Field Assessment of Cold Recycling Technologies Used for Pavement Preservation

by

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ABSTRACT

Cold recycling of asphalt concrete pavements is a widely practiced technique used to prolong pavement life as a low-cost preservation method. Research in recycled materials has become necessary to further evaluate the material properties and performance of this sustainable technology. The National Center for Asphalt Technology (NCAT) has constructed several test sections as part of its Pavement Preservation Study along Highway US-280 near Opelika, Alabama. The Highway US-280 preservation sections were built in the summer of 2015, and part of the experiment included four different sections using cold recycled techniques surfaced with a one-inch thin overlay. These cold recycled (CR) asphalt pavement included Cold In-place Recycling (CIR) and Cold Central Plant Recycling (CCPR), each one with asphalt emulsion and foamed asphalt as recycling agents. The functional and structural performance of the pavement recycling techniques was evaluated periodically. Rutting and IRI measurements were taken biweekly, deflections were taken quarterly, and crack maps were generated quarterly to assess the influence of CR pavement sections in the thin overlay surface.

Field performance measurements showed that the use of cold recycled materials influenced rut depths, falling into the fair threshold of the MAP-21 rating system during the first four and a half years of service. Cracking increase was detected in the last year of this study in the CCPR-emulsion section. Roughness of the section slightly increased over time in most of the sections.

FWD backcalculated modulus indicated the cold recycled sections have temperature-dependent behavior, with less temperature susceptibility in the CCPR foamed section. The obtained results were used to evaluate the structural contribution of the recycling technologies from a pavement design perspective. Based on empirical pavement design, it was determined that the recommended structural layer coefficients of the recycled materials ranged between 0.23 and 0.35.

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LIST OF ABBREVIATIONS

DOT	Department of Transportation
US	United States
NCHRP	National Cooperative Highway Research Program
FHWA	Federal Highway Administration
TRIP	National Transportation Research Nonprofit
RAP	Reclaimed Asphalt Pavement
NAPA	National Asphalt Pavement Association
ARRA	Asphalt Recycling and Reclamation Association
HR	Hot Recycling
HIR	Hot In-place Recycling
CR	Cold Recycling
CIR	Cold In-place Recycling
CCPR	Cold Central Plant Recycling
AC	Asphalt Concrete
SGC	Superpave Gyratory Compactor
ITS	Indirect Tensile Strength
OMC	Optimum Moisture Content
ER	Expansion Ratio
HL	Half-Life
ITS	Indirect Tensile Strength

TSR	Tensile Strength Ratio
PMS	Pavement Management System
NCHRP	National Cooperative Highway Research Program

CHAPTER 1: INTRODUCTION

1.1. BACKGROUND

The National System of Interstate and Defense Highways, now known as the interstate highway system, was established in 1956 by President Eisenhower, who signed the Federal-Aid Highway Act law. This Act proclaims that the federal government would pay for 90 percent of the cost of construction of Interstate Highways. The system had the goal to serve auto, truck, and strategic military needs. These paved roads were designed based primarily upon experience to meet relatively low traffic levels for design periods of 20 to 40 years. Since that time, heavy traffic volumes increased due to population growth and economic development, which conclude that many of these pavement structures have exceeded their original design life (National Research Council 2005).

During the expansion of the roadway network, the initial cost was the most important factor for construction decisions. Long-term cost analysis for maintenance and rehabilitation were not relevant for decision-making. As a result, nowadays, America's roads are often crowded, frequently in poor condition, and underfunded. According to TRIP (National Transportation Research Nonprofit), 21% of the nation's highways had poor pavement condition, and roads had a significant backlog of $420 billion in repairing existing highways in 2015. Owner agencies faced the reality of having insufficient funds to repair and maintain their pavement networks (American Society of Civil Engineers 2017).

As a response to this issue, pavement preservation was introduced to make better use of available funding. The concept of pavement preservation is to prioritize keeping reliable roads in good condition while spending minimum resources on roads in poor condition. Pavement preservation includes preventive maintenance, minor rehabilitation, and some routine maintenance activities that, when applied at the right time, can restore the function of the existing system and extend its service life (Asphalt Recycling & Reclaiming Association 2015).

The funds typically available for maintenance and rehabilitation have been below the minimum required to improve or at least maintain the overall condition of the pavement network. The primary funding source of highways is the federal motor fuel tax of 18.4 cents per gallon for gasoline, which has not increased since 1993. This factor, added to inflation over the years, has resulted in a search for new materials and technology methods to maintain the road network by effective resource management. Sustainable pavements have risen as an answer to this issue.

A sustainable pavement is a safe, efficient, and environmentally friendly road structure capable of meeting the needs of present road users without jeopardizing future generations (Chappat and Bilal 2003). The asphalt industry has been developing improved technologies and construction methods to minimize environmental impacts, maximize economic profits, and meet societal goals. This approach is called the triple-bottom line sustainable method (Asphalt Paving Association of Iowa 2008). The use of Reclaimed Asphalt Pavement (RAP) for recycling pavement techniques is an innovative technology in the asphalt industry.

The asphalt binder is the most expensive and economically variable material in an asphalt mixture. During the Arab oil embargo, the cost of crude oil rose, which, combined with savings through material replacement, generated an increase of RAP popularity in the 1970s. The Federal Highway Administration (FHWA) supplied partial funding to state agencies to build paving projects using RAP and to document the results. Therefore, guidelines for pavement recycling were generated during the late 1970s, 1980s, and 1990s (Copeland 2011).

According to the National Asphalt Pavement Association (NAPA) survey in 2018, asphalt pavement is the most common source of recycled material in North America. The survey data reports more than 82.2 million tons of RAP. Producers stated that the use of 98% of RAP is for new construction, pavement preservation, and rehabilitation. Also, 6.4 million tons of RAP were used as aggregate. At year-end 2018, some 110.3 million tons of RAP was estimated to be stockpiled for future use across the country. Asphalt recycling and reclaiming strategies allow agencies to achieve savings in energy consumption, reduce greenhouse gas emissions, and to use less non-renewable natural resources (Williams and Willis 2019).

Even though many recycling techniques are continuously developed in the industry, the Asphalt Recycling and Reclamation Association (ARRA) states that asphalt recycling and reclaiming methods can be categorized in five overall groups for practical purposes: (1) Hot Recycling, (2) Cold Recycling, (3) Full-Depth Reclamation, (4) Hot In-place Recycling, and (5) Cold Planing. Besides Hot Recycling, the remaining four categories have some sub-categories which further describe pavement recycling depending on the specific method or process used to obtain the recycled material. It is also is common to combine the use of different techniques in the same project (Asphalt Recycling & Reclaiming Association 2015).

ARRA defines Hot Recycling (HR) as the process of combining RAP with new or “virgin” aggregates, new asphalt binder, and recycling agents in a central plant to produce a recycled asphalt mixture. Also, Hot in-place Recycling (HIR) is defined as the process of reclaiming the existing asphalt pavement structure on-site, by a heating and softening process that allows scarifying or loosening the existing pavement, which is then thoroughly mixed. The use of RAP has been mostly applied to HR and HIR because of the lack of guidance on RAP use and documented information about the successful long-term performance of cold recycled mixes. Nevertheless, as asphalt binder costs continuously increase, and more emphasis is given to sustainable technologies, the asphalt community is reassessing the use of RAP in Cold Recycling (Vargas-Nordcbeck and Timm 2013).

Cold Recycling (CR) consists of recovering and reusing the existing asphalt pavement layers without the application of heat (Wirtgen Group 2012). This technique is divided into two subcategories: Cold In-place Recycling (CIR) and Cold Central Plant Recycling (CCPR). The two most commonly used recycling agents in cold recycled mixes are foamed asphalt and emulsified asphalt. Cold Recycling of asphalt concrete pavements is a practice that has been performed to prolong pavement life as a low-cost preservation method. Also, cold asphalt recycling falls into the three-bottom line approach of sustainability. CR is one way of increasing the effectiveness of existing budgets to maintain, preserve, rehabilitate and reconstruct more miles of roadway for each dollar spent, optimize the use of natural resources, reduce impacts on the greenhouse effect, improve health, and save money to taxpayers.

Cold Recycling techniques have become more prevalent by many highway agencies (Diefenderfer and Apeagyei 2014), which have gained experience by employing cold recycled mixes as a base layer in their roads as a part of a pavement rehabilitation program using thick hot mix asphalt (HMA) in the top layers. As no extensive studies have been performed on Cold Recycling as a pavement preservation approach, this research focuses on the structural and functional performance of cold recycled sections using a thin hot mix asphalt overlay as a surface layer.

1.2. OBJECTIVES

Given the need to better comprehend and model performance of cold recycled materials in pavement preservation, the general objective in the thesis is:

1. Evaluate the structural and functional performance of cold recycled asphalt pavement as a pavement preservation technique.

Additionally, other objectives of the research include:

2. Determine the structural contribution of CCPR and CIR; and
3. Compare the performance of the two recycling techniques- Cold In-place Recycling (CIR) and Cold Central Plant Recycling (CCPR), using foam and emulsion as a recycling agent.

1.3. SCOPE

To accomplish these objectives, four test sections were built along Highway US-280 in Opelika, Alabama, in September 2015. A total of four recycled sections were placed, including cold central-plant recycling with emulsified and foamed asphalt binders (i.e., CCPR-E and CCPR-F), and cold in-place recycling with emulsified and foamed asphalt binders (i.e., CIR-E and CIR-F), both technologies include an active filler. All the sections have 100% RAP, and a thin 1-inch overlay was placed as a surface layer.

Falling weight deflectometer (FWD) testing was performed to quantify the seasonal behavior of the pavement layer moduli. Roughness, cracking, and rutting was measured and analyzed throughout the experiment to determine if the recycling techniques and materials affect the functional and structural performance. Moduli backcalculation was estimated using the software ELMOD 6.

The data and observations presented in this study are part of the National Center for Asphalt Technology (NCAT) Pavement Preservation Group (PG) Study. This research effort is funded by multiple State Departments of Transportation (DOTs), the Federal Highway Administration (FHWA), and the Foundation for Pavement Preservation (FP2 Inc.) and aims at determining the life-extending benefit of various pavement preservation treatments under varying conditions.

1.4. ORGANIZATION OF THESIS

This thesis is structured into five chapters:

- Chapter 2. A literature review that focuses on the concept of Cold Recycling and the importance it has nowadays, a brief description of the different mix design methods, the different recycling agents, construction practices, performance of previous cold recycled mix projects, and past research related to assessing the structural behavior of cold recycled mixes.

- Chapter 3. Summarizes the methodology, which describes the experimental plan of the investigation, the Preservation Group (PG) study, the test location and its features, the four different mix designs, the construction process of the recycling mixes, data collection, and performance evaluation of the sections.
- Chapter 4 includes a summary of results from the structural and functional performance of the four test sections with a discussion of main findings, as well as an analysis of the structural characteristics of the recycled materials based on the results obtained from backcalculation, and its use for pavement design methods.
- Chapter 5 provides the main conclusion of the thesis and some recommendations for future research on cold recycled mixes.

CHAPTER 2: LITERATURE REVIEW

The Federal Highway Administration (FHWA) issued a "Memorandum on Pavement preservation Definitions" to provide clarification and consistency in the interpretation and evaluation of pavement preservation programs. According to FHWA, pavement preservation consists of "work that is planned and performed to improve or sustain the condition of the transportation facility in a state of good repair. Preservation activities generally do not add capacity or structural value, but do restore the overall condition of the transportation facility." (Waidelich 2016). Pavement preservation can include routine maintenance, preventive maintenance, and minor rehabilitation strategies, as shown in table 1.

Table 1. Pavement Preservation Guidelines (Geiger 2005)

	Maintenance & Rehabilitation Category	Type of Activity	Increase Capacity	Increase Strength	Reduce Aging	Restore Serviceability
	Construction	New Construction	X	X	X	X
		Reconstruction	X	X	X	X
	Rehabilitation	Major (Heavy) Rehabilitation		X	X	X
		Structural Overlay		X	X	X
Pavement Preservation		Minor (Light) Rehabilitation			X	X
	Maintenance	Preventive Maintenance			X	X
		Routine Maintenance				X
		Corrective (Reactive) Maintenance				X
		Catastrophic Maintenance				X

Preventive maintenance consists of any activity that is intended to preserve or extend the service life of pavement until a significant rehabilitation or complete reconstruction is required. As the pavement condition deteriorates, there comes the point when maintenance activities are no longer cost-effective, and rehabilitation is needed. Rehabilitation techniques are more expensive than maintenance activities, but the rehabilitated pavement condition will generally be equivalent to what was achieved during the initial construction. Activities that improve or restore the structural capacity of a pavement by adding or recycling the top layers of the pavement structure can be divided into major (Heavy), structural overlay, and minor (Light).

Preservation treatments are usually less expensive than other strategies, and when applied at the right time, they can prolong the performance life of the pavement resulting in the highest return on investment. Owner agencies that have a pavement management plan usually allocate funds to pavement preservation instead of only rehabilitation projects (Asphalt Recycling & Reclaiming Association 2015). The asphalt recycling and reclaiming strategies may include:

- Cold Planing or milling, where distinctive design equipment removes an existing asphalt pavement to the desired depth, longitudinal profile, and cross-slope. This technique can be used to restore friction, reduce roughness, or to eliminate the oxidized and distresses surface of an existing pavement.
- Cold Recycling, which is the reuse of the existing pavement after milling to a depth as thin as 2 inches and up to 5 inches, sizing and mixing the material with a bituminous recycling agent, and then placing and compacting the recycled mixture.
- Hot In-Place Recycling, which includes the process of heating, softening and loosening the existing asphalt pavement followed by mixing, typically with a rejuvenating agent,

and then placing and compacting. This method corrects oxidation, minor cracking, and other defects in the upper layer of the pavement.

- Full Depth Reclamation, a rehabilitation technique that pulverizes the entire thickness of the asphalt pavement and a defined portion of the underlying materials. A stabilizing agent that can be mechanical, bituminous, or chemical is added if required. The reclaimed sections are blended to provide an improved road base; an asphalt overlay is usually placed as the surface course.

This literature review focuses on Cold Recycling as a pavement preservation approach. This method consists of recycling asphalt pavement without the application of heat during the recycling process. When an asphalt overlay is applied on top of the recycled mixture, the activity is considered a minor rehabilitation (Asphalt Recycling & Reclaiming Association 2015). Asphalt recycling has several unique advantages over the traditional asphalt concrete (AC) overlay rehabilitation methods, which include:

- Reuse and conservation of non-renewable natural resources.
- Preservation of existing roadway geometry and clearances.
- Corrections to pavement profile and cross-slope.
- Improved pavement smoothness.
- Improved pavement physical properties by modification of existing aggregate gradation, and asphalt binder properties.
- Mitigation or elimination of non-load associated cracking.

2.1. COLD RECYCLING MIX DESIGN

Most State Highway Agencies do not have developed their own CR specifications. The practices range from simple empirical formulas to more sophisticated techniques with performance-based testing. The most straightforward practices are based on the amount and consistency of the recovered asphalt binder to predict an initial recycling additive content. The sophisticated methods include testing such as resilient modulus, stability, and moisture sensitivity tests. Testing often includes short-term and long-term curing conditions (Eller and Olson 2009; Kim et al. 2007; Muthen 1999).

The ultimate objective of mix design is to identify the most effective proportion of materials and achieve optimal pavement performance. Cold Recycling mix design procedures are based on different compaction methods; in general, the most popular are Marshall compaction and the Superpave Gyratory Compactor (SGC) (Gao et al. 2014; Liu et al. 2012; Scholz et al. 1990). The mixture tests also include a combination of Marshall stability and indirect tensile strength (ITS) under wet vs. dry conditions. The methods shown in this chapter are based on the manuals from the American Reclaiming and Recycling Association, Wirtgen, and the Asphalt Academy (Asphalt Academy 2009; Asphalt Recycling & Reclaiming Association 2015; Wirtgen Group 2012). The two targets to be determined from any type of mix design are the optimum moisture content (OMC) to achieve the maximum compacted density, and the second target is the optimum recycling agent content to fulfill the minimum strength criteria. The standard procedure of the mix design includes the following steps:

2.1.1. Sampling and RAP properties evaluation

The mix design procedure requires optimization not only in terms of volumetric and compaction characteristics but also involves the consideration of the physical properties of the RAP along the length, width, and depth of the road. It is thus essential that the material samples used during the mix design be representative of the materials in the asphalt layer recycled with a bituminous agent. When roadway sampling is required, sections with significant differences in materials should be treated as different sampling units (ARRA 2017).

There is a misconception that samples from RAP stockpiles are highly variable, so 100% of RAP content mixes will lead to more variability in the recycled mixture (West 2015). Nevertheless, Nady states that well-managed RAP stockpiles have a more consistent gradation than virgin aggregates (Nady 1997), and it was confirmed with data gathered by NCAT that RAP processed from different sources can also be just as stable in gradation and asphalt content as millings (West 2009). The most accurate way to sample existing RAP stockpiles is with the assistance of a front-end loader, as described in Section 5 of AASHTO R90 Standard Practice for Sampling Aggregate Products.

RAP is typically stored in one unfractionated stockpile, or two different stockpiles with an only coarse or fine fraction. In a research project in Minnesota, Eller and Olson (2009) suggested separating RAP into three different fractions and blend before mix design to reduce variability and segregation of the aggregates.

The tests needed from RAP for cold recycled mix design are asphalt binder content of the RAP, black-rock gradation, percent of fines (passing #200 sieve), flat, and elongate ratio, and the optimum water content. The University of Nevada Reno partnered with NCAT to evaluate several options for testing RAP properties (Hajj et al. 2012), three methods were used to determine asphalt contents and recover the aggregates for aggregate property tests: the ignition method, the centrifuge extraction method, and the reflux extraction method. The study results indicated that the ignition method yielded the most accurate asphalt contents for the RAP and provided the lowest testing variability compared to solvent extraction methods.

Different design manuals have distinct gradation test methods. The gradation test method for RAP/aggregate suggested by Wirtgen follows the ASTM D422 *Standard Test Method for Particle-Size Analysis of Soils with minor change* (Wirtgen Group 2012). ARRA suggests ASTM C136 *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates* and ASTM C117 *Standard Test Method for Materials Finer than No. 200 Sieve in Mineral Aggregates by Washing* to perform sieve analysis (ARRA 2016, 2017), and the gradation band recommended by Asphalt Academy follow South African National Roads Agency specifications (Asphalt Academy 2009).

Wirtgen suggests running a moisture/density relationship in the RAP. Different water contents are needed to calculate the optimum moisture content; this process allows a high density level to be achieved when the recycled material is compacted (Wirtgen Group 2012).

2.1.2. Recycling Agent

The two types of bitumen used as recycling agents in cold recycled mixes are foamed asphalt and emulsified asphalt. They are mixed with RAP at ambient temperature, and their stabilizing mechanism is different.

2.1.2.1. Emulsified Asphalt

An emulsion is a dispersion of small droplets of one liquid in another liquid. Asphalt emulsion is the dispersion of asphalt binder in water by an emulsifying agent. Figure 1 shows the manufacturing process of emulsion, where soap, water solution, and asphalt binder are blended by a colloid mill that shears the binder into microscopic particles. Asphalt emulsion can be classified according to the sign of the charge on the droplets as cationic (positive charge) or anionic (negative charge). Also, emulsions are categorized by the time to react with the aggregate; they can be rapid-setting (RS), medium-setting (MS), and slow-setting (SS). Emulsions are named according to ASTM D977 *Standard Specification for Emulsified Asphalt* and D2397 *Standard Specification for Cationic Emulsified Asphalt* and followed by numbers and text indicating the emulsion viscosity and residue properties, for example, a cationic RS of high viscosity and hard residual asphalt is denoted by the code CRS-2H (James 2016).

Emulsion “break” is a term used to describe the separation of the asphalt binder from the water after a set period following contact with the RAP mixture. The emulsion works as a lubricating agent, and after the break, the residual asphalt keeps the original properties of the virgin binder. The emulsion binder has the characteristics of coating the mixture with a thin film and bring initial strength to the RAP (Salomon 2016).

Figure 2 shows how the emulsion (represented by the blue color) disperses preferentially amongst the finer particles (represented by the black color) with the opposite charge, but not exclusively because it covers partially coarser aggregates (shown in gray). A chemical bond between the asphalt bitumen and the aggregate is promoted by the emulsifier (Asphalt Academy 2009).

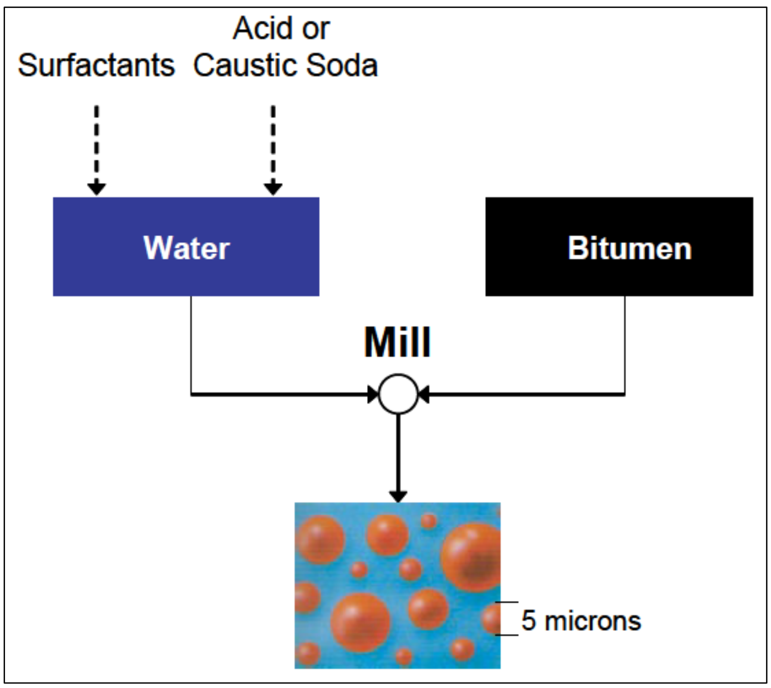

Figure 1. Manufacture of asphalt emulsion (Wirtgen Group 2012)

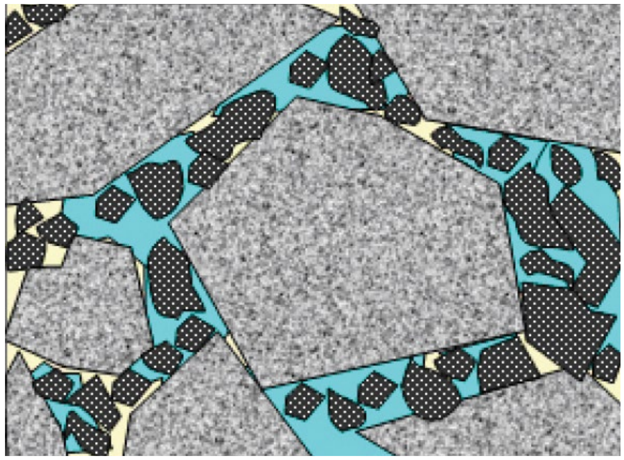

Figure 2. Aggregate and asphalt emulsion bond (Asphalt Academy 2009)

According to Gao et al. (2014), slow-setting asphalt emulsions are more compatible with the RAP materials and have long workability time to make good dispersion in the mix. Ameri and Behnood (2012) studied the effects of steel slag on the properties of cold recycled mixtures; the results showed that the anionic bitumen emulsion enhanced the compatibility with marginal materials effectively. The Nevada Department of Transportation has usually used CMS-2 for CR projects (Sanjeevan et al. 2014). Then, different types of emulsions can change for different states and regions, so it is essential to determine which type of emulsion will perform best for a particular application.

2.1.2.2. Foamed Asphalt

Foamed asphalt was initially developed by Csanyi in 1957 for full-depth reclamation to reduce the viscosity of asphalt binder to be able to mix with the reclaimed material (Csanyi 1957). Nowadays, the process consists of injecting a small amount of water into hot asphalt as it is combined with the recycled materials (Figure 3). When the hot binder and water blend, the asphalt expands as the water turns to steam, creating a thin asphalt film of about ten times more coating potential (Stroup-Gardiner 2011).

In Figure 4, the foamed asphalt content (represented by the blue color) is usually too low to coat all of the aggregate particles thoroughly, it distributes exclusively to the finer particles, producing “spot welds” of a mastic of bitumen droplets and the fines (represented by the black color), therefore the fines particles and passing #200 have significant role to form foam dispersion. The coarse RAP particles work as a structural skeleton (represented by the gray color). Another factor to consider for asphalt foam dispersion is the RAP moisture content before mixing (Schwartz and Khosravifar 2013).

