Using Rubberized Asphalt to Maintain High Volume Traffic Roads in Dickinson County

Final Report

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Dickinson County Road Commission

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EXECUTIVE SUMMARY

The report summarizes the application of rubber modified asphalt pavement with the dry process. A demonstration project with such dry process was paved in Dickinson, Michigan, in June 2019. Three sections were paved in the project: control leveling mixture and control surface mixture, control leveling mixture and rubber modified surface mixture, and rubber modified leveling mixture and rubber modified surface mixture. The loose asphalt mixes were transported to the MTU asphalt mixture lab for further testing. The loose asphalt mixes were compacted using Superpave gyratory compactor (SGC) to prepare cylinders. The noise measurement was conducted on three sections of the project and the comparison was provided.

The report included seven sections. The first section introduced the implementation of rubber modified asphalt mixture using the dry process on the demonstration project. The second section described the volumetric design of rubber modified asphalt mixture with the dry process. The third section evaluated the rutting characteristics of rubber modified asphalt mixture. The fourth section assessed the low temperature cracking property of rubber modified asphalt mixture. The fifth section described the performance assessment of the binder from the mixture. The sixth section introduced the detailed field application of the rubberized mixture with the dry process. The last section is the project summary, discussion, and conclusions. The major achievements of this project are listed as the follows:

(1) The volumetric design of rubber modified asphalt mixture using the dry process was proposed and implemented. In order to guarantee the consistency of the asphalt mixture design in the laboratory and the results of mixture construction in the plant, the aggregate gradation should be controlled strictly during the laboratory mixture design procedure. The compacted mix should be kept in the mold for 30 minutes during sample cooling in order to allow the final rubber/binder reactions to take place. The design asphalt binder content for the leveling layer and the surface layer asphalt mixture are 4.2% and 5.9%, respectively. Dry process rubber mix designs test well in the lab, as long as they are handled properly.

(2) The high temperature rutting performance of plant mixed and laboratory compacted asphalt mixture was assessed using the Hamburg wheel tracking device (HWTD) test. For the dense graded asphalt mixture with 10% rubber addition, the leveling layer asphalt mixture had bigger NMAS, higher RAP content, and less asphalt content than the surface layer

asphalt mixture. The leveling layer asphalt mixture had better rut resistance. The rubber modification significantly improved the rutting and moisture damage resistance of the asphalt mixture. Reducing the air void increased the density and the stiffness of the asphalt mixture.

(3) The low temperature cracking property of plant mixed and laboratory compacted asphalt mixture was evaluated using the disc shaped compact tension (DCT) test. For the dense graded asphalt mixture with 10% rubber addition, the smaller NMAS, lower RAP content, and higher asphalt content contributed to the better low temperature cracking property of the surface layer asphalt mixture than that of leveling layer asphalt mixture. The rubber modification in the asphalt mixture increased the thickness of the asphalt film on the aggregate surface and improved the bonding between the asphalt and the aggregate, and thus enhanced the low temperature cracking resistance of the asphalt mixture. When the test temperature increased from -24 ºC to -18 ºC, the fracture energy of the asphalt mixture was increased.

(4) The modified extraction procedure was proposed to improve the accuracy of the extraction procedure. The property of the base asphalt binder and extracted asphalt binder had a good correlation. Based on the DSR test and the ABCD test, the performance grade of the base asphalt was PG 58-34, the performance grade of the base asphalt with rubber was PG 70-34, the performance grade of the extracted asphalt binder from the leveling layer and surface layer without rubber was PG 64-28, and the performance grade of the extracted asphalt binder from the leveling layer and surface layer with rubber was PG 70-28. The aged asphalt binder and rubber in extracted asphalt binder guaranteed the asphalt binder to sustain heavy traffic load, which thus improved the permanent deformation resistance of asphalt binder. The laboratory performance of the dry processed rubber asphalt materials was better than that of conventional pavement materials.

(5) The construction experience of rubber modified asphalt mixture with the dry process in this project can provide guidance for the application of rubberized asphalt mixture in wet freeze climates. Dry process rubber performs well in the field within one-year observation. Dry process rubber can be easily produced by asphalt plant operators as proved by the construction of the project. The quality of the pavement using the dry process could be easily managed by the local agency. Dry process rubber offers MDOT, counties and cities

an opportunity to pave more for less expenses. Rubber can potentially extend road life and reduce life cycle cost, and the dry process has potential to replace polymer modification.

Based on the implementation of the project, the following lessons were learned:

(1) The quality control of mixture design in the laboratory is critical to the success of the application of rubber modified asphalt mixture in the field. During the application of the mixture, making proper adjustment of the mixture design based on the field compaction ability can improve the final pavement performance.

(2) The crumb rubber swells during the placement and compaction procedure and affects the volumetric property of the pavement. Additional compaction during the decreasing temperature period could guarantee the compaction density of the pavement meets the expected results.

(3) The original pavement may have structural distress before the placement of the new surface pavement. Documenting the original pavement condition in future projects can help to check the effectiveness of the new pavement so that the service life of the pavement can be enhanced.

CHAPTER 1: DESIGN AND CONSTRUCTION OF USING RUBBERIZED ASPHALT TO MAINTAIN HIGH VOLUME TRAFFIC ROADS

1.1 Background

In the wet/terminal blend process, fine, recycled tire rubber is added to asphalt binders. The rubber/binder blend is heated to high temperatures, typically for an hour or more. The rubber grains do not melt into the asphalt binder, and because they are denser than the heated asphalt binder, they will settle unless the binder is continuously agitated. The heated rubber absorbs the lighter ends of the heated binder, and the rubber grains soften and become sticky and swollen. The rubber/binder blend is shipped to asphalt plants in trucks, often without any additional agitation. The rubber/binder blend is pumped into a tank at an asphalt plant, and the blend is pumped into the mixture production. The rubberized pavements produced with this modified binder tend to last longer, resist rutting and cracking, and require less maintenance over the life of the pavement.

This technology has several issues. First, if rubber/asphalt blends are not properly agitated, the rubber will settle on trucks and in facility tanks. This can lead to the production of pavements with too little or too much rubber content, and that can damage pavement performance. There are no readily available and accepted technologies to measure the presence of rubber content in asphalt mixes, so quality control is a difficult issue. Second, wet process/terminal blend rubberized asphalt is difficult to produce and place without expensive mix modification. The rubber tends to accumulate in tanks and filters during production, which requires periodic cleaning. Finally, pavements made with wet process rubberized asphalt tend to be the most expensive modified asphalt pavements in terms of material cost, production and placement.

In our dry process, Elastiko^{R} is added at the asphalt mix facility, much like fine aggregate. The material is added to a typical drum plant using a precision loss-in-weight pneumatic feeder that blows rubber into the aggregate before the addition of a binder. The feed rate is precise (plus or minus 1.5% in the field), and the addition of rubber during the mixing process eliminates any settling issues. During production, surge storage, and transport, the heated rubber absorbs the lighter components of the heated binder. No special tanks, pumps or filters are required, and there is no risk of separation. In the field, these rubberized pavements tend to last longer than the standard hot mix, they resist rutting and cracking, and they require less maintenance over the life of the pavement.

Dry process rubberized pavements have been compared to both wet process rubber pavements and polymer modified pavements in the field, and the performance of all modified pavements is comparable. The dry process asphalt places and compacts like a standard hot mix asphalt. Finally, pavements made with the dry process asphalt tend to be the least expensive modified asphalt pavements in terms of material cost, production, and placement. Table 1-1 shows the comparison between the wet/terminal blend process and the dry process (e.g., Elastiko^{R}).

Table 1-1. Wet/Terminal Blend vs. Dry Process (e.g. ElastikoR)

Process	Wet/Terminal Blend	Dry Process (e.g., Elastiko^{R})
Description	Fine, recycled tire rubber is added to asphalt binders	Elastiko^{R} is added at the asphalt mix facility to partly replace fine aggregates
Pros	1. Longer performance than standard hot mix 2. Rutting and cracking resistance 3. Less maintenance	1. No special tanks, pumps, filters, clean-out procedures 2. No risk of rubber separation 3. Interlinked, proven feeder system 4. Excellent quality control 5. Warm mix temperature required for placement 6. Comparable performance with both wet process rubber pavements and polymer modified pavements 7. Least expensive modified asphalt pavements in terms of material cost, production and placement
Cons	1. Rubber separation without properly agitation 2. Difficulty in producing and placing without expensive mix modification 3. Most expensive modified asphalt pavements in terms of material cost, production, and placement	Lack of experience in mix design and production as well as long-term performance. Therefore, the goal of this study is to evaluate the mix design and production as well as field performance with a county road commission.

As far as equipment goes, we use a modified loss-in-weight fiber machine to feed the crumb rubber through the RAP Collar. Other than a forklift to add chemicals and the chemicals themselves, no other equipment is required for asphalt modification. We can bring our equipment on-site to a plant and be operational in a few hours. We can shut down modification with a switch, and we can detach and move the equipment onto a truck in about two hours. This is a simple process for asphalt plants. Figure 1-1 shows the rubber feeder system. Figure 1-2 shows that the paving operation is the same as the conventional asphalt pavement.

Figure 1-1. Rubber feeder system

Figure 1-2 Paving operation – same as the conventional asphalt pavement

1.2 Construction of trial sections

The trial sections were paved on County Road 607 (aka Bass Lake Road), From North of Twin Falls Access Road to North of Badwater Lane in Dickinson County. The conventional and GTR base and overlay were constructed for a comparison study. Figure 1-3 shows the locations of the trial section in Dickinson County. The construction details of the trial sections is demonstrated below. Figure 1-4 shows the project plan of the trial sections in Dickinson County.

Figure 1-3 The locations of the trial section in Dickinson County

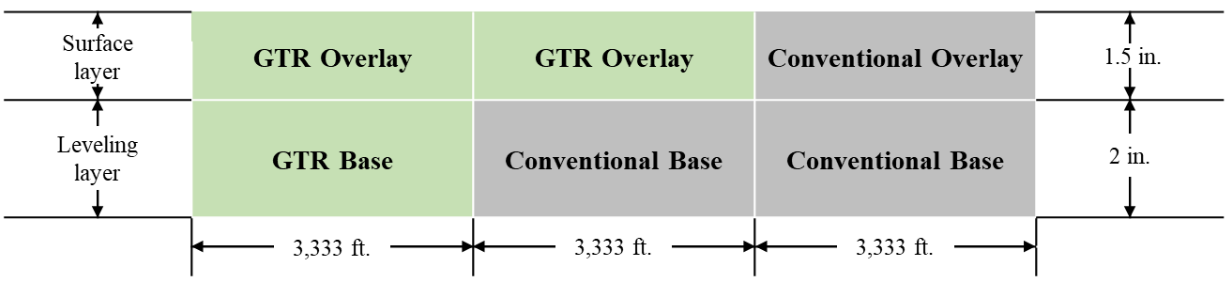


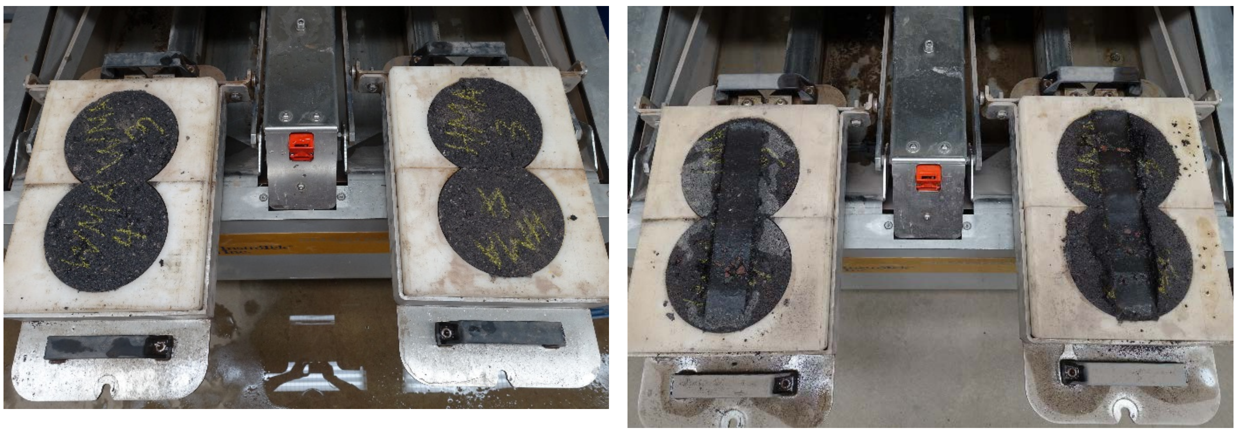
Figure 1-4. The project plan for the GTR and conventional overlay and base

1.3 Field sampling

Field samples were collected for the later lab analysis. In detail, the samples collected from the Dickinson County included the loose aggregate, asphalt binder, drilled cores, and slabs of asphalt mixture. The loose aggregate and asphalt were directly collected from the plant. 42 cores with a diameter of 6 inches were drilled for the field. 3 slabs with the area of 4 square foot were collected.

1.4 Laboratory testing for field collected samples

The field collected samples were transported to the MTU asphalt mixture lab for further testing. The loose asphalt mixes were compacted using Superpave gyratory compactor (SGC) to prepare cylinders, as shown in Figure 1-5. The cylinder samples were used to evaluate anti-rutting performance, moisture susceptibility, and low temperature performance. The corresponding tests included the Hamburg wheel tracking device (HWTD) test, tensile strength ratio (TSR) test, and disc-shaped compact tension (DCT) test.



(a) Before test (b) After test

Figure 1-5: The HWTD test for the cylinder samples

1.5 Summary

This Chapter introduced the implementation of GTR-HMA on trial road sections in Michigan with a dry mix method. The details in the asphalt mix design, material section, construction, field sampling, and lab testing were documented. Since this project was using GTR dry mix HMA in Michigan, the success of this project will provide guidance and help explore the implementation of asphalt pavement containing scrap tire rubber.

CHAPTER 2: VOLUMETRIC DESIGN OF RUBBERIZED ASPHALT MIXTURE WITH THE DRY PROCESS

2.1 Introduction

The dry process is a more cost-effective method than the wet process in adopting a large quantity of crumb rubber without modifying the mixing plant. But the most severe difficulty of the application is the consistency issue of the asphalt mixture during the production due to the interaction between the asphalt binder and rubber particle. The mechanism of the rubber-asphalt binder interaction was that the light component of the asphalt binder was absorbed by the rubber particle, thus resulted in the swelling of the rubber particle in the asphalt mixture. The swelling effect resisted the compaction effort and impeded the mixture from achieving the target density, thus posed a potential threat of distress, such as cracking or rutting. The interaction procedure was influenced by the rubber particle size, asphalt binder content, and the curing time [1]. Higher rubber-asphalt binder interaction degree occurred at asphalt mixture produced with finer rubber particle and higher asphalt content, and the swelling process mainly happened in the first four hour curing time [2].

The volumetric property of the asphalt mixture with rubber was influenced by the swelling process, which affected the target density of the asphalt mixture and thus weakened the consistency of the asphalt mixture. The compaction temperatures significantly influenced the volumetric properties of the crumb rubber modified (CRM) mixtures. The compaction temperature was influenced the viscosity of the asphalt binder and the air void content of the asphalt mixture [3]. The air void and the voids in mineral aggregate were two critical parameters that influenced the rutting performance of the rubber modified asphalt mixtures [4]. Provided curing time for the loose rubber asphalt mixture is helpful for the compaction, but the production cost will increase. One option to reduce the swelling effect on the rubber modified asphalt mixture is using the gap graded mixture design. The mineral filler content was lesser, and the air void content was higher, which provided potential space for the rubber swelling [5]. Some researchers tried to use pre-treated rubber to reduce the swelling influence during the compaction procedure. By using reacted and activated rubber (RAR) in ultra-thin asphalt mixture overlay, the swelling process could be initiated before the compaction of the asphalt mixture, thus guaranteed the compatibility and consistency of the asphalt mixture [6]. Different processed rubber particles had different rubber-asphalt interaction procedures. In this research, the Elastiko 100 is a liquid surface treatment for crumb rubber. The volumetric property of Elastiko 100 treated rubber asphalt mixture is unknown. RAP content in the mixture design also influenced the interaction between the rubber particle and asphalt binder. The aim of this chapter is to conduct the volumetric design of the rubber modified asphalt mixture with RAP using the dry process. This chapter will discuss the volumetric design of rubber modified asphalt mixture with the dry process based on the Superpave mixture design procedure for the leveling layer and the surface layer asphalt mixture with rubber.

2.2 Materials

2.2.1 Aggregate

In order to save project costs, the aggregates used in this research and construction site were taken from local quarries. The aggregate types used in the leveling layer (LVSP) included 5/8" chip, natural sand, manufacture sand, screen sand, reclaimed asphalt pavement materials (RAP), and break down filler (diameter less than 75 μm). The gradation of different types of aggregate used in LVSP is shown in Figure 2-1. The physical properties of the aggregate are shown in Table 2-1. The aggregate types used in the surface layer (5E1) included 1/2" chip, natural sand, manufacture sand, ISP, RAP, and break down filler (diameter less than 75 μm). The gradation of different types of aggregate used in 5E1 is shown in Figure 2-2, and the physical properties of the aggregate are shown in Table 2-2.

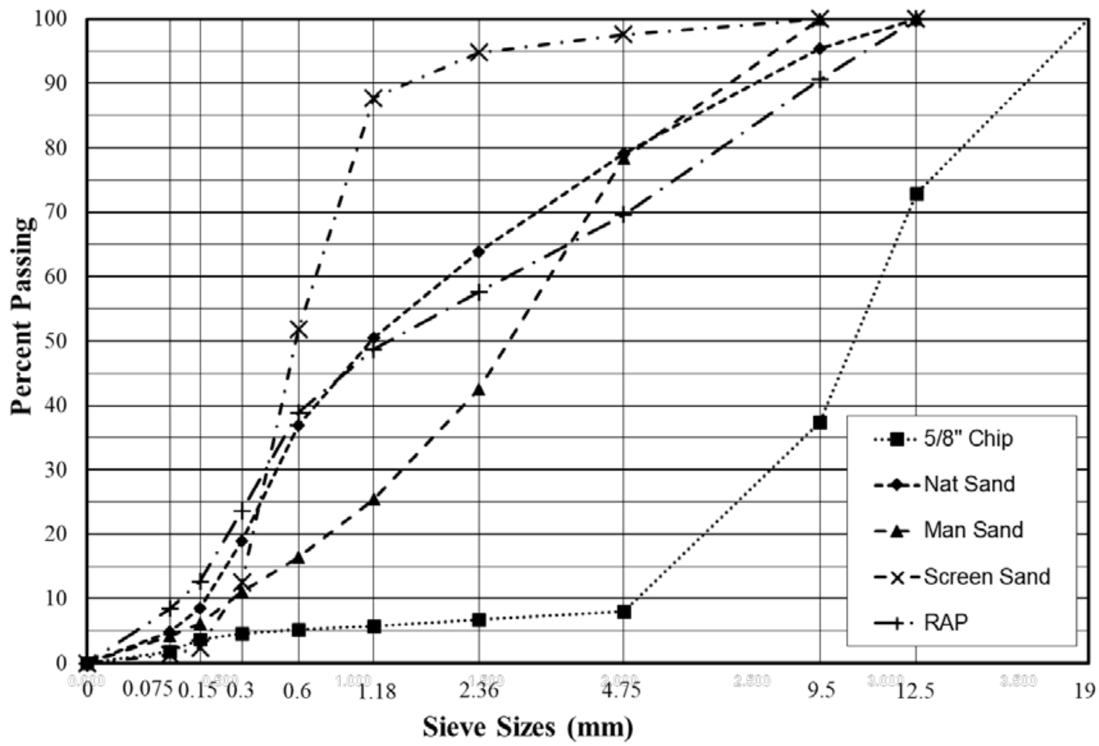


Figure 2-1. The gradation of different types of aggregate used in LVSP

Table 2-1. The properties of different types of aggregate used in LVSP

	5/8" Chip	Natural sand	Manufacture sand	Screen sand	RAP
1 Face crush（%）	89	35.5	100	11.5	82.9
Angularity index	-	41.5	47	41	43.8
AWI value #16	265	223	268	-	240
Combined calc. Gsb	2.704	2.700	2.719	2.641	2.709
Fine bulk S.G.	-	2.7	2.714	2.641	2.709
Soft particles（%）	0.8	1.4	0.3	14.5	1.9

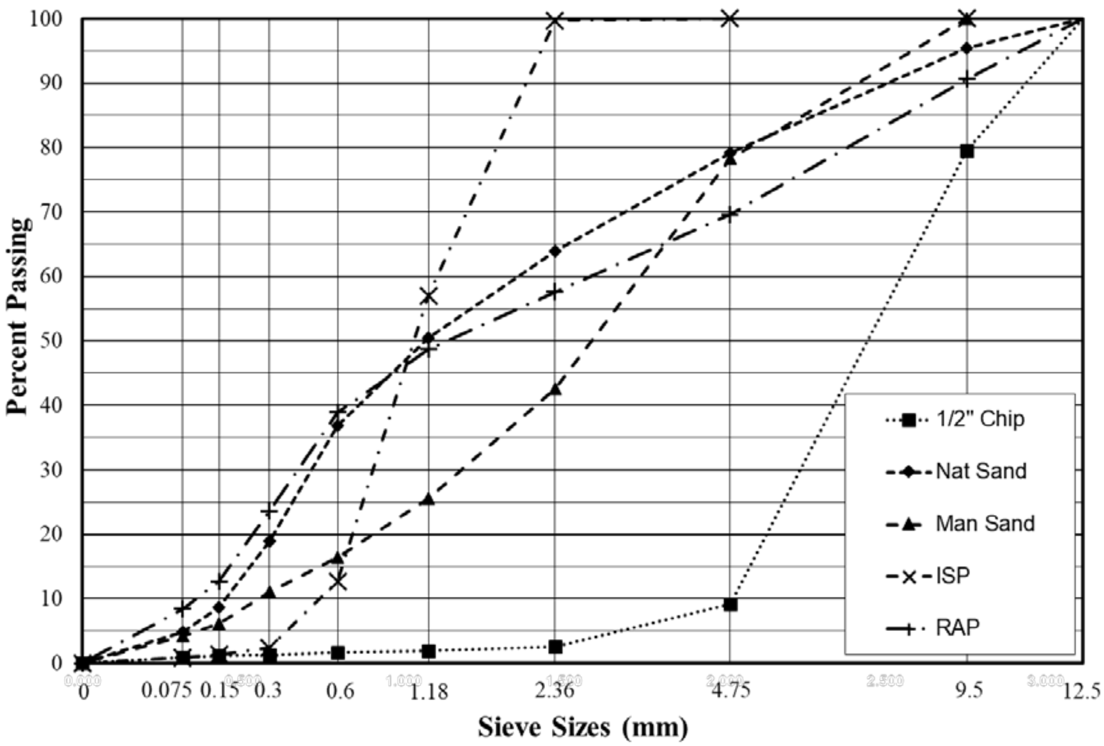


Figure 2-2. The gradation of different types of aggregate used in 5E1

Table 2-2. The properties of different types of aggregate used in 5E1

	½" Chip	Natural sand	Manufacture sand	ISP	RAP
1 Face crush（%）	95	35.5	100	-	82.9
Angularity index	-	42	48	41	44
AWI value #16	265	222	267	493	240
Combined calc. Gsb	2.716	2.700	2.712	2.765	2.709
Fine bulk S.G.	-	2.7	2.714	2.765	2.708
Soft particles（%）	0.8	1.5	0.9	-	1.9

A representative RAP was extracted, and the gradation of RAP aggregate was determined based on the extracted aggregate. The percentage of aggregate passing certain sieves for RAP aggregate is shown in Table 2-3.

Table 2-3. Percentage of aggregate extracted from RAP passing different sieves

	3/8"	# 4	# 8	# 16	# 30	# 50	# 100	# 200
Design	90.6%	69.6%	57.6%	48.7%	38.9%	23.6%	12.7%	8.4%
Measure	83.1%	66.6%	54.3%	45.1%	36.9%	24.4%	13.7%	9.6%

The asphalt binder in RAP was determined based on the extraction test. The measured asphalt binder content in RAP is 4.57%, and the designed value is 4.15%. The measured content is slightly higher than the design value due to more fine aggregate adopted in the sample used in the test.

2.2.2 Asphalt binder

The PG 58-34 asphalt binder was used in the project, and the basic properties of the asphalt binder is shown in Table 2-4.

Table 2-4. The basic properties of asphalt binder PG 58-34

Properties	Test result	Standard value
Viscosity (135 ºC， Pa·S,)	0.398	< 3.0
Unaged $G^{*}/\sin\delta$ (58 ºC, kPa)	1.542	> 1.0
RTFO aged $G^{*}/\sin\delta$ (58 ºC, kPa)	4.147	> 2.2
PAV aged $G^{*}\times\sin\delta$ (16 ºC, kPa)	2346	<5000

2.2.3 Dry process rubber

The crumb rubber used in the project was treated with Elastiko 100, as shown in Figure 2-3. The chemically engineered crumb rubber product used in production complied with an ASTM 30 minus specification (ASTM 5644-01) using ASTM compliant screen sizing on samples (ASTM E11-15) and dry, free of visible debris or other contaminants. The crumb rubber surfaces were chemically engineered to improve the workability of the rubber-modified asphalt during handling, placement, and compaction. The basic properties of Elastiko 100 treated rubber are shown in Table 2-5. The gradation of Elastiko 100 processed crumb rubber is shown in Table 2-6.

Figure 2-3. The Elastiko 100 treated crumb rubber

Table 2-5. The basic property of ELASTIKO 100 processed crumb rubber

Property	Measured result
Primary ingredient	ASTM-30 recycled crumb rubber
Appearance	Black, fine-grained crumb material with white flecks
Specific gravity	1.15 ± 0.04
Flash point	246 ºC
Auto-Ignition temperature	371 ºC
Solubility in water	0.1-1.0%

Table 2-6. The gradation of Elastiko 100 processed crumb rubber

Sieve	Percent Passing
No. 16 (1.18mm)	100
No. 30 (0.6mm)	96 - 99
No. 40 (0.425mm)	70 - 74
No. 100 (0.150mm)	42 - 48
No. 200 (0.075mm)	0 - 12

2.2.4 Anti-stripping agent

In order to guarantee the moisture susceptibility of asphalt mixture, ZycoTherm-SP (ZT-SP) anti-stripping agent was adopted in this study to improve the anti-stripping property between the asphalt and aggregate in the asphalt mixture. During the application procedure, ZT-SP is added to the asphalt and stirred for 2 minutes to fully mix with the asphalt. For rubber modified asphalt or mixtures containing RAP, the recommended dosage by the vendor is shown in Table 2-7.

Table 2-7. The recommended dosage of ZT-SP under different conditions

Binder type	Indicative ZT-SP dosage（%）
Crumb rubber modified asphalt binder	0.1
RAP 20%	0.08
RAP 35%	0.1
RAP 45%	0.125

2.3 Volumetric design

The volumetric design of rubberized asphalt mixture design followed the Superpave mixture design. The difference between the volumetric design of rubber modified asphalt mixture and conventional asphalt mixture is the interaction between the rubber particle and asphalt binder in rubber modified asphalt mixture. The swell of the crumb rubber affected the volumetric properties of the asphalt mixture. The quality control of the laboratory design is critical to guarantee that design results could successfully represent the mixture construction in the plant.

2.3.1 Prepare aggregate trail blend gradations

The volumetric design of the leveling layer and surface layer asphalt mixture design criteria are displayed in Table 2-8.

Table 2-8. The design criteria of different asphalt mixture types

Design criteria	Mixture type	
	Surface layer	Leveling layer
Design number of gyration (N_{design})	76	45
Percent of maximum specific gravity (%G_{mm}) at N_{design}	96	96
VMA min % at N_{design} (based on G_{sb})	15	14
VFA at N_{design}（%）	65-78	70-80
Tensile strength ratio（TSR）	≥ 80%	≥ 80%

The gradation requirements of different asphalt mixture types are shown in Table 2-9. The surface layer asphalt mixture gradation restricted zone is shown in Table 2-10. Normally, the aggregate gradation should avoid passing through the restricted zone, but the mixture aggregate gradation can pass through the gradation restricted zone above the maximum density line. There is no restricted zone requirement for the leveling layer asphalt mixture.

Table 2-9. The gradation restrictions of different asphalt mixture types

Sieve size	Percent passing（Control point）	
	Surface layer	Leveling layer
¾ inch (19 mm)		100
½ inch (12.5 mm)	100	75 - 95
3/8 inch (9.5 mm)	90 - 100	60 - 90
No. 4 (4.75 mm)	≤ 90	45 - 80
No. 8 (2.36 mm)	32 - 67	30 - 65
No. 16 (1.18 mm)		20 - 50
No. 30 (0.6 mm)		15 - 40
No. 50 (0.3 mm)		10 - 25
No. 100 (0.15 mm)		5 - 15
No. 200 (0.075 mm)		3 - 6

Table 2-10. The percent passing restricted zone for surface layer asphalt mixture

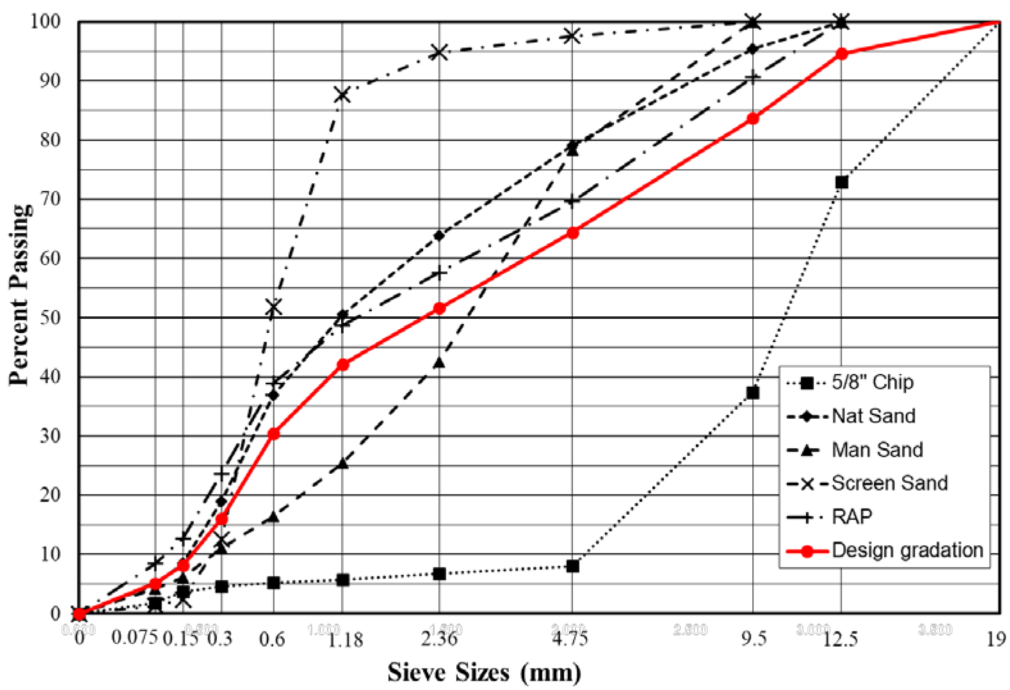
Sieve size	Percent passing
No. 8 (2.36 mm)	47.2
No. 16 (1.18 mm)	31.6 - 37.6
No. 30 (0.6 mm)	23.5 - 27.5
No. 50 (0.3 mm)	18.7

2.3.2 Determination of aggregate gradation

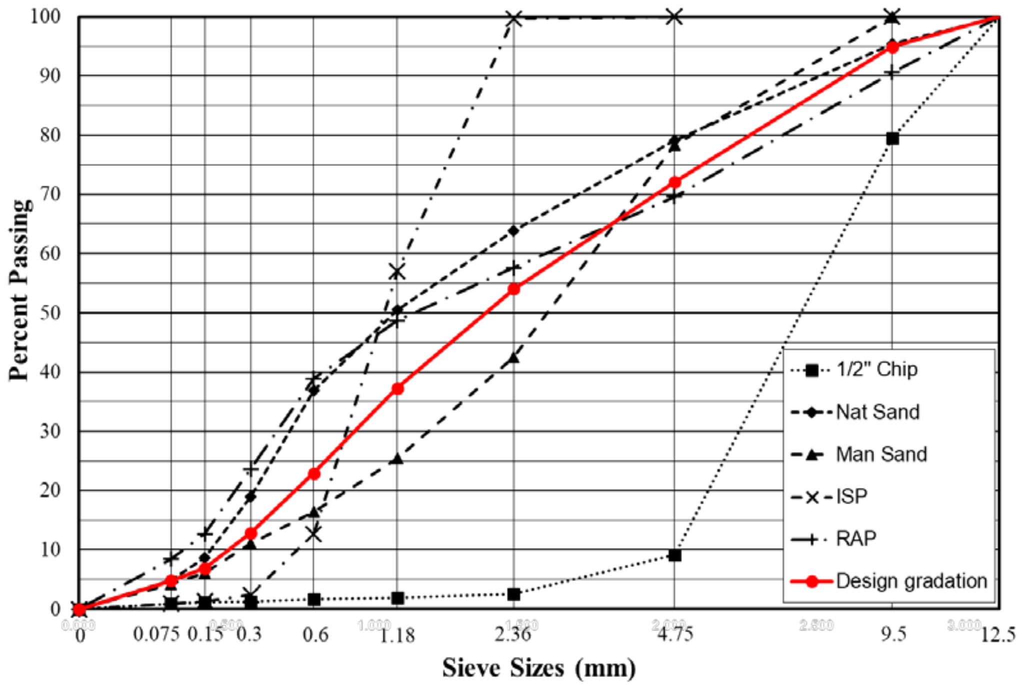
The aggregate percentage of different mixture designs are presented in Table 2-11. The final gradation design of two types of asphalt mixtures are displayed in Figure 2-4. The nominal maximum aggregate size (NMAS) of the surface layer asphalt mixture is 9.5 mm, and the NMAS of the leveling layer asphalt mixture is 12.5 mm.

