100% RECYCLED MIXTURES USING COLD IN-PLACE RECYCLING (CIR), HOT IN-PLACE RECYCLING (HIR) AND COLD CENTRAL PLANT RECYCLING (CCPR)

Research Final Report from University of Tennessee, Knoxville | Baoshan Huang, Yuetan Ma, Pawel Polaczyk | May 30, 2022

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16. Abstract

The increasing costs of asphalt binder and aggregates have put much pressure on the highway maintenance budgets. Pavement managers are seeking alternative cost-effective approaches to rehabilitate the roads. In-place recycling techniques, including hot in-place recycling (HIR) and cold in-place recycling (CIR), encouraged consuming 100% RAP from the existing pavement, which allowed the utilization of suitable recycling agents to rejuvenate the aged asphalt. However, the cost-effectiveness and blending mechanisms of using these techniques remained concerns. In this research project, the UT research teams aimed to conduct a comprehensive evaluation of using the in-place recycling techniques, investigate the blending mechanisms and recycling efficiency of using HIR techniques, and provide useful recommendations to TDOT for further constructions. Results showed that HIR and CIR techniques were suitable for low-volume roads, which could save 20.1% and 15.9% cost in the whole life cycle, respectively. Pavement with in-place recycling techniques tended to encounter severe cracking issues. For CIR, an adequate curing period and appropriate cement usages could improve the cracking resistance, moisture resistance, and stiffness of the CIR mixes. For HIR, the mobilized RAP content ranged from 20% to 30%, contributing to a change of effective binder quality and an increase in the effective asphalt content. The binder quality significantly influenced the stiffness of the HIR mixes, while the effective asphalt content determined the cracking resistance of HIR mixes. An appropriate heating temperature and adequate of using recycling agent could improve the cracking resistance of the HIR mixes.

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Executive Summary

The increasing costs of asphalt binder and aggregates have put much pressure on the highway maintenance budgets. Recycling has played a significant role in the pavement rehabilitation of state highway agencies. According to the asphalt pavement survey on recycled materials conducted by National Asphalt Pavement Association (NAPA), the total estimated tons of recycled asphalt pavement (RAP) used in the asphalt mixtures was 76.2 million in 2017, and the total estimated amount of stockpiled RAP at the end of the 2017 construction season was about 102.1 million tons. The survey also reported that about 0.89 million tons of the RAP were created in Tennessee in 2017 (Williams et al., 2018). Pavement managers are seeking alternative cost-effective approaches to rehabilitate the roads. Currently, there exist several asphalt recycling techniques consisting of two basic approaches: (a) inclusion of the milled reclaimed asphalt pavement (RAP) to replace a proportion of aggregates in hot mix asphalt (HMA) in asphalt plant, and (b) the in-place recycling, including cold in-place recycling (CIR) and hot in-place recycling (HIR). In-place recycling technique encouraged consuming 100% RAP from the existing pavement, which was allowed by utilizing suitable recycling agents to rejuvenate the aged asphalt.

However, some concerns remained for the utilization of in-place recycling techniques in pavement rehabilitation: 1. What are the performance differences between the asphalt mixtures after in-place recycling and conventional HMA mixtures? 2. Is it cost-effective to apply the in-place recycling techniques to pavement rehabilitation compared to the conventional HMA surface milling & filling? 3. How can the performance of the asphalt mixtures after in-place recycling be improved? 4. How much RAP binder can be available for coating the aggregates during HIR? 5 What factors determine the performance of HIR mixes?

In this research project, the University of Tennessee research team aimed to conduct a comprehensive evaluation of using the in-place recycling techniques, investigate the blending mechanisms and recycling efficiency of using HIR techniques, and provide valuable recommendations to the Tennessee Department of Transportation (TDOT) for future constructions. To achieve these objectives, the research team participated in three HIR and two CIR field projects. The loose HIR and CIR materials were collected and compacted for performance testing. The HMA plant mixes from the same region were also collected for comparison. ME software was adopted to predict the pavement life after rehabilitation with in-place recycling techniques and traditional HMA milling & filling. Life cycle cost analysis (LCCA) was applied to evaluate the cost-effectiveness of the pavement with different rehabilitation techniques in the whole life cycle. Furthermore, the research team explored the blending mechanism of HIR mixes with 100% RAP and developed a method to quantify the mobilized RAP content in HIR mixes to evaluate the blending efficiency of HIR mixes. The influence of mobilized RAP binder content on the effective binder quality and mixture performances was also investigated in this project.

Key Findings

Based on the experimental and simulation results, the key findings from this research project were summarized as follows:

- Compared to HMA (D-mix), HIR mixes showed acceptable rutting and moisture resistance. Cracking resistance is the main issue that HIR mixes would encounter. The pavement after HIR surface treatment would yield a larger value of roughness index and encounter severe fatigue cracking as well as low-temperature cracking issues. LCCA results reflected that HIR surface rehabilitation was cost-effective only for low-volume roads, which could save 52.9% initial cost and 20.1% life cycle cost than conventional HMA milling & filling.
- Compared to the HMA (BM2-mix) for the binder layer, CIR mixes tended to encounter rutting and cracking problems. A sufficient curing period would improve the CIR mixes' stiffness, cracking resistance, and moisture resistance. From pavement prediction results, the pavement after CIR rehabilitation would increase the IRI value and have severe cracking issues. LCCA results showed that the CIR technique was also cost-effective for the low volume road, which could save 43.9% initial cost and 15.9% life cycle cost than conventional HMA milling & filling. The high-level traffic conditions would damage the CIR rehabilitated pavement more quickly, and CIR technique was not cost-effective compared to the conventional HMA milling & filling.
- The mobilized RAP content represented the recycling efficiency of the HIR mixes. Based on this research, the mobilized RAP content for HIR mixes located at a range of 20% to 30%. An appropriate increase of temperature and additive dosages could improve the RAP binder mobilization and the recycling efficiency of the HIR mixes.
- The effective asphalt content was considered a significant factor that dominated the cracking behavior of the HIR mixes. Even though the higher mobilized RAP binder content theoretically resulted in more brittle binder blends and cracking potential, the mobilized RAP content would also increase the effective asphalt content, which improved the aggregates coating and the cracking resistance of the HIR mixes.

Key Recommendations

Based on the conclusions, the potential recommendations were summarized as follows:

- Increase the mixing and compaction temperature to 130° C (current is 110° C) for HIR construction since higher temperature would improve the binder mobilization and the cracking resistance of the HIR mixes.
- Increase the dosage of asphalt emulsion and primarily focus on the cracking resistance during HIR mix design. Based on the experimental results, HIR mixes would mainly encounter cracking issues. Hence, more dosages of asphalt emulsion could be added to improve the cracking resistance of HIR mixes. Additionally, a performance-based mix design (e.g., balance mix design) is recommended to focus more on the cracking resistance of the HIR mixes.
- If possible, increase the mix temperature to 150 ~160° C and use virgin asphalt during HIR construction, which could significantly improve RAP binder mobilization and the cracking resistance of HIR mixes.
- Apply the CIR technique only in a low-volume road and ensure the curing period after paving. For the CIR mix design, increase cement usage to improve the strength of the base layer.

- Conduct pavement monitoring for HIR and CIR sections during the service period, which could provide valid field data for the pavement performance evaluation for in-place recycling techniques.

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Glossary of Key Terms and Acronyms

RAP	Reclaimed asphalt pavement
HMA	Hot mix asphalt
HIR	Hot in-place recycling
CIR	Cold in-place recycling
LCCA	Life cycle cost analysis
NAPA	National asphalt pavement association
CSS	Cationic slow setting
DSR	Dynamic shear rheometer
BBR	Bending beam rheometer
SEM	Scanning electron microscopy
SGC	Superpave gyratory compactor
GPC	Gel permeation chromatography
LMS	Large molecular size
FTIR	Fourier transform infrared spectroscopy
FN	Flow number
AMPT	Asphalt mixture performance tester
IDT	Indirect tension
APA	Asphalt pavement analyzer
AADTT	Annual daily truck traffic
IRI	International roughness index
$DCSE_f$	Dissipated creep strain energy
FE	Fracture energy
EE	Elastic energy
TSR	Tensile strength ratio

Chapter 1 Introduction

1.1 Problem statement

With the increasing cost of petroleum products, the construction costs, and the limitation of the federal budgets for pavement preservation, recycling has played a significant role in pavement rehabilitation of state highway agencies (SHAs) (Ma and Huang, 2020; O'Sullivan, 2009). According to the asphalt pavement survey on recycled materials conducted by National Asphalt Pavement Association (NAPA), the total estimated tons of recycled asphalt pavement (RAP) used in the asphalt mixtures was 76.2 million in 2017, and the total estimated amount of stockpiled RAP at the end of the 2017 construction season was about 102.1 million tons. The survey also reported that about 0.89 million tons of the RAP were created in Tennessee in 2017 (Williams et al., 2018). Thus, there is an immediate need to look into some cost-effective methods and alternative techniques to rehabilitate distressed pavements. The in-place recycling techniques may prove to be an economical and sustainable pavement rehabilitation method to use recycled materials. The incorporation of suitable rejuvenators or emulsions to the aged binders can save the binder costs and the related environmental impacts.

Previous Tennessee Department of Transportation (TDOT)-University of Tennessee (UT) joint research has established that current TDOT limits on recycled material percentages are already near their maximums. Technologies are available that utilize 100% of asphalt millings into mixtures at ambient temperatures, known as cold in-place recycling (CIR) and hot in-place recycling (HIR). HIR and CIR provide an alternative to using excessive RAP stockpiles in a beneficial application. However, these technologies are new to TDOT; TDOT has little to no experience with the mixed design and performance of these technologies. For TDOT to successfully adopt and implement these technologies, there is a need to look into the materials, mix design, construction, and performance of these 100% recycled mixtures.

1.2 Research scope

The scope of the research work includes:

- To complete a synthesis of literature review on CIR, HIR, and Department of Transportation (DOT) survey on these 100% recycling technologies in the US, especially in the Southeastern region;
- To perform field testing and monitor pavement performance of 100% asphalt recycling projects;
- To conduct a series of laboratory tests on asphalt millings, raw materials, and 100% recycled asphalt mixtures to test their properties and performance; and
- To conduct a cost-effectiveness analysis.

To achieve the aforementioned scope, the UT research team worked with TDOT Materials and Tests Division engineers to identify possible projects of 100% recycled asphalt mixtures. After that, the research team visited the job sites to collect the raw materials for further laboratory test evaluations.

For HIR mixes, the research team collected the materials before and after HIR procedures and brought them to the UT lab. For CIR mixes, the research team used the UT mobile laboratory to make samples onsite since asphalt emulsion of foamed asphalt would break during the transportation. Performance tests including Asphalt mixture performance tester (AMPT) tests, Superpave indirect tensile (IDT) tests, Asphalt pavement analyzer (APA) tests, moisture susceptibility tests were adopted to evaluate the rutting, cracking, and moisture susceptibility of the HIR and CIR mixes. The hot mix asphalt (HMA) plant mixes, D-mix, and BM2 mix, with similar gradation as HIR and CIR mixes in the same region, were also collected for comparison.

ME software was used to conduct the pavement life prediction for the pavement rehabilitation with HIR, CIR techniques, and the conventional milling & filling techniques. The life cycle cost analysis (LCCA) was conducted to evaluate the cost-effectiveness of the HIR and CIR techniques.

Furthermore, the research team explored the blending mechanism of HIR mixes with 100% RAP and developed a method to quantify the mobilized RAP content in HIR mixes to evaluate the blending efficiency of HIR mixes. The influence of mobilized RAP content on the effective binder quality and mixture performances was also investigated in this project. Such areas were not presented in any other research projects.

Chapter 2 Literature Review

2.1 HIR technique

The HIR technique is one of the rehabilitation techniques for recycled asphalt pavements with the application of in situ heat during the recycling process. It consists of heating and remixing the recycled materials with the addition of the emulsion, rejuvenator, or additives to meet the required performance of the mixtures. The HIR process is suitable for the pavement with minimal distresses but no structural distresses. Different types of HIR processes are applied for pavement maintenance based on the severity of distress. In general, surface recycling (heater scarification), repaving, and remixing are the basic HIR process organized by the Asphalt Recycling and Reclaiming Association (ARRA) (Ma et al., 2020a; Recycling, 1992). The procedures of these steps are shown in **Figure 2-1**. Surface recycling is the process that the existing road surface is heated and scarified, followed by applying the rejuvenating agents and compaction technique, which is suitable for pavements subjected to minor cracks within 25-50 mm in depth. Repaving is often used when surface recycling fails to restore the pavement condition, requiring an asphalt concrete (AC) overlay of 25-50 mm. Remixing is usually conducted when significant modifications of the existing mix are required due to the change in aggregate gradation, aggregate abrasion, binder content, binder rheology, and mixture volumetric (Stroup-Gardiner, 2012).

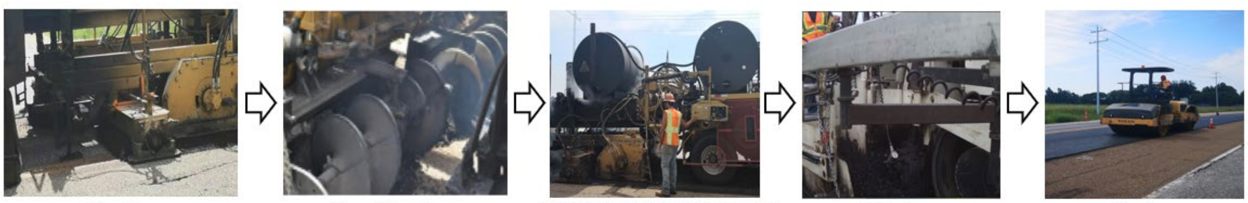


Figure 2-1. Hot in-place recycling procedures.

A series of studies have explored the concept of incorporating RAP (up to 30%) during HMA production in asphalt plants (Huang et al., 2005b; Huang et al., 2011; Huang et al., 2004). The brittleness of the RAP binder is considered a significant factor that would attenuate the cracking resistance of asphalt mixtures, hence the pavement's durability (Ma et al., 2020b; Zhao et al., 2012). The aforementioned problems might exaggerate during HIR procedures since 100% RAP is commonly used for rehabilitation. Zhong et al. investigated the binder aging level during HIR of asphalt pavement. They concluded that extra aging caused by HIR was not significant and the aging level of the bottom part is smaller than the surface part (Zhong et al., 2021). To improve the properties of the HIR mix, recycling agents were adopted to recover the chemical and mechanical properties of the RAP binder in HIR (Hafeez et al., 2014; Li et al., 2014). Ali and Bonaquist suggested that the addition of recycling agents could restore the recycled binder PG grade close to the original condition (Hesham and Bonaquist, 2012). A new additive, named Styrene-butadiene rubber latex, was also introduced to enhance the moisture susceptibility and low-temperature cracking resistance of the HIR mix (Li et al., 2016). Preheating conditions, including temperatures and heating times, were critical to the performance of HIR mixes (Liu et al., 2019). Ali and Grzybowski conducted a case study of LCCA between HIR and conventional pavement rehabilitation. Projections reflected that HIR could result in an over 40% reduction of the initial cost (Ali and Grzybowski, 2012). Cao et al. assessed the cost and environmental concerns between HIR and the conventional milling & filling techniques with assumed service life, indicating that HIR could save 5% cost and reduce 16% of the overall environmental impacts (Cao et al., 2019).

2.2 Blending efficiency of asphalt mixtures

A major concern of using 100% RAP materials in asphalt mixtures is how much RAP binder can be activated and mobilized to coat the aggregates since RAP binder has become stiff and brittle which is difficult to be activated. In terms of the production of traditional HMA, a certain percent of RAP (up to 30%) is usually incorporated into the asphalt mixtures (Huang et al., 2005a). Theoretically, it is expected that the virgin binder can diffuse into the RAP binder to achieve a homogenous binder blend, which wraps over both virgin and RAP aggregates (Zhao et al., 2015). However, In the real blending scenarios of HMA, as shown in **Figure 2-2**, the "black rock" lies in the innermost layer, while part of the viscous RAP is mobilized with the virgin binder, which is used to coat both virgin and RAP aggregates (Zhao et al., 2016). Therefore, the effective mobilized RAP consists of the RAP binder activated by high temperatures and part of the viscous RAP that is mobilized by the virgin asphalt or additives.

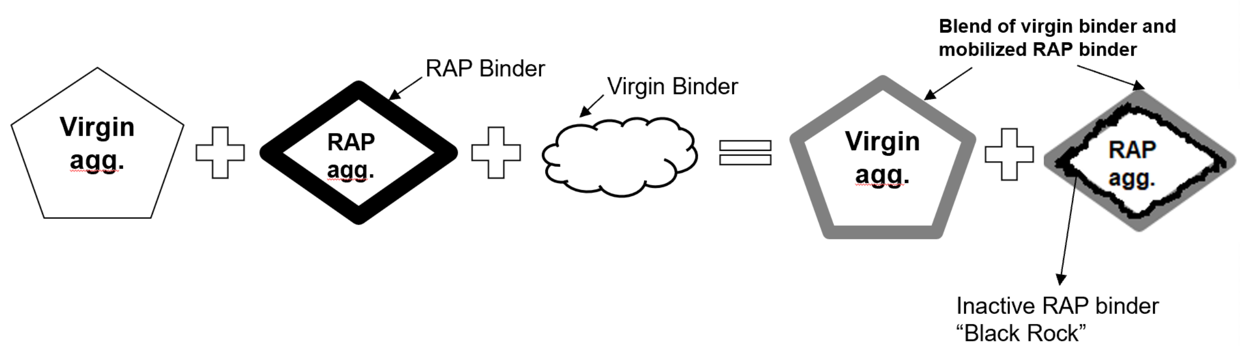


Figure 2-2. Real blending between virgin and aged binder in HMA.

In terms of the RAP binder during HIR procedures, some of the RAP binders are scraped off by the auger system during heat scarification. With the addition of the recycling agents such as asphalt emulsion and rejuvenator, part of the viscous RAP is rejuvenated and mobilized to glue the aggregates. In this scenario, the effective RAP binder includes a proportion of the scraped RAP binder, the mobilized RAP binder through heating, and part of the viscous RAP mobilized by additives (Ma et al., 2021a). Compared to the blending scenario of HMA, HIR techniques severely restrict mobilization of the RAP binder since 100% RAP is promoted to be utilized in the field. Hence, to improve the efficiency of asphalt mixtures during HIR construction, approaches need to be developed to characterize and quantify the mobilized RAP content.

2.3 CIR technique

CIR is usually performed at a depth of 2 to 6 inches of the asphalt pavement using a train of equipment such as a tanker, a cold recycler, a paver, and rollers. CIR consists of pulverizing the asphalt layer, adding the stabilizing agents, mixing of the stabilizer and pulverized materials, and compacting the mixtures. Figure 3 presents the basic procedures of CIR in the field project. The recycling agents used for CIR are similar to those for full-depth reclamation (FDR), including lime, fly ash, cement, asphalt emulsion, and foamed asphalt. The active fillers are often incorporated with the stabilizers to improve the water-resistance and to achieve a relatively high early strength (Manual). A thin HMA layer is placed over the recycled layer once the recycled pavement is compacted. The advantage of CIR technology is to reduce the number of materials transported to the job site and minimize the environmental effect, compared with other techniques. Furthermore, it can completely repair different pavement distresses such as rutting, reflection cracks, potholes, and prolong the service life of the asphalt pavements.

2.3.1 CIR mix design

In the beginning, a number of studies were mainly focused on the mix design of the cold mixtures. Foamed and emulsified asphalt were commonly used to stabilize the cold mix. Different curing temperatures and curing conditions were also evaluated during the mix design. Kim et al. finally proposed a developed mix design method, which was widely used in cold recycling (Kim and Lee, 2011; Kim et al., 2007). The sketches of the mix design include the collection of RAP materials from a job site, evaluation of RAP characteristics, separation of RAP materials (different sizes), selection of RAP gradations, asphalt binder, determinations of moisture content, mix design (foamed asphalt content or emulsified asphalt content, curing conditions), volumetric characteristics, IDT test (Dry and wet condition), final optimum mix.

Because of the complex composition of the final cold recycled mixture, mainly consisting of RAP materials, aggregate, asphalt types, or cementitious stabilization agents, the performance of rehabilitated pavement highly depends on the properties of its components. Therefore, it is necessary to evaluate the raw materials of the CR mixture to be used during construction.

2.3.2 RAP materials

The properties of RAP materials should be evaluated during the mix design, including the moisture content, gradation, asphalt binder content. Since the moisture content has a significant effect on the final performance of the pavement, the moisture of the RAP in the field should be considered to calculate the total amount of water in the mix design. Since the RAP binder can affect the coating of the asphalt mixtures, it is necessary to identify whether the RAP binder is active or not through the penetration test, viscosity, and softening point. Also, the gradation of the RAP should be tested to evaluate if it meets the requirements of the designed gradation. Further adjustment of the gradation combining the virgin aggregates should be applied before the in-place paving.

2.3.3 Asphalt types

Emulsified and foamed asphalt are commonly used in cold recycling. In general, there exist three types of commonly used emulsions: cationic slow setting (CSS-1), HFMS-2s (High-flow medium-setting with solvent), and high-float medium-setting emulsion modified with a polymer (HFMS-2P). Many studies are focused on the influence of different types of asphalt emulsion on the binder properties and the performance of the mixtures. Pouliot et al. (2003) investigated the influence of the cationic slow setting (CSS-1) and anionic slow setting (SS-1) emulsions with cement on the mechanical properties of the mortals. Results showed that the CSS-1 of the mortars had higher elastic modulus and strengths than SS-1 (Pouliot et al., 2003). Marasteanu et al. (2006) studied the rheological properties of the asphalt emulsion residues of different types of the asphalt emulsions, including cationic rapid setting emulsion modified with a polymer (CRS-2P), a proprietary engineered emulsion (EE), and high-float medium-setting emulsion modified with a polymer (HFMS-2P) through dynamic shear rheometer (DSR), Bending beam rheometer

(BBR), and direct tension. Results revealed that CSS-1 had the most rapid stress accumulation, followed by CRS-2P and EE, with the highest thermal stress between -10 and 0 ° C. Kim and Lee (2012) conducted a study about the influence of the different types of asphalt emulsion on the dynamic modulus, flow number, and flow time of the asphalt mixtures.

Foamed asphalt is a physical process in which a predetermined amount of cold water is injected into hot asphalt in a series of individual expansion chambers in a base recycling machine. The expanded asphalt will have a high surface area, which is beneficial to coat the aggregates. Previous studies were mostly about the foaming temperatures and the performance characterization of the asphalt mixtures. Kim et al. investigated the foaming temperature and RAP temperature effect on the performance of cold mix during the mix design. It was recommended that the CIR-foam mix design should be conducted using the RAP materials heated to a relatively high temperature (Kim and Lee, 2011). Ministry of Transportation Ontario constructed a 7-km CIR-emulsion road and a first 5-km CIR-foamed trial section to evaluate the differences between the emulsified asphalt mixtures and foamed asphalt mixtures (Chan et al., 2009). The performance data collected by the Falling Weight Deflectometer and Automatic Road Analyzer of those two trail sections indicated that the performance of BSM-foam the BSM-emulsion are similar.

2.3.4 Cementitious materials

The cementitious materials used for cold recycling consist of Portland cement, lime, fly ash, ground blast furnace slag, etc. The primary objective of using cementitious materials is to increase the comprehensive strength of the mixtures.

Cement is the most commonly used in cold recycling. A number of studies have been conducted to evaluate cement usage, including the optimum cement content during the mix design (Gao et al., 2014) and Scanning Electron Microscopy (SEM) analysis of the strength development (Lin et al., 2017). Loizos et al. (2009) found that using cement in the field project could solve the cracking problem (Papavasiliou and Loizos, 2009). Curtis et al. (2007) concluded that the utilization of cement could accelerate the breaking time of the emulsion and reduce the construction time in the field project (Berthelot et al., 2007).

Lime is preferred to use for stabilizing with the asphalt, which can improve the strength of the mixtures. Working with clay, incorporating lime can reduce the plasticity of the mixtures and reduce the swelling of the clay. Niazi et al. (2007) compared the effect of cement and lime on the emulsified asphalt mixtures. It was revealed that if hydrated lime was employed in the mixtures, the strength improvement was better (Niazi and Jalili, 2009). Kansas Department of Transportation conducted a field project that compared lime and class C fly ash in the two traffic lines. The test sections reflected lime could replace the class C fly ash and significantly enhance tensile strength and moisture resistance (Cross and Du, 1998). However, the slurry construction for lime was more complicated.

Fly ash is a coal combustion product that can be used in CIR projects to enhance the bearing capacity. Li et al. (2013) evaluated the performance of fly ash stabilized cold mixtures and conducted the life cycle cost analysis. Results showed that incorporating class C fly ash could significantly reduce the cost and energy consumption (Li, Xiaojun et al., 2013). Li et al. (2012) also explored the performances of the cement and fly ash stabilized mixtures. It was reflected that by using the fly ash, the optimum moisture content and maximum dry density of the mixtures were significantly influenced (Li, Xiangguo et al., 2013).

2.4 Research gaps

As summarized from the literature reviews, there exist limited studies on the following areas:

1. A comprehensive performance evaluation between HIR & HMA and CIR & HMA.
2. A life cycle cost analysis of HIR and CIR to evaluate whether it is cost-effective to apply HIR or CIR for pavement rehabilitation.
3. The blending mechanism or binder mobilization during HIR techniques.
4. A method to quantify the mobilized RAP content of HIR mixes.
5. Approaches to improve the performance of HIR and CIR mixes.

Chapter 3 Methodology

3.1 Field experiments

3.1.1 Infrared camera

Temperature is of great significance during HIR techniques. The infrared camera was adopted to evaluate the temperature distribution of all the HIR procedures. Then the thermal images were imported into the FLIR software for image analysis. The detailed figures and tables were summarized in **Figure A-1**.

3.1.2 UT mobile trailer

For CIR projects, the UT mobile trailer containing two Superpave Gyratory Compactor (SGC) compactors was transported into the project, and the CIR specimens were made directly in the field to avoid the breakage of the asphalt emulsion or foamed asphalt. **Figure A-2** shows the UT mobile trailer and specimen compaction in the field project.

3.2 Asphalt binder tests

3.2.1 Dynamic shear rheometer (DSR) test

The DSR frequency sweep test was performed on all the binder blends to explore the rheological properties at three different temperatures of 10 °C, 25°C, and 40°C and angular frequency of 0.1-100 rad/s for two replicates. The master curves were created at a reference temperature of 25 °C. The rheological properties are characterized by complex shear modulus (G*) and phase angle (δ) at a different reduced frequency. G* is considered as the sample's total resistance to deformation when repeatedly sheared. A larger complex modulus usually has better resistance to flow deformation. The phase angle, defined as the time lag between the applied shear stress and the resulting shear strain, reflects the viscous response of the asphalt (Ma et al., 2021b; Ma et al., 2021c).

The time-temperature superposition principle is used to construct the master curves of the binder blends. Asphalt is considered a viscoelastic material and is highly related to temperature change. The shift factor (α_T) is defined as the ratio between the reduced frequency at the reference temperature (ω_r) and the frequency at the desired temperature (ω), which can be expressed as:

$$\alpha_T = \frac{\omega_r}{\omega} \tag{1}$$

A second-order polynomial relation mode is used to expresses $\log(\alpha_T)$ in terms of temperature (Kutay and Jamrah, 2013):

$$log(\alpha_T) = a_1 T^2 + a_2 T + a_3 \tag{2}$$

where a_1, a_2 and a_3 are regression coefficients.

A sigmoidal model is utilized to construct the master curves:

$$\log(|G^*|) = \delta + \frac{\alpha}{1+e^{\beta+\gamma(\log f)}} \tag{3}$$

where G^*= dynamic modulus (Pa); f = reduced frequency (Hz); δ = the minimum value of G^*; δ+α = the maximum value of G^*; β and γ = constants.

A master curve is plotted at a reference temperature as the shift factors are calculated.

3.2.2 Gel permeation chromatography (GPC)

Gel permeation chromatography (GPC) was conducted to detect the change of molecular size distribution of the asphalt. The solution for GPC testing was prepared at a concentration of 1 mg of asphalt per 1mL of tetrahydrofuran (THF), then injected into a 2-mL vial. The vial was inserted in an automatic sample injector, and the concentration of the components was recorded in the eluent during the testing. The chromatogram, which shows the relationship between the elution time and the refractive index, was obtained from GPC testing.

GPC method was commonly used to evaluate the blending efficiency of the asphalt mixtures because it can differentiate the aged binder from the virgin binder due to a higher portion of large molecule in the aged binder (Bowers et al., 2014; Ding et al., 2016a, 2018; Ding et al., 2016b). A large molecular size (LMS) fraction, a medium molecular size fraction (MMS), and a small molecular size fraction (SMS) are divided by the molecular weight distribution of asphalt. Since the elution times in the GPC chromatogram can be converted to molecular weights using the calibration curve, a reasonable method to calculate the LMS was developed based on the molecular weights (Daly et al., 2013). Daly et al. addressed the binder component fractions, including the maltenes (molecular weight less than 3000), asphaltenes (molecular weight from 3000 to 19000), and polymers (molecular weight greater than 19000) (Daly et al., 2013). The peak of LMS (%) difference was always found around 3000 Dalton and was considered as the large molecule threshold. The LMS fraction was selected for the components with molecular weights greater or equal to 3000 Dalton, while the SMS fraction consisted of the proportions of less than 1000 Dalton. The proportion of fraction was calculated by the ratios of the fraction area to the total area of the chromatogram. Correspondingly, the LMS (%) can be calculated using equation (4). Two replicate tests were conducted for each sample.

$$\mathrm{LMS}\ (\%) = \frac{Curve\ area\ where\ molecular\ weights\ larger\ than\ Large\ molecular\ threashold}{Total\ area} \quad (4)$$

3.2.3 Fourier transform infrared spectroscopy (FTIR)

FTIR provides a convenient approach to characterize the blending efficiency of the asphalt binders at molecular levels (Zhao et al., 2015). The absorbed radiation will be converted into vibration and rotational energy by the molecules in asphalt samples. A receiver can detect the signals and form a spectrum afterward, revealing a certain molecular fingerprint of each functional group in asphalt. In asphalt binder chemistry, the carbonyl bond (C=O) exhibits at a wavelength around 1700 $cm^{-1,}$ while the sulfoxide bond (S=O) exhibits a wavelength around 1000 cm^{-1} in the asphalt spectrum. Both chemical bonds could gauge the level of asphalt oxidation. Studies showed that the aged asphalt is prone to have higher C=O and S=O compared with the virgin binder (Hou et al., 2018; Ma et al., 2022; Ma et al., 2021d). The blended binder virgin and aged binder would fall in the middle of the two regions. The degree of blending can be calculated using the peak areas (Sreeram et al., 2018).

Figure A-3 illustrates the FTIR spectrum of the virgin emulsion and RAP binder used in this study. Studies concluded that the carbonyl band had been better correlated with the level of long-term aging. The virgin and aged binder are more explicit to evaluate the aging level (Bowers et al., 2014; Sreeram et al., 2018). The two-point method is used to calculate the degree of blending using the peak areas. As shown in **Figure A-3**, the carbonyl area is firstly selected in the spectrum curves. The baseline is constructed by connecting the lower limit (1672.567 cm-1) and the upper limit (1729,537 cm-1) of the area. It can be seen that the virgin binder presents little C=O bonds, whereas the RAP binder undergoes a significant change of C=O due to the long service period on the pavement (Dony et al., 2016). However, the reference area with a lower limit of 2760.417 cm-1 and a higher limit of 3007.286 cm-1 remain stable in both virgin and RAP binder. Therefore, the degree of blending can be calculated based on the ratio of the carbonyl area to the reference area, which is measured using the OMNIC software.

The degree of blending of HMA refers to the ratio of the mobilized RAP binder in the mixtures (α) to the total RAP binder in the mixtures (β) (Coffey et al., 2013). In this study, the degree of blending is noted as the effective mobilized RAP content, which can be calculated according to equation (6).

$$Effective\ mobilized\ RAP\ content = \frac{\alpha}{\beta} \times 100\% \tag{5}$$

where β is relatively easy to determine as the mass of total RAP binder in the mixtures. By giving the carbonyl index determined by carbonyl area and reference area, α can be calculated as the following equations:

$$\alpha = \frac{CI_{Mix} - CI_{Virgin}}{CI_{RAP} - CI_{Virgin}} \tag{6}$$

where,

α = the proportion of mobilized RAP binder in the mixtures

$$CI = \frac{Carbonyl\ Area}{Reference\ Area} \tag{7}$$

CI_{Mix}= Carbonyl index of the binder extracted from virgin coarse aggregates

CI_{Virgin} = Carbonyl index of the asphalt emulsion

CI_{RAP} = Carbonyl index of the RAP binder

3.2.4 Surface free energy method

Surface free energy is the magnitude of work required to create a unit area of a material in a vacuum. In the asphalt-aggregate system, the cohesion and adhesion strengths are determined by the surface energy at the asphalt-aggregate interface. Based on the Good-van Oss-Chaudhury theory proposed by Van Oss et al., the total surface energy can be expressed in the form of a Lewis acidic component (Γ^{+}), a Lewis basic component (Γ^{-}), and Lifshitz-van der Walls components (Γ^{LW}) (Van Oss et al., 1988). The total surface free energy (Γ^{Total}) can be expressed in terms of a Lifshitz-van der Walls and an acid-base (Γ^{AB}) component using the following equations:

$$\Gamma^{Total} = \Gamma^{LW} + \Gamma^{AB} \tag{9}$$

where, $\Gamma^{AB} = 2\sqrt{\Gamma^{+}\Gamma^{-}}$ (10)

The wetting and spreading of a liquid over a surface, determined by the contact angle (θ), proved to be directly related to the surface free energy. The sessile drop method was adopted to measure the contact angle of the effective binder blends (Howson et al., 2009; Little and Bhasin, 2006). The prepared binder blends were firstly reheated and poured over the glass slides. Then the samples were dried until a homogenous thin asphalt layer was achieved. A drop of approximately 5 μL probe liquid was dispensed at room temperature over the asphalt sample. Finally, a digital image was captured, and the contact angle was measured using the image processing software. The average value of the three replicates was used for surface energy calculation. The suitable probe liquids should have high surface free energy values, be immiscible with asphalt binder and have a broader range of surface energy components. Distilled water, ethylene glycol, and formamide were selected for contact angle measurement. The surface energy components of the three probe liquids are given in **Table 3-1** (Cheng et al., 2002).