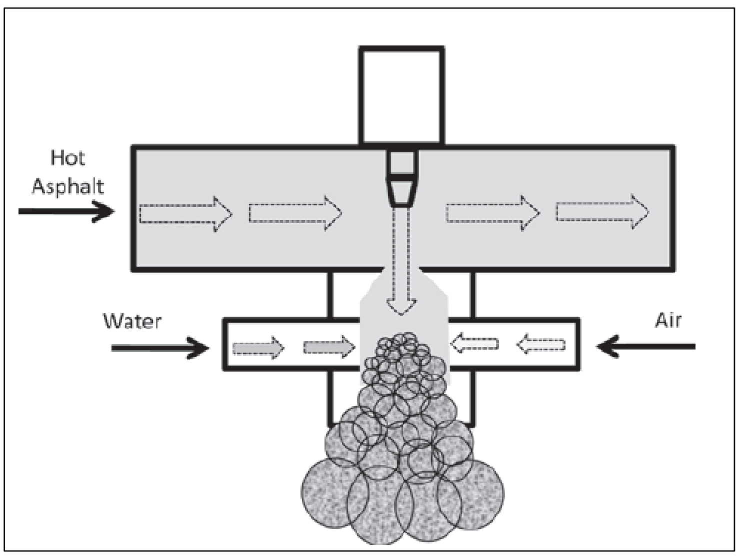

Figure 3. Foamed asphalt process (Wirtgen Group 2012)

Foamed asphalt is produced in a specialized device that can heat binder to a specific temperature and pressurize the binder, air, and water at the desired pressure. The foaming process causes an expansion of asphalt binder, reducing surface tension and viscosity. The two key parameters to evaluate foaming are the expansion ratio (ER), and half-life (HL).

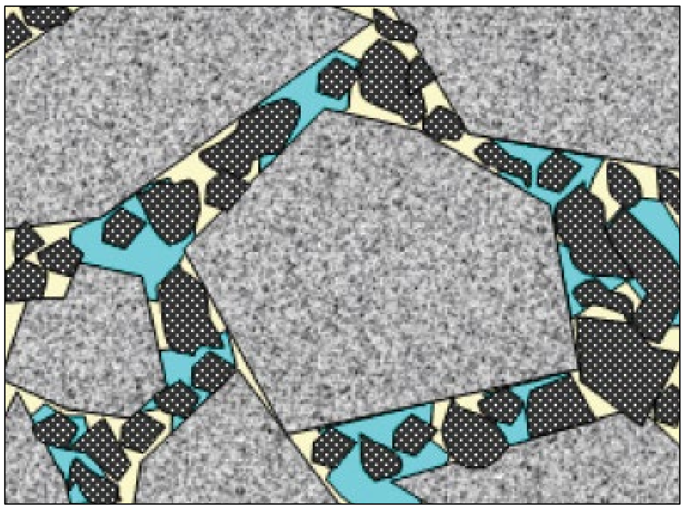

Figure 4. Aggregate and foam asphalt bond (Asphalt Academy 2009)

These characteristics are used to determine the required binder temperature and foaming water content to achieve adequate foaming. The expansion ratio is described as the ratio between the maximum volume of the foam relative to the original amount of asphalt. The HL is the time duration for the foamed binder to collapse into half of its maximum capacity (Asphalt Academy 2009; Wirtgen Group 2012).

More significant expansions and longer half-life can provide a better dispersion for the granular materials (David Newcomb et al. 2015). The quality of the foamed asphalt mix is deeply related to the quality of the foam, which is measured by the expansion ratio and the half-life, and 100% RAP mixes can be prepared with lower percentages of fines (Marquis et al. 2002). A summary of the main differences between the recycling agents is shown in Table 2.

Table 2. Comparison between recycling agents (Wirtgen Group 2012)

Factor	Recycling agent	
	Emulsion	Foamed
Asphalt mixing Temperature	20°C to 70°C	160°C to 180°C (Before foaming)
Aggregate temperature during mixing	Ambient (>10°C)	Ambient (>15°C)
Moisture content during mixing	OMC plus 1% minus emulsion addition	70% to 90% of OMC
Aggregate coating	Coating of finer particles (some coarse particles).	Coating of finest particles only. Increased cohesion from the bitumen/fines mortar
Construction and compaction temperature	Ambient (>5°C)	Ambient (>10°C)
Air Voids	10% to 15%	10% to 15%
Rate of initial strength gain	Slow (moisture loss)	Medium (moisture loss)
Modification of bitumen	Yes	No
Important bitumen parameters	*Emulsion type *Residual bitumen *Breaking time	*Expansion ratio *Half-life

2.1.3. Active Filler

The term for the filler that chemically modified the mix properties is called active filler. They can be cement, hydrated lime, fly ash and other. They work improving adhesion between the RAP and emulsion, improving dispersion in foam recycling mix by increasing the number of fines, increase stiffness and accelerated curing of the cold recycling mix. Commonly, the maximum amount of cement is 1%.

2.1.4. Mixing and Compacting

The batch of the materials depends on the amount needed by the design methodology, before mixing the RAP has to be set to mixing temperature, generally at room temperature. The temperature of the emulsion is the one recommended by manufacturer specifications. Mixing of test specimens can be done manually, with a mechanical bucket mixer, or with a laboratory size pugmill. Moisture added to the mix is typically 1.5 to 3.0% (ARRA 2016). The RAP is compacted at different emulsion contents to estimate the optimum emulsion content.

Specimens for Marshall testing shall be compacted to 4 inches (100 mm) in diameter, using either 75 blows per side by a Marshall hammer or with 30 gyrations using a Superpave gyratory compactor (SGC) at 1.25° angle, 87 psi stress. Thirty gyrations are broadly accepted in many states, including California, Illinois, Iowa, Kansas, and many others except for Texas, which specifies 35 gyrations. Lee and Kim (2003) selected a compaction effort of 25 gyrations as the gyration level that would achieve a similar density as the one with a 75-blow Marshall hammer.

Foamed asphalt has to be set to the temperature defined previously during ER and HL test. A proper mixing time between 20 and 60 seconds for foamed mixes is suggested (Gu et al. 2019). The Wirtgen manual recommends mixing the materials at a moisture content range from 70 to 90% of the OMC determined by the standard Proctor test (Wirtgen Group 2012). Lee (1981) recommended using a moisture content equal to 65 to 85 percent of the OMC.

Different methods have been used to compact cold recycled foamed asphalt mixtures for design. The methods include the modified proctor test method, Marshall hammer compaction of 75 blows that is popular in Maryland (Schwartz and Khosravifar 2013), gyratory compaction method generally at 30 gyrations and others. According to Kim et al. (2007), using a gyratory compactor rather than a Marshall hammer produces more consistent mixtures for various foamed asphalt contents.

2.1.5. Strength and Moisture testing

A total of 6 specimens at each emulsified asphalt content are prepared for Marshall stability testing, three for dry specimens, and three moisture conditioned specimens. For the Marshall stability test, the moisture-conditioned samples are soaked in a 25 ± 1°C water bath for 23 hours, followed by a 1-hour soak in a 40 ± 1°C water bath. The Marshall stability ratio is calculated by dividing the average conditioned Marshall stability by the average dry Marshall stability for each emulsified asphalt content (AASHTO 2017a). A required minimum retained Marshall stability ratio value is typically 0.70. Table 3 shows a summary of the mix design requirements (ARRA 2016; Wirtgen Group 2012)

Table 3 Recommended mix design requirements for Emulsion Mix (AASHTO 2017b)

Test Method	Criteria	Property
High-Temperature Validation		
Marshall Stability AASHTO T 245 (ASTM D6927)	Minimum 1,250 lb. (5,560 N)	Cured Stability
Retained Marshall Stability based on Moisture Conditioning	Minimum 0.70	Resistance to Moisture Induced Damage
Evaluation of Existing Binder		
Recovery of Binder from RAP AASHTO T 319 (ASTM D5404)	Used for Penetration Testing	Recovery of the binder
Penetration of Bituminous Materials AASHTO T 49 (ASTM D5)	Report Only	Softness of Existing Binder

The Indirect Tensile Strength (ITS) is tested for foamed asphalt mix specimens; this procedure follows AASHTO T283. Moisture conditioning is conducted on three compacted, cured specimens at each foamed asphalt recycling agent content, and then samples are submerged in a 25 ± 1 °C water bath for 24 hours and tested immediately after removal from the water bath. TSR is a design criterion defined as the average moisture conditioned specimen strength divided by the average dry specimen strength. Table 4 shows other mix design criteria according to ARRA guidelines.

Table 4. Recommended mix design requirements for Foamed Mix (ARRA 2017)

Test Method	Criteria	Property
Indirect Tensile Strength AASHTO T 283 (ASTM D4867)	Minimum 45 psi (310 KPa)	Cured Strength
Tensile Strength Ratio based on Moisture Conditioning AASHTO T 283 (ASTM D4867)	Minimum 0.70	Resistance to Moisture Induced Damage
Evaluation of Existing Binder		
Recovery of Binder from RAP AASHTO T 319 (ASTM D5404)	Used for Penetration Testing	Recovery of the binder
Penetration of Bituminous Materials AASHTO T 49 (ASTM D5)	Report Only	Softness of Existing Binder

2.2. COLD RECYCLING CONSTRUCTION

2.2.1. Cold In-place Recycling

Cold In-place Recycling (CIR) is a maintenance/rehabilitation process that occurs within the roadway. This method pulverizes and recycles the top of the asphalt layer using a continuous train operation then mixes the RAP with a recycling agent and repaved in place. The thickness is often between 50-125 mm (2-5 inches). CIR uses 100 percent of the reclaimed asphalt produced during the milling process. Also, CIR significantly reduces materials trucking, natural resources, and lower project costs (Asphalt Recycling & Reclaiming Association 2015; Cross and West 2018; Gu et al. 2019).

The CIR method has the advantage of being able to repair a variety of pavement distresses such as potholes, rutting, and cracks, prolonging the service life of asphalt pavement, and improving ride comfort. Moreover, this process reduces energy consumption, shortens the construction project period due to high production rates, and relieves environmental pollution by using a high percentage of the recycled materials (Wirtgen Group 2012; Xiao et al. 2018). Another benefit is that CIR would not result in raising the grade; therefore, this process can be applied to projects with various entrances, side roads, and intersections (Lane and Kazmierowski 2005).

CIR requires various recycling agent tankers, cold planing devices, crushing/screening or sizing units, mixers, pavers, and rollers. A train is defined as the combined equipment spreads out over a considerable distance. Depending on the project's scope, the CIR process can be a single-train, two-unit, or multi-unit CIR train. The most common is a single unit train that does not contain screening and crushing units; the mixture can be placed by a screed attached to the individual unit or using an asphalt paver, as shown in Figure 5.

CIR can vary in the process of how RAP is obtained and sized, the recycling agents and additives employed, and how the mixture is placed (Asphalt Recycling & Reclaiming Association 2015; Cross and West 2018). Unlike the HIR recycling trains, one or two nurse trucks are usually in front of the recycling profiler and mixer unit to provide a continuous supply of liquids for the mix (recycling agents, water). The recycling unit mills, processes, and mixes the recycled materials and then transfers them to a paver. Standard compaction practices are used to place and compact the mixture (Stroup-Gardiner 2011).

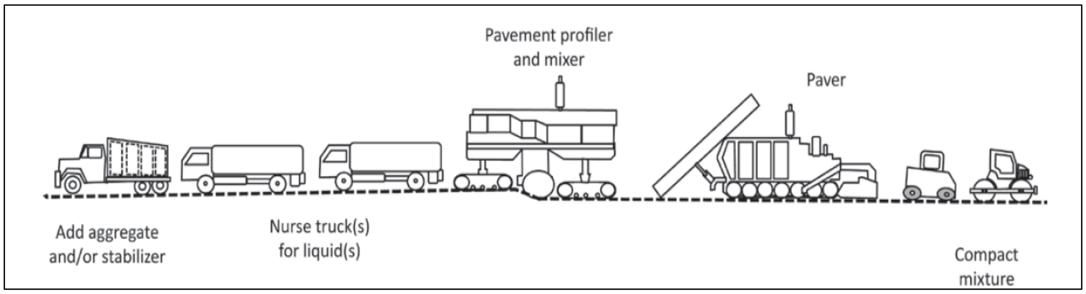

Figure 5. Diagram of a CIR train (Stroup-Gardiner 2011)

2.2.2. Cold Central Plant Recycling

The first step of the CCPR method is milling the existing asphalt section and then stockpiled the material for later use in the plant. CCPR differs from CIR because the asphalt recycling process occurs at a mobile or central plant location (Gu et al. 2019). Other differences of CCPR are that it allows materials from an existing pavement to be selected and pre-treated (crushing and screening), stockpiled, and tested before mixing, thereby increasing the level of confidence that can be achieved during placement (Wirtgen Group 2012). On the other hand, CCPR is similar to the CIR method because it shares some benefits like cost reduction, environmental protection, and shortening construction periods.

Figure 6 shows the CCPR processes in which the asphalt recycling takes place at a central location using a stationary cold recycled mix plant and stockpiled RAP materials. CCPR plants include a belt scale, a computer-controlled recycling agent system, an additive system, and a pugmill for mixing of the final product. CCPR mixtures can be immediately transported in dump trucks or belly dump trucks to the paving operations or stockpiled for later use (Cross and West 2018). Nevertheless, storing the CCPR material is not recommended when asphalt emulsion is used as the primary recycling additive (Asphalt Academy 2009). Placement of the CCPR mixture on layer thickness ranges from 3 to 6 inches (75 to 150 millimeters) is conducted with conventional asphalt pavers, but a motor grader could also be used (Asphalt Recycling & Reclaiming Association 2015).

Figure 6. Cold Central Plant Recycling

The most common recycling agent used in CCPR production is asphalt emulsion. Still, foamed asphalt with mineral additive has increased in the last years (Wirtgen Group 2012). The mixing mechanism is different between emulsion and foamed asphalt.

Control of temperature and moisture is relevant to proportioning and blending the recycled materials accurately. CCPR mixtures require more compaction cycles than hot mix asphalt due to the high internal friction, higher viscosity of the aged binder, and colder compaction temperatures (Asphalt Recycling & Reclaiming Association 2015).

2.3. COLD RECYCLING PERFORMANCE

The fluctuating economy and the need for high-quality natural aggregates and petroleum have increased the demand for cost-effective alternatives to virgin paving materials. Irrespective of the type of recycling agent, there are two recycling construction method alternatives: Cold central plant recycling (CCPR) and cold in-place recycling (CIR). Over the years, Cold Recycling of asphalt concrete pavements has become a widely practiced construction method adopted by different agencies across the United States; they provide owner agencies with cost-effective and sustainable approaches to restore their aged asphalt pavements. CIR and CCPR have been shown to speed up project delivery and alleviate construction traffic congestion, which reduces user costs (Cross and West 2018; Vargas-Nordcbeck and Rahman 2019). The use of recycling techniques to rehabilitate and maintain pavement in-situ started in the 1930s, and over the years, agencies gained experience by research and case studies.

In 1984, the Ohio Department of Transportation studied two low-volume roads to analyze the long-term performance of cold mix recycling with satisfactory results in one of the selected sites according to field cores and deflection measurements (Dudley et al. 1987). The Pennsylvania Department of Transportation by the end of 1985 had paved around 90 CR rural roads, including CIR and CCPR using emulsion, and a single seal coat was applied as the wearing course with the aims of providing a standard specification for selecting suitable candidate projects. One of the recommendations was to obtain optimum moisture content of the RAP so the asphalt emulsion can be adequately dispersed in the mix (Kandhal and Koehler 1987).

Moreover, since 1984 the New Mexico DOT has conducted over 120 CIR projects using polymer-modified emulsion as a recycling agent, with an asphalt overlay that was applied to the top of the recycled base. The pavement condition index (PCI) was used to evaluate road performance (McKeen, Hanson, and Stokes, 1998). The Kansas Department of Transportation started employing Cold Recycling procedures in 1986 to rehabilitate flexible pavements at a rate of 80 to 160 km per year, from 1990 to 1992, fly ash was added to four test pavements towards reducing the potential for moisture damage and wheel path rutting (Cross and Fager 1995). Additionally, in 1997 an experimental partial-depth cold in-place project was initiated in Kansas on Highway US-283. This CIR project involved two test sections in assessing the effectiveness of fly ash and the performance of the rehabilitated pavement using asphalt emulsions technology on a low volume condition. The results showed that the fly ash section cracked before the emulsion section, and both were not susceptible to rutting (Thomas et al. 2000).

In 1986, Iowa DOT paved its first CIR road. Subsequently, more CIR roads were built with an HMA overlay. Most of the CIR roads were successful, but others reached failure before it was expected, research and experience over the years concluded that the main reason for the failed projects was paving in poor subgrade sections (Kim et al. 2010). Eighteen roads were chosen to evaluate short-term performance between 1997 and 1998. The research experiment found that CIR roads generally performed well and predicted service life of 18 years (Jahren Charles T. et al. 1999).

Furthermore, 24 test roads were selected in 2010 to evaluate the long-term performance of CIR sections built between 1986 and 2004. The results support the theory that the CIR layer acts as a stress-relieving layer where a reduced value of CIR modulus and a higher amount of air voids indicates better performance (Chen et al. 2010).

The Nevada Department of Transportation started using the CIR technology in 1995 to rehabilitate roads of low and medium traffic volume; the recycled layer is treated as a stabilized base course followed by a thin HMA overlay. A mix design procedure based on the Hveem design method was developed in 2004 and implemented on three field projects, which showed excellent performance (Sebaaly et al. 2004). A research was published in 2006 about the success The Nevada Department of Transportation has had for more than 30 years using a Pavement Management System (PMS) where CIR and FDR procedures were applied for low and medium-volume roads. During that period, the agency had saved more than $600 M compared to complete reconstruction costs (Bemanian et al. 2006). Also, NDOT applied CIR techniques to high volume traffic roads using HMA overlay plus a surface layer which improve rutting and cracking resistance compared to applying only a surface treatment in low-volume roads (Sanjeevan et al. 2014)

A rural airport in Florida restored an aging runway first built in the 1950s by applying cold in-place recycling. The original road had poor drainage, exposure to heat, and heavy use over the years. The project was a partnership between the Florida DOT and the contractor, the construction was completed in 1997, and after an inspection in 2002, no further maintenance was needed (Polak 2003). Caltrans applied chip seals on the top of CIR with foamed asphalt to improve farm roads in inadequate conditions in San Joaquin Valley. This experiment extended the life of the pavement by ten years (Kuennen 2003).

The use of Class C fly ash in CIR to improve the structural capacity of asphalt pavement base layers was analyzed in Wisconsin. Nondestructive deflection tests were performed in a study section to analyze the structural performance; the researchers concluded that using recycled materials saved hauling costs (Wen et al. 2003). A section of 3.6 miles on the southbound direction of I-81 in Virginia was rehabilitated using CIR, CCPR, and FDR techniques in 2011; the objective was to gain experience in Virginia DOT personnel in a cold recycled mix design and field evaluation. The research project concluded that the combination of the construction techniques accelerated the time window in that the AC overlay can be paved after the construction of the CCPR layer (Diefenderfer and Apeagyei 2014).

In Minnesota, two different CIR processes were compared in 2002. One was the conventional CIR process, and the other one included a sampling protocol, a new mix design including performance testing of laboratory-prepared samples, and an engineered asphalt emulsion. The new process exhibited superior performance in terms of raveling, thermal cracking, and moisture susceptibility (Forsberg et al. 2002). Cox and Howard (2016) investigated the effects of binder systems (single or multiple component binder systems) on CIR construction. As a result of the experiment, they created a CIR design framework. Also, The Mississippi DOT evaluated the CIR moisture-density relationship on Highway US-49. The main research finding is that moisture content had no significant effect on RAP dry density evaluation (Cox et al. 2015).

The National Center for Asphalt Technology (NCAT) constructed cold central plant recycled (CCPR) structural sections in its 2012 research cycle. The results from the experiment show that under accelerated loading condition, the functional and structural performance of foamed asphalt CCPR with 100% RAP performed similarly to conventional hot asphalt mix (Timm et al. 2018). Researchers have tested emulsion and foamed recycled mixes under mechanistic tests to analyze short- and long-term performance at various testing temperatures and loading conditions in the lab (Birgisson et al. 2004; Brown et al. 2004; Kim and Lee 2012).

2.4. STRUCTURAL PAVEMENT DESIGN

Pavement design methods have been developed, starting with empirical methods moving to more mechanistic ones that require computer software to analyze. Pavement design using cold recycled material has been a challenge to many researchers to model the material properties of the mix accurately. The lack of quantitative values for the engineering properties of CIR/CCPR materials that can be used with confidence in pavement structural design is a significant impediment to the more widespread use of these sustainable strategies. The Mechanistic-Empirical (ME) pavement design method predicts specific modes of distress resulting from modeling pavement responses to loadings under various conditions (Timm et al. 2014).

The NCHRP research report 863 studied the dynamic modulus, a relevant input in the mechanistic-empirical pavement design methodology, using different recycling agents and additives on CIR, CCPR, and FDR. The results show that the materials have a similar range of dynamic modulus at intermediate and high frequencies (Schwartz and Diefenderfer 2017). This conclusion is supported by a similar trend observed in Virginia based on FWD testing (Diefenderfer et al. 2016). Nevertheless, different research shows that CIR-emulsion is not as sensitive to temperature and loading frequency as HMA according to dynamic modulus testing, and the variation of CIR using foam is due to the residual asphalt content. (Kim et al. 2009; Kim and Lee 2012).

The variability of results presented by researchers in modeling recycled materials, in addition to the lack of field performance calibration for this materials have stopped the agencies from moving forward to a mechanistic-empirical approach; therefore, DOTs continue using the American Association of State Highway and Transportation Officials (AASHTO) design method issued in 1993 as their primary design tool. A survey from 2014 reports that 40 agencies used more than one pavement design method, 48 agencies use empirical design methods, and only 13 use the ME design approach (Pierce and McGovern 2014).

The AASHTO 93 empirical design is based primarily upon observations from the American Association of State Highway Officials (AASHO) Road Test conducted from 1958-1960 in Ottawa, Illinois. AASHTO introduced the pavement structural number (SN) concept. The SN symbolizes the pavement structure on top of the subgrade determined by the product of specific layer thicknesses (Di), their respective structural coefficients (ai), and drainage coefficients (mi). The structural coefficients relate the relative load-carrying capacity of different materials; also, a direct correlation may be determined between the structural layer coefficient and the elastic modulus of each layer analyzed (Highway Research Board 1962). Therefore, the structural capacity for each pavement layer can be represented by an SN as the product of the layer thickness and the structural coefficient layer and the drainage coefficient. As mentioned above, the AASHTO design method is the most used design approach by agencies, mainly for overlay design. This approach is accomplished using falling weight deflectometer (FWD) testing for layer properties.

The structural layer coefficient has been determined based on their relationship with the elastic modulus. Research has been done to correlate some recycled material tested in the laboratory to describe the structural contribution of a specific pavement layer. Some layer coefficients have been proposed for a limited number of recycled materials, though the recommended values are mostly based on the results from laboratory testing. Another method based on the relationship mentioned before is to compute the elastic modulus from deflection testing on the field (Díaz-Sánchez et al. 2017).

2.4.1. Structural Coefficients

Over the years, research has been conducted to determine the structural layer coefficients for recycled materials through laboratory testing and backcalculated moduli from deflections. During the summer of 1981, in Indiana, an experimental section was constructed to evaluate construction with the foamed technology. Layer coefficients were computed through deflection data, and the results after one day of construction varied from 0.13-0.30

in. $^{-1}$ (Wijk and Wood 1983). A laboratory study evaluated the behavior of cold-recycled asphalt paving mixtures by using asphalt emulsion and foamed asphalt as the recycling agents. Resilient modulus results show a similar performance of the mixes, and layer coefficients ranged from 0.10 to 0.16 cm^{-1} (0.25–0.40 in. $^{-1}$) were computed (Tia and Wood 1983). In Maryland, a study evaluated foamed asphalt as a base material to explain the distinct mechanical behavior, and it was compared to a granular base. The recycled foamed base reported a modulus in the range of 191–343 ksi and a structural layer coefficient of 0.142 cm^{-1} (0.36 in. $^{-1}$), however, the granular aggregate base had a resilient modulus in the range of 20-40 ksi (Khosravifar et al. 2015). These results are significantly low compared to the 449–1000 ksi range for dynamic modulus of a typical HMA at 218°C and a 10 Hz loading frequency (Huang 1993).

Three projects in Maine were selected to determine the structural strength of foamed asphalt CR layers by conducting FWD tests. From the calculated moduli, the structural coefficients were between 0.22-0.35 in. $^{-1}$ (Marquis et al. 2003). The New Mexico State Highway and Transportation Department started to use CIR as a rehabilitation alternative for flexible pavements since 1984 and based on performance results, the structural coefficient determined by the DOT was 0.30 in. $^{-1}$ (McKeen et al. 1998). The Nevada DOT conducted an extensive field-testing program that used the FWD to establish a structural coefficient for the CIR layer, the conclusion assigned a structural coefficient of 0.26 in. $^{-1}$ (Sebaaly et al. 2004).

Romanoschi et al. (2004) estimated a structural layer coefficient of 0.18 for full-depth reclamation using foamed asphalt based on the results of full-scale accelerated pavement testing in Kansas. During the 2011 construction season, VDOT rehabilitated an interstate using CIR and CCPR; this research concluded that CIR and CCPR properties were not statistically different in terms of resilient modulus. Moreover, they have temperature dependency behavior similar to HMA (Apeagyei and Diefenderfer 2013). The study used deflection testing and laboratory measurements of the resilient modulus and indirect tensile strength of CCPR-foam field cores to estimate layer coefficients; the results ranged from 0.36–0.48 in.$^{-1}$ (Diefenderfer and Apeagyei 2014).

In 2012, two full-scale pavement sections were built at NCAT to assess the structural contribution of CCPR with 100% RAP and foamed asphalt as a recycling agent. The layer coefficients were found to fluctuate from 0.36 to 0.39 in.$^{-1}$ (Díaz-Sánchez et al. 2017).

