8-2018

A Framework for Comparative Life-Cycle Evaluation of Alternative Pavement Types

Saeed Alqadhi
Purdue University

A FRAMEWORK FOR COMPARATIVE LIFE-CYCLE EVALUATION OF ALTERNATIVE PAVEMENT TYPES

by

Saeed Alqadhi

A Dissertation

Submitted to the Faculty of Purdue University

In Partial Fulfillment of the Requirements for the degree of

Doctor of Philosophy

Lyles School of Civil Engineering

West Lafayette, Indiana

August 2018

THE PURDUE UNIVERSITY GRADUATE SCHOOL
STATEMENT OF COMMITTEE APPROVAL

Dr. Samuel Labi, Chair

Lyles School of Civil Engineering

Dr. Kumares C. Sinha

Lyles School of Civil Engineering

Dr. John E. Haddock

Lyles School of Civil Engineering

Dr. William Crossley

School of Aeronautics and Astronautics

Approved by:

Dr. Dulcy Abraham

Head of the Graduate Program

Dedicated to

My parents, Dhafer and Fawzya

My wife, Fatimah

and

My children, Jana, Abdulaziz, and Hala

ACKNOWLEDGMENTS

First and foremost, I am thankful to Allah the Ever-Living, the Sustainer of all existence, for giving me the strength and patience to complete my doctoral research.

I am sincerely grateful to my major advisor, Professor Samuel Labi, for his unconditional support and guidance throughout my studies. His scholarly contributions to my research were invaluable and his words of encouragement helped me persevere. I consider myself fortunate to have had the opportunity to work with someone of such outstanding knowledge, modesty, and kindness. I also give special thanks to Professor Kumares C. Sinha for the insightful and refreshingly unconventional ideas he shared with me whenever I sought his advice. In addition, I am very grateful to Professor John E. Haddock and Professor William Crossley for their significant contributions to my research and for the valuable courses I took with them at Purdue.

To my parents, Dhafer and Fawzya, I am eternally grateful for all of the emotional and financial support you have given me throughout my life. I would not have been able to accomplish what I have without your guidance and prayers. To my mother, your strength and your gentle soul taught me that anything can be accomplished with persistence and hard work. To my father, thank you for always believing in me and inspiring me to pursue a doctorate in engineering. I have done this for you, and I hope I can make you proud. I am especially grateful to my wife Fatimah and my children, Jana, Abdulaziz, and Hala, who have been supportive in every possible way. Thank you for your patience and forbearance during my years of study at Purdue.

Finally, I owe a great debt of gratitude to King Khalid University, my sponsor, for the financial support I received during my studies at Purdue.

.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

ABSTRACT

Author: Alqadhi, Saeed, D. PhD
Institution: Purdue University
Degree Received: August 2018
Title: A Framework for Comparative Life-Cycle Evaluation of Alternative Pavement Types
Major Professor: Samuel Labi

Researchers and practitioners agree that the selection of an appropriate pavement surface material type should be made based on a comprehensive evaluation that incorporates the costs and benefits associated with each alternative for the stakeholder. The most appropriate material type generally is the most cost-effective alternative over the pavement life cycle. Hypothetically, the most appropriate material type will vary across the various geographical regions of the U.S. because material costs and performance are influenced by the deterioration agents at play and the construction costs in a region. To address this issue, this dissertation proposes a comprehensive methodology to identify the most appropriate choice of pavement material type under different climatic and traffic conditions and thereby establish the conditions under which any one of two pavement materials can be considered superior. The case study of this dissertation uses data for an interstate highway section from the Long-Term Pavement Performance (LTPP) program database. The stakeholder costs include the agency cost, the user cost, and the community cost. The benefits (effectiveness) were evaluated using the concept of an area bounded by a performance curve and a pre-determined threshold. For each of the four LTPP zones and the two material types, the optimal maintenance and rehabilitation (M&R) schedule was established, and the corresponding optimal life cycle cost-effectiveness was determined using both deterministic and probabilistic sensitivity analysis. The results using the former approach suggest that the most cost-effective pavement material types in wet climates and dry climates are rigid and flexible, respectively, irrespective of the discount rate. When the latter approach was used, the flexible pavement material was found to be the stochastically-dominant pavement material type irrespective of the climatic zone or discount rate. This framework can be scaled down to a state or scaled up to the national or continental level, given the availability of cost, traffic loading, pavement condition, and environmental datasets.

CHAPTER 1. INTRODUCTION

1.1. Study Background

During new construction, major rehabilitation, or reconstruction of pavements, highway agencies often need to decide between two major surface material types for the new pavement: asphalt concrete (AC) and Portland cement concrete (PCC). In making this decision, agencies consider a variety of factors, including the availability of raw materials, the frequency of future work zones, constructability and maintainability, initial costs incurred by the agency, future costs of maintenance, and so on. Some of these criteria can be encapsulated within an economic efficiency framework. While some agencies have traditionally made decisions based on the initial agency cost alone, most have realized that such decisions are not truly optimal and thus are seeking long-term solutions that are more sustainable over a highway pavement's life cycle. Thus, there is a growing need for more robust highway investment evaluation techniques. In situations where all of the evaluation factors can be monetized, the economic concept of life cycle cost analysis (LCCA) is a promising evaluation technique for identifying optimal choices among competing alternatives (Darter et al., 1985; Rangaraju et al., 2005; Reigle et al., 2002; Walls and Smith, 1998).

A large majority (over 94%) of paved roads in the U.S. have an AC surface (NAPA, 2018), possibly due to the ease of construction and maintenance or because AC is considered to have lower initial costs than PCC (Embacher et al., 2001). The latter supposition is particularly relevant since many agencies still make decisions based on initial cost only. From a technical perspective, the superiority of one material type over another is largely ambiguous. Some claim that PCC is more cost-effective in the long term compared to hot mix asphalt (HMA). This claim might be based on the fact that PCC has a relatively longer service life of 30 years (in general) compared to HMA's 20 years (Labi et al., 2003), and requires less frequent maintenance and rehabilitation (M&R). The initial construction costs of PCC are generally higher than those of HMA, which might lead decision-makers in some transportation agencies to prefer HMA pavement (Sasraku-Neequaye et al., 2017). Sometimes when a PCC pavement deteriorates, particularly one built with jointed concrete, the entire concrete slab at the defect location must be replaced. A few studies assert that PCC pavements require less intense or frequent maintenance over their life cycles, but the evidence has been mixed (Wimsatt et al., 2009). For example, in certain instances where jointed

PCC patching is performed, the entire concrete slab must be replaced, which results in a treatment associated with high costs.

The M&R treatments received by a rigid or flexible pavement over its life cycle influences its life cycle cost and therefore influences the choice of one surface material type over another. This life cycle schedule may be based on existing practice or optimal practice. The optimal scheduling of M&R treatments yields better cost-effective outcomes compared to the traditional scheduling method (Al-Mansour and Sinha, 1994; Kuennen, 2005; O'Brien, 1989).

1.2. Problem Statement

AC (flexible) and PCC (rigid) pavements are the typical material choices for pavement construction, maintenance, and rehabilitation. In the U.S., the roads are categorized as follows: 63% flexible pavement, 4% rigid pavement, and 33% unpaved roads, (e.g., dirt, gravel, or aggregate) (WAPA, 2018). The focus of this dissertation is paved roads, both flexible and rigid pavement. Issues including durability, traffic loadings, safety, fuel consumption, ease of construction, and maintenance often are considered when selecting the material. Some of these factors can be monetized, which facilitates their inclusion in economic assessments. Highway agencies often seek to keep their asset expenditures as low as possible in order to save money in the short term; however, pavement conditions can deteriorate rapidly, making the cost of maintenance higher in the long term. Highway users are affected by deteriorated pavements and incur higher user costs due to poor pavement conditions; and neighboring communities also may be affected by higher noise levels caused by contact between vehicle tires with pavement surfaces when the pavement is experiencing advanced deterioration. However, if the highway agency carries out frequent repairs and maintenance, users may experience greater comfort (smoother rides) despite the high cost of maintenance and the frequent inconveniences for road users. This strategy of frequent maintenance also can reduce noise pollution, but at the same time, it may have an adverse net impact on air quality since more maintenance treatments can lead to the release of more greenhouse gas emissions into the surrounding environment.

An optimal M&R schedule is one that achieves the maximum net benefit from the applied treatment(s). Choosing the most cost-effective pavement material can be justified by comparing the optimal (or typically by using state-of-the-practice) M&R profile associated with each pavement type. The choice of a pavement surface material is made not for the entire network but separately for each individual project, using case-by-case assessment studies.

The important factors affecting pavement design at the project level are traffic volume and axle loading, pavement service life, and temperature and precipitation rates. Although extensive research has been carried out on pavement M&R, several key issues have not been addressed:

1. Lack of a comprehensive stochastic LCCA and multi-criteria decision-making (MCDM) framework for choosing the most cost-effective pavement surface material considering the varying conditions of traffic and climatic regions.
2. Lack of project-level LCCA that includes assessment of the community costs along with the agency and user costs.
3. Lack of incorporation of the effects of the community costs or benefits on the optimal profiles of M&R activities, including full life cycle assessments of greenhouse gas emissions and energy consumption as well as noise.

1.3. Study Objective and Scope

The main objective of this dissertation was to develop a framework for investigating the conditions under which one pavement material type (flexible versus rigid) is superior to the other in terms of its overall life cycle cost-effectiveness. The objective was achieved by building an optimal life cycle M&R activity profile for a given material type in each climatic zone. The framework can help identify the most feasible pavement material type for a given set of conditions. The overall cost includes the agency costs of initial construction, maintenance, and rehabilitation; the user work zone costs of delay costs and vehicle operating costs (VOC) and the normal operations VOC costs; and the community costs of air and noise pollution. The two measures of effectiveness (MOEs) used are the pavement life and the area bounded by the pavement performance curve. Although this framework is applicable for any highway functional class, interstate highways are used in the case study of this dissertation. The initial pavement thickness, for both types of pavement materials, assumed in the case study are consistence with the Long-Term Pavement Performance (LTPP) interstates sections. However, this framework can be applied for any given pavement thickness.

1.4. Overview of the Study Approach

The flow of this dissertation is shown in Figure 1.1. First, the pavement type and material families were defined. The next step was to identify all of the possible M&R activities for the material type in question. The pavement performance models then were built and used to evaluate the effectiveness of the preservation treatments. Two different criteria were applied to evaluate the effectiveness of each treatment: the estimated life of the M&R treatment and the increased area bounded by the pavement performance curve due to the treatment. The life cycle costs (LCC) of each treatment then were estimated based on existing cost models for the agency, user, and community costs. The cost-effectiveness of all the candidate life cycle activity profiles were subsequently evaluated using monetized or non-monetized benefits and the optimal activity life cycle schedule was identified through the genetic algorithm (GA) optimization technique. Sensitivity analysis was then applied on the optimal M&R schedule using deterministic and probabilistic approaches, which was repeated for the other pavement material, as shown in Figure 1.1. For example, when the process started with a flexible material (material A in this case), then the other pavement material (material B) was a rigid pavement. The superior pavement material was selected after the sensitivity analysis was conducted for both pavement alternatives.

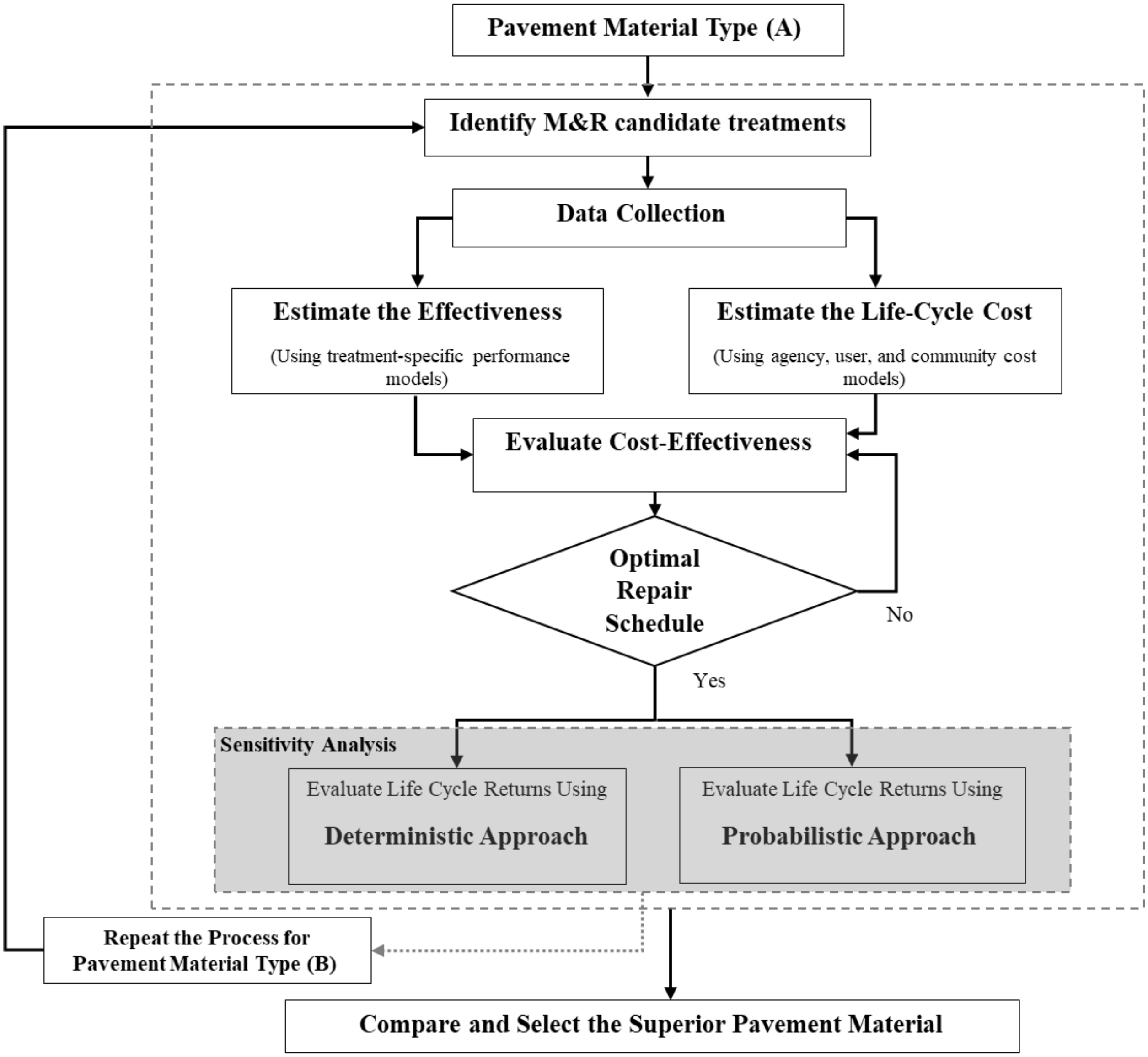


Figure 1.1: Study approach for the deterministic and probabilistic scenarios

1.5. Organization of the Dissertation

This dissertation consists of six chapters. Chapter 1 presented background information on the different pavement material types and provided the problem statement, scope, and objective of the research as well as an overview of the study approach. Chapter 2 summarizes the literature review, classifies different pavement material types and functional classes, and briefly describes the M&R treatments. Chapter 3 outlines the study framework and presents an algorithm developed to select the optimal M&R schedule. Chapter 4 presents the results of the pavement performance models for each M&R treatment in each LTPP region. The cost results also are presented in this chapter. In Chapter 5, the methodologies and techniques used to identify the optimal M&R schedule, along with the optimal M&R treatment schedules, are explained. This chapter also discusses probabilistic sensitivity analysis. Finally, the research summary and conclusions and future work and recommendations are presented in Chapter 6.

CHAPTER 2. DISCUSSION OF THE MATERIAL TYPE ALTERNATIVES

2.1. Introduction

Pavements are typically classified in two main categories based on their topmost surface material: (1) asphalt concrete pavement (or flexible pavement) (AC) and (2) Portland cement concrete pavement (or rigid pavement) (PCC). Pavements also are classified based on the functional class of the highway, and highways are further grouped on the basis of their locations (i.e., whether they are rural or urban or whether they function as interstates, freeways, arterial, collectors or local roads). The framework is general and can be used for roads of different functional classes; but for the case study in this dissertation, interstate was the functional class used.

2.2. Criteria for Establishing Pavement Families

Several methodologies exist for classifying pavements. Pavements can be classified on the basis of the material used for the wearing surface, the administrative jurisdiction, the design or construction type, the functional class of the roadway, and the climatic region where the pavement is located. Pavements have long been classified according to two types based on the dominant material used for the surface layer: flexible pavements and rigid pavements. The administrative classification is adopted for the purposes of denoting the level of government responsibility for a road. Classification of pavements based on design type is determined using the geometric features of the highway that are considered useful for highway design procedures. For the functional classification of highways, pavements are grouped by the type of service they provide. Several past studies have adopted the functional classification of highways according to the following NHS categories: Interstate highway, NHS non-Interstate highway, and non-NHS (AASHTO, 2011; Irfan et al., 2009; Labi & Sinha, 2005; Lamptey, 2004). The classification method used in this dissertation is based on the material used for the surface layer of the pavement (dominant material).

2.2.1. Comparing Pavement Surface Material Types: A General Discussion

Pavements in the U.S. can be classified based on the surface layer: unbound aggregate, asphalt concrete, Portland cement concrete, and composite pavement. The main function of pavements is to reduce the surface stress transferred by the tire point load or the circular uniform pressure to an acceptable level before it reaches the soil (subgrade) (Papagiannakis et al., 2008). The primary difference between flexible and rigid pavements is the way they distribute the load from the surface layer to the subgrade. Asphalt pavement distribute the traffic load throughout the pavement layers (surface, base, and subbase) and is called "flexible" because the entire pavement structure bends when a load is applied (BCE, 2017). On the other hand, rigid pavements carry and distribute the traffic load throughout a wider area of the subgrade. Rigid pavements react to traffic loads as a bridge over the subgrade, where most of the load is carried by the concrete slab (ACPA, 2018) while flexible pavements decrease the intensity of the traffic load as the pavement depth increases. The high modulus of the elasticity of rigid pavements, compared to flexible pavements modulus, allows the concrete slab to bear most of the traffic load (Papagiannakis, 2008).

The asphalt binder used in asphalt concrete pavement can be divided into two categories: natural asphalt and petroleum asphalt. In the case study of this dissertation, the latter is considered because it is more commonly used in the U.S. An asphalt binder is an organic material obtained from the fractional distillation of crude oil (Nagayach, 2015), The asphalt binder is a viscoelastic material that becomes brittle and vulnerable to cracking at low temperatures and plastic and vulnerable to shear-related failures at high temperatures (Brown, 2009). This susceptibility to temperature changes intensifies with high traffic loads, thus requiring more frequent maintenance. An asphalt binder is mixed with different sizes of aggregate to form asphalt concrete. For rigid pavement, the binder material is Portland cement, which is obtained by crushing specific types of stones containing hydraulic calcium silicates, calcium aluminate, calcium aluminoferrites, and calcium sulfate (gypsum) (Kosmatka et al., 2002). Portland cement hardens when it chemically reacts with water and forms a stone-like mass (paste).

The American Concrete Pavement Association (ACPA) argues that rigid pavement is preferable due not only to life cycle cost-effectiveness but also to enhanced safety, durability, and texture smoothness (ACPA, 1998). Safety is improved with rigid pavement because it is more visible (particularly at night), does not rut, and provides better skid resistance texture (Wimsatt, 2009). The notion that rigid pavements have longer lives is also supported by Labi & Sinha (2003) and Walls and Smith (1998). The fuel consumption of trucks driving on rigid pavement is reduced from 5% to 25% because of the absence of truck wheel deflection (Zaniewski, 1989). Also oil spillage on asphalt pavement is harmful in extremely hot weather because both asphalt and oil are oil-based materials, and oil spillage has a negligible effect on PCC, even in hot weather conditions.

The nighttime visibility of roadway markings is determined by the markings' ability to reflect light. Daytime visibility is related to the contrast between the roadway marking and the pavement surface. Pavement markings are more visible to the human eye when applied on asphalt pavements, which have a dark-colored surface. For PCC, pavement markings need to be applied on top of a black marking material to enhance visibility during the day, but it doubles the cost of the markings (Gates et al., 2003). For PCC pavements, the whitish-colored surface reflects the sunlight much better compared to AC pavements, thus helping to lower the outside air temperature (HIG, 2018). However, PCC pavement's whitish surface causes undesirable glare reflected from sunlight, which may affect humans during the daytime. According to FHWA, both AC and PCC can provide safe (including skid resistance), durable, and low-noise pavements when designed and constructed according to the prescribed standards (FHWA, 2005). There are different points of view on whether the flexible or the rigid pavement material is superior in terms of long-term performance. As the differences between pavement materials can be kept under control during the pavement design and construction processes, the cost-effectiveness of repairs over a pavement's remaining life becomes the main criterion for comparing the two pavement types. Rigid pavements are stiffer than flexible pavements, which can lead to lower deflection levels under the wheel path due to traffic loadings and therefore can provide surfaces with better rolling resistance (Lenngren, 2014). This would make the rigid pavement the superior material to use in terms of material properties, especially when heavy truck traffic is combined with roads with steep slopes, intersections, or major checkpoints.

2.2.1.1. Asphalt Concrete or Flexible Pavement

A flexible pavement is the mixture of an asphaltic (or bituminous) binder and aggregates and typically is formed in several layers (Figure 2.1) (AASHTO, 1993; INDOT, 2013). The pavement may exist as a bituminous surface treatment for low-volume roads or as an HMA surface for higher functional class highways or for roads with high traffic volumes (Papagiannakis, 2008). The major purposes of the top layer, known as the wearing surface, include the following: limiting the amount of moisture entering the pavement structure; providing a long-lasting friction layer throughout the pavement's life; achieving a smooth pavement surface; and providing structural support for all the pavement layers (INDOT, 2013). The soil, also called the subgrade, typically is compacted to achieve maximum density, and, in some cases, six to eight inches of the subgrade are scarified and blended with other materials such as lime, Portland cement, and fly ash to enhance their physical and engineering properties (Papagiannakis, 2008). A subbase layer is placed directly on top of the subgrade. This subbase layer is comprised of treated or untreated granular materials (typically crushed aggregate). In terms of bearing capacity, the subbase layer has better engineering properties and a higher resilience modulus than the subgrade material (AASHTO, 1993). The base layer is placed on top of the subbase layer and under the surface layer and typically consists of high-quality crushed stone aggregates. This layer may or may not be stabilized using different types of cementitious materials (Papagiannakis, 2008). The top layer, or the wearing surface, is made of asphalt concrete, which is a mixture of asphalt cement and crushed aggregates. The basic function of the surface layer is to protect the base layer from wheel abrasion and to act as a waterproof layer for the whole pavement structure. Furthermore, asphalt concrete provides a skid-resistant layer to help vehicles stop safely (AASHTO, 1993; Mannering et al., 2007). The load transformation applied on flexible pavements mainly is distributed through the pavement layers into the subgrade (Yoder et al., 1975).

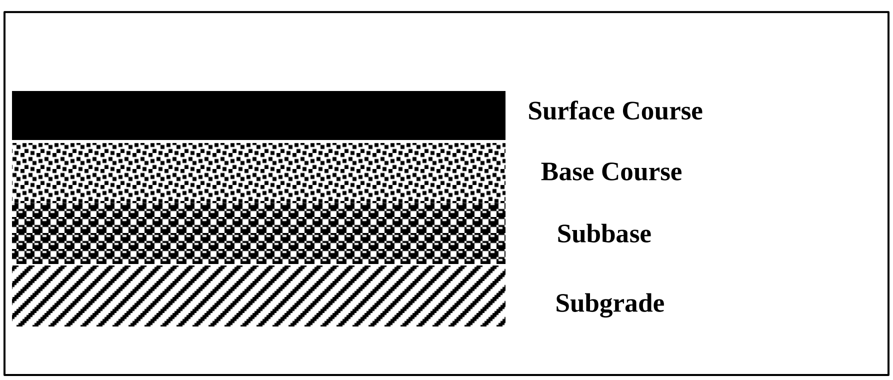


Figure 2.1: Flexible pavement typical cross-section

2.2.1.2. Portland Cement Concrete or Rigid Pavement

A rigid pavement consists of a wearing surface and a base layer placed on top of a compacted subgrade (Figure 2.2) (AASHTO, 1993; INDOT, 2013). The subbase consists of at least one layer of granular or stabilized material. The subbase is comprised of at least one layer of granular or stabilized material. This layer is optional only when the subgrade has strong physical components and good engineering properties (equivalent to the subbase quality) or the expected traffic loading does not exceed one million 18-kip ESAL's (Equivalent Single Axle Load) (AASHTO, 1993; Papagiannakis, 2008). The subgrade can be modified or stabilized, if necessary, by blending its material with a variety of cementitious materials. For rigid pavements, the traffic load is distributed over a wider area across the subgrade because the load is transferred by the bending action of the entire concrete slab. The load is distributed in this way because of the pavement's stiffness and high modulus of elasticity. There are different types of Portland cement concrete pavement, such as jointed plain concrete pavement (JPCP), jointed reinforced concrete pavement (JRCP), continuously reinforced concrete pavement (CRCP), and precast pre-stressed concrete pavement (PPCP) (ACPA, 1998). For JPCP, the joints are spaced so that the natural occurrence of concrete cracking can be controlled. Dowel bars are the load transfer device connecting two slabs at the contraction joints. The steel used in JRCP is not intended for structural support but rather to allow for a greater distance between two consecutive slabs for better control of concrete cracking. Well-designed and well-constructed CRCP is known to have good performance compared to other rigid pavement types. Its initial cost is relatively high compared to JPCP or JRCP because of the extra amount of steel required in CRCP. However, it has superior long-term effectiveness and is considered to be more cost-effective than traditional rigid pavement (ACPA, 1998; Plei, 2012). PPCP is a relatively new technology and has been found to have several benefits: reduction in user delays during construction, including shorter work zone durations; improvements in quality and performance; reduction in slab thickness; and extension of the construction season (Merritt et al., 2008).

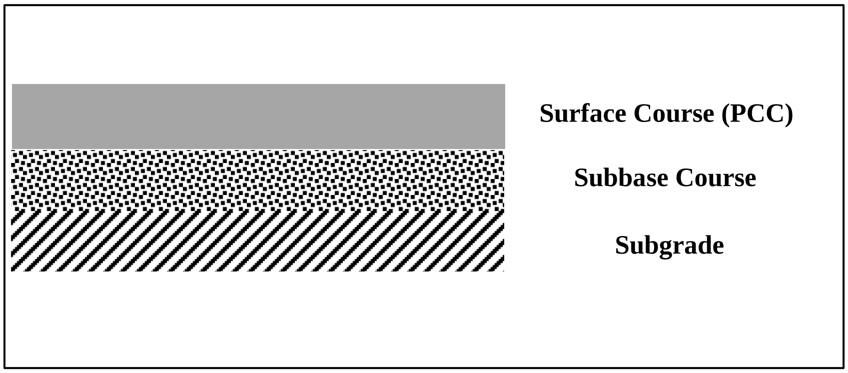


Figure 2.2: Rigid pavement typical cross-section

2.2.1.3. Composite Pavement

Pavements that combine flexible and rigid pavement are called composite pavements. The common practice for constructing composite pavements during the new construction phase involves placing AC on top of PCC. For highways with heavy traffic volumes, composite pavements often are considered a cost-effective alternative (Nunn, 2004). Composite pavement is also known as semi-rigid pavement in some countries, and it is widely used on highways that have high traffic volumes. This pavement may consist of AC on top of PCC (often called perpetual pavement) or PCC on top of AC, although the latter is rarely used for new construction because of its high initial cost. In Europe, composite pavements have shown potential to be a cost-effective pavement alternative for highways with heavy traffic volumes (Flintsch et al., 2013). Although some may argue that composite pavement should be categorized in a separate family of pavements, in this dissertation it is considered to be part of the rigid pavement family, especially composite pavement made of AC overlays on top of existing PCC pavement. This assumption is consistent with the methodology commonly found in LTPP studies, which tend to assume that AC overlays applied on PCC pavements are best understood as rigid pavement rehabilitation treatments.

2.3. Treatments Options for Each Material Type

Several M&R treatments can be applied to flexible and rigid pavements to enhance their functional and structural performance. In this section, some of the most common treatments for each pavement material are presented.

2.3.1. Flexible Pavement

The M&R treatments presented in the following sections have been prescribed by FHWA, and some were specifically chosen for the LTPP program. The specific pavement study (SPS) designated SPS-3 was developed to measure the performance of different flexible pavements that had received maintenance treatments in comparison to untreated control sections. SPS-5 was conducted to compare the performance of different flexible pavement rehabilitation treatments (Hall et al., 2002).

2.3.1.1. New Construction of Flexible Pavement

New construction (or reconstruction) of asphalt pavement is typically required when the pavement reaches a point where no other treatments (maintenance or rehabilitation) are cost-effective. It is expected that the structural properties of the underlying layers of the pavement surface worsen after several decades of accumulated heavy traffic loadings. The treatment needed in such cases is full-depth HMA concrete reconstruction, where all the layers of the old pavement are removed and replaced with an HMA surface course. This surface layer is placed over a new base course (stabilized or not), which then is placed on top of a new subbase layer consisting of crushed stone (WSDOT, 2012).

2.3.1.2. Pavement Maintenance Treatment Options for Flexible Pavement

Maintenance is defined as the use of methods and techniques to preserve pavement condition and improve safety and ride quality (FHWA, 2016; Hall, 2002). Maintenance activities can be classified as preventive or corrective. Preventive maintenance treatments protect pavements and delay deterioration. Corrective maintenance treatments fix pavement distress (Brown, 2009; Hicks et al., 1999). The following are some of the common preventive and corrective maintenance options:

a) **Crack Sealing** involves cleaning a crack and filling it with sealant to prevent water from penetrating into the pavement structure (Zaniewski et al., 1996). It is a periodic treatment suitable for several crack types, such as longitudinal cracks, transverse cracks, reflection cracks, edge cracks, and block cracks. Crack sealing is not commonly applied to fatigue (or alligator) cracks; for such cracks, seal coating or overlays are more suitable (Brown, 2009). Typically, crack sealing is applied on roads that carry any traffic volume, as long as adequate traffic controls are provided (INDOT, 2013).

b) **Fog Sealing** is the application of bituminous material on the surface layer of the pavement to restore the asphaltic qualities of the AC pavement (Zaniewski, 1996). It is applied to the pavement surface to improve waterproofing and to prevent stone loss due to raveling (INDOT, 2013). The main benefit of fog sealing is that it is a low-cost surface treatment that helps postpone the need for a major surface treatment for one or two years if applied on a structurally-sound pavement. A primary concern regarding fog sealing is avoiding excess application of the asphalt cement, which can cause loss of friction between the road surface and the tire (Brown, 2009).

c) **Chip Sealing** is a single or multiple applications of an asphaltic binder covered by single-sized aggregate over a pavement surface, or, in some cases, over a base course layer (Brown, 2009; Zaniewski, 1996). This treatment is not recommended for pavement with structural deficiencies, but for pavement without such deficiencies, it can improve the friction of the wearing surface and provide waterproofing (INDOT, 2013).

d) **Thin Cold Seal** is a mixture (normally mixed at the job site in specifically designed construction trucks) of emulsified asphalt, water, well-graded fine aggregate, and mineral fillers. Two major examples of thin cold seals are slurry seal and micro-surfacing (Zaniewski, 1996). Slurry seal is used to fill surface cracks and to provide a thin surface to improve skid resistance. Its layer thickness is approximately equal to the maximum aggregate size (the maximum thickness is slightly greater than ¼ in.) (Brown, 2009). Micro-surfacing, which provides a new wearing surface to reduce the oxidization of asphalt pavements and to fill ruts (INDOT, 2013), is recommended for reducing roughness on pavements with an international roughness index (IRI) value of 130 or less, given that the pavement is in good structural condition.

e) **Patching** is one of the most common treatments for repairing potholes and other localized distresses, such as fatigue (or alligator) cracking. There are two types of patching: partial depth and full depth. Partial-depth patching is applied by removing the surface layer and replacing it with new asphalt concrete. Full-depth patching involves the removal of all the underlying layers of the pavement (surface, base, and subbase) down to the subbase layer (Brown, 2009).

f) **Thin Overlay** is applied by profile milling the old asphalt surface and overlaying the new asphalt layer at a thickness usually not exceeding 1.6 inches (sometimes reaching 1.8 inches). This treatment is applied on top of old pavement having minor defects, but it is not for pavement having significant potholes (INDOT, 2013). It is a common treatment for reducing pavement surface roughness, delaying pavement deterioration, improving pavement friction, and enhancing structural capability by increasing pavement thickness (Brown, 2009).

In the SPS-3 study, four flexible pavement treatments were applied and compared to an untreated control section. These experiments were carried out at 81 sites across the U.S. and Canada between 1990 and 1991. The preventive maintenance treatments studied for the SPS-3 experiment included crack sealing, chip seals, slurry seal, and thin HMA overlay, which are listed in Table 2.1. Other recent preventive maintenance treatments, such as micro-surfacing, were intentionally not considered (Morian et al., 1998).

The specifications of the material used for slurry, chip, and crack seals were the same in the four LTPP Program regions (Hall et al., 2002).

Table 2.1: SPS-3 experimental sections

Test Section Number	Treatment
310	Thin Overlay
320	Slurry Seal
330	Crack Seal
340	Control
350	Chip Seal

2.3.1.3. Rehabilitation Treatment Options for Flexible Pavement

Rehabilitation is defined as treatments applied to a pavement to enhance the functional or structural aspects of its condition, thus extending its service life (Hicks et al, 1999). Pavement rehabilitation is the recommended action when the condition of the pavement has reached a point at which preventive and corrective maintenance are no longer effective. Pavement rehabilitation is a method for enhancing the functional and structural features of pavements in terms of ride quality and pavement condition, which can considerably extending pavement service life (Hall et al., 2001). Pavement rehabilitation treatments include resurfacing, restoration, and rehabilitation (3R) (INDOT, 2013). Rehabilitation techniques vary based on the type of pavement distress and the local conditions of the pavement. There are several alternatives for rehabilitation, including hot/cold in-place recycling, full- or partial-depth repair, and asphalt overlays. This dissertation focuses on asphalt overlays due to the availability of the data for such treatment in the LTPP database. The overlay types are as follows:

a) **Functional Overlay:** A functional overlay is an HMA overlay applied to correct deficiencies such as high pavement roughness and low friction. It can be applied directly onto the existing pavement, with or without milling. The purpose of a functional overlay is to improve a highway’s serviceability, which deteriorates due to traffic and environmental conditions. The thickness of a functional overlay varies from one agency to another, but typically it can reach up to 3 inches with single or multiple courses (INDOT, 2013). On a structurally sound pavement, a functional overlay may restore the pavement smoothness to a like-new condition. A functional overlay may improve the structural capacity of the pavement to a degree, although this is not the primary goal of the treatment.

b) **Structural Overlay:** Additional structural support is required when a pavement becomes structurally deficient due to traffic loads that exceed the expected load in the initial structural design. Like functional overlays, structural overlays increase the service life of the pavement, provide a smooth ride to the motoring public, and improve the skid resistance of the pavement. The thickness of a structural overlay normally exceeds that of a functional overlay because of the former's need to structurally support the pavement. As such, structural overlays are expected to extend the service life of a pavement more than functional overlays. The design process for a structural overlay is based on evaluating the existing pavement using deflection analysis, component analysis, and mechanistic analysis. Deflection analysis measures and analyzes the pavement deflections with respect to traffic loading. Component analysis converts the thickness of all pavement layers to an equivalent HMA layer so that the required thickness then can be estimated using the relationship between subgrade strength, pavement structure, and traffic loading. Mechanistic analysis uses the properties of the pavement materials and damage criteria to assess pavement distresses such as rutting and alligator cracking (Brown, 2009).

One of the objectives of the LTPP studies was to evaluate or developing new techniques and strategies for flexible pavement rehabilitation. SPS-5 examined flexible pavement rehabilitation strategies and the factors affecting the performance of overlays. These factors included the extent of surface preparation, overlay thickness, overlay material, traffic, environment, location, and condition of existing pavement. The SPS-5 sites were distributed throughout the four LTPP climatic regions (wet freeze, wet non-freeze, dry freeze, and dry non-freeze zones). This collaborative project was directed by the Strategic Highway Research Program (SHRP), FHWA, and the U.S. and Canadian highway agencies (McGhee, 1994).

SPS-5 examined different structural features of rehabilitation, such as the type of the surface preparation before overlay, the type of material used (virgin asphalt mixture or recycled mixture), and the thickness of the overlay. In Table 2.2, the minimum surface preparation indicates that only patching was performed before the overlay. Intensive preparation indicates that two inches of the existing pavement were milled off and patched in order to correct localized deficiencies. According to the SPS-5 experimental plan, the recycled mixture was to contain 30 percent of the reclaimed asphalt pavement (RAP), which is the milled material from sections where intensive surface preparation was performed (Hall, 2002).

There were 18 SPS-5 experimental projects throughout the U.S. and Canada. The required length for the test sections was 500 feet (152.4 m) with a fine-sand subgrade and a minimum traffic loading of 85,000 equivalent single axle loads (ESALs) per year (Hall, 2002).

Table 2.2: SPS-5 experimental sections (Hall, 2002)

Experiment	**Pre-overlay Preparation**	**Overlay Mix. Type**	**Thickness (in)**
SPS-501	Control	-	0
SPS-502	Minimal	Recycled	2
SPS-503	Minimal	Recycled	5
SPS-504	Minimal	Virgin	5
SPS-505	Minimal	Virgin	2
SPS-506	Intensive	Virgin	2
SPS-507	Intensive	Virgin	5
SPS-508	Intensive	Recycled	5
SPS-509	Intensive	Recycled	2

2.3.2. Rigid Pavement

The treatments and techniques for t M&R of rigid pavements vary by state agency. This section discusses M&R strategies based on the most common practices approved by the FHWA and state transportation agencies. In addition, this section discusses the M&R methods used as part of the LTPP experiments.

2.3.2.1. New Construction of Rigid Pavement

New construction or reconstruction of rigid pavements is often warranted when the existing rigid pavement has reached the end of its service life. To construct a new rigid pavement, the old slab is removed and the new PCC surface layer then is placed directly on top of the base or subbase layer. In certain cases, the existing concrete slab is removed or rubblized to yield a new subgrade, and the new PCC layer then is applied on top of the subgrade. After the joints are established according to design standards, a sealing material is applied to the joints to prevent water penetration (WSDOT, 2012). Rigid pavements are typically constructed on interstates and other NHS roads where a high volume of truck traffic is expected. JPCP, the predominant type of rigid pavement used in most states was used in the LTPP studies. A drainage layer of crushed rock base or subbase is an important design feature of these pavements because poor drainage causes pumping problems and, consequently, subgrade failure (INDOT, 2013).

2.3.2.2. Maintenance Treatment Options for Rigid Pavements

Rigid pavements are considered to be more durable compared to flexible pavements and have been found to need fewer and less intense routine and preventive maintenance treatments (Wimsatt, 2009). Common maintenance treatments for rigid pavements include:

a) **Joint and Crack Sealing** is the process of cleaning and sealing cracks or joints on a rigid pavement surface slab. This treatment prevents moisture and debris from penetrating the pavement structure. The treatment typically is applied if 10% or more of the joints have loose, missing, or depressed sealants. This deterioration is more likely to happen to pavements older than ten years. In such cases, the affected joints should be sawed and the old joint seals removed and replaced with new seals (INDOT, 2013).

b) **Diamond Grinding** is the shallow-depth removal of the rigid pavement surface using diamond saw blades. This treatment is used to address faulting at joints and cracks, to eliminate bumps (Hall, 2002; Labi and Sinha, 2003), improve skid resistance, correct surface defects, and promote drainage. Typically, the joint seals are removed prior to or in conjunction with texture grinding operations.

c) **Grooving** is carried out using diamond saw blades but with more spacing between the blades than is necessary with diamond grinding. The main reason for this treatment is to improve the pavement friction under wet conditions and to increase the surface drainage capabilities of the rigid pavement. It is commonly applied on highway ramps or vertical and horizontal curves, especially when the cross slopes are improperly constructed (Hall, 2002).

d) **Partial-Depth Patching** is the repair of localized distressed patches to improve ride quality. Up to one-third of the concrete pavement surface depth is sawed and replaced with a normal or high-early-strength concrete mixture. Any unsound material is removed before applying the new concrete mixture (INDOT, 2013). Partial-depth patching can be used for localized distresses and as a part of pavement restoration or when preparing a pavement for an overlay (Hall, 2002).

e) **Full-Depth Patching** is the localized repair of the full width of a rigid pavement that is at least six feet long. This treatment is used for distresses related to structural deficiencies, defective materials, or construction problems (Hall, 2002). The purpose of the treatment is to improve ride quality and replace deteriorated surfaces and joints (INDOT, 2013). When quick opening to traffic is required, the material used for the fully patched area is a high-early-strength concrete mixture.

f) **Slab Replacement** is a common treatment for JPCP when individual slabs fail to support the traffic load while the rest are in reasonably good structural condition. Slab replacement is a cost-effective treatment when 90% of the pavement is in good condition (Bautista et al., 2008).

Other maintenance treatments can be used, such as load transfer restoration using load transfer devices such as dowel bars that are suitable for repairing wide cracks. Another treatment, slab stabilization, is used to fill the small voids underneath the concrete slab with flowable asphalt or concrete materials (ACPA, 1998; INDOT, 2013).

SPS-4 was a study of preventive maintenance conducted by the Strategic Highway Research Program (SHRP) under the LTPP Program. One of the purposes of this dissertation was to specify methods for finding the best time to apply the most effective maintenance treatments. Evaluating the effectiveness of individual treatments was another goal of the dissertation. The preventive maintenance treatments applied to the rigid pavements of the SPS-4 test sections included joint/crack sealing, undersealing (also known as sub-sealing or slab stabilization), surface grinding and grooving, partial-depth patching at joints/cracks, and full-depth patching at joints/cracks. A total of 31 SPS-4 sites in the U.S. and Canada were studied until 1991 (Hall et al., 2001; Morian et al., 1998).

2.3.2.3. Rehabilitation Treatment Options for Rigid Pavements

As is the case with flexible pavement rehabilitation, rigid pavement rehabilitation is needed when preventive maintenance is no longer a cost-effective option. Unlike pavement maintenance, rehabilitation results in a major improvement to the pavement structure. Pavements are expected to have better performance and longer service life when a rehabilitation treatment, rather than a maintenance treatment, is applied. Besides improving the structural support of the pavement, rehabilitation treatments provide a smoother ride and improve skid resistance, as do some of the preventive maintenance treatments, such as thin overlay. Different rehabilitation treatments are applied based on different pavement distresses. Some of the common treatments are presented in this section.

- **a) HMA Functional Overlays** are typically preceded by partial- or full-depth patching of the existing rigid pavement. HMA overlays on rigid pavement are designed and executed similarly to those used on flexible pavement. These treatments are used a wearing surfaces to improve ride quality and surface friction with no further structural support to the pavement structure (Hall et al., 2001; Irfan, 2010).
- b) **HMA Structural Overlays** are placed on top of the rigid pavement after either partial- or full-depth patching of the existing rigid pavement. The structural deficiency approach described in the 1993 AASHTO Design Guide is the most common design method for asphalt and concrete overlays. Structural HMA overlays are applied when a substantial increase in the structural capacity of the pavement is needed (Hall et al., 2001). The overlay thickness can be as much as six or eight inches (Irfan, Khurshid, & Labi, 2009).
- c) **Crack-and-Seat PCC Slab with HMA Overlay** is a common treatment for JPCP, which involves cracking the existing JPCP slab into smaller blocks (roughly three to five feet long by six feet wide) and overlaying the broken slab with asphalt concrete. The purpose of breaking the concrete slab into smaller pieces is to limit its vertical and horizontal movement. The new slab sizes, however, are large enough to maintain the slab’s minimal structural integrity (Irfan et al., 2012). Reflective cracking is a result of the horizontal strain caused by the thermal expansion of the concrete slab and the vertical strain on the slab due to frequent heavy traffic loading above the joints (CDOT, 2015).

d) **PCC Overlays (Bonded / Unbonded):** The two common PCC overlays are bonded and unbonded concrete overlays. Bonded concrete overlays require intensive preparation of the existing surface to ensure a strong bond between the existing and the new concrete slabs. The typical thickness of a bonded concrete overlay is four inches, which is considered relatively thin. PCC slurry and grout are the most common bonding materials (McGhee, 1994). This type of overlay is only occasionally used because it is designed to treat pavements in fair-to-good condition (ACPA, 1998; Hall et al., 2001). Unbonded concrete overlays are extremely thick PCC overlays (five to 12 inches) placed on top of an intermediate layer separating the old and new concrete slabs. The separation layer typically used is a thin concrete layer. The intentional separation of the two concrete layers allows them to act independently, and, as a result, the distresses on the old pavement slab are not reflected in the new one. Surface repair and preparation are minimal in this treatment, which makes it a good candidate for badly deteriorated rigid pavements (ACPA, 1998). Unbonded concrete overlay is considered a reasonable alternative to rigid pavement reconstruction when a shorter construction schedule is needed (Hall, 2001). Both types of PCC overlays provide additional structural support for the pavement and improve the wearing surface's functional properties.

e) **PCC Rubblization and HMA Overlay** involves breaking the concrete slab into small pieces so that the slab works as an aggregate base course with improved structural capacity and placing an HMA overlay over the new base. This is a typical treatment for preventing reflective cracking in newly-applied HMA layers (MNDOT, 2014). A survey of 38 U.S. states showed that rubblized PCC with HMA overlay provided superior performance compared to cracking-and-seating with HMA overlay in terms of reducing reflective cracking (Ksaibati et al., 1998). This treatment is not recommended if the subgrade requires substantial structural improvement (MNDOT, 2014).

SPS-6 was conducted to investigate the performance of different rehabilitation treatments on existing JPCP and JRCP (Table 2.3). Similar to other SPS studies, SPS-6 was implemented in the four different climatic regions of the U.S. and Canada on both fine- and course-grained subgrades (Ambroz et al., 2005). SPS-6 was conducted by SHRP under the LTPP Program.

Table 2.3: SPS-6 experimental sections

Experiment	Pre-overlay Preparation	Thickness of AC overlay (in)
SPS-601	Control	0
SPS-602	Minimal	0
SPS-603	Minimal	4
SPS-604	Minimal with saw and seal	4
SPS-605	Intensive	0
SPS-606	Intensive	4
SPS-607	Crack/break and seat	4
SPS-608	Crack/break and seat	8

All the rehabilitation overlays in the SPS-6 study were asphalt concrete overlays over existing rigid pavements. As shown in Table 2.3, SPS-6 had eight major experimental sections, with one control section (601) that received routine maintenance but not rehabilitation. Sections 602 and 605 both received no overlay and only received “minimal preparation” and “intensive preparation,” respectively. Sections 603 and 604 both received “minimal preparation” and had four-inch overlays, while section 604 had sawed and sealed joints. The only four-inch overlay that received intensive preparation was section 606. Finally, sections 607 and 608 were cracked or broken and seated with four and eight-inch overlays, respectively (Hall et al., 2002).

The routine maintenance applied on the control sections included joint and crack sealing and limited patching. Minimal preparation included crack repair and seals, limited patching, joint stabilization, and diamond grinding in severe faulting cases. These are typical preparation activities performed by highway agencies prior to placing overlays. Intensive preparations included sub-sealing, sub-drainage, joint repair and sealing, full-depth repair with restoration of load transfer, diamond grinding, and shoulder rehabilitation. Sections receiving AC overlays were not treated with diamond grinding or joint/crack sealing. Crack-and-seat or break-and-seat are mechanical techniques used to minimize reflective cracking. The cracking and seating process is used with JPCP, while the breaking and seating process is used with JRCP. The intent of these processes is to create full-depth hairline cracks on the PCC slabs and throughout the reinforcement, when present, to achieve full material separation. Sawing and sealing of section 604 was conducted directly above the existing joints and cracks of the rigid pavement (Ambroz, 2005).

2.4. Review of Previous Comparative Evaluation of Material Types

The common method for comparing the two typical pavement materials (asphalt and concrete) is Life Cycle Cost Analysis (LCCA). LCCA is defined by the Federal Highway Administration (FHWA) as an analysis technique based on a sound principle of economic analysis that is used to evaluate the overall economic efficiency between two or more competing alternatives in light of initial and discounted future agency, user, and other relevant costs (Walls and Smith, 1998). In recent years, the concept of LCCA has been used by highway agencies in their planning and budgeting processes to decide on future pavement treatments in terms of their cost-effectiveness (Darter et al, 1985). LCCA also has been used as a tool to evaluate proposed preservation treatments during planning and scheduling (Reigle, 2002). The outcomes of LCCA not only reveal which treatment is superior to the other, but it also clarifies how to implement the most cost-effective treatment scheduling for a given project (Rangaraju, 2005). Several performance indicators of LCCA can be used, including the net present value (NPV), equivalent uniform annual cost (EUAC), and internal rate of return (IRR) (Walls and Smith, 1998). According to the National Highway System Designation Act in 1995, LCCA is required to be conducted for (NHS) segments costing twenty-five million dollars or more. Although the Transportation Equity Act for the 21st Century (TEA-21) removed the requirement of conducting LCCA for transportation projects funded by the federal government, highway agencies still are encouraged to implement this technique for NHS projects.

LCCA can be implemented at two primary levels: the project level and the network level. The optimal profile of treatment activities is determined using project level analysis, but the allocation and availability of funds are typically not considered at this level of analysis (Ozbay et al., 2003; Tighe, 2001). All fund allocations and limitations along with their related polices are considered at the network level of LCCA (Ravirala et al., 2002; Zhang et al., 2012). This dissertation focuses on the LCCA technique at the project level since it aims to define the optimal profile of M&R activities. According to the FHWA, LCCA is conducted using the following steps: (1) establish an alternative pavement treatment plan for a pre-defined analysis period; (2)

determine M&R treatment scheduling; (3) estimate the cost incurred by the highway agency; (4) estimate the cost incurred by highway users; (5) establish the cash flow diagram for the alternative scenarios; (6) calculate the NPV; (7) analyze the results; and (8) evaluate alternative strategies (Walls and Smith, 1998). Further discussion about the methodology used in this dissertation is presented in Chapter 3.

Advancements in LCCA research in the last 15 years have resulted mainly from funding and encouragement from the FHWA and the National Highway System Designation Act of 1995 (Chan et al., 2008; Swei, 2012). A 2001 study compared asphalt and concrete pavements using LCCA on a low-volume road and concluded that, in most cases, PCC pavement is the most cost-effective pavement material when agency cost is the only cost component included (Embacher, 2001). Another study that compared the agency cost of flexible and rigid pavements yielded similar findings: the rigid pavement was found to be the most cost-effective paving material choice (Adow et al., 2011). The Asphalt Pavement Alliance (APA) presented a synthesis LCCA study in 2005 comparing HMA with PCC using historical agency cost data. The study examined interstate highway sections in Ohio, Kansas, and Iowa with similar traffic loadings and ages of the pavement alternatives. Although there were no signs that the study included the user costs, the construction M&R costs were included. The study concluded that the present worth of HMA is lower than that of PCC by 10% in the initial costs and by 25% in the rest-of-life costs (Villacres, 2005). A research conducted by APA using the LTPP database to estimate the service life of flexible pavements concluded that the median age was 17 years (Quintus et al., 2005). APA supported the use of LCCA as a decision-making tool for the phases of initial cost, maintenance, and rehabilitation, using NPV as the indicator. According to APA, several advantages are expected when flexible pavements are used, including low initial and rest-of-life costs, speed and flexibility of AC pavement construction, adoptability to a variety of traffic loading levels, long pavement life, and 100% recyclability (APA, 2004).

Another study was conducted to select the best pavement material for heavy vehicles based on the agency cost of constructing a specific roadway section for pre-determined loading (Uljarevic et al., 2016). This study started from the pavement design phase, using the same loading to design asphalt and concrete pavements. The results showed that rigid pavements cost 29% more than flexible pavements. In a study conducted to determine the effects of different traffic loading levels and different soil conditions on the pavement material selection process (Akakin et al., 1983), the authors concluded that rigid pavement was the more cost-effective alternative. According to Sullivan and Moss, (2014), rigid pavement is not only superior to HMA, but it is even more cost-effective than warm mix asphalt concrete when the initial cost of construction was the only cost component considered. Another study addressing the agency cost of flexible and rigid pavement material selections was conducted in India, where once again PCC pavement was found to be the more cost-effective choice (Mohod et al., 2016). The initial cost of asphalt concrete was found to be lower than the PCC alternative; however, ultimately the LCCA of PCC was found to be lower (Sasraku-Neequaye, 2017).

Wimsatt (2009) argued that rigid pavement lasts longer, providing a more cost-effective choice of pavement material. The American Concrete Pavement Association (ACPA) introduced an LCCA guide in 2002 to compare the pavement options based on the agency costs (construction, maintenance, rehabilitation, and salvage costs) and the user costs (delay, vehicle operation, and safety costs), using present worth (PW) and EUAC as the indicators. A comprehensive case study using these guidelines was conducted in several states, and the results suggested that rigid pavement lasted 1.6 to 2.6 times longer than flexible pavement and was found to be 14% to 250% more cost-effective than asphalt pavement (ACPA, 2002).

A study of the different LCCA approaches compared the Alabama Department of Transportation (ALDOT) method, the National Center for Asphalt Technology (NCAT) method, and the University of Alabama (UA) method (West et al., 2012). The three LCCA techniques were applied on six different interstate highway sections with different design strategies. The input entries for each method were consistent across the six highway interstate sections. Considering agency cost alone, NCAT found AC to be the most cost-effective pavement material. Only one interstate highway section considering rehabilitation cost alone was in favor of PCC; AC was the most cost-effective material for the other five highway sections. The UA methodology results found that three highway sections were in favor of AC, two were in favor of PCC (one of them had rehabilitation treatment only), and one was slightly in favor of PCC (only 5% lower than AC). It is worth mentioning that the UA method did not include the user costs in its calculation.

According to the result of the survey conducted by Wimsatt et al., (2009), 94% of agencies utilize the LCCA technique in their decision making process. About 60% of the responding states indicated that the user cost were not included in their analysis. Those who included user cost, or said that they would include it in future analyses, indicated that only travel time delay and VOC at work-zones would be included in user cost. Most LCCA studies were conducted at a project level where the choice of pavement material was made on a case by case basis; few studies included all of the cost and benefit components. The inclusion of agency and user cost, and community costs in a few cases, was more common when researchers were trying to reach the optimal schedule of treatment ativities. Some studies such as Swei (2012) considered the deterministic and stochastic approaches of the LCCA technique without considering the different cost components (agency, user, and community). This dissertation used a comprehensive methodology including all cost and benefit components (see Chapter 3).

2.5. Chapter Summary

In this chapter, pavements were classified based on their surface types. Generally, three different types of pavement were discussed: flexible, rigid, and composite. M&R treatments for both flexible and rigid pavements were identified and briefly discussed. LTPP Program M&R treatments also were discussed to lay the groundwork for further discussion of these treatments in later chapters. This discussion helped in the development of pavement performance models for each type of LTPP treatment and in the analysis of the effects of each individual treatment in the four LTPP climatic regions. It is therefore essential to identify pavement families and all possible M&R treatments in order to identify the optimal M&R schedule in each climatic region. Finally, a review of LCCA past studies that evaluated or selected the pavement material type were presented.

CHAPTER 3. STUDY METHODOLOGY AND FRAMEWORK

3.1. Introduction

This dissertation addresses this important question in pavement management: what is the most cost-effective pavement material (AC or PCC) and corresponding M&R schedule over the pavement's life-cycle? To address this research question, a systematic framework first was developed to incorporate all the factors that are expected to influence the optimal solution. The process started with a comprehensive review of the related literature followed by data collection, identification of pavement categories, definition of all the M&R candidate treatments, development of performance models for all treatments, identification of existing cost models for M&R activities, a cost-effectiveness analysis, and an LCCA using deterministic and stochastic approaches in conjunction with multi-criteria decision making tools. To ensure a consistent method of comparison between the two different pavement materials, an optimal profile for each pavement type needed to be developed. Therefore, the cost components in this dissertation included agency cost, work zone user costs (travel time and vehicle operating costs (VOC), and community costs (costs associated with both air and noise pollution). The first component of the effectiveness (benefit) analysis was the non-monetized effectiveness, which was evaluated by determining the area under the pavement performance curves. Monetized effectiveness, which addresses the agency and user cost savings, also was used in this dissertation to evaluate the effectiveness of various M&R treatments.

3.2. Data Collection

The LTPP dataset was one of the primary sources of the data used in this dissertation. The LTPP dataset includes pavement condition data, climatic data, operational characteristic data, traffic data, and others. The purpose of using the LTPP data was to develop a comprehensive dataset for all the M&R candidate treatments from different climatic zones to examine the effects of treatments under different climatic conditions. The data required to build the pavement performance models for the M&R treatments, using both flexible and rigid pavements, were found in the SPS 3, SPS 4, SPS 5, and SPS 6 studies' datasets, which were briefly discussed in Chapter 2. These data were related to the pavement condition, such as cracking, pavement roughness, faulting, spalling, and other types of pavement distress. Construction cost models based on the contract data available in the literature from the Indiana Department of Transportation (INDOT) were adopted in this dissertation for initial construction, maintenance, and rehabilitation activities. Duration models that were used to evaluate user costs (for travel time and VOC) also were adopted from existing research (Ahmed, 2012a; Irfan, 2010). Greenhouse gas emissions and energy consumption were estimated using the Athena Impact Estimator for Highways. Noise barrier costs were obtained from various past studies based on the barrier design, material, or height (Sinha and Labi, 2007).

3.3. Effectiveness Analysis

In preserving their highway pavements, agencies are motivated primarily by the need to ensure a certain minimum level of service for their customers. This minimum level of service may be understood to include duly correcting any structural or functional pavement deficiencies, reducing the rate of physical deterioration, enhancing user safety by improving pavement condition, and increasing pavement longevity. The realization of these objectives constitutes an essential goal of highway agencies and often is reflected in their mission statements. Therefore, the following questions are posed and addressed routinely by highway agencies: How many years can be added to a pavement's lifespan by carrying out a preservation treatment? To what extent can a maintenance or rehabilitation treatment improve the pavement condition?

In answering questions such as these, highway managers seek to measure objectively the degree to which an M&R activity accomplishes not only the specific objectives associated with a treatment but also the broader goals of the agency. The effectiveness of a certain maintenance or rehabilitation treatment can be assessed in the long term or the short term (R. Smith et al., 1993), both of which are useful in pavement management because they help agencies compare the benefits of alternative rehabilitation materials, procedures, and work sources (in-house versus contract). Methods showing long-term effectiveness include enhancing pavement life, upgrading the pavement condition, and reducing the costs of routine maintenance in the years after treatment. A long-term effectiveness evaluation is especially essential when engaging in long-term programming and planning; for example, information regarding a rehabilitation treatment's service life can help an agency estimate when the next reconstruction or major rehabilitation would be required, which enables the agency to create a reliable budget and schedule for future rehabilitation and reconstruction procedures.

This research focuses on the long-term effectiveness of M&R treatments. A measure of effectiveness (MOE) is regarded as a performance measure that can be used to evaluate the effectiveness of preventive M&R treatments. MOEs help elucidate the concerns not only of the agency but also of other highway pavement stakeholders (users and community members) (Sinha and Labi, 2018).

3.3.1. General Procedure for Assessing the Effectiveness of M&R

There are three fundamental questions that must be addressed related to evaluating M&R efficacy. (1) How must effectiveness be assessed and what performance indicators must be used? (2) On what basis can a maintenance or rehabilitation activity be considered effective? (3) When maintenance or rehabilitation treatments are regarded as effective, how can this success be translated and used to enhance the function of former pavements, considering the natural and operating environments and the M&R treatments required?

Based on these three sequential questions, the steps for evaluating effectiveness can be regarded as follows (Alqadhi et al., 2016; Sinha and Labi, 2018):

Step 1: Choose a suitable MOE for the maintenance or rehabilitation treatment, such as extending pavement life.

Step 2: Choose a suitable performance indicator for the pavement in question. The performance indicator must show the shortcoming that the maintenance or rehabilitation treatment should address. Examples of performance indicators are the cracking index, IRI, faulting, and rutting.

Step 3. Compute the MOE values for every pavement section where the same maintenance or rehabilitation treatment was applied.

Step 4. Check whether the M&R activity was found significant from a statistical perspective on the basis of MOE values regarding the chosen performance indicator. This can be accomplished by examining the null hypothesis that the mean MOE value is zero versus the alternative hypothesis that the mean is more than zero at the selected range of confidence interval. The MOE value regarding the chosen PI was determined for every pavement that received the treatment. Since the reported values of the PIs are average values taken across a significantly greater number of pavements, the dispersal of the MOE values can be regarded as a statistical sampling distribution of the means. Based on this assumption, the Alternate Hypothesis (H_A) and Null Hypothesis (H_0) for the treatment effectiveness, regarding the chosen PI and the MOE, can be expressed as follows (Sinha and Labi, 2018):

H_0: $\mu_{MOE} \leq 0$ (ineffective treatment)

H_A: $\mu_{MOE} > 0$ (effective treatment)

Step 5. After confirming the effectiveness of the M&R treatment, the next step is to use the different MOE values to make a wider statement regarding the effectiveness of the treatment. This can be accomplished by using one of the three primary forms: simple average value, statistical model (deterministic or probabilistic), or probability distribution. Further discussion is presented in the research (Sinha and Labi, 2018).

3.3.2. Non-Monetized Measures of Long-Term Effectiveness

This section describes step one of the framework expressed in the section above. The MOEs that can be used in assessing effectiveness incorporate several features: an enhancement of the average pavement performance over the treatment life; the evaluated life of a specific treatment; an extension of the pavement life because of the implemented treatment; an increase in the area covered by the curve of pavement performance because of the treatment; a reduction in the likelihood of initiating an undesired event or specific distress; and a decrease in the routine maintenance cost as a result of the treatment (Lavrenz et al., 2014). The treatment service life, the enhancement of the average pavement performance over the life of the treatment, and the area bounded by the performance curve methods are the focus of this dissertation.

3.3.2.1. Treatment Service Life

The life of an M&R treatment can be regarded as the time required for the recipient pavement to be returned to the predetermined condition threshold. The condition threshold can be determined in terms of a performance indicator that the treatment was intended to address. The treatment life's duration is dependent on the condition of the pavement, the intensity of the treatment, the agency's policy on loading, the climate, and triggers. The treatment service life is dependent not just on these aspects but also on funding availability. Treatment life can be evaluated using several strategies, which include the following: (1) an age-based strategy, or the time between two consecutive treatments and (2) a condition-based strategy, which evaluates the preserved pavement in terms of reverting back to a predefined performance threshold. The treatment life is viewed as a MOE for evaluating the treatment effectiveness implemented on a certain pavement section (Geoffroy, 1996; Hall, 2002; Khurshid et al., 2008; Labi et al., 2006; Mamlouk et al., 1998; O'Brien, 1989; Raza, 1994). Smith et al. (1993) assessed several M&R treatments, using different performance indicators that included individual measures of pavement distress such as skid resistance and roughness. They compared the treatments based on the time reserved for the treated pavement in reaching the threshold degree of the PIs. These two approaches are discussed below.

(i) The age-based approach

The number of years passing between the treatment and the upcoming similar or greater level of treatment is evaluated through the contact records (Li and Sinha, 2004). The age-based approach focuses on the actual life-span of the treatment, rather than making assumptions about treatment life and terminating the collection requirement of pavement conditions. The limitations of this approach outweigh its benefits. For instance, the life span of a maintenance or rehabilitation treatment may be affected by other factors such as regulatory changes, obsolescence, or changes in consumer values and attitudes (Lemer, 1996). Thus, the motivations behind executing a new treatment before the end of the life of its first treatment may include greater traffic loading, termination of safety problems related to defective design, and socio-economic changes making the pavement obsolete (Ford et al., 2011). The maintenance or rehabilitation treatment age reported in the contract dataset might not necessarily reflect the actual treatment age.

(ii) The condition-based approach

The goal of this approach can be defined as reaching the estimated time required for the pavement to return to a preassigned threshold condition after an M&R treatment is implemented. The condition-based approach is comprised of two methods: the aggregate approach and the disaggregate approach. In the aggregate approach, one performance model is established for all the pavements that received the same treatment, and treatment life is assessed by calculating the time required for the performance curve to reach the pre-specified threshold (Irfan, 2010; Lamptey, 2004). Using regression models, the factors impacting the pavement condition are determined at a specific confidence level. In the disaggregate strategy, pavement performance is monitored over time for every individual pavement receiving the same treatment. The treatment life is determined when the performance of the pavement falls under a predetermined threshold condition. This process can be repeated for a specific number of pavement sections receiving the same treatment. Then, the average treatment life of each pavement section is evaluated for this treatment, after which a deterministic model can be established where the service life is determined as a function of climate, traffic, layer thickness, or similar explanatory variables. After evaluating the treatment service life applied to every pavement section, probabilistic models are established using survival analysis (Irfan, 2010).

3.3.2.2. Enhancement in Average Condition of the Highway Pavement

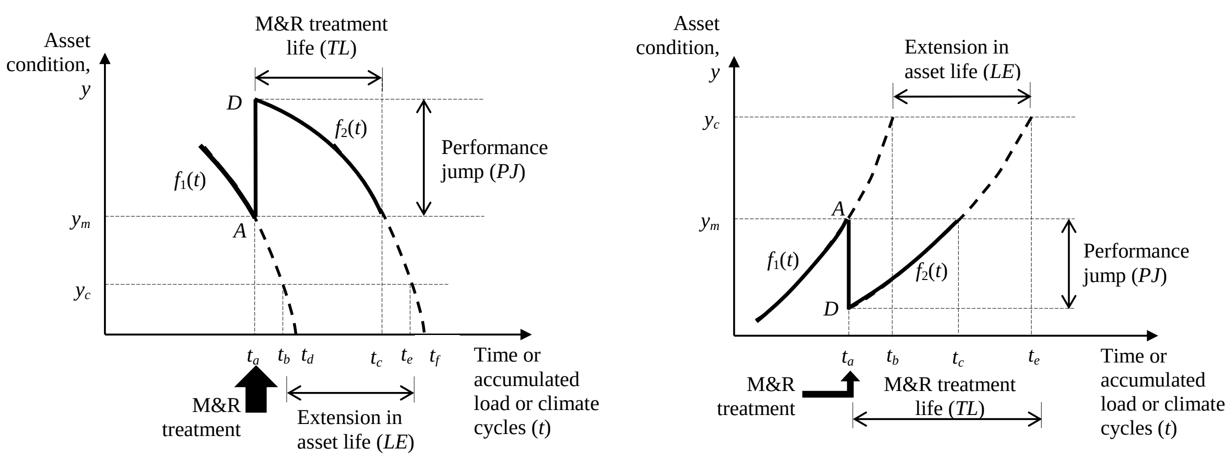
This MOE tests the level of distress as an absolute value relative to the level immediately before the treatment. The effectiveness can be measured by observing the average pavement condition with a well-defined performance indicator throughout the treatment life until the condition falls under a predetermined threshold. Developing pavement performance models based on data collected from different pavement sections that received the same maintenance or rehabilitation treatment is an alternative method for evaluating the annual average pavement condition. Next, the enhancement in the average condition of the pavement after the treatment, ψ, can be evaluated by calculating the percentage of change in the regular condition as compared to the condition prior to treatment.

$$\Psi = 100 * \frac{\left(\frac{1}{t_T}(PI_0 + PI_1 + \cdots + PI_T) - PI_i\right)}{PI_i} \tag{3.1}$$

where PI_0, represents the performance indicators of the pavement condition immediately after implementing a treatment, PI_T is the performance indicator when the pavement reaches the predetermined threshold or trigger value. While PI_i indicates the pavement condition at a given year, i and t_T represent the target periods during which the post-treatment condition is evaluated.

3.3.2.3. Area Bounded by Performance Curve

Many researchers have recognized that the area bounded by the performance curve and the threshold line encompasses the effectiveness concepts of (1) the treatment life and (2) the average performance of the pavement after it has received the treatment (Alqadhi et al., 2016). As such, this MOE may be the most appropriate way of assessing rehabilitation effectiveness. Like the other MOEs (M&R treatment life and pavement life extension), this MOE value can be determined using aggregate or disaggregate techniques. For the aggregate techniques, PIs that received the maintenance or rehabilitation treatment in question are monitored in several pavement sections. To do this, a graph of the condition measurements versus time is plotted, the area bounded by the performance plot first is determined for each section, and the average of these areas then is determined. For the disaggregate techniques, only one performance curve is developed using data from all the pavement sections that received the treatment, and then the area bounded by the curve is determined using calculus or coordinate geometry. Two rules apply for Figure 3.1: (i), the treatment effectiveness is the area under the curve, that is, the area bounded by the curve and the horizontal line projected from the threshold condition level; and (ii), the treatment effectiveness is the area over the curve, that is, the area bounded by the curve and the horizontal line projected from the threshold condition level (Labi et al., 2008).



(i) Non-increasing performance indicators (ii) Non-decreasing performance indicators

Figure 3.1: Relationships between treatment application and pavement life extension

This MOE has been applied widely in the area of pavement management, where it often has been used as a surrogate for pavement-related road user costs (Feighan et al., 1987; Geoffroy, 1996; Joseph, 1992). The following is the mathematical expression for the treatment effectiveness, in terms of the area bounded by the curve (Irfan, 2010):

For non-decreasing performance indicators (general form):

$$AOC_{CT,i} = \left\{\left[PI_{max} * \left(t_{CT(max)} - t_{CT(trig)}\right) - \int_{0}^{t_{CT(max)}} PI_{CT,i}dt\right] - \left[PI_{max} * \left(t_{pCT(max)} - t_{CT(trig)}\right) - \int_{t_{CT(trig)}}^{t_{pCT(max)}} PI_{CT,0}dt\right]\right\} \tag{3.2}$$

For non-increasing performance indicators (general form):

$$AUC_{CT,i} = \left\{\int_{0}^{t_{CT(max)}} PI_{CT,i}dt - \left[\int_{t_{CT(trig)}}^{t_{pCT(max)}} PI_{CT,0}dt - PI_{max} * \left(t_{pCT(max)} - t_{CT(trig)}\right)\right] - PI_{max} * \left(t_{CT(max)} - t_{CT(trig)}\right)\right\} \tag{3.3}$$

where $AOC_{CT,i}$ is the area over the performance indicator curve generated by a treatment and bounded by the performance indicator threshold, $AUC_{CT,i}$ is the area under the performance indicator curve generated by the $x = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a}$ treatment(s) bounded by the performance indicator threshold, and $PI_{CT,0}$ is the new construction (or do-nothing strategy) performance indicator. As shown in Figure 3.2, $PI_{CT,i}$ represents the post-treatment performance curve (model) for a treatment (*i*) in a given year (*t*). In Figures 3.3 and 3.4, $T_{CT(trig)}$ is the difference between the time at which treatment *i* reaches a trigger value for the PI and the time at which the previous treatment (i-1)th was implemented ($t_{CT(trig)}$ = t(i), the actual treatment life), $T_{CT(max)}$ represents the age (*t*) at the maximum allowable performance for the triggered treatment (*s*), $t_{pCT(max)}$ represents the age (*t*) at the maximum allowable performance for the pre-treatment (*pCT*), and t_{max} is the age (*t*) at the maximum allowable performance (end of pavement life).

The case study in this dissertation focuses only on a non-decreasing performance indicator, IRI. The suggested performance indicator for the non-increasing function is known as the present serviceability index (PSI).

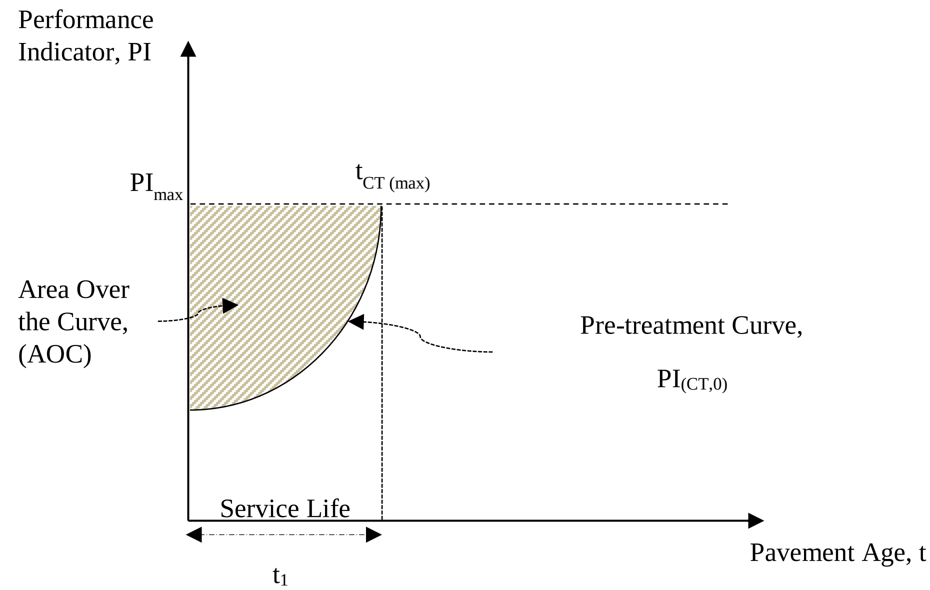


Figure 3.2: Pavement life cycle profile for do-nothing strategy

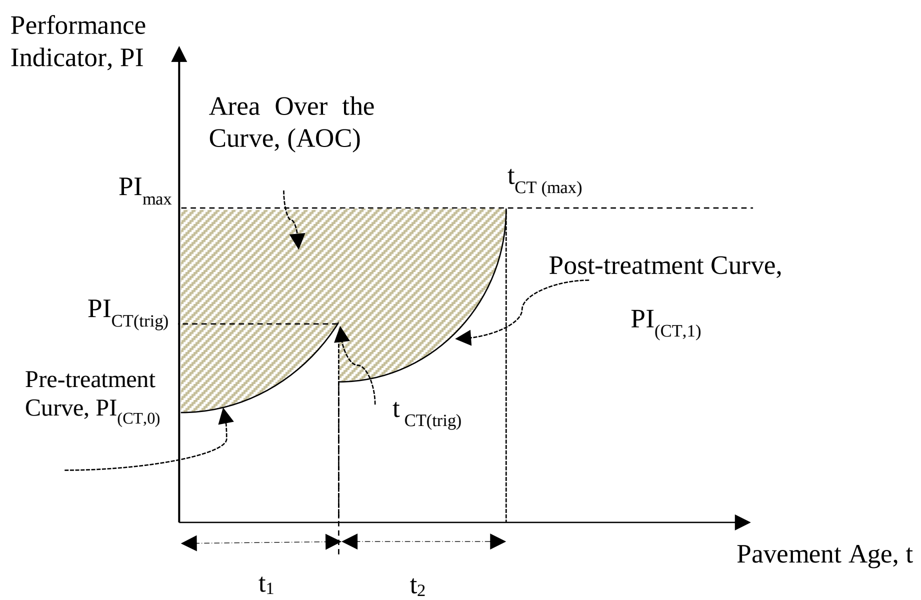


Figure 3.3: Pavement life cycle profile for the strategy with one major treatment

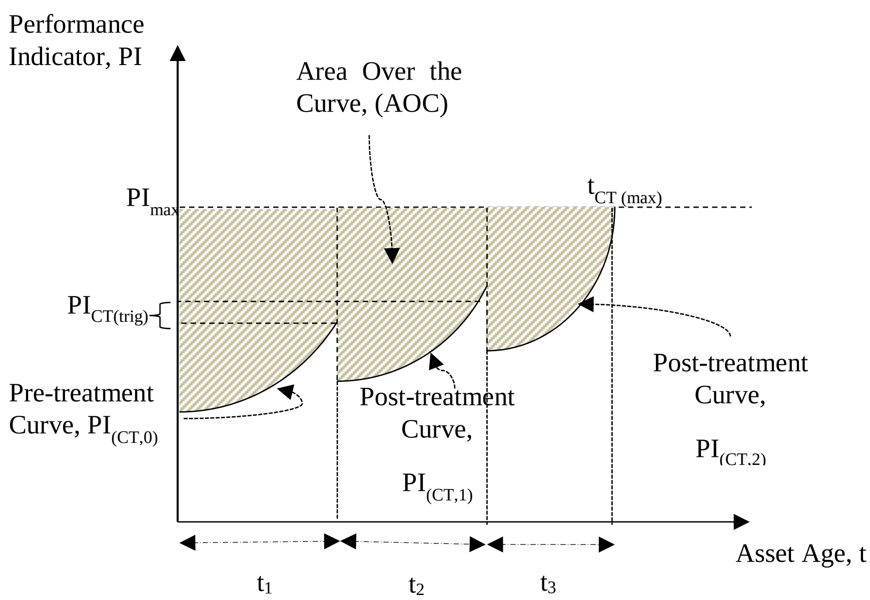


Figure 3.4: Pavement life cycle profile for the strategy with two major treatments

3.3.3. Monetized Long-Term Measures of Effectiveness

By improving the pavement condition, the agency (construction and M&R) and user costs (travel time, safety, and vehicle operating costs) are reduced, benefiting both the agency and the highway users. These benefits can be quantified by evaluating the reduction (savings) in costs gained by the agency and roadway users, as well as the community surrounding the highway location.

3.3.3.1. Agency Benefits

The frequency of routine maintenance increases when a maintenance or rehabilitation treatment is delayed due to budget constraints or other reasons. The reduction in the costs of these routine maintenance treatments can be expressed as benefits. In the literature, models have been developed to estimate the agency costs of several routine maintenance treatments, helping us to quantify agency benefits (Al-Mansour, 1994). Agency cost is estimated as a function of pavement condition:

$$Log\ ARM = a + b * (PSI) \tag{3.4}$$

where *ARM* is the annual routine maintenance spending in dollars/lane-mile, PSI is the pavement condition PI at the time of maintenance, and a and b are the estimated regression parameters. Equation (3.5) is used to convert PSI to IRI (Gulen et al., 1994):

$$PSI = 9e^{-0.008747*IRI} \tag{3.5}$$

Lu and Tolliver (2013) developed a direct relationship between the annual expenditure (*ARM*) and the pavement condition where the PI is represented by IRI (Equation 3.6). The annual routine maintenance expenses for interstate highways was found to be between \$400 and \$1,800/lane-mile (Volovski et al., 2017). Volovski's study presented panel models with random effect that predicted the annual routine maintenance expenditure for interstates (Equation 3.7), U.S., and states roads based on the pavement age, pavement type, climate, and traffic variables.

$$ARM = 10^{3.78-2.15EXP(-0.26*IRI)} + 10^{3.53-2.3EXP(-0.26*IRI)} \tag{3.6}$$

$$\begin{aligned} AMEX = (\beta_0 + \beta_1 RI + \beta_2 TL + \beta_3 \ln(SS) + \beta_4 Age + \beta_5 AOU + \beta_6 RPI \\ + \beta_7 LMR + \beta_8 RIN)^2 \end{aligned} \tag{3.7}$$

where *RI* is the rural road indicator (binary, 0 or 1), *TL* is the percentage of commercial vehicles from the total traffic volume (%), *SS* is the segment size (lane-km), *Age* represents the pavement age in years, *AOU* is the amount of usage or the traffic volume in 1,000s of AADT, *RPI* is the rigid pavement indicator, *LMR* is an indicator (binary) if the last major rehabilitation included construction of a new travel lane (1) or not (0), *RIN* is an indicator if the last major rehabilitation included rubblization (1) or not (0), and β_0 *to* β_8 are the model parameters.