Table 3-1. Surface free energy components of different probe liquids.

Probe liquids	Γ_{Total} (mJ/m^2)	Γ_L^{LW} (mJ/m^2)	Γ_L^{+} (mJ/m^2)	Γ_L^{-} (mJ/m^2)
Water	72.8	21.8	25.5	25.5
Glycerol	64	34	3.92	57.4
Formamide	58	39	2.28	39.6
Ethylene glycol	48.3	29.3	1.92	47

The relationship between contact angle and surface free energy can be expressed as Young-Dupré equation:

$$\Gamma_L(1+cos\theta)=2(\sqrt{\Gamma_S^{LW}\Gamma_L^{LW}}+\sqrt{\Gamma_S^{+}\Gamma_L^{-}}+\sqrt{\Gamma_S^{-}\Gamma_L^{+}}) \quad (11)$$

where L and S represent the probe liquids and the solid asphalt, respectively.

3.3 Asphalt mixture tests

3.3.1 Asphalt mixture performance tester (AMPT)

Two types of tests, the dynamic modulus test and the flow number test (FN), are included in the AMPT tests following the procedures NCHRP 9-29 and NCHRP 9-19, respectively (Bonaquist, 2011; Witczak, 2002). The compacted specimen with a diameter of 100 mm and a thickness of 150 mm is placed in an environmental chamber with an appropriate testing temperature, shown in **Figure A-4**. A haversine compressive stress is applied to the test specimen at a specific frequency. Three linear variable displacement transducers are used at 120° angles to capture the deformation of the specimen during the test. The complex modulus, $|E^*|$ is used to define the stress-to-strain relationship for a linear viscoelastic mixture specimen. Dynamic modulus tests at different loading frequencies and temperatures are used to create a master curve.

Flow number test (FN) is aimed to evaluate the creep characteristics of mixtures subjected to a repeated-load permanent deformation. The flow number denoted as the point at which the permanent strain rate is at the minimum value, and tertiary flow begins has been correlated with the rutting resistance of the mixtures (Witczak, 2002).

During the flow number test, a sample at a specific temperature is tested under repeated haversine axial compressive load of 0.1 second every 1.0 second for a maximum of 10,000 cycles or until deformation of 50,000 microstrains is reached. The point in the permanent strain curve where the accumulation rate of permanent strain reaches a minimum value has been defined as the flow number, as shown in **Figure A-5**.

The test can be conducted with or without confining pressure. The permanent axial strain obtained during the test is measured as the load cycles. The flow number is defined as the number of load cycles in relation to the minimum rate of change of axial strain. It can be related to resistance to permanent deformation (rutting). The flow number test should be performed on the samples that are 150-mm in height and 100-mm in diameter. The final sample is cored and cut from a larger sample around 170-mm in height and 150-mm in diameter. The test is performed at a specific temperature, and the testing chamber should equilibrate temperature for at least 1 hour. Follow the AMPT software to start the test, and when the test is finished, the AMPT will unload itself. The AMPT software performs the calculation of the flow number for every specimen. The operator should calculate the average.

3.3.2 Superpave Indirect tension (IDT) test

This study conducted two types of Superpave indirect tension (IDT) tests: the resilience modulus test and the indirect tensile strength test. Strain gages were used to obtain the vertical and horizontal strain readings. The resilient modulus and the indirect tensile strength tests were usually employed at 25° C. **Figure A-6** shows the test setup of the Superpave IDT test.

3.3.2.1 Resilient modulus test

The resilient modulus test was performed on the cylindrical samples by applying a repeated peak-load resulting in horizontal deformations within the range of 200–300 micro strains. Each load cycle consists of 0.1-s load application followed by a 0.9-s rest period. The load and deformation were continuously recorded, and the resilient modulus can be calculated as follows:

$$M_R = \frac{P \times GL}{\Delta H \times t \times D \times C_{cmpl}} \tag{12}$$

Where M_R = resilience modulus; P = maximum load; GL = gage length; ΔH = horizontal deformation; t = thickness of specimen; D = diameter of specimen; C_{cmpl} = $0.6354\,(\mathrm{X/Y})^{-1} - 0.332$; nondimensional creep compliance factor and $\mathrm{X/Y}$ = the ratio of horizontal to vertical deformation

3.3.2.2 Indirect tension strength test

The IDT strength test was used to determine tensile strength and strain of the mixture specimens compacted to 7 ± 1 % air voids. Cylindrical specimens with 150 mm in diameter and 50 mm in thickness were monotonically loaded to failure along the vertical diametric axis at the constant rate of 50 mm/min. The load and deformation were continuously recorded during the test. The peak load was determined, which was used to compute indirect tensile stress at failure using the following equation:

$$S_t = \frac{2 \times P \times C_{sx}}{\pi \times t \times D} \tag{13}$$

Where S_t = indirect tensile strength; P = peak load; $C_{sx} = 0.948 - 0.01114 \times (t/D) - 0.2693 \times \upsilon + 1.436 \times (t/D) \times \upsilon$ Where $\upsilon = -0.1 + 1.48 \times (X/Y)^2 - 0.778 \times (t/D)^2 \times (X/Y)^2$; *D*, *t*, and (*X/Y*) are the same as described above.

According to the stress-strain response curve from the IDT strength test (**Figure A-7**), the dissipated creep strain energy threshold ($\mathrm{DCSE_f}$) can be determined afterward:

$$\mathrm{DCSE_f} = FE - EE \tag{14}$$

Where *FE* = fracture energy, which represents the area under the stress-strain curve to the failure strain ε_f and *EE* = elastic energy.

$$FE = \int_0^{\varepsilon_f} S(\varepsilon) \tag{15}$$

$$EE = \frac{1}{2} S_t(\varepsilon_f - \varepsilon_0) \tag{16}$$

3.3.3 Asphalt pavement analyzer (APA) rutting test

The Asphalt Pavement Analyzer (APA) is widely used to evaluate the rutting resistance of the asphalt mixtures. Based on AASHTO TP 63-09 procedures, the wheel is tracked across the sample for 8000 cycles using a 100-lb load and a 100-psi hose pressure. In this study, the specimens were conditioned in the molds at 64 ° C for 6 h, and the rut depths at 8000 cycles were recorded. **Figure A-8** shows the specimens after the APA rutting test.

3.3.4 Moisture susceptibility test

Two types of moisture susceptibility tests were conducted to evaluate the moisture resistance of HIR, and HMA mixes, the immersion-conditioned (IM) test and the freeze-thaw-conditioned (F-T) test. The asphalt mixtures were compacted into a cylinder with a diameter of 150 mm and a height of 50 mm at 7% $\pm$ 0.5% air voids. One group of specimens was immersed in a water bath at 60 ° C for 24 hours, followed by conditioning at 25 ° C for 2 hours, whereas the other group of specimens was conditioned in the freeze-thaw machine for one cycle before further IDT strength testing. Tensile strength ratio (TSR) between the conditioned and unconditioned specimens was calculated to evaluate the moisture resistance of the HIR and HMA mixes.

3.3.5 Ideal-CT Test

The Ideal-CT test was adopted to evaluate the cracking resistance of the HIR mixes following the ASTM D8225 standard (Zhou et al., 2017). The specimens were compacted at a diameter of 150 mm and a height of 75 mm and were centered in the Material Test System (MTS). The load was applied at a 50 mm/min rate during the test. The load and load-line displacement (LLD) were collected and plotted for CTindex calculations. The work of failure (Wf) is firstly calculated by integrating the area under the load and LLD curves. Then, the failure energy (Gf) is obtained by dividing the work of failure by the cross-sectional area. Finally, the CTindex is derived from the parameters in the load-displacement curves using equation (17):

$$CT_{index} = \frac{t}{62} \times \frac{l_{75}}{D} \times \frac{G_f}{|m_{75}|} \times 10^6 \tag{17}$$

where CTindex is the cracking tolerance index; l_{75} is the displacement at 75% of the peak load after failure; $|m_{75}|$ is the slope of the tangential zone around the 75% peak load point after the peak; t is the specimen thickness (mm); D is the specimen diameter (mm).

3.4 ME pavement design

3.4.1 HIR & HMA (D-mix) pavement model

The ME software was utilized to predict the performance and life cycle between the pavement after HIR surface treatment and the construction of a new HMA surface layer. The pavement performance models were locally calibrated to ensure the reliability of the prediction (Gong et al., 2017; Zhou et al., 2013). The traffic, climate, and pavement structures were determined according to the field project condition. The pavement layer and thickness were adopted following the TDOT pavement design guide. The average two-way annual daily truck traffic (AADTT) was calculated as 2000. To simulate a larger volume of traffic conditions, the pavement performance was also predicted with an AADTT of 4000. The design life was set as 20 years. The pavement structure models are presented in **Figure A-9**. D-mix, a dense mix of HMA, is applied as the surface layer, while BM2-mix is used as the binder layer in Tennessee. Afterward, the pavement design life was reduced until all the pavement performance satisfied the pavement design criteria, which determined the pavement's life cycle.

3.4.2 CIR & HMA (BM2 mix) pavement model

ME software was also used to predict the pavement life with CIR treatment and BM2 binder layer replacement. A 2-inch overlay with HMA was constructed on the top of the binder layer. The traffic volume was set as 2000 and 4000 AADTT to simulate different traffic conditions. The climate conditions were similar to HIR pavement. The pavement structure models are presented in **Figure A-10**.

3.4.3 Design criteria and threshold

A series of pavement performances were analyzed at a design life of 20 years, including international roughness index (IRI), top-down fatigue cracking, bottom-up fatigue cracking, thermal cracking, and pavement deformations. The performance criteria were summarized in **Table 3-2** to achieve a fair pavement condition with Tennessee pavement design criteria (**Table 3-3**).

Table 3-2. Design parameters thresholds in ME design.

Design input	*Limit*	*Reliability*
IRI (in/mile)	170	90
AC top-down fatigue cracking (% lane area)	20	90
AC bottom-up fatigue cracking (% lane area)	20	90
AC thermal cracking (ft/mile)	1000	90
Permanent deformation – total pavement	0.4	90
Permanent deformation AC only (in)	0.25	90

Table 3-3. Tennessee pavement design criteria.

Design input	*Limit*	*Reliability*
IRI (in/mile)	170	90
AC top-down fatigue cracking (% lane area)	20	90
AC bottom-up fatigue cracking (% lane area)	20	90
AC thermal cracking (ft/mile)	1000	90
Permanent deformation – total pavement	0.4	90
Permanent deformation AC only (in)	0.25	90

3.5 Experimental design and outline

In this section, the experimental design and each specific task are presented. The flow charts were constructed to clearly show the contents of each task.

3.5.1 Task 1: A Comparative Study on Pavement Rehabilitation using HIR & HMA (D-mix) for surface layer: Performance Evaluation, Pavement Life Prediction, and Life Cycle Cost Analysis

Figure A-11 shows the flow chart of task 1. This task aims to conduct comprehensive comparisons between HIR and HMA mixes and pavement, including performance evaluation, pavement life prediction, and LCCA. To achieve this, HIR mixes from three different projects were collected and recompacted in the lab. For comparison, one common HMA surface mix was also obtained from the asphalt plant. Asphalt Mixture Performance Tester (AMPT), Superpave indirect tensile strength (IDT) tests, tensile strength ratio (TSR) tests were adopted for performance evaluation. Field cores were collected to assess the in-place construction qualities. ME software was utilized for modeling and pavement life prediction.

3.5.2 Task 2: A Comparative Study on Pavement Rehabilitation using CIR & HMA (BM2 mix) for binder layer: Performance Evaluation, Pavement Life Prediction, and Life Cycle Cost Analysis

Similar to task 1, task 2 was to conduct a comprehensive comparison between CIR and HMA (BM2 mix) for the binder layer including performance evaluation at different curing period, pavement life prediction using ME software, and life cycle cost analysis for the whole life cycle. **Figure A-12** shows the flow chart of task 2.

3.5.3 Task 3: Temperature effect on the performance and binder mobilization of HIR mixes

This task aims to investigate the influence of mixing and compaction temperature on the performance of the HIR mix. To achieve this, the loose mixes, after being incorporated with additives at 110 ° C were collected in the field project. In the lab, the mixtures were remixed and compacted to a target air void using Superpave Gyratory Compactor (SGC) at 110° C, 120° C, and 130° C in the lab. The Asphalt Pavement Analyzer (APA) tests, the Superpave indirect tension (IDT) tests, the asphalt mixture performance tester (AMPT), and moisture susceptibility evaluation were used to characterize the performances of the asphalt mixtures compacted at different temperatures. Also, a staged extraction test was used to identify the mobilization of RAP binders and the blending mechanism of the binder blends under different mixing temperatures. **Figure A-13** and **Figure A-14** show the flow charts of mixture performance evaluation and binder mobilization in task 3, respectively.

3.5.4 Task 4: Quantify and maximize the binder mobilization and recycling efficiency of HIR mixes

The objective of this task was to simulate the HIR procedures and introduce a straightforward approach to quantifying the effective mobilized RAP content of the HIR mix based on the ignition loss of the RAP binder. The HIR process was simulated with the application of the torch and mixer, in which the temperature distribution was also captured through the infrared camera. The ignition oven method was used to obtain the RAP binder's ignition loss directly. The influence of various factors on the effective mobilized RAP content was considered, including the temperatures, the types of additives, the additives content, and the heating method. The FTIR method was also adopted to compare the effective mobilized RAP content with the ignition oven method. **Figure A-15** shows the flow chart of task 4.

3.5.4.1 Heating methodology

Oven heating methodology is used to simulate the asphalt mixing procedures in the asphalt plant; however, as for the HIR train in the field, the heating units apply fire to directly heat the pavement in a short period. The use of fire will activate more but may burn a little RAP binder, which may influence the mobilization rate of RAP. Therefore, the torch heating procedure is designed to simulate the fire heating in the lab. A torch is controlled to directly blow the asphalt mixtures in the asphalt mixing machine, while a digital infrared thermometer is used to detect the temperature. The infrared camera is also applied to capture the temperature distribution of the asphalt mixtures until reaching the target temperature.

3.5.4.2 Effective mobilized RAP content quantification

The effective mobilized RAP content refers to the activated RAP binder that is used to coat aggregates. The asphalt ignition furnace was applied to determine the asphalt content of asphalt mixtures using the loss on ignition following ASTM D6307. In order to directly simulate the blending process and quantify the proportion of effective RAP content, virgin aggregates that retained ½ and 3/8 inches were selected to replace part of coarse RAP in the mix while keeping the fine RAP that passed No. 4 sieve since it is convenient to differentiate and separate the coarse virgin aggregates and fine RAP for further tests.

There exist two scenarios in quantifying the effective mobilized RAP content (**Figure A-16**): 1. "Dry" mixing, the blend including coarse virgin aggregates and fine RAP only, which is to simulate the heat scarification and scrape off RAP during HIR procedures. 2. "Wet" mixing, the mixtures with the incorporation of additives or asphalt (rejuvenator, asphalt emulsion, and commercial asphalt PG 64-22 used in this study), aiming at simulating the injection of additives in the HIR technique. **Figure A-16** (a) presents the blending condition of "dry" mixing. Before and after "dry" mixing, the fine RAP was picked out and fully burnt in the ignition furnace. The effective mobilized RAP content could be obtained by the difference of the ignition loss of the RAP materials before and after ignition. Further, the calculation of effective mobilized RAP content would be more complicated under the scenarios with the incorporation of rejuvenator, asphalt emulsion, and commercial virgin asphalt (PG 64-22), since additives or asphalt binder are capable of diffusing into the RAP binder, which can help with the activation and mobilization. As shown in **Figure A-16** (b), the asphalt emulsion tends to adhere to the surface of both virgin and RAP aggregates as well. Therefore, after the "wet" mixing, the ignition loss of the collected fine RAP consists of the mobilized RAP content and the attached additives or virgin binder. Approaches are needed to get rid of this proportion of asphalt emulsion films. Previous studies usually applied the surface area of different sizes of aggregates to roughly estimate the mass of asphalt film around the aggregate particles (Craus and Ishai, 1977; Ding et al., 2016a). In this study, as shown in **Figure A-16** (c), a relatively direct method was developed for the asphalt emulsion film calculation, which is to substitute all the fine RAP with a similar gradation of virgin aggregates for mixing and burning the collected fine virgin aggregates afterward. The mass of the asphalt emulsion film could be determined through the ignition loss of the collected virgin aggregates. Based on this approach, various conditions, including temperatures, additives dosages, and heating methods, were considered to quantify the effective mobilized RAP content of 100% RAP after HIR procedures.

3.5.5 Task 5: Influence of mobilized RAP content on the effective binder quality and performance of HIR mixes

This task aims to investigate the influence of the mobilized RAP content on the effective binder quality and the performance of HIR mixes. The loose RAP materials were collected in an HIR project after heat scarification. The ignition oven method was adopted to quantify the mobilization rate of the RAP binder and obtain the effective binder content. The effective binder blends were prepared based on the proportion of the mobilized RAP binder and the dosages of the recycling agent, followed by characterizing binder qualities through a dynamic shear rheometer (DSR). The performances of HIR mixes at different mobilization conditions were tested using Asphalt Mixture Performance Tester (AMPT) and Ideal-CT cracking tests. **Table 3-4** summarizes the design of mobilized RAP quantification for 100% RAP with different mixing conditions, including temperature, asphalt types, and dosages. **Figure A-17** presents the flow chart of task 5.

Table 3-4. Effective mobilized RAP quantification under different mixing conditions.

Mixture label	*Temperatures (° C)*	*Asphalt types*	*Dosages (%)*
RA11013	110	ARA-3P	1.3
RA11026	110	ARA-3P	2.6
RA11039	110	ARA-3P	3.9
RA13013	130	ARA-3P	1.3
RA13026	130	ARA-3P	2.6
RA13039	130	ARA-3P	3.9
V16013	160	PG-6422	1.3
V16026	160	PG-6422	2.6
V16039	160	PG-6422	3.9

Chapter 4 Results and Discussion

4.1 Surface layer rehabilitation between HIR and HMA

4.1.1 Performance evaluation

4.1.1.1 Sieve analysis and asphalt content

HIR and HMA surface mixes were collected from three different construction sections and asphalt plants in Tennessee, respectively. All the materials came from a certain region, indicating similar aggregate types. The grain size distributions of the raw materials are shown in **Figure 4-1**. Both HIR and HMA mixes follow a similar gradation, which generally matches the typical surface mix in Tennessee. One cationic asphalt emulsion was used as a recycling agent to restore the binder properties of RAP. The asphalt contents of HIR and HMA mixes were determined through the centrifuge extraction test. As shown in **Table 4-1**, the asphalt contents of HIR mixes are generally higher than HMA since HIR mix contains the asphalt binder in RAP and asphalt emulsion. The incorporation of recycling agents would soften and activate more RAP binders and increase the effective binder contents of HIR mixes (Ma et al., 2021a). The asphalt contents in three HIR mixes represent the different dosages of asphalt emulsion incorporated during construction. The loose HIR mixes were reheated and recompacted for performance testing. Two replicates of AMPT tests and three replicates of IDT tests were conducted. Twelve field cores with a diameter of 150 mm were also extracted from the pavements before and after HIR procedures in each section, followed by cutting the top 50 mm for further performance testing.

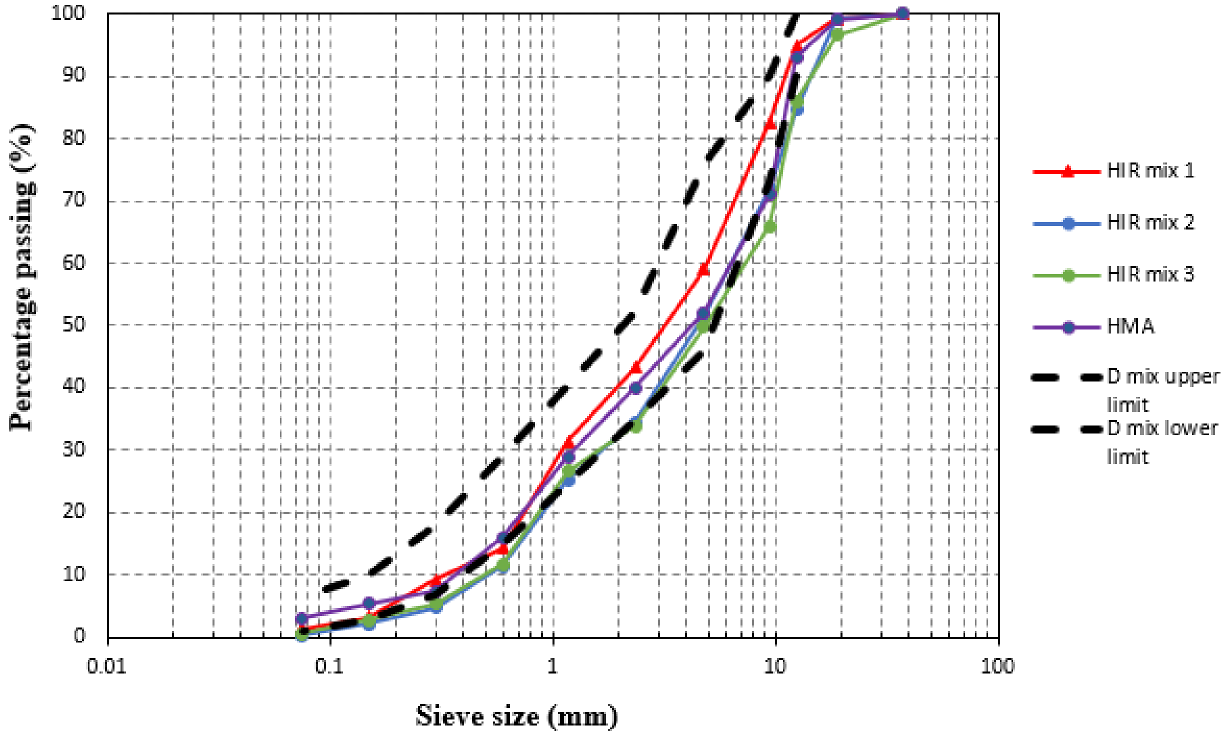


Figure 4-1. The grain size distributions of raw materials.

Table 4-1. Asphalt content of HIR and HMA mixes.

Type of Mixes	*HIR mix 1*	*HIR mix 2*	*HIR mix 3*
Asphalt content (%)	6.93	7.32	7.68

4.1.1.2 AMPT test results.

Figure 4-2 plots the dynamic modulus master curves of HIR and HMA mixes. The former study reflects that the dynamic modulus of asphalt mixtures is sensitive to the change of asphalt binders (Huang et al., 2008). The mixtures with higher asphalt content present a lower dynamic modulus at all temperatures and frequency sweeps in HIR mixes. A larger proportion of HIR binder generates a softer asphalt mixture, indicating more rutting potential of the pavements. Compared to HIR mixes, HMA mixes present a larger dynamic modulus value at low temperatures (high frequency) but a lower value at high temperatures (low frequency). It can also be noticed that HMA mix has less asphalt content than HIR mixes but provides stiffer performances of the asphalt mixtures, especially at low temperatures. Such differences might attribute to the better adhesion and coating capability between the virgin asphalt and the aggregates. As the temperature increases, the virgin binder tends to be softened more severely than the RAP binder, indicating more rutting issues of the asphalt mixtures. A lower flow number value of HMA, obtained at 54.4 ° C in **Figure 4-3**, also validates more rutting potential of HMA pavements at high service temperatures.

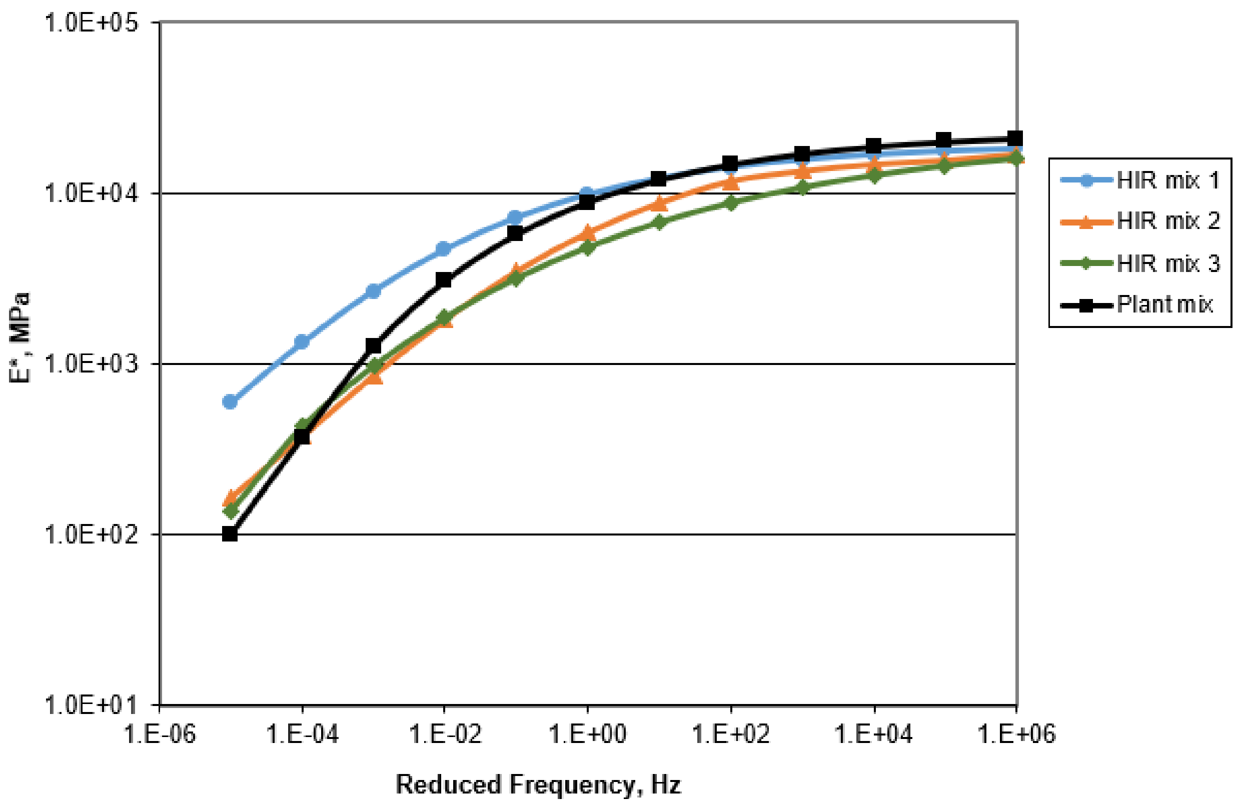


Figure 4-2. Dynamic modulus master curves of different asphalt mixtures.

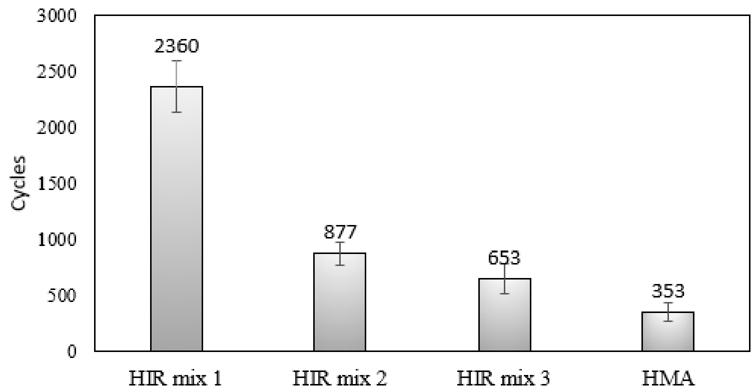
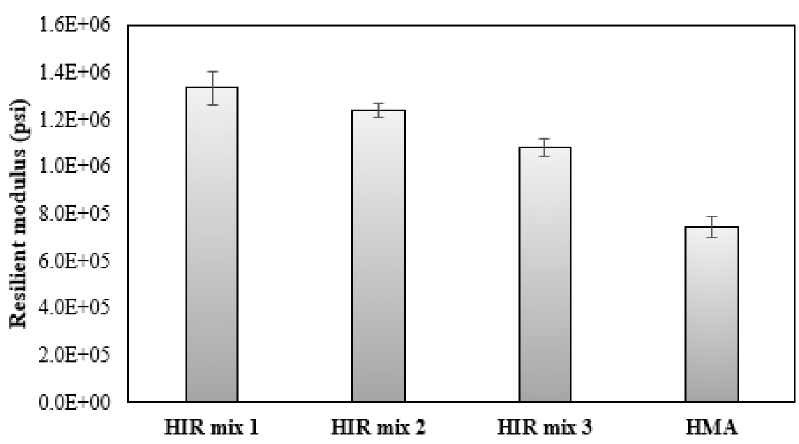


Figure 4-3. Flow number test results of different asphalt mixtures.

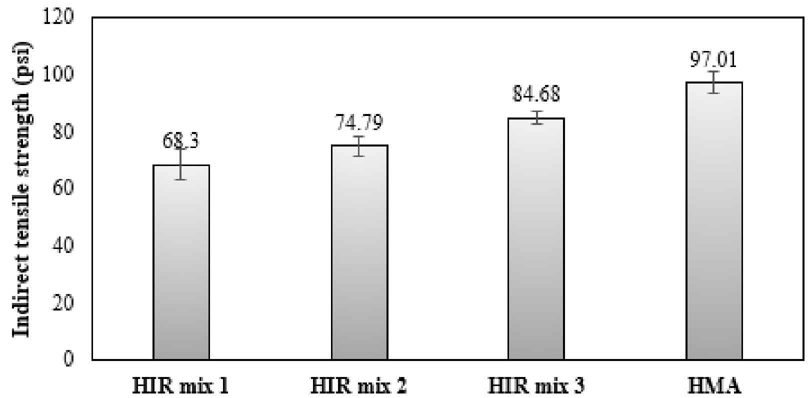
4.1.1.3 Superpave IDT results

Figure 4-4 (a), (b), and (c) illustrate the resilient modulus, IDT strength, and dissipated creep strain energy threshold ($DCSE_f$) test results of HIR and HMA mixes at room temperatures. The resilient modulus test is applied to characterize the elastic behavior of asphalt mixtures. As for HIR mixes, it is shown that HIR mix with higher asphalt content has a lower resilient modulus, indicating the increasing ductility of the asphalt mixtures. HMA mixes exhibit the lowest resilient modulus compared to HIR mixes due to the better ductility of virgin binder than the aged binder.

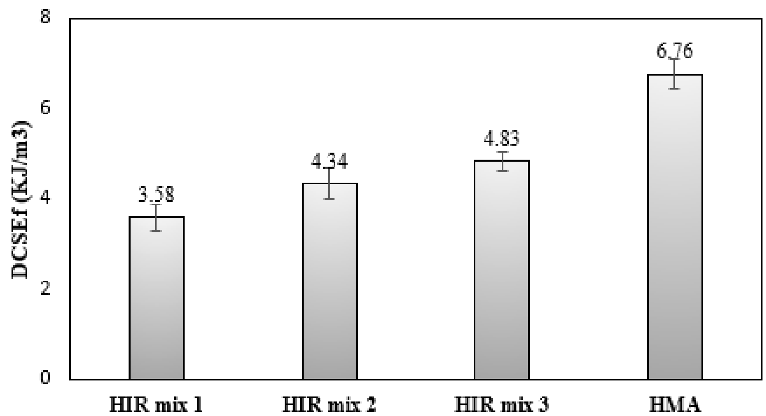
$DCSE_f$ reflects a threshold for cracking initiation and propagation of the asphalt mixtures. In general, asphalt mixtures with a higher $DCSE_f$ value have improved cracking resistance (Zhang et al., 2001b). As shown in **Figure 4-4** (b) and (c), the IDT strength and $DCSE_f$ of the HMA mix are much higher than the HIR mixes, even though it contains the lowest asphalt content. Therefore, the HMA mix tends to have better cracking resistance than the HIR mix. The high production temperature of HMA with virgin asphalt would contribute to an improved coating and adhesion between the binder and aggregates. However, in terms of HIR mix, the incomplete activation of RAP binder through fire heating, the low mixing, compaction temperature, and the large proportion of RAP binder would result in a weaker coating with aggregates, and hence the compromised cracking resistance of asphalt mixtures. In addition, the major concern of the HIR mix is the cracking issue, which can be potentially improved by using more additives and increasing the mixing temperatures.



(a)



(b)



(c)

Figure 4-4. Superpave IDT test results of HIR and HMA mixes, (a) Resilient modulus; (b) Indirect tensile strength; (c) $DCSE_f$.

4.1.1.4 Moisture susceptibility test results

The immersion conditioned and freeze-thaw conditioned tests were adopted to evaluate the moisture resistance of the asphalt mixtures, denoted as TSR (IM) and TSR (F-T) in **Figure 4-5**, respectively. Freeze-thaw conditioning has more severe moisture damage than the immersion conditioning method. It can be seen that the HMA mix has better moisture resistance than the HIR mix regarding both moisture damage scenarios. The TSR values for HIR mix with different asphalt content did not vary significantly. The TSR values for HIR mix underwater immersion conditions could satisfy the minimum criteria for Superpave asphalt mixture design (TSR $\geqslant$ 0.80). Further strategies should be considered to improve the moisture resistance of HIR mixes, especially for areas with significant temperature differences.

