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Cold Reclamation and Recycling Techniques to Achieve Perpetual Pavements

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16. Abstract (Limit: 250 words)

This study evaluates Cold In-place Recycling (CIR) for developing sustainable and cost-effective perpetual pavements. As part of the 2022 NRRA construction, four test sections were constructed at the MnROAD mainline section to utilize CIR and additionally assess the effects of incorporating rejuvenator in cold recycled asphalt materials. Two sections included bituminous layer over aggregate base, while the other two included bituminous overlay over stabilized full depth reclamation (SFDR) base layer. For two sections, rejuvenator was incorporated to evaluate its impact on the performance of the cold recycled (CR) layer. Laboratory tests conducted after one year of service showed that even though rejuvenator improved binder fatigue resistance, its benefits were less evident at the mixture level, where cracking resistance declined. Perpetual pavement analysis indicated bottom-up cracking potential in all test sections but suggested that reasonable adjustments to overlay or CIR thickness could achieve perpetual behavior. The contrasting outcomes between binder, mixture, and Falling Weight Deflectometer (FWD) testing highlight the need for further research to fully understand the effect of rejuvenator on the CR layer/material behavior. Additional testing, focusing on fatigue, rutting, and low-temperature performance, is recommended to refine rejuvenator use in CIR applications and optimize cold recycling techniques for perpetual pavement construction.

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EXECUTIVE SUMMARY

This report explores the use of cold reclamation and recycling techniques to develop perpetual pavements, focusing on sustainable, cost-effective, and environmentally friendly solutions for road rehabilitation.

At the project outset, a survey was distributed to NRRA agency members to gather information on existing rehabilitation practices. The survey addressed the use of cold recycling by various states, current pavement design methodologies for both perpetual and cold recycled pavements, treatment approaches for cold recycled layers, and methods for tracking the performance of cold recycled pavements. It also captured agency preferences for rehabilitating/reconstructing cold recycled pavement structures. Responses highlighted agency preferences for rehabilitating cold recycled pavements and revealed that CIR (Cold In-place Recycling), FDR (Full Depth Reclamation), and SFDR (Stabilized Full Depth Reclamation) were associated with favorable outcomes in terms of pavement longevity and performance. Interviews with representatives from Illinois, Mississippi, and North Dakota Departments of Transportation (DOTs) confirmed these observations and indicated growing interest in CIR and CCPR (Cold Central Plant Recycling) techniques, although these methods remain relatively new to some DOTs.

As part of the 2022 NRRA construction, four CIR test sections were built at the MnROAD mainline section. Test sections 2201 and 2202 featured bituminous layer over aggregate base, while test sections 2207 and 2208 included bituminous overlay over SFDR layer. For test sections 2201 and 2207, rejuvenator emulsion was added to engineered emulsion to evaluate its impact on the performance of the cold recycled (CR) layer.

After one year in service, cores were taken from the test sections for laboratory evaluation of the CR materials. Test included Marshall stability, low-temperature analysis, and Illinois Flexibility Index Tests (I-FIT). While rejuvenation showed potential for performance improvement during mix design stage, post-service testing indicated no significant improvement and, in some cases, a decline in cracking resistance. We evaluated the reclaimed asphalt pavement (RAP) binders that were extracted from RAP, together with the base binder that was used to produce emulsion, and base binders + rejuvenator. Tests included: rheological characterization, Multiple Stress Creep and Recovery (MSCR), and Linear Amplitude Sweep (LAS) tests on a Dynamic Shear Rheometer (DSR). Significant findings regarding materials recovered and extracted during construction (test section 2201-RAP and test section 2202-RAP) demonstrated similar stiffness and resistance to cracking and rutting. With regard to in-service binders, rejuvenator-treated test section 2201 showed superior fatigue resistance performance in comparison to test section 2202. However, a reduced resistance to rutting was observed for test section 2201 in comparison to test section 2202.

Falling Weight Deflectometer (FWD) testing was performed by MnDOT and MnROAD on the test sections at various periods. The overall results, on average, showed a better performance of the CR layer and pavement structure for test section 2201 compared with test section 2202 in terms of back-calculated stiffness, Surface Curvature Index (SCI), and peak deflection. Conversely, test section 2207 showed a worse performance compared with 2208.

Perpetual analysis results from both driving and passing lanes showed that all test sections are expected to experience bottom-up cracking at some point, particularly on already in-place layers prior to CIR application for sections 2201 and 2202 and on CIR and in-place layers for section 2207 and 2208. Additional perpetual analyses were performed to reassess these sections in terms of bottom-up cracking and quantify the adjustments in thickness required to match them more closely with or below the perpetual design limit. Through trial and error, the bituminous layer and CIR thicknesses were adjusted across all sections and the perpetual design thickness was determined. Results showed that perpetual behavior could be achieved by increasing overlay thickness by one to two inches, CIR depth by one inch, or a combination of both.

Due to inconsistencies among binder, mixture, and FWD testing results, definitive conclusions on the rejuvenator's efficacy remain unclear. One explanation could be that the rejuvenator improvement is evident when dealing with binder only. In the mixture phase, with only about 2 percent emulsion, this improvement is either diminished or rejuvenation could even result in some deterioration in mixture performance. These findings suggest a need for further testing on CIR mixtures containing rejuvenators, particularly for fatigue, rutting, and low-temperature performance. Additional studies and refinements to the mix design process are recommended to optimize the use of rejuvenators in CIR applications.

Chapter 1: Literature Review

1.1 Reclamation and Recycling Techniques

The roadway network has grown in the past decades, and the need for rehabilitation of existing asphalt pavements has increased. Because of the limited budget and rising energy prices, searching for cost-effective and efficient rehabilitation methods has become an important issue for agencies [1]. With the increasing availability of reclaimed asphalt pavement (RAP), decreasing availability of high-quality virgin aggregate that meets strict specifications, and the environmental impact of pavement materials, there is a need for alternative cost-effective, high-quality, and environmentally friendly materials [2,3]. In the past 25 years, asphalt recycling and reclamation techniques were proved to be the technically and environmentally preferred way of rehabilitating existing pavements [1]. Recycling and reclamation of existing asphalt pavements reduce the cost of construction, preserve the existing pavement geometry, conserve asphalt, aggregate, and energy and preserve the environment [4,5].

Asphalt recycling can be divided into three broad categories [1]:

- Cold Planing (CP)
- Hot In-Place Recycling (HIR)
- Cold Recycling (CR)

Within these broad categories, seven sub-categories can be defined, as shown in Figure 1-1. These methods can be used in conjunction with one another or other maintenance and rehabilitation techniques such as asphalt overlay or surface treatment to address a wide variety of pavement defects [5]. Table 1-1 provides a general guideline for selecting recycling and reclamation methods [1]. The abbreviations are defined in Figure 1-1.

Figure 1-2 shows the recycling and reclaiming techniques suitable for different Pavement Condition Index (PCI) ranges [6]. Surface treatments are generally used for pavements that are in very good condition. HIR and Cold In-place recycling (CIR) are typically used to treat structurally sound pavements that need ride quality improvements. FDR/SFDR are generally utilized to rehabilitate poor-condition pavements that require structural repair.

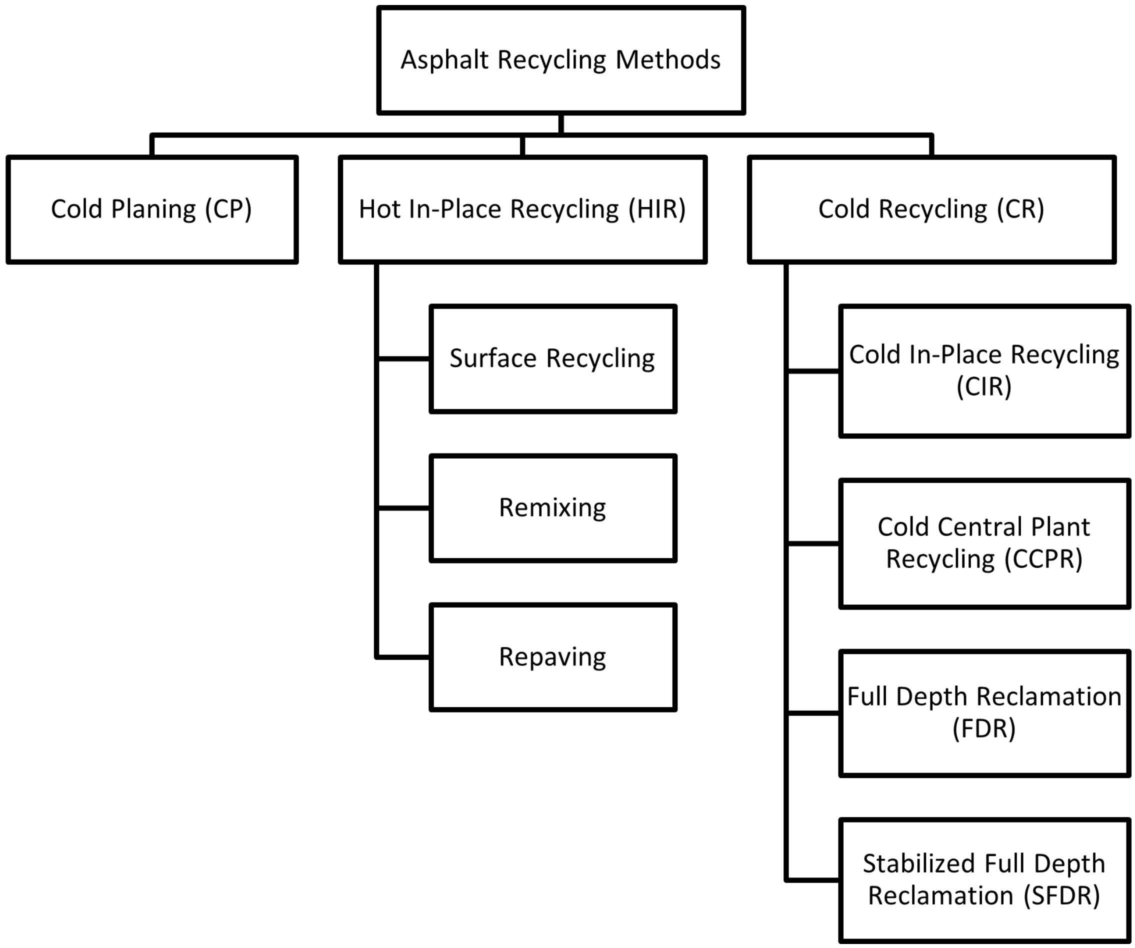
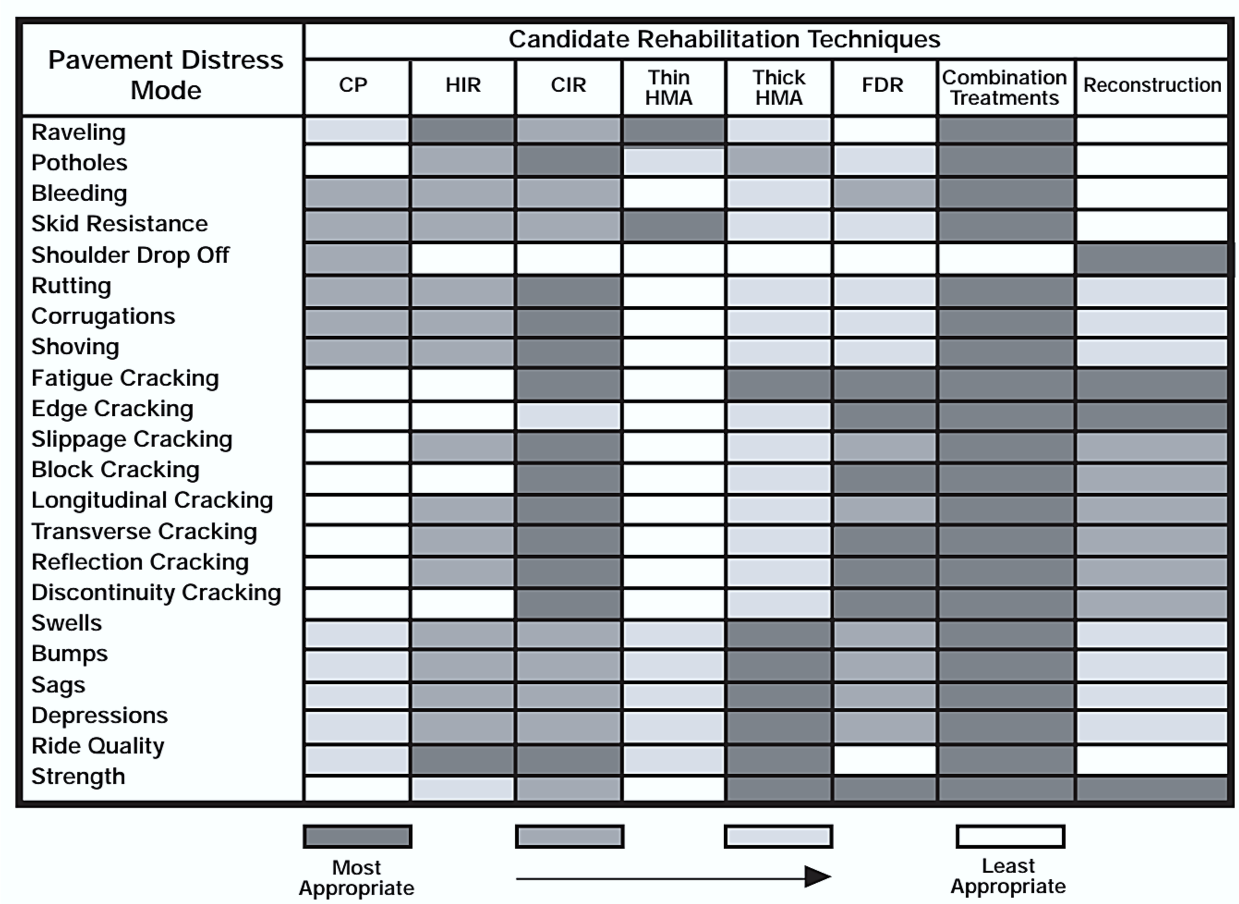


Figure 1-1 Asphalt recycling methods

Table 1.1 Recycling and reclamation methods selection guideline [1]



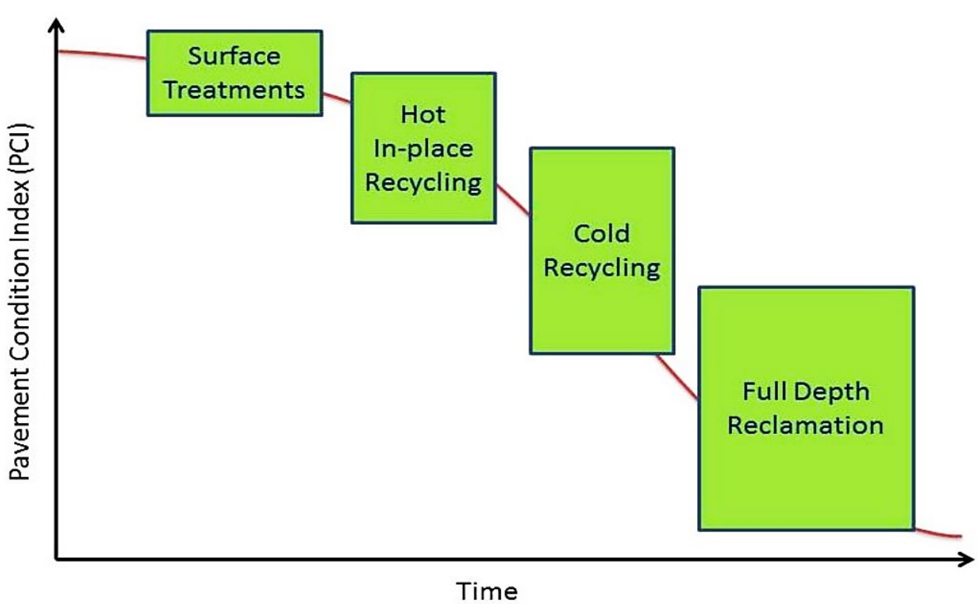


Figure 1-2 Pavement condition index and type of asphalt recycling and reclamation techniques [6]

Asphalt recycling and reclamation have the following benefits:

- Enhancement of road performance with superior materials and increase in workers' safety in cold recycling methods [5]
- Preservation of existing roadway geometry and pavement profile, and cross-slope correction [1]
- Reduction in the cost of new materials, transportation, and energy use make them less expensive than the traditional methods [5]
- Reduction in the use of non-renewable materials, energy sources, and landfilling [1]
- Fuel consumption and greenhouse gas emission reduction due to the elimination of material transportation for in-place methods [2]
- Reduction in user delays and traffic flow disruption due to the decreased construction time [1]
- Elimination of material transportation time for in-place methods [5]
- Mitigation/elimination of reflective cracking with some techniques such as FDR/SFDR [1]

The objective of this research is to assess the utilization of the existing cold recycling methods to achieve perpetual pavement. The cold recycling methods will be discussed in detail in this chapter.

1.1.1 Cold In-place Recycling (CIR)

CIR is one of the cold recycling methods of rehabilitation that is completely performed on the site. CIR involves reusing the existing asphalt pavement materials without applying heat [4]. CIR generally uses 100 percent of the RAP produced during the recycling process [1]. Asphalt emulsion or foamed asphalt, proportioned as a percentage of the weight of the RAP, is added as a recycling agent. Except for recycling agents, no transportation of materials is required; therefore, hauling costs are relatively low in the CIR technique. Compared to cold central plant recycling (CCPR), CIR is more suitable for roads located far from a central plant [4].

CIR is used when non-load-related distresses are high in number and severity and/or extend down from the pavement surface. It can be combined with asphalt overlay to address load-related distresses [2], and can treat most pavement distresses, improve ride quality, eliminate dust problems, minimize hauling, and be used for pavement widening [4].

The typical treatment depth for CIR is 3 to 4 inches [5]. Depths as thin as 2 inches are possible with good underlying support. Also, treatment depths up to 5 inches are possible provided proper compaction can be achieved [1]. Thicker depths may be possible with multiple passes [2] and using chemical additives such as cement, lime, kiln dust, or fly ash to improve the stability of the treated layer and moisture damage resistance [1]. Lime or Portland cement can be added in dry or slurry form. The slurry method eliminates dust problems and allows better control of the amount of the recycling agent [1]. It should be noted that cold refers to the temperature of milling and aggregates. CIR can still use hot binders with the foamed asphalt process [2].

1.1.1.1 Project Selection Criteria

Although CIR can address most pavement distresses, cracked, structurally sound pavements that have well-drained bases are the best candidates [5]. The following pavement distresses can be corrected or mitigated by CIR [1]:

- Raveling
- Potholes
- Bleeding
- Skid resistance
- Rutting
- Corrugations
- Shoving
- Fatigue, block, and edge cracking
- Slippage, longitudinal, and transverse thermal cracking
- Reflection and discontinuity cracking
- Poor ride quality caused by sags, bumps, swells, and depressions

In general, to eliminate pavement distresses, the root cause(s) of them need to be addressed during the rehabilitation process. Table 1-2 shows a detailed description of distresses that can be corrected or mitigated by the CR technique [5].

Table 1.2 CR applicability [5]

Condition	Distresses	CR Applicability
Surface Defects	Raveling	Yes
	Potholes	Yes
	Bleeding	Yes
	Skid Resistance	Yes
Deformations	Shoulder Drop Off	No
	Rutting - Wear	Yes
	Rutting - Mix Instability	Possible, see note a
	Rutting - Deep Structural	Possible, see note b
	Corrugations	Yes
	Shoving	Possible, see note a
Load Associated Cracking	Fatigue – Bottom Up	Possible, see note c
	Fatigue – Top Down	Possible, see note c
	Edge	Possible, see note d
	Slippage	Possible, see note e
Non-Load Associated Cracking	Block	Yes
	Longitudinal	Yes
	Transverse	Yes
	Reflective	Yes
Combined Cracking	Joint Reflection	Possible, see note f
	Discontinuity	Yes

Condition	Distresses	CR Applicability
Base/Subgrade Deficiencies	Swells, Bumps, Sags Depressions	Possible, see note g
Roughness	Ride Quality	Yes
Other Criteria	All Levels of Traffic	Yes, see note h
	Rural	Yes
	Urban	Yes, see note i
	Stripping	Possible, see note a
	Poor Drainage	No, see note j

Notes:

a) It can be corrected by adding cement, lime, and/or new aggregate but it needs to be verified by a mix design.
b) Not with CIR but it can be addressed with CCPR and correction of the underlying layers.
c) Ensure that structural requirements can be met. An asphalt overlay may be needed.
d) Need to provide shoulder confinement after performing.
e) If treatment depth exceeds the slippage plane.
f) It may not correct, but it will mitigate.
g) CIR may not correct but may mitigate. However, it can be addressed with CCPR and correction of the underlying layer.
h) If correct pavement structural design is undertaken as part of the process to ensure that the effects of future traffic are considered and if the CR mixture has sufficient early and long-term strength. Additives such as cement or lime may be needed to improve early strength gain.
i) Geometric constraints may affect the type of recycling units used or whether CIR or CCPR is used.
j) Poor drainage must be corrected for CR, or any other pavement treatment, to ensure adequate performance.

Typically, at least 70 percent of the existing asphalt layer should be treated to mitigate reflective cracking. In general, a minimum of 1 inch and preferably 2 inches of asphalt or 6 inches of aggregate base should remain to support the weight of the recycling train. Shallow treatment depth will increase the probability of the reoccurrence of reflective cracking. Wear rutting is easily treated with CR. Minor instability rutting can also be treated if appropriate recycling agents, modifiers, and/or granular materials are utilized [1].

At first, CR was used on low to medium traffic volume roadways, but it is now routinely used on higher traffic volume roadways, including interstate pavements [1]. Agencies are less likely to consider CIR for roads with traffic less than 5,000 average annual daily traffic (AADT). This may be due to the lack of proper support for recycling equipment on thinner low-volume roads [2].

There are no climate restrictions for CIR, so it can be used in all four climate zones, including dry freeze, dry no freeze, wet freeze, and wet no freeze [5]. However, CIR is preferred in hot, dry climates because wet, cool weather lengthens the curing time in emulsion-based CIR [2].

1.1.1.2 Materials and Mix Design

Based on a survey in 2011, state agencies usually do not perform mix designs for their CIR projects. When they do, they use Marshall, Superpave, Wirtgen, or other methods [2]. Some AASHTO standards have been developed recently. These include:

- **AASHTO M 352**: *Standard Specification for Materials Used in Cold Recycled Mixtures with Emulsified Asphalt*
- **AASHTO R 117**: *Standard Practice for Emulsified Asphalt Content of Cold Recycled Mixture Designs*
- **AASHTO MP 38**: *Standard Specification for Mix Design of Cold Recycled Mixture with Foamed Asphalt*
- **AASHTO PP 94**: *Standard Practice for Determination of Optimum Asphalt Content of Cold Recycled Mixture with Foamed Asphalt*

Most CIR mix designs typically consist of the following steps [5]:

- Taking samples from the existing asphalt pavement
- Determining binder content, gradation, and percent crushed particles of the extracted aggregate
- Determining aged binder properties, if desired
- Crushing materials to produce RAP and determining the gradation
- Selecting the type and grade of bituminous recycling agent (emulsion or foamed asphalt)
- Selecting type and amount of recycling additives if required (corrective aggregate, cement, etc.)
- Preparing and testing specimens
- Establishing job mix formula

Field adjustments to the recycling agent design content can be made in order to obtain optimum performance, if necessary.

For a proper mixture design procedure for CIR recycled materials, several issues need to be considered, including [7]:

- RAP gradation: fine, medium, or coarse
- Curing conditions: open or sealed, temperature, and length of curing
- Testing conditions: as cured or wet (soaked or vacuum saturation)
- Testing procedures: Expansion Ratio and Half-life for foamed asphalt, Marshall, Indirect Tensile Strength (ITS), Tensile Strength Ratio (TSR), Unconfined Compressive Strength (UCS), modulus
- Design criteria: minimum strength (wet or dry) – retained strength (wet strength divided by dry strength)

The most common mix design methods include [7,8]:

a. Emulsions

   - MnDOT: 4-in. gyratory with 30 gyrations
   - Caltrans: 75 blow Marshall

- Iowa DOT: 4-in. gyratory with 30 revolutions
- SemMaterials: 6-in. gyratory with 30 gyrations
- Wirtgen: 75 blow Marshall
- Ontario Ministry of Transportation (MTO): 75 blow Marshall

b. Foamed asphalt

- MnDOT: 4-in. gyratory with 30 gyrations
- WisDOT: gyratory with 30 gyrations or 75 blow Marshall
- Iowa DOT: 4-in. gyratory with 25 revolutions
- Wirtgen: 75 blow Marshall
- Ontario MTO: 75 blow Marshall

The mix design procedures include the curing process to represent field conditions. Some of the mix design methods vary in their curing procedure [7,8]:

a. Emulsions
    - MnDOT: Cure unsealed samples at 140 °F for 16 hr. min, 48 hr. max to constant weight
    - Caltrans: Cure unsealed samples at 140 °F to constant weight
    - Iowa DOT: Cure unsealed samples for 48 hr. at 140 °F
    - SemMaterials: Cure unsealed samples for 72 hr. at 140 °F
    - Wirtgen: 72 hr. at 104 °F. Specimens are compacted at the equilibrium/field moisture content and cured in sealed containers for 48 hr. at 104 °F before testing for high traffic (> 5 MESALs) roads.
    - Ontario MTO: 48 h at 140 °F, soaked for 24 hr. at 77 °F, or vacuum saturated for 60 min at 50 mmHg.
b. Foamed asphalt
    - MnDOT: 72 h at 104 °F
    - WisDOT: follows AASHTO PP 94 standard
    - Iowa DOT: 72 h at 104 °F
    - Wirtgen: Same as for emulsions
    - Ontario MTO: Same as for emulsions

For specimens compacted at the equilibrium/field moisture content, the curing condition typically results in a dry specimen, except for the Wirtgen approach.

Various properties of the CIR mixtures are determined once the specimens have cured [7,8]:

a. Emulsions
    - MnDOT: Marshall stability at 104 °F (> 1,250 lb.) and Tensile Strength Ratio (TSR) (> 0.7), Indirect Tensile Test (IDT)
    - Caltrans: Marshall stability (dry) at 104 °F (> 1,250 lb.) and TSR (> 0.7)
    - Iowa DOT: Marshall stability at 100 °F (> 1,000 lb.)
    - SemMaterials: ITS, resilient modulus, and modified cohesiometer
        - 35 to 40 psi minimum (dry)

    - 20 to 25 psi minimum (wet)
    - TSR > 0.7
    - 120 to 150 ksi minimum for resilient modulus at 77 °F
  - Ontario MTO: ITS (dry and wet)
    - 50 psi minimum (dry)
    - 25 psi minimum (wet)
    - TSR > 0.5

b. Foamed asphalt:
  - MnDOT: Cured Marshall stability at 104 °F (> 1,250 lb.) and TSR (> 0.7) after soaking based on cured stability, IDT
  - WisDOT: follows AASHTO PP 94 standard
  - Ontario MTO: ITS (dry and wet)
    - 50 psi minimum (dry)
    - 25 psi minimum (wet)
    - TSR > 0.5

1.1.1.3 Construction

Generally, CIR construction consists of the following steps [4]:

- Preparation of construction area
- Milling the existing asphalt pavement
- Addition of the recycling agent and other additives (if needed)
- Placing the recycled mixture
- Curing of the recycled layer (if needed)
- Compaction and placement of surface course

CIR uses several pieces of equipment that form a continuous train, including tanker trucks, milling machines, crushing and screening units, mixers, pavers, and rollers. There are various types of CIR trains that are different in how the RAP is removed and sized, how recycling agents are added, and how they are mixed and compacted [1]. These include single-unit trains, two-unit trains, and multi-unit trains [5].

In the single-unit CIR train, the removal of RAP, mixing, and addition of recycling additive is performed in a milling machine. The material placement is performed by a screed attached to the back of the unit. Two-unit CIR train consists of a full-lane milling machine and a mix paver. The milling machine removes, sizes, and transfers the RAP material into the mixing machine. The mix paver mixes the materials and performs the placement and initial compaction of recycled materials. The multi-unit train consists of a very large full-lane milling machine, a screening and crushing unit, and a pugmill mixer. The milling machine removes the RAP, the screening and crushing unit performs the final sizing of the RAP, and materials are added and mixed in the pugmill mixer. The final mix is deposited in a windrow and placed with conventional bituminous pavers [1]. Usually, One or two nurse trucks in front of the profiler/mixer provide a continuous flow of liquids (water and recycling agent) for the mix [2]. Figure 1-3 shows the typical equipment in the CIR recycling train. Figure 1-4 shows the components of a pavement profiler and mixer.

Figure 1-3 Typical CIR recycling equipment [2]

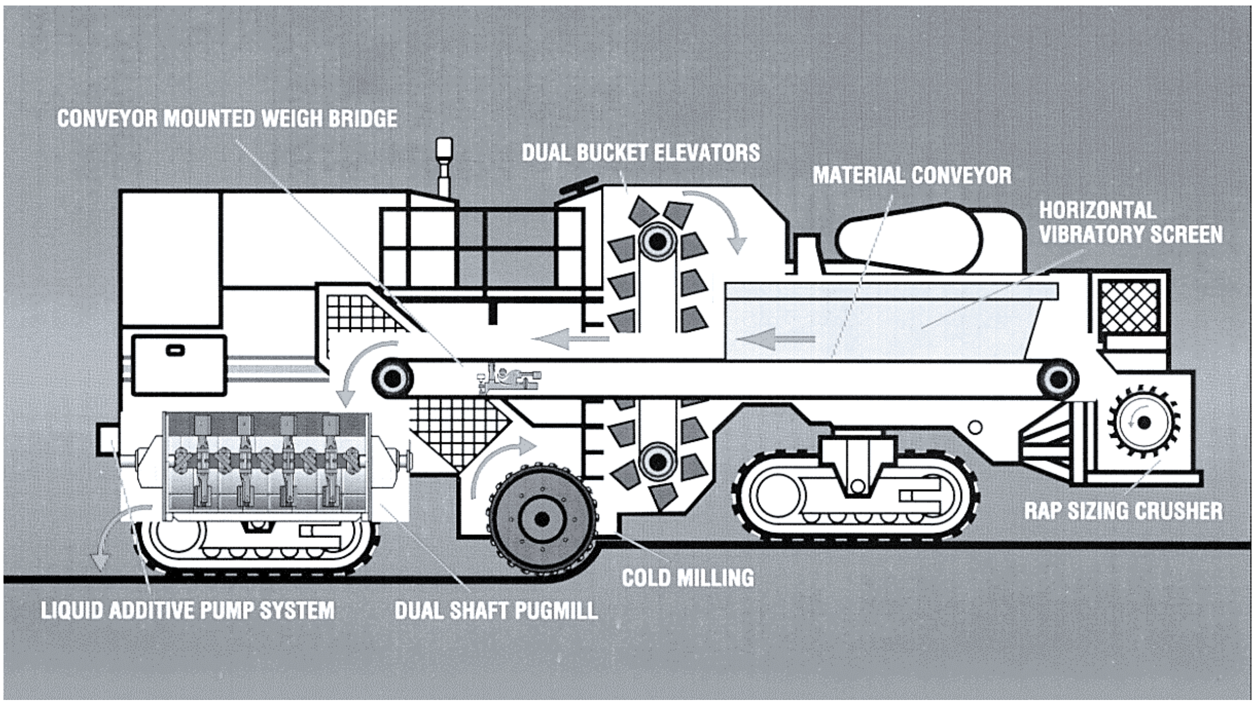


Figure 1-4 Pavement profiler and mixer components [4]

Compaction of cold recycled material usually needs more energy than conventional bituminous due to high internal friction between mixed particles, higher viscosity of aged binder, and colder compaction temperatures. Compaction usually is performed with a large pneumatic-tired roller and vibrating steel drum rollers. Three methods are available for determining the target density for compaction of CR mixtures: percent of laboratory compacted density, percent of field compacted density, or percent of control strip density. Target densities for recycled mix compaction are usually established by using rolling control strips or by the field compaction of density specimens using Marshall, Proctor compactors, or gyratory compactors. Since it is generally not possible to obtain cores during construction, the compacted density is measured using a nuclear density gauge. Backscatter measurement is typically used for control strips, but for density checks for specification compliance, direct transmission measurement is preferred. The density obtained will be a "wet density" since conversion to a true "dry density" by the gauge is not possible with CR mixtures. This is because the gauge cannot distinguish between the construction water and emulsion water, and therefore, the calculated dry density will be misleading. Typical specification requirements using field compacted target densities are 93 to 96 percent compaction on a wet density basis. Specifications typically require 96 to 97 percent of the target density using the control strip [1]. After compaction, the road can be opened to traffic shortly (typically less than an hour) [5].

1.1.1.4 Performance

The expected service life of CIR with a bituminous overlay is about 7 to 20 years which is equivalent to the service life of thick bituminous lift [1]. The expected service life of CIR with surface treatment is 6 to 10 years [1]. The expected service life and performance of CIR projects highly depend on the treatment depth and type and depth of asphalt overlay [1]. Based on a study in Ontario, the overall performance of the CIR project was reported to be satisfactory and minimal problems were encountered. FWD results showed that the strength characteristics of CIR rehabilitated pavement have improved in time, and significant improvements in ride quality were observed after rehabilitation. Ride quality measurements of CIR rehabilitated pavements were concluded to be better than those with conventional rehabilitation methods [9]. The long-term field performance of CIR projects in Nevada showed that CIR is an effective and recommended technique for low and medium-traffic roads that produces a more stable and flexible base, resulting in reduced reflecting cracking, thermal cracking, and rutting. It was also concluded that the CIR process also results in a higher and more-stable Pavement Serviceability Index (PSI) [10].

Performance assessment of CIR projects in northwestern Pennsylvania indicated that CIR provides service lives up to 160 percent of the 10-year design life provided by conventional mill and overlay projects in the same area and performs two to three times better than conventional overlay projects in resistance against reflection cracking. With good construction controls, the recycled material from the CIR process can develop stiffness values similar to conventional bituminous materials. In addition, the CIR material, like other coarse-grained materials, appears to be stress-sensitive, which means its stiffness increases with the increase in traffic load. This characteristic can be an important factor in the observed good performance and reflection cracking control. Back-calculated modulus data, however, indicated a gradual decrease in material stiffness over time which becomes significant ten years after construction [11].

Ogbo et al. [12] performed pavement analysis on various kinds of cold recycled mixtures made of three stabilizing agents (Portland cement, foamed asphalt, and emulsion) that were assessed separately and in combination. The pavement analysis was carried out to compare pavement structures with equal structural capacity utilizing measured resilient modulus values as input in MnPAVE. The results of this investigation demonstrated that while different material combinations may produce equal stiffness when examined at a single temperature, other performance parameters like cracking resistance may vary. The study demonstrated the reliance of MnPAVE analysis on the supplied stiffness values and may assume that the behavior of all material combinations would be identical.

Kuchiisi et al. [13] compared the measured and predicted strain of two sections where one section comprised of a thin layer of gap-graded asphalt mix of only 30 mm and a 250mm base layer consisting of

2 percent foamed asphalt and 1 percent hydrated lime, and the other section comprising of 125mm asphalt concrete together with a 250mm base layer containing 3 percent foamed asphalt and 2 percent cement active filler. Both sections bonded with a semi-infinite layer comprised of the remaining infrastructure. FWD load applications were conducted along the longitudinal direction of the asphalt cut for in-situ strain measurements, while strain gauges were employed for the field strains. The gap-graded asphalt mix section was considered linear elastic, while the 125mm asphalt concrete section was considered nonlinear elastic. KENLAYER and AEMC software were used to compare the results of the predicted and measured strains. Figure 1-5 shows the representative sections and the position of the strain gauges (SG). According to the comparison of predicted and measured strains, a partially adherent scenario in the simulations would more accurately depict the pavement response for the gap-graded asphalt mix section. On the other hand, calculated strains on the 125 mm asphalt concrete structure differ from the measured strains. Temperature influence, material mechanical behavior, and inherent field measurement variability could all have contributed to the disparity between measured and calculated strains. According to this study, treating this material as either linear or nonlinear elastic results in noticeably different stresses and strains within the pavement structure. The simulations display reduced vertical stresses on top of the remaining structure and lowered horizontal strains in the asphalt and base layers for the nonlinear situation. However, it was suggested that modeling such mixture layers as a nonlinear elastic material could lead to even more precise material characterization and better-designed pavement structures. Also, the resilient modulus model type was found to affect the pavement response. The confining pressure and Pezo's model, respectively, present the lowest and highest horizontal strains within the RAP-modified layer. Because Pezo's model provides a better material characterization, the authors recommend using it in pavement analysis simulations to account for the RAP-modified layer nonlinear elastic behavior.

