

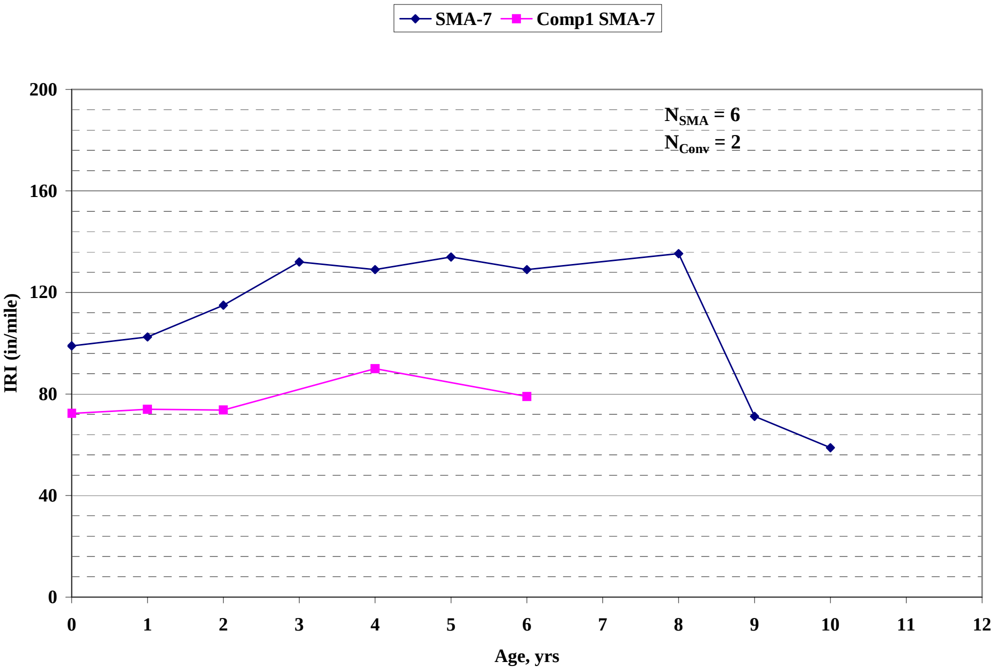

Figure A-42. IRI plot for comparison 7 (I-894 EB/WB, Milwaukee County and I-94 EB /WB, Milwaukee County).

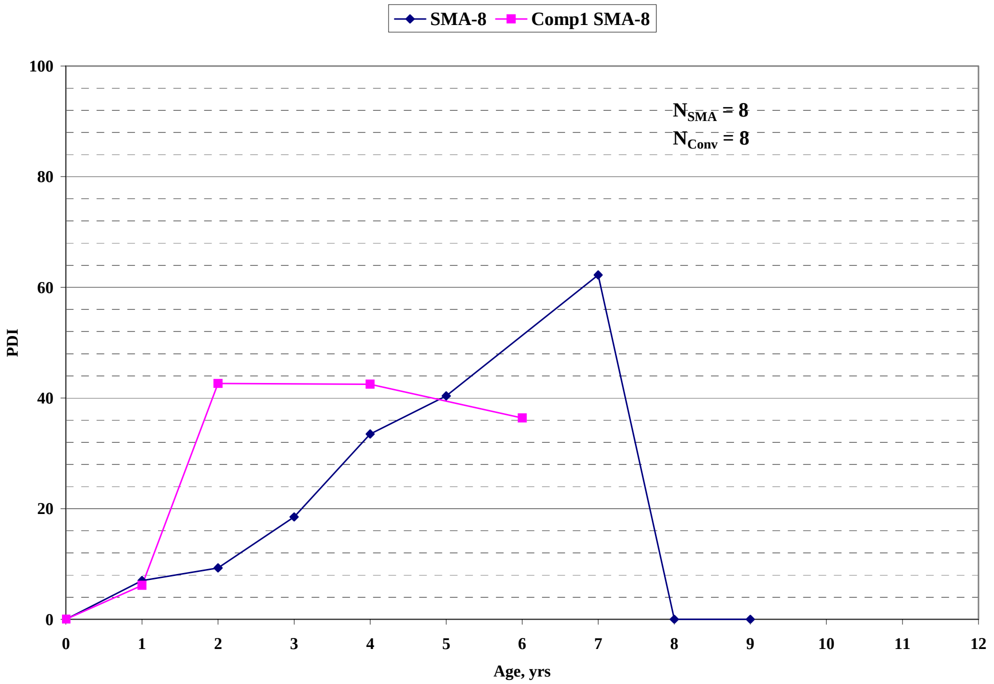

Figure A-43. PDI plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

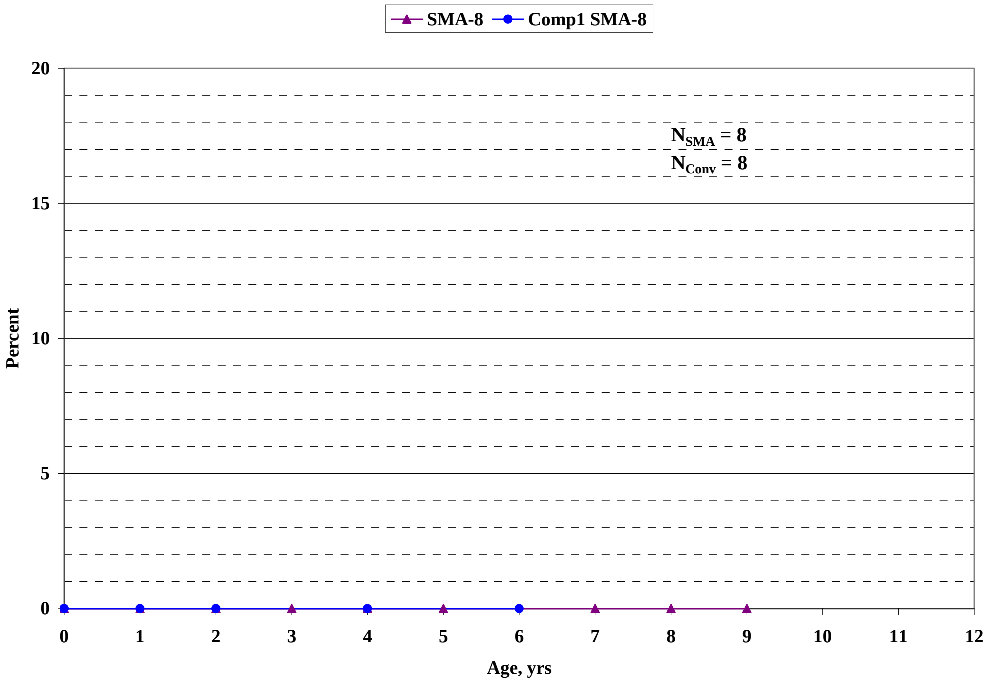

Figure A-44. Alligator cracking plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

