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Development of holding strategies for deteriorated low-volume roads and evaluation of performance of Iowa test sections

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Development of holding strategies for deteriorated low-volume roads and evaluation of performance of Iowa test sections

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ABSTRACT

Highway agencies in Iowa are challenged by the imbalance between available financial resources for pavement maintenance and the growing number of roads that are due for major rehabilitation or reconstruction. Funding priority is usually given to those roads that are part of the National-Highway-System (NHS). Rural low-volume roads (LVR) are usually not included in the NHS and may be managed by state highway authorities, counties, or townships. These LVRs provide accesses to the areas that are less populated; and, are sometimes allowed to deteriorate to a poor condition due to lack of funding for rehabilitation. Highway agencies are interested in holding strategies that are more aggressive than preventative maintenance but not as extensive as rehabilitation, to improve such roadways. The Iowa Department of Transportation (IDOT) constructed test sections using ten holding strategy treatments to aid in the development of appropriate guidelines.

Holding strategy treatments used various combinations of thin asphalt layers, surface treatments, and in-place recycling technologies, including cold in-place recycling (CIR) and full-depth reclamation (FDR). This dissertation presents a comprehensive study on these test sections based on various engineering criteria, including cost and life expectancy, and functional and structural performance measures. The current implementation of the technologies that were incorporated into the holding strategies was also reviewed. The findings in these studies show that the concept of holding strategy can be successfully achieved by selecting treatments that use a combination of various widely used thin hot mix overlays, thin surface treatments, and in-place recycling technologies. A treatment selection tool and recommendations to the structure design and safety performance are also developed in this dissertation.

CHAPTER 1. INTRODUCTION

1.1. Background

Highway agencies in the United States are facing the challenge of maintaining the pavement condition of highway network. The available financial resources for pavement rehabilitation grow slowly in comparison to the deterioration rate of the highway network. From 1999 to 2006, the non-interstate primary highways in poor condition had increased by more than 60% in Iowa (Iowa DOT 2008). In 2013, the highways that received a good rating were less than 47% in Iowa primary roadway system. It is estimated that the shortfall of annual transportation funding for meeting the most critical needs in Iowa is $215 million.

This challenge is more critical for low-volume roads than roads that carry higher traffic volume. Compared to roads with high traffic volume, low-volume roads usually have lower funding priorities. The *Moving Ahead for Progress in the 21st Century Act* (MAP-21) provides federal funding to state highway agencies to improve conditions of their infrastructure. MAP-21 established performance targets for National Highway System (NHS) which includes primarily interstates and primary roads that carry relatively high traffic volumes (FHWA 2014). No performance targets had set for secondary and local roads which usually carry low traffic volume. According to the Minnesota Department of Transportation (MnDOT) pavement condition report (MnDOT 2015), in 2014, 4.4% of the roads which are not included in the national highway system (NHS) in Minnesota were in poor condition with regard to ride quality. Meanwhile, the percentages of interstates and the other NHS roads with poor ride quality were 1.9% and 3%, respectively. With current funding level, it is projected that more than 10% of the Minnesota non-NHS roads will have poor ride quality condition by 2018.

In the past, the pavement maintenance strategy used by highway agencies was a "worst-first" strategy. The worst-first strategy refers as investing financial resources on major rehabilitation or reconstruction projects for roads that are in a poor or very poor condition. This strategy usually involves high costs for thick asphalt overlays or base material improvements. It was recently realized by pavement engineers and researchers that considerable savings can be obtained by implementing pavement preservation concept. A pavement preservation strategy involves applying preventive maintenance treatments, which usually are considerably lower in cost compared to major rehabilitation and reconstruction projects, to pavements that are still in good condition with a planned schedule. The treatments used for pavement preservation are usually thin surface treatments such as chip seal which prolong the service life of the surface or near-surface layer without adding significant structural capacity to the pavement structure. Many states such as California and Michigan have balanced both pavement preservation, and rehabilitation and reconstruction into a "mix-of-fixes" strategy in which the condition of each road is evaluated and treatments are applied with the goal of maximizing the performance of the road network and minimizing long range costs. The "mix-of-fixes" strategy includes three levels of treatments: reconstruction, rehabilitation, and preventive maintenance (Galehouse 2003). Reconstruction and rehabilitation are undertaken to roads with severe base and subgrade damage and insufficient structural capacity. Preventive maintenance is applied to roads with minor distresses which are only found in the surface layer. The minimum life extensions recommended for the three levels of treatments are 20, 10, and 5 years, respectively (Galehouse 2003) (Caltrans 2013).

One challenge for the "mix-of-fixes" strategy is that the successful use of preventive maintenance requires optimum timing which means that action should be taken relatively early in the road's service life. Premature or delayed maintenance activities will result in unnecessarily high maintenance costs. Many organizations have developed trigger values for preventive maintenance, rehabilitation, and reconstruction based on pavement performance evaluated through various pavement condition survey methods and non-destructive testing (NDT) (Hicks, Seeds and Peshkin 2000) (Smith 2001). However, highway agencies sometimes fail to apply appropriate treatment when the trigger value is reached for a particular road because financial resources are insufficient. It is desirable to extend the "time window" by maintaining the road conditions using holding strategies. A holding strategy is defined as the pavement management strategy which postpones major rehabilitation or reconstruction of a deteriorated road section with applications of treatments that are more aggressive than preventive maintenance treatments, with lower cost and most likely shorter service lives when compared to rehabilitation strategies (Yu, Jahren and Williams 2015). Holding strategies provide highway agencies flexibility in funding allocations and help the transit from a "worst-first" strategy to pavement preservation. The long-term goal of adopting holding strategies is to improve the overall condition of the highway system.

1.2. Problem Statement & Objectives

Holding strategies as a pavement management concept became more and more an item of interest in recent years; however, the treatments that can be used to meet the goals of holding strategies have not been studied in the context of holding strategies. The definition of holding strategy requires the treatments to have lower construction costs than traditional rehabilitation or reconstruction methods, which usually involve a thick hot mix asphalt (HMA) layer and, sometimes, replacement of the base material which results in considerably high construction costs. Some pavement maintenance treatments, such as thin or ultrathin asphalt overlays, thin surface treatments, and in-place recycling, have relatively lower costs compared to those traditional rehabilitation and reconstruction methods. These lower-cost pavement maintenance treatments or treatment combinations may be utilized to achieve the goals of holding strategies.

The thin or ultrathin asphalt overlays and thin surface treatments are usually used as preventive maintenance treatments for pavements that are in good condition. However, pavements where the use of holding strategies would be desirable are suffering from relatively severe deterioration that is beyond the scope of pavement preservation. Applying such treatments to heavily deteriorated roads is not considered to be cost-effective from the pavement preservation perspective, because such treatments are not expected to extend the service life of a deteriorated road with what is usually thought to be good performance for a substantial time. However, there was no quantitative studies have been found that specifically address the application of pavement maintenance treatments to heavily deteriorated roads.

In-place recycling technologies, such as cold in-place recycling (CIR) and full-depth reclamation (FDR), destroy distress patterns, do not add to the thickness of pavement structure, and rejuvenate aged binder by recycling agents which produce a stable base for surface layers. The typical practice in Iowa is to apply a thick asphalt overlay (more than 3 inches) to the CIR or FDR treated pavement. Few documented projects have specified the placement of a thin asphalt overlay or thin surface treatment over the recycled layer.

Because of the lack of historical performance records and quantitative cost-effectiveness study with regarding these treatments applied to severely deteriorated roads, the potential of these treatments as holding strategies requires verification. The objectives of this study are: 1. to develop holding strategy treatments through evaluation of the performance and cost-effectiveness; and 2. to develop a decision tool to assist the selection of appropriate holding strategies for a specific scenario.

1.3. Research Methodology

To achieve the primary objectives of this research project, a test road that includes ten test sections using various treatments that are proposed to be holding strategies was constructed. A comprehensive investigation of the construction technologies used in the treatments and the functional and structural performance and cost-effectiveness of the test sections was performed.

The research project described herein was completed in seven phases. The first phase includes a thorough literature review covering lower-cost pavement maintenance technologies that can be potentially used as part of a holding strategy and documentation of projects that were previously constructed using similar treatments as those envisioned in this study. The literature review was performed through the review of relevant publications from international journals, conference papers and proceedings, government website, and other documents.

The second phase of this research work involved the evaluation of the pre-construction pavement condition of the test road. Historical documents regarding the road, including construction plans and the pavement management information system (PMIS) database, were reviewed. A pre-construction pavement condition survey was also conducted. The goal of this phase was to understand the distresses that were present and their causes as well the geometry, traffic volume, pavement structure, and historical performance of this road.

In Phase 3, the construction of the test sections were documented. The design, construction procedures, quality control/assurance measures, and material quantities and construction costs were recorded. The day-to-day construction activities were also documented through photographs, videos, and the inspector's diary.

The fourth phase of this research project involved the evaluation of the post-construction functional performance of the test sections. This part of the research work includes post-construction pavement condition surveys and surface characterization tests. The pavement condition surveys measure the extent of surficial distresses and defects through visual observations and survey tools. The surface characterization tests evaluate the roughness of the test sections and the characteristics of micro and macro-texture of the test section surfaces.

Phase 5 involved the investigation of the structural performance of the treatments using in-situ non-destructive tests as well as laboratory tests employed on field core samples. Falling weight deflectometer (FWD) tests were performed before and after the construction of the test sections. The FWD test results were used to estimate the effective structural number (SN_{eff}) and compare the SN_{eff} of each test section with that of the road before construction. Laboratory dynamic modulus tests were carried out to evaluate the stiffness behavior of each layer in the treatments. The structural coefficients of the treatment layers were also determined using the dynamic modulus results and FWD results backcalculations.

Phase 6 involved the execution of a life-cycle-cost-analysis (LCCA) for various holding strategies that were applied to the test sections. Based on the LCCA results and the findings from the literature review on the suitability and construction limitations of the technologies that were used to construct the test sections, the treatment methods that could be utilized for holding strategies are recommended; and a decision tool was developed for treatment selection.

Lastly, Phase 7 is the culmination of the previous phases that are documented in this doctoral dissertation. The tests and analysis results are synthesized in order to draw conclusions; and the findings are presented in four journal papers.

1.4. Dissertation Organization

This dissertation is divided into seven chapters, composed of an introduction, literature review, four journal articles, and a summary. The introductory chapter provides background information about holding strategies as well as a discussion of the problems that this dissertation addresses and summarizes the methodology for the subsequent research effort. The literature review chapter addresses the definition, advantages and limitations, and life expectancies of the technologies that can be potentially used for holding strategies and documents the investigation of treatment methods similar to those envisioned under this project. Each of Chapters 3 through 6 includes an individual paper discussing one facet of this investigation. The conclusion chapter provides an overview of the efforts that was intended to achieve the goals of this study and summarizes the findings and recommendations that are intended to improve the future implementations of holding strategies.

Chapter 3 is a journal article that introduces the general concept of holding strategies and documents the design and construction of the test sections. This article also discusses the performance of various different surface types during severe winter weathers and proposes a preliminary decision table for holding strategy selection. This article has been published in the *Journal of the Transportation Research Board*. The goal of this article is to raise practitioner and researcher interest and awareness about the possibility of using holding strategies.

Chapter 4 is a journal article that documents an investigation of the test section performance and life-cycle cost-effectiveness; it uses pavement condition survey results and LCCA to identify treatment methods that are appropriate candidates for holding strategies. An improved decision tool is also developed as assist with treatment selection.

Chapter 5 is an article which investigates the functional performance of the test sections in terms of surface characterizations. Differences of the micro and macro-textures and roughness between chip seal and asphalt surface are discussed. The influences of pavement structure, traffic, and snow removal activities on various types of surface are also documented.

Chapter 6 presents an article that evaluates the structural performance of the test sections through field and laboratory material testing. The influences of the holding strategy treatments on pavement structural capacity are discussed. The structural layer coefficients of individual treatment layers are also estimated.

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CHAPTER 2. LITERATURE REVIEW

Treatment methods that are specifically designed to be used as holding strategies have not been well developed. Technologies that can be potentially used for holding strategies include thin and ultrathin asphalt overlays, thin surface treatments (TST), and in-place recycling. This investigation work focuses on treatment methods that involve combinations of these technologies. This chapter consists of a thorough review of the individual technologies and a search for similar treatment combinations that had been used elsewhere. It is found that although the individual treatments have been successfully used and widely accepted elsewhere, documented use of these treatments or combinations thereof for severely deteriorated roads are few.

2.1. Thin Asphalt Layer

Thin asphalt layers include thin and ultrathin asphalt overlays and thin asphalt interlayers. Thin asphalt overlay usually refers to asphalt surface course with layer thickness less than 1.5 inches (Caltrans 2008) (Dave 2011) (Huddleston 2009) (Sauber 2009). The California Department of Transportation (Caltrans) defines the layer thickness of thin asphalt overlay as less than 1.25 inches (Caltrans 2008). Ultrathin asphalt overlays usually have a lift thickness less than 1 inch (Caltrans 2008) (Dave 2011) (Huddleston 2009) (Sauber 2009). Thin asphalt overlays are usually used for pavement preservation. This treatment is effective in improving pavement function and correcting surficial deficiencies such as raveling, non-load related cracking, and rutting or shoving that is only limited to the surface layer (Newcomb 2009). Newcomb reviewed various studies on the performance of thin overlays (Newcomb 2009). The studies were conducted in various years from 1994 to 2009 and a wide range of locations including various states in US, and Austria and Canada. The results show that the life expectancy of thin asphalt overlays ranges from 5 to 16 years. Lower life-cycle costs were also attributed to thin asphalt overlays when compared to other preventive maintenance treatments (Chou, Datta and Pulugurta 2008).

A commonly used type of ultrathin overlay is the ultrathin bonded wearing course, also known as the open-graded friction course (OGFC). OGFC uses high quality gap-graded aggregate and polymer-modified asphalt binder. The typical lift thickness is between 15 mm (0.6 inch) to 20 mm (0.8 inch) (Gilbert, Olivier and Gale 2004). The ultrathin asphalt layer is placed onto a thick polymer-modified asphalt tack coat which improves the bond strength between the ultrathin layer and the underlying pavement surface. Special paving equipment is used to apply the tack coat and the OGFC in a single pass. OGFC improves functionality of roads that are losing skid resistance and where roughness is an issue; it also provides a waterproofing layer that protects the underlying pavement structure from water damage. The life expectancy of OGFC is between 8 and 12 years (Gilbert, Olivier and Gale 2004).

When the thin and ultrathin asphalt overlays are used for pavement preservation, it is often required that the underlying pavements have a sound structure; and that distresses are minor. Pavements with evidence of insufficient structure, such as longitudinal cracking on wheel paths, rutting in base layer, and alligator cracking, should be treated with more aggressive treatments in comparison to thin and ultrathin overlays. Newcomb recommends that thin asphalt overlays should be used for pavements with distress that extends for less than 10 percent of the project (Newcomb 2009). For OGFC, the candidate roads should have a remaining life of 6 to 8 years (Gilbert, Olivier and Gale 2004). If the treatments are used as holding strategies, these criteria will not be met. No quantitative study of thin and ultrathin overlays being employed on severely deteriorated pavements was found by the author.

A thin asphalt interlayer is typically used as a stress relief layer to minimize reflective cracking (Montestruque, et al. 2012) (Laurent and Serfass 1993). The thin interlayer is usually placed between the cracked pavement surface and the new surface course. The typical lift thickness is 20 mm (0.8 inch) to 30 mm (1.2 inch) (Montestruque, et al. 2012). The asphalt mixture consists of fine aggregate (usually less than 3/8 inch) and high percent polymer-modified asphalt (up to 7.5%). The purpose of such mix design is to create a strong and highly flexible layer which absorbs part of the crack wall movement and reduces shear and tensile stresses at the interface of the layers above existing cracks (Montestruque, et al. 2012). Sometimes, a geosynthetic membrane is applied in combination with the thin asphalt interlayer to further improve the anti-reflective cracking capability (Montestruque, et al. 2012).

The thin mat thicknesses of the thin asphalt overlay and interlayer produce additional quality control issues in comparison to the conventional asphalt overlays (Newcomb 2009). Fine aggregate gradation requires additional monitoring of aggregate moisture for possible impacts on asphalt content. It is difficult to measure the in-place mat density. Readings from a density gauge become inconsistent and less accurate if the layer thickness is less than 1 inch. Core samples are also difficult to obtain. Special attention should be paid to pavement temperature during compaction. The mat temperature for thin overlays decreases faster than that for thicker asphalt layers. It is important to maintain a fast and consistent compaction operation and perform the construction during favorable weather conditions.

2.2. Thin Surface Treatment

A TST is also known as a light surface treatment (LST) or a bituminous surface treatment (BST) (Dayamba, Jahren and Yu 2015). A TST is a thin layer of liquid asphalt and aggregate cover with an application thickness less than one half inch (Li, et al. 2007). TSTs are usually used in pavement preservation to seal minor cracks, correct surface defects, improve road functionality, and provide a waterproofing layer which prolongs the road service life. TSTs are considered to have no structural capacity during pavement design (Peshkin, Hoerner and Zimmerman 2004). In some places, TSTs were used on aggregate-surfaced roads to control dust, improve functionality, and decreased maintenance difficulties (Dayamba, Jahren and Yu 2015).

A variety of treatments are considered as TSTs including chip seal, slurry seal, cape seal, sand seal, Otta seal, and others.

A chip seal is constructed by applying an asphalt emulsion on road surface and covering it with single-sized aggregate particles. Rollers are used to embed aggregate particles into the asphalt layer in order to achieve the target embedment. The embedment rate refers to the percent of the height of the aggregate to which the asphalt rises. An optimum embedment of 70% is usually desirable (Caltrans 2008) (SME 2012). Sometimes, a chip seal using polymer modified asphalt is used as a stress absorbing interlayer (Caltrans 2008). In such case, the chip seal is placed between the existing pavement surface and the asphalt overlay to prevent cracks from reflecting through. A double chip seal is also used to provide additional protection for the underlying pavement structures. A double chip seal consists of two applications of chip seal. The aggregate of the upper layer usually has smaller particle size than that of the lower layer. The life expectancy of chip seal ranges from 3 to 5 years (Nantung, Ji and Shields 2011) (Maher, et al. 2005).

A slurry seal is a thin layer that is a mixture of asphalt emulsion, fine graded aggregate, mineral filler, water, and additives (ISSA 2001). The aggregate used for slurry seal is required to pass a 3/8-inch sieve. Slurry seal is constructed with a paver designed specifically for applying slurry seal treatment. A microsurfacing is a special type of slurry seal, that uses a rapid setting polymer modified asphalt emulsion and high quality aggregates which produce a stiffer mixture that requires less curing time in comparison to slurry seal. Microsurfacing is used for circumstances where a slurry seal fall short of meeting the requirements imposed by high traffic volume and limited road closure times. It is also used as a reactive treatment for rut filling (Zhang and Tian 2014). The life expectancy of slurry seal and microsurfacing ranges from 3 to 8 years (Nantung, Ji and Shields 2011) (Maher, et al. 2005).

A cape seal is constructed by applying a slurry seal over a chip seal. This combined treatment is more protective for the pavement structure than either of the individual treatments. The smoother texture of the slurry seal also mitigates concerns regarding lower drivability of a chip seal surface.

A sand seal is similar to a chip seal and is constructed by applying an asphalt emulsion film this is covered with sand size fine aggregate. A sand seal is often used as a temporary treatment to restore surface texture and repair raveling (WSDOT 2003). Due to the small particle size of aggregate, sand seal has a smooth surface texture. The treatment is recommended for use in areas where a high quality aggregate source is not available in the vicinity (Greening, Gourley and Tournee 2001).

An Otta seal is constructed by placing a thick application of relatively soft asphalt emulsion and covering it with a graded aggregate (Johnson and Pantelis 2008). The construction process is similar to that of chip seal. Rollers are used to embed the aggregate into the binder layer. Otta seal applications can often use relatively low quality, locally-available aggregate, and sometimes provide cost savings (Johnson and Pantelis 2008). The gradation of aggregate is usually coarser than that of the aggregate for a sand seal. The treatment can be used in areas where a quality aggregate source is not available. An Otta seal often has higher tolerance for construction faults than other TSTs. The end product of an Otta seal can be more effective in retarding the aging of the asphalt in the underlying layers in comparison to a chip seal (Overby and Pinard 2013).

Liu et al. conducted a study on various TSTs that are used for pavement preservation in Kansas (Liu, Hossain and Miller 2010). The definition of TST in Liu et al.'s research includes the TSTs that are defined in this report as well as thin asphalt overlays. The study analyzed the performance data of all roads which received a TST in Kansas from 1992 to 2006. The results indicate that the service life of TSTs on high-volume roads is significantly shorter compared to TSTs on lower-volume roads. In comparisons to thin asphalt overlays, chip seal appears to have a lower service life. The average service life of chip seal on non-Interstate highways is 5 years. Slurry seals on Interstate highways exhibited higher service lives in comparison to chip seals; while the service lives of slurry seals and chip seals on non-Interstate highways were comparable. Chip seal had the lowest annual cost among all the treatments that were compared. The equivalent uniform annual cost (EUAC) of chip seal is less than half of the EUAC of slurry seal and less than 20% of the EUAC of 3-inch overlay.

A research conducted by Wang et al. quantified the cost-benefits of various types of TSTs in Pennsylvania including crack sealing, chip seal, microsurfacing, thin overlay, and NovaChip (similar to OGFC) (Wang, Morian and Frith 2013). The study compares the EUAC of each TST with a do-nothing alternative using the Pennsylvania Pavement Management Information System (PMIS) data from 1998 to 2008. It was found that crack sealing had the highest benefit-cost ratio; while NovaChip had the lowest benefit-cost ratio. The EUAC of the TSTs varied with the condition of the existing pavement when treatments were applied. In order to quantify the effects of existing pavement condition on the service life extensions provided by the TSTs, performance models were established using Pennsylvania overall pavement index (OPI). The results indicated that the pavement life benefits of TSTs started to decrease significantly when the OPI of existing pavement decreased below a trigger value. The trigger values for chip seal and microsurfacing on highways with less than 2000 average daily traffic (ADT) are about 85 and 90, respectively. Such OPI values typically occur at 5 to 6 years after initial construction. The life extensions at optimum timing are 4 and 7 years for chip seal and microsurfacing, respectively.

