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Effect of Asphalt Rejuvenating Agent on Aged Reclaimed Asphalt Pavement and Binder Properties

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The objective of this research was to investigate the effects of rejuvenation on HIR performance characteristics by assessing critical performance indicators such as stiffness, permanent deformation, moisture susceptibility, and cracking resistance. A two-step experimental program was designed that included mechanical property measurements of the HIR mixture and rheological properties of the extracted binder. The level of mixing occurring between new and aged binder with ARA was also investigated. HIR samples were obtained from three Kansas Department of Transportation projects, and Hamburg wheel-tracking device, dynamic modulus, flow number, Texas overlay, thermal stress restrained specimen, and moisture susceptibility tests were conducted on mixtures with and without ARA. Rheological studies on the extracted binder included dynamic shear rheometer and bending beam rheometer tests. The miscibility of new and aged binder was investigated using scanning electron microscope (SEM) images, energy dispersive X-ray spectroscopy (EDXS), and the exudation droplet test (EDT). Study results showed significant variability in the mechanical performance of HIR mixtures, which was attributed to the variability of binders as observed in EDT, SEM, and EDXS studies.

Life-cycle cost analysis (LCCA) showed that HIR is an economic maintenance alternative for asphalt projects in Kansas. LCCA results showed that pavement design strategies with HIR activities will result in alternatives with lower net present values when compared to alternatives without HIR maintenance activities.

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Abstract

Hot in-place recycling (HIR) preserves distressed asphalt pavements while minimizing use of virgin binder and aggregates. The final quality of an HIR mixture depends on the characteristics of the original binder, aging of the pavement surface during service, and whether or not new binder or rejuvenator was added to the mixture. An HIR mixture should maintain desired properties for additional service periods, making asphalt binder modification inevitable. Asphalt binder modifications in HIR are commonly performed by adding an asphalt rejuvenating agent (ARA). However, ARA may adversely affect the qualities of new HIR and potentially fail to improve the quality of the final surface.

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Life-cycle cost analysis (LCCA) showed that HIR is an economic maintenance alternative for asphalt projects in Kansas. LCCA results showed that pavement design strategies with HIR activities will result in alternatives with lower net present values when compared to alternatives without HIR maintenance activities.

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Chapter 1: Introduction

1.1 In-Place Recycling

Incorporating higher quantities of reclaimed asphalt pavement (RAP) in construction of new pavements as a part of concentrated efforts to secure sustainable roadway systems is a contemporary approach to achieving viability in constructing new roadway pavements. In-place recycling, a maintenance method for hot-mix asphalt (HMA) pavements, can improve functional performance and ride quality of pavement by correcting surface deficiencies such as rutting, raveling, and surface cracking by modifying the top 25–40 mm of the existing surface. In addition to being cost-effective and environmentally sound, in-place recycling minimizes construction time and traffic flow disruption, making it an increasingly desirable maintenance alternative for HMA pavements (Stroup-Gardiner, 2011).

In-place recycling techniques such as hot in-place recycling (HIR), cold in-place recycling (CIR), and full-depth reclamation (FDR) are appropriate for various types, levels, and severities of distresses on HMA pavement. Each in-place recycling method is applicable for a certain period in the pavement life as the surface ages in service. Figure 1.1 illustrates the relationship between the pavement condition and the type of applicable in-place recycling method. A combination of stress severity and overall existing HMA pavement thickness generally determines the suitable in-place recycling methods (Stroup-Gardiner, 2011).

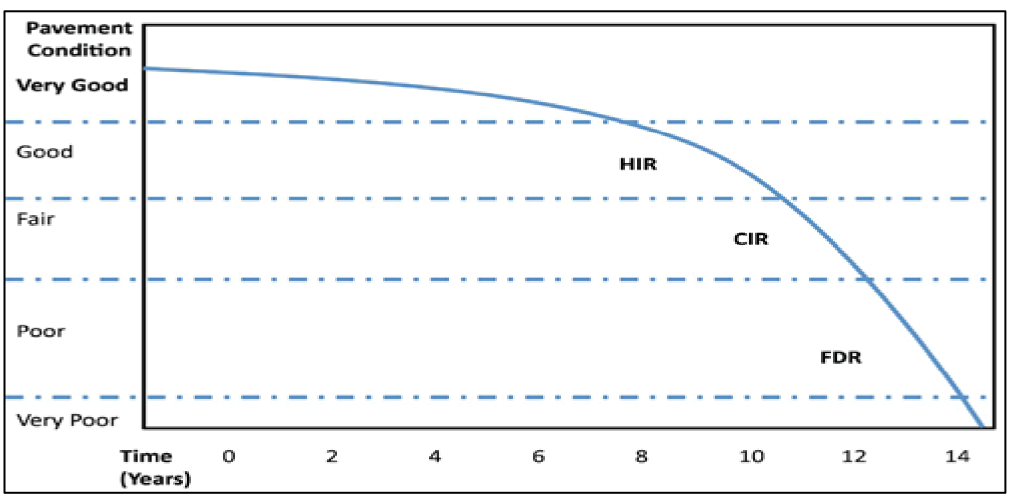


Figure 1.1: Pavement Condition and Applicable In-Place Recycling Method
Source: Stroup-Gardiner (2011)

1.1.1 Cold In-Place Recycling

CIR is the process of in-place recycling of an asphalt pavement with a train of equipment that can range in size from a single unit to multi-unit train. CIR is often used on roads with low traffic volume or secondary roads in which a central hot-mix plant may not be convenient for producing HMA for an overlay. In CIR, the top 25–75 mm of existing pavement are milled and removed, mixed with additives and occasionally virgin binder and aggregates, and then laid on the pavement surface. As the name implies, no heating is required in the CIR process. In addition to asphalt emulsions that are most commonly used in CIR, recycling agents, cutback asphalts, foamed asphalt, and chemical additives are feasible additives in order to achieve desirable CIR properties. Depending on structural requirements of the final pavement, CIR may receive a wearing course such as a chip seal or a thin HMA overlay. New aggregates or dry additives added to the mix are spread on the existing pavement right before milling, and additional liquids (recycling agents and water) are added to the mixture to allow proper mixing, aggregate coating, and compaction (Asphalt Recycling and Reclaiming Association [ARRA], 2015).

Added moisture and water associated with emulsion must be allowed to evaporate after the pavement is placed, making the CIR process slower than HIR and FDR. Curing time for CIR ranges from 2 days to 2 weeks depending on weather conditions (Kandhal & Mallick, 1997). CIR mixtures are also stiffer than heated asphalt mixtures and require additional compaction. Even with increased compaction, densities identical to HMA mixtures cannot be achieved with CIR. Total air voids in CIR mixtures range between 9% and 15%, while desirable HMA pavement has 7% air voids. High percentages of air voids can cause premature damage if a wearing surface is not applied when the curing process is complete (ARRA, 2015). Figure 1.2 shows a schematic of a single-unit CIR train.

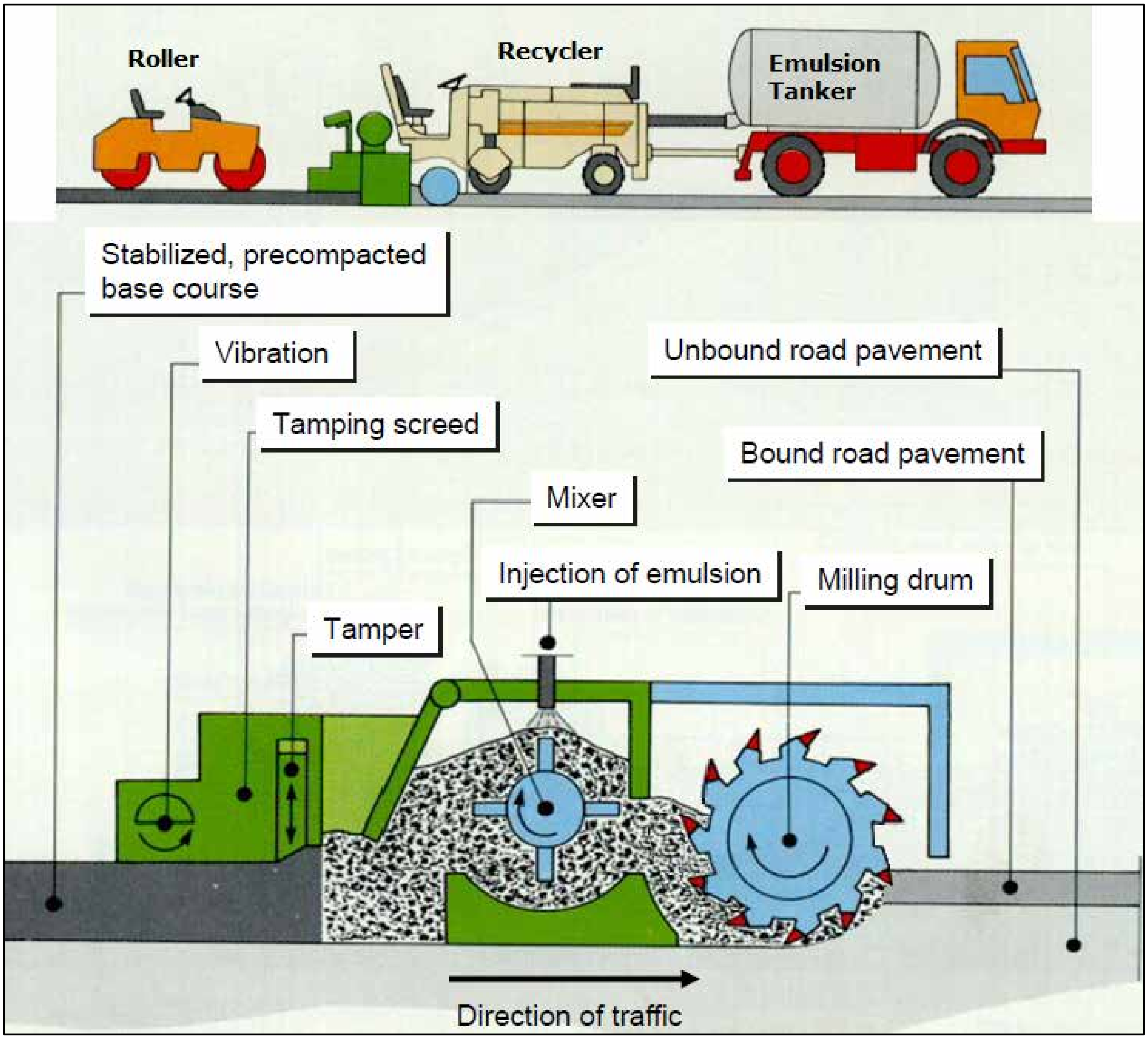


Figure 1.2: Schematic of a Single-Unit CIR Train
Source: Kandhal and Mallick (1997)

1.1.2 Full-Depth Reclamation

FDR is a rehabilitation method in which the full thickness of the HMA pavement and a predefined portion of the underlying layers, including base, subbase, and potentially subgrade, are pulverized and homogeneously blended to provide an upgraded, uniform bed that acts as the base layer for a new surface course (Stroup-Gardiner, 2011). Similar to CIR, FDR requires no heat. If the in-place material is insufficient to provide the desired quality as the base layer for future pavement structure, new materials may be imported and included in the modification and stabilization of the treated base layer. Mechanical, chemical, and bituminous stabilization can be used in FDR. Mechanical stabilization requires the addition of granular material such as virgin aggregates, recycled asphalt, or crushed portland cement concrete. Chemical stabilization requires the addition of lime, portland cement, fly ash, lime or cement kiln dust, calcium/magnesium chloride, or many other chemical products. Bituminous stabilization is accomplished using liquid asphalt, asphalt emulsion, and foamed asphalt. A combination of mechanical, chemical, and bituminous modification methods may be used to increase the stability of the treated base. Although treatment depths vary depending on the thickness of the existing pavement structure, they generally range between 100 and 300 mm.

As illustrated in Figure 1.1, FDR is a rehabilitation alternative for pavements in poor condition when CIR and HIR are not feasible. Advantages of FDR mentioned in the literature include sustainability and conservation of energy and nonrenewable resources, elimination of pavement distress in base and subbase (aggregate gradation, rutting, cracks, bumps, and drainage), high production rates, and economical feasibility. Once the existing pavement is pulverized and stabilized, a final wearing surface of asphalt concrete or portland cement concrete is constructed on top of it (Stroup-Gardiner, 2011).

1.1.3 Hot In-Place Recycling

HIR, an in-place method for HMA pavement recycling, preserves or maintains distressed asphalt pavements while minimizing use of virgin binder and aggregates. In HIR, the existing surface is heated immediately before milling, mixed with new materials and additives to improve properties of the reclaimed asphalt pavement (RAP), and then placed and compacted as a new surface, all in one continuous operation. The new materials and additives may include new aggregates to adjust the final gradation or new binders and asphalt rejuvenating agents (ARAs) to soften the aged binder in RAP and increase the flexibility of the final asphalt concrete layer (Stroup-Gardiner, 2011; ARRA, 2015).

HIR is an applicable treatment when a majority of pavement distresses are at the low-severity level and confined to the upper 25–40 mm of the surface with no evidence of structural problems such as longitudinal cracking in the wheel path, alligator cracking, or edge cracking.

HIR can be used on roads with less than 5,000 average annual daily traffic (AADT) as long as the subgrade can support the weight of the recycling train. It is the preferred in-place recycling method for roads with AADT higher than 30,000. In Kansas, the decision on whether or not the subgrade could support the weight of the recycling equipment was previously made based on the thickness and condition of the existing pavement structure. The standardized procedure included coring and dynamic cone penetrometer (DCP) test of the subgrade. However, extensive use of HIR has replaced the previous method of evaluation in Kansas. Existing cores and pavement distress are evaluated and the decision about the subgrade strength is made based on past Kansas Department of Transportation (KDOT) experiences.

The HIR train consists of a preheating unit, a heating and recycling unit, and a roller unit. Figure 1.3 shows the HIR train as it replaces aged asphalt on the top pavement layer with a new recycled layer. While the preheating unit heats up the old HMA pavement surface, the heating and recycling unit applies more heat and scarifies the HMA pavement. Preheating units should uniformly increase the pavement surface temperature across the treatment area, or mat, without leaving hot or scorching spots. The length of time that pavement is exposed to reduced heat must be increased in order to minimize damage to existing asphalt binder and aggregates when heating. This extended time can be accomplished by slowing the rate at which the heat source moves over the pavement or increasing the number of heat sources. Slowing the heat source, however, may lead to reduced production rates and increased costs. Therefore, contractors optimize production using additional preheaters.

The original HIR process utilized direct contact between the pavement surface and flame in order to heat the surface, but radiant or infrared heating is currently used to avoid damaging the asphalt binder and causing undesirable emissions. Although propane is the most commonly used fuel for indirect heating, new hot air/low infrared heating units use diesel fuel. Emission control systems have been developed to control gaseous hydrocarbons generated in the HIR process. Fumes generated during the heating process are collected and incinerated in a combustion chamber or after burner. The hydrocarbons/combustible materials are reduced primarily to carbon dioxide and water vapor. The emission systems are designed to reduce or eliminate the opacity, irritants, and particulates to a level that complies with air pollution standards. HIR trains use emission control systems that have met very strict emission regulations of Washington and California. Figure 1.4 shows a heating recycling unit with a propane tank on top. Depending on the method of recycling, the surface temperature can elevate to 110–150 °C, requiring multiple rows of spring-loaded scarifiers to scarify the heated pavement.

HIR benefits routinely documented in the literature include minimized material costs, conserved virgin binder and aggregates, eliminated disposal components, reduced fuel consumption and emission, minimized traffic disruptions, shortened lane closure times, maintained height clearance, and resolved existing material problems such as moisture damage (Stroup-Gardiner, 2011; ARRA, 2015). However, HIR cannot be used in tight turns, steep grades, castings, and areas that require lane widening. HIR also may not be applicable to recycling pavements with multiple seal coats, crumb rubber surface treatments, porous HMA mixes, or HMA surfaces with excessive crack sealant (Stroup-Gardiner, 2011; ARRA, 2015).

The Asphalt Recycling and Reclamation Association (ARRA) recognizes three basic types of HIR methods: resurfacing (surface recycling), repaving, and remixing. In order to eliminate the effect of oxidative hardening due to long-term aging at service and heating during the recycling process, recycling agents may be added in all three methods, but virgin aggregate is used only in repaving and remixing operations. The three processes are described in the following sections.

1.1.3.1 Resurfacing

Resurfacing is a rehabilitation process that restores cracked, brittle, and irregular pavement in preparation for a final thin wearing course. A thin-lift HMA overlay is commonly placed on top of a wearing course comprised of surface recycled pavement on low-volume roads. A conventional HMA layer is 25–40 mm, while a thin HMA layer is approximately 12.5 mm. Placement of a thin HMA layer is only possible if the recycled roadway is level, rejuvenated, and free of cracks. Resurfacing is applicable only on pavements with stable and adequate bases that can accommodate the weight of the HIR equipment and require modifications only to the top 25–50 mm of the asphalt course. Since no virgin aggregates or asphalt binder are added in the resurfacing process, ARA governs the modification of existing HMA pavement. Parameters such as ambient temperature and wind conditions as well as characteristics of the treated HMA pavement and its moisture content can influence the production rate of resurfacing, which is typically no less than 1.5 m/min (meters per minute) and no more than 15 m/min.

Although scarification depths between 20 and 40 mm are attainable in resurfacing, existing pavement is typically scarified to 20–25 mm by increasing the surface temperature to 110–150 °C.

The main purpose of resurfacing is to eliminate surface cracks and distortions, but it also restores the pavement surface to a desirable line, grade, and cross section, thereby ensuring good drainage and improved surface frictional resistance. Resurfacing has also proven effective in removing reflective cracks when used prior to HMA overlay (ARRA, 2015).

Figure 1.3: HIR Train (Preheating and Heating and Recycling Units)

Figure 1.4: Heating and Recycling Unit in HIR Train

1.1.3.2 Remixing

Remixing is the preferred HIR method when significant modifications are required for the physical properties of existing asphalt pavement. In the remixing process, changes can be made to aggregate gradation, aggregate abrasion/friction number, asphalt binder content and rheology, and asphalt mixture volumetric and stability properties. The pavement surface can be reinforced with mixture modifications, and the strengthened layer functions as the wearing course for high traffic volume applications. Remixing is classified into single- and multi-stage methods.

The single-stage remixing method was developed in the late 1970s and early 1980s in Europe and Japan. Steady improvements have been made to the equipment since its inception. In the single-stage method, heating softens existing HMA pavement and then the full treatment depth is scarified. Loose material is directed into a mixing chamber consisting of pugmill or a mixing drum where materials are exposed to further heating and drying before transportation to the mix chamber. Treatment depths for the single-stage method are commonly between 25 and 50 mm, with 40 mm being most common.

The multi-stage method, developed in the United States in the late 1980s and early 1990s, is continuously being improved. Once existing HMA pavement is softened in multi-stage remixing, the loose windrowed material passes below the next heating unit or a slat conveyor picks up the material and carries it over the heating bed of the next unit. The sequence continues until the required remix treatment depth is achieved and all loose material is moved into the mixing chamber. Treatment depths for the multi-stage method range between 40 and 75 mm, with 50 mm being most common.

The average temperature of the loosened mix as it enters the mixing chamber should be 120–150 °C. In all equipment setups, the measured amount of rejuvenating agent, admixture, or virgin HMA is added prior to the mixing phase. Although the exact location for mixing materials may change, the location is always at or prior to the mixing chamber. In order to maximize the dispersion time with the aged asphalt binder, the recycling agent is added as early as possible in the process. In order to ensure reliable and exact control over the quantity of additives, a computer-controlled system linked to the forward operating speed of the HIR train adds the admixture and recycling agent to the loosened material.

Admixture and rejuvenating agent application rates vary depending on the existing asphalt condition, mix design requirements, and type of rejuvenating agent, but rejuvenating agent application rates of up to 2 liters per square meter (lit/m^2) are possible. The admixture application rate is limited to a maximum of approximately 30% by weight of the recycled mix or 55 kilograms per square meter (kg/m^2).

In the remixing process, the underlying pavement is usually heated to 50–80 °C and the recycled mix is warmed to 110–130 °C after placement, resulting in a thermal bond between the two layers. In addition, the heating beds that typically extend beyond the scarification width by 100–150 mm heat and soften the adjacent material, thereby providing a thermally-integrated bond between the existing HMA pavement and the recycled mix, and resulting in a seamless longitudinal joint that is resistant to environmental and traffic effects.

Similar to resurfacing, parameters such as ambient temperature, wind conditions, and characteristics of the existing asphalt pavement and its moisture content, as well as potential admixtures and additives added to the mixture, cause remixing production rates to vary from 1.5 to 10.7 m/min. Rubberized chip seals and asphalt pavements containing crumb rubber and polymer-modified asphalt binders have also been remixed by increasing the heating temperature, using different compaction equipment (than that used for HIR in HMA), and exercising more control over emissions without significant difficulties.

1.1.3.3 Repaving

Repaving is used when resurfacing and remixing cannot restore the pavement profile or surface requirements, such as friction number, or when conventional HMA overlay operation is not practical. Repaving is also used when a very thin HMA or specialty mix is required as a wearing course or when more than 50 mm of overlay is necessary in order for the pavement to carry the required traffic load.

Repaving is classified into single-pass and multiple-pass methods. In the single-pass method, existing HMA pavement is recycled while the last unit in the HIR train places an integral overlay of new HMA/specialty mix on the screeded but uncompacted recycled mix, compacting both layers simultaneously. In the multiple-pass method, the last unit in the HIR train screeds and places the recycled mix; another paver follows immediately behind and places the conventional HMA/specialty mix onto the uncompacted recycled mixture. The recycled mix and new overlay material are then compacted as one thick lift.

Although treatment depth in repaving varies based on the method used, it typically ranges from 25 to 50 mm. Treatment depth of existing HMA pavement is also related to the thickness of the integral HMA overlay or specialty mix that must be placed. A combined thickness of integral overlay and underlying recycled mix greater than 75–100 mm can result in increased placement, compaction, and smoothness difficulties. The common practice is to use a 25–50 mm recycled depth and a 25–50 mm integral overlay thickness. The same methods for adding additives and rejuvenating agents in resurfacing are applicable in repaving. The production rate of repaving is very similar to remixing: approximately 1.5–10.7 m/min.

1.2 Problem Statement

The road network in the United States is the largest in the world, with more than 6.4 million kilometers of paved public road (Federal Highway Administration [FHWA], 2010). As of 2010, annual vehicle miles traveled (VMT) was more than 3 trillion in the United States. While HMA pavements comprise 95% of the paved roads in the United States (Washington Asphalt Pavement Association, n.d.), more than half of the nation’s roads are in poor and fair conditions and are in need of maintenance (FHWA, 2009a, 2009b).

In 1987, the Strategic Highway Research Program (SHRP) began to conduct studies to specify, test, and design asphalt materials and mixtures. The new mixture design system, unveiled in 1993, was named Superpave, which stands for Superior Performing Asphalt Pavements. Superpave represents an improved system for specifying the components of asphalt concrete, mixture design and analysis, and HMA pavement performance prediction. The new asphalt binder evaluation system developed by Superpave, performance grade (PG) asphalt binders, has been adopted by most states in the United States and many other countries. Considering that a flexible pavement service life is typically between 15 and 20 years and that Superpave is almost 20 years old at the time of this writing, it is understandable that a majority of original Superpave mixtures have started to reach the end of their performance lives. The struggle now is to assess if the existing distressed pavements can be effectively recycled while meeting performance requirements for a pavement that must remain in service for 10 additional years of service life (Ali, McCarthy, & Welker, 2013).

Research projects pertaining to RAP and application of RAP in the construction of new HMA pavement have been conducted since the 1970s. Recycling pavements, however, took a new form with the introduction of Superpave and its volumetric and mix design requirements. User agencies such as state Departments of Transportation limited the use of RAP to 20% in a given mix design, with a national average of 13%. RAP limitations and steps taken to increase its usage are discussed in detail in the literature review chapter of this document. HMA is thought to be 100% recyclable, and the use of ARA could potentially resolve all performance issues recycled HMA may face in its service life (Zaumanis, Mallick, & Frank, 2014; Mallick, O'Sullivan, Tao, & Frank, 2010). However, ARA potentially adversely affects rutting resistance of the final surface or fails to improve the quality of the final surface. Mixed effects of the ARAs are hypothesized to be more influenced by product application than by product performance (Karlsson & Isacsson, 2006).

HIR technology is increasingly used in Kansas to recycle 70–100% of existing asphalt pavements. However, a comprehensive research project that included mechanical performance and binder consistency evaluations was needed to assess and investigate possible approaches to modify HIR mixture performances.

1.3 Objectives

The main objectives of this research included:

1. Investigating the effects of rejuvenation on critical performance indicators of HIR mixture, such as stiffness, permanent deformation, moisture susceptibility, and resistance to cracking;
2. Studying the miscibility between aged binder and ARA and between virgin binder and aged binder in the presence of ARA;
3. Studying the correlation between the level of mixing, as obtained by scanning electron microscope (SEM) images, energy dispersive X-ray spectroscopy (EDXS), and the exudation droplet test (EDT) and HIR mixture performance, as evaluated by mechanical laboratory tests; and
4. Conducting life-cycle cost analysis on alternative pavement design strategies to evaluate overall long-term economic efficiency of HIR.

1.4 Organization of the Report

This report is divided into seven chapters, including this introductory chapter (Chapter 1). Chapter 2 is a comprehensive literature review of RAP usage in the construction of new pavements, including limitations and solutions. A laboratory experimental plan is discussed in detail in Chapter 3, and performance testing, including asphalt mixture performance testing, asphalt binder rheology, and miscibility assessment, is described in Chapter 4. Chapter 5 provides results and discussion, and Chapter 6 includes pavement maintenance and life-cycle cost analysis. Conclusions and recommendations for better HIR practices in the future are provided in Chapter 7.

Chapter 2: Literature Review

This literature review surveys applicable theory, laboratory experiments, and field investigations related to rejuvenation effects on aged binder qualities in RAP and subsequent rejuvenation effects on mixtures with high RAP quantities, as in HIR. This literature review also includes RAP consumption background, restrictions on increasing RAP quantity, and solutions for RAP restrictions and modifications.

2.1 Background

Almost all paving asphalt cement in use today is obtained from processed petroleum crude oils. A majority of refineries in the United States are located near oceans for required transport supply or are equipped with pipelines from marine terminals. Since a considerable amount of crude oil used in the United States is imported from foreign countries, the global supply of crude oil is essential to the supply of asphalt (Brown et al., 2009). Crude oil prices significantly increased as a result of the 1973 oil crisis, thereby increasing the attractiveness of RAP usage. A field demonstration project carried out in New Jersey in 1979 with 50% RAP determined project performance to be “extremely well.” Problems with pavement performance, production technology, and undesirable emissions in other projects, however, reduced the research and implementation of high RAP mixtures in the late 1970s and early 1980s. Although Superpave mix design was introduced in 1993, it did not include a procedure for incorporating RAP, so state agencies were unwilling to use it in Superpave mixtures. The problem of not having specific instructions for including RAP in the HMA mix design was not addressed until 7 years later in 2000 when the National Cooperative Highway Research Program (NCHRP) Project 9-12 led to the development of the current procedure, American Association of State Highway and Transportation Officials (AASHTO) M 323 (2012), which considers the use of RAP in Superpave mix design (Zaumanis & Mallick, 2015).

Recycling is more demanding than conventional paving regarding mixture design, mechanical mixing, and paving procedures. In addition, material properties are more variable in RAP compared to virgin aggregates and virgin binder. RAP, however, is being used at record level all across the United States. Nevertheless, currently RAP is used in unbound base layers, on road shouders, and rural roads, and a relatively small percentage (13%) is used in construction of new HMA pavements. Using higher proportions of RAP is a step forward in saving non-renewable raw materials and reducing energy consumption for transportation and manufacturing processes (Zaumanis, Mallick, & Frank, 2014; Dony, Colin, Bruneau, Drouadaine, & Navaro, 2013). Considering material and construction costs, it has been estimated that use of RAP provides savings ranging from 14% to 34% for RAP content varying between 20% and 50% (Daniel & Lachance, 2005). While contractors are willing to use higher percentages of RAP, the conflicting results of past studies led state agencies to not allow more than 25% RAP in the mixture (Daniel & Lachance, 2005; Al-Qadi, Carpenter, Roberts, Ozer, & Aurangzeb, 2009; Mogawer, Austerman, Engstrom, & Bonaquist, 2009). RAP mixtures often do not follow expected performance trends of virgin materials. While original bitumen and asphalt mixture properties change during years of service (binder in RAP is stiffer than bitumen used originally in new construction, the proportion of fine and coarse materials changes, and materials of various origins may have been mixed into RAP during processing; Karlsson & Isacsson, 2006), the technology and knowledge for recycling 100% of RAP is already available (Zaumanis, Mallick, & Frank, 2014; Dony et al., 2013). The three main areas of hesitation regarding use of high proportions of RAP in new roadway mixtures include aging of the existing binder in RAP, miscibility of new and aged binder or blending, and aggregate gradation control.