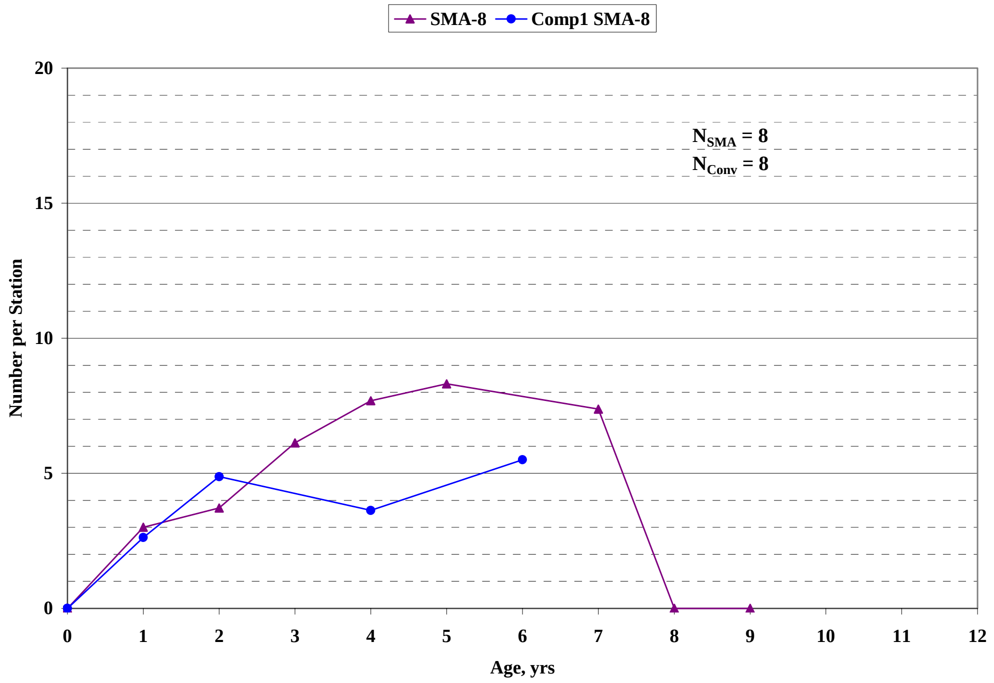

Figure A-45. Transverse cracking plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

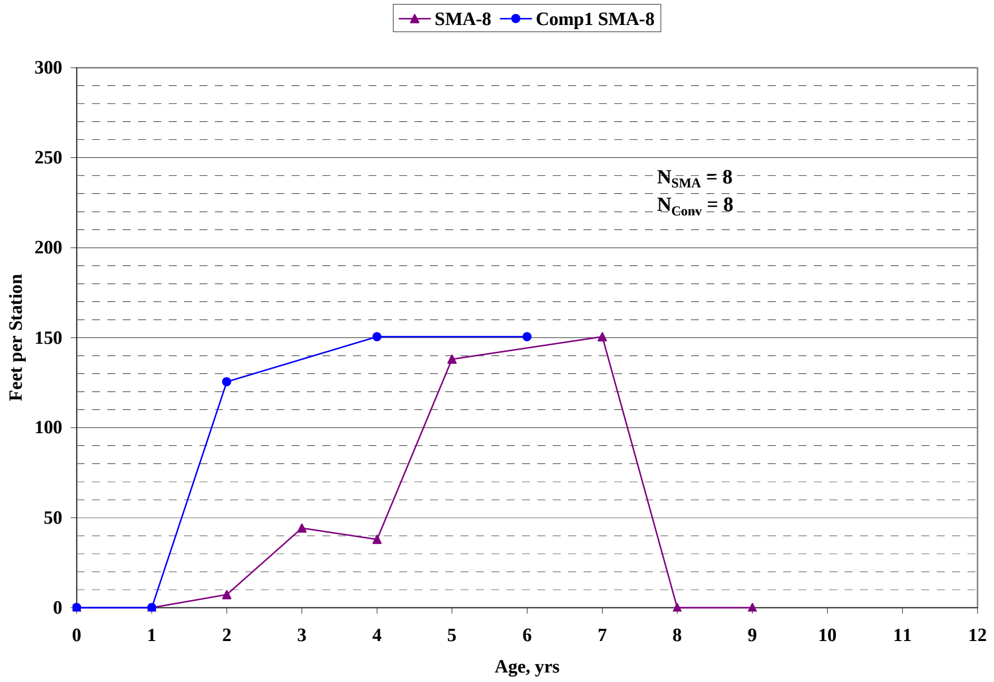

Figure A-46. Longitudinal cracking plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

