Holding Strategies for Low-Volume State Routes – Phase I

Final Report
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CONSTRUCTION MANAGEMENT AND TECHNOLOGY

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16. Abstract

The overall pavement condition of Iowa's highway network has been deteriorating in the past decade due to aging facilities, increasing traffic, and lack of financial resources. Due to insufficient funding, rehabilitation or reconstruction is delayed for low-volume roads in need of repairs. Some lower cost treatments, which may have shorter life expectancies in comparison to traditional rehabilitation or reconstruction methods, have been considered inappropriate for use with severely deteriorated pavements. However, these treatments could be applied to "hold" these pavements in an acceptable condition until funding for rehabilitation or reconstruction is available. Such holding strategies would likely increase the flexibility in allocating funds and improve the overall condition of the highway network in Iowa.

In order to develop treatments that can be used to fulfill the goal of a holding strategy, nine test sections were constructed on a 13 mile low-volume asphalt road segment in 2013. Proposed holding strategy treatments using various combinations of thin and ultrathin asphalt overlays, in-place recycling technologies, and chip seals were applied to remedy the poor surface conditions of the pavements. A series of pavement condition surveys, in situ and laboratory material tests, and surface characterizations were performed to evaluate the structural and functional performance of the test sections.

Based on the performance of the test sections, the life expectancies of the various treatments were estimated and lifecycle costs were analyzed. The lifecycle cost analysis results indicate that 8 of the 10 proposed treatments can be used as candidate holding strategy treatments to address conditions that are like the test. The other treatments had lower cost-effectiveness for these test sections compared to traditional pavement rehabilitation methods. However, they could be more cost effective in circumstances that are better matched to their advantages.

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HOLDING STRATEGIES FOR LOW-VOLUME STATE ROUTES

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EXECUTIVE SUMMARY

The Iowa Department of Transportation (DOT) sponsored a research project that involved constructing nine test sections on a 13-mile low-volume asphalt road in 2013. The aim of this research project was to develop holding strategies as a potential solution to a critical challenge facing the state.

The overall pavement condition of Iowa's highway network has been deteriorating in the past decade, primarily due to aging facilities, increasing traffic, and lack of financial resources. Maintenance efforts for low-volume roads that are due for rehabilitation or reconstruction are sometimes postponed due to insufficient funding.

Some lower cost treatments, which may have shorter life expectancies in comparison to traditional rehabilitation or reconstruction methods, have been considered inappropriate for use with severely deteriorated pavement. However, these treatments could be applied to these pavements to "hold" them in an acceptable condition until funding for rehabilitation or reconstruction is available. Such holding strategies would likely increase the flexibility in allocating funds and improve the overall condition of the highway network in Iowa.

The holding strategy treatments described in this report include various combinations of thin and ultrathin asphalt overlays, in-place recycling technologies, and chip seals. This report documents the construction and six-year performance of the test sections. The performance was evaluated by pavement condition surveys, in situ nondestructive structural tests, laboratory material tests, and various surface characterizations.

The pavement condition surveys indicated that longitudinal cracking, rutting, raveling, edge breaks, and roughness in the existing pavements have been successfully corrected by the holding strategy treatments. The predominant distress type found in the test sections was reflective transverse cracking that developed from the cracking patterns that remained in the remaining layers of the original pavement sections.

Recycling technologies were the most effective treatments in preventing reflective cracking. A thin interlayer with an ultrathin asphalt overlay method and a two-inch asphalt overlay exhibited satisfactory performance against reflective transverse cracking. The sections that were scarified and covered with thin asphalt overlays developed more transverse cracking in comparison to the other test sections.

Applying chip seals over various treatments improved their ability to prevent reflective cracking. Loss of cover aggregate caused by snow plowing operations and traffic was observed with chip seals that were applied to rough surfaces, such as scarified pavements or full-depth reclamation (FDR) layers. The surface characteristics of the asphalt pavements and chip seals were evaluated using a dynamic friction tester (DFT) and the sand patch test (SPT). From a safety perspective, the functionality of chip seals is comparable to that of an asphalt surface. However, chip seals have higher macro-texture in comparison to an asphalt surface, which can lead to an increased noise level and faster tire wear. Some localized distress at bridge approaches was also observed where chip seals were the final surface over cold in-place recycling (CIR) and FDR

The influences of the holding strategy treatments on the test sections' structural capacity were investigated using a falling weight deflectometer (FWD) test and a dynamic modulus (E*) test. The structural evaluation indicated that the holding strategy treatments tend to temporarily decrease pavement structural capacity. The test sections regained their stiffness within two years after construction. The treatments involving a CIR or FDR layer exhibited the greatest decrease in pavement structural capacity shortly after construction, and pavements recovered to the original stiffness level that existed before construction within two years.

The lifecycle costs of the various holding strategy treatments were estimated and compared to those of a traditional 3-in. asphalt concrete overlay rehabilitation method. The lifecycle cost analysis (LCCA) results indicated that the equivalent annual cost (EAC) of the FDR and CIR with a double chip seal surface was projected to be higher than that of the 3-in. overlay strategy. The lifecycle costs of the other holding strategies are projected to be less than or equal to the lifecycle cost of the 3-in. overlay method.

INTRODUCTION

The Iowa Department of Transportation (DOT) is facing a challenge in maintaining the pavement condition of its highway network. The available financial resources for pavement rehabilitation have grown slowly in comparison to the deterioration rate of the highway network.

From 1999 through 2006, the number of non-Interstate primary highways in poor condition increased by more than 60% (Iowa DOT 2008). In 2013, the highways that received a good rating constituted less than 47% of the Iowa primary roadway system (ASCE Iowa Section 2015). The American Society of Civil Engineers (ASCE) graded the overall condition of Iowa public roads as C- (ASCE Iowa Section 2015). A C- grade indicates that the condition is marginally adequate for current use.

Increased investment is needed to maintain the current condition of the 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 for roads that carry a higher volume of traffic. Compared to roads with higher traffic volumes, low-volume roads usually have lower funding priorities.

The Moving Ahead for Progress in the 21st Century (MAP-21) Act provides federal funding to state highway agencies to improve the conditions of their transportation infrastructure. MAP-21 established performance targets for the national highway system (NHS), which includes Interstate highways and primary roads (FHWA 2014). No performance targets were set for secondary and local roads, which usually carry low traffic volumes.

In the past, the pavement maintenance strategy used by many highway agencies has been a worst-first strategy. The worst-first strategy describes investing financial resources in major rehabilitation or reconstruction projects for roads that are in poor or very poor condition. This strategy usually involves high costs for thick asphalt or concrete overlays or base material improvements followed by the placement of completely new pavement sections.

Pavement engineers and researchers have recently realized that considerable savings can be obtained by adopting a pavement preservation approach. A pavement preservation strategy involves applying preventive maintenance treatments, which usually have considerably lower costs in comparison to major rehabilitation and reconstruction projects, to pavements that are still in good condition and following a planned schedule.

The treatments used for pavement preservation are usually thin surface treatments (TSTs) such as chip seals, slurry seals, microsurfacing, and thin overlays, which prolong the service life of the surface or near-surface layer without adding significant structural capacity to the pavement structure.

Many states, like California and Michigan, have incorporated pavement preservation and the worst-first strategy into a mix-of-fixes strategy, in which the condition of each road is evaluated and maintenance treatments can be applied appropriately. The mix-of-fixes strategy includes three levels of treatments: reconstruction, rehabilitation, and preventive maintenance (Galehouse 2003).

Reconstruction and rehabilitation are undertaken on roads with severe base and subgrade damage and insufficient structural capacity. Preventive maintenance is applied to roads with minor distresses that 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 preventive maintenance requires optimum timing. Premature or delayed maintenance activities 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 nondestructive testing (NDT) (Hicks et al. 2000, Smith 2001). However, highway agencies sometimes fail to apply appropriate treatments when a trigger value is reached for a particular road because of insufficient financial resources.

It is desirable to extend the time window by maintaining the road conditions using holding strategies. A holding strategy is defined as a pavement management strategy that postpones major rehabilitation or reconstruction of a deteriorated road section using treatments that are more aggressive than preventive maintenance treatments and that have lower costs and, most likely, shorter service lives compared to rehabilitation strategies (Yu et al. 2015).

Holding strategies could likely provide highway agencies flexibility in funding allocations and help effect a transition 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.

A complete holding strategy process includes five steps: project recognition, treatment selection, design and construction, maintenance, and late-life reactive maintenance (Yu et al. 2015). This process starts with a network-level analysis of the maintenance needs for the highway system.

Roads are sorted into categories according to the required type of maintenance. Then, maintenance funding can be assigned to roads with high priorities. For roads that are due for major rehabilitation or reconstruction but will not receive adequate funding, the use of holding strategies can be considered.

The treatment selection step includes a project-level analysis of holding strategy alternatives for a particular project. An appropriate holding strategy treatment is selected based on the intended holding time, traffic and environmental conditions, types of distresses on the existing pavement, and cost.

The design and construction step involves pre-construction testing to verify the site conditions, proper design and construction activities, and the execution of quality control and assurance measures.

The maintenance step includes a schedule of routine maintenance such as seal coating, crack filling, patching, and so forth.

Late-life reactive maintenance requires the design of appropriate treatments to be applied at the end of the designed holding time. Depending on funding availability, the late-life reactive maintenance can be a major rehabilitation, a reconstruction, or another holding strategy.

In an effort to develop detailed approaches for the five steps of the holding strategy, the Iowa DOT constructed a test road on IA 93 in 2013. The original road was a two-lane full-depth asphalt highway with an average annual daily traffic (AADT) of 1,040 vehicles per day (vpd). The existing pavement had a rough surface and was suffering from various surficial distresses. Ten test sections were constructed using various treatments proposed as holding strategies. The treatments included combinations of thin asphalt layers, in-place recycling technologies, and thin surface treatments. Table 1 summarizes the treatments applied to the test sections on IA 93.

Table 1. IA 93 holding strategy treatments

Section Number	Base Treatment	Surface Treatment	Section Length (miles)
1	1 in. scarification	1.5 in. HMA overlay	1.3
2	1 in. scarification	1.5 in. HMA overlay and single chip seal	2.0
3	1 in. scarification and 1 in. interlayer course	0.75 in. ultrathin HMA overlay	2.2
4	8 in. full-depth reclamation	1.5 in. HMA overlay	1.0
5	8 in. full-depth reclamation	double chip seal	0.4
6	2.5 in. cold-in-place recycling	double chip seal	1.4
7	2.5 in. cold-in-place recycling	1.5 in. HMA overlay	1.6
8	none	2 in. HMA overlay	1.4
9	1 in. leveling and strengthening course	single chip seal	1.9
10	1 in. scarification	single chip seal	0.3

HMA = hot-mix asphalt

LITERATURE REVIEW

A literature search was conducted to ascertain if approaches similar to the holding strategy approach have been used elsewhere. The researchers found that although some of the individual treatments have been used successfully and are widely accepted elsewhere, the combinations of treatments and the approach proposed in this study have found limited previous use.

Thin Asphalt Layer

Thin asphalt layers include thin and ultrathin asphalt overlays and thin asphalt interlayers. A thin asphalt overlay usually refers to an asphalt surface course with a layer thickness of 1.5 in. or less (Caltrans 2008, Dave 2011, Huddleston 2009, Sauber 2009). The California Department of Transportation (Caltrans) defines the layer thickness of a thin asphalt overlay as less than 1.25 in. (Caltrans 2008). Ultrathin asphalt overlays usually have a lift thickness of less than 1 in. (Caltrans 2008, Dave 2011, Huddleston 2009, Sauber 2009).

A thin asphalt overlay is 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 2009).

Newcomb (2009) reviewed various studies on the performance of thin overlays. These studies were conducted during various years from 1994 through 2009 and included a wide range of locations, including various locations in the US, Austria, and Canada. The results showed that the life expectancy of thin asphalt overlays ranges from 5 to 16 years. Lower lifecycle costs were also recognized for thin asphalt overlays compared to other preventive maintenance treatments (Chou et al. 2008).

A commonly used type of ultrathin overlay is an ultrathin bonded wearing course, also known as an open-graded friction course (OGFC). An OGFC uses high-quality gap-graded aggregate and a polymer-modified asphalt binder. The typical lift thickness is between 15 mm (0.6 in.) and 20 mm (0.8 in.) (Gilbert et al. 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.

An OGFC improves the functionality of roads that are losing skid resistance and for which roughness is an issue; it also provides a waterproofing layer that protects the underlying pavement structure from water damage. The life expectancy of an OGFC is between 8 and 12 years (Gilbert et al. 2004).

When 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 wheelpaths, rutting in the base layer, and alligator cracking, should be treated with more aggressive treatments than thin and ultrathin overlays. Newcomb (2009) recommends that thin asphalt overlays should be used for pavements with distress that extends for less than 10% of the project. For an OGFC, the candidate roads should have a remaining life of 6 to 8 years (Gilbert et al. 2004).

If the treatments are used as holding strategies, these criteria will not be met. The researchers did not find any quantitative research involving thin and ultrathin overlays being employed on severely deteriorated pavements.

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 in.) to 30 mm (1.2 in.) (Montestruque et al. 2012). The asphalt mixture consists of fine aggregates (usually less than 3/8 in.) and a high percentage of polymer-modified asphalt (up to 7.5%). The purpose of such a mix design is to create a strong and highly flexible layer that absorbs part of the crack wall movement and reduces shear and tensile stresses at the interface of the layers above the 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 compared to conventional asphalt overlays (Newcomb 2009). The 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 in. Cored samples are also difficult to obtain. Special attention should be paid to pavement temperature during compaction. The mat temperature decreases faster for thin layers than for thicker asphalt layers. It is important to maintain a fast and consistent operation of compaction and perform construction during favorable weather conditions.

Thin Surface Treatment

A TST is also known as a light surface treatment (LST) or a bituminous surface treatment (BST) (Dayamba et al. 2015). A TST is a thin layer of liquid asphalt and aggregate with an application thickness less than 1/2 in. (Li et al. 2007). TSTs are usually used for pavement preservation to seal minor cracks, correct surface defects, improve road functionalities, and provide a waterproofing layer that prolongs the road service life. TSTs are considered to have no structural capacity during pavement design (Peshkin et al. 2004). In some places, TSTs have been used on aggregate-surfaced roads to provide dust control and functional improvements, as well as reduce maintenance requirements (Dayamba et al. 2015).

A variety of treatments are considered to be TSTs, including chip seals, slurry seals, cape seals, sand seals, Otta seals, etc.

A chip seal is constructed by applying an asphalt emulsion on the road surface and covering it with single-sized aggregate. Rollers are used to embed aggregate particles into the asphalt layer 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 employed as a stress absorbing interlayer (Caltrans 2008). In such a 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 a smaller particle size than that of the lower layer. The life expectancy of a chip seal ranges from 3 to 5 years (Nantung et al. 2011, Maher et al. 2005).

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

A sand seal is similar to a chip seal and is constructed by applying an asphalt emulsion film that 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 the aggregate, a sand seal has a relatively smooth surface texture. The treatment is recommended for use in areas where a high-quality aggregate source is not available in the vicinity (Greening et al. 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 a chip seal. Pneumatic rollers are used to embed the aggregate into the binder layer. Otta seal applications can often use relatively low-quality locally available aggregate and can sometimes provide cost savings (Johnson and Pantelis 2008). The gradation of the 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 than a chip seal in retarding the aging of the asphalt in the underlying layers (Overby and Pinard 2013).