2.2 Restrictions for Increasing RAP Quantity

2.2.1 Asphalt Binder Aging

Asphalt cement is a by-product of the petroleum distillation process. Petroleum crude oil is a plant life transformation that has occurred over millions of years under varied temperatures and pressure conditions. Although all petroleum is basically comprised of hydrocarbons, the nature and composition can vary from one source to another. Asphalt cement, a distilled product from crude oil, has varying chemical compositions and properties based on its crude oil source. Aging increases asphalt binder viscosity and leads to increased stiffness of the asphalt binder. Viscosity can be defined as the resistance to flow of a fluid. Stiff binders are known to be brittle and susceptible to disintegration and cracking failures (McDaniel, Soleymani, Anderson, Turner, & Peterson, 2000; McDaniel, Soleymani, & Shah, 2002; Mohammad, Cooper, & Elseifi, 2011).

Asphalt binder is composed of two main components: asphaltenes and maltenes. Asphaltenes (A) are defined as the fraction of asphalt binder that is insoluble in n-pentane, which serves as a bodying agent. Maltene, the common name for whatever remains in the asphalt binder once the asphaltenes have precipitated, is composed of four bodies (Brown et al., 2009):

1. **Nitrogen base (N)**: components of highly reactive resins that act as a peptizer for strongly associated asphaltenes in the solvating phase.
2. **First acidiffins (A_1)**: components of resinous hydrocarbons that act as a solvent for peptized asphaltenes.
3. **Second acidiffins (A_2)**: components of slightly unsaturated hydrocarbons that also serve as a solvent for peptized asphaltenes.
4. **Paraffins (P)**: the oily constituent of maltenes that acts as a gelling agent for asphalt components.

Asphalt binder aging is primarily associated with a reaction with oxygen in the air and loss of volatiles, leading to increased viscosity and stiffness. A and P asphalt components are the most stable, while components N, A_1, and A_2 are more susceptible to oxidation in descending order. As aging occurs, the ratio of asphaltenes and maltenes changes, leading to increased stiffness and viscosity and decreased ductility of the asphalt binder. In other words, during oxidation the N components more quickly convert to A components than A_1 and A_2 components. The process of oxidation increases the A fraction and reduces the N, A_1, and A_2 components over time. The ratio of chemically active to less reactive components in asphalt binder is an indicator of the oxidation rate. The ratio, which is called the maltenes parameter, is defined as provided in Equation 2.1 (Boyer, 2000; Zaumanis & Mallick, 2015):

$$\textbf{Maltenes Parameter} = \frac{(\mathbf{N}+\mathbf{A_1})}{(\mathbf{P}+\mathbf{A_2})} \qquad \textbf{Equation 2.1}$$

Rheological properties of binder types depend strongly on asphaltene content. If the temperature is constant, the viscosity increases as the concentration of asphaltene increases. The viscosity, however, is influenced by the amount and the shape of asphaltene particles. Asphaltenes are stacks of plate-like sheets comprised of aromatic/naphthenic ring structures. At high temperatures, the size and shape of asphaltenes change due to broken hydrogen bonds that held the sheets/stacks together. As the temperature increases, asphaltene component separation begins and continues until the structure can no longer be divided and the unit sheet of condensed aromatic and naphthenic ring is reached. The broken bonds cause decreased viscosity of asphalt binder with increased temperature. As the hot asphalt binder cools, reassociation between asphaltenes forms extended sheets that interact with other chemical types and other asphaltene stacks to form distinct asphaltene particles. Intermolecular and intramolecular attractions between asphaltenes and other entities result in molecular shape changes that lead to increased viscosities in asphalt binder. Asphaltenes constitute 5–25% of asphalt binder volume depending on the source of crude oil. Therefore, asphalt binders demonstrate unique susceptibilities to aging (Hunter, Self, Read, Gerlis, & Taylor, 2015).

Asphalt binder hardening occurs in two phases: short-term aging and long-term aging. The two phases are detailed in the following sections.

2.2.1.1 Short-Term Aging

The first major change in viscosity occurs when asphalt is mixed with aggregates at temperatures generally ranging from 135 °C to 163 °C and then transported and laid. Aging that occurs during mixing, transportation, and compaction is called short-term aging because it occurs in a couple of hours (short term) from mixing. Short-term aging is caused by the following (Zaumanis & Mallick, 2015; Hunter et al., 2015):

- Oxidation: Asphalt binder oxidizes when it comes into contact with atmospheric oxygen. Polar hydroxyl, carbonyl, and carboxylic groups are formed during oxidation. Large, complex molecules form, increasing asphalt binder hardness and decreasing flexibility. The degree of oxidation in short-term aging is highly dependent on the temperature of mixture and time of mixing. The degree of oxidation doubles for each 10 °C increase in temperature above 100 °C.

- Loss of volatile fractions: The evaporation of volatile components from the asphalt binder is highly dependent on the pavement surface temperature and exposure conditions. Hard asphalt binder used in road construction is relatively involatile, so hardening caused by volatilization in HMA mixture production is rather small.
- Exudative hardening: Exudative hardening occurs when oily components (asphaltenes and resins) exude from the asphalt binder into mineral aggregates. Exudative hardening is a function of the exudation tendency of the asphalt binder and the porosity of the aggregates.

As mentioned, a primary component of the Superpave system is performance-based tests and specifications for asphalt binders. Superpave binder specifications require that the asphalt binder be tested after simulating aging during three critical stages: 1) transportation, storage, and handling prior to mixing with aggregates; 2) short-term aging; and 3) long-term aging in service. The rolling thin-film oven (RTFO) test, which is utilized in Superpave for short-term aging simulation, estimates the mass of volatiles lost during 85 minutes at 165 °C. The mass of volatiles lost is related to the amount of aging that may occur during HMA production and construction. The rotational viscometer (RV) test is used to test the binder once it has undergone short-term aging. The RV test was adopted for Superpave in order to determine asphalt binder viscosity at high construction temperatures (above 100 °C) to ensure that the binder is sufficiently fluid to mix with aggregate at the mixing temperature. Superpave binder specifications limit binder viscosity to 30 g/cm.s at 135 °C (Brown et al., 2009).

2.2.1.2 Long-Term Aging

The age hardening process of HMA pavement continues long after the pavement is laid and opened to traffic. The rate of age hardening decreases once asphalt concrete mixing and placement is complete, but it continues for the first 2–3 years until the pavement reaches its limiting density under the traffic load. After this point, the rate of age hardening in asphalt binder continues to decrease, requiring longer time periods to distinguish changes in rheological properties of the asphalt binder. Increased air voids in the asphalt mixture are known to accelerate age hardening. If the air void content in the laid HMA pavement is higher than the designed air void, water and light can readily penetrate the structure. When asphalt binder is mixed with aggregates in the mixer, the aggregates are expected to be coated with a thin film of asphalt layer 5–15 µm thick. The asphalt binder content or asphalt film thickness of a mixture significantly impacts aging on the road. Thickness of the oxidized asphalt binder in ambient temperature (between 40 °C and 60 °C) is limited to approximately 4 µm. Therefore, minimum asphalt binder film thickness of 6–8 µm is required for acceptable performance of HMA mixtures (Hunter et al., 2015). Age hardening is also influenced by pavement layer position within the road cross section, with the surface layer aging faster than layers underneath.

Figure 2.1 shows a plot of viscosity change versus time at 60 °C for six asphalt mixtures in a study conducted by Kandhal, Sandvig, Koehler, and Wenger (1973). The figure proves that asphalt binders age at unique rates.

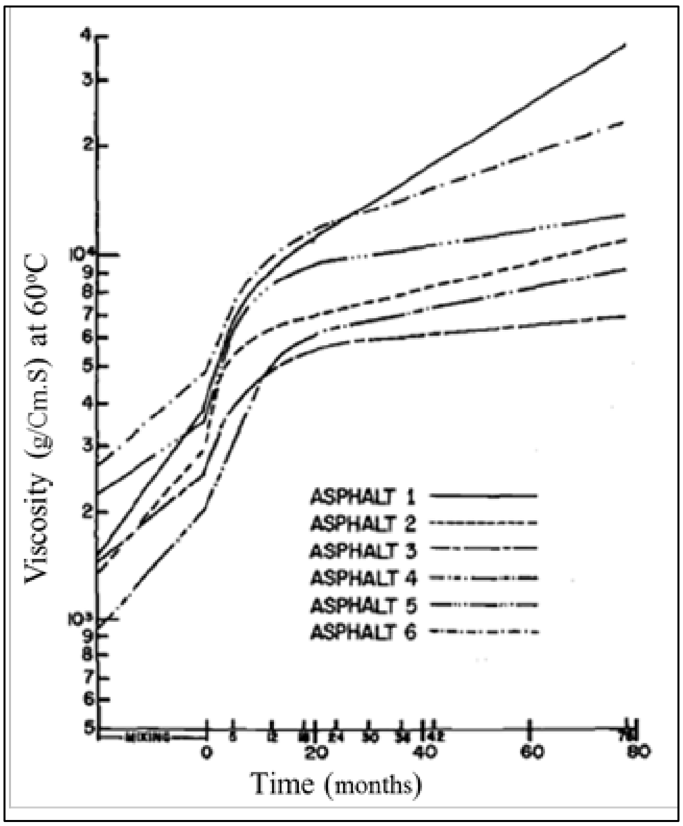


Figure 2.1: Typical Changes of Asphalt Binder Viscosity Over Time
Source: Kandhal et al. (1973)

Long-term aging mechanism has been recognized as a combination of the following factors (Zaumanis & Mallick, 2015; Brown et al., 2009; Hunter et al., 2015):

- Oxidation: Exposure to a constant supply of fresh air allows continued asphalt binder oxidation during years of service. Asphalt binder at the surface of pavement structure, however, oxidizes much faster than asphalt binder in the deep layers. Continuous contact with fresh air also exposes asphalt binder at the surface to heat which, as discussed, accelerates asphalt hardening.
- Polymerization: Polymerization is defined as the combination of like molecules to form larger molecules that cause progressive hardening in the asphalt binder.
- Photo-oxidation: Oxidation of 4–5-μm-thick asphalt binder film that rapidly forms on the pavement surface due to natural ultraviolet (UV) radiation is called photo-oxidation. UV radiation is absorbed into the upper 10 μm of asphalt binder film, and the skin formation (on the asphalt binder) can protect the asphalt layer from further oxidation. However, the thin skin layer is soluble in rain water and can be washed away, exposing fresh binder to oxidation.
- Thixotropy: Thixotropy is defined as progressive hardening caused by formation of a structure within the asphalt binder. Thixotropy, which is attributed to the reorientation of asphalt binder molecules, can be reversed, to some extent, by reheating and working the material. Physical hardening generally occurs on pavements with little or no traffic, and its magnitude depends on asphalt binder composition.
- Syneresis: Syneresis is an exudation reaction in which thin oily liquids are exuded to the surface of the asphalt binder film. When oily liquids are separated from the body of the asphalt binder, the remaining component is hard and brittle.

The design methodology of Superpave results in an asphalt mixture with optimized performance regarding three HMA pavement distresses: rutting, fatigue cracking, and thermal cracking. Rutting, defined as permanent deformation in the wheel path, occurs typically at high temperatures. Fatigue cracking, defined as interconnected cracking with a pattern resembling alligator skin, occurs at intermediate temperature. Low-temperature cracking manifested as equally spaced transverse cracks on asphalt pavement occurs at low temperatures.

The pressure aging vessel (PAV) is the selected device for simulating long-term aging (5–10 years in service) in Superpave. Since the asphalt in service has already undergone short-term aging, the PAV is used to age RTFO residue. After aging in PAV, a dynamic shear rheometer (DSR) is used to characterize the viscous and elastic behavior of aged asphalt binders at high and intermediate service temperatures. DSR measures the complex shear modulus (G^*) and phase angle (δ) of asphalt binders at desired temperature and frequency of loading. G^*, which is defined as the total resistance of asphalt binder to deformation, is composed of elastic and viscous components, with δ being the angle between the two components (Brown et al., 2009). Figure 2.2 illustrates the viscous and elastic components of G* and δ.

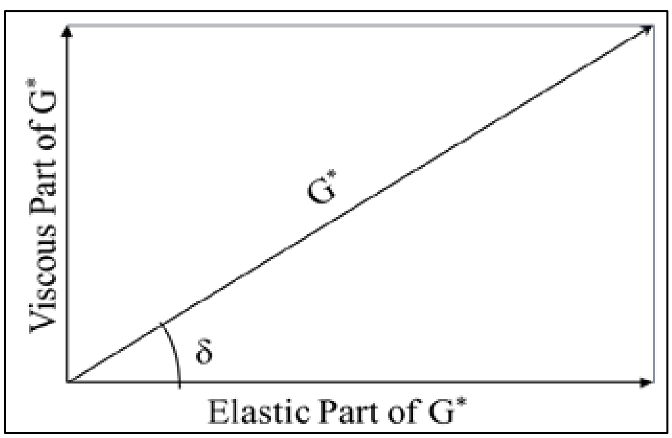


Figure 2.2: Viscous and Elastic Components of G* and δ
Source: Brown et al. (2009)

Temperature and frequency of loading significantly affect G^* and δ. All Superpave DSR tests are conducted at a frequency of 10 rad/sec, which is equivalent to 1.59 Hz. The testing temperature is defined by the specific service temperature to which the asphalt binder is exposed.

A weather database of approximately 7,500 North American weather stations over a 20-year period is used to determine the anticipated high and intermediate temperatures that the asphalt binder will experience in its service.

As mentioned, fatigue cracking is a phenomenon that occurs in intermediate temperatures. Therefore, in order to determine the asphalt binder’s ability to resist fatigue cracking, PAV-aged samples are tested at intermediate design temperature. In order to estimate rutting resistance, asphalt binder is tested in DSR at two states: un-aged (neat) and RTFO-aged. Asphalt binder is tested at its maximum design temperature to predict rutting resistance (Brown et al., 2009). Rutting susceptibility of asphalt binder is not tested on PAV-aged samples because stiff binders are less susceptible to rutting. As a result, in order to make predictions of worst-case rutting scenarios, the asphalt binder is tested on its un-aged and short-term-aged conditions at the highest temperature at which it is expected to serve.

The DSR test parameter has the following three specification requirements (Brown et al., 2009):

1. When un-aged asphalt binder is tested, the minimum value of $G^*/\text{Sin } \delta$ should be 1 kPa.
2. When RTFO residue is tested, the value of $G^*/\text{Sin } \delta$ (the same asphalt binder) should be no less than 2.2 kPa.
3. When PAV-aged asphalt binder is tested, the value of $G^*\text{Sin } \delta$ should be less than 5,000 kPa.

Superpave adopted the bending beam rheometer (BBR) test to test asphalt binder susceptibility to low-temperature cracking. Low-temperature cracks form when stresses accumulate within the asphalt layer due to contraction of the asphalt binder caused by exposure to low temperatures. If the contraction occurs rapidly enough, the amount of accumulated stress can exceed the stress-relaxation ability of the HMA pavement, causing cracks to develop to relieve stress. Thermal cracking can result from one thermal cycle below the critical cracking temperature of the binder, from thermal cycling, and from repeated temperature cycling at temperatures above the critical cracking temperature. BBR uses a transient creep load, applied in the bending mode, in order to load an asphalt beam sample (held at a constant low temperature submerged in water). The test is performed at the lowest temperature expected for the pavement. As binder stiffness increases during aging, the ability of HMA pavements to relieve thermal stress by flow decreases, increasing the propensity for thermal cracking. The age-hardening issue is addressed in Superpave by conducting the BBR test on RTFO- and PAV-aged samples. The two parameters obtained from the BBR test are S(t) and m, which are analogous to G^* and δ in the DSR test. S(t) is defined as the asphalt binder stiffness obtained at time *t*, and m is the slope of the stiffness curve. Numerical limits for these two parameters in Superpave are a maximum of 300 MPa for S(60) and a minimum m value of 0.3, where S(60) is the stiffness of the asphalt binder 60 seconds after loading.

2.2.2 RAP Aggregate Gradation and Properties

The addition of RAP to the HMA mixture affects mixture performance in terms of rutting and cracking because aged binder and old aggregates in RAP have properties that differ from the virgin binder and aggregates. Original characteristics of aggregates used in the existing HMA pavement change during milling and processing operations involved in obtaining RAP. The removal of old pavement requires milling, ripping, and sometimes crushing, which produces an excessive amount of fine aggregates, making it difficult to control volumetric properties of the final mixture, especially when high amounts of RAP are added to the mixture. Furthermore, RAP segregates when it is stockpiled in asphalt plants. Segregation is defined as the nonuniform distribution of coarse and fine aggregates in the stockpile. The finer fraction of segregated RAP contains higher asphalt content (than coarser fraction) due to an increased surface area, which complicates the air void control in the HMA mix (Sabahfar & Hossain, 2015). RAP typically is one of the aggregates used in mix design, and the same specifications and requirements for virgin aggregates are applied to RAP. Previous research has shown that the effect of stiff binder in RAP becomes more pronounced as the percentage of RAP increases in the mix. Although the addition of up to 20% RAP to the HMA mixture has an insignificant effect on mixture characteristics, 40% RAP significantly influences mixture properties (McDaniel et al., 2000). Higher percentages of RAP complicate conformance to Superpave specifications to the point that McDaniel et al. (2002) concluded that RAP usage in HMA mixtures should be restricted to

40–50% because the high fine content of RAP prevents fulfillment of Superpave volumetric requirements.

2.2.3 Blending of Old and Virgin Asphalt Binders

RAP consists of aged asphalt binder and aggregates, and the amount of blending between the aged and the virgin binder is unknown when RAP is mixed with virgin binder. Reliability of the assumption that a uniform mixture can be obtained once the mixing process is complete is also unclear. Total blending, black rock, and actual situation scenarios for blending levels have been considered in the literature. Total blending assumes that the RAP binder blends completely with the virgin binder. Black rock assumes that the RAP binder does not blend with the virgin binder and that RAP remains as a black rock in the mix. The third scenario assumes that the binder in the aged RAP blends with the virgin binder, but not completely. Experience has shown that when small quantities of RAP (less than 20%) are used in the mix, RAP behaves like a black rock and influences mixture volumetric and performance properties via aggregate gradation. When higher quantities of RAP (more than 20%) are used, the black rock analogy is no longer valid since the aged binder in RAP blends with the virgin binder in sufficiently large quantities to significantly affect final mixture properties. In reality, actual blending that occurs between virgin and RAP binders is unknown and the effective contribution of RAP binder to the total binder content of the asphalt mixture is unidentified (Zaumanis, Mallick, Poulikakos, & Frank, 2014; McDaniel & Anderson, 2001).

The current design procedure for HMA recycling assumes that the aged binder fully blends with the virgin binder. Inaccurate assumption of blending can complicate the mix design process and pavement performance expectations. If the aged binder in RAP does not blend with ARA (or blends to a lesser degree than speculated), the actual asphalt content of the recycled mixture is lower than design-specified optimal content. In addition, because the blending does not occur as expected, the viscosity of the final asphalt differs from the predicted viscosity. However, if ARA diffuses into the aged asphalt binder to a higher extent than desired and predicted, the asphalt mixture can become too soft (Zaumanis, Mallick, & Frank, 2013; Xu, Xu, & Ji, 2014). Stiff binder in RAP is expected to positively influence the rutting performance of

HMA mixture while making it more vulnerable to fatigue and thermal cracking. In an HMA plant, RAP is usually mixed with virgin asphalt and aggregates for less than 1 minute. Regardless of the blending situation, laboratory evaluations have shown that although rutting performance typically improves due to the stiffening effect of aged binder, fatigue and thermal performance decline (Mohammad et al., 2011; McDaniel et al., 2002) or become inconsistent (Norton et al., 2013). Others have alternatively concluded that the inclusion of up to 30% RAP can improve the fatigue performance of asphalt mixtures (Huang, Li, Vukosavljevic, Shu, & Egan, 2005).

If aged asphalt binder cannot fully blend with virgin asphalt binder, it forms a coating layer around the aggregate. Long-term aging causes the coating layer to be much stiffer than the virgin binder, resulting in formation of a composite-layered system that contains RAP in the HMA mixture. The composite structure reduces the stress concentration and potentially enhances performance of asphalt mixture (Huang et al., 2005). Figure 2.3 shows the three scenarios for the blending of virgin and aged asphalt binders (Xu et al., 2014).

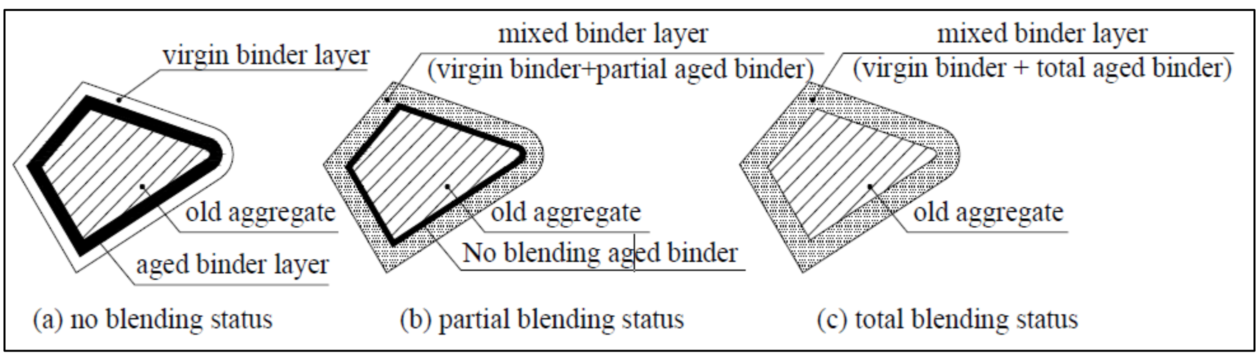


Figure 2.3: Three Scenarios for Blending of Virgin and Aged Binder
Source: Xu et al. (2014)

Blending between aged and virgin binders is time-dependent and continues until a stable blending state is reached. Changes in blending state lead to changes in the performance of pavement containing RAP. Carpenter and Wolosick (1980) studied the resilient modulus and rutting resistance of HMA mixtures containing RAP. Results showed that both parameters were significantly lower 2 weeks after completion of mixing and compaction. These findings were verified by research that showed that the blending of virgin and aged binder or the diffusion of new binder into stiffened old binder can continue for 3–6 months after paving is complete (Xu et al., 2014; Tran, Taylor, & Willis, 2012; Huang et al., 2005; Carpenter & Wolosick, 1980). Consequently, pavement performance indicators that are acceptable at the time of mixing and compaction may not be acceptable several months later, and premature pavement distresses could occur. Figure 2.4 shows changes in the degree of blending over time and the influence it can have on the performance of an HMA mixture containing RAP (Xu et al., 2014).

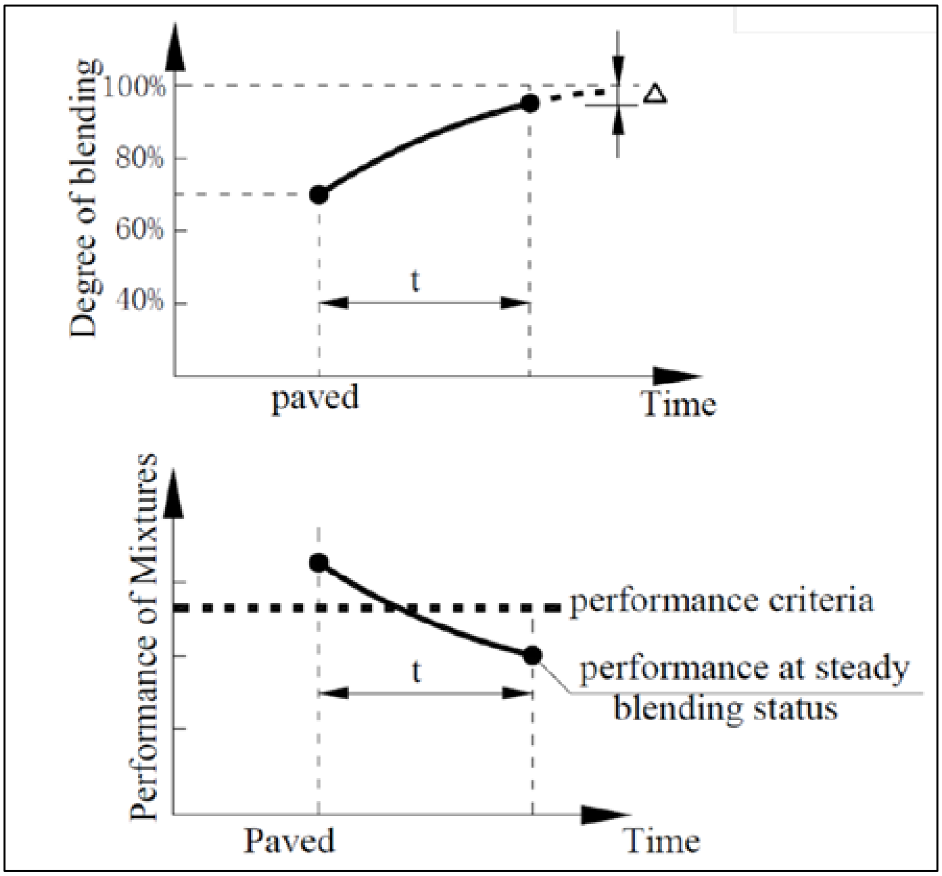


Figure 2.4: Changes in Degree of Blending Over Time and the Influence on Performance
Source: Xu et al. (2014)

Changes in degree of blending and consequences related to long-term pavement performance require blending assessment, as it occurs, between the aged and virgin binders. Measurement methods for evaluating the level of blending are discussed in the following sections.

2.2.3.1 Binder-Marked Method

The binder-marked method measures blending between aged and virgin binders by manipulating molecules in order to detect and recognize those molecules. Molecules can be manipulated by altering the atoms (e.g., neutron radiation can be used to produce C13 or D isotopes) or by slightly changing the chemical composition of the binder. Changing atoms is not popular in asphalt binder studies because it potentially produces unsatisfactory results (Xu et al., 2014).

Lee, Terrel, and Mahoney (1983) used EDXS to track the blending of aged and virgin binder using titanium. Results showed a clear interlayer between the virgin and aged binders. EDXS, which is a chemical microanalysis technique for element identification in material analysis, can be mounted on SEM, transmission electron microscopes (TEM), and scanning transmission electron microscopes (STEM). EDXS uses the primary beam of the microscope to generate characteristic X-rays and then detects X-rays emitted from the sample during bombardment by an electron beam in order to characterize the elemental composition of the analyzed sample (Hollerith et al., 2004).

Many researchers have used image technology to observe the blending process between aged and virgin binders. Nguyen (2009) photographed surfaces of specimens cut from a compacted sample made with RAP and virgin binder. Homogeneity of the mixture was examined using a virgin binder with a color that differed from the RAP binder (i.e., the virgin binder was dyed red using iron oxide pigment). Experimental results indicated that the level of homogeneity and stiffness of the asphalt mixture are significantly influenced by the time, method, and temperature of mixing. Extended mixing time improves the homogeneity and reduces variation in stiffness values of RAP mixtures. In addition, the conclusion was made that even at extreme conditions of mixing prepared in laboratory, which would never exist in the field, RAP binder does not blend completely with the virgin binder (Nguyen, 2009).

Research conducted by Navaro et al. (2012) proposed an observation method using image analysis under white light and UV light to analyze the state of blending, resulting in suggested measurement techniques to quantify the level of blending between aged and virgin binders. Virgin binder in this research was synthetic and fluorescent under UV light, and experiments were carried out on asphalt mixtures made at different temperatures and with different mixing times. Microscopic observations showed that the reduction in RAP clusters (composed of very small particles of RAP) is closely linked to the combined effect of production temperature and mixing time.

Studies have shown that marker selection is essential when various methods of binder marking are used. The marker should not alter the diffusion properties of the marked binder in order to avoid significantly decreasing the reliability of observations made at various conditions (Xu et al., 2014).

2.2.3.2 Difference-Identified Methods

Difference-identified methods evaluate the level of blending between aged and virgin binders by capturing identifiable differences between the binders. The difference-identified method directly observes the blending process between the aged binder and new binder without altering atomic structure or chemical composition of the binder (Xu et al., 2014). Techniques that characterize the chemical composition of asphalt binder include gas chromatography, thermogravimetric analysis, mass spectroscopy, differential scanning calorimetry, nuclear magnetic resonance, gel permeation chromatography, and Fourier-transform infrared spectroscopy. Techniques used to study asphalt binder microstructure include X-ray diffraction, size-exclusion chromatography, and various microscopy techniques such as SEM, TEM, phase-contrast, polarized light, laser scanning, fluorescence, and atomic force microscopy (AFM; Allen, Little, & Bhasin, 2012). Electron microscopy functions similarly to light microscopy except that a focused beam of electrons is used instead of light and magnetic solid optical lenses. When the primary electron beam hits the specimen in SEM, backscattered or secondary electrons are ejected from the specimen. A detector collects the backscattered electrons and converts them into an image of the specimen. Signal intensity is converted into a brighter or darker portion of the image. Therefore, an image obtained by SEM is from the surface or near surface because electron beams with different energy levels can penetrate into various specimen surface depths (Elseifi, Al-Qadi, Yang, & Carpenter, 2008).

