GEORGIA DOT RESEARCH PROJECT 14-06

FINAL REPORT

AN ENHANCED GDOT PAVEMENT PRESERVATION GUIDE WITH OPTIMAL TIMING OF PAVEMENT PRESERVATION

Georgia Department of Transportation

OFFICE OF PERFORMANCE-BASED MANAGEMENT AND RESEARCH

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16. Abstract:

The objectives of this research are 1) to develop the first version of the comprehensive pavement preservation guide, covering project selection, specification, material selection, construction procedure, QA/QC, along with the web-based pavement preservation interactive tool (PPIT) that enables GDOT to cost-effectively preserve its pavements; and 2) to conduct field tests and long-term performance monitoring using 3D technology to study the effectiveness of crack sealing and fog seal in support of GDOT's pavement preservation criteria and policies. This research project developed the first version of the comprehensive pavement preservation guide along with the web-based pavement preservation interactive tool (PPIT). Based on crack sealing tests in Hawkinsville, southern GA, and Covington, central GA, crack sealing can effectively retard crack growth by 40% – 128% on crack sealed test sections, which demonstrates very good crack sealing effectiveness. Results show that the crack sealing effectiveness is good for higher pretreatment COPACES rating and supports GDOT's new policy of increasing COPACES ratings for its crack sealing application method. Based on fog seal tests on I-475 with different raveling conditions (very light, light, medium, and severe with a loss of aggregates ranging from 0 – 20% under GDOT's Severity Level 1), the skid number (SN) decreases about 45% immediately after the fog seal application and recovers back to the requirement level (SN = 30) in a week. The fog seal treatment is promising in reducing the rate of aggregate loss for medium and severe raveling conditions (under GDOT's raveling Severity Level 1 with loss of aggregate around 10% – 20%) by 30% – 90% and 40% – 70%, respectively.

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GDOT Research Project No. 14-06

Final Report

AN ENHANCED GDOT PAVEMENT PRESERVATION GUIDE WITH OPTIMAL TIMING OF PAVEMENT PRESERVATION

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
		ILLUMINATION		
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
^oC	Celsius	1.8C+32	Fahrenheit	^oF
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

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EXECUTIVE SUMMARY

The experienced pavement experts in the Georgia Department of Transportation (GDOT) possess a great amount of tacit knowledge regarding pavement preservation, but before those experts retire, there is an urgent need to collect and document their knowledge. In addition, other state Department of Transportations' (DOT) experience and knowledge of pavement preservation techniques and their treatment criteria and performance need to be studied. As GDOT increases the number of its pavement preservation outsourcing projects, a pavement preservation guide containing comprehensive and consistent information about project selection, specification, material selection, construction procedure, and Quality Assurance (QA)/Quality Control(QC) is essential and is urgently needed for GDOT to perform cost-effective pavement maintenance (right treatment, right time, and right location) and achieve the best pavement performance for the least amount of money while meeting Moving Ahead for Progress in the 21st Century Act's (MAP-21) requirements.

The objectives of this research are to develop the first version of the comprehensive pavement preservation guide, covering project selection, specification, material selection, construction procedure, QA/QC, along with the web-based pavement preservation interactive tool (PPIT) that enables GDOT to cost-effectively preserve its pavements and to conduct field tests and long-term performance monitoring using 3D technology to study the two most commonly implemented pavement preservation methods (i.e. crack sealing and fog seal), identified by GDOT. This information will be used to evaluate the feasibility of applying fog seal to high traffic volume interstate highways and to study crack sealing effectiveness and adequate treatment timing in support of GDOT's pavement preservation criteria and policies.

CONCLUSIONS

The research findings of this study are summarized in the three parts below:

Part 1: Development of an Interactive Pavement Preservation Guide

This research project developed the first version of the comprehensive pavement preservation guide along with the web-based pavement preservation interactive tool (PPIT) that can be used by GDOT's Office of Maintenance and seven different district offices at any time (24 hours a day and 7 days a week) and in any place. Different asphalt and concrete pavement preservation methods have been incorporated into the PPIT and are presented in this report. For a detailed introduction and tutorial for PPIT, please see APPENDIX A. For asphalt pavements, this report presents chip seal, crack seal, fog seal, micro-milling, micro-surfacing, thin overlay, white topping, pothole patching, open-graded interlayer, full-depth reclamation, hot in-place recycling, cold in-place recycling, and ultra-thin bounded wearing course. For concrete pavements, this report presents diamond grinding, dowel bar retrofit, full-depth repair, joint sealing, and partial depth repair. For each pavement preservation method, this report presents the treatment method, treatment selection criteria, material design, construction, corresponding considerations, and performance and limitations.

Part 2: Crack Sealing Field Test Study

The second part of this report presents the field test of crack seal. Nine selected field test sites with different locations (six in Hawkinsville, southern GA; three in Covington, central GA), different pretreatment pavement conditions (COPACES ratings of 66-98) and different roadway environment factors (thicknesses 2.5-10 inch; AADT 930-12,200; truck percentage 4%-26%)

have been monitored quarterly using 3D technology over the 3 past years right after the crack sealing application. To quantitatively compute the crack sealing effectiveness, each site has both crack sealed test sections and non-sealed control sections to compare the differences in crack length growth. Based on detailed analysis results of 3 years of data on nine test sites, the following findings are presented:

- Crack sealing can effectively retard crack growth by 40%-128% on crack sealed test sections compared to the non-sealed control sections, which demonstrates very good crack sealing effectiveness.

- A higher pretreatment COPACES rating pavement provides better crack sealing effectiveness based on a breakdown study with 3 clusters of pretreatment COPACES ratings (66-69, 79-85, 93-98) having a 40%-48%, 52%-72%, 82%-124% crack sealing effectiveness, respectively. This finding supports GDOT's new policy of increasing its crack sealing criteria from a maximum COPACES rating of 85-90.

- Crack sealing effectiveness increases when truck traffic (AADTT) decreases; however, the finding is not conclusive. In addition, there is no clear trend for the impact of pavement thickness on crack sealing effectiveness.

- Results show that crack sealing effectiveness is high for Severity Level 1 block/transverse (B/T) cracking no matter the crack length or percentage. More and higher severity levels of longitudinal cracking in wheel paths (GDOT defined it as load crack), and higher severity level of B/T cracking (Severity Level 2 B/T cracking) result in lower crack sealing effectiveness.

Part 3: Fog Seal Field Test Study

The third part of this report presents the field test study of fog seal. Thirteen test sites on I-475 with different raveling conditions (very light, light, medium, severe with a loss of aggregates ranging from 0%-20%; GDOT defined all of them to be Severity Level 1) were selected for conducting various field tests, including locked-wheel friction test, IRI test, and visual inspection of aggregate loss test. The following tests summarize the major research findings:

Friction Tests

- A pavement's surface friction decreases right after fog seal application. Based on measurement results, the skid number (SN) decreases about 45% immediately after the fog seal application.

- The SN recovers to 30-35 in 2-4 days, and 35 or above in 5-7 days after fog seal application. Results show the surface friction of pavements with high traffic volume and truck percentage recover faster than the pavements with low-traffic volume.

- Results show that a higher friction value in the outside lane with high truck traffic volume compared to the inside lane that is likely caused by different loss of aggregates. In addition, measurements show that the friction is higher in summer and lower in winter.

IRI Tests

- The test site (I-475) has "very good" smoothness with the IRI being less than 60 in/mile based on the IRI measurements after September 2014, except that part of the pavement in southbound Lane 3 has an IRI greater than 60 in/mile, which could be due to severe raveling conditions or joint conditions between asphalt and concrete pavements.

- A slight increase in IRI right after fog seal application can be observed on most of the pavement based on the short-term IRI performance before and after fog seal applications. However, the difference is insignificant (within 6 in/mile difference).

- For the long-term IRI performance, no clear trend of increase or decrease can be observed over the entire lane. Still, the variation of IRI measurements in each mile increases from Lane 1 to Lane 3, which could be caused by the variation of aggregate loss conditions along different lanes with different truck percentages.

- Results show that seasonal changes have no obvious impact on IRI. Results show that the aggregate loss increases with higher traffic volume and truck percentage, and IRI increases with the increase of aggregate loss. However, it is not conclusive.

Durability Tests

The durability tests are conducted to measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and the fog sealed test sections based on the quantitative measurements of aggregate loss falling into the rumble strips. The following conclusions can be made about fog seal effectiveness:

- The fog seal treatment is promising in reducing the rate of aggregate loss for medium and severe raveling conditions by 30%-90% and 40%-70%, respectively. (The medium and severe raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 10%-15% and 15-20%, respectively.)

- The fog seal effectiveness on very light and light raveling conditions is not apparent because the quantities aggregates loss on non-fog seal and fog seal sections are both small. (The very light and light raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 0%-5% and 5-10%, respectively.)

RECOMMENDATIONS

The following presents the recommendations for improving the pavement preservation method guide and future research for performing fog seal and crack seal studies.

Recommendation of Crack Sealing Performance Study

- It is recommended that crack growth monitoring on the crack sealing test sites be extended to better understand the terminal life of crack sealing (e.g. 3 years, 5 years).

- Our crack sealing effectiveness study focuses on the low severity level of cracking. It is recommended that additional test sites with severe cracks (e.g. Severity Level 3) be studied for more detailed effectiveness impacts.

- It is recommended that the impact of the crack sealing area on the pavement friction reduction be studied to establish a criterion for the maximum crack sealing area.

- It recommended that the impact of pavement thickness on crack sealing effectiveness on additional test sites with different thicknesses, but similar COPACES ratings and traffic volume be studied.

- It is recommended that quantitatively measuring the impact of the pretreatment crack characteristics (e.g. crack length, direction, position, density) and different types of crack (e.g. load crack, block crack) and severity levels be accomplished. This will enable state DOTs to optimize their crack sealing effectiveness with the detailed crack characteristic-based criteria.

Recommendation of Fog Seal Performance Study

- Permeability of surface friction course is still a concern. Due to the application of fog seal, potential safety issues could be raised because of the increasing chance of splash, spray, and, even, hydroplaning. It is noted that Permeameter is not a good means for measuring permeability on porous surface. It is suggested that splash and spray tests be conducted, and they should be monitored by cameras.

- Further study is needed to assess the durability of pavements after fog seal. In this research, visual inspection and sample weighing were used to measure aggregate loss over time. This method is not completely accurate, since the results are largely affected by environment and human measurements. The 3D pavement surface data was collected. It is recommended a more accurate method to quantitatively compute the loss of aggregates, using the already collected 3D pavement surface data, be developed.

- As recommended by GDOT engineers, additional field tests are needed to evaluate the performance of fog seal on interstate highway pavements at different ages and in different locations. Although the test results are useful for uncovering some characteristics and trends of pavement performance, they cannot be easily extended to other pavements of different ages or in different locations. Especially, if the optimal timing of fog seal is to be accurately determined, more field tests and long-term monitoring on diverse pavement samples are needed.

CHAPTER 1. INTRODUCTION

RESEARCH BACKGROUND AND RESEARCH NEED

A comprehensive pavement preservation guide that defines pavement preservation methods, their application criteria, and performance is essential for the Georgia Department of Transportation (GDOT) to perform cost-effective pavement maintenance and achieve the best pavement performance with less money. Figure 1 shows a typical pavement performance curve using an exponential trend. Pavement condition drops slowly in the early stages of a pavement's service life but then deteriorates very quickly in later stages as it reaches the end of its service life. As illustrated in Figure 1, the pavement condition drops 40% during the first 75% of its service life; then, it drops another 40% in the next 12% of its service life. Thus, applying the right pavement preservation treatment at the right time is critical to prolonging the pavement's life; it will cost up to 80% less at the end of the first 75% of a pavement's life than anytime thereafter.

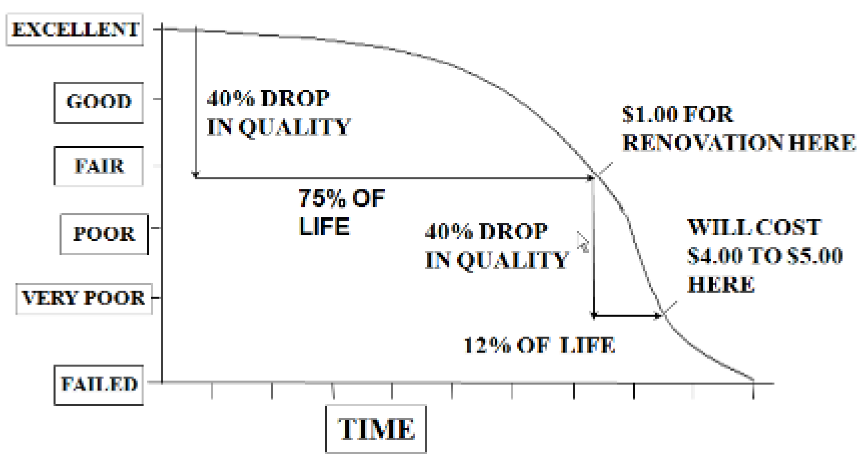


Figure 1. Graph. Typical pavement performance curve.

The experienced pavement experts in GDOT possess a great amount of tacit knowledge regarding pavement preservation, but before those experts retire, there is an urgent need to collect and document their knowledge. In addition, other state DOTs' experience and knowledge of pavement preservation techniques and their treatment criteria and performance also need to be studied. As GDOT increases the number of its pavement preservation outsourcing projects, a pavement preservation guide containing comprehensive and consistent information about project selection, specification, material selection, construction procedure, and QA/QC is essential and is urgently needed for GDOT to perform cost-effective pavement maintenance (right treatment, right time, and right location) and achieve the best pavement performance for the least amount of money while meeting MAP-21's requirements.

To meet stringent budget requirements, GDOT has been practicing various pavement preservation technologies on Georgia's highways for many years. However, a comprehensive, up-to-date pavement preservation guide based on GDOT's pavement distress protocol to select the right treatment on the right pavement at the right time is lacking.

In addition, to determine the best performance for those frequently applied pavement preservation techniques, such as crack seal, fog seal, and road tests based on GDOT's pavement distress protocol need to be studied, especially the right treatment timing.

RESEARCH OBJECTIVES

The objective of this research is to produce the first version of an enhanced, comprehensive pavement preservation guide along with a pavement preservation interactive tool (PPIT) (covering project selection, pavement preservation methods, specifications, materials selection, construction procedures, and QA/QC) for GDOT to cost-effectively preserve its pavements while meeting MAP-21's requirements. The major tasks include the following:

1) Identify the requirements of the pavement preservation guide;
2) Conduct interviews with GDOT pavement experts and review the update-to-date pavement preservation techniques and their performance and treatment timing, as recommended by the Federal Highway Administration (FHWA), other state DOTs, research organizations, and researchers;
3) Conduct in-depth data mining and statistical analysis of GDOT's historical pavement distress data;
4) Conduct field tests with long-term performance monitoring on fog seal and crack sealing in the field to study the optimal timing for applying the most frequently used pavement preservation techniques;
5) Develop the first version of an enhanced pavement preservation guide along with a web-based pavement preservation interactive tool (PPIT) for the pavement preservation guide;
6) Summarize research findings and develop a final report.

REPORT ORGANIZATION

This report is organized as follows. CHAPTER 1. presents the research background and research need of the study. CHAPTER 2. presents the literature review of asphalt pavement preservation methods. CHAPTER 3. presents the literature review of concrete pavement preservation methods. CHAPTER 4. presents the crack sealing performance study using historical COPACES data. CHAPTER 5. presents the field test study of crack sealing performance. CHAPTER 6. presents a field test study of fog seal performance. CHAPTER 7. presents the conclusions and recommendations. APPENDIX A includes the detailed introduction to and tutorial for the PPIT. APPENDIX B contains the GDOT Pavement Condition Evaluation System (PACES) Manual.

CHAPTER 2. LITERATURE REVIEW OF ASPHALT PAVEMENT PRESERVATION METHODS

This chapter reviews the prevailing asphalt pavement preservation treatments for developing GDOT's pavement preservation guide. Each treatment method is presented based on the contents of 1) what the treatment is, 2) how to select a project for this treatment, 3) material design, 4) construction procedures and their considerations, and 5) performance and limitations. These contents are also incorporated to develop the web-based pavement preservation interactive tool (PPIT) for GDOT's pavement preservation guide. For a detailed introduction to and tutorial for the PPIT, please see APPENDIX A.

FOG SEAL

What is Fog Seal?

Defined by the Asphalt Emulsion Manufacturers Association (AEMA), fog seal is "a light spray application of dilute asphalt emulsion used primarily to seal an existing asphalt surface to reduce raveling and enrich dry and weathered surfaces" (AI & AMEA, 2014).

Fog seal is also used in chip seal application to hold chips in place, which is also called flush costs. This can help prevent vehicle damage from flying chips. Fog seal can seal small cracks and prevent moisture penetration.

GDOT has initiated a study of using fog seal to preserve raveled Open Grade Friction Course (OGFC) or Porous European Mix (PEM) on high-traffic-volume interstate highways since 2014. OGFC or PEM provides an excellent riding experience and enhances roadway safety in wet weather and is widely used on Georgia's interstate highways. For OGFC and PEM under heavy traffic conditions, raveling, which is the dislodging of coarse aggregates, becomes the major type of distress. In the past, as a major maintenance method for correcting raveling issues, GDOT has milled the surface friction course and part of the underlying asphalt layer, then resurfaced it. To slow down the pavement deterioration and prolong the life of OGFC pavements, GDOT has actively sought alternative preventive maintenance methods and is conducting relevant studies. To prevent further pavement deterioration caused by raveling, GDOT has studied the application of fog seal on Interstate Highway 475, a bypass around Macon, Georgia.

How to Select Projects for Fog Seal?

The California Department of Transportation (Caltrans) Maintenance Technical Advisory Guide (MTAG) is often referred to as a source for some major pavement preservation techniques (Caltrans, 2003). The following content was summarized from this guide:

1) As a pavement preservation technique, fog seals will not correct distresses such as cracking, base failures, excessive stone already lost, or any other severe pavement defects. It is used to seal pavement surfaces and extend pavement life, but it cannot provide extra loading capacity. Thus, the pavement structure should be in good condition, and the pavement surface should not be severely deteriorated.

2) Fog seal can be used on raveled pavement as a method of holding stones in place or on aged pavement surfaces as a method of rejuvenating hardened asphalt. However, there is not a systematic, objective method to quantifying the degree of raveling or aging in a pavement. Moreover, the best treatment timing of fog seal is still unknown. Visual

inspection and engineers' judgments are still the major approach for determining fog seal treatment.

3) Fog seal can be applied on traveled ways, shoulders, gores, or dikes. However, the requirement of emulsion penetration for various surfaces is different. On a traveled way, fog seals should only be used where surface penetration of the emulsion is expected. In contrast, on shoulders, gores, or dikes, penetration is desirable, but it is not essential.

4) In general, traffic level is not a determining factor except in job set up. For situations requiring that the sealed pavement be opened to traffic shortly after the application of the seal, a blotter coat of sand may be used to prevent pick-up. However, fog seal use is restricted on heavily trafficked roads as the friction may be reduced after its application (TPPC, 2013).

Figure 2, Figure 3, and Figure 4 show some typical situations where fog seals are suitable.

Figure 2. Photo. Aged dense graded HMA.

Figure 3. Photo. Open graded HMA.

Figure 4. Photo. Chip seal.

In contrast, dense graded HMA with closed surface texture is not suitable for fog seal (Figure 5).

Figure 5. Photo. Dense graded HMA with closed surface texture.

Material Design

Dilute asphalt emulsion is used for fog seal. In the original asphalt emulsion, the percentage of water is up to 43%. Then, an equal volume (to the original emulsion) of water (1:1) is added to dilute the original asphalt emulsion. If fog seal is used for rejuvenating an aged pavement surface, rejuvenating oil is also added to soften the stiffened old asphalt. To facilitate the dilute asphalt emulsion to penetrate pavement surface, two properties are important: 1) it should have low viscosity; and it should break slowly.

Two types of asphalt emulsions are used, cationic and anionic (TxDOT, 2006). The asphalt droplets carry cationic charges in a cationic asphalt emulsion. Similarly, they carry anionic charges in an anionic asphalt emulsion. These two types of asphalt emulsions behave differently when asphalt breaks. Since a pavement surface is normally negatively charged, a cationic asphalt emulsion can break through the attraction between asphalt droplets and a pavement surface and the loss of water due to evaporation. For anionic asphalt emulsion, evaporation is the only way for asphalt breaking. Thus, if all other conditions are same (humidity, pavement surface, etc.), a cationic asphalt emulsion can cure faster than an anionic one. In terms of asphalt setting time, asphalt emulsions are further classified as rapid-setting (RS), medium-setting (MS), and slow-setting (SS). In terms of ASTM D977 and D2397, CRS, CMS, and CSS indicate cationic emulsions, whereas RS, MS, and SS means anionic counterparts are followed by numbers and text indicating the emulsion viscosity and residue properties. For example, SS-1H means anionic slow-setting emulsion with low viscosity and a hard asphalt residue. The grades of asphalt emulsion usually used in fog seals are SS-1, SS-1h, CSS-1, or CSS-1h.

For high-stress pavements, polymers are added to modify the conventional asphalt emulsions. Polymers provide a stronger bond with better long-term performance (Muncy, 2013).

Construction Procedures and Considerations

FHWA and FP2 published a checklist for fog seal in 2002 (FHWA, 2002), which is a good guideline for fog seal construction. Caltrans adopted this guideline with necessary modifications (Caltrans, 2003). The following is a summary**:**

Weather Considerations

Warm conditions with little to no chance of rain are necessary to ensure successful applications. Fog seals should not be applied when the atmospheric temperature is below 50°F and the pavement temperature is below 59°F.

Traffic Considerations

Traffic control should be in place before work forces and equipment enters onto the roadway or into the work zone. Under ideal conditions, including increasing air and surface temperatures, it is suggested that traffic be kept off the fog seal material for at least two hours and acceptable skid test values are achieved.

Surface Preparation

Pavement surfaces must be clean and dry. Dust, dirt, and debris need to be removed using a road sweeper, power broom. If flushing is required, it should be completed 24 hours prior to the application.

Operations

The emulsion should be diluted no more than 24 hours before its intended use. This is to avoid settlement of the diluted emulsion. Water is always added to the emulsion and not the other way around. The emulsion may be circulated using a centrifugal or other suitable pump to ensure uniformity.

The emulsion is generally sprayed at ambient temperature, but it can be heated to 122°F maximum.

The emulsion application rate is pavement surface dependent. Typical application rates for diluted emulsion (1:1) range from 0.03-0.22 gal/yd2 depending on the surface conditions (Caltrans, 2003). Ideally, one-half of the application should be sprayed in each direction to prevent build up on one side of stones only (this is particularly important in the case of chip seals) and rough surfaces. Build up on one side can result in a slippery surface and an inadequate binder to fully enrich the surface or hold the stone.

Table 1. AMEA recommendations for application rates (Caltrans, 2003).

% Original Emulsion	Dilution Rate	Tight Surface*		Open Surface**	
		(l/m^2)	(gal/yd^2)	(l/m^2)	(gal/yd^2)
50	1:1	0.15 - 0.5	0.03 – 0.11	0.4 - 1.0	0.09 – 0.22

The following method can be used to estimate emulsion application rate (Muncy, 2013):

1) Take a 1L can of diluted emulsion and pour it evenly over an area about 1 m^2 (or take a 1 quart can of the diluted emulsion and pour it evenly over an area about 1 yd^2).

2) If the emulsion is not absorbed into the surface (after 2-3 minutes), decrease the amount and apply to a new 1 m^2 (or 1 yd^2) area. Repeat the trials until the approximate application rate is found.

3) If the surface looks like it will absorb more emulsion, increase the amount and apply over a new 1 m^2 (or 1 yd^2) area. Repeat trials until the approximate application rate is found.

4) Sand blotters may be used at approximately 1 kg/m^2 to allow early opening to traffic. Sweeping may be required. Even with sand cover, traffic control may be required to keep speeds down.

Performance and Limitations

Fog seals are an inexpensive way of arresting raveling and adding binder back into aged surfaces. They can also hold chips in place in fresh chip seals, (or older chip seals beginning to lose rock) reducing the potential for vehicle damage.

Fog seals are not useful as seal coats on tight surfaces without the addition of aggregates, as they will reduce surface texture and may create a slippery surface. Fog seals should not be used on rubberized asphalt concrete (RAC) or polymer modified mixes unless the pavements are over five years old, as these binders age at different rates.

The application of fog seals is also limited by weather. A cutoff date in the fall (e.g. September 1st) will ensure that rain will not be a factor and that the emulsion will fully cure before freezing conditions are encountered. In addition, seal coats applied in the winter have less time to penetrate the pavement and are more prone to cause slick surface conditions.

The service lives of fog seals could be 1-3 years depending on various factors (type of seal, original pavement conditions, traffic, roadway geometry, etc.) (TPPC, 2013).

CRACK SEALING/FILLING

What is Crack Sealing/Filling?

Crack sealing and filling are common treatment methods that can effectively reduce or prevent water infiltration caused by cracks in asphalt pavements. Crack sealing and filling do not improve structural conditions of the pavements but can retard irreparable damage to the subgrade from water intrusion. It has been reported that crack sealing can extend the life of a pavement up to 4 years (Wu et al., 2010). Possible limitations and negative impacts of crack sealing and filling include undesirable visual impacts and increased pavement roughness. Crack sealing and filling have different applications; the main factor is determining the appropriate application for working cracks or non-working cracks (working cracks have horizontal and/or vertical movements greater than non-working cracks, and the movement criteria is different, as shown in Table 2).

Table 2. Criterial of working and non-working cracks.

	Working cracking	**Non-working cracking**	**Movement criteria**
Michigan (Gesford) **Washington State** (ERICKSON 1992) **Maryland** (MDSHA 2014)	1)transverse cracks 2)some longitudinal 3)diagonal cracks	1. Diagonal cracks, 2. Most longitudinal cracks 3. Some block cracks	0.100 inch (2.5 mm)
SHRP (Thomas P. Wilson and Romine 1994)	1. Transverse thermal 2. Transverse reflective 3. Longitudinal reflective 4. Longitudinal cold-joint	1. Longitudinal edge 2. Longitudinal reflective 3. Longitudinal cold-joint 4. Distantly spaced block	0.100 inch (2.5mm)
NCHRP (Decker, 2014)			0.125 inch (3.0 mm)
California (Caltrans 2003)			0.250 inch (6.0 mm)

Crack sealing uses materials that are “placed into and/or above ‘working’ cracks in order to prevent intrusion of water and incompressible substances (e.g. dirt) into the cracks”; and often involves routing the crack prior to cleaning (Decker, 2014).

Crack filling is the placement of materials “into “non-working” cracks to substantially reduce infiltration of water and reinforce adjacent cracks” (Decker, 2014). Crack sealing is more common for transverse cracks (such as transverse thermal and reflective cracks, diagonal cracks, and certain longitudinal reflective cracks), and crack filling is more common for longitudinal cracking (such as typically longitudinal cold-joint and reflective cracks, edge cracks, and distantly spaced block cracks) (Wu et al., 2010).

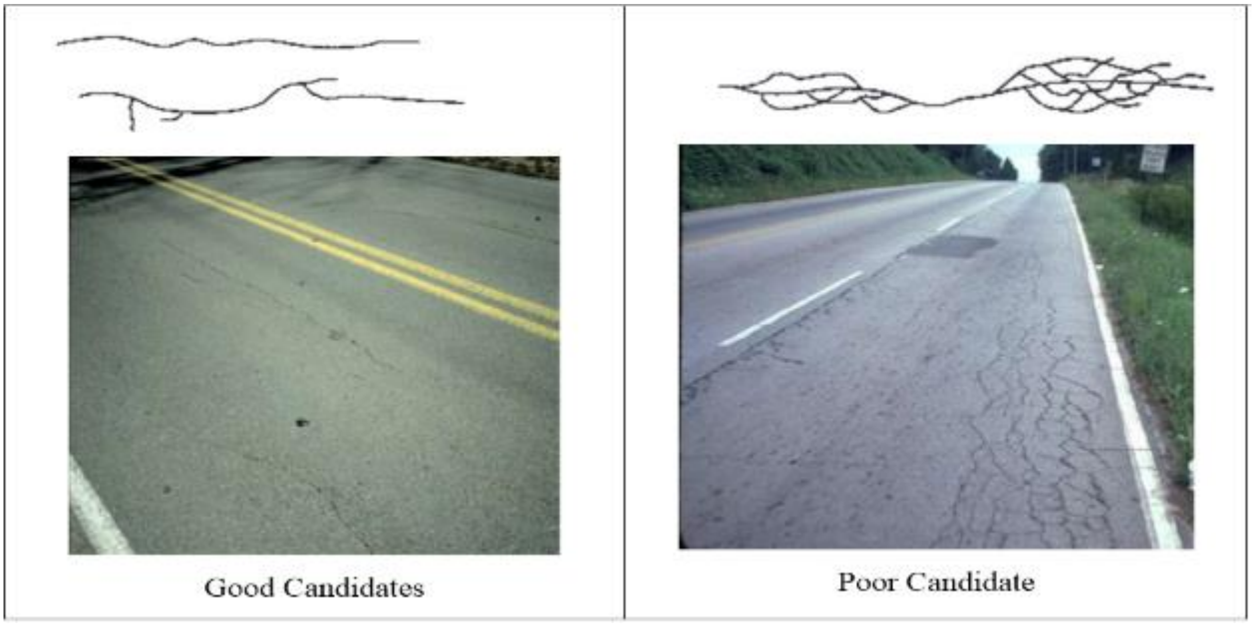
In general, crack sealing, which is intended to prevent water penetration, requires more effort to accomplish than crack filling. Although clear distinctions between these two treatment methods can be found in the literature, in practice, many state DOTs, counties, and cities do not make specific distinctions between the two, and recent research has reported that crack sealing is generally performed in northern climates, and crack filling is more predominate in southern climates (Decker, 2014). This is logical since the colder temperatures would lead to cracks that would move (contract) more. Although GDOT considers Type M sealant as a crack filler and Type S sealant as a crack sealer in the specifications, they can be used for both kinds of cracks. The characteristics of crack sealing and filling are shown in Table 3.

Table 3. Characteristic of Crack seanling and filling.

Advantages	Disadvantages
• Reduce water infiltration • Decrease further crack deterioration (e.g., spalling at crack) • Reduce or delay moisture damage • Slow pavement deterioration (roughness increases, potholes and depression formation) • Performance well in all climate conditions • Performance is not significantly affected by varying ADT or truck levels • Quick opening to traffic • Substantial life-cycle cost savings	• Over-application can cause a reduction in skid resistance • Poor appearance and visibility • No structural improvement

How to Select Projects for Crack Sealing/ Filling?

Crack sealing and filling are preventive maintenance treatment methods that are used to reduce and/or prevent water penetration along pavement surface cracks (e.g., longitudinal, transverse, and diagonal cracks) into sub-layers. The pavement, in general, needs to be in good/fair condition, i.e., no alligator cracking (in GDOT's COPACES, it is load cracking Severity Level 3 or 4). Figure 6 shows a good and poor candidate for crack sealing. These techniques are considered effective when applied to cracks less than 1 inch in width; they should not be considered if the pavement is structurally defective, e.g. the pavement has alligator cracking (load cracking Severity Level 3 or 4) and/or excessive rutting or if the pavement is scheduled to be resurfaced or replaced. Crack sealing/filling does not add structural capacity.



Good Candidates

Poor Candidate

Figure 6. Photo. Cracking Characteristics of Good and Poor Candidates (Decker, 2014; GDOT, 2007)

A best practice by Decker noted the criteria in the Table 4 (Decker, 2010). Edge deterioration relates to the edge of the crack. It was noted that it was very difficult to identify the horizontal movement, as it is season dependent.

Table 4. Typical criteria for crack sealing and filling (Decker, 2014)

	Crack Sealing	Crack Filling
Applicable Width	0.12"-1.00"	0.12"-1.00"
Edge Deterioration	<25%	<50%
Annual Horizontal Movement	>0.12" Working	<0.12" Non-Working
Appropriate Type of Crack	Transverse Thermal Transverse Reflective Longitudinal Reflective Longitudinal Cold Joint	Longitudinal Reflective Longitudinal Cold Joint Longitudinal Edge Block, distantly spaced

Material Design

Table 5 illustrates different material types used for crack sealing and filling. Modified asphalt emulsions, liquid asphalt, and rubberized asphalts are the most common types. Temperature and traffic considerations go into selecting the proper material for a project. Approved asphalt-rubber materials for GDOT are located on GDOT's Qualified Products List (QPL) 92 and defined in GDOT specification Section 407.

Table 5. Properties associated with various material types (Decker, 2014)

Property	Material Type*							
	Asphalt Emulsion	Polymer-Modified Emulsion	Asphalt Cement	Fiberized Asphalt	Asphalt Rubber	Rubberized Asphalt	Low-Modulus Rubberized Asphalt	Self-Leveling Silicone
Short Preparation	✓	✓						✓✓
Quick & Easy to Place	✓	✓	✓✓	✓✓	✓✓	✓✓	✓✓	
Short Cure Time			✓✓	✓✓	✓✓	✓✓	✓✓	✓
Adhesiveness	✓✓	✓	✓✓	✓	✓	✓	✓	✓
Cohesiveness					✓	✓	✓✓	✓
Resistance to Softening and Flow (cured state)		✓			✓	✓	✓✓	✓✓
Flexibility		✓			✓	✓	✓✓	✓✓
Elasticity		✓			✓	✓	✓	✓✓
Resistance to Aging and Weathering						✓	✓	✓✓
Resistance to Tracking and Abrasion					✓	✓✓	✓	

*✓ Applicable; ✓✓ Very Applicable

Construction Procedures and Considerations

Weather Considerations

It is recommended that crack sealing and filling be conducted when the temperature is moderately cool (e.g., in the spring or fall) and the crack be filled just below the top. The reason for this is shown Figure 7, where the material remains stressed in all seasons if applied in the winter. Manufacturer and agency instructions and requirements specific to sealant use and ambient/surface temperature (typically 40 °F and rising) for routing and sealing should be followed. The application should not proceed if there is moisture on the pavement surface or in the cracks or if rain is imminent.

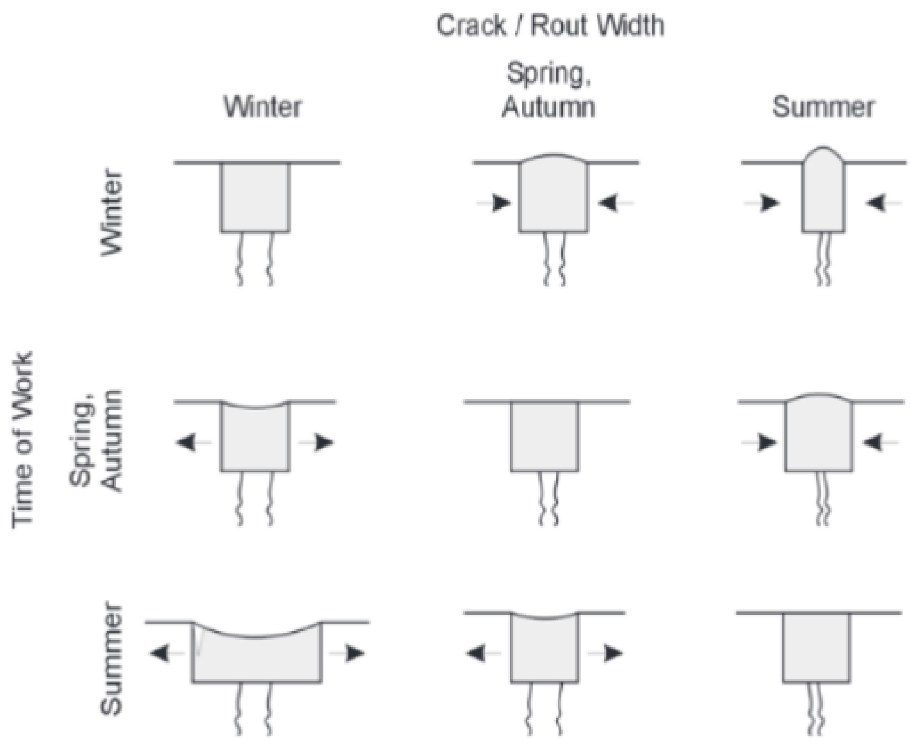


Figure 7. Graph. Seasonal Impact on Sealing (Decker, 2014)

Traffic Considerations

An appropriate traffic control plan should be developed and maintained before and during the crack sealing/filling operations. The work zone should be constructed according to GDOT requirements. Traffic should not be opened until the sealant has adequately cured and will not be picked up by vehicle tires.

Surface Preparation

Pavement surface and cracks should be dry and clean. Any other pavement distresses should be repaired before applying crack sealing/filling. An air compressor/blower should be used to clean dust and debris before applying sealing materials. If needed, a hot-air lance should be used to ensure that no moisture remains on the pavement surface or in cracks.

Reservoir Cutting/Routing

Routing is typically used for crack sealing operations. A square or rectangular reservoir is cut along and centered around the crack on the pavement surface. Manufacturer and/or agency requirements regarding the dimensions of the reservoir should be followed. The crack and reservoir should be cleaned the same way the surface is prepared, as described above.

Sealant Application

Sealant application varies according to the configuration of the cracks and the techniques used. Examples of crack treatment techniques include flush fill, over-band, reservoir, combination, and backer rod.

Manufacturers' and agencies' installation temperature and configurations requirements should be followed. Sealant should be heated and constantly remain at the minimum temperature required for pouring or application, but it should not exceed the material's safe heating temperature. Sealant should be continuously agitated to ensure the uniformity of the material.

The reservoir should be filled from the bottom up. Then, the sealant should be finished and shaped according to manufacturer and agency requirements. Blotter materials (e.g., sand) should be applied sufficiently on crack treatment materials to prevent wheel tracking if the pavement must be open to traffic before materials are fully cured.

Cure time after crack sealing/filling is usually less than 1 hour (OPE, 2001). A sufficient amount of blotter material must be applied to protect the uncured crack treatment material from tracking. Fine sand (the most common used blotter), toilet paper, talcum powder, and limestone dust are commercial products often used. These blotters should be applied immediately after finishing so that they can stick to the material and serve as temporary covers. Care must be taken not to over-apply dust and powder materials.

Performance and Limitations

Pavement condition

Crack sealing and filling is intended to be performed while the pavement is still in relatively good condition. A pavement with a PACES rating of 80 to 90 with Level 1 cracking (including load, block, and reflective cracking) would be most appropriate, but a pavement with a rating between 70 and 80 with predominately block cracking Level 2 or less would also be a suitable candidate. Pavements with Severity Level 3 or 4 load cracking should not be considered.

Crack Characteristics

Crack characteristics, such as the crack type and the microscopic features of pavement cracks, including crack width, crack length, and crack orientation, are important factors in determining the effectiveness of the crack treatment because they directly affect the timing for crack treatment and the sealant selection, routing etc. Reflective cracking, due to movement of underlying concrete slabs, may need to be routed prior to sealing to provide some extension of pavement life.

Crack sealing/filling techniques

Cracks must be clean and dry to allow the sealant material to adhere to the sides of the crack. The sealant must be properly prepared (handling and heating) and the crack not underfilled or overfilled.

CHIP SEAL

What is Chip Seal?

A chip seal is the application of a thin layer of bituminous binder; it is immediately followed by the application of a layer of crushed gravel (chips), which is compacted and embedded into the binder using a pneumatic (rubber tired) roller. A chip seal is a pavement preservation treatment that addresses pavement distresses such as raveling and minor cracks, and it provides a new skid resistant wearing surface. Advantages of chip seals include improved skid resistance, rapidity and ease of construction, and good durability, all which help extend the service life of pavements (CDOT, 2008).

Figure 8 shows the use of chip seals, in terms of traffic condition and overall performance, in the United States (Gransberg et al, 2005).

Figure 8. Graph. Summary of a Survey Regarding to Chip Seal Usage and Performance (Gransberg et at., 2005)

Chip seals are typically more economical than regular asphalt mixtures. They can protect the subgrade by preventing water from penetrating through cracks. They can restore skid-resistance to worn or polished pavements. They have also been used as a crack-relief interlayer along with an asphalt mixture resurfacing to reduce reflective cracking. A chip seal, basically, consists of two types of materials: binders and aggregates. Depending on the surface texture, traffic conditions, and climatic conditions, different types and designs of chip seal can be applied. The most common types of chip seal and their designs are summarized below (CDOT, 2008; Gransberg et al., 2005; Shuler et al., 2011):

Single Chip Seal:

A single-course chip seal is the most common type of chip seals. It consists of a single layer of bituminous binder with uniformly graded aggregate, as shown in Figure 9:

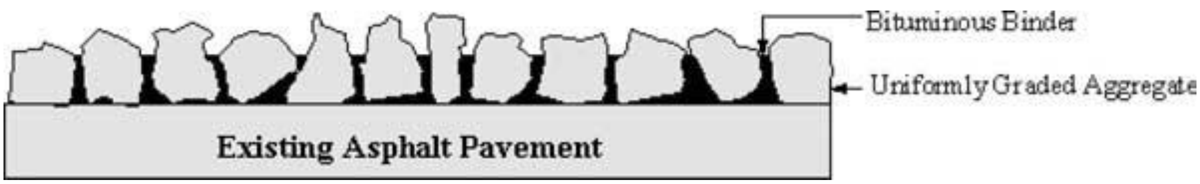


Figure 9. Graph. Single Chip Seal (Gransberg et al., 2005)

Double/Multiple Chip Seal:

A double or multiple chip seal is a seal coating consisting of multiple applications of bituminous binder and aggregate. As shown in Figure 10, a double chip seal consists of a layer of bituminous binder, a layer of uniform graded aggregate, another layer of binder, and another layer of smaller aggregate.

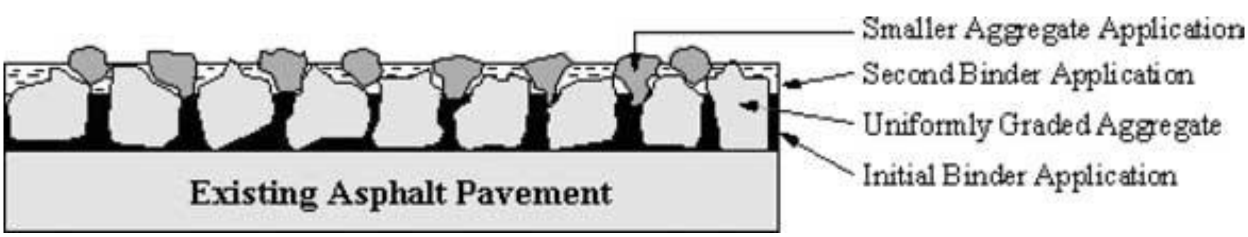


Figure 10. Graph. Double Chip Seal (Gransberg et al., 2005)

Racked-in Seal (Single Chip Seal with Choke Stone):

This type of chip seal consists of a single chip seal covered with a crushed fine aggregate applied to the surface to lock in the voids of the seal. As shown in Figure 11, the choke stone provides interlock between the binder and the seal, which then prevents the loss of aggregate before the binder becomes fully cured.

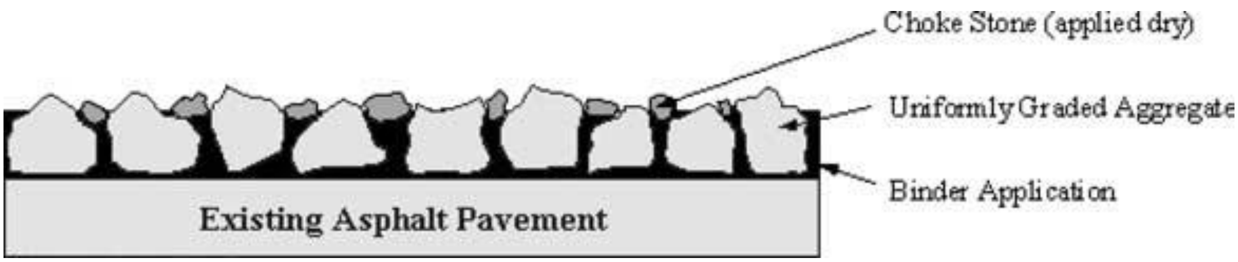


Figure 11. Graph. Racked-in Seal (Gransberg et al., 2005)

Inverted Seal:

This type of chip seal is a double chip seal. However, the difference between an inverted seal and a regular double chip is that its larger aggregate is applied after the application of smaller aggregate. This type of seal is used to repair or correct an existing bleeding surface. Details are shown in Figure 12:

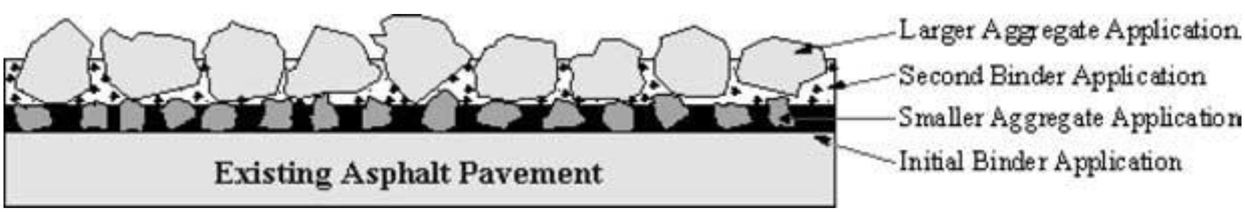


Figure 12. Graph. Inverted Seal (Gransberg et al., 2005)

Geotextile-Reinforced Seal:

For this type of seal, a layer of geotextile product is placed on the existing pavement prior to the application of a single chip seal. It is used to enhance the performance of a seal over extremely oxidized or thermal cracked surfaces. Details are shown in Figure 13:

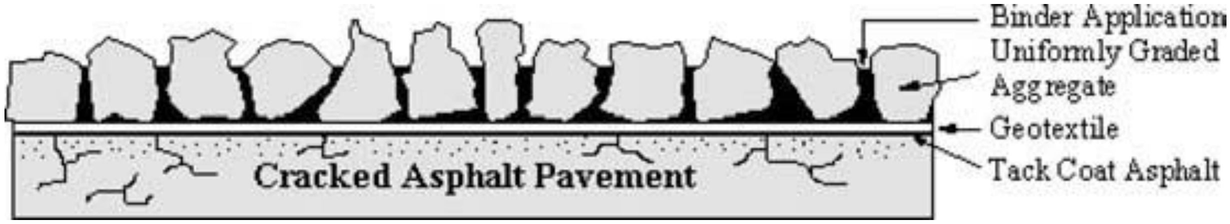


Figure 13. Graph. Geotextile-Reinforced Seal (Gransberg et al., 2005)

How to Select Projects for Chip Seal

Chip seal is a pavement preservation treatment that addresses pavement distresses, such as raveling, flushing, minor cracks, and it provides a new skid resistant wearing surface for aged pavements. Advantages of chip seals include improved skid resistance, rapidity and ease of construction, and good durability that help extend the service life of pavements (NCPP).

Chip seals are not designed for enhancing the structural capacity of pavements; instead, they are a means of preventive maintenance that can effectively enhance pavement surface conditions and extend pavement service life when applied to pavements with limited or no distress (Gransberg et al., 2005; Peshkin et al., 2004; Shuler et al., 2011). Pavements with severe distresses or pavements that are extensively deteriorated, on the other hand, are not suitable for chip seal application. Chip seals cannot bridge cracks wider than 1/4 inch, nor can they properly address ruts greater than 3/8 inch in depth. Chip seals can produce a rough, open texture that may produce high noise, and they may not be as smooth as asphalt mixtures. Stones may be dislodged in the early stages of service, which can cause broken windshields if the chip seal is not properly constructed. Using modified binders and sand seals with the chip seal can alleviate some of these concerns. Good and poor candidates are shown in Figure 14, Figure 15, Figure 16:

Figure 14. Photo. Chip Seal Good and Poor Candidates (NCPP)

Figure 15. Photo. Poor Candidates for Chip Seal

Figure 16. Photo. Good candidates for scrub seal with cape overlay

Material Design

The chip seal bituminous material (emulsion or asphalt cement) and aggregate gradation should follow the standard specifications in Section 424 of GDOT Supplemental Specifications. Aggregates and emulsions/asphalt binders must come from approved suppliers. Hauling costs should be considered when there is no approved aggregate source near the proposed project.

Materials must be handled on site properly to prevent contamination from dirt or other deleterious materials.

Traffic levels affect bituminous material application rates. Roads with lower traffic volumes can tolerate higher bituminous liquid application rates. Roads with higher traffic volumes or higher truck traffic require slightly lower bituminous liquid application rates to avoid bleeding. The application rates in the specifications should be followed.

Construction Procedures and Considerations

Weather and proper methods are critical for chip seal performance.

Weather Considerations

It is required that chip seals be constructed when the temperature is warm. The standard specification, Section 424, limits construction to between April 15 and October 15 and, also, requires a minimum ambient and pavement temperature. I If there is moisture on the pavement surface or if rain or freezing temperatures are imminent, the application should not be done. The relative humidity can affect the amount of time for an asphalt emulsion to cure; therefore, the specification has a caution when the temperature is warm and the relative humidity is high. In addition, high temperatures may also lead to lower viscosity in the binder, causing aggregates to be picked up by the rubber tires of the rollers.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the chip seal operations. The work zone should be constructed according to GDOT requirements. The pavement should not be opened to traffic until the bituminous material has cured sufficiently to ensure that the aggregate will not be loosened, dislodged, or whipped off by slow moving traffic (GDOT Specification Section 424).

Low-speed traffic may be allowed to travel on the fresh seal after the sweep; a pilot vehicle should be used to control the speed. Traffic will continue to compact the seal to its full embedment over time.

Surface Preparation

1) Before chip sealing is applied, the existing pavement surface should be structurally sound.

2) Patching and/or crack sealing may be needed if alligator cracking or longitudinal/transverse cracking are present.

3) The surface should be primed; a light application of asphalt emulsion diluted 50:50 with water on the surface should be sprayed on, if required, so that the actual binder for the chip seal will not be absorbed into the surface and cause loss of aggregate after the pavement is opened to traffic.

4) The pavement surface should be cleaned with a power broom, power blower, hand brooms, or other means so that it is free of rocks, dust, and debris.

Chip Seal Application

Binder (Emulsion) Application

1) The binder distributor should be calibrated before spraying. To ensure it is properly functioning, it should be checked for having a volume level gauge, a sample valve, and a hand hose. Equipment is shown in Figure 17:

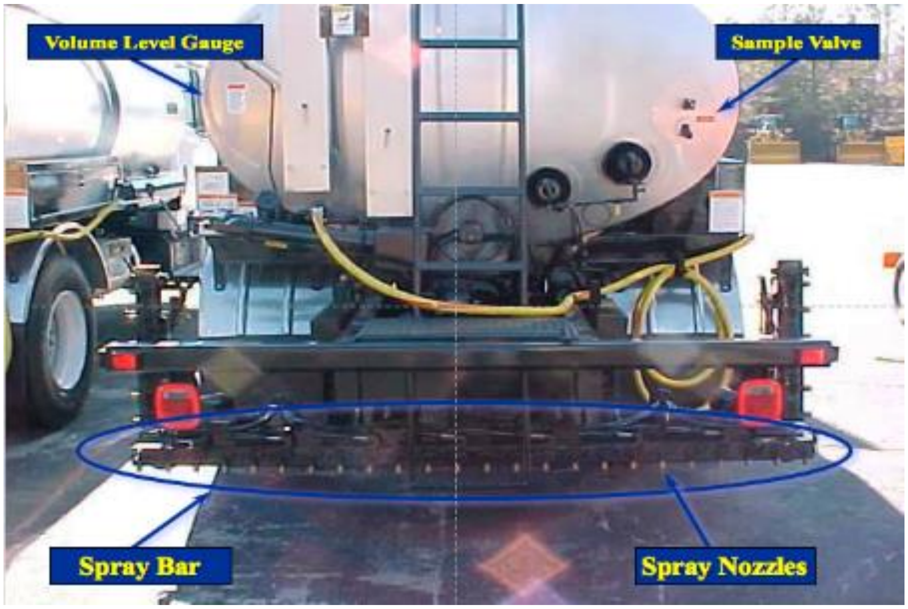


Figure 17. Photo. Equipment of Binder Application

2) The spray pattern should be checked for uniformity and proper double overlap.

3) The binder application should be started and stopped by spraying on top of heavy paper that helps create a clear transverse joint between the chip seal and the existing pavement. Proper double overlap of distributor spray pattern is shown in Figure 18:

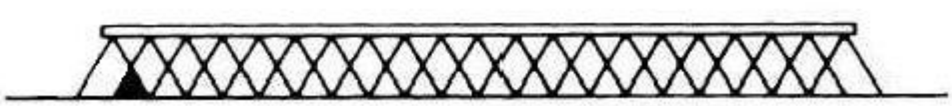


Figure 18. Graph. Proper double overlap of distributor spray pattern

Aggregate (Chip) Application

1) Chips should be applied immediately before the fresh emulsion sets.

2) Truck movements on the fresh emulsion should be minimized.

3) The aggregate spreader should have a scalping screen to remove any large particles. The aggregate spreaders should be checked for proper calibration and wear. An aggregate spreader is shown in Figure 19:

Figure 19. Photo. Aggregate Spreader

Rolling

1) Rubber tire (pneumatic) rollers provide the initial compaction of the aggregate. A steel-wheeled roller may be used if it does not fracture the aggregate, but a minimum of two rubber-tired rollers are preferred. Rubber-tired rollers are shown in Figure 20.

2) The speed of rolling is a key consideration. In hot weather, rolling should be expedited to ensure proper embedment of aggregate. However, if rollers travel too fast, proper embedment will not be achieved.

Figure 20. Photo. Rubber-tired Rollers

Brooming

1) The new surface should be cleaned with a broom to remove any excess chips as soon as possible after rolling and before the pavement is opened to traffic. A power broom is shown in Figure 21.

2) During a multiple chip seal operation, sweeping is needed between the application of courses.

3) Care must be taken not to unseat bonded stone when sweeping.

Figure 21. Photo. Power Broom

Opening to Traffic

Traffic on an individual course should not be allowed until the bituminous material has cooled or set enough to ensure that the aggregates will not be loosened, dislodged, or whipped off by slow-moving traffic.

Performances and Limitations

Loose or Excess Stone

Loose stone from a chip seal operation has numerous negative consequences:

1) Reduction of skid resistance;

2) Possible windshield damage;

3) Obscuring of striping;

4) Dislodgement of stone that would not otherwise be disturbed;

5) Negative public perception of chip seals.

Loose stone can be caused by using improper types or grades of bituminous materials, paving when temperature conditions are not proper, improper or insufficient rolling, excessive application of stone, failure to adequately sweep the finished surface, or a combination of any of these factors.

Bleeding

Bleeding is caused by uneven coverage of bituminous materials. Proper calibration and operation of the distributor should prevent bleeding. A bleeding joint and a bleeding wheel path are shown in Figure 22.

Figure 22. Photo. Bleeding Joint and Bleeding Wheel Paths

Streaking/Raveling

Streaking raveling is usually the result of allowing traffic on the chip seal too early. It will also result in loose stones, as shown on the edge of the shoulder in Figure 23:

Figure 23. Photo. Streaking/Raveling

Corrugations

Corrugations can be the result of improper/excessive aggregate application rates. Corrugations are shown in Figure 24.

Figure 24. Photo. Corrugations

MICRO-SURFACING

What is Micro-surfacing?

Micro-surfacing is a type of slurry seal that uses a polymer-modified asphalt emulsion and is placed by a specialized machine. It cures through chemical reaction, faster than slurry seal, and it can also be thicker than slurry seal. Micro-surfacing can be used in slurry seal applications that need quick cure time or thicker layers. They can also be used on pavements that are just beyond the cracking or rutting limits of slurry seals (SHRP 2, 2011). Micro-surfacing does not have as much liquid binder as slurry seal, and it uses a modified binder, so special equipment is required to apply the material properly. Micro-surfacing is typically more expensive than slurry seal due to the special equipment required and the modified binder used. “Micro-surfacing performs best when applied to correct surface friction, oxidation, raveling, and/or rutting on pavements that have adequate structural capacity” (NCHRP, 2010).

Micro-surfacing used in Georgia can be coarser than that used for slurry seals, but it is still finer than typical chip seals. Slurry seals use finer aggregate (~4.75 mm /passing #4 sieve max size) than chip seals to allow the mixture to flow and be spread as a thin wearing surface. Maximum aggregate size for micro-surfacing is 3/16 to 3/8 inch; chip seals are typically 3/8 to 1 inch.

Material used as mineral filler may include Portland cement or hydrated lime. The mixture is placed continuously over an existing pavement. The treatment seals the pavement, restores surface texture, fills minor ruts, and can reduce continued raveling of the surface. Slurry seal/micro-surfacing Types I, II, and III are defined by the International Slurry Surfacing Association (ISSA) and are based on the range of the aggregate gradation, with Type I, the finest, and Type III, the coarsest gradation. Type II is recommended for roadways that have raveling and oxidation and moderate to heavy traffic volumes. Type III is appropriate for arterial streets and highways to fill minor surface irregularities, correct raveling and oxidation, and restore surface friction (NCHRP, 2010).

GDOT's aggregate gradation for micro-surfacing in standard specification Section 428 includes a Type I and Type II, but it should be noted that GDOT's Type I gradations align closer to the ISSA Type II, and GDOT's Type II gradations are closer to the ISSA Type III designation. GDOT slurry seal gradation is different than the GDOT micro-surfacing Types I or II, but it is very similar to ISSA Type II. GDOT does not have a comparable ISSA Type I gradation in the slurry seal or micro-surfacing specification. Both GDOT Types I and II micro-surfacing allow some coarser material (between 4.75 mm and 9.50 mm/ #4 sieve to 3/8 inch). Like slurry seal, the mixture should be designed based on the specific materials to be used. Criteria and test methods are provided in Standard Specification Section 428.

How to Select Projects for Micro-surfacing

Slurry seals can seal and extend the life of an aged pavement. Pavements that are in good condition but on which the asphalt surface has oxidized are the best candidates for slurry seal. Slurry seal is not recommended for pavements with wide cracks (block cracking Severity Level 3 or 4) or alligator cracking (load cracking Severity Level 3 or 4) and/or excessive rutting (> 1⁄4 inch), or if the pavement is already exhibiting subgrade failure due to moisture (i.e. deep patching required). A slurry seal does not add structural capacity. Poor candidates for slurry seal are shown in Figure 25.

Figure 25. Photo. Characteristics of Poor Candidates for Slurry Seal (GDOT, 2007)

Micro-surfacing can be used on pavements that were considered for slurry seal but have higher levels of raveling or rutting than recommended for slurry seal. Pavements that are in relatively good condition but which have asphalt surfaces that have raveled, oxidized, or have become slightly rutted are the best candidates. Micro-surfacing is not recommended for pavements with wide cracks (block cracking Severity Level 3 or 4) or alligator cracking (load cracking Severity Level 4) and/or excessive rutting (>1/2 inch), or if the pavement already is exhibiting subgrade failure due to moisture (i.e. deep patching required). Micro-surfacing, although it can be thicker than a slurry seal, still does not add structural capacity. Good candidates for micro-surfacing are shown in Figure 26.

Figure 26. Photo. Good Candidates for Micro-surfacing

Material Design

The cold thin surface paving system consists of a water-based polymer modified asphalt emulsion(about 7% by weight), 100% crushed fine aggregate (about 95% by weight), mineral filler (about 1% of weight of total dry mix), water (about 6%-11% of dry mix), and polymer additives (about 3% by weight).

The State of California Department of Transportation (Caltrans) Maintenance Technical Advisory Guide (MTAG) (Caltrans, 2003) gives some requirements for material design. The following content was summarized from this guide:

1) There are two types of asphalt emulsion, polymer modified anionic (PMQS-1h) and polymer modified cationic (PMCQS-1h). Cationic emulsions are typically used in micro-surfacing. The minimum amount required will be based on asphalt weight content and will be certified by the emulsion supplier. In general, three percent (3%) polymer solids, based on asphalt weight, is considered minimum (Caltrans, 2003).

2) In addition, the asphalt emulsion should be slow-breaking and quick-setting. Slow-breaking gives the emulsion enough time to go into the surface and adhere with it. Quick-setting guarantees the emulsion adheres with the surface and avoids running off the roads.

3) High quality, clean, 100 percent crushed aggregate should be used for micro-surfacing. The maximum aggregate size for micro-surfacing is typically 9.50 mm (3/8 inch). There are usually three types of aggregate. Table 6 shows the gradation of these three types. Type I is too fine to be used for micro-surfacing. Types II and III are usually used for micro-surfacing.

Table 6. Gradations of Three Types of Aggregate (Caltrans, 2003)

Sieve	Type I percentage passing	Type II percentage passing	Type III percentage passing
3/8 in (9.5mm)	-	100	100
No.4 (4.75mm)	100	94-100	70-90
No.8 (2.36mm)	90-100	65-90	45-70
No.16 (1.18mm)	60-90	40-70	28-50
No.30 (600µm)	40-65	25-50	19-34
No.200 (75µm)	10-20	5-15	5-15

Table 7 shows the different usage of Type II and Type III. Type II is fine, and Type III is coarse; Type II is used for general resurfacing and sealing, and Type III is used for high-volume roadway resurfacing, rut filling, and high friction results. Limestone, slag, silicate, granite, flint, and basalt aggregates are typically used in micro-surfacing.

Table 7. Different Usage of Type II and Type III (Caltrans, 2003)

Application	Aggregate Type II	Aggregate Type III
Void Filling	•	
Wearing Course AADT < 100	•	
Wearing Course AADT < 1,000	•	•
Wearing Course AADT < 20,000	•	•
Minor Shape Correction 0.4 – 0.8 inch (10 – 20 mm)		•
Application Rates in lbs of dry aggregate per square yard	10 - 15	20 - 25

1) Mineral filler shall be any recognized brand of non-air entrained Portland cement or hydrated lime that is free from lumps. The type and amount of mineral filler needed shall be determined by a laboratory mix design and will be considered as part of the aggregate gradation. An increase or decrease of less than one percent (1%) may be permitted when the micro-surfacing is being applied if it is found to be necessary for better consistency or set times. Hydrated lime, fly ash, Portland cement, kiln dust, limestone dust, bag house fines, and crushed rock screenings can all be used as mineral filler, according to NCHRP.

2) There are three kinds of water in the asphalt mix: water in the asphalt emulsion, water in the mineral filler, and additional water. The additional water shall be potable and free of harmful soluble salts or reactive chemicals and any other contaminants so that the asphalt will not separate from the emulsion before the micro-surfacing is placed. The amount of water should be 6% to 11% by weight in case there are difficulties in construction due to too little or slow setting if too much.

3) Typical additives used in micro-surfacing include emulsifier solutions, aluminum sulfate, aluminum chloride, borax, ammonium sulfate, inorganic salts, and amines. They generally act as break control and are useful when temperatures rise during the day.

Construction Procedures and Considerations

Slurry Seal

Any hairline cracks should be sealed, but the sealant should not be excessive. Elevated areas should be milled. A tack coat should be applied immediately prior to the slurry seal. The slurry seal should be spread over the pavement in a continuous manner.

Weather Considerations

It is recommended that slurry seal be applied when the temperature is warm. The standard specification Section 427 provides seasonal limitations on the placement of slurry seal. The acceptable dates generally range from April to October, with the tightest time frames for the northern zones, and the widest range (April 1st to October 31st) for the southern zones. If there is moisture on pavement surfaces or in cracks or if rain is imminent, slurry seal should not be applied Figure 27 shows the operations of slurry seal.

Figure 27. Photo. Slurry Seal Operation (Source: Bill Evans, Ergon)

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the slurry seal operations. The work zone should be constructed according to GDOT requirements. The pavement should not be opened to traffic until the slurry seal has cured enough to withstand marring and tearing, and no water will be pumped to the surface (GDOT Specification Section 427).

Surface Preparation

Pavement surfaces should be dry and clean. Any cracks wider than 1/8 inch (3.00 mm) should be sealed.

Equipment Inspection

The machine to place the slurry seal should be checked for calibration, be relatively clean, and not have worn parts.

Slurry Seal Application

The specifications for installation temperatures should be followed. A tack coat should be applied to the pavement prior to the slurry seal. The surface should be pre-wetted by fogging, if necessary. The application rate and consistency of the slurry seal should be verified at the start of the process and periodically during the application.

Micro-Surfacing

Hairline cracks should be sealed but the sealant must not be excessive. Elevated areas should be milled. Minor ruts should be filled with a rut box as shown in the figure below using a leveling course. A tack coat should be applied immediately prior to placement. The pavement should be micro-surfaced uniformly in a continuous manner. Figure 28 shows the rut-filling box used to level ruts.

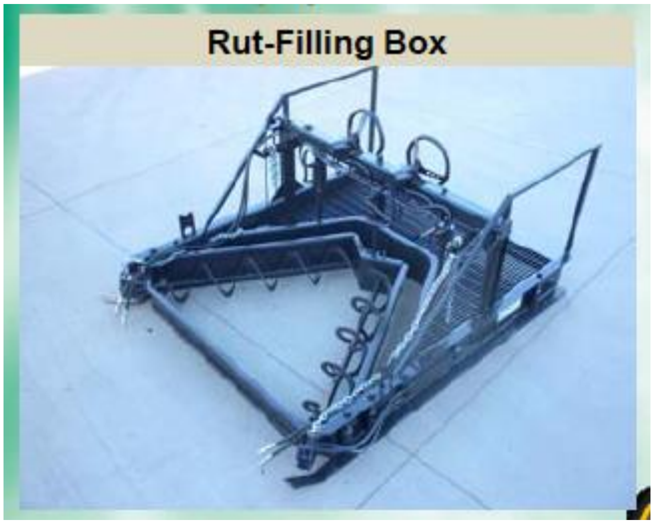


Figure 28. Photo. Rut-Filling Box Used to Level Ruts (NCPP, 2017)

Weather Considerations

It is recommended that micro-surfacing be conducted when the temperature is warm. The standard specification Section 428 requires a minimum ambient and pavement temperature. I If there is moisture on the pavement surface or in cracks or if rain or freezing temperatures are imminent, micro-surfacing should be accomplished Figure 29 shows the operation of micro-surfacing.

Figure 29. Photo. Micro-surfacing Operation (NCPP, 2017)

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the micro-surfacing operations. The work zone should be constructed according to GDOT requirements. The pavement should not be opened to traffic until the micro-surfacing has cured enough to withstand marring and tearing (GDOT Specification Section 428).

Surface Preparation

Pavement surfaces should be dry and clean. Any cracks wider than 1/8 inch (3.00 mm) should be sealed. Existing ruts should be filled with a leveling course. A rut box must be used for any ruts over 1/2 inch (12.70 mm). Ruts should be filled as noted in Figure 30.

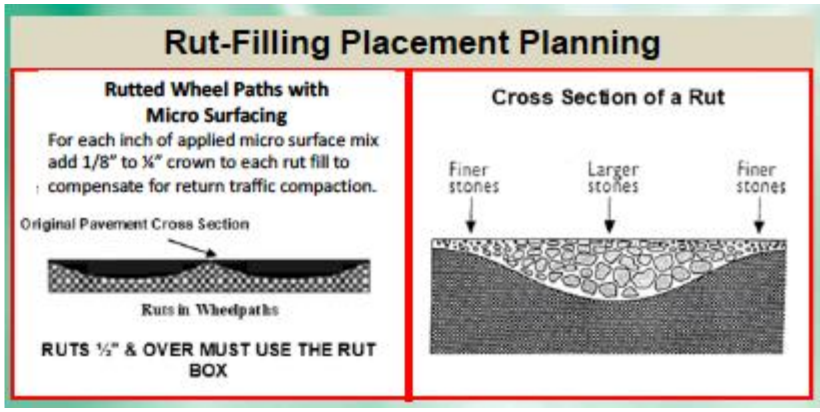


Figure 30. Graph. Rut-Filling Placement Planning (NCPP, 2017)

Equipment Inspection

The machine to applies the micro-surfacing should be checked for calibration, be relatively clean, and not have worn parts. Gauges or counters should be provided to check the output of materials, as shown in Figure 31:

Figure 31. Graph. Yield Checks (NCPP, 2017)

Micro-surfacing Application

Specifications for installation temperatures should be followed. A tack coat should be applied to the pavement prior to application of the micro-surfacing. The surface should be pre-wet by fogging, if necessary. The beginning application rate should be verified and checked consistency during the application. The mixture should be placed with a mechanical-type spreader box equipped with paddles or other devices to agitate and spread the material uniformly.

Performance

Slurry Seal

Pavement condition

Slurry seals are best for pavements that are still in relatively good condition but have oxidized due to age. A pavement with a PACES rating of 80 to 90 with Level 1 cracking (including load, block, and reflective cracking) would be most appropriate, but a pavement with a rating between 70 and 80 with predominately block cracking Level 2 or less would also be a suitable candidate. Pavements with Severity Level 3 or 4 load cracking should not be considered.

Construction Characteristics

Consistency of the mixture must be maintained during mixing and application. Lumping, balling, or unmixed aggregate will cause unacceptable performance. Segregation of the mixture or excessive liquid in the mixture will produce inconsistent performance.

Micro-Surfacing

Pavement condition

Micro-surfacing is best for pavements that are still in relatively good condition but have distresses beyond a slurry seal treatment. A pavement with a PACES rating of 75-85 with Level 1 or 2 cracking (including load, block, and reflective cracking) would be most appropriate, but a pavement with a rating between 70 and 80 with predominately block cracking Level 3 or less would also be a suitable candidate. Pavements with Severity Level 4 load cracking should not be considered.

Construction Characteristics

Rut filling and leveling must be performed at least 24 hours before micro-surfacing. Consistency of the mixture must be maintained during mixing and placement. Lumping, balling, or unmixed aggregate will cause unacceptable performance. Segregation of the mixture or excessive liquid in the mixture will produce inconsistent performance. The finished surface should have a uniform texture free of excessive scratch marks, tears, or other surface irregularities.

THIN OVERLAY

What is Thin Overlay?

Defined by the National Asphalt Pavement Association (NAPA), thin overlay is “a time-proven method of extending the life of pavement structures that are still in serviceable shape, which is 1.50 inch or less in thickness, and comprised of aggregate having a small nominal maximum aggregate size, generally 12.50 mm or less” (NAPA, 2012). Thin overlay is often carried out in conjunction with other corrective activities, such as milling and leveling. It is considered as pavement preservation because thin layer does not add structural capacity.

