DEPARTMENT OF TRANSPORTATION

Cost Estimate of B vs. C Grade Asphalt Binders

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16. Abstract (Limit: 250 words)

Polymer-modified binders (PMB) have been shown over the decades to improve the mechanical properties of asphalt mixtures compared to unmodified binders. Considering the higher initial cost of PMB, selecting the best alternative is very important, especially for local agencies given their limited budgets. A challenge in the materials selection process for low-volume roads is the limited information available, which could allow engineers to determine whether using PMB is cost-effective. In this research, we investigate the use of PG 58H-34 PMB binders (grade C) and PG58S-28 unmodified binders (grade B) for low-volume roads in Minnesota. Historical pavement performance data are analyzed to compare the field performance of modified and unmodified mixtures. Laboratory experiments are performed to compare the low-temperature cracking properties of polymer-modified and unmodified binders and mixtures commonly used in Minnesota. Based on the experimental results, a life-cycle cost analysis (LCCA) is performed comparing the use of polymer-modified and unmodified binders for low-volume roads in Minnesota. The results show that using PMBs for new construction is expected to extend the pavement service life by 6 years, and that using PMB is more cost-effective than using unmodified binders for low-traffic roads.

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COST ESTIMATE OF B VS. C GRADE ASPHALT BINDERS

FINAL REPORT

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EXECUTIVE SUMMARY

Polymer-modified binders (PMB) have been shown over the decades to improve the mechanical properties of asphalt mixtures compared to unmodified binders. There is not enough information on the cost-effectiveness of PMBs for low-volume roads, considering their higher initial cost. This research investigated the use of PMBs for low-volume roads in Minnesota. First, historical pavement performance data, including Ride Quality Index (RQI) and Surface Rating (SR), were analyzed to study the effect of polymer-modification on pavement performance. The result showed that pavements constructed using PMBs performed better than pavements constructed using unmodified binders. Then laboratory experiments were performed to compare the low-temperature cracking properties of polymer-modified PG 58H-34 and unmodified PG 58S-28 binders and mixtures commonly used in Minnesota. The experiments consisted of the Bending Beam Rheometer (BBR) creep and strength tests for binders, and the Disc-shaped Compact Tension (DCT) test for mixtures. The experimental results showed that the polymer-modified binders and mixtures generally had higher low-temperature cracking resistance than the unmodified ones. Based on the experimental results, a life-cycle cost analysis (LCCA) was performed comparing the use of polymer-modified and unmodified binders for a low-volume road in Minnesota. The results showed that using PG 58H-34 PMB for new construction was expected to extend the pavement service life by 6 years, which indicates and using this binder will be more cost-effective than using unmodified PG 58S-28 binders, for low-traffic roads.

CHAPTER 1: INTRODUCTION

1.1 BACKGROUND

Polymer-modified binders (PMB) have been shown over the decades to improve the mechanical properties of asphalt mixtures compared to unmodified binders, including, for example, low-temperature cracking, fatigue cracking, and rutting.

In cold climate states like Minnesota, low-temperature transverse cracking in winter represents the most prevalent distress in asphalt pavements (Marasteanu, et al. 2007). One option to mitigate the adverse effects of transverse cracking is to use PMBs to reduce the occurrence of transverse cracks and, thus, maintenance needs. Another option is to use less expensive unmodified binders and perform more frequent repairs. Selecting the best alternative is very important, given the limited budgets of local agencies. While for high-traffic roads, the choice of superior products is more obvious, for lower-volume roads, there is limited information to allow engineers to determine whether using PMBs is more cost-effective, considering its higher initial cost.

1.2 OBJECTIVE AND ORGANIZATION

This research investigates the benefits of using PMBs for low-volume roads in Minnesota. In Chapter 2, a comprehensive literature review is conducted focusing on the performance and usage of PMBs. In Chapter 3, a data analysis is performed based on historical pavement management data for pavements with modified and unmodified mixtures. In Chapter 4, a laboratory experimental study is performed to compare the low-temperature cracking properties of PMBs and unmodified binders. In Chapter 5, a laboratory experimental study is performed on the low-temperature cracking properties of polymer-modified and unmodified mixtures. In Chapter 6, a life-cycle cost analysis (LCCA) is performed, comparing the use of polymer-modified and unmodified binders for low-volume roads in Minnesota.

CHAPTER 2: LITERATURE REVIEW

In this chapter, a literature review on Polymer Modified Binder (PMB) is performed, including an introduction to polymer modifiers, laboratory and field performance of PMB, cost-benefit of PMB, and the use of PMB for low-volume roads.

2.1 INTRODUCTION TO PMB

Polymer modification of asphalt binder is the incorporation of polymers in virgin asphalt binder by mechanical mixing or chemical reaction. Compared with virgin binder, PMB enhances pavement performance, including rutting, low-temperature cracking, fatigue cracking, stripping, and aging (Yildirim 2005). During the last half century, PMB use has increased all over the world, to address the rapid increase in traffic load and volume (Zhu et al. 2014).

The first pavement test section with PMB was built in France in the 1930s. The use of PMB started in the U.S. in the 1950s (King et al. 1999). Before the 1970s, PMB was less used in the U.S. compared to European countries, where the presence of large contractors, who could take the risk of providing longer warranties, motivated a greater interest in decreased life-cycle cost, even at higher initial costs (Terrel and Walter 1989). Since the 1980s, the use of PMB, by the US asphalt paving industry, has consistently increased (Yildirim 2005).

The polymers used for asphalt modification cover a broad range of materials that can be classified into two general categories: plastomers and elastomers. Typical plastomers for asphalt modifiers include: Polyethylene (PE), Polypropylene (PP), Ethylene-Vinyl Acetate (EVA), and Ethylene-Butyl Acrylate (EBA). The advantages of plastomers include: 1) relatively low cost, 2) high resistance to rutting, and 3) relatively good storage stability, while the disadvantages are: 1) limited improvement in low-temperature properties, 2) limited improvement in elastic recovery (Zhu et al. 2014, Isacsson and Lu 1995, Baldino et al. 2012, Brovelli et al. 2013, Arslan et al. 2014).

Typical elastomers for asphalt modification include natural rubbers and synthetic rubbers, such as Styrene-Butadiene-Styrene [SBS], and styrene-butadiene-rubber [SBR]. Due to the improvement in elastic recovery, elastomers can increase the resistance to permanent deformation (rutting) (Airey 2003, Airey 2004a, Lucena et al. 2004). However, compared with plastomers, elastomers are more prone to decomposition and thus reduce the resistance to aging (Lu et al. 1999, Zhang et al. 2010, Cortizo et al. 2004, Mouillet et al. 2008, Naskar et al. 2013, Behera et al. 2013).

Among all polymer modifiers, SBS has attracted the most attention, and represents the most used modifier. It is known for improving both high temperature and low temperature properties of binders (Zhu et al. 2014). The crosslink structure of SBS contributes to the improvements in the elasticity and strength of SBS modified binders (Lucena et al. 2004). However, SBS has low compatibility with some virgin binders (Wang et al. 2010, Wen et al. 2002, Galooyak et al. 2010) and low resistance to heat, oxidation and ultraviolet radiation (Li et al. 2010, Collins et al. 1992).

2.2 LABORATORY INVESTIGATION OF PMB

2.2.1 Rutting

Airey (2004b) evaluated the rutting and fatigue performance of EVA and SBS polymer modified mixtures by performing laboratory experiments using the Nottingham Asphalt Tester (NAT). The results indicated an improved rutting and fatigue performance for the EVA and SBS modified mixture, compared to the unmodified mixture.

Tayfur et al. (2007) studied the rutting performance of PMB mixtures. An unmodified and five modified mixtures were investigated. Amorphous polyalphaolefin, cellulose fiber, polyolefin, bituminous cellulose fiber and SBS were used as modifiers. The Laboratoire Central des Ponts et Chaussées (LCPC) wheel tracking tests were performed. The results showed that PMB mixtures were more resistant to rutting than the unmodified mixture, with the SBS mixtures having the highest rutting resistance.

Brovelli et al. (2015) studied the rutting resistance of two types of PMB mixtures containing amorphous polyolefin polymer and a particular polymer obtained by combining LDPE (low density polyethylene) and EVA (ethyl-vinyl-acetate). Rutting tests were performed by a wheel tracking device. Stiffness and fatigue tests were carried out to characterize the performance of the asphalt mixtures. The testing showed that polymer modification in this study improved rutting resistance without compromising the stiffness and fatigue behavior.

2.2.2 Fatigue

Mohammad et al. (2001) studied the effect of PMB on the fatigue life of the mixtures by indirect tensile cyclic loading test. Five types of polymers (SBS, SBR, styrene- ethylene-butylene-styrene [SEBS], Elvaloy, and crumb rubber [CRM]) were studied. The results showed that the addition of polymer increases the laboratory fatigue life of asphalt mixtures. The author argued that the improvement in the fatigue lives is mainly due to the improvements in the rheological properties of the binders. A model was developed correlating the laboratory fatigue life with the rheological properties of binders.

Souliman et al. (2016) studied the effects of Asphalt Rubber (AR) and PMB on fatigue behavior of mixtures. Strain controlled fatigue tests were conducted on the beam fatigue tests setup. The results indicated that the AR and PMB mixtures had much longer fatigue lives compared to the reference (unmodified) mixture. A cost-effectiveness analysis was performed and showed that the AR and PMB asphalt mixtures exhibited significantly higher cost-effectiveness compared to unmodified mixtures, although AR and PMB increases the initial cost of the materials.

Vamegh et al. (2019) studied the fatigue resistance of asphalt mixtures modified by SBR/PP polymer blends and SBS. The results of bending beam fatigue test and indirect tensile fatigue test showed that polymer modification can increase fatigue life of asphalt mixture. More specifically, the polymer blends of SBR and PP showed better fatigue life than SBS modified and unmodified samples.

2.2.3 Low Temperature Cracking Resistance

Li et al. (2008) studied the polymer modifcation on the low temperature cracking resistance of asphalt mixtures using two test methods: the Semi Circular Bending (SCB) test and Disc-Shaped Compact Tension (DCT) test. Three modifiers were studied: SBS, Black Max, and Elvaloy. It was found that all PMB mixtures had higher low temperature cracking resistance than unmodified mixtures. Among the three modifiers studied, SBS and Black Max showed better performance than Elvaloy.

Huang et al. (2011) studied the effect of RAP and SBS on mechanical performance of asphalt mixtures using SCB, indirect tensile test (IDT), and beam fatigue test. It was found that mixtures containing PMB (PG 70-22) resulted in higher tensile strengths than unmodified binder mixtures (PG 64-22). Moreover, it was found that the inclusion of RAP generally decreased the crack resistance. The decrease in crack resistance was more significant for the mixtures with unmodified binder than the mixtures with SBS PMB.

Hill and Buttlar (2016) studied the effect of SBS PMB on the fracture process zone (FPZ) size using digital image correlation (DIC) technology. It was found that polymer modification generally increases the size of the FPZ, which demonstrated enhanced low temperature performance at the microscale.

Bonaquist et al. (2016) studied the effect of PMB on durability of asphalt mixtures in Wisconsin. Experimental results of SCB tests showed that mixtures produced with PMB have higher resistance to cracking than mixtures with unmodified binder. This study recommended the use of PMB in all surface course mixtures and in mixtures containing RAP in Wisconsin.

2.3 FIELD PERFORMANCE OF PMB

Many laboratory experimental studies have shown the superior performance of PMB to regular binders. To validate the enhanced performance of PMB in real pavements, a number of field studies have been performed.

In 1990, a survey was conducted to examine the field performance of polymer modified asphalt pavements (Button 1992). 30 pavements from 14 states in the U.S. were investigated. The polymer modifiers studied included SBR, SBS, PE, EVA, and tire rubber. Most pavements investigated were less than 5 years old, and the results indicated no significant differences between the different types of PMB mixtures.

Ponniah and Kennepohl (1996) analyzed the field performance of PMBs in Ontario, Canada, that included Neoprene, scrap tire rubber, Vestoplast-S, Kraton 4460, Styrelf, and polyethylene. Results showed that PMB pavement sections performed better than unmodified asphalt with respect to rutting. With regard to cracking, polymers used with 85 to 100 penetration base asphalt did not perform better than the unmodified control sections. However, PMB with soft grade (150 to 200 pen) base binders improved low-temperature performance, compared to the control section. Life-cycle cost analysis indicated that PMB is cost-effective if the cost of polymer modification did not exceed the cost of conventional asphalt binder by 100 percent.

Mississippi Department of Transportation studied the effect of PMB on rutting performance of asphalt pavements (Albritton et al. 1999). It was observed that modified mixtures required a higher mixing temperature than regular mixtures. Field performance confirmed that the modified mixtures had better rutting resistance than regular mixtures. Laboratory testing results also showed that Asphalt Pavement Analyzer (APA) test results correlated well with field rutting measurements for most of the polymer modifiers considered in this study. Based on that, the authors recommended APA to be used for rutting performance prediction.

In 2003, a joint research effort between Indiana Department of Transportation and Purdue University investigated the effect of PMB on rutting and cracking performance (McDaniel and Shah 2003). The modifiers evaluated included PAC, Novophalt, multigrade asphalt cement (MGAC), polyester fibers, Neoprene, SBR and asphalt rubber (AR). A detailed description is shown in Table 2.1.

Table 2.1 Modifiers Used in the Study (McDaniel and Shah 2003)

Modifier	Supplier	Type of Modifier	Polymer Content (% of binder)
Asphalt Rubber (AR)	Asphalt Rubber Systems	Wet Process Crumb Rubber	20 ~ 3%
Multigrade Asphalt Cement 20-40 (MGAC)	Asphalt Materials	Gelled Asphalt	NA
Neoprene (Neo)	DuPont	Synthetic Rubber	2%
Novophalt (PE)	Novophalt America	Low density polyethylene	5%
Polyester Fibers (Fiber)	BoniFibers	Fiber	*
PAC20 (PAC)	Styrelf (Now Koch Materials)	Prereacted SB block copolymer	NA
Ultrapave (SBR)	Textile Rubber	SBR Latex	3%

*Fibers added at rate of 5 lbs/ton for base and intermediate and 7.5 lbs/ton of surface mixtures.
NA = Not applicable to MGAC, Not available for PAC (proprietary information).

After 11 years of service, the field sections were all performing well in terms of rutting. There were marked differences, however, between the various sections in terms of cracking. Some of the sections cracked extensively within 3-6 years after construction. Other sections were still performing well after 11 years. The best performers included the SBR, PAC and AR. A second tier of performance included the Neoprene, Fibers and MGAC. The worst performers were the unmodified control sections and the Novophalt. Laboratory testing indicated that all of the modifiers stiffened the binder at high temperature, which explains the good rutting resistance observed in the field. It was also observed that the indirect tensile testing of the modified and control mixtures did not correlate with the observed field performance.

Asphalt Institute (2005) performed a study comparing the performance of overlays constructed with PMB to similar overlays constructed with unmodified binder. It was concluded that the use of polymer modified binders reduced all forms of distress, increasing the life of flexible pavements by 2 to 10 years.

Von Quintus et al. (2007) investigated 36 pavement sections, including both roadway and accelerated pavement test sections, to quantify the benefit of using PMB. The results showed that the use of polymer modification reduces the occurrence of distresses like rutting, fatigue cracking, and thermal cracking, which extended the pavement service lives by 5 to 10 years. A clear bias was found between the predicted and measured distress values for the sections with PMB mixtures, when using the mechanistic-empirical distress prediction models, which suggested the need for different calibration factors in PMB mixtures for use in rutting and fatigue cracking prediction equations.

Lu et al. (2014) studied the performance of PMB on a high traffic volume test road. Field cores were taken after about 10-year service. Laboratory testing revealed that the SBS PMB retained better rheological properties in comparison with conventional ones: higher strain recovery and lower non-recoverable compliance at high temperatures, and lower stiffness at low temperatures. The SBS PMB had good aging resistance, shown by both laboratory aging tests and field aging. Moreover, the SBS PMB significantly enhanced fatigue behavior.

Chen et al. (2018) performed a study in Taiwan to investigate the effect of SBS PMB on field performance after 6 years of service. The results of laboratory testing indicated that the morphology of SBS PMB was influenced by storage temperature and polymer content. The formation of an interlocked continuous network was shown to enhance the rheological properties of PMB. Field performance evaluations showed that none of the test sections had obvious rutting. Notable differences were observed in the cracking behavior. The test section with the highly-modified binder had a much better resistance to cracking. The field measurements on cracking corresponded well with the test results of the semi-circular bend test in the laboratory.

Ahmed et al. (2019) studied the field performance of SBS PMB after 9 years of service on pavements in Sweden. It was found that the unmodified mixtures exhibited considerable aging and the SBS-modified mixtures were less affected by aging. Furthermore, the SBS-modified mixture had significantly better fatigue resistance than the conventional mixture.

Virginia Transportation Research Council evaluated the effectiveness of using high polymer-modified (HP) binders in surface asphalt mixtures for mitigating cracking (Habbouche et al. 2021). Distress survey data collected from the Virginia Department of Transportation (VDOT) Pavement Management System compared HP field sections to the control PMB sections. The HP sections showed the most promising performance 5 years after construction (2015-2020). It was noted that, in general, none of the evaluated mixtures (HP or PMb) were able to stop reflective cracking totally. The service lives of HP and PMB overlays were estimated. Overall, PMA and HP overlays had an average predicted service life of 6.2 and 8.3 years, respectively.

2.4 COST-BENEFIT ANALYSIS

As shown in the previous section, numerous studies have concluded that PMB improves pavement performance. Considering the high initial cost of PMB, a number of studies were performed to investigate the cost-effectiveness of PMB using life-cycle analysis.

Hicks and Epps (2000) studied the cost-effectiveness of implementing crumb rubber modified asphalt binder in Arizona and California by the Life-cycle cost analysis (LCCA). Randomness in the input variables was also considered. The results showed that, in most of the application cases investigated in this study, crumb rubber modified binder is more cost-effective than virgin binders, while for some low traffic volume roads it is not. The authors pointed out that the LCCA results were very sensitive to the input values, such as the service life of a mixture. The input values used in this study were determined based on the experience of highway agencies. Thus, the authors suggested that long-term field performance data should be used in the future to calibrate or refine the results of LCCA.

Asphalt Institute (Buncher, 2009) conducted a LCCA to compare polymer-modified mixtures with unmodified HMA mixtures in terms of their cost-effectiveness. The benefit of PMB on pavement performance was estimated through field performance data. Based on the better performance of PMB, compared with virgin binders, fewer maintenance and rehabilitation activities were scheduled for polymer-modified mixtures. The LCCA results showed a potential saving ranging from 4.5% to 14% when polymer-modified mixtures were used compared with unmodified HMA mixtures.