Liu et al. (2010) conducted a study on various TSTs used for pavement preservation in Kansas. The definition of TST in Liu et al.'s (2010) 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 that 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 than that of TSTs on lower-volume roads. In comparison to thin asphalt overlays, chip seals appear to have a lower service life. The average service life of chip seal on non-Interstate highways is five years. Slurry seals on Interstate highways exhibit higher service lives than chip seals, while the service lives of slurry seals and chip seals on non-Interstate highways are comparable. Chip seals have the lowest annual cost among all the treatments compared. The equivalent uniform annual cost (EUAC) of a chip seal is less than half of the EUAC of a slurry seal and less than 20% of the EUAC of a 3 in. overlay.

A study conducted by Wang et al. (2013) quantified the costs-benefits of various types of TSTs in Pennsylvania, including crack sealing, chip seals, microsurfacing, thin overlays, and NovaChip (similar to OGFC). The study compared the EUAC of each TST with a do-nothing alternative using Pennsylvania Pavement Management Information System (PMIS) data from 1998 to 2008. It was found that crack sealing has the highest benefit-cost ratio, while NovaChip has the lowest benefit-cost ratio. The EUAC of the TSTs varies with the condition of the existing pavement when treatments were applied. In order to quantify the effects of the existing pavement condition on the service life extensions provided by the TSTs, performance models were established using the Pennsylvania overall pavement index (OPI). The results indicate that the pavement life benefits of TSTs start to decrease significantly when the OPI of the existing pavement decreases below a trigger value. The trigger values for chip seals and microsurfacing on highways with an average daily traffic (ADT) of less than 2,000 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 seals 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 condition. Few case studies were found for TSTs used on deteriorated pavements as a rehabilitation treatment.

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 to be 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 production of new materials and hauling, handling, and storage. The required hours of labor and time for a rehabilitation project are also decreased.

Hot In-Place Recycling

HIR uses a heating unit to soften the existing pavement by heating it to between 110°C and 150°C (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 on roads with a sound structure. The treatment depth is typically 3/4 to 1 in. and does not exceed 2 in. (Finn 1980). The efficiency of the heating unit is significantly affected by surface treatments, such as chip seals (Pierce 1996). The removal of surface treatments may be required before HIR is performed. Because the existing pavements of IA 93 and many other roads in Iowa are maintained with surface treatments and have cracking depths greater than 1 in., HIR may have less application as a holding strategy treatment than other in-place recycling technologies.

Cold In-Place Recycling

CIR is an in-place recycling technology that pulverizes, adds recycling agents to, mixes, spreads, and compacts 2 to 5 in. 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 (AC) 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 in. of the existing pavement layer in order to support the load from the construction equipment that performs the recycling (FHWA 2011a). Local experience in Iowa is that retaining 3 in. of existing pavement is better, and checking for adequate subgrade support before construction is a preferred practice. This process is also known as 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 the 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 the RAP to the desired particle size. The multiple-unit trains have a higher production rate and consistency than single-unit machines (Caltrans 2008). However, multiple-unit trains have difficulty in negotiating turns and corners, which are more frequently encountered in urban areas than 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 in. 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 of about 45% to 75% have been realized when using CIR as an alternative to the conventional overlay method (FHWA 2011a, Jahren et al. 1998). The life expectancy of CIR ranges from 7 to more than 20 years (FHWA 2011a, 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 lose moisture and gain strength. A favorable working environment is critical to the success of construction. Many state agencies have specified weather restrictions for CIR construction. Typically, an ambient temperature above 15°C (59°F) and dry weather conditions are desirable. During construction, the bearing strength is temporarily decreased. Weak spots may fail to support the construction equipment and cause failure in the base and subgrade. Such failures can be repaired with an asphalt overlay or a replacement of the weak materials at the locations of the failures. Asphalt stripping was problematic for CIR sections in Kansas (FHWA 2011a), and lime slurry was used to mitigate the stripping issues and improve the overall performance.

The structural layer coefficients of CIR are usually smaller than those of new hot-mix asphalt (HMA). The results f the Association of State Highway and Transportation Officials (AASHTO) road test suggest that an appropriate layer coefficient for CIR would range from 0.3 to 0.35 (AASHTO 1986). Some state agencies use layer coefficients ranging from 0.25 to 0.28 (FHWA 2011a). 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).

Full-Depth Reclamation

FDR, which is also known as full-depth cold recycling, is a process that involves pulverization of the entire asphalt pavement layer and a portion of the underlying aggregate base. The recycled materials are then mixed and placed as a base layer. The treatment depth is typically 6 to 9 in. and seldom greater than 12 in. 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 the RAP material gradation, plasticity index, fines content, and the extent to which the asphalt binder in the RAP material has aged. Virgin aggregate can be added if there is a need for additional structural capacity or lane widening.

FDR can be performed using some of the same equipment and processes as CIR. The single-unit machine and the two-pass operation are more popular than multiple-unit trains and the single-pass operation (Thompson et al. 2009). The primary reason is that the single-unit machine performs pulverization, sizing, mixing, and placing around the rotary drum without requiring the transport of the 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 the 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 a base layer with the same thickness, the use of FDR can result in savings of 90% with regard to new materials and 80% with regard to diesel fuel (PCA 2005). A lifecycle cost analysis (LCCA) conducted by Diefenderfer and Apeagyei (2011) indicates that pavement maintenance strategies involving SFDR were about 16% less costly than conventional mill and fill strategies during a 50 year analysis period. FDR is usually used in combination with an AC overlay. The treatment provides a service life that is comparable to that of a reconstruction project.

The factors to be considered in FDR construction, such as the need for curing time and adequate subgrade support, should be the same as those used for CIR construction. The required minimum temperature for SFDR using chemical stabilizers is typically 4°C (39.2°F) to 7°C (44.6°F) (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 the SFDR layer be less than 2.5%, or 50% of the optimum moisture content determined from the proctor test

(Illinois DOT 2012). Many state agencies also establish rolling criteria to ensure that adequate compaction is achieved. The Iowa DOT requires the field density at 75% of the FDR mat depth to be higher than 92% of the laboratory density for secondary roads and requires the field density at the 2 in. depth to be higher than 97% of the density at 75% of the mat depth (Iowa DOT 2012).

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

Nevada Rehabilitation Alternatives Research

Test sections in Nevada were constructed on five low-volume roads using CIR, SFDR, cold-mix asphalt, and various surface treatments. The roads are two-lane rural highways that carry an ADT of 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 in. overlay and chip seal surface, and nine CIR sections with a chip seal surface. The SFDR sections with a 1.5 in. 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 in. 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 a stabilization agent. 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 using roughness measurements, condition surveys, and falling weight deflectometer (FWD) tests. The sections were monitored for three to four years. The results show 17% to 62% performance improvements for the SFDR treatments and 2% to 43% performance improvements for 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 for the SFDR sections. A LCCA was also performed for the CIR treatments using a 20 year analysis period and a 4% discount rate. The cost analysis results indicated that an average cost savings of $100,000 per centerline mile was realized by using CIR and chip seal for rehabilitation during a 20 year lifecycle.

OBJECTIVES AND METHODOLOGY

The main objective of this study was to evaluate the performance of the various holding strategy test sections and their cost-effectiveness. This report summarizes the two-year performance observation of the treatment sections. The performance of the test sections was evaluated through a series of pavement condition surveys, structural evaluation, and surface characteristics tests. A LCCA was conducted to quantify the cost-effectiveness of each treatment method. Based on the treatment performance and LCCA results, recommendations were made to assist in the decision process for selecting appropriate holding strategies.

Pavement Condition Survey

Pavement condition surveys were performed by following the Federal Highway Administration (FHWA) pavement distress identification manual (Miller and Bellinger 2003). Surficial distresses, such as transverse and longitudinal cracking, fatigue cracking, raveling, and rutting, were evaluated. The majority of the evaluations involved visual inspection. The severity level of the cracks was determined using a caliper. Rutting was measured in the wheelpaths using a deflectometer equipped with a 4 ft straight edge and a vertical ruler (Figure 1).

Figure 1. Deflectometer for measuring rutting

Three survey sections were randomly assigned to each test section for most test sections. The standard survey section was 500 ft long. All of the survey sections were located in the eastbound lane. The location of the survey sections also depended on the terrain and geometry of the road. Road segments that may cause changes in traffic speed or concentration of runoff water can result in biased observations or safety concerns. Such road segments may include bridges, intersections, vertical curves at the bottom of a sag, sharp horizontal curves, or treatment transition areas. These segments of the road were exempted from the pavement condition survey. Because of these limitations, some test sections failed to provide three survey sections with the orthodox section length. Table 2 summarizes the length and location of each survey section.

Table 2. Assigned survey sections

Test Section	Survey Section #	Section Length (ft)	Location (miles)
1	1-1	500	0.6
	1-2	500	0.8
	1-3	500	1.0
2	2-1	500	1.4
	2-2	500	1.7
	2-3	500	2.3
3	3-1	500	3.5
	3-2	500	4.4
	3-3	500	5.1
4	4-1	500	5.6
	4-2	500	5.9
	4-3	500	6.2
5	5	715	6.6
6	6-1	500	7.0
	6-2	500	7.4
	6-3	500	7.9
7	7-1	500	8.5
	7-2	500	9.0
	7-3	500	9.5
8	8-1	500	10.0
	8-2	500	10.5
	8-3	500	10.9
9	9-1	750	11.6
	9-2	750	12.5
10	10	450	13.8

Location = distance from the beginning of the project at the intersection of Y Avenue and IA 93 in Sumner

Structural Evaluation

The pavement structural capacity was evaluated using the FWD and laboratory dynamic modulus (E*) tests. The objective of the structural evaluation was to understand the influences of the holding strategy treatments on pavement structural capacity.

FWD Test

The FWD tests were performed by Iowa DOT pavement management professionals in October 2012, November 2013, and September 2015. The tests were carried out about every half-mile along both traffic lanes in 2012 and 2013. The pavement deflection data were obtained at 52 and 48 locations in 2012 and 2013, respectively. In 2015, the FWD tests were conducted at a minimum of five locations in each test section. The number of the FWD tests performed in each section is summarized in Table 3.

Table 3. Number of FWD tests in different test sections

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

AC = asphalt concrete
CIR = cold in-place recycling
FDR = full-depth reclamation

During each test, the FWD applied impact loads at two stress levels, 12 ksi and 15 ksi, onto a 5.9 in. diameter loading plate. Nine geophones placed every 12 in. from the loading center were used to capture the deflections of the pavement surface and establish deflection basins. The results were interpreted using the BAKFAA software developed by Federal Aviation Administration (FAA) to estimate the modulus of all pavement layers.

E Test*

The E* tests were performed on specimens prepared from field core samples. The E* tests used an indirect tensile strength (IDT) test setup with strain measurement devices (linear variable differential transformers [LVDTs]) mounted in the vertical and horizontal directions (Figure 2).

Figure 2. IDT dynamic modulus testing setup

The testing principles and procedure details were provided by Kim et al. (2004). The existing pavement layers before construction of the holding strategies and the FDR layers were too thick for this type of testing setup. Therefore, the core samples of these layers were shaped to make 4 in. diameter cylinder specimens and were tested by following the standard procedures specified in ASTM D3497. The E* values were measured at 0.4°C, 17.1°C, and 33.8°C for each specimen. The sample cores procured from Section 5 and the CIR layer of Section 7 were destroyed during coring due to their low strengths. Three replications were conducted for each pavement layer in the other sections.

Surface Characterization

The surface characteristics were evaluated using a dynamic friction tester (DFT), the sand patch test (SPT), and a smartphone-based international roughness index (IRI) measuring system. These surface characteristics influence road functional performance, such as friction, noise generation, tire wear, and fuel economy, which is related to passengers' safety, level of comfort, and user costs.

DFT

The DFT measures the friction coefficient of a pavement surface using a portable measuring system. The device has three rubber sliders attached to a circular plate. During testing, the circular plate is driven to rotate by a motor and causes the rubber sliders to move relative to the pavement surface. The torque required to maintain a particular rotating speed is measured and converted to the friction force. The friction coefficient can then be calculated through the relation between the weight of the equipment that is carried by the rubber sliders and the friction force. The DFT can measure the friction coefficient in both dry and wet surface conditions. The friction coefficient in the wet condition is more critical than that of the dry condition in the matter of safety. The test procedures were performed according to ASTM E1911.

SPT

The SPT measures the macro-texture of a pavement surface using sand particles. Macro-texture affects the friction during high-speed skidding or when water is present. Hysteresis caused by tire deformation due to the pavement macro-texture accounts for more than 95% of the overall friction at speeds higher than 65 mph (PIARC 1987). A high macro-texture also facilitates drainage and reduces hydroplaning, which occurs when a water film develops at the pavement-tire interface and causes a considerable decrease in friction. During testing, a cylinder is used to measure an amount of sand with a 45 ml uncompacted volume. The sand is poured onto the pavement surface and spread to form a sand circle. The average diameter of the circle is then measured. The mean texture depth (MTD) can be calculated by knowing the volume of sand and dividing by the area of the sand circle.

IRI

IRI is an important roughness indicator that is used by many highway organizations and agencies. It is usually measured from longitudinal road profiles using a quarter-car vehicle mathematical model. The conventional IRI test is costly and time consuming. A smartphone-based application, Roadroid, recently developed by Swedish scientists Hans Jones and Lars Forslof, was used instead of the conventional IRI profiler to estimate the roughness of the test sections. 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). During testing, a smartphone was attached to the windshield of a mid-size car (2014 Ford Taurus) using a car mount for a mobile phone. The application is able to calibrate the vibration readings for vehicle type and speed. However, in order to achieve a higher consistency of data, the vehicle speed was maintained at 50 miles per hour while the smartphone recorded the readings.

TEST SECTION CONSTRUCTION

The field test sections were constructed on highway IA 93 between Sumner and Fayette in Iowa (see Figure 3).