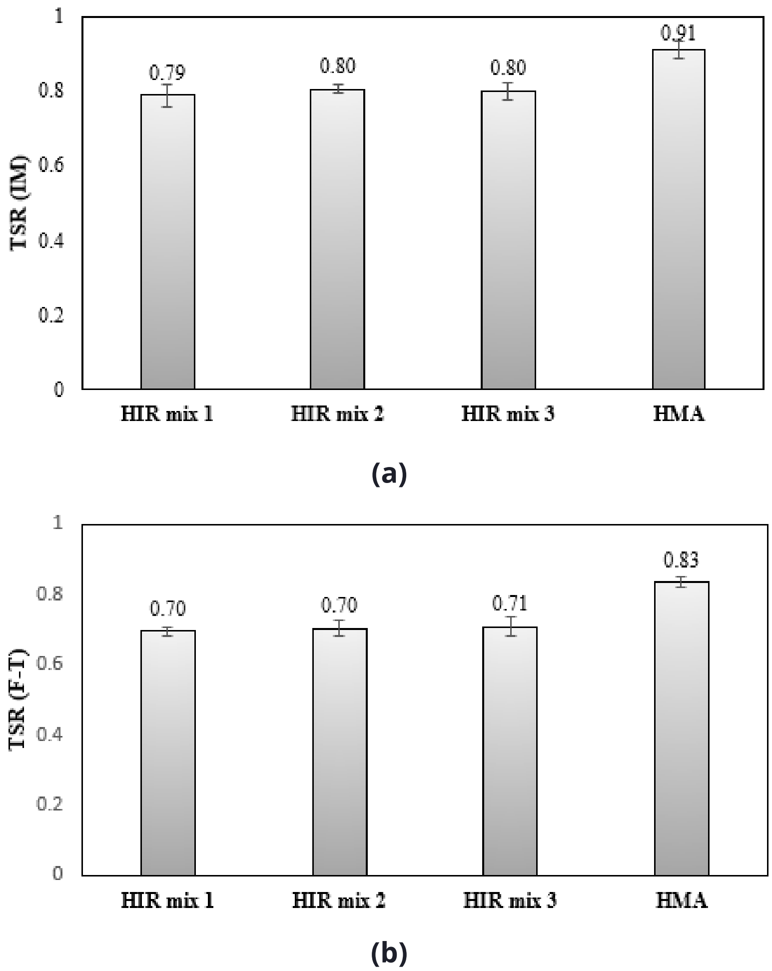
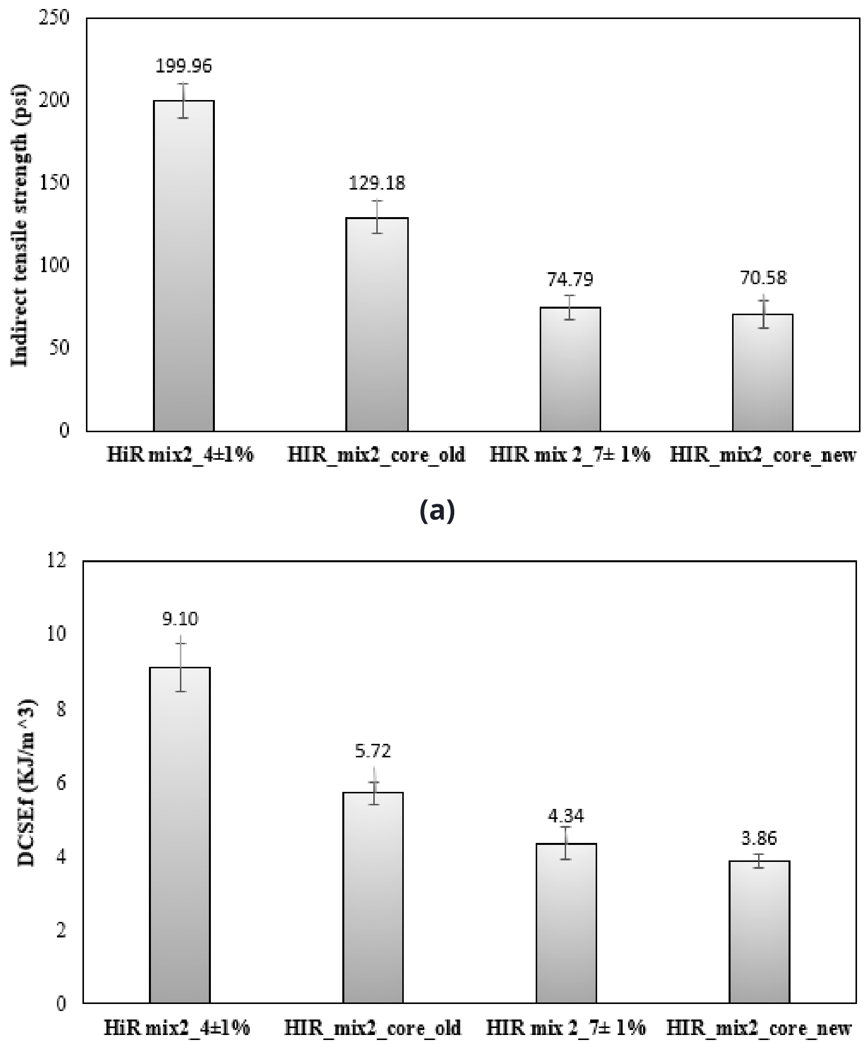


Figure 4-5. Moisture susceptibility tests of HIR and HMA mixes, (a) TSR (IM); (b) TSR (F-T).

4.1.1.5 Field cores evaluation

Field cores from one HIR section were collected from the pavement before and after HIR surface treatment. The air voids of old and new cores are around 4% and 7%, respectively. The cores were trimmed from the top 50 mm for further Superpave IDT tests. As a comparison, the collected HIR loose mixes were also compacted at 7% and 4% air voids in the lab to simulate the pavement condition after construction and long service life. **Figure 4-6** (a) and (b) illustrate the indirect tensile strength and $DCSE_f$ test results of the cores and the lab compacted specimens. It can be noticed that the IDT strength and dissipated creep strain energy of the old pavement decrease around 40%, corresponding to their initial conditions. This means that the quality of the pavement surface was mitigated due to the cracking issues during service lives, which required surface rehabilitation. The new cores and lab compacted HIR mix present similar values of the IDT strength and $DCSE_f$, indicating a good quality control of the pavement surface treatment through HIR procedures.



(b)

Figure 4-6. Performance comparison between field cores and lab specimens, (a) IDT strength, (b) DCSE$_f$.

4.1.2 Pavement life prediction

The ME software was adopted to predict the pavement's service life through HIR and HMA surface rehabilitation considering different pavement performance criteria. The input parameters are shown in the Appendices. The pavement performances after 20 years are predicted in **Table 4-2.** Compared to the pavements constructed by the new HMA surface layer, the pavements after HIR procedures presented similar permanent deformations but higher fatigue cracking area and thermal cracking length. Both the bottom-up fatigue cracking area and the thermal cracking length would not satisfy the criteria. Therefore, the pavement after HIR procedures was not susceptible to rutting issues, whereas it tended to encounter bottom-up fatigue cracking problems, especially thermal cracking. The pavement prediction results agree with the laboratory performance test results.

Table 4-3 presents the life cycle of pavements at two traffic conditions using different rehabilitation approaches. At 2000 AADTT, pavement's life cycle after the HIR technique is around eight years, four years shorter than the pavement with the new HMA surface layer. When the traffic increases up to 4000 AADTT, the pavement life with PG 64-22 was reduced to 9 years, and the pavement life with HIR surface rehabilitation decreased to 4 years. The LCCA of each rehabilitation techniques were presented in the next section.

Table 4-2. Pavement performance after 20 years.

Performance criteria	*PG 64-22*	*HIR*	*Limit*	*Reliability*
IRI (in/mile)	158.24	172.87	170	90
AC top-down fatigue cracking (% lane area)	4.69	14.13	20	90
AC bottom-up fatigue cracking (% lane area)	58.44	64.47	20	90
AC thermal cracking (ft/mile)	934.59	1727.52	1000	90
Permanent deformation - AC only (in)	0.22	0.20	0.25	90

Table 4-3. Life cycles of the pavement after different rehabilitation approaches.

Traffic level (AADTT)	*PG 64-22*	*HIR*
2000	12 years	8 years
4000	9 years	4 years

4.1.3 Life cycle cost analysis (LCCA)

The Estimated Uniform Annual Cost (EUAC) method expresses life cycle costs as an annualized estimation of cash flow instead of a lump-sum estimation of the present value. The EUAC values of HIR and conventional HMA were calculated using the following equation:

$$EUAC = Cost\ \frac{i(1+i)^n}{(1+i)^n-1} \tag{18}$$

where i = discount rate, and n = number of years

The EUAC method can be applied to evaluate the economic value of different alternatives, reflecting the life cycle cost analysis of the pavement. As for the 2-inch surface layer, the average cost of the HIR section is 4.02 \$/yard2 (19778.1\$/lane mile), while that of the convention section via HMA milling and & filling is \$12.8/yard2 (\$62975.05/lane mile) in Tennessee. Correspondingly, assuming a discount rate of 4%, the EUAC of HIR over 8 years and the conventional pavement over 12 years are \$0.60/yard2 (\$2937.60/lane mile), and \$1.36/yard2 (\$6710.13 /lane mile), respectively. Therefore, pavement rehabilitated through HIR could offer up to 56% initial cost savings. **Figure 4-7** and **Figure 4-8** display the life cycle of pavement after HIR and HMA milling & filling at different traffic conditions. At 2000 AADTT, pavement with HIR surface rehabilitation could save 21.5% cost in the whole life cycle, while when the traffic increased to 4000 AADTT, the pavement with HIR surface rehabilitation could cost 25.6% more than traditional HMA milling & filling techniques. Hence, HIR could be more cost-effective than HMA milling & filling techniques for low volume roads.

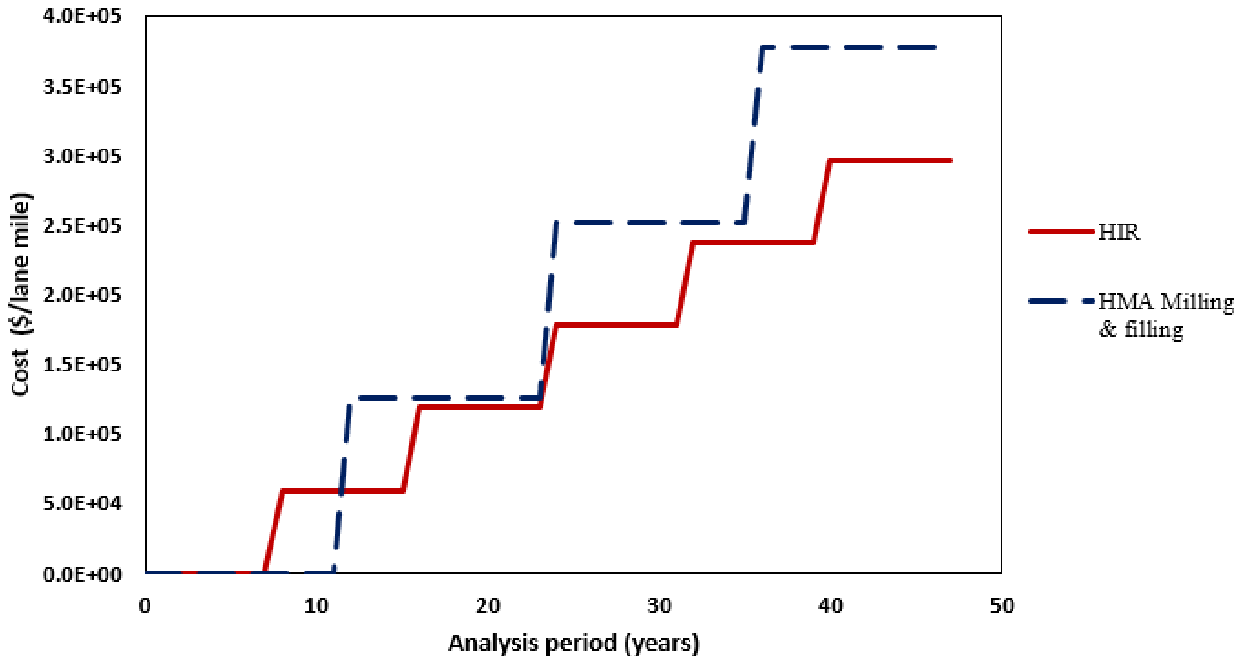


Figure 4-7. LCCA of pavement rehabilitation at 2000 AADTT.

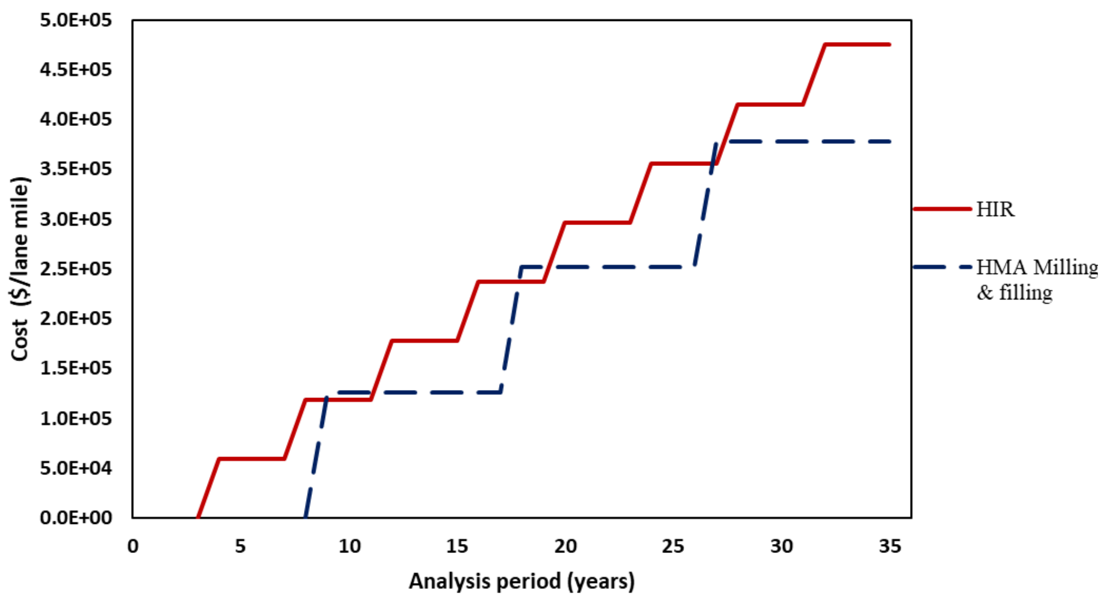


Figure 4-8. LCCA of pavement rehabilitation at 4000 AADTT.

4.2 Binder layer rehabilitation between CIR and HMA

CIR techniques are usually used to rehabilitate down to the base layer of the existing pavement. As for the CIR projects in region 2 and region 4, 1% cement was initially spread on the pavement surface; 2% water was sprayed, followed by scarifying the pavement down to 4 inches and mixing the loose mixes with water and asphalt emulsion (region 2) or foamed asphalt (region 4). Hence, the pavement layer after CIR rehabilitation can be treated as the new binder layer of the pavement, which is similar to traditional BM2 mixes in Tennessee. An additional HMA surface layer is usually placed on the CIR layer. How different are the performances between the CIR and BM2 mixes? A comprehensive performance evaluation between the CIR mix and the conventional BM2 mix have been evaluated. ME software was also adopted to predict pavement life using the CIR technique and HMA milling & filling. The LCCA of the pavement with two techniques was also conducted for the pavement with the two rehabilitation techniques.

4.2.1 Performance evaluation

4.2.1.1 Sieve analysis

The gradation chart of different mixtures is shown in **Figure 4-9**. It can be seen that the gradations of two CIR mixes are between the upper and lower limit of BM2 mixes. The CIR mixes in region 2 have coarser gradation than in region 4. The gradation of CIR mixes and BM2 mixes are comparable for further performance testing.

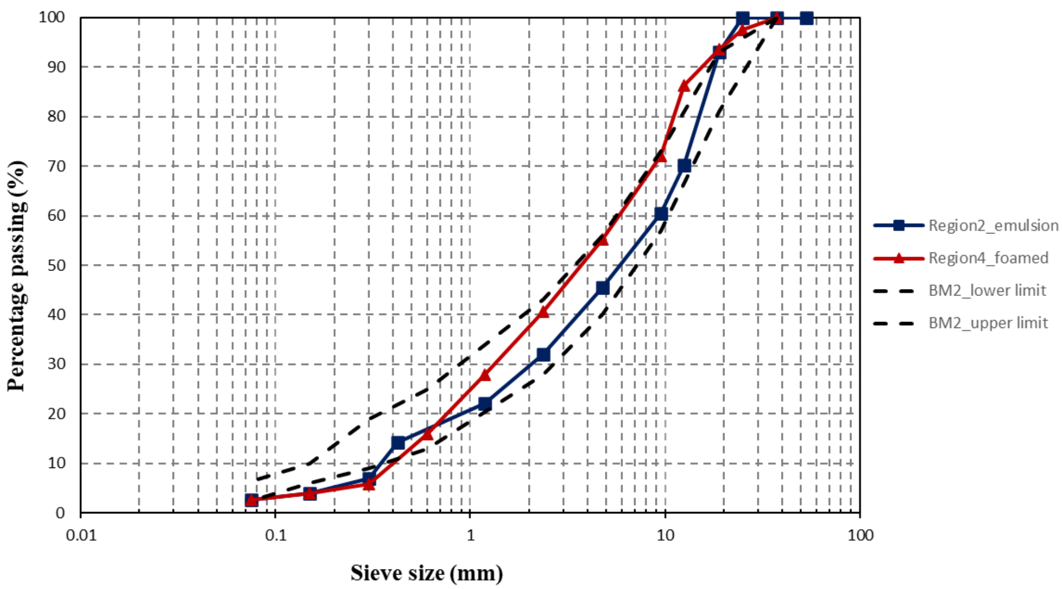
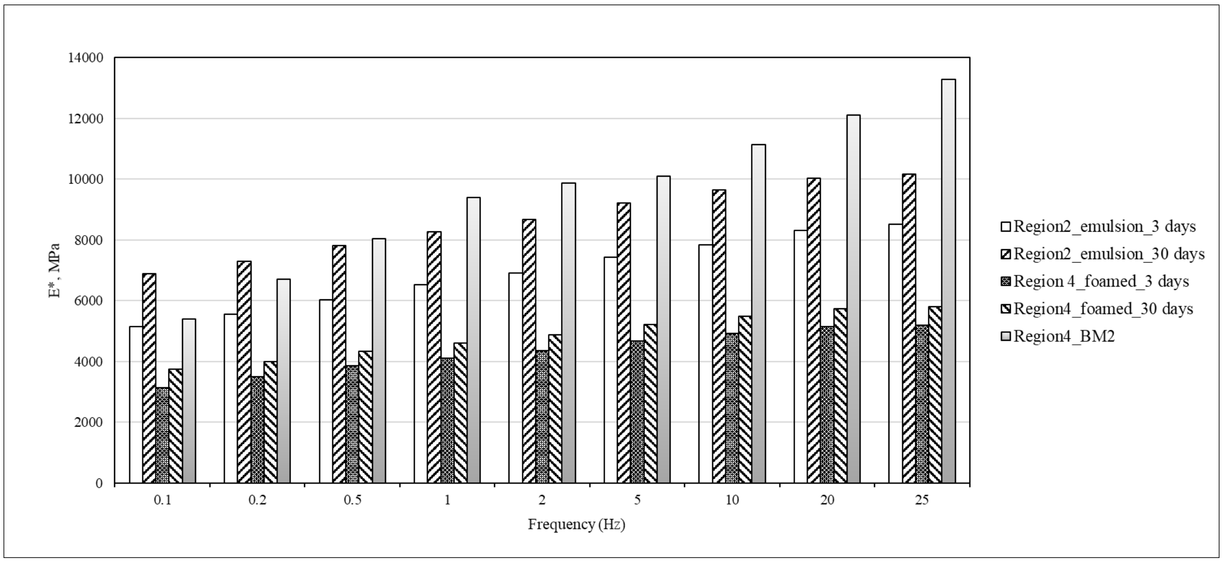


Figure 4-9. Gradation chart of different mixes.

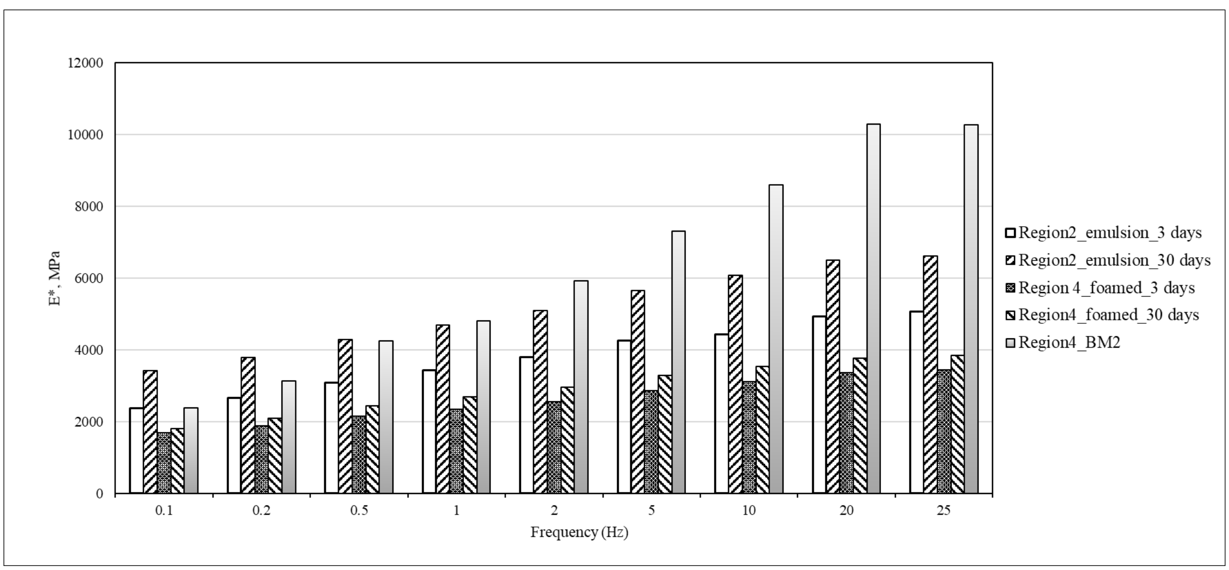
The loose CIR mixes were collected from the construction site and compacted through the SGC in the mobile trailer. Proctor test was firstly used to obtain the optimum moisture test and maximum dry density in which the gyrations were determined to reach the optimum compaction at maximum dry density. The compacted specimens were carefully sealed and brought into the UT lab for additional curing. The specimens were cured 3 days and 30 days, corresponding to the short and long period, respectively. After curing, the specimens were used for further performance tests, including resilient modulus test, IDT strength test, TSR tests, and AMPT tests. Three replicates were conducted for each test. The test results are listed as follows:

4.2.1.2 AMPT test results

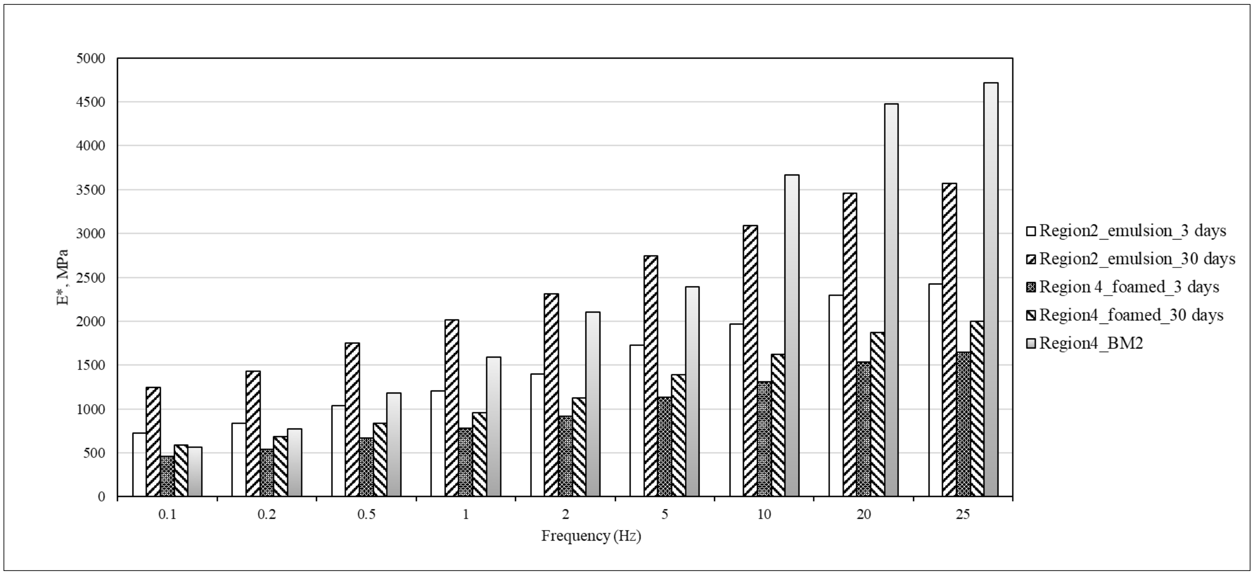
Figure 4-10 presents the dynamic modulus results of different mixes at three temperatures. From the figure, a longer curing period increased the dynamic modulus of the CIR mixes. However, compared with the BM2 mixes, CIR mixes after a long curing period still had smaller dynamic modulus, especially at low temperature, indicating less adhesion between asphalt and aggregates and more rutting potential. **Figure 4-11** shows the flow number of the CIR mixes after the 30-days curing period and BM2 mixes. It can be found that BM2 mixes had a significantly higher flow number than CIR mixes, indicating that pavement with CIR rehabilitation techniques might encounter rutting issue during service life.



(a)



(b)



(c)

Figure 4-10. Dynamic modulus of asphalt mixtures of two curing periods at different temperature. (a) 4C, (b)20C, (c) 40C.

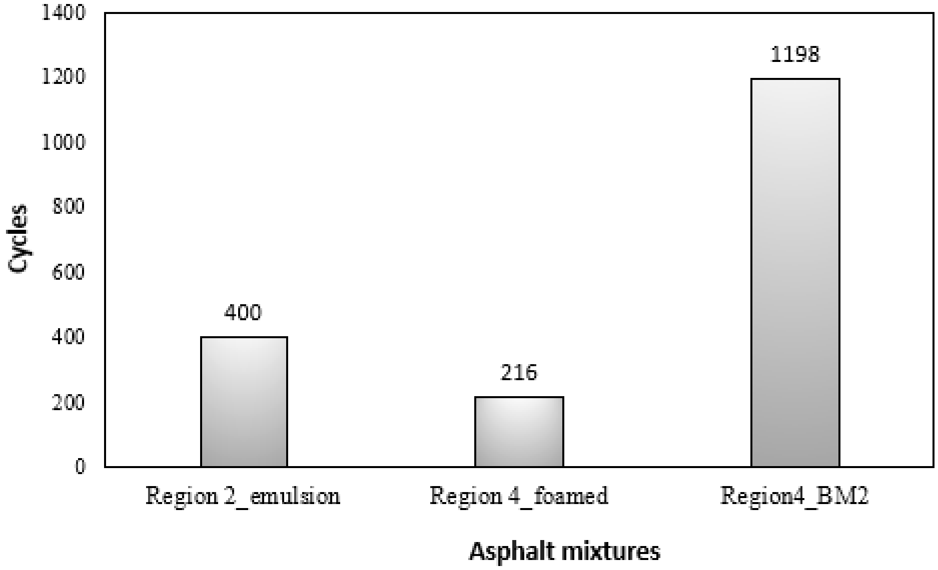
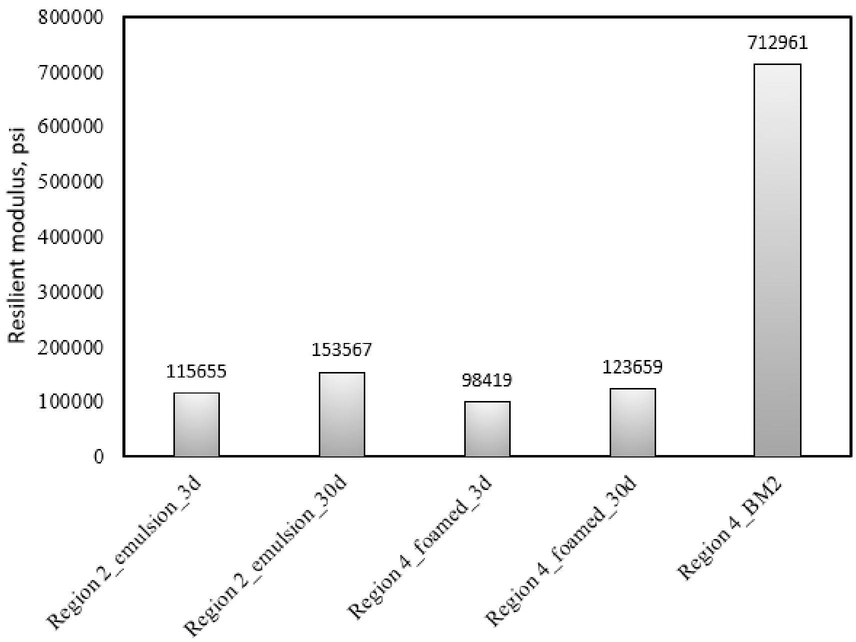


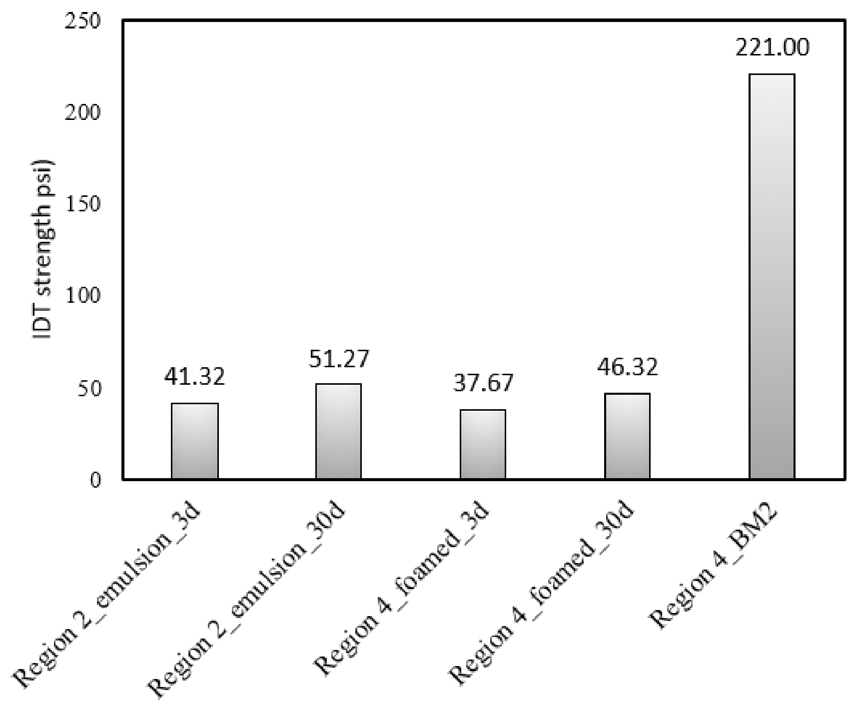
Figure 4-11. Flow number of CIR mixes after 30-days curing period and BM2 mixes.

4.2.1.3 Superpave IDT results

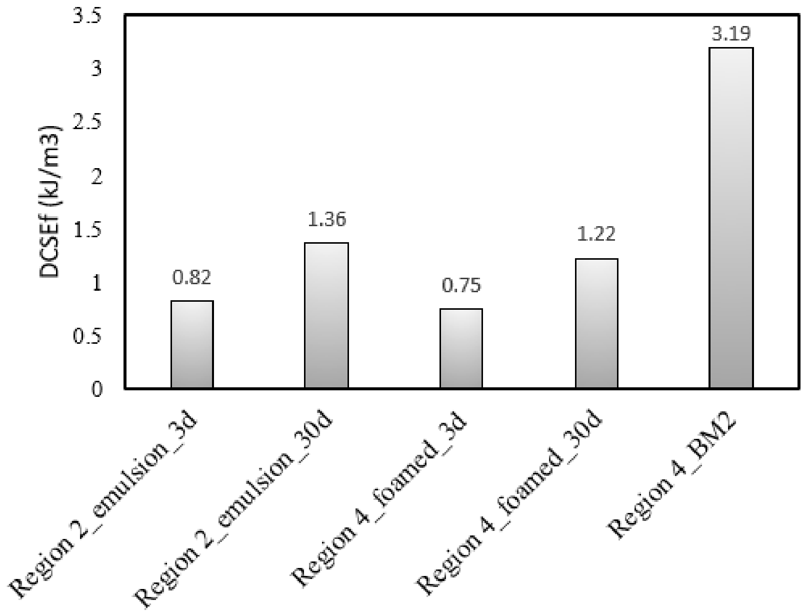
Figure 4-12 (a), (b), (c) illustrate the resilient modulus results, IDT strength results, dissipated creep strain energy ($DCSE_f$) results, respectively. It can be summarized from the results that a longer curing period could increase the resilient modulus, IDT strength, and $DSCE_f$ of the CIR mixes, reflecting improved cracking resistance and moisture resistance. Hence, a longer curing period is recommended to ensure a better cracking resistance of the CIR mixes. However, compared to conventional BM2 asphalt mixtures, CIR mixes would have significantly lower cracking resistance.


Resilient modulus, psi
800000
700000
600000
500000
400000
300000
200000
100000
0
115655
153567
98419
123659
712961
Region 2_emulsion_3d
Region 2_emulsion_30d
Region 4_foamed_3d
Region 4_foamed_30d
Region 4_BM2

(a)


IDT strength psi)
250
200
150
100
50
0
41.32
51.27
37.67
46.32
221.00
Region 2_emulsion_3d
Region 2_emulsion_30d
Region 4_foamed_3d
Region 4_foamed_30d
Region 4_BM2

(b)



(c)

Figure 4-12. Superpave IDT test results.