3.3.3.2.User Benefits (VOC) During Normal Operation

In the literature, it is common to define the direct expenses of vehicle operations as the vehicle operating cost (VOC), which is a function of the vehicle type, fuel type, longitudinal grade, vehicle speed, delay, speed changes, roadway horizontal curvature, and road surface conditions (Sinha and Labi, 2011). The focus of this dissertation is finding the difference in VOC costs for different pavement materials during normal operations. Not all of the factors listed above are related to the pavement material used, whether surfaced with asphalt or concrete materials. The road surface is expected to make the most difference between VOCs on flexible pavement or rigid pavement. Rough pavement leads to more frequent vehicle maintenance, resulting in higher repair and maintenance costs. Also, rough pavement causes greater resistance for the vehicle tires, which increases the rate of fuel consumption compared with smoother pavement surfaces (Sinha and labi, 2007).

Both flexible and rigid pavements have different surface roughness values, both initially and throughout their service lives. Studies have correlated the increase in pavement roughness with an increase in VOC. For example, a study in New Zealand related VOC to pavement roughness (Opus, 1999) (Figure 3.5). This study found two types of costs related to VOC: (1) a smooth road base cost (0.41 $/vehicle-mile) and (2) an additional cost incurred only when the IRI exceeds 100. Another model, developed by Barnes and Langworthy in 2003, explains the relationship between VOC and pavement roughness (only when the IRI values are above 80 in./mile), as shown in Equation (3.8). The equation includes a VOC adjustment multiplier, *m*, for each pavement material type:

$$m = 0.001 * \left(\frac{IRI - 80}{10}\right)^2 + 0.018 * \left(\frac{IRI - 80}{10}\right) + 0.9991 \tag{3.8}$$

In order to estimate the VOC for flexible and rigid pavements, the pavement condition should first be evaluated, using IRI as a performance indicator. This can be done by using pavement performance models for each treatment (details are provided in Section 3.5.1.2).

Pavement roughness decreases when a maintenance or rehabilitation treatment is carried out, which causes a reduction in the VOC costs incurred by highway users and represents another way of evaluating the user benefits (savings) of maintenance or rehabilitation work (AASHTO, 2003).

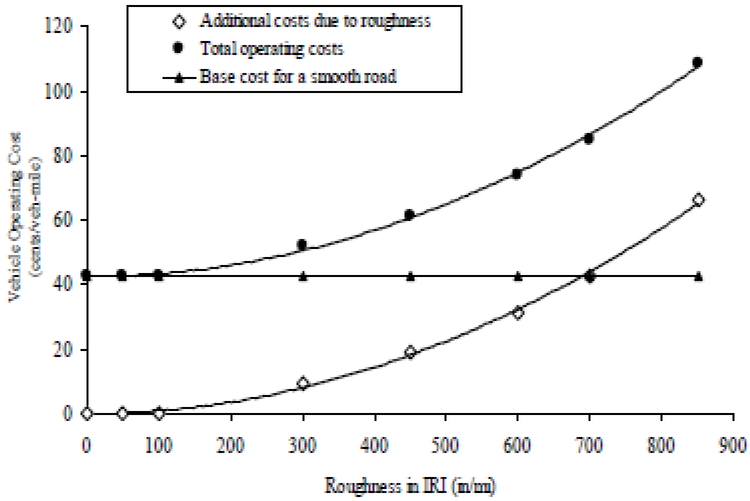


Figure 3.5: Relationship between pavement roughness and VOC

3.3.3.3. Community Benefits

One way to estimate community benefits is by evaluating the cost reduction due to the maintenance or rehabilitation treatment. Community cost is defined as the cost incurred by individuals located in the highway vicinity. There are several types of costs under this category: (1) noise, (2) air pollution, (3) water pollution, and (4) other ecological damage. The scope of this dissertation includes estimating the impact of different M&R treatments on air quality and noise level, assuming that other environmental resource costs are the same. The benefits are estimated by evaluating the reduction in cost associated with noise and air pollution brought about by implementing a certain treatment. In other words, benefits are estimated by evaluating the difference between the cost before and after applying the maintenance or rehabilitation treatment. Evaluations of community cost (Alqadhi et al., 2018) are presented under the cost estimation section in this dissertation. The following equations are used to evaluate the community benefits associated with reducing air and noise pollution, respectively:

$$S_A = APC_{Base} - APC_{After} \tag{3.9}$$

$$S_N = NPC_{Base} - NPC_{After} \tag{3.10}$$

While S_A represents the savings due to the air quality improvement resulting from pavement condition improvement from M&R treatment implementation ($), APC_{Base} is the air pollution cost ($) for the base case (where no treatment is applied). In addition, APC_{After} is the air pollution cost after implementing maintenance or rehabilitation treatment(s), S_N is the savings resulting from a reduction in noise level ($), NPC_{Base} is the noise pollution cost ($) with the base case (no treatment applied), NPC_{After} is the noise pollution cost after implementing maintenance or rehabilitation treatment(s).

The estimated savings for the costs related to air and noise pollution are evaluated based on the change in pavement condition before and after M&R treatments are applied. The relationship between air and noise pollution and pavement roughness is presented in the community cost evaluation section in this dissertation.

3.4. Performance Indicators

A PI or performance measure is a quantifiable tool for assessing asset condition. Performance indicators can measure the functional status and structural integrity of a pavement. It is vital to identify a performance measure before collecting data or building pavement performance models. Pavement performance indicators are classified in two major categories: (1) serviceability indices such as IRI, PSI, and rut depth, and (2) indices based on the distress condition, such as pavement condition rating (PCR), cracks, potholes, and slab faulting. IRI is considered the most popular pavement condition measure (Mannering, 2007; Robbins, 2016). IRI establishes uniformity in the physical measurement of pavement roughness and is a widely accepted measure of pavement condition in the U.S. and around the world. IRI measurement procedures were developed by the World Bank in Brazil (Sayers, 1986). IRI is a non-decreasing measure expressed in inches/mile (or m/km), which means that the road is getting rougher when the IRI value increases. By contrast, PSI is defined as a non-increasing pavement condition measure. It is measured using a panel of raters who drive over a highway segment and rate the pavement on a scale of 1 to 5, with 1 representing the worst condition and 5 representing a smooth and perfect pavement. Due to the common use of IRI as a good measure of pavement condition, and since the LTPP dataset documented the pavement section profile in IRI values, this dissertation used IRI to determine the response variable for the pavement performance models.

3.5. Pavement Performance Models

One of the primary purposes of developing performance models for pavement is to predict its future behavior and determine the pavement deterioration over time. Accumulative traffic loads, environmental factors (e.g., temperature, freeze index, precipitation), design, and construction all are the factors that affect pavement condition. Pavement performance models help the asset manager to decide when interventions are needed. Pavement performance models typically are developed to predict the following: primary responses (i.e., stress, strain, or deflection); structural integrity (i.e., rutting, alligator cracking, and faulting); and functional performance (based on user perceptions such as comfort level) (Irfan, 2010).

Pavement performance models can be evaluated by adopting empirical, mechanistic, or mechanistic-empirical methods. The empirical method is determined with statistical models based on observed historical data. The mechanistic method is evaluated based on one of the mechanics theories (i.e., visco-elastic theory). The mechanistic-empirical method analyzes features such as stress or strain, which are estimated by the mechanistic model (Feighan, 1987; Haas et al., 1994; Lytton, 1987; Wadsworth, 1990). All of these methods, each already adopted by highway agencies, fall under two main categories: deterministic modeling or probabilistic modeling (Haas, 1994). In this dissertation, the empirical method is combined with the deterministic approach to build statistical models that predict pavement performance.

A variety of research studies have developed pavement performance models for flexible and rigid pavements. Treatment-specific prediction models are essential for estimating the potential benefits of each treatment. These models have been introduced and analyzed by a number of researchers (Al-Mansour et al., 1994; Gulen, 1994; Sedat et al., 2001; Irfan, 2010; Irfan et al., 2009; Khurshid, 2010; Labi & Sinha, 2003; Lamptey, 2004; Lamptey et al., 2005; Livneh, 1996; Rajagopal & George, 1991; and Sebaaly et al., 1995). These models were developed using different performance indicators with a variety of explanatory variables that included traffic loading, environmental factors, pavement thickness, mixture design, and type of routine maintenance applied.

One of the earliest studies using the LTPP dataset to assess flexible pavement rehabilitation treatments (SPS 5) concluded that no definite results were found (Daleiden et al., 1998). That study was inconclusive due to the absence of pavement distress data since the pavement sections in the LTPP experiment were relatively newly constructed. The treatment service life spans of several rehabilitation activities have been estimated at eight to 15 years, based on the prediction models developed using the LTPP dataset for SPS 5 (Hall, 2001). The SPS 5 studies examined pavement roughness with IRI as a measure of pavement condition (Perera et al., 1999; Perera et al., 2006). The results of these studies suggest the following conclusions: 1) pavement roughness is not significantly influenced by pre-existing conditions; 2) pre-overlay methods of preparation were found to be insignificant; 3) two-inch and five-inch overlays of asphalt concrete in the early stages have similar levels of effectiveness; and 4) pavement pre-overlay conditions significantly influence pavement roughness, especially during the early life of the pavement overlay. Another study by Hall et al., (2002) was conducted on the rehabilitation treatments of flexible pavement, and reached the following conclusions: 1) five-inch asphalt overlay performed better than two-inch overlay; there is a strong relationship between pre- and post-IRI and 2) the influences of pavement age and average air temperature can grow stronger over time. Ahmed et al. (2013) used the data from the LTPP western region of the SPS 5 study to evaluate the performance of flexible pavement in that region, using aggregate and disaggregate methodologies by comparing the pavement rehabilitation treatments for asphalt concrete pavements with different overlay thicknesses and different levels of preparation before the overlay treatment was applied. Their study concluded that the five-inch overlay was superior to the two-inch overlay, in terms of long-term effectiveness in the western LTPP region. Also, before implementing an overlay, they found that an intensive preparation method was found more favorable rather than a minimal one.

Several studies have been conducted to evaluate the effectiveness of rigid pavement M&R treatments. Hall et al. (1993) performed a comparative study of several load-transfer treatments on an interstate in Florida. In addition, for the SPS 6 study, the effectiveness of the rigid pavement treatment on the LTPP data was evaluated on the Pennsylvania pavement test section (Morian et al., 2003). Hall et al. (2002) evaluated the effectiveness of rigid pavement treatments and flexible treatments for the SPS 6 study. The study arrived at a number of conclusions regarding long-term effectiveness: 1) the level of preparation of the pavement before applying the overlay treatment was found to be insignificant (minimal vs. intensive); 2) no relationship was detected between pre-treatment IRI and post-treatment IRI; and 3) the IRI difference between the control sections and the rehabilitated sections grew intensely with an increase in the accumulated heavy truck traffic. Khurshid et al. (2008) studied the pavement performance of asphalt overlays on rigid pavements based on five of the SPS 6 treatments across the LTPP regions. Their study suggests that the pavement condition before the rehabilitation treatment can have a significant influence on the post-treatment pavement performance. Also, their results shed light on the long-term effects of weather severity and traffic loading on pavement performance while the effect of pavement location across the different climatic zones was found to be negligible.

Most of the pavement performance studies based on the LTPP datasets were carried out during the early stages of pavement life. Indeed, only a few studies considered the long-term effectiveness of pavement M&R treatments across different climatic zones. Recently, models of flexible and rigid pavements were developed using the artificial neural network (ANN) modeling technique to predict pavement roughness in IRI and the LTPP database (Gopisetti, 2017). The models used in Gopisetti's study represented selected individual sections across the climatic zones but did not present models for each treatment individually. This study demonstrated how to predict IRI from different sets of attributes, such as traffic load and climatic inputs, without specifying the behavior of the individual treatments. A number of studies were conducted to compare linear regression and ANN models using the LTPP database (Abdelaziz et al., 2018; Jaafar et al., 2016). These studies used pavement age, cracking, and rut depth as explanatory variables to predict pavement roughness. These models are not treatment-specific models, but rather aim to find a better prediction method for pavement condition in general based on specific attributes. The need remains for a more comprehensive study that considers treatment-specific models across different climatic zones to study the behavior of each treatment in the different climatic conditions.

3.5.1.1. Development of Performance Jump Models

To develop an optimal profile schedule of M&R activities, performance models first must be developed for each M&R treatment for use as inputs for the optimization process. This dissertation presents post-treatment performance models (PTPM) as well as performance jump models (PJM).

Ideally, different model forms are tested both for PTPM and PJM for each LTPP climatic zone. Only PJM models have been estimated from the aggregated data from all the climatic zones in relation to M&R treatments on AC and PCC pavements. The aggregation of data from all the climatic zones were collected with limitations since the number of observations for each treatment at each climatic zone were insufficient. The forms used for performance jump models are linear, exponential, power, and logarithmic, as shown below:

$$IRI_{Drop_i} = \alpha + \beta \times IRI_{Pre} \tag{3.11}$$

$$IRI_{Drop_i} = \alpha \times e^{\beta \times IRI_{Pre}} \tag{3.12}$$

$$IRI_{Drop_i} = \alpha \times {IRI_{Pre}}^{\beta} \tag{3.13}$$

$$IRI_{Drop_i} = \alpha + \beta \times [\ln IRI_{Pre}] \tag{3.14}$$

where IRI_{Drop} is the sudden drop in the IRI value due to treatment *i* implementation, IRI_{Pre} is the trigger (pre) value of the pavement condition when treatment *i* was applied, α is the constant term, and β is the specific parameter for the model explanatory variables.

3.5.1.2. Post-Treatment Pavement Performance Models

In this dissertation, the effectiveness of each M&R treatment was evaluated using regression prediction models. Treatment-specific models were developed for each treatment at each climatic zone included in the LTPP database. Pavement performance models mainly have been developed for flexible pavement preventive maintenance (SPS 310 and SPS 320) and rehabilitation treatments (SPS 5). Also, using IRI as the performance indicator, prediction models of the pavement condition were estimated for rigid pavement rehabilitation (SPS 6). In general, a direct correlation between pavement roughness and rigid pavement maintenance treatments was difficult to detect using such models. Using the LTPP dataset, models weredeveloped for some of the applied rigid maintenance treatments where significant correlation with IRI was detected.

The LTPP program study was conducted across the U.S. and Canada in four well-defined climate zones. For all the types of model forms used in this dissertation, annual measurements of pavement roughness, represented in IRI, were collected from the datasets as the dependent variables. For each treatment at each one of the four climatic zones, IRI as well as other explanatory variables, such as traffic volume and environmental data, were reported, which qualified the database as a cross-sectional dataset. In addition, the data can be described as a time-series set of data because of its variation over time. The most suitable description of this dataset is panel data because it combines the characteristics of both cross-sectional and time-series data. Pooled data, or panel data, permit analysts to capture the specific behaviors of the data that cannot be identified only by cross-sectional or time-series data (Hsiao, 1986; Washington et al., 2010).

Ordinary least squares regression models cannot capture two main characteristics: heterogeneity and serial correlation. The first and most crucial issue needing to be addressed when comparing cross-sectional or time-series data with panel data is heterogeneity bias (Greene, 2000; Hausman et al., 1981). Heterogeneity is the variation across cross-sectional units and does not necessarily represent an actual dataset. Why not? This issue of heterogeneity in the LTPP database is manifested in repeated pavement sections for a single treatment. For example, a rehabilitation treatment of a flexible pavement (SPS 507) with a five-inch asphalt overlay in a wet-freeze zone has five different sections. Each of these pavement sections has from five to 19 years of observations, which qualifies the dataset as an unbalanced panel dataset. To avoid inconsistencies in model inferences, heterogeneity must be taken into consideration (Ghahari et al., 2018; Greene, 2000). The second issue is the serial correlation of the disturbance term, which is a result of correlation across time. An unbiased and slandered error in the regression estimates is expected if the serial correlation is not accounted for (Washington, 2010). This error results in increased t-statistics for some of the variables and makes them seem statistically significant.

The general form of the Ordinary Least Squares (OLS), which does not account for heterogeneity and serial correlation, is presented below:

$$y_i = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_n X_n + \varepsilon_i \tag{3.15}$$

where y_i is the dependent variable for model i, β_0 is the constant term, $\beta_1$to β_n are the parameter estimates, and ε_i is the error term.

Fixed-effect and random-effect are used to resolve the issues of heterogeneity and autocorrelation, also known as serial correlation. An estimate of fixed-effect is used to correct for heterogeneity through a binary variable, also referred to as dummy variable, applied to each pavement section. The primary shortcoming of using the fixed-effect is that the estimated models can be used only in the same climatic zones from which the data are extracted. This means that a thin overlay (SPS 310) model in the wet-freeze zone, for example, cannot be used to estimate the pavement performance of the same type of treatment in another climatic zone, such as a dry-freeze zone. Since models already have been developed for each treatment type across all climatic zones, a panel model with fixed-effect estimation is sufficient for our purposes in this dissertation. Random-effect is considered to be a better modeling prediction method if these models need to be used in areas other than those in which the data were collected (present more general forms). The following is the general form of the panel model (Ghahari, 2018; Greene, 2000; Hsiao, 1986; Washington, 2010):

$$
\begin{aligned}
&Y_{it} = \alpha + \boldsymbol{\beta_{it} X_{it}} + \mu_i + \lambda_t + \nu_{it} \\
&\mu_i \approx N\left(0, \sigma_\mu^2\right) \\
&\lambda_t \approx N\left(0, \sigma_\lambda^2\right) \\
&\nu_{it} \approx N(0, \sigma_\nu^2) \\
&\sum[x_{it}\mu_i] = 0
\end{aligned}
\tag{3.16}
$$

where Y_{it} is the dependent variable for *i* cross-sectional unit (in IRI values, in/mile), α is the constant term, $\boldsymbol{\beta_{it}}$ is the vector of the parameter estimates, $\boldsymbol{X_{it}}$ is the explanatory variable estimates, μ_i is the unobserved group effect, λ_t is the unobserved time effect, and ν_{it} is the random disturbance term.

Different forms of post-treatment pavement performance models have been developed in the literature, and most of them follow the exponential form. Some of the model forms developed in this dissertation include the following:

$$IRI = e^{\alpha + \beta * ATT * t + \gamma * FI * t} \tag{3.17}$$

$$IRI = e^{\alpha + \beta * t} \tag{3.18}$$

$$IRI = e^{\alpha + \beta * ATT * t + \gamma * \mathrm{AATEM} * t} \tag{3.19}$$

$$IRI = e^{\alpha + \beta * ATT * t} \tag{3.20}$$

where *IRI* represents the pavement condition in inches per mile for *i* activity at year *t*, *ATT* is the annual truck traffic of a specific pavement section (represented in millions of trucks), FI is the average annual freeze index (represented in thousands of degree-days), AATEM is the accumalated effect of the average annual temperature (t* average annual temperature in Fahrenheit)/ 1,000), and α, β, and γ are the model parameters.

3.6. Cost Analysis

This dissertation considered three cost categories that affect alternative action decisions and differ across material types: agency, user, and community costs. The agency cost considers construction, rehabilitation, maintenance, and salvage. The user cost is the cost incurred by the road user during normal operations and in work zones. Costs related to work zones include travel time delay costs and VOC, which are related to the work zone duration. The community costs consist of noise and air pollution. The noise cost is calculated as the cost of the noise barrier needed to mitigate the traffic noise, and the air pollution cost is estimated by monetizing the social damage associated with the global warming potential (GWP) of the greenhouse gas (GHG) emissions evaluated as carbon dioxide (CO_2) equivalent emitted during the life cycle assessment (LCA) phases of the pavement materials.

3.6.1. Agency Cost Estimation

Agency cost is defined as the expenditures incurred by the asset owner or operator, usually a public agency that provides the transportation service. This cost comprises seven stages: advance planning, preliminary engineering, final design, right-of-way acquisition and preparation, construction, operation, and preservation and maintenance (Sinha and Labi, 2007). In this dissertation, only (re)construction, maintenance, and rehabilitation activities are considered. The initial construction cost is no longer considered as the only criterion used for evaluating and selecting transportation projects; rather, all the incurred costs throughout the life cycle of the project should be considered. Maintenance costs “are incurred to preserve the capital investments made in the pavement infrastructure and to ensure that the pavement provides a satisfactory level of service to its users” (Lamptey, 2005). The reconstruction and rehabilitation costs are “the costs incurred in all phases of the design and construction of the facility” (Lamptey, 2005). The average cost of individual treatments might be sufficient for planning purposes and is commonly expressed in dollars per unit area (dollars/m^2 or dollars/ft^2) or dollars per lane-mile. The average agency cost for a specific treatment can be expressed as shown below:

$$Agency\ Cost = UC * L * N \tag{3.21}$$

where the agency cost is the total preservation cost in 2017 constant dollars, *UC* is the average unit cost in dollars/lane-mile, *L* is the length of the project in miles, and *N* is the number of lanes.

Costs can be adjusted for inflation using the FHWA construction price index (CPI), as shown in the following equation (Walls, 1998):

$$C_{AY} = C_{BY} \times \frac{I_{AY}}{I_{BY}} \tag{3.22}$$

where C_{AY} is the cost for the required year, C_{BY} is the cost for the reference year, I_{AY} is the index for the year of the analysis, and I_{BY} is the index for the reference year. Special cost adjustments also should be considered, as shown below.

$$C_{SQ} = C_{RS} \times \frac{I_{SQ}}{I_{RS}} \tag{3.23}$$

where C_{SQ} is the cost of the activity in the state in question, C_{RS} is the cost of the activity in the reference state, I_{SQ} is the index corresponding to the state in question, and I_{RS} is the index corresponding to the reference state.

Statistical models of the agency costs have been developed to consider pavement condition, project length, and number of lanes with two functional forms, as shown in Equations (3.24) and (3.25) (Irfan, 2010).

$$Agency\ Cost = \alpha . length^{\beta} . N^{\gamma} . \left[ln\left(PI_{trig}\right)\right]^{\delta} \tag{3.24}$$

$$Agency\ Cost = \alpha + (\beta . length) + (\gamma . N) + \left(\delta . \left[ln\left(PI_{trig}\right)\right]\right) \tag{3.25}$$

where the agency cost of a pavement treatment is in millions of 2017 constant dollars, length is the length of construction in miles, *N* is the number of lanes, PI_{trig} is the pavement condition before applying the treatment (surface roughness in IRI in in./mile), and α, β, γ, and δ are the parameter estimates of the model's explanatory variables.

3.6.2. User Cost Estimation

User cost represents the costs incurred by the highway user over the life of the project. This cost is dependent on highway improvements and the related M&R strategies over the analysis period.

This cost determines the total cost of a transportation project and can be divided based on category (work zone and normal operation user costs) or component (vehicle operating costs, travel time costs, crash costs, and environmental costs) (Lamptey, 2005). In this dissertation, only the work zone user costs are discussed; more specifically, travel time delay and vehicle operating costs. User costs during normal operations are considered invariant across different types of treatments and for different pavement materials (Hall et al., 2001; Maurer et al., 2007; Shober & Friedrichs, 1998). In this dissertation, the safety cost is assumed to be the same for both flexible and rigid pavements.

3.6.2.1. Costs Caused by Travel Time Delay at Work Zones

The delay costs incurred by highway users are a direct result of the construction duration of each M&R treatment (Irfan, 2009). This delay, caused by partial or full closure of the highway, leads to an increase in road user travel time, which can be converted to a monetary cost. Delay costs due to reductions in speed limit for each pavement treatment were considered in order to estimate work zone costs. These delay costs were assessed using the following expression:

$$Delay\ Cost = D_{WZ}.\sum_{j}^{J}(V_j.\Delta T_j.DC_j) \qquad (3.26)$$

where V_j is the number of vehicles delayed by the speed change at the work zone for each vehicle class, ΔT_j is the travel time difference in hours for the speed change of vehicle j, DC_j is the travel time delay cost rate in dollars/mile, D_{WZ} is the time taken for each treatment in days, and j is the vehicle class. The travel time delay cost for each vehicle class must be updated to 2017 constant dollars using the FHWA consumer price index (Walls, 1998).

Duration models for maintenance, rehabilitation, and new construction activities were developed following the general form (Irfan, 2010):

$$D_{WZ} = e^{\alpha+\sum_{k}^{K} B_k.X_k} \qquad (3.27)$$

where D_{WZ} is the treatment duration in days, α is a constant term, B_k is the parameter estimate of the model's explanatory variables, and X_k is a vector of the explanatory variables.

3.6.2.2. Vehicle Operating Cost (VOC) at Work Zones

Several factors affect VOC, including vehicle type, fuel type, longitudinal grade, vehicle speed, delay (during normal operations), speed change, horizontal curvature, and road surface condition (Sinha and Labi, 2011). Vehicle operating costs at work zones are a special case of the VOC during normal operations. Only travel time delay and speed (or speed reduction) affect roadway user costs at work zones. In this section, VOC due to speed reductions in the work zone are considered. The fuel VOC change is calculated using the following expression:

$$VOC\ Cost = D_{WZ}.\sum_{j}^{J}(V_j.\Delta T_j.g_j.p_j) \qquad (3.28)$$

where V_j is the number of vehicles delayed by the speed change at the work zone for each vehicle class, ΔT_j is the travel time difference due to the speed change for vehicle j in hours, D_{WZ} is the time taken for each treatment in days, g_j is the fuel consumption in gallons per hour of delay, p_j is the average fuel price in dollars per gallon, and j is the vehicle class (Irfan, 2010).

3.6.3. Community Cost Estimation

The community cost is the cost incurred by individuals living or working in the vicinity of a highway under repair. It includes the costs associated with air pollution, noise pollution, water pollution, and other ecological degradations. This dissertation estimated the effects of constructing and maintaining flexible and rigid pavements on air quality and noise level, assuming that other environmental resource costs were the same for both pavement material types.

3.6.3.1. Air Pollution Cost

The primary producer of carbon monoxide and other hazardous gases in the U.S., more than any other industries, is the transportation sector, mainly due to fuel consumption (Bennett et al., 2001). Air pollutants can be categorized as (i) criteria pollutants and (ii) greenhouse gases (GHG). Pollutant emissions are caused by stationary sources (during construction/repair) or mobile sources (mainly from vehicles). The six most common criteria air pollutants that may directly affect human health include: ozone (O_3); particulate matter (PM); carbon monoxide (CO); nitrogen (NOx);

sulfur dioxide (SO_2); and lead (Pb). Greenhouse gases (GHG) can be defined as atmospheric gases that have the ability to trap heat. The most common GHG include carbon dioxide (CO_2), methane (CH_4), nitrous oxide (N_2O), and fluorinated gases. CO_2 emissions come mostly from petroleum fuels, gasoline in particular, and account for 80% of the total greenhouse gas emissions (K. C. Sinha, 2007).

Since this dissertation addresses the monetary consequences of flexible and rigid pavement materials, the mobile sources of pollutants are considered the same across both forms of pavement. It is reasonable to assume that the same mobile emissions given that both pavement types are expected to have the same traffic volume would have the same level of mobile emissions. Full LCA of greenhouse gas emissions and fuel consumption was undertaken in this dissertation, beginning with the acquisition of raw materials and concluding with the end of pavement life.

LCA is a methodological assessment of the potential environmental burdens of a pavement and its impacts on factors including climate change, human health, and fossil fuel depletion (Rebitzer et al., 2004). LCA covers the environmental aspects and potential impacts of the pavement throughout its service life, from material acquisition through production, construction, M&R, and eventually product disposal. This methodology was developed based on the International Standards Organization (ISO), and specifically the ISO 14040 and 14044 series. There are four basic phases of LCA (Gopalakrishnan et al., 2014; ISO, 1997):

1. **Goal and scope definition:** This phase describes the system boundaries and functional unit selections. Clear definition of the levels of detail and the overall time constraints are required. All assumptions and data sources must be specified and all questions must be addressed. Any limitations in this process also need to be clearly addressed.
2. **Life cycle inventory (LCI) analysis:** This stage results in an estimation of the resource consumption and waste quantities associated with the production of flexible and rigid pavements and includes collecting all data inputs (e.g., raw materials, water, and energy usage) and outputs (e.g., waste and atmospheric emission) in the defined system. The raw materials considered are asphalt binders (bitumen), cement, aggregates, steel, and other supplementary cementitious materials such as fly ash and slag. The system boundary, as presented in Figure 3.7, needs to be defined prior to developing the LCI and its limits (Leng et al., 2017).

3. **Life cycle impact assessment:** This stage produces an evaluation of the life cycle impact on different categories, such as global warming, fossil fuel depletion, and human health. This step must specify an indicator to measure the impact of pavement materials on human health. The global warming potential (GWP) of the GHG and energy consumption are chosen because of their huge impact on the environment. The GWP emission usually is presented in kilograms of carbon dioxide equivalent emission (kg of CO2 equivalent).
4. **Life cycle interpretation:** This stage compares the performance scores of all the impact categories to evaluate the results. A summary of the LCA four basic phases is shown in Figure 3.6 (BSI, 2006).

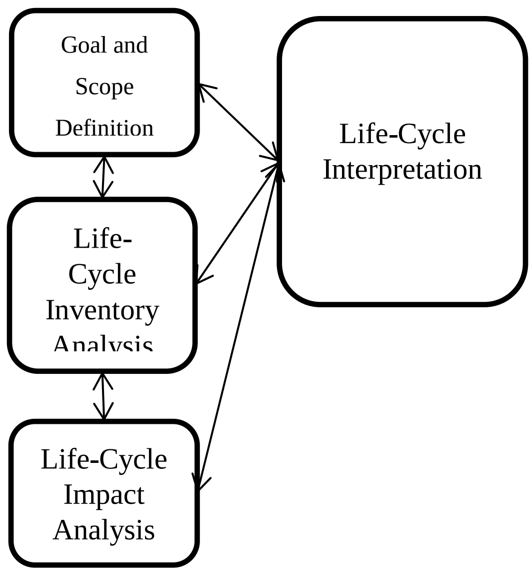


Figure 3.6: Life-cycle assessment framework

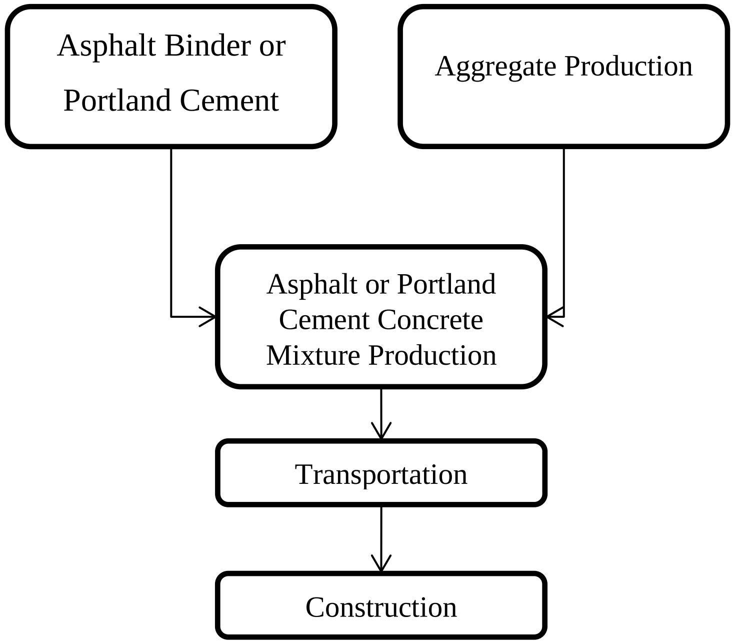


Figure 3.7: LCI system boundary

The scope of this part of the dissertation included energy consumption and the GWP of the GHG emissions associated with acquisition of raw materials such as binders, aggregates, and additives. This part also examines the production of asphalt binders and Portland cement, production of aggregates, asphalt and PCC plant operation, and construction, M&R, transportation, and product disposal.

A comparable LCA study was conducted in Finland to compare asphalt concrete and jointed plain Portland cement concrete pavements (JPCP) (Hakkinen et al., 1996). CO_2 emissions, energy consumption, and other environmental criteria were evaluated. The study also considered an upstream supply chain of AC and PCC, along with the construction phase. The Finish study concluded that the CO_2 emission of rigid pavement is 40 to 60% more when compared to the emission from flexible pavement. In addition, the study suggested that the energy consumption of asphalt pavement is twice that of the energy consumed by PCC pavement.

A study developed at Carnegie Melon University comparing hot mix asphalt (HMA) and continuous reinforced concrete pavement (CRCP) showed that HMA consumes 40% more energy than CRCP (Horvath et al., 1998). This study did not account for the feedstock energy of asphalt production. The study also suggested that CRCP has an adverse impact on the environment that is greater than the other environmental outputs. The combined effect of life cycle air quality and energy consumption of AC is 90% higher compared to the effect of PCC (Roudebush, 1999). A comprehensive study done by Stripple (2001) included all phases of the LCA process, except the use phase. Their study compared three different pavement materials: (1) HMA, (2) cold mix asphalt (CMA) and (3) JPCP. The results showed that the CO_2 emissions of HMA and CMA were 17% lower than that of JPCP. Also, the energy consumption of HMA was 34% lower than that of JPCP.

Several studies conducted with different system boundaries concluded that rigid pavement has a lower environmental impact on air quality, energy consumption, or both air quality and energy consumption (Nisbet et al., 2001; Treloar et al., 2004). By contrast, other studies produced opposite results. This contradiction in the research (Berthiaume et al., 1999; Hoang et al., 2005; Roudebush, 1999; Stripple, 2001; Zapata et al., 2005) motivated this dissertation to use a comprehensive system boundary. The contradictory results were caused by defining the system boundaries in certain ways, which eventually affected the results, or by using different data sources. A well-defined comprehensive system boundary, where all phases are considered, leads to a more conclusive outcome. The Athena (2006) study, conducted in six locations within the U.S. and Canada, is arguably the most comprehensive study comparing asphalt and concrete pavements. The study concluded that asphalt concrete pavements consume more energy than rigid pavements. Most of the asphalt energy consumption came from the feedstock process. The rigid pavement GHG emissions were found to be 11% more than asphalt pavement emissions. Other comprehensive studies reached the same results as the Athena (2006) study (Aurangzeb et al., 2014; Chan, 2007; Hakkinen, 1996; Horvath, 1998; Weiland et al., 2010).

A number of LCA assessment models have been developed in compliance with ISO protocol. These models are used as tools to quantify the life cycle effects of emission, energy consumption, and other environmental effects. Some of these models, such as GaBi and SimaPro, are available commercially and widely used. These two software programs have built-in databases and licenses that are required to gain access to the databases. By contrast, PaLATE is an open source tool that was developed by the University of California, Berkley in 2004 and updated in 2011; it contains the LCA's material, construction, maintenance, and end of life phases. The output of this model is limited to energy use. Other tools for X are available as well, including Portland

Cement Association (Cement), Eurobitume (Bitumen), PAS 2050, aspect, PE-2, CHANGER, and the Athena Impact Estimator of Highways (Gopalakrishnan, 2014; Thakkar, 2016).

Athena Impact Estimator for Highways is a software that tracts the following LCA phases: materials, manufacturing processes, construction, vehicle usage, M&R stages, and transportation (Athena, 2014). It has the flexibility to allow users to input different design parameters either for asphalt or concrete pavements. The Roadway Designer tool has the ability to deal with a variety of different surfaces and base and sub-base materials. This tool follows environmental impact assessment measures in compliance with U.S. EPA regulations. The latest version of this software reports the following: global warming potential, total primary energy, non-renewable primary energy, fossil fuel consumption, acidification potential, HH criteria, Ozone depletion potential, smog potential, and eutrophication potential. This tool is used in this dissertation to estimate the GWP of the GHG and energy consumption quantities. The outputs of different types of treatments are presented in Chapter 4.

The next step is to evaluate the environmental impact of flexible and rigid pavements by monetizing their emission amounts and assessing how much energy is consumed during the entire life of the pavement. Three different approaches have been used to examine these questions in monetary forms: (1) considering the cost of cleaning up the air surrounding the polluting source; (2) estimating the social damage cost of air pollution on the emission source vicinity; and (3) estimating the cost based on an individual's willingness to pay for damages. The social damage approach is adopted in this dissertation. The first performance indicator is the GWP of the GHG, measured by the CO_2 equivalent gas emission. The second PI is the quantity of energy consumed by the entire process. These values of emission and energy consumption are estimated using Athena Impact Estimator for Highways software. The air pollution cost then is estimated using the following equations:

$$APC = GWP_{Cost} + EC_{Cost} \tag{3.29}$$

$$GWP_{Cost} = E * SDC \tag{3.30}$$

$$EC_{Cost} = E_{Con} * EUC \tag{3.31}$$

where *APC* is the cost of air pollution ($/lane-mile), EC_{Cost} is the energy consumption cost ($/Lane-Mile), *SDC* is the estimated social damage cost ($/metric ton), *E* is the emissions of CO_2 equivalent (metric tons) for one mile of pavement production, E_{Con} is the energy consumed for one lane-mile of pavement in mega joules (MJ), and *EUC* is the average energy unit cost ($/MJ).

Air Quality Savings (Benefits) Related to Pavement Roughness Improvement

Analysis of the impact of mobile sources of emission is useful in estimating the benefits of pavement overlay on air quality (Sinha and Labi, 2007). Pavement roughness has an indirect impact on greenhouse gas (GHG) emissions, since the pavement condition affects vehicle speed and vehicle speed affects GHG emission. Paterson & Watanatada(1985) concluded that drivers tend to increase their speed as pavement conditions improve. This means that the average speed will increase with the reduction in IRI value. Other studies indicated that travel speed is affected by pavement roughness only when it exceeds 290 in/mi (Kalembo et al., 2012). According to a study by Wang et al. (2014), there is an insignificant impact of pavement roughness on free flow speed (only 0.48 to 0.64 km/h change in speed for each unit increase of IRI m/km). This small change in free-flow speed due to pavement condition improvement will have a negligible effect on pollutant emissions, including GHG. Since pavement roughness is not going to exceed 290 in/mile the change in the average speed is negligible, and therefore the community benefit associated with air quality could be ignored (Alqadhi et al., 2018).

On the contrary, a one-unit increase in pavement roughness represented in the IRI (m/km) leads to a 2-3 percent increase in fuel consumption of passenger cars, irrespective of vehicle speed. The increase in fuel consumption for heavy trucks is 2-3 percent at 35 mph and 1-2 percent at 70 mph (Chatti et al., 2012). Because of this relationship between pavement condition and fuel consumption, evaluation of atmospheric emissions during certain maintenance or rehabilitation treatments is possible. To understand these atmospheric emissions, the fuel consumption on a given road segment must be evaluated using two scenarios: implementing a treatment and no treatment. The amount of chemical emissions due to fuel consumption is then evaluated and monetized based on its adverse impact on human health. Different costs are assigned based on the severity of health outcomes (Pellecuer et al., 2014a).

The first step is to estimate the fuel consumption (*FC*) of vehicle class *i*, in (mL/km), based on the HDM-4 model equations developed by Chatti and Zaabar (2012). Only the general form of their model is presented in this dissertation:

$$FC_i = \frac{1000}{vi} * \{max[\alpha_i, \varepsilon_i * (P_{tot} * (1 + dFuel))]\} \tag{3.32}$$

where v_i represent the speed of the vehicle (m/s), α_i is fuel consumption (mL/s), ε_i is the engine efficiency (mL/kW/s), *dFuel* is the ratio of excess fuel caused by congestion, P_{tot} is the power (kW), and *i* is the vehicle class. The detailed equations, parameters and assumption are presented in Chatti and Zaabar's book (2012).

The most common emissions that have negative impacts on the environment due to fuel consumptions are: carbon monoxide (CO), hydrocarbons (HC), sulfur dioxide (SO_2), nitrogen oxides (NO_x), particulate matter that has a diameter of 10 µm or less, and carbon dioxide (CO_2). Only (CO_2) emission is evaluated using Joumard et al. (2007), and the rest of the emissions are evaluated using Bennett and Greenwood (2001):

$$TPE_{i,j} = EOE_{i,j} * CPF_{i,j} \tag{3.33}$$

$$ER_j = \frac{1}{8.64 * 10^{10}} * \sum\nolimits_i TPE_{i,j} * a_i * AADT \tag{3.34}$$

where $TPE_{i,j}$ is the tailpipe emission (g/km), $EOE_{i,j}$ is the engine emission (g/km), $CPF_{i,j}$ is the catalyst pass fraction, ER_j is the immediate emission rate for tailpipe emission (µg/s/km), $8.64*10^{10}$ is a conversion factor to convert (g/day/km) to (µg/s/km), and *i* and *j* are the vehicle class and emission types, respectively.

After estimating the emission rates, the emission concentration must be evaluated. Atmospheric emissions are placed in two categories: (1) short-range emissions (e.g., PM_{10} and $PM_{2.5}$), and (2) long-range emissions (e.g., GHG). On the one hand, short-range emissions are the only required dispersion estimations because the severity of their impacts depends on the emissions concentration at the receptor location. On the other hand, the impact of long-range emissions depends on the global concentration of the gas. Hanna et al. (1982) showed that the plume dispersion model can be used to estimate short-range emissions since highway traffic is considered to be a continuous linear source of emission. The additional concentration due to traffic is estimated as shown below (Venkatram et al., 2006):

$$c_j(x^{eff}) = \sqrt{\frac{2}{\pi}} * \frac{ER_j * 10^3}{U * cos\theta * \sigma_Z(x^{eff})} \tag{3.35}$$

$$\sigma_Z(x^{eff}) = 0.14 + (1 + 0.0003 * x^{eff})^{-1/2} \tag{3.36}$$

where $C_j(x^{eff})$ is the traffic additional emission concentration j (µg/m^3), x^{eff}effective is the receptor-highway downwind distance (m), U represents the average wind velocity (m/s), Θ is the wind-blow angle to the highway, and $\sigma_Z(x^{eff})$ is the parameter of the vertical dispersion (Hanna, 1982).

The impact of atmospheric emissions has three major dimensions: human welfare, building and infrastructure, and corps (Pellecuer, 2014a). Several health outcomes are included to monetize the effect of atmospheric emissions on human health (Table 3.1). The annual number of additional cases impacted by air pollution are estimated as follows (Künzli et al., 2000):

$$N_{j,h} = CRF_{j,h} * C_j * N_h * P \tag{3.37}$$

where $N_{j,h}$ is the annual number of additional cases caused by traffic induced emissions, $CRF_{j,h}$ is the concentration response function (µg^{-1} . m^3), N_h is the base annual number of cases, P is the population number affected by emissions from the traffic stream, and j and h are the types of emission and health outcomes, respectively.

The total cost of air pollution is comprised of (1) damage costs caused by global warming; (2) costs from biodiversity loss; and (3) cleaning and renovation costs due to building façade soiling and erosion (Pellecuer, 2014a).

$$\begin{aligned} APC = \sum_h AHC_h * N_h + ADC_{CO_2} * 31.536 * ER_{CO_2} * L \\ + \sum_j BLC_j * C_j + BDC_{PM} * 31.536 * ER_{PM} * L \end{aligned} \tag{3.38}$$

where APC is the air pollution cost (\$), AHC_h is the cost of the health outcome (\$/case, in 2017 constant Dollars), ADC_{CO2} is the additional cost of CO_2 (\$/g, in 2017 constant Dollars), ER_{CO2} is the CO_2 emission rate (µg/s/km), L is the highway section length (km), BLC_j is the cost of biodiversity loss related to emissions concentration change (\$/ton, in 2017 constant Dollars) (Table 3.2), BDC_{PM} is the cost of building damage due to the change in the concentration of PM per ton of PM (\$/ton, in 2017 constant Dollars), ER_{PM} is the PM emission rate (µg/s/km), 31.536 is a factor used to convert (μg/s/km) to (g/year/km), and *j* and *h* are types of emission and health outcomes, respectively.

The change in fuel consumption arising from alterations in the pavement condition is a major input factor of air pollution savings of M&R treatments. Equation (3.9) can be used to evaluate the expected air quality savings resulting from pavement condition improvement.

Table 3.1: The average costs of each case of pollution-related health outcome (2017 constant US dollars)

Health Outcome	*AHC*	
Mortality	$	17,867,251
Respiratory Hospital admission	$	5,613
Cardiac Hospital Admission	$	6,949
Respiratory emergency visit	$	4,282
Cardiac emergency visit	$	5,880
Restricted Activity day	$	359
Asthma Symptom day	$	127
Acute respiratory symptom day	$	104
Adult Bronchitis case	$	569,492
Child Bronchitis case	$	664

Table 3.2: Other air pollution costs (2017 constant US dollars)

	Average Cost	
ADC_{CO2} ($/ g)	$	698
BLC_{NOx} ($/ton)	$	1,616
BLC_{SO2} ($/ton)	$	323
BDC_{PM} ($/ton)	$	398

3.6.3.2. Noise Pollution Cost

People working or living in the vicinity of a highway may be vulnerable to the adverse impact of traffic noise. Excessive exposure to traffic noise for a long period of time causes serious health problems that negatively affect quality of life (Rasmussen et al., 2007). Safe noise levels should not exceed 70 db(A) for 24 hours, according to the World Health Organization (WHO, 2000). Permanent damage, even hearing loss, is expected when the noise level reaches 85 db (A) or higher (Loss, 2015). Several countermeasures can be implemented to mitigate the noise level, such as noise barriers that provide a buffer zone and pavements with a quieter design. Of all alternative techniques, a noise barrier is the most common mitigation method. The effect of this countermeasure is evaluated in this dissertation.

Traffic noise is evaluated using the Barry and Reagan (1978) equation. First, the basic reference energy mean noise emission level (REMEL) is evaluated for different vehicle classes, and then several adjustment factors including traffic flow, distance, finite roadway, and shield adjustment are examined (Sinha and Labi, 2007). The noise impact at any receptor position will follow Equation (3.39) if the distance between the centerline of the highway and the receptor exceeds 15 meters.

$$L_{eq}(h)_i = (L_0)_{E,i} + 10\ log_{10}\frac{N_i \pi D_0}{S_i T} + 10\ log_{10}\left(\frac{D_0}{D}\right)^{1+\alpha} + 10\ log_{10}\left(\frac{\varphi\alpha(\emptyset_1, \emptyset_2)}{\pi}\right) + \Delta_S \tag{3.39}$$

where L_{eq} represents the hourly equivalent sound level for the *ith* vehicle class, $(L_0)_{E,i}$ is the REMEL for vehicle class i, N_i is the number of vehicle of class i passing a point at time T, S_i is the average speed (km/h) for vehicle class i, T is the time required to measure L_{eq}, D represents the perpendicular distance from the traffic lane centerline to the receptor, D_0 is the reference distance from the source of emission, α is a parameter for the site condition, Ψ is the adjustment factor for the highway finite-length, and Δ_s represents the shield adjustment factor (if a noise barrier exists, dBA).

If the noise level of a given project exceeds the limits stipulated in the FHWA guidelines, noise mitigation actions are implemented according to the FHWA's noise abatement criteria (FHWA, 2011). The amount of noise reduction required varies from one state to the next. For example, INDOT established a goal to reduce noise by at least seven dBA for pavements designed after noise barriers were implemented in the state of Indiana (INDOT, 2011). Noise barrier walls are the most common abatement measures and they use various types of materials, such as concrete blocks and wood. If needed, the height of the barrier is determined by finding the hourly equivalent sound level for each vehicle class. The height of the noise barrier is the key input for estimating the cost of the noise barrier, which is one of the surrogate measures for noise pollution cost. The average unit costs for different building materials can be obtained from the literature, and cost can be presented in dollars per square foot or in millions per mile length of a noise barrier. For the purposes of this study, a statistical model was built based on historical data from several states where the cost, in millions, was a function of the noise barrier length in miles (Sinha and Labi, 2011):

$$Noise\ Barrier\ Cost = -0.7269 \times ln(Barrier\ Length) \tag{3.40}$$

Noise Pollution Savings (Benefits) Related to Pavement Roughness Improvement

Note that the noise level before and after implementing any overlay will be the same since Barry and Reagan's 1978 equation does not capture the effects of pavement condition on noise level. Very few studies have investigated the relationship between pavement condition and traffic noise. In one such study, a statistical model developed by Pozder (2012) relates pavement conditions to equivalent noise levels. The pavement condition rating (PCR) was the performance indicator used. The study concluded that noise level is degraded by 0.035 dB with a one point change in pavement surface (Pozder, 2012). The performance indicator of pavement condition used in this dissertation is IRI. The relationship between the pavement condition index (PCI), also known as PCR, and the IRI is shown below (Arhin et al., 2015):

$$\mathrm{PCI} = -0.215\ (\mathrm{IRI}) + 110.73 \tag{3.41}$$

Based on this relationship, the reduction of noise level during the life cycle of a certain treatment is evaluated. The cost associated with the noise level can be indirectly evaluated since the relationship between pavement age and noise cost is established (Pellecuer, 2014a) (Pellecuer et al., 2014b). The noise level is estimated through the traffic noise model (TNM) 2.5 developed by FHWA and based on Barry and Reagan's Equation (3.39). Noise mitigation action is needed if the noise level exceeds the allowable levels. However, an adjustment factor is required since TNM does not incorporate the aging effect of the pavement surface on the noise emission (Bendtsen et al., 2010):

$$\Delta L=0.25*\Delta L_{Age}+A+\left[\frac{0.75*\Delta L_{ADT}*AADT}{10^{6}*N}\right] \tag{3.42}$$

where ΔL is the adjustment factor related to pavement aging (ΔL= 0.4), ΔL_{Age} is the increase in the noise level of the age component (dB/year), A is the pavement age (years), ΔL_{ADT} is the increase in the noise level of the traffic load component (ΔL_{ADT} = 0.21, in dBA/year), *AADT* represents the average annual daily traffic (both directions), N stands for number of lanes, $E_{0_{i,s}}$ is the sound energy of vehicle class i (from TNM in J), and $E_{i,s}$ is the sound energy when the pavement aging effect is considered (J) (Bendtsen, 2010).

The impact of noise variation during any given hour of the day is estimated by using the following equation

$$TE_{k}=\sum_{i} 0.0476*E_{i}*\frac{a_{i}*b_{k}*AADT}{v_{i}} \tag{3.43}$$

where a_i represents the percentage of vehicle class i in the traffic stream (a_{Auto}=38.324 and a_{Truck}= 0.227), b_k is the proportion of AADT for the *kth* hour, and v_i is the vehicle speed (m/s) (Pellecuer, 2014a).

Assuming that the noise source is 15 m from the receptor, the estimated noise level of the *kth* hour is L_{0_k}. Also, the noise level is adjusted to take into consideration the effect of the distance between the highway and adjacent houses as well as the impact of certain ground characteristics on noise propagation, as shown in Equation (3.45) (Menge et al., 1998).

$$L_{0_{k}}=10*\log(TE_{k}) \tag{3.44}$$

$$L_{k}(x)=L_{0_{k}}+10*\log\left(\frac{15}{x}\right)^{1+\varphi} \tag{3.45}$$

where x is the distance between the centerline of the highway to the receptor (in meters), and φ is the ground absorption factor (φ= zero).

This dissertation addresses the impact of traffic noise on human beings in two dimensions: its effect on human health and the amount of annoyance (Pellecuer, 2014a). Traffic noise may lead to a number of health consequences: (1) myocardial infraction; (2) angina pectoris; and (3) hypertension (Davies et al., 2012; Staatsen et al., 2004). The average noise level (L_{den}) is the indicator used to evaluate the effect of traffic noise day and night on health outcomes. Then, the number of noise emission-related cases for each health outcome N_i is evaluated

$$L_{den} = 10 * log\left\{\frac{\sum_{k=1}^{24} 10^{\frac{L_k(x)+P_{den_k}}{10}}}{24}\right\} \tag{3.46}$$

$$N_i = a_i * (L_{den} - b_i) * \frac{P}{1000} \tag{3.47}$$

where L_{den_s} is the average noise level (dB), $\mathrm{P_{den_k}}$ represents the hour of the day, P represent the number of people exposed to traffic noise, and a_i and b_i are parameters adopted from Staatsen et al. (2004).

The degree of severity of traffic noise is represented as the percentage of people who are exposed to different levels of annoyance (low, medium or high) (Miedema et al., 2001):

$$AP = a_{AP} * (L_{den} - L_{AP})^3 + b_{AP} * (L_{den} - L_{AP})^2 + c_{AP} * (L_{den} - L_{AP}) \tag{3.48}$$

where AP is the percentage of individuals lightly annoyed, annoyed or highly annoyed by traffic noise per year (%), and a_{AP}, b_{AP}, c_{AP}, and L_{AP} are parameters presented by Miedema and Oudshoorn, (2001). Health effects and annoyance caused by traffic noise are monetized based on the following equation (Pellecuer, 2014a):

$$NPC = \left\{\sum_i HC_i * N_{s,i} + \delta * \sum_{AP} IHC_{AP} * AP\right\} \tag{3.49}$$

where NPC is the noise pollution cost estimation (\$), HC_h is the cost of one case of health outcome (in 2017 constant Dollars), δ is a binary coefficient (0 when L_{den} is higher than 70 dB, and 1 otherwise), and IHC_{AP} is the cost of an individual if lightly annoyed, annoyed or highly annoyed per year (in 2017 constant Dollars) (Bickel et al., 2006).

Table 3.3: The average costs of each case of health outcome (in $2017 constant US dollars)

Myocardial infarction:		
Fatal, years of life lost (YOLL)	$	3,315,438
Non-fatal, days in hospital	$	64,411
Angina Pectoris:		
Days in hospital	$	33,067
Hypertension:		
Days in hospital	$	3,212
Sleep disturbance road traffic	$	1,406

Source: Bickel et al., 20016

Table 3.4: Annoyance valuations (in $2017 constant US dollars)

		IHC
Lightly Annoyed	$	82
Annoyed	$	187
Highly Annoyed	$	187

These costs are estimated based on willingness to pay and willingness to accept compensation approaches. The cost estimation is based on the effect of several health problems on those living in the vicinity of the roadway. Benefits are estimated by evaluating the reduction in noise cost through certain treatments. In other words, benefits are estimated by evaluating the difference between the noise cost before and after applying the maintenance or rehabilitation treatment during normal operations, as presented in Equation (3.10).

3.7. Cost-Effectiveness Analysis

3.7.1. Non-Monetized Values of Effectiveness

This dissertation uses non-monetized values of effectiveness: the estimated life of rehabilitation treatments, the increase in average performance of pavement over the treatment life, and the increased area bounded by the pavement performance curve due to treatment. The area bounded by the performance curve and the threshold line encompasses both of the effectiveness concepts of (1) the average performance of the pavement after it has received the treatment and (2) the treatment life. As such, this measure is the most appropriate for assessing M&R effectiveness. Only the areas of the non-decreasing performance indicators are presented in this dissertation as a measure of benefits:

$$Benefit = AOC_{NC} + \sum_{s=1}^{m} \sum_{i=1}^{n} x_{s,i} * [AOC_{s,i}] \qquad (3.50)$$

where AOC_{NC} is the additional area over the performance curve generated by the new construction (or do-nothing scenario) treatment and bounded by the performance indicator threshold, $AOC_{s,i}$ is the additional area over the performance curve generated by treatment *s* and bounded by the performance indicator threshold, *m* is the number of alternative M&R treatments, *n* is the number of possible times preservations may be triggered during the analysis period, and $x_{s,i}$ is the number of maintenance/rehabilitation treatments.

3.7.2. Life Cycle Cost Analysis

Agencies are moving toward adopting the concept of pavement life cycle in their planning and budgeting processes for future pavement investments. The concept of LCCA has increasingly been used to determine pavement effectiveness and to identify the appropriate pavement treatments (Darter et al., 1985; Ozbay, 2003; Ravirala, 2002; Walls and Smith, 1998; Zhang, 2012). The purpose of this dissertation is to establish a comprehensive framework of all the costs that might be incurred by the transportation facility's stakeholders. The stakeholders include the facility owner (often a government agency), transportation users, and the community surrounding the highway facility. All costs related to the agency, the users, and the surrounding community are considered when the overall cost of a certain highway investment is evaluated. The decision regarding whether to initiate highway construction, maintenance, or rehabilitation is typically made by the transportation agency. This seems to suggest that the agency cost is more important to the decision-maker than the user and community costs.

The present worth cost (PWC) converts all the costs expected in the future to their equivalent values in target year dollars; equivalent uniform annual cost (EUAC) is used to denote the annual incurred cost over the life of a project. The equation for the present worth factor (PWF) and the capital recovery factor (CRF) are presented in Equation (3.51) and (3.52):

$$\mathrm{PWF} = \frac{1}{(1+\mathrm{i})^{\mathrm{N}}} \tag{3.51}$$

$$CRF = \left(\frac{i\,(1+i)^N}{(1+i)^N - 1}\right) \tag{3.52}$$

where *i* is the discount rate and *N* is the number of years from initial construction.

In its 2013 design manual, INDOT defined salvage as “the construction cost of the last cycle times the ratio of the remaining service years to the last cycle design life.” The salvage value can be calculated using the following equation:

$$SV = \$ \cdot \left(\frac{RL}{DL}\right) \tag{3.53}$$

where *$* is the construction cost of the last cycle, *RL* is the remaining service life (years), and *DL* is the design life of the last cycle in years. This dissertation uses this equation to account for the pavement salvage value.

The total cost of a certain M&R activity profile is presented below:

$$\begin{aligned} Cost = \; & (w_{AC} * AC_{new} + w_{UC} * (WZUC_{TTD} + WZUC_{VOC})_{new} + w_{CC} \\ & * CC_{new}) \\ & + \sum_{CT=1}^{m} \sum_{i=1}^{n} X_{CT,i} \\ & * \left[PWF_{i\%,Nt(i)}\right. \\ & * \left(w_{AC} * w_{AC} * AC_{CT,i} + w_{UC}\right. \\ & \left.\left. * (WZUC_{TTD} + WZUC_{VOC})_{CT,i} + w_{CC} * CC_{CT,i}\right)\right] \end{aligned} \tag{3.54}$$

where AC_{new} is the total agency cost of the new construction, $WZUC_{TTD}$ is the travel time delay cost at the work-zone for the new/re-construction, $WZUC_{VOC}$ is the VOC at the work-zone for the new/re-construction, CC_{new} is the total community cost for the new/re-construction (evaluated by adding the cost of the noise barrier, if needed), $AC_{CT,i}$ is the total agency cost for candidate treatment candidate treatment *CT* (*CT* = 1, 2, 3, …*m*) of stage *i* (*i* = 1, 2, 3, …, *n*), $WZUC_{TTD}$ represents the travel time delay cost at work-zone for treatment *CT*, $WZUC_{VOC}$ is the VOC at work-zone for treatment *CT*, CC_{new} is the total community cost for treatment *CT*, $x_{CT,i}$ is the binary integer

(0 or 1) when treatment CT is applied at stage i, and w_{AC}, w_{UC}, and w_{CC} are the assigned weights for agency, user, and community costs.

3.7.3. Cost-Effectiveness (Benefit)

By quantifying both the cost and benefit of a project, decision-makers can make their choices based on a solid foundation. Several methods for determining the cost-effectiveness of a project are available, starting with calculating the simple difference between the total costs and benefits. Units ought to be the same for both costs and benefits. NPV and EUAC are other measures of cost-effectiveness for which all units must be monetary. These methods are appropriate for evaluating cost-effectiveness when the benefits are expressed in a monetized form. When the cost and effectiveness are expressed in different units, benefit-cost (B/C) ratio is the suitable method of evaluation (e.g., a ratio of benefits, represented by AOC or area to LCC, represented by EUAC or dollars). The B/C ratio is the evaluation criteria used in this study because it can accommodate both monetized and non-monetized benefits. A number of past studies have used the B/C ratio to evaluate non-monetized benefits (Irfan, 2009; Labi et al., 2005; Labi et al., 2005; Morian, 2003; Peshkin et al., 2004).

$$\text{Incremental Benefit/Cost Ratio} = \frac{\text{Incremental Benefits (Monetized or Nonmonetized)}}{\text{Incremental Costs}} \qquad (3.55)$$

3.8. Methods for Establishing the Weights of Different Cost Types

There are two key aspects of the multi-criteria decision making process: (1) establishing weights for the performance criteria and (2) scaling the performance criteria (Sinha and Labi, 2007). Weighting assigns relative weights to each evaluation criterion to reflect its importance compared to other criteria. Scaling establishes a common unit so that all performance criteria can be expressed in the same units, enabling comparison between different alternatives. Agency, user, and community costs are expressed in monetary values (dollars), but not all of them are equally important to the decision-maker. Therefore, different weights should be assigned to each cost according to its importance, and a weighting technique is adopted for this purpose. Weighting methods include equal weighting, direct weighting, the Delphi approach, the gamble method, pairwise comparison, and value swinging. Although other methods are available, only a combination of the Delphi approach and the direct weighting and the pairwise comparison are used in this dissertation.

Direct Weighting and the Delphi Approach: For the direct weighting method, the decision-maker directly assigns a numerical weight to each performance criteria, using two different approaches. The first approach is called point allocation, where the decision-maker assigns a weight on a scale of 0 to 10, 0 being the least important and 10 being the most important. Scaling is the other method of direct rating, which involves the simple ordering of performance criteria according to their importance. The Delphi approach is applied after conducting the first iteration, where an assessment of the responses to a survey is implemented and the mean and standard deviation are developed for each question in the survey. The survey is then distributed again to the same respondents, and each question is supplemented with the mean and standard deviation resulting from the previous survey. Normally, the standard deviation is expected to be lower in the second iteration. This process takes at least two rounds to achieve stable values (Dalkey et al., 1963).

Pairwise Comparison using the Analytical Hierarchy Process: A common tool for pairwise comparison of performance criteria is the analytical hierarchy process (AHP) developed by Saaty (1977). This process is a powerful method for guiding the decision-maker through a complex decision-making process. The method requires illustrating the decision-maker’s preferences as ratio values on a well-defined scale, as shown in Table 3.5. The procedure is important for finding pairwise comparison matrices that are required to be consistent. The AHP method also can be enhanced using the Delphi technique (Sinha et al., 2009).

Table 3.5: Pairwise comparison matrix

Comparison	A to B Ratio
A is ***extremely more*** important than **B**	9
A is ***strongly more*** important than **B**	7
A is ***moderately more*** important than **B**	5
A is ***slightly more*** important than **B**	3
A is ***equally*** important to **B**	1
A is ***slightly less*** important than **B**	1/3
A is ***moderately less*** important than **B**	1/5
A is ***strongly less*** important than **B**	1/7
A is ***extremely less*** important than **B**	1/9

3.9. M&R Scheduling

Implementing maintenance and/or rehabilitation treatments at optimal times extends pavement service life and improves ride quality for roadway users. Historically, M&R strategies have been based on different approaches and factors, such as expert opinion, existing (historical) M&R practices, and analytical methods. Expert opinion is normally discovered through surveys and is considered subjective and inconsistent (Gschösser et al., 2013; Lamptey, 2004; Lamptey, 2005; Walls, 1998). Despite the convenience of simply continuing existing (historical) M&R practices, changes in polices or funding may easily influence practices and yield methodologies that are not cost-effective (Lamptey et al., 2005). At the project level, several studies have adopted numerical optimization methodologies to determine the optimal types of treatments to be applied and the exact timing of such treatments that will prolong pavement service life (Friesz et al., 1979; Irfan, 2012; Lamptey et al., 2005; Markow et al., 1985; Mamlouk and Zaniewski, 2000; Peshkin, 2004; Zaniewski, 1996). The reason for developing an optimal profile of preservation activities is to maximize the return on investment, with benefits expressed as an extension of the service life of the asset, which can be accomplished by using funds and resources efficiently. Therefore, this dissertation focuses on project-level optimization of M&R schedules using agency, user, and community costs.

An integer programming technique (a common method of optimization in transportation applications) is used when all or some of the design variables ought to be integers. An optimization methodology at the project level was developed by Lamptey et al. (2004) to schedule preventive maintenance treatments between two major rehabilitation treatments. Integer programming formulation was used in this study to compare different maintenance alternatives by maximizing both the agency cost and the user cost. Simultaneously, pavement condition was maintained at specified performance levels and within certain budgetary constraints. Irfan et al. (2012) developed an optimal schedule of M&R activities by analyzing the agency and user costs at the project level. The formulation used was based on a mixed-integer nonlinear programming technique that accounted for the non-integer variables and overcame the presence of nonlinear objective functions and/or constraints.

The optimal solution in a dynamic programming technique is comprised of several partial solutions for every sub-problem considered. This optimization technique splits a complex problem into a set of smaller and simpler problems (Winston et al., 2003). Dynamic optimization models have been used to optimize maintenance activities based on agency and user costs while the objective function maximizes the utility associated with the road user (Friesz, 1979). Using dynamic optimization techniques, a model for deriving the pavement's optimal initial thickness and the thicknesses and timing of subsequent overlays was developed by minimizing the costs (agency and user) while maintaining the pavement condition at the desired level (Mamlouk and Zaniewski, 2000).

The genetic algorithm (GA) is a heuristic search method used to find optimal solutions in which the function value is the only form used in the search process. GA does not require evaluation of the function's derivatives, nor does it require the function to be continuous. The algorithm is applicable to all types of design variables: discrete (integer or binary), continuous, and non-differentiable. Unlike a derivative-based optimization algorithm, GA determines the near-global optimum solutions. Although GA does not guarantee exact global optimal solutions, this issue can be resolved, to some extent, by executing the algorithm a number of times and by allowing it to run for a longer period of time (Arora, 2012). At the network level, Fwa et al. (1996) and Pilson et al. (1999) developed approaches for finding the optimal preservation treatments (M&R) in light of costs (agency and user) using a genetic algorithm. More recently, an integrated system of maintenance, repair, and rehabilitation was presented by Liu et al. (2017) to optimize agency and user costs without violating certain pavement condition thresholds; they used a GA optimization tool both for network and project-level problems (Zaniewski & Mamlouk, 1996a).

3.10. Problem Formulation

A number of M&R strategies have been established based on cost-effectiveness. Mathematical optimization methods have been used at both the network and project levels. One of the objectives of this dissertation is to develop an optimal profile of M&R activities for flexible and rigid pavements. To this end, a cost-effectiveness analysis is conducted based on the optimal profile of each pavement type to evaluate the most cost-effective pavement material type. Several studies have assessed the optimal M&R schedules at the project level using agency cost only (Abaza, 2002; Pilson, 1999) or using both agency and user costs to analyze M&R activities (Friesz, 1979; Irfan, 2012; Geoffery Lamptey, 2005; Markow, 1985; Mamlouk et al., 2000; Peshkin, 2004). The purpose of this dissertation is to optimize the M&R schedule at the project level using the agency, user, and community costs. Specific trigger values are assigned to initiate rehabilitation M&R activities; for example, see Figure 3.8, in which two treatments are implemented over the pavement service life. The example presented in this figure shows that the optimization problem consists of two stages. Identification of these stages is the question of interest. The proposed optimization method can be adjusted to any number of stages, based on the planner’s judgment. Flexible and rigid pavements are expected to last no more than 30 years, which makes it unlikely that they will receive more than two major preservations during their service lifetimes. For this reason, strategies having up to three stages are presented in this dissertation.

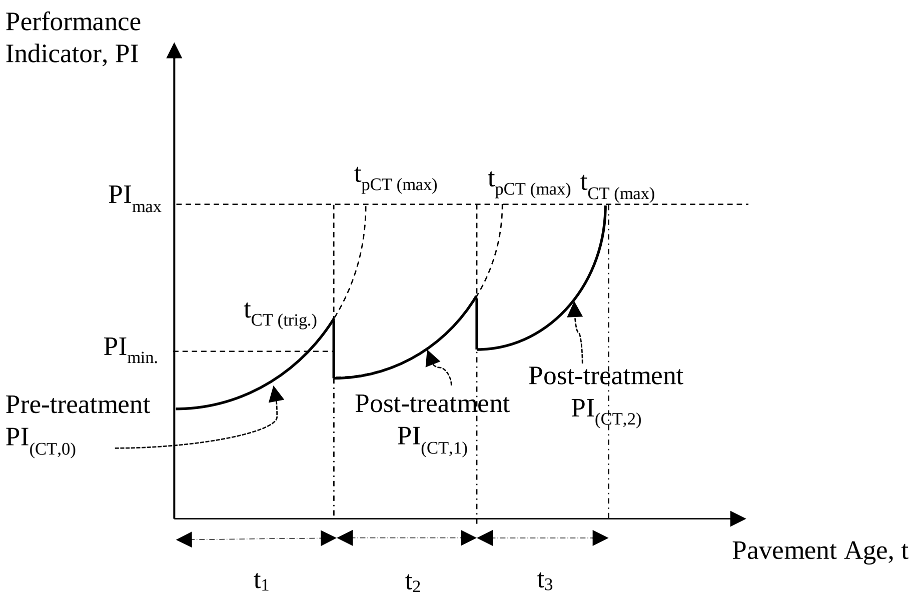


Figure 3.8: Treatment preassigned-trigger values based on given PI

Based on common practice, no major preservation (preventive maintenance or rehabilitation) is expected to be implemented in the first few years after new construction. This is represented as t_1 in Figure 3.8. The performance threshold that the pavement is not allowed to exceed (PI_{max}) is designated based on agency standards. $PI_{CT,0}$ is the new construction (s) performance at time t, and $PI_{CT,i}$ represents the pavement performance after implementing candidate treatment CT at time t. The time $t_{CT(trig)}$ is triggered by the pre-treatment performance curve for the succeeding treatment CT, while $t_{pCT(max)}$ represents the time it took treatment CT to reach the maximum allowable performance (PI_{max}). Treatment life is simply defined as the time between two consecutive preservation treatments, and t_i is the treatment service life of (t-1).

Pavement roughness is the performance measure adopted in this formulation; and a non-decreasing performance measure called the international roughness index (IRI) is used as the performance indicator. A few assumptions are made regarding the triggering of M&R activities:

1. No treatment is required when IRI is less than PI_{min} (Figure 3.8).
2. Each profile of M&R activities should start with a new construction activity (Stage 1 in Figure 3.8) followed by one or more preventive M&R treatment(s).
3. At the end of each stage, a M&R treatment may be implemented.

4. Preventive maintenance is a candidate treatment to be applied only when IRI is between PI_{min} and a PI value that is specified by an individual agency's standards.
5. When the pavement reaches a deterioration rate at which preventative maintenance is no longer an effective treatment, a rehabilitation activity should take place.
6. A decision regarding treatment must be reached at each stage (in this case, Stages 2 and 3 in Figure 3.8).

The decision to choose a certain treatment is made based on performance models that are functions of traffic, climate, and pavement age. The total cost of each treatment also is considered because the objective function is presented in terms of cost-effectiveness (in this case, the B/C ratio). All treatment decisions at all stages are optimized by maximizing the cost-effectiveness, as shown in Equation (3.56). The final decision is made based on two criteria: (1) finding the right year for treatment to be applied and (2) finding the appropriate treatment to apply.

The decision variables are related to the performance indicator, PI, models of the treatment at a pre-determined threshold. Each treatment has a unique benefit function, and the PI monitors its performance until it reaches the threshold. The area bounded by the curve and the threshold is the method used to evaluate this benefit. Also, every treatment has its own cost function (or average value). Taking into consideration both the benefits and the costs of each treatment applied leads to different overall outcomes. The decision variables can be expressed as the time, t, it takes a treatment to reach the threshold PI. This is can be simply evaluated since the relationship between the pavement PI and time are given in Equations (3.17) to (3.20). The treatment PI threshold (or time) are considered continuous variables while the type of treatment, the other decision variable X shown in Equation (3.56), is considered an integer variable.

The objective function for this optimization formulation was defined based on the cost-effectiveness economical concept. The objective is to reach to the optimal schedule of treatment activities by considering both the costs and benefits of each treatment applied. The overall benefit is measured by evaluating the area bounded by the performance curve, and the cost component is comprised of the agency, user, and community costs. Each treatment cost component is brought to the initial year of the analysis through the present worth method, and the area bounded by the performance curve and threshold is the surrogate for the benefit of the treatments for the entire analysis period. The measures used to maximize the cost-effectiveness must not violate the lower and upper bounds of the pavement performance curves. These bound values are assigned by the agency according to their own specifications. The objective function implicitly contains the additional performance constraints of minimum treatment performance level, the absolute performance threshold of each stage, and the type of treatment. The first constraint, shown in Equation (3.57), is used to ensure that no treatment for the newly constructed pavement reaches a condition that is superior to $PI_{\min}$. Equation (3.58) is a constraint ensuring that the performance of all treatments, including the performance of new construction, will not exceed the absolute performance threshold of PI_{max}. The constraint in Equation (3.59) indicates that only one treatment should be chosen at the end of each stage *i*. The objective function and the constraints are presented below:

$$Maximizing\ f(x) = \mathrm{z} = \frac{Benefit}{Cost}$$

$$Z = Max \frac{AOC_{new} + \sum_{CT=1}^{m}\sum_{i=1}^{n} x_{CT,i} * \left[AOC_{CT,i}\right]}{\left\{\begin{matrix}(w_{AC} * AC_{new} + w_{UC} * (WZUC_{TTD} + WZUC_{VOC})_{new} + w_{CC} * CC_{new}) + \\ \sum_{CT=1}^{m}\sum_{i=1}^{n} X_{CT,i} * \left[PWF_{i\%,Nt(i)} * \begin{pmatrix} w_{AC} * AC_{CT,i} + w_{UC} * (WZUC_{TTD} + WZUC_{VOC})_{CT,i} + \\ w_{CC} * CC_{CT,i}\end{pmatrix}\right]\end{matrix}\right\}} \tag{3.56}$$

Subjected to:

$$PI_{CT,t}\left(\bar{X}_{CT,0\ldots t-1}\right) \geq PI_{min},\ \forall s, t \tag{3.57}$$

$$PI_{CT,t}\left(\bar{X}_{CT,0\ldots t-1}\right) \leq PI_{max},\ \forall s, t \tag{3.58}$$

$$\sum_{CT=1}^{m} X_{CT,i} = 1, \forall i\ \ t \tag{3.59}$$

As discussed earlier, the benefit is determined by evaluating the area bounded by the performance curve and the pre-determined threshold. The performance indicator used is IRI, which is a non-decreasing performance measure. This means that the area of interest is the area over the performance curve and below the assigned threshold. The area in question can be calculated using the following equations:

$$AOC_{new} = PI_{max} * t_{new} - \int_{0}^{t_{new}} PI_{new,t} dt \tag{3.60}$$

$$AOC_{CT,i} = PI_{max} * t_i - \int_{0}^{t_i} PI_{CT,t} dt \tag{3.61}$$

The performance indicator of a certain treatment is measured by the developed model for pavement roughness. Several exponential statistical forms have been developed, where the traffic loading, freeze index, and pavement age are the factors influencing pavement roughness. The model form presented below is an example of the PI function used in the optimization model:

$$PI_{new,t} = e^{\alpha_{new} + \beta_{new} * ATT * t + \gamma_{new} * FI * t} \tag{3.62}$$

$$PI_{CT,t} = e^{\alpha + \beta * ATT * t + \gamma * FI * t} \tag{3.63}$$

where $AOC_{CT,i}$ is the additional area over the performance curve generated by a candidate treatment CT (CT = 1, 2, 3, …m) and confined by the pre-determined performance indicator threshold (AOC_{new} is the area for the new construction), $PI_{CT,0}$ is the new construction performance indicator, $PI_{CT,i}$ is the treatment performance curve (model) of stage i (i = 1, 2, 3, …, n) for candidate treatment CT at year t (these are the post-treatment models using IRI as the PI), $PI_{new,i}$ is the treatment curve of the new construction (see equations presented in Tables 4.3 to 4.19), $x_{CT,i}$ is the binary integer (0 or 1) when treatment CT is applied at stage i, AC_{new} is the total agency cost of the new construction, $WZUC_{TTD}$ is the travel time delay cost at the work-zone for the new/re-construction, $WZUC_{VOC}$ is the VOC at the work-zone for the new/re-construction, CC_{new} is the total community cost for the new/re-construction (evaluated by adding the cost of the noise barrier, if needed), $AC_{CT,i}$ is the total agency cost for candidate treatment CT, $WZUC_{TTD}$ represents the travel time delay cost at work-zone for treatment CT, $WZUC_{VOC}$ is the VOC at work-zone for treatment CT, CC_{new} is the total community cost for treatment CT, and w_{AC}, w_{UC}, and w_{CC} are the assigned weights for agency, user, and community costs.

The optimal profile of M&R treatments for flexible and rigid pavements were evaluated at each one of the four climatic zones. This was done to set the groundwork for unbiased comparisons between the pavement types in terms of which one is the most cost-effective across different climatic zones. The optimal solution follows the deterministic approach for reaching the optimal profiles of M&R activities for both pavement materials. After the optimal profiles of activities are determined, deterministic and probabilistic analysis were conducted based on the deterministic optimal profile. The stochastic optimization method is not considered at this stage, and further discussion on this topic is presented in Chapter 5.

3.11. Optimization Solution Method

The objective function of the optimization problem in this dissertation is the ratio of the incremental benefit over the incremental cost. This is a function of the PI over the pavement service life. The objective function is a piecewise function, as shown in Figure 3.9, and is represented by more than one sub-function. For the same instant of time (at $t_{s(trig)}$), the function has two values of PI. The behavior of the piecewise function is determined based on the behavior of its sub-functions. Every sub-function has an interval that is a part of the main function domain. In the one-stage scenario, two functions are shown in the figure, and the same concept is applied to the two-stage scenario.

