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DEVELOPING PAVEMENT PRESERVATION AND MITIGATION STRATEGIES USING PAVEMENT ME DESIGN GUIDE FOR RHODE ISLAND DOT

by

Sean Coffey

A Thesis

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Thesis Chair: Yusuf Mehta, Ph.D.

Dedication

I would like to dedicate this to my Grandfather Bernard Lombardelli who passed away as I was completing my research.

Acknowledgement

I would like to express my appreciation to Professor Yusuf Mehta for his guidance and help throughout this research. I would also like to thank my parents, Dona and Dennis Coffey, and Sarah Gettings for being supportive through this whole process.

Abstract

Sean Coffey
PAVEMENT DESIGN AND MITIGATION FOR RIDOT USING PAVEME
2012/13
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The objective of the study is to develop pavement preservation strategies using Pavement ME. The International Roughness Index (IRI), Longitudinal Cracking, and Permanent Deformation (Rutting) in both the asphalt layer and the total structure were predicted using Pavement ME to determine the sensitivity of flexible pavement performance with varying pavement parameters, such as thickness of the surface and existing hot mix asphalt (HMA) layers, traffic volume, binder grade of the surface HMA layer and the base layer modulus. The surface HMA layer thickness and traffic volume were significant at 95% confidence level for both types of rutting and terminal IRI. The existing HMA layer thickness was significant for all four distresses. All five properties were significant with a 90% confidence level for terminal IRI. Performance curves were developed using the pavement management scoring system for the overall pavement and individual distresses. Two traffic distributions were identified from the weigh-in-motion stations for this analysis. The first distribution had a 22% higher concentration of Class 9 trucks and the second distribution had over 80% of Class 5 trucks. The crack seal and stress absorbing membrane interlayer alternatives were the most cost effective in all cases. Chip Seal was cost effective for the three and five layer structures with the Class 9 heavy truck distribution and with four layer structure for the Class 5 heavy truck distribution.

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Chapter 1

Introduction and Approach

1.1 Introduction

The mechanistic empirical design guide also called MEPDG, DarwinME, or Pavement ME Design is on the path to be approved by American Association of State Highway Transportation Officials (AASHTO). In the near future and state agencies, including Rhode Island Department of Transportation (RIDOT), must be able to design using the new guide. Transitioning to a mechanistic-empirical design process represents a huge paradigm shift for the majority of State DOTs, such as Rhode Island. This will require a tremendous amount of education, training, new testing equipment and specifications, and data collection. Most importantly, it will require better communication and coordination between the design engineers, materials engineers, traffic engineers and consultants. This communication level, for the different transportation agencies, is essential to collecting and maintaining the data needed for optimizing pavement designs while continuing to validate and calibrate the MEPDG performance prediction models. Therefore, a significant amount of time, resources, and funding is needed in order to facilitate implementation of the MEPDG by State DOTs. Nonetheless, the adoption of the MEPDG design process has the potential for providing a substantial long-term savings based on the sheer magnitude of annual expenditures for highway pavements.

Before State DOTs, including RIDOT, can fully implement Pavement ME; however, there is a need to conduct analyses with Pavement ME to determine which factors influence predicted pavement performance in the state of Rhode Island and

Pavement ME’s use in pavement preservation management. Determining the sensitivity of these factors on predicted performance is a critical step in identifying appropriate pavement preservation strategies.

1.2 Goal

This study was initiated to complete the following objectives:

- Identify key factors that might impact the Pavement ME–predicted flexible pavement performance; and
- Identify cost-effective pavement preservation strategies for RIDOT pavement management system.

1.3 Research Approach

To achieve the overall goal, the research approach will be broken up into two phases. The first phase will analyze the overall predicted performance of selected RIDOT roadways, and conduct sensitivity analysis of the effects of various parameters on predicted pavement performance distresses, and then a statistical analysis will be conducted to determine the significance of the varying parameters on performance. The list of tasks to complete the first phase is listed below.

- Analyze pavement performance by varying different parameters depending on the roadway being analyzed. The purpose of this task was to determine when the pavement structure passes reliability requirements.
- Sensitivity of predicted performance to input parameters
    - *Parameter: Vehicle distribution.* The purpose of this task is to analyze the impact of predicted performance between the two types of vehicle distributions, a national default vehicle distribution versus a local

distribution obtained from Weigh-in-motion sites in the state of Rhode Island.

  - *Parameter: Binder grades*. The purpose of this task is to determine the impact of varying the binder grade of the surface layer typically used in the state of RI on the predicted performance
  - *Parameter: Layer Thickness of surface and existing hot mix asphalt later*. The purpose of this task is to determine the impact that the thickness of the new surface layer and existing Hot Mix Asphalt (HMA) layer have on predicted pavement performance.
  - *Parameter: Traffic volume*. The purpose of this task is to determine the impact of average annual daily truck traffic (AADTT) on predicted performance in the state of Rhode Island.

- Conduct statistical analysis of all these Pavement ME comparison results with SPSS statistical analysis software to determine the relative impact of these parameters on the predicted performance.
- Compare the predicted performance curves to determine the sensitivity of input parameters on pavement performance curves.
- Determine the sensitivity of the performance of actual RIDOT mix on traffic and layer thicknesses using level I analysis to confirm the accuracy of level III analysis.
- Determine the sensitivity of subbase material properties on predicted performance.

The second phase focuses on pavement preservation and management. This analysis will analyze historic roadways structures in Rhode Island and develop cost efficient management plans for a broad range of roadways. The steps necessary for this process are as follows.

- Determine when various pavement management system (PMS) triggers are reached. The purpose of this task is to determine the timeline when the triggers will be reached and appropriate for use.
- Complete cost analysis to determine most efficient plan or alternatives to extend pavement performance. The purpose of this task is to optimize use of funds on Rhode Island roadways by developing a life span management plan.

The overall process of the study can be seen below in Figure 1.1.

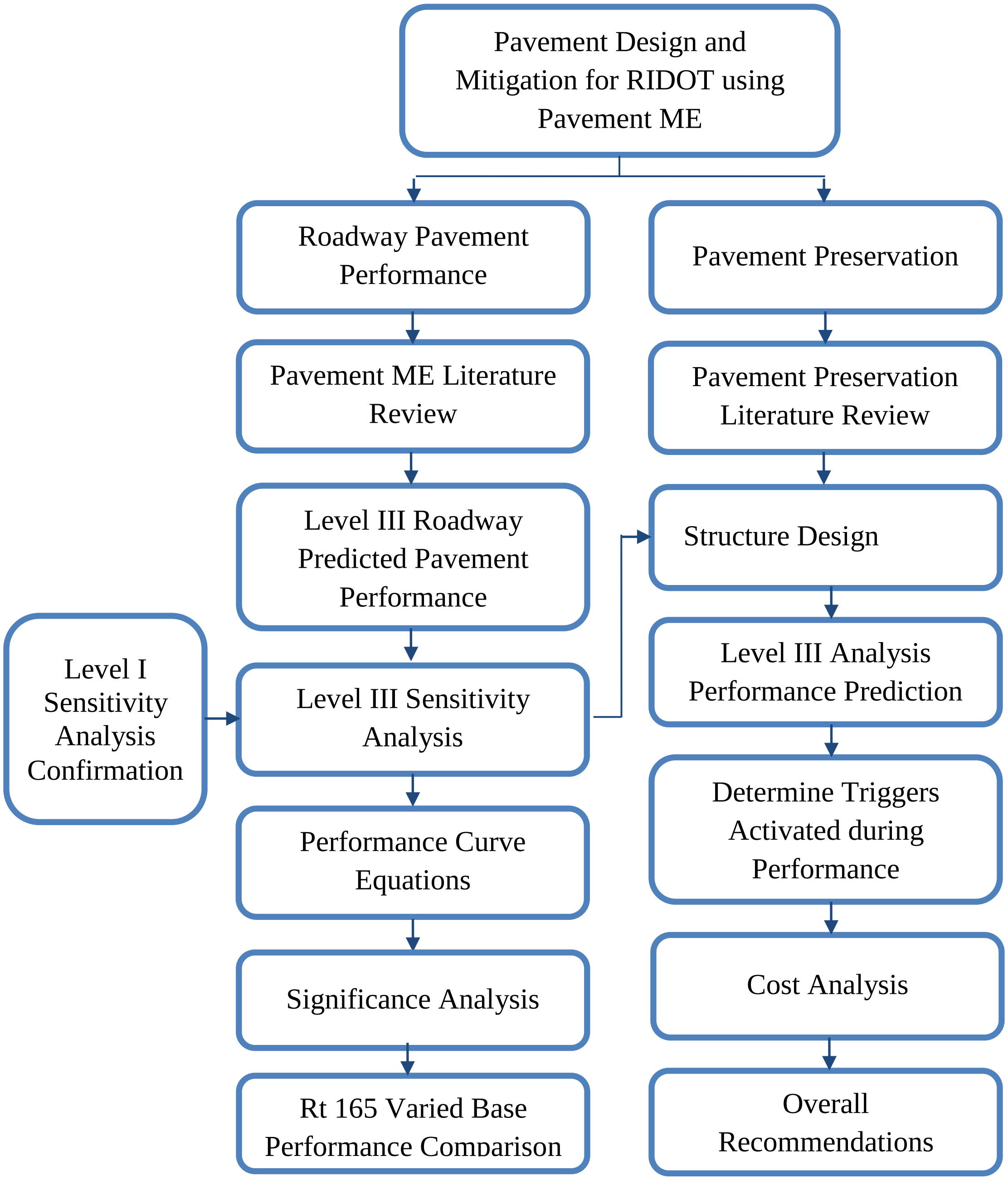

Figure 1.1 RIDOT Study Research Approach Flow Chart

Chapter 2

Pavement ME and Pavement Preservation Literature Review

2.1 Pavement ME

The need for MEPDG was due to deficiencies in the current AASHTO design guide and road test. Some of the deficiencies include traffic volumes being lower than realistic volumes, limited material properties, climatic effects, and reliability causing over design [1]. The guide was developed to account for any variable that can affect the pavement performance including such properties as climatic stations across the United States, material properties, traffic/truck properties, and damage accumulation. The guide used an iterative process to determine the overall distresses for flexible and rigid pavement based on those properties [1]. Models for each distress were developed to fit existing sections across the United States. The models allowed the use of national defaults for inputs to localized data collected from a specific location/project [1]. As the models were being generated, a program, MEPDG, now Pavement ME, was developed to calculated pavement performance based on the models. The guide was developed and published for the pavement industry to evaluate the accuracy and use of the models. Through evaluations of the guide and new information being collected, MEPDG updates the models for the guide and Pavement ME to improve on the overall effectiveness.

Since the inception of MEPDG, several research studies have been conducted to assess the different aspects of the guide and Pavement ME software. One main aspect researchers have focused on for their research is assessing the quality of the guide/software and improving it. The other main aspect looks at new test protocols or improving data collection methods to ensure better data for use with MEPDG process and the Pavement ME software. Table 2.1 shows previous studies that have evaluated the implementation of Pavement ME and the effects of varying key factors on pavement performance. Several studies [2, 3, 4, 5] have shown that different models, such as climatic models and distress prediction models, have their limitations but have been improving through revisions of the guide. Other studies [6, 7] have shown that developing new testing methods and updating data collection resources have improved the quality of collected data; thus, allowing the Pavement ME Design models to predict performance more accurately.

Table 2.1: Previous Studies' Key Findings on Pavement ME Design Implementation

Study	**Key Findings**
Jadoun et al., 2012 [8]	Local calibration factors/data improve the overall accuracy of the predict performance
Mohammad et al., 2011 [9]	Variability in the Dynamic Complex Modulus is about proportional to changes in performance if below 10%
Delgadillo et al., 2011 [10]	MEPDG can design more appropriate structures for the design life preferred especially if locally calibrated
Haider et al., 2011 [11]	Local traffic inputs are important for a more accurate pavement performance
Li et al., 2011 [12]	Using MEPDG for a catalog of structures for Low Volume Roads is a practical method of design
Romanoschi et al., 2011 [13]	Site Specific data can produce differences in performance when compared to using state or default values provided by MEPDG.

The above studies looked at two major aspects: factors known to affect performance or local calibration to make the performance prediction more accurate. In this study, elements of both of these aspects were investigated. In particular, the study at hand identifies parameters for Rhode Island conditions that have maximum impact on pavement performance. This will provide an invaluable tool to the state agency to prioritize their resources while developing the most cost-effective design.

2.2 Pavement Preservation

2.2.1 Pavement Preservation Mitigation Alternatives. There are many types of mitigation alternatives that can be utilized by different transportation divisions around the world. In this study; however, focus was given to the mitigation alternatives that are currently used by RIDOT in their pavement management system. Those alternatives are listed below.

- Crack Seal
- Level and Overlay
- Mill and Overlay (with and without a friction course)
- Paver Placed Elastomeric Surface Treatment (PPEST)
- Rubberized Chip Seal
- Reclamation
- Reconstruction
- Stress Absorbing Membrane Interlayer (SAMI)

Each one of the alternatives has their own strengths and weaknesses. In the following sections, include a discussion of each of these alternatives along with their common applications and possible pavement condition triggers. The pavement condition trigger is a way to quantify the current state of the pavement that would help identify the pavement preservation strategy needed to improve the current condition of the pavement. In addition, cost analysis will also be conducted, which will be based on averages of available data; as prices of alternatives can vary greatly from year to year.

2.2.1.1 Crack Seal. Crack sealing is the process of closing cracks that are typically opening and closing with temperature variation. This adds no structural support to the pavement [14] as it only helps in reducing damage from water and debris getting into the cracks. On a HMA pavement this type of mitigation is implemented typically two to four years after construction. Crack sealing is an alternative meant to treat longitudinal, transverse, and minor block cracking [14]. This treatment can be repeatedly used but the results can be more variable with more repetitions [15]. Depending on the type of crack sealant and crack sealing process this alternative can add up to three years or two to six years [14, 15]. The typical cost is about $4,550 per lane mile [16, 17].

2.2.1.2 Level and Overlay. The level and overlay mitigation alternative is a process in which a fresh layer of asphalt is added to bring the roadway to its original profile, leveled, and then an overlay is added on top of the newly leveled roadway. This alternative adds about 1.5-2" of HMA on top of the existing roadway surface. This mitigation alternative improves the overall quality of the roadway while adding some structural support to the pavement though it is not an actual structural layer [15]. Level and overlay helps with most types of distresses except fatigue cracking. It helps level out any slight rutting and seals cracks that might exist in the pavement. This mitigation alternative also improves the overall ride quality [14]. Depending on the roadway, level and overlay can add between 5 to 10 years to a roadway's life [15]. The typical cost is $84,150 per lane mile [16, 17].

2.2.1.3 Mill and Overlay. Similar to the level and overlay mitigation alternative, the mill and overlay increases the thickness of the pavement and also helps in extending its life. However, this mitigation alternative focuses more on rutting as it helps level off the pavement where rutting might be an issue. Mill and overlay is commonly used when the pavement layer bonds are compromised. This alternative is popular for applications where changing the pavement elevation is not possible due to curbing, the gutter or other physical constraints [15]. A friction course can be added to improve the pavement's surface friction where needed. The typical cost without a friction course is between $101,000 and $105,000 per lane mile depending on the milling method used [16, 17]. The friction course adds on another $37,000 per lane mile [16, 17].

2.2.1.4 Paver Placed Elastomeric Surface Treatment (PPEST). This mitigation alternative is a thin HMA layer (1±1/4") with a tack coat put down right before application of the thin HMA layer. As required by RIDOT specifications, the aggregate gradation used in preparing the HMA layer, used for this alternative, is an open gradation with most of the aggregates are retained on the #4 or # 8 sieves. The asphalt binder utilized for this alternative is modified with crumb rubber to help seal the asphalt [18]. PPEST is usually done for roadways with moderate cracking of any form except reflection cracking [15]. PPEST is for roadways with light rutting and cracks that are less than a ¼" wide. If those distresses are not light then the rutted area should be leveled and the cracks should be sealed. This treatment adds about 4-6 years to an asphalt roadways life but there is not a large quantity of data confirming the exact length of addition years expected [15]. The typical cost is about $86,280 per lane mile [16, 17].

2.2.1.5 Rubberized Chip Seal. A rubberized chip seal is an alternative that starts with the application of a thin asphalt layer modified with rubber. Then aggregate chips are imbedded into the asphalt layer [14]. The chip seal can also be applied in a double application where two chip seals are layed right on top of each other. Chip seals are typically used to seal various types of moderate cracking issues in asphalt and will assist in slowing the oxidation process of the existing asphalt. Double chip seals are used when the price of filling all of the cracks is higher than the cost of a single chip seal. Chip Seals can add 3-7 years to a pavement's life with double chip seals are more likely to be on the higher end of that range [15]. The typical cost is about $40,750 per lane mile [16, 17].

2.2.1.6 Reclamation and Reconstruction. Reclamation and reconstruction mitigation alternatives are similar and hence provide overall similar outcomes. Both remove a layer or layers that are failing or failed and replace it with a brand new layer. This is usually required when the roadway is in severe distress due to various combinations of cracking and rutting. Reclamation and reconstruction are usually the last preferred alternatives as they are time consuming and expensive. A project in Rhode Island in 2009 had the cost of reclamation around $350,200 per lane mile [19]. Since this is not a preferred alternative, there is limited cost data on reclamation.

2.2.1.7 SAMI. Not much information has been found on this alternative except that it is a chip seal followed by an overlay consisting of gap graded mix [20]. There is limited information as this alternative is a combination of an overlay and a rubberized chip seal. No RIDOT specification has been found or provided by RIDOT for this alternative. From this information and other alternative pricing the alternative would cost about $124,900 per lane mile [16, 17].

2.2.2 Pavement Preservation Management. The pavement preservation process consists of multiple steps that are developed for each agency as defined by US Department of Transportation (USDOT) and the Federal Highway Administration (FHWA) [21]. This involves five steps:

The first step is to establish a trigger or threshold system for serviceability of a road. This is done to determine when a roadway cannot be rehabilitated and needs to be reconstructed or reclaimed. This would be a point that a distress or a combination of distresses is at a severity level that is too expensive to repair when compared to the cost of reconstruction/reclamation. The second step is to establish a base-line current conditions and performance predictions. The current condition is to know the severity level of each distress monitored and what repair alternatives will be needed in the near future. The performance predictions are conducted to more efficiently maintain the roadways within the network as it can help identify a preliminary timeline of when repairs should be applied [21].

2.2.2.1 Roadway Current Condition Rating System. Current conditions will help set up a rating system for the roadway based on pavement distresses such as rutting or different types of cracking. This system is developed based on the agencies needs as certain distress will have different weightage depending on the location of the agency. For example, in states with heavy traffic, the larger volumes of traffic could cause rutting issues. If that is the case, that particular state DOT(s) could put a higher emphasis on rutting in their weightage rating system than other distresses. For example, states, such as Florida which typically has higher temperature averages compared to New England states, thermal cracking is not as much of an issue so the respective weightage for thermal cracking will be less than other distresses. Thus, the respective weights would be agency dependent, due to varying factors, such as, climate and traffic. Also, the way each distress is scored and rated will be dependent on the type of location the agency controls such as urban versus rural. For urban roads, the triggers will be more stringent compared to rural roads as they typically have heavier loading and generally have higher importance within the roadway network. In regards to the type of roadway, interstates will have a stricter trigger system due to its higher use than the local county road that is used mostly by those in the few surrounding municipalities.

2.2.2.2 Performance Prediction Models. Pavement performance predictions are typically based on past experience of the agency which can also affect which prediction model they use for performance predictions [21]. The prediction models could be developed in a multitude of ways. Some basic ways are through purely mechanistic models, mechanistic-empirical models, regression models and subjective models [22]. Purely mechanistic models are completely based on stresses, strains or deflections within a structure. Mechanistic-empirical is the method Pavement ME uses where it combines the mechanical responses with the historic empirical data. Regression models are based on relating observed/measured deterioration to parameters such as material properties, loading properties, and/or climatic factors. Subjective models are a group of models that use probability to determine the likelihood of different pavement conditions to occur. Regression models are traditional ways to model predicted performance as it is based on the respective transportation agencies historical database for pavement performance. Pavement ME software based on the mechanistic-empirical design philosophy has become increasingly popular, as mentioned earlier, due to the large amount of input parameters that impact pavement performance. Performance models can be for the overall structure's performance or individual distresses. It is common for performance prediction models to be developed for individual distresses or a combination of multiple distresses representing the overall structure's performance. The performance model for the overall structure is called the remaining service life (RSL) of the roadway.

2.2.2.3 Remaining Service Life (RSL). RSL can be dependent on the overall pavement structure or an individual distress. The purpose of identifying the RSL for a pavement structure is to determine the number of years before the pavement can be considered unserviceable or unsafe to drive on. A brand new pavement structure's RSL will be equal to the design life of the roadway [23, 24]. Determining how the RSL changes over time can be done on a road-by-road basis, which is a traditional method, or through the entire network. Determining how the RSL changes over time would be accomplished by updating the performance model with new data for the roadway. Updating the performance model can also help determine the life extension of different repair alternatives as the extension can vary based on agencies resources [25]. In this study, the road-by-road basis will be the process used as Pavement ME limits the model to one roadway. RIDOT currently has a system to determine RSL through their current Pavement Management System (PMS). The RIDOT PMS will be discussed in a later section explaining their respective trigger and scoring system.

***2.2.2.4 Initial Cost Effective Alternative Development*.** The third step in pavement preservation process would be how to prioritize the pavement preservation alternatives for an overall strategy. This research will focus on a multi-year prioritization as it compares alternatives better and takes into account the performance curves created from the RIDOT scoring system [21]. The first step to prioritize alternatives would be to determine when the exact triggers for repair alternatives are reached. This could be as simple as triggers for severity of the repairs such as minor rehabilitation versus specific alternatives, such as rubberized chip seal. The next step would be to determine the cost of each alternative along with the performance/years added to the overall pavement from that alternative. Knowing when the different levels of rehabilitation are triggered along with their cost helps determine if it is more cost-efficient to do multiple minor rehabilitations or a singular major rehabilitation. This could be expanded even further by comparing specific alternatives and their costs using the same basic concept. Once this information is collected a cost analysis can be completed. This first analysis would be looking at each alternative individually and developing a strategy for each. This examines the cost effectiveness of each alternative as each has varying costs [26].