Previous investigations regarding TSTs were primarily focused on when TSTs are used as a preventive maintenance treatment. In order for the treatments to be effective and achieve the maximum cost-benefits, the candidate roads need to be in good conditions. Few case studies were found for TSTs used on deteriorated pavements as a rehabilitation treatment.

2.3. In-place Recycling

In-place recycling technologies are usually used for rehabilitation of deteriorated asphalt pavements. The commonly used in-place recycling methods include hot in-place recycling (HIR), cold in-place recycling (CIR), and full depth reclamation (FDR). In-place recycling technologies are considered environmentally friendly and lower-cost alternatives to the conventional overlay method of reconstruction. Old pavement materials are recycled and used immediately after the recycling process to produce new materials in place. Therefore, the cost, energy, and resource savings can be realized by eliminating the effects of production of new materials, hauling, and handling and storage. The required hours of labor and time for a rehabilitation project are also decreased.

2.3.1. Hot in-place recycling

HIR uses a heating unit to soften the existing pavement by heating it to 110 ℃ to 150 ℃ (FHWA 2005). A grinding unit is used to pick up the heated pavement and convey it to a mixing unit where virgin aggregate and binder are added to produce the recycled materials. HIR is used for treating surface distresses and defects of roads with a sound structure. The treatment depth is typically 3/4 to 1 inch and does not exceed 2 inches (Finn 1980). The efficiency of heating unit is significantly affected by surface treatments, such as chip seal (Pierce 1996). Removal of surface treatments may be required before HIR is performed. Because the existing pavements of IA-93 and many other roads in Iowa were maintained with surface treatments and have cracking depth greater than 1 inch, HIR may have less application as a holding strategy treatment in comparison to other in-place recycling technologies.

2.3.2. Cold in-place recycling

CIR is an in-place recycling technology which pulverizes, adds recycling agents, mixes, spreads, and compacts 2 to 5 inches of the existing asphalt pavement by using a cold recycling train which consists of cold-milling machines, crushers, screeners, pugmills, and pavers to produce a recycled asphalt concrete layer. Virgin aggregates may be needed if an increase in pavement thickness or width is required. The process usually requires the retention of at least 1 inch of the existing pavement layer in order to support the load from the construction equipment that perform the recycling (FHWA 2011). It is also known as the partial-depth cold recycling.

The CIR construction process includes pulverization, sizing, mixing, and paving. This process can be performed by a single machine or a multiple-unit train. The single-unit machine usually performs CIR construction in a two-pass procedure. During the first pass, the machine pulverizes the existing pavement and reduces the size of the recycled asphalt pavement (RAP). During the second pass, the RAP is mixed with recycling agents and placed on road. The multiple-unit train consists of a pavement profiler, a crusher, a pugmill, and a paver. Each step in the CIR process is carried out by a single piece of equipment; and all steps are completed in one pass. Sometimes, a two-unit train is also used for CIR construction. The two-unit train consists of a pugmill mixer-paver which is capable of mixing and paving. A milling machine is required to process RAP to the desired particle size. The multiple-unit trains have higher production rate and consistency than the single-unit machines (Caltrans 2008). However, the multiple-unit trains have difficult in negotiating turns and corners which are more frequently encountered in urban areas in comparison to rural areas.

CIR can be used to correct various surface defects and pavement distresses. As part of a pavement rehabilitation project, CIR is applied as a base preparation treatment before an overlay is placed. A 1.5 to 4-inch overlay is typically constructed over the CIR layer. CIR has been successfully implemented in many states in the US and in other countries. Considerable cost savings about 45 to 75% were recognized by using CIR as an alternative of the conventional overlay method (FHWA 2011) (Jahren, et al. 1998). The life expectancy of CIR ranges from 7 to more than 20 years (FHWA 2011) (Jahren, et al. 1998).

The commonly used recycling agents for CIR include asphalt emulsions and foamed asphalt. Adequate curing time is required in order for the CIR layer to loose moisture and gain strength. A favorable working environment is critical to the success of construction. Many state agencies have specified weather restrictions for CIR constructions. Typically, an ambient temperature above 15 ℃ (59 ℉) and dry weather condition are desirable. During construction, the bearing strength is temporarily decreased. Weak spots may fail to support the construction equipment and cause failure in base and subgrade. Such failure can be repaired with an asphalt overlay or a replacement of the weak materials at the failure spots. Asphalt stripping was problematic for CIR sections in Kansas (FHWA 2011); and lime slurry was used to mitigate the stripping issue and improve the overall performance.

The structural layer coefficients of CIR are usually smaller than the layer coefficient of new HMA. The AASHTO road test results suggested that an appropriate layer coefficient for CIR would range from 0.3 to 0.35 (AASHTO 1986). Some state agencies use a layer coefficients ranging from 0.25 to 0.28 (FHWA 2011). There is no single nationally accepted mix design method that has been adopted for CIR mixtures. However, many organizations have developed CIR mix design methods based on Marshall, Hveem, or Superpave Gyratory methods (Epps and Allen 1990).

2.3.3. Full-depth reclamation

FDR which is also known as the full-depth cold recycling is a process which involves which pulverizes the entire asphalt pavement layer and a portion of the underlying aggregate base. Then the recycled materials are mixed and placed as a base layer. The treatment depth is typically 6 to 9 inches and seldom greater than 12 inches. Stabilization agents are sometimes used in FDR to create a stabilized full-depth reclamation (SFDR) layer. Commonly used stabilizers include bituminous stabilization agents, such as various asphalt emulsions and foamed asphalt, and chemical stabilization agents, such as fly ash, cement, lime, and calcium/magnesium chlorides. The selection of a stabilizer type is usually based on RAP material gradation, the plasticity index, fines content, and the extent to which the asphalt binder in RAP material has aged. Virgin aggregate can be added if there is a need for additional structural capacity or lane widening.

FDR can be constructed using some of the same equipment and process as CIR. The single-unit machine and the two-pass operation are more popular than the multiple-unit trains and the single-pass operation (Thompson, Garcia and Carpenter 2009). The primary reason is that the single-unit machine performs pulverization, sizing, mixing, and placing around the rotary drum without requiring the transport RAP materials to other pieces of construction equipment; this lessens the possibility of subgrade failure due to construction equipment loads.

FDR is effective in correcting various functional and structural distresses. The treatment is able to completely eliminate cracking patterns of any crack type (top-down or bottom-up); this mitigates reflective cracking. FDR can also improve the pavement structural capacity by increasing the base layer thickness. Compared to a reconstruction project for base layer with the same thickness, the use of FDR can result in savings of 90% with regard to new materials and 80% for diesel fuel (PCA 2005). A life-cycle-cost-analysis (LCCA) conducted by Diefenderfer et al. indicated that the pavement maintenance strategies involving SFDR are about 16% less costly in comparison to the conventional mill and fill strategies during a 50-year analysis period (Diefenderfer and Apeagyei 2011). FDR is usually used in combination with an AC overlay. The treatment provides a service life that is comparable to that of reconstruction project.

The factors, such as the need for curing time and the need for adequate subgrade support, should be considered in FDR construction as they are for CIR construction. The required minimum temperature for SFDR using chemical stabilizers is typically 4 ℃ (39.2 ℉) to 7 ℃

(44.6 ℉) (Morian and Scheetz 2012). The weather and temperature requirements for bituminous SFDR are the same as those for CIR. The Illinois DOT requires that the moisture content of SFDR layer is less than 2.5% or 50% of the optimum moisture content determined from the proctor test (Illinois DOT Bureau of Local Roads and Streets 2012). Many state agencies also establish rolling criteria to assure adequate compaction is achieved. IADOT requires the field density at 75% of the FDR mat depth to be higher than 92% of laboratory density for secondary roads; and the field density at the 2-inch depth to be higher than 97% of the density at the 75% mat depth (Iowa Department of Transportation Highway Division Specification Section 2012).

The FDR layers without introduction of stabilization agents are considered to have the same structural capacity as an aggregate base. SFDR has higher structural capacity than that of FDR material. The layer coefficient of SFDR ranges from 0.16 to 0.22 (Nantung, Ji and Shields 2011); and is dependent on type of stabilization agent that has been used. In Minnesota, a granular equivalence value of 1.5 is used in the design of SFDR thickness (Tang, Cao and Labuz 2012). The mix design for an SFDR mixture is often developed using the judgment of an experienced personal. Many mix design methods that are developed for cold recycled pavement materials can be also used for both CIR and SFDR (Epps and Allen 1990).

2.4. Nevada Rehabilitation Alternatives Research

CIR and FDR are effective pavement rehabilitation technologies that can be used as lower-cost alternatives of the conventional HMA overlay and base reconstruction. However, the treatments are usually overlaid with a 2 to 4-inch of HMA. Few references were found that investigated the performance of CIR or FDR that are covered by a TST. In these records about the recycling and surfacing treatments, the road sections in Nevada were found to be in the most similar traffic conditions, treatment methods, and pavement conditions to those for IA-93 test sections. The performance of these test sections was investigated by Maurer et al. (Maurer, Bemanian and Polish 2007).

The Nevada test sections were constructed on five low-volume roads using CIR, SFDR, cold mix asphalt, and various surface treatments. The roads are two-lane rural highways which carry an ADT less than 400. The existing pavements were suffering from fatigue cracking, transverse and non-wheelpath longitudinal cracking, and raveling. The test sections include four SFDR sections with a chip seal surface, four SFDR sections with a 1.5-inch overlay and chip seal surface, and 9 CIR sections with a chip seal surface. The SFDR sections with 1.5-inch overlay and chip seal surface used two proprietary products as the recycling agents. These sections were originally designed to be covered with a chip seal surface. However, construction failures and early-age performance issues were encountered; and a 1.5-inch overlay was applied as a corrective measure. The other SFDR sections were stabilized with cement or an asphalt emulsion. The CIR sections were constructed using asphalt emulsion as stabilization agents. Some of the CIR sections used a proprietary polymer-modified asphalt emulsion; while the other CIR sections used a CMS-2S asphalt emulsion.

The performance of the test sections was evaluated by considering roughness measurements, condition surveys, and falling weight deflectometer (FWD) tests. The sections were monitored for 3 to 4 years. The results show 17% to 62% performance improvements by the SFDR treatments and 2% to 43% by the CIR treatments. The average improvements for roughness were 14% for the SFDR sections and 20 to 30% for the CIR sections. The FWD results also indicated 36 to 72% structural improvements on the SFDR sections. A LCCA was also performed for the CIR treatments with a 20-year analysis period and a 4% discount rate. The cost analysis results indicated that an average cost saving of $100,000 per centerline mile was realized by using CIR and chip seal for rehabilitation in a 20-year life-cycle.

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CHAPTER 3. DEVELOPMENT OF HOLDING STRATEGIES FOR DETERIORATED LOW VOLUME ROADS – An Introduction to Test Sections in Iowa

Modified from a paper submitted in the *Journal of the Transportation Research Board*

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3.1. Abstract

Heavily deteriorated low volume roads in the United States are often facing the plight of insufficient maintenance funding. Funding priority is usually given to those roads that are part of the National-Highway-System (NHS). Rural highways that have the geometry and speed limits (typically less than 65 mph and more than 40 mph) intended for lower traffic volume are not included in the NHS and may be managed by state highway authorities, counties, or townships. These low-volume roads (LVR) provide accesses to the areas that are less populated and typically have traffic counts of less than 1500 average daily traffic (ADT). Highway agencies are interested in holding strategies that are more aggressive than preventative maintenance but not as extensive as rehabilitation, to improve such roadways. The Iowa Department of Transportation (IDOT) constructed test sections using ten holding strategy treatments to aid in the development of appropriate guidelines. Holding strategy treatments use various combinations of thin surfacing technologies and base recycling and strengthening treatments for flexible pavement rehabilitation and preservation, including single and multiple layer chip seals, thin and ultra-thin asphalt overlays, cold in-place recycling, and full-depth reclamation. This paper summarizes the

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construction of the test sections and recent observations of the performance and maintenance of the treatments. Based on the construction costs and treatment life expectations from the literature, a draft decision table is proposed for the selection of holding strategy treatments. The decision table recommends that the most cost-effective holding strategy treatment among the ten treatments that were constructed for a specified holding period. The decision table will be validated in the future with further performance observations.

Key words: holding strategy, low-volume road, pavement maintenance.

3.2. Introduction

Maintaining the pavement condition of highways at an acceptable condition is becoming a pressing challenge for many highway agencies. The maintenance priority is usually given to roadways in the national highway system (NHS) in order to arrive at the performance targets established for the *Moving Ahead for Progress in the 21st Century Act* (MAP-21) (FHWA 2014). The non-NHS roads, which are mostly lower volume paved roads, often lack adequate funding for rehabilitation due to the low funding priority. A low-volume road (LVR) discussed within the scope of this paper is two-lane local roads and rural collectors with less than 1500 average daily traffic (ADT) and speed limit between 40 mph and 65 mph. The LVRs provide vital accesses to the areas with low population density. Due to the budget restraints, only a few selected non-NHS roads are typically rehabilitated while only reactive maintenances, such as crack sealing or patching, are performed on the others. The roads that are selected to be maintained generally have been the target of relatively more complaints because they have higher traffic volumes and poorer conditions. The LVRs for which rehabilitation has not been funded are often allowed to deteriorate until budgets are allocated to those roads. Some of the roads have been neglected for a considerable time; and have considerable pavement distresses such as severe cracking, rutting, potholes, and raveling decrease travel comfort and increase vehicle required maintenance and possibility of accidents. In some states, some roads eventually degraded to serviceability levels of unpaved surfaces. It was reported that about half of counties in Michigan had converted paved LVRs to gravel and more than 100 miles of asphalt roads were converted to unpaved roads in South Dakota in 2009 (Etter 2010).Such conversions being contemplated elsewhere in the United States (Batheja 2013).In an attempt to avoid such a disappointing situation, it would be desirable to develop holding strategies that can be integrated into pavement management systems (PMS).

This paper introduces an ongoing holding strategy research project sponsored by the Iowa DOT; it summarizes the construction of test sections and provides a preliminary decision table for the engineers to make initial selections of possible holding strategy treatments. The authors' intentions are to raise engineer and researcher interest and awareness about the possibility of using holding strategies. As of this writing further research involving material characterization, cost analysis, and performance observations is continuing.

3.3. Background

A holding strategy is a pavement management concept which implies postponement of major rehabilitation or reconstruction of a deteriorated road section with applications of treatments that are more aggressive than preventative maintenance treatments, with lower cost and most likely shorter service lives when compared to rehabilitation strategies. A holding strategy is sometimes considered as a special pavement preservation method which provides solutions in certain circumstances when the budget is limited and the long-term cost-effectiveness is not the primary decision factor (Luhr, et al. 2010) (Metropolitan Government of Nashville and Davidson County 1999). However, it is beneficial for decision makers to understand the differences between a holding strategy and a typical preventive maintenance treatment. The primary benefits of a preventive maintenance strategy are improvement of functionality for pavements in relatively good condition over a long service life that provides a favorable rate of return on investments to the road system. A "good" condition usually refers to a pavement condition index (PCI) of 70 and above according to the ASTM PCI rating scale (ASTM 2007). However, a holding strategy addresses pavements that are in poor condition due to budget shortages by providing moderate improvements for moderate costs. The expectation is that if all roads can be maintained in at least moderately good condition through the judicious use of holding strategies, budgets can be more easily maintained for preventive maintenance activities, rather than being diverted to the rehabilitation actives necessary to execute a “worst first” strategy.

Selection of a holding strategy should include five components: project recognition, treatment selection, design and construction, maintenance and late-life reactive maintenance. The project selection component (network level analysis) involves using the current PMS to recognize the road sections that are eligible for holding strategy application. The treatment selection component (project level analysis) compares various alternatives and selects the right treatment based on the cost-effectiveness, availability of technology and material, intended life extension, climate, and traffic conditions. The design and construction component includes proper design, construction, and quality control/assurance procedures to assure the quality of the treatment. The maintenance component should include preventive and routine maintenance appropriate for the situation at hand. Late-life reactive maintenance component may be necessary if funding is not available for rehabilitation or the application of another holding strategy at the end of the service life.

Sometimes, the terminology “holding strategy” is used to describe reactive maintenance activities, such as crack filling and patching, that are applied to extend the service life of a road that is due for rehabilitation. Such maintenance activities typically do not include a comprehensive base treatment; therefore, they are not in the scope of the holding strategies described in this paper. The Ministry of Transportation of Ontario (MTO) has implemented a holding strategy as one of the three types of alternative maintenance and rehabilitation strategies in its PMS since 1985 (Kazmierowski 2001). The three maintenance and rehabilitation strategy types are the preferred strategy, the holding strategy, and the deferred strategy. The preferred strategy is the optimal solution of a life-cycle cost analysis (LCCA). If the preferred strategy cannot be carried out due to budget limits, the holding strategy can be chosen to delay the needs for the preferred strategy. The maximum expected postponement of the preferred strategy by the MTO holding strategy is 3 years. If the need for the delay is more than 3 years, the MTO would use the deferred strategy which envisions the application of no treatments during the postponement period and employs an expensive treatment thereafter. The holding strategies envisioned in the present paper differ from the ones developed by the MTO, because the holding strategies envisioned in the present paper are expected to last 7 to 12 years.

3.4. Holding Strategy Treatments and Expected Life

The philosophy of a holding strategy requires that the treatment be inexpensive, protective to the existing pavement system, and provide at least moderate improvement to road performance. For aged LVRs in the Midwest states, the pavements can suffer from non-load-related surface defects such as thermal cracking or raveling. These surface distresses may not dramatically decrease the load-bearing capacity of the pavement; but they could significantly lower vehicle speeds (Dell'Acqua and Russo 2011) and travel comfort. Consequentially, the willingness of drivers to travel the route may be decreased causing a decrease in traffic volume. Although the materials of the pavement surface layer may be severely damaged, the base and subgrade materials are sometimes still in an acceptable condition which is adequate to support the current and projected traffic. A few in-place recycling technologies and various thin surfacing treatments can be used to correct the surface distresses without reconstruction of the base and subgrade or placement a thick overlay which result in higher construction costs. The existing pavement can be treated with scarification, hot/cold in-place recycling (HIR or CIR), or full depth reclamation (FDR) in order to eliminate the cracks that may reflect through the surface layer and other defects which would influence the performance of the surface treatments. Sometimes, the construction of an overlay, interlayer, or leveling course can also serve as part of a holding strategy solution. A thin surfacing treatment, such as chip seal or microsurfacing, can be employed in conjunction with the previously mentioned treatments to improve skid resistance and ride quality and create a waterproof or water resistant layer to protect the underlying materials from oxidization and water damage. Crack sealing and patching sometimes can effectively preserve the pavement condition for a short period. Such treatments can be also considered a holding strategy treatment for a short time life extension. There are also holding strategy treatments for Portland cement concrete (PCC) pavements, such as diamond grinding, dowel bar retrofit, etc. Such PCC treatments are beyond the scope of this paper as the focus is on flexible pavement holding strategies.

Life extension of pavements using thin maintenance surfaces and base treatments has been studied extensively. However, the research usually focuses on the life extension for a single treatment type that is applied to a road in relatively good condition. Combinations of various thin surfaces and base treatments applied to roads in relatively poor condition have not yet been well investigated. Table 3.1 and Table 3.2 summarize the expected life extensions and unit costs of various holding strategy treatments (treatments are defined in Appendix). Because the data were collected from projects which have various pavement structures, deterioration conditions, project locations, and traffic conditions, large variations were found in the expected life extensions and the unit costs. Readers should be aware that the treatments were used for preventive maintenance in most of the projects. If the same treatment was used for a holding strategy, the expected life extension should approach the lower end of the expectation range since the pavement would be noticeably more deteriorated.

Table 3.1. Expected Life Extension in Years of Various Treatments

Treatment	**Geoffroy (1996)** (Geoffroy 1996)	**Hicks et al. (2000)** (Hicks, Seeds and Peshkin 2000)	**Maher et al. (2005)** (Maher, et al. 2005)	**Huang (2009)** (Huang and Dong 2009)	**Wu et al. (2010)** (Wu, et al. 2010)	**Michigan DOT (2011)** (Galehouse 2003)
Crack Sealing		2 to 5		up to 3	0 to 4	up to 3
Thin Asphalt Overlay		2 to 12		9 to 12	3 to 23	5 to 10
Chip Seal	4 to 7	3 to 7	3 to 5	3 to 5	3 to 8	3 to 6
Double Chip Seal			4 to 8			4 to 7
Microsurfacing	4 to 7	3 to 9	5 to 8	7 to 9	3 to 8	3 to 5
Slurry Seal	1 to 6	3 to 7	3 to 8	3 to 8	4 to 7	
Fog Seal		2 to 4	1 to 3		4 to 5	
Otta Seal			4 to 8	4 to 8		
Double Otta Seal			8 to 15			
Cold In-place Recycling			6 to 20		4 to 17	
Hot In-place Recycling			6 to 15		3 to 8	
Full Depth Reclamation			7 to 20		10 to 20	

Table 3.2. Cost per Square Meter of Various Treatments

Treatment	**Hicks et al. (2000)** (Hicks, Seeds and Peshkin 2000)	**Maher et al. (2005)** (Maher, et al. 2005)	**Huang (2009)** (Huang and Dong 2009)
Crack Sealing			$1 to $5
Thin Asphalt Overlay	$2.09	$1 to $1.5	$2.1 to $2.4
Chip Seal	$0.85	$1.5 to $3	$0.84 to $1.14
Microsurfacing	$1.25	$3.1 to $3.9	
Slurry Seal	$1.08	$0.9 to $1.8	$0.9 to $1.8
Fog Seal	$0.54	$0.25 to $0.6	
Otta Seal		$2 to $2.7	$2 to $2.7
Cold In-place Recycling		$4.2 to $4.8	
Hot In-place Recycling		$1.5 to $3.9	
Full Depth Reclamation		$5 to $8	

3.5. Research Objectives

The Iowa DOT sponsored a research project in 2013 with the objective developing guidelines for applying holding strategies. The methodology included the construction of test sections on a representative road segment to minimize the variations in the pavement and subgrade conditions, traffic volume and type, climate, construction process and materials. The selected road is a noticeably deteriorated low-volume two-lane rural highway which has a flexible pavement system. This research documents the construction process and will investigate possible life extensions and the construction and maintenance costs of various holding strategy treatments.