Table 2-11. The aggregate percentage of different mixture design

Aggregate percentage of LVSP design							
Aggregate type	5/8" Chip	Natural sand	Manufacture sand	Screen sand	RAP	Breakdown	Combined gradation
Blend %	20%	32.5%	12%	10%	25%	0.5%	100%
Aggregate percentage of 5E1 design							
Aggregate type	½" Chip	Natural sand	Manufacture sand	ISP	RAP	Breakdown	Combined gradation
Blend %	12%	22.5%	33%	15%	17%	0.5%	100%



(a) the LVSP mixture design



(b) The 5E1 mixture design

Figure 2-4. The gradation design of two types of asphalt mixture

Due to the uneven distribution of the aggregate in the quarry, the aggregate gradation obtained from the quarry to the laboratory may differ from the actual aggregate gradation. In order to guarantee the consistency of the asphalt mixture design in the laboratory and the results of mixture construction in the plant, the aggregate gradation should be controlled strictly during the laboratory mixture design procedure.

Firstly, different types of aggregate were sieved in the laboratory. Then, the mass of each type of aggregate to prepare one sample was determined based on the mass of one sample and percentage of different types of aggregate in Table 2-11. Finally, the mass of each size aggregate in each type was determined based on Figure 2-1 and Figure 2-2. In order to ensure the repeatability of the test results, at least two test samples were prepared for each test condition.

2.3.3 Rubberized asphalt mixture preparation

The shearing mixing method was used to compact the rubberized asphalt mixture in the lab. Following are the listed steps to prepare sample:

1. Calculate the total amounts of asphalt binder and rubber required to produce the desired sample size.

2. Pre-heat the aggregate to within a range of 360 to 400 ºF (182.2 to 204.4 ºC).

3. Pre-heat the binder to 350 ºF (176.7 ºC). Add the Elastiko to the pre-heated binder. Place the combined rubber and binder in a high-speed, shearing mixer and mix for 30 minutes. Maintain the binder at 350 ºF.

4. Mixing the aggregate and the mixed binder thoroughly in a bucket mixer, but do not mix the materials so long that the temperature of the mix drops substantially. Then place the mixed sample in a preheated 335 ºF (168.3 ºC) oven and short-term aged for two hours immediately.

5. At the completion of the 2-hour period in the oven, allow the mix to cool to 320 ºF (160 ºC). Use the Superpave gyratory compactor to prepare the specimen.

6. Keep the compacted mix in the mold for 30 minutes during sample cooling in order to allow the final rubber/binder reactions to take place, preferably under a fan to facilitate cooling. Covering the mold with a heavily weighted metal plate or similar device to keep the product in the mold is required in order to preserve densities.

2.3.4 Determination of the design binder content

The mixture with the designed gradation at each of the following three binder contents are prepared: (1) the estimated binder content, P_b (design); (2) 0.5 percent below P_b; (3) 0.5 percent above P_b. The design binder content included the crumb rubber because the rubber interacted with rubber in the lab specimen preparation procedure. Following the procedures in Section 2.3.3 to prepare the specimens. Determine the bulk specific gravity (G_{mb}) and theoretical maximum specific gravity (G_{mm}) according to AASHTO T 275 and T 209. Determine the binder content that produces the air void content (V_a) of 4.0 percent at N_{design} gyrations using the following steps:

(1) Obtain Va, VMA, and VFA at N_{design} according to Equations 2.1, 2.2, and 2.3:

$$V_a = 100(1-G_{mb}/G_{mm}) \qquad (2.1)$$

$$VMA = 100 - (G_{mb}\times P_s/G_{sb}) \qquad (2.2)$$

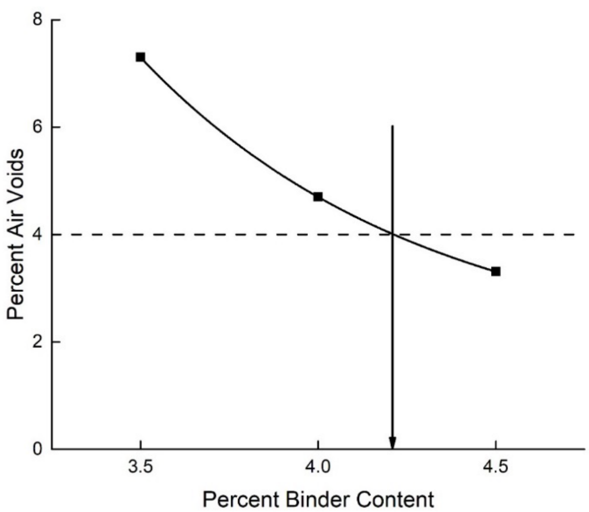
$$VFA = 100\ (VMA\text{-}V_a/VMA) \tag{2.3}$$

(2) Determine the sample density at $N_{initial}$ ($\%G_{mm\text{-}initial}$), using Equation 2.4:

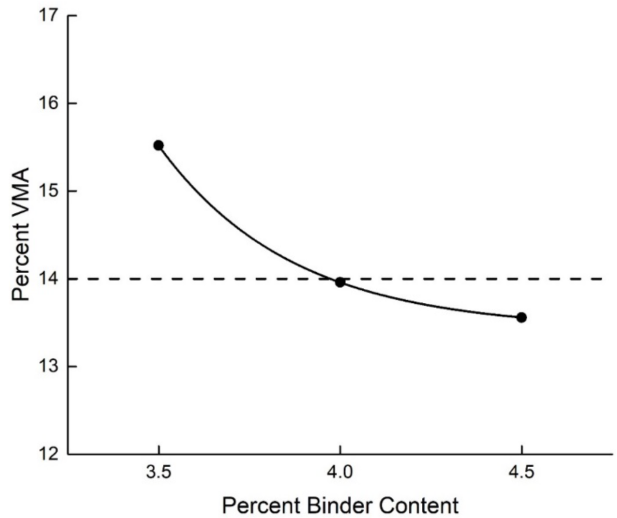
$$\%G_{mm\text{-}initial} = 100\ (G_{mb} \times h_d/G_{mm}h_i) \tag{2.4}$$

(3) Get the average *Va*, VMA, and VFA, and density at N_{design} for samples under different binder contents. Obtain the binder content that the *Va* value is equal to 4.0 percent.

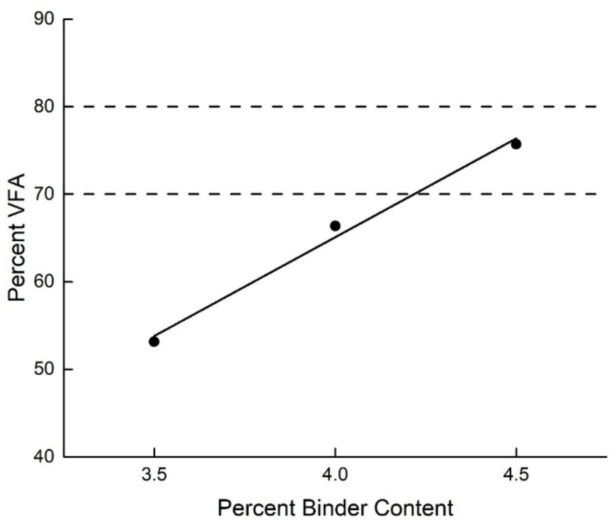
The average *Va*, VMA, and VFA, and density at N_{design} for samples under different binder contents of the leveling layer mixture design are shown in Figure 2-5. The volumetric design result for different asphalt binder contents are shown in Table 2-12.



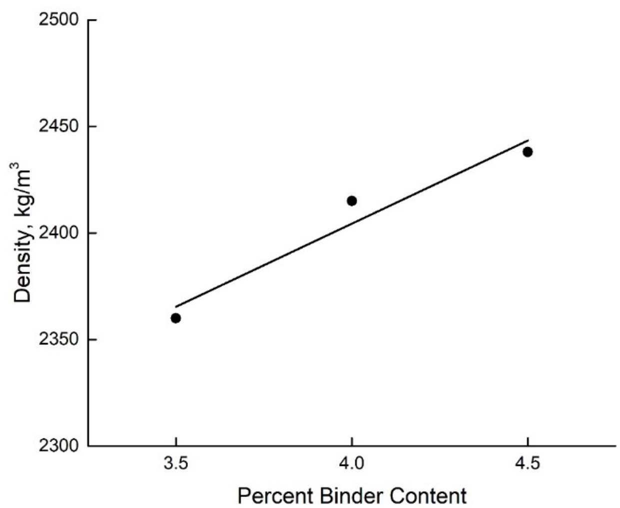
(a) Air void



(b) VMA



(c) VFA



(d) Density

Figure 2-5. The relation between binder content and volumetric parameters of the leveling layer mixture design

Table 2-12. Volumetric design result – leveling layer

P_b (%)	V_a (%) [1]	VMA (%) [2]	VFA (%) [3]	Density at N_{design} (kg/m^3)
3.5	7.27	15.52	53.15	2360
4.0	4.70	13.96	66.37	2415

4.5	3.29	13.56	75.72	2438

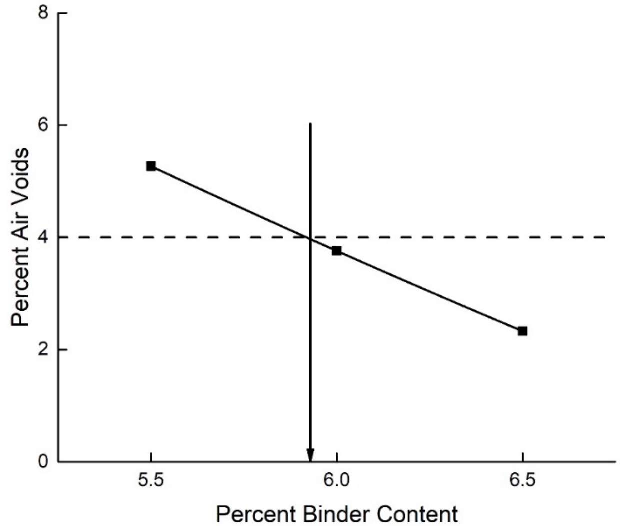
Note: 1. The design binder content is 4.2% (4.0 % air void), see Figure 2-5.

2. Based on the VMA min 14% at N_{design} (Table 2-12), asphalt binder content should be lower than 3.97%.

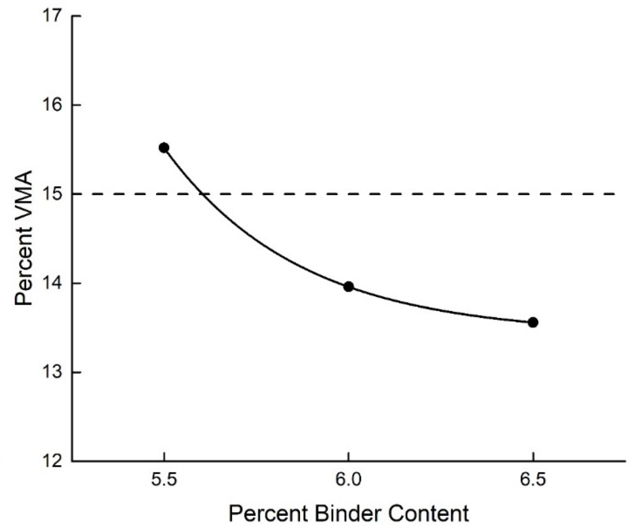
3. Based on the VFA 70%-80% at N_{design} (Table 2-12), asphalt binder content should be higher than 4.2%, the same as the condition of 4.0% air void.

Based on the Table 2-12 result, the air void of asphalt mixture with 4.2% binder content is 4.0%. The 4.2% asphalt binder meets the VFA criteria. However, the VMA does not meet the criteria. Based on the data result, the asphalt binder content should be lower than 3.97% to meet the criteria. Due to the adoption of rubber, the asphalt binder content selected 4.2% for the moisture susceptibility performance evaluation.

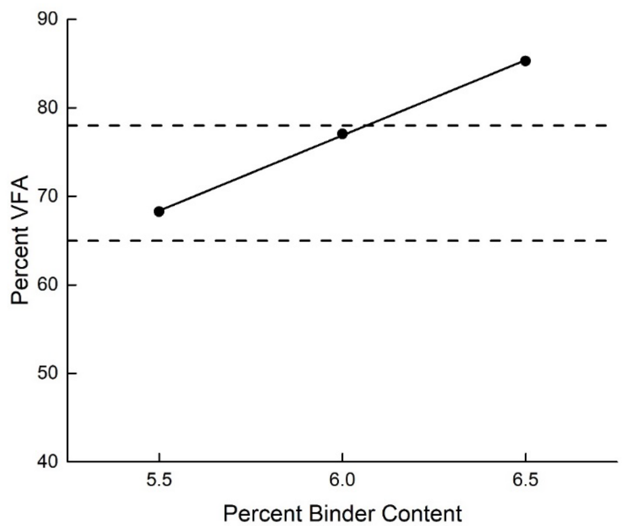
The average V_a, VMA, and VFA, and density at N_{design} for samples under different binder contents of the surface layer mixture design are shown in Figure 2-6. The volumetric design result for different asphalt binder contents are shown in Table 2-13.



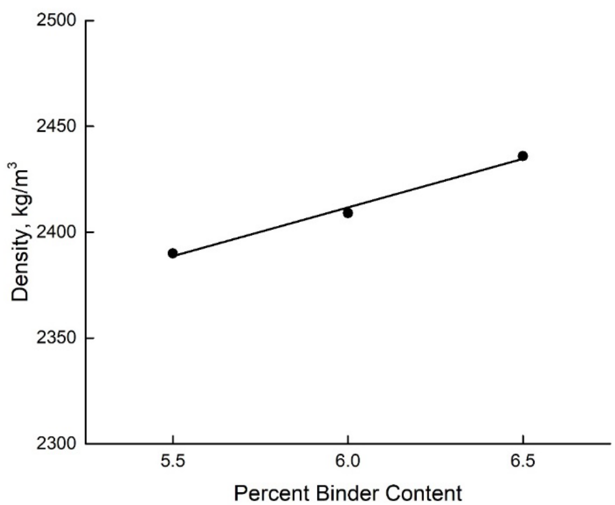
(a) Air void



(b) VMA



(c) VFA



(d) Density

Figure 2-6. The relation between binder content and volumetric parameters of the surface layer mixture design

Table 2-13. Volumetric design result- surface layer

P_b (%)	V_a (%) [1]	VMA (%) [2]	VFA (%) [3]	Density at N_{design} (kg/m^3)
5.5	5.27	16.62	68.28	2390
6.0	3.76	16.35	77.04	2409

6.5	2.33	15.81	85.29	2436

Note: 1. The design binder content is 5.9% (4.0 % air void), see Figure 2-6.

2. Based on the VMA min 15% at N_{design} (Table 2-13), asphalt binder content should be lower than 5.6%.

3. Based on the VFA 65%-78% at N_{design} (Table 2-13), asphalt binder content should be lower than 6.1%.

Based on the Table 2-13 result, the air void of asphalt mixture with 5.9% binder content is 4.0%. The 5.9% asphalt binder meets the VFA criteria. However, the VMA does not meet the criteria. Based on the data result, the asphalt binder content should be lower than 5.6% to meet the criteria. Due to the adoption of rubber, the asphalt binder content selected 5.9% for the moisture susceptibility performance evaluation.

2.3.5 Moisture susceptibility evaluation

In order to improve the moisture resistance, dosages of 0.125% and 0.08% ZT-SP (anti-stripping agent) (by weight of binder) were selected for LVSP and 5E1, respectively. The moisture susceptibility of LVSP and 5E1 asphalt mixture design all met the standard restriction (TSR>0.8). The TSR results of LVSP and 5E1 asphalt mixture design are shown in Table 2-14, the TSR values of LVSP and 5E1 asphalt mixture met the standard restriction.

Table 2-14. The TSR test results of LVSP and 5E1 asphalt mixture

Sample	Average tensile strength of unconditioned samples	Average tensile strength of conditioned samples	TSR
LVSP	175.08	151.44	0.86
5E1	165.7	149.79	0.90

2.4The final mixture design

The final aggregate gradation of two types of asphalt mixture is shown in Figure 2-4 and Table 2-11. The RAP content in the surface layer asphalt mixture and the leveling layer asphalt mixture are 17% and 25%, respectively. The NMAS of the surface layer asphalt mixture is 9.5 mm, and the NMAS of the leveling layer asphalt mixture is 12.5 mm. The design asphalt binder content for the leveling layer and the surface layer asphalt mixture are 4.2% and 5.9%, respectively. The dosages of an anti-stripping agent ZT-SP for the leveling layer asphalt mixture and the surface layer asphalt mixture were 0.125% and 0.08%, respectively.

2.5 Conclusion

This chapter introduced the volumetric design of rubberized asphalt mixture with the dry process, the aggregate trail blend gradations preparation, aggregate gradation determination, rubberized asphalt mixture preparation, design binder content determination, and moisture susceptibility evaluation were proposed. The following conclusions could provide guidance for future rubberized asphalt mixture applications.

(1) In order to guarantee the consistency of the asphalt mixture design in the laboratory and the results of mixture construction in the plant, the aggregate gradation should be controlled strictly during the laboratory mixture design procedure.

(2) The compacted mix should be kept in the mold for 30 minutes during sample cooling in order to allow the final rubber/binder reactions to take place.

(3) The design asphalt binder content for the leveling layer and the surface layer asphalt mixture are 4.2% and 5.9%, respectively.

CHAPTER 3: HIGH TEMPERATURE RUTTING PERFORMANCE EVALUATION OF RUBBERIZED ASPHALT MIXTURE

3.1 Introduction

In 2017, about 287.3 million tires were discarded in the US, in which 88.6% of these scrap tires are light-duty tires, and the remaining are commercial tires. The key scrap tire markets included tire-derived fuel (TDF), ground rubber, civil engineering, and other markets [7]. Scrap tires can be used in highway engineering based on different types of rubber products. Shredded or chipped tires can be applied in embankment construction as light weight fill materials. The whole tires can be used for stabilizing roadside shoulders and constructing retaining walls. Ground rubber can be used to partially replace fine aggregate in asphalt pavement [8]. The crumb rubber can be adopted to modify asphalt binder to enhance the high temperature performance of the asphalt mixture with the wet process [9]. Normally, in the dry process, rubber particles were adopted in gap graded asphalt mixture as friction course of asphalt pavement [10].

The dry process is mainly used in hot mix asphalt pavement, and only limited research has been studied for the application of the dry process in the cold mix, chip seals, and surface treatment [11]. The dry process was firstly adopted in the US. with the PlusRive technology. The PlusRide dry process technology was adopted in the Department of Transportation agencies (DOT) in different states, which proved that the strategy was not a cost-effective method [12]. Some test sections performed well, while some did not have comparable performance to conventional asphalt mixture. The dry process can use batch and drum-dryer plants to incorporate rubber into the asphalt mixture, but the mixing temperature and duration need to be adjusted [13]. Unlike conventional asphalt mixture construction, rubber asphalt mixture compaction with dry process needs a finishing roller to compact the mixture until the temperature decreased to 60 ºC. The reaction between the rubber and asphalt binder at elevated temperature will swell the mixture. The quality of rubber modified asphalt mixture with the dry process need to be controlled by monitoring the rubber gradation, rubber percentage, rubber pretreatment, and mixing time. There are still many unsolved issues regarding the application of rubber with the dry process. More field data related to the dry process are needed to evaluate the performance. The influence of the dry process on the environment and the cost-effectiveness needed to be validated. The application of the dry process under different climate conditions with different mixture designs and different raw materials can provide more reliable data for the guidance of further construction.

3.2 Test materials and test method

3.2.1 Test materials

Chapter 2 described the volumetric mixture design of rubberized asphalt mixture using the dry process. The leveling layer and the surface layer used different mixture gradation and asphalt binder content. In order to compare the rutting and moisture performance of rubberized asphalt mixture with the dry process, the leveling layer and the surface layer asphalt mixture with and without rubber were prepared. The gradation and asphalt binder content of asphalt mixture with and without rubber were the same. Totally four types asphalt mixtures were prepared, which included surface layer (5E1) asphalt mixture with rubber, surface layer (5E1) asphalt mixture without rubber, leveling layer (LVSP) asphalt mixture with rubber, and leveling layer (LVSP) asphalt mixture without rubber.

The specimens were prepared with the Superpave gyratory compactor (SGC). The diameter and height of the sample were 150 mm and 60 mm, respectively. The air void content of the specimen was 7.0 ± 1.0%.

The edge of the SGC samples was cut to fit in the cylindrical specimen mounting system, as shown in Figure 3-1. The system was mounted rigidly to the HWTD.

Figure 3-1. The specimen and the cylindrical specimen mounting system

3.2.2 Test method

The HWTD was adopted to evaluate the premature deformation of asphalt mixture because of the structure weakness, asphalt binder stiffness inadequate, or moisture sensitivity, as shown in Figure 3-2. The rutting and moisture susceptibility of asphalt mixture was evaluated according to AASHTO T 324. A reciprocating rolling wheel applied a force back and forth on a submerged sample. The diameter and the width of the steel wheel are 8 in. and 1.85 in., respectively. The load on the steel wheel is 158 ± 1 lb. The wheel passes through the sample 52 ± 2 passes per minute. The maximum speed of the wheel is 1ft/s when the wheel reaches the midpoint of the specimen. The linear variable differential transducer (LVDT) device gets the depth of the wheel during the test procedure until the test ends. The rut depth and the number of passes to failure were measured during the test procedure. The water bath temperature was 50 ºC since the high temperature specification of the PG binder was 58 ºC.

Figure 3-2. The Hamburg Wheel Tracking Device

The HWTT parameters are shown in Figure 3-3, and the definition of these parameters are explained below:

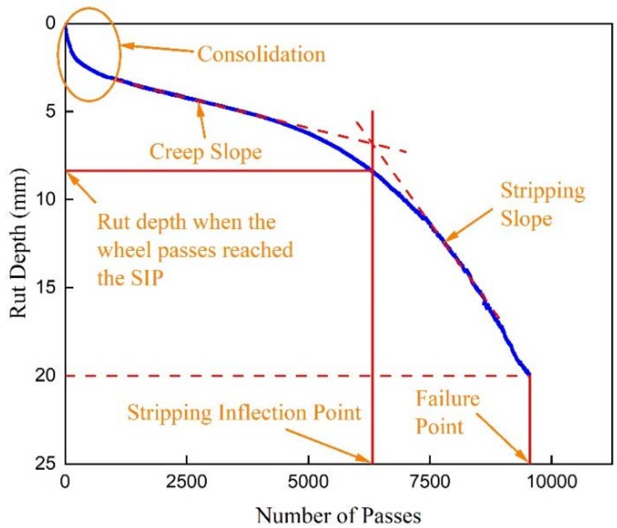


Figure 3-3. The number of passes and rut depth curve and HWTT parameters

(1) Consolidation. The rut depth accumulated during the first 1000 cycles was due to the consolidation.
(2) Creep slope (m_R). The creep slope is the first steady slope section, which reflected the rutting characteristics of the asphalt mixture without the existence of moisture damage.
(3) Stripping inflection point (SIP). The SIP was the intercept of the creep slope and stripping slope. The number of passes that SIP occurs (N_s) can be used to quantify the moisture damage of asphalt mixture. The rut depth of the sample when the wheel passes reached the SIP (t_S) can also reflect the rutting possibility of the asphalt mixture without the influence of moisture damage. The asphalt mixture with lower N_s is more susceptible to moisture damage. The asphalt mixture with lower t_S has better rutting resistance.
(4) Stripping slope (m_S). The stripping slope is the second steady slope section, which reflected the combined effect of the rutting and the moisture damage on the specimen.
(5) Failure point. The test ends after the rut depth accumulation is 20 mm, or when the number of wheel passes achieved 20,000. The number of wheel pass when the sample reached the failure point (N_f) could directly reflect the rutting and moisture damage performance of the asphalt mixture.

Different asphalt mixtures have different rutting susceptibility and moisture susceptibility during the rutting test. Using the N_f or N_s may be not suitable to compare the rutting susceptibility between different asphalt mixtures. While the ratio of N_s and N_f ($R_{stripping}$) could effectively assess the moisture damage of different asphalt mixtures, as shown in Equation 1. The $R_{stripping}$ reflected the number of wheel passing percentage before moisture damage was considered. Asphalt mixture with lower $R_{stripping}$ had better moisture damage resistance during the rutting test.

$$R_{stripping} = N_s / N_f \tag{3.1}$$

3.3 Results and analysis

3.3.1 The correlation between the number of passes and the rut depth

The rut depth-number of passes curve of the different asphalt mixtures is displayed in Figure 3-4. The 5E1 asphalt mixture without rubber, 5E1 asphalt mixture with rubber, and LVSP asphalt mixture without rubber terminated after the rutting depth reached 20 mm, and the numbers of passes are 4360, 9552, and 6372, respectively. While the rutting depth of the LVSP asphalt mixture with rubber was 7.5 mm after 20,000 wheel passes. The air voids of four types of asphalt mixtures are about 7.5%, the influence of air void on the rutting performance was not considered. The rutting resistance of the LVSP asphalt mixture was higher than that of the 5E1 asphalt mixture. The NMAS of LVSP and 5E1 asphalt mixture were 12.5 mm and 9.5 mm, respectively. The higher NMAS contributed to the improved rutting resistance of asphalt mixture. The RAP content of LVSP and 5E1 asphalt mixture were 25% and 17%, respectively. The higher RAP content in the LVSP asphalt mixture also enhanced the rutting resistance of asphalt mixture. The rutting performance was enhanced after the addition of rubber. The addition of rubber increased the stiffness and stability of asphalt mixture, thus improved the rutting resistance of asphalt mixture.

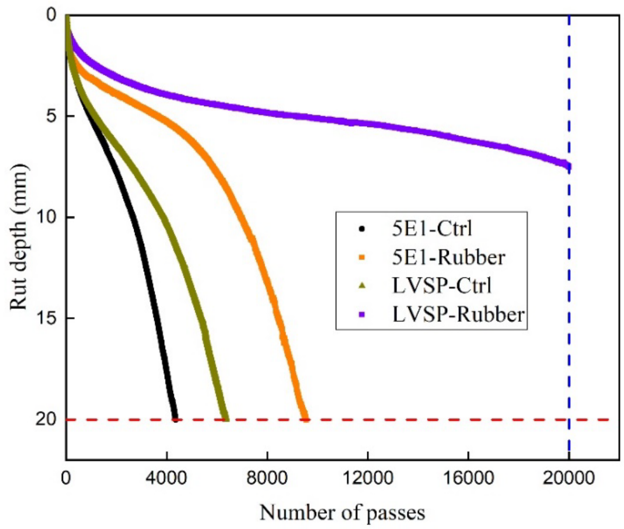


Figure 3-4. Correlation between rut depth and number of passes of different asphalt mixtures

The creep slope, stripping slope, and t_S of different asphalt mixtures are presented in Figure 3-5. The stripping point of LVSP asphalt mixture with rubber does not occur until the number of passes reached 20,000. The LVSP asphalt mixture with rubber does not have a stripping slope. The creep slope and stripping slope of 5E1 and LVSP asphalt mixture decreased after the addition of rubber. After the addition of rubber, the creep slope and stripping slope of 5E1 asphalt mixture decreased by 70.8% and 43.3%, respectively. The creep slope of the LVSP asphalt mixture decreased by 88.2% after the addition of rubber. The addition of rubber increased the stiffness of asphalt mixture, thus reduced the rutting depth increasing rate and reduced the creep slope and stripping slope. The decreasing rate of stripping slope was slower because the moisture damage significantly weakened the rutting performance of the asphalt mixture. The rutting resistance improvement of rubber to the asphalt mixture after the moisture damage occurred was influenced.

The LVSP asphalt mixture had a lower creep slope and stripping slope than the 5E1 asphalt mixture. The higher NMAS and RAP content in the LVSP asphalt mixture improved the skeleton structure and rutting resistance of asphalt mixture thus decreased the creep slope and stripping slope. But the 5E1 asphalt mixture with rubber had lower creep slope and stripping slope than LVSP asphalt mixture without rubber. The addition of rubber significantly improved the stiffness of the asphalt mixture, thus enhanced the rutting and moisture damage resistance of asphalt mixture. The stiffness improvement effect was stronger than the influence of asphalt mixture gradation and RAP addition.