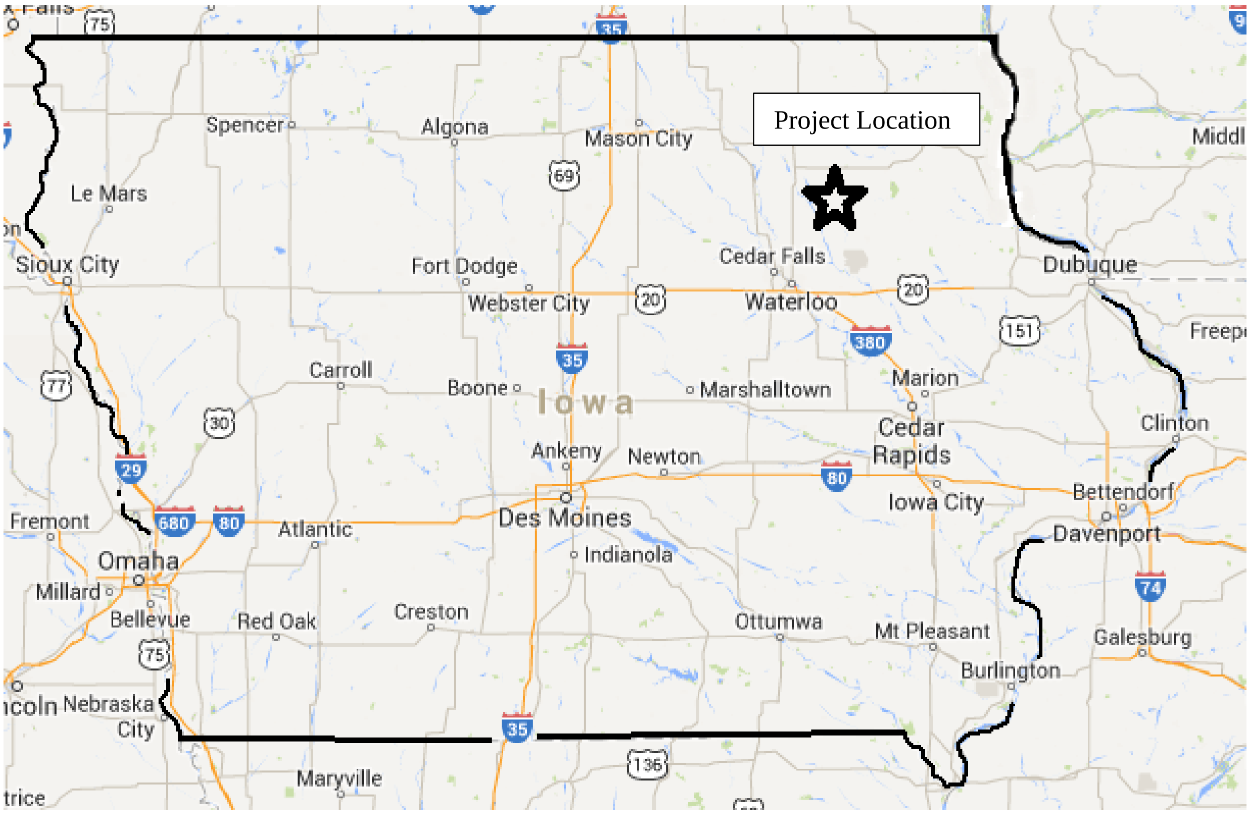

Image © Google 2014

Figure 3. Project location in Iowa

IA 93 is a state highway on the Iowa primary road system. The road connects Fayette, Sumner, Tripoli, and US 63. The primary industry in this region is agriculture. The AADT in 2011 was 1,040 vpd, with 8% trucks. The design hour volume (dhv) and equivalent single-axle loads (ESALs) are 10 vehicles per hour (vph) and 475,960, respectively. The road is subjected to heavy oversized farm traffic during harvest seasons. A construction project was performed on the east segment of IA 93 (see Figure 4) and is the subject of this report.

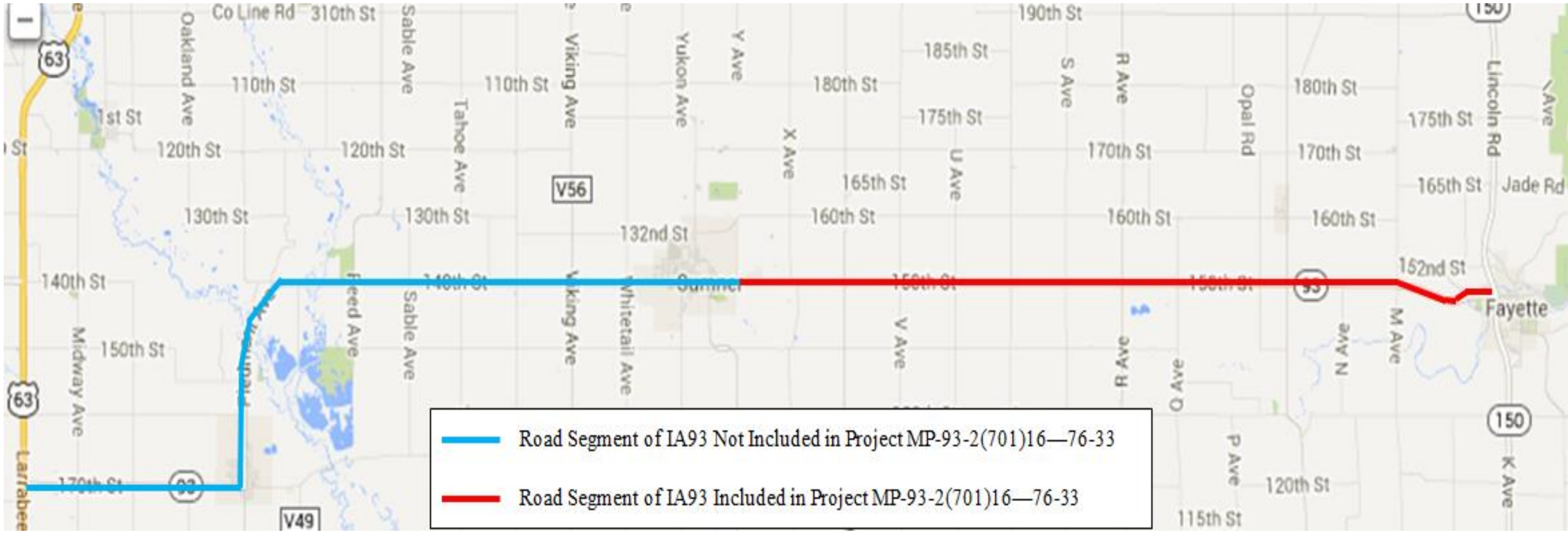

Image © Google 2014

Figure 4. Project location on IA 93 from Y Avenue in Sumner east to IA 150 in Fayette

The length of the project is 13.6 miles. About one quarter-mile of the road section in the Municipality of Fayette is operated as an urban street with two traffic lanes and two parking lanes, while the rest of the project is operated as a rural two-lane highway. The lane width is 12 ft for the rural sections, and the urban section has 12 ft traffic lanes with 6 ft parking lanes.

Pavement Management

The road section was first built in 1951. The original pavement structure consists of a 6 in. thick granular base layer and a 0.75 in. thick asphalt (seal coat) surface. The road was resurfaced in 1971 with a 2.5 in. type "B" asphalt concrete binder course and a 2 in. type "B" asphalt concrete surface course. The more recent surface maintenance treatments were two seal coat layers placed in 1990 and 2006, respectively. Field coring for this research project showed that the average thickness for the existing asphalt pavement is 7 in. Some areas have pavement thicknesses exceeding 8 in. It is possible that an asphalt overlay or some asphalt maintenance patching was placed in the road and not entered into the pavement maintenance records. The structure of the existing pavement system as presented in the pavement management records is sketched in Figure 5.

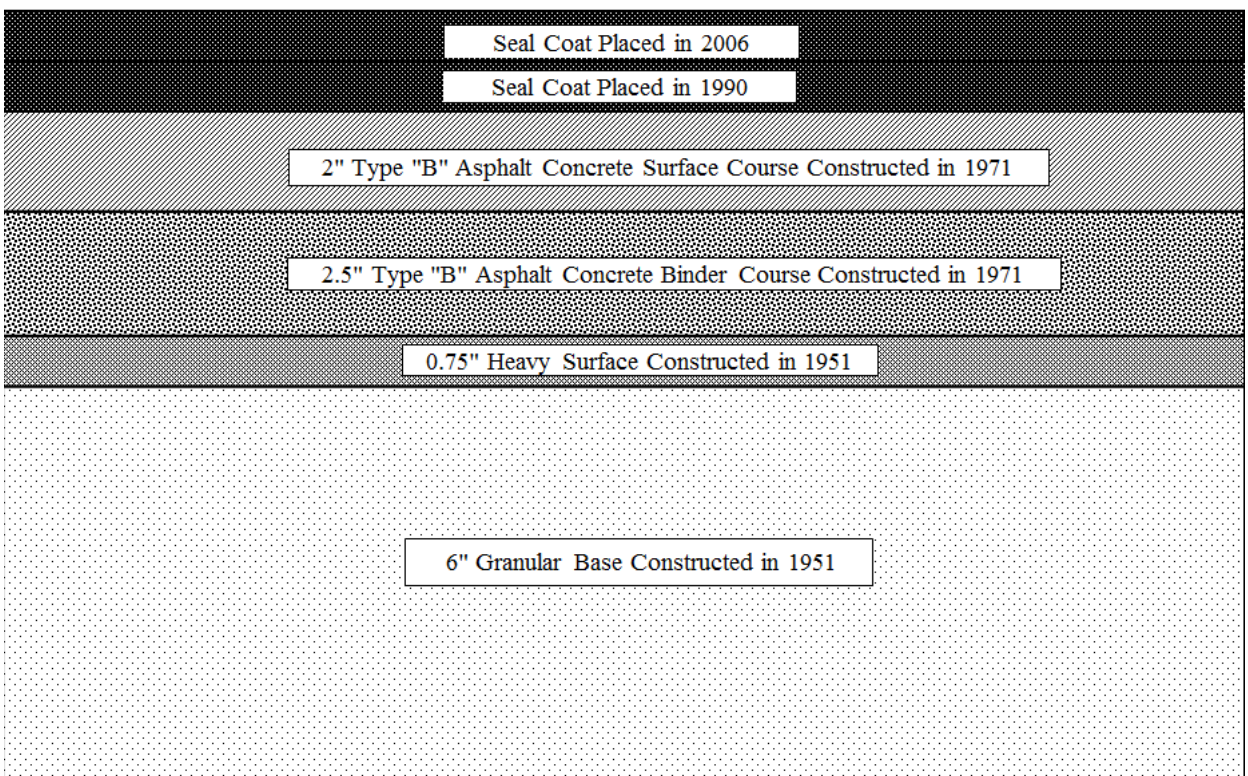

Figure 5. Structure of existing pavement

The Iowa DOT pavement management system records indicate that the pre-construction pavement was in poor condition. The US Army Corps of Engineers' pavement condition index (PCI) was evaluated in 2012 by following the standard ASTM procedures (ASTM International 2011). The results show that the average PCI for the existing pavement was 32. The IRI measured in the same year suggests that the current drivability fails to meet the FHWA definitions for acceptable ride quality. For the purpose of improving pavement performance, the FHWA proposed the IRI values of 95 in. per mile and 170 in. per mile as the primary and secondary goal for pavements, respectively, in 2006 (FHWA 2011b). The average IRI in 2011 for IA 93 was 246 in. per mile. A pavement distress survey was conducted prior to construction. The survey results suggest that the Iowa DOT’s records of present pavement condition may underestimate the level of road deterioration. The predominant distress type found in the pre-construction survey was transverse and longitudinal cracking. Field core samples showed that all cracks initiated from the pavement surface. Other distress types observed included bleeding, potholes, raveling, and edge breaks. The Iowa DOT’s records indicate that the average rut depth for this road section was 4 mm in 2012.

As part of this research project, various holding strategy treatments were applied to 10 test sections. A summary of the treatment methods is shown in Table 1, and the test section locations are shown in Figure 6.

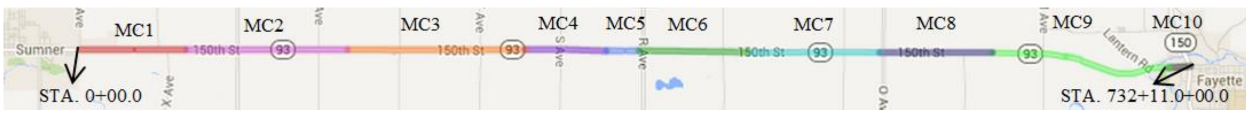

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Figure 6. Rehabilitation treatment section locations

The beginning station, 0+00, was set at the intersection of Y Avenue and IA 93. The ending station was set at the intersection of IA 150 and IA 93. Sections 1 through 9 are on the rural segments of the road, and Section 10 is on the urban segment.

The construction process and quality control and assurance procedures are documented for each construction treatment in the following paragraphs. The material types and quantities and project costs are also summarized.

Materials

HMA Concrete

The project documents specified Type A asphalt concretes for the interlayer, leveling and strengthening course, and surface course with a design traffic load of 1 million ESALs. The HMA overlays with layer thicknesses of 1.5 or 2 in. included an aggregate blend with a 0.5 in. maximum aggregate size (MAS) and a PG58-28 binder at a 5.3% binder content. This mix design was applied to the surface courses in Sections 1, 2, 4, 7, and 8. The leveling and strengthening course in Section 9 included 3/8 in. MAS aggregate and the same PG58-28 binder at a 6.3% asphalt content. Section 3 used various types of 3/8 in. MAS aggregates for the interlayer and ultrathin surface course. The asphalt binders used for the interlayer and surface course are PG64-34 at a 7.4% asphalt content and PG76-34 at a 6.7% asphalt content, respectively. During construction, a material shortage occurred for the PG76-34 binder. About 420 ft of the surface on the eastbound lane was paved using the PG58-28 binder.

The source aggregate was produced by Paul Niemann Construction Company. The asphalt binders (PG58-28, PG64-34, and PG76-34) were sourced from Midwest Industrial Asphalt in LaCrosse, Wisconsin. The aggregate batch design and blended gradation are provided in Table 4 and Table 5.

Table 4. Source aggregate type and blend percentage for HMA concrete mixes

1.5 in. and 2 in. Surface Course Mix		0.75 in. Ultrathin Surface Course Mix	
Aggregate	**Percent in Mix**	**Aggregate**	**Percent in Mix**
1/2 in. ACC Stone	15	3/8 in. ACC Stone	48
1/2 in. Washed Chips	25	3/16 in. Washed Manufactured Sand	32
3/16 in. Washed Manufactured Sand	10	Washed Concrete Sand	7
Washed Concrete Sand	30	3/8 in. Washed Chip	13
RAP	20		
1 in. Interlayer Course Mix		**1 in. Leveling and Strengthening Course Mix**	
Aggregate	**Percent in Mix**	**Aggregate**	**Percent in Mix**
3/8 in. ACC Stone	40	3/8 in. ACC Stone	30
3/16 in. Washed Manufactured Sand	20	3/8 in. Washed Chips	15
Washed Concrete Sand	25	3/8 in. Washed Manufactured Sand	30
Agricultural Lime	15	Washed Concrete Sand	25

ACC = asphalt cement concrete

Table 5. Blended aggregate gradation

Sieve Size	Percent Passing, %			
	1.5 in. and 2 in. Surface Course Mix	0.75 in. Ultrathin Surface Course Mix	1 in. Interlayer Course Mix	1 in. Leveling and Strengthening Course Mix
3/4 in.	100.0	100.0	100.0	100.0
1/2 in.	99.0	100.0	100.0	100.0
3/8 in.	90.0	100.0	100.0	100.0
#4	64.0	76.0	84.0	78.0
#8	48.0	50.0	63.0	53.0
#16	38.0	34.0	47.0	39.0
#30	28.0	28.0	35.0	30.0
#50	12.0	18.0	21.0	16.0
#100	6.0	9.8	11.0	7.7
#200	3.8	4.9	6.5	3.6

CIR

The cold-in-place recycled material consisted of the pulverized existing asphalt pavement mixed with foamed PG52-34 asphalt binder at an average application rate of 0.73 tons per station. The binder material was produced by TexPar Energy, LLC in Davenport, Iowa.

FDR

The full-depth reclamation material consisted of the pulverized existing asphalt pavement mixed with 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%. The binder material was produced by Bituminous Materials and Supply L.P. in Tama, Iowa.

Seal Coat

The seal coat binder specified was CRS-2P asphalt emulsion. The average application rate was 0.38 gallons per square yard for the rural sections. In the urban section, a higher application rate of 0.6 gallons per square yard was applied. The cover aggregate for the seal coat treatment had a 1/2 in. MAS and was sourced from Platte Quarry in Bremer County, Iowa. The average application rate of the aggregate cover was 29 pounds per square yard for the rural sections and 48 pounds per square yard for the urban section.