4.2.1.4 Moisture susceptibility test results

Figure 4-13 presents the TSR results for CIR mixes with different curing periods and the BM2 mixes. It can be seen that a longer curing period will significantly improve the TSR values and the moisture resistance of the CIR mixes, which satisfied the TSR criteria (≥ 80%) of the asphalt mixtures.

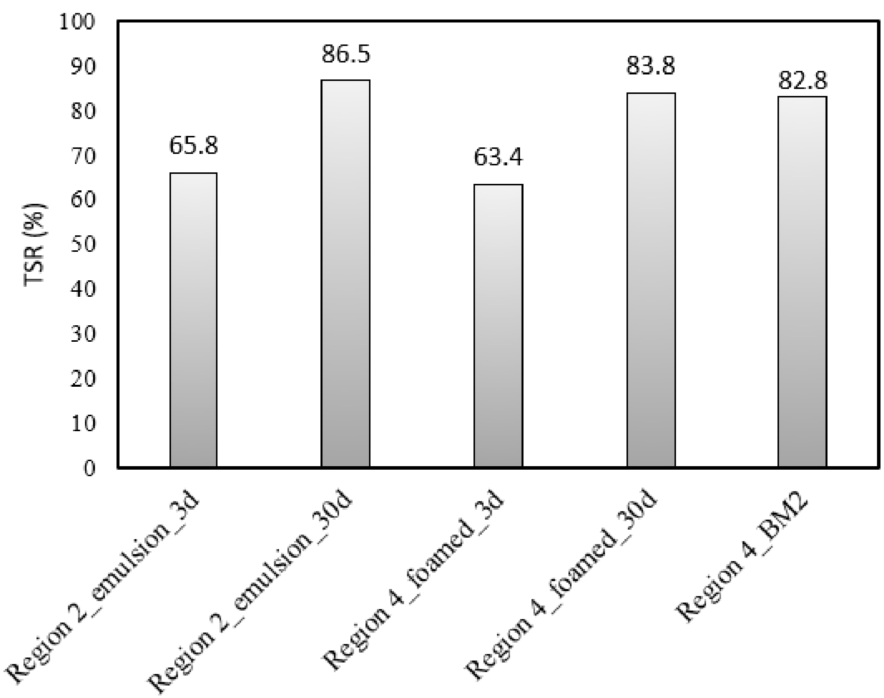


Figure 4-13. TSR results for CIR mixes with different curing periods and the BM2 mixes.

4.2.2 Performance life prediction

Table 4-4 shows the pavement performance with CIR techniques and traditional new binder layer rehabilitation under different traffic conditions after 20 years. Compared with the new BM2 layer, pavement with CIR rehabilitation techniques tend to have worse IRI and more cracking potential. **Table 4-5** summarizes the whole life cycle of the pavement with different rehabilitation techniques under different traffic conditions. It can be found that pavement rehabilitation with CIR techniques will reduce the pavement life by over 50%. The pavement can only last for 2 years if the AADTT increases to 4000, indicating that CIR techniques might be suitable for the low volume road (low traffic volume). The life cycle cost analysis for different rehabilitation methods was conducted in the next section.

Table 4-4. Performance prediction of the pavement after 20 years under different traffic conditions.

Performance criteria	***BM2_2000***	***BM2_4000***	***CIR_2000***	***CIR_4000***	***Limit***	***Reliability***
IRI (in/mile)	161.34	183.7	189.32	200.97	170	90
AC top-down fatigue cracking (% lane area)	13.91	14.16	13.97	14.16	20	90
AC bottom-up fatigue cracking (% lane area)	35.94	76.51	93.51	100	20	90
Permanent deformation - total pavement (in)	0.43	0.49	0.42	0.47	0.4	90

Table 4-5. Life cycle of the pavement with different rehabilitation techniques under different traffic conditions.

Rehabilitation methods	***BM2_2000***	***BM2_4000***	***CIR_2000***	***CIR_4000***
Pavement life	12 years	6 years	7 years	2 years

4.2.3 Life cycle cost analysis (LCCA)

TDOT did not have the cost data of CIR projects, the CIR cost from literatures were obtained for LCCA. Offenbacker et al. conducted a study to evaluate the economic and environmental cost analysis of CIR and conventional HMA milling (Offenbacker et al., 2021). **Figure 4-14** summarizes their test results. It shows that the total cost for HMA mill & overlay is approximately $82300/lane mile, while the average total cost for CIR is $46150/lane mile, reflecting an initial 43.9% cost saving of the pavement.

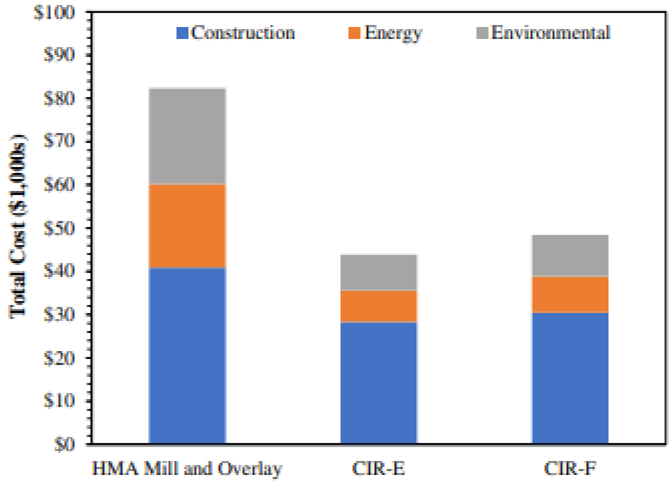
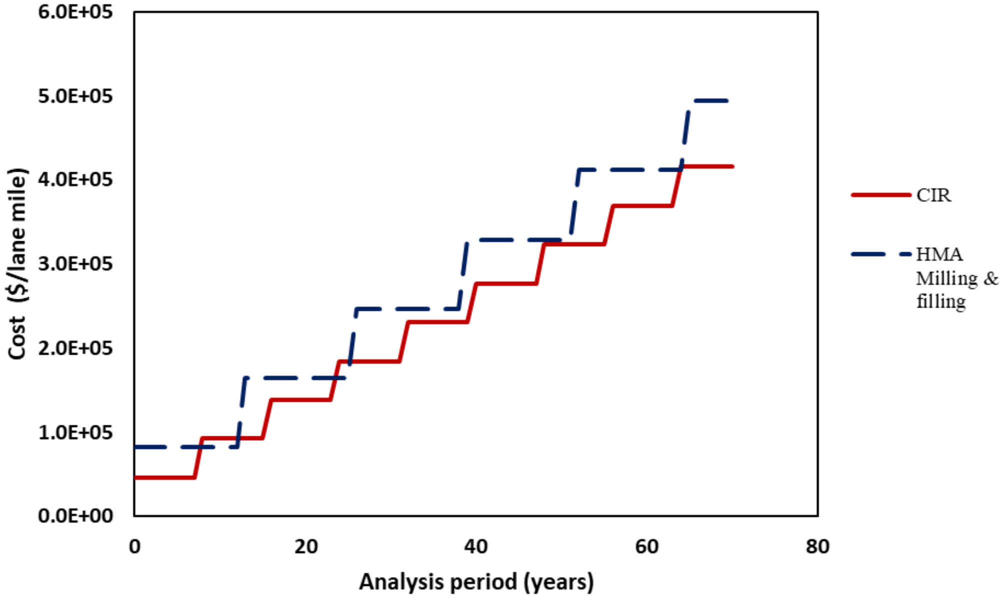
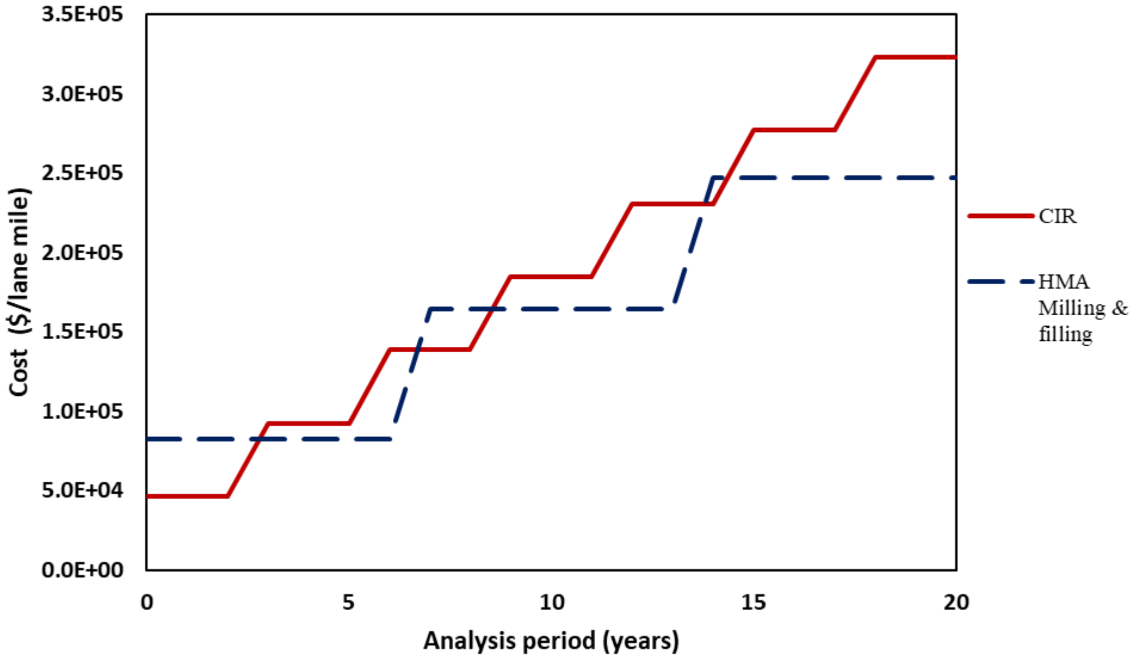


Figure 4-14. Total costs for 2-inch HMA overlay, CIR-E with HMA overlay, and CIR-F with HMA overlay (Offenbacker et al., 2021).

Figure 4-15 (a) and (b) show the LCCA of the pavement with two rehabilitation techniques at 2000 AADTT and 4000 AADTT, respectively. It can be found that CIR technique would result in 15.9% cost saving in the whole life cycle (70 years) compared with the conventional HMA milling & filling. However, when the traffic volume increases to 4000 AADTT, the CIR technique would consume 30.8% more cost than the conventional milling & filling. Hence, CIR techniques would contribute to the cost saving and can be applied to the pavement with low traffic volume.



(a)



(b)

Figure 4-15. LCCA of pavement with two rehabilitation techniques under different traffic load: (a) 2000 AADTT; (b) 4000 AADTT.

4.3 Temperature effect on the performance and binder mobilization of HIR mixes

4.3.1 Performance evaluation

4.3.1.1 AMPT test results

Figure 4-16 plots the dynamic modulus master curves of the HIR mixtures heated at three temperatures in a log scale. It can be observed that mixtures compacted at 130 ° C show the lowest dynamic modulus at all frequency stages, followed by the mixtures conditioned at 120 ° C and 110 ° C. Previous research revealed that the dynamic modulus test is sensitive to the small change of the asphalt binder content (Huang et al., 2008). Therefore, for the 100% HIR mix, the high heating temperature may activate more aged asphalt and accelerate the diffusion of the additives into the RAP binder. A more effective asphalt binder would be activated to soften the mixtures, making the specimens easier to deform during the dynamic modulus test.

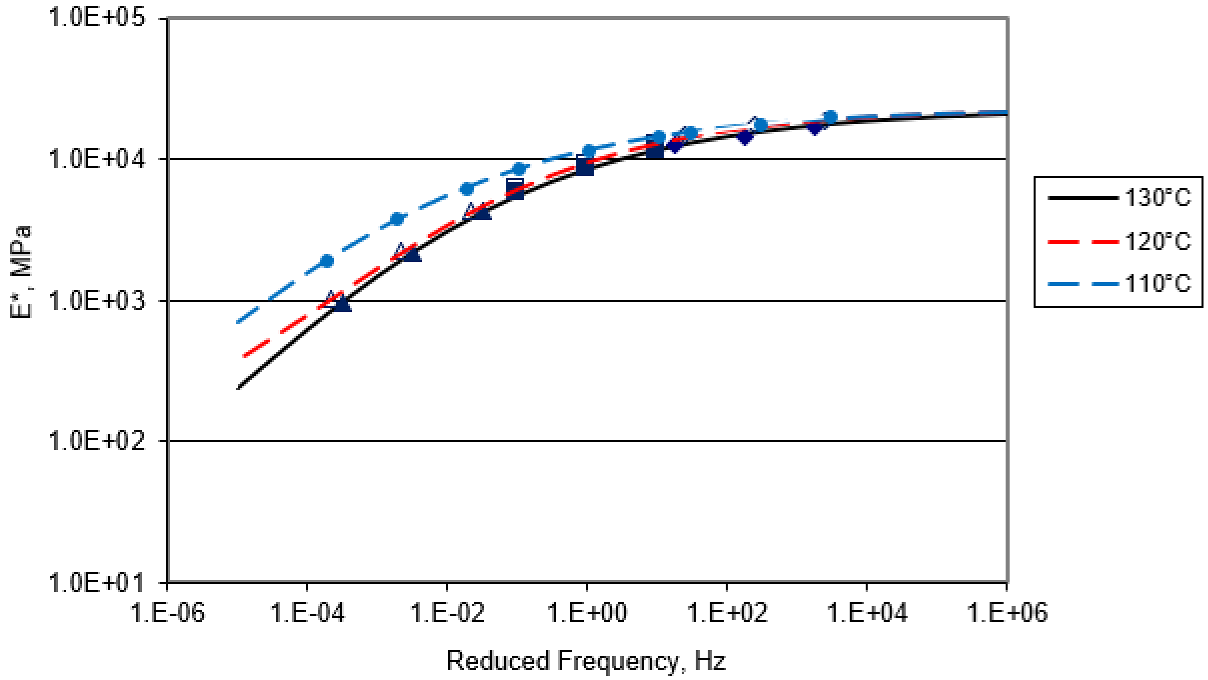


Figure 4-16. Dynamic modulus master curve of the HIR mixtures at the reference temperature of 20 ° C.

A flow number test was also conducted to explore the high-temperature performances of the HIR mixtures compacted at three temperatures. The test was performed at 54.4 ° C, which was the average highest temperature in Tennessee. **Figure 4-17** shows that the flow numbers of mixes conditioned at 110 ° C, 120 ° C, and 130 ° C are 2360, 2049, and 1504, respectively. Mixtures compacted at a higher temperature will compromise the rutting resistance, indicated by a lower flow number. Higher compaction temperature may also facilitate the curing and breaking of emulsified asphalt. More additives were possible to diffuse into the aged binder at high temperatures. Therefore, the activated RAP asphalt was able to weaken the rutting performances of the mixtures. The rutting resistance of the mixtures will be evaluated further by the APA rutting test.

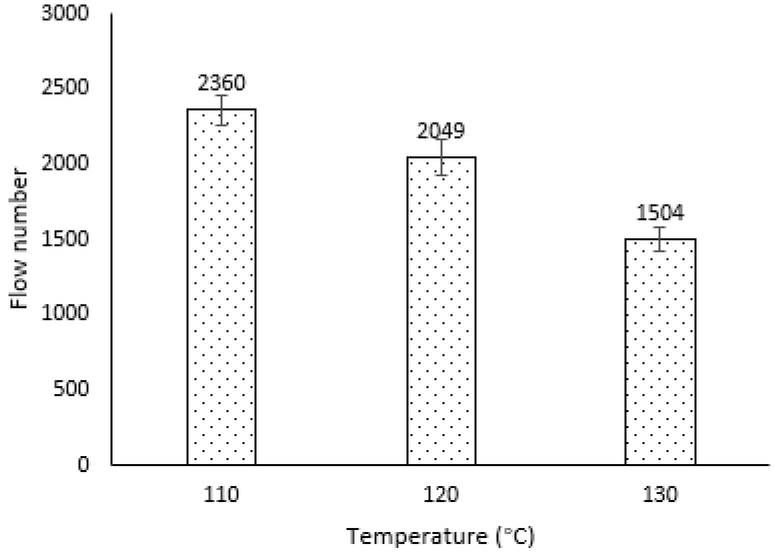


Figure 4-17. Flow number results of the HIR mix.

4.3.1.2 APA rutting test results.

Figure 4-18 presents the APA rutting test results. As expected, the HIR mixtures heated at high temperatures are more susceptible to rutting, which is indicated by a larger rut depth value. The rut depths of the mixtures are in accordance with the dynamic modulus and flow number results.

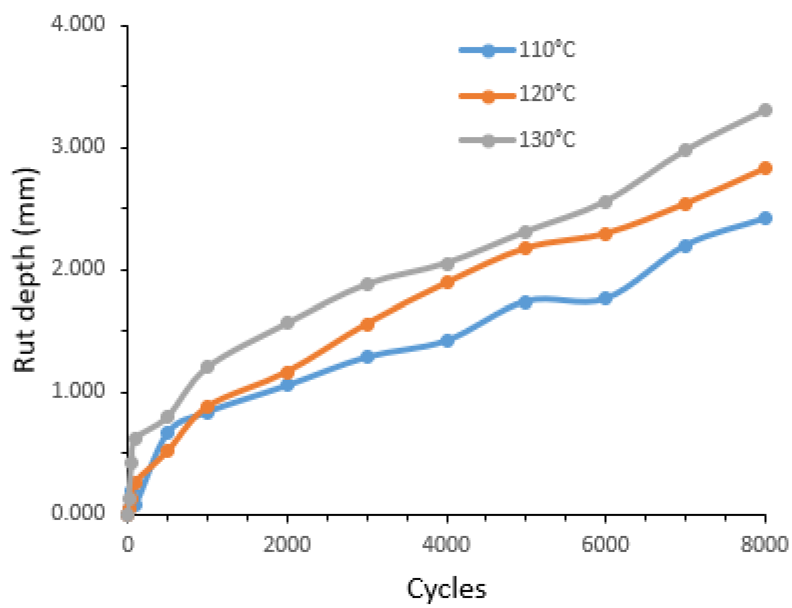


Figure 4-18. APA rutting test results.

4.3.1.3 Superpave IDT DSCE$_f$ test results

The dissipated creep strain energy (DCSE$_f$) exhibits a threshold for crack initiation and propagation. The increase of DCSE$_f$ results in improving the cracking resistance of the asphalt mixtures (Zhang et al., 2001a). **Figure 4-19** presents the DCSE$_f$ results of the HIR mix at different heating temperatures. The higher mixing temperature resulted in a larger DCSE$_f$ value of the HIR mix, indicating that the cracking resistance of the mixtures was improved. Other Superpave IDT test results were provided in the appendices. Based on the cracking behavior of HIR mixes, the higher mixing temperature would improve the diffusion of the additives to the aged binder and facilitate the RAP binder mobilization. The binder mobilized under different temperatures was validated through the GPC test in the next section.

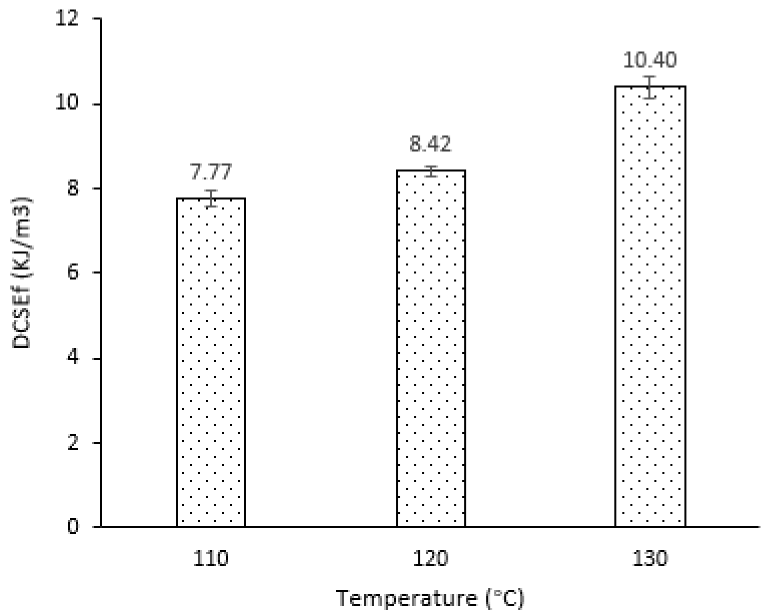


Figure 4-19. DCSE$_f$ results of HIR mixes at different temperatures.

4.3.2 Binder mobilization

Based on the performance test results, high mixing temperatures are likely to accelerate the diffusion of the additives into the RAP binders. The mobilized RAP binders increase the amount of effective asphalt content to achieve better-coated asphalt mixtures. In this section, the staged extraction test was conducted to validate the mobilization of RAP binders and the blending mechanism of the binder blends. The outer and inner layers of the HIR mix blended at three temperatures were separated by being submerged into TCE for 30 seconds and then 3 mins, respectively. The differences of the LMS fractions between the inner and outer layers were obtained by the GPC test.

Table 4-6 shows the results of staged extraction and GPC test results. The emulsified asphalt and RAP binder LMS fractions were 20.6% and 33.8%, respectively. Regardless of the temperature, all the outer and inner layers had a value of LMS fractions between them with a lower value for the outer layer, indicating that the outer layer contained more additives, whereas the inner layer had more RAP binders. The differences of LMS fractions of inner and outer layers reflected the capability of the emulsified asphalt diffusing into the RAP binder. The smaller value indicated that the blending is more efficient, and a larger proportion of RAP binder was mobilized. It can be observed that the increase of mixing temperature will decrease the difference of LMS fractions, which means that high temperatures could facilitate the diffusion of additives into the aged binder and improve the blending efficiency of the binders. As a result, more RAP binders would be activated to coat the aggregates at higher temperatures. Therefore, the cracking and moisture resistance performances of the asphalt mixture could be improved, but the higher content of activated asphalt would cause some rutting issues.

Table 4-6. Stage extraction and GPC test results.

Material types	*Mixing temperature (°C)*	*Large molecular size fraction (%)*		
		Outer layer	Inner layer	Difference
HIR mix	110	25.8	31.7	5.9
HIR mix	120	26.3	29.3	3.0
HIR mix	130	27.1	28.4	1.4
Emulsified asphalt	--	20.6		--
RAP	--	33.8		--

4.4 Quantifying and maximizing the binder mobilization and recycling efficiency of HIR mixes

4.4.1 Ignition oven test results

4.4.1.1 Mobilized RAP content through "Dry" blending

Figure 4-20 shows the effective mobilized RAP content for "dry" blending at different temperatures. In this case, the RAP binder was activated and mobilized mainly with the assistance of temperatures. Three mass proportions of virgin and RAP aggregates were considered to determine the amount of raw material in further tests. It is evident that temperature performs a significant role in mobilizing the RAP binder to coat aggregates. At lower temperatures (110° C), it is difficult to diffuse the RAP binder, while at higher temperatures up to 190° C, over 25% of the RAP binder can be mobilized under since the higher temperature can reduce the viscosity of the stiff RAP binder. Furthermore, a larger proportion of RAP in the mixtures present less effective mobilized RAP content because the mobilized RAP binder has a limited virgin aggregate surface to coat and adhere to the surface of RAP materials instead. Such phenomena are becoming more explicit at higher temperatures. Hence, more virgin aggregates should be incorporated to apply adequate area for mobilized RAP binder. There exist few differences in the effective mobilized RAP content between the mass ratio of virgin aggregates and RAP aggregates =1:1 and 2:1. Finally, 1000g virgin aggregates and 1000g RAP aggregates were adopted for further tests in this study considering the similar gradation of the HIR mix in the field.

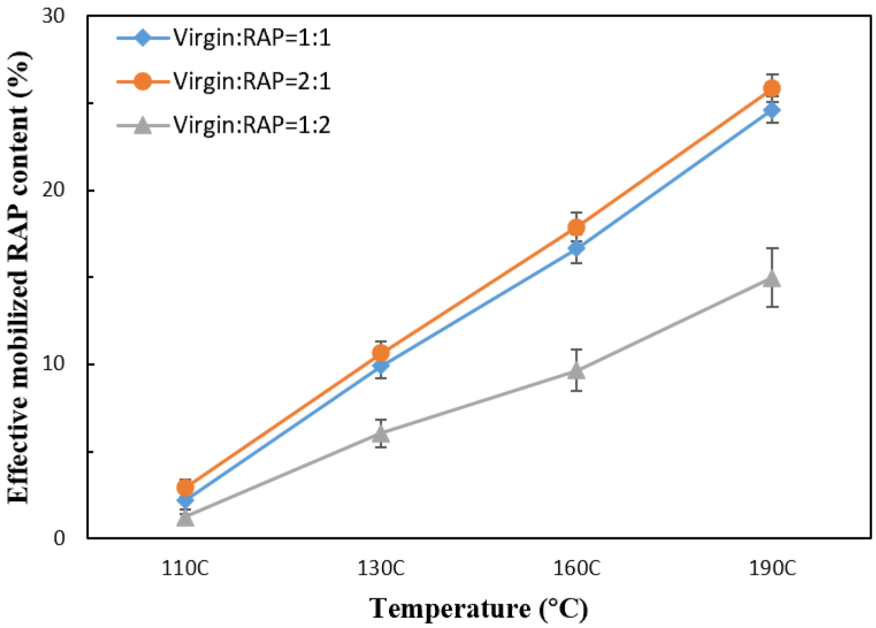


Figure 4-20. The effective mobilized RAP content for "dry" blending at different temperatures.

4.4.1.2 Mobilized RAP content with different additives

Different additives, including rejuvenator and asphalt emulsion, were also used to evaluate the changes of mobilized RAP content at different temperatures. Virgin and RAP aggregates were heated in the oven first, then sprayed the additives. The rejuvenator and asphalt emulsion dosage was determined based on the field application, which was 1.0% and 2.6% mass of the RAP materials, respectively. As shown in **Figure 4-21**, compared to the "dry" blending, both rejuvenator and asphalt emulsion could significantly improve the mobilization of the RAP binder. At lower temperatures, asphalt emulsion performed slightly higher mobilization of RAP binder than rejuvenator; however, rejuvenator tended to work more efficiently at higher temperatures since asphalt emulsion will experience more rapid breakage and evaporation at high temperatures, which compromise the blending efficiency.

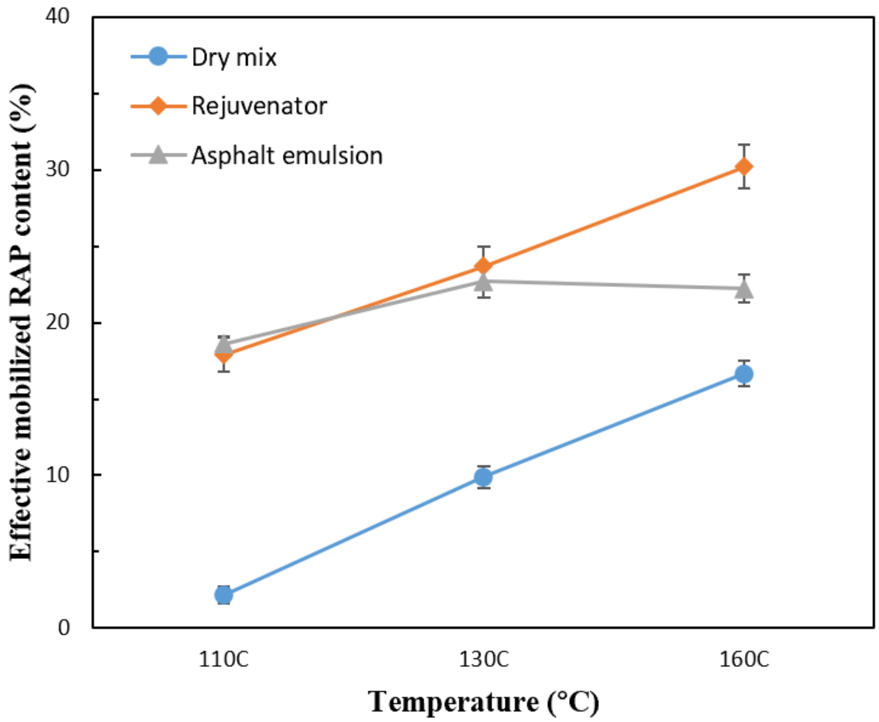


Figure 4-21. Effective mobilized RAP content with different additives.

4.4.1.3 Mobilized RAP content with different emulsion dosages

More RAP binder is expected to be mobilized to coat aggregates by incorporating a larger proportion of asphalt emulsion in HIR. Different contents of asphalt emulsion were also considered to explore whether the amount of asphalt emulsion can affect the mobilized RAP content and whether the ignition oven method is suitable for recognizing these differences. **Figure 4-22** compares the effective mobilized RAP content with various content of asphalt emulsion under three oven heating temperatures. It is clearly shown that higher dosages of asphalt emulsion will activate more RAP binder. Three dosages exhibit similar trends at three temperatures, indicating that an optimum temperature for RAP mobilization with asphalt emulsion during HIR procedures lies around 130 ° C since excessive heating is prone to evaporate the water and break the asphalt emulsion. Furthermore, it can be noticed that the effective mobilized RAP content for 1.3% asphalt emulsion at 130° C is close to the 2.6% asphalt emulsion at 110° C. Therefore, a balanced blending efficiency exists regarding the temperature and amount of asphalt emulsion in 100% RAP during HIR mix design. To improve the mobilization efficiency of RAP, energy-saving pavement rehabilitation can be achieved by adding more content asphalt emulsion without increasing temperatures.

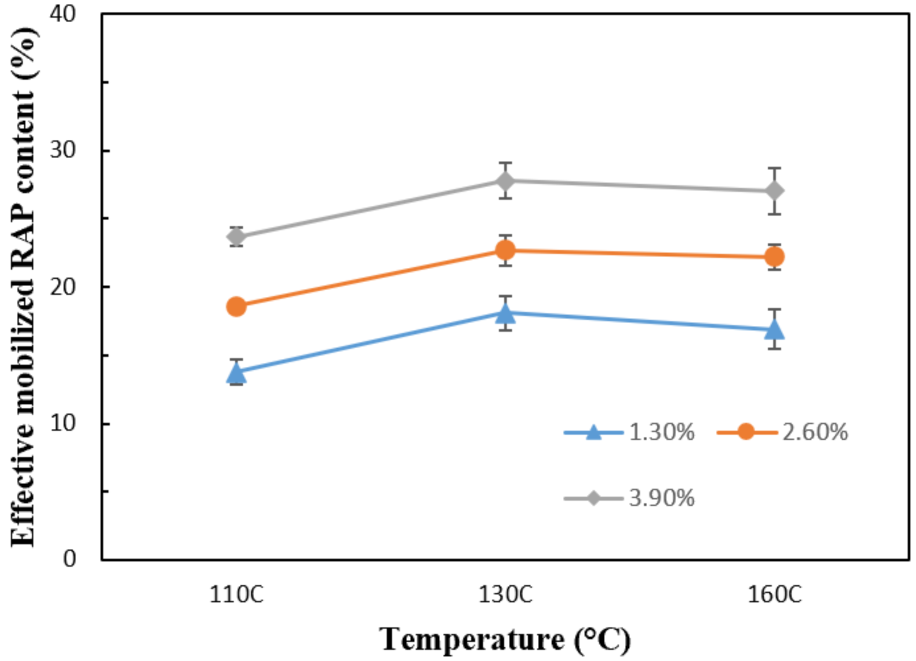


Figure 4-22. Effective mobilized RAP content with various dosages of asphalt emulsion under different oven heating temperatures.

4.4.1.4 Mobilized RAP content with different heating methods

In field HIR projects, the heating units provide fire heating to soften and activate RAP, which is different from the oven heating in the lab. To better understand the blending efficiency scenario of 100% RAP with the HIR technique, torch blow tests were developed to simulate the fire heating in the field. **Figure 4-23** presents the effective mobilized content of the HIR mix with 2.6% asphalt emulsion through both oven and fire heating at different temperatures. As shown in Figure 15, HIR mix with fire heating has higher effective mobilized RAP content than the ones heated with the oven. Similar to the oven heating, fire heating gives a larger increase of mobilized RAP content from 110° C to 130° C, but a smooth increase (or slight decrease) from 130° C to 160° C due to the breakage of asphalt emulsion. Therefore, on the one hand, the fire heating method can increase the temperature rapidly and activate more RAP binder within a short period, while on the other hand, it would break the asphalt emulsion quicker.

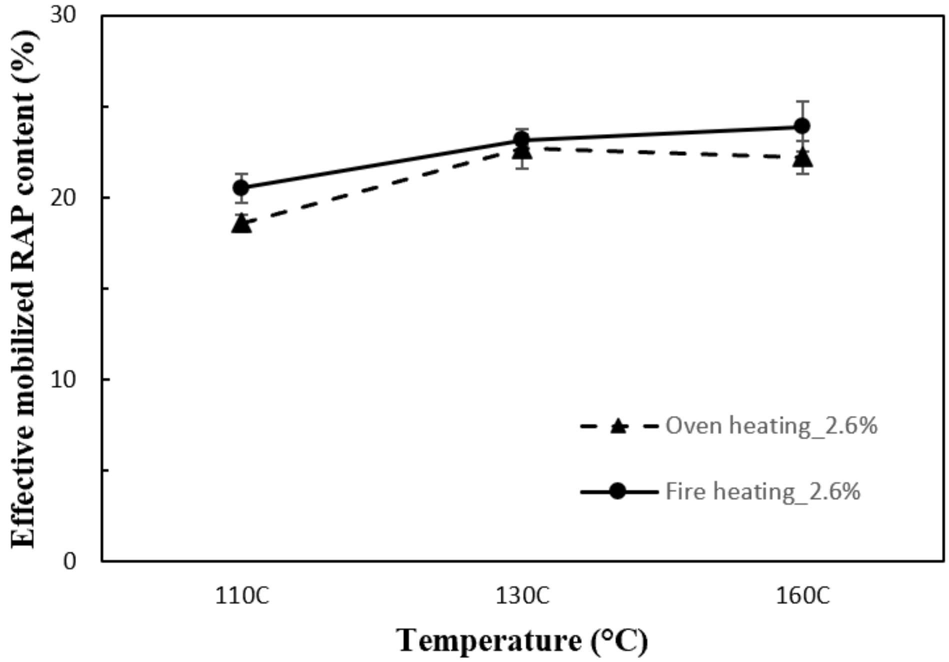


Figure 4-23. The effective mobilized content of the HIR mix with 2.6% asphalt emulsion through both oven and fire heating at different temperatures.