2.5. SUMMARY

This chapter defined pavement preservation and their different techniques focusing on cold recycling as a preservation technique. Also, the literature review gives a brief overview of mix design methods going from simple empirical to more sophisticated procedures with performance-based testing. Differences between the two recycling agents (emulsion and foam) were described in the chapter, which includes production, mixing temperature, RAP coating, and others.

A description of the two main recycling techniques was included in this study. A summary of the performance of recycled sections in different DOTs was given. A literature-based summary of their documented structural performance was provided.

The main results related to the layer coefficient found in the literature reviewed are summarized in Table 5. The reason for a wide range of cold recycled coefficients may be because most agencies do not have a CR specification, resulting in non-optimized designs. Therefore, this thesis focuses on the long-term performance of cold recycled asphalt mixtures as a pavement preservation technique regardless of construction methods or recycling agents.

Table 5. Structural coefficients for recycled materials

Source	Recycling Method	Layer Coefficient
Tia and Wood (1983)	Laboratory mixing	0.25 - 0.40
Wijk and Wood (1983)	CIR	0.13 - 0.30
Mckeen et al. (1998)	CIR	0.30
Marquis (2003)	CIR	0.22-0.35
Sebaaly et al. (2004)	CIR	0.26
Romanoschi et al. (2004)	FDR	0.18
Khosravifar et al. (2015)	CIR	0.36
Apeagyei and Diefenderfer (2014)	CCPR	0.36 - 0.48
(Díaz-Sánchez et al. (2017)	CCPR	0.36 - 0.39

CHAPTER 3: METHODOLOGY

This chapter discusses the methods used to accomplish the objectives of this research. The evaluation of the structural contribution and functional performance of Cold Recycling techniques under realistic conditions is the most proper way to assess the evolution of pavement properties and distresses overtime under representative traffic loading. Therefore, the estimation of layer coefficients for the recycled layers is reliable to be used as an input in the empirical pavement design method. Thus, four test sections were built along Highway US-280 in Opelika, Alabama.

Cold recycled mix design and pavement construction information of the recycled sections are reported, followed by the data collection techniques and data processing. Cold recycled treatments were applied to full-scale test sections in 2015. Routine testing was carried out to monitor the performance of cold recycled treatment sections. Roughness, cracking, and rutting was measured and analyzed throughout the experiment; also, falling weight deflectometer (FWD) testing was performed to examine the structural condition of the pavement. The data collected from FWD testing were used to backcalculate the layer moduli. The FWD files obtained from the Dynatest device were used for backcalculation with ELMOD 6 software. Finally, a statistical analysis was performed on the measured parameters of the cold recycled sections using Minitab 17 software to measure the influence of the construction techniques and recycling agents in the performance of the recycled sections.

3.1. PRESERVATION GROUP STUDY

In the summer of 2012, NCAT designed the Pavement Preservation Group (PG) study as part of NCAT's fifth research cycle. The PG Study was initiated in response to the growing need for agencies to obtain reliable performance data for different pavement preservation treatments that would allow agencies to make objective decisions regarding treatment selection. Although pavement preservation treatments have been applied to test sections on the NCAT Pavement Test Track, the PG study sections were placed on off-Track roadways to eliminate any effects resulting from the accelerated rate of axle load repetitions (Powell 2016). The PG Study began with 23 preservation treatments in Lee County Road 159, a low-traffic volume road in Alabama.

Preservation treatments were placed on a section of Highway US-280, a higher volume roadway, during the 2015 research cycle. NCAT has also partnered with the Minnesota Department of Transportation's Road Research Facility (MnROAD) to quantify the benefits of pavement preservation in both northern and southern climates. In 2016, several treatments were replicated on low and high traffic volume roadways near Pease, Minnesota. The Preservation Group test locations are shown in Table 6.

Table 6. Test Locations Summary

Roadway	**LR -159**	**US-280**	**CSAH 8**	**US-169**
Location	Auburn, AL	Opelika, AL	Pease, MN	Pease, MN
Traffic Volume	Low	High	Low	High
Number of treatments	23	34	22	21
Year Treated	2012	2015	2016	2016

3.2. TEST SECTIONS

For the 2015 research cycle, 34 pavement preservation treatments or treatment combinations were placed on the outside lane of a 4-mile section of Highway US-280 in Opelika, Alabama. The two-lane highway has a total length of 392 miles, and the road goes from Blichton, Georgia to Birmingham, Alabama, and is the main connector between the cities of Birmingham and Auburn, AL. Figure 7 shows the location and a view of the test sections placed on this site. The accumulated traffic load since the summer of 2015 is approximate 2.8 million ESALs. The reported annual average daily traffic (AADT) in 2015 was 18,300 vehicles per day (vpd), and by 2018 was 20,083 vpd with a truck volume of 16%.

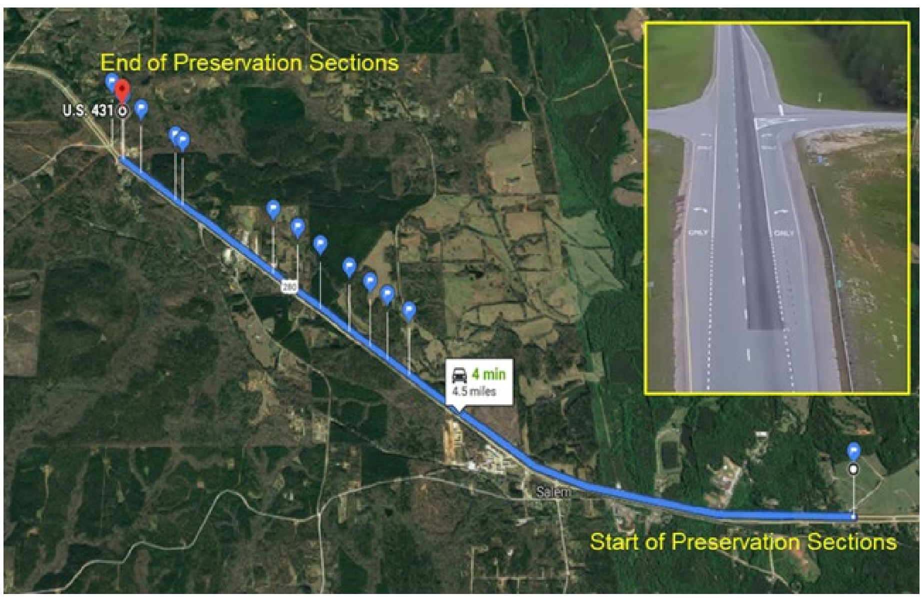

Figure 7. Location and Overview of Highway US-280. (Reproduced from Google Earth, 2020)

At the time of construction, the existing pavement on Highway US-280 was nine years old and had an average hot mix asphalt of 6.8 inches on top of a granular base of ten inches. The 46 sections have a length of 0.1 miles each and are located on the outside lane of the two-lane eastbound highway. Each section is subdivided into 32 subsections measuring 5.5 by 16.5 feet. There are six control sections with low and high levels of cracking, rutting, IRI and texture, and six empty sections. Additionally, there are 11 segments of the road, which were part of the Long-Term Pavement Performance (LTPP) project; these sections add variability to the layer thicknesses of the paved road. Collected data from cores, LTPP database, and construction records corroborate the layer thickness variability.

The location of the four recycled sections included in this study corresponds to sections 40, 41, 43, and 44 on the Highway US-280 test site, situated between mileposts 131.9 and 132.4. The treatments placed include cold central-plant recycling with emulsified and foamed asphalt binders (i.e., CCPR-E and CCPR-F), and cold in-place recycling with emulsified and foamed asphalt binders (i.e., CIR-E and CIR-F). All the sections have 100% RAP, and a thin 1-inch overlay was paved as a surface layer. The complete pavement structure was determined by Ground Penetrating Radar (GPR) testing validated with core logs. The composition of the recycled sections is shown in Table 7, Figures 8, and 9.

Table 7. Average layer thicknesses of cold recycled sections

Section	Treatment Type	Milepost	Overlay, inch	CR, inch	HMA, inch	Granular Base, inch
40	CCPR-F	131.90	1.00	4.00	3.40	11.10
41	CCPR-E	132.00	1.00	4.00	8.70	8.00
43	CIR-E	132.20	1.00	4.00	12.50	0.00
44	CIR-F	132.30	1.00	4.00	7.80	0.00

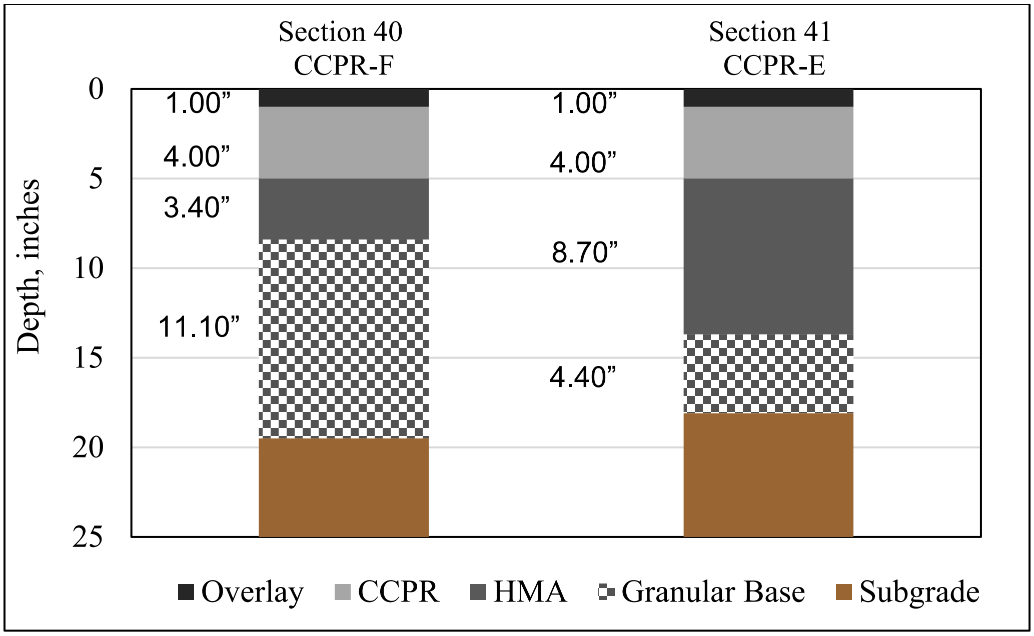

Figure 8. Average layer thicknesses of Cold Central Plant Recycled sections

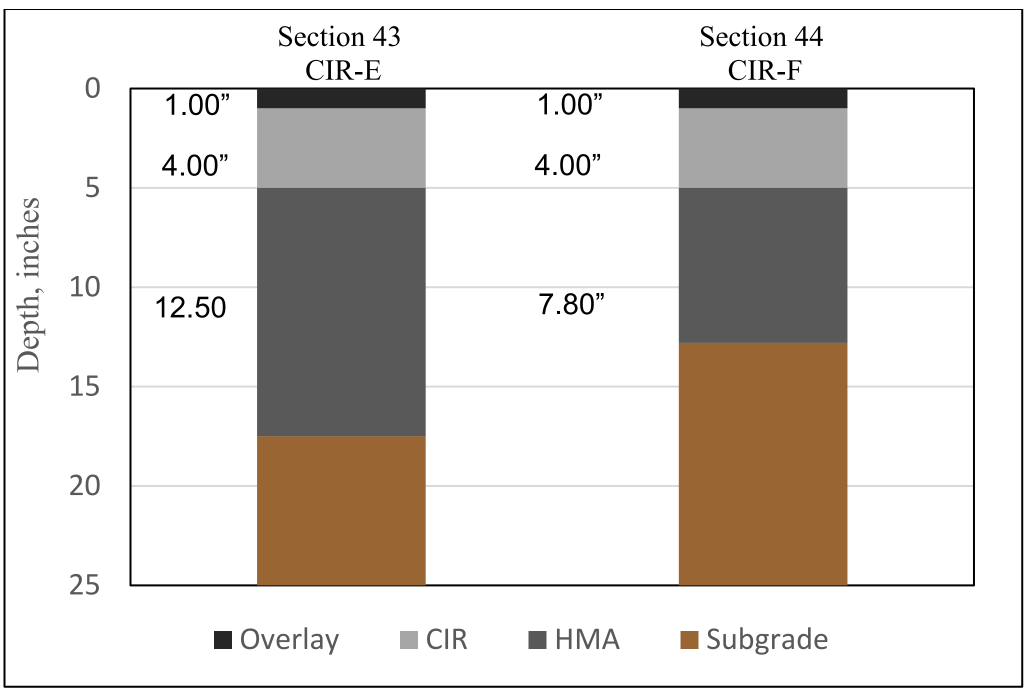

Figure 9. Average layer thicknesses of Cold In-Place Recycling sections

3.2.1. Mix design

The two common recycling agents used for this research were foamed asphalt and asphalt emulsions. The reduced viscosity of recycling agents allows mixing with ambient temperature RAP. Cold recycled mixtures can be produced using two production techniques, cold central plant recycling (CCPR), and cold in-place recycling (CIR) using processing and mixing equipment on the roadway.

Cold Recycling mix design guidelines proposed by the Asphalt Recycling & Reclaiming Association (ARRA) were followed to design both the foamed asphalt and asphalt emulsion mixtures (ARRA 2016, 2017). East Alabama Paving (EAP) milled, crushed, screened, and stockpiled the RAP; then, the material was sampled from EAP's stockpile. The RAP source was from a previous construction project on Highway US-280 in Opelika, Alabama. The RAP binder content ranged from 4.9 to 5.2 percent, and the performance grade of the recovered binder was PG 100-10. A PG 67-22 binder from Birmingham, AL, was used for foaming while a PG 64-22 binder from Parsons, TN was used for the emulsions.

NCAT designed the foamed asphalt mixtures. The Wirtgen's laboratory-scale WLB 10-S model foaming plant was used to produce the foamed asphalt. The asphalt was foamed at 170°C and 1.3 percent water to obtain a foamed asphalt with an 8.5 expansion ratio and 6-second half-life. The twin-shaft pugmill WLM 30 model was used to mix RAP with foamed asphalt binder at a room temperature of 25 ± 2 °C. The optimum foamed asphalt content was determined by testing three foamed asphalt contents (2.0, 2.5, 3.0 percent) to produce different trial mixtures.

After mixing, the samples were compacted to 63.5±2.5 mm height for 35 gyrations using a Superpave gyratory compactor in a 100-mm diameter mold (Gu et al. 2019). The compaction effort was based on previous field construction projects performed by Wirtgen America to match field compaction effort. The specimens were extruded from the molds after compaction, and then cured in an oven at 40 ± 1 °C for 72 hours and cooled at 25 ± 2 °C for 24 hours. Compacted and cured specimens were tested for indirect tensile strength (ITS) in both dry and wet conditions following AASHTO T283 without freeze-thaw conditioning. The design followed the ARRA criteria of a minimum dry strength of 45 psi and a minimum tensile strength ratio of 70 percent.

The optimum foamed asphalt content was 2.2 percent by weight of dry RAP, and the total water content was 7.2% by the weight of dry RAP for the CCPR mixture. The foamed asphalt content was 1.8% by the weight of dry RAP, and the total water content was 4.9% by the weight of dry RAP for the CIR mixture. A dosage of 1.5 percent Type I/II Portland cement was added to reduce moisture susceptibility for both asphalt mixes, this amount of active filler is greater than typical values. High rates of cement might affect the cracking resistance of the sections.

The methods and criteria used for the emulsion mix designs were established by Ingevity Corporation; these techniques were based on experience and ARRA design procedures. The design method considers the dynamic modulus test as part of the selection of the optimum asphalt content. Ergon Corporation provided the emulsion, which was made using Ingevity's INDULIN W-5 (a cationic slow-set emulsifier) at a dosage rate of 1.0% active emulsifier and 2% of polymers.

The emulsion was blended with RAP at three different contents (2, 3, and 4 percent) for comparison. Therefore, the mixtures were mixed and compacted for Marshall stability testing at dry and wet conditions. All the mixtures with different contents passed the minimum 1,250 lbf stability criteria, and a minimum of 70 percent retained stability. The optimum emulsion content was selected by testing the asphalt for dynamic modulus (E*), fracture energy by the disc-shaped compact tension test, and raveling test.

The emulsions contained 62 percent residual asphalt, also the emulsion content was designed as 3.0 percent, and the total water content was 7.0 percent by weight of dry RAP for the CCPR mixture. Whereas for the CIR mixture, the design emulsified asphalt content was determined as 3.2 percent, and the total water content was selected as 4.4 percent by weight of dry RAP. The same cement type and amount used in the foamed asphalt mixtures were added.

A summary of the mix designs is shown in Table 8, the asphalt content shown in the table is the residual asphalt for the emulsion designs, and the compaction is the ratio of dry density between field cores and laboratory QC samples. The foamed and emulsion mixtures have almost the same gradation for CCPR and CIR; the gradation curves are located to the left of the maximum density line, which indicates a fine gradation. The burned gradations of cold recycled techniques are shown in Figure 10, and they follow the ARRA criteria.

Table 8. Summary of Cold recycled mix designs

Mixture	Asphalt (%)	Water (%)	Compaction (%)	Dry ITS (psi)	Wet ITS (psi)	TSR (%)
CCPR foamed	2.2	7.2	98.3	52.2	48.1	92.1
CIR foamed	1.8	4.9	96.6	69.5	68.7	98.8
Mixture	**Asphalt (%)**	**Water (%)**	**Compaction (%)**	**Dry Stability (lbf)**	**Wet Stability (lbf)**	**Retained Stability (%)**
CCPR emulsion	1.9	7.0	102.4	2175.0	2025.0	93.1
CIR emulsion	2.0	4.4	96.7	2275.0	2225.0	97.8

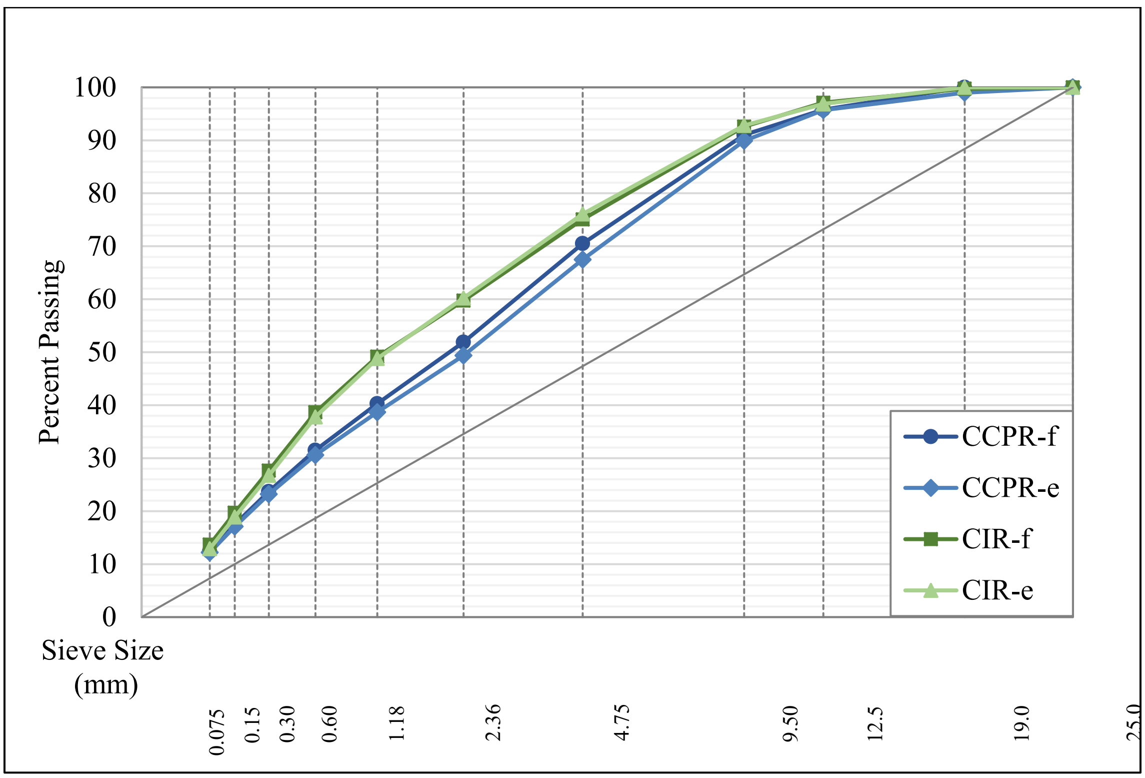

Figure 10. Burned gradation of the aggregates from the cold recycled mixtures

The thin overlays placed on top of the recycled sections have the same hot mix asphalt mixture. The nominal maximum aggregate size (NMAS) was 4.75 inches; the dense-graded mix had a design binder content of 6.1 percent; the base binder had a performance grade of PG 67-22. The mixture was designed under the Superpave method with a compaction effort of 75 gyrations. The aggregates consisted of limestone and sand, 11 percent of fractionated fine RAP, and 3 percent of recycled asphalt shingles (RAS), for a total binder replacement of 20 percent.

3.2.2. Construction

The main contractor for the construction of the four recycled sections as part of the preservation study was East Alabama Paving. The recycling equipment was supplied by Wirtgen America Inc. and supported by Wirtgen personnel. The construction was scheduled for the week of September 7, 2015. A Wirtgen KM220 mobile mixing plant was used to produce foam and emulsion CCPR mix. A Wirtgen 3800 CR in-place recycling machine was used to produce foam and emulsion CIR mix. The construction of the CCPR sections started on Wednesday, September 9; the weather during the construction day was mostly clear, with a temperature of around 90°F and 55% relative humidity. The sections were milled to a depth of 5.4 inches (Figure 11) to allow for the placing of CCPR produced material, to accommodate the fluff of recycling and the thin overlay.

The CCPR was carried out using a Wirtgen KMA220 Cold Mix Recycling Plant, positioned at the NCAT test track (Figure 12). The foam CCPR was placed on Section 40 at mile marker 131.9; the recycled RAP was loaded directly to trucks via the discharge conveyor and transported to the construction site. The cold recycled mix was paved using a Roadtec paver to the required depth and levels and immediately compacted behind the screed using tandem steel drum vibratory rollers.

Figure 11. Milling of the CCPR sections

Figure 12. Production of CCPR at NCAT test track

Following the foamed RAP section, the KMA220 was set up for the Emulsion section following the mix design. The emulsion CCPR was placed on Section 41 at milepost 132.0, the paving of the emulsion recycled RAP executed precisely as the foam section with similar compaction results (Figure 13).

After compaction, both sections were sealed with a tack coat; once cured, the lane was opened to full unrestricted speed traffic (Figure 14).

Figure 13. Paving of the CCPR foam section

Figure 14. Tack coat application for both sections

On September 10, 2015, there was a concern that the tanker truck carrying the PG 64-22 asphalt still contained leftover remnants of bitumen from the trackless tack emulsion. Therefore, it was decided to bring in the emulsion tanker and make section 43 the CIR emulsion section.

The weather was cloudy, with a temperature of 79°F and 84% relative humidity. The workday started by pre-milling to a depth of 1.4 inches, with 1.5% cement spread on the surface in each of the two 500 ft sections (Figure 15). The CIR was carried out using a Wirtgen 3800CR full lane recycling machine; the 3800CR being was set up for emulsion in the first section. The resulting homogeneous recycled mix was loaded directly into the hopper of the Roadtec paver via a rear discharge conveyor (Figure 16). The material was then paved and compacted exactly as the CCPR material. The CCPR emulsion was placed at mile marker 132.2.

Figure 15. Pre-milling of the CIR sections

Figure 16. Recycled material loaded into the paver

On September 11, 2015, the 3800 CR was switched to foamed asphalt, and the foam-based CIR was placed on Section 44 at milepost 132.3. Finally, the thin overlay was placed and compacted on all the sections (Figure 17).

Quality control was performed to verify the quality of produced cold recycled mixtures. The recycled mixtures were sampled close to the paver auger during construction and tested for total water content, added asphalt content (subtracting RAP binder content from total binder content), and aggregate gradation. Samples were also compacted and cured for the ITS test (or stability test). Both the foamed mixture's strength and emulsion mixture's stability met ARRA's recommended criteria.

Figure 17. Compaction of the thin overlay

3.3. DATA COLLECTION

Asphalt pavement performance depends on many variables, such as traffic, materials, and construction practices. Continuous testing is relevant for reliable results and pavement management. The evaluation is based on a set of visual evaluation methods and non-destructive testing (NDT). Field data collection has been in progress since September 2015. Data collection was performed periodically for the test sections. The data collected included roughness, rutting, cracking, friction, texture, and deflection testing. Lane closures are required to test on Highway US-280 due to the traffic level and safety of the crew.

3.3.1. Functional Performance testing

Pavement condition is assessed by collecting functional performance data over time. Rut depth and roughness were obtained biweekly using a data collection vehicle equipped with an inertial profiler, a laser rut measurement system (LRMS), and high-resolution cameras. The inertial profiler collected the longitudinal profile of the pavement surface for both wheel paths (Figure 18), and images for crack mapping were taken quarterly.

The LRMS systems can obtain the transverse profile and measure rut depths, while multiple laser sensors measure macrotexture data by the average mean texture depth (MTD) according to ASTM E1845. Additionally, the high-resolution camera for pavement scanning is used to monitor cracking. Friction was measured monthly by ALDOT on each section with a locked-wheel skid trailer; the test was conducted using a ribbed tire at 40 mph on a wet surface following ASTM E 274.