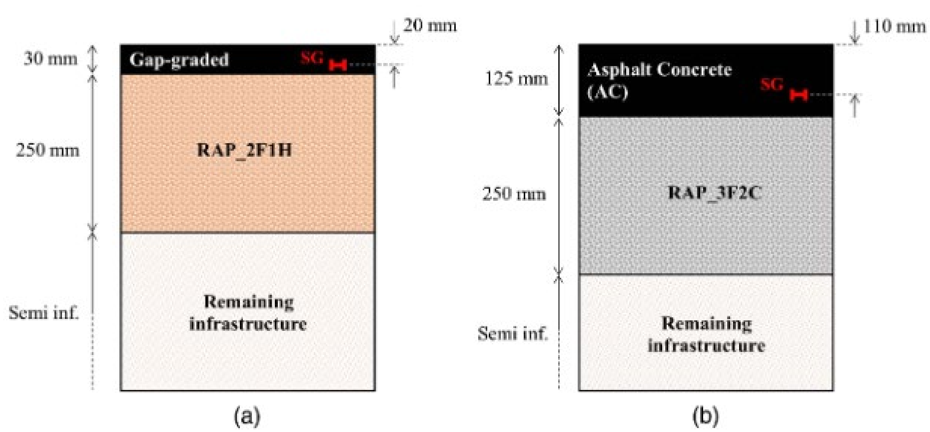


Figure 1-5 Test section thickness and layer combination used in Kuchiisi et al. study (SG: strain gauge) [13]

Preti et al. [14] simulated a pavement structure made up of a cold recycled mixture treated with cement and foamed asphalt to better understand the impact of cold recycled layers on the overall pavement response to traffic loading in terms of pavement deformation accumulation on the surface. With the use of the Mohr-Coulomb failure criterion and the perfectly plastic flow rule, this study suggested using elastic-perfectly plastic models for subsurface materials in the design and analysis of flexible pavement systems. In addition, the effect of temperature on the mechanical response of cold recycled mixtures was demonstrated, which also integrated a realistic temperature distribution with depth into the multilayer pavement model. It was established that this effect could not be ignored for a proper evaluation of the pavement. Comparable rutting performance of cold recycled treated base layer to those using traditional granular base layers was presented in this study, and it was concluded that the CR layer placed above crushed aggregate granular material in the pavement structure shows very positive results. As a case study in evaluating laboratory performance and structural contribution of cold recycled versus hot mixed intermediate and base course asphalt layers based on simplified viscoelastic continuum damage (S-VECD) fatigue results using the D^R (damage rate) failure criterion, Nemati et al. [15] concluded that CR mixtures are anticipated to have better fatigue resistance than bituminous mixtures.

The long-term performance of cold recycled pavements in Nevada with two methods, including CIR followed by a bituminous overlay and surface treatment for high-volume roads and CIR only surface treatment for low-volume roads using three emulsion technologies, including CMS-2S (a cationic, medium-setting asphalt emulsion), Reflex (a solvent-free emulsion), and PASS (a polymerized asphalt surface sealer emulsion), indicated that major distresses in CIR pavements are transverse and longitudinal cracking. Most surface-treated pavements and half of the bituminous overlaid pavements experienced transverse cracking. One-third of bituminous overlayed CIR pavements and two-thirds of surface-treated pavements experienced longitudinal cracking. Although bituminous-overlaid CIR pavements experienced significantly less rutting and roughness, rutting and roughness reduction of surface-treated CIR pavements were not significant. CIR pavements with CMS-2S and PASS emulsion perform remarkedly better than those treated with Reflex emulsion on low-volume roads. CIR pavements with CMS-2S emulsion and bituminous overlay of more than 3 inches had more service life than those with less than 3 inches of bituminous overlay. In conclusion, CIR pavements with bituminous overlay and surface treatment on high-volume roads performed much better than CIR pavement with only surface treatment on low-volume roads. It was also concluded that the thickness of the bituminous overlay crucially affects the long-term performance of CIR pavements [16].

1.1.2 Cold Central Plant Recycling (CCPR)

CCPR is one of the cold recycling rehabilitation techniques in which newly milled or existing RAP materials are combined with foamed or emulsified bitumen, additives, and/or recycling agents in a central cold mix recycling plant to produce a recycled pavement material that can be placed by conventional asphalt paving equipment [17]. The stationary plant can be a specifically designed plant or a stationary configured CIR train without the cold milling unit [5].

CCPR is suitable for locations where in-place recycling of pavement is not an option due to logistical reasons, high amounts/rates of production are required, and close control of the mix design is needed [4,5]. Also, because CCPR is not performed in place, base improvement can be made using other procedures, such as FDR, prior to the placement of CCPR materials [17,18]. CCPR can be used to provide a base course for reconstruction or new construction projects [17]. One of the advantages of CCPR is that significant structural improvement can be made without changing the vertical or horizontal geometry of the roadway [4]. The typical layer thickness for CCPR is 2 to 6 inches [1]. The total thickness of the recycled layer can be increased by using multiple lifts [17]. In addition to emulsified asphalt, foamed asphalt, cutbacks, fly ash, hydrated lime, and Portland cement can be used for the stabilization of recycled material [4].

Typically, asphalt recycling occurs at a central stationary cold recycling plant where a stockpile of RAP is available. However, there are other variations, such as onsite CCPR and imported CCPR. In onsite CCPR, milled RAP from a roadway project is transported to the processing site, processed, and then the CCPR mix is returned to the roadway for placement and compaction. In imported CCPR, milled RAP from a roadway project is transported to the processing site and then transferred to be placed in a different roadway. In central facility CCPR, RAP from various projects transports to a central plant, stockpiled, and/or transported to other roads for placement and compaction [5].

1.1.2.1 Project Selection Criteria

CCPR can correct almost all types of pavement distress except fatigue cracking and those caused by base failures. Correcting underlying materials is necessary to address fatigue cracking and base-related failures. Ride quality and reflection cracking are among the most important distresses that can be corrected with CCPR [4].

1.1.2.2 Materials and Mix Design

The mix design for CCPR is the same as for CIR. Adjustments may be required in the field to the recycling agent design content to obtain optimum performance.

1.1.2.3 Construction

CCPR construction usually consists of five basic steps [1]:

- Removal of the existing pavement
- Crushing and stockpiling
- Mixing and adding new aggregate and/or recycling additives
- Laydown and compaction
- Placing surface course as required

Detailed CCPR construction steps are shown in Figure 1-6.

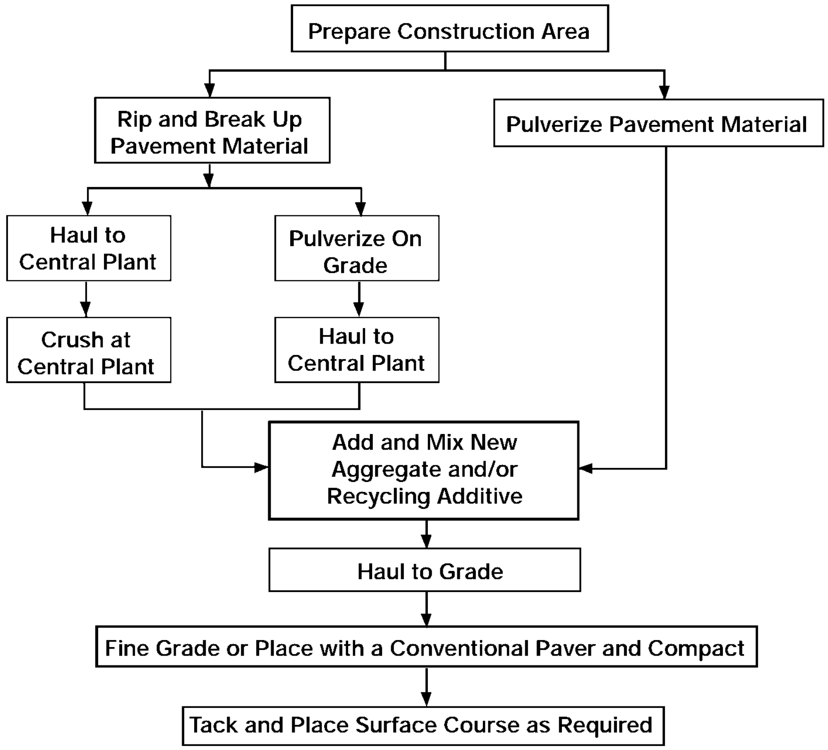


Figure 1-6 CCPR construction sequence [1]

In the first step, the existing pavement is ripped, scarified, pulverized, or milled to the desired depth. The existing pavement is either ripped and hauled to the central plant for crushing and sizing or ripped, broken, and pulverized in place and then transported to a central plant for mixing. The first method results in better control of RAP gradation and prevents oversizing. The second method needs special equipment and a considerable amount of traffic control. The third option is to cold mill the existing pavement and transfer the RAP to the central plant. This method has a high production rate and provides better control of the removal depth but may result in RAP oversizing [4].

Crushing and screening units reduce the size of broken pavement to acceptable limits. The RAP materials are stockpiled for mixing or future use. Drum, batch, or typically continuous plant may be used for mixing [4]. New aggregate is also collected at the plant site to be used if required. The CCPR plant consists of several cold feed bins for the RAP and new aggregate, a belt scale, an accurate computer-controlled liquid recycling agent, additives, a water system connected to a pugmill where materials are mixed, and a hopper for storage and loading. If the CCPR mix is stockpiled for future use, a conveyor/belt stacker is used [1]. CCPR mix is transported with haul trucks to the site, where the existing pavement is cleaned and tacked prior to the placement of the mix. Usually, the CCPR mix is placed using asphalt pavers, but a motor grader could also be used if smoothness is not important [5]. The CCPR mix is then compacted using large-sized rubber-tired rollers and vibratory steel-wheel rollers. Although for some very low-traffic roadways a single or double seal coat is sometimes enough, the CCPR mix is generally overlayed with bituminous [4].

1.1.2.4 Performance

The expected service life of CCPR with surface treatment is 6 to 8 years, and the expected service life of CCPR with an HMA overlay is about 12 to 15 years which is equivalent to the service life of thick bituminous lift [1].

Performance evaluation of CCPR sections on I-80 and I-64 in Virginia indicated that when properly designed and constructed, CCPR performs similarly to conventional asphalt pavements and can be used on high-traffic volume roads [19]. Schwartz et al. [20] investigated the material properties of CCPR materials using a bituminous stabilizing agent for mechanistic-empirical design. Typical values of Dynamic Modulus (|E*|) and Repeated Load Permanent Deformation (RLPD) structural properties under field-mixed, field-compacted, and field-cured conditions via laboratory testing of field cores taken 12 or more months after placement was evaluated. One CCPR project was stabilized using emulsified asphalt, and two CCPR projects were stabilized using emulsified asphalt and cement. Performance predictions were performed using pavement mechanistic-empirical (M-E) design V2.0 with laboratory-measured material properties. It was concluded that the permanent deformation of CCPR mixtures might be similar to CIR and conventional base asphalt mixtures but slightly more than FDR mixtures. Typical dynamic modulus values of CCPR mixtures to use as an input in Pavement M-E design were suggested. The predicted rutting of cold-recycled sections fell within the acceptable limit. Only sections with a thin bituminous overlay exhibited poor rutting performance. Cold recycled sections with a bituminous overly of 2 inches or more resulted in predicted rutting comparable to those of conventional bituminous rehabilitated sections [20].

Vaitkus et al. [21] evaluated the performance of CCPR rehabilitated pavements using a falling weight deflectometer. The CCPR mix was stabilized with 3.1 percent foamed asphalt and 2.3 percent cement. The bearing capacity (surface modulus) of CCPR base pavement gradually increased over time up to 47 percent after four years of operation. This is while the M-E pavement design procedure was not able to address this increase in surface moduli in predicting the durability (the remaining life) of the pavement. It was concluded that the M-E design procedure needs to be revised to design CCPR accurately [21].

The effect of thin overlays on the structural performance of cold recycled base layer for high-volume roads in Alabama was evaluated by Rahman & Vargas-Nordcbeck [18]. Based on the three years of rutting and IRI data, CCPR sections resulted in more rutting than CIR sections, and sections stabilized using foamed asphalt resulted in more rutting than emulsified asphalt stabilized sections. IRI of untreated base sections was lower than the cold recycled section. The thin overlay surface of cold recycled bases was concluded to be more susceptible to deterioration of ride quality under heavy traffic. After the first year of service, no significant cracking was observed in the test sections except for a low-severity cracking on the foamed CCPR section [18].

Bowers et al. [22] investigated the negative consequences of stockpiling with respect to the mechanical properties of CCPR materials. They found that the CCPR mix becomes less workable, as defined by the number of gyrations required to compact a CCPR test specimen, as curing time increases. The indirect tensile strength and dynamic modulus decreased exponentially within the first three days of stockpiling and then reached a steady value. Based on this study, a CCPR mixture produced using foamed asphalt as the recycling agent and cement as the active filler can be stockpiled for no more than 24 hours [22].

1.1.3 Full Depth Reclamation (FDR)

FDR is an in-place rehabilitation technique in which the full depth of the existing bituminous layer and a predetermined thickness of the base or subbase is pulverized to produce homogenous base materials [1]. Since the base/subbase layer can vary in thickness and also may be contaminated by the underlying subgrade, it is recommended to limit the base/subbase inclusion to 1 to 2 inches only. The treatment depth of FDR is generally between 4 to 12 inches [1]. FDR is mainly used when a milling depth of over 6 inches and/or additional pavement strength is required. Achieving adequate compaction is reported to be difficult in lifts thicker than 12 to 14 inches [2].

FDR consists of pulverization of reclamation of existing pavement material, adding new materials if required, mixing, grading/shaping, compaction, and applying a wearing course [1]. FDR is used when the structural strength of the processed mixture is sufficient enough that enhancing with stabilizing additives is not necessary [23]. Crushed stone or gravel may be added to the mix to improve the gradation and strength of reclaimed materials. However, adding crushed stone or gravel may not be the most cost-effective method since the increased strength may not be enough or long-lasting [24]. Usually, FDR is used to rehabilitate roads that are in poor condition and encountered deeper structural deficiencies, but it can also be used to increase the structural capacity of pavements that are in good condition [5]. FDR addresses both pavement structural and base problems [24] and is suitable for addressing almost all forms of distresses [2].

FDR/SFDR use has grown significantly over the past decade in the United States. Figure 1-7 shows the use and the size (Lane-miles per year) of FDR/SFDR rehabilitation based on a survey in 2011 [2]. The numbers on the figure show the maximum acceptable traffic level (AADT) that each State considers for FDR/SFDR projects.

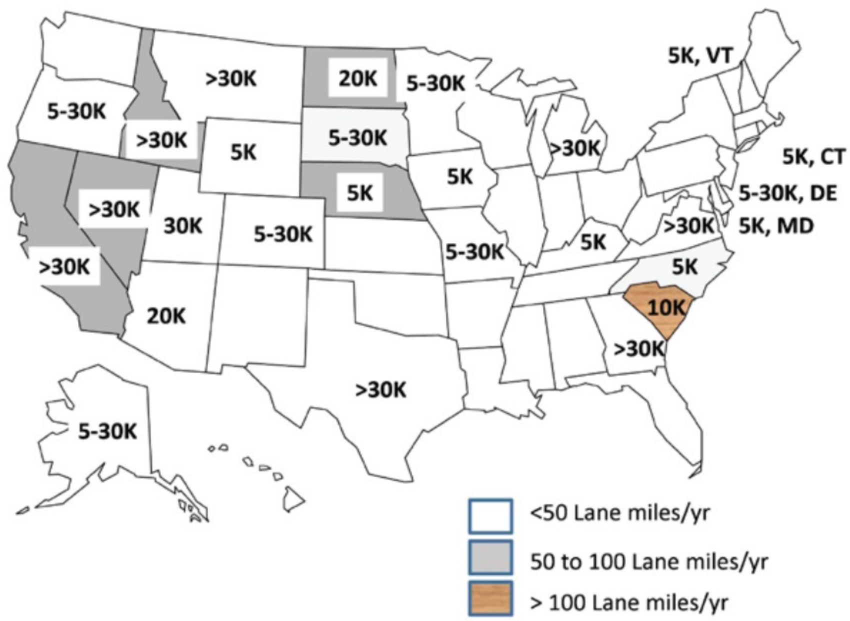


Figure 1-7 Size of FDR/SFDR (lane-miles per year) and maximum acceptable traffic levels per State [2]

FDR is a good alternative to traditional rehabilitation and reconstruction because it is fast, inexpensive, and provides up to 25 years of life extension [5,25]. The service life of FDR is usually restrained by the service life of the wearing course and not the FDR mixture itself [5]. FDR is also advantageous because it can improve the pavement structure without changing the geometry of the pavement and shoulder reconstruction, be used for pavement widening, and is environmentally desired [4].

1.1.3.1 Project Selection Criteria

FDR is typically used to rehabilitate roads that are in poor condition and encountered deeper structural deficiencies [5] and is suitable for addressing almost all pavement distresses [2]. The following pavement distresses can be treated by FDR [1]:

- Fatigue, edge, slippage, block, longitudinal, reflection cracking, and discontinuity
- Poor ride quality due to bumps, sags, swells, and depressions
- Rutting, corrugations, and shoving
- Loss of bonding and stripping
- Loss of surface integrity due to raveling, potholes, and bleeding
- Excessive shoulder drop-off
- Inadequate structural capacity

Although wear rutting can be easily treated with FDR, other rehabilitation methods usually are more economical [1]. FDR is recommended for pavements with deep rutting, drainage, and/or base and subgrade problems [4]. If it is related to weak/wet subgrade, FDR cannot address structural rutting unless subgrade stabilization/improvement and bituminous surface course application are considered [1].

In the past, FDR was limited to low and medium-traffic roads because there was no effective way to pulverize thick pavements of high-volume roads [1]. However, with the newer/larger equipment, many States have successfully used FDR for roads with traffic levels over 30,000 AADT [2]. FDR has been successfully used for airfield pavements, high-volume highways, heavy-duty parking, and industrial yards [5].

FDR use is almost independent of the climatic condition [2], and it can be used in all four climate zones, including dry freeze, dry no freeze, wet freeze, and wet no freeze [5].

1.1.3.2 Construction

Figure 1-8 shows the general FDR construction steps. There are two general processes. In the first process, the existing pavement is broken and pulverized with a motor grader or dozer. In multi-step sequence, materials may need size reduction with sheep foot or similar roller, crusher/compactor units attached to motor grader, hammermill (or impact breaker), or rotary mixers. This method may need multiple passes to achieve size reduction and may cause a lack of uniformity in the depth of cut. In a two-step sequence, the breaking, pulverization, and sizing of materials are combined with cold milling or a pulverizing machine. This method provides more control over the depth of cut and can pulverize the existing pavement in a single pass [4]. The second process, which involves a central plant, is an old method because rotary mixers could not effectively pulverize un-ripped pavements in the past. Nowadays, the widespread use of cold milling machines led to the production of large, productive reclaimers [1,4].

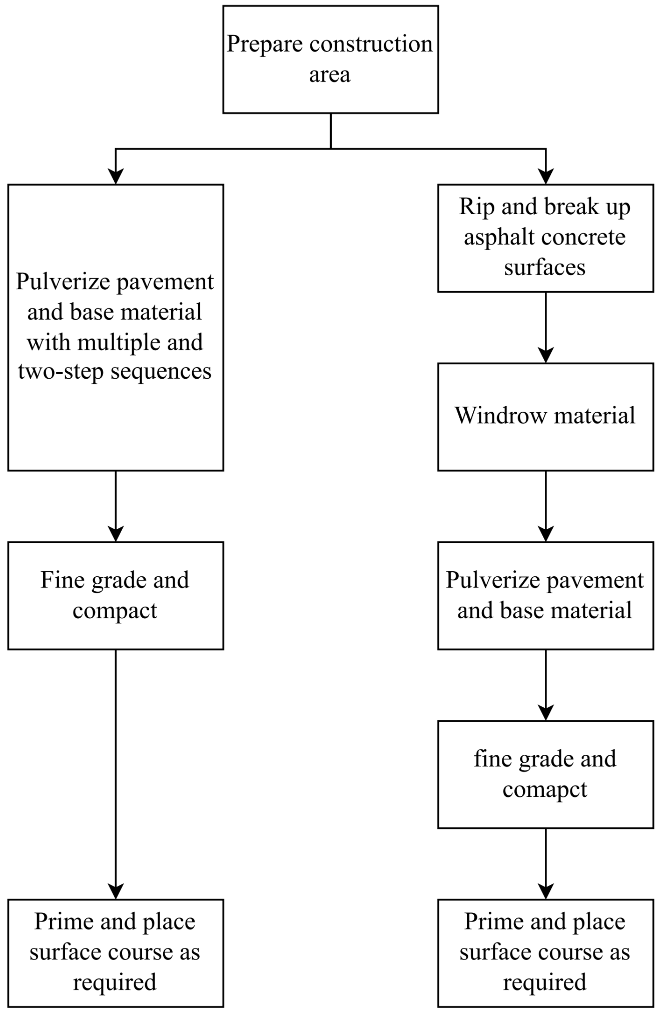


Figure 1-8 FDR construction steps [4]

Haul trucks may be used to provide additional granular materials to increase the thickness of the reclaimed layer, modify the gradation of material, or provide mechanical stabilization. A water truck with a spray bar is used to add moisture to the reclaimed materials that are not at the Optimum Moisture Content (OMC) for compaction. A motor grader is used to spread, shape, and aerate reclaimed materials if they are above their OMC [1].

Compaction is achieved with static steel-wheel, pneumatic-tired, and vibratory rollers, or a combination of them [4]. Because of the thickness and materials properties of reclaimed materials, compaction is usually performed in three steps with heavy equipment. Firstly, large pneumatic, smooth vibrating drum or padfoot rollers may be used for the initial compaction. A pneumatic roller is used for the intermediate compaction, and a smooth vibrating drum or static steel roller is used for finishing rolling [1]. Usually, a bituminous overlay or a single or double surface treatment is applied over the cold-recycled asphalt base materials [4].

1.1.3.3 Performance

The expected service life of FDR with surface treatment is 7 to 10 years, and the expected service life of FDR with a bituminous overlay is up to 20 years which is equivalent to new construction service life [1].

The performance of an FDR section was compared with a cement-stabilized SFDR section in California using the Accelerated Loading Test (ALT). The SFDR section was stabilized with 5 percent Portland cement. A rut depth of approximately 0.5 inches was recorded on the FDR section after approximately 490,000 ESALs were applied, compared with a rut depth of 0.12 inches on the SFDR section after more than 43.3 million ESALs. Despite being subjected to more than 37 million additional ESALs, measured and back-calculated stiffnesses were significantly higher for the SFDR section compared with the FDR section. No cracking was observed on either section. Elastic deflection at the bottom of the SFDR layer after 43.3 million ESALs was approximately the same as that at the bottom of the FDR layer after 490,000 ESALs [26].

Beesam and Torres-Machi [27] analyzed eight FDR and three SFDR projects in Colorado and compared their long-term performance with M-E predictions. They used deflection data from the falling weight deflectometer to back-calculate the resilient modulus of FDR materials and collected up to 11 years of IRI measurements. They concluded that current M-E inputs for FDR materials underestimate their actual performance and lead to a conservative design. They recommended that the strength modulus of non-stabilized FDR should increase from 30,000 psi to 37,300 psi and to 160,000 psi for emulsion-stabilized FDR (SFDR) [27].

1.1.4 Stabilized Full-Depth Reclamation (SFDR)

SFDR is a variation of FDR in which the pulverized materials are mixed with stabilizing agent(s) to produce a stabilized base course [23]. SFDR consists of pulverization or reclamation of existing pavement material, adding stabilizing additives, mixing, grading/shaping, compaction, and applying a wearing course [24]. Similar to FDR, the treatment depth of SFDR is generally between 4 to 12 inches, but it is typically used when milling depths over 6 inches are required [1,2]. Stabilizing additives are used when the reclaimed materials do not have the mechanical properties and/or structural strength to support the predicted loads [23]. SFDR is a better solution than FDR for roads that are in very poor condition or need structural upgrading for carrying more traffic loads [28]. Mechanical stabilizing additives such as crushed stone or gravel, chemical stabilizing additives such as Portland cement, hydrated lime, calcium chloride, magnesium chloride, fly ash, and/or bitumen additives such as slow or medium set asphalt emulsion or foamed (expanded) asphalt can be used to enhance the properties of reclaimed materials [1,23,24,29]. Also, geosynthetics and proprietary products, such as BaseOne®, have been used to enhance the performance of materials [30,31].

1.1.4.1 Project Selection Criteria

In addition to distresses that can be treated with FDR, instability rutting can be addressed with SFDR if the appropriate stabilizing agent(s) and/or granular material are considered in the mix design [1].

1.1.4.2 Materials and Mix Design

Like CIR, there is no nationally accepted mix design method for SFDR [1]. Based on a survey in 2011, most of the respondents do not do mix design for their SFDR projects, and the rest use Marshall, Superpave, Wirtgen, or other methods. Similar to CIR, the most common mix design methods are emulsion or foamed asphalt-based methods [2].

Laboratory mix design needs to be performed to optimize the type/quantity of stabilizing agent(s) and the physical properties of the reclaimed asphalt mix. To increase the reclaimed materials load-carrying capability of the reclaimed mix, different stabilizing agent(s) are often added to improve the mixture strength, durability, and/or moisture susceptibility. There are four categories of stabilizing agents [1]:

- Mechanical: virgin aggregates, RAP, and/or crushed/reclaimed Portland cement concrete
- Chemical: calcium chloride, magnesium chloride, lime (hydrated or quick lime), fly ash (type C or F), kiln dust (cement/CKD or lime/LKD), Portland cement (dry or slurry), or other chemical products
- Bituminous: asphalt emulsion, emulsified recycling agent, or a foamed/expanded asphalt
- Combination of two or more of the above

The mechanical mix design consists of determining the Optimum Moisture Content (OMC) to maximize the strength properties of the material. To determine the OMC and corresponding Maximum Dry Density (MDD), the standard or modified Proctor or similar tests can be used [1].

In chemical mix design, proportions of reclaimed material, chemical stabilizing agent, and moisture content are established by means of trial mixes. The OMC and MDD are then selected by the standard or modified Proctor or similar tests for the given chemical stabilizing agent application rate. A laboratory curing period between 2 to 4 hours is commonly used before compacting the samples. The strength and durability properties of the reclaimed mix are then evaluated. Finally, the lowest chemical stabilizing agent application rate that produces strength more than the required minimum is selected. Some owner agencies may also verify the durability and moisture sensitivity of the stabilized reclaimed mix [1].

In bituminous mix design, if an asphalt emulsion is selected as the stabilizing agent, the Optimum Fluid Content (OFC) is established using the standard of modified Proctor or similar tests with 50 percent water and 50 percent emulsion or with the minimum amount of water needed for good emulsion coating. Optimum bitumen content is then determined by adding a range of emulsion and water to oven-dried samples so that the determined OFC is maintained constant. The Marshall, Hveem, or other test methods for cold mixes are used to form and compact the stabilized mixture. After compaction, the samples are cured at room temperature [1].

If an asphalt foam is selected as the stabilizing agent, the foaming characteristics of the asphalt binder are determined, and the OMC is established. The objective is to maximize the expansion ratio and the half-life of the foamed asphalt by determining the percentage of water needed for a given asphalt binder temperature. The next step is to determine the foamed asphalt content. The Marshall, Hveem, or other test methods for cold mixes are used to form and compact the stabilized mixture.

The final step of both emulsion and asphalt foam mix design processes is to evaluate the strength and moisture sensitivity properties of the stabilized mix by determining the Marshall Stability, Hveem Stability, Indirect Tensile Strength (ITS), or other strength tests. The foamed asphalt content at which the soaked strength is maximum is the Design Bitumen Content (DBC), provided the minimum moisture sensitivity value is exceeded. Other tests, such as resilient modulus, dynamic creep, etc., may be performed on the cured stabilized reclaimed mixture at the DBC [1].

1.1.4.3 Construction

Generally, SFDR consists of the following steps [1]:

- Pulverization and sizing of the existing asphalt-bound layers
- Incorporation and mixing of the underlying granular base, subbase, or subgrade soil
- Application of mechanical, chemical, bituminous, or combination stabilizing agent(s), if required
- Mixing of the reclaimed materials and stabilizing agent(s)
- Initial or breakdown compaction
- Rough grading or initial shaping
- Intermediate compaction
- Intermediate shaping
- Final compaction
- Final trimming or tight blading
- Removal of all loose material
- Curing
- Application of seal or wearing course

Figure 1-9 shows the general SFDR construction steps. There are four different processes. Similar to FDR, the second process (from the left) involves a central plant. The other three are in-place processes. The resulting material can be processed further for resizing and mixing with recycling agents and new materials if required. Choosing between the in-place and central plant methods depends on equipment availability, roadway condition, and economic consideration. In-place recycling is generally more economical [4]. The first process from the left in Figure 1-9 is similar to that of FDR, except it involves adding and mixing stabilizing agents. The mixing is generally performed with a blade mixer or a transverse-shaft mixer in a multiple-step sequence. Cold milling machines are equipped with a pump and metering system to serve as a mixing unit in two-step sequences [4]. The single machine (unit) and equipment train processes are similar to those of the CIR technique.

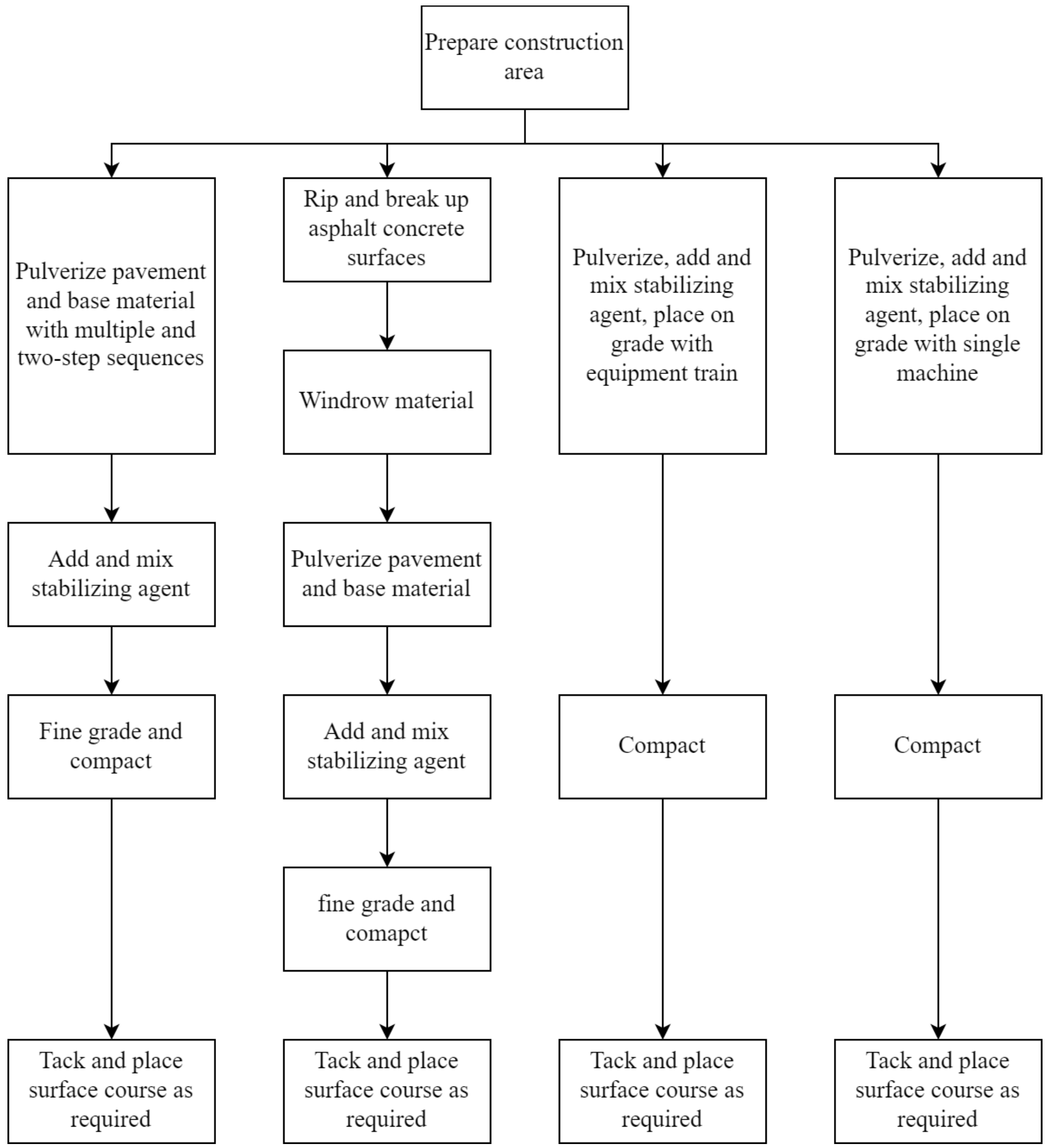


Figure 1-9 SFDR construction steps [4]

1.1.4.4 Performance

Three engineered emulsion SFDR sections (Cells 2, 3, and 4) constructed at the Minnesota Road Research Facility (MnRoad) on I-94 in 2008 were analyzed by Johanneck and Dai [32]. Each test section was designed for 3.5 million ESALs for 5 years. As of June 30, 2012, the sections had been subjected to approximately 2.2 million ESALs, greater than 60 percent of the design life. Different design emulsion contents were considered for the SFDR layers. SFDR layer of Cells 2, 3, and 4 was stabilized using 4 percent, 3 percent, and 0.75 percent Engineered Emulsion (EE), respectively. Strain gauges were used to measure the strain responses at the bottom of the bituminous and SFDR layers. The measured data indicated that the bottom of the bituminous and SFDR layers were under tensile strain and the tensile strains under the SFDR layers were much greater than the bituminous layers showing the stabilized layer transferred strain deeper into the pavement structure successfully. BISAR simulations indicated that the horizontal strains under the bituminous layer were reduced by approximately 50 percent compared to a bituminous over a granular base system. This reduction in tensile strain could lead to better performance and increased service life. In-field performance measurement of IRI, cracking, and rutting indicated that test sections were performing satisfactorily. No cracking was observed in Cells 2 and 4. A single crack was observed in Cell 3, which is believed to have propagated from the shoulder. No thermal cracking was observed. After about 60 percent of the design life, rutting measurements were 0.27 inches for Cells 2 and 3 and 0.3 for Cell 4. DARWin-M-E rutting and IRI predictions using project-specific inputs were close to field measurements. Overall, we concluded that SFDR is advantageous in reducing the bituminous tensile strains and leads to better pavement performance than a traditional structure of bituminous over a granular base [32]. As of February 2019, the sections had been subjected to more than 8.5 million ESALs, greatly exceeding their design ESALs. After 7 years in service, all cells had a "Good" (above 85) Pavement Condition Index (PCI) [33].

Schwartz et al. [20] investigated the material properties of SFDR, CIR, and CCPR materials using different recycling/stabilizing agents and chemical additives for mechanistic-empirical design. Typical values of Dynamic Modulus (|E*|) and Repeated Load Permanent Deformation (RLPD) structural properties under field-mixed, field-compacted, and field-cured conditions via laboratory testing of field cores taken 12 or more months after placement were evaluated. All three recycling processes had similar ranges of dynamic modulus values at intermediate and high reduced frequencies; thus, lower layer coefficient or dynamic modulus values that agencies may consider for SFDR mixtures may be too conservative. SFDR mixtures showed less temperature dependency and higher stiffness at low reduced frequencies. This is probably because the temperature dependency of stiffness is controlled by the existing RAP binder, which is the main component of CIR and CCPR. SFDR mixtures were found to have lower permanent deformations than CCPR and CIR. Typical dynamic modulus values of SFDR mixtures as an input in Pavement M-E design were suggested [20].

The first-year performance of CIR pavement sections in Wisconsin showed no surface distresses. Fly ash-stabilized base materials had more structural capacity than untreated and emulsion-stabilized materials. Based on the allowable traffic estimated, the increased structural capacity of the fly ash test section was interpreted as a performance life increase of 58 and 28 percent compared to untreated and emulsion-stabilized test sections, respectively [34].

The 10-year performance of SFDR projects in Nevada indicated that they were performing well. All were constructed using 2 percent or 3 percent (if soft subgrade is encountered) cement as a stabilizing agent. One project was experiencing non-wheel path longitudinal cracking, which was attributed to the asphalt surface and not the SFDR process. The 10 to 15-year performance indicated that most projects performed well except for several projects that experienced transverse or fatigue cracking. The cracking was reflected through the SFDR layer and was not top-down cracking. SFDR projects older than 15 years were rehabilitated or experienced reflection cracking [35].

1.1.5 Cost Benefits and Environmental Impacts

Asphalt recycling and reclamation techniques have proven to be cost-effective and environmentally friendly [1]. CIR and CCPR can reuse 100 percent of the existing materials and reduce greenhouse gas emissions by up to 90 percent and 50 percent, respectively. FDR/SFDR can reuse 100 percent of the existing materials and reduce importing and exporting of materials by 90 percent [5]. HIR, CIR, CCPR, and FDR are reported to be 20 to 40 percent, 20 to 50 percent, 20 to 50 percent, and 40 to 80 percent more cost-effective than conventional maintenance/rehabilitation techniques, respectively [5]. The life cycle cost analysis of high RAP (60 percent) hot and foamed cold recycled mixtures using data provided by FDOT demonstrated that foamed cold recycled mixture is the preferable option in terms of cost-effectiveness compared to the hot recycled mixture [36].