4.4.1.5 Mobilized RAP content between HIR and HMA

The above results confirm that an ignition oven method is an acceptable approach to quantifying the effective mobilized RAP content of the HIR mix, which directly focused on the mass loss variations of the RAP binder. It is expected that such a method can be applied in HMA (PG 64-22 as a virgin binder). **Figure 4-24** compares the effective mobilized RAP content with asphalt emulsion and HMA at 160° C. Mixtures with 3.9% virgin asphalt present the highest effective mobilized RAP content over 50%, over two times that of mixtures with asphalt emulsion. The virgin asphalt will not break or evaporate at high temperatures, resulting in a continuous ability to diffuse into the RAP binder and assist the mobilization afterward. The amount of asphalt used in HMA should be adjusted carefully since it can significantly influence the blending efficiency and the effective mobilized RAP content.

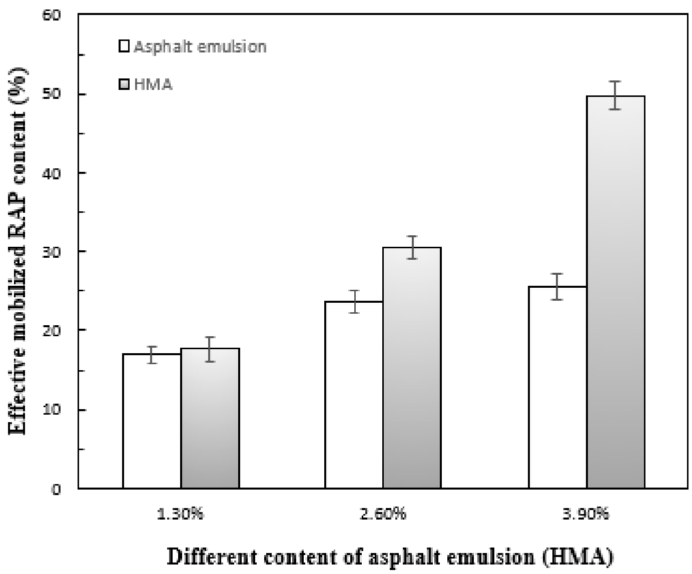


Figure 4-24. The effective mobilized RAP content with asphalt emulsion and HMA at 160° C.

4.4.2 FTIR test results

4.4.2.1 Mobilized RAP content quantification through FTIR

The ignition oven method offers an effective approach to quantify the effective mobilized RAP content, which aims at measuring the ignition loss differences of RAP materials. Besides, FTIR is a commonly used method of quantifying the degree of blending based on the extracted binder blends of the virgin aggregates. **Figure 4-25** shows that the 3D contour of CI and effective mobilized RAP content was constructed considering temperature and asphalt emulsion content. It can be seen that the effective mobilized RAP content is positively correlated with both asphalt emulsion content and temperatures, which matches the pattern obtained by the ignition oven method. Mixtures with the addition of 3.9% asphalt emulsion at 160° C exhibits the highest effective mobilized RAP content, close to 50%.

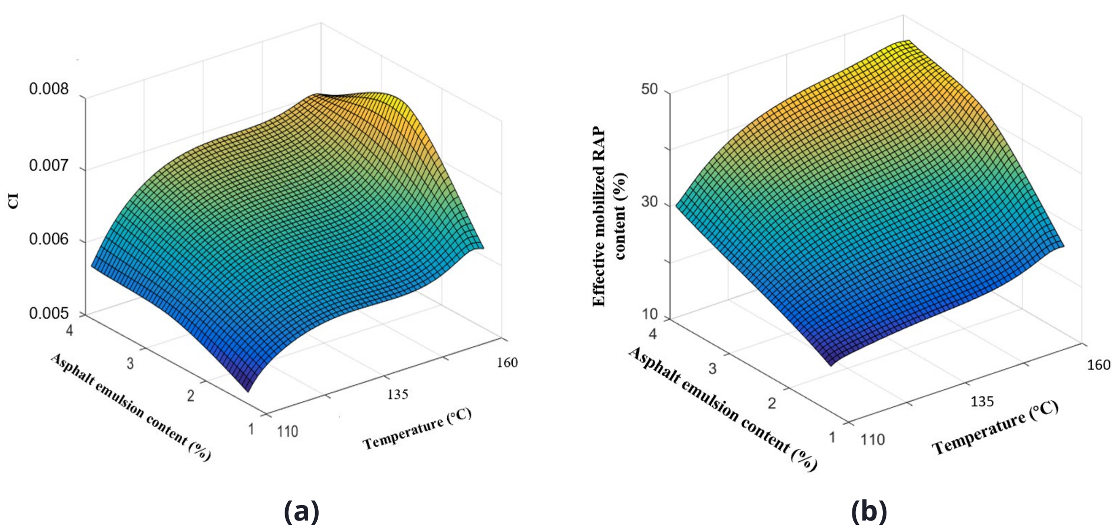


Figure 4-25. The contour of the CI and effective mobilized RAP content under different conditions using the FTIR method, (a) CI, (b) Effective mobilized RAP content (%).

4.4.2.2 Correlation analysis between ignition oven method and FTIR method

Figure 4-26 summarized the linear correlation between the mobilized RAP content results obtained by FTIR and ignition oven methods at different testing conditions. A linear relationship was discovered between the two testing methods, and the trendline was constructed with a R^2 = 0.8161, indicating a relatively strong fitness of the data. Therefore, an accurate mobilized RAP content can be estimated through the ignition oven method under the circumstance where the FTIR method is hard to be applied.

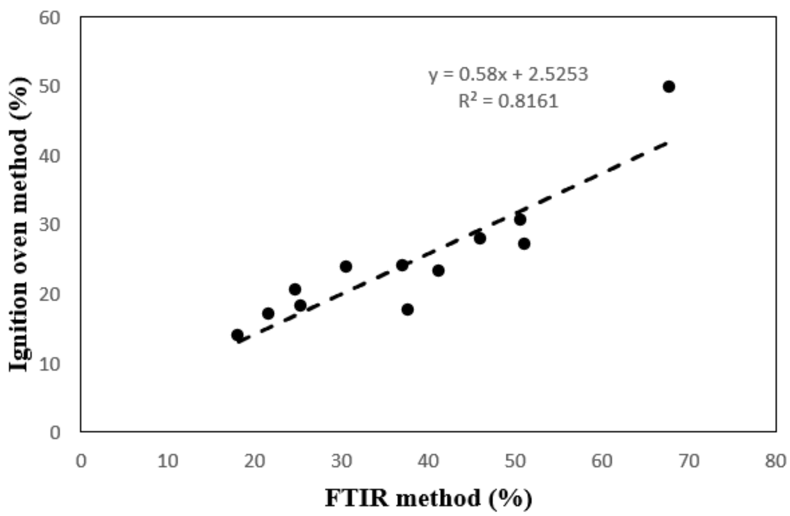


Figure 4-26. Relationship between the FTIR method results and ignition oven method results.

4.5 Influence of mobilized RAP content on the effective binder quality and performance of HIR mixes

4.5.1 Mobilized RAP content and effective asphalt content quantification

The mobilized RAP content of HIR mixes was determined at three mixing temperatures and different dosages of recycling agents using the ignition oven method. **Table 4-7** shows the results of mobilized RAP content and the RAP binder ratio for HIR mixes with different dosages of recycling agents at three mixing temperatures. The effective asphalt content consists of the effective mobilized RAP content and the asphalt content in the recycling agent or virgin binder. The proportion of the virgin and RAP binder in the effective binder blends might affect the quality of the effective binder blends and hence the HIR mixes' performance. As shown in **Table 4-7**, it can be seen that the increase of temperature and recycling agent could mobilize more RAP binder, contributing to a larger value of effective asphalt content.

Regarding the HIR mixes with the same dosages of recycling agents, a higher temperature would increase the effective RAP binder ratio in the effective binder blends. However, considering the HIR mixes blended at the same temperature, a higher dosage of recycling agent would lower the RAP binder ratio in the effective binder blends. Furthermore, compared with the HIR mixes with the recycling agents, the addition of virgin binder would improve the mobilization of the RAP binder but also lower the RAP binder ratio in the effective binder blends since no water or additives appear in the virgin binder.

Table 4-7. Effective mobilized RAP content for HIR mixes.

Mixture label	***Temperature (° C)***	***Dosages (%)***	***Mobilized RAP content (%)***	***Effective asphalt content (%)***	***Effective RAP binder ratio***
RA11013	110	1.3	1.14	1.79	0.64
RA11026	110	2.6	1.54	2.84	0.54
RA11039	110	3.9	1.96	3.91	0.50
RA13013	130	1.3	1.50	2.15	0.70
RA13026	130	2.6	1.88	3.18	0.59
RA13039	130	3.9	2.42	4.37	0.55
V16013	160	1.3	1.40	2.76	0.52
V16026	160	2.6	2.52	5.12	0.49
V16039	160	3.9	4.12	8.02	0.51

4.5.2 Binder quality characterization

4.5.2.1 DSR temperature sweep test results

The effective binder blends were manually prepared for binder quality characterization at different temperatures. **Figure 4-27** shows the effective binder blends' DSR temperature sweep test results. The rutting parameter G*/sinδ was used to characterize the rheological properties of the effective binder blends. As shown in **Figure 4-27**, the effective binder blends with virgin asphalt and mobilized RAP binder present the highest rutting parameter, indicating the stiffer binder blends. The addition of different dosages of the virgin binder would not vary the quality of the effective binder blends. Regarding the incorporation of recycling agents, adding more dosages could lower the rutting parameters of the binder blends. At the same time, the increase of temperature could facilitate the mobilization of the RAP binders, which increased the rutting parameter of the effective binder blends. The DSR temperature sweep test results agree with the effective RAP binder ratio in **Table 4-7**, reflecting that the preparation procedures are representative of the quality of the effective binder blends.

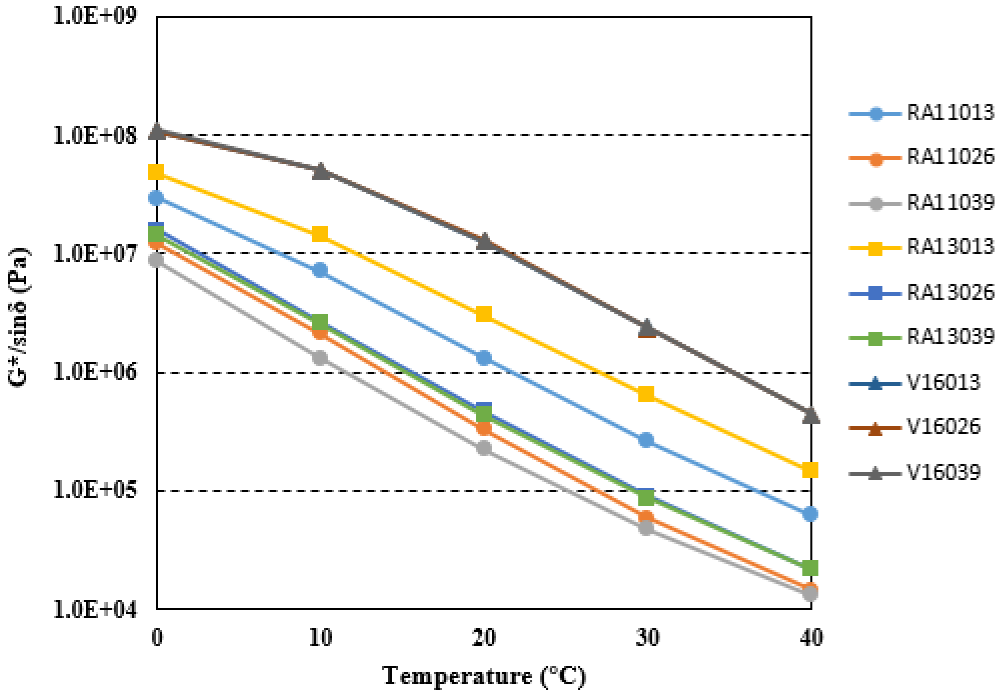


Figure 4-27. Temperature sweep test results of the effective binder blends.

4.5.2.2 Surface free energy of the effective binder blends

Table 4-8 displays the surface energy of the effective binder blends in different HIR mixes. For the HIR mixes with virgin asphalt, the effective RAP binder ratios in the effective binder blends are similar among V16013, V16026, and V16039. Hence, one type of binder blend with a RAP binder ratio of 0.5 was prepared for contact angle and surface energy testing. From **Table 4-8**, more RAP binder ratios in the effective binder blends would lower surface free energy. It can also be found that the total surface free energy of the effective binder blends increases with higher additive dosages, indicating a higher cohesion of the binder blends and better adhesion between the binder blends and aggregates. Increasing the mixing temperature would lower the surface energy of effective binder blends at the same additive dosages, indicated by a larger RAP binder ratio. However, the high RAP mobilization also increased the effective asphalt content, which might cause the different performances of the HIR mixes. To further investigate which factors dominate the mixture performances, performance tests including AMPT tests and Ideal CT test were adopted to evaluate the viscoelastic behaviors and cracking performances of the HIR mixes.

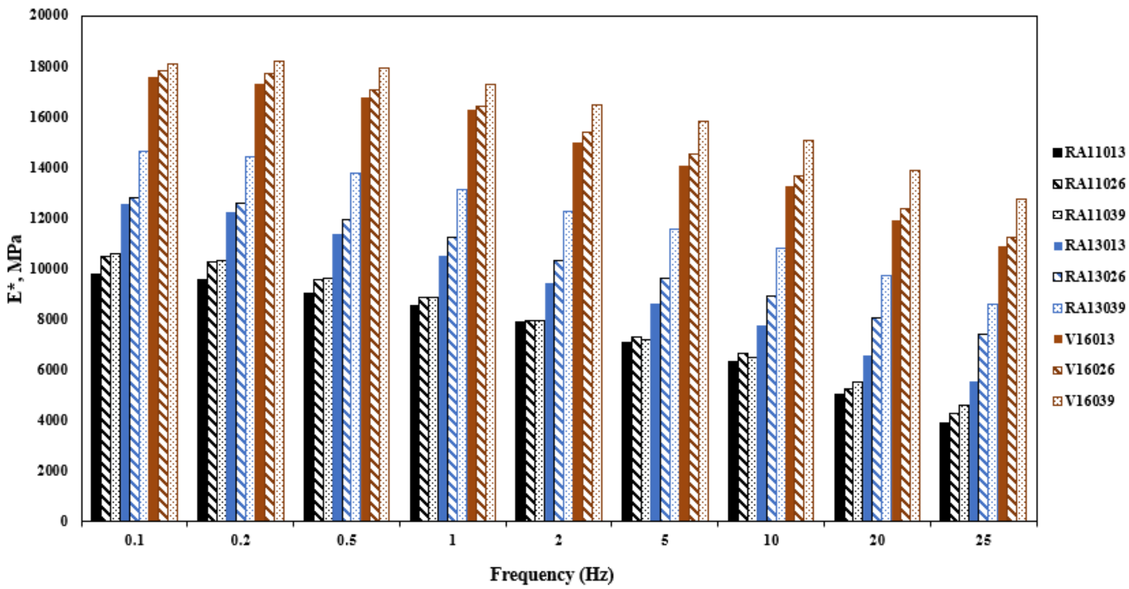
Table 4-8. Surface energy results of the effective binder blends in different HIR mixes.

Effective binder blends	$\Gamma_L^{LW}\,(mJ/m^2)$	$\Gamma_L^{+}\,(mJ/m^2)$	$\Gamma_L^{-}\,(mJ/m^2)$	$\Gamma^{AB}\,(mJ/m^2)$	$\Gamma_{Total}\,(mJ/m^2)$	*SD*
RA11013	16.9	0.53	3.8	2.84	19.74	0.12
RA11026	18.01	0.43	4.3	2.72	20.73	0.16
RA11039	19.56	0.21	4.7	1.99	21.55	0.03
RA13013	15.9	0.5	3.92	2.80	18.70	0.18
RA13026	17.23	0.45	4.12	2.72	19.95	0.25
RA13039	18.3	0.32	4.65	2.44	20.74	0.04
V160	20.12	0.26	4.95	2.27	22.39	0.13

4.5.3 Mixture performance characterization

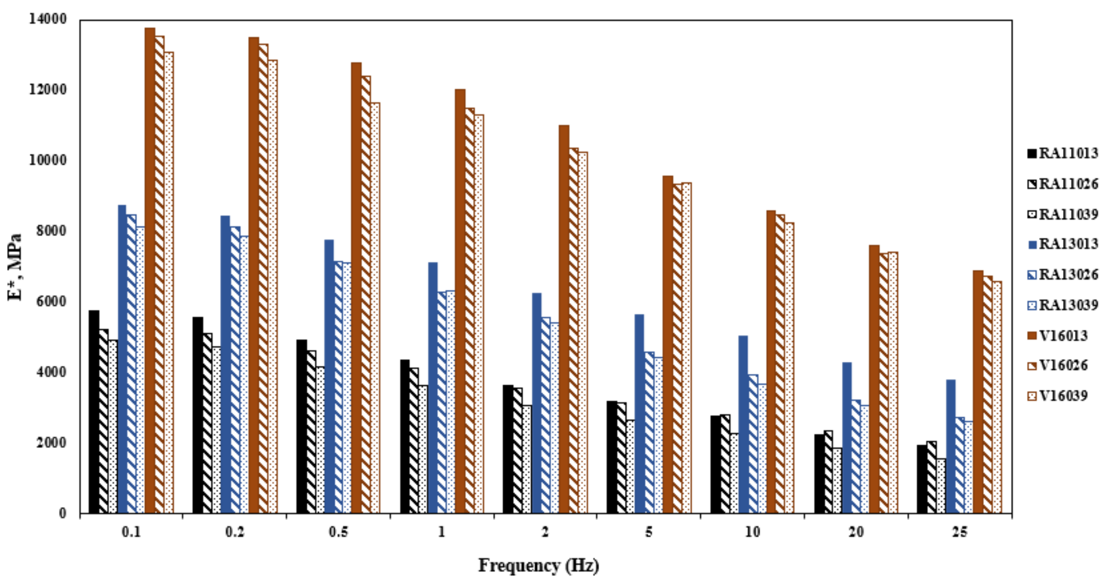
4.5.3.1 AMPT test results

Figure 4-28 (a), (b), and (c) present the dynamic modulus results of the HIR mixes conditioned at 4° C, 20° C, and 40° C, respectively. As for the use of recycling agents, it can be found that more dosages would increase the dynamic modulus of the HIR mixes at low temperatures (4° C). According to DSR results, high dosages of recycling agents would lower the effective binder ratio and hence the modulus of the effective binder blends, which might result in a lower dynamic modulus of the asphalt mixtures. However, the mobilized RAP content also significantly increased the effective asphalt content in the HIR mixes. The surface-free energy of effective binder blends also increased with a higher dosage of recycling agents. At low testing temperatures, the higher asphalt content and surface-free energy of the effective binder blends would increase the interaction area and the coating between asphalt and aggregates, which could strengthen the asphalt-aggregates system. Furthermore, increasing the mixing temperature could facilitate the RAP binder mobilization, leading to a higher effective RAP binder ratio and effective asphalt content. The higher effective asphalt content and the stiffness of effective binder blends could contribute to an increased dynamic modulus of HIR mixes. Similarly, HIR mixes with virgin binder showed the highest dynamic modulus. As shown in **Figure 4-28** (b) and (c), when the testing temperature increases, the dynamic modulus of the HIR mixes was reduced with the higher dosages of additives, which was different from the trend at low temperatures (4 ° C). The elevated temperatures could soften the effective binder blends and weaken the rutting resistance of the asphalt mixtures. The high asphalt content would cause more asphalt migration and also a lower dynamic modulus of asphalt mixtures (Daniel and Lachance, 2005). Hence, both effective binder quality and effective asphalt content could affect the dynamic modulus of HIR mixes. More effective asphalt content could improve the asphalt-aggregates coating system at low temperatures but mitigate the rutting resistance of HIR mixes at high temperatures.



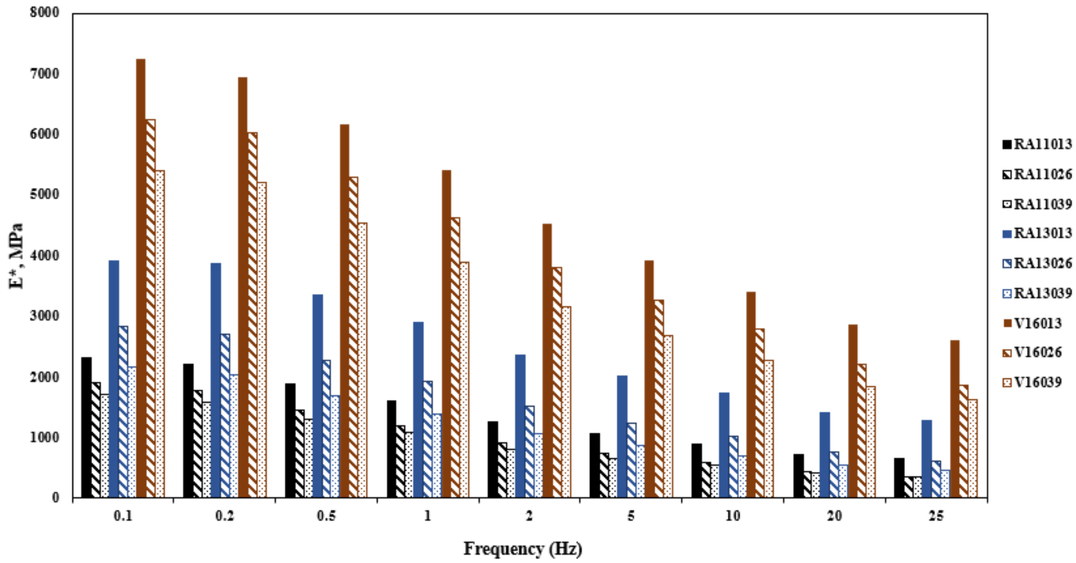
E*, MPa
Frequency (Hz)
RA11013
RA11026
RA11039
RA13013
RA13026
RA13039
V16013
V16026
V16039

(a)



E*, MPa
Frequency (Hz)
RA11013
RA11026
RA11039
RA13013
RA13026
RA13039
V16013
V16026
V16039

(b)



(c)

Figure 4-28 Dynamic modulus test results of HIR mixes at different temperatures; (a) 4 ° C; (b) 20° C; (c) 40 ° C.

Figure 4-29 shows the flow number test results of the HIR mixes. Tests were conducted at 54.4 ° C to simulate the high-temperature performances. Flow number exhibited a similar pattern compared to the dynamic modulus test results at high temperatures, which further demonstrated that the binder quality played a significant role in the stiffness and high-temperature performances of the HIR mixes. In terms of HIR mixes with the virgin asphalt at the same RAP binder ratios, a higher asphalt content could cause more rutting potential. It is necessary to point out that the HIR mixes present relatively high flow numbers than the traditional HMA mixes based on literature (Huang et al., 2008; Polaczyk et al., 2021). As mentioned previously, HIR mixes contain 100% RAP, which increases the stiffness of the asphalt mixtures. Hence, the HIR mixes are not susceptible to rutting issues.

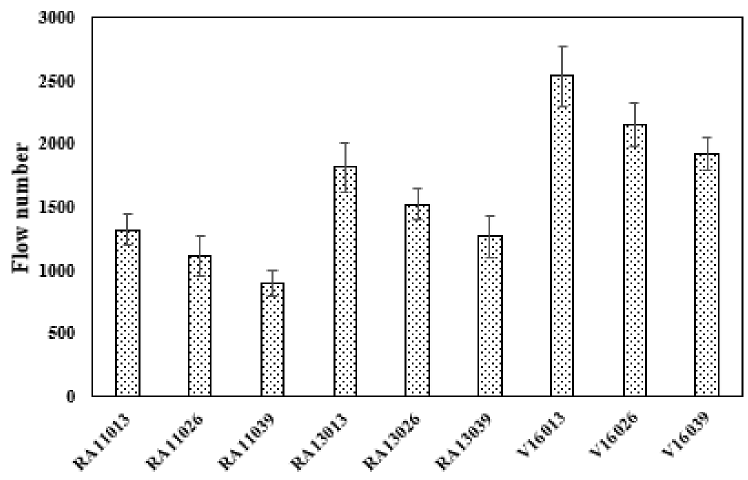


Figure 4-29. Flow number test results of HIR mix at 54.4 ° C.

4.5.3.2 Ideal-CT test results

Cracking is considered the main concern for asphalt mixtures containing a high percentage of RAP (Huang et al., 2004). The 100% use of RAP in HIR mixes tends to exaggerate the cracking issue. **Figure 4-30** shows the Ideal-CT test results of different HIR mixes. CT_{index} was obtained from the average of three replicates. At a certain mixing temperature, the use of more additives or virgin asphalt facilitated the mobilization of the RAP binder and improved the cracking resistance of the HIR mixes, indicated by a larger value of $CT_{index.}$ Similarly, the increase in temperature also resulted in a higher CTindex of the HIR mixes with a certain additive dosage. Regarding the mobilized RAP content in Table 4-7 and the binder quality characterization, more RAP binder mobilization would increase the modulus and decrease the surface energy of the effective binder blends, which might lower the cracking resistance of the HIR mixes. Specifically, the effective RAP binder ratio for RA13026 is higher than RA11026, reflecting that the effective binder blend of RA13026 is stiffer than RA11026. Theoretically, the stiffer binder blends would cause more cracking issues in the asphalt mixtures. However, the CT_{index} of RA13026 is higher than RA11026, indicating a higher cracking resistance. The aforementioned results could be attributed to the effective asphalt content in HIR mixes. Specifically, the mobilized RAP binder would also contribute to an increase in the effective asphalt content. As shown in Table 4-7, the effective asphalt content for RA13026 is higher than RA13013. More effective asphalt content could improve the asphalt-aggregates coating system, the ductility of the asphalt mixture, and the cracking resistance of HIR mixes. Hence, under such circumstances, the effective asphalt content played a more important role in the cracking behavior of the HIR mixes than the effective RAP binder ratio. The use of virgin binder would significantly improve the binder quality, and the cracking resistance of HIR mixes was notably improved when the virgin binder was adequate during mixing.

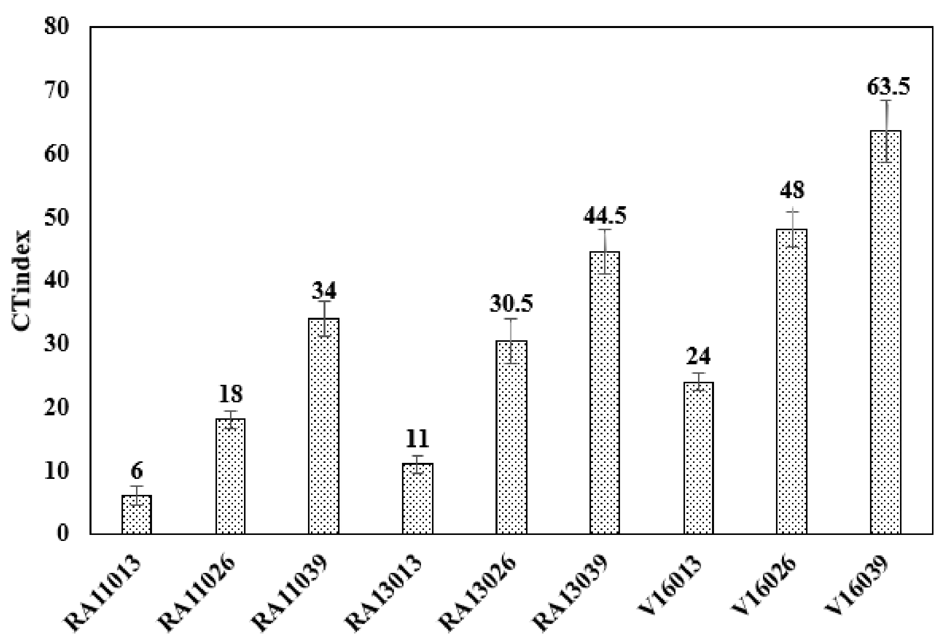


Figure 4-30. Ideal CT test results of different HIR mixes.

Chapter 5 Conclusion and Recommendations

In this research project, the loose HIR and CIR mixes were collected for the performance evaluation and comparison with the HMA mixes. With the performance testing results, ME software was adopted to predict the pavement life with HIR, CIR rehabilitation, and conventional HMA milling & filling techniques. LCCA was further conducted at different traffic levels to evaluate whether it was cost-effective to apply the in-place recycling techniques to the pavement rehabilitation. Additionally, the blending mechanism and recycling efficiency with HIR techniques were investigated, and a new method of quantifying the mobilized RAP binder content was developed. Finally, approaches to improve the binder mobilization and performances of HIR mixes were provided. Based on the test results, the following conclusions can be summarized:

(1) HIR mixes showed acceptable rutting and moisture resistance. Cracking resistance is the main issue that HIR mixes would encounter. HMA has a stronger coating between asphalt and aggregates than the HIR mix even with the lower asphalt binder content, indicating higher IDT strength and $DCSE_f$.

(2) The $DCSE_f$ of field cores reflected that the cracking resistance of the existing pavement surface before HIR rehabilitation decreased more than 40%. HIR technique showed consistent construction qualities as lab mixes, which could restore the cracking resistance of existing pavement.

(3) ME prediction results indicated that pavement after HIR surface treatment would yield a larger value of roughness index and encounter severe fatigue cracking as well as low-temperature cracking issues. LCCA results reflected that HIR surface rehabilitation was cost-effective only for low-volume roads. It would save over 50% initial cost and 20% life cycle cost than conventional HMA milling & filling.

(4) Compared to the HMA (BM2 mix) for the binder layer, CIR mixes tended to encounter rutting and cracking problems. A sufficient curing period would improve the stiffness, cracking resistance, and moisture resistance of the CIR mixes' stiffness, cracking resistance, and moisture resistance.

(5) ME prediction results indicated that pavement after CIR rehabilitation would increase the IRI value and have severe cracking issues. LCCA results showed that the CIR technique was also cost-effective for the low-volume road. The high-level traffic conditions would damage the CIR rehabilitated pavement more quickly.

(6) A higher mixing temperature of HIR mixes could improve the diffusion of additives into the RAP binder and facilitate the RAP mobilization. The higher mobilized asphalt content could improve the cracking and moisture resistance of the HIR mixes.

(7) The ignition oven method was a valid and convenient approach to quantify the mobilized RAP content in HIR mixes by looking at the mass loss of the RAP binder. The ignition oven method presented comparable results with the FTIR method for mobilized RAP content quantification.

(8) The mobilized RAP content represented the recycling efficiency of the HIR mixes. Based on this research, the mobilized RAP content for HIR mixes ranged from 20% to 30%. An appropriate

increase of temperature and additive dosages could improve the RAP binder mobilization and the recycling efficiency of the HIR mixes.

(9) An overheat of HIR mixes might cause the breakage of the recycling agent and restrict the rejuvenation effect, which lowers the mobilized RAP content. 130° C was recommended as the mixing temperature during field construction.

(10) The mobilized RAP content for 100% RAP could reach 60% when the virgin asphalt was used at a mixing temperature of 160 ° C.

(11) The mobilized RAP binder content could alter the qualities of the effective binder blends and the effective asphalt content, which is highly related to the performance of the HIR mixes.

(12) The higher proportion of RAP binder in the effective binder blends would increase the stiffness and lower the surface free energy of the binder blends. The binder qualities, including the rheological properties and surface free energy, are highly related to the dynamic modulus of the HIR mixes.

(13) The effective asphalt content was considered a significant factor that dominated the cracking behavior of the HIR mixes. Even though the higher mobilized RAP binder content theoretically resulted in more brittle binder blends and cracking potential, the mobilized RAP content would also increase the effective asphalt content, which improved the aggregates coating and the cracking resistance of the HIR mixes.

Based on the conclusions, the recommendations are provided to TDOT as follows:

(1) Increase the mixing and compaction temperature to 130° C (current is 110° C) for HIR construction, since higher temperature would improve the binder mobilization and also the cracking resistance of the HIR mixes.

(2) Increase the dosage of using asphalt emulsion and especially focus on the cracking resistance during HIR mix design. Based on the experimental results, HIR mixes would mainly encounter cracking issues. Hence, more dosages of asphalt emulsion could be added to improve the cracking resistance of HIR mixes. Additionally, a performance-based mix design (e.g. balance mix design) is recommended to focus more on the cracking resistance of the HIR mixes.

(3) If possible, increase the mix temperature to 150 ~160° C and use virgin asphalt during HIR construction, which could significantly improve RAP binder mobilization and the cracking resistance of HIR mixes.

(4) Apply the CIR technique only in a low-volume road and ensure the curing period after paving. For the CIR mix design, increase cement usage to improve the strength of the base layer.

(5) Conduct pavement monitoring for HIR and CIR sections during the service period, which could provide valid field data for the pavement performance evaluation for in-place recycling techniques.

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Appendices

Appendix A: DOT survey results

Questionnaire preamble

The University of Tennessee (UT) is performing a research project entitled “100% Recycled Mixtures Using Cold In-Place Recycling (CIR), Hot In-Place Recycling (HIR) and Cold Central Plant Recycling (CCPR)” along with the Tennessee Department of Transportation (TDOT). One of the research objectives is to perform field testing and evaluate the pavement performance of 100% asphalt recycling projects.

The purpose of this survey is to determine the current practices of state DOTs regarding the three techniques (HIR, CIR, and CCPR) to seek advice and opinions in this area. Survey results will be shared with industry, government agencies, and officials to promote recycling and sustainable paving technologies.

