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THE LABORATORY PERFORMANCE AND AGING CHARACTERISTICS OF
RUBBER MODIFIED ASPHALT MIXTURE USING A DRY PROCESS

By
Dongdong Ge

A DISSERTATION
Submitted in partial fulfillment of the requirements for the degree of
DOCTOR OF PHILOSOPHY
In Civil Engineering

MICHIGAN TECHNOLOGICAL UNIVERSITY

2020

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This dissertation has been approved in partial fulfillment of the requirements for the Degree of DOCTOR OF PHILOSOPHY in Civil Engineering.

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Preface

The dissertation is organized as follows. Introduction of crumb rubber modified asphalt and asphalt aging were described in Chapter 1. Chapter 2-4 evaluated the performance of the rubber modified asphalt mixture. The volumetric design of rubber modified asphalt mixture using the dry process was illustrated in Chapter 2. The high temperature rutting performance of rubber modified asphalt mixture using the dry process was evaluated in Chapter 3. The low temperature cracking performance of rubber modified asphalt mixture using the dry process was presented in Chapter 4. Chapter 5-6 investigated the asphalt binder extraction procedure and evaluated the extracted asphalt binder. The influences of TCE on the base asphalt and PAV aged asphalt binder were demonstrated in Chapter 5. The performance of asphalt binder extracted from rubber modified asphalt mixture using the dry process was presented in Chapter 6. The comparison between the extracted asphalt binder and base asphalt binder with rubber was studied in the chapter 6. Chapter 7 covered the summary, conclusion, and future research work.

Part of the work in Chapter 3 was accepted in the 4th International Conference on Transportation Geotechnics (ICTG) in Chicago, Illinois, USA. Part of the work in Chapter 4 was accepted in the RILEM International Symposium on Bituminous Materials – ISBM LYON 2020 in Lyon, France. Chapter 5 is from a previous article published by the Journal of Cleaner Production, Volume 232. The title of this paper is “The performance of asphalt binder with trichloroethylene: improving the efficiency of using reclaimed asphalt pavement” by Dongdong Ge, Zhanping You, Siyu Chen, Chaochao Liu, Junfeng Gao, and Songtao Lv.

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List of abbreviations

CRM	Crumb Rubber Modified
SMA	Stone Mastic Asphalt Mixture
OGFC	Open Graded Friction Course
RTFO	Rolling Thin Film Oven
PAV	Pressure Aging Vessel
DSR	Dynamic Shear Rheometer
MSCR	Multiple Stress Creep and Recovery
ABCD	Asphalt Binder Cracking Device
DCT	Disc Shaped Compact Tension
SCB	Semi-Circular Bend
CMOD	Crack Mouth Opening Displacement
FTIR	Fourier-transform Infrared Spectroscopy
TCE	Trichloroethylene
XRD	X-ray Diffraction
RAP	Reclaimed Asphalt Pavement
ISP	Intermediate shingle particle
NMAS	Nominal Maximum Aggregate Size
Pb	Design binder content
Va	Air void content
VFA	Voids Filled with Asphalt
VMA	Voids in the mineral aggregate
TSR	tensile strength ratio

LVSP	Leveling layer asphalt mixture
5E1	Surface layer asphalt mixture
TDF	Tire-derived fuel
SGC	Superpave gyratory compactor
HWTD	Hamburg wheel tracking device
LVDT	Linear variable differential transducer
m_R	Creep slope
SIP	Stripping inflection point
m_s	Stripping slope
N_s	Number of passes that SIP occurs
N_f	Number of passes that sample fails
ANOVA	Analysis of variance
SHRP	Strategic Highway Research Program
TI	Toughness index
WMA	Warm Mix Asphalt
Jnr	Non-recoverable creep compliance

Abstract

The accumulation of waste tires generates severe environmental issues. Using crumb rubber processed from waste tires on the pavement could relieve the pressure of waste tire on the environment and improve the performance of the pavement as well. The application of crumb rubber on the pavement using the dry process was proved to be a cost-effective strategy in some projects, however, the interaction between the rubber and asphalt binder was unknown in the rubber modified asphalt mixture using the dry process, especially when reclaimed asphalt pavement was adopted. The performance improvement of the rubber particle to the asphalt mixture was closely related to the interaction. The influence of aging on the characteristics of rubber modified asphalt mixture using the dry process is critical to its long-term performance. The scope of the dissertation is to evaluate the performance of rubber modified asphalt mixture using a dry process when reclaimed asphalt pavement was adopted. The influence of aging on the characteristics of the rubber modified asphalt mixture and the extracted asphalt binder were investigated.

This dissertation proposed the volumetric design of rubber modified asphalt mixture using the dry process. The aggregate gradation determination, rubber modified asphalt mixture preparation, design binder content determination, and moisture susceptibility evaluation were proposed. The quality control of the laboratory mixture design procedure was critical to guarantee the consistency of the asphalt mixture design in the laboratory and the results of mixture construction in the plant. The high temperature rutting performance and the low temperature cracking performance of plant mixed and laboratory compacted asphalt mixture was evaluated. The influence of various factors, which included mixture design parameters of the asphalt mixture (nominal maximum aggregate size, reclaimed asphalt pavement content, and asphalt content), rubber modification, mixture air void content, test temperature, and aging procedure, on the rutting and low temperature characteristics of asphalt mixtures were evaluated. Based on the evaluation of different assessment parameters, the rubber addition and the rubber

addition and aging were the two most critical factors that influenced the rutting and low temperature cracking performance of different asphalt mixtures.

The effect of the Trichloroethylene solvent in asphalt binder was investigated, and the extracted asphalt binder was assessed and compared with the base asphalt binder and rubber modified base asphalt binder. Both the existence of Trichloroethylene and the content of Trichloroethylene had a significant influence on the $|G^*|$ of unaged and PAV aged asphalt binder. The Trichloroethylene in the extracted asphalt binder significantly influenced the actual performance of the asphalt binder in the reclaimed asphalt binder, thus potentially compromising the success mixture design. The modified extraction procedure was proposed to improve the accuracy of the extraction procedure. The rubber particles in the asphalt mixture using the dry process were not totally interacted with asphalt binder, only the interacted rubber particles were extracted, and the influence of the interacted rubber particle was assessed. The aged asphalt binder weakened the low temperature performance of the extracted asphalt binder. 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.

1 Introduction

This chapter presents the literature review related to the topic of this dissertation on the aging performance of rubber modified asphalt mixture using the dry process. The crumb rubber modified asphalt binder was summarized. The aging of asphalt binder and asphalt mixture were discussed. Finally, the objectives of the dissertation were proposed based upon the knowledge gap and industry demand.

1.1 Crumb rubber modified (CRM) asphalt binder and asphalt mixture

1.1.1 Background

The growth of vehicle production increased the disposal of waste tires. The recycling of vehicle tires has attracted more and more attention because of the environmental issues during the waste tires disposal process. Using waste tires to produce crumb rubbers was proved to be a promising direction, which could be used to modify asphalt binder.

The crumb rubber with high elastic property absorbs loading energy and generates large deformation during the loading period, and recovers to original condition after loading was released. The high elastic characteristics of the crumb was originated from its molecular structure. Rubber is a kind of polymer materials and was also regarded as elastomer [1]. The elastomer rubber has the following characteristics: (a) the long-chain molecules guarantees the free rotation of molecules around the bonds which joins neighbor molecules. (b) The joined molecules by the cross-link with the chemical or mechanical method formed three-dimensional spatial structure network. (c) The force between the molecules are small that molecules could move freely even though the molecules are cross-linked [2].

As a viscoelastic material, the deformation response of rubber to load is influenced by the temperature and loading strain rate. The rubber displayed the elastic property at the low loading strain rate or above the ambient temperature. The deformation response of rubber to load was sensitive to the temperature under the elastic conditions. The elastic property

differences between the rubber and asphalt binder was due to the long-chain cross linked molecules in rubber [3]. Rubber has better ductility than asphalt binder at low temperature or high loading strain rates.

Compared with other polymer modified asphalt binders, the complicate mixing equipment was not strictly needed for CRM asphalt binder, and the restriction is negligible to modify asphalt binder. The cost of CRM asphalt binder is less, and the performance of CRM asphalt binder is comparable compared with other polymer modified asphalt binders. Asphalt binder modified with crumb rubber with the wet process could enhance the rutting resistance and fatigue cracking resistance of asphalt mixture [4]. The benefits of using CRM asphalt binder are less sensitive to the change of temperatures, better deformation resistance at high temperatures, improved aging resistance characteristics, and better fatigue distress resistance. The addition of crumb rubber improved the asphalt binder properties, which included penetration, softening point, and viscosity [5]. Jeong discovered that by adding 20% crumb rubber, the viscosity of asphalt binder increased 550%, and the stiffness increased 225% [6]. Rubber Pavement Association discovered that the tire noise decreased 50% if the rubber was used in open graded asphalt mixture. The life span of the pavement with CRM asphalt binder was expanded.

1.1.2 Rubber modification procedure

CRM asphalt binder and asphalt mixture is normally produced by modifying asphalt binders with crumb rubber from ground tires under certain temperature and durations. The rubber particles swelled when mixed with asphalt binder, and the particle size swelled up to 3-5 times after absorbed the light component of asphalt binder, then the rubber in asphalt binder decomposed [7]. The high temperature and low temperature performance of CRM asphalt binder were influenced by the rubber particle dispersion, the rubber dissolution in the molecule scale, and the interaction between the rubber particle and asphalt binder [8]. The rubber particles in asphalt binder will decompose when too high temperature or too long mixing duration were used to conduct the asphalt

modification. The depolymerization or devulcanization occur when rubber particle dissolve into asphalt binder. The viscosity of the rubber modified asphalt binder was decreased during the process [9]. Normally, two different strategies are adopted to use the crumb rubber in asphalt mixture, the dry process and the wet process.

1.1.2.1 Wet process

The wet processes modified the asphalt binder with crumb rubber before the binder is mixed with aggregate to produce mixture [10]. The high shear mechanism completely depolymerized rubber particles, and the rubber particles were totally digested into asphalt binder. The interaction between the rubber particle and asphalt binder during the wet process could significantly improve the rutting and cracking resistance of the pavement [11]. The viscosity of the asphalt binder was increased after modification with rubber using the wet process, thus increased the difficulty during the production and placement, and the production cost was also increased. One big problem during the application of the rubber asphalt mixture using the wet process is the segregation of rubber particles in the modified asphalt binder. Mechanical agitation is needed to guarantee the uniformity of rubber particle in the modified asphalt binder.

1.1.2.2 Dry process

The dry process mixed the crumb rubber with aggregate before contacting asphalt binder during the production of asphalt mixture. The crumb rubber is not completely reacted with asphalt binder during the dry process. No modification of the mixing equipment is needed in the plant to add rubber particle using the dry process. The rubber segregation in the asphalt mixture was not a problem because rubber particles were not pre-interacted with asphalt binder [12]. But the interaction between rubber particle and asphalt binder swelled the rubber particles during the production and placement, thus influenced the volumetric property of the asphalt mixture if adopted improperly. The performance of the mixture produced by mixing asphalt binder and fine rubber particles before adding aggregate was higher than that of the mixture that produced using the dry process. Mixing asphalt binder with rubber particle before adding aggregate could guarantee the

rubber particle could adequately interact with asphalt binder, thus improve the rubber particle modification efficiency.

1.1.2.3 Influence factors

The performance of rubber modified asphalt was highly influenced by test conditions and internal factors. The mixing conditions included mixing temperature and mixing time. The internal factors included asphalt binder characteristics, crumb rubber property, rubber content, and rubber size [13]. Appropriate processing parameters guaranteed the successful production of CRM asphalt binder. Table 1-1 displayed the key variables in producing CRM asphalt binder that are suggested by the California Department of Transportation (Caltrans), South African Bitumen Association (Sabita), and Association of Australian and New Zealand Road Transport and Traffic Authorities (Austroads) [13].

Table 1-1. Summary of the CRM asphalt binder specification requirement

Properties	Caltrans 2006	Sabita 2007	Austroads 2007
Asphalt Penetration (dmm)	120-150	60-100	85-100
Rubber passing sieve size (mm)	2.36	1.18	2.36
Rubber content (%)	18-22	18-24	15-18
Extender oil (%)	2.5-6	0-4	0
Calcium carbonate /talc (%)	-	0-4	-
Mixing temperature (°C)	190-220	180-220	195
Mixing speed (rpm)	-	3000	-
Mixing time (min)	45-60	-	30-45

a. Production procedure (mixing temperature, duration, and shear speed)

The mixing temperature and duration are critical parameters during CRM asphalt binder production. Higher mixing temperature and duration prompted the interaction between asphalt binder and crumb rubber, especially when coarse rubber particles and high rubber

content were adopted when producing CRM asphalt binder [14]. Higher mixing temperature and duration guaranteed the stability of CRM asphalt binder to avoid segregation and settling issues during the application of CRM asphalt binder [15]. The rubber modified asphalt binder properties was not significantly influenced by the increase of mixing time, which ranged between 30 minutes and 60 minutes. But the viscosity, softening point, ductility, elastic recovery, and complex shear modulus was influenced by the blending temperature [16]. However, the distribution of rubber particles in the asphalt binder will be improved with the increase of mixing temperature and mixing duration. The rubber depolymerization rate was also influenced by the rubber particle size, curing time, and mixing rate. Table 1-2 summarized the mixing variables adopted by different transportation agencies in the US and Canada. In order to balance the storage compatibility and performance of CRM asphalt binder, Attia recommended using fine rubber particles to produce terminal blended CRM asphalt binder [14]. Terminal blended CRM asphalt binder used high shear mechanism to totally depolymerize rubber particles, thus increased the compatibility between rubber particles and asphalt binders.

Table 1-2. Mixing variables adopted by some transportation agencies

Agency	Mixing temperature (°C)	Mixing time (min)	Crumb rubber content
Ontario, Canada	180	45	18-20 %
California	190-226 (adding rubber)	45	20 ± 2%
	190-218 (mixing)		
Arizona	180-205 (adding rubber)	≥ 60	≥ 20%
	165-190 (mixing)		
Florida	168-190	≥ 30	≥ 20%
New Jersey	180-205 (adding rubber)	≥ 60	≥ 17%
	165-190 (mixing)		

The shear mixing speed also affected the properties of CRM asphalt binder. The property of asphalt binder prepared with a high shear mixer was better than that mixed with a

propeller-type mixer. The mixing speed influenced the viscosity and softening point of the CRM asphalt binder. The higher shear mixing procedure improved the dispersion property of the rubber particle in the asphalt binder. The high shear mixing speed increased the possibility that the swelled rubber particles could be chopped into smaller rubber particles. The high shear mixing speed decreased the rubber particle size, thus increased the interaction speed between the asphalt binder and rubber particle [17]. High shear mixing speed improved the low temperature performance, and the low shear mixing speed enhanced the medium and high temperature properties. What's more, the storage stability and the dispersion of rubber particles can also be enhanced by high shear mixing speed [18].

b. Asphalt binder property

The property of pure asphalt binder had a significant influence on the performance of CRM asphalt binder. The rubber particle mixed with a soft binder with a high light component will swell higher thus had better low and high temperature properties than mixed with hard binder at the same rubber particle content [3]. While the CRM asphalt binder prepared with hard asphalt binder normally had higher high temperature stiffness, and the CRM asphalt binder with soft asphalt binder normally had better low temperature stiffness.

Henia analyzed the influence of asphalt binder different components of saturates, aromatics, resin, and asphaltenes on the viscoelastic properties of CRM asphalt binder. The research found that aromatics lubricated the asphalt; thus asphalt binders with high aromatics had higher temperature sensitivity. The quality of aromatic also influenced the swelling property of the rubber particles and the consistency of mix properties [19].

c. Rubber property

The rubber property, especially the microstructure, influences the performance of CRM asphalt binder. The microstructure of the rubber surface can be improved by several methods. By using the fluorescence microscope, the microstructure of three different rubber modified asphalt binders, which included general crumb rubber, dynamic

desulfurized crumb rubber, and high-speed agitation desulfurized crumb rubber, were investigated. The dynamic desulfuration destroyed the sulfurized bond of the crumb rubber, thus improved the swelling capacity of crumb rubber in asphalt binder. But the reunion of rubber particles during dynamic desulfuration also restricted the swelling of dynamic desulfurized crumb rubber in bitumen. The high-speed agitation desulfuration process not only improved the swelling capacity of crumb rubber in bitumen but also prevented the reunion of rubber particles. The high-speed agitation desulfurized crumb rubber modified asphalt binder possessed the best property [20]. Microwave irradiation cleaved the vulcanization network, thus enhanced the surface activity of rubber particles and improved the viscoelastic property and storage stability of CRM asphalt binder [21]. Hot water can be used to remove the light component in rubber particles, thus reduced the differences between rubber and asphalt binder and improved the compatibility [22]. The microstructure of rubber modified recycled hot mix asphalt could be assessed with the X-ray CT technology [23]. The distribution of the recycled aggregate and the new aggregated were evaluated with the X-ray CT technology. The binder film thickness was calculated, and asphalt film thickness increased with the decrease in aggregate particle size at the same asphalt dosage.

The manufacturing technology of rubber also influenced the surface microstructure. The porous of crumb rubber produced at ambient grinding was bigger, while the crumb rubber produced with cryogenic method had relatively angular, smooth, cracking shape with the smaller porous surface [24]. The CRM asphalt binder with rubber produced with ambient grinding method had a bigger surface area than produced with the cryogenic method, thus had a bigger swelling rate and better performance [25]. The high porous structure enhanced the light component absorption of the asphalt binder and improved the viscosity, complex modulus, better rutting resistance and low temperature cracking resistance [26]. What's more, the cost of cryogenic tire grinding was higher due to liquid nitrogen was needed during the rubber production [27].

The vulcanization of the rubber influenced the mixing temperature and duration used to produce CRM asphalt binder [28]. The optimum mixing duration for slightly vulcanized

rubber particles was influenced by the mixing temperature [29]. Unvulcanized rubber particles only need 10 minutes at 160 °C to achieve the same mixed binder. The unvulcanized rubber was easier to disperse into asphalt binder due to the condition of rubber and the fineness of powder [30]. The vulcanized rubber was difficult to disperse because the rubber powder after the vulcanization procedure had a coarse surface [31].

d. Rubber content and rubber size (gradation)

Except for the mixing temperature and mixing duration, crumb rubber size also had significant influence on the interaction between rubber and asphalt binder. Fine rubber particles swelled faster due to the bigger surface area, thus accelerated the depolymerization process. Sun analyzed the factors that influenced the viscosity of CRM asphalt binder, and found the following effects are decreased in turn, which are crumb rubber content, mixing temperature, rubber size, and mixing duration [32]. CRM asphalt binder with smaller rubber size had higher viscosity and resilience due to higher surface area. The combined rubber particle size and content influenced the low temperature performance of CRM asphalt binder [33]. Different rubber particle sizes may need different optimum rubber particle content to guarantee the performance of CRM asphalt binder [34]. Based on the high temperature rheological properties evaluation of CRM asphalt binder with five rubber asphalt dosages and three rubber particle meshes, the rutting resistance of crumb rubber modified asphalt was influenced by the rubber content, rubber size, and aging condition [35, 36]. With the increase of rubber content, the viscosity of the asphalt binder was significantly increased, and the low temperature stiffness was also influenced. The modification effect was better if finer crumb rubber was adopted to modify asphalt. There is no significant performance difference between CRM asphalt binders with 20% and 25% crumb rubber content [37]. By using reacted and activated rubber to modified asphalt binder, five rubber dosages were adopted, and the maximum dosage was 40% by weight of modified asphalt binder. The rutting resistance, fatigue resistance, and low temperature stability were significantly improved [38]. Based on gel permeation chromatography analysis on CRM asphalt binder, higher crumb rubber content (15% to 20%) decreased the small molecular ratio because the

more light component was absorbed from asphalt binder during the mixing procedure [39].