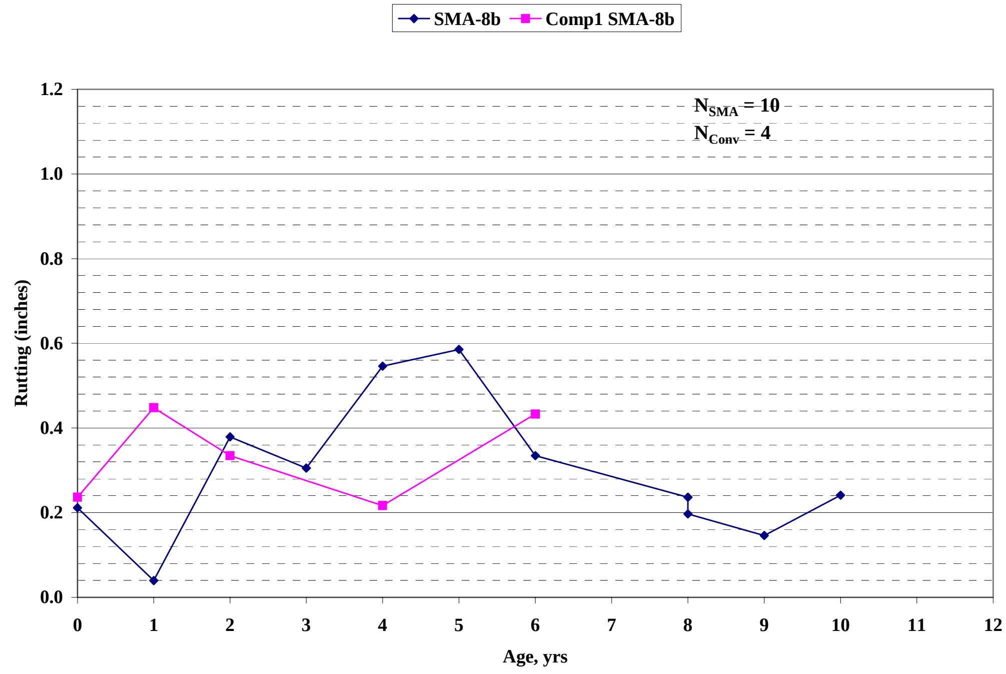

Figure A-47. Rutting plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

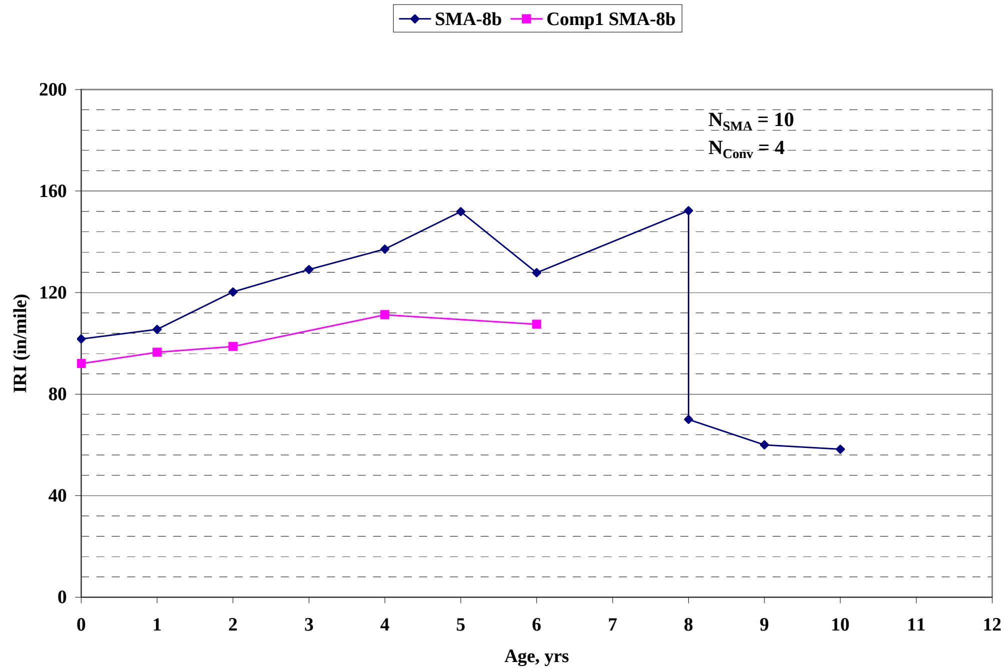

Figure A-48. IRI plot for comparison 8 (I-43 NB/SB, Milwaukee County and I-94 EB/WB, Milwaukee County).

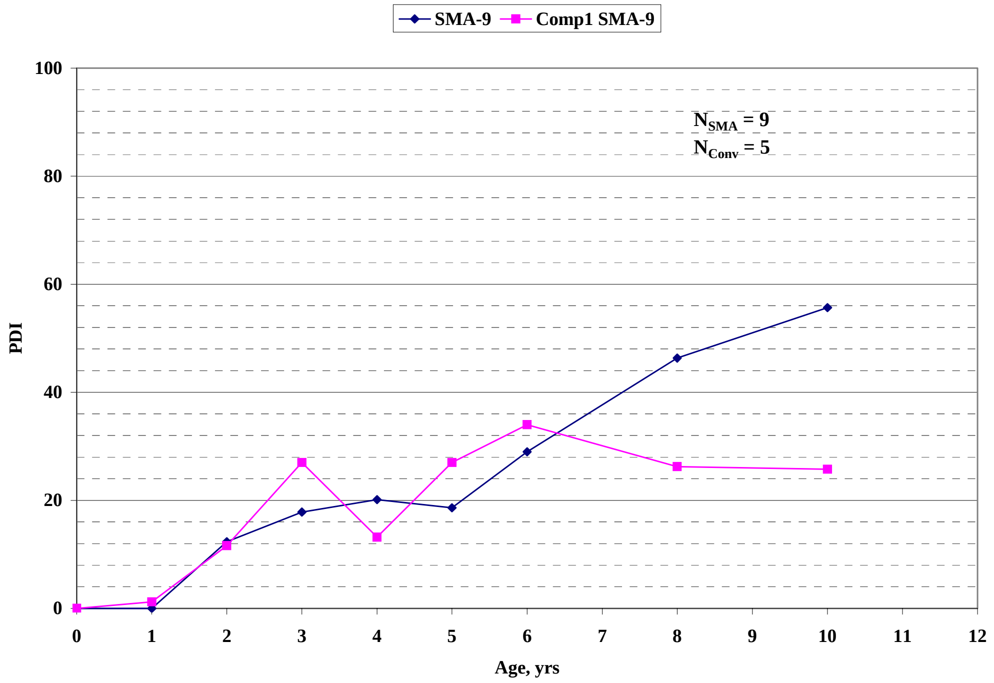

Figure A-49. PDI plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

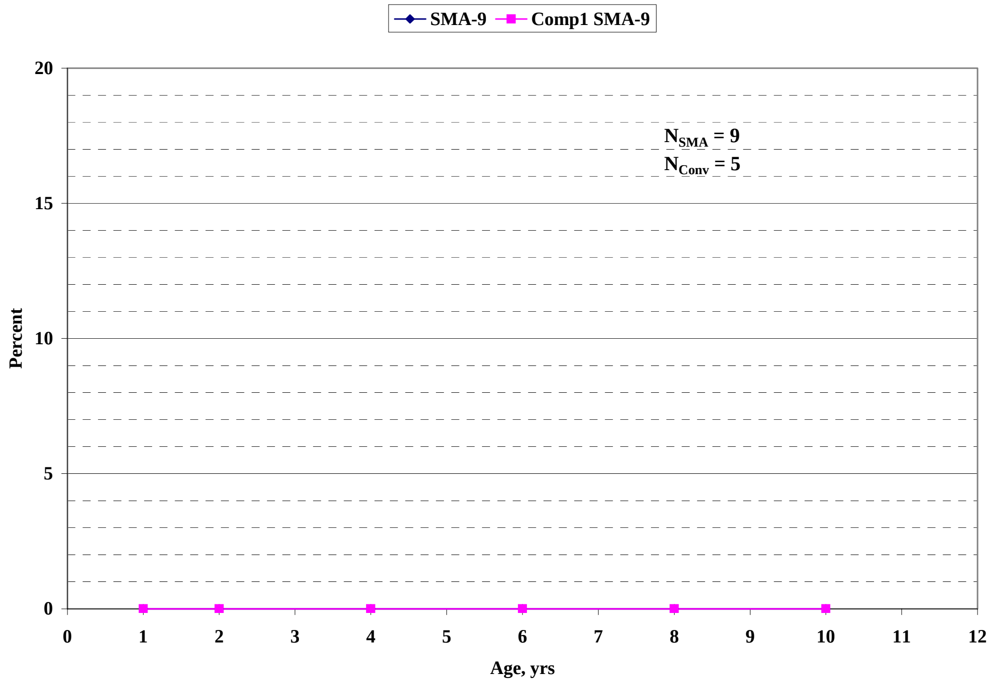

Figure A-50. Alligator cracking plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