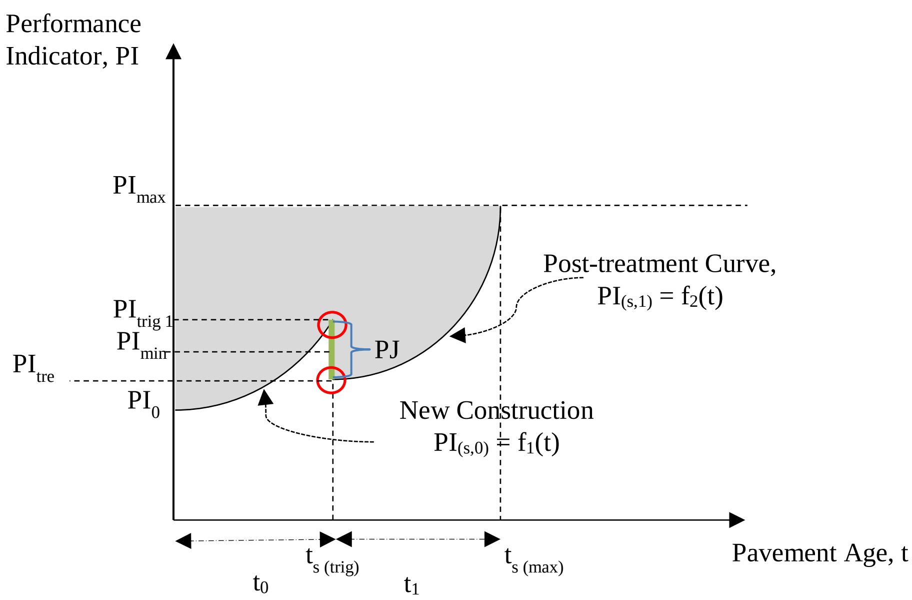


Figure 3.9: Pavement life cycle profile with single stage

The mathematical expression of the piecewise objective function regarding the performance indicator and time are presented as in equation 3.64 and 3.65.

$$F(PI) = \begin{cases} f_1(PI) & PI_0 \quad \leq PI \leq PI_{tirg} \\ f_2(PI) & PI_{tre} \leq PI \leq PI_{max} \end{cases} \tag{3.64}$$

$$(t) = \begin{cases} f_1(t) & 0 \leq t \leq t_{s(trig)} \\ f_2(\mathrm{t}) & t_{s(trig)} \leq t \leq t_{s(max)} \end{cases} \tag{3.65}$$

where PI is the performance indicator, PI is the performance indicator of a treatment, PI_0 is the performance indicator at (t = 0), PI_{trig} is the trigger value for applying a treatment, PI_{tre} is the performance indicator value immediately after implementing the treatment, PI_{max} is the pre-determined maximum (threshold) performance indicator, t represent the pavement age, $t_{(trig)}$ is the time where the treatment is implemented, and t_{max} is the time where the pavement reaches the PI threshold.

In order to determine the optimal performance indicator at which the treatment needs to be applied, the objective function must be differentiable. If a function is proven to be differentiable, then this function must be continuous; it is important to note, though, that not all continuous functions are differentiable. The continuity check is proposed because some of the non-differentiable functions, such as the one caused by discontinuity that could be eliminated, may become differentiable. The test of the piecewise function with an exponential form of the similar pavement performance model was carried out by Alinizzi (2017). His study proved that the discontinuity of the function is a jump discontinuity at the performance jump point, which means that the objective function, F(PI), is not differentiable at this point. However, the two sub-functions, f_1 and f_2, are differentiable at their defined domains because they are exponential functions. It is thus established that the exponential functions are differentiable. The proof that f_1 and f_2 are differentiable in their domain and concave functions is presented by Alinizzi (2017). This proves that the objective function has a global solution.

3.11.1. Optimization Problem Complexity

Scheduling M&R activities throughout the pavement's service life can be a complex task. The decision variable used in this dissertation is the pavement PI threshold where the best combination of M&R treatment(s) are needed at the optimal timings (or at the optimal PI thresholds). The complexity of the optimization problem is based on the number of candidate treatments, CT, to be applied at the optimal time, T (or PI threshold). The optimal profile of M&R activities is based on the combined cost-effectiveness of all treatments applied to a single scheduling activity profile. Suppose that a preservation treatment is needed at a certain stage where the PI reaches to a threshold value and there are four candidate treatments, some of which will last for 20 years. The possible number of solutions at this stage alone is CT^T (which is 4^{20} = 10,995,116,280 possible solutions) (Gao et al., 2009; Irfan, 2010; Morin et al., 1976). When the performance indicator's upper and lower bounds are applied to the optimization problem, the expected service life of the treatment will be reduced, and the number of the possible solution will become much lower. If a thin overlay is expected to perform for four years, based on a given threshold, then the number of possible solutions will be only 4^4, or 256 possible solutions. However, some of the rehabilitation treatment performance models developed in Chapter 4 have service lives of up to 25 years, and such treatments will have a large number of possible solutions. Such treatments are ignored, and only treatments with reasonable service lives are selected as candidate treatments for the optimization problem.

The existence of a global solution directed us to use a heuristic optimization method, which does not necessarily converge to optimal solutions. Heuristic (approximate) optimization methods may not reach exact optimal solutions, but they will provide a global optimal solution, which is sufficient for the scheduling problem at hand. In this dissertation, GA was used to identify the optimal profile (M&R treatments) for flexible and rigid pavements. When there are four different candidate treatments at one stage and each treatment has ten years of service life (4^{10}), GA is computationally efficient. GA is best used when the gradient of the objective function and/or the constraints cannot easily be evaluated. It can handle continuous, integer discrete, and categorical discrete variables and provide a near-global optimum (not a local minimum) (Taha, 2005). GAs have a robust search capability that allows them to handle pavement management system (PMS) problems with large numbers of constraints (Fwa, 1996).

The number of possible solutions grows exponentially when treatments last longer without violating the enforced performance thresholds. This will make the deterministic optimization problem computationally more complex. Although stochastic optimization accounts for the uncertainty involved in life cycle analysis and optimization inputs, adopting such a method would further complicate the problem. The pavement treatment performance models developed earlier are deterministic, and the pavement condition, measured in IRI, is a function of treatment age, traffic loading, and other environmental explanatory variables. One way to conduct stochastic optimization is to solve the presented scheduling problem by using probabilistic instead of deterministic models when evaluating pavement performance. In order to follow the suggested methodology in this dissertation, probabilistic models should be developed for each M&R treatment at each climatic zone. Although this method is outside of the scope of this dissertation, it is suitable for future research.

3.12. Sensitivity Analysis Using Deterministic and Probabilistic Approaches

Evaluating the cost-effectiveness of the optimal profile is the next step of the methodology proposed in this dissertation. Two methods are suggested: (1) a deterministic approach and (2) a probabilistic approach. The deterministic method includes an evaluation of the NPV or EUAC of the optimal profile of each pavement material based on predetermined inputs. Several inputs for the agency, user, and community costs are assumed based on common practice, such as using a 4 % interest rate for the LCCA calculations (INDOT, 2013). Additionally, the average unit costs (or cost models) of M&R treatments are used as a single input without considering the uncertainty incorporated into the costs when the costs were evaluated.

The probabilistic approach is used because of the uncertainty about input variables for the LCCA. The deterministic approach considers only one entry for each variable, but the approach can be enhanced by conducting a sensitivity analysis. Even with the use of a sensitivity analysis, however, the uncertainty of some entries still cannot be addressed. The uncertainty areas of a LCCA can be exposed using risk analysis. Risk analysis combines two methods: (1) the probabilistic entries of the uncertain input variables for LCCA and (2) computer simulation to capture the risk of the LCCA outcomes. The results are presented in a probability distribution that describes the range of the outputs. This approach enables the decision-maker to have a full range of all possible values along with the probability associated with each individual outcome. This approach has been adopted by the FHWA and includes the following steps: (1) problem identification; (2) quantification of the uncertain inputs using the proper probability distribution; (3) implementation of the computer simulation; (4) analysis of the results and inferences; and (5) decision determination (Walls and Smith, 1998). Further discussion of these five steps is presented in Chapter 5.

3.13. Chapter Summary

This chapter presented the methodology used in this dissertation, beginning with a discussion of data acquisition for flexible and concrete pavement materials using the LTPP database. As described, effectiveness is a way of quantifying benefits that can be evaluated using monetized and non-monetized approaches. The non-monetized MOEs include calculating the estimated life of the rehabilitation treatment, the increase in average performance of the pavement over the treatment life, and the increased area bounded by the pavement performance curve due to the treatment. These monetized benefits are quantified by evaluating the reduction (savings) in costs incurred by the agency and roadway users. Three cost categories are considered in this dissertation: agency, user, and community. The agency cost includes the costs of construction, rehabilitation, maintenance, and salvage. The user cost is the cost incurred by the road user during normal operations and in work zones. The work zones costs include travel time delay cost and VOC. Community costs consist of noise and air pollution costs. The noise cost is calculated as the cost of the noise barrier needed to mitigate the traffic noise, and the air pollution cost is estimated by monetizing the social damage associated with the global warming potential of greenhouse gas emitted during the LCA phases and the cost of energy consumed during this process. Cost-effectiveness is evaluated based on monetized and non-monetized effectiveness values amalgamated using the B/C ratio method. The optimal life cycle activity profiles were identified using the genetic algorithm technique.

CHAPTER 4. RESULTS OF THE EFFECTIVENESS AND COST ANALYSIS

4.1. Introduction

Pavement performance models are essential inputs for determining an optimal schedule of M&R activity treatments. The condition of the pavement can be evaluated before and after applying any preservation activity by using the proper MOE. A short-term MOE such as a performance jump was used for this purpose. Other MOEs used in this dissertation to assess long-term treatment benefits were treatment life, average performance of a pavement over its service life, and the area bounded by the pavement performance curve after treatment. To achieve the optimal schedule of M&R activities, cost models were estimated. This chapter presents the performance and cost models for all the flexible and rigid pavement M&R treatments, which are essential inputs to the optimization problem.

4.2. Data for the Pavement Performance Models

The main source of data for the performance jump and post-treatment performance trend models was the LTPP dataset. This dataset provided data on pavement conditions related to distresses such as crack severity and length, as well as on rut depth. In the LTPP dataset, the condition of each highway section is measured and recorded each year over several years. Some missing data were noticed; however, in situations where the IRI value for a specific year was missing due to limited resources or technical difficulties, a linear interpolation was assumed to find the missing data points. This technique was judged to be adequate for modeling purposes, instead of using the dataset with all of the missing data points. In some pavement sections, the IRI values do not exhibit a realistic representation of the pavement condition; for example, when the IRI values for three consecutive years are 85, 67, and 92 inches/mile. The sudden drop in pavement roughness (85 to 67 inches/mile) was unrealistic, especially given that the records did not show that any type of treatment was applied during that specific year. Such unlikely values were eliminated from the dataset to avoid contaminating the performance models.

4.3. Performance Model Results

This section reviews the results of the PJMs and PTPMs developed using the LTPP data for states in wet freeze, wet non-freeze, dry freeze, and dry non-freeze zones. For each treatment, maintenance, or rehabilitation activity, data from the states in the same climatic region were grouped so that one model would represent the performance of that treatment for all the pavements in that specific climatic zone. Performance models then were developed for flexible pavement maintenance (SPS 3) and rehabilitation (SPS 5) treatments as well as rigid rehabilitation (SPS 6) treatments. This dissertation focuses on preventive M&R treatments, where the effect of routine maintenance on pavement condition (in terms of IRI values) is assumed in most cases to be negligible, with a few exceptions. Only a few models were developed for the LTPP rigid pavement maintenance treatments (SPS 4) because (1) most of those treatments are localized treatments that have minimal effects on pavement roughness (e.g., joint/crack sealing, partial- and full-depth patching, etc.), and (2) in instances where several maintenance activities are applied to the same pavement section at the same time, it is difficult to evaluate the effect of the individual treatments on the pavement condition.

4.3.1. Performance Jump

Ideally, the pavement condition should improve after treatment implementation, which was the case with most of the sections examined in this study. However, a few pavement sections experienced sudden deterioration just after receiving a preventive maintenance treatment. This may have occurred because the pavements sections were in such poor condition that a thin overlay was not the right treatment to apply (rehabilitation may have been more appropriate) or because the construction methods used in these instances were poor. Such cases were eliminated from the PJM modeling dataset.

The number of samples was another issue of concern since only a few sections were available to build performance jump models (ten or fewer for maintenance and five or fewer for rehabilitation activities in the wet-freezing zone). In such instances, the data from all the climatic zones were aggregated to build one trend model for each individual treatment because some treatments had as few as nine observations, even after aggregating the data from different pavement sections in different climatic zones. This aggregation of the dataset was done only with pavement jump models. The results of both the performance jump models and the average decreases in IRI values (improvement in condition) are presented in Table 4.1 for flexible pavement M&R and in Table 4.2 for rigid pavement rehabilitation. The performance jump model forms were presented in Chapter 3 (see Equations 3.15 to 3.18).

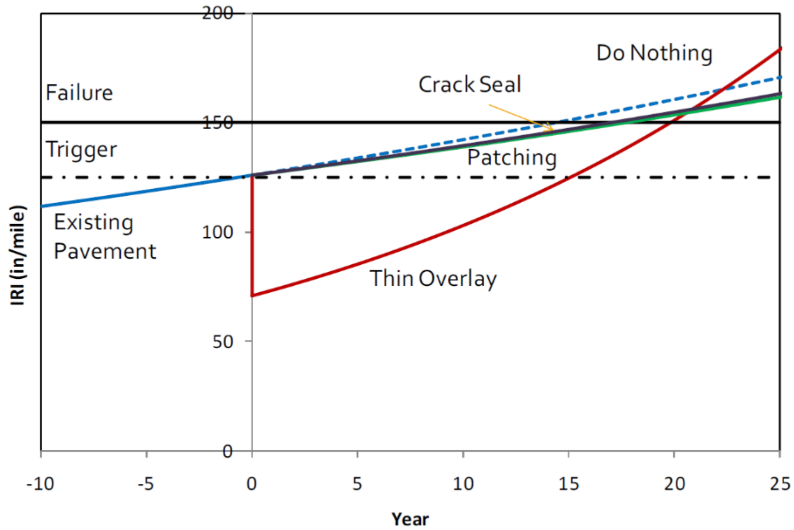


Figure 4.1: Performance of maintenance treatments

Preventive M&R activities are expected to provide considerable reduction in IRI, which translates to the performance improvement resulting from such treatments. Routine maintenance treatments are expected to improve pavement performance in the long term, but no performance jumps (sudden decrease in IRI value) are expected after such treatments. Figure 4.1 compares a thin overlay (a preventive maintenance treatment) to crack sealing and patching performance (routine maintenance treatment) (Ong et al., 2011). This figure shows a sudden decrease in IRI for thin overlay treatments, whereas other routine maintenance treatments show improvement in pavement condition without a sudden reduction caused by implementing these treatments.

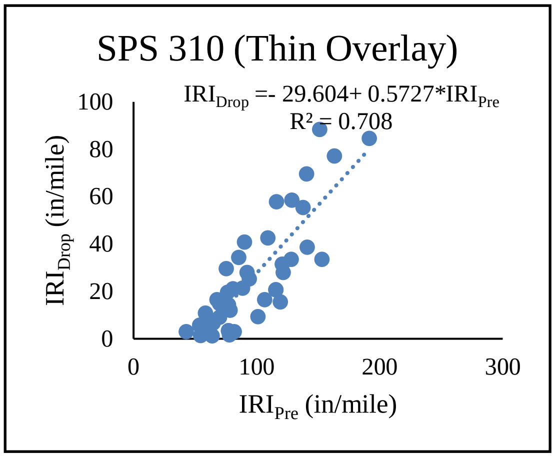
As expected, the average drop in IRI values (performance jump) for rehabilitation AC overlays (SPS 5) produced better performance improvement compared to maintenance treatments, as shown in Figure 4.1. The average IRI drop due to AC rehabilitation overlays was almost twice the IRI drop caused by thin overlay treatments. The relationship between the performance jump and the pre-treatment condition for thin overlays and other rehabilitation AC overlays (SPS 5) is shown in Figure 4.2. As mentioned above, the number of sections was too small to build robust performance jump models, and the models presented are intended to show only the trend of the treatments for the different pre-treatment pavement conditions. Because of the small number of samples in this dataset, the average decrease in IRI values was used as the performance indicator for the performance jump inputs in the optimization problem. The 8-inch AC overlay on top of an existing rigid pavement with cracked or broken and seated joints (SPS 608) had the best performance jump compared to the other 4-inch overlays. The other performance results of the rigid pavement rehabilitation activities are shown in Figure 4.3.

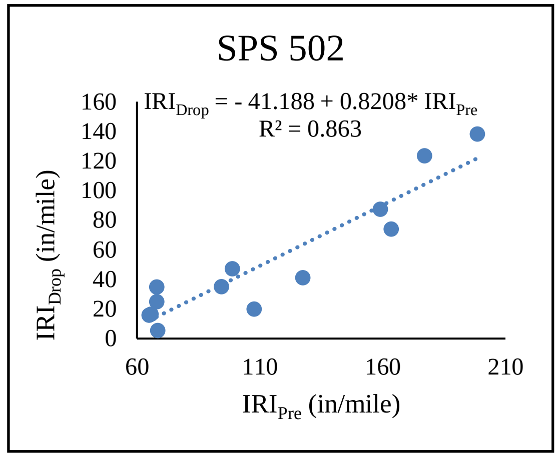
Table 4.1: Performance jump models for flexible pavements (SPS 3 and SPS 5)

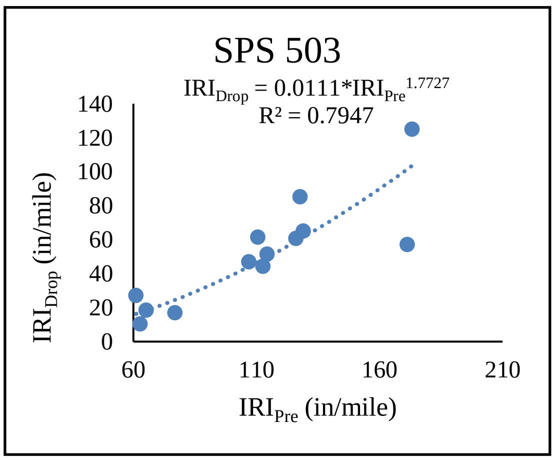
Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Average IRI_{Pre} (in/mile)	**Average IRI_{Drop} (in/mile)**
SPS 310	Linear	α	-29.604	-4.915	44	0.708	97.45	**26.02**
		β	0.573	9.784				
SPS 502	Linear	α	-41.188	-3.550	13	0.863	112.01	**50.25**
		β	0.821	8.318				
SPS 503	Power	α	0.011	0.789	13	0.795	110.58	**51.57**
		β	1.773	6.525				
SPS 504	Linear	α	-28.601	-2.949	13	0.878	110.18	**53.63**
		β	0.746	8.913				
SPS 505	Linear	α	-12.184	-0.763	13	0.567	100.54	**45.90**
		β	0.578	3.794				
SPS 506	Power	α	0.201	0.705	11	0.613	94.42	**45.25**
		β	1.183	3.774				
SPS 507	Exponential	α	11.484	5.217	12	0.845	102.36	**50.07**
		β	0.013	7.375				
SPS 508	Logarithmic	α	-301.748	-7.379	13	0.868	97.82	**44.48**
		β	76.319	8.486				
SPS 509	Linear	α	-29.965	-2.436	12	0.850	110.81	**56.54**
		β	0.781	7.536				

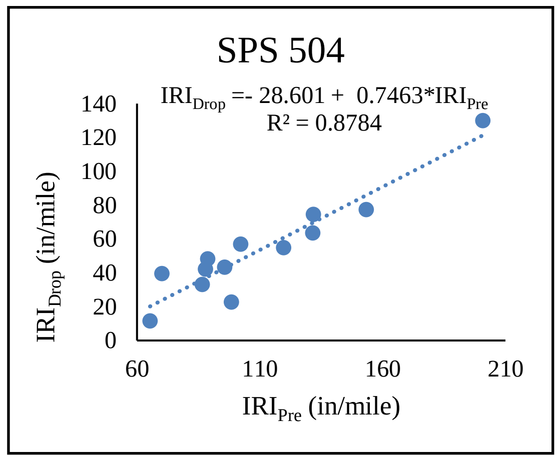
Table 4.2: Performance jump models for rigid pavements (SPS 6)

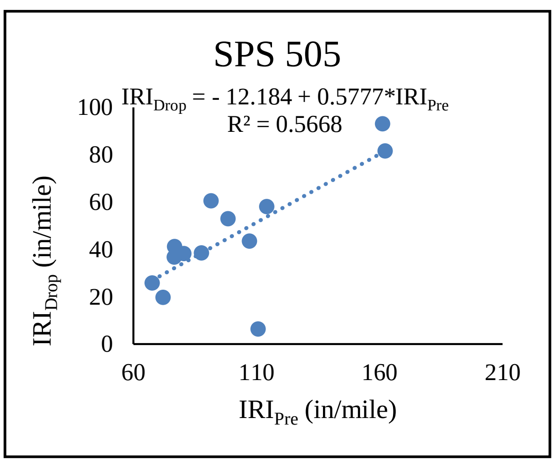
Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Average IRI_{Pre} (in/mile)	**Average IRI_{Drop} (in/mile)**
SPS 603	Linear	α	-54.267	-2.664	10	0.841	137.40	**75.47**
		β	0.944	6.499				
SPS 604	Exponential	α	16.942	8.053	10	0.945	139.22	**76.79**
		β	0.01	11.767				
SPS 606	Linear	α	-58.312	-2.886	9	0.871	141.16	**77.52**
		β	0.962	6.871				
SPS 607	Exponential	α	15.72	3.951	10	0.809	134.52	**69.21**
		β	0.011	5.819				
SPS 608	Logarithmic	α	-766.532	-16.202	10	0.976	142.14	**81.29**
		β	172.426	17.949				

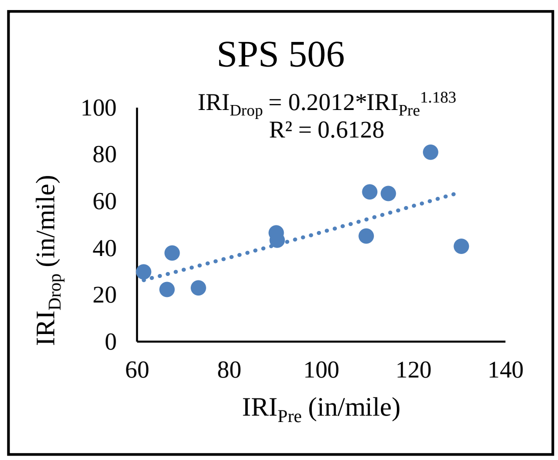

SPS 310 (Thin Overlay)
IRI$_{Drop}$ =- 29.604+ 0.5727*IRI$_{Pre}$
R² = 0.708
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)


SPS 502
IRI$_{Drop}$ = - 41.188 + 0.8208* IRI$_{Pre}$
R² = 0.863
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)


SPS 503
IRI$_{Drop}$ = 0.0111*IRI$_{Pre}^{1.7727}$
R² = 0.7947
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)


SPS 504
IRI$_{Drop}$ =- 28.601 + 0.7463*IRI$_{Pre}$
R² = 0.8784
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)


SPS 505
IRI$_{Drop}$ = - 12.184 + 0.5777*IRI$_{Pre}$
R² = 0.5668
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)


SPS 506
IRI$_{Drop}$ = 0.2012*IRI$_{Pre}^{1.183}$
R² = 0.6128
IRI$_{Drop}$ (in/mile)
IRI$_{Pre}$ (in/mile)

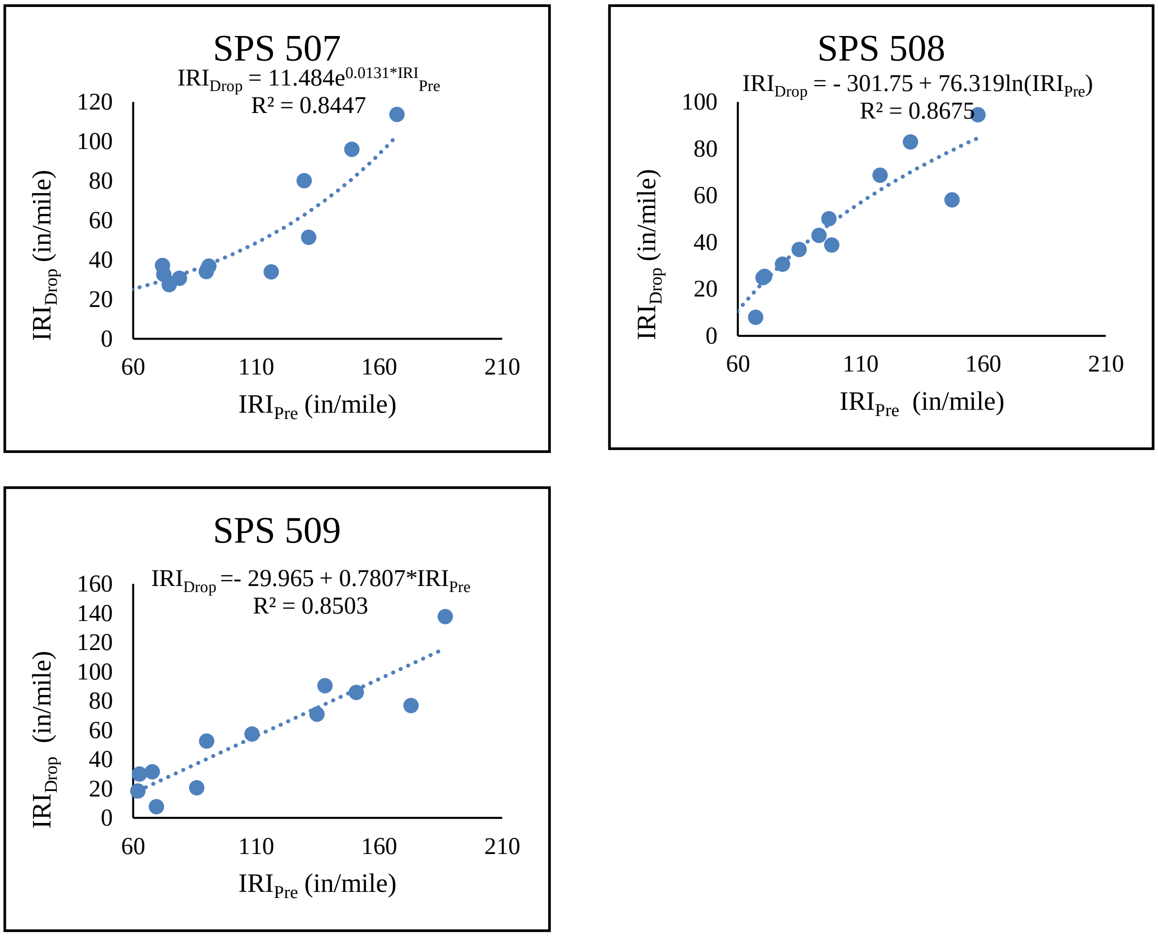


Figure 4.2: Relationship between performance jump and pre-treatment pavement condition for flexible pavement maintenance (SPS 3) and rehabilitation (SPS 5) treatments

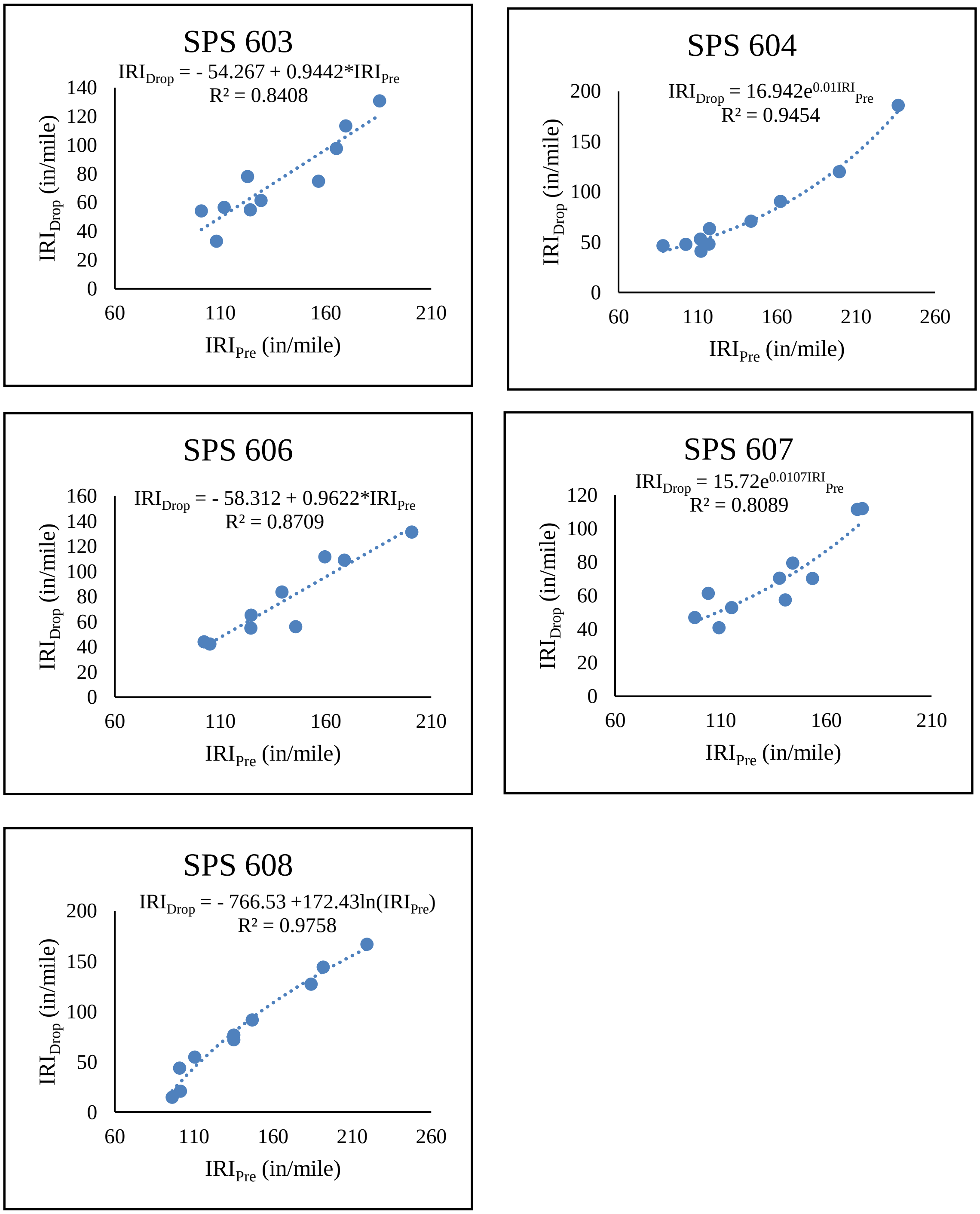


Figure 4.3: Relationship between performance jump and pre-treatment pavement condition for rigid pavement rehabilitation (SPS 6) treatments

4.3.2. Post-Treatment Performance

The pavement conditions of the LTPP sections were observed and monitored throughout their service lives using a uniform performance indicator. In this chapter, the IRI (in./mile) was used as the performance indicator. Performance models were developed for each specific treatment to capture the pavement behavior under different conditions. The models presented in this dissertation represent the LTPP wet freeze, wet non-freeze, dry freeze, and dry non-freeze zones. The performance models provided crucial input for the development of an optimal M&R schedule. Several factors were considered during the modeling process, such as the average annual precipitation, average annual temperature, average freezing index, freeze and thaw cycles, average annual daily traffic (AADT), average annual truck traffic (AATT), and pavement thickness. The exponential model was found to be the best form describing the pavement's condition over its entire service life. Four exponential forms are used in this section (Table 4.3).

Table 4.3: Pavement performance model forms

Model Mathematical Expression	Model Form
$IRI = e^{\alpha+\beta*ATT*t+\gamma*FI*t}$	1
$IRI = e^{\alpha+\beta*t}$	2
$IRI = e^{\alpha+\beta*ATT*t+\gamma*\mathrm{AATEM}*t}$	3
$IRI = e^{\alpha+\beta*ATT*t}$	4

IRI represents the pavement condition in (in/mile) for *i* activity at year *t*, *ATT* is the annual truck traffic of a specific pavement section (represented in millions of trucks), FI is the average annual freeze index (represented in thousands of degree-days), AATEM is the accumalated effect of the average annual temperature ((t* average annual temperature in Fahrenheit)/ 1,000), and α, β, and γ are the model parameters.

The second performance model form shows the strong effect of pavement age alone on pavement condition, without consideration of any other factors. These age-based models were used to estimate the results of the best fitted models for flexible pavement M&R activities and rigid pavement rehabilitation activities (Appendix A). Age-based models do not reflect the actual pavement condition because they suggest that the IRI values of thin overlays in a wet-freeze zone, for example, are less than 100 in./mile, even after 20 years of applying that treatment. For this reason, the results from these models are presented in Appendix A. The purpose of developing these models is to show how different treatments behave throughout their service lives relative to each other. Thin AC overlays are shown to perform better and experience sudden decreases in the pavement roughness when compared with the other maintenance treatments. Also, 5-inch overlays (recycled [SPS 508] or virgin [SPS 507]) with intensive surface preparation tend to perform better than other rehabilitation treatments, although only from the pavement age point of view. Similarly, the performance of 8-inch AC overlays (SPS 608) on rigid pavements was superior to the other rigid pavement rehabilitation treatments. Using the pavement performance models, these overlays were observed in the wet-freeze zone and other similar but not identical zones, as well as in the other climatic zones when the age-based models were used.

Several iterations were performed to find the best fitting line representing the best possible regression model. The exponential form (1) was found to be most reasonable because it accounted not only for the effects of the annual truck traffic and average annual freezing index of each year, but it also considered the accumulated truck traffic and accumulated freezing indices. In general, this model form was found to be more significant in the two freezing zones (wet-freeze and dry-freeze). In general, the model form (4) was found to be more suitable for studying non-freezing zones (wet-non-freeze and dry-non-freeze).

It is important to note that these models were developed with the LTPP dataset, which is an unbalanced panel dataset where each pavement section *i* has a number of observations *j*. All the models were developed as panel models with fixed effects both for age-based models and other model forms. The goal of this dissertation is to find a model that maximizes the R^2 with significant estimated parameters that have high t-ratios, while maintaining the Durbin-Watson statistics close to 2. Both 90% and 95% levels of confidence were used with critical t-ratio values of 1.645 and 1.96. All the coefficients presented in the tables (Table 4.4 to Table 4.19 as well as the tables presented in the Appendix) were found to be significant, mainly at the 90% confidence interval.

Table 4.4: Performance models for flexible pavement maintenance (SPS 3), wet-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310		α	4.160	105.667	85	0.9007	0.8858	96.73	**21.01**
	1	β	0.034	3.672					
		γ	0.017	4.824					
SPS-320		α	4.340	130.966	91	0.8002	0.7734		
	1	β	0.022	1.899				-	-
		γ	0.027	8.785					
SPS-330		α	4.623	102.674	91	0.8663	0.8496		
	4	β	0.038	3.031				-	-
SPS-340		α	4.425	99.183	84	0.8393	0.8148		
	1	β	0.047	3.505				-	-
		γ	0.033	6.873					
SPS-350		α	4.340	118.894	87	0.8695	0.8504		
	1	β	0.027	2.725				-	-
		γ	0.014	3.993					

Table 4.5: Performance models for flexible pavement rehabilitation (SPS 5), wet-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501		α	4.531	88.474	56	0.9338	0.9257		
	1	β	0.058	5.683				-	-
		γ	0.012	5.900					
SPS-502		α	4.109	66.600	66	0.7856	0.7638	104.53	**41.18**
	1	β	0.037	2.839					
		γ	0.009	2.526					
SPS-503		α	3.038	93.122	67	0.8338	0.8172	125.96	**71.98**
	1	β	0.024	2.526					
		γ	0.017	6.221					
SPS-504		α	4.172	75.537	65	0.9053	0.8955	130.35	**73.56**
	1	β	0.044	5.595					
		γ	0.008	3.254					
SPS-505		α	4.149	65.126	65	0.9139	0.9050	115.81	**58.24**
	1	β	0.031	3.739					
		γ	0.010	3.890					
SPS-506		α	4.024	104.390	65	0.8890	0.8775	91.43	**43.51**
	1	β	0.037	4.953					
		γ	0.008	3.754					
SPS-507		α	4.183	144.819	67	0.7933	0.7764	119.39	**66.21**
	4	β	0.019	3.054					
SPS-508		α	3.943	136.479	65	0.9467	0.9411	106.59	**54.62**
	1	β	0.013	2.961					
		γ	0.010	7.358					
SPS-509		α	4.177	84.706	65	0.8833	0.8713	120.08	**63.95**
	1	β	0.020	2.531					
		γ	0.018	7.555					

Table 4.6: Performance models for rigid pavement maintenance (SPS 4), wet-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2
Joint Sealing	**1**	α	4.622	72.573	64	0.984	0.982
		β	0.014	2.885			
		γ	0.013	4.212			
Crack Sealing	**1**	α	4.749	1102.6	12	0.943	0.9305
		β	0.063	8.199			
		γ	-0.006	-2.199			
Partial-Depth Patching	**1**	α	4.708	75.387	24	0.9621	0.9515
		β	0.072	4.957			
		γ	-0.030	-2.570			
Full-Depth Patching	**4**	α	4.655	30.52	14	0.9842	0.9795
		β	0.091	5.169			

Table 4.7: Performance models for rigid pavement rehabilitation (SPS 6), wet-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	**1**	α	4.882	146.42	53	0.882	.86.71	-	-
		β	0.034	9.326					
		γ	0.01	1.877					
SPS-602	**1**	α	4.935	128.77	58	0.662	0.6217	-	-
		β	0.03	6.357					
		γ	0.042	4.296					
SPS-603	**1**	α	4.150	185.26	89	0.8122	0.8562	130.46	**61.96**
		β	0.031	10.097					
		γ	0.010	2.854					
SPS-604	**1**	α	4.206	278.29	89	0.8863	0.8765	133.80	**65.72**
		β	0.026	9.705					
		γ	0.010	3.219					
SPS-606	**1**	α	4.138	291.51	89	0.9276	0.9214	147.27	**83.75**
		β	0.030	13.410					
		γ	0.008	3.228					
SPS-607	**1**	α	4.148	199.35	77	0.8492	0.8363	142.78	**73.10**
		β	0.019	6.461					
		γ	0.007	2.089					
SPS-608	**1**	α	4.151	179	85	0.9206	0.9134	119.31	**54.55**
		β	0.005	3.009					
		γ	0.006	2.899					

Table 4.8: Performance models for flexible pavement maintenance (SPS 3), wet-non-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	**4**	α	4.341	91.964	24	0.6721	0.6229	89.68	**23.10**
		β	0.143	4.275					
SPS-320	**4**	α	4.658	46.429	34	0.9281	0.9209	-	-
		β	0.044	3.306					
SPS-330	**4**	α	4.369	37.558	26	0.6238	0.5725	-	-
		β	0.262	3.246					
SPS-340	**4**	α	4.192	145.205	33	0.6047	0.5638	-	-
		β	0.093	5.421					
SPS-350	**4**	α	4.333	85.436	25	0.5257	0.4580	-	-
		β	0.099	2.318					

Table 4.9: Performance models for flexible pavement rehabilitation (SPS 5), wet-non-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	**4**	α	4.184	101.715	10	0.8793	0.8642	-	-
		β	0.090	7.634					
SPS-502	**4**	α	4.104	85.549	63	0.8113	0.7983	91.29	**37.67**
		β	0.034	6.688					
SPS-503	**4**	α	3.970	94.584	39	0.8287	0.8140	90.05	**28.31**
		β	0.023	6.072					
SPS-504	**4**	α	4.024	59.360	58	0.5053	0.4680	94.42	**44.15**
		β	0.023	2.282					
SPS-505	**4**	α	4.028	63.515	63	0.9374	0.9331	87.77	**31.98**
		β	0.024	6.298					
SPS-506	**4**	α	3.835	61.330	60	0.9496	0.9459	81.50	**32.76**
		β	0.026	7.556					
SPS-507	**4**	α	3.926	81.730	66	0.9252	0.9203	84.60	**36.42**
		β	0.033	10.440					
SPS-508	**4**	α	4.128	99.055	61	0.9341	0.9294	85.84	**26.85**
		β	0.021	8.386					
SPS-509	**4**	α	3.837	92.610	56	0.8859	0.8793	102.95	**48.87**
		β	0.029	9.111					

Table 4.10: Performance models for rigid pavement maintenance (SPS 4), wet-non-freeze zone

Treatment	Model Form	Coefficients		t-stat	No. Obs.	R^2	Adj.-R^2
Joint Sealing	**4**	α	4.63	185.28	34	0.756	0.7227
		β	0.02	1.79			

Table 4.11: Performance models for rigid pavement rehabilitation (SPS 6), wet-non-freeze zone

Treatment	Model Form	Coefficients		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	**4**	α	5.067	43.494	18	0.967	0.9626	-	-
		β	0.04	10.638					
SPS-602	**4**	α	4.238	70.103	16	0.993	0.9914	-	-
		β	0.051	40.795					
SPS-603	**4**	α	4.312	45.192	16	0.9628	0.9571	115.54	**55.57**
		β	0.016	3.933					
SPS-604	**4**	α	4.169	48.647	16	0.9506	0.9430	100.14	**43.82**
		β	0.053	13.536					
SPS-606	**4**	α	4.428	41.176	16	0.9602	0.9541	152.67	**83.79**
		β	0.014	2.778					
SPS-607	**4**	α	4.551	20.711	12	0.8638	0.8335	128.84	**65.70**
		β	0.127	5.621					
SPS-608	**4**	α	4.029	134.51	16	0.9363	0.9266	169.24	**117.92**
		β	0.005	3.175					

Table 4.12: Performance models for flexible pavement maintenance (SPS 3), dry-freeze zone

Treatment	Model Form	Coefficients		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	**1**	α	4.346	77.492	56	0.6656	0.6087	96.75	**29.91**
		β	0.564	5.224					
		γ	0.041	2.558					
SPS-320	**4**	α	4.673	56.086	83	0.9342	0.9261	-	-
		β	0.317	5.440					
SPS-330	**4**	α	4.491	49.290	83	0.9268	0.9178	-	-
		β	0.425	6.408					
SPS-340	**1**	α	4.640	27.591	28	0.8296	0.8000	-	-
		β	0.450	3.514					
		γ	0.082	2.689					
SPS-350	**1**	α	4.678	47.044	83	0.9486	0.9415	-	-
		β	0.219	3.635					
		γ	0.016	1.985					

Table 4.13: Performance models for flexible pavement rehabilitation (SPS 5), dry-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	**1**	α	3.757	86.122	9	0.9649	0.9532	-	-
		β	0.062	5.387					
		γ	0.115	7.130					
SPS-502	**1**	α	4.006	55.502	20	0.8120	0.7767	96.63	**41.09**
		β	0.068	3.908					
		γ	0.036	2.858					
SPS-503	**1**	α	3.979	57.224	20	0.9350	0.9228	112.46	**56.42**
		β	0.017	2.383					
		γ	0.010	1.841					
SPS-504	**1**	α	3.870	124.712	20	0.8681	0.8434	91.81	**42.67**
		β	0.031	5.007					
		γ	0.008	1.848					
SPS-505	**1**	α	3.728	61.716	20	0.7616	0.7169	77.68	**32.16**
		β	0.061	3.120					
		γ	0.050	3.555					
SPS-506	**4**	α	3.890	41.227	19	0.9388	0.9311	123.74	**81.04**
		β	0.141	10.136					
SPS-507	**4**	α	4.085	67.911	19	0.4400	0.3710	74.76	**27.43**
		β	0.074	3.322					
SPS-508	**4**	α	3.880	172.124	20	0.3975	0.3266	101.51	**52.82**
		β	0.028	3.251					
SPS-509	**1**	α	3.939	59.951	20	0.5346	0.4473	62.03	**18.31**
		β	0.041	1.781					
		γ	0.037	2.210					

Table 4.14: Performance models for rigid pavement maintenance (SPS 4), dry-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2
Joint Sealing	**4**	α	4.508	99.017	49	0.979	0.9751
		β	0.085	9.338			
Partial-Depth Patching	**4**	α	4.687	40.7	13	0.9942	0.9931
		β	0.036	4.313			

Table 4.15: Performance models for rigid pavement rehabilitation (SPS 6), dry-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-603		α	4.157	91.234	30	0.8617	0.8458	165.05	**97.57**
	1	β	0.053	9.636					
		γ	0.016	5.993					
SPS-604		α	4.207	84.08	31	0.9232	0.9146	165.05	**120.07**
	1	β	0.026	10.058					
		γ	0.009	6.013					
SPS-606		α	4.197	153.02	31	0.8485	0.8251	165.05	**111.57**
	1	β	0.024	8.993					
		γ	0.009	5.856					
SPS-607		α	3.996	91.091	31	0.9361	0.9289	144.21	**79.39**
	1	β	0.042	10.847					
		γ	0.029	12.313					
SPS-608		α	4.054	154.98	31	0.8624	0.8471	219.29	**166.95**
	1	β	0.021	7.617					
		γ	0.014	8.496					

Table 4.16: Performance models for flexible pavement maintenance (SPS 3), dry-non-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	**4**	α	3.870	143.324	16	0.8954	0.8794	99.97	**31.62**
		β	0.339	9.849					
SPS-320	**4**	α	4.337	42.787	40	0.9934	0.9927	-	-
		β	0.130	9.255					
SPS-330	**4**	α	4.103	40.415	39	0.9843	0.9825	-	-
		β	0.228	8.838					
SPS-340	**4**	α	4.105	40.717	33	0.9879	0.9867	-	-
		β	0.217	8.296					
SPS-350	**4**	α	4.435	119.303	31	0.7946	0.7718	-	-
		β	0.256	74.150					

Table 4.17: Performance models for flexible pavement rehabilitation (SPS 5), dry-non-freeze zone

Treatment	Model Form	*Coefficients*		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	**4**	α	4.428	32.975	29	0.9700	0.9650	-	-
		β	0.088	7.682					
SPS-502	**4**	α	4.385	43.361	54	0.9083	0.9008	161.70	**88.87**
		β	0.929	10.304					
SPS-503	**4**	α	4.098	50.303	53	0.9133	0.9061	116.20	**52.14**
		β	0.065	9.123					
SPS-504	**4**	α	4.142	62.824	60	0.9638	0.9611	116.56	**46.99**
		β	0.033	11.116					
SPS-505	**4**	α	4.151	51.073	53	0.9200	0.9133	112.46	**57.17**
		β	0.062	9.051					
SPS-506	**1**	α	4.019	46.589	53	0.9247	0.9167	104.86	**51.72**
		β	0.051	8.186					
		γ	0.108	2.208					
SPS-507	**4**	α	4.119	74.334	60	0.8992	0.8919	112.55	**54.28**
		β	0.037	8.806					
SPS-508	**4**	α	4.024	73.830	53	0.9440	0.9393	99.66	**48.93**
		β	0.039	10.375					
SPS-509	**4**	α	4.129	39.565	53	0.8450	0.8321	125.20	**69.66**
		β	0.106	8.348					

Table 4.18: Performance models for rigid pavement maintenance (SPS 4), dry-non-freeze zone

Treatment	Model Form	Coefficients		t-stat	No. Obs.	R^2	Adj.-R^2
Joint Sealing	**4**	α	4.544	195.5	23	0.758	0.7342
		β	0.022	4.015			
Crack Sealing	**4**	α	4.673	420.31	7	0.658	0.59
		β	0.032	3.104			
Partial-Depth Patching	**4**	α	4.207	177.42	30	0.9153	0.9090
		β	0.055	13.796			

Table 4.19: Performance models for rigid pavement rehabilitation (SPS 6), dry-non-freeze zone

Treatment	Model Form	Coefficients		t-stat	No. Obs.	R^2	Adj.-R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	**4**	α	4.765	506.56	19	0.973	0.9716	-	-
		β	0.029	24.833					
SPS-602	**4**	α	4.325	97.773	31	0.946	0.9425	-	-
		β	0.063	20.02					
SPS-603	**4**	α	4.013	114.06	39	0.8646	0.8530	152.02	**99.39**
		β	0.063	14.607					
SPS-604	**4**	α	4.174	73.391	42	0.7201	0.6980	152.34	**99.12**
		β	0.041	8.866					
SPS-606	**4**	α	4.309	82.266	42	0.7325	0.7114	125.35	**65.03**
		β	0.035	8.812					
SPS-607	**4**	α	4.139	0.05	42	0.8147	0.8001	127.29	**66.36**
		β	0.050	12.588					
SPS-608	**4**	α	4.175	72.902	43	0.7923	0.7763	128.81	**63.97**
		β	0.033	9.505					

4.3.3. Service Life

The treatment service life was evaluated by solving the mathematical form of each pavement performance model for the unknown time (t) it takes the pavement condition to reach a threshold.

The response variable, which is the performance indicator of the pavement condition, is IRI in (in/mile). This value needs to be substituted in each model form to get the related service life of each treatment. The most reasonable substitution values are the IRI maximum trigger values for M&R treatments. The maximum trigger values for M&R are 130 and 160 (in/mile), respectively. The following is a list of equations for evaluating a treatment service life based on the four model forms developed for flexible and rigid pavement treatments:

$$t = \frac{log(IRI) - \alpha}{\beta * ATT + \gamma * FI} \tag{4.1}$$

$$t = \frac{log(IRI) - \alpha}{\beta} \tag{4.2}$$

$$t = \frac{log(IRI) - \alpha}{\beta * ATT + \gamma * AATEM} \tag{4.3}$$

$$t = \frac{log(IRI) - \alpha}{\beta * ATT} \tag{4.4}$$

The model parameters α, β, and γ are presented in Table 4.4 through Table 4.19, and the other explanatory variables are assumed separately, according to their climatic zone. The model forms used to evaluate service life are: (1) model form 1 for the wet-freeze and dry-freeze zones, with a few exceptions where model form 4 is used for some treatments; and (2) model form 4 for wet non-freeze and dry non-freeze zones (because the freezing index was found to be insignificant in such climates). For consistency purposes, the annual truck traffic was assumed to be 3.468 million for all the zones. The average values at each zone were assumed for the freezing index and average annual temperature. The average freezing indices were 915 and 987 for the wet freeze and dry freeze zones, respectively; and the average annual temperature for the wet non-freeze and dry non-freeze zones was 68 Fahrenheit. The results for treatment service lives are presented in Figure 4.4 and Figure 4.5. Several acronyms are used in Figure 4.4 and Figure 4.5: *M* for minimal and *I* for Intensive preparation before applying the overlay, *R* for the recycled mix, V for the virgin mix, C for crack, B for break, S for Seat, and S&S for saw & seal.

Table 4.20: Treatment service life ranges from the literature

Treatment Type	Service Life (yrs)	Source
Crack sealing	2.2	(Feighan, 1986)
	(3-5)	(Brown, 1988)
	(6-8)	(Morian, 1997)
	(1-3)	(INDOT, 2013)
Chip sealing	(1-6)	(Shuler, 1984)
	4	(Feighan, 1986)
	(3-6)	(Parker, 1993)
	(4-7)	(Raza, 1994)
	(6-10)	(Morian, 1997)
	4	(INDOT, 2013)
Slurry Seal	(1-6)	(Shuler, 1984)
	(3-6)	(Brown, 1988)
	(7-10)	(Morian, 1997)
Micro-surfacing	(4-6)	(Shuler, 1984)
	(5-7)	(Raza, 1994)
	7	(Irfan, 2010)
	6	(Bilal, 2010)
	8	(INDOT, 2013)
Thin HMA overlay	< 6	(Shuler, 1984)
	8	(Joseph, 1992)
	(8-11)	(Raza, 1994)
	(6-11)	(Morian, 1997)
	9	(INDOT, 2013)
HMA overlay (Functional)	12	(Irfan, 2010)
	15	(INDOT, 2013)
HMA overlay 4-5 in (Structural)	11	(Irfan, 2010)
	18	(INDOT, 2013)
Asphalt pavement patching	(1-3)	(Johnson, 2000)
PCC Patching	10	(Irfan, 2010)
	8	(Anwaar, 2012)
Diamond Grinding	(16-17)	(Caltrans, 2005)
	14	(Caltrans, 2008)
Repair PCC & AC Overlay	14	(Irfan, 2010)
	15	(Anwaar, 2012)
Load Transfer Restoration (Dowel Bar Retrofit)	15	(Pierce, 2003)
	(10-15)	(Gulden, 2003)
	(8-15)	(Caltrans, 2008)

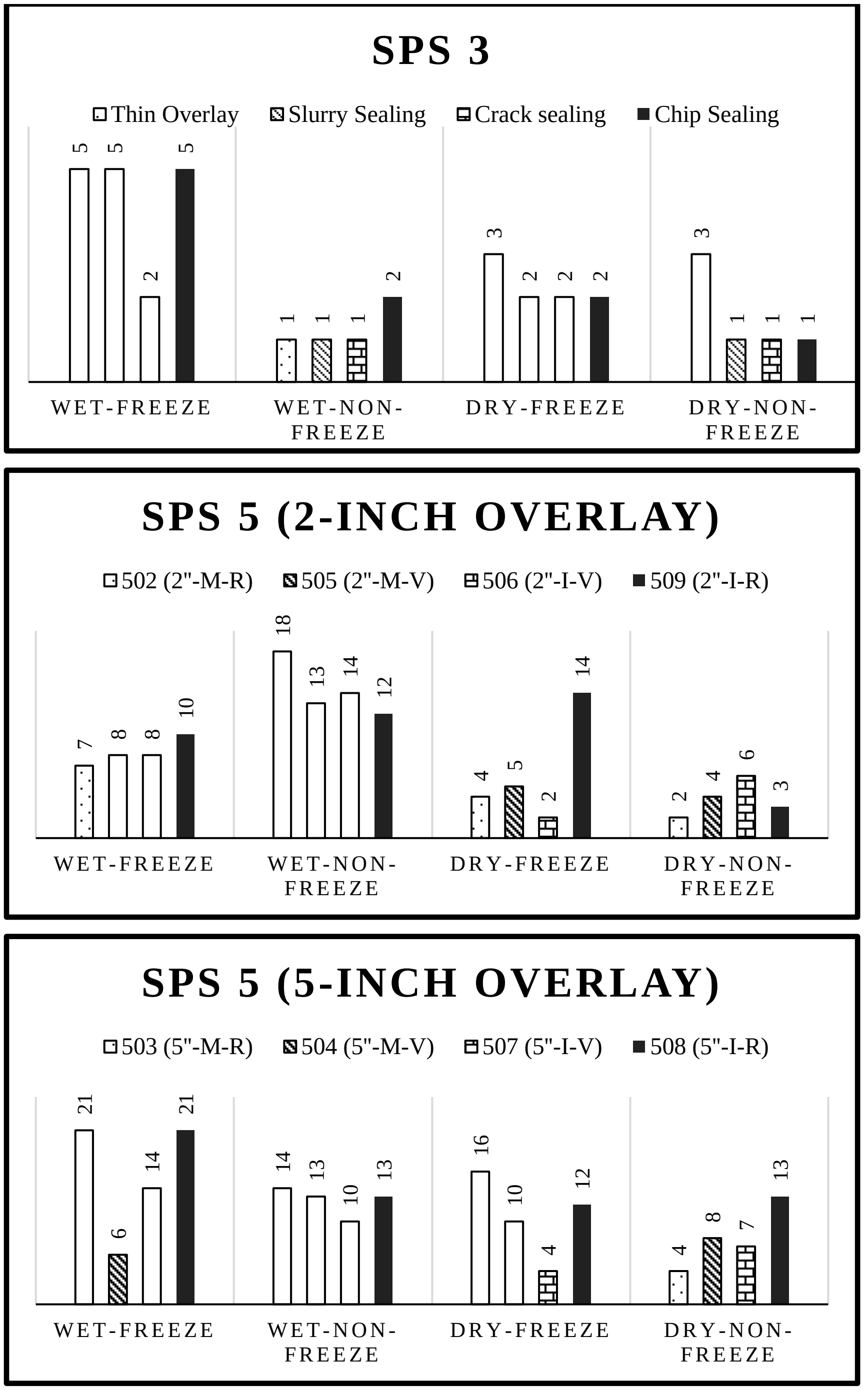


Figure 4.4: Service life of flexible maintenance (SPS 3) and rehabilitation (SPS 5) treatments

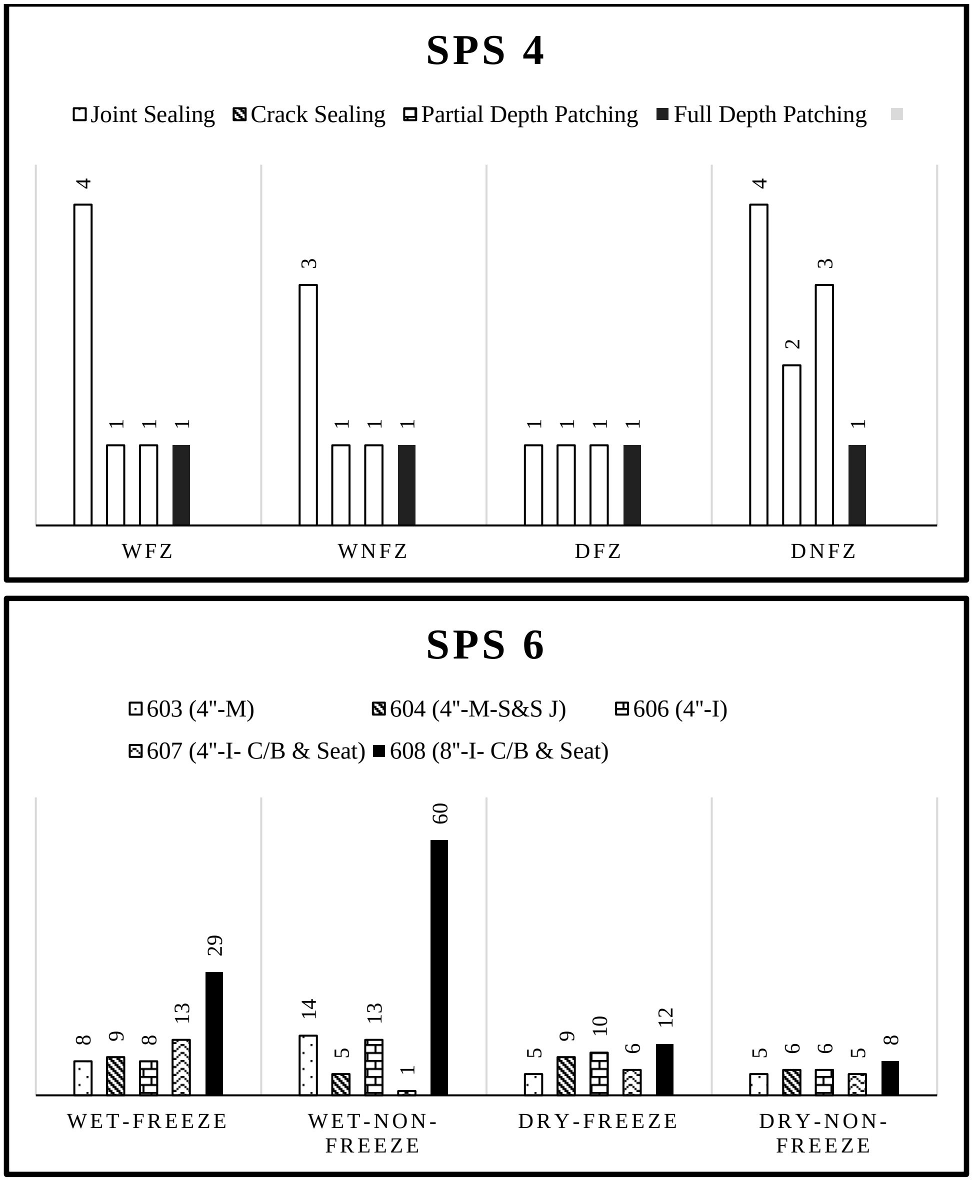


Figure 4.5: Service life for rigid maintenance (SPS 4) and rehabilitation (SPS 6) treatments

This dissertation's pavement performance models for M&R treatments on flexible and rigid pavements were developed using the LTPP dataset. As discussed earlier, prediction models are expected to result in a few unrealistic outcomes caused by dataset limitations. Some of the unreasonable service lives shown in Figure 4.4 and Figure 4.5 may have been caused by applying the same truck traffic load across all the climatic zone models. Performance models were built based on different ranges of traffic loading since applying only one traffic load may result in such behavior. This may be one reason that some of these service lives were out of the expected ranges found in the literature, as presented in Table 4.20. The purpose behind using only one level of truck traffic loading is to trace the treatment effectiveness across the different climatic zones under the same traffic conditions.

Several conclusions can be drawn from examining the treatment service life model outputs. Thin overlay was superior to other maintenance treatments with longer service lives, except in wet non-freeze zones. Slurry and chip sealing performed as well as thin overlay only in wet-freeze zones. In addition, crack sealing, slurry sealing, and chip sealing treatments behaved similarly across different climatic zones. The model results show that a 2-inch virgin mix overlay with minimal preparation performed more effectively than a recycled mix with the same preparation method, except for in the wet-non-freeze zone. In freezing zones, a wet and dry 2-inch recycled mix was superior, while a 2-inch virgin mix was superior in non-freezing zones. Based on the model outcomes, 5-inch overlays performed better in wet zones. Unexpectedly, recycled 5-inch overlays, in general, performed better than the virgin mixes. Rigid pavement maintenance treatments behaved similarly across the climatic zones, which may be a result of their insignificant effects on pavement roughness. It is clear that, regarding service life, an 8-inch AC overlay applied to PCC pavement was a more effective treatment than a 4-inch overlay. Since the data extracted from the wet-non-freeze zone showed very little change in IRI values over the years, especially with the 8-inch overlay, the model outcome with its long service life was unreasonable. Finally, the method of preparation before implementing the overlay for SPS 6 treatments was insignificant. The models used as inputs for the optimization process are the ones with sound and reasonable service lives that match those found in the literature. All the models resulting in extreme service lives were excluded from the optimization process.

4.4. Cost Component and Models

4.4.1. Agency Cost

The agency costs in this dissertation included construction/reconstruction, maintenance, and rehabilitation costs. The cost can be presented either as an average unit costs for treatments, based on historical data and agency records of bids and contracts, or as cost models based on those datasets. The following section shows both cost representations for the wet-freeze, wet-non-freeze, dry-freeze, and dry-non-freeze LTPP climate regions.

4.4.1.1. Average Treatment Unit Costs for all Climatic Zones

Calculating the average unit costs can be a simple exercise that is useful at the initial stages of planning. These costs are extracted from historical data for a variety of contracting projects from different agencies. The average unit costs of (re)construction, maintenance, and rehabilitation on flexible and rigid pavements are shown in Table 4.20 and Table 4.21, respectively (Ahmed, 2012; Irfan, 2010; Wu et al., 2010). A performance evaluation of various M&R treatments conducted under the supervision of FHWA used the cost and pavement performance data from states representing all four climatic regions of the LTPP Program (Wu et al., 2010).

Average unit cost is easily calculated using Equation 3.21, which simply multiplies the average unit cost by the length of the segment, multiplied by the number of lanes (see Chapter 3). All the costs must be adjusted for inflation using Equation 3.22 and for location or special adjustments, as in Equation 3.23.

Table 4.21: Agency unit costs for flexible pavement treatments

Treatment	**Unit Cost in 2017 Constant dollars ($/Lane-Mile)** *Mean*	*std. dev.*	**number of samples**	
Reconstruction Cost	$ 2,126,974	$ 283,409	23	*
Crack Sealing	$ 3,851	$ 4,233	17	**
Fog Seals	$ 303,056	$ -	2	***
Chip Sealing (Seal Coating)	$ 11,124	$ 12,940	3	**
Slurry Seals	$ 46,612	$ -	1	***
Micro-surfacing	$ 31,541	$ 5,956	7	*
Thin HMA Overlay	$ 115,204	$ 45,021	6	*
Functional HMA Overlay	$ 141,042	$ 96,312	18	*
Structural HMA Overlay	$ 242,983	$ 122,685	16	*
Resurfacing (Partial 3R)	$ 209,194	$ 163,155	13	**
Mill Full Depth & AC Overlay	$ 269,833	$ 130,451	13	*

* Irfan (2010), ** Ahmed (2012b), *** Zhang (2010)

Table 4.22: Agency unit costs for rigid pavement treatments

Treatment	**Unit Cost in 2017 Constant dollars ($/Lane-Mile)** *Mean*	*std. dev.*	**number of samples**	
Reconstruction Cost	$ 2,841,304	$ 231,150	19	*
Cleaning and Joint Sealing	$ 22,061	$ -	1	***
Diamond Grinding	$ 312,554	$ -	3	***
Partial Depth Repair	$ 137,295	$ -	2	***
Full Depth Repair	$ 146,332	$ -	3	***
Repair PCC & HMA Overlay	$ 178,460	$ 164,060	36	*
HMA Functional Overlay on Concrete	$ 122,422	$ 127,751	14	**
Concrete Pavement Restoration (CPR) Techniques	$ 205,298	$ 236,756	7	**
PCC Overlay on PCC Pavement	$ 516,071	$ 41,435	11	*
Crack and Seat PCC & HMA Overlay	$ 444,429	$ 127,075	6	*
Rubblize PCC & HMA Overlay	$ 1,061,171	$ 252,633	16	*

* Irfan (2010), ** Ahmed (2012b), *** Zhang (2010)

4.4.1.2. Cost Models for M&R Treatments

As discussed in Chapter 3, different statistical forms have been developed, as shown in Equations 3.22 and 3.23. These models were built as functions of project length, number of lanes, and, most importantly, pre-treatment pavement condition:

$$Agency\ Cost = \alpha . length^{\beta} . N^{\gamma} . \left[ln\left(PI_{trig}\right)\right]^{\delta} \qquad (4.5)$$

$$Agency\ Cost = \alpha + (\beta . length) + (\gamma . N) + \left(\delta . \left[ln\left(PI_{trig}\right)\right]\right) \qquad (4.6)$$

Again, all the costs must be adjusted for inflation, using Equation 3.22, and for location, or special adjustments as in Equation 3.23. The model parameter estimates are presented in Table 4.23, where thin overlays and functional HMA overlays follow the form of Equation 3.24, and structural HMA overlays and resurfacing follow the functional form of Equation 3.25 (Irfan, 2010).

Table 4.23: Statistical cost models for flexible pavement treatments

Treatment Type	**Parameter Estimates of Model Explanatory Variables**				**R^2**
	α	**β**	**γ**	**δ**	
Thin Overlay	0.106	0.814	1.334	4.261	0.884
Function HMA Overlay	24.446	0.662	0.243	1.736	0.735
Structural Overlay	-11.697	0.251	1.159	1.856	0.819
Resurfacing (3R)	0.098	0.690	0.458	4.867	0.576

4.4.2. User Cost Estimation Results

This dissertation discusses work zone user cost only because of the previously noted assumption that user cost due to normal operations is considered to be the same for both flexible and rigid pavements. As discussed in Chapter 3, the duration model is an essential step in estimating work zone user cost associated with travel time delay and work zone user cost associated with VOC. The general form of the duration model is expressed in Equation 3.27 in Chapter 3.

$$D_{WZ} = e^{\alpha + \sum_{k}^{K} B_k . X_k} \qquad (4.7)$$

The specific duration models for each preservation activity are shown in Equations 4.8, 4.9, and 4.10:

$$D_{Maintenance} = e^{4.87+0.299*Cost+0.268*Contract\_Type} \tag{4.8}$$

$$D_{Resurfacing} = e^{4.60+0.340*Cost+0.253*Contract\_Type} \tag{4.9}$$

$$D_{Construction} = e^{4.70+0.307*COst+0.237*Contract\_Type} \tag{4.10}$$

where Cost is the total agency cost of a project in millions of dollars and Contract Type is an indicator variable. Contract Type is 0 when the project is specified to be completed in a given number of days and 1 when the project is given a fixed deadline (Irfan, 2010).

4.4.2.1. Travel Time Delay Cost

Travel time delay cost is a direct result of the speed limit reduction in work zones during construction. The estimation of the delay cost is shown below:

$$Delay\ Cost = D_{WZ}.\sum_{j}^{J}(V_j.\Delta T_j.DC_j) \tag{4.11}$$

where V_j is the number of vehicles within each vehicle class delayed by the speed change at the work zone, ΔT_j is the travel time difference due to the speed change for vehicle j in hours, DC_j is the travel time delay cost rate in dollars/mile, D_{WZ} is the time taken for each treatment in days, and j is the vehicle class. The travel time delay cost for each vehicle class was updated to 2017 constant dollars using the FHWA consumer price index (Walls and Smith, 1998).

4.4.2.2. Vehicle Operating Cost

In addition to travel time delay cost, the VOC in work zones is related to the reduction in speed limit and traffic volume during construction. The VOC in work zones was calculated using Equation 4.12:

$$VOC\ Cost = D_{WZ}.\sum_{j}^{J}(V_j.\Delta T_j.g_j.p_j) \tag{4.12}$$

where V_j is the number of vehicles within each vehicle class delayed by the speed change at the work zone, ΔT_j is the travel time difference due to the speed change for vehicle j in hours, D_{WZ} is the time taken for each treatment in days, g_j is the fuel consumption in gallons per hour of delay, p_j is the average fuel price in dollars per gallon, and j is the vehicle class (Irfan, 2010; Sinha and Labi, 2007).

4.4.3. Community Cost Estimation Results

The community cost component consists of the costs associated with mitigating the adverse impacts of air and noise pollution. In this section, the evaluation methods for each community cost element are presented. A sample calculation of the community cost is provided in the case study (Chapter 5).

4.4.3.1. Cost Associated with Air Pollution

The monetary value of the effect of air pollution on air quality can be determined using any one of three methods: (1) the cost of cleaning up the air around the polluting source; (2) the social damage costs of air pollution; and (3) the amount a community is willing to pay to mitigate the effects of air pollution. In this dissertation, social damage cost is examined using a CO_2 equivalent, and energy consumption serves as the performance measure.

LCA follows four typical steps to calculate the amount of emission and energy consumed for both asphalt and concrete pavements. The goal is to quantify the GWP of the GHG as well as the energy consumption. The scope is to evaluate the environmental impact of newly constructed M&R treatments on different pavement materials. The evaluation in this dissertation was based on a unit function of one lane-mile of pavement. The assigned system boundaries are material acquisition (such as asphalt binder, cement, steel, and aggregates); mixture production; transportation; and construction (including M&R).

The life cycle inventory starts with raw material acquisition, where all the materials used in the pavement are identified. The material used in extracting and manufacturing the asphalt binder and the Portland cement must be examined in order to evaluate the expected emission and energy used at this stage. Emission and energy usage resulting from the aggregates manufacturing process also are accounted for in the LCI estimation. At the pavement mixture production stage, the aggregate, asphalt binder or cement (based on pavement type), and mixture additives are the inputs that need to be considered.

The transportation stages considered in this dissertation are (1) transporting raw materials from the site, such as transporting aggregates from the quarry to the asphalt mixture plant; and (2) transporting pavement to the construction site. The main source of emission and energy consumption during the construction stage is the construction equipment, such as paving, rolling and compacting, and sawing and milling equipment. Estimations of equipment emission and energy usage are based on the built-in database of the Athena IE software. All air emissions (reported in CO2 equivalent) and energy consumed (reported in MJs) in the LCI phases were estimated based on the Athena software.

The global warming potential of greenhouse gas emissions and energy consumption for the newly constructed pavements are presented in Table 4.24 and Table 4.27, respectively. The results show that the total GWP emission for rigid pavement was 22.73% more when compared to flexible pavement (Figure 4.6). However, it is also important to note that rigid pavement consumed only 50.78% of the energy when compared to flexible pavement (Figure 4.7). These results are consistent with previous literature that investigated broader system boundaries (Aurangzeb, 2014; Chan, 2007; Hakkinen et al., 1996; Horvath et al., 1998; Weiland et al., 2010). The comparison between pavement materials in terms of emission and energy consumption was highly affected by the M&R treatments implemented on a certain highway pavement. Therefore, a project level estimate of the environmental impact of each pavement material was calculated. The GWP and energy consumption levels for M&R treatments on both pavement materials are presented in the following tables.