2.2.2.5 Strategy Development. The fourth step is more complicated as it involves a multi-alternative plan to optimize funds and the benefits for the respective alternatives. There are a few methods to determine the strategy to use in pavement preservation. The three main ones presented by Federal Highway Administration (FHWA) are decision trees, decision matrices, and programmed rules [21]. Decision trees and decision matrices are similar except the trees are similar to a flow chart while the matrices are presented in a table. Decision trees have the user follow different branches that help specify the condition of roadway through the pavement index and types of distresses evident in the pavement. Decision matrices summarize the different alternatives and their effectiveness in certain situations such as the roadway condition, the added structural benefit, and effectiveness for different distresses. These two methods could be as simple or as complicated as necessary depending on the data available [21]. The programmed rules method is requirements for an alternative to be used similar to the triggers determined for the current condition rating system step mentioned earlier. The rules could even be based upon the triggers. For this study, a decision matrix will be used as it is the most logical choice for the data available for this project. Like the trigger system developed earlier in the process, the alternatives listed in the matrix can be as simple as the severity of the rehabilitation or as specific as using a PPEST alternative.

2.2.2.6 Overall Cost Effective Alternatives Development. The fifth step would be prioritizing and determining which alternatives are best. This is accomplished by looking at effectiveness/benefits and cost. Effectiveness is the added area under the performance curve while benefit could be the same as effectiveness or it could be based on other factors related to the alternatives. Cost looks at the overall life cycle cost analysis for the different alternatives.

There are two common techniques (marginal cost effectiveness approach and incremental benefit/cost analysis approach) that can be used to prioritize specific alternatives [21]. The marginal cost effectiveness method involves a step by step process that uses effectiveness-cost ratios to efficiently spend a budget for rehabilitation. Incremental benefit/cost (IBC) analysis plots the alternative's benefits versus the costs on a graph. The benefits are based on different parameters determined to be important and are available for the analysis. These might include: the cost efficiency of the alternative, effectiveness to solve the problem being mitigated, or any other parameter affecting the alternatives. Ultimately the units for benefit can vary from dollars to years added to unit-less if it is a rating system. The IBC method will use the alternatives that appear highest on the plot as they added the most benefit per unit of cost. The cost, according to FHWA, as reported by Shahin et al. (1985) [27] , should be measured in equivalent uniform annual costs (EUAC) as it brings the different alternatives that are triggered at different times to an equal basis. According to the Transportation Economics Committee of the Transportation Research Board, the cost is the discounted cost of the alternative the respective year being analyzed or the net present value as it would produce the same results [28].

The alternatives shown below in Figure 2.1 are examples of different pavement preservation alternatives such as chip seal and PPEST. The IBC line connects the highest alternatives to create the efficiency frontier and no points shall be above the efficiency frontier. According to the FHWA, the alternatives should be connected in a way that “no line segment has a bigger slope than the previous line segment” [21]. In the example below in Figure 2.1, the dotted line to the right of the square alternative has a bigger slope then the previous dotted line. If that last point was not included then it would be above the efficiency frontier so the diamond alternative must be excluded from the IBC for it to form correctly. In other words, it might be better to just wait longer and not conduct the intermediate alternative because the overall benefit is the same, regardless of whether that alternative is implemented. The last alternative on the right is not included because the line segment between points cannot be negative as it would be adding cost while losing benefit. Through this method the most efficient alternatives are determined and can be chosen depending on salary restrictions or controlling distresses.

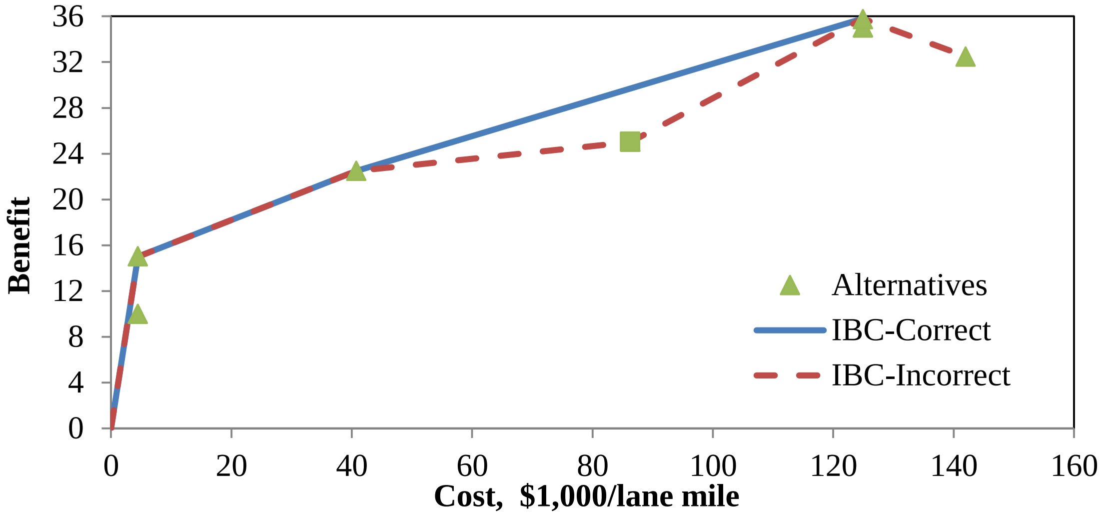

Figure 2.1 IBC Example

The incremental benefit/cost analysis would be the most effective for this study as a budget for the roadway repairs is not included in this study. Any further optimization of the pavement preservation system would need to be at a network level and look at all roadways needing preservation over the respective analysis period being used by the transportation authority. This research is specifically looking at the initial individual roadway prioritization and optimization.

Chapter 3

RIDOT Pavement Performance using Pavement ME Design Level III

3.1 Roadway Pavement Performance

In order to determine the effectiveness of Pavement ME for RIDOT in terms of quality information, it is necessary to first utilize the Pavement ME to design (i.e. determine pavement layers' thicknesses) existing roads that are about to be repaired. The first step in determining the effectiveness of Pavement ME for RIDOT is to apply it to existing roadways about to be repaired and determine the necessary thicknesses for a stable design. Two types of analyses were done to accomplish this initial step for RIDOT. The first analysis was done on three roadways using the same structure but different traffic loads and speeds. The second analysis evaluated two design structures for a highway to determine which would perform best for a given set of traffic conditions. The roadway locations specified by RIDOT can be seen below in Figure 3.1.

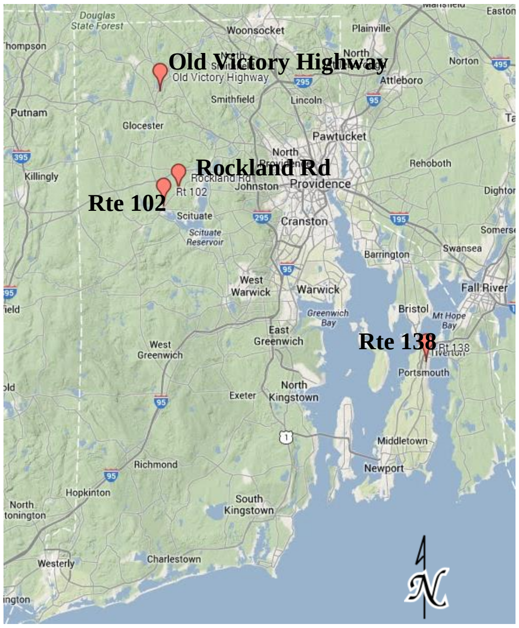

Figure 3.1 Roadway Location

3.1.1 Pavement ME Design Analysis for Rte 138, Old Victory Rd, and Rockland Rd. A mechanistic empirical analysis was conducted on three roads; Rte. 138, Old Victory Highway and Rockland Road to determine an appropriate thickness of the base layer, existing HMA layer, for the three roadways listed above to receive a 2-inch overlay. From the information provided by RIDOT, the layers would be the base structure shown below in Table 3.1. Using a back-calculation program, BAKFAA, the modulus of the 3 layers below the new HMA layer were back-calculated from falling weight deflectometer (FWD) deflection data and can be seen in Table 3.1. The results, presented in Table 3.1, are representative of typical values found for pavement structures in Rhode Island. These moduli values were used in the predict performance for an overlay structure with varying HMA existing base thicknesses, up to the existing thickness, with a 7” or 10” subbase. Rte. 138 varied speed, specified by RIDOT, instead of varying the existing base layer thickness. In all three sections the performance of the pavements with a thickness of subbase of 7” and 10” were similar. The climatic data used was based on weather in Providence, Rhode Island. It is worth noting that a 50% reliability levels was utilized to complete all of the Pavement ME Design simulations discussed above. This was selected because it similar to the reliability level typically used in roadway design.

Table 3.1: Baseline Pavement Structure

Layer Material	Thickness (in.)	PG Information/ Modulus (psi)	
HMA (PG 64-22)	Varying	HMA Performance Grade:	76-28
		CPR 3/4” sieve:	0
		CPR 3/8” sieve:	17
		CPR #4 sieve:	36
		% Passing #200 sieve:	4.5
Pulverized in Place-AC	7” or 10”	59,722	
A-1-a Gravel	12	23,160	
A-1-a Subgrade	Semi-infinite	14,082	

The traffic inputs for all roads are shown below in Table 3.2. The number of lanes in the design direction is the number of lanes in each direction of the roadway. Percent of trucks in the design direction is usually 50%, assuming that over the design life, 50% of the traffic is in each direction. To be conservative in the design, 60% of the traffic was used in the design direction. Percent of trucks in design lane is the number of trucks that are in the design lane. Usually that number is relatively high for two lanes because the majority of trucks are in the slow lane.

Table 3.2: Traffic Inputs for All three Roads

Parameter	Rte 138	Old Victory Highway	Rockland Rd
Initial two-way AADTT	3015	350	192
Number of lanes in design direction	1		
Percent of trucks in design direction (%)	60		
Percent of trucks in design lane (%)	100		
Operational speed (mph)	45 and 60 Comparison	60	60

3.1.1.1 Route 138. The results of are below in Table.3.3 and Table 3.4. Each setup of results is broken down by the operational speed used then the distress predicted and its corresponding reliability. The structure passed reliability for both speeds and both subbase thicknesses. The rutting was within 10% of the reliability target for both subbase thicknesses either for AC rutting and/or total rutting.

3.1.1.2 Old Victory Highway. The results are shown below in Table 3.5 and Table 3.6. Each setup of results it is broken down by the base layer thickness used then the distress predicted and its corresponding reliability. For both subbase thicknesses, it was found that the existing HMA layer could be completely removed and the structure would still pass the reliability target.

3.1.1.3 Rockland Road. The results of the tests are below in Table 3.7 and Table 3.8a. Each setup of results it is broken down by the base layer thickness used then the distress predicted and its corresponding reliability. For both subbase thicknesses, it was found that the existing HMA layer could be completely removed and fall within the reliability target. Due to the low AADTT, the reliability for all distress for this roadway were about 98% and greater.

Table.3.3: Varying Operational Speed w/ a 7" Subbase and 2" Overlay (Rte. 138)

Performance Criteria	**Distress Target**	**45 mph**		**60 mph**	
		Distress Predicted	**Reliability Predicted**	**Distress Predicted**	**Reliability Predicted**
Terminal IRI (in/mi)	172	127.8	89.99	127.2	90.37
AC Surface Down Cracking (Long Cracking) (ft./mile)	2,000	688	73.23	681	73.36
AC Bottom Up Cracking (Alligator Cracking) (%)	25	9.8	85.88	9.6	86.18
Permanent Deformation (AC Only) (in)	0.25	0.24	57.12	0.23	62.42
Permanent Deformation (Total Pavement) (in)	0.75	0.74	52.35	0.73	56.61

Table 3.4: Varying Operational Speed w/ a 10" Subbase and 2" Overlay (Rte. 138)

Performance Criteria	**Distress Target**	**45 mph**		**60 mph**	
		Distress Predicted	**Reliability Predicted**	**Distress Predicted**	**Reliability Predicted**
Terminal IRI (in/mi)	172	125	91.68	124.3	92.04
AC Surface Down Cracking (Long. Cracking) (ft/mile)	2,000	303	81.2	294	81.44
AC Bottom Up Cracking (Alligator Cracking) (%)	25	7.4	89.34	7.3	89.54
Permanent Deformation (AC Only) (in)	0.25	0.24	56.63	0.23	62.59
Permanent Deformation (Total Pavement) (in)	0.75	0.71	63.2	0.7	67.73

Table 3.5: Varying Thicknesses of HMA base layer w/ a 7" Subbase (Old Victory)

Performance Criteria	**Distress Target**	**No HMA Layer**		**1 in HMA Layer**		**2 in HMA Layer**	
		Distress Pred.	**Reliab. Pred.**	**Distress Pred.**	**Reliab. Pred.**	**Distress Pred.**	**Reliab. Pred.**
Terminal IRI (in/mi)	172	121.1	93.77	116	95.72	111.3	97.42
AC Surface Down Cracking (LongCracking) (ft./mile)	2,000	12.2	99.01	698	73.04	566	75.54
AC Bottom Up Cracking (Alligator Cracking) (%)	25	1.3	95.34	3.8	93.33	3.4	93.86
Permanent Deformation (AC Only) (in)	0.25	0.11	99.97	0.11	99.97	0.08	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.68	74.26	0.56	97.92	0.47	99.98

Table 3.6: Varying Thicknesses of HMA Base Layer w/ a 10" Subbase (Old Victory)

Performance Criteria	**Distress Target**	**No HMA Layer**		**1 in HMA Layer**		**2 in HMA Layer**	
		Distress Pred.	**Reliab. Pred.**	**Distress Pred.**	**Reliab. Pred.**	**Distress Pred.**	**Reliab. Pred.**
Terminal IRI (in/mi)	172	120.7	93.94	116.1	95.89	111	97.48
AC Surface Down Cracking (Long Cracking) (ft./mile)	2,000	10.6	99.27	460	77.65	388	79.17
AC Bottom Up Cracking (Alligator Cracking) (%)	25	1.1	95.52	3.2	93.84	3	94.31
Permanent Deformation (AC Only) (in)	0.25	0.11	99.98	0.11	99.98	0.08	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.69	73.78	0.56	97.95	0.47	99.98

Table 3.7: Varying Thicknesses of HMA Base Layer w/ a 7" Subbase (Rockland Rd)

Performance Criteria	**Distress Target**	**No HMA Layer**		**1 in HMA Layer**		**2 in HMA Layer**	
		Distress Pred.	**Reliab Pred.**	**Distress Pred.**	**Reliab Pred.**	**Distress Pred.**	**Reliab, Pred.**
Terminal IRI (in/mi)	172	109.8	97.97	107.5	98.46	105.2	98.85
AC Surface Down Cracking (LongCracking) (ft./mile)	2,000	0.4	99.999	11.1	99.19	11.1	99.19
AC Bottom Up Cracking (Alligator Cracking) (%)	25	0	99.999	0.2	99.999	.4	99.999
Permanent Deformation (AC Only) (in)	0.25	0.06	99.999	0.07	99.999	0.06	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.48	99.83	0.42	99.999	0.37	99.999

Table 3.8: Varying Thicknesses of HMA base layer w/ a 10" Subbase (Rockland Rd)

Performance Criteria	**Distress Target**	**No HMA Layer**		**1 in HMA Layer**		**2 in HMA Layer**	
		Distress Pred.	**Reliab. Pred.**	**Distress Pred.**	**Reliab Pred.**	**Distress Pred.**	**Reliab Pred.**
Terminal IRI (in/mi)	172	108.5	98.25	106.5	98.64	104.5	98.96
AC Surface Down Cracking (Long Cracking) (ft./mile)	2,000	0.5	99.999	4.8	99.94	4.5	99.95
AC Bottom Up Cracking (Alligator Cracking) (%)	25	0	99.999	0.2	99.999	0.3	99.999
Permanent Deformation (AC Only) (in)	0.25	0.06	99.999	0.07	99.999	0.06	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.45	99.99	0.4	99.999	0.35	99.999

3.1.2 Pavement ME Design Analysis for Rte. 102. Mechanistic empirical analysis was done for Rte. 102 using 2 design options provided by RIDOT. From the information provided by RIDOT, the layers would be the structures seen below in Table 3.9. The same modulus values from the previous study were used. These were used in Pavement ME Design to predict performance of the two structures seen below in Table 3.9. Structure 1 was to replace the existing HMA layer with a new HMA layer as it would save money in leveling the sides of the roads to match existing structures. Structure 2 was adding an HMA overlay to the existing structure. The climatic data used was based on weather in Providence, Rhode Island. The traffic inputs for Rte. 102 are below in Table 3.11. Two traffic volumes from RI were given, one for Coventry Rd and one for Foster Rd. A reliability of 95% was specified for this analysis instead of the 50% reliability from the last analysis.

Table 3.9: Pavement Structures

Layer Material	**Thickness (inches)**		**Modulus (psi)**
	Structure 1	**Structure 2**	
HMA (PG 76-28)-New Layer	3.5	2	Inputs calculated from Table 3.10
HMA (PG 64-22)-Existing	----	3.5	Inputs calculated from Table 3.10
Pulverized in Place-AC	5		59,722
A-1-a Gravel	9.5		23,160
A-1-a Subgrade	Semi-infinite		14,082

Table 3.10: Inputs for the Surface Layer

HMA Performance Grade:	76-28 (Surface Layer)	64-22 (Existing layer)
Cumulative % Retained 3/4” sieve:	0	0
Cumulative % Retained 3/8” sieve:	17	17
Cumulative % Retained #4 sieve:	36	36
% Passing #200 sieve:	4.5	4.5

Table 3.11: Traffic Inputs for Rte. 102

Roadway	**Parameter**	**Value**
Coventry	Initial two-way AADTT	93
Foster	Initial two-way AADTT	75
Both	Number of lanes in design direction	1
	Percent of trucks in design direction (%)	60
	Percent of trucks in design lane (%)	100
	Operational speed (mph)	60

In Table 3.12 and Table 3.13 are the results of the Pavement ME Design analyses for the two structures provided and the 2 traffic volumes provided. The thickness of the replacement HMA layer in Structure 1 did not meet reliability criteria. The asphalt top-down cracking failed reliability for both traffic volumes. Just to pass the default 90% reliability set by Pavement ME Design the 5.5 in. replacement HMA layer was needed to achieve 91.98% reliability. Structure 2 passed for both of the traffic volumes at the default 90% Pavement ME Design reliability target. This does not meet the 95% reliability but to reach that reliability level a 2.5 in. overlay is needed instead of the standard 2”.

Table 3.12: Results for replacing the 3.5" layer

Performance Criteria	Distress Target	Coventry AADTT		Foster AADTT	
		Distress Predicted	**Reliability Predicted**	**Distress Predicted**	**Reliability Predicted**
Terminal IRI (in/mi)	172	110.4	97.78	109.4	98.06
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	**309**	**81.05**	**225**	**83.39**
AC Bottom Up Cracking (Alligator Cracking) (%)	25	1.1	99.89	0.9	99.999
Permanent Deformation (AC Only) (in	0.25	0.07	99.999	0.06	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.48	99.96	0.46	99.99

Table 3.13: Results for adding a 2" overlay on the existing HMA layer

Performance Criteria	Distress Target	Coventry AADTT		Foster AADTT	
		Distress Predicted	**Reliability Predicted**	**Distress Predicted**	**Reliability Predicted**
Terminal IRI (in/mi)	172	105.9	98.74	105.2	98.85
AC Surface Down Cracking (Long. Cracking) (ft/mile)	2,000	59.2	91.98	42.8	93.91
AC Bottom Up Cracking (Alligator Cracking) (%)	25	0.3	99.999	0.3	99.999
Permanent Deformation (AC Only) (in)	0.25	0.06	99.999	0.06	99.999
Permanent Deformation (Total Pavement) (in)	0.75	0.39	99.999	0.37	99.999

3.2 Pavement ME Design Level III Sensitivity Analysis

The analysis in the previous section, clearly demonstrated that in many cases sections thinner than what was initially recommended were sufficient to pass the Pavement ME criteria. The analyses lead to a financial savings of about $200,000 [29].

Since that analysis was a success, the next step was to perform a sensitivity analysis to determine key factors that affected pavement performance. The sensitivity of various input parameters that affect pavement performance was analyzed. These parameters include traffic volume, thickness of layers, and stiffness of individual layers. The input parameters used in the Pavement ME-Design analysis were chosen to closely represent Rhode Island conditions. The surface layer binder performance grade (PG) range used in this study ranged from PG 52-28 to PG 82-28. The higher PG grade was varied to account for varied traffic and the lower PG was kept constant to represent Rhode Island's cold climate.

The average annual daily truck traffic (AADTT) values were obtained from Rhode Island traffic maps [30]. The AADT was varied from 500 to 4,000. Typical traffic stayed below 2,000 but 4,000 was used for most of the analysis to be able to help accentuate the effects of the changes. The other traffic parameters used was 2 lanes in the design direction, 50% of trucks in the design direction and 95% of the trucks in the design lane. The subgrade layer soil types, A-1-a and A-1-b, were chosen as they are common soils found in that area. The modulus values used were typical values for Rhode Island or were obtained from back-calculation of FWD data of existing roadways [30]. The weather conditions used were from Providence, Rhode Island for a 20 year analysis period. The design life was identified as the duration until the roadway reaches the Fair rating, 84.1-75.6 out of 100, according to RIDOT's pavement performance scale. This scale is based off of several forms of distress but not all are predicted by Pavement ME-Design so the rating compared in the results is based off of only the results accurately predicted in Pavement ME-Design. The full extent of the RIDOT performance scale will be discussed later. The results of this phase of the study with alligator cracking included can be found in Appendix A.

3.2.1 Vehicle Distribution. This initial analysis was done to determine if a default distribution could be used in the analysis. The default vehicle distribution was used as one distribution. The other distribution was from a Rhode Island weigh in motion station [30]. This distribution was a typical vehicle distribution with a high percentage of the truck traffic with class 5 & 9 trucks. The two distributions can be seen below in Table 3.14. For this part of the analysis, the structure analyzed can be seen listed below. The modulus of the asphalt layers in a level III analysis are based on the performance grades and gradation seen in Table 3.15. The modulus for the A-1-b base is 12,374 psi and the modulus for the last two layers is 20000 psi. These were based on typical values for RI or default values for Pavement ME.