Lee and Kim (2010) investigated the cost-effectiveness of using polymer modification for chip seals applied on high traffic volume roads. Laboratory tests on rutting, bleeding, and aggregate retention were performed. Results indicated that polymer modified chip seals improve all these mechanical properties, which tend to extend the service life of the pavement. LCCA result indicated that polymer-modified chip seals become a cost-effective solution if the polymer modification can extend the service life of the chip seal from 5 years to more than 7 years, which, according to the authors, was highly possible given their laboratory test results.

Souliman et al. (2016) investigated the cost-effectiveness of using PMB for improving fatigue performance. A polymer-modified mixture and an unmodified mixture used on a high traffic volume road in Sweden were compared. Flexural bending beam fatigue tests were performed to evaluate the fatigue behavior of mixtures. Then, based on the beam fatigue test results, the mechanistic-empirical analysis was applied to estimate the fatigue life of pavements. The cost-effectiveness was computed as the ratio between the fatigue life and total cost. The results showed that although the polymer-modification increases the cost of the material, it is more cost-effective in the long-term than the unmodified mixture.

2.5 CASE STUDIES OF USING PMB FOR LOW-VOLUME ROADS

Although PMB has been shown to significantly improve pavement performance, their use for low-volume roads is limited, due to their high initial cost and the reduced number of users who benefit from its use (Leiva-Villacorta et al. 2019). Only four relevant studies devoted to the use of PMB for low-volume roads were identified.

In 2000, Hicks and Epps (2000) studied the cost-effectiveness of crumb rubber modified asphalt binder by performing LCCA for different application situations in Arizona and California, including some low-volume roads cases. The results showed that crumb rubber modified mixtures might be less cost-effective than unmodified mixtures for low traffic volume roads. This conclusion was drawn based on the estimation of the service lives of crumb rubber modified mixtures and unmodified mixtures, which were determined based on the experience of highway agencies, so the authors suggested further studies to be performed on this topic.

A study in Norway (Pay 2017) investigated the possibility of implementing PMB on Norwegian low-volume roads to reduce rutting distresses. Laboratory experiment tests showed the improved resistance of PMB mixtures to rutting. The author therefore inferred that the use of PMB in low-traffic asphalt pavements could increase service lifetime. The cost-benefit of using PMB on low-volume roads was not discussed in this study.

Moreno-Navarro et al. (2017) studied the use of PMB to rehabilitate light and medium traffic volume roads. Laboratory tests were performed to estimate the long-term performance of a PMB mixture and a traditional unmodified mixture. Based on the test results, a structural analysis was performed to calculate the stress and strain distribution in a pavement structure, and to estimate the service lives of the PMB mixture and the unmodified mixture. The results showed that the PMB mixture would have a service life four times longer than that of the unmodified mixture. As a result, PMB mixture would be more cost effective than the unmodified mixture to rehabilitate the low volume road.

2.6 CONCLUSIONS

In this chapter, a literature review on the performance and cost-effectiveness of PMB was performed. The following conclusions were drawn:

- Many laboratory and field studies have shown the improved performance of PMB mixtures, including rutting, cracking, and fatigue resistance.
- Field studies comparing different polymer modifiers showed that rutting is typically not an issue for PMB mixtures. However, the response to other distresses, such as cracking, fatigue and aging, is different for different modifiers. Most studies have shown that SBS modified binder achieved better cracking, fatigue and aging properties than other modifiers.
- Most cost-benefit investigations of PMB used LCCA. The main difficulty for performing LCCA is the estimation of the service lives for different PMBs, which is typically done based on experience. A rational method for that is needed.
- Previous studies showed that PMB is cost-effective for most application scenarios for high traffic volume roads, while for low traffic volume roads, in some instances this is not true.

- Very few research efforts have investigated the use of PMB for low-volume roads. From the limited studies, no consensus was achieved regarding the cost-effectiveness of using PMB for low-volume roads and this topic is still under investigation.

CHAPTER 3: ANALYSIS OF FIELD PERFORMANCE DATA

In this Chapter, the historical pavement performance data were analyzed regarding the use of B and C binders.

3.1 PAVEMENT MANAGEMENT DATA

Field performance data of pavement sections with B and C binders was obtained from the Office of Materials and Road Research at MnDOT. The data consists of 816 records of Ride Quality Index (RQI) and 428 records of Surface Rating (SR). RQI quantifies pavement surface smoothness, while SR quantifies the pavement surface distresses. For each index, a higher value indicates a better pavement condition. The analysis is focused on the data of new construction projects.

3.2 STATISTICAL ANALYSIS OF RQI AND SR

The distribution of the RQI and SR data for different ages (1~4 years after construction) were shown in Figure 3.1. We expect both RQI and SR to decrease as the pavement ages, but the effect of age on performance (RQI and SR) is not very significant at least for the first four years after construction, as shown in Figure 3.1.

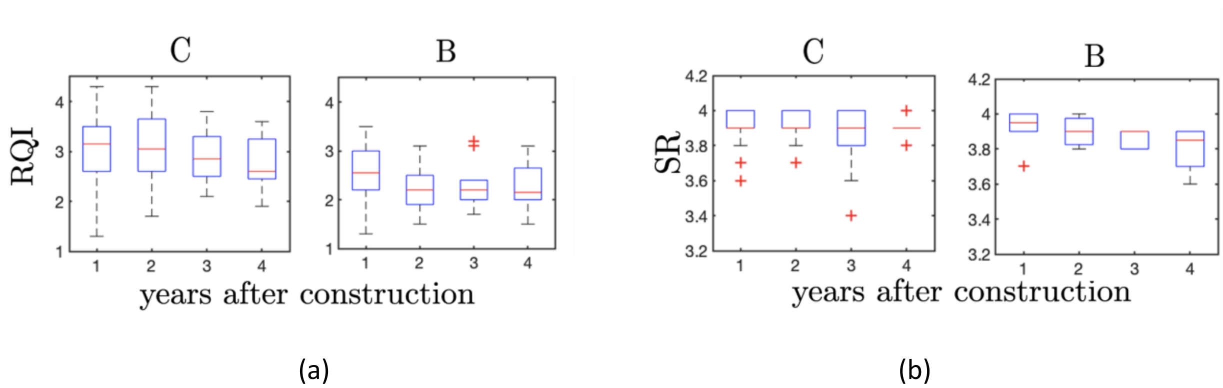
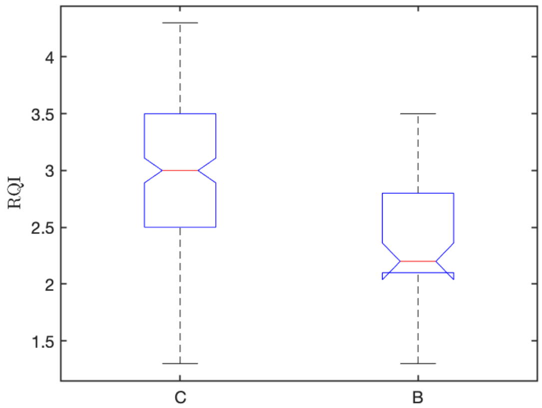
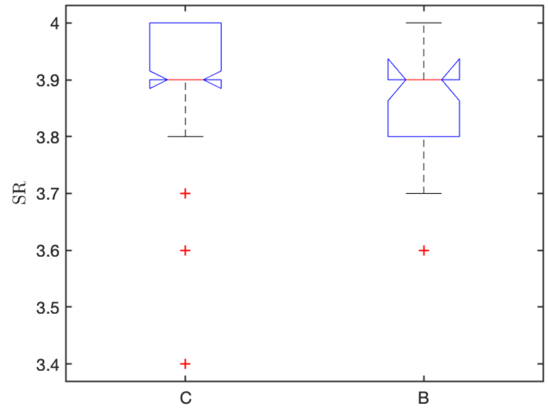


Figure 3.1 Box plots of (a) RQI, (b) SR

Since the effect of age in the first four years is minor as shown in Figure 3.1, the effect of binder grade is investigated considering the data of all ages (1-4 years), as shown in Figure 3.2. It is seen that projects used C binder clearly have a higher mean RQI than that used B binders. For SR, projects used C binders have slightly higher mean value than that used B binders, but the difference is not significant.



(a)



(b)

Figure 3.2 Box plots of all data of (a) RQI, (b) SR.

Analysis of Variance (ANOVA) was performed to check the effects binder grades on RQI and SR. The result is shown in Table 3.1 and Table 3.2 respectively. It confirms that the effect of binder grade on RQI is statically significant, given the p-value is lower than 0.001, while its effect on SR is not statically significant, since the p-value, 0.31 is large than the significant level of 0.05.

Table 3.1 Two-Way ANOVA Table for RQI

Source	SS	df	MS	F ratio	p-value
Binder grade	14.3	1	14.3	43.5	<0.001
Error	82.8	252	0.3		
Total	97.1	253			

Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

Table 3.2 Two-Way ANOVA Table for SR

Source	SS	df	MS	F ratio	p-value
Binder grade	0.01	1	0.01	1.06	0.31
Error	1.43	119	0.01		
Total	1.44	120			

Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

In summary, the statistical analysis shows that, in general, new construction projects that used C (polymer-modified) binders performed better than those who used B (unmodified) binders.

CHAPTER 4: EXPERIMENTAL INVESTIGATION OF ASPHALT BINDERS

In this Chapter, the research team contacted the Chemical Laboratory at the Office of Materials and Road Research (OMRR) and obtained extra material from B and C binders that had been submitted to the Chem Lab during the previous construction season. These binders were experimentally evaluated by the BBR creep test and a newly developed BBR strength test that uses a modified BBR equipment called BBR-Pro. The second test procedure characterizes the failure properties of binders by strength and failure strain of binders. Based on the experimental results, the low-temperature performance of the B and C binders were compared.

4.1 MATERIAL INFORMATION AND EXPERIMENTAL PLAN

4.1.1 Material Information

Fourteen binders were selected by the research team, including seven type B and seven type C binders. All C binders are polymer modified binders, while all B binders are unmodified binders. More information, including its S and m values, of the 14 binders is listed in Table 4.1.

Table 4.1 Information of the binders

#	Binder ID	PG	Binder Type	Polymer Modification	S and m, PGLT+10	
					S(60s), MPa	m(60s)
1	131	58H-34	C	Yes	274	0.349
2	145	58H-34	C	Yes	94	0.331
3	301	58H-34	C	Yes	226	0.299
4	348	58H-34	C	Yes	275	0.316
5	391	58H-34	C	Yes	118	0.319
6	26	58H-34	C	Yes	122	0.339
7	350	58H-34	C	Yes	265	0.307
8	79	58S-28	B	No	133	0.31
9	93	58S-28	B	No	267	0.326
10	126	58S-28	B	No	167	0.299
11	146	58S-28	B	No	107	0.305
12	147	58S-28	B	No	101	0.346
13	294	58S-28	B	No	272	0.328
14	323	58S-28	B	No	254	0.303

4.1.2 Experimental Plan

First, a BBR creep test (AASHTO T313-19) was performed for 240 seconds, followed by 240 seconds of recovery, At the end of the recovery time, a BBR strength test (Marasteanu et al. 2017) was performed on the same specimen until it failed. The BBR strength test evaluates the failure properties of asphalt binders, i.e., the strength and failure strain. For each binder, two temperature levels were investigated, the PG low temperature (PGLT) + 10 °C and PGLT + 4 °C. For each binder and test temperature, three replicates were tested.

4.2 BBR CREEP TEST RESULTS

The BBR creep test characterizes the rheological properties of binders at low temperatures. The creep compliance is calculated based on the BBR creep test data. The creep compliance for one of the binders, the binder 131, is shown in Figure 4.1 as an example.

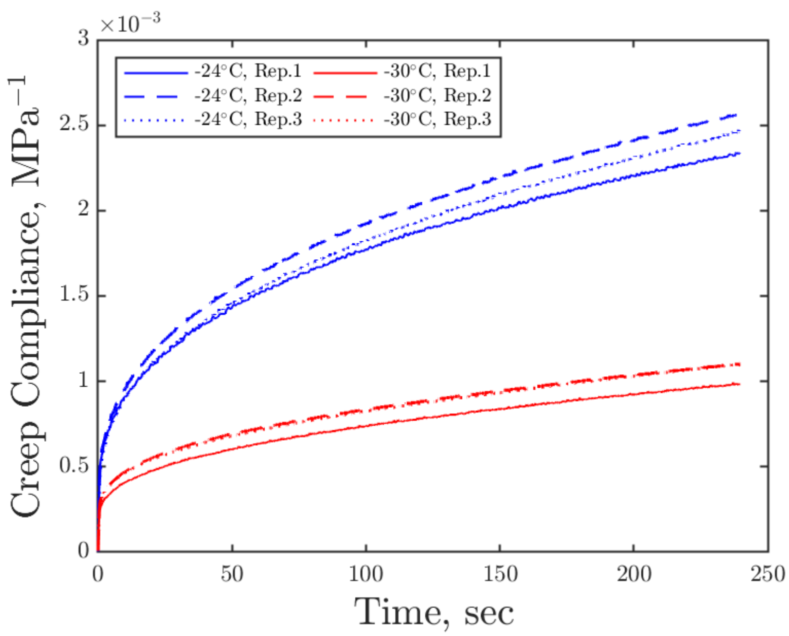


Figure 4.1 Creep compliance of the binder 131 at -24 °C and -30 °C

4.2.1 S and m Values

From the creep compliance, the creep stiffness S(t) (the inverse of creep compliance) and the m-value (the slope of log S(t) vs. log(t)) are calculated. The S and m-value at 60 seconds, S(60s) and m(60s), are the parameters used to determine the low temperature performance grade of binders (AASHTO T313-19). The results of S(60s) and m(60s) are calculated and listed in Table 4.2. It is seen that at the temperature PGLT + 10 °C, the criteria for S and m, S(60s) ≤ 300 MPa and m(60s) ≥ 0.3, are both satisfied, while at a lower temperature PGLT + 4 °C, at least one of the criteria is not satisfied. This confirmed the correctness of the low temperature performance grade of the selected binders.

Table 4.2 S and m values of the binders

Binder Type	Binder ID	Temp, °C	Ave. S(60s) MPa	Std. S(60s) MPa	Ave. m(60s)	Std. m(60s)
C	131	-24	190.27	7.79	0.31	0.0097
		-30	438.46	22.06	0.28	0.0064
	145	-24	255.04	7.17	0.30	0.0060
		-30	538.66	29.79	0.25	0.0038
	301	-24	225.33	13.73	0.34	0.0052
		-30	461.62	17.52	0.28	0.0163
	348	-24	274.95	23.56	0.31	0.0099
		-30	663.36	25.09	0.24	0.0087
	391	-24	228.65	5.58	0.33	0.0072
		-30	504.40	22.38	0.28	0.0060
	26	-24	166.76	13.43	0.37	0.0136
		-30	411.65	72.13	0.27	0.0206
	350	-24	157.20	9.30	0.38	0.0042
		-30	378.89	13.51	0.28	0.0068
B	79	-18	168.28	5.49	0.34	0.0158
		-24	395.91	18.57	0.28	0.0102
	93	-18	202.46	6.27	0.33	0.0057
		-24	457.25	14.51	0.28	0.0090
	126	-18	198.17	7.85	0.34	0.0092
		-24	451.84	16.15	0.27	0.0078
	146	-18	192.05	8.13	0.33	0.0120
		-24	396.22	52.82	0.29	0.0115
	147	-18	193.82	11.60	0.32	0.0052
		-24	387.06	43.70	0.28	0.0189
	294	-18	194.12	19.00	0.34	0.0066
		-24	477.83	39.20	0.28	0.0044
	323	-18	196.91	15.19	0.35	0.0152
		-24	419.78	71.43	0.26	0.0260

4.3 BBR STRENGTH TEST RESULTS

The BBR strength test characterizes the failure properties of binders by measuring two parameters: strength and the failure strain (Marasteanu et al. 2017, Matias De Oliveira et al. 2019, Yan et al. 2020). Figure 4.2 shows the BBR strength test data of Binder 131. The stress-strain curves of other binders are similar to that shown in Figure 4.2. It is seen that the stress-strain curves at the two test temperatures are clearly different from each other, which is due to the difference in the rheological properties of the binder at the two temperatures. The end of the stress-strain curve is the failure point. The corresponding stress and strain at the failure point are the strength and the failure strain, respectively. The x and y coordinates of the circles in Figure 4.2 show the average failure strain and average strength of the three replicates. The horizontal and vertical error bars show the standard deviation of the failure strain and strength, respectively. The strength and failure strain results of all the binders are listed in Table 4.3.

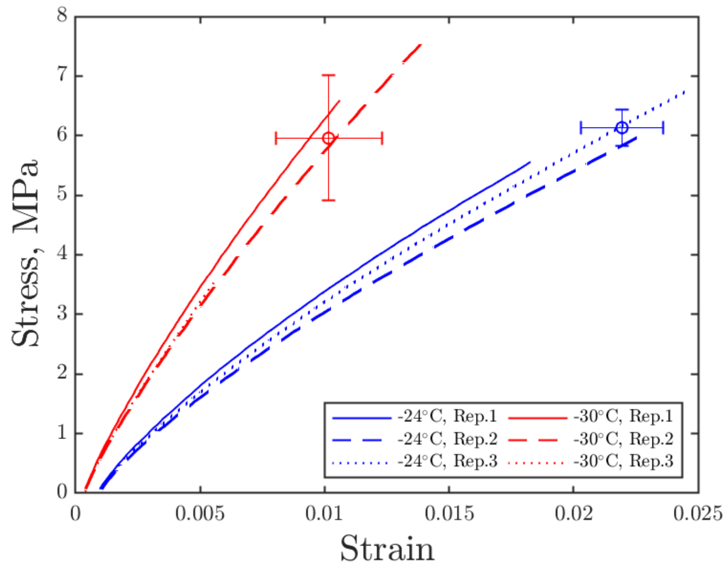


Figure 4.2 Stress – Strain relationship of the binder 131 at -24 °C and -30 °C, obtained from BBR strength test.