How to Select Project for Thin Overlay

Thin overlay is used to replace only the top, worn-out asphaltic layer; the underlying layer is in good condition. The rule of thumb for selecting a thin overlay project is the underlying layers must be structurally sound. A PACES rating of 70 is typically the trigger point for thin overlay. The following are guidelines for the selection of thin overlay projects:

1) Thin overlays should only be used on structurally sound pavements where minor defects are present without base failures; excessive stones have already been lost; or there are any other severe pavement defects.

2) Some distresses (including rutting, cracking (longitudinal or transverse, alligator cracking), potholes, raveling, polished aggregate, flushing/ bleeding, etc.) should be considered before using thin overlay.

3) Rutting: limited rutting (less than 1/2 inch (12.70 mm)). Rutting greater than 1/2 inch (12.70 mm) is a concern and requires milling.

4) Cracking: Load cracking Levels 1, 2, and 3 (limited) and block cracking Levels 1 and 2 can be considered for thin overlay. Limited load cracking Levels 3 and 4 can be considered for thin overlay with combined treatments, such as deep patching or milling.

5) Thin overlay projects that are not cost effective tend to be those performed on very poor pavements. The benefits also decrease if the existing pavement is still in excellent condition.

Therefore, when there are good structural conditions, selecting roads, including those with moderate or high severity levels, distresses for thin overlay maintenance is very important.

Bad and good candidates for thin overlay are shown in Figure 32 and Figure 33.

Figure 32. Photo. Bad Candidates for Thin Overlay

Figure 33. Photo. Good Candidate for Thin Overlay

Material Design

The types of HMA applied can be dense graded, open graded, and stone matrix mixes with or without high percentages of polymer-modified binders. This section focuses on dense graded layers and does not address other mix types, such as open-graded mixes. These materials and their construction requirements are described in Sections 400, 424, 824, and 828 of the Standard Specifications, 2013 Edition, in Special Provisions 400 and 828, issued February 24, 2006, and in Special Provisions 424 and 824 issued March 6, 2006. According to GDOT's Asphalt Pavement Selection Guidelines, mix types are distinguished according to nominal maximum size of the aggregate in millimeters, i.e. 9.5 relates to an aggregate size of 9.5 millimeters, or 3/8 inch. Selection guidelines for asphalt pavement are presented in two categories: low to medium volume and medium to high volume, as shown in Table 8.

Table 8. Selection Guidelines for Asphalt Pavement

Category	Traffic	Surface Type
Low to Medium	TPD < 100 or ADT < 800	BST
	TPD < 100 or ADT < 1,000*	4.75 HMA
	TPD < 200 or ADT < 2,000*	9.5 Type I HMA
Medium to High	TPD < 200 and 2,000 < ADT < 10,000	9.5 Type II HMA
	TPD > 200 or ADT > 10,000	12.5 HMA
TPD = trucks per day ADT = average daily traffic *Note: ADT is conservative and can be exceeded with accurate information on TPD		

Construction Procedures and Considerations

Weather Considerations

When the overlay is very thin, it may cool from both the bottom and the top. The cooling time is twice as fast for a thicker one. So, the time available for the contractor to obtain density has been reduced. For this reason, production temperatures may need to be greater for thin overlay. The asphaltic concrete OGFC mix or PEM should not be laid when air temperatures are below 55 °F (13 °C).

Traffic Considerations

A proper traffic control plan should be developed and maintained before and during the thin overlay operations. The work zone should be constructed according to GDOT requirements. The pavement can be opened to traffic after the mix has been compacted and the mat has cooled to 60 °C (140 °F) or to the agency's required temperature. It takes about 10-20 minutes for pavement using UTBWC to allow traffic.

Surface Preparation

Pavement surfaces must be clean and dry. The dust, dirt, and debris need to be removed using a road sweeper or power broom. Paving should not begin if rain is imminent. In addition, distresses (e.g., cracks, rutting, etc.) should be treated for maximum performance. Cracks greater than 7.9 mm (5/16 inch) should be sealed within 3.2 mm (1/8 inch) or flush with existing surfaces. Alligator cracks and potholes should be removed and patched. Rutting should be milled where it is the result of mix deficiencies and milled or leveled where it is due to wear or post construction consolidation.

Tack coat application

A uniform tack coat is essential for helping create a bond between the newly applied surface mix and the existing surface. Based on survey responses, tack application rates vary from 0.02 to 0.2 gal/yd^2 (0.1 to 0.96 L/m^2). The rate varies depending on whether the overlay is dense-graded mix or UTBWC and whether the tack coat is emulsion, asphalt cement, or a special “trackless” type tack coat. The rate may also depend on the type of surface being placed and whether the existing surface has been milled.

For example, Tennessee applies 0.08- 0.12 gal/ yd^2 on a milled surface and 0.05 -0.1 gal/ yd^2 on a non-milled surface; Kansas uses 0.03 gal/ yd^2 for 4.75-mm NMAS mix and 0.13 gal/yd^2 for UTBWC; and Louisiana uses 0.12 gal/yd^2 for OGFC. If an emulsion is used for tack coat, the specified rate is usually based on the residual asphalt amount. Georgia found that slippage may be more of a problem when emulsions are used during hot summer paving, and the agency has required asphalt cement for tack coat for the last 30 years. Of the milling speed alone, the combination of the milling machine speed, the drum rotation frequency, and the teeth inspection needed to be adjusted based on the underlying material to meet the RVD requirement. On an I-95 project, the milled materials accumulated on the teeth and resulted in a rough milled surface. A soap solution was added to the water sprayed on the front of the milling drum to clean the teeth. In addition, the contractor developed a daily routine to check and replace worn-out teeth to ensure a smooth micro-milled surface.

Rolling

There are several stages of rolling used for dense graded mixtures. Because thin layers lose temperature rapidly, the rolling temperatures must be strictly monitored. The stages for compaction include initial breakdown using a vibratory roller, kneading compaction using a pneumatic roller, and finishing using a static roller (Caltrans, 2003).

Performance and Limitations

Thin overlay is intended to replace the worn-out surface layer (less than 1 1/2 inch) when the underlying layer is still structurally sound. A pavement with a PACES rating less or equal to 70 with moderate cracking (e.g., Levels 1, 2, and 3 load cracking and Levels 1 and 2 block cracking) and limited rutting (< 1/2 inch) would be most appropriate, but a pavement with localized rutting or load cracking Levels 3 and 4 would also be a suitable candidate with proper deep patching. Pavements with severe rutting or structure damage should not be considered.

MICRO-MILLING

What is Micro-milling?

In 2007, GDOT developed a new pavement preservation method using micro-milling in conjunction with a thin overly to cost-effectively replace a porous asphalt layer (e.g., OGFC and PEM) on interstate highways without replacing or damaging an underlying layer (e.g., SMA) that is still in good condition. The micro-milling and thin overlay method was first applied on I-75 near Perry, Georgia. The project was approximately 15.6 centerline-miles (93.6 lane-miles with 3 lanes in each direction) and the estimated saving was more than $5 million (approximately $59,000 per lane-mile).

Georgia's interstate highways are commonly constructed with a porous asphalt layer (e.g., 3⁄4 inch of OGFC or 1 1⁄4 inch of PEM) on top of dense-graded or gap-graded asphalt pavements (Jared & Hines, 2014). When the porous asphalt layer wears out after 10 to 12 years, the common practice in Georgia has been to mill out the porous asphalt layer and approximately 3⁄4 inch to 1 1⁄4 inch of the dense-graded asphalt beneath it by using conventional milling (Jared & Hines, 2014; Lai et al., 2009; Lai, 2011; Lai et al., 2012). However, the milled dense-graded asphalt layer is typically still in good condition. Replacing only the porous asphalt layer directly over the milled surface has rarely been done because of two concerns: 1) the potential for poor bonding between the porous asphalt layer and the rough milled surface and 2) the potential for water to enter the porous asphalt layer and become trapped in the valleys of the rough milled surface. These situations could cause delamination of the porous asphalt layer. Therefore, the practice was to also mill and replace (typically 1 1⁄2 inch) of the underlying asphalt mixture to provide a smooth surface to bond with the porous asphalt layer.

Micro-milling has been used by states to correct roughness and/or to provide a comfortable ride on a milled surface before it is overlaid. Compared to the conventional milling, micro-milling uses a milling drum with tighter spacing (2/10 inch), more teeth (700-1,000), and a special pattern to produce a finer and smoother texture on the milled surface. Figure 34 shows the pavement textures on conventionally milled and micro-milled surfaces. Currently, only a few states have developed or are developing formal specifications. These states include New York, Rhode Island, Georgia, Tennessee, Alabama, Virginia, Indiana, South Carolina, and California (Asphalt Contractor, 2011). GDOT has taken a step further to develop Specification Section 432 to ensure the quality of the micro-milled pavement surface texture. A new performance indicator, ridge-to-valley-depth (RVD), was developed through research studies to control the quality of the micro-milled surface texture. Today, GDOT has applied micro-milling and thin overlay on I-75, I-95, I-285, etc., to cost-effectively replace the worn-out porous layer (e.g., OGFC and PEM). Figure 34 shows the difference between conventional-milled surface and micro-milled surface.

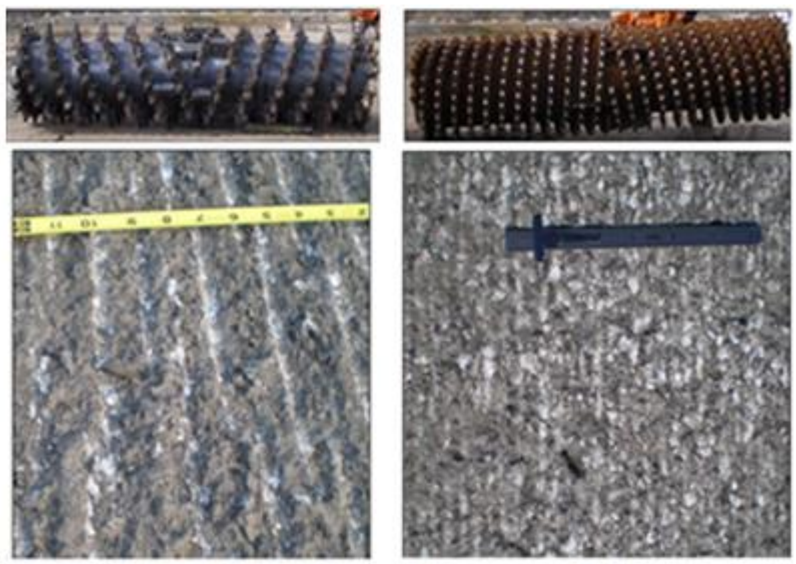

(a) Conventional- milled surface (b) Micro-milled surface

Figure 34. Photo. Conventional Milling vs. Micro-milling

How to Select Projects for Micro-milling

Micro-milling and thin overlay is used to replace only the top, worn-out porous asphalt layer (e.g., OGFC and PEM) when the underlying layer (e.g., SMA) is in good condition. The rule of thumb for selecting micro-milling and thin overlay projects is the underlying layers must be structurally sound. The following are guidelines for the selection of micro-milling projects:

1) The porous layer (e.g., OGFC and PED) wears out. For example, extensive raveling on the surface layer.

2) Three factors are used to check for micro-milling suitability: core, rutting, and the Hamburg test.

3) Rutting: limited rutting (less than 1⁄4 inch). Rutting greater than 1⁄4 inch is a concern.

4) Core: the deformation of each layer must be measured to identify the underneath pavement structure.

5) Hamburg test: 12.50 mm is the failure threshold.

Material Design

Approved materials for GDOT are located on GDOT's Qualified Products List (QPL) and defined in GDOT's Specification Section 400. Note that besides the material, the quality of the micro-milled pavement surface texture is critical for the performance of micro-milling and thin overlay. GDOT Specification Section 432, "Mill Asphalt Concrete Pavement," (GDOT 2005) is used for micro-milling.

Construction Procedures and Considerations

Weather Considerations

There is no specific weather consideration for micro-milling operations other than non-rainy days. The asphaltic concrete OGFC mix or PEM should not be laid when air temperatures are below 55 °F (13 °C).

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the micro-milling and thin overlay operations. The work zone should be constructed according to GDOT requirements. Note that GDOT allows the micro-milled surface to be opened to traffic as a temporary riding surface for up to 3 days before being paved, except in rain event days where construction is not permitted.

Surface Preparation

Distresses that are deeper than the micro-milled depth need to be repaired to maximize the pavement service life and performance. The isolated spots with severe and deep cracks should be deep patched before the micro-milling operation.

Micro-milling Operations

A fine, smooth micro-milled surface texture is the key for providing good bonding between the porous asphalt layer and the milled surface. Thus, a test section is used to ensure the quality of micro-milled surface.

Micro-milling on a test section

Prior to construction, the contractor should perform micro-milling on a 1,000-ft test section with a uniformly textured surface and cross section that is approved by the project engineer. The test section allows contractors to adjust the operations, such as milling speed and drum speed (revolution per minute, RPM), to achieve a smooth micro-milled surface texture. When the requirements (including RVD and HCS IRI) are exceeded in the test section, work is halted, and the contractor must submit a plan detailing the steps/actions to be taken. If approved by GDOT's engineer, the contractor will use another 1000-ft test section to achieve the requirement. The failed test section will also be re-milled to achieve compliance with the requirements.

Micro-milling operation

During the micro-milling operation, the micro-milling depth is measured frequently using a gauge, and the quality of the milled surface is visually inspected. The engineer also checks the slope and cross slope using a straight edge to ensure the milled surface aligns with the design. Figure 35 shows the operation of micro-milling.

(a) Micro-milling (b) Checking the micro-milled surface

Figure 35. Photo. Micro-milling Operation

The following are the observations and lessons learned from previous projects:

1) The micro-milled surface texture is rougher on the gap-graded SMA than on the dense-graded HMA. However, stringent requirements were met on both projects.

2) A variable milling depth, instead of “fixed” depth, is used as a standard for the micro-milling and thin overlay method. This is to ensure the entire depth of OGFC/PEM can be removed and no residuals (or scabs) are left on the micro-milled surface.

3) Instead of the milling speed alone, the combination of the milling machine speed, the drum rotation frequency, and the teeth inspection must be adjusted based on the underlying material to meet the RVD requirement. On the I-95 project, the milled materials accumulated on the teeth and resulted in a rough milled surface. A soap solution was added to the water sprayed on the front of the milling drum to clean the teeth. In addition, the contractor developed a daily routine to check and replace worn-out teeth to ensure a smooth micro-milled surface.

Cleaning a micro-milled surface

A clean milled surface free of loose aggregates, dirt, and dust is essential to ensure adequate bonding of the binder to the existing base layer. A cleaning process should immediately follow the micro-milling operation to remove loose aggregates and dust from the micro-milled surface. The contractor should sweep the surface dust using a power broom. Compared to cleaning a conventionally milled surface, the broom has to move slower to clean the micro-milled surface because the particles from the micro-milling operation are small. Figure 36 shows the operation of cleaning a micro-milled surface.

Figure 36. Photo. Cleaning a Micro-milled Surface

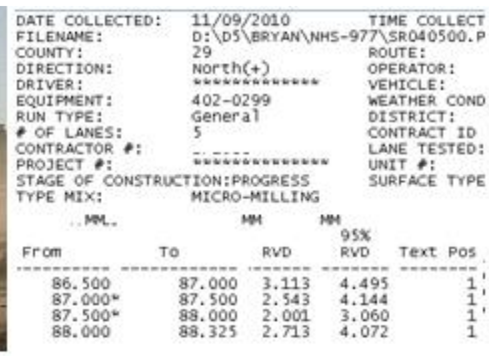
Quality control on micro-milled surface

The RVD and IRI were measured on micro-milled surfaces using the laser road profiler to ensure quality. RVD is defined as the difference in height between the ridge (highest) and valley (lowest) points within the base length of 100.0 mm. Figure 37 shows GDOT's laser road profiler measuring the RVD and IRI. The following are the requirements for the micro-milled pavement surface texture:

1) Any areas exceeding 1/8 in (3.2 mm) between the ridge and valley of the mat surface or failing to meet pavement surface acceptance testing using the Laser Road Profiler shall require the underlying layer to be removed and replaced with material as directed by the engineer at no additional cost to GDOT. All corrective work shall be performed on a minimum 1000-ft section.

2) The indices for the smoothness of the milled surface measured have a target value of 825 mm/km and must not exceed the correction index of 900 mm/km.

3) The cross slope must be uniform, and no depressions or slope misalignments greater than 1/4 inch per 12 ft. (6 mm in 3.6 m) can exist when the slope is tested with a straightedge placed perpendicularly to the center line.