Various forms of micro characterization techniques have been used to define blending as it occurs between aged and virgin asphalt binders. Nahar et al. (2013) used AFM to identify the blending zone, proving the presence of a blending zone at the interface of the two types of asphalt binder used in the experiment. However, samples used in the study by Nahar et al. were only exposed to thermal load, and no real-world mechanical blending (mixing) was applied. Based on study results, they concluded that the interface of RAP and virgin binder can be identified. Bowers, Huang, Shu, and Miller (2014) used gel permeation chromatography and Fourier transform infrared spectroscopy and a staged-extraction technique to study blending of virgin and aged binder in RAP. Results indicated a certain degree of blending that was not uniform throughout the binder.

Elseifi et al. (2008) used SEM to study the surface morphology of HMA, and Yao et al. (2013) used SEM to investigate the microstructure of nano-modified asphalt binder. SEM images increased understanding of microstructural changes of the studied asphalt binder by providing high resolution pictures of the various transformation stages occurring within the asphalt binder matrix.

2.2.3.3 Staged Extraction Method

Staged extraction, or progressive extraction, evaluates the degree of blending between RAP and virgin binder. In the staged extraction method, the RAP aggregate that is supposedly covered with asphalt binders of different age and stiffness is soaked in asphalt solvent for a certain period of time. The asphalt binder is extracted by layer from the aggregates, and performance differences among binders of various layers indirectly denote blending between the virgin and aged binders (Xu et al., 2014; Zhao, Huang, & Shu, 2015). Figure 2.5 shows a RAP aggregate with exaggerated distinct layers of surrounding asphalt.

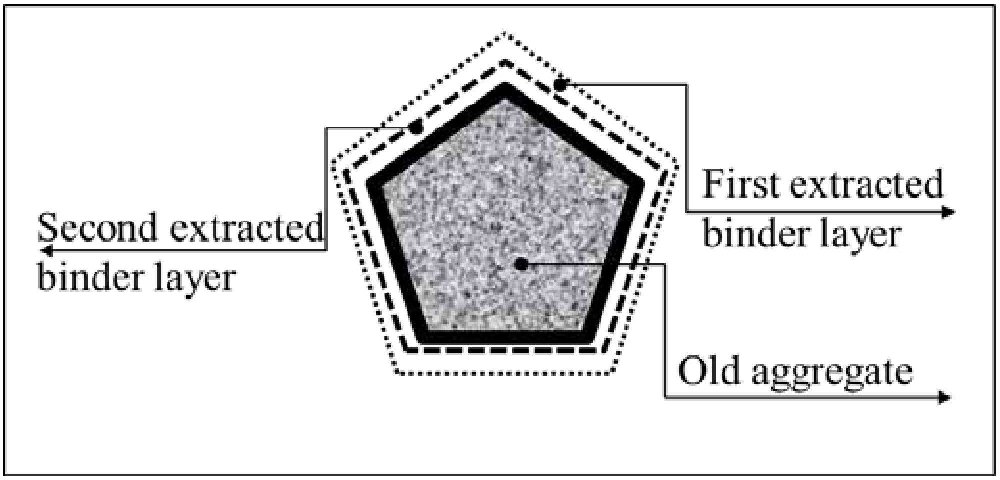


Figure 2.5: Staged Extraction Analogy for a Single RAP Aggregate
Source: Xu et al. (2014)

The staged extraction method, which was first introduced by Zearly in 1979, was adopted and modified by other researchers. Noureldin and Wood (1987) designed a four-layer model (as opposed to the previous two-layer system), concluding that only the outermost layers of RAP blended fully with virgin binder, with the innermost layer being stiffer than the second and third layers.

Use of the staged extraction method is associated with concerns that limit the reliability of test results. The concerns pertain to solvent use and whether the solvent type or remaining solvent in the recovered binder can affect the test results. Also, the method is of no value if the binder that coats the RAP aggregate is not uniformly distributed. RAP material is naturally diverse, but its variability can complicate evaluations made by staged extraction operation. Although it can approximately classify RAP-surrounding binder into several layers, the rules of classifications in the staged extraction method are subjective and depend on researcher experience (Xu et al., 2014; Zhao et al., 2015). Therefore, use of the staged extraction method to study the level of blending that occurs between the aged and virgin binder has been limited.

2.2.3.4 Indirect Performance Measurement Method

Indirect performance measurement methods characterize the blending degree between aged and virgin binders by comparing differences in mixture performance under total blending status and real blending status. The dynamic modulus $|E^*|$ is the performance characteristic commonly used to predict the $|E^*|$ of the mixture if total blending has occurred. The blending degree can be determined via comparison between the predicted and measured $|E^*|$. The degree of blending has also been assessed using indirect tensile strength and unconfined compressive tests on asphalt mixtures (Xu et al., 2014).

The purpose of indirect performance measurement methods is to model performance of the total blending condition and then compare the total blending to what is obtained from laboratory test results. However, the level of blending in this method is only one of many factors that affect mixture performance. For example, aggregate properties, temperature and time of mixing, and changes in mixture volumetric properties can significantly influence mixture performance.

2.2.4 Distresses Associated with High RAP Quantities

When RAP is used in pavement construction, the final surface should show a similar or improved field performance compared to conventional asphalt pavement. Recent research has shown that using high RAP quantities (higher than 25%) in HMA mixture can influence the pavement performance properties in different manners. This section describes common distresses associated with high RAP mixtures and provides current findings regarding the performance characteristics of high RAP mixtures.

2.2.4.1 Cracking

As asphalt binder oxidizes during its service life, it becomes stiffer and more susceptible to fatigue and low-temperature cracking. NCHRP Project 9-12 by McDaniel et al. (2000) concluded that RAP mixture properties at low RAP content do not differ significantly from mixtures with no RAP. In mixtures with high RAP content, however, indirect tensile strength and beam fatigue test results indicated increased stiffness, leading to cracking if no adjustments are made in the mix design. Bennert and Dongré (2010) reported that an assumed linear increase in stiffness based on RAP amount increments can be inaccurate when the mixture contains high proportions of RAP. In such cases, the mix stiffness and magnitude of cracking depend on the degree of blending between the virgin binder and RAP binder (Zaumanis, Mallick, & Frank, 2014).

Mogawer, Austerman, and Roussel (2011) investigated stiffness and low-temperature cracking of plant-produced HMA mixtures with 40% RAP. Results showed that the stiffness of mixtures containing RAP increased by as much as 49% compared to the virgin mixture. However, low-temperature cracking for virgin and RAP mixtures did not differ significantly. Shah, McDaniel, Huber, and Gallivan (2007) observed the same trend in low-temperature cracking. West, Willis, and Marasteanu (2013) evaluated the use of 55% RAP mixes, proving that stiffness measured by dynamic modulus at various temperatures and frequencies increased by 25–60% compared to virgin mixtures. The research also concluded that fracture energy, which is an indicator of fatigue cracking susceptibility of the HMA mixture, was better for virgin mixes compared to mixtures with high RAP content. Critical low-temperature cracking analysis using BBR on mixture beams indicated that mixtures with high RAP content performed similarly to virgin mixtures in similar climates (Zaumanis, Mallick, & Frank, 2014).

Al-Qadi, Aurangzeb, Carpenter, Pine, and Trepanier (2012) and McDaniel, Shah, and Huber (2012) found that mixtures containing 40% or more RAP showed increased fatigue life compared to conventional mixtures. Shu, Huang, and Vukosavljevic (2008) reported similar results. Relationships developed between laboratory test results and test track findings at the National Center for Asphalt Technology (NCAT) suggested that 50% RAP should demonstrate better fatigue performance than the virgin control mix (West et al., 2012). The NCAT test track results also indicated that the increased stiffness of mixtures with high RAP content can reduce critical tensile strains in the pavement structure, thereby benefitting the structural design of perpetual pavements (Zaumanis, Mallick, & Frank, 2014).

2.2.4.2 Moisture Susceptibility

Moisture susceptibility of an asphalt mixture is generally evaluated by comparing the tensile strength of conditioned samples (exposed to one cycle of freeze and thaw) to the tensile strength of unconditioned samples (not exposed to moisture). The higher the tensile strength ratio

(TSR), the less susceptible the mixture is to moisture damage. Minimum TSR for asphalt mixtures in Kansas is 80%; the mixture is unacceptable if the minimum requirement is not met.

Mixtures with RAP are expected to be less susceptible to moisture because RAP aggregate is covered with a layer of asphalt. However, asphalt binder coverage is just one factor that influences asphalt binder behavior in the presence of moisture. For example, if the original recycled pavement demonstrated weak adhesion between aggregate and asphalt binder, the problem is likely to reoccur if adhesion additives are not added to the new (re-made with RAP) HMA mixture. Production technology can also significantly affect the stripping performance of asphalt pavements. For example, insufficient blending of RAP and virgin binder or a low discharge temperature has been shown to increase asphalt mixture susceptibility to moisture damage (Zaumanis, Mallick, & Frank, 2014).

Results of moisture susceptibility evaluations of mixtures containing RAP are contradictory. Mogawer et al. (2012) showed that the moisture susceptibility of mixtures containing 40% RAP was equal to or better than the moisture susceptibility of virgin mixtures. The authors also noted that moisture damage resistance improved as RAP content in the mixtures increased. Al-Qadi et al. (2012) observed a similar trend. However, results obtained by Elwardany and Daniel (2011) confirmed that the potential for moisture damage of RAP mixtures generally is not greater than the potential for moisture damage of a conventional HMA. The NCHRP 9-46 study by West et al. (2013) evaluated moisture susceptibility of RAP mixtures with various binder grades, multiple aggregates, and RAP content. Several mixtures with high RAP content failed to pass the minimum requirement of 80% TSR, but the addition of an antistripping agent improved the performance to the required level. However, the conditioned and unconditioned tensile strength of mixtures with high RAP content always exceeded tensile strengths of virgin mixtures. Researchers suggested that the sole use of TSR to assess moisture damage may be misleading, and recommended a certain threshold of tensile strength (Zaumanis, Mallick, & Frank, 2014). Sabahfar and Hossain (2015) studied two sources of RAP, with RAP content changing from 20% to 40%. TSR results showed a decreasing pattern of TSR with RAP increments in the mixture for both sources, with only one RAP source passing the minimum requirement of 80% TSR.

2.2.4.3 Permanent Deformation

Permanent deformation, or rutting, is not a distress associated with RAP mixtures since a stiff binder is expected to improve rutting performance. Adding an improper amount or improper type of ARA, however, can create rutting issues on HMA pavements containing high RAP contents. Studies conducted by Carpenter and Wolosick (1980) and Noureldin and Wood (1987) showed that ARA (or soft virgin binder) continues to penetrate (diffuse) in the aged binder film long after pavement placement. The dominant effect from the soft outer layer may lead to permanent deformation in early stages of pavement life until equilibrium is reached.

West et al. (2012) showed that mixtures paved at the NCAT test track with 45% RAP demonstrated excellent rutting performance, even when a soft asphalt binder was used. Mogawer et al. (2012) also proved that plant-produced HMA mixtures containing 40% RAP showed excellent rutting resistance. Research by Tran et al. (2012) found no detrimental effect of 12% rejuvenating agent (by mass of the aged asphalt content) on the performance of laboratory-produced HMA mixtures with 50% RAP (Zaumanis, Mallick, & Frank, 2014).

2.3 Solutions for RAP Restrictions and Modifications

As mentioned, asphalt binder aging, RAP aggregate properties, and uncertainties regarding the level of blending that occurs between the virgin and aged binders are the primary reasons high proportions of RAP are not used in HMA mixtures. The following section discusses the probable solutions for these issues.

2.3.1 Rejuvenation of Aged Binder

Stiffened aged binder is the main obstacle preventing increased RAP content of HMA mixtures because mixtures with high RAP content are perceived to be susceptible to fatigue and thermal cracking. ARAs are commonly used to reduce binder viscosity and to attain desired mixture workability by recovering necessary performance properties of the aged binder. ARA is a hydrocarbon product with selected physical characteristics that can restore required characteristics (maltenes parameter) of oxidized asphalt binder. Recycling agents are also referred to as softening agents, soft asphalt, recycling oil, or aromatic oil. In addition to requirements for the chemical composition of ARA (restoring necessary components of the aged binder), it must also have a high flash point, be easily dispersed, demonstrate low volatile loss during hot mixing, resist hardening, and be uniform from batch to batch (O'Sullivan, 2011). Successful use of an ARA should reverse the RAP binder aging process, restore asphalt binder properties for another service period, and make the RAP binder effectively available to the mix (Zaumanis et al., 2013). In other words, the aged binder in RAP must be rejuvenated so that it obtains chemical and rheological characteristics similar to virgin binder in order to participate in the HMA production process. ARAs restore the original ratio of asphaltenes to maltenes by increasing the peptizing power of the maltene phase (Dony et al., 2013).

Industrial process oil, soft asphalt binders, asphalt flux oil, lube stock, and slurry oil are examples of rejuvenating agents used in asphalt recycling. Industrial process oils (lubricating and extender oils) are commonly used due to their high proportion of maltene constituents, which restores the rheological properties of the oxidized RAP binder. As asphalt recycling increases in popularity, new products have been developed in the United States that can provide maltenes without an aromatic content in order to eliminate environmental concerns associated with use of oil-based products. Products such as Hydrogreen are produced from plant materials and claimed to be 100% green (Association of Asphalt Paving Technologists, 2011). Hydrogreen mimics the maltenes phase of the asphalt binder and supplements the maltenes component to produce the performance effects of rejuvenation, asphaltenes dispersion, and viscosity reduction, as well as improvement in low-temperature ductility. However, the introduction of new products prompts consideration of the compatibility of the rejuvenator and the aged binder. This compatibility should be sufficiently high to ensure rejuvenator diffusion and binder restoration (O'Sullivan, 2011).

Carpenter and Wolosick (1980) categorized the mechanism or diffusion process of an ARA into four steps. In the first step, the rejuvenator forms a low-viscosity layer around the RAP aggregate coated with highly aged binder. In the second step, the rejuvenating agent begins to penetrate into the aged binder layer, resulting in a decreased amount of raw rejuvenator coating the particles and softening the aged binder. The aged binder consumes the rejuvenator in the third step, and penetration continues when viscosity of the inner layer decreases and viscosity of the outer layer gradually increases. In the fourth step, equilibrium is reached for a majority of the recycled binder film once sufficient time has passed (Im, Zhou, Lee, & Scullion, 2014).

Although use of ARA has obvious economic and environmental benefits (by allowing more RAP in HMA mixtures), some state agencies are reluctant to allow the use of ARA due to potential rutting damage ARAs can cause by excessively softening the asphalt binder. If blending between ARA and the aged binder is incomplete, a microlayer with low stiffness can form on the RAP surface, eventually leading to premature rut damage. Rejuvenators should provide short-term and long-term properties required for a pavement (Zaumanis, Mallick, & Frank, 2014).

In the short term, ARA should rapidly diffuse into the RAP binder, mobilize the aged, stiffened binder, and produce a uniform coating. A rejuvenating agent should soften the RAP binder to produce a workable mixture that can be easily placed and compacted to the required density without producing harmful emissions. A majority of the diffusion process should be completed before allowing traffic on the pavement in order to avoid friction reduction and susceptibility to rutting. In the long term, ARAs should rejuvenate chemical and physical properties of the aged binder and maintain stability for an additional service period. Adjustments in binder rheology should reduce asphalt binder susceptibility to fatigue and low-temperature cracking. The binder also should not become excessively soft, thereby making the pavement structure susceptible to rutting. The new binder that is a combination of the aged binder and the rejuvenating agent (or soft binder) should provide sufficient adhesion and cohesion in the mixture to prevent moisture damage and raveling (Zaumanis, Mallick, & Frank, 2014).

Although identical terminology is often used, softening the aged binder and rejuvenating the aged binder are distinctive processes. Softening agents lower the viscosity of the aged asphalt binder, whereas ARAs restore physical and chemical properties of the aged binder (Karlsson & Isacsson, 2006).

In the 1970s, the Air Force Weapons Laboratory, Naval Civil Engineering Laboratory, and the United States Navy conducted studies to evaluate the effect of the ARA Reclamite on performance characteristics of asphalt pavements. All three studies concluded that the rejuvenating agent improved the performance of asphalt concrete pavement (Karlsson & Isacsson, 2006). In 1976, the Army Corps of Engineers evaluated five ARA products. After 4 years in service, three of the ARAs softened the aged binder and significantly decreased its viscosity, but two other rejuvenating agents hardened the aged binder (Karlsson & Isacsson, 2006). Asli, Ahmadinia, Zargar, and Karim (2012) used waste cooking oil as an ARA, demonstrating rejuvenation of the aged asphalt binder and similar final physical properties to the original asphalt binder. Dony et al. (2013) studied three ARAs: aromatic oil, vegetable oil, and traditional soft-grade asphalt binder. Results of their study showed that, for high RAP content, RAP binder can be rejuvenated using all three ARAs. They also determined that the behavior of rejuvenated binders may vary during short-term and long-term stages of aging.

Zaumanis, Mallick, Poulikakos, & Frank (2014) studied the performance of six ARAs with 100% RAP: waste vegetable oil, waste vegetable grease, organic oil, distilled tall oil, aromatic extract, and waste engine oil. The dosage of each ARA was 12% by mass of the aged asphalt content. Five out of the six ARAs successfully reduced the binder performance grade (PG) to the desired level (waste engine oil failed), and all ARAs showed excellent rutting resistance and longer fatigue life compared to the virgin mixtures. Most ARAs also lowered critical cracking temperature compared to the virgin mixtures. During HIR production, RAP is typically mixed with ARA (occasionally with virgin aggregates and virgin binder) for a short mixing time (often under 1 minute). The aged asphalt binder is expected to attain necessary viscosity, blend with the ARA, and mobilize. In order for the entire HIR process to be completed in the short mixing time, a sufficient amount of ARA should diffuse into the aged asphalt in order to restore its properties to the required level. Blending and diffusion between RAP and ARA is a function of RAP source and its properties, production temperature, properties of ARA, and mixing time and technique.

Inaccurate assumption of blending can complicate the mix design process and pavement performance expectations. If the aged binder in RAP does not blend with ARA (or blends to a lesser degree than speculated), the actual asphalt content of the recycled mixture is lower than design-specified optimal content. In addition, because the blending does not occur as expected, the viscosity of the final asphalt differs from the predicted viscosity. However, if ARA diffuses into the aged asphalt binder to a higher extent than desired and predicted, the asphalt mixture can become too soft (Zaumanis et al., 2013; Xu et al., 2014). The mixed effects of ARAs on final pavement qualifications are considered to be more influenced by product application than product performance. Therefore, in addition to meticulous selection of ARA based on RAP properties, consideration of mixing time and temperature is necessary to ensure short-term and long-term properties for an HIR pavement (Karlsson & Isacsson, 2006; Zaumanis et al., 2013).

2.3.2 RAP Aggregate Fractionation

Original characteristics of aggregates used in existing HMA pavement changes during milling and processing operations for RAP production. The removal of old pavement requires milling, ripping, and sometimes crushing, which produces an excessive amount of fine aggregates, making it difficult to control volumetric properties of the final mixture, especially when high proportions of RAP are used (Shannon, Lee, Tang, Williams, & Schram, 2013).

Fractionation is a process in which RAP is separated into at least two sizes, typically one coarse and one fine fraction, in order to ensure consistency in the RAP aggregate. Previous research has shown improved retention of mixture stiffness with fractionation (Colbert & You, 2012), enhanced performance (higher fractured energy; Norton et al., 2013), and efficiently controlled volumetric properties (Shannon et al., 2013). Sabahfar and Hossain (2015) showed that fractionation did not significantly improve moisture susceptibility or significantly affect the stiffness of mixtures with high RAP content. However, they did observe significant improvements in rutting performance of fractionated RAP mixtures.

2.4 Summary

Using higher proportions of RAP in construction of new pavement structures is a step forward in meeting the needs for saving non-renewable raw materials and reducing energy consumption for transportation and manufacturing processes. While contractors are willing to use higher percentages of RAP, the conflicting results of past studies were not assuring for state agencies to allow more than 25% RAP in the HMA mixture and high percentages of RAP are not commonly used in practice. Three main areas of hesitation in using higher proportions of RAP in constructon of new pavement structures (aging of the existing binder in RAP, blending of new and aged binder, and aggregate gradation control) and HMA pavement distresses associated with high RAP quantities in the mixture were discussed in detail in Section 2.2. Available solutions for RAP restrictions (rejuvenation of aged binder and RAP aggregate fractionation) were discussed in Section 2.3.

Considering the conflicting test results of studies conducted on cracking and moisture susceptibility performance of HMA mixtures with high RAP contents, the argument of the safe RAP quantity or the best way of modifying its quality has not settled yet. Nonetheless, it is believed that RAP is 100% recyclable and some researchers would argue that the technology and knowledge for recycling 100% of RAP is already available.

Chapter 3: Laboratory Experimental Plan

3.1 Introduction

The objective of this research was to investigate the effects of rejuvenation on HIR performance characteristics by assessing critical performance indicators of asphalt mixtures including stiffness, permanent deformation, moisture susceptibility, and cracking resistance. A two-step experimental program was designed that included measurements of properties of the HIR mixture and rheological properties of the extracted binder. In addition, the level of blending that occurs between the new and aged binder in presence of ARA was investigated using SEM, EDXS, and EDT. Performance indicatory tests included the Hamburg wheel-tracking device (HWTD), dynamic modulus, flow number, Texas Overlay (OT), moisture susceptibility (KT-56), and thermal stress restrained specimen test (TSRST). In order to evaluate the temperature susceptibility of HIR mixtures, the OT test was conducted at normal temperature (21 °C) and at low temperature (4 °C). The TSRST was also used to assess the low-temperature characteristics of HIR mixtures. Binder rheological studies included DSR and BBR tests. The EDT was used to evaluate the level of uniformity in the extracted binder, and miscibility of the virgin and aged binders was investigated using SEM images and EDXS. Figure 3.1 illustrates the research plan flowchart.

3.2 Sample Collection

HIR mixtures for this study were obtained during road construction that utilized HIR. Sample HIR materials were collected from in front of and behind the HIR paver. Material collected in front of the paver was not mixed with ARA, but material collected behind the paver was already mixed with ARA and formed the final pavement layer after compaction. Samples were obtained from three projects in north central, southwest, and southeast Kansas. Projects were selected based on geographical distribution and feasibility of sample collection during construction of the HIR layer. Figure 3.2 illustrates sample collection at the locations in Kansas. The southwestern project (US-56) contained large proportions of sand and crushed gravel, the north central project (K-14) contained crushed limestone, and the southeast project (US-59) contained very hard, crushed limestone. Since no ARA was added to samples obtained in front of the paver, those samples were designated as “w/o ARA,” while samples obtained from behind the paver were mixed with ARA and labeled “with ARA.” Figure 3.3 shows the sample locations in north central, southwest, and southeast Kansas.

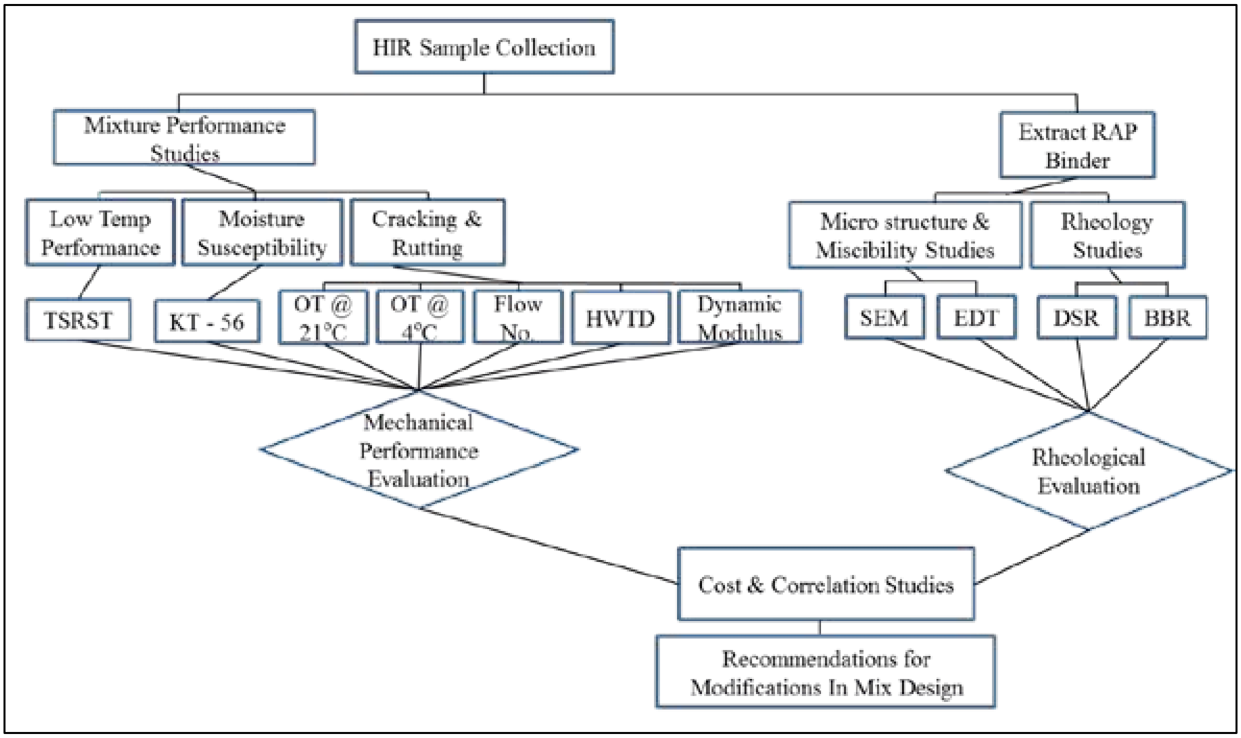


Figure 3.1: Research Plan Flowchart

Figure 3.2: HIR Sample Collection in Front of (Left) and Behind (Right) the Paver

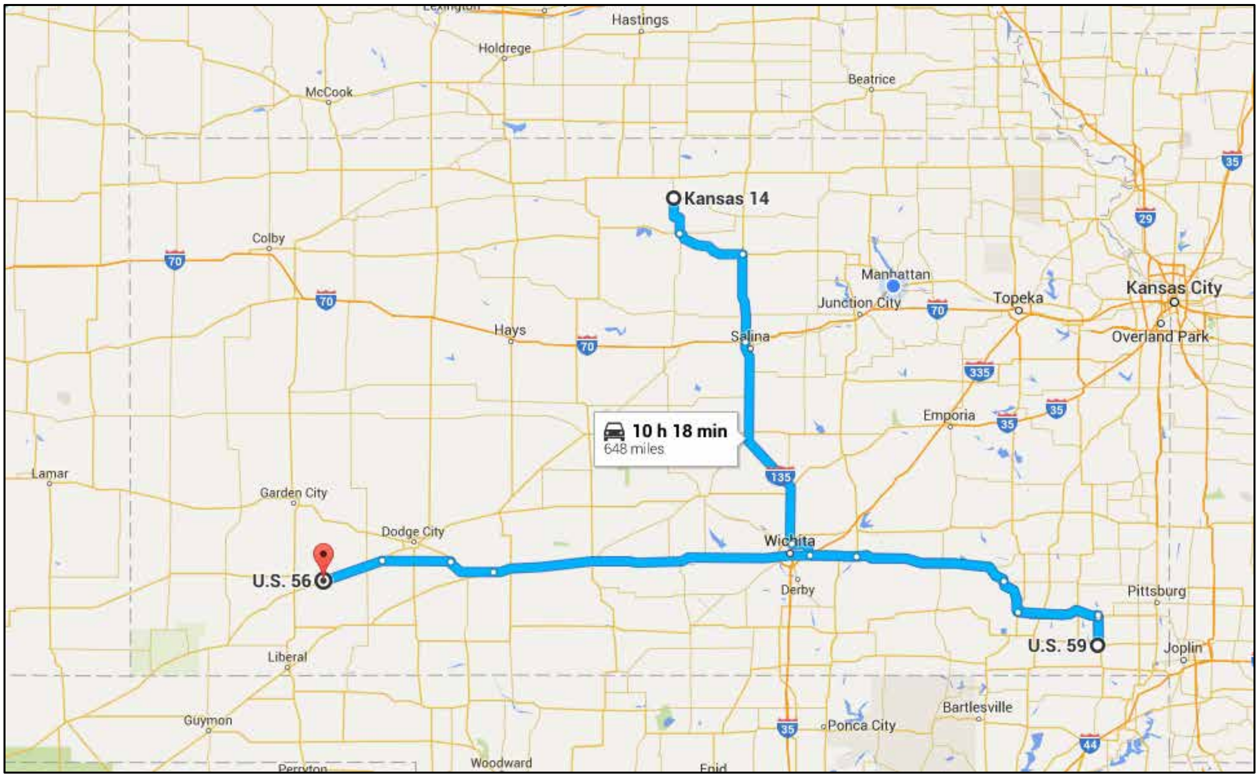


Figure 3.3: Location of Collected HIR Materials in Kansas

3.3 HIR Mixture Properties and Sample Preparation

Collected loose materials were stored in the laboratory. In order to prepare samples for the performance tests, the loose mix was heated to temperatures ranging between 132 °C and 138 °C based on mix design criteria and compacted in a mold with a diameter of 150 mm to a desired height at 7% air voids using the Superpave gyratory compactor (SGC). Maximum theoretical specific gravity (G_{mm}) of the loose mix and bulk specific gravity (G_{mb}) of the compacted specimens were obtained to ensure achievement of 7% target air void (% air voids = 100 - G_{mm}/G_{mb}*100). This air void content was selected to replicate the in-place density of an HIR mixture. Compacted samples were cored and trimmed to prepare the required specimens for the mechanical tests. Asphalt content of the HIR mixture, true binder PG of the rejuvenated asphalt binders, and the AADTs of the three locations are provided in Table 3.1. US-56 and US-59 are non-interstate major travel corridors and carry higher annual average daily truck traffic (AADTT) than K-14. Asphalt binder was extracted from HIR mixtures via the solvent extraction test method, using toluene as solvent according to ASTM D2172 (2011).