Table 4.24: Global warming potential emission in CO_2 equivalent for new construction

Treatment	GWP in CO_2 equivalent (kg)			CO_2 Cost ($/Lane-Mile)
	Raw Material, Production & Construction	Transportation	Total	
HMA	5.76E+05	3.06E+04	6.07E+05	$ 19,504
PCC	7.24E+05	2.04E+04	7.45E+05	$ 23,945

Table 4.25: Global warming potential emission in CO_2 equivalent for flexible pavement treatments

Treatment	GWP in CO_2 equivalent (kg)			CO_2 Cost ($/Lane-Mile)
	Raw Material, Production & Construction	Transportation	Total	
Crack sealing	6.14E+02	1.32E+01	6.27E+02	$ 20.15
Patching	1.30E+03	2.00E+02	1.50E+03	$ 48.24
1" Overlay	7.80E+04	7.06E+02	7.87E+04	$ 2,528.92
2" Overlay	1.52E+05	1.27E+03	1.53E+05	$ 4,926.60
3" Overlay	2.26E+05	1.84E+03	2.28E+05	$ 7,324.27
4" Overlay	3.00E+05	2.41E+03	3.02E+05	$ 9,721.95
5" Overlay	3.74E+05	2.98E+03	3.77E+05	$ 12,119.63
8" Overlay	8.38E+05	3.54E+04	8.74E+05	$ 28,091.30

Table 4.26: Global warming potential emission in CO_2 equivalent for rigid pavement treatments

Treatment	GWP in CO_2 equivalent (kg)			CO_2 Cost ($/Lane-Mile)
	Raw Material, Production & Construction	Transportation	Total	
Diamond Grinding	1.85E+04	6.08E+02	1.91E+04	$ 613.89
Joint Sealing	2.72E+03	2.28E+01	2.74E+03	$ 88.13
Partial Depth Patching	7.01E+03	2.87E+02	7.29E+03	$ 234.51
Full Depth Patching	2.70E+04	5.40E+02	2.75E+04	$ 885.10
Asphalt Crack Sealing	9.14E+03	4.93E+01	9.19E+03	$ 295.33

Table 4.27: Non-renewable energy consumption for new construction

Treatment	Non-Renewable Energy (MJ) Raw Material, Production & Construction	Transportation	Total	CO2 Cost ($/Lane-Mile)
HMA	1.98E+07	4.44E+05	2.02E+07	$ 404,120
PCC	9.44E+06	2.96E+05	9.74E+06	$ 194,713

Table 4.28: Non-renewable energy consumption for flexible pavement treatments

Treatment	Non-Renewable Energy (MJ) Raw Material, Production & Construction	Transportation	Total	CO2 Cost ($/Lane-Mile)
Crack sealing	5.59E+04	1.92E+02	5.61E+04	$ 1,121
Patching	1.00E+05	2.90E+03	1.03E+05	$ 2,061
1" Overlay	1.78E+06	1.03E+04	1.79E+06	$ 35,766
2" Overlay	3.09E+06	1.85E+04	3.11E+06	$ 62,187
3" Overlay	4.40E+06	2.68E+04	4.43E+06	$ 88,607
4" Overlay	5.72E+06	3.50E+04	5.75E+06	$ 115,027
5" Overlay	7.03E+06	4.33E+04	7.07E+06	$ 141,448
8" Overlay	3.04E+07	5.14E+05	3.09E+07	$ 618,553

Table 4.29: Non-renewable energy consumption for rigid pavement treatments

Treatment	Non-Renewable Energy (MJ) Raw Material, Production & Construction	Transportation	Total	CO2 Cost ($/Lane-Mile)
Diamond Grinding	2.69E+05	8.83E+03	2.77E+05	$ 5,547
Joint Sealing	5.69E+04	3.31E+02	5.72E+04	$ 1,144
Partial Depth Patching	9.79E+04	4.17E+03	1.02E+05	$ 2,042
Full Depth Patching	3.81E+05	7.84E+03	3.88E+05	$ 7,768
Asphalt Crack Sealing	6.91E+05	7.17E+02	6.92E+05	$ 13,834

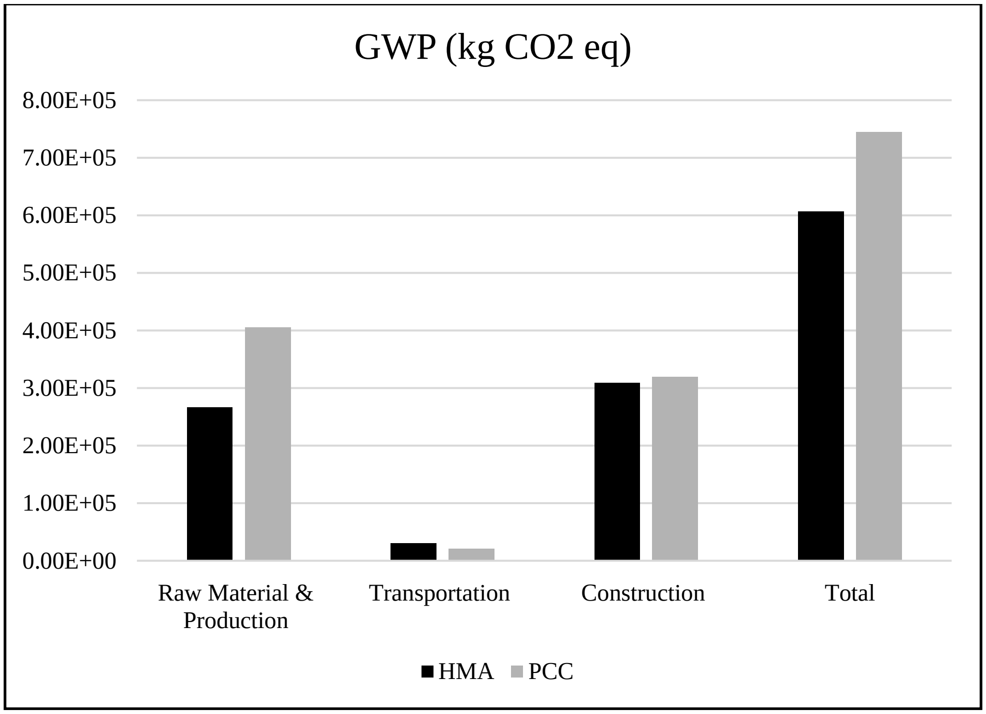


Figure 4.6: GWP of the greenhouse gas emissions for flexible and rigid pavements

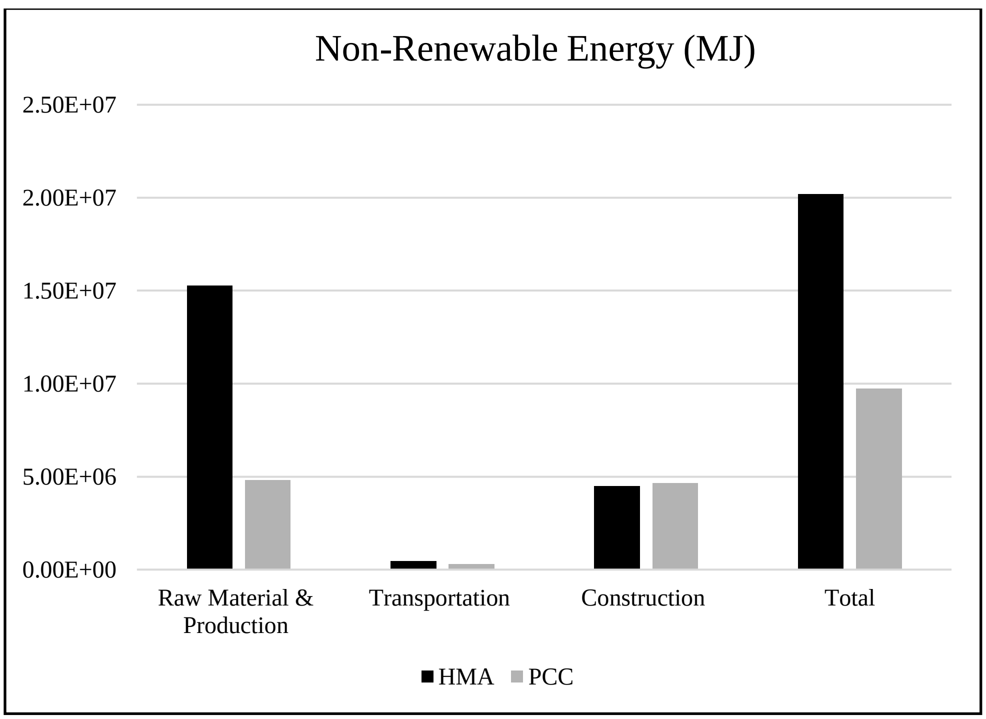


Figure 4.7: Energy consumption for flexible and rigid pavements

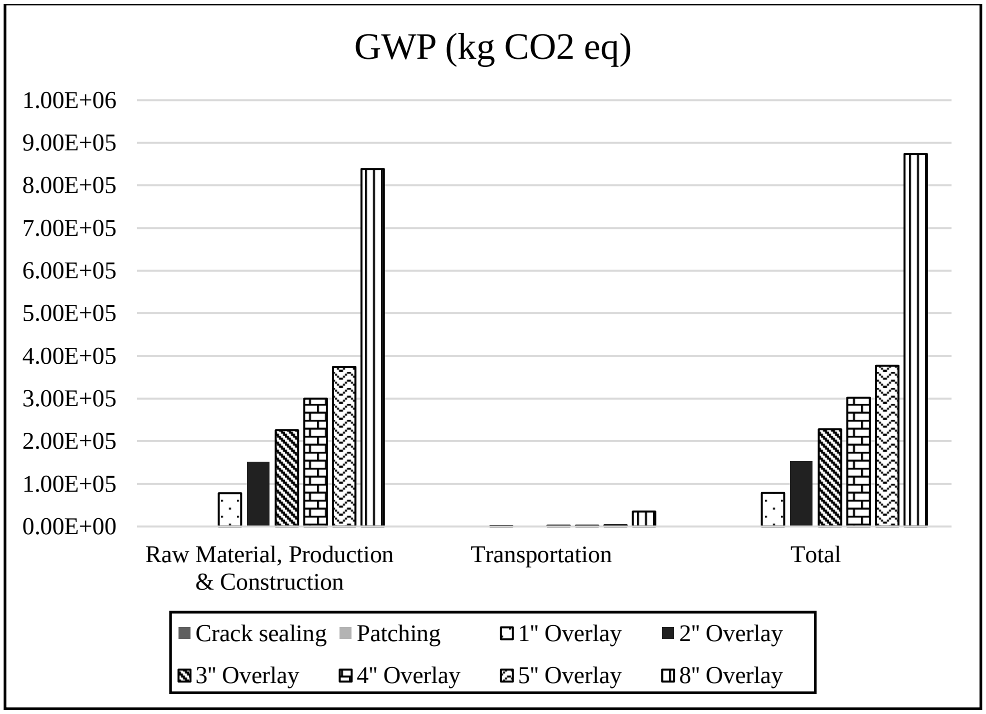


Figure 4.8: GWP of the greenhouse gas emissions for flexible pavement treatments

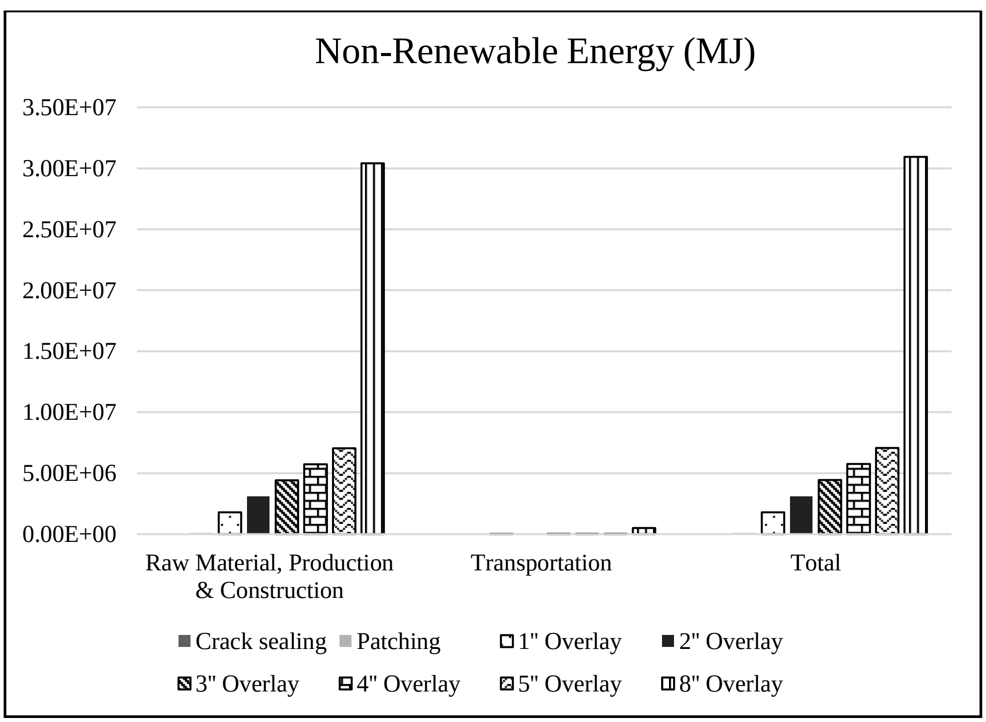


Figure 4.9: Energy consumption for flexible pavement treatments

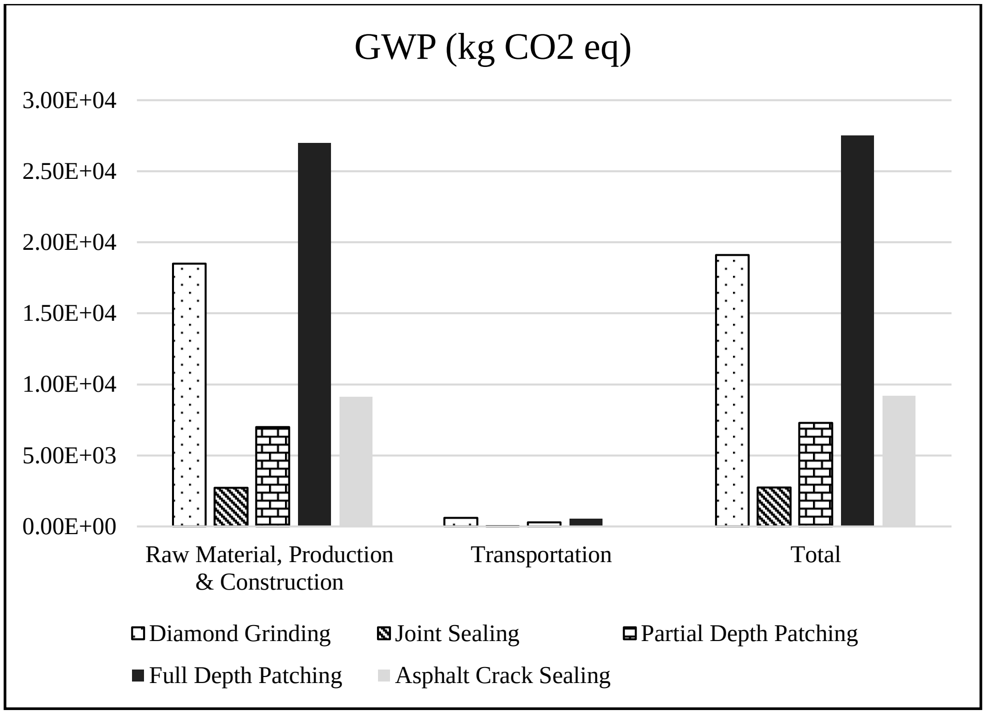


Figure 4.10: GWP of the greenhouse gas emissions for rigid pavement treatments

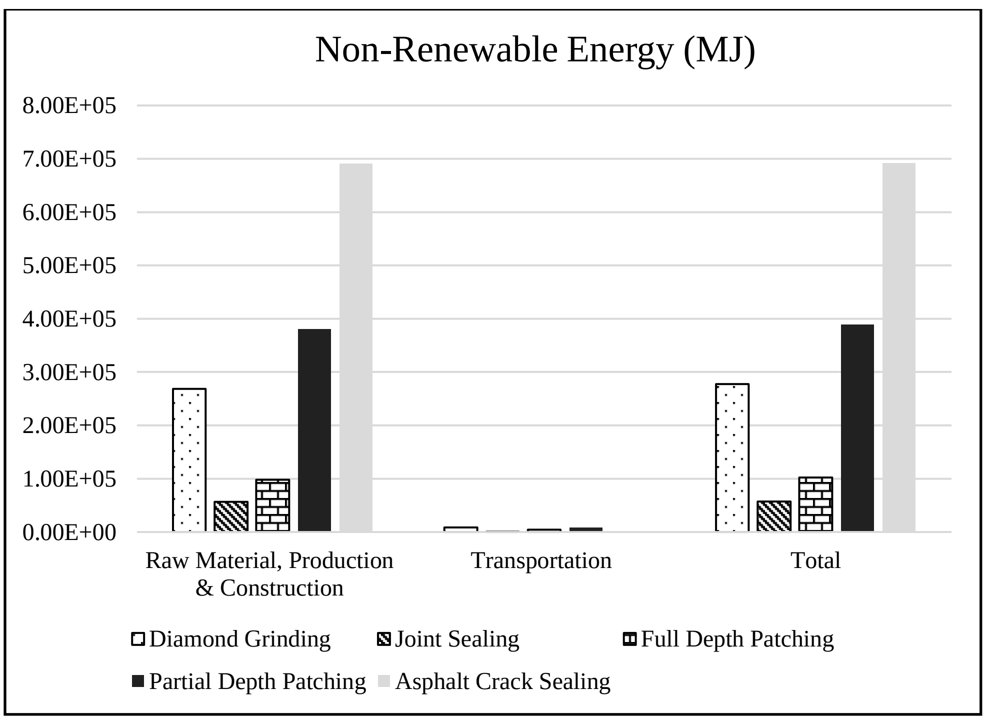


Figure 4.11: Energy consumption for rigid pavement treatments

4.4.3.2. Cost Associated with Noise Pollution

The cost resulting from highway noise was calculated in terms of the cost of the countermeasures used to reduce the noise level for the community near the highway. Different mitigation techniques are available, including constructing a noise barrier or expanding the right-of-way on the highway. Mitigating the noise level by building a noise barrier is the method examined in this dissertation.

The first step in evaluating the cost associated with noise level is to determine whether the FHWA has set a noise level threshold for the functional class of the highway in question. As shown in Figure 4.12, no noise impact evaluation is needed for activities in categories F and G, where the highway traffic noise does not affect developed land (activity category F) or where the activity is located near undeveloped land (activity category G). An equation developed by Barry and Regan (1978) (Equation 3.39 in Chapter 3) was used to evaluate the sound (noise) level for all vehicle types on the highways that fall into categories A, B, C, D, and E, as shown in Table 4.30 (Bahrami, 2010). If the noise level exceeds FHWA standards, mitigating action needs to be taken, and in such cases, a noise barrier is used. The noise barrier's success in reducing noise depends on (1) the noise barrier's height and (2) the construction material used for the barrier. The noise barrier height should be determined first, and then the material of that barrier should be selected based on a LCCA of the barrier's structure life. Figure 4.12 presents the process of evaluating noise cost based on the cost of the noise barrier (Bahrami, 2010). The cost of noise pollution may be equated with the life cycle cost of the noise barrier, including the barrier maintenance cost. The initial cost of the barrier can be calculated using the unit cost per area/volume or by using the statistical model introduced earlier in Chapter 3 (Section 3.5.1.2). The estimated value of the noise barrier is presented in the case study section of Chapter 5.

Table 4.30: Noise abatement criteria

Activity Category	Activity Leq(h)	Criteria L10(h)	Evaluation Location	Activity Description
A	57	60	Exterior	Lands on which serenity and quiet are of extraordinary significance and serve an important public need and where the preservation of those qualities is essential if the area is to continue to serve its intended purpose.
B	67	70	Exterior	Residential
C	67	70	Exterior	Active sport areas, amphitheaters, auditoriums, campgrounds, cemeteries, day care centers, hospitals, libraries, medical facilities, parks, picnic areas, places of worship, playgrounds, public meeting rooms, public or nonprofit institutional structures, radio studios, recording studios, recreation areas, Section 4(f) sites, schools, television studios, trails, and trail crossings.
D	52	55	Interior	Auditoriums, day care centers, hospitals, libraries, medical facilities, places of worship, public meeting rooms, public or nonprofit institutional structures, radio studios, recording studios, schools, and television studios.
E	72	75	Exterior	Hotels, motels, offices, restaurants/bars, and other developed lands, properties or activities not included in A-D or F.
F	-			Agriculture, airports, bus yards, emergency services, industrial, logging, maintenance facilities, manufacturing, mining, rail yards, retail facilities, shipyards, utilities (water resources, water treatment, electrical), and warehousing.
G	-			Undeveloped lands that are not permitted.

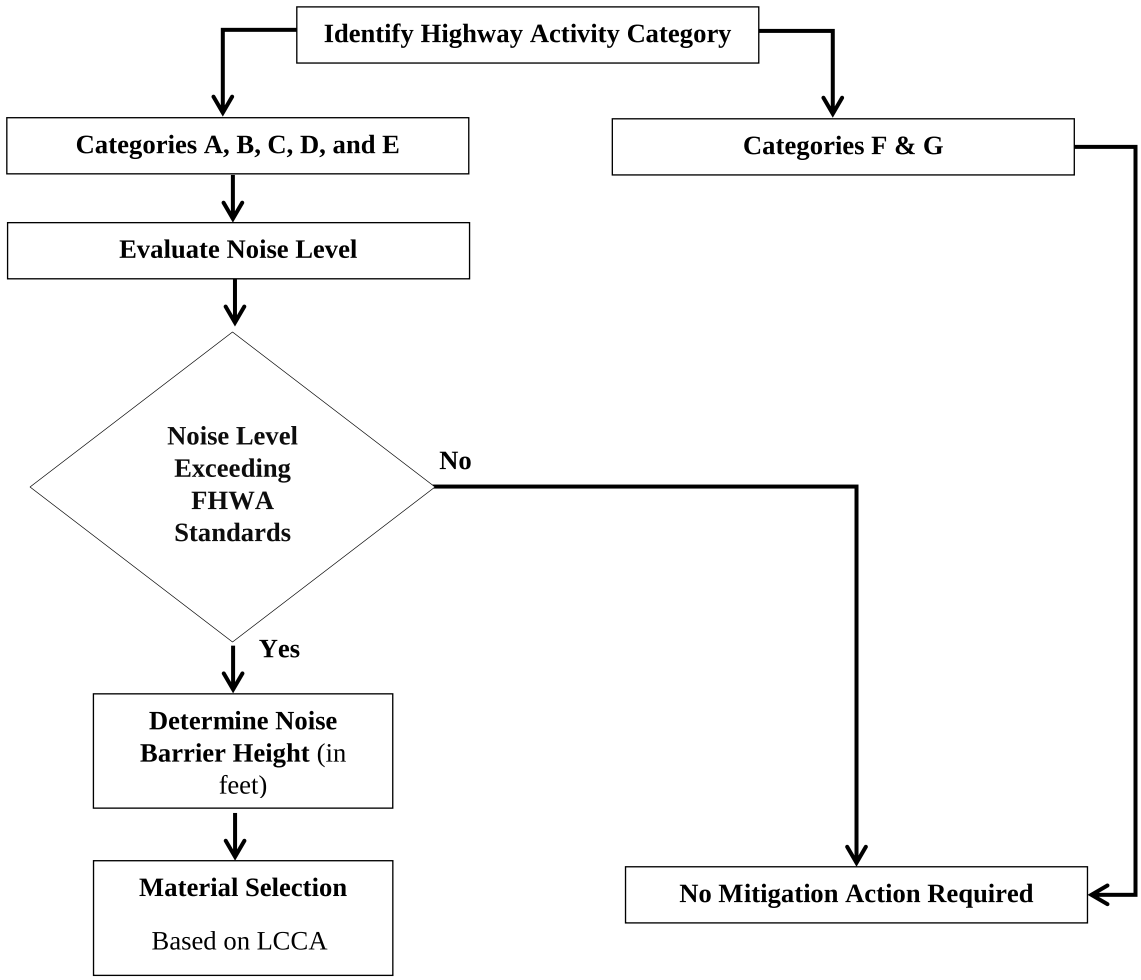


Figure 4.12: Procedure for evaluating the effect of noise barriers

4.5. The Effect of Performance and Cost Models on the Optimal Treatment Profile

The statistical relationships developed in this chapter to predict pavement performance and to estimate costs are key inputs in the optimization problem. These models are crucial for determining the optimal treatment profile for a certain pavement type. Using goodness of fit test indicators as the sole criteria for choosing the best optimal profile could lead to misleading results. The treatment service life should fall within the range found in the literature. For example, asphalt concrete functional overlay should have a treatment life of nine to 15 years, depending on the threshold adopted. Using treatment SPS 506 in the dry-freeze zone as a functional overlay with only two years of service life would highly affect the choice of the optimal profile because the service life is unrealistic. Using unrealistic inputs eventually leads to unrealistic optimal profiles, which make it difficult to compare the two pavement materials (asphalt and concrete). The models adopted from the literature for estimating the agency cost for each treatment are essential in the optimization process as well. Reasonable values for treatment benefits, evaluated with the performance models, and for costs are critical for reaching the optimal profile of a pavement treatment. After developing several models for a number of pavement M&R treatments in each LTPP climatic zone, only the models with sound service lives are used to develop optimal profiles of pavement treatments.

4.6. Chapter Summary

This chapter presented the results of the pavement performance and performance jump models. Average improvements resulting from the application of each individual treatment were presented, and attention was given to compensating for any weaknesses in the performance jump models due to the small number of observations for these models. Two major forms of pavement performance models were introduced: (1) models based on pavement age only (Appendix A) and (2) models based on the cumulative traffic and climate conditions over the pavement's service life. Also, the average unit costs of different preservation treatments were provided from different sources. All the types of cost models used to obtain the agency and user costs were presented in this chapter. The two major components of community cost were identified as (1) air pollution cost and (2) noise cost. The impact of air pollution on the environment was determined using two criteria: (1) the global warming potential of greenhouse gas emission and (2) energy consumption for flexible and rigid pavements using the LCA methodology. The cost from noise pollution was estimated based on noise barriers built according to FHWA noise abatement criteria. Once the need for a noise barrier is justified, the cost of noise pollution, or the life cycle cost of the noise barrier, can be estimated. All of these components are key inputs for the analysis and evaluation of cost-effectiveness.

CHAPTER 5. RESULTS OF THE DETERMINISTIC AND PROBABILISTIC APPROACHES OF THE SENSITIVITY ANALYSIS

5.1. Introduction

The methods and results for creating optimal profiles of flexible and rigid pavements are presented in this chapter for the wet freeze, wet non-freeze, dry freeze, and dry non-freeze climatic zones. The optimal timing and type of maintenance or rehabilitation treatment to be applied was ascertained through the deterministic optimization approach. In the literature, several methods have been used to determine the optimal scheduling of M&R activities based on cost-effectiveness techniques, where agency cost is considered to be the main component of the total cost. In this chapter, the costs included in the optimization model are the agency, user, and community costs. The agency costs were evaluated using one of two approaches: average unit cost and cost models adopted from the literature. The two components of user cost (travel time and VOC) in the work-zone were evaluated referencing the duration models of each treatment and other major inputs. The noise cost, the first element of community cost, was evaluated in terms of the cost of the noise barrier. The second element of the community cost is the air pollution cost. Air pollution cost was estimated by evaluating the social damage of the GWP of the GHG, represented by the carbon dioxide equivalent emissions during the entire LCA phase as well as the estimated cost of the energy consumed during these phases. The summation of all the costs for each treatment applied during the analysis period was estimated, and the benefit each treatment was evaluated using the performance models presented in Chapter 4. Finally, the benefits were evaluated using the concept of the area bounded by the performance curve and the threshold line. The cost-effectiveness of any profile can be evaluated based on the estimation method described above. This chapter presents the optimization method used to reach the optimal profile of M&R activities based on the cost-effectiveness criteria.

5.2. Optimization Inputs

Performance thresholds are considered significant inputs for optimizing and scheduling the most cost-effective treatments. The maximum allowable performance indicator (threshold), PI_{max}, and the minimum pavement performance indicator, PI_{min}, where no treatments are to be applied under that value, both are determined based on the highway agency requirements. Although some assumptions were made based on common practices, the methodology proposed in this dissertation can be adjusted to any range of PI inputs. The candidate treatments considered are preventive maintenance (thin overlay, chip sealing, and slurry sealing) and rehabilitation (functional and structural overlays) treatments. The PI ranges assigned to preventive M&R activities were 100 to 130 (in/mile) and 100 to 160 (in/mile), respectively. These ranges are based on the consensus within the field that preventive maintenance treatments are considered ineffective if they are applied on top of pavements in poor condition.

The other inputs required for reaching an optimal activity profile were the AADT, truck traffic percentage, average annual freeze index, and average annual temperature. All of these inputs are related to the explanatory variables of the developed treatment performance models presented in Chapters 3 and 4. The average values used for these inputs were obtained from each zone's dataset. The traffic loading was fixed across the states to investigate the effect of the same traffic across different climatic regions. The average freeze indices (in thousands) for wet freeze, wet non-freeze, dry freeze, and dry non-freeze zones were 0.915, 0.015, 0.987, and 0.028, respectively. The AADT was assumed to be 50,000, and the truck percentage was 19%, which means that the accumulated truck traffic was 3.468 (in millions). Along with the treatment costs, the discount rate was another critical factor to consider in each climatic zone. These costs, represented as average values or models, were presented in Chapter 4.

5.3. Proposed Candidate Treatments for Maintenance and Rehabilitation Optimization

The procedure for identifying the most cost-efficient pavement material follows the flowchart in Figure 1.1. A case study was conducted to compare the optimal profiles of flexible and rigid pavement cost-effectiveness. The following candidate treatments were considered for flexible and rigid pavements:

a. *New construction*:

For flexible pavement, a control section of flexible pavement with no treatment (SPS 501) is a reasonable surrogate to measure the performance of new construction. However, treatment of a 5-inch section of virgin AC overlay (SPS 504) was used instead because of the unrealistic performance model of SPS 501, which indicated that the treatment should be applied as early as the first year after the pavement is constructed. The results of these models were consistent across the different climatic zones. For rigid pavement, the assumed equivalent treatment for new construction was a 4-inch AC overlay with sawed-and-sealed joints and minimal surface preparations (SPS 604).

b. *Maintenance Treatments*: Thin overlay (SPS 310) and slurry sealing (SPS 320) were the candidate treatments chosen for flexible pavement. Joint sealing and crack sealing (SPS 4) were the treatments adopted for rigid pavement.

c. *Rehabilitation Treatments*:

1. Functional Overlay: The functional overlay selected for asphalt pavements in all the zones except the dry-freeze zone was a 2-inch (average) overlay with intensive preparation and a virgin mix called SPS 506. For the dry-freeze zone, a 2-inch (average) AC overlay with intensive preparation and a recycled mix called SPS 509 was chosen. For the rigid pavement, a 4-inch (average) AC overlay with cracking (or breaking) and seating of the PCC slab (SPS 607) was selected for all zones except the wet non-freeze zone. For the wet non-freeze zone, a 4-inch (average) AC overlay over PCC with minimal preparation (SPS 603) was used.
2. Structural Overlay: A 5-inch (average) AC overlay with intensive preparation and virgin mix SPS 507 was the dominant asphalt structural overlay treatment used in all zones except the dry-freeze zone. For dry-freeze zone , a 2-inch (average) AC overlay with intensive preparation and recycled mix SPS 509 was used. The rigid pavement structural treatment was SPS 608, which is comprised of an 8-inch AC overlay with cracking or breaking and seating of the PCC slab. This treatment was used in all the zones except the wet non-freeze zone, where SPS 604 was used instead.

The process of selecting the candidate treatments followed a systematic and consistent methodology to avoid bias. When a specific treatment was identified as the flexible functional overlay, for example, this treatment was assigned to this role for all the climatic zones. However, the selected treatment sometimes behaves unexpectedly in one climatic zone (e.g., unreasonable service life), forcing the use a different though similar treatment in that particular climatic region. As mentioned above, SPS 507 was chosen as the functional overlay treatment for the flexible pavement. In the dry-freeze zone, however, SPS 507 was known to have a service life of only four years, which is unrealistic. In this case, a similar treatment called SPS 509 was used, which has the same thickness and preparation method but consists of recycled asphalt instead of a virgin mix. This choice was mainly caused by the type and integrity of the data used to build the performance models. If the dataset does not reflect the actual condition of the pavement, then the statistical models will be misleading. Treatments with such problems were ignored and disqualified from becoming the candidate treatment.

The effectiveness of each candidate treatment was determined by calculating the area over the curve of the performance models and bounded by the performance threshold. Since the performance indicator is a non-decreasing MOE, the area over the curve is used to evaluate the effectiveness of the different treatments. The equation representing AOC is shown in Equation 3.2. The performance model forms used in this study are presented in Chapter 4 (Table 4.3, and the coefficients are presented in Table 4.4 to Table 4.19).

5.4. Optimal Profile Results

In this section, the optimal profiles or schedules of M&R activities for flexible and rigid pavements are presented. The optimal schedule of treatments is based on the overall cost-effectiveness of all the treatments administered to asphalt and concrete pavements in each LTPP climatic region. The profiles were evaluated based on a deterministic optimization approach using a genetic algorithm search method. At each of the two stages of these hypothetical examples, four alternative treatments were available: (1) thin overlay, (2) slurry sealing, (3) functional overlay, and (4) structural overlay. The optimal schedule of M&R treatments in this section was obtained by using the average values for the major inputs, such as traffic loadings and freeze indices. The sensitivity of these optimal schedules to traffic and climatic conditions are discussed in Sections 5.5 and 5.6.

5.4.1. Wet-Freeze Climatic Zone

The expected output of the optimization process in this dissertation is the determination of the most cost-effective type of treatment and when to apply that treatment. The optimal profile shows that a functional overlay is the optimal treatment at the first stage and a structural overlay is optimal for the second. According to the optimal profile, the functional and structural overlays should be implemented in years 4 and 11, respectively, as shown in Figure 5.1. The pavement reaches the performance threshold (IRI = 160 in/mile) at year 25, the year the pavement needs to be reconstructed.

The optimal life cycle schedule for rigid pavement is shown in Figure 5.2, which uses the same inputs as flexible pavement. The best treatment to apply to rigid pavement, as for flexible pavement, during the first stage is a functional overlay, and the best treatment for the second stage is a structural overlay. The treatments are implemented in years 6 and 17. For this profile, the pavement service life is 45 years, at which point the pavement condition is critical and reconstruction of the highway (or major rehabilitation) should take place.

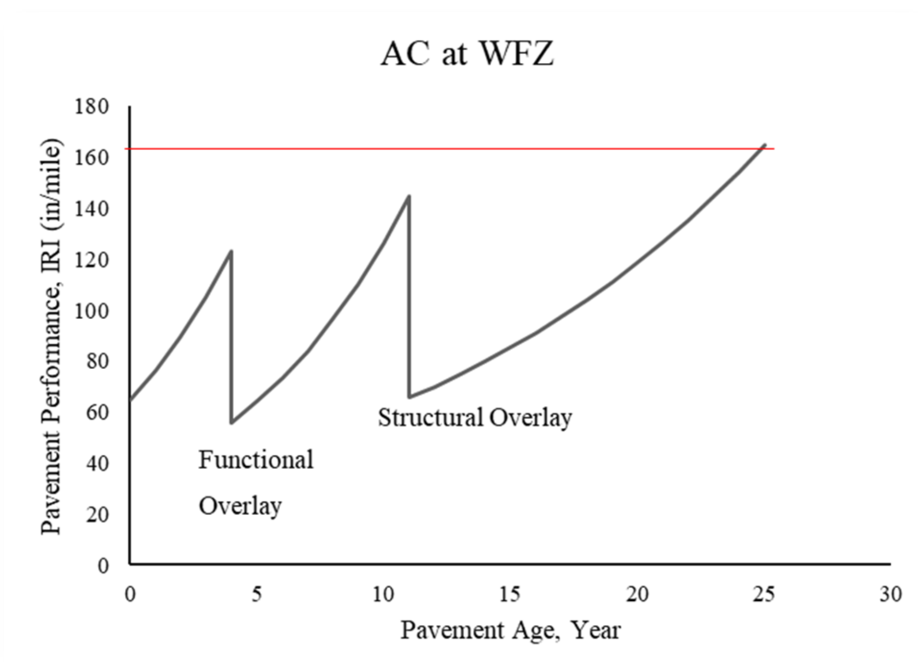


Figure 5.1: Optimal profile involving two flexible pavement preservation treatments, wet-freeze zone

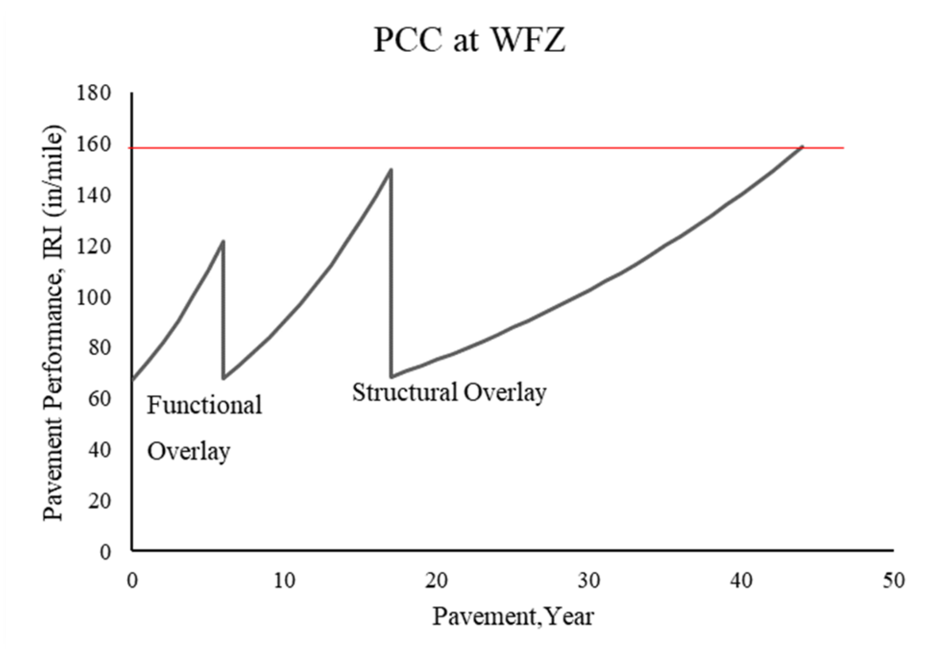


Figure 5.2: Optimal profile involving two rigid pavement preservation treatments, wet-freeze zone

5.4.2. Wet-Non-Freeze Climatic Zone

In this zone, a functional overlay of the flexible pavement is the dominant type of treatment during both stages. The functional overlay should be applied at years 10 and 23 based on this scenario, as shown in Figure 5.3. Reconstruction or major rehabilitation is required at year 36, when the pavement condition reaches the maximum performance threshold (IRI = 160 in/mile).

The functional overlay is also the treatment that must be implemented for rigid pavement at both stages. This treatment should take place at years 4 and 17, as shown in Figure 5.4. Based on this two-stage treatment scenario, the pavement condition will reach the maximum allowable performance indicator (IRI =160 in/mile) at year 30.

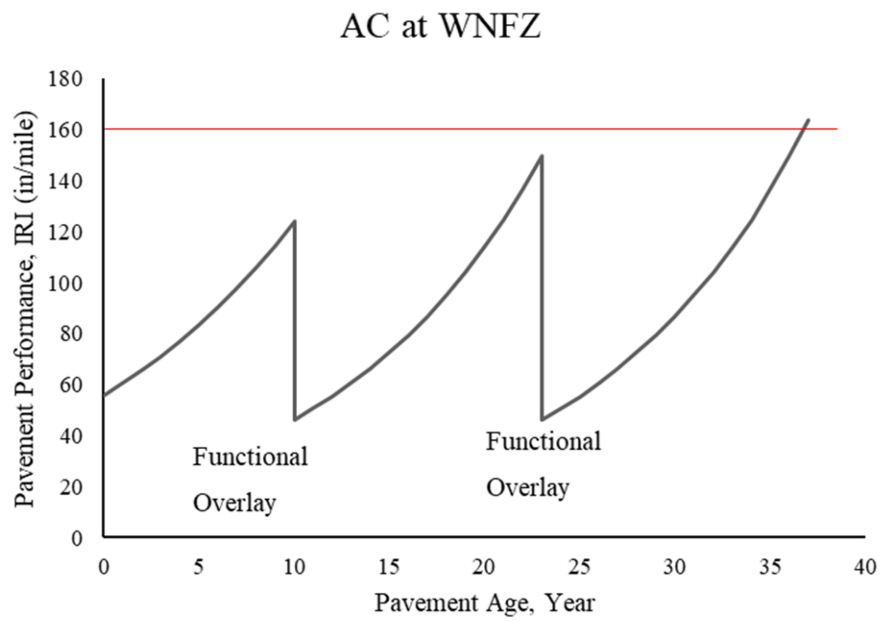


Figure 5.3: Optimal profile involving two flexible pavement preservation treatments, wet non-freeze zone

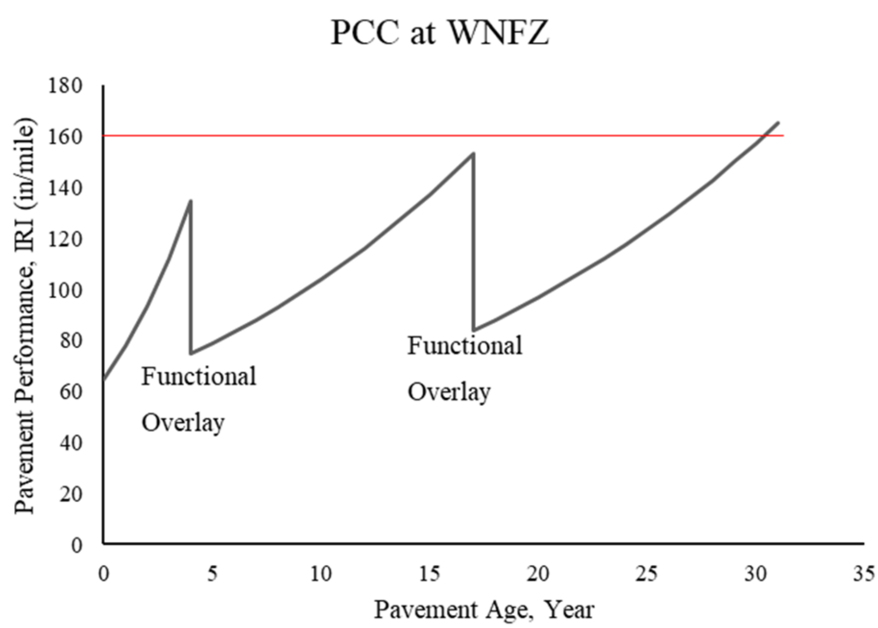


Figure 5.4: Optimal profile involving two rigid pavement preservation treatments, wet non-freeze zone

5.4.3. Dry-Freeze Climatic Zone

A structural overlay of flexible pavement is the dominant type of treatment in the dry-freeze zone. Based on the optimal schedule of treatments presented in Figure 5.5, the structural overlay must be implemented in years 8 and 14 of the pavement's life. If these two treatments are implemented at the pre-determined maximum performance indicator (IRI = 160 in/mile), the pavement's life will be extended to 27 years.

Rigid pavement requires two different treatments at each stage: functional and structural overlays at stages 1 and 2, respectively. These treatments are to be applied at years 11 and 14 of the pavement's service life, as shown in Figure 5.6. The pavement is expected to reach its performanc threshold at year 25.

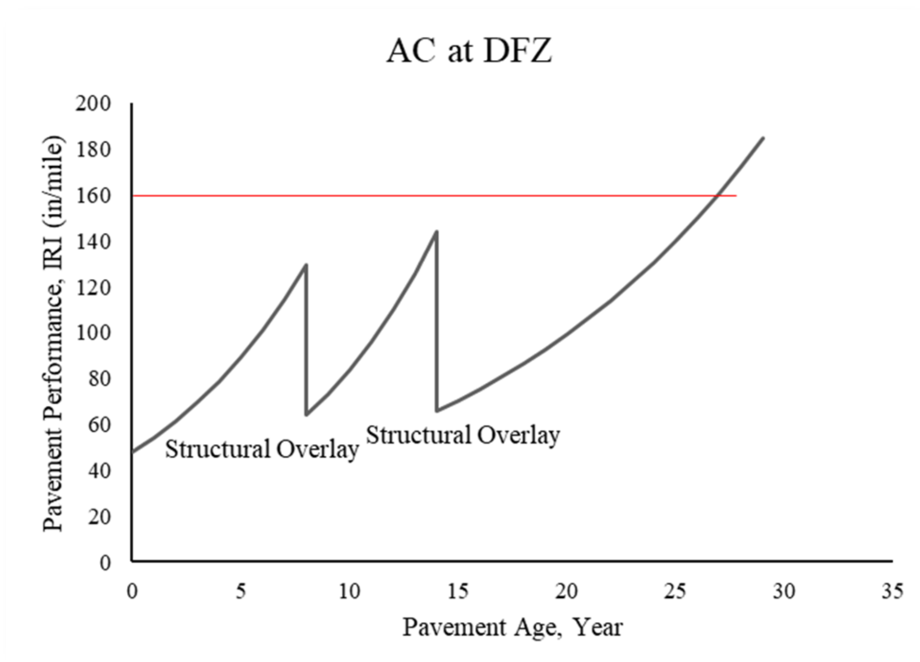


Figure 5.5: Optimal profile involving two flexible pavement preservation treatments, dry freeze zone

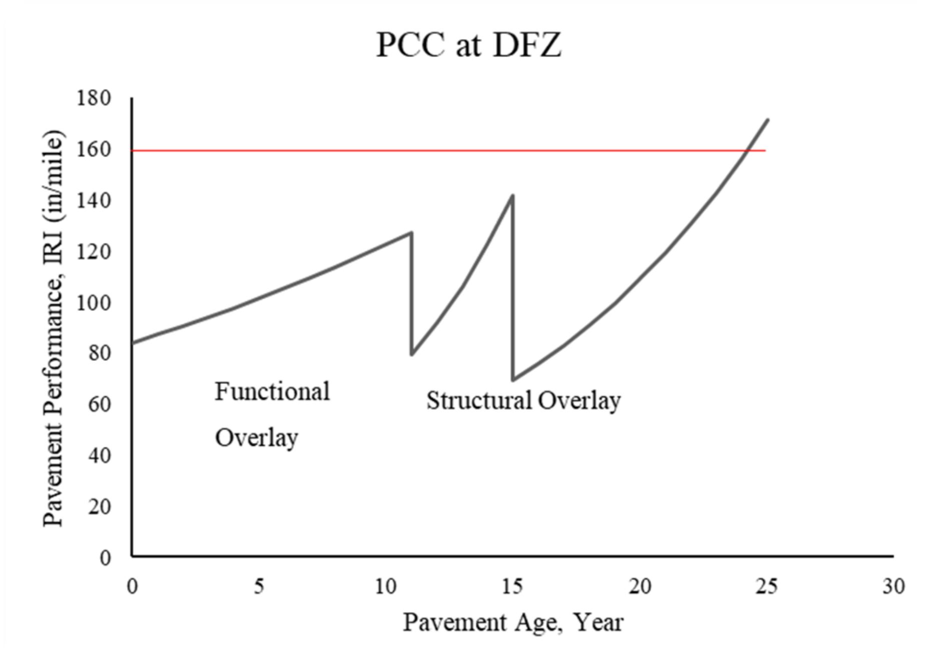


Figure 5.6: Optimal profile involving two rigid pavement preservation treatments, dry freeze zone

5.4.4. Dry-Non-Freeze Climatic Zone

Functional and structural overlays should be implemented on the asphalt pavement in the dry non-freeze climatic zone. As shown in Figure 5.7, the treatments must be completed at years 6 and 10 for the functional and structural treatments, respectively. Reconstruction or major rehabilitation is required at year 17 when the pavement condition reaches the maximum performance threshold.

In the dry non-freeze climatic zone, the structural overlay treatment should be applied during both stages. The structural overlay is to be applied at years 4 and 11, as shown in Figure 5.8. This profile reaches the end of its service life at year 18 when the performance threshold is at 160 in/mile.

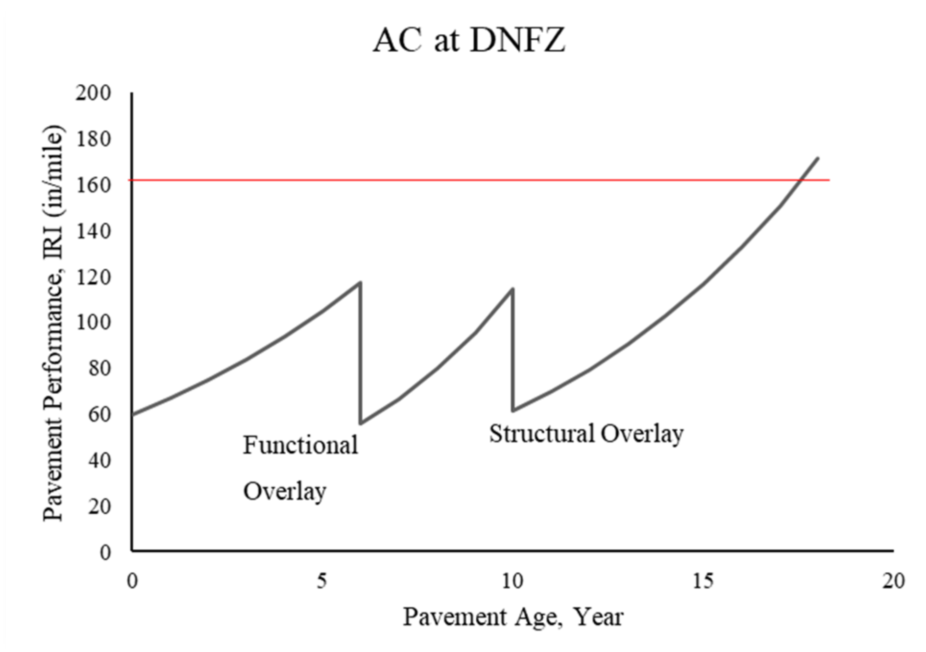


Figure 5.7: Optimal profile involving two flexible pavement preservation treatments, dry non-freeze zone

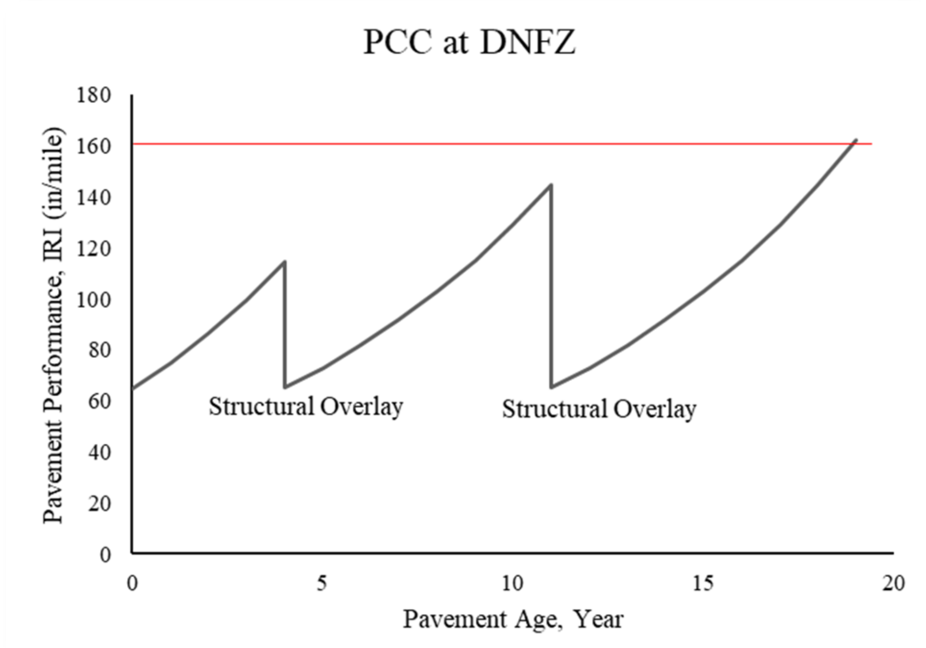


Figure 5.8: Optimal profile involving two rigid pavement preservation treatments, dry non-freeze zone

5.4.5. Comparison of the Optimal Profiles Across the Climatic Zones

It is worth mentioning that no preventive maintenance was chosen as a treatment in any of the optimal profiles across the different climatic regions, perhaps because preventive maintenance is less cost-effective than a functional overlay. In general, a functional overlay extends the life of the pavement almost twice as much as the thin overlay treatment with only about a 22% increase in treatment cost. As presented in the figures above, three treatment combinations were reported: functional overlay for both stages, structural overlay for both stages, or functional overlay for the first stage and structural overlay for the second stage. These treatment plans were chosen based on the best overall cost-effectiveness of the entire profile.

It is reasonable that a functional overlay should be applied during the first stage, which is when the pavement condition is better compared to the pavement condition during the second stage, when the pavement deteriorates faster and major rehabilitation is expected soon. In some cases, primarily wet non-freeze zones, a functional overlay is chosen for both stages; and in the case of wet-non-freeze zones, a functional overlay is ideal because of its longer service life. The range for the functional overlay in this zone is 12 to 18 years, which explains why this treatment appears to be more attractive than the structural overlay, even during the second stage. This treatment will extend the pavement service life while incurring a relatively low agency cost compared to a structural overlay, which costs about 72% more. The pavement service life for both the flexible and rigid pavements varies based on the nature of the treatments and their behavioral changes in different environmental conditions. These optimal profiles are highly sensitive to the prediction models of the treatments. The optimal schedule will best represent the real-life scenarios, reflecting the actual performance of the pavement.

5.4.6. Comparison of the Optimal M&R Schedules Developed in this Dissertation vs. Schedules Developed in the Literature and in the State of Practice

In this dissertation, the optimal M&R schedules for the different climatic zones were developed at the average levels of the input variables used for the analysis. These variables include the threshold pavement condition (for each preservation treatment), and the average levels of annual daily traffic, truck traffic percentage, annual freeze index, and annual temperature. To investigate the effect of the same traffic level across the different climatic regions, the traffic loading levels were kept fixed across the climatic regions. Generally, the research found that the optimal schedule is very sensitive to the levels of the input variables. Similar results were found when these results were compared to the schedules developed in the literature, particularly for flexible pavements in the LTPP wet freeze zone (Figure 5.1). In Irfan's (2010) study in Indiana (a wet freeze state), the inputs were similar to those of the current dissertation (the major exception being the inclusion of community costs in the analysis), and the optimal schedule involved the application of thin HMA overlay in year 11, functional overlay in year 20, and end-of-life in year 31. In the current dissertation, the optimal schedule for that LTPP zone is: functional overlay in year 4, structural overlay in year 11, and end-of-life in year 25. The optimal profile is expected to be different for different levels of the input variables and different datasets used to build the treatment-specific performance and cost models. Between these two optimal profiles, the difference in the pavement service life is only 6 years. Comparing two optimal schedules with two distinctly different inputs generally not always practical. However, this comparison was carried out in this section of the dissertation in order to throw more light on the transferability of the optimal schedules developed in this dissertation and how it compared to the schedules found in the literature.

The results of the dissertation were also compared with the state of practice. Most states transportation agencies are adopting M&R schedules based on common (historical) practice or expert opinion (Ahmed, 2012). The typical M&R schedule consist of several preventive maintenance and few major rehabilitation treatments. An example of this is the schedule provided in the INDOT manual for concrete and asphalt pavements (INDOT, 2013): for the flexible pavement, the M&R schedule contains 10 crack sealing treatments, 4 joint seal treatments, and 2 mill and fill functional overlay (rehabilitation) treatments over the 50-year analysis period. For rigid pavements, the M&R schedule consists of joint seals applied at years 8, 16, and 24, HMA overlay applied at years 30 and 42, and crack seals at years 30, 36, 39, 45 and 48 for the same analysis period.

The developed M&R schedule for the desert-like climatic region (for example, in the state of California) starts with preventive maintenance at years 18 and 41, and HMA overlay (rehabilitation) at years 23 and 46 for 55 years of analysis (CADOT, 2013). For flexible pavements, similar M&R schedules were developed for California based on the different climatic regions within that state: 2 preventive maintenance and 2 rehabilitation treatments applied at within the analysis period. For rigid pavements, the M&R schedule for the 55 years analysis period contains 3 concrete rehabilitation treatments at years 25, 30 and 40, and slab replacement at year 45. The other M&R alternative schedule for the rigid is to apply 2 AC overlays at year 30 and 38, and the slab is replaced at year 45.

Other states have developed their pavement M&R schedules based on the actual practice and based on their agency's standards. Across the state practices, it is common to observe 3 to 4 major preservation treatments over an analysis period of 40–50 years. In this dissertation, the flexible pavement service life was 17–36 years with only two major preservation treatments, and the rigid pavement had a service life of 18–45 years with 2 preservation treatments applied. In general, the optimal profiles developed in this dissertation had only 2 preservation treatments but provided a pavement life of as much as 30–40 years while the state of practice typically had 3 to 5 major preservation treatments. It is not easy to compare the optimal schedules developed in this dissertation with the actual (state of practice) because the former is developed based on predetermined levels of the input variable whereas the actual practice may not have the same combinations of the levels of the input variables. In addition, this dissertation developed schedules based on LCA criteria while those in the practice may be based on a different set of goals. Nevertheless, the optimal schedule can serve as a good reflection of the data quality and scheduling decisions based on the optimal profile can be expected to be more reliable.

5.5. The LCCA Deterministic Approach

The deterministic approach uses inputs to those used to obtain the optimal schedule for all the cost components. Pavement age was derived from the optimal profile for each pavement material using the interest rate of 4%. Cost models were used to determine optimal preservation schedules; however, the average unit cost of treatments could safely be used at this stage, as earlier presented in Table 4.21 and Table 4.22. The same user cost assumptions used for the optimal preservation profiles also were adopted for the LCCA deterministic approach: a duration model for each treatment using an average fuel price of $2.40 per gallon; a fuel consumption rate of 2.04 gallons/hour for automobiles and 20.73 gallons/hour for trucks; an automobile delay rate of $13 per hour and a corresponding truck rate of $24 per day; an average speed of 70 mph (65 mph for trucks); a work zone speed of 45 mph (40 mph for trucks); and an AADT of 50,000 with 19%

truck traffic. The costs associated with air pollution were evaluated based on the GWP of the greenhouse gas emissions represented in CO_2 equivalents and the energy consumption used throughout the LCA phases. Regarding CO_2, the estimated social cost of manufacturing one ton of pavement was $32.15 in 2017 constant dollars with a 4% interest rate. The average cost of energy consumption was assumed to be $0.02 per MJ. The roadway geometry assumptions were based on interstate highways standards. The cost associated with noise pollution was estimated based on the height of the noise barrier and the relationship presented in Equation 3.40 (Sinha and Labi, 2007). The assumed maintenance cost for the noise barrier was 5% of the value of the asset annually.

After obtaining the optimal schedule, the EUAC was evaluated for both the AC and PCC pavements. EUAC was used because the pavement service lives of flexible and rigid pavements are not equal. One might argue that EUAC is a method suitable only for evaluating costs, not benefits. Although the EUAC does not explicitly evaluate effectiveness, it does implicitly consider effectiveness when obtaining the optimal activity profile with the area over the curve technique. Cost efficiency was evaluated using two methods: (1) the deterministic approach and (2) the risk analysis approach.

5.5.1. Wet-Freeze Climatic Zone

The objective function was designed to achieve the maximum B/C ratio separately for each pavement material. Using a cost-effectiveness method in which the benefits are evaluated using the area over the curve and the costs are estimated using EUAC, rigid pavements were found to be superior to flexible pavements in this climatic region. The optimal solutions for both pavement materials are presented in Figure 5.9, using different discount rates. The figure shows that the B/C ratio increased as the discount rate increased. For example, assuming everything else remained constant, if the discount rate were to increase from 1% to 2%, the B/C would increase by 0.01 unit as shown below:

$$Benefit - Cost\ Ratio = \frac{Benefit}{Cost} = \frac{AOC}{PW} = \frac{AOC}{(1 + i\%)^{-N}}$$

$$Benefit - Cost\ Ratio = \frac{AOC}{PW} = \frac{1}{(1+0.01)^{-1}} = 1.01$$

$$Benefit - Cost\ Ratio = \frac{AOC}{PW} = \frac{1}{(1+0.02)^{-1}} = 1.02$$

$$Increase\ in\ Benefit - Cost\ Ratio = 1.02 - 1.01 = 0.01$$

The effect of the traffic volume and the freezing index on the choice of pavement material in the LTPP wet-freeze zone also was investigated, and the results are shown in Figure 5.10 and Figure 5.11. As expected, an increase in truck traffic affected the attractiveness of both pavement materials. Figure 5.10 shows a sudden reduction in the B/C ratio when the truck traffic exceeded 1,049 trucks per day, while the B/C ratio continued to decrease at a slower rate. This sudden decrease was caused by the inclusion of the noise barrier cost after the truck traffic exceeded 1,050 trucks per day. Before this cutoff point, the noise barrier was not required because the noise level did not exceed the pre-defined noise level threshold. The effect of truck traffic and the proportions of PCC and AC B/C ratios are shown in Figure 5.11. The dashed red line in this figure represents the borderline between flexible and rigid pavement superiority. Anything above that line indicated the rigid pavement's superiority, and anything below it indicated the superiority of the flexible pavement. However, flexible pavement was found to be the superior pavement material only for roads with low traffic volume and an annual freezing index of 4,000, as well as for roads with AADT under 10,000 (or 2,000 ADTT). Also, the difference between the PCC and AC pavements was found to be negligible (less than 8% superiority of PCC over AC). The PCC pavement was the dominant choice for the LTPP wet-freeze zone when the truck traffic exceeded 2,000 trucks per day.

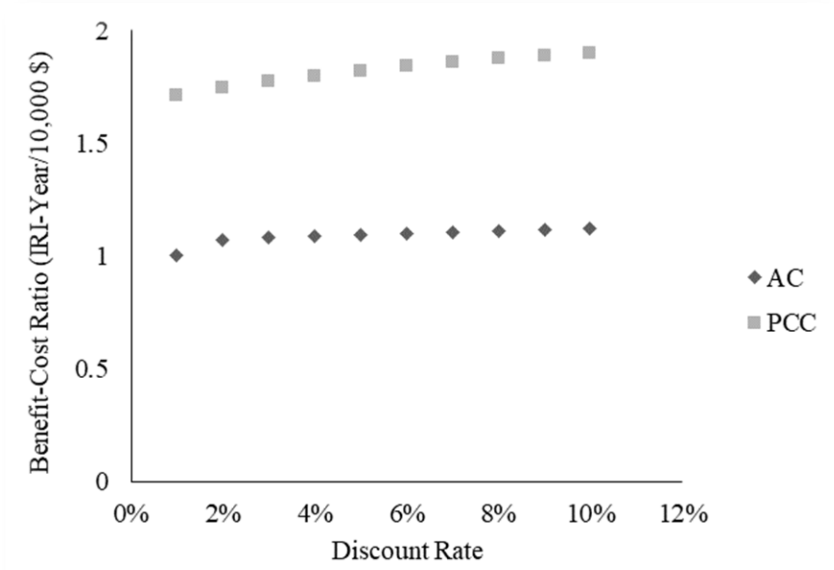


Figure 5.9: Effect of discount rate on the relative attractiveness between AC and PCC pavements, wet freeze zone

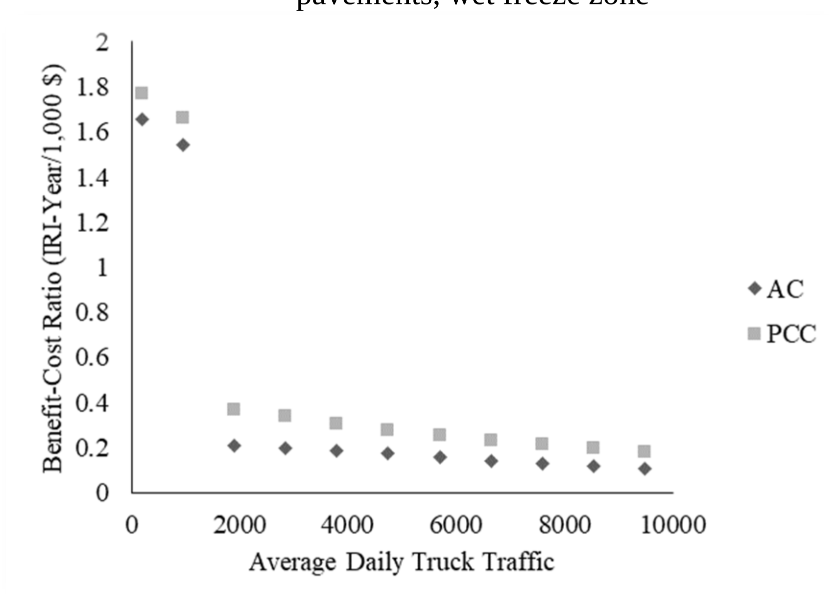


Figure 5.10: Effect of truck traffic on the relative attractiveness between AC and PCC pavements, wet-freeze zone

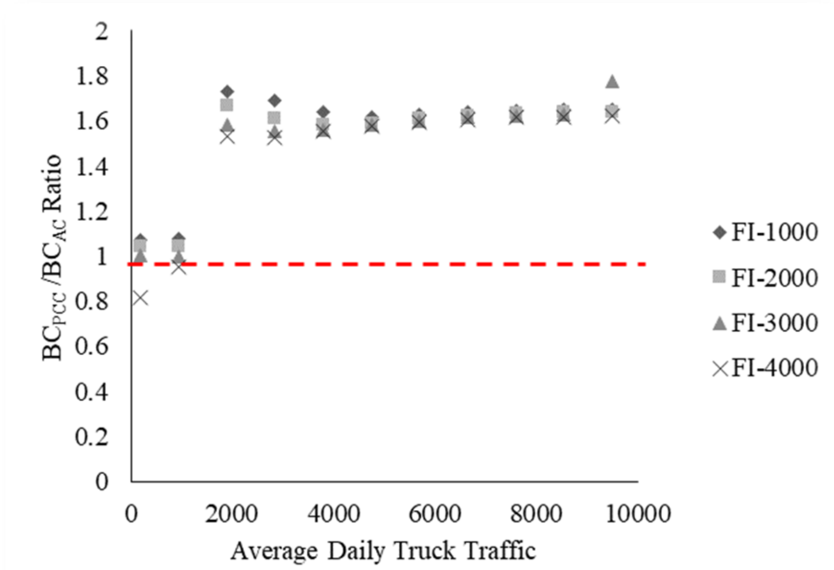


Figure 5.11: Effect of truck traffic on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC, wet-freeze zone

5.5.2. Wet Non-Freeze Climatic Zone

In this climatic region, the rigid pavement also was found to be superior to the flexible pavement in terms of cost-effectiveness. As shown in Figure 5.12, the effect of the discount rate was similar to that in the wet freeze zone, with about the same difference in magnitude between the asphalt and concrete pavements (1 unit of B/C ratio in favor of PCC pavement). Similarly, the B/C ratio followed the same trend for the effect of the truck traffic volume as in the wet-freeze zone. A sudden reduction in the B/C ratio appeared after the truck traffic exceeded 2,000 trucks per day. At that point, the reduction rate started to slow down, as shown in Figure 5.13. As discussed in the previous section, the sudden decrease in the B/C ratio resulted from including the noise barrier cost when the truck traffic exceeded a certain noise level threshold. The main difference in this climatic region is that the freeze index was found to be insignificant and only truck traffic and pavement age were the major factors predicting pavement performance. In this case, PCC pavement was always superior to AC in terms of cost-effectiveness, as shown in Figure 5.14. For example, at 2,000 or so trucks/day, the rigid pavement had about twice the attractiveness of AC. This level then started to decrease to a point at which PCC was 46% more superior to AC in the 10,000 trucks/day scenario.

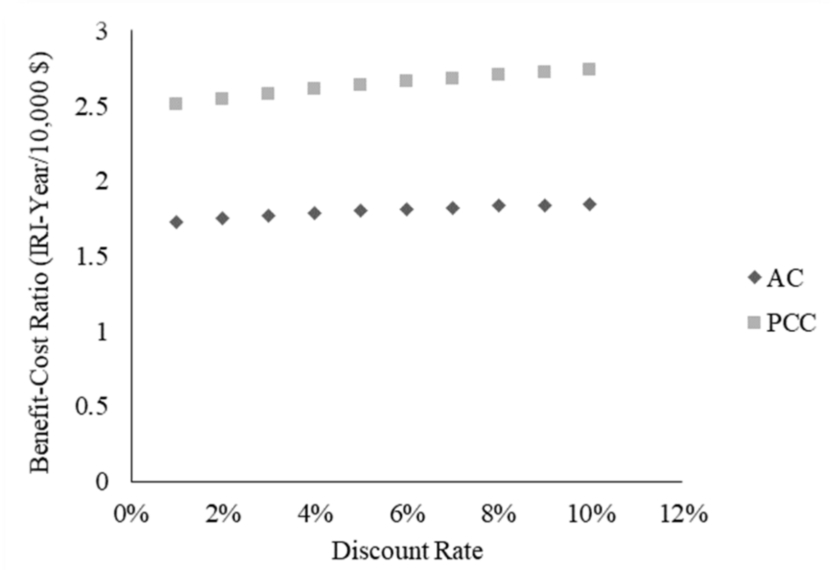


Figure 5.12: Effect of discount rate on the relative attractiveness between AC and PCC pavements, wet non-freeze zone

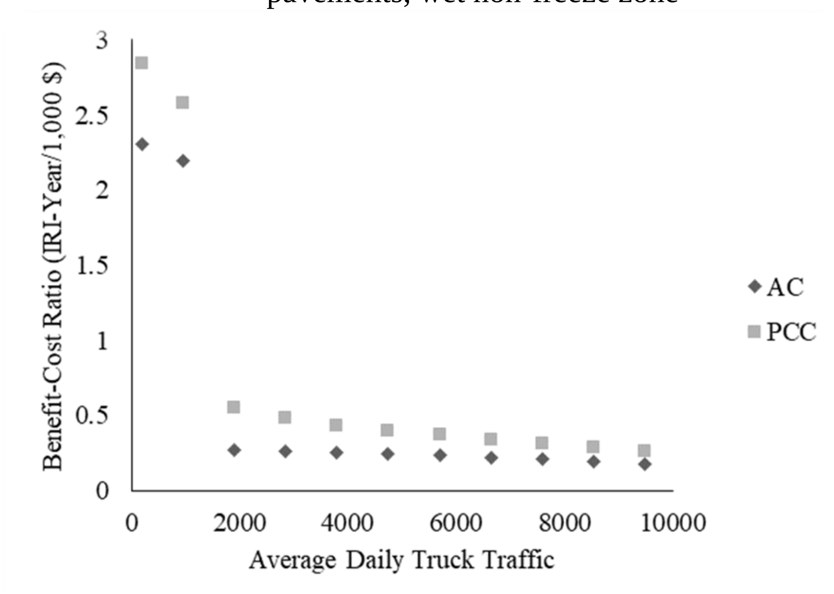


Figure 5.13: Effect of truck traffic on the relative attractiveness between AC and PCC pavements, wet non-freeze zone

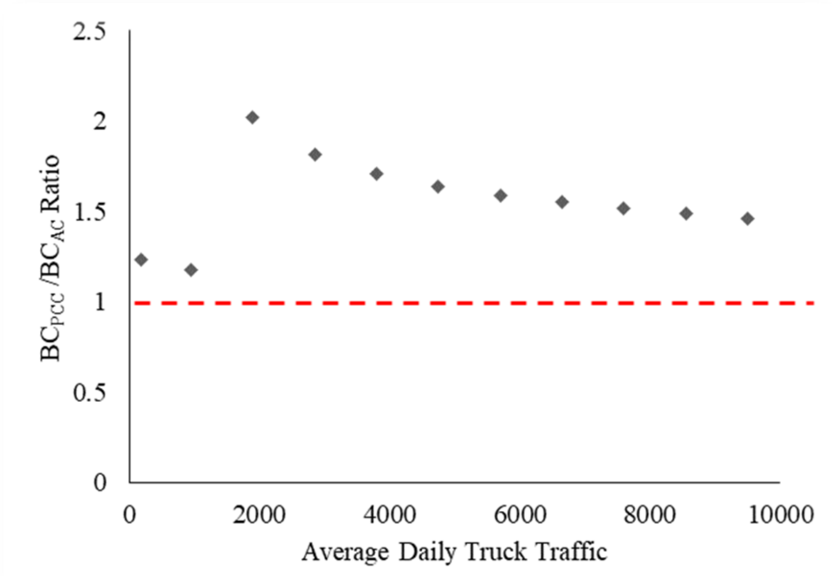


Figure 5.14: Effect of truck traffic on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC, wet non-freeze zone

5.5.3. Dry-Freeze Climatic Zone

Asphalt pavement was the best choice of pavement material in the dry-freeze zone, even with high freeze indices. In the dry-freeze climatic region, the B/C ratio increased as the discount rate increased with the superior flexible pavement, as shown in Figure 5.15. The effect of traffic loading on the B/C ratio is shown in Figure 5.16. Although the effect of the freeze index was found to be significant in this climatic region, it had a similar effect, no matter how high the freeze index was, as shown in Figure 5.17. Flexible pavement was found to be the best choice of paving material in the dry-freeze zone. The superiority range of the flexible pavement over rigid pavement was between 22% and 51%.

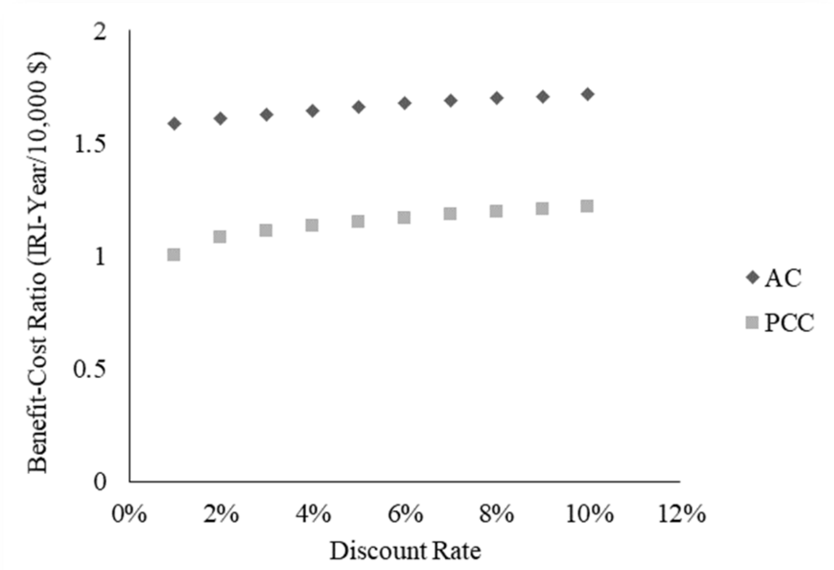


Figure 5.15: Effect of discount rate on the relative attractiveness between AC and PCC pavements, dry freeze zone

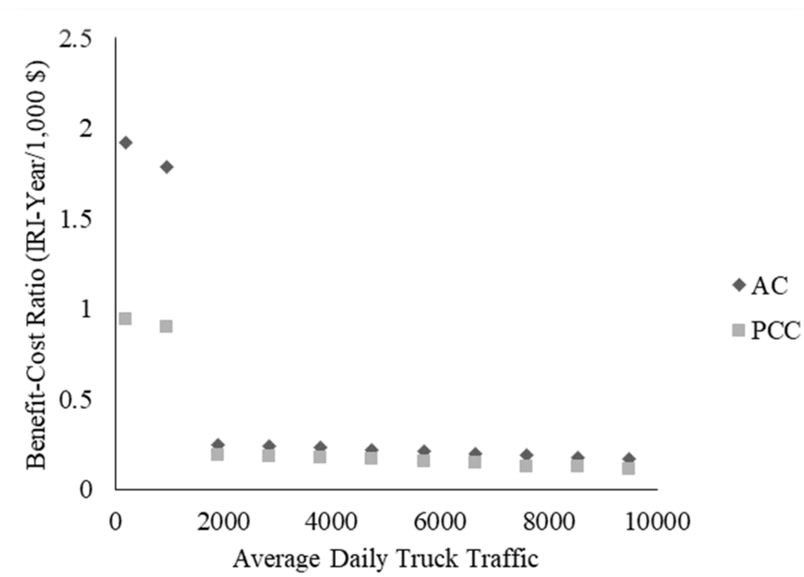


Figure 5.16: Effect of truck traffic on the relative attractiveness between AC and PCC pavements, dry freeze zone

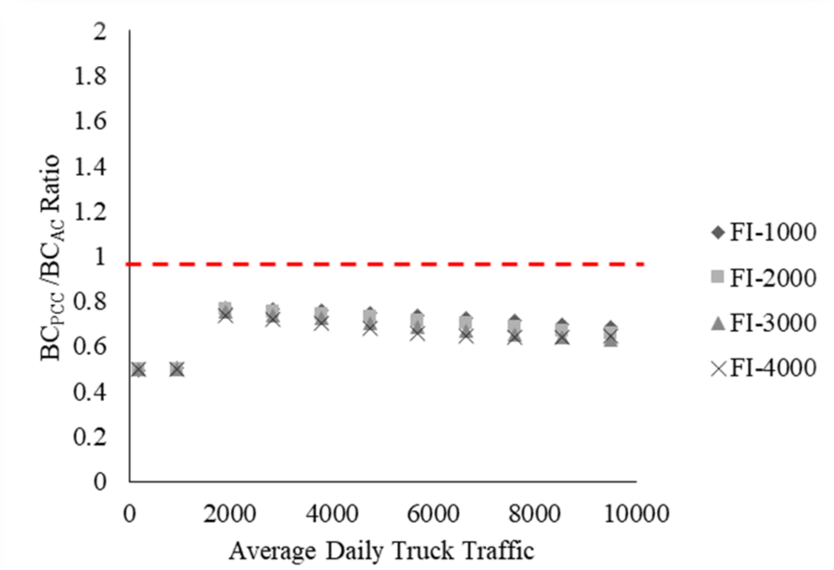


Figure 5.17: Effect of truck traffic on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC, dry freeze zone

5.5.4. Dry-Non-Freeze Climatic Zone

Flexible pavement also was found to be superior to rigid pavement in the dry-non-freeze zone. The flexible and rigid pavement B/C ratio increased with an increase in the discount rate, which revealed a smaller difference in magnitude between AC and PCC, compared with the discount rate in other regions (0.2 on average) (see Figure 5.18). The effect of the traffic loading on the B/C ratio of flexible and rigid pavements is shown in Figure 5.19. Similar to the wet-non-freeze zone, the freezing index in this zone was found to be insignificant, and only truck traffic and pavement age were major factors in predicting pavement performance. Flexible pavement was found to be superior to rigid pavement with different levels of traffic loadings, as shown in Figure 5.20. The flexible pavement in this climatic region was about 23% more superior to the rigid pavement, especially when truck traffic loading exceeded 2,000 trucks/day.

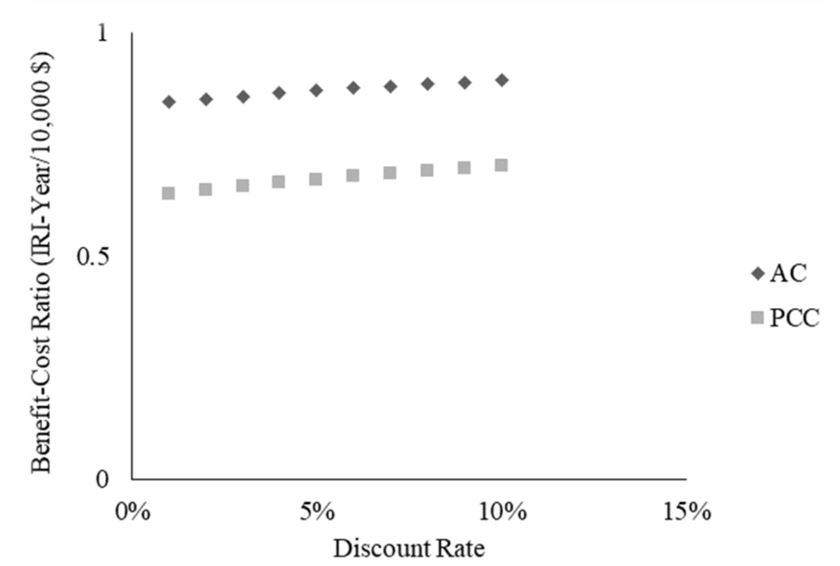


Figure 5.18: Effect of discount rate on the relative attractiveness between AC and PCC pavements, dry non-freeze zone

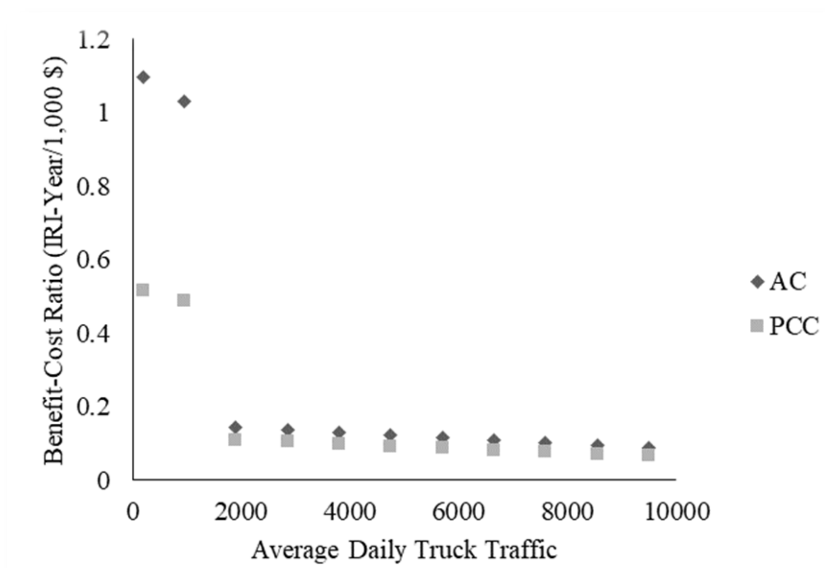


Figure 5.19: Effect of truck traffic on the relative attractiveness between AC and PCC pavements, dry non-freeze zone

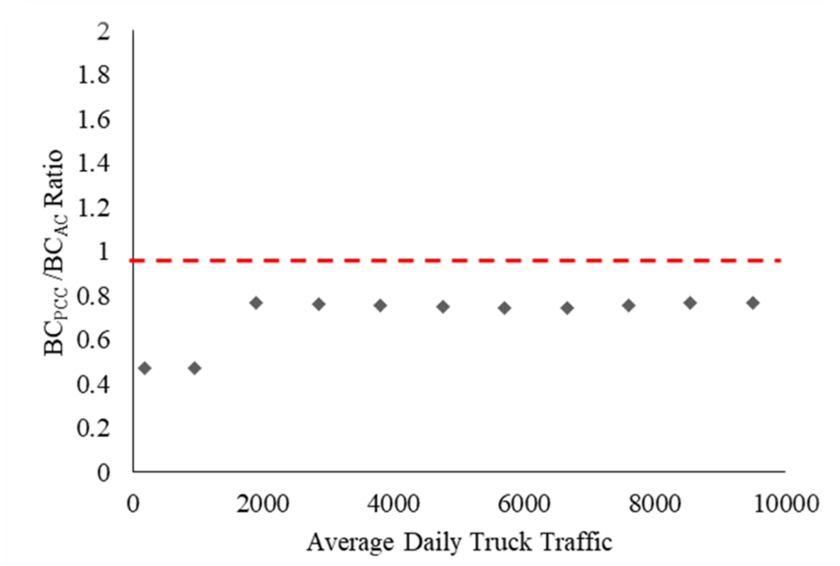


Figure 5.20: Effect of truck traffic on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC, dry non-freeze zone

5.5.5. Comparison across Different Traffic and Climatic Conditions

The optimal schedules of M&R treatments were determined for the LTPP climatic regions based on the cost-effectiveness of the overall profile. The effect of the average daily truck traffic on the choice of the pavement material in the four LTPP climatic zones, including the effect of the average annual freeze index (when found significant), is shown in Figure 5.21. Using the output of the deterministic optimization method, the results of the deterministic approach show that rigid pavement is the better choice of paving material for wet climates (wet-freeze and wet-non-freeze) and flexible pavement is more suitable for dry climates (dry-freeze and dry-non-freeze). Each point in Figure 5.21 represent an optimal solution with different M&R schedules. It appears that choosing paving material heavily depends on the presence and absence of moisture. The freeze index had a small impact when it comes to selecting the pavement material type (asphalt or concrete). Based on Figure 5.21, this dissertation concluded that rigid pavement is the best material choice for wet climates with heavy truck traffic and flexible pavement is more desirable for dry climates.