The Federal Highway Administration (FHWA) proposed performance measurements to assess the condition of the pavement based on the percentage of pavements in good and poor condition. The Moving Ahead for Progress in the 21st Century Act (MAP-21) legislation was issued by the FHWA in January 2017, which required that performance measures be established by the State Departments of Transportation (DOTs). The condition of the pavements is to be determined based on the International Roughness Index (IRI), cracking percent, and average rutting.

Figure 18. Automated Road Analyzing Vehicle

The condition evaluation of the test sections is based on the three performance measures adopted from MAP-21. This approach is essential to evaluate the road in a way that is consistent among agencies (Visintine et al. 2018). The thresholds of the condition categories for the performance indicator established by the MAP-21 act for good, fair, and poor are shown in Table 9. This evaluation requires pavement segments of 0.1 miles, the same as the length of the test sections in this study.

Table 9. Condition thresholds for MAP-21 Performance Measures

Condition Rating	% of Area Cracked	Rutting, in (cm)	IRI, in/mi (m/km)
Good	< 5%	< 0.20 (0.51)	< 95 (1.5)
Fair	5 – 20%	0.20 – 0.40	95 – 170
Poor	> 20%	> 0.40 (1.02)	> 170 (2.7)

3.3.2. Deflections

Deflection testing is performed following ASTM D 4694 *Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device*. The lane closure of Highway US 280 for FWD testing is made quarterly. There are three designated random locations for FWD testing within each test section. An example of the locations for test section 43 can be observed in Figure 19. The FWD test is performed on the same locations on each test date.

Figure 19. Random location of test section 43 (Reproduced from Google Earth, 2019)

At each random location, testing was performed in the center of the wheel paths. A Dynatest 8000 FWD model device is used by NCAT for deflection testing (Figure 20). At the time of testing, three replicates at four load levels (approximately 6, 9, 12, and 16 kips) were applied with a plate radius of 5.91 inches at each location. Surface and air temperatures were recorded, the load level will depend of the height of the drop, an acceptable height drop range is between 90 and 110 percent.

At the start of every testing day, the FWD device applied a warmup dropping session in order to check for variability within drops. The variability under standard specification is 3% within replicates of the load magnitude of a target magnitude load. Common oversights during testing are seating errors, which are due to debris or rough surface texture; these can be addressed by cleaning the pads or checking the spring and performing seating drops to adapt to the asphalt surface. The FWD device is subjected to calibration to prevent systematic errors; monthly relative calibration and annual reference calibration are achieved following the AASHTO R32 *Standard Practice for Calibrating the Load Cell and Deflection Sensors for a Falling Weight Deflectometer*.

Figure 20. FWD testing on Highway US-280

A set of geophones are arranged to measure the deflection basin around the center of the load plate. The FWD device was used with a total of nine geophones, spaced, as shown in Table 10.

Table 10. Geophone offsets

Sensor Number	Offset from Load Center, in.
1	0
2	8
3	12
4	18
5	24
6	36
7	48
8	60
9	72

3.3.3. Ground Penetrating Radar (GPR)

On May 19, 2020, Bhate Geosciences Corporation completed the GPR fieldwork of the pavement layer thicknesses at the 0.1-mile sections of Highway US-280 (Figure 21). The test was performed using air-launched GPR following the applicable requirements of the ASTM D4748-10 *Standard Test Method for Determining the Thickness of Bound Pavement Layers using Short-Pulse Radar*. The GPR scanned the layer thickness every six inches. Existing core data provided by NCAT was used to validate the layer thicknesses obtained using GPR. The GPR collection instrument records the two-way travel time of the wave produced from an antenna as it travels from the transmitter to the reflector (pavement material interface) and back to the receiver. The dielectric constant of the material determines the velocity of the GPR wave.

Figure 21. GPR calibration

The profile of the sections determined by GPR testing is displayed in Figure 22, and Tables 11 and 12 indicate the pavement structure per random location of each test section calculated as an average of 20 ft forward and backward from the test location to account for the variability.

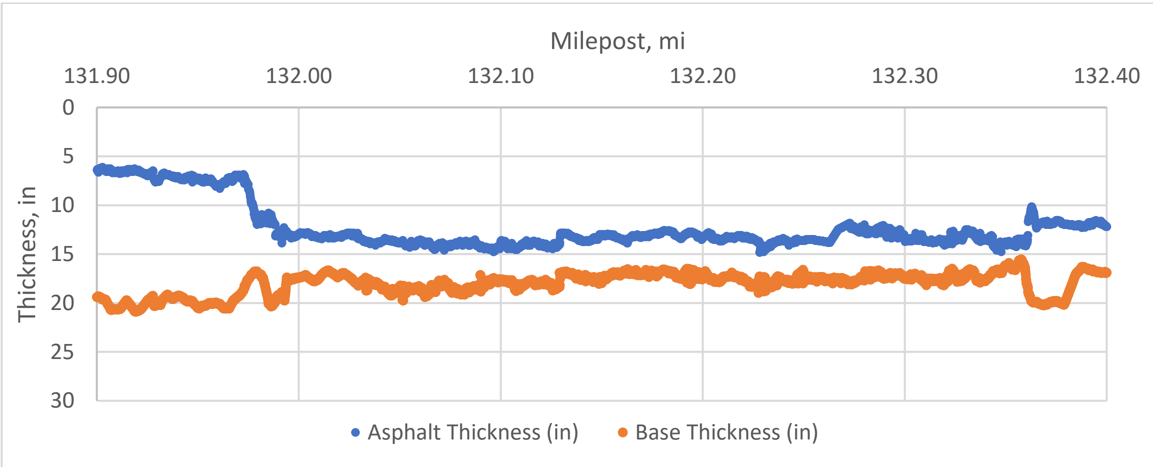

Figure 22. GPR profile

Table 11. Layer thickness per random location-Sections 40 and 41

Random Location	Milepost	Asphalt, inch	Base, inch
40-1	131.93	6.91	13.01
40-2	131.94	7.22	12.29
40-3	131.98	10.26	6.90
41-1	132.01	13.05	8.00
41-2	132.06	13.84	8.00
41-3	132.08	13.89	8.00

Table 12. Layer thickness per random location-Sections 43 and 44

Random Location	Milepost	Asphalt, inch
43-1	132.23	18.10
43-2	132.26	17.59
43-3	132.30	17.21
44-1	132.31	17.64
44-2	132.36	13.22
44-3	132.37	11.76

3.4. STRUCTURAL CAPACITY

The development of the structural capacity analysis and the structural contribution assessment for each technique follows the flow chart presented in Figure 23. This process started with data collection using the FWD device. At the time of testing, the FWD operator checks the device to prevent any testing error. The load pulse generated by the FWD momentarily deforms the pavement under the load plate into a dish or bowl shape which is measured by the geophones, this resultant deflection shape is called the deflection basin.

The second step for the analysis is to evaluate the raw deflection data. The collected deflection files were stored in a database. This process is essential to filter outlier values that can be caused by the presence of cracking on the surface or irregularities, cleanliness of the surface, bedrock, deflections beyond the mechanical limit, shallow stiff layer, and water table. A typical error is when a geophone that is far away from the load plate measures a higher deflection than the geophone closer to the load; this error leads to erroneous backcalculated modulus. Another filter to the database is that from the three load levels (6,000 to 12,000 lb.) that have been tested at each random location (3 replicates per load level), only the data obtained for the second load level (approximately 9,000 lb.) were used for structural capacity evaluation.

The third step in the process is to import the filtered FWD files into the ELMOD 6 software and create a database in the software. The recycled sections are defined in the software. Also, the thickness and seed modulus for each layer are entered as inputs. The bituminous layers (cold recycled material and HMA) were combined as one for this computation. Later, the replicates of the load levels were selected to perform the moduli backcalculation in the software. The software uses the Deflection Basin Fit method to backcalculate the moduli of the combined layers (Ep) and resilient modulus of the subgrade (Mr). This method requires a forward calculation program within ELMOD 6.

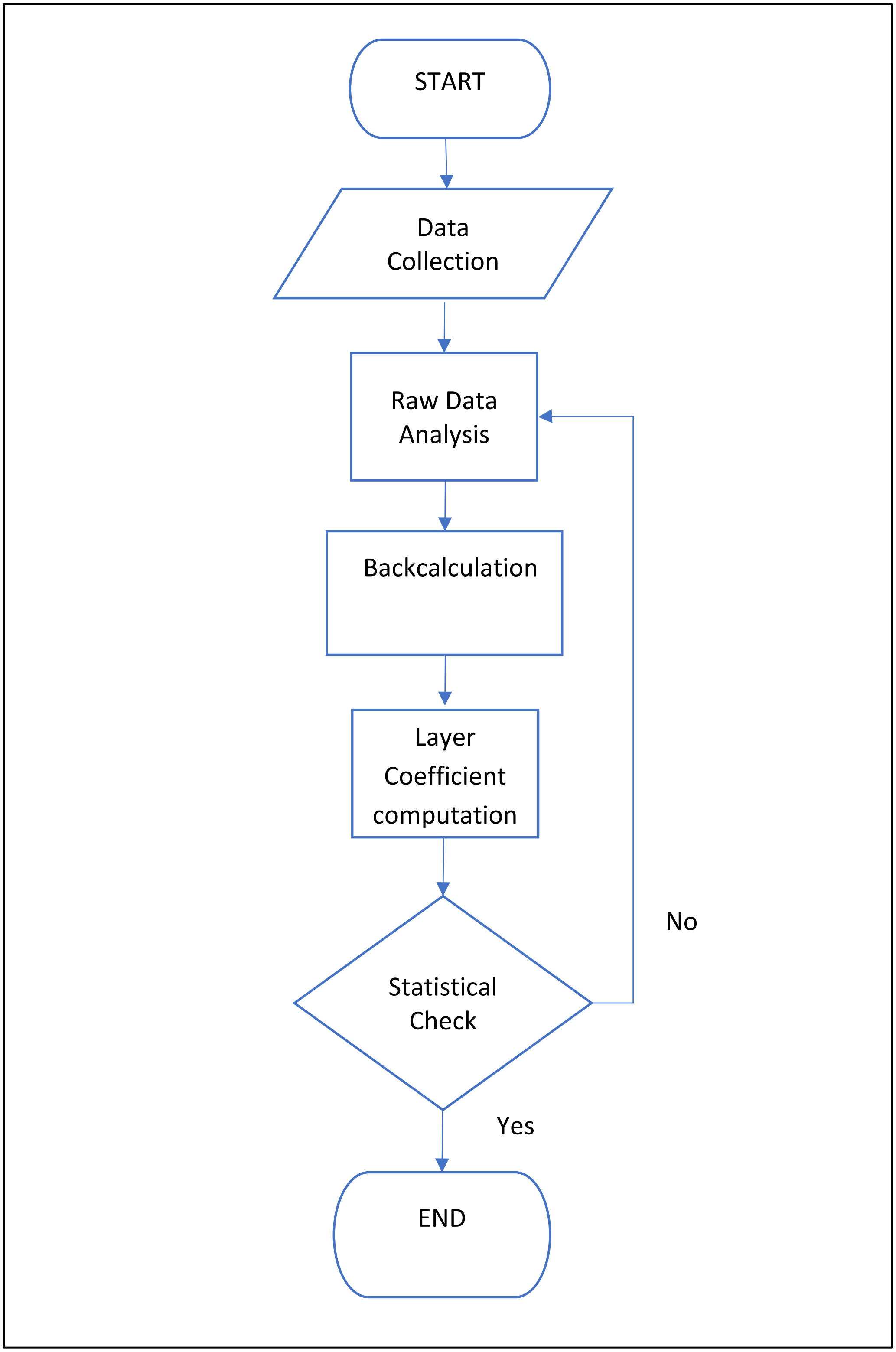

Figure 23.Flowchart of the structural analysis process

This program utilizes Odemark-Boussinesq equations to generate a database of deflection basins for different combinations of layer moduli, specified layer thicknesses, and loading conditions. The Odemark-Boussinesq model follows the underlying assumptions that the layers are homogeneous, isotropic, and linearly elastic. The measured deflection basin is compared with the deflection basins in the database using a search algorithm, and a set of moduli are interpolated from the layer moduli that produces the closest calculated deflection basins in the database.

The number of basins required to obtain a suitable database depends on the number of layers and the expected moduli ranges. The selection of the search range in the ELMOD 6 software can be modified on the offset (10 to 50%), and the steps (1 to 10) search options. The software interface can be seen in Figure 24. This research considered an offset of 30% and two steps for the search range. Finally, the program optimizes the solution by seeking to minimize the relative sum of squared differences between the measured and calculated surface deflections.

The backcalculation results are exported in an Excel file; it is important to notice that the results are for the test conditions, without any adjustment for temperature or seasonal effects. Surface temperatures recordings at the time of testing were used as an input to Bell's equation to compute the asphalt temperature in the middle of the layer as a function of the previous average daily air temperature. The average air temperature was collected using the LTTP database for Opelika, AL.

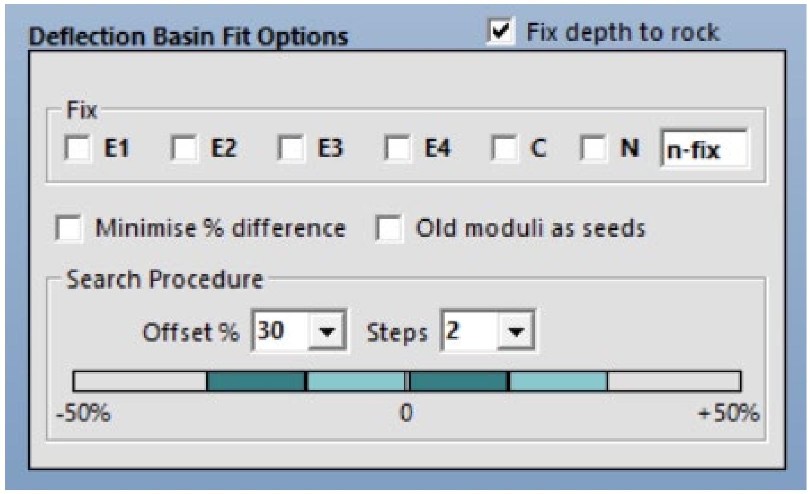

Figure 24. Deflection basin fit options on ELMOD 6

Temperature correction factors to convert composite pavement modulus at any temperature to 68 °F were determined for each test section using correlation equations obtained in Excel between Ep and temperature. The following equations were used for the calculation:

$$E_{pT} = k_1 e^{k_2 T} \quad [1]$$

$$E_{p68} = E_{pT} e^{k_2(68-T)} \quad [2]$$

Where,

E_{pT} = Backcalculated Composite Pavement Modulus at T temp., psi

E_{pT68} = Backcalculated Composite Pavement Modulus at 68°F, psi

T = Mid depth temperature of pavement section, F

k_1, k_2 = correlation coefficient

The next step in the structural analysis process is to compute the layer coefficient of the cold recycled layers. The method used to calculate layer coefficients in Sections 43 and 44, which there are no granular materials, is by using the AASHTO 1993 AC over AC overlay design method because the composite pavement modulus is over the subgrade. The Effective Structural Number (SN_{eff}) calculation requires the backcalculated composite pavement modulus standardized at 68°F (Ep_{68}) following Equation 3.

$$SN_{eff} = 0.0045 \times D \sqrt[2]{E_{p68}} \qquad [3]$$

Where,

SN_{eff} = Effective Structural Number from the existing pavement section

D = Total pavement thickness over subgrade, in

Ep_{68} = Backcalculated Standardized Composite Pavement Modulus, psi

The layer coefficient in Sections 40 and 41 was computed using Equation 4, which is a correlation between the layer coefficient and the elastic modulus of AC from the AASHO Road test. This method was initially used by Schwartz and Khosravifar (2013) and later applied by Díaz-Sánchez et al. (2017) and Diefenderfer and Apeagyei (2014).

$$a = 0.1665 \times \ln(E_{68}) - 1.7309 \qquad [4]$$

Where,

a = Composite Pavement structural coefficient

Ep_{68} = Backcalculated Standardized Composite Pavement Modulus, psi

Then, using the layer coefficient (a) computed from Equation 4, the AC/RAP layer SN was calculated according to Equation 5.

$$SN_{AC/RAP} = D_{AC/RAP} \times a \qquad [5]$$

Where,

$SN_{\mathrm{AC/RAP}}$ = Effective Structural Number

$D_{\mathrm{AC/RAP}}$ = Thickness of asphalt concrete layers, in

a = Composite Pavement structural coefficient

Subsequently, the Recycled Structural Number (SN_R) is calculated by the difference between the SN_{eff} (Section 43 and 44) or $SN_{AC/RAP}$ (Sections 40 and 41); and the product of the thickness and structural coefficient of the HMA layers (Equation 6). A typical loss of 20% of the asphalt structural coefficient (a_{AC}) is considered for low alligator and transverse cracking severity in accordance with the AASHTO method. Also, an NCAT study recommends an asphalt layer coefficient of 0.54 for new asphalt mixes in Alabama (Peters-Davis and Timm 2011).

$$SN_R = SN_{eff} - D_R \times a_{AC} \quad [6]$$

Where,

SN_R = Recycled Structural Number

SN_{eff} = Effective Structural Number

D_R = thickness of asphalt concrete layers, in

a_{AC} = Asphalt concrete structural coefficient

Finally, the layer coefficient for the cold recycled mix layers is computed by dividing the SN_R by their layer thickness. The last step in the process is to check for steady results by comparing the average layer coefficients within random locations and cold recycled test sections using statistical techniques. If the results are not adequate, the inputs for the backcalculation process in step 3 should be evaluated and then continue to the normal process flow.

3.5. SUMMARY

Four test sections were built in 2015 on Highway US-280 as part of a more extensive study conducted by the NCAT. This research evaluates the field performance of cold recycled materials (CCPR-E, CCPR-F, CIR-E, and CIR-F) under real conditions and evaluates its structural contribution in a pavement structure.

The functional performance of the four test sections was tested by collecting rut depth, cracking, and ride quality data on a routine basis over 4.5 years. The structural performance was evaluated using FWD testing, and the data were filtered, analyzed, and backcalculated to assess the performance over time under actual weather and traffic conditions.

Finally, a statistical analysis was performed to assess the effect of the CR method on the structural and functional performance of the treatments.

CHAPTER 4: RESULTS

4.1. INTRODUCTION

NCAT has monitored the structural and functional performance of the four recycled sections in Highway US 280 periodically. The functional performance was determined by cracking, rut depth measurements, and ride quality for each test section. A falling-weight deflectometer was used to conduct deflection testing to assess the structural performance of the four recycled test sections over time. The structural layer coefficient was tested for statistical analysis using a general linear model and compared by the Tukey test.

4.2. STRUCTURAL CAPACITY

Data collected for structural capacity analysis of the sections started in the last quarter of 2015. From the three different load levels collected, only the load level 2, which is 9,000 lbf was used for the analysis. This load level is often applied for most highway pavement testing, research so that it will be easier to compare results with other project sites. Also, this load level represents a standard 18,000 lbf axle load.

4.2.1. Raw data analysis

The next step following the flow chart previously shown in Figure 22 is to evaluate raw deflection data. The collected deflection files were stored in a database for analysis. This process is critical to filter outlier values that can be caused by shallow bedrock, shallow stiff layer, water table, and others. Due to the amount of collected data at each geophone per random location, a straightforward procedure to filter data is by using the deflection measured by the first geophone located on top of the load plate, known as D_0. Figure 25 shows the D_0 deflections of the recycled sections distributed quarterly over time. The data displayed represents the average of the random locations and the repetitions at each section.

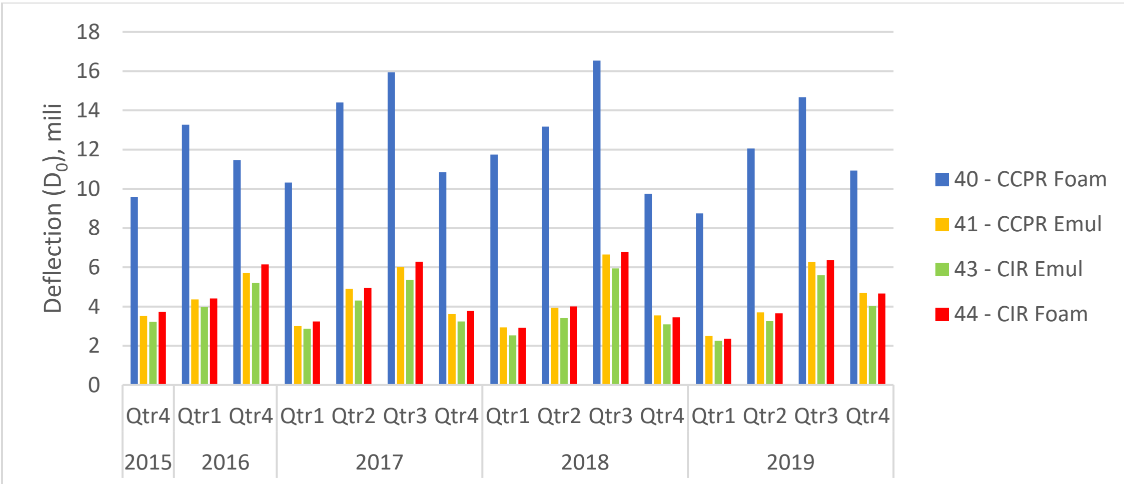

Figure 25. Deflections measured at the first geophone (D_0)

The layers influence the center deflection underneath the surface; therefore, it is expected that the sections have different performance under load drops because they have different structures. It can be observed that section CCPR-F (40) consistently has the highest deflections in comparison with the others. This expected due to the thinner asphalt layer in this section.

A more detailed evaluation of Section CCPR-F (40) can be observed in Figure 26. It can be noticed that random locations one and two (RL-1 and RL-2) exhibited higher deflections than RL-3, resulting in an average deflection higher than the other recycled sections. A non-uniform pavement structure can explain this variability throughout the length of the section, which can be confirmed by the data displayed in the graphic where RL-3 has the lowest deflections.

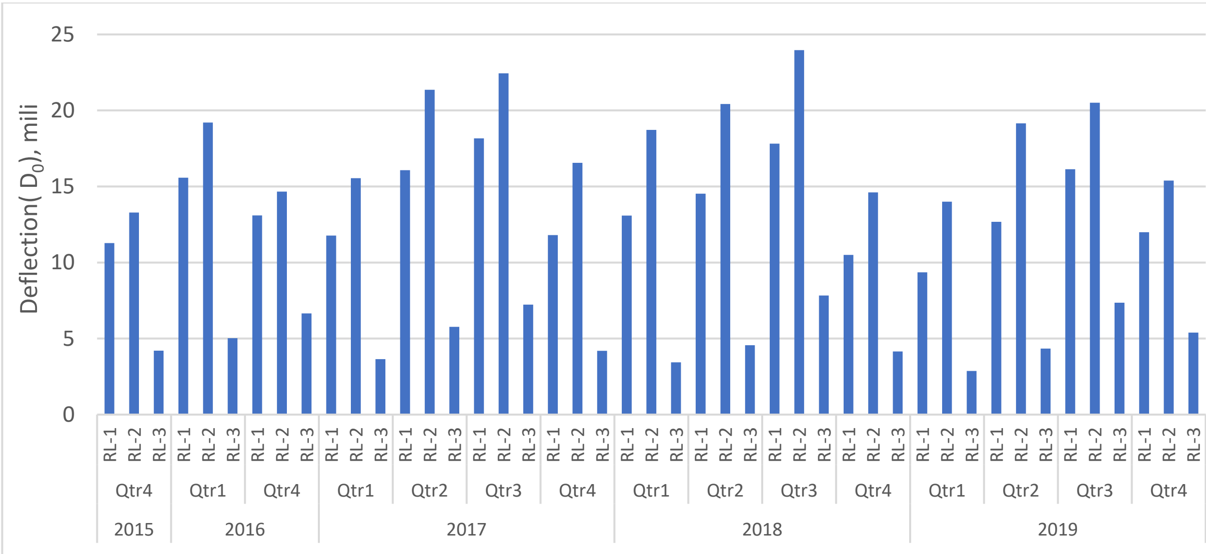

Figure 26. Deflections (D_0) on random locations of section 40

Another method to analyze structural capacity using raw deflection data is by the surface modulus concept. The analysis is based on Boussinesq's original closed-form equations; this is very useful for estimating subgrade modulus and for diagnosing stress-sensitive subgrade material. This approach uses the deflection measured by the nine geophones, and not only the center deflection, which is dependent on the moduli of each layer below. This assessment can be used to determine if the subgrade or the upper materials influence CCPR-F (40) section. The surface moduli are calculated at horizontal distance r, which is determined by the location of the geophones. The outer deflections fall within a zone where only the subgrade contributes to the surface moduli calculation (Horak 2007).

The relationship between the vertical deflection and the surface modulus is described by Equations 7 and 8.

$$E_0 = \frac{2P(1-\mu^2)}{\pi a D_0} \qquad [7]$$

$$E_r = \frac{P(1-\mu^2)}{\pi r D_r} \qquad [8]$$

Where,

E_0= Surface modulus at the center plate, psi

E_r= Surface modulus at the center plate at distance r, psi

P= Applied load, lbs

r= radial distance from center load, inch

D = surface deflection, inch

The surface modulus results in Figure 27 (tested in December 2015) show a similar normal trend and a linear elastic subgrade in sections CCPR-E (41), CIR-E (43), and CIR-F (44). On the other hand, CCPR-F (40) surface moduli might indicate that the subgrade strongly influences the pavement structure. Sensors 6 to 9 (r =36 to 72 inches) represent the surface moduli under the influence of the subgrade; those results could imply the presence of shallow bedrock, stiff clay layers, or even a shallow water table based on the literature. Surface moduli assessed in different months during the research shows the same tendency.