CIR is proved to be a cost-effective rehabilitation technique that can provide superior performance to conventional rehabilitation techniques with a cost-saving of up to 30 to 60 percent [11]. The economic analysis of CCPR test sections at the NCAT test track in 2012 indicated that by using recycled materials, cost savings of up to 31 percent could be achieved compared to dense-graded sections [37].

An environmental and economic evaluation of CIR and Cold In-place Recycled Expanded Asphalt Mix (CIREAM) techniques in Canada showed that CIR and CIREAM have 42 percent less initial cost compared to traditional rehabilitation techniques (i.e., mill and overlay) and are still cost-effective over a 50-years analysis period with a discount rate of 5 percent. They also emit significantly fewer GHGs compared to traditional rehabilitation techniques [38].

Braham A. [39] compared the 50-year life cycle cost analysis of SFDR versus traditional pavement Maintenance and Rehabilitation (M&R) strategies in Arkansas. Traditional M&R strategies include chip seal, 2-inch bituminous overlay, mill and fill, remove and replace, and reconstruct. SFDR projects were stabilized using foamed asphalt, emulsion, and cement. User costs were also calculated. It was concluded that SFDR was economically competitive with traditional M&R strategies, often more economical than a mill and fill while providing a significant structural gain. User delay significantly increased the cost of FDR. The traffic capacity of SFDR quickly increased at treatment depths greater than 10 inches. SFDR was less economical on roadways with higher ratios of bound to unbound materials [39].

A comparison of SFDR mixes stabilized with emulsion and emulsion plus lime indicated that although the costs of mixes with emulsion and lime are slightly higher, the improvement in rutting, stripping, and shear strength performance may justify the use of emulsion and lime to stabilize SFDR mixes [40]. The 50-year life cycle cost analysis of SFDR versus traditional pavement rehabilitation techniques in Virginia indicated that SFDR could save approximately $10 million and $30.5 million on primary and secondary roads, respectively [41].

1.1.6 Summary

The use of cold recycling and reclamation techniques has become widespread in recent years because they reduce the use of non-renewable resources, are cost-effective, efficient, fast to construct, and are environmentally friendly. Cold recycling techniques are categorized into four main techniques:

- Cold In-Place Recycling (CIR)
- Cold Central Plant Recycling (CCPR)
- Full Depth Reclamation (FDR)
- Stabilized Full-Depth Reclamation (SFDR)

CIR is an in-place recycling technique that involves reusing the existing asphalt pavement materials without the application of heat. Usually, an asphalt emulsion is used to stabilize the recycled materials. CIR can address most pavement distresses but cracked, structurally sound pavements that have well-drained bases are the best candidates. CIR construction consists of milling the existing pavement, adding recycling agent(s) or virgin aggregate, placing and compacting the recycled mix, and applying a surface course. The expected service life of CIR is 6 to 15 years and highly depends on the treatment depth and the thickness of the asphalt overlay [1]. If properly constructed, CIR can perform equal to or better than conventional rehabilitation methods.

CCPR is a cold recycling technique in which materials and/or recycling agent(s) are mixed in a central plant or a stationary unit. CCPR is useful for cases where in-place recycling is not possible, or a high amount/rate of production or close control of the mix design is needed. CCPR can address almost all types of pavement distresses, but it is mainly used to correct reflective cracking. CCPR consists of removing the existing pavement layers and crushing (on grade or at the plant). Mixing and adding new aggregate and/or recycling additives at the plant, laydown and compaction, and placing a surface course if required. The expected service life of CCPR projects is 6 to 15 years, and their performance is comparable to conventional rehabilitation techniques [1].

FDR is an in-place rehabilitation technique in which the full depth of the existing pavement and a predetermined thickness of the base, subbase, or subgrade is pulverized to produce base materials. FDR is used when the structural strength of the processed mixture is sufficient enough that enhancing with stabilizing additives is not necessary. FDR can address both pavement and base problems, is suitable to address almost all forms of distresses, and is usually used to rehabilitate pavements that are in poor condition with deep structural deficiencies. FDR consists of pulverization of reclamation of existing pavement material, adding new materials if required, mixing, grading/shaping, compacting, and applying a wearing course. The expected service life of FDR with asphalt overlay is up to 20 years which is equivalent to new construction service life.

SFDR is a variation of FDR in which the pulverized materials are mixed with stabilizing agents to produce a stabilized base course. Stabilizing additives such as gravel, Portland cement, hydrated lime, calcium chloride, magnesium chloride, fly ash, slow or medium set asphalt emulsion, or foamed (expanded) asphalt are used to enhance the mechanical properties and/or structural strength of reclaimed materials. SFDR is a better solution than FDR for roads that are in very poor condition or need structural upgrading to carry more traffic loads. SFDR consists of pulverization of reclamation of existing pavement material, adding stabilizing additives if required, mixing, grading/shaping, compacting, and applying a wearing course. The performance of SFDR is generally good. SFDR can perform better than a traditional structure of bituminous over a granular base since it transfers tensile strains deeper into the pavement structure.

1.2 Perpetual Pavements

Perpetual pavement, also referred to as long-lasting asphalt pavement, is an asphalt pavement that is designed to last at least 50 years without needing major structural rehabilitation or reconstruction. Periodic surface renewals, however, may be applied to address surface distresses. This concept is not new, and full-depth and deep-strength asphalt pavements constructed since the 1960s have been successful in providing long service lives. Perpetual pavement is mainly achieved by proper structural design and selecting appropriate material for each layer in the pavement structure [42,43].

Perpetual pavements should have enough structural strength and thickness to prevent fatigue cracking, structural rutting, and permanent deformation. They should also be durable to resist damage from traffic loading and the environment [44,45]. Perpetual pavements are cost-effective because they only need minor rehabilitation throughout their life. Minor rehabilitation also requires less construction time which reduces user time delay. Perpetual pavements have low environmental impacts because eliminating major rehabilitation and reconstruction means using less non-renewable materials and fewer energy sources [46,47].

1.2.1 Structure and Materials

Generally, a perpetual pavement structure consists of three bituminous layers over a solid foundation. Bituminous layers consist of a high-quality surface bituminous, such as hot-mix- asphalt (HMA) or stone matrix asphalt (SMA), or open-graded friction course (OGFC) layer, an intermediate high-modulus rut-resistant layer, and a flexible fatigue-resistant bituminous base. The maximum tensile strain occurs at the bottom of the bituminous base [42]. Figure 1-10 shows the general concept of the perpetual pavement structure.

Table 1.3 Perpetual pavement structure concept [48]

Pavement Layers	Pavement Structure
Surface	High-Quality HMA, SMA or OGFC 1.5 – 3 inches
Intermediate	High Modulus Rut Resistant 4 – 7 inches
Bituminous Base	Flexible Fatigue Resistant 3 – 4 inches
Pavement Foundation	-

The pavement foundation or subgrade is one of the most critical components of perpetual pavements. The foundation should provide a solid platform to support paving and compaction equipment during construction and bear traffic loads and climatic changes during the performance period [42]. Soil stabilization and/or treatment may be used to increase the structural capacity of the foundation. Materials for this layer include sand or sandy-gravel subgrades, stabilized fine-grained subgrade, un-stabilized or stabilized granular base materials, or rubblized concrete [46,49].

The bituminous base layer should resist fatigue cracking from bending under repeated traffic loads. Mixtures with high binder contents (to an extent) can improve the fatigue resistance characteristics of the bituminous layer. High binder or rich asphalt mixtures with appropriate layer thickness could result in a bituminous base layer that can resist fatigue cracking of the bottom layer [50]. Generally, the performance of CR base layers is significantly influenced by the RAP binder content, emulsion type & content, and RAP source [15]. Another method that eliminates the use of different mixtures for intermediate and base layers is to design a thick base layer in order to minimize the tensile strain at the bottom of the bituminous layer so that cumulative damage would not occur [51]. The intermediate or binder layer should be durable and stable to resist rutting. Stability in this layer can be achieved by stone-on-stone contact of coarse aggregates [42]. The internal friction and the aggregate skeleton can be obtained by using a large Nominal Maximum Aggregate Size (NMAS) aggregate. The Superpave method may be used for NMAS of up to 37.5 mm [52].

The surface layer should resist rutting and cracking, provide friction, mitigate splash and spray, and minimize the tire noise. SMA, Superpave dense-graded mixture, or OGFC could be selected to consider these requirements. In case of heavy traffic volumes, the need for rutting and wear resistance, impermeability, and durability may suggest the use of SMA. In lighter traffic situations, the use of a dense-graded Superpave mixture is more appropriate. OGFC, which has a higher percentage of air voids, allows the water to drain from the surface of the pavement and thus is used to improve wet-weather friction of the surface layer [42].

1.2.2 Design

Perpetual pavement design requires a different approach than empirical pavement design, which relies on the relationship between pavement performance observation, traffic, material quality indicator, and layer thickness. Perpetual pavements are designed with the mechanistic-empirical design approach that is based on the analysis of the pavement responses such as stresses, strains, and displacements [46]. This approach essentially aims to determine the pavement's reaction to traffic and climate, locate critical points in the pavement structure, and design against certain failures and distresses by selecting appropriate materials and layer thicknesses [42]. Figure 1-11 shows the procedure of perpetual pavement design.

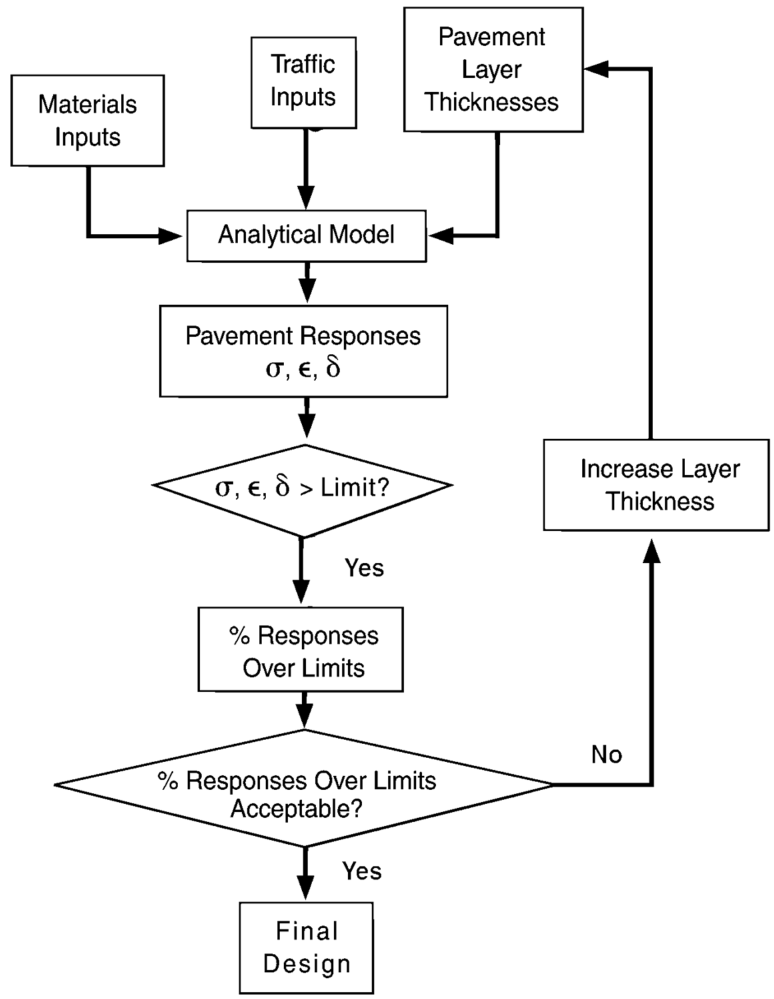


Figure 1-10 Perpetual pavement design procedure [46]

1.2.2.1 Limiting Pavement Response

The approach to designing perpetual pavements is to define the critical pavement responses below which structural damage is zero in the M-E procedure. Most perpetual pavement approaches use structural rutting and bottom-up fatigue cracking for this purpose [46].

Structural rutting is the large permanent deformation of the granular base and/or subgrade under the loading traffic. Structural rutting is rare but expensive to repair [46]. Harvey et al. [53] and Walubita et al. [45] used vertical compressive strain at the top of the subgrade as the limiting pavement design parameter. They stated that if the compressive strain at the top of the subgrade remains below 200 $\mu\varepsilon$ (microstrain), permanent deformation will not occur.

Bottom-up fatigue cracking occurs at the bottom of the asphalt layer and may propagate to the surface, resulting in surface deterioration, pumping, and rutting [46]. One approach for designing perpetual pavement is to reduce the strain at the bottom of the asphalt layer and relocate the strain to the pavement surface, where treating cracks is easier. Fatigue Endurance Limit (FEL) is the tensile strain below which no fatigue damage is accumulated. FEL is commonly used as a limiting strain criterion to design perpetual pavements. Figure 1-12 shows the concept of FEL. The most common value suggested for FEL is 70 $\mu\varepsilon$ (microstrain). Many studies have been conducted to determine the FEL of asphalt mixtures. Table 1-3 summarizes the reported FEL values.

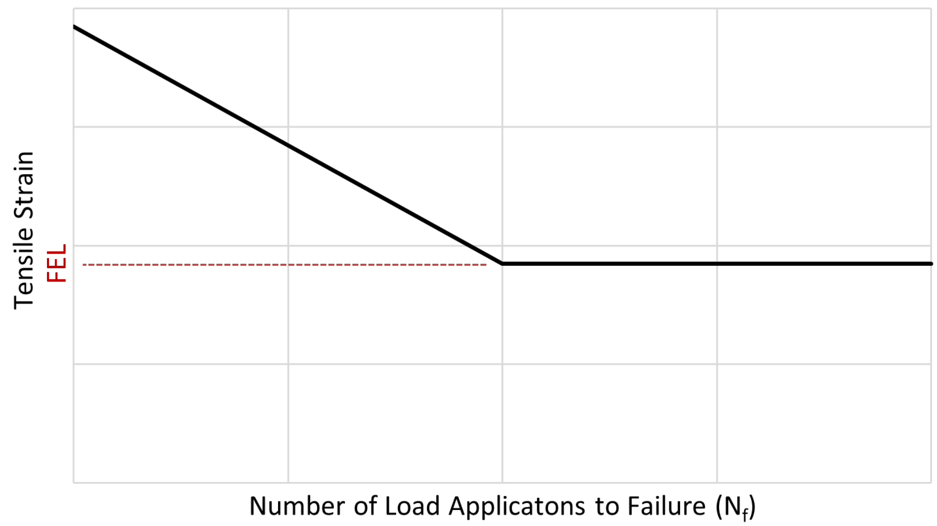


Figure 1-11 Fatigue endurance limit concept

Table 1.4 Suggested FEL values

FEL	Reference
70 $\mu\varepsilon$	Monismith et al., Carpenter et al., Lee et al. [54–56]
FEL @ 25 °C = 11.31 ln (OT) + 21.63 R^2=0.88 OT = Overlay tester cycles to failure	Hu et al. [57]
20 – 180 $\mu\varepsilon$	Norouzi et al. [58]
200 $\mu\varepsilon$	Sanchit et al. [59]

FEL is dependent on the temperature, the number of load cycles, and the mixture components and properties [57,58]. Willis and Timm [60] recommended that a strain distribution be used instead of using an individual fatigue design endurance limit since a single strain value may not accurately quantify what the pavement is experiencing.

Table 1-4 shows the upper bound of the proposed strain distribution. Using a strain distribution allows pavement engineers to design thinner pavements that still perform adequately [60].

Table 1.5 Fatigue design endurance limit for perpetual pavements [60]

Percentile	Upper Bound Fatigue Limit
99%	394
95%	346
90%	310
85%	282
80%	263
75%	247
70%	232
65%	218
60%	205
55%	193
50%	181
45%	168
40%	155
35%	143
30%	132
25%	122
20%	112
15%	101
10%	90
5%	72
1%	49

1.2.2.2 PerRoad

Timm and Newcomb [47] developed a perpetual pavement design methodology for the Asphalt Pavement Alliance, embodied in the computer program, PerRoad. This design procedure incorporates M-E analysis and Monte Carlo simulation to calculate probabilistic pavement reaction to loading and evaluate potential damage. This method is similar to M-E design approaches in which pavement cross-section, material properties, and loading configurations are entered into a mechanistic model, and pavement responses are calculated at critical locations. Next, pavement responses are transformed to predict the allowable number of loads (N_f). The main difference between PerRoad and other M-E approaches is that PerRoad considers the N_f infinite below the threshold response resulting in zero damage accumulation. Another feature of PerRoad is that it can consider variability in material properties, layer thickness, and traffic loading configurations. This input variability is modeled using Monte Carlo simulations to create random pavement structures, apply different loads based on the probability distribution, and generate distributions of pavement responses. Figure 1-13 shows the PerRoad perpetual pavement design procedure [47]. PerRoad is available for download at https://www.driveasphalt.org/.

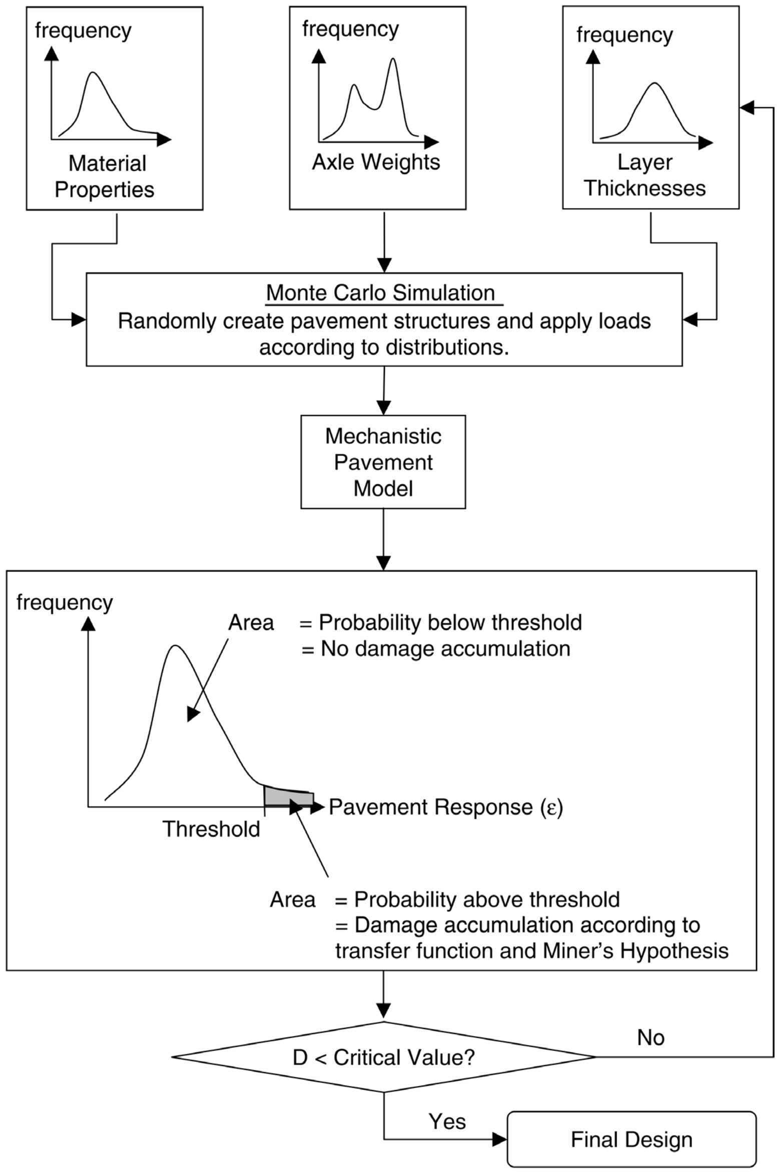


Figure 1-12 PerRoad perpetual pavement design procedure using Monte Carlo simulation [47]

1.2.2.3 Mechanistic-Empirical Pavement Design Guide (MEPDG)

MEPDG, developed under the NCHRP 1-37A project, presented a modern approach to pavement design. The MEPDG considers inputs that affect the performance of pavements, including traffic, climate, pavement structure, and material properties. MEPDG, similar to other M-E pavement design procedures, predicts critical pavement responses (strains, stresses, and deflections) and uses transfer functions and pavement distress models to translate these responses to pavement performance. MEPDG is considerably more complex than empirical pavement design methods and incorporates hundreds of variables and parameters to characterize materials and predict pavement performance [61,62].

MEPDG consists of three pavement design and analysis stages: evaluation, analysis, and strategy selection. Figure 1-14 shows these three stages in more detail [61].

MEPDG can be used to design perpetual pavements. Many studies used the results of MEPDG to predict the performance of perpetual pavements [46,63–65]. Although MEPDG can consider design lives of more than 50 years, it should be noted that few pavements with more than 30 years of performance data were used in the global calibration of prediction models. MEPDG considers the FEL as a material property for bituminous layers. The FEL is assumed to be independent of the layer temperature and mixture modulus, and a single value is used for all bituminous layers. The FEL was excluded from the global calibration and should only be used with the re-calibration of the fatigue cracking model [61].

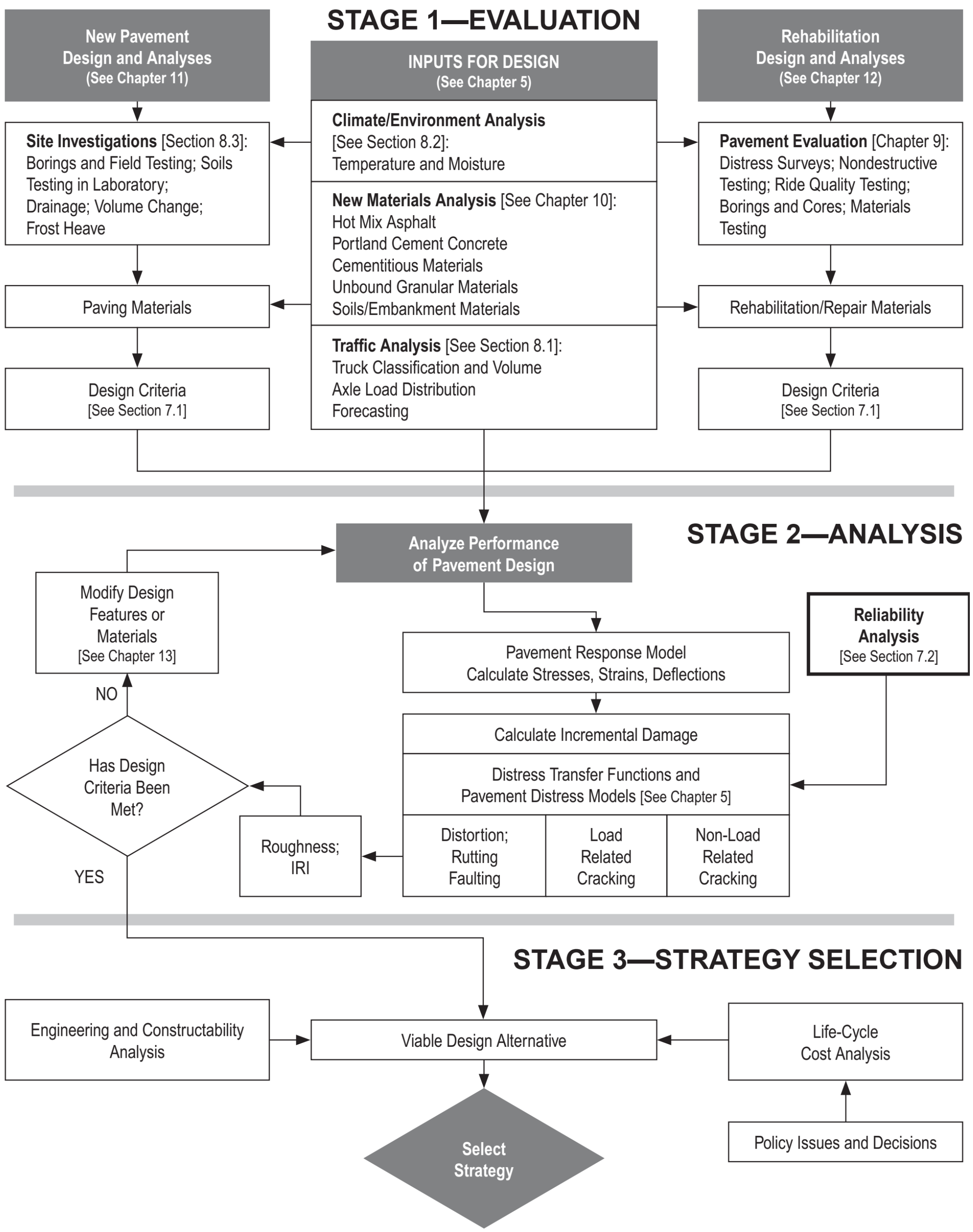


Figure 1-13 Flowchart of pavement design and analysis stages for MEPDG [61]

1.2.2.4 Other Design Methods

Lee et al. [56] developed a simplified design procedure for determining layer thickness in long-life pavements with a design life of more than 40 years. A synthetic database was established using ILLIPAVE finite element program with a different combination of layer thickness and modulus to develop a model that predicts critical pavement response. This method suggests the minimum layer thickness and layer modulus based on the limited strain concept using the database. The FEL value of 70 $\mu\varepsilon$ for the bottom of the asphalt layer and a limiting compressive strain of 200 $\mu\varepsilon$ was considered as the design criteria. Figure 1-15 shows the long-life pavement design procedure. It was concluded that as the thickness and the modulus of the asphalt concrete layer increase, the tensile strain at the bottom of the bituminous base decreases. It was suggested that the thickness of the bituminous base layer should be at least 175 mm (6.9 in.), and the modulus of the asphalt mix should be greater than 3.5 GPa (507 ksi) for a long-life pavement with a minimum design life of 40 years. Accelerated Pavement Testing (APT) measurements and field testing were used to validate the design procedure. Validation showed that the procedure could predict the measured strain levels within an acceptable range [56].

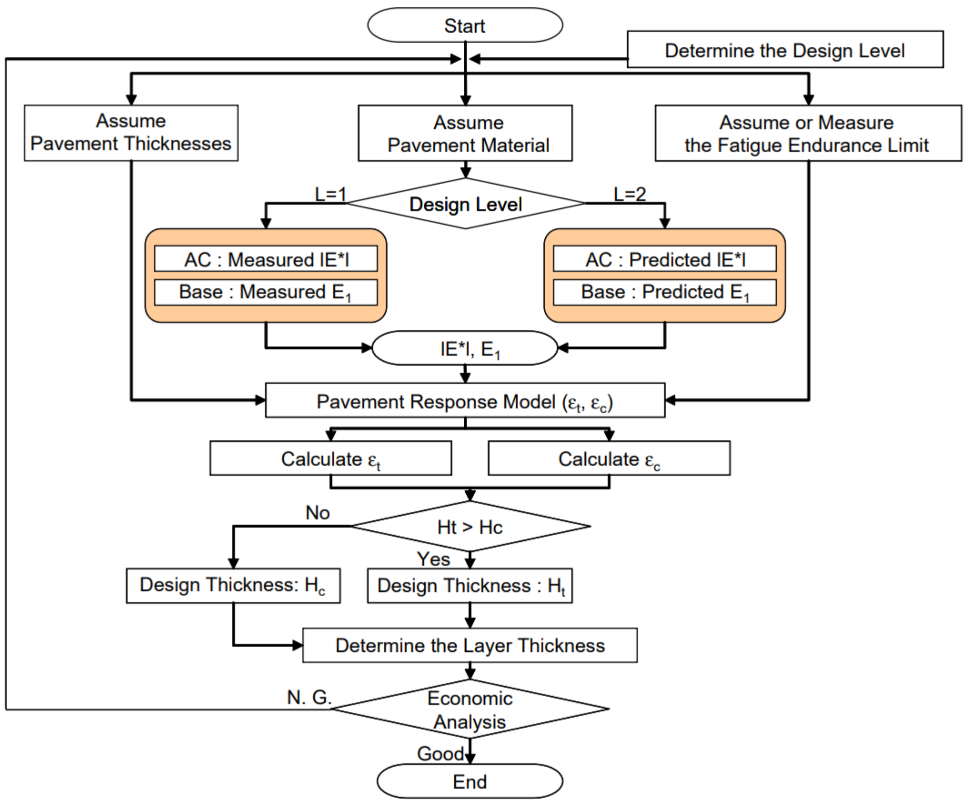


Figure 1-14 Simplified design procedure for long-life pavements proposed by Lee et al. [56]

1.2.3 Construction

The construction of perpetual pavements is generally similar to conventional pavements [46]. However, it requires special attention to details, the quality of the build, and the compaction of pavement layers. Modern testing methods should be used during the construction process to receive continuous feedback on the quality of materials and construction [48].

Several factors ensure the longevity of pavement structure in the construction of perpetual pavements, including [46]:

- Preparation of a strong and uniform foundation
- Achievement of the optimum density in the asphalt mix

- Uniformity in asphalt mix design, production, and placement
- The good bonding between all pavement layers
- Quality control throughout the construction process

The foundation should be solid and uniform to carry the construction equipment loads. Volume changes in the foundation soil should be minimized by using stabilizers or soil mixing [48]. In situ foundation testing is recommended to control both the quality and consistency [51]. Appropriate material handling during production, transport, and laydown can prevent segregation in coarse aggregate mixtures [48]. Interlayer friction between the foundation and the asphalt base layer should be provided in order to avoid compaction problems in the asphalt base layer. Compaction of asphalt layers is crucial since low density reduces fatigue life and resistance, thereby increasing cracking [46]. Proper density in bituminous layers can be achieved by implementing Superpave and the successful application of SMA. Volumetric control of the mixtures by the contractor will ensure the final product consistency and quality [48].

1.2.4 Performance, Cost Benefits, and Environmental Impacts

To maintain perpetual pavement in its optimal state, periodic pavement monitoring and resurfacing are necessary to ensure that surface distresses such as top-down fatigue cracking, thermal cracking, rutting, and surface wear are restricted to the top few inches of the surface course [42,46]. The resurfaced layer should be similar to the original surface in rut resistance, durability, thermal cracking resistance, and wear resistance [48]. The binder PG of the asphalt layers should be appropriate for the project traffic level and climate of the area. The binder PG of the top layer should be bumped to at least one high temperature greater than what is normally used in the area to avoid rutting [42]. Depending upon availability and cost, the low-temperature grade should be that typically used in the area for a 95 or 99 percent reliability to resist thermal cracking [48].

Perpetual pavements have proven to be economically advantageous compared to conventional or concrete pavements [46]. The Life Cycle Cost (LCC) of perpetual pavement was compared to conventional asphalt pavement by Amini et al. [64] for various traffic and weather conditions for an analysis period of 40 years. The perpetual structure consists of an SMA surface layer, a bituminous intermediate layer, and a bituminous base layer above a granular foundation layer. The perpetual pavements were designed using AASHTOWare pavement M-E design by considering an FEL value of 70 $\mu\varepsilon$ as the limiting response criteria. It was concluded that perpetual pavements have 4- to 20-percent less LCC than conventional asphalt pavements. This might be due to less maintenance and rehabilitation costs, elimination of reconstruction, and reduced delay and user costs.

Life Cycle Cost Analysis (LCCA) and sustainability assessment of perpetual pavements in Canada indicated that although perpetual pavements have higher initial costs compared to conventional asphalt pavements, they have cost benefits in the long term. Perpetual pavements are more sustainable, especially for roads with heavy traffic. Reduction in the maintenance schedule of perpetual pavements also reduces non-renewable resource consumption and pollution [49].

Environmental impacts of conventional and perpetual pavements with an analysis period of 40 years were compared by Liu et al. [66] in China with respect to energy consumption and carbon dioxide (CO_2) emissions. The study showed that perpetual pavement could reduce energy consumption and CO_2 emissions by up to 14.5 percent and 18.5 percent, respectively. This is mainly achieved by reduced maintenance despite having more environmental impacts due to increased asphalt thickness. The maintenance phase of perpetual pavement resulted in approximately 60 percent less energy consumption and CO_2 emissions compared to conventional asphalt pavement.

1.2.5 Using Recycling and Reclamation Techniques for Building Perpetual Pavements

There are not many studies conducted on using recycling and reclamation techniques for achieving perpetual pavements. In one study done by Timm et al. [17], perpetual pavement analysis of three NCAT cement-stabilized CCPR sections was performed to determine if they meet perpetual design criteria with respect to bottom-up fatigue cracking. Test sections include an SMA surface and Superpave dense-graded AC above a CCPR layer over a granular aggregate base for Sections N3 and N4 and a cement-stabilized base (SFDR) for Section S12. Figure 1-16 shows the as-built thickness and layer combination of the test sections. Asphalt strain gauges were installed at the bottom of the CCPR layer to measure bending. Earth pressure cells were also installed at the bottom of the CCPR layer and the top of the subgrade to measure vertical stress distribution. The CCPR materials passed the 70 percent retained Indirect Tensile Strength (ITS) and a dry ITS of 45 psi requirements. Approximately 10 MESALs were applied during the first two years of the study (2012–2014), and an additional 10 MESALs was applied during the second test cycle (2015–2017). PerRoad software was used to simulate and compare the predicted tensile strains to field measurements. It was concluded that based on both simulation and field measurement results, the SFDR section (S12) is expected to be perpetual, while the other two sections are expected to experience bottom-up fatigue cracking. PerRoad simulation showed that the N3 section, which has a thicker Superpave AC layer compared with the N4 section, could be designed to behave as perpetual by increasing the thickness of AC/CCPR layers [17].

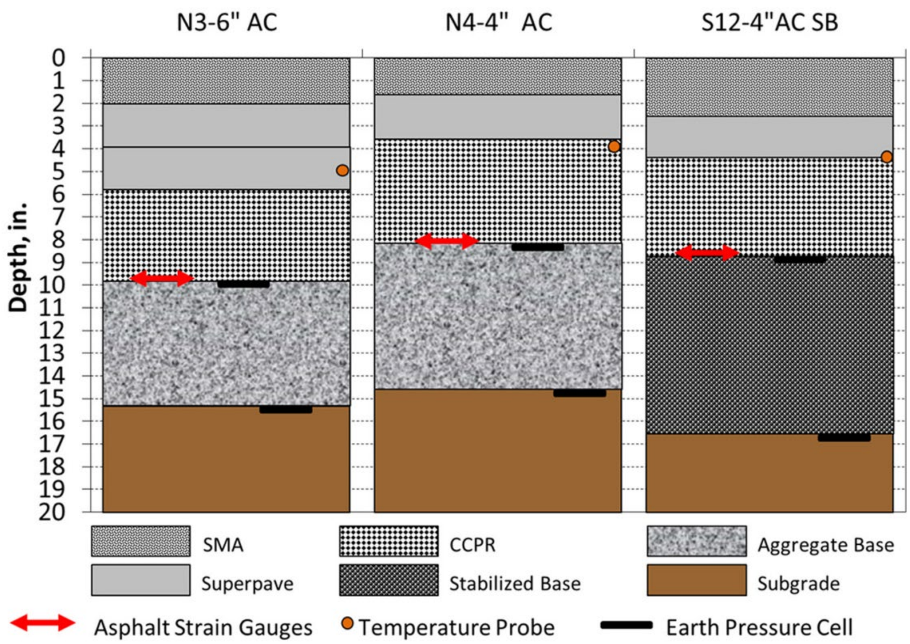


Figure 1-15 Test section as-built thickness and layer combination used in Timm et al. study [17]

1.2.6 Summary

Perpetual pavements are designed and constructed to last at least 50 years without needing major structural rehabilitation or reconstruction. Periodic resurfacing may be applied to address surface distresses. Perpetual pavement is achieved by appropriate structural design, material selection, and construction quality. The structure of perpetual pavements consists of three bituminous layers, including a high-quality surface bituminous, an intermediate high-modulus rut-resistant layer, and a flexible fatigue-resistant bituminous base over a solid and durable foundation.

Perpetual pavements are designed with the mechanistic-empirical design approach that is based on the analysis of the pavement responses and designing against certain failures and distresses by selecting appropriate materials and layer thicknesses.

The construction of perpetual pavements is generally similar to conventional pavements, but special attention should be given to details and the quality of the build. Preparation of a strong and uniform foundation, uniformity of asphalt mixes, appropriate material handling during production, transport, and placement, good interlayer friction, and quality control throughout the construction process ensure the longevity of pavement structure in the construction of perpetual pavements. Perpetual pavements not only can serve under different traffic and climatic conditions for the long term but also have lower life cycle costs and environmental impact than conventional pavements.

There are not a lot of studies conducted on using recycling and reclamation techniques for achieving perpetual pavements. In one study, it was shown that SFDR could behave as perpetual. Also, using thick enough AC overlays on top of the CCPR layer can lead to perpetual behavior.