Technical difficulties associated with determining the high side of the PG grade of the RAP binder prevented acquisition of complete PG grades for K-14 and US-56 HIR mixtures without ARA. The technical difficulties were not caused by RAP binder since, based on PG grades of the rejuvenated binders, US-59 had the stiffest binder but its high-side PG grade was determined. In order to provide a common property of all recovered binders, Figure 3.4 illustrates δ, |G*|, and |G*|/sinδ obtained at 58 °C from a dynamic shear rheometer (DSR) test conducted on non-aged extracted binder. More details regarding δ, |G*|, |G*|/sinδ, and the DSR test can be found in Brown et al. (2009) and Chapter 2 of this report.

Optimum ARA emulsion content was chosen to be 1.25% (by weight of the mix) for the required PGs, translating into an application rate of 1.45 ± 0.2 lit/m^2. Optimum emulsion content was chosen based on rutting and cracking resistance of the mixture as assessed by the asphalt pavement analyzer (APA) and indirect tension tests (IDTs), with restrictive limit of 1.25% by weight of the mix. As shown in Table 3.1, the binder PG of mixtures with ARA was far from 70-28, the desirable binder PG for surface mixtures (within a depth of 100 mm) in most locations in Kansas. Although more than 1.25% ARA can be allowed, if decided that field conditions warrant the increase, KDOT ARA application rates are commonly limited to 1.25% by weight of the mix.

Ergon, Inc., in Jackson, Mississippi, provided Reclamite as the ARA for all three locations (on US-56, US-59, and K-14 highways). Reclamite, a polymer-modified emulsion with a naphthenic oil base formulated from the same light oils and resins that are volatile fractions of asphalt binder, restores the asphaltene/maltene balance of an aged asphalt mixture. Specifications and results of quality control tests conducted on Reclamite are provided in Table 3.2.

Table 3.1: Asphalt Binder and Aggregate Properties

Location	US-56		US-59		K-14	
	W/o ARA	With ARA	W/o ARA	With ARA	W/o ARA	With ARA
% Asphalt content (solvent extraction)	5.92	6.51	5.53	6.07	4.96	5.21
Binder PG	*-12	85 -18	104-10	94-15	*-5	86 -13
AADT	1,870		2,040		1,180	
AADTT	545		300		240	

*Technical difficulties in obtaining binder high side PG

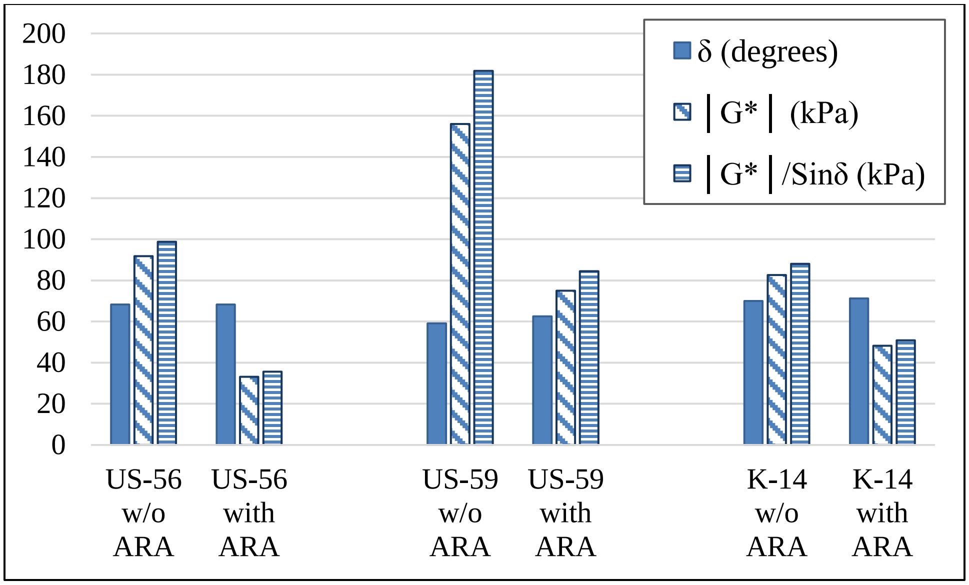


Figure 3.4: DSR Test Results at 58 °C for Non-Aged Extracted Binder of HIR Mixtures

Table 3.2: ARA Quality Control Test Results

Test Type	Specification		Test Results
	Minimum	Maximum	
Asphaltenes, % Max	-----	25	18.09
Elastic Recovery at 4 °C	60	-----	75
Oil Distillate by Volume of Emulsion, %	-----	2.00	0.5
Penetration at 4 °C, 100g, 5 sec	50	150	96
Residue by Distillation, 177 °C, 20 min hold	60	-----	63.3
Saybolt Viscosity, 25 °C, SSf	15	100	30.1
Sieve Test, %	-----	0.10	0.0
Storage Stability, 24 hrs %	-----	1.00	-0.6

Chapter 4: Performance Testing

4.1 Asphalt Mixture Performance Testing

As mentioned, asphalt mixture performance tests conducted in this study included HWTD, dynamic modulus, flow number, OT (at 21 °C and 4 °C), moisture susceptibility (KT-56), and TSRST. The selected tests are discussed in the following sections.

4.1.1 Hamburg Wheel-Tracking Device Test

The HWTD test is currently widely used to predict rutting and stripping potential of asphalt mixtures (Hafeez, Ozer, & Al-Qadi, 2014). The Hamburg wheel tracker manufactured by PMW, Inc., of Salina, Kansas (now Troxler, Inc.), was used in this study (Figure 4.1). This device, which can test two specimens simultaneously, is operated by rolling a pair of steel wheels across the surface of specimens submerged in a water bath at 50 °C. The wheels have diameters of 204 mm and widths of 47 mm. The device operates at approximately 50 wheel passes/min, and the load applied by each wheel is approximately 705±22 N. Specimens used in this test were compacted to 7±1% air voids using an SGC. The specimens were 150 mm in diameter and 62 mm in height. Rut depth was measured automatically and continuously at 11 points along the wheel path of each sample with a linear variable differential transformer (LVDT) with an accuracy of 0.01 mm. HWTD automatically stops the test if the preset number of cycles is reached or if the rut depth measured by the LVDTs reaches 20 mm for an individual specimen. Figure 4.2 shows HWTD test samples before and after test completion when maximum rut depth is reached.

Figure 4.1: HWTD Test Setup

Figure 4.2: HWTD Samples Before (Left) and After (Right) Completion of Test

4.1.2 Dynamic Modulus Test

HIR mixture resistance to permanent deformation and fatigue cracking can be characterized by measuring dynamic modulus $|E^*|$ and phase angle (δ). Dynamic modulus defines a relationship between stress and strain for a linear viscoelastic material under sinusoidal loading that can be used in the Mechanistic Empirical Pavement Design Guide (now AASHTOWare Pavement ME Design) to model pavement performance (Hafeez et al., 2014).

Dynamic modulus does not provide direct estimation of cracking potential of HMA mixture, but it is commonly used in asphalt mixture stiffness evaluation. The addition of ARA was expected to soften the HIR mixture and lower the $|E^*|$. Stiff mixtures are commonly associated with improved rutting and declined fatigue and thermal cracking performances (Mogawer et al., 2015). The dynamic modulus test was used in this study to assess HIR mixture stiffness as influenced by stiff binder in the mixture or changed due to added ARA. Using an asphalt mixture performance tester (AMPT) machine, dynamic modulus tests were conducted on specimens cored and trimmed to have diameters of 100 mm and heights of 150 mm from SGC compacted samples with diameters of 150 mm and heights of 170 mm. The original samples were fabricated at an air void level of 8.5±1% using SGC to obtain an air void level of 7±1% in dynamic modulus cores (the air void is 1%–1.5% lower in cores). Figure 4.3 shows a sample fabricated in the SGC and a dynamic modulus test sample cored and trimmed from it. The range of dynamic load in the dynamic modulus test was 10–690 KPa, with higher load assigned for lower test temperatures. The adjusted dynamic load imposed axial strains between 50 and 150 microstrains in order to keep the deformation in the range of elastic behavior of asphalt mixture.

Figure 4.3: Original and Transformed Dynamic Modulus Test Samples

In the dynamic modulus test, three LVDTs were attached to the sample for axial deformation data collection, providing an estimated limit of accuracy of 13.1%. Figure 4.4 shows specimen setup and LVDT connections.

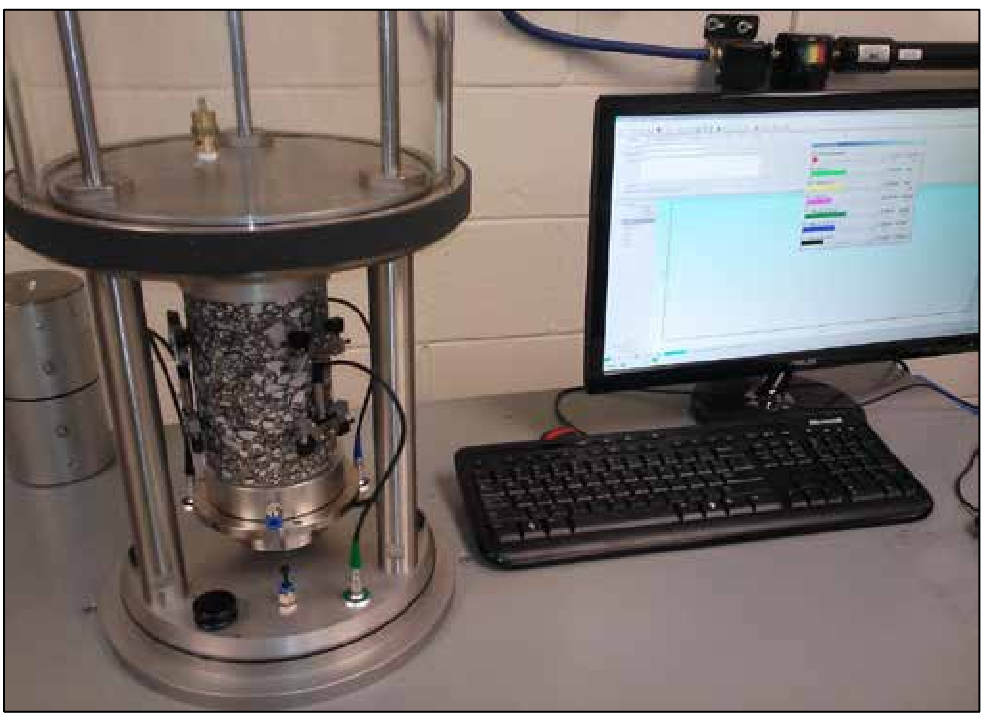


Figure 4.4: Dynamic Modulus Sample Setup in AMPT Machine with Attached LVDTs

4.1.3 Flow Number Test

In addition to the dynamic modulus and flow time tests, the flow number test is one of the tests identified in NCHRP Project 9-19 as a simple performance test related to the rutting resistance of HMA mixtures. Flow number, defined as the number of load pulses when the minimum rate of change in permanent strain occurs during the repeated load test, is determined by differentiation of permanent strain versus the number of load cycle curves (Bhasin, Button, & Chowdhury, 2003). The flow number test differs from the flow time test in loading. In the flow number test, a dynamic load is applied and periodic recovery periods are provided for the specimen. The flow time test measures the viscoelastic response of an HMA specimen under a static stress level.

The flow number test applies repeated dynamic load in a sinusoidal wave for 0.1 second, followed by a dwell or rest period of 0.9 second. Based on the test temperature, the stress level is selected to capture creep characteristics of all mixtures. The flow number test in this study was conducted at a temperature of 21 °C. As illustrated in Figure 4.5, flow number corresponds to the minimum value of the rate of change of axial strain when plotted against the number of cycles on a log-log scale. Flow number was found to have a good correlation with field rutting; the higher number of cycles in the flow number test implied less rutting potential (Bhasin et al., 2003). The flow number test sample preparation process is identical to the dynamic modulus test. Three replicates were tested for each HIR mixture.

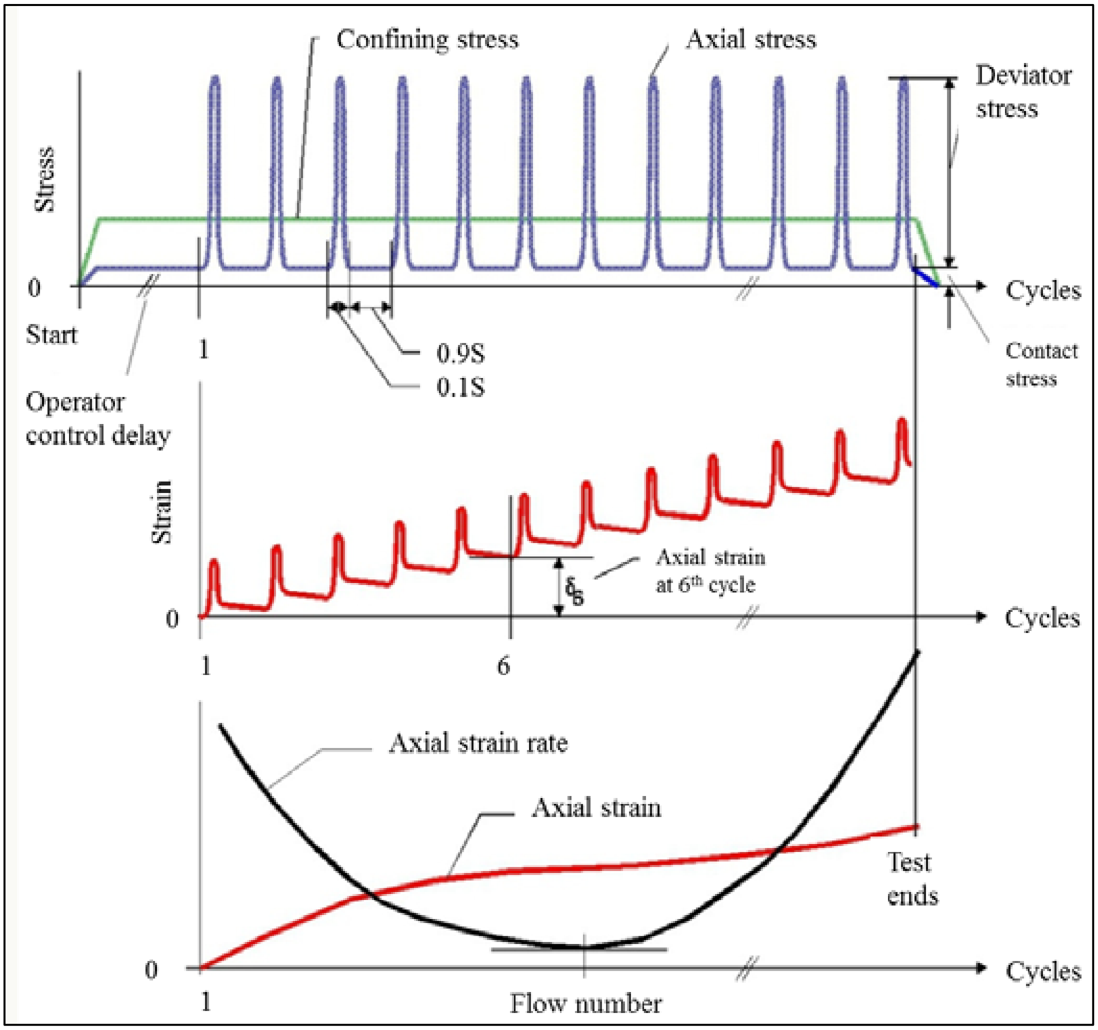


Figure 4.5: Stress Application and Timing in the Flow Number Test

4.1.4 Texas Overlay Test

Asphalt pavement experiences repeated loading, unloading, and rest periods due to traffic loads and environmental conditions. Crack initiation at the bottom of the asphalt pavement layer and its propagation to the top of the layer (and vice versa) is a dominant distress mechanism in HMA pavement structures. Reflection cracking is generally defined as propagation of an existing crack or joint pattern from existing pavements to new overlays (Koohi, Luo, Lytton, & Scullion, 2013; Han, Gautam, Pokharel, & Parsons, 2013). Reflective cracking typically initiates at the bottom of the overlay and propagates through the asphalt pavement layer to the surface. Reflective cracks are typically generated by the bending and shearing action of traffic loads.

Asphalt pavement cracks when it cannot withstand tensile and shear stresses created by vertical and horizontal movements of cracked or jointed surface underneath (Zhou, Hu, & Scullion, 2006; Li, Oh, Naik, Simate, & Walubita, 2014).

Various approaches have been used to assess cracking resistance of HMA mixtures, including conventional fatigue crack analysis such as the flexural bending beam fatigue test (FBBFT), IDT, and the semicircular bending (SCB) test. However, most test methods have limitations because they are semi-empirical and often time-consuming. Field data required for validation of these test methods are scarce, and some test protocols have only recently gained popularity and are not fully developed (Koohi et al., 2013; Li et al., 2014). For example, sample preparation in FBBFT requires a rolling wheel or linear kneading compactor, which is more complicated than a gyratory compactor.

Unlike FBBFT, IDT, and SCB tests, the OT test directly models crack growth and propagation using asphalt specimen thickness. The simplicity of the OT test has led to its use as an index test for fatigue since crack growth is the active mechanism in reflection cracking and fatigue. Zhou et al. (2006) proposed a balanced mix design concept with HWTD and OT tests for rutting and cracking evaluations, respectively (Koohi et al., 2013). Research findings by Zhou et al. (2006) indicated that OT test results were in good correlation with field performance of dense-graded HMA mixtures in Texas. Their study also revealed that the OT test was promising for evaluation of fatigue cracking resistance and low-temperature cracking resistance of HMA mixtures.

OT test configuration consists of one fixed steel plate and one steel plate that can move back and forth to apply desired repeating displacement. The specimen is glued onto the two metallic plates, leaving a 2-mm gap to allow actual loading and testing and simulate transverse cracks or joints on an existing pavement. The test operates in a displacement-controlled loading mode to induce horizontal movement in the mobile plate in order to simulate the opening and closing of joints or cracks in old pavements beneath an overlay (Li et al., 2014).

In this study, OT tests were conducted using the AMPT machine at 4 °C and 21 °C. Although 21 °C is the typical temperature for OT tests, HIR samples were also tested at 4 °C to study the effect of low temperature on crack initiation and propagation in the HIR layer and to determine whether the addition of ARA would change the cracking propensity of HIR mixtures at low temperatures. The OT test setup in AMPT, loading configuration, and an example of a test specimen are shown in Figure 4.6.

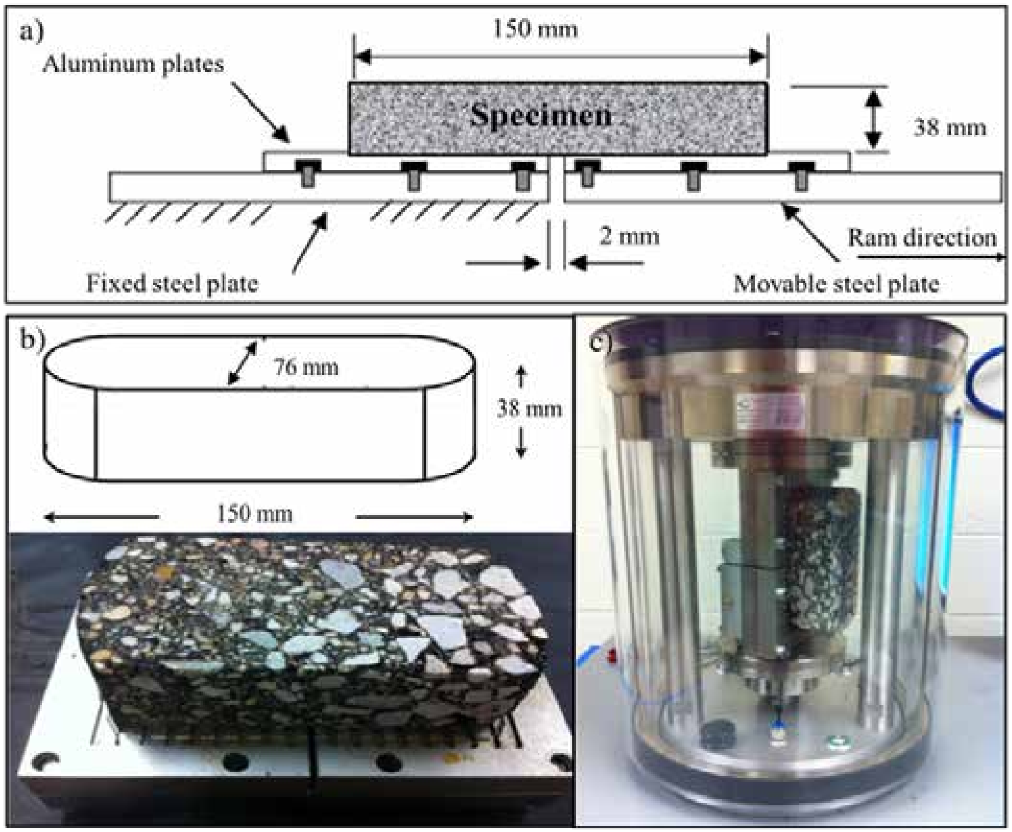


Figure 4.6: OT Test Setup: (a) Load Configuration, (b) Sample Dimensions, (c) AMPT Setup

4.1.5 Moisture Susceptibility Test

The moisture susceptibility test evaluates the effect of saturation, accelerated water conditioning, and freeze-thaw on HIR samples. This study utilized the Kansas Test Method KT-56 (2012), "Resistance of Compacted Asphalt Mixture to Moisture Induced Damage," commonly known as the modified Lottman test. KT-56 is a minor modification of the AASHTO T 283 (2014) test method. HIR mixtures with and without ARA were tested to investigate the effect of ARA as the pavement was exposed to freeze–thaw cycles. For the modified Lottman test, specimens must be 150 mm in diameter and 95±5 mm in height, and six specimens with

7±0.5% air void compacted in an SGC are required. The final result of the test was the ratio of the average tensile strength of three partially saturated specimens that experienced one cycle of freeze-thaw (wet samples) to the average tensile strength of three specimens that were not exposed to moisture (dry samples). According to KDOT specifications, a mixture passes minimum requirements if this TSR exceeds 80%. Figure 4.7 shows the steps involved in conditioning wet samples and the final tensile strength test.

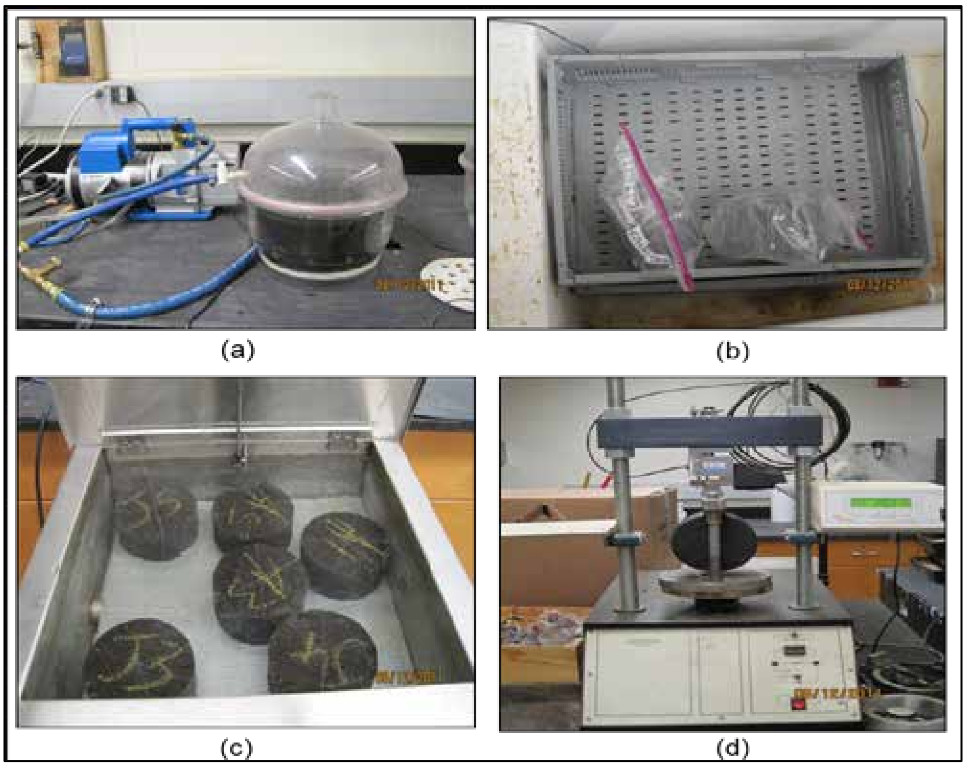


Figure 4.7: KT-56 Test Procedure: (a) Vacuum Saturation, (b) Freezing (Minimum of 16 hrs), (c) Thawing (24 hrs in Water Bath at 60 °C), (d) Testing Tensile Strength

4.1.6 Thermal Stress Restrained Specimen Test

Low-temperature cracking, one of the most common distresses on asphalt pavements, occurs when tensile stresses in asphalt pavement (due to low temperatures) exceed the strength of the asphalt concrete mixture and cause microcracks at the surface and edges of the pavement structure. At cold temperatures or repeated temperature cycles, the crack penetrates the full depth and width of the asphalt concrete layer. Current Superpave specifications based on linear viscoelastic analysis of creep and strength data do not allow timely prediction of crack propagation and do not take into account the effects of traffic loading, variable aging throughout the asphalt layer, and thermal behavior of the pavement as a system; appropriate analysis should be based on the concept of fracture mechanics. However, no preferred test method currently exists to investigate fracture resistance of asphalt materials (Jung & Vinson, 1994; Marasteanu et al., 2007).

TSRST results were shown to be excellent indicators of low-temperature cracking resistance of asphalt mixtures. Asphalt and aggregate type, air void content, specimen size, degree of aging, stress relaxation, and cooling rate were found to be the influential factors in TSRST (Jung & Vinson, 1994). In order to evaluate low-temperature performance characteristics of HIR mixtures as influenced by the addition of ARA, mixtures with and without ARA were tested according to the AASHTO TP 10 (1993) test method in which the cooling rate was set to 10 °C/h and tensile stress was induced for every 2.5 µm contraction. Three replicates were tested for each HIR mixture. TSRST setup, loading configuration, and illustration of a test specimen are provided in Figure 4.8.

Figure 4.8: TSRST Setup: (a) Testing Equipment, (b) Load Cell (Inside the Chamber), (c) Test Sample Glued to the Top and Bottom Plates

4.2 Asphalt Binder Rheology and Miscibility Assessment

4.2.1 Bending Beam Rheometer and Dynamic Shear Rheometer

BBR measures low-temperature stiffness and relaxation properties of asphalt binder, and DSR characterizes the viscous and elastic behavior of asphalt binders at medium to high temperatures. The KDOT Materials and Research Center Chemistry Laboratory had previously completed BBR and DSR tests on binder extracted from HIR mixtures for all study locations in Kansas. However, due to anomalies in the existing data, especially in high temperature grading, additional investigation is required before these BBR and DSR test results can be meaningfully presented.

4.2.2 Exudation Droplet Test

Oxidation is the primary cause of asphalt binder aging during its years of service. Because it is in continuous contact with fresh air and heat, the asphalt binder at the surface of pavement oxidizes much faster than the asphalt binder in deep pavement layers. HIR replaces the top 25 to 50 mm of aged asphalt pavement that contains highly aged asphalt binder. Because aging changes the microstructure of asphalt binder, ARA is used to mobilize the aged binder in the mix by building a homogeneous binder matrix when interacting with the aged binder in the mix.

EDT evaluates the compatibility and characteristics of the new microstructure by assessing the exudation tendency of an asphalt binder. Some asphalt binders exude an oily component when in contact with mineral aggregates with specific void ratios; this oily component can leak into the stone and be detected. Exudation is defined as the loss of oily material that exudes from the binder into the aggregate, representing an unbalanced amount and type of low molecular weight component in the asphalt binder compared to the amount and type of asphaltenes. In order for ARA to sustainably rejuvenate aged binder, it must remain embedded in the asphalt binder matrix and not migrate to the aggregates (Porter, 1991; Root & Moore, 1992; Hunter et al., 2015).