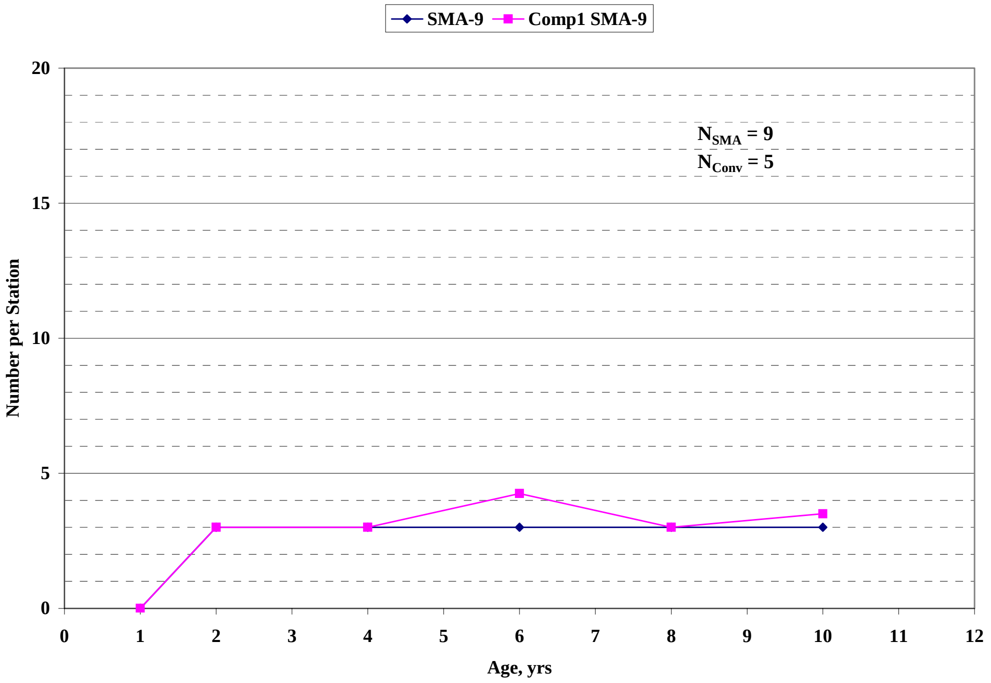

Figure A-51. Transverse cracking plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

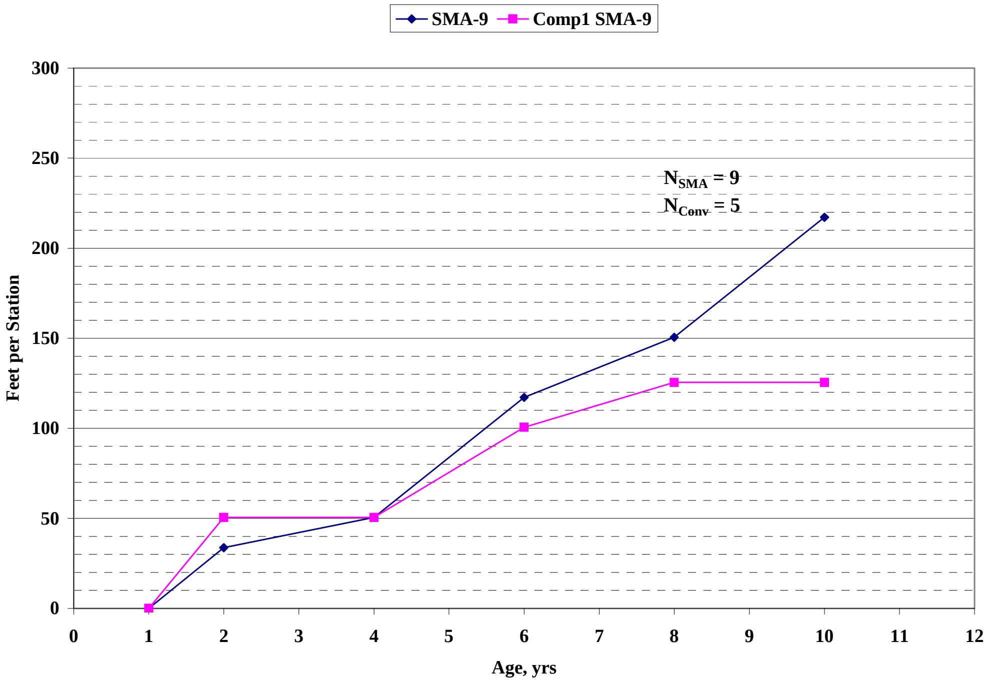

Figure A-52. Longitudinal cracking plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