Construction and Quality Control/Assurance Procedures

Scarification

Scarification operations commenced on June 26, 2013. Four workdays were required to complete these operations for Sections 1, 2, 3, and 10. The existing pavement was milled with a profiler to the design depth and profile. The profiler used in this project is capable of milling the full width of one traffic lane; this satisfies the requirement in Article 2214.03.A2 of the Iowa DOT Standard Specifications that the number of milling passes for one traffic lane cannot exceed two (Iowa DOT 2012). The milling depth was constrained by the need to match the scarification depths at the longitudinal joint and to keep the slope of the scarified surface within the specified tolerances. Construction specifications require that the mismatch in the scarification depths at the longitudinal joint cannot exceed 1/4 in. (Iowa DOT 2012). The minimum and maximum allowable slopes for the scarified surface are specified in the contact documents as 2% and 3%, respectively. An Iowa DOT construction technician performed profile checks at random locations to ensure that the milling requirements were achieved.

The milled pavement materials were transported by a discharge conveyor to a truck in front of the profiler (Figure 7).

Figure 7. Pavement scarification operation

Water was sprayed on the milling drum in order to control dust and cool down the drum. A water tank truck accompanied the profiler to provide a continuous source of water. Before it was opened to traffic, the scarified surface was cleaned with a rotary broom, as specified by Article 2214.03.B5 of the Iowa DOT Standard Specifications (Iowa DOT 2012). Field observations indicated that the milling process eliminated most of the cracks in the existing pavement surface. Some cracks, mostly transverse cracks, were found to have propagated deeper than the milling depth and remained evident in the scarified surface (Figure 8).

Figure 8. Cracks that had propagated deeper than milling depth

CIR

Cold in-place recycling operations commenced on July 23 and were completed on July 26 in 2013. The operation was performed with a CIR train, which included a milling machine, a crushing and screening unit, a pug mill, and an oil tank trailer (Figure 9).

Figure 9. Cold in-place recycling train with milling machine (top left), crushing and screening unit (top right), pug mill (bottom left), and oil tank trailer (bottom right)

The milling machine milled the existing pavement to a 2.5 in. depth, and the RAP was conveyed to the crushing and screening unit. This unit further crushed large chucks of RAP into smaller particles to meet the specification requirements for RAP gradation. Section 2318 of the Iowa DOT Standard Specifications requires 98% to 100% of RAP particles to pass the 1.5 in. sieve and 90% to 100% of RAP particles to pass the 1 in. sieve (Iowa DOT 2012).

The processed RAP was then conveyed to the pug mill, where it was blended with foamed asphalt. The asphalt binder was supplied by an oil tank trailer that was attached behind the pug mill. The average temperature reading for the oil in the tank trailer was slightly above 300°F, which was within the specification-required range of 310°F ± 20°F (Iowa DOT 2012). The blended mixture was placed in a windrow for a paver to lay down as a recycled asphalt layer. A pneumatic tire roller and a steel wheel roller, as specified in Article 2318.03.A5 of the Iowa DOT Standard Specifications (Iowa DOT 2012), were used to compact the CIR layer to the required density. Iowa specifications require the density of the CIR layer to be 94% of the laboratory density or higher.

The laboratory density was determined by measuring the compacted densities of moist CIR samples and finding their corresponding moisture contents. The field-procured CIR samples were compacted with the required compactive efforts using a gyratory or Marshall compactor.

The density of each compacted sample was measured and corrected for the moisture content to provide a dry density. The average dry density was used as a quality control criterion to compare with nuclear density gauge readings for the compacted CIR layer. Details of the density control process are specified in Article 504 of the Iowa DOT Instructional Memorandums for Materials (I.M.s) (Iowa DOT 2013). Ten nuclear gauge density readings were taken at random locations, as required by I.M. 334 (Iowa DOT 2013). The field tests were performed by the contractor, and the laboratory tests were performed by the District 2 Materials Laboratory in Mason City. The test results are summarized in Table 6.

Table 6. Density control tests results for CIR

Testing Date	7/23	7/24	7/25	7/26
Corrected Laboratory Dry Density, pcf	126.6	127.1	127.1	127.1
Average Corrected Field Dry Density, pcf	122.9	123.7	123.3	121.0
Percent of Laboratory Density, %	97.1	97.3	97.0	95.2
Coefficient of Variation for Field Dry Densities, %	2.3	1.3	1.4	1.1
Percent moisture, %	5.2	4.8	4.7	5.1

FDR

FDR operations were performed in two phases. Phase I construction included a pulverizing machine (Figure 10) that pulverized the full depth of the existing pavement.

Figure 10. Pulverizing machine for FDR

The required gradation for the pulverized pavement is the same as that for CIR material. A sheepsfoot compactor was used to compact the pulverized material until the required compaction was achieved. The initial sizing pulverization passes commenced on July 31 and ended on August 1, 2013.

Phase II operations included incorporating fly ash and foamed asphalt into the pulverized RAP, followed by compaction and finished grading. A truck loaded with fly ash was used to place fly ash at a rate of 2.4 tons per station. Fly ash and sized RAP were mixed with foamed asphalt using the same pulverizing machine that was utilized for the initial sizing. An oil tank trailer and a water tank trailer were attached to the pulverizing machine to supply asphalt binder and water for foaming (Figure 11).

Figure 11. FDR train

Initial rolling was performed with a sheepsfoot compactor immediately after the mixing process. A motor grader (Figure 12) was operated to eliminate the sheepsfoot roller marks and to perform rough grading of the compacted surface.

Figure 12. Motor grader used for finishing FDR

A steel wheel compactor and a pneumatic tire roller were then used to finish compacting the FDR layer to the required density. According to Article 2116.03.E1 of the Iowa DOT Standard Specifications, the field density measured at a depth of 75% of the FDR layer thickness (6 in. for this project) is required to be 94% of the target density or greater, and the field density measured at a depth of 25% of the FDR layer thickness is required to be greater than 97% of the field density at the 75% thickness (Iowa DOT 2012). The field density measured at 2 in. is required to be 97% of the field density at 6 in. The density control process is referenced against corrected dry densities. The method used to obtain these corrected dry densities is specified in I.M. 504 (Iowa DOT 2013). The test results for density control are summarized in Table 7.

Table 7. Density control tests results for FDR

Testing Date	8/3	8/6	8/7
Corrected Laboratory Dry Density, pcf	133.4	134.0	130.1
Average Corrected Field Dry Density (6 in.), pcf	132.7	134.0	130.1
Average Corrected Field Dry Density (2 in.), pcf	131.3	132.6	131.7
Percent of Laboratory Density (6 in.), %	99.4	100.0	100.0
Percent of the Density at 6 in. (2 in.), %	98.9	98.9	100.7
Coefficient of Variation for Field Dry Densities (6 in.), %	2.1	2.2	1.5
Coefficient of Variation for Field Dry Densities (6 in.), %	2.9	2.3	1.3
Percent Moisture at (6 in.), %	5.9	5.9	4.8

After compaction was completed, the motor grader performed final grading of the FDR layer surface to ensure that the design cross slope and profile were achieved. According to Article 2116.03.F4 of the Iowa DOT Standard Specifications, the cross slope for this project must be within a range of 2% to 3%, and the profile variation cannot exceed 1 in. (Iowa DOT 2012). The specifications also require a minimum of a 6 in. overlap between the two pulverization paths in the eastbound and westbound lanes at the centerline. The overlap width for this project was 1 ft.

The planned date for phase II construction was August 2, 2013. However, the temperature readings for the oil tank trailers were 287°F and 277°F, which were below the allowable temperature range of 320°F ± 20°F. The readings at the plant before transportation were above 328°F. The suspected cause for the low temperature readings was the long hauling distance. Asphalt binder for foaming was obtained from a different supplier located closer to the project site, and the temperature criteria were then met. The actual phase II construction was commenced on August 3, 2013. FDR construction operations were paused on August 4 and 5 due to the weekend and rainy weather. The operations were resumed on August 6 and completed on August 7 in 2013.

HMA Paving

HMA materials for the surface course, interlayer, and leveling and strengthening courses were placed according to Article 2303.03 of the Iowa DOT Standard Specifications (Iowa DOT 2012). All HMA paving operations commenced on August 7 and finished on August 16, 2013. The laydown dates for each section are listed in Table 8.

Table 8. HMA paving construction dates

Section	Date	Course	Thickness (in.)
1	8/7	surface	1.5
1	8/8	surface	1.5
2	8/8	surface	1.5
2	8/9	surface	1.5
4	8/9	surface	1.5
8	8/12	surface	2
9	8/13	leveling and strengthening	1
9	8/14	leveling and strengthening	1
3	8/14	interlayer	1
3	8/15	surface	0.75
4	8/15	surface	1.5
7	8/16	surface	1.5

Prior to the placement of the HMA materials, a thin film of tack coat was sprayed onto the road surface in order to ensure bonding between the HMA layer and the underlying surface. HMA mixtures were produced in a portable asphalt plant that was located northwest of the intersection of X Avenue and IA 93. Bottom dump trucks were used to haul the HMA materials to the construction area and place them in a windrow. A windrow pickup unit was used to transfer the materials from the windrow to the paver, which formed the materials to the design thicknesses (Figure 13).

Figure 13. Bottom dump truck and paver

Pneumatic tire rollers and steel wheel rollers compacted the HMA layers. The Class I compaction method (Article 2303.03.C5 of the Iowa DOT Standard Specifications) was applied to the surface courses (Iowa DOT 2012). The leveling and strengthening course and interlayer were compacted using the Class II compaction method according to Article 2303.03.C5 of the Iowa DOT Standard Specifications (Iowa DOT 2012). For Class I compaction, in-place air voids and layer thicknesses were used as quality characteristics for the computation of pay factors. Air void tests and thickness measurements were performed in accordance with Article 2303.03.D of the Iowa DOT Standard Specifications (Iowa DOT 2012). The test results are summarized in Table 9.

Table 9. HMA quality control characteristics

Date	Average Percent Air Voids (%)	Thickness Quality Index
8/7	8.1	0.33
8/8	6.5	2.06
8/9	5.8	0.84
8/12	5.3	1.35
8/16	6.5	1.24

Seal Coat

Before the bituminous seal coat was applied, the road surface was cleaned with a rotary broom. An oil distributor then sprayed emulsified asphalt at a temperature between 150°F and 185°F onto the road surface. A chip spreader followed the oil distributor and placed the cover aggregate. The aggregate cover was compacted by two pneumatic rollers to embed the aggregate into the bituminous film. An overview of the seal coat treatment operation is shown in Figure 14.

Figure 14. Construction of seal coat

After two hours of curing, the aggregate was sufficiently adhered to the asphalt and the road was opened to traffic. Before opening to traffic, the rotary broom was used to sweep away loose aggregate. Diluted emulsified asphalt was then sprayed onto the road surface to control dust (Figure 15).

Figure 15. Dust control for seal coat

The construction of the seal coat commenced on August 28 and was completed September 5, 2013.

Shoulder

A Type B granular shoulder treatment, as specified in Section 2121 of the Iowa DOT Standard Specifications (Iowa DOT 2012) was applied to the pavement edge for this project. Damp aggregate was placed on the road shoulder with a laydown vehicle as shown in Figure 16.

Figure 16. Shoulder aggregate placement vehicle

After placement of the shoulder aggregate, the shoulder was compacted according to Article 2121.03.C3 of the Iowa DOT Standard Specifications. Shoulder construction commenced on August 17 and was completed August 20, 2013.

Traffic Control

Traffic control operations were conducted according to Article 1107.08 of the Iowa DOT Standard Specifications, “Public Convenience and Safety” (Iowa DOT 2012). The roadway was completely closed in both lanes during FDR operations. A detour was established over adjacent roads. For other operations, single-lane traffic control and a pilot car and flaggers were used to lead traffic through the work zone safely (Figure 17).

Figure 17. Single-lane pilot car traffic control

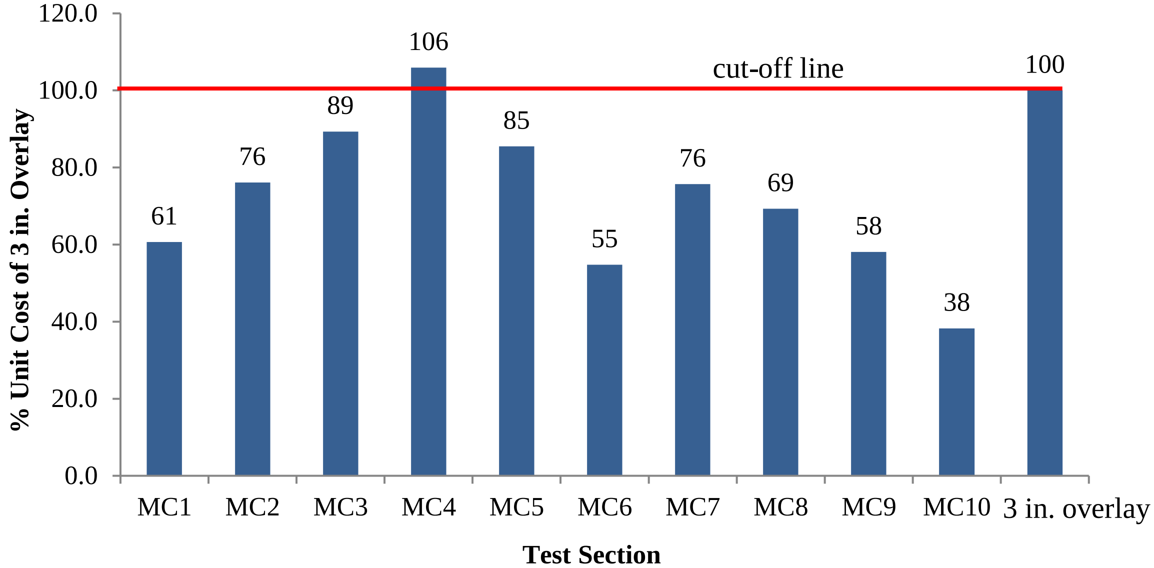
Quantity and Cost

The total cost for the project was $1,692,157. Table 10 summarizes the cost and material quantity for each bid item.

Table 10. Construction costs and material quantities

Item	Unit	Rural Quantity	Rural Unit Price	Urban Quantity	Urban Unit Price	Cost
Mobilization	LS	1	49121.5	1	1002.48	50124
Pavement Scarification	SY	78866	0.8	4893	0.8	67007
Painted Pavement Mark	STA	4136	6.9	57.132	6.9	28930
Flagger	EACH	96.5	325	8	325	33963
Pilot Car	EACH	18	490	1	490	9310
Full-depth reclamation	SY	19360	3.06			59313
Fly Ash	TON	180.4	45			8119
Cold-in-Place Recycling	SY	41600	1.98			82368
Safety Closure	EACH	9	100			900
Traffic Control	LS	1	9692.2	1	197.8	9890
Granular Shoulder Type B	TON	3034	21.49			65193
Clean Preparation of Base	MILE	8.8	500	0.3	500	4550
HMA Level/Strengthen	TON	1609	36.25			58337
HMA Interlayer, 3/8 in.	TON	1573	40.69			64019
HMA Surface, 1/2 in.	TON	9423	32.7			308134
Asphalt Binder, PG58-28	TON	660.6	545			360005
HMA Pavement Sample	LS	1	2500			2500
Aggregate Cover, 1/2 in.	TON	1504	21.3	82.63	21.3	33799
Ultrathin Lift Surface, 3/8 in.	TON	1316	41.88			55121
Pay Factor Adjustment for Field Voids	EACH	11675	1			11675
Binder, CRS-2P	GAL	39547	3.15	2068	3.15	131087
Asphalt Emulsion for Dust Control	GAL	1127	8.3	112	8.3	10285
Price Adjustment for Non-complying Traffic Control	EACH	-1500	1			-1500
Foamed Asphalt	TON	122.8	600.85			73796
Asphalt Binder, PG76-34	TON	89.1	853			75977
Asphalt Binder, PG64-34	TON	115.4	700			80752
Pay Adj I/D-HMA PAV'T Smoothness	EACH	8500	1			8500
Total Cost					**$**	**1692157**

A comparison between the costs for the various test sections is shown in Figure 18.