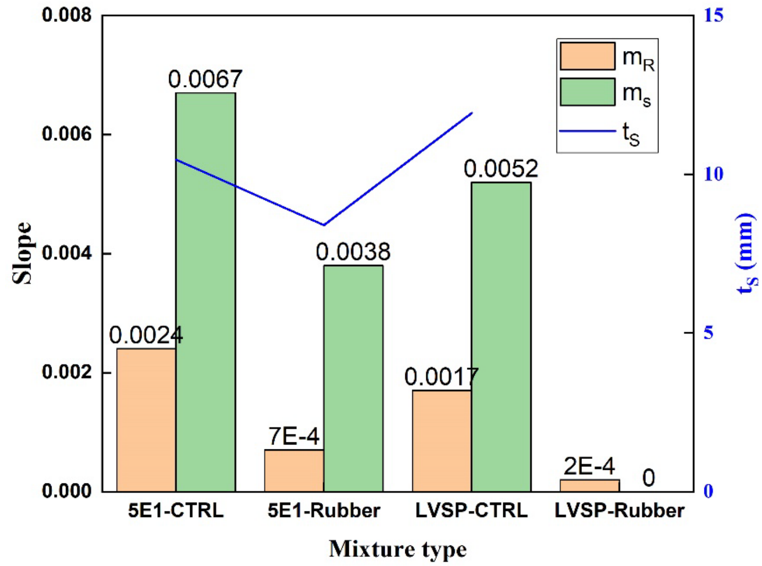


Figure 3-5. The creep slope, stripping slope and t_S of different asphalt mixtures

The N_s, N_f, and $R_{stripping}$ of different asphalt mixtures are shown in Figure 3-4. The LVSP asphalt mixture with rubber does not have a stripping point and N_s. The N_s and N_f of 5E1 asphalt mixture were increased after the addition of rubber. The N_f of LVSP asphalt mixture was increased after the addition of rubber. The addition of rubber increased the number of wheel passing needed to reach the moisture damage and fail point. The N_s is not linearly correlated to the $R_{stripping}$, because the N_f was also considered in the calculation of $R_{stripping}$. The $R_{stripping}$ increased slightly for 5E1 asphalt mixture after the addition of rubber. The moisture damage was slightly decreased, but the N_f of 5E1 asphalt mixture with rubber almost 2.2 times higher than the N_f of 5E1 asphalt mixture without rubber. The overall rutting resistance and moisture damage resistance of 5E1 asphalt mixture was increased after the addition of rubber. The LVSP asphalt mixture had weaker moisture damage resistance than the 5E1 asphalt mixture. The lower asphalt binder content and higher RAP content of LVSP asphalt mixture may be the main reasons for the weaker moisture damage resistance.

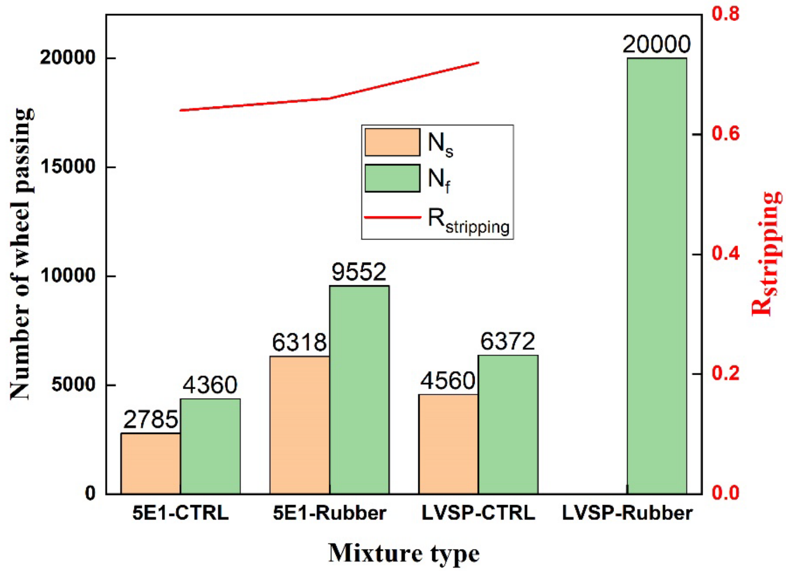


Figure 3-6. The N_S, N_F, and $R_{stripping}$ of different asphalt mixtures

3.3.2 The main factors affecting the rutting test of asphalt mixtures

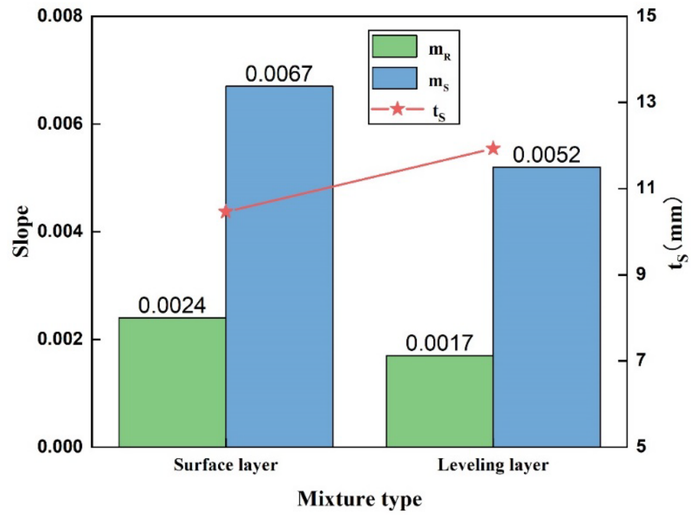
The rutting and moisture resistance of asphalt mixtures were affected by the mixture design parameters of the asphalt mixture (NMAS, RAP content, and asphalt content), rubber modification, and mixture air void content. This section will analyze the influence of various factors on the rutting and moisture performance of asphalt mixtures.

3.3.2.1 Mixture design parameters of asphalt mixture (NMAS, RAP content and asphalt content)

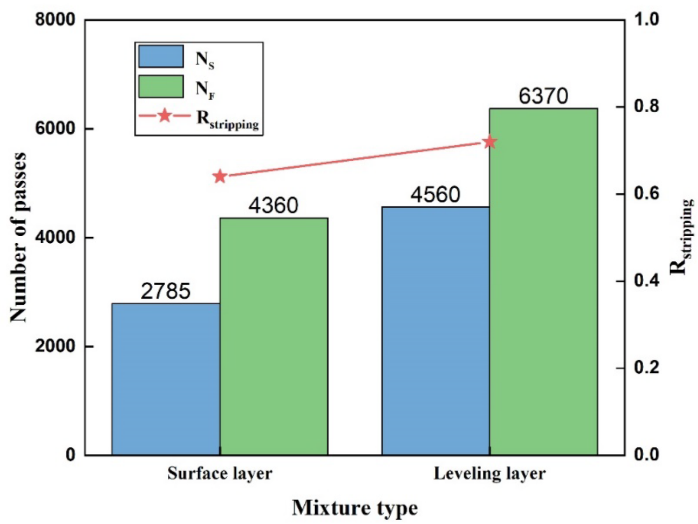
The aggregate gradation of the asphalt mixture affects the structure of the mixture, thus affects the stress response of the asphalt mixture under traffic loads. The NMAS of the surface layer asphalt mixture is 9.5 mm. The NMAS of the leveling layer asphalt mixture is 12.5 mm. The RAP content of the leveling layer asphalt mixture and the surface layer asphalt mixture is 25% and 17%, respectively. The asphalt content of the surface layer asphalt mixture and the leveling layer asphalt mixture is 5.9% and 4.2%, respectively.

The rutting parameters of the surface layer and leveling layer asphalt mixtures without rubber are shown in Figure 3-7. The average air void ratio of the surface layer asphalt mixture test samples is 7.4%, and the average air void ratio of the leveling layer asphalt mixture test samples is 7.5%. The effect of air void on the rutting and moisture susceptibility of asphalt mixtures was not considered. The surface layer asphalt mixture without rubber had a higher creep slope and stripping slope than the leveling layer asphalt mixture without rubber. The stripping slopes of the surface layer asphalt mixture and the leveling layer asphalt mixture are 2.8 times and 3.1 times that of the creep slopes, respectively (Figure 3-7 (a)). The creep slope only reflected the deformation performance of the asphalt mixture under wheel load, while the stripping slope characterized the deformation performance of the asphalt mixture under the combined effect of the wheel load and the moisture damage. Due to the moisture damage influence, the rutting resistance of the asphalt mixture was significantly reduced. The leveling layer asphalt mixture had stronger rutting and moisture damage resistance than the surface layer asphalt mixture.

The number of passes required for the leveling layer asphalt mixture to reach the stripping point is 164% of the surface layer asphalt mixture. The number of passes required for the leveling layer asphalt mixture to fail is 146% of the surface layer asphalt mixture (Figure 3-7 (b)). The rut depth is slightly higher when the leveling layer asphalt mixture reaches the SIP. The parameter $R_{stripping}$ of the leveling layer asphalt mixture is higher than that of the surface layer asphalt mixture. The NMAS of the surface layer asphalt mixture and the leveling layer asphalt mixture are 9.5 mm and 12.5 mm, respectively. The gradation design of the leveling layer asphalt mixture had a larger NMAS, thus had a stronger mixture structure and better permanent deformation resistance under load. The leveling layer asphalt mixture required more numbers of passes to the SIP and fail during the rutting test. The RAP content of the leveling layer asphalt mixture and the surface layer asphalt mixture are 25% and 17%, respectively. Since the aged asphalt in the RAP will improve the high temperature deformation resistance of the mixture, the higher RAP content in the leveling layer asphalt mixture improved the deformation resistance of the mixture. The asphalt content of the surface layer asphalt mixture and the leveling layer asphalt mixture are 5.9% and 4.2%, respectively. The higher asphalt content increased the asphalt film thickness on the surface of aggregate, and the aggregate with thicker asphalt film is more susceptible to deform during the rutting test, thus resulted in asphalt mixture with weaker rutting resistance. Compared with the surface layer asphalt mixture, the leveling layer asphalt mixture had bigger NMAS, higher RAP content, and less asphalt content. The leveling layer asphalt mixture had better rut resistance.



(a) creep slope, stripping slope and t_S



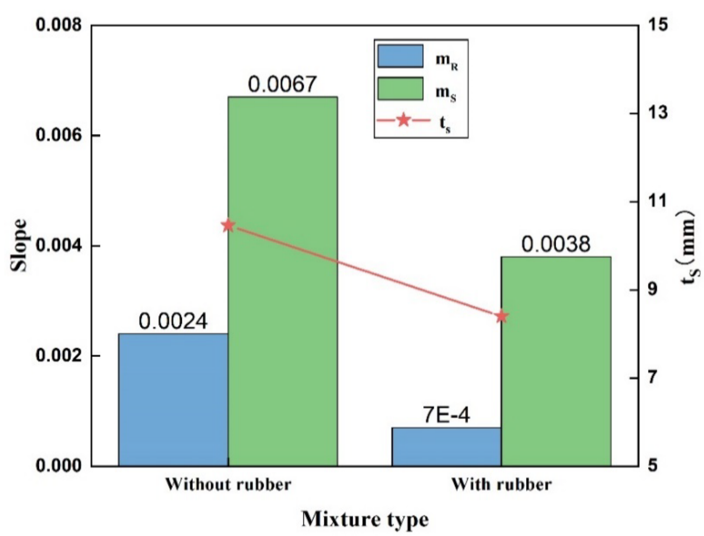
(b) N_S, N_F, and $R_{stripping}$

Figure 3-7. The rutting parameters of the surface layer and leveling layer asphalt mixtures without rubber

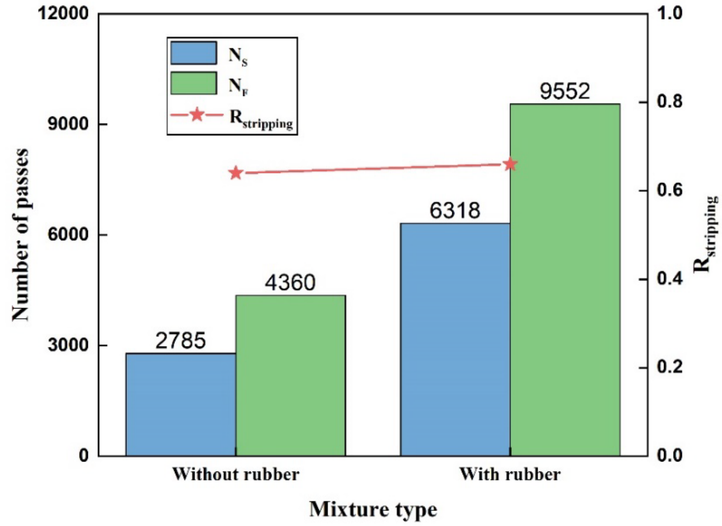
3.3.2.3 Rubber modification

The rutting parameters of the surface layer asphalt mixture with and without rubber are shown in Figure 3-8. The average air void of the surface layer asphalt mixture test specimens without rubber modification is 7.4%, and the average void ratio of the surface layer asphalt mixture test specimens with rubber modification is 7.5%. The influence of air void on the rutting and moisture susceptibility of asphalt mixtures was not considered. The rubber modification could significantly improve the rutting resistance of the surface layer asphalt mixture. After the rubber modification, the creep slope and stripping slope of the surface layer asphalt mixture were reduced to 29% and 57% that of asphalt mixture without rubber, respectively. The stripping slope of the surface layer asphalt mixture with and without rubber modification are 2.8 times and 5.4 times of the creep slope, respectively (Figure 3-8 (a)). The rut depth increasing rate of the surface layer asphalt mixture with rubber increased significantly after moisture damage. This is because the rubber modification greatly reduced the creep slope and thus increased the ratio of the stripping slope and the creep slope. The stripping slope of asphalt mixture with rubber was still lower than that of the asphalt mixture without rubber. The t_S of the asphalt mixture with rubber was slightly smaller than that of the asphalt mixture without rubber. This is because the creep slope of the surface layer asphalt mixture with rubber was reduced, and the rut depth at which moisture damage occurred was reduced.

The number of passes required for the asphalt mixture without rubber to reach the SIP was 2785. The rutting and moisture perfromance of the asphalt mixture with rubber are significantly improved, and the number of passes required to reach the SIP was increased to 6318 (Figure 3-8 (b)). The number of passes required for surface layer asphalt mixture with rubber to failure was 2.2 times than that of the surface layer asphalt mixture with rubber. The $R_{stripping}$ of the surface layer asphalt mixture with rubber was similar to that of the surface layer asphalt mixture without rubber. The rubber modification significantly improved the rutting and moisture damage resistance of the surface layer asphalt mixture.



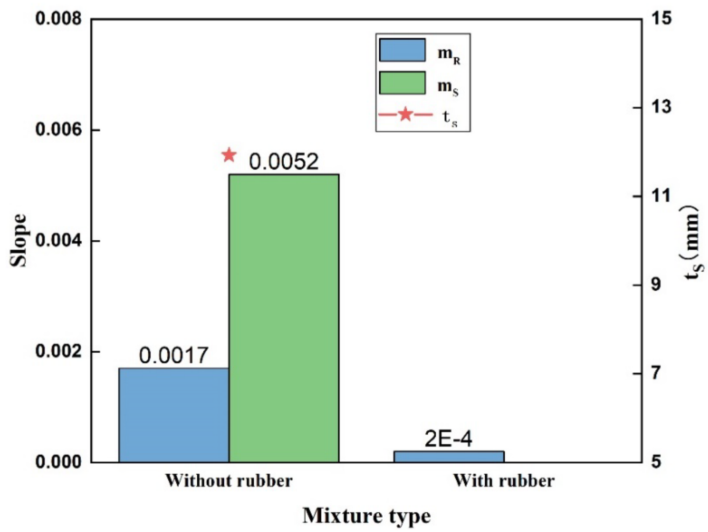
(a) creep slope, stripping slope and t_S



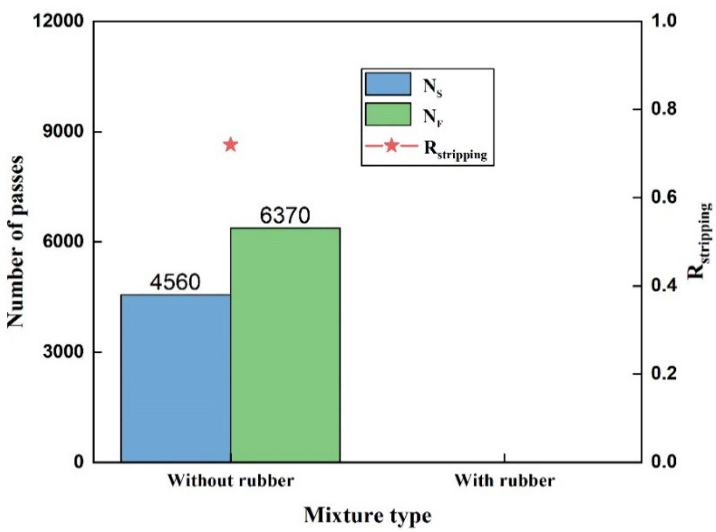
(b) N_S, N_F, and $R_{stripping}$

Figure 3-8. The rutting parameters of the surface layer asphalt mixtures with and without rubber

The rutting parameters of the leveling layer asphalt mixture with and without rubber are shown in Figure 3-9. The average air void of the leveling layer asphalt mixture test specimens without rubber modification is 7.5%, and the average void ratio of the leveling layer asphalt mixture test specimens with rubber modification is 7.4%. The influence of air void on the rutting and moisture susceptibility of asphalt mixtures was not considered. The rut depth of the leveling layer asphalt mixture without rubber reached 20 mm when the number of passes reached 6370. The rutting test of the leveling layer asphalt mixture with rubber terminated when the number of passes reached 20,000, and the rut depth was 7.48 mm (Figure 3-4). The leveling layer asphalt mixture did not reach the SIP until the end of the test, and there was no stripping slope and failure point. The creep slope of the leveling layer asphalt mixture without rubber was 0.0017. The creep slope reduced to only 12% after the asphalt mixture was modified with rubber (Figure 3-9). By comparing the rutting test results of the leveling layer and the surface layer asphalt mixture with and without rubber, the rubber modification significantly improved the rutting and moisture damage resistance of the asphalt mixture.



(a) creep slope, stripping slope and t_S



(b) N_S, N_F, and $R_{stripping}$

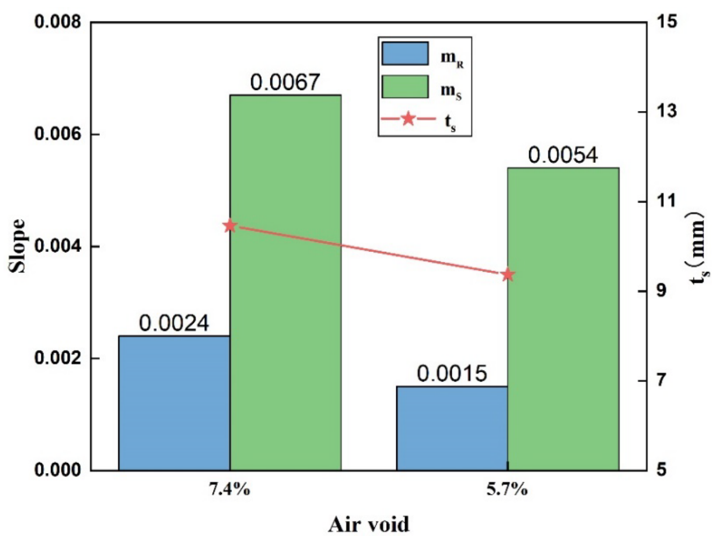
Figure 3-9. The rutting parameters of the leveling layer asphalt mixtures with and without rubber

3.3.2.3 Mixture air void content

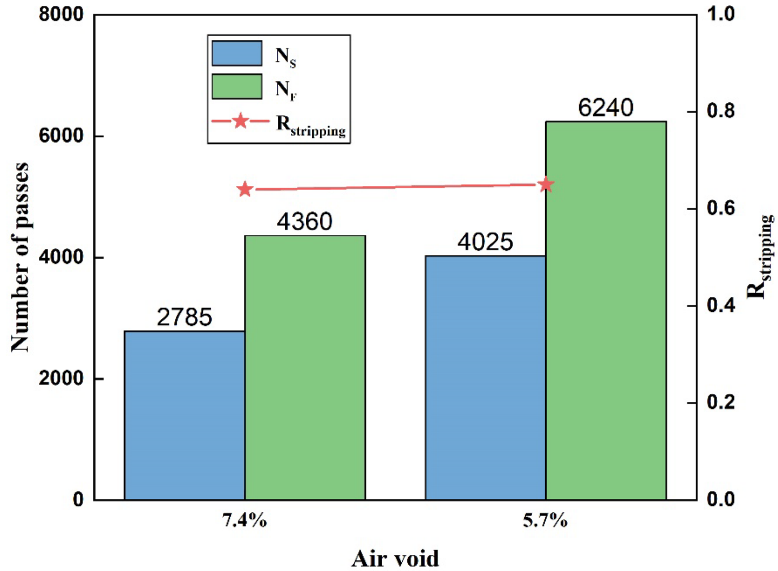
Based on the AASHTO T324, the air void of the laboratory prepared samples for the HWTT should be 7.0 ± 1.0%. In order to evaluate the influence of air void content on the rutting and moisture perfromance, samples out of the air void suggestion range were prepared. The influence of air void content on rutting parameters of the surface layer asphalt mixture without rubber is shown in Figure 3-10.

The air void contents of the surface layer asphalt mixture without rubber test specimens were 7.4% and 5.7%. When the air void reduced from 7.4% to 5.7%, the creep slope and stripping slope of the surface layer asphalt mixture without rubber reduced 38% and 19%, respectively (Figure 3-10 (a)). As the air void of the test specimen decreased, the rutting resistance of the surface layer asphalt mixture without rubber improved significantly. The stripping slopes of the 7.4% and 5.7% surface layer asphalt mixture without rubber were 2.8 and 3.6 times the creep slope, respectively. After reducing the air void, the t_S of the sample was reduced. This is because the creep slope of the 5.7% air void surface layer asphalt mixture without rubber was reduced, and the rutting resistance of the sample was improved.

The number of passes required to reach the SIP of the 7.4% air void surface layer asphalt mixture without rubber was 2785 (Figure 3-10 (b)). The rutting resistance of the surface layer asphalt mixture without rubber was improved significantly after the air void was reduced. The number of passes required to reach the SIP increased to 4025 after decreasing the air void. The N_S and N_F of the 5.7% air void surface layer asphalt mixture without rubber was about 1.4 times than that of the 7.4% air void surface layer asphalt mixture without rubber. The $R_{stripping}$ of the two air void contents surface layer asphalt mixture without rubber was similar. When the air void of the surface layer asphalt mixture without rubber reduced from 7.4% to 5.7%, the rutting and moisture damage resistance of the asphalt mixture improved about 1.4 times. Reducing the air void increased the density of the asphalt mixture, increased the stiffness of the asphalt mixture, thus improved the rutting and moisture damage resistance of the asphalt mixture.



(a) creep slope, stripping slope and t_S



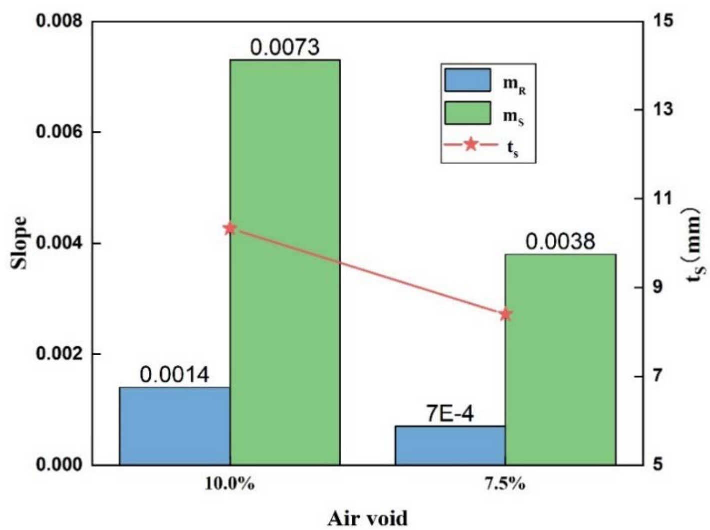
(b) N_S, N_F, and $R_{stripping}$

Figure 3-10. The influence of air void content on rutting parameters of the surface layer asphalt mixture without rubber

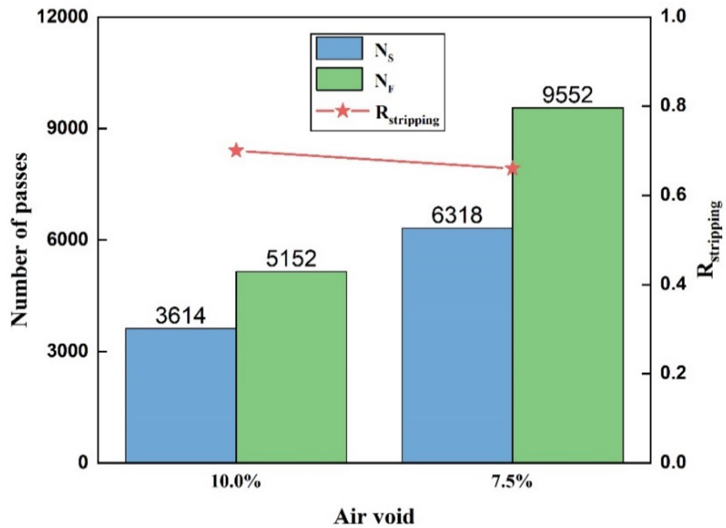
The influence of air void content on rutting parameters of the surface layer asphalt mixture with rubber is shown in Figure 3-11. The air void contents of the surface layer asphalt mixture with rubber test specimens were 7.5% and 10.0%. When the air void increased from 7.5% to 10.0%, the creep slope and stripping slope of the surface layer asphalt mixture with rubber increased 100% and 92%, respectively (Figure 3-11 (a)). As the air void of the test specimen increased, the rutting resistance of the surface layer asphalt mixture with rubber reduced significantly. As the air void of the samples increased, water was easier to penetrate into the asphalt mixture during the rutting test. Under the effect of the wheel load, the water pressure weakened the bond between the asphalt and the aggregate and significantly increased the stripping slope. The stripping slopes of the 7.5% and 10.0% air void surface layer asphalt mixture with rubber were 5.4 times and 5.2 times of the creep slopes, respectively. After increasing the air void, the t_S of the sample was increased. This is because the creep slope of the 10.0% air void surface layer asphalt mixture with rubber was increased, and the rutting resistance of the sample was decreased.

The N_s the 7.5% void ratio surface layer asphalt mixture with rubber was 6,318. The rutting resistance of the surface layer asphalt mixture with rubber was weakened significantly after the air void was increased. The number of passes required to reach the SIP decreased 43% after increasing the air void. The N_S and N_F of the 10.0% air void surface layer asphalt mixture with rubber was about 57% that of the 7.5% air void surface layer asphalt mixture with rubber. The $R_{stripping}$ of the two air void contents surface layer asphalt mixture with rubber was similar. When the air void of the surface layer asphalt mixture with rubber increased from 7.5% to 10.0%, the rutting and moisture damage resistance of the asphalt mixture decreased by about 50%.

Increasing the air void of the asphalt mixture could reduce the stiffness of the asphalt mixture. After the moisture damage occurs, the larger void ratio increases the possibility of moisture entering the asphalt mixture and weakens the adhesion between the asphalt and the aggregate. During the construction of the project, controlling the compaction density of the asphalt mixture and reducing the void ratio of the asphalt mixture could effectively improve the rutting and moisture damage resistance of the asphalt mixture.



(a) creep slope, stripping slope and t_S



(b) N_S, N_F, and $R_{stripping}$

Figure 3-11. The influence of air void content on rutting parameters of the surface layer asphalt mixture with rubber

3.4 Conclusion

This chapter analyzed the high temperature rutting performance of different asphalt mixtures. The influence of mixture design factor, rubber addition, and air void were compared, and the analytical results were proposed. The main conclusions are as follows:

(1) Compared with the surface layer asphalt mixture, the leveling layer asphalt mixture had bigger NMAS, higher RAP content, and less asphalt content. The leveling layer asphalt mixture had better rut resistance. The creep slope, number of passes to SIP and failure were significantly influenced by the mixture design factor.

(2) The rubber modification significantly improved the rutting and moisture damage resistance of the asphalt mixture. The creep slope, stripping slope, and the number of passes to SIP and fail were significantly influenced by the rubber modification.

(3) Reducing the air void increased the density of the asphalt mixture, increased the stiffness of the asphalt mixture, thus improved the rutting and moisture damage resistance of the asphalt mixture. The number of passes to SIP and fail were significantly influenced by the mixture air void.

CHAPTER 4: LOW TEMPERATURE CRACKING PERFORMANCE EVALUATION OF RUBBERIZED ASPHALT MIXTURE

4.1 Introduction

Low temperature cracking is the main asphalt mixture distress that occurs in wet-freeze regions. The cracking caused by low temperature accelerates the distress of the asphalt mixture pavement and reduces the surface life of the pavement [14]. Therefore, the evaluation of the low-temperature cracking performance of asphalt mixtures is particularly important for the pavement applied in the wet-freeze regions [15].

In the past few decades, different test methods have been adopted to study the low temperature cracking performance of asphalt mixture and to choose materials that have adequate low temperature cracking resistance. The indirect tensile test, which was similar to the Brazilian test, was proposed by the SHRP project to evaluate the creep compliance and low temperature performance of asphalt mixtures [16]. The semi-circular bend (SCB) and the disc-shaped compact tension (DCT) have been used by researchers to evaluate the low temperature crack resistance of asphalt mixtures [17, 18]. The fracture energy of SCB samples should be greater than 350 J/m^2, and the fracture toughness should be higher than kPa·m$^{1/2}$[19]. The DCT test had standard restrictions based on different traffic levels, the fracture energy of asphalt mixture used in low volume road should be greater than 400 J/m^2, while the DCT test fracture energy of asphalt mixture under heavy traffic conditions should be greater than 690 J/m^2 [20].

4.2 Test materials and test method

4.2.1 Test materials and test sample preparation

Chapter 1 describes the volumetric mixture design of rubberized asphalt mixture with the dry process. The leveling layer and the surface layer use different mixture gradation and asphalt binder content. In order to compare the rutting and moisture susceptibility of rubberized asphalt mixture with the dry process, the leveling layer and the surface layer asphalt mixture with and without rubber were prepared. The gradation and asphalt binder content of asphalt mixture with and without rubber were the same. Totally four types asphalt mixtures were prepared, which included surface layer (5E1) asphalt mixture with rubber, surface layer (5E1) asphalt mixture without rubber, leveling layer (LVSP) asphalt mixture with rubber, and leveling layer (LVSP) asphalt mixture without rubber.

The specimens were prepared with the Superpave gyratory compactor (SGC). The diameter and height of the sample were 150 mm and 120 mm, respectively (Figure 4-1 (a)).

(a) SGC sample

(b) Two 50 mm thickness sample

(c) Final DCT test sample

Figure 4-1. Sample preparation procedure of DCT test

During the compaction of the SGC specimen, the surface of the test specimen was not flat. The upper and lower surfaces of the asphalt mixture prepared by the SGC were cut by the saw to reduce the influence on the test results. Finally, two specimens with a diameter of 150 mm and a thickness of 50 ± 5 mm were obtained (Figure 4-1 (b)). Two cores with a diameter of 25 mm were drilled symmetrically on the test piece, and the initial ligament length of 62.5 ± 2.5 mm was cut along the central line of the specimen (Figure 4-1 (c)). The dimensions of the standard test specimen for the DCT test are shown in Figure 4-2 and Table 4-1. The air void content of the laboratory compacted specimen was 7.0 ± 1.0%.

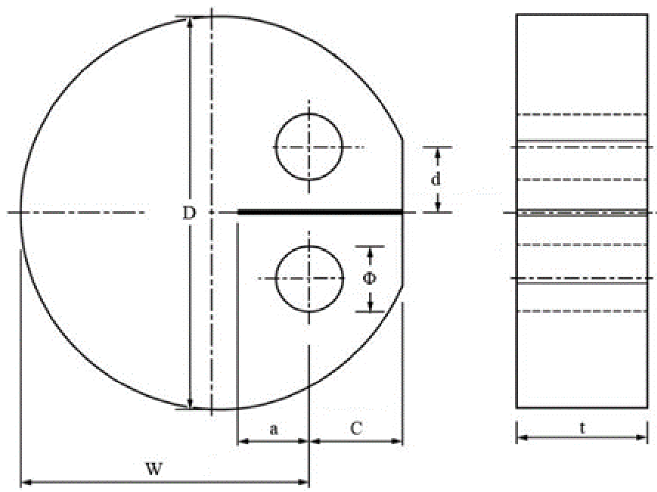


Figure 4-2. The geometrical dimensions of the DCT test specimen

Table 4-1.The value of different dimensions of the DCT test specimen

Dimensions	Value (mm)
D	150
W	110
a	27.5
C	35
d	25
Φ	25
t	50

4.2.2 Test method

4.3.2.1 Test conditions

In this study, the DCT test equipment produced by TestQuip LLC was adopted to assess the low-temperature cracking property of the asphalt mixture, as shown in Figure 4-3. The DCT test could be applied to quantify the cracking resistance of the asphalt mixture, and the influence of cracking on the service life could be assessed. The test temperature included -24 ºC and -18 ºC. The test was performed with a constant crack mouth opening displacement (CMOD) rate of 1 mm/min. The test was stopped after the load returned to 0.1 kN. The CMOD after the peak load reflected the propagation of cracking inside the sample during the DCT test. The creep characteristics of the asphalt mixture are negligible at this test loading condition.