3.6. Construction of Test Sections

3.6.1. Project Location and Existing Pavement Condition

The test sections were constructed on the Highway IA 93 between Sumner and Fayette in Iowa. The road has an average annual daily traffic (AADT) of 1040 with 3 percent trucks. Historical traffic count data has indicated that the ADT of the road has not noticeably changed since 2001 (Iowa DOT 2014). The international roughness index (IRI) measured in 2012 was 246 inches per mile. A preconstruction pavement distress survey documented extensive transverse and fatigue cracks, raveling, and severe edge breaks on the existing pavement. Top-down cracking is the predominant distress type. A statistical summary of the observed transverse and longitudinal cracks from 23 randomly assigned survey sections is shown in Table 3.3. The experimental road is arbitrarily divided into three segments: the eastern segment, the middle segment, and the western segment (Figure 3.1). The eastern and western segments are in the vicinity of the municipal areas of Sumner and Fayette, respectively. In comparison to the middle segment, they are expected to have higher traffic volume and lower operating speed due to their proximities to the residential areas and rolling terrain and horizontal curves in the western segment. The eastern and western segments are 2 miles each; and the middle segment is 9.2 miles. Pavement distresses within each segment are considered evenly distributed along the distance; and this assumption was confirmed with visual inspection of the pavement condition before the construction of the treatments. A Student's t-test is conducted to compare the density of cracking between various segments; and the results are shown in Table 3.3 and Table 3.4. The results indicate that the selected experimental road does not have a segment which has deteriorated more significantly than the other parts of the road. A few potholes were also observed. A present serviceability index (PSI) of 1.9 was estimated from the pavement distress survey results. The roadway was built in 1951. The original pavement consisted of a 6-inch rolled stone base and a 0.75-inch chip seal surface. The Iowa DOT pavement management information system (PMIS) shows that a 4.5-inch thick hot mix asphalt (HMA) overlay was placed on the original pavement in 1971. However, field observations during construction of the FDR sections showed that the actual pavement thickness is between 7 and 8 inches. The Iowa DOT PMIS might miss documenting an overlay. The pavement was resurfaced with two seal coats (chip seals) in 1990 and 2006, respectively.

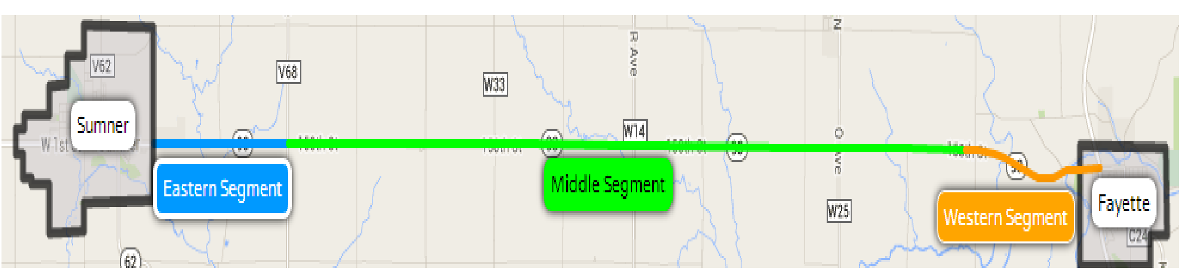

Figure 3.1. IA93 Distress Survey Segments

Table 3.3. Cracking Type and Density of the Existing Pavement

Cracking Type	Crack Density, ft/1000 ft^2			Crack Severity
	Min.	Max.	Mean	
Transverse	90	184	142	23% Low, 46% Medium, 31% High
Longitudinal	7	32	71	23% Low, 41% Medium, 37% High

Table 3.4. Distribution Along the Length of the Existing Pavement

Segment	Number of Survey Sections in the Segment	Mean Cracking Density, ft/1000 ft^2	Cracking Density Coefficient of Variation	Statistical Level*
Eastern	6	31	18.4%	A
Middle	14	38	22.2%	A
Western	4	39	21.1%	A

***:** Levels not represented by same letter are significantly different.

3.6.2. Test Sections and Treatment Methods

The research methodology included constructing and observing as many treatments as the budget and the road length would reasonably allow. Ten test sections were constructed between June and September in 2013. A summary of the base treatment and surface treatments are presented in Table 3.5. Test Sections 1 through 9 are located in the rural part of the IA 93 which has one 12-foot traffic lane in each direction. Test Section 10 is a small town urban segment which has a 12-foot traffic lane and a 6-foot parking lane in each direction. The construction of the urban segment was requested by the municipality of Fayette.

Table 3.5. Treatment Sections and Methods

Section Number	Base Treatment	Surface Treatment	Section Length, Mile
MC1	1" scarification	1.5" HMA overlay	1.3
MC2	1" scarification	1.5" HMA overlay and single chip seal	2.0
MC3	1" scarification and 1" interlayer course	0.75" ultra-thin HMA overlay	2.2
MC4	8" full depth reclamation	1.5" HMA overlay	1.0
MC5	8" full depth reclamation	double chip seal	0.4
MC6	2.5" cold-in-place recycling	double chip seal	1.4
MC7	2.5" cold-in-place recycling	1.5" HMA overlay	1.6
MC8	none	2" HMA overlay	1.4
MC9	1" leveling and strengthening course	single chip seal	1.9
MC10	1" scarification	single chip seal	0.3

3.6.3. Mix Design and Materials

3.6.3.1. HMA overlay

A typical overlay design procedure was followed to design the aggregate gradations and binder percentages of the HMA surface and interlayer courses. The same mix design and materials were used for the 1.5 and 2-inch HMA layers. The designs for the ultra-thin surface, the interlayer course, and the leveling and strengthening course are different to account for the thin thickness of the lifts. The aggregate gradation and the binder performance grade and content of each type of mix are summarized in Table 3.6 and Table 3.7.

Table 3.6. HMA Mix Design: Aggregate Gradation

Aggregate Gradation				
	Percent Passing, %			
Sieve Size	1.5" and 2" Surface Course Mix	0.75" Ultra-thin Surface Course Mix	1" Interlayer Course Mix	1" Leveling and Strengthening Course Mix
3/4"	100	100	100	100
1/2"	99	100	100	100
3/8"	90	100	100	100
#4	64	76	84	78
#8	48	50	63	53
#16	38	34	47	39
#30	28	28	35	30
#50	12	18	21	16
#100	6.1	9.8	11	7.7
#200	3.9	4.9	6.5	3.6

Table 3.7. HMA Mix Design: Binder Content and Grade

Mix Type	Binder Content, %	Binder Grade
1.5" and 2" Surface Course Mix	5.3	PG58-28
0.75" Ultra-thin Surface Course Mix	6.7	PG76-34
1" Interlayer Course Mix	7.4	PG64-34
1" Leveling and Strengthening Course Mix	6.3	PG58-28

3.6.3.2. CIR

The cold-in-place recycled material was processed by pulverizing the existing asphalt pavement mixed with a foamed PG52-34 asphalt binder at an average application rate of 0.73 tons per station which results in a binder content of 2.4%.

3.6.3.3. FDR

The full depth reclamation material was processed by pulverizing the existing asphalt pavement mixed with a foamed PG52-34 binder and Class C fly ash. The average application rates for the asphalt binder and fly ash were 2.8% and 2% by dry mass of mixture, respectively. The actual virgin asphalt content ranged from 2.7% to 2.9%.

3.6.3.4. Chip seal

The cover aggregate for the chip seal was 0.5-inch limestone. The average aggregate application rate was 29 pounds per square yard for the rural sections and 48 pounds per square yard for the urban section. A CRS-2P asphalt emulsion was applied at rates of 0.38 and 0.6 gallon per square yard for the rural and urban sections, respectively. The double chip seal sections were constructed with two applications of the asphalt emulsion and cover aggregate, the first course using ½ in aggregate and the second course using 3/8 in aggregate. The quality was primarily controlled by controlling the material application rates.

3.7. Field Observations

A post-construction survey was conducted a week after the completion of the construction of all test sections. The survey found that the pavement conditions were significantly improved through the construction of the holding strategy treatments. The distresses that were observed in the pre-construction survey were successfully removed. Figure 3.2 illustrates a comparison between the surface conditions before and after the treatment at the same location.

One of the research interests is the winter performance of the test sections. An interview with the winter maintenance staff that maintains the test sections indicated that the asphalt surface and chip seal surface have distinctive behaviors in severe winter weather. The chip seal with the rough surface provides better traction/skid resistance for traffic during snow events in comparison to the other smoother asphalt surfaces. However, the plowing operations shears off high spots on the chip seal surface and pulls out loose aggregate as illustrated in Figure 3.3. This process increases the attrition rate of the plowing. The plowing operations also improve the surface traction for vehicles in most cases. The aggregate after plowing has fractured surfaces which increase the contact area between vehicle tires and the road surface. The area of snow and ice pack is also significantly decreased. However, in some sections where the loss of aggregate is very noticeable, the plowing may cause loss of road surface friction. It was also noticed that that the temperature change of the chip seal surface is considerably slower than that of the conventional asphalt pavement surface. On the days in which the temperature fluctuates above and below the freezing point, the melted water on the asphalt surface refreezes when temperature drops and forms a slick ice layer. For the chip seal surface, it takes longer for the surface temperature to rise to the melting point and. The formation of ice is also slower on a chip seal surface. This allows enough time for the DOT operator to react appropriately. However, if the temperature is above 32 °F and steady, the asphalt surface melts the snow more quickly and dries out the water faster than the chip seal surface. Because the accumulated snow in the voids between the aggregate pieces on the chip seal is difficult to remove for the snowplows, the chip seal tends to have more melted water when the temperature is high. The surface irregularity of the chip seal reduces the wind speed at the surface which further decreases the evaporation rate of water. The situation is hazardous if additional precipitation falls on the wet pavement section. A wet surface accumulated snow much faster than a dry surface which exposes traffic to a higher risk. Abrasives might be useful in such a case; meanwhile an asphalt surface can be treated plowing, possibly attempting to scrape ice with an underbody plow. In addition, the pavement markings on the chip seal surface can be removed by snowplows and traffic noticeably faster in comparison to an asphalt surface. Faded pavement markings can be associated with increased risk of traffic accidents. Figure 3.3 shows a comparison between the pavement markings on a chip seal and an asphalt surface. The aforementioned differences would affect the annual maintenance costs of the two surface types and may be considered as part of an LCCA assessment.

Figure 3.2. Comparison Between Surface Conditions Before and After Treatment

Figure 3.3. Snow Plowing Effects on Chip Seal and Asphalt Pavement Surface: (a) before plowing, (b) after plowing, (c) chip seal surface, and (d) asphalt pavement surface

3.8. Construction Cost and Treatment Selection Recommendation

The total construction cost of the 10 test sections was $1,692,157 over the approximately 15 miles, which yields an average unit cost of $10.8 per square meter. The average unit cost is high compared to the typical costs from the literature search results in Table 3.2. Considering that each section used two or three maintenance treatments and the relative short section length which results in increased mobilization costs, it seems likely that the costs could be reduced to a potentially acceptable level for a holding strategy if one combination of treatments was selected for a longer length of road. The current practices in Iowa are typically to use 3 to 4.5 inches of an asphalt overlay as the major pavement rehabilitation method. Thus, an estimated unit cost for a 3-inch HMA overlay is used for comparison purposes for each of the holding strategy costs (Figure 3.4). The comparisons indicate that the unit cost for the full depth reclamation with 1.5-inch HMA overlay (MC4) exceeds the unit cost of a 3-inch HMA overlay. Such a treatment method is considered a major rehabilitation method rather than a holding strategy treatment due to the high cost. The costs of the other treatments fulfill the essential purpose of a holding strategy. Note that the results of the cost comparison could change substantially due to local economic conditions and the length of the section to be treated. With the construction cost information and the treatment life expectations from the literature, a preliminary decision table could be developed to help with the initial planning for holding strategy projects (Table 3.8). Users could select treatment methods based on cost and expected life extension of the alternatives. It is intended that the holding life for each treatment method could be estimated after analyzing the results of future observations from the test sections described in this paper.

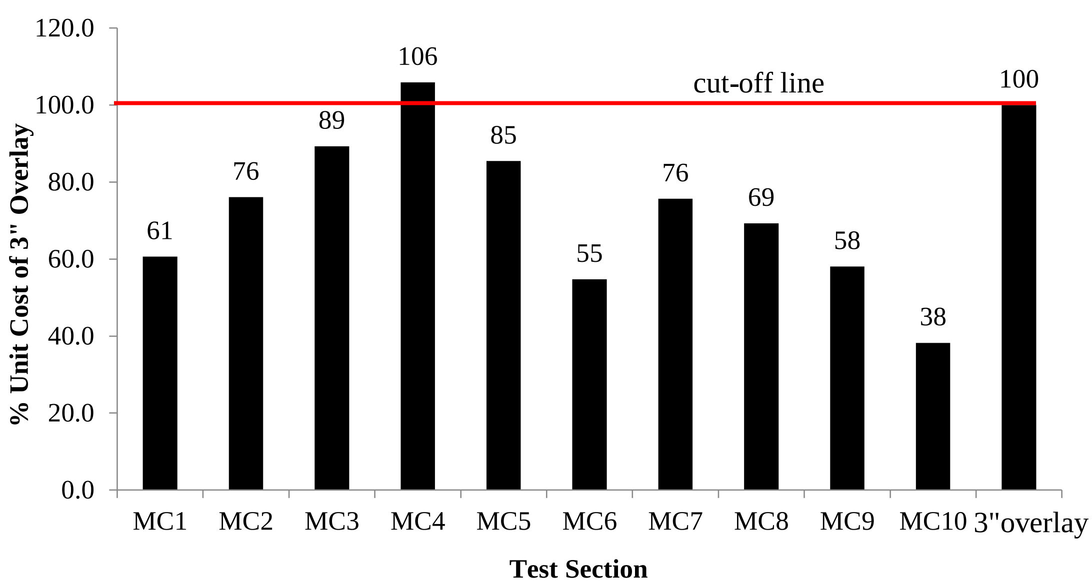

Figure 3.4. Unit Cost Comparison with 3-inch Overlay

Table 3.8. Preliminary Selection Table

Years for Holding	Proposed Treatment
1	scarification + chip seal
2 – 4	CIR + double chip seal
5 – 7	CIR + thin overlay
8 – 10	scarification + interlayer + ultra-thin overlay

3.9. Conclusion and Future Research Plan

The Iowa DOT holding strategy project involved the construction of ten test sections on a low volume road which was approaching the end of its service life. Various base and surface treatments were used to construct the test sections in order to evaluate the feasibility of using the treatment methods to fulfill of the goals of a holding strategy. Among the ten treatment methods, one appears to be too expensive to use as a holding strategy treatment in this case study; and the other nine methods were found to be good candidates. The concept of holding strategies is practical in terms of cost effectiveness and road condition improvements. This paper proposed a preliminary decision table based on the expected life extensions and the construction costs for selecting the proper holding strategy treatments.

Some implementation issues which would benefit from further research were found during this study. Winter maintenance experience indicates that the roads with a chip seal surface perform differently from those with an asphalt surface during snow precipitation events in terms of the snow removal method, snow holding behavior, and resulting safety outcomes. Additional information is needed for evaluating differences in safety and the winter maintenance costs.

Future research on the test sections will include regular periodic monitoring of pavement performance, an accounting of maintenance costs, and conducting laboratory evaluation of the material properties. Since the condition of the underlying support layers is critical in terms of estimating the amount of possible life extension of holding strategies, falling weight deflectometer analyses and consideration of base preparation activities (i.e., scarification, CIR, and FDR treatments) will also be investigated. The tracking of maintenance costs will be used to complete the cost-effectiveness analysis. A more reliable decision tool based on actual field performance of the holding strategy treatments will be developed.

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3.11. Appendix

Crack sealing: fill the cracks of a pavement using liquid asphalt crack fillers in order to prevent further propagation of the crack and water damage to the underneath pavement structures.

Thin asphalt overlay: a HMA overlay with less than 1.5 inches thickness.

Chip seal: a surface sealing consisting a rapid setting asphalt emulsion and a single-sized aggregate cover.

Slurry seal: a mixture of emulsified asphalt and crushed rock that can be spread over an existing road surface as a surface sealing technology. A slurry seal consists of a graded aggregate, an asphalt emulsion, mineral fillers, and additives.

Microsurfacing: A special type of slurry seal which uses polymer-modified asphalt and high quality aggregate.

Otta seal: an asphalt surface treatment constructed by placing a graded local aggregate on an application of a relatively soft asphalt binder.

Fog seal: "a light spray application of dilute asphalt emulsion used primarily to seal an existing asphalt surface to reduce raveling and enrich dry and weathered surfaces" (Caltrans 2008).

Cold in-place recycling: an asphalt pavement rehabilitation technique which pulverizes 2 to 5 inches of the existing pavement and mixes the recycled pavement materials with emulsified or foamed asphalt at the construction site. The mixture is placed as a rejuvenated asphalt surface.

Hot in-place recycling: an on-site asphalt pavement rehabilitation method which softens the existing pavement with heat and scarifies 3/4 to 2 inches of the softened pavement. The scarified materials are reused with virgin asphalt and/or aggregate to pave a new road surface.

Full-depth reclamation: "a recycling method where all of the asphalt pavement section and a predetermined amount of underlying materials are treated to produce a stabilized base course" (Asphalt Recycling & Reclaiming Association 1992).

CHAPTER 4. DEVELOPMENT OF SELECTION STRATEGY OF FLEXIBLE PAVEMENT HOLDING STRATEGIES FOR LOW-VOLUME ROADS

Modified from a paper submitted to the *International Journal of Pavement Engineering*

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4.1. Abstract

Local highway agencies in Iowa are challenged by the imbalance between available financial resources for non-NHS (national highway system) road repair and the growing number of roads due for major rehabilitation or reconstruction. Holding strategy provides cost-effective alternatives to traditional rehabilitation or reconstruction treatments and allows highway agencies to allocate the maintenance budget with some degree of flexibility. In order to develop treatment methods that can be used for holding strategies, Iowa DOT constructed test sections in 2013 on IA-93. Various combinations of recycling technologies, thin and ultrathin asphalt overlays, and surface treatment (chip seal) were used to develop treatments that can be possibly implemented for holding strategies.

A series of pavement condition surveys were performed to evaluate the performance of each treatment method. This paper summarizes the results of the pavement condition surveys. These treatments successfully corrected surface defects on existing pavement and considerably improved ride quality. Various levels of reflective racking were observed for some sections. Cold in-place recycling (CIR) and full-depth reclamation (FDR) were found to be particularly

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effective for crack mitigation. Loss of cover aggregate was found for the section which received a FDR and double chip seal treatment. An Life Cycle Cost Analysis (LCCA) is conducted to compare the cost-effectiveness of the holding strategies and a conventional rehabilitation treatment. Based on the LCCA results, a decision table is proposed as a reference to assist in the selection of appropriate holding strategy.

Key words: holding strategy, low-volume road, life-cycle-cost-analysis, decision table, pavement maintenance.

4.2. Introduction

Local highway agencies in Iowa are challenged by the imbalance between available financial resources for non-NHS (national highway system) road repair and the growing number of roads that are due for major rehabilitation or reconstruction. According to the *Iowa Infrastructure 2015 Report Card* (Mulholland and Crawford 2015), about 9% of the non-NHS roads are in poor condition with regard to pavement performance. The total budget shortfall for Iowa's road network has been estimated at $1.6 billion; and an additional $215 million is required for the most critical needs. Over the past decade, about 59% of the funding for highway condition and safety features improvements was received from the Federal Highway Trust Fund under the MAP-21 bill. Limited resources from such federal sources were used for the non-NHS roads, because MAP-21 only established performance targets for interstates and major highways in NHS. Therefore, local agencies are only able to renew a portion of the roads that require attention. The average pavement condition will decrease which increases the costs of social and economic activities and will negatively impact local economies, if this situation continues.

Non-NHS roads often carry lower traffic volumes in comparison to NHS highways. The pavements of non-NHS roads in the Midwest states usually fail because of surface defects and non-load related distresses. Non-load related distresses can be corrected by using treatments that are able to mitigate distress patterns and provide a waterproof surface without noticeably increasing the structural capacity of the pavement. A thick asphalt overlay, usually over 3 inches, as part of a traditional mill and fill strategy is often less cost-effective for treating the non-load related distresses. This provides an opportunity in which cost-savings and average pavement condition improvements for the states' highway network can be achieved by applying holding strategy treatments that are more aggressive than preventive maintenance; which, have relatively lower costs in comparison to a reconstruction alternative or a conventional rehabilitation method.

There is a variety of technologies and treatment combinations that may serve as holding strategy treatments. These treatments vary in their costs and the amount that they can extend service lives. The Iowa Department of Transportation (IADOT) constructed nine test sections using various holding strategy treatments, including combinations of thin and ultrathin asphalt overlays, in-place recycling technologies, and thin surface treatments, in 2013. The performance of the test sections were evaluated every one half year. In this paper, the performance observations are summarized; and the costs and benefits of these holding strategy treatments are evaluated. A decision tool is proposed for selecting appropriate holding strategies for low-volume highways with a full-depth asphalt structure.

4.3. Background

The concept of a holding strategy was previously introduced (Yu, Jahren and Williams 2015); it is defined as a pavement management concept which implies postponement of major rehabilitation or reconstruction of a deteriorated road section with applications of treatments that are more aggressive than preventive maintenance treatments, with lower cost and most likely shorter service lives when compared to rehabilitation or reconstruction treatments. Holding strategies can be used for roads that require rehabilitation or reconstruction activities; in situations where the agency does not have adequate financial resources to fund the required effort. Holding strategy treatments are intended to maintain the functionalities of pavements at an acceptable level until sufficient funding is available.

Non-NHS highways which carry low-volume traffic are the primary candidates for holding strategies. These roads can often have a sound structure even though their surface conditions are disappointing. Because structural capacity improvement is not the primary goal of holding strategies, combinations of various thin asphalt layers, surface treatments, and recycling technologies can be used to develop holding strategy treatments.