Section 10 included a 6 ft parking lane in each direction, while the other sections did not.

Figure 18. Test section construction cost comparison

Because the test sections are extremely short in comparison to typical road projects, direct comparisons of the construction costs of the various test sections are not instructional for the selection of a holding strategy. Therefore, the unit costs per mile were compared using relative values. The cost for a hypothetical 3 in. overlay was estimated based on the actual cost of the 2 in. overlay section and was chosen as the threshold.

TEST RESULTS

The results of the pavement condition surveys, structural capacity tests, and surface characterization tests are summarized and discussed in this chapter.

Test Section Performance

Pavement condition surveys were conducted before and after construction. The survey schedule is shown in Table 11.

Table 11. Pavement condition survey schedule

Survey	Date(s)
Pre-Construction	July 2013
Project Construction	August and September 2013
1st Post-Construction	September 2013
2nd Post-Construction	April 2014
3rd Post-Construction	November 2014
4th Post-Construction	April 2015
5th Post-Construction	December 2018
6th Post Construction	October 2019

The pre-construction survey showed that the existing pavement was suffering from severe thermal cracking, raveling, edge breaks, and a rough surface (see Figure 19).

Figure 19. Existing pavement surface condition

However, no signs of severe load-related failures, such as considerable longitudinal cracking in the wheelpaths, bottom-up cracking, or severe fatigue cracking, were found. The first post-construction survey showed that the construction was successful. The later post-construction surveys indicated that surficial distresses started to occur during the winter following the construction.

The condition surveys conducted according to the schedule shown in Table 11 indicated that the primary distress type was transverse cracking. The densities of the transverse cracking observed in the post-construction surveys are compared with the pre-construction crack density in Figure 20.

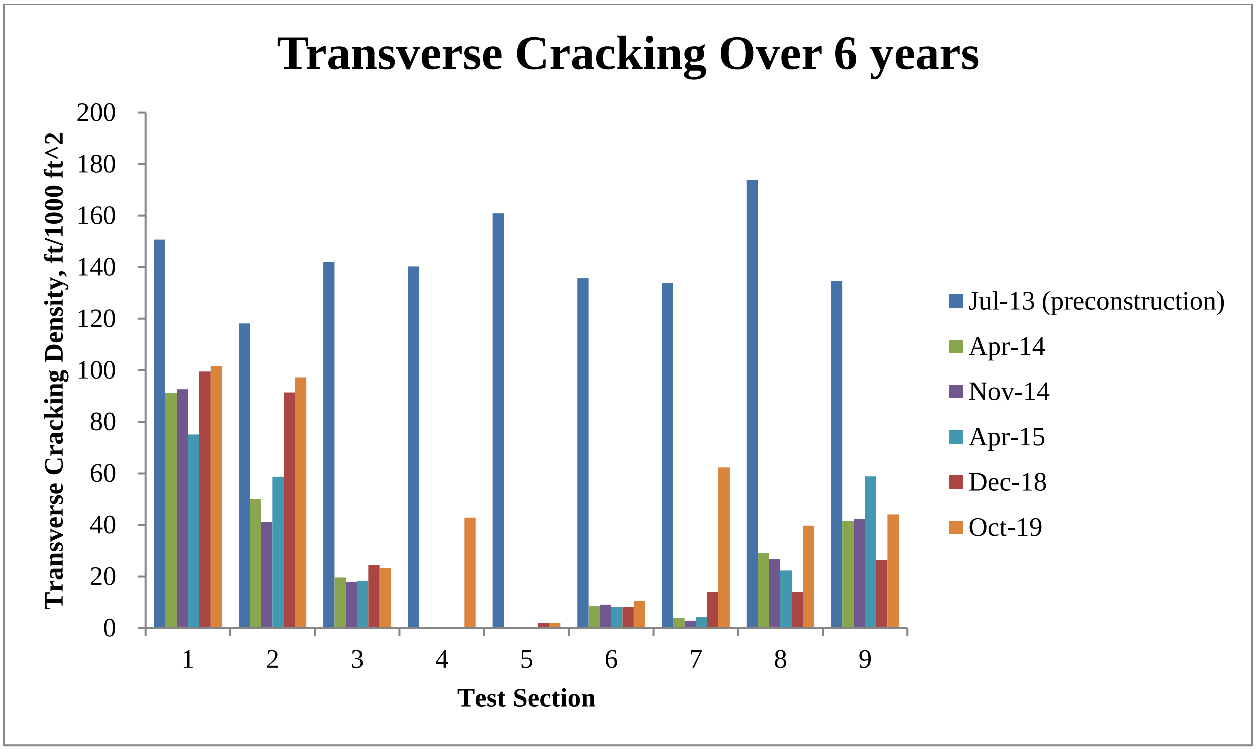


Figure 20. Transverse cracking densities

Because Section 10 has a different geometry, traffic speed, and pavement structure, as suggested by the core samples from the other test sections, the performance of Section 10 is discussed in a separate paragraph and is not compared with the other sections. Field core samples procured in July 2015 also suggested that the transverse cracks in the sections that received milling as the base preparation method or that did not receive a base treatment were reflective cracks that developed from the crack pattern remaining in the existing pavement structure. The crack numbers did not considerably change after the second post-construction survey. The average transverse cracking density of the April 2015 survey was compared to the cracking density of the existing pavement for each test section. A crack reduction value was calculated as the percent of transverse cracking in the existing pavement that is successfully corrected by the holding strategy treatments. The test sections were grouped according to their crack reduction values and compared to reveal the primary factors that contribute to better performance. The grouped sections and contributing factors are shown in Table 12 in order from the highest to the lowest crack reduction value.

Table 12. Contributing factors to transverse cracking correction ability

Test Section	Crack Reduction (%)	Contributing Factor
4, 5, 6, 7	> 95	Aggressive recycling treatments for the existing pavement (CIR and FDR)
3, 8	80–95	High-quality asphalt overlay material and thicker overlay lift
2, 9	60–80	Chip seal
1	< 60	

CIR = cold in-place recycling
FDR = full-depth reclamation

At the December 2018 survey, the crack reduction percentages for FDR and CIR sections were 92 for transverse cracks and 97 for longitudinal cracks. This order also indicates the influence level of the contributing factors.

At the time of the December 2018 and October 2019 surveys, it was noted that 80% of all cracks had been sealed in accordance with established maintenance practices, and these sealed cracks were included in the cracking density calculations.

The sections where recycling technologies were used had the most satisfactory performance in terms of transverse cracking mitigation. The sections where CIR and FDR were used without an asphalt overlay exhibited a cracking correction ability comparable to that of sections where recycling technologies and an asphalt overlay were used. Higher quality or thicker asphalt overlays were the most effective treatments for reducing transverse cracking for the sections that did not receive CIR or FDR treatments. The 1 in. milling did not appear to considerably influence crack mitigation.

Figure 21 and Figure 22 summarize the longitudinal cracking and rutting observed in the pavement condition surveys.

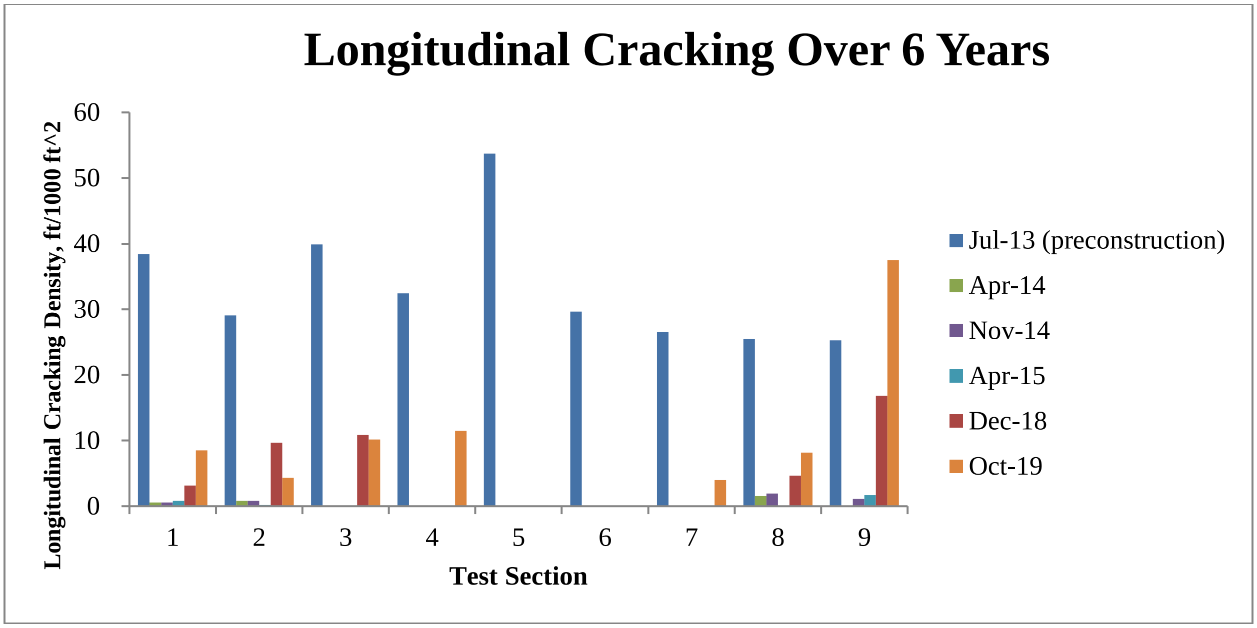


Figure 21. Longitudinal cracking densities

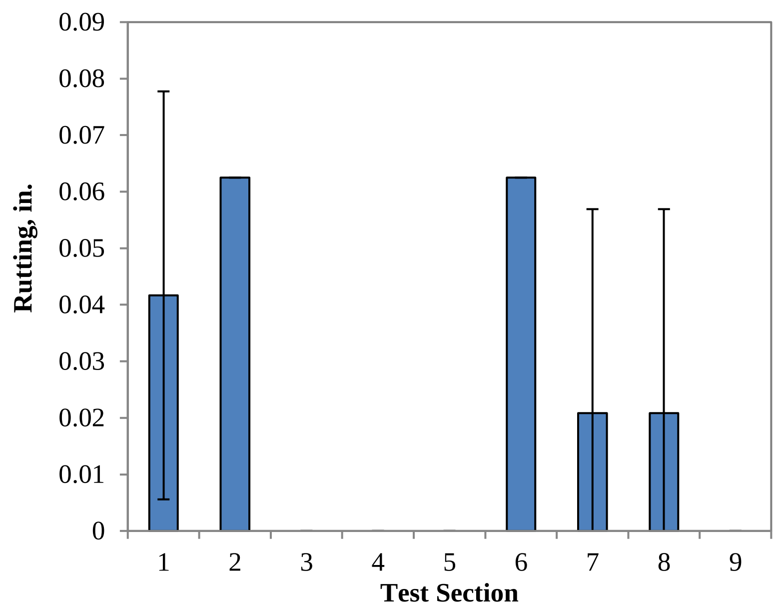


Figure 22. Average rutting on IA 93 test sections

All of the sections exhibited satisfactory performance in terms of longitudinal cracking mitigation and rutting resistance. Figure 22 shows the average rutting for each test section; the error bars show the standard deviation (an indication of variability) of the measurements as of April 2015. The large error bars for Sections 1, 7, and 8 indicate greater variability with regard to the amount of rutting that occurs along the length of the test section, thus implying that the rutting was localized in certain areas of these test sections. Spot checking for rutting was conducted in December 2018 and October 2019, and no measurable changes were recorded.

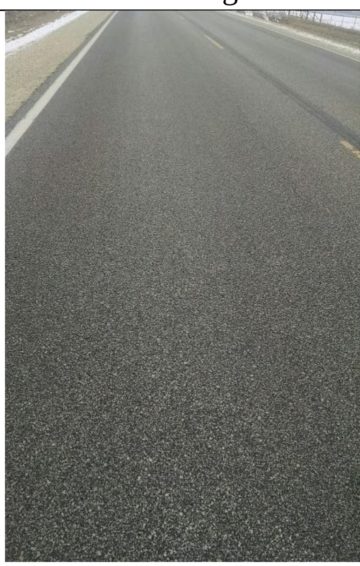
A severe loss of chip seal cover aggregate was found in the wheel paths of Section 5. In some areas, the aggregate loss had led to small potholes. This was especially evident at a bridge approach where compaction of the FDR materials might have been difficult. Loss of aggregate did not occur in the other chip seal surfaces. A comparison of the chip seal surface appearances is shown in Figure 23.

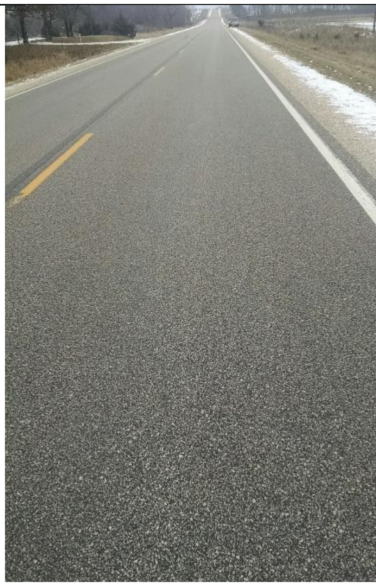
Figure 23. Appearance of four chip seal surfaces

Snow plowing operations are believed to be the primary cause for the aggregate loss problem. The relatively low bond strength between the chip seal aggregate and the full-depth reclaimed layer may have resulted in the performance difference between the chip seal in Section 5 and the chip seals in other test sections.

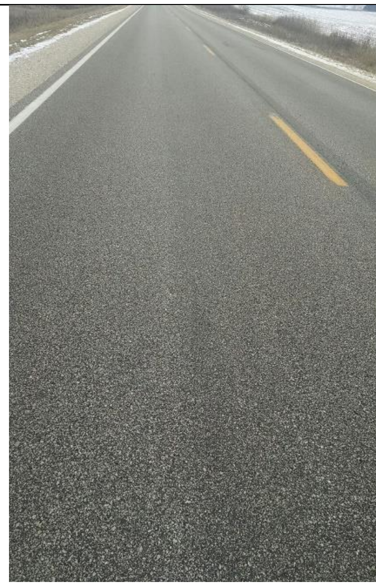
The urban segment of IA 93 was not considered as a formal test section for this investigation. However, the following description of post construction conditions as of April 2015 is provided for interested readers. This section had more severe transverse and longitudinal cracking before treatment construction. The transverse and longitudinal cracking densities were 204 ft/1,000 ft^2 and 174 ft/1,000 ft^2, respectively. (The average cracking density in the rural segment was 143 ft/1,000 ft^2 for transverse cracking and 33 ft/1,000 ft^2 for longitudinal cracking.) The three most recent pavement condition surveys showed that the average transverse and longitudinal cracking densities were 21/1,000 ft^2 and 24 ft/1,000 ft^2. Although the cracking performance of Section 10 was satisfactory, the treatment is not considered to be completely satisfactory due to the severe loss of chip seal aggregate on the 6 ft parking lane. A heavier application of emulsion on the parking lane would likely address this situation. Since there is less traffic in a parking lane that will embed the aggregate, additional emulsion is often helpful.