The crumb rubber gradation also influenced the property of the modified binder and distress resistance. Dynamic Shear Rheometer (DSR) and Multiple Stress Creep and Recovery (MSCR) were conducted on CRM asphalt binder with two viscosities graded VG30 and VG40 asphalt binder. The rutting resistance of the CRM asphalt binder was better for finer crumb rubber gradation. Since the surface area of fine crumb rubber was higher, more asphalt binder components were absorbed by the crumb rubber particles. Thus the swell effect due to the interaction between asphalt binder and rubber particles was more significant for fine rubber particles than that with coarse rubber particles [40].

1.1.3 Performance of CRM asphalt binder and asphalt mixture

1.1.3.1 Rheological property of CRM asphalt binder

a. Physical property and stiffness

The penetration of CRM asphalt binder decreased with the increase of rubber content, which indicated that the rubber modification of asphalt binder significantly increased the stiffness of asphalt binder. The penetration of asphalt increased with the increase of shearing duration; the longer mixing time prompted better dispersion of the rubber particles, thus increased the homogenization of CRM asphalt binder [41].

Elastic recovery performance is critical for the fatigue and rutting resistance of asphalt binder. The elastic recovery could also be used to assess the polymer modification effect on the asphalt binder. The rubber modification could improve the elastic recovery property of the asphalt binder. Rubber modification could increase the rutting resistance and ductility of the asphalt binder. The rubber modification may affect the aging performance of the asphalt binder. The high molecules in rubber particle may decrease the compatibility of the modified asphalt binder [42].

The softening point of the asphalt binder increased with the increase of the crumb rubber content. The rubber content may influence the component percentage of asphalt binder. The component percentage of asphalt binder influence the property of the asphalt binder, and the temperature susceptibility was also affected [43]. The influence of rubber content on the softening point of asphalt binder was more significant than the influence of asphalt binder type and particle size. The softening point of the asphalt binder reflected the stiffness of the asphalt binder, thus affect the elastic recovery of the asphalt binder [44].

b. Viscoelastic property

Asphalt binder was regarded as a kind of viscoelastic material due to the elastic and viscous property of the asphalt binder. Figure 1-1 reflected the correlation between the stress and the strain of a viscoelastic material. Part of the strain was recovered after the stress was released, but the unrecovered strain resulted in a permanent deformation [29]. The rheological property of the asphalt binder was influenced by the temperature and the loading condition due to the viscoelastic characteristics. The most common distress occurred on the pavement that related to temperature included rutting and cracking [45]. By using the dynamic shear rheometer (DSR), the rheological performance of the asphalt binder under different temperatures, loading stresses could be assessed by using the frequency sweeps or temperature sweeps.

(a) Applied stress

(b) Viscoelastic response

Figure 1-1. The general behavior of viscoelastic material under constant loading stress

[29]

Rubber modification could improve the viscoelastic characteristics of the asphalt binder. The viscosity of the rubber modified asphalt binder was higher than that of unmodified asphalt binder. The permanent deformation resistance and low temperature cracking resistance of the rubber modified asphalt binder was better than base asphalt binder. The viscoelastic property of rubber modified asphalt binder with 9% rubber content was similar to 3% SBS modified asphalt binder at -10 °C. When the test temperature increased to 75 °C, the SBS content increased to 7% [46]. Based on the DSR test on CRM asphalt binder with different rubber content and mixing condition, the deformation susceptibility of CRM asphalt binder was less sensitive to stress than unmodified asphalt. The correlation between the rheological parameters and softening point was not significantly influenced by the mixing condition [43]. Based on the DSR and Bending Beam Rheometer test results, the fatigue performance of the rubber modified asphalt binder was significantly improved. The low temperature cracking characteristics of the asphalt binder was enhanced after rubber modification. The rheological performance of the asphalt binder was improved with the increase of rubber content [47].

c. Viscosity

Viscosity reflects the flow resistance of asphalt binder and helps evaluate the asphalt binder if the pumping and mixing procedure will be affected. The viscosity of asphalt binder at high temperatures affected the performance of the asphalt mixture at the manufacture temperature and the construction temperature. The pumpability of the asphalt binder reflected the easiness of the asphalt binder to be transferred between the storage facility and the manufacture plant. The mixability of the asphalt binder can be used to assess the simplicity of the asphalt binder to coat aggregate and mixing with other components in the manufacture plant. The workability of the asphalt binder could be used to evaluate the easiness of the asphalt mixture to be placed and compacted in the field [37]. The viscosity of the asphalt binder at the placement temperature significantly affect the final performance of the mixture [48]. The viscosity of the asphalt binder at the compaction or mixing temperatures affects the stability and workability of the asphalt mixture [49].

The rubber modification increased the viscosity of asphalt binder, the uniformity and the deformation resistance of the asphalt mixture was increased with the increase of the rubber content. The viscosity of the rubber asphalt binder increased with the increasing rubber content, especially when 15 wt% of it was added [50]. The viscosity of the rubber modified asphalt binder was influenced by the property of the crumb rubber. The viscosity of rubber modified asphalt binder at 95 °C was 20 times higher than that of the base asphalt binder when 30% vulcanized rubber was added [51]. The influence of devulcanized rubber on the viscosity of the asphalt binder was ignorable. No gel was formed during the rubber mixing procedure based on the viscosity test result [52].

1.1.3.2 Storage stability of CRM asphalt binder

a. Evaluation of storage stability

Due to the composition inconsistent in the rubber modified asphalt binder, and the density difference between asphalt binder and rubber particle, the rubber particles in the

asphalt binder tend to segregate in the rubber modified asphalt binder storage tank [53]. The rubber particle size is the main parameter that affect the storage stability of the rubber modified asphalt binder. The rubber modified asphalt binder with lager rubber particle size is more prone to have storage stability issues [54].

Rubber modified asphalt binder produced with finer rubber particles had better storage stability. The mixing condition also affect the storage stability of the asphalt binder, rubber modified asphalt binder produced at higher temperature with higher shearing rate had better storage stability. Mixing with pressure also improve the storage stability [9]. The storage stability was affected by the production process, the asphalt mixture produced using the dry process with large rubber size had better storage stability than that of terminal blended mixture. During the dry process, the rubber particles only partly interacted with asphalt binder, thus the segregation possibility was decreased [55].

b. Problems related to the storage stability of CRM asphalt binder

The application of the rubber modified asphalt binder was affected by its storage stability. The segregation of rubber particles in the rubber modified asphalt binder may clog the pump valve. The production procedure was affected by the storage problem and the property of the rubber modified asphalt binder was weakened [31]. Mixing the rubber modified asphalt binder with a stirrer continuously could effectively avoid the segregation possibility in the rubber modified asphalt binder. But the mixing process will waste energy and increase the production cost [56].

c. Storage stability improvement method

The stability of rubber modified asphalt binder can be improved with the following methods. (a) decrease the rubber particle size; (b) decrease the density difference between the rubber particle and asphalt binder; (c) increase the viscosity of asphalt binder. The best way to improve the stability of rubber modified asphalt can be achieved when physical and chemical interactions occurred between the rubber particle and asphalt binder after rubber added to the asphalt binder [57]. Such compounds may include transpolyoctenamer rubber, the mixture of sulfur and SBS copolymer, peroxides and

acrylate-glycidyl copolymers, high-density polyethylene or low-density polyethylene, or the maleic anhydride-grafted ethylene-octene copolymer (POE-g-MA). Using furfuraldehyde or acrylic acid to treat the surface of rubber particle could also improve the storage stability of rubber modified asphalt binder [55].

1.1.3.3 Modify asphalt binder with both crumb rubber and other additives

The workability of the CRM asphalt binder was lower than that of the conventional asphalt, because of the higher viscosity. By adding other additives, the workability of the CRM asphalt binder could be improved, such as the WMA additive. The influence of warm-mix additive on the performance of rubber modified asphalt binder at high temperatures was affected by the rubber content [58]. By using different types of WMA additives, which included Sasobit, RH, and Advera, the viscosity, low temperature rheological property, and high failure temperature of asphalt modified with the 40 mesh rubber at different contents assessed [59]. The viscosity of the CRM binder modified with those three WMA additives were all reduced, the influence of RH on the viscosity reduction was most obvious. Sasobit had the most remarkable effect on complex shear modulus of both unaged and RTFO-aged CRM asphalt binders. RH only had a significant influence on RTFO-aged binders. The influence of the Advera was indistinctive [60]. The low temperature performance of the CRM asphalt binder was significantly improved by the higher crumb rubber content. Advera enhanced the low-temperature performance by improving the stress relaxation capacity [61]. Another advantage of using the WMA additive is to reduce the emission generated due to the application of crumb rubber in the asphalt binder. The addition of crumb rubber greatly increased the hazardous emission. Temperature is the most influential factor affecting emissions. Using the WMA additive could reduce the production temperature, thus reduced the temperature influence of using crumb rubber [62].

The performance of CRM asphalt binder could also be improved by adopting other additives. Bio-oil derived from corn stover increased the viscosity and the high temperature performance of CRM asphalt binder. The workability of the CRM asphalt

binder could also be improved by adopting bio-oil. What's more, the effect of bio-oil on the high-temperature sensitivity was related to rubber particle size [63]. The combined modification with polymer and crumb rubber could improve the rheological characteristics of the asphalt binder. By assessing the linear viscoelastic and large strain rheology of modified asphalt binder, the Lissajous-Bowditch plots and non-linear measurement could provide guidance in assessing the performance grading [64]. The modification effect of asphalt binder with waste tire rubber (WTR) and recycled polyethylene (RPE) was significantly affected by the WTR and RPE content. The high temperature property was increased remarkably without sacrificed the low temperature characteristics [65].

1.1.3.4 Characterization of CRM asphalt mixture

The rheological properties of asphalt binder indicated the pavement performance. The rheological performance of asphalt binder at high temperatures could be used to evaluate the rutting resistance of the asphalt mixture. The fatigue performance of the asphalt pavement could be evaluated by evaluating the rheological performance of the asphalt binder at intermediate temperatures. The low temperature performance of the asphalt mixture could be quantified with the rheological performance of the asphalt binder at low temperatures.

a. Cracking (thermal cracking and fatigue cracking)

The cracking of the asphalt pavement mainly has two distress types, thermal cracking and fatigue cracking. The thermal cracking mainly occurs on the pavement when significant temperature change happened. The significant change of temperature will generate tensile stress on the asphalt pavement. The thermal cracking will happen when the tensile stress exceeded the tensile strength of the pavement, together with the stress generated due to the traffic load. The fatigue cracking is mainly due to the repeated stress generated due to the traffic load. The passing of the traffic load imposes damage to the asphalt pavement, the damage will accumulate with the increase of traffic load passing by. The performance of the pavement will decrease with the damage accumulation. The cracking initiation

occurred once the traffic load exceed the tensile strength of the asphalt pavement, and the cracking mainly occurs on the surface of the asphalt mixture [66].

The use of crumb rubber modified with asphalt binder could significantly improve the asphalt mixture fatigue damage resistance. The improvement effect was better if using a smaller rubber particle size. The fatigue life of asphalt mixture was also influenced by the loading frequency. The fatigue life was extended and the crack grow rate was reduced with the increasing of loading frequency. The asphalt mixture with gap gradation had better fatigue resistance [67]. Rubber modification could improve the rheological performance of the asphalt binder is part of the reason that the performance of the rubber asphalt mixture was better than that of the unmodified asphalt mixture [68].

By using the extended finite element method, the low temperature cracking propagation of rubber modified asphalt mixture was analyzed. The cracking of the SCB asphalt mixture sample was characterized, and the rubber concentration had a significant influence on the cracking performance [69]. The influence of rubber content, rubber particle size, and temperature on the low temperature cracking resistance of the rubber modified asphalt mixture was evaluated with the semi-circular bending (SCB) test. The improvement of finer rubber particles on the low temperature performance was more significant than that of the large rubber particle. The performance reached the peak value at the dosage of 20% addition [70].

b. Rutting

Rutting in one of the main distress type occurred in the asphalt pavement, and the rutting mainly included the consolidation rutting and the instability rutting. The consolidation mainly happened along the wheel path due to the load applied to the pavement. The air void decrease of the pavement due to the traffic load was the main reason of the consolidation rutting. The permanent deformation on the base or subbase of the pavement may also lead to the consolidation of the pavement. The instability rutting mainly occurred on the top 2 inches of the asphalt mixture because of the asphalt mixture performance [71].

The increase of the tire load, tire pressure and traffic volume is the main reason of the increasing of the rutting deformation. The use of rubber modified asphalt binder could significantly improve the rutting resistance of the asphalt mixture. The crumb rubber modified asphalt binder had higher stiffness than that of unmodified asphalt at the test temperature of 60 °C. The gyratory shear strength and rutting resistance of the rubber modified asphalt mixture was better than that of mixture without modification based on the loaded wheel test [72].

1.1.3.5 CRM asphalt mixture performance improvement strategy

The viscosity of the crumb rubber modified asphalt binder was higher than unmodified asphalt binder, thus increased the production temperature and the production cost. The increased production temperature also increased the environmental pollution. The equipment modification may also be needed to process rubber modified asphalt mixture.

(1) Warm mix rubber modified asphalt mixture

The WMA technologies could reduce the production temperature of the asphalt mixture, thus increase the possibility of adding more RAP to the asphalt mixture [73]. Adding RAP could increase the stiffness of the asphalt mixture, but the disadvantage of the aged asphalt binder could be compensated by using the WMA additives [74]. Using warm mix rubber modified asphalt mixture could save the fuel consumption during the production of asphalt mixture and reduce the hazard emission. By using Evotherm as warm mix additive, the fatigue, moisture damage, rutting and thermal cracking resistance of warm mix rubber modified asphalt mixture had comparable or better performance than conventional asphalt mixture [75].

(2) Chemical modification

The interaction between rubber particle and asphalt binder could be improved by using activators or chemicals. The chemical bond between rubber particle and asphalt binder could improve the property of rubber modified asphalt binder [76]. The transpolyoctenamer rubber (TOR) could enhance the interaction between the sulfur from

rubber particle and sulfur from asphalt binder [77]. The rubber modified asphalt binder produced with the TOR had good distribution property, thus improve the performance of rubber modified asphalt binder [78]. The swelling of the rubber particle in the asphalt mixture was restricted, and the strength of the asphalt mixture was improved by using cement to coat rubber particle. Using cement treated No. 16 rubber to replace 50% aggregate had the best improvement [79]. Polyurethane rubber particle mixtures exhibit excellent rutting resistance, and the polyurethane rubber particle mixture has excellent high temperature stability. What's more, the polyurethane rubber particle mixture had well-interconnected pores and an excellent drainage effect. The noise of the pavement could be reduced by using a polyurethane rubber particle mixture [80, 81].

(3) Physical treatment

The compatibility between rubber particle and asphalt binder was restricted by the property of rubber particle. Using physical treatment strategy, the surface property of rubber particle could be improved, which included microwave treatment and matrix asphalt preblending process. The surface improvement of rubber particle could increase the adhesion between rubber particle and asphalt binder could enhance the low temperature property of rubber modified asphalt binder [82]. By using the Reacted and Activated Rubber (RAR) and reclaimed asphalt pavement, the Superpave mix design and Marshall mix design were adopted to design the mixture, the rutting and moisture damage resistance of the asphalt mixture was improved [83]. The RAR in the gap graded mixture design as the ultra-thin hot mix asphalt overlay for wet-freeze climates was assessed. The moisture damage resistance, low temperature cracking resistance, fatigue cracking resistance, rutting resistance, and noise reduction was favorable [84].

1.2 Asphalt binder and asphalt mixture aging

1.2.1 Background

1.2.1.1 Introduction

Reclaimed asphalt pavement (RAP) was the aged asphalt pavement materials that could be used in the new pavement construction. Using RAP on the intermediate or surface layer could use the asphalt binder from RAP to partially replace the virgin asphalt, thus reduced the cost of the asphalt mixture production [85]. However, different agencies had restrictions on the content of RAP used in the mixture production. Normally the RAP was treated as aggregate, and the asphalt binder from RAP was not considered. Due to the limitation of knowledge and experience, using high RAP content on the asphalt mixture still have a lot barriers.

With the increasing amount of reclaimed asphalt pavement (RAP), the use of RAP in asphalt pavement has substantial environmental and economic benefits. The use of RAP affects the aging performance of mixtures; the aging effect on stiffness decreased as RAP content increased, and the significant difference occurred at high temperatures and low frequencies [86]. Evan compared different distress criteria, including rutting, cracking, and ride quality of pavement with RAP and virgin materials. Results showed that pavement with RAP had a better rutting performance, but worse cracking and ride quality. The difference was not significant, which proved that the pavement with high RAP content had comparable performance to that of pavement with virgin materials [87]. As the content of RAP in the mixture increased, the amount of aged asphalt binders also increased, which negatively influenced the performance of asphalt mixtures [88]. In coping with the problem, many researchers tried to add rejuvenators to aged asphalt materials to chemically restore the properties of aged binders [89-91].

1.2.1.2 Environmental issues

Asphalt binder and asphalt mixture recycling could save the energy consumption and conserve natural resources. Conservation of natural resources and sustainable

consumption is vital for sustainable development. By proper mixture design, aggregates can be partially or totally replaced by RAP. Recycled waste materials are considered ideal for surfacing secondary roads and do not see it as a high worth product. Using RAP in the construction of new asphalt pavement as aggregate could reduce the influence of RAP as waste materials and reduce the influence of RAP to the environment [92].

1.2.1.3 Cost benefits

The aggregate and aged asphalt binders in RAP material could effectively replace the new asphalt pavement construction materials, thus reduce the cost of new aggregate and asphalt binders used in the pavement construction. The cost of recycled mixtures is mainly included the cost used to process RAP, to test RAP performance. The adoption of rejuvenator to process and pollution control also increase the cost when using RAP in the asphalt pavement construction. The cost of the pavement construction could reduce 50%-70% if 100% RAP materials were adopted in the construction [93].