Table 4.3 Strength and failure strain of the binders

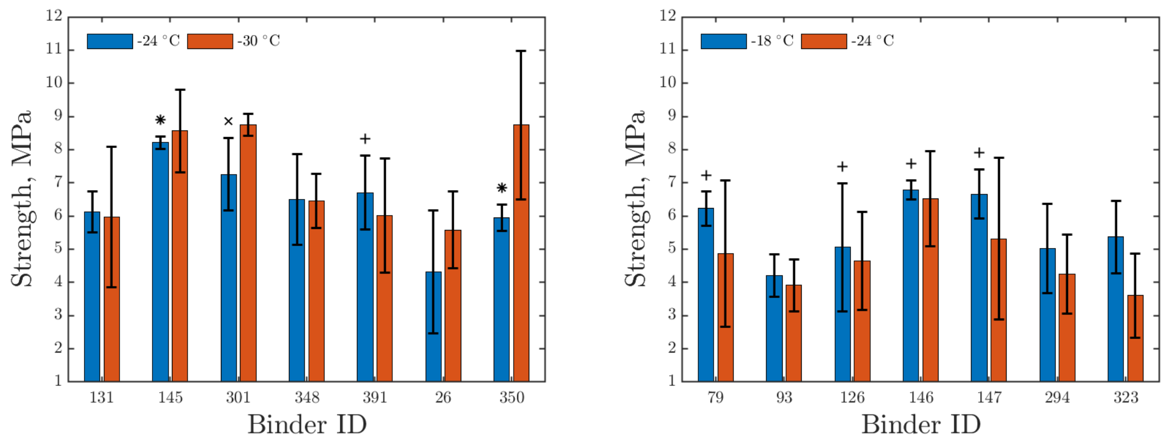
Binder Type	Binder ID	Temp, °C	Ave. σ_f, MPa	Std. σ_f, MPa	Ave. ε_F	Std. ε_F	Note*
C	131	-24	6.13	0.61	0.022	0.0033	
		-30	5.96	2.11	0.010	0.0043	
	145	-24	8.21	0.18	0.025	0.0014	3
		-30	8.57	1.25	0.012	0.0023	
	301	-24	7.26	1.09	0.023	0.0037	2
		-30	8.75	0.33	0.014	0.0012	
	348	-24	6.50	1.37	0.016	0.0042	
		-30	6.45	0.81	0.007	0.0011	
	391	-24	6.70	1.12	0.020	0.0040	1
		-30	6.02	1.73	0.009	0.0038	
	26	-24	4.32	1.85	0.016	0.0079	
		-30	5.58	1.15	0.009	0.0028	
	350	-24	5.95	0.39	0.025	0.0004	3
		-30	8.74	2.24	0.017	0.0052	
B	79	-18	6.22	0.51	0.023	0.0020	1
		-24	4.87	2.20	0.008	0.0040	
	93	-18	4.20	0.64	0.013	0.0023	
		-24	3.92	0.78	0.006	0.0012	
	126	-18	5.06	1.93	0.016	0.0074	1
		-24	4.64	1.48	0.007	0.0025	
	146	-18	6.79	0.29	0.023	0.0012	1
		-24	6.52	1.44	0.012	0.0015	
	147	-18	6.66	0.74	0.023	0.0027	1
		-24	5.32	2.43	0.010	0.0053	
	294	-18	5.01	1.35	0.015	0.0047	
		-24	4.25	1.20	0.006	0.0018	
	323	-18	5.37	1.09	0.018	0.0052	
		-24	3.61	1.27	0.005	0.0017	

It is important to note that some samples did not fail at the maximum load applied by the BBR device. In Table 4.3, the numbers listed in the column “Note” denote the number of replicates that did not fail. If the sample does not fail, the real strength and failure strain would be higher than the measured values.

4.3.1 Effect of Temperature on Failure Properties

The effect of the testing temperature on the failure properties of binders can be seen in Figure 4.3andFigure 4.4. The markers in Figure 4.3and Figure 4.4mean the number of samples that did not fail, with ‘+’, ‘x’, and ‘*’ indicating the number being 1, 2, and 3, respectively. In the cases with markers, the real strength/failure strain would be higher than the value shown by the heights of bars. The error bars show the standard deviations of the values.

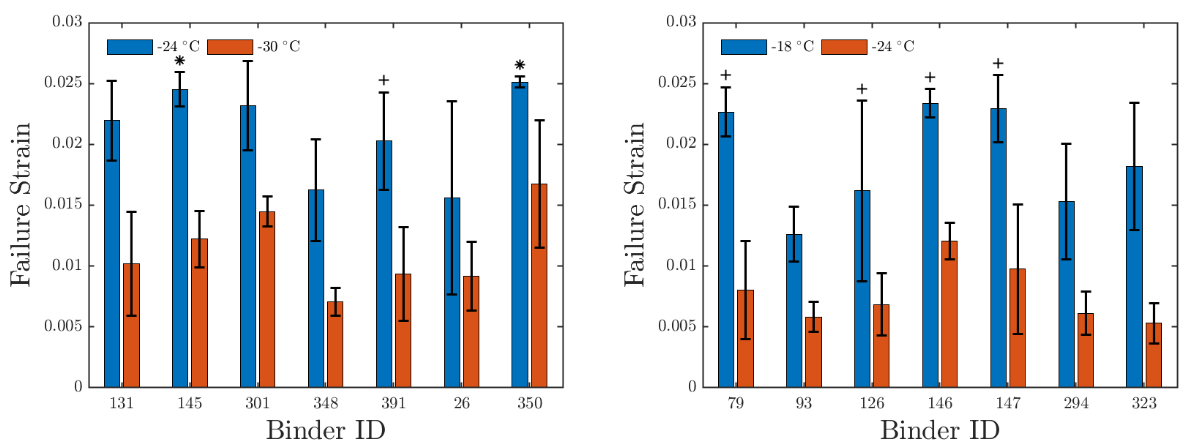
The effect of the testing temperature on the strength of binders is shown in Figure 4.3. For C binders (Figure 4.3(a)), the ranking of the strength with respect to temperature is inconsistent. Three out of seven binders had higher average strength at the higher test temperature, while the rest of the binders were the opposite. For B binders (Figure 4.3(b)), all the binders had higher average strength at the higher test temperature. However, considering the variability of the data, the difference in strength between the two temperatures is not significant.



(a) (b)

Figure 4.3 Strength of the binders at two temperatures (a) C binders, (b) B binders.

Figure 4.4 shows that test temperature has a strong effect on failure strain of binders. It is seen that a 6 °C increase in temperature can almost double the failure strain of the binders.



(a) (b)

Figure 4.4 Failure strain of the binders at two temperatures (a) C binders, (b) B binders.

4.4 ΔT_c

In the past decade, a new parameter called ΔT_c has gained considerable attention among both researchers and highway agencies. It is believed that ΔT_c is related to non-load related cracking distresses in asphalt pavements, e.g., due to aging of the binder (Asphalt Institute 2019). Currently, the implementation of ΔT_c is still under debate, while it is generally adopted that ΔT_c is a good research tool for forensic studies (McDaniel and Shah 2019). Therefore, in this study, the ΔT_c values of the 14 binders are calculated. The relationship between ΔT_c and other properties (thermal stress and failure properties) are investigated.

To calculate ΔT_c, the S(60s) and m(60s) values are used to estimate the critical temperatures at which S(60s) = 300 MPa, denoted as T_{cS}, and the critical temperatures at which m(60s) = 0.3, denoted as T_{cm}. The ΔT_c parameter is then calculated as $T_{cS} - T_{cm}$. Therefore, ΔT_c indicates whether the performance grade of the binder is governed by creep stiffness S or creep rate, the m-value. When ΔT_c is positive, the binder is referred to as being "S-controlled" (failing the S criterion at a warmer temperature than the m criterion), while a negative ΔT_c indicates the binder is "m-controlled" (fails m criterion at a warmer temperature than the S criterion). The absolute magnitude of the ΔT_c indicates the degree to which the binder is S- or m-controlled. Studies have observed that creep stiffness and m-value may not change at the same rate due to aging. Rather, the loss in relaxation (m-value) may have a more significant effect on cracking performance than the increase in creep stiffness (Anderson et al. 2011, Asphalt Institute 2019). Thus, higher ΔT_c (more S-controlled) is desired. Criteria have been proposed for ΔT_c. For example, $\Delta T_c \geq -5$ °C for 40 hours PAV aged binders is required by AASHTO PP 78-17 and $\Delta T_c \geq -2$ °C for 20 hours PAV aged binders is required by Utah DOT (Asphalt Institute 2019).

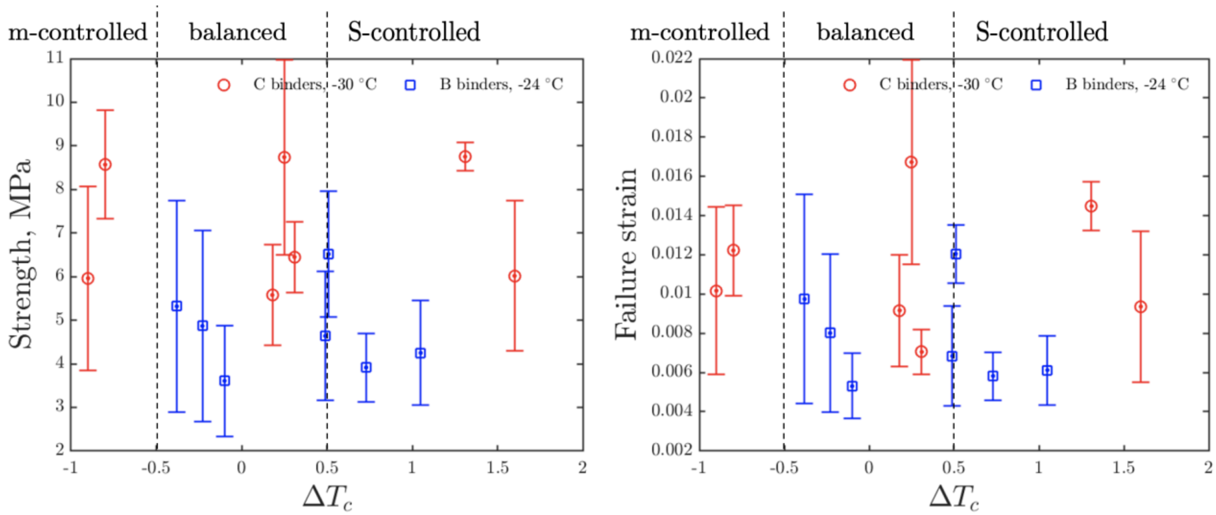
The calculated critical temperatures and the ΔT_c are listed in Table 4.4. The PAV aging period was 20 hours for all the binders. It is seen that all the binders satisfy $\Delta T_c \geq -2$ °C, the criterion currently used by Utah DOT (Asphalt Institute 2019). According to the values of ΔT_c, the binders are classified into three categories, $\Delta T_c < -0.5$ °C, -0.5 °C ≤ $\Delta T_c \leq 0.5$ °C, and $\Delta T_c > 0.5$ °C. The three categories are denoted as "m-controlled", "balanced", and "S-controlled", respectively. The classification of the binders is also shown in Table 4.4.

Table 4.4 Critical temperatures and ΔT_c of the binders

Binder Type	Binder ID	T_{cS}, °C	T_{cm}, °C	ΔT_c, °C	Classification
C	131	-37.19	-36.29	-0.90	m-controlled
	145	-35.29	-34.49	-0.80	m-controlled
	301	-36.33	-37.64	1.31	S-controlled
	348	-34.54	-34.85	0.31	Balanced
	391	-36.00	-37.61	1.60	S-controlled
	26	-37.77	-37.95	0.18	Balanced
	350	-38.32	-38.57	0.25	Balanced
B	79	-32.02	-31.79	-0.23	Balanced
	93	-30.85	-31.57	0.73	S-controlled
	126	-31.02	-31.51	0.49	Balanced
	146	-31.63	-32.14	0.51	S-controlled
	147	-31.73	-31.35	-0.38	Balanced
	294	-30.87	-31.91	1.05	S-controlled
	323	-31.33	-31.23	-0.10	Balanced

The ΔT_c data was further used to determine if there are any correlations with the failure properties of binders. Only the data calculated at the lower temperatures (-30 °C for C binders and -24 °C for B binders) was used in the analysis since at the higher test temperatures (-24 °C for C binders and -18 °C for B binders) some specimens did not fail in the BBR strength test.

The relationship between ΔT_c and strength is shown in Figure 4.5(a). It is seen that there is no clear trend between ΔT_c and strength, and there is no significant difference in strength between the three different categories. The relationship between ΔT_c and failure strain is shown in Figure 4.5(b). Similarly, no clear trend can be identified between ΔT_c and failure strain, and no significant difference in failure strain can be identified between the three different categories.



(a) (b)

Figure 4.5 The relationship of ΔT_c with (a) strength, (b) failure strain.

4.5 THERMAL STRESS CALCULATION

Using the low temperature rheological properties obtained from the BBR creep test, we can calculate the accumulation of thermal stress under a certain temperature history. We consider a special temperature history, which mimic the temperature drop in winter. At t=0, we assume the binder is at a temperature T_1 and is stress-free. When t>0, temperature decreases with the increase in time at a constant rate α_1, i.e.,

$$T(t) = T_1 - \alpha_1 t \qquad \text{Eq. 1}$$

Therefore, the thermal strain can be calculated as.

$$\varepsilon_T(t) = \alpha_2 \alpha_1 t \qquad \text{Eq. 2}$$

where α_2= coefficient of thermal expansion. In this study we consider the special case where, T_1 = 20 °C, α_1 = 20 °C /hr, and α_2 = 2×10^{-5} $(°\mathrm{C})^{-1}$

Two effects must be considered in the thermal stress calculation. First, the stress relaxation of viscoelasticity must be considered, which can be characterized by the relaxation modulus $E(t)$. Second, as the temperature is changing, the effect of temperature on the relaxation modulus must be considered. In other words, the relaxation modulus must be considered as a function of both time and temperature, i.e., $E(t, T)$. The method proposed by Marasteanu (2004) was followed to evaluate the thermal stress.

For the effect of temperature on relaxation modulus, it is assumed that the temperature's effect is only on changing the intrinsic time scale of the material, and ignore the effect of temperature on density, glassy modulus, etc. As a result, the effect of temperature on material properties can be captured by a change of time scale. A time scale named "reduced time", ξ, can be defined in proportion to the intrinsic time scale of the material:

$$\frac{dt}{\alpha(T_0)} = \frac{d\xi}{\alpha(T)} \quad \text{Eq. 3}$$

where $\alpha(T)$ = intrinsic time scale of the material as a function of temperature T, T_0 = the reference temperature at which the regular time scales t and ξ are equal.

According to thermodynamics, the effect of T on the intrinsic time scale is in an exponential form, i.e., $\alpha(T) = \exp(f(T))$. For simplicity, it is common to assume a linear form for the $f(T)$, i.e.,

$$\alpha(T) = \exp(\beta - CT) \quad \text{Eq. 4}$$

where β and C are coefficients. The ratio, $\alpha(T_0)/\alpha(T)$, is needed in the following calculation which equals $\exp(C(T + T_0))$. It is seen that the coefficient β cancelled out, so β is not needed for the calculation.

According to Eq. 3, the reduced time can be related to the regular time t by:

$$\xi(t) = \frac{1}{\alpha(T_0)} \int_0^t \alpha(T(\tau))\, d\tau \quad \text{Eq. 5}$$

For the specific temperature history in Eq. 1, the corresponding reduced time ξ can be calculated by substituting Eq. 1 and Eq. 4 into Eq. 5.

$$\xi(t) = \frac{\exp\left(C(T_1 - T_0)\right)}{\alpha_1 C}(1 - \exp(-\alpha_1 C t)) \quad \text{Eq. 6}$$

In the reduced time, the effect of temperature on relaxation modulus is cancelled out by by the change of the time scale, i.e.,

$$E(t, T) = E(\xi, T_0) \quad \text{Eq. 7}$$

Therefore, the temperature varying process in the regular time scale can be transformed to an isothermal process in the reduced time scale, so the thermal stress can be calculated by simply performing the viscoelastic convolution in the reduced time scale:

$$\sigma_T(\xi) = \int_0^{\xi} \frac{d\varepsilon(\xi')}{d\xi'} E(\xi - \xi', T_0) d\xi' \quad \text{Eq. 8}$$

Changing the time variable from ξ to t by the relationship $\xi(t)$ in Eq. 8, we can get the thermal stress as a function of regular time t.

$$\sigma_T(t) = \int_0^{t} \frac{d\varepsilon(\tau)}{d\tau} E(\xi(t) - \xi(\tau), T_0) d\tau \quad \text{Eq. 9}$$

Substituting Eq. 2 into Eq. 5, we get.

$$\sigma_T(t) = \alpha_1 \alpha_2 \int_0^t E(\xi(t) - \xi(\tau), T_0) d\tau \qquad \text{Eq. 10}$$

Therefore, if the relaxation modulus at different temperatures $E(t, T)$ is known, then the thermal stress σ_T can be calculated by numerically integrating Eq. 10.

The relaxation modulus $E(t, T)$ can be obtained by fitting the Christensen–Anderson–Marasteanu (CAM) model to the experimental data of BBR creep test of multiple temperatures. The CAM model has the following form:

$$E(t, T) = E_g \left(1 + \left(\frac{\tau(t,T)}{t_c}\right)^v\right)^{-\frac{w}{v}} \qquad \text{Eq. 11}$$

$$\tau(t, T) = t \exp(C(T + T_0)) \qquad \text{Eq. 12}$$

The parameters C, E_g, t_c, v, w can be obtained by curve fitting.

The calculation of the thermal stress of the Binder 131 is demonstrated here as an example. First, the creep compliance $D(t)$ data (shown in Figure 4.1) are converted to relaxation modulus by numerically solving the convolution integral.

$$\int_0^t E(\tau) D(t - \tau) d\tau = t \qquad \text{Eq. 13}$$

Then, the CAM model is fitted to the relaxation modulus. The result is shown in Figure 4.6.

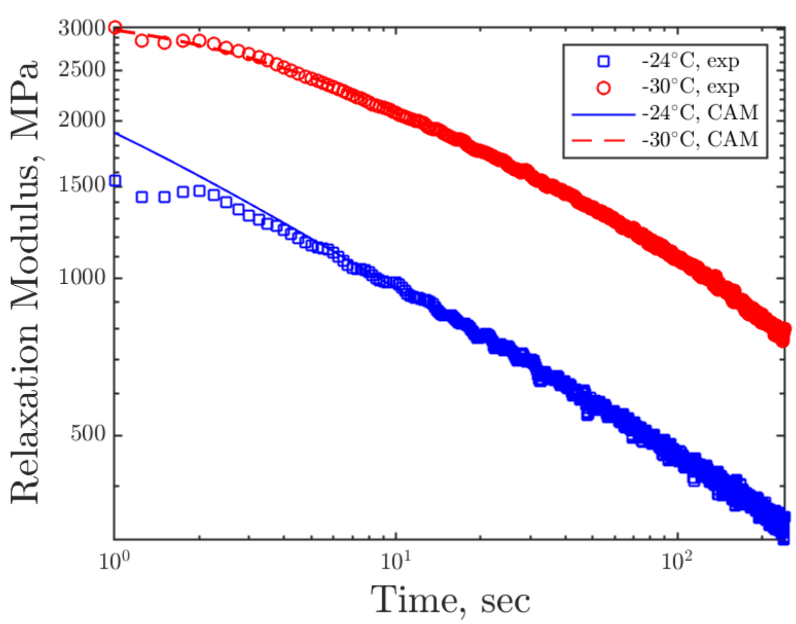


Figure 4.6 Relaxation modulus of Binder 131 fitted by CAM model

Substituting the fitted CAM model to Eq. 10, the thermal stress can be calculated. The result is shown in Figure 4.7. It is seen that thermal stress increases with the decrease in temperature.