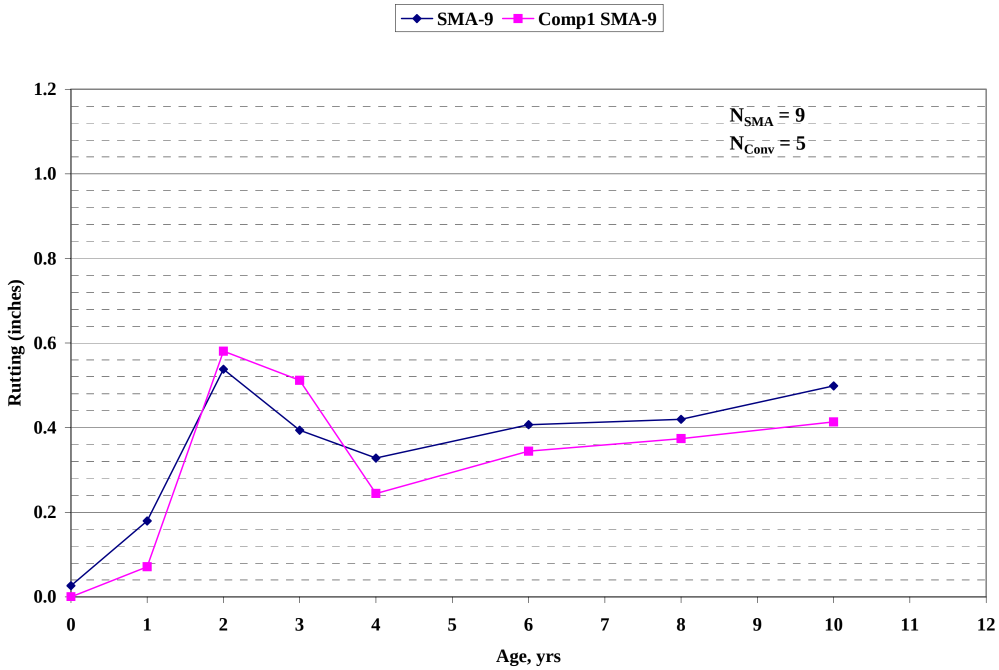

Figure A-53. Rutting plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

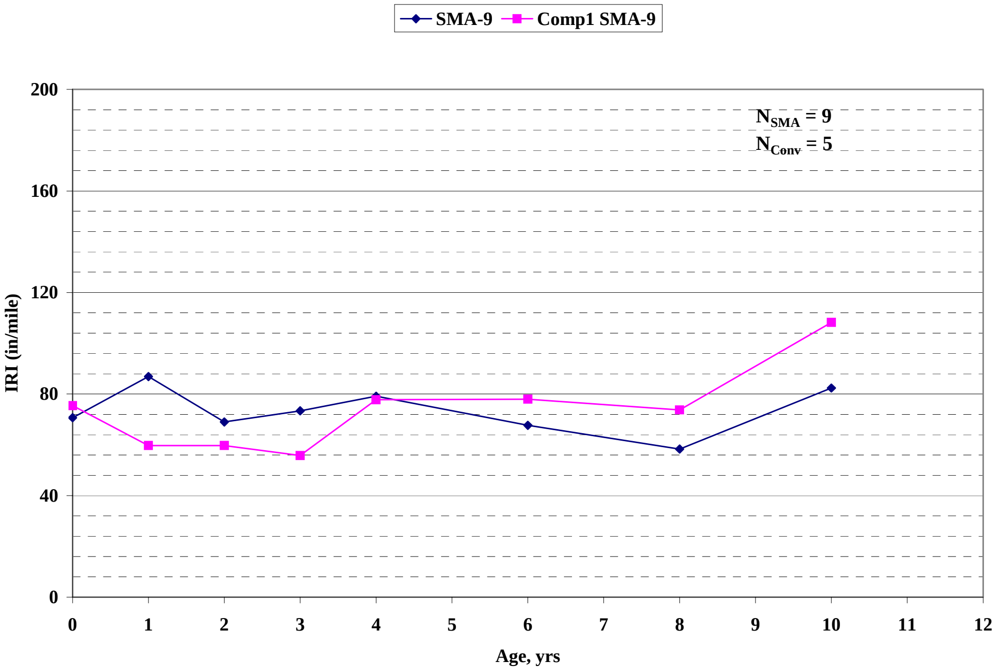

Figure A-54. IRI plot for comparison 9 (STH-21 EB/WB, Juneau County and STH-21 EB/WB, Juneau County).

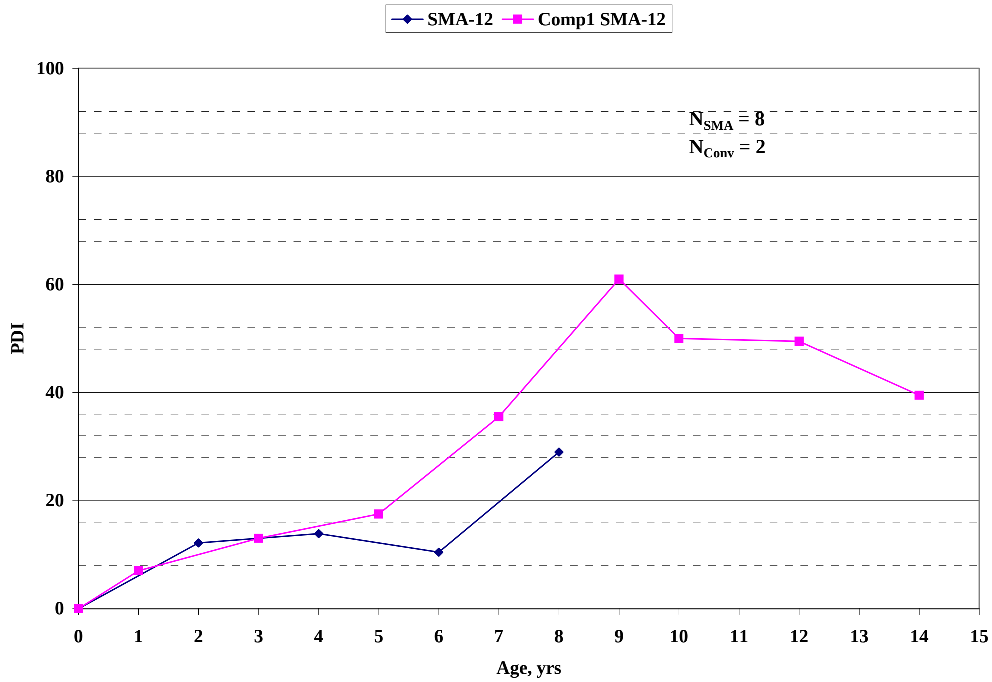

Figure A-55. PDI plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

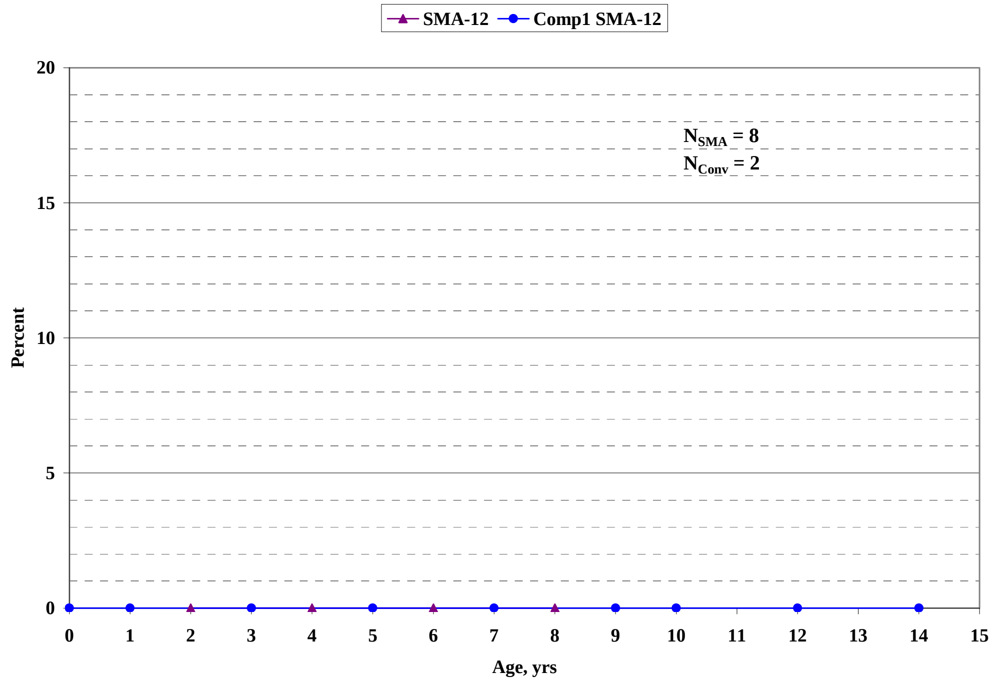

Figure A-56. Alligator cracking plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

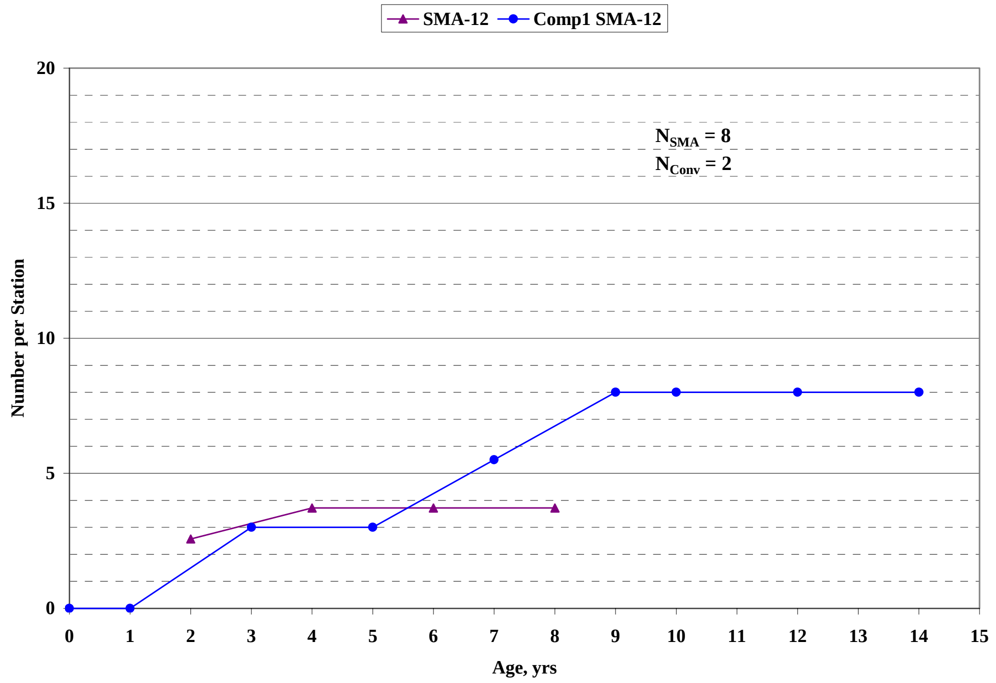

Figure A-57. Transverse cracking plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