Chapter 2: Current Practices on Reclamation and Recycling Techniques

This chapter provides details on current practices employed by NRRA agency members on reclamation and recycling techniques to achieve perpetual pavements characteristics.

2.1 NRRA Agency Members Survey

A survey was developed using the University of New Hampshire Qualtrics Survey system and distributed to all NRRA agency members. This was done to identify current rehabilitation approaches, including cold recycling, used by states, current pavement design methodologies used for both perpetual pavements and cold recycled pavements, treatment approaches for cold recycled layers, and methodologies for tracking cold recycled treated pavement performance together with agency preference for rehabilitating/reconstructing cold recycled pavement structures. The survey is included in Appendix A of this report. A total of 11 responses were received. A summary of the findings is discussed in the following sub-sections.

2.1.1 Cold Recycling Techniques

The survey data shows that a large percentage of NRRA member agencies utilize cold recycling technology (Figure 2-1) for rehabilitation projects. Cold recycling techniques used include CIR, CCPR, and FDR/SFDR with CIR and FDR/SFDR being the most common (Figure 2-2).

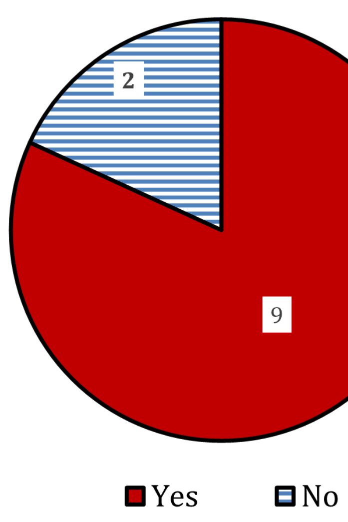


Figure 2-1 Cold recycling utilization

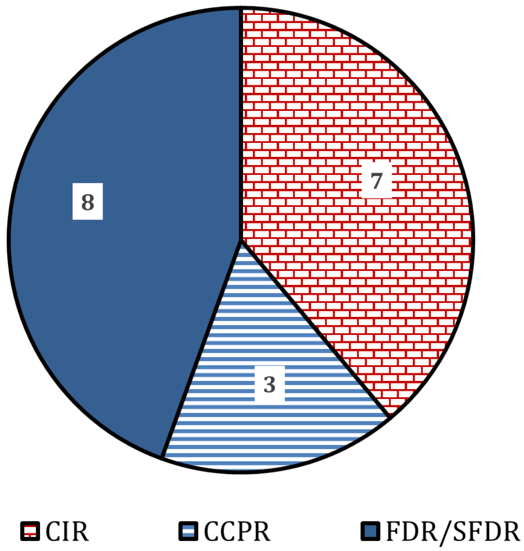


Figure 2-2 Cold recycling technique

Processes for selecting these recycling techniques for specific rehabilitation projects vary amongst agency members. Table 2-1 provides summary of typical processes and/or criterion for each technique's deployment.

Table 2.1 Cold recycling selection criterion as reported by member agencies

NRRA Member Agencies	CIR	CCPR	FDR/SFDR
Caltrans	Severity of pavement distress	Severity of pavement distress	Severity of pavement distress
Illinois DOT	*See Note b*	*See Note b*	*See Note b*
LRRB[(1)] /RIC[(2)]	*See Note a*	*See Note a*	Failing pavement (Decent subgrade)
Minnesota DOT	Pavement distresses limited to bituminous layer	Sections that require subgrade construction or roadway realignment	Problem with subgrade of existing structure
Mississippi DOT	*See Note a*	*See Note a*	Developed decision trees derived from the agency's pavement management data
Montana DOT	Based on what is observed on field cores	*See Note a*	Based on what is observed on field cores

NRRA Member Agencies	CIR	CCPR	FDR/SFDR
Nebraska DOT	Weak pavement with full strip out in cores	*See Note a*	Weak pavement with full strip out in cores
North Dakota DOT	High crack pattern on existing pavement surface also lacking sufficient thickness	*See Note a*	Overlays no longer performing satisfactorily, and extensive treatment is needed
Wisconsin DOT	Severely distressed bituminous layers	*See Note a*	Selection criteria in development

Notes:

a. Agency does not use this technique
b. No selection criterion was reported.

[1] LRRB: Local Road Research Board
[2] RIC: Research Implementation Committee

2.1.1.1 Treatments

Responders indicated various treatments that are used for recycled sections. As presented in Figure 2-3, CIR layers are most commonly treated using Portland cement. In addition, depending on the needs of a particular project, emulsion, foamed asphalt, and lime (as active filler with emulsion or foamed asphalt) are used. As for CCPR (Figure 2-4), the majority of member agencies use either emulsion, Portland cement or foamed asphalt across various rehabilitation projects. FDR/SFDR (Figure 2-5) on the other hand is most treated with Portland cement. As for agencies that responded to "Other (Please specify)", one agency indicated the use of emulsion, 0.5 percent cement and geosynthetics across projects depending on the needs of the project. Another agency mentioned the use of emulsion, foamed asphalt and cement which also depends on the needs of the project. The use of proprietary materials (recycling agent) was indicated by one agency for FDR/SFDR treated layers, primarily when the material cost of stabilizer is about 10 percent of asphalt binder cost.

Some agencies also pointed out that roads treated with FDR are just reclamation without any added materials or stabilizing agent. One responder points to a pavement pulverization process followed by smoothening and then compaction with seldom any stabilizing agent used. Another responder mentioned that across FDR rehabilitation projects, there is a mixture of non-stabilized FDRs and cement stabilized FDRs. The cement stabilized FDRs are particularly used when a thinner overall section is preferred and/or the subgrade is questionable.

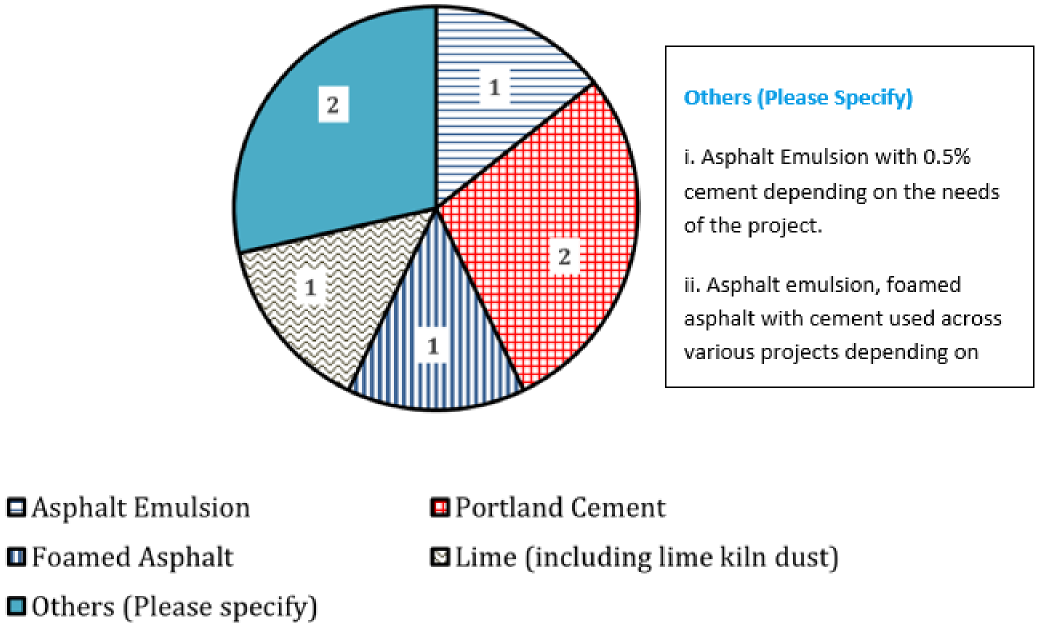


Figure 2-3 CIR treatment

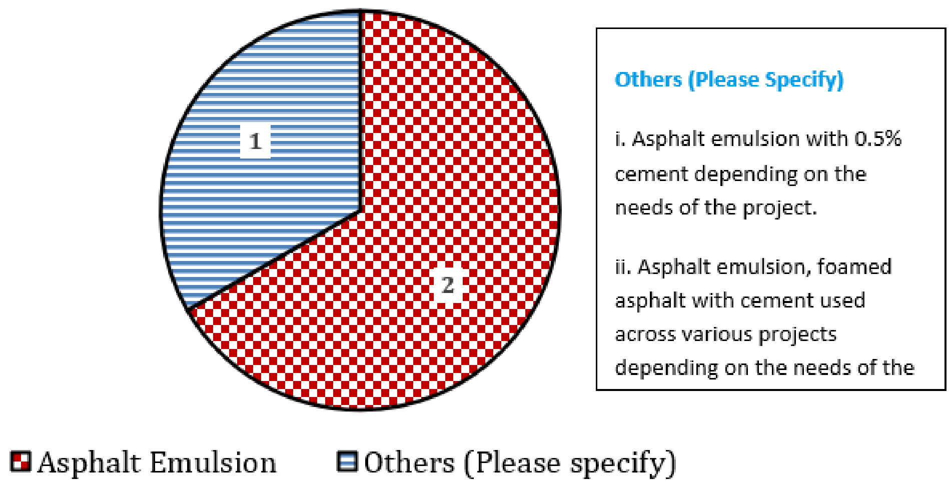


Figure 2-4 CCPR treatment

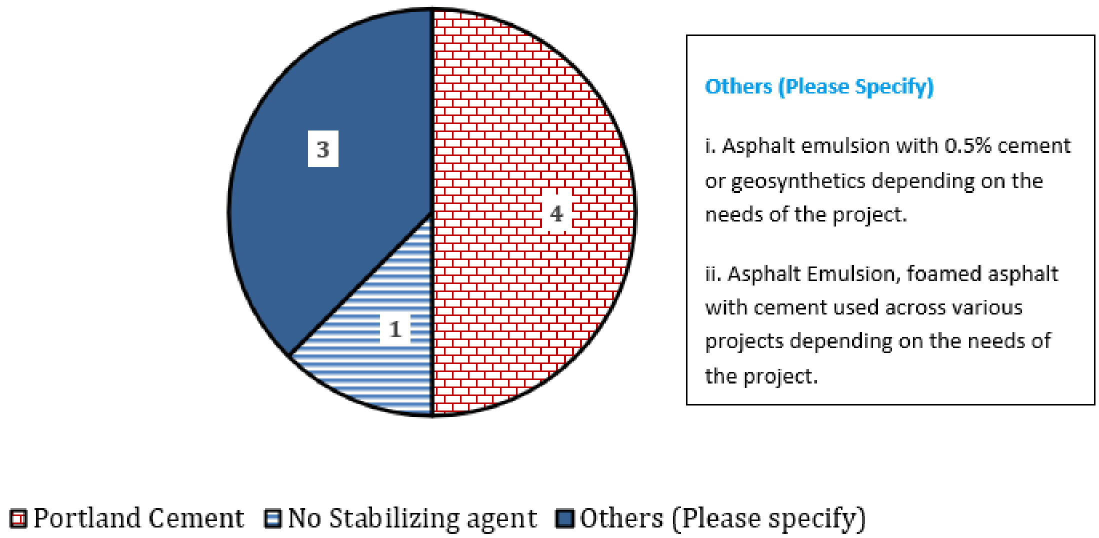


Figure 2-5 FDR/SFDR treatment

2.1.2 Pavement Design

In designing thicknesses of cold recycled layers (CIR, CCPR, FDR/SFDR) and wearing course/overlays, several approaches are employed by responding agencies. Table 2-2 and Table 2-3 present the approaches employed by all agency survey responders and the pavement design manual/guide respectively.

Table 2.2 Cold recycled Layer and overlay thickness design approach

NRRA Member Agencies	CIR	CCPR	FDR/SFDR
Caltrans	In-house design (CalME)	In-house design (CalME)	In-house design (CalME)
Illinois DOT	AASHTO 1993	AASHTO 1993	*See Note c*
LRRB/RIC	*See Note c*	*See Note c*	AASHTO 1993
Minnesota DOT	In-house design (MnPAVE)	In-house design (MnPAVE)	In-house design (MnPAVE)
Mississippi DOT	*See Note c*	*See Note c*	AASHTO 1993
Montana DOT	AASHTO 1993	*See Note c*	AASHTO 1993

NRRA Member Agencies	CIR	CCPR	FDR/SFDR
Nebraska DOT	AASHTO 1993, AASHTOWare PMED	*See Note c*	Designed having 4 inches of asphalt concrete overlay over FDR. *See Note a*
North Dakota DOT	AASHTO 1993	*See Note c*	AASHTO 1993
Wisconsin DOT	In-house design (WisPAVE) *See Note b*	*See Note c*	In-house design (WisPAVE) See Note b

Notes:

a. This approach was utilized based on the premise of observed performance of the pavement structure after 12-15 years where FDR was seen not to bound well with 2-3 inches of asphalt concrete creating fatigue cracking distress.
b. AASHTO 1972 used in developing the WisPAVE package
c. Agency does not use this technique

Table 2.3 Pavement design manual

NRRA Member Agencies	Pavement Design Manual Online Address	Last Date Modified
Illinois DOT	IDOT BLRS CIR Procedure	April, 2012
Mississippi DOT	Mississippi DOT Pavement Design Procedures	December 31, 2019
Minnesota DOT	Manual - Pavement Design - Materials & Road Research - MnDOT	May 5, 2022
Montana DOT	Montana DOT Pavement Design Manual	November, 2018
Wisconsin DOT	Standardized Special Provisions for Engineering Design Facilities Development Manual- Pavements	May 15, 2019

As shown in Figure 2-6, Mechanistic-Empirical approach (such as PerROAD) is most often used in designing perpetual pavements amongst member agencies. Just one case exists where AASHTO 93 is employed for flexible perpetual pavement design and AASHTOWare PMED for rigid perpetual pavement design.

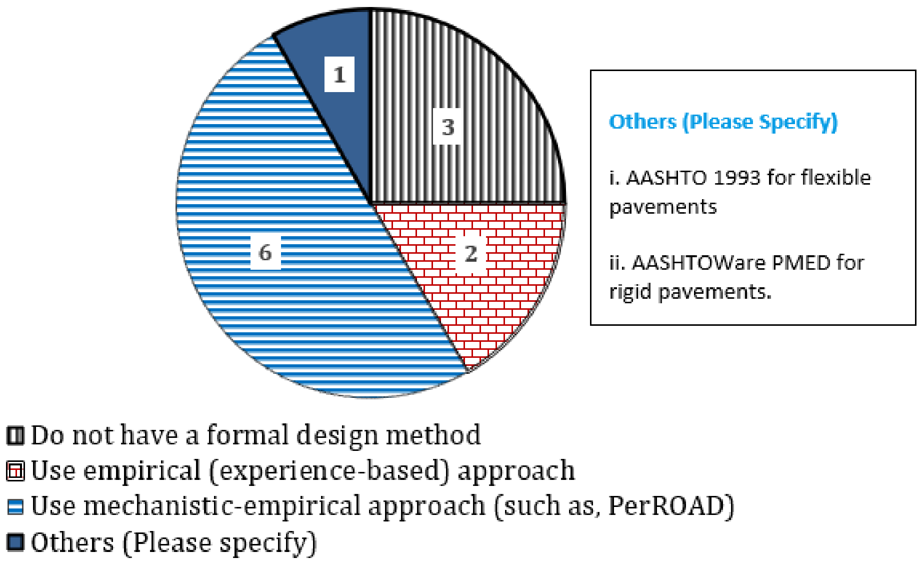


Figure 2-6 Perpetual Pavement Design approach

2.1.3 Performance Tracking of Cold Recycled Pavements

Information gathered from the survey indicates that efforts are in place to track performance of cold recycled treated pavements (Figure 2-7). Although no reports on performance tracking are available as of the time of the survey, one respondent indicated lessons learned from an FDR treated pavement in a construction monitoring report. From the details gathered, it was suggested that extensive testing should be carried out on a project site prior to commencement of construction to properly understand the existing pavement structure. The testing should primarily aim to determine the limitations of the underlying materials and detect potential problems associated with the underlying structural components.

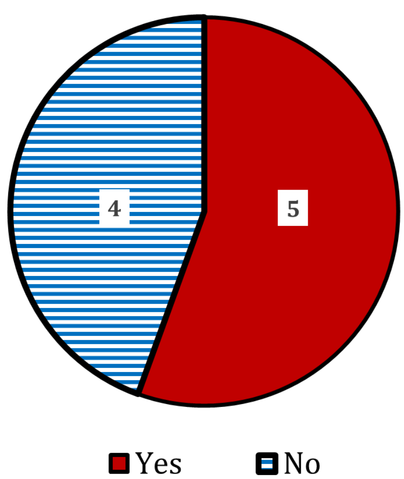


Figure 2-7 Performance tracking of cold recycled pavements

2.1.4 Post-Cold Recycling Rehabilitation Considerations

Member agencies have identified the most preferred and/or best approach for rehabilitating or reconstructing cold recycled treated pavements when they reach end of life. Details are summarized in Table 2-4.

Table 2.4 Post-cold recycling rehabilitation considerations

NRRA Member Agencies	CIR	CCPR	FDR/SFDR
Caltrans	Reconstruction, FDR	Reconstruction, FDR	Reconstruction
Illinois DOT	*See Note b*	*See Note c*	*See Note c*
LRRB/RIC	*See Note b*	*See Note b*	FDR
Minnesota DOT	More of CIR or FDR	More of CIR, FDR, or roadway realignment	CIR or FDR
Mississippi DOT	*See Note b*	*See Note b*	Asphalt Resurfacing *See Note d*
Montana DOT	Reconstruction	*See Note b*	Reconstruction
Nebraska DOT	Mill off existing asphalt overlay and add 3 to 4 inches of asphalt concrete	*See Note b*	Mill off existing asphalt overlay and add 3 to 4 inches of asphalt concrete
North Dakota DOT	*See Note c*	*See Note b*	Maintenance with mills and overlays *See Note a*
Wisconsin DOT	Pavement replacement	*See Note b*	*See Note c*

Notes:

a. FDR on existing FDR pavement has been done. However, most roadways with FDR segments are being maintained.
b. Agency does not use this technique.
c. Cold recycled treated pavements are still relatively new. Not applicable yet.
d. The pavement is still relatively new and not applicable yet. Foreseeing that the pavement will be perpetual.

2.2 NRRA Agency Members Interview

Following the survey of current practices used by NRRA member agencies on reclamation and recycling techniques to achieve perpetual pavements, select NRRA member agencies were contacted to provide additional details on their experiences with cold recycling. The representatives from Illinois, Mississippi and North Dakota Departments of Transportation were interviewed. The focus of these interviews was to obtain additional information on these agencies' practices for pavement design inputs, stabilizing/recycling agent specifications, and expected pavement life and performance of cold recycled pavements. The following sub-sections provide synopsis of the interviews.

2.2.1 North Dakota DOT (NDDOT)

CIR experience for NDDOT is limited as there has only been one CIR project completed in recent years (2021). CIR is currently not a preferred recycling technology for NDDOT primarily due to lack of historic experience with CIR and some issues associated with preserving pavement geometry. Further, there may be some concerns associated with the reflective cracking potential when there may be severe cracking in existing structures. Due to concerns with severe cracking in existing pavements, the NDDOT has commonly used FDR for rehabilitation projects. Stabilizing agents have not been utilized frequently in FDR, but Portland cement is typically used on select FDR projects. If there is cement stabilization, which is normally limited to avoid creating a stiff layer, cement percentages are determined by the mix design. For both CIR and FDR, 0.25 is set as the layer coefficient for the AASHTO empirical pavement design procedure. These recycled sections are typically expected to yield 20 years of service life with good performance.

2.2.2 Mississippi DOT (MSDOT)

In the past 12 years, around eight SFDR rehabilitation projects have been completed by MSDOT. A significant extent of stabilization is performed on all pavement projects across the state due to significant soil stability concerns. For every subgrade exposure, cement is added for stabilization. Most four-lane and even two-lane sections have two stabilized layers in the pavement structure, but these rehabilitations are frequently performed on older pavements with only an un-stabilized granular base. Mississippi Test Method 25 is the MSDOT specification for soil-cement stabilization. This same approach is also used for stabilizing cold recycled and reclaimed layers. This approach specifies the technique of compaction for these layers, namely the Proctor method when reclamation is used with a significant amount of fines and soil materials with RAP. In addition, the Superpave gyratory technique is employed if the RAP material consists mostly of RAP without a significant amount of soil, and this is based primarily on experience and not defined in the test method. In terms of pavement design inputs, these SFDR layers are assumed to have an AASHTO empirical design method layer coefficient value of 0.20, which is same as value used by MSDOT for typical soil-cement layers. It was noted that the majority of SFDR treated pavements do achieve higher performance than their counterparts with only soil-cement stabilized layers. The average life of SFDR treated pavements within MSDOT’s current pavement management system is approximately 13-14 years. It is anticipated that these SFDR-treated pavements would not require rehabilitation during their design lives, and if any rehabilitation is required, it will mostly be in the form of a wearing course replacement.

Future rehabilitation projects incorporating CCPR and FDR are presently being explored by MSDOT. Current constraints on CCPR development are a lack of storage space (stockpiling area).

2.2.3 Illinois DOT (IDOT)

The preference for CIR and CCPR over FDR is more common IDOT practice, however, there are a significant number of FDR treatments conducted in Illinois by the local agencies. Since most state-level roads are composite (i.e., asphalt over PCC), FDR treatments have not yet been considered for them. In terms of pavement design inputs, the AASHTO empirical design process layer coefficient value for these CIR, CCPR recycled layers is set at 0.28 and the overlay/wearing course is 0.40. From a design standpoint, these recycled structures are predicted to have a 15-year life, with budget being the determining factor in selecting whether to design for a longer life. One of the IDOT CIR project (US Route 24) information has been shared with the research team, this project has performed very well over last 12 years. Figure 2-8 illustrates the relationship between the performance of a composite pavement (asphalt overlay on PCC) as predicted by IDOT's pavement management system against the US 24 CIR pavement in terms of IDOT's Condition Rating Survey measure.

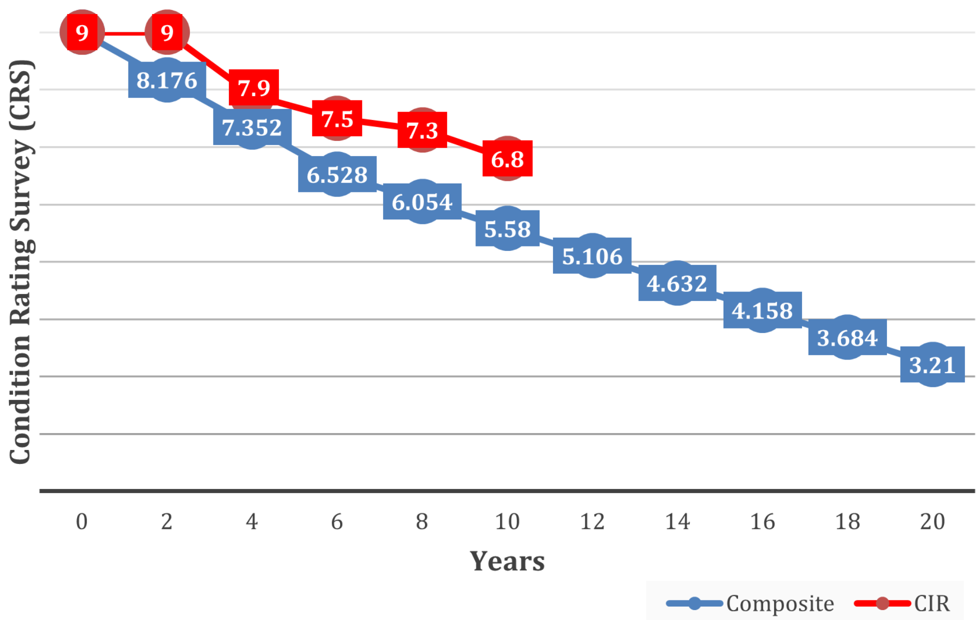


Figure 2-8 Illinois DOT US 24 condition rating survey; CIR vs. composite pavement performance

The interviews with three NRRA agencies provided the research team with added information that supplements the information gathered through literature review and agency survey. Interviews reaffirmed that agencies have seen very favorable pavement longevity and performance after rehabilitating distressed asphalt pavements using CIR, FDR, and SFDR. It was also confirmed that CIR and CCPR techniques are still relatively new for some of the DOTs and that there is willingness to use these in future. Lastly, the use of AASHTO empirical pavement design approach is still predominant when using cold recycled layers to rehabilitate pavements.

Chapter 3: Laboratory Testing

3.1 Test Sections

According to the 2022 MnROAD Construction Activities report [67], four CIR test sections were constructed as part of the 2022 NRRA MnROAD construction. These test sections include 2201 (constructed in old test section 101), 2202 (constructed in old test section 201), 2207 and 2208 (constructed in old test section 4) on MnROAD I-94 WB mainline. Test sections 101 and 201 were resurfaced in 2017 while maintaining the existing aggregate base and subgrade from the original construction in 1994. Test section 4 was rehabilitated using SFDR technique in 2008 while maintaining its original subgrade from 1994. The test sections have a cross section of 38 feet including 9 feet for driving shoulder, 12 feet for driving lanes, 13 feet for passing lane, and 4 feet for passing shoulder [67]. Figure 3-1 presents the typical sections for the study test sections.

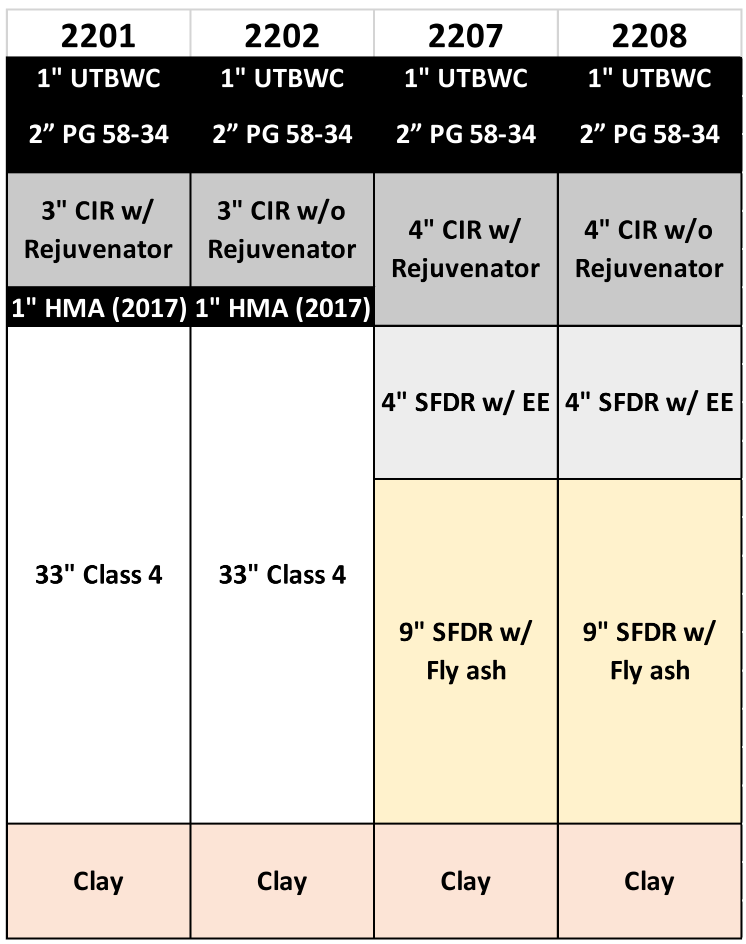


Figure 3-1 Typical sections for test sections 2201, 2202, 2207, and 2208 (UTBWC: Ultra-thin Bonded Wearing Course; CIR: Cold In-place Recycling; SFDR: Stabilized Full-Depth Reclamation; EE: Engineered Emulsion; HMA: Hot Mix Asphalt; PG: Performance Grade)

For test sections 2201 and 2202, the existing bituminous surface was initially milled in such a way that 2 to 2.5 inches of in-place bituminous layer remained. In order to construct a 3-inch CIR mat, sufficient RAP millings from the same section were added to the milled surface prior to the CIR operation. These two sections received 2 inches of bituminous overlay (PG 58-34) followed by one inch of Ultra-thin Bonded Wearing Course (UTBWC). For test section 2201, rejuvenator was added to the Engineering Emulsion (EE) [67].

For test section 2207 and 2208, first the existing bituminous overlay was entirely milled to the surface of the SFDR layer. Next, 4 inches of CIR was performed on the old SFDR layer. Similar to 2201 and 2202, these two sections also received 2 inches of bituminous overlay (PG 58-34) followed by one inch of UTBWC. For test section 2207, rejuvenator was added to the EE [67].

The rejuvenator emulsion was combined with EE at two percent by weight rejuvenator emulsion and 98 percent by weight EE for both 2201 and 2207 test sections. In this process, the rejuvenator emulsion was added to the finished EE emulsion.

3.2 Original Mix Designs

The original laboratory CIR mix designs were performed by the mix design consultant selected by the contractor in accordance with MnDOT Grading and Base Manual. Mix designs were initially performed on test sections 2202 and 2208. Test sections 2201 and 2207 which contained rejuvenator emulsion, were verified at the optimum emulsion contents obtained from 2202 and 2208 mix designs, respectively. Table 3-1 presents the RAP gradations for the test sections which are substantially similar. Table 3-2 summarizes the mix designs results.

Table 3.1 RAP gradation of the test sections

Sieve Size		Percent Passing	
mm	**inch**	**2201 and 2202**	**2207 and 2208**
25.0	1	100	100
19.0	3/4	93	93
12.5	1/2	70	70
9.5	3/8	60	60
4.75	#4	45	45
2.36	#8	28	28
1.18	#16	14	17
0.600	#30	6	8
0.300	#50	2	3
0.150	#100	1	1
0.075	#200	1	1

Table 3.2 CIR mix designs summary

Test Section	2201	2202	2207	2208	CIR Specification Limits
Optimum Emulsion Content (%)	2.2	2.2	1.7	1.7	--
Rejuvenator	Yes	No	Yes	No	--
Cement (%)	0.5	0.5	0.5	0.5	--
Bulk Density, lb/ft^3	130.1	132.8	130.6	132.5	--
Air voids (%)	16.8	15.1	15.7	13.8	--
Marshall Stability, dry (lb)	1574	1341	1411	1279	**1250, Min**
Marshall Stability, soaked (lb)	1155	1186	1085	1093	--
Retained stability (%)	73	88	77	85	**70, Min**
Raveling, Mass Loss (%)	--	1.2	--	0.6	**2.0, Max**
Critical low temp (°C)	--	-29	--	-31	**Report**

As Table 3-2 shows, the mixtures with rejuvenator (2201 and 2207) had higher dry Marshall stabilities compared to the companions without rejuvenator (2202 and 2208) while the wet Marshall stabilities were about the same. Test sections 2202 and 2208 resulted in higher retained stabilities.

3.3 Mixture Properties after One-Year in Service

Cores were extracted from the test section by NRRA after one year of service from both the driving and passing lanes to study the in-place CR materials properties. From each test section, 25 6-inch diameter cores and 5 4-inch diameter cores were collected.

The cores were shipped to Braun Intertec lab where they were cut at appropriate heights to produce CIR pucks. Several tests and measurements were conducted on the CIR mixtures including bulk density, air void content, Marshall stability (dry and soaked), critical low temperature, and Illinois Flexibility Index Test (I-FIT). The 4-inch pucks were used for Marshall stability testing: two specimens for each condition (dry and wet). All other tests were performed on the 6-inch diameter pucks. For each test section, 6 semi-circular specimens were tested for I-FIT.

Table 3-3 provides some of the mixture testing results for the study mixtures after one year of service. Unfortunately, the 4-inch cores received from 2202 were not thick enough to produce intact CIR pucks and therefore, Marshall testing was not achievable for this section.

Table 3.3 CIR properties after one year of service

Test Section	2201	2202	2207	2208
Bulk Density, lb/ft^3	134.1	--	131.2	135.9
Air voids (%)	11.9	--	13.9	13.8
Marshall Stability, dry (lb)	3400	--	2065	2315
Marshall Stability, soaked (lb)	1380	--	795	1100
Retained stability (%)	41	--	38	48
Critical low temp (°C)	-22	-22	-24	-23

Figure 3-2 presents the bulk densities of the test sections from their original mix designs and after one year of service. The values over the bars show the percent differences. As this figure shows, bulk densities are comparable with the maximum difference of 3 percent.

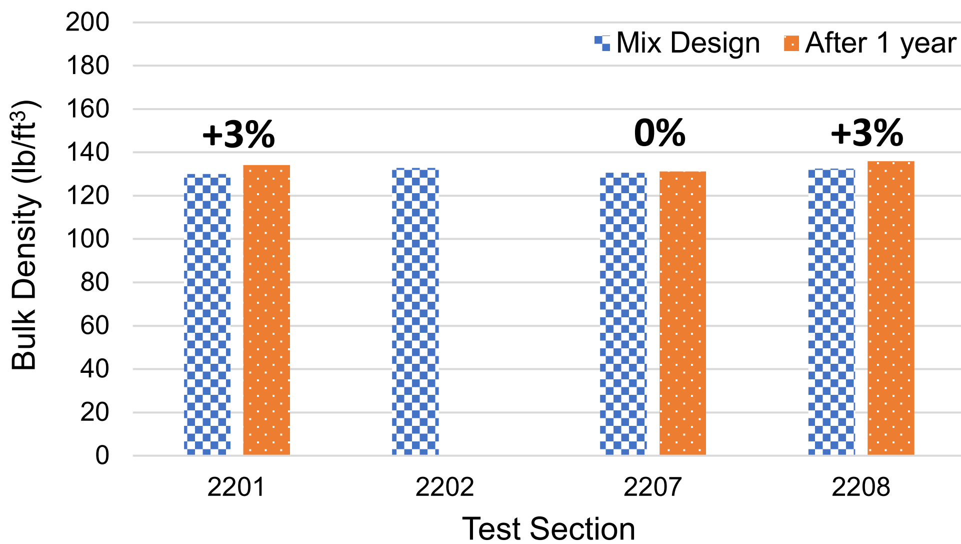


Figure 3-2 Bulk density comparison: mix design vs. one year after service

Figure 3-3 illustrates the air void content of the test sections from their original mix designs and after one year of service. As this figure suggests, the field air void was generally lower than what was achieved during mix design procedure in the lab. This may suggest that either the CIR layer has received relatively higher compaction effort during construction compared with the mix design procedure compaction or the traffic has reduced the air void of the CIR layer by the wheel load compaction during the first years of service.

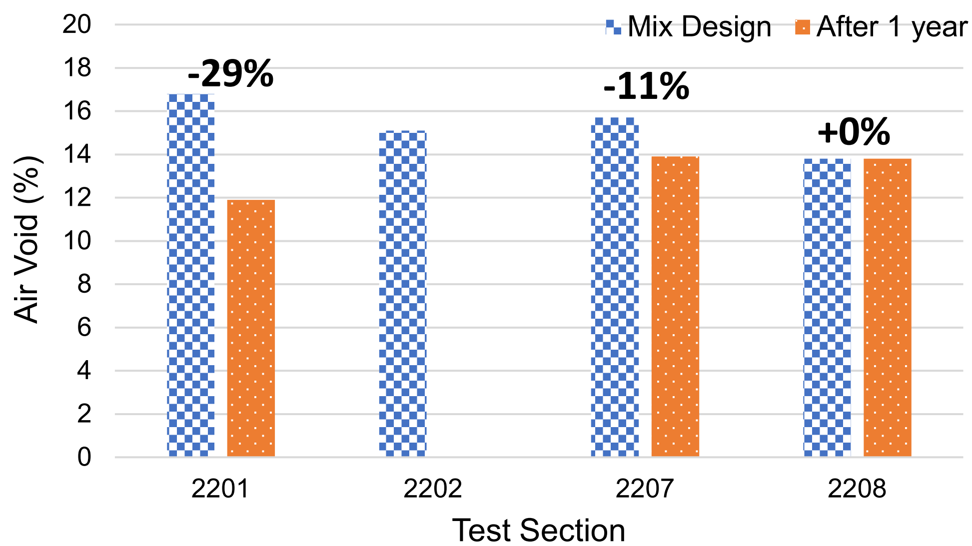


Figure 3-3 Air void content comparison: mix design vs. 1 year after service

Figure 3-4 presents the Marshall dry stability of the test sections from their original mix designs and after one year of service. As this figure shows, dry stabilities after one year of service are significantly higher than the original mix designs which is in agreement with the decrease observed in air void contents of the mixtures after one year of service, except for test section 2208.