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DIRECTION:          North(+)            OPERATOR:
DRIVER:             **************      VEHICLE:
EQUIPMENT:          402-0299            WEATHER COND
RUN TYPE:           General             DISTRICT:
# OF LANES:         5                   CONTRACT ID
CONTRACTOR #:       _._____             LANE TESTED:
PROJECT #:          ***************     UNIT #:
STAGE OF CONSTRUCTION:PROGRESS          SURFACE TYPE
TYPE MIX:           MICRO-MILLING
     ..MM..              MM       MM
                                  95%
 From          To        RVD      RVD    Text Pos
---------- ----------- ------- ------- --------
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    87.000*     87.500   2.543   4.144        1
    87.500*     88.000   2.001   3.060        1
    88.000      88.325   2.713   4.072        1
```

Figure 37. Graph. Measure RVD Using GDOT's Laser Road Profiler

Paving operation

After the micro-milled surface was cleaned, a tack coat using straight PG binder was applied, and an OGFC layer was placed on top. The tack coat rate of approximately 0.08 gal/yd2 was suggested for micro-milled surfaces for PEM overlay paving. Following the tack coat application, the PEM was placed on top and compacted using a rubber-tired roller and finished with a steel wheel roller.

Performance and Limitations

Pavement condition

Micro-milling and thin overlay have replaced the worn-out OGFC/PEM while the underlying layer is still structurally sound. A pavement with a PACES rating of 70 to 80 with Levels 1 and 2 raveling and limited rutting (< 1⁄4;) would be most appropriate, but a pavement with localized rutting would also be a suitable candidate with proper deep patching. Pavements with severe rutting or structure damage should not be considered.

Micro-milling surface quality

A fine, smooth micro-milled surface texture is the key for micro-milling and thin overlay method. It provides good bonding between the porous asphalt layer and the milled surface and prevents water from being trapped in the valleys of the milled surface, which could result in delamination and premature distresses. Therefore, it is essential to use a test section, a variable mill depth, a quality control based on Specification Section 432 to achieve a smooth micro-milled surface texture.

WHITE TOPPING

What is White Topping?

White topping allows for an alternative to asphalt overlays for existing asphalt pavements, especially where recurring rutting or shoving has been demonstrated or is expected due to truck loadings (either experienced or expected). White topping in Georgia has commonly been associated with intersection areas that have recurrent or excessive rutting due to stopping and turning movements from heavy trucks, but the method can be used for any length of pavement. The word “white topping” has been replaced in recent literature with the terms “unbonded concrete overlay over asphalt” (UCOA) or “bonded concrete overlay over asphalt” (BCOA). UCOA was known as conventional white topping and typically is now basically designed like a new pavement on a stable base, so the thickness can range from 4-11 inch. BCOA was once termed ultra-thin white topping, and it typically is 2-6 inch in thickness.

How to Select Projects for White Topping

Asphalt intersections or asphalt pavements with mild to severe distresses from rutting and shoving (including PACES Severity Level 3 corrugation/pushing) are ideal candidates, shown in Figure 38, but it can be used on almost any type of asphalt pavement if properly designed.

Figure 38. Photo. Ideal Candidates for White Topping (CP Tech Center, 2017)

For the purposes of this pavement preservation guide, the focus will be on ultra-thin white topping (now called BCOA) for intersections. Due to the presences of curbs and gutters in intersecting roadways and businesses commonly located at intersections, milling the existing asphalt and replacing it with concrete is the typical method of construction of BCOAs in intersections. In this case the best candidate is an intersection that has experienced rutting in the wheel paths or through the intersection. Intersections that have been milled or overlaid more than once due to rutting are prime project candidates. It is essential that a minimum amount of asphalt remains after milling; therefore, cores should be taken in the intersection prior to considering an intersection site for BCOA. The Pavement Design section of the Office of Materials and Testing should also be consulted for a thickness design based on the anticipated truck traffic at the intersection. Based on a typical BCOA of 4 inch, a minimum of 6 inch of asphalt should exist, although 7 inch is preferable. Joints are cut into the pavement just like concrete pavement, but due to the differences in shapes of intersections, the location of the joints need special attention. Intersection BCOAs need to be laid out prior to construction to make sure that the joint spacing is as consistent as possible and to address special areas like tapers of turning lanes and turning radius corners. Joints should not be placed in wheel paths and should provide for square panels as much as possible. Panel size should be proportional to the concrete/panel thickness, i.e. 4 inch panels should be placed so that the panel size is 4 ft by 4 ft.

Material Design

The materials for concrete overlays are the same materials used for concrete pavements or concrete ramps, except that structural (polypropylene) pavement fibers are also used to increase the toughness of the thin slab. The fibers do not increase the strength of the concrete, but they do assist in holding the panels together. This also helps control shrinkage and crack growth in the panels (CP Tech Center, 2017). Concrete mix design requirements, material requirements, and fiber specifications are included in Special Provision 453.

Construction Procedures and Considerations

Weather Considerations

As with any paving operation, construction should not be planned when inclement or rainy weather is anticipated. Due to the thin concrete layer that is applied, it is also detrimental to construct the BCOA when the temperature is extremely hot, like in the middle of a typical Georgia sunny summer day. The thin slabs can curl and warp due to high temperature or moisture differentials as they are curing, potentially causing early cracking. Early morning or late evening construction would be preferable.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the BCOA construction. The work zone should be constructed according to GDOT requirements.

Construction may be best planned for nighttime or weekend work to minimize disruptions and provide for optimum weather conditions.

Surface Preparation

Pavement should be milled to the depth of the BCOA thickness, typically 4 inch. There should be no isolated areas of distress after milling; areas should be checked for soundness and repair if necessary. The milled surface should be cleaned and wetted just prior to concrete placement. The intent is for the concrete to bond to the milled asphalt.

Performance and Limitations

Pavement Considerations

Concrete can bridge minor soft spots but will crack if the foundation is too soft. Non-uniformity of the pavement area prior to concrete placement will result in cracking. Therefore, any deep patching should be performed prior to concrete placement, and the milled asphalt should be as clean and even as possible. The surface should not have any ponded water prior to concrete placement.

Joint Layout

Joints should be laid out in a pattern of small square panels. Longitudinal joints should not be placed in the wheel paths, and the size of the squares should be less than 1.5 times the thickness of the concrete overlay. Steel dowels or tie bars should not be used due to the thinness of the concrete (CP Tech Center, 2004).

Finishing and Curing

The concrete should be placed, vibrated, and finished to grade so that a smooth surface meets the straightedge requirements according to GDOT specifications. Curing the concrete uniformly with curing compound should be accomplished.

POTHOLE PATCHING

What is Pothole Patching?

Pothole patching is the process of filling potholes in the asphalt pavement. Pothole is the localized distress in an asphalt-surfaced pavement. Because of the action of climate and traffic on the pavement, some pieces of asphalt pavement become weakened. Then they are removed under the action of traffic, so a pothole occurs (Wilson & Romine, 1994). The pothole patching helps control further deterioration and expensive repair of the pavement.

GDOT mainly use four types of the pothole patching: cold-mix patching, hot-mix patching, mastic patching, and spray-injection patching. The differences are the materials used. Spray-injection patching blasts the asphalt into the pothole to avoid the additional compaction.

How to Select Projects for Pothole Patching

Distresses Suitable for Repair by Patching

For the most part, the definitions of the distress terms used in this paper correspond to those in the Distress Identification Manual for the Long-Term Performance Program, commonly referred to as the DIM. Many states use this manual and, therefore, they are familiar with the terminology.

In asphalt pavements, the most common distresses that can be repaired by patching include potholes, deterioration around cracks, delamination, rutting, or raveling. The DIM does not include deterioration around a crack as a distress, but it does include high severity cracking that can describe this type of deterioration. In this document, the term “delamination” refers to the separation of one layer of an asphalt pavement from the underlying layer; some refer to this as "peeling." In the DIM, this would be categorized as a pothole. One distinguishing feature of a delamination versus a pothole is that a delamination has a flat bottom at the top of the underlying layer, whereas a pothole is bowl-shaped. Figure 39 illustrates potholes and Figure 40 illustrates delamination.

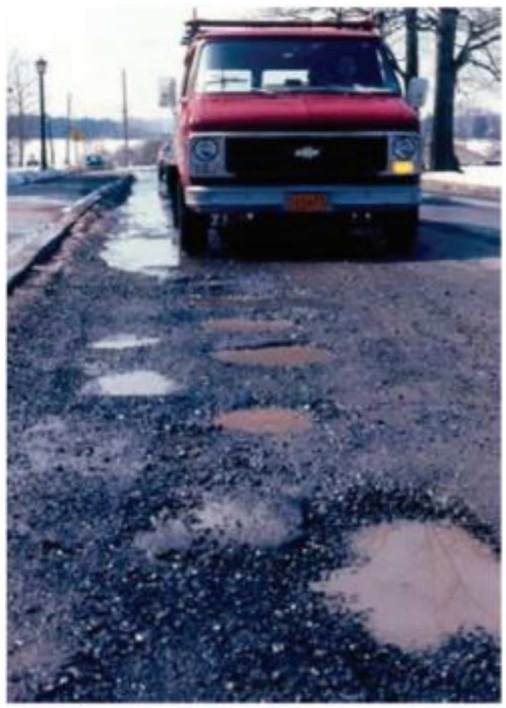


Figure 39. Photo. Potholes Caused by Poor Drainage

Figure 40. Photo. Delamination of Surface Layer

In concrete pavements, some of the distresses that can be addressed through patching include deterioration around cracks (such as durability “D” cracks, map cracking, and longitudinal or transverse cracks), scaling, pop outs, and blowups. In addition, jointed concrete pavements may experience joint spalling, corner breaks, faulting, and damage caused by water pumping that can be repaired by patching. Continuously reinforced concrete may be subject to punch-outs, as well. As with asphalt pavements, the DIM does not consider deterioration around cracks as a distress type on concrete pavement; however, deterioration around a crack could be considered analogous to spalling at a joint.

Material Design

Hot-mix

The types of HMA applied can be dense graded, open graded, and stone matrix mixes with or without high percentages of polymer-modified binders. This section focuses on dense graded layers and does not address other mix types, such as open-graded mixes. These materials and their construction requirements are described in Sections 400, 424, 824, and 828 of the Standard Specifications, 2013 Edition, in Special Provisions 400 and 828 issued February 24, 2006, and in Special Provisions 424 and 824 issued March 6, 2006.

According to GDOT's Asphalt Pavement Selection Guidelines, mix types are distinguished according to nominal maximum size of the aggregate in millimeters (i.e. 9.5 relates to an aggregate size of 9.5 mm, or 3/8 inch). Table 9 shows the selection guidelines for asphalt pavement presented in two categories, low to medium volume and medium to high volume.

Table 9. Selection Guidelines for Asphalt Pavement

Category	Traffic	Surface Type
Low to Medium	TPD < 100 or ADT < 800	BST
	TPD < 100 or ADT < 1,000*	4.75 HMA
	TPD < 200 or ADT < 2,000*	9.5 Type I HMA
Medium to High	TPD < 200 and 2,000 < ADT < 10,000	9.5 Type II HMA
	TPD > 200 or ADT > 10,000	12.5 HMA

TPD = trucks per day
ADT = average daily traffic
*Note: ADT is conservative and can be exceeded with accurate information on TPD

Cold-mix

Cold-mixes for patching use the same designation as regular asphaltic concrete mixtures (i.e. 9.5 mm), but there may be slightly different requirements; therefore, the cold mix specification 401 should be referred to for the required composition of cold-mix types.

Mastic Patching

Requirements for mastic patching materials are found in Specification 827. Materials used should provide a well-bonded, flexible, durable, and traffic resistant repair when properly applied.

Spray Injection

Emulsion: When the surface temperature is more than 50 °F, R rapid set CRS-2 or HFE150 should be used. When the surface temperature is less than 50 °F, M medium set CMS-1 or 2 should be used.

Aggregates: Standard state spec. rock for chip sealing has a common size of 1/4 inch, 3/8 inch, 1/2 inch, or 5/8 inch. For rideability and making a tight mix, smaller sizes should be used, except for shoulders.

Construction Procedures and Considerations

Cold-mix

Weather Considerations

The recommended temperatures for aggregate and bituminous materials to ensure proper mixing are CMS-2: 140-160 °F (60-70 °C); PG 64-22: 300-350 °F (150-175 °C); MC-250: 100-225 °F (40-105 °C); and Aggregates 200-225 °F (95-105 °C).

Traffic Considerations

Pothole patching is typically a relatively quick process; therefore, it should be performed using temporary traffic control measures. In high-traffic areas, construction may be best planned for nighttime or weekend work to minimize disruptions, but since pothole repair can sometimes be time-sensitive (depending upon the size and location of the pothole), that is not always possible.

Surface Preparation

The pothole should be cleaned and all loose debris removed. In most cases, the area should be dry before filling. Application of a light amount of liquid tack may be performed before filling the pothole.

Mastic Patching

The pavement patching mastic component material is combined into a melter/applicator pot, heated, mixed, and applied to the area requiring repair as recommended by the manufacturer. The material should be heated to no greater than 410 °F. The equipment should produce and maintain a homogenous mixture of modified asphalt binder and aggregates at a uniform temperature without gradation-related or temperature-related segregation in the pavement patching mastic. The material must provide a repair that can be opened to traffic once the pavement patching mastic has cooled and solidified.

Spray Injection

The spray injection method uses specialized trailer- or truck-mounted equipment to blow water and debris from the pothole, spray a tack coat into the hole, blow asphalt and aggregate together into the hole, then cover the patch with a layer of aggregate. Because the aggregate and emulsion are propelled into the patch area with high pressure air, no further compaction is necessary. Spray patching can be used on asphalt or concrete pavements and is sometimes done under adverse conditions because of the speed with which it can be accomplished.

Performance and Limitations

Patches may fail because of the materials used, installation issues, or simply because the roadway continues to deteriorate. The patches themselves may fail, as described by Anderson et al. in 1988 and shown in Table 10.

Table 10. Failure Symptoms and Mechanisms (Anderson et al. 1988)

Symptom	Failure Mechanism
In Stockpile	
Poor Workability	Binder too stiff; excessive fines or dirty aggregate; mix too coarse or too fine
Binder Draindown	Binder too soft; stockpiled or mixed at high temperatures
Stripping	Inadequate binder coating during mixing; cold or wet aggregate
Clumpy Mixture	Binder cures prematurely
Cold Weather Stiffness	Significant binder susceptibility to temperature; excessive fines or dirty aggregate; mix too coarse or too fine
During Placement	
Poor Workability	Binder too stiff; excessive fines or dirty aggregate; mix too coarse or too fine
Poor Stability	Binder too soft or excessive binder; insufficient voids in mineral aggregate; poor aggregate interlock
Excessive Softening (when used with hot box)	Binder too soft
In Service	
Pushing, Shoving	Poor compaction; binder too soft or excessive binder; significant binder susceptibility to temperature; contaminated mixture; slow curing rate; moisture damage; insufficient voids in the mineral aggregate; poor aggregate interlock
Dishing	Poor compaction
Raveling	Poor compaction; binder too soft; poor mixture cohesion; poor aggregate interlock; aggregate binder absorption; moisture damage; excessive fines or dirty aggregate; mix too coarse or too fine
Freeze-Thaw Deterioration	Mix too permeable; poor mix cohesion; moisture damage
Poor Skid Resistance	Excessive binder; aggregate not skid resistant; gradation too dense
Shrinkage or Lack of Adhesion to Sides of the Hole	Poor adhesion; tack coat not used or mix not self-tacking; poor hole preparation

The study also found that the proprietary cold mix materials performed significantly better than a local patching mix. Spray injection was not found to be cost-effective.

OPEN GRADED INTERLAYER

What is Open Graded Interlayer?

Open graded interlayer (OGI) is an open graded mixture placed at a lift thickness that allows for stone-to-stone contact of the mixture. This provides a stable mixture while also allowing for an open mix. The mixture is designed to provide 18 – 23% in-place air voids in the mix. It is also a highly interconnected void structure that will allow for the dissipation of stresses and strains within the interlayer without transferring the stress into the surface layer. GDOT uses a bituminous plant-produced asphaltic concrete OGI. The mixture can serve as asphaltic concrete leveling course over irregular surfaces. OGI not only mitigates existing cracking within asphaltic concrete pavements, but it also serves many purposes, including adding a limited structural benefit and providing a leveling course.

How to Select Projects for Open Graded Interlayer

OGI is used as an interlayer for crack relief when milling and inlaying highly cracked pavements. Interlayers are intended to reduce reflective cracking from underlying pavement by dissipating the stress and movement from the underlying cracking. Projects that are candidates for an interlayer typically have higher levels of Types 2 and 3 cracking or minimal levels of Type 4 cracking. Heavily cracked pavements should be appropriately evaluated for more extensive rehabilitation prior to placing an interlayer, as not all movement can be arrested with an interlayer treatment. Figure 41 shows OGI project before OGI and after OGI.

Figure 41. Photo. OGI Project (a) Before OGI and (b) After OGI (GDOT)

Material Design

The OGI mixture shall be formulated to contain approximately 18 to 23 percent in-place air voids after compaction. Approved mixtures that meet the mixture control tolerances and design criteria should be used. Detailed criteria are in Specification 415.

Construction Procedures and Considerations

Weather Considerations

The use of emulsion has specific weather and seasonal limitations. Ambient temperatures should not be less than 50 ºF (10 ºC) for 48 hours immediately prior to application, and there should be no forecast of ambient temperature less than 50 ºF (10 ºC) for 48 hours immediately following application. The ambient temperature and the road surface temperature should be at least 60 ºF (16 ºC) and stable at the time of application. No exceptions are permitted, except as authorized by an engineer. Asphalt cement should not be applied to a wet surface.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the OGI construction. The work zone should be constructed according to GDOT requirements. Construction may be best planned for nighttime or weekend work to minimize disruptions and provide for optimum weather conditions.

Surface Preparation

Pavement may be milled to the depth anticipated for the open graded interlayer. Excessive dirt or contaminants should be removed from the surface.

Performance and Limitations

Weather and seasonal conditions can affect the performance of the open graded interlayer. As with any type of asphalt layer, if an emulsion is used for tack, the OGI should not be placed until the tack emulsion has completely set. Existing pavement temperatures can adversely affect the timing of the emulsion set. Traffic should not be placed on OGI directly, if possible; instead, the work should be planned so that the surface course is applied before the road is released to traffic if possible.

FULL DEPTH RECLAMATION

What is Full Depth Reclamation?

Full depth reclamation (FDR) is a major rehabilitation option for severely distressed pavements that recycles the existing material in place to form a new, stronger base. FDR involves pulverizing, mixing, and compacting an existing asphalt surface, the underlying base, and, potentially, a small portion of the subgrade to create a base for a new pavement. The base can then be paved with an asphalt or concrete surface layer. Stabilizing materials, such as lime, cement or asphalt, are typically mixed into the FDR material to improve the stability of the new base.

How to Select Projects for Full Depth Reclamation

Asphalt pavements with severe distress from fatigue cracking (PACES Severity Level 3 or 4 cracking or bottom-up cracking), severe rutting and shoving (including PACES Severity Level 3 corrugation/pushing), or numerous weak spots (typically showing up as numerous patches) are ideal candidates. Figure 42 shows the pavement distress suitable for FDR.

a) Alligator (fatigue) cracking b) Excessive patching

Figure 42. Photo. Pavement Distress suitable for FDR (CP Tech Center)

The limit of a typical FDR machine is 18 inch, so distress in a pavement caused due to issues deeper than 18 inch will not be addressed with FDR. FDR can be used for asphalt pavements thicker than 10 inch, but pavements that thick are typically milled down to at most 10 inch before FDR is performed. In those cases, the pavement is milled before performing the FDR, and the milled asphalt is used as RAP in other pavements or in the final surface layer. Pre-milling could also address distress issues that are slightly lower than 18 inch into the pavement. FDR can also be used to increase the ability of a pavement to withstand higher loading than the pavement's original design by increasing thickness or strength of the pavement layers.

Material Design

It is necessary to use some type of stabilization additive in FDR. The amount and type of stabilizer material needed is based on the existing in-place material properties and is designed in a laboratory setting, similar to an asphalt or concrete mix design. Guidelines on how to perform the mix design can be found in industry or research reports (ref: CP Tech Center for cement and

UCPRC or CDOT for asphalt/bituminous). In-place materials that are too fine or too plastic even after mixing may not be good candidates for FDR. While most additives work well with the in-place asphalt, some additives work better with certain types of in-place base or subgrade materials. High sulfate soils, high organic content soils, or low pH soils can affect the hydration of cement additives. GDOT recommends a minimum pH value of 4.0 for soils that will be incorporated into an FDR layer.

Construction Procedures and Considerations

Consistency in mixing, compacting, and curing the FDR treated pavement is important in the ultimate success of FDR. As noted earlier, the asphalt can be milled prior to FDR, or FDR can be performed directly from the surface. A road reclaimer is used to dig up and mix the pavement, as shown in the Figure 43. The reclaimer mills up and mixes the in-place material. The stabilizer additive can be added during the mixing. The additive can also be added after initial mixing and then the material is remixed. Proper mixing will provide a uniform, consistently sized material. Compaction of the remixed material is necessary and is performed using typical compaction equipment (i.e. sheepsfoot, vibrating, and/or pneumatic rollers). The compacted surface should be moist cured or sealed with a bituminous type sealer. Figure 43 shows the schematic of a roadway reclaimer.

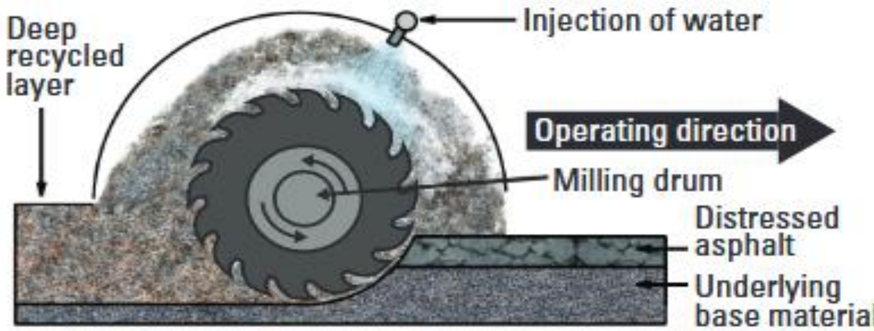


Figure 43. Graph. Schematic of a Roadway Reclaimer (CP Tech Center, 2017)

Weather Considerations

As with any paving operation, construction should not be planned when there is inclement or rainy weather anticipated. In addition, minimum temperature restrictions may be necessary to allow for the proper compaction and curing of the FDR base. The GDOT specification for specific weather or temperature restrictions should be consulted.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the FDR construction. The work zone should be constructed according to GDOT requirements. Construction may be best planned for nighttime or weekend work to minimize disruptions and provide for optimum weather conditions. Using cement in the mix-design can assist in providing earlier strength gain for opening to traffic sooner.

Surface Preparation

Pavement may be milled to the depth anticipated for the FDR, or the FDR can take place from the surface. Excessive dirt or contaminants should be removed from the surface.

Performance and Limitations

Drainage Issues

FDR can be used to improve cross-slope but does not address more general drainage issues. Drainage should be improved prior to performing FDR.

Improper Mix Design or Inadequate Mixing or Compaction

Improper mix design can prevent recycled materials from achieving the desired consistency and strength. Laboratory testing using actual in-place materials is imperative in achieving a proper mix design. Inadequate mixing or compaction can also affect strength and consistency. Quality assurance procedures, such as in-place density or coring, is necessary to ensure that the material has been handled properly.

Inadequate Finishing and Curing

Allowing traffic on the surface too soon or inadequate treatment of the in-place mixture can cause rutting, raveling or inability to get proper smoothness on the final surface mix.

HOT IN-PLACE RECYCLING

What is Hot In-place Recycling?

Hot in-place recycling (HIR) is a potential in-place alternative to milling and inlaying an asphalt pavement to improve surface conditions. HIR only addresses surface distresses that are within the top few inches of the surface. HIR involves heating, scarifying or milling the surface, mixing, and relaying an existing asphalt surface on site using the in-place asphalt material. A rejuvenating material is applied during the mixing process. Additional aggregate may need to be added in the mixing step if the existing surface mix is already very fine, as the scarification process reduces the gradation of the asphalt mixture. Additional asphalt may also need to be added if the existing surface mix is already very highly oxidized, as the rejuvenators may not provide adequate binder alone in this case. The HIR treated surface can be left as the final surface or an asphalt overlay can be placed over the finished HIR. Three different processes are identified for HIR:

1) Surface recycling - (3/4 to 1 1/2 inch depth) the pavement is first heated to soften it, the surface is scarified or milled and left on the surface, and the material is mixed with a rejuvenator and picked up and placed and compacted with typical asphalt paving equipment.

2) Remixing - (up to 3 inches depth) similar to surface recycling except that additional aggregate, asphalt, or both are added in the mixing process.

3) Repaving - surface recycling and a new asphalt overlay are performed in a single-pass method, and the HIR and new asphalt are compacted at the same time.

Figure 44 shows HIR project before HIR and 7 years after HIR.

(a) (b)

Figure 44. Photo. HIR Project (a) Before HIR and (b) 7 Years Later (NCAT, 2011)

How to Select Projects for Hot In-place Recycling

Asphalt pavements with distresses related to oxidization or non-structural raveling are ideal candidates. The pavements may have minor distresses from cracking (PACES Severity Level 1 or at most Level 2 block cracking), minimal rutting and shoving (PACES Severity Level 1 rutting), or very minimal patching. Pavements with high severity fatigue (alligator) cracking or structural distress, like base or subgrade failures, are not suitable candidates for HIR. Asphalt pavements that have had multiple crack seal treatments, also, may not be suitable projects, depending upon the material used for crack sealing, since the heating process can affect the sealant to the point of making it flammable or cause excessive smoke during the heating process. The type of surface mix in place can also retard the heating effect; therefore, HIR should also not be used on rubber modified mixes or open-graded surface mixes. Figure 45 shows the pavement distress suitable for HIR.

Figure 45. Photo. Pavement distress suitable for HIR (NCAT, 2011)

Since the HIR process involves heating the pavement, roadways with very shallow utilities or roadways with bushes or other flammable materials, or landscaping hanging over or close to the roadway could pose a hazard. Figure 46 shows good candidates for HIR.

Figure 46. Photo. Good candidates for HIR

Material Design

It is essential that a mix design be performed prior to actual construction. The mix design should use the in-place material and address the use of a rejuvenating agent and/or any additional aggregate or asphalt or an asphalt modifier (polymer). Any additional material must come from GDOT approved sources or be approved by the Office of Materials and Tests prior to use. Special Provision 403 addresses the minimum performance criteria that is expected for the mix design.

Construction Procedures and Considerations

Consistency in heating, mixing, and compacting the HIR treated pavement is important in the ultimate success of HIR. The heating units need to provide the temperature necessary to soften the pavement to the depth desired while not damaging the asphalt, the aggregate, or the surrounding properties. The mixing equipment needs to also provide heat for the mixing process and have the capability to accurately blend additional materials at the proper rate (such as added asphalt or aggregate). The paver needs to be able to perform like a typical asphalt paver, consistently distribute the material along the screed, control cross slope and longitudinal grade, and provide a uniform final surface. Controlling smoothness of HIR surfaces that are not going to be overlaid with a new asphalt surface is important and has been identified as a construction issue in the past. Figure 47 shows the typical sequence for HIR processes.

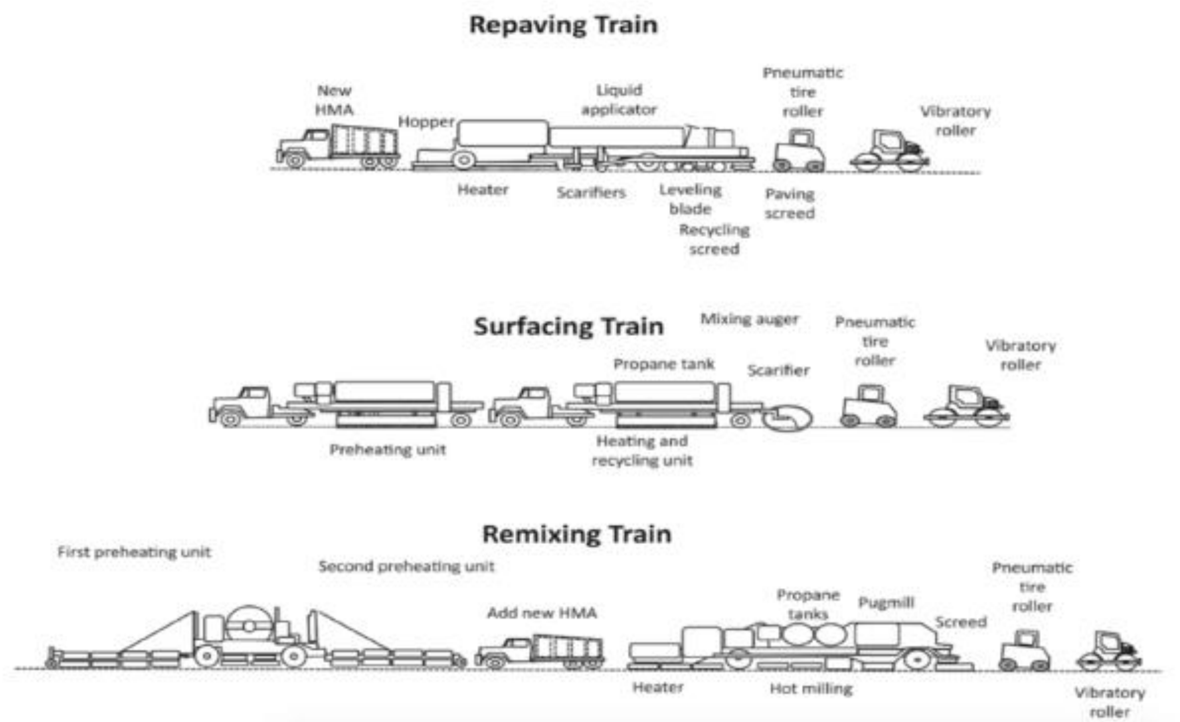


Figure 47. Graph. Typical Sequence for HIR Processes (NCHRP 421, 2011)

Weather Considerations

As with any paving operation, construction should not be planned when there is inclement or rainy weather anticipated. In addition, minimum temperature restrictions may be necessary to allow for the proper heating and softening of the asphalt surface. The GDOT specifications for specific weather or temperature restrictions should be consulted.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the HIR process. The work zone should be constructed according to GDOT requirements. Construction may be best planned for weekend work to minimize disruptions and provide for optimum weather conditions.

Surface Preparation

Excessive dirt or contaminants should be removed from the surface. Anything that could affect the milling/scarifying operation should be removed prior to construction (i.e rpms or thermoplastic paint markings).

Performance and Limitations

Poor Project Selection Criteria

Structural deficiencies of the pavement will not be remedied using HIR, and depending upon the severity, structural distresses have been found to reappear weeks to months after HIR treatment. HIR can be used to improve cross-slope but does not address more general drainage issues. Drainage should be improved prior to performing HIR. Pavements that have had geotextile fabrics placed near the surface are not suitable candidates for HIR.

Improper Mix Design or Inadequate Heating, Mixing or Compaction

Improper mix design can prevent recycled materials from achieving the desired consistency and strength. Laboratory testing using actual in-place materials is imperative in achieving a proper mix design. Inadequate heating, mixing, or compaction can also affect strength and consistency. Quality assurance procedures, such as in-place density or coring, are necessary to ensure that the material has been handled properly. Thickness is part of the mix-design process, and care needs to be taken so that the HIR thickness does not match up with an existing layer boundary so that scabbing of the layer occurs, which can lead to delamination.

Contractor Experience

Other states' experience with HIR has been mixed; some projects have failed early and others have lasted like an asphalt mill and overlay. Project selection is considered one of the reasons for this inconsistency. Contractor experience may be another. Unlike typical asphalt paving, HIR is more of a specialty construction with specialty equipment (heaters and scarifies), and so the final result can be different based on the experience of the contractor performing the work.

COLD IN-PLACE RECYCLING

What is Cold In-place Recycling?

Cold in-place recycling (CIR) is a potential in-place alternative to milling and inlaying an asphalt pavement to improve surface conditions. It provides agencies with the ability to optimize the value of in-place materials and minimize construction time and traffic flow disruptions. It also reduces the number of construction vehicles moving in and out of the construction area. In recent years, petroleum and aggregate economics and supply have increased the need for high-quality, cost-effective alternatives to virgin paving mixtures. Distresses in the upper 2 to 4 inch can be minimized using CIR. CIR is a process that uses cold milling of surface and remixing with the addition of asphalt emulsion, Portland cement, foamed asphalt, or other additives to improve the properties of the reclaimed asphalt pavement, followed by placing and compacting the new mix in one continuous operation.

How to Select Projects for Cold In-place Recycling

Different methods of recycling are applicable to different types, levels, and severity, and, consequently, different periods in the pavement life. Typically, CIR is used when there is a higher number, type, and severity of non-load-related distresses that may extend farther down from the surface. CIR with an overlay can be used to address some load-related distresses. Figure 48 shows the pavement condition and type of appropriate in-place recycling method.

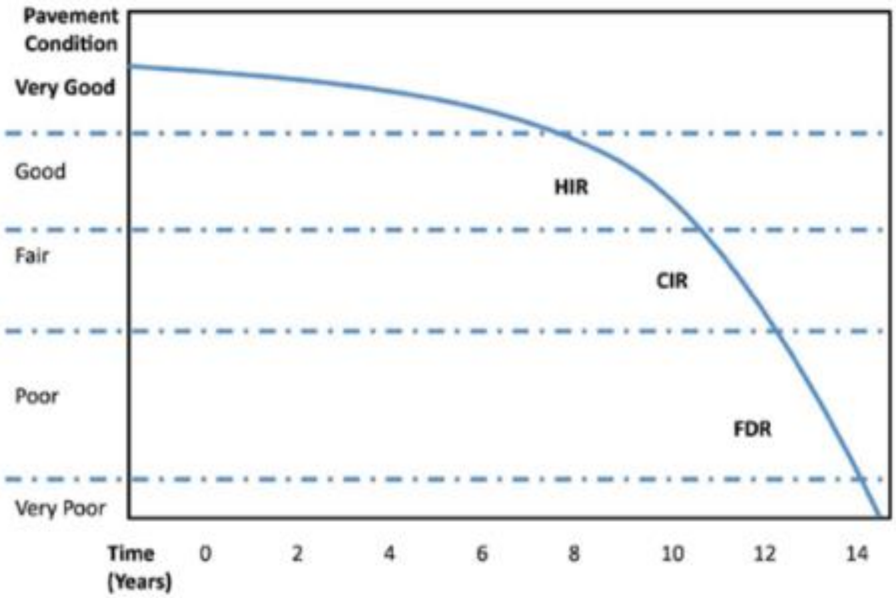


Figure 48. Graph. Pavement Condition and Type of In-place Recycling Method

Figure 49 shows the good candidates for CIR.

Figure 49. Photo. Good candidates for CIR

Material Design

In-Situ Layer Properties

In-situ properties are needed to evaluate the need for different designs for different segments for a project. The ability of the underlying layers to support the construction equipment, and the variability in layer thicknesses that can affect a reasonable selection of milling depths needs to be determined. The initial use for this testing is to determine the ability of the subgrade to support the weight of the recycling equipment, to evaluate needs for increased structural capacity, to provide information for the structural design, and to identify sections in need of different treatments. Georgia agencies use coring to determine thickness for CIR. The availability or collection of in-place material properties information needs to be considered when developing the project design, specifications, and agency estimates of project costs.

RAP Properties

RAP binder content, RAP binder properties, and RAP aggregate gradation are needed for the appropriate selection of grades of new aggregates, new binders, recycling agents, and additives. The most common agency preconstruction laboratory testing focuses on RAP gradations and binder contents, material properties, and recovered binder properties. Preconstruction testing is key to designing recycling mixes. The time needed for this testing, as well as the costs to the project need to be considered in developing cost estimates and project timelines.

New Materials and Additives

A range of new materials and additives can be used to produce desired mix properties and early performance. New aggregates may be added to adjust the final gradation. New binders (paving-grade asphalts, emulsions) are used to soften aged asphalt in the RAP and provide more flexibility of the final asphalt concrete layer. Recycling agents and rejuvenators can be used instead of the new binders to improve the binder performance properties. Although each material can be added individually, it is common practice to introduce new aggregates and asphalt by adding new HMA to the recycled materials. Additives and stabilizers can be added to improve stiffness, moisture resistance, and rut resistance; reduce raveling; help dry moist RAP and soils; and control the rate of set of emulsions.

Mix Designs

The most commonly used mix design methods vary by the in-place recycling process. CIR is based on emulsion or foamed asphalt methods, which include:

1) EEs
    a. Caltrans: 75 blow Marshall
    b. Iowa DOT: 4 inch gyratory with 30 revolutions
    c. SemMaterials: 6 inch gyratory with 30 gyrations
2) Emulsions
    a. Wirtgen: 75 blow Marshall

b. Ontario Ministry of Transportation (MTO): 75 blow Marshall

3) Foamed asphalt

a. Iowa DOT: 4 inch gyratory with 25 revolutions

b. Wirtgen: 75 blow Marshall

c. Ontario MTO: 75 blow Marshall

Because mix design is intended to represent filed conditions, curing periods before testing are included in emulsion and foamed asphalt mix designs. As with the compaction methods, each mix design method varies in its curing procedures:

1) EEs

a. Caltrans: Cure at 140ºF to constant weight

b. Iowa DOT: 48 h at 140ºF

c. SemMaterials: 72 h at 140ºF

2) Emulsions

a. Wirtgen: 72 h at 104ºF. For high traffic (i.e., greater than 5 million ESALs), the specimens are compacted at the anticipated final field moisture content and cured in sealed containers for 40 h at 104ºF.

b. Ontario MTO: 48 h at 140ºF, soaked for 24 h at 77ºF, or vacuum saturated for 60 min at mmHg pressure.

3) Foamed asphalt

    a. Iowa DOT: 72 h at 105ºF

    b. Wirtgen: Same as for emulsions

    c. Ontario MTO: Same as for emulsions

Construction Procedures and Considerations

CIR mills only the existing HMA pavement surface. Screening decks and onboard crushers size the reclaimed asphalt pavement. The sized material is then transferred to a twin-shaft pugmill and mixed with the emulsion or foamed asphalt.

Standard haul trucks are used to provide new aggregates or new HMA. Unlike the HIR recycling trains, one or two nurse trucks are usually in front of the recycling profiler and mixer unit to provide a continuous supply of liquids for the mix. The recycling unit mills, processes, and mixes the recycled materials and then transfers them to a paver. Standard compaction practices are used to place and compact them. Figure 50 shows the typical equipment in a CIR recycling train.

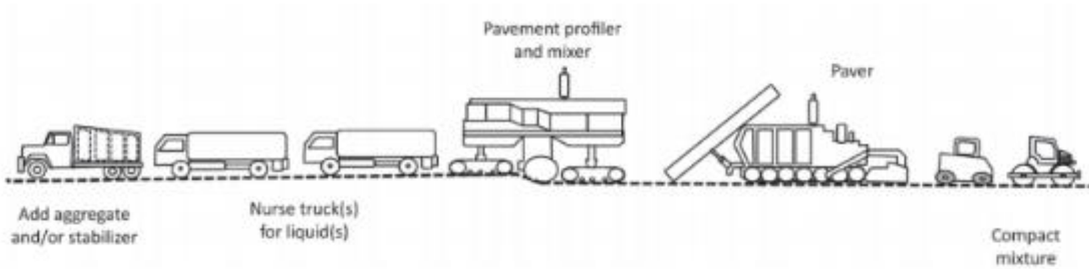


Figure 50. Graph. Typical equipment in CIR Recycling Train (FHWA 1997; ARRA 2001)

Weather Considerations

As with any paving operation, construction should not be planned when there is inclement or rainy weather anticipated. In addition, minimum temperature restrictions may be necessary to allow for the proper compaction and curing of the CIR base. The GDOT specification for specific weather or temperature restrictions should be consulted.

Curing conditions for CIR range from

– a minimum of 14 days to maximum of 30 days or

– when moisture content is below 1.0%.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the CIR construction. The work zone should be constructed according to GDOT requirements. Construction may be best planned for nighttime or weekend work to minimize disruptions and provide for optimum weather conditions. Using cement in the mix-design can assist in providing earlier strength gain for opening to traffic sooner.

Surface Preparation

Equipment used in front of recycling and for compaction after recycling is that typically used in conventional maintenance and HMA overlay placement projects. Surface preparation is not specific to a particular recycling process.

Performance and Limitations

Wet Weather Condition

Wet weather conditions would limit CIR as follows:

1) Rainy weather interrupts construction work, which requires moving large, slow equipment units on and off the project. Parking large equipment during construction is an issue because of the size and very slow speed of the equipment.

2) Possible performance issues exist if rainy weather sets in before work is complete.

3) Wet, cool weather delays the use of emulsion-based CIR and lengthens the curing time.

ULTRA-THIN BONDED WEARING COURSE

What is Ultra-Thin Bonded Wearing Course?

Ultra-thin Bonded Wearing Course (UTBWC), also known as NovaChpTM, is designed for use as a high-performance surface treatment over structurally sound asphalt or concrete pavements. UTBWC consists of a heavy spray application of a polymer-modified asphalt emulsion membrane, followed immediately by an ultra-thin gap-graded hot mix asphalt (HMA). Both applications are quickly completed in one pass by a specially designed paving machine. Once the hot mixed asphalt is placed, a 10-ton steel wheeled vibratory roller is used for compaction. The area can be opened to regular traffic usually within 15 minutes after compaction is completed.

The benefits of UTBWC include the following:

1) Superior bonding of the new surface to the existing surface.

2) Impermeable membrane helps seal out water.

3) Reduces noise.

4) Improves skid resistance.

5) Maintains overhead clearances, curbs, and drainage profiles.