A thin asphalt overlay refers to any asphalt concrete (AC) layer with a lift thickness smaller than 1.5 inches. An AC layer less than 1 inch thick is usually called ultrathin asphalt overlay (Caltrans 2008). Thin asphalt overlays are usually used for pavement preservation. The treatment is effective for improving pavement functionalities and correcting surficial deficiencies such as raveling, non-load related cracking, and rutting or shoving that is only limited to the surface layer. Newcomb reviewed various studies on the performance of thin asphalt overlays in the United States, Canada, and Europe from 1994 to 2009 (Newcomb 2009). It was found that the life expectancy for thin asphalt overlays used as preventive maintenance treatments was 5 to 16 years. A commonly used type of ultrathin overlay is an ultrathin bonded wearing course, which also known as the open-graded friction course (OGFC). An OGFC uses high quality gap-graded aggregates and polymer-modified asphalt binder to produce such mixes. The OGFC layer usually has a lift thickness between 0.6 to 0.8 inches; and is placed on a thick polymer-modified asphalt tack coat (Gilbert, Olivier and Gale 2004). The treatment was found effective in improving the skid resistance and roughness of the pavement surface. If thin and ultrathin overlays are to be used for pavement preservation, the existing pavement distresses are must be minor. It is recommended that thin asphalt overlays be used for pavements with distress involve less than 10 percent of the projects (Newcomb 2009); and OGFC are applied for pavements with 6 to 8 years remaining service lives (Gilbert, Olivier and Gale 2004). Such requirements are usually not met if the treatments are used as holding strategies for pavements with severely deteriorated surfaces. The authors did not find any documented projects that involve applying a thin or ultrathin asphalt overlay on a road that had a pavement condition that was not suitable for preventive maintenance.

Thin surface treatment (TST) refers to application of a bituminous surface layer less than one half inch thick (Li, et al. 2007). These treatments provide waterproof surface layers which protect the underlying structures from water damage and oxidization. A list of commonly used TSTs and treatment descriptions is shown in Table 4.1. Liu et al. conducted a study on various TSTs used for pavement preservation in Kansas (Liu, Hossain and Miller 2010); and found that the service lives of TSTs on high-volume roads are significantly shorter compared to TSTs on lower-volume roads. The average service life of chip seal on non-Interstate highways was 5 years. Slurry seal on Interstate highways exhibited higher service life than chip seal; while the service lives of slurry seal and chip seal on non-Interstate highways were comparable. Previous investigations regarding TSTs have been primarily focused on TSTs that are used as preventive maintenance treatments. In order for the treatments to be effective and achieve the maximum cost-benefits, the candidate roads must be in good condition. Few case studies were found for instances where TSTs were used as pavement rehabilitation treatments.

Table 4.1. Commonly Used Thin Surface Treatments

Treatment	Description
Chip Seal	Constructed by applying an asphalt emulsion on road surface and covering with single-sized aggregate.
Slurry Seal	A thin film of mixture of asphalt emulsion, fine graded aggregate, mineral filler, water, and additives. Constructed use special purpose mixer-paver equipment.
Microsurfacing	A special type of slurry seal which uses polymer-modified asphalt emulsion and high quality aggregate.
Sand Seal	Constructed by applying emulsion film which is covered with fine aggregate. Usually used as a temporary treatment to restore surface texture and repair raveling (WSDOT 2003).
Otta Seal	A thick application of relatively soft asphalt emulsion covered with graded aggregate from local sources. The gradation of aggregate is usually coarser than that of the aggregate for sand seal.
Cape Seal	Constructed by applying a slurry seal on a chip seal surface.

Cold in-place recycling (CIR) and full-depth reclamation (FDR) are popular recycling technologies used for pavement rehabilitation. CIR and FDR recycle old pavement materials to produce new materials in place. This process requires less cost and resources associated with new material production as well as costs for hauling, handling, and storage of recycled asphalt pavement (RAP) in comparison to a traditional mill-and-fill rehabilitation method or reconstruction.

CIR is constructed by pulverizing 2 to 5 inches of the existing bituminous material. The RAP materials are ground to smaller size particles and mixed with recycling agents. Commonly used recycling agents include asphalt emulsion and foamed asphalt. Virgin aggregates may be needed if an increase in pavement thickness is required or in a widening project. CIR is usually used in a rehabilitation project as a base preparation treatment for asphalt overlay. The treatment eliminates the distresses on the existing pavement and reduces reflective cracking. CIR has been successfully implemented in many states in the US and in other countries. The life expectancy of CIR ranges from 7 to more than 20 years (FHWA 2011) (Jahren, et al. 1998).

FDR corrects pavement distresses with a more aggressive method than CIR. The entire layer of the bituminous material and a portion of the underlying base materials are pulverized to produce a new base layer. The application thickness is usually 6 to 12 inches. Foamed asphalt, asphalt emulsion, cement, fly ash, and other stabilizers are sometimes added to improve the strength of the FDR layer. This process allows the distress patterns in the existing pavement to be completely destroyed. Therefore, it can be used to address bottom-up cracking, cracking that extends to a considerable depth, and other distresses that cannot be effectively corrected with CIR. The life-cycle-cost-analysis (LCCA) conducted by Diefenderfer et al. indicated that the pavement maintenance strategies involving stabilized full-depth reclamation (SFDR) are about 16% less costly than the conventional mill and fill strategies in a 50-year analysis period (Diefenderfer and Apeagyei 2011). FDR is usually used in combination with an AC overlay. The treatment provides comparable service life as a reconstruction project.

Throughout the brief review of these technologies, it was found thin asphalt overlays and TSTs had been primarily used for pavement preservation. However, these treatments require the existing pavement to be in a good condition to achieve their best interests. In-place recycling technologies, such as CIR and FDR, provide lower-cost alternatives to traditional rehabilitation or reconstruction methods to improve the pavement condition to a level that thin overlays and TSTs can be constructed. Therefore, thin overlays, TSTs, and the combinations of these treatments and recycling technologies can be used to develop holding strategy treatments that are designed for different holding time periods.

4.4. Objectives and Methodology

As part of the IADOT's efforts to develop holding strategies, nine test sections including a control section which received a conventional 2-inch overlay treatment were constructed on a 13-mile segment of IA-93. The treatments and test section numbers are listed in Table 4.2. An additional section was also constructed in the municipality of Fayette. The treatment constructed for this section included milling down one inch of the existing pavement and applying a chip seal to the milled surface. Because the geometry, traffic speed, and pavement structure of this section are different from those of the other sections, this additional section is not included in this paper for analyses and comparisons. The objective of this study is to evaluate the performance of these proposed holding strategy treatments and develop a decision tool to assist highway agencies in selecting an appropriate holding strategy.

Table 4.2. IA-93 Holding Strategy Test Sections

Section #	Treatment	Length, Mile
1	1" scarification + 1.5" AC overlay	1.3
2	1" scarification + 1.5" AC overlay + single chip seal	2.0
3	1" scarification + 1" interlayer course + 0.75" ultra-thin AC overlay	2.2
4	8" FDR + 1.5" AC overlay	1.0
5	8" FDR + double chip seal	0.4
6	2.5" CIR + double chip seal	1.4
7	2.5" CIR + 1.5" AC overlay	1.6
8	2" AC overlay	1.4
9	1" leveling and strengthening course + single chip seal	1.9

IA-93 is a two-lane highway that has an average daily traffic (ADT) of 1040 and 3 percent trucks. The road has two 12-foot traffic lanes and a speed limit of 55 miles per hour (mph). The existing pavement had 7 to 8 inches of bituminous material and 6 inches of aggregate base. Before the construction of the test sections in 2013, the road was suffering from severe surface defects, transverse cracking, and edge breaks; it had a very rough surface with an international roughness index (IRI) of 246 inches per mile. However, signs of structural failure, such as fatigue cracking, extensive longitudinal cracking on wheel paths, and rutting, were few on the existing pavement. Pavement cores also indicated the cracks were initiated from the pavement surface.

The 1.5 and 2-inch surface courses were made with PG58-28 asphalt binder and aggregate with 3/8-inch nominal maximum aggregate size (NMAS). The ultrathin surface course, 1-inch interlayer, and 1-inch leveling course used smaller aggregate sizes with a 4.75 mm NMAS. The interlayer and ultrathin surface course was designed to withstand the traffic load and prevent reflective cracking despite the thin lift thicknesses. High quality asphalt binders and higher binder contents were used to produce hot-mix-asphalt (HMA) mixtures. The ultrathin overlay used 6.7% binder content and a polymer-modified PG76-34 binder; and the interlayer used 7.4% binder content and a PG64-34 binder. The CIR and FDR were constructed using foamed asphalt as the recycling agent. Fly ash was also applied as a stabilizer for FDR sections.

Pavement condition surveys were conducted before and after the construction. A schedule of the surveys is shown in Table 4.3. In each survey, the test section performance was examined for cracking, raveling, potholes, rutting, and other distresses. A mobile phone-based roughness measurement system was employed in the 4th post-construction survey to estimate the IRI of the test sections. This IRI measurement system was reported to have 81% correlation (R-square value) with the traditional laser measurement systems (Forslof and Jones 2015).

Table 4.3. Pavement Condition Survey Schedule

Preconstruction Survey	July 2013
Project Construction	August and September 2013
1st Post-Construction Survey	September 2013
2nd Post-Construction Survey	April 2014
3rd Post-Construction Survey	November 2014
4th Post-Construction Survey	April 2015

The pavement condition survey results are used to compare the effectiveness of various holding strategy treatments to correct distresses and also to estimate the life expectancies of the treatment. Life-cycle-cost-analysis (LCCA) was conducted to evaluate the cost-effectiveness of each treatment. The decision tool was developed to consider the treatment life expectancy, life-cycle cost-effectiveness, and constructability.

4.5. Test Section Performance

The construction of the test sections was successful. The post-construction survey that was conducted immediately after construction showed that all of the sections were free from distresses. The follow-on surveys that were executed during the first 2 years after construction showed that the predominant distress type was transverse cracking. Figure 4.1 summarizes the transverse cracking densities of the test sections. Sections 4 through 7 exhibited the least amount of transverse cracking among all test sections. Sample cores reveal that most cracks were reflective cracks caused by the crack pattern that remained in the existing pavement. This indicates the recycling technologies are more effective in correcting the cracking pattern in comparison to milling and filling. It also indicates that the ability to correct the cracking pattern is related to the application depth of the treatment. For these sections which had received aggressive base treatments, such as CIR or FDR, a chip seal surface that was applied directed on the prepared base had performance was comparable as that of a similar test section that had an

AC layer instead of the chip seal. The thickness of the AC overlay influences the performance of the other test sections. The three sections with 1 to 1.5-inch thin AC overlays (Section 1, 2 and 9) exhibited more transverse cracking than Section 8 which had a 2-inch overlay. Section 3 has an AC interlayer and an ultrathin overlay which are made with high quality asphalt binders at high binder contents. This mixture was designed to retard the development of reflective cracking. Figure 4.1 shows Section 3 had less transverse cracking in comparison to Section 8. In addition, according to the observation herein, chip seal can be effective improving the reflective cracking resistance of a pavement. Section 9 performed better than Section 1 in terms of crack resistance. Although the overlay thickness of Section 9 is one half inch thinner in comparison to that of Section 1, the addition of chip seal layer seemed offset by the influence of the thinner overlay thickness and possibly providing this section the better resistance against reflective cracking. Less cracking was also found for Section 2 in comparison to Section 1. Section 2 has the same structure as Section 1; except for Section 2 also has a chip seal layer. However, the better performance of Section 2 may have also resulted from having less transverse cracking density on the existing pavement. Figure 4.1 also shows the cracks appeared after the first winter following construction; and cracking densities did not noticeably change during the first two years. For some sections, lower cracking densities were observed at a later time. This was most likely caused by measurement errors during pavement surveys.

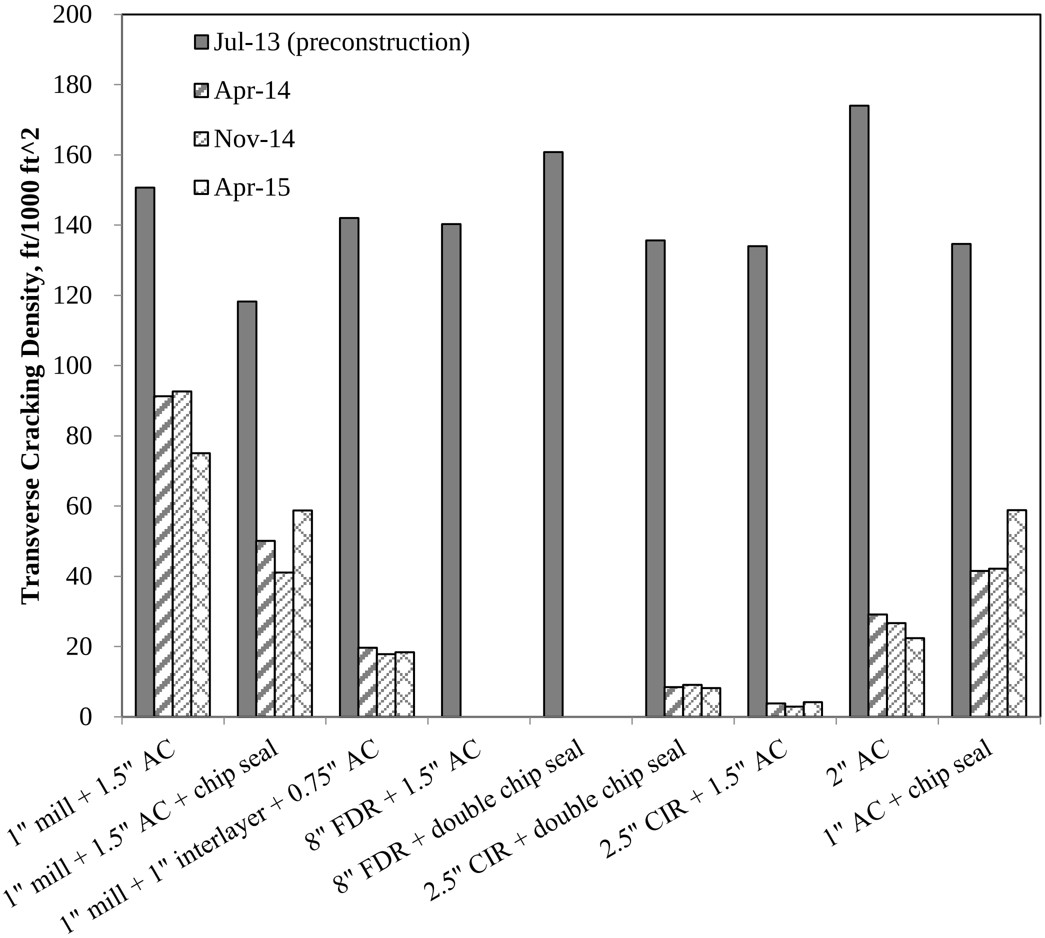

Figure 4.1. Transverse Cracking on IA-93 Test Sections

All of the sections exhibited satisfactory performance in terms of longitudinal cracking, rutting, and roughness. Measurements of these types of distresses are shown in Figures 4.2 through 4.4, respectively. There were no longitudinal cracks found for Sections 3 through 7. For the other rural sections, the longitudinal cracks were very few.

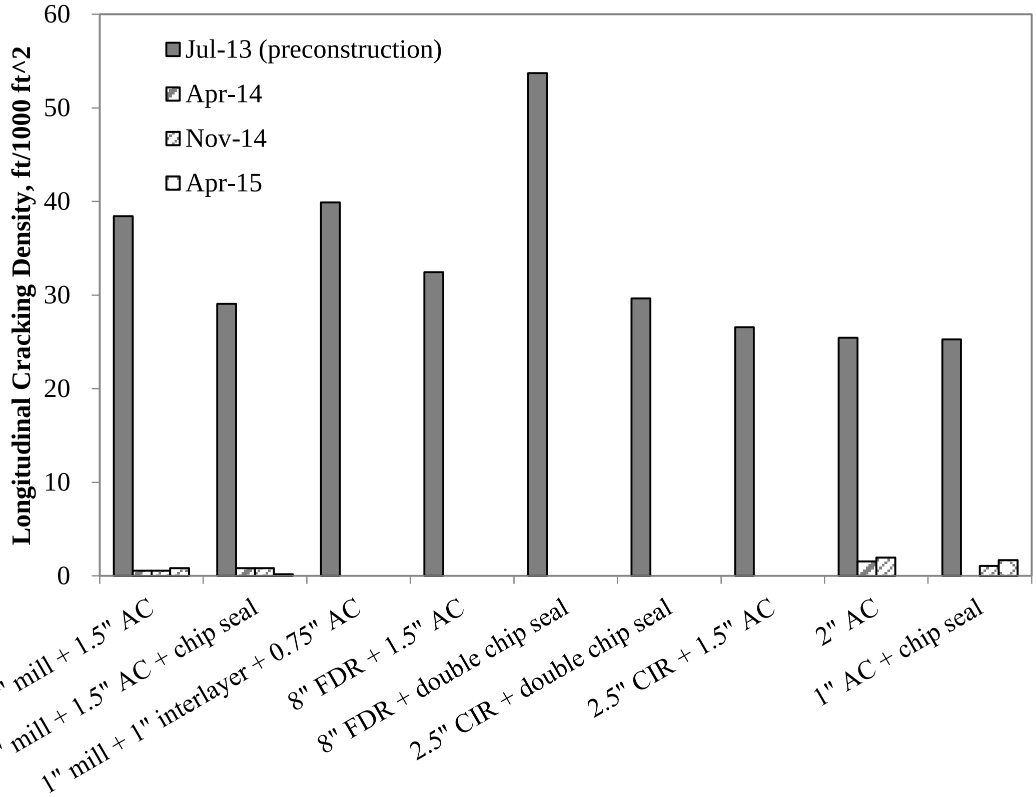

Figure 4.2. Longitudinal Cracking on IA-93 Test Sections

All sections exhibited no or very minor (less than 2 mm) rutting. The error bars in Figure 4.3 show the standard deviations of the measurements. The large error bars for Sections 1, 7, and 8 indicate that rutting was localized to those three sections.

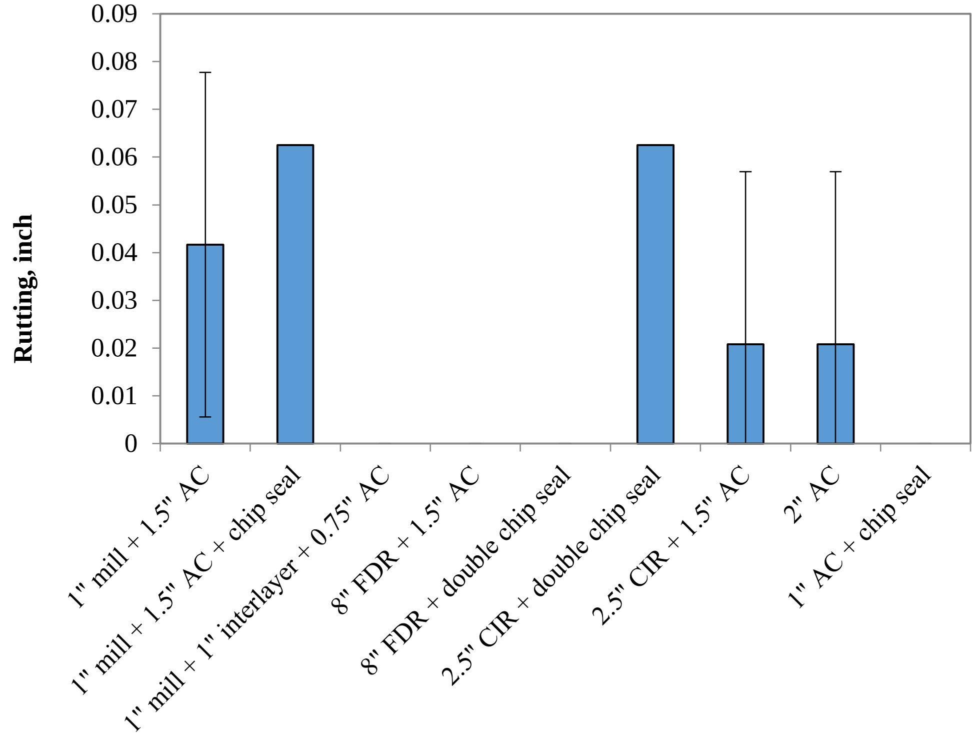

Figure 4.3. Rutting on IA-93 Test Sections

Pavements with an IRI less than 95 inches per mile are typically considered as having a very smooth surfaces. All test sections had IRI values less than 95 inches per mile; and were exhibiting very good ride quality. Figure 4.4 indicates that the sections which do not have an AC layer (Sections 5 and 6) exhibited slightly higher IRI values in comparison to the other sections which include an AC layer.

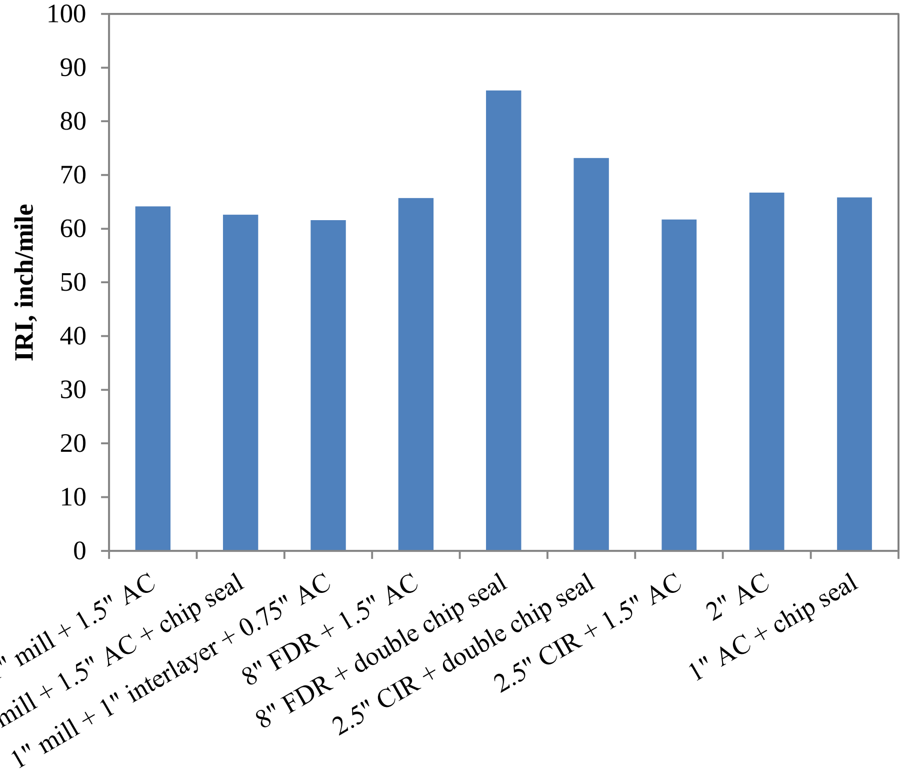

Figure 4.4. International Roughness Index of IA-93 Test Sections.