- PG76-28 HMA Surface Layer- 2- 4”
- PG 64-22 Existing HMA Layer-6”
- A-1-b base - 7”
- Crushed Gravel Subbase- 5”
- A-1-b Subgrade- Semi-Infinite

Table 3.14: Vehicle Distributions

Vehicle Class	**Default Distribution**	**RIDOT Distribution**
	AADTT (%)	**AADTT (%)**
Class 4	1.8%	1.5%
Class 5	24.6%	37.8%
Class 6	7.6%	5.5%
Class 7	0.5%	0.2%
Class 8	5.0%	12.7%
Class 9	31.3%	40.5%
Class 10	9.8%	0.5%
Class 11	0.8%	1.1%
Class 12	3.3%	0.1%
Class 13	15.3%	0.1%

Table 3.15: Pavement ME Design Input for the HMA Layers

	Surface Layer	Existing Layer
Binder Performance Grade:	PG 76-28 (initially)	PG 64-22
Cumulative % Retained 3/4” sieve:	0	0
Cumulative % Retained 3/8” sieve:	13	13
Cumulative % Retained #4 sieve:	40	40
% Passing #200 sieve:	6	6

3.2.1.1 Impact of Vehicle Distribution on Predicted Performance. The surface layer thickness was varied from 2 to 4”. Table 3.16 shows the Pavement ME Design results when the thickness of the surface layer was varied. Increasing the surface layer from 2 to 4” did not significantly impact the structures performance. The AADTT was assumed to be 4,000. Going from the default distribution to the RIDOT distribution had an impact on pavement performance. The decrease in number of class 13 trucks and the increase in number of class 5 and 9 trucks brought all of the distresses down. The difference in years until the Fair rating between RIDOT and default distribution ranged between 1.83 to 2.83 additional years. The RIDOT distribution is more realistic to what is to be expected in Rhode Island and will be used for the rest of the analyses.

Table 3.16: Output from Varied Top Layer and Vehicle Distribution

Performance Criteria	**Distress Target**	**RIDOT Distribution**			**Default Distribution**		
		2”	**3”**	**4”**	**2”**	**3”**	**4”**
		Distress Predicted			**Distress Predicted**		
Terminal IRI (in/mi)	172	122.4	115.67	112.47	128.84	120.76	116.48
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	4,250	1,250	266	6,720	2,780	701
Permanent Deformation (AC Only) (in)	0.25	0.37	0.29	0.25	0.44	0.34	0.3
Permanent Deformation (Total Pavement) (in)	0.75	0.71	0.59	0.52	0.83	0.69	0.62
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		3.83	5.83	8.83	2	3.83	6

3.2.2 Pavement ME Design Level III Analysis on Predicted Performance. As mentioned earlier, RIDOT vehicle distribution should be used in the analyses for this research. The basic traffic inputs, such as the number of lanes and the percentage of trucks in the design lane, will remain the same as the initial analysis except the AADTT. The structure and modulus values used for the next set of analyses are shown below in Table.3.17. The gradation information for the HMA layers remains the same as used above along with the performance grade of the existing layer.

Table.3.17: Level III Analysis Pavement Structure

Layer Material	**Thickness (in.)**	**Modulus (psi)**
HMA	2	Same as previous analysis
HMA (Existing)	6	Same as previous analysis
A-1-b Base	7	12,374
Crushed Gravel	5	20,000
A-1-b Subgrade	Semi- Infinite	20,000

3.2.2.1 Impact of Binder Grade on Predicted Performance. The binders used for the surface layer sensitivity ranged from PG 52-28 to PG 82-28. The traffic level used for this analysis was an AADTT of 4,000. The results shown in Table 3.18 indicate that there is no significant effect on the performance with the variation in binder grade of the surface layer. The surface down cracking, longitudinal cracking, when increased from PG52 and PG82 decreased by 36%. There is a decrease in rutting of the asphalt concrete layer by 14% while the rest of the distresses decreased by less than 10%. Since the amount of time added to the years until the Fair Rating is only a year overall, the impact of the binder grades is minimal.

Table 3.18: Binder Grade Impact Results

Performance Criteria	**Distress Target**	**PG52 - 28**	**PG58 - 28**	**PG64 - 28**	**PG70 - 28**	**PG76 -28**	**PG82 - 28**
		Distress Predicted					
Terminal IRI (in/mi)	172	126.23	125.07	124.26	123.51	122.4	121.83
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	6,190	5,680	5,260	4,860	4,250	3,940
Permanent Deformation (AC Only) (in)	0.25	0.42	0.41	0.4	0.39	0.37	0.36
Permanent Deformation (Total Pavement) (in)	0.75	0.77	0.75	0.74	0.72	0.71	0.7
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		2.92	3	3.25	3.75	3.83	3.92

3.2.2.2 Impact of a Higher Layer 3 Modulus on Predicted Performance. It was shown in certain structures that have a higher modulus value, greater than 30,000 psi, in the base layer is anticipated for Rhode Island [30]. Due to this, additional Pavement ME-Design runs were done with the base layer modulus of up to 46,259 psi, which was obtained from back-calculation of a similar roadway. This is to analyze thickness of overlays on roadways with possibly a higher modulus base layer. Since it was found that the binder performance grade did not have a significant impact, a Performance Grade of PG64-28, typical for RI, was used for this analysis. The results are shown in Table 3.19 below. The modulus of base layer was most sensitive to surface down cracking, longitudinal cracking, of the asphalt concrete (AC). With each increase of the modulus by about 8,000 psi increments, the cracking reduced by about a third. This is a significant decrease while all of the other distresses stayed within 10% from the initial modulus to the final modulus. While the impact on the longitudinal cracking can be seen with the improved base modulus, the overall effectiveness in extending the life of the pavement is not large as it only adds 1.33 years between all of the modulus values.

Table 3.19: Binder Grade Impact Results w/ Higher Modulus for Layer 3

Performance Criteria	**Modulus psi**	**12,374**	**20,845**	**29,316**	**37,786**	**46,259**
	Distress Target	**Distress Predicted**				
Terminal IRI (in/mi)	172	124.26	122.09	121.08	120.27	119.73
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	5,260	3,530	2,330	1,520	1,000
Permanent Deformation(ACOnly)(in)	0.25	0.4	0.4	0.41	0.41	0.41
Permanent Deformation (Total Pavement) (in)	0.75	0.74	0.71	0.7	0.7	0.69
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		3.25	3.92	4	4.08	4.58

3.2.2.3 Impact of Surface Layer Thickness on Predicted Performance. The new HMA layer was increased to determine if the performance would be significantly impacted. The base layer modulus was kept at 46,259 psi. The results can be seen in

Table 3.20. The increase of the New HMA layer had significant impact on every distress except terminal IRI. With each additional inch of new HMA the longitudinal cracking decreased by 75 to 85%. Each additional inch of new HMA decreased the AC rutting by 10-20% with a total decrease of AC rutting of 40% when the thickness is increased from 2” to 6”. The increase of HMA thickness improved the total rutting by 35% when going from 2” to 6”. Most of the improvement in rutting was due to the AC rutting reduction and only about 0.07” of the rutting improvement were from the granular soils. The benefit of adding a thicker top layer can be seen as going from 2” to 6” increases the years it took reach a Fair rating by 7.2 years. The biggest increase in the years to reach the Fair rating was 2.5 years when thickness was increased from 2 to 3”.

Table 3.20: Pavement ME Design Surface Layer Thickness Impact Results

Performance Criteria	**Thickness, in.**	**2**	**3**	**4**	**5**	**6**
	Distress Target	**Distress Predicted**				
Terminal IRI (in/mi)	172	119.73	115.55	113.24	111.33	109.31
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	1,000	233	53.8	8.96	1.34
Permanent Deformation (AC Only) (in)	0.25	0.41	0.34	0.31	0.28	0.24
Permanent Deformation (Total Pavement) (in)	0.75	0.69	0.6	0.54	0.5	0.45
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		4.58	7.08	8.58	9.92	11.75

3.2.2.4 Impact of Existing HMA Layer Thickness. The purpose of this next sensitivity analysis was to see the effect on the performance if the existing HMA layer was milled down further from the initial 6” thickness. The base layer modulus is still kept at 46,259 psi. The results are shown in Table 3.21. As expected, the smaller the existing HMA layer is the higher the distresses increased. Terminal IRI and AC rutting distresses increased by the same increment that the existing HMA layer increased. Longitudinal cracking increased with each decrease in thickness but each increase was smaller than the previous increase. The total rutting distress increase grew with each additional inch of the existing HMA layer milled. The decrease in years until the Fair rating between each inch milled down is less than or equal to a year per inch milled.

Table 3.21: Pavement ME Design Existing HMA Layer Thickness Impact Results

Performance Criteria	**Thickness, in.**	**6**	**5**	**4**	**3**	**2**
	Distress Target	**Distress Predicted**				
Terminal IRI (in/mi)	172	119.73	122.98	126.83	130.78	134.71
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	1,000	2,750	5,200	7,160	8,240
Permanent Deformation (AC Only) (in)	0.25	0.41	0.44	0.46	0.49	0.51
Permanent Deformation (Total Pavement) (in)	0.75	0.69	0.74	0.8	0.87	0.95
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		4.58	3.83	2.83	1.92	1.5

3.2.2.5 Impact of Traffic. This final sensitivity analysis kept the initial thicknesses of the base layer to 6” and the modulus was kept at 46,259 psi. Table.3.22 shows the results of this analysis. The analysis looked at lower traffic levels as it is more realistic to Rhode Island traffic patterns. The decrease in traffic wasn’t proportional to the decrease in longitudinal cracking. The decrease in longitudinal cracking distress was about a 10 to15% higher decrease than the decrease in traffic volume. Both AC and total rutting followed the opposite pattern. The decrease in distress between each traffic volume was less than the decrease in traffic itself. For IRI, even though the smallest traffic volume, 500, was an eighth of the largest traffic volume, 4,000, the IRI distress only decreased by about 11%. Having an accurate account of traffic clearly helped in decreasing the overall thickness of the structure needed as it can be seen the decrease in traffic on an average added 10 years.

Table.3.22: Pavement ME Design Traffic Comparison Results

Performance Criteria	**AADTT**	**500**	**1,000**	**1,500**	**2,000**	**4,000**
	Distress Target	**Distress Predicted**				
Terminal IRI (in/mi)	172	106.29	109.53	111.89	113.83	119.73
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	46.80	133	244	373	1,000
Permanent Deformation (AC Only) (in)	0.25	0.15	0.21	0.26	0.3	0.41
Permanent Deformation (Total Pavement) (in)	0.75	0.37	0.45	0.51	0.55	0.69
Years until Fair RIDOT Score Reach Based on IRI, Longitudinal Cracking and Total Rutting		14.25	11.25	9	7	4.58

3.3 Pavement ME Design Level III Statistical Analysis

After all of the level III Pavement ME Design runs, the sensitivity of the parameters on the pavement performance was determined. Table 3.23 summarizes all the Pavement ME Design runs and the extent that the parameters vary. In order to determine the effect of the parameter on the pavement performance a statistical analysis was conducted. The effect of the parameters to the pavement performance will help RIDOT identify the parameters that affect pavement performance most and can provide tools to the designer/RIDOT engineers to minimize the pavement distresses. The statistical analysis software called Statistical Package for Social Science (SPSS) by IBM was used to analyze the data.

The study used the SPSS software General Linear model with Univariate analysis for statistical data analysis. It was determined from the initial results obtained from the SPSS program that this data cannot be used for statistical analysis. It is because each individual parameter had no replicate Pavement ME Design runs to put into SPSS format. The data was reduced in order to use it as an input for the SPSS software with a 95% confidence/significance level.

Therefore, the data was “modified” in order to use it for SPSS program. In order to do that, two variables of every individual parameter were combined and were counted as two replicates. For example, the first layer thickness has two variables, one that is 2” thick and one 3inches thick. These variables are combined and generate an “effective” 2.5inch variable. This was to gain two replicates by creating an upper and lower limit for the 2.5inch thick predicted performance based on the 2” and 3” thick Pavement ME runs. Therefore, we could obtain two replicates of the 2.5inches thick first layer. The only consequence is that we cannot determine the difference in performance due to a 2” thickness versus 3” thickness. However, this is the only way to analyze the data considering only a single run could be conducted for each set of input parameters.

Table 3.23: Pavement ME Design Level III Summary

Layer Thickness					AADTT	Binder Grade		Modulus, psi		
New HMA	**Exist. HMA**	**Base**	**Sub-base**	**Sub-Grade**		**New HMA**	**Existing HMA**	**Base**	**Sub-base**	**Sub-Grade**
2"	2"	7"	5"	Semi-inf.	500	PG52-28	PG 64-22	12,374	20,000	20,000
3"	3"				1,000	PG58-28		20,845		
4"	4"				1,500	PG64-28		29,316		
5"	5"				2,000	PG70-28		37,786		
6"	6"				4,000	PG76-28		46,259		
						PG82-28				

3.3.1 Statistical Data Analysis. The statistical analysis showed that there were several factors affecting the pavement performance however due to restricted replicate data the following parameters were evaluated in SPSS software sensitivity analysis.

- New HMA Thickness
- Existing HMA Thickness
- Traffic
- New HMA Binder Grade
- Modulus of the Base Layer

Furthermore, analyses of the following major distresses were done to check the effect of the above parameters. Only the effects of the above parameters on the distresses below were analyzed and not the interaction between distresses.

- AC Rutting
- Longitudinal Cracking
- Terminal IRI
- Total Rutting

The highlighted values are the parameters that were significant within the 95% significance level for that distress.

Table 3.24: Impact of Various Factors on Predicted Performance Distresses

Source	**AC Rutting**		**Longitudinal Cracking**		**Terminal IRI**		**Total Rutting**	
	F	**Sig.**	**F**	**Sig.**	**F**	**Sig.**	**F**	**Sig.**
New HMA Thickness	57.78	.000	3.14	.071	36.83	.000	46.28	.000
Existing HMA Thickness	19.68	.000	84.25	.000	62.95	.000	43.36	.000
Traffic	135.72	.000	2.47	.116	58.28	.000	78.70	.000
Binder Grade of New HMA layer	3.68	.048	4.13	.036	3.03	.077	2.00	.168
Base Layer Modulus	.24	.792	32.82	.000	7.31	.006	2.10	.155

3.3.2 Statistical Analysis Summary of Findings.

For each pavement distress the corresponding parameters and their effect on the pavement performance are shown below

AC Rutting:

- All of the parameters involving the asphalt layers were significant along with the traffic volume.
- The binder grade of new HMA layer was statistically significant at 95% confidence level.
- Modulus of base layer was only significant at about an 80% confidence level.

Longitudinal Cracking:

- Existing HMA thickness and the modulus of base layer had the highest sensitivity to longitudinal cracking

- Binder grade of new HMA layer had a significant effect at 95% confidence level.
- Within the range of thickness evaluated, the new HMA thickness did not have a significance effect on longitudinal cracking.
- At the traffic volumes evaluated, traffic was not significant at 95% confidence level.

Terminal IRI:

- The thickness of the two HMA layers, the base layer modulus and traffic was significant at 95% confidence level.
- Binder grade of the New HMA was significant at 90% confidence level.

Total Rutting:

- The thickness of the two HMA layers and traffic had a high significance
- Binder grade of new HMA layer and modulus of base layer were statistically significant at 85% significance level.

A study, done by the University of Maryland in connection with Iowa State University, found similar sensitivity results, for the parameters we tested, for an HMA structure [31].

3.4 Comparison Analysis Best Fit Performance Curves

The years until the Fair Rating variable from the sensitivity analysis were determined by the performance curve for each Pavement ME run. Performance curves were developed to see how the variables affected the overall performance curve equations. This was done by using the RIDOT scoring system to get the overall performance curve equations of the top variable sensitivity analyses, surface HMA layer thickness, existing HMA layer thickness, and traffic. The curves are comprised of the monthly predicted performance given my Pavement ME over the 20 year design life.

The time dependent performance curves for the surface HMA layer thickness, existing HMA layer thickness, and traffic are shown below in Figure 3.2, Figure 3.3, and Figure 3.4. Each figure's legend shows the variation of each parameter in the respective order that the performance curve is in the figure itself. For example, in Figure 3.2 the top performance curve is the 2" thick surface HMA curve and proceeds to the bottom performance curve is the 6" thick surface HMA curve, following of the order of the legend below in Table 3.25: is a summary of the equations for the time dependent performance curves.

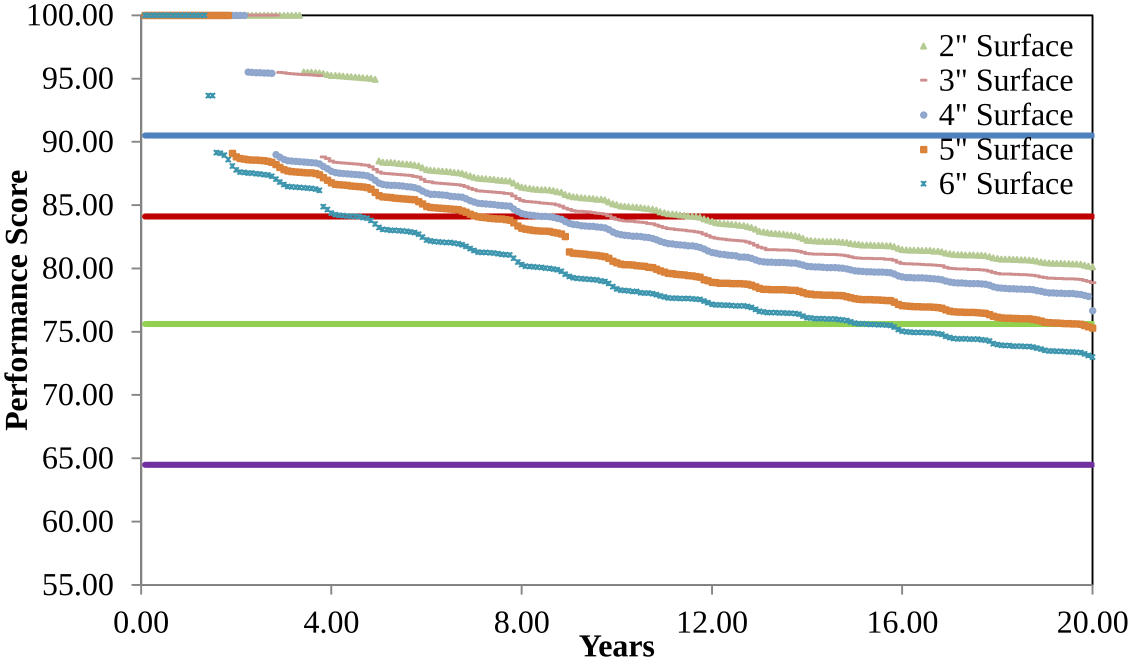

Figure 3.2 Surface HMA Layer Thickness Performance Curves

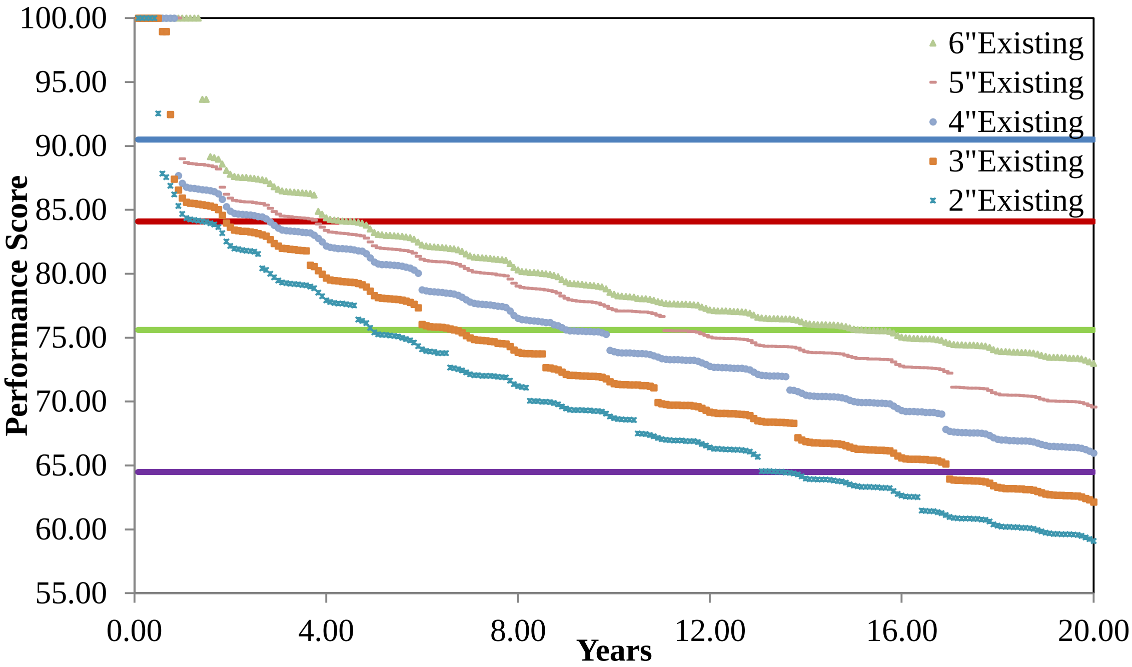

Figure 3.3 Existing HMA Layer Thickness Performance Curves

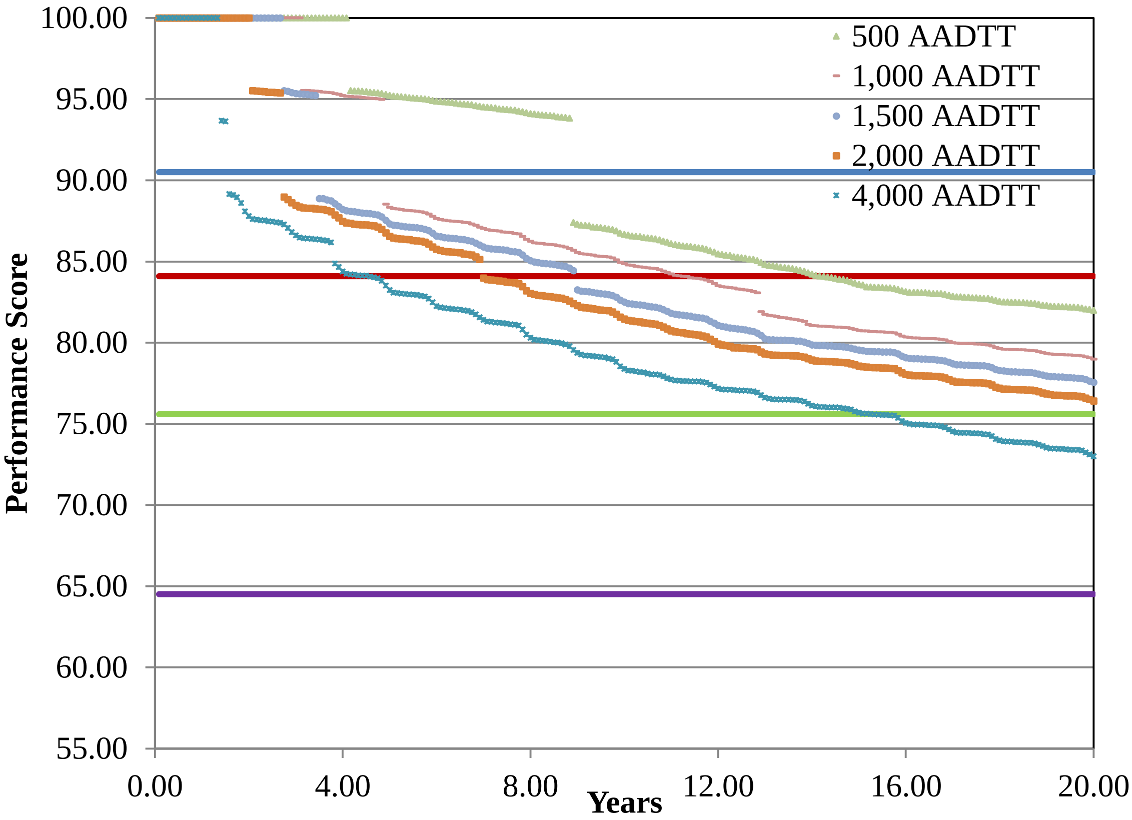

Figure 3.4 Traffic Performance Curves

Table 3.25: Performance Curve Equations

Surf. HMA	Equation AADTT of 4000	Exist. HMA	Equations AADTT of 4000	Traffic, AADTT	Equation Surf: 2” Exist: 6”
2”	$0.040x^2-1.55x+89$	2”	$0.077x^2-2.95x+89$	4000	$0.040x^2-1.55x+89$
3”	$0.022x^2-1.15x+89$	3”	$0.061x^2-2.50x+89$	2000	$0.028x^2-1.20x+89$
4”	$0.020x^2-0.99x+89$	4”	$0.043x^2-1.98x+89$	1500	$0.024x^2-1.09x+89$
5”	$0.020x^2-0.94x+89$	5”	$0.033x^2-1.61x+89$	1000	$0.017x^2-0.94x+89$
6”	$0.020x^2-0.89x+89$	6”	$0.040x^2-1.55x+89$	500	$-0.502x+87.105$
Avg. R^2	0.995	Avg. R^2	0.983	Avg. R^2	Quad. Eqs: 0.991 Linear Eq: 0.976

From the equations above, the performance curves do not start at a perfect score of 100. This is due to the way that the rutting and IRI is scored. The rutting and IRI have an initial score of 100 but once past a specific threshold they get a deduction of 14 and 10, respectively. These sudden drops in score caused large 10 point drops in the overall score as can be seen in the three figures above. By visual inspection, it can be seen from the figures that the performance curves stabilized once that sudden drop occurred and while there were still drops in score due to the cracking scoring system, these additional drops were minor drops of 1 or 2 points as shown in the figures above. The quadratic coefficient explains the initial drop of the performance curve, however as the years progress, there is a steady drop which is explained by the linear coefficient. The higher the quadratic coefficient, larger is the drop in the initial years, and higher the linear coefficient, larger is the drop in the later years.