Responding states

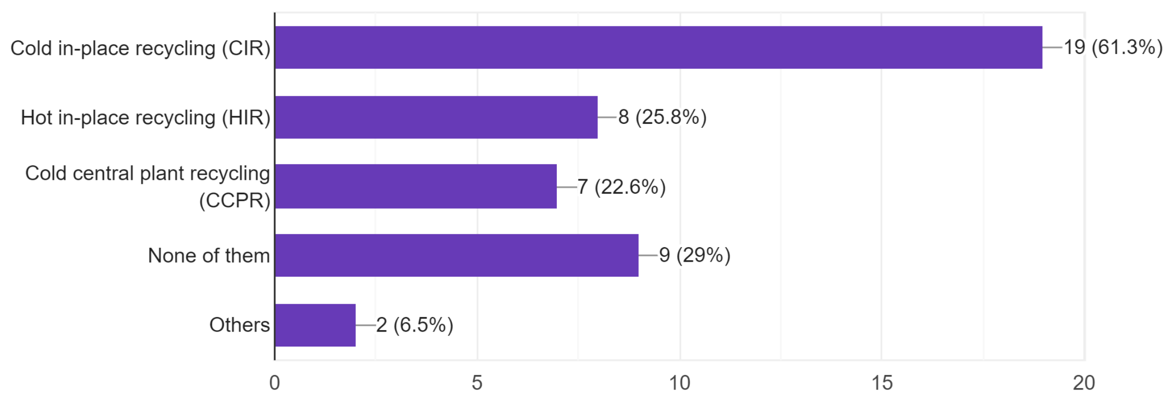
The survey was sent to 52 DOTs (including the District of Columbia and Puerto Rico) listed in Table A-1. Among them, 31 states responded to the survey.

Table A-1. States response to the survey.

No.	***States***	***State responded***	***State not responded***
1	Alaska	1	Alberta
2	Arkansas	1	Arizona
3	Colorado	1	Connecticut
4	Indiana	1	Idaho
5	Kansas	1	Maine
6	Maryland	1	North Dakota
7	Montana	1	Puerto Rico*
8	Mississippi	1	Massachusetts
9	Missouri	1	Michigan
10	North Carolina	1	Minnesota
11	New York State	1	Texas
12	Rhode Island	1	Nebraska
13	South Carolina	1	Nevada
14	Utah	1	New Hampshire
15	Virginia	1	New Jersey
16	Washington, D.C.	1	Oklahoma
17	Wisconsin	1	Oregon

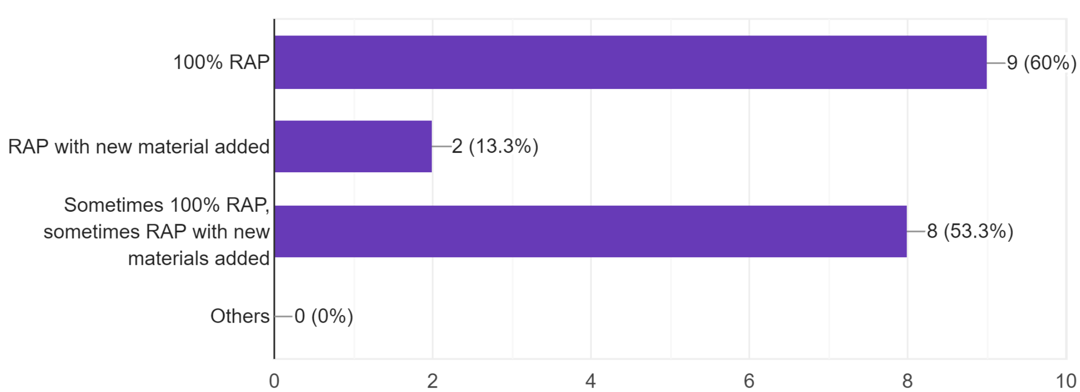
18	Louisiana	1	South Dakota
19	Ohio	1	Vermont
20	California	1	Wyoming
21	New Mexico	1	Hawaii
22	Florida	1	
23	Georgia	1	
24	Illinois	1	
25	Iowa	1	
26	Kentucky	1	
27	Pennsylvania	1	

<u>1. Among the three recycling technologies, which technology is mostly used in your state?</u>



Notes: Other technologies: Full-depth reclamation (FDR)

<u>2. For the HIR practice in your state, what types of recycled materials do you often use?</u>



Notes: Not too many states have used the HIR technique for pavement rehabilitation.

3. What mixing temperature does your state use in the HIR construction?

Possible answers:

- Kansas does not require a mixing temperature. The HIR is to be a minimum of 190°F behind the paver.
- Min 225 F.
- 235°F – 300°F
- 275 – 325 deg F
- We have not yet ha a project, but the county job, here are the specifications from the county contract Equipment: It shall be capable of heating the pavement surface to a temperature high enough to allow scarification to the required depth without breaking aggregate particles or charring the pavement (225-325 degrees F) Heating and repaving process: The removed material shall have a temperature between 115 degrees F and 265 degrees F, unless otherwise directed by the Engineer
- Usually around 300°F. The contractor can choose the mixing temperature.

Notes: The mixing and paving temperature should be at least 225F. However, both the temperature and the performance of mixtures should reach a balance, since more heating would cause more smoke and burn the asphalt.

4. How many passes of the heating-milling units are often used during the HIR process in your state?

Possible answers: 1 pass; 2 or 3 passes; 1 or 2 passes; 1 train with 3 heaters; 1 train including 4 hot milling units.

Notes: The heating train consisting of one preheating unit, and four heat milling units worked well in Tennessee.

5. What type of additives does your state often use for HIR?

Possible answers:

- Ergon and Vance Brothers are pre-qualified. Specification 1205 is linked. I believe the ARA is Reclamite®.
- Contractor designed; unsure of amounts.
- ERA 25
- ARA

Notes: In Tennessee, ARA3P from Ergon was used.

6. What is the amount of additive does your state often use for HIR? eg: 0.5 gallons/ft^2

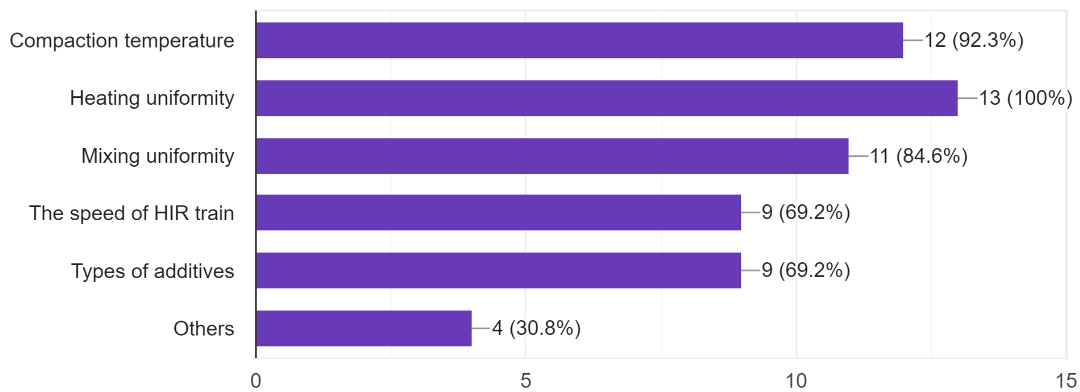
Possible answers:

- Approximately 0.175 gal/sy/inch
- Contractor designed; unsure of amounts.
- Depends on in-situ material.
- 0.1 – 0.2 Gal/SY

- The contractor determines the amount of rejuvenator necessary to meet our specification requirement for recovered binder. Recovered binder must meet one of the two criteria below. - Penetration of the recovered asphalt material in the asphalt mixture (determined in accordance with AASHTO T 49), within the range of 40 – 80 dmm - Viscosity of the recovered asphalt material in the asphalt mixture (determined in accordance with AASHTO T 202), within the range of 5,000 to 15,000 poises.

Notes: The amount of additive was determined based on the viscosity requirements of asphalt, RAP properties, and mix design.

7. In the HIR technique, which of the factor (or factors) do you think is (are) critical?



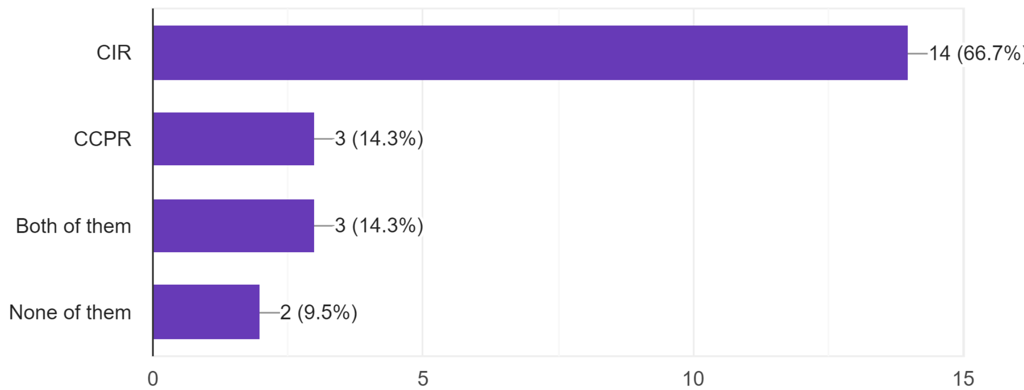
Notes: Other factors including milling depth; mix design properties;

8. What is the challenge for HIR construction?

Possible answers:

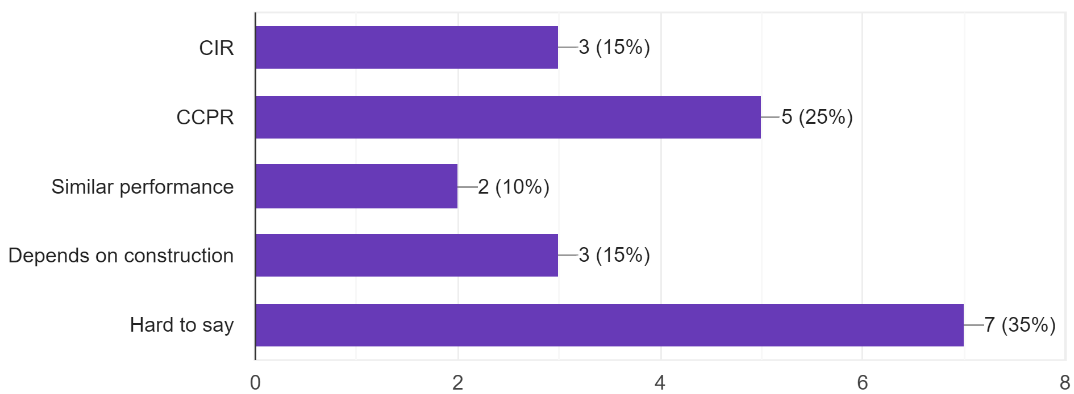
- Maintaining appropriate heat through the process - balance between destroying the rock (too cold) and destroying the binder (too hot)
- Needs a final surface cap that will often exceed the cost of a thin lift overlay or single lift mill/fill.
- Our contracting community is fighting it tooth and nail because all the equipment is "out of state". Even proving it's the best economical decision, with out of state mob, isn't working. It's very political in our state.
- Burning vegetation
- We don't have a local contractor who performs this, we are not seeing a cost advantage, and of the structural testing we performed on the county job is showing results which do not give us confidence.

9. As for CIR and CCPR, which type of recycling does your state often use?



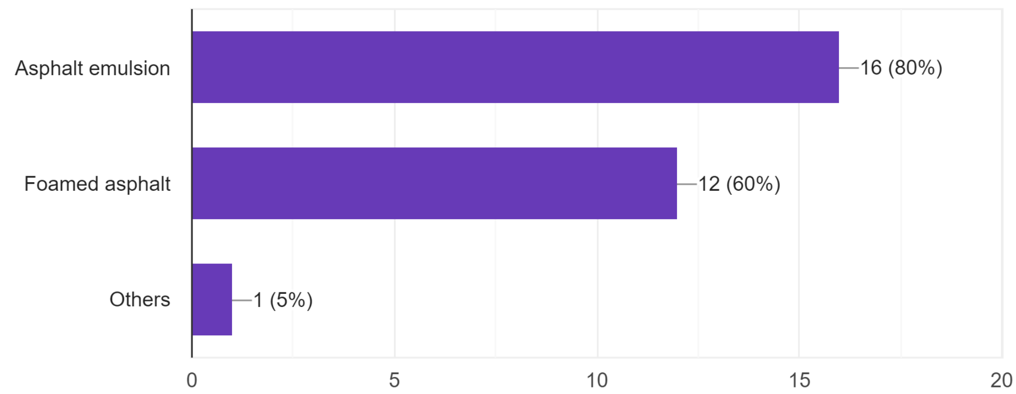
Notes: most of the states used CIR instead of CCPR. Also, some of the states used FDR, which milled together will the subgrade soil.

10. As for CIR and CCPR, which type of recycling do you think can give a better performance of the pavement?



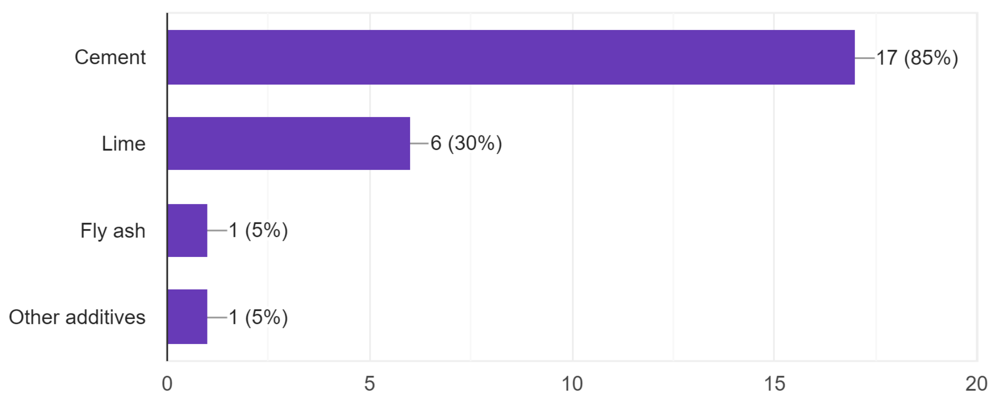
Notes: CCPR could adjust gradation of the mixtures, but states have less experience of doing CCPR.

11. In cold recycling (CIR or CCPR), which type of asphalt stabilization agents are commonly used in your state?



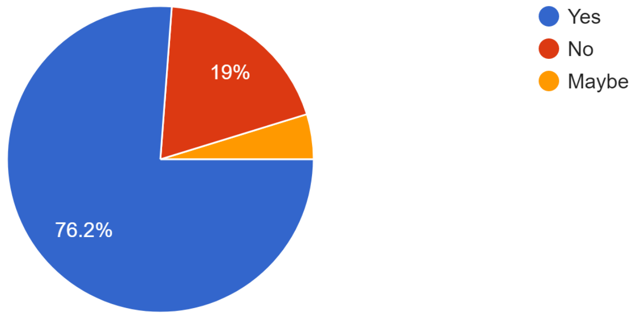
Notes: Asphalt emulsion was widely used. Some states used foamed asphalt to save money. Each type works well.

12. In cold recycling (CIR or CCPR), which type of cementitious agents are commonly used in your state?



Notes: Most states used cement and lime, which would increase the strength and moisture resistance. Cement would have less setting time, while lime would have less cracking.

13. Do you think the milling will affect the gradation of RAP significantly in cold recycling (CIR or CCPR)?



Notes: Milling will affect the gradation of RAP, especially the speed of the milling train. Indiana DOT specified the gradation of milling as follows: "The milling would need to pulverize the material to meet the proposed design (and ongoing QC testing of the reclaimed base). We require at a minimum the pulverization shall produce a gradation that has 100% passing the 1 ½" sieve."

14. What is the curing condition that you often use for CIR and CCPR in your state? Please specify the curing temperature and curing time.

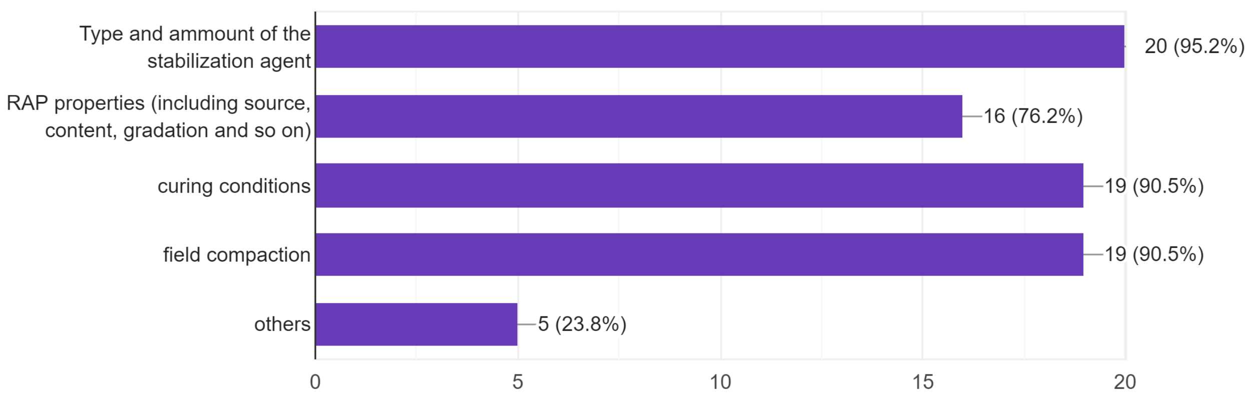
Possible answers:

- Complete between May 1 and September 30, when the ambient air temperature is greater than 50°F and rising. Curing period ends when the moisture content is below 2%
- Above 50 F and rising and above freezing for 48 hours after placement. Curing time is until the moisture level is less than 2%.
- Mix Design per CR 201 and 202, Field 40 deg plus for 3 days (foam) or 10 days (emulsion).
- We like to have the mat ready to open in a couple hours after placement. We have stability tests that help us check to see if it is ready to open to traffic. Temperatures of the compacted material need to be between 80 and 120 F.
- Construction of CIR shall not be carried out before June 1 or after August 31 of any year except upon written order of the Project Engineer. Begin recycling when the forecasted air temperature is 60°F and rising at the project site. Cease recycling operations if the air or pavement temperatures drop below 60°F. Do not begin CIR if the forecasted air temperature will drop below 40°F or rain is anticipated within 48 hours of mixing.
- Same as our soil-cement specifications. Section 308 of our standard specifications.
- CDOT Specification "…Before placing the sealing emulsion or hot mix asphalt overlay, the cold recycled pavement shall be allowed to cure until the free moisture is reduced to 1 percent free moisture or less, by total weight of mix…"
- Temperature must be above 50 F. For the initial cure, traffic may be permitted a minimum of four hours after final compaction; the first two must be daylight hours. Final cure will be considered complete when the moisture content drops 50% from the final compaction moisture content, and the CIR is satisfactorily proof rolled as directed.

- In the project we completed, curing was completed by placing an emulsion and sand over the completed mixture until it was paved, which was about 48 hours on average. This occurred in September weather of upstate South Carolina.
- Max 50% of the optimum water content, min operation air & material temp is 50F
- 3 days and moisture measured at mid-depth of the recycled section is 2.0 percent or less, or 2) 10 days without rain.
- Pavement temps must be above 50F with ambient temps above 35F for seven days in a row. We require less than 3.0% moisture remaining in the mixture or in place for 10 consecutive days without rainfall before the material can be paved over.

Notes: Overall, the temperature should be greater than 50F, and the curing time should be determined based on the moisture content and weather conditions.

<u>15. In cold recycling (CIR or CCPR) technique, which of the factor (or factors) do you think is (are) critical?</u>



<u>16. What is the challenge for the CIR, and CCPR technique?</u>

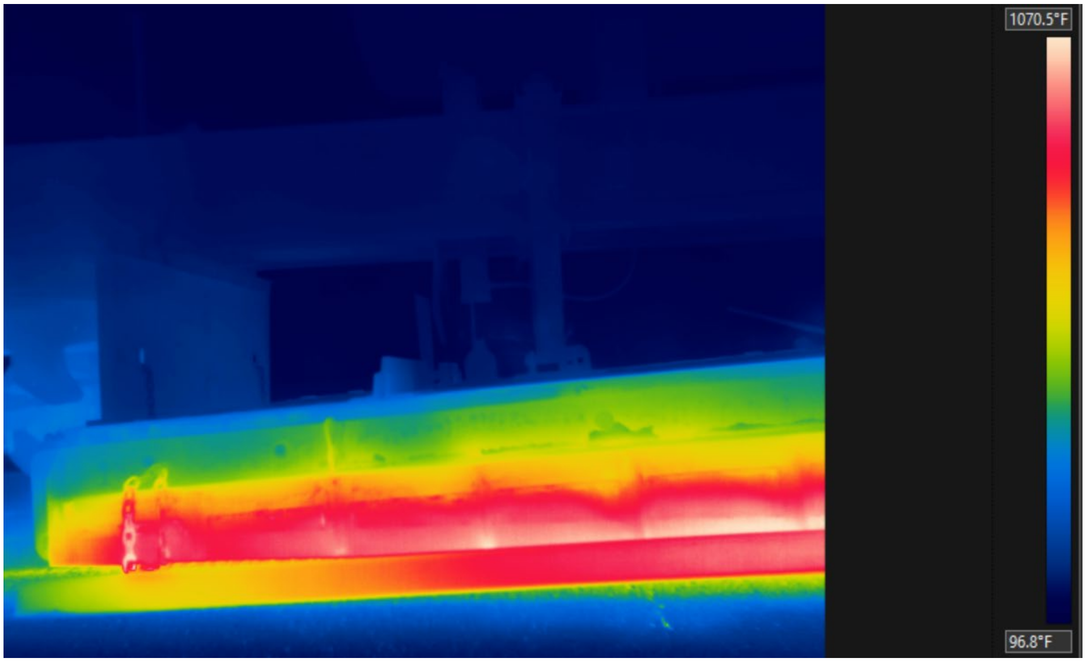
Possible answers:

- That it not be prone to stripping
- Emulsions are very temperature sensitive.
- Staff familiarity, our recycling projects are very few and far between so that a crew may only see one in their career. So we have to “re-learn” every time.
- Getting Contractors to follow the Specifications and Industry Standards
- The proper project selection and maintenance of traffic.
- Ensuring an agency has an experience contractor on the project.
- Mixture consistency in the field
- Achieving proper compaction
- Maintenance of Traffic.
- Lack of local contractors and fear of changes (new technologies). We feel our project was very successful, but there is still hesitation in many of our offices.
- Need faster and easier methods to accept the materials.
- Inspection, project selection, materials variabilities
- Proper field investigation, opening to traffic (raveling), and sufficient curing time prior to opening to heavy vehicles.

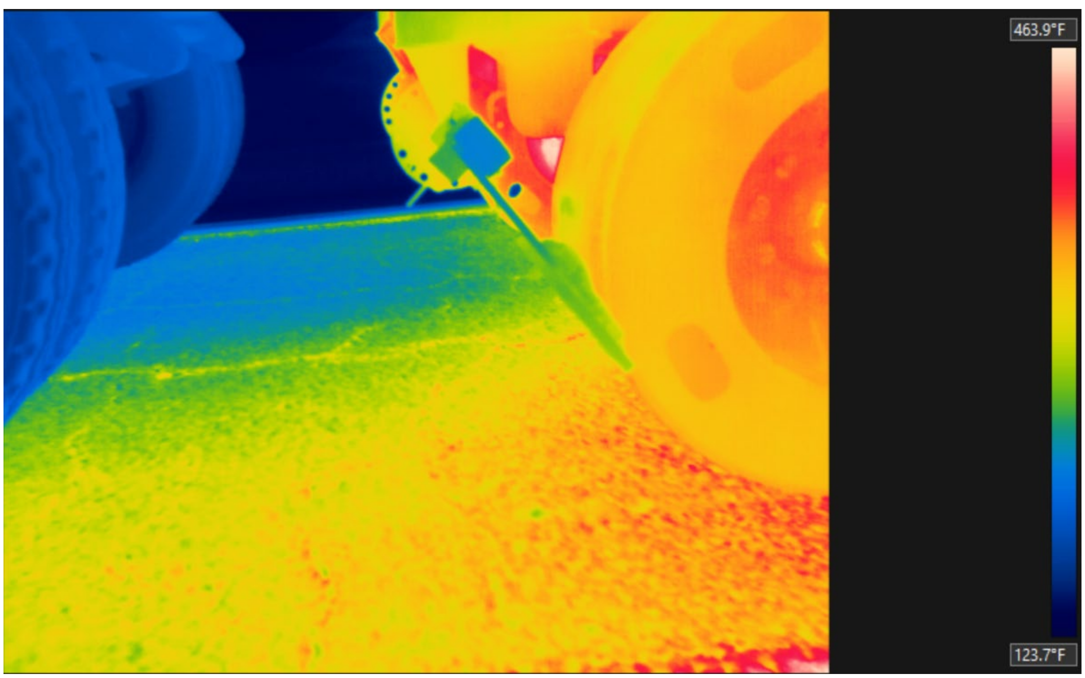
- Proper candidate selection (or will fail). A (sub) contractor with experience. Proper establishment of the road profile cross slope. Grader should be used, but most try to make it up with the pre surface course milling. Can often cut a few inches out on the roadway edge of the reclaimed material we just paid for (and reduce the structure). Saturated subbase that will result in soft spots/failure.

Appendix B: Infrared camera results

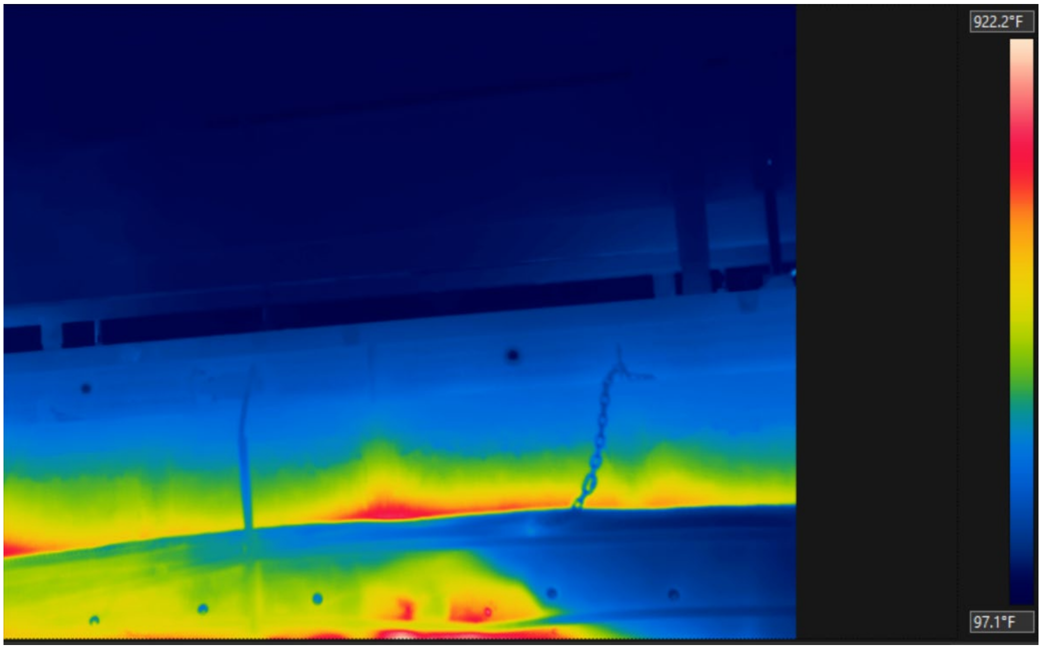
The temperature distributions during the HIR process were captured using an infrared camera and presented in Figure B-1(a) to (m). The highest temperature that the heater could reach and the average pavement temperature were summarized in Table B-1



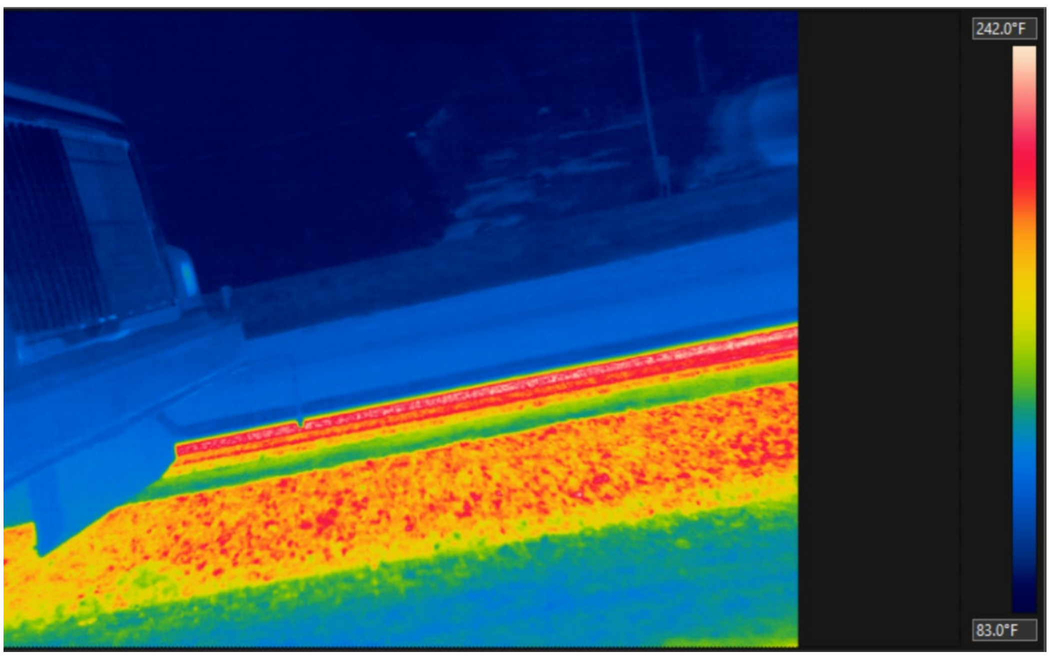
(a) Temperature distribution during the first heating.



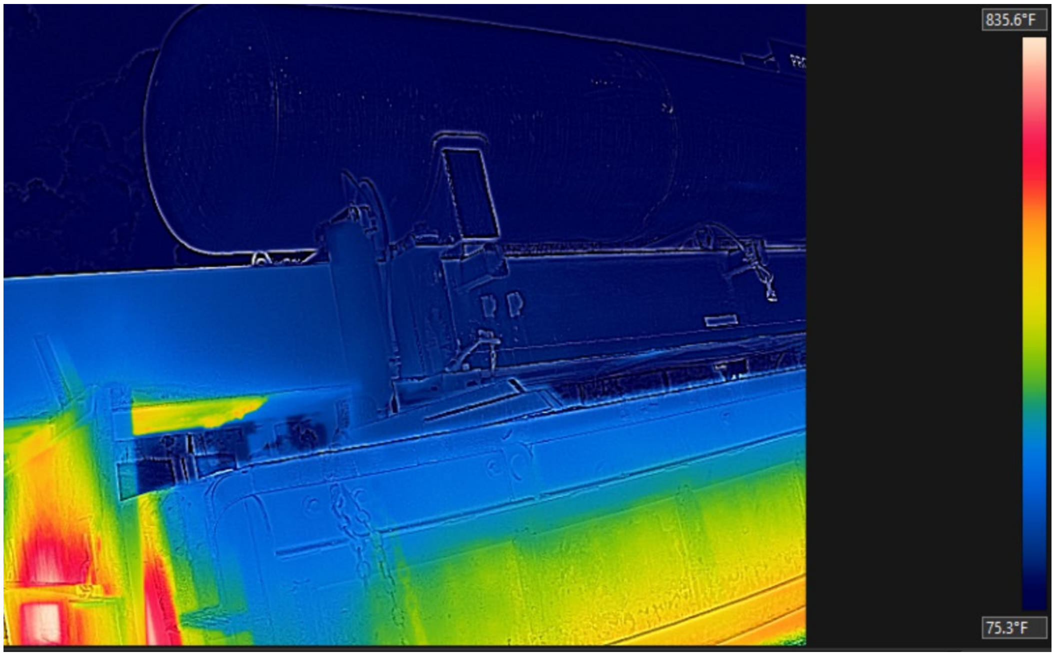
(b) Pavement temperature distribution after the first heating.



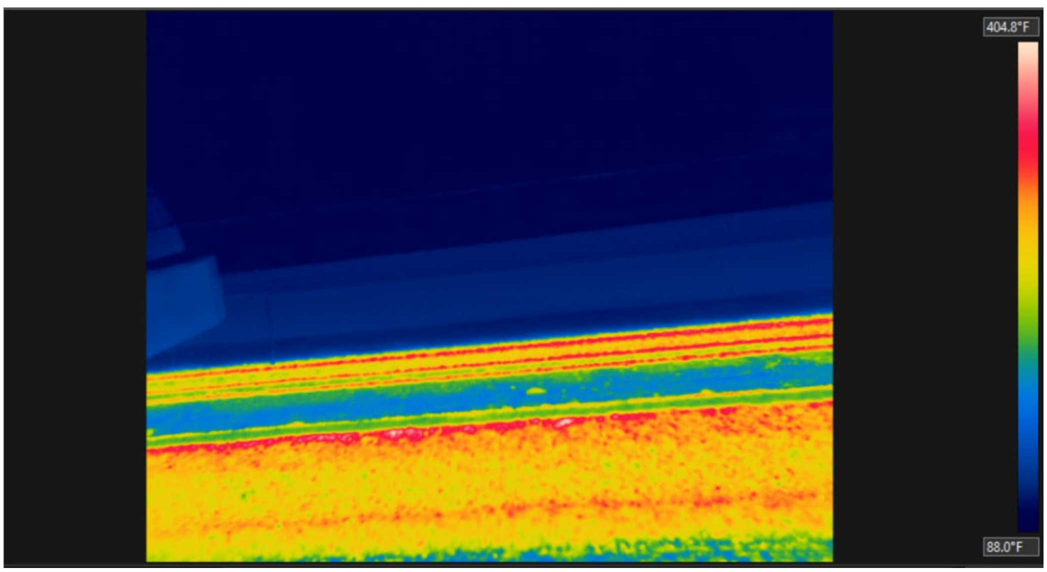
(c) Temperature distribution during the second heating.