Severe loss of chip seal cover aggregate was identified in the wheel paths of Section 5. In some areas, the aggregate loss had resulted in small potholes forming. This section had chip seal applied over an FDR layer; however, chip seals for the other sections were placed on either an AC layer or a CIR layer. In comparisons to AC and CIR layer, the FDR layer has rougher surface texture, which may result in lower bonding strength between the chip seal and the substrate, especially if dust collects in low spots. Therefore, apparently the cover aggregate on Section 5 can be removed by traffic and snow plow operations more easily than aggregates on those sections with apparently better bond strength between the chip seal surface and layers below. The surface conditions of the chip seal with stripped cover aggregate and the chip seals without this problem are compared in Figure 4.5.

Figure 4.5. Surface Condition of Chip Seals

Although sections with more cracks may exhibit lower early-age IRI, these sections are more likely to fail in a shorter amount of time than sections with less cracks because cracking allows water and air to penetrate the surface layer which results in water damage and oxidization which accelerate the deterioration process. Therefore, the ability to correct cracking indicates treatment's capacity to restore pavement condition and extend pavement life. The amount of cracks that appeared in the first two years after construction was compared to the amount of cracks on the existing pavement before construction. A percent crack reduction was calculated for each section in Table 4.4. It is assumed that the treatments that are capable of reducing cracks on the existing pavement by more than 95% of cracks, that is comparable to that a service life extension of a reconstructed pavement. According to the percent crack reductions, the life expectancy of each section is predicted in Table 4.4.

Table 4.4. Crack Reduction and Treatment Life Expectancy

Test Section	Crack Reduction, %	Life Expectancy, years
1	54	3-5
2	66	5-8
3	90	8-15
4	100	15-25
5	100	15-25
6	95	15-25
7	98	15-25
8	86	8-15
9	70	5-8

4.6. Life Cycle Cost Analysis

In order to evaluate the cost-effectiveness of the holding strategies, a hypothetical scenario is created assuming that a low-volume road that is similar to IA-93 is due for reconstruction. Various rehabilitation strategies including traditional mill and fill method and that the holding strategy treatments discussed in this paper are applied to remedy the road condition. No other major maintenance treatments are scheduled for treatments that have an AC surface. Because chip seal wears out under traffic and snow removal operations relatively faster than AC, additional seal coats are planned for the treatments that involve a chip seal surface. The holding strategy treatment used for Section 5 requires more frequent chip seal application in comparison to other methods because the performance of Section 5 indicates that the surface of this treatment wears out more rapidly. The rehabilitation alternatives are summarized in Table 4.5.

Table 4.5. Rehabilitation Alternatives and Scheduled Maintenance Activities

Alternative	Treatment Method	Routine Activity	Expected Life Extension
A	1” scarification + 1.5” AC overlay	Crack filling	4
B	1” scarification + 1.5” AC overlay + single chip seal	Crack filling x 2; chip seal	7
C	1” scarification + 1” interlayer course + 0.75” ultra-thin AC overlay	Crack filling x 3	12
D	8” FDR + 1.5” AC overlay	Crack filling x 3	20
E	8” FDR + double chip seal	Crack filling x 6; chip seal x 6	20
F	2.5” CIR + double chip seal	Crack filling x 3; chip seal x 3	20
G	2.5” CIR + 1.5” AC overlay	Crack filling x 3	20
H	2” AC overlay	Crack filling x 3	12
I	1” leveling and strengthening course + single chip seal	Crack filling x 2; chip seal	7
J	4” mill + 5” AC	Crack filling x 3	20

The costs of the test section constructions are used to estimate the costs of the alternatives. The costs for maintenance activities are estimated from the bid prices for several maintenance projects in Iowa and neighboring states. The costs for treatment construction and maintenance as well as the equivalent annual costs (EAC) are summarized in Table 4.6. The costs estimation indicates that the construction costs for Alternatives A through I are 39 to 63% lower than the construction cost for Alternative J. Alternatives A, B, C, E, and I show higher EAC than the conventional mill and fill method. Compared to Alternative J, these strategies, except for Alternative E, provide less service life extensions. Therefore, these alternatives can be used only as a temporary treatment to restore pavement functionality for a limited time period. If sufficient funding is granted, other long-term alternatives should be selected to improve the pavement condition. The other holding strategy alternatives involving CIR or FDR provide comparable service lives in comparison to the traditional rehabilitation method. The EAC of these alternatives are lower than the EAC of Alternative J. Therefore, these treatments can be used as alternative rehabilitation methods in the situations that a long-term solution is desirable. Chip seal tends to have higher maintenance costs than AC. For Alternatives E and F, the increase in maintenance costs offset the cost savings from the lower construction costs in comparisons to Alternatives D and G; and result in higher EAC. However, the applications of chip seal as a surface maintenance method may considerably extend pavement service life, which may result in that the actual cost-effectiveness of the treatments involving a chip seal is underestimated. Long-term performance observation is needed to investigate the influence of such routine maintenance activities on pavement service life and cost-effectiveness.

Table 4.6. Costs per Road Mile for Alternative Strategies

	Alternative	Construction Cost, $	Maintenance Cost, $	EAC, $
A	1” scarification + 1.5” AC overlay	104,200	500	26,200
B	1” scarification + 1.5” AC overlay + single chip seal	130,700	26,000	22,400
C	1” scarification + 1” interlayer + 0.75” ultra-thin overlay	153,500	1,400	12,900
D	8” FDR + 1.5” AC overlay	181,900	1,400	9,200
E	8” FDR + double chip seal	146,800	150,300	14,900
F	2.5” CIR + double chip seal	94,000	75,100	8,500
G	2.5” CIR + 1.5” AC overlay	130,000	1,400	6,600
H	2” AC overlay	119,000	1,400	10,000
I	1” leveling and strengthening course + single chip seal	99,800	1,000	14,400
J	4” mill + 5” AC	255,500	1,000	12,800

4.7. Treatment Selection Tool

Holding strategies are applied when funding is insufficient for agencies to afford traditional rehabilitation or reconstruction. This requires that the treatments to have considerably lower construction costs and comparable life-cycle cost-effectiveness in comparison to a conventional “permanent” solution. Table 4.6 provides construction cost estimates for the nine treatments that were used for the IA-93 project are considerably lower than the 4-inch mill and 5-inch overlay method; however, some alternatives show significantly higher EAC. The alternatives with an EAC that is higher than $15,000 are considered to be less cost-effective and disqualified for being used as holding strategy treatments. The constructability, life expectancies, and cost-effectiveness of the other treatments are compared; and a decision table (Table 4.7) is proposed to select the appropriate holding strategy for a particular situation.

The life expectancies of the alternative strategies are estimated based on the short-term performance of the IA-93 test sections. The LCCA results and this treatment selection tool are only suggested for use as references to assist the holding strategy selection process. Long-term performance observations are required in order to develop a more reliable decision tool.

Table 4.7. Decision Table for Holding Strategy Selection

Evaluate structural adequacy through pavement survey, core samples, or non-destructive tests.	No structural improvement needed (no severe longitudinal cracking on wheel paths; no severe fatigue cracking; cracks initiated from pavement surface; no high severity rutting)	Consider Type-A strategies
	Need a structural improvement (severe longitudinal cracking on wheel paths, fatigue cracking, cracks initiated from bottom, and high severity rutting)	Consider Type-B strategies

Type-A Strategy

(Alternatives in the left columns have higher priorities than alternatives in the right columns. Consider lower priority alternatives if the conditions do not allow the use of higher priority alternatives or the technologies for higher priority alternatives are not locally available.)

Required holding time less than 8 years

Alternative	CIR + thin overlay	CIR + double chip seal	FDR + thin overlay	2-inch overlay	FDR + double chip seal	milling + interlayer + ultrathin overlay	1-inch overlay + chip seal

Required holding time 8 to 15 years

Alternative	CIR + thin overlay	CIR + double chip seal	FDR + thin overlay	2-inch overlay	FDR + double chip seal	milling + interlayer + ultrathin overlay

Required holding time more than 15 years

Alternative	CIR + thin overlay	CIR + double chip seal	FDR + thin overlay	FDR + double chip seal

Type-B Strategy

(Alternatives in the left columns have higher priorities than alternatives in the right columns. Consider lower priority alternatives, if the conditions do not allow the use of higher priority alternatives or the technologies for higher priority alternatives are not locally available.)

Alternative	CIR + overlay	FDR + overlay	mill + fill or reconstruction

Restrictive Conditions

CIR + thin overlay	• Pavement thickness is too small (higher risk of construction failure for bituminous material thickness less than 6 inches) or base and subgrade strength are too low to support construction equipment. • Budget for initial construction is low.
CIR + double chip seal	• Pavement thickness is too small (higher risk of construction failure for bituminous material thickness less than 6 inches) or base and subgrade strength are too low to support construction equipment. • Budget for maintenance is low. • Smooth surface is desired by users.
FDR + thin overlay	• Pavement thickness is too large and exceeds the maximum reclamation capacity of the equipment (typically 12 inches). • Budget for initial construction is low.
2-inch overlay	• Shoulder width is too narrow due to high pavement elevation. Increase in pavement thickness may result in high costs to accommodate slope changes in shoulder.
FDR + double chip seal	• Pavement thickness is too large and exceeds the maximum reclamation capacity of the equipment (typically 12 inches). • Budget for maintenance is low. • Smooth surface is desired by users.
milling + interlayer + ultrathin overlay	• Budget for initial construction is low.
CIR + overlay	• Pavement thickness is too small (higher risk of construction failure for bituminous material thickness less than 6 inches) or base and subgrade strength is too low to support construction equipment.
FDR + overlay	• Pavement thickness is too large and exceeds the maximum reclamation capacity of the equipment (typically 12 inches).

4.8. Conclusion

Budget shortfall has been one of the biggest challenges for maintaining the condition of highway networks, especially for low-volume roads. Holding strategy provides cost-effective alternatives of traditional rehabilitation or reconstruction treatments; and allows highway agencies to allocate maintenance budget with some degrees of flexibility. In order to develop treatment methods that can be used for holding strategies, Iowa DOT constructed test sections in 2013 on IA-93. A series of pavement condition surveys were performed to evaluate the performance of each treatment method. This paper summarizes the results of the pavement condition surveys. An LCCA is conducted to compare the cost-effectiveness of the holding strategies and a conventional rehabilitation treatment. Based on the LCCA results, a decision table is proposed as a reference to assist selection of the appropriate holding strategy.

The pavement condition surveys indicated the holding strategy treatments successfully corrected surficial distresses of the existing pavement. However, cracking patterns had not been eliminated in some sections which leaded to presences of reflective cracking after the first winter. During the 2-year monitoring of test section performance, the number of cracks did not considerably change. This indicates that reflective cracking develops rapidly; while, new cracks occur at a slower rate.

Recycling technologies, including CIR and FDR, which allow cracking patterns at a greater depth can be treated exhibited the best performance in terms of cracking mitigation. The interlayer and ultrathin overlay and the 2-inch overlay had moderate capability of preventing reflective cracking without aggressive surface preparation treatments. A chip seal seemed to be effective in reducing reflective cracking and prolonging pavement life. In addition, the benefits of applying a surface preparation involving 1-inch milling depth had very limited effect on section performance.

All treatments successfully corrected the surface defects of the existing pavement and improved ride quality. Although some sections had extensive reflective cracking, their IRI values indicated the pavements were in good condition. The sections with an AC layer, regardless of the thickness, had lower IRI values than the sections that have had a chip seal applied directly on the recycled base.

Loss of chip seal cover aggregate was observed for the test section that has received a FDR and double chip seal treatment. The rough surface of the FDR layer seems to have increased the risk of chip seal damage caused by traffic and snow plowing operations.

The LCCA results indicate that the EACs of the 1-inch milling and 1.5-inch overlay methods are considerably higher in comparison to the EAC of the traditional mill and fill method. Such methods are not recommended for holding strategies. The other holding strategy treatments had an equivalent level or considerably lower EAC than the traditional method. Meanwhile, the initial construction costs of all of the treatments are 39 to 63% lower than the construction cost of the traditional method.

A decision table was developed based on the LCCA results, treatment performance, and constructability. This decision tool is recommended for decision makers to use as a reference. Future research and long-term performance data is needed in order to develop a more reliable decision tool that can be used as a guideline for holding strategy selection.

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CHAPTER 5. FIELD STUDY OF SURFACE CHARACTERISTICS OF CHIP SEAL AND ASPHALT CONCRETE WITH VARIOUS UNDERLYING STRUCTURES

Modified from a paper to be submitted to the *Journal of the Performance of Constructed Facilities*

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5.1. Abstract

The increased interest in pavement preservation and lower-cost rehabilitation alternatives has resulted in increased implementation of chip seal treatments. Pavements with chip seals have distinguished surface characteristics compared to asphalt concrete (AC) surfaces. Such differences can result in different road functional performance, such as friction, noise generation, tire wear, and fuel economy, which influence passengers' safety, level of comfort, and user costs. The surface characteristics of chip seals and bitumen surfaces have been studied extensively. However, little research has focused on the influences of the pavement structure and base treatment method on the surface layer functional characteristics. This paper investigates the different surface behavior of chip seals and AC surfaces. The surface characteristics are evaluated using friction coefficient, mean texture depth (MTD), and international roughness index (IRI). A dynamic friction tester (DFT) was used to measure the friction coefficients; and the IRI was estimated with a smartphone-based roughness measurement system. The surface characterization tests were performed for ten test sections on a low-volume full-depth asphalt road in Iowa. The test sections include five chip seal sections and five AC surfaced sections with

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varying pavement structures. Some sections contain a recycled base layer treated with cold in-place recycling (CIR) or full-depth reclamation (FDR). This paper focuses on the influences of surface type, pavement structure, and traffic on road surface characteristics. The findings indicated that chip seals are comparable to asphalt concrete in terms of surface performance such as skid resistance and roughness. However, the chip seal layer applied directly on a full-depth reclamation base was found to suffer from a loss of the macro-texture.

Key words: chip seal, holding strategy, friction coefficient, surface texture, asphalt.

5.2. Introduction

The increased interest in pavement preservation and lower-cost rehabilitation alternatives has resulted in increased implementation of chip seal treatments. Compared to pavements with a hot-mix-asphalt (HMA) surface, pavements with chip seal treatments have different surface characteristics. Such differences can result in different road functional performance, such as friction, noise generation, tire wear, and fuel economy, which influence passengers' safety, level of comfort, and user costs. The surface characteristics of chip seal and bitumen surface have been studied extensively. However, little research has focused on the influence of the pavement structure and base treatment method on the surface layer functional characteristics.

Pavements with different structures are expected to have different service lives given the same traffic loading and climatic conditions. The rates of serviceability loss, which is primarily due to by the changes in surface characteristics, may also be substantially different. Such changes in surface characteristics may be driven by a complicated mechanism. For example, pavements with different thicknesses may have different air voids after a certain amount of traffic even though their initial air voids may be the same. The variations in air voids may result in distinctively different levels in permeability which affects the drainage behavior and results in different levels of skid resistance. Chip seal surfaces with different underlying structures may have different bonding strengths at the interface of the chip seal layer and the underlying structures. Therefore, their surfaces may experience differences in aggregate stripping caused by traffic or snow plow operations. The knowledge of such differences provides useful information when a decision maker is selecting a rehabilitation treatment at the project level or a state highway agency is planning for its pavement management strategy at the network level.

This paper evaluates the surface characteristics of ten test sections over a 13 mile stretch on a low-volume state highway in Iowa. The test sections include asphalt layers with different thicknesses, and various chip seal and recycling technologies including cold in-place recycling (CIR) and full-depth reclamation (FDR). The surface characteristics of the test sections were evaluated using a dynamic friction tester (DFT), the sand patch test (SPT), and a smartphone-based international roughness index (IRI) measuring system. The effects of pavement structure on surface characteristics and the correlation between different tests are discussed in this paper.

5.3. Background

Road functionality is affected by pavement surface texture and roughness. Depending on the wavelength and amplitude, the pavement surface textures are usually discussed at three levels: the micro-texture, macro-texture, and mega-texture levels. The texture wavelength and amplitude at each level are summarized in Table 5.1 (PIARC 1987). Micro and macro-texture are related to aggregate properties. The micro-texture of pavement is determined by the surface roughness and surficial voids of aggregate particles. The particle size, shape, and gradation control the pavement macro-texture. The mega-texture is usually affected by pavement distresses and defects. The effects of each type of surface texture on pavement functionalities have been well discussed by Sandburg (1998), Henry (2000), Flintsch et al. (2002), and Hall et al. (2009). Micro-texture and macro-texture dominate the tire-surface interface behavior which determines the friction, noise, and tire attrition performance of pavements. The noise generation is primarily influenced by the macro-texture; while, the friction and tire attrition depend on both micro and macro textures. Roughness describes the vertical variations of the road profile. Mega-texture and roughness affect the vehicle fuel efficiency and the passengers' experiences about the smoothness of the pavement surface.

Table 5.1. Surface Texture Levels at Various Wavelengths and Amplitudes

Surface Texture Level	Wavelength	Peak-to-Peak Amplitude
Micro-texture	<0.5mm	0.001 to 0.5mm
Macro-texture	0.5 to 50 mm	0.1 to 20 mm
Mega-texture	20 to 500 mm	0.1 to 50 mm

The micro-texture of a pavement is usually difficult to measure directly. However, this material property is related to pavement friction and controls the magnitude of friction at low slipping speeds (Flintsch, et al. 2002). Friction is usually measured using full-scale test tire devices, such as the locked wheel friction tester, the fixed-slip tester, and the variable slip tester, or the dynamic friction tester (DFT). The coefficient of friction is measured and converted to a friction number (FN) or an international friction index (IFI). The value of FN or IFI is dependent upon the type of the slider or tire of the friction testing device. However, a general increase in friction was found for deteriorated road surfaces which received a chip seal treatment. Seneviratne and Bergener (1994) (Thompson, Garcia and Carpenter 2009) indicated that an average improvement of 24 in FN value for the pavements treated with chip seals. Li et al. conducted comprehensive research on the long-term behavior of the surface characteristics of various preventive maintenance treatments, including chip seals, fog seals, microsurfacing, thin asphalt overlays, and ultrathin bounded wearing courses (UBWC) (Li, et al. 2011). The study showed that the FN values measured at 40 miles per hour [FN(40)] after the chip seal treatments were 16 to 43% higher than the FN(40) values before the treatments. A higher slipping speed typically results in a smaller FN value (Flintsch, et al. 2002). Both studies indicate that the FN(40) value of a newly constructed chip seal surfaces varies from 40 to 70. Seneviratne and Bergener found there was no correlation between the FN values before and after the chip seal treatments. However, the results of Li et al.'s study show that the chip seal surface has a higher FN value if the FN of the pavement before construction is higher. Li et al. also evaluated the effects of the service time on the FN value. The FN values of various treatments exhibited a decreasing trend over a long period of time due to traffic loading. The friction of the fine-graded thin asphalt layer rapidly decreased after construction. The initial FN(40) values of the thin asphalt layers varied between 35 and 52; while, the FN values were 36 to 48% lower than the initial FN values. The UBWC showed a relatively more stable friction performance with a FN(40) of 48 to 59. There was no considerable decrease in the FN value observed in the 48 months after the completion of construction. However, the newly placed surfaces of fog seal and microsurfacing sections experienced a curing phase where friction increased gradually over time. This effect offsets the initial friction loss caused by the traffic. The duration of the curing process is different for varying surface types. The typical curing durations for fog seal and microsurfacing are 6 and 12 months, respectively. The chip seal surfaces also had a curing stage in the first 12 months after construction. At the curing stage, aggregates were lost from the surface because of the immature bonding strength between the aggregate and emulsion. This results in a rapid decrease in friction with the initial FN(40) values decreasing by 10 to 30% in 12 months.

Macro-texture affects the friction at high slipping speeds or when water is present. Hysteresis caused by tire deformation due to the pavement macro-texture accounts for more than 95 percent of the overall friction at a speed higher than 65 mph (PIARC 1987). High macro-texture also facilitates drainage and reduces hydroplaning which forms a water film at the pavement-tire interface and causes a significant decrease in friction. Macro-texture can be evaluated by mean texture depth (MTD) or mean profile depth (MPD) using the sand patch test (SPT) or a laser profiler, respectively. MTD and MPD are the average pavement surface profile from the highest point. MTD is measured in two dimensions; while MPD is tested in one dimension. The typical MPD values for an asphalt surface ranges from 0.4 to 2.5 mm (Rada, et al. 2013). New Zealand has established the failure criteria for the MTD of chip seal surfaces: the minimum MTD is 0.7 mm for roads with a speed limit less than 44 mph and 0.9 mm for roads with higher speed limits (Pierce and Kebede 2015). The noise generation is higher when the macro-texture is higher. A linear correlation between the noise levels measured at the vehicle underbody and the pavement MTD was established by Saykin in 2011; and indicated the noise energy level was increased by approximately 8% when the surface macro-texture was increased from 0.5 mm to 1.5 mm.

The mega-texture and roughness are typically evaluated by the international roughness index (IRI). IRI serves as an important performance indicator in many state highway agencies' pavement management system; and is used in combination with distress survey results to compute a pavement quality index (PQI). IRI is usually measured with a profiler that measures the variation of the road longitudinal profile. Pavements with an IRI value smaller than 95 inches per mile are typically considered having “good” to “very good” ride quality; while, an IRI value greater than 170 inches per mile is generally unacceptable. A recent smartphone-based application (Roadroid) was developed by Swedish scientists Hans Jones and Lars Forslof as a lower-cost alternative of the conventional IRI profiler method for IRI measurments. This smartphone application collects vibration data from the built-in acceleration sensor of the smartphone and correlates the vibration readings to IRI. The application is able to provide 80% reliability for an information quality level (IQL) of 3 which can be used for program analysis or detailed planning (Jonhes and Forslof 2014).

5.4. Research Objectives and Methodology

The primary objective of this research is to evaluate the surface characteristics of chip seal and asphalt pavement applications with various pavement structures. The research focuses on the influences of pavement structure and traffic on pavement surface characteristics at three levels: micro-texture, macro-texture, and roughness.