1.2.2 Asphalt binder aging

1.2.2.1 Mechanism related to asphalt aging

The aging mechanism of asphalt binders mainly included three aspects: volatilization of light component in the asphalt, oxidation of the hydrocarbon in the asphalt binder, and steric hardening of the asphalt [94]. Volatilization and oxidation are irreversible because chemical reaction occurred, and the steric hardening process can be reversed by changing the temperature to reconstruct the molecular structure [95].

a. volatilization

The high temperature during the production and placement of the asphalt mixture prompted the evaporation of the light components of the asphalt mixture [96]. The evaporation procedure was significantly influenced by the temperature and the evaporation rate increased dramatically with the increase of the temperature. The asphalt film thickness covered the aggregate affect the heat transfer in the asphalt binder. The

aging of the asphalt binder was influenced by the film thickness of the asphalt binder. The aging of the asphalt binder was improved with the decrease of the film thickness [97]. The volatilization of asphalt binder may generate fume or steam and decrease the mass of the asphalt binder. The decrease of the light component weight percentage in the asphalt binder will result in the increase of the viscosity and the decrease of the flow ability of the asphalt binder [98].

b. oxidation

The hydrocarbon component of the asphalt binder oxidized during the surface in the asphalt pavement. The oxidation process mostly begins on the surface of the pavement, the oxidation process will harden the asphalt binder, thus increases the possibility of distress generated on the top surface of the asphalt mixture. The oxidation process will occur on the lower layer of the asphalt mixture once top-down cracking happens [99]. The ultraviolet radiation from the environment could break the chemical bond in the asphalt binder, thus decrease the oxidation resistance of the asphalt binder. The effect of ultraviolet radiation on the asphalt binder could be minimized by adding ultraviolet radiation additive [100]. Carbonyl and sulfoxide component generated during the aging process and decreased the cracking resistance of the asphalt binder [101]. By analyzing the oxidation of asphalt binders in HMA, the property of aggregates and mixtures had significant influence on the aging property of asphalt mixtures [102]. Asphalt oxidation will increase the brittleness of the asphalt binder and increase the possibility of cracking [103]. The oxidation process also weakened the fatigue resistance of asphalt mixture [104, 105]. The wood waste bio-oil had a significant higher oxygen content than petroleum asphalt. And the aging of bio-asphalt is quicker than petroleum asphalt in production and service [106-108].

c. steric hardening

Steric hardening reflects the hardening effect of asphalt binder with the change of temperature. The steric hardening procedure happens every day because of the change of

temperature. The viscosity and the volume of the asphalt binder changes with the steric hardening procedure [109].

1.2.2.2 Property changing during the aging procedure

The aging procedure increase the stiffness of the asphalt binder, and the effect was determined by the temperature and the original property of asphalt binder. The aging effect was improved with the increase of the temperature. The asphalt with high stiffness was less affected by the aging procedure at lower temperature. For different asphalt binder, the critical aging temperature is different, and relationship could be observed between the DSR and FTIR result of aged asphalt binder [110, 111]. The aging mechanism of asphalt binder and the asphalt from SGC compacted mixtures is different. The aging of asphalt mixtures in PAV should cost three times longer to have the same aging condition as asphalt binder [112]. The stiffness of the asphalt binder was changed during the aging procedure, thus affect the fatigue performance of the asphalt mixture [113]. The aging effect of the asphalt mixture was also affected by the void content of the asphalt mixture. The oxidation effect was more significant for mixture with higher air void [114]. The short-term aging and long-term aging had significant influence on the aging resistance of the asphalt mixture. The condition of enough aging is critical to the rutting property of the asphalt mixture [115, 116].

1.2.2.3 Laboratory aging

a. RTFO

The asphalt binders will be aged under the high temperatures during the mixing procedure in the plant. The high temperature during the transportation and placement procedure of the asphalt mixture also aged asphalt. During the mixing process, the heated aggregate was covered by the thin asphalt film, and part of the asphalt components were absorbed by the aggregate. This term of aging is called short-term aging. In the laboratory, the rolling thin film oven (RTFO) procedure is used to simulate these kind of aging [117, 118].

b. PAV

The aging procedure continues after the construction of the pavement, and oxidation is the main aging type due to the relatively lower temperature of the pavement during the service period. The aging of the asphalt pavement during the service period was influenced by the air void of the asphalt mixture and the position of the asphalt mixture in the pavement structure. The aging degree of the asphalt mixture near the surface of the asphalt pavement was higher. The pressure aging vessel (PAV) was adopted in the laboratory to simulate the long-term aging of the asphalt mixture during the service period [119, 120].

1.2.3 Asphalt mixture aging

1.2.3.1 *The laboratory procedure to simulate the asphalt mixture aging*

With the increase of the pavement service life, the pavement was aged gradually, the cracking performance of the pavement was decreased. Based on the AASHTO R30, the aging of asphalt mixture in the laboratory was proposed to simulate the aging procedure of asphalt mixture in the field. The short-term aging was conducted on laboratory prepared loose mix with the thickness of 25-50mm, in the oven at 135 °C for 4 hours. Long-term aging was conducted on the compacted specimen. The compacted sample was cooled to room temperature for 16 hours after compaction and then set in the oven at 85 °C for 120 hours [121].

1.2.3.2 *The application of recycled asphalt pavement*

The aged asphalt pavement can be used as aggregate to produce new asphalt mixture. The production of the mixture contains RAP was normally realized by overheating the new aggregate, and the heat transfer from the new aggregate to RAP could dry and heat RAP. The procedure could guarantee the RAP didn't heated by the flame directly. The heating condition of the new aggregate was influenced by the moisture condition of the aggregate and the RAP content [122].

c. The limitations

(1) Property of aged binder

During the aging process, the component of asphalt changes increased the stiffness and viscosity of the binder. The performance of the mixture may not be as expected if designed improperly, thus reduced the service life of the road. The cost of the pavement maintenance fee would also increase, and energy waste occurred due to the decrease of pavement performance [123]. By mixing the asphalt binder with the extracted asphalt binder from RAP, the viscosity of the mixed binders was increased with the increase of the RAP content. The stiffness of mixed binder was increased with the addition of RAP [124].

(2) Blending degree and diffusion

The blending degree and diffusion efficiency of asphalt with RAP was influenced by different parameters, which included the RAP resource and binder property, production condition, and production duration [125]. The blending degree could be improved by reduce the size of RAP, increase the mixing temperature and duration, and use WMA additives [93].

(3) RAP aggregate property

The gradation of RAP was not easy to control because the percentage of fine aggregate was influenced by the milling procedure. The volumetric property of the asphalt mixture was also influenced by the RAP aggregate property [126].

d. Distress associated with high RAP content

The performance of the asphalt mixture with high RAP content was significantly decreased, thus the field application of mixture with high RAP was restricted [127].

(1) Cracking

The performance of the asphalt mixture with RAP was similar as that without RAP when the RAP content was low. When the RAP content was high, the tensile strength of the

asphalt mixture was reduced and the stiffness of the asphalt mixture was increased. The possibility that fatigue cracking occurred on the asphalt mixture was increased. The stiffness of the asphalt mixture increased 49% when 40% RAP was adopted in the asphalt mixture design [128].

(2) Water susceptibility

The mixture produced with high RAP content had higher stripping possibility than mixture produced without RAP. However, the pavement resource used to produce RAP also affect the moisture resistance of pavement significantly. The stripping problems will occur if old recycled pavement had a stripping problem, and no adhesion additives added to the new mix. The blending efficiency between the RAP and new asphalt binder or mixing temperature also affect the moisture susceptibility of the asphalt mixture [129].

(3) Rutting

The stiffness of the pavement with high RAP content was increased, thus the permanent deformation resistance of the asphalt pavement was improved [130]. The rutting resistance of the pavement need to be assessed if rejuvenator or softer asphalt binder was adopted to process the aged RAP.

e. Performance improvement strategy

(1) Use softer binder

Using the softer binder with lower viscosity could reduce the stiffness of the asphalt binder after mixing with RAP. The high stiffness of aged asphalt binder from RAP could be decreased when mixed with softer binder. The high temperature grade of the base asphalt binder will increase after mixing with high RAP content in the mixture [131].

(2) Add rejuvenator

By mixing RAP with rejuvenator, the stiffness of the mixture with high RAP content could be reduced, thus improve the cracking resistance of the mixture without reducing the rutting resistance of the asphalt mixture. Different rejuvenators have been studied by

researchers, the main mechanism of the adoption of rejuvenator is restore the content ratio in the asphalt binder [132]. The performance of the aged asphalt binder from RAP could be partially recovered to unaged condition, thus significantly improved the performance of the asphalt mixture with high RAP content.

(3) Use WMA

Using WMA additive could reduce the viscosity of the asphalt binder, thus decrease the mixing and production temperature. Using WMA additive in the asphalt mixture with high RAP content could reduce the stiffness increase due to the aged asphalt binder from RAP. The aging effect due to high RAP content was also reduced [133].

f. RAP management

The RAP management is critical to the quality control of mixture design with RAP. Proper management could effectively reduce the dust content and moisture ratio in RAP. The mixture design parameters, which included the gradation and asphalt binder content, are also affect by RAP management [134].

(1) RAP milling and processing

The property of RAP was influenced by the crushing equipment, mill depth. And the processing speed also affect the quality of the RAP. The processed RAP could be used directly in the field mixture design or processed in the mixing plant before used in the mixture design. Process the milled RAP with screen process is beneficial to control the quality of mixture with RAP. The RAP that milled incomplete is normally needed to be processed before the application in the mixture design because the oversize RAP cannot be separate into smaller sizes during the mixing procedure [135].

(2) Storage of RAP

During the storage of RAP, attention need to be paid to the moisture condition of RAP because water is not easy to be moved out, and normally the moisture content of RAP stockpiles is higher than that of aggregate. The performance of RAP from different

projects are normally not the same. Arrange the RAP stockpile from different resources in different places is good to control the quality of the RAP. If RAP from different resources have to store in the same location due to the limitation of storage area, mixing RAP from different resources completely could help control the consistence of RAP property in the mixture design [136].

1.3 Objectives of the dissertation

Only limited research focused on the performance and aging characteristics of rubber modified asphalt mixture using the dry process. The aging characteristics of the long-term service condition of the rubber modified asphalt mixture using the dry process was unknown. The major goal of this study is to understand the performance of the rubber modified asphalt mixture using the dry process. The influence of aging on the characteristics of the rubber modified asphalt mixture and the extracted asphalt binder were investigated. The objectives of this study are summarized as follows:

- a. To investigate the volumetric design of the rubber modified asphalt mixture with RAP using the dry process. The quality control strategies to guarantee the laboratory design successfully represent the field application were proposed.
- b. To study the rutting and moisture susceptibility of rubber modified asphalt mixture using the dry process. The influence of factors, which included mixture design parameters, rubber modification, mixture air void, and mixture aging, were studied.
- c. To examine the low temperature cracking performance of rubber modified asphalt mixture using the dry process. The influence of factors, which included mixture design parameters, rubber modification, test temperature, and mixture aging, were evaluated.
- d. To assess the effect of solvent in asphalt binder and to propose a method to determine the actual performance of asphalt binder extracted from asphalt mixture.

- e. To discover the performance of the extracted asphalt binder from the rubber modified asphalt mixture using the dry process. The interaction degree between the rubber particles and the influence of the aged asphalt binder from RAP were investigated.

2 Design Asphalt Mixture with Rubber Using the Dry Process

Extractive Summary

The interaction between the rubber particle and asphalt binder resulted in the swelling of the rubber particle in the asphalt mixture. The rubber particle directly interacted with asphalt binder during the mixing procedure, and the interaction process continued during the production and the placement procedure. 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. RAP content in the mixture design also affected the interaction between the rubber particle and asphalt binder. The scope of this chapter is to conduct the volumetric design of the rubber modified asphalt mixture using the dry process when RAP was adopted. The challenge during the volumetric design in the laboratory is how to guarantee the laboratory design successfully represent the field application. In this chapter, the volumetric design procedures were proposed, and the quality control strategies were proposed. 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 were conducted. The volumetric design procedure could provide a guidance for future volumetric design of the rubber modified asphalt mixture with RAP using 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 [137]. 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 [124].

The volumetric property of the asphalt mixture with rubber was influenced by the swelling process, which affected the target density of the asphalt mixture, thus weakened the consistency of the asphalt mixture. The compaction temperatures significantly influenced the volumetric properties of the CRM mixtures. The compaction temperature influenced the viscosity of the asphalt binder and the air void content of the asphalt mixture [138]. The air void and the voids in mineral aggregate were two critical parameters that influenced the rutting performance of the rubber modified asphalt mixtures [139]. 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 [140]. Some researchers tried to use pre-treated rubber to reduce the swelling influence during the compaction procedure. By using RAR in ultra-thin hot mix asphalt overlay, the swelling process could be initiated before the compaction of the asphalt mixture, thus guaranteed the compatibility and consistency of the asphalt mixture [84]. 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 percent passing is based on the weight ratio of the aggregate passing certain sieve to the total aggregate weight. 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, intermediate shingle particle (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%, the designed value is 4.15%. The measured content is slightly higher than the design value due to more fine aggregate was adopted in the sample used to do the test.

2.2.2 Asphalt binder

The PG 58-34 asphalt binder was used in the project, and the basic properties of 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 °C, Pa·S _e)	0.398	< 3.0
Unaged $G^*/\sin \delta$ (58 °C, kPa)	1.542	> 1.0
RTFO aged $G^*/\sin \delta$ (58 °C, kPa)	4.147	> 2.2
PAV aged $G^* \times \sin \delta$ (16 °C, kPa)	2346	<5000

2.2.3 Engineered crumb 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 are 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 °C
Auto-Ignition temperature	371 °C
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 resistance of asphalt mixture, ZycTherm-SP (ZT-SP) anti-stripping agent was used in this study to improve the anti-stripping performance 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 rubber modified asphalt mixture with RAP 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 crumb rubber 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 the design results could successfully represent the mixture construction in the plant.

2.3.1 Prepare aggregate trial 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)	$\geq 80\%$	$\geq 80\%$

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 crossing 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 shown in Table 2-11. The final gradation designs of two types of asphalt mixture are shown in Figure 2-5. The nominal maximum aggregate size of the surface layer asphalt mixture is 9.5 mm, and the nominal maximum aggregate size 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 aggregate to prepare one sample was determined based on the mass of one sample and 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 three test samples were prepared for each test condition.

2.3.3 Rubber modified asphalt mixture preparation

The shearing mixing method was used to produce the rubber modified asphalt mixture in the lab. Following 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 °F (182.2 to 204.4 °C).
3. Pre-heat the binder to 350 °F (176.7 °C). 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 °F.
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 °F (168.3 °C) 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 °F (160 °C). 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

Mixtures were prepared with the gradation selected in previous section and with the following asphalt binder content: (1) the estimated binder content, P_b ; (2) 0.5% lower than P_b ; (3) 0.5% higher than P_b . The design binder content included the crumb rubber because the rubber has 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 design binder content that generate sample with the air void content (V_a) of 4.0% at N_{design} gyrations by following the procedures:

(1) Determine the V_a , VMA, and VFA at N_{design} according to Equations 2.1, 2.2, and 2.3:

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

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

$$VFA = 100 (VMA - V_a / VMA) \quad (2.3)$$

(2) Calculate the average relative density of specimens at $N_{initial}$ (% $G_{mm-initial}$), according to Equation 2.4:

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

(3) Plot the relation between binder content and the average V_a , VMA, and VFA, and relative density at N_{design} . Finally, obtain the optimum asphalt binder content that produce mixture with 4.0% air void.

The relation between asphalt binder content and the average V_a , VMA, and VFA, and relative density at N_{design} of the leveling layer mixture design are shown in Figure 2-5. The volumetric design result for different asphalt binder contents is 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 (LVSP) mixture design

Table 2-12. Volumetric design result – leveling layer

Pb (%)	Va (%) 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 relation between asphalt binder content and the average Va , VMA, and VFA, and relative density at N_{design} of the surface layer mixture design are shown in Figure 2-6. The volumetric design result for different asphalt binder contents is 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 (5E1) mixture design

Table 2-13. Volumetric design result- the 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), as shown in 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) [141]. 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 dry subset (psi)	Average tensile strength of conditioned subset (psi)	TSR
LVSP	175.08	151.44	0.86
5E1	165.7	149.79	0.90

2.4 The 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 nominal maximum aggregate size of the surface layer asphalt mixture is 9.5 mm, and the nominal maximum aggregate size 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 Chapter summary

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

3 High Temperature Rutting Performance Evaluation of Rubber Modified Asphalt Mixture Using the Dry Process

Extractive Summary

This chapter evaluated the rutting performance of rubber modified asphalt mixture using the dry process. The mixtures were produced in the plant with the dry process and compacted in the laboratory. Four types of asphalt mixtures, including LVSP and 5E1 gradation with and without crumb rubber, were produced in a plant. The volumetric design of the asphalt mixture used the design in chapter 2. The rutting performance of four types of asphalt mixtures was evaluated using the Hamburg wheel track test (HWTT). For asphalt mixture, the rutting performance of asphalt mixture with rubber was improved compared with base asphalt mixture. The rutting performance of LVSP mixture was higher than that of the 5E1 mixture. The number of wheel passes of the 5E1 mixture with rubber was 2.2 times higher than the 5E1 mixture without rubber. The rutting test of the LVSP mixture without rubber terminated after the rutting depth reached 20 mm, and the number of wheel passes was less than 20000. While the rutting depth of the LVSP mixture with rubber was 7.5 mm after 20000 wheel passes. The excellent performance of asphalt mixture with rubber in high temperature suggests that it can be an alternative material for asphalt pavement in high-volume traffic area.

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 [142]. 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 treated as a fine aggregate substitute in asphalt pavement [143]. The crumb rubber can be used to modify asphalt

binder to increase the high temperature performance of the asphalt mixture with the wet process [144]. Normally, in the dry process, ground rubber particles were added into asphalt mixture as fine aggregate in gap graded friction course of asphalt pavement [145].

The dry process is mainly used in hot mix asphalt pavement, and only limited research studied the application in the cold mix, chip seals, and surface treatment [146]. The dry process was firstly adopted in the US. with the PlusRive technology. The PlusRide dry process technology was adopted in different states DOT, which proved that the strategy was not a cost-effective method [147]. Some test sections performed well, while some did not have comparable performance than 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 [148]. 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 °C. 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 Summary of test materials

Chapter 2 described the volumetric mixture design of rubber modified asphalt mixture with 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 susceptibility of rubber modified 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 the rubber.

The specimens were compacted 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 laboratory compacted specimen was $7.0 \pm 1.0\%$. In order to guarantee the repeatability of the test results, at least three replicate test samples were prepared for each test condition.

In order to study the effect of long-term aging on the rutting and moisture damage resistance of asphalt mixtures, the SGC specimens were long-term aged in the oven at $85 \pm 3^\circ\text{C}$ for 120 ± 0.5 hours before conducting the Hamburg wheel tracking device (HWTD) test, based on the AASHTO R30 standard.