In the EDT process, small droplets of binder (extracted from HIR mixtures) were placed on marble plates (50 × 50 mm with five circular recesses, 10 mm in diameter). These droplets were kept in an oven at 60 °C for 96 hours under nitrogen flow, and then the binder droplets were examined under ultraviolet (UV) light using a microscope. If a ring was observed around the asphalt droplets and the ring was larger than 1.5 mm in width, the asphalt binder was considered exuded (Hunter et al., 2015). Wider rings are representative of undesirable nonhomogeneous and unbalanced mixtures. Figure 4.9 shows EDT samples under normal and UV light with visible rings due to exudation.

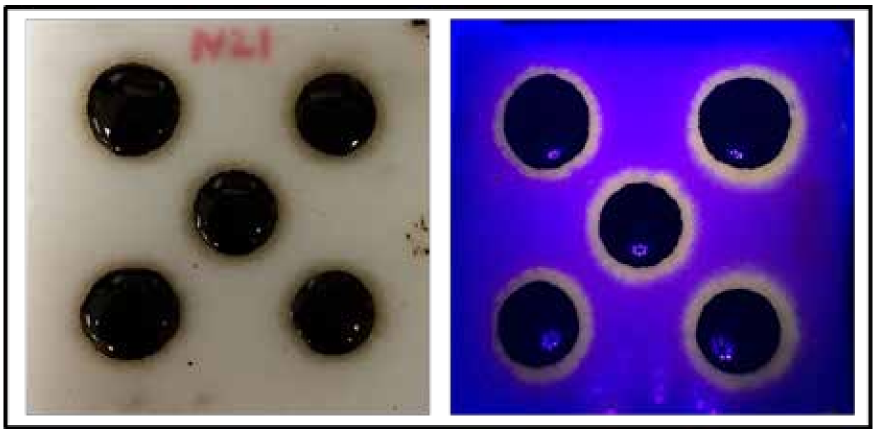


Figure 4.9: EDT Samples under Normal (Left) and UV (Right) Light with Clear Rings around Asphalt Droplets

4.2.3 Scanning Electron Microscopy and Energy Dispersive X-Ray Spectroscopy

SEM is a technique in microscopy that employs electron beams to produce magnified images with high resolution. In producing images using a microscope, employing either light beams or electron beams, diffraction limits the resolution and quality of image. Diffraction is bending of waves (light waves) when reaching a narrow opening or an obstacle and is dependent on the wavelength of the beam and size of opening or obstacle. The range of optical wavelengths, from deep UV to infrared (IR), are in the range of hundreds of nanometers. Electron beams have wavelengths in fraction of nanometers. The dependence of diffraction on the wavelength of the beam makes electron beams more appropriate than beams of wavelengths in the optical region. SEM makes it possible to produce images of objects in the micro to nanometer range with relatively lower diffraction effects by employing the energy of electron beams (Goldstein et al., 2012).

To create an SEM image, electron beams, generated by an electron gun, are decelerated on the solid sample surface and reflected. The detectors collect the electrons that come from the sample (either direct scattering or emitted from the sample). The energy of the detected electrons together with their intensity and location of emission is used to create an image. Present day SEMs are also capable of performing analyses of selected point locations on the sample for determining chemical compositions, using EDXS. EDXS gives specific quantity and distribution information about the composition of elements in the sample (Rinaldini, Schuetz, Partl, Tebaldi, & Poulikakos, 2014). The versatility of SEM and X-ray analysis is originated in the diverse interactions that the beam of electrons goes through within the specimen. The interactions can reveal information on the specimen's composition, topography, crystallography, and other properties. The electron-specimen interactions can be divided into two categories: elastic scattering and inelastic scattering (Goldstein et al., 2012).

Elastic scattering affects the trajectory of electron beam inside the specimen without altering the electron's kinetic energy and causes the electron backscattering phenomenon. Electron backscattering forms important imaging signals in SEM. Elastic scattering changes the direction of the electron velocity but the magnitude of the velocity remains unchanged, meaning that the electron's kinetic energy, $E = m_e \upsilon^2/2$ (m_e is the electron mass), remains unchanged while the electron deviates from its original path by angle $Ø_e$ (the subscript denotes "elastic"). $Ø_e$ can take any value in the range of 0°−180°, with the average value per interaction of about 2°−5°. Thus, in elastic scattering, the electron continues to move in approximately the same direction. Elastic scattering is also strongly dependent on atomic number and incident electron energy. The occurrence probability of elastic scattering increases when specimens with higher atomic numbers are tested, and decreases when high energy electron beams are used (Goldstein et al., 2012).

In inelastic scattering, some energy transfers from the beam of electrons to the atoms of the specimen which results in the generation of secondary electrons, characteristics and continuum X-rays, Auger electrons, electron hole pairs in semiconductor and insulators, long-wavelength electromagnetic radiation in the visible, UV, and IR regions of the spectrum, lattice vibrations, and electron oscillations in metals (Goldstein et al., 2012). All the mentioned products, generated by electron beam and sample interactions, can be used to drive information on the nature of the specimen including what atomic species are present within the region affected by the electron beam (Reimer, 1998). However, only secondary electrons and characteristics and continuum X-rays are discussed in this study since those are the principle processes used for microstructural analysis of binder samples in this study.

Inelastic scattering of the energetic electron can lead to the promotion of loosely bound electrons from the valence band to the conduction band with enough kinetic energy for subsequent motion through the solid. This specimen electron, designated as secondary electron, propagates through the specimen and gets subjected to inelastic scattering and energy loss. If the secondary electron retains sufficient energy when it reaches the surface to overcome the surface barrier energy, it will escape from the solid as a secondary electron and will be detected by the secondary electron detectors in SEM (Reimer, 1998).

Interaction of the electron beam with atoms in the sample causes shell transitions which result in the emission of an X-ray. Shell transitions occur when an element is bombarded with high-energy particles (electrons in SEM) and strike a bound electron in an atom and cause it to eject from the inner shell of the atom. After the electron has been ejected, the atom is left with a vacant energy level. Outer-shell electrons fall into the inner shell, emitting quantized photons with an energy level equivalent to the energy difference between the higher and lower states. The transition from higher to lower energy levels produces X-rays with energy characteristics of the parent element and detection and measurement of the energy characteristics permits energy dispersive X-ray spectroscopy analysis (EDXS). EXDS can provide rapid qualitative or, with adequate equipment, quantitative analysis of elemental composition with a sampling depth of 1-2 microns.

Sample preparation for SEM included putting a small droplet of desired binder (extracted from US-56, US-59, or K-14 mixtures) on a silicon wafer and mounting the silicon wafer on an aluminum holder or stub using double-sided conductive tapes. Figure 4.10 shows extracted asphalt binder samples prepared to be placed in the SEM. Asphalt binder is nonconductive and has high resistivity. Material of high resistivity will rapidly charge under the electron beam and can cause a dielectric breakdown in certain regions of the specimen, which leads to complex image artifacts commonly referred to as “charging.” Impacting the secondary electrons, charging will degrade the resolving power and analytical capabilities of the system by introducing astigmatism instabilities, excessive brightness, and spurious X-ray signals (Goldstein et al., 2012). Creating a conductive layer of metal on the sample inhibits charging, reduces thermal damage, and improves the secondary electron signal required for topographic examination in the SEM. In this study, Palladium (Pd) was the metal used to create the conductive layer to prevent charging.

Figure 4.11 shows an SEM image of US-59 extracted binder. Figure 4.11 is taken from uncoated samples (Figure 4.10) and formed by backscattered electrons. Figure 4.12 shows the EDXS analysis on two spots on the US-59 uncoated sample.

Figure 4.13 shows US-56 extracted binder samples under the Pd spray and coated with Pd. Figure 4.14 shows an SEM image of US-56 extracted binder taken from Pd coated samples (Figure 4.13) formed by employing secondary electrons. Detailed discussion of SEM and EDXS analysis is provided in Chapter 5.

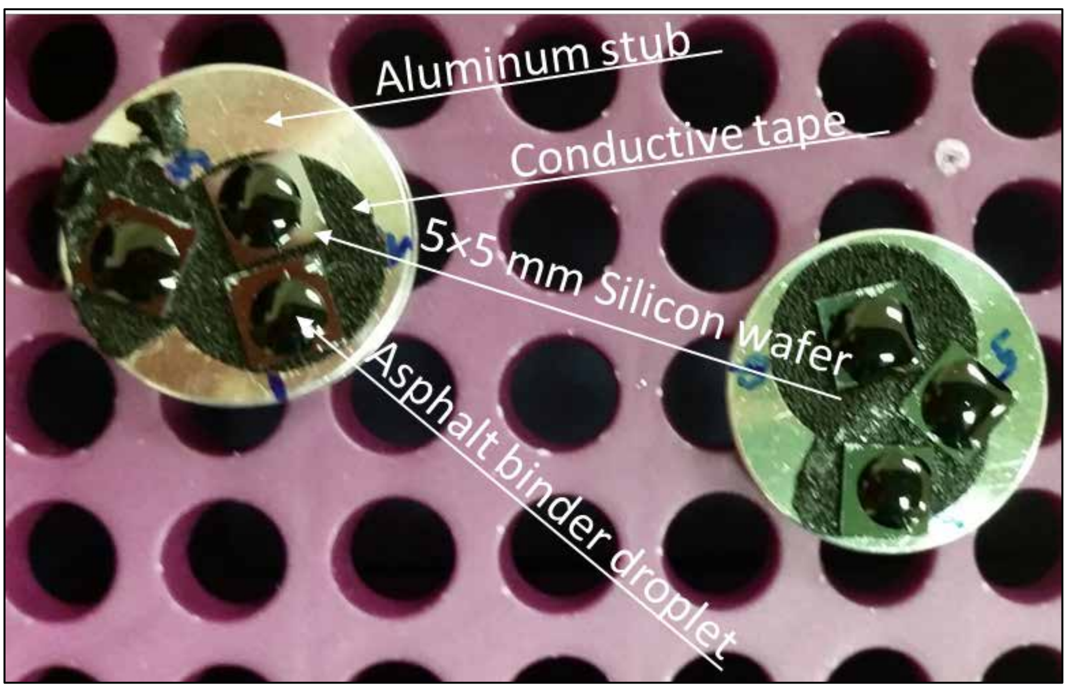


Figure 4.10: Extracted Asphalt Binder Samples Prepared for SEM

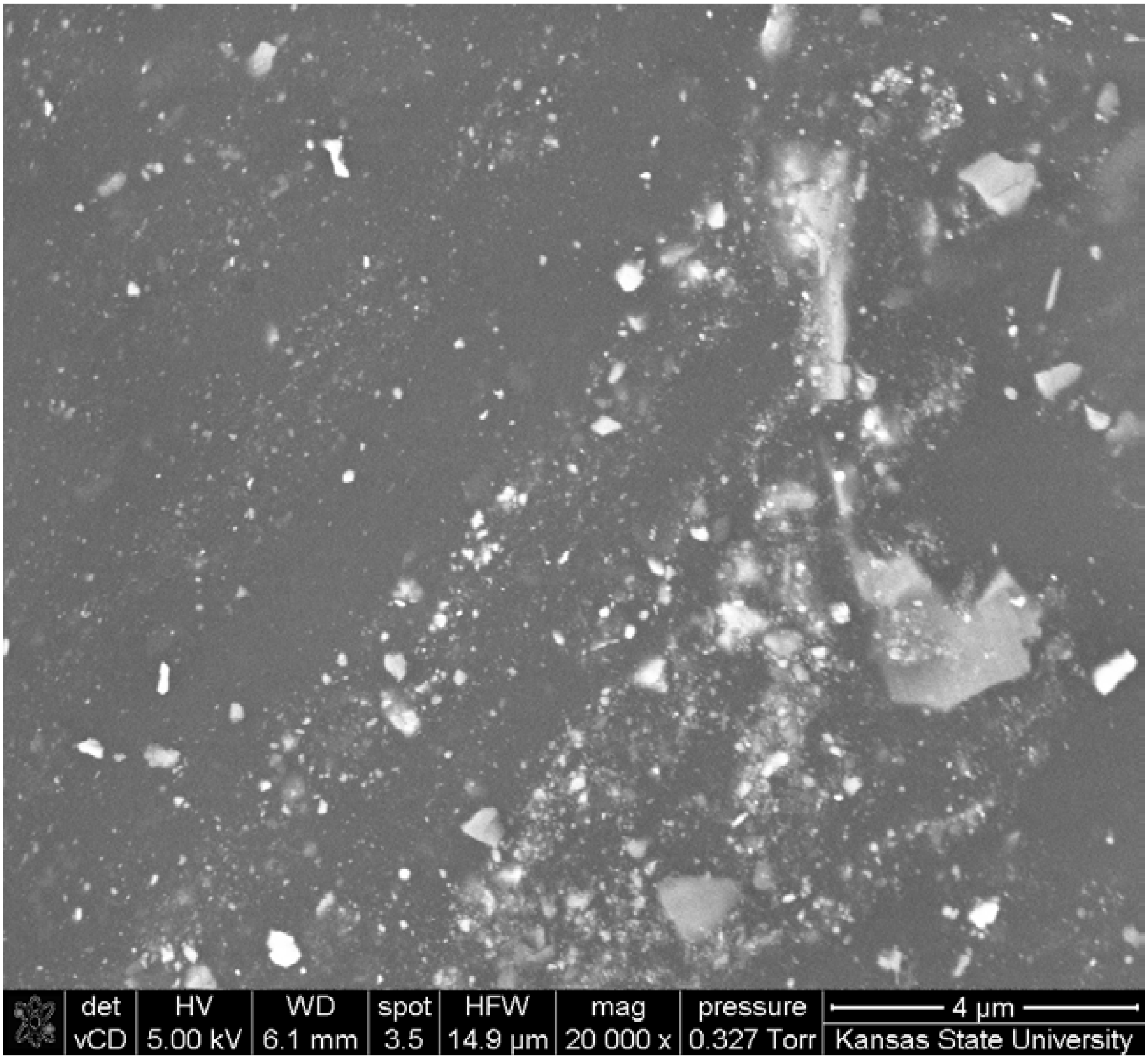
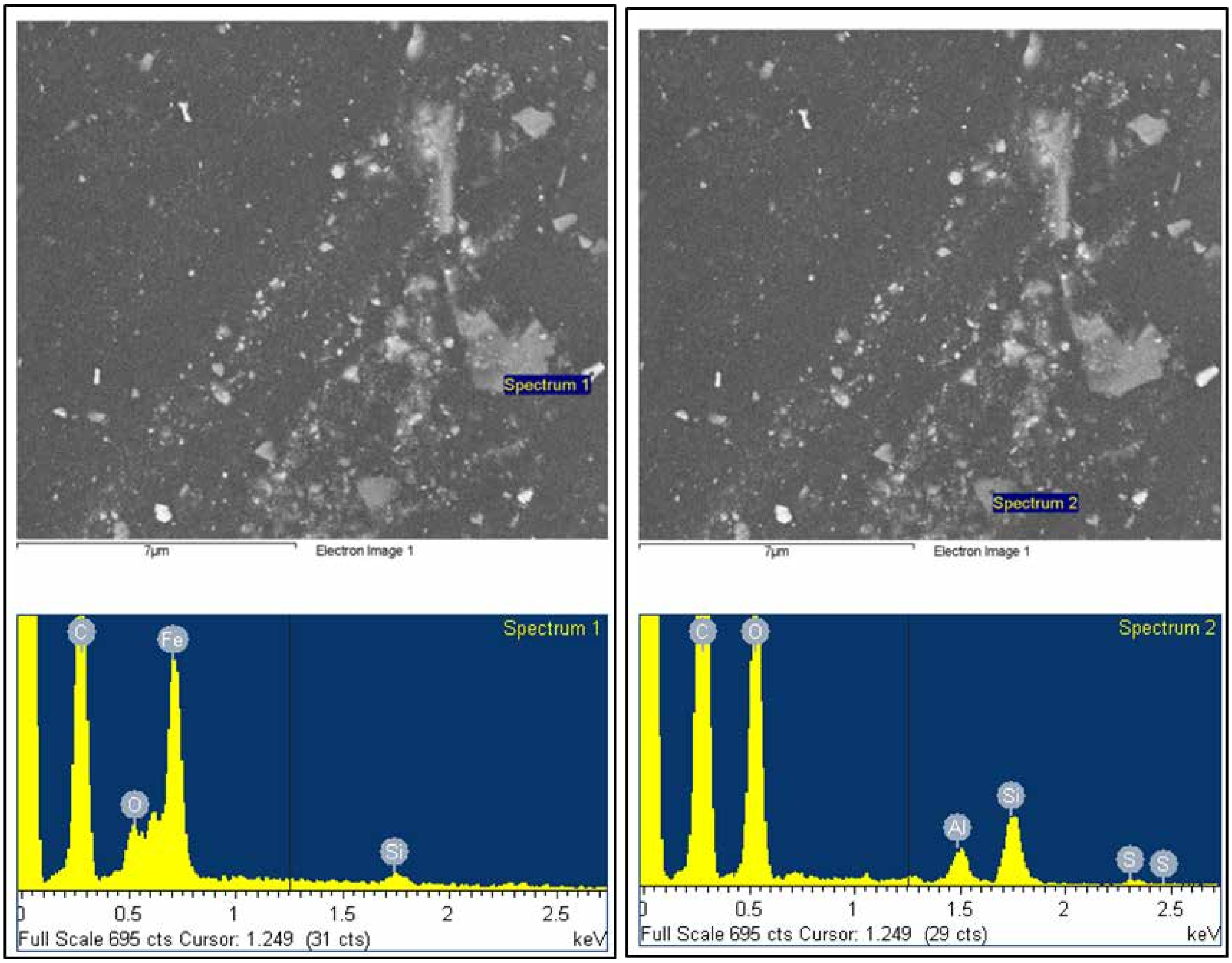


Figure 4.11: SEM Image of US-59 Extracted Binder



Element	Weight%	Atomic%
C K	26.48	59.91
O K	3.01	5.12
Si K	1.37	1.33
Fe L	69.14	33.65

Element	Weight%	Atomic%
C K	51.44	62.31
O K	32.12	29.21
Al K	3.53	1.91
Si K	11.20	5.80
S K	1.70	0.77

Figure 4.12: EDXS Analysis of Two Spots on the Binder Extracted from US-59 Mixture

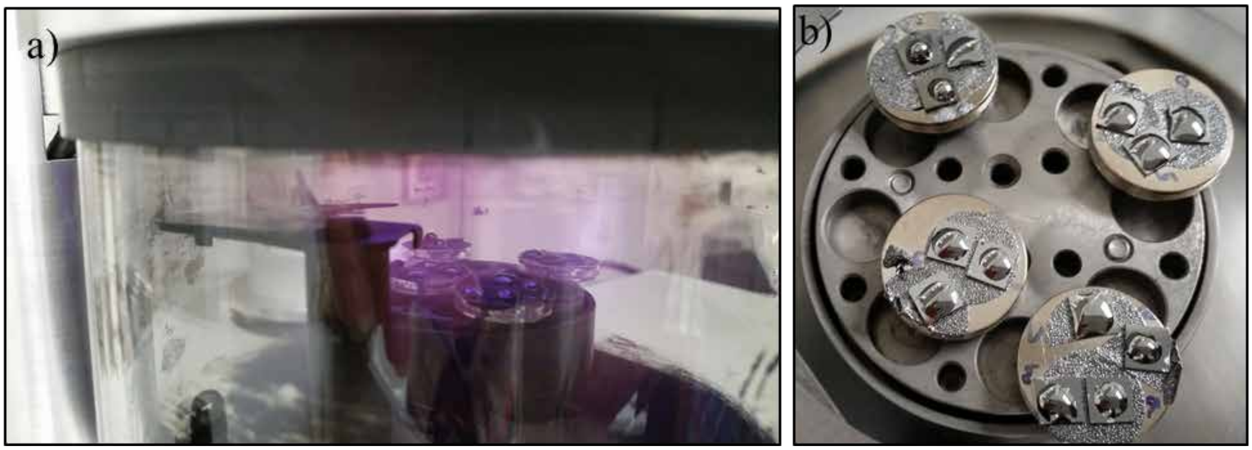


Figure 4.13: US-56 Extracted Binder Samples: (a) Under Pd Spray, (b) Coated with Pd

Figure 4.14: SEM Image of US-56 Extracted Binder

Chapter 5: Results and Discussion

5.1 HIR Mixture Test Results

5.1.1 Hamburg Wheel-Tracking Device Test Results

Each HWTD test requires four compacted samples. Because the HWTD test was conducted three times for each HIR mixture in this study, a minimum of 12 samples were compacted for each HIR mixture. In the HWTD test specimen, rut depth typically increases as the number of wheel passes increases; lower rut depths indicate less rutting and/or stripping susceptibility. HIR mixtures without ARA showed expected improved resistance to rutting due to the hardening effect of the aged binder. KDOT set the standard for acceptable HWTD test results at a maximum rut depth of 12.5 mm at 10,000 wheel passes. This study adopted a criterion of 40,000 wheel passes in order to compare various HIR mixtures. HWTD test results are provided in Table 5.1. The KDOT minimum requirement was met by all HIR mixtures, and no stripping or moisture damage was observed at the test termination point, indicating that the addition of ARA did not increase rutting risk for the pavement. The HIR mixture for the US-59 project was somewhat aged/dry (causing difficulty in compaction), potentially causing the high rut resistance of that mixture (less than 3 mm after 40,000 wheel passes).

Table 5.1: Comparison of HWTD Rut Depth Test Results

HIR Mixture	Number of Wheel Passes (Rut depth)					
	Left Wheel			Right Wheel		
US-56 w/o ARA	40,000 (4 mm)	40,000 (10 mm)	33,574 (20 mm)	40,000 (5 mm)	40,000 (3 mm)	40,000 (3 mm)
US-56 with ARA	26,006 (20 mm)	31,366 (20 mm)	32,200 (20 mm)	28,296 (20 mm)	29,136 (20 mm)	40,000 (20 mm)
US-59 w/o ARA	40,000 (3 mm)	40,000 (3 mm)	40,000 (4 mm)	40,000 (3 mm)	40,000 (3 mm)	40,000 (3 mm)
US-59 with ARA	40,000 (3 mm)	40,000 (3 mm)	40,000 (2 mm)	40,000 (3 mm)	40,000 (3 mm)	40,000 (2 mm)
K-14 w/o ARA	40,000 (15 mm)	40,000 (19 mm)	40,000 (6 mm)	35,227 (20 mm)	40,000 (6 mm)	40,000 (11 mm)
K-14 with ARA	30,632 (20 mm)	40,000 (14 mm)	28,812 (20 mm)	35,650 (20 mm)	37,482 (20 mm)	25,044 (20 mm)

5.1.2 Dynamic Modulus Test Results

Dynamic modulus tests were conducted following AASHTO TP 62-07 (2007) in a temperature-controlled chamber at three temperatures (4 °C, 21 °C, and 37 °C) and six frequencies (25, 10, 5, 1, 0.5, and 0.1 Hz) using an AMPT. For graphical analysis and straightforward interpretation of test data, $|E^*|$ master curves were generated by shifting data according to the time–temperature superposition principle described in AASHTO PP 61 (2013; reference temperature of 20 °C). Three samples were fabricated for each mix design, and average test results were used for comparison. Dynamic modulus master curves for all three HIR mixtures (with and without ARA) are provided in Figure 5.1, which shows that the softening effect of ARA was detected as lower dynamic moduli were obtained for mixtures with ARA. High stiffness of the US-59 mixture without ARA was also noticeable in the dynamic modulus test results. Comparison of these dynamic modulus results with the binder test results show K-14 being the softest but stiffer in the mix, while US-56 is softest in mix and fairly soft in binder testing. Overall, the results indicate that the addition of ARA reduces the stiffness of the binder and the mix.

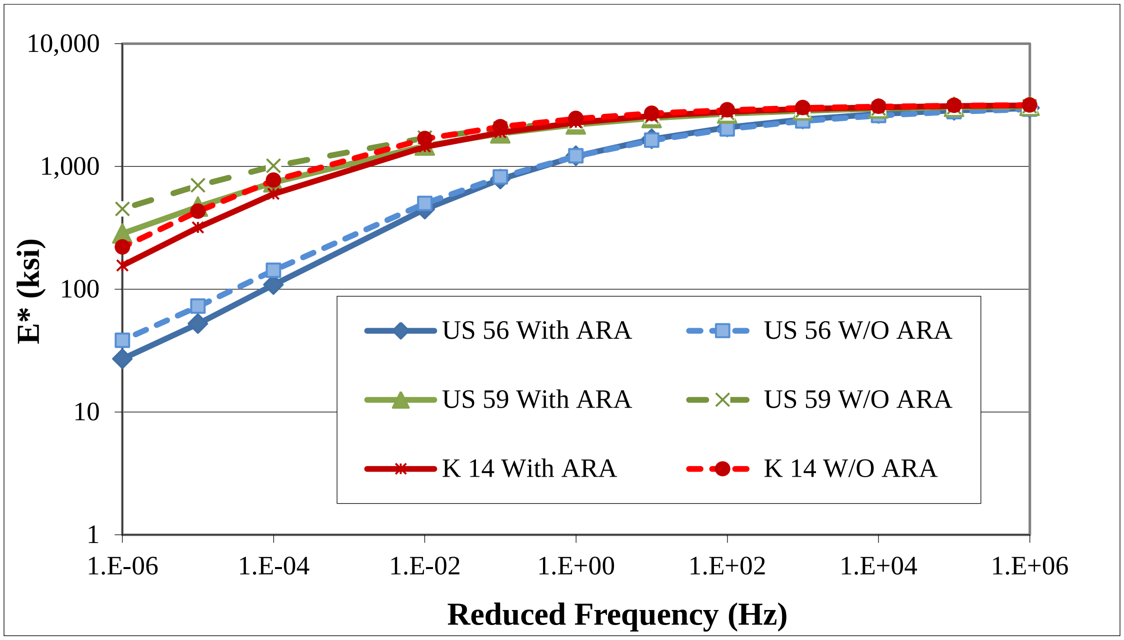


Figure 5.1: Dynamic Modulus Master Curves for HIR Mixtures With and Without ARA

5.1.3 Texas Overlay Test Results

The current OT test results analysis method requires load cycles to be counted to failure; failure criterion is satisfied when the initial maximum load is reduced by 93% (Zhou et al., 2006). If the 93% load reduction does not occur, the test ceases when 1,000 cycles are reached. Theoretically, the more OT cycles that can be endured before failure, the better the crack-resistance potential of the asphalt mixture. Research studies conducted primarily with Texas dense-graded mixes have shown that HMA mixes that last at least 300 cycles are acceptable (Li et al., 2014). This criterion was adopted in this study, and HIR mixtures that endured over 300 OT load cycles were assumed to have adequate crack resistance. Three replicates were fabricated for each HIR mixture in order to evaluate cracking resistance via the OT test. However, due to fluctuation in OT test results and subsequent higher coefficients of variations, a definite conclusion regarding the performance of HIR mixtures could not be drawn. In addition, due to a shortage of HIR material, no more than one extra specimen could be made for each mixture. OT test results are tabulated in Table 5.2. The fourth specimen for all mixtures was the extra sample that was fabricated later. Bold numbers in Table 5.2 indicate samples that failed to meet the minimum requirement of 300 cycles before failure.

Table 5.2 shows an unexpected trend in OT test results: the addition of ARA had no effect on performance improvements of HIR mixtures and very high variability of results. Outcomes of the OT test in the literature, however, indicate that OT test results are generally reliable with acceptable low variability (Li et al., 2014; Guercio, Mehta, & McCarthy, 2015). The study conducted by Im et al. (2014) showed that the addition of rejuvenating agent improved the number of OT cycles from 110% to 300% based on the type of rejuvenator added. Their findings contradict results obtained in this study for US-59 and K-14 mixtures. The addition of ARA improved US-56 mixture performance, with the coefficient of variation in the acceptable range for the OT test (10–25%; Guercio et al., 2015). The study conducted by Li et al. (2014) suggested that heterogeneity in the mixture, poor workability, and high stiffness of the asphalt binder can cause high variabilities in OT test results. The same study suggests that mixtures with high asphalt binder content showed better performance compared to HMA mixtures with relatively lower asphalt binder content. High asphalt binder content led to improved ductility and elasticity in the mix and allowed the mixture to sustain repeated tensile loading prior to crack failure.

Table 5.2: OT Test Results Conducted at 4 °C and 21 °C

HIR Mix		21 °C					4 °C				
		Number of OT Cycles									
		1st	2nd	3rd	4th	Avg.	1st	2nd	3rd	4th	Avg.
US-56	With ARA	647	946	1000	1000	898	1,000	1,000	1,000	1,000	1,000
	W/o ARA	1,000	1,000	1,000	1,000	1,000	280	6	247	148	170
US-59	With ARA	290	1,000	1000	187	619	1,000	185	1,000	243	607
	W/o ARA	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**1,000**	**129**	**1,000**	**61**	548
K-14	With ARA	**249**	**1,000**	**413**	**192**	**464**	**1,000**	**23**	**1,000**	**157**	545
	W/o ARA	967	1,000	304	415	672	1,000	1,000	1,000	1,000	1,000

Results provided in Table 5.2 follow the same trend as discussed in the literature. The US-56 mixture with highest binder content (Table 3.1) and more consistent binder structure (Table 5.5 and EDT section) demonstrated performance improvement after the addition of ARA, while high stiffness and low workability, low binder contents (even after the addition of ARA), and binder inconsistencies did not contribute to consistent repeatable test results for US-59 and K-14 mixtures.