As the surface and existing HMA thickness increases or traffic decreases upto AADTT of 1000, the rate of initial drop and the rate of drop of steady decreases, indicating that the overall performance of pavement deteriorates slowly.

At AADTT of 500, the performance curve no longer appears to be quadratic form, but it is a linear reduction in performance curve. The anomaly in the trend is of the coefficients is an artifact of curve fitting. The linear function, performance=-0.502x+87.105 is more appropriate at this AADTT level. This indicates that the quadratic function may only be valid for AADTT of 1000 and above for the structures evaluated. With a longer design life, instead of the 20 year used, a quadratic function may become valid but for this case the performance curve was linear. Also unlike the other performance curves this curve stabilizes after it drops down to a performance score of 87.105.

The surface HMA layer performance curves were quite consistent for four of the five performance curves except for the 2” thickness. The x^2 coefficient of the quadratic equation was 0.04 for 2” thickness surface HMA. As the surface layer thickness increased, the x^2 coefficient stabilized at 0.020 while the x coefficient is beginning to level off around 0.89.

The traffic performance curves had similarities between all of them except when the AADTT was 500. While they are not identical like the surface HMA layer curves they do have some similarities. As the traffic decreases both coefficients decreases though the decrease is not proportional with the decrease in traffic. With each drop in traffic level, the coefficients remain similar (first coefficient stays within 0.006 and the second coefficient stays within 0.1), except when AADTT is at 500. The performance curves were most sensitive to the thickness of existing HMA layer. At higher thicknesses, the equations were consistent, but once the layer gets too thin (i.e. less than 4 in.) the coefficients start increasing significantly as seen in Table 3.25. This shows that sensitivity of performance curves is structure dependent. As it can be seen with the surface HMA layer thickness and the traffic, one performance curve may be suitable for multiple structures. As the surface layer thickness increases (i.e. greater than 2 in.) the sensitivity of the performance curves to the thickness decreases.

3.5 Pavement ME Design Level I Analysis Background

Pavement ME Design Level I analysis was used to evaluate the accuracy and sensitivity of predicted performance using the Level III analysis. Level I analysis takes into account the dynamic complex modulus of the asphalt mix along with basic dynamic shear rheometer (DSR) binder properties of the extracted binders from the mix. Structures analyzed were based on long term pavement performance sections (LTPP) sections in the state of Rhode Island. This analysis looked at two possible repair/reconstruction techniques or setups for the two structures. Then the performance of the road was analyzed to determine how accurate Pavement ME can predict performance for RIDOT using level I analysis.

The dynamic complex modulus for the mix was collected and the master curve was developed for use in the analysis. The binder properties of the binder used were also collected. Table 3.26 shows the results of the dynamic complex modulus test and Figure 3.5 shows the master curve used for the Pavement ME Design analysis based on the dynamic complex modulus results. Table 3.27 shows the DSR results for the binder properties and Table 3.28 shows the mix gradation.

Table 3.26: Dynamic Complex Modulus (ksi) Test Results

		Frequencies, Hz						
	Temp °C	10	5	2	1	0.5	0.2	0.1
DCM HMA Mix	4	10,173	9,303	8,081	7,206	6,352	5,301	4,603
	20	4,489	3,755	2,911	2,356	1,873	1,336	1,029
	40	764	556	350	251	180	122	92

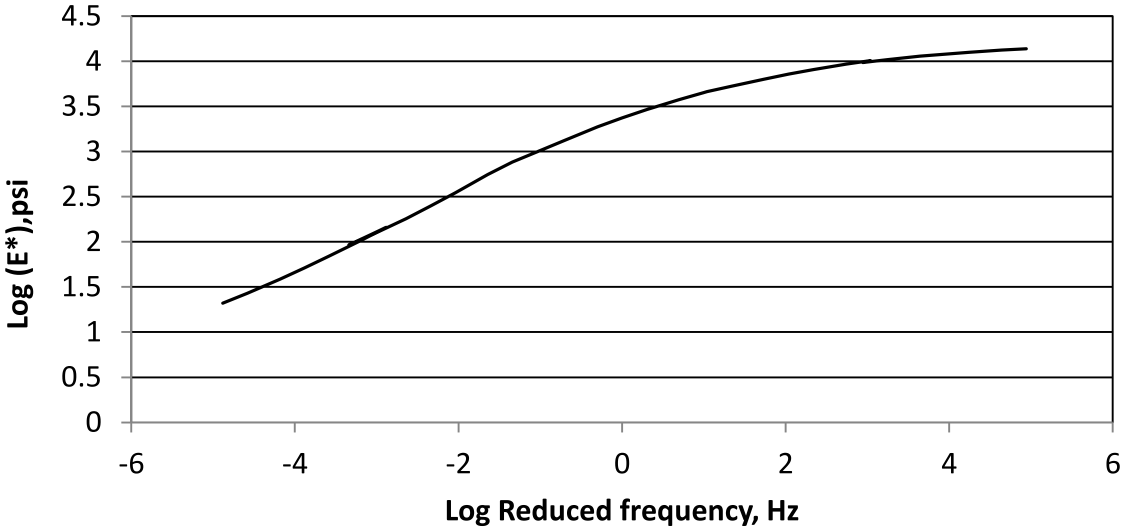

Figure 3.5 DCM HMA Master Curve

Table 3.27: DSR Binder Properties

Temp, °F	**Angular Frequency = 10 rad/sec**	
	G*. kPa	**Delta (°)**
147	3.51	83.0
158	1.61	84.9
169	0.77	86.5

Table 3.28: RIDOT Mix Gradation

Sieve Size#	Cumulative Percent Passing
1/2	100.0%
3/8	97.2%
#4	70.1%
#8	43.5%
#16	28.5%
#30	20.2%
#50	14.0%
#100	7.8%
#200	3.0%
Pan	0.0%

3.5.1 Pavement ME Design Level I Analysis. The two structures used for this analysis can be seen in Table 3.29 below. The two traffic levels used were with an AADTT of 3,015, high traffic, and 350, low traffic. The traffic levels had 60% of the trucks in the design direction, with 100% of the trucks in the design lane using the default vehicle distribution. Rutting and Fatigue Cracking were the focus of the analysis. In Table 3.30 and Table 3.31 show the final 20 year performance of the varying structures broken down by structure and setup.

Table 3.29: Varying Pavement ME Design Structures

Structure 1			
	Thicknesses		
Material	Setup 1	Setup 2	Modulus Value
DCM HMA Layer	Varying (2”, 3”, 4”)	2” Overlay	Level I analysis
HMA Existing Layer	6.5”	Varying (4”, 5”, 6”)	Inputs from
A-1-a	12”		23,160 psi
A-1-a	Semi-Infinite		14,082 psi

Structure 2			
	Thicknesses		
Material	Setup 1	Setup 2	Modulus Value
DCM HMA Layer	Varying (2”, 3”, 4”)	2” Overlay	Level I analysis
Cold RAP including Milling	8”	Varying (5”, 6”, 7”)	59,722 psi
A-1-a	8”		23,160 psi
A-1-a	Semi-Infinite		14,082 psi

Table 3.30: Predicted Fatigue Cracking, Top Down Cracking, at 20 years

	High Traffic		**Low Traffic**	
Structure 1	**Maximum Damage (%)**	**Maximum Cracking (ft./mi)**	**Maximum Damage (%)**	**Maximum Cracking (ft./mi)**
Setup 1-4” top	0.17	0.66	0.020	0.03
Setup 1-3” top	0.73	5.91	0.084	0.22
Setup 1-2” top	2.61	41.2	0.303	1.57
Setup 2-6” base	4.86	106	0.565	4.04
Setup 2-5” base	13.4	474	1.550	18.7
Setup 2-4” base	31.5	1,560	3.660	68.7
Structure 2				
Setup 1-4” top	18.0	727	2.090	29.5
Setup 1-3” top	15.9	610	1.850	24.5
Setup 1-2” top	2.15	30.8	0.250	1.17
Setup 2-7” base	2.04	28.4	0.237	1.08
Setup 2-6” base	2.11	29.9	0.245	1.14
Setup 2-5” base	2.23	32.5	0.259	1.23

Table 3.31: Predicted Rutting at 20 years

	High Traffic			Low Traffic		
Structure 1	**Top Layer (in.)**	**Base Layer (in.)**	**Total (in.)**	**Top Layer (in.)**	**Base Layer (in.)**	**Total (in.)**
Setup 1-4” top	0.164	0.007	0.444	0.058	0.002	0.266
Setup 1-3” top	0.150	0.025	0.468	0.054	0.009	0.283
Setup 1-2” top	0.100	0.075	0.492	0.036	0.027	0.300
Setup 2-6” base	0.103	0.082	0.517	0.037	0.029	0.315
Setup 2-5” base	0.100	0.095	0.558	0.036	0.034	0.343
Setup 2-4” base	0.093	0.114	0.603	0.033	0.041	0.372
Structure 2						
Setup 1-4” top	0.273	0.076	0.706	0.097	0.058	0.426
Setup 1-3” top	0.290	0.102	0.790	0.103	0.078	0.482
Setup 1-2” top	0.254	0.147	0.857	0.091	0.112	0.546
Setup 2-7” base	0.247	0.136	0.876	0.088	0.103	0.564
Setup 2-6” base	0.257	0.123	0.906	0.092	0.093	0.582
Setup 2-5” base	0.249	0.104	0.913	0.089	0.079	0.591

3.5.2 Pavement ME Design Level I Pavement Performance.

3.5.2.1 Fatigue Cracking. At the higher traffic level, AADTT of 3,015, with thicker surface layer, the fatigue cracking damage was minimized. Structure 2 had 100 times more fatigue cracking than Structure 1 for the 4” thick surface performance. Structure 2 was less affected by changes in the base layer’s thickness with a variation of about 10% while Structure 1 varied by 650%. At the lower traffic level, AADTT of 350, the same trends occurred as the higher traffic level.

3.5.2.2 Rutting. Rutting was more sensitive to variations in the thickness of the top layer. The base layer overall had minimal rutting for all setups and traffic levels compared to the top layer. For Structure 1, with both setups and traffic levels, with an increase of thickness in the top layer or base layer the top layer took more rutting damage similar to the sensitivity analysis. For Structure 2 with both setups and both traffic levels, the rutting damage initially increased and then decreased as the thickness of the top or base layer increased. This is most likely due because the top layer initially is not taking as much of the damage, then taking more damage as it is getting thicker, and finally reducing the damage because it has gotten thicker.

3.6 Rte. 165 Varied Base Material Performance Comparison

A current project by RIDOT is being completed on Rte. 165 in western Rhode Island looking at the effects of varied subbase material. This project is being completed to help understand their respective performances so the best performing material can be used for future projects. This is to help prepare the roadways to the growing truck traffic traveling through Rhode Island. To test the overall accuracy for the use of Pavement ME in RIDOT the individual sections were modeled using the information provided to be able to compare the results to the results from the field test. The subbase test sections being looked at are a control asphalt subbase, a Portland Cement Concrete (PCC) subbase, a CaCl reclaimed subbase, a bituminous stabilized subbase, and a geo-grid test section. These different subbases are being place monitored by RIDOT for different distresses. It was also a part of the project to model the test sections in Pavement ME to compare to measured results. Information about the different materials was provided [32], weigh-in-motion data for a nearby roadway, Rte. 95, was used [33], and the traffic level, 5,800 AADT, was from found on their online traffic flow map [34]. The vehicle distribution can be found below in Table 3.32*:* . The percent truck traffic was determined by looking at state percentages which gave about 4-5% trucks [33]. The WIM station closest to Rte. 165, Rte. 95, had 4.1% trucks so the same was used for Rte. 165 bringing the AADTT to 240.

The test sections were modeled as close to the field in Pavement ME. However, there were some restrictions that required minor material modeling changes. The PCC layer became a cement stabilized layer as PCC in Pavement ME is only for overlays and the CaCl reclaimed subbase was just a reclaimed layer. There were no stabilized bituminous layer options but due to the material it should perform somewhere between the asphalt control section and the reclaimed layer. The material being stabilized by the bituminous material was the same as the reclaimed layer. Also there is currently no way to accurate model a geo-grid layer in Pavement ME. The default gradations provided were used but still followed the RIDOT asphalt gradation control points. Below in Table 3.33: is the structure setup used for the experiment and the model. The last layer, labeled as the subgrade, in the construction was 6” and no other information for the layers below were provided so this layer was taken as the lowest semi-infinite layer.

Table 3.32: Rt 165 Distribution

Vehicle Class	% AADTT
4	6.62
5	68.59
6	9.37
7	1.64
8	3.41
9	10.1
10	0.23
11	0.0
12	0.0
13	0.04

Table 3.33: Rt 165 Structure

Material	Thickness	Modulus/PG Grade
HMA 12.5 mm	2”	PG 64-28
HMA 19 mm	2.5”	PG64-22
HMA 9.5 mm or Cement Stabilized or Cold Recycled Asphalt	8”	PG64-28 or 2,000,000 psi or 30,000 psi
Crushed Gravel	1”	25,000 psi
A-1-a Subgrade	Semi-Infinite	18,000 psi

The crushed gravel was the original subbase and was mixed in within the recycled asphalt subbase. The final 20 year predicted performance of the three subbase materials is below in Table 3.34.

Table 3.34: Rt 165 Subbase Predicted 20 year performance

Performance Criteria	Material	Asphalt (Control)	Cement Stabilized	Cold Recycled Asphalt
	Distress Target	Distress Predicted		
Terminal IRI (in/mi)	172	133.33	135.2	143.05
AC Surface Down Cracking (Long. Cracking) (ft./mile)	2,000	259.99	261.32	2,072.13
Permanent Deformation (AC Only) (in)	0.25	0.07	0.14	0.11
Permanent Deformation (Total Pavement) (in)	0.75	0.26	0.30	0.47

As it can be seen from the table above, the control asphalt layer performed the best out of the three pavements. All three passed with 90% confidence except the longitudinal cracking in the recycled asphalt subbase but the confidence was at 89%. The asphalt had the lowest distress compared to the other two subbases. The difference between the asphalt and the cement stabilized layer are minimal except in the amount of asphalt deformation. The cement layer helped limit the permanent deformation but caused half of the deformation to occur in the top two asphalt layers. This could help for minimizing the cost of repair as it is focusing the distresses to the top two layers. The cold recycled asphalt had more cracking than the target crack and the most deformation making this the least preferred subbase. Even though it is just reclaimed material from the old structure it has over 2,000 ft/mile of cracking. Overall the asphalt performed the best but the cement stabilized layer focused the distress in the easier accessed top layers

Chapter 4

Pavement Preservation Management and Cost Analysis

4.1 RIDOT Pavement Management System

To complete a cost analysis for mitigation alternatives the management system used needs to be explained. Rhode Island Department of Transportation has a comprehensive monitoring and scoring system. Below in Table 4.1, the previous scoring system and the current scoring system used by RIDOT are broken up by their respective weight [35]. The previous system had a relative similar weight across the board for the different distresses that were monitored. Since then they have updated the system, the weights are similar for cracking, rutting, and roughness. This allows the weightage to be similar than the previous scoring which had 60% of the score just for cracking. The 2013 system does not account for edge cracking, patch failure, or bleeding. All three of these are still monitored but do not contribute to the Pavement Structural Health Index (PSHI).

Table 4.1: Pavement Scoring System Breakdown

Monitored Distresses		2012 Scoring	2013 Scoring
International Roughness Index		10	30
Total Rutting		10	30
Longitudinal	Cracking	10	7
Transverse		10	7
Alligator		20	16
Block		15	10
Edge		5	-
Patch Failure		15	-
Bleeding		5	-

Each distress has its own scoring system based on a score out of 100 points. Then each of the distress score are averaged together by the respective weights mentioned previously. The total test section used to score a roadway is based on a 328 foot section or 100 meters [35]. In Figure 4.1, the rutting scoring system is graphically represented. The score system shown does not go to 100 because below 0.28" the score is 100 than it goes to the respective equation [35]. Once the rutting gets to 1" in severity the scoring system gets harsher. Once the rutting reaches 1.5" and higher the score is 0.

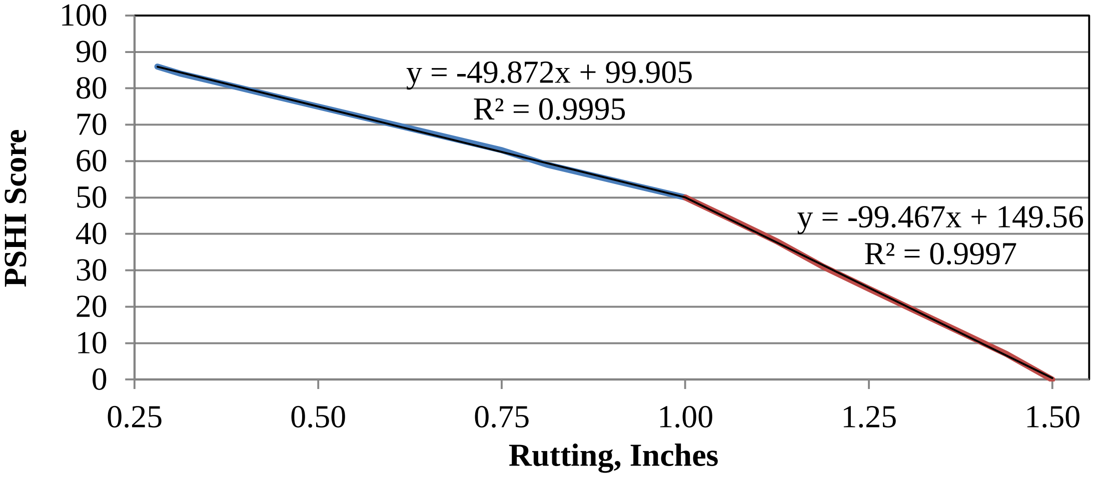

Figure 4.1 Rutting Distress Rating Score System

The IRI system is similar to rutting where there are different ranges of linear scoring relationships. Below 75inches/mile the score for IRI is 100. The transition between the different ranges occurs at 95inches/mile and 250 inches/mile as seen in Figure 4.2 [35]. When the IRI reaches 400 inches/mile and higher the pavement score is reduced to 0.