Ten test sections were constructed on a 13-mile segment on IA 93 in 2013. The original pavement structure includes 7 to 8 inches of asphalt pavement and 6 inches of aggregate base. The road carries an average daily traffic of 1040 with 3 percent truck traffic. The project consists of two segments. The rural segment has two 12-foot traffic lanes and a speed limit of 55 mph. The urban segment is located in the vicinity of the Fayette municipality and has a 12-foot traffic lane and a 6-foot parking lane for each direction. The speed limit for the urban segment is 45 mph. The road was suffering from various pavement distresses and surface defects. A series of rehabilitation treatments were constructed. Table 5.2 summarizes the applied treatments for each section and the pavement structures. Test sections 1 through 9 were established on the rural segment; while, Section 10 was constructed on the urban segment.

Table 5.2. Test Section Rehabilitation Treatments

Section Number	Base Treatment	Surface Treatment
1	1” scarification	1.5” HMA overlay
2	1” scarification	1.5” HMA overlay and single chip seal
3	1” scarification and 1” interlayer course	0.75” ultra-thin HMA overlay
4	8” full depth reclamation	1.5” HMA overlay
5	8” full depth reclamation	double chip seal
6	2.5” cold-in-place recycling	double chip seal
7	2.5” cold-in-place recycling	1.5” HMA overlay
8	none	2” HMA overlay
9	1” leveling and strengthening course	single chip seal
10	1” scarification	single chip seal

The test sections were subjected to traffic loads, weathering, and snow removal operations, such as snow plowing and deicing. Surface characterization tests were performed in April 2015 – 19 months after construction. At the time of testing, the test sections showed different levels of surface deteriorations. DFT, SPT, and the Roadroid IRI tests were conducted to capture the pavement surface characteristics at the three texture levels.

The DFT and SPT were performed at three random locations for each test section. All tests were conducted on the eastbound traffic lane. Both the outer wheelpath and the middle of the lane were tested. The pavement friction was tested at a slip speed of 60 km/h and in dry and wet conditions, respectively. For, the SPT tests, five sand patches were made at each location and lane position for testing. The average diameters were used to calculate the MTD of the pavement. The experimental design of the DFT and SPT tests for each test section is shown in Table 5.3.

The IRI test was performed by attaching a mobile phone with the vehicle windshield through a smartphone car-mount. A mid-size wagon was used for this test. The vehicle speed was maintained at 50 mph during the data collection.

Table 5.3. Experimental Design Table for DFT and SPT

Test		Random Location					
		1		2		3	
		Wheelpath	Middle of the Lane	Wheelpath	Middle of the Lane	Wheelpath	Middle of the Lane
Dynamic Friction Test	Dry	x*	x	x	x	x	x
	Wet	x	x	x	x	x	x
Sand Patch Test		xxxxx	xxxxx	xxxxx	xxxxx	xxxxx	xxxxx

*: x represents one replicate of the test.

5.5. Surface Characteristics Testing Results

5.5.1. DFT

The average friction coefficients at various testing conditions are shown in Figure 5.1. The error bars indicate the 95% confidence intervals of the mean friction coefficients. The dry friction coefficients of the sections with an asphalt concrete (AC) surface were higher than the friction coefficients of the chip seal surface sections. However, the friction coefficients of the AC surfaces and chip seal surfaces are not very different for the wet condition. For the dry condition, the pavement friction is primarily from the adhesion between pavement surface and tire which is controlled by the micro-texture of pavement surface. When water is present, a water film can form at the pavement-tire interface resulting in hydroplaning. The water film reduces the contact force between the tire and pavement surfaces. The high macro-texture of the chip seal surface results in large energy loss through hysteresis and provides a higher friction coefficient than the AC surface. The rough surface of the chip seal also facilitates water drainage which reduces the amount of water at the pavement-tire interface; therefore, nearly offsets the friction differences between the asphalt and chip seal surfaces.

Traffic appears to have an influence on pavement friction. The dry friction coefficients were higher in the wheelpath compared to the friction coefficients on the lane centerline for the asphalt surface. However, there are few differences in the friction coefficients measured in the wet condition between the wheelpath and the lane centerline. This indicates that traffic may result in micro-texture changes in wheelpaths of asphalt surfaces. For chip seal surfaces, the dry friction coefficients were not affected by the trafficking effects. However, four of the five chip seal sections showed smaller friction coefficients in the wheelpath than those in the lane centerline in the wet condition. Rutting in wheelpaths and loss of macro-texture may lead to the differences in the wet friction coefficients between the wheelpath and the lane centerline. Minor localized rutting was found for Sections 1, 2, and 6. Significant loss of surface macro-texture was found for Section 5. These sections had lower friction coefficients in the wheelpath than in the lane centerline. However, the wheelpath friction for Section 9 was also lower than the lane centerline friction, even though neither rutting nor macro-texture loss was observed for Section 5. Figure 5.1 suggests that the friction coefficients measured at different locations of the same section are variable. The lower friction coefficient on the wheelpath in Section 9 may be a result of the variations in the friction measurements.

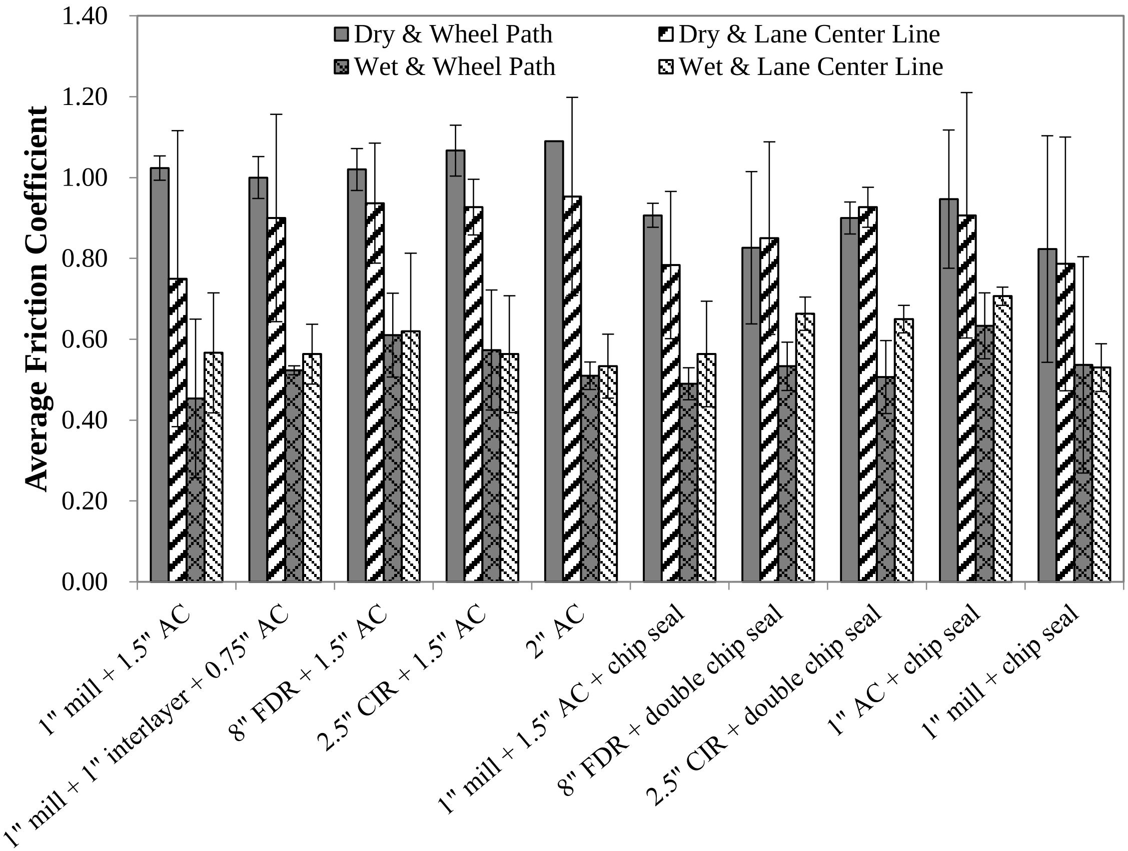

Figure 5.1. DFT Results at Various Testing Conditions

The influences of pavement structure on the friction behavior were also investigated with comparisons between test sections with the same surface type and varying structures. The results are summarized in Table 5.4. The friction coefficient levels are denoted by cell colors. Pavement structures with different cell colors indicate a statistical difference between their friction coefficients. For the sections with an asphalt surface, the lane centerline has the same level of friction coefficient. However, the friction coefficients in the wheelpath are different. Because the influence of traffic is higher in the wheelpaths than the lane centerline, this indicates that pavement structure affects the influence of traffic on surface friction; however, it has little influence on the friction changes caused by other factors. Sections with a chip seal surface exhibited a distinctive difference in friction coefficients. Differences in friction coefficients are found on both the wheelpath and lane centerline. The authors suspect that this may be caused by snowplow operations. Unlike traffic loads which are applied primarily on the wheelpaths, snowplow operations can cause road deterioration over the entire lane width. Compared to the asphalt surface, the chip seal surface is more susceptible to such road damage due to its rough texture. There are no statistically significant differences found for the friction coefficients of the chip seal sections in the dry condition. Because the dry friction behavior is primarily controlled by pavement micro-texture, the similarities of the friction coefficients in the dry condition indicate that the pavement structure does not significantly affect the micro-texture behavior of chip seal surfaces. Pavement friction in the wheelpaths and in the wet condition is usually a concern for safety. Table 5.4 also indicates that the FDR and CIR were able to improve the friction performance of an asphalt overlay. Layer thickness does not significantly affect the pavement safety performance.

Table 5.4. Statistical Comparisons of Friction Coefficients

Surface Type	Pavement Structure Comparison	Dry & Wheelpath	Wet & Wheelpath	Dry & Lane Centerline	Wet & Lane Centerline
AC	1" mill + 1.5" AC	1.02	0.45	0.75	0.57
	1" mill + 1" interlayer + 0.75" AC	1.00	0.52	0.90	0.56
	8" FDR + 1.5" AC	1.02	0.61	0.94	0.62
	2.5" CIR + 1.5" AC	1.07	0.57	0.93	0.56
	2" AC	1.09	0.51	0.95	0.53
Chip Seal	1.5" AC	0.91	0.49	0.78	0.56
	8" FDR	0.83	0.53	0.85	0.66
	2.5" CIR	0.9	0.51	0.93	0.65
	1" AC	0.95	0.63	0.91	0.71
	1" mill	0.82	0.54	0.79	0.53

5.5.2. SPT

The results of the SPT test are summarized in Figure 5.2. The MTD of the chip seal surfaces is considerably higher than that of the asphalt surfaces. The average MTD is 0.602 mm and 1.079 mm for the asphalt and chip seal surfaces, respectively. The measurements taken in the wheelpath are very similar as the measurements tested in the center of the lane for the asphalt surfaced sections. Greater differences in the friction coefficients of the chip seal surfaced sections were observed between the wheelpath and the lane centerline. These differences are statistically significant for the FDR and double chip seal section at the 95% confidence level as quantified by the MTD between the lane centerline and the wheelpath. Trafficking is believed to be the primary cause of the difference between the MTD values of the wheelpath and the lane centerline. The FDR layer is usually considered to be similar to a stabilized aggregate base, rather than an HMA material. Compared to a HMA layer, the FDR layer has lower bond strength to other asphaltic materials. The author observed that the weaker bond strength may lead to a loss of the cover aggregate under traffic loading for chip seals applied directly on FDR layers. For the other test sections, which have an asphalt surface or the chip seal surface was applied on an asphalt layer, the layer thickness and base treatment type have no influence on the SPT results.

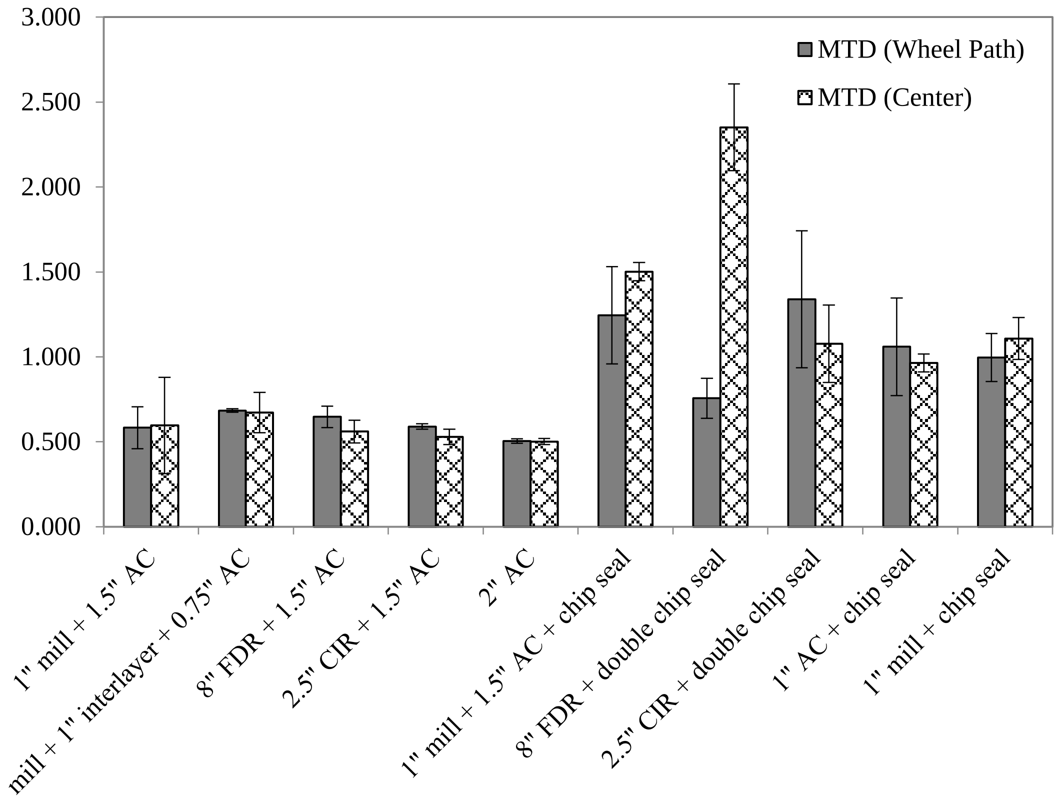

Figure 5.2. Mean Texture Depths

5.5.3. IRI

The existing pavement prior to the construction treatments had an IRI value of 246 inches per mile. Figure 5.3 shows that all sections have a good rideability in terms of roughness after the rehabilitation treatments were applied. Comparisons are performed for sections with the same base treatment types but different surface types as well as sections with the same surface types with different base treatments. The results of the comparisons are shown in Table 5.5. It can be found that an asphalt overlay is critical to correct pavement roughness. The sections with an asphalt layer exhibited much lower roughness than the sections which did not receive an asphalt overlay treatment. For the sections which do not include an asphalt layer, the base treatment type also influences pavement roughness. The 1-inch milling showed the minimum roughness improvement; while the CIR treatment provides the greatest roughness improvements among the three base treatment types: CIR, FDR, and scarification. For sections that have an asphalt overlay, the influences of base treatment type are minimal. It is also found that the addition of a chip seal layer does not significantly affect pavement roughness.

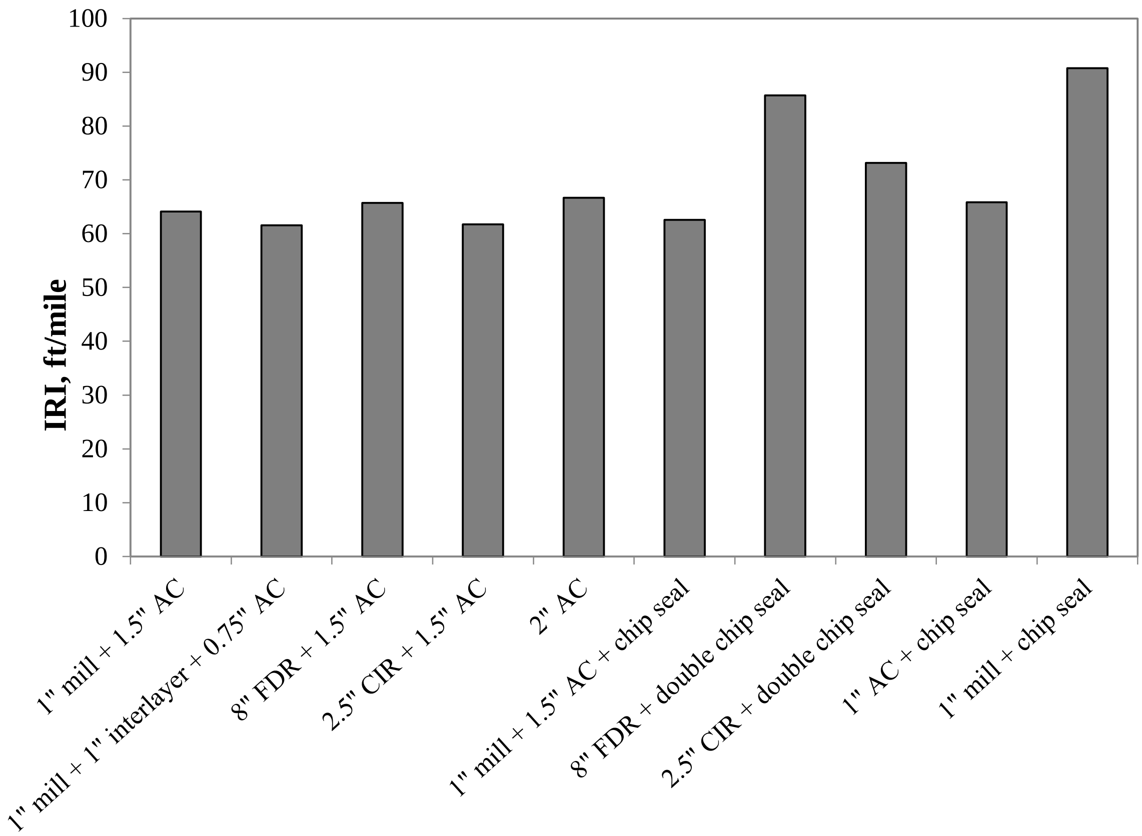

Figure 5.3. IRI Measured from Roadroid

Table 5.5. Comparison of IRI between Sections with Different Surface and Base Types

Base Treatment	Surface Type Comparison	Difference Between IRI Values, ft/mile
1” mill	1.5” AC vs. Chip Seal	-26.6
2.5” CIR	1.5” AC vs. Chip Seal	-11.4
8” FDR	1.5” AC vs. Chip Seal	-20.0
None	2” AC vs. 1” AC + Chip Seal	0.8
1” mill	1.5” AC vs. 1.5” AC + Chip Seal	1.5
Surface Treatment	**Base Type Comparison**	**Difference Between IRI Values, ft/mile**
1.5” AC	1” mill vs. 2.5” CIR	2.4
1.5” AC	1” mill vs. 8” FDR	-1.6
Chip Seal	1” mill vs. 2.5” CIR	17.6
Chip Seal	1” mill vs. 8” FDR	5.0

5.6. Correlation between Tests

Figure 5.4 illustrates the correlations of the DFT results with the MTD and IRI values. The results show little correlations between the DFT measurements and MTD or pavement roughness. A decreasing trend was found for the dry friction coefficients and an increasing trend was found for the wet friction coefficient as the MTD or IRI increases. However, the degrees of correlations for these trends are very low. Figure 5.5 compares MTD values measured in the wheelpath with the IRI readings. An increasing trend was found for MTD as the IRI increases. This indicates that the IRI values measured by the smartphone application may be affected by the surface macro-texture of the pavement. The weak correlations between these surface characteristics tests indicate that pavement textures at different scales (micro, macro, and mega-texture or roughness) do not necessarily correlate. This conclusion conforms to a study of correlations between pavement surface characteristics by Yero et al. (2012). In Yero et al.’s investigation, the friction, texture depth, and roughness of six test roads with three surface types (asphalt wearing course, stone matrix asphalt, and surface dressing) were measured by the British Pendulum Tester, the SPT, and the Australian Road Research Board (ARRB) walking profilometer, respectively. The results of Yero et al.'s study showed the R-square values for the friction-MTD, the friction-roughness, and the MTD-roughness correlation were ranging between 0.12 and 0.29, 0.12 and 0.38, and 0.07 and 0.51, respectively. Yero et al. only tested these surface characteristics in the dry condition; and found that the correlations between these surface characteristics tended to be stronger for the surface dressed surfaces than that for the other surface types.

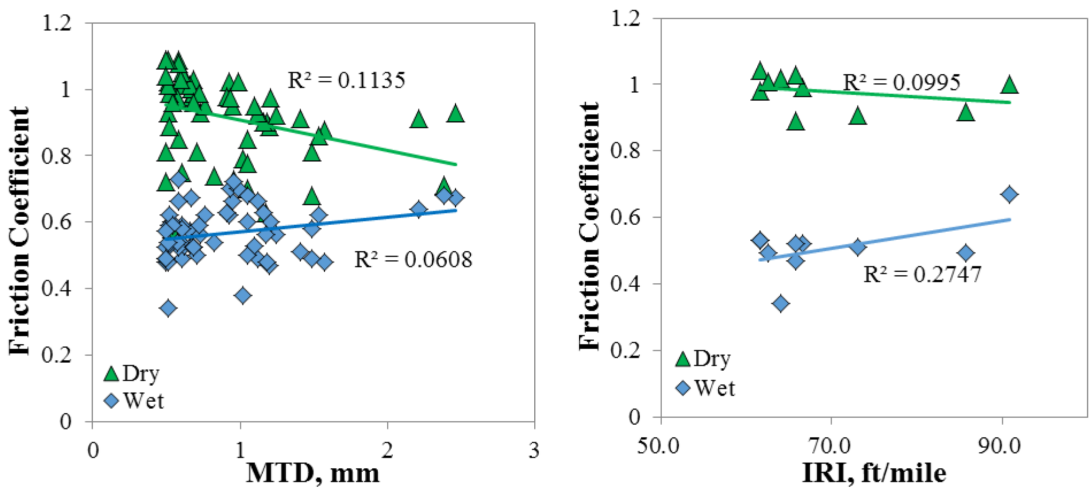

Figure 5.4. Correlations of Friction Coefficient with MTD and IRI

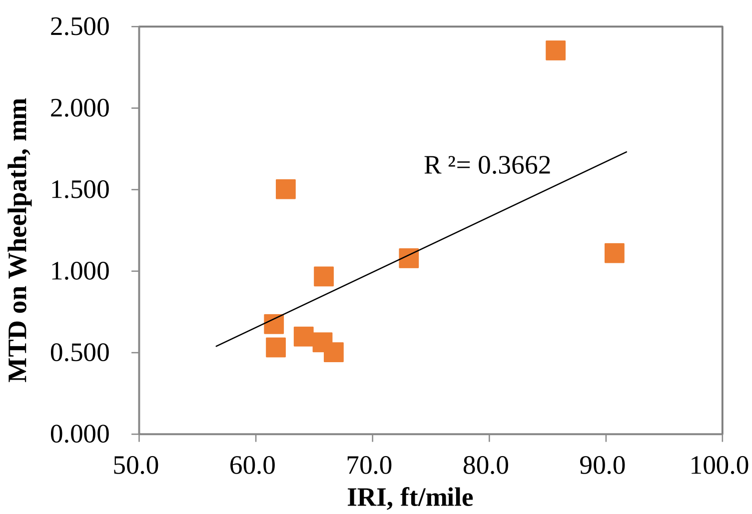

Figure 5.5. Correlation between MTD and IRI

5.7. Conclusion

This paper investigated the surface characteristics of ten test sections on a low-volume road in Iowa. The test sections were treated with various rehabilitation strategy treatments and included various surface types and pavement structures. The road surface textures at various levels were evaluated with DFT, SPT, and a smartphone-based application. The influences of surface type, pavement structure, and traffic loading on pavement surface characteristics were discussed.