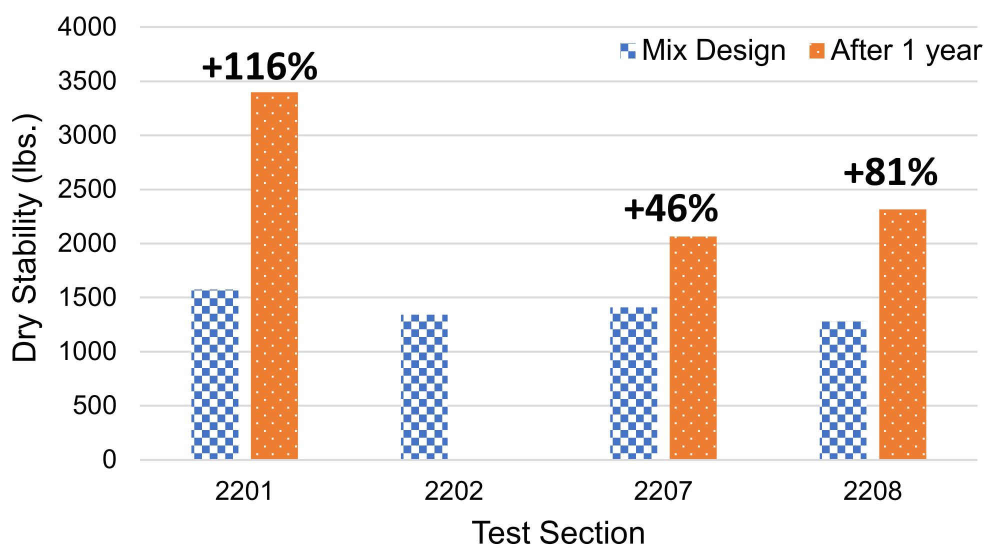


Figure 3-4 Marshall dry density comparison: mix design vs. one year after service

Figure 3-5 illustrates the Marshall wet stability of the test sections from their original mix designs and one year after service. As this figure suggests, wet stability differences were not consistent among the test sections; 2201 was higher, 2207 was lower, and 2208 was about the same as the original mix design results.

Figure 3-6 shows the Marshall retained stabilities which are significantly lower after one year of service compared with the original mix design values. This is mainly due to the notable increase in dry densities as shown in Figure 3-4. By definition, the retained stability is the ratio of the density of the conditioned samples to the density of the dry samples. A jump in the dry density, increases the denominator, and therefore, reduces the ratio.

As Table 3-2 shows, the CIR specification specifies a minimum dry stability of 1,250 pounds and a minimum retained stability of 70 percent. Even though the retained stability of after one year of service mixtures fell below 70 percent, it should be noted that the wet densities are still above 875 pounds (1,250 x 70% = 875) for all the test sections. As such, the mixtures are expected to perform adequately in terms of moisture susceptibility.

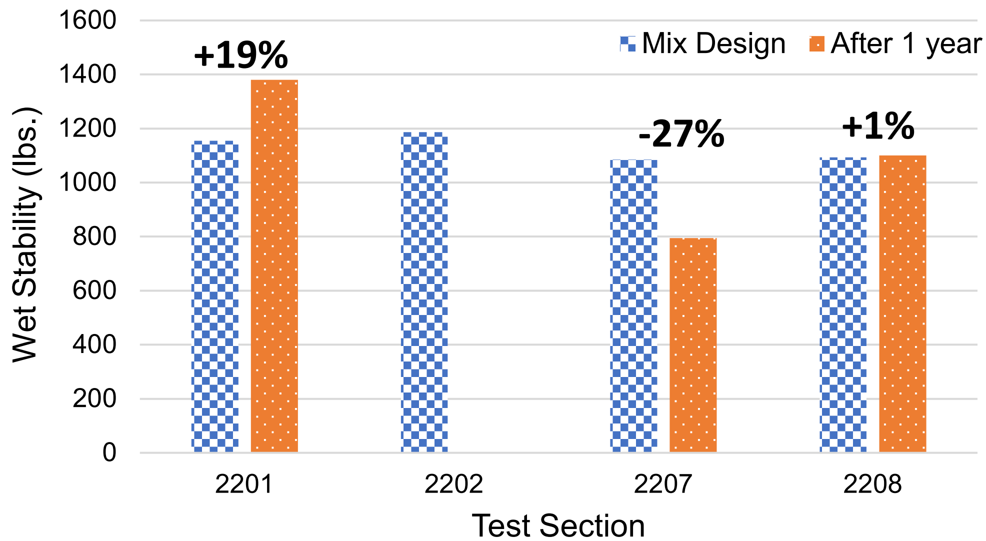


Figure 3-5 Marshall wet density comparison: mix design vs. one year after service

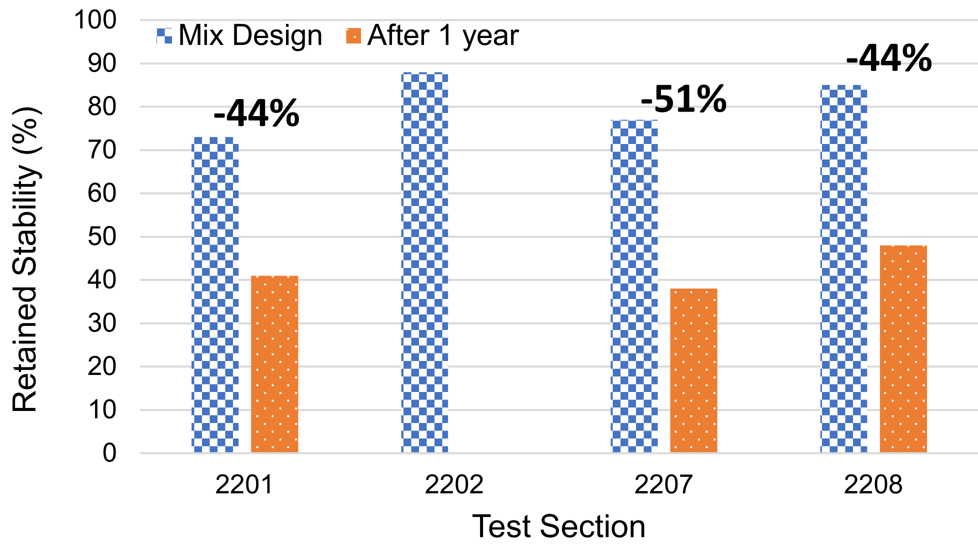


Figure 3-6 Marshall retained stability comparison: mix design vs. one year after service

Figure 3-7 presents critical low temperatures of the test sections one year after service which are relatively close and in the range of -24 to -22°C. Figure 3-8 compares the critical low temperature of test sections 2202 and 2208 one year after service with their original mix design values. As this figure shows, the critical low temperatures in the case one year after service are about 25 percent higher in both cases. This suggests a worse low temperature performance of the mixtures after one year of service compared with the original mix designs.

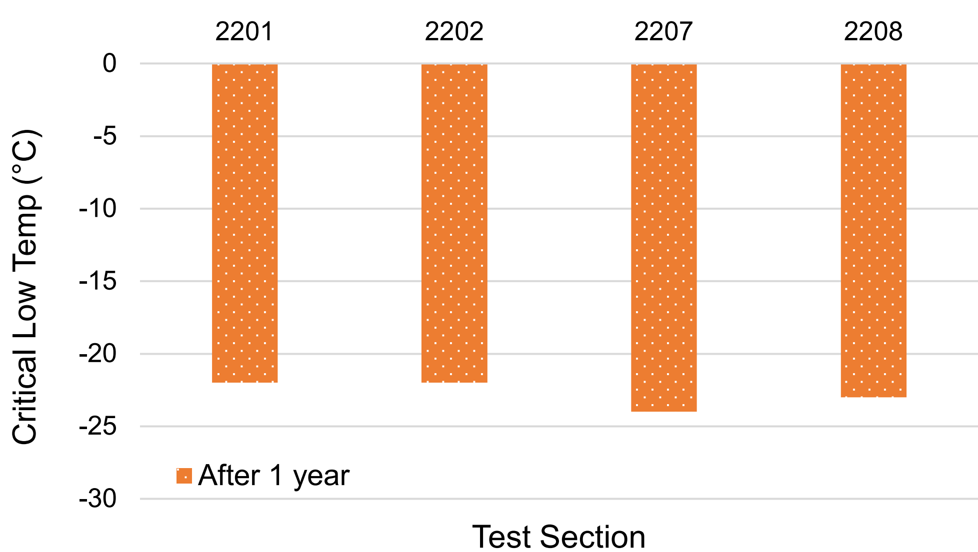


Figure 3-7 Critical low temp of study mixtures after one year of service

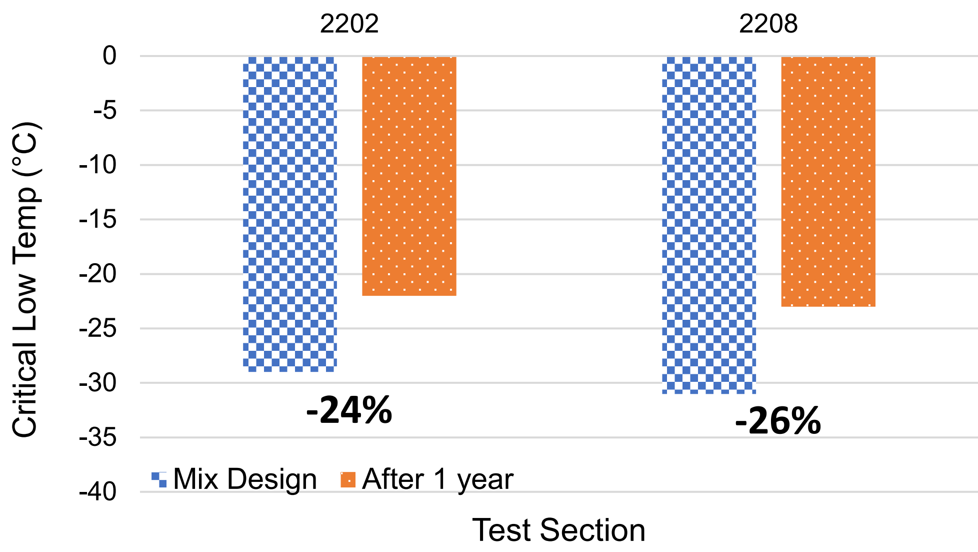


Figure 3-8 Critical low temperature comparison: mix design vs. after one year of service for 2202 and 2208

I-FIT testing was conducted in accordance with AASHTO T-393 to determine the fracture potential of the mixtures after one year of service. Figure 3-9 shows the peak loads of the study mixtures. The error bars show one standard deviation. As Figure 3-9 shows, 2202 shows significantly higher peak load among all the mixtures. The mixtures without rejuvenator (2202 and 2208) show higher average peak loads compared to the companions with rejuvenator (2201 and 2207). Although 2207 and 2208 results seem to have no statistically significant difference.

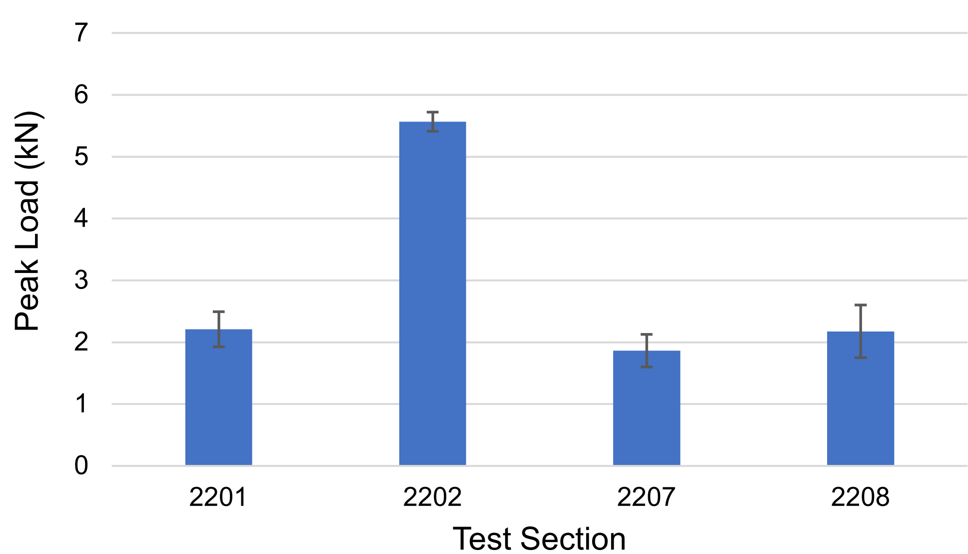


Figure 3-9 Peak load of the study mixtures during I-FIT

Figure 3-10 presents the fracture energy of the study mixtures. As this figure shows, like the peak load, 2202 has the highest fracture energy among the tested mixtures. The mixtures without rejuvenator

(2202 and 2208) show higher average fracture energies compared to the companions with rejuvenator (2201 and 2207). Although 2207 and 2208 results appear to have no statistically significant difference.

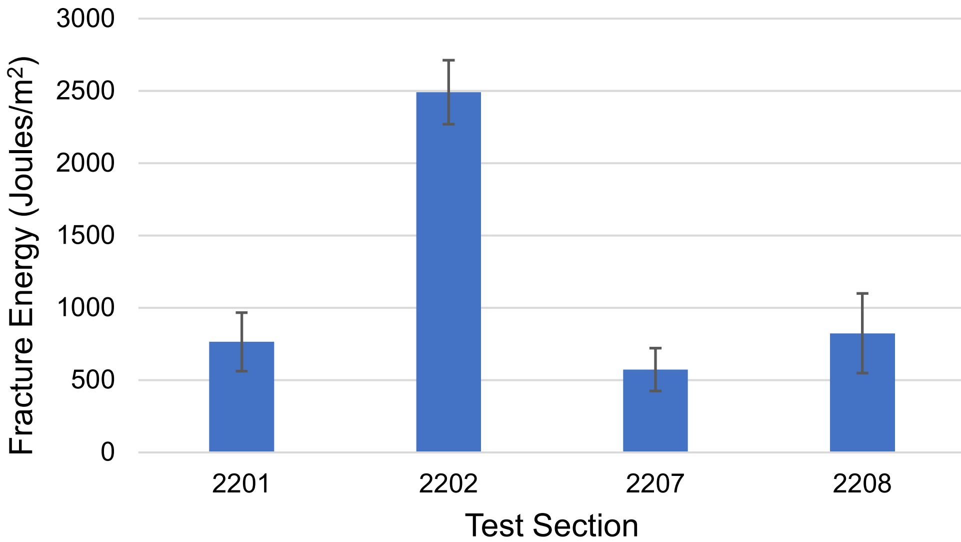


Figure 3-10 Fracture energy of the study mixtures from I-FIT

Figure 3-11 presents the flexibility index of the study mixtures. In general, the larger the FI number, the more resistant the mixture is to premature overall cracking-related damages. As Figure 3-11 shows, the mixtures without rejuvenator (2202 and 2208) show higher average flexibility indices, and therefore better crack resistance, compared to those with rejuvenator companions (2201 and 2207).

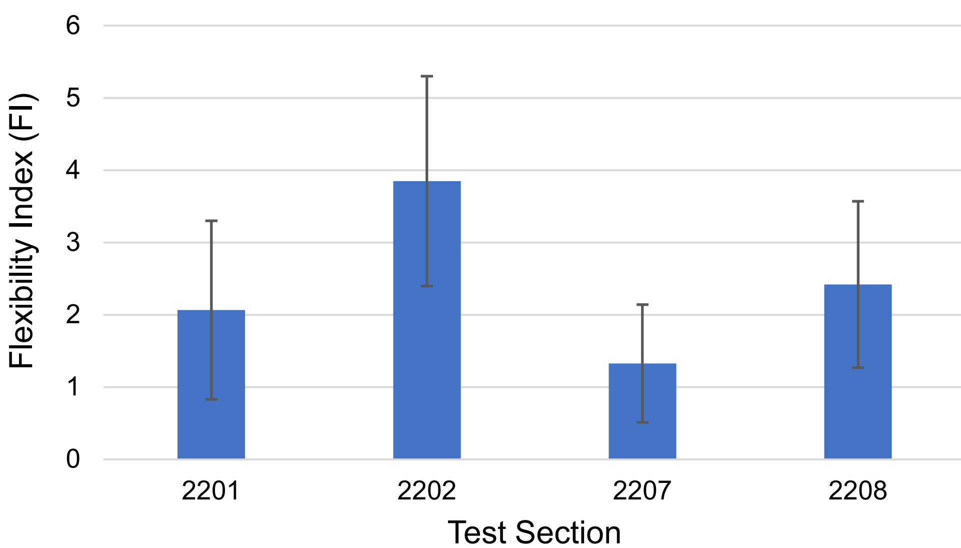


Figure 3-11 Flexibility index of the study mixtures from I-FIT

As it can be seen from the I-FIT results, 2202 has significantly higher peak-load and fracture energy among the tests sections. Unfortunately, information on the Marshall stability, density, and air void of this test section is not available at the time of writing this report to better understand the root cause of this difference as the 4-inch cores were collected from this test section were not thick enough to produce intact CIR pucks.

Overall, even though the rejuvenation showed some potential for performance improvement in the mix design phase with increase in the dry Marshall stabilities (Table 3-2), the after one year of service mixture testing either do not show a remarkable change in the performance of the mixture in the presence of rejuvenator or show some deteriorations in some cases (e.g., lower flexibility indices of with rejuvenator mixtures compared with without rejuvenator mixtures).

3.4 Binder Testing

3.4.1 Testing Method Overview

The binder testing methods conducted in this task include rheological characterization, Multiple Stress Creep and Recovery (MSCR), and Linear Amplitude Sweep (LAS) using the Dynamic Shear Rheometer (DSR). Details of the various tests are described further in subsequent sections. The materials evaluated using these tests include the extracted and recovered binders from the Reclaimed Asphalt Pavement (RAP) sampled at the time of construction from the test sections 2201 and 2202 (designated as test section 2201-RAP and test section 2202-RAP), the base binder, the base binder with added rejuvenator, and the extracted and recovered binders from the test sections after one year of service (test section 2201-1yr and test section 2202-1yr). For both MSCR and temperature/frequency sweep tests, the base binder was tested unaged. Table 3-4 summarizes the tests and the materials evaluated.

Table 3.4 Binder testing method and materials

Test Methods	Materials				Significance
	RAP	Base Binder	Base Binder + rejuvenator	Component materials after 1-yr in service	
Temperature and frequency sweep	X	X	X	X	Develop complex shear modulus and phase angle master curves and determine rheological indices (e.g., Glover-Rowe parameter)
Multiple Stress Creep and Recovery (MSCR)	X	X	X	X	Determine permanent deformation performance parameter of non-recoverable shear compliance (J_{nr}) and strain tolerance by measurement of percent strain recovery measures.
Linear Amplitude Sweep (LAS)	X			X	Determine viscoelastic continuum damage-based performance parameters for the binders that have been shown to correlate with the fatigue damage potential.

3.4.1.1 Temperature and Frequency Sweep

The temperature-frequency sweep was performed using the DSR with a 4-mm plate to measure the performance grade and the rheological parameters of the recovered RAP binder materials and base binders. This test encompasses a broad temperature range (-18°C to 40°C, in 3-degree increments) and frequency range (15 frequencies from 100 rad/sec to 0.2 rad/sec) by utilizing the appropriate strain level (within linear viscoelastic range) at each combination of test temperature and frequency. The isotherm tests are performed from coldest to warmest temperature and from highest to lowest frequency. The complex shear modulus and phase angle master curve is then constructed using RHEA software following Equation 3-1.

$$G^* = G_g^* \left(1 + \left(\frac{\omega}{\omega_c} \right)^{\log(2)/_R} \right)^{R/_{\log(2)}}$$

Equation 3-1

In which:

G^* = Complex modulus (Pa) at frequency ω (rad/s),

G^*_g = Glassy modulus asymptote variable (assumed 1GPa),

ω_c = Cross-over frequency, and

R = Rheological Index or "R-Value".

R-value as depicted in Equation 3-1 is an important parameter for viscoelastic material characterization. A higher R-value is indicative of increased brittleness and susceptibility to cracking in aged binders.

The Glover-Rowe [68] parameter is calculated from the DSR measurements using Equation 3-2. This parameter has a strong correlation with ductility and is therefore used to estimate fatigue and thermal cracking resistance.

$$G - R = \frac{G^*(\cos\delta)^2}{\sin\delta}$$

Equation 3-2

In which:

G^* = Complex shear modulus at 25°C, 0.005 rad/s, and

δ = phase angle value at 25°C, 0.005 rad/s.

A lower G-R value suggests more resistance to durability cracking. A limiting value of 180 kPa is given for the initiation of cracking, a second value of 600 kPa is suggested for the development of severe cracking (i.e., block cracking) [68].

3.4.1.2 Multiple Stress Creep and Recovery (MSCR)

This test method measures the mechanical properties of the binder materials. This technique involves a creep and recovery test in which the binders were subjected to a constant load for a predetermined amount of time before being allowed to recover. Twenty (20) cycles were executed at a 0.1 kPa stress level, followed by ten cycles at a 3.2 kPa stress level. The primary result of MSCR is the non-recoverable creep compliance J_{nr}, which represents the binder strain response to the applied stress levels. Greater values of J_{nr} indicate a greater susceptibility to plastic deformation and, hence, a decreased resistance to rutting. Percent recovery (%Recovery) is likewise derived from MSCR and reflects the proportion of strain that is recovered at each stress level. Lower values of J_{nr} indicate increased rutting resistance and more demanding applications. J_{nr} values determine traffic levels for binder selection. Traffic threshold values for binder selection are categorized as "S" for standard traffic with traffic levels less than 10 million Equivalent Single Axle Loads (ESALs) or speeds greater than 70 km/h, "H" for heavy with traffic levels between 10 and 30 million ESALs or speeds between 20 – 70 km/h, "V" for very heavy with traffic levels greater than 30 million ESALs or speeds lower than 20 km/h, and "E" for extremely heavy with traffic levels greater than 30 million ESALs, in order of decreasing J_{nr} and increasing rutting resistance. The testing temperature for all binders tested following Minnesota climate are summarized in Table 3-5.

Table 3.5 MSCR test temperature for binders

Binder	Testing temperature
Test section 2201-RAP	58°C
Test section 2202-RAP	58°C
Test section 2201-1yr	58°C
Test section 2202-1yr	58°C
Base Binder	46°C, 52°C and 58°C
Base Binder + rejuvenator	46°C, 52°C and 58°C

3.4.1.3 Linear Amplitude Sweep (LAS) test

The LAS test measures the binder's resistance to fatigue damage. This is an oscillatory strain sweep test that causes damage to the binder by applying load amplitudes that increase linearly. The LAS test comprises of two steps: first, a frequency sweep performed to determine the undamaged material's properties and the binder's rheological characteristics. A linear amplitude strain sweep is then used to assess the damage characteristics of the binder. In accordance with AASHTO TP101, frequency sweeps were performed with a strain amplitude of 0.1 percent and a frequency range of 0.1 to 30 Hz. The test temperature was set at 19°C for all the extracted and recovered binders. This temperature was selected to accurately capture binder failure, as elevated temperatures would cause plastic flow rather than failure due to microcracking. In addition, lower temperatures would lead to a loss of adhesion between the binder sample and the DSR plates. The amplitude sweep test was performed at a 10 Hz frequency. The testing technique involved delivering a strain that increased linearly from 0 percent to 30 percent over 3,100 loading cycles. The tests were performed using a DSR device with a parallel plate of 8 mm diameter and a 2 mm gap. Using Equation 3-3 through Equation 3-5, the integrity (C(t)) and damage accumulation (D(t)) curves (fatigue law, also known as Damage Characteristic Curve (DCC)) of the binder sample during the test were determined. The relationship between C(t) and D(t) can be fit to the power law as shown in Equation 3-5.

$$C(t) = \frac{G^* \sin\delta(t)}{G^* \sin\delta_{initial}}$$

Equation 3-3

$$D(t) = \sum_{i=1}^{N} [\gamma_0^2 \pi (C_{i-1} - C_1)]^{\frac{\alpha}{1+\alpha}} (t_i - t_{i-1})^{\frac{1}{1+\alpha}}$$

Equation 3-4

$$C(t) = C_0 - C_1(D)^{C_2}$$

Equation 3-5

In which:

$G*$ = Complex modulus,
δ =Phase angle,
γ_0= The applied strain,
α is calculated from the frequency sweep test, and
t = test time.
C_0, C_0, C_0 = model fitting parameters

Using Equation 3-6 through Equation 3-9, the number of cycles until failure is computed. Under the current AASHTO TP101, the definition of failure for the LAS test is defined as a 35 percent reduction in the initial modulus.

$$N_f = A(\gamma_{max})^B$$

Equation 3-6

$$A = \frac{f\left(D_f\right)^k}{k(\pi I_D C_1 C_2)^\alpha}$$

Equation 3-7

$$B = 2\alpha$$

Equation 3-8

$$D_f = 0.35(\frac{C_0}{C_1})^{\left(\frac{1}{C_2}\right)}$$

Equation 3-9

In which:

A and *B* = Viscoelastic Continuum Damage (VECD) model coefficients,
γ_{max} = applied strain,
D_f = Damage at failure @ 35% reduction in $G^* \sin\delta$,
f = Loading frequency (10 Hz), and
α = fitting coefficient from log (G') versus log (w).

Given the obtained *A* and *B* parameters from the LAS test, the N_f versus strain curve may be computed using Equation 3-6. Figure 3-12 shows a typical plot. In general, the *A* parameter measures the material's ability to maintain its integrity during repeated loading cycles and cumulative damage. This parameter has a direct relationship with the storage modulus. In other words, as the storage modulus drops during loading cycles, the *A* parameter decreases, indicating a reduced capacity of the binder to maintain its integrity. The *B* parameter characterizes the asphalt binder's sensitivity to strain level. Larger absolute values of *B* imply that the fatigue life declines at a greater rate (steeper (negative) slope of the N_f-strain curve as depicted in Figure 3-12) as strain level increases. In general, fatigue-resistant binders have greater *A* values and lower absolute *B* values [69].

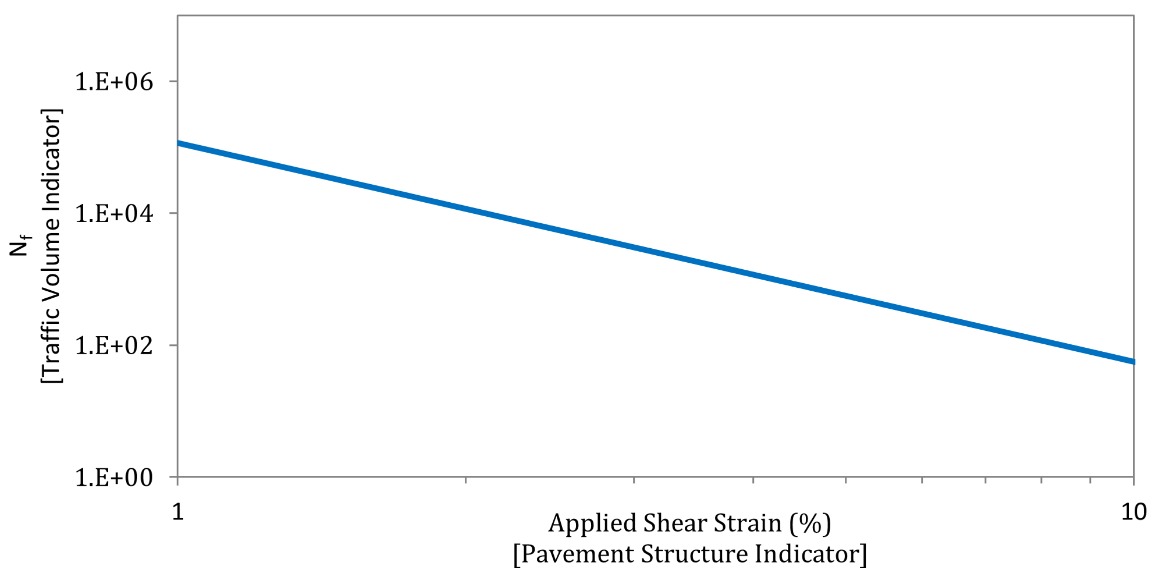


Figure 3-12 Typical fatigue characterization (N_f-strain) plot from the LAS test

Zhang et al., [70] developed three fatigue performance parameters for asphalt binders based on viscoelastic continuum damage (VECD) and failure energy from the LAS test results. These are average reduction in integrity up to failure (I^R), strain tolerance (ϵ_T) and strain energy tolerance (E_f). The I^R parameter evaluates the fatigue properties of asphalt binders with different ageing conditions. The calculation of I^R is shown in Equation 3-10 below:

$$I^R = \frac{\int_0^{N_f}(1-C)dN}{N_f}$$

Equation 3-10

In which

N_f = number of load cycles to failure (defined as peak stress point from the LAS test, and
C = Pseudo stiffness, calculated by Equation 3-3.

As specified in this study, a higher I^R value is preferred, indicating the material's capability to resist fatigue cracking. The strain tolerance (ϵ_T) captures the important post-peak behavior of binder during the LAS test which is ignored by the I^R parameter. In order to determine ϵ_T, an 'end' point for the data to be included in the analysis must be specified. On the post-peak portion of the stress-strain curve, various points are examined, including the inflection point (the point with the lowest negative slope on the stress-strain curve where the shear stress drops considerably), 50 percent of the maximum stress point, and the 25 percent maximum stress point. ϵ_T is the strain level that corresponds with the specified 'end' point. The strain energy tolerance (E_f) is an energy-based index for asphalt binders employed to evaluate the fatigue properties. As shown in Figure 3-13, the area under the stress-strain curve to the end point (25 percent of maximum stress) is defined as the strain energy tolerance (E_f). These three developed parameters are explored in this report.

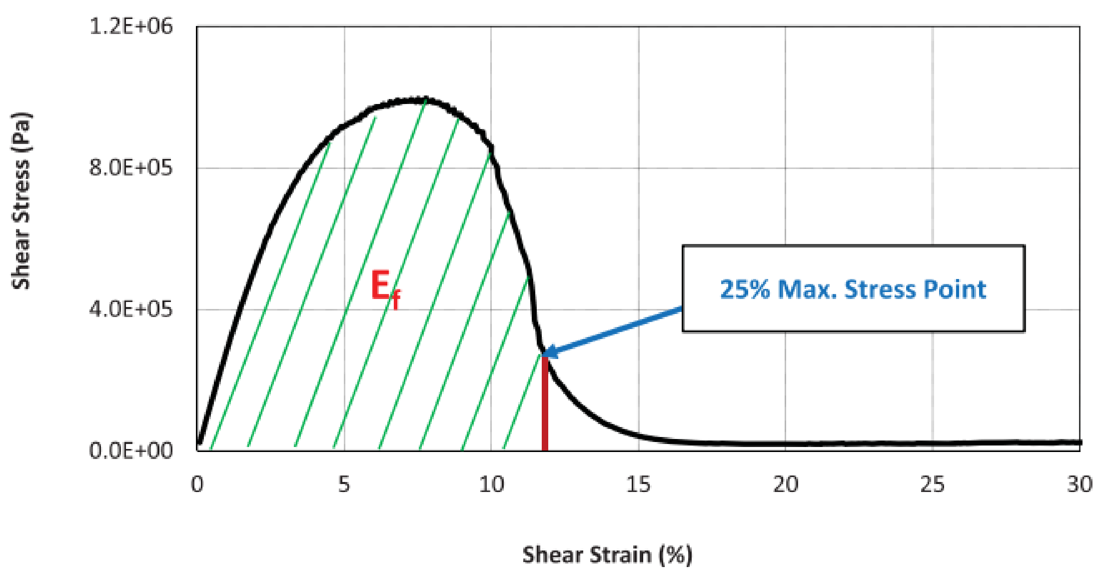


Figure 3-13 Illustration of E_f calculation [70]

3.4.2 Results and Discussions

This section focuses on the binder testing results from the base binders, base binders + rejuvenator, and the extracted and recovered binders from the RAP and 1-yr service materials (Test section 2201-RAP, Test section 2202-RAP, Test section 2201-1yr, and Test section 2202-1yr).

3.4.2.1 Temperature and frequency sweep

$|G^*|$ and phase angle master curves for all evaluated binders are presented in Figure 3-14 and Figure 3-15, respectively. From Figure 3-14, all binders appear to be comparable in terms of stiffness at higher reduced frequency (low temperature). A noticeable difference is seen when comparing the lower reduced frequency (high temperature), where both the base binder and binder + rejuvenator exhibit lower $|G^*|$ values relative to the component materials derived during construction and after one year in service. Alongside the complex modulus, the analysis of phase angle demonstrates similar characteristics for all materials at higher reduced frequency. However, at lower reduced frequency, both the base binder and the base binder with rejuvenator exhibit a significantly higher phase angle.

According to Equation 3-2, the Glover-Rowe parameter for each material was derived from master curve data at a frequency of 0.005 rad/s and reference temperature of 15°C. Figure 3-16(a-b) shows the G-R points in the Black space and corresponding G-R values presented in a bar graph. The evolution of the field-observed cracking distresses has been connected to the final locations in Black Space [71]. Test section 2201-RAP, test section 2202-RAP, test section 2201-1yr, and test section 2202-1yr all appear to be comparable. The near-threshold cracking behavior observed in both Test section 2201-RAP and Test section 2202-RAP can be attributed to their aged condition, the presence of a rejuvenator to mitigate this effect is not apparent given the comparable nature of all materials (binders after 1-yr in service). The reduced $|G^*|$ implies that both base binder materials have a softening effect, which leads to a reduction in cracking. The change in phase angle signifies the modification, consequently a high relaxation capability. The G-R parameter shown in Figure 3-16(b) are derived from the master curve analysis. Since there was a single master curve for each binder evaluated, no statistical analysis can be performed.