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 Hamburg Wheel Tracking Device (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 failure susceptibility of asphalt mixture because of the aggregate 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, which was submerged in temperature-controlled water under wheel load, 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 specimen 52 ± 2 passes per minute. The maximum speed of the wheel is 1 ft/s, when the wheel reached the midpoint of the specimen. The linear variable differential transducer (LVDT) device measures 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 °C since the high temperature specification of the PG binder was 58 °C.

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 HWT test 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 slope of the first steady-state section of the curve [149]. The creep slope reflects the rutting potential 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 assess the moisture damage resistance of the asphalt mixture [150]. The rut depth of the sample when the wheel passes reached the SIP (t_s) can also reflect the rutting susceptibility of the asphalt mixture without the influence of moisture damage. The asphalt mixture with lower N_s is more prone to occur the 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-state section of the curve. The stripping slope reflected the combined effect of the rutting and the moisture damage on the specimen.
- (5) Failure point. The test terminated after the rut depth reached 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 resistance 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 ($R_{\text{stripping}}$) could effectively evaluate the moisture susceptibility of different asphalt mixtures, as shown in Equation 3.1. The $R_{\text{stripping}}$ reflected the number of wheel passing percentage before moisture damage was considered. Asphalt mixture with lower $R_{\text{stripping}}$ had better moisture damage resistance during the rutting test.

$$R_{\text{stripping}} = N_s / N_f \quad (3.1)$$

3.3 Testing 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 are shown 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 nominal maximum aggregate size (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 shown 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 resistance 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 susceptibility of asphalt mixtures were affected by the mixture design parameters of the asphalt mixture (nominal maximum aggregate size (NMAS), RAP content, and asphalt content), rubber modification, mixture air void content, and aging procedure. This section will analyze the influence of various factors on the rutting and moisture susceptibility of asphalt mixtures.

The statistical analysis was conducted to evaluate the influence of mixture design parameters, rubber modification, mixture air void content, and aging on different rutting parameters. The statistical analysis was conducted using the linear model to get the analysis of variance (ANOVA). The dependent variables were the rutting test parameters, and the fixed variables were mixture design parameters, rubber modification, mixture air void content, and aging. An alpha value of 0.05 was selected to obtain a 95% confidence level. The F-value and P-values were obtained through the ANOVA. The F-value is

determined by the ratio between the variance between groups to the variance within groups. Thus, a higher F-value is positive for a significant effect. The F-values were obtained from the F-test, and the corresponding P-values (possibility) were also obtained. The P-value is used to determine if it is statistically significant, with a 95% confidence. If the P-value is lower than 0.05, the null hypothesis is rejected, and there is a significant effect statistically. Otherwise, there is no significant effect.

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 are 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 did not consider. 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 the 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 failure 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

The statistical analysis was conducted to evaluate the effect of mixture design parameters on different rutting test parameters, as shown in Table 3-1. It is found that the creep slope, number of passes to SIP and failure were significantly influenced by the mixture

design parameters. The stripping slope, the rut depth at the SIP (t_s), and $R_{stripping}$ were not significantly influenced by the mixture design parameters. But the P-value of t_s was only slightly higher than 0.05.

Table 3-1. Test of mixture design parameters effect on rutting parameters

Parameters	F-value	P-value	Significant effect
Creep slope	33.8	0.028	Yes
Stripping slope	4.945	0.156	No
t_s	17.37	0.053	No
N_s	12309	<0.001	Yes
N_F	70.87	0.014	Yes
$R_{stripping}$	13.31	0.068	No

3.3.2.2 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 is 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 damage resistance 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 that of the surface layer asphalt mixture without 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_r , 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

The statistical analysis was conducted to evaluate the effect of rubber addition on different rutting test parameters, as shown in Table 3-2. It is found that the creep slope, stripping slope, and the number of passes to SIP and failure were significantly influenced

by the rubber modification. The rut depth at the SIP (t_s) and $R_{stripping}$ were not significantly influenced by the rubber modification.

Table 3-2. Test of rubber addition effect on rutting parameters

Parameters	F-value	P-value	Significant effect
Creep slope	256	0.004	Yes
Stripping slope	26.56	0.036	Yes
t_s	0.092	0.790	No
N_s	218.5	0.005	Yes
N_F	505.7	0.002	Yes
$R_{stripping}$	8.77	0.098	No

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 $7.0 \pm 1.0\%$. In order to evaluate the influence of air void content on the rutting and moisture susceptibility, 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 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 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 of 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%.

Increased the air void of the asphalt mixture could reduce the stiffness and permanent deformation resistance of the asphalt mixture. After the moisture damage occurs, the larger void ratio increased the possibility of moisture entering the asphalt mixture and weakened 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

The statistical analysis was conducted to evaluate the effect of mixture air void on different rutting test parameters, as shown in Table 3-3. It is found that the number of passes to SIP and failure were significantly influenced by the mixture air void. The creep slope, stripping slope, rut depth at the SIP (t_s), and $R_{stripping}$ were not significantly influenced by the mixture air void.

Table 3-3. Test of air void effect on rutting parameters

Parameters	F-value	P-value	Significant effect
Creep slope	7.759	0.108	No
Stripping slope	5.281	0.148	No
t_s	0.464	0.566	No
N_S	283.7	0.004	Yes
N_F	25.387	0.037	Yes
$R_{stripping}$	1.133	0.399	No

3.3.2.4 Mixture aging

The influence of aging on rutting parameters of the surface layer asphalt mixture is shown in Figure 3-12. The average air void contents of the unaged and long-term aged surface layer asphalt mixture without rubber were 7.4% and 6.9%, respectively. The effect of air void on the rutting resistance of asphalt mixture with different aging degrees was not considered. Compared with unaged asphalt mixture, after long-term aging, the creep slope and stripping slope of the surface layer asphalt mixture without rubber reduced to 33% and 58%, respectively. The aging procedure significantly improved the rutting resistance of the asphalt mixture. The stripping slopes of the unaged and long-term aged surface layer asphalt mixture without rubber were 2.8 times and 4.9 times of the creep slopes, respectively. The creep slope of the long-term aged surface layer asphalt mixture without rubber was significantly reduced, thus the ratio of the stripping slope to the creep slope was increased. The number of passes required for the surface layer asphalt mixture without rubber to the SIP was 2785. After aging, the surface layer asphalt mixture without rubber significantly improved the rutting resistance. The number of passes required to SIP increased about 1.5 times. The t_s of the asphalt mixture with two different aging conditions was similar. After aging, the number of passes for surface layer asphalt mixture without rubber to fail was 234% of the unaged asphalt mixture. The $R_{stripping}$ of the two aging conditions samples was similar. The aging of the asphalt mixture significantly increased the stiffness of the surface layer asphalt mixture without rubber, reduced its creep slope and stripping slope, and improved the rutting resistance of the asphalt mixture.

The average air void contents of the unaged and long-term aged surface layer asphalt mixture with rubber were 7.5% and 7.4%, respectively. The effect of air void content on the rutting resistance of asphalt mixture with different aging degrees was not considered. After long-term aging of the asphalt mixture, the creep slope of the surface layer asphalt mixture with rubber reduced to 14% of the unaged asphalt mixture. After long-term aging of the surface layer asphalt mixture with rubber, the asphalt mixture did not reach the SIP until the end of the test (the number of passes reached 20,000 times). The rut depth of the

asphalt mixture after 20,000 passes was 4.75 mm. The long-term aged surface layer asphalt mixture with rubber didn't have the stripping slope and failure point. The rut depth of the unaged surface layer asphalt mixture with rubber reached 20 mm after 9552 passes. The aging procedure could significantly improve the rutting resistance of the surface layer asphalt mixture with rubber. The influence of aging on the moisture susceptibility of the surface layer asphalt mixture with rubber need more test to validate because there is no moisture damage data after the aging procedure.

By comparing the rut depth and the number of passes of the long-term aged surface layer asphalt mixture without rubber and unaged surface layer asphalt mixture with rubber, the two materials had similar creep slope, stripping slope, and the number of passes to SIP and failure. Both rubber modification and long-term aging could improve the rutting resistance of the asphalt mixture in the surface layer, and more tests are needed to verify the mechanism of the effects.

(a) creep slope, stripping slope and t_S

(b) N_s , N_f , and $R_{stripping}$

Figure 3-12. The influence of aging on rutting parameters of the surface layer asphalt mixture

The influence of aging on rutting parameters of the leveling layer asphalt mixture is shown in Figure 3-13. The average air void contents of the unaged and long-term aged leveling layer asphalt mixture without rubber were 7.5% and 7.1%, respectively. The effect of air void on the rutting resistance of asphalt mixture with different aging degrees was not considered. Compared with unaged asphalt mixture, after long-term aging, the creep slope and stripping slope of the leveling layer asphalt mixture without rubber reduced to 23% and 42%, respectively. The aging procedure significantly improved the rutting resistance of the asphalt mixture. The stripping slopes of the unaged and long-term aged leveling layer asphalt mixture without rubber were 3.1 times and 5.5 times of the creep slopes, respectively. The creep slope of the long-term aged leveling layer asphalt mixture without rubber was significantly reduced, thus the ratio of the stripping slope to the creep slope was increased. The number of passes required for the leveling layer asphalt mixture without rubber to the SIP was 4560. After aging, the leveling layer asphalt mixture without rubber significantly improved the rutting resistance. The number of passes required to SIP increased about 3.0 times. The t_s of the asphalt mixture decreased slightly after the aging procedure. After aging, the number of passes for

leveling layer asphalt mixture without rubber to fail was 295% of the unaged asphalt mixture. The $R_{stripping}$ of the asphalt mixture increased slightly after the aging procedure. The aging of the asphalt mixture significantly increased the stiffness of the leveling layer asphalt mixture without rubber, reduced its creep slope and stripping slope, and improved the rutting resistance of the asphalt mixture.

The average air void contents of the unaged and long-term aged leveling layer asphalt mixture with rubber were 7.4% and 7.5%, respectively. The effect of air void content on the rutting resistance of asphalt mixture with different aging degrees was not considered. After long-term aging of the asphalt mixture, the creep slope of the leveling layer asphalt mixture with rubber reduced to 50% of the unaged asphalt mixture.

The asphalt mixtures with two different aging conditions did not reach the SIP until the end of the test. The leveling layer asphalt mixture with rubber didn't have the stripping slope and failure point. The rut depth of the unaged and aged leveling layer asphalt mixture after 20,000 passes were 7.5 mm and 5.14 mm, respectively. The influence of aging on the moisture susceptibility of the leveling layer asphalt mixture with rubber need more test to validate because there is no moisture damage data after the aging procedure.

(a) creep slope, stripping slope and t_s

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

Figure 3-13. The influence of aging on rutting parameters of the leveling layer asphalt mixture

The statistical analysis was conducted to evaluate the effect of mixture aging on different rutting test parameters, as shown in Table 3-4. It is found that the creep slope, number of passes to SIP and failure were significantly influenced by the mixture design parameters. The stripping slope, rut depth at the SIP (t_s), and $R_{stripping}$ were not significantly influenced by the mixture design parameters.

Table 3-4. Test of aging effect on rutting parameters

Parameters	F-value	P-value	Significant effect
Creep slope	150	0.001	Yes
Stripping slope	1.402	0.321	No
t_s	0.006	0.945	No
N_s	33.05	0.010	Yes
N_F	89.20	0.003	Yes
$R_{stripping}$	6.231	0.088	No

3.3.3 The two-way interactive effect analysis of different parameters

Based on the ANOVA of the previous chapter, the creep slope, stripping slope, the number of passes to SIP and failure were significantly affected by mixture design factors, rubber modification, mixture air void content, and aging. The rut depth at the SIP and $R_{stripping}$ were not influenced by these parameters. It is necessary to find whether these factors had a significant interactive effect on those rutting parameters. Thus, the two-way interactive ANOVA analysis was conducted. During the rutting test, the leveling layer asphalt mixture with rubber didn't reach the SIP until the end of the test, and the leveling layer didn't have the stripping slope and number of passes to SIP. The surface layer asphalt mixture with rubber after aging also didn't have the stripping slope and number of passes to SIP. The lack of data will significantly influence the analytical test result. Thus, the interactive effect of mixture design factor and rubber addition on stripping slope and number of passes to SIP was not analyzed. The two-way interactive ANOVA analysis of the interactive effect of different mixture factors on creep slope and number of passes to fail were conducted.

(1) The mixture design factor and rubber modification interactive effect

The interactive effect of mixture design factor and rubber modification was analyzed, and the two-way ANOVA results are shown in Table 3-5. Based on the results, mixture design factor and rubber addition had a significant influence on the creep slope of different asphalt mixtures, but there was no significant interactive effect of these two parameters on the creep slope of asphalt mixture. While the mixture design factor and rubber addition had a significant effect on the number of passes to fail of different asphalt mixtures. And the interaction between mixture design factor and rubber modification was significant.

Table 3-5. The interactive effect of mixture design factor and rubber addition on rutting parameters

Parameters		F-value	P-value	Significant effect
Creep slope	Mixture design	49	0.002	Yes
	Rubber	386.8	3.9E-5	Yes
	Interaction	2.778	0.171	No
Number of passes to fail	Mixture design	2733.7	8.0E-7	Yes
	Rubber	6570.3	1.4E-7	Yes
	Interaction	1376.4	3.2E-6	Yes

(2) The air void and rubber modification interactive effect

The interactive effect of air void and rubber modification was analyzed, and the two-way ANOVA results are shown in Table 3-6. The air void and rubber addition had a significant effect on the creep slope of different asphalt mixtures. And there was a significant interactive effect of those two factors. None of those two factors had a significant effect on the number of passes to fail, and there was no significant interactive effect of those two factors. This may be because some asphalt mixtures failed when the number of passes reached 20000, while some samples failed when the rutting depth reached 20 mm. Different test fail criteria were met at the end of the test, thus influenced the analytical test results.

Table 3-6. The interactive effect of air void and rubber addition on rutting parameters

Parameters		F-value	P-value	Significant effect
Creep slope	Air void	4.8E30	2.1E-31	Yes
	Rubber	2.74E30	3.6E-31	Yes
	Interaction	2.74E30	3.6E-31	Yes
Number of passes to fail	Air void	0.2562	0.6630	No
	Rubber	0.2378	0.8079	No
	Interaction	0.2378	0.8079	No

(3) The aging and rubber modification interactive effect

The interactive effect of aging and rubber modification was analyzed, and the two-way ANOVA results are shown in Table 3-7. At the 0.05 level, the aging and rubber addition had a significant effect on creep slope and number of passes to fail of different asphalt mixtures. And the interaction between aging and rubber modification was significant.

Table 3-7. The interactive effect of aging and rubber addition on rutting parameters

Parameters		F-value	P-value	Significant
Creep slope	Aging	441.8	3.0E-5	Yes
	Rubber	336.2	5.2E-5	Yes
	Interaction	105.8	5.0E-4	Yes
Number of passes to fail	Aging	1496.2	2.7E-6	Yes
	Rubber	887.5	7.6E-6	Yes
	Interaction	52.1	0.002	Yes

(4) The aging and mixture design factor interactive effect

The interactive effect of aging and mixture design factor was analyzed, and the two-way ANOVA results are shown in Table 3-8. The aging and mixture design factor had a significant effect on creep slope and number of passes to fail of different asphalt mixtures. And the interaction between aging and mixture design factor was significant.

Table 3-8. The interactive effect of aging and mixture design factor on rutting parameters

Parameters		F-value	P-value	Significant
Creep slope	Mixture design	313.6	3.9E-4	Yes
	Aging	14.4	0.032	Yes
	Interaction	672.4	1.3E-4	Yes
Number of passes to fail	Mixture design	241.2	5.8E-4	Yes
	Aging	1170.5	5.5E-5	Yes
	Interaction	87.9	0.003	Yes

The interaction between aging and air void, and mixture design factor and air void were not displayed, due to no significant effect of these factors were found, and there was no significant interaction effect occurred between those factors. Based on the two-way ANOVA interactive analysis of different factors, the rubber addition and aging were two more critical factors that influenced the rutting performance of different asphalt mixtures. The definition of test end criteria also brought difficulty to the analytical result comparison. A parameter combined with the rut depth and the number of passes to fail may improve the possibility of the test result comparison between different asphalt mixtures.

3.4 Chapter summary

This chapter analyzed the high temperature rutting performance of different asphalt mixtures. The influence of mixture design factor, rubber addition, air void, and aging

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 NMA, 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 failure 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 failure were significantly influenced by the mixture air void.
- (4) The rutting resistance of different asphalt mixture was improved after the aging procedure. The influence of aging on the moisture susceptibility of the leveling layer asphalt mixture with rubber need more test to validate because there is no moisture damage data after the aging procedure. The creep slope, number of passes to SIP and failure were significantly influenced by the mixture design parameters.
- (5) Based on the two-way ANOVA interactive analysis of different factors, the rubber addition and aging were two more critical factors that influenced the rutting performance of different asphalt mixtures.

4 Low Temperature Cracking Performance Evaluation of Rubber Modified Asphalt Mixture Using the Dry Process¹

Extractive Summary

The low temperature cracking is the main distresses occurred in asphalt pavement in cold weather climates. The main scope of this chapter is to evaluate the low temperature fracture properties of asphalt mixture with rubber using the dry process and to compare with those of asphalt mixture without rubber. The mixtures were produced in the plant with the dry process and compacted in the laboratory. Four types of asphalt mixtures, including LVSP and 5E1 gradation with and without crumb rubber, were produced in the plant in Iron Mountain, MI. The volumetric design of the mixture followed the Superpave mixture design shown in chapter 2. The low temperature performance of four types of asphalt mixtures was evaluated using the Disk-shaped Compact Tension (DCT) test at -24 °C. The low temperature performance of asphalt mixture with rubber was improved compared with base asphalt mixture. The low temperature performance of 5E1 mixture was higher than that of the LVSP mixture. The reason may be due to the higher asphalt binder content and lower RAP content of the 5E1 mixture. The cracking energy of the 5E1 mixture with rubber was 1.52 times higher than the 5E1 mixture without rubber. The cracking energy of the LVSP mixture with rubber was 1.37 times higher than the LVSP mixture without rubber. The excellent performance of asphalt mixture with rubber in low temperature suggests that it can be an alternative material for asphalt pavement in cold weather climates.

4.1 Introduction

Low temperature cracking is the main asphalt mixture distress that occurred in wet-freeze regions. The cracking caused by low temperature accelerated the distress of the asphalt

¹ Part of the text accepted in the RILEM International Symposium on Bituminous Materials – ISBM LYON 2020 in Lyon, France.

mixture pavement and reduced the surface life of the pavement [151]. Therefore, the evaluation of the low-temperature cracking performance of asphalt mixtures is particularly important for the pavement applied in the wet-freeze regions [152].

In the past few decades, different test methods were adopted to study the low temperature cracking performance of asphalt mixture and to choose materials that had 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 [153]. The semi-circular bend (SCB) and the disc-shaped compact tension (DCT) were used by researchers to evaluate the low temperature crack resistance of asphalt mixtures [154, 155]. The fracture energy of SCB samples should be greater than 350 J/m^2 , and the fracture toughness should be higher than $\text{kPa}\cdot\text{m}^{1/2}$ [156]. 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 [157].