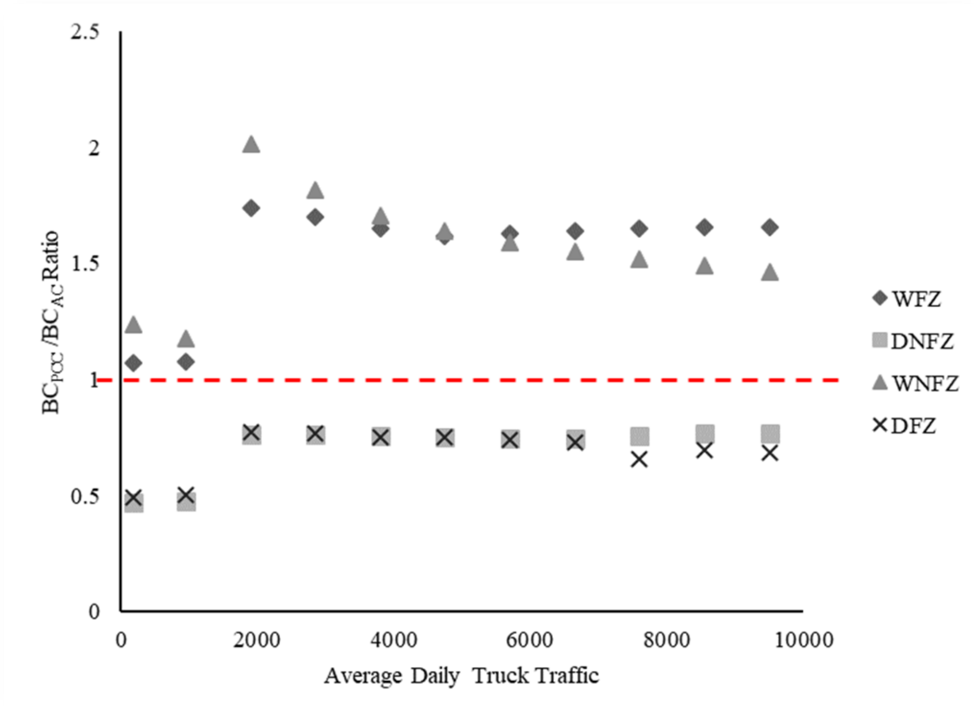


Figure 5.21: Effect of truck traffic on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC across the climatic zones

The optimal schedule of M&R was highly sensitive to traffic loadings and any major change in traffic volume likely could alter the optimal activity profile. Traffic volume was divided into three major categories: low (less than 10,000 vehicle/day), medium (between 10,000 and 20,000 vehicle/day), and high (more than 20,000 vehicle/day) traffic volume to investigate its effect of the optimal solution across the four LTPP climatic zones (Smith et al., 2011). The B/C ratios of the optimal M&R schedules of these different traffic load categories are presented in Figure 5.22 and 5.23 for flexible and rigid pavements, respectively. As expected, the results show that the attractiveness (the magnitude) of the B/C ratio of the optimal profile would decrease as the traffic volume increases because of the change in the optimal M&R schedule. This change in the optimal M&R schedule is the result of the change in the traffic volume. Pavements deteriorate faster with heavy traffic volumes, which then would require more frequent (and higher level) M&R treatments that would increase the cost and lead to a reduction in the B/C ratio.

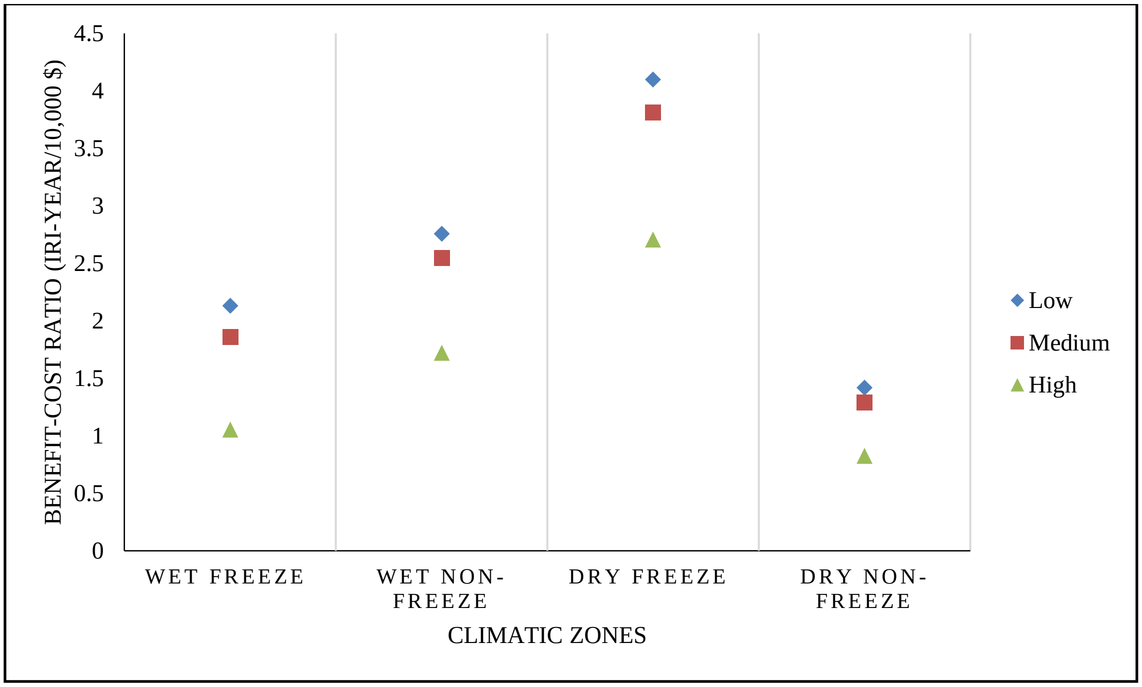


Figure 5.22: Effect of different truck traffic (low, medium, and high) on the B/C ratio of the AC pavement across the climatic zones

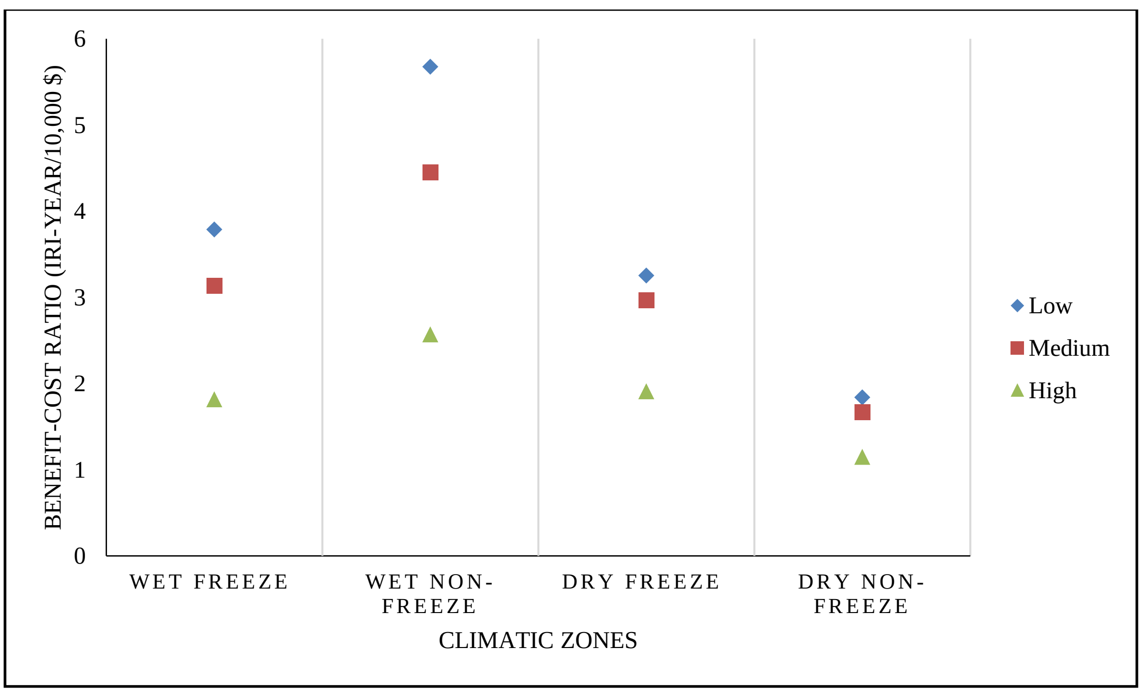


Figure 5.23: Effect of different truck traffic (low, medium, and high) on the B/C ratio of the PCC pavement across the climatic zones

5.6. The Stochastic LCCA Approach

The probabilistic approach was considered in this research because of the uncertainty associated with the input variables for LCCA. As discussed in Chapter 3, risk analysis (i.e., the probabilistic approach) combines two methods: (1) the probabilistic entries of uncertain input variables for LCCA; and (2) computer simulations to capture the risk of the LCCA outcomes. The results of this method are presented as probability distributions that illustrate the range of the outcomes of the analysis. The steps in the probabilistic method are as follows (Walls and Smith, 1998):

1. Identify the problem
2. Quantify the uncertain inputs using the proper probability distribution
3. Implement the computer simulations
4. Analyze the results
5. Make a decision

The first step already has been explored by obtaining the optimal schedule of preservation activities throughout a pavement's service life. The second step requires assuming the proper distribution of the major inputs for all of the cost components. It is safe to assume that both the average unit costs of the pavement preservation treatments and the interest (discount) rates are normally distributed, since raw data of the preservation unit costs are not available. The unit costs adopted from the literature include the average, minimum, and maximum values, as well as the standard deviation of the costs of each particular preservation treatment. The range of the discount rate input is between 1% and 10% for all cost components in this dissertation.

The next step was to perform simulations of the given scenarios. Simulation was considered to enhance the sensitivity analysis in that different randomly selected values from the implemented type of probability distribution were used to estimate the discrete outputs. Monte Carlo simulation is the process of incorporating random numbers into a data sample based on the probability distribution (Rubinstein et al., 1981). This method requires the use of the cumulative probability distribution of the input distribution. Random numbers along the y-axis of the cumulative distribution chart are generated using a uniform distribution, where all values have an equal probability of being selected. The x-axis in this case study shows the EUAC of each cost component (including agency, user, and community costs).

Each iteration represents a possible output and was subjected to careful statistical analysis. Then, samples were drawn from the probability distribution until the given number of iterations was completed or the simulation process converged, whichever happened first. The assumed number of iterations was 10,000 because Monte Carlo simulation requires a large number of samples to guarantee enough representation of samples with a low probability value. The change in the average total cost when 1,000 rather than 10,000 iterations were used was negligible (less than 1%), which is why 10,000 iterations was a sufficient number of iterations to use in this dissertation. Histograms of the normal distribution were assumed, and the cumulative probability distribution of the risk profile for the total cost (including agency, user, and community) in this case study for the four LTPP climatic regions are presented in Figure 5.24 to Figure 5.31. The histograms of every cost component at each climatic zone are presented in Appendix B.

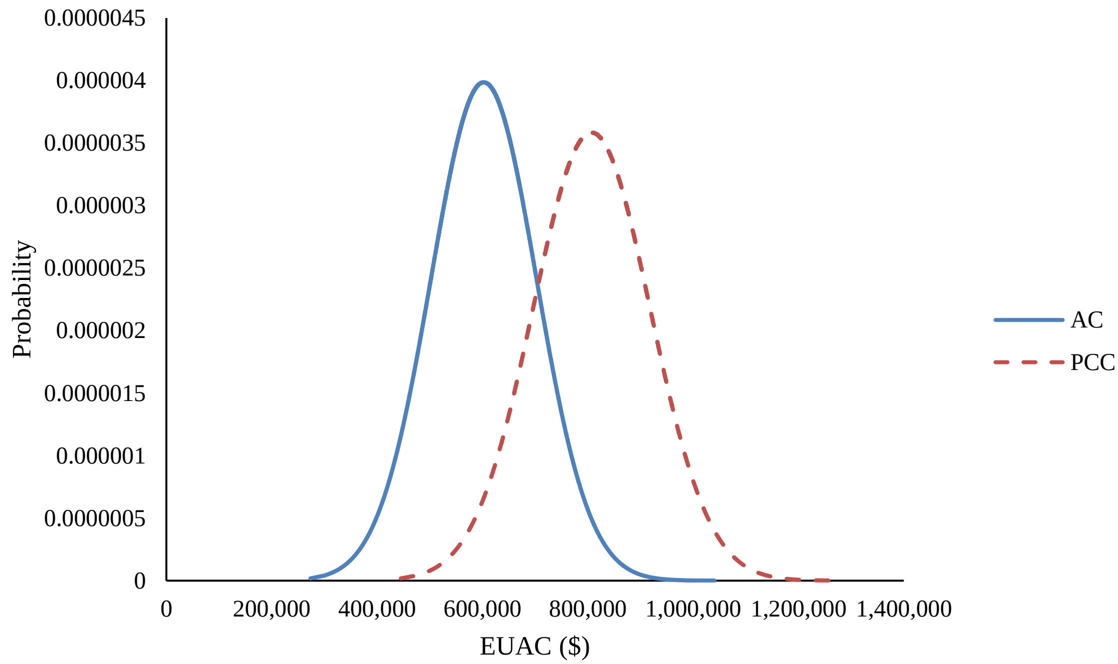


Figure 5.24: Comparative EUAC probability distribution of the total cost for AC and PCC pavements, wet freeze zone

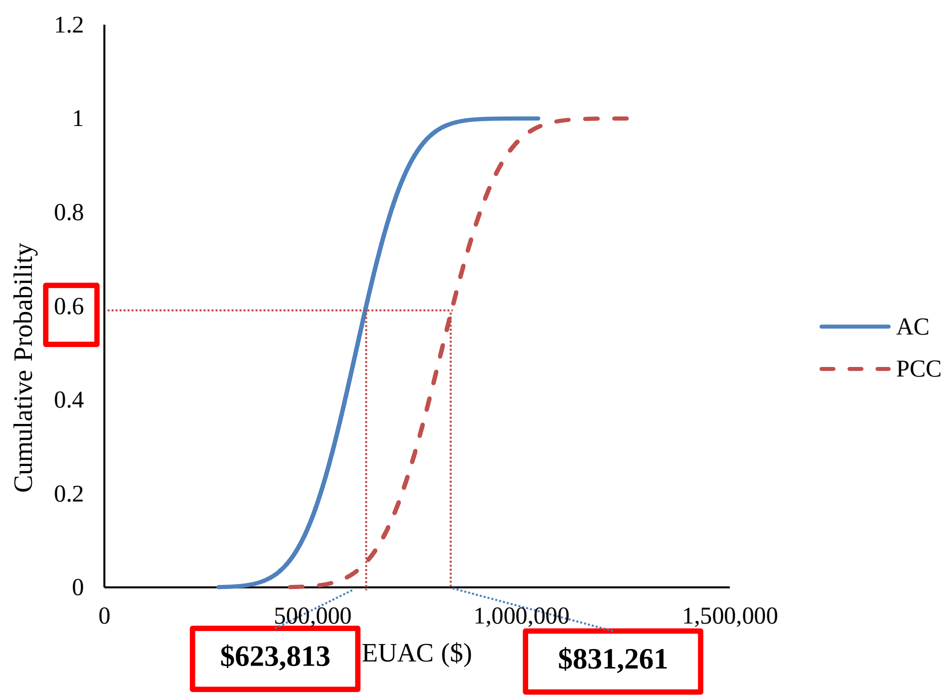


Figure 5.25: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements, wet freeze zone

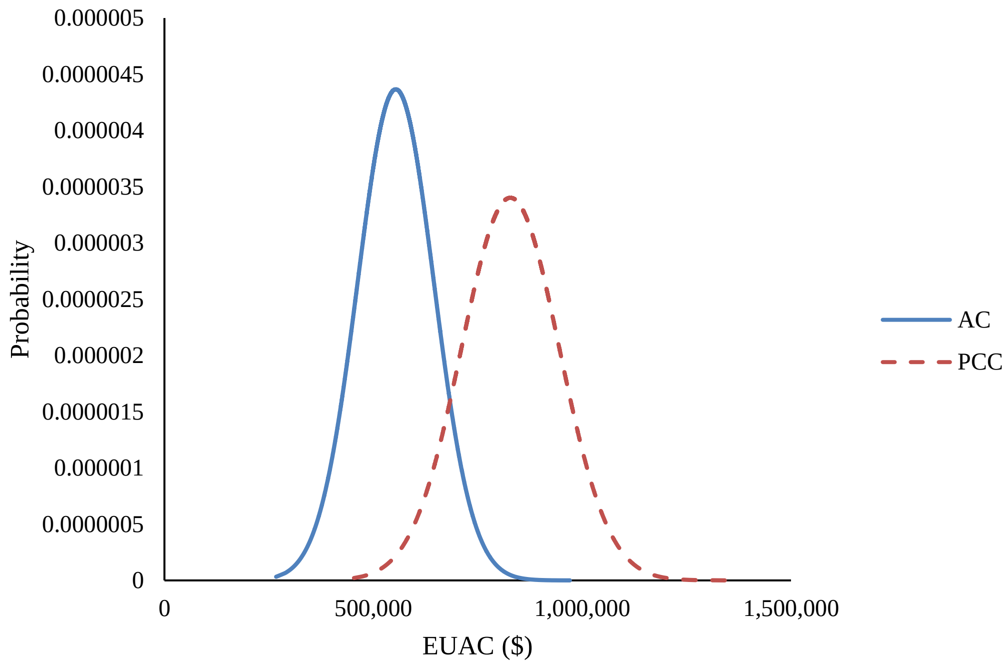


Figure 5.26: Comparative EUAC probability distribution of the total cost for AC and PCC pavements, wet non-freeze zone

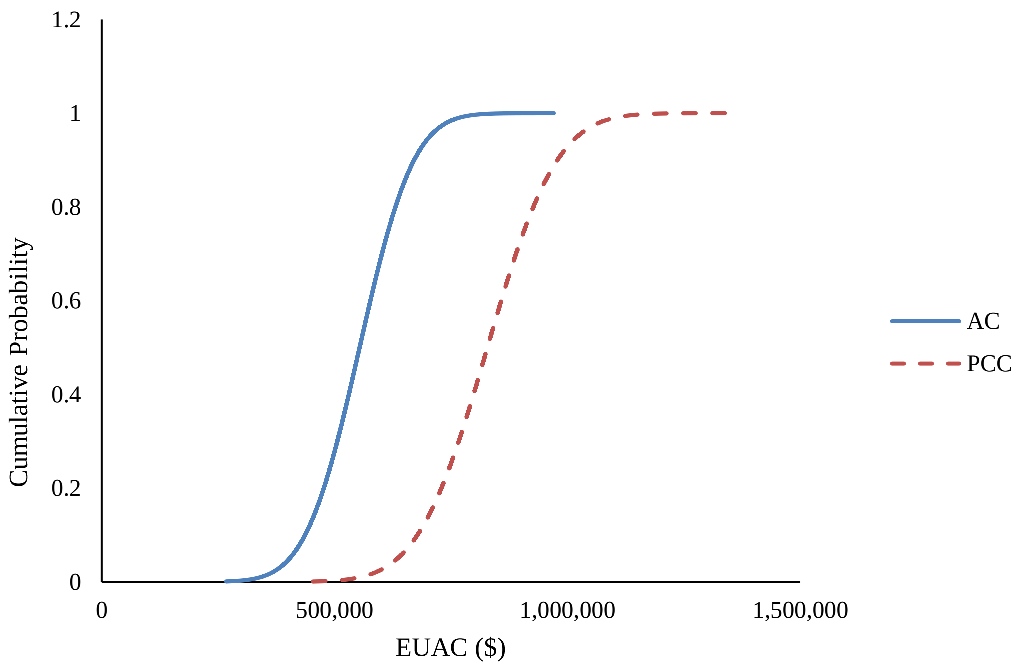


Figure 5.27: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements, wet non-freeze zone

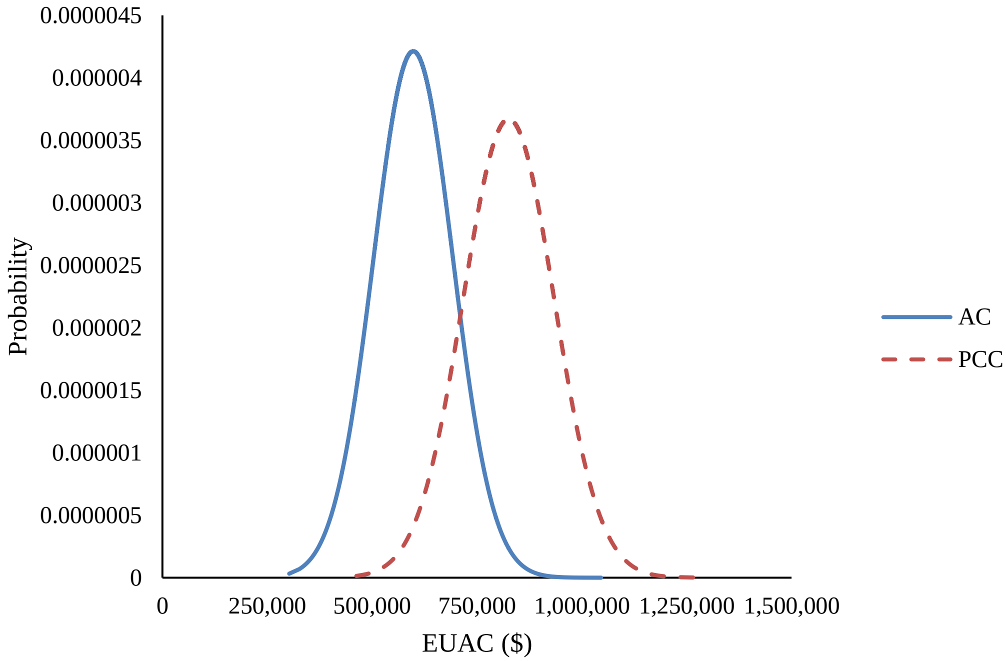


Figure 5.28: Comparative EUAC probability distribution of the total cost for AC and PCC pavements, dry freeze zone

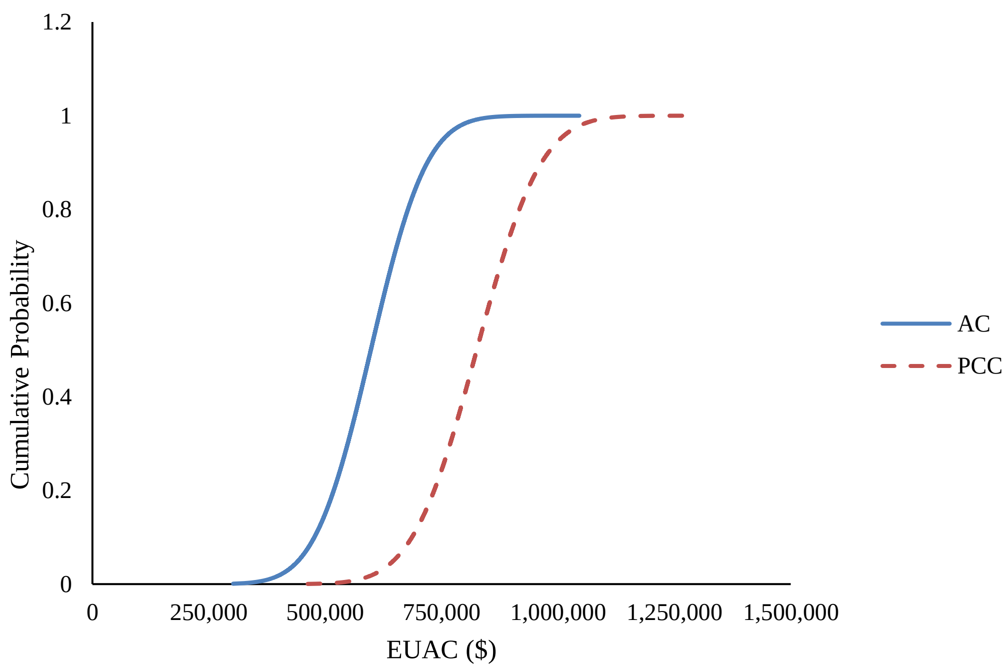


Figure 5.29: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements, dry freeze zone

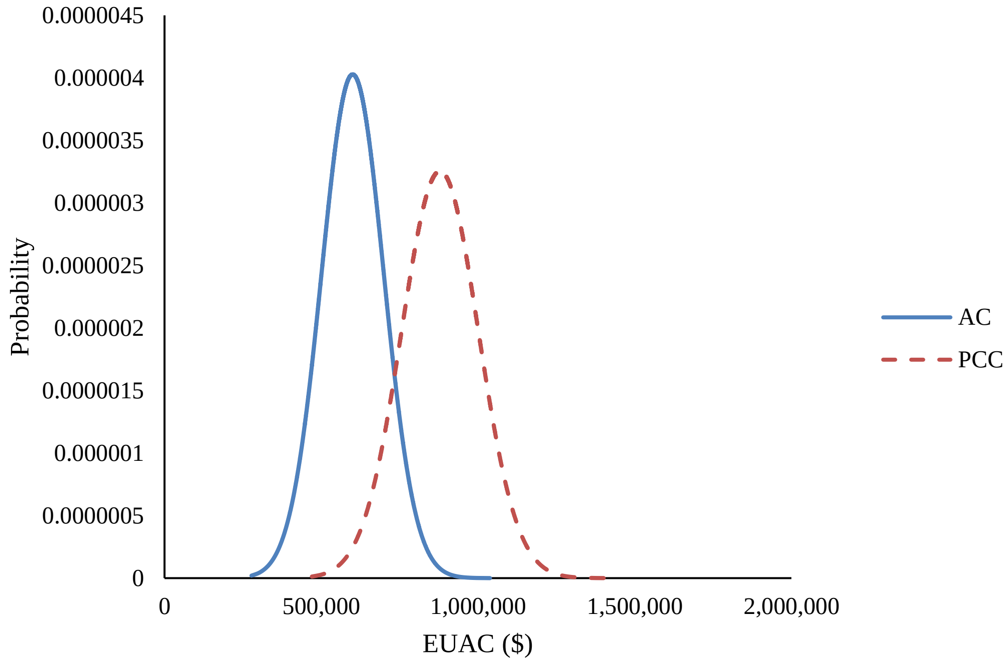


Figure 5.30: Comparative EUAC probability distribution of the total cost for AC and PCC pavements, dry non-freeze zone

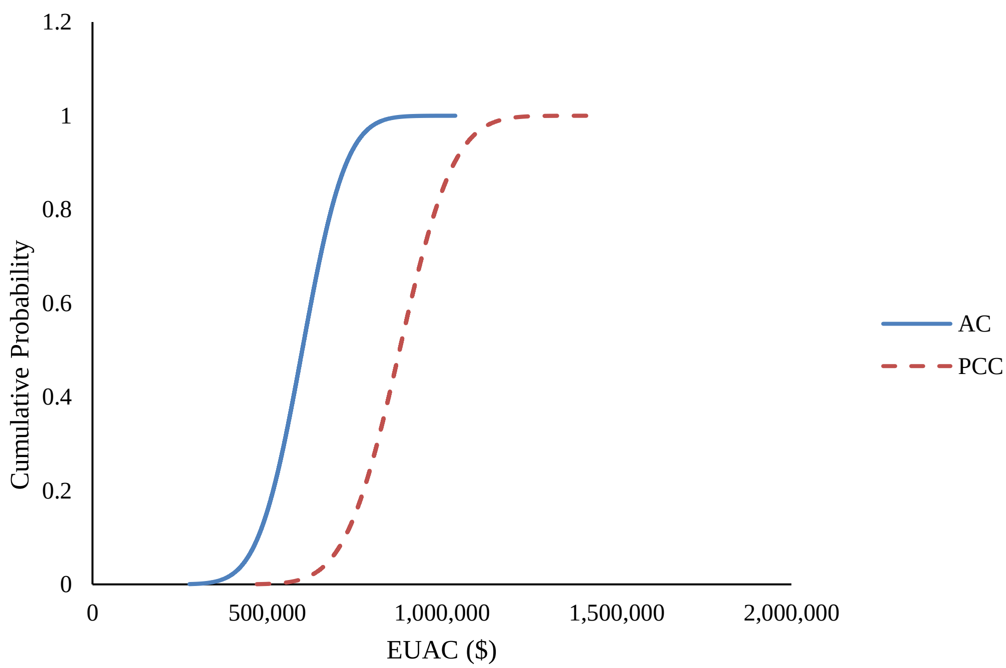


Figure 5.31: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements, dry non-freeze

Table 5.1: Summary of statistics of the EUAC of the total cost per lane-mile

	Flexible	Rigid
	Wet-Freeze Zone	
Mean	$ 602,539	$ 807,233
Standard Deviation	$ 100,096	$ 111,315
Minimum	$ 273,642	$ 445,244
Maximum	$ 1,040,458	$ 1,273,251
	Wet-Non-Freeze Zone	
Mean	$ 553,827	$ 828,192
Standard Deviation	$ 91,378	$ 117,270
Minimum	$ 267,427	$ 453,760
Maximum	$ 970,587	$ 1,348,703
	Dry-Freeze Zone	
Mean	$ 598,037	$ 825,565
Standard Deviation	$ 94,713	$ 108,717
Minimum	$ 302,366	$ 462,471
Maximum	$ 1,045,740	$ 1,280,127
	Dry-Non-Freeze Zone	
Mean	$ 600,294	$ 878,833
Standard Deviation	$ 99,053	$ 122,586
Minimum	$ 277,613	$ 470,287
Maximum	$ 1,038,230	$ 1,431,430

For most of the cost components (agency, user, and noise costs), the flexible pavement costs were lower than the rigid pavement costs, except for the costs associated with air pollution, and they have almost 100% probability. In Figure 5.25, which represents the wet-freeze zone, there is a 60% probability that flexible and rigid pavement costs will be less than $623,813 and $831,261, respectively. This means that after processing 10,000 iterations, 60% of the calculated EUAC values for these cost components were lower than $623,813 for AC and $831,261 for PCC. In this figure, the EUAC of flexible pavement is less than rigid pavement 100% of the time, when compared at the same probability. When this approach is used, the same trend can be traced in all the climatic zones (Figure 5.25, Figure 5.27, Figure 5.29, and Figure 5.31), which indicates that the most cost-effective pavement material across the different climatic zones is flexible pavement. In general, the steeper the slope of the curve on the cumulative probability charts, the lower the variability. The flexible and rigid pavement slopes in Figure 5.25 showed similar low variabilities, and the flexible pavement always had a lower cost compared to the rigid pavement, with 100% probability.

A potential reason why flexible pavement was superior, in terms of cost-effectiveness, is that the percentage of agency cost contribution to the total cost of rigid pavement is about 10% more than that of flexible pavement. Across the LTPP's four climatic regions, the agency cost associated with rigid pavement is almost twice the agency cost with flexible pavement, and the total cost of rigid pavement ranges from 34% to 50% more than the flexible pavement's total cost. The agency cost percentages of the total cost were 22% and 31% to 32% for flexible and rigid pavements, respectively. The user cost magnitude was higher for flexible pavement, with a range of 36% to 39% of the total cost compared to 29 to 31% of the total cost for rigid pavement. Community cost contributed the most, as a percentage of the total cost with 39% to 42% for the flexible pavement and 37% to 39% for the rigid pavement. Adding one dollar of agency cost to one dollar of user and community costs (equal weights for all cost components) would result in bias against the agency cost (which is the most important cost type, at least from the agency perspective). The effect of different weight assignments are discussed in the following section.

5.7. The Effect of Different Weights of Costs on the Optimal Profile

Two suggested methodologies were introduced in Chapter 3 (Section 3.8) for determining weight assignments for different types of costs: (1) the combination of the Delphi approach and direct weighting, and (2) the pairwise comparison using the Analytical Hierarchy Process (AHP). These methods are survey-based methods, where only one set of weights (for agency, user, and community costs) is to be reached at the end of the process. These methods are presented as guidelines for future studies. In this section, several combinations of weights were assumed across the different climatic regions in order to further investigate the effect of different weights on the pavement material selection.

A previous study suggested that $0.6286 of the user cost should be added to each dollar of the agency cost using the AHP weighting technique (Sinha et al., 2009). According to this study, the agency and user cost weights are 0.6140 and 0.3860, respectively. If we assumed equal weights for the user and community costs, then the weight assignment would be 0.6140 for the agency cost, 0.1930 for the user cost, and 0.1930 for the community cost. For simplicity, a scenario was introduced among the different scenarios of cost weight combinations with the following weights: 0.60 for the agency cost, 0.20 for the user cost, and 0.02 for the community cost. Several weight combinations were used to illustrate the effect of weight assignments on the optimal pavement material selection process, as shown in Table 5.2.

Agency cost is an essential cost component and cannot be overlooked in any given weight assignment scenario. The real question pertains to the magnitude of the weight given for each cost component. Equal weights for the agency, user, and community costs were assumed in the optimal profiles (treatment schedules) of the asphalt and concrete pavements presented in Section 5.6. The optimal profile experienced some changing of treatment scheduling when different weights were assigned to the three major cost components. Also, the magnitude of the B/C ratio of each pavement's optimal profile changed due to different weight assignments, which resulted in different optimal scheduling of M&R treatments. However, the effect of different weight combinations of the selection of the pavement material was of more interest in this dissertation. Previously, the results showed that rigid pavements are more cost-effective in the wet climates and the flexible pavements are superior in the dry climates. This finding held even after considering 18 different weight combinations as shown in Figure 5.32. This figure illustrates how the wet climate favored the rigid pavements and how the flexible pavements were found to be more suitable for dry climates even with different weight assignments for the agency, user, and community costs.

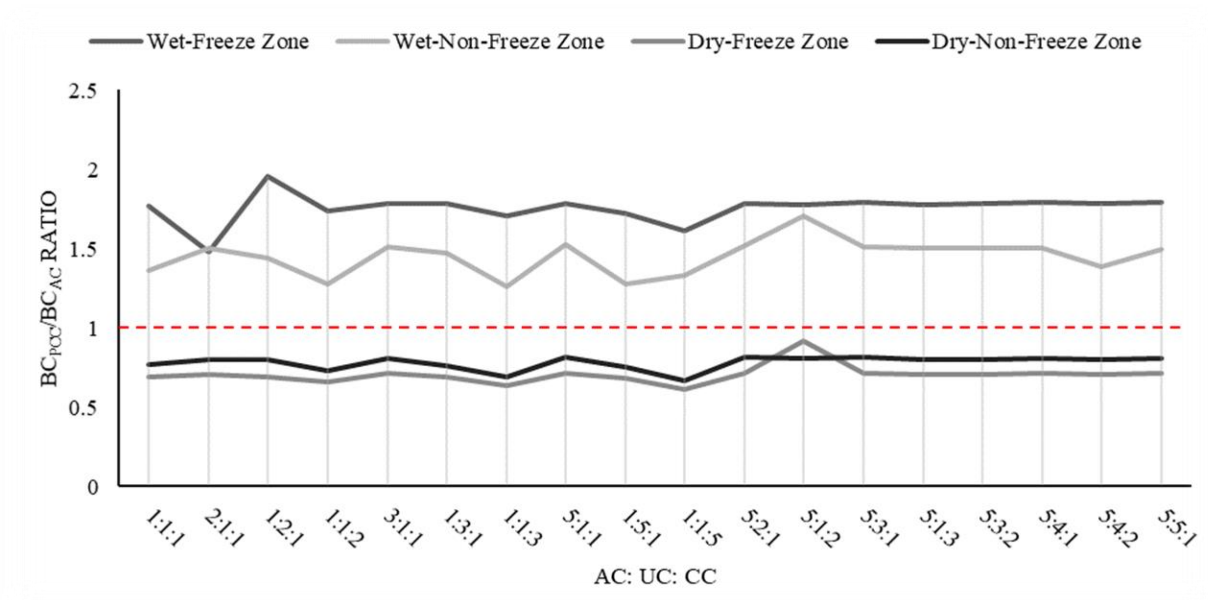


Figure 5.32: Effect of different weight combinations on the proportion of the relative attractiveness of PCC to the relative attractiveness of AC across the climatic zones

Table 5.2: The proportion of the B/C ratio of PCC to the B/C ratio of AC with different weight combinations of the stakeholder costs

Ratio	Weights			BC_{PCC} / BC_{AC} Ratio			
	AC	UC	CC	Wet-Freeze Zone	Wet-Non-Freeze Zone	Dry-Freeze Zone	Dry-Non-Freeze Zone
1:1:1	0.33	0.33	0.33	1.77	1.36	0.69	0.77
2:1:1	0.50	0.25	0.25	1.48	1.50	0.70	0.79
1:2:1	0.25	0.50	0.25	1.95	1.44	0.68	0.80
1:1:2	0.25	0.25	0.50	1.74	1.27	0.66	0.73
3:1:1	0.60	0.20	0.20	1.78	1.51	0.71	0.81
1:3:1	0.20	0.60	0.20	1.78	1.47	0.68	0.76
1:1:3	0.20	0.20	0.60	1.71	1.26	0.64	0.69
5:1:1	0.71	0.14	0.14	1.79	1.52	0.71	0.81
1:5:1	0.14	0.71	0.14	1.72	1.27	0.68	0.75
1:1:5	0.14	0.14	0.71	1.61	1.33	0.61	0.66
5:2:1	0.63	0.25	0.13	1.79	1.52	0.71	0.81
5:1:2	0.63	0.13	0.25	1.78	1.71	0.92	0.80
5:3:1	0.56	0.33	0.11	1.79	1.51	0.71	0.81
5:1:3	0.56	0.11	0.33	1.77	1.50	0.70	0.79
5:3:2	0.50	0.30	0.20	1.78	1.50	0.71	0.80
5:4:1	0.50	0.40	0.10	1.79	1.50	0.71	0.81
5:4:2	0.45	0.36	0.18	1.78	1.39	0.71	0.80
5:5:1	0.45	0.45	0.09	1.79	1.50	0.71	0.81

The optimal material choice was not affected by the change in the weights assigned to the agency, user, and community costs as shown in Figure 5.32. The B/C ratios of the optimal M&R schedules were changing as the different weights assigned were altered. However, this change did not affect the type of treatment selected nor the time to apply them with the given 18 weight assignment scenarios. The change in type of treatment and when to apply a treatment was basically noticed when the traffic volume was changed. For example, the optimal M&R schedule for the equal weight scenario (AC to UC to CC is 1:1:1) and for the scenario where the agency cost is five times the user and the community cost (AC to UC to CC is 5:1:1) was found to be the same for the flexible pavement in the wet freeze zone (Figure 5.1). The only change noticed was in the B/C ratio, which was caused by the change in the cost magnitude (by changing weights for the cost components), which means that the change in the weight assigned had a small effect on the time and type of treatment applied.

5.8. Chapter Summary

In this chapter, the optimal schedules for M&R treatments for asphalt and concrete pavements were presented across the four LTPP climatic regions. The optimal profiles were evaluated using a deterministic optimization approach and a genetic algorithm search method. The aim of the objective function was to maximize the B/C ratio of the overall M&R treatments applied over the pavement life. The benefits were evaluated using the area bounded by the performance curve and by calculating the pre-determined performance threshold. The treatment-specific performance curves presented in Chapter 4 were used to find the benefit area. The components of the agency, user, and community costs were estimated based on the models and the evaluated values presented in Chapter 4. An optimal profile for each pavement material (AC and PCC) was developed at every one of the four climatic zones based on the assumed (average) inputs. The results of this deterministic approach indicated that rigid pavement was the most cost-effective pavement material in wet freeze and wet non-freeze zones, while flexible pavement was more suitable for dry freeze and dry non-freeze zones. The probabilistic approach results show that flexible pavement was the most cost-effective pavement across all the LTPP climatic zones. The pavement material selection for each climatic zone was not affected by using different weight combinations of agency, user, and community costs.

CHAPTER 6. SUMMARY, CONCLUSIONS, AND FUTURE WORK

6.1. Introduction

This dissertation addressed the broad question of how to determine whether asphalt or concrete is the more cost-effective pavement type. Deciding which pavement type is superior requires analyzing various factors unique to each situation. In order to decide which pavement material is more cost-effective, a systematic methodology of evaluation is needed, and the economic comparison between flexible and rigid pavements needs to be more efficient than it has been in the past. This dissertation proposed a framework for selecting the most cost-effective pavement material under a given set of conditions. Developing M&R treatment optimal schedules allows an accurate comparison to be made so that the cost-effectiveness can be estimated for each pavement type. Optimal profiles of M&R treatments were developed in this dissertation to maximize the B/C ratio of the overall treatments applied on a pavement section for the pavement's life, including newly-constructed pavement. A brief summary of the proposed methodology and the results of this dissertation are presented in the subsequent sections.

6.2. Research Summary

The primary purpose of this dissertation was to develop a methodology to help identify the conditions under which each pavement material type is superior to the other as far as its overall life cycle cost-effectiveness. The proposed framework begins by building the optimal life cycle M&R activity profile for each material type in different climatic zones. Optimal M&R treatment schedules for flexible and rigid pavements then were developed in the four LTPP climatic regions: wet freeze zone, wet non-freeze zone, dry freeze-zone, and dry non-freeze zone. The purpose of these optimal profiles was to understand the effects of the environmental conditions on pavement behavior and, eventually, to study the effects of the environmental conditions on the selection of the more cost-effective pavement material. The two main element inputs of the optimization process were the benefits and the costs; and the objective function was constructed to maximize the B/C ratio of the life cycle M&R activity profile. The benefits were evaluated using the area bounded by the performance curve and the pre-determined performance indicator threshold.

Treatment-specific performance models for different LTPP climatic regions were developed to predict pavement behavior over the treatment service life.

The first and, arguably, most critical cost category, is the agency cost, which includes the initial construction, maintenance, rehabilitation, and salvage costs. The user cost considered in this dissertation was the work zone costs (travel time delay and VOC). The community cost is comprised of the air and noise pollution costs. The air pollution cost was evaluated based on the global warming potential of greenhouse gas emissions and the amount of energy consumed throughout the life cycle assessment phase; and the noise cost was the same as the life cycle cost of constructing a noise barrier (if needed).

6.2.1. Pavement Material Types and Preservation Treatments

In the proposed methodology, the first step is to identify the pavement material type. In this dissertation, the pavements were classified based on the wearing surface material type (AC or PCC). The decision-maker may begin the analysis with either flexible or rigid pavements. The next step is to identify all the candidate M&R treatments for the material type in question. Several M&R treatments can be considered for both asphalt and concrete pavements. There are two categories of maintenance treatments: preventive and corrective. The preventive maintenance treatments protect the pavement and delay pavement deterioration while the corrective maintenance treatments fix specific pavement distresses.

Flexible pavement maintenance treatments include crack sealing, fog sealing, patching, chip sealing, slurry sealing, micro-surfacing, and thin overlay. Some of the standard maintenance treatments for rigid pavement include joint and crack sealing, diamond grinding, grooving, partial depth patching, full depth patching, and slab replacement. Rehabilitation treatments are recommended when the pavement condition reaches the point at which maintenance treatments no longer are effective. Flexible rehabilitation treatments are classified based on pavement condition enhancement (functional or structural). Rigid pavement rehabilitation treatments include HMA functional overlay, HMA structural overlay, PCC crack and seat with HMA overlay, PCC overlay, and PCC rubblization and HMA overlay.

Acquiring a comprehensive dataset containing pavement condition, traffic loading, and other environmental factors for the same treatment applied in different climatic regions was not an easy task. Fortunately, the pavement performance dataset obtained from LTPP program, one of the primary studies of the Strategic Highway Research Program (SHRP), was publicly available. The data were collected from four major climatic zones in North America: wet freeze, wet non-freeze, dry freeze, and dry non-freeze. The required data to build treatment-specific performance models were found in specific SPS studies.

For flexible pavement, the preventive maintenance treatments studied in the SPS 3 experiment included crack sealing, chip seals, slurry seal, and thin HMA overlay. The LTPP rehabilitation treatments for flexible pavement (SPS 5) were implemented with different structural features of rehabilitation, such as the type of surface preparation before overlay (minimal or intensive), the type of material used (virgin asphalt mixture or recycled), and the thickness of the overlay (two to five inches).

The LTPP preventive maintenance treatments applied to rigid pavement (SPS 4) included joint/crack sealing, surface grinding and grooving, partial-depth patching at joints/cracks, and full-depth patching at joints/cracks. All of the rehabilitation treatments used in the LTPP program for rigid pavement (SPS 6) were asphalt concrete overlays over jointed plain concrete pavement (JPCP) or jointed reinforced concrete pavement (JRCP). Different preparation methods were applied prior to the overlay implementation (minimal and intensive preparations and crack or break and seat). The overlay thickness for all (SPS 6) treatments was four inches of AC, except for SPS 608, which used 8 inches of AC overlay.

6.2.2. Effectiveness and Performance Models

Objective measures are needed to evaluate the effectiveness and benefit level of particular preservation treatments. The effectiveness of individual maintenance or rehabilitation treatments can be assessed in the short term or long term. The long-term MOEs include pavement condition improvement, pavement life extension, and reduction of routine maintenance in the years following the treatment applied. The long-term effectiveness measures are crucial during the planning and programming stage of the highway asset. For example, knowing the treatment service life helps the highway agency plan in advance the implementation of the next preservation treatment. The focus of this dissertation was on the long-term MOEs, although some models of pavement performance jump trend models (short-term MOEs) also were evaluated here.

The long-term MOE can be divided into monetized and non-monetized MOEs. The non-monetized MOEs used in this dissertation were the treatment service life and the area bounded by the performance curve. The treatment service life is the time required for the pavement condition to return to a predetermined condition threshold. These condition thresholds can be determined with performance indicators assigned by the highway agencies. Two strategies were used to evaluate treatment life: (1) an age-based strategy, which is the time between two treatments; and (2) a condition-based strategy, which is the time required for the pavement to return to a pre-existing condition after the preservation treatment is applied. The area bounded by the performance curve and the pre-determined threshold combined the effectiveness concepts of (1) treatment life, and (2) increase in average condition after implementing a preservation treatment. This method is arguably the best one for analyzing the M&R treatments and therefore was used in this dissertation.

The monetized benefits can be estimated by evaluating the reduction in the agency (including construction, maintenance, and rehabilitation) and user (travel time delay and VOC) costs due to pavement condition improvement. The highways agency's savings (benefits) can be evaluated by estimating the reduction in the routine maintenance due to a preservation treatment. The routine maintenance expenditure was estimated as a function of the pavement condition. The user benefit is the difference in the VOC during normal operation with or without implementing a preservation treatment. The cost for the community associated with air and noise pollution are related to the pavement condition. Specifically, this dissertation relates increases in air and noise pollution with deterioration of pavement condition. The savings therefore can be evaluated by considering the difference in the costs associated with air and noise pollution before and after applying preservation treatments.

The performance indicator used in this dissertation is pavement roughness measured in IRI (in/mile). The PI is used to quantify the asset condition, and IRI is a standard measure of pavement condition. The treatment-specific performance models were developed to predict the future behavior of M&R treatments across LTPP climatic zones. Also, performance jump models were developed for each preventive M&R treatment. In addition, the LTPP database was used to build treatment performance models for flexible and rigid pavements. The M&R treatments for flexible pavement were developed according to SPS 3 and SPS 5, respectively. Rigid pavement performance models also were developed for maintenance (SPS 4) and rehabilitation (SPS 6). A panel model with fixed effect was the modeling technique used for the treatment-specific performance models. When more than one pavement section is considered for each treatment, a heterogeneity issue arises that can be corrected by estimating the fixed-effect. The response variable of all the models is the pavement condition (represented in IRI values). Several explanatory variables considered for developing the models included accumulated truck traffic loading; pavement age; and environmental variables including average freeze index and average annual temperature. The performance jump (trend) models and the treatment-specific performance models for each treatment across the LTPP climatic zones were presented in Chapter 4.

6.2.3. Cost Components

As mentioned earlier, this study considered the following three cost classes that differ across pavement material type: agency cost, user cost, and community cost. The agency cost is comprised of several cost categories, but the focus of this dissertation was on the initial M&R costs of the highway. These costs usually are incurred by public agencies providing transportation services to their communities. The M&R costs were included to further explore these costs throughout the life cycle of the project. Both average and statistical cost models were adopted from the literature and subjected to inflation and special cost adjustment factors. For the purposes of this dissertation, user cost was defined as the cost incurred by roadway users at work zones. Travel time delay and VOC in the work zone were the two components considered for user cost. Duration models of individual treatments were essential for estimating work-zone user costs. The two components of community cost considered in this dissertation were air and noise pollution costs. The noise cost was evaluated based on the need to construct a noise barrier; therefore, the cost of constructing and maintaining the noise barrier is considered as the noise pollution cost. The FHWA noise abatement criteria include the construction of a noise barrier. The air pollution cost was estimated by monetizing the social effect of the GWP of greenhouse gas emissions manifested in carbon dioxide equivalent emission as well as the energy consumed during the combined LCA phases. LCA includes environmental characteristics and potential impacts throughout the pavement’s service life, from material acquisition through production, construction, transportation, M&R, and eventually product disposal. The four basic phases of LCA considered in this study were the goal and the scope definition, the life cycle inventory, the life cycle impact assessment, and the life cycle interpretation. The results for all the cost components were presented in Chapter 4.

6.2.4. Developing the Optimal Profile of Treatment Activities

The process of selecting pavement materials must include assessment of all the costs and benefits for the alternatives (flexible or rigid). Several methods are available to calculate the cost-effectiveness at a project level. In this dissertation, NPV and EUAC were the MOE methods implemented because the benefits are expressed in a monetized form. The incremental B/C ratio is the evaluation criteria used when cost and effectiveness are expressed in different units (non-monetized benefits).

The numerical optimization methodology was adopted to ascertain the best type of treatment(s) to be applied and the exact timing of such treatment(s) for the pavement's service life. The primary reason for developing an optimal profile of M&R activities is to maximize the return on investment and use resources as efficiently as possible. The optimal profile of activities was evaluated at the project-level, where all the costs and benefits were accounted for. The pavement performance indicators, represented by the specified IRI (in/mile) at pre-determined thresholds, were the decision variables for this optimization problem. The PI threshold is indicated by continuous variables while the type of treatment is shown in integer variables. Different trigger values were assigned to initiate maintenance or rehabilitation activities: 100-130 in/mile and 100 to 160 in/mile for M&R, respectively. The objective function was established to maximize the incremental B/C ratio of the entire profile of activities throughout the pavement service life. The benefits were evaluated using the area bounded by the pavement performance curve and the pre-determined PI thresholds, while the cost components were estimated by aggregating the agency, user, and community costs of each treatment applied throughout the pavement life. Several constraints were enforced: (1) minimum and maximum PI must not be violated; (2) no treatment is to be applied when the pavement is newly constructed; and (3) only one treatment is to be implemented at each stage. The treatment-specific performance models developed in Chapter 4 were used to evaluate the incremental benefits of each treatment.

The objective function was considered as a piecewise function, which is a function comprised of a number of sub-functions and has two values at any point in time. It was shown that, when treatments were applied, there was "jump discontinuity" in the objective function at the performance jump points, which means that while the primary function was not differentiable, the sub-functions were found differentiable in their domains. In this situation, the objective function has a global solution. One reason for selecting GA optimization was its ability to handle the discontinuity issue related to the objective function. Also, the design variables of GA can be discrete, continuous, or even non-differentiable. The global optimal is not guaranteed when using GA; however, the near-global optimal solution is expected. The optimal profiles for flexible and rigid pavements across LTPP climatic regions were evaluated and presented in Chapter 5.

6.2.5. Deterministic and Probabilistic Approaches

The two suggested approaches for comparing AC and PCC are the deterministic and the probabilistic approaches, which were applied based on the optimal schedule of activities for flexible and rigid pavements. The deterministic approach was evaluated using the EUAC of the optimal profile of each pavement type based on pre-determined inputs. Some of these inputs, including the agency, user, and community costs, were presented in Chapter 4. The probabilistic method was introduced due to the uncertainty of the input variables. Using the deterministic approach with sensitivity analysis could not take care of the uncertainty issue. For that reason, stochastic (risk) analysis was used to address the uncertainty of the inputs; this uncertainty was overcome by using the probabilistic entry of the variable and studying the uncertainty with a computer simulation to assess the risk related to LCCA outcomes. The results were presented in a probability distribution that described a range of outputs instead of single values. When following this method, a decision-maker is able to anticipate each outcome with its associated probability.

6.3. Research Conclusion

The most cost-effective pavement material was determined with the optimal schedule of M&R treatments for a given highway project. Changes in climatic conditions play a vital role in determining the best pavement material to be selected, especially its economic attractiveness. Based on the deterministic approach, rigid pavement was found to be more cost-effective in wet climates (wet-freeze and wet-non-freeze zones) while flexible pavement was found to be more suitable for dry climates (dry-freeze and dry-non-freeze zones). This means that the primary factor affecting pavement material selection is the presence of moisture. Other environmental factors, such as freeze index and average temperature, have limited impacts on pavement material selection. Although the freeze index was found to be significant in freezing climates, it had only a small impact on pavement material selection. Truck traffic loading is an essential factor that affects pavement type selection in different climatic conditions. In wet climates, the superiority of the rigid pavement over the flexible pavement increased as the truck traffic volume increased. Furthermore, the economic attractiveness of flexible pavement in dry climates declined as the truck traffic volume increased.

Flexible pavement was the dominant pavement material across the four LTPP climatic zones when the stochastic approach was implemented. Even though the EUAC does not explicitly consider the benefits, the effectiveness (benefits) of each M&R treatment were considered implicitly when the optimal activity profiles were developed using the area over the curve measure. The results of this approach show that rigid pavement can cost 34 to 50% more than flexible pavement. The uncertainty of the treatment service life, which is one of the input variables, was not considered in this approach because of the deterministic nature of the statistical prediction models of treatment service life. If the uncertainty related to such variables needs to be considered, then a stochastic optimization method is one way to do so. There is a need to develop a probabilistic performance model for capturing the uncertainty of the treatment's expected life; however, this model is out of the scope of the present dissertation and may be considered in future work.

6.4. Future Work

The framework proposed in this dissertation can be scaled down to a state level or scaled up to the national or continental level, as long as the datasets of costs and pavement and climatic conditions are available. To estimate user cost accurately, treatment-specific duration models should be developed out of the same datasets used for the pavement performance models. For the purposes of this research, treatment-specific performance models using panel models with fixed effect were used. Ideally, these performance models would be expanded by using random effect models, resulting in pavement performance models that could be used to study any given climatic region. Another way to enhance this framework is to develop probabilistic treatment-specific performance models to incorporate the uncertainty of treatment life. Developing these performance models utilizing the LTPP database would be challenging because a dataset is needed that represents different climatic conditions. The stochastic optimization method accounts for the uncertainty of treatment life and other optimization inputs. This approach could be used in future work as an alternative to the probabilistic approach presented in this dissertation and to incorporate the uncertainty of input variables. The suggested probabilistic models could be used to solve the stochastic optimization problem involved in scheduling treatments.

REFERENCES

AASHTO. (1993). *Guide for design of pavement structures*. (A. A. of S. H. and TransportationOfficials, Ed.). Washington, D.C.: America Association of State Highway and TransportationOfficials.

AASHTO. (2003). *User and non-user benefit analysis for highways*. American Association of State Highway and Transportation Officials. Retrieved from https://www1.browntechnical.org/products/user-and-non-user-benefit-analysis-for-highways-3rd-edition?variant=1052751003667&gclid=EAIaIQobChMI6pe7-LqQ2gIVhlmGCh2xsg4oEAYYASABEgKrPvD_BwE

AASHTO. (2011). *A policy on geometric design of highways and streets, 2011.* American Association of State Highway and Transportation Officials.

Abaza, K. A. (2002). Optimum Flexible Pavement Life-Cycle Analysis Model. *Journal of Transportation Engineering*, *128*(6), 542–549. https://doi.org/10.1061/(ASCE)0733-947X(2002)128:6(542)

Abdelaziz, N., Abd El-Hakim, R. T., El-Badawy, S. M., & Afify, H. A. (2018). International Roughness Index prediction model for flexible pavements. *International Journal of Pavement Engineering*, 1–12. https://doi.org/10.1080/10298436.2018.1441414

ACPA. (1998). *Rubblizing of Concrete Pavements :A Discussion of its Use*. American Concrete Pavement Association.

ACPA. (2002). *Life Cycle Cost Analysis: A Guide for Comparing Alternate Pavement Analysis*. Washington D.C.

ACPA. (2018). Differences Between Concrete and Asphalt Pavement. Retrieved May 17, 2018, from http://overlays.acpa.org/Concrete_Pavement/Technical/Fundamentals/Differences_Between_Concrete_and_Asphalt.asp

Adow, M. A.-A. O., Allotey, S. E., & Sasraku-Neequaye, B. K. (2011). Comparative Cost Analysis between Asphalt Pavement and Concrete Pavement in Road Construction: A Case study using Concrete grade 35. *Civil and Environmental Research*, *7*(10), 94–104. Retrieved from http://www.iiste.org/Journals/index.php/CER/article/view/26238

Ahmed, A. (2012). Pavement damage cost estimation using highway agency maintenance, rehabilitation, and reconstruction strategies. Retrieved from http://search.proquest.com/openview/f0f4330dafec8bedcb85fa74c9c64b2d/1?pq-origsite=gscholar&cbl=18750&diss=y

Ahmed, A., Labi, S., Li, Z., & Shields, T. (2013). Aggregate and disaggregate statistical evaluation of the performance-based effectiveness of long-term pavement performance specific pavement study-5 (LTPP SPS-5) flexible pavement rehabilitation treatments. *Structure and Infrastructure Engineering*, *9*(2), 172–187. https://doi.org/10.1080/15732479.2010.524224

Akakin, T., Engin, Y., & Ucar, S. (1983). Initial cost comparison of rigid and flexible pavements: under different traffic and soil conditions. Retrieved from http://ich.cl/descargas/wp-content/uploads/12-07-16_PAV_GPH_19-INITIAL-COST-COMPARISON-OF-RIGID.pdf

Al-Mansour, A. I., & Sinha, K. C. (1994). Economic analysis of effectiveness of pavement preventive maintenance. *Transportation Research Record*, (1442), 31–37. Retrieved from http://www.scopus.com/inward/record.url?eid=2-s2.0-0028532294&partnerID=40&md5=7bd915bd143e38d5770fd9bae3d6ebfe

Alinizzi, M. (2017). *FRAMEWORK FOR OPTIMIZING THE MAINTENANCE AND REHABILITATION SCHEDULES OF INTERDEPENDENT INFRASTRUCTURE SYSTEMS*. Purdue University.

Alqadhi, S., Ghahari, S. A., Woldemariam, W., Volovski, M., & Chen, S. (2018). Costs and Benefits of Highway Pavement Resurfacing: Interstate 465 Case Study. *Infrastructure Asset Management*, 1–40. https://doi.org/10.1680/jinam.17.00036

Alqadhi, S., Hoyos, J. M., Qiao, Y., & Labi, and S. (2016). Measurement of Pavement Rehabilitation Effectiveness: Constructs and Comparison of Current Findings to Existing Literature. In *Eighth International Conference on Maintenance and Rehabilitation of Pavements* (p. 214). Singapore: RESEARCH PUBLISHING Blk 12 Lorong Bakar Batu, #02-11, Singapore 348745. https://doi.org/10.3850/978-981-11-0449-7-214-cd

Ambroz, J. K., & Darter, M. I. (2005). Rehabilitation of Jointed Portland Cement Concrete Pavements : SPS-6 — Initial Evaluation and Analysis, (October).

APA. (2004). *Pavement Type Selection Processes*. Lexington, Kentucky.

Arhin, S., & Williams, L. (2015). Predicting pavement condition index using international roughness index in a dense urban area. *Article.Sapub.Org*. Retrieved from http://article.sapub.org/10.5923.j.jce.20150501.02.html

Arora, J. (2012). *Genetic algorithms for optimum design*. *Academic Press Waltham, UK*. Retrieved from https://scholar.google.com/scholar?hl=en&as_sdt=0%2C15&q=Arora%2C+Jasbir.+Introduction+to+optimum+design.+Academic+Press%2C+2012.&btnG=

Athena. (2006). *A Life Cycle Perspective on Concrete and Asphalt Roadways: Embodied Primary Energy and Global Warming Potential*. Ottawa, ON, Canada.

Athena. (2014). The Athena impact estimator for highways. Ottawa, Canada: Athena Sustainable Materials Institute.

Aurangzeb, Q., Al-Qadi, I. L., Ozer, H., & Yang, R. (2014). Hybrid life cycle assessment for asphalt mixtures with high RAP content. *Resources, Conservation and Recycling*, *83*, 77–86. https://doi.org/10.1016/J.RESCONREC.2013.12.004

Bahrami, A. (2010). Procedures for Abatement of Highway Traffic Noise and Construction Noise / Rules and Regulations. *Federal Register*, *75*(133). Retrieved from http://www.archives.gov/

Barnes, G., & Langworthy, P. (2003). The Per-mile Costs of Operating Automobiles and Trucks. Retrieved from https://conservancy.umn.edu/handle/11299/909

Bautista, F. E., & Basheer, I. (2008). Jointed Plain Concrete Pavement (JPCP) Preservation and Rehabilitation Design Guide.

BCE. (2017). Comparison between Flexible Pavement and rigid pavement in highway construction - Basic Civil Engineering. Retrieved May 17, 2018, from http://www.basiccivilengineering.com/2015/04/comparison-between-flexible-pavement.html

Bendtsen, H., Lu, Q., & Kohler, E. (2010). Acoustic Aging of Asphalt Pavements: A Californian/Danish Comparison. Retrieved from https://www.researchgate.net/profile/Qing_Lu18/publication/270570683_Acoustic_aging_of_asphalt_pavement_A_Californian_Danish_comparison/links/54ad923a0cf2213c5fe412c6.pdf

Bennett, C., & Greenwood, I. (2001). Modelling road user and environmental effects in HDM-4. *Researchgate.Net*.

Berthiaume, R., & Bouchard, C. (1999). EXERGY ANALYSIS OF THE ENVIRONMENTAL IMPACT OF PAVING MATERIAL MANUFACTURE. *Transactions of the Canadian Society for Mechanical Engineering*, *23*(1B), 187–196. https://doi.org/10.1139/tcsme-1999-0013

Bickel, P., Friedrich, R., Link, H., Stewart, L., & Nash, C. (2006). Introducing environmental externalities into transport pricing: Measurement and implications. *Transport Reviews*, *26*(4), 389–415. https://doi.org/10.1080/01441640600602039

Brown, E. R. (2009). *Hot mix asphalt : materials, mixture design and construction*. NAPA Research and Education Foundation.

BSI. (2006). 14040: Environmental management–life cycle assessment–principles and framework. London.

CADOT. (2013). *LIFE-CYCLE COST ANALYSIS PROCEDURES MANUAL State of California Department of Transportation Division of Maintenance Pavement Program LCCA Procedures Manual*. Retrieved from http://www.dot.ca.gov/hq/maint/Pavement/Offices/Pavement_Engineering/LCCA_Docs/LCCA_25CA_Manual_Final_Aug_1_2013_v2.pdf

CDOT. (2015). Concrete Pavement Guide Part 4: Rehabilitation Strategies. Retrieved from http://www.dot.ca.gov/hq/maint/Pavement/Offices/Pavement_Engineering/CPG/CPG_Ch410_Crack-Seat-Overlay.pdf

Chan, A., Keoleian, G., & Gabler, E. (2008). Evaluation of Life-Cycle Cost Analysis Practices Used by the Michigan Department of Transportation. *Journal of Transportation Engineering*, *134*(6), 236–245. https://doi.org/10.1061/(ASCE)0733-947X(2008)134:6(236)

Chan, A. W.-C. (2007, April 30). *Economic and Environmental Evaluations of Life-Cycle Cost Analysis Practice: A Case Study of Michigan DOT Pavement Projects*. Retrieved from https://deepblue.lib.umich.edu/handle/2027.42/50469

Chatti, K., & Zaabar, I. (2012). *Estimating the Effects of Pavement Condition on Vehicle Operating Costs*. National Academies Press. https://doi.org/10.17226/22808

Daleiden, J., Simpson, A., & Rauhut, J. (1998). *Rehabilitation performance trends: Early observations from long-term pavement performance (LTPP) specific pavement studies (SPS)*. Retrieved from https://trid.trb.org/view/473214

Dalkey, N., & Helmer, O. (1963). An Experimental Application of the DELPHI Method to the Use of Experts. *Management Science*, *9*(3), 458–467. https://doi.org/10.1287/mnsc.9.3.458

Darter, M. I., Smith, R. E., & Shahin, M. Y. (1985). Use of life-cycle costing analysis as a basis for determining the cost-effectiveness of maintenance and rehabilitation treatments for developing a network level assignment procedure. *In Proceedings, North American Pavement Management Conference, Toronto*.

Davies, H., & Kamp, I. (2012). Noise and cardiovascular disease: A review of the literature 2008-2011. *Noise and Health*, *14*(61), 287. https://doi.org/10.4103/1463-1741.104895

Embacher, R. A., & Snyder, M. B. (2001). Life-cycle cost comparison of asphalt and concrete pavements on low-volume roads case study comparisons. *Transportation Research Record*, (1749), 28–37. Retrieved from http://www.scopus.com/inward/record.url?eid=2-s2.0-0035201823&partnerID=tZOtx3y1

Feighan, K. J., Shahin, M. Y., & Sinha, K. C. (1987). A dynamic programming approach to optimization for pavement management systems. *2nd North America Pavement Management Conference*.

FHWA. (2005). Technical Advisory T 5040.36 Surface Texture for Asphalt and Concrete Pavements - Pavements - Federal Highway Administration. Retrieved May 21, 2018, from https://www.fhwa.dot.gov/pavement/t504036.cfm#background

FHWA. (2011). Highway Traffic Noise: Analysis and Abatement Guidance, (June 2010), 75.

FHWA. (2016). Guidance on Highway Preservation And Maintenance. Retrieved from https://www.fhwa.dot.gov/preservation/memos/160225.cfm

Flintsch, G. W., Diefenderfer, B. K., & Nunez, O. (2013). Composite Pavement Systems: Synthesis of Design and Construction Practices.

Ford, K., Arman, M., Labi, S., Sinha, K., & Shirole, A. (2011). *Methodology for estimating life expectancies of highway assets*. Washington, DC. Retrieved from https://engineering.purdue.edu/LEHA/NCHRP 08-71/Work Reports/08-71 Draft Final Report June 10 2011.pdf

Friesz, T. L., & Enrique Fernandez, J. (1979). A model of optimal transport maintenance with demand responsiveness. *Transportation Research Part B: Methodological*, *13*(4), 317–339. https://doi.org/10.1016/0191-2615(79)90025-0

Fwa, T. F., Chan, W. T., & Tan, C. Y. (1996). Genetic-Algorithm Programming of Road Maintenance and Rehabilitation. *Journal of Transportation Engineering, 122*(3), 246–253. https://doi.org/10.1061/(ASCE)0733-947X(1996)122:3(246)

Gao, L., & Zhang, Z. (2009). Approximate Dynamic Programming Approach to Network-Level Budget Planning and Allocation for Pavement Infrastructure. In *Transportation Research Board 88th Annual Meeting*. Washington DC, United States. Retrieved from https://trid.trb.org/view/881691

Gates, T. J., Hawkins, H. G., & Rose, E. R. (2003). *EFFECTIVE PAVEMENT MARKING MATERIALS AND APPLICATIONS FOR PORTLAND CEMENT CONCRETE ROADWAYS*. Austin, Texas . Retrieved from https://static.tti.tamu.edu/tti.tamu.edu/documents/4150-2.pdf

Geoffroy, D. N. (1996). *Cost-effective preventive pavement maintenance*. *NCHRP Synthesis of Highway Practice*. National Academy Press. Retrieved from https://trid.trb.org/view/461832

Ghahari, S., Volovski, M., Alqadhi, S., & Alinizzi, M. (2018). Estimation of Annual Repair Expenditure for Interstate Highway Bridges. *Infrastructure Asset Management*, 1–33. https://doi.org/10.1680/jinam.17.00021

Gopalakrishnan, K., Steyn, W. J., & Harvey, J. (2014). *Climate change, energy, sustainability and pavements*.

Gopisetti, L. (2017). *International Roughness Index Prediction of Flexible and Rigid Pavements Using Climate and Traffic Data*. Bradley University.

Greene, W. H. (2000). *Econometric analysis (International edition).*

Gschösser, F., & Wallbaum, H. (2013). Life cycle assessment of representative swiss road pavements for national roads with an accompanying life cycle cost analysis. *Environmental Science and Technology*, *47*(15), 8453–8461. https://doi.org/10.1021/es400309e

Gulen, S., Woods, R., & … J. W. (1994). Correlation of present serviceability ratings with international roughness index. *Onlinepubs.Trb.Org*. Retrieved from http://onlinepubs.trb.org/Onlinepubs/trr/1994/1435/1435.pdf#page=35

Gulen, S., & Zhu, K. (2001). DEVELOPMENT OF IMPROVED PAVEMENT PERFORMANCE PREDICTION MODELS FOR THE INDIANA PAVEMENT MANAGEMENT SYSTEM. Retrieved from https://docs.lib.purdue.edu/cgi/viewcontent.cgi?referer=https://scholar.google.com/&httpsredir=1&article=1540&context=jtrp

Haas, R. C. G. (Ralph C. G. ., Hudson, W. R., & Zaniewski, J. P. (1994). *Modern pavement management*. Krieger Pub. Co. Retrieved from https://trid.trb.org/view/388787

Hakkinen, T., & Makela, K. (1996). *Environmental adaption of concrete: Environmental impact of concrete and asphalt pavements*. Finland.

Hall, K., Darter, M., & Armaghani, J. (1993). *Performance monitoring of joint load transfer restoration*. Retrieved from https://trid.trb.org/view/382542

Hall, K. T., Correa, C. E., Carpenter, S. H., & Elliott, R. P. (2001). Rehabilitation strategies for highway pavements. *Final Report, NCHRP Project*, *35*(May), 1–38.

Hall, K. T., Simpson, A. L., & Correa, C. E. (2002). LTPP Data Analysis: Effectiveness of Maintenance and Rehabilitation Options. *NCHRP Web Document*, *47*(June), 191.

Hanna, S. R., Briggs, G. A., & Hosker, R. P. (1982). Handbook on ATMOSPHERIC DIFFUSION Air-Pollution Meteorology in Complex Terrain. *Handbook on Atmospheric Diffusion*, 1–7.

Hausman, J. A., & Taylor, W. E. (1981). Panel Data and Unobservable Individual Effects. *Econometrica*, *49*(6), 1377. https://doi.org/10.2307/1911406

Hicks, R., Moulthrop, J., & Daleiden, J. (1999). Selecting a Preventive Maintenance Treatment for Flexible Pavements. *Transportation Research Record: Journal of the Transportation Research Board*, *1680*, 1–12. https://doi.org/10.3141/1680-01

HIG. (2018). Cool Pavements. Retrieved from https://heatisland.lbl.gov/coolscience/cool-pavements

Hoang, T., Jullien, A., Ventura, A., & … Y. C. (2005). A global methodology for sustainable road-Application to the environmental assessment of French highway. In *10DBMC International Conference On Durability of Building Materials and Components* (pp. 17–20). Lyon (France),. Retrieved from http://www.irbnet.de/daten/iconda/06089002852.pdf

Horvath, A., & Hendrickson, C. (1998). Comparison of Environmental Implications of Asphalt and Steel-Reinforced Concrete Pavements. *Transportation Research Record*, *1626*(98), 105–113. https://doi.org/10.3141/1626-13

Hsiao, C. (1986). *Analysis of Panel Data, Econometric Society Monograph No. 11*. Cambridge, UK: Cambridge University Press. Retrieved from https://scholar.google.com/scholar?hl=en&as_sdt=0%2C15&q=analysis+of+panel+data+hsiao+1986&btnG=&oq=analysis+of+panel+data+hsiao+198

INDOT. (2011). Indiana Department of Transportation Traffic Noise Analysis Procedure. Retrieved from https://www.in.gov/indot/files/INDOT_Noise_Policy_June_2011.pdf

INDOT. (2013). *Pavement and Underdrain Design Elements* (Vol. 03).

Irfan, M. (2010). A framework for developing optimal pavement life-cycle activity profiles. *Theses and Dissertations Available from ProQuest*. Retrieved from https://docs.lib.purdue.edu/dissertations/AAI3418399

Irfan, M., Khurshid, M. B., Bai, Q., Labi, S., & Morin, T. L. (2012). Establishing optimal project-level strategies for pavement maintenance and rehabilitation - A framework and case study. *Engineering Optimization*, *44*(5), 565–589. https://doi.org/10.1080/0305215X.2011.588226

Irfan, M., Khurshid, M., & Labi, S. (2009). Determining the Service Life of Thin Hot-Mix Asphalt Overlay by Means of Different Performance Indicators. *Transportation Research Record: Journal of the Transportation Research Board*, *2108*(Table 1), 37–45. https://doi.org/10.3141/2108-04

Irfan, M., Khurshid, M., Labi, S., & Flora, W. (2009). Evaluating the cost effectiveness of flexible rehabilitation treatments using different performance criteria. *Journal of Transportation …*, *135*(October), 753–763. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000041

ISO. (1997). *Environmental Management - Life Cycle Assessment - Principles And Framework, ISO 14040*.

Joseph, P. (1992). CRACK SEALING IN FLEXIBLE PAVEMENTS: A LIFE-CYCLE COST ANALYSIS. Retrieved from https://trid.trb.org/view/368081

Joumard, R., André, J., Rapone, M., & Zallinger, M. (2007). Emission factor modelling and database for light vehicles. *Researchgate.Net*. Retrieved from https://www.researchgate.net/profile/Michel_Andre/publication/265322236_Emission_factor_modelling_and_database_for_light_vehicles_-_Artemis_deliverable_3/links/5476fc580cf2778985b09f16/Emission-factor-modelling-and-database-for-light-vehicles-Artemis-deli

Kalembo, C., Jeihani, M., & Saka, A. (2012). Evaluation of the impact of pavement roughness on vehicle gas emissions in Baltimore County. *Docs.Trb.Org*.

Khurshid, M. (2010). *A FRAMEWORK FOR ESTABLISHING OPTIMAL PERFORMANCE THRESHOLDS FOR HIGHWAY ASSET INTERVENTIONS*. Retrieved from https://search.proquest.com/openview/3af65e40e90dc31a118a40c8e0e1d500/1?pq-origsite=gscholar&cbl=18750&diss=y

Khurshid, M. B., Irfan, M., Labi, S. A., & Sinha, K. C. (2008). Effectiveness of Asphaltic Concrete Overlays of Rigid Highway Pavements Using LTPP SPS-6 Data. Retrieved from https://trid.trb.org/view/848876

Kosmatka, S., Panarese, W., & Kerkhoff, B. (2002). *Design and control of concrete mixtures*. Skokie, Illinois: Portland Cement Association. Retrieved from http://cement.org/bookstore/supporting/cd100/eb001frt.pdf

Ksaibati, K., McNamara, R., Miley, W., Florida, P., & Armaghani, J. M. (1998). Evaluating the ride number as a pavement management roughness index, (August), 22.

Kuennen, T. (2005). Making high-volume roads last longer. *Better Roads 75, 4*. Retrieved from https://trid.trb.org/view/755746

Künzli, N., Kaiser, R., Medina, S., Studnicka, M., Chanel, O., Filliger, P., … Sommer, H. (2000). Public-health impact of outdoor and traffic-related air pollution: a European assessment. *The Lancet*, *356*(9232), 795–801. https://doi.org/10.1016/S0140-6736(00)02653-2

Labi, S. A., Lamptey, G., Kong, S.-H., & Nunoo, C. (2006). Long-Term Benefits of Microsurfacing Applications in Indiana - Methodology and Case Study. Retrieved from https://trid.trb.org/view/777431

Labi, S., Lamptey, G., Konduri, S., & Sinha, K. (2005). Part 1: Pavement Management: Analysis of Long-Term Effectiveness of Thin Hot-Mix Asphaltic Concrete Overlay Treatments. *Transportation Research Record: Journal of the Transportation Research Board*, *1940*(1), 1–12. https://doi.org/10.3141/1940-01

Labi, S., Reddy Venga, B., Kalafatas, G., & Ong, G. P. (2008). Relationships between Long-Term and Short-Term Effectiveness of Transportation Pavement Treatments. Athens, Greece: 10th International Conference on Applications of Advanced Technologies in Transportation (AATT).

Labi, S., & Sinha, K. C. (2003). The Effectiveness of Maintenance and its Impact on Capital Expenditures. Indiana, USA: Joint Transportation Research Program, Indiana. *Department of Transportation and Purdue University, West Lafayette, Indiana*, *7*(2). https://doi.org/10.5703/1288284313331.This

Labi, S., & Sinha, K. C. (2005). Life-Cycle Evaluation of Flexible Pavement Preventive Maintenance. *Journal of Transportation Engineering*, *131*(10), 744–751. https://doi.org/10.1061/(ASCE)0733-947X(2005)131:10(744)

Lamptey, G. (2004). *Optimal scheduling of pavement preventive maintenance using life cycle cost analysis*. *Purdue University West Lafayette, IN*.

Lamptey, G., Ahmad, M., Labi, S., & Sinha, K. (2005). *Life Cycle Cost Analysis for INDOT Pavement Design Procedures*. *JTRP Technical Reports*. https://doi.org/10.5703/1288284313261

Lavrenz, S., Hoyos, J., & Labi, S. (2014). Constructs for Quantifying the Long-term Effectiveness of Civil Infrastructure Interventions (pp. 379–406). https://doi.org/10.1201/b17073-14

Lemer, A. C. (1996). Infrastructure Obsolescence and Design Service Life. *Journal of Infrastructure Systems*, *2*(4), 153–161. https://doi.org/10.1061/(ASCE)1076-0342(1996)2:4(153)

Leng, Z., Al-Qadi, I. L., & Cao, R. (2017). Life-cycle economic and environmental assessment of warm stone mastic asphalt. *Transportmetrica A: Transport Science*, *9935*(November), 1–14. https://doi.org/10.1080/23249935.2017.1390707

Lenngren, C. (2014). DIFFERENT PAVEMENT TYPES AND ROLLING RESISTANCE. *International Concrete Sustainability Conference Boston*. https://doi.org/10.13140/2.1.3520.2889

Li, Z., & Sinha, K. C. (2004). Methodology for Multicriteria Decision Making in Highway Asset Management System Goals in Asset Management. *Transportation Research Record*, *1885*, 79–87.

Liu, X., Choy, D., Cui, Q., & Schwartz, C. (2017). Capitalizing green pavement a method and valuation. *The International Pavement Life-Cycle Assessment Symposium Convened*, 56. https://doi.org/doi:10.1201/9781315159324-26

Livneh, M. (1996). Deterioration Modeling for Unlaid and Overlaid Pavements. *Transportation Research Record: Journal of the Transportation Research Board*, *1524*, 177–184. https://doi.org/10.3141/1524-21

Loss, N. (2015). Dangerous decibels focuses on noise-induced hearing loss. Retrieved from https://scholar.google.com/scholar?hl=en&as_sdt=0%2C15&q=68.%09Noise+Induced+Hearing+Loss%3A+Dangerous+decibels+focus+on+noise-induced+hearing+loss.+%282015%2C+February+25%29.+Retrieved+from%3A+http%3A%2F%2Fwww.dangerousdecibels.org%2Feducation%2Finformat

Lu, P., & Tolliver, D. (2013). Multiobjective Pavement-Preservation Decision Making with Simulated Constraint Boundary Programming. *Journal of Transportation Engineering*, *139*(9), 880–888. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000573

Lytton, R. (1987). Concepts of pavement performance prediction and modeling. In *2nd North American Conference on Managing 1987*. Retrieved from http://mail.tku.edu.tw/yinghaur/lee/pms/all-handouts/G.pdf

Mamlouk, M., & Zaniewski, J. (1998). Pavement Preventive Maintenance: Description, Effectiveness, and Treatments. In *Flexible Pavement Rehabilitation and Maintenance* (pp. 121-121–15). 100 Barr Harbor Drive, PO Box C700, West Conshohocken, PA 19428-2959: ASTM International. https://doi.org/10.1520/STP12856S

Mannering, F., Kilareski, W., & Washburn, S. (2007). *Principles of highway engineering and traffic analysis*.