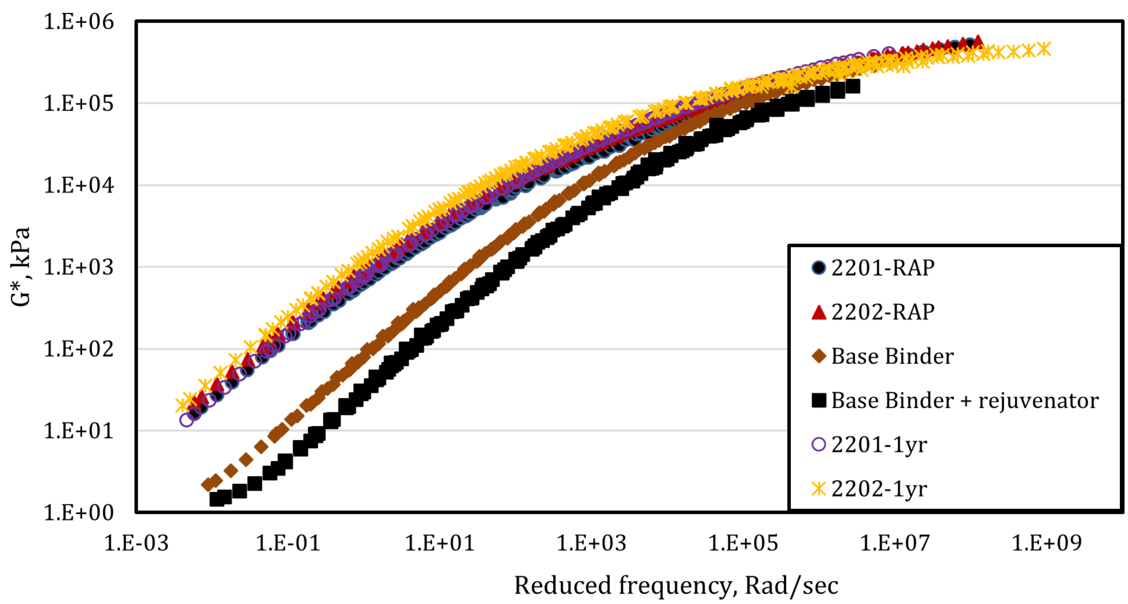


Figure 3-14 Complex modulus master curves for all binders @ 25°C

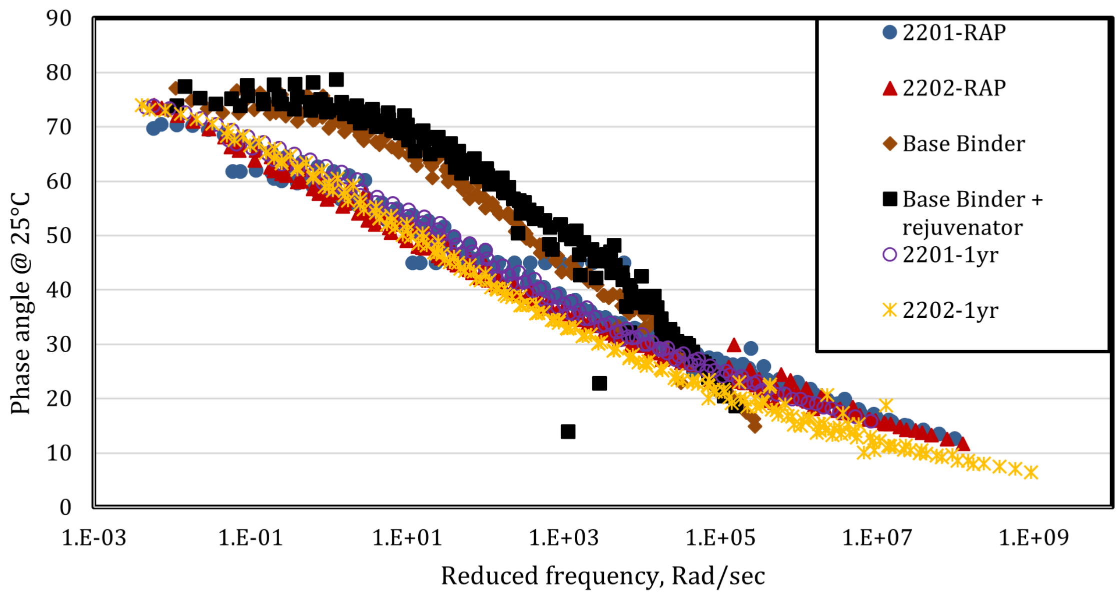
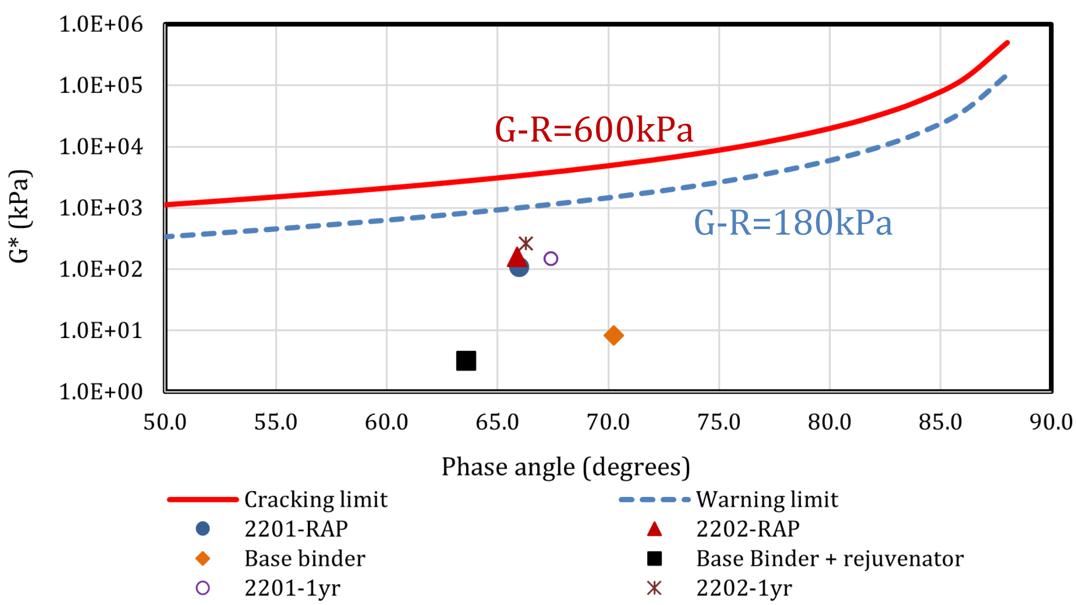
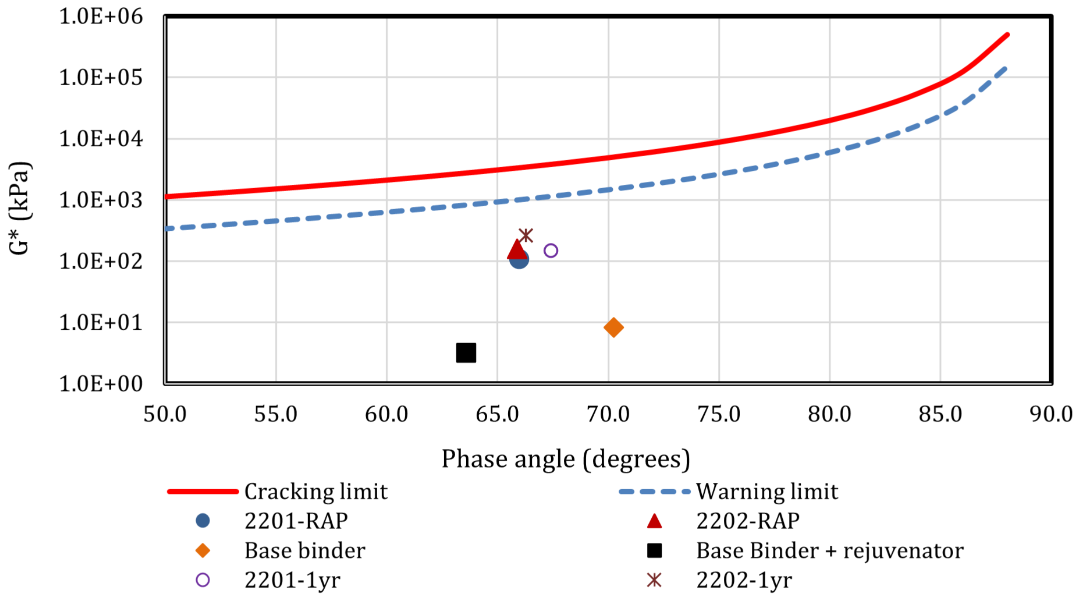


Figure 3-15 Phase angle master curve for all binders @ 25°C



(a)



(b)

Figure 3-16 (a) Black space for all binders, and (b) G-R values for all materials

3.4.2.2 Multiple creep stress and recovery (MSCR)

Non-recoverable creep compliance (J_{nr}) and elastic recovery were determined from the MSCR test results at stress levels of 0.1 kPa and 3.2 kPa and the binders were subsequently graded according to AASHTO M332. All binders were tested at a temperature of 58°C representative of the Minnesota climate PG requirement. Since neither base binder nor base binder + rejuvenator had measurable recovery at 0.1 kPa and 3.2 kPa stress levels, the MSCR test was conducted at selected lower temperatures (46°C and 52°C) to detect any potential recovery from the binders and subsequent non-recoverable creep compliance. Elastic recovery results and non-recoverable creep compliance (J_{nr}) at the 0.1 kPa stress level for all binders are presented in Figure 3-17 and Figure 3-18, respectively.

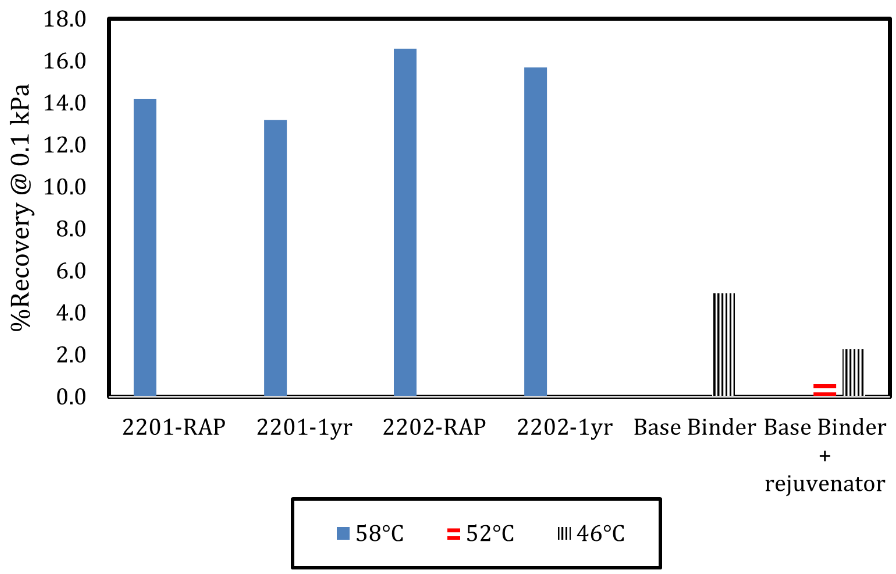


Figure 3-17 %Recovery at 0.1 kPa for all binders and test temperatures.

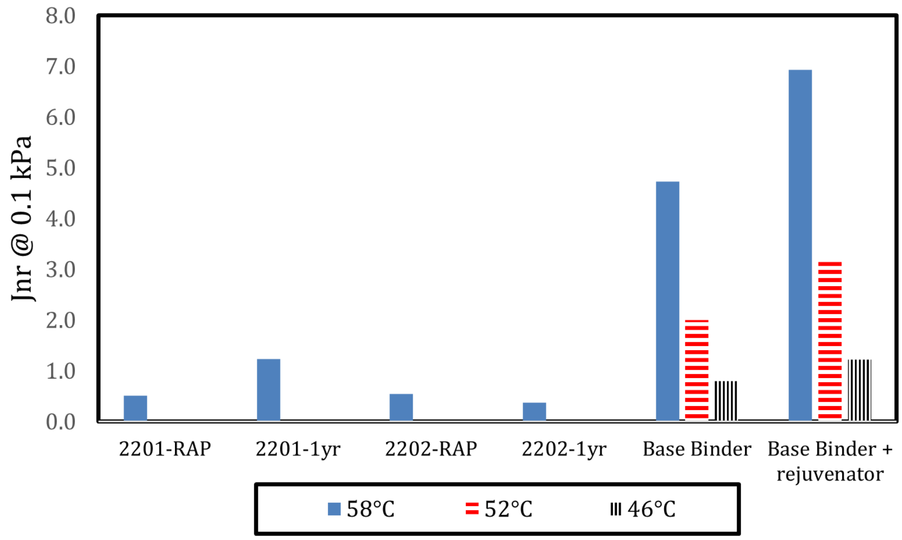


Figure 3-18 Jnr at 0.1 kPa for all binders and test temperatures.

Comparing materials extracted and recovered at the time of construction, Test section 2202-RAP shows better elastic recovery compared to Test section 2201-RAP at 0.1 kPa stress level. Same conclusions are consistent for the binders extracted and recovered 1-yr after service: Test section 2202-1yr showing better elastic recovery compared to Test section 2201-1yr. No elastic recovery was observed for either the base binder or the base binder + rejuvenator. J_{nr} at 0.1 kPa for Test section 2201-RAP shows better rutting resistance than Test section 2202-RAP, though not significant and may be comparable. Same applies to base binder in comparison with base binder + rejuvenator.

As for 3.2 kPa stress level (Figure 3-19 and Figure 3-20), Test section 2201-RAP shows better elastic recovery compared to Test section 2202-RAP. The opposite is the case for the 1-yr service binders where Test section 2202-1yr shows better elastic recovery as compared to Test section 2201-1yr. The softening nature of the base binder + rejuvenator was clearly captured by the very high J_{nr} value, which falls outside the 4.5 kPa^{-1} limit as specified in AASHTO M332. Both Test section 2201-RAP and Test section 2202-RAP are classified as "V" or very heavy, with Test section 2201-RAP exhibiting better rutting resistance than Test section 2202-RAP. Following 1-yr of service, Test section 2202-1yr binder shows better rutting resistance than Test section 2201-1yr, indicating a reverse effect from the rejuvenator on Test section 2201-1yr. Test section 2201-1yr is thus classified as “H” heavy while Test section 2202-1yr is classified as “E” extreme.

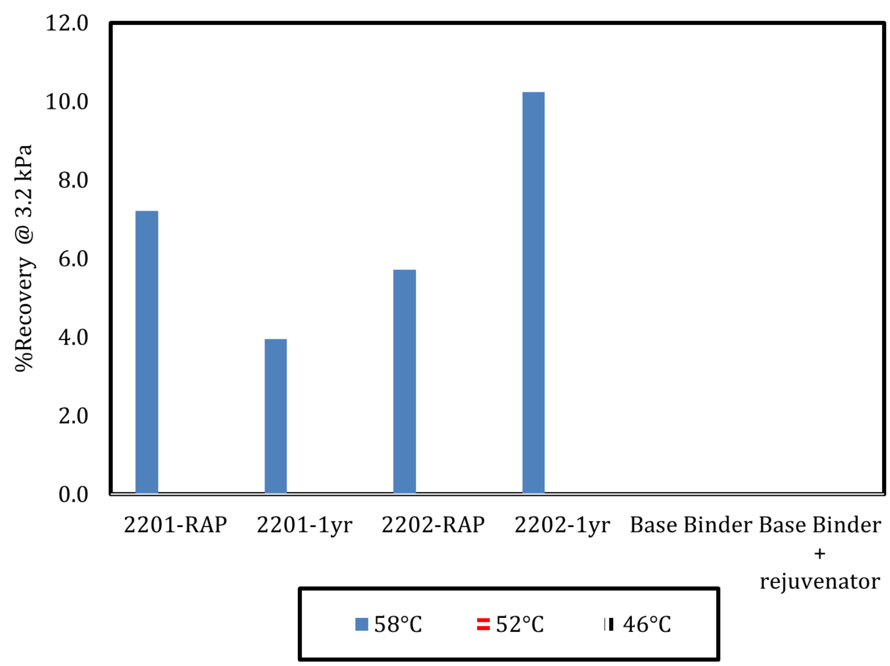


Figure 3-19 Percent recovery at 3.2 kPa for all binders and test temperatures

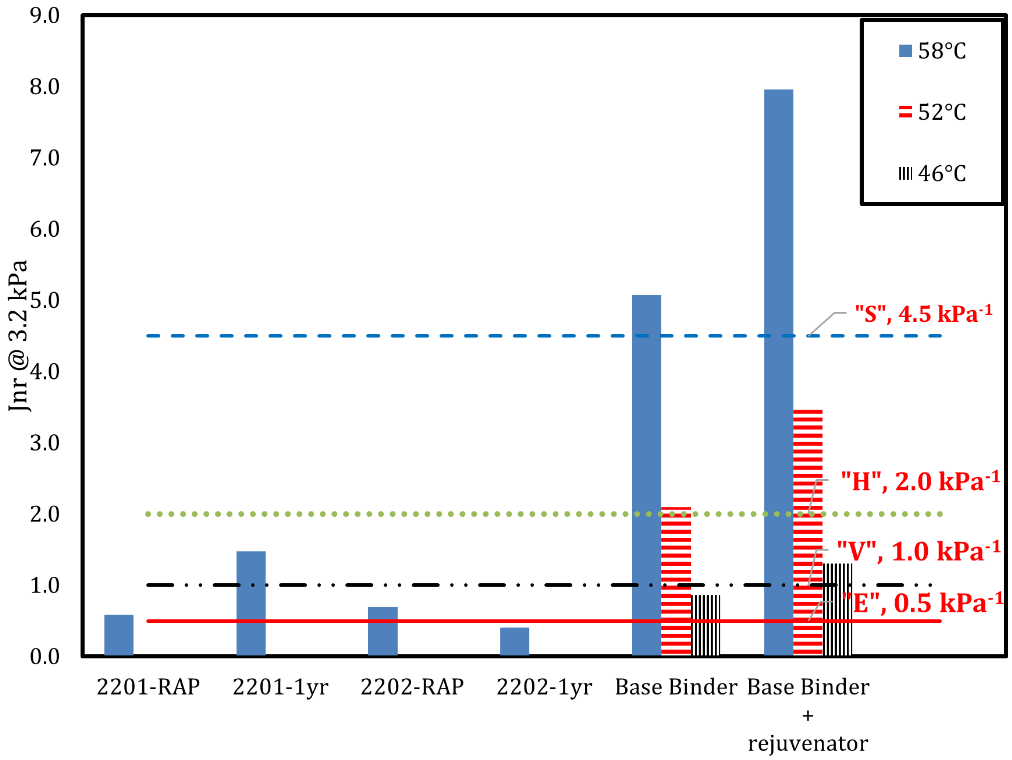


Figure 3-20 Non-recoverable compliance (Jnr) at 3.2kPa

3.4.2.3 Linear Amplitude Sweep (LAS)

The fatigue properties are evaluated using N_f, A, $|B|$ parameter and the developed parameters from Zhang et al., [70]. These results are shown in Table 3-6. Both RAP binders appear to have similar properties. For Test section 2201-1yr and Test section 2202-1yr, Test section 2201-1yr appears to have better fatigue cracking resistance from the N_f value obtained at both 2.5 percent and 5 percent strain relative to Test section 2202-1yr. Based on the LAS parameters (*A* and *B*), it is observed that Test section 2201-1yr has higher value on the *A* parameter which indicates its superior fatigue performance. In terms of the developed parameters by Zhang et al. [70], Test section 2201-1yr as well shows better capability to resist fatigue cracking.

Table 3.6 Binder parameters at test temperature 19C

Binders	N_f-2.5% Strain	N_f-5% Strain	A	$\|B\|$	ϵ_T	E_f (10^5)	I^R
2201-RAP	7,220	457	277,731	3.98	16.94	94.5	0.18
2201-1yr	4,124	442	78,894	3.20	14.16	43.2	0.13
2202-RAP	5,589	562	116,537	3.32	17.94	89.8	0.19
2202-1yr	385	30	11,359	3.70	5.94	28.6	0.05

The stress-strain curves and resulting strain values at max stress are shown in Figure 3-21 and Table 3-7 respectively. After peak stress, Test section 2201-RAP, Test section 2202-RAP and Test section 2201-1yr show a gentle post-peak slope while Test section 2202-1yr shows a fast drop. Test section 2202-1yr show better response at high strain levels but not relatively significant. Test section 2201-1yr on the other hand shows higher strain tolerance.

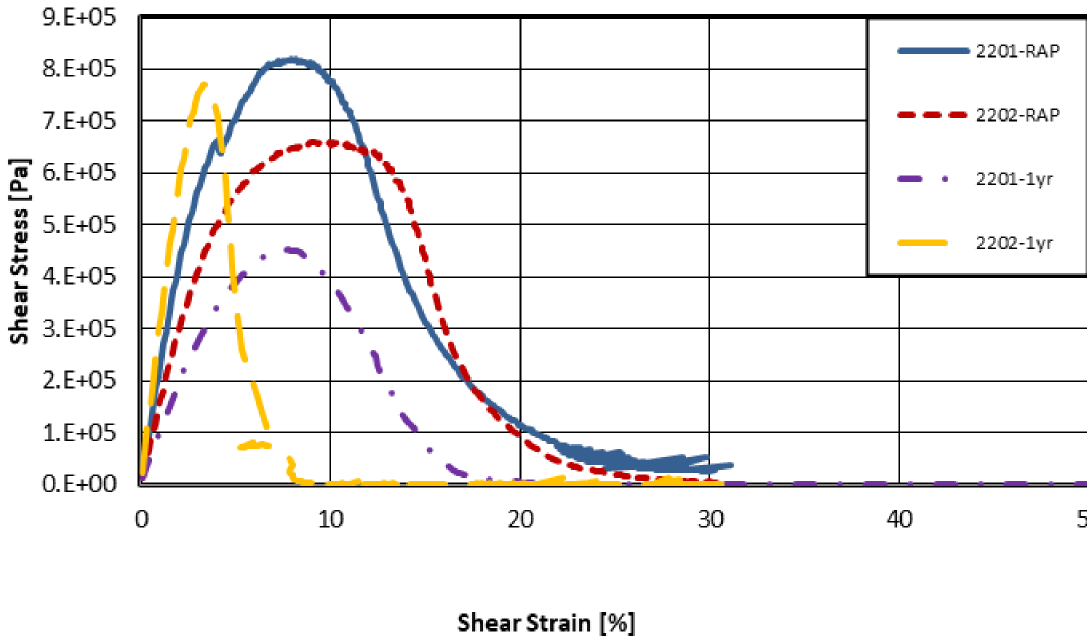


Figure 3-21 Stress-strain plot

Table 3.7 Strain levels at maximum stress

Material	Strain levels @ maximum stress (%)
2201-RAP	7.96
2201-1yr	7.75
2202-RAP	9.19
2202-1yr	3.38

Fatigue performance of the binders using the viscoelastic continuum damage (VECD) are presented in Figure 3-22. Test section 2201-RAP, Test section 2202-RAP, and Test section 2201-1yr demonstrate similar high fatigue life at low strain levels following an intersection change leading to the behavior change. Test section 2202-1yr on the other hand appears to have lower fatigue life at low strain levels.

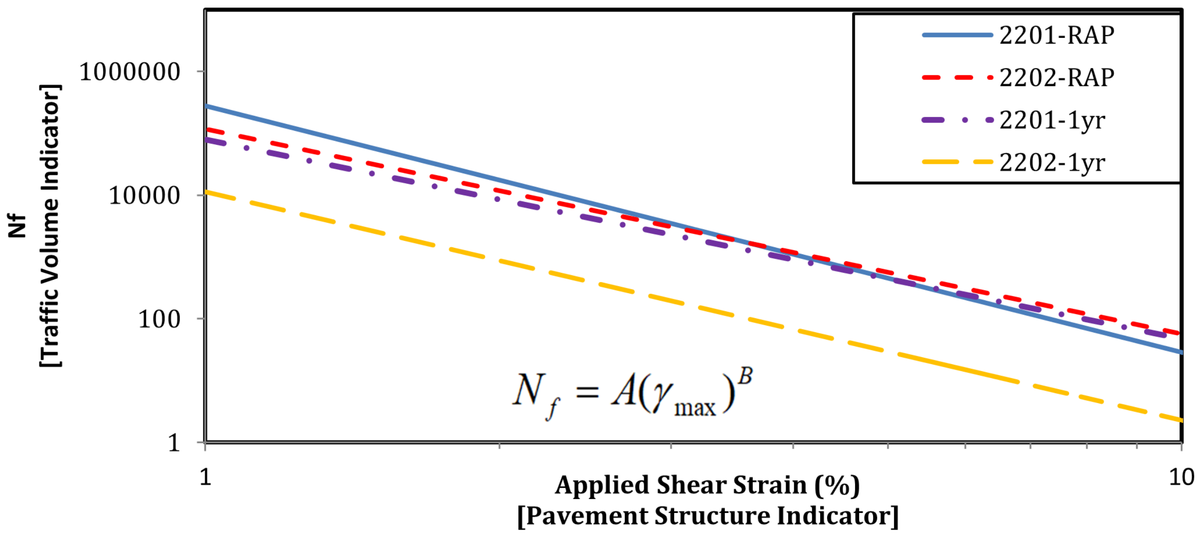


Figure 3-22 Fatigue characterization

Figure 3-23 represents the damage curve and shows integrity (*C*) versus damage intensity (*D*). As a criterion representing damage to the sample at each stage, the *D* parameter can be indicative of a material resistance to fatigue [72]. Test section 2201-RAP, Test section 2202-RAP, and Test section 2201-1yr show comparable damage intensity and consequently experience the same fatigue life. Test section 2202-1yr deviates from this trend.

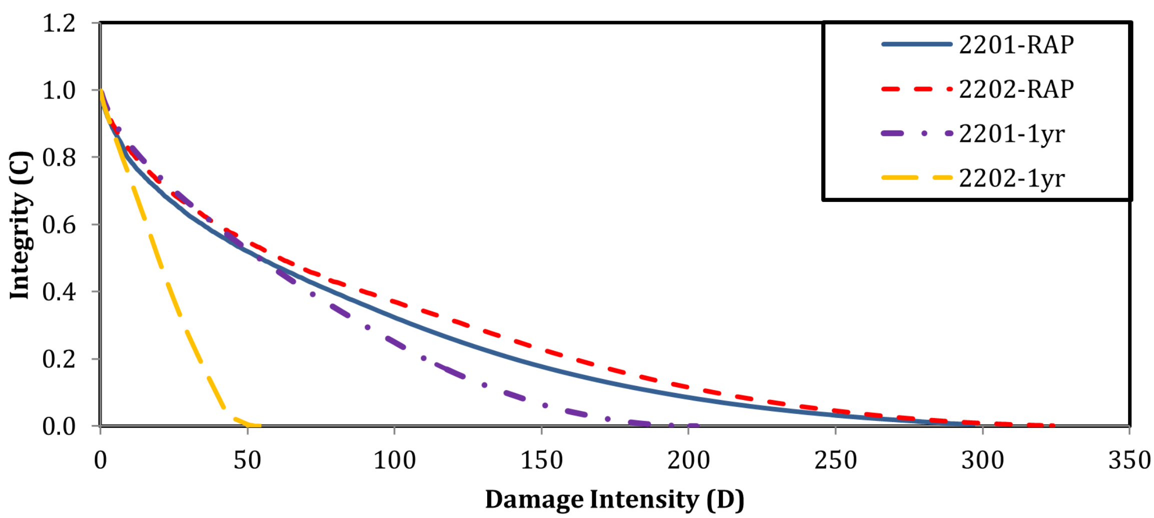


Figure 3-23 VECD damage curve

3.5 Summary

Bituminous cores were extracted from the study after one year of service and several laboratory testing were conducted on them. The mixture testing included Marshall stability (dry and soaked), critical low temperature, and Illinois Flexibility Index Test (I-FIT). Overall, even though the rejuvenation showed some potential for performance improvement in the mix design phase with an increase in the dry Marshall stabilities, the mixture testing results after one year of service either did not show a remarkable change in the performance of the mixture in the presence of rejuvenator or showed some deterioration in cracking resistance.

The evaluation of the RAP binders together with base binder and base binders + rejuvenator involved rheological characterization, Multiple Stress Creep and Recovery (MSCR), and Linear Amplitude Sweep (LAS) tests on a Dynamic Shear Rheometer (DSR). Significant findings regarding materials recovered and extracted during construction (Test section 2201-RAP and Test section 2202-RAP) demonstrate similar stiffness and resistance to cracking and rutting. With regard to in-service binders, rejuvenator-treated Test section 2201 showed superior fatigue resistance performance in comparison to Test section 2202. However, a reduced resistance to rutting is observed for Test section 2201 in comparison to Test section 2202.

Because of the contrasting outcomes between binder and mixture testing results on the effect of rejuvenator, drawing a definite conclusion does not seem to be totally justified. One explanation could be that the rejuvenator improvement is evident when dealing with binder only. In the mixture phase, with only about 2 percent emulsion, this improvement is either diminished or rejuvenation could even result in some deterioration in mixture performance. These differing results suggest that the CIR mixtures containing rejuvenator may need further investigation and/or adjustment. More extensive mixture testing including fatigue, rutting, and low-temperature tests on CIR mixtures containing rejuvenator is recommended.

Chapter 4: Construction Monitoring Results

In this chapter, the construction monitoring activities and testing results including the contractor Quality Control (QC) and Intelligent Construction Technologies (ICT) are compiled and reported. In addition, Falling Weight Deflectometer (FWD) testing results are analyzed, and the results are reported here.

4.1 Contractor Quality Control Testing

The QC testing included gradations (entire and simple), depth check, moisture content, yield checks (emulsion and cement), control strip, and compaction.

Table 4-1 shows the entire gradation results for the four test sections along with their mix design data. As this table shows, the in-place RAP gradation was coarser than what was used in the mix design process for 2202. All other in-place RAP gradations were slightly finer than their companion mix designs.

Two simple gradations were performed per test section for a total of 8 simple gradations. All simple gradations showed that 100 percent of RAP samples pass the 1.25-inch sieve.

Depth checks were performed at two locations per test section for a total of 8 depth checks. As expected, 2201 and 2202 CIR depths were found to be 3 inches, and 2207 and 2208 CIR depths were determined to be 4 inches.

The in-place moisture content was measured at two locations and was between 2.0 and 2.5 percent.

Table 4.1 The in-place RAP gradations

Sieve Size (mm)	Sieve Size (inch)	2201 In-place	2201 Mix Design	2202 In-place	2202 Mix Design	2207 In-place	2207 Mix Design	2208 In-place	2208 Mix Design
37.5	1 1/2	100	100	100	100	100	100	100	100
25.0	1	98	100	82	100	96	100	96	100
19.0	3/4	95	93	59	93	91	93	94	93
9.5	3/8	70	60	41	60	67	60	79	60
4.75	#4	47	45	26	45	42	45	57	45
2.0	#10	29	--	11	--	26	--	37	--
0.6	#30	11	6	--	6	11	8	16	8

Table 4-2 shows emulsion yield checks for the test sections. As this table shows, the emulsion application rate was higher than the mix design application rate for 2207 and 2208 while this application rate for 2201 and 2202 was pretty close to the mix design value.

Table 4.2 Yield checks for emulsion

Test Section	Field Application Rate (gal/SQ YD)	Mix Design Application Rate (gal/SQ YD)
2201	0.95	0.96 (2.2%)
2202	0.95	0.96 (2.2%)
2207	0.95	0.76 (1.7%)
2208	0.95	0.76 (1.7%)

Table 4-3 shows cement yield checks for the test sections. As this table shows, the cement application rate was higher than the mix design application rate for 2201 and 2202, while this application rate for 2207 and 2208 was the same as the mix design value.

Table 4.3 Yield checks for cement

Test Section	Field Application Rate (%)	Mix Design Application Rate (%)
2201	0.6	0.5
2202	0.6	0.5
2207	0.5	0.5
2208	0.5	0.5

Two control strips were created: one for Test sections 2201 and 2202, and one for Test sections 2207 and 2208. For Test sections 2201 and 2202, a maximum wet density of 132.7 pcf was achieved with 3 passes of rubber tire roller and 4 passes of smooth drum roller (i.e., total of 7 passes). For Test sections 2207 and 2208, a maximum wet density of 136.9 pcf was achieved with 4 passes of rubber tire roller and 4 passes of smooth drum roller (i.e., total of 8 passes).

Four nuclear compaction tests were performed per test section. Table 4-4 summarizes the compaction test results. As this table shows, all 16 points have exceeded the minimum required compaction of 98 percent. The average in-place relative compaction for all test sections was about 102 percent.

Table 4.4 Compaction test results

Test Section	Nuclear Density (pcf)	Relative Compaction (%)	Pass/Fail
2201(a)	133.9	101	Pass
	133.5	101	Pass
	133.2	100	Pass
	133.4	101	Pass
2202(b)	139.1	105	Pass
	143.2	108	Pass
	137.1	103	Pass
	136.4	103	Pass
2207(c)	139.0	102	Pass
	138.2	101	Pass
	139.2	102	Pass
	140.8	103	Pass
2208 (d)	136.0	99	Pass
	139.0	102	Pass
	133.6	98	Pass
	137.6	101	Pass

(a) Compaction target from control strip – 132.7 pcf
(b) Compaction target from control strip – 132.7 pcf
(c) Compaction target from control strip – 136.9 pcf
(d) Compaction target from control strip – 136.9 pcf

It should be noted that in addition to the contractor QC testing, MnROAD also conducted Light Weight Deflectometer (LWD) tests on the CIR layers at approximately 2, 24, 48 and 60 hours after the construction which showed an increase in the modulus of the CIR layer overtime for all the test sections. The International Roughness Index (IRI) was also measured on the final bituminous layers. The detailed LWD and IRI results can be found in 2022 MnROAD Construction Activities report [67].

4.2 Intelligent Construction Technologies (ICT) Testing

The Intelligent Compaction Technologies (ICT) was performed during compaction of the CIR layers. The ICT includes compaction using an instrumented roller; a self-propelled roller integrated with a global navigation satellite system and onboard documentation system that can display real-time color-coded maps of roller location, number of passes, roller speeds, and amplitude and vibration frequencies of the roller drum.

Table 4-5 shows the number of roller passes per test section. As this table shows, the test sections have received similar compaction in terms of number of roller passes; 89 to 93 percent of the test sections area has received more than 5 roller passes with the average of 92 percent. Three (3) percentage of the project area has received 4 roller passes and 3 percentage of the project area has received 3 roller passes. Only 1 percent of the project area has received 2 or a smaller number of roller passes. Figure 4-1 also shows pie charts of number of roller passes for each test section.

Table 4.5 Number of roller passes

Test Section	Offset	5+ passes (%)	4 passes (%)	3 passes (%)	2 passes (%)	1 pass (%)	No passes (%)
2201	CL-12R	93	1	0	6	0	0
2201	12L-CL	93	4	2	0	0	0
2201	Average	93	2	1	3	0	0
2202	CL-12R	93	4	1	1	1	0
2202	12L-CL	90	2	4	4	0	1
2202	Average	92	3	2	2	1	0
2207	CL-12R	92	1	7	0	0	0
2207	12L-CL	93	5	1	0	0	0
2207	Average	92	3	4	0	0	0
2208	CL-12R	95	3	2	0	0	0
2208	12L-CL	89	2	6	1	1	1
2208	average	92	2	4	1	0	0
	Average	**92**	**3**	**3**	**1**	**0**	**0**

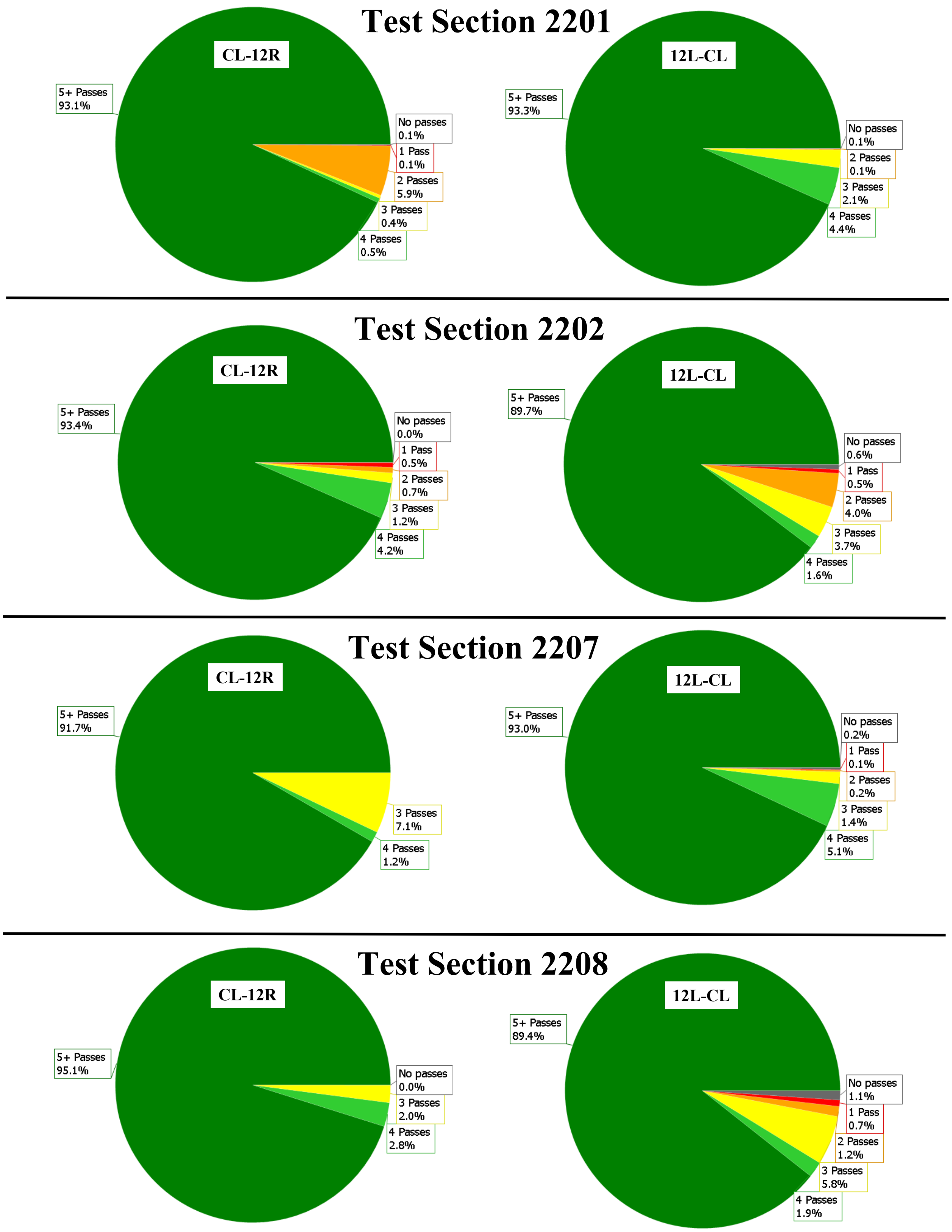


Figure 4-1 Number of roller passes per test section

4.3 Falling Weight Deflectometer (FWD) Testing

This section presents the back-calculated stiffness results for the CIR layer from the falling weight deflectometer tests conducted at various periods on the test sections. A Dynatest 8000 FWD setup was used to measure the deflection of the pavement surface on the inner wheel path, mid-lane, and outer wheel path in the direction of travel on both driving and passing lanes. Only measurements on the mid-lane and outer-wheel paths are presented here. FWD tests were performed at 5 intervals, 15.24 m along the length of Test sections 2201, 2207 and 2208 and 4 intervals of roughly 13.72 m along the length of Test section 2202. Three drops (corresponding to approximately 27 kN, 40 kN, and 55 kN) were performed at each testing campaign using an FWD loading plate of 300 mm. Data from the 40 kN FWD drop, which represents the standard single-axle wheel load, was used for the analysis.

Results of this analysis are from the test campaigns conducted 2 months (October 2022), 3 months (November 2022), 7 months (March 2023), and 10 months (June 2023) after construction for Test sections 2201 and 2202, and 3 months (November 2022), 7 months (March 2023), 10 months (June 2023), and 13 months (August 2023) for Test sections 2207 and 2208. The pavement cross section of all test sections is as shown in Figure 3-1.