5.1.4 Flow Number Test Results

The flow number (FN) test in this study was conducted with deviator stress and contact stress of 207 kPa and 5 kPa, respectively. Recommended temperatures for the FN test range from 25 °C to 60 °C. The test is commonly conducted at 54 °C in Kansas, which is the average 7-day maximum pavement temperature 20 mm below the surface at 50% reliability. The HIR mixtures in this study, however, had already been tested for rutting resistance with HWTD while submerged in water at 50 °C but did not show any rutting tendency. OT test results at 21 °C, though, showed considerable fluctuations. Therefore, in order to investigate the range of FN cycles the samples could endure before failure and to evaluate the creep tendency of HIR mixtures, the FN test was conducted until failure or 10,000 cycles, whichever came first. The minimum recommended FN requirement is 190 cycles for a traffic level of 10–30 million equivalent single axle loads (ESALs; NCHRP, 2012; Saghebfar, 2014).

While three replicates were initially tested for each HIR mixture, one or two more samples were fabricated due to high fluctuation in FN test results, and outliers were identified based on all samples tested for one mixture. Similar to dynamic modulus test samples, FN samples were fabricated using SGC at an air void level of 8.5±1% to obtain an air void level of 7±1 % in the cores (the air void level is 1%–1.5% lower in cores than in the SGC-compacted samples).

FN test results and statistical parameters are provided in Table 5.3, in which the bold cells represent outliers that were excluded in further analysis. Some test results that were not outliers were also deleted because they were out of the spectrum of other results in the same group. Statistical analysis conducted on FN test results (95% level of confidence) showed significant changes (not necessarily improvements) in the performances of US-56 and US-59 mixtures when ARA was added.

Very similar to OT test results, the US-56 mixture with highest binder content and stiffest binder demonstrated performance improvement after the addition of ARA. For US-59 mixture, the addition of ARA has resulted in lower number of FN cycles and higher coefficient of variation which can be attributed to the mixture being heterogeneous. The addition of ARA has improved the performance of K-14 mixture, although not significantly. Also, based on FN test results, K-14 mixture with ARA has a more homogeneous structure.

Table 5.3: Flow Number Test Results and Statistical Parameters

HIR Mixture	US-56		US-59		K-14	
	with ARA	w/o ARA	with ARA	w/o ARA	with ARA	w/o ARA
Flow Number Cycles	3,165	2,368	2,956	7,321	1,872	1,205
	3,346	793	6,240	7,241	2,433	2,253
	3,875	1719	3,788	7,233	**413**	**9,086**
					1,886	1,165
		1,590	5,839			
Mean	3,462	1,618	4,706	7,265	2,064	1,541
Std. Dev.	369	647	1584	49	320	617
COV (%)	11	40	34	1	16	40
p-value (with ARA vs. w/o)	0.005		0.048		0.28	

5.1.5 Moisture Susceptibility Test Results

The moisture susceptibility test was conducted to evaluate the effect of saturation, accelerated water conditioning, and freeze-thaw on HIR samples using the Kansas Test Method KT-56. The test is a minor modification of AASHTO T 283, with a mandatory freeze cycle. Moisture susceptibility test results are tabulated in Table 5.4 and illustrated in Figure 5.2. Although TSR improved when ARA was added to two out of three HIR mixtures, statistical analysis showed the improvement was significant only for the US-56 mixture at 5% level of significance (p-value = 0.03).

Table 5.4: Tensile Strength Test Results for Mixtures With and Without ARA

HIR Mix	Dry Strength (kPa)	Wet Strength (kPa)	%TSR
US-56 w/o ARA	999.9	706.9	70.7
	1033.8	742.8	71.8
	1031.1	823.5	79.9
US-59 w/o ARA	1802.2	1340	74.4
	1665.7	1348.2	80.9
	1552.2	1329.4	85.6
K-14 w/o ARA	1804.8	1012.7	56.1
	1815.4	1022.7	56.3
	1832.9	983.9	53.7
US-56 with ARA	906.9	828.7	91.4
	874.6	795.3	90.9
	843.4	793.5	94.1
US-59 with ARA	1466.5	1220.2	83.2
	1482.0	1274.3	86.0
	1492.3	1325.6	88.8
K-14 with ARA	1993.9	933.0	46.8
	1796.4	1017.2	56.6
	1630.5	972.0	59.6

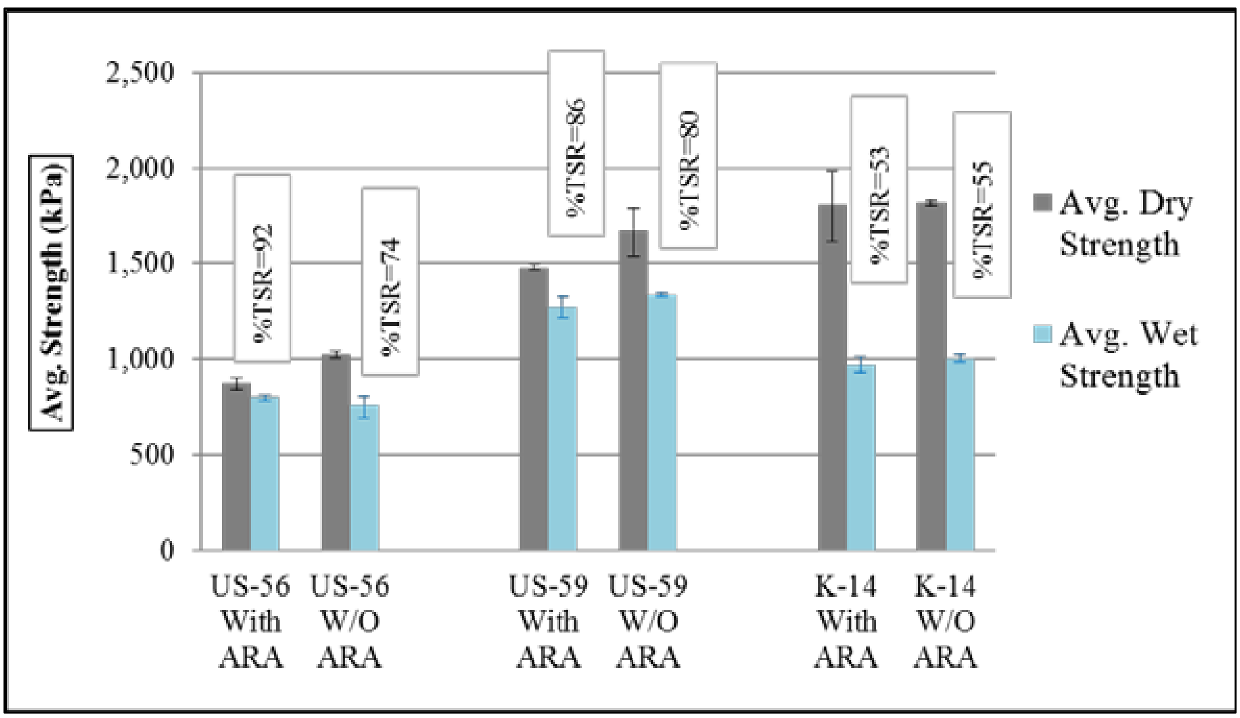


Figure 5.2: Average TSR Results for Wet and Dry Samples

5.1.6 Thermal Stress Restrained Specimen Test Results

TSRST results for HIR mixtures in this study are provided in Figure 5.3. For US-56 and US-59 mixtures, the average failure temperature was lower for mixtures with ARA. The trend for the K-14 mixture, however, was similar to the trend for the moisture susceptibility test. Temperature reduction caused by the addition of ARA was statistically insignificant for US-56 and US-59 mixtures, with p-values of 0.6 and 0.3, respectively. Although the K-14 mixture without ARA performed better in TSRST, performance of mixtures with and without ARA did not differ significantly (p-value = 0.6).

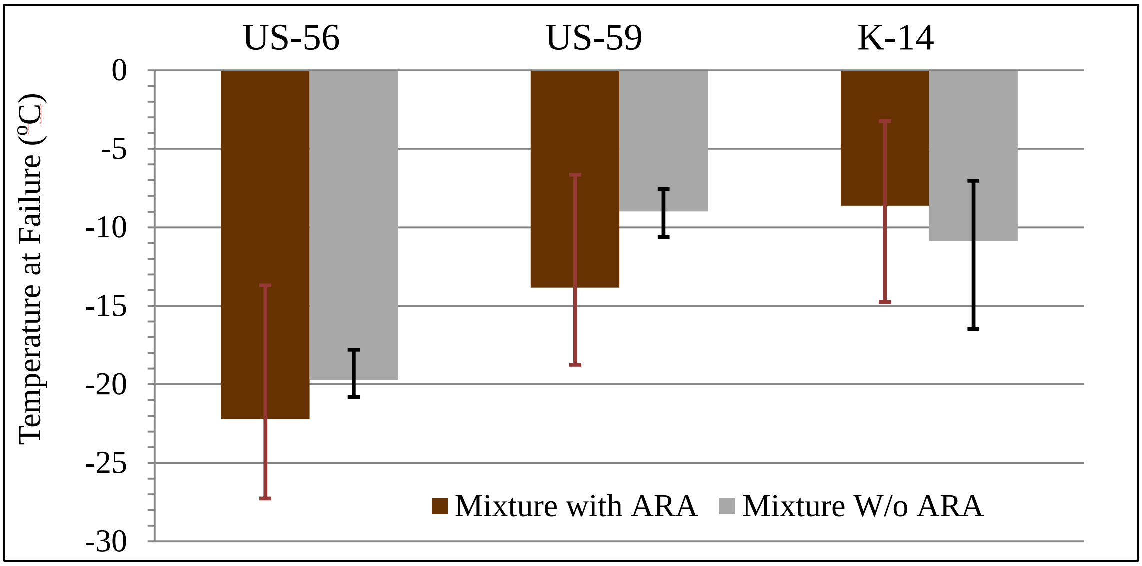


Figure 5.3: TSRST Results for Mixtures With and Without ARA

5.2 Rheology and Miscibility Assessments

5.2.1 Exudation Droplet Test Results

Discrepancies in mechanical test results of HIR mixtures prompted use of EDT in order to understand binder consistency in each HIR mixture. EDT results in Table 5.5 show mixtures with ARA had wider rings (addition of ARA causes heterogeneity that can be detected by EDT) and US-59 and K-14 mixtures with ARA had rings wider than 1.5 mm (nonuniformity in binder extracted from field samples). It should be mentioned that for the results provided in Table 5.5, binder samples were heated only up to 60 °C (to be able to obtain a droplet of the sample) and were not mixed. Heating and mixing was avoided with the purpose of conducting the EDT on samples that are representatives of the HIR binder at field.

In order to investigate the possibility of obtaining a uniform mixture of aged binder and ARA, extracted binder for the US-59 mixture without ARA was mixed with Reclamite and virgin binder in the laboratory by controlling time and temperature of mixing, and EDT was repeated. Added to the US-59 extracted binder were 5% ARA and 20% virgin binder (by the weight of the binder). Results of mixing under laboratory conditions are provided in Table 5.6. Laboratory-controlled conditions allowed acquisition of a homogeneous mixture of ARA/virgin binder with aged HIR extracted binder. The heating and mixing process included heating the aged binder from HIR for 40 minutes before it reached 135–140 °C, adding the ARA/virgin binder and mixing (with a hand mixer) for 40 seconds, returning the mixture to the oven for 10 minutes to allow bubbles to emerge, and finally pouring binder droplets on to the marble plates.

Table 5.5: EDT Results for HIR Binder (Extracted from Field Samples)

HIR Mixture	Ring Diameter (mm)	
	With ARA	W/o ARA
US-56	1.4	1.3
US-59	1.8	1.7
K-14	1.5	1.2

Table 5.6: EDT Results for HIR Binder Mixed with ARA and Virgin Binder under Laboratory Condition

Combined HIR Mixture	Ring Diameter (mm)
US-59 w/o ARA + 5% ARA	1.3
US-59 w/o ARA + 20% virgin binder	1.3

EDT results show that the US-56 mixture had acceptable binder consistency (ring diameter less than 1.5 mm before and after the addition of ARA) and showed consistent performance improvements as evaluated by OT, TSRST, FN, and moisture susceptibility tests.

Exudation rings larger than 1.5 mm in US-59 extracted binder demonstrate a nonuniform binder structure that has not improved after the addition of ARA. OT and flow number test results did not show any performance improvement after the addition of ARA, mainly due to the reason that no improvement was made in the binder structure after the addition of ARA. It should be mentioned that moisture susceptibility performance improvement of the US-59 mixture after the addition of ARA was not statistically significant.

For the K-14 mixture, EDT results of extracted binder are in an acceptable range but the mechanical tests of K-14 mixture did not show improvements after the addition of ARA. Considering that the K-14 mixture had the lowest binder content among the three locations, the unimproved or declined performance in mechanical test results after the addition of ARA might have been caused by binder inadequacy in the mixture (preventing the diffusion of ARA in the short time of mixing in the HIR process).

EDT results in Table 5.5 also show that by sufficiently mixing aged asphalt binder and ARA (controlling time and temperature of mixing), it is possible to achieve a binder that has enough uniformity to pass EDT criteria (ring diameter less than 1.5 mm).

5.2.2 Scanning Electron Microscopy and Energy Dispersive X-Ray Spectroscopy Results

Mechanical test results discussed so far have provided an overview of the performance characteristics of each mixture, which have proven to be inconsistent for mixtures collected at various locations. Variations in test results were speculated to be caused by binder inconsistencies or improper diffusion of ARA in aged asphalt binder.

SEM images of US-59 extracted binder (with and without ARA) are provided in Figure 5.4. The image gray level depends on the chemical composition of the material, with higher ordinal number atoms appearing in lighter colors. Figure 5.4 also shows the heterogeneous structural of US-59 binder on images with high magnification. Figure 5.5 shows SEM images of US-59 binder with higher magnification in which protuberance and recess on the samples can be identified. The heterogeneous texture of US-59 binder, observed in Figure 5.4 and Figure 5.5, confirms the EDT results (nonuniform binder structure for US-59).

The study by Wang et al. (2015) on surface morphologies and phase contrasts showed protuberance and recess produce strong phase contrast, indicating difference in properties from the surrounding substance. EDXS was used in conjunction with SEM to further investigate the elemental differences at various spots of each sample. EDXS results of the spots identified on Figure 5.4 and Figure 5.5 are provided in Table 5.6. The ARA was also analyzed by SEM, but no SEM image of ARA is provided since no significant spot was identifiable on the uniform gray texture of the sample. The existence of silica (Si) in the tested spots can be attributed to the aggregates from which the binder samples were extracted. In the US-59 binder, however, lead (Pb), iron (Fe), and aluminum (Al) were identified on frequent bright spots on the samples (Table 5.6). The EDXS analysis was conducted on US-56 extracted binder samples and while Si was frequently detected on tested spots, no heavy metals were detected.

While exploring the source and effects of Pb, Fe, and Al on the HIR performance characteristics is beyond the scope of this research, EDXS and SEM in this study showed a great deal of discrepancy in the US-59 binder structure, as confirmed by EDT and mechanical test results.

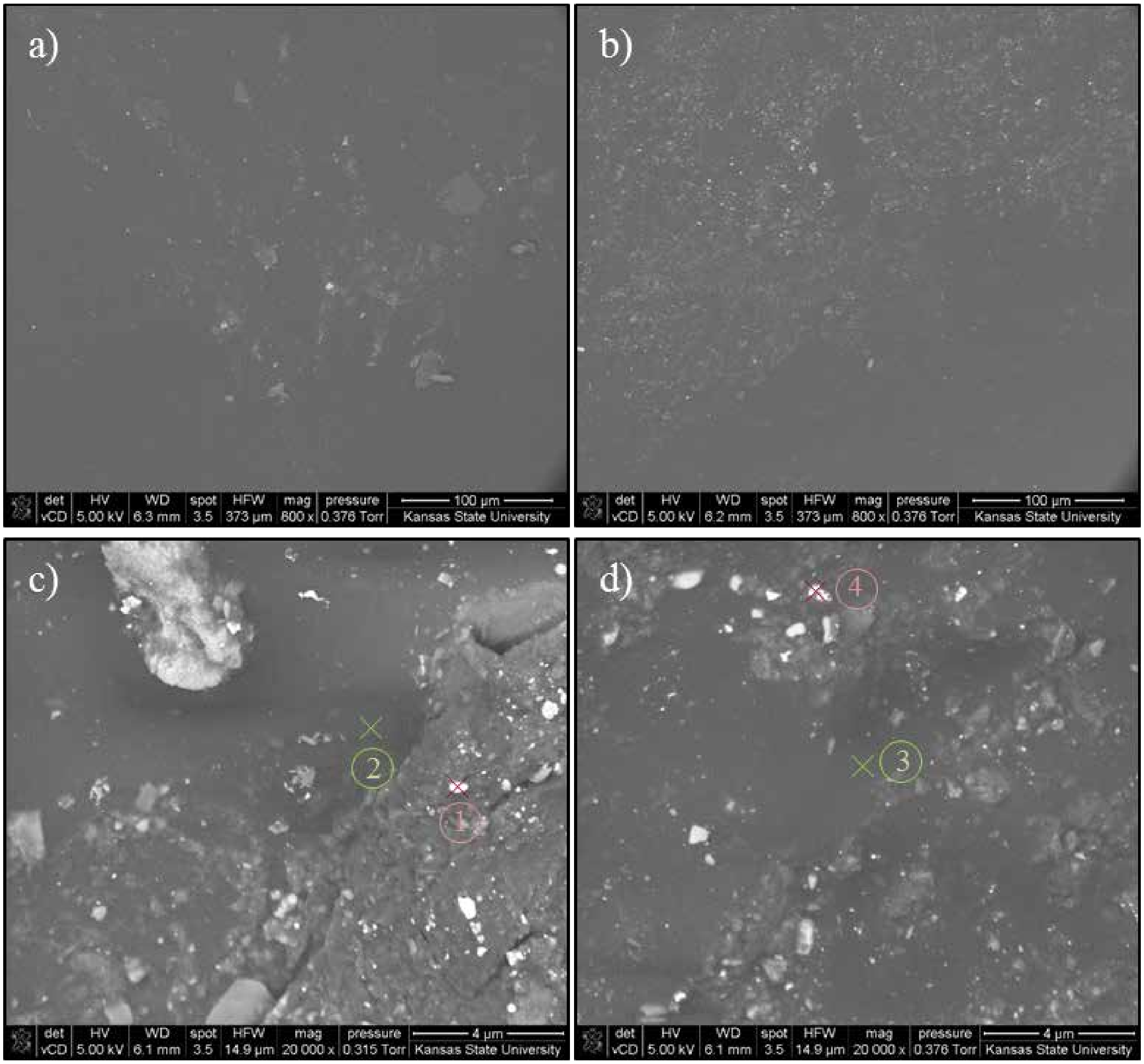


Figure 5.4: SEM Images: (a) US-59 w/o ARA, (b) US-59 with ARA, (c) US-59 w/o ARA (Higher Magnification), (d) US-59 with ARA (Higher Magnification)

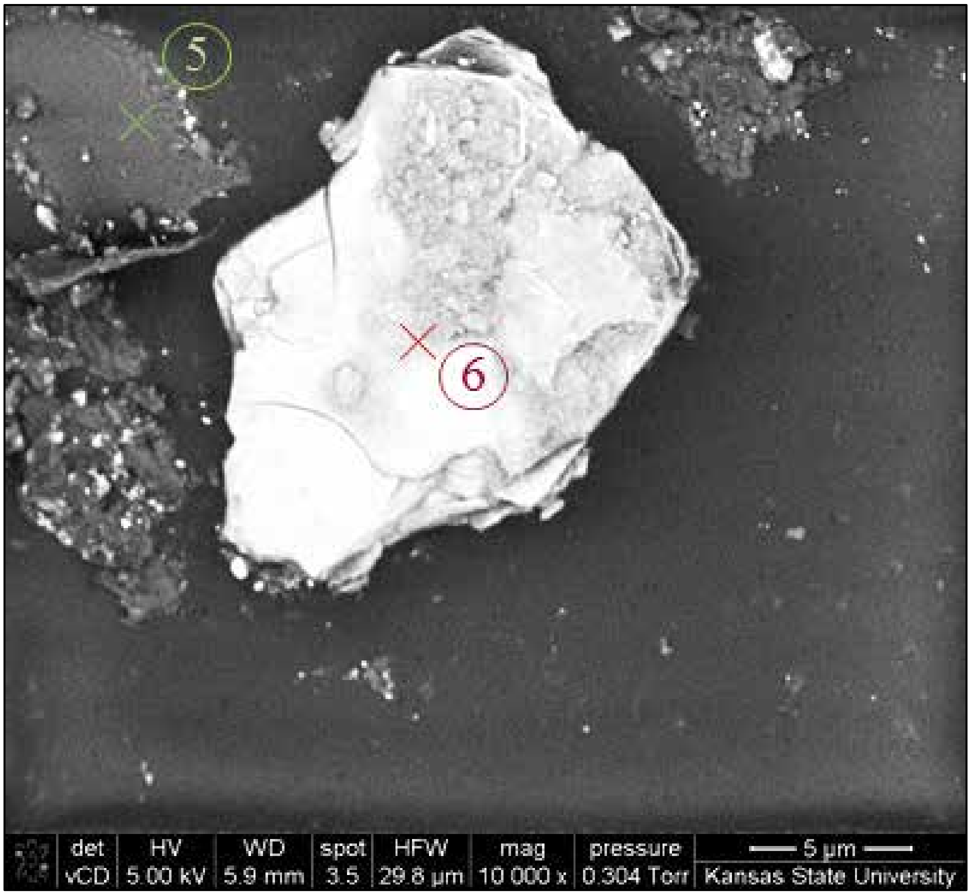


Figure 5.5: High Magnification SEM Image of US-59 Binder

Table 5.7: EDXS Elemental Analysis on Extracted Binder from US-59 Mixture

Atomic Percentage of Elements Recognized					
Point 1	**Point 2**	**Point 3**	**Point 4**	**Point 5**	**Point 6**
C: 68% O: 25% Pb: 7%	C: 86% O: 6% Al: 4% Si: 1% S: 3%	C: 91% O: 6% Si: 2% S: 1%	C: 75% O: 13% Al: 9% Si: 2% S: 1%	C: 91% O: 6% Na: 2% S: 1%	C: 41% O: 40% Al: 3% Si: 11% Fe: 3% Mg: 2%

Chapter 6: Pavement Maintenance and Life-Cycle Cost Analysis

Increasing costs of highway pavement construction, repair, maintenance, and rehabilitation, as well as shortfalls in highway revenue, have forced highway agencies to integrate various pavement preservation programs in order to attain long-term, cost-effective investments. Life-cycle cost analysis (LCCA) is based on the principles of economic analysis, which aids evaluation of overall long-term economic efficiency among competing alternatives (Lamptey, Ahmad, Labi, & Sinha, 2005). The Federal Highway Administration (FHWA) has always encouraged the use of LCCA when analyzing all major investment decisions in order to improve efficiency of those decisions.

LCCA for highway investment decisions was first introduced in 1960 by the American Association of State Highway Officials (AASHO). In 1963, a study was developed to organize available data on vehicle operating costs into a format that highway planners could use to develop LCCA (Winfrey, 1963). The NCHRP, Texas Transportation Institute (TTI), and the Center for Highway Transportation also promoted LCCA during the 1960s and developed a methodology and computer program to analyze and rank alternative flexible (and later rigid) pavement designs by overall life-cycle cost. The current FHWA position on LCCA for pavement design, which includes consideration of LCCA in pavement design and engineering, is based on the Intermodal Surface Transportation Equity Act of 1991 (ISTEA). Other LCCA driving forces include the National Highway System (NHS) Designation Act of 1995, which required states to conduct LCCA on NHS projects with costs higher than a certain threshold, and the 1998 Transportation Equity Act for the 21st Century (TEA-21; Walls & Smith, 1998).

Although local highway departments have historically emphasized building new pavements, increasing budget constraints have altered the focus to maintaining and preserving existing pavement surfaces, resulting in three types of pavement maintenance operations: preventive maintenance, corrective maintenance, and emergency maintenance (Johnson, 2000). Figure 6.1 demonstrates the differences in three maintenance types.

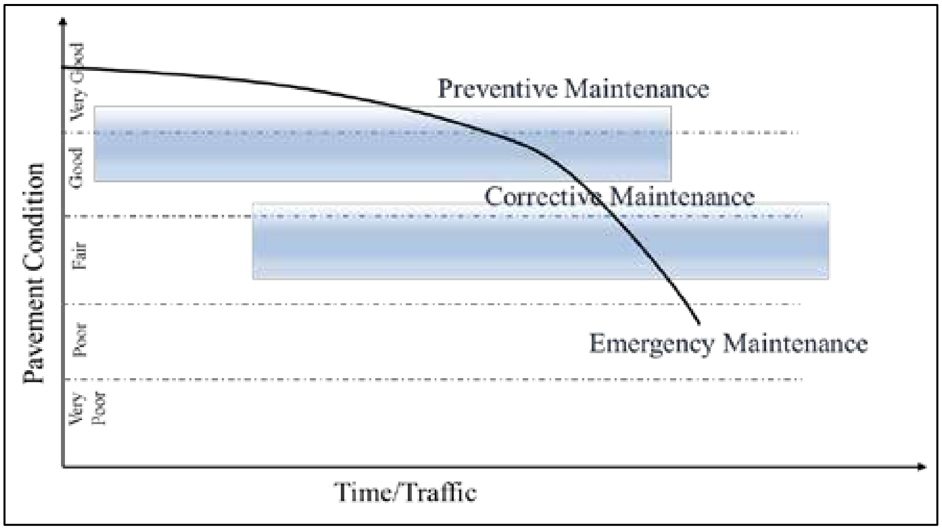


Figure 6.1: Pavement Maintenance Categories
Source: Johnson (2000)

The definition of each maintenance type and common treatments are provided in the following sections.

6.1 Maintenance Types and Common Techniques

6.1.1 Preventive Maintenance

Preventive maintenance maintains or improves a pavement's functional condition by delaying progressive failure of the pavement and reducing the need for routine maintenance and service activities. Preventive treatments are effective when applied to relatively young pavements in good condition. Application of preventive treatments at an inappropriate time in the service life of a pavement structure could lead to the flawed conclusion that preventive maintenance does not work. Pavements begin to deteriorate as they are exposed to traffic and environmental forces. This deterioration in HMA pavements occurs in the form of rutting, cracking, loss of surface texture, and increased roughness. Preventive maintenance guarantees the mentioned deterioration modes can be predicted and at least partially mitigated before they occur, ensuring a higher level of service, reduced user costs, and increased safety will be achieved due to proper application of preventive maintenance, thereby leading to delayed need for rehabilitation and subsequent life-cycle cost savings. Typical pavement preventive maintenance treatments include crack treatment, fog seal, slurry seal, scrub seal, microsurfacing, chip seal, thin overlay, and ultrathin bonded asphalt layer (UBAS; Peshkin, Hoerner, & Zimmerman, 2004).

6.1.1.1 Crack Treatment

Crack treatment fills and seals asphalt pavement cracks using a polymer-modified asphalt binder (i.e., thermoplastic bituminous material that softens when exposed to heat and hardens upon cooling). Crack sealing and crack filling are intended to prevent the intrusion of moisture through existing cracks. Prior to application of the sealant, proper crack cleaning is essential to ensure a good bond and maximum performance, and placement should be performed during cool, dry weather conditions (Peshkin et al., 2004). Figure 6.2 shows a crack-sealing operation on asphalt pavement.

Figure 6.2: Crack Treatment Operation on HMA Pavement
Source: Minnesota Department of Transportation (2014)

6.1.1.2 Fog Seal

Fog seal involves spraying a diluted water-based asphalt emulsion onto the pavement surface. The emulsified asphalt allows a very light layer to be evenly spread on the surface. The water evaporates, leaving just enough asphalt to fill in tiny cracks, cover the oxidized surface asphalt, waterproof the surface, and provide some pavement edge-shoulder delineation. Fog seal cannot be used on surfaces with poor skid resistance because it further lowers the skid resistance (Peshkin et al., 2004). Figure 6.3 illustrates application of fog seal on the asphalt concrete pavement.

Figure 6.3: Application of Fog Seal on the Asphalt Concrete Pavement
Source: United States Department of Agriculture Forest Services (1999)

6.1.1.3 Slurry Seal

Slurry seal is a mixture of well-graded aggregate mixed with asphalt emulsion. This treatment is used when the pavement primarily suffers from excessive oxidation and hardening of the existing surface (Chapter 2). Slurry seals retard surface raveling (loss of bond between aggregate particles and asphalt binder), seal minor cracks, and improve surface friction (Peshkin et al., 2004). Figure 6.4 shows the application of slurry seal on asphalt pavement.

Figure 6.4: Application of Slurry Seal on Asphalt Pavement
Source: Delaware Center for Transportation (n.d.)

6.1.1.4 Scrub Seal

Scrub seal is a four-step process that rejuvenates the asphalt surface and fills voids and surface cracks. The four-step process includes: (1) application of a layer of polymer-modified asphalt emulsion that is broomed into the voids and cracks of the pavement; (2) application of sand or small-sized aggregate; (3) a second application of polymer-modified asphalt (by brooming); and (4) rolling with a pneumatic-tired roller. As with fog seal, scrub seal should not be used on tight surfaces because it reduces skid resistance (Peshkin et al., 2004). Figure 6.5 shows the brooms used in scrub seal and the application of slurry seal on asphalt concrete pavement.