Markow, M. J., & Balta, W. S. (1985). Optimal Rehabilitation Frequencies for Highway Pavements. *Texas Research and Development Company*, 1–9. Retrieved from http://onlinepubs.trb.org/Onlinepubs/trr/1985/1035/1035-004.pdf

Maurer, G., Bemanian, S., & Polish, P. (2007). Alternative Strategies for Rehabilitation of Low-Volume Roads in Nevada. *Transportation Research Record: Journal of the Transportation Research Board*, *1989*, 309–320. https://doi.org/10.3141/1989-78

McGhee, K. (1994). Portland Cement Concrete Resurfacing, NCHRP Synthesis of Highway Practice 204. *Transportation Research Board, Washington, DC*.

Menge, C., Rossano, C., Anderson, G., & Bajdek, C. (1998). FHWA traffic noise model, Version 1.0 technical manual.

Merritt, D. K., Rogers, R. B., & Rasmussen, R. O. (2008). Construction of a Precast Prestressed Concrete Pavement Demonstration Project on Interstate 57 near Sikeston, Missouri, 133. https://doi.org/10.1017/CBO9781107415324.004

Michael S. Mamlouk, John P. Zaniewski, W. H. (2000). Analysis and design optimization of flexible pavement. *Journal of Transportation Engineering.*, *126*(2), 161–167.

Miedema, H. M. E., & Oudshoorn, C. G. M. (2001). Annoyance from transportation noise. *Environmental Health Perspectives*, *109*(4), 409–416. https://doi.org/10.1289/ehp.01109409

MNDOT. (2014). MnDOT Pavement Design Manual MNDOT PAVEMENT DESIGN MANUAL.

Mohamed Jaafar, Z. F. Bin, Uddin, W., & Najjar, Y. (2016). Asphalt Pavement Roughness Modeling Using the Artificial Neural Network and Linear Regression Approaches for LTPP Southern Region. *Transportation Research Board 95th Annual MeetingTransportation Research Board*, *No. 16-419*. Retrieved from https://trid.trb.org/view/1393569

Mohod, M., & Kadam, K. (2016). A Comparative Study on Rigid and Flexible Pavement: A Review. *IOSR Journal of Mechanical and Civil Engineering (IOSR-JMCE), Volume 13,*(3 Ver. VII), 84–88. https://doi.org/10.9790/1684-1303078488

Morian, D. A., Gibson, S. D., & Epps, J. A. (1998, March 1). Maintaining flexible pavements : the long term pavement performance experiment SPS-3 5-year data analysis. United States. Federal Highway Administration. Retrieved from https://rosap.ntl.bts.gov/view/dot/4551

Morian, D., Coleman, L., Frith, D., Stoffels, S., & Dawood, D. (2003). Pennsylvania SPS-6 Performance at 10 Years: Evaluation of Concrete Pavement Rehabilitation Strategies. *Transportation Research Record: Journal of the Transportation Research Board*, *1823*, 28–38. https://doi.org/10.3141/1823-04

Morin, T. L., & Marsten, R. E. (1976). An Algorithm for Nonlinear Knapsack Problems. *Management Science*, *22*(10), 1147–1158. https://doi.org/10.1287/mnsc.22.10.1147

Nagayach, R. (2015). What are the differences between concrete, cement, and asphalt? - Quora. Retrieved May 20, 2018, from https://www.quora.com/What-are-the-differences-between-concrete-cement-and-asphalt

NAPA. (2018). National Asphalt Pavement Association: Engineering Overview. Retrieved May 14, 2018, from http://www.asphaltpavement.org/index.php?option=com_content&view=article&id=14&Itemid=33

Nisbet, M. A., Marceau, M. L., VanGeem, M. G., & Gajda, J. (2001). *Environmental life cycle inventory of Portland cement concrete and asphalt concrete pavements*.

Nunn, M. (2004). Development of a more versatile approach to flexible and flexible composite pavement design. Retrieved from https://trid.trb.org/view/1159034

O'Brien, L. (1989). *Evolution and benefits of preventive maintenance strategies*. Retrieved from https://trid.trb.org/view/312563

Ong, G. P. (Raymond), Nantung, T., & Sinha, K. (2011). *Indiana Pavement Preservation Program*. *JTRP Technical Reports*. https://doi.org/10.5703/1288284314258

Opus. (1999). *Review of VOC-Pavement Roughness Relationships Contained in Transfund's Project Evaluation Manual*. Lower Hutt, New Zealand.

Ozbay, K., Parker, N., & Jawad, D. (2003). *Guidelines for life cycle cost analysis*. Retrieved from https://trid.trb.org/view/798636

Papagiannakis, A. T., & Masad, E. (2008). *Pavement design and materials*. John Wiley.

Paterson, W., & T Watanatada. (1985). Relationships between vehicle speed, ride quality, and road roughness. *Astm.Org*. Retrieved from http://www.astm.org/DIGITAL_LIBRARY/STP/PAGES/STP34596S.htm

Pellecuer, L., Assaf, G. J., & St-Jacques, M. (2014a). Influence of Pavement Condition on Environmental Costs. *Journal of Transportation Engineering*, *140*(10), 1–15. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000721

Pellecuer, L., Assaf, G. J., & St-Jacques, M. (2014b). Life cycle environmental benefits of pavement surface maintenance. *Canadian Journal of Civil Engineering*, *41*(8), 695–702. https://doi.org/10.1139/cjce-2013-0508

Perera, R., & Kohn, S. (1999). International Roughness Index of Asphalt Concrete Overlays: Analysis of Data from Long-Term Pavement Performance Program SPS-5 Projects. *Transportation Research Record: Journal of the Transportation Research Board*, *1655*, 100–109. https://doi.org/10.3141/1655-14

Perera, R. W., & Kohn, S. D. (2006). Ride Quality Performance of Asphalt Concrete Pavements Subjected to Different Rehabilitation Strategies. In *Airfield and Highway Pavement* (pp. 789–800). Reston, VA: American Society of Civil Engineers. https://doi.org/10.1061/40838(191)67

Peshkin, D., Hoerner, T., & Zimmerman, K. (2004). *Optimal timing of pavement preventive maintenance treatment applications*. Retrieved from https://books.google.com/books?hl=en&lr=&id=QBLxd-PGn18C&oi=fnd&pg=PA4&dq=Peshkin,+D.G.,+Hoerner,+T.E.+and+Zimmerman,+K.A.,.+Optimal+timing+of+pavement+preventive+maintenance+treatment+applications.+Washington,+DC.:+Federal+Highway+Administration,+2004.&o

Pilson, C., Hudson, W., & Anderson, V. (1999). Multiobjective Optimization in Pavement Management by Using Genetic Algorithms and Efficient Surfaces. *Transportation Research Record: Journal of the Transportation Research Board*, *1655*, 42–48. https://doi.org/10.3141/1655-07

Plei, M., & Tayabji, S. (2012). *Continuously Reinforced Concrete Pavement Performance and Best Practices*. Retrieved from https://trid.trb.org/view/1264832

Pozder, M. (2012). Research on the Effects of Pavement Surface Condition on Road Traffic Noise. *Archives for Technical Sciences*, *1*(8), 57–64. https://doi.org/10.7251/afts.2013.0508.057P

Quintus, V., L, H., Mallela, J., & Jiang, J. (2005). *Expected Service Life and Performance Characteristics of HMA Pavements in LTPP*. *Asphalt Pavement Alliance*. Retrieved from https://trid.trb.org/view/813989

Rajagopal, A. S., & George, K. P. (1991). Pavement Maintenance Effectiveness. *Transportation Research Record*, *1276*, 62–68.

Rangaraju, P., Amirkhanian, S., & Guven, Z. (2005). *Life cycle cost analysis for pavement type selection*. Retrieved from http://www.scdot.scltap.org/wp-content/uploads/2017/05/SPR656Final.pdf

Rasmussen, R. O., Bernhard, R. J., Sandberg, U., & Mun, E. P. (2007). The Little Book of Quieter Pavements. Retrieved from https://trid.trb.org/view/870845

Ravirala, V., & Grivas, D. A. (2002). State Increment Method of Life-Cycle Cost Analysis for Highway Management. *Journal of Infrastructure Systems*, *1*(3), 151–159. https://doi.org/10.1061/(ASCE)1076-0342(1995)1:3(151)

Raza, H. (1994). *STATE-OF-THE-PRACTICE DESIGN, CONSTRUCTION, AND PERFORMANCE OF MICRO-SURFACING. FINAL REPORT*. Washington, DC United States. Retrieved from https://trid.trb.org/view/404828

Rebitzer, G., Ekvall, T., Frischknecht, R., & Hunkeler, D. (2004). Life cycle assessment: Part 1: Framework, goal and scope definition, inventory analysis, and applications. *Elsevier*. Retrieved from https://www.sciencedirect.com/science/article/pii/S0160412003002459

Reigle, J., & Zaniewski, J. (2002). Risk-Based Life-Cycle Cost Analysis for Project-Level Pavement Management. *Transportation Research Record: Journal of the Transportation Research Board*, *1816*, 34–42. https://doi.org/10.3141/1816-05

Robbins, M. (2016). *A Synthesis Report: Value of Pavement Smoothness and Ride Quality to Roadway Users and the Impact of Pavement Roughness on Vehicle Operating Costs*. *pavementpreservation.org*. Auburn, Alabama. Retrieved from https://www.pavementpreservation.org/wp-content/uploads/2016/05/rep16-03.pdf

Roudebush, W. H. (1999). *Environmental value engineering assessment of concrete and asphalt pavement*. Skokie, IL.

Rubinstein, R. Y., & Y., R. (1981). *Simulation and the monte carlo method*. Wiley. Retrieved from https://dl.acm.org/citation.cfm?id=539488

Saaty, T. L. (1977). A scaling method for priorities in hierarchical structures. *Journal of Mathematical Psychology*, *15*(3), 234–281. https://doi.org/10.1016/0022-2496(77)90033-5

Sasraku-Neequaye, B., & Amedegbe-Doe, D. (2017). COMPARATIVE COST ANALYSIS BETWEEN ASPHALT PAVEMENT AND CONCRETE PAVEMENT IN ROAD CONSTRUCTION (A Case study of Comparative Cost analysis between asphalt pavement and Concrete grade 42 pavement in Road Construction.). *International Journal of Advanced Engineering Research and Technology*, *5*(6). Retrieved from http://www.ijaert.org/wp-content/uploads/2017/06/150227.pdf

Sayers, M. (1986). Guidelines for conducting and calibrating road roughness measurements.

Sebaaly, P., Lani, S., Record, A. H.-T. research, & 1995, U. (995). Performance models for flexible pavement maintenance treatments. *Transportation Research Record*, *1508*. Retrieved from https://trid.trb.org/view/453079

Shober, S., & Friedrichs, D. (1998). Pavement Preservation Strategy. *Transportation Research Record: Journal of the Transportation Research Board*, *1643*, 44–53. https://doi.org/10.3141/1643-07

Sinha, K.C., Labi, S. (2018). *Highway Infrastructure Asset Management*. Boca Raton, FL: CRC Press.

Sinha, K. C., & Labi, S. (2007). Evaluation of Transportation Projects and Programs Using Multiple Criteria. *Transportation Decision Making: Principles of Project Evaluation and Programming*, 449–482.

Sinha, K. C., Patidar, V., Li, Z., Labi, S., & Thompson, P. D. (2009). Establishing the Weights of Performance Criteria: Case Studies in Transportation Facility Management. *Journal of Transportation Engineering*, *135*(9), 619–631. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000039

Sinha, K., & Labi, S. (2007). *Transportation decision making: Principles of project evaluation and programming*. Retrieved from https://books.google.com/books?hl=en&lr=&id=870LEFbxZTQC&oi=fnd&pg=PR7&dq=Sinha+K.+C.,+Labi,+S.+Transportation+Decision-making,+Principles+of+Project+Evaluation+and+Programming.+NJ:+Wiley,+Hoboken,+2007.&ots=-KUBdRQp82&sig=Oz5WKvdsyhQ5c4Q2h2aHTfi_E10

Smith, K. L., & Peshkin, D. G. (2011). Pavement Preservation on High-Traffic-Volume Roadways. *Transportation Research Record: Journal of the Transportation Research Board*, *2235*(1), 54–65. https://doi.org/10.3141/2235-07

Smith, R., Freeman, T., & Pendleton, O. (1993). *PAVEMENT MAINTENANCE EFFECTIVENESS*. Washington, DC. Retrieved from https://trid.trb.org/view/386726

Staatsen, B., Nijland, H., Kempen, E., Hollander, A., Franssen, A., & Kamp, I. (2004). Assessment of health impacts and policy options in relation to transport-related noise exposures. Bilthoven: RIVM, 2004. Retrieved from http://www.rivm.nl/bibliotheek/rapporten/815120002.pdf

Stripple, H. (2001). Life cycle assessment of Road. *Swedish Environmental Research Institute IVL*, (March), 96p. https://doi.org/10.1177/0734242X10379146

Sullivan, E. J., & Moss, A. (2014). *Paving Cost Comparisons: Warm-Mix Asphalt Versus Concrete PCA Market Intelligence*. Retrieved from http://www.cement.org/docs/default-source/market-economics-pdfs/issues-trends/paving_cost_comparisons_flash.pdf?sfvrsn=4

Swei, O. (2012). *Incorporating uncertainty in the Life Cycle Cost Analysis of pavements*. Massachusetts Institute of Technology. Retrieved from https://dspace.mit.edu/handle/1721.1/78541

Taha, H. (2005). *Operations research: An introduction (for VTU)*. Retrieved from https://scholar.google.com/scholar?hl=en&as_sdt=0%2C15&q=Operations+Research%3A+An+Introduction+%289th+edition%29+by+Hamdy+A+Taha&btnG=

Thakkar, C. (2016). *DEVELOPMENT OF LIFE CYCLE ASSESSMENT TOOL FOR PAVEMENT SUSTAINABILITY ANALYSIS*. The State University of New Jersey - New Brunswick. Retrieved from https://search.proquest.com/openview/a93402125b7007c71ee725735069a1a1/1?pq-origsite=gscholar&cbl=18750&diss=y

Tighe, S. (2001). Guidelines for Probabilistic Pavement Life Cycle Cost Analysis. *Transportation Research Record: Journal of the Transportation Research Board*, *1769*, 28–38. https://doi.org/10.3141/1769-04

Treloar, G. J., Love, P. E. D., & Crawford, R. H. (2004). Hybrid Life-Cycle Inventory for Road Construction and Use. *Journal of Construction Engineering and Management*, *130*(1), 43–49. https://doi.org/10.1061/(ASCE)0733-9364(2004)130:1(43)

Uljarevic, M., & Supic, S. (2016). COMPARATIVE ANALYSIS OF FLEXIBLE AND RIGID PAVEMENT DESIGN. In *4th INTERNATIONAL CONFERENCE* (pp. 591–601). Journal of Faculty of Civil Engineering. https://doi.org/10.14415/konferencijaGFS 2016.060

Venkatram, A., & Horst, T. W. (2006). Approximating dispersion from a finite line source. *Atmospheric Environment*, *40*(13), 2401–2408. https://doi.org/10.1016/J.ATMOSENV.2005.12.014

Villacres, J. N. (2005). Pavement Life-Cycle Cost Studies Using Actual Cost Data – A Synthesis. Retrieved from https://trid.trb.org/view/813988

Volovski, M., Murillo-Hoyos, J., Saeed, T. U., & Labi, S. (2017). Estimation of Routine Maintenance Expenditures for Highway Pavement Segments: Accounting for Heterogeneity Using Random-Effects Models. *Journal of Transportation Engineering, Part A: Systems*, *143*(5), 04017006. https://doi.org/10.1061/JTEPBS.0000041

Wadsworth, H. M. (1990). *Handbook of statistical methods for engineers and scientists*. McGraw-Hill Pub. Co. Retrieved from http://www.sidalc.net/cgi-bin/wxis.exe/?IsisScript=FCL.xis&method=post&formato=2&cantidad=1&expresion=mfn=001001

Walls, J., & Smith, M. R. (1998). Life-cycle cost analysis in pavement design: in search of better investment decisions, (FHWA/SA-98-079). Retrieved from https://trid.trb.org/view/1159632

Wang, T., Harvey, J., Lea, J., & Kim, C. (2014). Impact of Pavement Roughness on Vehicle Free-Flow Speed. *Journal of Transportation Engineering*, *140*(9), 04014039. https://doi.org/10.1061/(ASCE)TE.1943-5436.0000689

WAPA. (2018). Pavement Facts | Washington Asphalt Pavement Association. Retrieved July 2, 2018, from http://www.asphaltwa.com/welcome-facts/

Washington, S., Karlaftis, M. G., & Mannering, F. L. (2010). *Statistical and econometric methods for transportation data analysis*. Retrieved from https://books.google.com/books?id=OZrMBQAAQBAJ&dq=Washington+S,+Karlaftis+M+and+Mannering+F+(2010)+Statistical+and+Econometric+Methods+for+Transportation+Data+Analysis.+CRC+Press,+Boca+Raton,+FL,+USA.&lr=&source=gbs_navlinks_s

Weiland, C. D., Muench, S. T., Engineering, U. of W. D. of C. and E., & Center, W. S. T. (2010, February 1). Life cycle assessment of Portland cement concrete interstate highway rehabilitation and replacement. Washington (State). Dept. of Transportation. Office of Research and Library Services. Retrieved from https://rosap.ntl.bts.gov/view/dot/22404

West, R., Tran, N., Musselman, M., Skolnik, J., & Brooks, M. (2012). *A Review of the Alabama Department of Transportation's Policies and Procedures for Life-Cycle Cost Analysis for Pavement Type Selection*. Retrieved from http://www.eng.auburn.edu/research/centers/ncat/files/technical-reports/rep13-06.pdf

WHO. (2000). World Health Organisation – Guidelines for Community Noise. Retrieved from https://infrastructure.planninginspectorate.gov.uk/document/2322958

Wimsatt, A., Chang-Albitres, C., & Krugler, P. (2009). Considerations for Rigid Vs. Flexible Pavement Designs when Allowed as Alternate Bids: Technical Report, *7*(2). Retrieved from http://d2dtl5nnlpfr0r.cloudfront.net/tti.tamu.edu/documents/0-6085-1.pdf

Winston, W., Venkataramanan, M., & Goldberg, J. (2003). *Introduction to mathematical programming*. Retrieved from http://www.philadelphia.edu.jo/math/syllabi/250492b.pdf

WSDOT. (2012). *Washington State Department of Transportation: Design Manual*.

Wu, Z. P. D. P. E., Groeger, J. L., Simpson, A. L. P. D. P. E., & Hicks, R. G. P. D. P. E. (2010). Performance Evaluation of Various Rehabilitation and Preservation Treatments, 90.

Yoder, E., & Witczak, M. (1975). *Principles of pavement design*.

Zaniewski, J. P. (1989). *Effect of pavement surface type on fuel consumption*. Skokie, IL.

Zaniewski, J. P., & Mamlouk, M. S. (1996). PAVEMENT MAINTENANCE EFFECTIVENESS - PREVENTIVE MAINTENANCE TREATMENTS. PARTICIPANT'S HANDBOOK. Retrieved from https://trid.trb.org/view/472777

Zapata, P., & Gambatese, J. A. (2005). Energy Consumption of Asphalt and Reinforced Concrete Pavement Materials and Construction. *Journal of Infrastructure Systems*, *11*(1), 9–20. https://doi.org/10.1061/(ASCE)1076-0342(2005)11:1(9)

Zhang, H., Keoleian, G. A., & Lepech, M. D. (2012). Network-Level Pavement Asset Management System Integrated with Life-Cycle Analysis and Life-Cycle Optimization. *Journal of Infrastructure Systems*, *19*(1), 99–107. https://doi.org/10.1061/(ASCE)IS.1943-555X.0000093

APPENDIX A. Age-based Performance Models

Age-based Performance Models for Flexible Pavement Maintenance (SPS 3) for wet-freeze zone

Treatment	Coefficients		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	α	4.254	91.010	85	0.8924	96.73	**21.01**
	β	0.021	7.013				
SPS-320	α	4.444	100.340	87	0.8560	-	-
	β	0.020	4.228				
SPS-330	α	4.655	77.450	91	0.8901	-	-
	β	0.020	5.331				
SPS-340	α	4.587	85.340	84	0.7869	-	-
	β	0.034	7.176				
SPS-350	α	4.477	101.729	87	0.8560	-	-
	β	0.016	5.186				

Age-based Performance Models for Flexible Pavement Rehabilitation (SPS 5) for wet-freeze zone

Treatment	Coefficients		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	α	4.544	59.733	56	0.9499	-	-
	β	0.034	12.891				
SPS-502	α	4.063	65.040	66	0.7958	104.53	**41.18**
	β	0.020	5.277				
SPS-503	α	4.006	69.410	67	0.7988	125.96	**71.98**
	β	0.022	6.715				
SPS-504	α	4.103	64.246	65	0.9243	130.35	**73.56**
	β	0.022	9.773				
SPS-505	α	4.082	58.475	65	0.9117	115.81	**58.24**
	β	0.020	8.063				
SPS-506	α	4.024	70.043	65	0.9157	91.43	**43.51**
	β	0.021	9.959				
SPS-507	α	4.097	103.953	67	0.8386	119.39	**66.21**
	β	0.010	5.388				
SPS-508	α	3.971	78.903	65	0.9341	106.59	**54.62**
	β	0.013	7.965				
SPS-509	α	4.115	67.568	65	0.8601	120.08	**63.95**
	β	0.023	8.113				

Age-based Performance Models for Rigid Pavement Maintenance (SPS 4) for wet-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2
Joint Sealing	*α*	4.63	48.457	64	0.9871
	β	0.017	8.567		
Crack Sealing	*α*	4.744	523.37	12	0.7748
	β	0.007	5.866		
Partial-Depth Patching	*α*	4.685	65.37	24	0.9335
	β	0.015	2.770		
Full-Depth Patching	*α*	4.875	31.879	14	0.9874
	β	0.036	6.004		

Age-based Performance Models for Rigid Pavement Rehabilitation (SPS 6) for wet-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	*α*	4.837	143.99	53	0.84	-	**-**
	β	0.048	14.558				
SPS-602	*α*	4.818	104.13	58	0.759	-	-
	β	0.042	8.804				
SPS-603	*α*	4.110	139.67	89	0.8091	130.46	**61.96**
	β	0.038	20.537				
SPS-604	*α*	4.154	194.75	89	0.9130	133.80	**65.72**
	β	0.033	23.934				
SPS-606	*α*	4.085	156.75	89	0.9318	147.27	**83.75**
	β	0.035	27.137				
SPS-607	*α*	4.107	150.08	77	0.8375	142.78	**73.10**
	β	0.024	12.684				
SPS-608	*α*	4.135	158.67	85	0.9352	119.31	**54.55**
	β	0.010	11.078				

Age-based Performance Models for Flexible Pavement Maintenance (SPS 3) for wet-non- freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	α	4.189	100.824	24	0.8379	89.68	**23.10**
	β	0.043	7.577				
SPS-320	α	4.523	29.190	34	0.9536	-	-
	β	0.030	5.783				
SPS-330	α	3.954	37.160	26	0.8355	-	-
	β	0.088	7.160				
SPS-340	α	4.156	122.262	33	0.7087	-	-
	β	0.024	7.088				
SPS-350	α	4.186	80.415	25	0.7283	-	-
	β	0.039	5.003				

Age-based Performance Models for Flexible Pavement Rehabilitation (SPS 5) for wet-non- freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	α	4.142	113.660	10	0.9187	-	-
	β	0.056	9.510				
SPS-502	α	3.872	59.655	63	0.9401	91.29	**37.67**
	β	0.029	16.289				
SPS-503	α	3.719	60.394	39	0.8942	90.05	**28.31**
	β	0.028	9.019				
SPS-504	α	3.963	36.048	58	0.4703	94.42	**44.15**
	β	0.011	1.162				
SPS-505	α	3.891	41.974	63	0.9802	87.77	**31.98**
	β	0.022	15.824				
SPS-506	α	3.866	40.119	60	0.9509	81.50	**32.76**
	β	0.018	7.751				
SPS-507	α	3.829	54.794	66	0.8930	84.60	**36.42**
	β	0.017	7.607				
SPS-508	α	4.012	67.925	61	0.9374	85.84	**26.85**
	β	0.014	8.767				
SPS-509	α	3.633	66.257	56	0.8704	102.95	**48.87**
	β	0.017	8.175				

Age-based Performance Models for Rigid Pavement Maintenance (SPS 4) for wet-non- freeze zone

Treatment	Coefficients		t-stat	No. Obs.	R^2
Joint Sealing	α	4.617	111.895	34	0.8063
	β	0.013			

Age-based Performance Models for Rigid Pavement Rehabilitation (SPS 6) for wet-non- freeze zone

Treatment	Coefficients		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	α	4.754	30.048	18	0.877	-	-
	β	0.0489	4.399				
SPS-602	α	4.055	69.254	16	0.75	-	-
	β	0.067	6.057				
SPS-603	α	4.018	26.078	16	0.9901	115.54	**55.57**
	β	0.032	9.687				
SPS-604	α	3.933	42.55	16	0.7489	100.14	**43.82**
	β	0.068	5.061				
SPS-605	α	3.976	67.816	16	0.7863	-	-
	β	0.079	6.828				
SPS-606	α	4.118	22.485	16	0.9719	152.67	**83.79**
	β	0.026	4.039				
SPS-607	α	3.868	18.659	12	0.9821	128.84	**65.70**
	β	0.205	17.298				
SPS-608	α	3.933	83.401	16	0.9821	169.24	**117.92**
	β	0.011	8.301				

Age-based Performance Models for Flexible Pavement Maintenance (SPS 3) for dry-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	α	4.214	78.561	56	0.7233	96.75	**29.91**
	β	0.068	8.771				
SPS-320	α	4.638	42.304	83	0.9516	-	-
	β	0.036	8.148				
SPS-330	α	4.271	38.834	53	0.8832	-	-
	β	0.071	8.787				
SPS-340	α	4.293	31.203	28	0.8890	-	-
	β	0.086	8.250				
SPS-350	α	4.699	42.776	83	0.9547	-	-
	β	0.029	6.821				

Age-based Performance Models for Flexible Pavement Rehabilitation (SPS 5) for dry-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	α	3.816	48.106	9	0.8332	-	-
	β	0.083	5.914				
SPS-502	α	3.972	54.419	20	0.8536	96.63	**41.09**
	β	0.060	8.259				
SPS-503	α	3.977	52.170	20	0.9554	112.46	**56.42**
	β	0.017	5.981				
SPS-504	α	3.838	108.173	20	0.9063	91.81	**42.67**
	β	0.022	9.092				
SPS-505	α	3.702	0.064	20	0.7693	77.68	**32.16**
	β	0.064	7.062				
SPS-506	α	3.894	26.007	19	0.8964	123.74	**81.04**
	β	0.072	7.360				
SPS-507	α	4.031	52.827	19	0.3651	74.76	**27.43**
	β	0.036	2.797				
SPS-508	α	3.832	159.882	20	0.6215	101.51	**52.82**
	β	0.020	5.185				
SPS-509	α	3.890	64.322	20	0.6643	62.03	**18.31**
	β	0.050	5.503				

Age-based Performance Models for Rigid Pavement Maintenance (SPS 4) for dry-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2
Joint Sealing	α	4.602	74.754	49	0.9684
	β	0.018	6.71		
Crack Sealing	α	5.116	223.829	48	0.9377
	β	0.02	14.67		
Partial-Depth Patching	α	4.785	38.779	13	0.9900
	β	0.008	2.593		

Age-based Performance Models for Rigid Pavement Rehabilitation (SPS 6) for dry-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	α	5.078	162.87	11	0.858	-	-
	β	0.034	7.373				
SPS-602	α	4.117	144	10	0.975	-	-
	β	0.081	17.565				
SPS-603	α	4.159	177.86	19	0.9661	165.05	**97.57**
	β	0.045	21.994				
SPS-604	α	4.391	246.12	19	0.9380	165.05	**120.07**
	β	0.025	16.034				
SPS-605	α	4.044	83.374	10	0.8832	-	-
	β	0.061	7.777				
SPS-606	α	4.206	174.92	19	0.8962	165.05	**111.57**
	β	0.026	12.112				
SPS-607	α	4.001	85.32	19	0.9476	144.21	**79.39**
	β	0.072	17.541				
SPS-608	α	3.994	174.03	19	0.9524	219.29	**166.95**
	β	0.037	18.437				

Age-based Performance Models for Flexible Pavement Maintenance (SPS 3) for dry-non-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-310	α	3.876	43.189	16	0.4669	99.97	**31.62**
	β	0.046	2.929				
SPS-320	α	4.020	41.118	40	0.9932	-	-
	β	0.015	9.085				
SPS-330	α	4.004	36.380	39	0.9727	-	-
	β	0.020	5.508				
SPS-340	α	4.000	36.198	33	0.9923	-	-
	β	0.020	11.168				
SPS-350	α	4.402	101.079	31	0.5664	-	-
	β	0.020	3.441				

Age-based Performance Models for Flexible Pavement Rehabilitation (SPS 5) for dry-non-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-501	α	3.937	24.667	29	0.9855	-	-
	β	0.058	12.161				
SPS-502	α	3.918	45.102	54	0.9493	161.70	**88.87**
	β	0.055	15.227				
SPS-503	α	3.772	46.430	53	0.9415	116.20	**52.14**
	β	0.037	12.104				
SPS-504	α	3.890	49.644	60	0.9477	116.56	**46.99**
	β	0.019	8.286				
SPS-505	α	3.880	45.437	53	0.9046	112.46	**57.17**
	β	0.031	7.808				
SPS-506	α	3.776	49.238	53	0.9351	104.86	**51.72**
	β	0.030	9.914				
SPS-507	α	3.922	59.538	60	0.8552	112.55	**54.28**
	β	0.021	6.099				
SPS-508	α	3.799	67.667	53	0.9282	99.66	**48.93**
	β	0.019	8.572				
SPS-509	α	3.735	37.954	53	0.9181	125.20	**69.66**
	β	0.065	13.219				

Age-based Performance Models for Rigid Pavement Maintenance (SPS 4) for dry-non-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2
Joint Sealing	α	4.547	159.283	23	0.7111
	β	0.007	3.197		
Crack Sealing	α	4.666	378.2	7	0.6931
	β	0.009	3.36		
Partial-Depth Patching	α	4.167	142.353	30	0.9099
	β	0.021	13.316		
Full-Depth Patching	α	5.051	53.784	14	0.9782
	β	0.022	6.589		

Age-based Performance Models for Rigid Pavement Rehabilitation (SPS 6) for dry-non-freeze zone

Treatment	*Coefficients*		t-stat	No. Obs.	R^2	IRI PRE (in/mile)	Average IRI Drop (in/mile)
SPS-601	α	4.733	395.62	19	0.965	-	-
	β	0.023	21.783				
SPS-602	α	4.394	55.652	31	0.915	-	-
	β	0.056	15.559				
SPS-603	α	3.926	52.738	39	0.8046	152.02	**99.39**
	β	0.081	11.710				
SPS-604	α	4.017	63.336	42	0.7803	152.34	**99.12**
	β	0.059	10.513				
SPS-605	α	4.177	45.269	34	0.8672	-	-
	β	0.082	13.462				
SPS-606	α	4.197	65.281	42	0.7726	125.35	**65.03**
	β	0.051	9.894				
SPS-607	α	4.062	54.32	42	0.5647	127.29	**66.36**
	β	0.057	6.754				
SPS-608	α	3.962	81.419	43	0.8237	128.81	**63.97**
	β	0.048	10.647				

APPENDIX B. Stochastic Dominance Curves

Wet-Freeze Zone

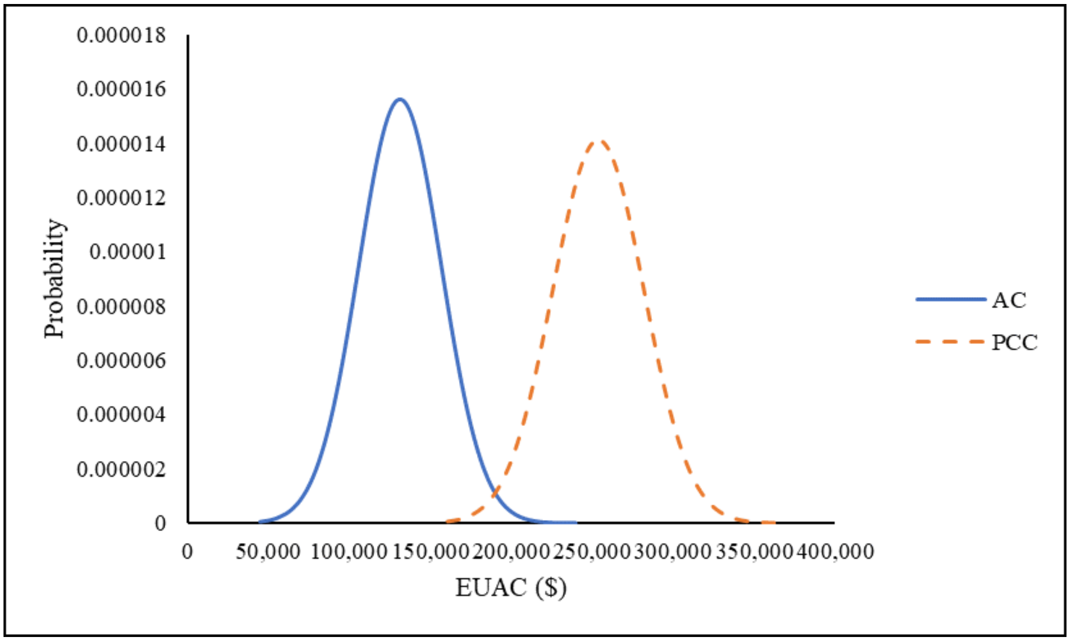


Figure B.1: Comparative EUAC probability distribution of the agency cost for AC and PCC pavements

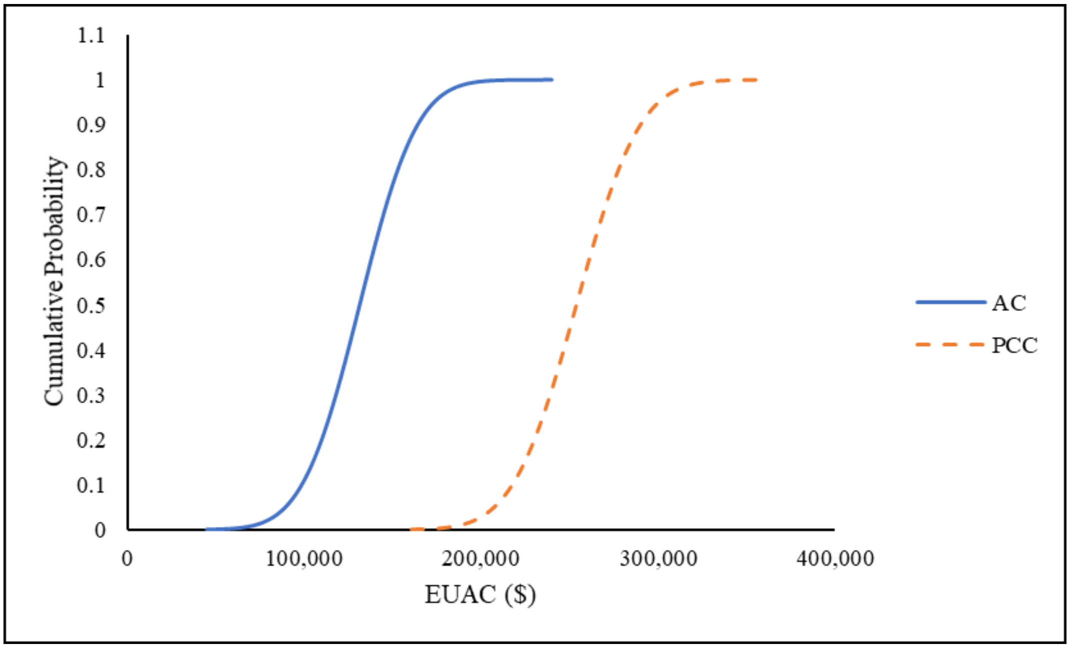


Figure B.2: Cumulative risk profile of the EUAC of the agency cost for AC and PCC pavements

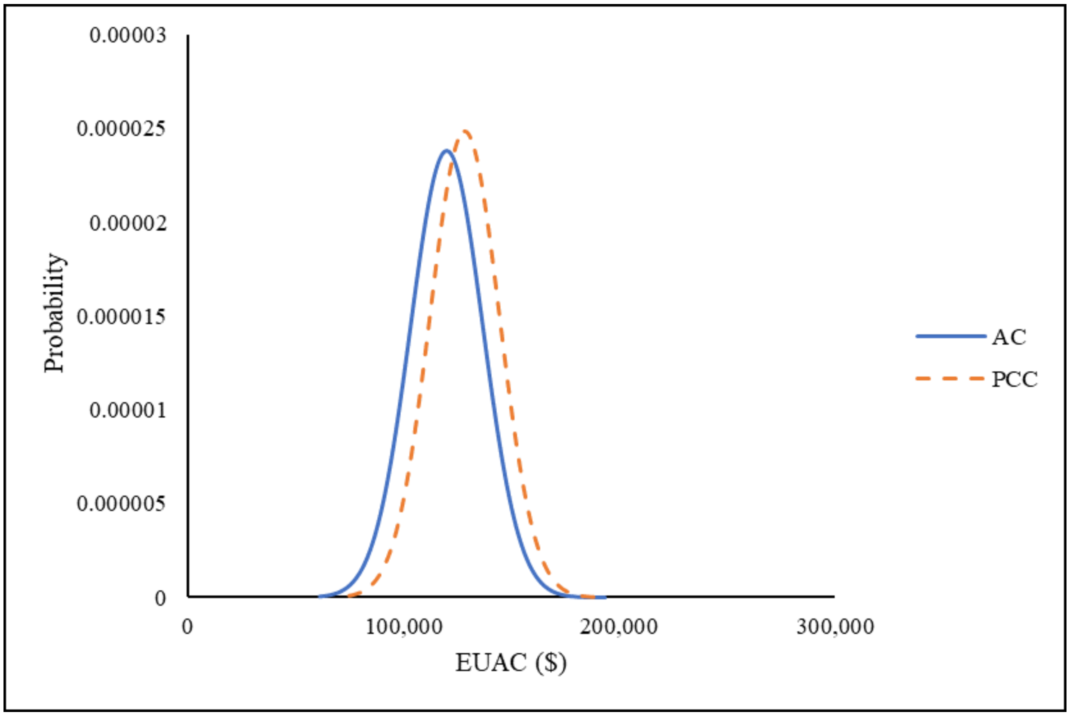


Figure B.3: Comparative EUAC probability distribution of the user cost (travel time delay cost) for AC and PCC pavements

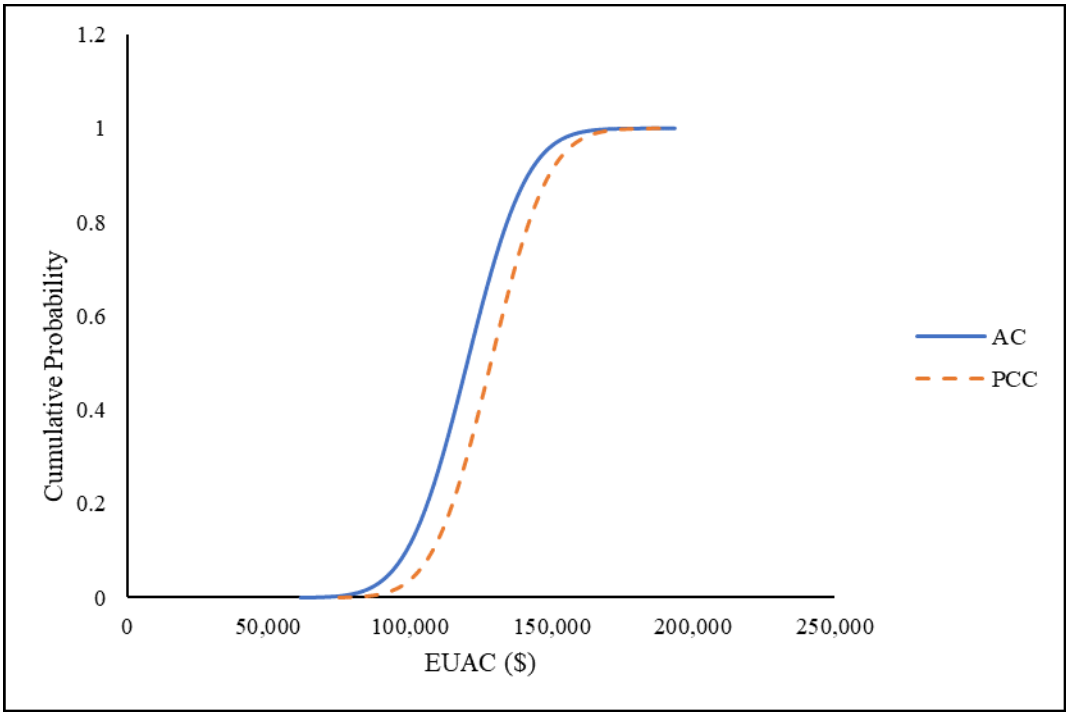


Figure B.4: Cumulative risk profile of the EUAC of the user cost (travel time delay cost) for AC and PCC pavements

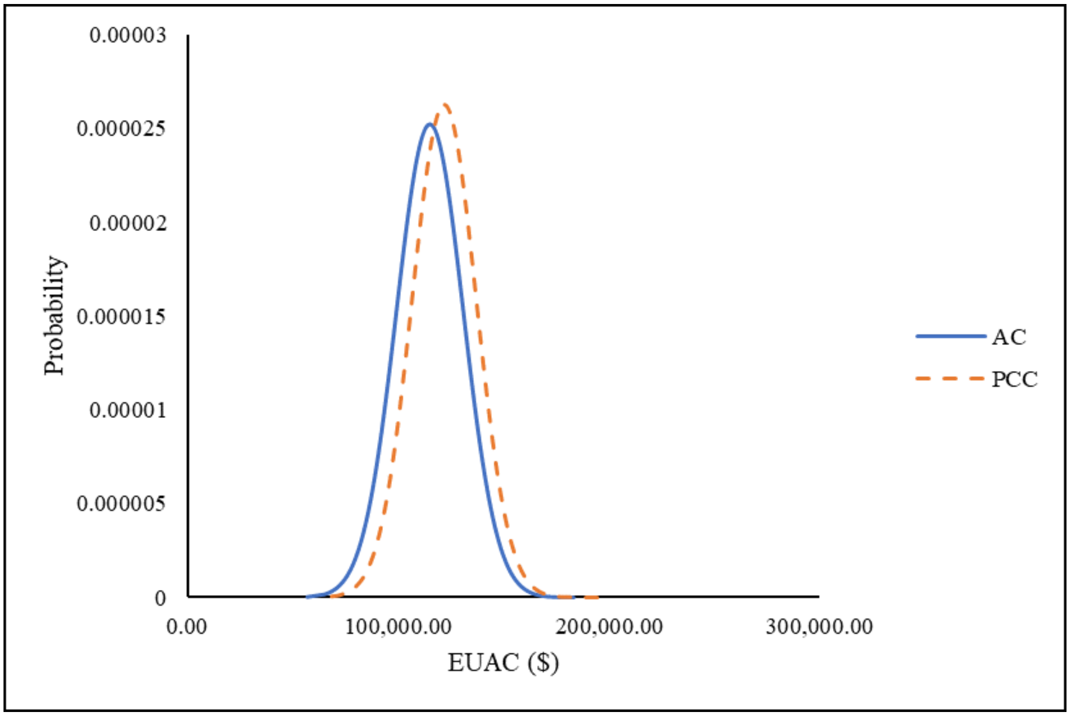


Figure B.5: Comparative EUAC probability distribution of the user cost (VOC) for AC and PCC pavements

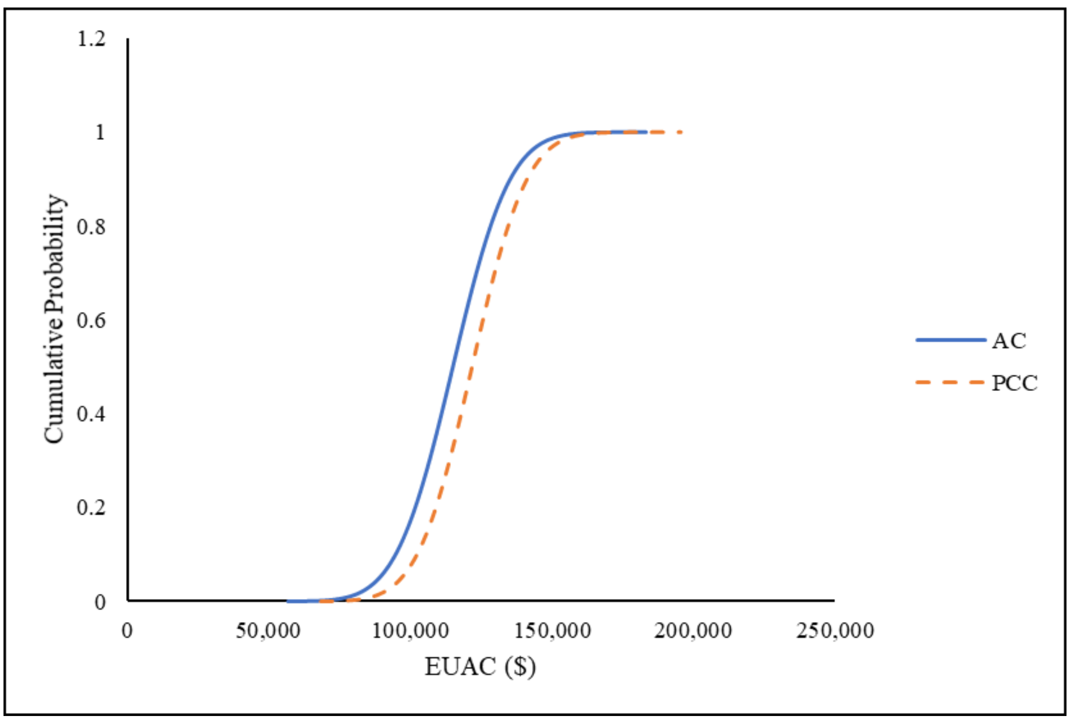


Figure B.6: Cumulative Risk profile of the EUAC of the user cost (VOC) for AC and PCC pavements

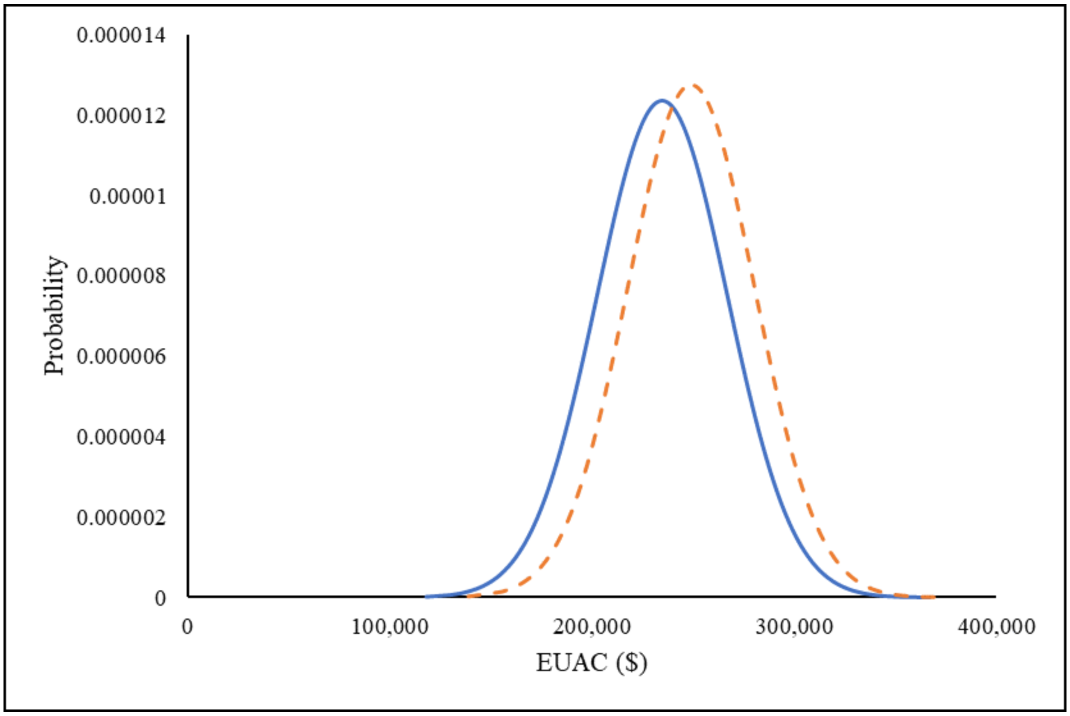


Figure B.7: Comparative EUAC probability distribution of the user cost for AC and PCC pavements

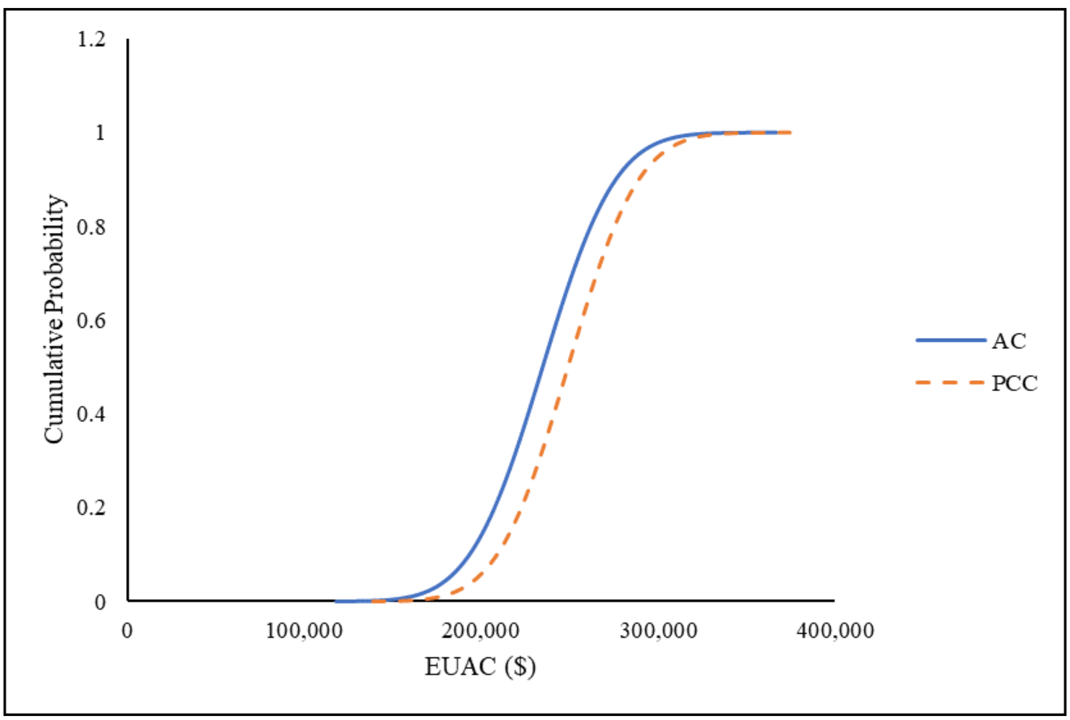


Figure B.8: Cumulative risk profile of the EUAC of the user cost for AC and PCC pavements

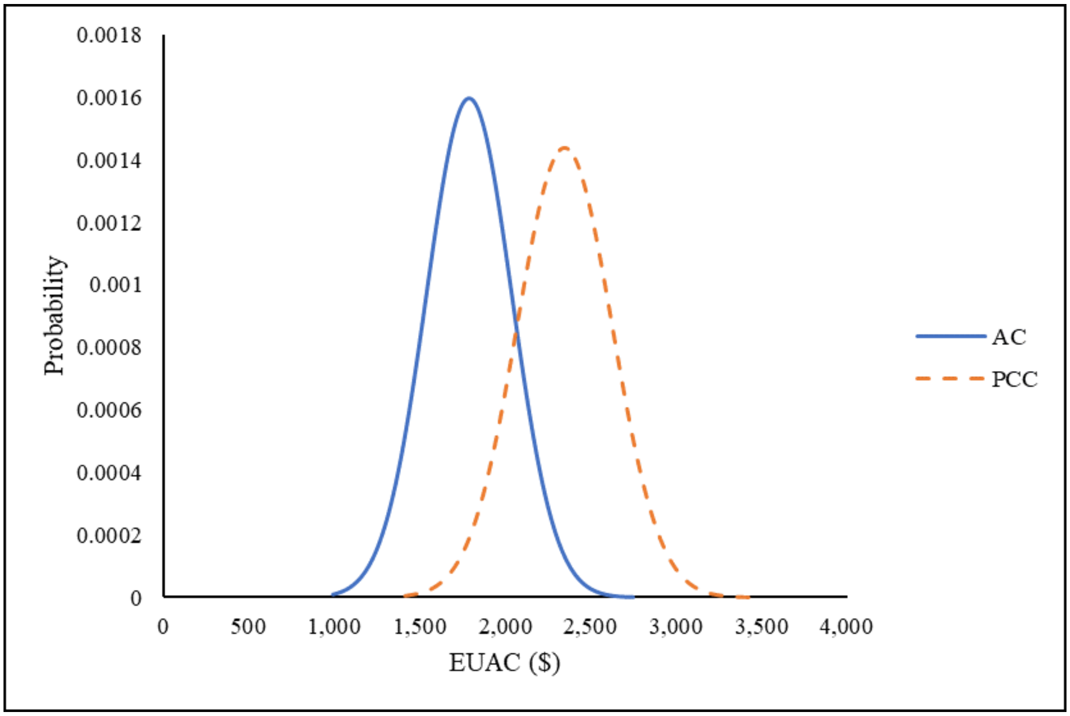


Figure B.9: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

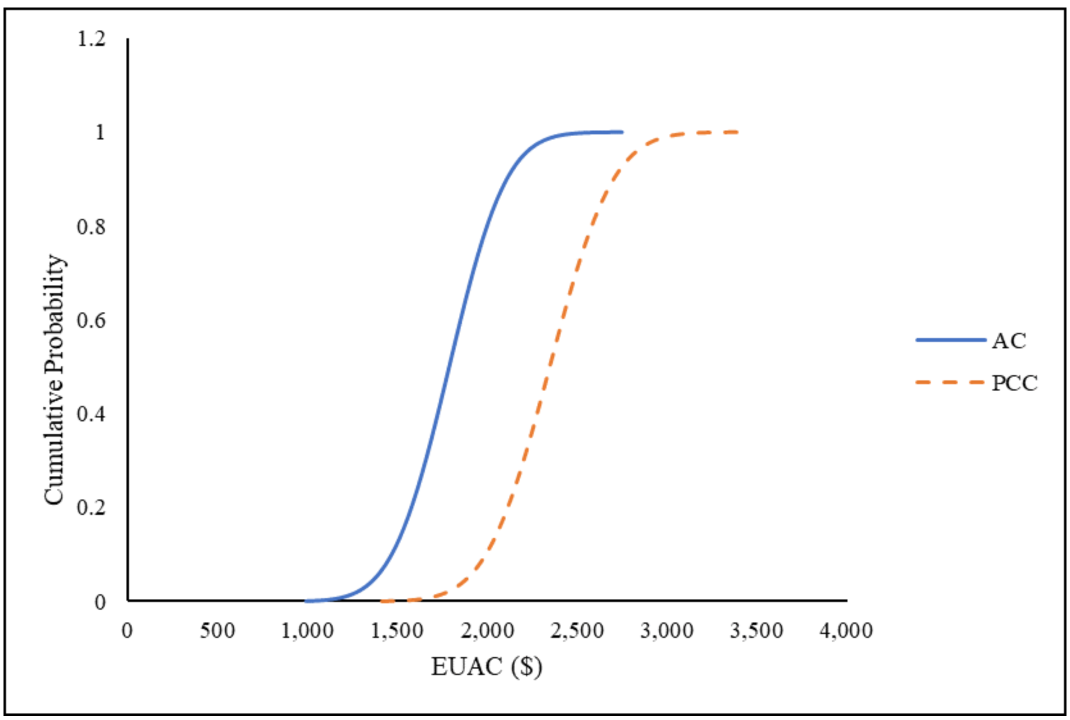


Figure B.10: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

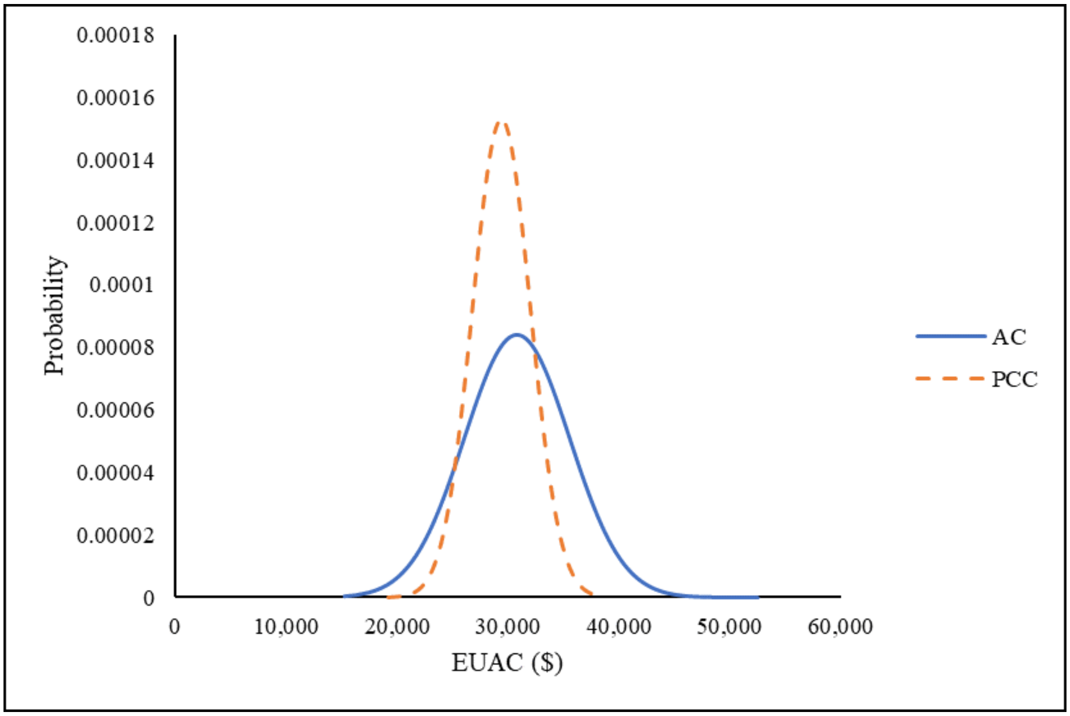


Figure B.11: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-energy consumption) for AC and PCC pavements

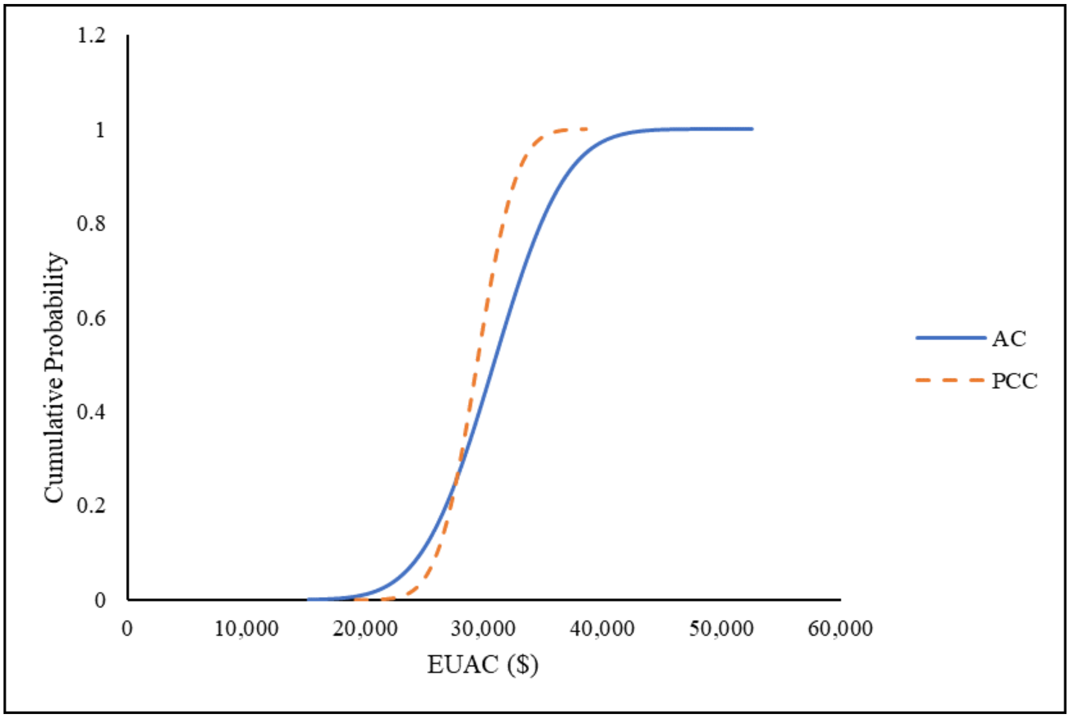


Figure B.12: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution- energy consumption) for AC and PCC pavements

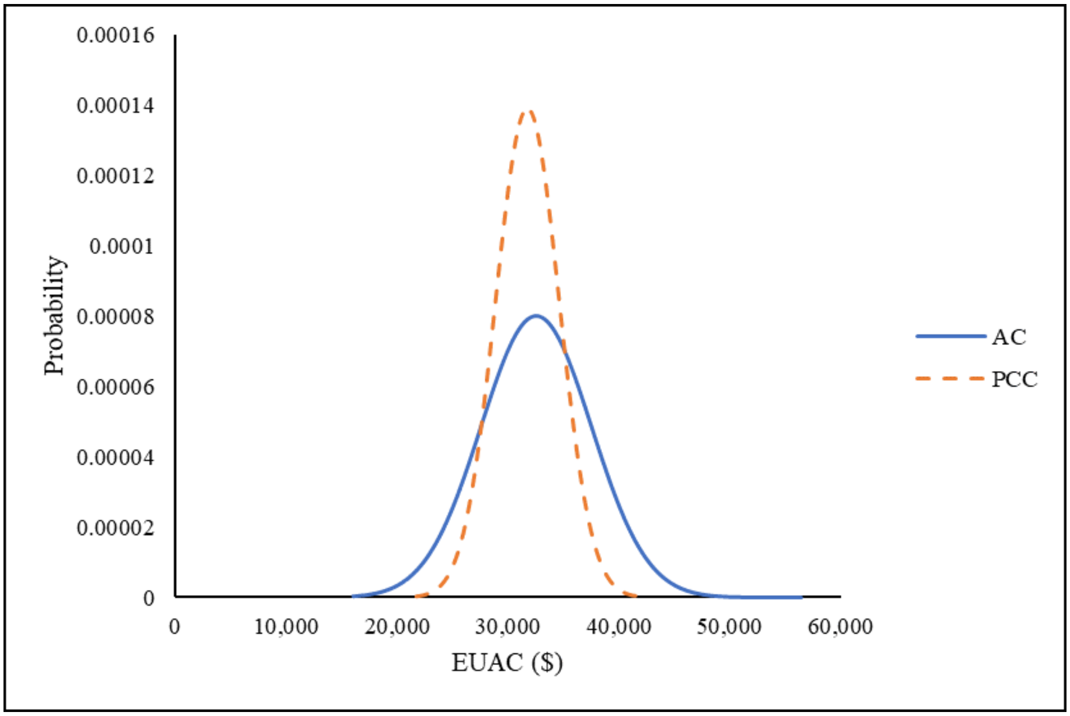


Figure B.13: Comparative EUAC probability distribution of the community cost (cost associated with air pollution) for AC and PCC pavements

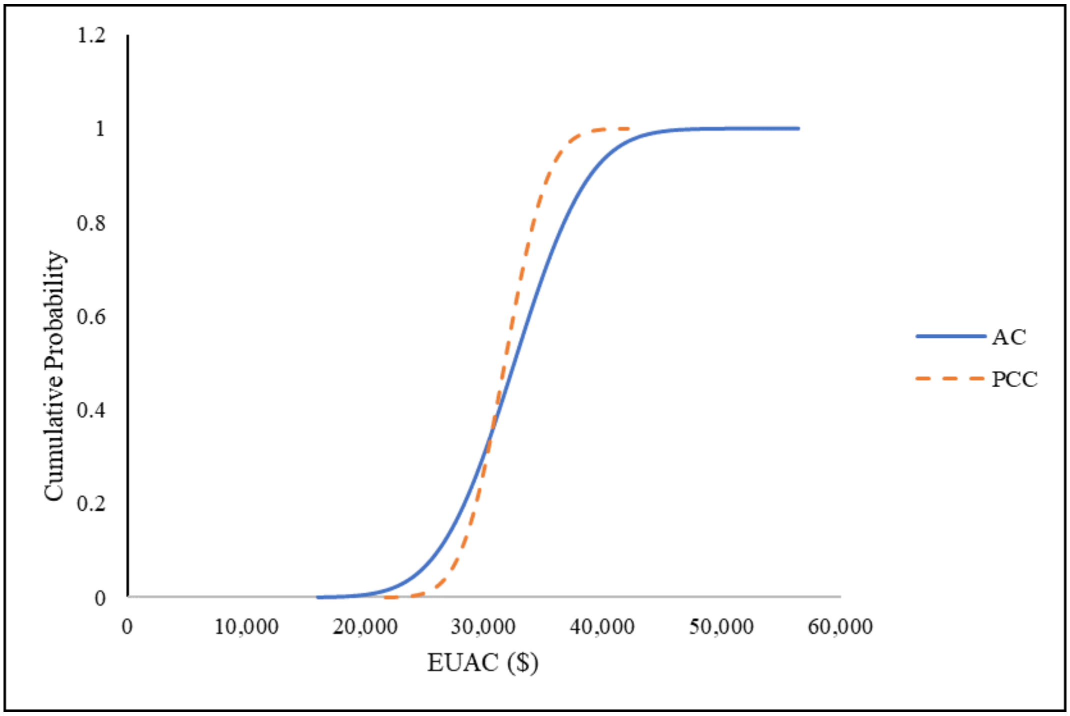


Figure B.14: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution) for AC and PCC pavements

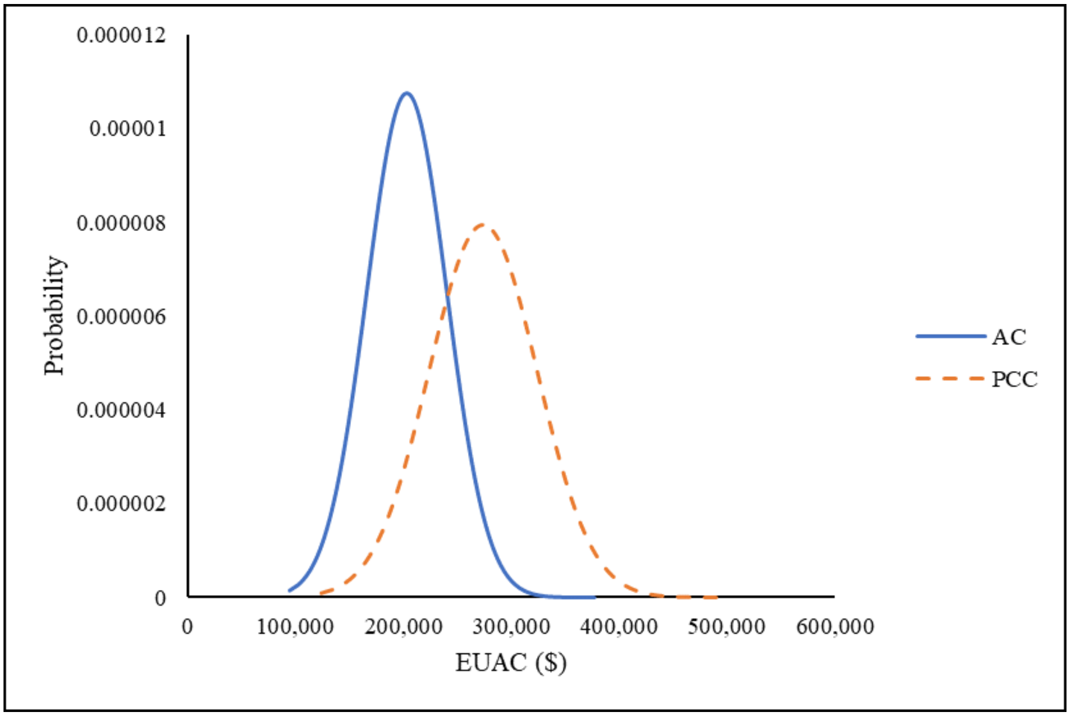


Figure B.15: Comparative EUAC probability distribution of the community cost (cost associated with noise pollution) for AC and PCC pavements

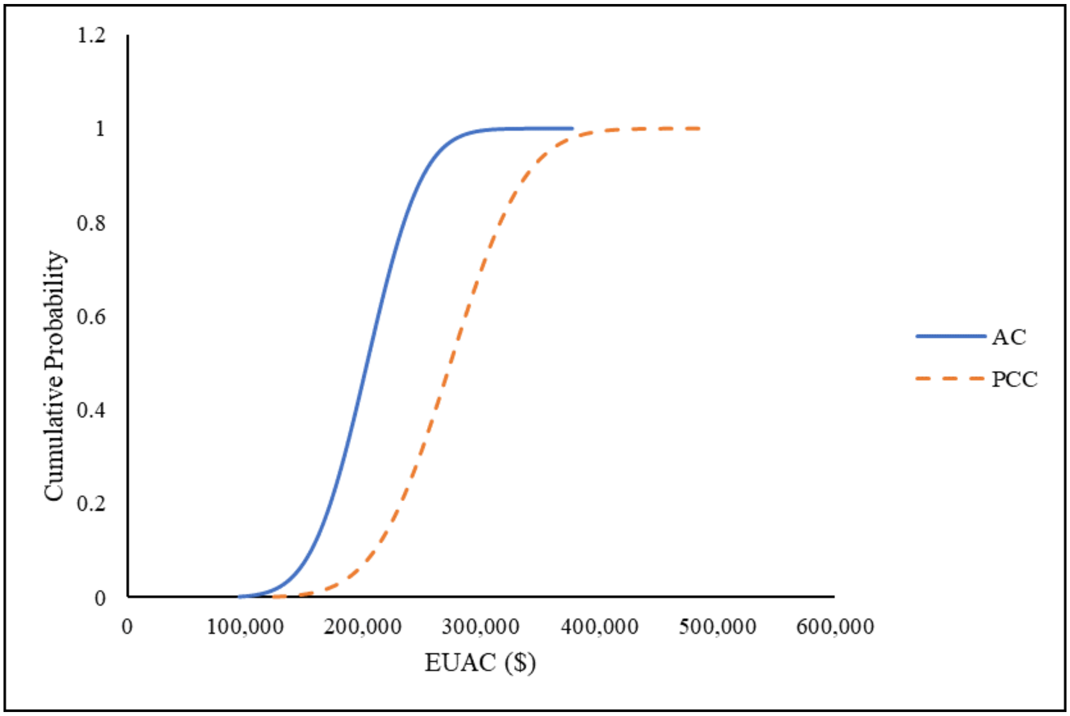


Figure B.16: Cumulative risk profile of the EUAC of the community cost (cost associated with noise pollution) for AC and PCC pavements

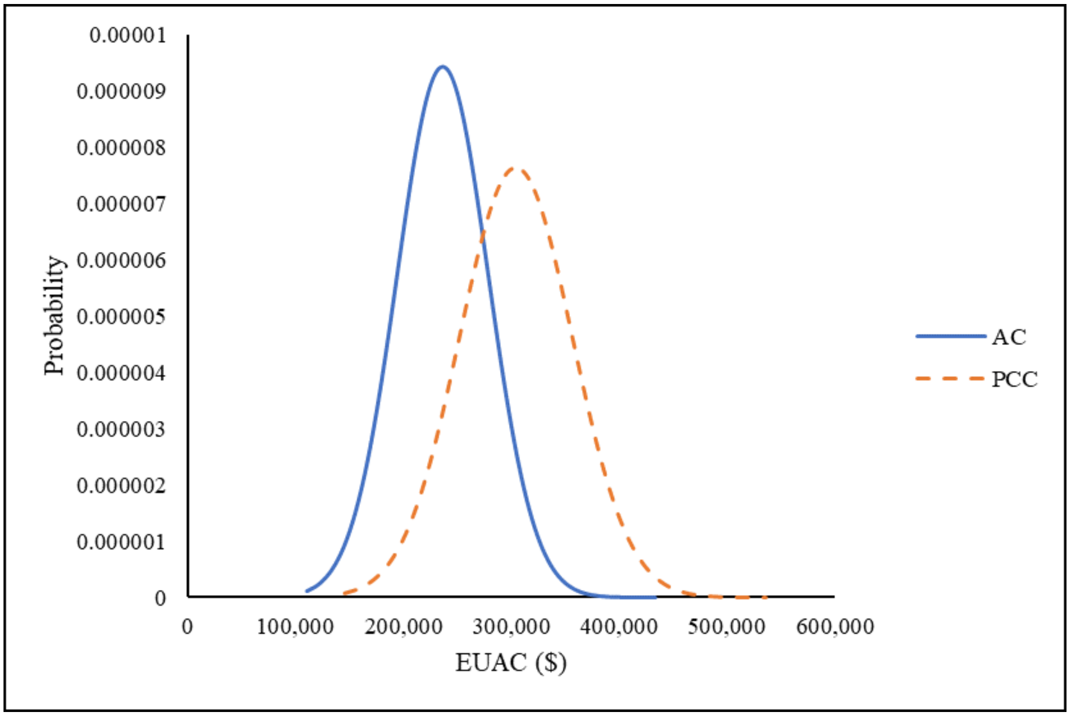


Figure B.17: Comparative EUAC probability distribution of the community cost for AC and PCC pavements

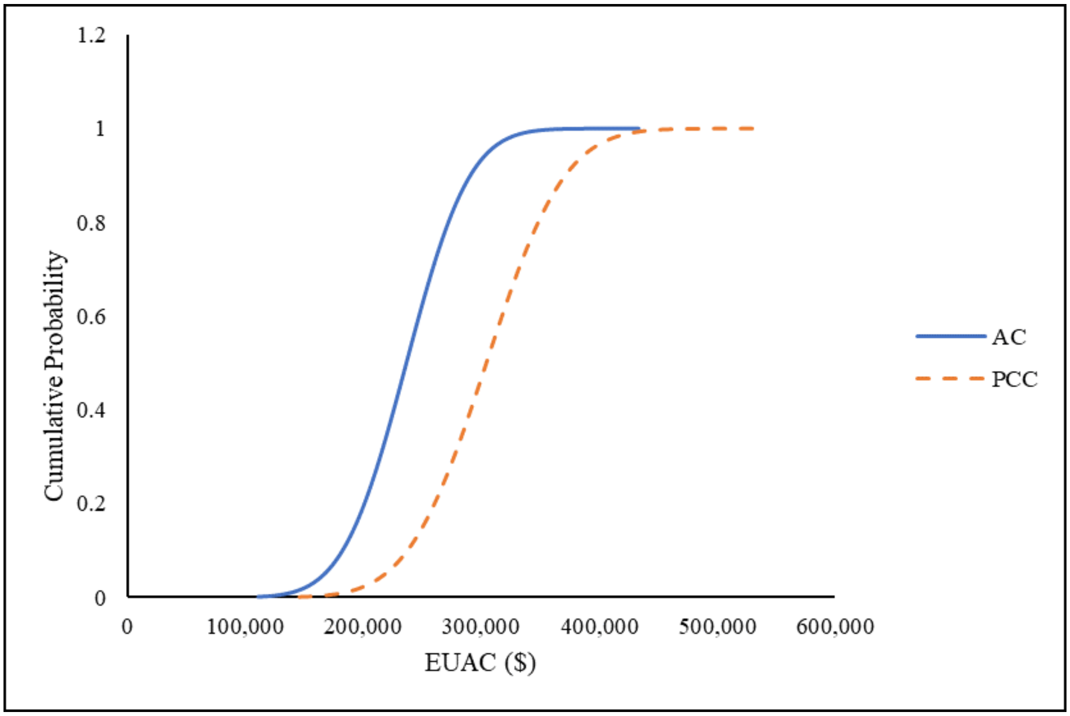


Figure B.18: Cumulative risk profile of the EUAC of the community cost for AC and PCC pavements

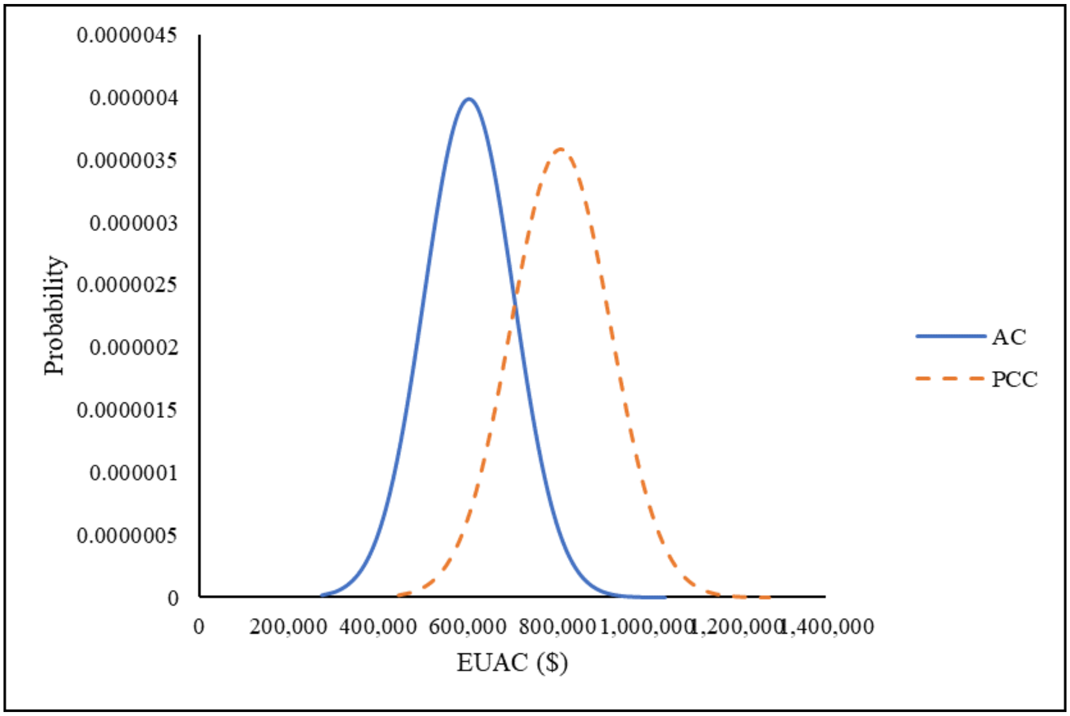


Figure B.19: Comparative EUAC probability distribution of the total cost for AC and PCC pavements