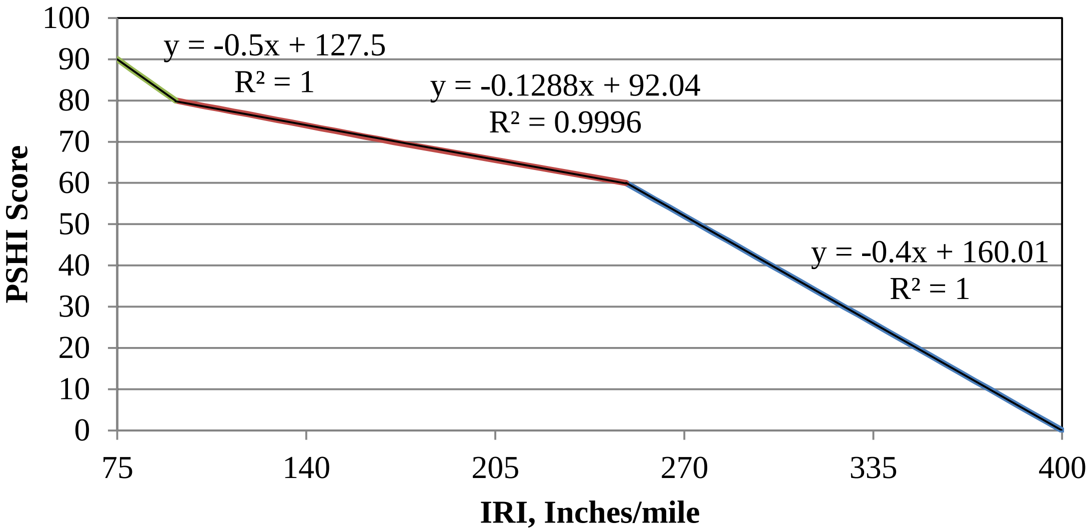

Figure 4.2 IRI Distress Rating Score System

The longitudinal cracking scale specifies a range of values instead of the linear relationship like in IRI or rutting. The ranges are based on the amount of linear feet of longitudinal cracking in the 328 foot test section as seen below in Table 4.2 [35]. The other forms of cracking monitored follow a similar scoring system and some used square footage of cracking instead of linear footage. Each cracking system was also based on severity from low to high severity depending on the crack width. The ranges become stricter and harsher as the severity raises. For purposes of this study, medium severity will be used. This is because, when looking at distress data for routes in Rhode Island, about 67% of the cracking is within the medium severity criteria and Pavement ME does not specify crack width. Pavement ME is constantly changing as their models for the different distresses improve. For this study, the models for the three distresses discussed here produce results that are believed to be accurate and reliable. Alligator cracking is modeled by Pavement ME but the output remained nearly zero for all analysis and was not included as it skewed the performance curves upwards considering it is weighted at 16%. It was beyond the scope of this study to determine if this was due to any issues of the model in Pavement ME. Using only rutting, IRI, and longitudinal cracking provides a more distinct difference between the performances of the structures analyzed. Since only 3 of the 6 distresses are producing reliable results they will be used in producing the pavement score for the modeled roads. The three distresses, rutting, IRI, and longitudinal cracking, make up 67% of RIDOT's 2013 scoring system. The pavement score used in comparisons will be based out of 67 instead the 100. This is to allow the comparison to still be out of 100%. The results of this phase of the study with alligator cracking included can be found in Appendix A.

Table 4.2: Medium Severity Longitudinal Cracking

Medium Severity	
ft.	Deduction
0-3.28	0
3.28-65.62	-10
65.62-131.23	-20
131.23-196.85	-30
196.85-262.47	-40
262.47-328.08	-50
328.08-393.7	-60
393.7-459.32	-70
459.32-524.93	-80
529.93-590.55	-90
590.55	-100

The combined score is given an overall rating based on the ranges below [35].

- Excellent: 90.5 – 100
- Good: 84.1 – 90.5

- Fair: 75.6 – 84.1
- Poor: 64.5 – 75.6
- Failed: 0- 64.5

Not only do the pavements get an overall quality score but the individual scores are taken into account by their pavement repair trigger system. The respective alternative chosen for use in the repair are based on an intricate system of different triggers. Table 4.3 below shows various triggers for each alternative [35]. The triggers that are useable with the three distresses being used for the cost analysis are numbered in Table 4.3. Also the roadway classification varies the triggers, such as principal arterials versus interstates. The triggers take into account for all of the weighted distresses individually except for rutting. Each alternative has a trigger which is based on the overall pavement score so rutting is still accounted for in the triggers though not individually. Since rutting is not a major issue for Rhode Island taking it into account is not as important as the different types of cracking and IRI. The alternatives again are listed below.

- Crack Seal
- Level and Overlay
- Mill and Overlay(with and without a friction course)
- Paver Placed Elastomeric Surface Treatment(PPEST)
- Rubberized Chip Seal
- Reclamation
- Reconstruction
- Stress Absorbing Membrane Interlayer (SAMI)

Table 4.3: *Trigger System*

Crack seal- Interstate	PPEST - Principal Arterials
1 IRI>=80 & 90>Longitudinal>=70 &90>Transverse>=70	Alligator>80 & 90>Block≥70 15 80>IRI≥60 & 80>Longitudinal≥60 & 80>Transverse≥60 & Alligator >80
2 80>PSHI>=75	
	16 80>PSHI≥65
Crack seal- Prin. Artls & Non NHS	
3 IRI>=65 & 90>Longitudinal>=60 90>Transverse>=65 & Alligator>=60	PPEST - Non NHS routes Alligator>70 & 75>Block≥50
4 PSHI>=85	17 IRI<70 & Longitudinal≥50 Transverse≥50 & Alligator>70
Level and Overlay-Principal Arterials	18 80>PSHI≥65
5 80>IRI>=40 & Long. Cracking <60 & Transverse<60 & Alligator >=80	 Rubberized chip seal - Non NHS (Rural)
Block Cracking < 50	Alligator≥70 & 70>Block≥50
6 70>PSHI>=60	19 IRI≥65 & 70>Longitudinal ≥50 70≥Transverse>50 &90>Alligator≥70
Level and Overlay	20 85>PSHI≥75
7 PSHI<70 & Longitudinal<50 Transverse <50 & 90>Alligator>65	 Reclaim - Prin. Arterials/ NonNHS (Rural)
Block Cracking < 50	Alligator <70
8 PSHI<70 And Alligator ≥60	Block ≤50
	21 PSHI<60
Mill & overlay w/FC- Interstates	
9 IRI<90 & Longitudinal <80 Transverse <80	Reconstruct Prin. Artls/Non NHS (Urban) 22 IRI<60 & Alligator<70
Block ≤75 Alligator >80	Block Cracking≤50
10 PSHI <85	23 PSHI<60
Mill and overlay - Principal arterials	SAMI- Principal Arterials
11 80>IRI≥40 & Longitudinal <60 Transverse<60 & Alligator≥80	Alligator >70 & Block <50 24 IRI<80 & Longitudinal <70 Alligator≥70
Block≤50 & Alligator>80	25 80>IRI≥40 & Transverse <70 Alligator≥70
12 70>PSHI≥55	26 80>PSHI≥70
Mill and overlay - Non NHS routes	SAMI- Non NHS Routes
13 IRI<70& Longitudinal<50 Transverse<50 & 90>Alligator≥65	Alligator>55 & Block<50 27 IRI<70 & Longitudinal<70 80>Alligator>55
Block ≤50 & Alligator>65	28 IRI<70 & Transverse<70 80>Alligator>55
14 70>PSHI≥55	29 80>PSHI≥70

Using the triggers and their respective weights, ranges were developed for each mitigation alternative as seen in Figure 4.3. This was done to identify which alternatives get triggered first. All of the alternatives, except mill and overlay with a friction course and reclamation and reconstruction, had a range of PSHI scores as a trigger but they usually fell within the range formed by other triggers. The triggers are reached in 4 groups as seen below.

- Crack Seal & PPEST
- Mill and Overlay with a Friction Course, Rubberized Chip Seal, & SAMI
- Level and Overlay & Mill and Overlay
- Reclamation and Reconstruction

The ranges make sense as the initial mitigation alternatives address mostly cracking related distresses. Then the roughness and rutting mitigation alternatives start getting triggered as the overall pavement performance worsens. This information will be useful later when optimizing a repair plan as the order that these alternatives are triggered can help prioritize certain alternatives over others. Though it will also be taken into account that rubberized chip seal and the reclamation alternatives are specifically for rural roadways that only make up about 20% of roadways in Rhode Island [36].

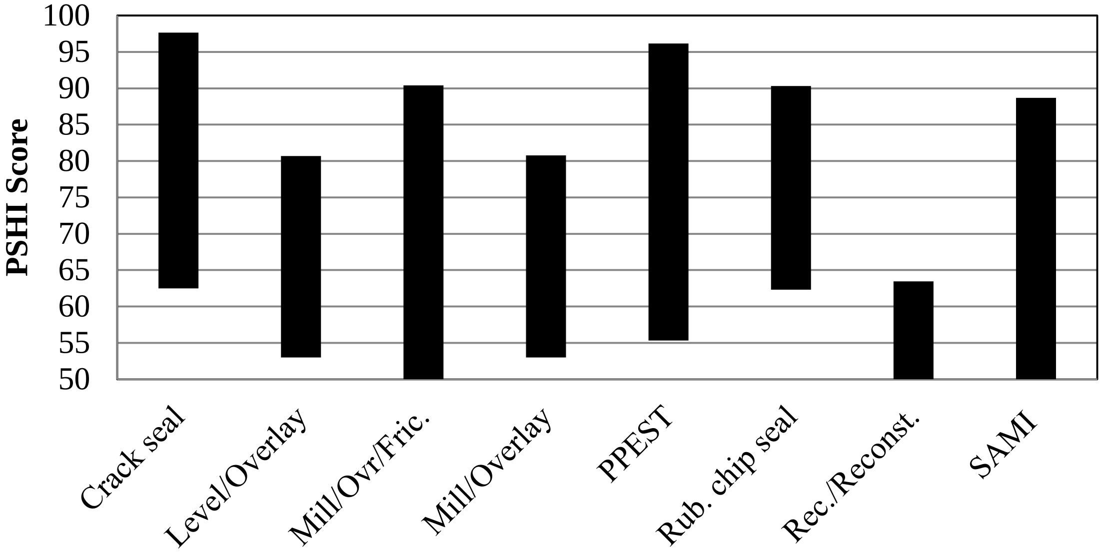

Figure 4.3 Mitigation Alternative Trigger Range

4.2 RIDOT Pavement Preservation Analysis Set up

The first step is to identify accurate structure for pavement preservation modeling is to look at historic data from RIDOT. In a design parameter paper released by RIDOT in August 2003 had useful structure data for the development of these structures [21]. It included a table of structural data on several roadways including Rte 2, Rte146N, Upper College Road, Roger Williams Way, Rte 107, Rte138, Charles Street, and Rte 146S. It included information on the different layers including a basic descriptor, thickness, modulus, Poisson's Ratio, and density. Though for the asphalt layers it did not include performance grade it did include the modulus so similarities between modulus values could be seen. This could indicate possibility of different layers sharing a similar or the same performance grade. The actual performance grades were picked from the Rte. 165 analysis as the values were being used in a current project. Also default Pavement ME gradations will be used as they were found to follow RIDOT gradations as found out in the Rte 165 analysis. The subbase and subgrade modulus values did vary by approximately 10,000 and 5,000 psi, respectively. The average and the median values for both layers were within 200 psi and because of this the average values will be used for both layers. The materials used for modeling were picked from data provided in the Rte. 165 analysis and the RIDOT design parameter paper used for the structure development. The weigh-in-motion (WIM) data used for Rte. 165 was also used in this analysis [33]. From the traffic counts provided the AADT and percentage of trucks were determined for seven different roadways. Using the above data, a vehicle distribution was determined. The variation between distributions was significant; therefore, two vehicle distributions were developed. The first distribution had a 22% higher concentration of Class 9 trucks and the second distribution had over 80% of trucks as a Class 5 truck. The largest AADTT found from the weigh-in-motion stations will be used for the analysis. Through this data, three different structures, two vehicle distributions, and one AADTT were picked. The summary of pavement structures and the two distributions can be seen below in Table 4.4 and Table 4.5, respectively. The AADTT of 2100 was selected. A structure based on the statistical analysis was developed. The structure thicknesses were the same as the base structure before any analysis was done bringing the total amount of structures up to four different structures. The PG was kept at PG64-28 as it was more representative of the binder grade found in RI. The modulus of the third layer was kept at the default value of 38,000 psi as it is more realistic for A-1-b being the third layer. The locations of the WIM stations and the historic structures used to develop the structure and traffic information for this study can be seen in Figure 4.4.

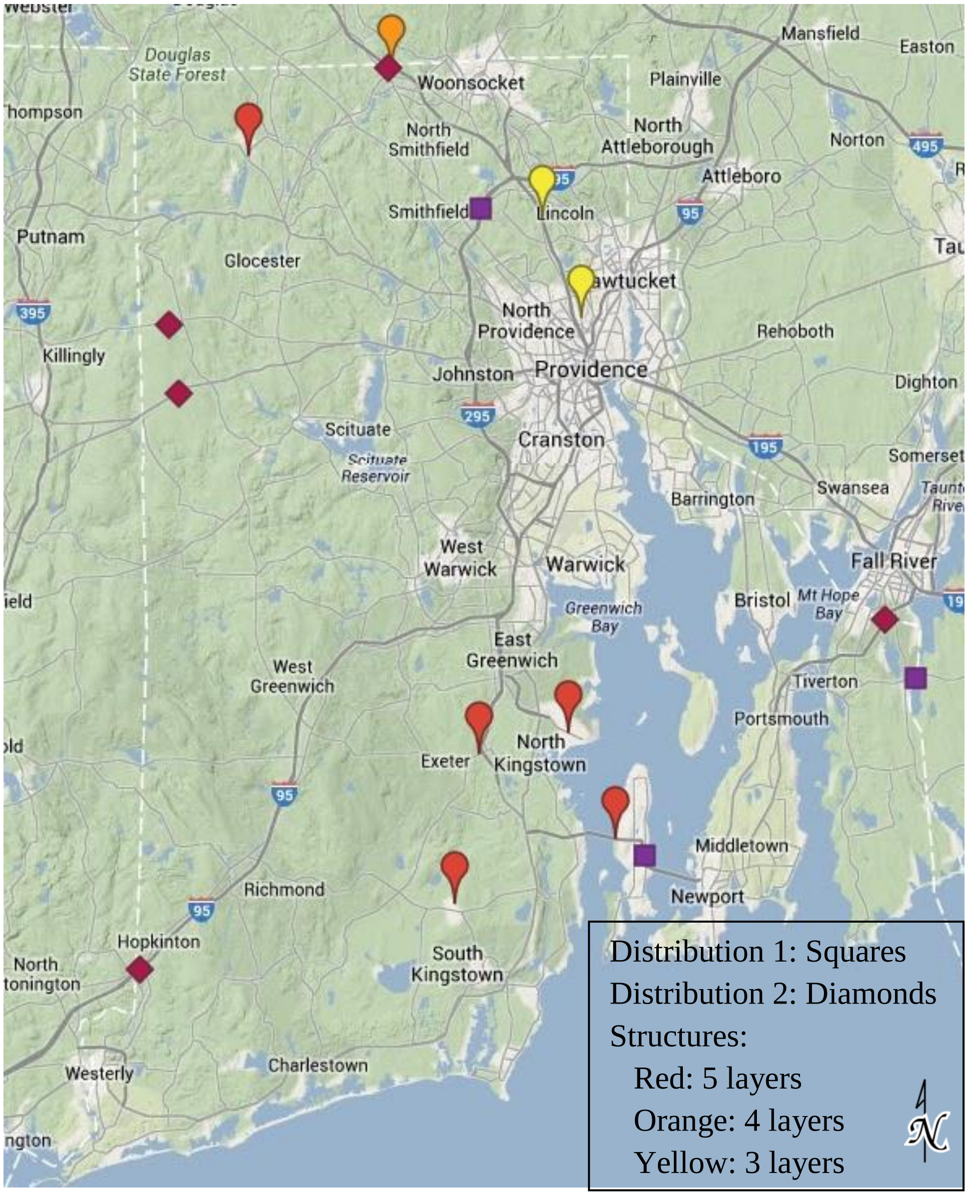

Figure 4.4 WIM Stations and Historic Structure Locations

Table 4.4: Pavement Preservation Structures

Historic Pavement Structures				
Material	**Thickness**			**Modulus or PG Grade**
	Struc 1	Struc 2	Struc 3	
HMA	8	2	2	PG 64-28
			2	PG64-22
		8	4	PG 64-28
Crushed Gravel	12”			17,000 psi
A-1-b Subgrade	Semi-Infinite			12,500 psi
Base Structure for Sensitivity Analysis- Struc 4				
HMA	2			PG 64-28
	6			PG 64-22
A-1-b	7			38,000 psi
Crushed Gravel	5			20,000 psi
A-1-b	Semi-Infinite			20,000 psi

Table 4.5: Pavement Preservation Vehicle Distribution

Vehicle Class	Distribution 1	Distribution 2
Class 4	2.52%	2.31%
Class 5	52.41%	83.01%
Class 6	7.54%	6.05%
Class 7	0.98%	0.95%
Class 8	7.55%	2.24%
Class 9	27.44%	5.27%
Class 10	0.72%	0.14%
Class 11	0.51%	0.02%
Class 12	0.20%	0.00%
Class 13	0.11%	0.01%

4.3 RIDOT Pavement Preservation Analysis

The four structures were run with both distributions in Pavement ME and three distresses, rutting, IRI, and longitudinal cracking, were analyzed. Each distress was scored according to their respective system and then combined according to their respective weightage. Each of the runs was checked for 29 useable triggers from Table 4.3. They were checked over the twenty year life span for when each trigger was activated by individual or combined distresses. Out of the 29 triggers, nine triggers were activated and out of those nine, five different repair alternatives were trigged. The five alternatives triggered were crack seal, mill and overlay with a friction course, PPEST, chip seal, and SAMI. When each trigger was activated for the four structures can be seen in Table 4.6 below. All of the different structures triggered all nine except for structure 2 and 4 using truck distribution 2. Overall, rutting was the distress that had the lowest rated score due to its severity. The IRI rated score controlled for a noticeable portion of the 20 year life span was specifically for structure 2 and 4 using truck distribution 2. Whenever the IRI rated score controlled, the rutting rated score was within 2 to 3 points of the IRI rated score.

Table 4.6: Timing of When Each Trigger was Activated

	Structure	1		2		3		4	
	Distribution	1	2	1	2	1	2	1	2
Trigger	Repair	Year Triggered							
2	Crack Seal	10.08	16.75	15.92	--	10.00	16.42	14.92	--
4	Crack Seal	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	Mill/Ovly/FC	5.00	7.25	7.08	9.92	4.92	7.08	7.83	11.08
16	PPEST	10.08	16.75	15.92	--	10.00	16.42	14.92	--
18	PPEST	10.08	16.75	15.92	--	10.00	16.42	14.92	--
20	Chip Seal	5.00	7.25	7.08	9.92	4.92	7.08	7.83	11.08
25	SAMI	11.50	12.92	12.83	14.00	11.42	12.83	13.17	14.50
26	SAMI	10.08	16.75	15.92	--	10.00	16.42	14.92	--
29	SAMI	10.08	16.75	15.92	--	10.00	16.42	14.92	--

The triggers were all activated at different times. Trigger four’s requirements were a PSHI score of 85 or greater so it was activated from the start. Structure 1 and 3 had almost identical trigger points. Their respective triggers were all within a few months of each other. Structures 2 and 4 had a similar trend but the difference between the two structures was a year or less. The next step was to look at the repairs individually to see which ones were the most cost effective. Using the timing of when each trigger was activated and the years that were added for each alternative, how many times each alternative can be implemented before reaching the 20 year design life was calculated. The twenty year was used as the cutoff date to compare to the "do nothing" alternative where the cost would be $350,000 per lane mile to reconstruct as mentioned previously in literature review about the repair alternatives. The individual repair alternative costs listed below are provided by RIDOT's pavement management department [16,17]. These numbers were based on cost information from 1998-2013[16,17]. The overall costs calculated are ranges as the years added for each alternative is also a range due to variability in its effectiveness. The information for the time added by the SAMI alternative was estimated from the description of the alternative. As mentioned earlier, SAMI is a chip seal with an overlay on top. The chip seal adds 3 to 7 years to the performance and an overlay adds 5 to 10 years. To be conservative, the years added were assumed to be an average of the two alternatives, 5 to 7 years. The amount of repetitions for each alternative was rounded down to the nearest integer so that the alternatives benefit would be completely within the 20 year design life. At a minimum, each alternative needed to be implemented at least once. The total cost of using a given alternative multiple times to achieve a 20 year design life was calculated as a percentage of the reconstruction cost, which was $350,000 per lane mile, shown in Table 4.7. The highlighted alternatives are the top three cost effective alternatives with the lowest average cost expressed as a percentage of the reconstruction cost. The top three cost effective alternatives are based off of the average of the minimum and maximum of the range and not just the minimum. In the case of multiple triggers being tied for the average cost, both triggers will be highlighted. For example, in Structure 1 with distribution 1, triggers 25, 26 and 29 are tied for third in lowest average cost so all three are highlighted.

- Crack Seal-$4,500 per lane-mile
- Mill and Overlay with a friction course-$142,000 per lane-mile
- PPEST-$86,000 per lane-mile
- Rubberized Chip Seal-$41,000 per lane-mile
- Stress Absorbing Membrane Interlayer (SAMI)-$125,000 per lane-mile

Table 4.7: Individual Repair Alternative Cost Effectiveness.

	Structure	1				2			
Distribution/Range		1-Min	1-Max	2-Min	2-Max	1-Min	1-Max	2-Min	2-Max
Trigger	Repair	% of Reconstruction Cost							
2	Crack Seal	1.27	5.08	1.27	1.27	1.27	2.54	--	--
4	Crack Seal	3.81	12.71	3.81	12.71	3.81	12.71	3.81	12.71
10	Mill/Ovly/FC	40.55	121.64	40.55	81.10	40.55	81.10	40.55	81.10
16	PPEST	24.64	49.27	24.64	75.41	24.64	24.64	--	--
18	PPEST	24.64	49.27	24.64	75.41	24.64	24.64	--	--
20	Chip Seal	23.27	58.18	11.64	46.54	11.64	46.54	11.64	34.91
25	SAMI	35.67	35.67	35.67	64.37	35.67	35.67	35.67	35.67
26	SAMI	35.67	35.67	35.67	35.67	35.67	35.67	--	--
29	SAMI	35.67	35.67	35.67	35.67	35.67	35.67	--	--
	Structure	3				4			
	Distribution	1-Min	1-Max	2-Min	2-Max	1-Min	1-Max	2-Min	2-Max
Trigger	Repair	% of Reconstruction Cost							
2	Crack Seal	1.27	6.35	1.27	1.27	1.27	2.54	--	--
4	Crack Seal	3.81	12.71	3.81	12.71	3.81	12.71	3.81	12.71
10	Mill/Ovly/FC	40.55	121.64	40.55	81.10	40.55	81.10	40.55	40.55
16	PPEST	24.64	49.27	24.64	24.64	24.64	24.64	--	--
18	PPEST	24.64	49.27	24.64	24.64	24.64	24.64	--	--
20	Chip Seal	23.27	58.18	11.64	46.54	11.64	46.54	11.64	23.27
25	SAMI	35.67	35.67	35.67	35.67	35.67	35.67	35.67	35.67
26	SAMI	35.67	71.33	35.67	35.67	35.67	35.67	--	--
29	SAMI	35.67	71.33	35.67	35.6	35.67	35.67	--	--

Both crack seal alternatives were the top cost effective alternatives in all systems for both distributions. SAMI and chip seal were each in the top three cost effective alternatives four times. PPEST was in the top three twice. Overall, this makes sense as these four alternatives listed above are the four cheapest alternatives. Even though PPEST is cheaper than SAMI, SAMI was triggered later in the life span and added on average more years to its life span, thus, making it more cost efficient.