Figure 24 summarizes post-construction conditions as of December 2018; only minor changes occurred between this time and the last survey that was conducted in October 2019.

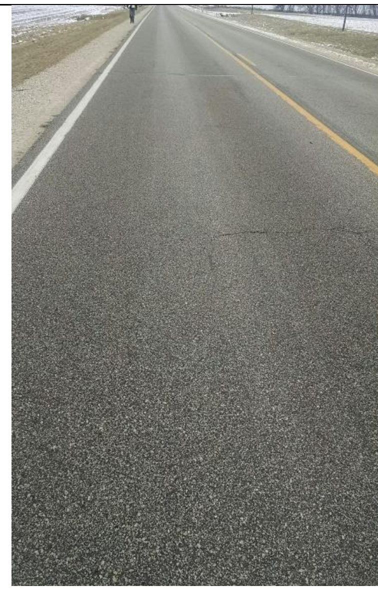
 Section 1: Most extensive cracking	 Section 2: Similar cracking to Section 1	 Section 3: Longitudinal cracks in the outer wheel path
 Section 4: No cracking	 Section 5: Some raveling between the wheel paths and pot holes	 Section 6: Minimal cracking



Section 7: Minimal cracking



Section 8: Minimal cracking



Section 9: Majority of longitudinal cracking in the wheel paths

Figure 24. Post- construction conditions of treatment sections

Structural Performance

Analyzing pavement structural capacity requires accurate thickness measurements for pavement layers. Sample cores were taken in 2015, and the cores were measured to estimate the actual thicknesses of the pavement layers in each section. The estimated layer thicknesses are summarized in Table 13.

Table 13. Estimated layer thickness measured from field cores

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

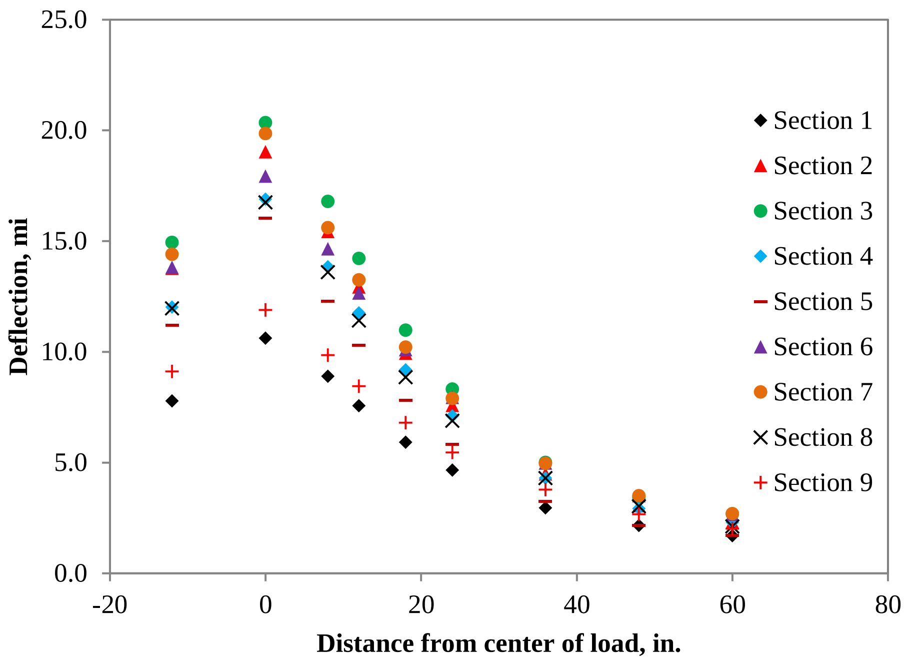
AC = asphalt concrete
CIR = cold in-place recycling
CS = chip seal
ExitPvt = existing pavement

The research team failed to obtain intact cores for Section 5 because the cores were destroyed during coring due to the low binding strength of the material. The thickness of the FDR layer in Section 5 was assumed to be 10 in. because the FDR layer in Section 4 was measured to be 10 in.

FWD Test

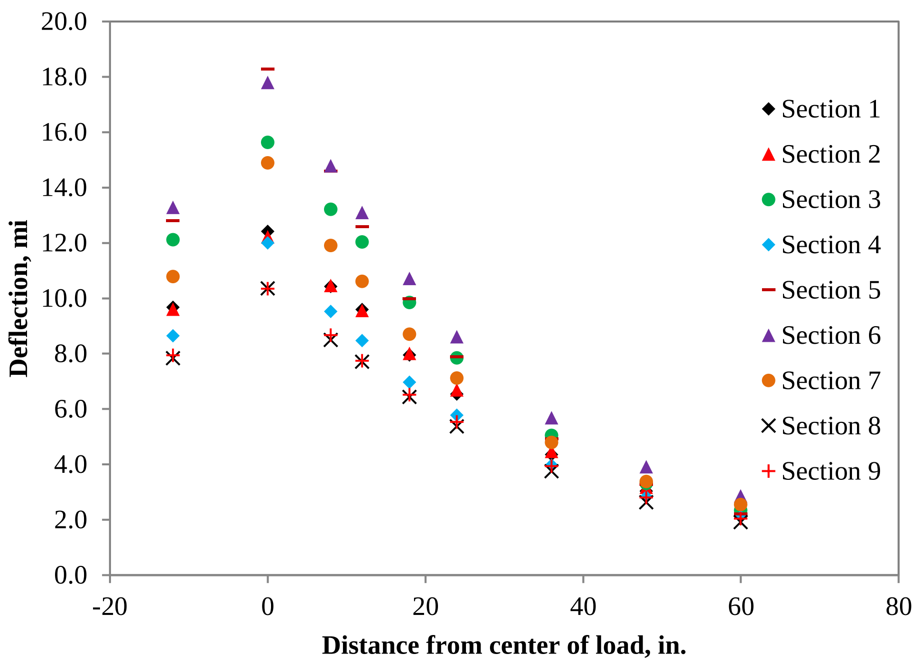
The deflections measured by each geophone at various locations within the same section were averaged to establish the depth and shape of the deflection basin for each of the test sections. The average depth and shape of the deflection basins for the test sections tested in 2012, 2013, and 2015 are shown in Figure 25.

Because Section 10 was not tested using the FWD before the holding strategy was applied, this section was not included in the structural performance analyses. The FWD testing results before construction of the holding strategies (2012) show that the deflections close to the center of the loads for Sections 1 and 9 were lower than those of the other sections. This indicates the possibility of a 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 among the deflection basins of various sections. This suggests that the various holding strategy treatments may influence pavement structural capacity differently.



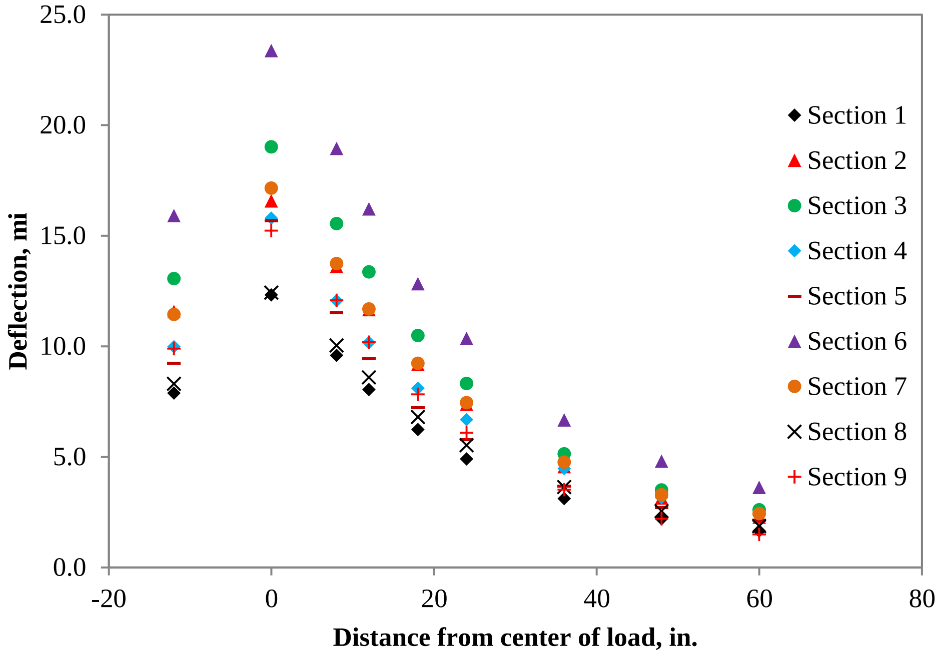
25.0
20.0
15.0
10.0
5.0
0.0
-20
0
20
40
60
80
Deflection, mi
Distance from center of load, in.
Section 1
Section 2
Section 3
Section 4
Section 5
Section 6
Section 7
Section 8
Section 9

2012



20.0
18.0
16.0
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
-20
0
20
40
60
80
Deflection, mi
Distance from center of load, in.
Section 1
Section 2
Section 3
Section 4
Section 5
Section 6
Section 7
Section 8
Section 9

2013



2015

Figure 25. FWD deflection basin at a load level of 12 kips

The BAKFAA software application was used to backcalculate the pavement layer modulus from the measured deflection basins. In the backcalculation process for a layered system, a thin layer, usually less than 2 in., is considered to be insensitive, which would indicate that the deflection of the thin layer is so small that a unique modulus value cannot be converged on. Therefore, each section was assumed to have a two-layer structure, including an asphalt pavement layer and a foundation (FND) layer. The AC layer consisted of all asphalt layers and CIR or FDR layers. The FND layer was a semi-infinite layer that included the aggregate base and subgrade. The backcalculated modulus of the AC layer was adjusted to a reference temperature of 25°C (77°F) using the Chen equation (Equation 1) (Lukanen et al. 2000). The test section moduli are summarized in Figure 26.

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

where:
E_{Tr} = modulus corrected to a reference temperature of T_r(°C)
E_T = modulus corrected to a reference temperature of T(°C)

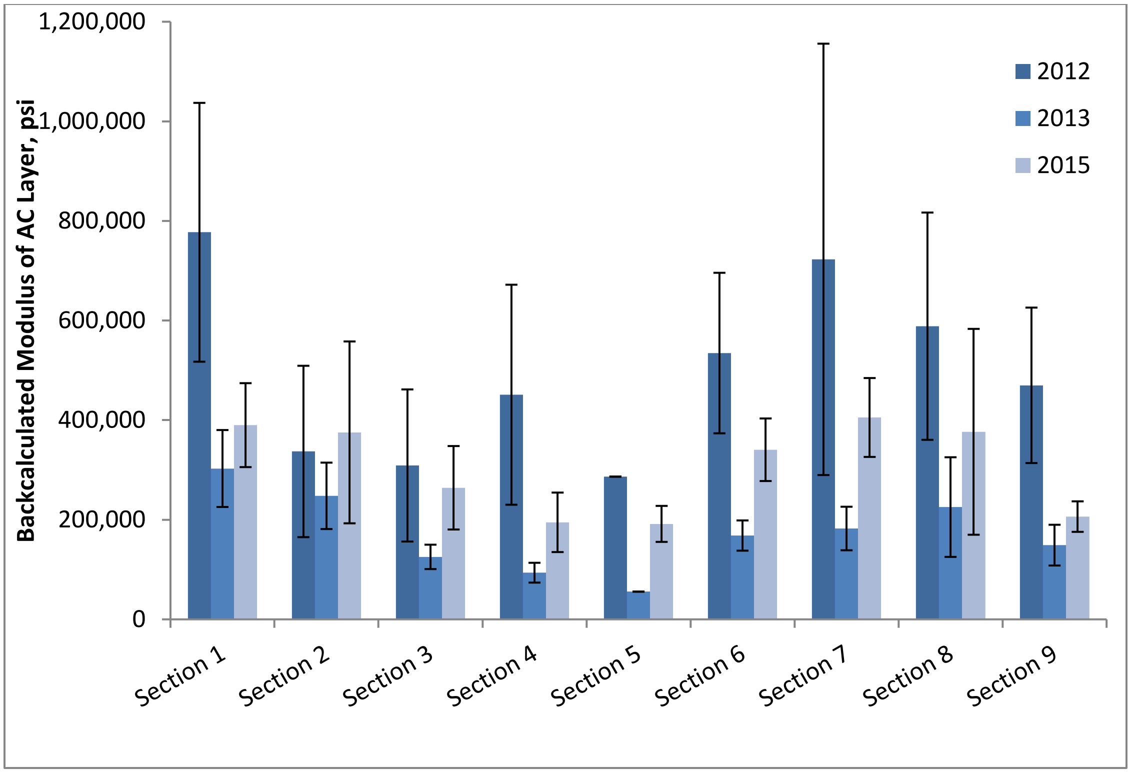


Figure 26. Backcalculated effective pavement modulus at 25°C

The results show that the overall pavement moduli decreased after construction. The pavement moduli may have decreased so drastically because the newly constructed pavement layers are softer than the pre-construction pavement and bias the temperature adjustment procedures. The tests were performed by experienced FWD operators using the same equipment and procedures, and therefore the systematic biases resulting from different testing procedures, equipment, and experimenters are presumed to have been 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 for moduli measured at lower temperatures, the Chen equation may underestimate the adjusted modulus at the reference temperature. 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 the pavement moduli was likely the introduction of softer layers into the pavement structure. The existing pavement had been aged and compacted by traffic for decades. Compared to the existing pavement, the newly constructed pavement layers are more flexible, which would yield lower combined moduli. Increases in the pavement moduli measured in 2015 compared with the moduli measured in 2013 also show that traffic and asphalt oxidization increased the moduli of the newly constructed pavement layers. Those increases are statistically significant for Sections 3, 4, 6, 7, and 9. The estimated moduli for the foundation layer are very similar from one year to another. The average modulus of the foundation layer is 21,000 psi.

E Test*

The E* test results were used to establish the E* master curve for each type of asphalt mixture. The E* values at the three testing temperatures were shifted to the 25°C reference temperature and fit with a sigmoidal model. The master curves are shown in Figure 27.