4.2 Test materials and test method

4.2.1 Test materials and test sample preparation

Chapter 2 described the volumetric mixture design of rubber modified asphalt mixture with the dry process. The leveling layer and the surface layer used different mixture gradation and asphalt binder content. In order to compare the low temperature cracking susceptibility of rubber modified 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)).

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 holes 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 red line in Figure 4.1 (c) is the loading direction, and the green line is the anticipated cracking direction. 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 $7.0 \pm 1.0\%$. In order to guarantee the repeatability of the test results, at least three replicate test specimens were prepared for each test condition.

In order to study the effect of long-term aging on the low temperature cracking resistance of asphalt mixtures, some SGC samples were long-term aged in the oven at $85 \pm 3^\circ\text{C}$ for 120 ± 0.5 hours before conducting the DCT test, based on the AASHTO R30 standard.

(a) SGC sample

(b) Two 50 mm thickness sample

(c) Final DCT test sample

Figure 4-1. The sample preparation procedure of DCT test

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

4.2.2.1 Test conditions

In this study, the DCT test equipment produced by TestQuip LLC was adopted to assess the low-temperature cracking performance of the asphalt mixture, as shown in Figure 4.3. The DCT test could be adopted to assess the cracking resistance of the asphalt mixture, and the influence of cracking on the service life could be assessed. According to ASTM D6373, the DCT test temperature should be 10 °C higher than the low Performance Grade (PG) temperature of the asphalt binder used in the asphalt mixture. The PG grade of asphalt binder is PG 58-34, so the test temperature chosen -24 °C. In order to analyze the effect of test temperature on the low-temperature cracking performance of asphalt mixture, the test was also conducted at -18 °C. The test specimens were conditioned at the test temperature for at least two hours before conducting the test to guarantee the test specimen temperature reached the target temperature. 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. 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 ($W_f^{Pre-peak}$) and the post-peak cracking work ($W_f^{Post-peak}$). The cracking work can be calculated using the quadrangle rule, as shown in Equation 4.1.

$$\begin{aligned}
 W_f &= W_f^{Pre-peak} + W_f^{Post-peak} \\
 &= \int_0^{\Delta F_{max}} F \cdot dx + \int_{\Delta F_{max}}^{\Delta F_{final}} F \cdot dx
 \end{aligned}
 \tag{4.1}$$

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),

ΔF_{final} 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.

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

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).

The post-peak slope could reflect the flexible property of the asphalt mixture during the DCT test, and the average post-peak slope (m_{average}) was calculated, as shown in Figure 5. The flexibility index (G_f/m_{average}) was proposed by Zhu to consider the softening behavior of the asphalt mixture during the fracture cracking procedure [158].

Figure 4-5. The flexibility index parameters of the DCT test

Perez Jimenez proposed a normalized fracture energy parameter (Toughness index, TI), which is calculated by multiplying the post-peak fracture energy with displacement between peak load and 50% peak load (as shown in Figure 5) [158]. The equation to calculate TI is shown below.

$$TI = (G_f^{Post-peak}) \cdot (\Delta_{mdp} - \Delta_{Fmax}) \cdot 10^{-3} \quad (4.3)$$

Where:

TI is the toughness index (J/m^2),

$G_f^{Post-peak}$ is the post-peak fracture energy (J/m^2),

Δ_{mdp} is the CMOD value at the 50% peak load (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 4-6. The loading direction and the anticipated cracking direction was

displayed in Figure 4.1 (c). 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 influenced 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 segregation of mixture. 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 4-6. The cracking path of the DCT test sample

The relationship between DCT test load and CMOD of different types of asphalt mixtures at -24 °C is shown in Figure 4.7. 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 nominal maximum aggregate size 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. The average fracture energy of the surface layer with rubber (5E1-Rubber) and the surface layer without rubber (5E1-CTRL) are 569 J/m^2 and 463.5 J/m^2 , respectively. The average fracture energy of the leveling layer with rubber (LVSP-Rubber) and the surface layer without rubber (LVSP-CTRL) are 526 J/m^2 and 385 J/m^2 , respectively. The fracture energy of the surface layer asphalt mixture was higher than that of the leveling layer. And the fracture energy was increased with the addition of the rubber particle. The maximum CMOD of the asphalt mixture had a similar changing trend as the fracture energy for different types of asphalt mixture. The correlation between the maximum CMOD and the fracture energy will be discussed later.

Figure 4-7. 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 performance of asphalt mixtures were affected by the mixture design factors of the asphalt mixture (nominal maximum aggregate size (NMAS), RAP content, and asphalt content), rubber modification, test temperature, and aging procedure. 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 various 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 are 25% and 17%, respectively. The asphalt content of the surface layer asphalt mixture and the leveling layer asphalt mixture are 5.9% and 4.2%, respectively.

The fracture energy of different asphalt mixtures at the test temperature of -24°C is shown in Figure 4.8. 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 are 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 cracking distress 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 are 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 4-8. The influence of mixture design on fracture energy

The peak load of different asphalt mixtures at the test temperature of -24°C is shown in Figure 4.9. 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 4-9. The influence of mixture design on peak load

The maximum CMOD of different asphalt mixtures at the test temperature of -24°C is shown in Figure 4.10. 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 4-10. The influence of mixture design on maximum CMOD

The statistical analysis was conducted to evaluate the effect of mixture design factors on fracture energy, peak load, and maximum CMOD, as shown in Table 4-2. It is found that at the 0.05 level, the fracture energy and maximum CMOD were significantly influenced by the mixture design factors. At the 0.05 level, the peak load was not significantly influenced by the mixture design factors.

Table 4-2. Test of mixture design factors effect on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	146.4	2.7E-4	Yes
Peak load	4.01	0.12	No
Maximum CMOD	115.2	4.3E-4	Yes

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 4-11. 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 °C to -18 °C. The fracture energy of the leveling layer increased by 19.7% when the test temperature increased from -24 °C to -18 °C. In the previous section, the influence of mixture design on the fracture energy was analyzed. 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 °C was still higher than the average fracture energy of the leveling layer at -18 °C. 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 4-11. 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 4.12. The peak load of test samples 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 °C to -18 °C. The average peak load of the leveling layer asphalt mixture decreased by 4.7% when the test temperature increased from -24 °C to -18 °C. 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 4-12. 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 4.13. 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 °C to -18 °C. The average maximum CMOD of the leveling layer asphalt mixture increased by 23.1% when the test temperature increased from -24 °C to -18 °C. With the increase of test temperature, the brittleness of the asphalt mixture decreased, and the elastic property of the asphalt mixture increased thus increased the maximum CMOD of asphalt mixture.

Figure 4-13. The influence of test temperature on maximum CMOD

The statistical analysis was conducted to evaluate the effect of test temperature on fracture energy, peak load, and maximum CMOD, as shown in Table 4-3. It is found that at the 0.05 level, the fracture energy and maximum CMOD were significantly influenced by the test temperature. At the 0.05 level, the peak load was not significantly influenced by the test temperature.

Table 4-3. Test of test temperature effect on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	73.4	3.7E-6	Yes
Peak load	2.98	0.096	No
Maximum CMOD	21.55	3.5E-4	Yes

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 4.14. 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, 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 did 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 °C was better than that at -18 °C. 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 4-14. The influence of rubber modification on fracture energy

The effect of the rubber modification on the peak load of different DCT test specimens is shown in Figure 4.15. The peak load of the asphalt mixtures increased slightly after adding rubber. At -24 °C, the average peak load of the surface layer and the leveling layer asphalt mixture increased 5.3% and 13.6%, respectively. At -18 °C, 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 4-15. The influence of rubber modification on peak load

The effect of rubber modification on the maximum CMOD of DCT test is shown in Figure 4.16. The maximum CMOD of asphalt mixtures was increased significantly after rubber modification. At the test temperature of -24 °C, 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 °C, 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 4-16. The influence of rubber modification on maximum CMOD

The statistical analysis was conducted to evaluate the effect of rubber modification on fracture energy, peak load, and maximum CMOD, as shown in Table 4-4. It is found that at the 0.05 level, the fracture energy, peak load, and maximum CMOD were significantly influenced by rubber modification.

Table 4-4. Test of rubber modification effect on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	36.86	2.4E-8	Yes
Peak load	15.11	1.5E-5	Yes
Maximum CMOD	11.16	1.3E-4	Yes

4.3.2.4 Asphalt mixture aging

The influence of aging on the fracture energy of different asphalt mixtures is shown in Figure 4.17. The low temperature fracture energy of asphalt mixtures reduced after the aging process. At the test temperature of -24°C , the average fracture energy of aged asphalt mixtures reduced to about 90% of the average fracture energy of unaged asphalt mixtures. At the test temperature of -18°C , the average fracture energy of aged asphalt mixtures reduced to about 95% of the average fracture energy of unaged asphalt mixtures. At a lower test temperature (-24°C), aging had a greater effect on the fracture energy of asphalt mixtures. After the aging of the asphalt mixture, the low temperature brittleness of the asphalt mixture became more obvious, and the low temperature cracking resistance decreased. The fracture energy increased when the test temperature increased from -24°C to -18°C . The fracture energy increase for aged asphalt mixture was higher than the unaged asphalt mixture. The influence of temperature change on the fracture energy was greater after the asphalt mixture aging.

For the surface layer asphalt mixture after aging, the rubber modification increased the fracture energy 12.4% and 16.9% at -24°C and -18°C , respectively. The improvement effect of rubber modification was slightly weaker than the unaged asphalt mixtures. A similar situation was observed for the leveling layer asphalt mixture, but the increase of fracture energy was higher. The aging process weakened the rubber modification effect on fracture energy enhancement. But the average fracture energy improvement of rubber modification for different asphalt mixtures under different test conditions is 23.9%. The rubber modification still significantly increased the low temperature cracking performance of asphalt mixtures.

Figure 4-17. The influence of aging on fracture energy of different asphalt mixtures

The influence of aging on the peak load of different asphalt mixtures is shown in Figure 4.18. The peak load of the asphalt mixtures increased slightly after the aging process. The increase was minimal except for the leveling layer asphalt mixture without rubber at -24 °C and the leveling layer asphalt mixture with and without rubber at -18°C. For the surface layer asphalt mixture after aging, the rubber modification increased the peak load of 6.9% and 8.0% at -24°C and -18°C, respectively. A similar situation was observed for the leveling layer asphalt mixture. The influence of aging on the peak load was not as significant as that on the fracture energy.

Figure 4-18. The influence of aging on peak load of different asphalt mixtures

The influence of aging on the maximum CMOD of different asphalt mixtures is shown in Figure 4.19. The maximum CMOD of the asphalt mixture was reduced after the aging of asphalt mixtures. At the test temperature of -24°C, the average maximum CMOD of aged asphalt mixtures reduced to about 90% of the average maximum CMOD of unaged asphalt mixtures. At the test temperature of -18°C, the average maximum CMOD of aged asphalt mixtures reduced to about 95% of the average maximum CMOD of unaged asphalt mixtures. At a lower test temperature (-24°C), aging had a greater effect on the maximum CMOD of asphalt mixtures. After the aging of the asphalt mixture, the elastic property of asphalt mixtures was reduced, and the deformation resistance of asphalt mixtures at low temperatures was reduced. The combined effect of low temperature and aging significantly reduced the deformation resistance of asphalt mixtures.

Figure 4-19. The influence of aging on maximum CMOD of different asphalt mixtures

The statistical analysis was conducted to evaluate the effect of the aging procedure on fracture energy, peak load, and maximum CMOD, as shown in Table 4-5. It is found that at the 0.05 level, the fracture energy, peak load, and maximum CMOD were significantly influenced by the aging procedure.

Table 4-5. Test of aging effect on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	13.28	1.6E-6	Yes
Peak load	3.967	0.0047	Yes
Maximum CMOD	5.748	0.0011	Yes

4.3.3 The two-way interactive effect analysis of different factors

Based on the one-way ANOVA, fracture energy, peak load, and maximum CMOD were significantly influenced by the asphalt mixture design factor, test temperature, rubber modification, and aging procedure. It is necessary to find whether these factors had a significant interactive effect on those low temperature cracking parameters. The two-way interactive ANOVA analysis was conducted between the mixture design factor, test temperature, rubber modification, and aging procedure. Based on two-way ANOVA analysis, the interactive effect occurred between rubber modification and aging procedure, and between mixture design factor and aging procedure.

(1) The mixture design factor and aging procedure interactive effect

The interactive effect of mixture design factor and aging procedure was analyzed, and the two-way ANOVA results are shown in Table 4-6. Based on the results, mixture design factor and aging procedure had a significant influence on the fracture energy of different asphalt mixtures, but there was no significant interactive effect of these two parameters on the peak load and maximum CMOD of asphalt mixture.

Table 4-6. The interactive effect of mixture design factor and aging procedure on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	7.401	0.026	Yes
Peak load	4.881	0.058	No
Maximum CMOD	2.817	0.132	No

(2) The Rubber modification and aging procedure interactive effect

The interactive effect of rubber modification and the aging procedure was analyzed, and the two-way ANOVA results are shown in Table 4-7. Based on the results, rubber modification and aging procedure 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.

Table 4-7. The interactive effect of rubber modification and aging procedure on low temperature cracking parameters

Parameters	F-value	P-value	Significant effect
Fracture energy	20.64	0.002	Yes
Peak load	1.432	0.266	No
Maximum CMOD	6.144	0.038	Yes

No significant interactive effect occurred between the other two factors on fracture energy, peak load, and maximum CMOD. Rubber modification and aging were two critical factors that influenced the low temperature cracking property of different asphalt mixture.

4.4 The correlation between fracture energy and other parameters

4.4.1 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 4-20. 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.

The aging procedure reduced the elastic property of the asphalt mixture at low temperatures. The maximum CMOD, fracture energy, and the low temperature cracking performance of the asphalt mixture were decreased after the aging procedure.

Figure 4-20. The correlation between the fracture energy and the maximum CMOD

(a) Load - CMOD correlation

(b) Load - Energy correlation

Figure 4-21. 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 is displayed in Figure 4-21. 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 and aging procedure 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 significant improvement in the fracture energy.

4.4.2 The correlation between fracture energy and post-peak fracture energy.

The correlation between the fracture energy and post-peak fracture energy of asphalt mixtures is shown in Figure 4.22. There was a very significant linear correlation between the fracture energy and the post-peak fracture energy ($p < 0.0001$), where the coefficient of determination $R^2 = 0.9745$. For different types of asphalt mixtures under different test temperatures, the post-peak fracture energy was approximately 86% of the overall fracture energy. During the DCT test, the load increased dramatically to the peak load with an increase of CMOD. The average of pre-peak fracture energy was only 14% of the total fracture energy. The load decreasing rate of different asphalt mixtures after the peak load was different from the continuing increase of the CMOD. The post-peak fracture energy reflected the dissipated energy during the propagation of crack inside the specimen.

Figure 4-22. The correlation between fracture energy and post-peak fracture energy

The correlation between the fracture energy and post-peak fracture energy of unaged and aged asphalt mixtures are shown in Figure 4.23. The post-peak fracture energy was approximately 87.5% of the total fracture energy for unaged asphalt mixture (Figure 4.23 (a)). The post-peak fracture energy was approximately 84.5% of the total fracture energy for aged asphalt mixture (Figure 4.23 (b)). The elastic property and the low temperature cracking property of the asphalt mixtures were reduced after the aging procedure. The capability of energy dissipation was reduced during the post-peak period. Thus the ratio of post-peak fracture energy to the total fracture energy was reduced.

(a) Unaged

(b) Aged

Figure 4-23. The correlation between fracture energy and post-peak fracture energy of unaged asphalt mixture

4.4.3 The correlation between the fracture energy and flexibility index (G_f/m_{average})

The correlation between the fracture energy and flexibility index (G_f/m_{average}) of asphalt mixtures is shown in Figure 4.24. The fracture energy had a very significant exponential

correlation with the flexibility index ($p < 0.0001$), in which the determination of coefficient $R^2 = 0.849$. The flexibility index of asphalt mixture can be used to characterize the elastic property and cracking resistance of asphalt mixture at low temperatures. Asphalt mixture with larger flexibility index had higher fracture energy and low temperature cracking resistance in the low temperature DCT test.

Figure 4-24. The correlation between the fracture energy and flexibility index ($G_f/m_{average}$)

4.4.4 The correlation between the fracture energy and toughness index (TI)

The correlation between the fracture energy and toughness index (TI) of asphalt mixtures is shown in Figure 4.25. The fracture energy had a very significant exponential correlation with the toughness index ($p < 0.0001$), in which the determination of coefficient $R^2 = 0.8909$. The flexibility index of asphalt mixtures increased exponentially with the increase of the fracture energy of different asphalt mixtures. Due to the significant increase of the flexibility index, the flexibility index could be easier to distinguish the low temperature property of different asphalt mixtures than the fracture energy.

Figure 4-25. The correlation between fracture energy and toughness index (TI)

The relationship between the flexibility index and the toughness index of asphalt mixtures is shown in Figure 4.26. The toughness index had a very significant linear correlation with the flexibility index ($p < 0.0001$), where the coefficient of determination $R^2 = 0.7636$.

The flexibility index, toughness index, and fracture energy could characterize the fracture process of asphalt mixture during the low temperature DCT test. The flexibility index assessed the low temperature cracking performance of asphalt mixtures with the slope of the load-CMOD curve after the asphalt mixture reached the peak load. The toughness index used the CMOD after the peak load and the deformation during the post-peak period to evaluate the low temperature cracking property of asphalt mixtures. The fracture energy could directly reflect the energy required to break the asphalt mixture to characterize the low temperature cracking performance of asphalt mixtures.

Figure 4-26. The correlation between flexibility index and toughness index (TI)

4.5 Chapter summary

In this chapter, the low temperature cracking properties of four types of plant mixed and laboratory compacted asphalt mixtures were evaluated with the DCT test. The influence of mixture design factor, test temperature, rubber modification, and aging 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 displayed as follows, according to the test results.

- (1) The cracking path do not follow the shortest failure path, and the cracking path was influenced 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 performance 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 adhesion between the asphalt and the aggregate, 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 °C to -18 °C, 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) The low temperature fracture energy of asphalt mixtures reduced after the aging process. At the test temperature of -24°C and -18°C, the average fracture energy of aged asphalt mixtures reduced to about 90% and 95% of the average fracture energy of unaged asphalt mixtures, respectively.
- (6) Based on the results, rubber modification and aging procedure 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.
- (7) There was a significant linear correlation between the fracture energy and the maximum CMOD and the post-peak fracture energy. The fracture energy had a significant exponential correlation with the flexibility index and the toughness index. The toughness index had a very significant linear correlation with the flexibility index.