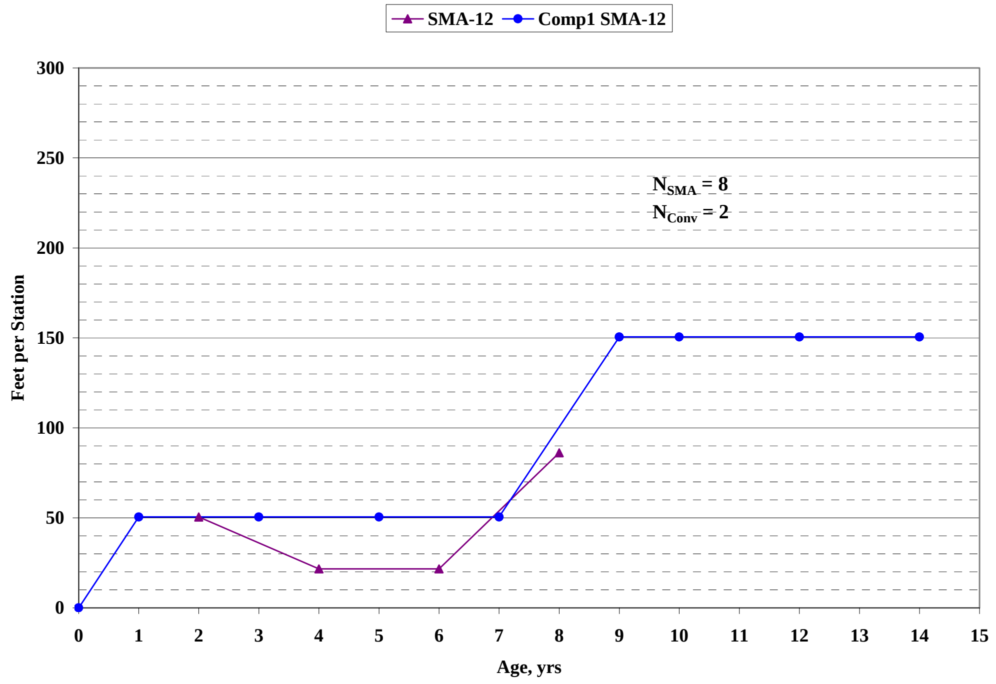

Figure A-58. Longitudinal cracking plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

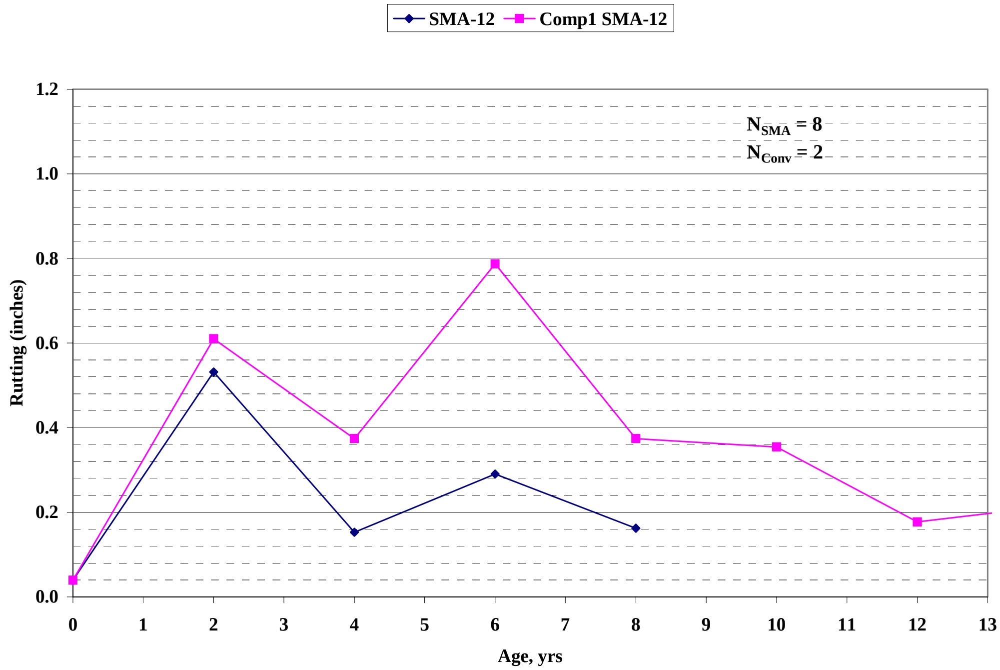

Figure A-59. Rutting plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

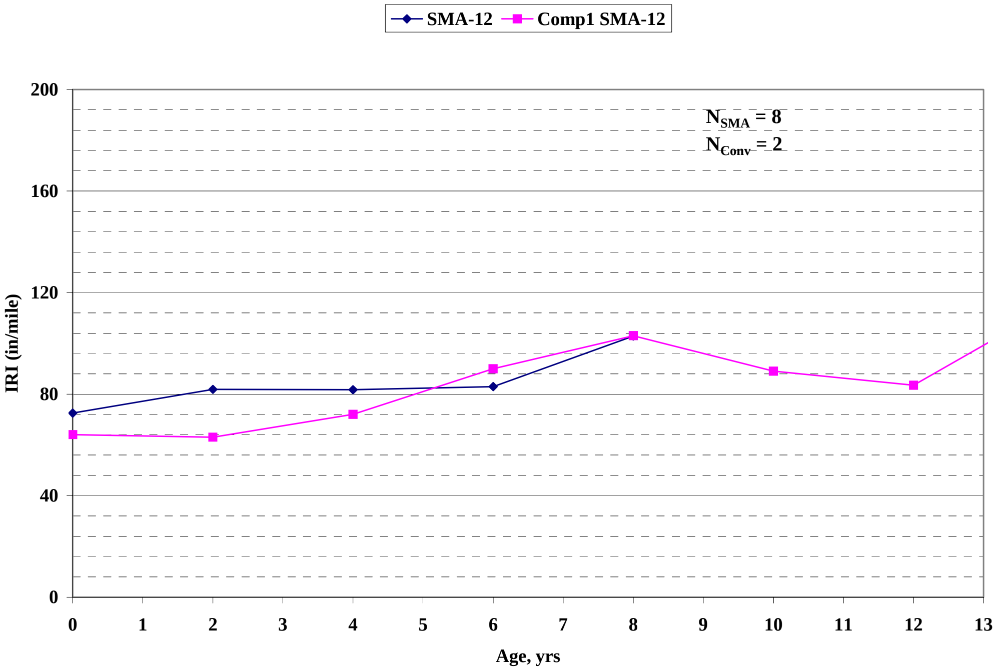

Figure A-60. IRI plot for comparison 10 (USH-41 NB/SB, Winnebago County and USH-41 NB/SB, Winnebago County).