6) Provides fast construction/return to traffic – lowers delay costs.

7) Minimizes user inconvenience.

Figure 51 shows the ultra-thin bonded wearing course:

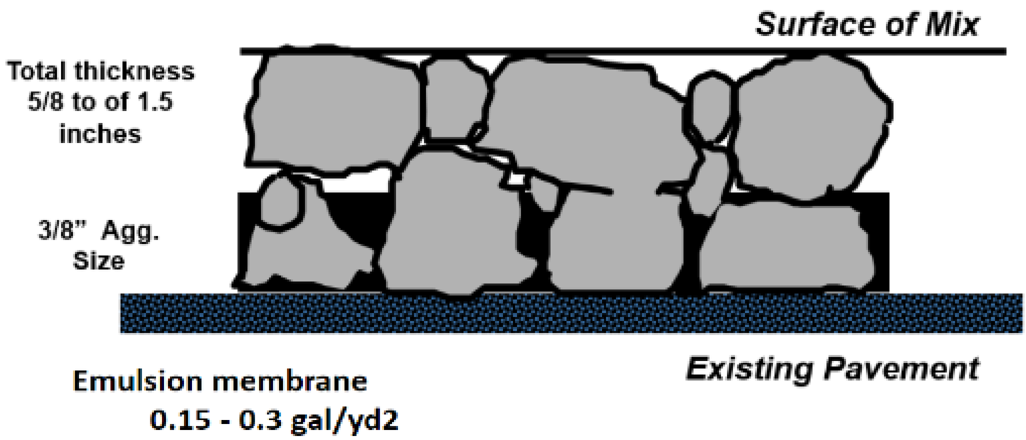


Figure 51. Graph. Ultra-thin Bonded Wearing Course (Noorvand & Estrada, 2015)

How to Select Projects for Ultra-Thin Bonded Wearing Course?

UTBWC is suitable for treating structurally sound pavement with slight to moderate cracking. It can be applied to high traffic volume roads. In addition, it can be considered an alternative to chip seals, micro-surfacing, and open-graded friction course (OGFC).

Material Design

Polymer modified emulsion and gap-graded HMA are two types of materials used in a UTBWC. The requirements for polymer modified emulsion are listed in Table 11.

Table 11. Requirements of Polymer Modified Emulsion (Ahmed, 2010)

Test on Emulsion	Method	Min.	Max.
Viscosity, Saybolt @ 25 °C (77 °F)	AASHTO T59	20	100
Storage Stability Test[1], 24h, %	AASHTO T59		1
Sieve Test	AASHTO T59		0.05
Residue by Distillation[2], %	AASHTO T59	63	
Oil Distillate by distillation, %	AASHTO T59		2
Demulsibility, 35 ml, 0.8% dioctyl sodium sulfosuccinate, %			
Tests of Residue From Distillation			
Penetration @ 25 °C (77 °F)	AASHTO T49	60	150
Solubility in trichloroethylene, %	AASHTO T44	97.5	
Elastic Recovery, %	AASHTO T301	60	
[1]Note: After standing undisturbed for 24 hours, the surface shall be a smooth homogenous color throughout. [2]Note: AASHTO T59 with modifications to include a 200 °C (392 °F) maximum temperature to be held for a period of 15 minutes.			

HMA consists of about 6% asphalt binder of PG 70-28. The coarse aggregate testing needs to meet the following requirements, shown in Table 12:

Table 12. Requirements of Polymer Modified Emulsion (Ahmed, 2010)

Tests	Method	Limit
Flat & Elongated Ratio @ 3:1, %	ASTM D 4791	25 max
% Crushed, single face	ASTM D 5821	95 min
% Crushed, two or more Mechanically crushed faces	ASTM D 5821	85 min
Micro-Deval, % loss	AASHTO TP58	18 max

Construction Procedures and Considerations

Weather and proper methods are critical for chip seal performance.

Weather/Environment Considerations

It is required that UTBWC be constructed when the temperature is warm. The standard Specification Section 424 limits construction to between April 15 and October 15 and, also, requires a minimum ambient and pavement temperature. I If there is moisture on the pavement surface or if rain or freezing temperatures are imminent, work should not proceed. The relative humidity can affect the amount of time for an asphalt emulsion to cure; therefore, the specification also has a caution when the temperature is warm and the relative humidity is high. In addition, high temperatures may also lead to lower viscosity in the binder, causing aggregate to be picked up by the rubber tires of the rollers.

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the UTBWC operations. The work zone should be constructed according to GDOT requirements.

Surface Preparation

Pavement surfaces shall be thoroughly cleaned prior to applying UTBWC. The non-working cracks wider than 1/4 inch and working cracks shall be sealed prior to applying the UTBWC. Pavement surfaces shall be free of fresh bituminous mix. The UTBWC shall not be applied until the sealant has cured. Curing time of the sealant shall be in accordance with the manufacturer's recommendations.

UTBWC Application

The paver for placing UTBWC is specially built, and it typically incorporates a receiving hopper, a feed conveyor, and a storage tank for polymer modified emulsion. The paver can place UTBWC at a rate of 10 ~ 30 m/min (30 ~ 90 ft/min). The screed of the paver should have the ability to crown the pavement at the center both positively and negatively, and it should have vertically adjustable extensions to accommodate the desired pavement profile. A polymer modified emulsion membrane is typically sprayed by the machine prior to the application of the UTBWC at the temperature of 5 ~ 80 °C (120 ~ 180 °F) and a spray rate of about 0.20 gal/yd^2.

During the compaction processes, a steel double drum asphalt roller can be used with a minimum weight of 10 metric tons. These rollers should be equipped with a functioning water system and scrapers to prevent adhesion of the fresh mix onto the roller drums.

Opening to Traffic

Do not allow traffic on an individual course until the bituminous material has cooled or set enough to ensure that the aggregates will not be loosened, dislodged, or whipped off by slow-moving traffic.

Performance and Limitations

Some results from combined tests of the wheel tracking test and the texture depth test proved that the designed wearing course mixture shows satisfactory skidding resistance and wearing resistance. Field texture depth test results confirmed that the designed wearing course mixture is promising to keep long-term skidding resistance during traffic loading.

Laboratory and field tests based on the test road indicated that the wearing course paved with the designed ultra-thin wearing course mixture can provide satisfactory waterproof and interlayer bonding effects, which are helpful for improving the pavement’s durability.

CHAPTER 3. LITERATURE REVIEW OF CONCRETE PAVEMENT PRESERVATION METHODS

This chapter reviews the prevailing concrete pavement preservation treatments for developing GDOT's pavement preservation guide. Each treatment method is presented based on the contents of 1) what the treatment is, 2) how to select project for this treatment, 3) material design, 4) construction procedures and their considerations, and 5) performance and limitations. These contents are also incorporated to develop the web-based pavement preservation interactive tool (PPIT) for GDOT's pavement preservation guide. The detailed introduction and tutorial of the PPIT is in APPENDIX A.

DIAMOND GRINDING

What is Diamond Grinding?

Diamond grinding is one of the pavement preservation techniques that are used for Portland Cement Concrete treatment. It was first used in 1956 on a new concrete taxiway at Davis Monthan Air Base in Tucson, Arizona, to correct localized profile problems (Rao et al., 1999). Continuous diamond grinding for pavement restoration dates has been first used in 1965 on a 19-year-old section of the San Bernardino Freeway (I-10) in California to eliminate excessive faulting (Neal et al., 1976). Diamond grinding can be used alone or with other preservation methods as a comprehensive CPR (comprehensive pavement rehabilitation) program in order to get smoother ride, road noise reduction, and friction improvement (Correa et al., 2001). A detailed CPR sequence is shown in Figure 52. Note that not all projects require every procedure, but the sequence should be maintained, and the most common procedures are in boxes shown in bold. Figure 53 shows a concrete pavement surface after diamond grinding by diamond saw blade.

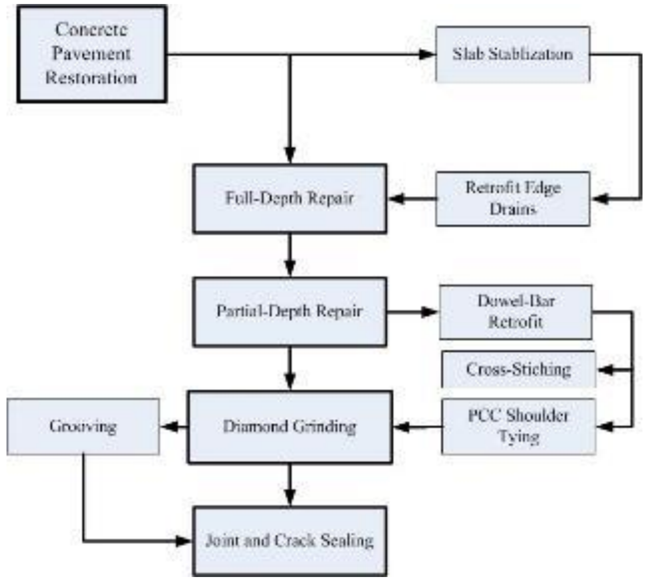


Figure 52. Graph. Sequence of CPR Techniques (ACPA, 1997)

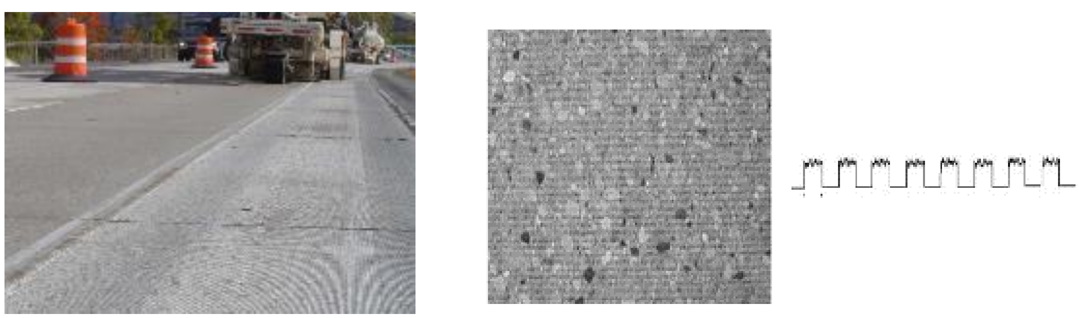


Figure 53. Graph. Concrete Pavement Surface after Grinding (Caltrans, 2007)

It is noted that diamond grinding and diamond grooving are two different grinding techniques, so here is a list the major differences between diamond grinding and diamond grooving. First, the purpose of these two techniques are different. Diamond grinding is mainly used to restore ride quality, improve skid resistance, and reduce tire-pavement interaction noise. However, diamond grooving is mainly used to produce channels to collect water and drain it from the pavement surface, thus reducing wet weather crashes caused by hydroplaning. Second is the difference of pavement surface texture after grinding and grooving. Figure 54 shows that the land areas of these two techniques are quite different, and the land area of diamond grooving is much longer than diamond grinding.

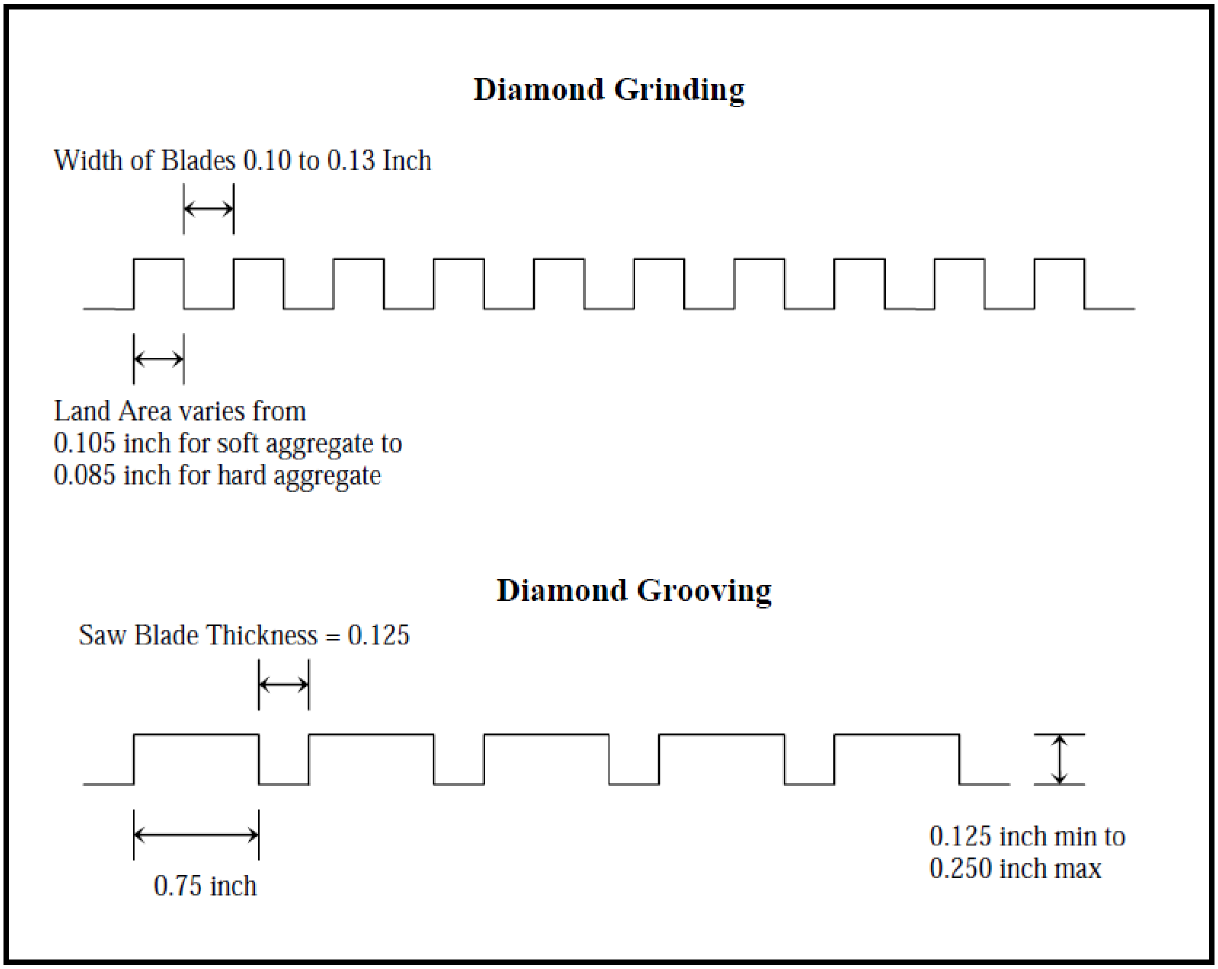


Figure 54. Diagram. Diamond Grinding and Diamond Grooving (NMDOT, 2007)

How to Select Project for Diamond Grinding

Suitable Candidates

This section lists suitable candidates for diamond grinding.

1) Faultings at joints and cracks of jointed plain concrete pavement (JPCP).

   Faulting is usually caused by heavy traffic loads, insufficient load transfer between adjacent slabs, and other factors. To correct this type of distress, people set up critical values of faulting, which will be addressed in section 1.4.2 while considering diamond grinding as the preservation method.

2) Built-in or construction roughness.

   Built-in roughness means that the surface of a newly constructed JPCP pavement does not meet smoothness specifications. In this case, diamond grinding can be used to correct this so-called built-in roughness problem.

3) Unacceptable tire-pavement interface noise.

   Overall longitudinal roughness result in tire-pavement interface noise directly. Diamond grinding corrects the textures of the worn surface with a new longitudinal texture, thus reducing roughness and providing a quieter ride.

4) Slab warping

   Slab warping at joints is caused by moisture gradient and construction curling in most cases. For example, in dry weather, slabs can become permanently warped at the joints. Because of long joint spacings and stiff base support, curled slabs are higher at the joints than at mid-panel, resulting in a bumpy, rough ride (Caltrans, 2007). Slab warping results in uncomfortable riding. In this case, diamond grinding can be used to restore the surface roughness, and after diamond grinding warping is not likely to recur.

5) Inadequate transverse slope

Diamond grinding can be used to restore the pavement transverse slope. For example, studded tires may wear the pavement surface so that water will aggregate in the wheel paths, which causes hydroplaning. In this case, diamond grinding can be used to remove the wheel path ruts and reduce hydroplaning (Caltrans, 2007).

Timing

Faulting Timing

Diamond grinding is used to correct the faulting distress of concrete pavement; however, it shouldn't be conducted if faulting reaches certain values. Table 13 shows the established threshold values for diamond grinding by NMDOT.

Table 13. Faulting Value Threshold (NMDOT, 2007)

Average Fault (Inches)	Comments
1/32	No roughness
1/16	Minor Faulting
3/32	Grinding project
1/8	Expedite Project
5/32 3/16 7/32	Discomfort begins
1/4	Grind Immediately

Also, if load transfer is deficient on the pavement, the pavement should be restored with dowel bar retrofit or undersealing first before diamond grinding is conducted. If not, the faulting problems will have a great possibility to occur again.

Usually, diamond grinding performs well when faulting is around 0.10 inch (2.5 mm), and diamond grinding should be done when faulting does not exceed 0.16 inch (4 mm) (Caltrans, 2007). It can also be seen from Concrete Pavement Preservation workshop in 2008 that the trigger value for faulting is 3 mm (1/8 inch).

Trigger and Limit Timing

It is not good to conduct CPR whether too early or too late. Proper timing is very important, thus timing for CPR with diamond grinding is set, which is known as the "window of opportunity." The "window of opportunity" is determined by performing annual pavement condition surveys and selecting trigger and limit values for conducting each CPR technique.

Trigger value: When a highway agency should consider diamond grinding and CPR.

Limit value: When the pavement has deteriorated so much that it is no longer cost effective to conduct grinding.

PSR (present serviceability rating): 5 (new) - 1 (extremely deteriorated condition).

Skid Resistance (f=Friction resistance /Load, unitless):

1) usually <30: it is time to take measures to correct.

2) >=30: acceptable for low volume roads, and

3) 31-35 means monitor pavement frequently, and

4) if >=35, it means acceptable for heavily traveled roads (Jayawickrama et al., 1996).

Table 14 and Table 15 show the timings for CPR with diamond grinding, which are often cited by many DOTs (Colorado DOT, 2010); these were originally seen by Correa and Wong in 2001.

Table 14. Trigger Values for Diamond Grinding (Correa et al., 2001)

	JPCP			JRCP			CRCP		
Traffic Volumes*	High	Med	Low	High	Med	Low	High	Med	Low
Faulting mm-avg (inches avg)	2.0 (0.08)	2.0 (0.08)	2.0 (0.08)	4.0 (0.16)	4.0 (0.16)	4.0 (0.16)	N.A.		
Skid Resistance	Minimum Local Acceptable Levels								
PSR	3.8	3.6	3.4	3.8	3.6	3.4	3.8	3.6	3.4
IRI m/km (in/mi)	1.0 (63)	1.2 (76)	1.4 (90)	1.0 (63)	1.2 (76)	1.4 (90)	1.0 (63)	1.2 (76)	1.4 (90)

*Volumes: High ADT>10,000; Med 3000<ADT<10,000; Low ADT <3,000

Table 15. Limit values for diamond grinding (Correa et al., 2001)

	JPCP			JRCP			CRCP		
Traffic Volumes*	High	Med	Low	High	Med	Low	High	Med	Low
Faulting mm-avg (inches avg)	9.0 (0.35)	12.0 (0.5)	15.0 (0.6)	9.0 (0.35)	12.0 (0.5)	15.0 (0.6)	N.A.		
Skid Resistance	Minimum Local Acceptable Levels								
PSR	3.0	2.5	2.0	3.0	2.5	2.0	3.0	2.5	2.0
IRI m/km (in/mi)	2.5 (160)	3.0 (190)	3.5 (222)	2.5 (160)	3.0 (190)	3.5 (222)	2.5 (160)	3.0 (190)	3.5 (222)

*Volumes: High ADT>10,000; Med 3000<ADT<10,000; Low ADT <3,000

MDOT Timing

The treatment thresholds for the Michigan DOT (MDOT) rigid pavements are included in its report in 2013. The threshold for diamond grinding is RSL (remaining service life): 12; DI (distress index): 15; RQI (ride quality index): 54 and IRI (international roughness index): 107. RQI IRI DI value reference for MDOT is shown in Figure 55 and Figure 56:

DI Values	Rating
0 – 25	Good
26 – 49	Fair
50 or Greater	poor

Figure 55. Graph. DI Value Reference for MDOT

RQI Value (MDOT)	IRI Value (in./mi.) (RQI Equiv.)	Rating
0-30	0 - 49	Excellent
31-50	50 - 81	Good
51-70	82 - 13	Fair
> 70	> 133	Poor

Figure 56. Graph. RQI, IRI, DI Value Reference for MDOT

Other Factors

When candidate projects are selected, the faulting critical value is considered, the timing window is also appropriate, and there are still other factors that should be considered to conduct diamond grinding:

1) Presence of severe drainage or erosion problems. These should be resolved before grinding.

2) Presence of progressive transverse slab cracking and corner breaks. This indicates a structural deterioration.

3) Joints and transverse cracks with a load transfer of less than 60 percent. In this case, the load transfer should be solved first by using retrofitted dowels or other methods prior to diamond grinding.

4) Concrete pavements suffer from durability problems, such as D-cracking, reactive aggregate, or freeze-thaw damage, that indicate that diamond grinding is not a suitable preservation technique, and that a more substantial rehabilitation strategy may be required (Correa et al., 2001).

Material Design

Diamond grinding equipment uses diamond saw blades that are gang mounted on a cutting head (Figure 57 and Figure 58). The three most important factors that are to be considered for a grinding machine are 1) weight of the machine; 2) the horsepower available to the grinding head, and 3) the grinding head (Correa & Wong 2001).

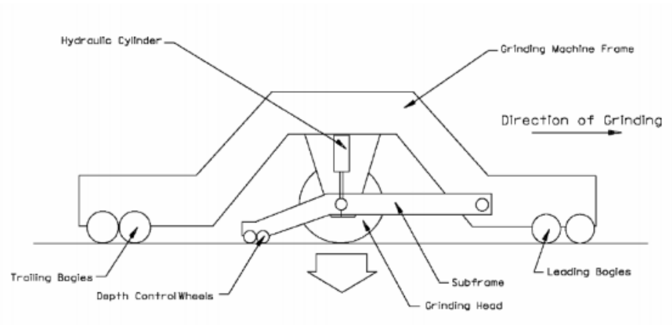


Figure 57. Photo/Diagram. Grinding Equipment

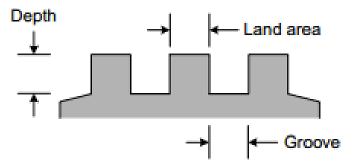
Figure 58. Photo. Diamond Saw Blade and Cutting Head (Caltrans DOT, 2008)

The front wheels pass over a bump or fault, and the centrally mounted cutting head will shave off the bump or fault. The rear wheels follow in the smooth path left by the grinding head. The cutting head usually has a width ranging from 910 to 970 mm (36 to 38 inch), but newer machines may go up to 1,200 mm (47 inch). Generally, appropriate texture is produced by a machine with spacing of 164 to 194 diamond blades per meter (50 to 60 per foot) of shaft (Correa et al., 2001).

From FHWA’s report in 2001, the 3 most important factors that a contractor should consider when choosing suitable saw blades are: diamond concentration, diamond size, and bond hardness. Typical diamond saw blade and cutting heads are shown in Figure 58.

Since the texture and friction caused by diamond-ground surfaces vary with the blade spacing, it is very important to choose the right blade. Also, blades will differ for soft and hard aggregate. A summary of typical groove widths, land areas, and depth of the pavement surfaces is shown below in Table 16. For example, typical values for California are shown in Table 17.

Table 16. Typical Grinding Texture for Different Aggregates (ACPA, 2006, InDOT, 2010)



	Range	Hard Aggregate	Soft Aggregate
Groove width	2.29 – 3.81 mm (0.090 – 0.150 in)	2.54 – 3.81 mm (0.100 – 0.150 in)	2.29 – 3.56 mm (0.090 – 0.140 in)
Land area	1.52 – 3.30 mm (0.060 – 0.130 in)	2.03 mm (0.080)	2.54 mm (0.100)
Depth	1.52 mm (0.060)	1.52 mm (0.060)	1.52 mm (0.060)
No. of Blades	165 – 200/m (50 – 60/ft)	175 – 200/m (53 – 60/ft)	165 – 180/m (50 – 54/ft)

Table 17. Typical values for diamond grinding design in California (Caltrans, 2007)

Parameter	Value
Groove	0.08 – 0.12 inch (2 – 3 mm)
Depth	0.06 – 0.08 inch (1.5 – 2 mm)
Number of Grooves	55 to 60/ft (180 – 200 / m)

Construction Procedures and Considerations

Usually, the pavement condition and roughness data are first used to determine whether it is appropriate to conduct diamond grinding (NMDOT, 2007).

First, the road must be assessed to see whether there will be reconstruction or not in the near future. If there will be reconstruction in the assessed road, it is not necessary to conduct diamond grinding.

Second, the pavement condition assessment is considered. Do structure-related problems exist?

If yes, diamond grinding is not appropriate. If not, does faulting exist? If yes, what is the reason for the faulting? If the reason for the faulting is structure related, it is not recommended to conduct diamond grinding directly.

Weather Requirements

“Air and/or surface temperature should meet minimum agency requirements (typically 35 °F [2 °C] and rising)” (Caltrans, 2007). Icy weather conditions should be avoided.

Traffic Control

The signs, devices, and setup must comply with the contract and MUTCD requirements. When diamond grinding is finished, all the equipment, signs, and personnel must be removed.

Common Problems Troubleshooting

After the selection of projects, the following lists the potential problems associated with diamond grinding and their corresponding solutions (NMDOT, 2007; Caltrans, 2007).

1) “Dogtails”

   “Dogtails” refers to the pavement area that are not ground due to lack of horizontal overlap. It is caused by weaving during grinding and can be solved by maintaining the required horizontal overlap between passes and steady steering.

2) “Holidays”

   “Holidays” refers to areas that are not ground, which is caused by isolated low spots in the pavement surface. This can be solved by lowering the grinding head and doing another pass, since, typically, it is required that 95% of the pavement surface be covered after grinding, which is also stated in Section 0.

3) Poor vertical match between passes

   This can be prevented by maintaining a constant down-pressure.

4) Too much or too little material removed near joints

   When this happens, wide gaps can be temporarily grouted to provide a smooth surface.

5) Too much slurry on the pavement during grinding

   This is caused by a vacuum unit or skirt surrounding the cutting head; when this phenomenon happens, the grinding operation should be stopped and equipment inspected immediately.

Field Operation

Once it is determined to conduct diamond grinding, the diamond grinding will be performed continuously along a traffic lane and, also, started and ended perpendicular to the pavement centerline. Typically, it includes the following 4 phases:

1) Preparation

   It must be ensured that there are no structural deficiencies before the diamond grinding operation.

2) Grinding

Grinding is always along a traffic lane and is a continuous operation. “A diamond saw blade with a cutting head of at least 36 inch in width is used to grind longitudinally. Several machines working together allow a lane to be completed in one pass, thus improving productivity in large projects. One machine and several passes with 2 inch of minimum overlap are used for small projects. The grinding equipment uses water to cool the cutting head” (InDOT, 2010). Generally, it is required that at least 95% percent of the whole area within any 3 foot by 100-foot test area should be ground after the diamond grinding procedure. “Isolated low spots of less than 2 square feet should not require texturing if lowering the cutting head would be required” (NMDOT, 2007). Since the cutting head is narrow, more than one pass will be needed for the grinding equipment.

3) Cleaning

The slurry/residue from the grinding operation must be properly removed.

4) Filling/Resealing

Joints and cracks should be sealed or filled.

Final Surface Finish

The ground pavement surface shall be uniform after diamond grinding. Table 18 shows the specifications by FHWA, 2015.

Table 18. FHWA Final Surface Specification

Parameter	Value
Grooves	2.00 and 4.00 mm (0.100 and 0.150 inch)
Land area	1.50 and 3.50 mm (0.065 and 0.125 inch)
Depth	1.50 mm (1/16 inch)

Adjusting the blade spacing may be necessary to achieve the specified texture listed above.

Thickness is one of the most sensitive factors affecting cracking performance of concrete pavements, and diamond grinding does reduce pavement thickness, so the thickness can be a concern to concrete pavement. Diamond grinding generally reduces slab thickness by 4.00 – 6.00 mm (0.100 – 0.250 in) (Rao et al. 1999; Drakopoulos et al., 1998).

A cracking model result indicated that a 5.00 mm (1/4 inch) reduction in slab thickness results in about 30 percent reduction in fatigue life if the concrete strength remains constant However, long-term strength of concrete is significantly higher than the design strength (typically the 28-day strength) (Rao et al., 1999). If the increase in concrete strength is considered, the small reduction in slab thickness has a negligible effect on service life. These results suggest that a typical concrete pavement may be ground up to three times without compromising its fatigue life (Rao et al., 1999). In practice, some states have ground concrete pavements up to three times without any problems (i.e., California and Georgia) (Correa et al., 2001).

Performance and Limitations

After the introduction of diamond grinding, the application of diamond grinding and project selection for diamond grinding performance is addressed in this section, since it is very important to evaluate the performance of diamond grinding both before and after diamond grinding projects. Various reports state that the field performance of diamond-ground sections is good (Caltrans, 2007; NMDOT, 2007). While considering the performance of diamond grinding, two indicators are listed here: service life extension, cost and faulting index.

Service Life Extension

Service life has many definitions. For example, one study by Rao et al. in 1999 shows that the average age at overlaying or reconstruction was 29.1 years, and the average service life of diamond ground surfaces was 11.4 years or 10.8 million ESALs. However, the sections in Rao's study include sections which have been ground more than once.

While considering the service life, the weather should also be considered. For example, in the same study, a 225 mm (9 inch) no-doweled jointed plain concrete pavement (JPCP) in a dry climate can sustain up to 20 million ESALs after grinding before the level of faulting becomes excessive, but similar pavements in a wet climate may require regrinding after 8 – 11 million ESALs.

So, the following sections will show the pavement performance using different criteria.

Service life based on IRI

"Nationwide, studies have shown that the average longevity of a diamond ground project is around 14 years, or about 11 years at an 80% certainty (reliability) level" (Rao et al., 1999).

Figure 59 shows the results of 26 California diamond grinding projects done by Caltrans in 2005. From Figure 59, the bigger the IRI before grinding, the bigger the improvement of IRI (the ratio of before grinding or after grinding IRI). From Figure 60, it is shown that

1) At 50% reliability prediction, the exponential relationship reveals that the average diamond-ground PCC pavement in California will last 16.8 years when the IRI ratio equals 1.78, which is the trigger value for rehabilitation (Stubstad et al., 2005);

2) At 80% reliability, the extension in service life is about 14 years. This study concludes that these results are reasonable, since the climatic conditions in California are comparatively favorable for rigid pavement performance (CalTrans, 2007).

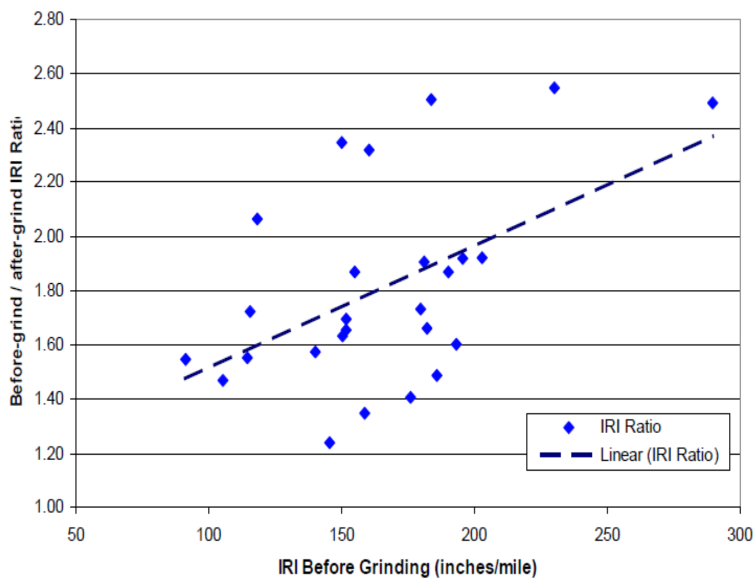


Figure 59. Graph. Improvement in Ride Quality (Stubstad et al., 2005)

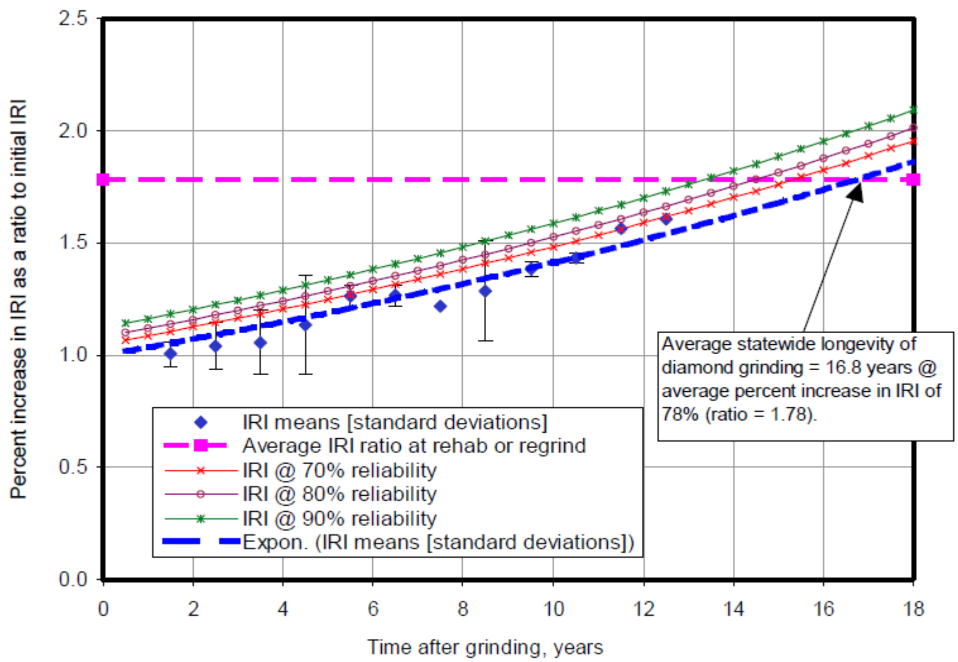


Figure 60. Graph. Expected survivability of diamond ground PCC pavements in California (Stubstad et al., 2005)

Service life based on EASL

The average service life of diamond-ground pavements based on survival analysis in the study done by Rao et al. 1999 is 37 years or 35 million ESALs since initial construction. Many sections in the database have survived 40 or more years, but some of these pavements were diamond-ground two or more times. So, further investigation is needed to state a more accurate number for service life based on EASL for one-time grinding.

Cost

Whether used alone or as part of an overall CPR program, diamond grinding costs between \$2.0 and \$8.0 per square meter (\$1.70 and \$6.70 per square yard). The cost can be as high as \$12/m^2

or \$10/yd^2) when concrete contains very hard river gravel (FHWA, 2001). The historical cost of diamond grinding by the Michigan DOT is \$4.88 (2008), \$3.37 (2007), \$3.49 (2006), \$4.84 (2005), an average of \$4.15 per yd^2.

Many state DOT's have found that the cost of diamond grinding is generally lower than the cost of an asphalt concrete overlay (Pierce, 1995; McGovern, 1995). For example, Washington conducted a cost comparison of retrofit dowel bars with grinding, 1.2 m (4 ft) tied shoulders with grinding, and 110 mm (4 inch) AC overlay, shown in Table 19, based on material costs (Pierce 1995).

Table 19. Washington Cost Comparison

Estimated costs of the rehabilitation alternatives	Costs
Retrofit dowel bars with diamond grinding (truck lane only)	\$73,000 per lane-km (\$117,450 per lane-mile)
Tied PCC shoulders with diamond grinding in the truck lane	\$69,100 per lane-km (\$111,200 per lane-mile)
AC overlay	\$118,300 per lane-km (\$190,300 per lane-mile)

Much of the cost savings can be attributed to the fact that diamond grinding can be applied only to lanes that need corrective treatment (FHWA, 2001).

GDOT found CPR with grinding to be 3-4 times as cost-effective as a 150 mm (6 inch) AC overlay (Rao et al., 1999).

Faulting Index Reduction

Other than the service life extension and cost analysis, Georgia DOT has used CPR with grinding for more than 20 years. GDOT experimented with the CMI Rotomill in 1976, the Klarcrete percussion device in 1978, and the Dynaplane cold planer in 1979 (Wouter, 2012) to compare with diamond grinding and finally found that only the diamond grinder could remove faulting without damaging the joint and did not produce an excessive noise. The faulting index for Georgia is shown in Table 20 data was analyzed for five major interstates from 1972 to 1996 for the same miles that were in service in 1972 in Georgia; the results are shown in Figure 61. It is shown that after 1996, all FI data are below 20, which means in fair condition after CPR.

Table 20. Faulting Index of Georgia

Faulting Index (FI)	Condition
15-20	Fair
20.1-25	Poor

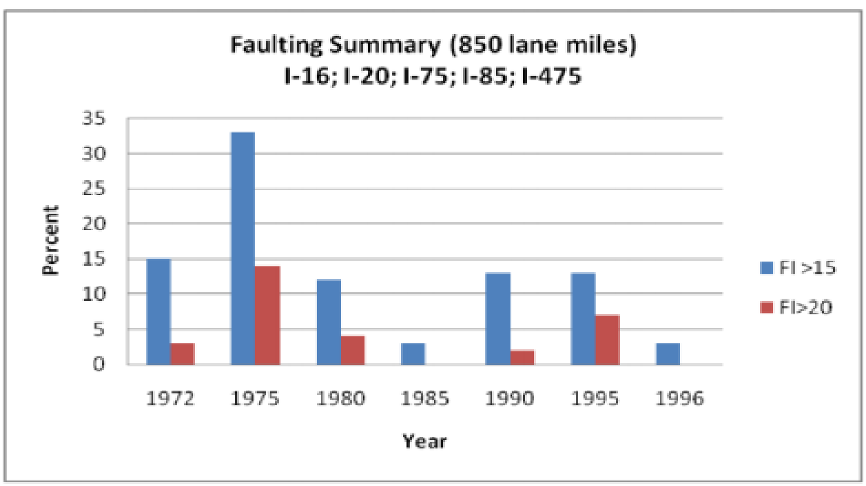


Figure 61. Graph. Faulting Index Change after CPR (Wouter, 2012)

PARTIAL DEPTH REPAIR

What is Partial Depth Repair?

Since the concrete pavement restoration (CPR) concept is widely known to almost all the transportation agencies, which is mentioned in previous chapter on diamond grinding, this chapter discusses the “Partial Depth Repair” topic. Seen in Figure 62, partial depth repair (PDR) is in the middle phase of CPR.

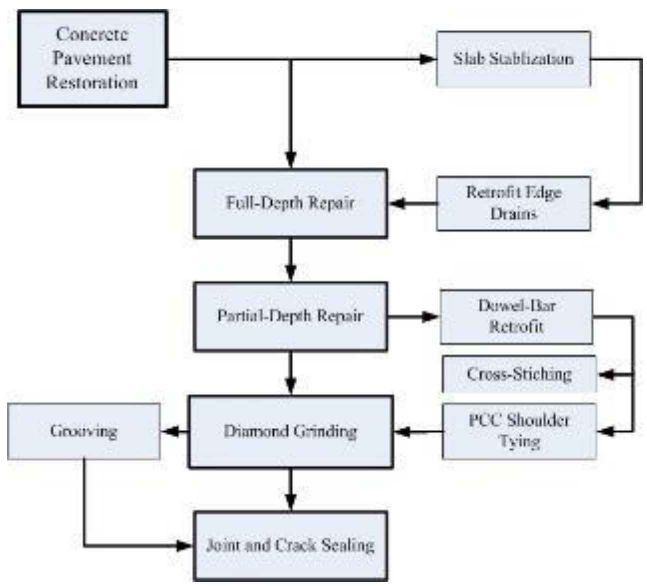


Figure 62. Graph. CPR Techniques (ACPA, 1997)

Partial depth repair (PDR) is used (in most cases) to treat spalling and surface scaling. If severe scaling is shown in a small areas, PDR can also be used (ACPA, 1998). It improves the rideability of concrete pavement and contributes to reduced moisture infiltration and intrusion of incompressible into joints (ACPA, 1997), and, also, it restores a uniform, well defined joint sealant reservoir prior to joint resealing (Frentress & Harrington, 2012).

Currently, almost all U.S. states (44 out of 49) use pavement patching except for five, mainly because that the probability of having freezing temperatures in those 5 states is very low and they don't have to consider patching as a major component of their maintenance programs. The distribution of the patching practice usage is shown in Figure 63 (McDaniel, 2014). A detailed survey discusses whether states monitor the performance of reactive and planned patches, whether they have an established method to track patch locations, and whether they have any QC/QA procedures at the time of patch placement (McDaniel, 2014). The distress problems that patching can treat are similar and the trigger timing for patching is also similar for states and local agencies. The engineering focus on patching is to determine 1) where to patch, 2) when to patch, 3) the type of patch, and 4) the situations in which patching is not appropriate (McDaniel, 2014).

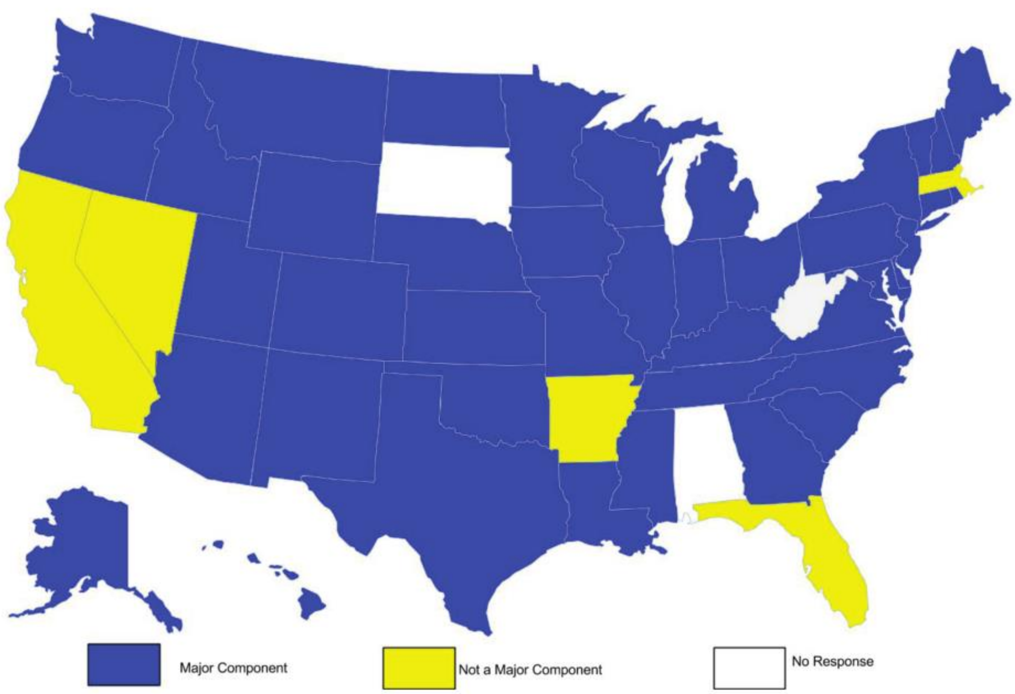


Figure 63. Graph. States considering patching a major activity (McDaniel, 2014)

The common procedure for partial depth repair is to first identify the removal area by chain dragging and hammering the surface to find the area limits. Then the removal area is marked out for further cleaning (sand blasting and air blasting). After that, concrete placement will be done to the removal area.

Partial-depth repairs usually includes 3 types: spot repair, long joint/crack repair, and bottom half repair. Spot repair and long joint/crack repair are standard partial-depth repairs; examples are shown in Figure 64 and Figure 65. Bottom half repairs are used for special repairs, including the bottom half of the slab. Spot repairs are usually between 10 inch and 6 ft long and typically used for pavements in which the existing load transfer devices (if any) are still functional (Daniel, 2011). Long joint/crack repairs are usually between 6 ft and one-third to one-half the depth of the concrete pavement in transverse and longitudinal joints. Figure 66 shows a deteriorated pavement corner for which bottom half repairs can be used. It should be noted that this repair is for work performed on the bottom half of the pavement; it should only be applied to edges or cross-joint locations and not exceed 18 inch (Daniel, 2011). For this study, we mainly focused on Type 1 and Type 2 PDR.

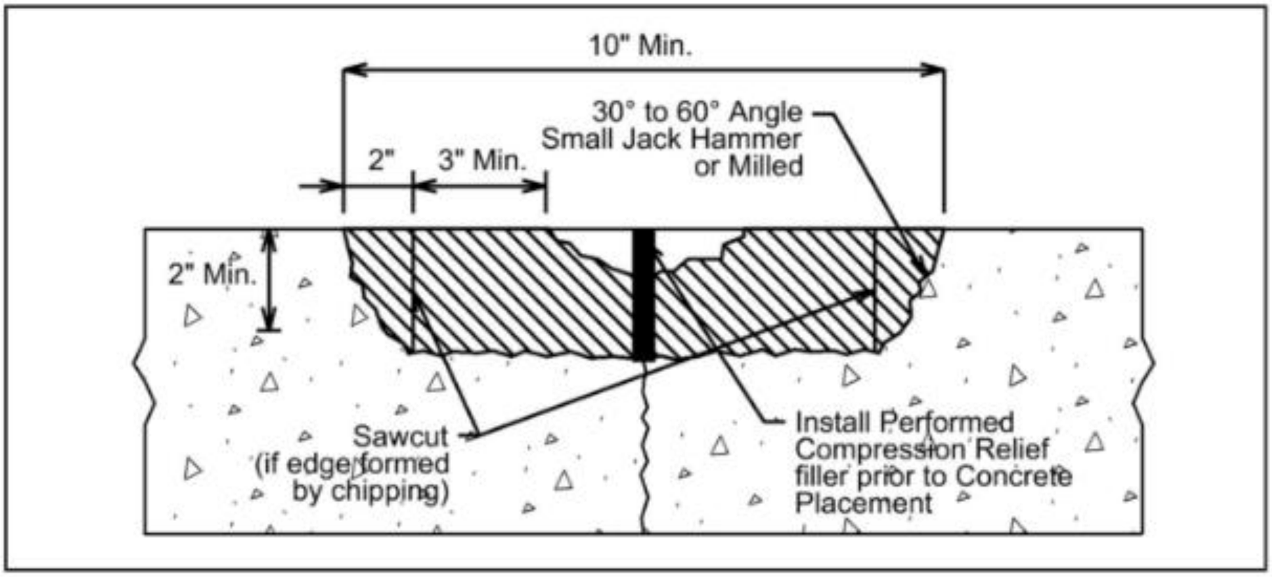


Figure 64. Graph. Partial Depth repair (Spot and Long Joint/Crack Repair) (Daniel, 2011)

Figure 65. Photo. Spot repair (Left) and Long Joint/Crack Repair (right) (Daniel, 2011)

Figure 66. Photo. Deteriorated Pavement Corner (Daniel, 2011)

Noticeably, especially for CPRP, because the full depth repair needs to cut to the full depth, which will affect the load transfer, PDR can keep the continuity of the longitudinal steel to maintain the transverse repair joints (Jungheum, 2012).

After a brief review of the partial depth patching, this guide will discuss the application of PDP in Section 1.2, including Functions, Project Selection (proper candidates), and Timing. After that, it will discuss Material , Equipment Inspection, and Weather Requirements, Traffic Control, and Construction steps that are introduced in detail. Finally, the Performance Evaluation and Cost are discussed. Following that, the performance section of the PDP and the cost of PDP will be discussed.

How to Select Project for Partial Depth Repair

The distress of concrete is either structural or functional. Typical structural distress includes cracking and joint deterioration, while typical functional distress includes loading, long joint spacing, shallow or late joint sawing, base or edge restraint, and joint lock-up (ACPA, 1997).

Suitable candidates for PDR

1) Concrete pavement with spalling, which is caused by the intrusion of incompressible materials into the joints and cracks by poor consolidation or inadequate curing, by localized areas of scaling, weak concrete, clay balls, or high steel.

2) Concrete pavement with surface scaling or deterioration in the top third of the slab caused by an inadequate air void system (FHWA, 2015; Frentress et al., 2012; Irene, 2012).

Not Suitable candidates for PDR

1) Concrete pavement with spalling of cracks caused by shrinkage, fatigue, or foundation movement.

2) Cases, either suitable or not suitable, by different authors.

3) Concrete pavement with spalling caused by dowel bar misalignment or lockup.

4) Spalling caused by D - cracking or reactive aggregate is recommend by some agencies but not by others (Irene, 2012).

Material Design

The FHWA/SHRP Manual of Practice, Materials, and Procedures for Rapid Repair of Partial-Depth Spalls in Concrete Pavements states that premature partial-depth patch failures can be attributed to a number of material-related causes, including the following (Wilson, 1999; Frentress et al., 2012):

1) Incompatibilities between the climatic conditions during repair replacement and the materials or procedures used.

2) Thermal incompatibility between the repair material and the pavement.

3) Extreme climatic conditions during the life of the repairs that are beyond the capabilities of the repair material.

4) Inadequate cure time prior to opening repaired pavements to traffic.

5) Incompatibility between the joint bond breaker and the joint sealant material.

Especially in Georgia, although PDR has been utilized since the 1960s and early 1970s, the patching material was identified as the root of the poor performance. It was found that some patching materials were incompatible with the thermal movement properties of the existing concrete (Gulden, 2012). It is clear that the choice of materials is very important. Generally, the material needed for PDR depends on several factors: the amount of time allowed for the repair, air temperature, the size of the repair, cost, and estimated performance (FHWA, 2005; Daniel, 2011). High-quality Portland cement concrete is generally the most appropriate material for

PDR, and it should be noted that while epoxy resin can also be used, the manufacturer's recommendations for placement should be followed (FHWA, 2005; Daniel, 2011).

Selection of proper repair mixture should be based on a determination of its desired properties, such as strength of the repair mixture at a given time (e.g., when the patch needs to be opened to traffic). Table 21 shows an example of the opening strength requirements for PDR. Besides that, two other properties that affect the short- and long-term performance of a patch mixture are shrinkage and the coefficient of thermal expansion (Frentress, 2012).

Table 21. Example of Opening Strength Requirements for PDR (Frentress, 2012)

	Opening Strength Requirements	
State	Flexural (psi)	Compressive (psi)
New York		1,527 (10.5 MPa)
Kansas	300	1,800
Missouri		1,600
Michigan	300	1,800
Minnesota	500	3,000
Colorado		2,500
Nebraska		3,625

A wide variety of rapid-setting and high-early-strength proprietary materials have been developed for PDR. The materials are easy to place, achieve exceptional early strength, and have been validated for use. For example, a concrete patch mix developed by the Minnesota DOT, called 3U18, has been very successful for more than 30 years (MnDOT, 2005; Daniel, 2011).

MnDOT did a research study on optimizing the material for partial PDR in 2014 with a 3 phase laboratory test (Ronald, 2014). Finally, MnDOT developed a laboratory testing-based acceptance procedure for PDR materials that can be used to repair rigid pavements.

Construction Procedures and Considerations

Weather Considerations

Manufacturers' installation instructions should be reviewed for requirements specific to the patch material being used. Air and surface temperatures must meet manufacturer's and contract requirements (typically 4 °C [40 °F] and rising) for concrete placement. Patching cannot proceed if rain is imminent.

Traffic Considerations

Signs, devices, and the work zone set up must be properly implemented, including trained personnel. The repaired pavement cannot be not opened to traffic until the patch material meets strength requirements. Also, signs and construction devices must be removed when the preservation is finished.

Construction steps

Construction steps are as follows:

1) Determine the repair boundaries

   The repair area is first defined by listening to the sound given by a chain, ball peen hammer, or steel pipe striking the deteriorated pavement. Then, the boundaries for sawing and milling will be marked based on the area determination. The boundaries should extend 2 – 4 inch beyond the visible distressed area, and the suggest values for the PDR are as follows: length: 15 inch; width: 10 inch; depth: 2 inch (Daniel, 2011).

Figure 67. Photo. Sounding deteriorated concrete using a hammer (left) and steel chain (right) (Frentress et al., 2012)

Figure 68. Photo. Deteriorated Pavement Marked for Sawing (Frentress et al., 2012)

2) Concrete Removal

Concrete removal is often achieved by two methods: saw cutting and milling. Saw cutting is the most common method. However, it is recommended that patches be prepared using milling rather than saw cutting, since saw cutting requires more time and more patching material to fill repair areas (FHWA, 1999; Mulvaney, 2014). Different kinds of removal methods are referred to by the report by Frentress and Harrington in 2012.

Figure 69. Photo. Removal Concrete with a Jackhammer (Jungheum, 2012)

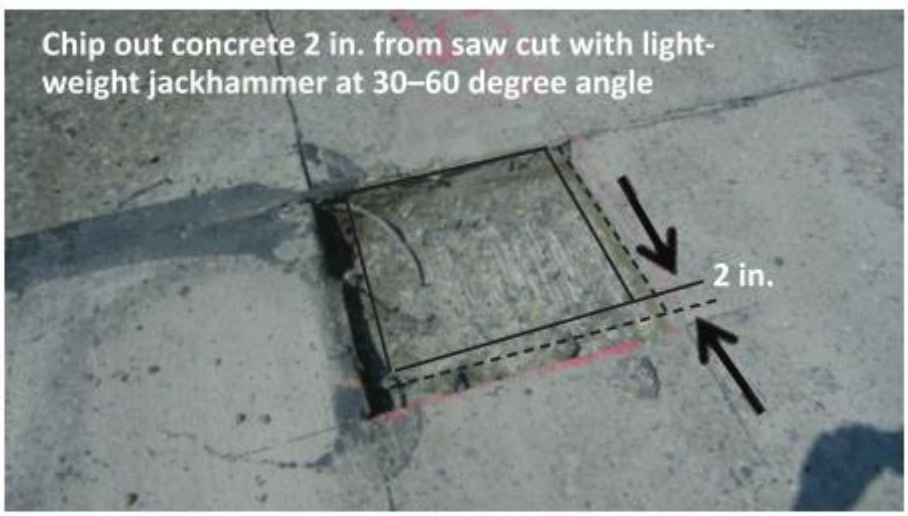


Figure 70. Photo. Saw-and-chip Removal (Frentress et al., 2012)

3) Prepare repair area.

Figure 71. Photo. Sweeping Loose Material (Frentress et al., 2012)

Figure 72. Photo. Sandblasting to Remove Loose Debris (Frentress et al., 2012)

Figure 73. Photo. Air Blasting to Remove Loose Debris (Frentress et al., 2012)

Figure 74. Photo. Using a Leaf Blower for Final Removal of Any Contaminants (Frentress et al., 2012)

4) Prepare joint.

   The fourth step is the installation of joint/crack compression relief material for Type 1 repairs and Type 2B crack repairs.

5) Apply bonding agent (do not allow to dry).

Figure 75. Photo. Placement of Cement Grout as Bonding Agent (Frentress et al., 2012)

6) Place patch material

Figure 76. Photo. Placement of Repair Material (Frentress et al., 2012)

7) Apply curing compound.

Figure 77. Photo. Curing of Repair Material (Frentress et al., 2012)

8) Optional diamond grinding.

9) Seal joints.

Figure 78. Photo. Joint sealing (Frentress et al., 2012)

Performance and Limitations

Using pavement life as a criteria, usually PDR can provide 5 to 15 years of service before further repairs are needed (Irene, 2012). Early studies of PDR show that 80 to 100 percent of properly installed repairs should perform well for 3 to 10 years. However, failure in as little as 2 or 3 years is possible if PDR is constructed with unsound materials or poor workmanship (Wilson, 1999; Smith et al., 2008).

The most frequent causes of early PDR failure include the following:

1) Improper repair materials;
2) Inadequate bond between the patch and existing pavement;
3) Insufficient consolidation;
4) Compression failure, which means the crushing of a repair due to expansion of the surrounding pavement during a freeze-thaw cycle;
5) Incompatible thermal expansion between the patch and existing pavement; and
6) Feathering of the patch material, which means the thin placement of patch materials because of shallow patch edges do not allow adequate depth of placement.

Because of such reasons as the condition of existing concrete, the extent of the deterioration, and future traffic loads, the service lives of various pavements can be very different and difficult to compare.

For example, Wisconsin did a test study of PDR performance by comparing it with other types of rehabilitation methods, which included HMA overlay, bonded concrete overlay, and full depth concrete repair. Their test of performance was monitored from 2000 to 2012. Due to the variations and inconsistencies of their results, they recommended for concrete pavements with surface deterioration at cracks and joints that material and specification changes be considered, including the requirement for a quality control plan/process and additional construction oversight when choosing PDR as a rehabilitation method (Irene, 2012).

DOWEL BAR RETROFIT

What is Dowel Bar Retrofit?

Dowel Bar Retrofit is one of the pavement preservation techniques used for Portland cement concrete treatment. It is used as a means to transfer load from one slab to the next. For jointed plain concrete pavements (JPCP), load transfer is the ability to transfer loads between adjacent slabs. The transfer occurs through a shear action, which is a combination of adjacent slabs' aggregates interlocking and dowel bars' mechanical actions. Efficient load transfer is very important to rigid pavements. Figure 79 shows examples of poor and good load transfer.

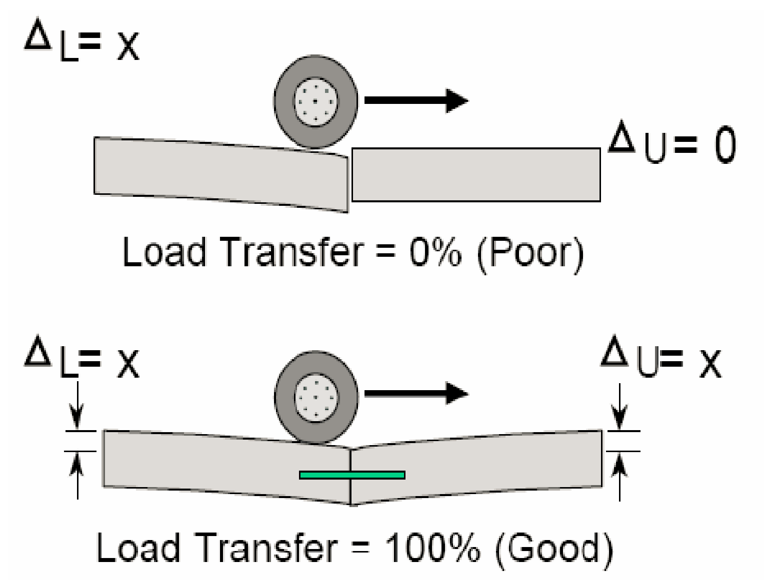


Figure 79. Graph. Load Transfer (Caltrans, 2006)

During the early days, the cost-effectiveness of dowel bar retrofit was not fully recognized, so older roadways have little or no effective means of load transferring between adjacent slabs, which resulted in slab faulting. This faulting creates a rough bumpy ride. Dowel bar retrofit is used to correct the faulting and bumping problem of concrete pavement. Figure 80 and Figure 81 show the bumping and faulting mechanism in a concrete pavement. Figure 82 shows a comparison of pavement before and after the dowel bar retrofit treatment. Since the 1970s, when Georgia started its DBR projects, this treatment method has been used in many states.