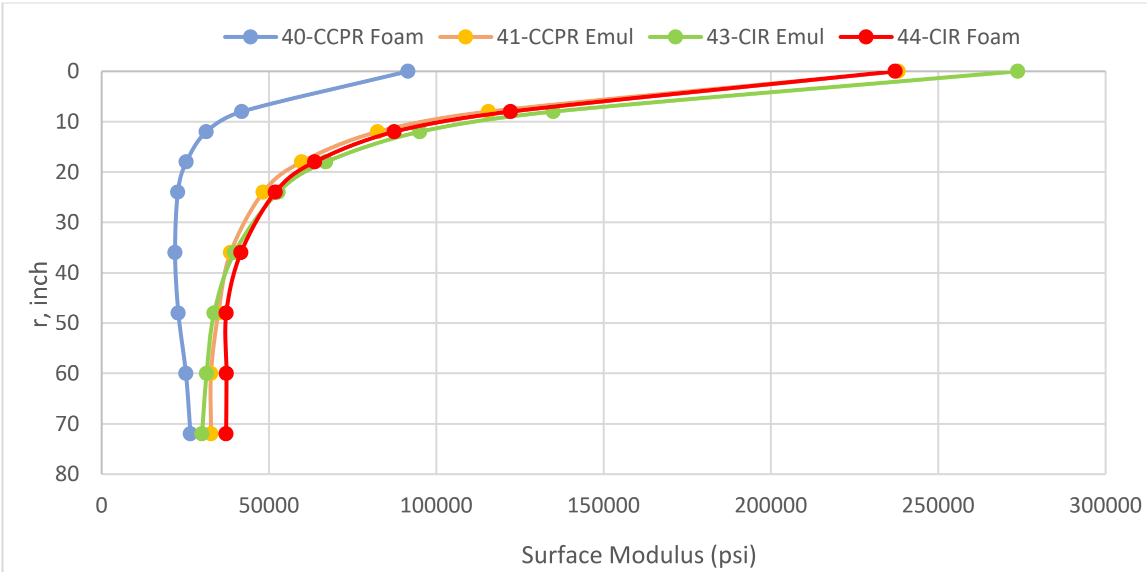

Figure 27. Recycled sections surface moduli

The effect of a shallow layer can be significant in deflections measured by the geophones; therefore, this matter can affect the backcalculation analysis since the subgrade is assumed to be a semi-infinite half-space, but in reality, the subgrade is only a few feet deep causing that the moduli calculated is underestimated. Generally, the effect of the shallow layer has little or no influence on the backcalculation process when the layer is deeper than 39.7 feet (Irwin 2002).

The depth of the shallow layer can be determined using the relationship presented in Equation 6 between the radius (in) and the deflection (mil.) tested at distance r. It is assumed that the deflection is expected to be zero when the radius is infinity. Therefore, Figure 28 shows a plot of deflections measured by sensor 6 to 9 at each random location of CCPR-F (40) section versus a/r, to the extent of the previous assumption if a/r has an intercept that is not zero, it indicates that an unknown layer may be present. The depth to the shallow layer is then determined at the radial distance where the deflection is zero.

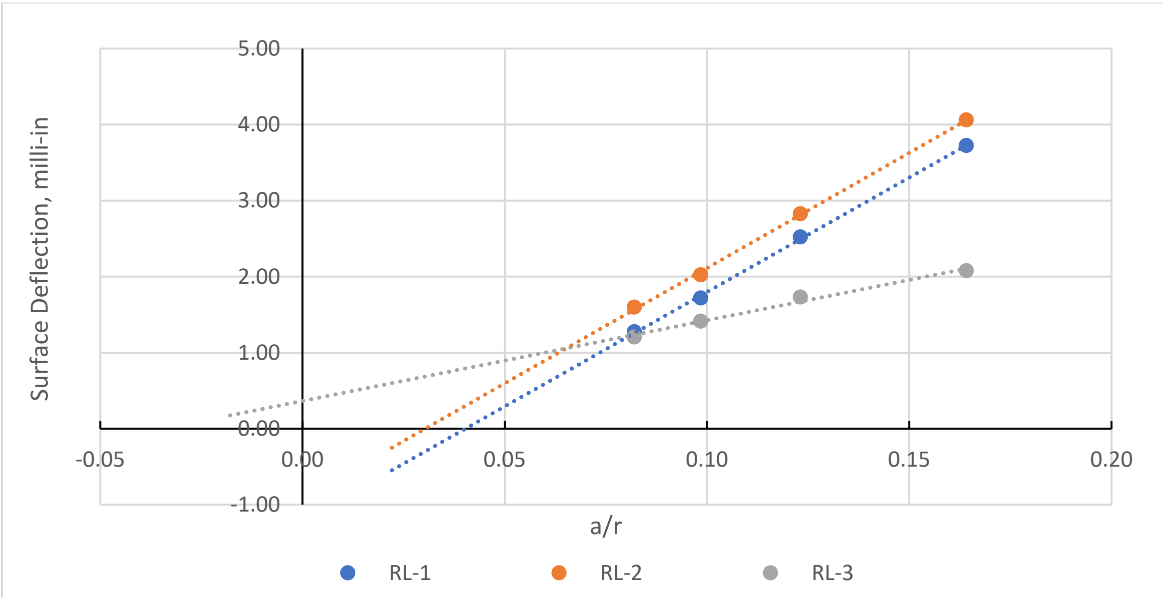

Figure 28. Prediction of depth to bedrock

The depth to shallow layer is shown in Table 13. Random locations 1 and 2 have a depth to a shallow layer of 12.2 ft and 16.2 ft, respectively, whereas random location 3 analysis concluded that there is no shallow layer present under the load. The other test sections were also verified for a shallow layer with negative results. Figure 29 shows a plot of the depth of the layer at each random location in the CCPR-F (40) section.

Table 13. Depth to a shallow layer

Random Location	Distance (ft)	m	b	Intercept	Shallow layer	depth (ft)
RL-1	136	30.108	-1.2134	0.0403	Yes	12.2
RL-2	220	30.326	-0.9206	0.0304	Yes	16.2
RL-3	409	10.614	0.3643	-0.0343	No	0.0

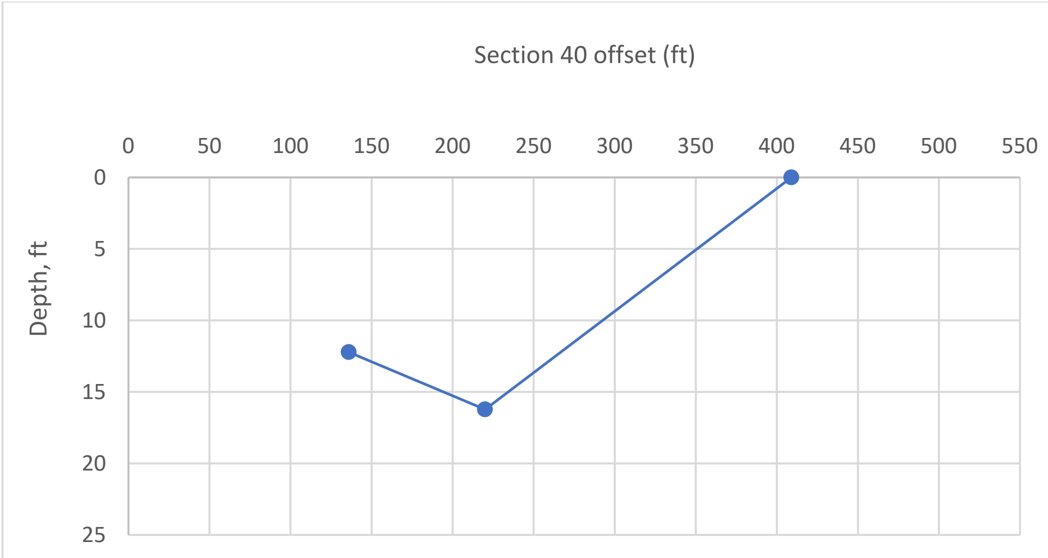

Figure 29. Depth to unknown layer at each random location

The shallow layer is present at a depth under 39.7 feet; therefore, it can affect the backcalculation process. As the depth to the shallow layer was computed, it can be modeled in the analysis to generate accurate results.

The GPR testing results for CCPR-F (40) section are presented in Figure 30. The thickness around the third random location, which is located around milepost 131.978, has a high standard deviation of 1.66 in due to the thickness variability, whereas the standard deviation of random location RL-1 and RL-2 are 0.38 in and 0.25 in, respectively. The variability of random location RL-3 is likely due to a pavement structure transition. Reliable results in the backcalculation are relevant for the recycled pavement analysis; therefore, random location RL-3 was not used for evaluation.

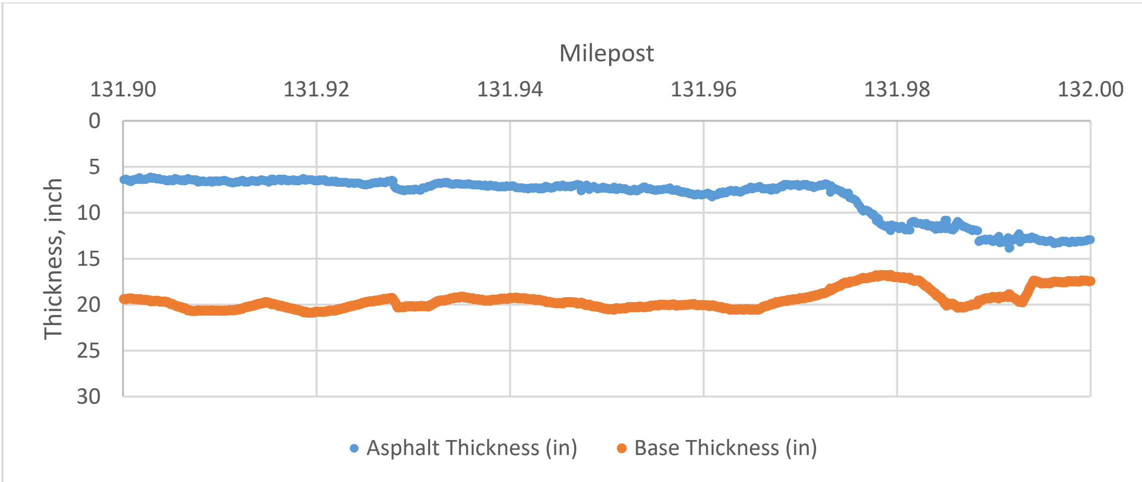

Figure 30. Section 40 GPR Layer Thickness

4.2.2. Backcalculation

The backcalculation of the layer moduli in the recycled sections was computed using the ELMOD 6 software, as mentioned in Chapter 3. The results were exported to an Excel file and temperature, and the time effect was analyzed using statistical tools.

Temperature and time-sensitivity of the backcalculated layer properties were analyzed to accomplish the objectives of this research. Mid-depth pavement temperatures were calculated based on Bell's equation for each random location.

A target load level of 9,000 pounds (actual loads ranging from 8,000 lbs. to 10,000 lbs.) was selected to analyze the temperature sensitivity of the asphalt pavement modulus (Ep). Figures 31 to 34 show the effect that temperature has on the Ep of the recycled sections. Also, Table 14 indicates the trendline equations for each section.

All the collected backcalculated Ep values for each pavement section have been plotted against the temperature in the horizontal axis. It can be concluded that the composite modulus of asphalt pavement (Ep) is temperature dependent. With the increase of temperature, the Elastic Modulus decreases exponentially. Nevertheless, CCPR-F (40) section has the lowest R-squared value, which indicates that temperature does not account for many variations in the composite modulus, this behavior can be explained by the layer composition of the test section where the old HMA has the thinnest thickness in association to the sections with values lower the 2.5 inches.

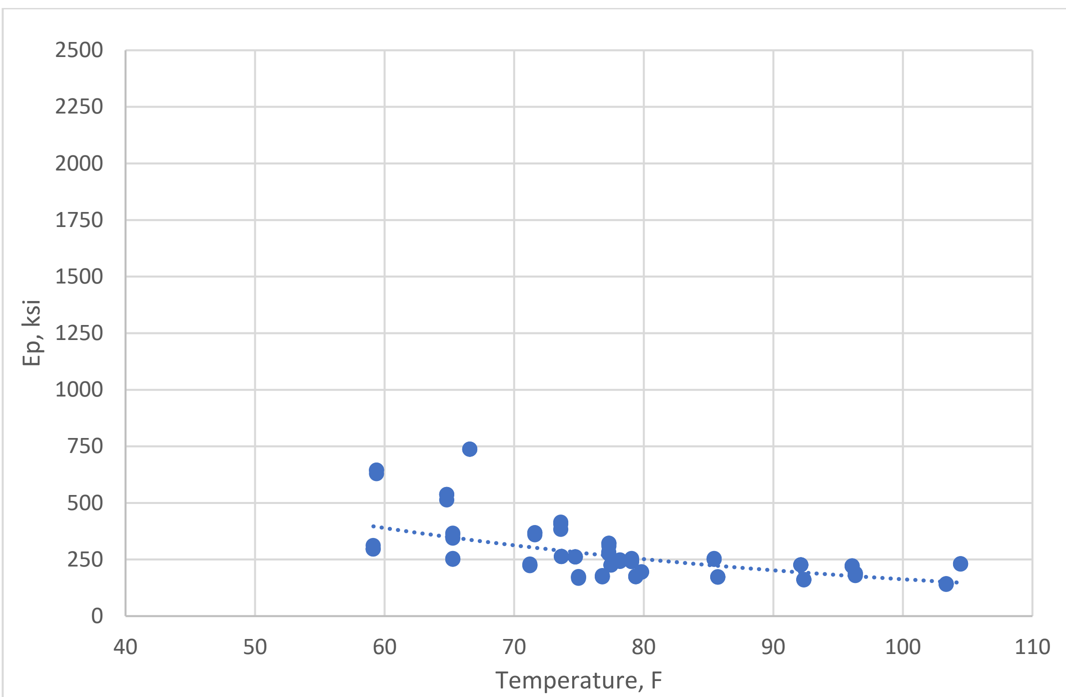

Figure 31. Temperature effect in Foamed CCPR section

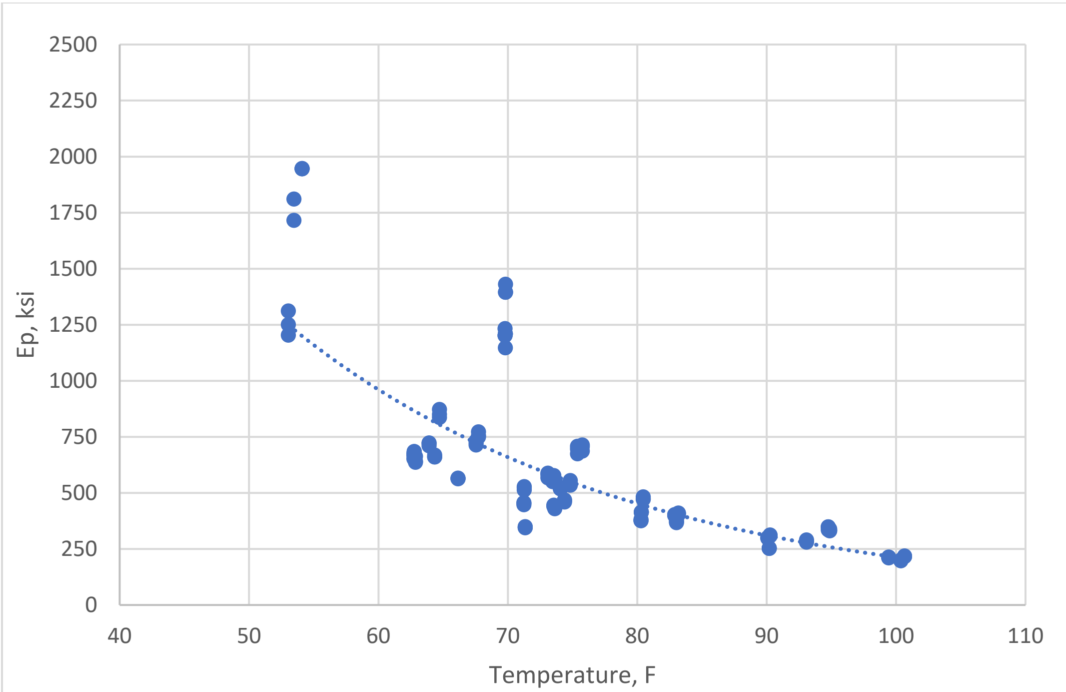

Figure 32. Temperature effect in Emulsified CCPR section

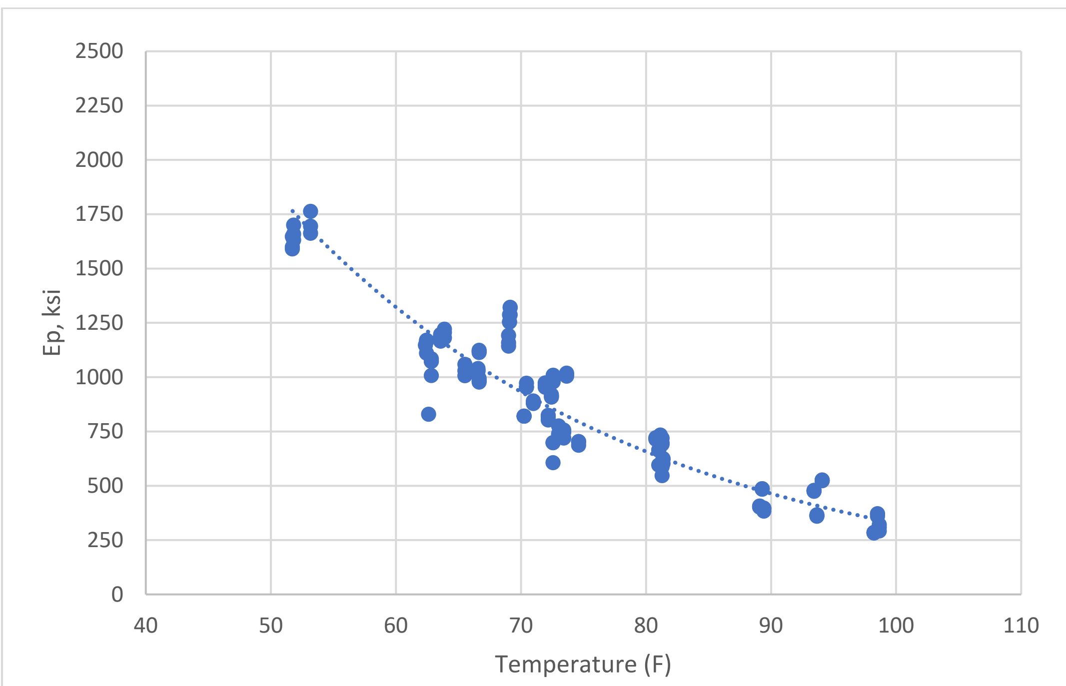

Figure 33. Temperature effect in Emulsified CIR section

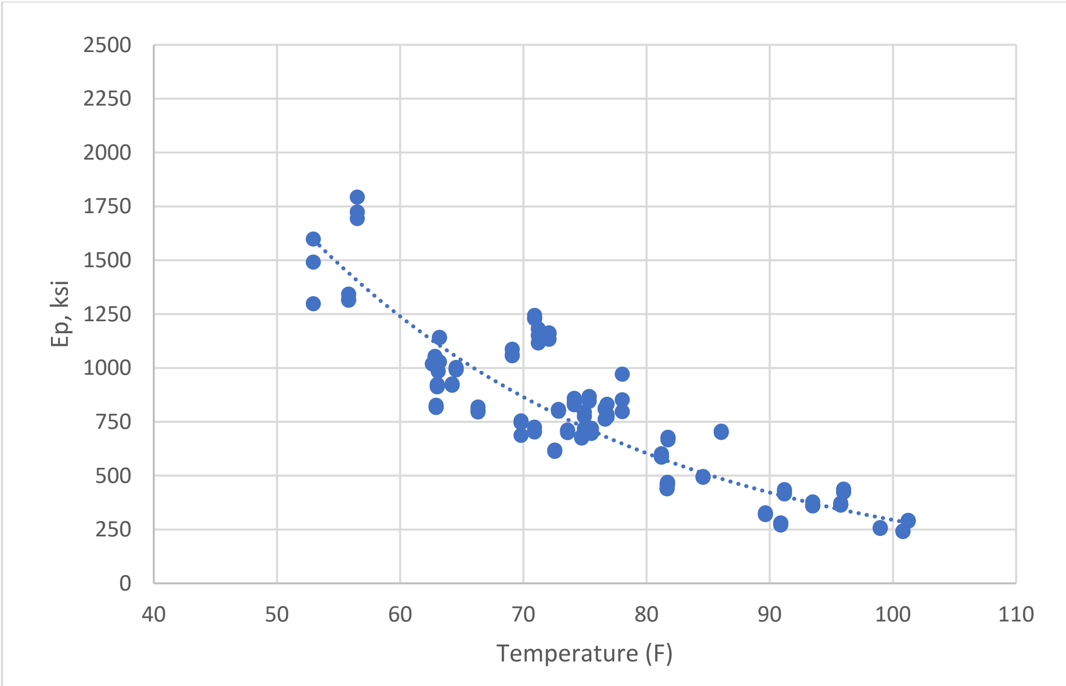

Figure 34. Temperature effect in Foamed CIR section

Table 14. Temperature trendline of the recycled sections

Section	R^2	Trendline equation
40-CCPR-F	0.44	$y = 1{,}437.398e^{-0.0218x}$
41-CCPR-E	0.75	$y = 9{,}193.578e^{-0.0376x}$
43-CIR-E	0.91	$y = 10{,}742.152e^{-0.0349x}$
44-CIR-F	0.83	$y = 10{,}703.747e^{-0.0359x}$

The backcalculated Ep was normalized to 68°F (Ep_{68}) using equations 1 and 2; the results were plotted along a timeline to observe the aging effect on the pavement per random location (Figure 35 to 38). The trendline of the composite modulus over time is presented in Table 15.