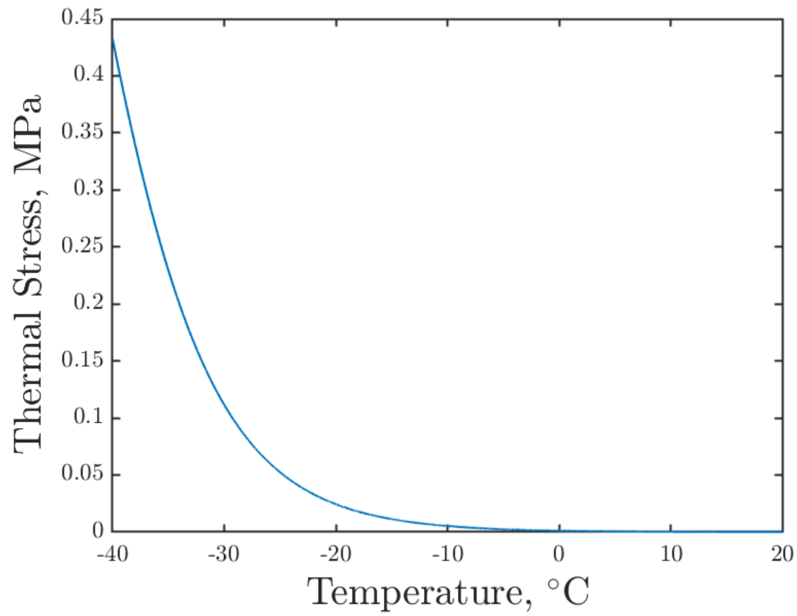


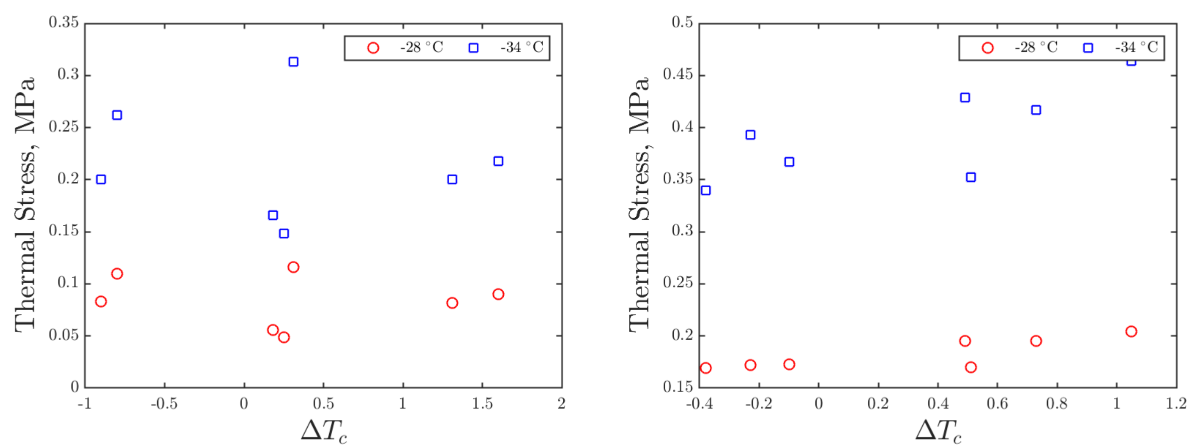
Figure 4.7 Thermal stress of Binder 131

The other binders have similar behaviors as that shown in Figure 4.7. The thermal stress results at two temperatures (-28 °C and -34 °C) are calculated and listed in Table 4.5.

Table 4.5 Predicted thermal stress of the binders at -28 and -34 °C

Binder Type	Binder ID	Thermal Stress, MPa, -28 °C	Thermal Stress, MPa, -34 °C
C	131	0.083	0.200
	145	0.110	0.262
	301	0.082	0.200
	348	0.116	0.313
	391	0.090	0.218
	26	0.056	0.166
	350	0.049	0.148
B	79	0.172	0.393
	93	0.195	0.417
	126	0.195	0.429
	146	0.170	0.352
	147	0.169	0.340
	294	0.204	0.464
	323	0.173	0.367

The relationships between thermal stress and ΔT_c are examined in Figure 4.8. As shown, the data are rather scattered, and no general trend can be identified between thermal stress and ΔT_c.



(a) (b)

Figure 4.8 Relationship of thermal stress with ΔT_c (a) C binders, (b) B binders.

4.6 COMPARISON OF C AND B BINDERS

4.6.1 Thermal Stress

Based on thermal stress calculated in Table 4.5, the thermal stress values of C and B binders are compared, as shown in Figure 4.9. It is seen that, at both the temperature levels (-28 °C and -34 °C), C binders clearly have lower thermal stress than B binders, and therefore are less prone to crack.

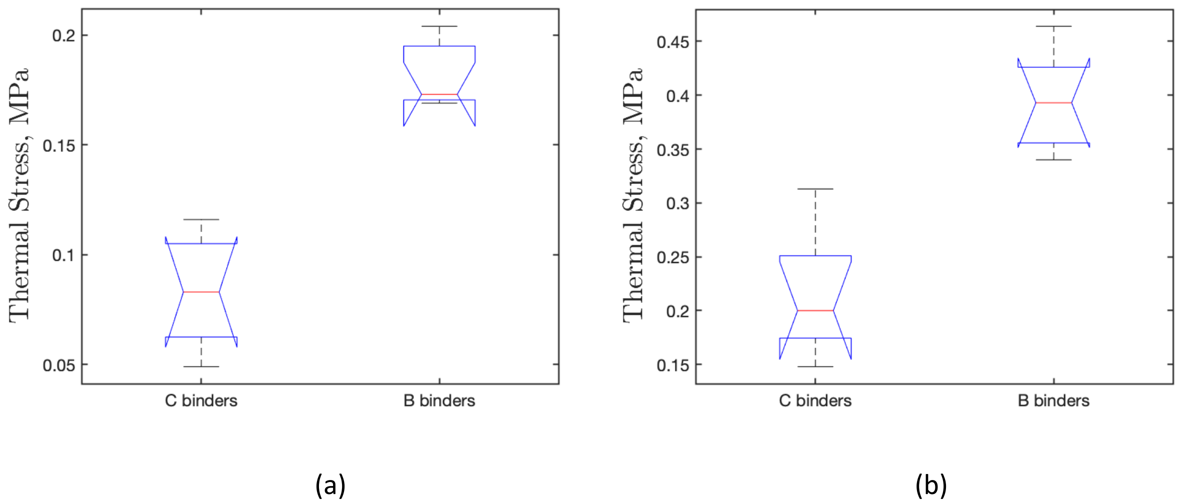


Figure 4.9 Box plot comparison of thermal stress between C and B binders, (a) -28 °C, (b) -34 °C.

An ANOVA was performed to further compare the thermal stress of C and B binders. The ANOVA results for the thermal stress at -28 °C and -34 °C are shown in Table 4.6 and Table 4.7 respectively. It is seen that the P-values are lower than 0.001 for the thermal stress at both temperatures, which confirms that the difference in thermal stress between the C and B binders is statistically significant. Since the thermal stress calculation is based on the rheological properties (relaxation modulus) of binders, it can be concluded that the lower thermal stress of C binders can be attributed to the lower relaxation modulus of C binders compared to that of B binders.

Table 4.6 ANOVA Table for comparing the thermal stress at -28 °C of C and B binders

Source	SS	df	MS	F ratio	p-value
Binder Type	0.0342	1	0.0342	81.07	<0.001
Error	0.00506	12	0.00042		
Total	0.03927	13			

Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

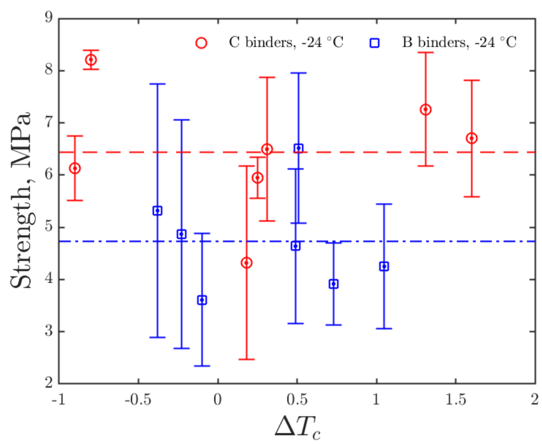
Table 4.7 ANOVA Table for comparing the thermal stress at -34 °C of C and B binders

Source	SS	df	MS	F ratio	p-value
Binder Type	0.1125	1	0.1125	43.24	<0.001
Error	0.03122	12	0.0026		
Total	0.14372	13			

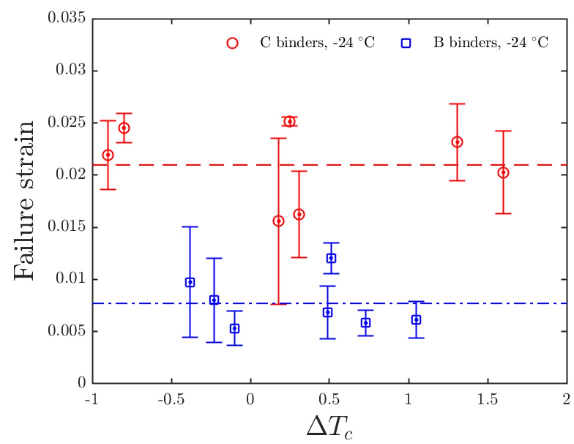
Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

4.6.2 Failure Properties

The strength and failure strain of C and B binders are compared in Figure 4.10. Data at the same temperature (-24 °C) were used for the comparison. It is seen that C binders have higher overall strength and failure strain than B binders, and therefore, resist cracking better.



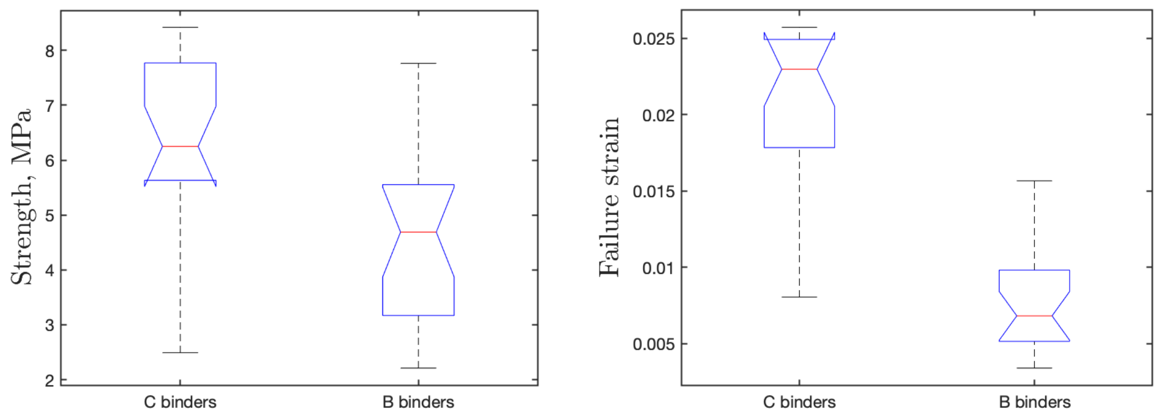
(a)



(b)

Figure 4.10 Comparison between C and B binders, (a) strength, (b) failure strain.

An ANOVA was performed to further check the difference in strength and failure strain between the C and B binders. The box plots comparisons of C and B binders are shown in Figure 4.11. The results of ANOVA are listed in Table 4.8 and Table 4.9.



(a) (b)

Figure 4.11 Box plots of the comparison between C and B binders, (a) strength, (b) failure strain.

Table 4.8 ANOVA Table for comparing the strength between C and B binders

Source	SS	df	MS	F ratio	p-value
Binder Type	30.621	1	30.6208	12.6	0.001
Error	97.229	40	2.4307		
Total	127.849	41			

Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

Table 4.9 ANOVA Table for comparing the failure strain of C and B binders

Source	SS	df	MS	F ratio	p-value
Binder Type	0.00185	1	0.00185	100.39	<0.001
Error	0.00074	40	0.00002		
Total	0.00259	41			

Note: SS = sum of squares. df = degrees of freedom. MS = mean square.

As shown, the P-values are 0.001 and <0.001 for strength and failure strain, respectively, which confirms that the differences in strength and failure strain between the C and B binders are statistically significant. This result indicates that C binders in general have higher resistance to failure than B binders. The higher failure resistance of C binders could be attributed to the polymer modification.

4.7 CONCLUSIONS

In this task, fourteen binders obtained from the Office of Materials and Road Research, including seven type C and seven type B. They were experimentally evaluated by the BBR creep test and the BBR strength test. Based on the experimental results, S and m values, thermal stress, strength, and failure strain of the binders were calculated. The low-temperature rheological and failure properties of the B and C binders were compared. The main conclusions are the following:

- The tested S and m values from BBR creep test confirmed the low temperature performance grades (LTPG) of the selected binders. Specifically, the LTPG are -34 °C for all C binders, and the LTPG are -28 °C for all B binders.
- The selected binders cover a wide range of ΔT_c, which enables the investigation of the relationship of ΔT_c with other properties, e.g., the thermal stress and failure properties. Based on the limited data of this study, no clear trend was identified between ΔT_c and those properties.
- The BBR strength test results showed that the increase in temperature has a strong effect on increasing the failure strain of the binder, while the effect of temperature on the strength of binders is not significant.
- The comparison between C and B binders shows that C binders clearly develop lower thermal stresses than B binders, which is due to the lower relaxation modulus of C binders than B binders. Moreover, C binders generally have higher strength and failure strain than B binders, which might be due to the polymer modification of the C binders.

CHAPTER 5: EXPERIMENTAL INVESTIGATION OF ASPHALT MIXTURES

In this Chapter, the fracture energy of asphalt mixes prepared with Binder Grades B and C were compared. Both plant- and lab-produced asphalt mixes were included in the study. The fracture energy was determined using the Disk-shaped Compact Tension (DCT) test (ASTM D-7313). Asphalt mixes were tested at the University of Minnesota Duluth (UMD). This set of asphalt mixes is referred to as UMD mixes in this report.

Additionally, DCT fracture energy results of 88 different asphalt mixes varied with binder grade (B and C), and traffic levels (3, 4, and 5), collected from MnDOT, were compared. The DCT test results were collected from the Office of the Materials and Road Research (OMRR). This set of mixes is referred to as OMRR mixes in this report.

5.1 MIXTURE INFORMATION

5.1.1 UMD Mixes

The four asphalt mixes with binder-C used in the study are plant-produced; two mixes were collected from Duluth's Northland constructors' plant. One mix was collected from the plant of Ulland Brothers at Duluth. The fourth binder-C mix was collected from the MnROAD; this mix was used in one of the test cells constructed during 2022 (Mix ID 2239) (NRRA projects). Table 5.1 presents the description of all the mixes tested at UMD. Each of the binder-C mixes has its binder-B counterpart, and together they are referred to as a Set. The DCT results of the binder-B and binder-C mixes in a set are compared. Out of four B mixes, three mixes (Sets 1-3) were prepared in the UMD laboratory using the same aggregates and binders used in their binder-C counterparts; materials were collected from the respective plants. The fourth binder-B mix (Set 4) was plant-produced as it was collected from the MnROAD. All the mixes used in the study were intended for the wearing course and designed as per Superpave or MnDOT's mix design criteria 2360 (MnDOT, 2020). As shown in Table 5.1, two sets (Sets 1 and 2) of mixes were designed for traffic level 3 (1-3 million ESALs), one set each for traffic level 4 (3-10 million ESALs) (Set 3) and 5 (10-30 million ESALs) (Set 4). Set 3 had aggregate size gradation B (SP 12.5) and all the others had aggregate size gradation A (SP 9.5).

Table 5.1 Description of Asphalt Mixes

Set no.	Mix. Designation	Plant information/ Material source	Traffic level	Aggregate gradation (SP max. aggregate size)
1	SPWEA330C (1)	Northland Constructors	3	A (SP 9.5)
	SPWEA330B (1)	Northland Constructors	3	A (SP 9.5)
2	SPWEA330C (2)	Ulland Brothers, Inc	3	A (SP 9.5)
	SPWEA330B (2)	Ulland Brothers, Inc	3	A (SP 9.5)
3	SPWEB440C	Northland Constructors	4	B (SP 12.5
	SPWEB440B	Northland Constructors	4	B (SP 12.5
4	SPWEAB540C	MnROAD	5	A (SP 9.5)
	SPWEAB540B	MnROAD	5	A (SP 9.5)

The aggregate gradations of Sets 1 through 3 are provided in Figure 5-1. As each of the three lab-produced binder-B mixes was prepared using the same aggregate gradation that was used for their binder-C counterpart (plant-mix), the gradations of mixes of a set are represented by a single curve. Mix design sheets of Sets 1 through 3 are provided in Appendix for more information.