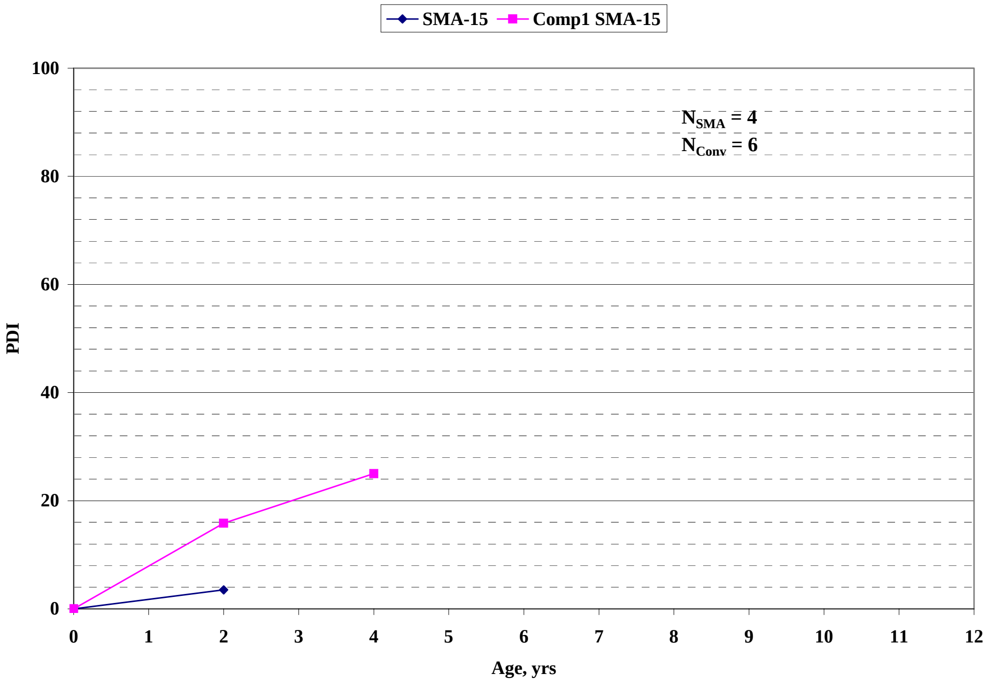

Figure A-61. PDI plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

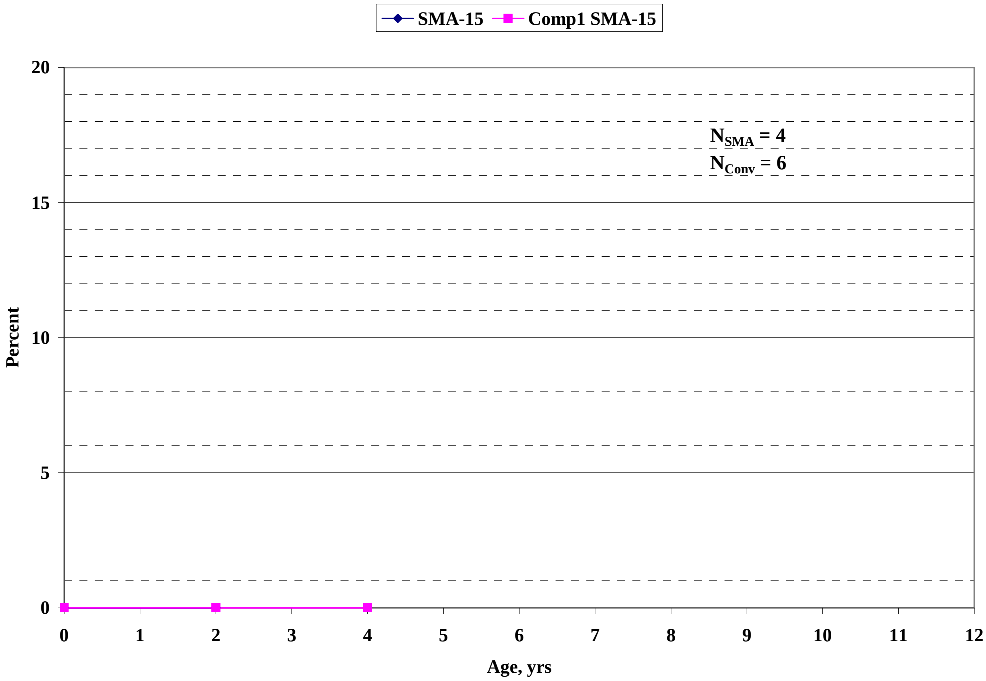

Figure A-62. Alligator cracking plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

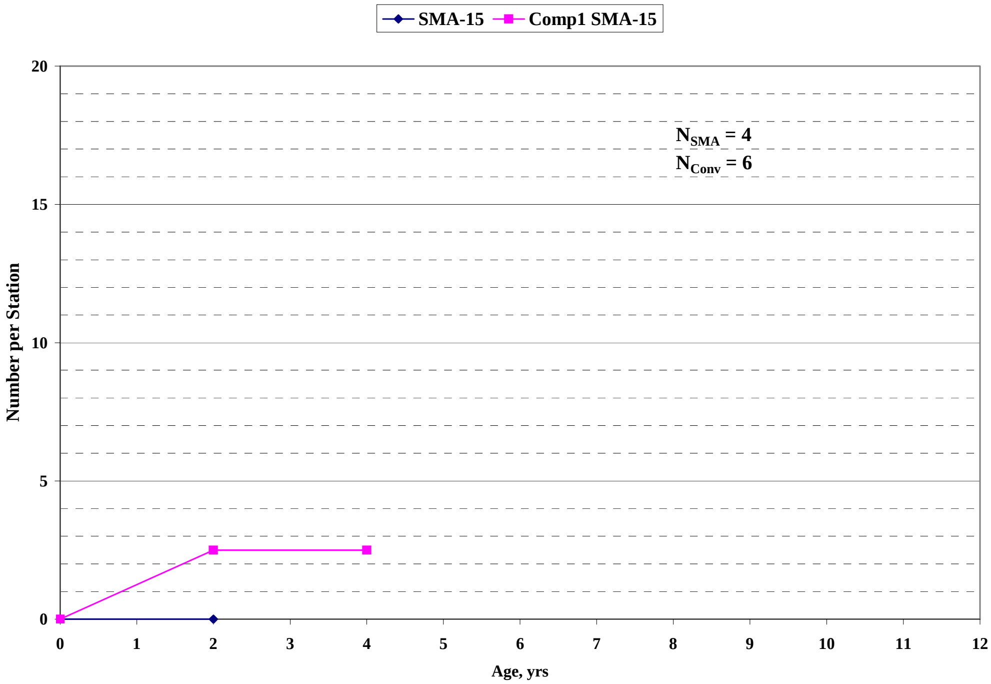

Figure A-63. Transverse cracking plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