4.3.1 Deflection Data Analysis

4.3.1.1 Deflection data processing (Outlier detection)

The initial stage of data processing involved verifying the appropriateness of individual basin deflections. Statistical comparisons were performed for each test section and test date to identify potential outliers in the basin deflections. This process utilized the peak deflection (D0), employing Tukey's fences methodology to identify and eliminate inappropriate basins. As a result, seven inappropriate basins were identified and removed from both the driving and passing lanes across all test campaigns.

4.3.1.2 Back-calculation inputs

Layered elastic principles combined with finite element simulations were employed using the Dynatest ELMOD6 program. To accurately assess the stiffness of pavement layers, the back-calculation analysis incorporated the following assumptions and inputs:

- Combination of the UTBWC and the bituminous layer to make a representative bituminous layer. This is necessary to ensure that there is higher accuracy of back-calculation process, and results are not influenced due to presence of too many layers and relatively thin layers.
- Seed moduli of the pavement layers. Table 4-6 outlines input values employed on the ELMOD6 program for back-calculation.

Each test section was modeled with five layers in the ELMOD6 package. Post-back-calculation, the root mean square (RMS) values were assessed to validate the reasonableness of the back-calculated stiffness. These RMS values averaged between 0 and 10 microns across all sections and testing phases, which were deemed acceptable. Given that testing was conducted at intervals along the length of each section, the average back-calculated modulus was used as the representative value for each test section.

Table 4.6 Input parameters on ELMOD for back-calculation

Layers/Materials	Seed/Initial Modulus (source)
3" Bituminous	4,000 MPa (LTPP)
3" - 4" CIR	2,000 MPa (Wirtgen Manual)
1" Existing Bituminous	6,000 MPa (approx. value from back-calculated moduli before CIR)
4" FDR + EE & FDR + Fly ash	500 MPa
33" Granular Base	ELMOD6 defined
Subgrade (Clay)	ELMOD6 defined

4.3.1.3 Moduli temperature correction

The effective temperature of the CIR layer and other temperature-sensitive layers (Bituminous, Existing Bituminous, FDR + EE) was calculated using the BELLS3 temperature model at one-third of the layer's thickness to the pavement surface, as described in Equation 4-1. This approach was deemed most appropriate as it complied with LTTP standards for FWD unit sensor spacing and drop sequence, and the testing length closely mirrored routine testing techniques. The moduli of each temperature-sensitive layer were subsequently adjusted to a reference temperature of 21°C using the asphalt temperature adjustment factor (ATAF) proposed by Lukanen et al. [73], as illustrated in Equation 4-2.

$$T_{bells3} = 0.95 + 0.892 * T_{surface} + (\log_{10}\left(\frac{H_{ac}}{3}\right) - 1.25) * (-0.448 * T_{surface} + 0.621 * T_{prev.day} + 1.83 * \sin(hr_{18} - 15.5)) + 0.042 * T_{surface} * \sin(hr_{18} - 13.5)$$

Equation 4-1

Where,

$T_{surface}$ = pavement surface temperature (°C)
$T_{prev.day}$ = average air temperature the day before testing (°C)
$\sin(hr_{18} - 15.5)$ = 18-hr sine function, 15.5 variation
$\sin(hr_{18} - 13.5)$ = 18-hr sine function, 13.5 variation

$$ATAF = 10^{slope(T_r - T_m)}$$

Equation 4-2

Where,
Slope = Log (Mr) = intercept + slope T (-0.021 used in this study following LTPP recommendations)
T_r = reference temperature, °C
T_m = measured temperature, °C

4.3.1.4 Deflection basin parameters

To further validate the back-calculated modulus of the CIR layer and assess the overall structural performance of each section, the peak deflection (D0) and surface curvature index (SCI) which indicate the condition of the base layer [74] were evaluated. The temperature normalized D0, developed by Kim and Park [75], and the SCI are presented in Equation 4-3.

$$D_{0adj/12adj} = D_{0/12} * 10^{(-C_0+Ar)*(H_{ac})(T-T_0)}$$

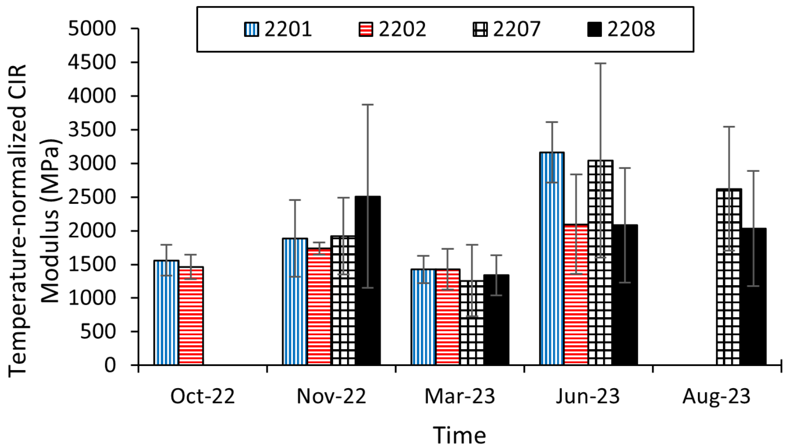
$$SCI = D_{0adj} - D_{12adj}$$

Equation 4-3

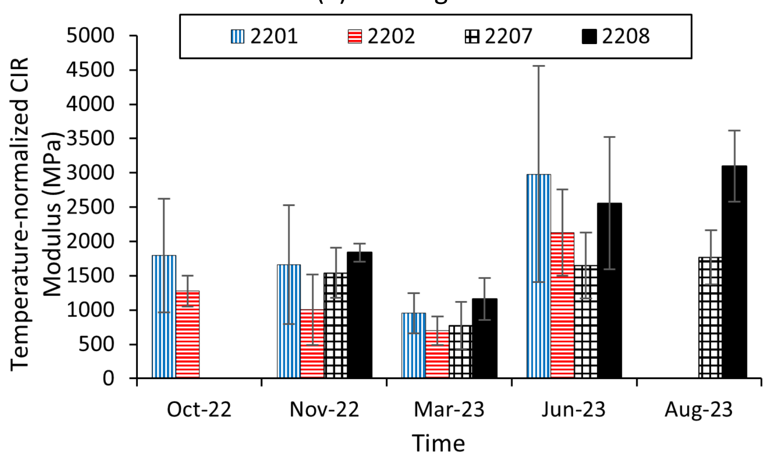
Where.
D_0 = measured deflection under the center of the load
D_{12} = measured deflection at 12-inch radial distance
C_0 = -5.47E-08
A= 4.65E-05
r = radial distance from the center of load
H_{ac} = depth of asphalt layer (in.)
T = measured effective pavement temperature
T_0 = reference temperature (21°C)
D_{0adj} = adjusted peak deflection
D_{12adj} = adjusted deflection at 12-inch (300 mm) radial distance

4.3.2 Back-calculation Results

Figure 4-2(a-b) and Figure 4-3(a-b) illustrate the back-calculated stiffness of the CIR layer for all test sections across various test campaigns, specifically for the outer wheel path and mid-lane, respectively. Error bars denote one standard deviation interval. Notably, test section 2201 exhibits higher average CIR stiffness values than test section 2202 across both driving and passing lanes for both the outer wheel path and mid-lane sections. Conversely, the analysis of test sections 2207 and 2208 across all testing periods reveals a general decrease in CIR layer stiffness for test section 2207, indicating a deviation from the mix design properties.

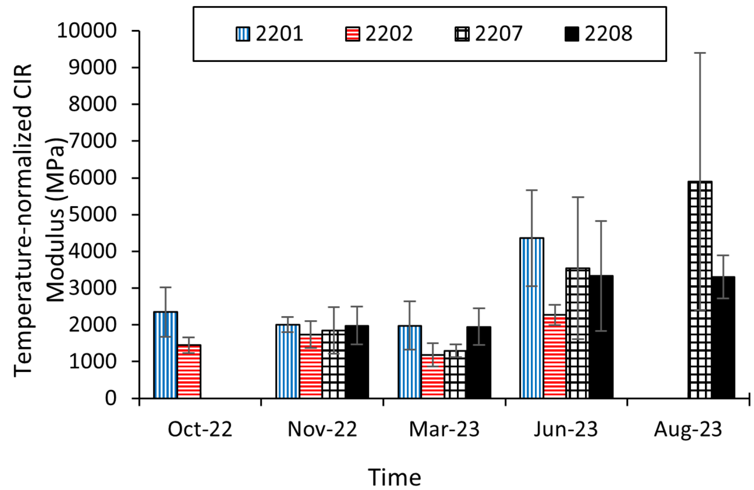


(a) Driving Lane

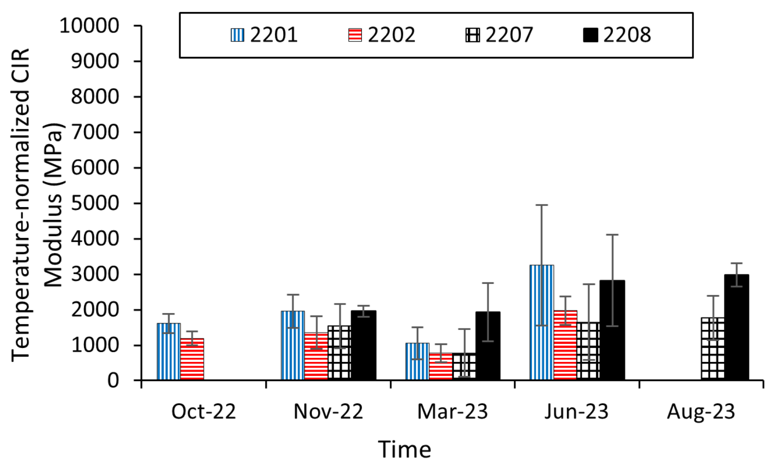


(b) Passing Lane

Figure 4-2: CIR back-calculated stiffness on the outer wheel path for: (a) driving lane and (b) passing lane



(a) Driving Lane



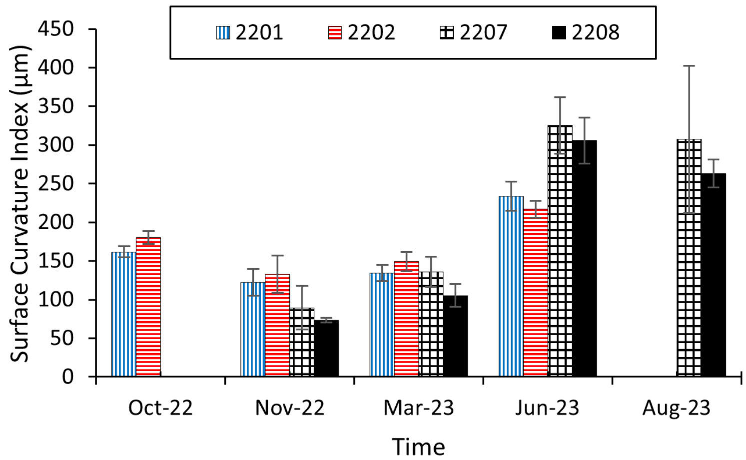
(b) Passing Lane

Figure 4-3: CIR back-calculated stiffness on the mid-lane for: (a) driving lane and (b) passing lane

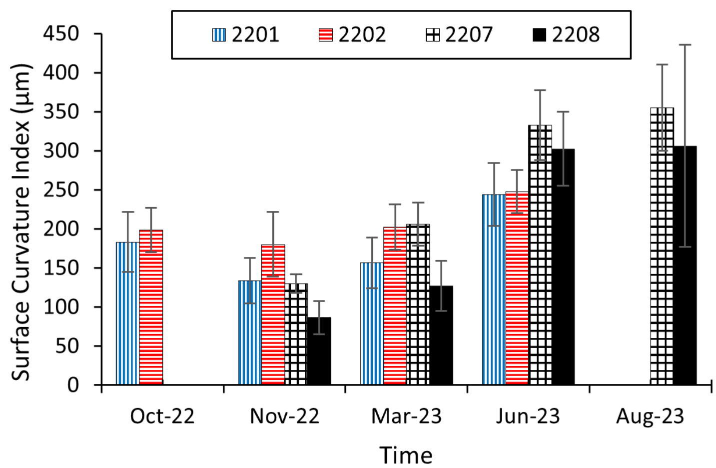
4.3.3 Deflection Basin Parameters

4.3.3.1 Surface curvature index (SCI)

Figure 4-4(a-b) and Figure 4-5(a-b) depict the SCI determined from the deflection basin for all sections, with error bars representing one standard deviation interval. Given that lower SCI values are preferable [74], test section 2201 demonstrates a better overall condition on average compared to test section 2202. Regarding Test sections 2207 and 2208, test section 2208 shows better condition on average across all testing periods for both the outer wheel path and mid-lane on the driving and passing lanes. These results further validate the findings from the back-calculated CIR moduli for all sections.

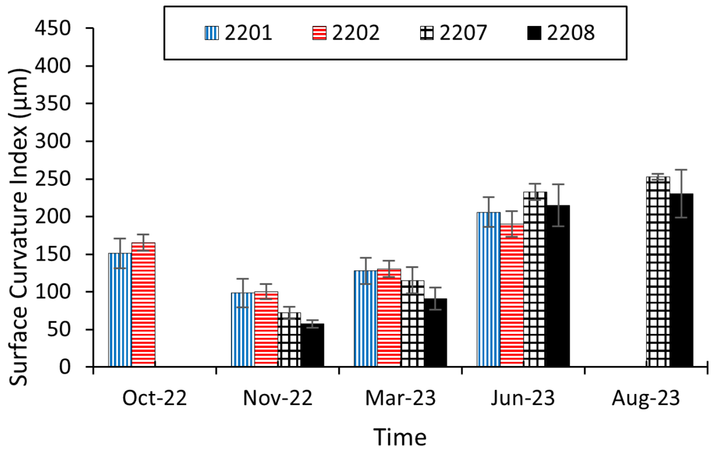


(a) Driving Lane

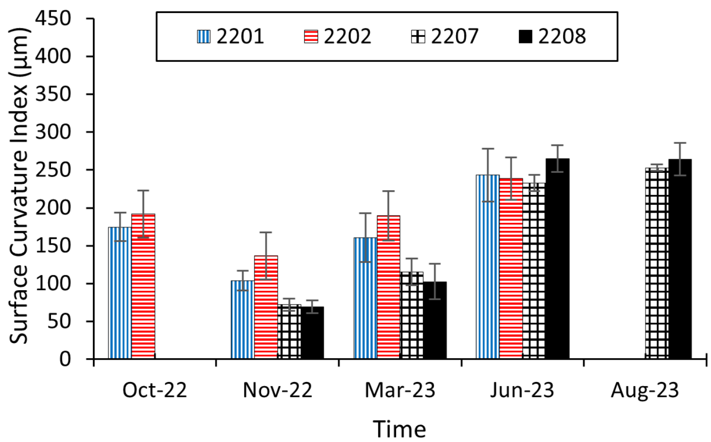


(b) Passing Lane

Figure 4-4: Outer wheel path SCI for: (a) driving lane and (b) passing lane



(a) Driving Lane

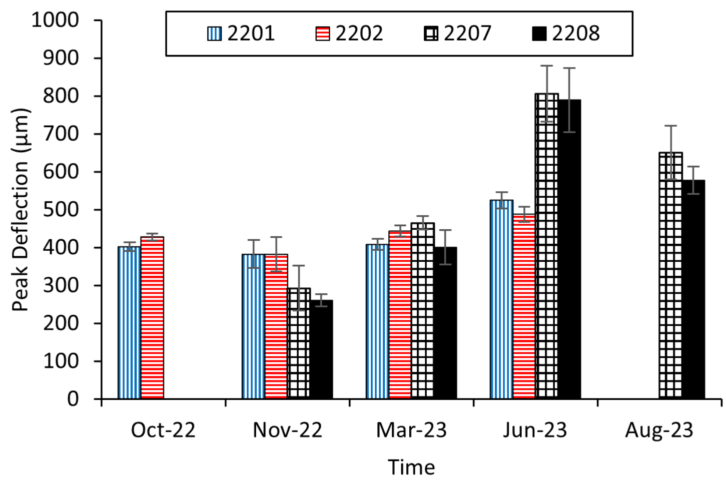


(b) Passing Lane

Figure 4-5: Mid-lane SCI for: (a) driving lane and (b) passing lane

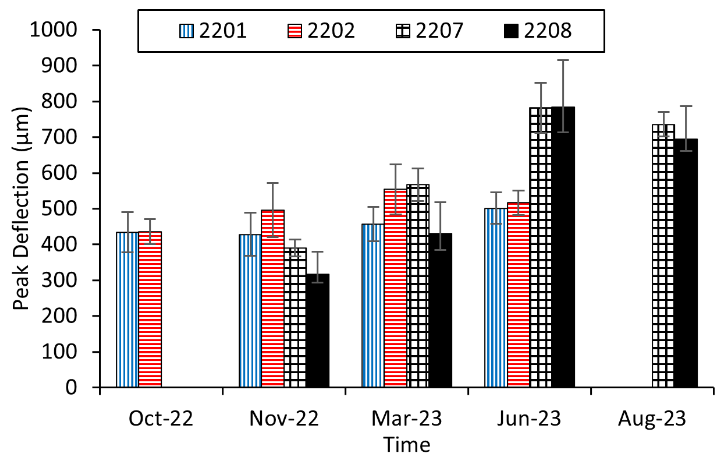
4.4 Overall Structural Condition (Peak Deflection)

The analysis presented in Figure 4-6(a-b) and Figure 4-7(a-b) highlights the overall structural performance of the sections in terms of peak deflection for both driving and passing lanes. Error bars represent one standard deviation interval. test section 2201 exhibits lower average peak deflection values compared to Test section 2202, indicating better structural performance. Conversely, Test section 2208 shows lower average peak deflection values than Test section 2207 across all testing periods for both the outer wheel path and mid-lane on the driving and passing lanes.



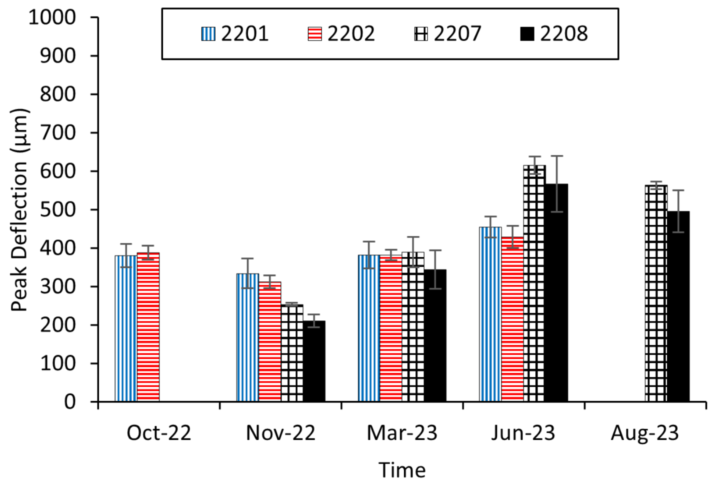
(a) Passing Lane

(b)

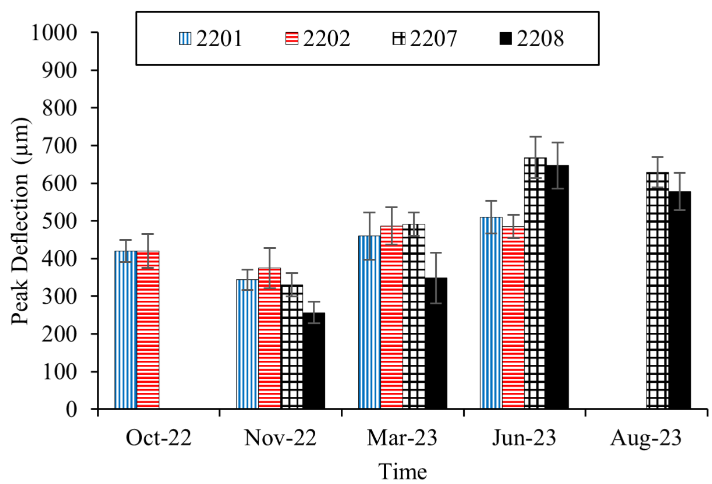


(c) Passing Lane

Figure 4-6: Outer wheelpath peak deflection for: (a) driving Lane (b) passing lane



(a) Driving Lane



(b) Passing Lane

Figure 4-7: Mid-lane peak deflection for: (a) driving lane and (b) passing lane

Chapter 5: Perpetual Pavement Mechanistic Analysis

This chapter outlines the results of the perpetual pavement analysis conducted on the MnROAD test sections. This was conducted to determine potential perpetual behavior of the test sections with respect to bottom-up fatigue cracking and subgrade rutting. Categorized in sections, details on the existing research for perpetual behavior analysis for cold recycled sections as well as relevant inputs employed for analysis are presented.

5.1 Introduction

Perpetual or long-lasting pavements require resilient base layers capable of withstanding heavy traffic load repetitions and large temperature variations. Also, these pavement structures are expected to have enough structural strength and thickness to prevent fatigue cracking and permanent deformation [76-77]. In addition, examining the fatigue characteristics of a pavement section exposed to significant truck traffic volumes is a customary procedure for conventionally designed asphalt pavements [78]. This evaluation of fatigue performance is also relevant for cold recycled layers, given their nature as semi-bonded materials.

Not many studies exist on the use of recycling and reclamation techniques for achieving perpetual pavements. In a study by Timm et al. [78], perpetual pavement analysis of three cement-stabilized cold CCPR sections was performed to determine if they meet perpetual design criteria with respect to bottom-up fatigue cracking. The test sections include a Stone Matrix Asphalt (SMA) surface and Superpave dense-graded asphalt concrete above the CCPR layer and over a granular aggregate base for two sections (called N3 and N4) and a cement stabilized base (SFDR) for the third section (called S12). Asphalt strain gauges were installed at the bottom of the CCPR layer to measure tensile strain. Earth pressure cells were also installed at the bottom of the CCPR layer and the top of the subgrade to measure vertical stress distribution. The CCPR materials passed the 70 percent retained Indirect Tensile Strength (ITS) and dry ITS of 310 kN/m^2 requirements. Approximately 10^7 Equivalent Single Axle Loads (ESALs) were applied during the first two years of the study (2012–2014), and an additional 10^7 ESALs were applied during the second test cycle (2015–2017). The PerRoad software was used to simulate and compare the predicted tensile strains to field measurements. It was concluded that the SFDR section (S12) is expected to be perpetual, while the other two sections are expected to experience bottom-up fatigue cracking. PerRoad simulation showed that the N3 section, which has a thicker Superpave Asphalt Concrete (AC) layer compared with the N4 section, could be designed to behave as perpetual by increasing the thickness of AC/CCPR layers [76].

5.2 Analysis Approach

To determine whether these pavement sections meet the criteria of perpetual pavements, perpetual characteristics analysis was conducted by employing strain distributions in accordance with developed criteria from the NCAT test track [79-80]. The PerRoad software which employs Monte Carlo simulation was used to construct simulated strain distributions in relation to bottom-up fatigue cracking and compressive strain metrics in terms of structural rutting tied to the subgrade. Input data for analysis include layer moduli- as determined through the in-situ FWD tests highlighted in Chapter 4 of this report, as well as layer thicknesses for all asphalt, cold recycled and existing pavement layers in each section, seasonal information (which accounts for the modulus variation across seasons), and traffic.

Prior to conducting the analysis, a check was made to ascertain whether the stiffness metrics of the existing underlying layers obtained from the FWD evaluation were in agreement with the range of stiffness results obtained from the Accelerated Plate Load Test (APLT) and COMP-Score® RT mapping tests conducted prior to the construction of the CIR layers and bituminous overlay. This was achieved through the following approach.

1. For test sections 2201 and 2202, the Class 4 granular layer and subgrade were combined to determine the composite resilient modulus (MR_{comp}) through Boussinesq's elastic solution for linear-elastic materials (Equation 5) for comparison with the APLT and COMP-Score® RT mapping obtained metrics. In addition, a comparison was made between the FWD determined modulus of the existing bituminous and results from both APLT and COMP-Score® RT tests.
2. For test sections 2207 and 2208, the FDR layers were combined to determine the MR_{comp} for comparison with the APLT and COMP-Score® RT mapping metrics. This was also concurrent with a comparison between the FWD determined modulus of the subgrade and results from both APLT and COMP-Score® RT tests.

$$\boldsymbol{MR_{comp}} = (\mathbf{1} - \boldsymbol{v}^{\mathbf{2}}) \times \boldsymbol{f} \times \boldsymbol{r} \times \frac{\boldsymbol{\sigma_0}}{\boldsymbol{\delta_0}} \qquad \text{Equation 5-1}$$

Where

v = 0.4
f = 2.0 Uniform stress distribution
r = 6-inch (based on 12-inch diameter plate)
σ_0 = Vertical Stress (30 psi)
δ_0 = Vertical displacement on the surface derived from Linear Elastic Analysis (inches)

The stiffness measurements for both the bituminous and FDR+EE layers were temperature-normalized to the average summertime temperature of 30°C before conducting the comparison. For further details on the APLT and COMP-Score® RT mapping tests, readers are directed to the MnROAD construction report [67]. The findings of this analysis are summarized in Table 5-1.

5.2.1 APLT, COMP-Score® RT mapping and FWD comparison results

Table 5.1 2201 & 2202 – Bituminous and Base + Subgrade Layers

Layer Type	APLT/COMP-Score® RT Mapping (psi)	FWD Determined Stiffness (psi)
Bituminous	40,105 – 250,000	120,115 – 200,924
Base + Subgrade	17,885 – 19,432	19,760 – 21,549

Table 5.2 2207 & 2208 – FDR and Subgrade Layers

Layer Type	APLT/COMP-Score® RT Mapping (psi)	FWD Determined Stiffness (psi)
FDR Layers	24,907 – 250,000	11,264 – 22,372
Subgrade	9,678 – 27,120	13,222 – 16,750

The analysis indicates that, for most pavement layers, the FWD stiffness metrics all fall within the range of stiffness values observed in tests conducted on the existing layers. Consequently, the FWD stiffness metrics were utilized in conducting the perpetual pavement analysis.

Tensile strain distribution was generated for all cracking prone layers, in terms of the bituminous, CIR and FDR layers in the pavement sections. It is particularly important to note that these criteria were not developed for CIR or FDR pavements, which renders their true applicability uncertain at this time. Nonetheless, they function as a spot check against conventional flexible pavements. Monitoring these sections will assist in confirming whether they can be implemented as-is or whether new criteria are required. PerRoad threshold tensile strain values in terms of strain distribution for perpetual behavior are depicted in Figure 5-1. Criteria for structural rutting was defined at 200 $\mu\varepsilon$ at a pass cumulative percentile of 50 percent as recommended in some references [45, 53].

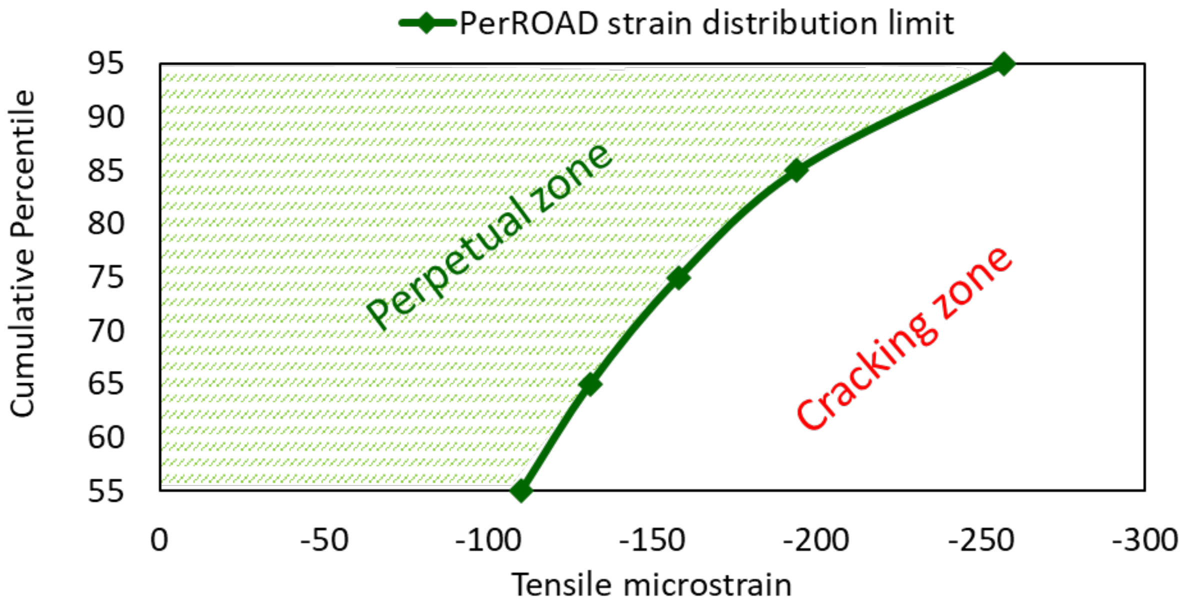


Figure 5-1: PerRoad tensile strain distribution for bottom-up fatigue cracking [80]

5.3 Perpetual Analysis Results

Results from both driving and passing lanes of all sections in terms of bottom-up fatigue cracking cumulative strain distributions are depicted in Figure 5-2. Responses above the PerRoad limit indicate the "cracking zone" while below this limit yields perpetual behavior (i.e., perpetual zone). Analysis on both driving and passing lane show that these sections are expected to experience bottom-up cracking at some point, particularly on already in-place layers prior to CR application for sections 2201 and 2202 and on CIR and in-place layers for section 2207 and 2208. Compressive strain in terms of structural rutting for all sections across both driving and passing lanes are observed to pass the 200 $\mu\varepsilon$ threshold criteria (100 $\mu\varepsilon$ at 100 percent cumulative percentile for all sections), indicating structural efficiency of the upper layers.

Additional perpetual analyses were performed to reassess these sections in terms of bottom-up cracking and quantify the adjustments in thickness required to match them more closely with or below the perpetual design limit. Through trial and error, the bituminous and CIR thicknesses were adjusted across all sections and the perpetual design thickness was determined. This is presented in Table 5-2.

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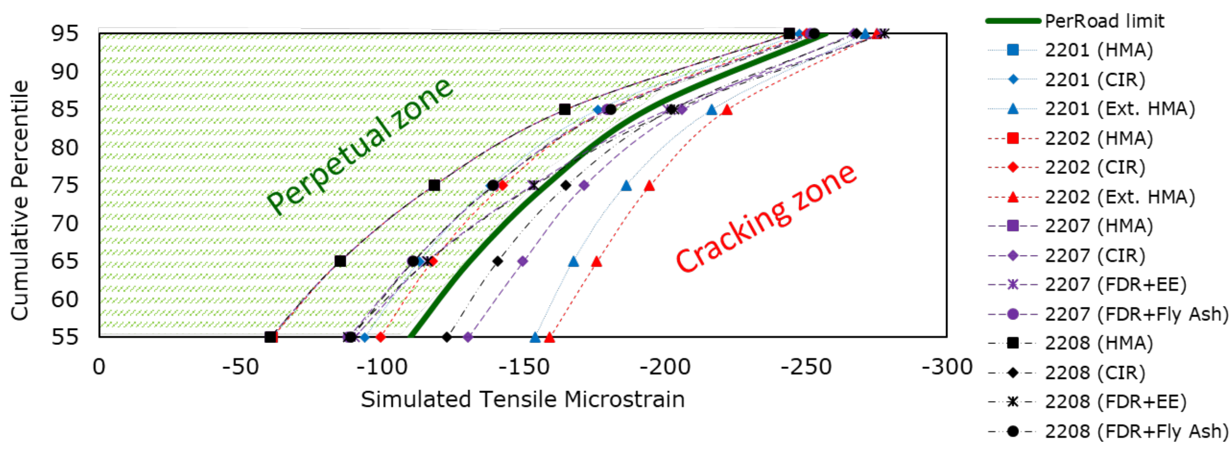
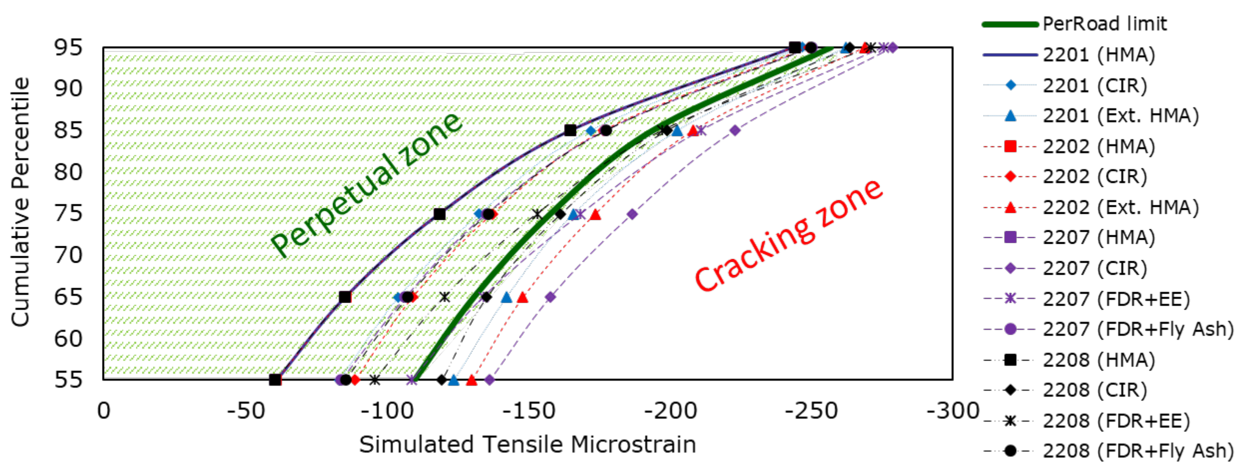


(a) Driving Lane



(b) Passing Lane

Figure 5-2: Bottom-up fatigue cracking cumulative strain distribution: (a) driving lane, and (b) passing lane

Table 5.3: Perpetual design thickness

Test Section	Lane	Optimized Design
2201	Driving	4” HMA + 4” CIR
	Passing	4” CIR
2202	Driving	4” HMA + 4” CIR
	Passing	4” CIR
2207	Driving	5.5” HMA
	Passing	5” HMA
2208	Driving	5” HMA
	Passing	5” HMA

Chapter 6: AASHTO Empirical Design Layer Coefficient Development for Reclaimed/Recycled Layers

This chapter outlines the methodology employed in the development of layer coefficients for reclaimed/recycled pavement layers to be utilized in the AASHTO empirical design procedure. Versions of the AASHTO empirical design approach since its first development based on the American Association of State Highway Officials (AASHO) road test performed in the late 1950s and early 1960s in Ottawa, Illinois include the AASHTO 1961, AASHTO 1972, AASHTO 1986 and AASHTO 1993 design guides. The later versions, which are both AASHTO 1986 and AASHTO 1993 design guides, are improvements of the earlier versions. Considering that the AASHTO 1993 design is the latest version and the most commonly utilized empirical approach till date, this guide was utilized in the layer coefficient development for this study. In addition, the layer coefficient development was specifically conducted for Cold In-place Recycled (CIR) layers. Procedures for the development and the derived coefficients are discussed in the following sub-sections.

6.1 Introduction

Major steps have been taken to switch from the empirical guide to a mechanistic-empirical pavement design guide (MEPDG) in the past three decades, however the high costs and lack of available database for regionally calibrating this design guide has become an issue for many of the state DOTs, not to additionally mention the limited efforts carried out and/or available database for reclaimed/recycled layers. The AASHTO 1993 design is however still being used as a reliable pavement structure design tool in many states in the United States and other countries around the world. Several researchers have explored investigations to the development of layer coefficient for reclaimed layers, particularly based on laboratory and field data [81-83]. The design equation is expressed in Equation 6-1, with design parameters discussed in subsequent sections.

$$log_{10}(W_{18}) = Z_R \times S_0 + 9.36 \times log_{10}(SN+1) - 0.20 + \frac{log_{10}\left(\frac{\Delta PSI}{4.2-1.5}\right)}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times log_{10}(M_R) - 8.07$$

Equation 6-1

In which,

$log_{10}(W_{18})$ = Number of the allowable equivalent single axle loads (18 kips) in design period
Z_R = z-statistic, determined based on level of reliability of the design
S_0 = Standard deviation of the design, based on level of accuracy of data collection
SN = Structural Number, indicating the overall load bearing capacity of subgrade soil
ΔPSI = Loss in serviceability
M_R = Resilient modulus of subgrade soil

6.1.1 Equivalent Single Axle Load (ESAL)- $log_{10}(W_{18})$

The design procedure for determining the thickness of the pavement layers is based on the total number of applied wheel loads over the pavement's design life. Given that axle loads and configurations vary among different vehicle types, the resulting damage they cause also differs. The cumulative equivalent damage over the pavement's design life is referred to as the Equivalent Single Axle Load (ESAL), which serves as the sole traffic factor in the pavement design process.