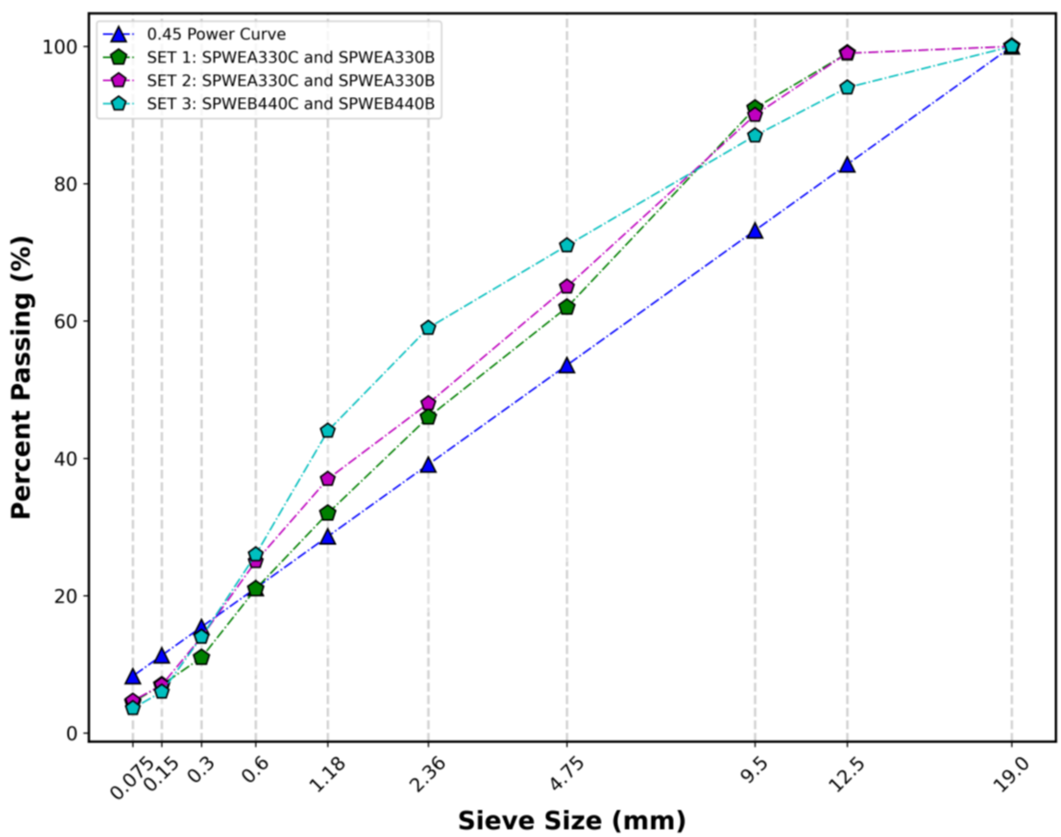


Figure 5-1 Gradations of the asphalt mixes

5.1.2 OMRR Mixes

DCT test results of total of 94 different asphalt mixes were collected from MnDOT, out of which 47 mixes were prepared with binder-B and the remaining had binder-C. Mixes were available for traffic levels 2 through 5. However, the vast majority (88 mixes) of the mixes were prepared for traffic levels 3 (44 mixes) and 4 (44 mixes). The mixes for traffic levels 2 and 5 (6 mixes) did not have their counterparts; therefore, they are not included in the comparison. All the 88 mixes considered were prepared for the wearing courses and consists of aggregate gradation A or B.

5.2 DCT TEST

Disc-shaped Compact Tension test was conducted to determine the fracture energy of asphalt mix samples at low temperatures. Test samples were compacted with 7±0.5% air voids. DCT samples were 50 mm (2 in) thick with a diameter of 150 mm (6 in). The rest of the specimen geometry is provided in Figure 5-2 (a). The unique geometry of the DCT test samples allows them to be loaded in tension. All the samples were tested at the recommended DCT test temperature, which is the PG minimum grade + 10°C, i.e., -18°C (-4°F) for the B mixes and -24°C (-11.2°F) for C mixes. Test temperature was regularly monitored using temperature gauges, and the samples were tested within ±0.2°C of the recommended temperature value.

The testing was conducted in accordance with the ASTM standard D-7313 on a Universal Testing Machine (UTM-30) by IPC Global equipped with a 30 kN (6,744 lb) servo-hydraulic labyrinth bearing actuator assembly and a dual-axis control and data acquisition system. As soon as the test temperature was reached, a seating load of 0.2 kN (45lb) was applied, and the test was run in Crack Mouth Opening Displacement (CMOD) controlled mode, i.e., applied force on the specimen was varied according to the rate of opening of the crack mouth at a rate of 1 mm/min (.04 in/min). Figure 5-2 (b) shows a photograph of DCT test in progress.

The thermal cracking performance of asphalt mixes can be determined by calculating the fracture energy. The fracture energy of a material is generally defined as the energy required to create a new unit fracture surface in the material. The DCT fracture energy (G_f) is determined by calculating the fracture work (W_f), which is the area under the load-CMOD curve. A typical load vs. CMOD curve is shown in Figure 5-2 (c). Fracture work is normalized by the ligament area to determine the fracture energy, as shown in Equation 5.1.

$$G_f = W_f/(t \times a) \qquad (5.1)$$

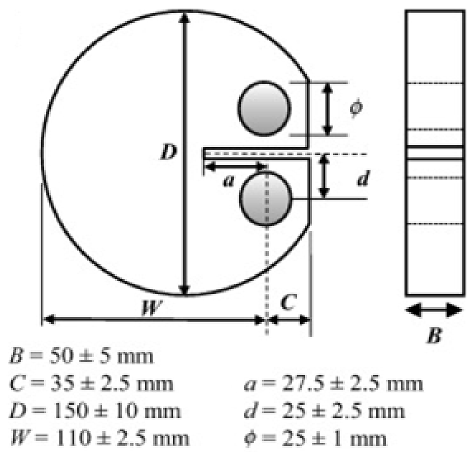
Where, G_f = Fracture Energy (J/mm^2), W_f = Work of fracture (J), t = thickness of the specimen (mm), and a = ligament length (mm) (ligament length is the remaining length along the pre-cut notch line).

The fracture work (W_f) in Equation 1 was calculated using Equation 5.2:

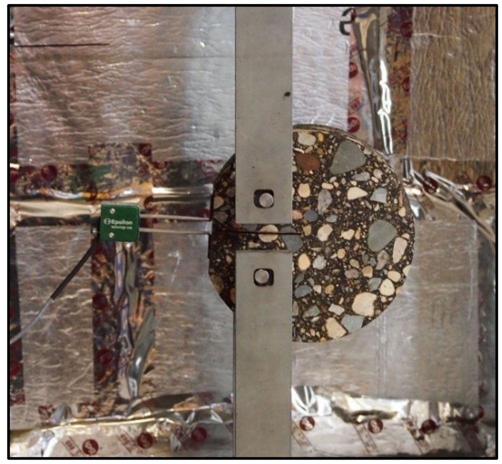
$$W_f = W_f^{Pre-peak} + W_f^{Post-peak} = \int_0^{\Delta_{Fmax}} F\,.\,du + \int_{\Delta_{Fmax}}^{\Delta_{Final}} F\,.\,du \qquad (5.2)$$

Where, u = CMOD, F = force, F_{max} = maximum force (at peak), and Δ_i = CMOD thresholds.

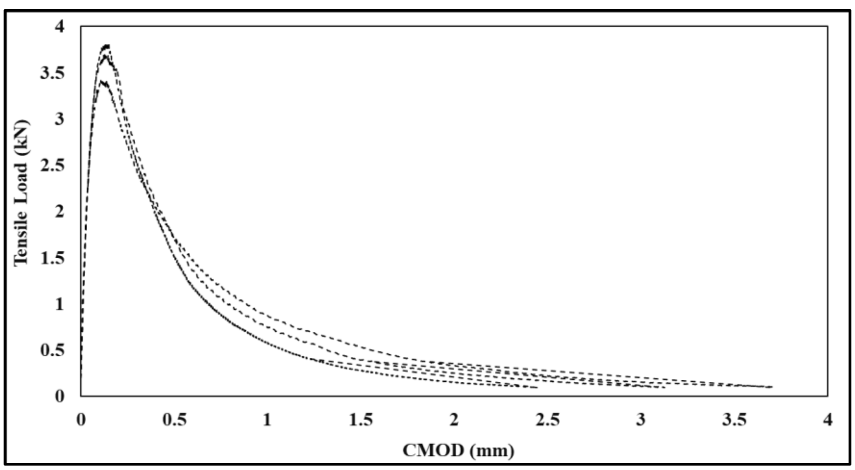
It may be noted that before performing any calculation on the load vs. CMOD curve, a polynomial pre-smoothing of the curve was done.



(a)



(b)



(c)

Figure 5-2 (a) DCT specimen geometry, (b) Sample installation, and (c) Load-CMOD curves generated from DCT testing.

5.3 UMD TEST RESULTS

For the UMD mixes, at least five replicate samples were tested for each mix. All the samples were prepared with 7±0.5% air voids. Gyratory compacted cylindrical samples were cut and sawed to the dimension of the test specimen.

5.3.1 Set 1

Figure 5-3 and Figure 5-4 show the DCT fracture energies (G_f) measured for the samples of Set 1. The average fracture energy value for the binder-C mixes is 536 J/m^2 with a standard deviation of 74 J/m^2; whereas, the average G_f for the B-binder mixes is 480 J/m^2 with a standard deviation of 158 J/m^2. The average G_f of the binder-C mix is approximately 14% higher than the binder-B mix.

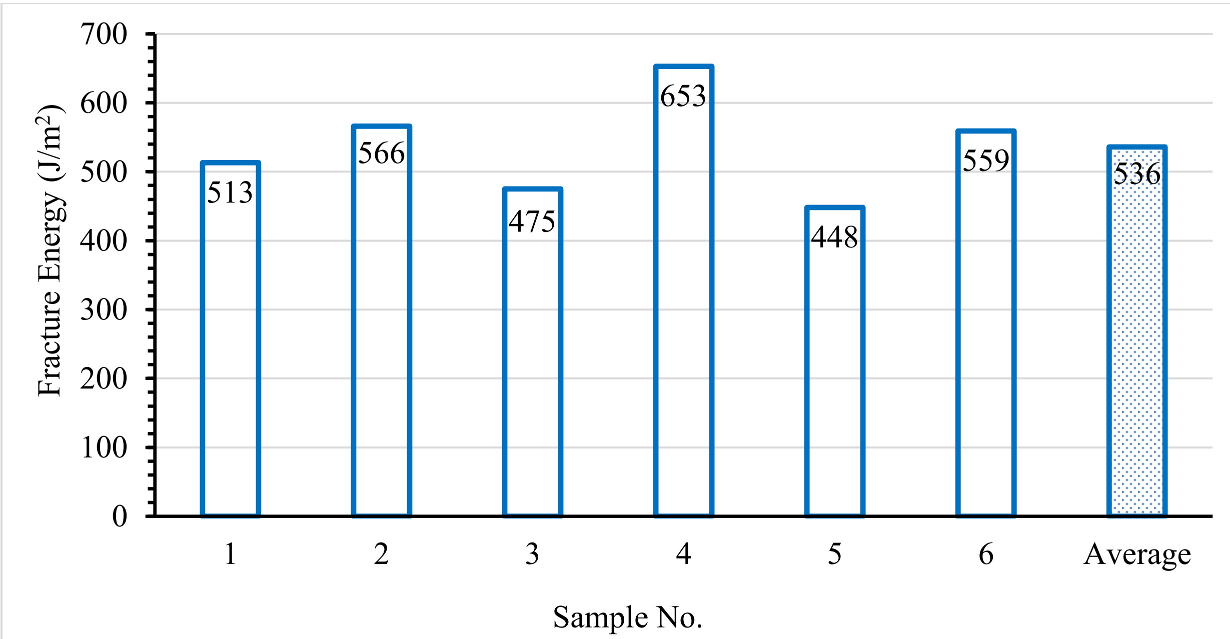


Figure 5-3 DCT Fracture energy (G_f) for the samples of Set 1 Mix with Binder-C: SPWEA330C (1)

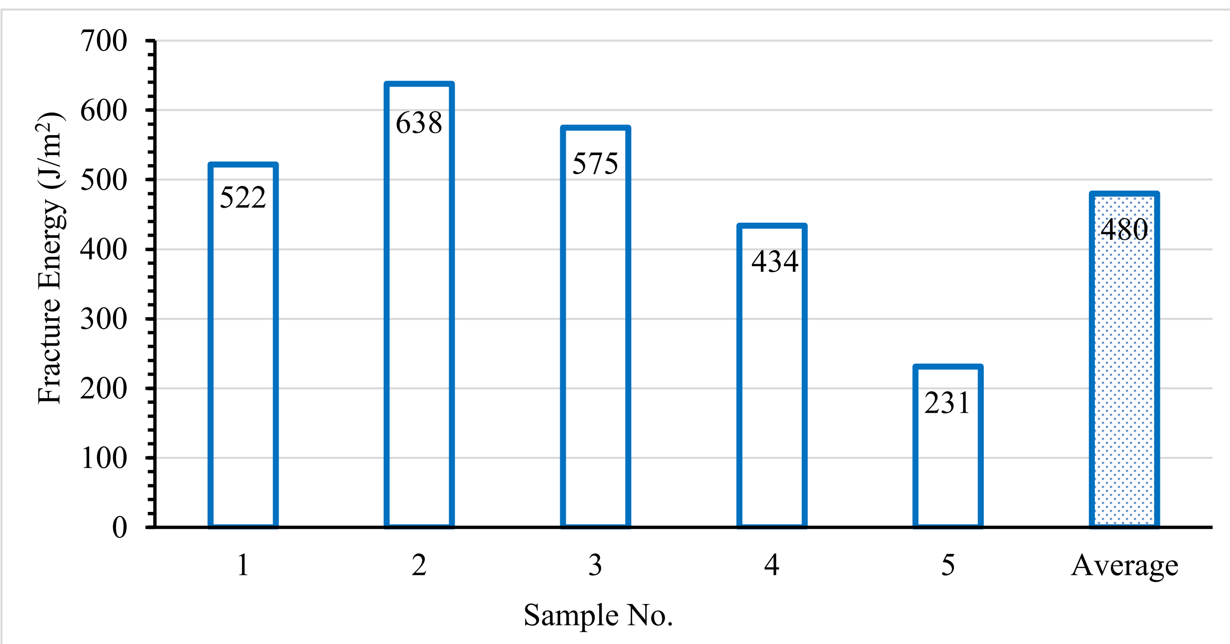


Figure 5-4 DCT Fracture energy (G_f) for the samples of Set 1 Mix with Binder-B: SPWEA330B (1)

5.3.2 Set 2

Figure 5-5 and Figure 5-6 show the values of G_f measured for the samples of Set 2. The average G_f for the binder-C mixes is 630 J/m^2 with a standard deviation of 145 J/m^2; whereas, the average G_f for the B-binder mixes is 503 J/m^2 with a standard deviation of 75 J/m^2. The average G_f of the binder-C mix is approximately 25% higher than the binder-B mix.

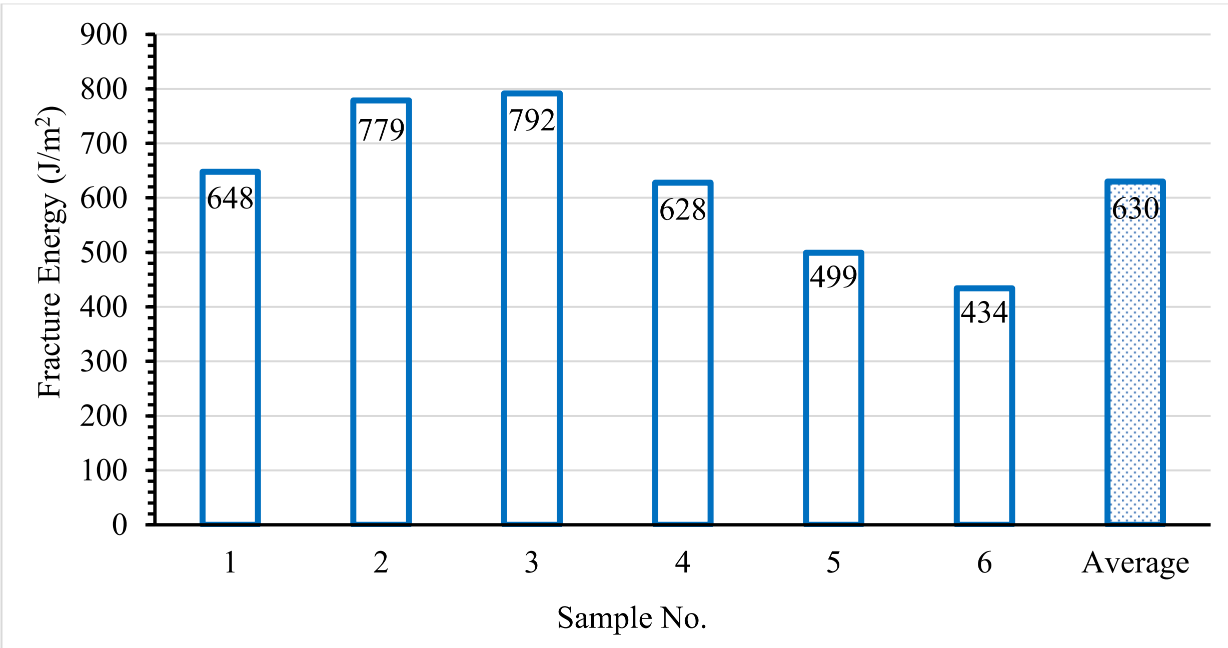


Figure 5-5 DCT Fracture energy (G_f) for the samples of Set 2 Mix with Binder-C: SPWEA330C (2)

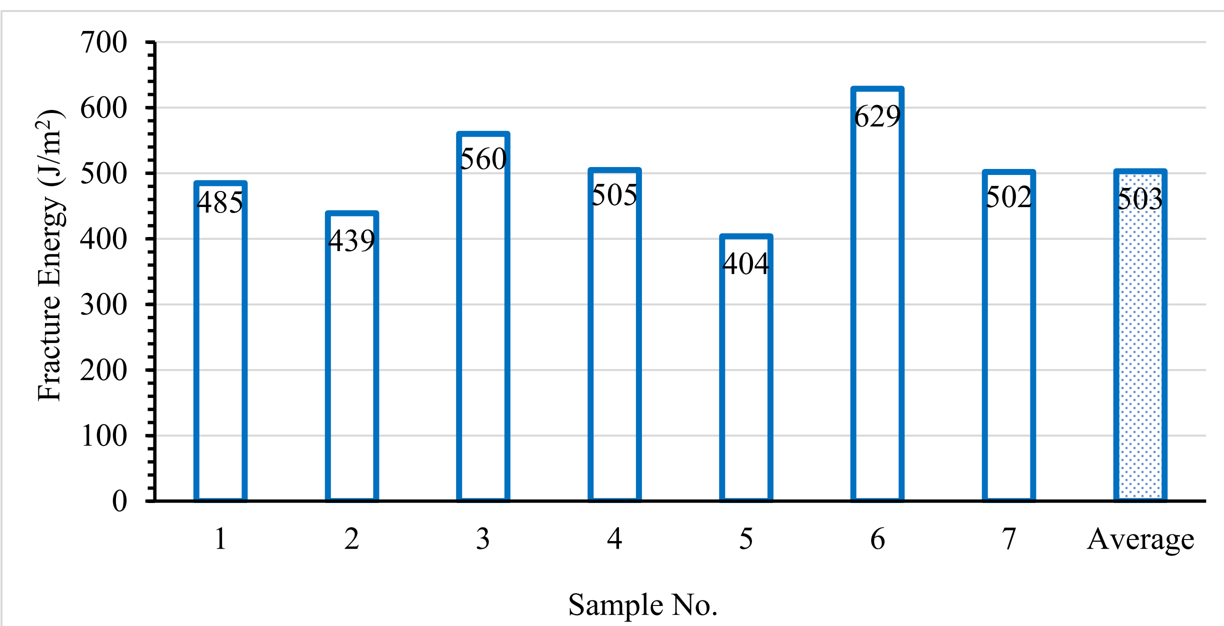


Figure 5-6 DCT Fracture energy (G_f) for the samples of Set 2 Mix with Binder-B: SPWEA330B (2)

5.3.3 Set 3

Figure 5-7 and Figure 5-8 present the G_f values measured for the samples of Set 3. The average fracture energy value for the binder-C mixes is 519 J/m^2 with a standard deviation of 94 J/m^2. The average G_f for the binder-B mixes is 448 J/m^2 with a standard deviation of 94 J/m^2. The average G_f of the binder-C mixes is approximately 16% higher than the binder-B mix.