Figure 6.5: Application of Scrub Seal with Scrub Seal Brooms Placed at Different Angles
Source: National Center for Asphalt Technology (2013)

6.1.1.5 Microsurfacing

Microsurfacing is similar to slurry seal in that it requires application of a mixture of water, asphalt emulsion, aggregate (mineral aggregates and mineral filler), and chemical additives on an existing asphalt concrete pavement surface. Polymer is commonly added to the asphalt emulsion to improve mixture properties. The major difference between slurry seal and microsurfacing is the breaking, or hardening, method. Slurry relies on water evaporation in the asphalt emulsion. The asphalt emulsion in microsurfacing, however, contains chemical additives that allow it to break without relying on the sun or heat for evaporation. Thus, microsurfacing hardens more quickly than slurry seals and can be used when conditions would not allow a successful slurry seal. Very shady streets or areas with high levels of traffic are good candidates for microsurfacing (Peshkin et al., 2004).

6.1.1.6 Chip Seal

Chip seal is a two-step process that includes: (1) direct application of asphalt emulsion on the surface, and (2) addition of a layer of crushed rock (chips) that is immediately rolled to embed the aggregate into the binder and orient it into an interlocking mosaic. Chip seal is named for the chips, or small crushed rock, placed on the surface of the pavement. Asphalt emulsions in chip seal applications contain paving asphalt, water, an emulsifying agent or surfactant, polymer, and sometimes a rejuvenator. The surfactant in asphalt emulsions keeps the paving asphalt drops in suspension until the emulsion is applied to the pavement surface when water in the asphalt emulsion begins to evaporate. The chips are immediately applied after the asphalt emulsion has been applied to the pavement surface. The polymer in the asphalt emulsion is a hardener that improves adhesion to the crushed rock and the pavement surface. Application of the rejuvenating agent softens the pavement and improves the bond between the two layers (Peshkin et al., 2004, Gransberg & James, 2005). Figure 6.6 shows the application of chip seal on an existing asphalt pavement surface.

Figure 6.6: Application of Chip Seal: Asphalt Emulsion Applied First, Followed by Chip Spreader (Driving Backwards) and Roller
Source: Pavement Interactive (n.d.)

6.1.1.7 Thin Overlay

Thin overlay, also called thin hot-mix overlay, involves application of a thin layer of plant-mixed combinations of asphalt binder and aggregates on the existing pavement surface. Thin overlay thickness ranges from 19 to 38 mm (0.75 to 1.50 in.), and dense-graded, open-graded, or stone matrix mixes can be used. Thin overlays improve ride quality, reduce oxidation of the pavement surface, provide surface drainage and friction, and correct surface irregularities (Johnson, 2000; Peshkin et al., 2004). Figure 6.7 shows a new thin HMA overlay mat being compacted.

Figure 6.7: Compaction of an HMA Overlay
Source: National Center for Asphalt Technology (2016)

6.1.1.8 UBAS

UBAS requires placement of a thin, gap-graded HMA layer over a special asphalt membrane (tack coat) on top of the existing asphalt pavement surface. UBAS thickness is 10–20 mm. The tack coat (heavy polymer-modified asphalt emulsion) prevents water leakage and provides a superior bond to the old asphalt. The overlay disperses water quickly off the surface, reduces roadway spray from vehicles, and provides increased visibility in wet weather. UBAS also has advantages of effective resolution of minor surface distresses, increased surface friction, and quick installation. Figure 6.8 shows a UBAS layer on top of the existing asphalt pavement surface.

Figure 6.8: UBAS Layer on Top of Existing Asphalt Pavement Surface
Source: Kandhal and Lockett (1998)

Preventive pavement maintenance is not a new concept, but it has not been readily applied because available preventive maintenance treatments were considered unsuitable for high-volume roadways and insufficient information was available about the performance and cost-effectiveness of preventive maintenance practices. In addition, lack of federal aid for maintenance encouraged agencies to allow pavements to undergo enough deterioration to qualify for rehabilitation funded by federal aid (Johnson, 2000). Figure 6.9 shows pavement conditions and the role that maintenance plays in cost efficiency and pavement quality. As shown in the figure, spending $1 on pavement preventive maintenance can eliminate or delay spending $6 on corrective maintenance and $25 on rehabilitation or emergency maintenance (Jackson, Dave, Sebaaly, & Porritt, 2005).

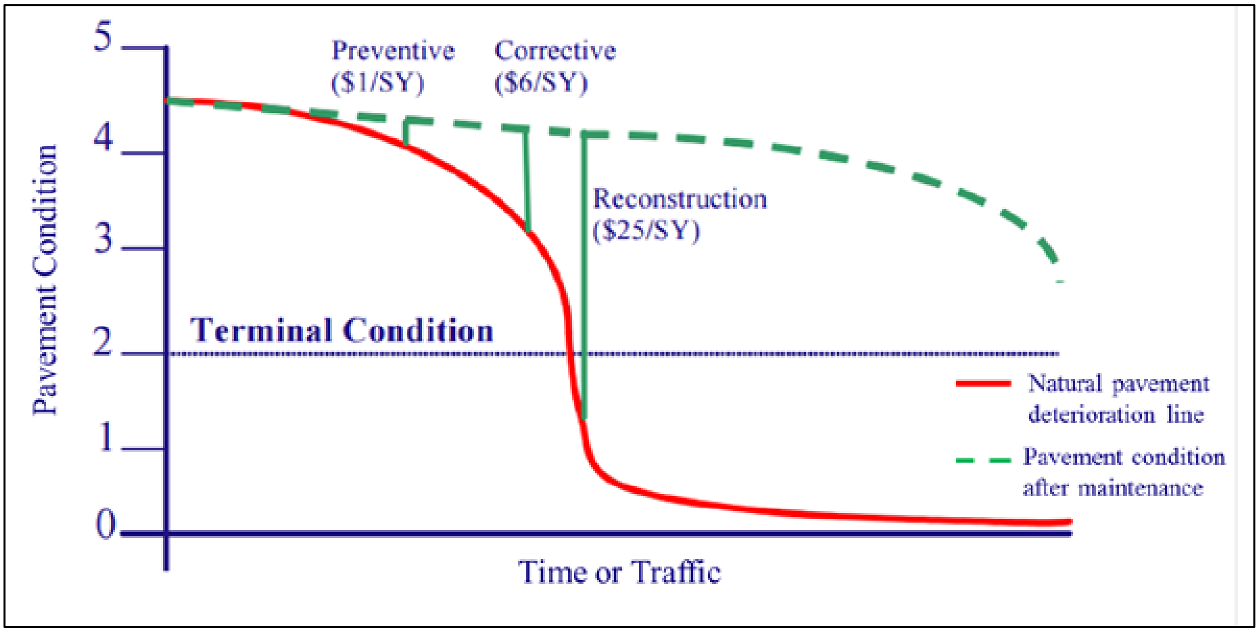


Figure 6.9: Pavement Conditions and Maintenance Related to Cost Efficiency and Pavement Quality
Source: Jackson et al. (2005)

Some highway agencies are also unwilling to conduct any treatments on pavements in good condition when a large backlog of pavements in poor condition exists within the system. Preventive maintenance is commonly ignored when potholes and other maintenance problems demand immediate actions and consume much of the limited maintenance budget. The public expects that problems such as potholes be fixed first, causing preventive maintenance work to be neglected. Winter maintenance also potentially restricts preventive maintenance funding. Preventive maintenance and snow and ice removal often operate under the same budget, but because of its direct effect on driver safety, winter maintenance is given a higher priority. Money that is left when winter maintenance is over may not be sufficient to fund an adequate preventive maintenance program (Johnson, 2000).

The United States Army Corps of Engineers (USACE) evaluated selected preventive maintenance surface treatments in order to find quantitative evidence that the application of preventive surface treatment can protect asphalt pavement from oxidation and aging. Oxidation is thought to be the most significant environmental effect contributing to the degradation of asphalt pavements. The USACE study showed that asphalt pavement preventive treatments can extend pavement life by 25% (5 years for a pavement with a 20-year lifespan), and reduce construction costs by approximately 10% and maintenance costs by 20% per year. The USACE study estimated that for a typical military airfield with one runway (cost approximately $2.5 million), the construction cost savings, not including maintenance savings, would be approximately $250,000 per year, or $1.2 million over a 5-year period. The United States Army has more than 150 runways in service, resulting in a 5-year savings of $180 million just for runways (Jackson et al., 2005).

6.1.2 Corrective Maintenance

Corrective maintenance is performed after a deficiency has occurred on the pavement in order to correct a specific pavement or area of distress (Peshkin et al., 2004). Corrective maintenance differs from preventive maintenance primarily in cost and timing, while some maintenance activities serve both functions (Brown et al., 2009). The basic difference between treatments is the pavement condition when treatment is applied, but no distinct boundaries indicate distinct differences in preventive and corrective maintenance or corrective and emergency maintenance categories (Figure 6.1).

Although preventive maintenance is performed when the pavement is still in good condition, corrective maintenance is performed when the pavement is in need of repair, making corrective maintenance more costly than preventive maintenance. Delays in maintenance increase pavement defects and severity of distresses, leading to higher correction costs resulting in significantly higher life-cycle costs for repair of localized areas of intensive cracking. If cracking has deteriorated the pavement to the point that disintegration of pavement material around the crack is occurring, the defective material must be removed and replaced (Brown et al., 2009).

Patching can be partial or full depth. Partial patching involves the removal of the distressed surface to the depth the surface is intact and replacing it with HMA. Full depth patching involves the complete removal of HMA layer down to subbase or even subgrade where the structure is intact. A patch made of HMA can be considered a long term repair and should last for years. If patching is used to fill potholes on the pavement in order to keep traffic moving, then it is considered as emergency maintenance and will serve as temporary before a permanent repair can be made (Brown et al., 2009).

Figure 6.10: Full Depth Patching on the Asphalt Pavement
Source: California Department of Transportation (2008)

6.1.3 Emergency Maintenance

Emergency maintenance is performed when an immediate pavement repair is required. Examples of emergency situations include blowouts (missing large sections of the pavement) and severe potholes that need immediate repair for safety or to allow traffic on the roadway. Emergency maintenance also includes treatments that hold the surface together until a more extensive rehabilitation or reconstruction treatment can be implemented. In emergency maintenance situations, typical considerations for choosing a treatment method, such as cost and long-term performance, are no longer mandatory. Cost is considered only after safety and required time for treatment application are considered. Materials that may not be acceptable when used in preventive or corrective maintenance activities become highly acceptable when used in an emergency situation (Johnson, 2000).

6.2 LCCA Procedure and Principles of Good Practice

Previous studies conducted on LCCA have suggested that proper application of LCCA techniques leads to more effective long-term investment decisions and more sustainable highway systems (Lamptey et al., 2005). LCCA should be conducted as early in the project development cycle as possible. The appropriate time for conducting LCCA for pavement design is during the project design stage. LCCA requires consideration of costs that differ among treatment alternatives. Costs common to all alternatives cancel out and do not need to be included in LCCA calculations. The following procedural steps comprise LCCA:

- Establish alternative pavement design strategies for the analysis period,
- Determine performance periods and activity timing,
- Estimate agency costs,
- Estimate user costs,
- Develop expenditure stream diagrams,
- Compute net present value (NPV),
- Analyze results, and
- Reevaluate design strategies.

The following sections briefly describe each step and its application in LCCA for this study.

6.2.1 Pavement Design Alternative Strategies for the Analysis Period

The primary purpose of an LCCA is to calculate the long-term effects of initial decisions on the future costs of maintenance and rehabilitation activities required for a predetermined minimum acceptable level of service for a specified time (Walls & Smith, 1998). The predetermined minimum acceptable level of service in Kansas is more than 80% of non-interstate and more than 85% of interstate pavements in good condition every year.

A pavement design strategy combines initial pavement design and necessary supporting maintenance and rehabilitation activities. The analysis period is the time horizon over which future costs are evaluated. The first step in conducting an LCCA of alternative pavement designs is to identify alternative pavement design strategies for the analysis period. The time period over which alternatives are evaluated should be long enough to reflect long-term cost differences associated with reasonable design strategies. FHWA's LCCA policy recommends a 35-year analysis period for all pavement projects. Regardless of the selected time span for LCCA, the analysis period should be the same for all alternatives (Walls & Smith, 1998). Figure 6.11 illustrates pavement design alternatives over the pavement design life.

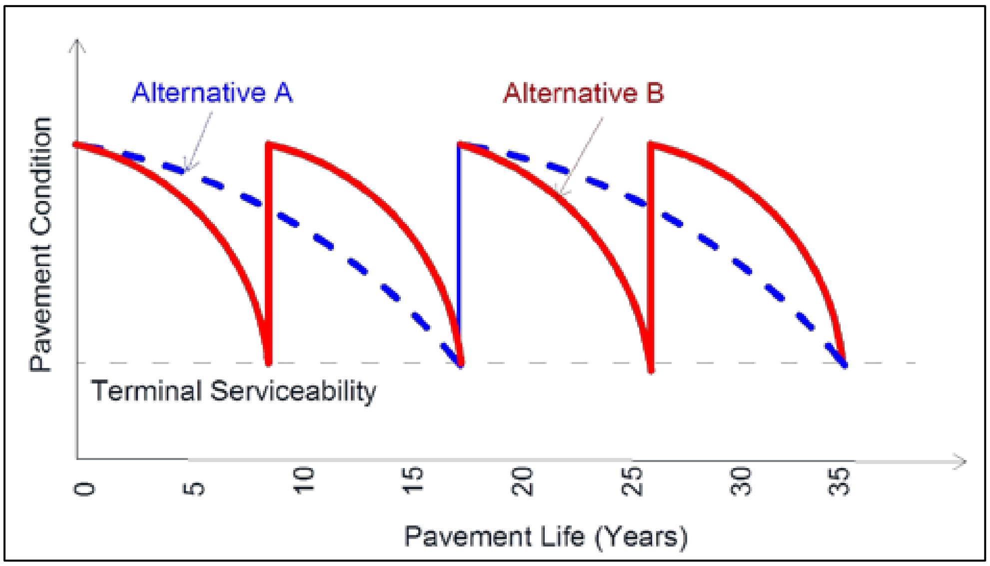


Figure 6.11: Pavement Design Alternatives and Pavement Design Life
Source: Walls and Smith (1998)

6.2.2 Performance Periods and Activity Timing

Performance life for the initial pavement design and subsequent rehabilitation activities significantly impacts LCCA results and directly affects the frequency of agency intervention on the highway facility, thereby affecting agency costs and user costs during construction and maintenance activities. State highway agencies (SHA) can determine specific performance information for various pavement strategies by analyzing pavement management data and past experiences. Operational pavement management systems (PMSs) can provide data and analysis techniques to assess pavement conditions and performance and traffic volumes to identify cost-effective strategies for short-term and long-term capital projects and maintenance programs (Walls & Smith, 1998).

6.2.3 Agency Costs

Agency cost estimations initially determine construction quantities/unit prices. Unit prices can be determined from SHA historical data on previously bid jobs of comparable scale. LCCA comparisons are always made between mutually exclusive competing alternatives, and LCCA considers only differential costs between alternatives. Costs common to all alternatives cancel out; these cost factors are generally noted and excluded from LCCA calculations. Agency costs include initial preliminary engineering, contract administration, construction supervision and construction costs, future routine and preventive maintenance, resurfacing and rehabilitation costs, and associated administrative costs (Walls & Smith, 1998).

6.2.4 User Costs

User costs are paid by the highway user over the life of the project. In LCCA, highway user costs are differential costs paid by the motoring public between competing alternative highway improvements and associated maintenance and rehabilitation strategies over the analysis period. For pavement design, user costs are further limited to user costs resulting from long-term pavement design decisions and supporting maintenance and rehabilitation implications. User costs are a combination of vehicle operating costs (VOCs), user delay costs, and crash costs (Walls & Smith, 1998).

6.2.5 Expenditure Stream Diagrams

Expenditure stream diagrams are graphical representations of expenditures over time, which are generally developed for each pavement design strategy to help visualize the extent and timing of expenditures. Costs are typically depicted as upward arrows at the time they occur during the analysis period, and benefits are represented as negative cost or downward arrows (Walls & Smith, 1998).

6.2.6 Net Present Value

LCCA is a form of economic analysis used to compare long-term economic efficiency of alternative investment options; NPV is the economic efficiency indicator of choice. Economic analysis focuses on the relationship between costs, timing of costs, and discount rates employed. Once all costs and timing have been developed, future costs must be discounted to the base year and added to the initial cost to determine the NPV for the LCCA alternative. The basic NPV formula for discounting discrete future amounts at various points in time back to a base year is provided in Equation 6.1 (Walls & Smith, 1998; Lamptey et al., 2005):

$$\boldsymbol{NPV} = \boldsymbol{Initial\ Cost} + \sum_{k=1}^{N} \boldsymbol{Rehab\ Cost_K}\left[\frac{1}{(1+i)^{n_K}}\right] \qquad \textbf{Equation 6.1}$$

Where:

i is the discount rate, and

n is years of expenditure.

When comparing alternatives, the alternative with the lower NPV should be selected as having more economic efficiency.

6.2.7 Analysis of Results

All LCCA outputs should, at a minimum, be subjected to a sensitivity analysis. Sensitivity analysis determines the influence of input assumptions, projections, and estimates on LCCA results. In a sensitivity analysis, major input values are varied (within some percentage of initial value or over a range of values) while all other input values are constant and the amount of change in results is noted. Input variables may then be ranked according to their effect on results. Sensitivity analysis allows the analyst to subjectively understand the impact of variability of individual inputs on overall LCCA results. Sensitivity analysis focuses on best-case/worst-case scenarios in order to rank outcomes. Most LCC sensitivity analyses, as a minimum, evaluate the influence of the discount rate (Walls & Smith, 1998; Labi & Sinha, 2003).

6.2.8 Design Strategy Reevaluation

Once NPVs have been computed for each alternative and sensitivity analysis has been performed, the analyst should reevaluate the competing design strategies. The overall benefit of

LCCA is not necessarily the results, but rather how the agency can use the information to modify proposed alternatives and develop more cost-effective strategies. LCCA results are just one of many factors that influence the ultimate selection of a pavement design strategy. The final decision may include factors outside the LCCA process, such as local politics, availability of funding, industry capability to perform required construction, and agency experience with a particular pavement type, as well as accuracy of the pavement design and rehabilitation models. Many assumptions, estimates, and projections contribute to the LCCA process. Variability associated with these inputs can significantly influence the reliability of LCCA results. The more accurate the forecast for future costs, pavement performance, and traffic for more than 30 years, the more reliable the LCCA outputs. A probabilistic risk analysis approach is needed to quantitatively capture uncertainty associated with LCCA forecasts (Walls & Smith, 1998; Lamptey et al., 2005).

6.2.9 Life-Cycle Cost Analysis of HIR

In order to establish alternative pavement design strategies, information regarding asphalt projects in Kansas in 2014 and 2015 was obtained from KDOT. This information included the history of pavement maintenance and rehabilitation for segments that have received any maintenance in 2014 or 2015. For the purpose of this study, the 30 years of maintenance history of five selected roadway segments were used to find out if applying HIR maintenance activities would have any economic justifications and lower NPVs. In Figure 6.12, the original scenario (Alternative 1) is the real maintenance history of the roadway segment (as extracted from KDOT data files). The second and third alternatives were fabricated by replacing actual activities conducted on the road segment by HIR activities. To conduct a comparison between alternatives, it is required to have an estimation of the cost and performance period of each real-life maintenance activity. KDOT asphalt project data sheets were used to acquire the data required for alternative comparisons. Average quantities were used whenever more than one cost or lifetime was associated with maintenance activities. Average costs and life expectancies for various alternatives are provided in Table 6.1, and expenditure stream diagrams for selected projects are provided in Figure 6.12.

To conduct life-cycle cost analysis, LCCA software provided by the asset management unit in FHWA was used (accessible at: http://www.fhwa.dot.gov/infrastructure/asstmgmt/lccasoft.cfm).

The software accounts for user costs by considering traffic type, terrain, AADT, capacity, speed limit, and the time required to complete the work activity. The software also takes posted speed limit, discount rate, and traffic growth rate into account when conducting the LCCA analysis for different alternatives. Similar to the speed limit in non-interstate major travel corridors in Kansas, 70 miles/hour was selected as the posted speed limit. Discount rate and traffic growth were chosen as 2% (uniform) and 4% (normal), respectively, after consulting with KDOT.

Table 6.1: Average Cost and Life Expectancy for Maintenance Alternatives

Treatment	Required time to complete the activity (hr/mile)	Avg. life expectancy (year)*	Avg. cost/mile ($)*
Chip seal	8	5	32,282
1" overlay	2	5	92,982
1.5" overlay	2.5	7	108,102
2" overlay	3	7	123,222
1" cold mill, 2" overlay	3	3	119,070
1.5" cold mill, 1.5" overlay	2.5	5	130,377
4.5" cold mill, 1.5" overlay	2.5	4	215,020
Rout and crack seal	10	5	7,000
2" HIR, 1" overlay	2	7	190,832
2" HIR, chipseal	8	5	95,719
1" HIR, 2" overlay	3	6	110,723
1" HIR, 1" overlay	2	6	110,723
2" HIR, seal coat	3	7	85,622

* based on KDOT 2014 and 2015 fiscal year data

Figure 6.13 shows the input page of LCCA software, illustrating general project inputs, traffic and roadway capacity inputs (were kept similar for all projects), and whether the probabilistic or deterministic type of analysis is used. The difference between probabilistic and deterministic analysis is that in deterministic analysis every input variable is treated as discrete fixed value. In reality, however, the majority of input variables are uncertain and it is important to be aware of the inherent uncertainty of the variables used as inputs into the analysis. For this reason, probabilistic approach was used in this study. Once all the input parameters for LCCA are estimated and put in place, the next step is to run a simulation of the model. In probabilistic analysis, a simulation is basically a rigorous extension of a sensitivity analysis that uses different randomly selected sets of values from the input probability distributions to calculate separate discrete results. By randomly drawing samples from the model's input distributions, the computer combines the variability inherent in the inputs and summarizes it in the form of a probability distribution. In this way, besides knowing the full range of possible values, the decision maker will have understanding of the relative probability of actual occurrence of any particular outcome. This is the type of information the decision maker needs to make reliable decisions. The results are arrayed in the form of a distribution covering all possible outcomes. The process of using random numbers to sample from probability distributions is known as Monte Carlo sampling (Walls & Smith, 1998).

The results of LCCA for five selected projects (Figure 6.12) are provided in Figures 6.14 to 6.18 and Table 6.2. Figures 6.14 to 6.18 show NPVs for agency costs, work zone user costs, and total costs, evaluated by considering uncertainties associated with each alternative. For instance, the original scenario in Project No. 2 (Figure 6.12) involves only one HIR activity (1" HIR + 1" overlay) as the last maintenance activity for the lifetime of the project. Alternative 2 is created by replacing 4^{th} project activity (1" overlay) by an HIR activity (2" HIR + Sealcoat). Figure 6.15 shows there is a 90% chance that agency costs associated with Alternative 2 are more than agency costs associated with Alternative 1. In other words, Alternative 2 has higher agency costs (by approximately $5,000 per mile) 90% of time during the 30 years of project lifetime when compared to Alternative 1. For the rest of the projects, the cost comparisons are very straightforward showing total costs associated with each project is highest for Alternative 1 (original scenario) and lower for Alternative 2 and Alternative 3, as more HIR maintenance activities are considered.

The LCCA software allows considering up to four alternatives for each project (only three are used in this study). Each alternative can include up to five work activities. Life expectancy and cost of each activity were defined based on the data provided in Table 6.1.

The project analysis period (General Project Inputs tab in Figure 6.13) can affect the NPV of different alternatives. Based on the available data on the history of each project (original scenario in Figure 6.12), analysis period was either 30 or 35 years for five projects under study but kept constant for different alternatives in each project. For instance, analysis period for Alternatives 1, 2, and 3 was 35 years for Project No. 1 and 30 years for Project No. 2. The difference in analysis period of Project No. 1 and Project No. 2 does not affect the LCCA, since the comparisons are made among different alternatives in the same project.

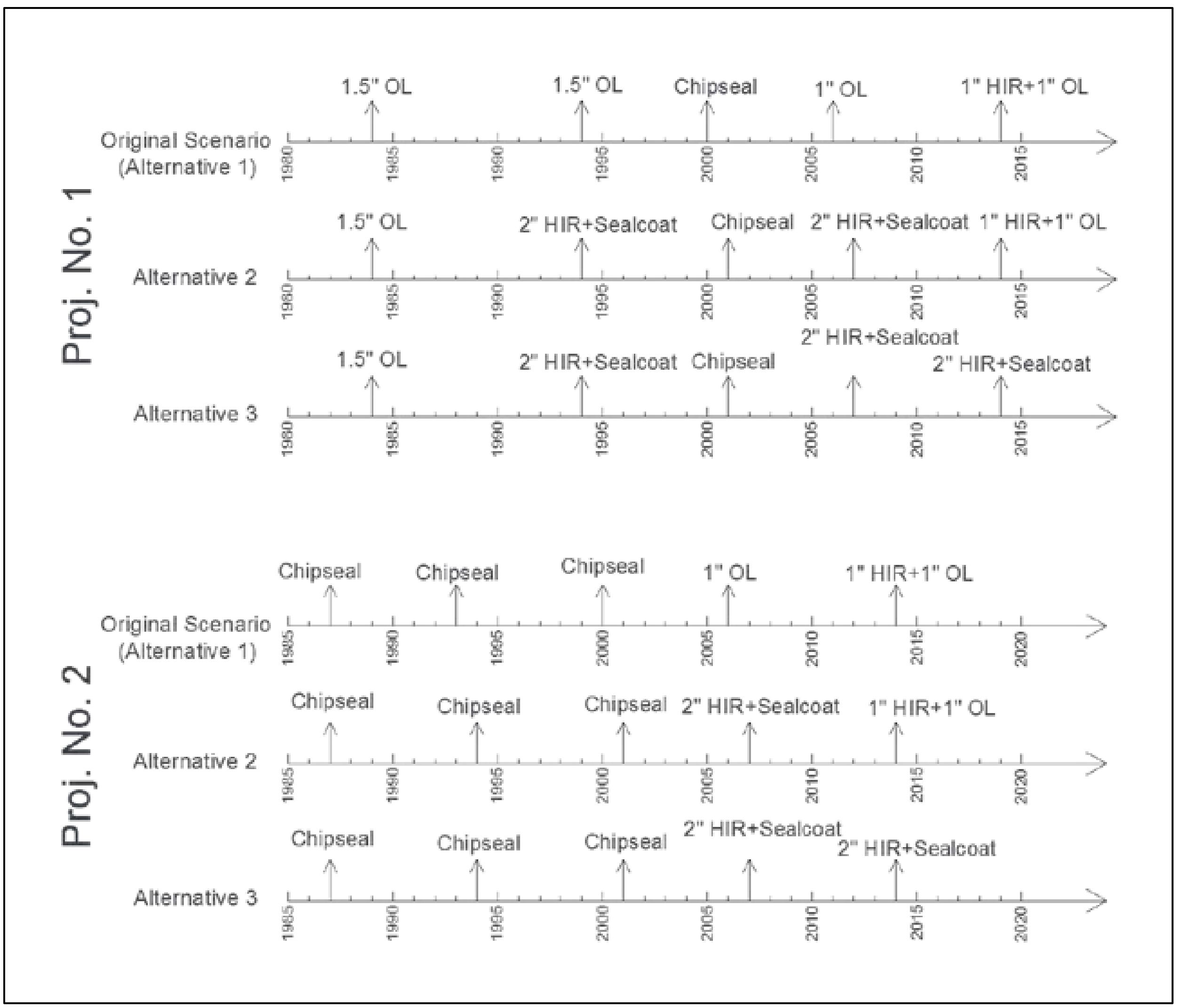


Figure 6.12: LCCA Alternatives for Selected Road Segments

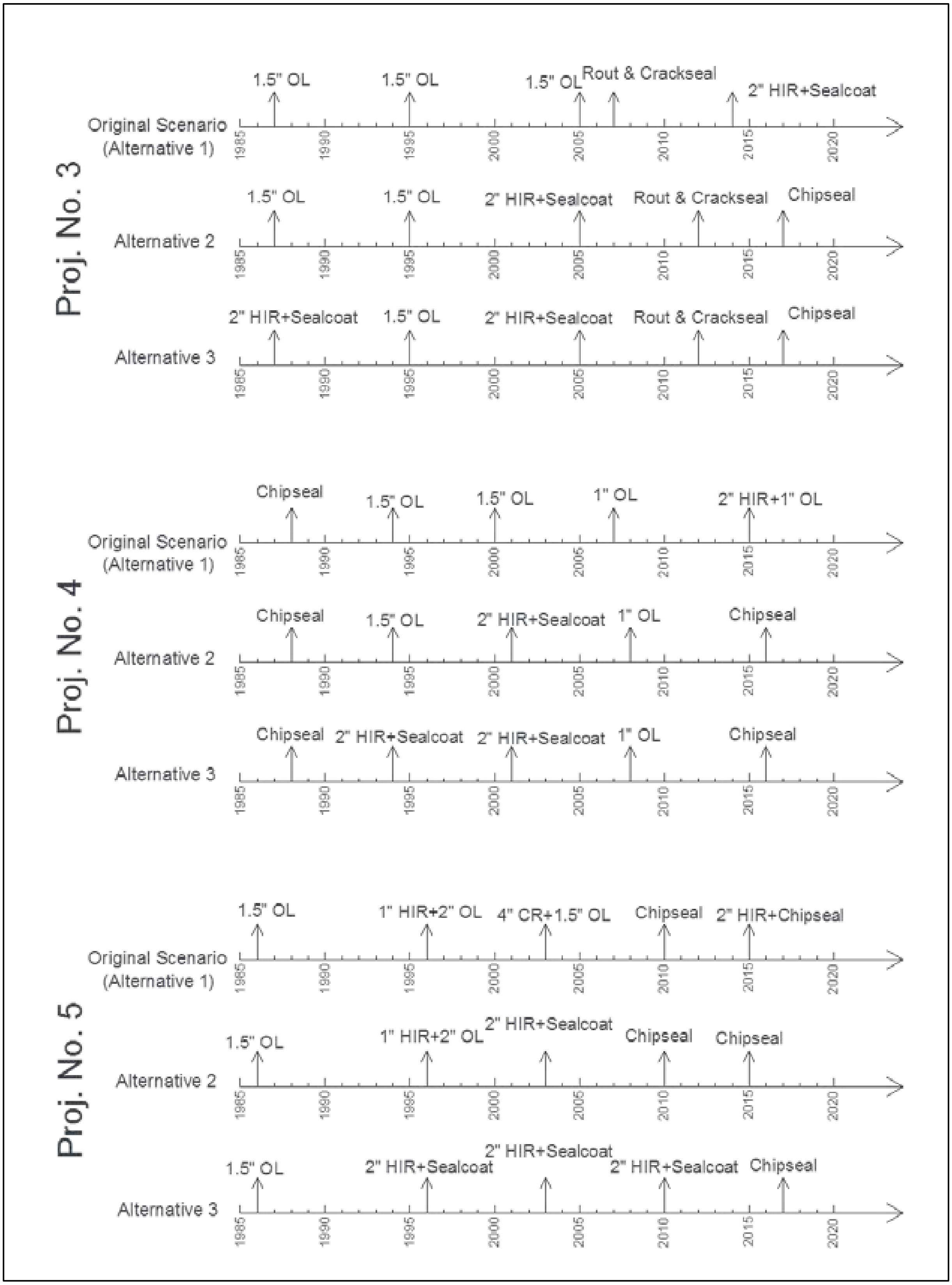


Figure 6.12: LCCA Alternatives for Selected Road Segments (Continued)