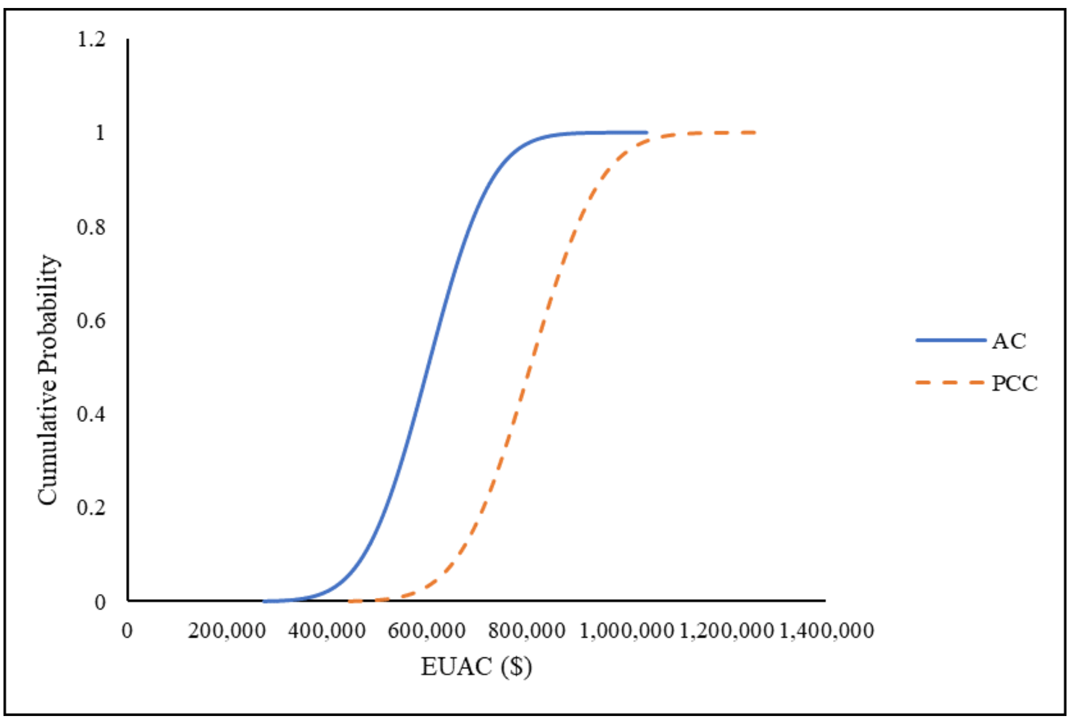


Figure B.20: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements

Wet-Non-Freeze Zone

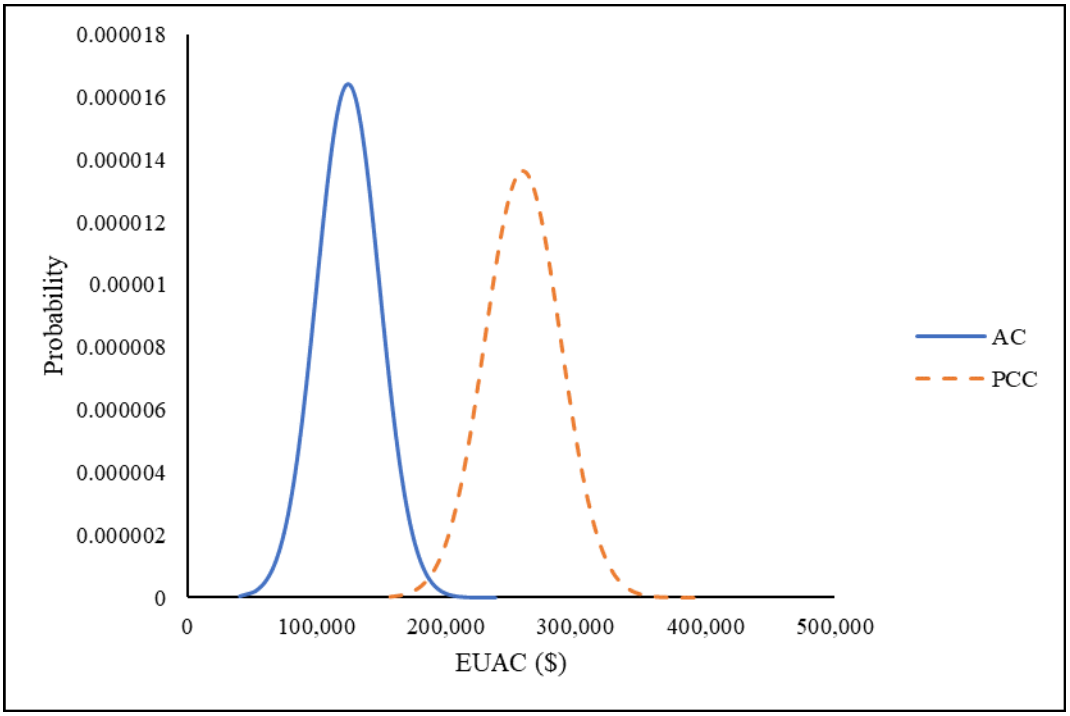


Figure B.21: Comparative EUAC probability distribution of the agency cost for AC and PCC pavements

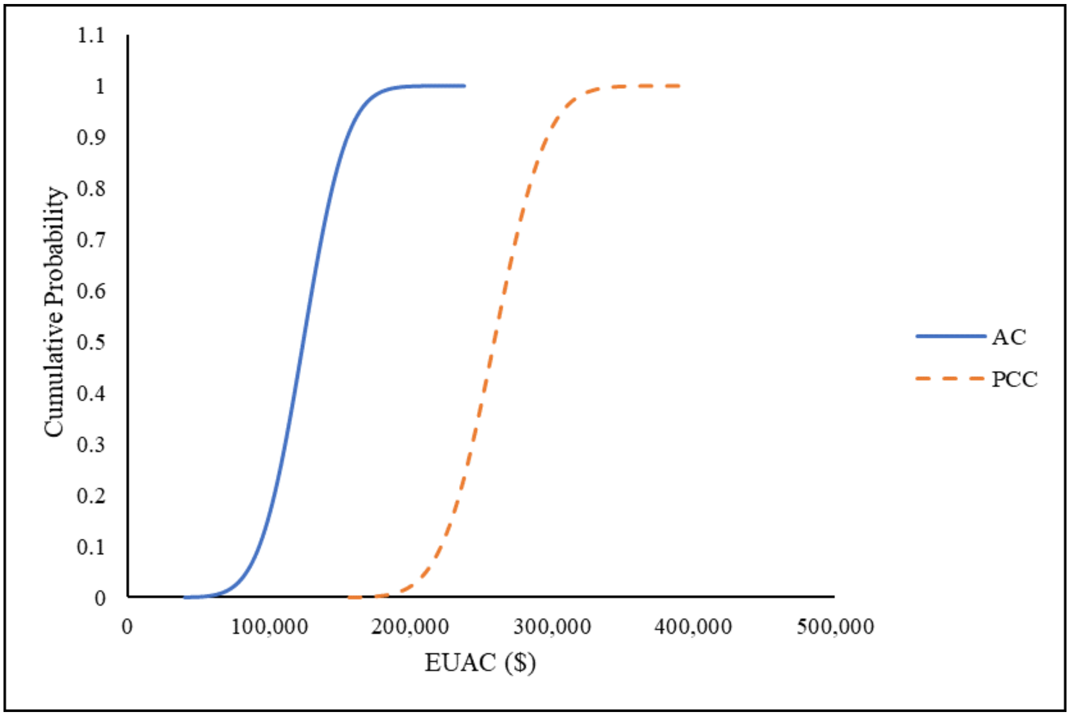


Figure B.22: Cumulative risk profile of the EUAC of the agency cost for AC and PCC pavements

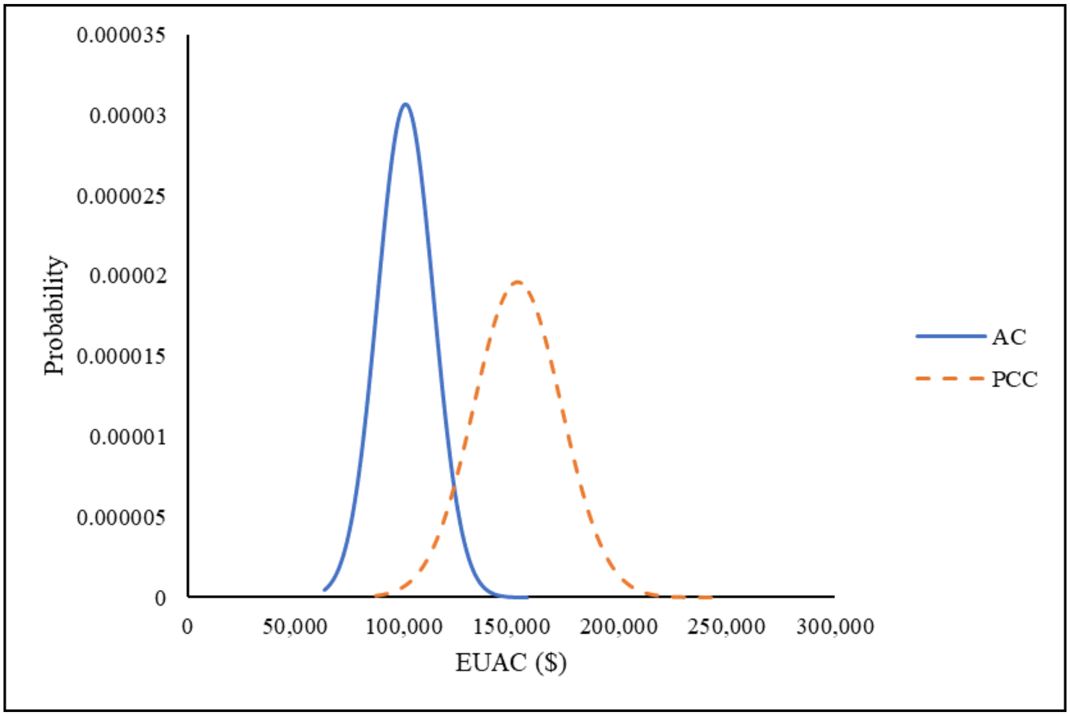


Figure B.23: Comparative EUAC probability distribution of the user cost (travel time delay cost) for AC and PCC pavements

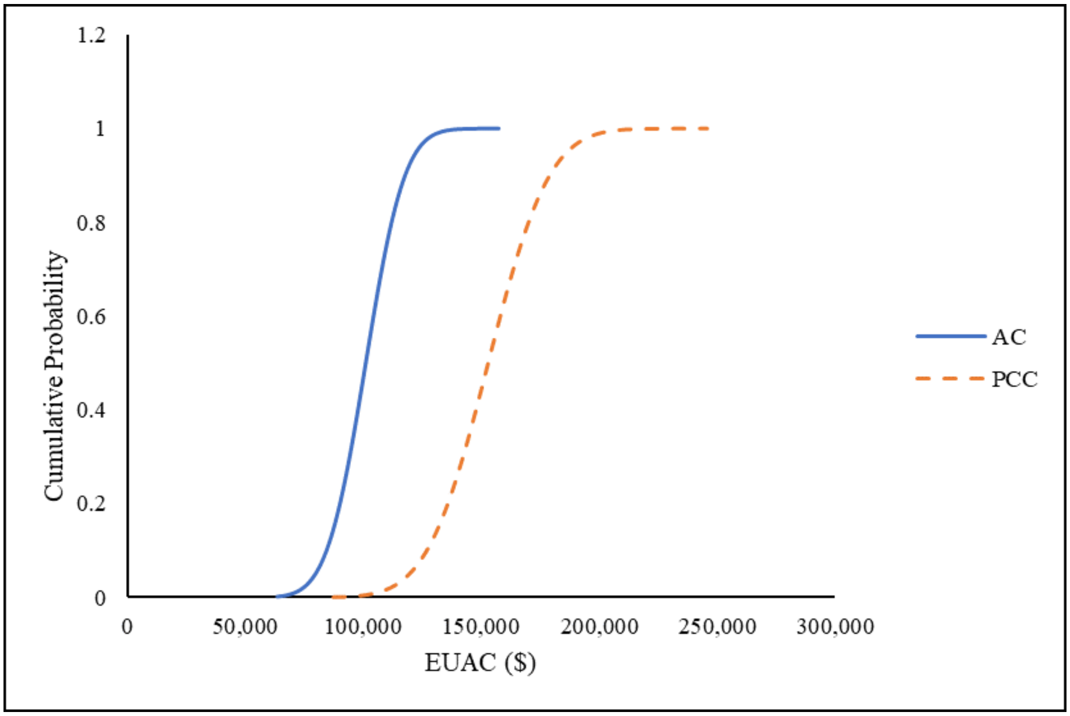


Figure B.24: Cumulative risk profile of the EUAC of the user cost (travel time delay cost) for AC and PCC pavements

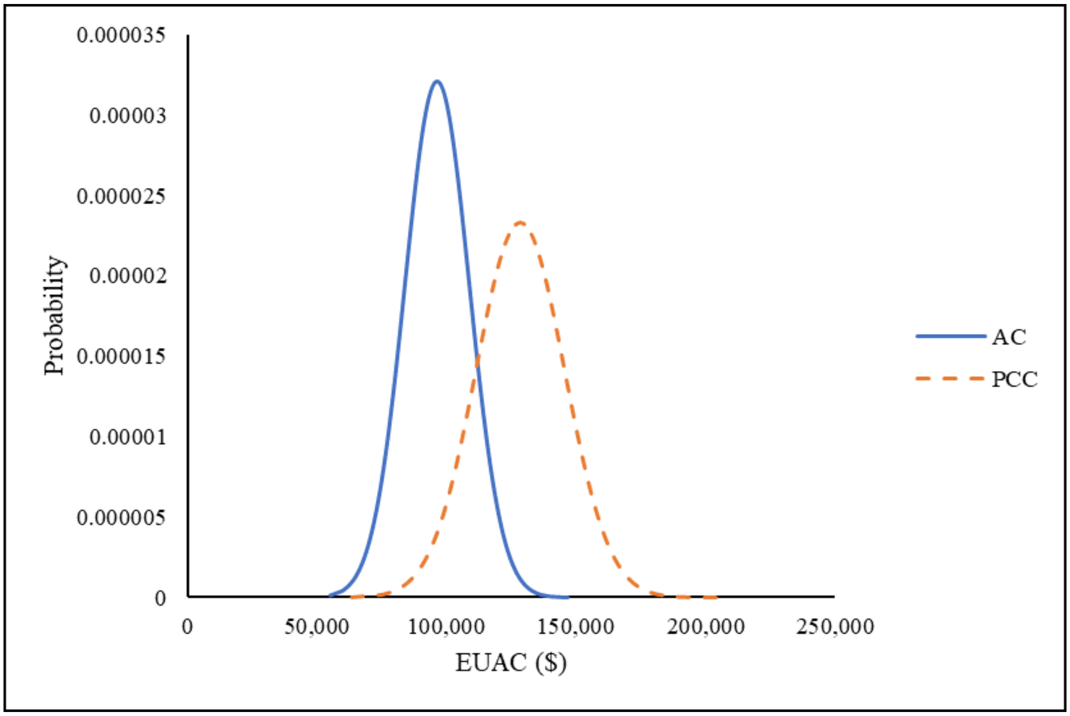


Figure B.25: Comparative EUAC probability distribution of the user cost (VOC) for AC and PCC pavements

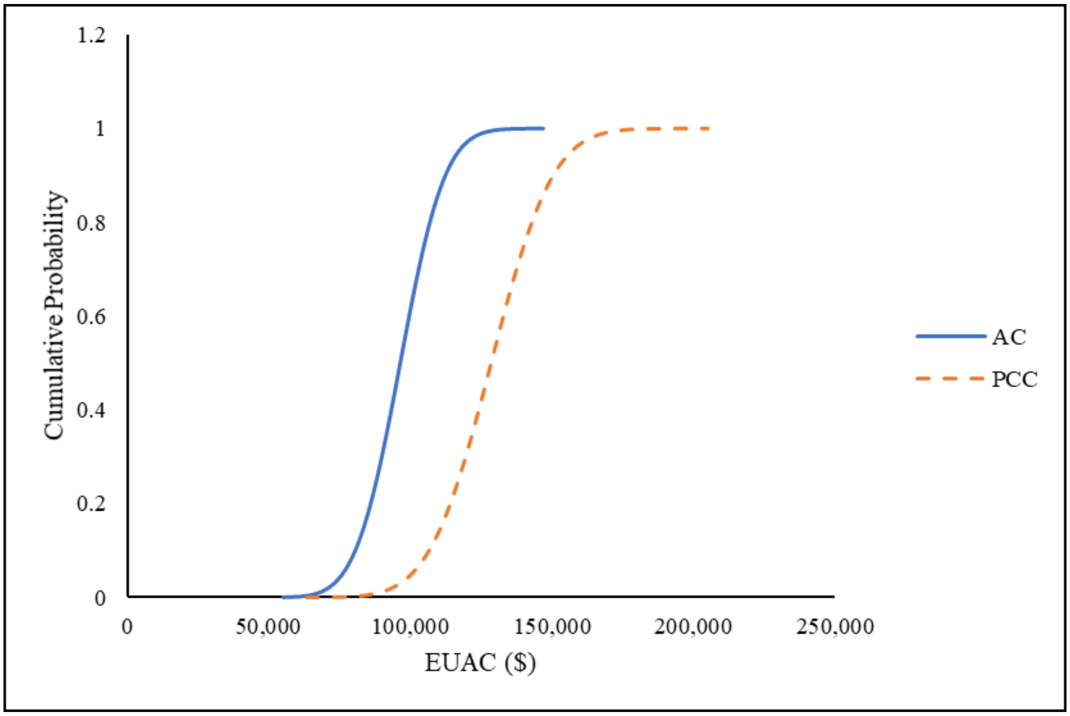


Figure B.26: Cumulative risk profile of the EUAC of the user cost (VOC) for AC and PCC pavements

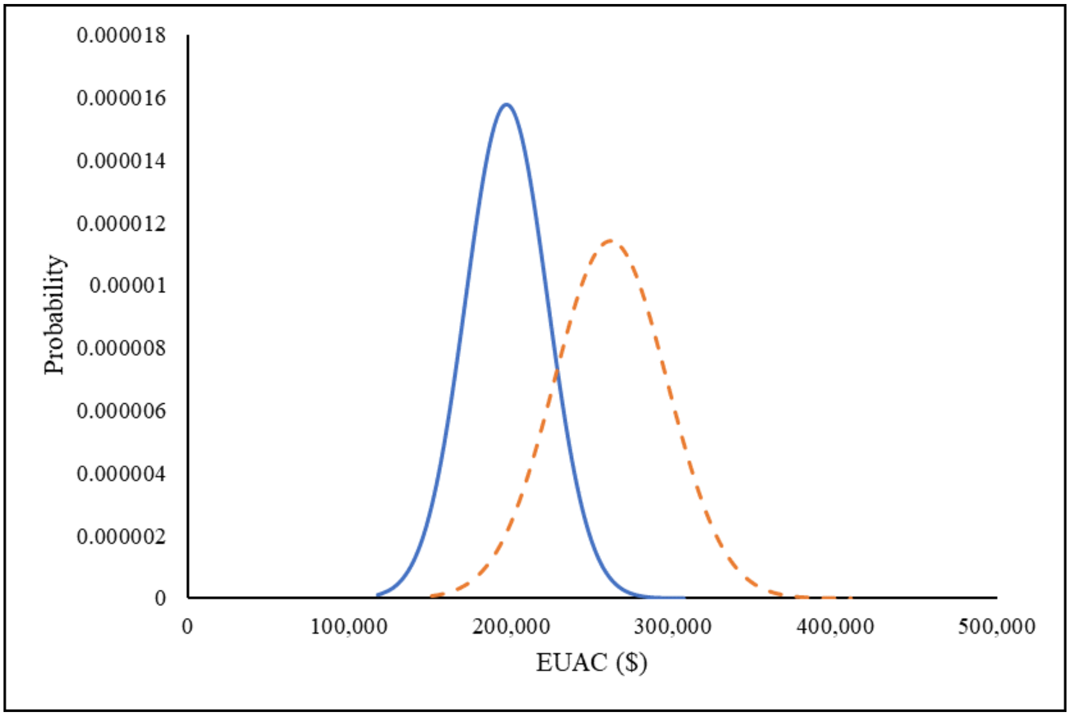


Figure B.27: Comparative EUAC probability distribution of the user cost for AC and PCC pavements

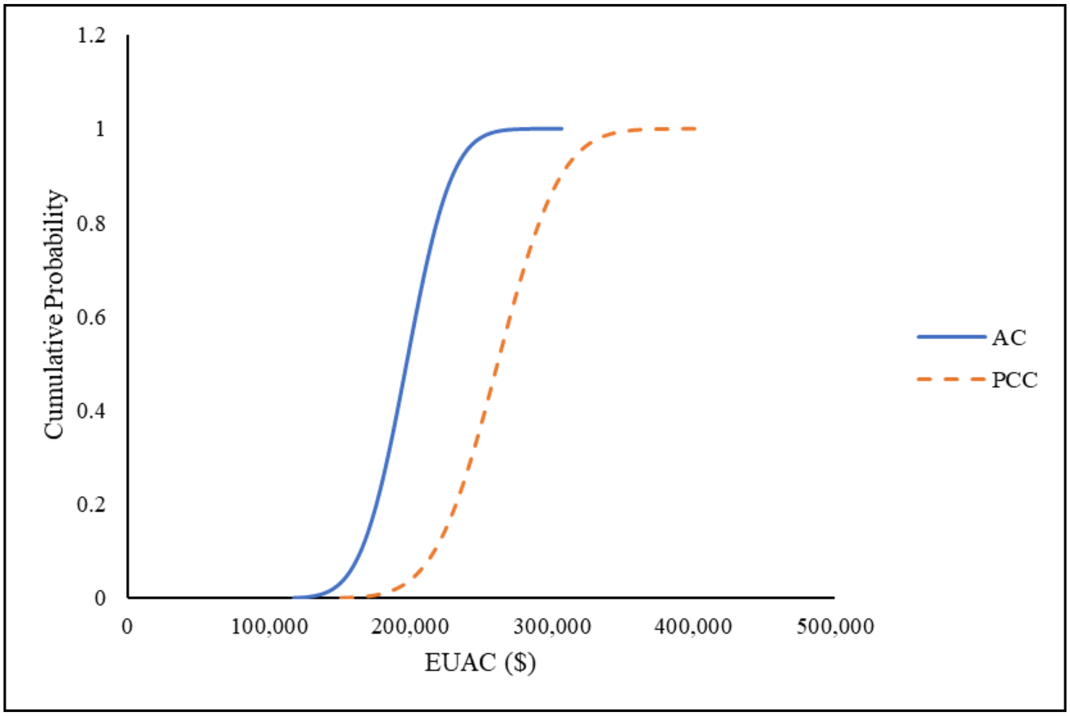


Figure B.28: Cumulative risk profile of the EUAC of the user cost for AC and PCC pavements

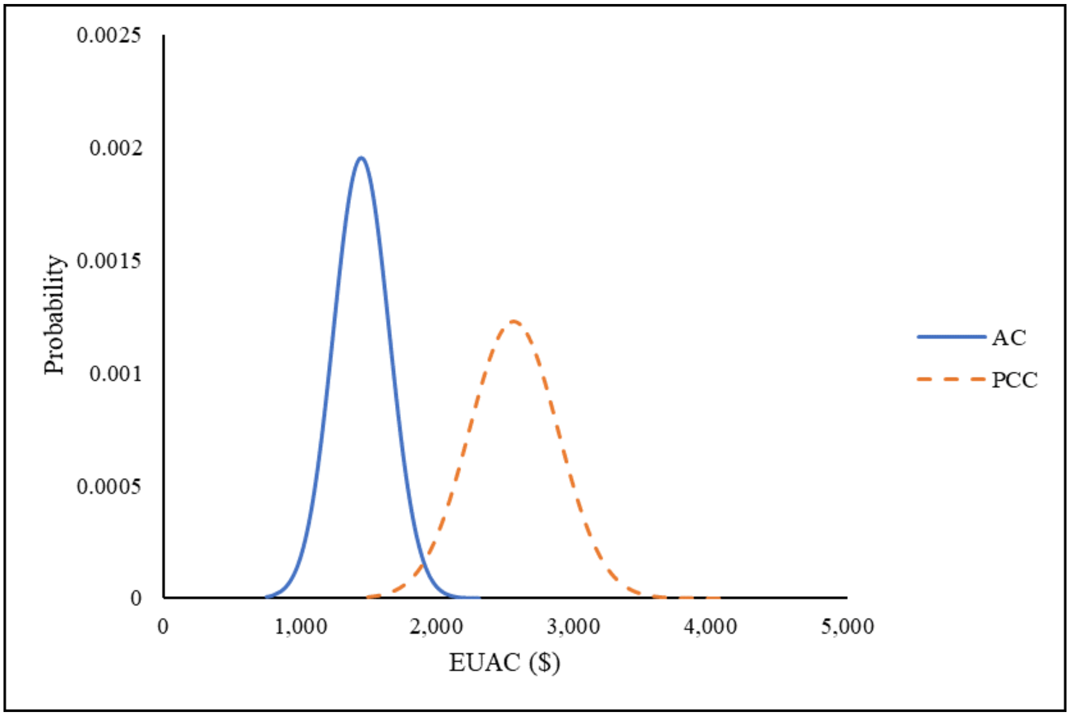


Figure B.29: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

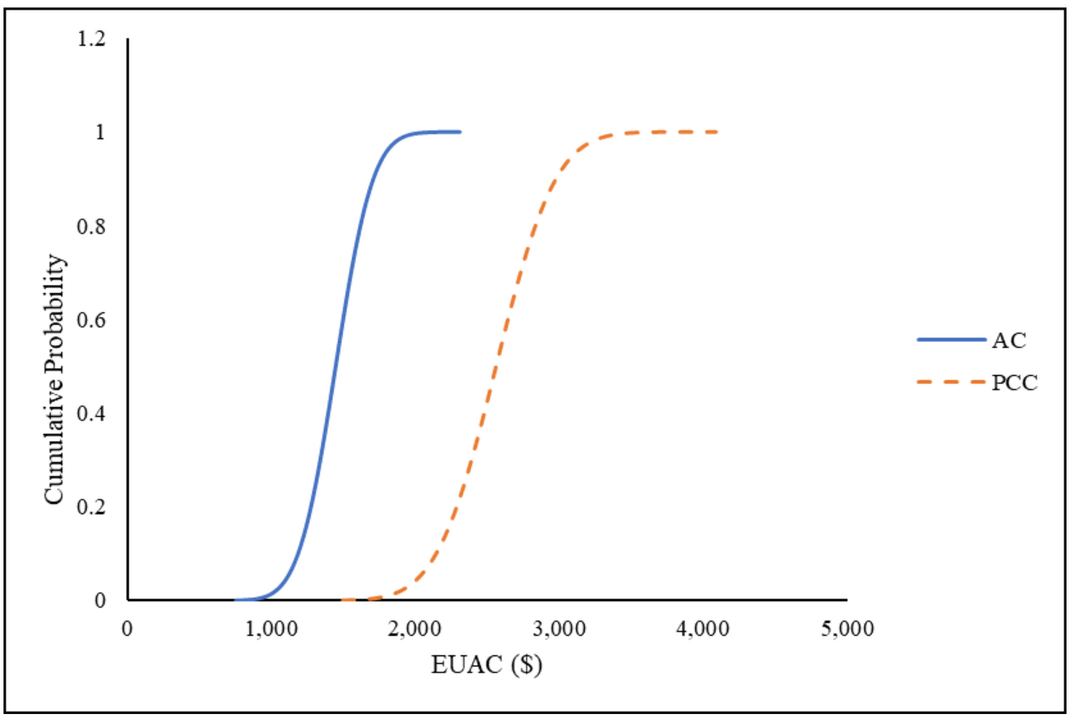


Figure B.30: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

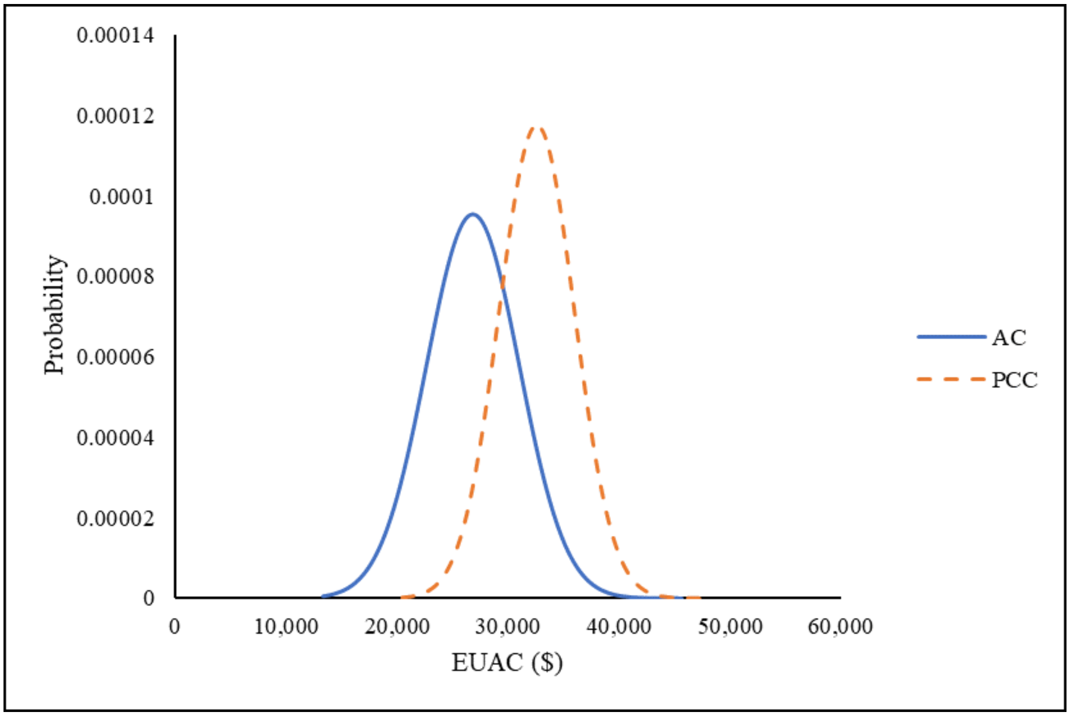


Figure B.31: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-energy consumption) for AC and PCC pavements

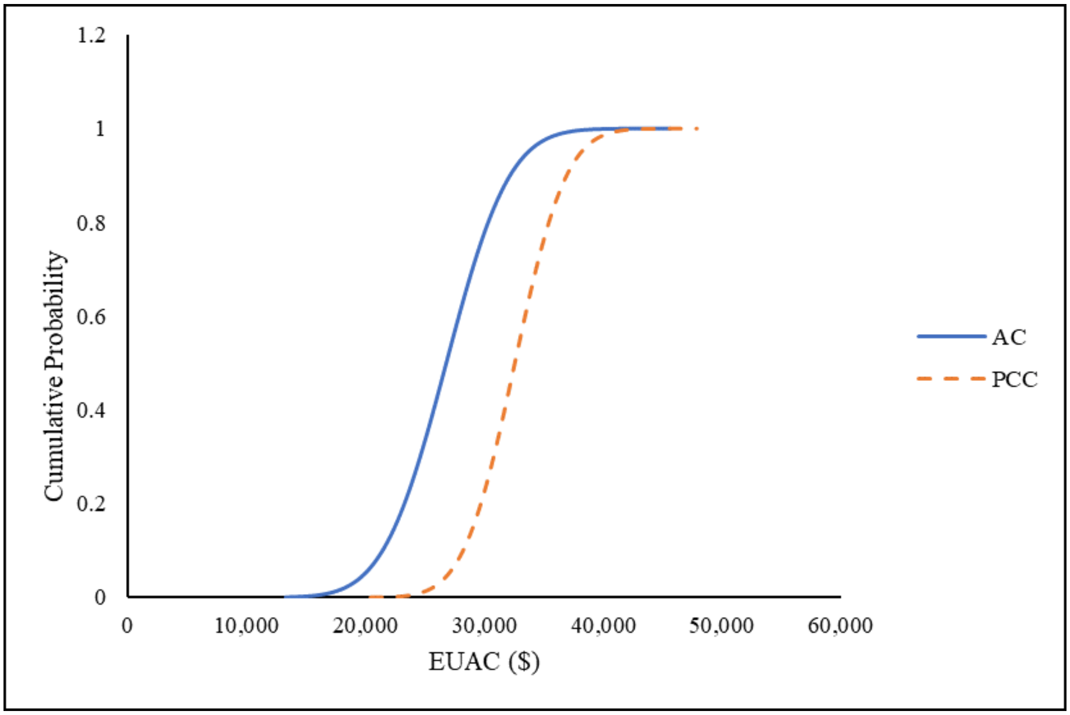


Figure B.32: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution- energy consumption) for AC and PCC pavements

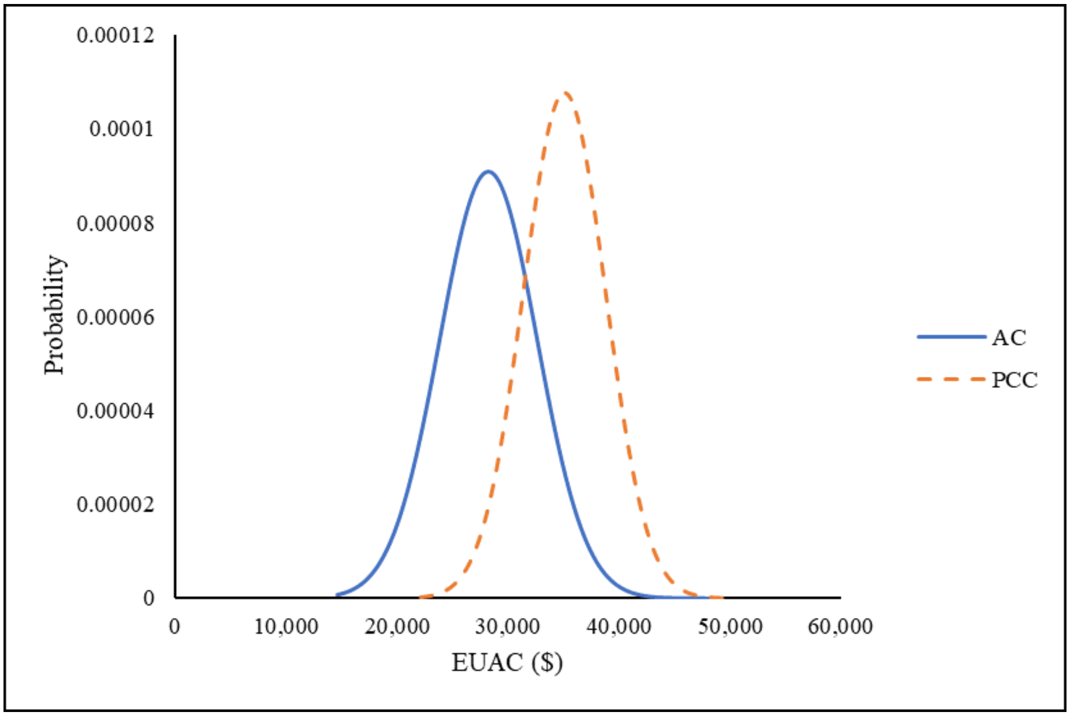


Figure B.33: Comparative EUAC probability distribution of the community cost (cost associated with air pollution) for AC and PCC pavements

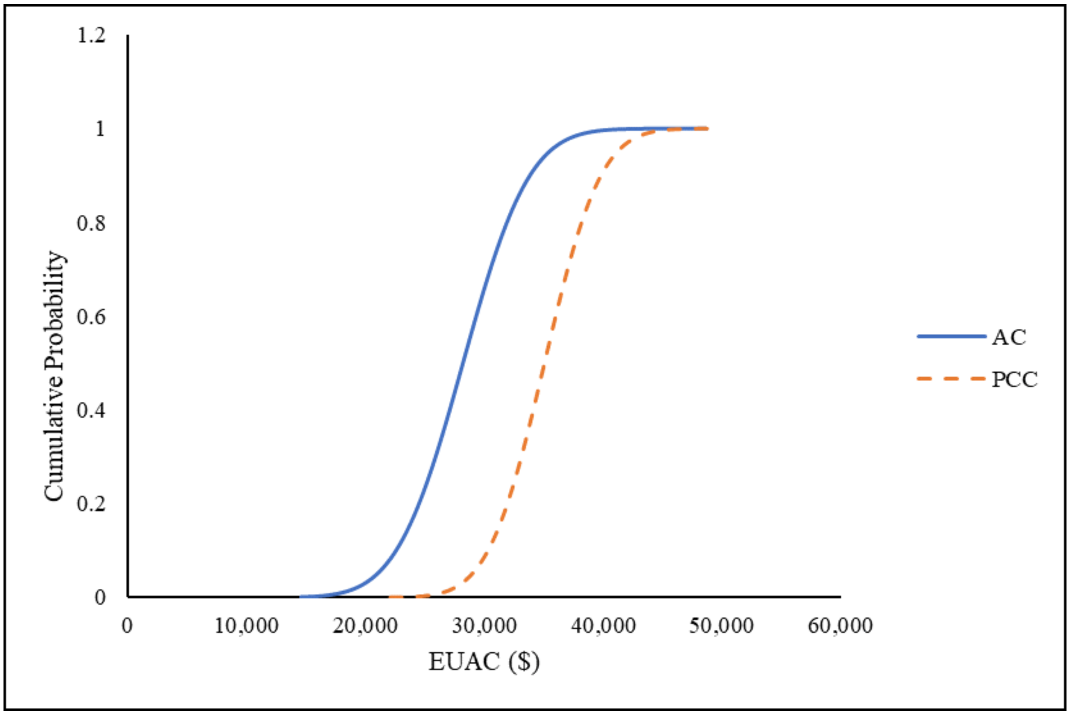


Figure B.34: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution) for AC and PCC pavements

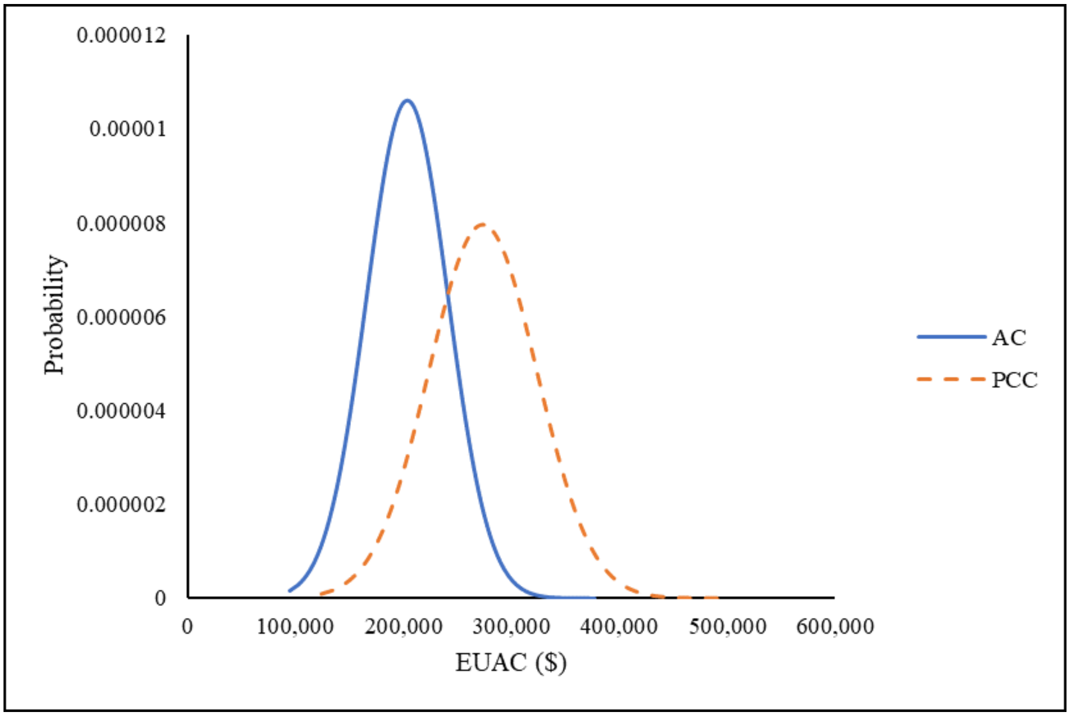


Figure B.35: Comparative EUAC probability distribution of the community cost (cost associated with noise pollution) for AC and PCC pavements

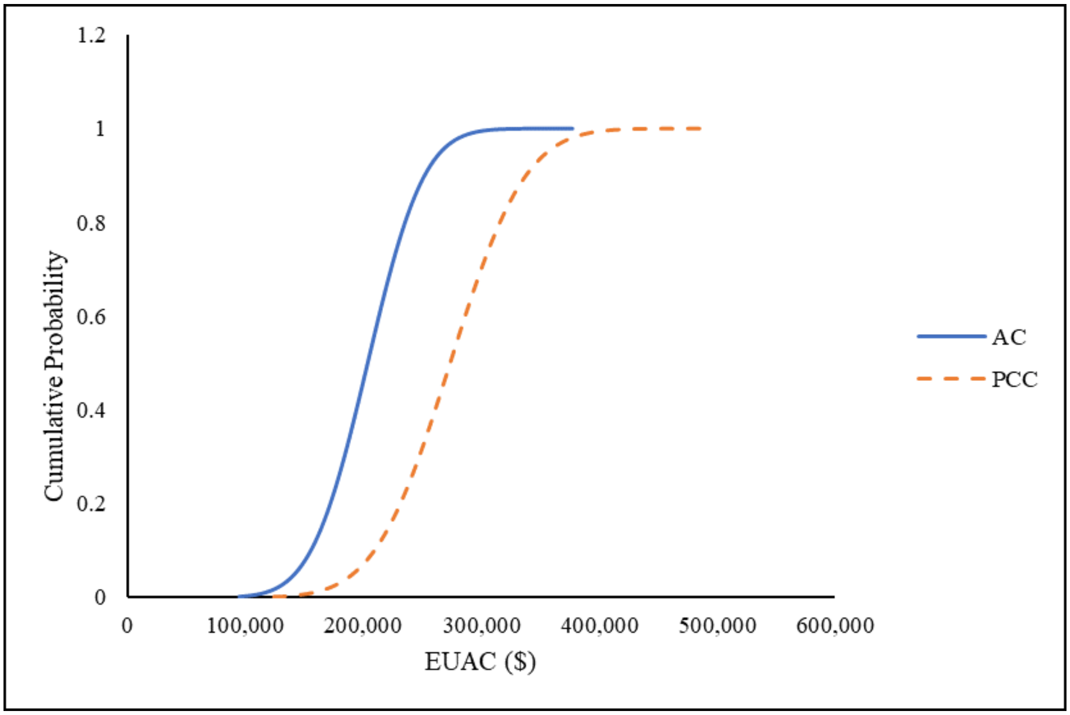


Figure B.36: Cumulative risk profile of the EUAC of the community cost (cost associated with noise pollution) for AC and PCC pavements

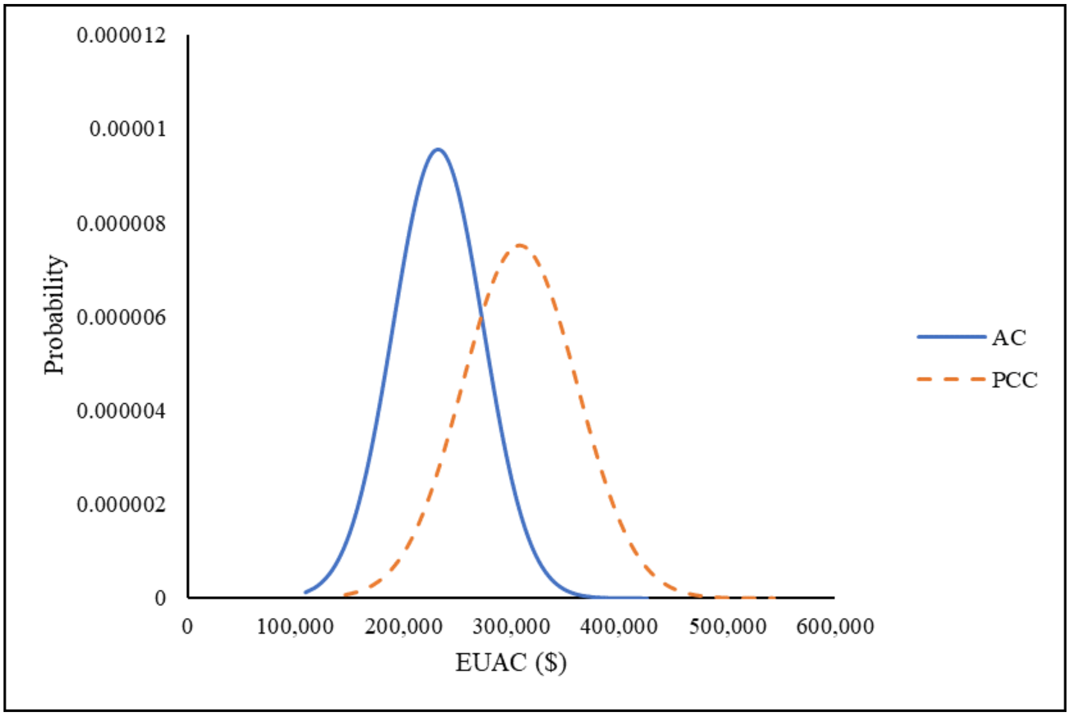


Figure B.37: Comparative EUAC probability distribution of the community cost for AC and PCC pavements

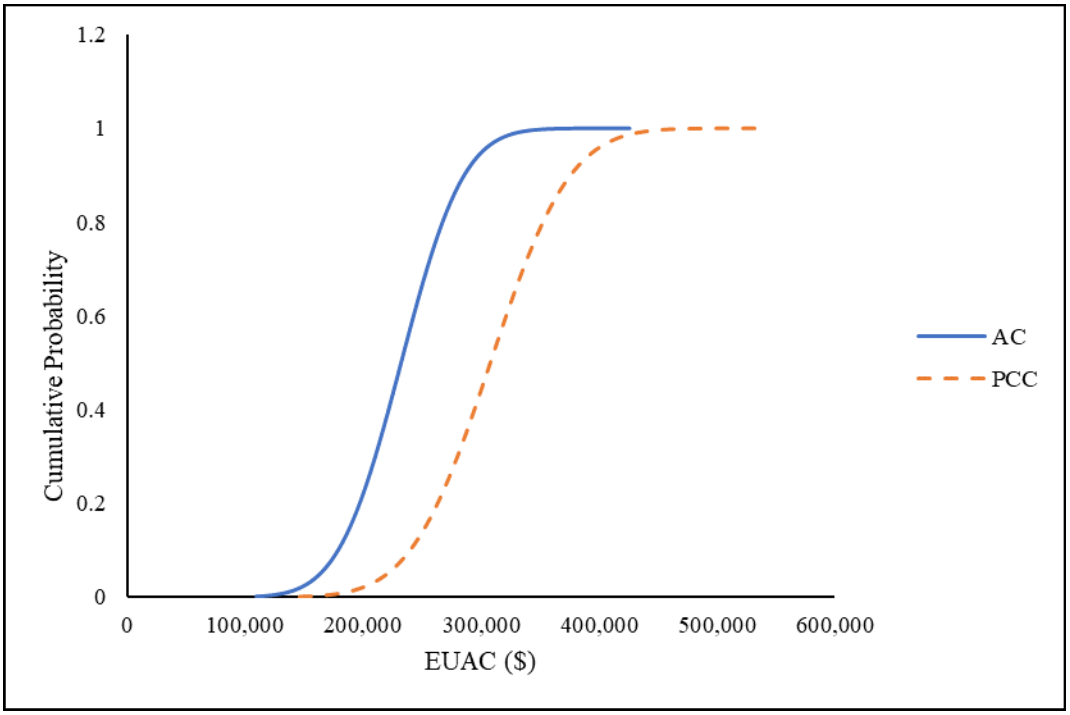


Figure B.38: Cumulative risk profile of the EUAC of the community cost for AC and PCC pavements

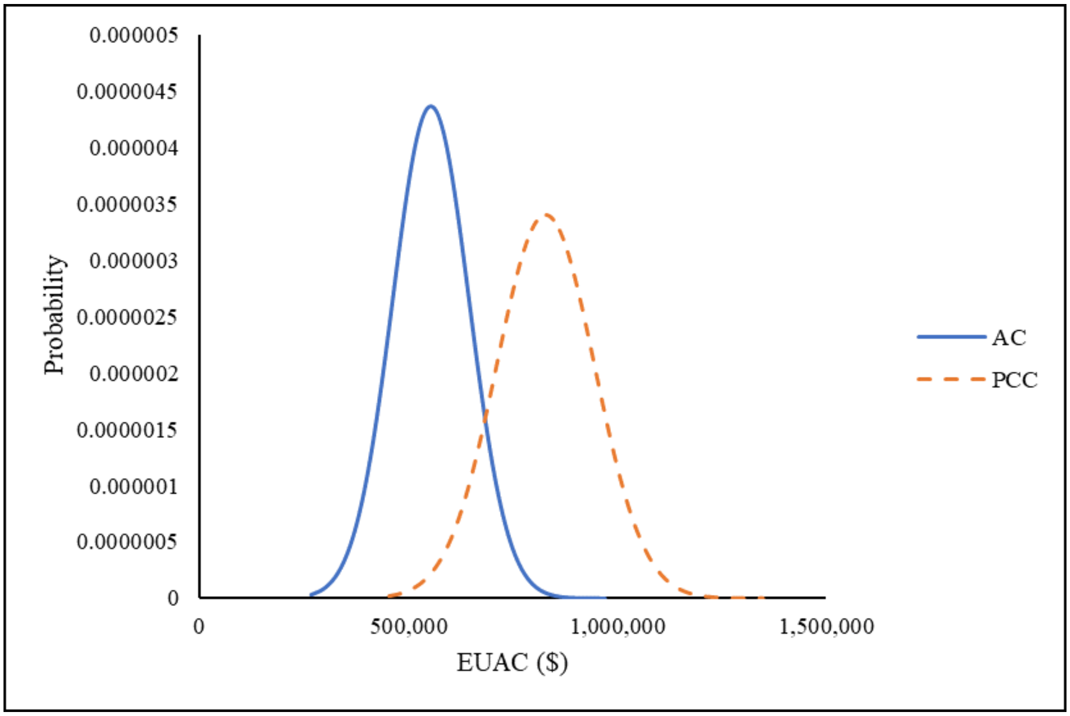


Figure B.39: Comparative EUAC probability distribution of the total cost for AC and PCC pavements

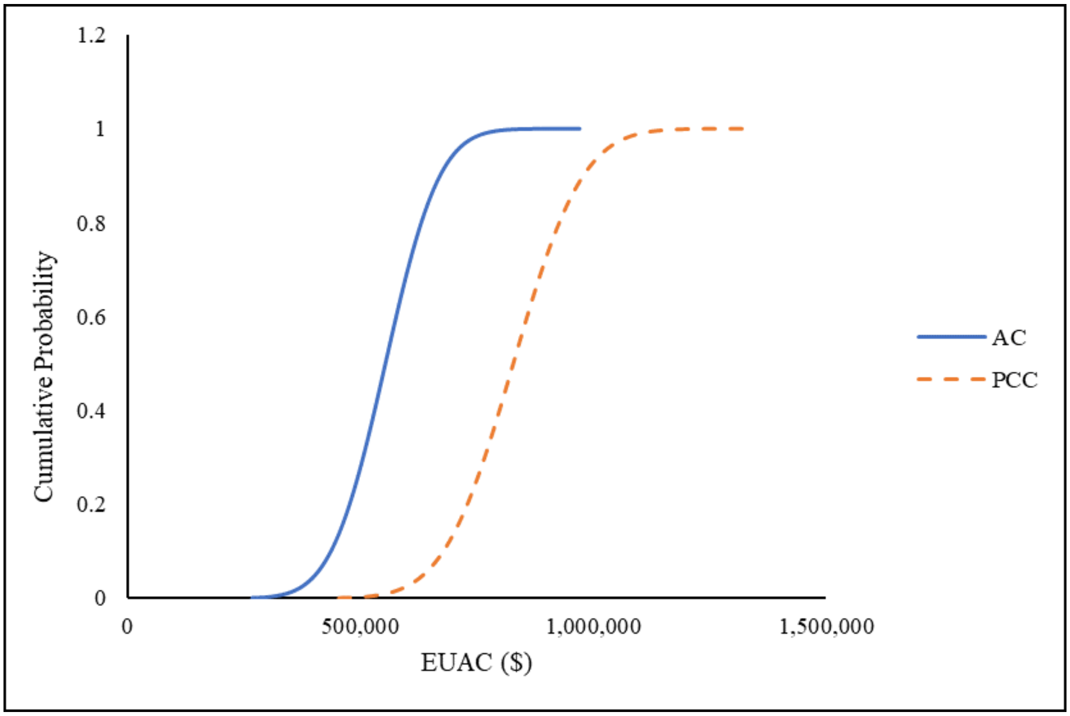


Figure B.40: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements

Dry-Freeze Zone

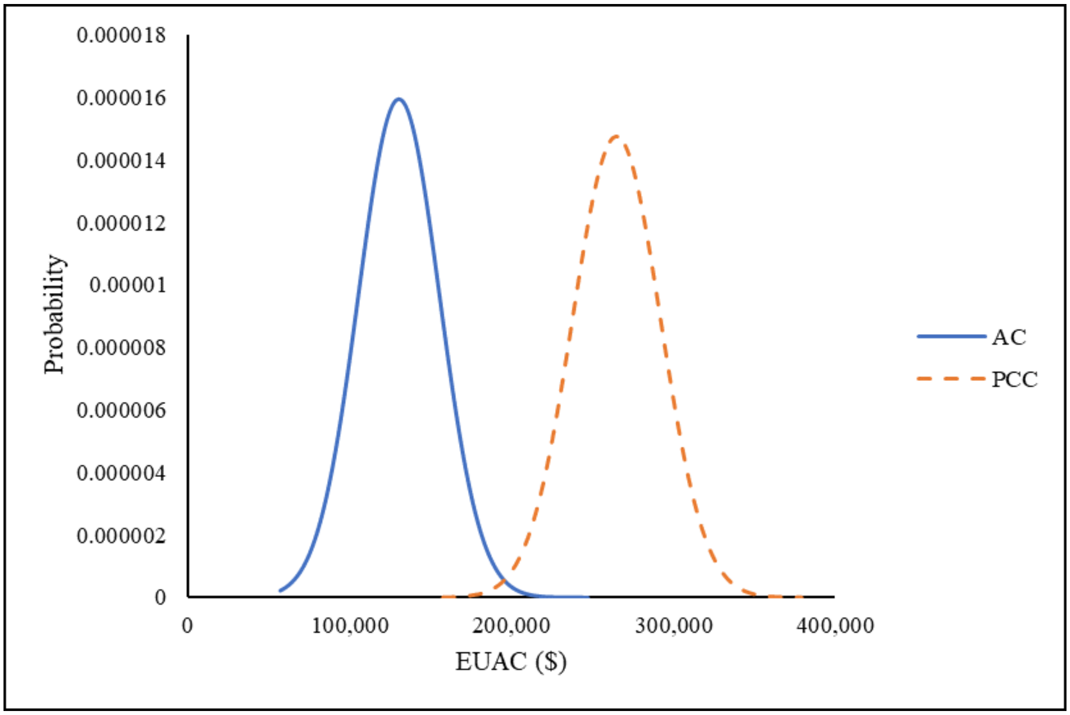


Figure B.41: Comparative EUAC probability distribution of the agency cost for AC and PCC pavements

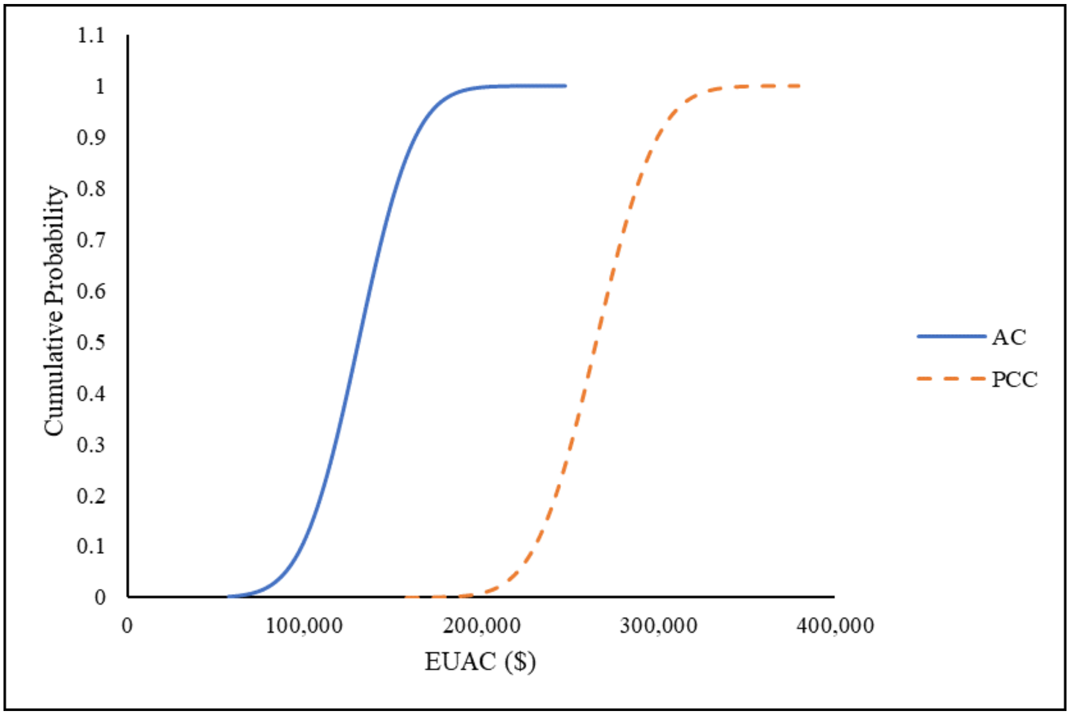


Figure B.42: Cumulative risk profile of the EUAC of the agency cost for AC and PCC pavements

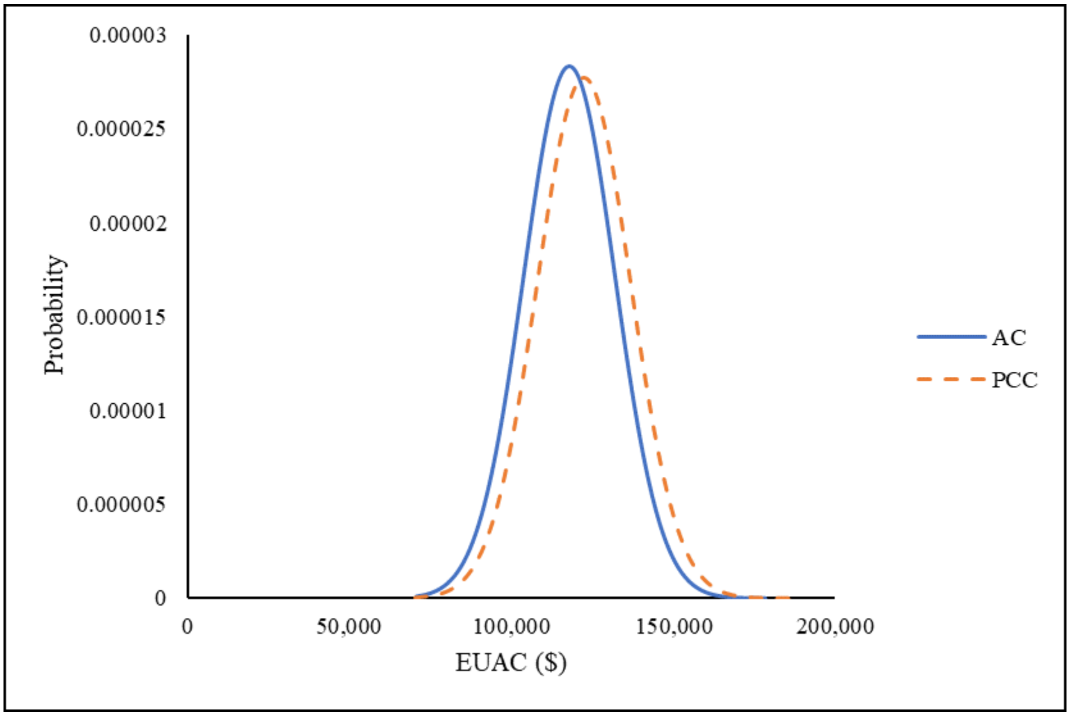


Figure B.43: Comparative EUAC probability distribution of the user cost (travel time delay cost) for AC and PCC pavements

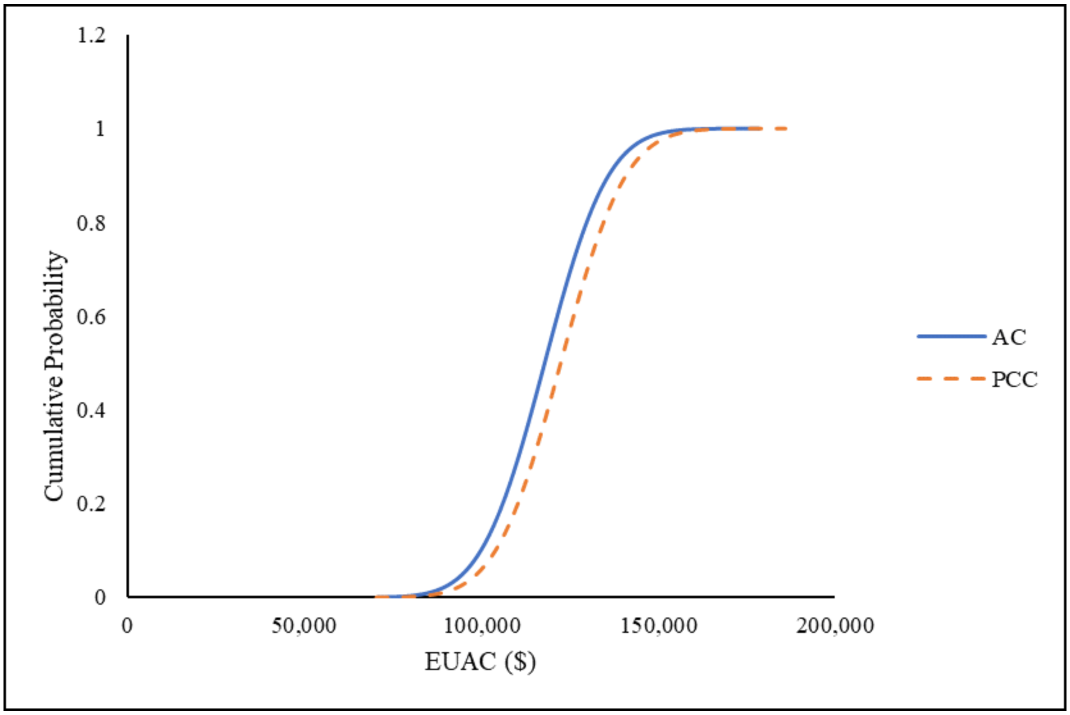


Figure B.44: Cumulative risk profile of the EUAC of the user cost (travel time delay cost) for AC and PCC pavements

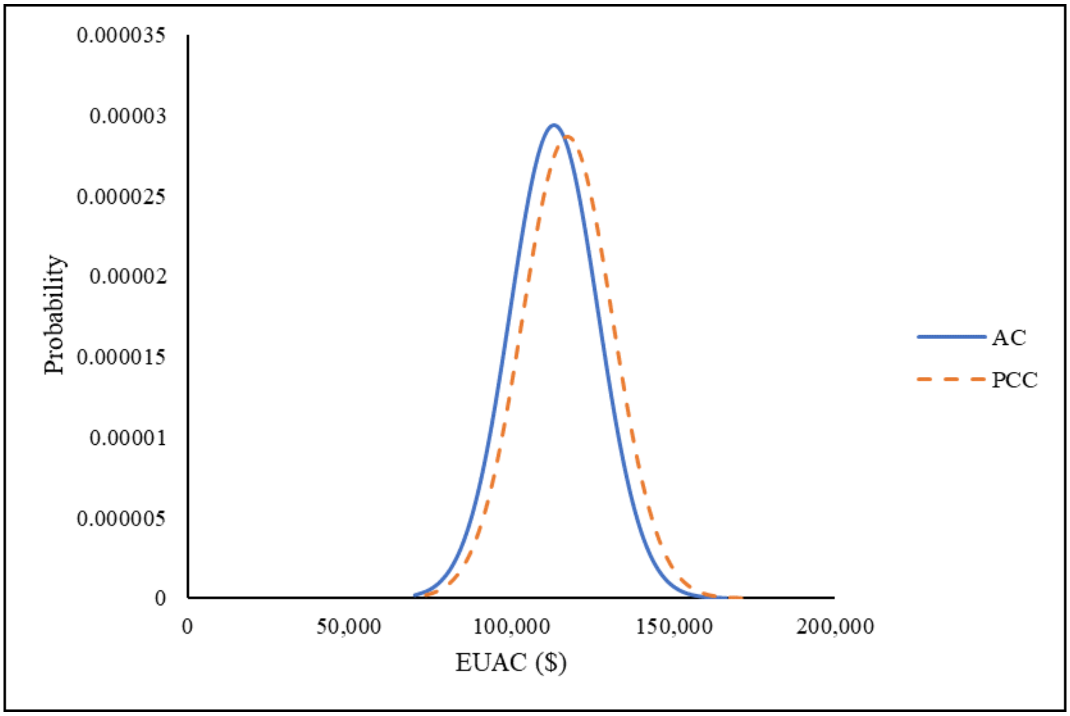


Figure B.45: Comparative EUAC probability distribution of the user cost (VOC) for AC and PCC pavements

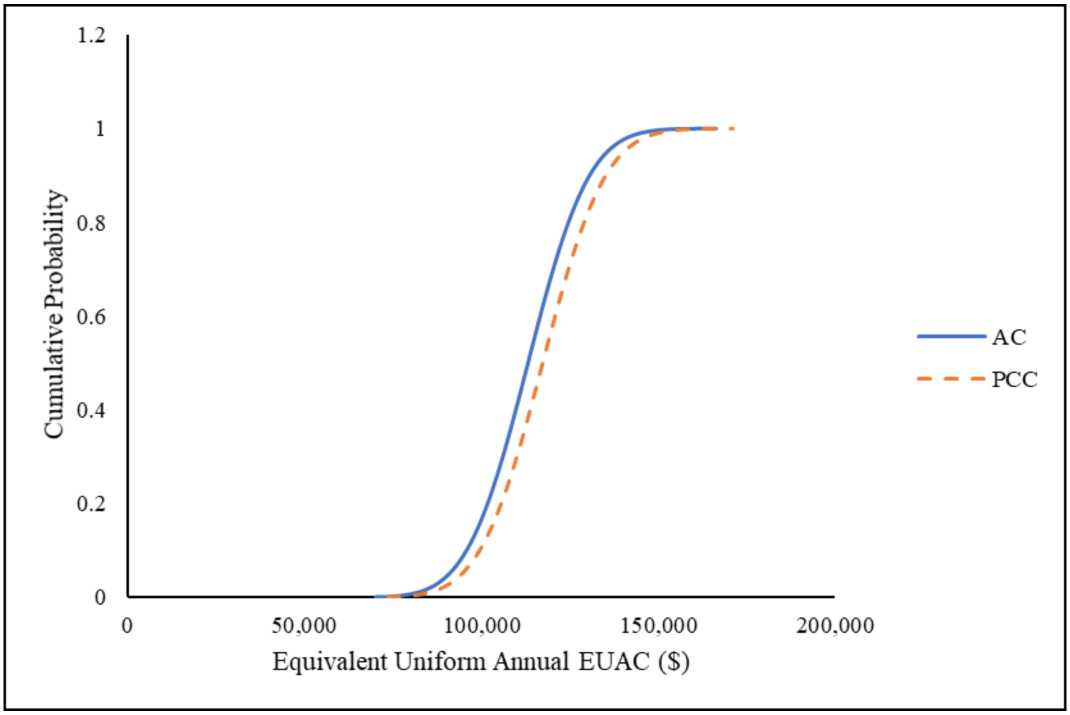


Figure B.46: Cumulative risk profile of the EUAC of the user cost (VOC) for AC and PCC pavements

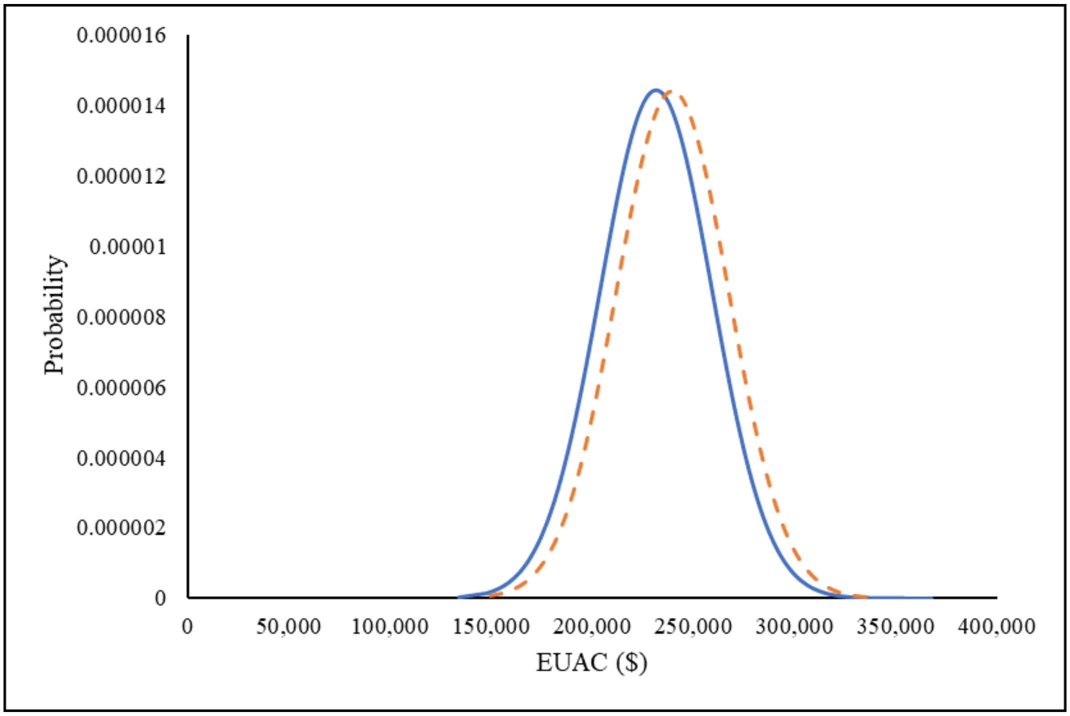


Figure B.47: Comparative EUAC probability distribution of the user cost for AC and PCC pavements

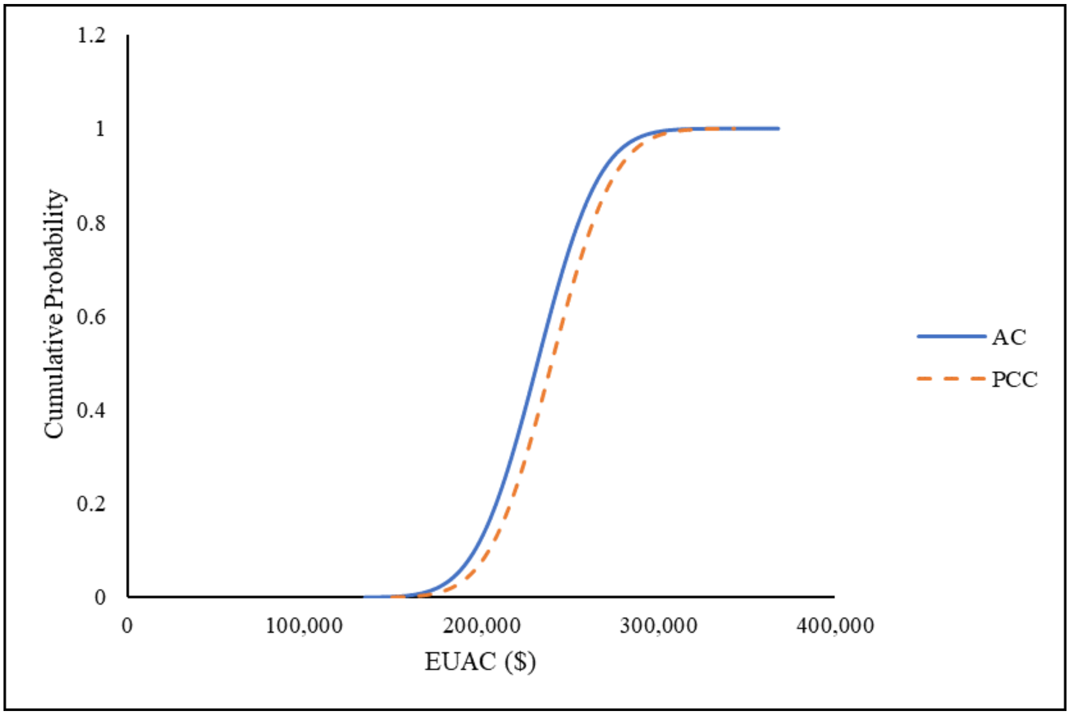


Figure B.48: Cumulative risk profile of the EUAC of the user cost for AC and PCC pavements

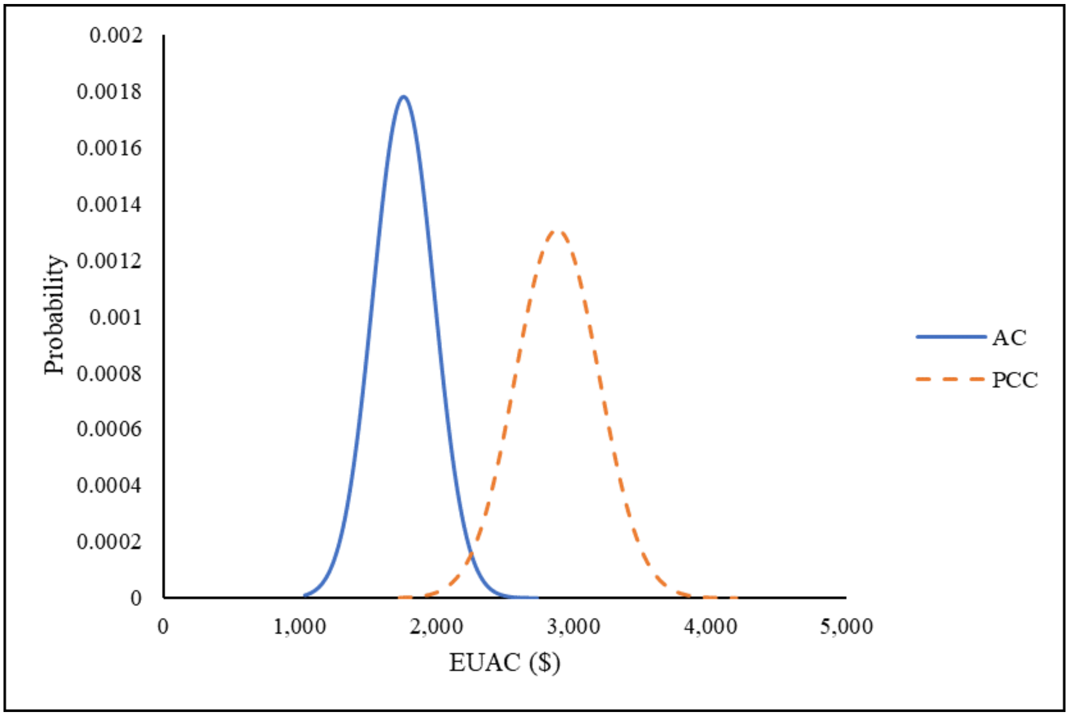


Figure B.49: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

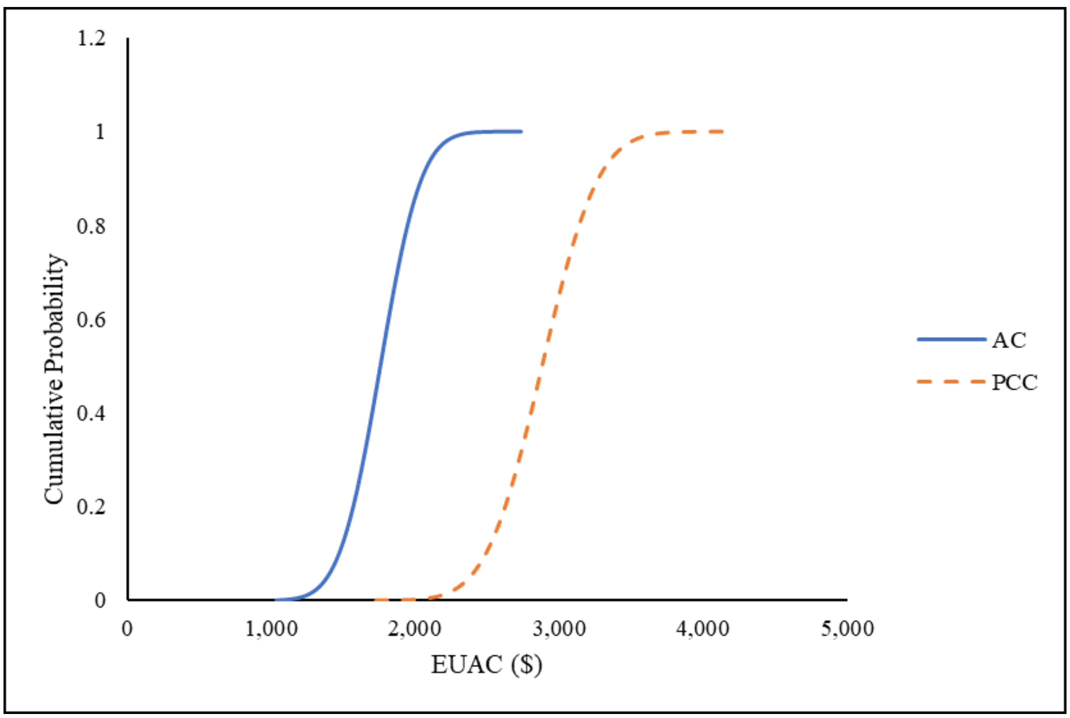


Figure B.50: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

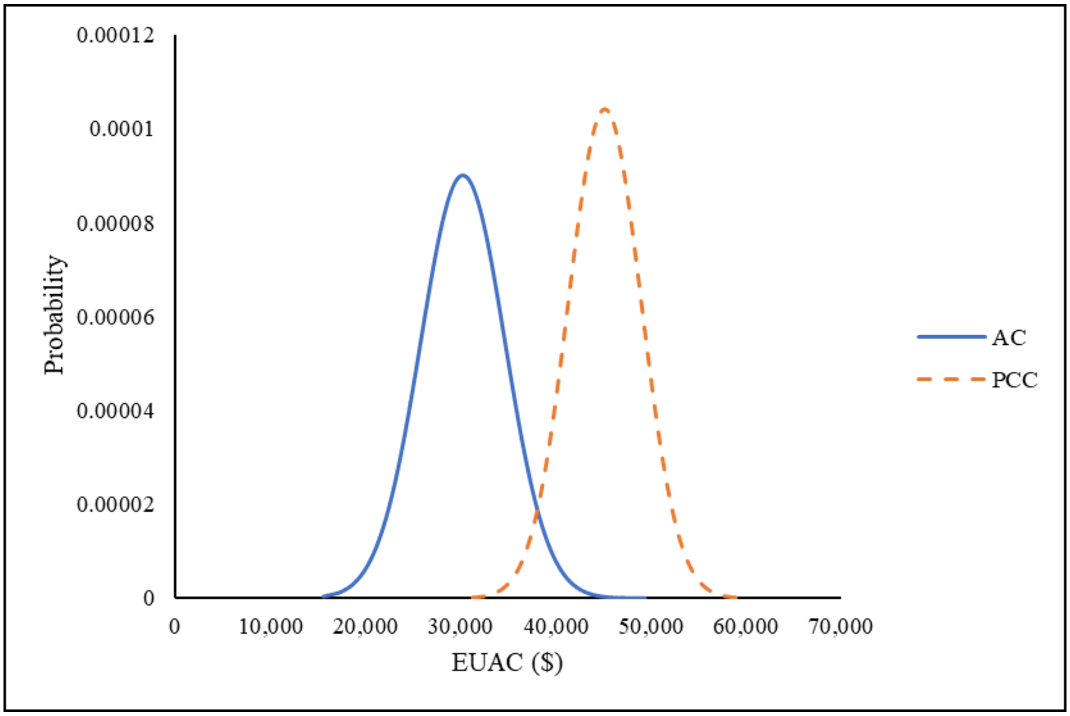


Figure B.51: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-energy consumption) for AC and PCC pavements