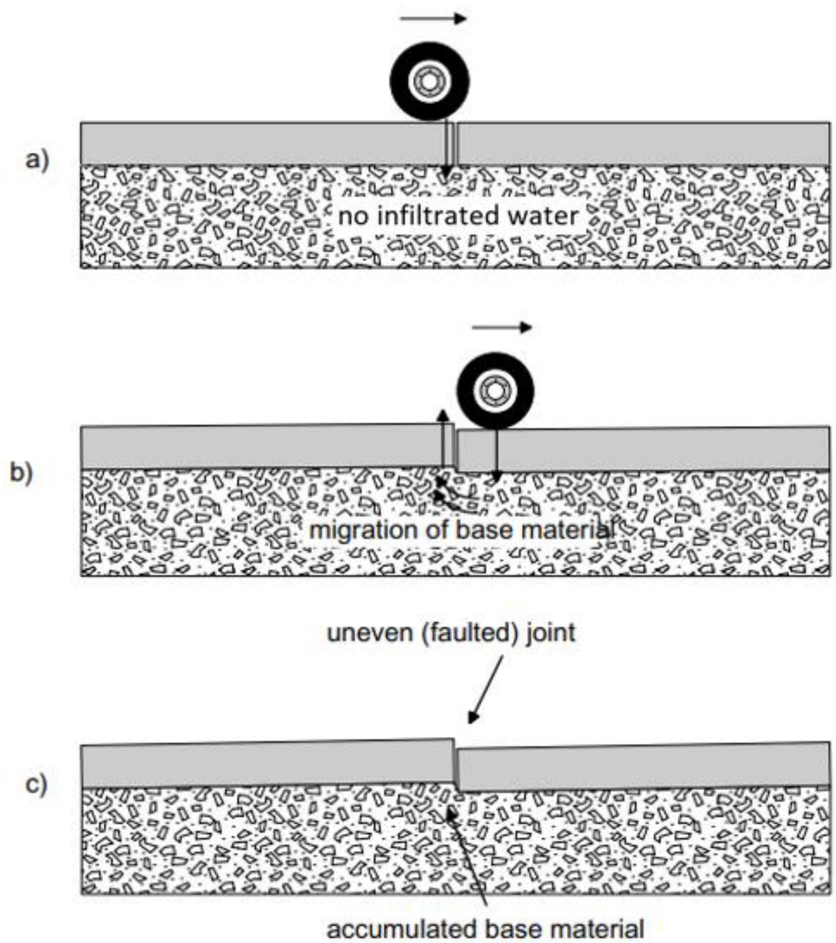


Figure 80. Graph. Faulting Mechanism of Rigid Pavements (Harvey et al. 2003)

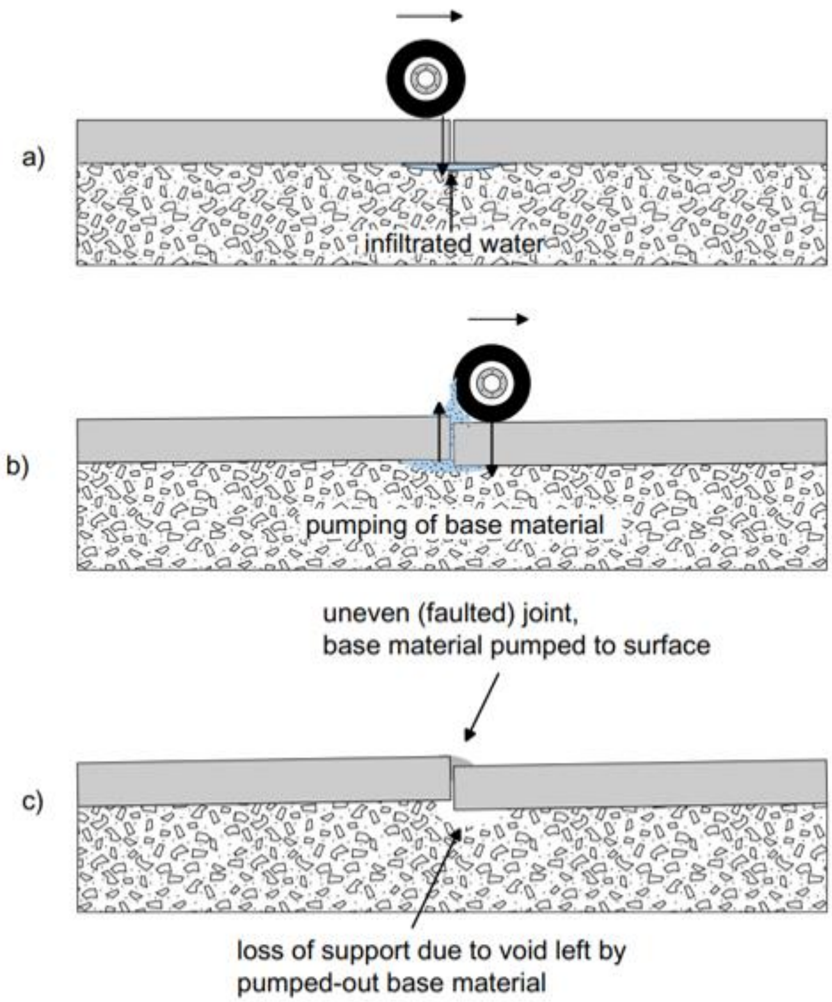


Figure 81. Graph. Bumping mechanism of rigid pavements (Harvey et al., 2003)

Figure 82. Photo. Completion of Dowel Bar Retrofit (Pavement Interactive, 2014)

How to Select Projects for Dowel Bar Retrofit?

Dowel bar retrofit is suitable for pavements that are structurally sound but show low load transfer at joints. It is stated by Caltrans that pavements with 1) little remaining life (extensive cracking of more than 10% Stage 3 cracking) and 2) high-severity joint defects are not suitable candidates.

Suitable Candidates

1) Structural condition of slabs should be good.

   Caltrans: D-cracking, alkali-silica reaction (ASR) or alkali-carbonate reaction (ACR) distress, multiple transverse cracking, or significant longitudinal cracking are poor candidates for DBR (Maintenance, 2008).

   WSDOT: Dowel-bar retrofit is appropriate on PCC pavements with <10% slab replacement.

2) Structural condition of base should be good.

   Caltrans: Slabs with a high deflection value at the joints are generally an indication of poor base condition (Maintenance, 2008).

3) Load transfer efficiency.

   Caltrans: LTE < 60 are good candidates. LTE is tested using the FWD, shown in Figure 83.

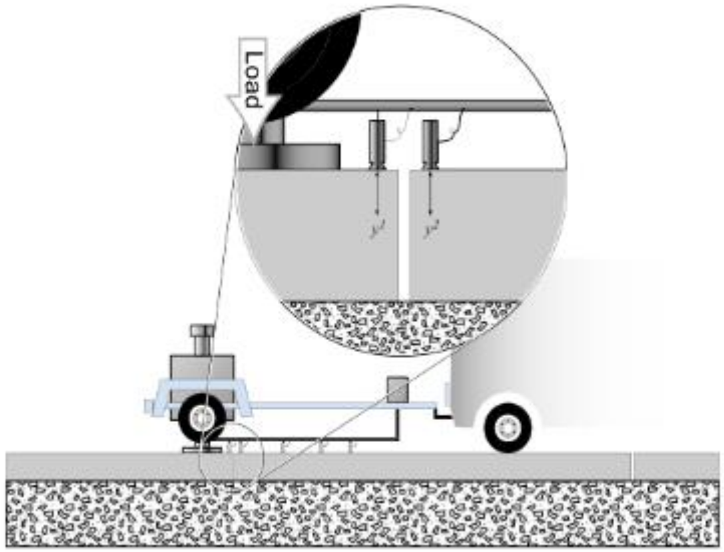


Figure 83. Graph. LTE testing using FWD (Pierce et al., 2003)

4) Faulting value

Caltrans: Pavements with faulting > 0.10 inch (2.5 mm) but < 1/2 inch (13.0 mm) are good candidates. Faulting < 0.10 inch (2.5 mm) does not warrant DBR. If faulting is greater than 1/2 inch (13.0 mm) and/or if the pavement is structurally inadequate, reconstruction should be considered (Pierce et al., 2003).

WSDOT: Dowel-bar retrofit is appropriate on PCC pavements having average faulting of 3.0 – 13.0 mm (1⁄8 to 1⁄2 inch).

5) Joints or cracks condition.

Caltrans: Joints or transverse cracks should exhibit low to moderate severity spalling or better (Maintenance, 2008).

Material Design

Once the dowels are placed, backfill materials are applied. Backfill materials should have similar thermal properties as the concrete, provide strong bonding to the existing concrete, be fast setting, have little shrinkage, and must develop enough strength to allow traffic in a short time. Both high early strength concrete and proprietary mixes have been used successfully. High early strength concrete usually contains Type III cement, accelerators, and aluminum powder. Accelerators and aluminum powder improve set times and reduce shrinkage. Aggregates in the mix should be small enough to allow the concrete to flow around the bar and consolidate properly.

Construction Procedures and Considerations

The procedure for dowel bar retrofit includes cutting slots, removing concrete from slots, cleaning slots, placing silicone at all joints/cracks, placing dowel bar assemblies in slots, placing patching materials, consolidating patching materials, finish patching materials, and diamond grinding. The field operation of dowel bar retrofit can be categorized into 1) slot preparation, 2) dowel bar preparation, 3) dowel bar placement, 4) patching, and 5) diamond grinding.

1) Preparing slots

Dowel bar retrofit equipment uses diamond-tipped saw blades to cut pavements. Figure 84 shows the saw blade and the concrete surface after slot cutting.

The saw cuts were approximately 5 1⁄2 inch deep (Bischoff et al., 2002). However, the depth of saw cutting should be determined carefully by the contractor. Usually, the contractor should measure the depth by placing a dowel bar in the dowel bar chairs. The depth is equal to half of the depth from the dowel to the bottom of the chairs and one half the pavement depth. Sometimes this measurement varies from the specification.

Slots: 1⁄2 inch wide and spaced 12 inch apart on the center (Pierce et al. 2003). Slots should be aligned to miss existing longitudinal cracks.

Figure 84. Photo. Cutting Slots (Bischoff et al., 2002)

After cutting slots, a jackhammer is used to clear the slots. A jackhammer can’t be used vertically to the plane of the pavement because it will punch through the floor of the slots. Many contractors use a 15-pound jackhammer.

After jackhammering, workers scraped the concrete debris from the slots with pickaxes.

Next, the slots were sandblasted and blown using compressed air. Concrete chunks, dirt, debris, water, and slurry should be all cleaned off after these steps. During the cleaning process, the public should be protected from sand and blasting debris, as shown in Figure 85.

Figure 85. Photo. Compressed Air Used to Blow Any Debris Out of the Slots (Bischoff et al., 2002)

2) Dowel bar preparation

There are mainly two types of dowel bars: 1) epoxy-coated steel dowel bars and 2) Nuovinox stainless steel-clad dowel bars.

For example, the Wisconsin DOT uses the following 2 types of dowel bars for one of their studies: 1) The standard epoxy-coated steel dowel bars, which are 18 inch long and 1 1⁄4 inch diameter; 2) the Nuovinox dowel bars, manufactured by Stelax Industries, Ltd., which are 15 inch long and 1 1⁄4 inch diameter. After preparation, each dowel bar is fitted with two end caps, two chairs, and a foam board (Bischoff et al., 2002). A more detailed categorization of types of dowel bars is introduced in Figure 86. Table 22 shows the state highway practices for dowel bar diameter (inch) by pavement thickness, which is summarized from 2009 National Concrete Consortium questionnaire responses (NCPTC, 2011).

Figure 86. Photo. Each dowel bar after preparation fitted with two end caps, two chairs, and a foam board (Bischoff et al.,2002)

Table 22. Dowel Bar Types According to Cost and Corrosion Resistance (WSDOT, 2011)

Dowel Bar Type	Cost	Corrosion Resistance
Solid stainless steel	Most expensive	Best corrosion resistance
Stainless steel clad	⇓	⇓
Stainless steel sleeve with epoxy coated insert	⇓	⇓
MMFX-2 steel (patented steel bar) and Jarden Lifejacket® dowel bar (zinc clad dowel bar)	⇓	⇓
Epoxy coated (AASHTO M-284)	⇓	⇓
Black steel (uncoated)	Least expensive	Worst corrosion resistance

Table 23. State highway agency practices for dowel bar diameter (inch) by pavement thickness

Slab Thickness (in.)	6.0	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5	11	11.5	12.0	12.5
California	1.250	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500	1.500	1.500	1.500	1.500
Iowa	0.750	0.750	0.750	0.750	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500	1.500
Illinois	1.000	1.000	1.250	1.250	1.500	1.500	1.500	1.500	1.500	1.500	1.500	1.500	1.500	1.500
Indiana	1.000	1.000	1.000	1.000	1.000	1.000	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.500
Michigan	1.000	1.000	1.000	1.000	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.500	1.500	1.500
Minnesota	1.000	1.000	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500
Missouri	N/A	N/A	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500
North Dakota	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500
Ohio	1.000	1.000	1.000	1.000	1.000	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500
Texas	N/A	N/A	N/A	N/A	1.000		1.125		1.250		1.375		1.500	
Wisconsin	N/A	N/A	1.000	1.000	1.250	1.250	1.250	1.250	1.500	1.500	1.500	1.500	1.500	1.500

Once the slots have been cleaned and the dowel bars prepared, the dowel bars are ready to be put into the slots. Contractors should make sure that the dowel bar placement has the correct position. Figure 87 and Figure 88 show the dowel bars that have been placed and aligned in slots over a transverse crack. Generally, 3 dowel bars are placed per wheel path because of the study done by Florida, which showed that three dowel bars performed as well as 5 dowel bars for faulting. Tayabji (1986) identified the following categories of dowel misalignment: horizontal translation, longitudinal translation (side shift), vertical translation (depth error), horizontal skew (horizontal misalignment), vertical tilt (vertical misalignment)

Figure 87. Photo. Dowel bars after placement

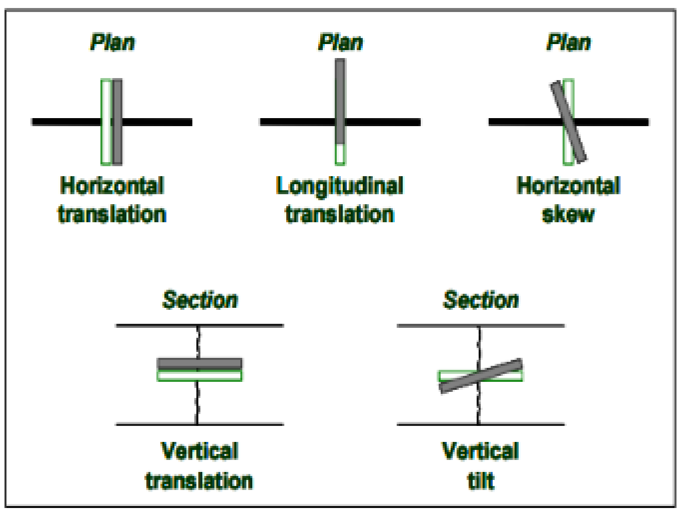


Figure 88. Graph. Dowel bar misalignment types

3) Patching

The slots are blown clean and wetted down with water before the patching material is poured into the slots. Then, the pathing material is run with mobile mixer until it reaches the desired consistency. Then, the material is backfilled into the slots and consolidated by a spud vibrator around the dowel bar. Figure 89, Figure 90, and Figure 91 show that a spud vibrator is inserted in the slot on each side of the foam board to consolidate the patch material around the dowel bar.

Figure 89. Photo. Slot preparation before patching (Bischoff et al., 2002)

Figure 90. Photo. Patching material consolidation (Bischoff et al., 2002)

Figure 91. Photo. Completed placement of patching material (Bischoff et al., 2002)

4) Diamond grinding

Diamond grinding should be within 10 days after the dowel bar placement in order to prevent road users' concerns about the surface roughness. Diamond grinding can restore the road surface smoothness. After diamond grinding, the grinding residue should be removed from the roadway surface immediately.

Issues

During the dowel bar retrofit process, there may be 3 kinds of issues: pavement issues, patch material issues, and dowel bar issues.

Pavement surfaces may have some issues, such as thin pavement, bottom-up deterioration, and wearing slabs (Bischoff et al., 2002).

Patching material issues include poor bonding and poor consolidation, which can be seen in Figure 92 and Figure 93, which show examples from MnDOT.

Dowel bar issues are mainly that the dowel bars are not in a uniform diameter size, which will make the end caps hard to fit.

Figure 92. Photo. Potential pavement issues for dowel bar retrofit (Bischoff et al., 2002)

Figure 93. Photo. Poor bonding and poor consolidation of patching material of MnDOT Specification 3U18.

Performance and Limitations

The performance of dowel bar retrofit can be expressed in terms of pavement life extension, and it is generally believed that DBR can extend pavement life by 10 to 15 years. There are many studies on DBR performance. Georgia and Puerto Rico started their DBR projects in the 1980s (FHWA, 1991; Gulden & Brown, 1987). Washington completed its first DBR project in 1992, and that state has over 10 years of performance data for such projects (Pierce et al, 2003). California also started DBR projects in the 1990s and has extensive performance data.

LTE (Load Transfer Efficiency)

1) LTE comparisons between before and after DBR in Figure 94 show that DBR and DG significantly improve load transfer by approximately 30%.

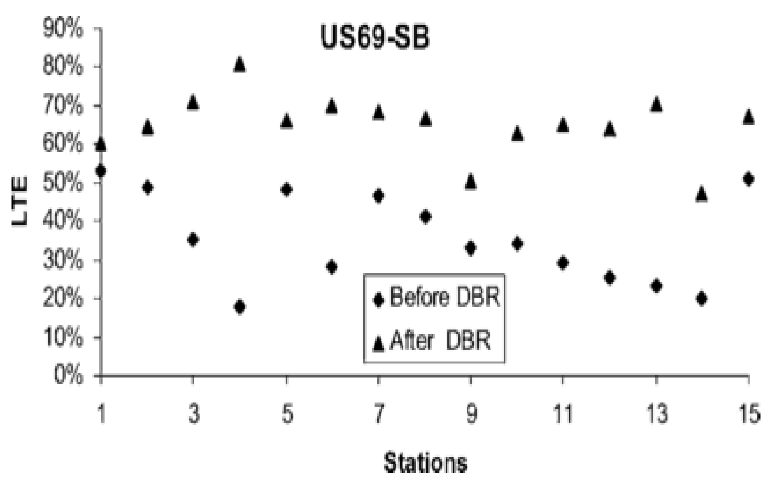


Figure 94. Graph. Improvement in ride quality after dowel bar retrofit

2) In a Minnesota study, it was found that after 14 years of heavy traffic and extreme weather, the mid-panel cracks on the TH 52 project still have an LTE ranging from 60 to

80 percent. Similar results are shown for retrofit dowel bar joints after 10 years of service on TH 23, a previously undoweled pavement now over 56 years old (Thomas & Bernard, 2009).

IRI

It is shown in Figure 95 that the overall average before the DG treatment is 2.98 m/km (189 inch/mile), and after the treatment, it is 1.86 m/km (118 inch/mile), which means that the DBR and DG treatment reduced IRI by 1.1 m/km (70 inch/mile). Figure 96 shows that IRI values decreased immediately after DBR; however, they increased 20 months after DBR (Chen et, al., 2008).

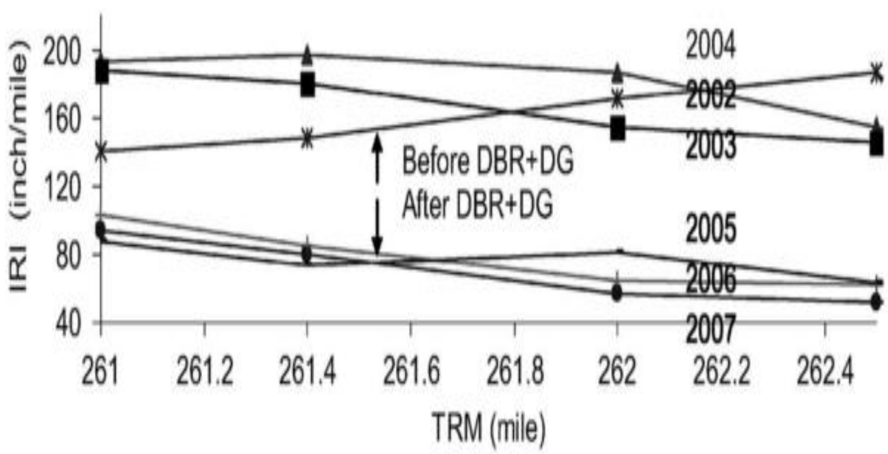


Figure 95. Graph. Improvement in ride quality after dowel bar retrofit (Chen et. al., 2008)

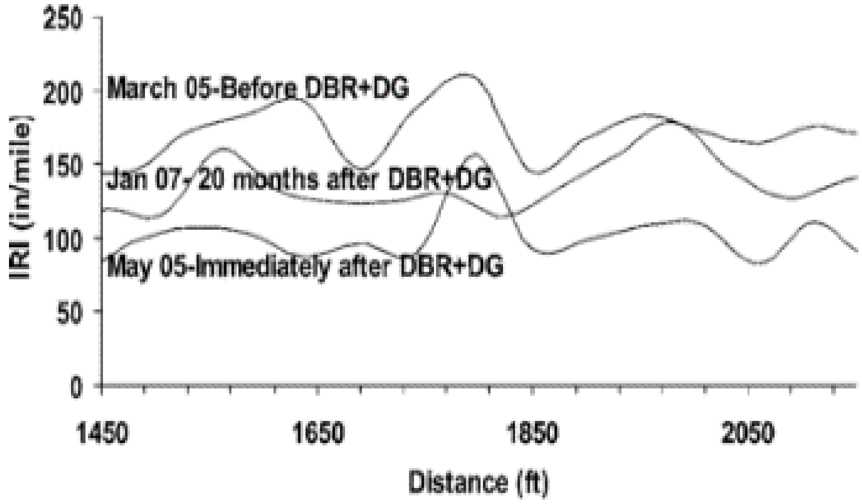


Figure 96. Graph. Comparisons of IRI values on US59 at different treatment times (Chen et. al., 2008)

Table 24. PDI and IRI Before and After DBR (Wisconsin DOT, 2010)

		PDI		IRI	
Highway	Direction	Before	After	Before	After
I-39 (Waushara County)	NB	14	0	1.84	0.67
I-39 (Waushara County)	SB	12	0	1.57	0.75
I-39 (Marquette County)	NB	24	0	1.36	0.85
USH 45	NB	6	6	1.65	0.94
USH 45	SB	6	4	3.52	1.28
STH 21	EB	6	0	2.32	0.90
STH 21	WB	6	0	2.10	0.93
STH 13		Data not available			
USH 18/151					

Cost

In 1997, the price had already dropped from $100 per dowel to $25 to $35 per dowel for routine installations (FHWA, 1997).

In a 2009 Wisconsin paper, the life-cycle cost comparison is listed at the 1999 value (Pierce, 2009).

Dowel bar retrofit, together with diamond grinding, in most cases is a cost-effective method for PCC treatment for concrete pavements that have the faulting and bumping distress but are otherwise structurally sound. The performance of this treatment method relies on appropriate project selection, dowel bar design features, patching materials, and construction quality. It has been proven by many agencies that DBR with diamond grinding can extend the concrete service life, improve rideability, and improve load transfer efficiency.

FULL DEPTH REPAIR

What is Full Depth Repair?

Full depth repair is one method of the concrete pavement restoration (CPR) program which is widely known and used by almost all transportation agencies. It is defined as the cast-in-place concrete repairs that extend the full-depth of the existing slab. Its purposes are to repair localized distress and to make the preparation for an overlay, as necessary, resulting in better rideability, structural integrity, and prevention of further deterioration.

How to choose project for Full Depth Repair

According to California DOT, full depth repair is suitable for transverse cracking with medium and high severity levels; longitudinal cracking with medium and high severity levels; corner breaks with low, medium, and high severity levels; spalling with medium, and high severity levels; blowup with low, medium. and high severity levels; D-Cracking with medium and high severity levels; and deterioration of existing repairs. According to ACPA, the suitable candidates are listed in Figure 97, Table 25, and Table 26.

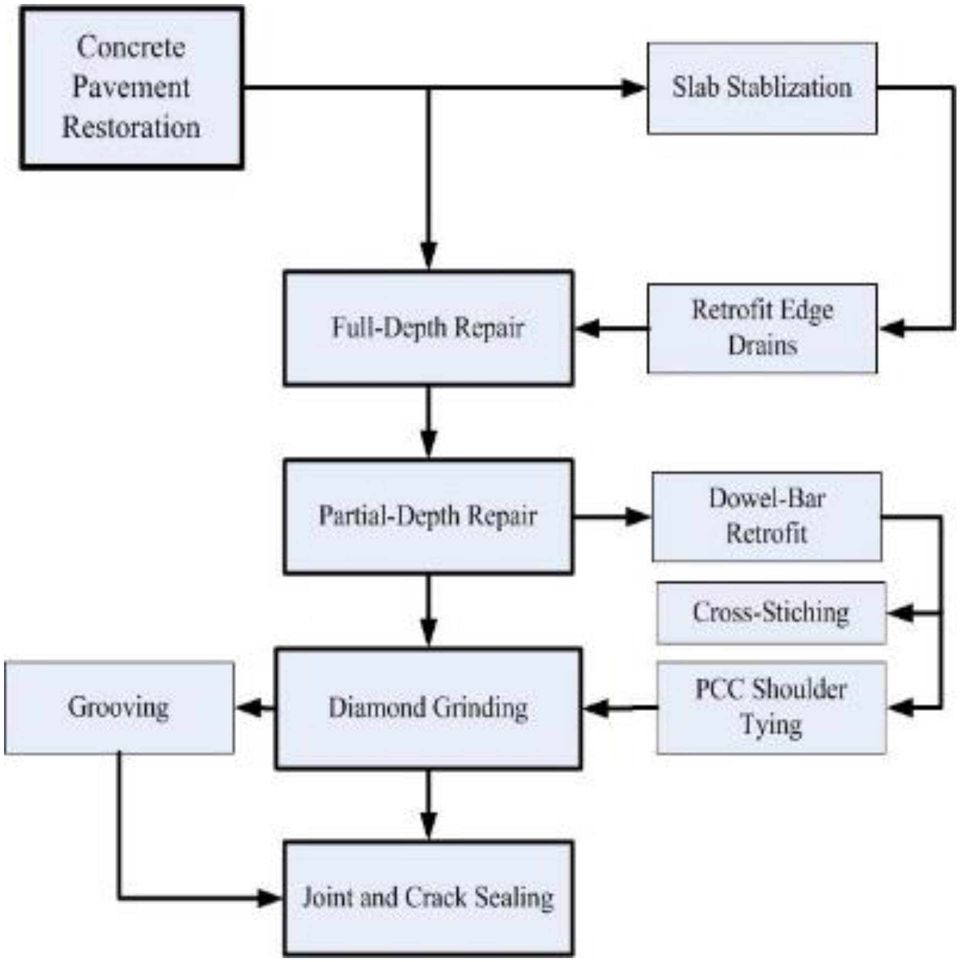


Figure 97. Graph. CPR techniques (ACPA, 1997)

Table 25. Types and Severity of distress that requires FDR (ACPA, 1995)

DISTRESS TYPE	MINIMUM SEVERITY LEVEL REQUIRING FULL-DEPTH REPAIR[2]
JOINTED PAVEMENT:	
Blowup	Low
Corner Break	Low
Durability (D-Cracking, Alkali-Silica Reactivity)	Moderate
Joint Deterioration[1]	Moderate (with faulting ≥ 6 mm [0.25 in])
Random Transverse Cracking[1]	Moderate (with faulting ≥ 6 mm [0.25 in])
Random Longitudinal Cracking[1]	High (with faulting ≥ 12 mm [0.5 in])
CONTINUOUSLY REINFORCED PAVEMENT:	
Blowup	Low
Durability (D-Cracking, Alkali-Silica Reactivity)	High
Punchouts	Moderate (with faulting ≥ 6 mm [0.25 in])
Random Transverse Cracking[1]	Moderate (with steel ruptures & faulting ≥ 6 mm [0.25 in])
Random Longitudinal Cracking[1]	High (with faulting ≥ 12 mm [0.5 in])

[1] *Partial-depth repair is recommended if deterioration is only within upper third of slab.*
[2] *For high-volume highway pavement (low-volume and low-speed roads may tolerate more deterioration; airport pavements may tolerate less deterioration due to concern over foreign-object damage).*

Table 26. WisDOT PCC Pavement FDR project selection

Distress Type	Pavement Type					
	JPCP	JRCP	CRCP	HMA overlaid JPCP	HMA overlaid JRCP	HMA overlaid CRCP
Linear (transverse and longitudinal) cracking and corner breaks	X	X	X			
Reflection cracking				X	X	X
Punchout			X			X
"D" cracking and alkali-silica reaction (ASR)	X	X	X	X	X	X
Joint spalling	X	X				
Blowup	X	X	X			

Material Design

Typical full-depth repair operations utilize concrete mixes containing 390-502 kg/m^3 (658-846 lbs/yd^3) of either cement Type I or Type III cement. A set-accelerator is frequently used to permit opening in 4 to 6 hours. Without the accelerator, these mixes allow opening in 12 to 72 hours. The use of proprietary concrete mixes is necessary to achieve opening times in as little as 2 hours. Using insulating blankets (or boards) during the first few hours after placement can improve the strength development of any mix. Regardless of the mix design used, the concrete mixture for full-depth repairs should have the following properties:

1) 6.5 ± 1.5 percent of entrained air in the concrete (less air may be permissible in no freeze areas); and

2) 50 to 100 mm (2 to 4 inch) slump.

Mixes using Type III cement may require slightly more water than a similar mix with Type I Portland cement. However, too much extra water may cause the concrete to suffer from high shrinkage during curing. A water-reducing admixture will disperse cement particles and reduce the water necessary for workability.

Calcium chloride ($CaCl_2$) or another accelerating chemical admixture is recommended for use as an accelerator in the patching concrete, provided that it is added as specified. It should be noted that initial set may occur within 30 minutes on warm days; therefore, only 1% of calcium chloride by weight of cement should be used when the air temperature exceeds 27° (80°). Up to 2% is acceptable in lower temperatures. For on-site mixing, calcium chloride in liquid form should be added to the mixer before other admixtures are added (except the air-entraining admixture). When using calcium chloride, considerations should be given to the remaining service life of the adjacent pavement and whether dowel bars and reinforcing steel are coated.

If calcium chloride or other accelerating admixture are being added at the plant and the concrete consistently arrives at the site too stiff, then the calcium chloride should be added at the site. If after the addition of calcium chloride at the site the concrete is still too stiff, the ready-mix plant operator should be notified to increase the slump an appropriate amount, up to 150 mm (6 inch).

Chemical admixture may be added to the concrete at the batch plant if the air temperatures are moderate (less than 20°C[68°F]) and the batch plant is less than 15 minutes from the project site. Non-chloride accelerators are recommended for CRCP and JRCP full-depth repairs (FHWA).

Performance and Limitations

The appropriateness of FDR's use, the quality of construction, and the properties of the repair material determine the performance of FDR.

Full-depth repair may perform well but not be cost-effective for a pavement that has little or no remaining structural life and will soon need resurfacing or reconstruction.

Concrete pavement restoration is estimated to have a service life in the range of 5 to 15 years, depending on the combination of techniques used and the extent, materials, and construction quality of each (NCHRP). The American Concrete Pavement Association estimates a service life of 10 to 15 years for full-depth repair on its own.

JOINT SEALING

What is Joint Sealing?

Joint resealing is a pavement preventive maintenance activity that serves two primary purposes. One purpose is to prevent moisture-related distresses by reducing the amount of moisture that can permeate a pavement structure. Too much moisture in the structure could result in distresses, such as pumping, loss of support, joint faulting, corner breaks, and concrete deterioration at or beneath the joint. The other purpose is to prevent the intrusion of incompressible materials, such as sand, pebbles, and other solid debris, so that pressure-related distresses, such as spalling and blowups, are prevented. Keeping joints sealed also helps to reduce noise emissions caused by "tire slap" or "joint slap" (Donavan, 2010), which is the sound tires make when they impact with the pavement joint (SNS, 2011).

Joint resealing is routinely performed by many highway agencies. The survey of the preservation practices of 50 highway agencies in the United States and Canada revealed that 55% of the agencies perform joint resealing on their medium-traffic rural roads, and 78% perform it on their high-traffic urban roads (Peshkin et al. 2011).

The general recommendation for pavements initially sealed at the time of construction is that the joints should continue to be regularly resealed over the life of the pavement in order to minimize the infiltration of moisture and incompressible materials. Joint resealing needs additional preparation of the joint channel, such as the provision of a designed reservoir, and the use of generally higher-quality materials.

How to choose project for Joint Sealing

Joint resealing should be performed when the existing sealant material is no longer performing its intended functions. The missing sealant, sealant that is in place but not bonded to the joint faces, and sealed joints that contain incompressible materials are caused by the loss of sealant material. Figure 98 shows some examples of joint sealant failures.

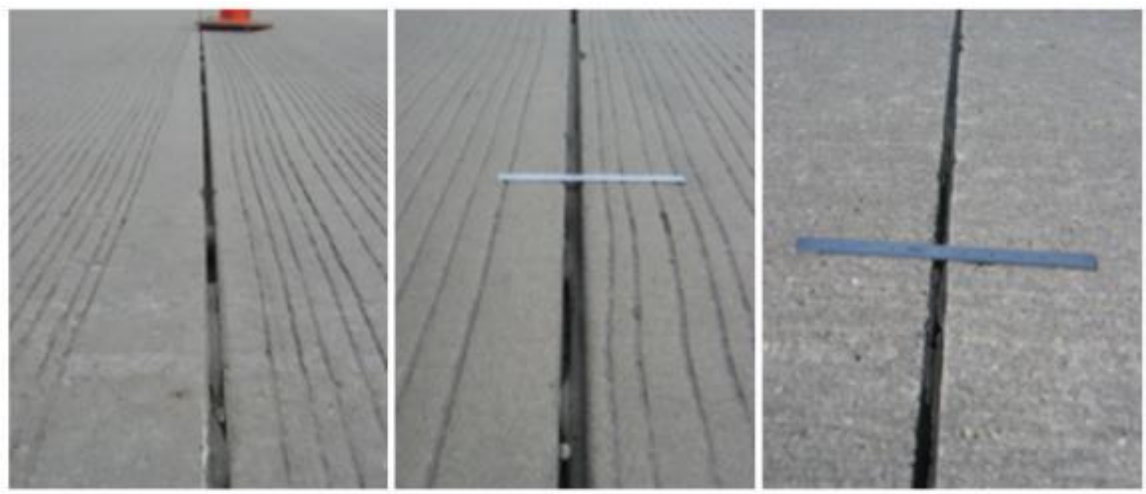


Figure 98. Photo. Examples of joint sealant failures (CP Tech Center)

Some agencies specify that joints be resealed when a certain amount of sealant material (typically 25-50% of the length) has failed to perform one or both of its primary functions. Some other agencies also base their decision on pavement type, pavement and sealant condition, and available funding (Evans et al., 1999). Furthermore, the pavement should still be performing well and be in relatively good condition. The optimum time of the year to perform joint resealing is generally during the spring and fall when moderate installation temperatures are prevalent and the joint width is near the middle of its working range; however, it is also important that the prevailing conditions are dry and that the threat of condensation is low. The greatest benefits from resealing are expected when the pavement is not severely deteriorated and when joint resealing is performed in conjunction with other pavement restoration activities, such as FDR, partial-depth repair, DBR, and diamond grinding.

Material Design

The material selection for joint resealing is dependent on climate condition, joint characteristics and spacing, traffic level and truck percent, and material availability and cost. Table 27 lists some of the hot-applied and cold-applied materials available for sealing joints in concrete pavements.

Table 27. Common Material Types and Specifications for Joint Sealing (CP Tech Center)

Material Type	Specification(s)	Description
Liquid, Hot-Applied Sealants		***Thermoplastic***
Polymerized/Rubberized Asphalts	ASTM D 6690, Type I (AASHTO M 324)	Moderate climates, 50% extension at 0°F (-18°C)
	ASTM D 6690, Type II (AASHTO M 324)	Most climates, 50% extension at -20°F (-29°C)
	ASTM D 6690, Type III (AASHTO M 324)	Most climates, 50% extension at -20°F (-29°C) with other special tests
	ASTM D 6690, Type IV (AASHTO M 324)	Very cold climates, 200% extension at -20°F (-29°C)
Liquid, Cold/Ambient-Applied Sealants		***Thermosetting***
Single-Component Silicone	ASTM D 5893, Type NS	Non-sag, toolable, low modulus
	ASTM D 5893, Type SL	Self-leveling, no tooling, low modulus
Two-Component Elastomeric Polymer (polysulfides, polyurethanes)	Fed Spec SS-S-200E, Type M	Jet-fuel resistant, jet-blast resistant, machine-applied fast-cure
	Fed Spec SS-S-200E, Type H	Jet-fuel resistant, jet-blast resistant, hand-mixed retarded-cure
Solid, Cold/Ambient-Applied Sealants		
Preformed Compression Seals –Polychloroprene Elastomeric (Neoprene) –Lubricant	ASTM D 2628 ASTM D 2835	Jet-fuel resistant preformed seal Used in installation of preformed seal
Expansion Joint Filler		
Preformed Filler Material	ASTM D 1751 (AASHTO M 213)	Bituminous, nonextruding, resilient
	ASTM D 1752, Types I–IV (AASHTO M 153)	Sponge rubber, cork, and recycled PVC
	ASTM D 994 (AASHTO M 33)	Bituminous
Backer Rod (if used)	ASTM D 5249	For hot- or cold-applied sealants

Hot-Applied Thermoplastic Sealant Materials

Thermoplastic sealants are bitumen-based materials that typically soften upon heating and harden upon cooling, usually without a change in chemical composition. These sealants have different elastic and thermal properties. They are also affected to some degree by weathering. Thermoplastic sealants are typically applied in a heated form on concrete pavements.

Polymer-modified/ground tire rubber-modified asphalt is the sealing industry standard. Different types and amounts of polymers and melted rubber are mixed into asphalt cement. The resulting sealants possess a large working range with respect to low-temperature extensibility and resistance to high-temperature softening and tracking. In recent years, softer grades of asphalt cement have been used in polymerized/rubberized asphalts to further improve low temperature extensibility. Figure 99 shows a transverse joint sealed with a hot-applied thermoplastic material.

Figure 99. Photo. Joint Sealed with Hot-applied Thermoplastic Material (CP Tech Center)

Cold-Applied Thermosetting Sealant Materials

Thermosetting sealants are typically one or two component materials that either set by the release of solvents or cure through a chemical reaction. Some of these sealants have shown potential for good performance, but the material costs are generally higher than standard polymerized or rubberized asphalt. However, thermosetting sealants are often placed thinner and have slightly lower labor and equipment costs because they require less time for daily preparation and cleanup. It has no initial material heating and no purging of lines and pumps.

A variety of thermosetting sealant materials is available, including polysulfides, polyurethanes, and silicones. Among these, silicones have been most widely used in pavement applications because they have demonstrated long-term performance capabilities. Polysulfide and polyurethane sealants are not widely used in highway sealing and resealing operations.

Silicone sealants are one-part cold-applied materials that exhibit good extensibility and strong resistance to weathering. These sealants have good bonding strength in combination with a low modulus, so they are able to be applied more thinly than the thermoplastic sealants. Figure 100 shows a project with both the transverse and longitudinal joints sealed with a silicone material.

Figure 100. Photo. Joint Sealed with Silicone Material (CP Tech Center)

Construction and Considerations

After the sealant material has been selected for a joint resealing project, the installation procedure should be given careful attention to ensure the performance of the sealant. It is noted that joint sealing is often required in conjunction with other preservation activities, and the same general steps are followed in those applications. In all cases, successful sealing projects require close attention to detail.

Transverse Joint Resealing

Hot-Applied Thermoplastic Sealant Materials

After the surface preparation and equipment installation, the sealant material should be installed in a uniform manner, filling the reservoir from the bottom up to avoid trapping any air bubbles. For recessed configurations, the joint reservoir should typically be filled no higher than 3–6 mm

(1/8 – 1/4 inch) below the surface of the pavement to allow room for sealant expansion during the summer when the joint closes, thus preventing the sealant from being pulled out by traffic. For flush-fill and overband configurations, the joint reservoir should be overfilled and the sealant struck-off as needed to form the specified configuration. Figure 101 shows the installation of a hot-applied sealant material in a joint resealing project.

Figure 101. Photo. Installation of hot-applied joint sealant

Cold-Applied Thermosetting Sealant Materials

These act in the same way as the thermoplastic materials; after the surface preparation, these silicone sealants should be installed in a uniform manner. The sealants should be installed form the bottom to the top of joint to ensure that no air is trapped. The non-self-leveling silicone sealants must be tooled to force the sealant around the backer rod and against the joint sidewalls. This tooling should also form a concave sealant surface with the lowest point being about 6 mm (1/4 inch) below the pavement surface. Successful tooling has been accomplished using such devices as a rubber hose on the end of a fiberglass rod or a piece of a large-diameter backer rod. Figure 102 shows the tooling operation.

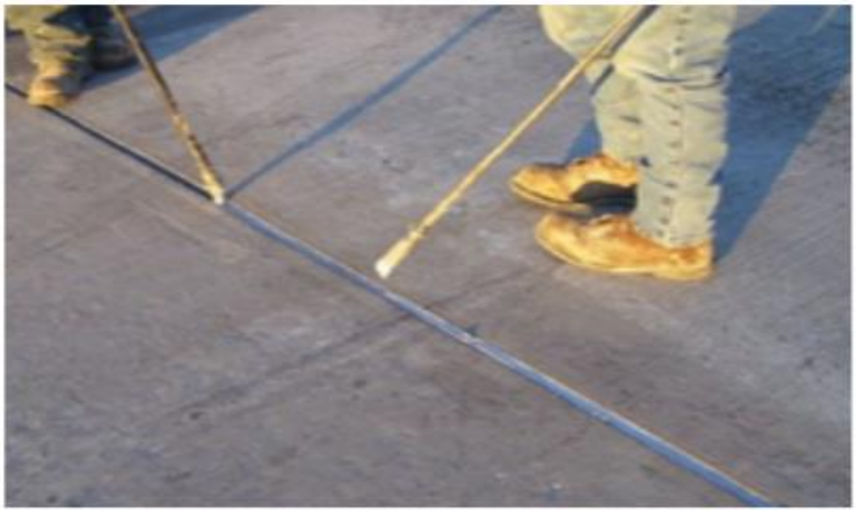


Figure 102. Photo. Sealant Placement at left followed by tooling operation at right (CP Tech Center)

Self-leveling silicone sealants do not require this tooling operation. However, workers should be cautious in the placement of the backer rod for self-leveling silicone sealants because the sealant can easily flow around a loose backer rod prior to curing and may flow out at the joint ends if not properly blocked.

When installing both silicone and thermoplastic sealants, such as in a project with silicone sealant in the transverse joints and hot-applied thermoplastic materials in the longitudinal joint, the silicone should be installed first to reduce the potential of contamination of the transverse joint during the longitudinal joint-sealing operations.

Polysulfide and polyurethane sealants require a curing period to gain their strength and resiliency, similar to silicone. In addition, these sealants require a special application nozzle and careful control of the application equipment.

Weather Considerations

Both hot-applied thermoplastic sealant materials and cold-applied thermosetting sealant materials should be applied only when the air temperature is at least 4ºC (40ºF) and rising (FHWA 2002).

Traffic Considerations

Proper traffic control plans should be developed and maintained before and during the joint resealing. Traffic should not be allowed on the newly sealed joints for about 30-60 minutes after sealant placement to avoid “tracking” of the sealant.

Surface Preparation

The first step of the joint resealing process is to remove old sealant from the joint, along with any incompressible materials. Initial removal can be done by any procedure that does not damage the joint itself. The second step is refacing. It provides a clean surface for bonding with the new sealant and to establish a reservoir with the desired shape factor.

Refacing is generally done using a water-cooled saw with diamond blades. These saws may use a single sawblade or may use multiple blades ganged together to provide the desired cutting width. Figure 103 shows ganged sawblades.

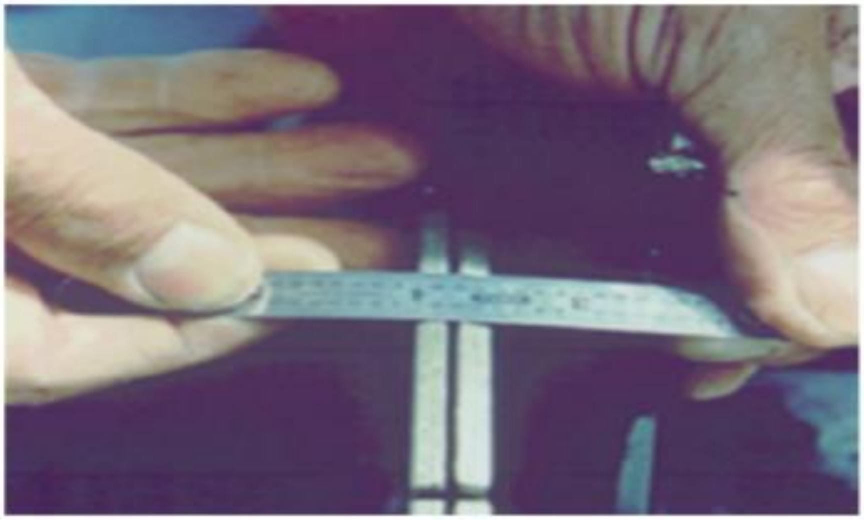


Figure 103. Photo. Ganged sawblades to provide desired cutting width (CP Tech Center)

Generally, a joint should be widened by no more than 2 mm (0.08 inch) during the refacing operation. This will limit the amount of concrete that is removed, increase production, and limit the width of the joint through successive joint resealing operations.

After the joint refacing, the joint should be cleaned with high-pressure air that is followed by light sandblasting. It helps to remove the laitance and any other residue on the joint faces. Air compressors used with the sandblasters must be equipped with working water and oil traps to prevent contamination of the joint bonding faces.

Figure 104. Photo. Sandblasting along a transverse joint (CP Tech Center)

Longitudinal Joint Resealing

Longitudinal joints between adjacent concrete pavement slabs and the longitudinal joint between the mainline concrete pavement and an HMA shoulder should also be addressed as part of a resealing operation. The procedures are almost the same as transverse joint resealing. However, there are some additional considerations.

Longitudinal joints between adjacent concrete slabs are found between adjacent traffic lanes or between a concrete mainline pavement and a concrete shoulder. Because of the limited amount of movement that occurs at these joints, no reservoir is formed, and they are sealed with a hot-poured thermoplastic material. The longitudinal joint between a concrete mainline pavement and an HMA shoulder can be a very difficult joint to seal. The differences in the thermal properties of each material and the differences in the structural cross section often result in large differential horizontal and vertical movements. Also, adding to this movement can be the curling/warping of the concrete, the lack of a tie between the concrete mainline and the HMA shoulder, and frost heave/swelling in the subgrade beneath the shoulder. A sufficiently wide reservoir must be cut in the existing HMA shoulder to allow for the anticipated vertical movements.

Performance and Limitations

The performance of silicone sealants is typically relevant to joint cleanliness, the presence of moisture, and tooling effectiveness. The materials that may contaminate the joint sidewalls include old sealant left on the joint or crack sidewalls, water-borne dust from the sawing operation, oil or water introduced by the compressed air stream, dust and dirt not removed during the cleaning operation, debris entering the joint after cleaning and prior to sealing, and other contaminants that may inhibit bonding, such as moisture condensation. The performance can be also affected by the type of aggregate in the existing concrete pavement. For example, some agencies have noted problems with the adhesion of silicone to concrete containing certain dolomitic aggregates, even when a primer was used (McGhee, 1995). Under such conditions, the use of the silicone sealant should be carefully considered.

CHAPTER 4. CRACK SEALING PERFORMANCE STUDY USING HISTORICAL COPACES DATA

The purpose of this chapter is to address the following three key research questions using GDOT's historical COPACES segment-level survey data from FY 1986 to FY 2015:

1) What are the current GDOT practices for crack sealing timing (e.g. pre-crack sealing COPACES rating)?

2) How many consecutive years of crack sealing survey notes are typically observed?

3) How long is pavement life extended on the crack-sealed segments compared to the non-crack-sealed segments? Would crack sealing extend pavement life? What is the optimal timing for applying the crack sealing treatment?

Three analyses, including a) GDOT's current crack sealing timing analysis, b) consecutive years of crack sealing survey notes' statistical analysis, and c) crack-sealed pavement life extension analysis were conducted in the data analysis section to address these key research questions.

This chapter is organized as 1) data processing, 2) data analysis, and 3) summary.

DATA PROCESSING

Data Description

The data was pulled from the Georgia Pavement Management System, which includes historical COPACES segment-level survey data from FY 1986 to FY 2015. Each entry in the system includes two types of information (segment location information and segment survey information), as described below:

- The segment location information has 14 fields, including the mile post start and end for each segment (“SEGMENTFROM” and “SEGMENTTO”), county number, route number, whether crack sealing was observed on the segment, and the segment rating.

- The segment distress information includes 29 fields with information related to the rutting, block cracking, loading, raveling, and reflective cracking and other distress information.

In addition to segment-level data provided in COPACES, the project-level location information data from COPACES was used to obtain additional information about the road characteristics.

Assumptions of Crack Sealing Information Extraction

To conduct crack sealing effectiveness analysis using segment-level COPACES data, the information about when and where the crack sealing is applied and the segment rating when crack sealing was applied, the COPACES data must be extracted from the dataset. Three categories (“YES,” “NO,” or “left blank”) of the “CRACK SEALED” field in the segment-level COPACES data determine whether crack sealing was observed during the segment-level survey. The following assumptions were followed to extract this information:

- When a segment initially transitions from a “NO” to “YES,” it is assumed that crack sealing was applied somewhere in between the two sequential FYs.

- When a segment transitions from “YES” to “NO” in consecutive years, it is assumed that the crack seal broke, treatment was applied (other than crack sealing), or reconstruction of the roadway occurred.

- If a “YES” is not observed and the “CRACK SEALED” field is empty within the COPACES survey system, it is assumed that no crack sealing was observed during that period of time.

Data Cleaning

This section details the data cleaning procedure used to ensure the quality and consistency of the raw segment-level COPACES data. There are four steps to conduct the data cleaning procedure: 1) zero segment length data removal, 2) segment rating missing data removal, 3) non-asphalt segment data removal, and 4) duplicate data removal. The raw data has 562,648 entries. After data cleaning was conducted, 455,342 entries remained to be used in the later data analysis section in this study. The number of segment entries removed at each step was 0, 391, 4914, and 102,001. Figure 105 provides a schematic of the data cleaning procedure.

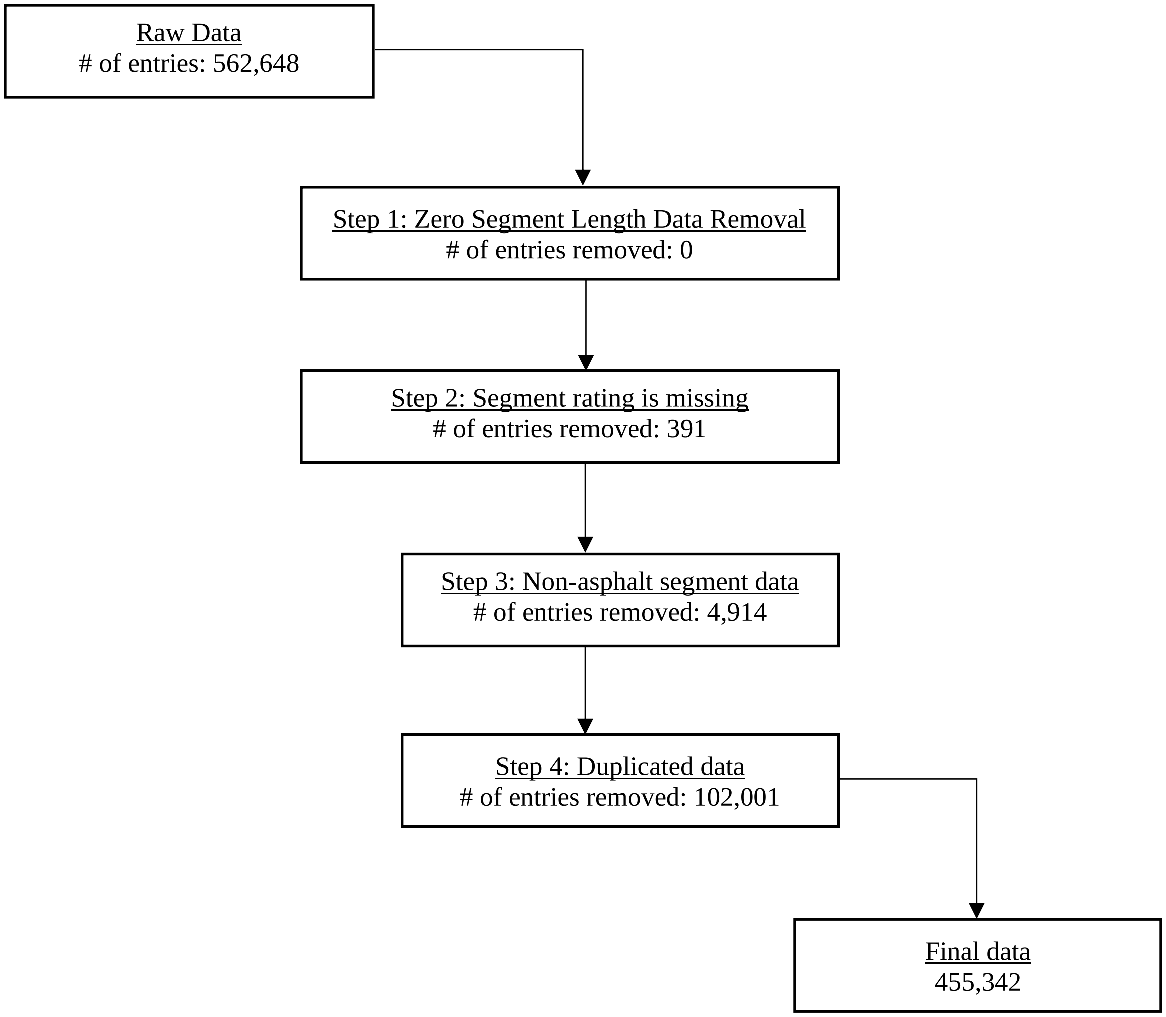


Figure 105. Graph. Data cleaning procedures

- Step 1: Zero Segment Length Data Removal. This step removed entries where MilePostFrom or MilePostTo were both 0's, which indicates a segment length of 0. The number of segment entries removed at this step is 0.

- Step 2: Segment Rating Missing Data Removal. This step removed entries with missing segment ratings. The number of segment entries removed at this step was 391.

- Step 3: Non-Asphalt Segment Data Removal. This step removed entries with non-asphalt projects. The number of segment entries removed at this step was 4,914.

- Step 4: Duplicated Data Removal. Once the previous three steps of data cleaning have been carried out, the RCLINK was created for each segment. A unique segment ID was created to identify entries that analyzed the same areas of the roadway by concatenating the RCLINK, SEGMENTFROM, SEGMENTTO and lane direction fields of each entry. Duplication within COPACES data occurs when there are multiple entries for a unique segment ID within the same year. To determine which entries to remove among the duplicates, the segment ratings before and after the year with the duplicate data were classified into one of three categories: 1) under construction or newly constructed (Segment Rating of 105 or 100), 2) a non-newly constructed segment (Segment Rating from 1-99), 3) missing or an also duplicate entry. Different scenarios created by combining these three categories were used for choosing which entry to keep. Finally, 102,001 of 562,648 entries duplicates were removed in this step.

DATA ANALYSIS

After the data preprocessing, 455,342 entries of segment-level data from FY 1986 to 2015 were used in the data analysis. The three analyses conducted were 1) GDOT's current crack sealing timing analysis, 2) consecutive years of crack sealing survey notes' statistical analysis, and 3) crack sealed pavement life extension analysis. These analyses are presented in this subsection to address the key research questions proposed at the beginning of this chapter.

Terminology was established to properly describe the segment-level data and the following analysis. Table 28 summarizes commonly used terms throughout this subsection as well as their respective definitions.

Table 28. Segment-level data definitions

Term	Definition
Start Year	First year crack sealing was observed on a segment (first year "CRACKSEALED" category was equal to "YES")
Consecutive Count	Total number of years in a row that crack sealing was observed on a segment
End Year	Last year crack sealing was observed on a segment (last year "CRACKSEALED" category was equal to "YES")
Life 70 of Crack Seal	The number of years until a pavement reaches a COPACES Segment Rating of 70 calculated from the Start Year of crack seal application

GDOT Current Crack Sealing Timing Analysis

The question of what are the current GDOT practices on crack sealing timing (e.g. pre-crack sealing COPACES rating) must be answered by plotting the distributions of the segment rating one year before the segment's being crack sealed. Based on the following plotted distribution, it is found that the mean segment rating at one year before crack sealing is 70, which might indicate the current GDOT practices on crack sealing timing is at a mean pre-crack sealing COPACES rating of 70. However, this is not conclusive because there is a lot of missing information at the segment level in the COPACES data.

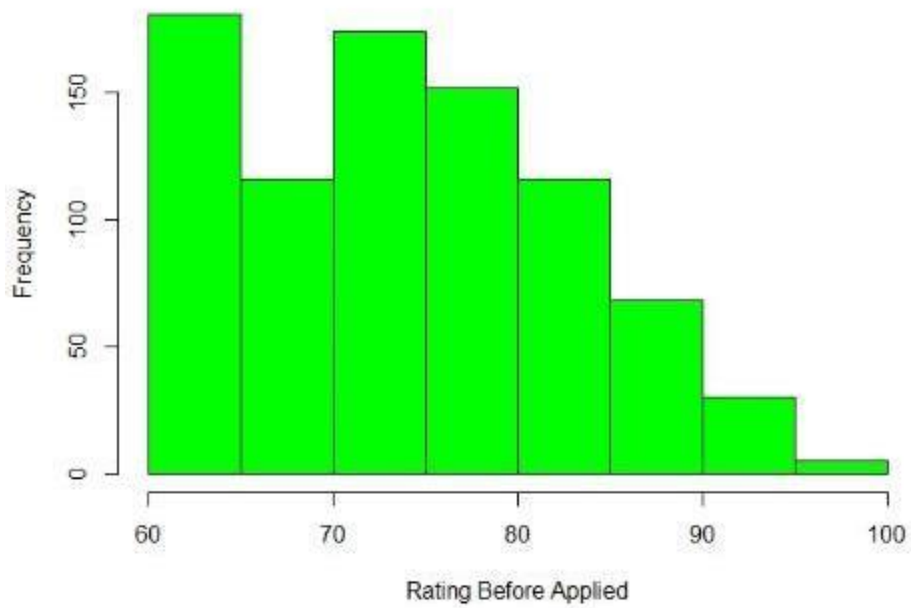


Figure 106. Graph. Distribution of segment ratings at one year before crack sealing

Statistical Analysis on Consecutive Years of "Crack Sealing" Survey Notes

To address the question of how many consecutive years of crack sealing survey notes are typically observed from the COPACES survey data, the figure below shows the distribution of consecutive years of crack sealing being observed. This could potentially link with crack sealing performance. Based on the plotted distribution, the mean value of the consecutive years of crack sealing is 2.087. However, this is not conclusive for using this information to interpret the crack sealing performance or life.

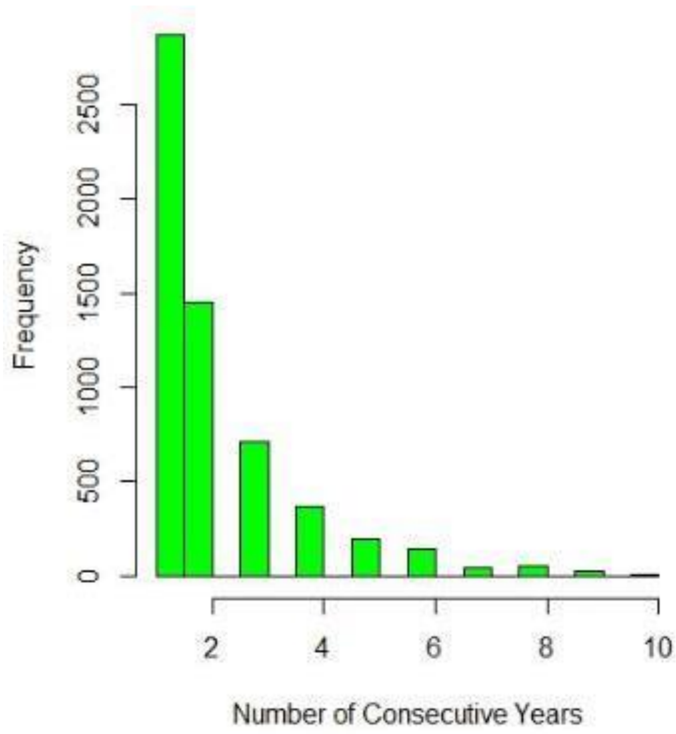


Figure 107. Graph. Distribution of number of consecutive years of crack sealing

Crack Sealed Pavement Life Extension Analysis

The crack sealed pavement life extension is determined by comparing the difference of Life 70 between crack sealed segments and non-sealed segments. Life 70 means the number of years until a pavement reaches a COPACES rating of 70, calculated from the start year of crack seal application. The following figure plots the consecutive COPACES rating of one segment from the start year of crack sealing application until it reaches 70. The survey conducted year was plotted on the x-axis while the segment rating was plotted on the y-axis.

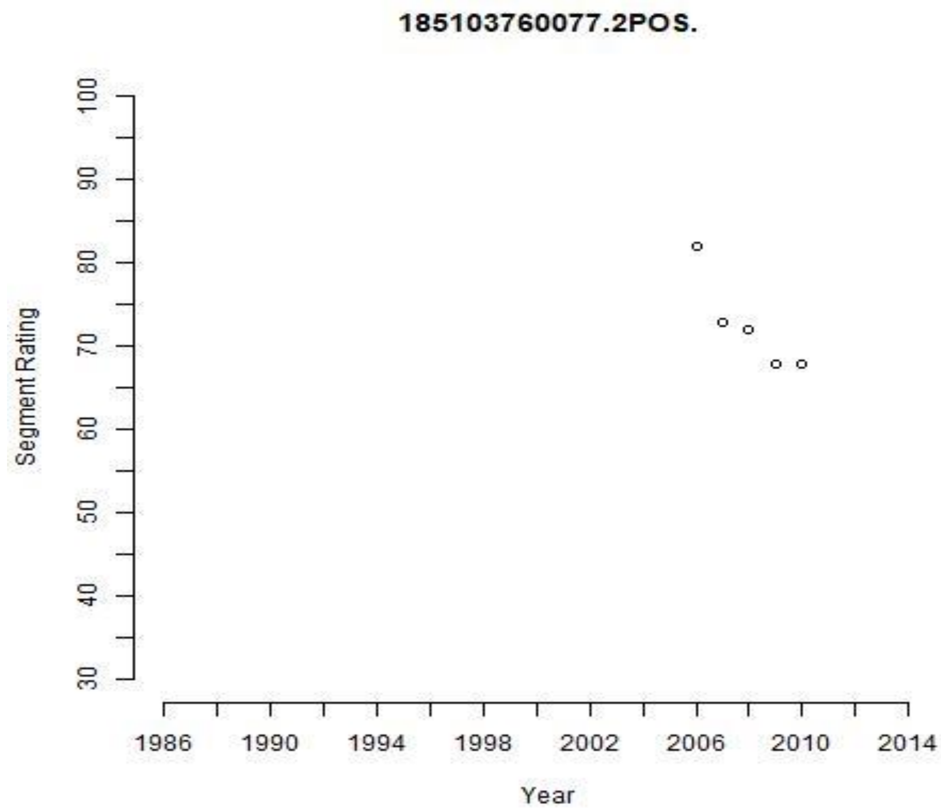


Figure 108. Graph. Plotting of one segment ratings over time

After plotting the segment ratings over time, the following assumptions have been made to calculate the Life 70 of each segment:

1) It is assumed that the deterioration curve for both crack-sealed and non-sealed segments is linear.

2) It is assumed that the minimum consecutive years required to properly predict the Life 70 rating is greater than 4.

3) It is assumed that the minimum acceptable R^2 value for a linear trend needed to properly predict Life 70 has to be greater than 0.50.

Based on these assumptions, linear regression for each segment was conducted. The calculated linear regression equation was then used to predict the Life 70 (number of years from the start year until that segment reached a Segment Rating of 70) for both crack-sealed segments and non-sealed segments. Meanwhile, the pre-crack sealing rating (Start Year Segment Rating) for each segment has also been recorded.

Then, crack sealed segments and non-sealed segments were separately grouped by the different pre-crack sealing ratings from 76 to 100 using 1 as an interval (e.g. 76, 77, …, 100). In each group of both crack sealed and non-sealed segments, the Life 70 were aggregated and averaged, which were recorded in the second and fourth columns in Table 29. The number of segments in each group were also recorded near the average Life 70. Finally, for each row (with the same pre-crack sealing rating) the difference of the average Life 70 between crack-sealed segments and non-sealed segments were calculated and recorded in the last column in Table 29.

Table 29 Summary of average Life 70

Start Year Segment Rating	Crack Sealed Segments		Non-Crack Sealed Segments		Crack Sealed Pavement Life Extension (Years)
	Average Life 70 (Years)	*Number of Segments*	*Average Life 70 (Years)*	*Number of Segments*	
76	2.48	17	2.42	77	0.05
77	3.00	14	2.10	98	0.90
78	2.75	18	2.91	71	-0.16
79	3.64	24	3.16	81	0.47
80	3.54	49	3.33	130	0.20
81	5.07	18	3.36	85	1.71
82	3.01	22	3.74	84	-0.73
83	3.60	24	4.32	137	-0.72
84	6.87	10	3.85	56	3.02
85	3.64	12	3.92	96	-0.28
86	4.50	11	4.10	49	0.40
87	3.70	12	3.84	46	-0.14
88	4.72	18	4.18	84	0.54
89	5.78	8	4.37	54	1.41
90	4.20	8	5.27	84	-1.07
91	2.95	4	5.15	37	-2.20
92	4.92	17	5.22	83	-0.30
93	4.15	3	5.13	13	-0.99
94	9.93	4	5.45	44	4.48
95	N/A	0	6.86	21	N/A
96	2.18	1	4.50	15	-2.32
97	N/A	0	8.75	4	N/A
98	4.24	6	5.06	12	-0.81
99	3.53	1	3.83	1	-0.31
100	3.93	2	9.35	6	-5.42

Meanwhile, the crack-sealed pavement life extension was also plotted with the corresponding pre-crack sealing pavement rating (Start Year Segment Rating) as shown in Figure 109**.**

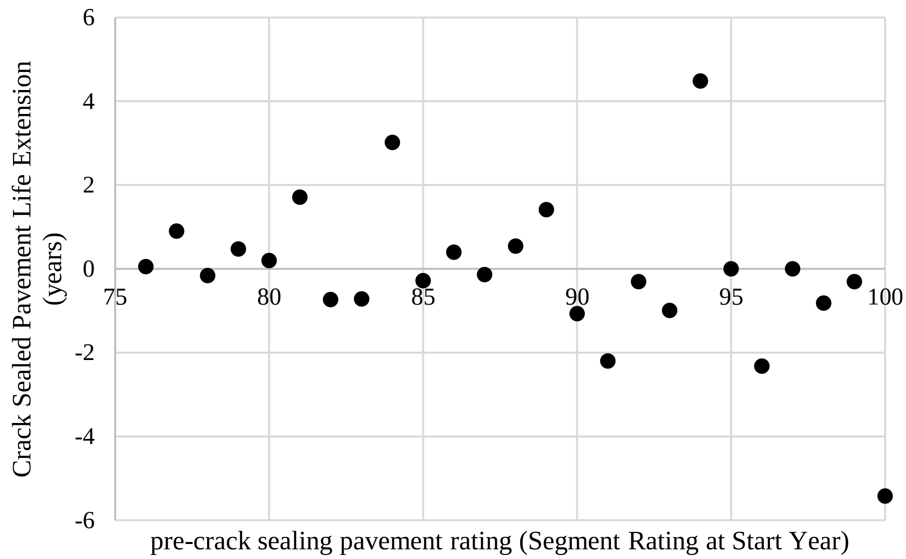


Figure 109. Graph. Crack sealed pavement extension at differing Start Year ratings

Based on results from Table 29 and Figure 109, it is not clear if the crack sealing is effective, and there is no clear trend on the optimal crack sealing treatment timing. More in-depth analysis (e.g. field tests) are recommended to study the crack sealed pavement life extension and the optimal timing of crack sealing.

SUMMARY

This chapter conducted three analyses, including 1) GDOT's current crack sealing timing analysis, 2) consecutive years of crack sealing survey notes' statistical analysis, and 3) crack sealed pavement life extension analysis using GDOT's historical COPACES segment-level survey data from FY 1986 to FY 2015, which tried to answer the proposed key research questions at the beginning of this chapter. However, due to the limitation of data quality, it is difficult to address these questions reliably and scientifically. So, only limited findings were observed in this study; they are presented below:

- The current GDOT practices on crack sealing timing are at a mean pre-crack sealing COPACES rating of 70. However, this is not conclusive because there is a lot of missing information at the segment level in the COPACES data.
- The mean value of the consecutive years of crack sealing is 2.087. However, this is not conclusive for using this information to interpret the crack sealing performance or life.
- It is not clear if the crack sealing is effective or not.
- It is not clear about the crack sealing treatment timing.

Limitations of Data

As stated throughout this chapter, the biggest limitation of data analysis is the quality of the data provided in COPACES at the network or segment level. The following issues were encountered in this chapter.

- As shown in data processing, a large amount of raw data (20%) was removed because of missing data and duplicated data.

    The segment-based data variability in the human pavement condition survey used different 100-ft segments in different years, thereby limiting the analysis of the crack-sealing performance.

Recommendations

Based on the analysis outcomes, it is not clear if the crack sealing is effective and the optimal crack sealing treatment timing. Due to the limitation of crack sealing data quality and data availability, there is a need to perform crack sealing field tests to answer the questions about crack sealing effectiveness in prolonging pavement life and the best timing for crack sealing if it is effective. Therefore, the crack sealing field tests with long-term performance monitoring is performed in CHAPTER 5.

CHAPTER 5. CRACK SEALING EFFECTIVENESS STUDY USING 3D LASER TECHNOLOGY

INTRODUCTION

Crack sealing is the most common asphalt pavement preservation method used by many departments of transportation (DOTs). According to a survey conducted by Decker in 2014, organizations of different sizes in the United States have annually spent from $100,000 to $10,000,000 on crack sealing/filling. The Indiana DOT spent approximately $4,000,000 per year (Fang et al., 2003), and the Alaska DOT spent more than $1,000,000 per year on crack sealing/filling (Mullin et al., 2014). As estimated by Eric Pitts, the former GDOT Maintenance Director and the former Chair of the AASHTO Subcommittee on Maintenance Pavement Technical Working Group, the annual cost of crack sealing is more than $10,000,000 for GDOT. Moreover, due to the continuous aging of pavements in the U.S. and the increasing labor and material costs, crack sealing is more cost-effective than other expensive pavement resurfacing methods for extending pavements' service life.

To achieve the highest cost-effectiveness and maximize the return on investment, especially under the stringent pavement maintenance budget conditions, it is essential to apply crack sealing treatments properly at the right time. If a treatment is applied too soon, only a little benefit can be added; if it is applied too late, it becomes ineffective (Peshkin et al., 2004). In addition, it could negatively impact pavement roughness if crack sealing is applied too early; skid-resistance might, also, be impacted negatively if crack sealing is applied too late.

Therefore, it is essential to quantitatively measure and compare the crack sealing method's effectiveness under different pavement pretreatment conditions (e.g. COPACES rating and crack characteristics) and roadway environment factors (e.g. pavement thickness, traffic volume) to determine the crack sealing effectiveness under different application timings and roadway environments, and verify and refine GDOT's crack sealing criteria based on test outcomes.

Due to the lack of technology and a good method that cost-effectively and quantitatively measures crack characteristics in the field (e.g. crack length, width, orientation, position, etc.), there have been only a limited number of studies on quantifying crack characteristics, including the measurement of crack length, for analyzing crack sealing effectiveness. Other crack characteristics, like crack width, are also valuable to be quantitatively measured in terms of crack conditions/timing for crack sealing application.

With the full-coverage and high-resolution that 3D laser technology provides, there is a great opportunity to extract both sealed and non-sealed crack characteristics with high granularity, including crack length, width, density, direction, and location, and to study the performance and crack sealing effectiveness. 3D laser technology uses the line laser-based 3D triangulation principle to capture 3D pavement surface images and collect 2D intensity images simultaneously. In this report, we use the term "3D laser technology" to represent 3D line laser-based imaging technology; further, we use "2D and 3D images" to represent two different types of images collected using 3D line laser technology. In this project, a 3D line laser system was used to collect both 2D intensity and 3D range images to quantitatively measure the crack length growth of crack sealed and non-sealed pavement sections to evaluate the crack sealing effectiveness; 2D intensity images were used to measure the sealed cracks, and 3D range images were used to detect non-sealed cracks (widths greater than 2 mm) to support the crack sealing effectiveness analysis.

In this study, we will use the term "crack sealing" to represent crack sealing/filling methods, though, according to the crack sealing/filling definition in GDOT's new preservation guild, the commonly used crack treatment method in Georgia is crack filling, which has no routing or cutting. Notice that in a recent survey of 28 state DOTs, 106 counties, and nine other highway agencies, 62% of the 157 respondents, made no distinction between various methods of crack sealing (Decker, 2014).

Research Objectives

GDOT has recently considered broadening its crack sealing criteria and policy to include pavements that have pavement condition ratings equal to and greater than 90. Being able to use 2D/3D pavement images to extract pavement surface crack characteristics (e.g. crack length, width, orientation, position, etc.) and conduct field coring to observe pavement internal crack patterns (e.g. crack depth, internal crack width) provides a great opportunity to address the following key research questions:

- How to quantify crack sealing effectiveness by leveraging the newly availed pavement surface crack characteristics (e.g. crack length, width, orientation, position, etc.), and how should it be measured?
- Is crack sealing effective for slowing down crack length growth? How effective is GDOT's crack sealing practice?

- What roadway factors (e.g. pavement thickness, traffic volume) impact crack sealing performance?

- What pretreatment pavement conditions (e.g. COPACES ratings and crack characteristics) determine the optimal timing for crack sealing? How effective is crack sealing when it is applied to pavements with a COPACES rating equal to or greater than 90? How effective is the crack sealing at various COPACES ratings?

- How does crack sealing impact the propagation of pavement internal crack patterns (e.g. crack depth, internal crack width)?

To answer these key research questions, this study addresses the following objectives:

- Designing and performing a field test study on real-world pavements to quantify crack sealing effectiveness by comparing the pavement surface crack length growth (quantitatively measured using 2D/3D pavement surface images) of sealed sections and non-sealed sections.

- Conducting a crack sealing timing analysis by studying the impacts of pretreatment pavement conditions (e.g. COPACES ratings and crack characteristics) and roadway factors (e.g. pavement thickness, traffic volume) to determine the crack sealing optimal timing.

- Verifying that crack sealing effectiveness is better at higher COPACES ratings (such as a rating of 90) to support new GDOT crack sealing policies and criteria that will significantly improve pavement maintenance decisions.

- Analyzing crack sealing impacts on the pavement internal crack propagation by using field coring data and pavement surface crack patterns (e.g. transverse or longitudinal cracks, newly grown cracks, etc.) information extracted from 2D/3D pavement surface images.

Literature Review

A comprehensive literature review of crack sealing has been performed in Chapter 2. In this subsection, studies relating to crack sealing effectiveness have been summarized to support the analysis in this chapter.

How is the crack sealing effectiveness defined and how is it measured?

Different measurement methods of crack sealing effectiveness have been proposed by many researchers. The literature review has revealed that the crack sealing effectiveness can usually be measured by comparing the difference between crack-sealed sections of pavement and non-sealed sections of pavement. Measurement methods of the crack sealing effectiveness could be 1) measuring the prolonged pavement service life (Rajagopal et al., 2011; Ponniah et al., 1996) by comparing how long it takes crack sealed sections and non-sealed sections to drop to a certain score in the pavement condition index (PCI); 2) measuring the gain in the pavement condition index (PCI) right after the crack sealing is applied (Rajagopal et al., 2011); and (3) comparing the difference in crack length growth among crack-sealed sections and non-sealed sections at specific timings after crack sealing has been applied (Lee et al., 2015; crafco.com, 2020).

The advantage of using the PCI for the first and second methods is that the PCI measurement method is mature and efficient; however, the disadvantage is that the PCI as an aggregated pavement performance index is easily biased by the other pavement distresses (e.g. rutting and raveling). In addition, another disadvantage of the second method (comparing the PCI gain right after sealing) is that some DOT protocols (e.g. GDOT's COPACES manual) do not differentiate sealed cracks and non-sealed cracks; so, there is no rating gain after cracks have been sealed. The third method makes a fair comparison of the crack growth by comparing the crack length growth and failed areas development, but due to the lack of an efficient method to quantitatively measure the crack characteristics, only a few studies using this method have been done, and most of them used the traditional measuring wheel (Zinke et al., 2006) and visual survey means (Vargas-Nordcbeck et al., 2020) to measure the crack length and failed areas. However, in one study, a Purdue University research team collaborated (Lee et al., 2015) with Indiana DOT (IDOT), and they used a digital highway data vehicle (DHDV) with PaveVision3d Ultra from Waylink Systems Co. to collect pavement 3D images to see the crack growth and the effectiveness of sealing. In addition, it was found that non-sealed/untreated control sections had usually been set up to compare with the crack sealed/treated test sections for a comparative study (Ponniah et al., 1996; Rajagopal et al., 2011).

Is crack sealing effective for slowing crack propagation?

Based on the method of measuring prolonged pavement service life, it was found that the majority of the studies indicated that crack sealing prolonged pavement lifecycles for 2-5 years if crack sealing was applied properly (Morian et al., 1997; Ponniah et al., 1996; Bausano et al., 2004), especially at the right timing (e.g. pretreatment pavement condition and crack characteristics) and under the right roadway environments (e.g. pavement thickness, pavement design, traffic volume, truck percentage). Extending pavement service life by applying crack sealing can save money by complementing expensive rehabilitation and reconstruction to achieve optimal asset management. The two mechanisms to explain how crack sealing slows down crack growth are:

1) preventing water from entering the base and subbase to prevent subgrade softening, and

2) preventing sand from entering the crack tips to prevent concentration stresses for growing the crack depth.

However, the real-world crack sealing effectiveness (based on GDOT's practices under different pavement condition variables) has not yet been verified. Therefore, research is needed to quantitatively measure and verify crack sealing effectiveness by comparing the difference of crack growth between non-sealed control sections and crack sealed test sections in a real-world environment.

What are the factors impacting crack sealing performance?

The possible factors impacting the effectiveness of crack sealing are summarized in CHAPTER 2. They are 1) pretreatment pavement condition (e.g. PCI, COPACES rating, etc.), 2) pretreatment crack characteristics (e.g. crack length), 3) pavement type and thickness, 4) crack sealing techniques, 5) sealant selection, 6) seasonality, 7) crack preparation methods, 8) crack sealing finishing methods, 9) traffic volume (e.g. AADT), and 10) routing geometry.

Because of the availability of data and the importance of these factors in Georgia's roadway environment, the research team has only presented a summary of the literature review concerning four selected factors (their impacts on crack sealing effectiveness are studied later):

- Pretreatment pavement condition.

  Pretreatment pavement conditions are typically measured using composite condition ratings (e.g. pavement condition index (PCI), pavement condition rating (PCR), etc.). By applying crack sealing under different pavement pretreatment conditions, the crack sealing effectiveness varies. State DOTs usually have their own rating systems to determine the crack sealing application timing. GDOT uses the Computerized Pavement Condition Evaluation System (COPACES) rating (GDOT, 2007) to determine crack sealing timing ($70 \leq$ COPACES ratings ≤ 85). Another comprehensive study showed that the maximum performance gain is achieved for all pavement and aggregate types when crack sealing is performed while the PCR of the existing pavement is in the range of 66-80 (Rajagopal et al., 2003). Pretreatment pavement condition is a key factor impacting crack sealing effectiveness; therefore, we selected ten test sites with different pretreatment COPACES ratings ranging from 66-95 for studying its impacts.

- Pretreatment crack characteristics.

  For pretreatment crack characteristics, the crack type, including crack length, crack width, and crack orientation (transverse or longitudinal), is an important factor (Masson et al., 1997). Our project collected detailed 3D data of cracks that had crack width, crack length, and crack orientation information.

- Pavement type and thickness.

  For pavement type, Rajagopal's study (Rajagopal et al., 2003) finds that the average performance gain in PCR for composite pavements is relatively higher than that of flexible pavements. Pavements with gravel in the surface layer were found to display relatively higher performance gain. Our research didn’t define the type of the pavement; however, to consider the impact of the original pavement design for crack sealing effectiveness, we did coring for every test site. From the coring data, we can determine the thickness of a pavement. Then, we can evaluate the impact of pavement thickness.

- Traffic volume.

  For traffic volume, Johnson et al. (2000) stated that load cracks were mainly caused by traffic load. They found that the crack sealing of pavements with low traffic performed better than the crack sealing of pavements with high traffic. We chose test sites with different traffic volumes to check if the crack sealing effectiveness of these sites varied.

What is the optimal timing for crack sealing?

The criteria used for determining crack sealing timing is different in different states. Many institutes and researchers have tried to create principles to guide optimal crack sealing treatment. However, there are no unified solutions, so far. The following paragraph lists some criteria that have been used by DOTs and some were developed by researchers.

Georgia DOT (GDOT, 2007) has detailed the criteria for the timing to select crack sealing as the pavement preservation method: 1) a COPACES rating of 80-85 (crack development is at an early stage); 2) the crack width is greater than 1/8 inch, and 3) transverse/block cracking is caused by weathering, and the early stages of longitudinal cracking are caused by loading. Michigan DOT’s standard of applying crack sealing is when the minimum life of a road is more than 10 years, the distress index (DI) is smaller than 15, the international roughness index (IRI) is less than 54, and rutting depth (RD) is smaller than 3 mm. The Connecticut Transportation Institute concluded that crack sealing should be applied when the pavement serviceability rating is between 6-7 (Zinke et al., 2006). The South Dakota DOT's guidelines (SDDOT, 2010) indicates that crack sealing should be applied when there is a crack severity level of low (e.g. fine parallel cracks) with all levels of extending and when a feasible treatment could be applied to the severity levels of medium (e.g. an alligator pattern has developed) with a low rating (i.e. 1%-9% of the wheel path is affected) and moderate (i.e. 10%-24% of the wheel path is affected) with all levels of extending. The crack density and percentage of crack length had been used as guidelines to determine the type of maintenance (Smith, 1999). The guidelines state that the crack treatments should be applied to pavements with low crack density having a moderate crack length (26%-50%) or to pavements with moderate crack density having low (0-25%) and moderate (26%-50%) crack length. Another principle for determining whether cracks should be sealed or filled or not is the condition of horizontal movement cracks (Wu et al.,2010). Working cracks with limited edge deterioration should be sealed, while non-working cracks should be filled. The Indiana Department of Transportation (INDOT) in 2016 used cores to determine actual crack depth for determining the crack sealing timing; INDOT said if the surface cracks do not exceed 3/8 inch, crack sealing would be an appropriate treatment method.

In summary, it can be concluded that the commonly used criteria indicators are 1) pretreatment pavement conditions (e.g. PCI, COPACES rating, age, distress Index, IRI), and 2) pretreatment crack characteristics (e.g. crack length, crack width, crack density, orientation, working or non-working crack). In this study, we will analyze the pretreatment pavement condition (using COPACES ratings) and pretreatment crack characteristics (using crack length and orientation) impacts on crack sealing effectiveness to verify some of these criteria.

Proposed Methodology

A methodology for quantifying crack sealing effectiveness is proposed in this study. It consists of four components: 1) field test design, 2) data collection and acquisition, 3) data processing, and 4) data analysis. A flowchart showing the four components of the proposed methodology is shown in Figure 110.

The field test design component includes the following: 1) design of two test scenarios (including both expedited and progressive test designs); 2) design of crack-sealed test sections and non-sealed control sections; and 3) site selection.

The data collection and acquisition component includes the following: 1) traffic volume and the pretreatment/initial COPACES rating data acquired from GDOT's database; 2) two instances (2016 and 2020) of field coring data collections; and 3) multiple runs of 2D/3D pavement image data collections, including one run before the crack sealing application to capture pretreatment crack characteristics and routine (every 3 months) data collection after crack sealing application.

The data processing component includes the following: 1) lane marking detection to find the range of interest (ROI); 2) multiple timestamp data registration; 3) crack map digitization; and 4) crack length computation.

The data analysis component includes the following: 1) plotting crack growth, 2) quantifying crack sealing effectiveness, 3) analyzing crack sealing timing, and 4) analyzing crack sealing impact on pavement internal crack width/depth growth.

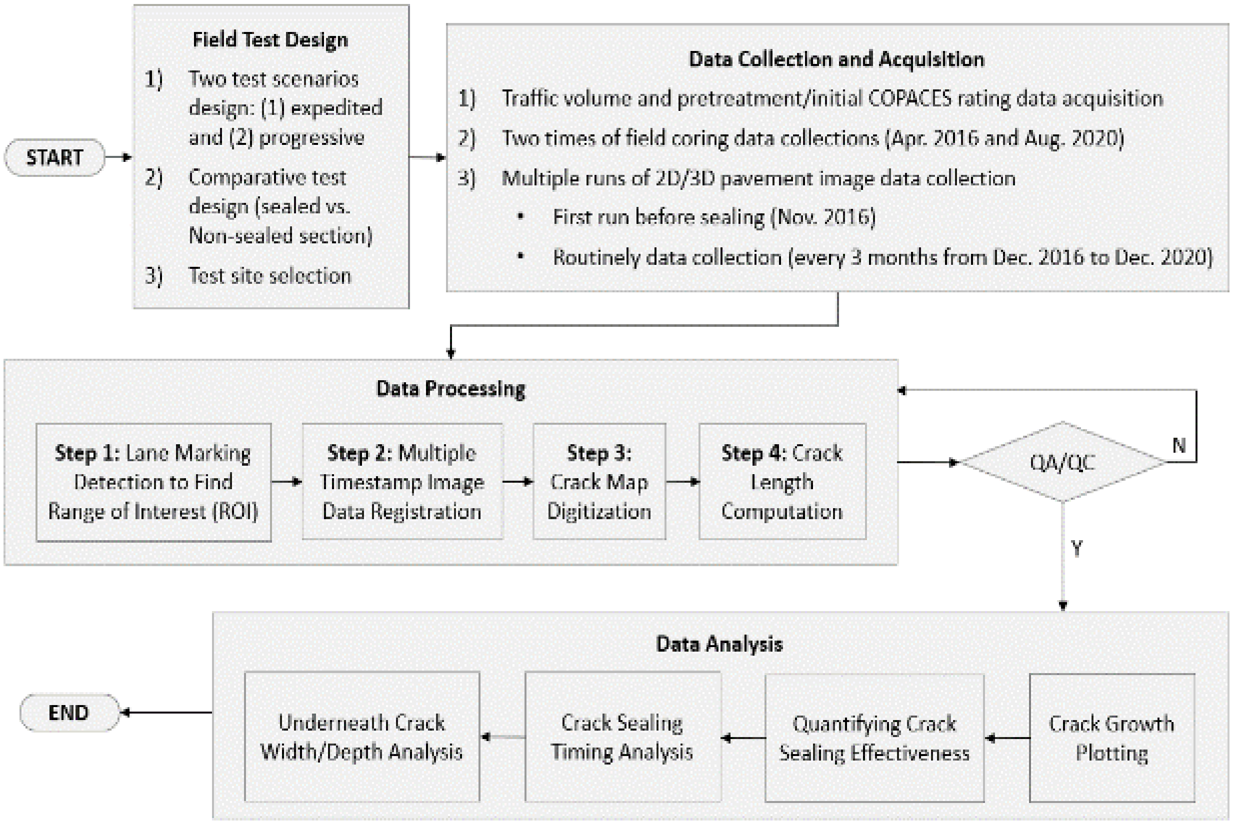


Figure 110. Graph. Flowchart: Components of the Proposed Methodology

Chapter Organization

This chapter is organized as follows. This section introduces the research background, research needs, literature review summary, research objectives, and the proposed methodology. The following section presents the field test design and data acquisition. Section 3 presents the data processing and data analysis and the results. Section 4 presents conclusions and recommendations.

FIELD TEST STUDY

In this section, the field test design and the corresponding considerations are introduced. Then, the field crack sealing operation is introduced. Finally, data collection and acquisition methods are introduced.

Field Test Design

The field test design component includes design of two test scenarios (the expedited and progressive test design), and site selection. The following subsections will introduce them in detail.

Design of Two Test Scenarios

The following three different types of pavement condition variables affecting crack sealing effectiveness are analyzed in this study:

1) Pavement thickness;

2) Traffic volume, indicated by the annual average daily traffic load (AADT) and truck traffic load (AADTT);

3) Crack sealing timing (pavement pretreatment condition), indicated by COPACES rating and crack characteristics (e.g. the amount of wheel-path longitudinal cracking, transverse cracking, and block cracking).

Two test design scenarios are presented in this subsection to support the data collection and analysis for this crack sealing effectiveness study. The first is an expedited test design scenario, and the second is a progressive test design scenario.

The first scenario (expedited test design) is designed to expedite the test outcomes for the crack sealing timing analysis; it applies crack sealing simultaneously at different locations with different pavement pretreatment conditions (e.g. COPACES ratings and crack characteristics).

From this, we can quantitatively compare the differences in crack sealing effectiveness for different crack sealing timings. This assumes that roadway environment factors, especially subgrade type and construction materials, are similar, although the pavement thickness and traffic volume of the pavement varies. The advantage is that we can obtain test outcomes in a short period of time, and we can also analyze the impact of pavement thickness and traffic volume; the disadvantage is that the roadway environment factors of these sections may not be the same.

The second scenario (progressive test design) is designed to have consistent/exactly-the-same roadway environment factors (e.g. pavement thickness, traffic volume, subgrade type, and construction material), but different crack sealing application timings (e.g. different years) with different pavement pretreatment conditions (e.g. COPACES rating and crack characteristics) to evaluate the crack sealing effectiveness. The advantage of the second scenario is that it controls the difference of roadway environmental factors for analyzing only the impact of crack sealing timing; the disadvantage is that it will take several years to perform the tests.

Thus, the one-time crack sealing layout and the multiple-time crack sealing layout are designed for these two tests. The detailed layouts are presented in the following subsection.

Moreover, to analyze the impacts of different pavement condition variables on the crack sealing effectiveness, the selection of test sites is considered and covers a wide range of pretreatment COPACES ratings (range 66-98), variable traffic volumes (AADT range 930-12,200; AADTT range 128-1,145, and different pavement thicknesses (2.5-10 inch). The detailed conditions and locations of the selected sites are presented in the site-selection subsection.

Design One-Time and Multiple-Time Crack Sealing Layout

To quantitatively measure and compare the crack sealing effectiveness under different conditions, both the one-time and multiple-time crack sealing layouts contain at least one non-sealed control section and several sealed test sections. This enables us to compare the difference of crack growth between non-sealed control sections and crack-sealed test sections to evaluate the "net gain," which indicates the crack sealing effectiveness.

Figure 111 shows the one-time crack sealing layout for the first test scenario (expedited test design). It includes two 500-ft sealed test sections and one 500-ft non-sealed control section. Notice that two sealed test sections are not adjacent but sealed simultaneously. One non-sealed control section is in the middle. So, with the one-time crack sealing layout at different test sites, we can assess effectiveness under different pretreatment pavement conditions (e.g. COPACES rating and crack characteristics) in a short period of time, although the assessment is indirect because the roadway factors (e.g. pavement thickness and traffic volume) might be different.

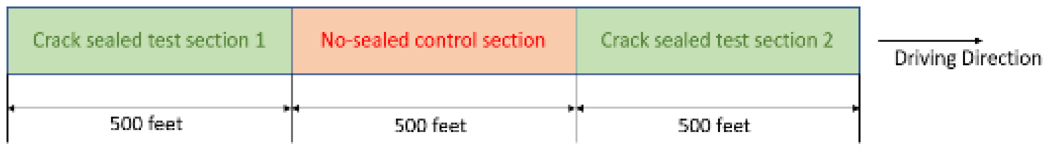


Figure 111. Graph. Test Site Layout with One-time Crack Sealing Applications

Figure 112 shows the multiple-time crack sealing layout for the second scenario (progressive test design). Several crack sealed test sections are sealed at different timings (e.g. different years) with different pretreatment pavement conditions (e.g. COPACES ratings and crack characteristics). In December 2016, only the first crack sealed test section (T1) was sealed. In February 2018, the second crack sealed test section (T2) was sealed. Thus, crack sealing at different timings (i.e., different years) with consistent/exactly-the-same roadway factors (e.g. pavement thickness, traffic volume, subgrade type, and construction material) could be analyzed.

Figure 112. Graph. Test Site Layout with Multiple-time Crack Sealing Applications

Table 30 summarizes the layout information of the 9 testing sites used in this study. Multiple-time layout Sites 2, 4, and 5 were originally designed for five crack sealing timings. However, before the third crack sealing application, they were sealed by normal pavement preservation. Therefore, only two crack sealing timings were implemented in our study.

Table 30. Summary of Site Layout

Site ID	Layout Type	Number of Sealed Sections	Number of Non-sealed sections	Sealed Timing
1	one-time	2	1	Dec. 2016
2	Multiple-time	2	2	Dec. 2016 and Feb. 2018
3	one-time	2	1	Dec. 2016
4	Multiple-time	2	2	Dec. 2016 and Feb. 2018
5	Multiple-time	2	2	Dec. 2016 and Feb. 2018
6	one-time	2	2	Dec. 2016
8	one-time	2	1	Dec. 2016
9	one-time	2	1	Dec. 2016
10	one-time	2	1	Dec. 2016

Site Selection

Following the test design presented previously, 10 test sites were originally selected from Georgia's state highways. Test Sites 1-6, which are located near Hawkinsville (in south Georgia), are shown in Figure 113. Test Sites 7-10 are near Covington (in central Georgia), as shown in Figure 114.

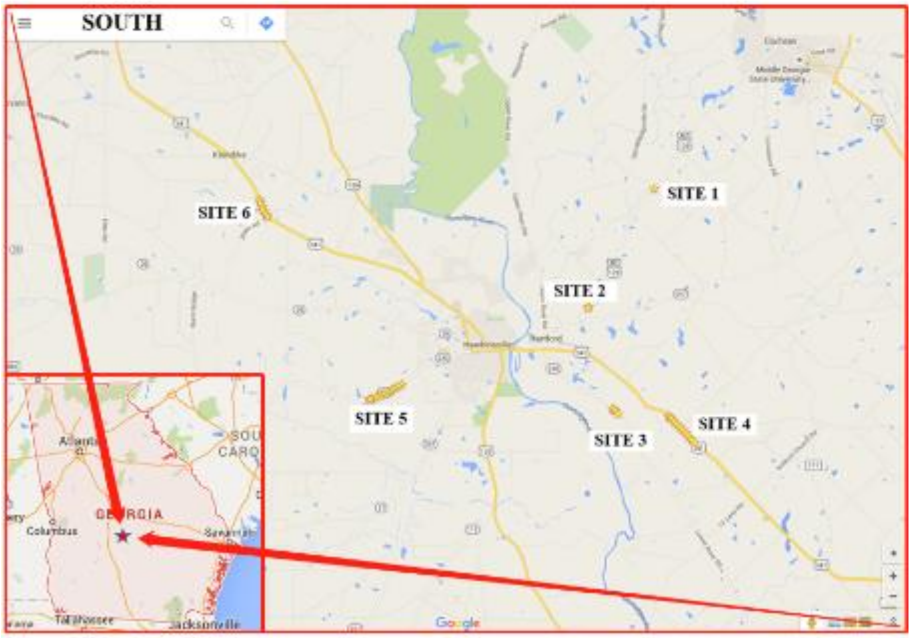


Figure 113. Graph. Location of Test Sites 1-6

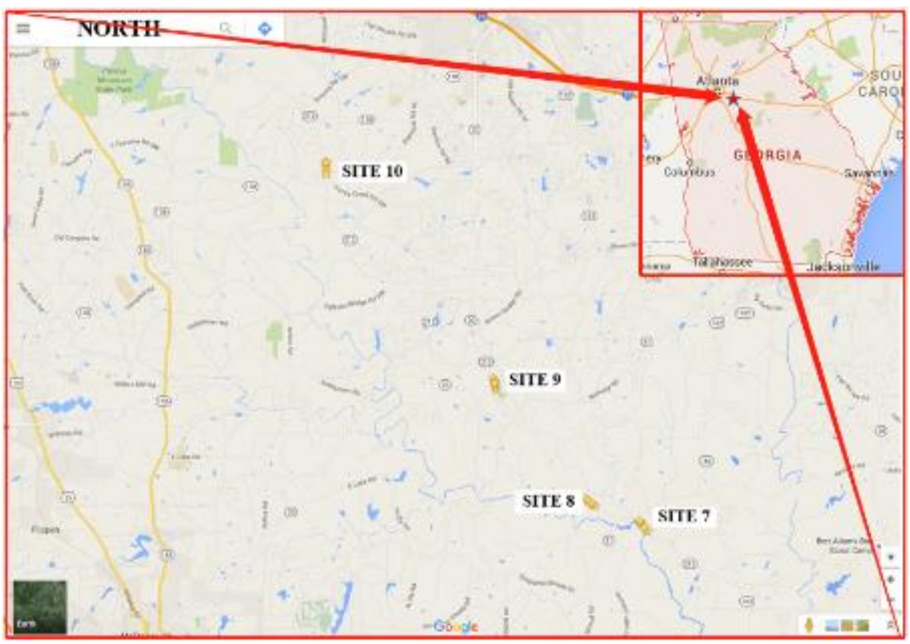


Figure 114. Graph. Location of Test Sites 7-10

Table 31 summarizes all test sites. All of the selected sites are asphalt pavements. The traffic volume ranges from low to high and is discussed in the next data collection section using annual average daily traffic (AADT) and annual average truck traffic (AADTT). To evaluate the impact of pretreatment pavement conditions, 4 sites are in good condition; one is in medium condition, and 5 are in poor condition.

Table 31. Summary of Selected Sites

Site #	Road Name	RCLINK	Mile Point	Pavement Structure	Traffic	Crack Condition	Total Length (ft.)	Test Sections
1	GA 26	0231002600	0.76	Composite	High	Poor	1500	2T/1C*
2	GA 26	2351011200	14.96	Composite	High	Good	4500	5T/4C
3	GA 230	2351023000	13.67	Thin AC layer	Low	Poor	3000	4T/2C
4	US 341	2351002700	16.70	Full depth	Medium	Good	4500	5T/4C
5	GA 230	2351023000	7.09	Thin AC layer	Low	Good	4500	5T/4C
6	US 341	2351001100	18.12	Full depth	High	Poor	2500	3T/2C
7**	GA 212	2171021200	6.66	Thin AC layer	High	Poor	1500	2T/1C
8	GA 212	2171021200	5.20	Thicker AC layer	High	Good	3000	4T/2C
9	GA 212	2171021200	1.55	Thicker AC layer	High	Medium	3000	4T/2C
10	GA 212	2471021200	2.89	Thicker AC layer	High	Poor	1500	2T/1C

* “T” represents the test section (crack sealed section); “C” means the control section.

** Site 7 was resurfaced in 2017; it is not included in the study.

Crack Sealing Field Operation

Starting in November 2016, the first crack sealing application was conducted by GDOT. Based on our communication with GDOT engineers, crack sealing by GDOT is typically applied from September to December and March to June because there is less thermal expansion and contraction to impact the crack sealing effectiveness. Type S asphalt-rubber material is used according to GDOT specification ("Section 407 – Asphalt-Rubber Joint and Crack Seal"). It is a hot-applied thermoplastic material that can seep in and fill small cracks due to its very low viscosity. Because the purpose of this study is to evaluate crack sealing effectiveness, GDOT's conventional method was adopted to seal cracks. There is no routing and cutting before crack sealing. As shown in Figure 115, the cracks were cleaned first using compressed air. Then, a hot-pour sealing method was used. According to GDOT practices, no routing is applied for the crack sealing on asphalt pavement. Please refer to Chapter 2 for the detailed crack sealing operation.

(a) Crack Clean (clean first) (b) Crack

*Crack sealing application operation at February 11th, 2020, at site 8 (Hwy 212, Covington, GA)

Figure 115. Photo. GDOT Crack Sealing Operation.

Data Collection and Acquisition

The data collection and acquisition component includes 1) traffic volume data, 2) initial COPACES rating data, 3) coring data for pavement thickness and subbase type, 4) 2D/3D pavement image data (including initial data collection before the crack sealing application and time-series data collection in quarterly collections), and 5) a summary of data collection and acquisition.

Traffic Volume Data Acquisition

Traffic volume is an important roadway factor impacting crack sealing effectiveness. The following table summarizes the traffic volume data using annual average daily traffic (AADT), percentage of the trucks, and annual average daily truck traffic (AADTT). Since trucks create much heavier loads on the pavement, they have a higher impact on the crack sealing effectiveness. Therefore, the annual average daily truck traffic (AADTT), calculated by AADT multiplied by the percentage of the trucks, is incorporated in this study.

Table 32. Summary of Traffic Volume Data

Site #	2016			2017			2018		
	AADT	Truck (%)	AADTT*	AADT	Truck (%)	AADTT*	AADT	Truck (%)	AADTT*
Site 1	4400	26%	1144	4540	N/A	N/A	4430	26%	1152
Site 2	4210	20%	842	4340	N/A	N/A	4480	24%	1075
Site 3	940	13%	122	970	N/A	N/A	1020	13%	133
Site 4	3900	17%	663	4020	N/A	N/A	3890	17%	661
Site 5	930	13%	121	960	N/A	N/A	1030	14%	144
Site 6	4710	15%	707	5230	14%	732	5260	14%	736
Site 8	6560	4%	262	6940	N/A	N/A	7270	6%	436
Site 9	12200	4%	488	12900	N/A	N/A	12900	N/A	N/A
Site 10	5530	3%	166	5850	N/A	N/A	8530	N/A	N/A

(* AADTT: Average Annual Daily Truck Traffic)

Pretreatment/Initial COPACES Rating Acquisition

The pretreatment/initial COPACES rating data is acquired from GDOT's historical COPACES database, which provides project-level data. The rating is queried from the database using the year, RC link, and milepost. The survey of the crack condition was collected using the initial 2D/3D pavement images collected by the Georgia Tech Sensing Vehicle (GTSV) in November 2016. The survey of the crack condition was based on GDOT's COPACES manual.

Table 33. Summary of Initial COPACES rating and Description of Crack Condition

Site #	Initial COPACES Rating	Extend Length Percentage of Cracks by Category				
		Wheel Path Load Crack			B/T Crack	
		Level 1	Level 2	Level 3	Level 1	Level 2
1	66	45%	45%	5%	0%	80%
2	95	5%	0%	0%	30%	0%
3	69	60%	20%	0%	10%	50%
4	93	10%	0%	0%	50%	0%
5	98	10%	0%	0%	20%	0%
6	69	60%	10%	0%	40%	20%
8	79	10%	0%	0%	50%	0%
9	85	20%	0%	0%	60%	0%
10	82	20%	0%	0%	30%	0%

Field Coring Data Collection

Two field coring operations were conducted by the collaboration of GDOT and Georgia Tech's research team in April 2016 and August 2020. The first-time field coring (April 2016) collected the pavement thickness information for all test sites to support the crack sealing timing analysis, and the baseline pavement internal crack width/depth information to support the analysis of crack sealing impact on pavement internal crack width/depth growth. The second field coring (August 2020) took additional cores near the cores collected April 2016 to compare the pavement internal crack width/depth from 2016-2020 to observe the crack width/depth growth.

For the first field coring (April 2016), 3 cores were taken for each test site. Two cores were from the left and right wheel-paths, respectively. Another core was taken between the two wheel-paths. If cracks occurred in the candidate areas of coring, the core was taken from the crack section to capture the pavement internal crack width/depth for the later internal crack width/depth growth analysis. Table 34 summarizes the information obtained from the first collected cores (April 2016) including 1) the pavement thickness at coring locations by measuring the height of the cores, 2) the surface crack patterns by observing the top of cores, and 3) the pavement internal crack width/depth by observing the sides of the cores. Figure 116 shows an example of the 3 cores collected in April 2016, from Site 1. In addition, subbase material for all sites was found to be soil-cement, as determined by GDOT engineers observing the photos of the cores collected first.

For the second field coring (August 2020), additional cores (near the cores first collected) were taken to observe the pavement internal crack width/depth growth for supporting the analysis of crack sealing impact on the pavement internal crack width/depth growth. To optimize the coring efforts, the second field coring was selected and focused on Site 8 and Site 9 because they have similar roadway environmental factors (i.e. traffic volume, pavement thickness, design, etc.), and the cores were located in the sealed and non-sealed sections for these two sites, which allowed us to compare the pavement internal crack width/depth growth difference between sealed and non-sealed cracks. Both sites were in the same lane of Georgia's State Route 212 (Site 8 is at mile point 5.2, and Site 9 is at mile point 1.55). Cores at Site 8 were from the sealed section; cores from Site 9 were from the non-sealed sections. A total of 28 additional cores were taken during the second field coring.

Figure 117 and Figure 118 show the layout of the sealed sections, non-sealed sections, and the first coring locations for Site 8 and Site 9, respectively. A red box in the figures represents the sealed sections; a circle with an inside cross represents the coring locations. Therefore, from Figure 117 and Figure 118, we can see that the first 3 collected cores from Site 8 are located in a sealed section, while the first 3 collected cores in Site 9 locate are in a non-sealed section.

Table 34. Summary of First Field Coring Data (Cored in April 2016)

Site-Core ID	Thickness (inch)	Descriptions of Top View	Descriptions of Side View
1-1	8.0	thin crack	6 inch deep crack and de-bonding
1-2	9.5	wide crack (1/8 inch)	3 1/2 inch deep crack
1-3	9.5	very wide crack (3/8 inch)	4 1/2 inch deep crack
2-1	6.5	very wide (1/4 inch)	very wide with branch, penetrated
2-2	10.0	very wide (1/4 inch)	broken to 2 layers, 6-inch penetrated crack
2-3	10.5	thin crack	broken to 3 layers, no penetrate crack
2-4	10.5	no crack	no crack
3-1	3.0	medium crack	penetrated crack with branch
3-2	2.5	wide crack (1/4 inch)	penetrated crack
3-3	2.5	no crack	no crack
3-4	3.0	no crack	no crack
3-5	4.0	thin crack	no crack
3-6	2.5	wide crack (1/4 inch)	penetrated
3-7	2.5	no crack	no crack
4-1	6.0	thin crack	2 inch deep crack
4-2	6.5	thin crack with branch	2 inch deep crack
4-3	6.6	no crack	no crack
4-4	5.0	no crack	stratified, broken
5-1	9.0	thin crack	no crack
5-2	7.0	wide crack (1/8 inch)	penetrated crack with branch
5-3	7.0	no crack	no crack
5-4	9.5	wide crack with branch (1/4 inch)	4 inch deep crack with layer stratified
6-1	6.5	medium crack	no crack
6-2	6.5	medium crack	penetrated, wide crack
6-3	6.5	no crack	stratified
8-1	8.0	no crack	no crack
8-2	8.0	wide crack (3/8 inch)	penetrated crack with layers stratified
8-3	8.0	no crack	no crack
9-1	7.7	thin crack	no crack
9-2	8.5	thin crack	penetrate crack
9-3	8.0	no crack	no crack
10-1	9.0	thin crack	2 inch deep crack
10-2	9.0	thin crack	no crack
10-3	8.5	no crack	no crack

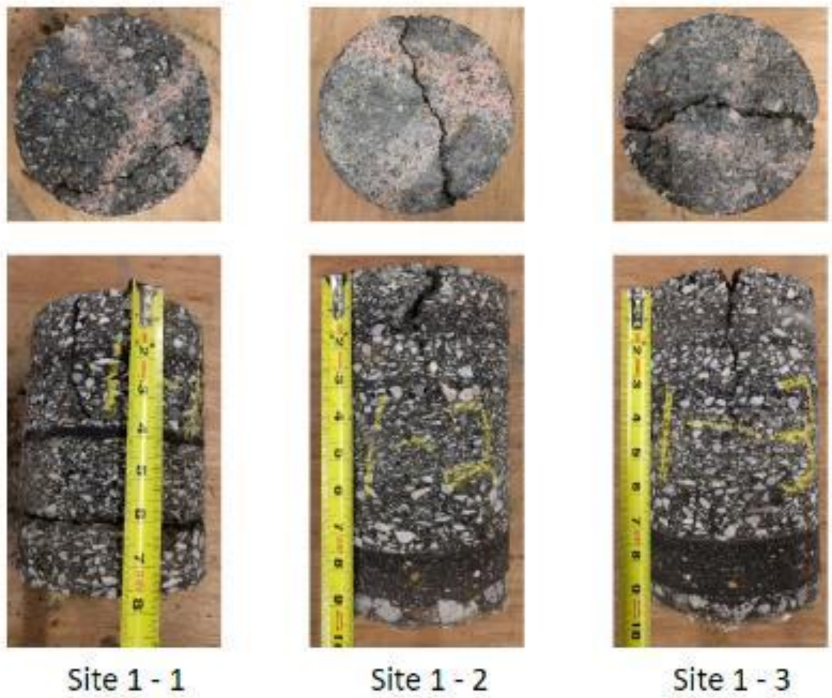


Figure 116. Photo. Three Cores from Site 1 (Collected in April 2016)

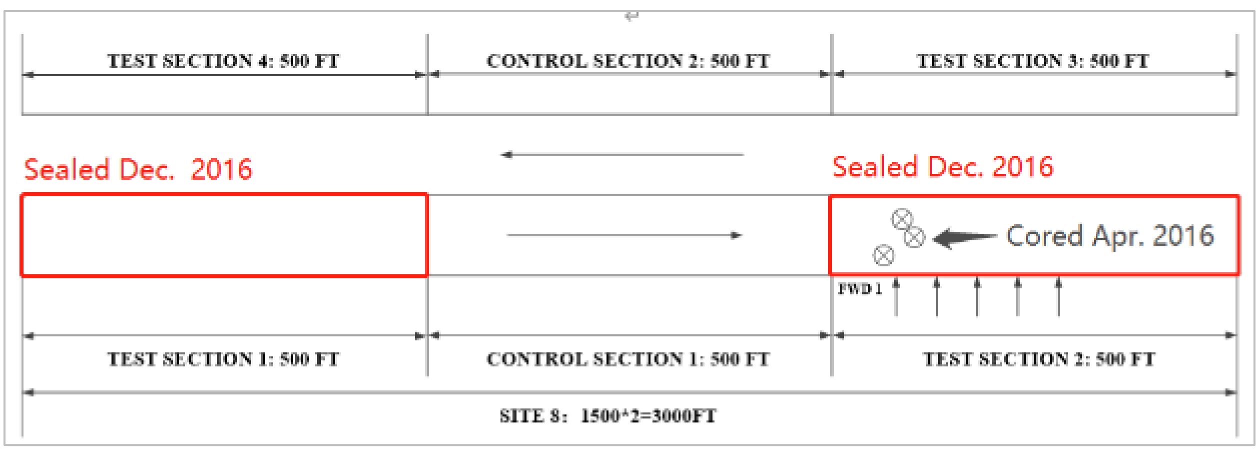


Figure 117. Graph. Sealed Sections and First Coring Locations at Site 8

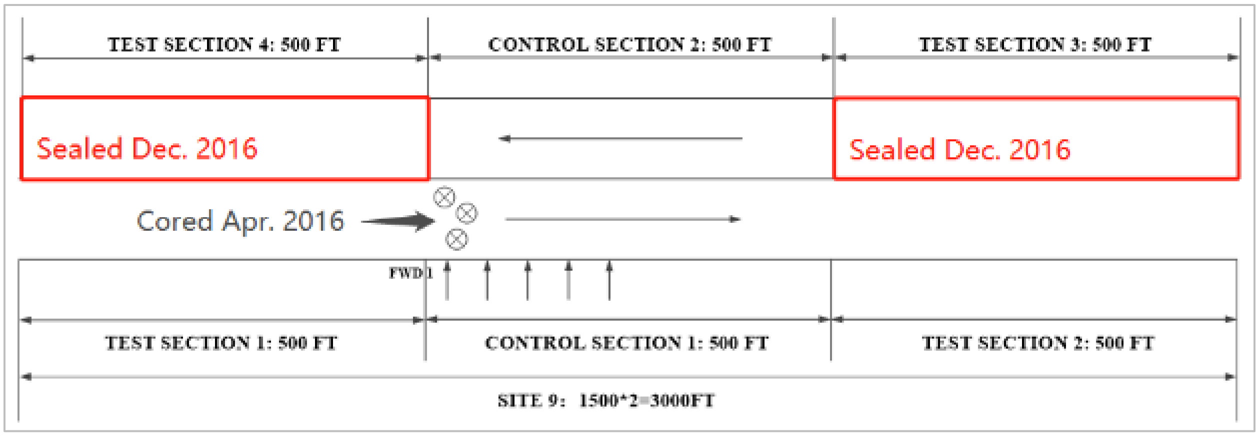


Figure 118. Graph. Sealed Sections and First Coring Locations at Site 9

2D, 3D Pavement Images Collection Using GTSV

Before the test sections were sealed, the initial 2D/3D pavement images data had been collected on November 3, 2016. Right after the test sections were crack sealed in late November 2016, the Georgia Tech (GaTech) Research Team started to collect pavement data quarterly on the selected sites. Instead of visual inspection, 2D/3D pavement images were collected using the GTSV. Table 35 shows the timestamps for data collection.

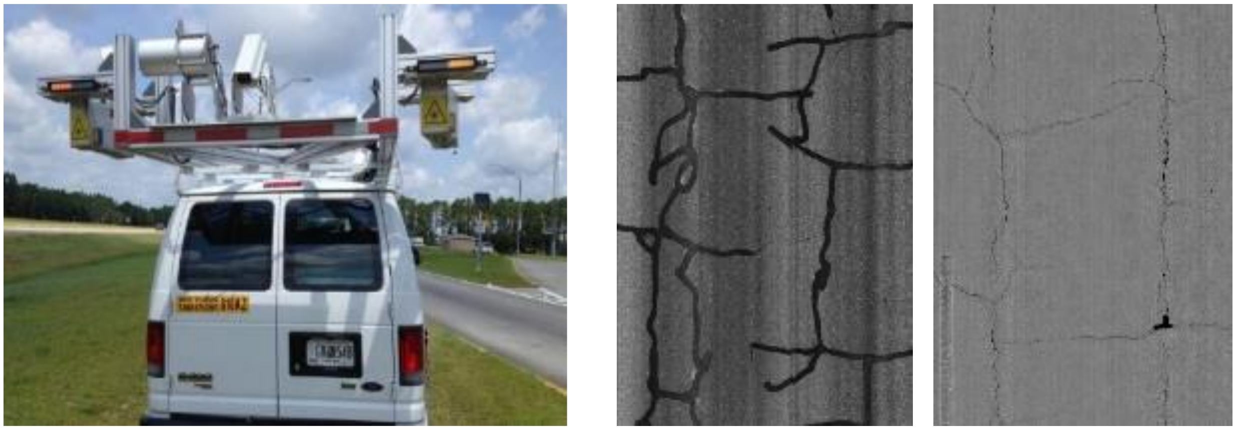
Table 35. Timestamps of 2D and3D Pavement Images Data Collection

Site 1-6 (South)		Site 8-10 (North)	
Date	Remark	Date	Remark
11/03/2016	Has not been sealed yet		
12/20/2016	First data after sealing	12/21/2016	First data after sealing
03/03/2017		03/10/2017	
06/28/2017		06/28/2017	
10/01/2017		09/26/2017	
12/12/2017		12/14/2017	
03/14/2018		03/14/2018	
10/17/2018			
01/30/2019		01/22/2019	
05/14/2019		05/02/2019	
09/03/2019		09/04/2019	
12/18/2019	Site 2,4,5 been sealed	12/19/2019	

The continuous, high-resolution 2D pavement surface intensity images and 3D pavement surface range images were collected simultaneously at a maximum speed of 100 km/h by two of line-laser scanning sensors mounted at the rear of the GTSV, shown in Figure 119(a). One frame of a 3D pavement surface image is composed of 1000 x 4160 points, with a 5 mm interval in the longitudinal direction and 1 mm interval in the transverse direction. The height value is stored in each cloud point with a resolution of 0.5 mm. The 2D pavement surface images have grey-level intensity images with a scale of 0-255 and have the same resolution as the 3D pavement surface images. 2D intensity pavement images are capable of capturing the sealed crack (shown in