5 The performance of asphalt binder with trichloroethylene (TCE): improving the efficiency of using reclaimed asphalt pavement (RAP) ²

Extractive Summary

Asphalt binder in reclaimed asphalt pavement (RAP) affects the design of asphalt mixture since most of the asphalt mixtures currently adopted quite an amount of RAP. The efficiency of using RAP in mixture design was influenced by the properties of asphalt binder in RAP. The goals of this chapter are to investigate the effect of solvent in asphalt binder and to propose a method to determine the actual performance of asphalt binder extracted from asphalt mixture. The Fourier-transform infrared spectroscopy (FTIR) test was used to analyze the functional groups of TCE and asphalt binder with TCE and to qualitatively and quantitatively evaluate the influence of solvent in asphalt binder. Dynamic Shear Rheometer (DSR) test was employed to evaluate the complex shear modulus (G^*) of asphalt binder with different TCE contents and to establish the relationship between the TCE contents and the G^* of asphalt binder. Based on the testing results, functional group =C-H of TCE can be used as an indicator to verify the existence of TCE in extracted asphalt binder. Both band areas $834\text{-}848\text{ cm}^{-1}$ and $918\text{-}938\text{ cm}^{-1}$ changed for PG 58-28 and Pressure Aging Vessel (PAV) aged PG 58-28 after the addition of TCE. Band area $918\text{-}938\text{ cm}^{-1}$ can better reflect the influence of TCE in asphalt binder. Band area $834\text{-}848\text{ cm}^{-1}$ does not change when TCE content was at a low level. Both the existence of TCE and the content of TCE had a significant influence on the $|G^*|$ of unaged and PAV aged PG 58-28 asphalt binder. A regression function was built to establish the relationship between TCE content and G^* . With the method developed in this study, the correlation between TCE amount the FTIR spectrum area was quantified. The actual complex shear modulus of asphalt binder without TCE can be back calculated. The method used in this study provided guidance for the determination

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of asphalt binder performance extracted from RAP, which can be beneficial to the design of asphalt mixtures with RAP.

5.1 Introduction

Using reclaimed asphalt pavement (RAP) in pavement construction can reduce construction budgets and retain environmental benefits. Dealing with RAP improperly will cause the waste of natural resources and severe environmental problems [92, 159]. The cost of asphalt binder and aggregate occupied the majority of expenses spent on pavement construction. The construction cost was highly dependent on the percentage of RAP used in pavement [160]. The costs of RAP mainly included RAP processing, rejuvenator adoption, material performance evaluation, and waste management. The fresh asphalt binder and aggregates used in pavement construction were reduced with the application of RAP materials [161]. The energy consumption during the construction of pavement was reduced because RAP was used during the production of asphalt mixture. At the same time, emissions were reduced during the heating process. The cost to handle the RAP materials in the landfill procedure was also decreased [162].

Even though the adoption of RAP materials relieved the pressure of waste RAP on the environment, and the cost of pavement was reduced, there were many difficulties during the asphalt mixture design containing RAP [163]. Without a proper design, the performance of the asphalt mixture with RAP cannot be guaranteed. Thus, the service life of the pavement will be influenced, and the maintenance cost will be increased [164]. The percentage of RAP in asphalt pavement was limited due to the performance of RAP. The cracking performance and rutting characteristics of asphalt mixture with RAP were highly dependent on the aging condition of RAP [165]. The fatigue cracking and low temperature cracking were more likely to occur on asphalt mixture with RAP, which had high stiffness. The rutting resistance of mixture with RAP was normally higher than a mixture without RAP because the aged asphalt binder in RAP increased the stiffness of RAP. Thus, it increased the rutting resistance of the mixture [166, 167]. The quality of the asphalt mixture used to produce RAP influenced the performance of the asphalt

mixture with RAP. The moisture susceptibility of the asphalt mixture with RAP was better when compared with asphalt mixture without RAP if the bonding between aggregate and asphalt binder was good in RAP. Special treatment should be considered if the original RAP had stripping issues [132].

During the production of asphalt mixture with RAP, the blending efficiency of asphalt binder in RAP was a critical issue, especially when the RAP was at a high percentage in the final asphalt mixture design [168, 169]. Previous studies used different strategies to evaluate the blending efficiency of asphalt mixture with RAP, which included gel permeation chromatography, fluorescence microscopy, molecular dynamics, and Fourier Transform Infrared Spectroscopy [170-173]. The influence of mixing time, mixing temperature, and the addition of Warm Mix Asphalt (WMA) additives on blending efficiency was evaluated by Bowers. The results found that the mixing time had minimal influence; the mixing temperature and the inclusion of WMA additive had a significant positive effect on the blending efficiency [174].

The other critical issue in asphalt mixture with RAP was the asphalt binder content and the asphalt binder property from RAP fraction [131]. The asphalt binder from RAP was normally acquired by extracting asphalt binder from the RAP, and then the asphalt binder content and asphalt binder property were determined [175, 176]. During asphalt binder extraction and recovery procedure, asphalt binder was dissolved in the solvent by soaking asphalt mixture in the solvent, in which trichloroethylene (TCE) was normally used. The RAP, which was washed with solvent, was centrifuged and filtered to separate fine aggregate. The asphalt binder solution was distilled under vacuum in a rotary evaporator, in which the solvent evaporated and asphalt binder remained in the flask. The amount of asphalt binder in RAP can be acquired, and the physical and chemical performance of recovered asphalt binder can be evaluated. The mineral filler can be separated with a high-quality filter process. The solvent may not be totally separated even when heat and pressure are used in the recovery procedure, especially when the asphalt binder is polymer modified [177]. The existence of the solvent in extracted asphalt binder can be evaluated with the Fourier-transform infrared spectroscopy (FTIR) test method [173].

FTIR has been widely used to evaluate the asphalt binder performance [120, 178, 179]. The chemical composition of asphalt binders can be verified by analyzing the infrared absorption of FTIR spectrum [106, 180]. The sequential dissolution of asphalt fraction with six different solvents during the asphalt extraction procedure was evaluated, which found the mean carbonyl index was not significantly influenced by solvent types [181]. The aging mechanism of bio-asphalt was investigated by comparing the carbonyl and sulphoxide compounds under different conditions [182]. FTIR is a reliable quantification method to evaluate the performance of the asphalt binder [183, 184].

5.2 Objectives and Scope

The research was motivated by the chemical solvents that were difficult to be completely separated from the asphalt binder during the solvent extraction procedure. The remaining solvent in asphalt binder had a great effect on the property of asphalt binder from RAP. The influence of the solvent remaining in the asphalt binder after the extraction process needs to be quantified. The primary objectives of this research are to investigate the effect of solvent in asphalt binder and to propose a method to determine the actual performance of asphalt binder extracted from asphalt mixture, thus improving the efficiency of using RAP. The Fourier-transform infrared spectroscopy (FTIR) test was conducted on TCE, asphalt binder, and asphalt binder with TCE to determine the influence of TCE contents on the functional groups of asphalt binder with TCE. Dynamic Shear Rheometer (DSR) was used to evaluate the influence of TCE contents on the complex shear modulus of unaged and aged asphalt binder with TCE. By combining the FTIR and DSR test results, this research provided guidance to estimate the actual complex shear modulus of asphalt binder extracted from the asphalt mixture.

5.3 Materials and Methods

5.3.1 Materials

Base asphalt binder with performance grading PG 58-28 and Pressure Aging Vessel (PAV) aged PG 58-28 were used in this study. Asphalt binder extraction procedure on

newly mixed asphalt mixture contains unaged asphalt binder can be used to examine the gradation and asphalt content. Before the design of asphalt mixture with RAP, the binder (PAV aged) performance in RAP needs to be determined [185]. Table 5-1 shows detailed properties of asphalt binder PG 58-28. The rotational viscosity, rutting parameter $G^*/\sin \delta$, and fatigue parameter $G^* \cdot \sin \delta$ met the Superpave PG 58-28 requirements.

Table 5-1. The properties of asphalt binder PG 58-28

Test properties	Test results	Specification
Specific gravity	1.030	—
Rotational viscosity @135 °C (Pa S)	0.320	<3.0
$G^*/\sin \delta$ @ 58 °C for virgin binder (kPa)	1.218	>1.0
$G^*/\sin \delta$ @ 58 °C for RTFO residue (kPa)	2.916	>2.2
$G^* \times \sin \delta$ @ 19 °C for PAV residue (kPa)	3,695	<5000

5.3.2 Sample preparation

In order to evaluate the influence of TCE in asphalt binder residue after extracting it from asphalt mixture, TCE with known content (1%, 2%, 3%, 4%, 5%, and 6% by weight of asphalt binder) was added to the PG 58-28 and PAV aged PG 58-28 asphalt binder. A high shear mixing machine was used to mix asphalt binder with TCE. The DSR and FTIR test samples were prepared immediately after the mixing procedure finished, in order to reduce the test errors because of the evaporation of TCE during the mixing process at high temperatures. The FTIR sample was prepared by taken a small amount of binder by a spatula and applied onto the Si chip directly. The asphalt sample was contacted with crystal during the test, and constant pressure was applied to the Si chip by the holding arm of the device.

5.3.3 FTIR methodology

The FTIR test method was used to analyze the functional groups of the asphalt binder. The molecular chemical bonds can be detected based on the interaction between the

infrared light beam and the functional groups in the asphalt binder. The functional group vibrated under certain patterns when the asphalt binder was charged by infrared light. By analyzing the absorbance of infrared light during certain wave numbers, the intensity of certain functional groups can be determined. In this research, the Jasco FT/IR 4200 FTIR spectrometer with MIRacle ATR accessory was used to evaluate functional group changes after TCE was mixed with asphalt binder. The ATR accessory made the FTIR less sensitive to the sample thickness so as to simplify the FTIR test sample preparation procedure. For each sample, 64 scans were adopted, and the spectra limit ranged from 600-4000 cm^{-1} with a resolution of 4 cm^{-1} . After each scan, ATR correction was used to reduce the error from the background. In order to guarantee the repeatability of the test results, at least three replicate test samples were scanned for each condition of the FTIR test. The detailed test parameters are displayed in Figure 5-1.

Figure 5-1. The test plan outlines for evaluating of influence of TCE

5.3.4 DSR test

DSR was used to evaluate the high-temperature rheological properties of asphalt binders. The PG 58-28 asphalt binder and asphalt binder with TCE samples were set between two

parallel plates with a gap of 1 mm and a diameter of 25 mm. Nine different temperature-sweeps (increasing from 34°C to 82°C with an interval of 6°C) were conducted for each sample. Six different frequencies (0.1Hz, 1Hz, 1.59Hz, 3Hz, 5Hz, 10 Hz) were used under each temperature. The test was conducted in a strain-controlled mode, with a 12% strain applied, and the test was in the linear viscoelastic range. The PAV aged PG 58-28 asphalt binder and PAV aged asphalt binder with TCE samples were set between two parallel plates with a gap of 2 mm and a diameter of 8 mm. Five different temperature-sweeps (increasing from 13°C to 25°C with an interval of 3°C) were conducted for PAV aged samples. Six different frequencies (0.1Hz, 1Hz, 1.59Hz, 3Hz, 5Hz, 10 Hz) were used under each temperature. The test was conducted in a strain-controlled mode, with a 1% strain applied, and the test was in the linear viscoelastic range. The detailed test parameters are displayed in Figure 5-1. In order to guarantee the repeatability of the test results, at least three replicate test samples were conducted for each asphalt binder type.

5.4 Results and Discussion

5.4.1 FTIR analysis of asphalt binder with different TCE contents

5.4.1.1 FTIR analysis of TCE

Figure 5-2 displays the FTIR spectrum of the TCE solvent. The peaks at 3010-3100 cm^{-1} identify the stretch =C-H alkene bonds. The peaks at 1620-1680 cm^{-1} represent the stretch C=C alkene bonds. The peaks at 675-1000 cm^{-1} reflect the bending =C-H alkene bonds. The peaks at 600-800 cm^{-1} demonstrate stretch C-Cl alkyl halide bonds [186]. The detailed functional groups of TCE are displayed in Table 5-2.

Figure 5-2. FTIR result of TCE

Table 5-2. The functional group of TCE

Functional group	Wave number /cm ⁻¹	Intensity
Alkene =C-H (stretch)	3010-3100	medium
Alkene =C-H (bending)	675-1000	strong
Alkene C=C (stretch)	1620-1680	variable
Alkyl Halide C-Cl (stretch)	600-800	strong

Figure 5-3 displays the FTIR spectrum of asphalt binder PG 58-28, PAV aged PG58-28, and TCE. Peaks between 2950 cm⁻¹ and 2850 cm⁻¹ represent the C–H aliphatic stretching. The peak at 1613.6 cm⁻¹ identifies as the aromatic C=C stretching effect. The C–H aliphatic bending in CH₂, and the C–H aliphatic bending in CH₃ are reflected at 1455.7 cm⁻¹ and 1376.6 cm⁻¹, respectively. The band around 1700 cm⁻¹ proves to be the stretching vibration of C=O, and the band around 1030 cm⁻¹ is an effect of the S=O vibration.

Figure 5-3. FTIR results of TCE and asphalt binder

5.4.1.2 FTIR analysis of unaged and PAV aged PG 58-28

The FTIR of PG 58-28 and PAV aged PG 58-28 with different TCE contents are displayed in Figure 5-4 and Figure 5-5, respectively. The alkane C–H bands at 1376.6 cm^{-1} , 1455.7 cm^{-1} , 2850 cm^{-1} , and 2950 cm^{-1} were not influenced by TCE in the PG 58-28 and PAV aged PG 58-28. In the PAV aged PG 58-28, the C=O band at 1700 cm^{-1} and S=O band at 1030 cm^{-1} were only influenced by the aging process.

Figure 5-4. FTIR of PG 58-28 with different TCE contents

Figure 5-5. FTIR of PAV aged PG 58-28 with different TCE contents

The =C-H in TCE affected the Alkene =C-H bonds in asphalt binder when TCE was added. The FTIR spectrum of PG58-28 with different TCE contents during wave number 820-860 cm^{-1} and 900-960 cm^{-1} are displayed in Figure 5-6 and Figure 5-7, respectively. Based on Figure 5-2, the FTIR of TCE during 675-1000 cm^{-1} has three peaks, which are 778 cm^{-1} , 837 cm^{-1} , and 928 cm^{-1} . By comparing the FTIR results of PG 58-28 with different TCE contents at these three peaks, some peaks were generated, as shown in Figure 5-6 and Figure 5-7. The same condition occurred in the FTIR results of PAV aged PG 58-28 asphalt binder with different TCE contents. Peaks occurred in PAV aged PG 58-28 with TCE, as shown in Figure 5-8 and Figure 5-9.

Figure 5-6. FTIR result of PG58-28 with TCE in the range of 820-860 cm^{-1}

Figure 5-7. FTIR result of PG58-28 with TCE in the range of 900-960 cm^{-1}

Figure 5-8. FTIR result of PAV aged PG58-28 with TCE in the range of 820-860 cm^{-1}

Figure 5-9. FTIR result of PAV aged PG58-28 with TCE in the range of 900-960 cm^{-1}

Figure 5-10 displays the infrared spectrum band area (green) for asphalt binder with two different TCE contents. The baseline (red), the upper limit, and the lower limit (blue) are presented in Figure 5-10. The upper limit and the lower limit were defined as the same wave numbers for different TCE contents. The baseline connected the intersection of the curve with the upper limit and the lower limit. The area below the baseline was not considered during the area calculation procedure if the baseline crossed the curve.

Figure 5-10. Infrared spectrum band area for wave number 834-848 cm⁻¹

The infrared area changed after PG 58-28 mixed with TCE is shown in Table 5-3. The band area was the average of three replicate samples, and the number in the bracket was the standard deviation. When the TCE content was higher than 3%, the linear relation between the TCE content and the band area 834-848 cm⁻¹ is displayed in equation (5.1).

$$\text{Band area} = 0.0119 \text{ TCE content} - 0.0336 \quad (R^2=0.9839) \quad (5.1)$$

For band area 834-848 cm⁻¹, the band area changing was minimal when TCE content was lower than 3%. The band area increased linearly with the increase of TCE content when the TCE content was higher than 3%.

For band area 918-938 cm⁻¹, the area was linearly proportional to the TCE percentage. The linear relation between the TCE content and band area 918-938 cm⁻¹ is displayed in equation (5.2).

$$\text{Band area} = 0.0093 \text{ TCE content} + 0.0099 \quad (R^2=0.9809) \quad (5.2)$$

When compared with the band area between 834-848 cm^{-1} and 918-938 cm^{-1} , the function group during 834-848 cm^{-1} maybe not as sensitive as the functional group during 918-938 cm^{-1} to the TCE content. Band area 918-938 cm^{-1} was better than band area 834-848 cm^{-1} in reflecting the influence of TCE in unaged asphalt binder.

Table 5-3. Area change after TCE was mixed with PG 58-28

Percentage of TCE / %	Wave number / cm^{-1}	
	834-848	918-938
1	0	0.022664 (0.002037)
2	0.000356 (0.000025)	0.026542 (0.002029)
3	0.001167 (0.000087)	0.036173 (0.002503)
4	0.016679 (0.001021)	0.044657 (0.03416)
5	0.024100 (0.001918)	0.056500 (0.003922)
6	0.038438 (0.002608)	0.067917 (0.004913)

The changes in the area after PAV aged PG 58-28 mixed with TCE are displayed in Table 4. The band area was the average of three replicate samples, and the number in the bracket was the standard deviation. For band area 834-848 cm^{-1} , the area was linearly proportional to the TCE content. The linear relation between the TCE content and band area 834-848 cm^{-1} is displayed in equation (5.3).

$$\text{Band area} = 0.0024 \text{ TCE content} - 0.0024 \quad (R^2=0.9257) \quad (5.3)$$

For band area 918-938 cm^{-1} , the area was linearly proportional to the TCE percentage. The linear relation between the TCE content and band area 918-938 cm^{-1} is displayed in equation (5.4).

$$\text{Band area} = 0.0076 \text{ TCE content} + 0.063 \quad (R^2=0.9537) \quad (5.4)$$

By comparing Equation 5.3 and 5.4 with Equation 5.1 and 5.2, the slope of the relation between band area and TCE content of PAV aged PG 58-28 was smaller than the slope of

unaged PG 58-28. The FTIR test method was harder to characterize the existence of TCE in PAV aged PG 58-28 than in the unaged PG 58-28.

Table 5-4. Area change after TCE mixed with PAV aged PG 58-28

Percentage of TCE / %	Wave number / cm^{-1}	
	834-848	918-938
1	0	0.0164 (0.0010)
2	0.0045 (0.0005)	0.0219 (0.0012)
3	0.0048 (0.0004)	0.0278 (0.0013)
4	0.0059 (0.0004)	0.0336 (0.0016)
5	0.0094 (0.0008)	0.0415 (0.0021)
6	0.0137 (0.0009)	0.0568 (0.0028)

Both band areas $834\text{-}848\text{ cm}^{-1}$ and $918\text{-}938\text{ cm}^{-1}$ changed for PG 58-28 and PAV aged PG 58-28 after the addition of TCE. By comparing equations 1-4, band area $918\text{-}938\text{ cm}^{-1}$ can better reflect the influence of TCE on asphalt binder. Band area $834\text{-}848\text{ cm}^{-1}$ didn't change when TCE content was low.