Table 15. Time trendline of the recycled sections

Section	m	Trendline equation
40-CCPR-F	-0.107	$y = -0.107x + 4{,}955$
41-CCPR-E	-0.119	$y = -0.119x + 5{,}885$
43-CIR-E	-0.016	$y = -0.016x + 1{,}676$
44-CIR-F	-0.035	$y = -0.035x + 2{,}458$

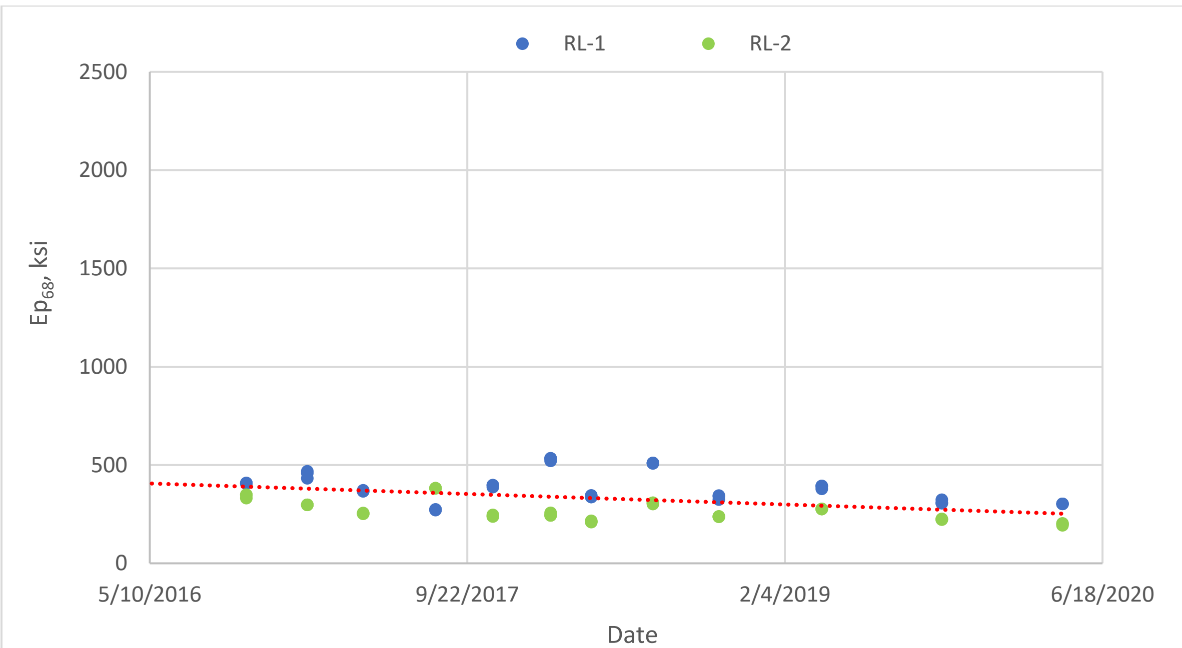

Figure 35. Time effect on the Foamed CCPR section

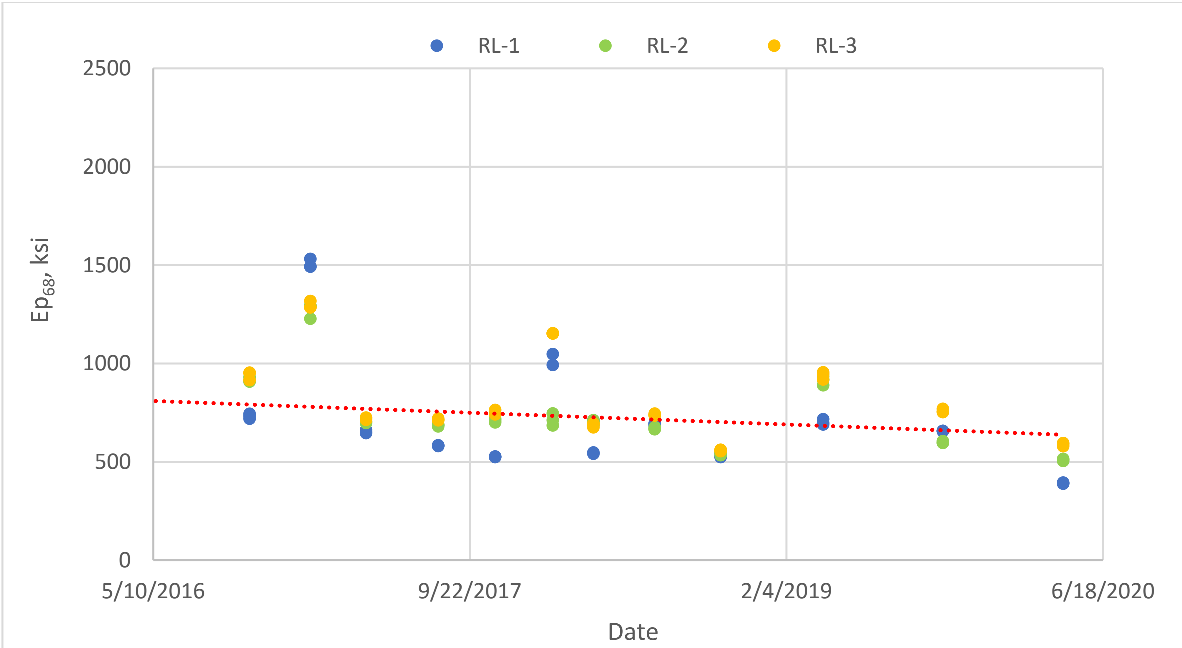

Figure 36. Time effect on the Emulsified CCPR section

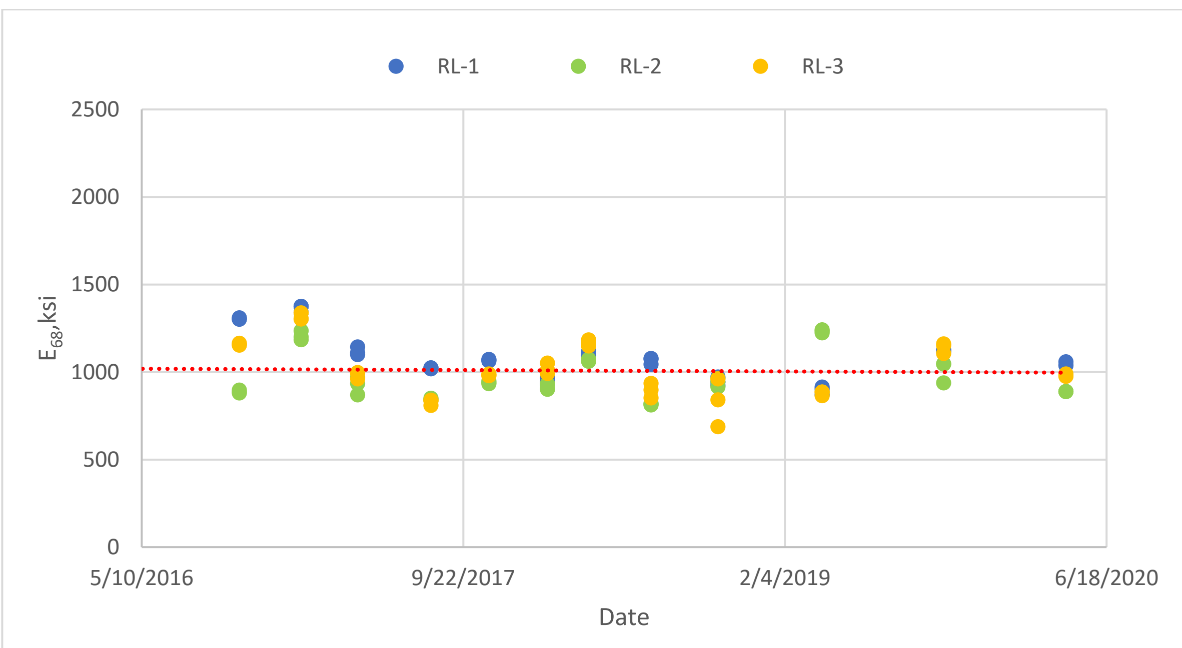

Figure 37. Time effect on the Emulsified CIR section

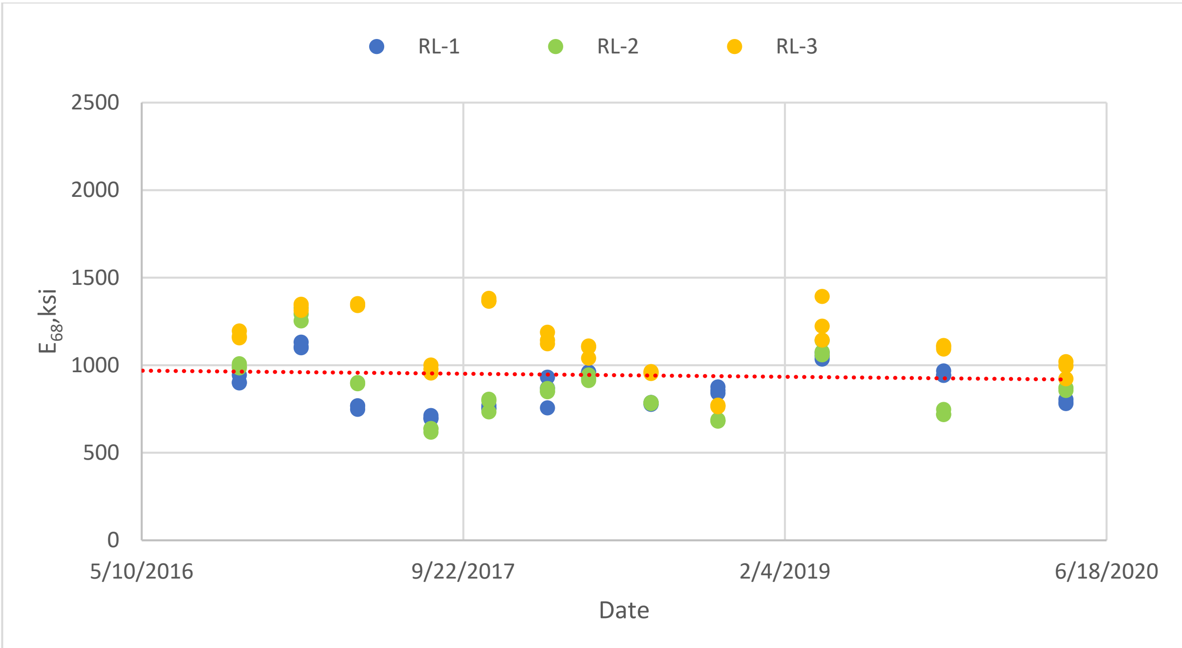

Figure 38. Time effect on the Foamed CIR section

When aging occurs, the normal behavior of hot mix asphalt layers is to become stiffer with high Ep values on top layers; this can be reflected with a positive trendline slope. The confidence interval of the regression analysis of the sections is presented in Table 16. The slope of sections CIR-E and CIR-F goes through zero, which indicates that statistically, there is no significant relationship between Ep_{68} and time.

On the other hand, P-value close to zero in sections CCPR-F (40) and CCPR-E (41) rejects the null hypothesis that the slope of the trendline is equal to zero (no effect). Based on the results, only CCPR-F(40) and CCPR-E (41) sections have statically decreased their composite pavement modulus over time; this behavior can be attributed to their recycled thickness because they are greater than the old asphalt layer. The combination of the old asphalt layer and the recycled one can generate the modulus decrease. A summary of the average, standard deviation, maximum and minimum values per random locations of Ep_{68} is presented in Table 17.

Table 16. Confidence Interval

Section	P-value	Upper C.I 95%	Lower C.I 95%
40-CCPR-F	0.000	-0.1570	-0.0570
41-CCPR-E	0.000	-0.2055	-0.0334
43-CIR-E	0.584	-0.0713	0.0403
44-CIR-F	0.359	-0.1104	0.0403

Table 17. Composite Asphalt modulus

Sections	**Average, Ksi**	**Std. Dev.**	**Max., Ksi**	**Min., Ksi**
CCPR Foam	**343**	**115**	**714**	**194**
RL-1	403	116	714	271
RL-2	283	77	501	194
CCPR Emulsion	**742**	**229**	**1532**	**390**
RL-1	709	269	1532	390
RL-2	722	194	1294	505
RL-3	796	213	1318	531
CIR Emulsion	**1010**	**143**	**1376**	**687**
RL-1	1070	130	1376	870
RL-2	962	132	1241	712
RL-3	998	147	1338	687
CIR Foam	**948**	**193**	**1392**	**617**
RL-1	867	123	1131	694
RL-2	875	166	1326	617
RL-3	1103	185	1392	762

The section with the greatest composite asphalt modulus value is the CIR emulsion, and the one with the minor value is the CCPR foamed section. More profound conclusions from the data provided are not suggested because each section has a different structure layer composition, although it is critical to mention that the largest standard deviation is in section

41, the CCPR emulsion. A more accurate comparison between sections is by the computation of the structural coefficient of each recycled section. This value is calculate using Equations 3 to 6, which separate the old asphalt layer and the thin overlay from the cold recycled mix.

Table 18 presents a statistical description of the recycled structural coefficient grouped by the method of construction and recycling agent. Boxplots of the sections are shown in Figure 39. Besides, Figures 40 to 43 help to visualize how replicates of layer coefficients are scattered through the study.

Table 18. Recycled structural coefficient

Section	Method	Recycling agent	N	Average	Std. Dev.	Max	Min
40	CCPR	Foam	84	0.31	0.092	0.539	0.153
41	CCPR	Emulsion	125	0.66	0.154	1.067	0.324
43	CIR	Emulsion	126	0.43	0.092	0.653	0.197
44	CIR	Foam	126	0.38	0.103	0.579	0.197

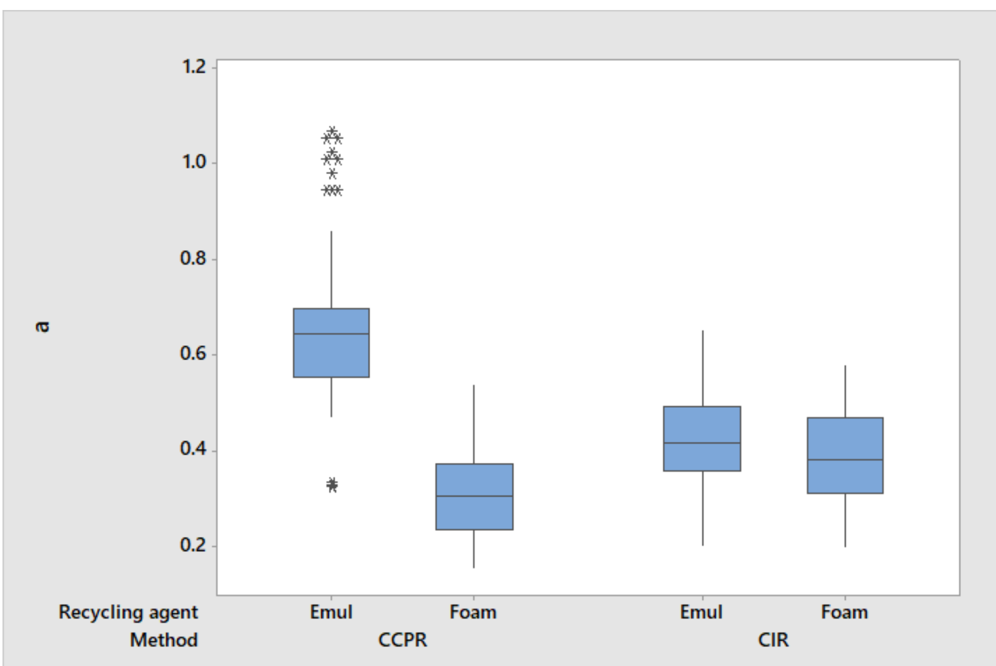

Figure 39. Boxplot of recycled structural layer coefficient

The layer coefficient results show a different behavior as the asphalt composite moduli; for instance, the CCPR Emulsion section has the largest layer coefficient value; however, the composite modulus of the same material is not the greatest of the sections. The CCPR foamed section has the lowest results in layer coefficient and composite pavement modulus.

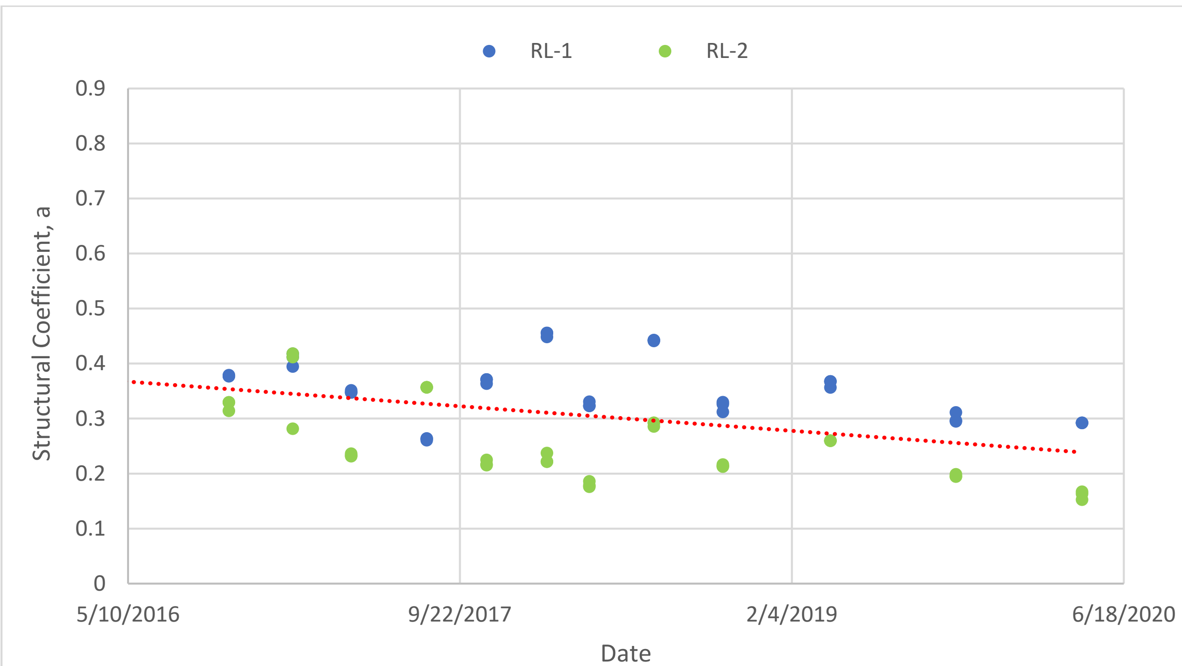

Figure 40. Time effect on Section CCPR-F (40) layer coefficient

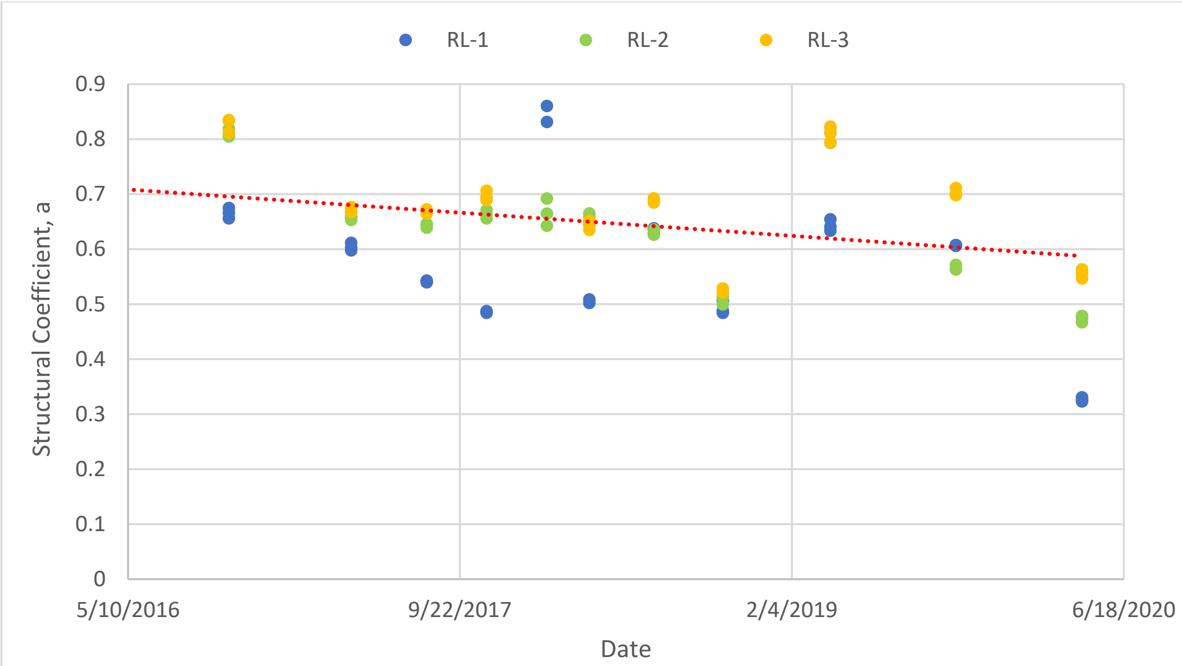

Figure 41. Time effect on Section CCPR-E (41) layer coefficient

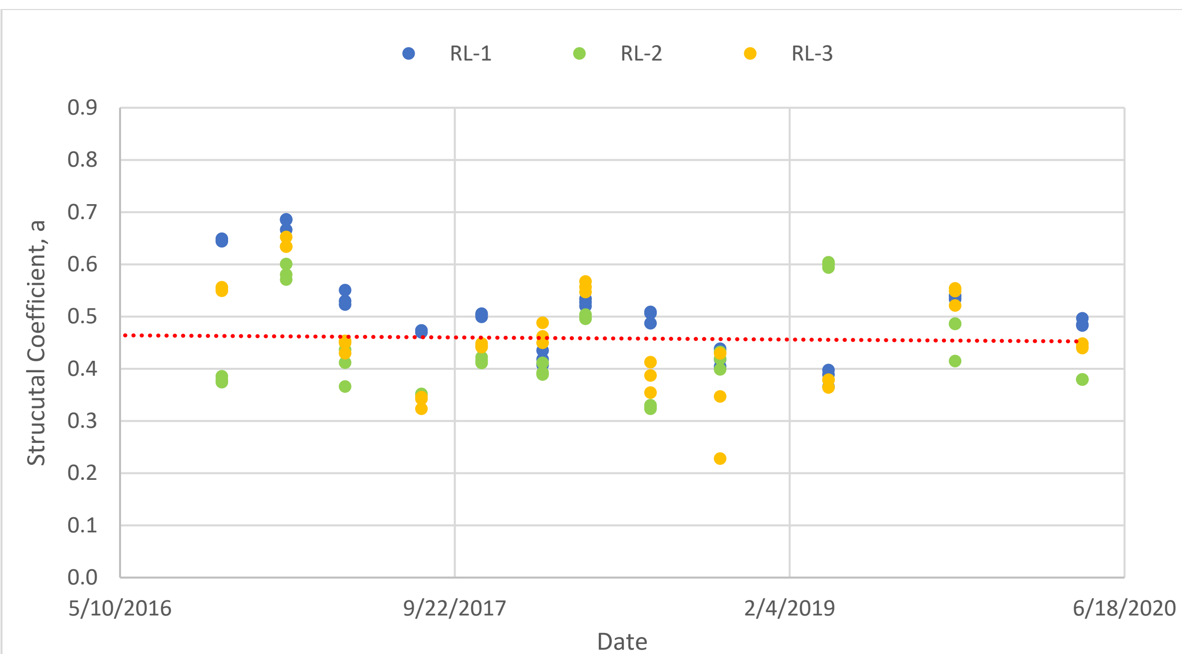

Figure 42. Time effect on Section CIR-E (43) layer coefficient

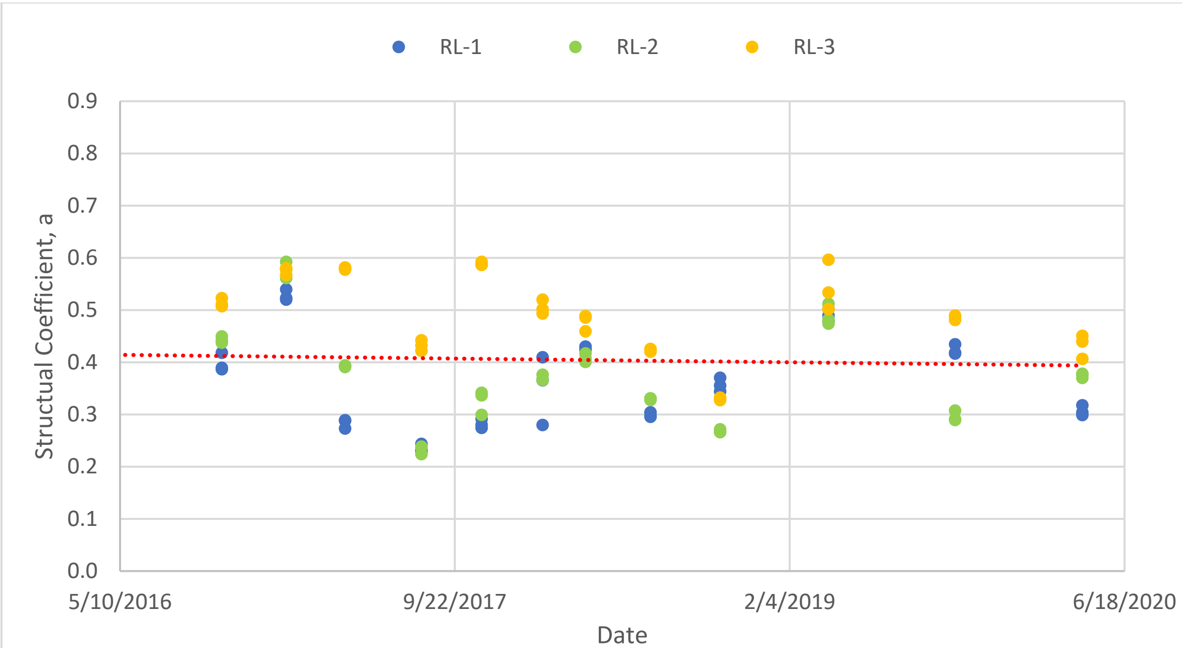

Figure 43. Time effect on Section CIR-F (44) layer coefficient

A general linear model is applied to analyze the effects of the construction method and recycling agent in the cold recycled performance; each factor has two levels (Table 18). The analysis of variance of the model using a significance level of 0.05 is presented in Table 19.

Table 19 Factor information

Factor	Type	Levels	Values
Method	Fixed	2	CCPR, CIR
Recycling agent	Fixed	2	Emulsion, Foam

Table 20. Analysis of Variance

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Method	1	0.7372	0.73724	56.33	0.000
Recycling agent	1	4.2358	4.23576	323.67	0.000
Method*Recycling agent	1	2.5051	2.50506	191.42	0.000
Error	457	5.9807	0.01309		
Total	460	13.5750			

All the factors mentioned have a p-value lower than 0.05; the analysis suggests that at 95% confidence, the method, recycling agent, and the combination of both have a significant impact in the structural coefficient calculations. The recycling agent factor has the largest F-value, which can be interpreted as the most significant factor in the analysis and can be easily observed in Figure 44, where the effect of the main effects is plotted. The main interaction plot (Figure 45) has nonparallel lines, which reflect that interaction occurs. This interaction effect suggests that the relationship between the construction method and structural coefficient depends on the recycling agent.