Figure 119(b)), while 3D range pavement images are capable of capturing non-sealed cracks greater than 2 mm wide (shown in Figure 119(c)). So, both sealed and non-sealed cracks can be captured well using the GTSV. After the raw data is collected, the detailed level of crack information can be extracted using automatic or semi-automatic crack detection methods.

With the advancement of sensor technology, 3D laser technology has become a mainstream technology for collecting high-resolution, 3D pavement surface data. A survey shows eighteen states use a 3D automated data collection system, and seventeen states said they plan to use it within two years (Zimmerman, 2017); the states expect to use 3D technology and 3D pavement surface data to automatically and semi-automatically extract different pavement distresses, including cracking, rutting, faulting, raveling, etc. The 3D pavement surface data has been used for detecting and measuring cracking (Tsai & Li, 2012; Jiang et al., 2015) and its deterioration (Jiang et al., 2016), rutting (Tsai & Li et al., 2013: Tsai & Wang et al., 2015), concrete joint faulting (Tsai & Wu et al., 2012), project-level micro-milling pavement surface texture construction quality control (Tsai & Wang, 2014), automated raveling detection and classification (Tsai & Wang, 2015), automatic pothole detection (Tsai & Chatterjee, 2017), and a new area-based faulting measurement with an enhanced accuracy (Geary & Tsai et al., 2018).



(a) Georgia Tech Sensing Vehicle (GTSV) (b) 2D Intensity (c) 3D Range Images

Figure 119. Photo. GTSV and Example of 2D, 3D Pavement Surface Images

Summary of Data Collection and Acquisition

Table 36 summarizes site conditions, which include 1) the location of the sites, 2) the pavement structure, 3) the pavement thickness, 4) a traffic description, 5) the annual average daily traffic (AADT), 6) the truck percentage, 7) the annual average daily truck traffic (AADTT), 8) a pavement condition description, and 9) a pretreatment/initial COPACES rating. These roadway environmental factors and pavement pretreatment conditions are used to support our correlation analysis for the crack sealing effectiveness study in the data analysis section.

Table 36. Summary of Site Conditions

Site#	Road Name	RCLINK	Mile Point	Pvmt. Structure	Pvmt. Thick. (inch)	Traffic	AADT (2018)	Truck	AADTT	Pavement Condition	Initial COPACES Rating
1	GA 26	231002600	0.76	Composite	9.50	High	4430	26%	1148	Poor	66
2	GA 26	2351011200	14.96	Composite	10.00	High	4480	24%	959	Good	95
3	GA 230	2351023000	13.67	Thin AC layer	2.50	Low	1020	13%	128	Poor	69
4	US 341	2351002700	16.70	Full depth	6.50	Medium	3890	17%	662	Good	93
5	GA 230	2351023000	7.09	Thin AC layer	8.00	Low	1030	14%	133	Good	98
6	US 341	2351001100	18.12	Full depth	6.50	High	5260	14%	722	Poor	69
8	GA 212	2171021200	5.20	Thicker AC layer	8.00	High	7270	6%	349	Good	79
9	GA 212	2171021200	1.55	Thicker AC layer	8.00	High	12900	4%	488	Medium	85
10	GA 212	2471021200	2.89	Thicker AC layer	9.00	High	8530	3%	166	Poor	82

DATA PROCESSING AND ANALYSIS

This section presents 1) pavement image data processing, 2) crack growth analysis, and 3) crack sealing effectiveness analysis. First, the pavement image data processing generated sealed and non-sealed crack length information from 2D/3D pavement images. Second, the crack growth analysis plotted the crack length growth trends over 3 testing years to compare the crack growth deterioration rates between the crack-sealed test section and non-sealed control pavement sections. Finally, in the crack sealing effectiveness analysis, we summarized and compared the crack sealing effectiveness of nine testing sites and discussed the impacts of different pavement condition variables to the crack sealing effectiveness.

Pavement Image Data Processing

To study crack length growth and change due to crack sealing, it is important to have an objective comparison of crack length growth among multiple timestamps of 2D/3D pavement images to support crack growth and crack sealing effectiveness analysis. Therefore, a sophisticated data processing procedure is needed. The data processing procedure is composed of the following steps: 1) automatic lane marking detection and modification, 2) multiple timestamps data registration, 3) semi-automatic crack map digitization, and 4) crack length computation.

Step 1: Automatic Lane Marking Detection and Modification

There are two purposes for accurately detecting the lane marking positions on the test roads. The first purpose is to confine the crack length counting region into the left and right lane markings to avoid counting other cracks located out of the lane (e.g. construction joints). The second purpose is to establish the lane boundary reference in the transverse direction to conduct the multiple timestamps data registration for determining a consistent region/area for crack length computation at multiple time stamps. Therefore, the accuracy of lane marking detection is important to ensure the accuracy of crack length computation and the consistency of crack length growth comparison among multiple timestamps.

The lane marking detection is automatically processed by the commercial software Roadinspects developed by the company, Pavemetrics. Although this automatic lane marking detection software can output accurate lane marking positions in most conditions, the occurrence of some extreme conditions (e.g. lane marking worn away, road exit, etc.) leads to wrong detection results. Therefore, visually checking and manual modification are conducted based on the automatic detection results to ensure the accuracy of detected lane marking positions.

Figure 120 is an example of a wrong lane marking detection that needs manual modification. The red dash line circle points out the wrongly detected lane marking that will impact the crack length calculation because the construction joint is included. So, we will manually modify the wrong detection lane marking based on the correct lane marking position in the adjacent images to keep the crack length calculation correct.

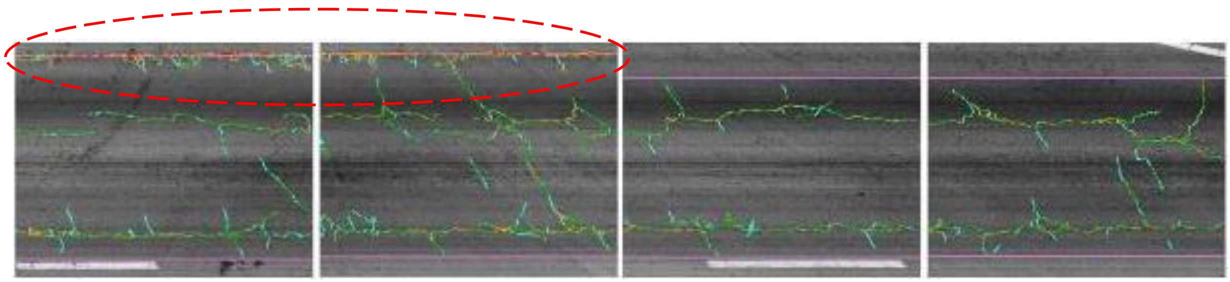


Figure 120. Graph. Example of Lane Marking Detection Problem

Step 2: Multiple Timestamp Pavement Surface Data Registration

The purpose of this step is to determine the region of interest (ROI) with a consistent boundary and area for the computation of crack lengths at different timestamps. The pavement surface image data collection systems cover only one lane when the vehicle is driving in the middle of that lane. In actual data collection, different driving behaviors cause different vehicle wandering patterns in different surveys, which further influences the coverage area in different surveys. Therefore, the ROI in the transverse direction cannot be guaranteed to be the same in different timestamps’ data at a certain location.

To solve this problem, this project adapted a multiple timestamp data registration method to compute the overlapping area (e.g. ROI) between different surveys at a certain location for a fair and consistent comparison. Figure 121 is an example of the concept or operation of overlapping the different timestamp data to find the ROI. The yellow box in the figure is the overlapped ROI between Survey 1 and Survey 2. The cracks will be digitized in the ROI region for the subsequent crack map generation to ensure an objective comparison of crack length computation among multiple timestamp data for a meaningful crack growth and deterioration analysis. The crack growth analysis in pavement management is questionable if a consistent ROI cannot be ensured (warranted) among the pavement data that has been collected at different timestamps.

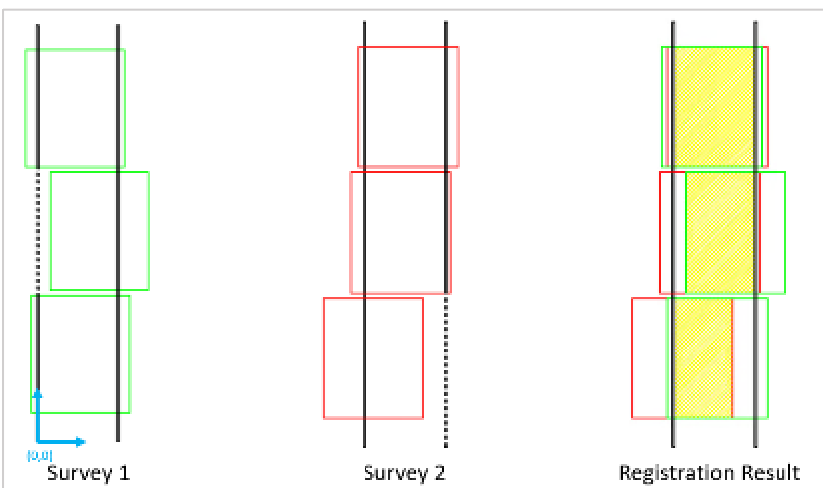


Figure 121. Graph. Example of Multiple Timestamp Data Registration

Step 3: Semi-Automatic Digitization of Crack Map

The purpose of this step is to semi-automatically digitalize the cracks for generating the crack map and for analyzing crack characteristics (e.g. length, width, orientation, position, etc.) in support of this crack sealing effectiveness study. The GaTech research team used a semi-automatic crack digitization tool based on the Minimal Paths algorithm that was developed by the GaTech research team's previous study to reduce the effort and potential errors introduced by human digitization. This will also reduce the variability of 2D/3D image quality in images collected at different timestamps. Compared to the fully automatic crack detection algorithms, the semi-automatic method has higher digitization accuracy and lower sensitivity to the slight change of 3D pavement image quality. Figure 122 is the user interface of the developed semi-automatic crack digitization tool. The blue line with nodes is used to represent the digitized crack. This tool can separately record x, y nodes of the sealed crack and non-sealed crack into an XML file, which can be further read and processed to calculate the total crack length.

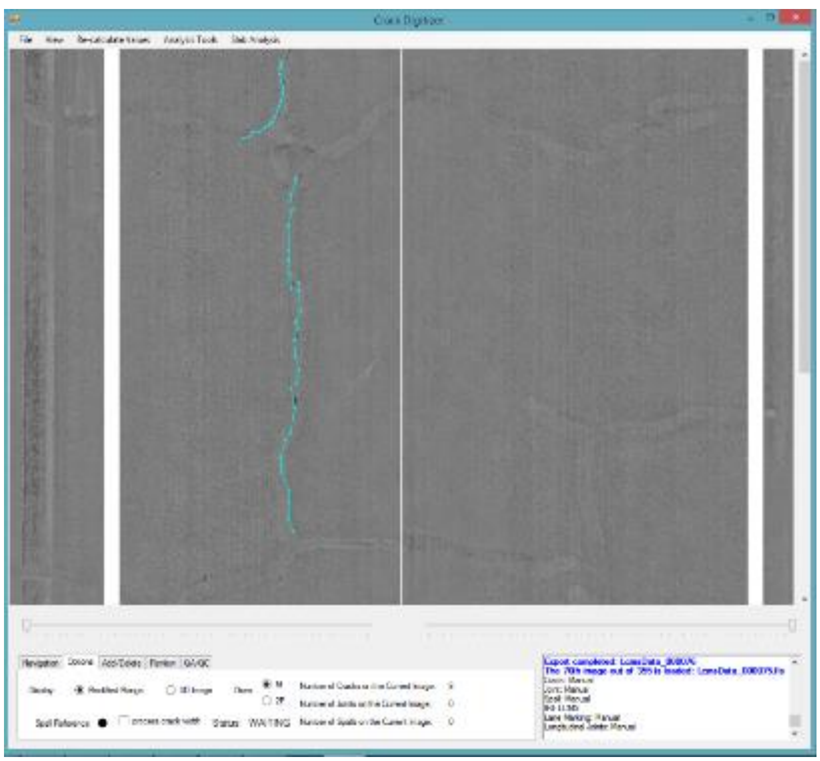


Figure 122. Graph. User Interface of Semi-Automatic Crack Digitization Tool

Step 4: Crack Length Computation

The purpose of this step is to compute the crack length, one of the crack characteristics, to support this crack sealing effectiveness study. The crack map, generated from the previous step, is stored in an XML file using the x, y coordinates of the nodes. By connecting the nodes and accumulating the distances between the two connected nodes, the length of each crack can be calculated. Then, the total crack length in the 500-ft section can be accumulated. The tool was developed using MATLAB by the GaTech research team to batch process the XML files and summarize the information as an easily readable file. Figure 123 is an example of the data format of the outputs for one 500-ft section at one site. Besides crack length, used as a crack indicator in this study, other crack characteristics (e.g. crack width, intersections, direction, location/position, etc.) can also be used as potential performance indicators in the future; they are, also, available in the output table. Crack length is the crack performance indicator used in this study to study crack deterioration and crack growth.

TimeStamp	Manual_Total_Intersection	Manual_Total_Length	Manual_Total_Longitudinal_Crack	Manual_Total_Transverse_Crack	Manual_WheelPath_Crack	Indirect_Non_WheelPath_Crack	Manual_Direct_Non_WheelPath_Crack	Avg_Crack_Width	Zone1_Crack	Zone2_Crack	Zone3_Crack	Zone4_Crack	Zone5_Crack
20161220	269	557410.605	323634.668	233775.938	435614.64	121795.964	117925.4345	3.92496	34346	223567	66065	212047	17514
20170303	572	604323.489	360006.119	244317.37	485225.07	119098.418	116740.4612	4.5208	34585	248982	63972	236243	18183
20170628	854	648327.976	384933.025	263394.951	512878.14	135449.833	125856.2446	4.57411	37828	258387	67289	254492	20740
20171001	682	718371.859	421213.014	297158.845	562881.24	155490.618	145499.8118	4.65928	42128	275718	79119	287163	24253
20171212	721	707782.927	418288.532	289494.395	560827.18	146955.743	143918.5618	4.62223	47045	278531	75497	282296	21377
20181017	966	743911.377	439945.842	303965.535	579766.76	164144.62	150661.2424	5.03042	38781	282939	85771	296827	26109
20190130	894	806308.706	462352.119	343956.588	629608.7	176700.004	170283.8296	5.05418	45336	306622	94172	322987	30776
20190514	1109	823194.196	453519.111	369675.085	649627.93	173566.265	158358.7327	5.09751	40512	333893	95519	315735	22327
20190903	1094	919640.541	506806.871	412833.67	716306.18	203334.366	185748.5811	5.2162	51002	339159	103340	377147	31406

Figure 123. Graph. Example of Data Format of Outputs

Crack Growth Plotting and Analysis

This subsection plots the crack length growth trends over 3 testing years (2017 – 2019) with two methods to compare the crack growth deterioration rates between the crack-sealed test section and the non-sealed control pavement sections. The first method uses the absolute crack length (a unit of meter) to plot the crack growth trends. However, due to the different pretreatment/initial crack length in different sites, the comparison of the crack deterioration rate between different sites is difficult. The second method uses a relative percentage of crack growth (a unit of percentage) to have the same comparison among all test sites by comparing the change between the crack length at two timestamps, which is also calculated. The following tables and figures summarize the actual crack length growth and the percentage crack length growth in the control and test sections. The actual crack length growth is calculated using the following algorithm:

$$\triangle CL = CL_{t2} - CL_{t1}$$

Where $\triangle CL =$ the actual crack length growth; $CL_{t2} =$ the crack length at the end of the test period; $CL_{t1} =$ the crack length at the beginning of the test period.

The percentage crack length growth is calculated using the following algorithm:

$$\triangle CL(\%) = \frac{CL_{t2} - CL_{t1}}{CL_{t1}}$$

Where $\triangle CL(\%) =$ the percentage crack length growth; $CL_{t2} =$ the crack length at the end of the test period; $CL_{t1} =$ the crack length at the beginning of the test period.

Then, two line charts for each site are plotted to compare the actual crack length growth and the percentage crack length growth, respectively. The x-axis is the data collection time. The y-axis for the crack length growth is the total crack length by meter. The y-axis for percentage crack length growth is the percentage crack length growth compared to the initial crack length. Different line colors represent different sections at one site. For the test section, if there is only one test section in each site, the red line represents this test section; if there is more than one test section, the orange line will represent the second test section. For control sections, two blue lines of different shades are used to represent these two control sections. The crack growth comparisons for all the testing sites are summarized in the tables and figures below. After this, a summarized comparison will be presented in the crack effectiveness analysis subsection.

To compare the crack sealing effectiveness between different sites, the “effectiveness” needs to be calculated. It is calculated using “percentage crack length growth of control (non-sealed) section” minus the “percentage growth of test (crack sealed) section.” The outcomes are presented in the next section.

Table 37. Crack Length Growth (Site 1)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section		
		Site 1 - C1 (meter)	Site 1 - T1* (meter)	Site 1 - T2* (meter)
2016-11-03 (Before Sealing)	0	566.1	672.9	633.0
12/20/2016	1	573.5	672.9	633.0
3/3/2017	4	607.8	673.3	634.7
6/28/2017	7	631.6	673.7	635.1
10/1/2017	11	667.6	674.4	636.5
12/12/2017	13	671.9	675.5	637.1
3/14/2018	16	690.4	678.9	640.0
10/17/2018	23	722.8	681.6	642.1
1/30/2019	26	759.7	686.2	646.4
5/14/2019	30	789.1	690.2	649.3
9/3/2019	34	809.1	693.5	656.0

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

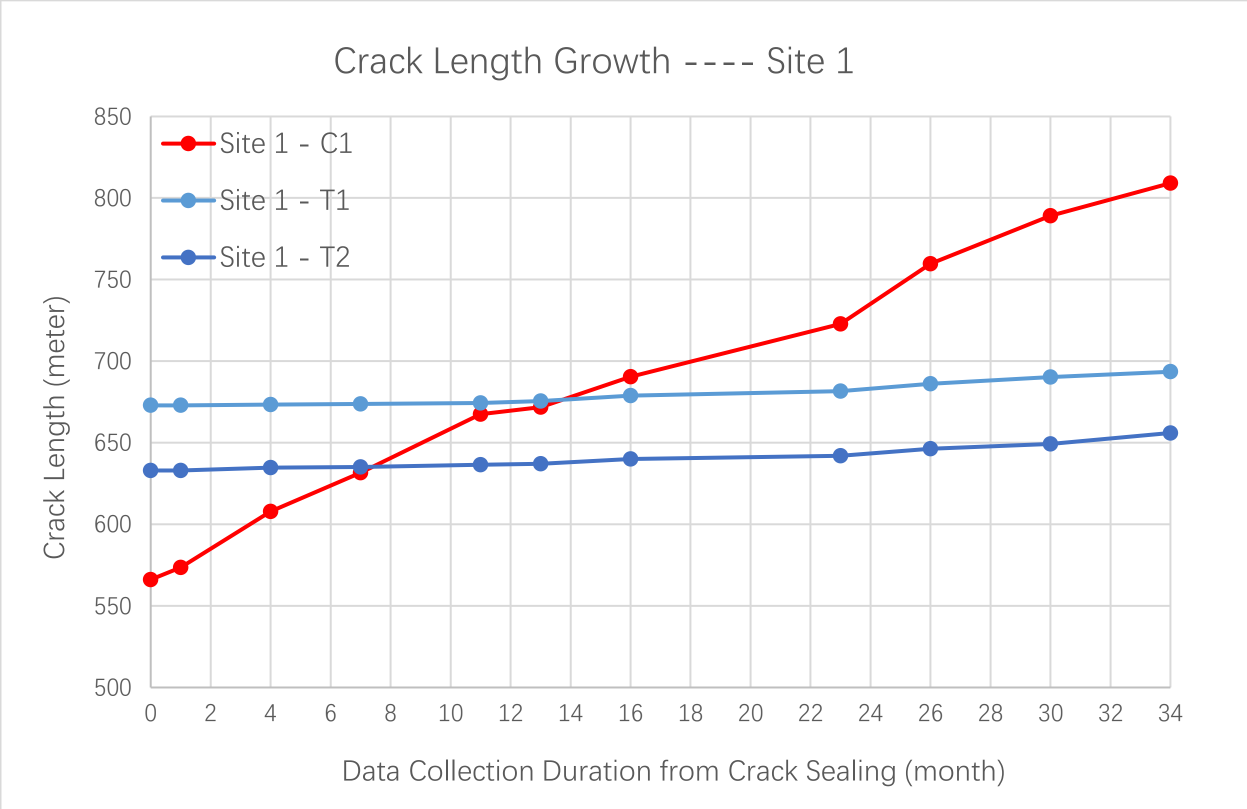


Figure 124. Graph. Crack Length Growth (Site 1)

Table 38. Percentage of Crack Length Growth (Site 1)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section		
		Site 1 - C1	Site 1 - T1*	Site 1 - T2*
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%
12/20/2016	1	1.31%	0.00%	0.00%
3/3/2017	4	7.36%	0.07%	0.27%
6/28/2017	7	11.56%	0.12%	0.33%
10/1/2017	11	17.93%	0.23%	0.56%
12/12/2017	13	18.68%	0.38%	0.65%
3/14/2018	16	21.95%	0.90%	1.11%
10/17/2018	23	27.68%	1.29%	1.43%
1/30/2019	26	34.21%	1.98%	2.11%
5/14/2019	30	39.39%	2.58%	2.57%
9/3/2019	34	42.92%	3.07%	3.64%

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

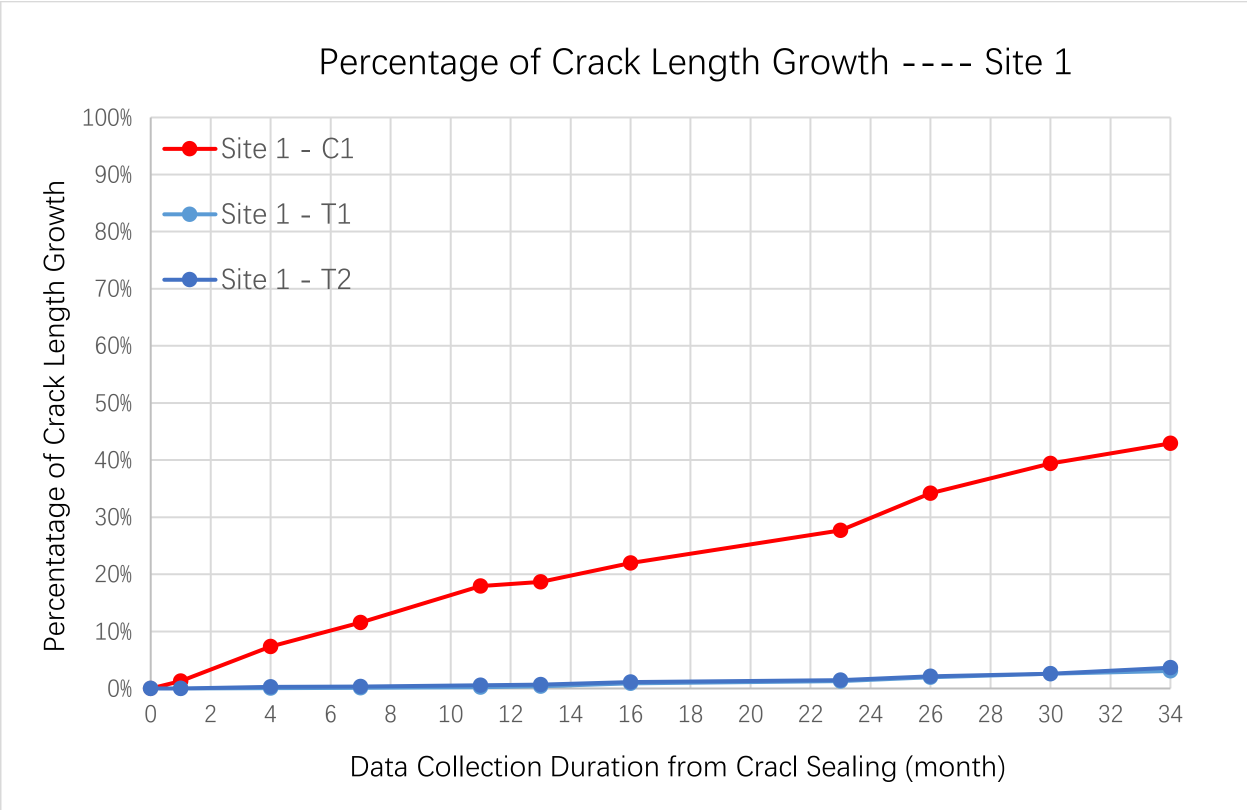


Figure 125. Graph. Percentage of Crack Length Growth (Site 1)

Table 39. Crack Length Growth (Site 2)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section			
		Site 2 - C2 (meter)	Site 2 - C4 (meter)	Site 2 - T1* (meter)	Site 2 - T2** (meter)
2016-11-03 (Before Sealing)	0	108.8	50.3	60.0	39.9
12/20/2016	1	111.7	52.4	60.0	42.7
3/3/2017	4	137.2	64.7	60.8	46.7
6/28/2017	7	138.0	80.3	61.8	49.6
10/1/2017	11	139.1	88.1	62.4	53.0
12/12/2017	13	140.3	95.2	62.7	55.9
3/14/2018	16	157.9	103.9	63.0	58.9
10/17/2018	23	173.6	115.9	63.6	60.8
1/30/2019	26	182.4	125.4	64.9	60.8
5/14/2019	30	188.3	132.5	65.0	60.9
9/3/2019	34	190.4	145.0	66.0	61.0

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

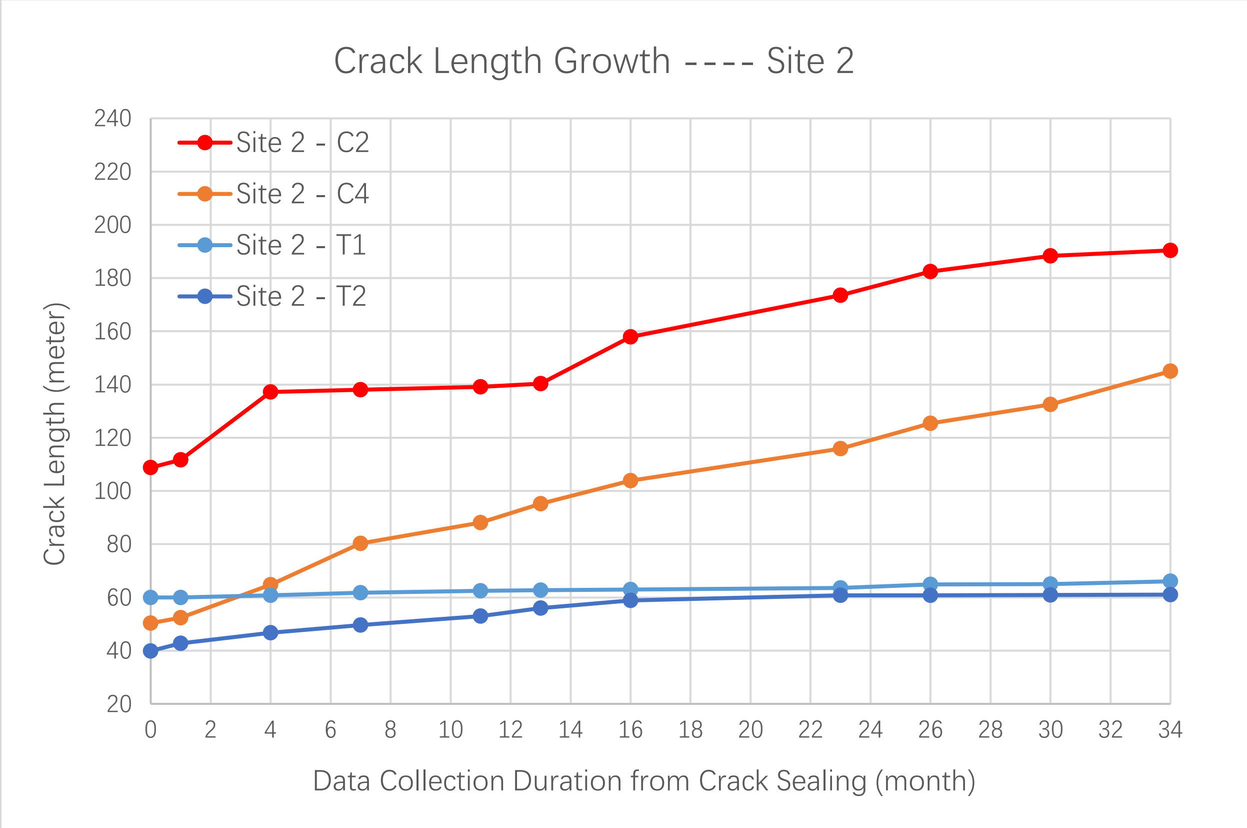


Figure 126. Graph. Crack Length Growth (Site 2)

Table 40. Percentage Crack Length Growth (Site 2)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section			
		Site 2 - C2	Site 2 - C4	Site 2 - T1*	Site 2 - T2**
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%	0.00%
12/20/2016	1	2.64%	4.14%	0.00%	7.04%
3/3/2017	4	26.12%	28.55%	1.37%	17.10%
6/28/2017	7	26.89%	59.56%	2.94%	24.37%
10/1/2017	11	27.87%	75.14%	4.06%	32.84%
12/12/2017	13	28.96%	89.10%	4.54%	40.23%
3/14/2018	16	45.15%	106.42%	5.04%	47.73%
10/17/2018	23	59.55%	130.24%	6.04%	52.46%
1/30/2019	26	67.66%	149.11%	8.20%	52.46%
5/14/2019	30	73.10%	163.24%	8.37%	52.69%
9/3/2019	34	75.02%	188.06%	10.09%	53.06%

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

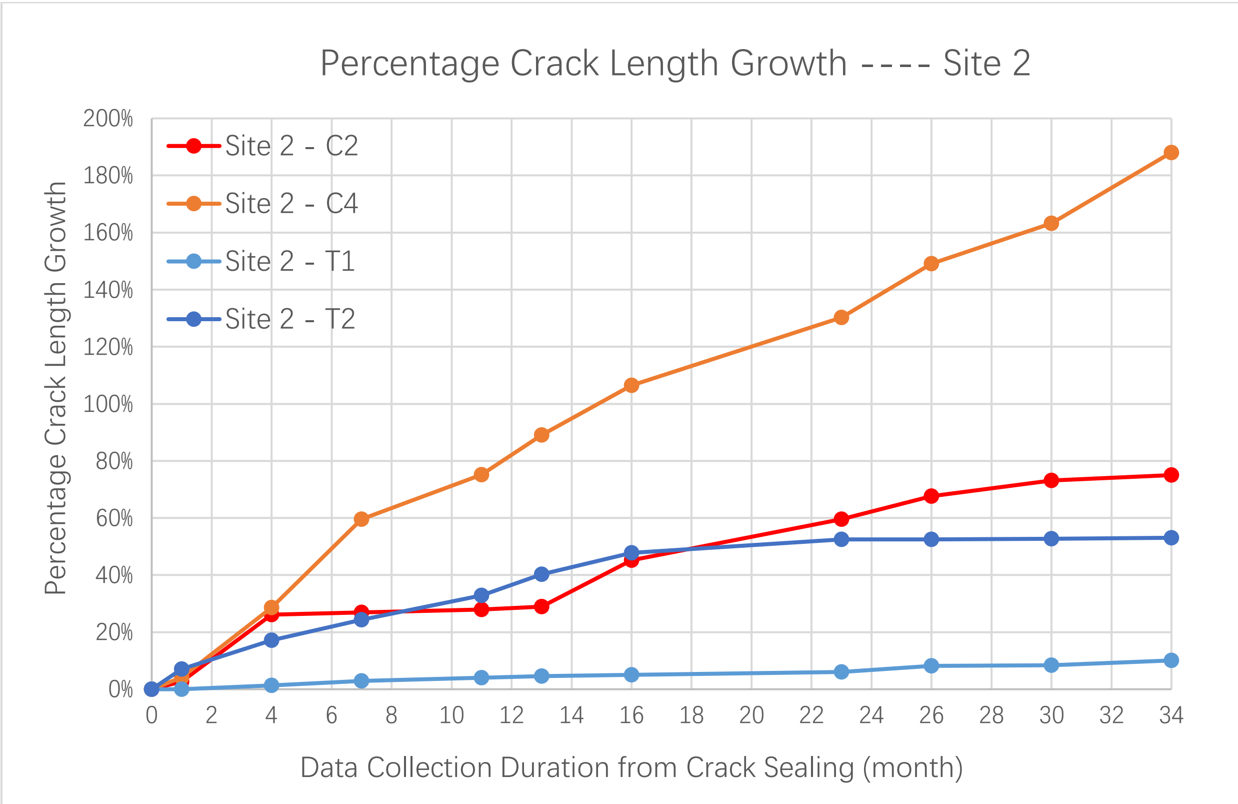


Figure 127. Graph. Percentage Crack Length Growth (Site 2)

Table 41. Crack Length Growth (Site 3)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section		
		Site 3 - C2 (meter)	Site 3 - T3* (meter)	Site 3 - T4* (meter)
2016-11-03 (Before Sealing)	0	237.0	493.5	313.2
12/20/2016	1	245.3	493.5	313.2
3/3/2017	4	261.0	494.3	315.5
6/28/2017	7	280.2	496.7	317.4
10/1/2017	11	290.7	500.0	317.7
12/12/2017	13	304.1	502.2	319.6
3/14/2018	16	317.5	502.4	321.2
10/17/2018	23	330.5	505.1	323.8
1/30/2019	26	346.0	509.5	326.1
5/14/2019	30	352.4	511.8	328.5
9/3/2019	34	361.4	513.5	330.4

(Note: *Crack sealing has been performed at the end of November 2016 for both T3 and T4)

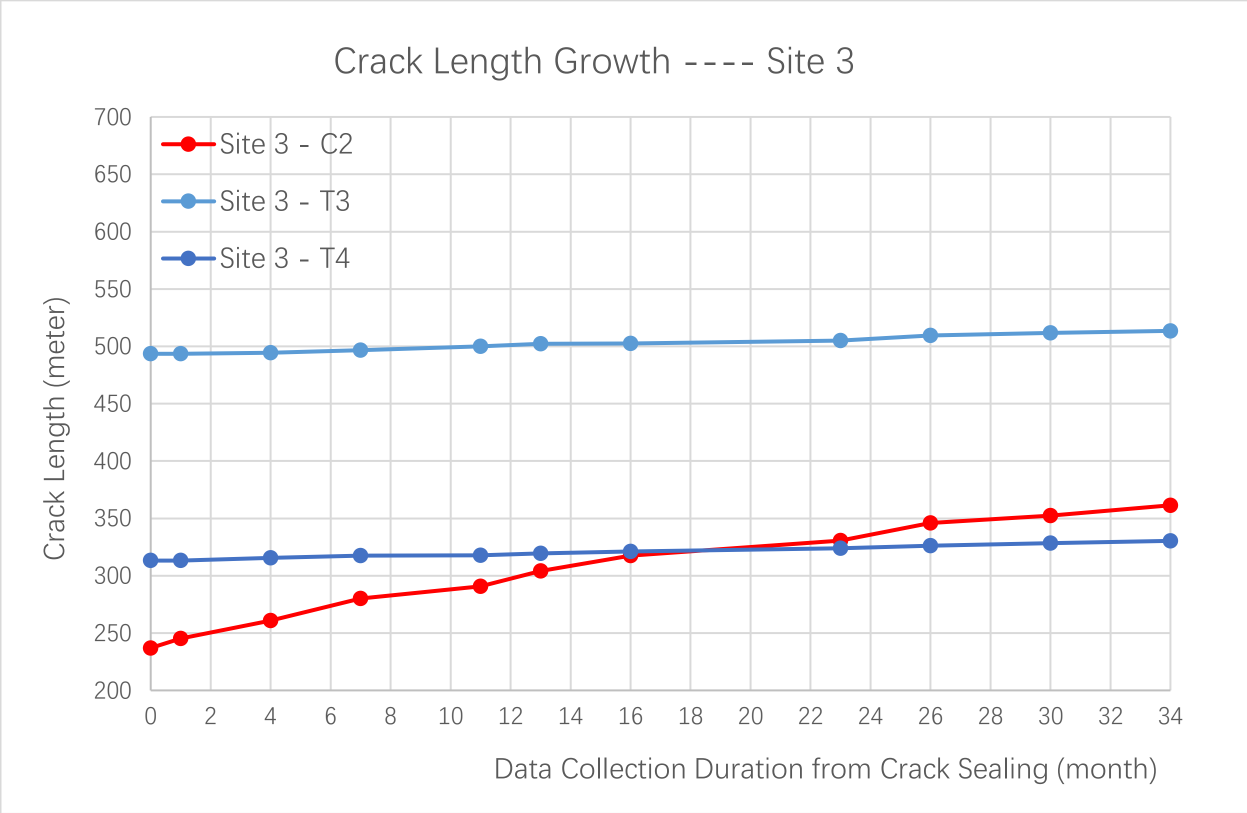


Figure 128. Graph. Crack Length Growth (Site 3)

Table 42. Percentage Crack Length Growth (Site 3)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section		
		Site 3 - C2	Site 3 - T3*	Site 3 - T4*
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%
12/20/2016	1	3.48%	0.00%	0.00%
3/3/2017	4	10.10%	0.17%	0.73%
6/28/2017	7	18.21%	0.66%	1.34%
10/1/2017	11	22.64%	1.32%	1.42%
12/12/2017	13	28.30%	1.77%	2.03%
3/14/2018	16	33.95%	1.81%	2.56%
10/17/2018	23	39.43%	2.36%	3.36%
1/30/2019	26	45.97%	3.25%	4.11%
5/14/2019	30	48.66%	3.71%	4.87%
9/3/2019	34	52.46%	4.06%	5.48%

(Note: *Crack sealing has been performed at the end of November 2016 for both T3 and T4)

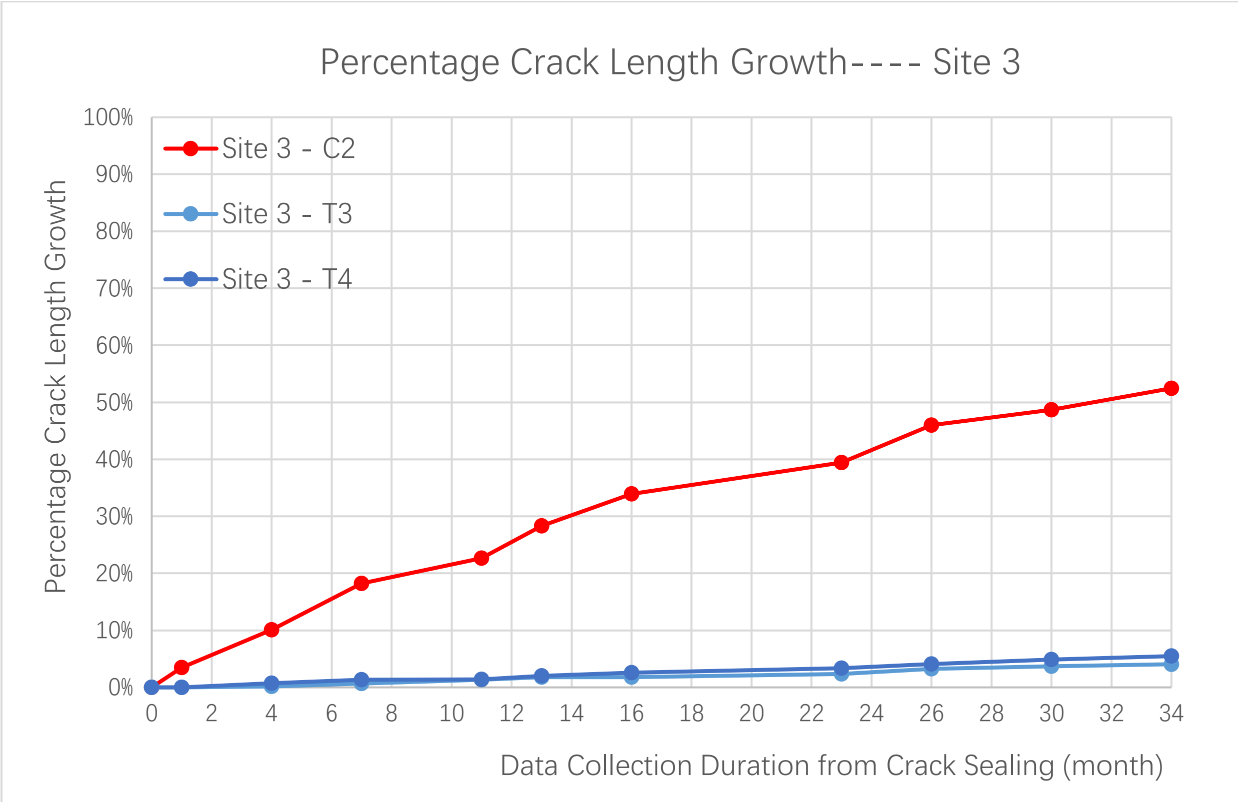


Figure 129. Graph. Percentage Crack Length Growth (Site 3)

Table 43. Crack Length Growth (Site 4)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section			
		Site 4 - C1 (meter)	Site 4 - C2 (meter)	Site 4 - T1* (meter)	Site 4 - T2** (meter)
2016-11-03 (Before Sealing)	0	154.5	49.1	320.7	52.1
12/20/2016	1	155.4	54.9	320.7	56.9
3/3/2017	4	188.8	63.9	320.7	62.0
6/28/2017	7	198.2	66.2	323.6	69.2
10/1/2017	11	201.4	70.6	324.1	74.1
12/12/2017	13	211.6	75.3	324.4	81.2
3/14/2018	16	221.6	78.4	325.8	86.8
10/17/2018	23	254.0	96.4	331.9	87.0
1/30/2019	26	295.3	114.2	341.8	88.9
5/14/2019	30	314.4	130.4	342.4	90.3
9/3/2019	34	325.3	145.7	343.8	91.7

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

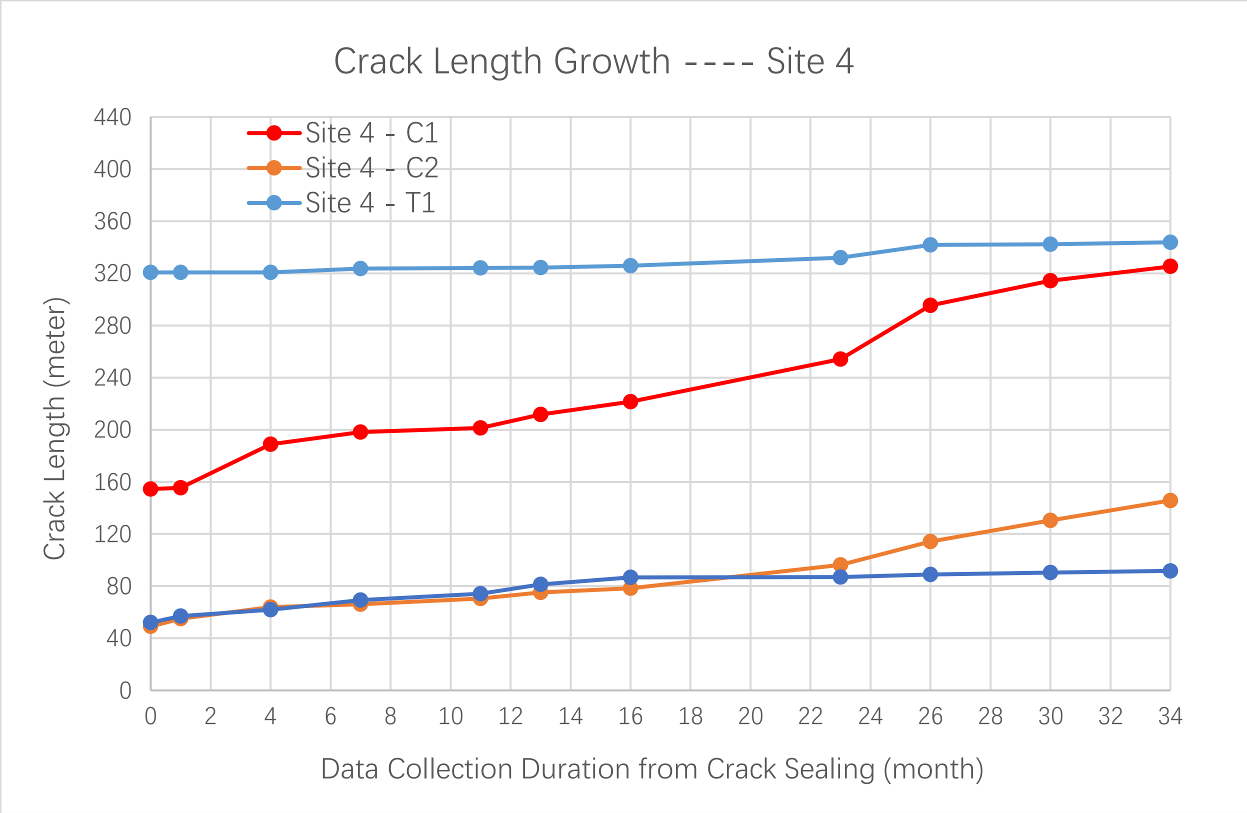


Figure 130. Graph. Crack Length Growth (Site 4)

Table 44. Percentage Crack Length Growth (Site 4)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section			
		Site 4 - C1	Site 4 - C2	Site 4 - T1*	Site 4 - T2**
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%	0.00%
12/20/2016	1	0.61%	11.96%	0.00%	9.28%
3/3/2017	4	22.20%	30.17%	0.00%	19.05%
6/28/2017	7	28.33%	34.87%	0.89%	32.92%
10/1/2017	11	30.37%	43.93%	1.04%	42.38%
12/12/2017	13	37.00%	53.47%	1.16%	56.04%
3/14/2018	16	43.47%	59.87%	1.59%	66.66%
10/17/2018	23	64.46%	96.53%	3.50%	67.11%
1/30/2019	26	91.19%	132.68%	6.59%	70.85%
5/14/2019	30	103.52%	165.76%	6.76%	73.41%
9/3/2019	34	110.57%	197.07%	7.21%	76.21%

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

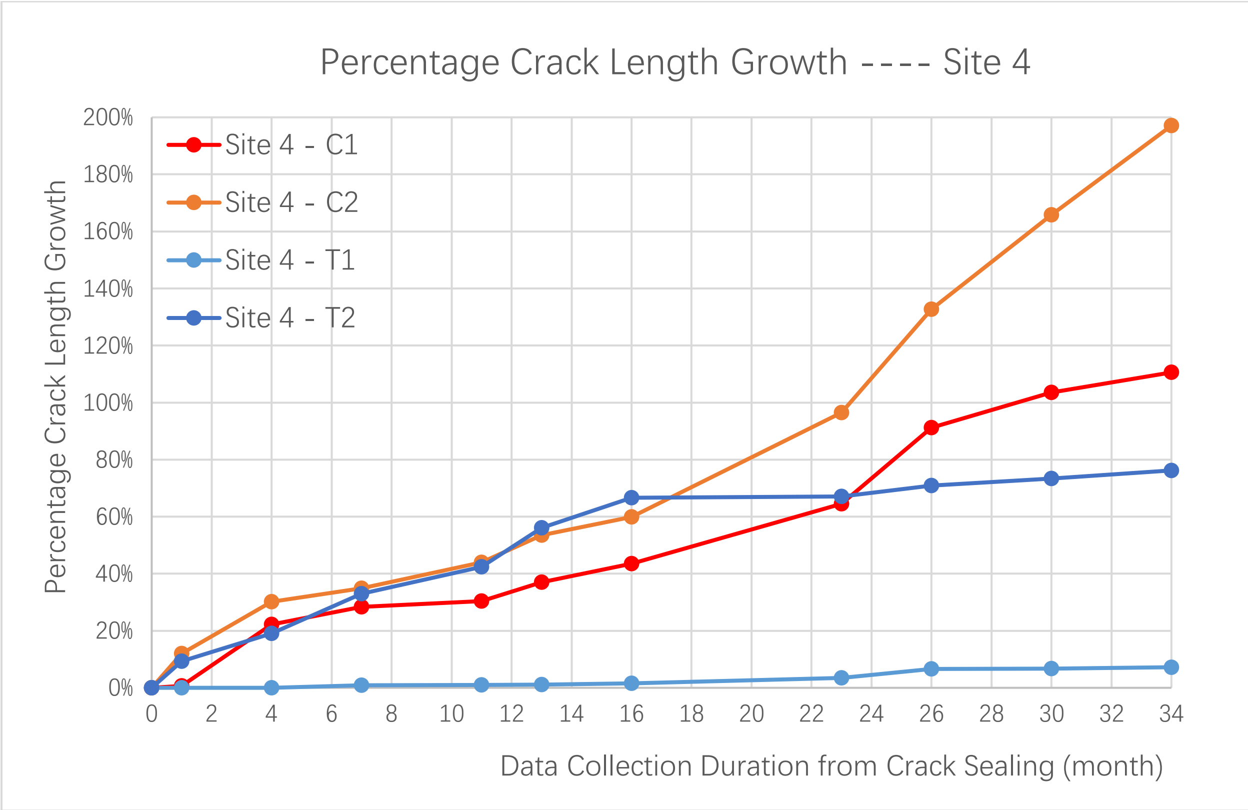


Figure 131. Graph. Percentage Crack Length Growth (Site 4)

Table 45. Crack Length Growth (Site 5)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section			
		Site 5 - C3 (meter)	Site 5 - C4 (meter)	Site 5 - T1* (meter)	Site 5 - T2** (meter)
2016-11-03 (Before Sealing)	0	50.6	81.6	81.1	32.2
12/20/2016	1	51.0	82.1	81.1	37.2
3/3/2017	4	53.9	84.9	81.1	43.5
6/28/2017	7	62.8	95.0	81.1	49.6
10/1/2017	11	64.6	105.2	81.1	57.3
12/12/2017	13	66.1	108.2	81.4	68.4
3/14/2018	16	76.3	122.1	83.1	78.9
10/17/2018	23	87.2	131.1	89.1	79.3
1/30/2019	26	93.2	141.9	93.1	80.5
5/14/2019	30	98.3	151.7	95.9	80.8
9/3/2019	34	103.8	163.7	97.8	81.0

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

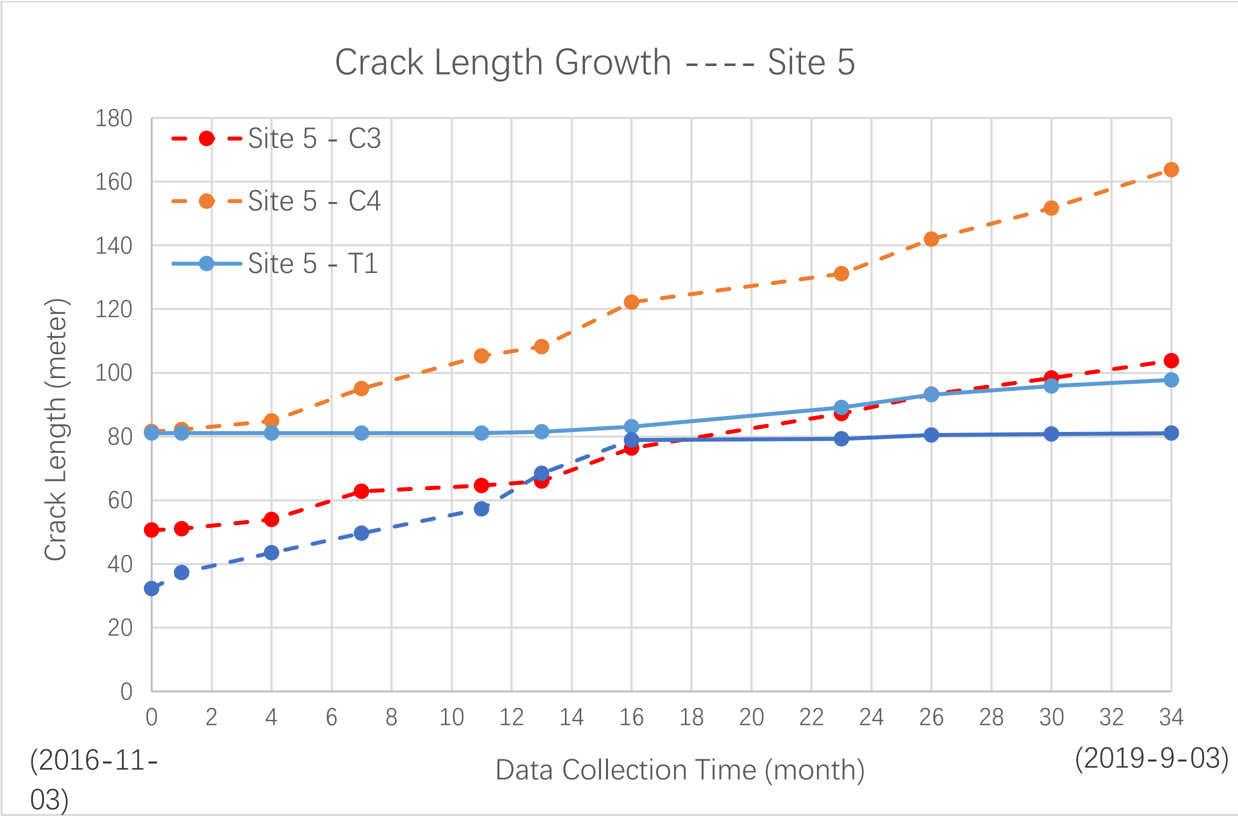


Figure 132. Graph. Crack Length Growth (Site 5)

Table 46. Percentage Crack Length Growth (Site 5)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section			
		Site 5 - C3	Site 5 - C4	Site 5 - T1*	Site 5 - T2**
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%	0.00%
12/20/2016	1	0.89%	0.72%	0.00%	15.57%
3/3/2017	4	6.60%	4.14%	0.00%	34.95%
6/28/2017	7	24.14%	16.48%	0.00%	54.06%
10/1/2017	11	27.60%	29.04%	0.00%	77.72%
12/12/2017	13	30.55%	32.66%	0.46%	112.23%
3/14/2018	16	50.85%	49.74%	2.51%	144.88%
10/17/2018	23	72.43%	60.75%	9.85%	146.10%
1/30/2019	26	84.17%	73.99%	14.81%	149.73%
5/14/2019	30	94.30%	85.97%	18.27%	150.76%
9/3/2019	34	105.18%	100.75%	20.59%	151.51%

(Note: *Crack sealing has been performed at the end of November 2016 for T1; ** at February 11th 2018 for T2)

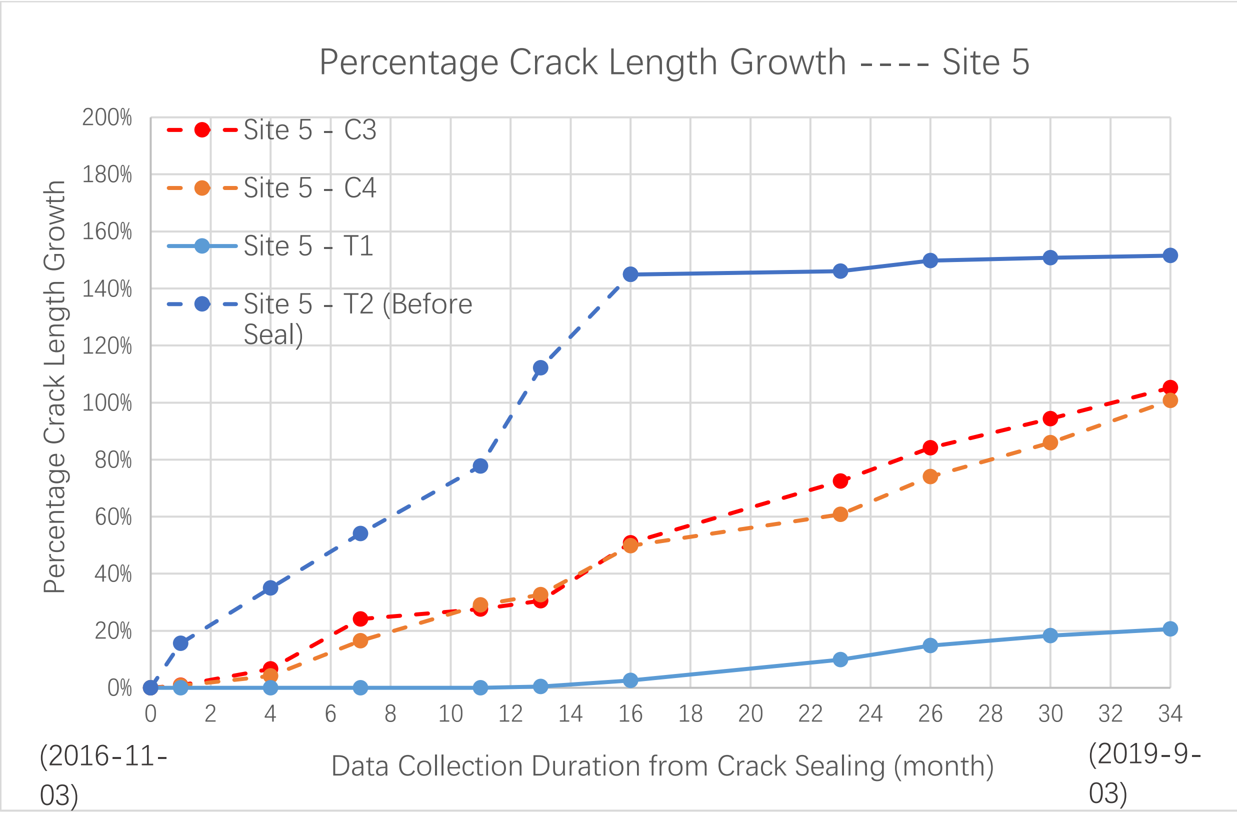


Figure 133. Graph. Percentage Crack Length Growth (Site 5)

Table 47. Crack Length Growth (Site 6)

Data Collection Time	Time After Sealing	Total Crack Length in 500-ft Section			
		Site 6 - C1	Site 6 - C2	Site 6 - T2*	Site 6 - T3*
	(month)	(meter)	(meter)	(meter)	(meter)
2016-11-03 (Before Sealing)	0	161.8	145.0	269.5	86.0
12/20/2016	1	181.8	151.3	269.5	86.0
3/3/2017	4	204.0	163.5	269.5	86.0
6/28/2017	7	207.6	169.9	272.4	88.1
10/1/2017	11	208.7	172.8	275.8	90.5
12/12/2017	13	210.3	174.4	280.7	93.4
3/14/2018	16	220.4	177.8	281.0	93.2
10/17/2018	23	233.6	181.8	282.8	93.5
1/30/2019	26	251.0	187.2	285.3	94.2
5/14/2019	30	264.9	193.8	290.7	94.9
9/3/2019	34	272.6	199.3	295.2	95.3

(Note: *Crack sealing has been performed at the end of November 2016 for both T2 and T3)

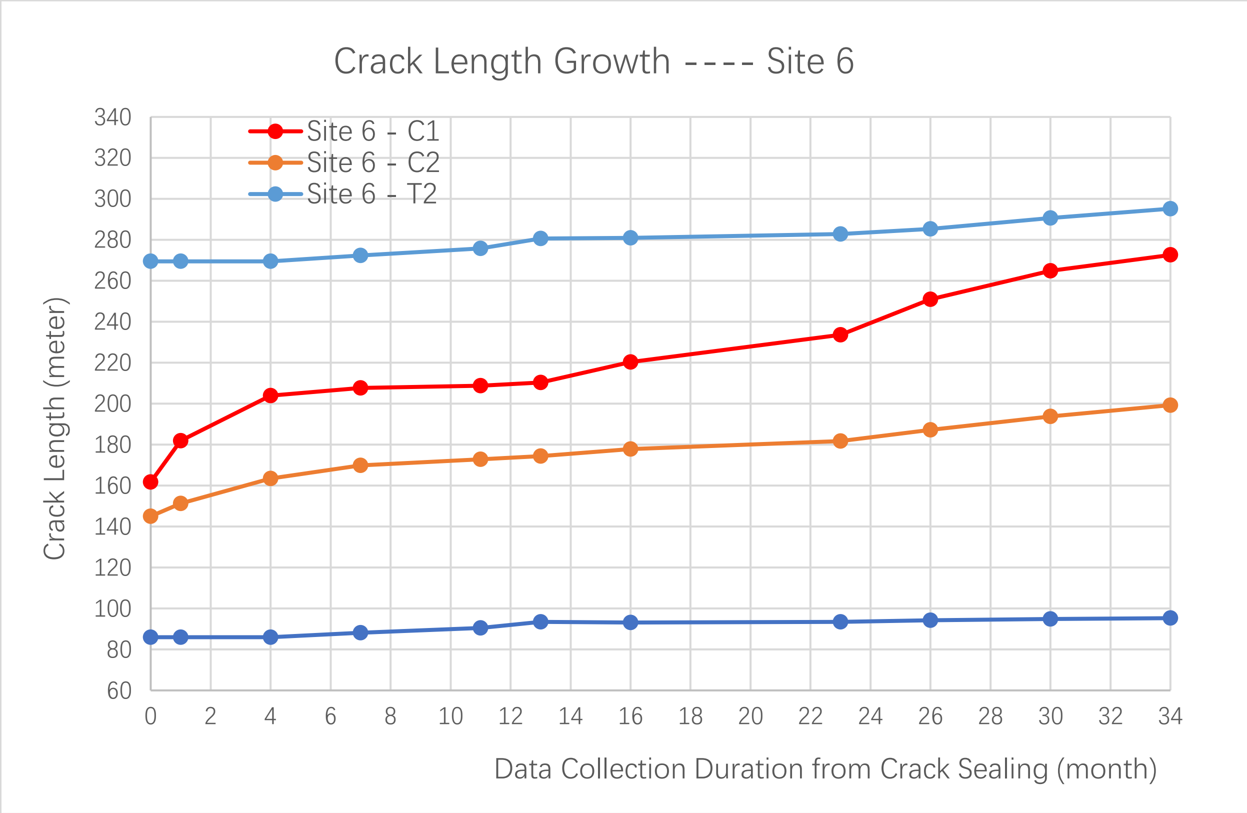


Figure 134. Graph. Crack Length Growth (Site 6)

Table 48. Percentage Crack Length Growth (Site 6)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section			
		Site 6 - C1	Site 6 - C2	Site 6 - T2	Site 6 - T3
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%	0.00%
12/20/2016	1	12.42%	4.37%	0.00%	0.00%
3/3/2017	4	26.10%	12.74%	0.00%	0.00%
6/28/2017	7	28.34%	17.19%	1.09%	2.45%
10/1/2017	11	29.02%	19.19%	2.34%	5.23%
12/12/2017	13	30.03%	20.26%	4.15%	8.64%
3/14/2018	16	36.24%	22.61%	4.26%	8.39%
10/17/2018	23	44.43%	25.35%	4.93%	8.73%
1/30/2019	26	55.16%	29.09%	5.87%	9.62%
5/14/2019	30	63.75%	33.64%	7.88%	10.38%
9/3/2019	34	68.52%	37.43%	9.54%	10.82%

(Note: *Crack sealing has been performed at the end of November 2016 for both T2 and T3)

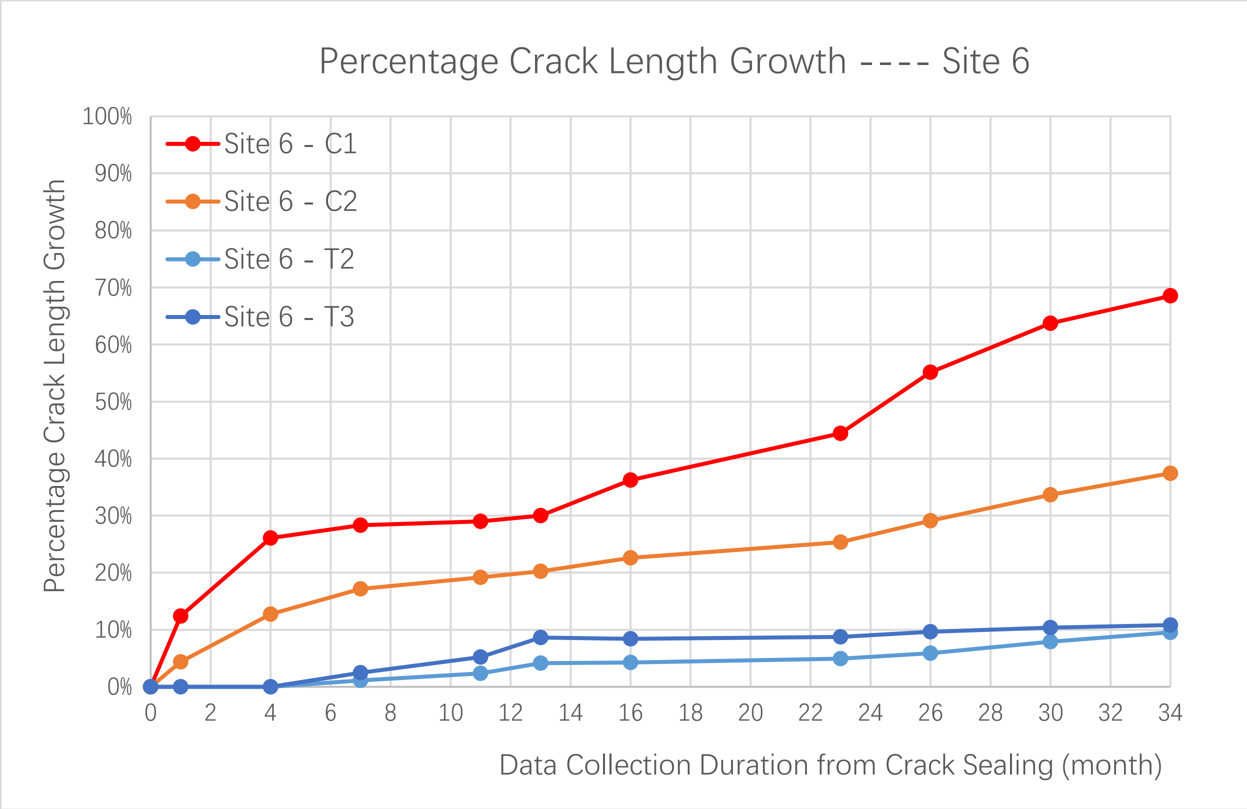


Figure 135. Graph. Percentage Crack Length Growth (Site 6)

Table 49. **Crack Length Growth (Site 8)**

Data Collection Time	Time After Sealing	Total Crack Length in 500-ft Section		
		Site 8 - C1	Site 8 - T1*	Site 8 - T2*
	(month)	(meter)	(meter)	(meter)
2016-11-03 (Before Sealing)	0	109.1	226.2	181.3
12/20/2016	1	109.1	226.2	181.3
3/3/2017	4	114.1	226.2	181.3
6/28/2017	7	144.0	226.9	181.3
10/1/2017	11	163.2	227.4	181.7
12/12/2017	13	167.2	228.1	181.7
3/14/2018	16	171.0	228.4	181.8
1/30/2019	26	180.0	229.2	182.0
5/14/2019	30	186.4	230.3	182.8
9/3/2019	34	191.7	232.4	183.9

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

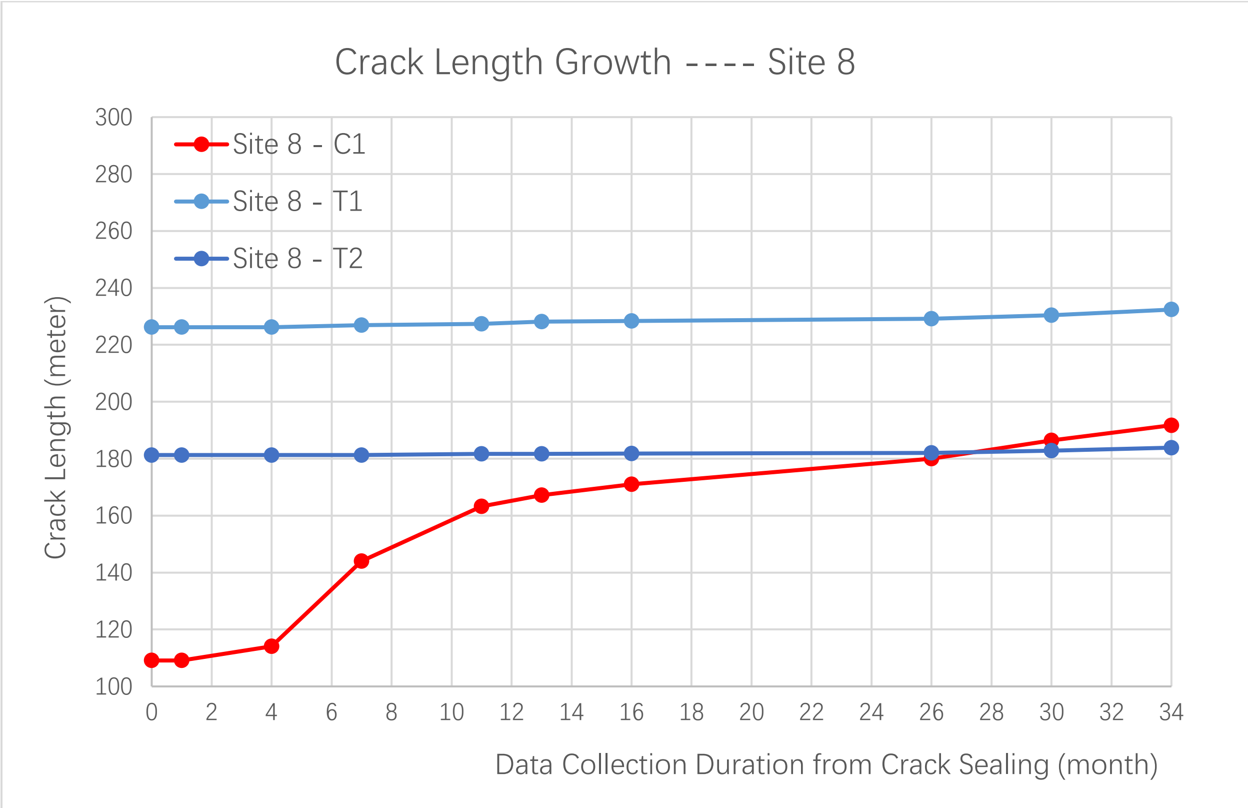


Figure 136. Graph. Crack Length Growth (Site 8)

Table 50. Percentage Crack Length Growth (Site 8)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section		
		Site 8 - C1	Site 8 - T1*	Site 8 - T2*
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%
12/20/2016	1	0.00%	0.00%	0.00%
3/3/2017	4	4.54%	0.00%	0.00%
6/28/2017	7	31.96%	0.33%	0.00%
10/1/2017	11	49.59%	0.53%	0.21%
12/12/2017	13	53.21%	0.85%	0.23%
3/14/2018	16	56.69%	0.98%	0.27%
1/30/2019	26	64.95%	1.34%	0.40%
5/14/2019	30	70.82%	1.84%	0.82%
9/3/2019	34	75.73%	2.74%	1.42%

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

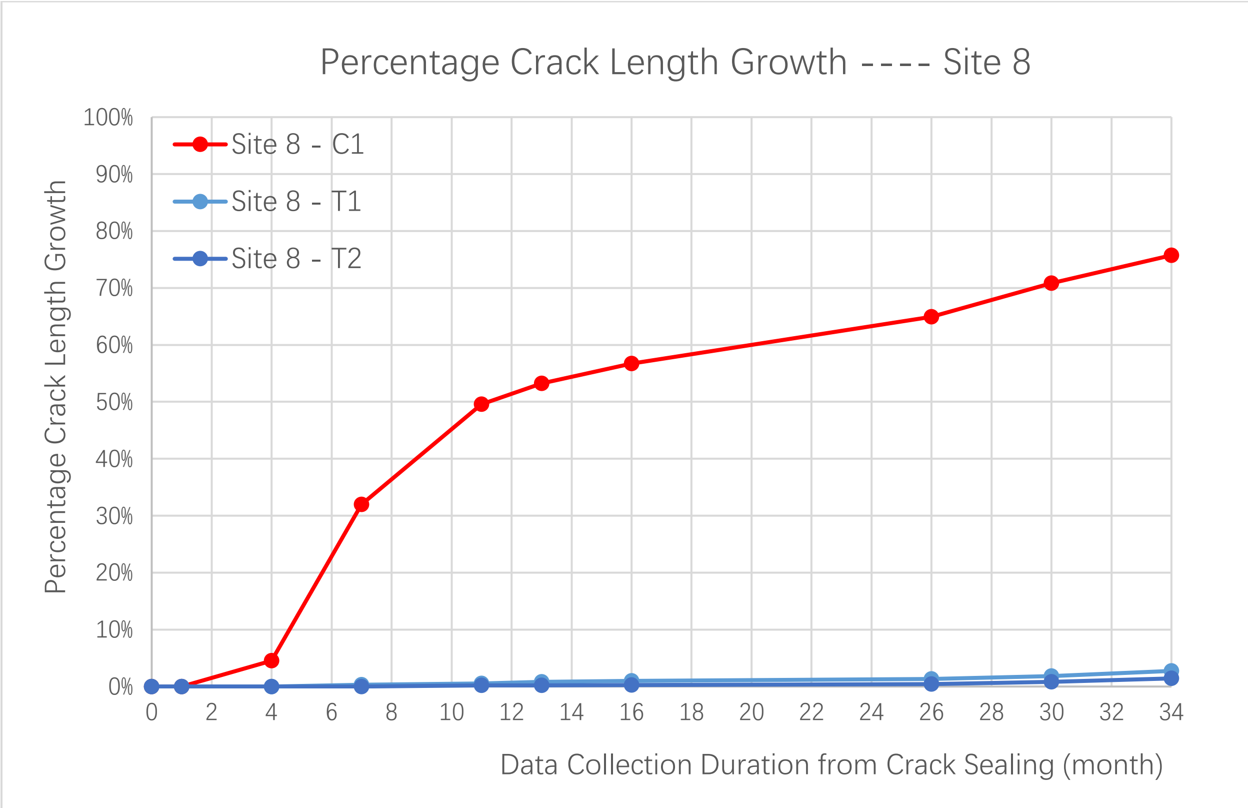


Figure 137. Graph. Percentage Crack Length Growth (Site 8)

Table 51. **Crack Length Growth (Site 9)**

Data Collection Time	Time After Sealing	Total Crack Length in 500-ft Section		
		Site 9 - C2	Site 9 - T3*	Site 9 - T4*
	(month)	(meter)	(meter)	(meter)
2016-11-03 (Before Sealing)	0	151.0	220.8	208.1
12/20/2016	1	151.0	220.8	208.1
3/3/2017	4	159.6	221.0	208.5
6/28/2017	7	184.0	222.2	209.4
10/1/2017	11	191.6	222.7	210.3
12/12/2017	13	197.9	223.3	211.3
3/14/2018	16	212.4	224.8	213.0
1/30/2019	26	224.7	227.0	216.0
5/14/2019	30	230.9	228.3	216.3
9/3/2019	34	236.3	230.5	217.9

(Note: *Crack sealing has been performed at the end of November 2016 for both T3 and T4)

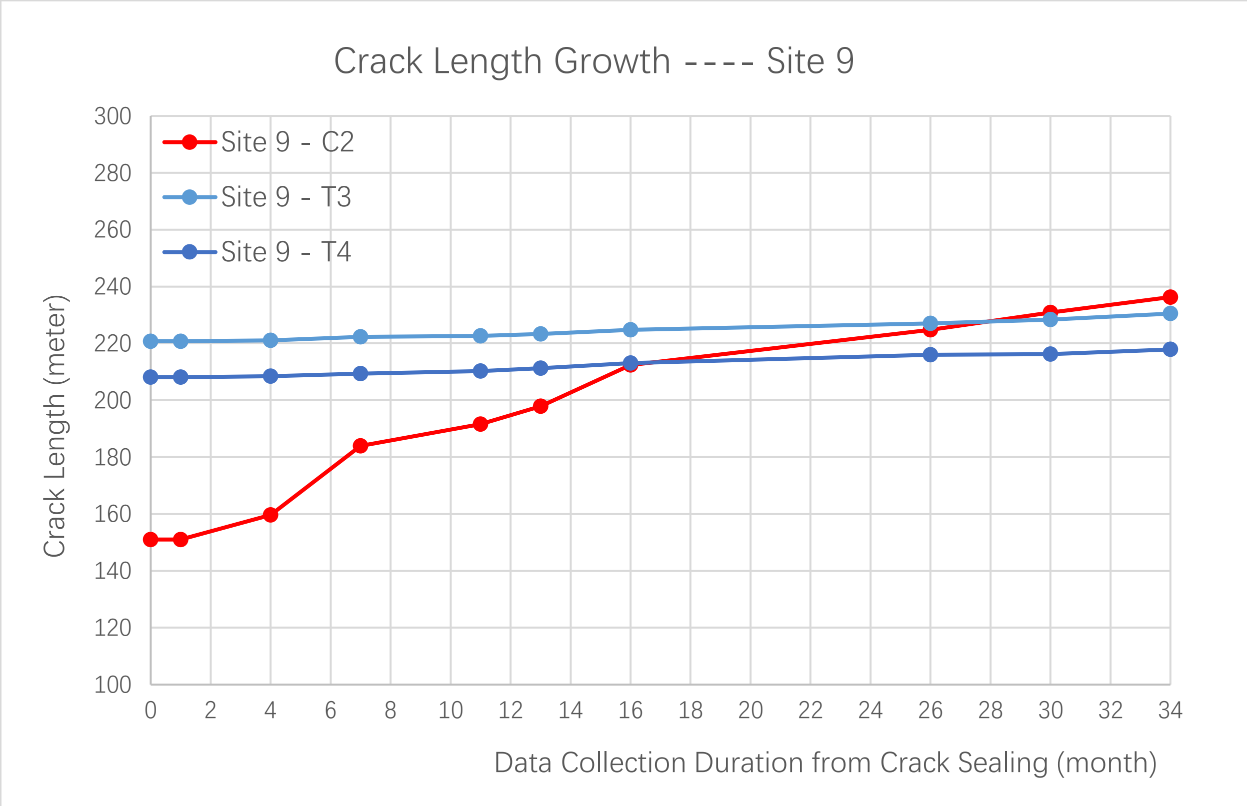


Figure 138. Graph. Crack Length Growth (Site 9)

Table 52. Percentage Crack Length Growth (Site 9)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section		
		Site 9 - C2	Site 9 - T3*	Site 9 - T4*
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%
12/20/2016	1	0.00%	0.00%	0.00%
3/3/2017	4	5.68%	0.12%	0.20%
6/28/2017	7	21.84%	0.67%	0.61%
10/1/2017	11	26.87%	0.87%	1.04%
12/12/2017	13	31.05%	1.14%	1.53%
3/14/2018	16	40.64%	1.82%	2.37%
1/30/2019	26	48.80%	2.81%	3.79%
5/14/2019	30	52.88%	3.42%	3.94%
9/3/2019	34	56.47%	4.40%	4.70%

(Note: *Crack sealing has been performed at the end of November 2016 for both T3 and T4)

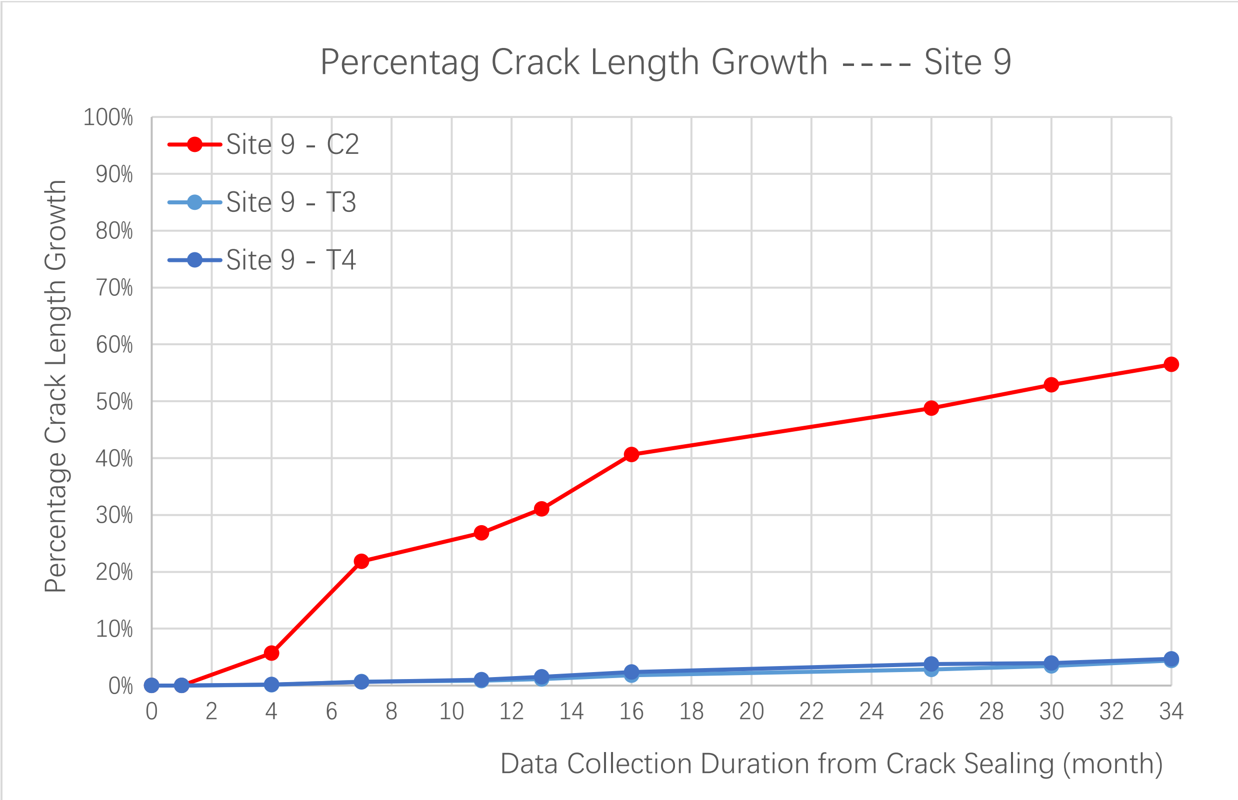


Figure 139. Graph. Percentage Crack Length Growth (Site 9)

Table 53. Crack Length Growth (Site 10)

Data Collection Time	Time After Sealing (month)	Total Crack Length in 500-ft Section		
		Site 10 - C1 (meter)	Site 10 - T1* (meter)	Site 10 - T2* (meter)
2016-11-03 (Before Sealing)	0	134.4	351.6	380.1
12/20/2016	1	134.4	351.6	380.1
3/3/2017	4	139.2	352.1	380.9
6/28/2017	7	163.1	352.4	382.0
10/1/2017	11	173.2	352.8	382.6
12/12/2017	13	183.9	353.2	383.0
3/14/2018	16	194.1	353.9	384.0
1/30/2019	26	216.6	355.9	385.8
5/14/2019	30	232.5	356.8	386.3
9/3/2019	34	241.4	359.0	387.9

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

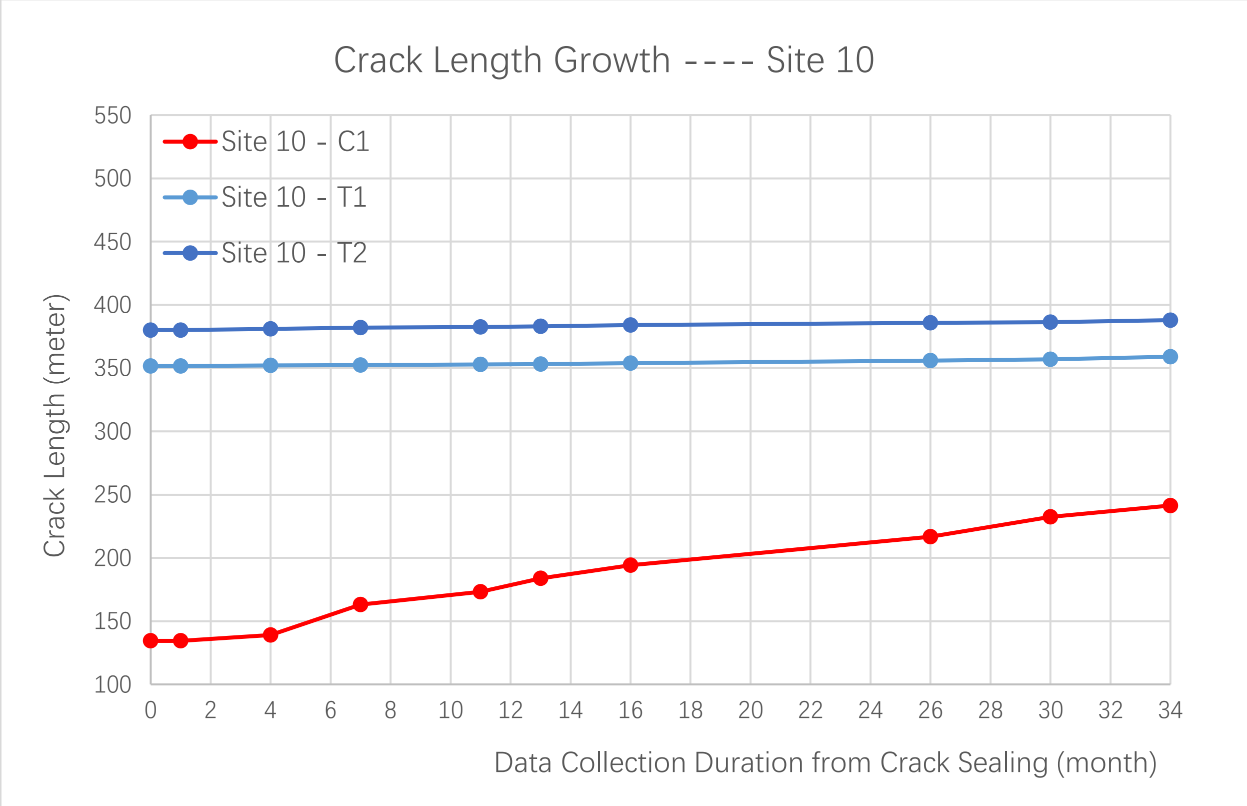


Figure 140. Graph. Crack Length Growth (Site 10)

Table 54. Percentage Crack Length Growth (Site 10)

Data Collection Time	Time After Sealing (month)	Percentage Crack Length Growth in 500-ft Section		
		Site 10 - C1	Site 10 - T1*	Site 10 - T2*
2016-11-03 (Before Sealing)	0	0.00%	0.00%	0.00%
12/20/2016	1	0.00%	0.00%	0.00%
3/3/2017	4	3.52%	0.13%	0.22%
6/28/2017	7	21.31%	0.23%	0.50%
10/1/2017	11	28.83%	0.34%	0.66%
12/12/2017	13	36.82%	0.45%	0.78%
3/14/2018	16	44.42%	0.64%	1.02%
1/30/2019	26	61.15%	1.22%	1.49%
5/14/2019	30	72.96%	1.48%	1.63%
9/3/2019	34	79.58%	2.10%	2.07%

(Note: *Crack sealing has been performed at the end of November 2016 for both T1 and T2)

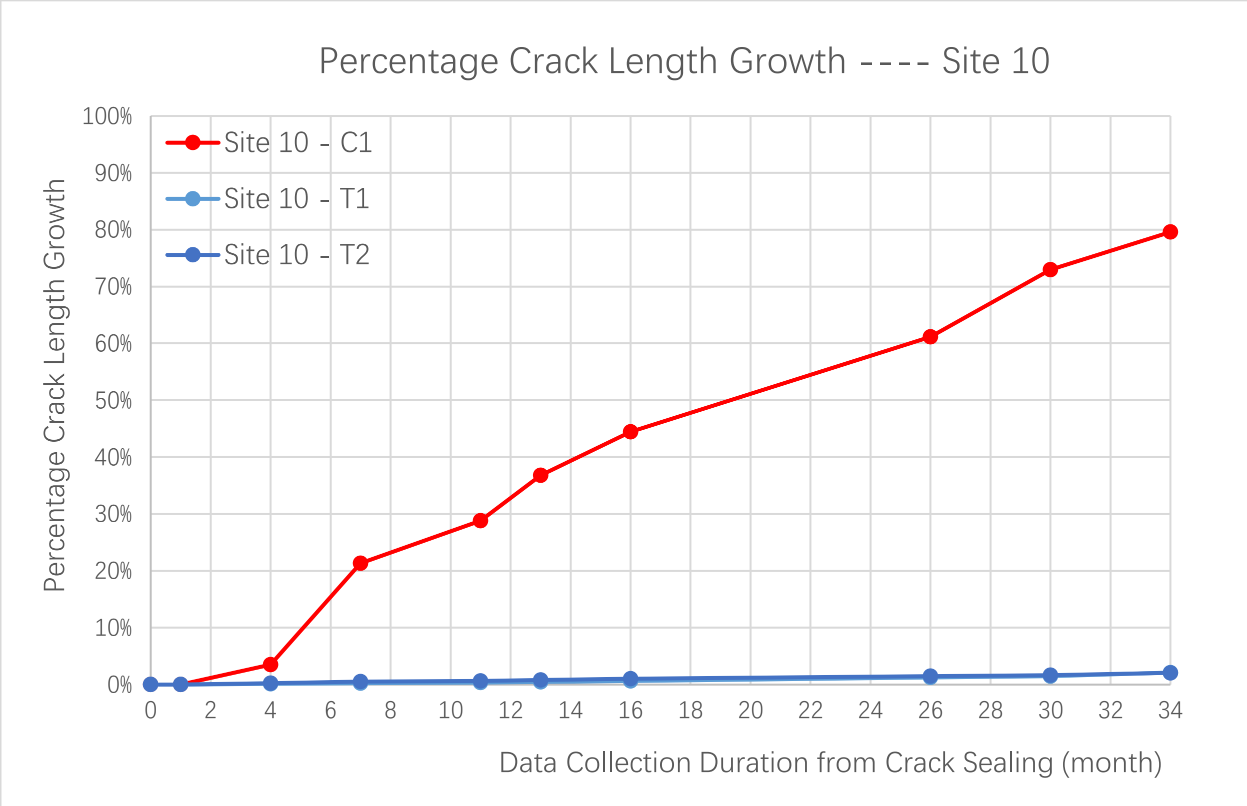


Figure 141. Graph. Percentage Crack Length Growth (Site 10)

Quantifying Crack Sealing Effectiveness

We quantitatively measure the crack sealing effectiveness by comparing the difference of long-term crack length growth between sealed and non-sealed sections. This can be calculated using the following algorithm:

$$CSE = \triangle\, CL(\%)_{Non-Sealed} - \triangle\, CL(\%)_{Sealed}$$

Where CSE = crack sealing effectiveness; $\triangle\, CL(\%)_{Non-Sealed}$ = the percentage crack length growth in non-sealed section; $\triangle\, CL(\%)_{Sealed}$ = the percentage crack length growth in the sealed test section.

The last column of Table 55 summarizes the calculated crack sealing effectiveness for all testing sites over a 3-year testing period according to the above algorithm. The values (i.e. the pretreatment/initial crack length, the absolute crack length growth, and the percentage crack length growth) used to compute crack sealing effectiveness are also summarized. In addition, pavement condition variables (i.e. pretreatment/initial COPACES rating, pavement thickness, AADT, truck percentage, and AADTT) are also incorporated in this table.

Table 55. Summary of Crack Sealing Effectiveness Analysis Results

Site #	Initial COPACES Rating	Pavement Thickness (inch)	Annual Average Daily Traffic (AADT) (2018)	Truck	Annual Average Daily Truck Traffic (AADTT) (2018)	Non-Sealed Section			Sealed Section			Crack Sealing Benefit (%)
						Initial Crack Length (meter)	Crack Length Growth (meter)	Percentage Crack Length Growth (%)	Initial Crack Length (meter)	Crack Length Growth (meter)	Percentage Crack Length Growth (%)	
1	66	9.5	4430	26%	1148	566	243	43%	653	22	3%	40%
2*	95	10.0	4480	24%	959	80	88	111%	60	6	10%	101%
3	69	2.5	1020	13%	128	237	124	52%	403	19	5%	48%
4*	93	6.5	3890	17%	662	102	134	131%	321	23	7%	124%
5*	98	8.0	1030	14%	133	66	68	102%	81	17	21%	82%
6*	69	6.5	5260	14%	722	153	83	54%	178	18	10%	44%
8	79	8.0	7270	6%	349	109	83	76%	204	4	2%	74%
9	85	8.0	12900	4%	488	151	85	56%	214	10	5%	52%
10	82	9.0	8530	3%	166	134	107	80%	366	8	2%	78%

(**If there are more than one non-sealed control section, we use the average crack length to calculate corresponding values (i.e. initial crack length, absolute crack length growth); similarly, if there are more than one crack sealed test section, we use the average crack length to calculate corresponding values; *"average annual daily truck traffic" (AADTT) is calculated using the "annual average daily traffic" (AADT) multiplied by the truck percentage; Table 1-1 is colored by the initial COPACES rating. Red is COPACES ≤ 70. Yellow is 70 < COPACES ≤ 90. Green is 90 < COPACES.)

By comparing the table values, we can observe the positive crack sealing effectiveness in the first 3 years after crack sealing on our selected test sites in Georgia. This is discussed below.

- For all sites, the percentage of crack length growth in the non-sealed control section is significantly greater than the crack sealed test section. By calculating the crack sealing effectiveness of all sites using the "percentage crack length growth on the non-sealed control section" minus the "percentage of crack length growth on the crack sealed test section," it can be seen that the minimum crack sealing effectiveness is 40% on Site 1, and the maximum crack sealing effectiveness is 124% on Site 4. Test outcomes show that crack sealing is very effective in slowing crack growth.

- Both the absolute actual and relative percentage crack length growth on the crack sealed testing sections are significantly low after 3 years of testing. The maximum absolute actual crack length growth on the crack sealed section is 23 meters in Site 4, which is equal to 7% of crack length growth over a 321-meter pretreatment/initial crack length. The maximum percentage crack length growth on the crack sealed section is 21% in Site 5, which is equal to a 17-meter crack growth over an 81-meter pretreatment/initial crack length.

- Results show that crack sealing is very effective (at least 40%) in retarding crack growth for pavement conditions with different initial COPACES ratings (ranging from 66-98), different pavement thicknesses (ranging from 2.5-10 inch), and different traffic volume (AADT ranging from 1,020-10,900; truck percent ranges from 3%-26%; AADTT ranges from 128-1,148).

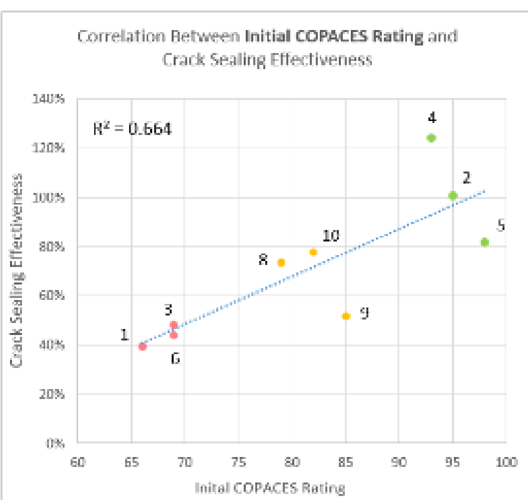
Crack Sealing Timing Analysis

Three aforementioned pavement condition variables (i.e. pretreatment/initial COPACES rating, traffic volume, and pavement thickness) that potentially impact the crack sealing effectiveness are analyzed below. The outcomes can help GDOT determine the adequate timing periods and conditions in which to apply crack sealing.

Pretreatment/Initial COPACES rating

The linear correlation analysis shows a trend (R-squared = 0.664, shown in Figure 142) that a higher COPACES rating will lead to higher effectiveness (if the impact of thickness and traffic is ignored). The breakdown analysis of different COPACES ratings on crack sealing effectiveness is shown below:

- With the COPACES rating range from 93-98, the effectiveness is high (ranging from 82%-124%).
- With the COPACES rating range from 79-85, the corresponding effectiveness is medium (ranging from 52%-74%).
- With the COPACES rating range from 66-69, the corresponding effectiveness is relatively low (ranging from 40%-48%).



Site #	Initial COPACES Rating	Thick-ness (inch)	Traffic Volume			Effective-ness (%)
			AADT (2018)	Truck (%)	AADTT (2018)	
5	98	8	1030	14%	133	82%
2	95	10	4480	24%	959	101%
4	93	6.5	3890	17%	662	124%
9	85	8	12900	4%	488	52%
10	82	9	8530	3%	166	78%
8	79	8	7270	6%	349	74%
3	69	2.5	1020	13%	128	48%
6	69	6.5	5260	14%	722	44%
1	66	9.5	4430	26%	1148	40%

Figure 142. Graph. Correlation Between Initial COPACES Rating and Crack Sealing Effectiveness

Traffic Volume

The direct correlation analysis (without grouping) shows the AADT and AADTT have no strong linear correlation with the crack sealing effectiveness (R-squared = 0.026 and 0.0005, respectively). However, if we group the data by COPACES rating as we did to control the variable (although the impact of thickness is still ignored), we observed two groups (COPACES ranges from 66-69 and 79-85) that show that the sites that have higher truck (AADTT) values will have a lower crack sealing effectiveness (shown in Figure 143).

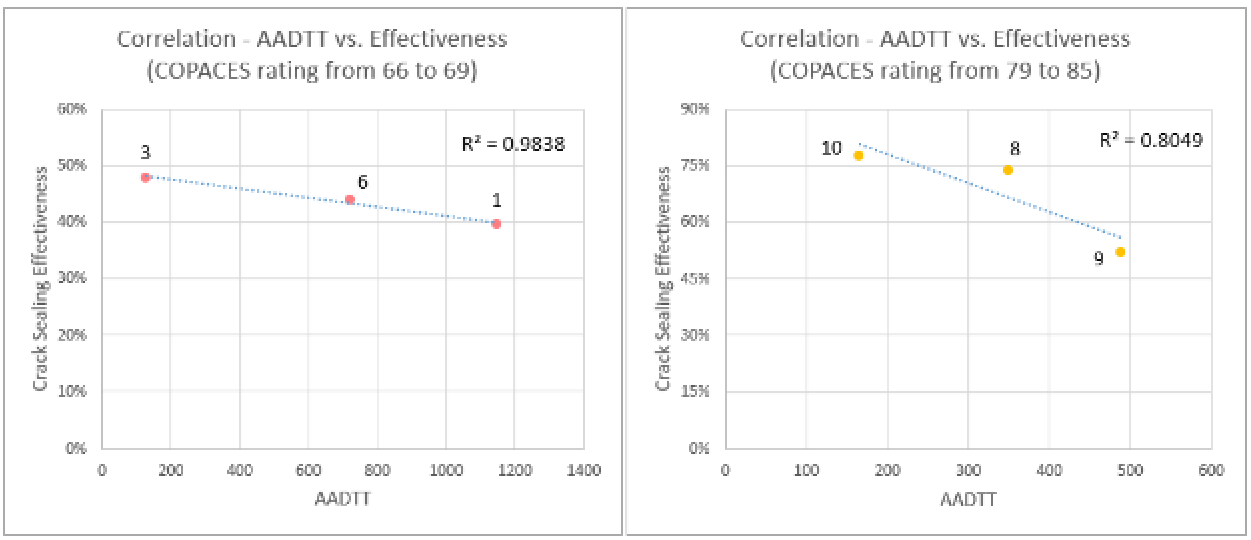


Figure 143. Graph. Correlation Between Traffic Volume (AADTT) and Crack Sealing Effectiveness

Pavement Thickness

There is no correlation observed between thickness and effectiveness whether grouped or not. Figure 144 plots the effectiveness by thickness and grouped by COPACES rating (the same grouping principle as the last two parts). So, the crack sealing effectiveness might not have much difference on different pavement thicknesses.

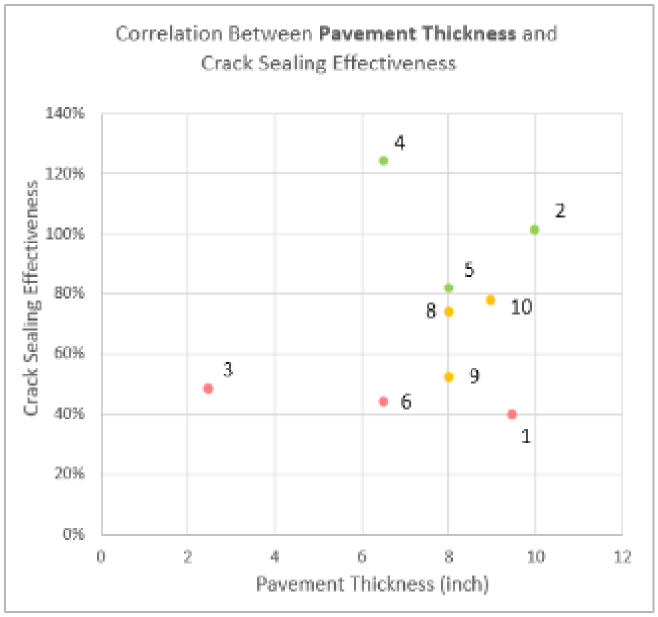


Figure 144. Graph. Correlation Between Pavement Thickness and Crack Sealing Effectiveness

In addition to the analysis of the three pavement condition variables, crack pattern analysis was also conducted. Table 56 reiterates the crack pattern results from the visual observation survey and incorporates the crack sealing effectiveness results simultaneously to analyze the impact of crack pattern (load crack (LC) and block/transverse (B/T) crack) on the sealing effectiveness. The results are presented below:

- Crack sealing still has a 40% effectiveness capability to retard crack growth, even though the pavement has a 45% level-1 LC, a 45% level-2, 5% level-3 LC, and an 80% level-2 B/T at Site 1, which has a 66 pretreatment COPACES rating.

- Crack sealing effectiveness is high (from 82%-124%) when there are small LC percentages (from 0-10%). See Sites 1,4, and 5.

- Crack sealing effectiveness is relatively low (40%-48%) when there are large and severe LC percentages (more than 70% of Level 1 plus Level 2 LC). Meanwhile, level-2 B/T usually occurs (20%-80%) in this case.

- The occurrence of Level-1 B/T crack has less negative impacts on the crack sealing effectiveness than the load crack and Level-2 B/T.

Table 56. Summary of Crack Pattern Analysis

Site #	Initial COPACES Rating	Extend Length Percentage of Cracks by Category					Crack Sealing Effectiveness
		Wheel Path Load Crack			B/T Crack		
		Level 1	Level 2	Level 3	Level 1	Level 2	
1	66	45%	45%	5%	0%	80%	40%
2	95	5%	0%	0%	30%	0%	101%
3	69	60%	20%	0%	10%	50%	48%
4	93	10%	0%	0%	50%	0%	124%
5	98	10%	0%	0%	20%	0%	82%
6	69	60%	10%	0%	40%	20%	44%
8	79	10%	0%	0%	50%	0%	74%
9	85	20%	0%	0%	60%	0%	52%
10	82	20%	0%	0%	30%	0%	78%

(Note: The percentage of load crack and B/T crack represents the initial/pretreatment percent of an extend/projection length over 500 feet in a testing section. The definition of load crack, block/transverse (B/T) crack correspond to the GDOT COPACES manual.)

Crack Sealing Impacts on Pavement Internal Crack Width/Depth Growth

This sub-section analyzes the crack sealing impact on the pavement internal crack width/depth growth; the analysis uses pavement surface crack patterns information extracted from 2D/3D pavement surface images and field coring data. First, pavement surface crack patterns are used to study the crack orientations (i.e. transverse or longitudinal) and crack growth path (i.e. new cracks originating from sealed or non-sealed cracks) to select crack sections; then, coring locations are identified. Based on the surface crack pattern information, two analysis scenarios are designed to study the pavement internal crack width/depth growth seen in the sealed and non-sealed cracks, and the new cracks that originated from sealed or non-sealed cracks. Finally, 6 cores (collected in April 2016) and 28 cores (collected in August 2020) from Site 8 and Site 9 are compared to visually estimate the crack sealing impact on pavement internal crack width/depth growth under two scenarios. The following discussion presents each scenario with its 1) purpose, 2) observations of crack sealing impact on pavement internal crack width/depth growth, and 3) coring locations and photos of cores.

Scenario 1

The purpose of Scenario 1 is to study the crack sealing impact on the pavement's internal crack width/depth growth. The pavement's internal crack width/depth growth is visually estimated by comparing the pavement's internal crack patterns in the previous cores (collected in April 2016) and in the new cores (collected in August 2020). Notice that new cores were taken near the previous cores for the comparison. The following figures include the side view of cores collected both in April 2016 and August 2020, and the coring location for each core. The cores on the longitudinal and transverse cracks are categorized, summarized, and compared separately.

This paragraph presents the cores taken from the longitudinal crack sections. The upper part of Figure 145 shows 4 cores taken from a sealed longitudinal crack section at Site 8. Of these 4 cores, one core was collected in April 2016 (labeled as “(0)” using yellow) and 3 cores were collected in August 2020 (labeled as “(1),” “(2),” and “(3)” using green). The lower part of Figure 145 is the pavement image that shows the pavement surface crack pattern and coring locations. By comparing the core collected in April 2016 with cores collected in August 2020 in Figure 145, we did not observe severe pavement internal crack width/depth growth on the sealed crack section at Site 8 after 4 years. However, we can clearly observe the pavement internal crack width growth in Figure 146, which shows the comparison of the cores on the non-sealed longitudinal crack section at Site 9. In Figure 146, according to the side view of the cores, we can see 1) the internal cracks are wider at the top of cores “(2)” and “(3),” and 2) the cracks on cores “(1),” “(2),” and “(3)” are much wider than core “(0).” Therefore, we visually estimate that the pavement internal crack width growth is clear for this non-sealed crack section. Also, we find the top part of the non-sealed crack is wider than the bottom part, which might be because of the interference of pavement extensions from non-compressible substances dropping from the pavement surface into the crack.

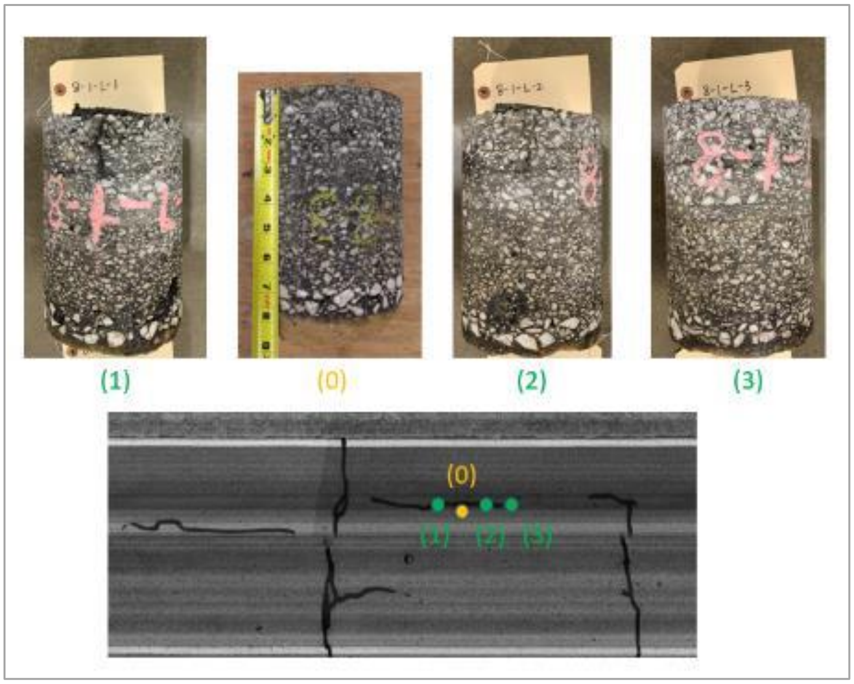


Figure 145. Photo. Cores on Sealed Longitudinal Crack (Site 8)

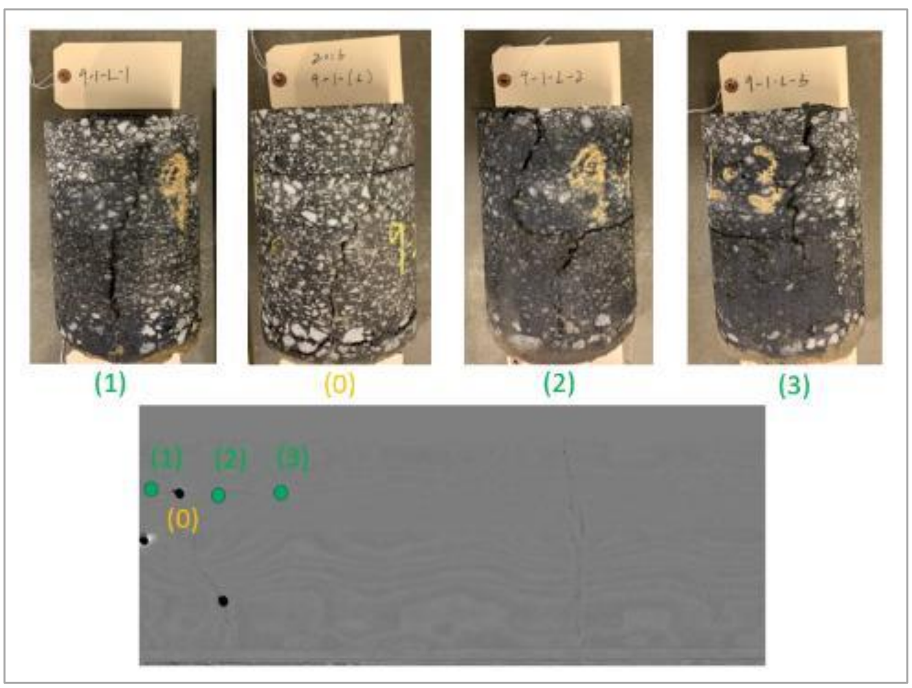


Figure 146. Photo. Cores on Non-Sealed Longitudinal Crack (Site 9)

This paragraph discusses the cores taken from the transverse crack sections. Figure 147 shows 4 cores taken from a sealed transverse crack section at Site 8. From this figure, we can see that the crack on the core collected in April 2016 (labeled as "0" using yellow) has already been penetrated and is severe. Therefore, it is hard for us to compare the pavement internal crack depth growth by comparing these 4 cores. However, we still can see that the cores "(1)" and "(3)" have thinner crack widths at the top and wider cracks at the bottom, which might be because of the crack sealing impact. Figure 148 shows 5 cores taken from a non-sealed transverse crack section at Site 9. From this figure, we can clearly see the crack width is wider at the top of the cores collected in August 2020 (labeled as "(1)," "(2)," "(3)," and "(4)"). Although cores "(2)" and "(3)" are broken because they could not be pulled out as complete cores, the appearance of brown soil on the broken cross-section indicates the pavement has internal cracks and has already been penetrated to the subbase of pavement. Therefore, by comparing the wide and penetrated cracks on the cores "(1)," "(2)," "(3)," and "(4)" collected in August 2020 with the tight and not fully penetrated crack in the core "(0) " collected in April 2016, we visually estimate that the pavement internal crack width/depth growth is clear for this non-sealed crack section.

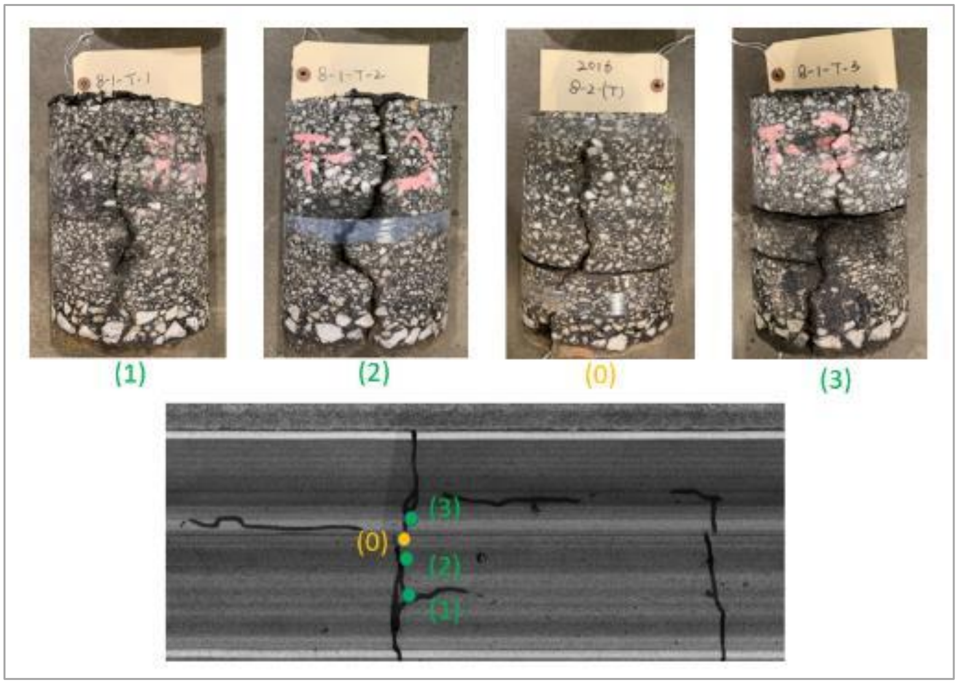


Figure 147. Photo. Transverse Crack, Sealed, Site 8

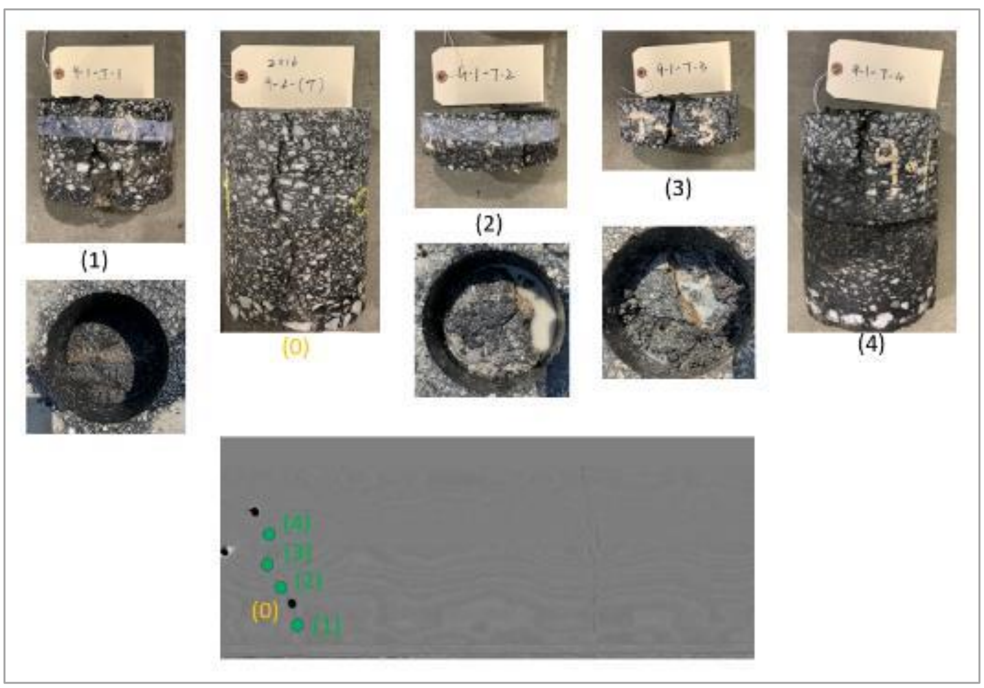


Figure 148. Photo. Transverse Crack, Non-sealed, Site 9

Scenario 2

The purpose of Scenario 2 is to compare the pavement internal crack width/depth growth for the new cracks that originated from sealed or non-sealed cracks to see how the crack sealing impact subsequently affects the crack growth pattern for both sealed and non-sealed cases. Pavement images collected in December 2016 were compared with pavement images collected in September 2019 to identify the new cracks. Then, based on the surface crack patterns (i.e. transverse or longitudinal cracks, sealed or non-sealed cracks, newly grown or non-grown cracks) to select the coring locations.

The following figures are from Site 8 and for Scenario 2. However, no clear difference was observed for cracks that originated from the sealed or non-sealed cracks.