By testing wave number $918\text{-}938\text{ cm}^{-1}$ of asphalt binder, the existence of TCE in asphalt binder can be determined. The TCE amount can be estimated through the calculation of area in the infrared spectrum, which provided a method to qualitatively evaluate the residue content of TCE in recovered asphalt binder.

5.4.2 DSR analysis

The DSR test was used to characterize the viscoelastic behavior of the asphalt binder. The complex shear modulus ($|G^*|$) and phase angle (δ) are two parameters to quantify the DSR test. $|G^*|$ reflects the shear deformation resistance of under repeated shearing force, while δ is the lag between the applied shear stress and the resulting shear strain. There

were nine different temperature-sweeps (increasing from 34°C to 82°C with an interval of 6°C) with six different frequencies (0.1Hz, 1Hz, 1.59Hz, 3Hz, 5Hz, 10 Hz) for each PG 58-28 with different TCE contents. The master curve can effectively predict $|G^*|$ and δ beyond the laboratory test conditions. A nonlinear least-squares regression model was used to regress the master curves of $|G^*|$ and δ under two different aging conditions [187].

5.4.2.1 DSR analysis of PG 58-28

The $|G^*|$ of PG 58-28 with different TCE contents is displayed in Figure 5-11. As shown in Figure 5-11, $|G^*|$ increased gradually with the increase of reduced frequency. The asphalt binder became softer when TCE was added, and $|G^*|$ decreased with the increase of TCE content. The $\log |G^*|$ was proportionally related to the logarithmic value of reduced frequency. The regression relation is displayed in equation (5.5). The regression coefficient in equation (5) for PG 58-28 asphalt binder with different TCE contents is displayed in Table 5-5. The coefficient of the determination is expressed as R^2 in Table 5-5. The coefficient A reflected the influence of frequency on the shear modulus of asphalt binder. The coefficient A increased slightly with the increase of TCE content. The influence of testing frequency on the test results increased with the increase of TCE content in asphalt binders. The coefficient B in equation (5) reflected the value change of $|G^*|$ with the addition of TCE. The coefficient B linearly decreased with the increase of TCE content. The coefficient B decreased around 1 after 6% TCE was added in asphalt binder. The maximum $|G^*|$ decreased about ten times when 6% TCE was added into PG 58-28 asphalt binder, which proved the significant influence of TCE on extracted asphalt binder.

$$\text{Log } |G^*| = A^* \log \text{ reduced frequency} + B \quad (5.5)$$

Figure 5-11. $|G^*|$ of PG 58-28 with different TCE contents

Table 5-5. The regression coefficient of PG 58-28 with different TCE contents

TCE content	A	B	R^2
0% TCE	0.8986	3.5	0.9965
1% TCE	0.9273	3.308	0.9985
2% TCE	0.9416	3.102	0.999
3% TCE	0.9521	2.942	0.9994
4% TCE	0.9542	2.808	0.9993
5% TCE	0.9585	2.679	0.9995
6% TCE	0.9626	2.581	0.9995

5.4.2.2 DSR analysis of PAV aged PG 58-28

The complex shear modulus of PAV aged PG 58-28 with different TCE contents is displayed in Figure 5.12. As shown in Figure 5.12, $|G^*|$ increased gradually with the increase of reduced frequency. The value of $|G^*|$ decreased dramatically when 1% TCE was added and then decreased proportionally with the increase of TCE content. The asphalt binder became softer when TCE was added, and $|G^*|$ decreased with the increase

of TCE content. The $\log |G^*|$ was proportionally related to the logarithmic value of reduced frequency. The equation (5) was also applicable to the regression relation between $\log |G^*|$ and the logarithmic value of reduced frequency. The regression coefficient in equation (5.5) for PAV aged PG 58-28 asphalt binder with different TCE contents are displayed in Table 5-6. The coefficient of the determination is expressed as R^2 in Table 5-6. The coefficient A increased slightly with the increase of TCE content. The coefficient B decreased dramatically after 1% TCE was added and decreased linearly with the continuous addition of TCE. The maximum $|G^*|$ decreased about 3.5 times after 6% TCE was added in the PAV aged PG 58-28 asphalt binder, which proved the significant influence of TCE on extracted asphalt binder.

Figure 5-12. $|G^*|$ of PAV aged PG 58-28 with different TCE contents

Table 5-6. The regression coefficient of PAV aged PG 58-28 with different TCE contents

TCE content	A	B	R ²
0% TCE	0.5471	6.553	0.9945
1% TCE	0.5882	6.259	0.9967
2% TCE	0.5923	6.201	0.9961
3% TCE	0.6069	6.123	0.9965
4% TCE	0.6094	6.082	0.9968
5% TCE	0.6162	6.054	0.9966
6% TCE	0.6167	6.013	0.9966

The influence of TCE on asphalt binder was obvious, the $|G^*|$ of the PG 58-28 asphalt binder decreased steadily with the increase of TCE content. The maximum $|G^*|$ decreased about 10 times when 6% TCE was added into the PG 58-28 asphalt binder. The $|G^*|$ of PAV aged PG 58-28 decreased dramatically after 1% TCE was added but decreased slightly with the increase of TCE content. The maximum $|G^*|$ decreased about 3.5 times after 6% TCE was added in the PAV aged PG 58-28 asphalt binder. Both the existence of TCE and the content of TCE had a significant influence on the $|G^*|$ of PG 58-28 asphalt binder. The existence of TCE had a more significant influence on the $|G^*|$ of PAV aged PG 58-28 asphalt binder.

A regression function was built to establish the relation between TCE content and $|G^*|$. The TCE amount can be estimated by calculating the FTIR spectrum area, and the actual complex shear modulus of asphalt binder without TCE can be back calculated, which provided guidance to determine the asphalt binder performance extracted from RAP.

5.4.2.3 Rutting parameter and fatigue parameter

The rutting parameter ($G^*/\sin \delta$) of asphalt binder with different TCE contents is displayed in Table 5-7. The $G^*/\sin \delta$ decreased with the increase of test temperature and TCE content. Based on the performance graded asphalt binder specification. The $G^*/\sin \delta$ should be smaller than 1 kPa. The base asphalt binder met the requirement at 58°C, the

asphalt binder with 2% TCE failed to meet the requirement at 58°C, and the asphalt binder with 4% TCE failed to meet the requirement at 52 °C. The rutting parameter of asphalt binder decreased with the increase of TCE content.

Table 5-7. $G^*/\sin\delta$ of PG 58-28 with different TCE contents

TCE content	$G^*/\sin\delta$ (Pa)		
	52 °C	58 °C	64 °C
0%	4252.3	1898.1	891.8
1%	2591.0	<u>1200.8</u>	591.7
2%	1638.0	790.5	405.1
3%	<u>1113.5</u>	554.5	292.1
4%	852.6	436.2	235.7
5%	622.3	323.4	177.9
6%	477.7	255.1	143.5

The fatigue parameter ($G^*\times\sin\delta$) of asphalt binder with different TCE contents is displayed in Table 5-8. The $G^*\times\sin\delta$ decreased with the increase of test temperature and TCE content. Based on the performance graded asphalt binder specification, the $G^*/\sin\delta$ should be smaller than 5000 kPa. The PAV aged asphalt binder without TCE failed to meet the requirement at 16°C. The PAV aged asphalt binder with TCE met the requirement at 16°C. The asphalt binder became softer after the addition of TCE, which is the reason why the fatigue performance of asphalt binder increased with the increase of TCE content.

Table 5-8. $G^*\times\sin\delta$ of PAV aged PG 58-28 with different TCE contents

TCE content	16 °C	19 °C	22 °C
0%	<u>5049.2</u>	3694.6	2636.9
1%	2714.3	1967.9	1413.4
2%	2431.9	1742.7	1217.0
3%	2056.5	1467.1	1020.0
4%	1896.8	1362.5	962.9
5%	1815.2	1287.0	884.7
6%	1664.9	1179.8	816.1

The existence of TCE in asphalt binders softened the asphalt binder, thus influenced the rutting potential and the fatigue performance of asphalt binder. The rutting parameter and fatigue parameter analysis further proved that the TCE significantly influenced the performance of extracted asphalt binder. The actual performance of the asphalt binder could be calculated based on the DSR test results. Even though such an observation was based upon limited samples and tests, it will be necessary to further investigate with different types of asphalt binders in order to verify such a relationship.

5.5 Chapter summary

Different TCE contents were chosen to mix with unaged and PAV aged PG 58-28 asphalt binder to evaluate the influence of TCE content on the performance of asphalt binder. Based on the FTIR test results, the content of TCE in asphalt binder can be quantitatively determined. The influence of different TCE contents on the complex shear modulus was analyzed. The actual performance of the asphalt binder can be determined based on FTIR and DSR tests. The main findings are displayed as follows, according to the DSR and FTIR test results.

- (1) Functional groups =C-H of TCE can be used as indicators to check for the existence of TCE in extracted asphalt binder.
- (2) Both band area $834\text{-}848\text{ cm}^{-1}$ and $918\text{-}938\text{ cm}^{-1}$ changed for PG 58-28 and PAV aged PG 58-28 after the addition of TCE.
- (3) Band area $918\text{-}938\text{ cm}^{-1}$ better reflected the influence of TCE in asphalt binder. Band area $834\text{-}848\text{ cm}^{-1}$ didn't change when the TCE content was low.
- (4) Both the existence of TCE and the content of TCE had a significant influence on the $|G^*|$ of unaged and PAV aged PG 58-28 asphalt binder. The maximum $|G^*|$ decreased about ten times when 6% TCE was added into the PG 58-28 asphalt binder. The maximum $|G^*|$ decreased about 3.5 times after 6% TCE was added in the PAV aged PG 58-28 asphalt binder.
- (5) A regression function was built to establish the relation between TCE content and $|G^*|$. The TCE amount could be estimated by calculating the FTIR spectrum area,

and the actual complex shear modulus of asphalt binder without TCE can be back calculated, which provided guidance to determine the asphalt binder performance extracted from RAP.

- (6) The existence of TCE in asphalt binder softened the asphalt binder, thus influenced the rutting potential and the fatigue performance of asphalt binders. The rutting parameter and fatigue parameter analysis further proved that the TCE significantly influenced the performance of extracted asphalt binder.

6 The performance evaluation of extracted asphalt binder

Extractive Summary

The interaction between the rubber and asphalt binder in the rubber modified asphalt mixture using a dry process lacks related research. By using the extraction procedure, the asphalt binder interacted with rubber could be obtained and assessed. The goal of this chapter is to promote a dry process technology with tire rubber from scrap tires by investigating the laboratory performance of asphalt binder extracted from rubber modified asphalt mixture. Four types of asphalt mixtures, which included two mixture designs with and without rubber, were produced in a plant. The property of extracted asphalt binder from the mixtures was compared with the base asphalt and the rubber modified asphalt. The dynamic shear rheological property, multiple stress creep and recovery property, and low temperature cracking property of different types of binders were evaluated. Based on the test results, the rubber addition significantly increased the high temperature performance and slightly improved the low temperature characteristics of the base asphalt. The aged asphalt binder in the reclaimed asphalt pavement (RAP) increased the high temperature property of the asphalt binder but decreased the low temperature characteristics. The contribution of rubber in extracted asphalt binder to the low temperature property was limited. However, the aged asphalt binder and rubber in extracted asphalt binder guaranteed the asphalt binder to sustain heavy traffic load at high temperatures. This rubber modified mixture with RAP may provide an appropriate and environmental-friendly pavement designed for heavy traffic conditions.

6.1 Introduction

The condition of the pavement was decreased with the increase of service life of the pavement. The pavement with different types of distress need maintenance or conservation. The old pavement with distress could be milled into smaller particles and used in the construction of the new pavement [168, 188]. The cost of the new pavement could be reduced if RAP was used, since the cost of the RAP materials is lower than new

materials. And the adoption of RAP in the new pavement construction also had environmental benefits [189, 190]. Using RAP to partially replace the new aggregate and asphalt binder could reduce the consumption of natural materials [191]. The energy waste and pollution to the environment would be decreased with the increase of RAP content in the new pavement construction [192]. The landfill needed for the disposal of RAP materials would be decreased with the increase of RAP in the pavement construction [193]. The performance of the pavement with RAP was sensitive to the success mixture design. The performance of the pavement with high RAP could be comparable as the pavement without RAP, if the mixture design was controlled, and the service life of the pavement would be extended [194-196].

The successful design of the asphalt mixture with RAP is highly related to the performance of asphalt binder extracted from RAP [197]. Centrifuge extraction and rotary evaporate recovery are the most common used test to obtain asphalt binder from RAP [198]. 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 an extraction equipment. The solvent mix of asphalt binder and the solvent was heated in a rotary evaporator to remove 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 original asphalt mixture and extracted aggregate. The property of the asphalt binder in the HMA or RAP could be evaluated using laboratory test procedure [199]. 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 extracted base binder. Based on the conclusions from the Chapter 5, the existence of TCE in the extracted asphalt binder had significant influence on the rheological property of the asphalt binder. The complex shear modulus will be used as 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.

6.2 Test Materials and Test Method

6.2.1 Test Materials

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 performance 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 °C for 30 mins.

6.2.2 Asphalt Binder Extraction

The automatic asphalt analyzer from the Controls group was used to was adopted to separate asphalt and aggregate from the asphalt mixture, as shown in Figure 6-1. 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 mixture, and 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 [200]. The rotary evaporation process followed the ASTM standard, and the test apparatus is shown in Figure 6-2. The TCE was evaporated, and the asphalt binder was left in the evaporating flask. The asphalt binder was received for a later test.

Figure 6-1. The automatic asphalt analyzer used to separate asphalt from the asphalt mixture

Figure 6-2. The rotary evaporator used to obtain asphalt binder from the asphalt-TCE solution

6.2.3 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 [201]. The Rolling Thin-Film Oven (RTFO) test in the laboratory could simulate the short-term aging of asphalt during this process, and the test apparatus is shown in Figure 6-3. The short-term aging test in this study was carried out in accordance with 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, 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 6-3. 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 [202]. The components in the asphalt undergone oxidation, evaporation, and dehydrogenation process. The Pressure Aging Vessel (PAV) in the laboratory could simulate the long-term aging of asphalt during the process, and the PAV test apparatus is shown in Figure 6-4. The long-term aging test in this study was carried out in accordance with 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 of the asphalt binder, 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 6-4. The pressure aging vessel (PAV) test apparatus

6.2.4 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 6-5. The complex shear modulus ($|G^*|$) and phase angle (δ) of the asphalt binder are measured from the DSR; $|G^*|$ reflected the shearing deformation resistance of asphalt binder under repeated shearing force, whereas δ reflected the delay between the applied shear stress and shear strain response [203]. The unaged and RTFO aged samples were set between two parallel plates with a diameter of 25 mm and a gap of 1 mm. The DSR temperature sweep was conducted at various temperatures ranging from 34 °C to 82 °C with an increment of 6 °C. Six frequencies (0.1, 1, 1.59, 3, 5, and 10 Hz) were adopted at each test temperature. The PAV aged samples were set between two parallel plates with a diameter of 8 mm and a gap of 2 mm. The DSR temperature sweep was conducted at various temperatures ranging from 13 °C to 25 °C with an increment of 3 °C. Six frequencies (0.1, 1, 1.59, 3, 5, and 10 Hz) were adopted at each test temperature. The tests were conducted in strain-controlled mode with a 12% strain rate for unaged asphalt binder, 10% strain rate for

RTFO aged asphalt binder, and 1.0% strain rate for PAV aged asphalt binder and the tests were in the linear viscoelastic range. In order to guarantee the repeatability of the test results, at least three replicate DSR test samples were conducted for each asphalt binder type.

Figure 6-5. The DSR test apparatus from Anton Paar

6.2.5 Multiple Stress Creep and Recovery (MSCR)

The Multiple Stress Creep and Recovery (MSCR) test was applied to determine the percent recovery and non-recoverable creep compliance of different asphalt binders. The MSCR test was conducted using the DSR at specific temperatures, and the test samples were RTFO aged with a diameter of 25 mm and a thickness of 1 mm. 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 [204]. In order to guarantee the repeatability of the test results, at least three replicate MSCR test samples were conducted for each asphalt binder type.

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 52°C, 58°C, 64°C, 70°C, and 76°C. 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 6-1.

Table 6-1. 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 less than the standard traffic load rate (>70 km/h)	$J_{nr_{3.2}} \leq 4.5 \text{ kPa}^{-1}$ $J_{nr_{diff}} \leq 75\%$
High (H)	Traffic levels of 10 to 30 million ESALs or slow-moving traffic (20 to 70 km/h)	$J_{nr_{3.2}} \leq 2.0 \text{ kPa}^{-1}$ $J_{nr_{diff}} \leq 75\%$
Very High (V)	Traffic levels of greater than 30 million ESALs or standing traffic (<20 km/h)	$J_{nr_{3.2}} \leq 1.0 \text{ kPa}^{-1}$ $J_{nr_{diff}} \leq 75\%$
Extreme high (E)	Traffic levels of greater than 30 million ESALs and standing traffic (<20 km/h)	$J_{nr_{3.2}} \leq 0.5 \text{ kPa}^{-1}$

The asphalt binder sample was loaded at constant stress (0.1 kPa or 3.2 kPa) for 1 s and then recover for 9 s. Twenty creep and recovery cycles were run at 0.1 kPa creep stress followed by ten creep and recovery cycles at 3.2 kPa creep stress. The first ten creep and recovery cycles at 0.1 kPa creep stress was used for specimen conditioning. The data from the second ten creep and recovery cycles at 0.1 kPa creep stress were collected and analyzed. The stress and strain data were recorded every 0.1 s during the creep cycle and recorded every 4.5 s during the recovery cycle. 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 6-6. The creep recovery test results of ten cycles under different creep stresses is shown in Figure 6-7.

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

Figure 6-7. 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.

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

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

$$R_{0.1}(\text{or } R_{3.2}) = \frac{\sum_{i=1}^{10} \varepsilon_r(0.1 \text{ or } 3.2, N)}{10} \quad (6.2)$$

The percent difference (R_{diff}) in recovery between 0.1 kPa and 3.2 kPa is calculated according to Equation 6.3

$$R_{diff} = \frac{R_{0.1} - R_{3.2}}{R_{0.1}} \cdot 100 \quad (6.3)$$

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

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

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

$$J_{nr,0.1kPa(\text{or } 3.2kPa)} = \frac{\sum_{i=1}^{10} J_{nr}(0.1 \text{ or } 3.2, N)}{10} \quad (6.5)$$

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

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

6.2.6 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 6-8. The operation of ABCD followed the AASHTO TP 92 test protocol. The ABCD test procedure is shown in Figure 6-9. The relation between strain and temperature of asphalt binder ring under certain

temperature decreasing rate was recorded [205]. The test temperature decreased from -20 °C to -60 °C with a decreasing rate of 10 °C/hour. The relation between the strain and temperature during the ABCD test is shown in Figure 6-10. The asphalt binder cracked when the temperature decreased to the crack temperature, as shown in Figure 6-9 (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 6-10. The PG grade by ABCD can be calculated according to Equation 6-8. In order to guarantee the repeatability of the test results, at least four replicate ABCD test samples were conducted for each asphalt binder type.