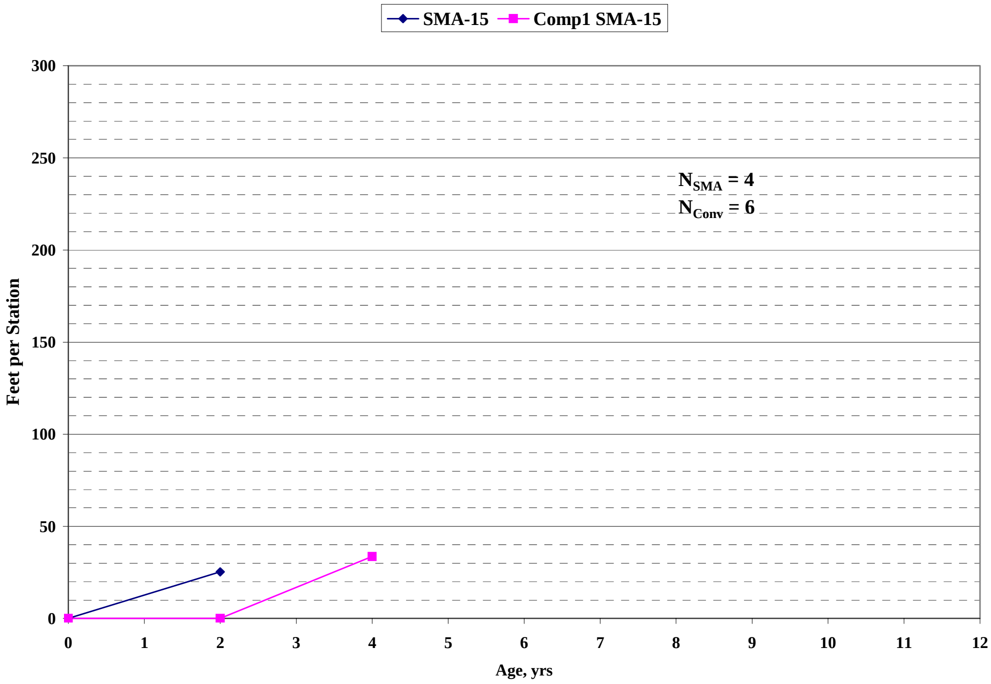

Figure A-64. Longitudinal cracking plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

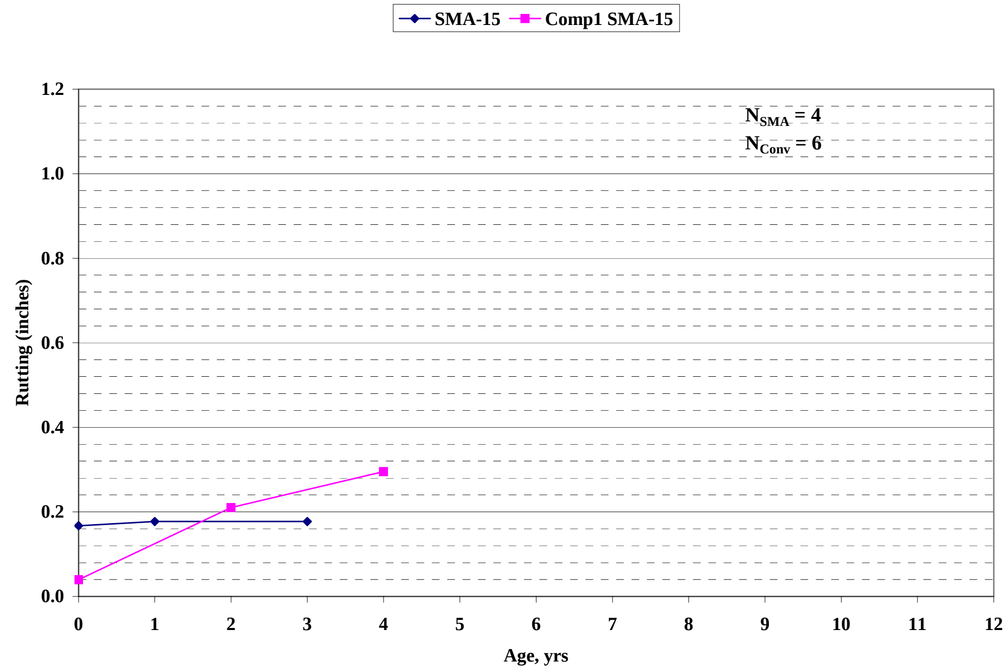

Figure A-65. Rutting plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

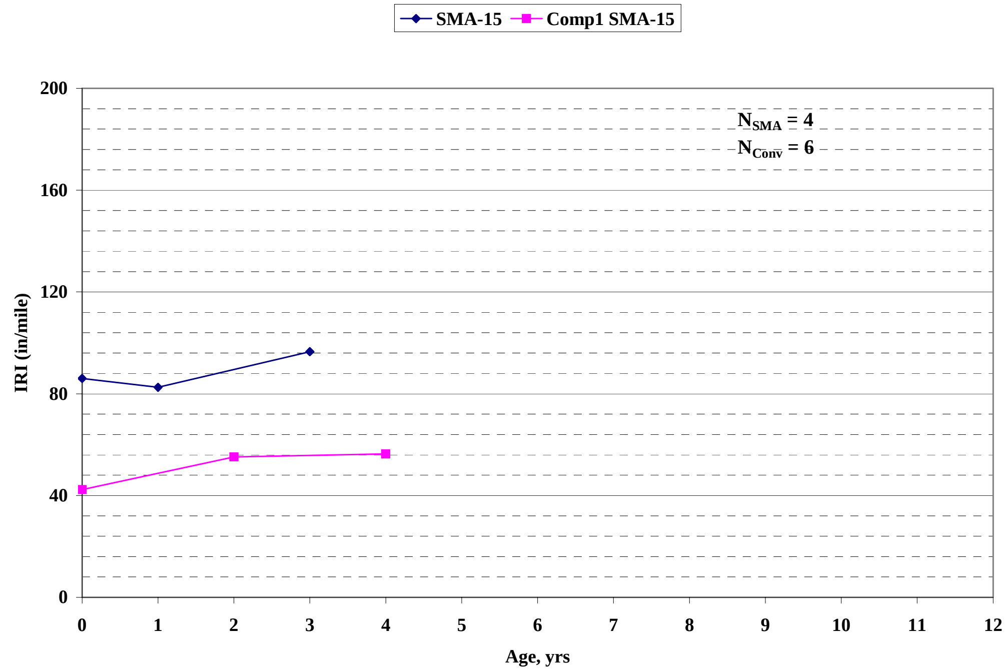

Figure A-66. IRI plot for comparison 11 (STH-29 EB/WB, Pierce County and STH-37 NB/SB, Pierce County).

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