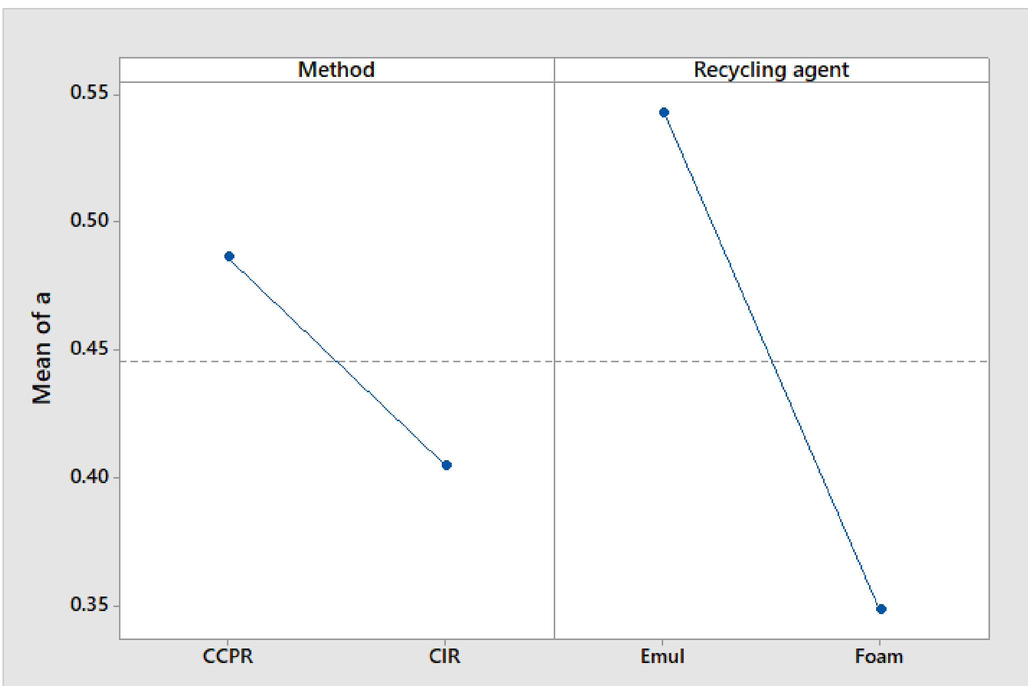

Figure 44. Main effects plot

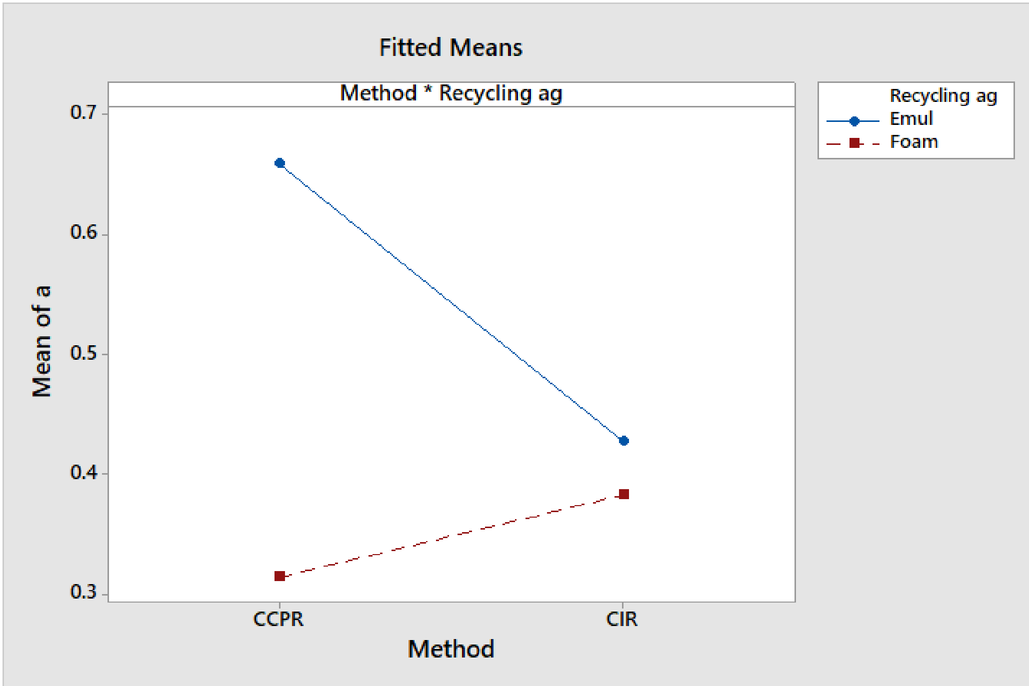

Figure 45. Interaction Plot

The average layer coefficient computed in the CCPR-E section is 0.66 in. $^{-1}$, this computed value is unexpected because it is greater than the layer coefficient assigned to a new HMA layer. Additionally, it is not similar to the values given in previous research studies, as can be seen in the literature review chapter (Table 5). A possible error in the computation of the layer coefficient is using wrong thicknesses when separating the composite pavement modulus into the recycled layer and old HMA layer. The CCPR-E section was filtered out of the statistical analysis because of the great unforeseen value, a large standard deviation for structural coefficient, and composite modulus, which all combined can affect the assessment of the remaining three sections.

The one-way analysis of variance (ANOVA) was applied to the three remaining sections to determine whether there are any statistically significant differences among their means (Table 21). The null hypothesis of the test is that all the means are equal, at 95% confidence, the hypothesis was rejected with a P-value lower than 0.05.

Table 21. ANOVA in sections 40, 43 and 44

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Section	2	0.6486	0.324296	35.04	0.00
Error	333	3.082	0.009255		
Total	335	3.7306			

The ANOVA test evaluates the variability between groups to determine if the mean differences are statistically significant. A post hoc test known as the Tukey test was applied in the model (Table 22), which compares the groups in pairs to determine which couples are meaningful in the analysis.

Table 22. Tukey test of the recycled sections

Section	N	Mean	Grouping		
40 CCPR-F	84	0.3143	A		
43 CIR-E	126	0.4277		B	
44 CIR-F	126	0.3828			C

The group results have a different letter, which means they are significantly different. The analysis requires three comparisons to cover all combinations at a significance level of 0.05. Table 23 below presents the comparisons, the difference between group means, and the adjusted p-value for each comparison. The p-values for each comparison are less than the significance level; the difference between group means is statistically significant. Figure 46 shows this comparison; if zero is contained within a confidence interval, it indicates that there is no statistical evidence that group means are different. None of the differences between group comparison have a zero value; then, the difference between those pairs of groups is statistically significant.

Table 23. Tukey test for difference of means

Group Comparison	Difference of Means	SE of Difference	Simultaneous 95% CI	T-Value	Adjusted P-Value
43 - 40	0.1134	0.0136	(0.0817, 0.1452)	8.37	0.000
44 - 40	0.0685	0.0136	(0.0368, 0.1002)	5.05	0.000
44 - 43	-0.045	0.0121	(-0.0733, -0.0166)	-3.71	0.001

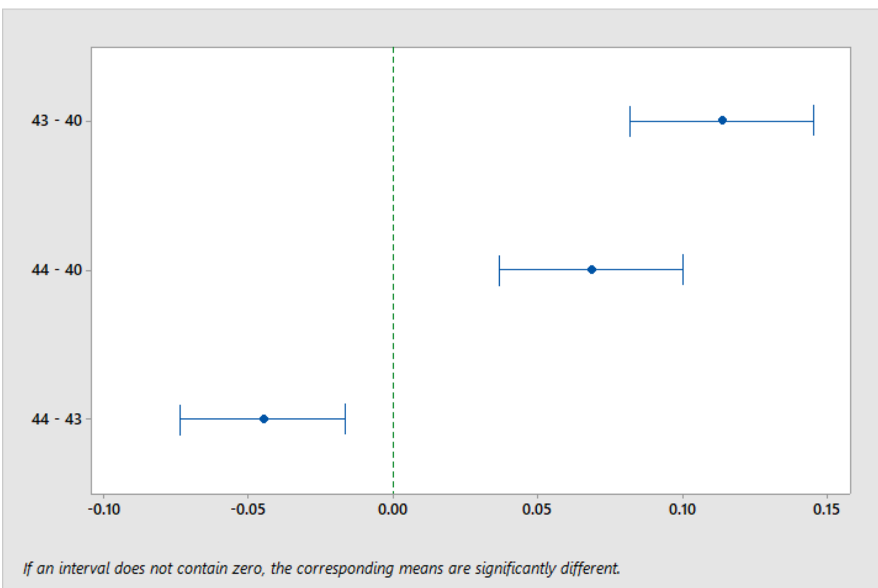

Figure 46. Differences of structural coefficient means

The recommended structural coefficients for the different techniques are presented in Table 24; values shown are the 25th percentile of the distribution per section as a conservative approach for structural pavement design, a conservative approach was selected to address the variability that design and construction can have to the overall performance of a cold recycled mix. A structural coefficient of 0.35 was selected for the CCPR Emulsion due to the enormous magnitude calculated in the section, which is larger than the structural coefficient of a new HMA layer. This value is also within the range and consistent with the existing literature.

Table 24. Recommended structural coefficients

Section	Method	Recycling agent	Structural coefficient
40	CCPR	Foam	0.23
41	CCPR	Emulsion	0.35
43	CIR	Emulsion	0.35
44	CIR	Foam	0.31

4.3. FUNCTIONAL PERFORMANCE

From October 2015 to April 2020, the accumulated traffic load is approximate 2.8 million ESALs. The functional performance of the recycled test sections at the closure of the quarterly test in 2020 is summarized in Table 24.

The sections have demonstrated a good international roughness index (IRI) performance; only the section CCPR-F (40) is in “fair” condition according to the MAP-21 criteria. There were some differences in the rutting resistance in the test sections, but the variations are not considered significant. Similarly, at the end of this research, the four recycled sections showed a minimum amount of cracking, but cracking resistance started to decrease in one section by the end of the study.

Table 25. Functional performance summary

Section	Description	Rutting (in.)	IRI (in./mi)	Cracking (%)
40	Foamed CCPR	0.27	82.2	4.2
41	Emulsion CCPR	0.16	77.6	0.2
43	Emulsion CIR	0.21	94.0	1.3
44	Foamed CIR	0.22	64.2	0.8

4.3.1. Rutting

The rutting performance of the test sections over 4.5 years of service is presented in Figure 28. As mentioned previously, by the time of this study, approximately 2.8 million ESALs had been applied to the test sections, and the reported annual average daily traffic (AADT) in 2018 was 20,083 vehicles. The figure shows two evident increases, the first one around 1.5 years, and the next one around 2.7 years. This behavior may be attributed to an increase in pavement temperatures over the summer months. The initial rut jump measure at the start of the study can be explained to the densification that occurs in the initial service phase causing a reduction of the air voids.

Although the data shows an increasing trend in all the sections, which is common in asphalt pavements open to high traffic volume, oscillating results are shown. This effect can be due to a seasonality effect, which can be observed during the third winter when the mean rut depth seems to maintain more constant values, and a measurement error can be attributed to this behavior.

Each section is divided into 32 subsections, which were used as replicates to evaluate for mean differences in rutting among the sections using the ANOVA test. October 5, 2015 (First date post-construction) and April 21, 2020 (Last date) were the days selected for the analysis (Table 25 and 26).

Table 26. ANOVA analysis post-construction

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Section	3	0.002278	0.000759	1.3	0.278
Error	124	0.072494	0.000585		
Total	127	0.074772			

Table 27. ANOVA analysis for the last date

Source	DF	Adj SS	Adj MS	F-Value	P-Value
Section	3	0.184	0.06134	1.32	0.272
Error	124	5.7772	0.04659		
Total	127	5.9612			

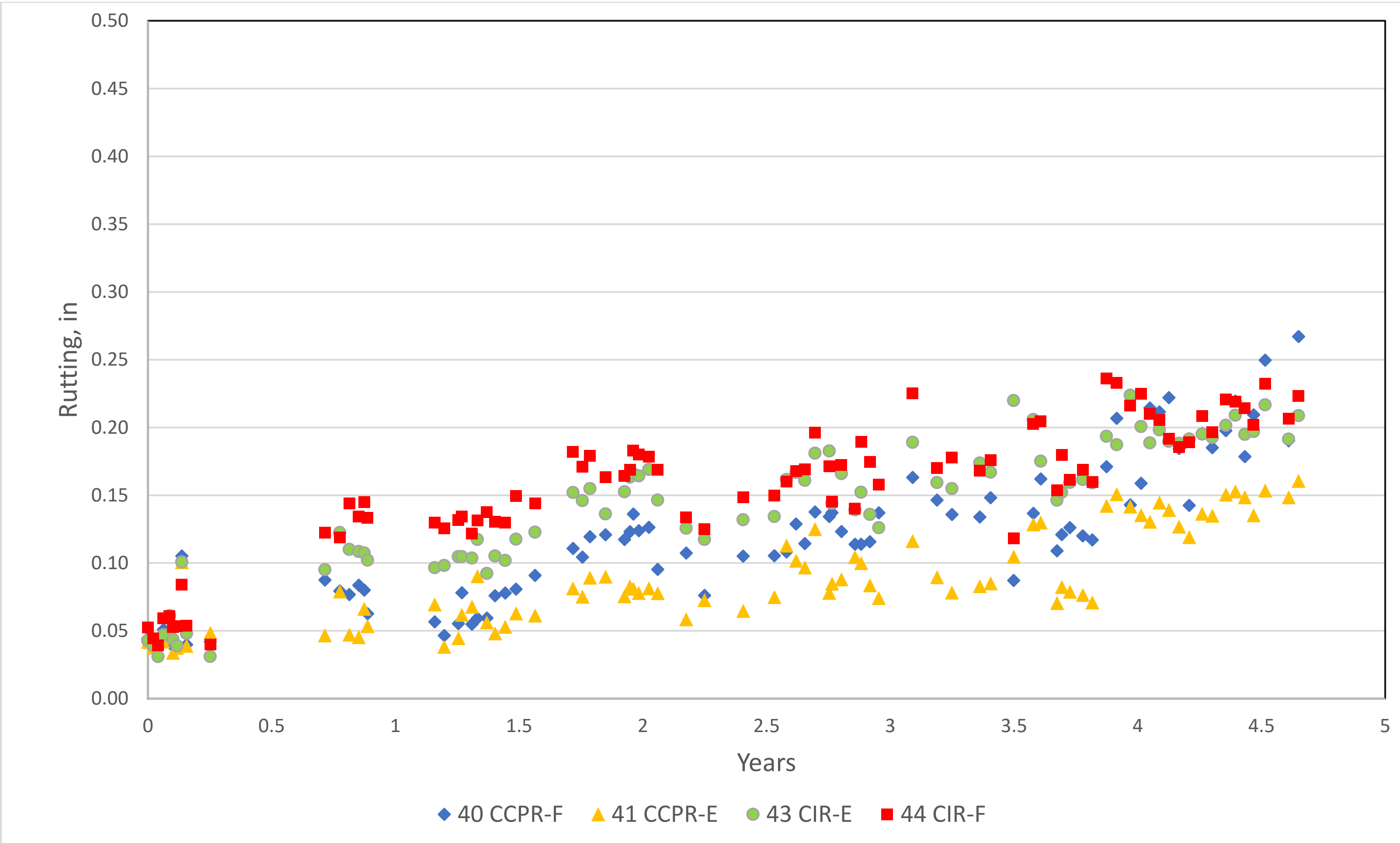

Figure 47. Rut performance of the recycled sections

The results show that on both dates, the P-value is higher than the significance level of 0.05. Hence there is not enough evidence to reject the null hypothesis, which is that the rutting means are all equal. The rut depth means for each date is shown in Table 28.

Table 28. Rutting average

Section	Post-construction rut depth means	Last date rut depth means
40-CCPR-F	0.04	0.27
41-CCPR-E	0.04	0.16
43-CIR-E	0.04	0.21
44-CIR-F	0.05	0.22

The average means have increased from post-construction to the last date, but all sections are statistically similar to each other on each of the dates. However, section CCPR-F is still in the "good" condition category being also the section with the highest structural coefficient values, while all the other three are in "fair" condition. By the end of this study, the mean rut depth of the four test sections was well below the poor criteria of 0.4 inches, established by the MAP-21 performance measures (Figure 48). The mean rut depths on April 2020 was less than the poor rutting criteria, which indicates the test sections had a good rutting performance after 4.5 years.

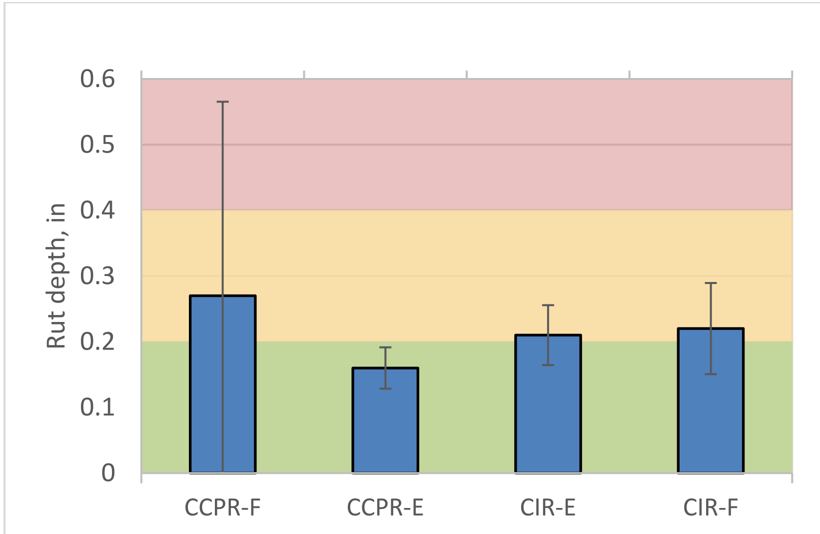

Figure 48. Mean rut depths of the recycled sections

Each recycled test section rutting performance can be shown in Figure 49 to 52. The foam CIR section was the first one to fall into the fair rutting condition at 3.1 years after construction, but the trend has been almost constant in the fourth year. The section that has exhibited better rutting resistance is section 41, which has been under the good indicator area for all the time analysis.

The section that has exhibited more rutting sensitivity along the study is section CIR-F, and the one which has the greatest rut depth by the end of the study is the CCPR-F section, this could be related to the low composite pavement moduli and the transition in its pavement structure along the section.

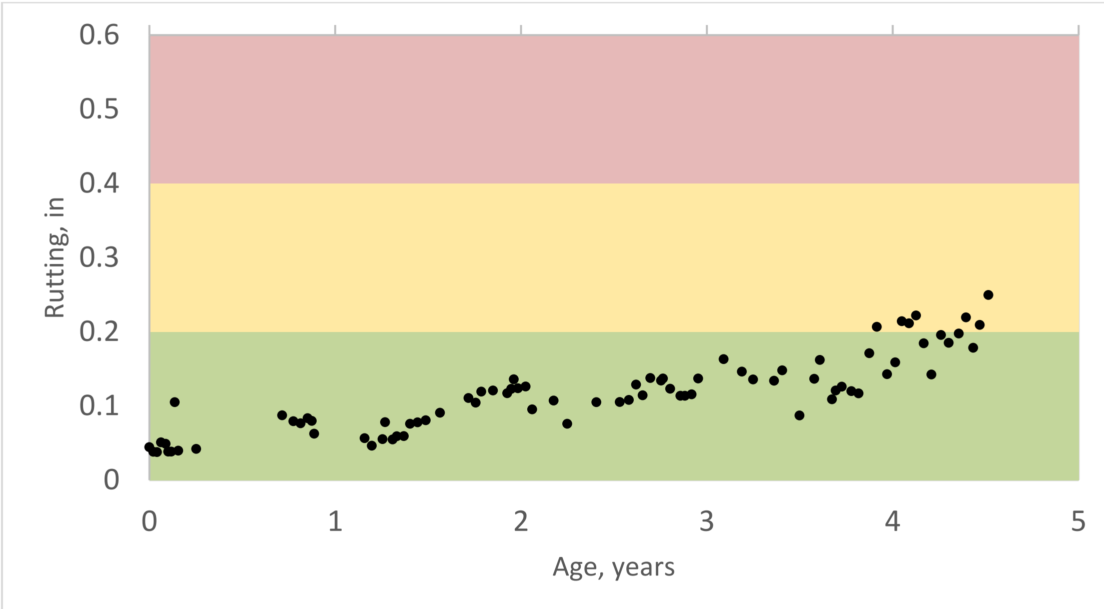

Figure 49. Thin overlay on foamed CCPR rut depth

In general, based on these results, all the recycled sections have had adequate rutting resistance under high traffic. Nevertheless, the more considerable rutting measure was approximately 0.25 inches in section CCPR-E, which is the one with the lowest composite pavement modulus and structural layer coefficient.

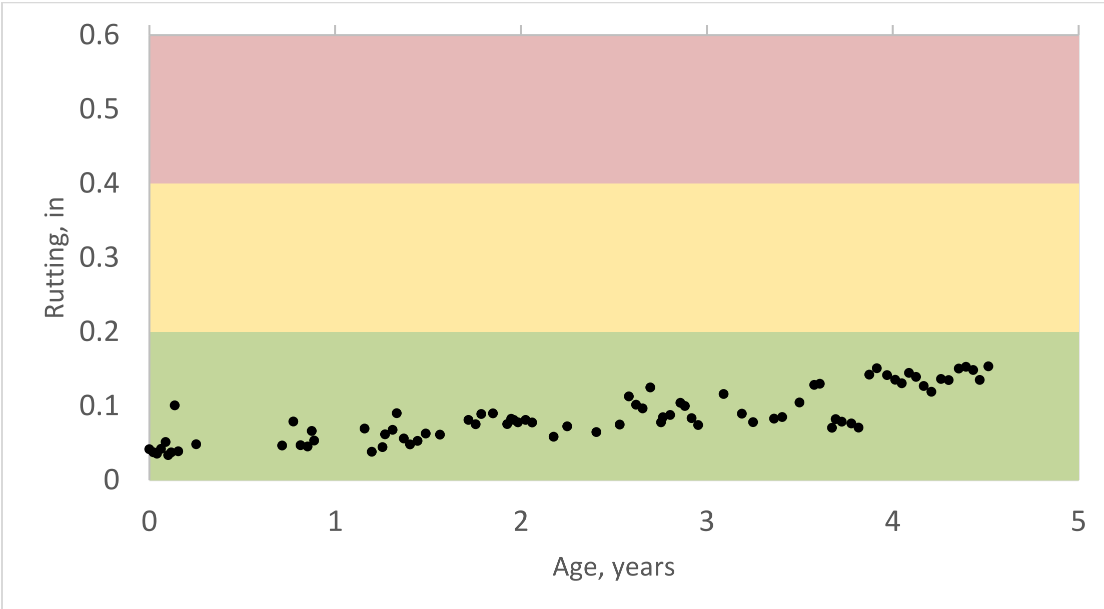

Figure 50. Thin overlay on emulsion CCPR rut depth

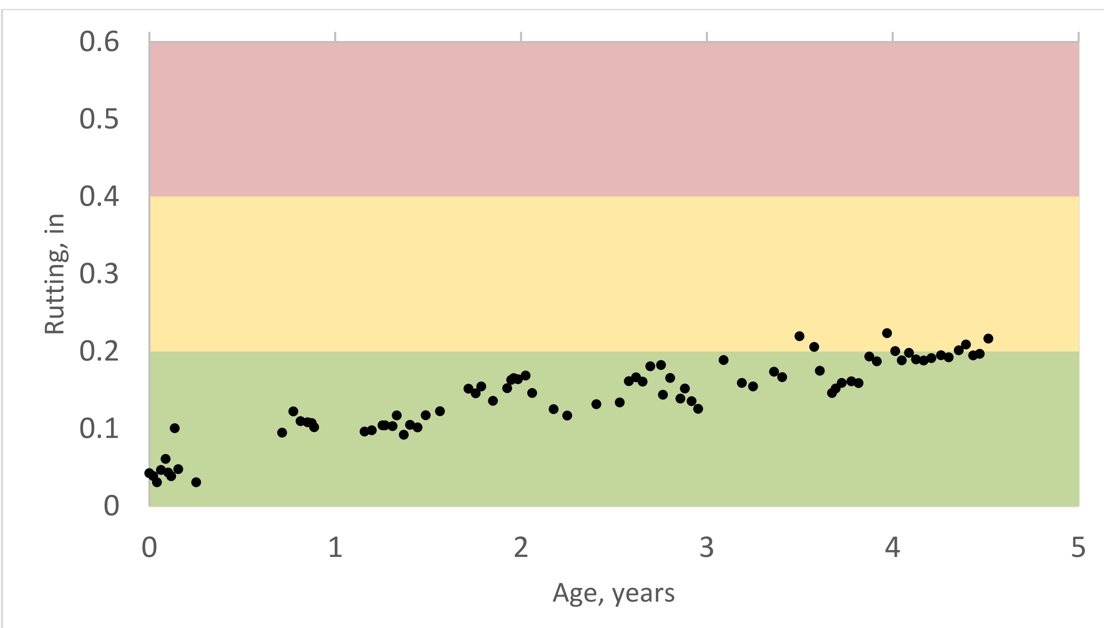

Figure 51. Thin overlay on emulsion CIR rut depth

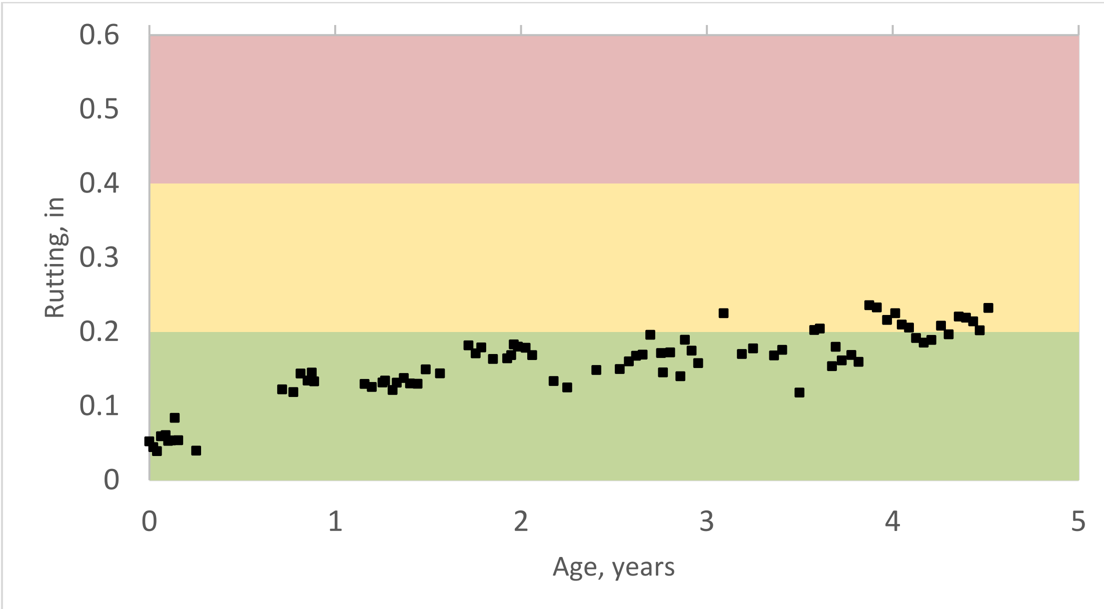

Figure 52. Thin overlay on foamed CIR rut depth

4.3.2. Roughness

Post-construction IRI was different for each section, but all of them were in “good” condition. By the end of this research, the four sections show a small increase in roughness. This behavior may be attributed to the loss of friction and texture in the surface layer due to axle load repetitions. In all cases, the measured IRI was maintained below the “fair” threshold of 95 in/mile established by the MAP-21 rating system. The CIR-F section appears to be more constant during the study, and the other sections have a different rate increase, being CCPR foamed section the most sensitive to change. The trendline of the variation in IRI over time is shown in Table 29.