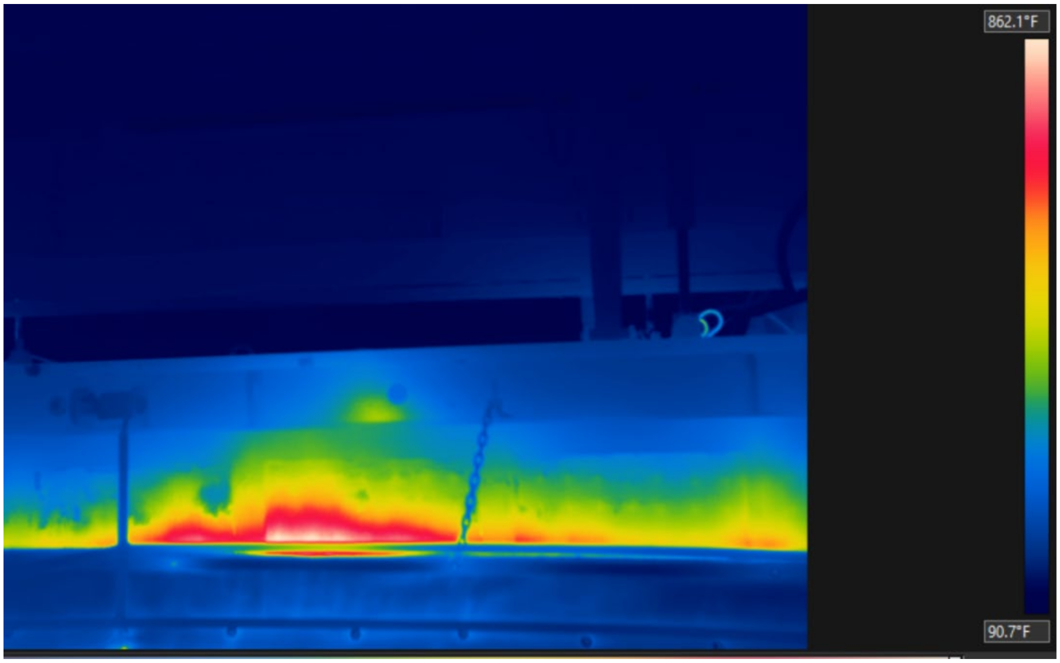
(d) Pavement temperature distribution after the second heating.



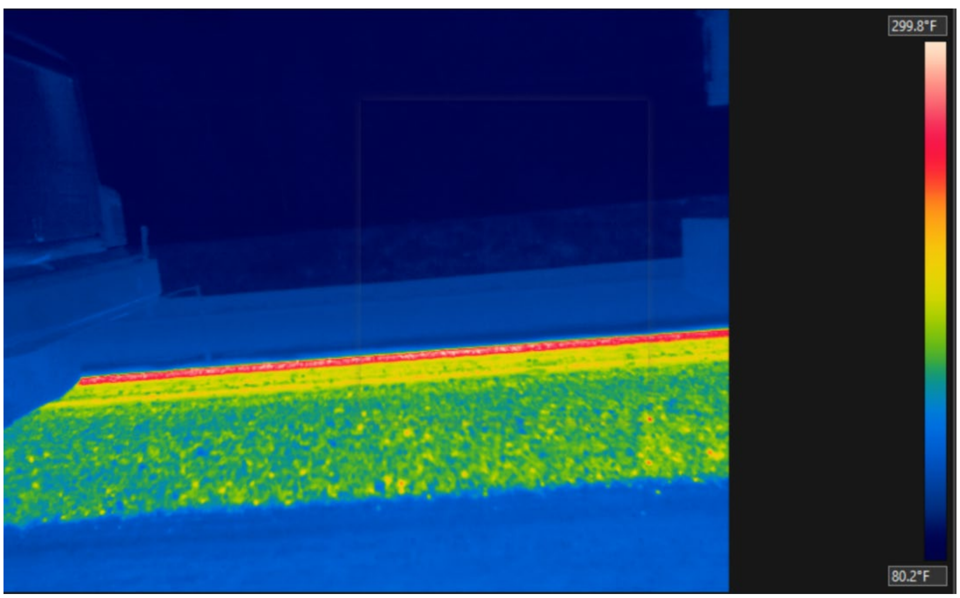
(e) Pavement temperature distribution during the third heating.



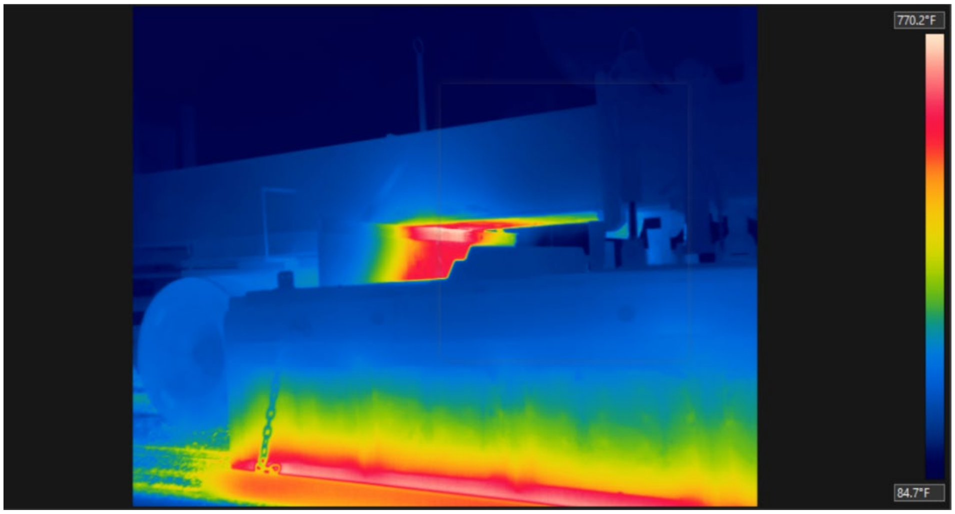
(f) Pavement temperature distribution after the third heating.



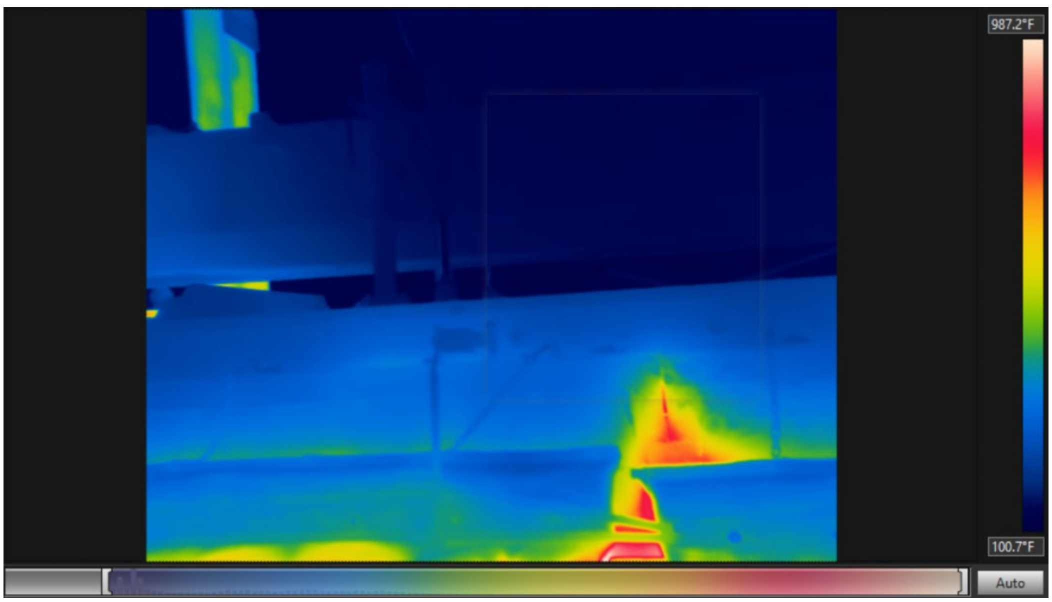
(g) Temperature distribution during the fourth heating.



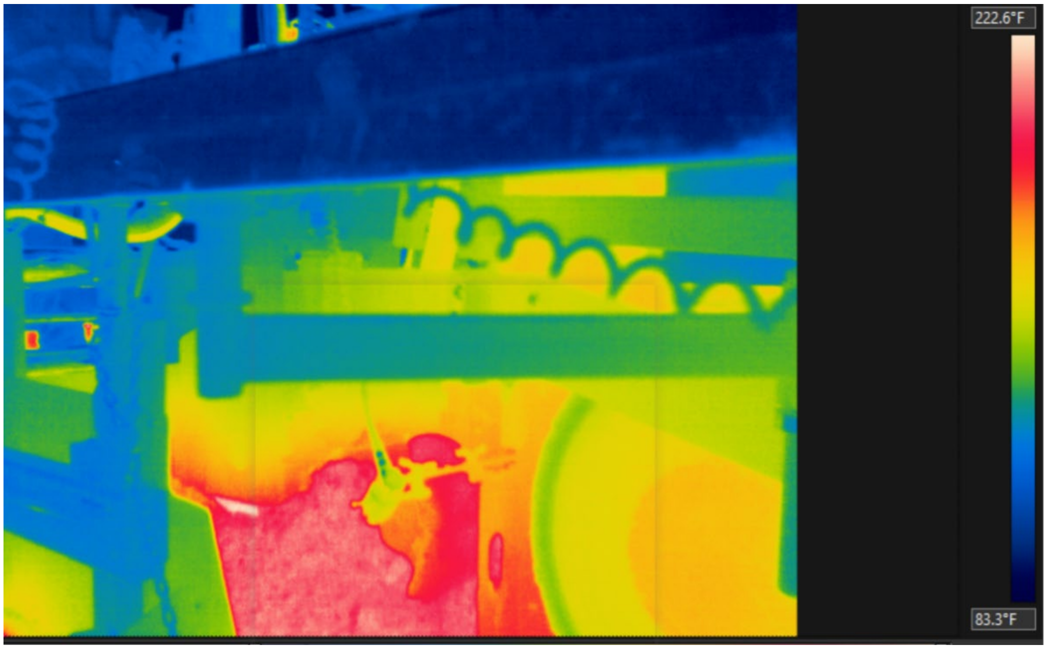
(h) Pavement temperature distribution after the fourth heating.



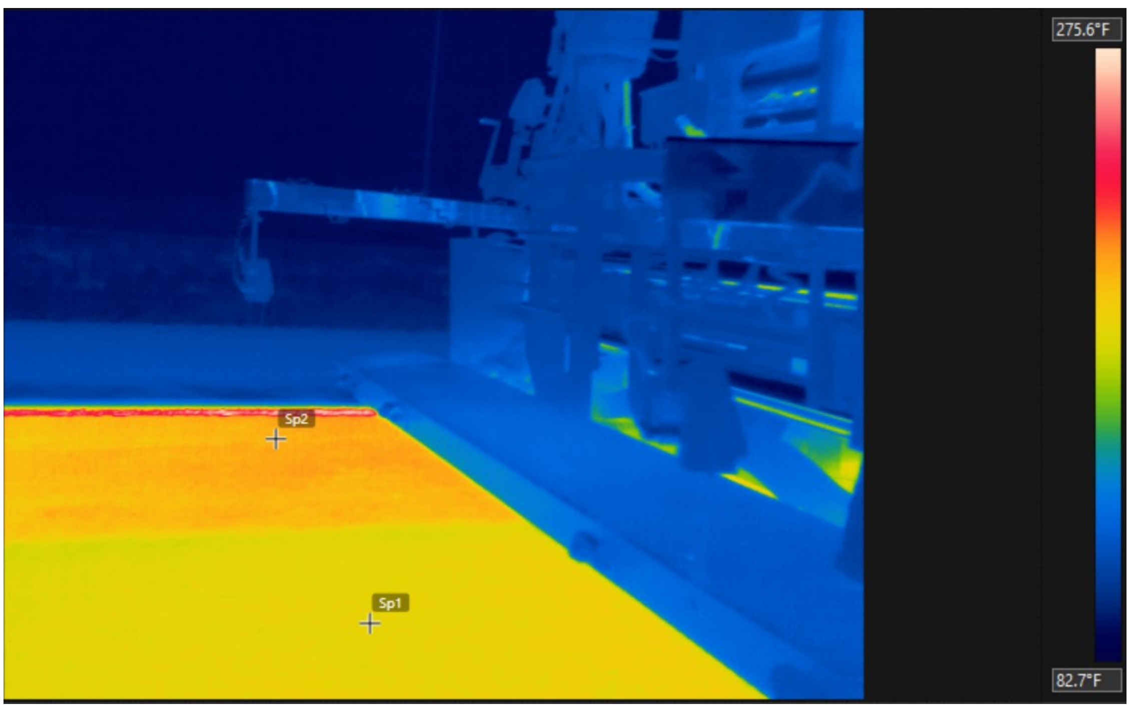
(i) Temperature distribution during the fifth heating.



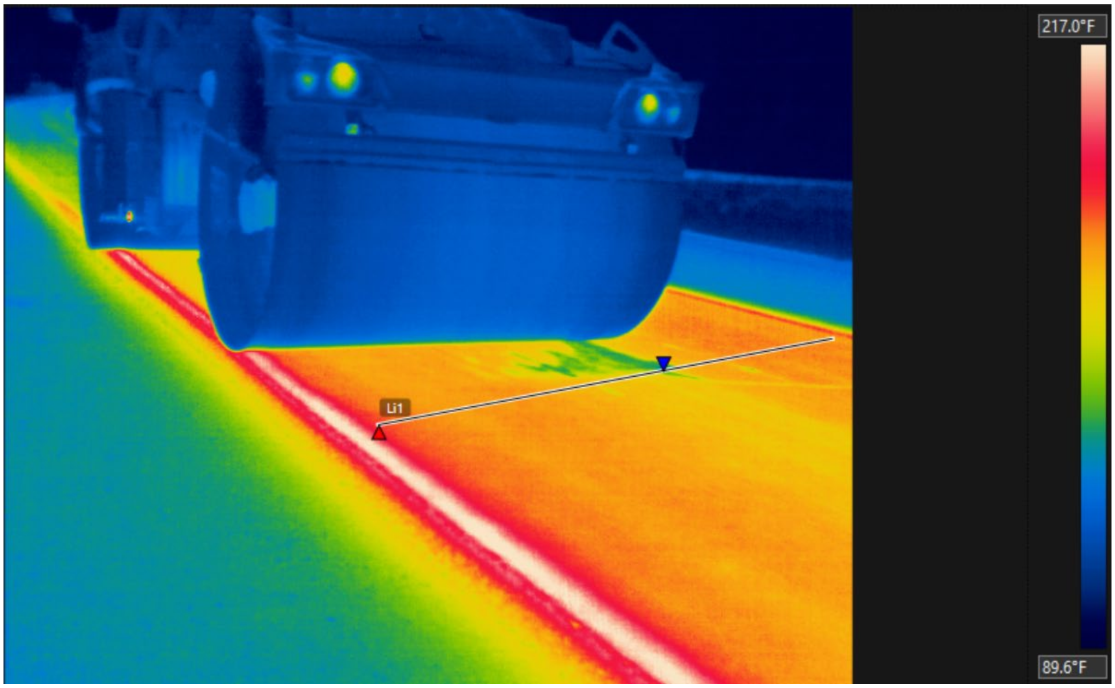
(j) Temperature distribution during the sixth heating.



(k) Temperature distribution of HIR mixes during mixing with additive.



(l) Pavement temperature distribution after paving.



(m) Pavement temperature distribution after last rolling.

Figure A-1. Temperature distribution of all HIR procedures.

Table A-2. Summary of temperature changes during the HIR procedures.

HIR procedures	*Temperature (°F)*	*Temperature (°C)*
First heating	1070.5	576.9
Pavement after the first heating	463.9	240
Second heating	922	494.4
Pavement after the second heating	193	89.4
Third heating	835.6	446.4
Pavement after the third heating	298	147.8
Fourth heating	862.1	461.7
Pavement after the fourth heating	197.9	92.1
Fifth heating	770.2	410.1
Six heating	987.2	530.7
Mixing with additive	222	106
After paving	208.9	98.3
Last rolling	178	81.1

Appendix C: Figures in Chapter 3

Figure A-2. UT mobile trailer and specimen compaction in the field.

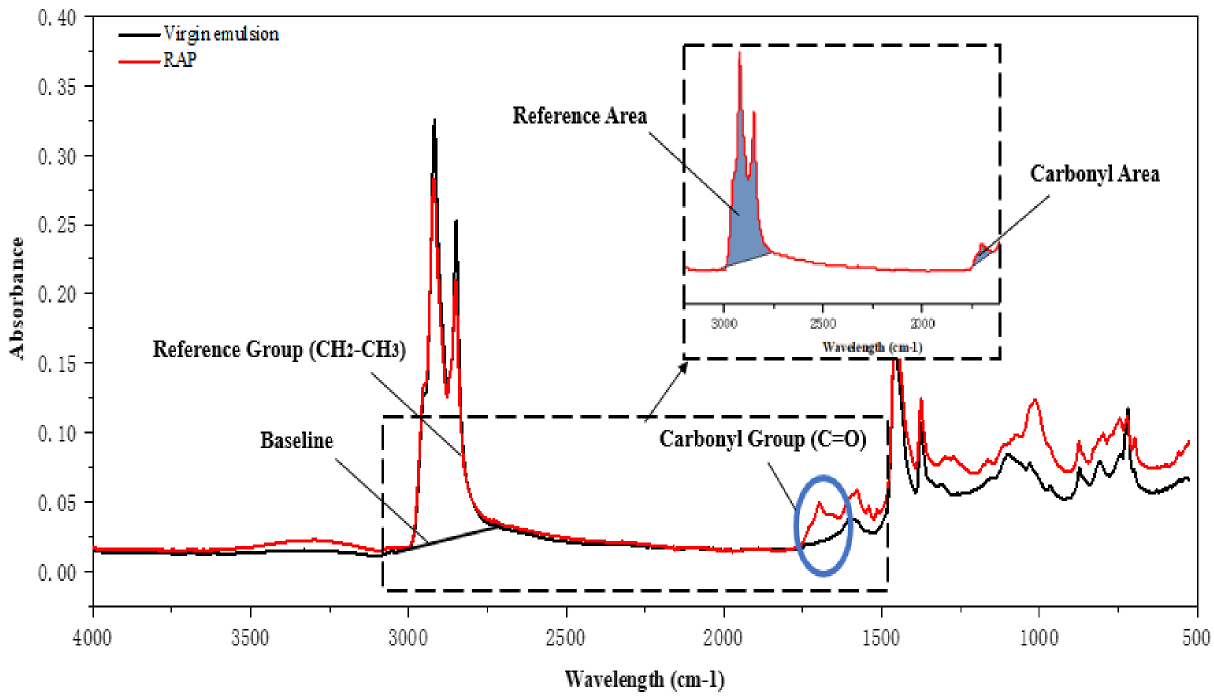


Figure A-3. FTIR spectrums of virgin binder and RAP binder with the peak area selections.

Figure A-4. Asphalt mixture performance tester (AMPT).

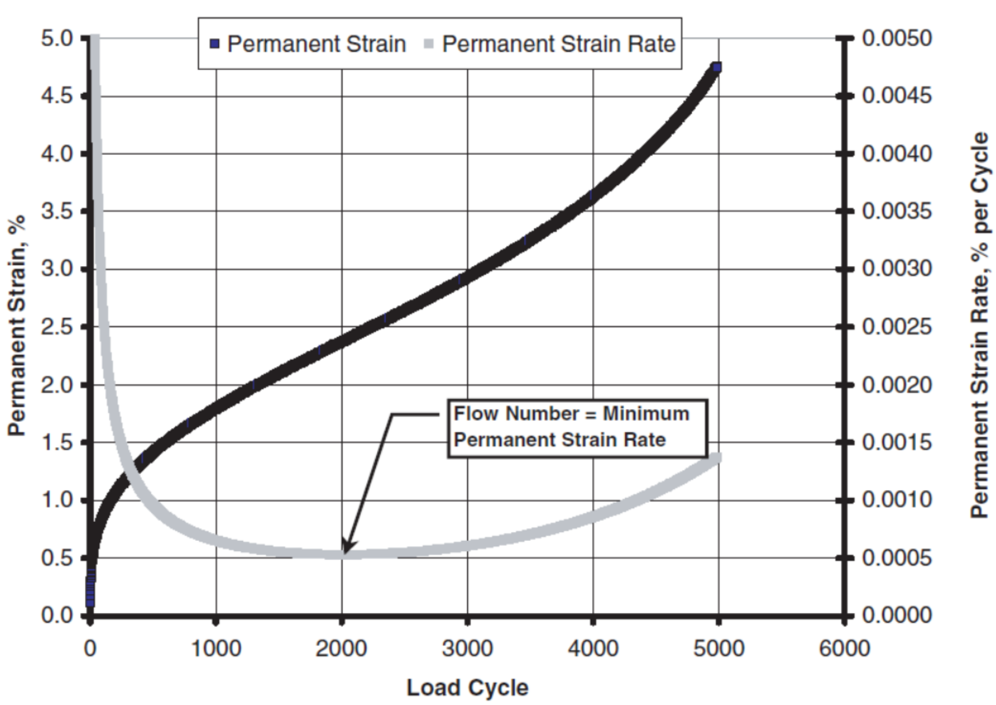


Figure A-5. Example of flow number determination (NCHRP673).

Figure A-6. Setup for Superpave IDT tests.

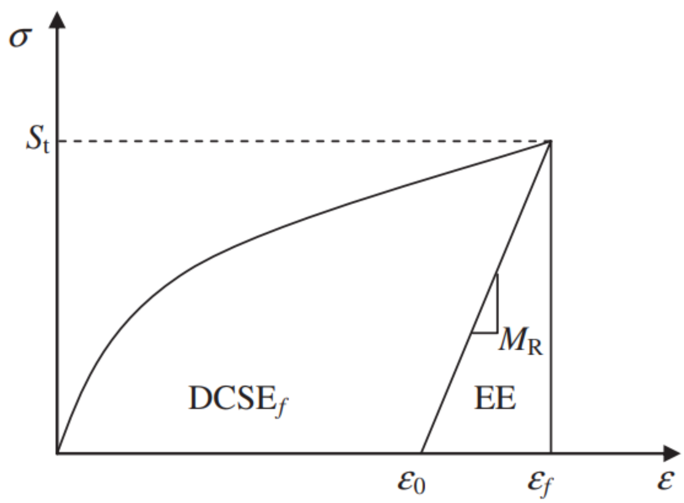


Figure A-7. Determination of creep strain energy (Chen and Huang, 2008)

Figure A-8. Test specimens after 8000 cycles.

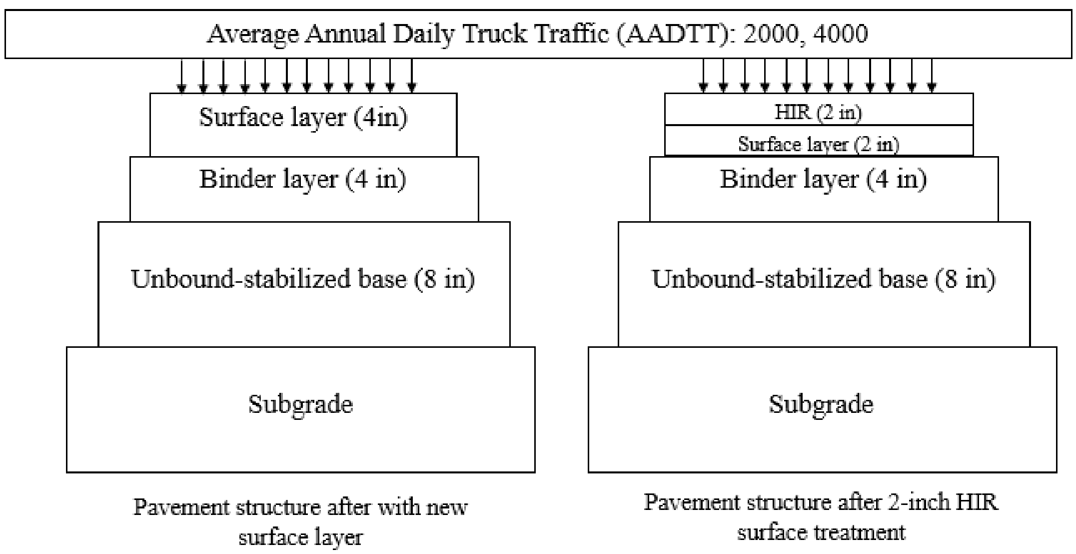


Figure A-9. Pavement structure models of two pavements; (a) New HMA surface layer; (b) after HIR surface treatment.

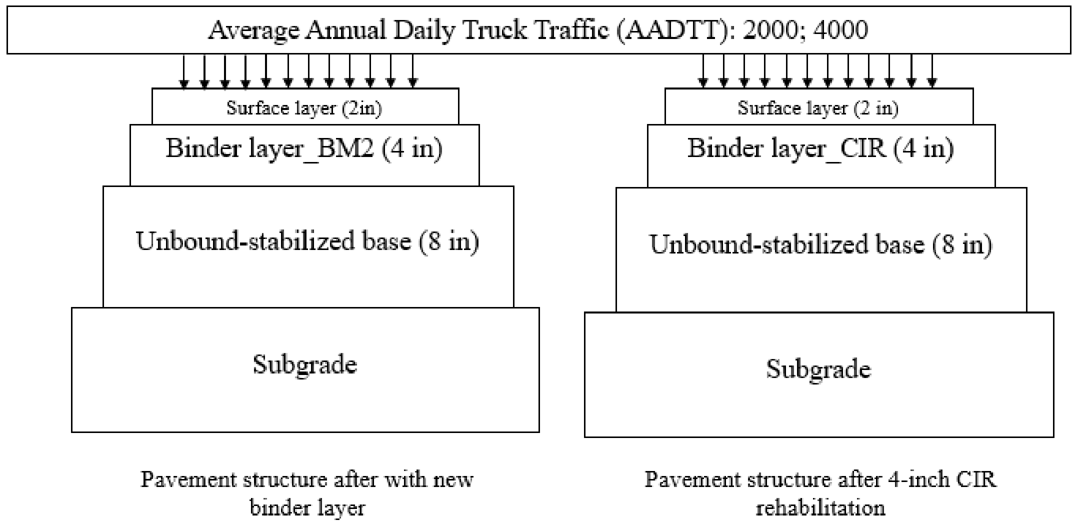


Figure A-10. Pavement models for rehabilitation with new BM2 layer (left) and CIR technique (right).

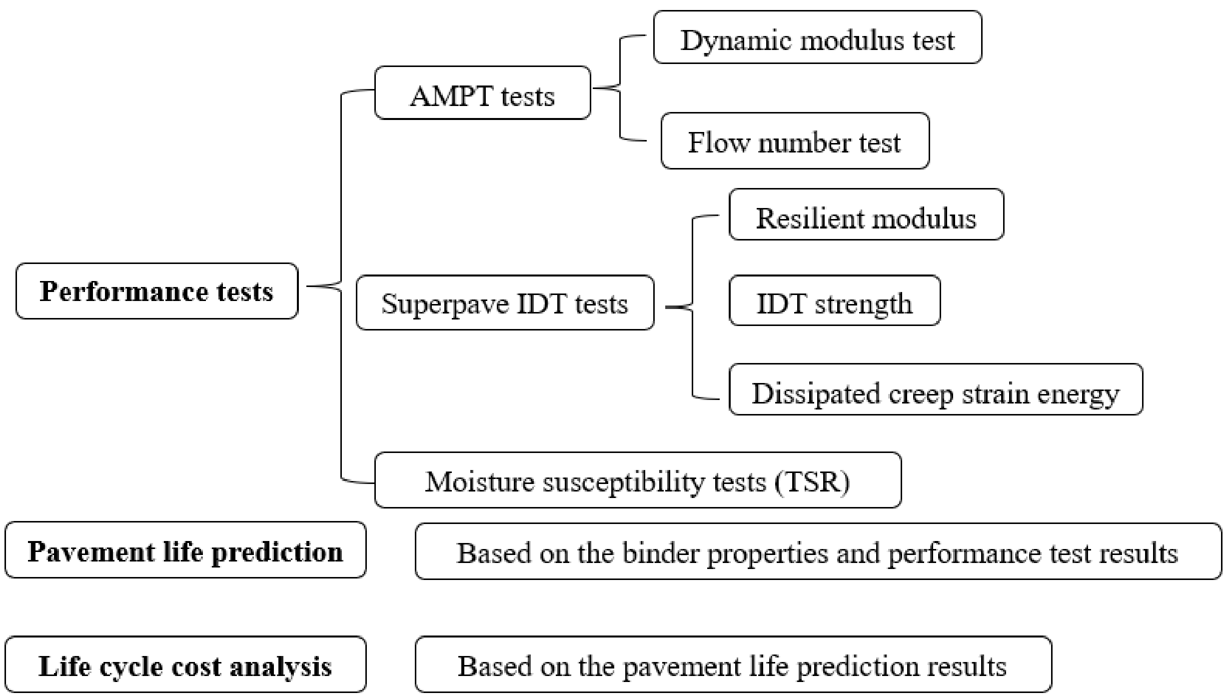


Figure A-11. Flow chart of task 1.

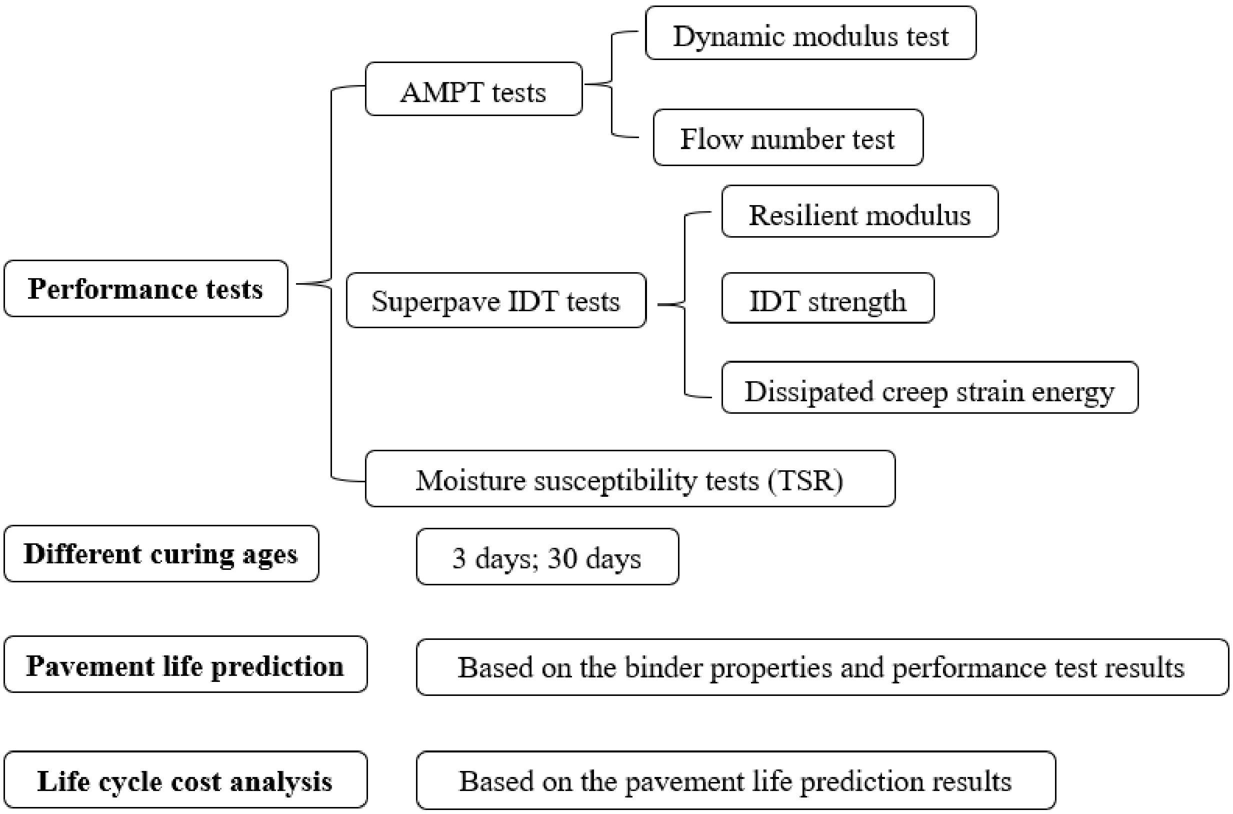


Figure A-12. Flow chart of task 2.