4.4 RIDOT Incremental Cost Benefit Analysis

The next step would be to look at each alternative with more specifics such as overall use and time until a trigger was activated. This would be done through an incremental cost benefit analysis as described earlier. In place of monetary benefits, a rating system will be used for the benefit rating as monetary benefits could not be determined. To develop a rating system, each repair alternative was scored out of ten points depending on its ability to repair each distress used for the analysis, rutting, IRI, and cracking. The cracking score was based on overall ability to repair all types of cracking. Since all types of cracking account for 40% while rutting and IRI are 30% each, it was decided to keep the three equally rated to follow the RIDOT rating system [35]. Even though overall cracking does have a 10% higher rating, this cracking is based on four different types of cracking while IRI and rutting are only based on one distress each. Due to that reasoning cracking was not given a higher weightage and was kept at an even level with IRI and rutting For this analysis, the IRI rating was based off of its ability to improve the friction for the roadway as improve in friction does help improve the IRI. This is because with higher roughness there is more water infiltration and standing water causing lose of friction. Below in Table 4.8 is the scoring for the individual alternatives. The score of ten means that the technique repaired various severities of that distress and zero meant that it did not contribute to the structural repair of the distress. A rating of a five means that it worked for lower severities and/or helped repair that distress in specific circumstances. As an example with PPEST, it received a 5 for rutting because road ways with higher severities of rutting the roadway need to be leveled before applying the PPEST.

Table 4.8: Distress Benefit Rating System

Trigger	Repair	Rutting	IRI/Friction	Cracking	Distress Benefit Subtotal
2 & 4	Crack Seal	0	0	10	10
10	Mill/Ovly/FC	10	10	10	30
16 & 18	PPEST	5	10	5	20
20	Chip Seal	0	10	10	20
25, 26, & 29	SAMI	10	10	10	30

It can be seen that the highest ratings are for mill and overlay with a friction course and SAMI. Both of these alternatives use a method to repair/remove cracks before adding an overlay. PPEST and Chip seal repair different distresses in varying degrees while crack seal focuses specifically on cracking distresses. A time-benefit factor was determined to be needed. This addressed the focus of this part of the study on determining the timing of the alternatives to be more cost effective. Waiting to use one alternative over another can produce savings as seen in the initial cost analysis above. To quantify this, a time-benefit factor was added. This time-benefit factor was the time taken until that alternative was activated and dividing that number by two. This was done to keep the highest score to ten considering the longest wait is 20 years, which is the design life. For example, a chip seal was triggered at the 5 year for the 3 layer structure with the first distribution so the time-benefit factor would be 2.5. This would make all the four benefit criteria rated, rutting, IRI/friction, cracking, and time, at the same scale and weight. Shown in Table 4.9 below are the overall benefit ratings for the different systems using the two traffic distributions.

Table 4.9: Overall Benefit Ratings

	Structure		1		2		3		4	
	Distribution		1	2	1	2	1	2	1	2
Trigger	Repair	Cost $/Ln,mi.	Total Benefit							
2	Crack Seal	4,450	15.0	18.4	18.0	--	15.0	18.2	17.5	--
4	Crack Seal		10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
10	Mill/Ovly/FC	142,000	32.5	33.6	33.5	35.0	32.5	33.5	33.9	35.5
16	PPEST	86,280	25.0	28.4	28.0	--	25.0	28.2	27.5	--
18	PPEST		25.0	28.4	28.0	--	25.0	28.2	27.5	--
20	Chip Seal	40,750	22.5	23.6	23.5	25.0	22.5	23.5	23.9	25.5
25	SAMI	124,900	35.8	36.5	36.4	37.0	35.7	36.4	36.6	37.3
26	SAMI		35.0	38.4	38.0	--	35.0	38.2	37.5	--
29	SAMI		35.0	38.4	38.0	--	35.0	38.2	37.5	--

The difference between each alternative is more noticeable for each structure and distribution. The benefit scores range from 10 to 38.4. The incremental cost benefit line was developed for each structure and distribution using the process described earlier in the in study. Figure 4.5 thru Figure 4.12 show the efficiency frontier for each structure and distribution with the triggers labeled by their specific reference number as seen in Table 4.9. From this the most cost efficient alternatives can be determined. Certain triggers were more efficient than other triggers of the same alternative specifically because they were triggered at a later date such as trigger 2 being more efficient than trigger 4. The triggers that were deemed the most cost efficient are summarized in Table 4.10.

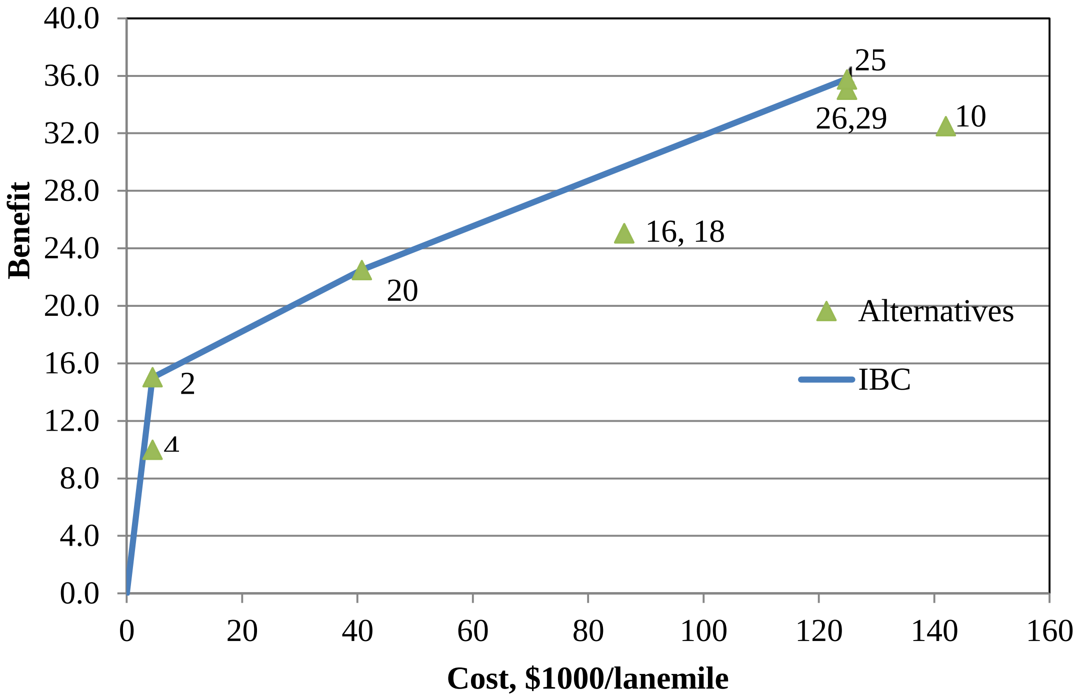

Figure 4.5 Incremental Benefit Cost Graph for Structure 1 Distribution 1

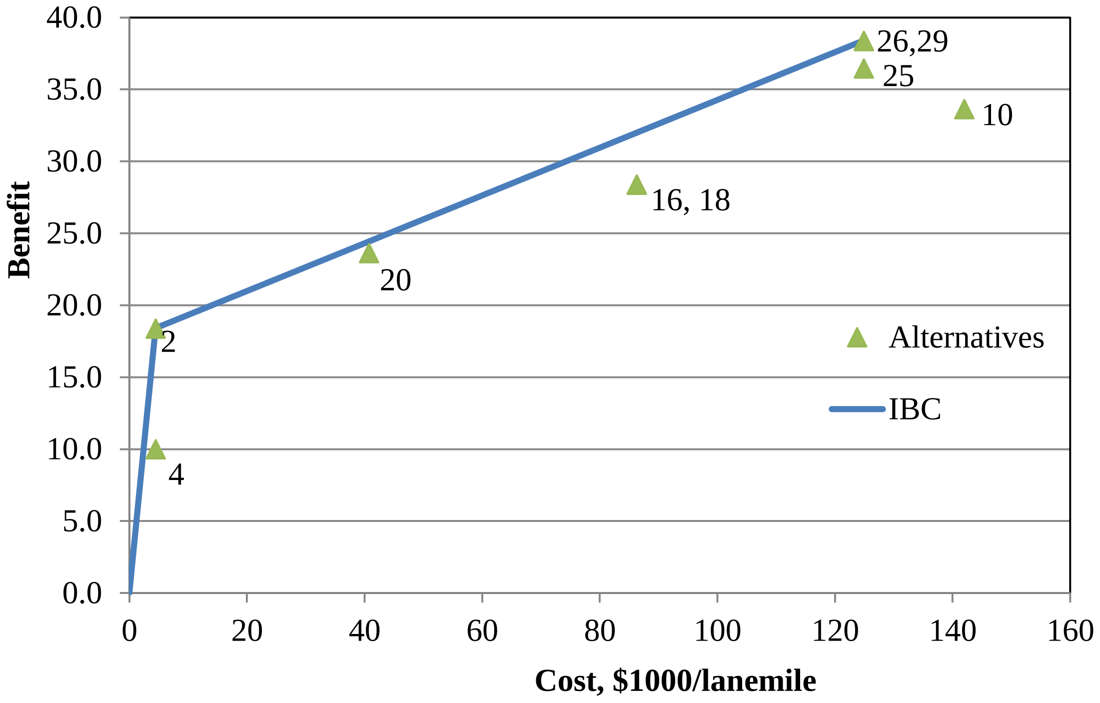

Figure 4.6 Incremental Benefit Cost Graph for Structure 1 Distribution 2

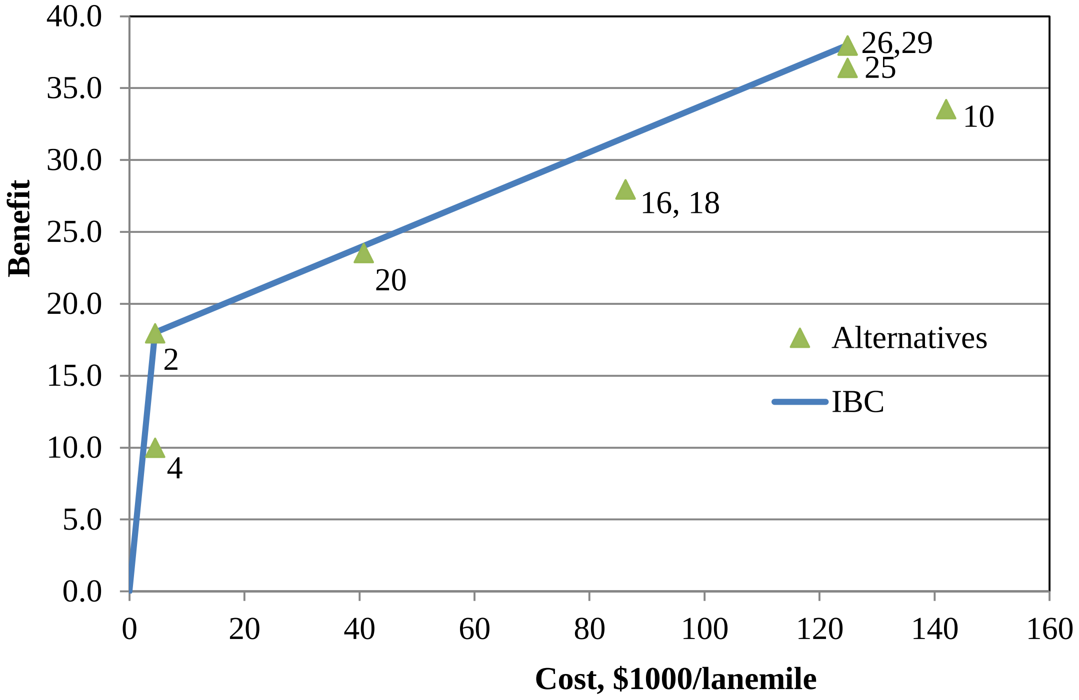

Figure 4.7 Incremental Benefit Cost Graph for Structure 2 Distribution 1

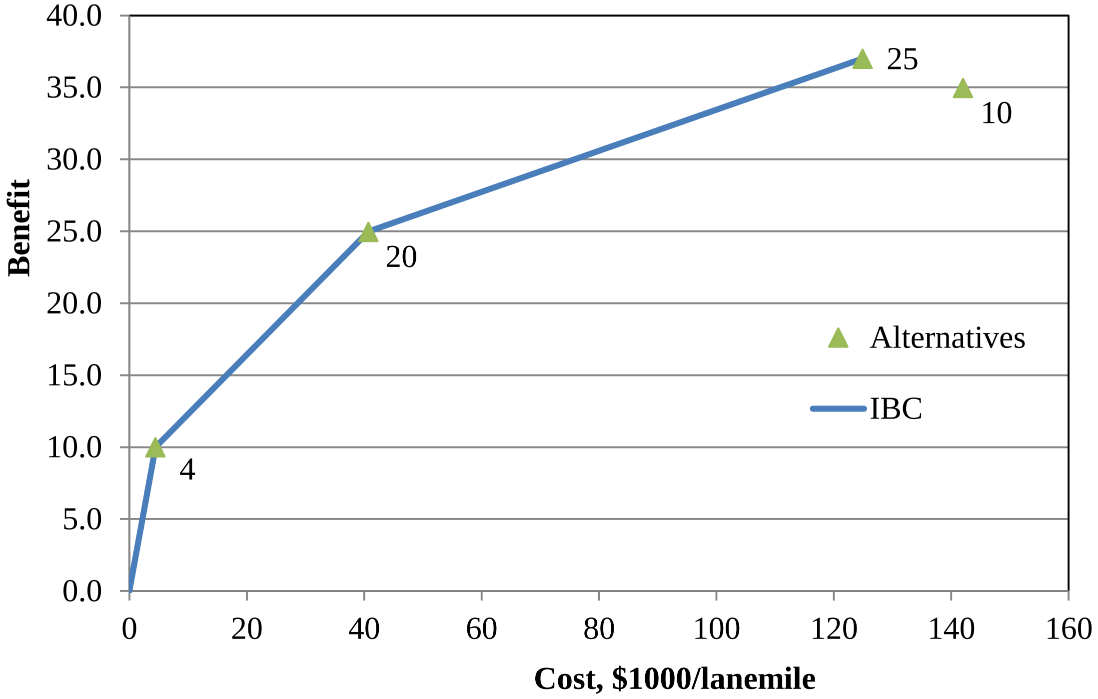

Figure 4.8 Incremental Benefit Cost Graph for Structure 2 Distribution 2

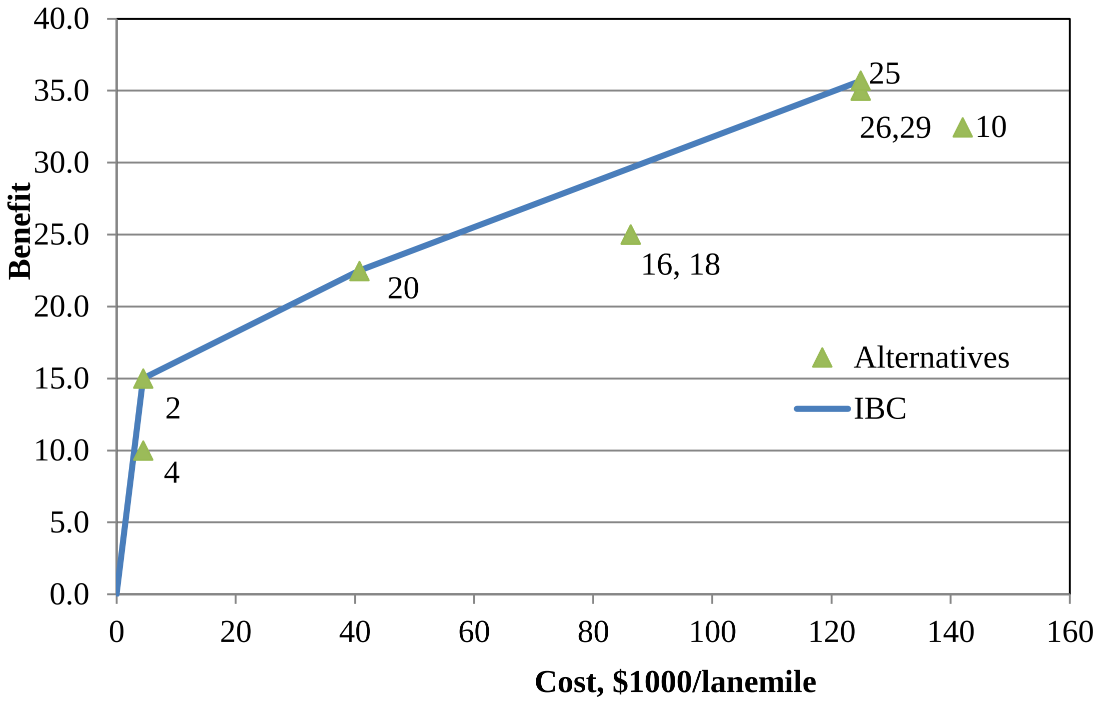

Figure 4.9 Incremental Benefit Cost Graph for Structure 3 Distribution 1

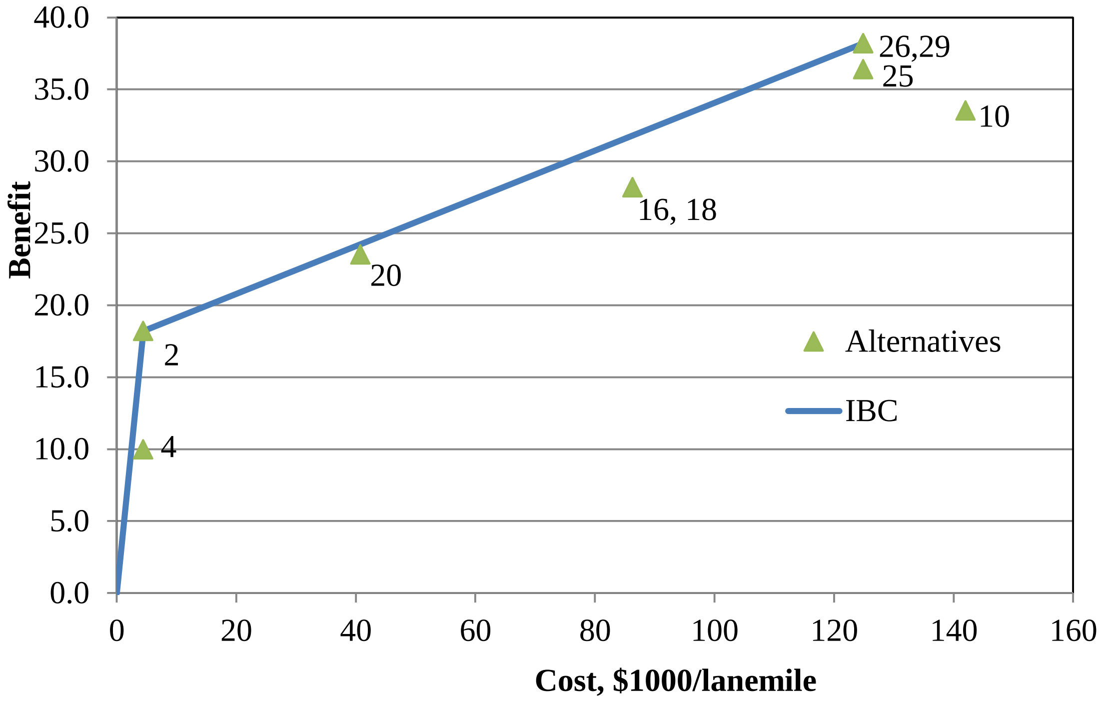

Figure 4.10 Incremental Benefit Cost Graph for Structure 3 Distribution 2

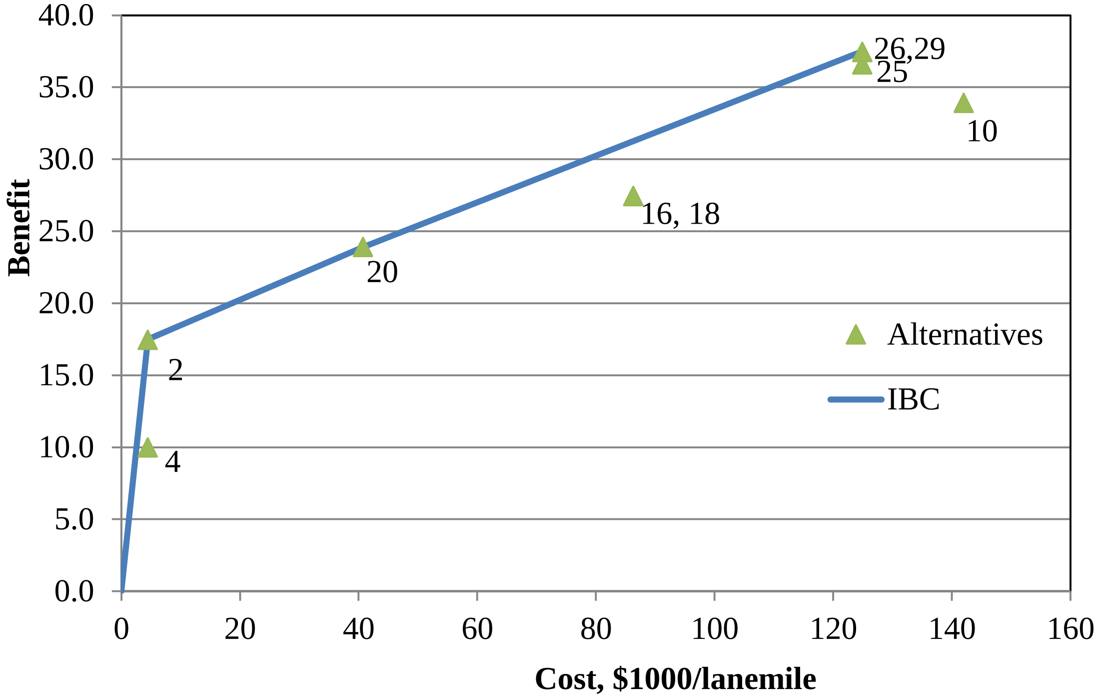

Figure 4.11 Incremental Benefit Cost Graph for Structure 4 Distribution 1

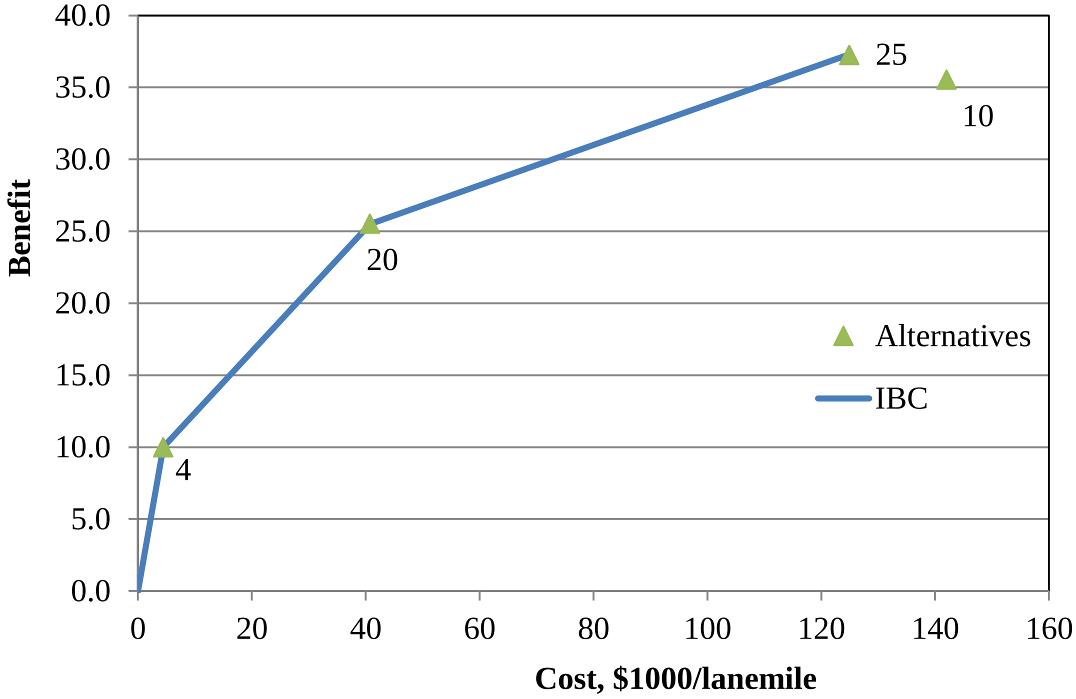

Figure 4.12 Incremental Benefit Cost Graph for Structure 4 Distribution 2

Table 4.10: Incremental Benefit Cost Efficient Alternatives

	Structure	1		2		3		4	
	Distribution	1	2	1	2	1	2	1	2
Trigger	Repair	Cost Efficient Alternatives							
2	Crack Seal	X	X	X	--	X	X	X	--
4	Crack Seal				X				X
10	Mill/Ovly/FC								
16	PPEST				--				--
18	PPEST				--				--
20	Chip Seal	X			X	X		X	X
25	SAMI	X			X	X			X
26	SAMI		X	X	--		X	X	--
29	SAMI		X	X	--		X	X	--