Table 29. Trendline of IRI variation over time

Section	m	Trendline equation
40-CCPR-F	3.920	$y = 3.920x + 61.96$
41-CCPR-E	4.078	$y = 4.078x + 52.40$
43-CIR-E	2.531	$y = 2.531x + 81.30$
44-CIR-F	-0.476	$y = -0.476x + 66.42$

The initial IRI of the CCPR foamed section was 62.0 in./mi, then the section experienced a constant increase in roughness level through the entire study to a level of 82.2 in./mi. The measured IRI was maintained below the 90 in/mil threshold established as "fair" criteria (Figure 53).

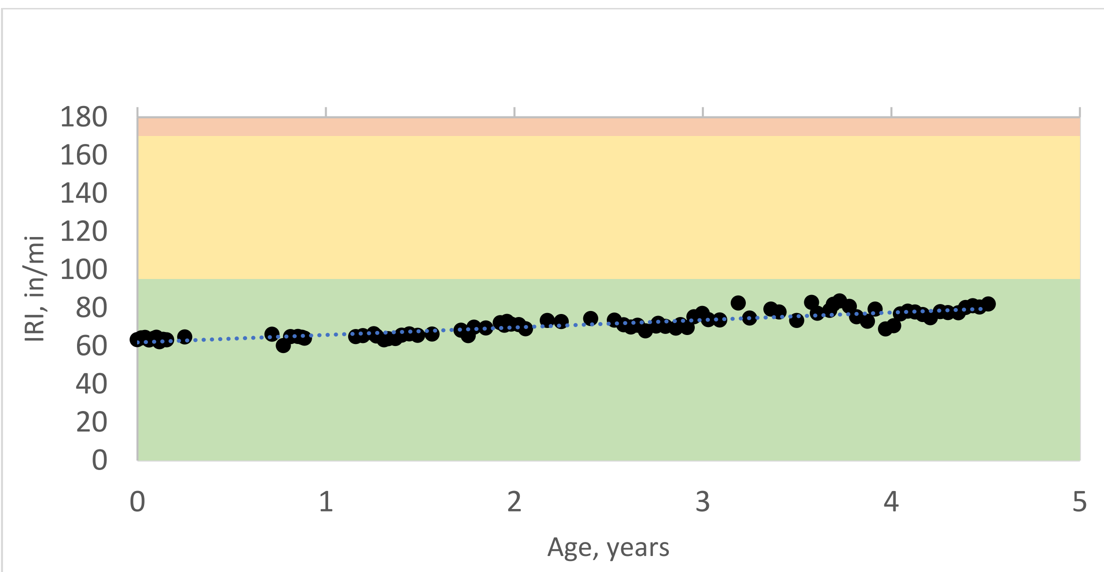

Figure 53. CCPR Foamed section roughness

The emulsion CCPR has the highest slope for IRI over time, but also, this section presents the lowest initial IRI measurement at 52.3 in./mi at the start of the study (Figure 54).

The CIR emulsion section showed higher initial IRI measurement at 78.1 in./mil. Moreover, it rose in the summer of 2017, falling into the “fair” category. However, the roughness level was steady at the end of the study (Figure 55).

The CCPR foamed section appears to be steadier over the study; its initial IRI value is 66.4 in./mi, which is similar to the CIR foamed section. However, the measured IRI did not change appreciably over time; actually, it presents a slightly negative slope that represents a roughness reduction (Figure 56).

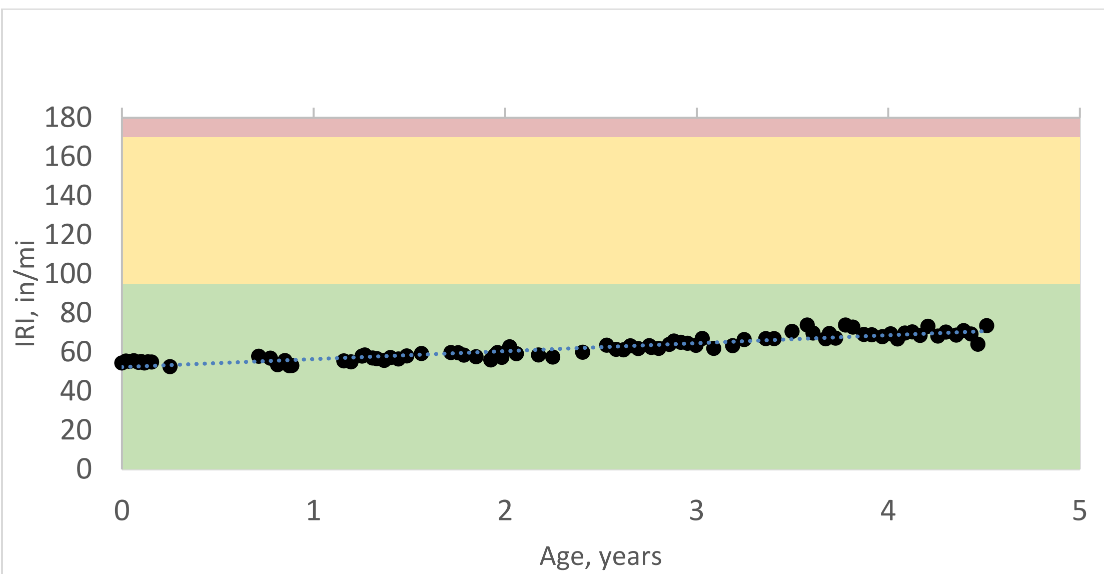

Figure 54. CCPR Emulsified section roughness

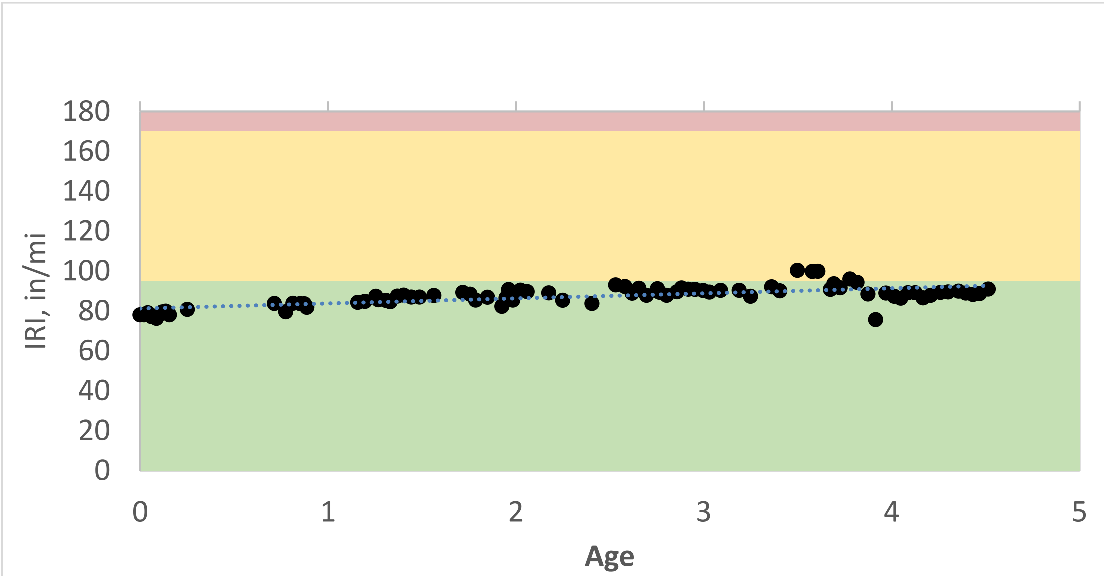

Figure 55. CIR Emulsified section roughness

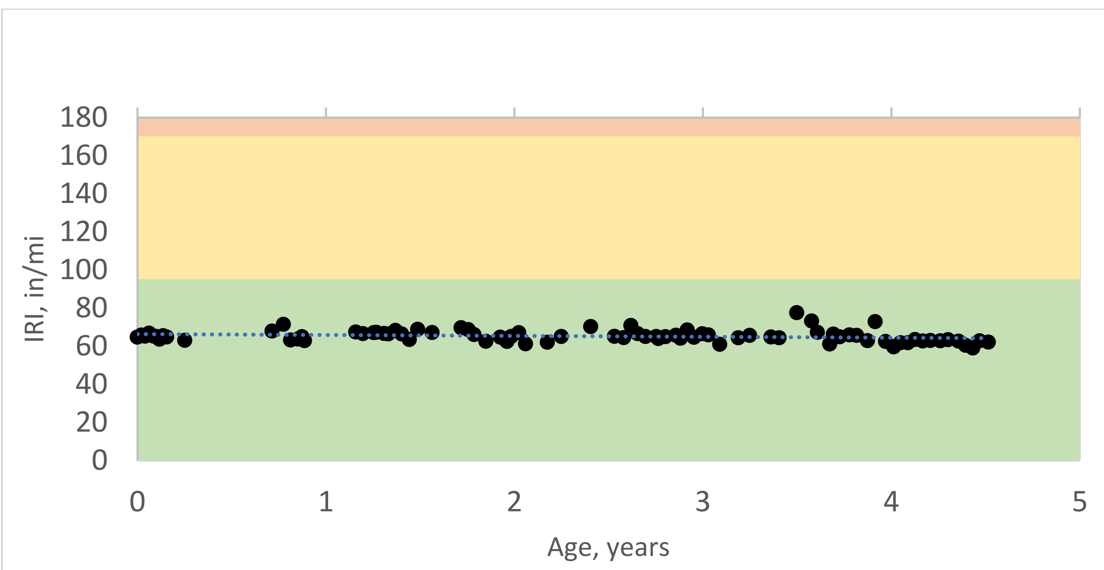

Figure 56. CIR Foamed section roughness

Since each section has a different initial IRI value, it is essential to compare each other by their roughness variation. Figure 57 shows the variation between the post-construction IRI and the last measure in April 2020. Section CCPR-F and CCPR-E have a similar shift with values of 18.9 and 23.2 in./mi respectively, nevertheless, as mentioned before, the CIR emulsion section has a minor roughness decrease. Also, each section was analyzed using a paired t-test, in which the null hypothesis is that the difference of means between post-construction IRI and the last measure equals zero (Table 30).

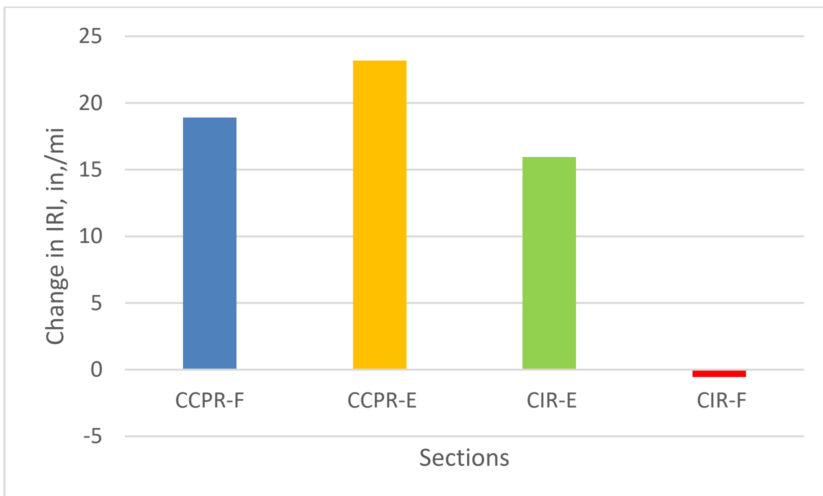

Figure 57. Roughness variation from post-construction to the last date

Table 30. IRI variation Pair T-test

Section	Mean	Std. Dev.	SE Mean	95% CI for μ_difference	T-Value	P-Value
40-CCPR-F	18.74	18.05	3.19	(-25.25, -12.24)	-4.53	0.000
41-CCPR-E	19.13	23.89	4.22	(-27.74, -10.51)	-5.88	0.000
43-CIR-E	12.89	25.04	4.43	(-21.91, -3.86)	-2.91	0.007
44-CIR-F	2.60	32.18	5.69	(-9.01, 14.20)	0.46	0.651

The results in Table 30 show that the null hypothesis in sections CCPR-F, CCPR-E, and CIR-E can be rejected with a confidence interval of 95%, section CCPR-E has the greatest T-value which can be interpreted as the section where the variation between dates is the most relevant. Finally, section CIR-F is the only one that cannot be rejected; then, it can be interpreted as a steady value over time.

4.3.3. Cracking

All the recycled sections performed well over the four and a half years of open traffic in Highway US-280. Most of the measured cracking was very fine hairline cracks that are only visible to the trained eye. Nevertheless, section CCPR-F (40) exhibited rapid growth in the amount of cracking in the last year. This increase of cracking can be related to the high deflections collected from FWD testing in the last year and the decrease of the composite pavement modulus during the study, the most considerable amount of cracking appears in the wheel path. A cracked section allows water to be trapped in the mix, which evolves in loss of cohesion. The last cracking measure in the CCPR-F section is 4.2%, falling close to the fair threshold, which is 5%. Even section 43 has a value slightly higher than 1.0% of cracking; it is too small to be significant for further statistical analysis by the end of this research.

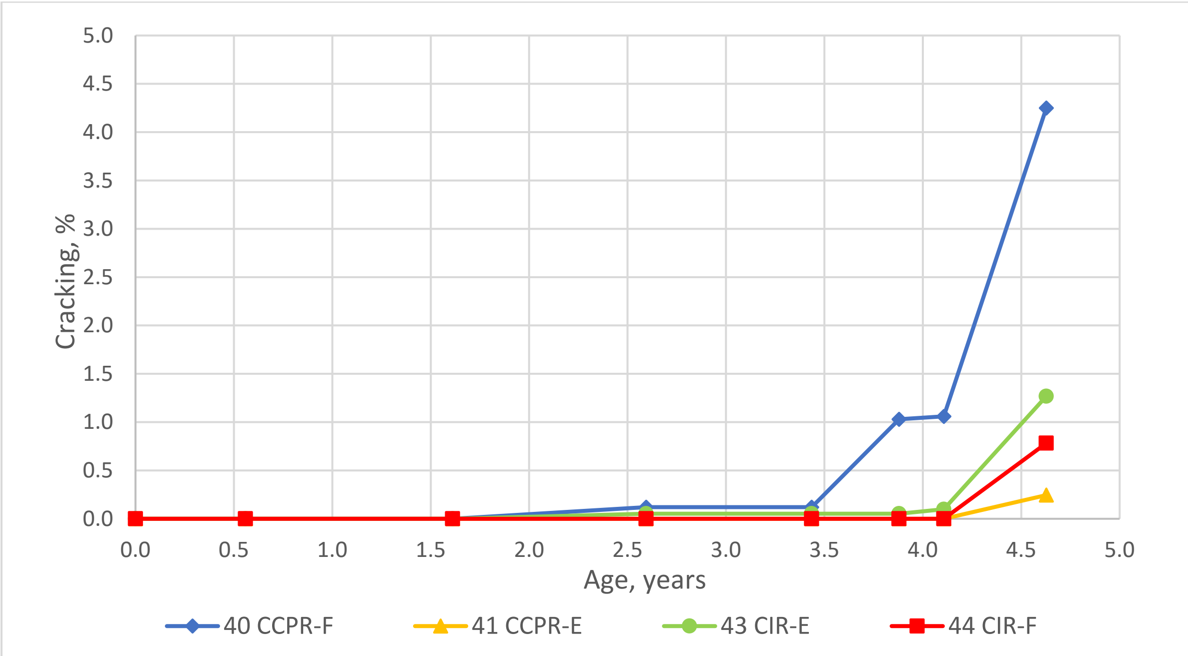

Figure 58 Cracking of the recycled sections.

The overall pavement condition using the MAP-21 rating analyzes each parameter with its threshold mentioned in Table 10. If all the parameters are "good," the section is good. If two parameters or more are considered "poor," then the section is poor. All other combinations result in a "fair" overall condition. A summary of the functional performance parameters of the recycled sections is shown in Tables 31. The overall condition of section CCPR-E reflects a good status of the pavement section; on the other hand, all the other sections fall into "fair" condition. Pictures of the sections after 4.5 years of service are shown in Figures 59 to 62. It can be observed that the cracking exhibited in section CCPR-F (40) is accompanied by pumping of fines, which may be related to the high content of fines in the mix, and the nature of foamed recycled mixes where the particles are not coated entirely.

Table 31. Overall condition of the Recycled Sections

Section	Parameter	Value	Rating	Overall Condition
40 CCPR-F	Cracking, %	4.2	GOOD	FAIR
	Rutting, in	0.27	FAIR	
	IRI, in/mi	82.2	GOOD	
41 CCPR-E	Cracking, %	0.2	GOOD	GOOD
	Rutting, in	0.16	GOOD	
	IRI, in/mi	77.6	GOOD	
43 CIR-E	Cracking, %	1.3	GOOD	FAIR
	Rutting, in	0.21	FAIR	
	IRI, in/mi	94	GOOD	
44 CIR-F	Cracking, %	0.8	GOOD	FAIR
	Rutting, in	0.22	FAIR	
	IRI, in/mi	64.2	GOOD	

Figure 59. Section 40 CCPR-F overview

Figure 60. Section 41 CCPR-E overview

Figure 61. Section 43 CIR-E overview

Figure 62. Section 44 CIR-F overview

4.4. SUMMARY

The sections have demonstrated a good international roughness index (IRI) performance. The rutting resistance shows an increase in rut depth, but the variations are not considered statically meaningful among the sections. Similarly, at the end of this research, the four recycled sections showed a minimum amount of cracking, but cracking resistance started to decrease in CCPR-F (40) by the end of the study. The CCPR-F (40), CCPR-E (41), and CIR-E (43) sections are categorized as fair under the MAP-21 rating.

Pavement modulus was backcalculated in all sections considering all the asphalt layers as composite pavement modulus. The combined AC/RAP lift was layer one, the aggregate base

(Sections 40 and 41) as layer two, and the subgrade as layer three. The CIR-E (43) and CIR-F (44) sections, without aggregate base, showed a strong influence of mid-depth pavement temperature on the modulus, and the thinnest AC section (Section 40) was found least temperature sensitive.

The composite pavement modulus values were normalized to a reference temperature of 68°F, CCPR F and CCPR-E sections have statically decreased their composite pavement modulus over time. However, the CIR-E (43) and CIR-F (44) sections have regular behavior overtime. The layer coefficient for the recycled sections was computed, and the structural coefficients were found to be statically different.

CHAPTER 5: CONCLUSIONS AND RECOMMENDATIONS

Four test sections were built in 2015 on Highway US-280 as part of a larger experiment called the Preservation Group study. This research evaluated the field performance of the cold recycled sections under real traffic conditions. The test sections were identified as sections CCPR-F (40), CCPR-E (41), CIR-E (43), and CIR-F (44). All the sections were comprised of a thin overlay, which was placed on top of the cold recycled material and an old HMA. Only sections 40 and 41 have a granular base below the asphalt concrete layers.

The general objective of the study was to evaluate the structural and functional performance of cold recycled asphalt pavement as a pavement preservation technique. Other objectives were to determine the structural contribution of the recycled technologies and to compare the performances of the two recycling techniques.

5.1. CONCLUSIONS

Based on the observations obtained throughout this study, the following conclusions can be made:

- The composite pavement modulus in the cold recycled sections is sensitive to temperature as HMA. Nevertheless, Section CCPR-F (40) has the lowest R-squared value in the regression analysis between the composite pavement modulus and temperature, which indicates that the temperature does not account for many variations in the composite modulus in the section; this conduct can be explained by the structural composition of the CCPR-F (40) section where the old HMA has the thinnest thickness compared to the other sections. Nevertheless, different research shows that CIR-emulsion is not as sensitive to temperature and loading frequency as HMA according to dynamic modulus testing, and the variation of CIR using foam is due to the residual asphalt content.

- The backcalculated Ep was normalized to 68°F (Ep_{68}). Even though some seasonal variability may affect the results, the variation of backcalculated pavement modulus over time showed no evidence of a structural decline in sections CIR-E (43) and CIR-F (44). In the confidence interval of the regression analysis of those sections, the 95% confidence interval for the slope goes through zero, which indicates that statistically, there is no significant relationship between Ep68 and time. On the other hand, CCPR-F (40) and CCPR-E (41) sections have statically decreased their composite pavement modulus over time, which may indicate deterioration of the pavement structure.
- A general linear model was applied to analyze the effects of the construction method and recycling agent in the structural coefficient of the recycled sections; each factor has two levels. All the factors mentioned have a p-value lower than 0.05; the analysis suggests that at 95% confidence, the method, recycling agent, and the interaction of them have a significant impact in the structural coefficient calculations. The most significant factor in the analysis was the recycling agent. The main interaction plot suggests that the relationship between the construction method and structural coefficient depends on the recycling agent.

- The average layer coefficient computed in the CCPR-E (41) section is 0.66 in. $^{-1}$, which is higher than any values presented in previous studies, this computed value is unexpected because it is greater than the layer coefficient assigned to a new HMA layer. A possible error in the computation of the layer coefficient is using wrong thicknesses when separating the composite pavement modulus into the recycled layer and old HMA layer.
- The three remaining sections were analyzed under an ANOVA test, the null hypothesis of the test is that all the means are equal, at 95% confidence, the hypothesis was rejected with a P-value lower than 0.05. A Tukey test was applied in the model to determine which sections were different. The test concluded that all the sections are significantly different from each other.
- The recommended structural coefficients suggested for structural pavement design were obtained by using the 25th percentile of the distribution for each section. A structural coefficient of 0.35 was selected for the CCPR-E (41) due to the high magnitude calculated in the section, which is larger than the structural coefficient of a new hot mix asphalt layer. The structural layer of the recycled materials ranged between 0.23 and 0.35.
- The functional performance of the four test sections was divided into three parameters. The mean rut depth of the four test sections was well below the poor criteria of 0.4 inches, established by the MAP-21 rating system. The average means have increased from post-construction to the last date of data collection, but there are

no statistical differences among the sections. However, section CCPR-E (41) is still in the "good" condition category being also the section with the highest structural coefficient, while all the other three sections are in "fair" condition.

- The roughness parameter for all four sections shows a normal increase. In all cases, the measured IRI was maintained below the fair threshold of 95 in/mile established by the MAP-21 rating system. This parameter was analyzed using a paired t-test for each section to evaluate a change between the post-construction data and the last one. Sections 40, 41, and 43 fail the test with a confidence interval of 95%, concluding there is a change in the IRI values overtime.
- The measured cracking was very fine hairline cracks. Nevertheless, section CCPR-F (40) exhibited rapid growth in the amount of cracking in the last year. This increase of cracking can be related to the high deflections collected from FWD testing in the last year and the decrease of the composite pavement modulus during the study. A cracked section allows water to be trapped in the mix, which evolves in loss of cohesion. This behavior is consistent with field performance in the section, which has the lowest rutting resistance in all sections.

5.2. RECOMMENDATIONS

The following recommendations could be made for future research:

- The PG study should continue traffic on the recycled sections and monitor the performance of the sections to obtain a long-term assessment.

- By the end of the study in Highway US-280, forensic analysis of the test sections that include performance tests would allow understanding the true failure mechanisms.
- A climate can be assessed in the cold recycled analysis. In the summer of 2019, similar cold recycled sections have been constructed in the Northern part of the PG study located in Minnesota; this will allow for incorporating climate as part of the analysis.
- Similar sections could be placed in the NCAT test track; this would address the performance of pavement preservation techniques under a high amount of heavy daily traffic. The sections would be instrumented with asphalt strain gauges, temperature probe, and earth pressure cell. They would allow continuous analysis of the pavement response under traffic and climate.

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