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

$$\text{PG grade by ABCD (}^{\circ}\text{C)} = 0.78 * (\text{ABCD cracking temperature, }^{\circ}\text{C)} - 0.9 \quad (6.8)$$

Figure 6-8. 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 6-9. The ABCD test procedure

Figure 6-10. The relation between the microstrain and temperature during the ABCD test

6.3 Test results and analysis

6.3.1 The rotary evaporation procedure validation

In the previous chapter, the results mentioned that the residue of TCE in asphalt binder had a significant influence on the $|G^*|$ of the asphalt binder. The change of $|G^*|$ could be used as an indicator to validate the existence of TCE in the extracted asphalt binder during the rotary evaporation procedure.

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 6-11. 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. In order to guarantee the repeatability of the test results, at least three replicate extraction tests were used for the extraction test. The average ratio of the $|G^*|$ of extracted asphalt binder to the $|G^*|$ of original asphalt binder was slightly lower than 60%.

Figure 6-11. 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 residues 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 set the air rate at 4L/min and rotate the glass container at the same temperature with the same rotate rate for 10 minutes. Finally, remove the asphalt binder in the glass container and stir the collected asphalt binder 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 6-12. 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, 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 6-12. The comparison between the original binder and extracted binder

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

6.3.2.1 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 were assessed. 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 6-13 and Figure 6-14. 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 6-13. The gradation of the surface layer asphalt mixture after the extraction procedure

Figure 6-14. 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 6-15. The color of the surface layer aggregate that from the mixture with rubber was darker than the aggregate from the mixture without rubber (Figure 6-15 (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 6-15 (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 6-15 (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 6-15. 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 6-16. The color of the leveling layer aggregate that from the mixture with rubber was darker than the aggregate from the mixture without rubber (Figure 6-16 (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 6-16 (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 6-16. The comparison between the leveling layer aggregate without and with rubber

6.3.2.2 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 6-17. 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 will 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% ($6.6\% = 5.9\% + 4.15\% \times 17\%$). The total asphalt binder content of 5E1 mixture with rubber was 6.08% if the rubber was not considered as part of asphalt binder ($5.9\% \times 0.91 + 4.15\% \times 17\% = 5.37 + 0.71 = 6.08\%$). 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.

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% ($5.24\% = 4.2\% + 4.15\% \times 25\%$). 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 ($4.2\% \times 0.91 + 4.15\% \times 25\% = 3.82 + 1.04 = 4.86\%$). 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.

Figure 6-17. The asphalt binder content determination based on the extraction procedure

6.3.3 The DSR analysis

6.3.3.1 The unaged asphalt binder

The master curve of different types of unaged asphalt binders is shown in Figure 6-18. A nonlinear least-squares regression model was used to plot the master curves of $|G^*|$ and δ [106]. The complex shear modulus master curve of different types of unaged asphalt binders is shown in Figure 6-18 (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 asphalt binder increased with the increase of the reduced frequency. Based on the time-temperature superposition principle, the property of asphalt binder at high reduced frequency could reflect the performance of asphalt binder at low temperatures. 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 of the reduced frequency. At the high frequency conditions (low temperature), the stiffness of extracted asphalt binder was higher due to the existence of an aged asphalt binder from RAP.

The phase angle master curve of different types of unaged asphalt binders is shown in Figure 6-18 (b). The phase angle of the asphalt binder decreased with the increase of the reduced frequency. The phase angle ranged between 73.2° and 81.8° 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 property of the asphalt binder was improved. The minimum phase angle reached to 53.2° for PG 58-34 asphalt binder with rubber. The phase angle of the extracted asphalt binders decreased dramatically with the increase of 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.

(a) The complex shear modulus master curve

(b) The phase angle master curve

Figure 6-18. The master curve of different types of unaged asphalt binder

The rutting parameter of different types of unaged asphalt binders is shown in Figure 6-19. 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 at the loading frequency of 10 rad/s. For unaged asphalt binder, the high temperature grade of base asphalt was 58 °C; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer without rubber was 64 °C, 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 °C. 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 high temperature grade of base asphalt increased 12 °C. 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 °C for extracted asphalt binder from mixtures with rubber.

Figure 6-19. The rutting parameter of different types of unaged asphalt binders

6.3.3.2 The RTFO aged asphalt binder

The mass loss of the different extracted binder types after RTFO aging procedure is shown in Figure 6-20. 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 6-20. 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 6-21 (a). The $|G^*|$ of asphalt binder increased with the increase of the reduced frequency. 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 6-21 (b). The phase angle of the asphalt binder decreased with the increase of the reduced frequency. 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 6-21. The master curve of different types of RTFO aged asphalt binder

The rutting parameter of different types of RTFO aged asphalt binders is shown in Figure 6-22. 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 at the loading frequency of 10 rad/s. For RTFO aged asphalt binder, the high temperature grade of base asphalt was 58 °C; the high temperature grade of extracted asphalt binder from the leveling layer and surface layer without rubber was 64 °C; the high temperature grade of extracted asphalt binder from the leveling layer and

surface layer with rubber was 70 °C, and the high temperature grade of base asphalt binder with rubber was 76 °C.

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

Figure 6-22. The rutting parameter of different types of RTFO aged asphalt binders

6.3.3.3 The PAV aged asphalt binder

The complex shear modulus master curve of different types of PAV aged asphalt binders is shown in Figure 6-23 (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 the stiffness of asphalt binder was increased, the influence 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 6-23 (b). The phase angle of the asphalt binder decreased with the increase of the reduced frequency. 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 6-23. The master curve of different types of PAV aged asphalt binder

6.3.4 MSCR

The average percent recovery and percent difference in the recovery of base asphalt binder after RTFO aging is shown in Figure 6-24 (a). The base asphalt binder 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. 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 °C to 70 °C. 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 °C under the stress level of 3.2 kPa. The percent difference in recovery under different stress levels increased with the increase of test temperature. The difference reached 100% at the test temperature of 70 °C.

The non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr-diff}$) of base asphalt binder after RTFO aging are shown in Figure 6-24 (b). The J_{nr} of asphalt binder was increased significantly with the increase of test temperature. The J_{nr} increased 12 times when the test temperature increased from 52 °C to 58 °C. The viscous percentage of asphalt binder increased with the increase of test temperature, the elastic recovery property was decreased, and thus the J_{nr} increased. The $J_{nr-diff}$ under different stress levels increased with the increase of test temperature.

Figure 6-24. 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 6-25 (a). The base asphalt binder with rubber 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. 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 °C to 70 °C. The average percent recovery of asphalt binder increased significantly after mixing with rubber. The asphalt binder with 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 °C and 58°C, 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 °C.

The non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance (J_{nr_diff}) of base asphalt binder with rubber after RTFO aging are shown in Figure 6-25 (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_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 6-25. 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 6-26 (a). The asphalt binder extracted from 5E1 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. By comparing Figure 6-24(a) and Figure 6-26(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 non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr-diff}$) of the asphalt binder extracted from 5E1 CTRL asphalt mixture after RTFO aging is shown in Figure 6-26 (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 $J_{nr-diff}$ 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 6-26. 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 6-27 (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.

The non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr-diff}$) of the asphalt binder extracted from 5E1 Rubber asphalt mixture after RTFO aging is shown in Figure 6-27 (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 $J_{nr-diff}$ of asphalt binder extracted from 5E1 Rubber asphalt mixture was similar to that from 5E1 CTRL asphalt mixture at low temperatures.

(a) Average percent recovery

(b) Creep compliance

Figure 6-27. The average percent recovery and creep compliance of 5E1 Rubber mixture extracted binder after RTFO aging

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 6-28 (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.

The non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr-diff}$) of the asphalt binder extracted from LVSP CTRL asphalt mixture after RTFO aging is shown in Figure 6-28 (b). The J_{nr} of asphalt binder extracted from the LVSP CTRL asphalt mixture was slightly lower than that from 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 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 $J_{nr-diff}$ of the asphalt binder extracted from the LVSP CTRL asphalt mixture was lower than that from the 5E1 CTRL asphalt mixture.

(a) Average percent recovery

(b) Creep compliance

Figure 6-28. The average percent recovery and creep compliance of LVSP CTRL mixture extracted binder after RTFO aging

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 6-29 (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 non-recoverable creep compliance (J_{nr}) and percent difference in non-recoverable creep compliance ($J_{nr-diff}$) of the asphalt binder extracted from LVSP Rubber asphalt mixture after RTFO aging is shown in Figure 6-28 (b). The J_{nr} 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 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 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 $J_{nr-diff}$ of the asphalt binder extracted from the LVSP Rubber asphalt mixture was lower than that from the 5E1 Rubber asphalt mixture.

Figure 6-29. 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 6-30. 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 6-1. The traffic level of base asphalt binder was very heavy at 52 °C, 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 °C, 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 °C, which was standard traffic condition. The extracted asphalt binder from 5E1 Rubber and

LVSP Rubber asphalt binder reduced to heavy traffic condition at 64 °C. The base asphalt binder could sustain very heavy traffic condition at 64 °C. At 70 °C, 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 6-30. The traffic level of different binders under different temperatures

6.3.5 ABCD

The low temperature performance of different asphalt binders was evaluated with the ABCD test. The crack temperature and the PG temperature based on ABCD is shown in Figure 6-31. The red dash line is the temperature line of -28 °C, and the blue dot line is the temperature line of -34 °C. The crack temperature of base asphalt and base asphalt binder with rubber was lower than -40 °C, and the crack temperature of extracted asphalt binder was higher than -40 °C. The crack temperature of the base asphalt binder with rubber was 3.9 °C lower than that of the base asphalt binder. The PG temperature of base

asphalt and base asphalt with rubber was lower than $-34\text{ }^{\circ}\text{C}$. The PG temperature of extracted asphalt binder was lower than $-28\text{ }^{\circ}\text{C}$ but higher than $-34\text{ }^{\circ}\text{C}$. The PG temperature of the base asphalt binder with rubber was $3\text{ }^{\circ}\text{C}$ 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.

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.

Figure 6-31. The crack temperature and PG temperature of different binders

The strain jump and fracture stress of different asphalt binders are shown in Figure 6-31. 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 6-32. The strain jump and fracture stress of different binders

6.4 Chapter 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.

7 Summary, conclusions, and future research work

The first two sections of this chapter displayed the summary of the tasks and main findings of the dissertation. The recommendation for future research on the aging performance of rubber modified asphalt mixture with the dry process and the extracted asphalt binder was proposed.

7.1 Summary

The aging of the asphalt mixture significantly influenced the performance of the asphalt mixture, thus affected the service life of the pavement. The application of rubber on the pavement reduced the environmental issue related to waste scrap tires as well improved the performance of the pavement. The literature review firstly summarized the modification procedure and modification mechanism of crumb rubber modified asphalt. The performance evaluation of the rubber modified asphalt binder and asphalt mixture was proposed. The performance improvement strategy was indicated in coping with problems that occurred during the application of rubber modified asphalt mixture. The laboratory procedure to simulate the asphalt binder and asphalt mixture aging was summarized, and the application of recycled asphalt pavement was proposed. The volumetric design of rubber modified asphalt mixture using the dry process was proposed. The aggregate gradation was determined based on the traffic volume and the layer of the mixture to be used on the pavement. The rubber modified asphalt mixture preparation procedure in the laboratory was proposed, and the rubber was premixed with asphalt binder due to the limitation of the machine in the laboratory. The asphalt binder content was determined based on the volumetric properties of the trial mixture design. The moisture susceptibility of the trial mixture design was evaluated.

The high temperature rutting performance of plant mixed and laboratory compacted asphalt mixture was evaluated using the Hamburg wheel tracking device (HWD). The correlation between the number of wheel passes and the rut depth was established. The influence of various factors, which included mixture design parameters of the asphalt mixture (nominal maximum aggregate size (NMAS), RAP content, and asphalt content),

rubber modification, mixture air void content, and aging procedure, on the rutting and moisture susceptibility of asphalt mixtures were evaluated. The two-way interactive effect analysis of different parameters was assessed. The low temperature cracking performance of plant mixed and laboratory compacted asphalt mixture was evaluated using the disc shaped compact tension (DCT) test. The influence of various factors, which included mixture design parameters of the asphalt mixture (nominal maximum aggregate size (NMAS), RAP content, and asphalt content), test temperature, rubber modification, and aging procedure, on the low temperature cracking characteristics of asphalt mixtures was evaluated. The two-way interactive effect analysis of different parameters was assessed. The correlation between fracture energy and the maximum CMOD, post-peak fracture energy, flexibility index, and toughness index was established.

Asphalt binder in reclaimed asphalt pavement (RAP) affects the design of asphalt mixture. Normally the extraction procedure was used to evaluate the performance of asphalt binder in the mixture. Based on my result and previous findings, the TCE may not evaporate totally during the distillation procedure. The effect of solvent in asphalt binder was evaluated by adding known dosage of TCE to quantify the influence. A method was proposed to determine the actual performance of asphalt binder extracted from asphalt mixture if no TCE exist in the residue. The Fourier-transform infrared spectroscopy (FTIR) test was used to analyze the functional groups of TCE and asphalt binder with TCE and to qualitatively and quantitatively evaluate the influence of solvent in asphalt binder. Dynamic Shear Rheometer (DSR) test was employed to evaluate the complex shear modulus (G^*) of asphalt binder with different TCE contents and to establish the relationship between the TCE contents and the G^* of asphalt binder. 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 performance of the four extracted asphalt binders and the base asphalt binder and base asphalt binder mixed with rubber were evaluated and compared. 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.

7.2 Conclusions

The rubber modified asphalt mixture using the dry process could bring environmental and economic benefits. The performance of rubber modified asphalt mixture using a dry process and the influence of aging characteristics lacks related research. Based on the study in the dissertation, the rutting, moisture stability, and the low temperature cracking properties of the dense graded rubber modified asphalt mixture with 10% rubber content were evaluated. The 10% rubber addition and the laboratory aging procedure had significant influence on the rutting and low temperature cracking characteristics of the asphalt mixture. The interaction between the asphalt binder and the rubber particle were assessed by evaluating the extracted asphalt binder. The aged asphalt binder from the RAP had significantly influence on the extracted asphalt binder. The main conclusions were summarized as follows:

- (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. 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.
- (2) For the dense graded asphalt mixture with 10% rubber addition, the leveling layer asphalt mixture had bigger NMAAS, higher RAP content, and less asphalt content than the surface layer asphalt mixture. 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. 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

failure were significantly influenced by the rubber modification. 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 failure were significantly influenced by the mixture air void. The rutting resistance of different asphalt mixture was improved after the aging procedure. Based on the two-way ANOVA interactive analysis of different factors, the rubber addition and aging were two more critical factors that influenced the rutting performance of different asphalt mixtures.

- (3) 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. For the dense graded asphalt mixture with 10% rubber addition, the smaller NMAAS, lower RAP content, and higher asphalt content contributed to the better low temperature cracking performance 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 surface of the aggregate, and improved the adhesion between the asphalt and the aggregate, thus enhanced the low temperature cracking resistance of the asphalt mixture. When the test temperature increased from -24 °C to -18 °C, the fracture energy of the asphalt mixture was increased, the average peak load was decreased, and the maximum CMOD of test specimens was increased. The low temperature fracture energy of asphalt mixtures reduced after the aging process. Based on the results, rubber modification and aging procedure 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 and the post-peak fracture energy. The fracture energy had a very significant exponential correlation with the flexibility index and the toughness index. The toughness index had a very significant linear correlation with the flexibility index.

- (4) By comparing the influence of different parameters on the high temperature and low temperature performance of rubber modified asphalt mixture using the dry process, the findings could provide some guidance for the future application of rubber asphalt mixtures. Some findings are in line with previous research results, and some new findings were proposed, especially the influence of rubber and aging. Asphalt mixture with bigger nominal maximum aggregate size and higher RAP content had improved high temperature rutting resistance but weakened low temperature cracking resistance. Decrease of the air void by controlling the compaction of the mixture could improve the high temperature rutting resistance of the mixture. Rubber modification could improve both the high temperature rutting and low temperature cracking performance of the asphalt mixture. The aging procedure could improve the high temperature rutting resistance, but the low temperature cracking resistance was weakened.
- (5) The functional groups $=C-H$ of TCE can be used as indicators to check for the existence of TCE in the extracted asphalt binder. Band area $918-938\text{ cm}^{-1}$ better reflected the influence of TCE in asphalt binder. Band area $834-848\text{ cm}^{-1}$ didn't change when the TCE content was low. Both the existence of TCE and the content of TCE had a significant influence on the $|G^*|$ of unaged and PAV aged PG 58-28 asphalt binder. The maximum $|G^*|$ decreased about ten times when 6% TCE was added into the PG 58-28 asphalt binder. The maximum $|G^*|$ decreased about 3.5 times after 6% TCE was added in the PAV aged PG 58-28 asphalt binder. A regression function was built to establish the relation between TCE content and $|G^*|$. The TCE amount could be estimated by calculating the FTIR spectrum area, and the actual complex shear modulus of asphalt binder without TCE can be back calculated, which provided guidance to determine the asphalt binder performance extracted from RAP. The existence of TCE in asphalt binder softened the asphalt binder, thus influenced the rutting potential and the fatigue performance of asphalt binder. The rutting parameter and fatigue parameter analysis further proved that the TCE significantly influenced the performance of extracted asphalt binder.

- (6) The modified extraction procedure was proposed to improve the accuracy of the extraction procedure. The difference between the asphalt binder and extracted asphalt binder had a good correlation. 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. 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, thus improved the permanent deformation resistance of asphalt binder.

7.3 Recommendation

The following recommendations related to the aging of rubber modified asphalt mixture using the dry process for future studies were proposed:

- (1) This study investigated limited rubber content and asphalt binder type. It is recommended to use different asphalt binders with different PG temperature grades and increase the rubber contents to examine the influence of asphalt binder type and rubber content during the dry process.
- (2) The mixture study adopted the dense graded asphalt mixture, the influence of the asphalt mixture gradation, such as SMA and OGFC, on the interaction between rubber and asphalt binder could be analyzed.
- (3) The contribution of asphalt binder from RAP on the performance of the final asphalt mixture needs to be studied. The interactive mechanism between the aged asphalt binder from the RAP and new asphalt binder need to be clarified

- (4) It is recommended to study the influence of TCE on the component of asphalt binder during the asphalt mixture extraction procedure, Using the SARA to study the components before and after extraction could find the extraction mechanism.
- (5) It is recommended to study the correlation between the performance of extracted asphalt binder after RTFO and PAV aging and the performance of asphalt binder extracted from long term aged asphalt mixture from the field mixed using the dry process.
- (6) The aging mechanism of rubber modified asphalt mixture using the dry process may be different from that using the wet process, using FTIR, X-Ray Diffraction (XRD) to study the mechanism from the micro-scale could help better understand the procedure.

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