Figure 4-3. DCT test equipment produced by the TestQuip LLC

4.2.2.2 Characterization parameters of the test results

The load-CMOD curve of the DCT test result can be obtained, as shown in Figure 4-4. The area under the curve was the work needed to break the sample until total failure (W_f). The W_f can be divided into the pre-peak cracking work ($W_f^{Pre\text{-}peak}$) and the post-peak cracking work ($W_f^{Post\text{-}peak}$). The cracking work can be calculated using the quadrangle rule, as shown in Equation 4.1.

$$W_f = W_f^{Pre-peak} + W_f^{Post-peak} \tag{4.1}$$

$$= \int_0^{\Delta Fmax} F \cdot dx + \int_{\Delta Fmax}^{\Delta Final} F \cdot dx$$

Where:

W_f is the cracking work, which is the area under the load-CMOD curve (mm-kN),

x is the CMOD (mm),

F is the load during the DCT test (kN),

ΔF_{max} is the CMOD value at the peak load (mm),

ΔFinal is the maximum CMOD value of the test sample (mm).

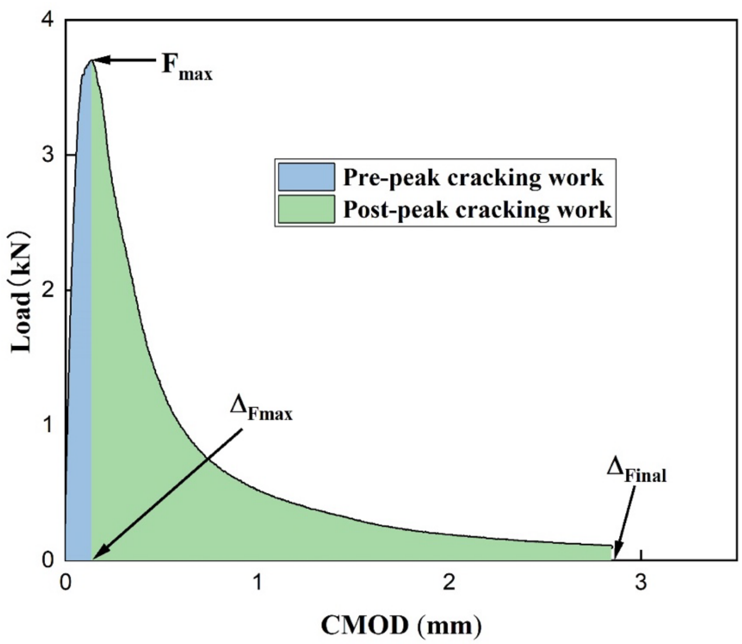


Figure 4-4. The load-CMOD curve and the cracking work

The Fracture energy (G_f) can be computed with Equation 4.2.

$$G_f = \frac{W_f}{t \times (W - a)} \tag{4.2}$$

Where:

G_f is the fracture energy (J/m^2),

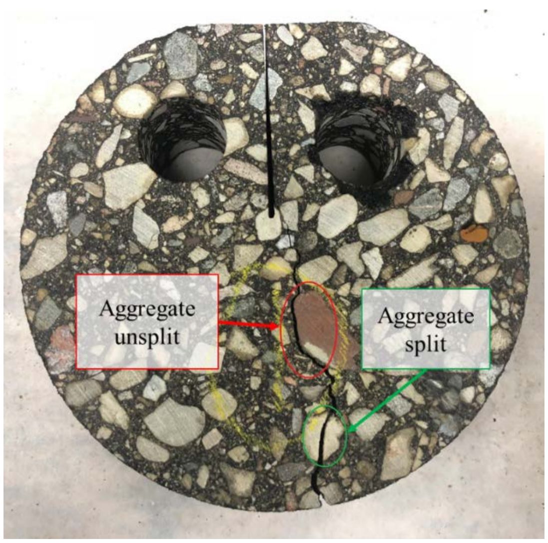
t is the thickness of the specimen (mm),

W-a is the initial ligament length (mm).

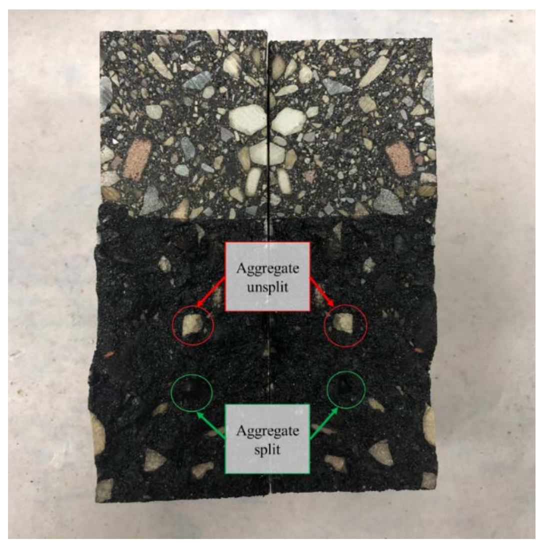
4.3 Results and analysis

4.3.1 The low temperature cracking characterization

The cracking path of the DCT test specimen from the surface view and inside the sample is shown in Figure 4-5. The split aggregate and unsplit aggregate can be seen on the surface of the test specimen and inside the test specimen. The cracking path didn't follow the shortest failure path, and the cracking path was affected by the aggregate distribution and the stiffness of the aggregate inside the mixture. In the computation of fracture energy in Equation 4.2, the hypothesis of the least cracking area was proposed. In the fracture energy validation in the lab, the results were influenced by many factors. One critical factor that should be noticed during the mixture production is the mixture segregation. What's more, the aggregate distribution was also influenced by the mixture segregation. Quality control of the mixture production in the lab and during the field application is critical to guarantee the low temperature cracking property consistency.



(a) The surface view of the cracking path



(b) The cracking path inside the sample

Figure 4-5. The cracking path of the DCT test sample

The relationship between DCT test load and CMOD of different types of asphalt mixtures at -24 ºC is shown in Figure 4-6. By comparing CMODs of different types of asphalt mixtures, it is found that the maximum deformation of the leveling layer asphalt mixture was smaller than that of the surface layer asphalt mixture. The NMAS of the leveling layer asphalt mixture was bigger, and the RAP content of the leveling layer asphalt mixture was higher, and the asphalt binder content of the leveling layer asphalt mixture was lower. Thus, the elastic performance of the leveling layer asphalt mixture was lower than that of the surface layer asphalt mixture. The leveling layer asphalt mixture had weaker deformation resistance during the DCT test, so the maximum deformation of the leveling layer asphalt mixture was smaller at the end of the DCT test. By comparing the same type of asphalt mixture with and without rubber, adding rubber could significantly increase the maximum deformation of the asphalt mixture when the specimen failed. Adding rubber to the asphalt mixture could significantly improve the elastic properties of the asphalt mixture, and enhance the elastic deformation property of the asphalt mixture during the DCT test. Therefore, the maximum deformation of the asphalt mixture with rubber at the end of the test was bigger.

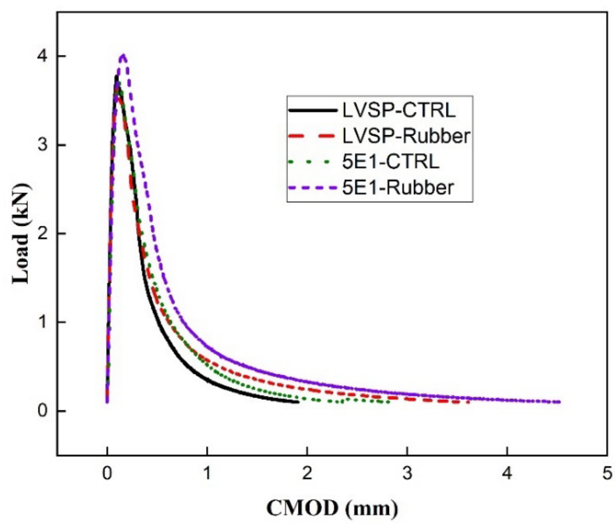


Figure 4-6. The relationship between load and CMOD of different types of asphalt mixtures

4.3.2 The factors affected the DCT test of the asphalt mixture

The low temperature cracking resistance of asphalt mixtures were affected by the mixture design factors of the asphalt mixture (NMAS, RAP content, and asphalt content), rubber modification, and test temperature. The air voids of different types of asphalt mixtures were about 7.0%, The influence of air void on the low temperature cracking performance of asphalt mixtures was not considered. This section will analyze the influence of different factors on the low temperature cracking property of asphalt mixtures.

4.3.2.1 Mixture design factors of asphalt mixture (NMAS, RAP content and asphalt content)

The NMAS, RAP content, and the asphalt content of the surface layer and the leveling layer asphalt mixtures were different. The NMAS of the surface layer asphalt mixture and the leveling layer asphalt mixture are 9.5 mm and 12.5 mm, respectively. The RAP content of the leveling layer asphalt mixture and the surface layer asphalt mixture is 25% and 17%, respectively. The asphalt content of the surface layer asphalt mixture and the leveling layer asphalt mixture is 5.9% and 4.2%, respectively.

The fracture energy at the test temperature of -24ºC is shown in Figure 4-7. Fracture energy could directly reflect the low temperature cracking properties of asphalt mixtures. The fracture energy of the surface layer asphalt mixture was 120% of the fracture energy of the leveling layer asphalt mixture when there was no rubber in the asphalt mixture. The fracture energy of the surface layer asphalt mixture was 108% of the fracture energy of the leveling layer asphalt mixture when the asphalt mixtures had rubber. The fracture energy of the surface layer asphalt mixture was significantly higher than that of the leveling layer asphalt mixture. The surface layer asphalt mixture had better low temperature cracking resistance than the leveling layer asphalt mixture. Rubber modification reduced the fracture energy differences between different types of asphalt mixtures. The subsequent section will display the detailed influence of rubber modification on the low temperature cracking performance of asphalt mixtures. The NMAS of the surface layer asphalt mixture and the leveling layer asphalt mixture are 9.5 mm and 12.5 mm, respectively. The specific surface area of the surface layer asphalt mixture was larger than that of the leveling layer asphalt mixture. The bonding performance between asphalt and aggregate in the surface layer asphalt mixture was better. From the perspective of NMAS of asphalt mixture, the surface layer asphalt mixture had better low temperature cracking performance. The RAP content of the leveling layer asphalt mixture and the surface layer asphalt mixture is 25% and 17%, respectively. The aged asphalt binder in the RAP increased the low temperature brittleness of the mixture, so the asphalt mixture with higher RAP was more prone to crack at low temperatures. From the perspective of the RAP content in the asphalt mixture, the low temperature cracking resistance of the surface layer asphalt mixture was better. The asphalt content of the surface layer asphalt mixture and the leveling layer asphalt mixture is 5.9% and 4.2%, respectively. The higher asphalt content increased the asphalt film thickness on the surface of aggregate, and the adhesion between the aggregate and asphalt was stronger. From the perspective of the asphalt content in the asphalt mixture, the low temperature cracking resistance of the surface layer asphalt mixture was better. Compared with the leveling layer asphalt mixture, the smaller NMAS, lower RAP content, and higher asphalt content contributed to the better low temperature cracking performance of the surface layer asphalt mixture.

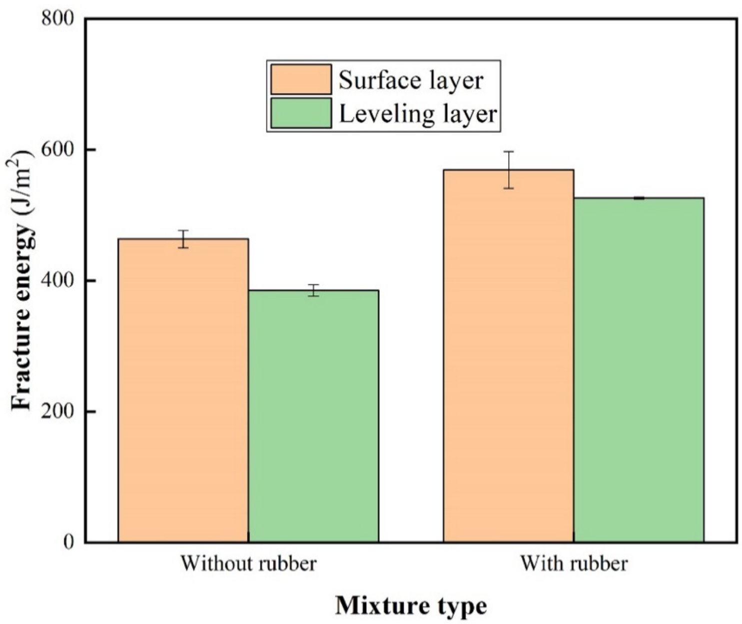


Figure 4-7. The influence of mixture design on fracture energy

The peak load of different asphalt mixtures at the test temperature of -24ºC is shown in Figure 4-8. The peak load of the surface layer asphalt mixture was 110% of the peak load of the leveling layer asphalt mixture when there was no rubber in the asphalt mixture. The peak load of the surface layer asphalt mixture was 102% of the peak load of the leveling layer asphalt mixture when the asphalt mixtures had rubber. The peak load differences between different asphalt mixtures were smaller compared with the fracture energy differences between different asphalt mixtures. The peak load differences between different asphalt mixtures were minimal when the asphalt mixture was modified with rubber. The peak load was not only affected by the mixture design factors of the asphalt mixture (NMAS, RAP content, and the asphalt content) but also influenced by the distribution of the aggregate in the asphalt mixture.

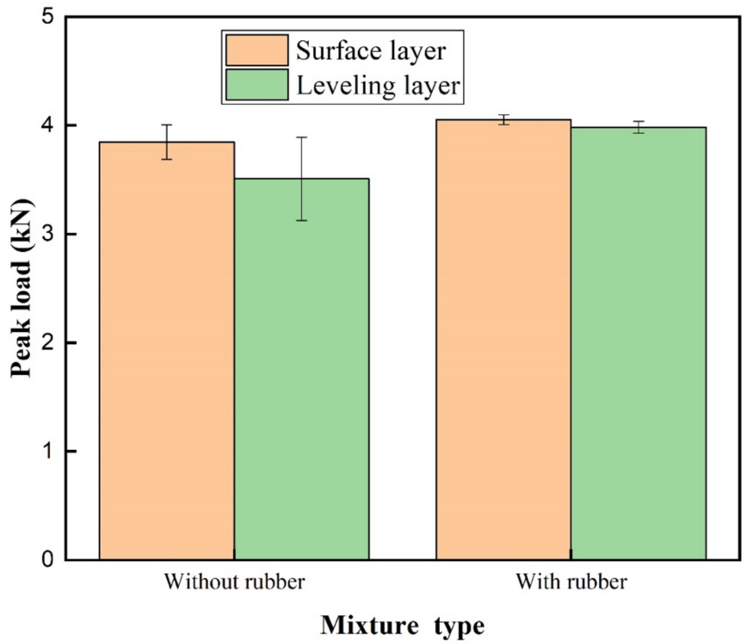


Figure 4-8. The influence of mixture design on peak load

The maximum CMOD of different asphalt mixtures at the test temperature of -24ºC is shown in Figure 4-9. The maximum CMOD could indirectly characterize the low temperature cracking resistance of the asphalt mixture. Asphalt mixture had higher low temperature cracking resistance if Asphalt mixture had higher maximum CMOD when the test specimen failed. The maximum CMOD of the surface layer asphalt mixture was 148% of the maximum CMOD of the leveling layer asphalt mixture when there was no rubber in the asphalt mixture. The maximum CMOD of the surface layer asphalt mixture was 117% of the maximum CMOD of the leveling layer asphalt mixture when the asphalt mixtures had rubber. The maximum CMOD of the surface layer asphalt mixture was significantly higher than that of the leveling layer asphalt mixture. The surface layer asphalt mixture had better low temperature cracking resistance than the leveling layer asphalt mixture. Rubber modification reduced the maximum CMOD differences between different types of asphalt mixtures.

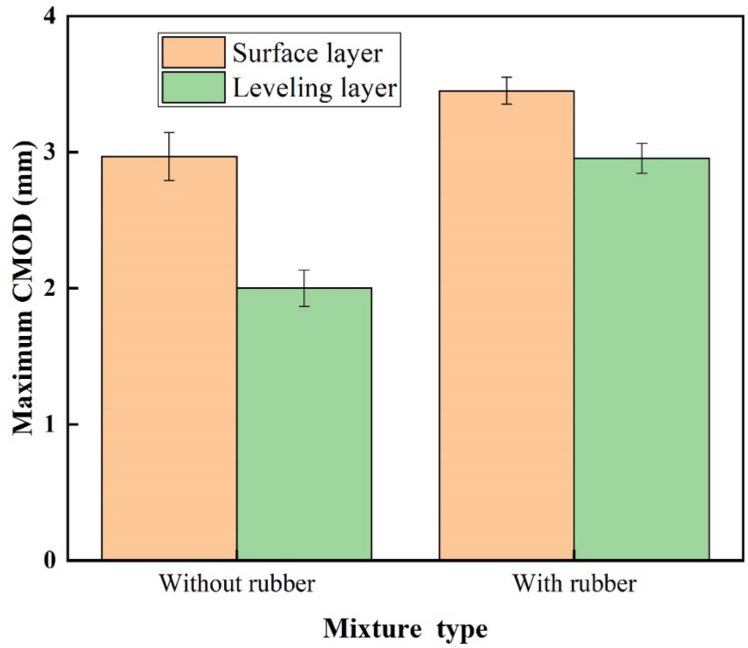


Figure 4-9. The influence of mixture design on maximum CMOD

4.3.2.2 Test temperature

The fracture energy of the surface layer and the leveling layer asphalt mixture without rubber under different test temperatures is shown in Figure 4-10. The fracture energy of test specimens increased with the increase of the test temperature. The fracture energy of the surface layer increased by 12.9% when the test temperature increased from -24 ºC to -18 ºC. The fracture energy of the leveling layer increased by 19.7% when the test temperature increased from -24 ºC to -18 ºC. In the previous section, the influence of mixture design on the fracture energy was analyzed, and the surface layer asphalt mixture had better low temperature cracking property than the leveling layer asphalt mixture. The fracture energy of the surface layer at -24 ºC was still higher than the average fracture energy of the leveling layer at -18 ºC. The asphalt mixtures exhibited more significant low temperature brittleness when the test temperature decreased, and the low temperature cracking resistance of the asphalt mixture decreased.

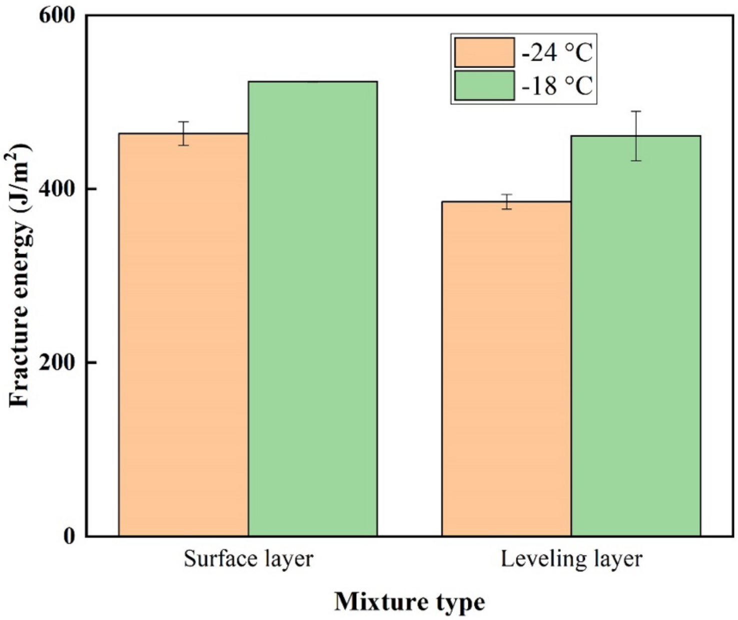


Figure 4-10. The influence of test temperature on fracture energy

The peak load of the surface layer and the leveling layer asphalt mixture without rubber under different test temperatures is shown in Figure 4-11. The peak load of test specimens decreased with the increase of the test temperature. The low temperature brittleness at low test temperature increased the load needed to break the test specimen, thus increased the peak load with the decrease of the test temperature. The average peak load of the surface layer asphalt mixture decreased by 13.5% when the test temperature increased from -24 ºC to -18 ºC. The average peak load of the leveling layer asphalt mixture decreased by 4.7% when the test temperature increased from -24 ºC to -18 ºC. The asphalt binder played an important role in the low-temperature performance of the asphalt mixture. The low temperature brittleness change of the asphalt mixture was significantly influenced by the asphalt binder in the asphalt mixture. The asphalt binder content of the surface layer asphalt mixture was higher. The surface layer asphalt mixture was more sensitive to the test temperature change than the leveling layer asphalt mixture. Therefore, the average peak load of the surface layer asphalt mixture changed more when the test temperature changed.

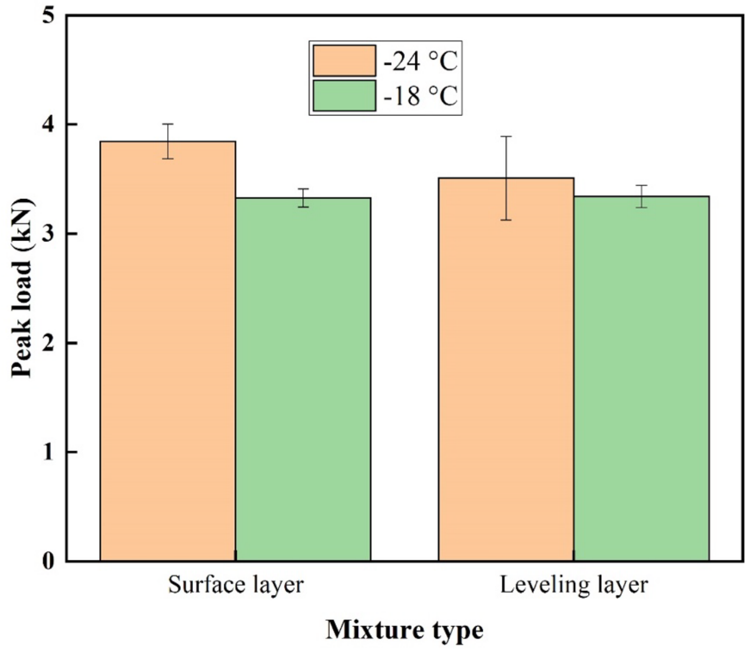


Figure 4-11. The influence of test temperature on peak load

The maximum CMOD of the surface layer and the leveling layer asphalt mixture without rubber under different test temperatures is shown in Figure 4-12. The maximum CMOD of test specimens increased with the increase of the test temperature. The average maximum CMOD of the surface layer asphalt mixture increased by 4.9% when the test temperature increased from -24 ºC to -18 ºC. The average maximum CMOD of the leveling layer asphalt mixture increased by 23.1% when the test temperature increased from -24 ºC to -18 ºC. With the increase of test temperature, the brittleness of the asphalt mixture decreased, and the elastic property of the asphalt mixture increased, and thus increased the maximum CMOD of asphalt mixture.

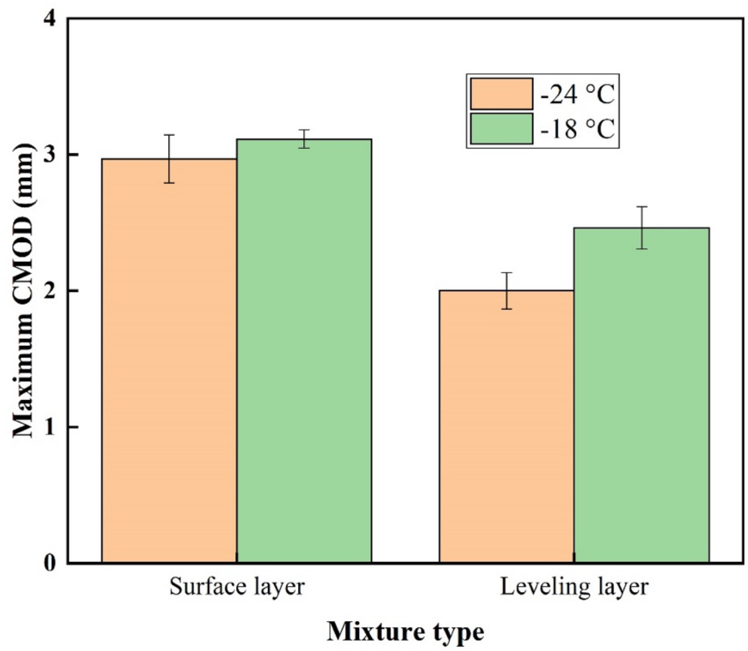


Figure 4-12. The influence of test temperature on maximum CMOD

4.3.2.3 Rubber modification

The effect of the rubber modification on the low-temperature fracture energy of the DCT test is shown in Figure 4-13. The low temperature fracture energy of asphalt mixtures was increased after rubber modification. The average fracture energy of the surface layer asphalt mixture increased to at least 18.4% higher after rubber modification. The fracture energy of the leveling layer asphalt mixture increased to at least 22.9% higher after rubber modification. The fracture energy of the surface layer without rubber was bigger, that's why the increasing rate was smaller after rubber modification. The rubber modification in the asphalt mixture increased the thickness of the asphalt film on the surface of the aggregate and improved the adhesion between the asphalt and the aggregate, and thus enhanced the low temperature cracking resistance of the asphalt mixture. The fracture energy differences between different mixture types were reduced after rubber modification. During the rubber modification with the dry process, the rubber particle that was not interacted with asphalt binder in the mixture filled the air void in the mixture as fine aggregate. The structure of the asphalt mixture was improved, and the fracture energy differences between different mixture designs were reduced. The fracture energy improvement effect due to rubber modification at the test temperature of -24 ºC was better than that at -18 ºC. The rubber modification improvement on the low temperature cracking performance of asphalt mixture at a lower temperature was better, which proved the effectiveness of rubber modification in improving the low temperature cracking performance of asphalt mixture.

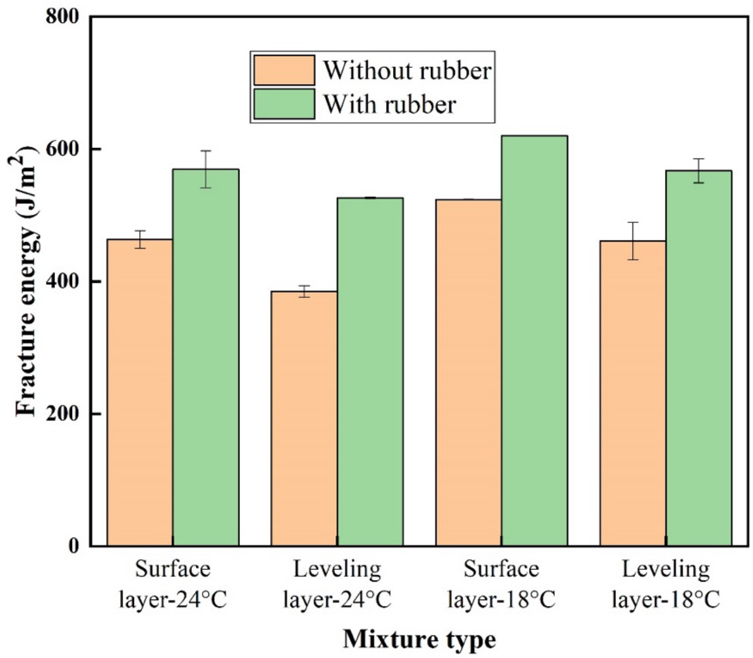


Figure 4-13. The influence of rubber modification on fracture energy

The effect of rubber modification on the peak load of different DCT test specimens is shown in Figure 4-14. The peak load of the asphalt mixtures increased slightly after adding rubber. At -24 ºC, the average peak load of the surface layer and the leveling layer asphalt mixture increased 5.3% and 13.6%, respectively. At -18 ºC, the average peak load of the surface layer and the leveling layer asphalt mixture increased 7.2% and 4.9%, respectively. The variance between parallel specimens of asphalt mixture without rubber was bigger. The peak load was influenced by the structure of the asphalt mixture, and the rubber modification increased the structural uniformity of the asphalt mixture.

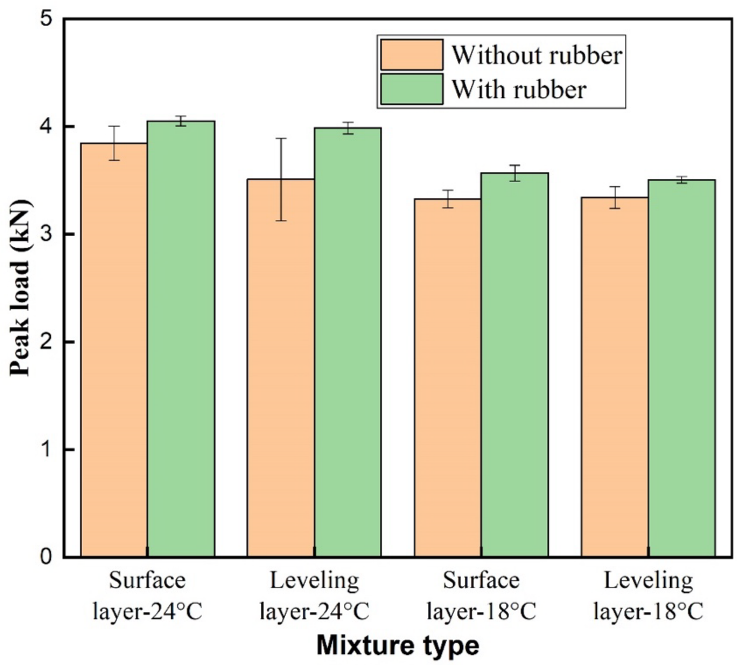


Figure 4-14. The influence of rubber modification on peak load

The effect of the rubber modification on the maximum CMOD of the DCT test is shown in Figure 4-15. The maximum CMOD of asphalt mixtures was increased significantly after rubber modification. At the test temperature of -24 ºC, the maximum CMOD of the surface layer and the leveling layer asphalt mixture increased 16.3% and 47.7%, respectively. At the test temperature of -18 ºC, the maximum CMOD of the surface layer and the leveling layer asphalt mixture increased 23.5% and 23.7%, respectively. The elastic property of the leveling layer without rubber was weak, and rubber modification dramatically improved the elastic property and the deformation performance of the asphalt mixture at low temperatures.