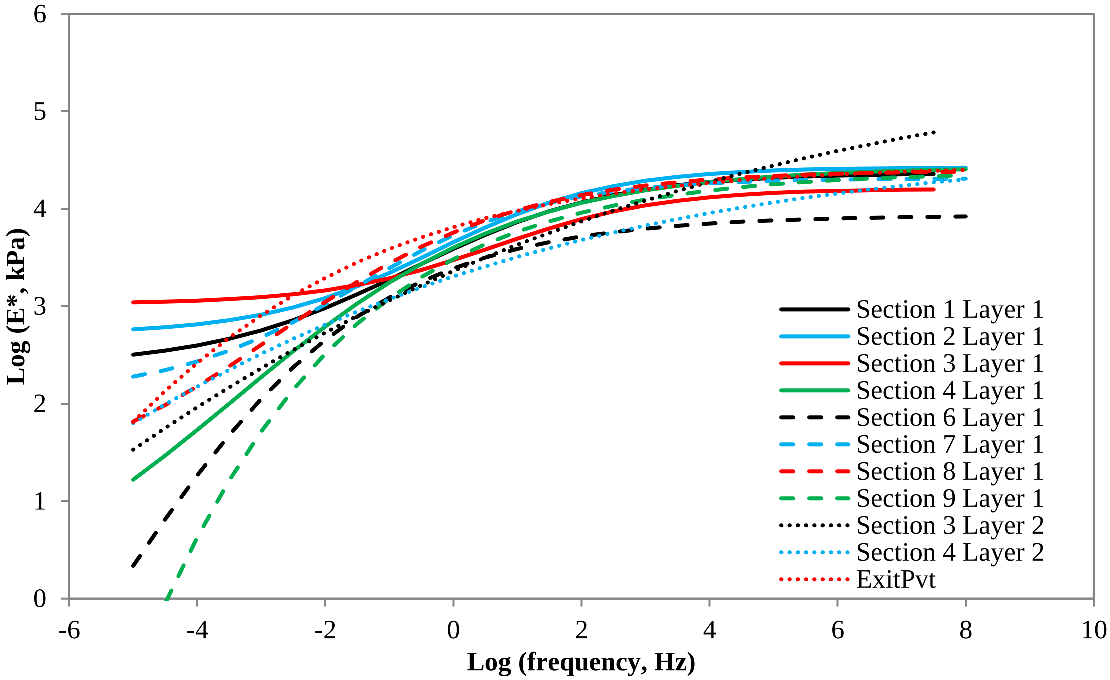


Figure 27. E* master curve

The master curves are used to estimate the E* values at 25°C and a 5.3 Hz frequency, which was chosen to simulate the pavement response under an impact load applied by the FWD (Loulizi et al. 2002). The estimated E* values under an impact load and at the reference temperature are summarized in Table 14.

Table 14. Dynamic modulus at 25°C and 5.3 Hz

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

AC = asphalt concrete
CIR = cold in-place recycling
CS = chip seal
FDR = full-depth reclamation

The newly constructed layers have lower moduli than the existing pavement layers. This finding agrees with the FWD testing results. The moduli of the CIR and FDR layers are considerably lower than those of the AC layers. Moreover, the addition of a chip seal does not seem to influence the modulus of asphalt pavement.

Structural Number and Layer Coefficient

The structural number for each pavement section based on the backcalculated modulus is calculated by multiplying the equivalent layer coefficient of the AC layer by the thickness of the AC layer. The structural number based on the measured E* is the sum of the products of the layer coefficients of the individual pavement layers and the layer thicknesses. The layer coefficient was estimated from the backcalculated modulus and measured E* using an empirical equation (Equation 2) developed by AASHTO.

$$a_i = a_s \left(\frac{E_i}{E_s}\right)^{1/3} \quad \textbf{(2)}$$

where:
a_i = structural layer coefficient of the pavement layer of interest
a_s = structural layer coefficient of a standard material (in this study, a_s is assumed to be 0.44)
E_i = modulus of the pavement layer of interest
E_s = corrected modulus of a standard material (in this study, E_s is assumed to be 3,000 MPa)

The estimated effective structural number (SN_{eff}) of each section is presented in Table 15.

Table 15. Effective structural number

Section	SN_{eff} estimated from FWD results 2012	2013	2015	SN_{eff} estimated from E*
1	3.7	2.8	3.1	3.9
2	2.9	2.8	3.2	4.0
3	3.1	2.5	3.2	4.4
4	3.3	3.1	3.9	4.6
5	2.9	2.2	3.3	3.8
6	3.1	2.2	2.8	3.2
7	2.9	2.4	3.1	3.5
8	3.1	3.1	3.6	4.7
9	4.0	3.1	3.5	5.4

The layer coefficients of individual layers estimated from the E* values are shown in Table 16.

Table 16. Layer coefficient estimated from E*

Section	Layer	Structural Layer Coefficient
1	1	0.49
2	1	0.46
3	1	0.45
4	1	0.47
6	1	0.41
7	1	0.50
8	1	0.54
9	1	0.43
3	2	0.39
4	2	0.38
Existing Pavement		0.54

SN_{eff} values estimated from E* are generally higher than SN_{eff} values estimated from FWD results. However, the SN_{eff} rankings for the test sections based on each of the two methods are in quite good agreement 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 relatively low stiffness of the CIR layers. The structural capacity of the FDR sections shortly after construction was considerably lower than that of 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 to that of the pavement before construction. Different levels of increase in structural capacity were also observed for the other sections.

Surface Characterization

The surface characterization tests were conducted in May 2015, 20 months after construction. The DFT evaluation and SPT were performed on the outside wheelpath and the lane centerline at three random locations for each test section. The friction coefficients were measured with a slider rotating at a speed of 60 km/h in both dry and wet conditions. The IRI was measured in the westbound lane, and the DFT evaluation and SPT were conducted in the eastbound lane.

DFT

The average friction coefficients in various testing conditions are shown in Figure 28.

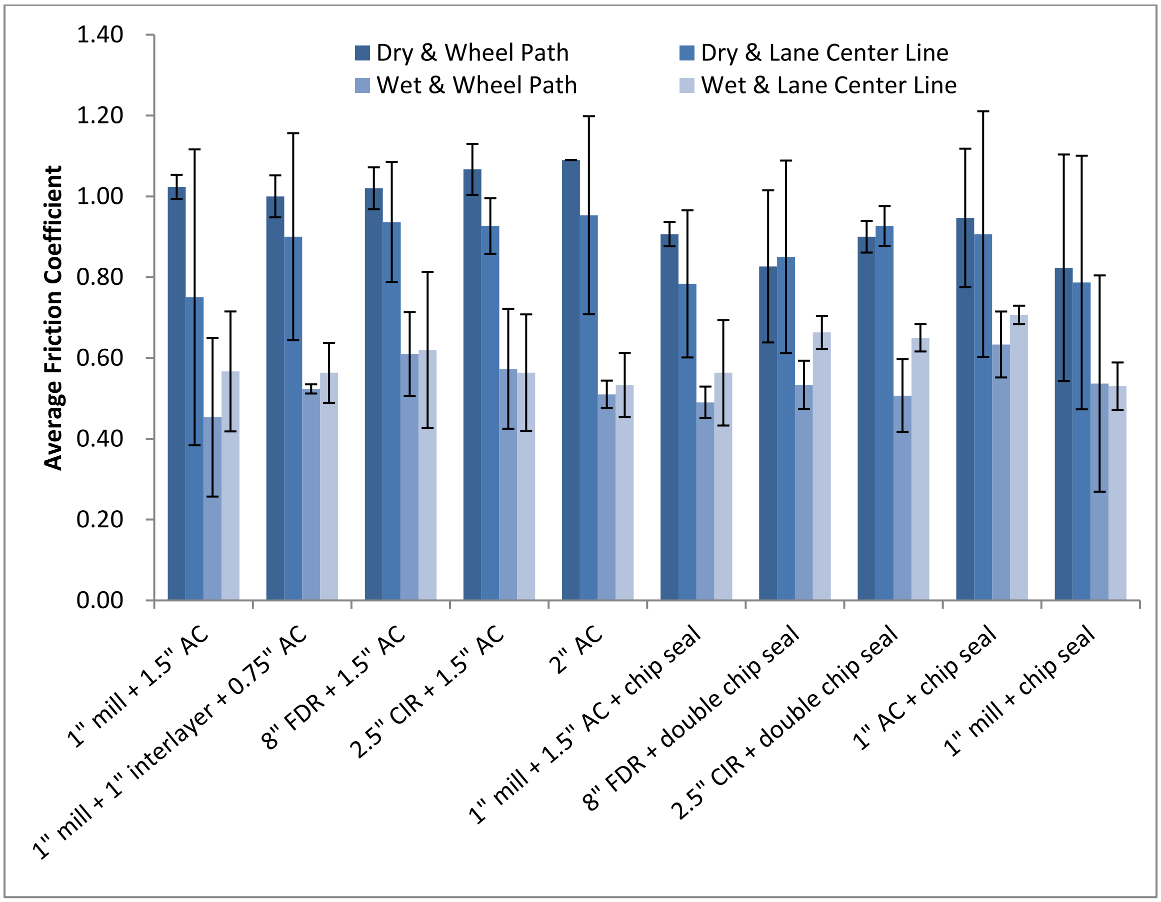


Figure 28. DFT results for various testing conditions

The error bars indicate the 95% confidence intervals of the mean friction coefficients. The dry friction coefficients of the sections with an AC surface were higher than the friction coefficients of the chip seal surface sections. The average dry friction coefficient was measured as 1.04 for the asphalt surfaces and 0.88 for the chip seal surfaces. Since the coefficient was greater than 1.00 for asphalt surfaces, there was likely a measurement error, so these measurements should be considered only as a relative comparison of the friction between asphalt and chip seal surfaces rather than indications of absolute amounts. Note that the friction coefficients for the AC surfaces and chip seal surfaces were not very different from each other in the wet condition. The average friction coefficient of all sections in the wet condition was 0.57.

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 surfaces. However, a few differences in the friction coefficients were measured in the wet condition between the wheelpath and the lane centerline. For the chip seal surfaces, the dry friction coefficients were not affected by whether or not the measurements were taken in the wheelpaths. However, four of the five chip seal sections showed smaller friction coefficients in the wheelpath than in the lane centerline in the wet condition.

SPT

The results of the SPT are summarized in Figure 29.

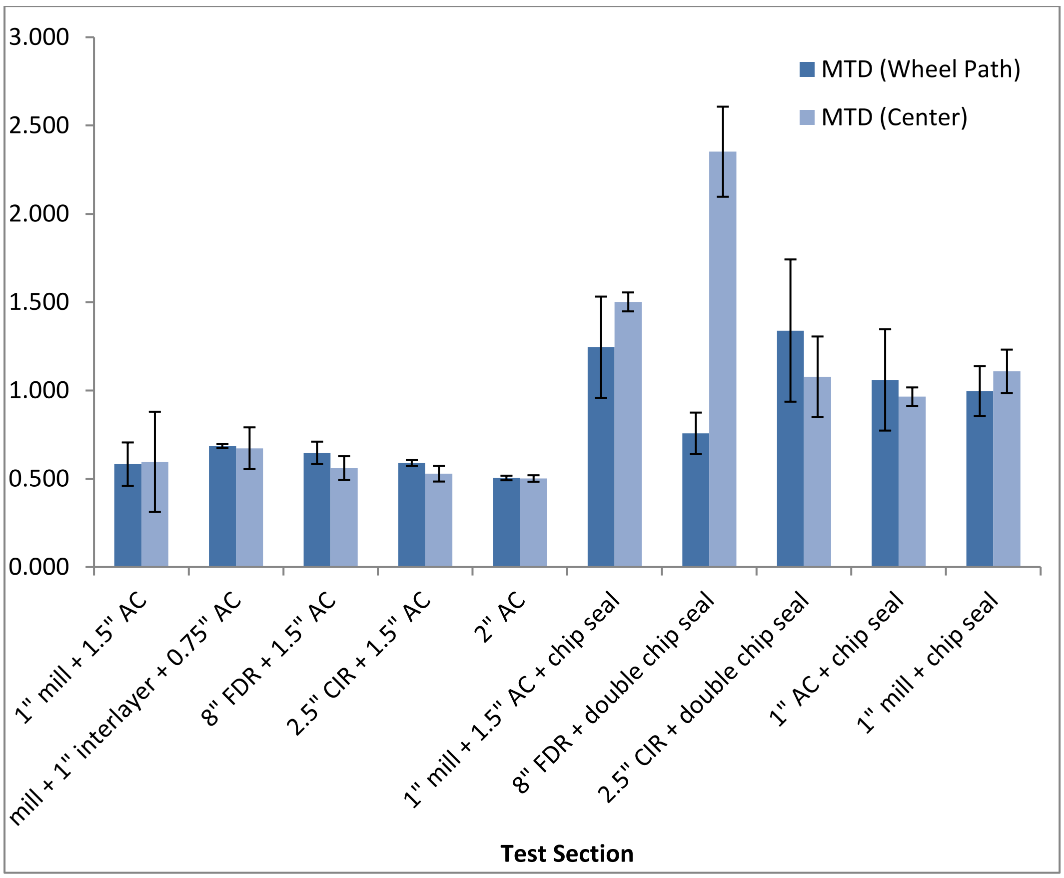


Figure 29. Mean texture depths

The MTD of the chip seal surfaces was considerably higher than that of the asphalt surfaces. The average MTD was 0.602 mm and 1.079 mm for the asphalt and chip seal surfaces, respectively. The measurements taken in the wheelpath were very similar to the measurements taken 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, and the loss of chip seal aggregate cover in Section 5 is believed to be the leading cause.

IRI

Figure 30 shows that the rideability has been considerably improved by the holding strategy treatments.

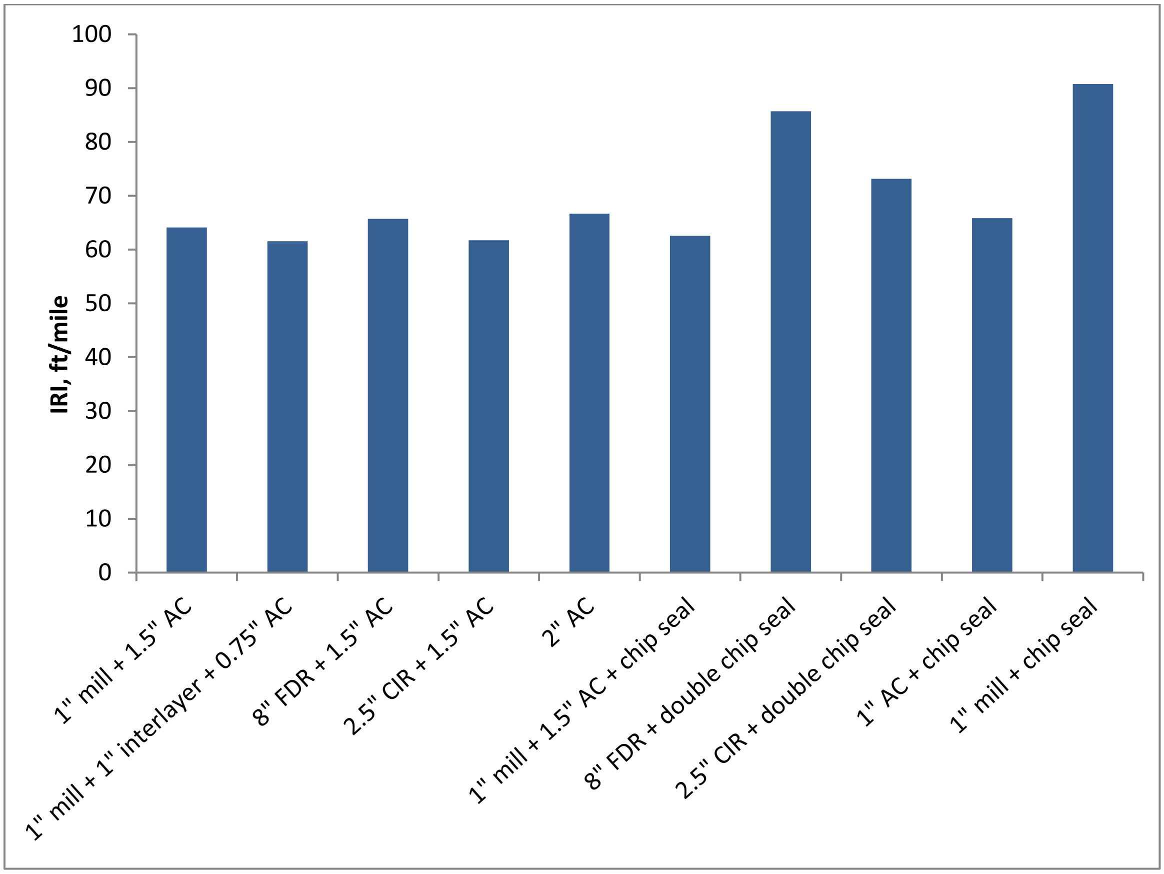


Figure 30. IRI measured from Roadroid

All sections were within the range of “good” (less than 95 in/mile) regarding surface roughness. The IRI values of Section 5 (FDR and double chip seal) and Section 10 (1 in. milling and chip seal) were higher than the IRI values of the other test sections. The average IRI was 88 in/mile for Sections 5 and 10 and was 65 in/mile for the other test sections.