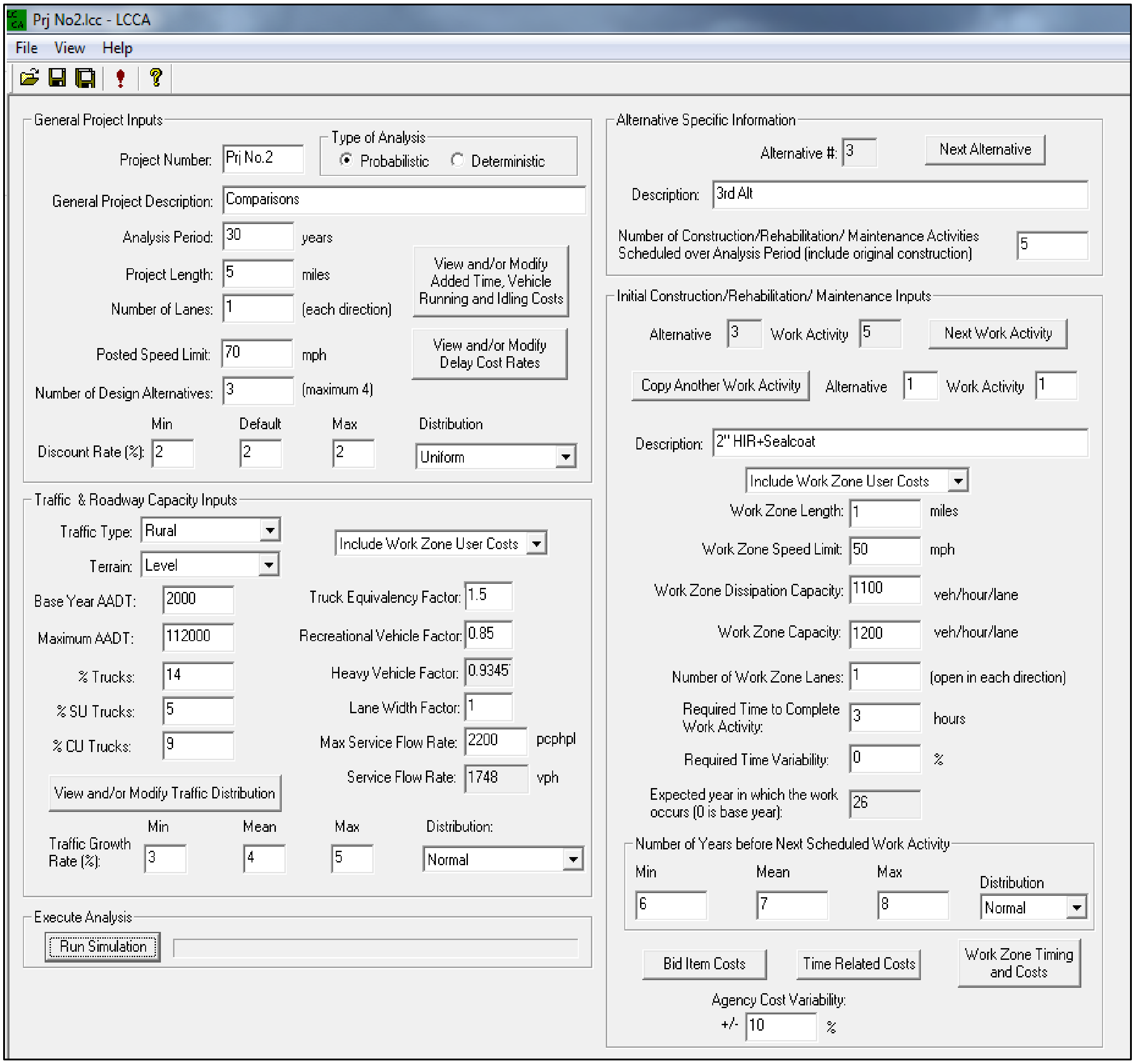


Figure 6.13: Screenshot of the LCCA Software Input Page

Table 6.2 shows the details of total costs NPVs and the amount of savings that could be made by including HIR activities in the lifetime of the asphalt project.

It should be mentioned that if deterministic analysis was chosen for LCCA, the output would have been one single deterministic result as provided in Figure 6.19.

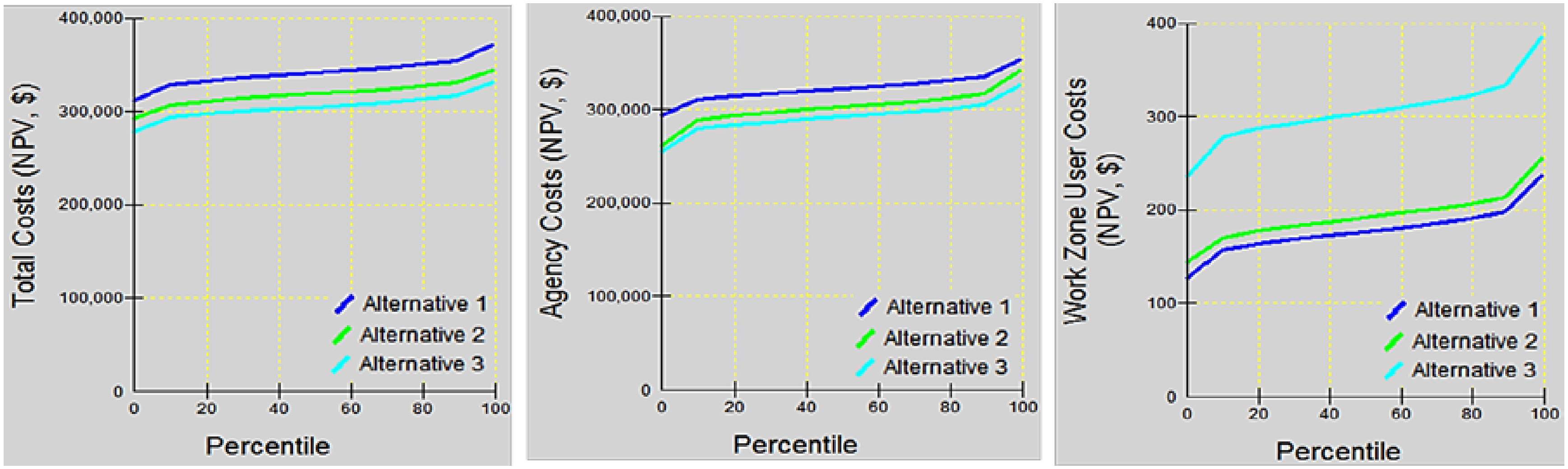


Figure 6.14: Comparisons of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 1

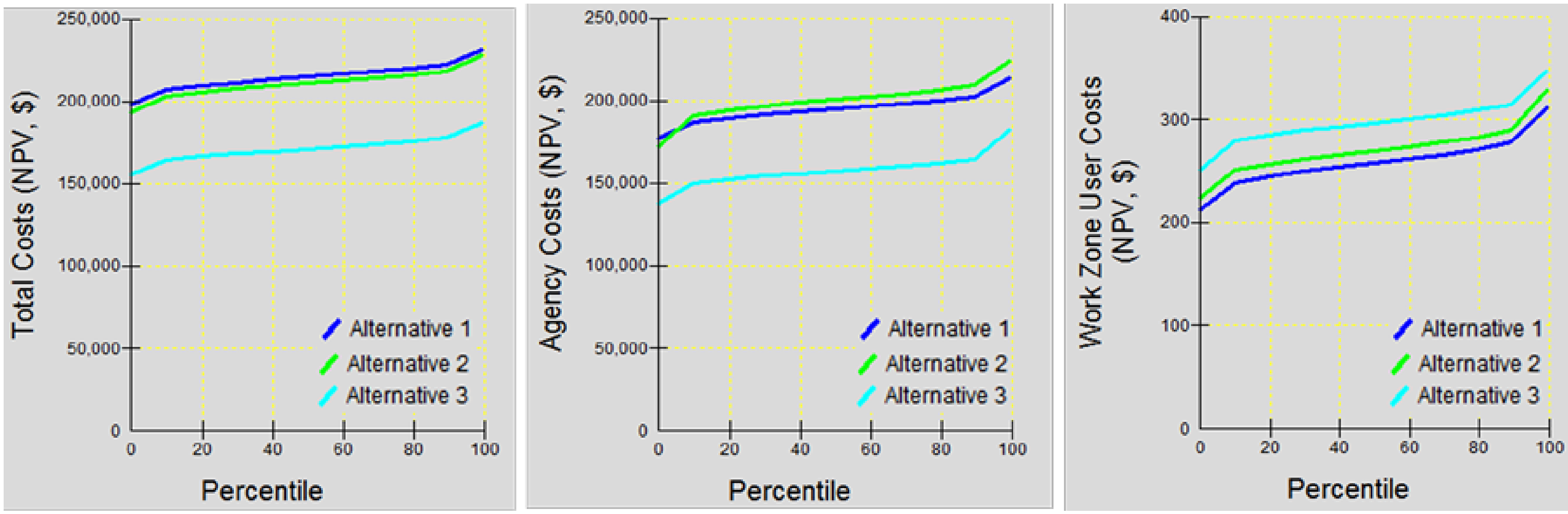


Figure 6.15: Comparisons of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 2

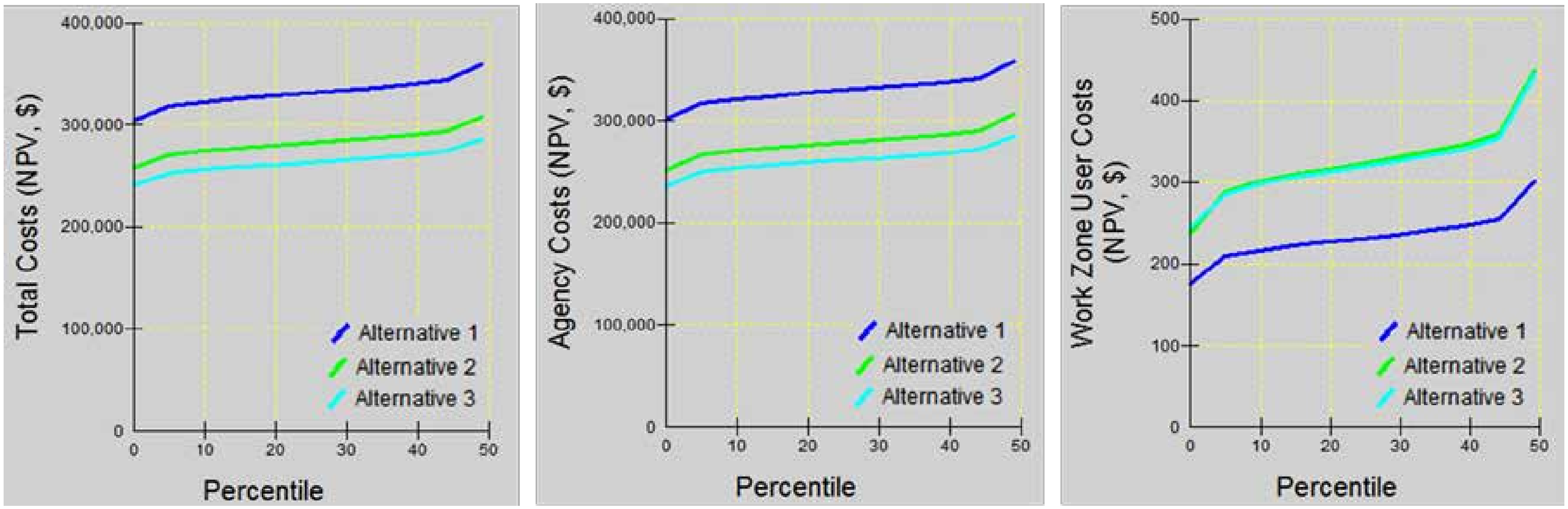


Figure 6.16: Comparisons of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 3

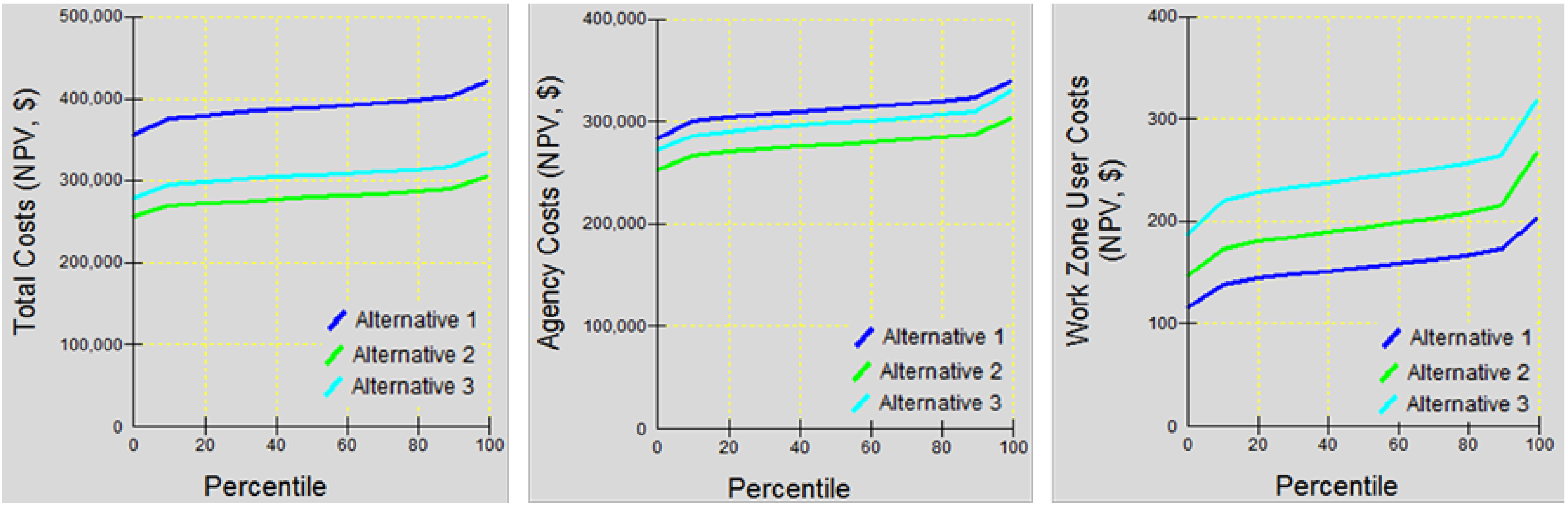


Figure 6.17: Comparisons of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 4

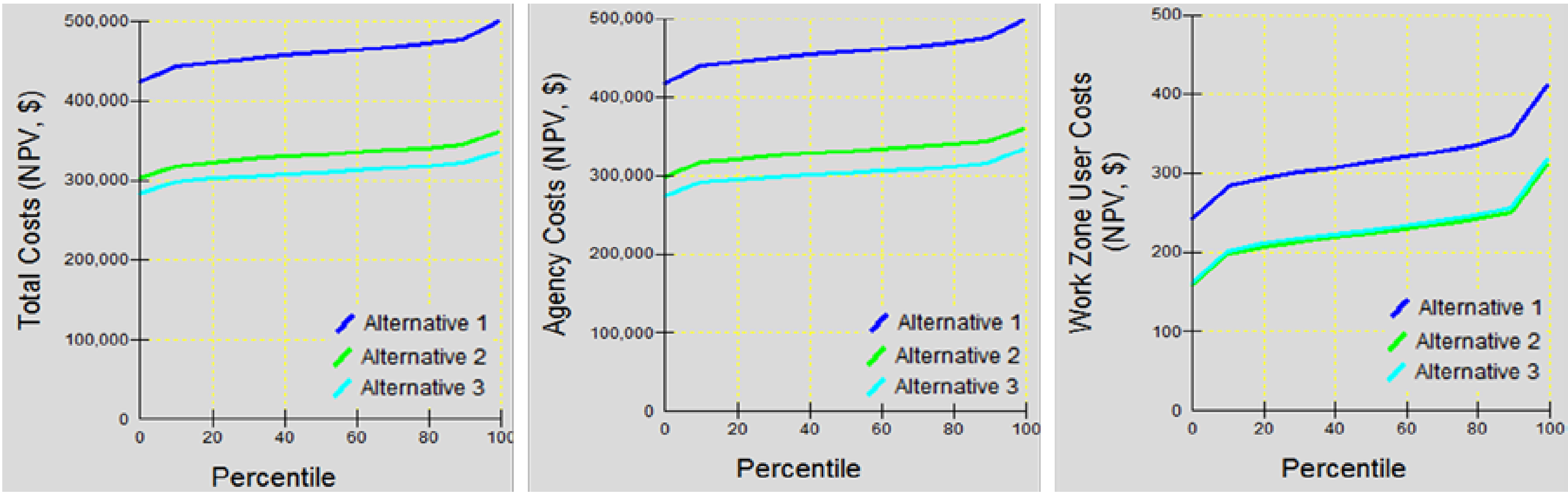


Figure 6.18: Comparisons of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 5

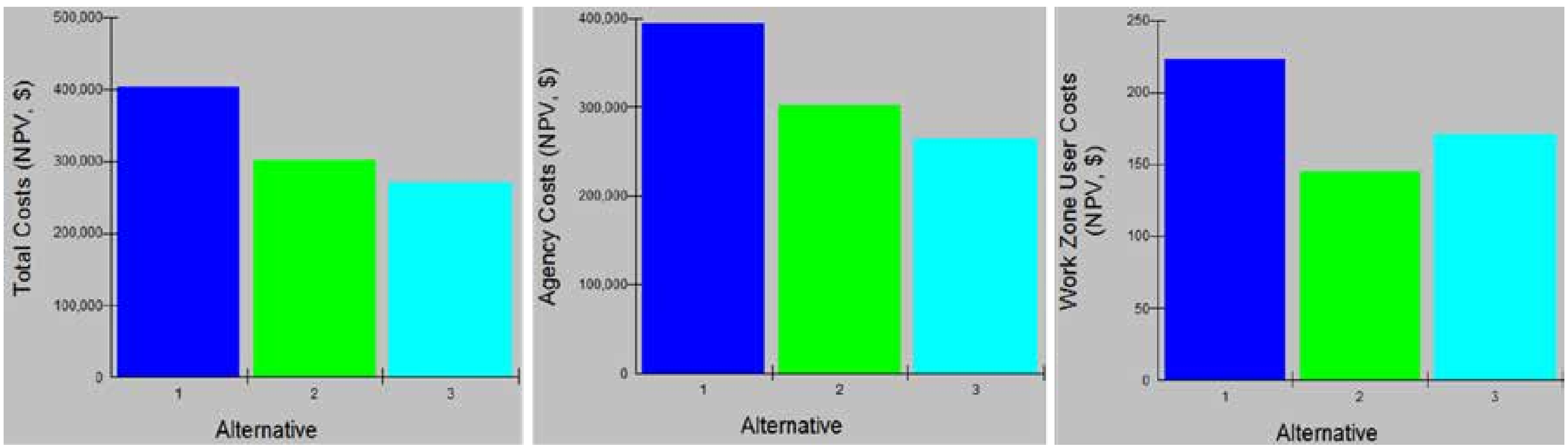


Figure 6.19: Deterministic Analysis Results of NPVs of Original Scenario (Alternative 1) and Alternatives 2 and 3 for Project 5

Table 6.2: Total Costs NPV Profiles for Alternatives 1, 2, and 3 and Associated Savings

	Percentile	Total Costs Net Present Value ($)			HIR Savings	
		Alternative 1	Alternative 2	Alternative 3	Alt 1 - Alt 2	Alt 1 - Alt 3
Project No. 1	0%	314,274	291,924	279,164	22,350	35,110
	10%	329,369	307,468	294,135	21,901	35,234
	20%	333,477	311,362	297,696	22,115	35,781
	30%	336,516	314,500	300,312	22,016	36,204
	40%	339,060	317,031	302,793	22,029	36,267
	50%	341,801	319,427	305,164	22,374	36,637
	60%	344,300	321,913	307,582	22,387	36,718
	70%	346,754	324,440	310,189	22,314	36,565
	80%	350,045	327,468	312,995	22,577	37,050
	90%	354,092	331,384	316,899	22,708	37,193
	100%	368,741	346,063	330,808	22,678	37,933
Project No. 2	0%	197,313	194,597	156,427	2,716	40,886
	10%	206,852	203,530	164,106	3,322	42,746
	20%	209,614	206,000	166,657	3,614	42,957
	30%	211,673	207,866	168,513	3,807	43,160
	40%	213,389	209,409	169,949	3,980	43,440
	50%	214,946	210,938	171,306	4,008	43,640
	60%	216,545	212,546	172,618	3,999	43,927
	70%	218,144	214,284	174,137	3,860	44,007
	80%	220,140	216,229	175,789	3,911	44,351
	90%	222,779	218,775	177,957	4,004	44,822
	100%	234,704	229,798	186,464	4,906	48,240
Project No. 3	0%	302,177	258,042	240,138	44,135	62,039
	10%	318,288	270,788	252,727	47,500	65,561
	20%	322,754	274,821	256,220	47,933	66,534
	30%	325,827	277,701	258,652	48,126	67,175
	40%	328,467	280,454	261,136	48,013	67,331
	50%	330,983	282,840	263,592	48,143	67,391
	60%	333,569	285,362	265,694	48,207	67,875
	70%	336,329	287,771	267,918	48,558	68,411
	80%	339,390	290,659	270,275	48,731	69,115
	90%	343,354	294,686	274,023	48,668	69,331
	100%	358,287	308,682	287,679	49,605	70,608

Table 6.2: Total Costs NPV Profiles for Alternatives 1, 2, and 3 and Associated Savings (Continued)

	Percentile	Total Costs Net Present Value ($)			HIR Savings	
		Alternative 1	Alternative 2	Alternative 3	Alt 1 - Alt 2	Alt 1 - Alt 3
Project No. 4	0%	357,963	257,161	276,641	100,802	81,322
	10%	375,215	269,327	294,589	105,888	80,626
	20%	379,872	272,478	298,428	107,394	81,444
	30%	383,302	275,238	301,251	108,064	82,051
	40%	386,309	277,771	303,493	108,538	82,816
	50%	389,112	279,703	305,717	109,409	83,395
	60%	391,852	282,014	307,819	109,838	84,033
	70%	394,613	284,025	310,157	110,588	84,456
	80%	398,047	286,548	313,128	111,499	84,919
	90%	402,914	290,349	316,916	112,565	85,998
	100%	422,242	303,123	331,196	119,119	91,046
Project No. 5	0%	423,017	290,394	265,079	132,623	157,938
	10%	442,608	317,513	281,806	125,095	160,802
	20%	448,516	322,441	285,610	126,075	162,906
	30%	453,020	326,005	288,590	127,015	164,430
	40%	457,032	329,138	291,309	127,894	165,723
	50%	460,794	331,837	293,713	128,957	167,081
	60%	464,459	334,683	296,129	129,776	168,330
	70%	468,475	337,677	298,789	130,798	169,686
	80%	472,973	341,105	301,900	131,868	171,073
	90%	478,677	345,507	306,919	133,170	171,758
	100%	497,547	362,923	335,500	134,624	162,047

As illustrated in Figures 6.14 to 6.18 and Table 6.2, LCCA shows that HIR is an economic maintenance alternative for asphalt projects (lower NPVs were associated with Alternative 2 and Alternative 3 for all five projects). Table 6.2 shows that pavement design strategies with HIR activities can provide considerable savings in the designed lifetime of the project.

Chapter 7: Conclusions and Recommendations

7.1 Conclusions

In HIR, existing pavement is heated and then the softened pavement is scarified and mixed with virgin aggregate and/or recycling agent and/or virgin asphalt mix. Various types of HIR processes are applicable based on the type of distress in the existing pavement. The three primary types of HIR processes are surface recycling, repaving, and remixing. Case histories for these three types show HIR effectively treats pavement distress at a much lower cost compared to conventional rehabilitation treatments. However, to ensure satisfactory performance of HIR operation, the cause of distress in the existing pavement must be determined and the applicability of HIR must be evaluated.

The primary objectives of this study were to investigate the effects of the addition of ARA on common performance indicators of HIR mixtures and to study the consistency and microstructure of the HIR binder when mixed with ARA. Based on obtained test results, the following findings were noted:

1. HWTD test results showed that although the addition of ARA has a slight softening effect on HIR mixture performance (lower number of wheel passes before reaching a rut depth of 20 mm for mixtures with ARA), HIR mixtures with ARA could certainly pass the Kansas requirement of a maximum rut depth of 12.5 mm after 10,000 wheel passes. Dynamic modulus and DSR ($|G^*|$ and $|G^*|/\sin\delta$) test results also confirmed the addition of ARA lowers the stiffness of HIR mixtures.
2. EDT results showed the US-56 mixture was the only mixture with acceptable binder consistency; the US-56 mixture with ARA showed consistent performance improvements (compared to the US-56 mixture without ARA) as evaluated by OT, TSRST, FN, and moisture susceptibility tests.
3. For US-59 and K-14, OT and flow number test results did not show any performance improvement after the addition of ARA. Low asphalt binder content, high stiffness, poor workability, and heterogeneity of the mixture

are speculated reasons for performance declines of US-59 and K-14 mixtures.

4. TSR and TSRST tests showed the same trend for all three locations; while the addition of ARA improved TSR and TSRST performance of US-56 and US-59 mixtures, K-14 mixture performance declined.
5. SEM and EDXS displays revealed a heterogeneous microstructure for US-59 extracted binder compared to the extracted binder of US-56.
6. LCCA showed that the inclusion of HIR activities into the maintenance plan of asphalt projects can lower NPVs when compared to previous maintenance practices. HIR is an economic alternative for many maintenance and rehabilitation activities commonly practiced in Kansas.

7.2 Recommendations

Although the improved performance of the US-56 mixture after the addition of ARA could have been the result of higher binder content and softer original binder (Table 3.1), thereby allowing easier diffusion of ARA, the declined or unimproved performances of the US-59 and K-14 mixtures demonstrate the need for HIR process improvements. The improvement of HIR process can include modification of ARA application rate and time/temperature of mixing (EDT results) or addition of virgin binder to allow for better diffusion of ARA (constant performance improvement in US-56 performance test results). While it is known that the content of asphalt binder and its microstructure affect the mechanical properties of asphalt binder, and consequently asphalt mixture, more research is needed to connect microstructural properties of asphalt binder with performance characteristics of asphalt mixture.

It is recommended that KDOT investigates possible specification changes in order to improve ARA mixing since the results of this study suggest that increased binder consistency leads to improved performance of HIR mixtures.

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