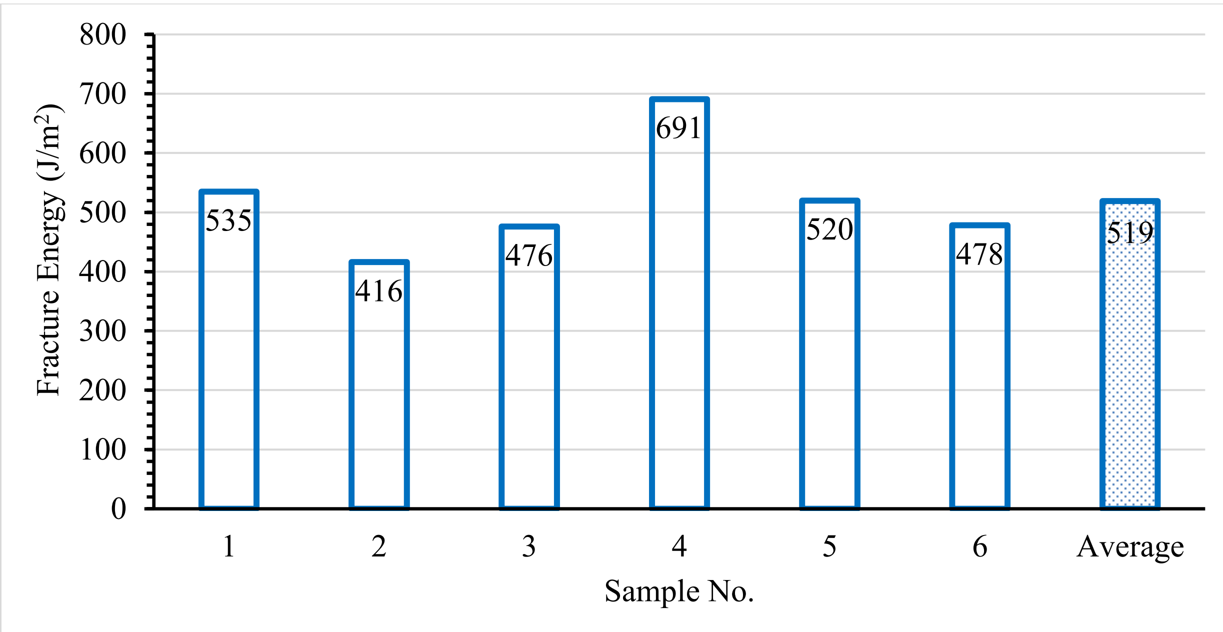


Figure 5-7 DCT Fracture energy (G_f) for the samples of Set 3 Mix with Binder-C: SPWEB440C

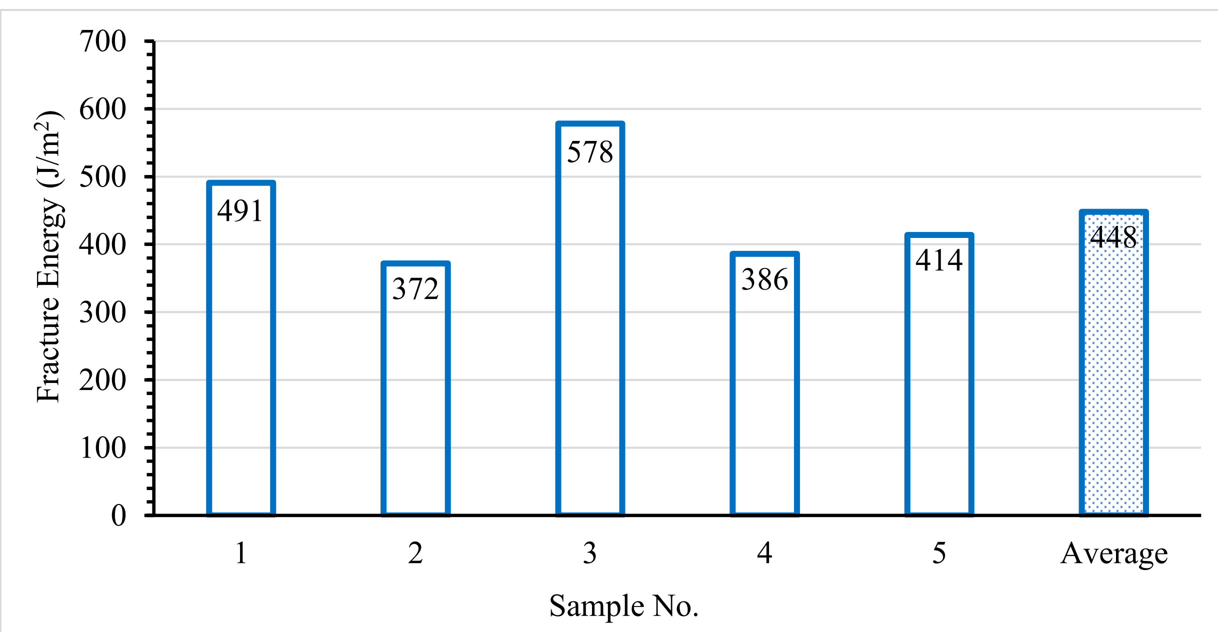


Figure 5-8 DCT Fracture energy (G_f) for the samples of Set 3 Mix with Binder-B: SPWEB440B

5.3.4 Set 4

Figure 5-9 and Figure 5-10 present the G_f values measured for the samples of Set 4. The average fracture energy value for the binder-C mix is 560 J/m^2 with a standard deviation of 80 J/m^2. The average G_f for the binder-B mix is 508 J/m^2 with a standard deviation of 49 J/m^2. The average G_f of the binder-C mix is approximately 10% higher than the binder-B mix. It may be recalled that the mixes in Set 4 were collected from MnROAD and the mix was prepared for traffic level 5.

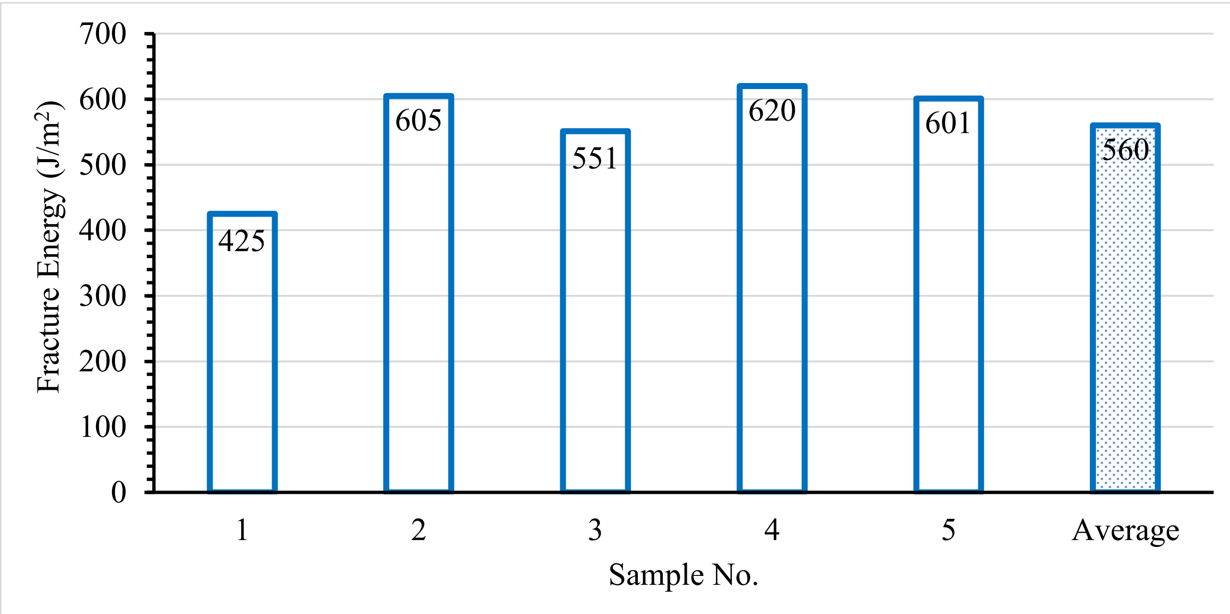


Figure 5-9 DCT Fracture energy (G_f) for the samples of Set 4 Mix with Binder-C: SPWEB540C

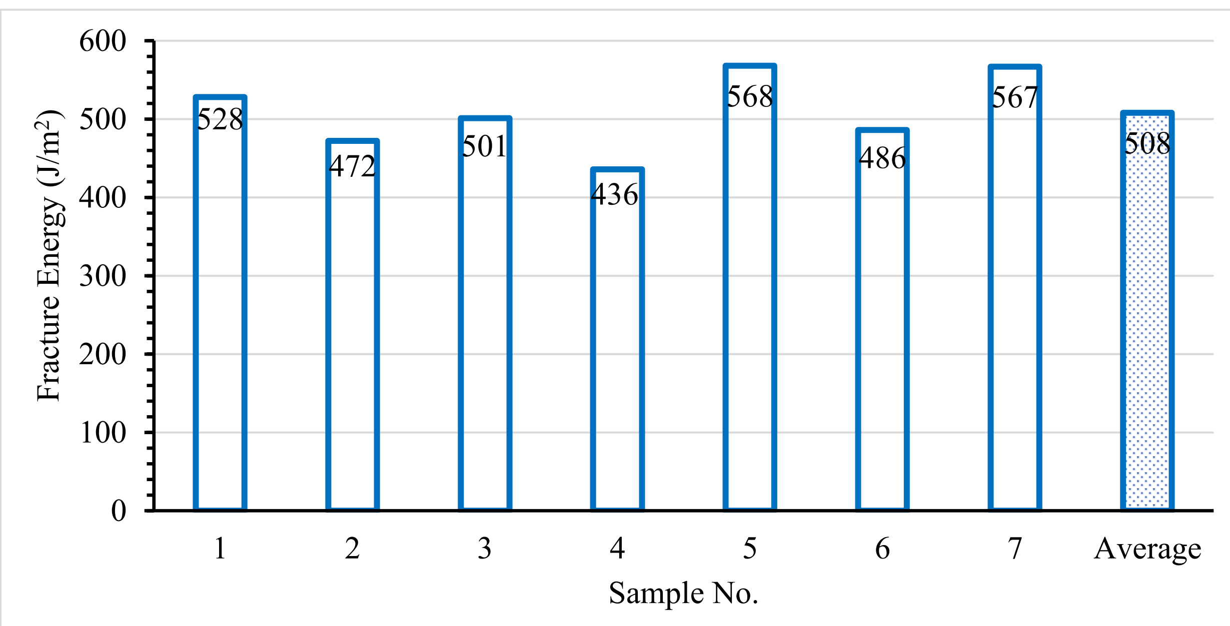


Figure 5-10 DCT Fracture energy (G_f) for the samples of Set 4 Mix with Binder-B SPWEB540B

5.3.5 Comparison of mechanical properties between the four sets of mixes

Figure 5-11 shows the comparison of all eight asphalt mixes tested at UMD. All four binder-C mixes exhibited higher G_f values than their binder-B counterparts. Set-2 mixes demonstrated the largest difference in the G_f values between the binder-C and binder-B mixes, approximately, 25%. It may be recalled the Set 2 mix is a traffic level 3 mix with aggregate gradation A, designed for 3% air voids.

Table 5.2 shows the results of the students' t-test for the G_f values of four different sets. The significance of the difference in the G_f values between the Binder-C and binder-B mixes was tested by the above-mentioned statistical test. It can be seen that only the Set 2 mixes showed a significant difference, with a p-value less than 0.05. The difference in the G_f values between the binder-C and binder-B mixes for the three other sets is statistically insignificant, with a p-value greater than 0.05.

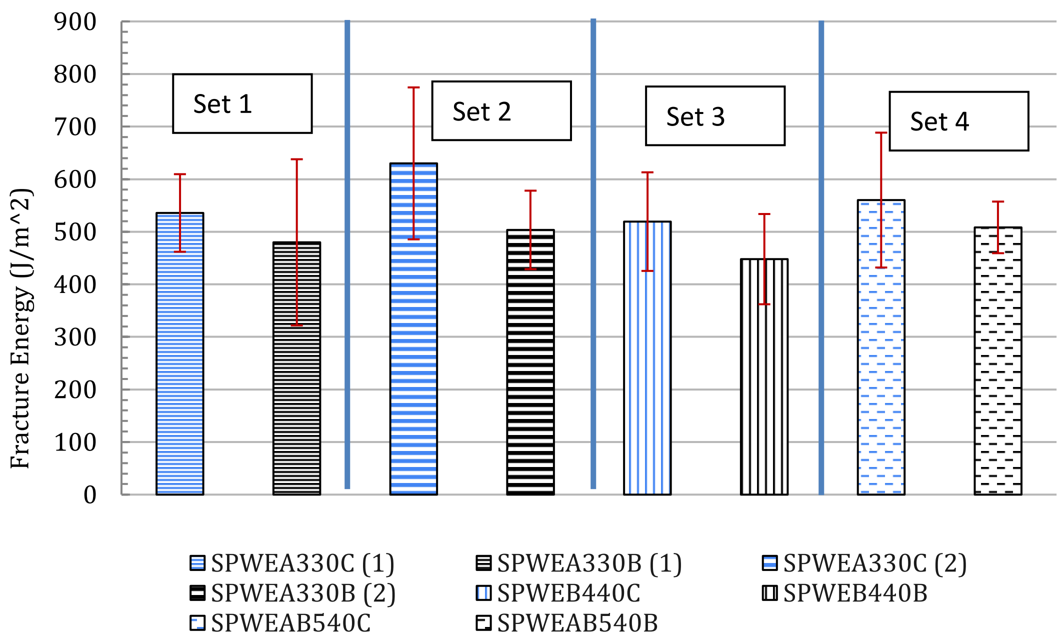


Figure 5-11 Comparison of Gf values of the eight mixes

Table 5.2 Student's paired t-test results

Set No.: Mix designation	Probability associated with a Student's paired t-Test, with a one-tailed distribution; unequal variance
Set 1: SPWEA330C and SPWEA330B	0.249
Set 2: SPWEA330C and SPWEA330C	0.046
Set 3: SPWEB440C and SPWEB440B	0.110
Set 4: SPWEAB540C and SPWEAB540B	0.12

5.4 OMRR Test Results

This section presents and discusses the fracture energy test results of the OMRR mixes. As mentioned before, 44 mixes for each binder-B and binder-C were considered in this analysis. Table 5.3 provides the number of mixes with respect to the traffic level. One disparity in this database is the significant differences in the number of mixes for a given traffic level. For traffic level 3, only 8 binder-C mixes are available, whereas for binder-B there are 38 mixes. For traffic level 4, there are 36 mixes for binder-C and 6 mixes for binder-B. Figure 5-12 presents the comparison of the G_f values of OMRR mixes. For traffic level 3, the average G_f value for binder-C mixes is 456 J/m^2, which is 48% larger than the G_f of the binder-B mixes, 310 J/m^2. The difference in the G_f values between the binder-B and binder-C mixes is 22% for the traffic level 4 mixes.

As the numbers of the mixes are inconsistent for a given traffic level, the weighted average of all binder-C mixes was compared with the weighted average of all the binder-B mixes. As shown in Table 5.3, the weighted average of all the binder-C mixes is 497 J/m^2 as compared to 324 J/m^2 for the binder-B mixes, which indicates that the binder-C mixes have approximately 53% larger DCT fracture energy.

Table 5.3 Statistics of OMRR mixes and their DCT fracture energy results

Binder type	Traffic level	DCT fracture energy results		
		Numbers of mixes	Average G_f (J/m^2)	Standard deviation (J/m^2)
Binder-C	3	8	456	50
	4	36	506	52
Binder-B	3	38	310	31
	4	6	414	57

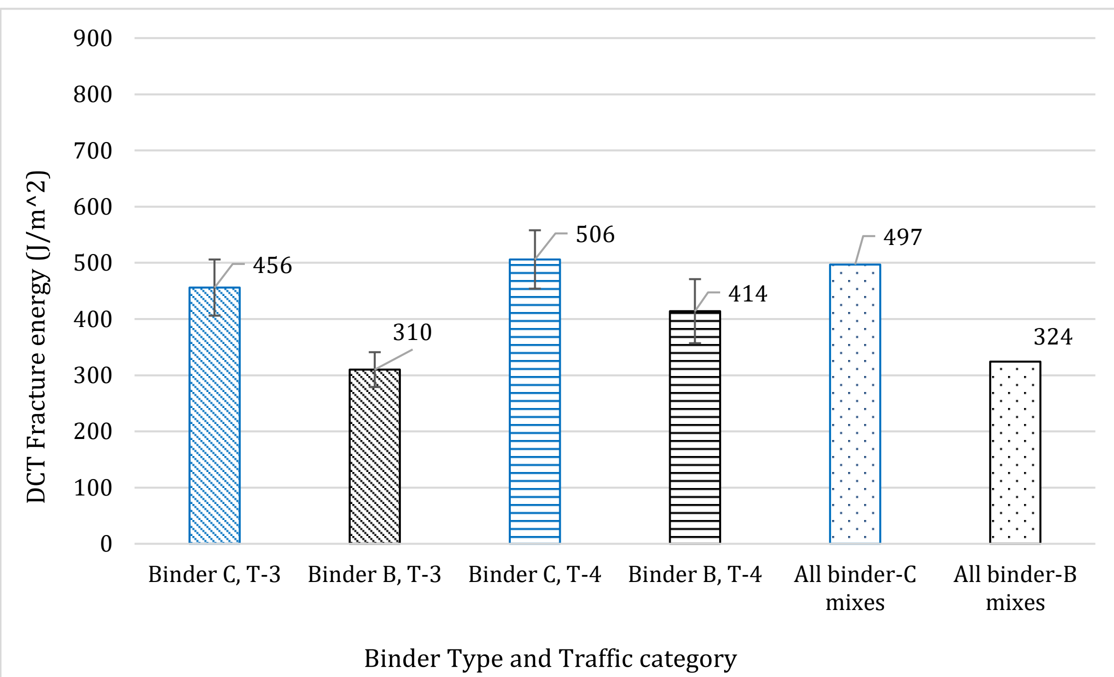


Figure 5-12 Comparison of Gf values of OMRR mixes

5.5 CONCLUSIONS

In this task, the fracture energies of the binder-B and binder-C mixes were compared. A total of 96 asphalt mixes were considered in the study. An equal number of the mixes were considered for binder-B and binder-C (48 mixes for each binder type). Out of 96 mixes, 8 mixes were tested at UMD and the test results of the other 88 mixes were collected from MnDOT's office of the Materials and Road Research (OMRR). Fracture energies were measured using the DCT test.