The three alternatives deemed most efficient were crack seal, chip seal, and SAMI. This table shows similar results to the initial cost analysis. Since PPEST had limitations to the distresses it could repair, it became less cost effective and was not part of the incremental cost benefit efficiency frontier. The similarities between Structure 1 and 3 and the similarities between Structures 2 and 4 can be seen here. The similarities carried over into which alternatives were more cost efficient by following the same pattern. The only difference was chip seal was cost efficient for the sensitivity analysis structure with distribution 1. Mill and overlay with a friction course was too costly for the amount of benefit added. SAMI added a similar benefit with less cost. The overall combination of the more efficient alternatives is going to depend on the distresses on the roadway and the overall budget for road repair.

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

The initial Pavement ME Design phase produced good results that showed some key factors to look at while designing pavement structures. The first analysis, analyzing actual RIDOT roadways, Rte138, Old Victory Highway, Rockland Rd, and Rte 102, showed the following results

- RIDOT structures needed a smaller base layer, 2” smaller, than expected, saving RIDOT about $200,000.
- Rte. 102 needed 2.5” on Structure 2 to achieve the specified 95% reliability

The second analysis, level III sensitivity analysis, showed the following results. The sensitivity analysis varied traffic distribution, new HMA layer PG, modulus of the base layer, thicknesses of the surface and existing HMA layer, and traffic volume.

- Rhode Island has about 80% of their truck traffic comprised of class 5 & 9 trucks.
- Using the RIDOT distribution was more realistic to conditions expected
- PG of binder of the New HMA only had a significant impact on longitudinal cracking and AC rutting.
- The modulus of the base layer affected the longitudinal cracking but had minimal impact on the other distresses.
- Increased thicknesses of the surface HMA layer did cause an overall decrease in both rutting performance by 35-40% and nearly 100% decrease in longitudinal cracking.

- Each additional inch of existing HMA thickness milled caused about a 3 and 5% increase in terminal IRI and AC rutting respectively.
- Longitudinal cracking increase by a few thousand ft/mile with each decreasing inch of existing HMA.
- Longitudinal cracking distress change between each traffic level was 10-15% higher than the change in traffic level itself while both rutting performances followed the opposite pattern.
- Traffic and new HMA layer thickness had the most significant impact on extending the pavement performance for the Fair rating goal used in this study

The statistical analysis determined the significance of the effects determined in the sensitivity analysis. This focused on the significance the parameters varied in the sensitivity analysis had on rutting, both AC and total, IRI, and longitudinal cracking.

- Thickness of the HMA layers and traffic has a significant impact on the pavement performance.
- Thickness of the existing HMA layers had a significant impact on all four distresses analyzed.
- New HMA Binder Grade and the base layer modulus had an impact on two of the four distresses but not all of the distresses evaluated.

The performance curves analysis resulted in the following findings. This analysis examined the effect the three significant parameters, the thickness of the two HMA layers and traffic, had on the overall pavement performance curve. A quadratic curve was fitted through the performance curves. The y-intercept of 89 was used as the performance curve stabilized once below a performance score of 89 due to the initial part of the RIDOT performance scoring system used to develop the curves.

- Surface HMA layer performance curves began to stabilize to one equation, see equation below, for a range of structures except for the 2” thickness.
  - Performance Index= $0.20\ x^2 + 0.89x + 89$
- Traffic performance curves had similarities between all of them except when AADTT was 500.
  - AADTT of 500, due to the lower loading, fit best with a linear equation of performance=$-0.502x+87.105$
- As the traffic decreases, the coefficient decrease through the decrease is not proportional with the decrease in traffic.
- With each drop in traffic level, the traffic performance curve equation’s coefficients stay close to the surface layer performance curve equation’s coefficients for each increase in surface thickness except for the AADTT of 500.
  - The x^2 coefficient stays within 0.006
  - The x coefficient stays within 0.1 if not exactly the same
- Performance curves were most sensitive to thickness of existing HMA layer
- With the surface HMA layer and the traffic, it shows that performance curves may stay similar for certain ranged of inputs before changing drastically.

The level I analysis was to determine the accuracy of the level III analysis and the overall sensitivity of Pavement. The analysis showed the following results.

- Fatigue cracking performance is sensitive to the layer thickness and traffic level.
- Rutting performance is sensitive to the top layer thickness and traffic level.

- Results followed trends found in level III results

The Rte. 165 analysis examined the sensitivity of subbase material properties on pavement performance.

- Control asphalt layer subbase layer performed the best out of the three pavements.
- All three subbases tested, asphalt, cement stabilized, and cold recycled asphalt, passed the 90% confidence reliability target except the longitudinal cracking in the recycled asphalt subbase but the confidence was still 89%.
- The difference in predicted performance between the asphalt and the cement stabilized layer are minimal
- The cold recycled asphalt had the highest distresses for IRI, longitudinal cracking and total rutting.
- Overall, the asphalt performed the best but the cement stabilized layer focused the distress in the easier accessed top layers.

This study agreed with the previous studies mentioned earlier in the literature review that some form of local calibration or using local inputs are helpful for using Pavement ME Design in the design of pavements. From these analyses, a cost analysis was completed for Rhode Island following historical data and the results from the sensitivity analyses. Through historic data and the results of the sensitivity analysis, four structures and two traffic distributions were developed. The four structures were structure 1, a 3 layered system, structure 2, a 4 layer system, structure 3, a 5 layer system, and structure 4, a structure based on the base structure used in the sensitivity analysis. The cost analysis was used to develop an “efficiency frontier” for each structure for both distributions. The efficiency frontier is formed by the repair alternatives that added the most benefit per unit cost. The benefit of each repair alternative was based off of a rating system. The rating system was based on the alternatives ability to repair three distresses, rutting, IRI/friction, and cracking, and a time dependent rating. The results showed the following findings.

- The first four alternatives triggered using the RIDOT scoring system are crack seal, PPEST, mill and overlay with a friction course, and rubberized chip seal
- Two traffic distributions were developed from WIM stations across the state
    - The first distribution had a 22% higher concentration of Class 9 trucks
    - The second distributions had over 80% of the trucks as Class 5 trucks
- From the initial cost analysis, crack seals were the most cost effective alternative for all structures and distributions.
- SAMI was cost effective for the first distribution for Structure 1 and 3 and was cost effective for the second distribution for Structure 2 and 4
- Chip seal was cost effective for the second distribution of Structure 1 and 4 and cost effective for both distributions with Structure2
- PPEST was cost effective only for the second distribution of Structure 3 and the first distribution of Structure 4
- From the incremental cost benefit analysis, crack seal and SAMI were cost effective for all structures and distribution
- Chip seal was cost effective for the first distribution for all structures except Structure 2 and was cost effective for the second distribution for Structure 2 and 4

5.2 Recommendations

The following recommendations are for all state agencies.

- Sensitivity analysis does not need to be completed unless the sensitivity of a specific parameter is needed for a project.
- A cost analysis for the repair alternatives should be done for any agency but the type of analysis depends on the information available.
    - If budgetary numbers are available then a marginal cost effectiveness analysis should be completed
    - If no budgetary numbers are available then an incremental cost benefit analysis should be completed.
        - Monetary benefits would give more definitive results as it can be directly compared to cost of a repair alternative
        - Nonmonetary benefits can be used but each rating system for the nonmonetary benefits will be dependent on needs which distresses are of more critical importance.
- Using Pavement ME as a preservation tool works best with localized data which can be done in multiple ways such as the following. With increasing the size of the state this is analyzed the localized data below should be broken up into different analysis regions.
    - Weigh-in-motion stations for more accurate traffic loading
    - Performance curves based on historic pavement data to help develop pavement structures that fit performance required of the project
    - Climatic stations

- Overall, this methodology could be implemented for any agency but depending on the different climate zones and traffic levels. The state being analyzed would be done for different regions rather than for the entire state.

Recommendations for Rhode Island DOT would include the following few steps listed below

- Focus on crack seal, rubberized chip seal, and SAMI repair alternatives as they are the most cost efficient alternatives
- SAMI should be developed more as it provides great benefit for repair with a cheaper cost when compared to mill and overlay with a friction course which provides similar benefits with higher cost.
- Overall, Pavement ME is a useful tool for preservation management for RIDOT.

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Appendix A

Analysis with Alligator Cracking

This appendix show the results from this study when alligator cracking is included. The alligator cracking results from the sensitivity analysis are listed below in Table A.1.

Table A.1 Sensitivity Analysis Alligator Cracking Distress Results

Binder Grade	PG52	PG58	PG64	PG70	PG76	PG80
Alligator Cracking (%)	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Base Layer Modulus, psi	12,372	20,845	29,316	37,786	46,259	
Alligator Cracking (%)	0.0000	0.0000	0.0000	0.0000	0.0000	
Surface Layer Thickness	2”	3”	4”	5”	6”	
Alligator Cracking (%)	0.0000	0.0001	0.0012	0.0029	0.0037	
Existing Layer Thickness	2”	3”	4”	5”	6”	
Alligator Cracking (%)	0.0002	0.0000	0.0000	0.0000	0.0037	
Traffic, AADTT	500	1000	1500	2000	4000	
Alligator Cracking (%)	0.0002	0.0000	0.0000	0.0000	0.0037	

Below in Table A.2, the time until the Fair Rating is reached was increased by the addition of alligator cracking. The range of values between PG 52 and PG 80 was 2.83 years instead of a year, when alligator was not included. For the extreme modulus values of 12,372-46,259 psi the range of years to reach fair rating increased from 1.33 years to 1.92 years. The years to reach fair rating for the range of surface layer thicknesses evaluated varied from 7.17 years to 11.83 years. The increase in the years to due to existing layer thickness’ range was from 3.08 years to 5.2 years. The traffic had the biggest increase from 9.67 years to 12+ years as the Fair Rating was not reached for the AADTT of 500.

Table A.2 Sensitivity Analysis Time until Fair Rating

Binder Grade	PG52	PG58	PG64	PG70	PG76	PG80
Without Alligator Cracking	2.92	3	3.25	3.75	3.83	3.92
With Alligator Cracking	5	5.83	6.08	7	7.58	7.83
Base Layer Modulus, psi	12,372	20,845	29,316	37,786	46,259	
Without Alligator Cracking	3.25	3.92	4	4.08	4.58	
With Alligator Cracking	6.08	7.67	7.92	8	8	
Surface Layer Thickness	2”	3”	4”	5”	6”	
Without Alligator Cracking	4.58	7.08	8.85	9.92	11.75	
With Alligator Cracking	8	10.25	13.92	16.5	19.83	
Existing Layer Thickness	2”	3”	4”	5”	6”	
Without Alligator Cracking	1.5	1.92	2.83	3.83	4.58	
With Alligator Cracking	2.8	3.8	5.9	7	8	
Traffic, AADTT	500	1,000	1,500	2,000	4,000	
Without Alligator Cracking	14.25	11.25	9	7	4.58	
With Alligator Cracking	N/A	16	13	12	8	

The relationships for the performance curve equations remained the same but the coefficients for the equations decreased. With the addition of the alligator cracking, instead of starting the pavement score at 89, due to the drops caused by rutting and IRI, it was started at 91 as can be seen below in Table A.3.

Table A.3 Performance Curve Equations with and without Alligator Cracking

Surf. HMA	Without Alligator Cracking	With Alligator Cracking
2”	$0.040x^2-1.55x+89$	$0.031x^2-1.23x+91$
3”	$0.022x^2-1.15x+89$	$0.015x^2-0.88x+91$
4”	$0.020x^2-0.99x+89$	$0.015x^2-0.77x+91$
5”	$0.020x^2-0.94x+89$	$0.015x^2-0.73x+91$
6”	$0.020x^2-0.89x+89$	$0.014x^2-0.69x+91$
Exist. HMA	Without Alligator Cracking	With Alligator Cracking
2”	$0.077x^2-2.95x+89$	$0.061x^2-2.36x+91$
3”	$0.061x^2-2.50x+89$	$0.048x^2-1.99x+91$
4”	$0.043x^2-1.98x+89$	$0.034x^2-1.57x+91$
5”	$0.033x^2-1.61x+89$	$0.026x^2-1.28x+91$
6”	$0.040x^2-1.55x+89$	$0.031x^2-1.23x+91$
Traffic, AADTT	Without Alligator Cracking	With Alligator Cracking
4,000	$0.040x^2-1.55x+89$	$0.031x^2-1.23x+91$
2,000	$0.028x^2-1.20x+89$	$0.021x^2-0.94x+91$
1,500	$0.024x^2-1.09x+89$	$0.018x^2-0.85x+91$
1,000	$0.017x^2-0.94x+89$	$0.012x^2-0.73x+91$
500	$0.061x^2-1.27x+89$	$0.005x^2-0.98x+91$

Eight new triggers were useable with alligator cracking and are listed in Table A.4. All of the new alligator cracking triggers were based on the alligator cracking score being above a specific number.

Table A.4 New Useable Triggers and Trigger Requirements

Trigger #	Repair Alternative	Trigger Distress Level Requirements
30	Mill and overlay with friction course	Block Cracking<=75 and Alligator Cracking >80
31	Mill and overlay	Block Cracking<=50 and Alligator Cracking >80
32	Mill and overlay	Block Cracking<=50 and Alligator Cracking >65
33	Paver Placed Elastomeric Surface Treatment (PPEST)	90>Block Cracking>=70 and Alligator Cracking>80
34	PPEST	75>Block Cracking>=50 and Alligator Cracking>70
35	Rubberized chip seal	70>Block Cracking>=50 and Alligator Cracking>=70
36	Reclamation	Alligator Cracking<70
37	SAMI	Block Cracking<50 & Alligator Cracking>55

With the addition of the alligator cracking raising PSHI, triggers 2, 16, 18, 26 and 29 were only activated for Structure 1 and 3 with the first distribution. All of the repairs were triggered later in the design life due to the higher PSHI scores. The new triggers that involved alligator cracking were trigger automatically, seen below in Table A.5, as they were based on the alligator cracking score being above a specific score for that distress.

Table A.5 Timing of When Each Trigger was Activated with Alligator Cracking

	Structure	1		2		3		4	
	Distribution	1	2	1	2	1	2	1	2
Trigger	Repair	Year Triggered							
2	Crack Seal	15.00	---	---	---	14.92	---	---	---
4	Crack Seal	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	Mill/Ovly/FC	7.17	11.08	12.33	16.92	6.67	9.83	11.92	17.83
16	PPEST	15.00	---	---	---	14.92	---	---	---
18	PPEST	15.00	---	---	---	14.92	---	---	---
20	Chip Seal	7.17	11.08	12.33	16.92	6.67	9.83	11.92	17.83
25	SAMI	11.50	12.92	12.83	14.00	11.42	12.83	13.17	14.50
26	SAMI	15.00	---	---	---	14.92	---	---	---
29	SAMI	15.00	---	---	---	14.92	---	---	---
30	Mill/Ovly/FC	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
31	Mill/Ovly	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
32	Mill/Ovly	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
33	PPEST	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
34	PPEST	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
35	ChipSeal	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
37	SAMI	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

The individual repair alternative costs listed below are provided by RIDOT's pavement management department [16, 17]. These numbers were based on cost information from 1998-2013[16,17].. Crack Seal stayed as the top cost effective alternative across all of the structures and distributions. Chip seal was now in the top three alternatives eight times instead of the previous four times and this includes one of the new chip seal triggers. SAMI was cost effective four times; same as when the analysis was completed without the alligator cracking. Following a similar trend as SAMI, PPEST was in the top three cost efficient alternatives twice again. The most noticeable change with the addition of alligator cracking, shown in Table A.6, is that mill and overlay with a friction course was cost effective once for Structure 2 with the first distribution.

- Crack Seal-$4,500 per lane-mile
- Mill & Overlay-$105,000 per lane-mile
- Mill & Overlay with a friction course-$142,000 per lane-mile
- PPEST-$86,000 per lane-mile
- Rubberized Chip Seal-$41,000 per lane-mile
- Stress Absorbing Membrane Interlayer (SAMI)-$125,000 per lane-mile

Table A.6 Individual Repair Alternatives Cost Effectiveness with Alligator Cracking

	Structure	1				2			
Distribution/Range		1-Min	1-Max	2-Min	2-Max	1-Min	1-Max	2-Min	2-Max
Trigger	Repair	% of Reconstruction Cost							
2	Crack Seal	1.27	2.54	---	---	---	---	---	---
4	Crack Seal	3.81	12.71	3.81	12.71	3.81	12.71	3.81	12.71
10	Mill/Ovly/FC	40.55	81.10	81.10	162.19	40.55	40.55	40.55	40.55
16	PPEST	24.64	75.41	---	---	---	---	---	---
18	PPEST	24.64	75.41	---	---	---	---	---	---
20	Chip Seal	11.64	46.54	23.27	69.82	11.64	23.27	11.64	11.64
25	SAMI	35.67	35.67	71.33	142.66	35.67	64.37	35.67	35.67
26	SAMI	35.67	64.37	---	---	---	---	---	---
29	SAMI	35.67	64.37	---	---	---	---	---	---
30	Mill/Ovly/FC	81.10	162.19	81.10	162.19	81.10	162.19	81.10	162.19
31	Mill/Ovly	59.97	119.93	59.97	119.93	59.97	119.93	59.97	119.93
32	Mill/Ovly	59.97	119.93	59.97	119.93	59.97	119.93	59.97	119.93
33	PPEST	73.91	123.19	73.91	123.19	73.91	123.19	73.91	123.19
34	PPEST	73.91	123.19	73.91	123.19	73.91	123.19	73.91	123.19
35	ChipSeal	23.27	69.82	23.27	69.82	23.27	69.82	23.27	69.82
37	SAMI	71.33	142.66	71.33	142.66	71.33	142.66	71.33	142.66
	Structure	3				4			
	Distribution	1-Min	1-Max	2-Min	2-Max	1-Min	1-Max	2-Min	2-Max
Trigger	Repair	% of Reconstruction Cost							
2	Crack Seal	1.27	2.54	---	---	---	---	---	---
4	Crack Seal	3.81	12.71	3.81	12.71	3.81	12.71	3.81	12.71
10	Mill/Ovly/FC	40.55	81.10	81.10	162.19	40.55	40.55	40.55	40.55
16	PPEST	24.64	24.64	---	---	---	---	---	---
18	PPEST	24.64	24.64	---	---	---	---	---	---
20	Chip Seal	11.64	46.54	23.27	69.82	11.64	23.27	11.64	11.64
25	SAMI	35.67	35.67	71.33	142.66	35.67	35.67	35.67	35.67
26	SAMI	35.67	35.67	---	---	---	---	---	---
29	SAMI	35.67	35.67	---	---	---	---	---	---
30	Mill/Ovly/FC	81.10	162.19	81.10	162.19	81.10	162.19	81.10	162.19
31	Mill/Ovly	59.97	119.93	59.97	119.93	59.97	119.93	59.97	119.93
32	Mill/Ovly	59.97	119.93	59.97	119.93	59.97	119.93	59.97	119.93
33	PPEST	73.91	123.19	73.91	123.19	73.91	123.19	73.91	123.19
34	PPEST	73.91	123.19	73.91	123.19	73.91	123.19	73.91	123.19
35	ChipSeal	23.27	69.82	23.27	69.82	23.27	69.82	23.27	69.82
37	SAMI	71.33	142.66	71.33	142.66	71.33	142.66	71.33	142.66

The only change in the overall benefit ratings, shown below in Table A.7, was that the timing factor was higher as all of the triggers were activated later in the design life as mentioned above.