The DFT results show that the friction coefficients in the dry condition were higher for the asphalt surface than for the chip seal surfaces. There were no significant differences in the friction coefficients between different surface types for the wet condition. Traffic is an influencing factor for pavement friction behavior. The average friction coefficients were generally higher for the asphalt surfaced sections in the dry condition and lower for the chip seal surfaced sections in the wet condition in the wheelpath than the friction coefficients in the lane centerline for the same sections at the same testing conditions. Pavement structure did not significantly affect the friction coefficients of the asphalt surfaced sections in the lane centerline. However, the friction coefficients of the asphalt surfaces in the wheelpath were found to be different. The asphalt surfaces with a CIR or FDR treated base showed better friction performance than the other sections in the wet condition. For the chip seal surfaced sections, the dry friction coefficients of the various sections showed no difference. However, different friction performance in the wet condition was observed in both the wheelpath and the lane centerline. The section with a 1-inch leveling course and a chip seal surface had the best friction performance among the chip sealed sections. The asphalt overlay thickness had no influence on pavement friction.

The MTDs of the chip seal surface were considerably higher than those of the asphalt surfaces. Pavement structure and traffic did not significantly influence the MTD value of the asphalt surfaces. Greater variations were observed for the MTDs of the chip seal surfaces. Significant loss of macro-texture in the wheelpath was found for the chip seal surface with a FDR base.

The IRI measurements indicated the test sections with an asphalt overlay had lower IRI values than the sections which did not include an asphalt layer, regardless the surface type. The sections with a CIR or FDR treated base showed better roughness than the sections that received a milling or no base treatment with the same surface type.

The friction coefficients, MTD, and IRI of each section were compared and no correlations were found between the DFT and SPT or IRI test results. An increasing trend was observed for the MTD value as the IRI values increase.

Based on the observations, the following conclusions can be drawn and are recommended to be considered in utilizing these combinations of treatments:

- The friction coefficient of chip seal in the dry condition is slightly lower than that of an asphalt surface; however, chip seals provide the same level of friction force as an asphalt surface if the pavement is wet;

- Traffic may change the micro-texture property of an asphalt surface, resulting in slight increases in the skid resistance for a dry pavement; however, it does not affect the pavement friction in the wet condition;
- Chip seals applied directly on a FDR layer may be subjected to a significant loss of macro-texture due to trafficking; and
- An asphalt overlay is the most effective treatment for greater roughness values. CIR is more effective for addressing roughness than FDR; and both methods are more effective than milling.

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CHAPTER 6. STRUCTURAL EVALUATION OF IOWA HOLDING STRATEGY TREATMENTS

Modified from a paper to be submitted to the *Road Materials and Pavement Design*

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6.1. Abstract

The increasing gap between the demand in pavement maintenance funding and the available budgetary resources is driving highway agencies to change their strategies for managing their pavement systems. A holding strategy program was initiated by the Iowa Department of Transportation to develop a different strategy of maintaining their aging pavements from the conventional mill and fill method. Ten test sections were constructed using various anticipated lower life-cycle cost rehabilitation technologies. Falling weight deflectormeter (FWD) and dynamic modulus (E*) testing were performed to assess the influences of the Iowa holding strategy treatments on pavement structure. It was found that the Iowa holding strategy treatments tend to decrease pavement structural capacity immediately after construction, especially for sections treated with cold in-place recycling (CIR) or full-depth reclamation (FDR). The structural capacities of the holding strategy sections increased in two years after construction and were comparable to those of the pavement before construction. The FDR section exhibited the greatest increase in layer modulus and structural capacity. For the purposes of a holding strategy treatment design, structural design based on FWD testing results

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is more conservative than design based on laboratory E* testing. The typically used layer coefficients for CIR and FDR (0.16 to 0.25) tend to be conservative for low-volume roads.

Key words: holding strategy, pavement rehabilitation, falling weight deflectometer, dynamic modulus, structural number.

6.2. Introduction

Highway agencies are responsible for maintaining their highway networks at an acceptable condition. With more than half a century of continuous expansion of the highway network in the United States, the scale of the current highway system has become too large to be maintained at satisfactory conditions with current budgetary resources. The state of pavement report for the California Department of Transportation (Caltrans) reported that only 23% of the needed funding is available to achieve their pavement condition targets in the next decade without increasing the current tax rate and funding policy (Caltrans 2013). The increasing gap between the demand for pavement maintenance funding and the available resources is driving highway agencies to consider changing their strategies for managing their pavement systems. The concept of "holding strategies" can be attractive to highway agencies that are seeking cost-effective solutions for roads that are approaching the end of their design life. Yu et al. introduced the concept of holding strategy and the preliminary research work on holding strategy test sections in Iowa (Yu, Jahren and Williams 2015). A holding strategy indicates pavement maintenance treatments that postpone major rehabilitation or reconstruction of a deteriorated road section with the application of lower cost alternatives of rehabilitation treatments. Traditional rehabilitation and reconstruction treatments restore pavement conditions and improve structural capacity to meet the projected traffic needs. However, the construction costs for such treatments are high; and therefore, cannot be afforded for all roads that are due for major rehabilitation or reconstruction. Preventive maintenance and minor rehabilitation treatments are relatively inexpensive. However, preventive maintenance and minor rehabilitation treatments require critical timing and pavements in good condition. The treatments for holding strategies need to be more aggressive than preventive maintenance treatments in order to be applied on severely deteriorated pavements and more economical than major rehabilitation. The holding strategy treatments used for the Iowa test sections involve various recycling technologies and thin surface treatments, such as cold in-place recycling (CIR), full depth reclamation (FDR), chip seal, and thin HMA overlay. Compared to the conventional rehabilitation method, holding strategy treatments do not significantly increase the structural capacity of a pavement system by the addition of substantive layer thicknesses. Meanwhile, the softening effect of the rejuvenation treatments (such as CIR and FDR) may result in lower modulus values of asphalt concrete (AC) layers which yield lower structural numbers (SN). Therefore, it is important that the influences of the holding strategy treatments on pavement structures be carefully evaluated to assure pavement systems adequately carry the design traffic.

6.3. Objectives and Methodology

Structural capacities of various asphalt materials and stabilized bases have been extensively studied. However, little research has been conducted for pavement structures similar to the Iowa holding strategy test sections. The conventional pavement structure usually involves a thick asphalt layer, typically more than 3 inches. Thin asphalt overlays (less than 2 inches) and surface treatments are believed to provide no structural benefits. However, a strong thin surface layer or absence of a surface course may divert the load transform pattern and stress distribution; therefore, potentially affecting the pavement's structural capacity. The primary objective of this paper is to investigate the structural capacity characteristics of the unconventional pavement structures of the Iowa holding strategy test sections. SN is used as the numerical indicator of pavement structural capacity. The effective structural numbers (SNeff) estimated using falling weight deflectometer (FWD) of the test sections before and after the construction of the holding strategy treatments are compared. Laboratory dynamic modulus tests were performed for individual pavement layers. The SN estimated from individual layer moduli is compared to the SNeff estimated from FWD testing.

6.4. Background

Tang et al. studied the seasonal change of the granular equivalency (GE) for FDR materials in Minnesota (Tang, Cao and Labuz 2012). GE is a structural capacity index used in the pavement design procedures of the Minnesota Department of Transportation (MnDOT). The GE of a pavement layer is calculated as the product of the granular equivalent factor and the layer thickness. The granular equivalent factor represents the relative stiffness of a pavement layer compared to the Minnesota Class 5 material. The typical GE factor for an AC material ranges from 2 to 2.25 (Stehr 2003). An FDR material was found to have a GE factor of 1 (Stehr 2003). The FWD tests were performed on seven FDR projects over a 3-year period. Each test section has 2 to 4 inches AC surface and 4 to 8 inches of a FDR base. The FDR base layer is either directly supported by the subgrade or 6 to 8 inches Class 5 aggregate subbase. The pavement moduli of the test sections were backcalculated from the deflection measurements. The results were used to estimate the GE factor of the FDR materials. The estimated GE values suggested the typical GE factor used in the existing pavement design method was conservative. A GE factor of 1.5 can be used for the stabilized FDR material in this study. It was also found that the stiffness of the FDR material was significantly affected by the spring thaw. Higher GE factors were observed in the summer and fall than in the spring.

Nantung et al. conducted a research on the structural number (SN) of an Indiana FDR project (Nantung, Ji and Shields 2011). Similar to the GE, SN is an indicator of pavement structural capacity that is adopted by the *AASHTO Guide for Design of Pavement Structures* (AASHTO 1993) (known as the 1993 AASHTO pavement design procedures in the following paragraphs) as well as many highway agencies. The SN is computed as the sum of the product of the layer coefficient, layer thickness, and drainage coefficient of each pavement layer. Since the layer coefficient is a stiffness index, pavement elastic modulus estimated from FWD test results can be used to estimate the layer coefficient for computation of the SN value. During the study, an 8-inch FDR base for a low-volume road was constructed in Indiana. FWD tests were conducted each year from 2007 to 2010 after the construction was completed. The analysis results showed that the asphalt stabilized FDR material had a layer coefficient that ranged from 0.16 to 0.22 shortly after the test section was placed. The layer coefficient had significantly increased by one year after the completion of construction; and showed no significant changes subsequently. The average as-constructed layer coefficient for the FDR layer was 0.21; and the average layer coefficient was 0.23 for the data measured in 2008 through 2010. With adjustments to the experimental layer coefficient to a traditional dense-graded granular base, the strength relationship was between a granular base and a FDR layer – the authors concluded a layer coefficient of 0.22 should be used for the design of a FDR treatment.

Diefenderfer and Apeagyei (2011) investigated the effective structural number (SN_{eff}) of pavements with an FDR base layer. The SN_{eff} is the "depreciated" structural number for a deteriorated pavement system. A coefficient is usually applied that considers the original layer coefficient of a new pavement in order to appropriately account for the influences of the pavement distresses on the pavement structural capacity that will likely occur late in a pavement's life. The 1993 AASHTO pavement design procedures (AASHTO 1993) introduced a method which estimates the SN_{eff} using FWD testing. Three FDR projects in Virginia were selected for this study with the test sections for these projects were treated with 8 to 10 inches of FDR and 1.5 to 3.75 inches AC. The deflection data was collected by conducting 10 to 12 FWD tests during 2 years after completion of each project. The results showed the SN_{eff} increased rapidly in the first 2 to 4 months after construction. It was observed that the SN_{eff} still increased 4 months after construction and approached a constant number after one year. The final SN_{eff} was about 15% to 45% higher than the SN_{eff} shortly after construction.

A comprehensive CIR study was conducted by Chen and Jahren (Chen, et al. 2010). Twenty four CIR projects in Iowa were evaluated through field observation and testing as well as laboratory testing. Based on a variety of testing and observation results, the authors concluded that the CIR layer behaves as a stress relief layer which exhibits better performance because it has a lower elastic modulus and higher air voids. The deflections from FWD testing were used to backcalculate the pavement layer moduli. The backcalculated CIR modulus ranged from 500 kips per square inch (ksi) to 14,500 ksi.

6.5. Iowa Holding Strategy Treatments

The Iowa Department of Transportation started a research project in 2013 to examine the feasibility of various treatments to be used as holding strategy treatments. The research project placed ten testing sections with total length of 13.6 miles on Highway IA 93. IA 93 is a lightly traveled two-lane rural highway with a flexible pavement system. The annual average daily traffic (AADT) on this road is 1040 with 3 percent heavy. A pavement condition survey conducted in July 2013 prior to construction indicated that the existing pavement was in poor condition with a pavement condition index (PCI) of 32. According to the ASTM designation, *standard practice for roads and parking lots pavement condition index surveys* (ASTM International 2011), roads with PCI values less than 55 are considered in poor condition and require maintenance to improve the roads' drivability. The predominant distress type of the existing pavement was top-down cracking. Severe raveling, potholes, and edge breaks were also observed during the pavement condition survey.

The technologies used as holding strategy treatments included various combinations of CIR, FDR, thin AC interlayers, scarification, chip seals, and thin and ultrathin AC overlays. The objective of the research is to evaluate holding strategy treatments that can be used to improve the serviceability and extend the service life of a deteriorated low-volume road. The treatment for each test section is summarized in Table 6.1. Field cores were procured in 2015 to verify the actual layer thicknesses; and the results are summarized in Table 6.2. The design of each material is shown in Table 6.3. Asphalt foaming technology was used for the CIR and FDR treatments. The aggregate gradation A, B, C, and D in Table 6.3 were designed using Superpave design procedures. The FDR material was stabilized with 2% fly ash. A modified Marshall Design method was used to design a mixture for FDR treatment. The aggregate for the single chip seal treatment was a 1/2-inch single sized limestone. The double chip seal treatment includes an additional chip seal layer made of 3/8-inch limestone on top of the bottom layer which has a 1/2-inch limestone aggregate cover. The aggregate gradation of each material is shown in Figure 6.1.

Table 6.1 Iowa Holding Strategy Treatments

Section	Base Treatment	Surface Treatment
1	1” scarification	1.5” AC overlay
2	1” scarification	1.5” AC overlay and single chip seal
3	1” scarification and 1” interlayer course	0.75” ultra-thin AC overlay
4	8” full depth reclamation	1.5” AC overlay
5	8” full depth reclamation	double chip seal
6	2.5” cold-in-place recycling	double chip seal
7	2.5” cold-in-place recycling	1.5” AC overlay
8	none	2” AC overlay
9	1” leveling and strengthening course	single chip seal
10	1” scarification	Single chip seal

Table 6.2 Actual Layer Thicknesses Measured from Field Cores

Section	Layer 1	Layer 2	Layer 3
1	1.4 inches AC	5.9 inches ExitPvt[1]	
2	1.5 inches AC+CS[2]	6.2 inches ExitPvt	
3	0.7 inches AC	1 inch AC	6.9 inches ExitPvt
4	1.7 inches AC	10 inches FDR	
5	Failed to obtain intact cores because of low material strength.		
6	3.2 inches CIR+CS	3.5 inches ExitPvt	
7	1.6 inches AC	2.7 inches CIR	2.9 inches ExitPvt
8	2.3 inches AC	6.4 inches ExitPvt	
9	1.4 inches AC+CS	8.8 inches ExitPvt	
10	4.5 inches ExitPvt+CS		

[1] ExitPvt states for existing pavement.
[2] CS states for chip seal.

Table 6.3 Material Mix Design

Material	Virgin Binder Content/Binder Application Rate	Binder Type	Aggregate Gradation
1.5" and 2" AC Overlay	5.3%	PG58-28	A
0.75" Ultra-thin AC Overlay	6.7%	PG76-34	B
1" AC Interlayer	7.4%	PG64-34	C
1" Levling and Strengthing Course	6.3%	PG58-28	D
Cold In-place Recycling	2.4%	PG52-34	
Full Depth Reclamation	2.8%	PG52-34	
Chip Seal	0.38 gallons per square yard for Treatments MC1 through 9; 0.6 gallons per square yard for Treatment MC10 (this application rate is doubled for double chip seal)	CRS-2P	

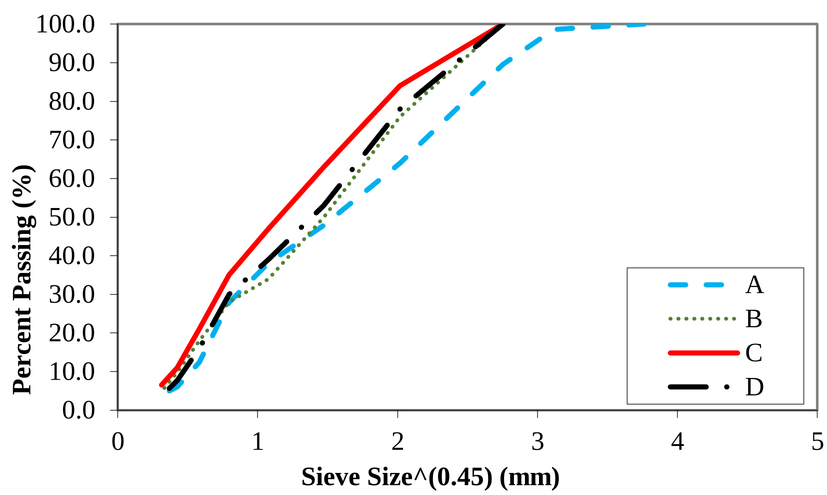


Figure 6.1 Gradations of aggregates in Iowa holding strategy treatments.

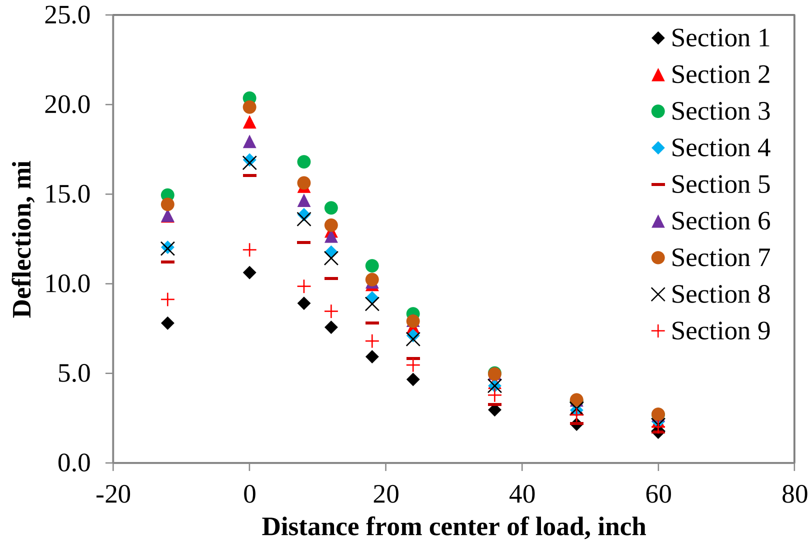
6.6. FWD Testing Results

FWD tests were performed on IA Highway 93 in October 2012, November 2013, and September 2015. The weather conditions were favorable for FWD testing which no precipitation was received in 48 hours before testing. The tests in 2012 and 2013 were carried out about every half mile along both traffic lanes. The pavement deflection data was obtained at 52 and 48 locations in 2012 and 2013, respectively. In 2015, FWD tests were conducted at a minimum of five locations in each test section. The number of FWD tests performed in each section is summarized in Table 6.4. Because FWD tests were not performed for Section 10 in 2012, influences of the treatment for Section 10 on pavement structure cannot be evaluated. The FWD tests were conducted with a trailer-towed FWD testing device. When a test was performed, an impact load was applied using a drop weight and a 12-inch diameter loading plate. Nine geophones were set at 12 inch intervals from the loading center to measure the deflection basin. The data was procured at two stress levels (12 ksi and 15 ksi). The backcalculations of the layer moduli show that the results at different stress levels are very similar. The average values of the backcalculated moduli at the two stress levels are used in this study to represent the pavement structural characteristics at each testing location.