The DCT test results of the mixes tested at UMD showed that the fracture energy of mixes with PMB (binder-C) can be 10 to 25% higher than mixes with unmodified binder (binder-B). It may be noted that three out of the four mixes with the binder-B were lab-produced (with 2 hours of aging). Whereas, all the binder-C mixes are plant-produced. Therefore, the difference in the results may not be entirely because of the difference in the binder type.

The differences in the DCT test results for the OMRR mixes were relatively more apparent. Differences were noticed in the DCT values as a function of traffic level and binder type. Traffic level 3 mixes showed relatively less fracture energy than the traffic level 4 mixes. For traffic level 4, binder-C mixes showed 48% higher fracture energy than binder-B mixes. For traffic level 3, the difference in fracture energy is 22%. It may be noted that the number of mixes for a given traffic level was hugely different; therefore, the weighted averages of all the binder-C mixes and all binder-B mixes were compared. It was found that binder-C mixes can provide ~50% more fracture energy compared to binder-B mixes when the data of all the 88 OMMR mixes were used for the comparison, irrespective of the traffic level.

CHAPTER 6: BENEFIT-COST ANALYSIS FOR THE USE OF B AND C BINDERS

In this Chapter, a life-cycle cost analysis (LCCA) is performed to compare the cost-effectiveness of asphalt mixtures prepared with binders B and C (referred to as B and C mixtures in this report). First, different methods used to perform Benefit-Cost Analysis (BCA) are discussed, and previous studies on the benefit-cost analysis of polymer-modified binders are reviewed. Then, the results from the laboratory and field studies performed in previous tasks are summarized. Based on this information, an LCCA is performed to compare B and C mixtures.

6.1 METHODS FOR BENEFIT-COST ANALYSIS

Benefit-Cost Analysis (BCA) quantifies the economic implications to ensure the best utilization of available funds. The most used BCA method in pavement engineering is the life-cycle cost analysis (LCCA). Other BCA methods include the Benefit-Cost Ratio and Incremental Benefit-Cost Ratio (Papagiannakis and Masad 2008).

LCCA is used to evaluate the total cost of an investment option over its entire life (Walls and Smith 1998). A technique known as "discounting" is used in LCCA to convert all costs throughout the project's life cycle into present dollars. The Net Present Cost (NPC) is the summation of all those discounted costs. The NPC can be used for comparing competing design alternatives for the identical analysis period.

LCCA is used when the alternatives provide the same level of performance. In other words, the difference between the benefits of different alternatives is negligible. Consequently, the cost-effectiveness of alternatives can be compared by only comparing costs. Unlike LCCA, general BCA considers the benefits of an improvement as well as its costs and, therefore, can be used to compare design alternatives that do not yield similar benefits, as well as to compare projects that accomplish different objectives (road realignment versus widening project). Moreover, BCA can determine whether a project should be undertaken (i.e., whether the project's life-cycle benefits will exceed its life-cycle costs) (FHWA 2002). The elements typically included in the LCCA and BCA are compared in Table 6.1.

Table 6.1 Comparison between LCCA and BCA (FHWA 2002)

Project Element	LCCA	BCA
Agency construction, rehabilitation, and maintenance expenditures	Yes	Yes
User costs during construction, rehabilitation, or maintenance	Yes	Yes
User costs during normal operations	Yes	Yes
User benefits resulting from project	No	Yes
Externalities resulting from project	No	Yes

The BCA is typically performed based on the concept of the Benefit-Cost Ratio (BCR). The benefits and costs are translated into present worth and divided to calculate the BCR. It can be used to determine the feasibility of a single alternative or to compare two or more alternatives, whereby the alternative with the largest BCR over 1.00 is best (Papagiannakis and Masad 2008). The main difficulty in using BCR is the estimation of the benefit. One method commonly used by researchers is to calculate the area under the performance-time curve to represent the benefit (Peshkin et al. 2004, Dawson et al. 2011, Munch et al. 2021).

A modification to the conventional BCR method is Incremental Benefit-Cost Ratio (IBCR). It compares the difference in benefits divided by the difference in costs between the two alternatives. If the ratio is larger than 1.00, the alternative with a higher cost is better. The method can be used to compare more than two alternatives by arranging them in order of increasing capital cost and comparing them two at a time. The better of the two alternatives in the first paired comparison competes with the following alternative until the best overall alternative is established (Papagiannakis and Masad 2008). Although the IBCR is a more complex analysis than the BCR, it yields more reasonable selections of alternatives (Riggs and West 1986).

Because of its relative simplicity, LCCA analysis is often preferred over BCA for economic analyses in practice (FHWA 2002). Most State DOTs use LCCA to some degree in selecting the preferred pavement alternative for major projects (Rangaraju et al. 2008), although LCCA is not applied to all pavement projects. Various software tools are available to assist in the analysis, with the FHWA's RealCost (FHWA 2010) being the most prevalent. MnDOT has developed a spreadsheet to perform LCCA, which is also used in this study.

The economic analyses of pavements present many challenges, since it is difficult to accurately estimate the timing and costs of many activities (e.g., user costs, work zone safety, environmental impacts, and the impact of local development) (Hallin et al. 2011). Utility theory and other forms of value engineering have been used to solve this issue. Another difficulty is how to quantify the randomness in economic analysis. The traditional deterministic analysis fails to adequately account for either the variability in actual initial costs and discount rates over time or the uncertainty in the timing and costs of planned maintenance and rehabilitation activities. The probabilistic approach is more realistic in that it uses statistical descriptions of the probable distribution of values for each input to account for the input-associated variability that creates uncertainty in the outputs of the analysis, which helps quantify the risk in any decisions that are made based on the outputs. A distribution of output values is produced to provide users with information for understanding the variability of the results and the confidence that can be placed in the analysis (FHWA 2002).

6.2 LITERATURE REVIEW ON LIFE-CYCLE COST ANALYSIS OF POLYMER-MODIFIED BINDERS

Ponniah and Kennepohl (1996) studied the field performance and life-cycle costs of PMB used in Ontario, Canada. The results show that pavement sections that use PMB outperform those that use unmodified binders in terms of resistance to rutting and low-temperature cracking, provided that the PMB has a soft-grade (150 to 200 pen) base asphalt. Based on the life-cycle cost analysis, it has been demonstrated that using PMB is a more cost-effective option than unmodified binder for extending pavement life by 2 to 3 years, provided that the cost of PMB does not exceed twice that of unmodified binder.

Hicks and Epps (2000) studied the cost-effectiveness of implementing crumb rubber-modified asphalt binders in Arizona and California by the life-cycle cost analysis (LCCA). The LCCA considered the effect of traffic volume. The results show that in most of the application cases investigated in this study, crumb rubber modified binder is more cost-effective than unmodified binders, while it is not for some low-traffic volume roads. The authors pointed out that the key for LCCA is the long-term performance prediction of pavements and suggested that more research be performed on long-term performance prediction.

Asphalt Institute performed a series of studies comparing polymer-modified mixtures' performance and life-cycle costs. The performance data from 36 pavement sections in the United States (including both roadway and accelerated pavement test sections) were analyzed (Von Quintus et al. 2007). The results show that the use of polymer modification reduces the occurrence of distresses like rutting, fatigue cracking, and thermal cracking, which extended the pavement service lives by 5 to 10 years. Following the field performance analysis, a life-cycle cost analysis was performed (Buncher and Rosenberger 2009) comparing polymer-modified mixtures with unmodified mixtures. Based on the better field performance of polymer-modified mixtures, fewer rehabilitation activities are scheduled for polymer-modified mixtures. The LCCA results show that using polymer-modified mixtures is more cost-effective. The LCCA results of this study are shown in Table 6.2. The authors also noted that this LCCA analysis did not consider user costs, which would be reduced with the longer service lives of polymer-modified mixtures. Considering user delay costs in the analysis would make the benefit of using polymer-modified mixtures even more apparent.

Table 6.2 LCCA results of a study of Asphalt Institute (Buncher and Rosenberger 2009)

Scenario	Initial Cost, $	Initial Cost Increase, %	LCC	LCC Savings, %
1) All layers unmodified	669K	-	1005	-
2) PMA for Wearing (2”) Course	682K	2	941	6.5
3) Perpetual Pavement: PMA for Wearing (2”) and Base (4”) Courses	709K	6	849	15.5
4) More Conservative Approach: PMA for Wearing (2”) and Binder (2.5”) Courses with the same activity schedule as Scenario 2	698K	4.5	964	4.5
5) More Conservative Approach: PMA for Wearing (2”), Binder (2.5”) and Base (4”) Courses with same activity schedule as Scenario 3	725K	8.5	864	14

Archilla (2008) studied the cost-effectiveness of using PMB in Hawaii, considering the unique geographic isolation of Hawaii compared to the US mainland. Life-cycle cost analyses were performed. The results indicate that for heavy load situations, PMB is a viable and cost-effective alternative. An Average Annual

Daily Truck Traffic (AADT) of 1,400 has been found as a possible threshold for using PMB mixtures. This study recommended the Hawaii DOT consider a policy where PMB mixtures are one of the main alternatives for heavy loading situations because, without a certain demand, it is unlikely that the industry would make the necessary investment for implementing PMB.

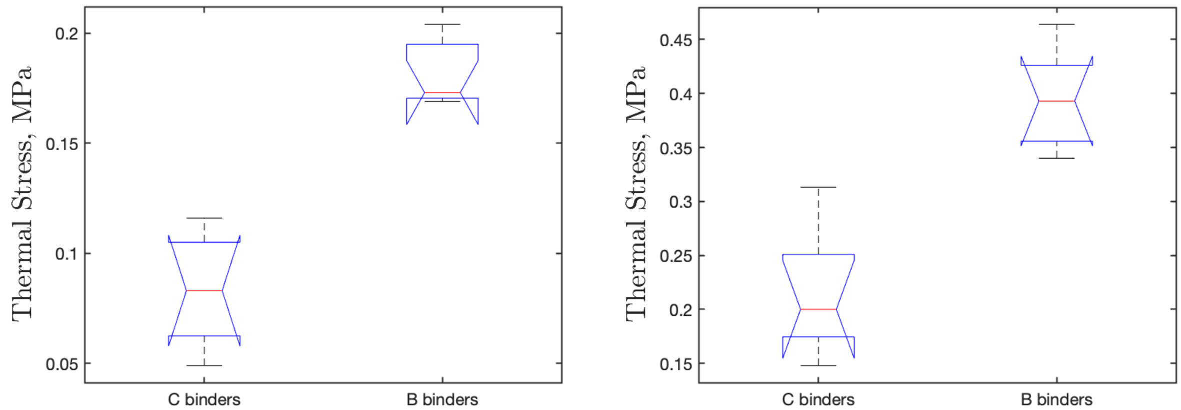
Lee and Kim (2010) investigated the cost-effectiveness of using polymer modification for chip seal. Laboratory tests on rutting, bleeding, and aggregate retention were performed. The results show that polymer-modified chip seal was helpful against the above-mentioned distresses, which tend to extend the pavement's service life. LCCA results show that polymer-modified chip seals would be a cost-effective solution if the polymer modification can extend the service life of the chip seal from 5 years to more than 7 years, which, according to the authors, is highly possible given their laboratory experimental results.

Souliman et al. (2016) investigated the cost-effectiveness of PMB in improving fatigue performance. A polymer-modified mixture and an unmodified mixture used on a high-traffic volume road in Sweden were compared. The flexural bending beam fatigue test was performed to evaluate the fatigue behavior of mixtures. Then, based on the beam fatigue test results, the mechanistic-empirical analysis was applied to estimate the fatigue life of pavements. The cost-effectiveness was computed as the ratio between the fatigue life and total cost. The results showed that although the polymer modification increases the cost of the material, it is more cost-effective in the long term than the unmodified mixture.

6.3 EFFECT OF BINDER GRADE ON LABORATORY AND FIELD CRACKING PERFORMANCE

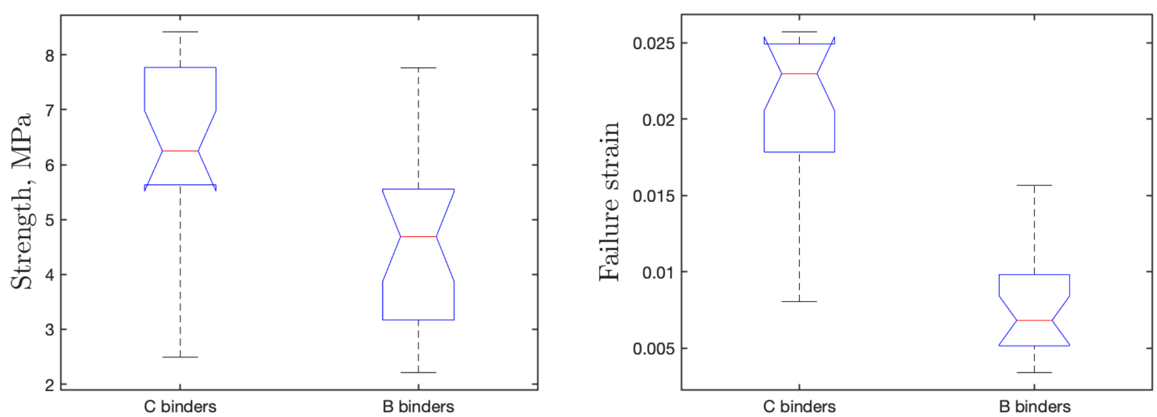
6.3.1 Laboratory Performance of Binders

In Task 4, the laboratory low-temperature performance of B and C binders were compared based on the BBR creep and strength test. In addition to the difference in low-temperature PG grade (-34°C for C binders and -28°C for B binders), the results show that C binders have lower thermal stresses and higher strength and failure strain than B binders, as shown in Figure 4.9 and Figure 4.11. Detailed data was included in Task 4 report. Based on the laboratory results, C binders are expected to have a better low-temperature cracking resistance in the field than B binders.



(a) (b)

Figure 6.1 Box plot comparison of thermal stress between C and B binders, (a) -28 °C, (b) -34 °C.



(a) (b)

Figure 6.2 Box plots of the comparison between C and B binders, (a) strength, (b) failure strain.

6.3.2 Laboratory Performance of Mixtures

In Task 5, the research group at University of Minnesota Duluth (UMD) performed DCT tests to compare the laboratory low-temperature cracking performance of B and C mixtures. In addition, the DCT test data (obtained from 2017 to 2019) at MnDOT Office of Materials and Road Research (OMRR) are analyzed to compare B and C mixtures. The DCT fracture energy results of OMRR and UMD are summarized in Table 6.3.

Table 6.3 DCT fracture energy (J/m^2) results of OMRR and UMD

		OMRR Data summary			UMD Data Summary			OMRR + UMD
	Traffic level	Number of mixes	Average	Std dev	Number of mixes	Average	Std dev	Average
Binder B	2	1	327	-	-	-	-	338
	3	38	310	31	2	491	-	
	4	6	414	57	1	447	-	
	5	2	445	18	1	508	-	
	All	44	324	-	4	484	-	
Binder C	2	-	-	-	-	-	-	502
	3	8	456	50	2	583	-	
	4	36	506	52	1	519	-	
	5	-	-	-	1	560	-	
	All	44	497	-	4	561	-	

The OMRR fracture energy data are shown in Figure 6.3. It is seen that C mixtures have significantly higher fracture energy than B mixtures. Moreover, it is seen that the traffic level of mixtures also affects fracture energy. More specifically, mixtures of traffic level 4 have higher fracture energy than that of traffic level 3. This is probably because higher traffic level mixtures use better quality aggregates (e.g., higher strength, hardness, and angularity) than lower traffic level mixtures.

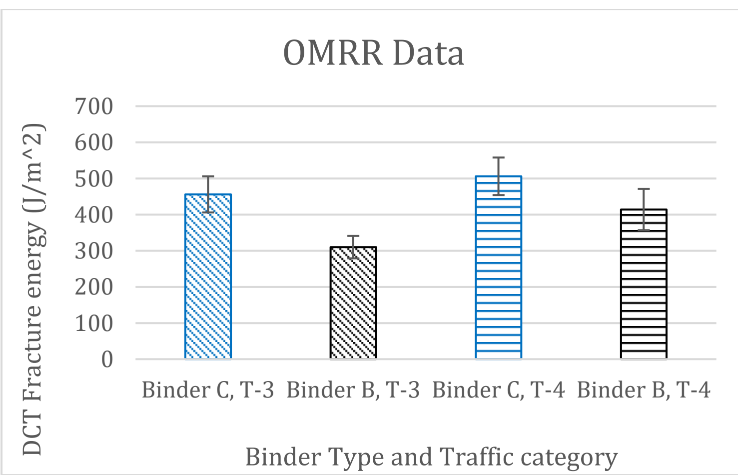


Figure 6.3 Fracture energy data of OMRR

The UMD fracture energy data are shown in Figure 6.4. Similar to OMRR data, it is seen that for each traffic level, C mixtures always have higher fracture energy than B mixtures. The effect of traffic level on fracture energy is not clear from the UMD data.

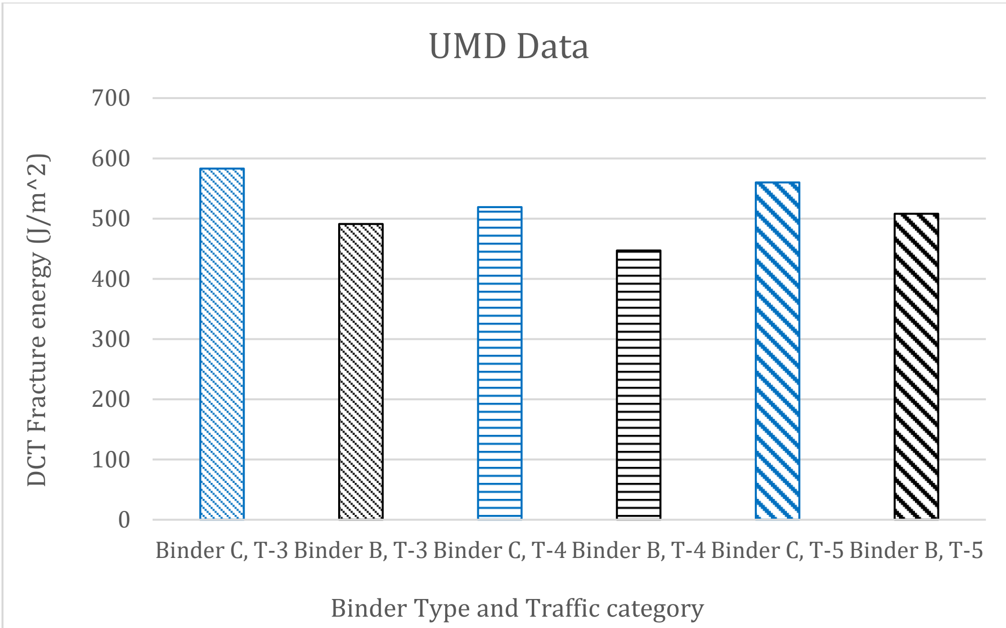


Figure 6.4 Fracture energy data of UMD

As shown in Table 6.3, considering the data of both OMRR and UMD, the overall average fracture energy for B and C mixtures are 338 and 502 J/m^2, respectively. These values will be used for estimating the field cracking performance of B and C mixtures and their rehabilitation schedules in section 6.4.2.