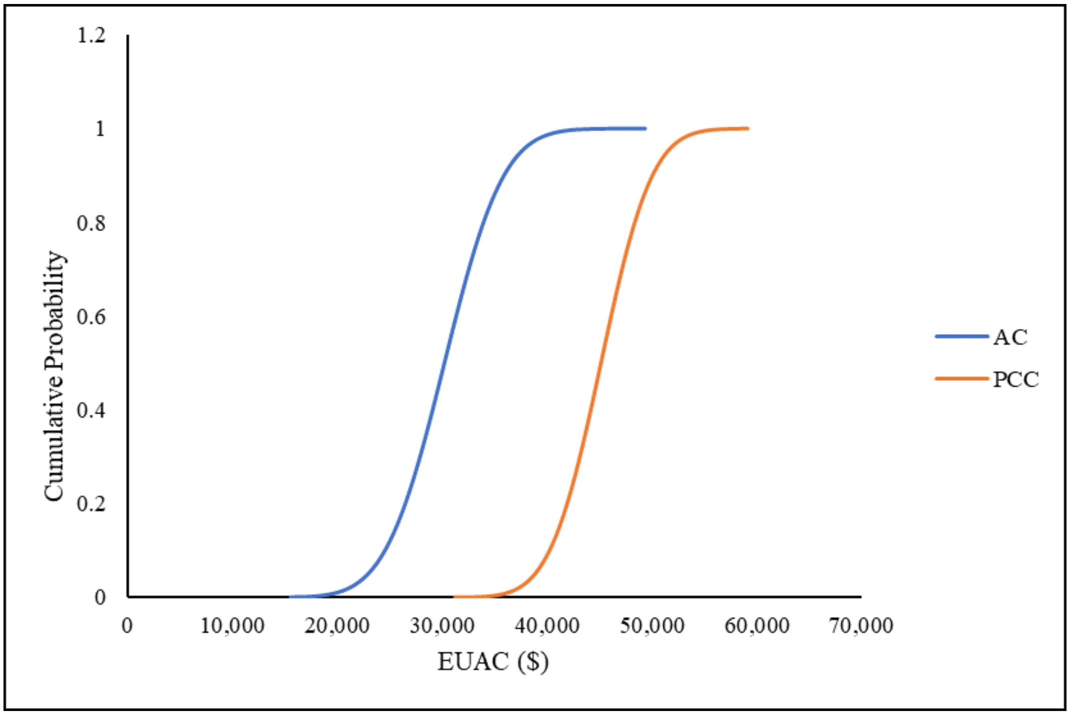


Figure B.52: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution- energy consumption) for AC and PCC pavements

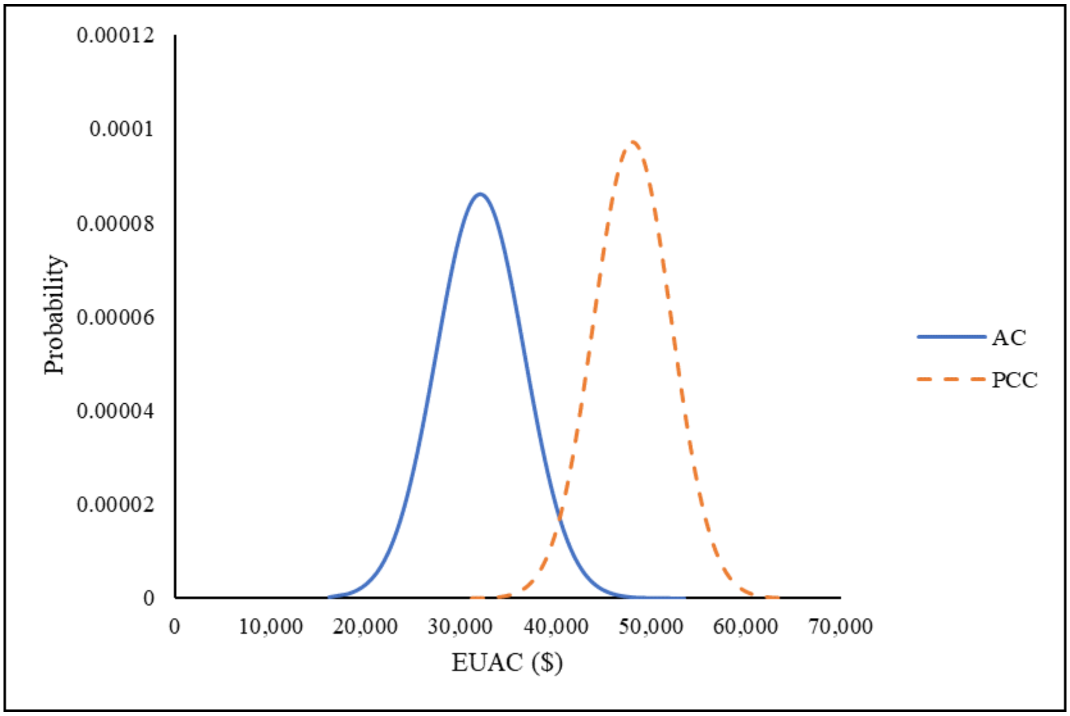


Figure B.53: Comparative EUAC probability distribution of the community cost (cost associated with air pollution) for AC and PCC pavements

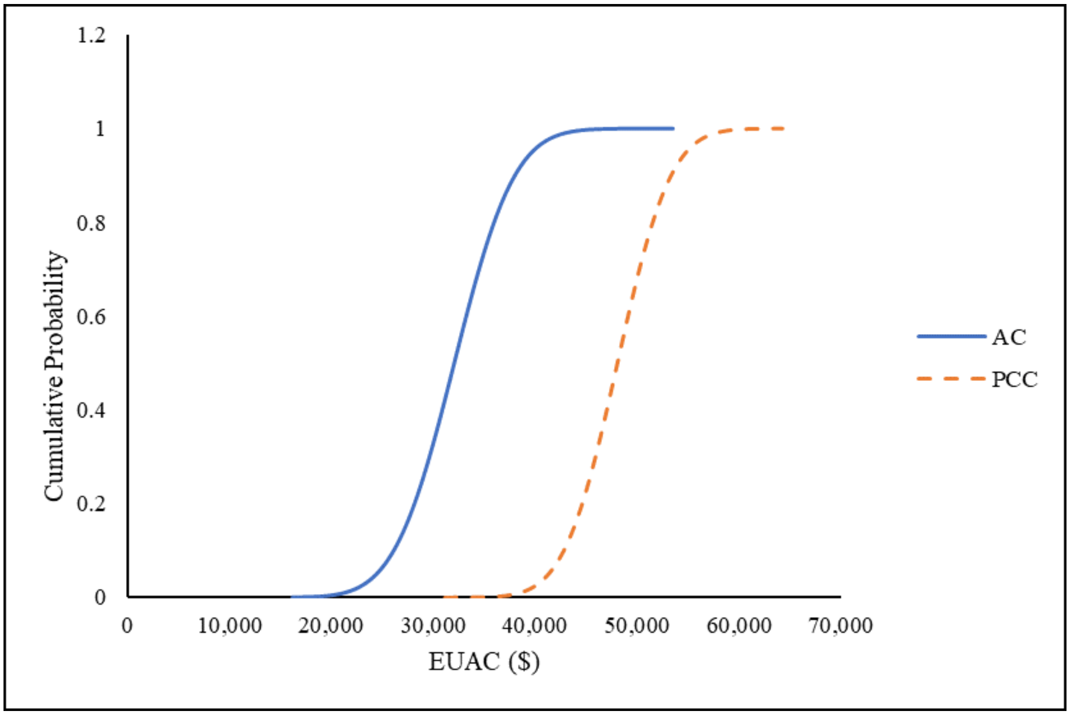


Figure B.54: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution) for AC and PCC pavements

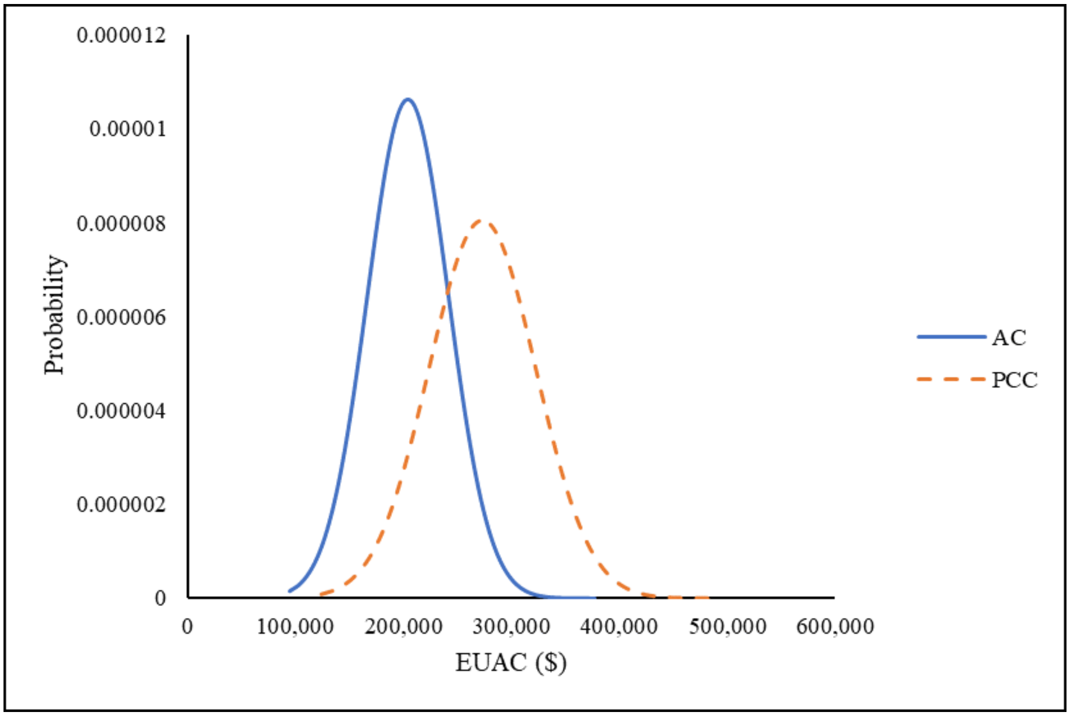


Figure B.55: Comparative EUAC probability distribution of the community cost (cost associated with noise pollution) for AC and PCC pavements

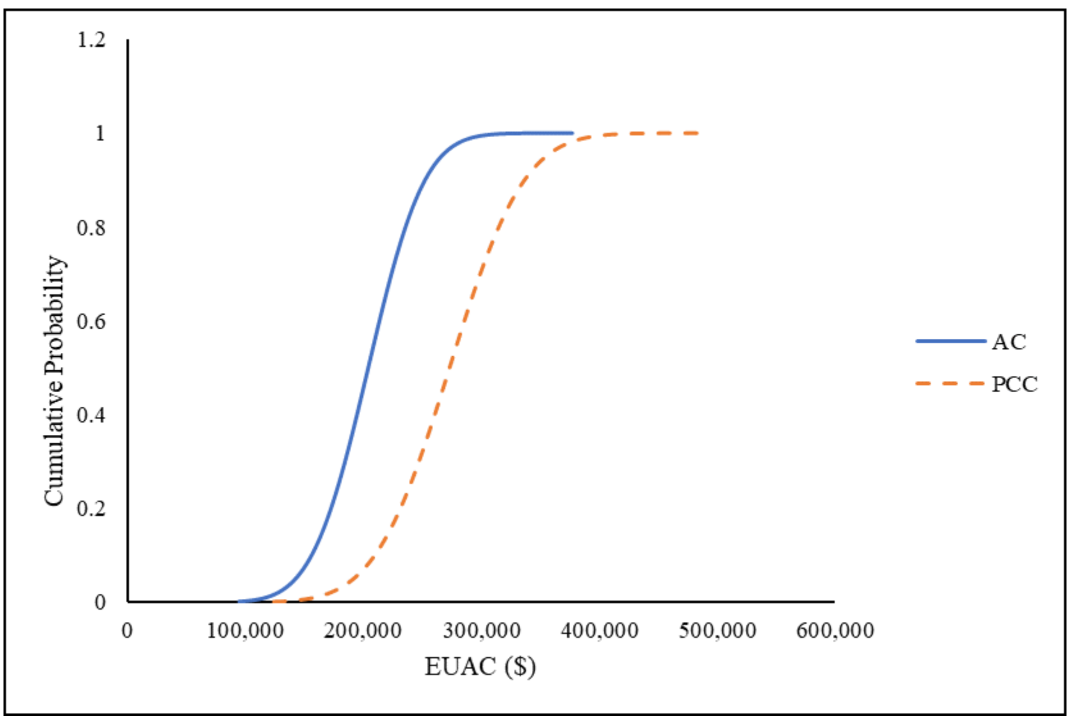


Figure B.56: Cumulative risk profile of the EUAC of the community cost (cost associated with noise pollution) for AC and PCC pavements

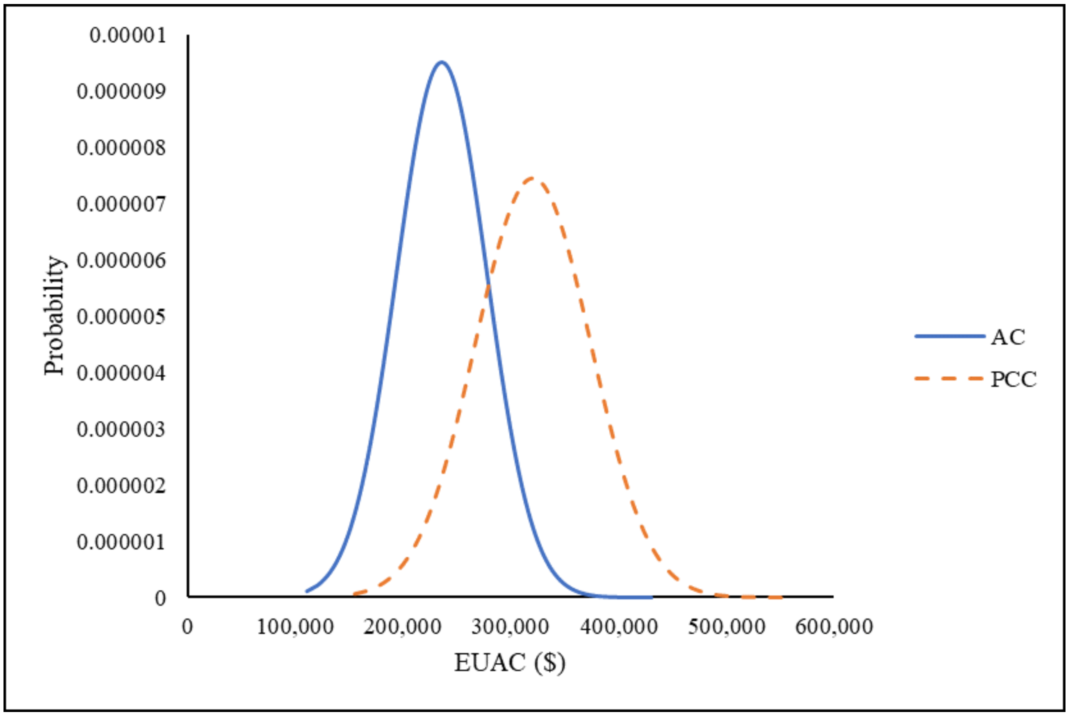


Figure B.57: Comparative EUAC probability distribution of the community cost for AC and PCC pavements

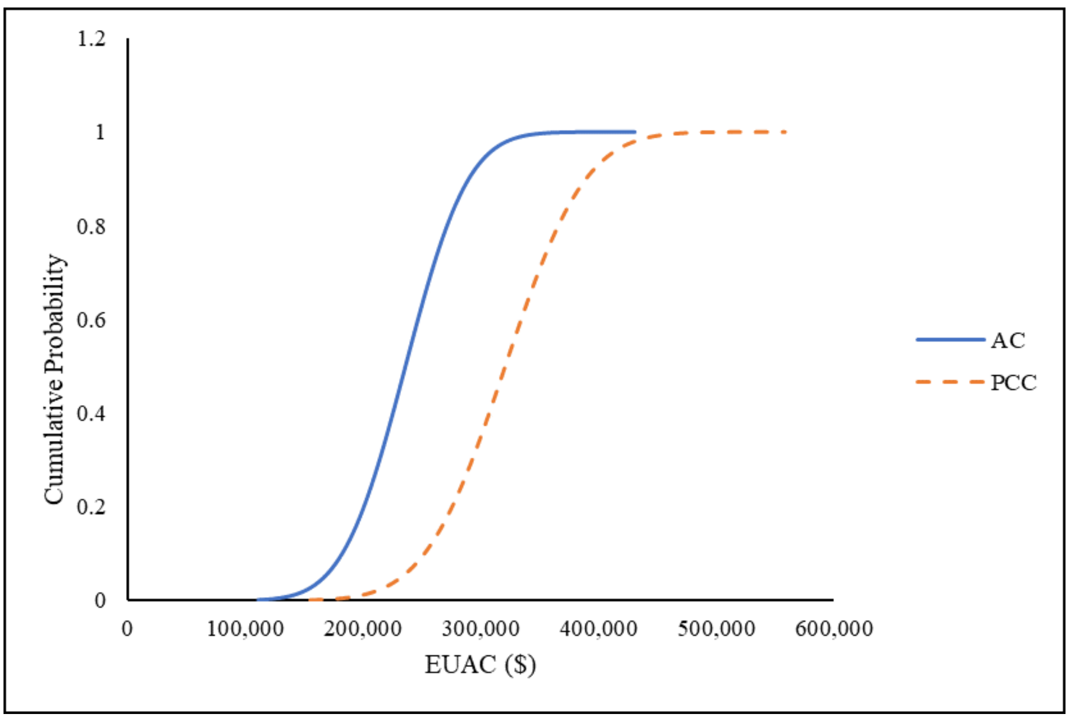


Figure B.58: Cumulative risk profile of the EUAC of the community cost for AC and PCC pavements

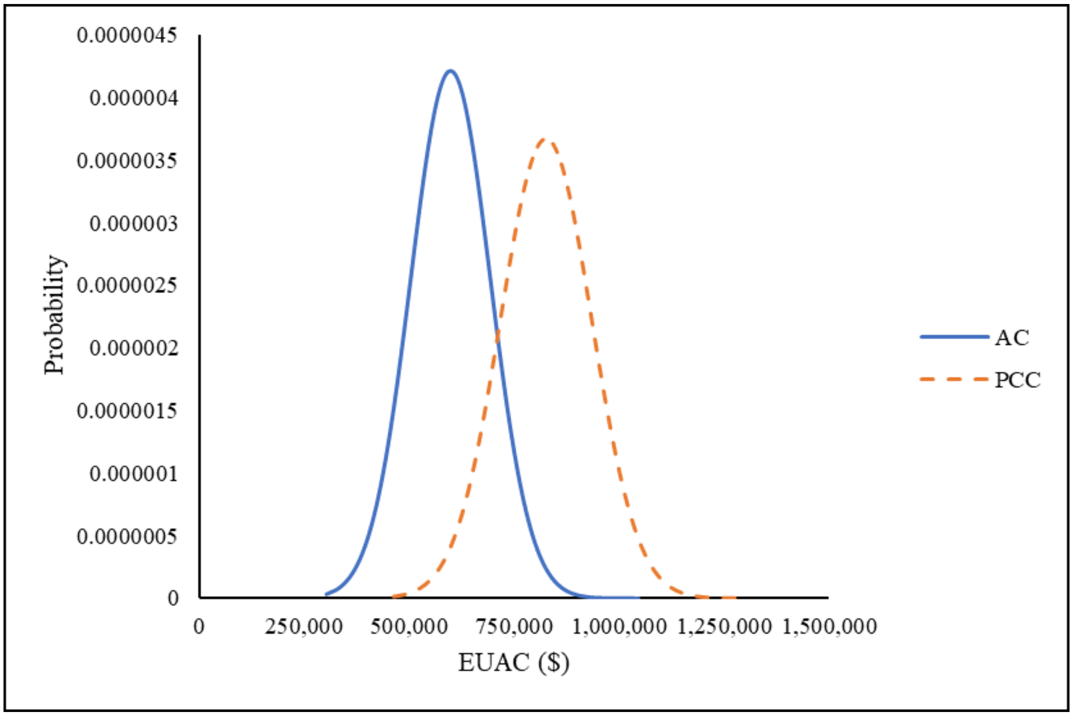


Figure B.59: Comparative EUAC probability distribution of the total cost for AC and PCC pavements

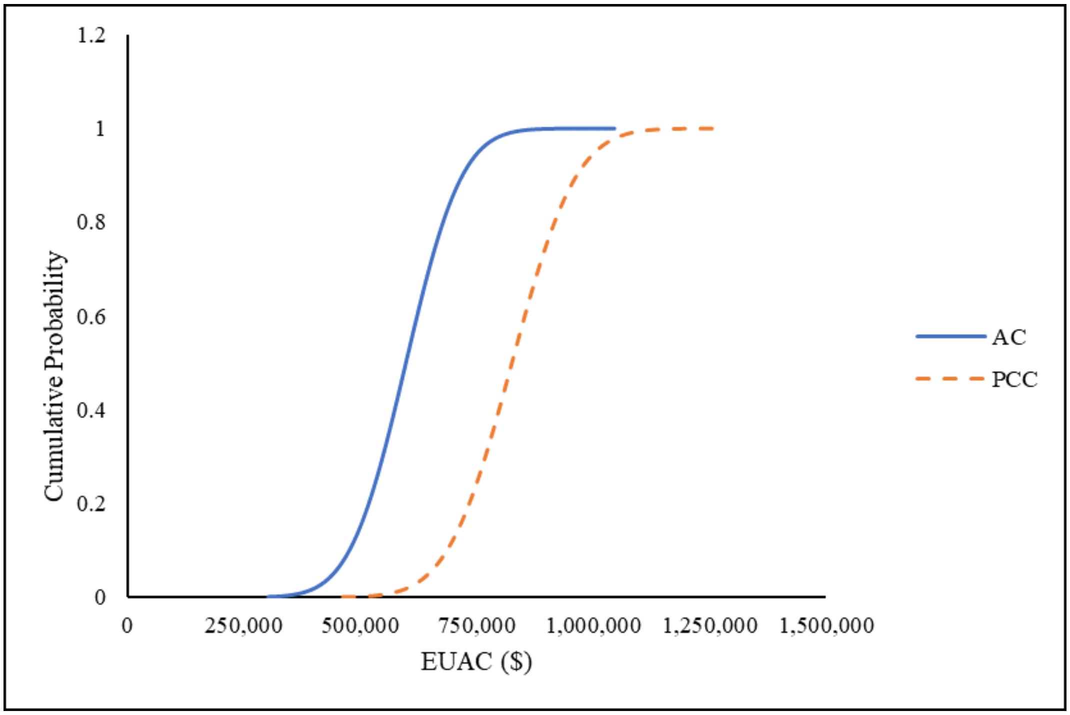


Figure B.60: Cumulative risk profile of the EUAC of the total cost for AC and PCC pavements

Dry-Non-Freeze Zone

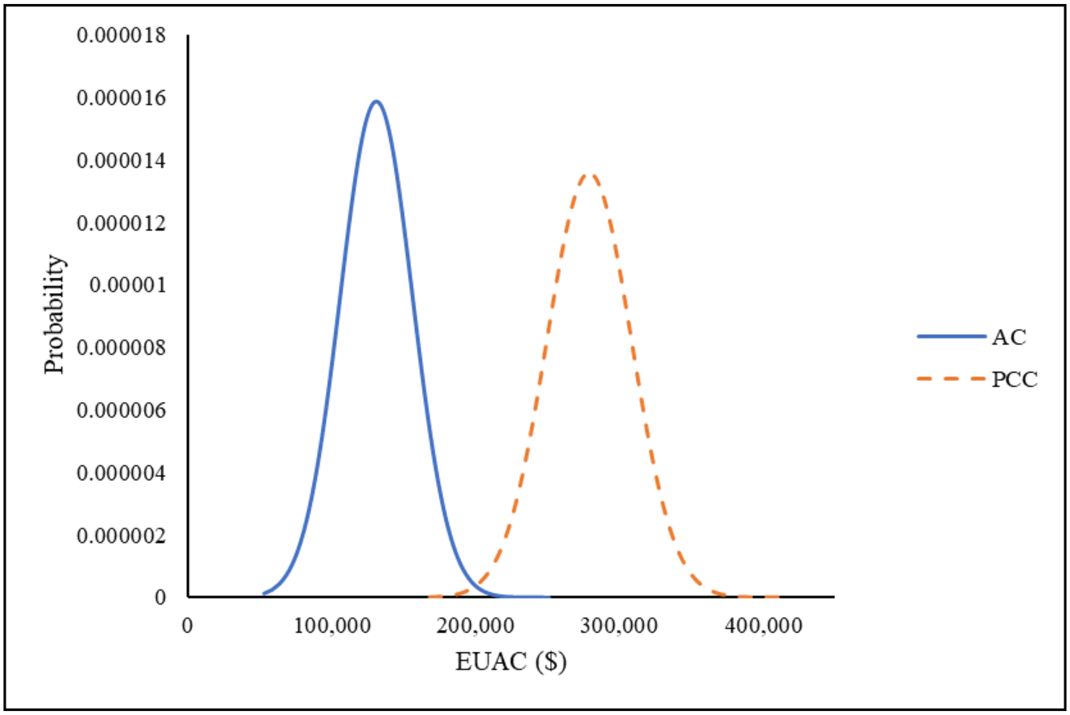


Figure B.61: Comparative EUAC probability distribution of the agency cost for AC and PCC pavements

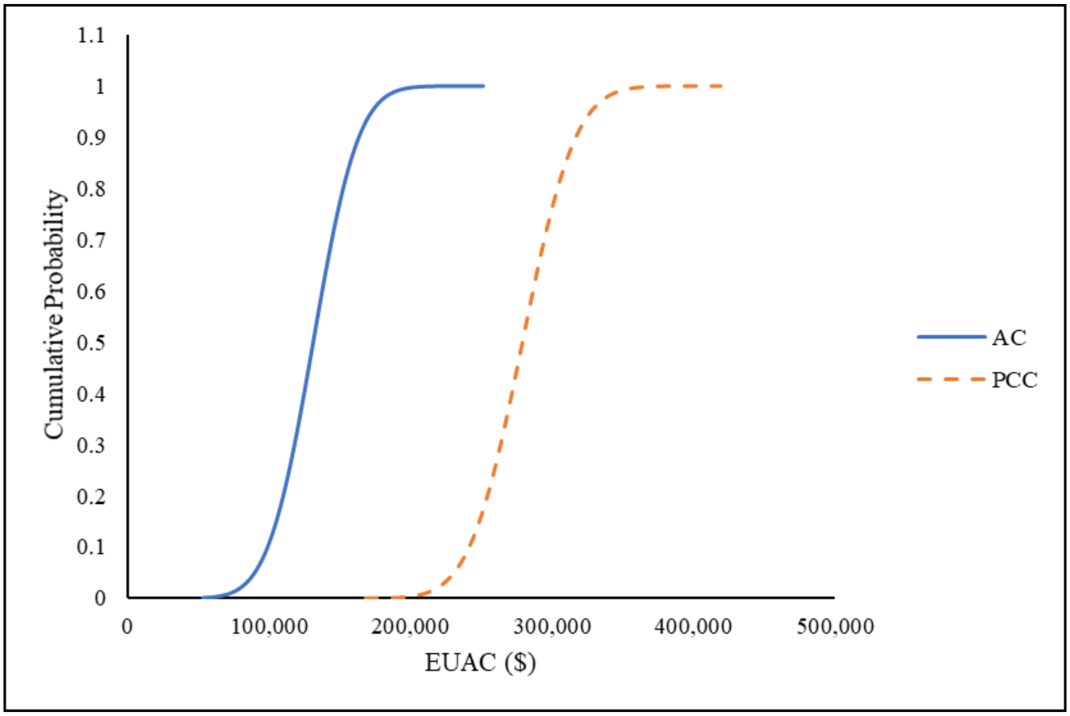


Figure B.62: Cumulative risk profile of the EUAC of the agency cost for AC and PCC pavements

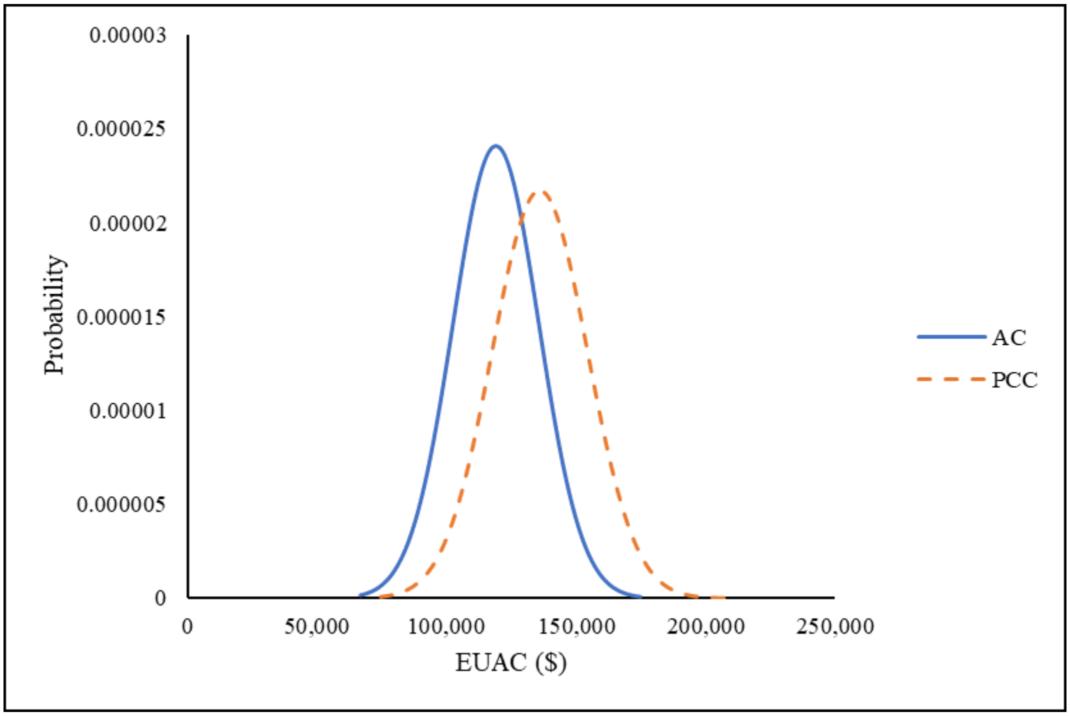


Figure B.63: Comparative EUAC probability distribution of the user cost (travel time delay cost) for AC and PCC pavements

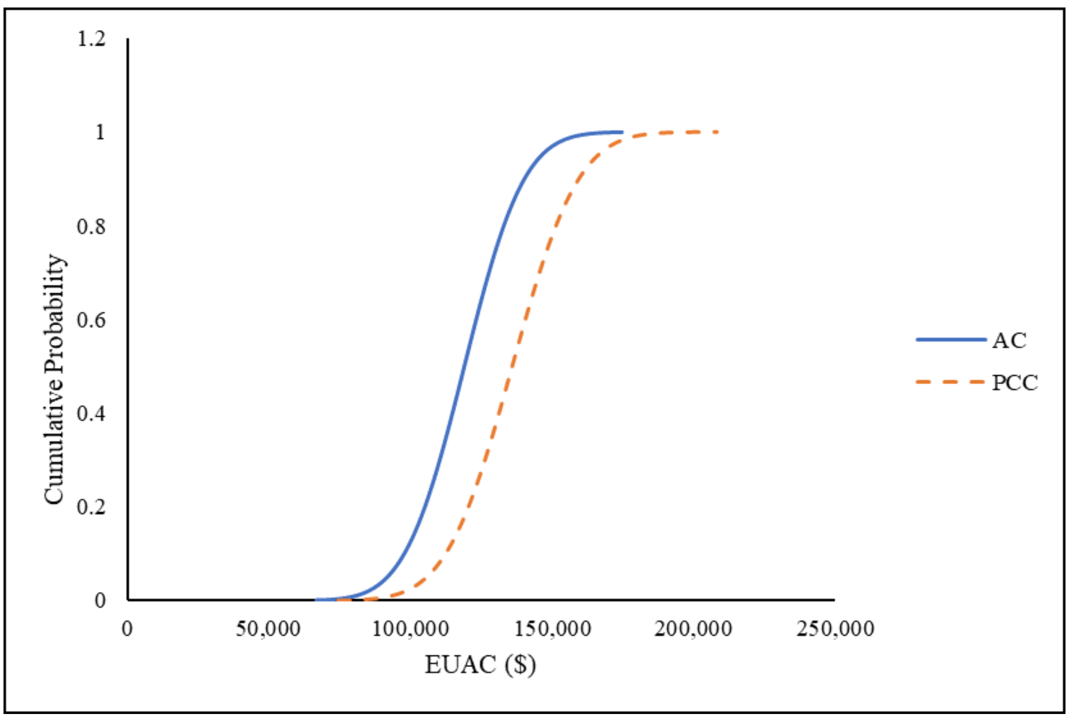


Figure B.64: Cumulative risk profile of the EUAC of the user cost (travel time delay cost) for AC and PCC pavements

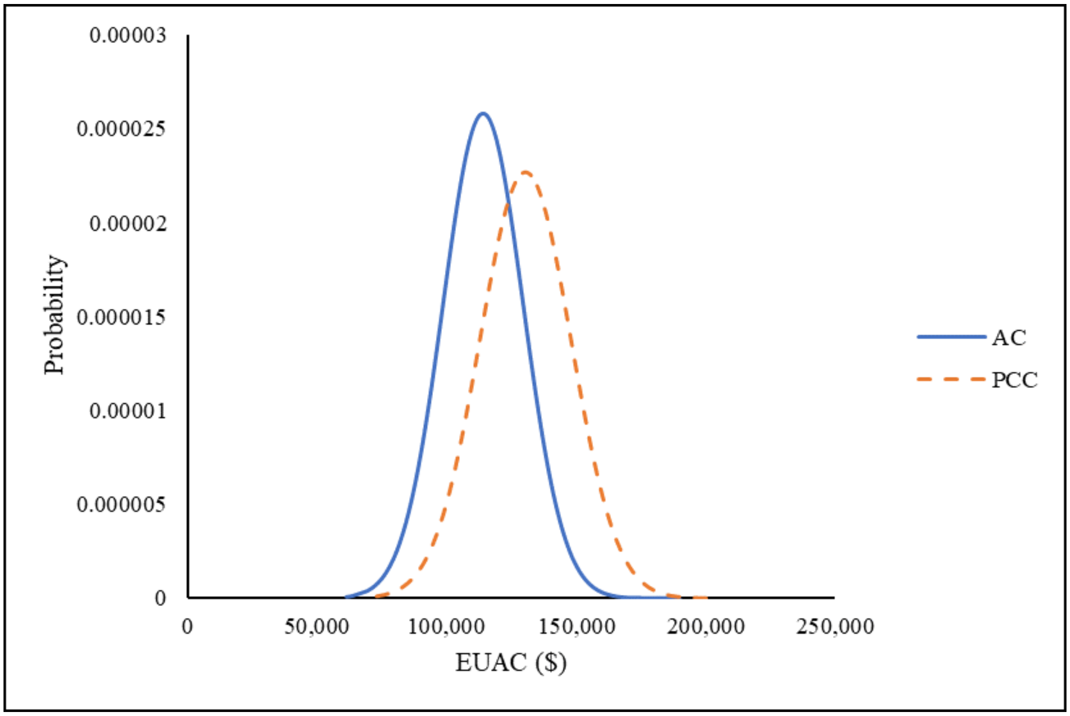


Figure B.65: Comparative EUAC probability distribution of the user cost (VOC) for AC and PCC pavements

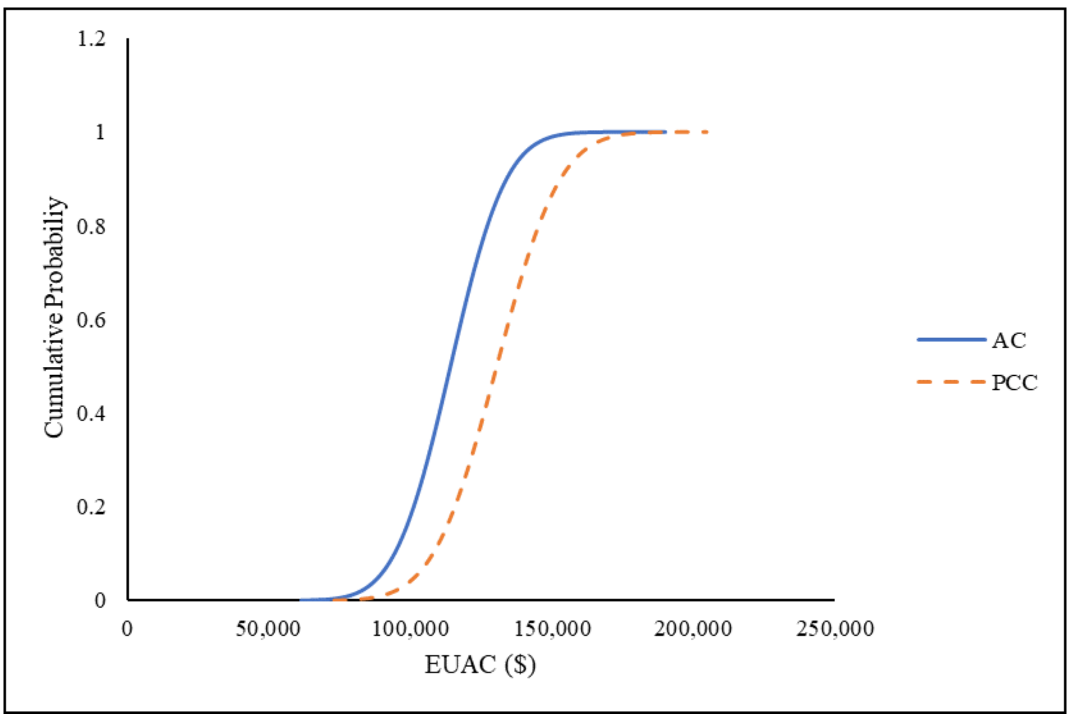


Figure B.66: Cumulative risk profile of the EUAC of the user cost (VOC) for AC and PCC pavements

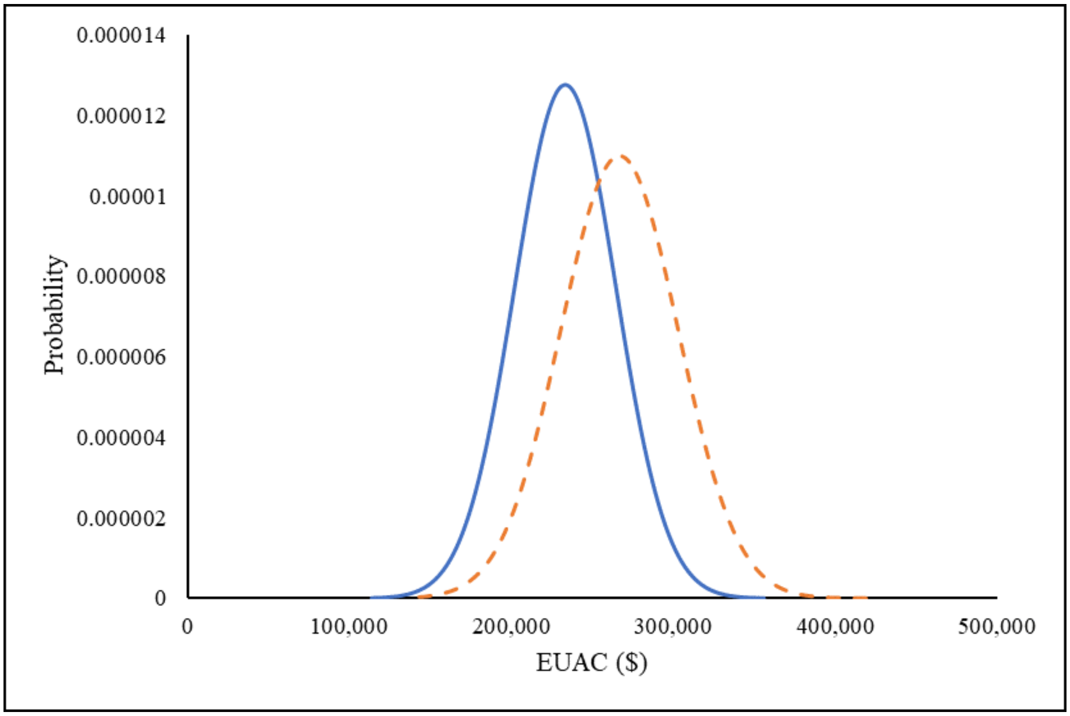


Figure B.67: Comparative EUAC probability distribution of the user cost for AC and PCC pavements

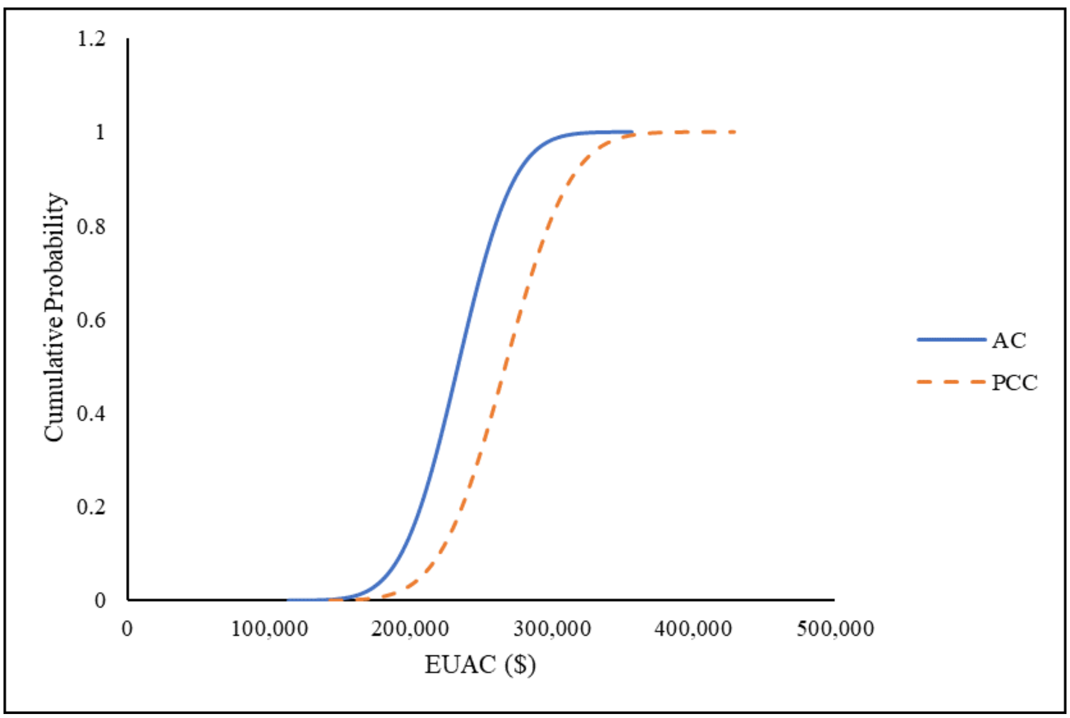


Figure B.68: Cumulative risk profile of the EUAC of the user cost for AC and PCC pavements

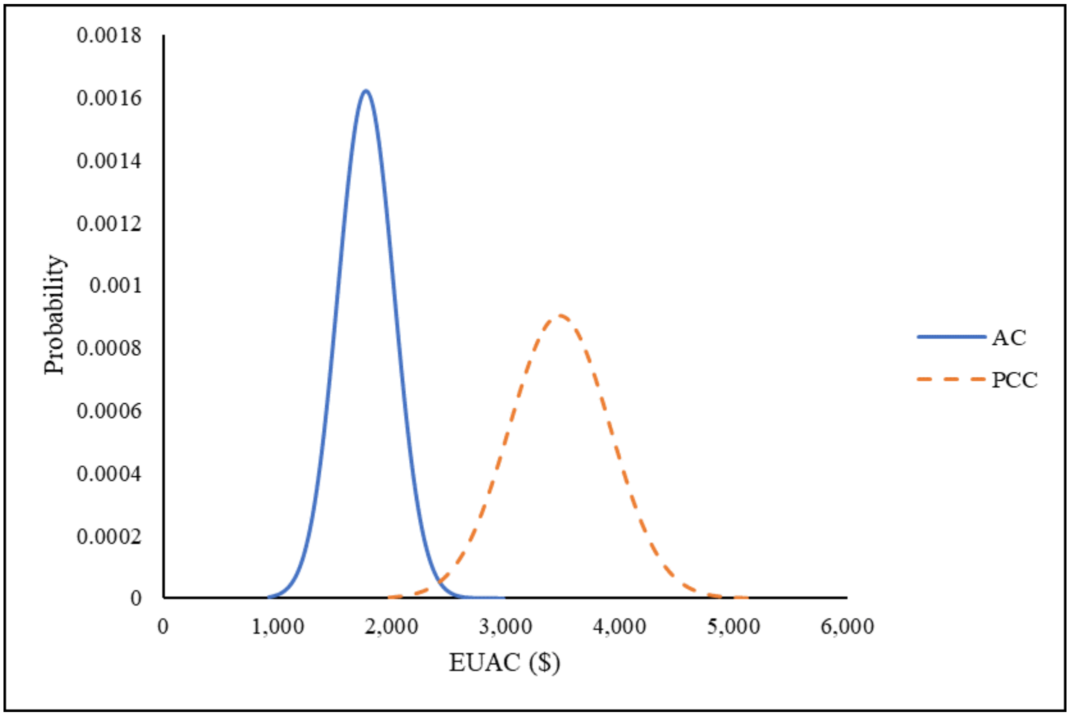


Figure B.69: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

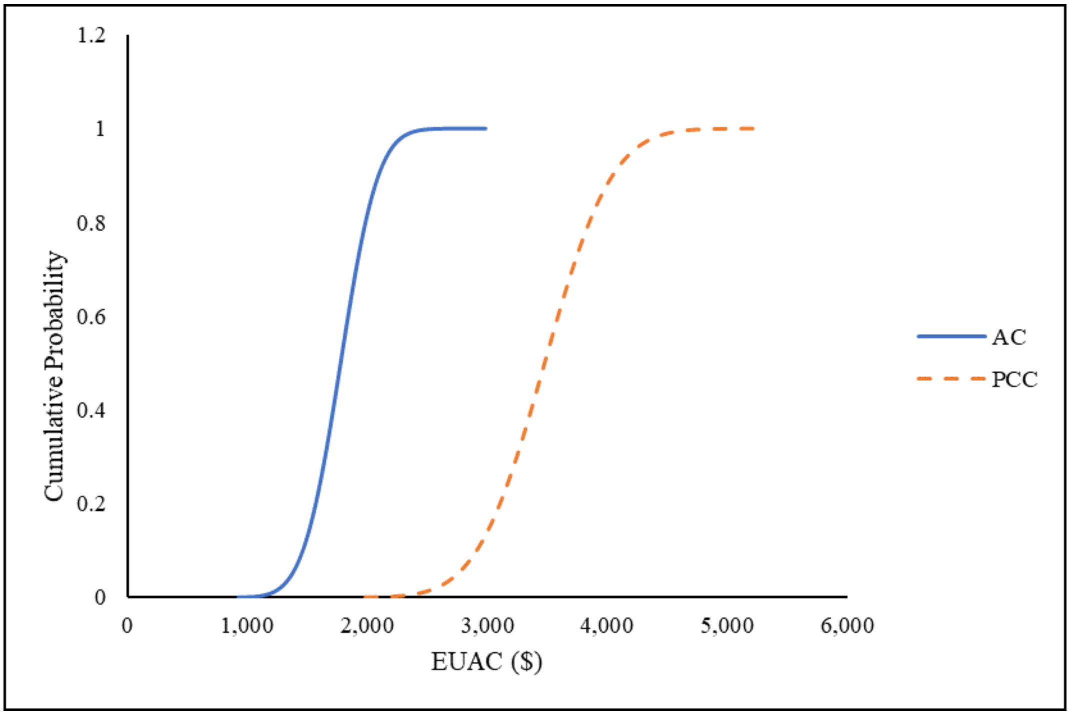


Figure B.70: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution-GWP of the GHG emission) for AC and PCC pavements

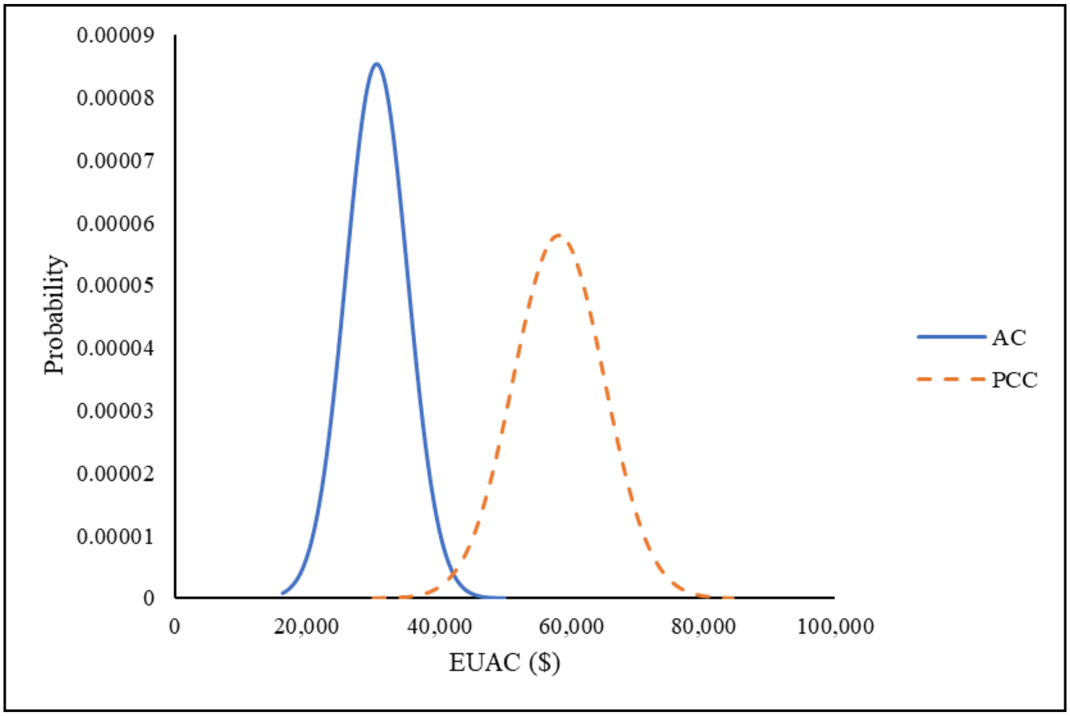


Figure B.71: Comparative EUAC probability distribution of the community cost (cost associated with air pollution-energy consumption) for AC and PCC pavements

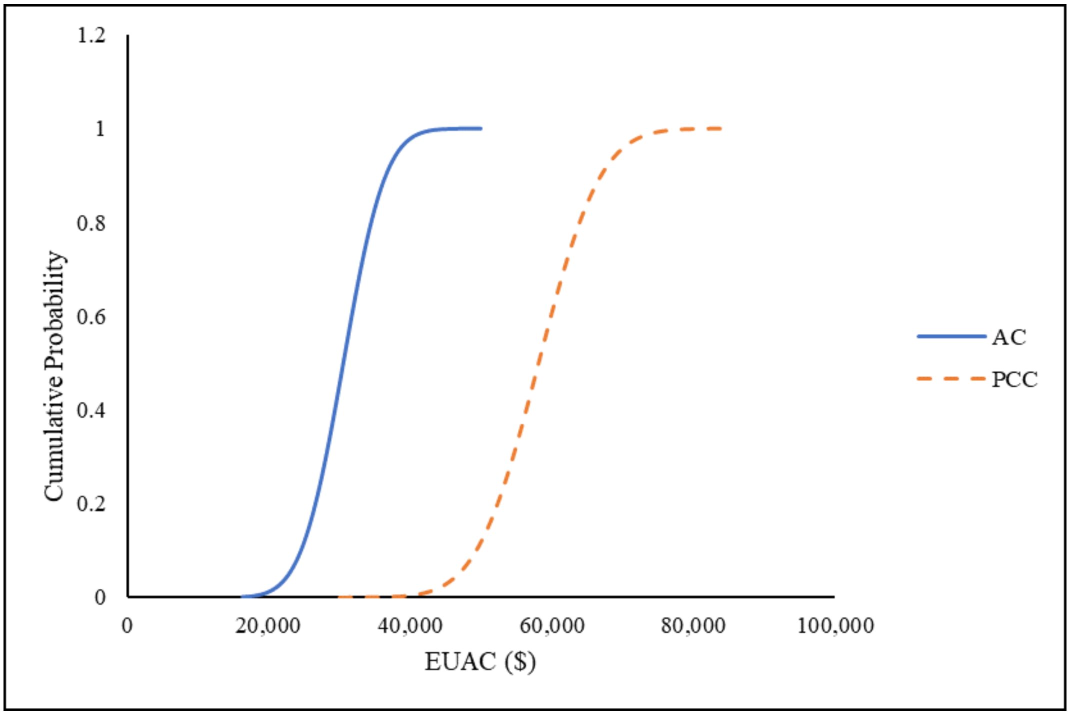


Figure B.72: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution- energy consumption) for AC and PCC pavements

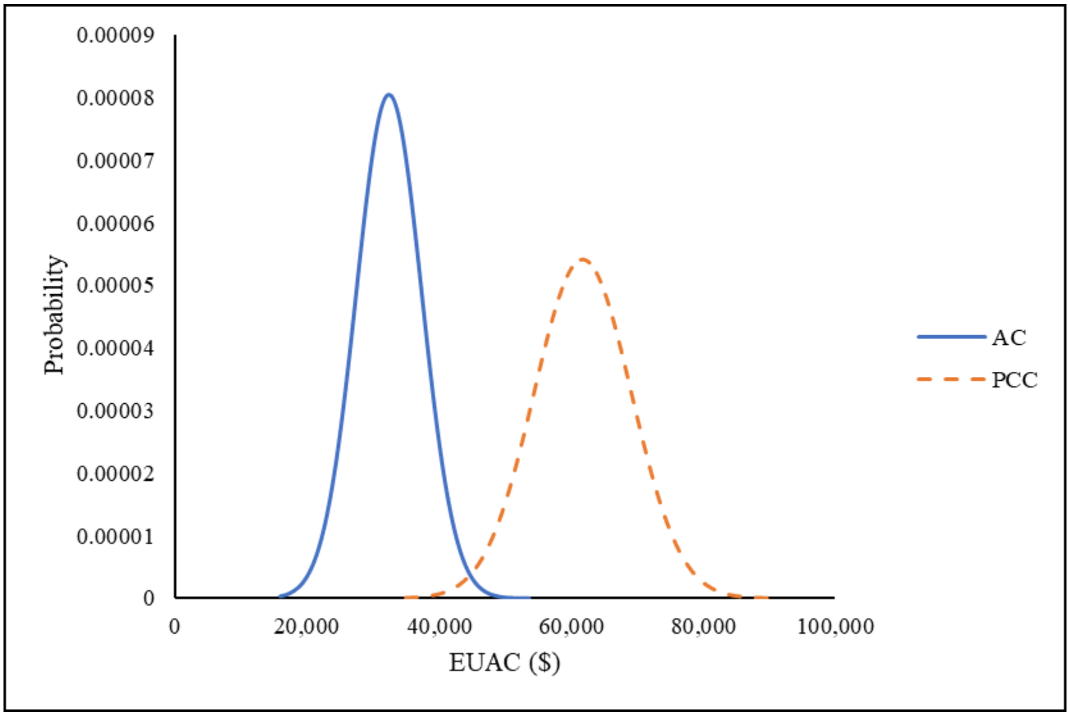


Figure B.73: Comparative EUAC probability distribution of the community cost (cost associated with air pollution) for AC and PCC pavements

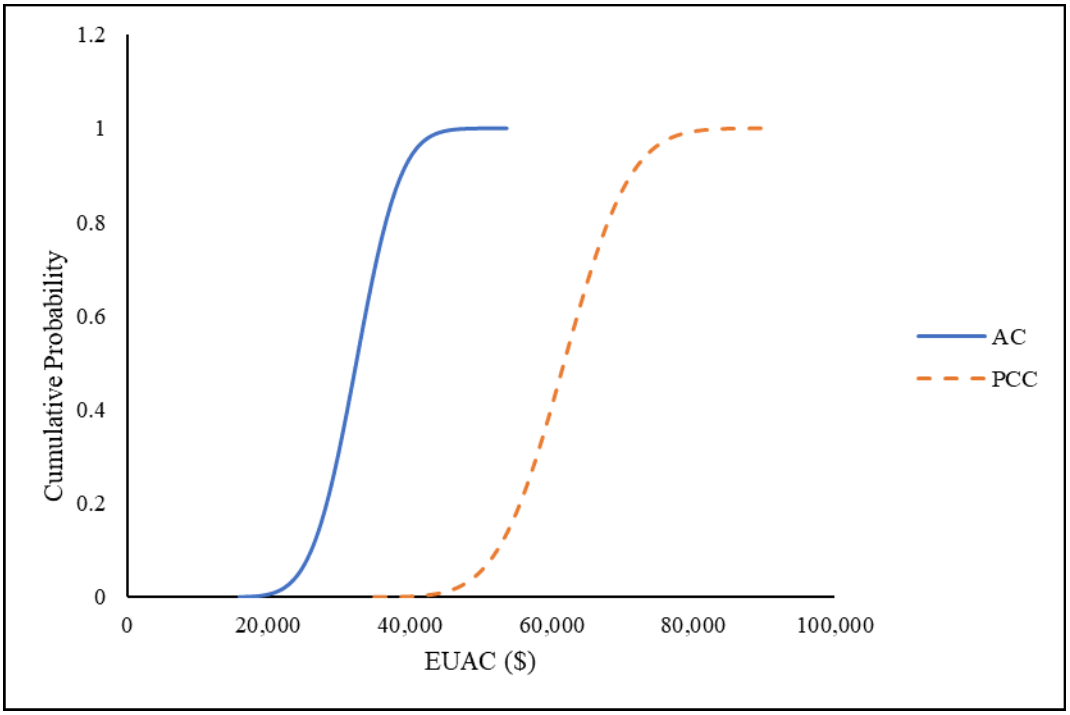


Figure B.74: Cumulative risk profile of the EUAC of the community cost (cost associated with air pollution) for AC and PCC pavements

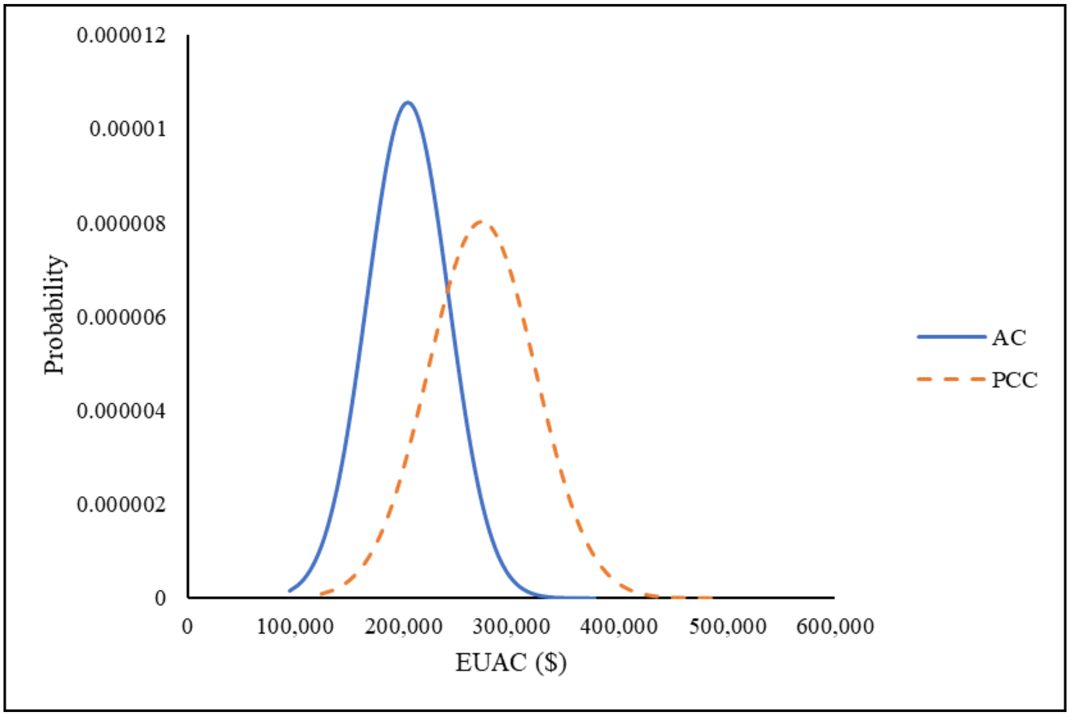


Figure B.75: Comparative EUAC probability distribution of the community cost (cost associated with noise pollution) for AC and PCC pavements

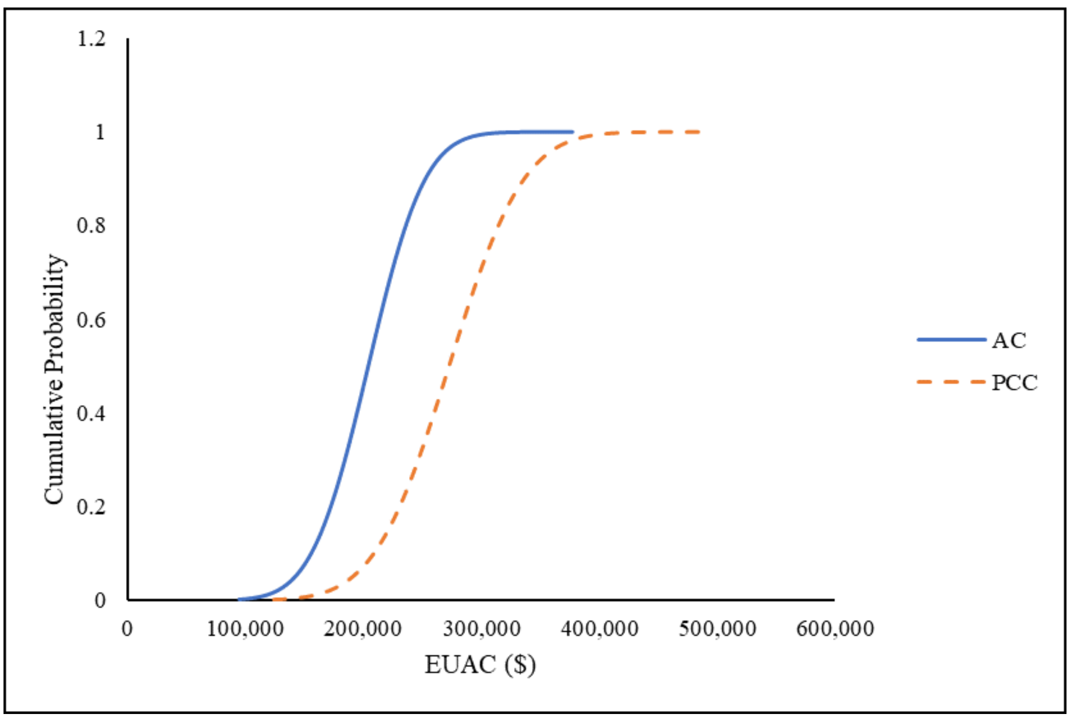


Figure B.76: Cumulative risk profile of the EUAC of the community cost (cost associated with noise pollution) for AC and PCC pavements

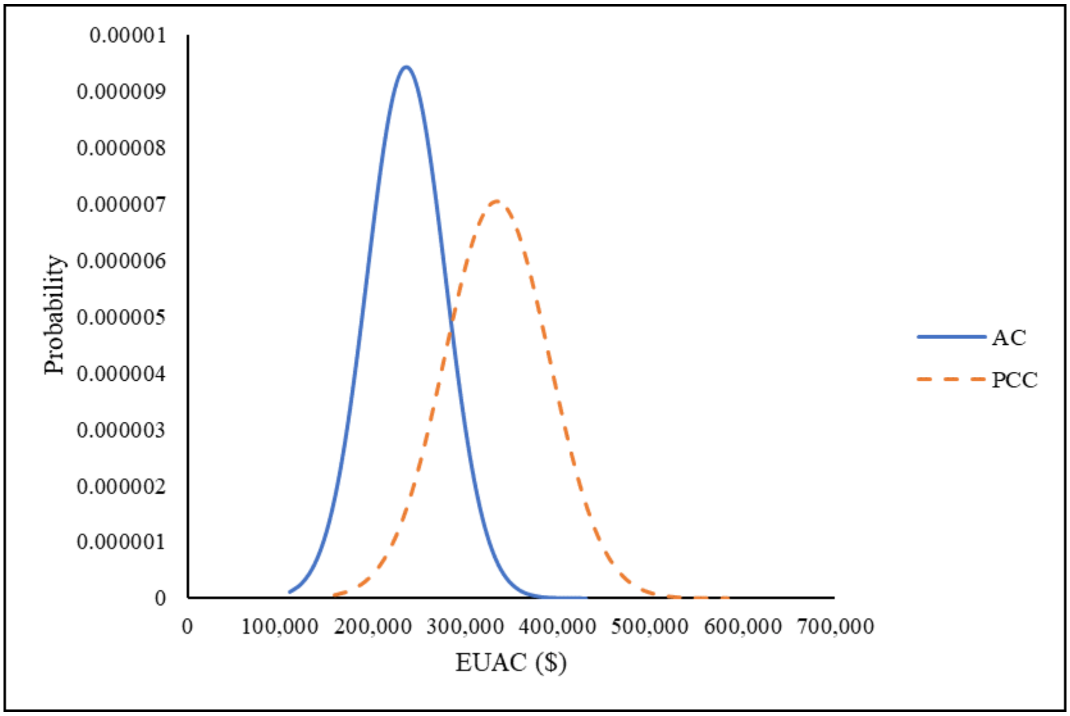


Figure B.77: Comparative EUAC probability distribution of the community cost for AC and PCC pavements

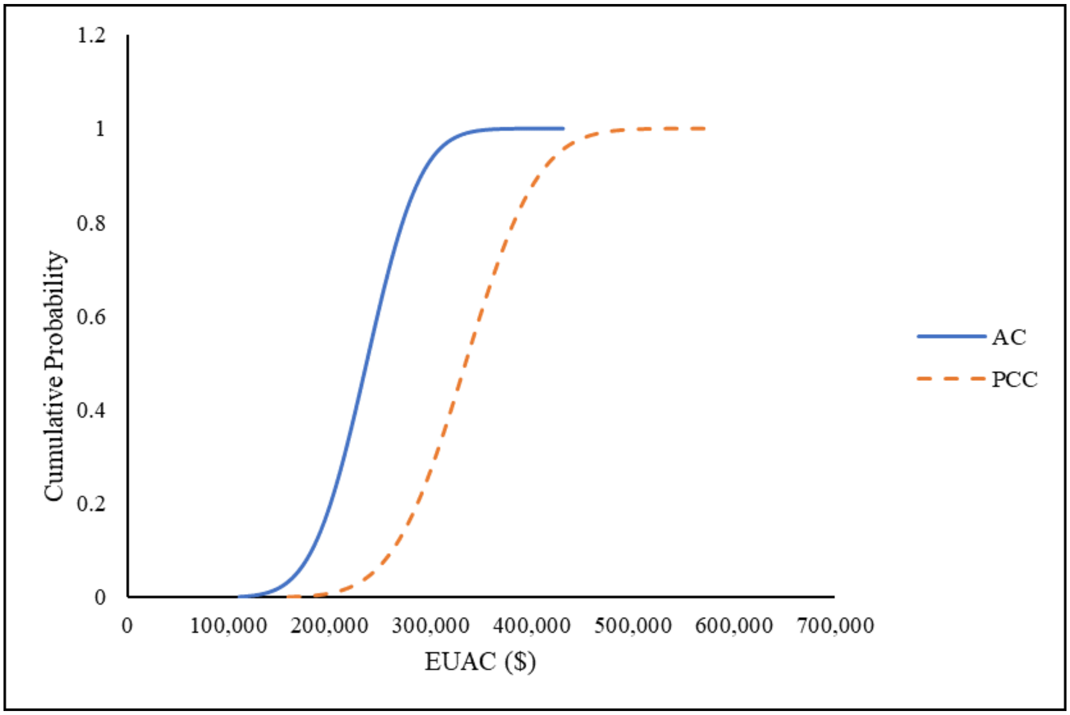


Figure B.78: Cumulative risk profile of the EUAC of the community cost for AC and PCC pavements

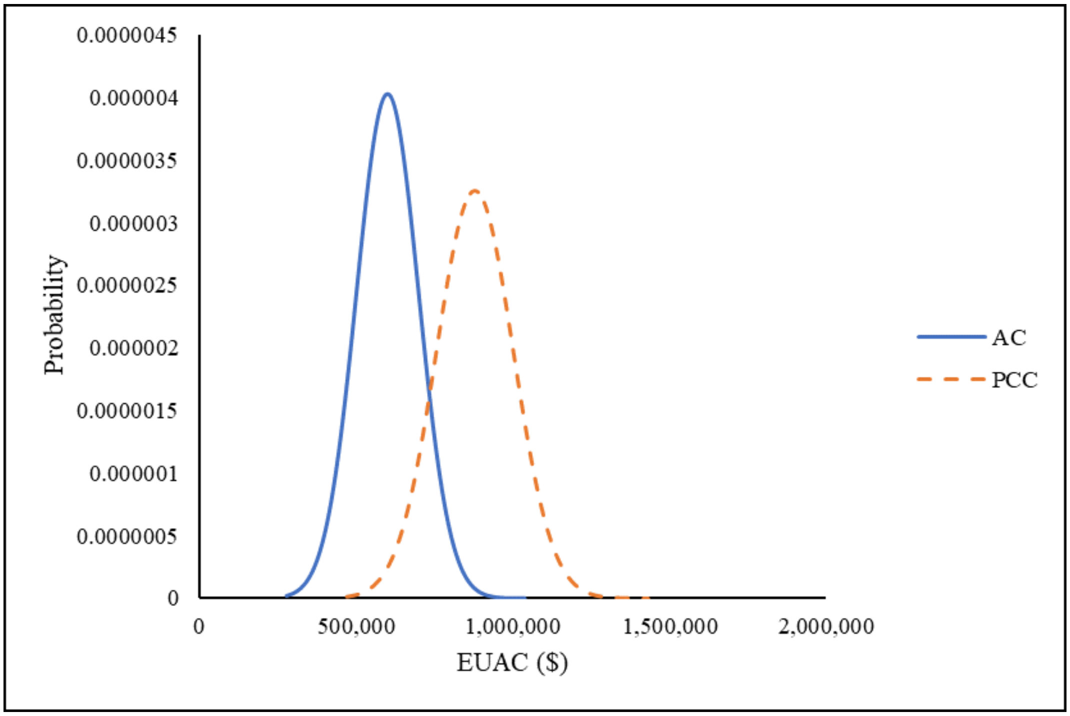


Figure B.79: Comparative EUAC probability distribution of the total cost for AC and PCC pavements

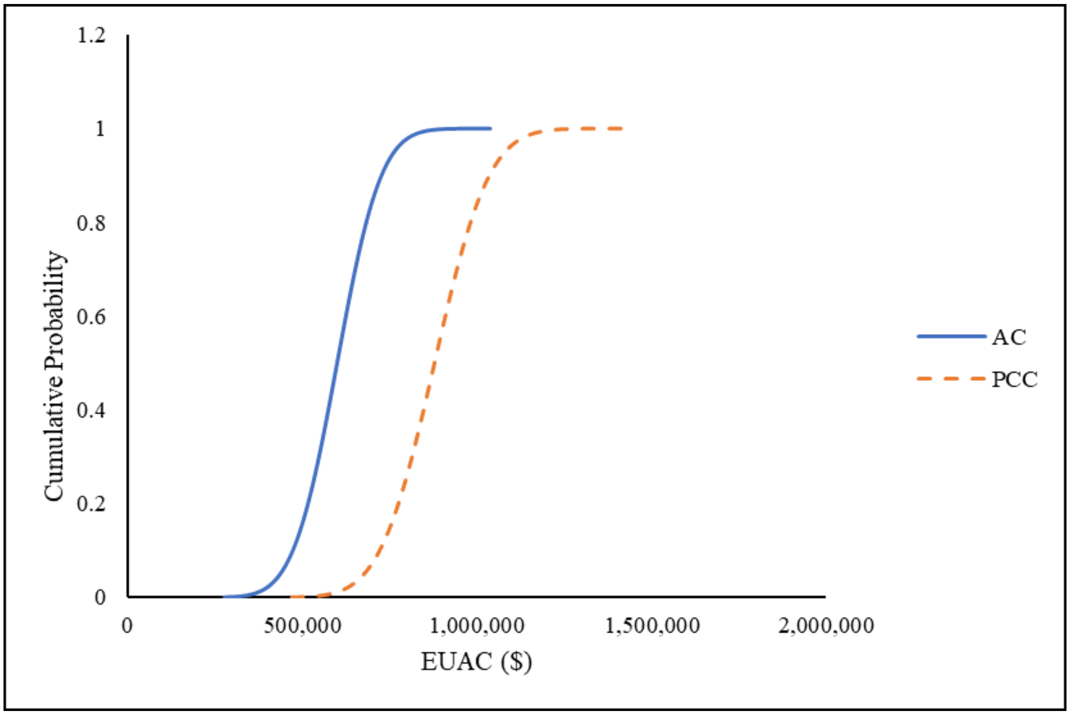


Figure B.80: cumulative risk profile of the EUAC of the total cost for AC and PCC pavements

PUBLICATIONS

1. Labi S, Saeed TU, Volovski M, **Alqadhi SD** (2015) *An Exploratory Discussion of the Impacts of Driverless Vehicle Operations on the Man-Made Environment*. Proceedings of the 1st International Conference on Mechanical and Transportation Engineering (ICMTE '15). ISBN: 978-1-61804-298-9. Kuala Lumpur, Malaysia. April 23-25, 2015.
2. **Alqadhi, Saeed**, Murillo Hoyos, Jackeline, Quiao, Yu, Labi, Samuel. "Measurement Of Pavement Rehabilitation Effectiveness: Constructs." *Eighth International Conference on Maintenance and Rehabilitation of Pavements* (2016).
3. Chen S, Saeed TU, **Al-Qadhi SD**, Labi S, Sinha KC (2016) *Safety Impacts of Pavement Rehabilitation at Multilane Highways*. Eighth International Conference on Maintenance and Rehabilitation of Pavements (MAIREPAV8), Singapore
4. **Alqadhi SD**, Saeed TU, Labi S (2016) *Evaluation of Alternative Materials for Interstate Highway Pavement Construction Using Life-Cycle Costing and Multiple Criteria Analysis.* Civil Engineering Conference in the Asia Region CECAR 7. Waikiki, Oahu, Hawaii | August 30 - September 2, 2016
5. Labi S, **Alqadhi SD**, Saeed TU, (2016) *Mathematical Constructs for Assessing the Effectiveness of Environmental Remediation Actions*. Civil Engineering Conference in the Asia Region CECAR 7. Waikiki, Oahu, Hawaii | August 30 - September 2, 2016
6. Chen, S., Saeed, T. U., **Alqadhi, S. D**., & Labi, S. (2017). Safety impacts of pavement surface roughness at two-lane and multi-lane highways: accounting for heterogeneity and seemingly unrelated correlation across crash severities. Transportmetrica A: Transport Science, 1-16.
7. Ghahari, S. A., Volovski, M., **Alqadhi, S.**, & Alinizzi, M. (2018). Estimation of annual repair expenditure for interstate highway bridges. Infrastructure Asset Management, 1-8.
8. **Alqadhi, S**., Ghahari, S. A., Chen, S., Volovski, M., & Woldemariam, W. (2018). Costs and benefits of highway resurfacing: a case study of Interstate 465 in Indiana, USA. Infrastructure Asset Management, 1-11.