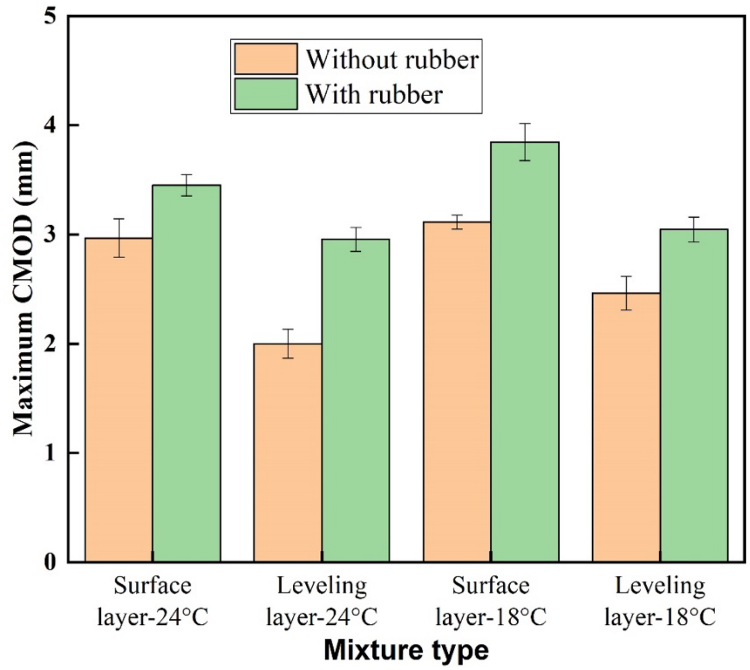


Figure 4-15. The influence of rubber modification on maximum CMOD

4.3.3 The correlation between the fracture energy and the maximum CMOD

The correlation between the fracture energy and the maximum CMOD of different asphalt mixtures under different test conditions is shown in Figure 4-19. There was a very significant linear correlation between the fracture energy and the maximum CMOD ($p < 0.0001$), where the coefficient of determination $R^2 = 0.7987$. For different asphalt mixture types under different test temperatures, the fracture energy of the asphalt mixture was mainly affected by the maximum CMOD. Because the value of the maximum CMOD determined the range of the area under the load-CMOD curve.

The previous section proved that rubber modification increased the elastic property of the asphalt mixture. The maximum CMOD, fracture energy, and the low temperature cracking performance of the asphalt mixture were increased after rubber modification.

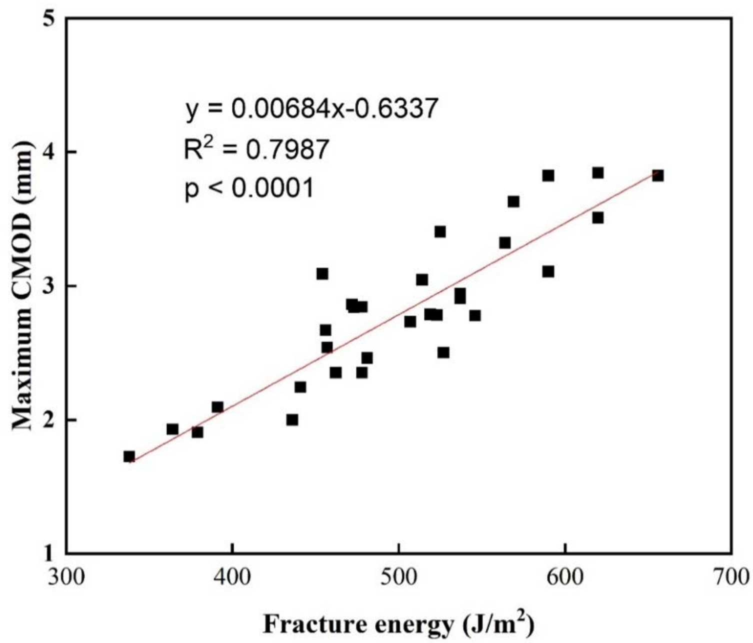
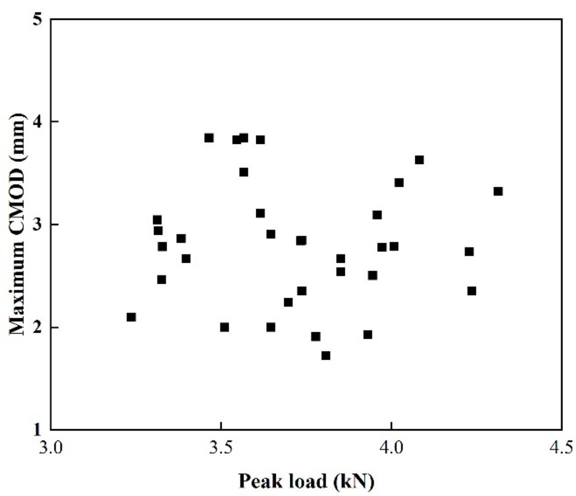
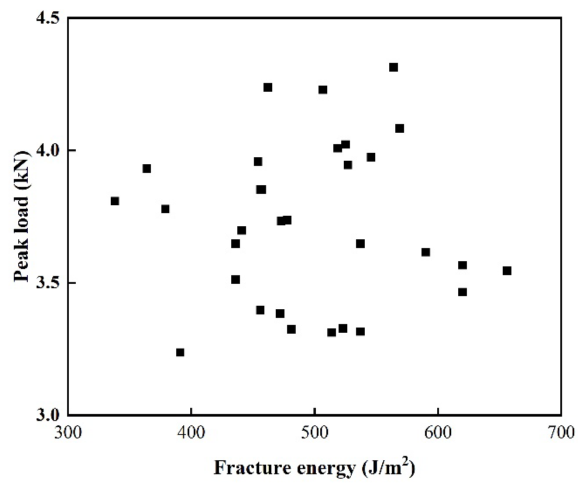


Figure 4-16. The correlation between the fracture energy and the maximum CMOD





(a) Load - CMOD correlation (b) Load - Energy correlation

Figure 4-17. The correlation between fracture energy, maximum CMOD, and peak load

The correlation between the peak load and the maximum CMOD, and between the peak load and the fracture energy are displayed in Figure 4-20. Based on the test results, the fracture energy and the maximum CMOD had no relation to the peak load. As analyzed in the previous section, the peak load also reflected the aggregate property and the uniformity of the asphalt mixture. The rubber modification slightly increased the peak load. After the specimens of the DCT test reached the peak load, the load sustained by the mixture was reduced significantly. Thus, the increase of peak load didn't contribute to the significant improvement in the fracture energy.

4.4 Conclusion

In this chapter, the low temperature cracking property of four types of plant mixed and laboratory compacted asphalt mixtures was evaluated with the DCT test. The influence of mixture design factor, test temperature, and rubber modification on DCT test parameters (fracture energy, peak load, and maximum CMOD) were analyzed. The correlation between fracture energy and maximum CMOD, post-peak fracture energy, flexibility index, and toughness index were established. The main findings are shown below.

(1) The cracking path didn't follow the shortest failure path, and the cracking path was affected by the aggregate distribution and the stiffness of the aggregate inside the mixture.

(2) Compared with the leveling layer asphalt mixture, the smaller NMAS, lower RAP content, and higher asphalt content contributed to the better low temperature cracking property of the surface layer asphalt mixture.

(3) The rubber modification in the asphalt mixture increased the thickness of the asphalt film on the surface of the aggregate and improved the bonding between the asphalt and the aggregate, and thus enhanced the low temperature cracking resistance of the asphalt mixture. The variance between parallel specimens of asphalt mixture without rubber was bigger. The peak load was influenced by the structure of the asphalt mixture, and the rubber modification increased the structural uniformity of the asphalt mixture.

(4) When the test temperature increased from -24 ºC to -18 ºC, the fracture energy of the asphalt mixture was increased, the average peak load was decreased, and the maximum CMOD of test specimens was increased.

(5) Based on the results, rubber modification had a significant influence on the fracture energy and the maximum CMOD of different asphalt mixtures, but there was no significant interactive effect of these two parameters on the peak load of asphalt mixture. There was a very significant linear correlation between the fracture energy and the maximum CMOD.

CHAPTER 5: THE PERFORMANCE EVALUATION OF EXTRACTED ASPHALT BINDER

5.1Introduction

The condition of the pavement was decreased with the increase of the service life of the pavement. The pavement with different types of distress needs maintenance or conservation. The old pavement with distress could be milled into smaller particles and used in the construction of the new pavement [21, 22]. The cost of the new pavement could be reduced if RAP was used since the cost of the RAP materials is lower than the new materials. And the adoption of RAP in the new pavement construction also had environmental benefits [23, 24]. Using RAP to partially replace the new aggregate and asphalt binder could reduce the consumption of natural materials [25]. The energy waste and pollution to the environment would be decreased with the increase of RAP content in the new pavement construction [26]. The landfill needed for the disposal of RAP materials would be decreased with the increase of RAP in the pavement construction [27]. The performance of the pavement with RAP was sensitive to the successful mixture design. The performance of the pavement with high RAP could be comparable to the pavement without RAP if the mixture design was controlled, and the service life of the pavement would be extended [28-30].

The successful design of the asphalt mixture with RAP is highly related to the performance of asphalt binder extracted from RAP [31]. Centrifuge extraction and rotary evaporate recovery are the most commonly used tests to obtain asphalt binder from RAP [32]. Normally, the asphalt mixture was submerged in the solvent to dissolve the asphalt binder from the asphalt mixture. The aggregate and asphalt binder were separated after the mixture was soaked in the solvent. The asphalt mixture is soaked in the solvent, and the asphalt was dissolved in the solvent in extraction equipment. The solvent mix of asphalt binder and the solvent was heated in a rotary evaporator to remove the solvent, and the asphalt binder was left in the flask. The content of asphalt binder in the asphalt mixture could be calculated based on the mass of the original asphalt mixture and extracted aggregate. The property of the asphalt binder in the HMA or RAP could be evaluated using a laboratory test procedure [33]. The separated aggregate can be used for gradation analysis.

The main purpose of the chapter is to evaluate the property of asphalt binder extracted from rubber modified asphalt mixture using the dry process. The standard recovery procedure will be assessed based on the comparison between the base binder and the extracted base binder. The complex shear modulus will be used as the main parameter to evaluate the efficiency of the standard extraction procedure. The asphalt binder will be extracted using the modified recovery procedure. The property of the extracted asphalt binder will be evaluated.

5.2 Test Materials and Test Method

5.2.1 Test material

Four types of asphalt mixtures (surface layer asphalt mixture with and without rubber, and leveling layer asphalt mixture with and without rubber) were extracted. Tests on extracted binders were conducted to assess the influence of rubber modification using the dry process on the property of asphalt binders. The PG 58-34 base asphalt and base asphalt mixed with crumb rubber used in the project were also evaluated. The rubber particle content in the field mixed asphalt mixtures was 10% of the content of the asphalt binder. The PG 58-34 base asphalt was mixed with rubber particles (10% of the content of the asphalt binder) in the lab at 168 ºC for 30 mins.

5.2.2 Asphalt Binder Extraction

The automatic asphalt analyzer from the Controlsgroup was adopted to separate asphalt and aggregate from the asphalt mixture, as shown in Figure 5-1. The automatic asphalt analyzer could conduct two types of the extraction process, the fast extraction process, and the standard extraction process. The standard extraction process included washing cycles and drying cycles. The standard extraction process was used to analyze the gradation of the mixture, and the asphalt binder was not collected. The preheated asphalt mixtures were separated into small pieces that were sealed in the washing drum. The fine aggregate passed the 0.075 mm (No. 200) sieve, and the asphalt-TCE solution was delivered to the centrifuge cup. After the drying cycles, the fine aggregate was left inside the centrifuge cup. While the fast extraction had a specimen soaking procedure before the washing cycles, the mixture inside the washing drum was soaked in the heated TCE solvent for 20 minutes. Trichloroethylene (TCE) and asphalt binder were received after specimen soaking procedure. The rotary evaporate procedure was conducted to remove TCE from the asphalt-TCE solution [34]. The rotary evaporation process followed the ASTM standard, and the test apparatus is shown in Figure 5-2. The TCE was evaporated, and the asphalt binder was left in the evaporating flask. The asphalt binder was received for a later test.

Figure 5-1. The automatic asphalt analyzer used to separate asphalt from the asphalt mixture

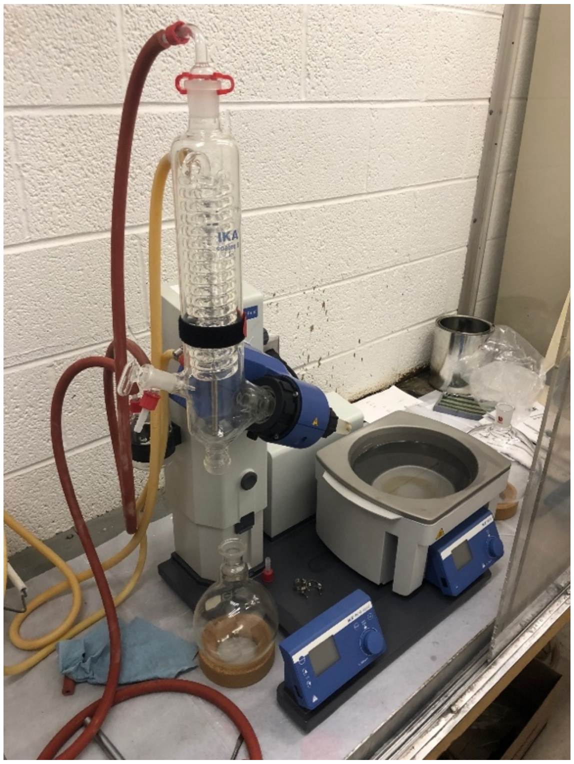


Figure 5-2. The rotary evaporator used to obtain asphalt binder from the asphalt-TCE solution

5.2.3 The aging procedure of extracted asphalt binder

During the asphalt mixture production and placement, the light component in the asphalt binder was evaporated or oxidized by interacting with oxygen at high temperatures [35]. The Rolling Thin-Film Oven (RTFO) test in the laboratory could reflect the short-term aging of asphalt during this procedure, and the test apparatus is shown in Figure 5-3. The short-term aging test was operated based on ASTM D2872 specification procedures. The bottles with about 35 g asphalt were spun at 163 °C for 85 minutes at a speed of 15 RPM. During the test, the air was blown into the bottle at a flow rate of 4000 mL/min to ensure the contact between the asphalt and the air. Finally, the short-term aged asphalt was collected for subsequent testing.

Figure 5-3. The rolling thin-film oven test apparatus

During the service life of the asphalt mixture, natural sunlight and temperature changes caused the physical and chemical reactions of the asphalt binder in the mixture [36]. The components in the asphalt had undergone oxidation, evaporation, and dehydrogenation process. The Pressure Aging Vessel (PAV) in the laboratory could reflect the long-term aging of asphalt binder during the process, and the PAV test apparatus is shown in Figure 5-4. The long-term aging test was operated based on ASTM D6521 specification procedures. RTFO aged sample was heated at atmospheric pressure of 2.10 MPa under 100 °C for 20 hours. After long-term aging, the degassing procedure was conducted for 15 minutes in a vacuum oven at 170 °C. Finally, the PAV aged asphalt was collected for subsequent testing.

Figure 5-4. The pressure aging vessel (PAV) test apparatus

5.2.4 Dynamic Shear Rheometer (DSR)

The Dynamic Shear Rheometer (DSR) was adopted to assess the viscoelastic performance of the asphalt binder. The MCR 302 Rheometer from Anton Paar was used to conduct the test, and the test apparatus is shown in Figure 5-5. The complex shear modulus ($|G^*|$) and phase angle (δ) of the asphalt binder are measured from the DSR [37]. The DSR test of the unaged and RTFO aged samples were operated under nine temperatures and six frequencies. The PAV aged test was operated under five temperatures and six frequencies.

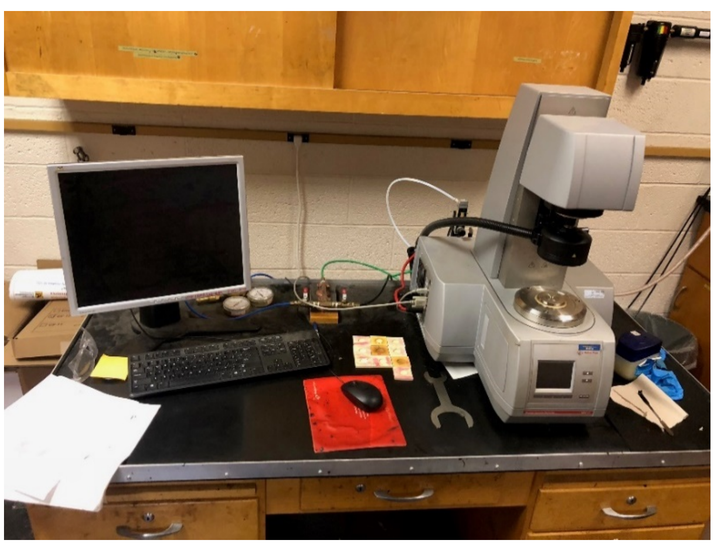


Figure 5-5. The DSR test apparatus from Anton Paar

5.2.5 Multiple Stress Creep and Recovery (MSCR)

The MSCR test was conducted using the DSR at specific temperatures, and the test samples were RTFO aged. The percent recovery reflected the elastic response of asphalt binder under shear creep and recovery at 0.1 kPa and 3.2 kPa stress levels. The non-recoverable creep compliance was the residual strain in asphalt binder after a creep and recovery cycle divided by the stress of 0.1 kPa or 3.2 kPa. The non-recoverable creep compliance could be used to reflect the permanent deformation of asphalt binder under repeated load [38].

The operation of MSCR followed the AASHTO T350 test protocol. The performance grade of asphalt binder is PG 58-34, the MSCR test temperature chose 52ºC, 58ºC, 64ºC, 70ºC, and 76ºC. The highest test temperature was different for different asphalt binders, and the test temperature was not increased once the asphalt binder failed to pass traffic grade at a certain temperature, according to AASHTO M 332. The test parameter and traffic condition for different traffic levels is shown in Table 5-1.

Table 5-1. The MSCR test parameter and traffic conditions under different traffic levels

Traffic level	Traffic condition	Test parameter
Standard (S)	Traffic levels < 10 million equivalent single-axle loads (ESALs) and traffic speed >70 km/h	$J_{nr\_3.2} \leq 4.5$ kPa^{-1} $J_{nr\_diff} \leq 75\%$
High (H)	10 million < ESALs < 30 million or 20 km/h < traffic speed < to 70 km/h	$J_{nr\_3.2} \leq 2.0$ kPa^{-1} $J_{nr\_diff} \leq 75\%$
Very High (V)	ESALs > 30 million or traffic speed <20 km/h	$J_{nr\_3.2} \leq 1.0$ kPa^{-1} $J_{nr\_diff} \leq 75\%$
Extreme high (E)	ESALs > 30 million and traffic speed <20 km/h	$J_{nr\_3.2} \leq 0.5$ kPa^{-1}

The initial strain(ε_0), peak strain at 1.0 s (ε_1), and the recoverable strain at 10.0 s (ε_{10}) during every creep and recovery cycles were recorded. The relation between the strain and time during one creep and recovery cycle is shown in Figure 5-6. The creep recovery test results of ten cycles under different creep stresses is shown in Figure 5-7.

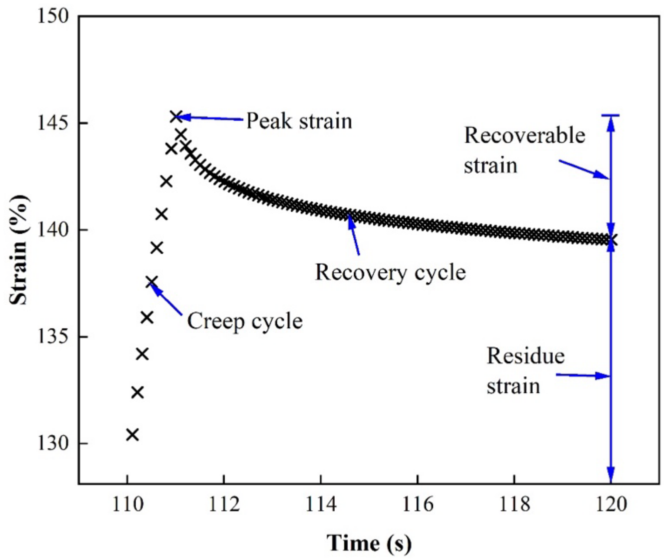
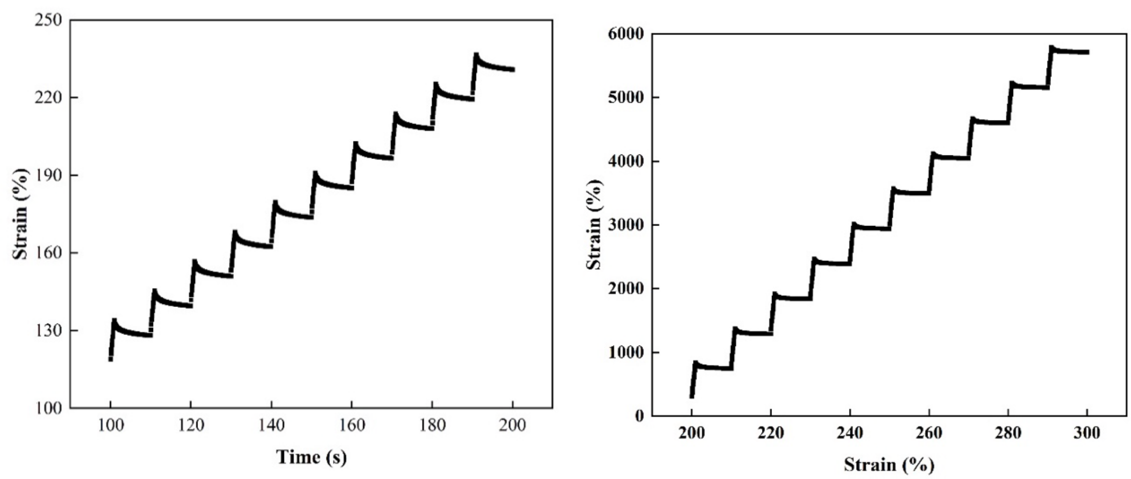


Figure 5-6. The strain- time curve during one creep recovery cycle



(a) 0.1 kPa (b) 3.2 kPa

Figure 5-7. The creep recovery test results under different creep stresses

The percent recovery under the creep stress of 0.1 kPa or 3.2 kPa (N = 1 to 10) was calculated with Equation 6.1.

$$\varepsilon_r(0.1\ or\ 3.2, N) = \frac{(\varepsilon_1 - \varepsilon_{10}) \cdot 100}{\varepsilon_1} \tag{6.1}$$

The average percent recovery at 0.1 kPa (or 3.2 kPa) is calculated based on Equation 6.2.

$$\text{R\_0.1(or R\_3.2)} = \frac{\sum_1^{10} \varepsilon_r(0.1\ or\ 3.2\ ,N)}{10} \tag{6.2}$$

The percent difference (R_diff) in recovery between 0.1 kPa and 3.2 kPa is calculated according to Equation 6.3

$$R_{diff} = \frac{R_{0.1} - R_{3.2}}{R_{0.1}} \cdot 100 \tag{6.3}$$

The non-recoverable creep compliance $J_{nr}(0.1$ or 3.2, N) is calculated according to Equation 6.4.

$$J_{nr}(0.1\ or\ 3.2, N) = \frac{\varepsilon_{10}}{0.1\ (\text{or } 3.2)} \tag{6.4}$$

The average non-recoverable creep compliance $J_{nr}0.1$ (or 3.2) is calculated according to Equation 6.5.

$$J_{nr,0.1kPa(or\ 3.2kPa)} = \frac{\sum_1^{10} J_{nr}(0.1\ or\ 3.2\ ,N)}{10} \tag{6.5}$$

The percent difference in non-recoverable creep compliance between 0.1 kPa and 3.2 kPa is calculated according to Equation 6.6.

$$J_{nr,diff} = \frac{J_{nr,3.2kPa} - J_{nr,0.1kPa}}{J_{nr,0.1kPa}} \cdot 100 \tag{6.6}$$

5.2.6 Asphalt Binder Cracking Device (ABCD)

The Asphalt Binder Cracking Device (ABCD) could directly assess the low temperature cracking property of asphalt binders, as shown in Figure 5-8. The operation of ABCD followed the AASHTO TP 92 test protocol. The ABCD test procedure is shown in Figure 5-9. The relation between strain and temperature of asphalt binder ring under certain temperature decreasing rate was recorded [39]. The relation between the strain and temperature during the ABCD test is displayed in Figure 5-10. The asphalt binder cracked when the temperature decreased to the crack temperature, as shown in Figure 5-9 (d). The strain jump was monitored, and the fracture stress in the asphalt binder is calculated with Equation 6.7. The relation between the microstrain and temperature during the ABCD test is shown in Figure 5-10. The PG grade based on ABCD can be obtained according to Equation 6.8.

$$Fracture\ stress\ (MPa) = (\text{Strain Jump}, \mu\varepsilon\ ) * 0.157 \tag{6.7}$$

$$PG\ grade\ by\ ABCD\ (°C) = 0.78 * (\text{ABCD cracking temperautre}, °C) - 0.9 \tag{6.8}$$

Figure 5-8. The Asphalt Binder Cracking Device (ABCD) test apparatus

(a) Sample in the mold

(b) Test sample

(c) Sample in the test chamber

(d) Failed sample

Figure 5-9. The ABCD test procedure

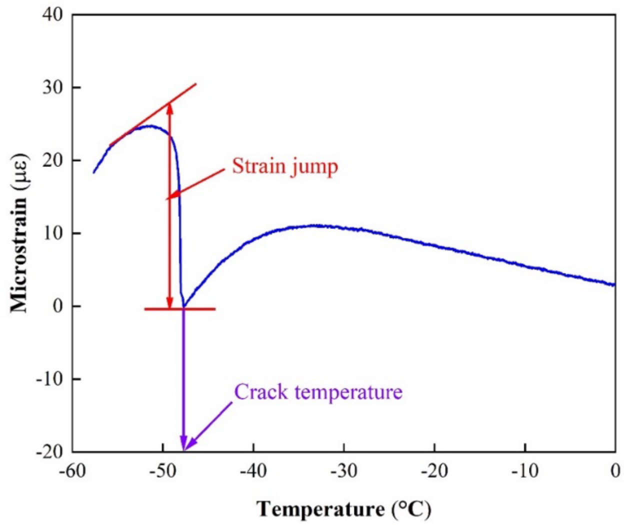


Figure 5-10. The relation between the microstrain and temperature during the ABCD test

5.3 Test results and analysis

5.3.1 The rotary evaporation procedure validation

The base asphalt with PG 58-34 used in the Dickinson project to produce asphalt mixture was mixed with TCE solvent, and the rotary evaporate procedure was conducted on the PG 58-34 asphalt - TCE mix. The operation of the extraction procedure followed the ASTM D5404. By comparing the |G*| of original PG 58-34 asphalt binder and extracted PG 58-34 asphalt binder, the extraction procedure and the efficiency could be validated.

The correlation between the |G*| of the original asphalt binder and extracted asphalt binder is shown in Figure 5-11. The |G*| of extracted asphalt binder was significantly lower than the original asphalt binder, which proved that the standard extraction procedure couldn't effectively remove all the TCE. The average ratio of the |G*| of extracted asphalt binder to the |G*| of original asphalt binder was slightly lower than 60%.

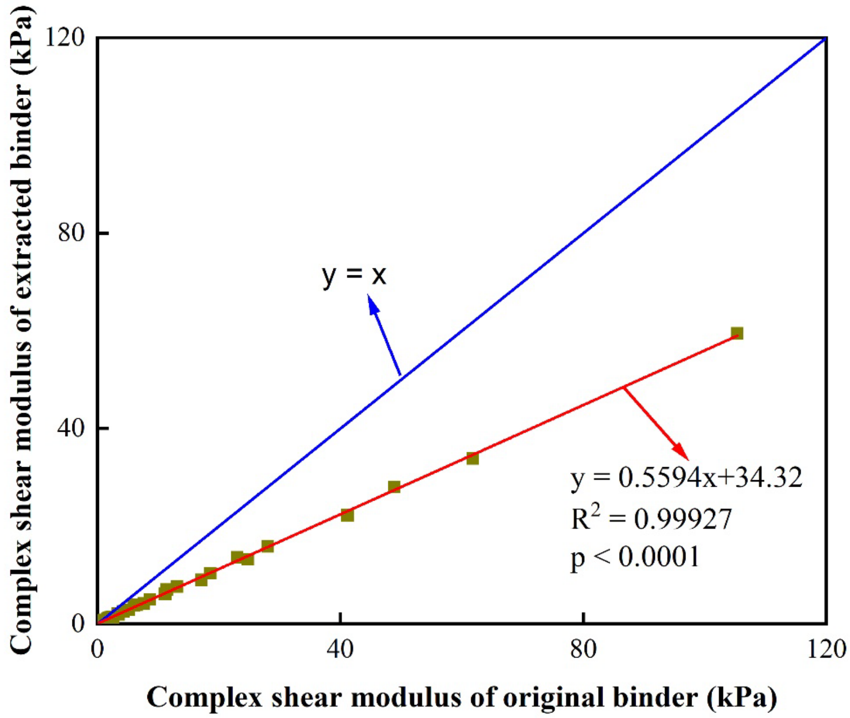


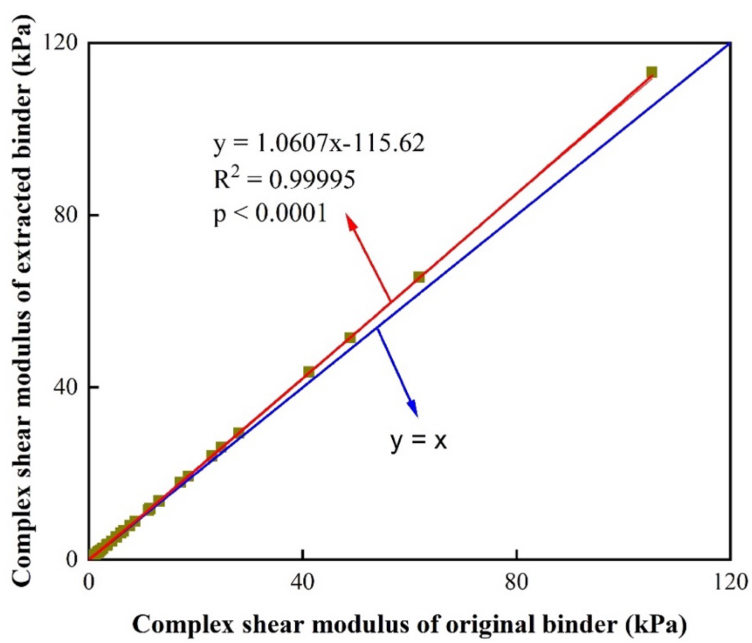
Figure 5-11. The relation between the complex shear modulus of the original binder and extracted binder

In the RTFO aging procedure, the thin-film asphalt binder was heated, and high pressure was applied. If applied with a proper condition, the procedure could effectively remove the residue TCE in asphalt binder without aging asphalt binder.

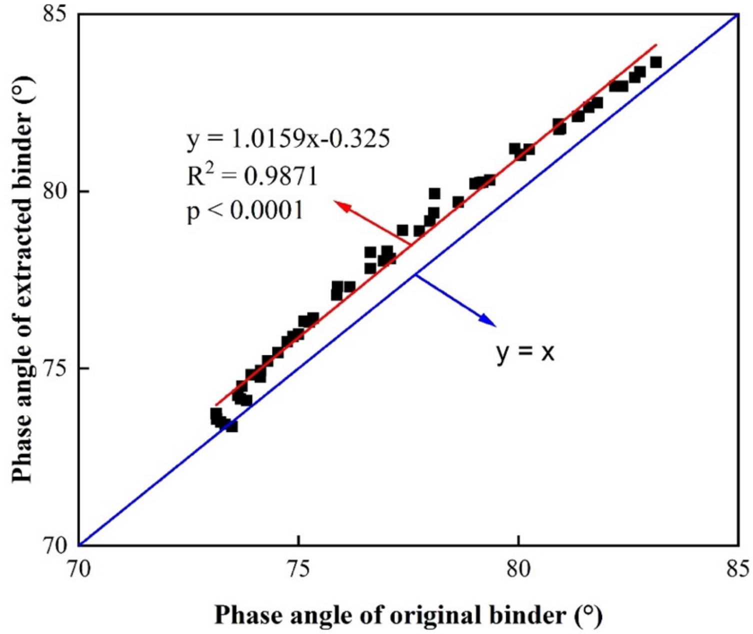
Based on the standard operation procedure in ASTM D2872, a glass container with 35g asphalt binders was rotated in the oven at 163 °C for 85 min. The modified procedure to remove the TCE resides is as follows. Firstly, 35g extracted asphalt binder containing TCE was poured into the glass container. The glass container was rotated at the rate of 15 r/min in the oven at the temperature of 163 °C for 5 min to guarantee the film thickness inside the glass container is even. Then the air rate was set at 4L/min and the glass container was rotated at the same temperature and the same rotation rate for 10 minutes. Finally, the collected asphalt binder was removed in the glass container and was stirred gently to homogenize the residue.