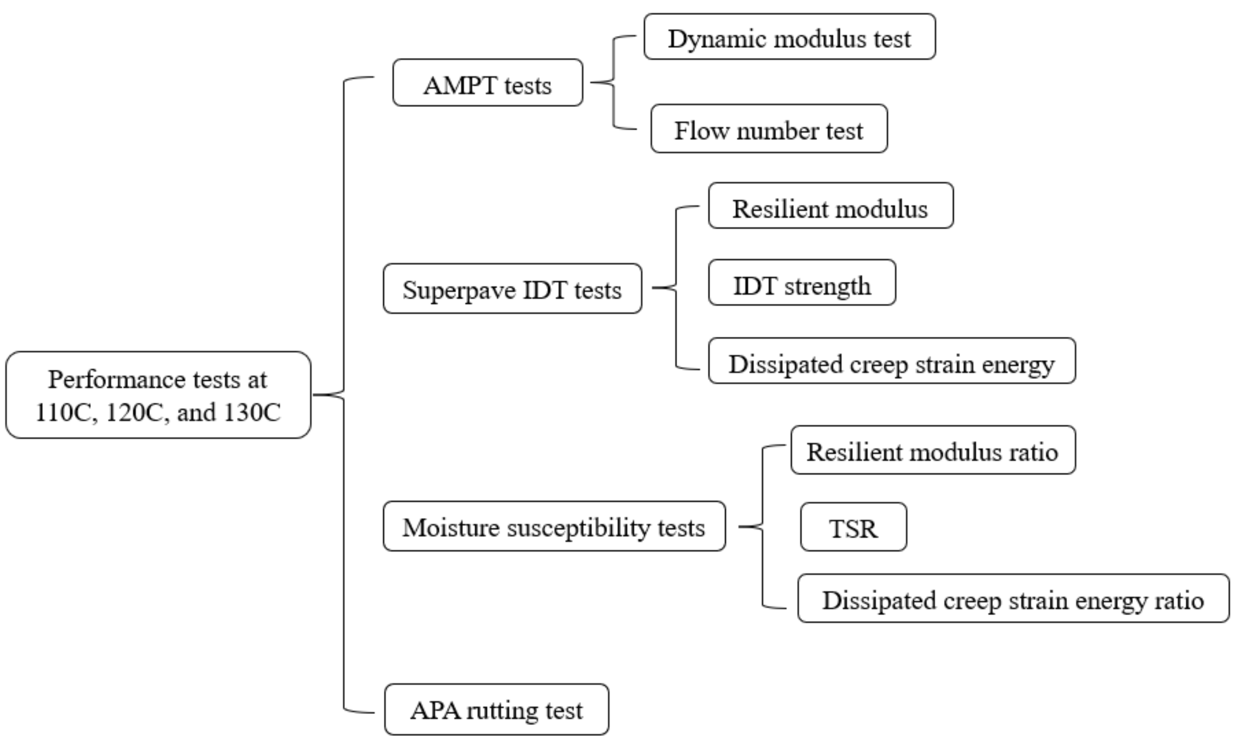


Figure A-13. Flow chart of temperature effect on the performance of HIR mixes.

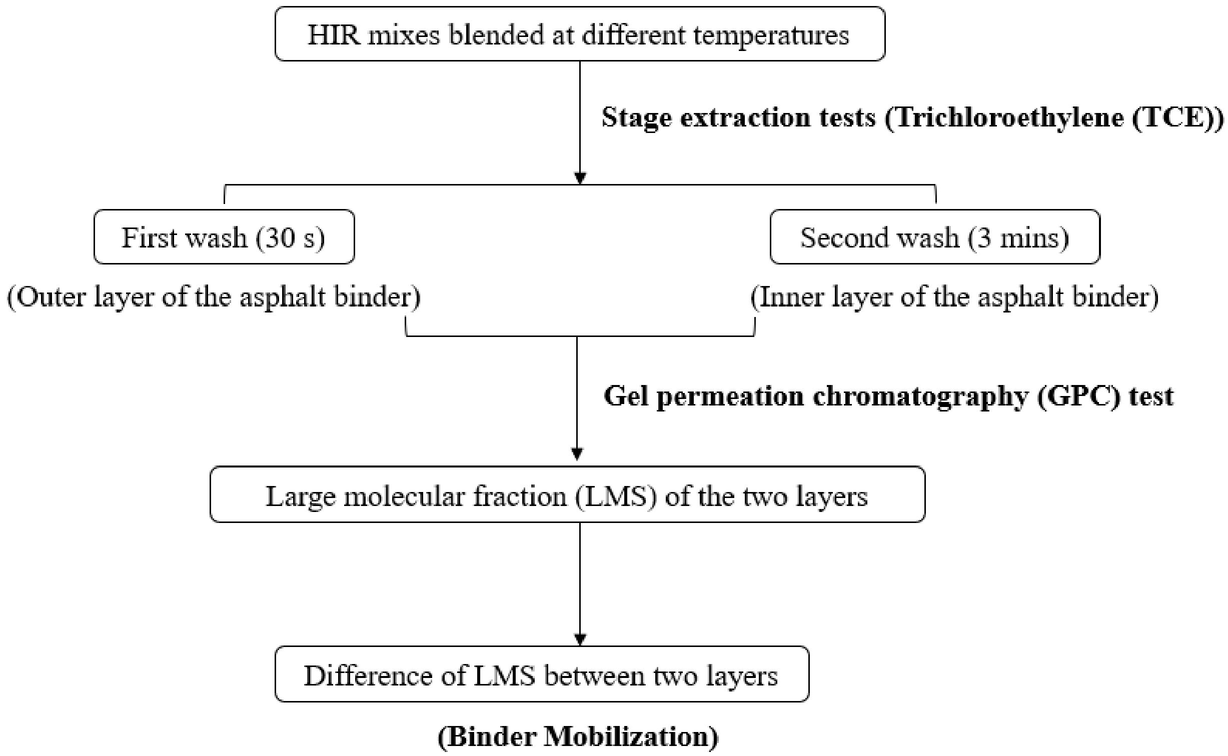


Figure A-14. Flow chart of temperature effect on the binder mobilization of HIR mixes.

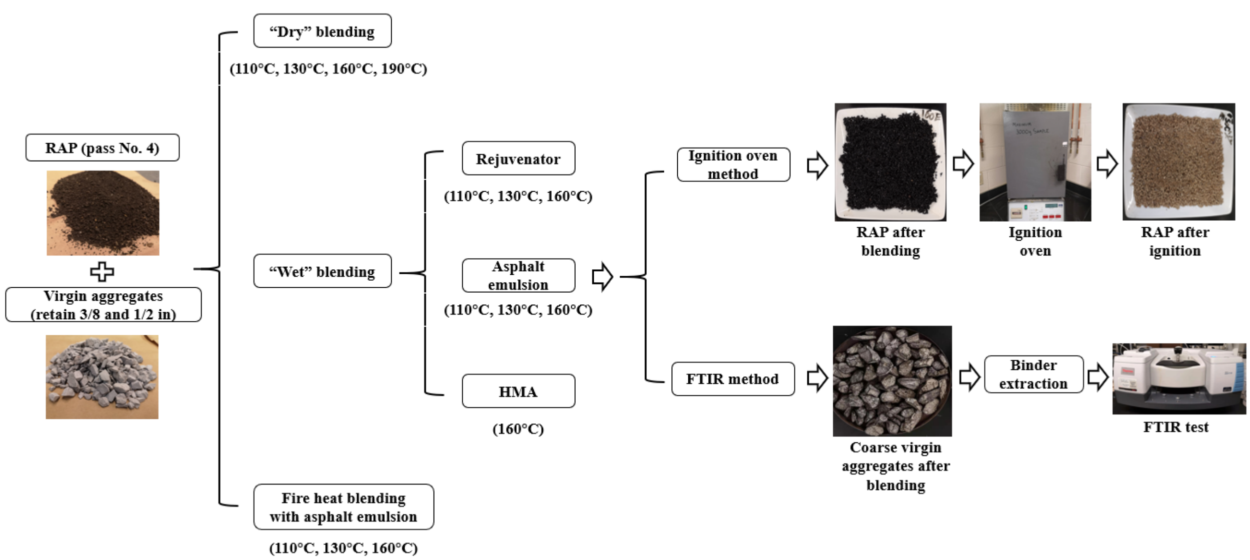


Figure A-15. The experimental design flow chart of task 4.

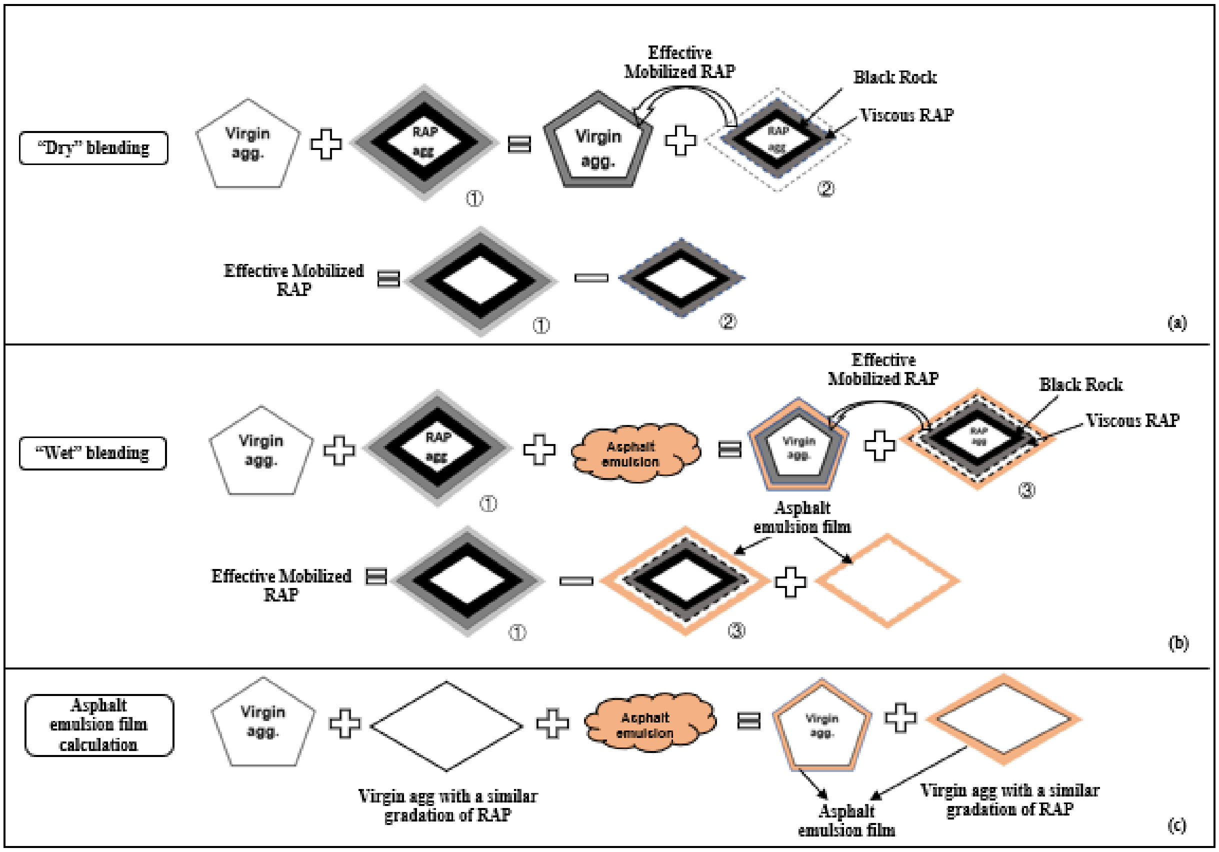


Figure A-16. Two scenarios of quantifying the effective mobilized RAP content. (a) "Dry" mixing; (b) "Wet" mixing; (c) Asphalt emulsion film calculation.

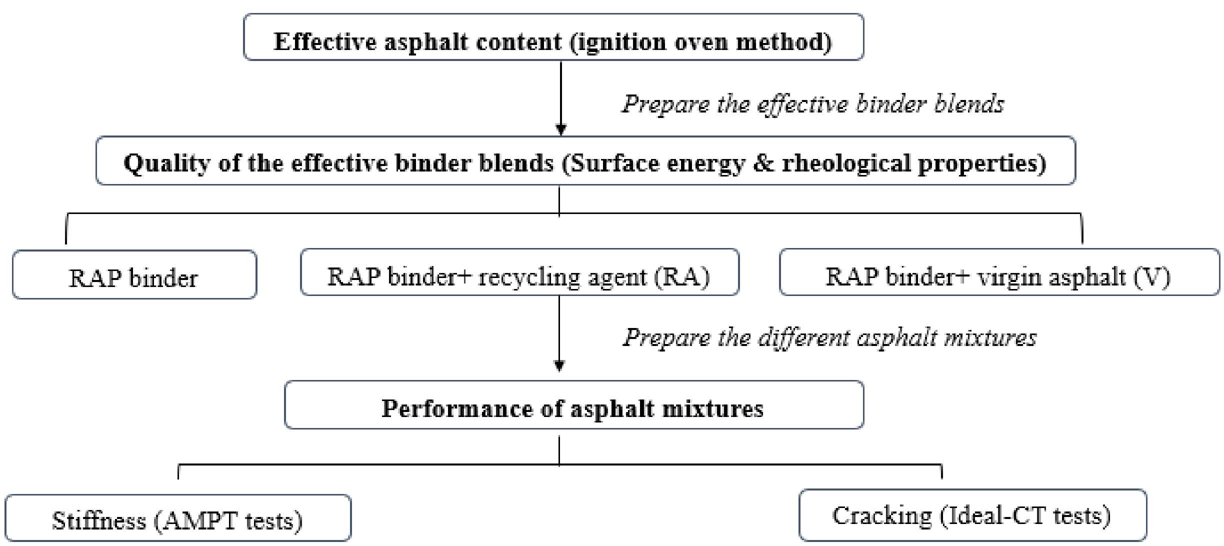


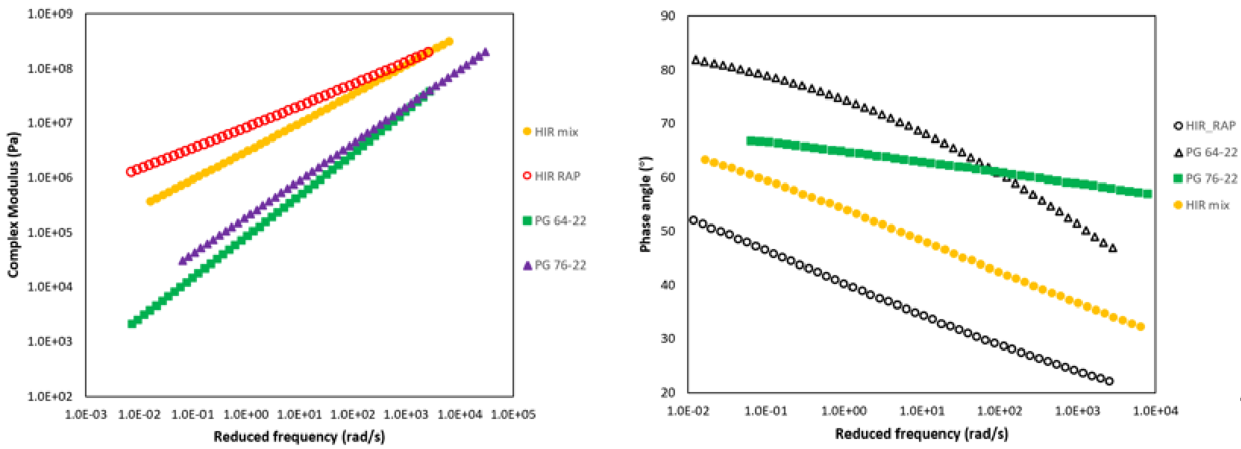
Figure A-17. Flow chart of task 5.

Appendix D: Additional figures in Chapter 4

Additional figures in section 4.1

Binder level-1 data inputs

As for binder properties, the binders of HIR mixes before and after rejuvenation were extracted and recovered for DSR frequency sweep tests at different temperatures. Asphalt is assumed to be fully blended with the RAP binder, which is the ideal blending scenario for HIR. DSR master curves and the input parameters are presented in Figure D-1 and Table D-1, respectively. It is shown in Figure 2 that the stiffness of the RAP binder is much higher than the base asphalt, especially in low frequency (high temperature). The addition of additives could soften the RAP binder and improve the rheological properties of the binder blends.



(a) (b)

Figure A-18. DSR master curves of different binder blends. (a) Complex modulus master curves; (b) phase angle master curves.

Table A-3. Binder properties design input.

PG 64-22				PG 76-22			
Temp (F)	Temp (°C)	G* (Pa)	Phase angle (°)	Temp (F)	Temp (°C)	G* (Pa)	Phase angle (°)
59	15	676435.5	65.3	59	15	1146239	61.8
77	25	116630.4	72.1	77	25	245153	64.9
95	35	15805	78.7	95	35	45049	64.9
HIR mix				HIR RAP			
Temp (F)	Temp (°C)	G* (Pa)	Phase angle (°)	Temp (F)	Temp (°C)	G* (Pa)	Phase angle (°)
59	15	14782760	43.63	59	15	29618406	31.32
77	25	6162031	51.34	77	25	11253999	38.39
95	35	862668	58.55	95	35	3055880	46.59

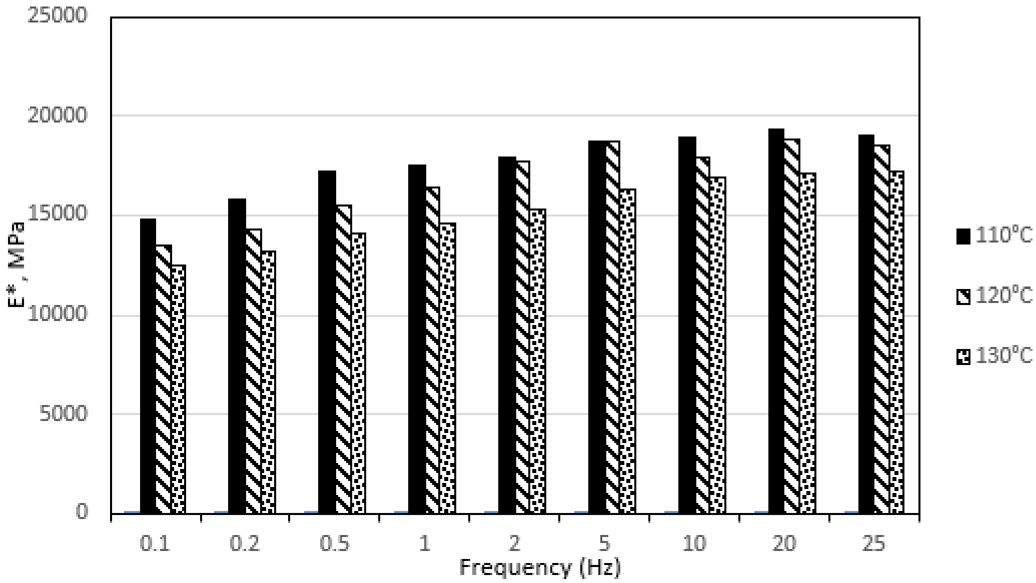
Mixture level-1 data inputs
The dynamic modulus inputs for different asphalt mixtures are shown in Table D-2.

Table A-4. Dynamic modulus of asphalt mixtures in different pavement layers.

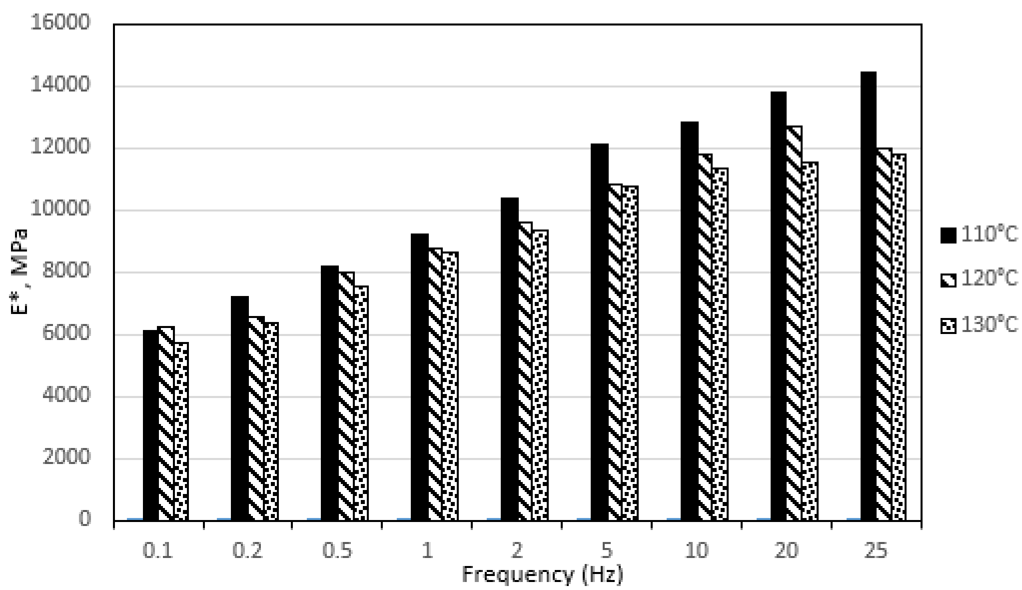
Dynamic modulus of HIR mix (psi)						
	Frequency (Hz)					
Temperature (°F)	0.1	0.5	1	5	10	25
14	2132499	2365736	2455186	2637494	2705520	2786409
40	1274491	1577604	1705456	1987229	2099961	2239597
70	465389	697235	797339	1078398	1206781	1379401
100	118114	202981	252804	404709	486601	609898
130	28435	50623	65020	115253	146316	198403
Dynamic modulus of D-mix with PG 64-22 (psi)						
	Frequency (Hz)					
Temperature (°F)	0.1	0.5	1	5	10	25
14	1751298	2076674	2203310	2461892	2557819	2670907
40	7922916	1135249	1290768	1650911	1799975	1987100
70	169227.5	323025.6	412935.4	676585.7	810901.1	1002730
100	23123.3	53610.5	75852.6	160645.3	215656.8	309022.5
130	3542.3	7965.7	11464.9	26980.4	38839.2	62101.8
Dynamic modulus of D-mix with PG 76-22 (psi)						
	Frequency (Hz)					
Temperature (°F)	0.1	0.5	1	5	10	25
14	2520385	2722695	2795313	2934413	2983130	3038682
40	1348727	1709169	1858036	2175995	2298423	2445601
70	256367	452466	556287	847530	990656	1190588
100	31996	58908	77229	144498	187781	261992

130	7208	10729	13062	21818	27808	38943
Dynamic modulus of BM-2 mix (psi)						
	Frequency (Hz)					
Temperature (°F)	0.1	0.5	1	5	10	25
14	2504737	2705062	2777478	2917199	2966520	3023069
40	1347969	1699086	1844327	2155694	2276201	2421692
70	271373	457767	560217	845515	985071	1179726
100	31920	59384	77993	145757	188989	262624
130	6571	10052	12371	21116	27111	38251

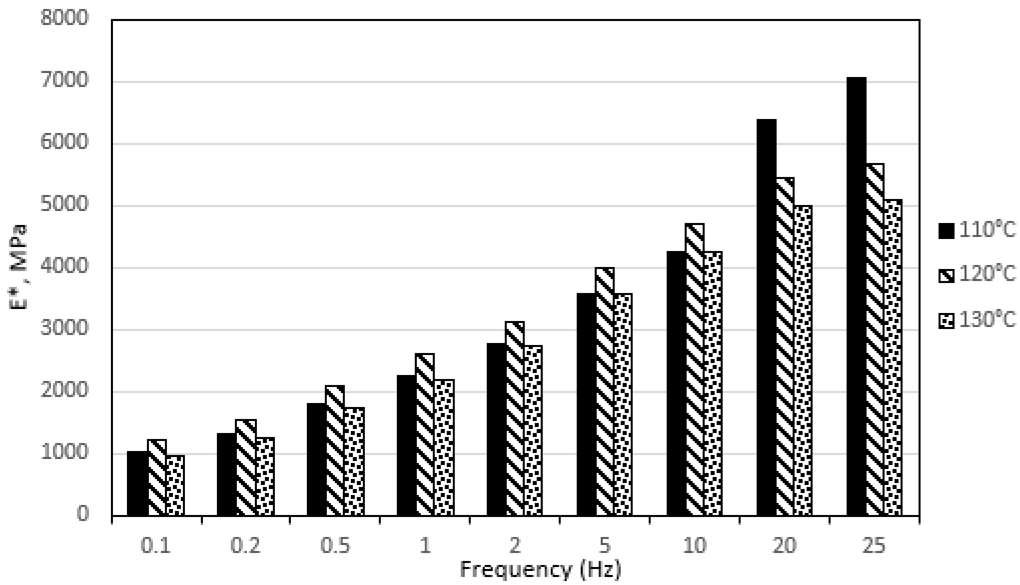
Additional figures in section 4.3



(a)

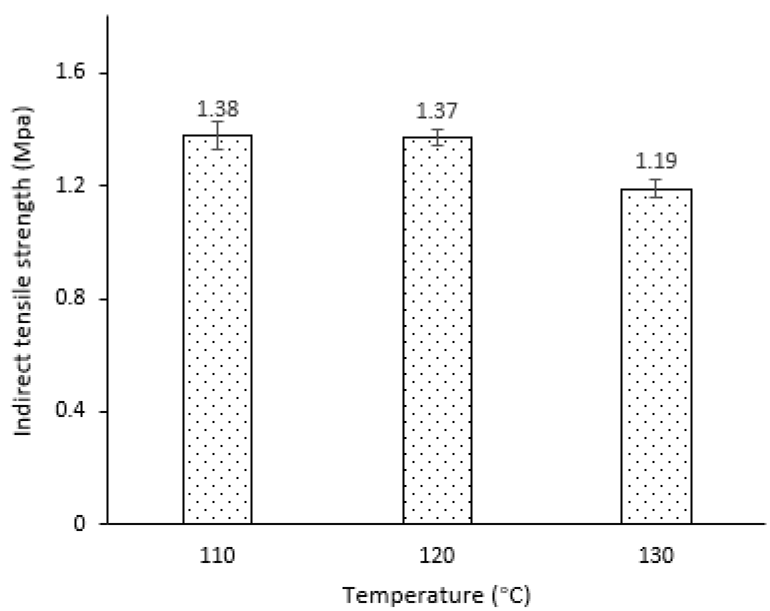


(b)

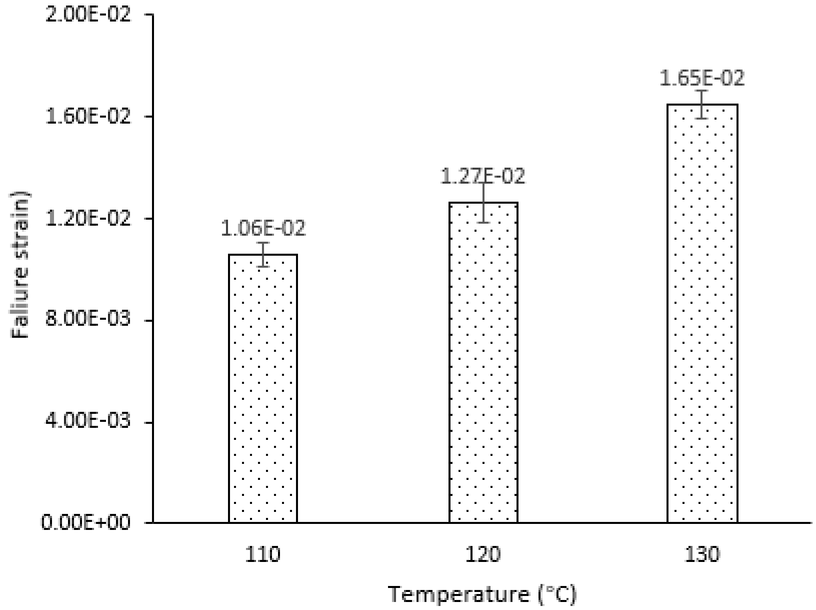


(c)

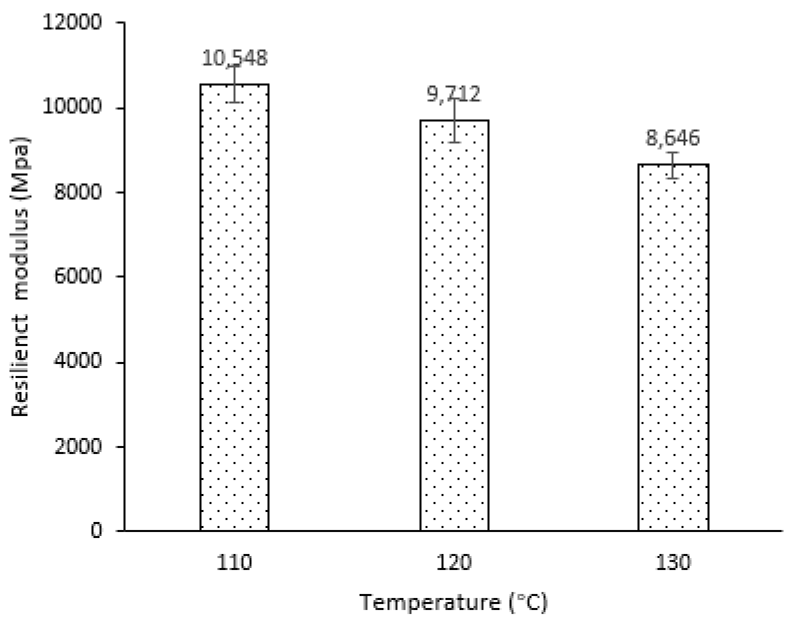
Figure A-19. Dynamic modulus of the HIR mixtures at different temperatures: (a)Tested at 4 °C, (b) tested at 20 °C, and (c) tested at 40 °C.



(a)

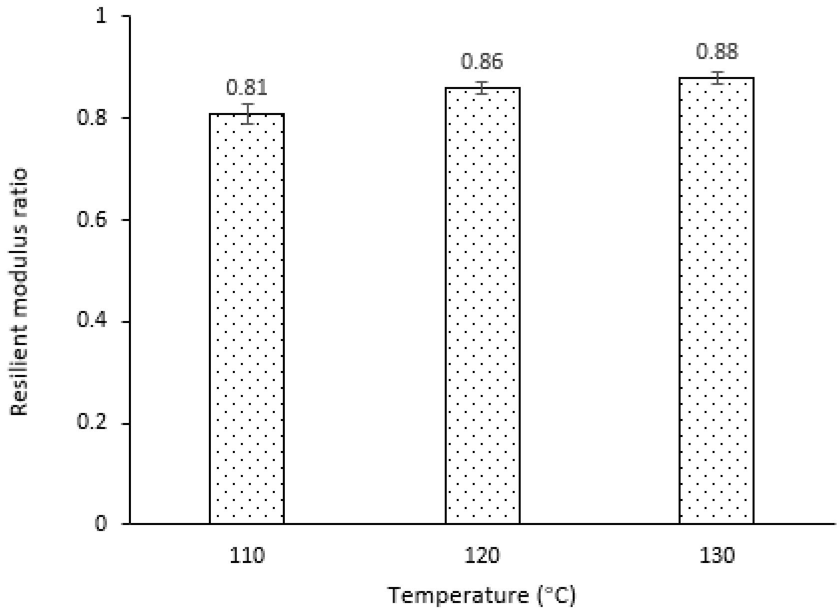


(b)

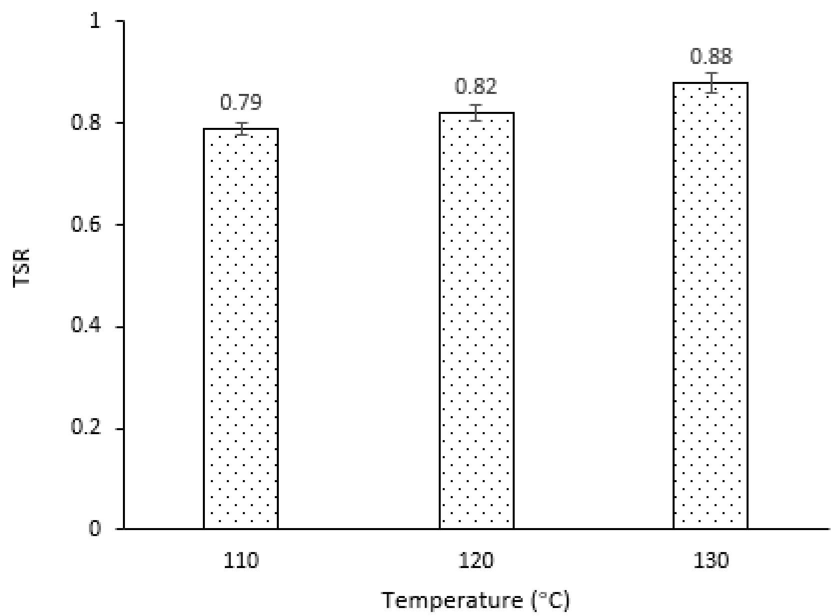


(c)

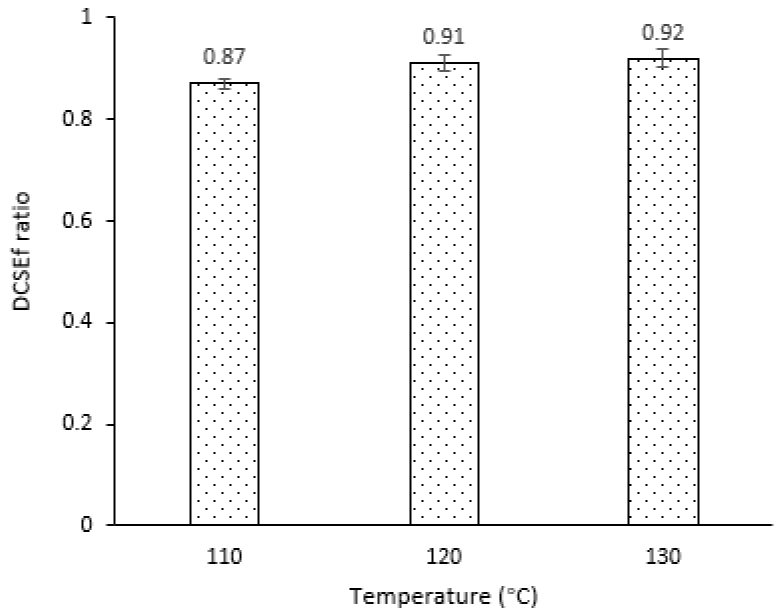
Figure A-20. Superpave IDT test results of HIR mixes at different temperatures; (a) Indirect tensile strength; (b) failure strain; (c) resilient modulus.


1
0.8
0.6
0.4
0.2
0
Resilient modulus ratio
0.81
0.86
0.88
110
120
130
Temperature (°C)

(a)


1
0.8
0.6
0.4
0.2
0
TSR
0.79
0.82
0.88
110
120
130
Temperature (°C)

(b)



(c)

Figure A-21. Moisture susceptibility test results of HIR mixes at different temperatures; (a) Resilient modulus ratio; (b) Tensile strength ratio (TSR) results; (c) Dissipated creep strain energy ratio (DCSEf).