LIFECYCLE COST ANALYSIS

A LCCA was conducted to evaluate the cost-effectiveness of the various holding strategy alternatives applied to the IA 93 test sections. These alternatives are summarized in Table 17.

Table 17. Rehabilitation alternatives and scheduled maintenance activities

Alternative	Treatment Method	Maintenance Activity	Expected Life Extension (yrs)
A	1 in. scarification + 1.5 in. AC overlay	Crack filling/sealing x 2	14
B	1 in. scarification + 1.5 in. AC overlay + single chip seal	Crack filling/sealing x 2	16
C	1 in. scarification + 1 in. interlayer course + 0.75 in. ultrathin AC overlay	Crack filling/sealing x 2	21
D	8 in. FDR + 1.5 in. AC overlay	Crack filling/sealing x 2	25
E	8 in. FDR + double chip seal	Chip seal x 2	18
F	2.5 in. CIR + double chip seal	Chip seal x 2	18
G	2.5 in. CIR + 1.5 in. AC overlay	Crack filling/sealing x 2	21
H	2 in. AC overlay	Crack filling/sealing x 2	15
I	1 in. leveling and strengthening course + single chip seal	Crack filling/sealing x 1	10
J	3 in. AC overlay	Crack filling x 3	22

AC = asphalt concrete
CIR = cold in-place recycling
FDR = full-depth reclamation

Based on the performance of the test sections, primarily the transverse crack reduction values, the life expectancies of the holding strategy treatments were estimated. Pre-planned maintenance activities such as crack filling and chip sealing were also included as part of the holding strategy alternatives. Chip sealing is scheduled for treatments that included a chip seal layer at construction in order to remedy the expected texture loss caused by traffic and snow plowing operations. This is consistent with the results of post construction observations. A control strategy, Alternative J, was also established in order to compare the cost-effectiveness of the proposed holding strategies to that of a conventional 3 in. AC overlay strategy. The following was considered in projecting expected life extension:

- Given the satisfactory performance of Alternative A. as of this writing, it is expected that its life will be at least twice its current age (2 x 7 = 14).

- Two years were added to the expected life of Alternative B in comparison to Alternative A to account for the added protection of the chip seal surface (14 + 2 = 16).

- Given the superior performance of Alternative C in crack transmission, five years were added to the expected life in comparison to Alternative B (16 + 5 = 21).

- Kim et al (2010) projected CIR pavements in Iowa would have a 21- to 25-year life for CIR sections with 3-in. AC overlays. Given that this pavement has excellent subgrade support, but the AC overlay was 1.5 in., using the low end of the range (21 years for Alternative G) seems advisable.

- Since the FDR sections have not experienced crack transmission, while the CIR sections have had a modest amount, it seems reasonable to project a 25-year life for the FDR section with a 1.5-in. AC overlay (Alternative D).

- Given the current condition of the pavement surfaces of Alternatives E and F, it seems reasonable that a chip seal every 6 years will be necessary to maintain them in acceptable condition. It seems reasonable after the life of two chip seal maintenance cycles have expired that theses sections would be considered to have reached a maximum life extension.

- Given the current condition of Alternative I, it seems reasonable to project a total life extension of 10 years.

- Data provided by the Iowa DOT indicates that a 3-in. functional AC overlay over full-depth asphalt can be expected to have an IRI of approximately 120 after 22 years of life. This test section pavement had a preconstruction IRI of 246 in/mi, which was clearly unacceptable. Given the preceding, an expected life extension of 22 years seems reasonable for Alternative J.

- Alternative H is an AC overlay with two-thirds the thickness of Alternative J, and the expected life extension of 15 years was projected because that is approximately two-thirds of 22 years.

- Two crack filling/sealing cycles were projected for pavements with approximately 15-year projected lives (A, B, and H) and one cycle was projected for Alternative I. Three cycles were projected for Alternative J during its 22-year life. Crack transmission has been delayed for Alternatives C, D, and G, so only two cycles are projected despite having projected lives of more than 20 years.

The cost of test section construction was used to estimate the costs of the alternatives. The equivalent annual cost (EAC) for each alternative was estimated, considering the projected life extension of each treatment and maintenance costs. Further detail regarding the estimates follows.

- The cost of bid items not directly associated with each test section, but necessary for the project (such as traffic control and mobilization) was allocated proportionally ~~be~~ using the direct cost of constructing each test section.

- The cost of a single chip seal per mile of road was estimated as being the difference in cost between Alternative A and B ($26,500).

- The Iowa DOT provided cost guidance suggesting that the cost of crack sealing/filling per mile is approximately 5% of the cost of Alternative J. This yielded a cost of $6,200 per mile.

- The applicable interest rate and inflation rate of the EAC calculations was set at 4%.

The costs for treatment construction and maintenance as well as the EACs are summarized in Table 18.

Table 18. Costs per road mile for alternative strategies

Alternative	Construction Cost ($)	Maintenance Cost ($)	EAC ($)	EAC Ratio to Alt. J
A	104,200	12,200	11,000	0.84
B	130,700	12,200	12,300	0.93
C	153,500	12,200	11,800	0.90
D	181,900	12,200	12,400	0.95
E	146,800	53,000	15,800	1.20
F	94,000	53,000	13,200	1.01
G	130,000	12,200	10,100	0.77
H	119,000	12,200	11,800	0.90
I	99,800	6,100	13,000	0.99
J	171,600	18,300	13,100	1.00

The cost estimation indicates that the construction costs for Alternatives A through I are 39% to 63% lower than the construction cost for Alternative J. Alternatives E and F show higher EACs in comparison to Alternative J, the conventional 3-in. AC overlay method. Compared to Alternative J, Alternatives C, D, and G provide comparable service lives and other alternatives provide shorter service lives. Therefore, alternatives A, B, and I can be used only as holding strategy treatments 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 alternatives involving CIR or FDR provide comparable service lives to the traditional rehabilitation method.

The EACs of all alternatives except E and F are lower than those of Alternative J. For these calculations, chip seal surfaces are expected to have higher maintenance costs than AC surfaces. For Alternatives E and F, the increase in maintenance costs offset the cost savings from the lower construction costs in comparison to Alternatives D and G, and the result is a higher predicted EAC.

Iowa has had very limited recent experience with double chip seals serving as a surface for FDR and CIR pavement sections on the primary network and limited experience on the secondary network. Long-term performance observations are needed to investigate how much life extension is possible for these alternatives.

The preceding projections were developed for roadways that have characteristics that are like those of IA 93: full-depth asphalt pavements with thicknesses of 4 to 8 in. having good subgrade support. The costs are provided according to the bid prices for this project in 2013. For comparison, tables and figures project relative cost ratios among the various alternatives that will likely remain applicable in the future if there are not relatively large changes in material, equipment, and labor costs among the alternatives.

CONCLUSIONS AND RECOMMENDATIONS

This report documents the construction and the first six years of performance of test sections on IA 93 that were established by the Iowa DOT to aid in the development of holding strategies that could postpone major rehabilitation or reconstruction for deteriorated low-volume asphalt pavements by utilizing treatments with relatively lower installation costs and reasonable lifecycle cost-effectiveness. Ten holding strategy treatments using various combinations of thin asphalt overlays, recycling technologies, and chip seals were constructed. The strategies' performance was evaluated by pavement condition surveys, in situ nondestructive structural tests, laboratory material tests, and various surface characterization tests.

Six pavement condition surveys were conducted, including one pre-construction survey and five post-construction surveys. The pre-construction survey showed that the existing pavement of IA 93 was suffering from severe non-load–related surficial distresses and edge breaks. The overall pavement condition was poor based on the pavement condition index. The post-construction surveys showed that the proposed holding strategy treatments successfully corrected longitudinal cracking, raveling, edge breaks, and rutting. The predominant distress type found in the test sections after construction was transverse cracking. These transverse cracks were formed over the winter after construction, and the density of the post-construction cracks did not considerably change during the six-year performance monitoring period. Field core samples suggest the transverse cracks in the CIR sections are new cracks in the surface layers. Field cores also confirmed that the transverse cracks in the other sections are reflective cracks that developed from the crack patterns that remained in the pre-construction pavement.

The recycling technologies, including CIR and FDR, were the most effective treatments for mitigating reflective cracking. The CIR and FDR sections had more than a 95% crack reduction compared to the cracking density before construction, and the cracks in the CIR and FDR sections were not reflective cracks. The CIR or FDR sections with a double chip seal surface exhibited comparable performance to the CIR or FDR sections with a 1.5 in. asphalt overlay, except for some loss of chip seal aggregate as discussed later.

The 2 in. asphalt overlay treatment and the treatment consisting of a 1 in. interlayer with a 0.75 in. ultrathin asphalt overlay reduced the cracking density in the existing pavement by 85% and 87%, respectively. The 1 in. milling and 1.5 in. asphalt overlay with chip seal treatment and the 1 in. leveling course with chip seal treatment had 20% and 55% crack reductions, respectively. The 1 in. milling and 1.5 in. asphalt overlay treatments had a 35% crack reduction value. All of the aforementioned cracks are of narrow width such that they are or can be easily sealed or filled as of this writing.

Loss of cover aggregate for the chip seal was also observed in the section constructed using FDR and a double chip seal treatment. Low bond strength between the chip seal surface and the FDR base is suspected to be the primary contributor to this type of surface defect. Additionally, some pothole-sized delamination was observed near bridge approach sections, where inadequate subgrade support might be an issue.

The DFT results indicate that the dry friction coefficient of asphalt surfaces was higher than that of the chip seals. The coefficient of friction of an asphalt surface in a wet condition was similar to that of a chip seal. The average dry friction coefficient was 1.04 for an asphalt surface and 0.88 for a chip seal surface. The average friction coefficient of all sections in the wet condition was 0.57.

The SPT results show that the chip seal surfaces have greater macro-texture than asphalt surfaces. The average MTD was 0.602 mm and 1.079 mm for the asphalt and chip seal surfaces, respectively. Considerably lower MTD measurements were found in the wheelpaths of the section that received the FDR and double chip seal treatment. The low MTD is considered to have resulted from the loss of cover aggregate.

The original pavement had an IRI value of over 200 in. per mile, which indicates poor rideability. The holding strategy treatments considerably improved rideability. All sections exhibited good surface roughness (IRIs less than 95 ft/mile). The IRI values of the FDR with double chip seal treatment and 1 in. milling with chip seal treatment were higher than those of the other test sections.

The effective structural numbers of the constructed test sections were estimated using FWD and E* tests. FWD tests were conducted in October 2012, November 2013, and September 2015 and provided field pavement structural assessments for the test sections before the construction of the holding strategy treatments, shortly after construction, and two years after construction, respectively. The FWD results indicate that aged pavement with severe surficial distresses on low-volume roads can retain a high structural capacity due to its high stiffness, which likely results from the compaction of materials by traffic loading and oxidization of the asphalt. Newly constructed pavement layers can potentially decrease the average stiffness of the pavement, resulting in a decrease in the pavement's structural capacity. Increased layer thickness can effectively offset this influence on pavement structure. However, treatments that include a recycled layer, such as CIR or FDR, may considerably lower the load carrying capacity of the pavement. An increase in pavement stiffness was observed two years after construction of the test sections. Sections that included an FDR layer exhibited the greatest improvements in stiffness. Therefore, it is recommended that, although the holding strategies do not considerably change the long-term pavement structural capacity, caution should be exercised with regard to heavy traffic loading shortly after the treatments are constructed, especially for treatments using CIR or FDR.

The projected lifecycle costs of the various holding strategy treatments were estimated and compared to that of a traditional 3-in. overlay method. The LCCA results indicate that the EAC of the CIR and FDR and double chip seal methods are higher than the EAC of the 3-in. overlay strategy. The lifecycle costs of the other holding strategies are lower than or equal to the lifecycle cost of the 3-in. overlay method. However, with more information on longer term performance and maintenance costs, the LCCA cost estimates may change considerably.

Based on the findings from this investigation, the following conclusions can be drawn, and recommendations can be made. (Note that these are limited to this set of test sections, which were applied to a full depth asphalt pavement with good subgrade support).

- Reflective transverse cracking is the primary early-age distress type for the holding strategy treatments involved in this study.

- The effectiveness of the methods in preventing reflective cracking, from the most effective to the least effective, are CIR or FDR, high-quality asphalt material (including interlayer), 2-in. AC overlay, leveling course and chip seal, and 1.5-in. mill and fill—with and without chip seal surface.

- FDR alternatives exhibited relatively high costs for these test sections; however, using this method for the relatively thick full-depth asphalt pavement section on IA 93 is challenging. FDR should still be considered for applications with thinner pavement sections with thick bases and possibly more modest subgrade support.

- From a safety perspective, the functionality of a chip seal is comparable to that of an asphalt surface. However, a chip seal has higher macro-texture than an asphalt surface, which can lead to an increased noise level and tire wear.

- A chip seal applied to an FDR or CIR layer in this investigation was susceptible to damage from snow plowing and traffic and may require frequent maintenance activities and increased maintenance costs. Providing a better bond between the chip seal and FDR or CIR surface would mitigate this concern. Also, some areas of delamination were observed near bridge approaches where subgrade support may be inadequate. While this method provides lower construction costs, the projected need to maintain the surface with further chip seals increase the EAC so that it exceeds those of other treatments.

- CIR or FDR with a thin asphalt overlay were projected to provide the longest life extension since they have developed few cracks so far. Also, negligible rutting was observed in these test sections. The CIR treatments have relatively low construction and projected lifecycle costs.

- All of the holding strategies extended provided life extension to IA 93 from 2013 through 2020. Construction costs ranged from $94,000 to $181,900 per road mile. Projected EACs. including maintenance, ranged from $10,100 to $13,200 per road mile, except for the FDR with a double chip seal, which was projected at $15,800. Projected life extensions ranged from 10 to 25 years.

This work provides decision makers a variety to of possible holding strategy choices with various construction costs, lifecycle costs, maintenance requirements, and surface characteristics from which to choose.

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