The DSR test was conducted on the extracted asphalt binder after removing the TCE residue. The comparison between the |G*| and δ of the original asphalt binder and extracted asphalt binder is shown in Figure 5-12. After the modified extraction procedure, the mean percentage difference for |G*| of the original asphalt binder and extracted asphalt binder was similar, and the extracted asphalt binder was 0.12% higher.

The phase angle of the extracted asphalt binder was slightly higher than the phase angle of the original asphalt binder, and the mean percentage difference for phase angle of extracted asphalt binder was 1.17% higher. Based on the test result, the property of extracted asphalt binder with the modified method was more reliable to reflect the property of the original asphalt binder. Three extraction tests were conducted on the asphalt mixed with the TCE, and the average standard deviation between different test results was 0.037, which proved that the modified extraction procedure was repeatable. The extraction procedure used to extract the field mixed asphalt mixture will use the modified extraction procedure.



(a) Complex shear modulus



(b) Phase angle

Figure 5-12. The comparison between the original binder and extracted binder

5.3.2 The gradation analysis and asphalt binder content of field mixed asphalt mixture

5.3.2.1 The gradation analysis of field mixed asphalt mixture

The gradation analysis was conducted on the plant mixed asphalt mixtures, four types of asphalt mixtures, which included surface layer asphalt mixture (5E1) with and without rubber, and leveling layer asphalt mixture (LVSP) with and without rubber. The capability of the automatic asphalt analyzer was about 2.5 kg asphalt mixtures per cycle. The gradation analysis results displayed the average results of several mixtures to eliminate the errors due to the segregation of the asphalt mixtures. The gradation results of the surface layer and the leveling layer asphalt mixtures are shown in Figure 5-13 and Figure 5-14. The design gradation used during the field application was also compared, and the gradation of the extracted asphalt mixture fit well with the original mixture design gradation.

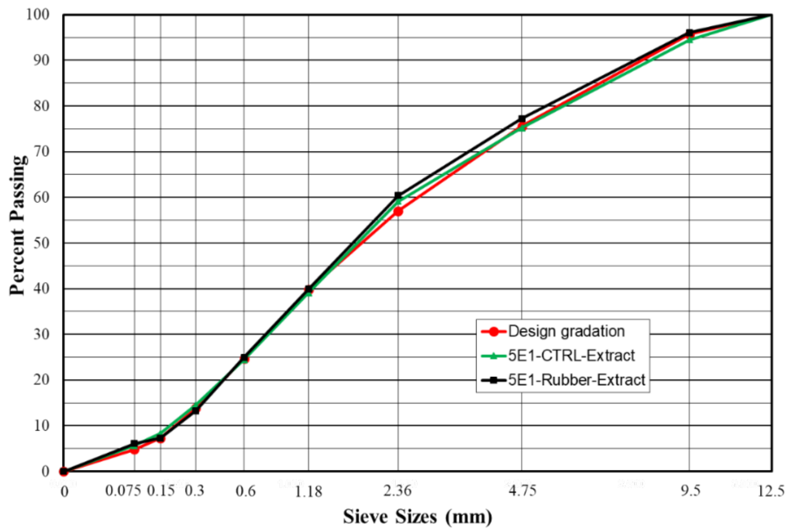


Figure 5-13. The gradation of the surface layer asphalt mixture after the extraction procedure

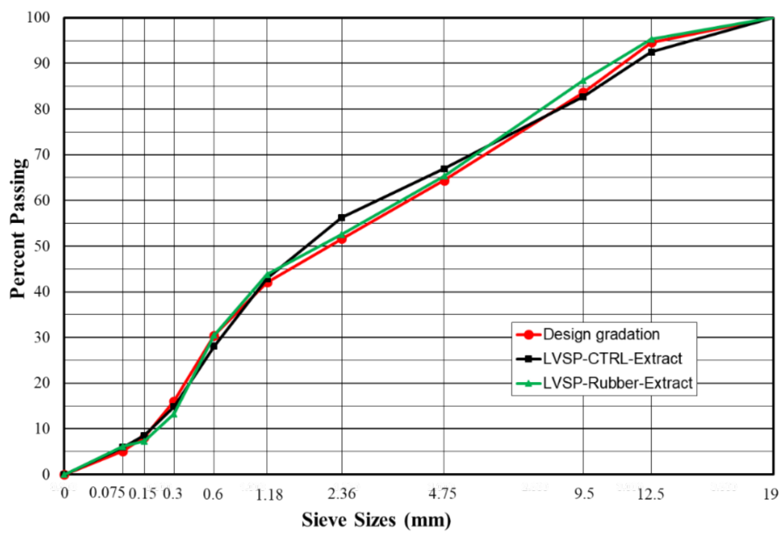
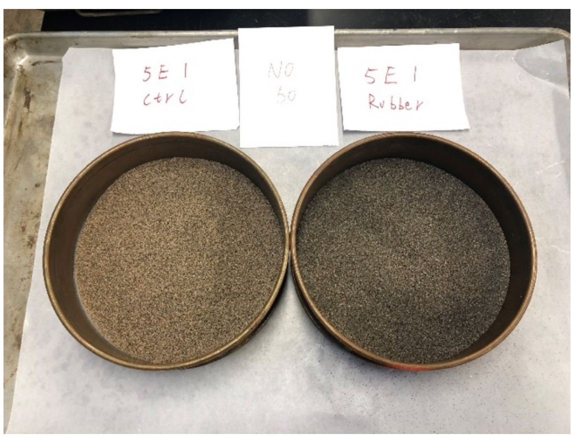
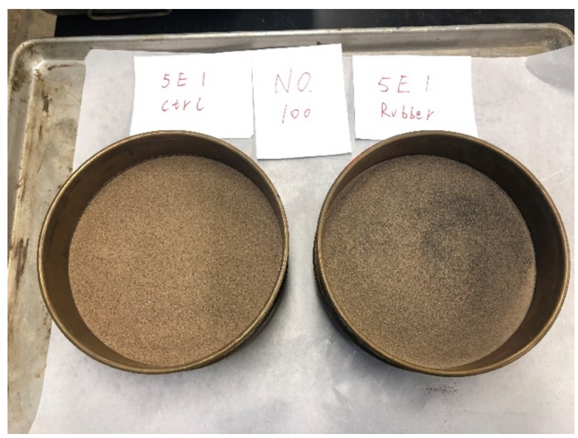


Figure 5-14. The gradation of the leveling layer asphalt mixture after the extraction procedure

The comparison between the surface layer without rubber (5E1-CTRL) and surface layer with rubber (5E1-Rubber) after the asphalt was washed by the TCE is shown in Figure 5-15. The color of the surface layer aggregate that from the mixture with rubber was darker than the aggregate from the mixture without rubber (Figure 5-15 (a)). The unreacted rubber particles that used to produce rubber modified asphalt mixture using the dry process were retained in the aggregate when the asphalt mixture was washed by the TCE. Some rubber particles that reacted with asphalt binder may also be washed out from the asphalt binder, and the assumption may need other test methods to validate. The aggregate that retained in the No. 50 and the No. 100 sieve from the surface layer asphalt mixtures with and without rubber were compared, as shown in Figure 5-15 (b) and (c). The density of the rubber particle was smaller than the density of the aggregate, and the rubber particle was on the surface layer of the sieve. Significant differences could be observed in Figure 5-15 (b) and (c), especially the aggregate that retained in the No.50. The difference was less significant for aggregate retained in the No.100 sieve.

(a) 5E1 CTRL vs 5E1 Rubber

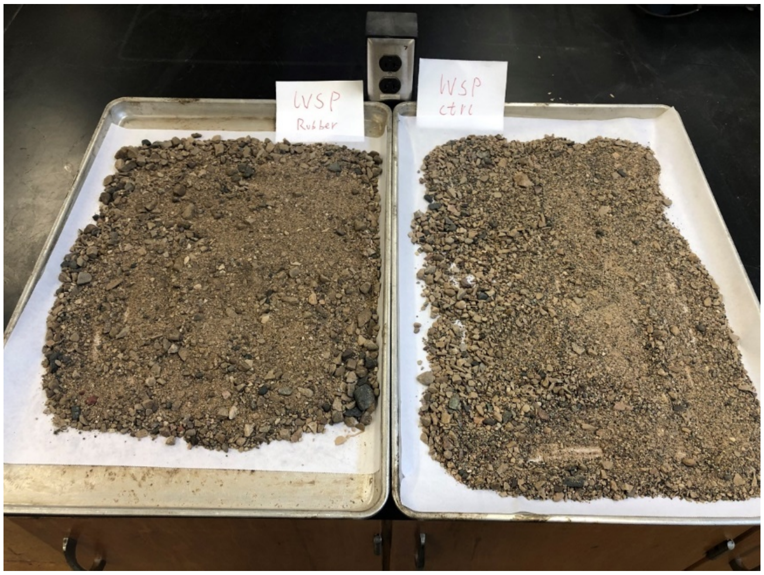




(b) Aggregate retained in No. 50 sieve (c) Aggregate retained in No. 100 sieve

Figure 5-15. The comparison between the surface layer aggregate without and with rubber

The comparison between the leveling layer without rubber (LVSP-CTRL) and leveling layer with rubber (LVSP-Rubber) after the asphalt was washed by the TCE is shown in Figure 5-16. The color of the leveling layer aggregate from the mixture with rubber was darker than the aggregate from the mixture without rubber (Figure 5-16 (a)). The aggregate that retained in the No. 50 and the No. 100 sieve from the leveling layer asphalt mixtures with and without rubber were compared, as shown in Figure 5-16 (b) and (c). The difference was more significant for aggregate retained in the No.50 sieve than the aggregate retained in the No. 100 sieve. The difference was less significant for the leveling layer aggregate than the surface layer aggregate. The asphalt binder content of the leveling layer mixture was lower, and the rubber content in the leveling layer asphalt mixture was lower. The rubber content was lesser in the leveling layer aggregate after asphalt binder was washed out; thus, the color difference was less significant.



(a) LVSP CTRL vs LVSP Rubber

(b) Aggregate retained in No. 50 sieve (c) Aggregate retained in No. 100 sieve

Figure 5-16. The comparison between the leveling layer aggregate without and with rubber

5.3.2.2 The asphalt binder content of field mixed asphalt mixture

The asphalt binder content in different asphalt mixtures was validated with the extraction procedure, and the asphalt binder content of different asphalt mixtures is shown in Figure 5-17.

For the surface layer (5E1) asphalt mixture, the design asphalt binder content is 5.9%, in which the asphalt binder in RAP is not included. In order to keep consistency, the design asphalt binder content for asphalt mixture with and without rubber was the same. Rubber was considered as part of the asphalt binder because the rubber would interact with asphalt binder. The binder content in RAP is 4.15%, 17% RAP is used in 5E1 design. The total asphalt binder content of 5E1 mixture design is 6.6% (6.6%=5.9%+4.15%*17%). The total asphalt binder content of 5E1 mixture with rubber was 6.08% if the rubber was not considered as part of asphalt binder (5.9%*0.91+4.15%*17%=5.37+0.71=6.08%). The average asphalt binder content of the surface layer asphalt mixture with and without rubber are 6.02% and 6.19%, respectively. The asphalt binder content was lower than the design asphalt binder content. The asphalt binder from RAP maybe not totally extracted. The rubber particle content was about 0.5%, but the asphalt binder content from the asphalt mixture with rubber was 0.17% lower than that from asphalt mixture without rubber. Part of rubber particles was interacted with asphalt binder and dissolved into the asphalt binder, only rubber particles that did not interact with asphalt binder were washed out and retained in the aggregate.

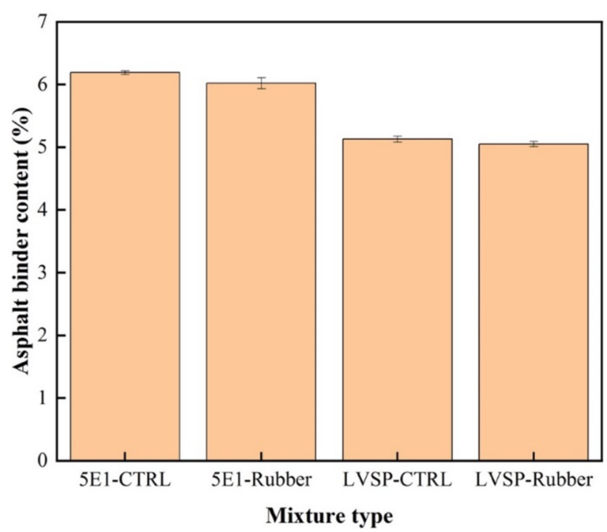


Figure 5-17. The asphalt binder content determination based on the extraction procedure

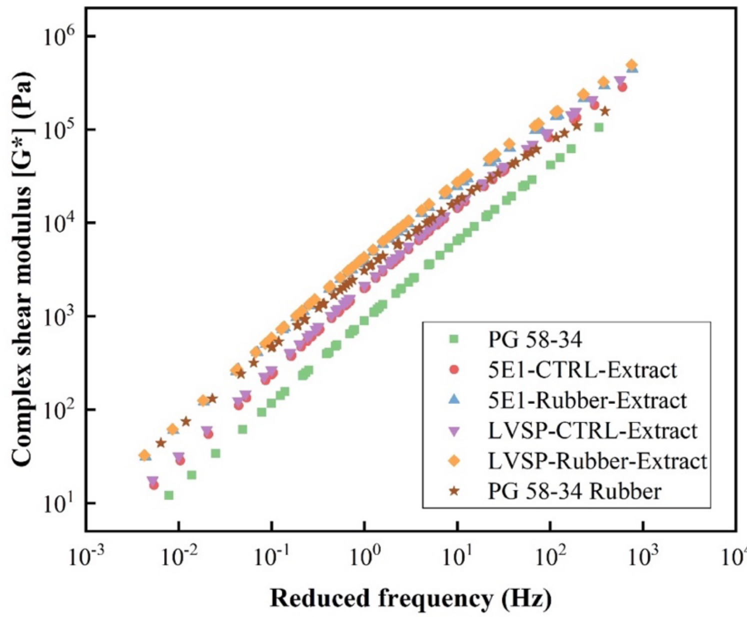
For the leveling layer (LVSP) asphalt mixture, the design asphalt binder content is 4.2%, in which the asphalt binder in RAP is not included. The binder content in RAP is 4.15%, 25% RAP is used in LVSP design. The total asphalt binder content of the LVSP mixture design is 5.24% (5.24%=4.2%+4.15%*25%). The total asphalt binder content of the leveling layer mixture with rubber was 4.86% if the rubber was not considered as part of asphalt binder

(4.2%*0.91+4.15%*25%=3.82+1.04=4.86%). The average asphalt binder content of the leveling layer asphalt mixture with and without rubber are 5.05% and 5.13%, respectively. The rubber particle content was about 0.38%, but the asphalt binder content from the asphalt mixture with rubber was 0.08% lower than that from asphalt mixture without rubber. Rubber particles that did not interact with asphalt binder were washed out by TCE solvent and retained in the aggregate.

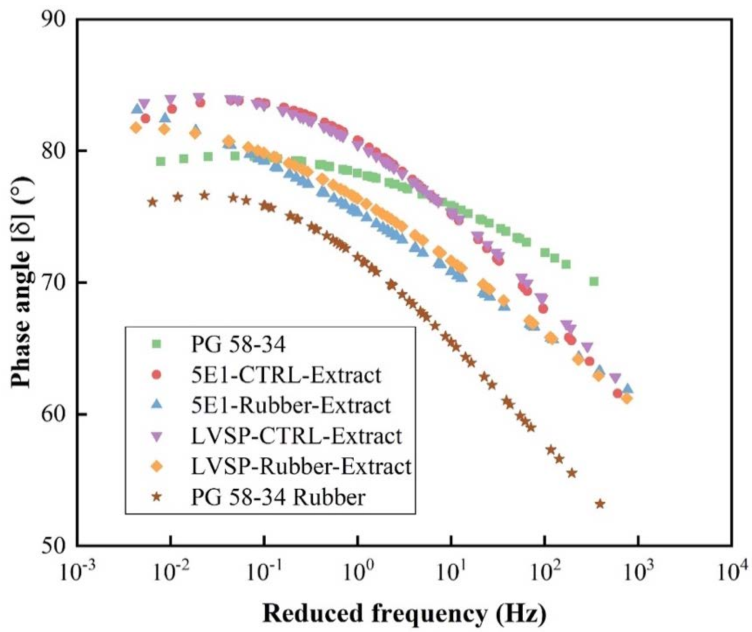
5.3.3 The DSR analysis

5.3.3.1 The unaged asphalt binder

The master curve of different types of unaged asphalt binders is shown in Figure 5-18. The complex shear modulus master curve of different types of unaged asphalt binders is shown in Figure 5-18 (a). The |G*| of all extracted asphalt binder was higher than the base asphalt. The |G*| of the asphalt binder extracted from the surface layer asphalt mixture and the leveling layer asphalt mixture were similar, but the |G*| of the asphalt binder extracted from the leveling layer asphalt mixture was slightly higher. The RAP content of the leveling layer was higher, and the new asphalt binder content was lower, the percentage of asphalt binder from the RAP in the extracted asphalt binder of the leveling layer asphalt mixture was higher. The |G*| of the asphalt binder from the RAP was higher; thus, the extracted asphalt binder from the leveling layer asphalt mixture had higher |G*|. The |G*| of the extracted asphalt binder from asphalt mixtures with rubber was higher than that of asphalt mixture without rubber. The light components from the asphalt binder interacted with the rubber particles, and the viscosity of the asphalt binder was increased, thus increased the |G*| of the extracted asphalt binder from asphalt mixtures with rubber. Due to the interaction between the rubber particle and asphalt binder, the |G*| of the PG 58-34 asphalt with rubber was higher. The |G*| of the PG 58-34 with rubber had similarly high values as the extracted asphalt mixture with rubber at the low reduced frequency regions. But the |G*| of the PG 58-34 with rubber had lower values than the extracted asphalt mixture without rubber at the high reduced frequency regions. What's more, the difference between the PG 58-34 with rubber and PG 58-34 base asphalt was reduced with the increase in the reduced frequency. At the high frequency conditions, the stiffness of extracted asphalt binder was higher because of the existence of the aged asphalt binder from RAP.



(a) The complex shear modulus master curve



(b) The phase angle master curve

Figure 5-18.The master curve of different types of unaged asphalt binder

The phase angle master curve of different types of unaged asphalt binders is shown in Figure 5-18 (b). The phase angle ranged between 73.2º and 81.8º under all temperatures and frequencies. The phase angle of the asphalt binder was reduced after mixing with rubber particles. The light component was absorbed by the rubber particle, and the elastic perfromance of the asphalt binder was improved. The minimum phase angle reached to 53.2º for PG 58-34 asphalt binder with rubber. The phase angle of the extracted asphalt binders decreased dramatically with the increase in the reduced frequency. At the low reduced frequency (high temperature) conditions, the phase angle of the extracted asphalt binder was higher than the PG 58-34 base asphalt binder. At the high reduced frequency (low temperature) conditions, the phase angle of the extracted asphalt binder was lower than the PG 58-34 base asphalt binder. The phase angle differences between the extracted asphalt binder from the surface layer asphalt mixture and the leveling layer asphalt mixture were minimal. Similar to the base asphalt binder and base asphalt binder with rubber, the phase angle of extracted asphalt binder from the mixture with rubber was lower than that from mixture without rubber. But the difference was minimal at the high reduced frequency regions. The aged asphalt binder from RAP increased the phase angle of the extracted asphalt binder. The viscosity of the asphalt binders was increased due to the aged binder from the RAP.

The rutting parameter of different types of unaged asphalt binders is shown in Figure 5-19. The rutting parameter of the base asphalt binder was the lowest. The rutting parameter of extracted asphalt binder from asphalt mixture without rubber was lower than that from asphalt mixture with rubber. Similar to the trend of the complex shear modulus, the rutting parameter of base asphalt with rubber had lower rutting parameters at low test temperatures and higher rutting parameters at high test temperatures. Based on the Superpave performance grade definition, the rutting parameter of the unaged asphalt binder should be higher than 1.0 kPa. For unaged asphalt binder, the high temperature grade of base asphalt was 58 ºC; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer without rubber was 64 ºC, and the high temperature grade of extracted asphalt binder from the leveling layer and surface layer with rubber and base asphalt with rubber was 70 ºC. The aged asphalt binder from RAP increased the complex shear modulus of extracted asphalt binder, thus increased the high temperature grade. The improvement of rubber addition was more significant. The results from the previous section mentioned that only part of rubber particles that interacted with asphalt binder was retained in asphalt binder after the extraction procedure. The combined effect of aged asphalt binder from RAP and part of interacted rubber particles increased the high temperature grade 12 ºC for extracted asphalt binder from mixtures with rubber.

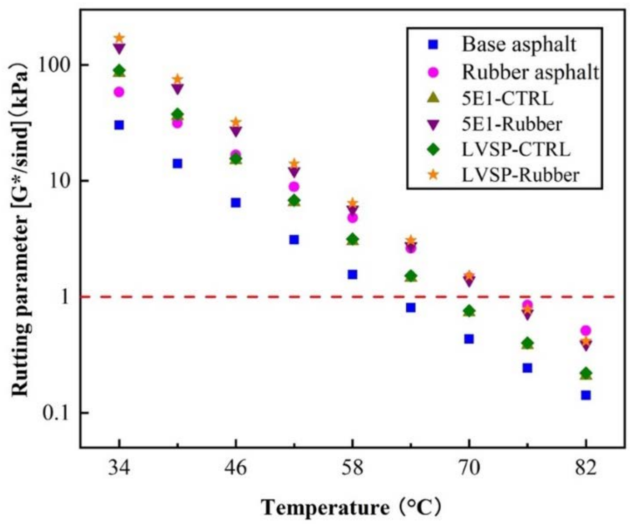


Figure 5-19. The rutting parameter of different types of unaged asphalt binders

5.3.3.2 The RTFO aged asphalt binder

The mass loss of the different extracted binder types after the RTFO aging procedure is shown in Figure 5-20. The mass loss of different extracted binder types was lower than the standard restriction of 1%. But the mass loss was very close to the limitation, especially for the surface layer asphalt mixture without rubber and the leveling layer asphalt mixture with rubber, the average mass loss was 0.95%.

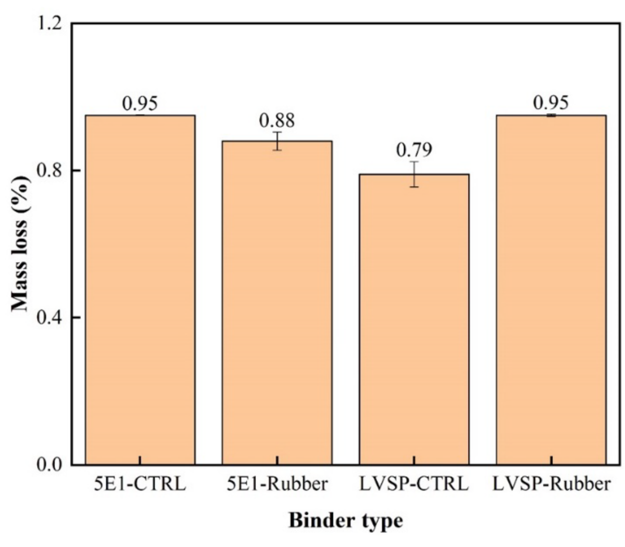
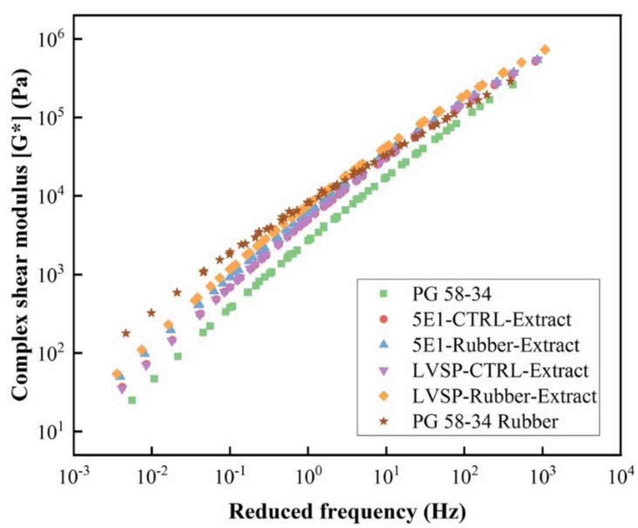


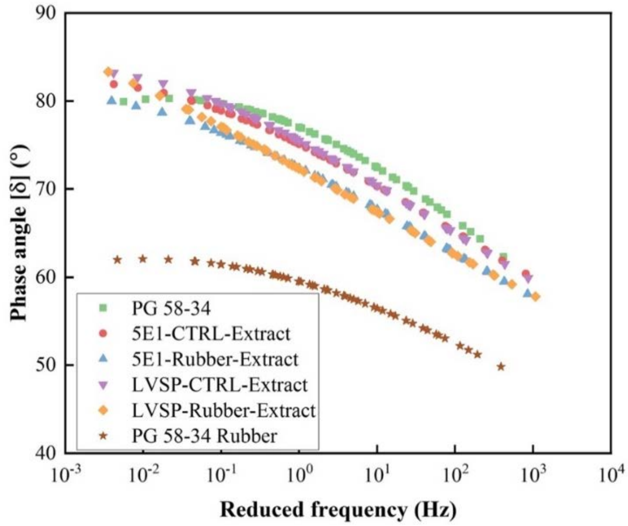
Figure 5-20. The mass loss of different extracted binder types after RTFO aging procedure

The complex shear modulus master curve of different types of RTFO aged asphalt binders are shown in Figure 5-21 (a). The |G*| of asphalt binder increased after RTFO aging. The |G*| of all extracted asphalt binder was higher than the base asphalt. The |G*| of the asphalt binder extracted from the leveling layer asphalt mixture was slightly higher than that from the surface layer asphalt mixture. The influence of aged asphalt binder from RAP was reduced after RTFO aging. The |G*| of the extracted asphalt binder from asphalt mixtures with rubber was higher than that of asphalt mixture without rubber. The |G*| of the PG 58-34 with rubber was higher than the extracted asphalt binder at the low reduced frequency regions. But the |G*| of the PG 58-34 with rubber had lower values than the extracted asphalt binder at the high reduced frequency regions. The difference between the PG 58-34 with rubber and PG 58-34 base asphalt was reduced with the increase of the reduced frequency.

The phase angle master curve of different types of RTFO aged asphalt binders are shown in Figure 5-21 (b). The phase angle of the asphalt binder decreased after RTFO aging. The phase angle of the asphalt binder with rubber was lower than asphalt binder without rubber because the elastic property of the asphalt binder with rubber was higher. The phase angle of the extracted asphalt binder was lower than the PG 58-34 base asphalt binder except at the low reduced frequency (high temperature) conditions. The phase angle of the extracted asphalt binder from the surface layer asphalt mixture was slightly lower than that from the leveling layer asphalt mixture. The phase angle of extracted asphalt binder from the mixture with rubber was lower than that from the mixture without rubber.



(a) The complex shear modulus master curve



(b) The phase angle master curve

Figure 5-21.The master curve of different types of RTFO aged asphalt binder

The rutting parameter of different types of RTFO aged asphalt binders is presented in Figure 5-22. The rutting parameter of the base asphalt binder was the lowest. The rutting parameter of extracted asphalt binder from asphalt mixture without rubber was lower than that from asphalt mixture with rubber. The rutting parameter of base asphalt with rubber had lower rutting parameters at low test temperatures and higher rutting parameters at high test temperatures than the extracted asphalt binder. Based on the Superpave performance grade definition, the rutting parameter of RTFO aged asphalt binder should be higher than 2.2 kPa. For RTFO aged asphalt binder, the high temperature grade of base asphalt was 58 ºC; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer without rubber was 64 ºC; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer with rubber was 70 ºC, and the high temperature grade of base asphalt binder with rubber was 76 ºC.

Based on the test results analysis of unaged and RTFO aged asphalt binder, the high temperature grade of base asphalt was 58 ºC; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer without rubber was 64 ºC, and the high temperature grade of extracted asphalt binder from the leveling layer and surface layer with rubber and base asphalt with rubber was 70 ºC.

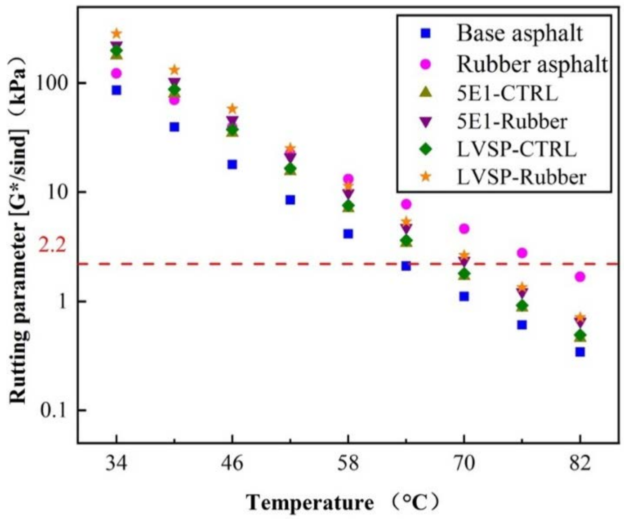
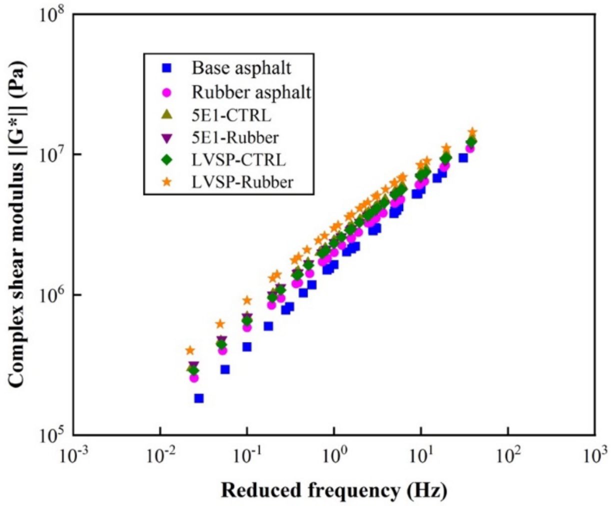


Figure 5-22. The rutting parameter of different types of RTFO aged asphalt binders

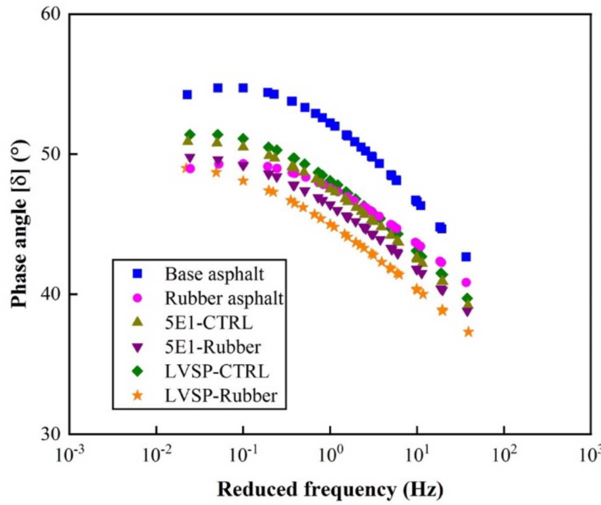
5.3.3.3 The PAV aged asphalt binder

The complex shear modulus master curve of different types of PAV aged asphalt binders is displayed in Figure 5-23 (a). The |G*| of asphalt binder increased with the increase of the reduced frequency. The |G*| of asphalt binder increased after PAV aging. The |G*| of all extracted asphalt binder was higher than the base asphalt binder with rubber and base asphalt. The |G*| of the base asphalt binder was increased after modification with rubber. The difference between the PG 58-34 with rubber and PG 58-34 base asphalt was reduced. After PAV aging, the light component in asphalt binder was oxidized, and increased the stiffness, and the effect of rubber on the |G*| of asphalt binder was reduced.