6.1.2 Present Serviceability Index (PSI)

The pavement serviceability is primarily evaluated by the ride quality experienced by road users. PSI is determined as the average of independent ratings by individuals assessing the pavement's ride quality, with scores ranging from 5 (best) to 0 (worst). This index is a crucial tool for Pavement Management Systems (PMS) to determine the appropriate timing for maintenance, rehabilitation, or reconstruction of the pavement. The initial serviceability of the pavement depends on its type and construction quality, with a typical value of 4.2 for flexible pavements. The terminal serviceability value for flexible pavements is usually set at 2.5.

6.1.3 Reliability

Reliability in the design process refers to the probability that the pavement will perform satisfactorily throughout its design life. This parameter varies based on the type and importance of the road. As a safety factor, reliability is incorporated into the AASHTO 1993 design guide to ensure that the actual ESALs over the design life do not exceed the estimated ESALs.

6.1.4 Structural Number (SN)

The structural number (SN) serves as a metric to evaluate its capacity to withstand applied loads. The SN is dependent on the type, quality, thickness, and drainage properties of the materials used in the pavement layers. Materials of lower strength necessitate a higher SN under identical loading and environmental conditions. The fundamental aim of pavement design is to protect the subgrade soil from undue stresses and prevent the infiltration of surface water, which can substantially reduce the soil's modulus and hasten pavement deterioration. The SN for a pavement with "n" layers is given by Equation 6-2, where the layer coefficients, or 'a-values', reflect the quality of the materials. Additionally, the drainage coefficient, or 'm-value', is determined by the effectiveness of drainage, varying from 0.4 to 1.4 based on material and moisture conditions.

$$SN = \sum_{i=1}^{n} a_i \times D_i \times m_i$$

Equation 6-2

Where,

SN = Structural number

a_i = Layer coefficient of the ith layer
D_i = Thickness of ith layer
m_i = Drainage coefficient of the ith layer

6.1.4.1 Layer coefficient (a-value)

The layer coefficient is described as an empirical indicator of the relationship between the structural number (SN) of a pavement structure and its layer thickness, demonstrating the relative capacity of a material to serve as a structural element of the pavement. For a specific pavement structure, the SN is first calculated using Equation 6-1. Subsequently, this SN value is correlated to the thicknesses of various layers through Equation 6-2. Through this process, the a-values, which are essentially regression coefficients, are established. These coefficients reflect the capacity of each layer to contribute to the overall pavement structure. Notably, the layer coefficient for asphalt materials depends not only on the properties and thickness of the asphalt itself but is also influenced by the properties of the underlying materials. This interaction highlights the complexity of designing pavement layers that are both effective and durable under varying operational and environmental conditions.

6.1.5 Subgrade resilient modulus

In the AASHTO 1993 design guide, the resilient modulus of subgrade is a critical factor influencing pavement design. The resilient modulus is particularly sensitive to temperature and moisture variations, which are more pronounced in cohesive soils and granular materials with a higher fraction of fines. Factors such as changes in the groundwater table level, precipitation, and seasonal temperature shifts, especially during freeze-thaw cycles, can significantly alter the resilient modulus. To account for these variations, the "effective resilient modulus" is calculated. This modulus represents an average value that reflects the potential damage to the pavement across different seasons, considering the fluctuating soil modulus values throughout the year. The effective resilient modulus serves as an equivalent value that would result in the same level of pavement damage as would occur if actual seasonal modulus values were applied in the design process.

6.2 Layer Coefficient Development Analysis Procedure

The general methodology used in this research is adapted from the performance-informed mechanistic approach developed by Nemati et al. [84] for asphalt mixtures. This approach was modified to specifically calibrate the layer coefficients for reclaimed and recycled layers, CIR layers in this case. Figure 6-1 illustrates the redefined methodology employed in this study.

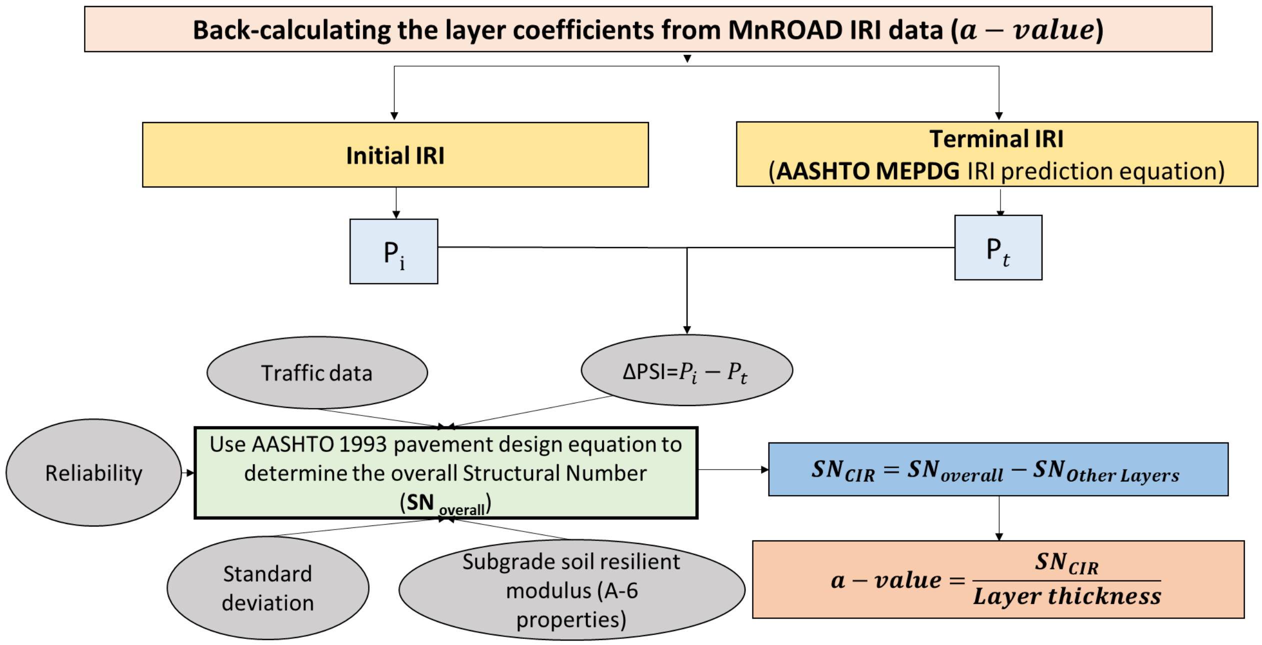


Figure 6-1: Methodology to layer coefficient development

Each step on the flowchart represents the necessary inputs required in evaluating the contributions of reclaimed/recycled layers (CIR) in the form of layer coefficients. To facilitate the a-value developments based on this approach, it is important to determine the necessary inputs based on available information for the test sections. All of these include integrating performance data in the context of field distress metrics for PSI derivation, material properties, as well as traffic information. Considering the extent of data/information available at the time of this report, some engineered decisions and assumptions needed to be made to establish a well-informed calibration for the a-values. These are discussed in the following subsections.

6.2.1 Material Properties

The material properties of all layers, as determined through the in-situ Falling Weight Deflectometer (FWD) measurements described in Chapter 4 of this report, were used for calibrating the a-values. Since the FWD testing campaign spanned climate seasons (fall, winter, spring, and summer), the stiffness properties of all temperature-sensitive layers across all sections were normalized to a reference temperature of 21°C using Equations 4-1 and 4-2. Following the normalization process, the resulting stiffness values of these layers and non-temperature sensitive stiffness metrics of the other layers from these different periods was used to conduct the a-value calibration. This approach provided a range of stiffness values to which the a-values can be determined.

It is important to indicate that the subgrade material stiffness was kept constant across all test campaigns and test sections, primarily adopting the AASHTO A-6 soil as determined from the USDA web soil survey using GPS coordinate of the test sections and confirmed through previous studies [85, 86]. Following determination of the representative material properties, a-values for the bituminous and aggregate base layers in section 2201 and 2202 for obtaining the structural number were simply derived using nomographs and equations developed for such purposes as part of the AASHTO empirical design procedure. The FDR + Fly Ash layer in sections 2207 and 2208 were modeled as an unbound base layer and as such, a-values for unbound materials as part of the empirical design procedure were adopted for these.

6.2.2 Conversion from a 5-layered to a 4-layered Pavement System

The pavement cross-sections for test sections 2201 and 2202 includes a thin (one-inch) bituminous layer beneath the CIR layer and above the granular layer and subgrade, creating a 5-layer system that resembles an inverted pavement. Analyzing this configuration could lead to unrealistic layer coefficients for the reclaimed layers. To address this, the pavement system was reconfigured to resemble a conventional flexible structure with four layers using the Odemark equivalent layer thickness model as expressed in Equation 6-3.

$$h_e = f \times h_1 \sqrt[3]{\frac{E_1(1-v_2^2)}{E_1(1-v_1^2)}}$$

Equation 6-3

Where.
h_e = Equivalent layer thickness (in, m)
f = Odemark's correction factor (0.9)
h_1 = thickness of upper layer (in, m)
E_1, E_2 = Elastic modulus of upper layer and lower layer, respectively. (MPa, psi)
v_1, v_2 = Poisson ration of upper and lower layer, respectively.

For sections 2201 and 2202, the thin bituminous layer and the granular layer were combined into a single representative layer, using the properties of the granular layer. The same approach was applied to sections 2207 and 2208, where both existing FDR layers were combined into a representative layer based on the properties of the FDR + Fly Ash layer. The final cross-sections of all test sections, following the application of the Odemark equivalent layer thickness model [87], are depicted in Figure 6-2.

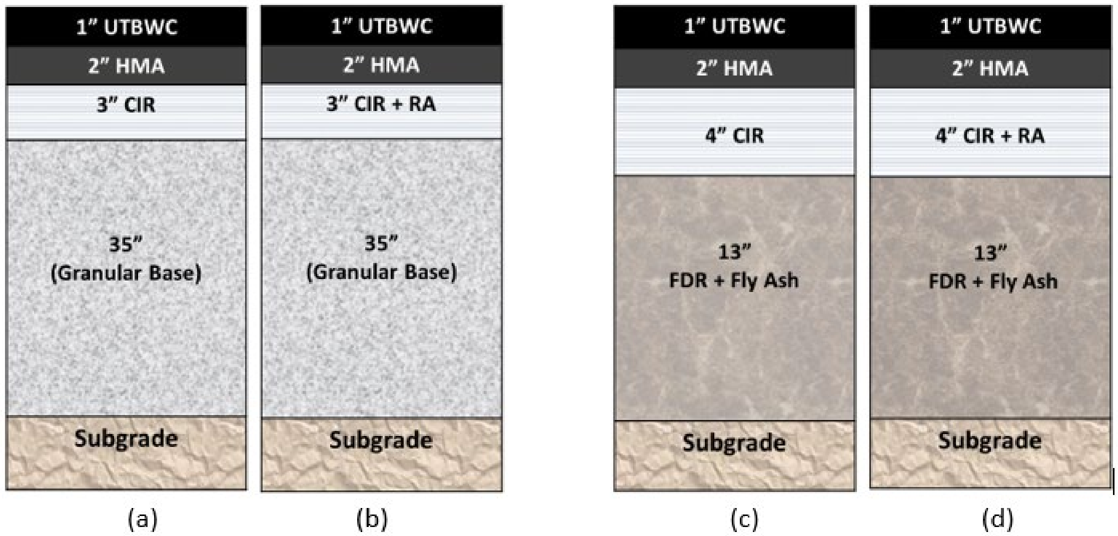


Figure 6-2: Re-modeled pavement sections based on Odemark's equivalent thickness for: (a) 2201, (b) 2202, (c) 2207, and (d) 2208

6.2.3 Field Distress Analysis (Present Serviceability Index Derivation)

One crucial step to evaluating the structural contributions of the CIR layer to the pavement structure is to assess the field performance of such pavement sections following exposure to traffic and/or environmental stressors. PSI, which is an input in the AASHTO empirical design guide, necessitates calibration in order to ensure that the actual performance metrics are used for the design of these pavement structures. The commonly adopted performance metric known as the International Roughness Index (IRI) was utilized in this case for back-calculation of the initial and terminal serviceability. Initial IRI metrics serving as the initial performance index determined for these sections after construction were recorded at 63, 82 and 47 in/mile for Test Sections 2201, 2202 and both 2207 and 2208, respectively. Since these sections were constructed in the year 2022, IRI metrics for the derivation of the terminal performance indices in the context of a 20-year design is not available. To address this data gap, the AASHTOWare Mechanistic-Empirical Pavement Design Guide's (AASHTO MEPDG) IRI prediction equation was utilized, leveraging critical indices such as, surface and structural distresses (cracking and rutting) as well as site conditions in terms of soil properties and climate means for deriving an all-in performance informed a-value. Equation 6-4 details the prediction model, and the predictor values adopted in this study.

$$IRI = IRI_0 + C_1(RD) + C_2(FC_{Total}) + C_3(TC) + C_4(SF)$$

Equation 6-4

Where.

IRI_0 = Initial IRI after construction, in./mile
RD = Average rut depth, in. (0.5 in. used in this analysis)

FC_{Total} = Area of fatigue cracking including alligator, longitudinal and reflective cracking, percent of total lane area. (10 percent lane area, AASHTOWare MEPDG recommended threshold value for interstate)
TC = Length of transverse cracking, ft./mile (500 ft./mi, AASHTOWare MEPDG recommended threshold value for interstate)
SF = Site factor (See Equation 6-5)
$C_{1,2,3,4}$ = Calibration factors ($C_1 = 40.0;\ C_2 = 0.40;\ C_3 = 0.008;\ C_4 = 0.015$)

$$SF = Age^{1.5}\left[ln\big((Precip+1)(FI+1)P_{02}\big) + ln\big((Precip+1)(PI+1)P_{200}\big)\right]$$

Equation 6-5

Where

Age = Pavement age, year
PI = Plasticity index of the soil, % (17.9%, obtained from USDA web soil survey using GPS coordinate of the test sections)
FI = Average annual freezing index, °F days (3,000°F days)
$Precip$ = Average annual precipitation or rainfall, in. (32 in.)
P_{02} = Percent passing the 0.02 mm sieve (65%, [88])
P_{200} = Percent passing the 0.075mm sieve (95%, [88])

Utilizing the information above, the 20-year terminal IRI for all sections were determined to be 127, 146 for sections 2201 and 2202, and 111 for both sections 2207 and 2208.

Following this analysis, the IRI and PSI correlation equation developed by Al-Omari and Dater [89] was used to convert the derived-IRI metrics to PSI. Equation 6-6 depicts the correlation equation.

$$PSI = 5e^{(-0.0038*IRI)}$$

Equation 6-6

Where *IRI* is in in./mile

With the nature of the power function equation, the initial serviceability can significantly vary based on this initial IRI value. It was decided to divide the initial distress metrics into three categories to have realistic initial PSI values based on the actual measured initial IRI values. Table 5-1 indicates these categories and corresponding initial distress metrics for analysis.

Table 6.1: Categories for defining serviceability index based on IRI metrics

Range of field measured IRI, (in./mile)	Suggested Initial Serviceability (Pi)
IRI < 45	5
45≤ IRI ≥ 55	4.5
IRI > 65	Correlation equation (Equation 6-6)

Table 6-2 summarizes the performance indices in terms of PSI for all sections utilized in the analysis.

Table 6.2: Serviceability performance indices for all sections

Test Section	Initial IRI	Terminal IRI	Initial Serviceability (P_i)	Terminal Serviceability (P_t)	ΔPSI
2201	63	127	3.94	3.08	0.85
2202	82	146	3.66	2.87	0.79
2207 & 2208	47	111	4.50	3.28	1.22

6.2.4 Reliability

Design reliability levels in this this study are defined at 75 percent and 95 percent to provide a range of reliability and corresponding a-values for design purposes. Corresponding z-statistic at the 75 percent and 95 percent reliability levels are -0.674 and -1.645, respectively.

6.2.5 Traffic (ESALs)

Traffic on the MnROAD section is not consistently continuous, as is typical on most roadways, due to frequent closures for field and/or forensic investigations. Nonetheless, traffic data available at the time of this report indicates an Average Daily Traffic (ADT) of 15,277 vehicles and an Average Daily Truck Traffic (ADTT) of 3,954 on the driving lane, and ADT of 13,908 and ADTT of 2,142 on the passing lane. Since these sections are a two-lane roadway, developments were based on the driving lane which is considered to experience the heaviest traffic. Considering a 20-year design period with a 1 percent annual traffic growth rate, the design Equivalent Single Axle Loads (ESALs) used for analysis was determined to be approximately 7 million.

6.3 Calibrated a-values for Cold In-Place Recycled Layers

Table 6-3 and 6-4 summarizes the calibrated a-values based on the 75 percent and 95 percent design reliability levels. As demonstrated through the analysis, a-values at 75 percent design reliability ranged 0.18-0.83 and 0.38-1.03 at the 95 percent reliability level. The relatively high a-values observed in certain cases across both design reliability levels can be attributed to the low stiffness values of the other pavement layers in the stiffness property database obtained from the FWD test campaigns. These lower stiffness values influence the back-calculation process, leading to a greater structural demand on the CIR layer. This effect is particularly notable since the overall structural number was based on an A-6 subgrade (modulus of 10 ksi) across each section and test campaign. To thus recommend representative a-values for design, statistical analysis using a normal distribution function was performed, considering the variations in stiffness properties across the FWD test campaigns. The proposed a-values within each design reliability level are defined at stiffness-based reliability levels of 85 percent, and 98 percent. These a-values are highlighted in Table 6-3 and Table 6-4.

By considering the magnitude of the FWD back-calculated stiffness values for the CIR layer across all test sections, this research thus recommends a-values for design based on the design and stiffness-based reliability levels:

- A-value corresponding to the 85 percent stiffness-based reliability for the 75 percent design reliability level
- A-value corresponding to the 98 percent stiffness-based reliability for the 95 percent design reliability level

Table 6.3: Calibrated layer coefficients for CIR layers at 75% design reliability

Test Section	Test Campaign	P_i	P_t	ΔPSI	$SN_{overall}$	$SN_{non-CIR}$	SN_{CIR}	a-value
2201	1	3.94	3.09	0.85	4.69	4.06	0.63	0.21
2201	2	3.94	3.09	0.85	4.69	2.86	1.83	0.61
2201	3	3.94	3.09	0.85	4.69	2.92	1.77	0.59
2201	4	3.94	3.09	0.85	4.69	2.94	1.75	0.58
2202	1	3.66	2.87	0.79	4.81	3.78	1.03	0.34
2202	2	3.66	2.87	0.79	4.81	3.06	1.75	0.58
2202	3	3.66	2.87	0.79	4.81	2.62	2.19	0.73
2202	4	3.66	2.87	0.79	4.81	3.62	1.19	0.40
2207	1	4.50	3.28	1.22	4.18	2.71	1.47	0.37
2207	2	4.50	3.28	1.22	4.18	1.73	2.45	0.61
2207	3	4.50	3.28	1.22	4.18	1.47	2.71	0.68
2207	4	4.50	3.28	1.22	4.18	3.43	0.75	0.19
2208	1	4.50	3.28	1.22	4.18	2.97	1.21	0.3
2208	2	4.50	3.28	1.22	4.18	2.30	1.87	0.46
2208	3	4.50	3.28	1.22	4.18	0.84	3.34	0.83
2208	4	4.50	3.28	1.22	4.18	2.27	1.90	0.48

P_i: Initial Serviceability

P_t: Terminal Serviceability

ΔPSI: Present Serviceability Index

$SN_{overall}$: Overall Pavement Structural Number

$SN_{non-CIR}$: Structural number for non-CIR layers

SN_{CIR}: Structural Number for CIR layers

Average a-value (50% Stiffness-based reliability)	**0.50**
Standard Deviation	**0.19**
85% Stiffness-based reliability	**0.31**
98% Stiffness-based reliability	**0.12**

Table 6.4: Calibrated layer coefficients for CIR layers at 95% design reliability

Test Section	Test Campaign	P_i	P_t	ΔPSI	$SN_{overall}$	$SN_{non-CIR}$	SN_{CIR}	a-value
2201	1	3.94	3.09	0.85	5.56	4.06	1.50	0.50
2201	2	3.94	3.09	0.85	5.56	2.86	2.69	0.90
2201	3	3.94	3.09	0.85	5.56	2.92	2.64	0.88
2201	4	3.94	3.09	0.85	5.56	2.94	2.62	0.87
2202	1	3.66	2.87	0.79	5.70	3.78	1.91	0.64
2202	2	3.66	2.87	0.79	5.70	3.06	2.64	0.88
2202	3	3.66	2.87	0.79	5.70	2.62	3.07	1.02
2202	4	3.66	2.87	0.79	5.70	3.62	2.08	0.69
2207	1	4.50	3.28	1.22	4.94	2.71	2.23	0.56
2207	2	4.50	3.28	1.22	4.94	1.73	3.21	0.80
2207	3	4.50	3.28	1.22	4.94	1.47	3.47	0.87
2207	4	4.50	3.28	1.22	4.94	3.43	1.51	0.38
2208	1	4.50	3.28	1.22	4.94	2.97	1.97	0.49
2208	2	4.50	3.28	1.22	4.94	2.30	2.64	0.65
2208	3	4.50	3.28	1.22	4.94	0.84	4.10	1.03
2208	4	4.50	3.28	1.22	4.94	2.27	2.67	0.67

P_i: Initial Serviceability

P_t: Terminal Serviceability

ΔPSI: Present Serviceability Index

$SN_{overall}$: Overall Pavement Structural Number

$SN_{non-CIR}$: Structural number for non-CIR layers

SN_{CIR}: Structural Number for CIR layers

Average a-value (50% Stiffness-based reliability)	**0.74**
Standard Deviation	**0.20**
85% Stiffness-based reliability	**0.54**
98% Stiffness-based reliability	**0.34**

Chapter 7: Summary and Conclusions

This study explored the use of cold reclamation and recycling techniques to develop perpetual pavements, focusing on sustainable, cost-effective, and environmentally friendly solutions for road rehabilitation.

In the beginning of the project, a survey was distributed to NRRA agency members to understand existing rehabilitation practices, including cold recycling, used by states, current pavement design methodologies used for both perpetual pavements and cold recycled pavements, treatment approaches for cold recycled layers, and methodologies for tracking cold recycled treated pavement performance together with agency preference for rehabilitating/reconstructing cold recycled pavement structures. Responses highlighted agency preferences for rehabilitating cold recycled pavements and revealed that CIR, FDR, and SFDR yielded favorable results for pavement longevity and performance. Interviews with representatives from Illinois, Mississippi, and North Dakota Departments of Transportation confirmed these observations and indicated growing interest in CIR and CCPR (Cold Central Plant Recycling) techniques, which remain relatively new to some DOTs.

As part of the 2022 NRRA construction, four CIR test sections were built at MnROAD. Test sections 2201 and 2202 featured bituminous over aggregate base, while 2207 and 2208 included bituminous overlay over SFDR layer. For test sections 2201 and 2207, rejuvenator emulsion was added to engineered emulsion to evaluate its impact on the performance of the CIR layer.

After one year of service, bituminous cores were extracted from the test sections for laboratory testing, including Marshall stability, low-temperature analysis, and Illinois Flexibility Index Tests (I-FIT). The field air void was generally lower than what was achieved during mix design procedure in the lab. This may suggest that either the CIR layer had received relatively higher compaction effort during construction compared with the mix design procedure compaction or the traffic had reduced the air void of the CIR layer by the wheel load compaction during the first years of service. The dry stabilities after one year of service were significantly higher than the original mix designs which agreed with the decrease observed in air void contents of the mixtures after one year of service. The critical low temperatures of the test sections one year after service were relatively close and in the range of -24 to -22°C but were about 25 percent higher compared to the mix design values. This suggests a worse low temperature performance of the mixtures after one year of service compared with the original mix designs. I-FIT results showed worse performance for rejuvenated sections in terms of peak load, fracture energy, and flexibility index. While rejuvenation showed potential for performance improvement during mix design stage, post-service testing indicated either no significant improvement or even a decline in cracking resistance.

The evaluation of the RAP binders together with base binder and base binders + rejuvenator involved rheological characterization, Multiple Stress Creep and Recovery (MSCR), and Linear Amplitude Sweep (LAS) tests on a Dynamic Shear Rheometer (DSR). The Black Space diagram showed Test section 2201-RAP, test section 2202-RAP, test section 2201-1yr, and test section 2202-1yr all appear to be comparable. The near-threshold cracking behavior observed in both Test section 2201-RAP and Test section 2202-RAP can be attributed to their aged condition, the presence of a rejuvenator to mitigate this effect is not apparent given the comparable nature of all materials (binders after 1-yr in service). Significant findings regarding materials recovered and extracted during construction (Test section 2201-RAP and Test section 2202-RAP) demonstrate similar stiffness and resistance to cracking and rutting. With regard to in-service binders, rejuvenator-treated Test section 2201 showed superior fatigue resistance performance in comparison to Test section 2202. However, a reduced resistance to rutting is observed for Test section 2201 in comparison to Test section 2202.

FWD testing was performed by MnDOT and MnROAD on the test sections at various periods. The back-calculated stiffness results for the CIR layer showed a higher stiffness for 2201 compared with 2202. Conversely, 2207 had a lower stiffness compared with 2208 across all testing periods. As for the SCI, Test section 2201 demonstrates a better overall condition on average compared to Test section 2202. Regarding Test sections 2207 and 2208, Test section 2208 shows better condition on average across all testing periods for both the outer wheel path and mid-lane on the driving and passing lanes. As for the peak deflections, Test section 2201 exhibits lower average peak deflection values compared to Test section 2202, indicating better structural performance. Conversely, Test section 2208 shows lower average peak deflection values than Test section 2207 across all testing periods for both the outer wheel path and mid-lane on the driving and passing lanes.

Perpetual analysis results from both driving and passing lanes showed that all test sections are expected to experience bottom-up cracking at some point, particularly on already in-place layers prior to CR application for sections 2201 and 2202 and on CIR and in-place layers for section 2207 and 2208. Additional perpetual analyses were performed to reassess these sections in terms of bottom-up cracking and quantify the adjustments in thickness required to match them more closely with or below the perpetual design limit. Through trial and error, the bituminous and CIR thicknesses were adjusted across all sections and the perpetual design thickness was determined. Results showed that perpetual behavior could be achieved by increasing overlay thickness by one to two inches, CIR depth by one inch, or a combination of both. Finally, representative Layer coefficients (a-values) were recommended within 75 and 95 percent design reliability levels at stiffness-based reliability levels of 85 percent, and 98 percent.

Due to inconsistencies among binder, mixture, and FWD testing results, definitive conclusions on the rejuvenator's efficacy remain unclear. One explanation could be that the rejuvenator improvement is evident when dealing with binder only. In the mixture phase, with only about 2 percent emulsion, this improvement is either diminished or rejuvenation could even result in some deterioration in mixture performance. These findings suggest a need for further testing on CIR mixtures containing rejuvenators, particularly for fatigue, rutting, and low-temperature performance. Additional studies and refinements to the mix design process are recommended to optimize the use of rejuvenators in CIR applications.

Although outside the scope of this study, it is important to emphasize that original MnROAD Cells 2 and 3 (test sections 2203, 2204, 2205, and 2206) received only surface treatments during 2022 NRRA MnROAD construction. As of this report, those pavements have reached year 17—over three times of their intended life—and yet these test sections remain in excellent condition with only the periodic surface treatments applied. This sustained performance supports the concept of perpetual pavement when using SFDR with Engineered Emulsion. It is therefore recommended to continue monitoring these test sections to determine how many additional years they will remain in service.

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Appendix A. Agency Survey

NRRA Perpetual Pavements Project - Survey

Reclamation and Recycling Techniques to Achieve Perpetual Pavements
This survey is being conducted in order to understand the current pavement design methodologies used by stakeholders for design of perpetual pavements as well as those with cold recycled layers as part of the National Road Research Alliance (NRRA) study on "Reclamation and Recycling Techniques to Achieve Perpetual Pavement Characteristics".

If you feel that this survey should be sent to someone else in your agency, please feel free to forward it to them. If you have any questions or concerns, please contact project PI Mohammad Sabouri (**msabouri@braunintertec.com**) or UNH Investigator Eshan Dave (**eshan.dave@unh.edu**).

1. Click "Next" to begin.

2. Agency

3. Full Name

4. Affiliation

5. Contact

   - Phone

   - e-mail address

6. What pavement design approach does your agency currently use for designing Perpetual Pavements?
   - Do not have a formal design method
   - Use empirical (experience-based) approach
   - Use mechanistic-empirical approach (such as, PerROAD)
   - Other (Please specify)

   [Display This Question:
   If What pavement design approach does your agency currently use for designing Perpetual Pavements? = Use empirical (experience-based) approach
   And What pavement design approach does your agency currently use for designing Perpetual Pavements? = Use mechanistic-empirical approach (such as, PerROAD)
   And What pavement design approach does your agency currently use for designing Perpetual Pavements? = Other (Please specify)[

7. Please provide weblink for design methods/standards for designing these Perpetual Pavements (If there are no links please move to "next" to upload file)

[Display This Question:
If What pavement design approach does your agency currently use for designing Perpetual Pavements? = Use empirical (experience-based) approach
And What pavement design approach does your agency currently use for designing Perpetual Pavements? = Use mechanistic-empirical approach (such as, PerROAD)
And And Please provide weblink for design methods/standards for designing these Perpetual Pavements (If there are no links please move to "next" to upload file) Text Response Is Empty
And What pavement design approach does your agency currently use for designing Perpetual Pavements? = Other (Please specify)
And What pavement design approach does your agency currently use for designing Perpetual Pavements? Text Response Is Not Empty]

8. Please upload the design methods/standards for designing these Perpetual Pavements
9. Has your agency utilized cold recycling techniques in rehabilitation of your pavements in last 5 years?
   - Yes
   - No

[Display This Question:
If Has your agency utilized cold recycling techniques in rehabilitation of your pavements in last 5... = Yes]

10. Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that apply)
    - CCPR
    - CIR
    - FDR/SFDR

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CCPR]

11. What pavement design approach is utilized to design thicknesses of cold recycled layer and wearing course/overlay by your agency for CCPR? (Select all that apply)
    - AAHSTO 1993
    - AASHTOWare PEMD
    - In-house Design (Please specify)
    - Others (Please specify)

[Display This Question:

If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CIR]

12. What pavement design approach is utilized to design thicknesses of cold recycled layer and wearing course/overlay by your agency for CIR? (Select all that apply)
    - AAHSTO 1993
    - AASHTOWare PEMD
    - In-house Design (Please specify)
    - Other (Please Specify)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = FDR/SFDR]

13. What pavement design approach is utilized to design thicknesses of cold recycled layer and wearing course/overlay by your agency for FDR/SFDR? (Select all that apply)
    - AAHSTO 1993
    - AASHTOWare PEMD
    - In-house Design (Please specify)
    - Other (Please Specify)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CCPR]

14. Please provide links of design methods/standards used by your agency for CCPR treated pavements (If there are no links, move to "next" to upload file)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CCPR
And And Please provide links of design methods/standards used by your agency for CCPR treated pavements (... Text Response Is Empty]

15. Please upload design methods/standards used by your agency for CCPR treated pavements

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CIR]

16. Please provide links of design methods/standards used by your agency for CIR treated pavements (If there are no links, move to "next" to upload file)

    [Display This Question:
    If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CIR
    And And Please provide links of design methods/standards used by your agency for CIR treated pavements (I... Text Response Is Empty]

17. Please upload design methods/standards used by your agency for CIR treated pavements

    [Display This Question:
    If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = FDR/SFDR]

18. Please provide links of design methods/standards used by your agency for FDR/SFDR treated pavements (If there are no links, move to "next" to upload file)

    [Display This Question:
    If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = FDR/SFDR
    And And Please provide links of design methods/standards used by your agency for FDR/SFDR treated pavemen... Text Response Is Empty

19. Please upload design methods/standards used by your agency for FDR/SFDR treated pavements

    [Display This Question:
    If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CCPR]

20. What is your agency's process for selecting CCPR as pavement rehabilitation technique (for example, pavement attributes/distresses that would result in CCPR to be considered)?

    [Display This Question:
    If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CIR]

21. What is your agency's process for selecting CIR as pavement rehabilitation technique (for example, pavement attributes/distresses that would result in CIR to be considered)?

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = FDR/SFDR]

22. What is your agency's process for selecting FDR/SFDR as pavement rehabilitation technique (for example, pavement attributes/distresses that would result in FDR/SFDR to be considered)?

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = CCPR]

23. On roads that are treated using the CCPR process, what type of stabilizing agent is used most commonly?
    - o Asphalt Emulsion
    - o Portland Cement
    - o Foamed Asphalt
    - o Lime (including lime kiln dust)
    - o No Stabilizing agent
    - o Others (Please specify)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = CIR]

24. On roads that are treated using the CIR process, what type of stabilizing agent is used most commonly?
    - o Asphalt Emulsion
    - o Portland Cement
    - o Foamed Asphalt
    - o Lime (including lime kiln dust)
    - o No Stabilizing agent
    - o Others (Please specify)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = FDR/SFDR]

25. On roads that are treated using the FDR/SFDR process, what type of stabilizing agent is used most commonly?
    - o Asphalt Emulsion

- Portland Cement
- Foamed Asphalt
- Lime (including lime kiln dust)
- No Stabilizing agent
- Others (Please specify)

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = FDR/SFDR]

26. What percentage of roads utilizing the FDR/SFDR process is just reclamation without any added materials or stabilizing agent?

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = CCPR]

27. Has your agency used any proprietary material in CCPR treated pavement? (such as, recycling agents)
    - Yes
    - No

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = CIR]

28. Has your agency used any proprietary material in CIR treated pavement? (such as, recycling agents)
    - Yes
    - No

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = FDR/SFDR]

29. Has your agency used any proprietary material in FDR/SFDR treated pavement? (such as, recycling agents)
    - Yes
    - No

[Display This Question:
If Has your agency used any proprietary material in CCPR treated pavement? (such as, recycling agents) = Yes

30. Please provide name(s) of proprietary material used in CCPR treated pavementa and indicate their purpose.
    Material _______________
    Purpose _______________

[Display This Question:
If Has your agency used any proprietary material in CIR treated pavement? (such as, recycling agents) = Yes]

31. Please provide name(s) of proprietary material used in CIR treated pavements and indicate their purpose.
    Material _______________
    Purpose _______________

[Display This Question:
If Has your agency used any proprietary material in FDR/SFDR treated pavement? (such as, recycling a... = Yes]

32. Please provide name(s) of proprietary material used in FDR/SFDR treated pavements and indicate their purpose.
    Material _______________
    Purpose _______________

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CCPR]

33. What rehabilitation techniques does your agency commonly use or plan to use on CCPR treated roads when they reach end of life?

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years? (Select all that... = CIR]

34. What rehabilitation techniques does your agency commonly use or plan to use on CIR treated roads when they reach end of life?

[Display This Question:
If Which cold recycling technique(s) have been used by your agency in last 5 years?
(Select all that... = FDR/SFDR]

35. What rehabilitation techniques does your agency commonly use or plan to use on FDR/SFDR treated roads when they reach end of life?

[Display This Question:
If Has your agency utilized cold recycling techniques in rehabilitation of your pavements in last 5... = Yes]

36. Has your agency undertaken any efforts to track field performances of cold recycling treated pavements in any projects?
   - o Yes
   - o No

[Display This Question:
If Has your agency undertaken any efforts to track field performances of cold recycling treated pave... = Yes]

37. If reports/data from performance tracking projects are available on the internet, we would appreciate if you would share those links with us.

   Please first select number of weblinks that you wish to share.
   - o 0
   - o 1
   - o 2
   - o 3
   - o 4
   - o 5

[Display This Question:
If If reports/data from performance tracking projects are available on the internet, we would apprec... != 0]

38. Please provide weblink of a report related to a project tracking performance of CIR,CCPR, FDR/SFDR (please provide link in this format "Name of project - link").

[Display This Question:
If Has your agency undertaken any efforts to track field performances of cold recycling treated pave... = Yes]

39. If you wish to upload reports/data with the performance tracking project(s), please select below the number of reports/files that you wish to upload
    - ○ 0
    - ○ 1
    - ○ 2
    - ○ 3
    - ○ 4
    - ○ 5

40. Please upload report(s) related to these project(s)

41. Can we contact you to obtain further information?
    - ○ Yes
    - ○ No

[Display This Question:
If Can we contact you to obtain further information? = Yes]

42. What communication methods are preferred (Select all that apply)?
    - ○ Zoom
    - ○ Phone Call
    - ○ Email
    - ○ Microsoft Teams

This is the end of the survey. Thank you for participating in this National Road Research Alliance (NRRA) survey for the study on "Reclamation and Recycling Techniques to Achieve Perpetual Pavement Characteristics". If you feel that this survey should be sent to someone else in your agency, please feel free to forward it to them. If you have any questions or concerns, please contact project PI Mohammad Sabouri (msabouri@braunintertec.com) or UNH Investigator Eshan Dave (eshan.dave@unh.edu).

If you are ready to submit the survey responses, please press "next".

Appendix B. Stiffness Back-calculation

Follow this link to Back-calculation stiffness results.xlsx