6.3.3 Field Performance

Task 3 has analyzed the effect of binder grade (B or C) on the Ride Quality Index (RQI) and Surface Rating (SR). The main conclusions are: (1) for new construction projects, C binder projects have higher smoothness (RQI) than B binder projects; (2) however, for overlay projects, B binder projects have higher smoothness (RQI) than C binder projects; (3) binder grades do not show a significant effect on pavement surface distresses (SR).

Previous studies have also investigated the correlation between asphalt material properties and field cracking performance. In a national pooled funded study on low-temperature cracking of asphalt pavements (Marasteanu et al. 2012), a relationship between SCB fracture energy and transverse cracking was identified, as shown in Figure 6.5. The severity of transverse cracking increases with the decrease in fracture energy in an exponential manner, and a limiting value of 350 J/m^2 was proposed for SCB fracture energy. The value was adjusted to 400 J/m^2 to account for aging effects.

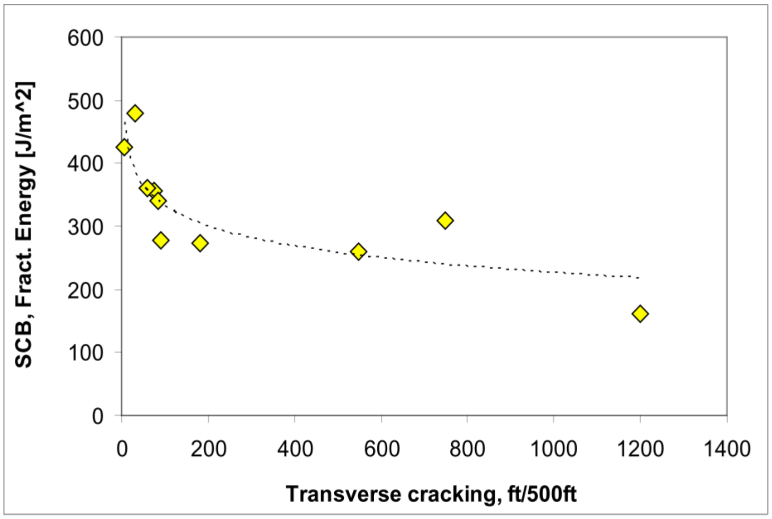


Figure 6.5 Relationship between SCB fracture energy and transverse cracking (Marasteanu et al. 2012)

In a previous MnDOT research project, Dave et al. (2015) studied the relationship between fracture energy and field cracking by DCT test. The results show that the asphalt mixtures with higher DCT fracture energy corresponded with pavements with a lower amount of transverse cracking. Figure 6.6 shows the relationship between the DCT fracture energy of field cores and the rate of transverse cracking of overlay projects (Oshone et al. 2019).

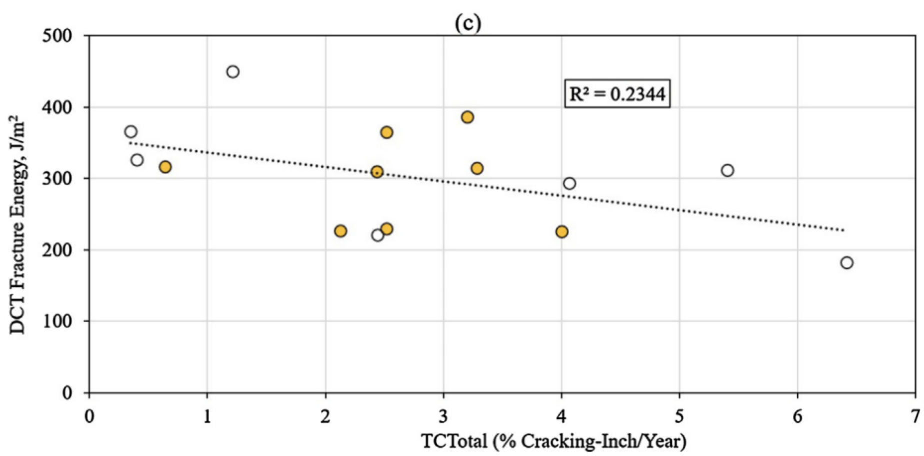


Figure 6.6 Relationship between DCT fracture energy and the rate of transverse cracking (Oshone et al. 2019)

The hollow symbols represent overlays on asphalt and concrete pavements, while the filled symbols represent mill and overlay on asphalt pavements. The x-axis is the total transverse cracking performance index (TCTotal), which is a representation of the rate of transverse cracking and is defined as:

$$TCTotal = \frac{Transverse\ cracking\ work}{Life\ at\ latest\ survey^2} \qquad \text{Eq. 6.1}$$

Where the transverse cracking work = Area under the transverse cracking and time curve, % cracking*year. An example of TCTotal calculation is shown in Figure 6.7.

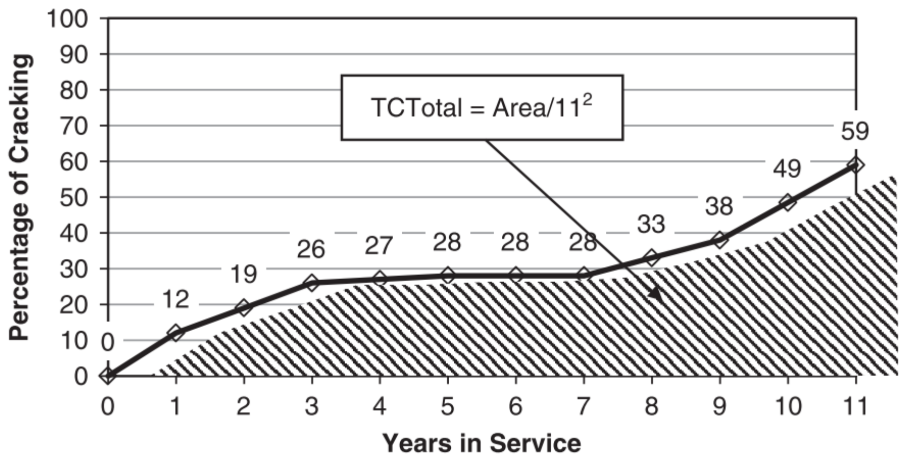


Figure 6.7 Schematic diagram of TCTotal calculation (Dave et al. 2016).

The linear relationship shown in Figure 6.6 provides a way to estimate the field transverse cracking performance based on laboratory fracture test results, which will be used in Section 6.4.2 for estimating the rehabilitation schedules of the design alternatives.

6.4 LIFE-CYCLE COST ANALYSIS FOR THE USE OF B AND C BINDERS

Based on the performance of B and C mixtures (binders) introduced in the previous sections, a life-cycle cost analysis (LCCA) is performed to compare the cost-effectiveness of B and C mixtures (binders) for being used in the wearing courses.

6.4.1 Alternatives and Construction Cost

A new construction project of low volume road is considered for LCCA. It is assumed that it is 12 feet wide, and its structure from bottom to top is 12" granular embankment, 10" aggregate base, and 4" wearing course. Based on the same pavement structure, we designed two alternatives, Alt-1 and Alt-2. The difference between them is that the Alt-1 uses C mixtures in the wearing course while the Alt-2 uses B mixture in the wearing course. The structures and construction costs of the two alternatives are listed in Table 6.4 and Table 6.5. The unit costs were obtained from the MnDOT LCCA spreadsheet (2021-2022) for District-2. Because the C mixture is more expensive ($84.2) than the B mixtures ($80.3), Alt-1 has a higher initial construction cost than Alt-2.

Table 6.4 Structure and construction cost of Alt-1, C mixture as wearing course

Layers	Depth (in)	Width (ft)	Quantity per mile*	Unit	Unit Price ($)	Total Price ($)
Subgrade Preparation			53	-	285.79	15,146.9
Select Granular Embankment MOD 7% (CV)	12	12	2346.7	CY	25.76	60,450.2
Aggregate Base (CV) Class 5	10	12	1955.6	CY	35.23	68,895.8
Wearing Course Mixture (9.5, C)	4	12	1591	TON	84.20	133,962.2
Total						**278,453.5**

Table 6.5 Structure and construction cost of Alt-2, B mixture as wearing course

Layers	Depth (in)	Width (ft)	Quantity per mile*	Unit	Unit Price ($)	Total Price ($)
Subgrade Preparation			53	-	285.79	15,146.9
Select Granular Embankment MOD 7% (CV)	12	12	2346.7	CY	25.76	60,450.2
Aggregate Base (CV) Class 5	10	12	1955.6	CY	35.23	68,895.8
Wearing Course Mixture (9.5, B)	4	12	1591	TON	80.30	127,757.3
Total						**272,250.1**

6.4.2 Rehabilitation Activities and Timing

According to Section 6.3, C mixtures (binders) have higher low-temperature cracking resistance than B mixtures, which would lead to different timing of rehabilitation for Alt-1 and Alt-2.

For Alt-1 (new constructions using C mixtures) MnDOT Pavement Design Manual (2019) (in Section 770) has suggested the rehabilitation activities and timing, assuming a service life of 20 years, as shown in

Table 6.6.

Table 6.6 Rehabilitation activities and schedule for new constructions using C mixtures (Alt-1) recommended by MnDOT (2019).

Age	Mainline Treatment	Mainline Quantity
0	Initial Construction	Initial Construction
8	Crack Treatment	16% Mainline Length

12	Chip Seal + Microsurfacing	40% Mainline Length
20	Mill Top lift + ½" & Overlay Mill Thickness +1.5"	100% Mainline Area
23	Crack Treatment	32% Mainline Length
27	Chip Seal	31% Mainline Length
35	End of 35-Year Analysis Period	2/17 Remaining Service Life

It is seen that a crack treatment is needed when 16% of the mainline is cracked at the age of 8 years. According to Eq. 6.1 and Figure 6.7, and assuming the crack accumulation is linear, this is equivalent to a cracking rate (TCTocal) of 1 %/yr, which is calculated as follows:

$$TCTotal = \frac{Transveser\ crack\ work}{Number\ of\ years^2} = \frac{0.5*16\%*8yr}{8^2\ yr^2} = 1\%/yr \qquad \text{Eq.6.2}$$

According to Figure 6.6, Figure 6.6a TCTotal of 1%/yr corresponds to a fracture energy of around 350 J/m^2. However, this value is lower than the average fracture energy for C mixtures obtained from laboratory testing (502 J/m^2) as seen in Section 3.3.2. It is believed that this inconsistency is due to the aging effect (Marasteanu et al. 2012, Braham et al. 2009). To reconcile this inconsistency, a reduction factor of 30% is introduced to convert the fracture energy of laboratory samples to that of field cores considering the aging effect. Considering this aging factor, the fracture energy values of B and C mixtures are reduced to 237 and 350 J/m^2, respectively. According to Figure 6.6, the TCTotal for B mixtures is estimated as 6%/yr. Assuming a linear accumulation of cracking, Alt-2 (B mixtures) would last for 1.3 years until they reach 16% of field cracking and need a crack treatment, which is more than 6 years earlier than Alt-1 (C mixtures). Based on this rough estimation, we assume that the Alt-2 has a service life of 14 years, and all activities are advanced for 6 years compared with the Alt-1. The detailed rehabilitation activities and schedule are shown in Table 6.7.

Table 6.7 Rehabilitation activities and schedule for new constructions using B mixtures (Alt-2).

Age	Mainline Treatment	Mainline Quantity
0	Initial Construction	Initial Construction
2	Crack Treatment	16% Mainline Length
6	Chip Seal + Microsurfacing	40% Mainline Length
14	Mill Top lift + ½" & Overlay Mill Thickness +1.5"	100% Mainline Area
16	Crack Treatment	32% Mainline Length
20	Chip Seal	31% Mainline Length
28	Mill Top lift + ½" & Overlay Mill Thickness +1.5"	100% Mainline Area
30	Crack Treatment	32% Mainline Length
34	Chip Seal	31% Mainline Length
35	End of 35-Year Analysis Period	7/14 Remaining Service Life

The costs of the maintenance and rehabilitation activities are obtained from the MnDOT LCCA spreadsheet (2021-2022) for District-2, which are listed in Table 6.8

Table 6.8 Costs of rehabilitation activities

Activities	Depth (in)	Width (ft)	Quantity per mile*	Unit	Unit Price ($)	Total Price ($)
Crack Treatment		12	7040	SY	0.46	3256
Chip Seal		12	7040	SY	1.48	10441
Microsurfacing		12	7040	SY	5.33	37517
Mill (3")		12	7040	SY	2.77	19512
Overlay	3	12	1193	TON	80.3	95826

6.4.3 Life-Cycle Cost

The calculation of the life-cycle costs is based on the Net Present Value (NPV) concept, which considers the discount rate i. For a cost C occurred at year n, its NPV is:

$$NPV = \frac{C}{(1+i)^n} \qquad \text{Eq. 6.3}$$

The total NPV is the summary of the NPV of each activity.

At the end of the analysis period, the remaining service life value (RSL) needs to be considered as a negative cost in the LCCA. The RSL can be calculated as:

$$RSL = C_{last} \times \frac{N_{RL}}{N_{SL}} \quad \text{Eq. 6.4}$$

C_{last} = cost of the last rehabilitation or reconstruction activity, e.g., an overlay. N_{RL} = unused service life, in years, of the last activity at the end of the analysis period. N_{SL} = service life of the last activity in years.

Based on the initial construction cost (Section 6.4.1) and the rehabilitation schedule and cost (Section 6.4.2), the life-cycle cost analysis (LCCA) is performed and detailed in Table 6.9. The analysis period is chosen as 35 years, and the discount rate is chosen as 0.66% according to the MnDOT LCCA spreadsheet (2021-2022) for District-2.

The result shows that the life-cycle cost of Alt-1 (C mixture) is about 14.4% lower than that of Alt-2 (B mixtures). Therefore, using the C mixture in wearing courses is more cost-effective than using B mixtures. It should be noted that this analysis does not consider user costs, such as the delay costs due to road repair, vehicle repair costs, etc. Alt-1 would have lower user costs because of its lower maintenance need. Considering user costs would make using C mixtures even more cost-effective.

Table 6.9 Life-cycle cost of the alternatives, analysis period = 35 years

Alt-1, C mixture, design life = 20 years				Alt-2, B mixture, design life = 15 years			
Activities	**Age**	**Cost ($)**	**NPV ($)**	**Activities**	**Age**	**Cost ($)**	**NPV ($)**
Initial Construction	0	278,454	278,454	**Initial Construction**	0	272,250	272,250
Crack Treatment	8	3,256	3,089	**Crack Treatment**	2	3,256	3,214
Chip Seal + Microsurfacing	12	47,958	44,318	**Chip Seal + Microsurfacing**	6	47,958	46,102
Mill and Overlay 3”	20	115,338	101,119	**Mill and Overlay 3”**	14	115,338	105,190
Crack Treatment	23	3,256	2,799	**Crack Treatment**	16	3,256	2,931
Chip Seal	27	10,441	8,742	**Chip Seal**	20	10,441	9,154
End of Analysis period (2/17 Remaining Life)	35	13,569	10,779	**Mill and Overlay 3”**	28	115,338	95,935
				Crack Treatment	30	3,256	2,673
				Chip Seal	34	10,441	8,349
				End of Analysis period (7/14 Remaining Life)	35	57,669	45,809
Life-Cycle Cost			**427,742**	**Life-Cycle Cost**			**499,989**

6.5 CONCLUSIONS

In this task, the previous studies on the benefit-cost analysis and the cost-effectiveness of polymer-modified binders are reviewed. Then, the laboratory and field performance of B and C mixtures are summarized based on previous tasks and relevant studies. Based on this information, an LCCA is performed comparing B and C mixtures. The main conclusions are summarized below:

1. Life-cycle cost analysis (LCCA) is the most feasible and commonly used economics analysis method in pavement engineering for comparing design alternatives. Therefore, an LCCA is performed in this study to compare the use of B and C mixtures.
2. Previous studies, as well as the experimental results in the previous tasks of this study, show that polymer-modified (C) mixtures (binders), in general, have better mechanical properties and are also more cost-effective than unmodified (B) mixtures (binders).
3. Based on the literature review comparing B and C mixtures, the experimental data of the previous tasks, and the recommendations of the MnDOT pavement design manual, it is assumed that using B mixtures for wearing courses of new construction would lead to a 14 years' service life which is 6 years shorter than using C mixtures. The maintenance and rehabilitation schedule would also be advanced by 6 years for B mixtures compared with C mixtures.
4. The LCCA result shows that, for new construction projects, using C mixtures can save about 14.4% in the total cost compared to using B mixtures. The user costs were not considered in the LCCA. However, if the user costs were considered, using C mixtures would be even more cost-effective than B mixtures.

CHAPTER 7: SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

In this study, the low-temperature cracking performance of PG 58H-34 PMBs (C binders) and PG58S-28 unmodified binders (B binders) was analyzed based on field performance data and laboratory experiments. Based on this information, a LCCA was performed to compare the cost-effectiveness of using PMBs and unmodified binders for new construction of low-volume roads in Minnesota. The main conclusions are summarized below:

1. The field performance data showed that, in general, new pavements constructed with C binders outperform the pavements constructed with B binders.
2. The laboratory experimental results showed that polymer-modified binders and mixtures have better low-temperature cracking resistance than unmodified binders and mixtures.
3. Based on the fracture energy test results of asphalt mixtures, and the correlation between the fracture energy and field cracking accumulation rate, it is estimated that using polymer-modified binders can extend the service life by 6 years compared with projects using unmodified binders.
4. The LCCA result showed that, for low-traffic roads in Minnesota, using C binders can save about 14.4% in total costs compared to using B binders. If user costs are considered, using C binders would be even more cost-effective than B binders, since less maintenance and rehabilitation activities are necessary.

This study also showed that a key component necessary to perform an accurate economic analysis in pavement engineering is the prediction of field performance, which generally requires bridging the gap between laboratory performance and field performance of mixtures.

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