The phase angle master curve of different types of PAV aged asphalt binders is shown in Figure 5-23 (b). The phase angle of the asphalt binder decreased after PAV aging. The phase angle of the extracted asphalt binder and asphalt binder with rubber was lower than the base asphalt binder. The phase angle of the extracted asphalt binder from the surface layer asphalt mixture without rubber was slightly lower than that from the leveling layer asphalt mixture without rubber. But the phase angle of the extracted asphalt binder from the surface layer asphalt mixture with rubber was slightly higher than that from the leveling layer asphalt mixture with rubber. The phase angle of extracted asphalt binder from the mixture with rubber was lower than that from mixture without rubber.



(a) The complex shear modulus master curve



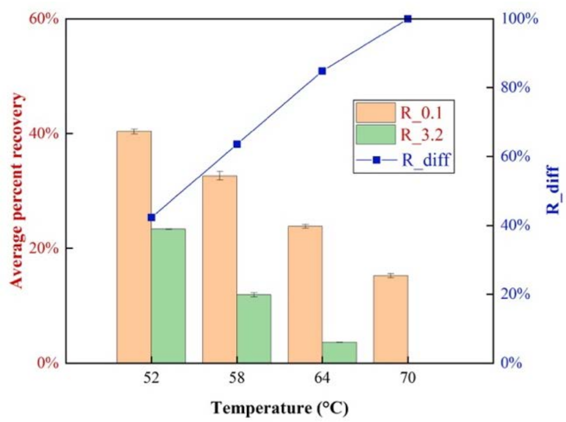
(b) The phase angle master curve

Figure 5-23. The master curve of different types of PAV aged asphalt binder

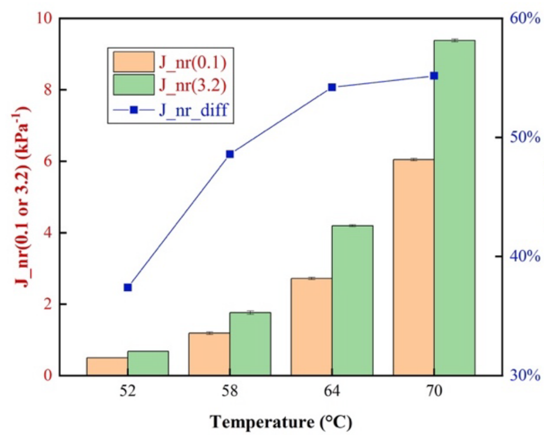
5.3.4 MSCR results

The average percent recovery and percent difference in the recovery of base asphalt binder after RTFO aging is shown in Figure 5-24 (a). The base asphalt binder failed at 70 °C, according to AASHTO M 332-14. Under the 0.1 kPa stress level, the average percent recovery of base asphalt reduced from 40.36% to 15.24% when the test temperature increased from 52 ºC to 70 ºC. The average percent recovery of asphalt binder reduced with the increase of stress level. Non-recoverable deformation was easier to occur in the asphalt binder at high stress levels, thus decreased the average percent recovery. The average percent recovery of the base asphalt reduced to 0 at the test temperature of 70 ºC under the stress level of 3.2 kPa. The percent difference in recovery under different stress levels increased as the temperature increased. The difference reached 100% at the test temperature of 70 ºC.

The non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr\text{-}diff}$) of base asphalt binder after RTFO aging are shown in Figure 5-24 (b). The J_{nr} of asphalt binder was increased significantly with the increase in the test temperature. The J_{nr} increased 12 times when the test temperature increased from 52 ºC to 58 ºC. The viscous percentage of asphalt binder increased as the temperature increased, the elastic recovery property was decreased, and thus the J_{nr} increased. The $J_{nr\text{-}diff}$ under different stress levels increased with the increase of test temperature.



(a) Average percent recovery

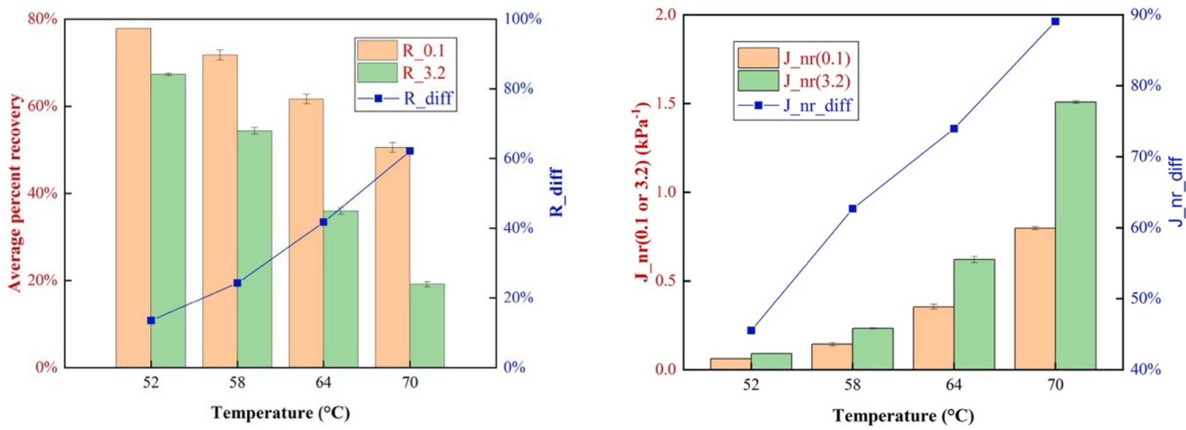


(b) Creep compliance

Figure 5-24. The average percent recovery and creep compliance of base asphalt after RTFO aging

The average percent recovery and percent difference in the recovery of base asphalt binder with rubber after RTFO aging is shown in Figure 5-25 (a). The base asphalt binder with rubber failed at 70 °C, according to AASHTO M 332-14. Under the 0.1 kPa stress level, the average percent recovery of base asphalt binder with rubber reduced from 77.84% to 50.53% when the test temperature increased from 52 ºC to 70 ºC. The average percent recovery of asphalt binder increased significantly after mixing with rubber. The asphalt binder with the rubber still had good elastic recovery property even at high temperatures and high stress level. Mixing asphalt binder with rubber could increase the elastic recovery property of asphalt binder, thus increased the rutting resistance of asphalt binder at high temperatures. The percent difference in recovery under different stress levels of asphalt binder with rubber was lower than that of the base asphalt binder. At the test temperature of 52 ºC and 58ºC, the percent difference in the recovery of base asphalt binder were 42.28% and 63.55%, respectively. But the percent difference in the recovery of base asphalt binder with rubber were 13.52% and 24.25%, respectively. The influence of stress level on the base asphalt binder with rubber was minimal at the test temperature of 52 ºC.

The J_{nr} and $J_{nr\text{-}diff}$ of base asphalt binder with rubber after RTFO aging are presented in Figure 5-25 (b). The J_{nr} of asphalt binder with rubber was lower than that of the base asphalt binder. The J_{nr} of the base asphalt binder was 7.5 times higher than that of the asphalt binder with rubber at the stress level of 0.1 kPa. At the stress level of 3.2 kPa, the J_{nr} of base asphalt binder was six times higher. The $J_{nr\text{-}diff}$ of asphalt binder with rubber was higher than that of the base asphalt binder. The J_{nr} of asphalt binder with rubber was lower, and a slight increase of J_{nr} had a significant influence on the percent difference under different stress levels. The elastic property and deformation resistance of the asphalt binder was significantly increased after mixing with rubber.

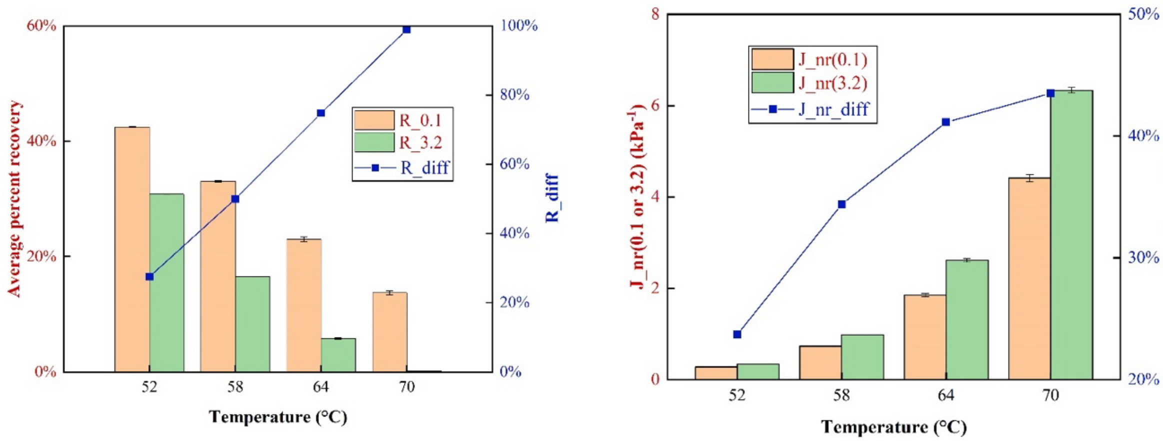


(a) Average percent recovery (b) Creep compliance

Figure 5-25. The average percent recovery and creep compliance of base asphalt with rubber after RTFO aging

The average percent recovery and percent difference in the recovery of the asphalt binder extracted from 5E1 CTRL asphalt mixture after RTFO aging are shown in Figure 5-26 (a). The asphalt binder extracted from 5E1 CTRL asphalt mixture failed at 70 °C, according to AASHTO M 332-14. By comparing Figure 5-24(a) and Figure 5-26(a), the average percent recovery of asphalt binder extracted from 5E1 CTRL asphalt mixture was higher than that of the base asphalt binder, especially at high stress level (3.2 kPa) condition. The aged asphalt binder from the RAP increased the stiffness of the extracted asphalt binder, thus increased the deformation resistance of the extracted asphalt binder. The percent difference in recovery under different stress levels increased linearly with the increase of test temperature. The percent difference in the recovery of asphalt binder extracted from 5E1 CTRL asphalt mixture was lower than that of the base asphalt binder at the same test temperature.

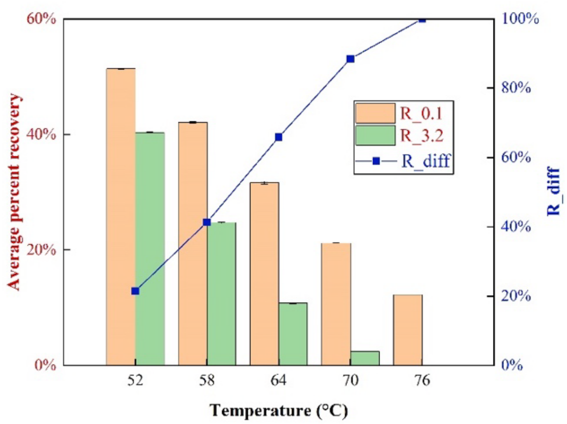
The J_{nr} and $J_{nr\text{-}diff}$ of the asphalt binder extracted from 5E1 CTRL asphalt mixture after RTFO aging is presented in Figure 5-26 (b). The J_{nr} of asphalt binder extracted from 5E1 CTRL asphalt mixture was lower than that of the base asphalt binder. The J_{nr} of the base asphalt binder was 1.5 times higher than that of the asphalt binder extracted from 5E1 CTRL asphalt mixture. The $J_{nr\text{-}diff}$ of asphalt binder extracted from 5E1 CTRL asphalt mixture was lower than that of the base asphalt binder. The aged asphalt binder from RAP increased the capability of asphalt binder to sustain the heavy load.



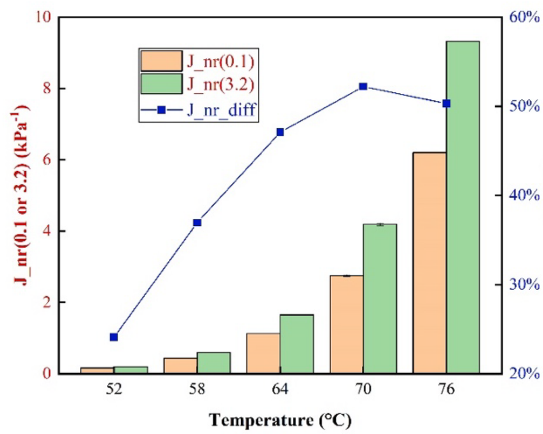
(a) Average percent recovery (b) Creep compliance

Figure 5-26. The average percent recovery and creep compliance of 5E1 CTRL mixture extracted binder after RTFO aging

The average percent recovery and percent difference in the recovery of the asphalt binder extracted from 5E1 Rubber asphalt mixture after RTFO aging is shown in Figure 5-27 (a). The asphalt binder extracted from 5E1 Rubber asphalt mixture failed at 76 °C, according to AASHTO M 332-14. The fail temperature of asphalt binder extracted from 5E1 Rubber asphalt mixture was 6 °C higher than that base asphalt with rubber and extracted asphalt binder from 5E1 CTRL asphalt mixture. The aged asphalt binder from the RAP increased the stiffness of the extracted asphalt binder, the rubber in the asphalt binder increased elastic property of the asphalt binder. The coupling effect of aged asphalt binder and rubber addition increased the property of the asphalt binder to sustain heavy stress load. The average percent recovery at 0.1 kPa stress levels decreased linearly with the increase of test temperature. The average percent recovery of asphalt binder extracted from 5E1 Rubber asphalt mixture was higher than that from 5E1 CTRL asphalt mixture. But the average percent recovery of asphalt binder extracted from 5E1 Rubber asphalt mixture was lower than that of base asphalt binder with rubber. The aged asphalt binder from the RAP decreased the elastic recovery property of the extracted asphalt binder. The percent difference in the recovery of asphalt binder extracted from 5E1 Rubber asphalt mixture was lower than that from 5E1 CTRL asphalt mixture at the same test temperature.



(a) Average percent recovery

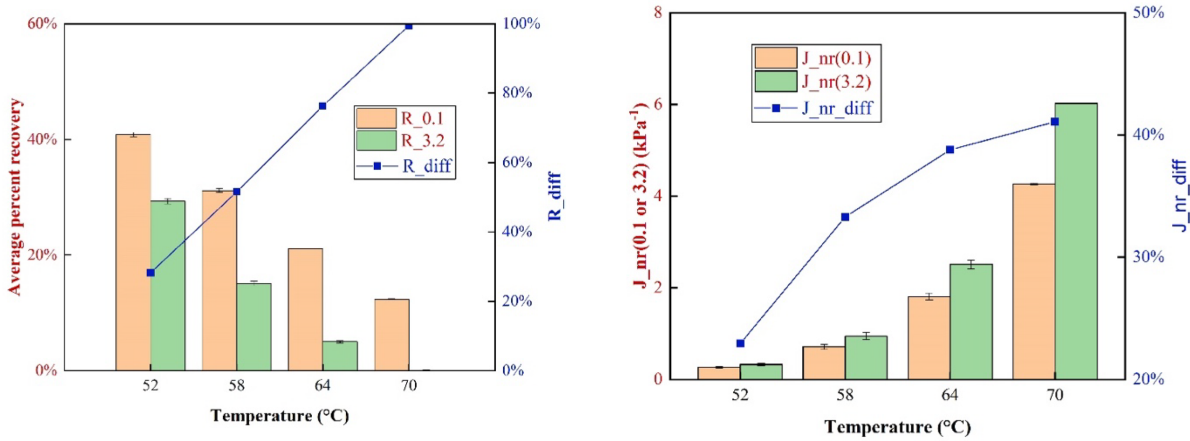


(b) Creep compliance

Figure 5-27. The average percent recovery and creep compliance of 5E1 Rubber mixture extracted binder after RTFO aging

The J_{nr} and $J_{nr\text{-}diff}$ of the asphalt binder extracted from 5E1 Rubber asphalt mixture after RTFO aging is presented in Figure 5-27 (b). The J_{nr} of asphalt binder extracted from 5E1 Rubber asphalt mixture was lower than that from 5E1 CTRL asphalt mixture. The J_{nr} of the asphalt binder extracted from 5E1 CTRL asphalt mixture was 1.5 times higher than that of the asphalt binder extracted from 5E1 Rubber asphalt mixture. The permanent deformation resistance of extracted asphalt binder under repeated load was increased after adding rubber. The J_{nr} of asphalt binder extracted from 5E1 Rubber asphalt mixture was higher than that of base asphalt binder with rubber. The elastic property of the extracted asphalt binder was weakened by the aged asphalt binder from RAP, thus increased the J_{nr} of extracted asphalt binder. The $J_{nr\text{-}diff}$ of asphalt binder extracted from 5E1 Rubber asphalt mixture was similar to that from 5E1 CTRL asphalt mixture at low temperatures.

The average percent recovery and percent difference in the recovery of the asphalt binder extracted from LVSP CTRL asphalt mixture after RTFO aging is shown in Figure 5-28 (a). The asphalt binder extracted from the LVSP CTRL asphalt mixture failed at 70 °C, according to AASHTO M 332-14. The average percent recovery at different stress levels decreased linearly with the increase of test temperature. The difference between the average percent recovery of asphalt binder extracted from LVSP CTRL asphalt mixture and that from 5E1 CTRL asphalt mixture was minimal. The influence of aged asphalt binder from RAP was decreased after adding rubber and after RTFO aging procedure. The percent difference in recovery increased linearly with the increase of test temperature.



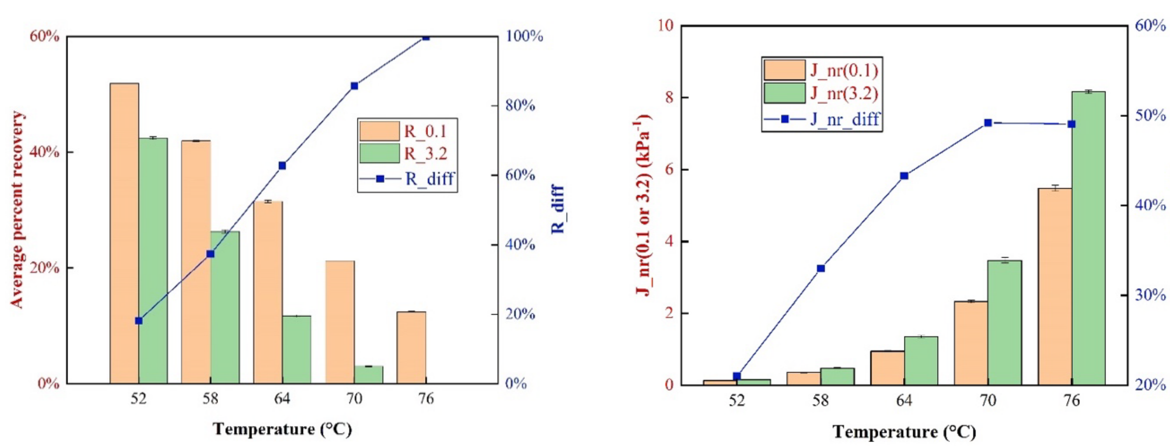
(a) Average percent recovery (b) Creep compliance

Figure 5-28. The average percent recovery and creep compliance of LVSP CTRL mixture extracted binder after RTFO aging

The Jnr and $J_{nr\text{-}diff}$ of the asphalt binder extracted from LVSP CTRL asphalt mixture after RTFO aging are shown in Figure 5-28 (b). The Jnr of asphalt binder extracted from the LVSP CTRL asphalt mixture was slightly lower than that from the 5E1 CTRL asphalt mixture. The aged asphalt binder content from the RAP was higher in LVSP CTRL asphalt mixture, and the stiffness of the asphalt binder extracted from the LVSP CTRL asphalt mixture was higher. The deformation resistance of extracted asphalt binder from LVSP CTRL asphalt mixture under repeated load was slightly increased. The $J_{nr\text{-}diff}$ of the asphalt binder extracted from the LVSP CTRL asphalt mixture was lower than that from the 5E1 CTRL asphalt mixture.

The average percent recovery and percent difference in the recovery of the asphalt binder extracted from LVSP Rubber asphalt mixture after RTFO aging is shown in Figure 5-29 (a). The asphalt binder extracted from the LVSP Rubber asphalt mixture failed at 76 °C, according to AASHTO M 332-14. The fail temperature of asphalt binder extracted from the LVSP Rubber asphalt mixture was 6 °C higher than that from the LVSP CTRL asphalt mixture. The average percent recovery at different stress levels decreased linearly with the increase of test temperature. The average percent recovery of asphalt binder extracted from the LVSP Rubber asphalt mixture was 10% higher than that from the LVSP CTRL asphalt mixture. The difference between the average percent recovery of asphalt binder extracted from the LVSP Rubber asphalt mixture and that from the 5E1 Rubber asphalt mixture was minimal. The influence of aged asphalt binder from RAP was decreased after adding rubber and after RTFO aging procedure. But the average percent recovery of asphalt binder extracted from the LVSP Rubber asphalt mixture was lower than that of base asphalt binder with rubber. The aged asphalt binder from the RAP decreased the elastic recovery property of the extracted asphalt binder. The percent difference in the recovery of asphalt binder extracted from the LVSP Rubber asphalt mixture was lower than that from LVSP CTRL asphalt mixture at the same test temperature.

The Jnr and $J_{nr\text{-}diff}$ of the asphalt binder extracted from LVSP Rubber asphalt mixture after RTFO aging is shown in Figure 5-29 (b). The Jnr of asphalt binder extracted from the LVSP Rubber asphalt mixture was slightly lower than that from the 5E1 Rubber asphalt mixture. The higher aged asphalt binder content from the RAP in LVSP Rubber asphalt mixture increased the deformation resistance of asphalt binder under repeated load. The Jnr of asphalt binder extracted from 5E1 Rubber asphalt mixture was lower than that from 5E1 CTRL asphalt mixture. The Jnr of the asphalt binder extracted from LVSP CTRL asphalt mixture was approximately two times higher than that of the asphalt binder extracted from LVSP Rubber asphalt mixture. The permanent deformation resistance of extracted asphalt binder under repeated load was increased after adding rubber. The $J_{nr\text{-}diff}$ of the asphalt binder extracted from the LVSP Rubber asphalt mixture was lower than that from the 5E1 Rubber asphalt mixture.



(a) Average percent recovery

(b) Creep compliance

Figure 5-29. The average percent recovery and creep compliance of LVSP Rubber mixture extracted binder after RTFO aging

The traffic level of different binders under different temperatures is shown in Figure 5-30. The traffic condition of "F" means the samples failed at the test temperature, and the "S", "H", "V", and "E" represented the traffic for Standard, Heavy, Very heavy, and Extreme heavy. The detailed expiation of different traffic levels is shown in Table 5-1. The traffic level of base asphalt binder was very heavy at 52 ºC, but the rest binders all belonged to extreme heavy traffic conditions. The rubber or aged asphalt binder from RAP increased the capability of the pavement to sustain heavy traffic load. At 58 ºC, the traffic level of all extracted asphalt binder decreased to very heavy, the base asphalt binder reduced to heavy traffic condition, but the base asphalt binder kept the extreme heavy traffic condition. The extracted asphalt binder from 5E1 CTRL and LVSP CTRL asphalt binder reduced to the same traffic condition as the base asphalt binder at 64 ºC, which was standard traffic condition. The extracted asphalt binder from 5E1 Rubber and LVSP Rubber asphalt binder reduced to heavy traffic condition at 64 ºC. The base asphalt binder could sustain very heavy traffic condition at 64 ºC. At 70 ºC, only the extracted asphalt binder from 5E1 Rubber and LVSP Rubber asphalt binder reduced to standard traffic condition, the rest binders failed according to AASHTO M 332-14. The aged asphalt binder and rubber in extracted asphalt binder guaranteed the asphalt binder to sustain heavy traffic load, thus improved the permanent deformation resistance of asphalt binder.

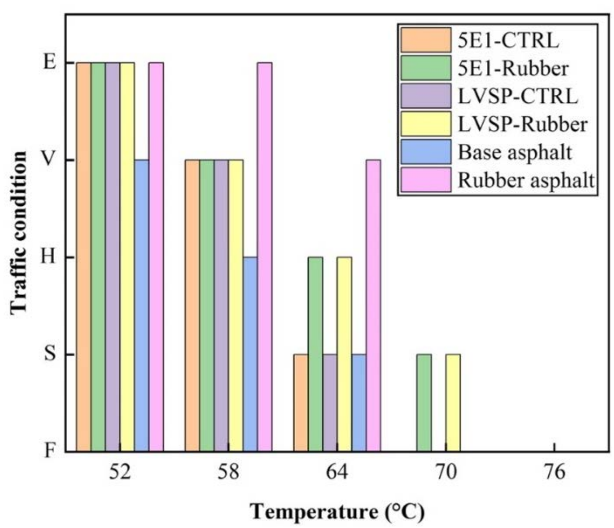


Figure 5-30. The traffic level of different binders under different temperatures

5.3.5 ABCD test analysis

The low temperature performance of different asphalt binders was assessed with the ABCD test. The crack temperature and the PG temperature based on ABCD is shown in Figure 5-31. The red dash line is the temperature line of -28 ºC, and the blue dot line is the temperature line of -34 ºC. The crack temperature of base asphalt and base asphalt binder with rubber was lower than -40 ºC, and the crack temperature of extracted asphalt binder was higher than -40 ºC. The crack temperature of the base asphalt binder with rubber was 3.9 ºC lower than that of the base asphalt binder. The PG temperature of base asphalt and base asphalt with rubber was lower than –34 ºC. The PG temperature of extracted asphalt binder was lower than -28 ºC but higher than -34 ºC. The PG temperature of the base asphalt binder with rubber was 3 ºC lower than that of the base asphalt binder. The rubber addition increased the low temperature cracking performance of the base asphalt. The low temperature cracking performance of the extracted asphalt binder was weaker than the base asphalt binder. The crack temperature and the PG temperature of extracted asphalt binder from asphalt mixture with rubber were lower than that from asphalt mixture without rubber. But only limited improvement was observed in the test. The difference between the different extracted asphalt binder was minimal. The aged asphalt binder from RAP decreased the low temperature cracking performance.

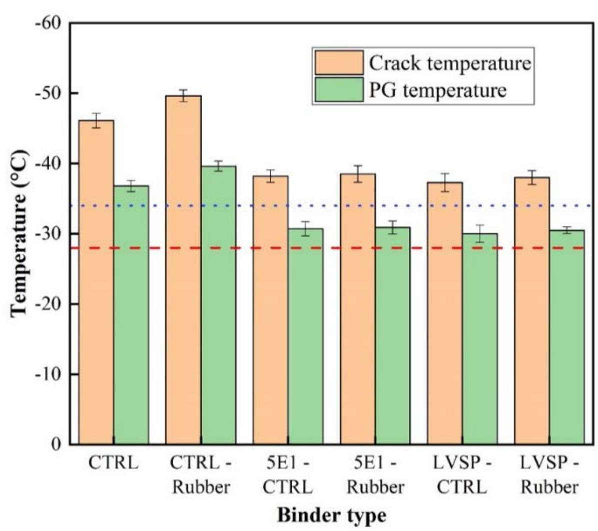


Figure 5-31. The crack temperature and PG temperature of different binders

Based on the DSR test and the ABCD test, the performance grade of the base asphalt was PG 58-34, the performance grade of the base asphalt with rubber was PG 70-34, the performance grade of the extracted asphalt binder from the leveling layer and surface layer without rubber was PG 64-28, and the performance grade of the extracted asphalt binder from the leveling layer and surface layer with rubber was PG 70-28.

The strain jump and fracture stress of different asphalt binders are shown in Figure 5-31. The strain jump and fracture stress of the base asphalt binder were significantly higher than the base asphalt binder with rubber and extracted asphalt binder. The crack temperature of the base asphalt was higher than the base asphalt binder with rubber. The base asphalt binder with rubber had lower strain jump and fracture stress even at a lower temperature. The stress dissipation capacity of the base asphalt binder with rubber was significantly better than that of the base asphalt. The extracted asphalt binder had a lower strain jump and fracture stress, but the crack temperature of extracted asphalt binder was lower. The difference between different extracted asphalt binders was minimal. The strain jump and fracture stress were highly related to the crack temperature of the asphalt binder. The strain jump and fracture stress of asphalt binder with rubber were slightly higher than that of extracted asphalt binder. The improvement effect of rubber on the low temperature cracking performance of asphalt binder was significant.

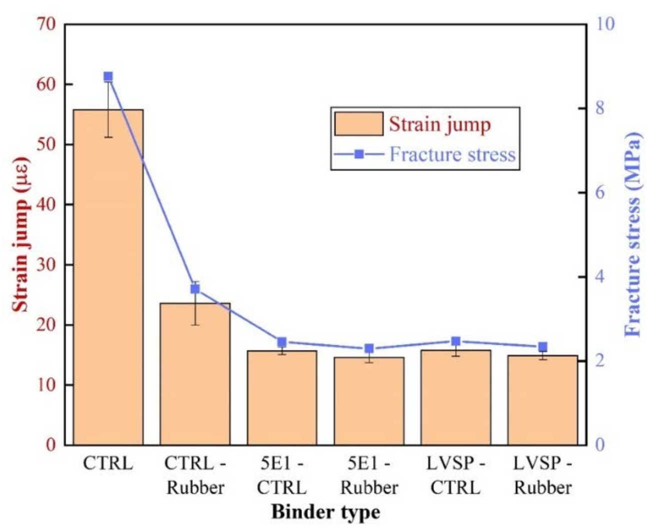


Figure 5-32. The strain jump and fracture stress of different binders

5.4 Summary

Four types of asphalt binders were extracted from field mixed asphalt mixtures, base asphalt binder, and base asphalt binder mixed with rubber were prepared. The short term aging, long term aging, dynamic shear rheological property, multiple Stress Creep and Recovery property, and low temperature cracking property of different types of asphalt binders were evaluated with the RTFO, PAV, DSR, MSCR, and ABCD. The main findings are as follows.

(1) The modified extraction procedure was proposed to improve the accuracy of the extraction procedure. The difference between the asphalt binder and the extracted asphalt binder had a good correlation.

(2) The gradation of the extracted asphalt mixture fits well with the original mixture design gradation. The asphalt binder content was slightly lower than the design content, and rubber particles that did not interact with asphalt binder were washed out and retained in the aggregate.

(3) Based on the DSR test and the ABCD test, the performance grade of the base asphalt was PG 58-34, the performance grade of the base asphalt with rubber was PG 70-34, the performance grade of the extracted asphalt binder from the leveling layer and surface layer without rubber was PG 64-28, and the performance grade of the extracted asphalt binder from the leveling layer and surface layer with rubber was PG 70-28.

(4) The aged asphalt binder and rubber in extracted asphalt binder guaranteed the asphalt binder to sustain heavy traffic load, thus improved the permanent deformation resistance of asphalt binder.