Table A.7 Overall Benefit Ratings

	Structure		1		2		3		4	
	Distribution		1	2	1	2	1	2	1	2
Trigger	Repair	Cost $/Ln,mi.	Total Benefit							
2	Crack Seal	4,450	17.5	---	---	---	17.5	---	---	---
4	Crack Seal		10.0	10.0	10.0	10.0	10.0	10.0	10.0	10.0
10	Mill/Ovly/FC	142,000	33.6	35.5	36.2	38.5	33.3	34.9	36.0	38.9
16	PPEST	86,280	27.5	---	---	---	27.5	---	---	---
18	PPEST		27.5	---	---	---	27.5	---	---	---
20	Chip Seal	40,750	23.6	25.5	26.2	28.5	23.3	24.9	26.0	28.9
25	SAMI	124,900	35.8	36.5	36.4	37.0	35.7	36.4	36.6	37.3
26	SAMI		37.5	---	---	---	37.5	---	---	---
29	SAMI		37.5	---	---	---	37.5	---	---	---
30	Mill/Ovly/FC	142,000	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
31	Mill/Ovly	105,000	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
32	Mill/Ovly	105,000	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0
33	PPEST	86,280	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
34	PPEST	86,280	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
35	ChipSeal	40,750	20.0	20.0	20.0	20.0	20.0	20.0	20.0	20.0
37	SAMI	124,900	30.0	30.0	30.0	30.0	30.0	30.0	30.0	30.0

The updated ICB figures are shown below in Figure A.1 thru Figure A.8. The ICB analysis results are shown below in Table A.8. The same three alternatives, crack seal, chip seal, and SAMI were all still cost-efficient alternatives. The major change due to the addition of alligator cracking was that mill and overlay with a friction course was included in the cost-efficient alternatives. Even though this alternative was included in the most efficient alternatives, overall the benefit added was just above zero but the added cost between mill and overlay with a friction course and the next cheapest alternative, SAMI, is $17,100. Since the added benefit is nearly zero, mill and overlay with a friction course is barely made it to the top for efficiency.

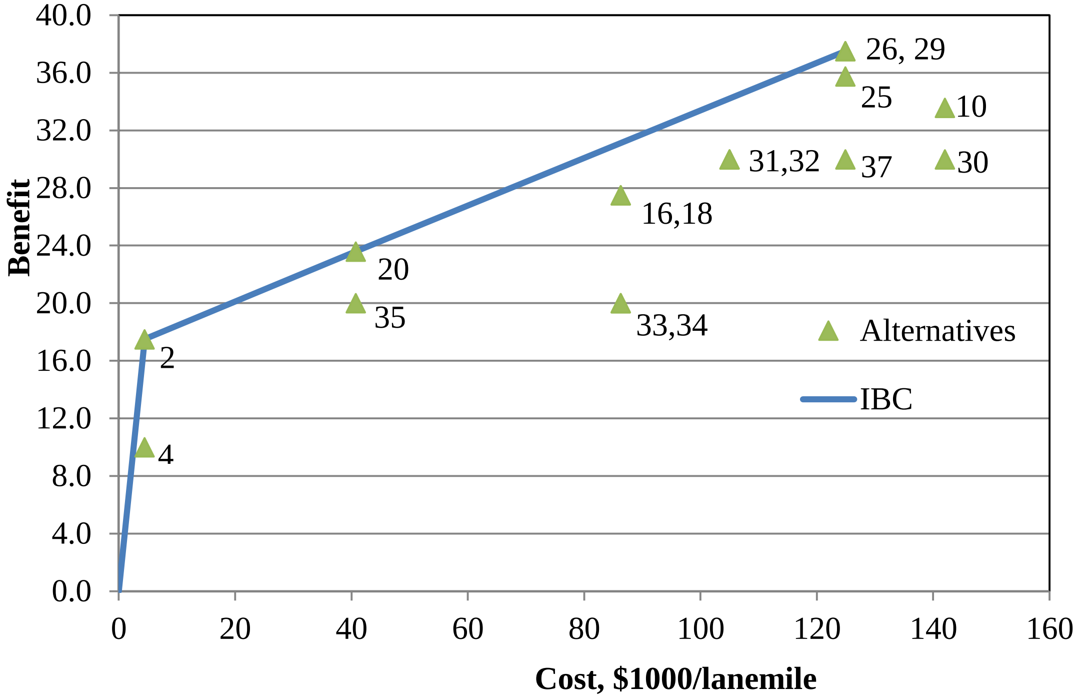

Figure A.1 Incremental Benefit Cost Graph for Structure 1 Distribution 1

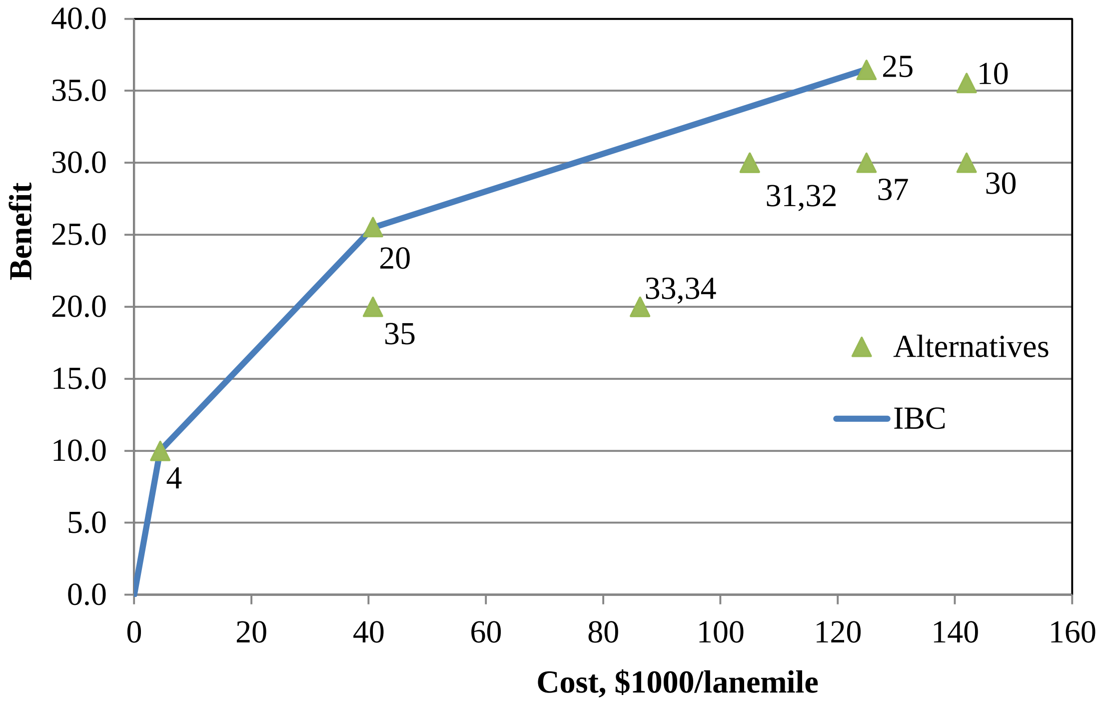

Figure A.2 Incremental Benefit Cost Graph for Structure 1 Distribution 2

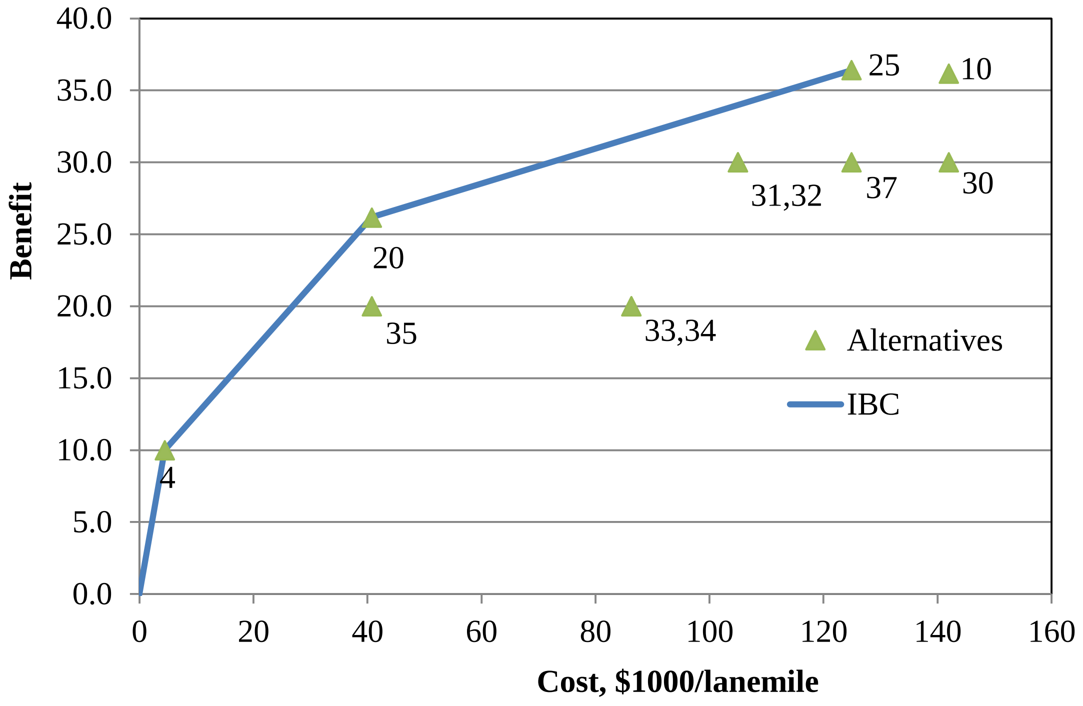

Figure A.3 Incremental Benefit Cost Graph for Structure 2 Distribution 1

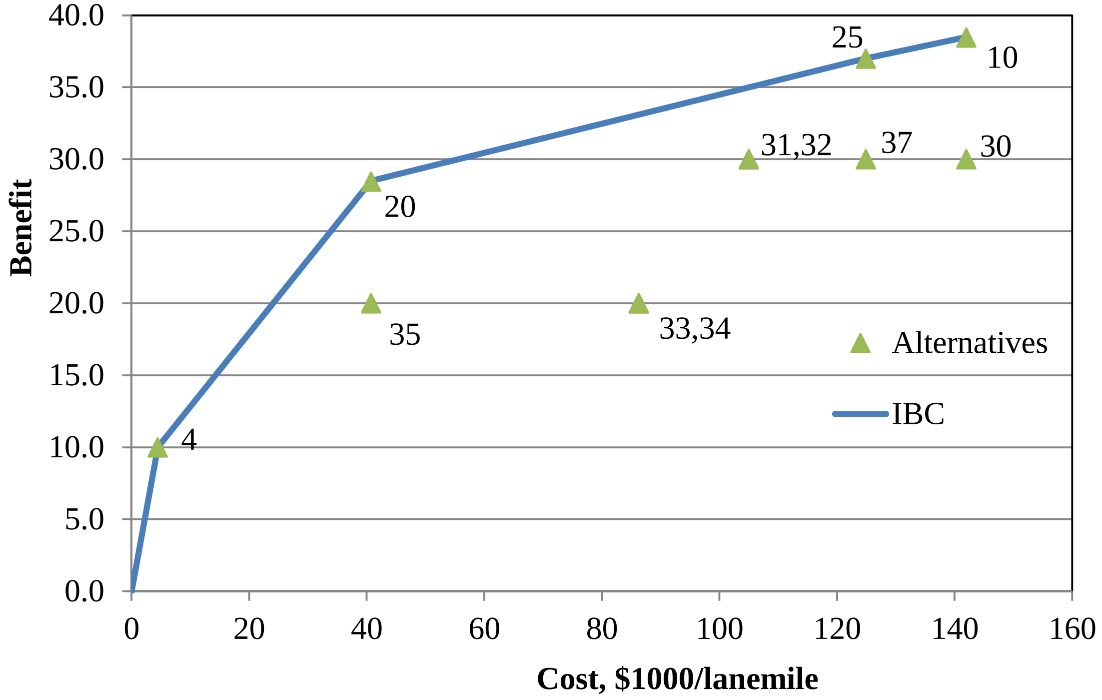

Figure A.4 Incremental Benefit Cost Graph for Structure 2 Distribution 2

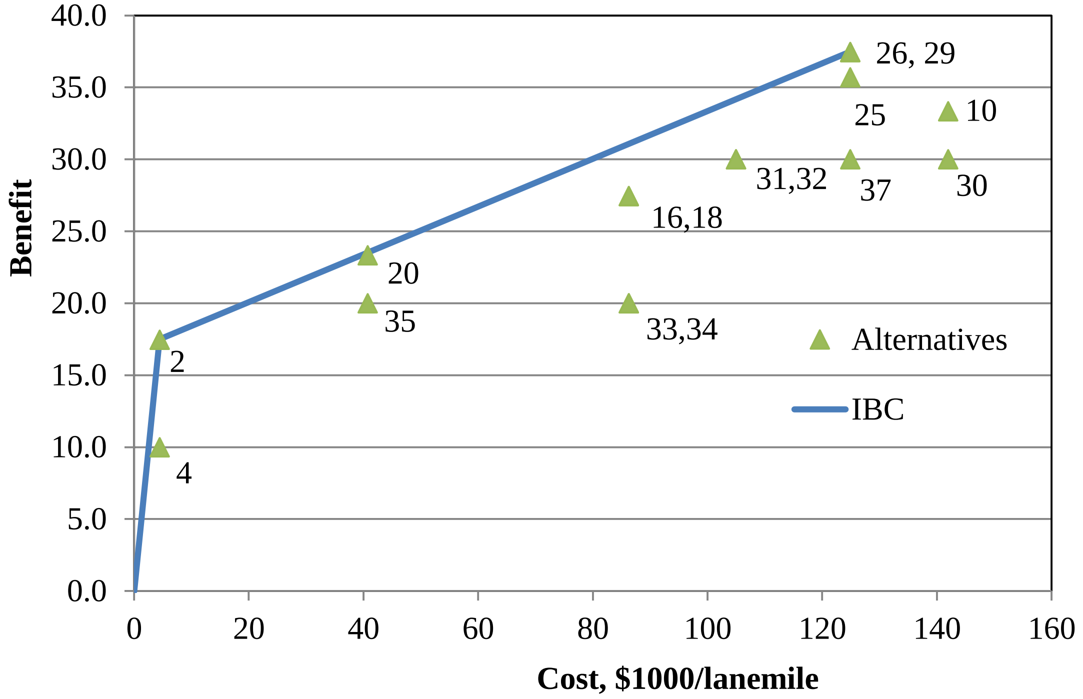

Figure A.5 Incremental Benefit Cost Graph for Structure 3 Distribution 1

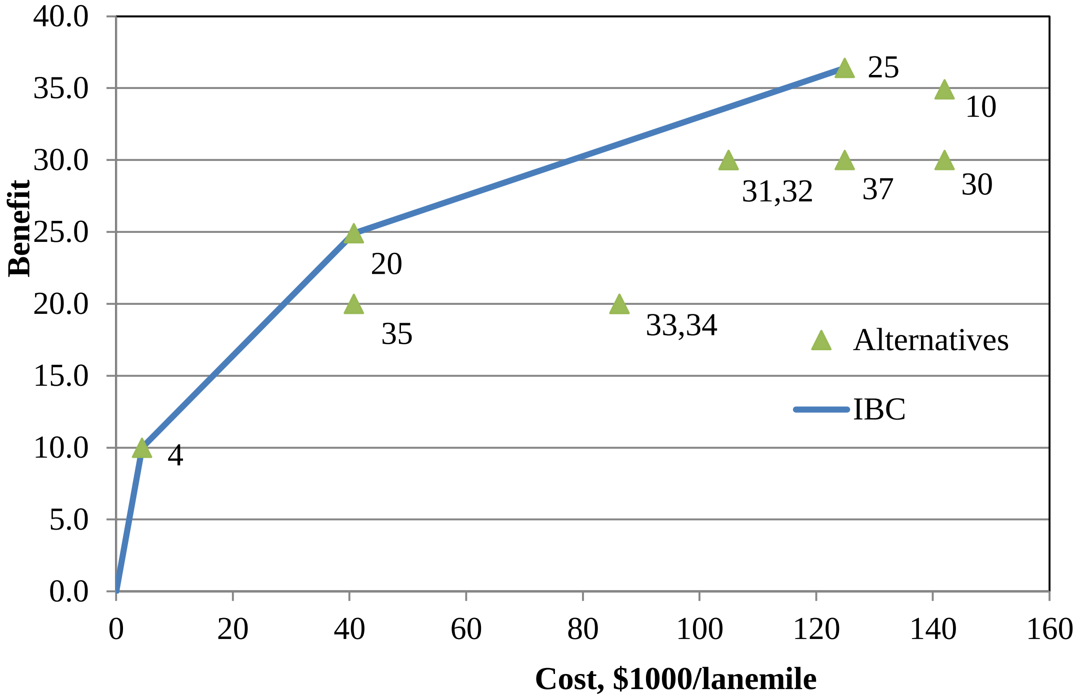

Figure A.6 Incremental Benefit Cost Graph for Structure 3 Distribution 2

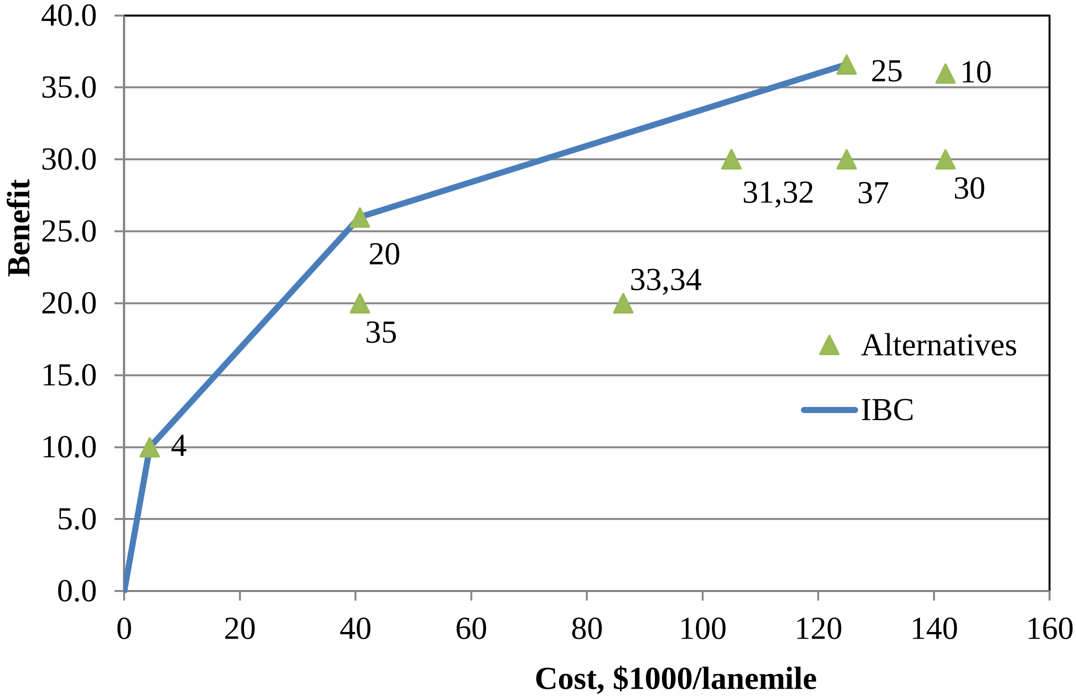

Figure A.7 Incremental Benefit Cost Graph for Structure 4 Distribution 1

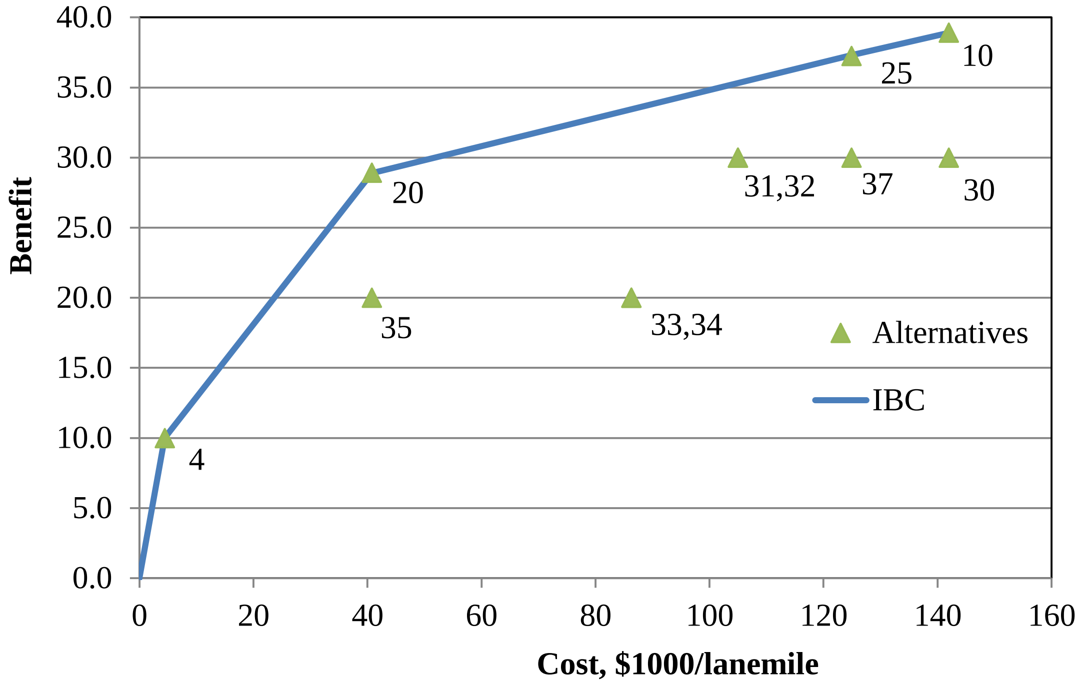

Figure A.8 Incremental Benefit Cost Graph for Structure 4 Distribution 2

Table A.8 Incremental Cost Benefit Efficient Alternatives with Alligator Cracking

	Structure	1		2		3		4	
	Distribution	1	2	1	2	1	2	1	2
Trigger	Repair	Cost Efficient Alternatives							
2	Crack Seal	X	---	---	---	X	---	---	---
4	Crack Seal		X	X	X		X	X	X
10	Mill/Ovly/FC				X				X
16	PPEST		---	---	---		---	---	---
18	PPEST		---	---	---		---	---	---
20	Chip Seal	X	X	X	X		X	X	X
25	SAMI		X	X	X		X	X	X
26	SAMI	X	---	---	---	X	---	---	---
29	SAMI	X	---	---	---	X	---	---	---
30	Mill/Ovly/FC								
31	Mill/Ovly								
32	Mill/Ovly								
33	PPEST								
34	PPEST								
35	ChipSeal								
37	SAMI								