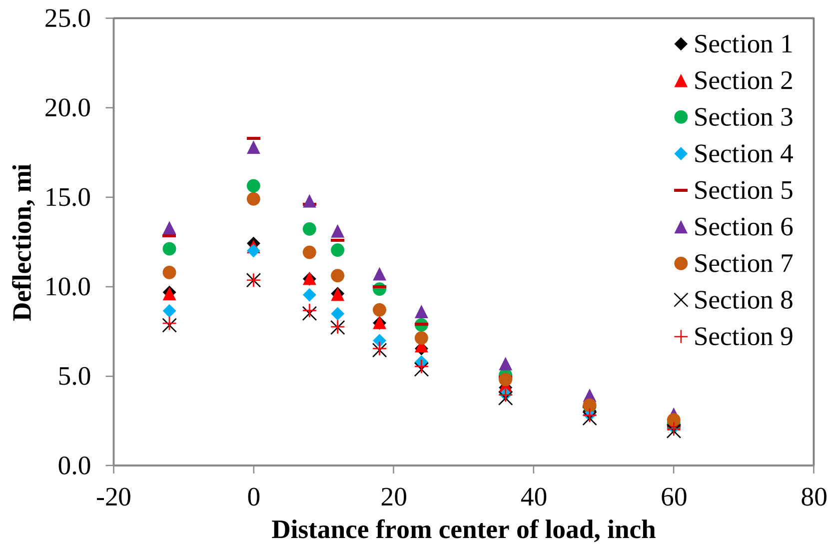
Table 6.4 Number of FWD Tests in Different Test Sections

Section	October, 2012	November, 2013	September, 2015	Treatment
1	4	3	5	1" scarification and 1.5" AC overlay
2	8	9	5	1" scarification + 1.5" AC overlay + chip seal
3	8	8	10	1" scarification + 1" interlayer + 3/4" AC overlay
4	5	5	5	8" FDR + 1.5" AC overlay
5	1	2	5	8" FDR + double chip seal
6	6	5	5	2.5" CIR + double chip seal
7	6	5	5	2.5" CIR + 1.5" AC overlay
8	6	5	5	2" AC overlay
9	8	6	5	1" AC leveling and strengthening + chip seal
10	0	1	5	1” scarification + chip seal
total	52	59	55	

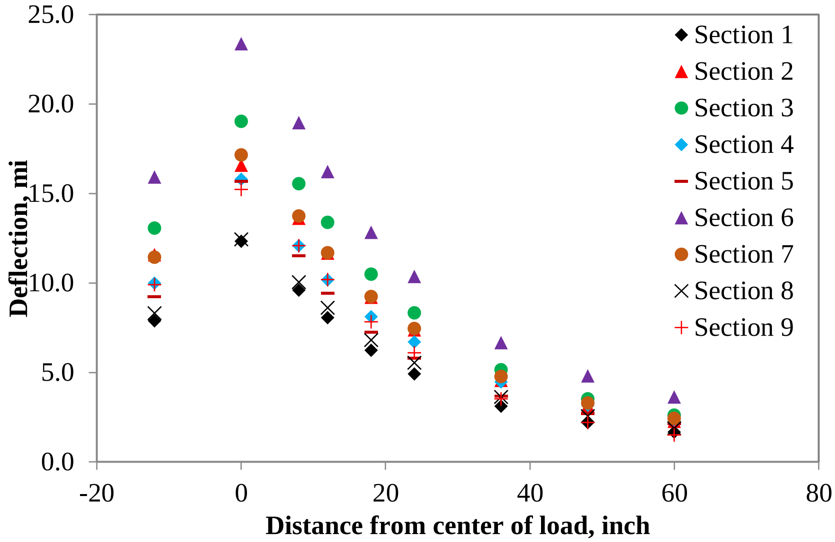
The average deflection basin at 12 kips load level of each section is shown in Figure 6.2. The FWD testing results before construction of the holding strategies show that the deflections close to the center of load at Sections 1 and 9 were lower than those of the other sections. This indicates the possibility of higher stiffness for the existing pavements in Sections 1 and 9. The deflection basins for the other sections were similar. The 2013 and 2015 results show more differences between the deflection basins of different sections. This suggests that the various holding strategy treatments may influence pavement structure capacity differently.



(a) 2012



(b) 2013



(c) 2015

Figure 6.2 FWD deflection basin at 12 kips load level.

The BAKFAA software developed by Federal Aviation Administration (FAA) for airport pavement design was applied to backcalculate the equivalent modulus of pavement layers. The backcalculated moduli were adjusted for pavement temperature using Chen's equation (Equation 1) (Chen, et al. 2000).

$$E_{Tr} = \frac{E_T}{(1.8T_r + 32)^{2.4462} \times (1.8T + 32)^{-2.4462}} \qquad \textbf{Equation 1}$$

where:

$E_{Tr} =$ modulus corrected to a reference temperature of $T_r(°C)$; and

$E_T =$ modulus determined from testing at a temperature of $T(°C)$.

The average pavement temperatures were computed from the measured pavement surface temperatures and the average air temperature of previous five days using the equation developed by Lukanen et al. (Equation 2) (Lukanen, Stubstad and Briggs 2000). A reference temperature of 25 ℃ (77 ℉) is chosen; and the backcalculated pavement moduli and standard deviation for each section are summarized in Figure 6.3.

$$T_d = 2.8 + 0.894 \times IR + (\log d - 1.5) \times [-0.54 \times IR + 0.77 \times (5day) + 3.763 \times \sin(hr - 18)] + \sin(hr - 14) \times (0.474 + 0.031 \times IR)$$

Equation 2

where:

$T_d =$ pavement temperature at depth d (°C);

$IR =$ infrared surface temperature (°C);

$d =$ depth at which material temperature is to be predicted (mm);

$5day =$ previous mean 5-day air temperature (°C); and

$hr =$ time of day in 24-hour system (radian).

The results show the overall pavement modulus decreased after construction. Causes for such decrease in pavement moduli may include that newly constructed pavement layers are softer than the existing pavement and bias in the temperature adjustment procedures. The tests were performed by experienced FWD operators using the same equipment and procedures. Any systematic biases would result from testing procedures, equipment, and personnel and expected to be small. In addition, the pavement temperatures in 2013 were much lower than those in 2012 and 2015. The lower pavement moduli in 2013 suggest that the Chen equation may underestimate the adjusted modulus at reference temperature for modulus measured at lower temperatures. However, the pavement temperatures in 2015 and 2012 were similar; yet, the pavement moduli in 2015 were considerably lower than those in 2012. Therefore, the primary contributor to the changes in pavement moduli is the introduction of softer layers into the pavement structure. The existing pavement has been aged and compacted by traffic for decades. Compared to the existing pavement, the newly constructed pavement layers are more flexible which yield lower combined modulus values. Increases in pavement moduli measured in 2015 compared with the moduli in 2013 also show that traffic and asphalt oxidization increased the moduli of newly constructed pavement layers. Such increases were statistically significant for Sections 3, 4, 6, 7, and 9.

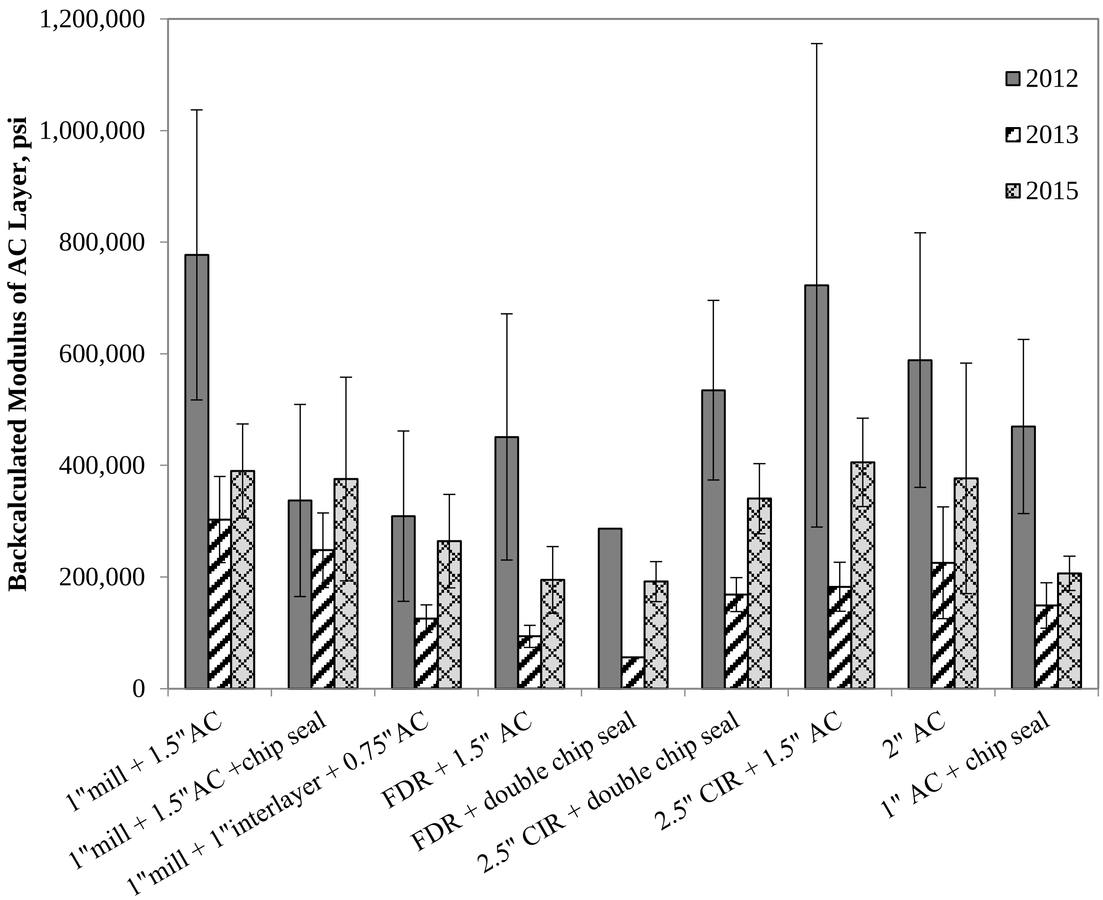


Figure 6.3 Backcalculated effective pavement modulus at 25 °C.

6.7. Laboratory Dynamic Modulus Testing Results

Dynamic modulus (E*) tests were performed on specimens prepared using field core samples. The E* testing used an indirect tensile strength (IDT) test setup with strain measurement devices (LVDT) mounted at both vertical and horizontal directions (Figure 6.4). The testing principle and procedure details were introduced by Kim et al. (2004). The pavement layers before construction of the holding strategies and the FDR layer are too thick for this type of testing setup. Core samples of these layers were prepared to make 4-inch diameter cylinder specimens and tested by following the standard procedures specified in ASTM D3497. The E* values were measured at 0.4 ℃, 17.1 ℃, and 33.8 ℃ for each specimen. The sample cores procured from Section 5 and the CIR layer of Section 7 were damaged during coring due to their low strength. Three replicate samples were tested for each pavement layer in the other sections. The E* master curves are established using the E* testing results. The master curves are used to estimate the E* values at 25 ℃ at various frequencies. The E* at 5.3 Hz can be used to simulate the pavement response under an impact load applied by FWD (Loulizi, et al. 2002). Table 6.5 summarizes the E* values estimated at 25 ℃ and 5.3Hz. The results presented in Table 6.5 indicate that the newly constructed layers have lower moduli than the existing pavement layers. This finding agrees the FWD testing results. The moduli of the CIR and FDR layers are considerably lower than those of the AC layers. Moreover, the chip seal layer does not seem to influence the modulus of the asphalt pavement.

Figure 6.4 IDT dynamic modulus testing setup.

Table 6.5 Dynamic Modulus at 25 ℃ and 5.3Hz

Section	Layer	E*, psi	Layer Description
1	1	585,558	1.4 inches AC
2	1	489,546	1.5 inches AC+CS
3	1	455,297	0.7 inches AC
4	1	521,821	1.7 inches AC
6	1	362,514	3.2 inches CIR+CS
7	1	654,330	1.6 inches AC
8	1	802,654	2.3 inches AC
9	1	419,901	1.4 inches AC+CS
3	2	309,010	1 inch AC
4	2	283,465	10 inches FDR
Existing Pavement		814,088	

6.8. Effective Structural Number and Layer Structural Coefficient

The layer coefficients for various materials involved in this study are determined from their moduli tested using FWD and laboratory dynamic modulus tests using the empirical correlation expressed by Equation 3. In this study, typical modulus value of 3,000 MPa and layer coefficient of 0.44 is assumed for the standard material.

$$a_i = a_s\left(\frac{E_i}{E_s}\right)^{1/3} \qquad \textbf{Equation 3}$$

where:

$a_i =$ structural layer coefficient of the interested pavement layer;
$a_s =$ structural layer coefficient of standard material;
$E_i =$ modulus of the interested pavement layer; and
$E_s =$ modulus of standard material.

The computed layer coefficients for individual pavement layers are shown in Table 6.6. The estimated SNeff are summarized in Table 6.7. The SNeff values estimated from E* are generally higher than the SNeff values estimated from FWD. However, the SNeff ranking for the test sections of the two methods are highly agreed with each other. Compared to the pavement structural capacity before construction of the holding strategies, the pavement structural capacity after construction was slightly lower. The CIR sections exhibited the greatest decrease in structural capacity due to the low stiffness of the CIR layers. The structural capacity of the FDR sections shortly after construction was significantly lower than the original pavement. However, the stiffness of the FDR layers increased considerably in two years; and the structural capacity two years after construction was comparable with that of the pavement before construction. Different levels of increase in structural capacity are also observed for the other sections.

Table 6.6 Layer Coefficient Estimated from E*

Section	Layer	Structural Layer Coefficient	Layer Description
1	1	0.49	1.4 inches asphalt surface course
2	1	0.46	1.5 inches asphalt surface course + chip seal
3	1	0.45	0.7 inches asphalt surface course
4	1	0.47	1.7 inches asphalt surface course
6	1	0.41	3.2 inches CIR+ double chip seal
7	1	0.50	1.6 inches asphalt surface course
8	1	0.54	2.3 inches asphalt surface course
9	1	0.43	1.4 inches asphalt surface course + chip seal
3	2	0.39	1 inch AC
4	2	0.38	10 inches FDR
Exiting Pavement		0.54	

Table 6.7 Effective Structural Number

Section	SNeff estimated from FWD results			SNeff estimated from E*(2015)	Test Section Description
	2012	2013	2015		
1	3.7	2.8	3.1	3.9	1"mill and 1.5"AC
2	2.9	2.8	3.2	4.0	1"mill + 1.5"AC + chip seal
3	3.1	2.5	3.2	4.4	1"mill + 1" interlayer + 3/4" AC
4	3.3	3.1	3.9	4.6	FDR + 1.5"AC
5	2.9	2.2	3.3	3.8	FDR + double chip seal
6	3.1	2.2	2.8	3.2	2.5" CIR + double chip seal
7	2.9	2.4	3.1	3.5	2.5"CIR + 1.5"AC
8	3.1	3.1	3.6	4.7	2"AC
9	4.0	3.1	3.5	5.4	1"AC + chip seal

6.9. Conclusion

The paper presents an in-situ NDT and laboratory evaluation of the structural capacity for the holding strategy treatments applied in Iowa. The effective structural numbers of the constructed test sections were estimated using FWD and E* tests and compared to the existing pavement before the treatments were applied. Layer coefficient of each treatment layer was also evaluated. According to the results, the following conclusions and recommendations are drawn:

- The structural capacity of the Iowa holding strategy sections was decreased after construction but recovered in two years post construction;
- Both FWD and E* tests can be used effectively for qualitative evaluation of pavement modulus. From a pavement design aspect, FWD testing is an effective tool for structural design in terms of providing reasonable and conservative information about pavement structural capacity;
- Chip seals do not seem to influence the modulus of asphalt pavement;
- The typical design structural coefficient for CIR and FDR used by many highway agencies is 0.16 to 0.25. This value is conservative for low-volume roads with sound structure;
- The FWD results indicate that aged pavement with severe surficial distresses for low-volume road can retain a high structural capacity due to high stiffness resulted from compaction by traffic loading and oxidization of asphalt. Aged pavement on a low-volume road can still have high stiffness and load carrying capacity. The AASHTO method for estimating structural capacity of deteriorated pavements based on surficial distresses may underestimate the actual structural capacity of such road; and
- Although the holding strategies do not considerably change the long-term pavement structural capacity, heavy traffic can be problematic for newly constructed roads using

holding strategies; especially roads using CIR or FDR until their structural capacity increases two years after construction.

6.10. References

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CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

7.1. Summary

The overall pavement condition of Iowa's highway network has been decreasing in the past decade, primarily caused by aging assets, increasing traffic, and lack of investment. Construction for low-volume roads that are due for rehabilitation or reconstruction is often postponed due to short falls in financial resources. Under these circumstances, it would be desirable to consider some lower-cost treatments, which may have a somewhat shorter life expectation than traditional rehabilitation or reconstruction methods for use on severely deteriorated pavement to "hold" the road at an acceptable level of performance until funding for rehabilitation or reconstruction is available. Such holding strategies may increase flexibility as budgets are allocated and improve the overall condition of highway network.

This dissertation reports an investigation that introduced the concept of holding strategies, investigated the feasibility of utilizing various lower-cost pavement maintenance treatments as holding strategies, and developed a decision tool for the selection of holding strategy treatments. This document is organized into four articles that summarize the important findings of this effort. Each article includes conclusions and recommendations that are drawn from the work reported herein. This last section presents the conclusions and recommendations resulted from individual articles; and discusses areas where future research efforts would be desirable.

7.2. Conclusions

The article presented in Chapter 3 indicates that the treatments proposed for the IA-93 test sections are financially feasible for use as holding strategies. With exception of the FDR with asphalt overlay, the cost of these treatments was 10 to 60% lower than that of a 3-inch asphalt overlay. The construction cost for FDR with an asphalt overlay treatment could be considerably lower if the required FDR thickness is smaller than the one required by the original pavement thickness in the test section for this project. The literature review shows that the individual technologies involved in the construction of the test sections are readily available and have been successfully used for pavement preservation and rehabilitation projects for decades. The documentation of the construction process also suggests that these technologies can be easily executed by local contractors. The LCCA results included in the article presented in Chapter 4 shows that, although these treatments can be economically constructed, some treatments are not recommended for use as holding strategies because their life-cycle cost-effectiveness is considerably lower than that of a conventional mill-and-fill rehabilitation method. The treatment methods that are economical to construct and cost effective for holding strategies include CIR or FDR with thin asphalt overlays or double chip seal, 2-inch asphalt overlay, thin asphalt interlayers with an ultrathin asphalt overlay, and a 1-inch leveling and strengthening course with chip seal.

The pavement condition survey results presented in Chapter 4 show that these treatments successfully corrected rutting, raveling, potholes, longitudinal cracking, and other surface defects. The predominant post-construction distress type on the test sections was reflective transverse cracking. The recycling technologies were the most effective treatments to prevent reflective cracking. The CIR and FDR sections reduced crack density by more than 95% compared to the pre-construction condition; and the cracks in the FDR sections were not reflective cracking. The CIR or FDR with a double chip seal surface exhibited performance that was comparable to that of CIR or FDR sections with a 1.5-inch asphalt overlay. However, the chip seal surface was rougher and resulted more tire noise. The 2-inch asphalt overlay and the 1-inch interlayer and

0.75-inch ultrathin asphalt overlay treatment reduced the cracking density in the existing pavement by 85% and 87%, respectively. The 1-inch milling and 1.5-inch asphalt overlay with chip seal treatment and the 1-inch leveling course with chip seal treatment had 58% and 65% crack reductions, respectively. These two treatments performed better in terms of reducing reflective cracking than the 1-inch milling and 1.5-inch asphalt overlay treatment which had 43% crack reduction value.

Surface characteristics as an important aspect of road functionality are investigated in the article presented in Chapter 5. From a safety perspective, the functionality of a chip seal is comparable to that of asphalt surface. The DFT results indicate that the dry friction coefficient of asphalt surface was slightly higher than that of chip seal. The average dry friction coefficient is 1.04 for asphalt surface and 0.88 for chip seal. The friction coefficient of asphalt surface in the wet condition was similar as the coefficient of friction for chip seal. The average coefficient of friction for all other sections in a wet condition is 0.57. However, chip seal had higher macro-texture in comparison to an asphalt surface; this can lead to an increase in tire noise and accelerate tire wear-out. The SPT results show that chip seal has a greater macro-texture than asphalt surface. The average MTD is 0.602 mm and 1.079 mm for the asphalt and chip seal surfaces, respectively. Considerably lower MTD was found in the wheelpaths for the section which received the FDR and double chip seal treatment in comparison to the other treatment involving chip seal. The low MTD is considered as the result of the loss of cover aggregate that was caused by snow removal activities. Low bond strength between the chip seal surface and the FDR base is observed by the author to be the primary contributor to this type of surface defect. Because the chip seals over FDR layer wear out faster than the chip seals in the other sections, the FDR with chip seal treatment will likely require more frequent maintenance activities, such as reapplication of chip seal, in comparison to the other treatments with a chip seal surface.

The original pavement had a poor rideability with an IRI value of over 200 inches per mile. The holding strategy treatments considerably improved the road rideability. All sections exhibited good (less than 95 inch/mile) surface roughness. The IRI values of the FDR with double chip seal treatment and 1-inch milling with chip seal treatment are higher than those of the other test sections. The ability to improve the roughness i of each individual treatment technology was also evaluated in Chapter 4. The treatments, from the most to the least effective for roughness correction, are asphalt overlay, CIR, and FDR.

Chapter 5 provides an evaluation of structural capacity of the holding strategy test sections using both field non-destructive testing and laboratory material testing. FWD tests were conducted in October 2012, November 2013, and September 2015, which provided field pavement structural assessment for pavement before the holding strategy treatments, shortly after construction, and two years after construction. The FWD results indicate that aged pavement with severe surficial distresses for low-volume road can retain a high structural capacity most likely due to the high stiffness that results from compaction by traffic as well as the oxidization of the asphalt binder. The newly constructed pavement layers can potentially decrease the average stiffness of pavement resulting in decrease in pavement structural capacity. Increased layer thickness can effectively offset this reduction in pavement structure. However, treatments that include a recycled layer, such as CIR or FDR, may considerably lower the structural capacity of pavement. An increase in pavement stiffness was observed in two years after construction for the test sections. Sections including a FDR layer exhibited the greatest improvements of stiffness. Therefore, it is recommended that although the holding strategies do not considerably change the long-term pavement structural capacity, caution should be exercised regarding the imposition of heavy traffic loadings shortly after the treatments, especially for treatments involving CIR or FDR.

The E* tests were conducted for core samples procured in 2015. The testing results show that the E* values of the newly constructed layers were considerably lower than those of the E* values for the existing pavement. The CIR and FDR layers had the lowest E* values. These findings were in agreement with the conclusions drawn from the FWD tests. However, the moduli that were documented as a result of the E* testing were higher than those that were predicted from the FWD testing. From a pavement design perspective, FWD testing is an effective tool for structural design in terms of providing reasonable and conservative information regarding pavement structural capacity.

7.3. Recommendations for Future Work

The research work presented in this dissertation documented the early-age (2-year) performance of the test sections. The LCCA was based on treatment life expectancy estimated by extrapolating of the short-term performance data and the average life expectancies of individual technologies. Future pavement condition surveys should be executed to document the long-term performance of the test sections in order to validate the estimated treatment service life predictions.

The LCCA was based on the assumption that all maintenance activities, such as crack filling and seal coat, are executed as scheduled to remedy issues regarding the rideability of the pavement and prevent damage which could accelerate road deterioration without considerably extend the service life of the road. However, these maintenance activities may influence the treatment life expectancy and affect the treatment's cost-effectiveness. This is especially true for the FDR with double chip seal treatment; frequent chip seal reapplication will likely be required to correct possible surface aggregate loss issues. Such maintenance activity can serve as preventive maintenance and, therefore, considerably improve the cost-effectiveness of this treatment. Further investigation on the effects of the maintenance activities are needed to improve the decision tool.

Future research should also be performed to establish trigger values for holding strategies. The trigger values can be based on pavement condition indicators, such as IRI, pavement condition index (PCI), ride quality index (RQI), and possibly a combination of measurements of individual types of pavement distresses. With such trigger values, holding strategies can be incorporated into the current pavement management program.