CHAPTER 6: THE FIELD APPLICATION OF RUBBERIZED ASPHALT WITH THE DRY PROCESS

6.1 Introduction

The project paved a trail road in Dickinson county in Michigan. The temperature during winter is low, and the annual snow is moderate. Based on the weather data from the local airport weather station, the temperature history of different months in Dickinson County is shown in Figure 6-1. Figure 6-1 displays the average high temperature, average low temperature, record high temperature, and record low temperature of different months. The average high temperature and the average low temperature reached the maximum value in June, which are 27ºC and 13ºC, respectively. The average high temperature and the average low temperature reached the minimum value in January, which are -4ºC and -16ºC, respectively. The record high temperature was 40ºC in June, and the record low temperature was -39ºC in Feburary.

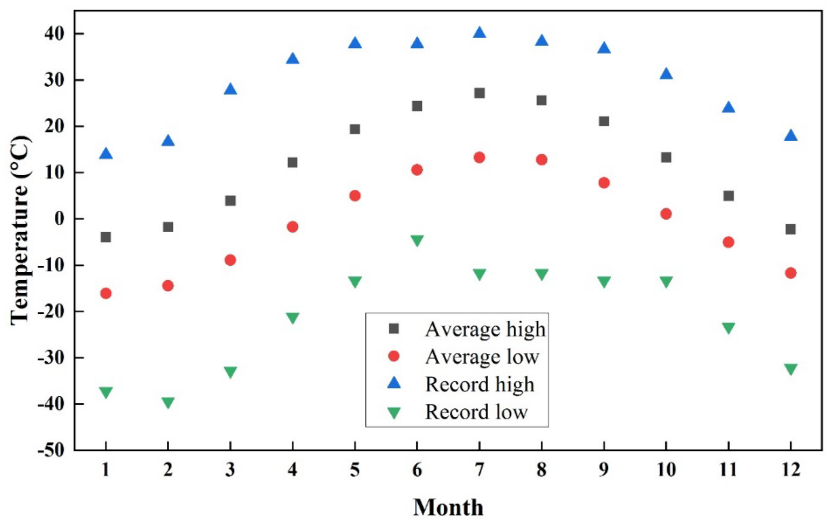


Figure 6-1. The monthly average temperature history in Dickinson county

The annual minimum temperature of the last 11 years based on the statistical data from the weather station is displayed in Figure 6-2. In the last 11 years, the minimum temperature of 6 years was lower than -28 ºC. The minimum temperature reached -32.2 ºC in the year of 2014. The project used the PG 58-34 base asphalt binder, and the asphalt type met the minimum temperature requirement.

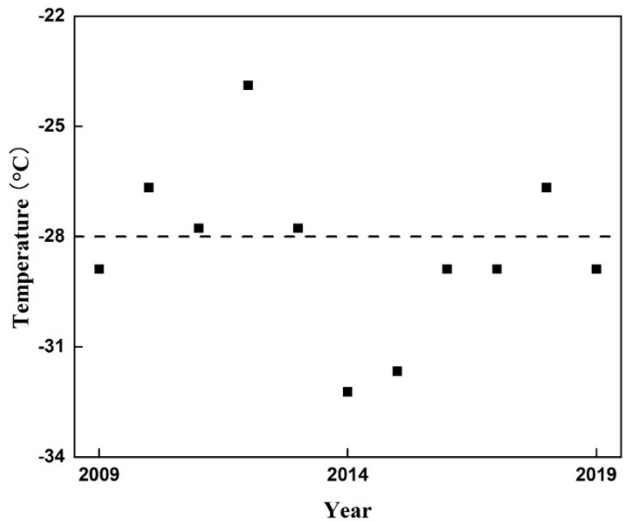


Figure 6-2. The annual minimum temperature of the last 11 years

The monthly average precipitation and snowfall history are shown in Figure 6-3. The maximum monthly average precipitation was 91.9 mm in September, and the minimum monthly average precipitation was 24.9 mm in February. On average, there was snowfall for seven months and no snowfall during June and October. The average snowfall during December and January both reached 3.56 m. The annual average snowfall was 1.5m based on the weather station statistic data.

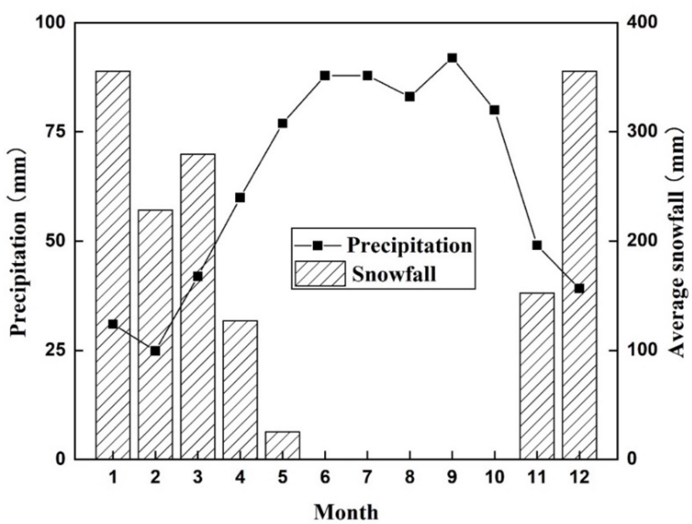


Figure 6-3. The monthly average precipitation and snowfall history

6.2 The field construction of rubberized asphalt mixture with the dry process

6.2.1 Preparation before the construction

Before the construction, the surface condition of the pavement was investigated and evaluated. Due to the influence of low temperature and the snowfall in the construction site, severe moisture damage, fatigue cracking, and low temperature cracking occurred on the surface of the pavement before the rehabilitation, as shown in Figure 6-4.

(a) low temperature cracking and fatigue cracking

(b) Moisture damage and fatigue cracking

Figure 6-4. Moisture damage, fatigue cracking, and low temperature cracking of the pavement before rehabilitation

In order to reduce the influence of moisture damage, fatigue cracking, and low temperature cracking on the new pavement, the upper layer of the old asphalt mixture was processed as the base layer of the new pavement. The surface 2-inch depth old pavement was crushed with a reclaimer machine (Figure 6-5 (a)). Then water was sprayed on the top of the crushed surface of the pavement in order to facilitate the compaction procedure (Figure 6-5 (b)). The reclaimed asphalt pavement was then compacted, and the surface of the old pavement was smoothed after compaction (Figure 6-5 (c) and (d)). Finally, the compacted old asphalt mixture surface layer was used as the base layer (Figure 6-5 (e)). The boundary of the new pavement was set in order to control the construction quality (Figure 6-5 (f)).

(a) Surface layer milling

(b) Water spray after milling

(c) Compaction of the old surface layer

(d) Leveling after compaction

(e) Old surface layer as the base layer

(f) Boundary control of the pavement

Figure 6-5. The processing procedure of the old pavement

6.2.2 Construction procedure

The hot mix asphalt plant has an independent cold aggregate feeding system (Figure 6-6 (a)) and a RAP feeding system (Figure 6-6 (b)). The cold aggregate and RAP materials were fed and heated in a drying drum according to the grading design mass percentage to remove the water in aggregate and RAP materials (Figure 6-6 (c)). The rubber construction process used in this research project was the dry process, an independent rubber feeding system was adopted at the mixing station (Figure 6-6 (d)). In order to control the rubber content during the mixture production procedure, the rubber feeding system was connected with the control room system, and the rubber particle feeding rate was controlled according to the amount of asphalt mixture produced in the asphalt plant. After the preparation of the mixture is completed, the mixture was elevated into the storage silo with a conveyor (Figure 6-6 (e)). The quality control in the hot asphalt mix plant is controlled by the control system (Figure 6-6 (f)).

(a) Aggregate feeding system

(b) RAP feeding system

(c) Drying drum

(d) Rubber feeding system

(e) Mixture storage silo

(f) Asphalt mixture control system

Figure 6-6. The asphalt mixture plant

Two layers were paved in the construction site, which included the leveling layer and surface layer, and the total length of the road was 1.9 miles. In order to compare the influence of rubber on different pavement structure design, the trail road was divided into three different sections, and the length of each section is 3,333 ft, as shown in Figure 6-7. The trail section #1 was asphalt pavement without the rubber. The trail section #2 used conventional asphalt mixture as the leveling layer and asphalt mixture with rubber as the surface layer. The trial section #3 was asphalt pavement with two layers using rubber.

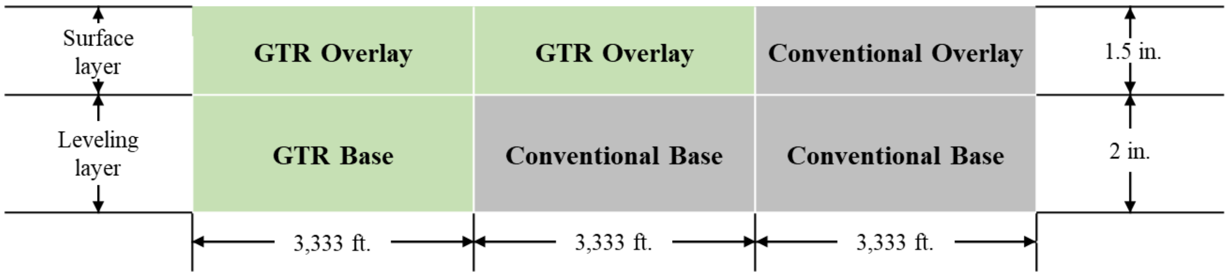


Figure 6-7. The asphalt pavement structure setting

During the construction process, the leveling layer of the asphalt pavement was firstly paved and compacted (Figure 6-8 (a)). In order to control the construction quality, the thickness of the leveling layer was monitored during the paving procedure (Figure 6-8 (b)). The nuclear density gauge was adopted to control the real-time compaction quality (Figure 6-8 (c)). As the leveling layer and surface layer used the same method to control the thickness and compaction during the paving and compaction procedure, the thickness and compaction control of the surface layer asphalt mixture was not displayed. After the compaction of the leveling layer, the emulsified asphalt was applied before the construction of the surface layer to ensure the bonding performance between different asphalt mixture layers (Figure 6-8 (d)). The surface layer was paved and compacted after the application and setting of the emulsified asphalt tack coat(Figure 6-8 (e)), and the final pavement condition was shown in Figure 6-8 (f). The traditional pavement compaction quality could be achieved by compacting the pavement at high temperatures. However, due to the swelling characteristics of the asphalt mixture with rubber, the air voids in the pavement would increase after the compaction and thus influence the final compaction density of the pavement. After traditional compaction at high temperatures, additional compaction must be performed when the pavement temperature decreases to about 70 °C to eliminate the rubber swelling effect in the asphalt pavement.

(a) Leveling layer compaction

(b) Leveling layer thickness control

(c) Leveling layer compaction density control

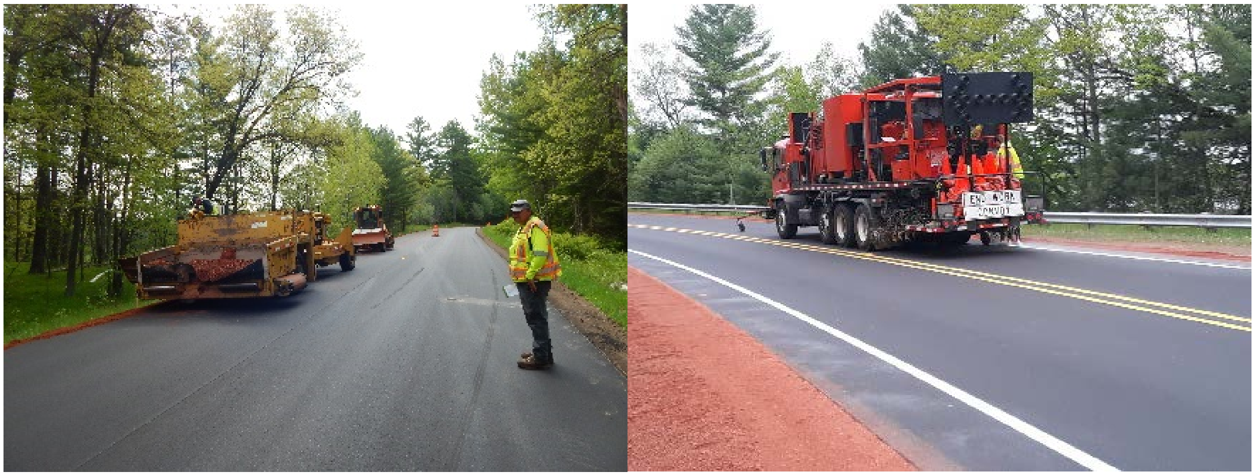
(d) Leveling emulsified asphalt application

(e) Surface layer compaction

(f) Final pavement condition

Figure 6-8. The asphalt mixture plant

After the completion of the construction, the graded crushed stone that met the state requirement was used as shoulder materials, and the shoulder was compacted with a roller compactor (Figure 6-9 (a)). Finally, road boundary markings were painted with road marking paint (Figure 6-9 (b)).



(a) Shoulder materials setting (b) Road mark setting

Figure 6-9. The pavement shoulder materials and road mark setting

6.3 The noise reduction evaluation of rubberized asphalt pavement with the dry process

The noise generated due to the interaction between tires and the road has a great impact on human health. The noise could decrease the judgment and reduce the work efficiency and productivity of human beings. What's more, the sleep of individuals could be significantly impacted by exposure to noise [40]. The response to different noise is different due to the intensity and frequency distribution differences between different noise. Human hearing is usually more sensitive to higher tones, such as emergency siren sounds or tire brake sounds, but less sensitive to lower tones, such as stereo pitch [41]. At the same sound intensity, higher tones have a greater influence on people than lower tones [42]. Prolonged exposure to high-intensity environmental noise can cause hearing impairment and thus affect human health. The unit of noise measurement is the decibel (dB), and the traffic noise normally varied between 55 dB and 80 dB [43]. The installation of sound insulation walls along roads could effectively reduce the influence of road noise on houses next to roads, but quieter roads can reduce the cost of sound insulation walls installation [44].

Under the noise logarithmic decibel scale, the noise energy is doubled when the noise increased 3 dB. The relationship between noise dB and noise energy is shown in Equation 6.1. According to Equation 5.1, when the noise decibel increased 10 dB, the noise energy increased by 10 times, and when the noise decibel increased by 20 dB, the noise energy increased by 100 times.

$$\Delta L_W = 10^{(\Delta I/10)} \qquad (6.1)$$

In which, ΔLw is the noise energy increase rate, and ΔI is the noise decibel increment.

6.3.1 Measuring instruments and measurement conditions

In this study, a noise meter that met the requirements of ANSI S1.4 Type 2 and IEC61672-1 Class 2 was used to evaluate the noise of asphalt pavement. The noise meter was calibrated at 1kHz and 94dB. The noise meter has a resolution of 0.1 dB and an accuracy of ± 1.5 dB. The distance between the noise meter and the vehicle during the measurement is 15 ft [6]. The noise on different road surfaces was measured, as shown in Figure 6-10. The noise measurement vehicle was a Dodge Grand Caravan, and the noise measurement was conducted under five different speeds: 20MPH, 30MPH, 40MPH, 50MPH, and 60MPH. A total of four noise data were collected at each speed. The temperature during the measurement was 15ºC, the wind speed was 27km / h, and the background noise was 46dB.

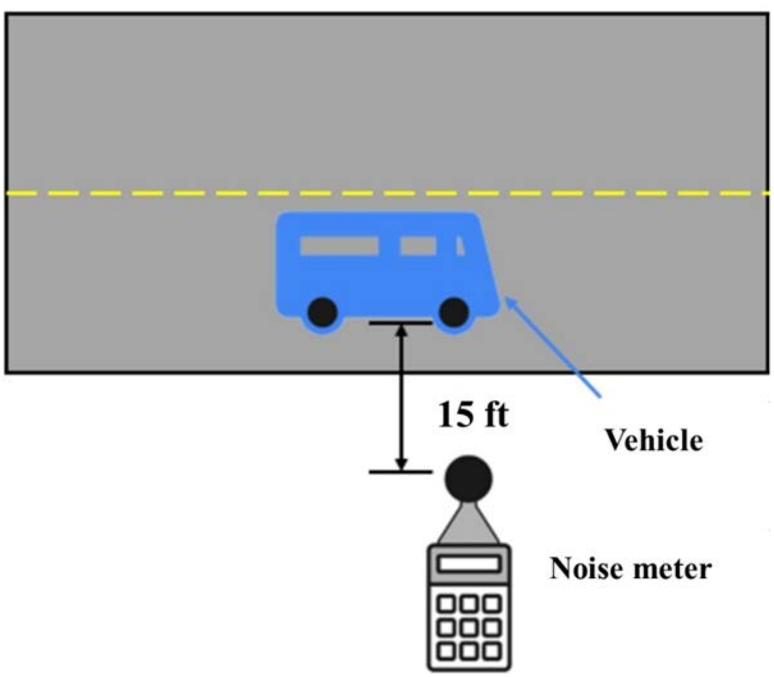


Figure 6-10. The noise measurement setting

6.3.2 Noise measurement results

This study measured the noise levels of three types of asphalt pavement test sections and old asphalt pavement. The noise results are displayed in Figure 6-11. The asphalt pavement noise increased linearly with the increase of vehicle speed. Compared with the old asphalt pavement, the noise reduced more than 5.5 dB under all vehicle speed conditions, and the noise reduced by 7.1 dB at the speed of 60 MPH. According to Equation 5.1, the noise energy of the newly paved asphalt pavement reduced to less than 28.3% of the noise energy of the old asphalt pavement. At the speed of 60 MPH, the noise energy reduced to 19.5% of the noise energy of old asphalt pavement.

The surface layers of road section # 2 and road section # 3 were all rubberized asphalt mixtures, and the noise results of the two road sections were similar. The surface layer of road section # 1 was the conventional asphalt mixture. Compared with road section # 2 and road section # 3, the noise of road section # 1 without rubber was higher. The noise of road sections # 2 and road section # 3 was 2dB lower than that of road section # 1, except the noise at the speed of 20 MPH. At the design speed of 50 MPH, the noise reduced more than 3dB, and the noise energy reduced more than 50%. The test results showed that the rubber modification could significantly reduce the noise of the asphalt mixture, compared with the conventional asphalt mixture the noise energy reduced more than 50% at the design speed of 50 MPH.

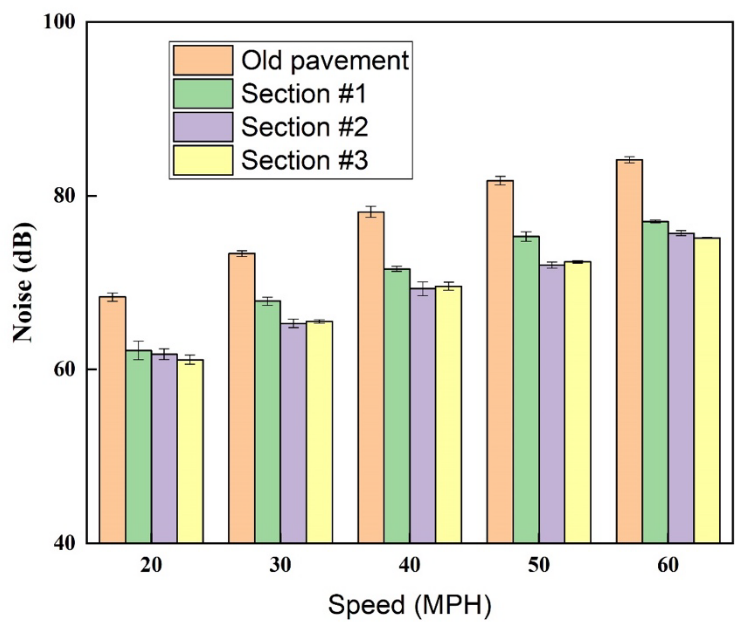


Figure 6-11. The noise test results of different types of pavement under different speeds

6.4 Evaluation of cost and environmental benefits of rubberized asphalt mixture with the dry process

6.4.1 Cost comparison of different types of asphalt mixtures

The design amounts of the leveling layer asphalt mixture without rubber, the surface layer asphalt mixture without rubber, the rubberized leveling layer asphalt mixture, and the rubberized surface layer asphalt mixture were 2230 tons, 840 tons, 1120 tons, and 1670 tons, respectively. The actual usage were 2114.21 tons, 861.81 tons, 1068.37 tons, and 1593.83 tons, respectively. The performance comparison of the rubberized asphalt mixture and conventional asphalt mixture presented in the previous chapter showed that rubber modification could significantly improve the high temperature rutting resistance and low temperature crack resistance of the asphalt mixture. The service life of the asphalt mixture could also be extended, and the maintenance costs of the asphalt mixture could be reduced. From the perspective of the life cycle assessment of asphalt mixtures, rubber modification was found to reduce the cost of asphalt mixtures, thus increasing the economic benefits of asphalt pavements. In modified asphalt projects, rubber is about $2-$3 per mix ton less expensive than polymers (for projects with a 2-grade bump). When compared to standard hot mix, our process costs about $3-$4 per mix ton more than the standard hot mix, but it can be paved thinner and will last longer than a standard hot mix.

6.4.2 Environmental benefits of rubberized asphalt mixture with the dry process

Scrap tires bring a significant influence on the environment, the heavy metals and chemicals contents pollute the environment again after the tires are degraded, and the surrounding soil and water quality get contaminated by scrap tires. Waste tires have become the main type of waste in landfills, which became a critical issue with waste disposal. Scrap tires are extremely flammable materials so they pose potential fire hazards. It is extremely difficult to extinguish once the tires catch fire. Scrap tires can easily become breeding grounds for mosquitoes and other pests, which in turn poses a serious health threat to human beings.

By using crumb rubber made from scrap tires in asphalt mixtures, the environmental impact of scrap tires can be significantly reduced. The weight of a scrap car tire is about 20 lbs. (9 kg) and about 40% by the weight of a scrap car tire is rubber. In this way, one scrap car tire contains about 8 lbs. (3.6 kg) rubber. The asphalt content of the surface layer asphalt mixture is 5.9%, the asphalt content of the leveling layer asphalt mixture is 4.2%, the rubber particles account for 10% of the mass of the asphalt binder. The tire consumption of the surface layer and the leveling layer rubber modified asphalt mixture are shown in Table 6-1 below. The surface layer of the asphalt mixture consumed 2370 scrap tires, and the leveling layer of the asphalt mixture consumed 1131 scrap tires. The total number of scrap tires used in this project is calculated to be 3501. If we assume average typical light truck tire yield 18 lbs. per tire, the tire use numbers should be 947 tires in the surface layer and 452 tires in the leveling layer.

Table 6-1. The estimated Tires used in the project

Asphalt Mixture	Estimated Asphalt mix (tons)	Estimated Asphalt binder (tons)	GTR used (tons)	Equivalent Number of Scrap Tires
Surface layer	1446	85.3	8.53	2370
Leveling layer	969	40.7	4.07	1131

6.5 Conclusions

This chapter introduced the engineering application of rubberized asphalt mixture with the dry process in wet freeze climates. The construction process of rubberized asphalt mixture with the dry process is introduced in detail. The noise reduction effect of different asphalt mixture types was compared and analyzed. Finally, the economic and environmental benefits of rubber modified asphalt mixture were evaluated. The conclusions are as follows.

(1) The construction experience of rubberized asphalt mixture with the dry process in this project can provide guidance for the application of rubberized asphalt mixture in wet freeze climates. Rubberized asphalt mixture has an elastic recovery phenomenon during paving and compaction procedure. Additional compaction is needed as the road temperature decreased to about 70ºC, which could guarantee the compact density of the asphalt mixture meet the standard.

(2) The noise of asphalt mixture pavement increased linearly with the increase in vehicle speed. The rubber modification with the dry process can significantly reduce the noise of the asphalt mixture, and the noise energy reduced to less than 50% at the design speed of 50 MPH.

(3) The rubberized asphalt mixture with the dry process could extend the service life of the asphalt mixture without increasing the construction cost, which thus reduced the maintenance cost

of the asphalt mixture. From the perspective of the life cycle assessment of asphalt mixtures, rubber modification can reduce the cost of asphalt mixtures, and thus help increase the economic benefits of asphalt pavements.

(4) The rubberized asphalt mixture with the dry process could significantly reduce the influence of waste tires on the environment. The total number of scrap tires used in this project is calculated to be 3501. Using rubber could not only improve the performance of asphalt mixtures but also reduces the environmental impact of scrap tires.

CHAPTER 7: PROJECT SUMMARY, DISCUSSION, AND CONCLUSIONS

KEY PROJECT SUMMARY

This evaluation focused on the use of engineered crumb rubber dry process modification in a light to medium-duty asphalt road in Northern Michigan. The project elements are outlined below.

Project Plan and Project Location

1. This project was designed to compare the cold weather performance of three sections of roadway in the Upper Peninsula of Michigan. The roadway sections included standard dense-graded mix designs and rubber-modified dense-graded mix designs.
2. The three sections of the roadway included a leveling layer and the overlay, all placed on milled, compacted RAP. The first section included a 2” thick standard 12.5 mm dense-graded mix design as a leveling layer and a 1.5” thick standard 9.5 mm dense-graded overlay. The second section included a 2” thick standard 12.5 mm dense-graded mix design as a leveling layer and a 1.5” thick rubber modified 9.5 mm dense-graded overlay. The third section included a 2” thick rubber modified 12.5 mm dense-graded mix design as a leveling layer and a 1.5” thick rubber modified 9.5 mm dense-graded overlay. In each rubber modified mix design, rubber was added at 10% by weight of the virgin binder.
3. The selected site for pavement evaluations was County Route 609 in the Upper Peninsula of Michigan, just south of Iron Mountain. Route 609 is a light-duty traffic road with some truck traffic.
4. The area climate exhibits extreme low temperatures and heavy winter snowfall. Severe thermal cracking, moisture, and salt intrusion into pavements are common in the area.
5. The existing pavement was in poor condition, with evidence of significant thermal and fatigue cracking. There was evidence of severe moisture damage as well. HMA base crushing and shaping was completed prior the asphalt construction.

Mixture Design and Testing

1. Two mixture designs were developed and evaluated for this project: a leveling layer (NMAS 12.5 mm) and a dense-graded surface layer (NMAS 9.5 mm). Each of the two designs were developed and produced with and without rubber content.
2. The design asphalt binder contents for the leveling layer and the surface layer asphalt mixture were 4.2% and 5.9%, respectively.
3. The RAP content of the mixes was 25% for the leveling layer and 17% in the surface layer. The useable binder content in the RAP is 4.15%.
4. PG58 -34 base binder was used in mix designs to meet the extreme cold weather condition.
5. Lab procedures for the mixtures using dry process rubber are not the same as standard asphalt mix designs. Crumb rubber absorbs binder lighter ends at mix temperatures above 132℃ （270 ℉), and the speed of binder uptake will be controlled by mix/binder temperature. In order to produce mixture designs to meet good performance in the field, lab designs should be developed in compliance with the manufacturer's lab procedures and specifications. Special attention should be paid to maintain proper mix temperatures, control sample swelling, and track curing time during sample preparation.
6. Unreacted rubber absorbs small amounts of the binder during mix production, storage, and transport. Small amounts of supplemental binder should be added in order to keep the mix design close to optimal binder content. The manufacturer's lab procedures and specifications regarding mix binder content should be reviewed and followed.
7. All four mix designs were developed with attention to the manufacturer's specifications and standard lab protocols. The mixes were subjected to a series of evaluations for comparative purposes. The following is a list of lab work summary:
    a. In a Hamburg test application, the addition of rubber to both mix designs significantly improved the rutting resistance and stability of each mix design. The creep and stripping slopes of both mixes were significantly improved with the addition of rubber. From a rutting and stripping perspective, all mix designs also showed improved results with a larger NMAS and reduced voids.
    b. The low temperature cracking characteristics for the four mix designs were evaluated through the use of the DCT test. With the addition of rubber, fracture energy went up approximately 20% in both mix designs (Leveling and Surface),

which means the rubber asphalt materials will resist low temperature cracks much better than the conventional asphalt mixtures.

c. From the DCT measurement, it can be observed the dry process rubber additions improved the elastic characteristics of the mix.

d. The base binder was PG58-34. Asphalt binders were extracted after production from the four mix designs. It was determined that the combination of RAP and rubber-modification produced binders with a performance grade of PG70-28 based on the ABCD and DSR test results. Rubber modification produced improved strain jump and fracture stress scoring in MSCR. Rubber modified binders had significantly improved stress dissipation capabilities, and it would support heavy traffic loads.

8. Our overall evaluation was that the addition of rubber enhanced the cold weather performance characteristics of the mix designs.

Mix Production and Placement

1. The design mixes were produced by Bacco Corporation using a portable plant located near the construction site.
2. Crushing and shaping of the existing pavement was completed and used as a base for new road construction.
3. Rubber material was injected with a fiber machine into a portable drum plant. Rubber feed rates were accurate to within less than 0.5% of the job mix formula during production. No material plant operational issues were reported during production.
4. A leveling course was laid and compacted, followed by an emulsified tack coat and a surface layer/overlay. After some adjustments during project startup, each rubber mix was produced and placed in compliance with the manufacturer's guidance on rubber content, mix temperature, and dwell time.
5. The rubber-modified mixes were transported and placed without any significant variations in normal operations during laydown and compaction. The rubberized pavements were compacted until the pavement temperature fell below 70℃ in order to make sure compaction was compliant. All finished pavements met the specified densities required by

the contract. The plant superintendent and field crews reported that the material was easy to produce and easy to place/compact (no separation, no build-up, no stickiness).

Post-Production Pavement Evaluations

1. The placement was completed in June of 2019.
2. Noise testing was performed on aged surface layers of standard asphalt pavement, the new unmodified surface asphalt, and the new surface asphalt modified by rubber. At various speeds, noise generated by the rubberized asphalt was lower to significantly lower than the new and aged asphalt pavements.
3. A performance evaluation of the pavement sections was completed in 2020. Researchers found that both the control and rubber modified pavements were performing well after one winter.
4. Additional pavement evaluations are planned for upcoming years.

OBSERVATIONS

1. The use of proper lab methods and procedures is critical in the mix design process. The manufacturer's specifications should be used during the mixture design and testing elements of a project.
2. The use of proper operational procedures is necessary to optimize field application and pavement performance. The manufacturer’s specifications should be consulted before production is started.
3. As noted, the crumb rubber swells on contact with the binder at temperatures above 132℃ (270 ℉) due to the uptake of binder lighter ends. Almost all of the swelling will occur over about a 10-minute period at mix temperatures of 163℃ (325 ℉). In order to avoid any swelling during compaction (which could impact pavement densities), the manufacturer’s specifications should be followed regarding plant temperatures and mix dwell times before placement and compaction. Placed pavements should be compacted at temperatures below 132℃ (270 ℉).
4. We would observe that when placing and compacting an overlay, the condition of the base pavement will play an important role in the performance of the overlay. Documenting base

pavement conditions is necessary in order to properly characterized new pavement performance over time.

CONCLUSIONS

We offer these conclusions:

1. The addition of 10% dry process rubber to the Leveling and Surface mix designs produced a two-grade PG high temperature bump.
2. The addition of 10% dry process rubber to the Leveling and Surface mix designs improved rutting and cracking (thermal and fatigue) resistance of those mix designs.
3. Lab procedures that control binder/mix temperatures, time, binder content, and gradation during mix preparation and evaluation are necessary in order to control dry process rubber reaction to the heated binder.
4. Lab test results suggest that the rubber-modified mix designs will perform better than standard hot mix designs in cold weather applications
5. Field production and engineering produced reliable, good quality rubberized mixes
    a. The dry process rubber injection process accurately fed rubber into the process during production.
    b. Field application of dry process rubber during production produced mixes that conformed to lab testing.
6. Initial field inspections – after one year of service – show that both the control and rubber modified pavements are holding up well. Additional evaluations are planned as the pavement ages.

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