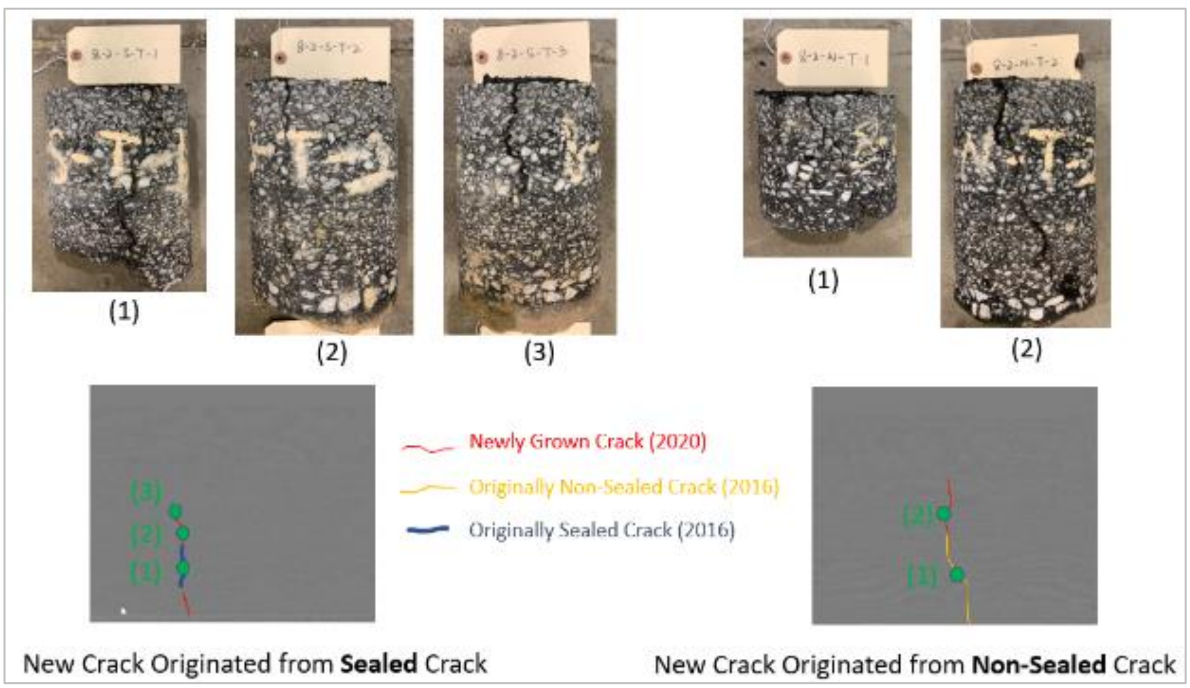


Figure 149. Photo. New Crack Originated from Sealed and Non-sealed Crack (Transverse, Site 8)

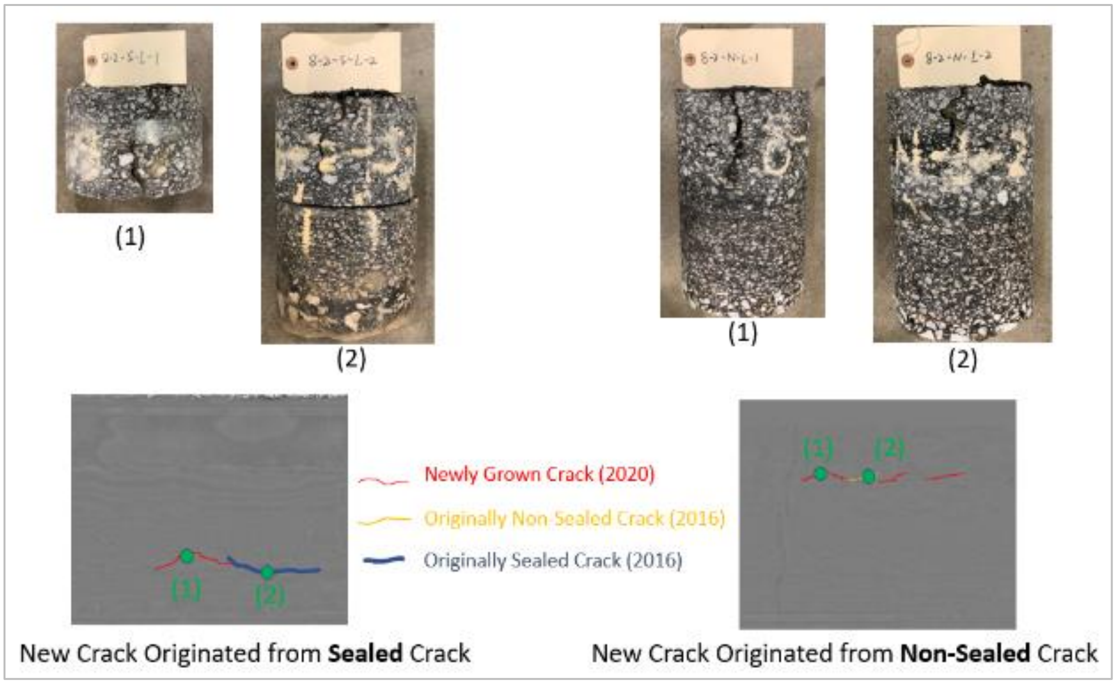


Figure 150. Photo. New Crack Originated from Sealed and Non-sealed Crack (Longitudinal, Site 8)

In addition, we have observed both bottom-up and top-down cracks, such as the top-down crack in Figure 145 and the bottom-up crack in Figure 147. We also observed some have a clear separation between two layers (cracks do not propagate down) and this might be due to no tack coat applied between these two layers, as shown in Figure 150.

Observations of Crack Sealing Impact on the Pavement Internal Crack Width/Depth

- Sealed cracks seem to have better pavement internal crack patterns (i.e. tighter width and shallower depth) than non-sealed cracks. Sealed cracks are usually tighter and thinner than non-sealed cracks when their crack depths are similar.

- The crack width clearly increases for non-sealed cracks (both transverse and longitudinal) but not for sealed cracks, as seen when comparing cores from April 2016 with new cores from August 2020.

- The crack width/depth of the newly grown cracks seems similar for the cracks originating from sealed or non-sealed cracks.

Other Observations from Field Coring

- We have observed both bottom-up and top-down cracks.

- De-bonding between two layers has been observed for some cores.

- Some cracks suddenly stopped growing because of debonding between old and new layers.

SUMMARY

This chapter presents a crack sealing effectiveness study. It presents the research needs and objectives, which are to quantitatively measure and compare the crack sealing effectiveness under different pavement condition variables, including 1) pavement thickness, 2) traffic load, and 3) crack sealing timing (e.g. pretreatment COPACES rating and crack pattern), for verifying and refining GDOT's crack sealing criteria based on test outcomes. Two test design scenarios (expedited and progressive test designs) were applied in this study. Both include a non-sealed control section and sealed test sections from the same site so the effectiveness difference between non-sealed and sealed sections could be objectively compared. For crack growth monitors, we used both absolute crack length growth and also the relative percentage crack length growth to observe the crack deterioration trends. Crack sealing effectiveness was then calculated using the “percentage crack length growth in non-sealed control section” minus the “percentage crack length growth in sealed test section” for each site. The crack sealing effectiveness calculated by the relative percentage crack growth enabled us to compare the sites with different pretreatment pavement conditions. Conclusions are drawn in this section according to the data analysis results and based on the 3-year test outcomes from comparisons between crack sealed test sections and non-sealed control sections at 9 selected test sites; the sites had different pretreatment COPACES ratings (ranging from 66-98) and were located at two different geographical regions in Georgia. Region 1 is located in Hawkinsville (southern Georgia) with Sites 1-6, and Region 2 is located in Covington (central Georgia) with Sites 8 to 10. The pavement corings were cut to get a better understanding of the pavement thickness (ranging from 2.5-10.0 inch); all of them had a soil-cement base. The traffic volume information was collected: AADT ranged from 930-2,200; the truck percentage ranged from 4%-26%, and the AADTT ranged from 128-1,148. Conclusions and recommendations for future research are discussed below.

Conclusions

Our literature review indicates that crack sealing can prolong pavement life. Crack sealing can prevent non-compressible objects (e.g. dirt) from falling into cracks, which leads to crack growth caused by expansion and contraction of vehicles’ tire pressure and temperature changes. In addition, crack sealing can prevent water, which will soften the subbase of pavement, from entering the subbase. However, a quantitative and scientific pavement crack growth evaluation based on long-term performance monitoring is lacking. This study has collected 3D pavement surface data to quantitatively evaluate crack growth quarterly over the past 3 years. Based on 3 years of long-term performance monitoring on crack length growth on 9 field test sites, the following findings are presented:

- Comparison of the performance between crack sealed and non-crack sealed sections shows that crack sealing can effectively prolong pavement life when the COPACES rating ranges from 66-98; it retards crack growth by 40%-128%.

- Results show that the crack sealing effectiveness is higher when applying crack sealing at a higher COPACES rating. Figure 142 shows the crack sealing effectiveness of 3 clusters of COPACES rating ranges from 66-69, 79-85, and 93-98, which are 40%-48%, 52%-72%, and 82%-124%, respectively. It shows a clear trend (R-squared = 0.664) that crack sealing effectiveness increases with higher pretreatment COPACES ratings. This actual performance finding suggests that GDOT should increase its crack sealing criteria from a COPACES rating of 85 to 90. This new policy will allow GDOT to prolong pavement life effectively.

- Further analyses on the impact of pavement thickness and traffic volume, especially AADT and AADTT, have been conducted by analyzing COPACES rating clusters of 66-69, 79-85, and 93-98. In two clusters (rated 66-69 and 79-85), results show crack sealing effectiveness increases when the truck traffic volume (AADTT) decreases. However, the finding is not conclusive. Additional tests are needed to confirm the impact of truck traffic. There is no clear trend for the impact of pavement thickness on crack sealing effectiveness.

- Additional observations of which types of cracks have higher crack sealing effectiveness show that the occurrence of more and severer pretreatment longitudinal cracking in non-wheel paths (GDOT defined it as load crack) and more Severity Level 2 block/transverse (B/T crack based on GDOT's definition) cracking will have lower crack sealing effectiveness. However, the different amount of Severity Level 1 block/transverse cracking makes no difference on the crack sealing effectiveness. Further studies can be performed to quantify the crack pattern difference in crack sealing effectiveness.

Recommendations

This section presents the following recommendations for future research:

- It is recommended to study the difference of extended pavement life because of different crack sealing timing (e.g. COPACES rating) by 1) continuous monitoring of the pavement performance (e.g. 3 years, 5 years), and 2) comparing the long-term COPACES rating difference between sealed and non-sealed sections.

- Additional test sites with severe cracks (e.g. Severity Level 3) should study crack sealing effectiveness of high cracking severity levels so that state DOTs can know the expected pavement life when crack sealing is applied to high cracking severity level pavements as a short-term patch.

- Study the impact of crack sealing on the reduction of pavement friction and establish a criterion for the maximum crack sealing area. This means that it is not adequate to apply crack sealing when the area of sealing is greater than a certain threshold value.

CHAPTER 6. FIELD TEST STUYD OF FOG SEAL PERFORMANCE

INTRODUCTION

Research Background

Open graded friction course (OGFC) has a porous structure that allows water to drain through it, which provides safety benefits by improving visibility and friction and reducing splash, spray, and hydroplaning risk in wet weather conditions (Onyango et al., 2017). It has been widely used on Georgia's interstate highways. Raveling is the dislodging of coarse aggregates in OGFC pavements. It has become the predominant pavement distresses in Georgia's interstate highway with OGFC pavements. The Georgia Department of Transportation (GDOT) has used traditional milling and overlay (remove both 7/8 inch OGFC and then 1.5 inch stone matrix asphalt, SMA underneath; and then overlay both layers), or micro-milling and thin overlay (remove only 7/8 inch OGFC and overlay only 7/8 inch OGFC) when the structure in the underlying layers is sound. To slow down the pavement deterioration (e.g. raveling) and prolong the life of OGFC pavements, GDOT has actively sought alternative preventive maintenance methods. In this research project, Georgia Tech has worked with GDOT to study the feasibility of slowing raveling (loss of aggregates) using fog seal. The field study of fog seal application has been performed on Interstate Highway 475 (I-475), a bypass around Macon, Georgia, from October 16, 2014, to December 4, 2014, by monitoring long-term fog seal performance.

Defined by the Asphalt Emulsion Manufacturers Association (AEMA), fog seal is "a light spray application of dilute asphalt emulsion used primarily to seal an existing asphalt surface to reduce raveling and enrich dry and weathered surfaces" (AI & AMEA, 2014). It is used on raveled pavements as a method of holding stone in place. However, the best treatment timing of fog seal is still unknown. Visual inspection and engineers' judgments still comprise the major approach for determining fog seal treatment (Caltrans, 2003). In general, traffic level is not a determining factor, except in job set up. However, fog seal use is restricted to heavily trafficked roads, as the friction may be reduced after its application (TxDOT, 2006). However, in the US, there is only very limited experience with applying fog seal to interstate highways that have high traffic volume.

I-475 is a high-traffic-volume interstate highway (AADT was about 40,000 to 50,000 in 2012). Figure 151 shows the location of the I-475 test site, which is located south of Atlanta near Macon, Georgia. The length of I-475 is 15 centerline miles. It has 3 northbound and 3 southbound lanes totaling 90 (15 × 6) lane miles. Based on GDOT's pavement survey using the Pavement Condition Evaluation System (PACES), the predominant distress on I-475 is raveling at Severity Level 1 (loss of aggregates approximately ranging from 0% to 20%).

To evaluate the feasibility of applying fog seal on high-traffic-volume interstate highways, GDOT has worked with the research team at the Georgia Institute of Technology (Georgia Tech) using the I-475 project as a case study to answer the following research questions:

1) Can fog seal be applied to interstate highways that have a heavy traffic volume, a raveling problem, safety concerns (reduced friction; splash & spray), and performance concerns (durability and smoothness)?

2) If the answer to Question 1 is yes, what is the right timing to apply fog seal that results in the best performance?

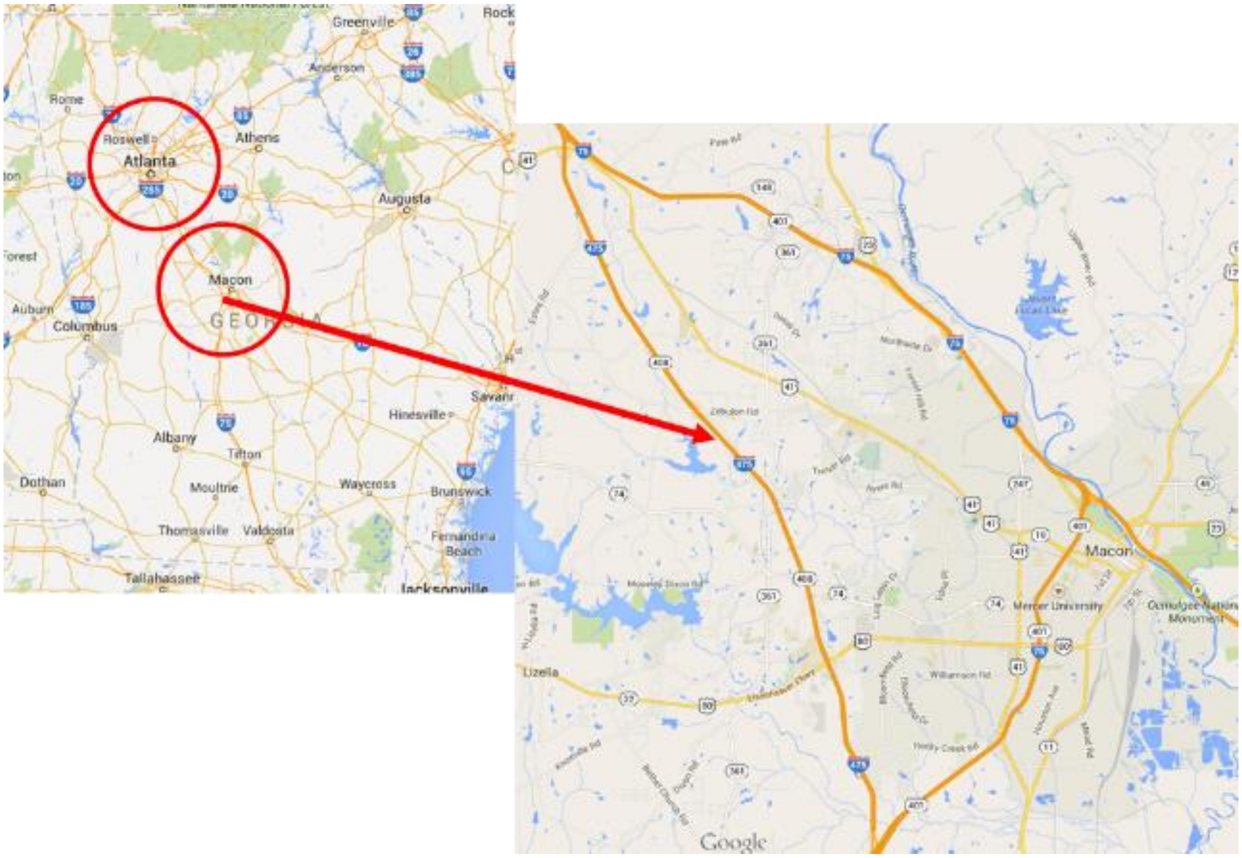


Figure 151. Graph. Location of I-475 Test Site

Research Objectives

The objectives of this research are as follows:

1) to study the feasibility of applying fog seal to high-traffic-volume interstate highways with raveling problems in terms of safety by evaluating friction and permeability;

2) to study the feasibility of applying fog seal to high-traffic-volume interstate highways with raveling problems in terms of performance by evaluating durability and smoothness; and

3) to evaluate the adequate timing (e.g. raveling conditions) of applying fog seal that results in better performance.

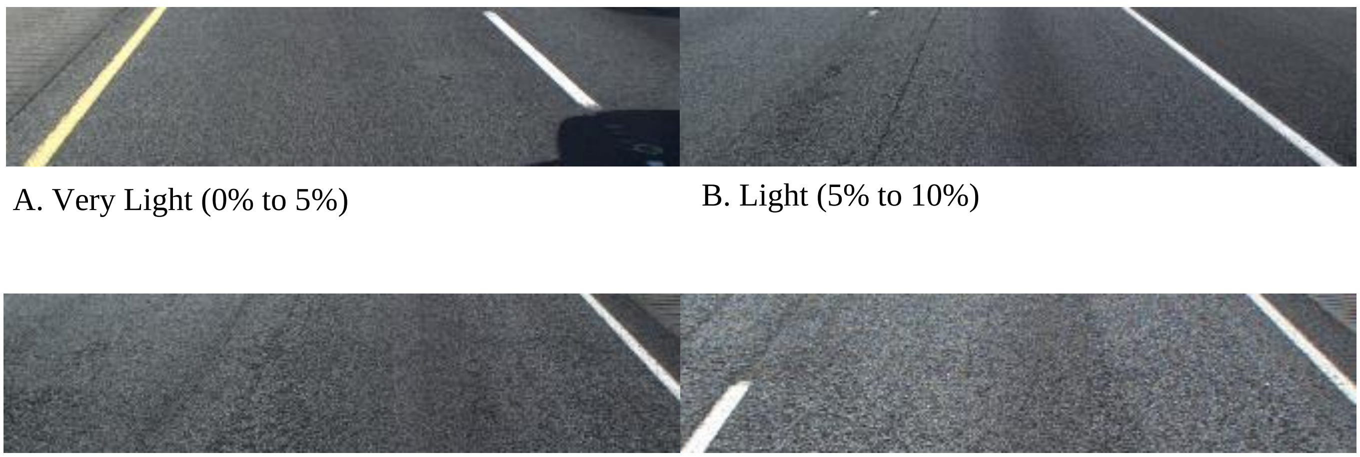
Chapter Organization

This chapter is organized as follows. The background and objectives of this study are presented in this section. Section 2 describes field test design. Section 3 presents friction tests and outcomes. Section 4 presents the IRI tests and outcomes. Section 5 presents visual inspection of aggregate loss. Section 6 presents conclusions and recommendations.

FIELD TEST DESIGN

Field Test Design Factors

To evaluate the performance of fog seal, two key factors are considered in the test site selection and design: 1) existing pavement conditions (i.e., raveling conditions), and 2) traffic load. Based on GDOT's PACES, the predominant distress on I-475 is raveling at Severity Level 1 (aggregates loss ranging from 0% to 20%), which is defined as the "loss of substantial number of stones" (Severity Level 2 is defined as "loss of most surface," and Severity Level 3 is defined as "loss of substantial portion of surface layer.") Since fog seal is used to enhance the bonding strength among aggregates in the surface friction course, it might be suitable for application on pavements with Severity Level 1 raveling. On the other hand, it is not suitable for Severity Levels 2 or 3 raveling conditions because of the loss of pavement friction course. Nevertheless, the definitions of raveling severity levels are subjective. They could result in a broad scope of pavement raveling conditions, which makes it inconvenient to study the timing of fog seal application. Thus, in this research, four sub-categories are defined for Severity Level 1 raveling, *Very Light* (0%-5%), *Light* (5%-10%), *Medium* (10%-15%), and *Severe* (15%-20%), based on the estimated percentage of aggregate loss. It should be noted that the percentage of aggregate loss computation using 3D laser data was not validated. However, the estimated percentage of aggregate loss can be used to further categorize pavements with Severity Level 1 raveling into four finer conditions. Thus, they can be used to select different test sites and to study the optimal timing of fog seal application. Figure 152 shows examples of the four sub-categories of Severity Level 1 raveling.



A. Very Light (0% to 5%)
B. Light (5% to 10%)
C. Medium (10% to 15%)
D. Severe (15% to 20%)

Figure 152. Photo. Four Sub-categories for Severity Level 1 Raveling

To consider the impact of traffic, test sites were selected in different lanes (inside lane – Lane 1; middle lane – Lane 2; and outside lane – Lane 3). Heavy trucks normally run in Lane 3; most vehicles running in Lane 1 are passenger cars. In the original design, two different spray rates for asphalt emulsion were suggested to test the performance difference. However, frequent changes of spray rate would interrupt the construction process. Thus, this design factor (spray rate) was not tested in actual testing. GDOT did do some testing in Lane 1 southbound (from milepost 4.12 to 1.11) using different spray rates. The spray rates for other sections and lanes were either 0.2 or 0.21gal/yd^2 (a small section in Lane 1 southbound, milepost 1.11 to 0, used a spray rate of 0.18 gal/yd^2).

Test Sites Design and Selection

Each test site was selected as a 1,500-ft pavement segment in a lane. The pavement conditions (raveling) were uniform on each test site. To evaluate the performance of fog seal, each test site was divided into a 500-ft non-fog sealed control section and a 1000-ft fog sealed test section, as shown in Figure 153. The 1000-ft fog sealed test section with one spray rate was supposed to have two 500-ft sections with two different spray rates. However, the design factor was not tested in actual testing, as mentioned in the previous section.

Figure 153. Graph. Layout of a Test Site

In terms of raveling conditions and traffic, 13 test sites were selected on I-475 southbound and northbound. Figure 154 shows the spatial locations of all 13 tests sites. Four test sites were selected in Lane 1; 1 test site was selected in Lane 2; and 8 sites were selected in Lane 3. The detailed location information and pavement condition for each test site can be found in Table 57.

Figure 154. Graph. Locations of 13 Test Sties

Table 57. List of 13 Test Sites

Site No.	Raveling Condition	Length (ft.)	Lane No.	Direction	RCLINK	MP From	MP To
1	Severe	1,500	3	SB	0211040800	7.832	7.547
2	Medium	1,500	3	SB	0211040800	11.095	10.811
3	Medium	1,500	3	SB	2071040800	0.111	0.008
4	Light	1,500	3	SB	2071040800	0.801	0.487
5	Light	1,500	3	SB	0211040800	5.558	5.267
6	Very Light	1,500	3	SB	0211040800	6.621	6.308
7	Very Light	1,500	1	SB	2071040800	2.006	1.692
8	Very Light	1,500	1	SB	0211040800	10.260	9.953
9	Light	1,500	3	NB	0211040800	1.433	1.749
10	Very Light	1,500	3	NB	0211040800	5.890	6.203
11	Light	1,500	2	NB	0211040800	3.190	3.506
12	Very Light	1,500	1	NB	0211040800	2.924	3.239
13	Very Light	1,500	1	NB	0211040800	6.211	6.525

Fog Seal Application and Testing

GDOT started fog seal application on I-475 on October 16, 2014, and completed it on December 4, 2014. Two trucks were used for fog seal application. Table 58 and Table 59 list the workload and spray rate for each truck on each day.

Table 58. Workload for Truck 1

TRUCK 1								
Date	Direction Lane	Spread Rate	Gallons Used	Shot / Load	Distance	Begin Milepost	End Milepost	Comments / Width FT.
10/16/2014	SB1	0.105	38	1	298	4.12	4.06	14
	SB1	0.060	12	2	100	4.06	4.04	14
	SB1	0.130	23	3	114	4.04	4.02	14
	SB1	0.065	13	4	125	4.02	4.00	14
	SB1	0.180	30	5	110	4.00	3.98	14
	SB1	0.180	1077	1	3828	3.98	3.26	14
	SB1	0.180	1302	2	4632	3.26	2.38	14
	SB1	0.180	1000	3	4400	2.38	1.55	14
	SB1	0.180	647	4	2297	1.55	1.11	14
10/20/2014	SB2	0.200	47	1	168	3.82	3.79	14
	SB2	0.200	1600	1	5465	3.79	2.75	14
	SB2	0.200	1604	2	5141	2.75	1.78	14
	SB2	0.200	1193	3	3812	1.78	1.06	14
	SB 3	0.200	1180	4	5217	5.84	4.85	11
10/21/2014	SB 3	0.200	1600	1	7459	4.85	3.44	11
	SB 3	0.200	363	2	1481	3.44	3.16	11
	SB 3	0.200	1600	2	6053	3.16	2.01	13
	SB 3	0.200	1300	3	5243	2.01	0.00	12
10/22/2014	SB 2	0.200	1100	1	3297	1.06	0.44	14
	SB2	0.200	1600	2	5324	0.44	0.20	14
	SB2	0.200	315	3	1045	0.20	0.00	14

(continued)

	SB 1	0.180	1266	1	4839	1.11	0.19	13
	SB 1	0.180	1500	2	5075	0.19	0.00	13
10/23/2014	NB 3	0.180	1224	1	5165	0.00	1.60	12
	NB 3	0.200	1600	2	6318	1.60	2.70	12
	NB 3	0.200	1718	3	6428	2.70	3.90	12
	NB 3	0.200	1425	4	5078	3.90	4.90	12
10/27/2014	NB 2	0.200	900	1	4458	0.00	1.20	11
	NB 2	0.200	1600	2	6995	1.20	2.50	11
	NB 1	0.200	1600	1	5408	0.00	1.40	14
	NB 1	0.200	1600	2	5498	1.40	2.00	14
	NB 1	0.200	24	3	76	2.00	2.50	14
	NB 2	0.200	339	3	1378	1.40	2.60	11
	NB 1	0.200	541	3	1731	2.50	3.10	14
10/28/2014	NB 2	0.200	1600	1	6387	2.90	3.70	12
	NB 2	0.200	1600	2	3441	3.70	4.80	12
	NB 1	0.200	1600	1	6397	2.90	3.50	14
	NB 1	0.200	1600	2	5777	3.50	4.80	14
10/29/2014	NB 3	0.200	1600	1	5983	4.80	6.00	13
	NB 3	0.210	1600	2	5455	6.00	7.30	13
	NB 3	0.210	1575	3	5715	7.30	8.10	13
11/3/2014	NB 1	0.210	1600	1	4995	5.00	6.20	14
	NB 1	0.210	1525	1	9503	6.20	7.30	14
	NB 2	0.210	1600	1	7639	5.00	6.00	11
	NB 2	0.210	1600	2	8088	6.00	7.30	11
11/4/2014	NB 2	0.210	1600	1	4967	7.30	8.30	13
	NB 2	0.210	1600	2	5096	8.30	9.30	13

(continued)

	NB 2	0.210	800	3	2909	9.30	9.90	13
	NB 2	0.210	1500	4	4237	9.90	10.80	13
11/5/2014	NB 2	0.210	1600	1	4354	10.80	11.80	13
	NB 2	0.210	800	2	2817	11.80	12.10	13
	NB 2	0.210	1600	3	5071	12.10	13.00	13
	NB 2	0.210	800	4	2230	13.80	13.30	13
	NB 3	0.210	800	4	2971	8.10	8.90	12
11/6/2014	NB 3	0.210	1600	1	5393	8.90	10.10	12
	NB 3	0.210	1600	2	5422	11.20	12.30	12
	NB 3	0.210	1600	3	5234	12.90	14.00	12
11/10/2014	NB 1	0.210	1600	1	4908	13.30	14.40	13
	SB 1	0.210	1600	2	4740	13.90	12.90	14
11/12/2014	SB 1	0.210	1600	1	4997	11.80	11.10	14
	SB 1	0.210	1600	2	4512	10.20	9.30	14
	SB 1	0.210	1600	3	4726	8.30	7.30	14
11/13/2014	SB 3	0.210	1600	1	5080	14.30	13.30	12
	SB 3	0.210	1600	2	5648	13.30	12.30	12
	SB 3	0.210	1600	3	6063	12.30	11.00	12
12/2/2014	SB 3	0.210	1600	1	4627	10.50	9.50	12
	SB 3	0.210	1600	2	4997	9.50	8.50	12
	SB 3	0.210	1600	3	5445	8.50	7.50	12
12/3/2014	SB 2	0.210	1600	1	5006	4.80	3.85	14
	SB 3	0.210	1600	2	8607	7.50	5.87	12
	SB2	0.210	75	3	175	3.85	3.82	14
	SB 3	0.210	75	3	175	5.87	5.84	12
12/4/2014	SB 1	0.210	1500	1	4610	5.00	4.12	14

Table 59. Workload for Truck 2

TRUCK 2								
Date	Direction Lane	Spread Rate	Gallons Used	Shot / Load	Distance	Begin Milepost	End Milepost	Comments / Width FT.
11/3/2014	NB 1	0.210	1600	1	4995	5.00	6.20	14
	NB 1	0.210	1500	2	4500	6.20	7.30	14
11/4/2014	NB 1	0.210	1620	1	4884	7.30	8.30	14
	NB 1	0.210	1000	2	3035	8.30	8.60	14
	NB 1	0.210	1640	3	5172	8.60	9.90	14
	NB 1	0.210	1600	4	4796	9.90	10.80	14
11/5/2014	NB 1	0.210	1600	1	5004	10.80	11.20	14
	NB 1	0.210	1621	2	4704	11.20	12.40	14
	NB 1	0.210	1400	3	4764	12.40	13.30	14
11/6/2014	NB 3	0.210	1600	1	5635	10.00	11.00	13
	NB 3	0.210	1500	2	4928	12.40	13.00	13
11/10/2014	NB 2	0.210	1600	1	5248	13.20	14.00	14
	NB 2	0.210	925	2	3152	13.90	15.00	13
	NB 1	0.210	1200	3	3644	14.20	15.00	14
11/12/2014	SB 1	0.210	1600	1	4615	14.90	13.90	14
	SB 1	0.210	1600	2	4012	12.90	11.80	14
	SB 1	0.210	1250	3	4056	11.10	10.20	14
	SB 1	0.210	1600	4	4724	9.30	8.30	14
	SB 1	0.210	450	5	1476	7.30	7.60	14
11/13/2014	SB 2	0.210	1600	1	5930	14.90	13.80	12
	SB 2	0.210	1600	2	8130	13.80	12.20	12

(continued)

	SB 2	0.210	1600	3	5510	12.20	11.20	12
	SB 2	0.210	925	4	3318	11.20	10.60	12
	SB 3	0.210	704	1	2710	11.40	10.50	12
12/2/2014	SB 2	0.210	1600	1L 1S	4992	10.50	9.40	14
	SB 2	0.210	1600	2	5026	9.40	8.20	14
	SB 2	0.210	1600	2L 1S	4482	8.20	8.60	14
	SB 2	0.210	1600	2	5306	8.60	7.10	14
	SB 3	0.210	600	3	2312	8.60	7.10	12
12/3/2014	SB 2	0.210	1600	1L 1S	4570	7.10	6.20	14
	SB 2	0.210	928	2	2828	5.50	4.80	14
	SB 3	0.210	657	3	2777	7.10	6.50	10
12/4/2014	SB1	0.210	1600	1	4852	7.30	6.30	14
	SB1	0.160	697	2	2800	5.50	5.00	14

To evaluate the performance of the fog seal, various tests, such as the locked-wheel friction tests, IRI tests, permeability tests, and aggregate loss evaluations, were conducted. Due to the need of traffic control, permeability tests were conducted before and right after fog seal application on both control and test sections. Friction tests and IRI tests were conducted by GDOT. The Georgia Tech research team conducted the fog seal durability tests to measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and fog-sealed test sections based on the quantitative measurements of aggregate loss falling into the rumble strips on Sites 1, 2, 4, and 6. It is noted that the permeability tests conducted by using a Permeameter were very inconsistent due to the porous OGFC surface. The measurement results were largely dependent on operators and cannot be used to objectively assess the water drainage capability of pavement OGFC layer. Thus, the permeability test results are excluded in this report. The following sections present the results of friction tests, IRI tests, and durability tests.

FRICTION TEST RESULTS

Introduction

The main objective of a friction test is to evaluate the feasibility, especially for safety, of applying fog seal on interstate highways. Through continuous testing, the following questions are to be addressed:

1) How will the friction drop right after fog seal is applied?

2) How soon will the friction recover to a desired level (e.g. SN = 35)?

3) How soon will the friction recover to its original level before fog seal?

4) How will fog seal affect the behavior of friction in the long term?

5) Will seasonal change (e.g. weather, air temperature) affect the behavior of friction in the long term?

6) Will traffic conditions (e.g. traffic volume, truck percentage) affect the behavior of friction in short term and long term?

To address the above questions, locked-wheel tests were conducted by GDOT over time. The locked-wheel tester used in this research is shown in Figure 155:

Figure 155. Photo. Locked-wheel Tester

To take a measurement, the locked-wheel tester is brought to the desired testing speed (typically 64 km/hr (40 mph)) and water is sprayed ahead of the test tire to wet the pavement surface. The test tire braking system is then actuated to lock the test tire. Instrumentation measures the friction force acting between the test tire and the pavement and reports the result as a skid number (SN).

The locked-wheel test was performed in the following two different methods:

1) The site-specific friction tests were the first data collection method. For a 1500-ft long test site, locked-wheel tests were done on six different spots, three on each wheel path. As illustrated in Figure 156, the surveyor collected two spots located in the control section and four spots in the fog seal test section. An example of collected SNs on Site 1 is shown in Figure 157. There were 13 selected field test sites, which are shown in Table 57.

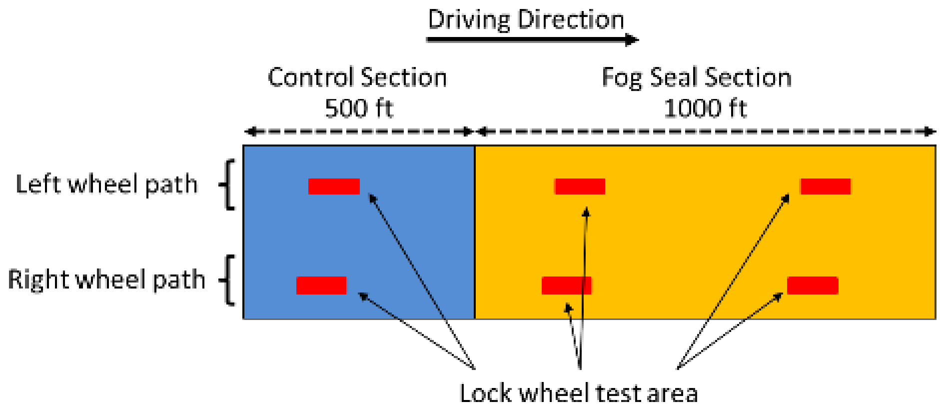


Figure 156. Graph. Locked-Wheel Test Design on a Test Site

Figure 157. Graph. Lock Wheel Test on Site 1

2) The entire I-475 friction tests were done in the second data collection method. Locked-wheel tests were performed on each wheel path at fixed intervals of 1 mile in each lane for the entirety of I-475. As shown in Figure 158, approximately 30 SNs were measured for each lane during one data collection.

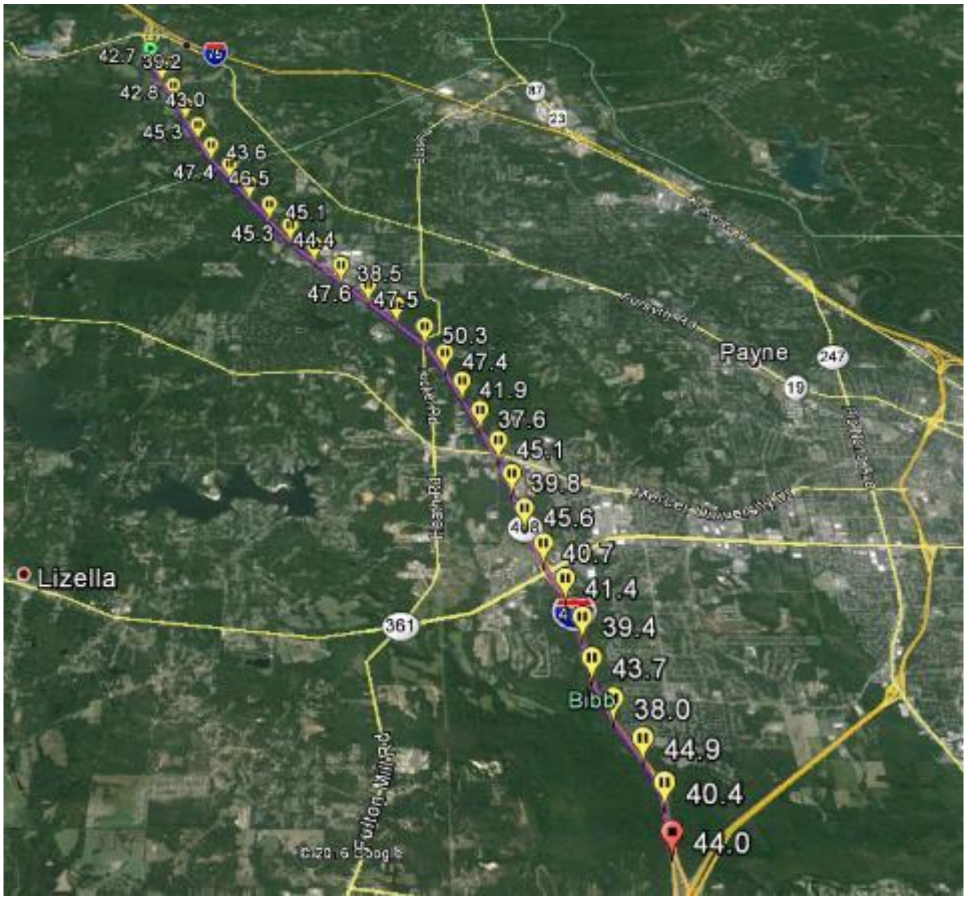


Figure 158. Graph. Lock Wheel Test along the Entire Lane (SB Lane 3)

Analysis of Short-term Friction Behavior

Short-term friction behavior indicates the change of pavement surface friction after fog seal application after one to several weeks. The following are the two questions to be addressed through the evaluation of short-term friction behavior:

1) How low does the friction drop immediately after fog seal application, and would it be below the desired level (e.g., SN = 35)?

2) How long does it take for friction to recover to a desired level of SN = 35?

For the short-term friction behavior, it is expected that the surface friction would decrease immediately after fog seal application. Under traffic conditions, the friction value will recover gradually to the original level (Li et al., 2011). Figure 159 illustrates the expected trend of friction behavior after fog seal application, which is also the original field test design for the friction testing (note: the actual testing schedule was different from the planned one).

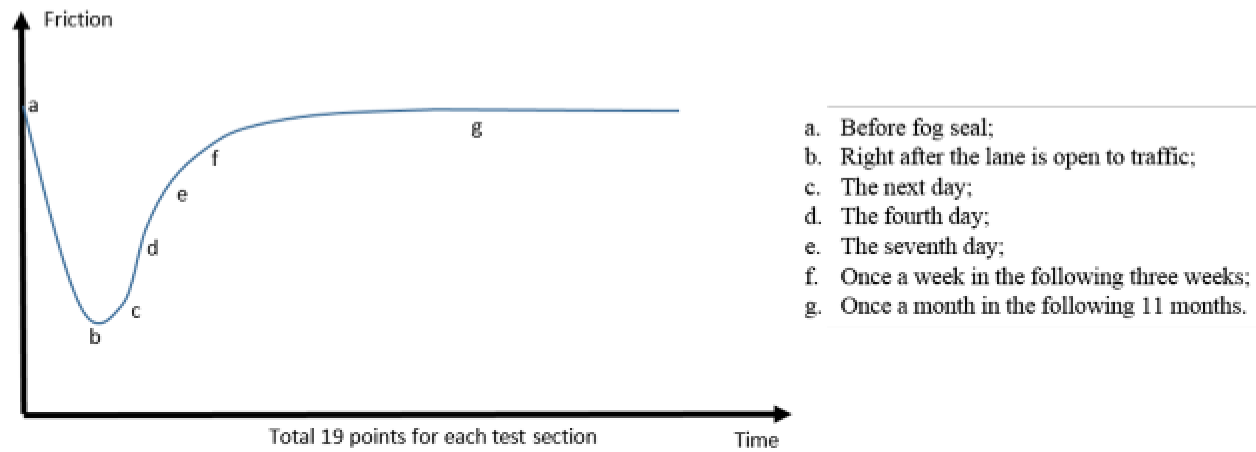


Figure 159. Graph. Expected Short-term Friction Behavior

To evaluate the short-term friction behavior of fog seal test sections, high-frequent (e.g. daily) site-specific friction tests were conducted on Site 6 (in Lane 3 on the outside lane with more truck traffic, ADT 9,685) and Site 8 (in Lane 1 on the inside lane with lower traffic volume and truck percentage, ADT 7,981). Both sites have similar raveling conditions (Very Light, with a 0% to 5% aggregate loss), while different traffic conditions (Site 6’s ADT is 9,685; Site 8’s ADT is 7,981). Detailed information on these two sites is shown in Figure 160.

Figure 160. Graph. Two Sites Selected for Short-term Analysis of Friction

Analysis of Short-Term Behavior on Site 6 (Lane 3, outside lane)

For Site 6 (the outside lane), fog seal was applied on December 2, 2014. The average SN on the left and right wheel path on the collection date are plotted in Figure 161; SNs of the non-fog sealed control section are represented by blue dots, while SNs of the fog-sealed test section are represented by yellow dots. The detailed data analyses are summarized below:

1) The SN before fog seal is 35.6 (measured on 11/13/2014). The SN drops 44% to a low value of 19.9 (measured on December 3, 2014) one day after the fog seal application on December 2, 2014.

2) Two days after fog seal, the SN recovered to above 30. It increased 72% to reach 34.2 on December 4, 2014.

3) Five days after fog seal, the SN increased to above 35. It increased 80% to reach 35.8 on December 7, 2014.

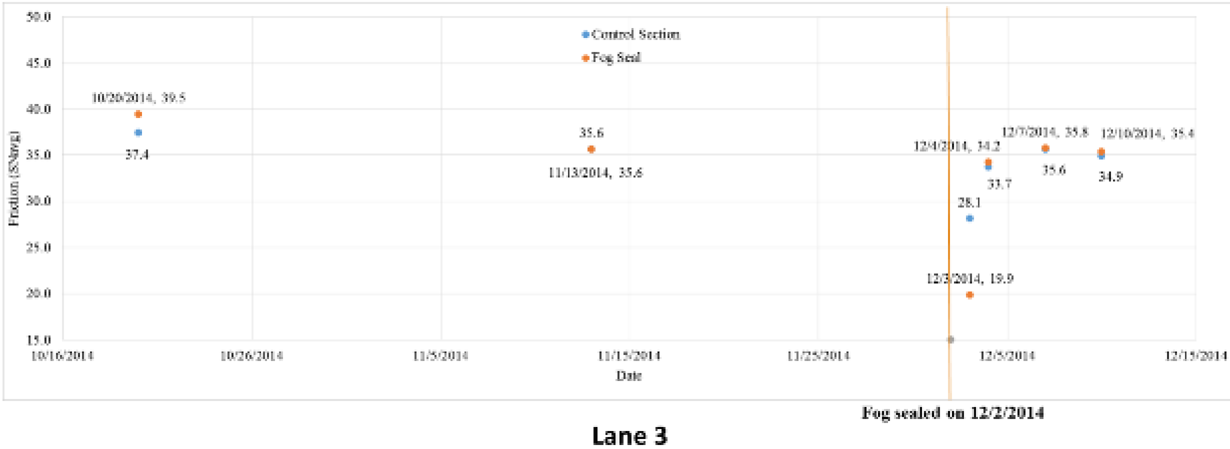


Figure 161. Graph. Short-term Friction Behavior on Site 6.

Analysis of Short-Term Behavior on Site 8 (lane 1, inside lane)

For Site 8 (the inside lane), fog seal was applied on November 12, 2014. The average SN on the left and right wheel path on the collection date are plotted in Figure 162; SNs of the non-fog sealed control section are represented by blue dots, while SNs of fog-sealed test section are represented by yellow dots. The detailed data analyses are summarized below:

1) The SN before fog seal is 46.7 (measured on November 10, 2014). On the day after fog seal, the SN dropped 45% to its lowest value of 25.8 (measured on November 12, 2014).

2) Four days after fog seal, the SN recovered to above 30.0. It increased 28% to reach 32.9 on November 16, 2014.

3) Seven days after fog seal, the SN increased to above 35.0. It increased 42% to reach 36.7 on November 19, 2014.

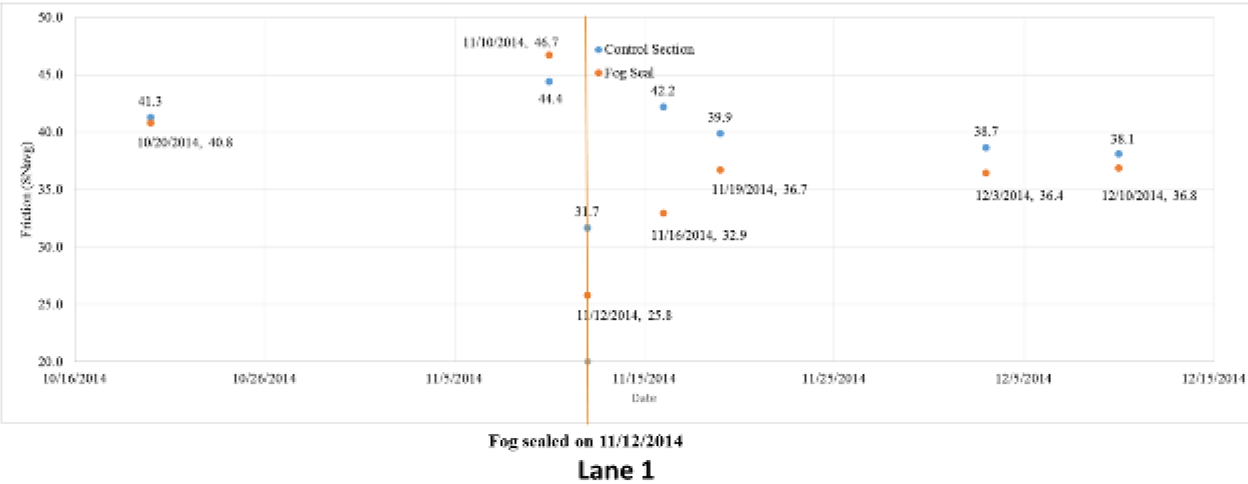


Figure 162. Graph. Short-term friction behavior on Site 8.

Comparison of Short-Term Behavior on Sites 8 and 6

Figure 163 shows the comparison of the short-term behaviors from Sites 6 and 8. Though the friction on Site 6 drops to a lower value than the one on Site 8, it recovers faster, which might be caused by the higher traffic and truck volumes on Site 6 (in the outside lane).

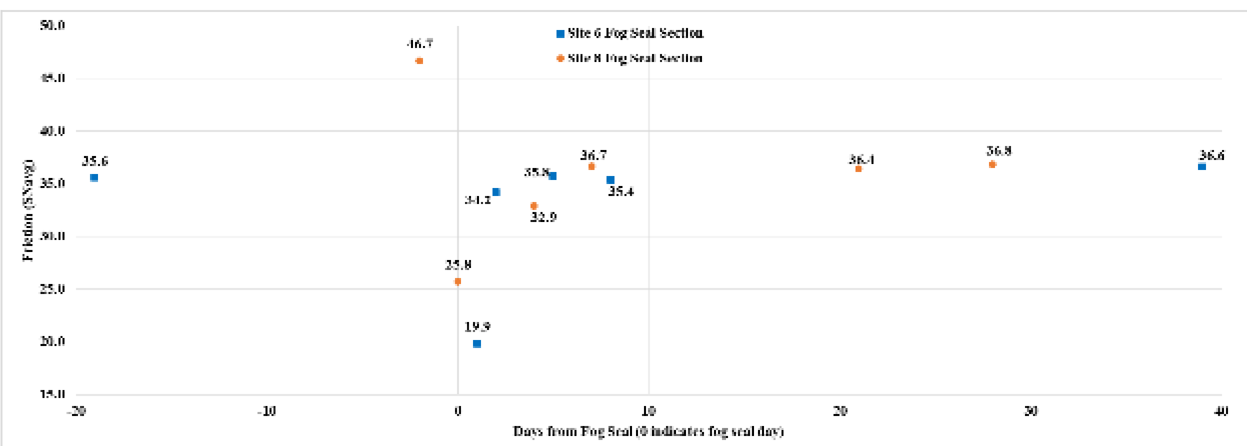


Figure 163. Graph. Comparison of short-term frictions behaviors on Sites 6 and 8.

Analysis of Long-term Friction Behavior

In addition to the short-term friction behavior analysis, a preliminary study of long-term friction analysis was conducted. After high-frequent (e.g. daily) site-based friction tests in the first week after fog seal application, low-frequent (e.g. weekly, monthly, and quarterly) site-based friction tests were conducted weekly for the first month and then monthly for the first year in 2015; they have been conducted quarterly since 2016.

Analysis of Long-Term Behavior on Site 6 (lane 3, outside lane)

On Site 6, the SN increased in a rough linear trend (1.671 per month) in the first 3 months after fog seal application (see Figure 164). The reason could be that the binder materials were removed by traffic. After 3 months of fog seal application, the SN decreased with a roughly linear trend (-0.6 per month) after three months, which could have been caused by the polishing process of aggregate particles.

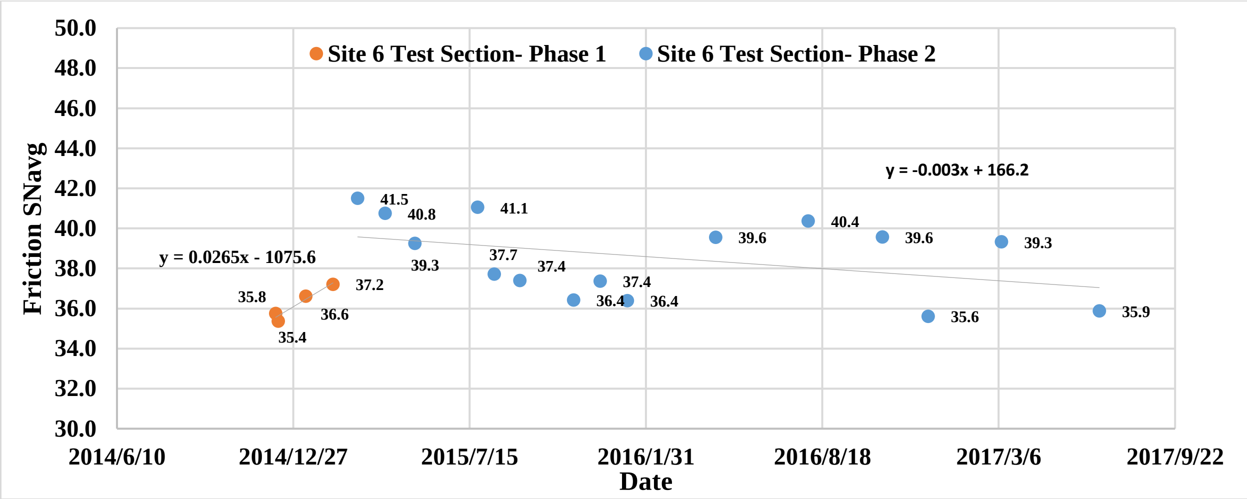


Figure 164. Graph. Long-term Friction Behavior on Site 6.

Analysis of Long-Term Behavior on Site 8 (lane 1, inside lane)

The SN increased slowly in a rough linear trend (1.026 per month) in the first 7 months after fog seal (see Figure 165). This could be because the binder materials were removed by traffic. After 7 months of fog seal application, the SN decreased with a rough linear trend (-1.194 per month). This could have been caused by the polishing process of aggregate particles.

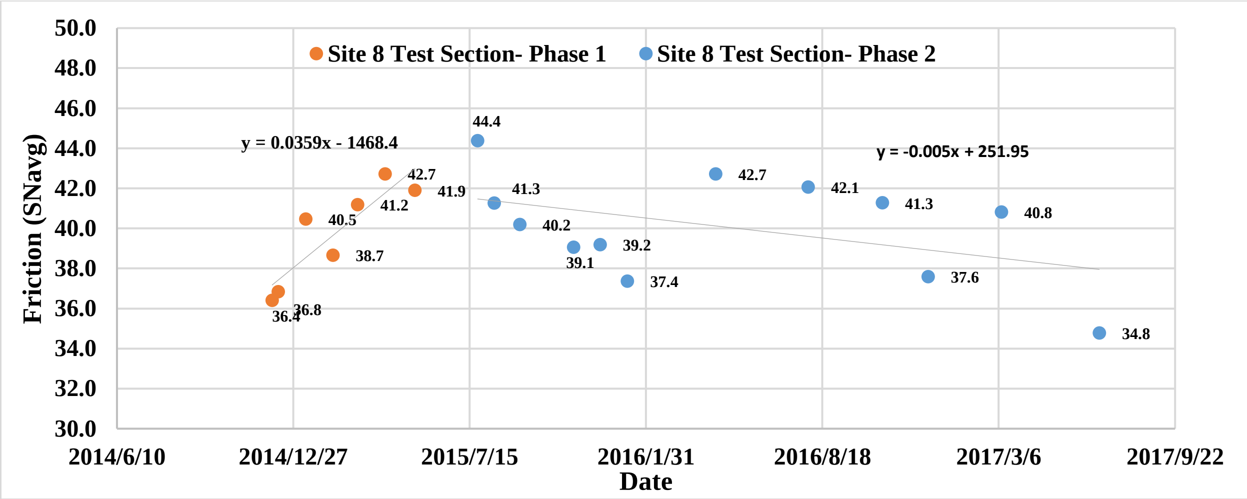


Figure 165. Graph. Long-term behavior of friction on Site 8.

Comparison of Long-Term Behavior on Sites 6 and 8

Sites 6 and 8 have similar long-term friction behaviors; the trend shows a steady increase in the first several months after fog seal application, followed by a decrease in the following months. As mentioned above, the increase of friction could be due to the asphalt emulsion materials being gradually removed by passing vehicles. On the other hand, the decreasing trend might be due to the surface aggregates being polished by tires.

Non-Fog Sealed Control Section vs. Fog Sealed Test Section

To evaluate whether fog seal affects the behavior of surface friction in the long-term, site-based friction tests were conducted to compare the frictions between non-fog sealed control sections and fog sealed test sections for Sites 1, 2, 4, and 6. All 4 sites are located on the outside lane (with similar traffic conditions); however, the pavement conditions varied. Site 1 had severe raveling (15% to 20%); Site 2 had medium raveling (10% to 15%); Site 4 had light raveling (5%

to 10%); and Site 6 had very light raveling (0% to 5%). Locations of the 4 selected sites are shown in Figure 166.

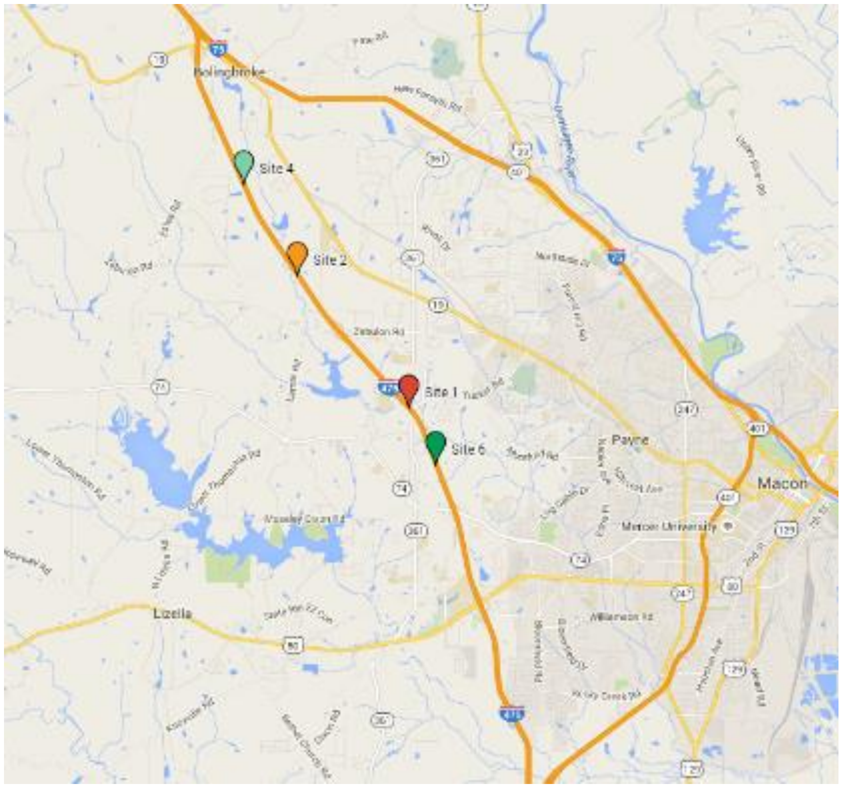
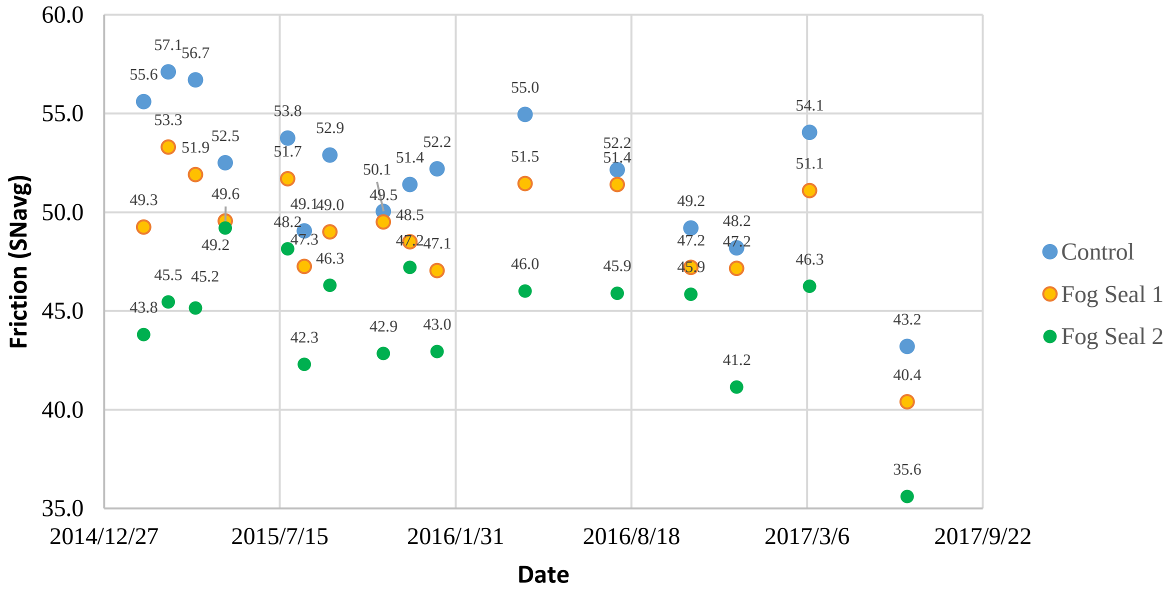
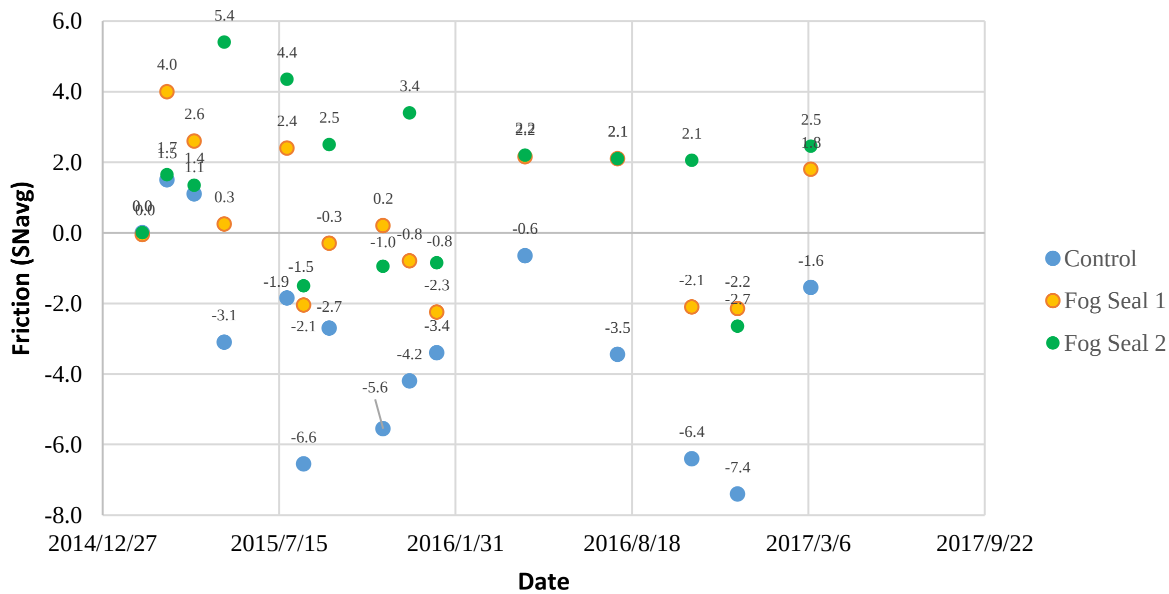


Figure 166. Graph. Locations of 4 Sites Selected for Comparison of Friction Behavior between Control and Fog Seal Sections

Figure 167, Figure 168, Figure 169, and Figure 170 compare friction in the control section to the friction in the two 500-ft test sections on Sites 1, 2, 4, and 6, respectively. The top figure shows the absolute SNs; the bottom figure shows the relative value using the first measurement as the base.

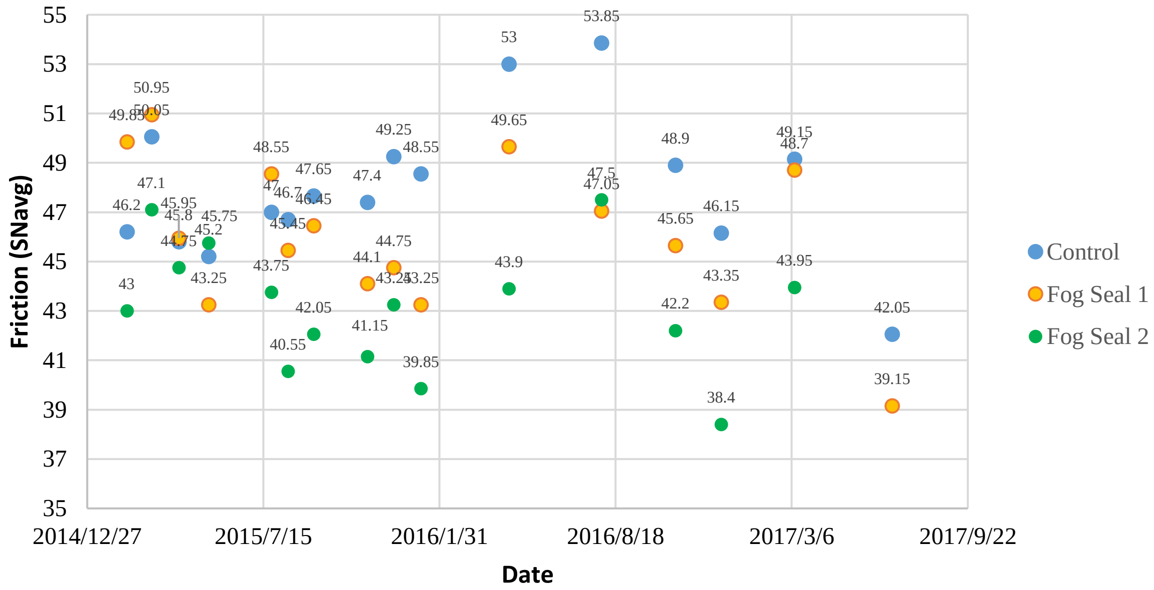


A. Site 1 – Absolute SN

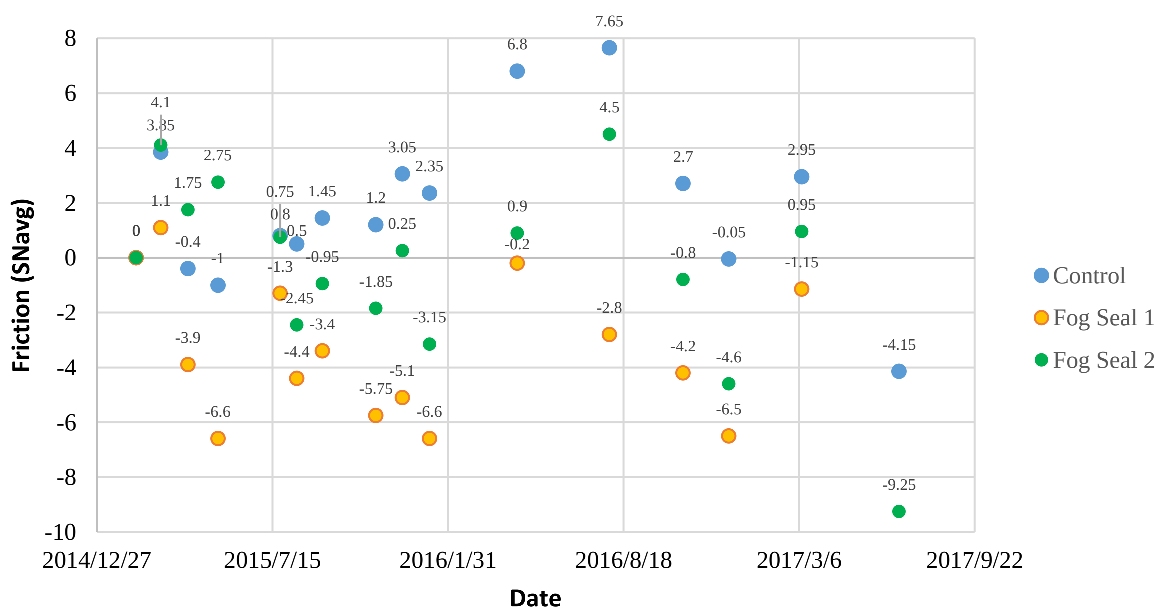


B. Site 1 – Relative SN

Figure 167. Graph. Comparison of Long-term Behavior between Control and Fog Seal Sections for Site 1 (top: absolute SN, bottom: relative SN)

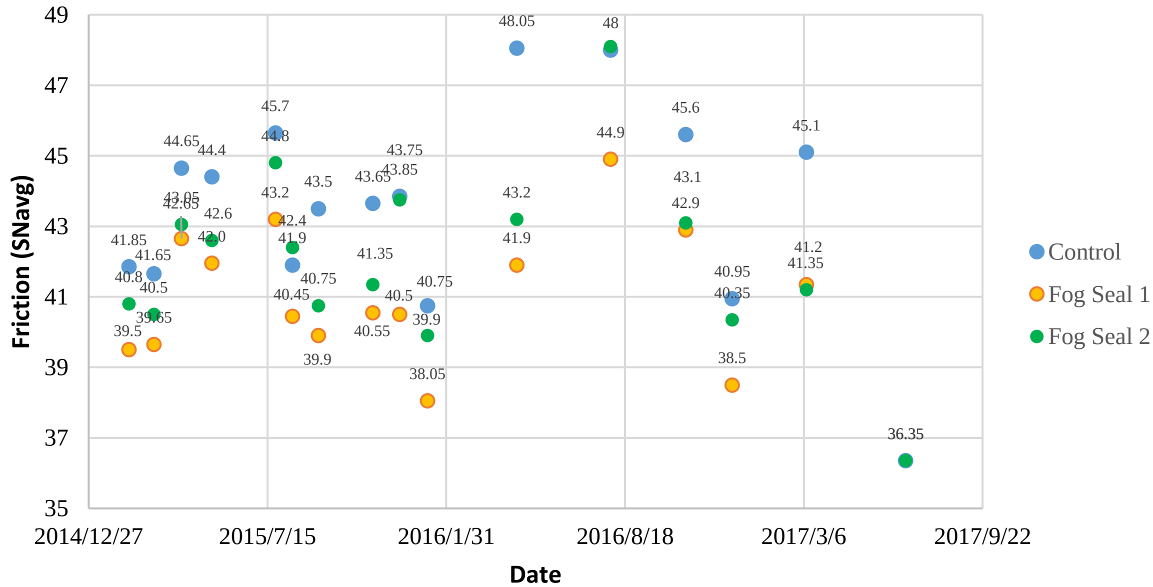


A. Site 2 – Absolute SN

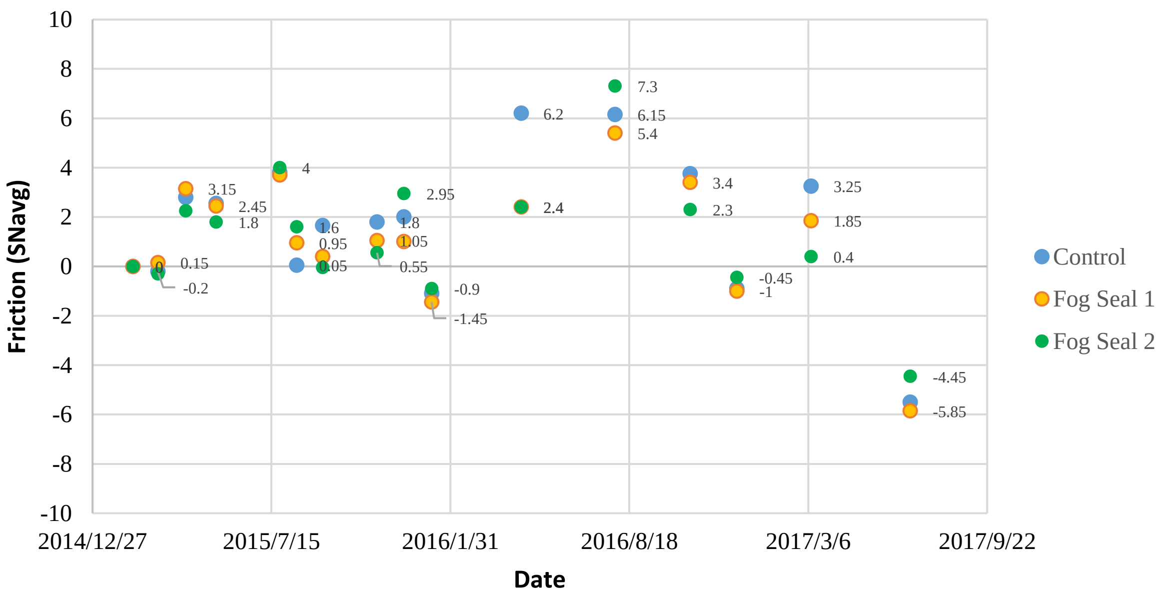


B. Site 2 – Relative SN

Figure 168. Graph. Comparison of Long-term Behavior between Control and Fog Seal Sections for Site 2 (top: absolute SN, bottom: relative SN)

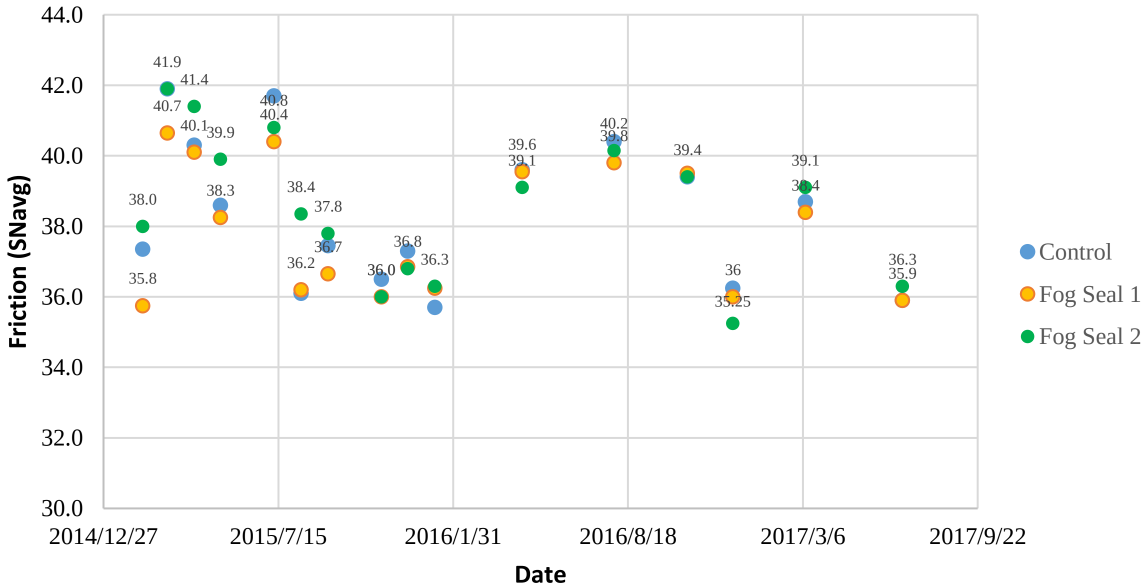


A. Site 4 – Absolute SN

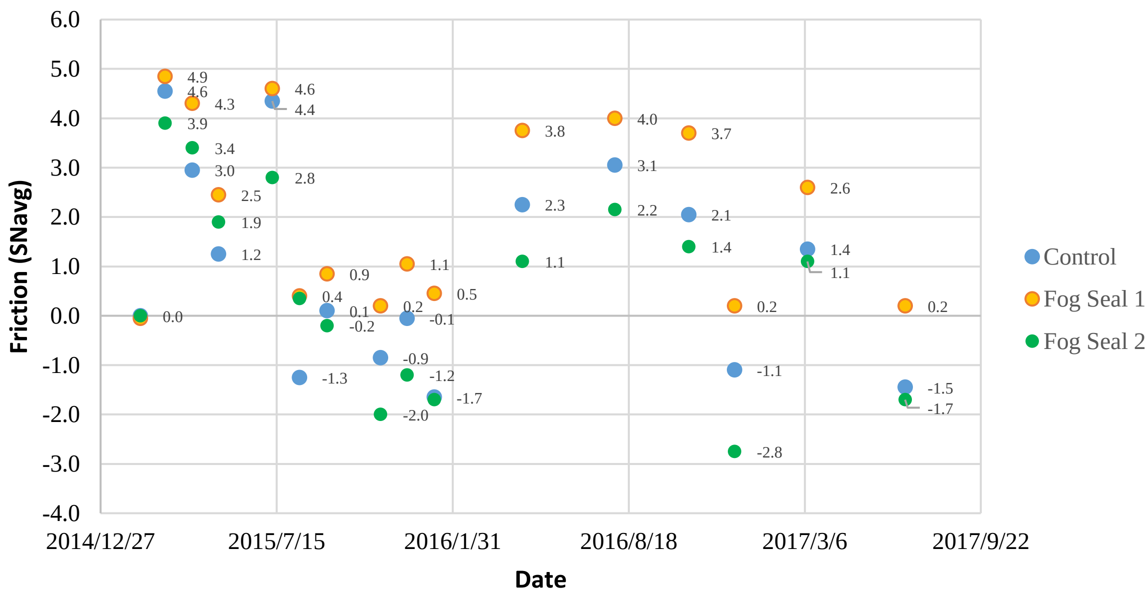


B. Site 4 – Relative SN

Figure 169. Graph. Comparison of Long-term Behavior between Control and Fog Seal Sections for Site 4 (top: absolute SN, bottom: relative SN)



A. Site 6 – Absolute SN



B. Site 6 – Relative SN

Figure 170. Graph. Comparison of Long-term Behavior between Control and Fog Seal Sections for Site 6 (top: absolute SN, bottom: relative SN)

Based on the comparison, the following summarizes the observations:

1) In 12 months after fog seal, the behavior of increasing-then-decreasing was found among SNs collected on 4 selected sites. This observation confirms the previous findings in the long-term analysis of Sites 6 and 8. The increasing trend could be the result of binders having been removed by traffic. The decreasing trend could be the result of the polishing process of aggregate particles. Yet another possible reason is that the surface friction was affected by seasonal changes, e.g. air temperature (Bianchini, 2011).

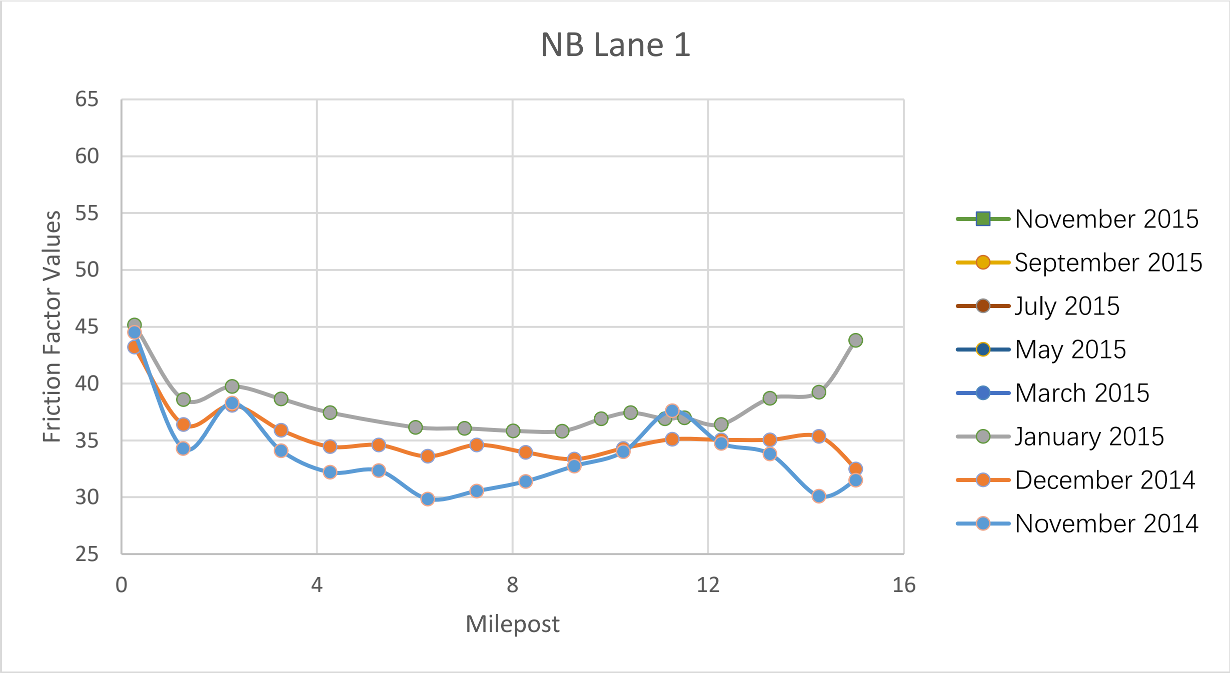
2) Although variation is observed for SN obtained at different dates, on the same collection date a higher SN can be generally found in the control section than in the fog seal test sections. This could indicate that the fog seal has a long-term (>=12 months) effect on surface friction.

3) High SNs were observed for the sites with severe aggregate loss. This could indicate that there is a correlation between surface friction and aggregate loss.

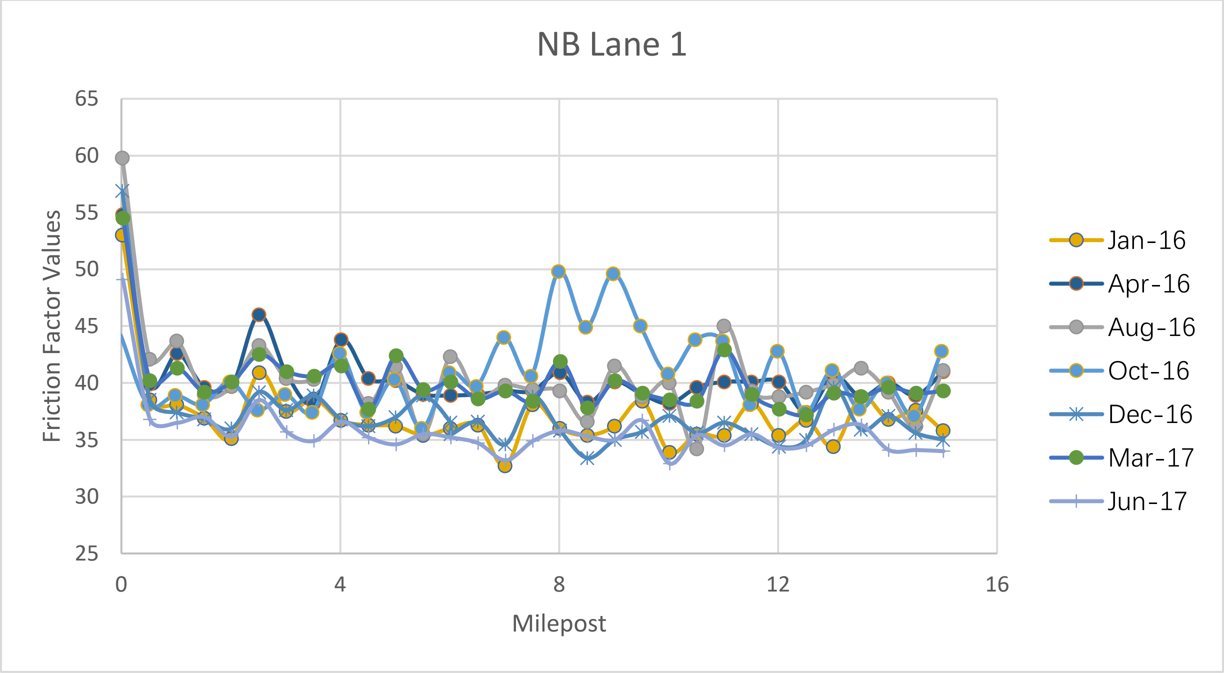
4) The difference between the SNs of control and test sections was larger for the sites with severe aggregate loss conditions.

Evaluation of Seasonal Impact on Friction

Based on the previous analysis, seasonal changes, such as air temperature, are likely to affect the surface friction. To further confirm the correlation between surface friction and seasonal changes, friction tests were conducted on the whole length of I-475. In each lane of I-475 (3 northbound lanes and 3 southbound lanes), the SN in each mile was extracted and plotted, as shown in Figure 171 to Figure 176.

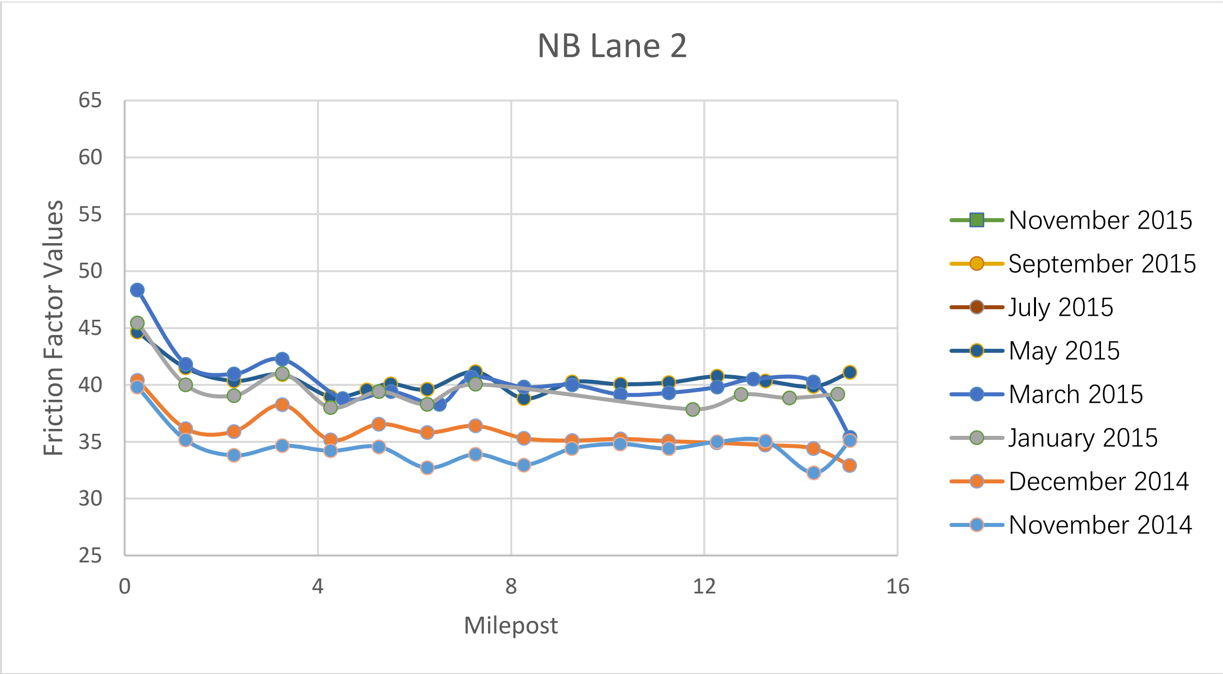


A. Data from November 2014 to November 2015

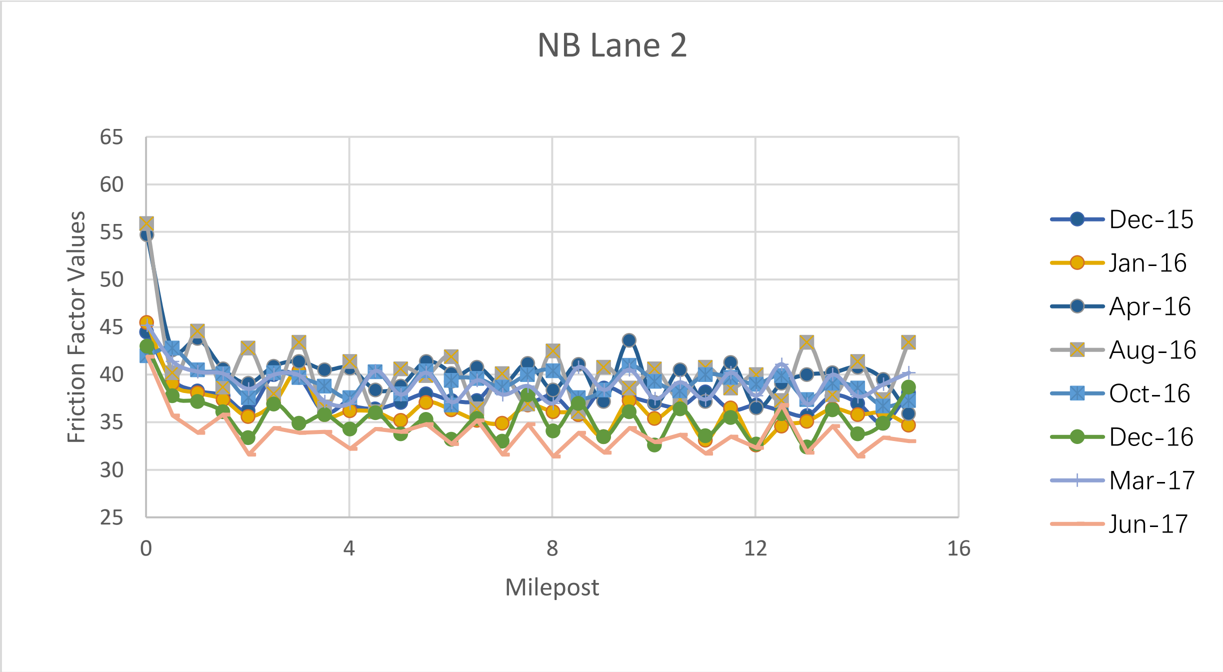


B. Data from January 2016 to June 2017

Figure 171. Graph. Comparison of Friction Measured at Different Dates at NB Lane 1

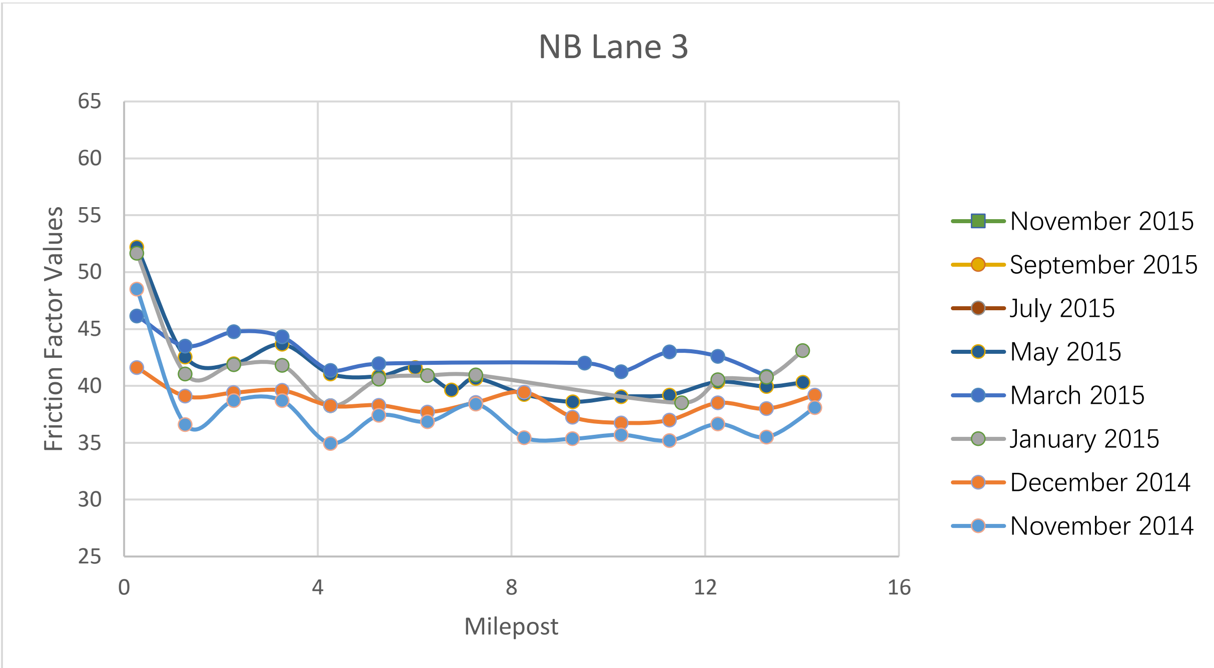


A. Data from November 2014 to November 2015

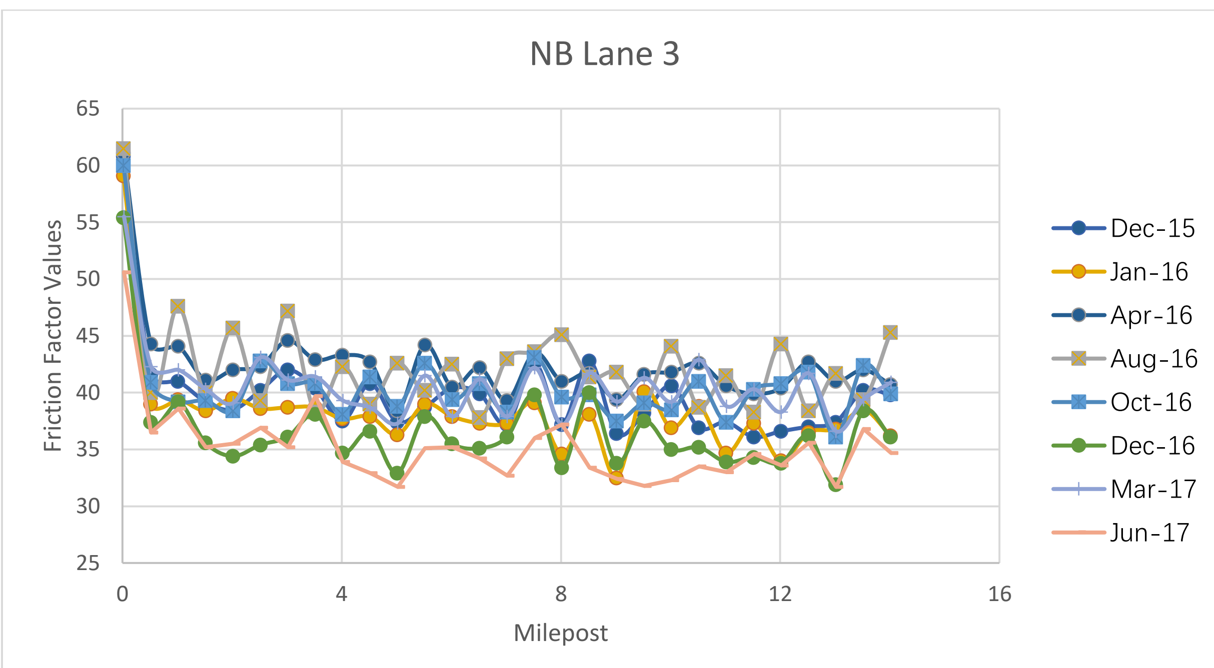


B. Data from December 2015 to June 2017

Figure 172. Graph. Comparison of Friction Measured at Different Dates at NB Lane 2

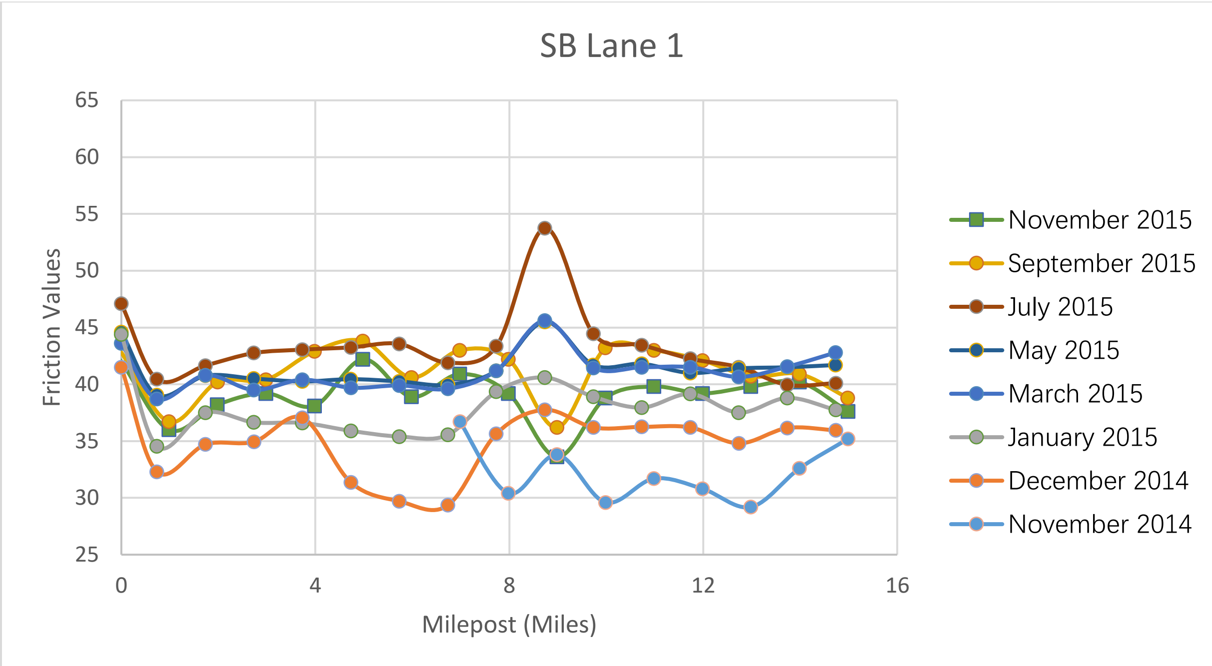


A. Data from November 2014 to November 2015

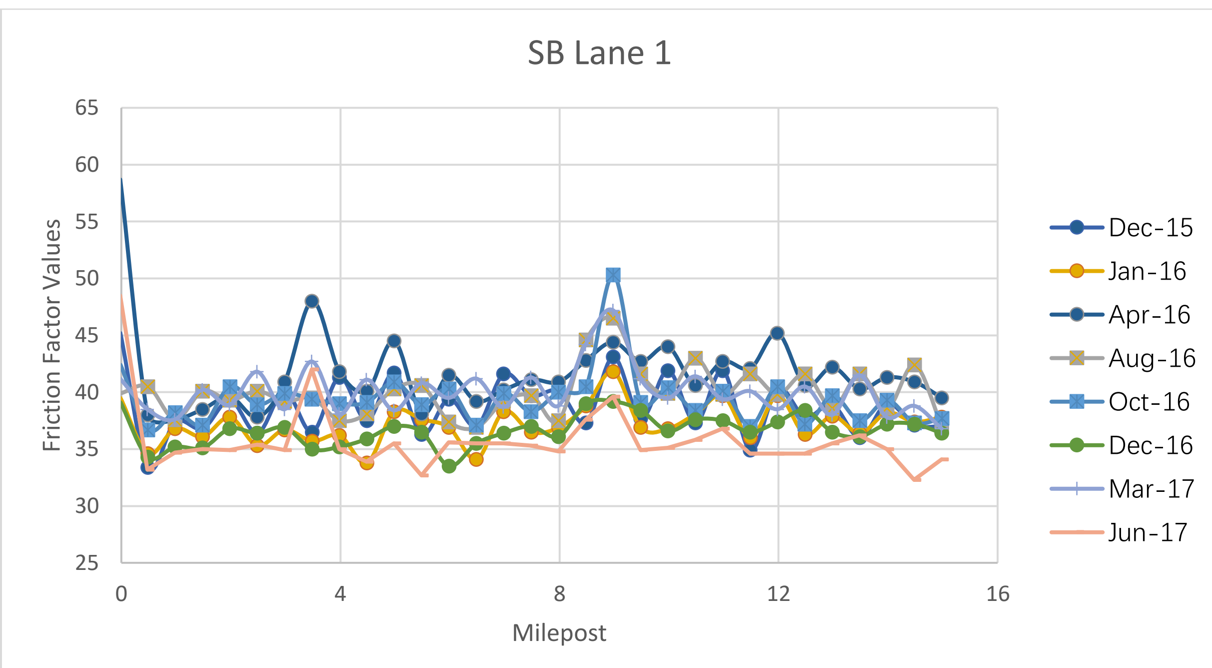


B. Data from December 2015 to June 2017

Figure 173. Graph. Comparison of Friction Measured at Different Dates at NB Lane 3

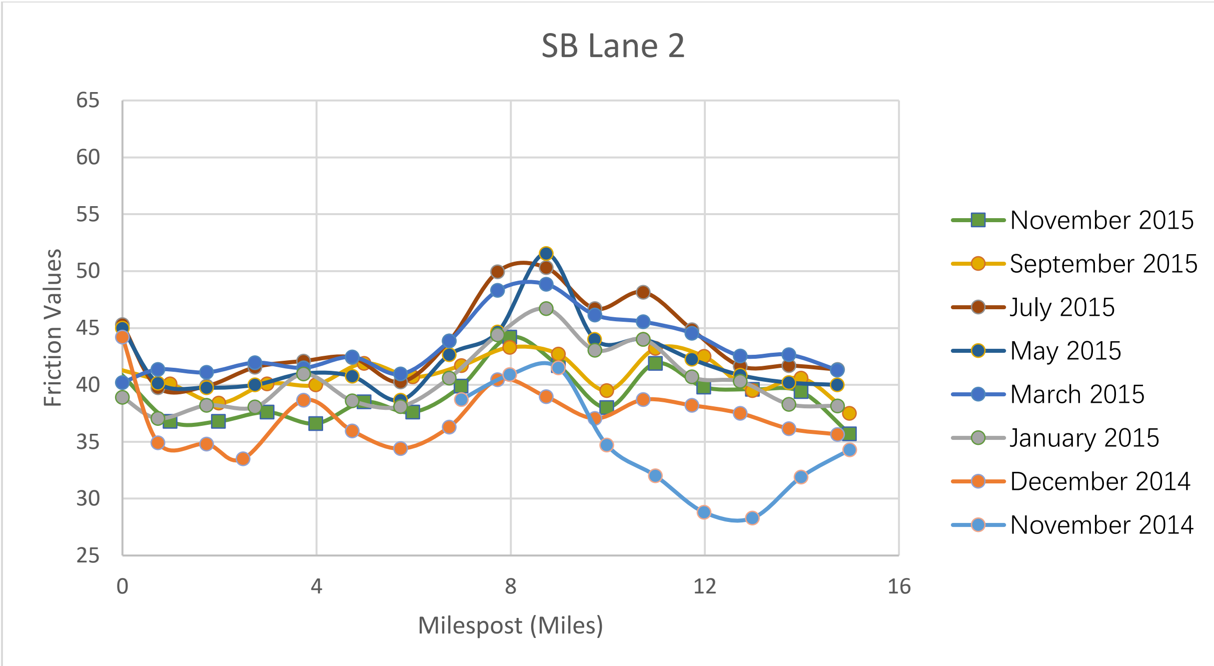


A. Data from November 2014 to November 2015

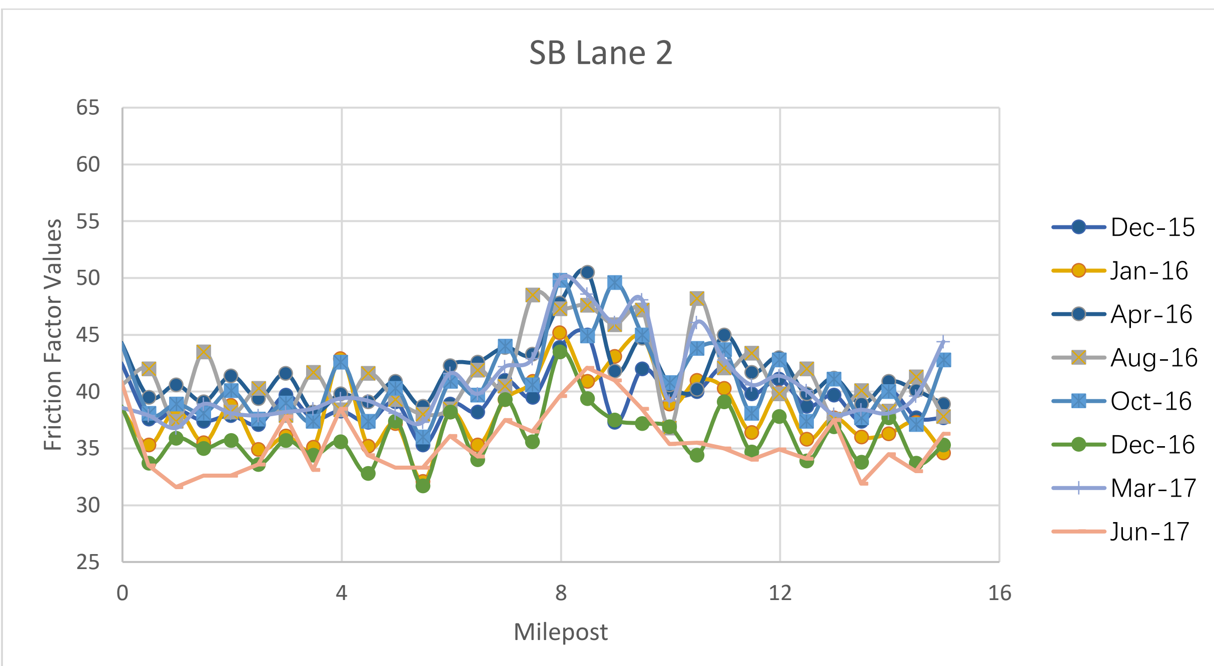


B. Data from December 2015 to June 2017

Figure 174. Graph. Comparison of Friction Measured at Different Dates at SB Lane 1

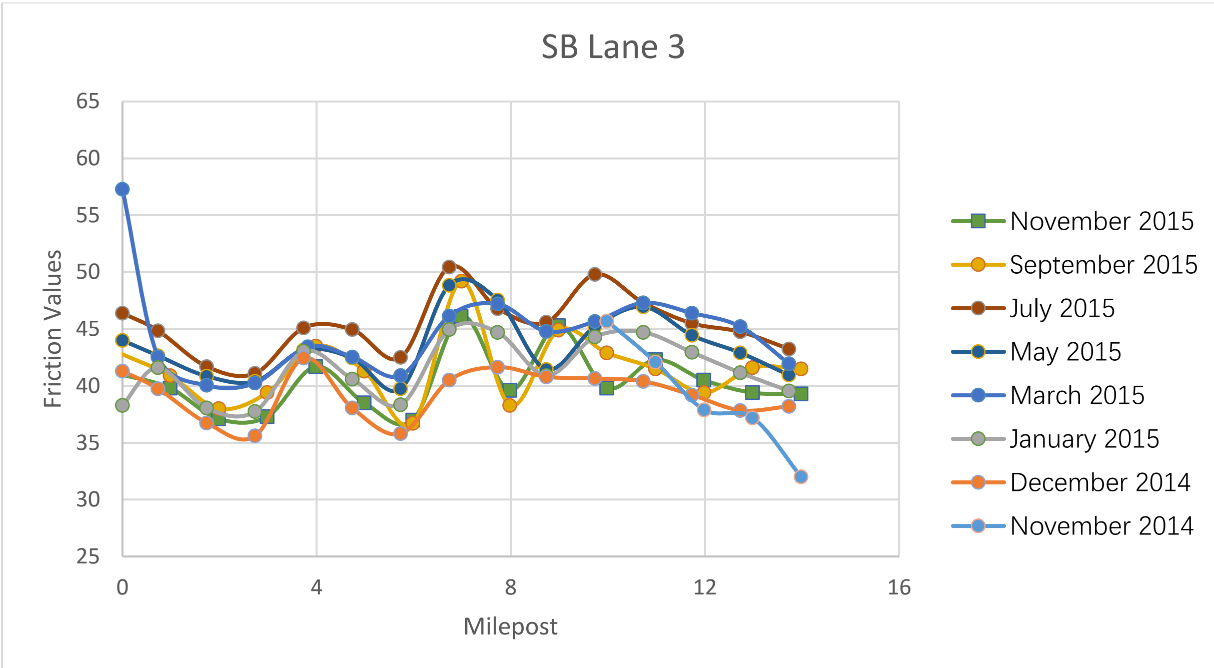


A. Data from November 2014 to November 2015

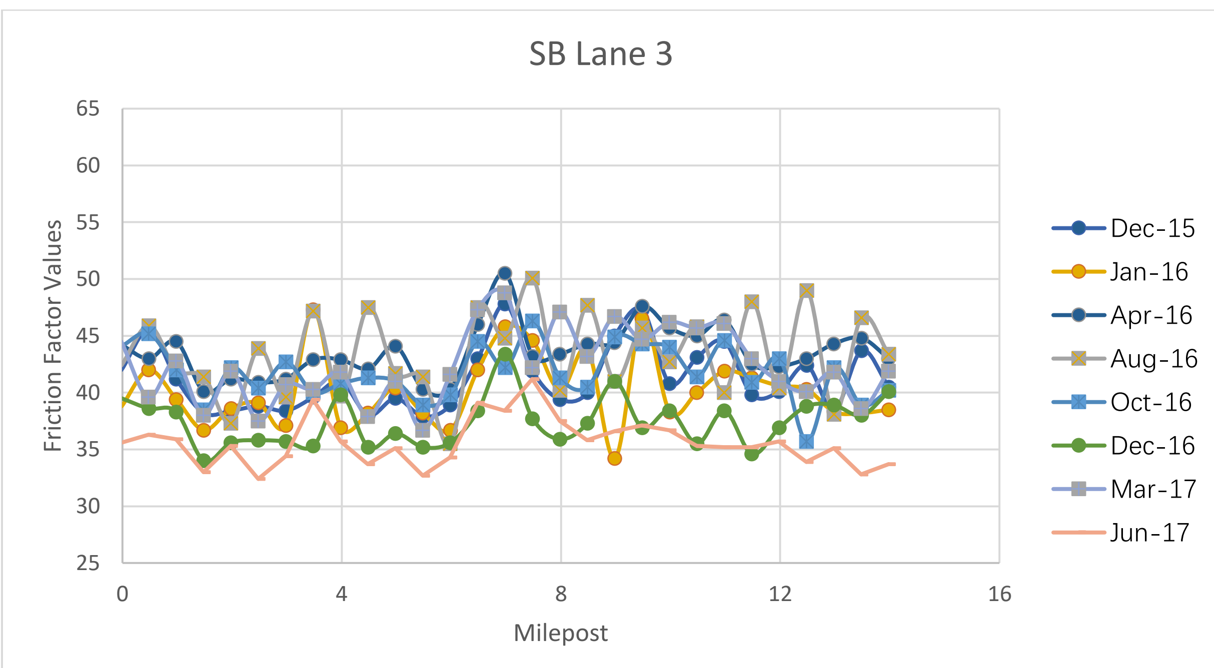


B. Data from December 2015 to June 2017

Figure 175. Graph. Comparison of Friction Measured at Different Dates at SB Lane 2



A. Data from November 2014 to November 2015



B. Data from December 2015 to June 2017

Figure 176. Graph. Comparison of Friction Measured at Different Dates at SB Lane 3

From the above figures, though there exists variation, a general trend can be observed that shows the SN increases in summer and decreases in winter for the same mile. Using Figure 171 as an example, the SN in each mile increased as the collection date went from November 2014 to July 2014. The SN reached a maximum along the entire lane in July 2014. Then, the SN kept decreasing until the data was collected in November 2015.

Similar trends can be found in other lanes, as well. In summary, the surface friction of the entire length of I-475 generally increased during the several months after fog seal. The increase ended between May 2015 and July 2015 for all northbound and southbound lanes. Then, SN decreased.

Evaluation of Traffic Impact on Friction

Apart from seasonal changes, another factor that could affect surface friction is traffic. All the I-475 friction tests in different lanes were conducted to evaluate the traffic impact on friction. The assumption was that the traffic condition varied in Lanes 1, 2 and 3. Lane 1 had light traffic volume and a low truck percentage. Lane 3 had heavy traffic volume and a high truck percentage. Lane 2 was in between. Figure 177 and Figure 178 compare the average SNs in all 6 in northbound and southbound lanes.

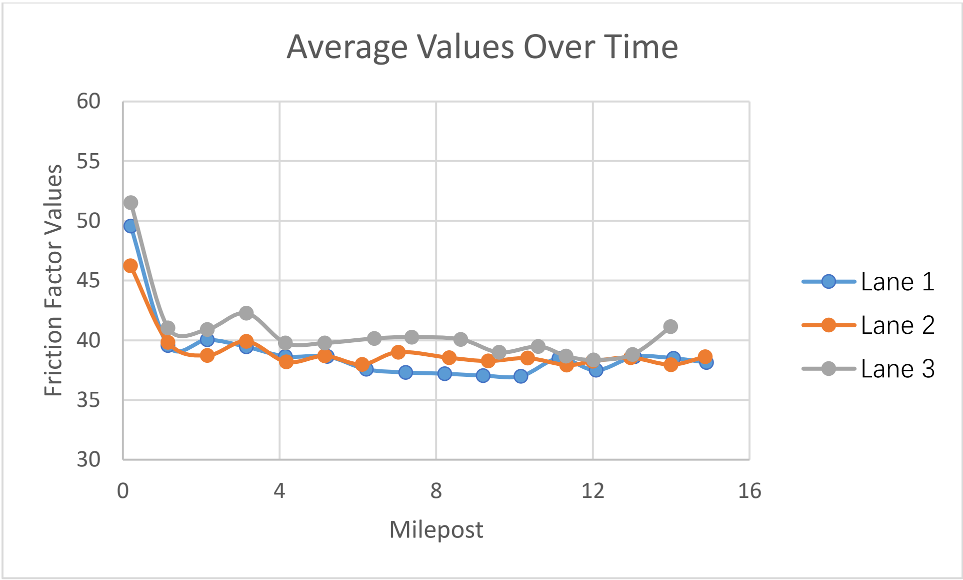


Figure 177. Graph. Comparison of Average Friction Measured at Different Lanes of Northbound

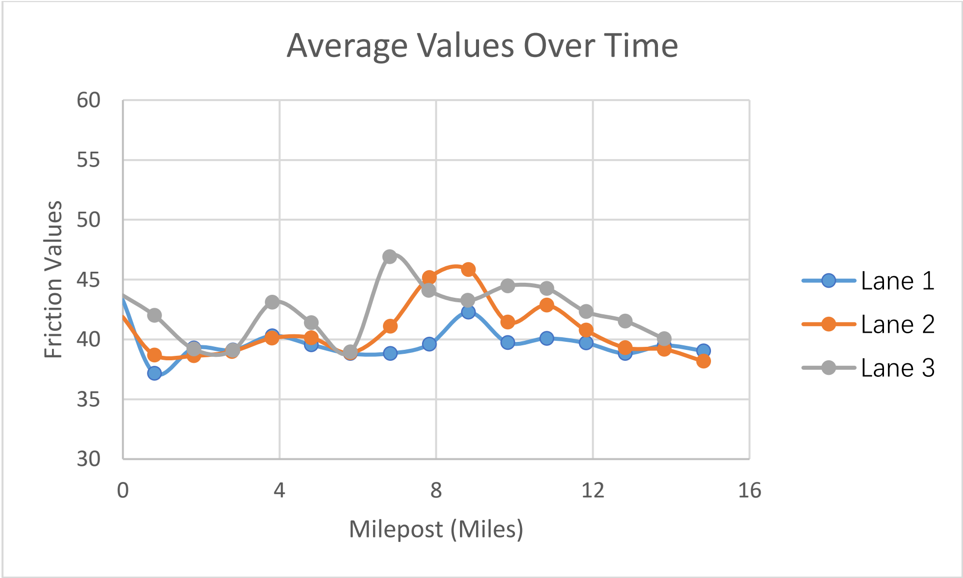


Figure 178. Graph. Comparison of Average Friction Measured at Different Lanes of Southbound

From Figure 177 and Figure 178, correlation between lane numbers and SN can be observed. Except for some distinct locations, the SNs in Lane 1 are generally less than the ones in Lane 2 and Lane 3. The SNs in Lane 2 are closer to the ones in Lane 3. However, Lane 3 generally has the highest SNs. Thus, traffic conditions could be a significant factor affecting OGFC pavement raveling conditions.

Summary

Through the analysis of friction data collected on I-475 before and after fog seal application, the following are some major findings:

1) A pavement's surface friction decreases right after fog seal application. Based on measurement results, the SN decreases about 45% immediately after fog seal application.

2) The SN recovers to 30 - 35 in 2-4 days, and to 35 or above in 5-7 days after fog seal application. Results show the surface friction of pavements with high traffic volume and truck percentage recover faster than the pavements with low-traffic volume.

3) Results show that the friction is higher in summer and lower in winter. In addition, measurements show that a higher friction value in the outside lane with high truck traffic volume when compared to the inside lane, which has a lower friction value; this is likely caused by different losses of aggregates.

SMOOTHNESS TEST RESULTS

Introduction

Pavement surface smoothness is measured by the International Roughness Index (IRI). In the research, other than a friction test, IRI is measured over time to evaluate pavement performance after applying fog seal on interstate highways. The purposes of smoothness tests were to address the following questions:

1) Will fog seal affect the IRI in the short term?
2) Will fog seal affect the IRI in the long term?
3) Are there any factors other than fog seal that could lead to the changes in IRI, e.g. seasonal changes or traffic conditions?

GDOT collected IRI data using the laser profiler shown in Figure 179. The data collection has been performed monthly over the entire 6 lanes of I-475 since September 2014. The obtained IRI measurements can be rated and placed into a series of categories (from very good to poor) according to the FHWA standard, as shown Figure 180. Note that the IRI value here is in inch/mile).

Figure 179. Photo. Laser Profiler for IRI Collection

60 80 100 120 140 160 180 200 220

IRI (in/mi)	<60	60-94	95-119	120-144	145-170	171-194	195-220	>220
FHWA Interstates	V.Good	Good	Fair	Mediocre		Poor		

Figure 180. Graph. FHWA IRI Interstate Ride Quality Ratings

Three IRI readings were collected at each location each time:

1) IRI 1: measurement in left wheel path, mm/km

2) IRI 2: measurement in right wheel path, mm/km

3) Hcs IRI: half-car simulation IRI, mm/km

In the following study, the Hcs IRI is used as the smoothness indicator of pavement surface; the unit is mm/km (1 mm/km = 0.06336 inch/mile).

Short-term IRI Analysis

To evaluate the impact of fog seal on IRI in the short term, we compared the IRI measurements in the same lane right before and after fog seal. To cover various traffic and pavement raveling conditions, IRI readings from the entire 6 lanes of I-475 were extracted and studied. The results are plotted in Figure 181 to Figure 186.

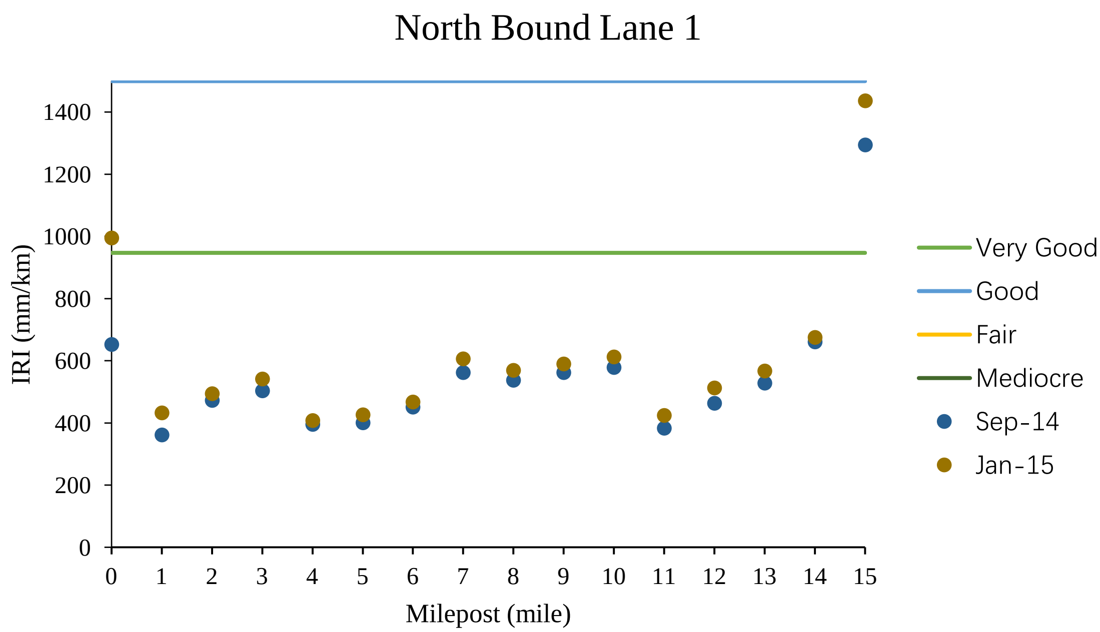


Figure 181. Graph. IRI readings of NB Lane 1 before and after Fog Seal

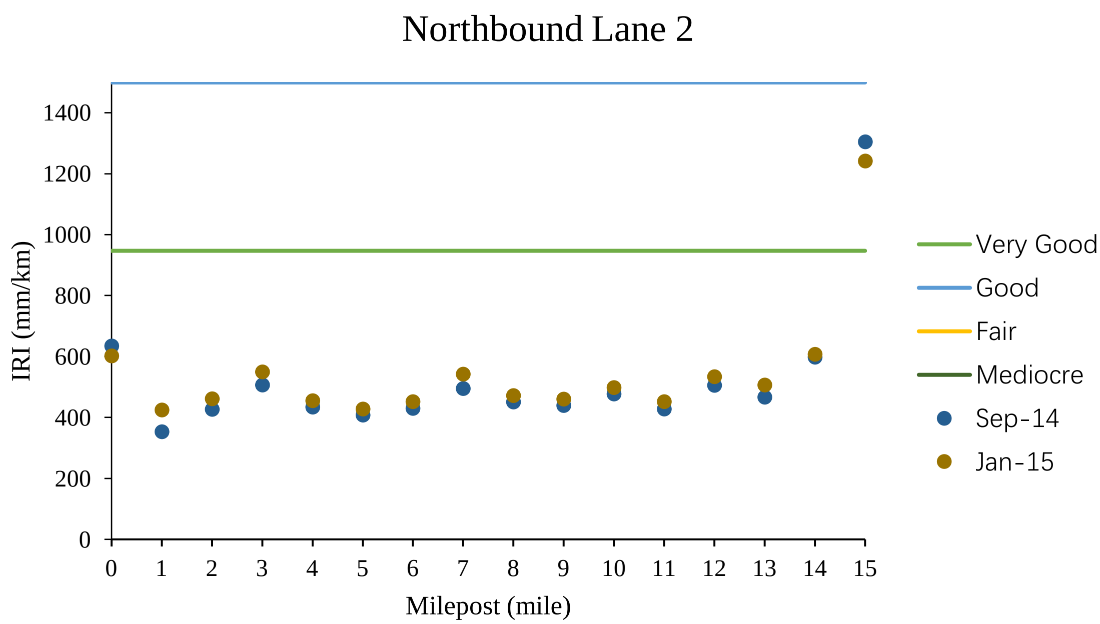


Figure 182. Graph. IRI readings of NB Lane 2 before and after Fog Seal

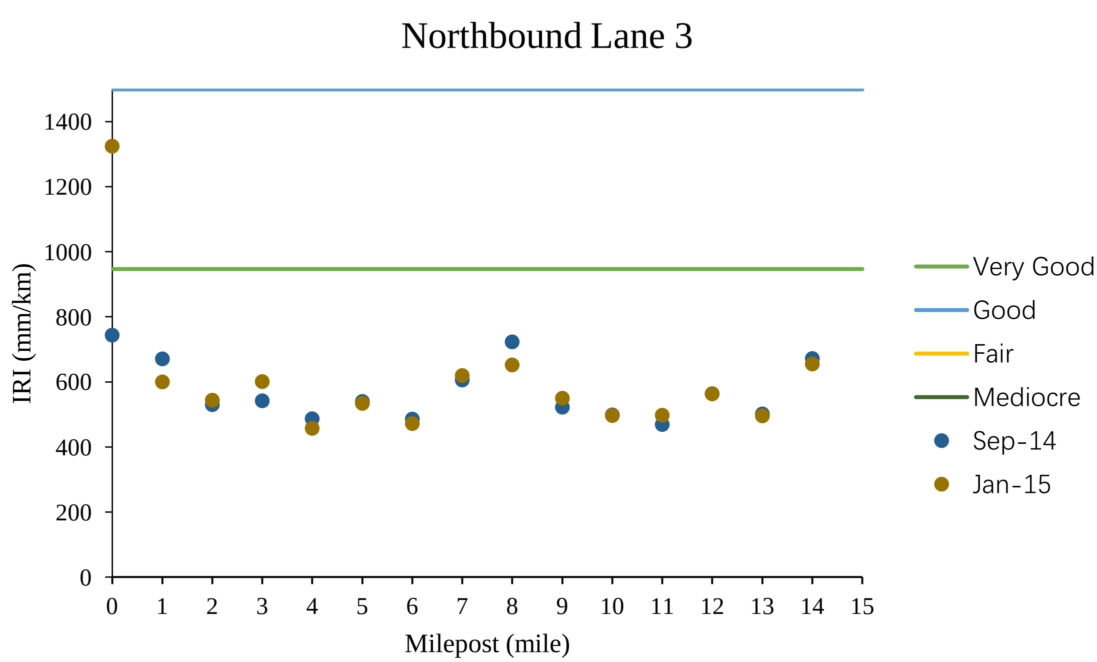


Figure 183. Graph. IRI readings of NB Lane 3 before and after Fog Seal

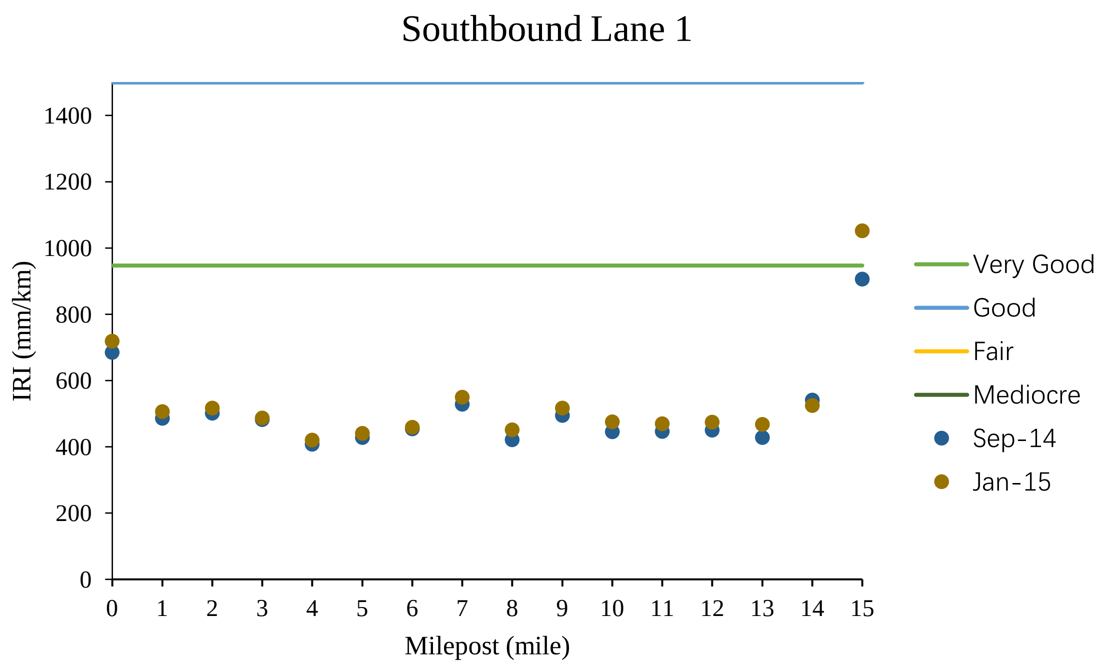


Figure 184. Graph. IRI readings of SB Lane 1 before and after Fog Seal

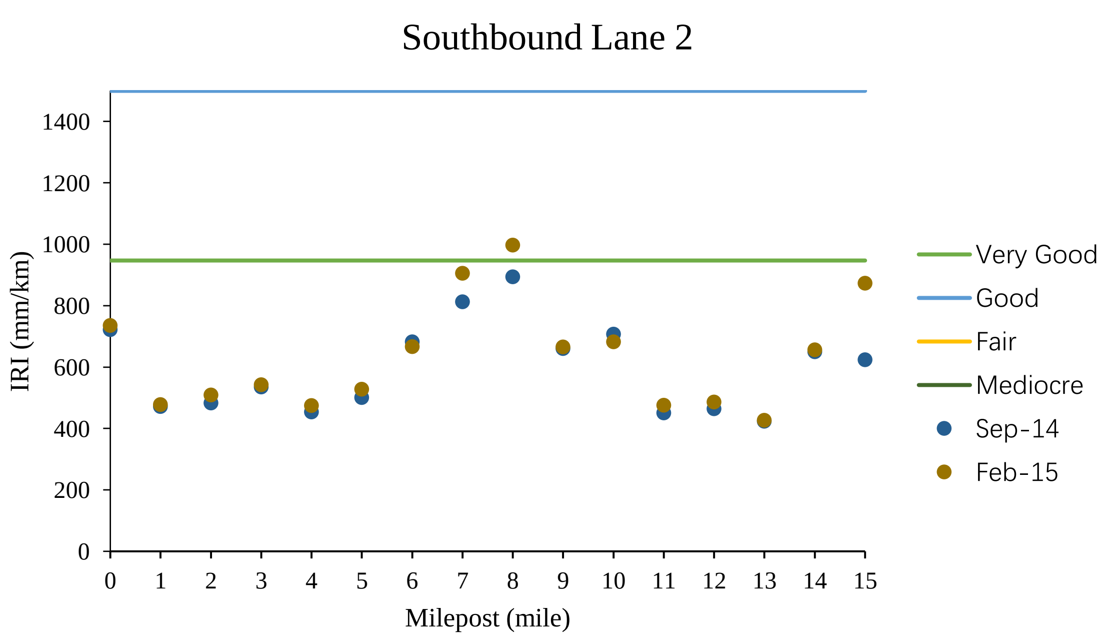


Figure 185. Graph. IRI readings of SB Lane 2 before and after Fog Seal

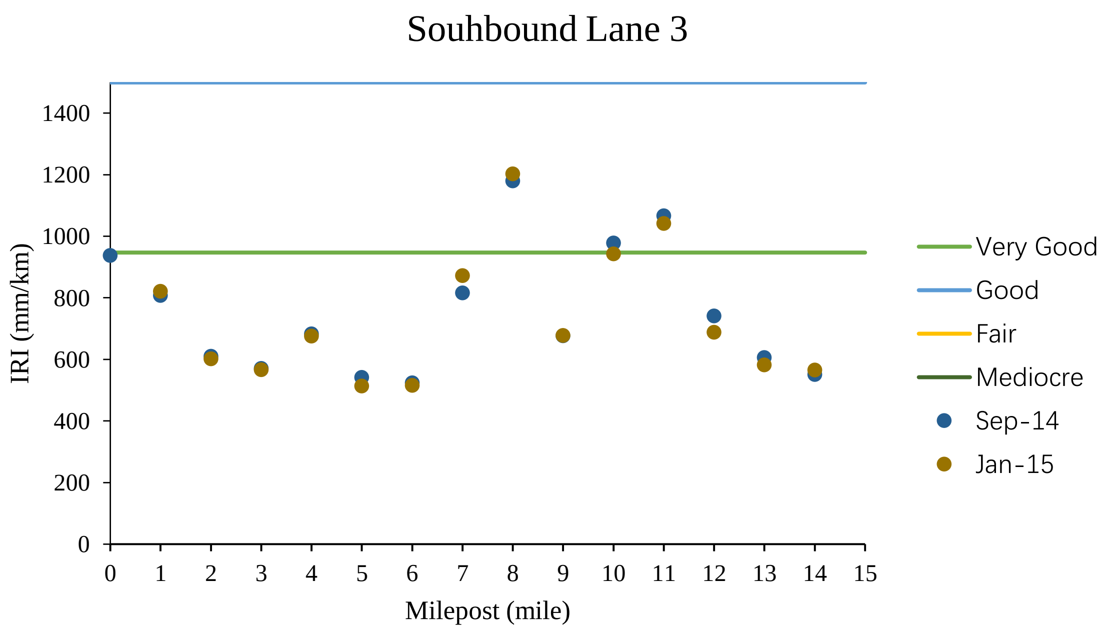


Figure 186. Graph. IRI readings of SB Lane 3 before and after Fog Seal

From Figure 181 to Figure 186, it can be seen that the IRI measurements after fog seal application are slightly higher than those before fog seal application. However, this difference is very insignificant if data variation is considered.

Long-term IRI Analysis

To evaluate the impact of fog seal on IRI in the long term, we can study the IRI measurements collected over an extended period of time. Figure 187 to Figure 192 show the IRI change along each lane over time.

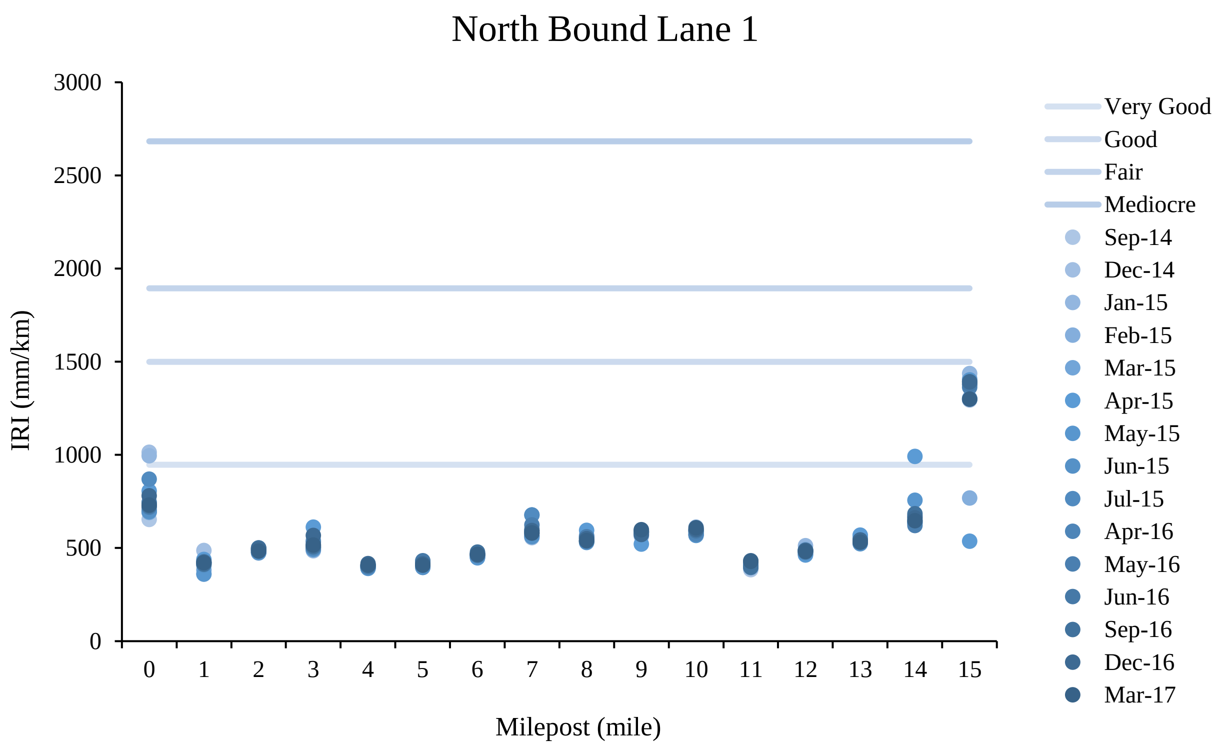


Figure 187. Graph. IRI Readings of NB Lane 1

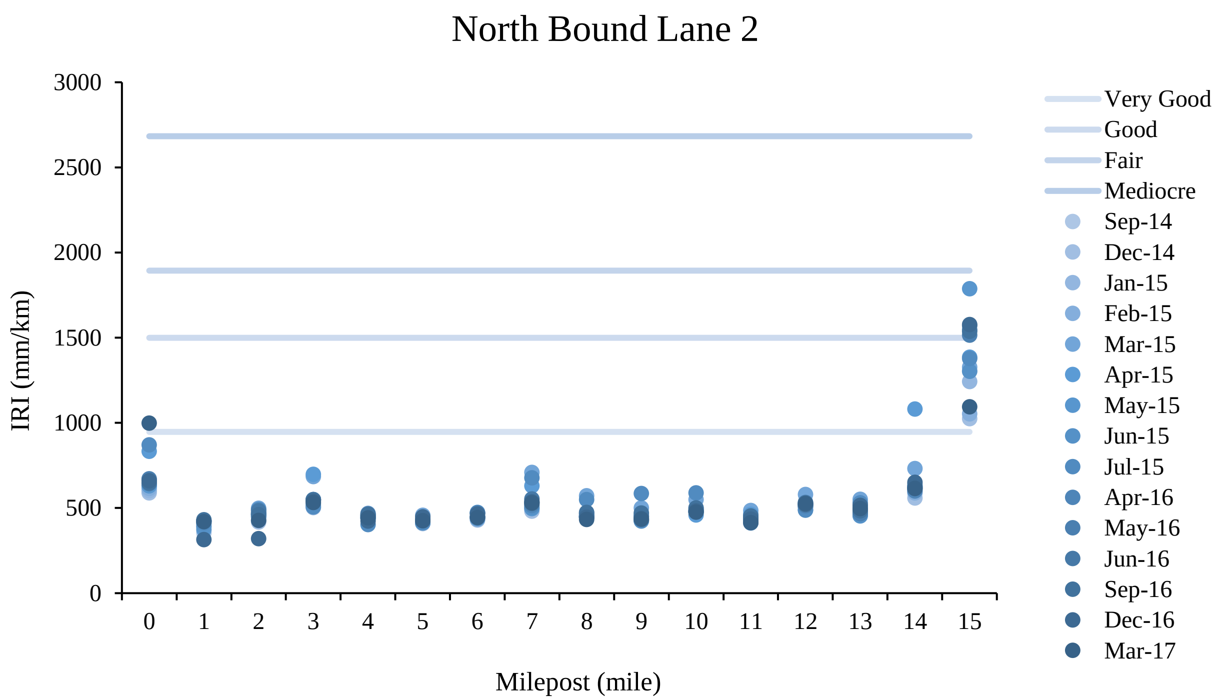


Figure 188. Graph. IRI Readings of NB Lane 2

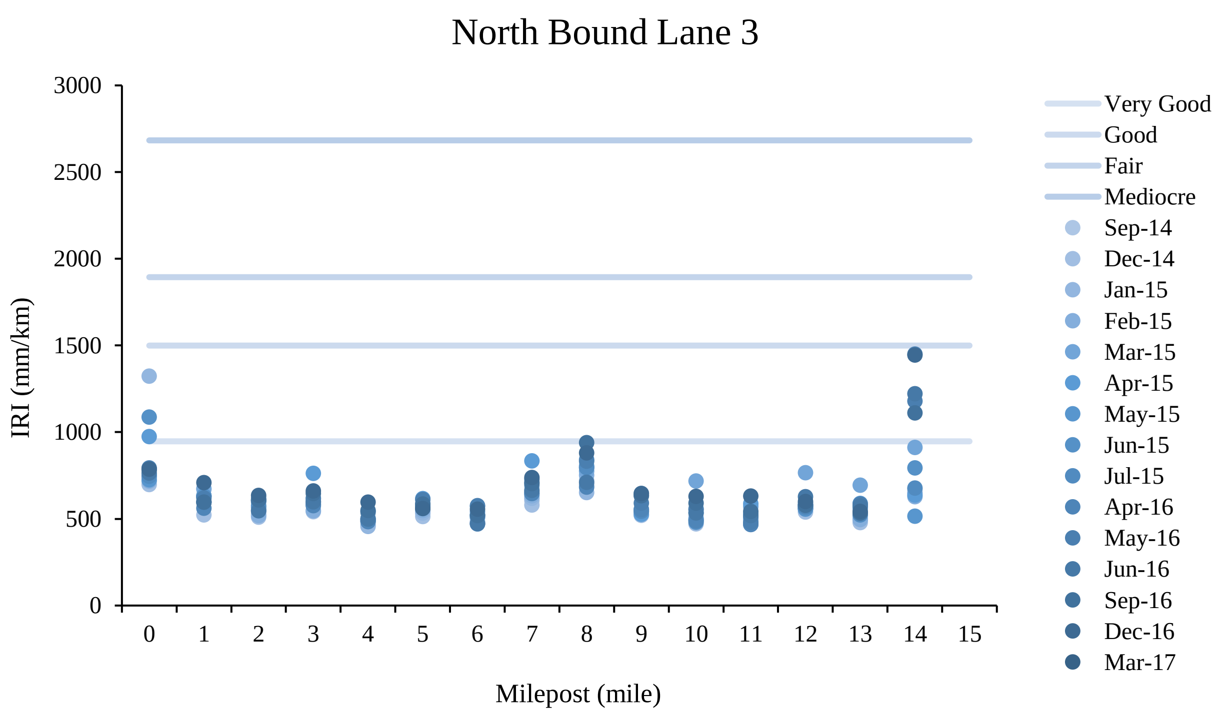


Figure 189. Graph. IRI Readings of NB Lane 3

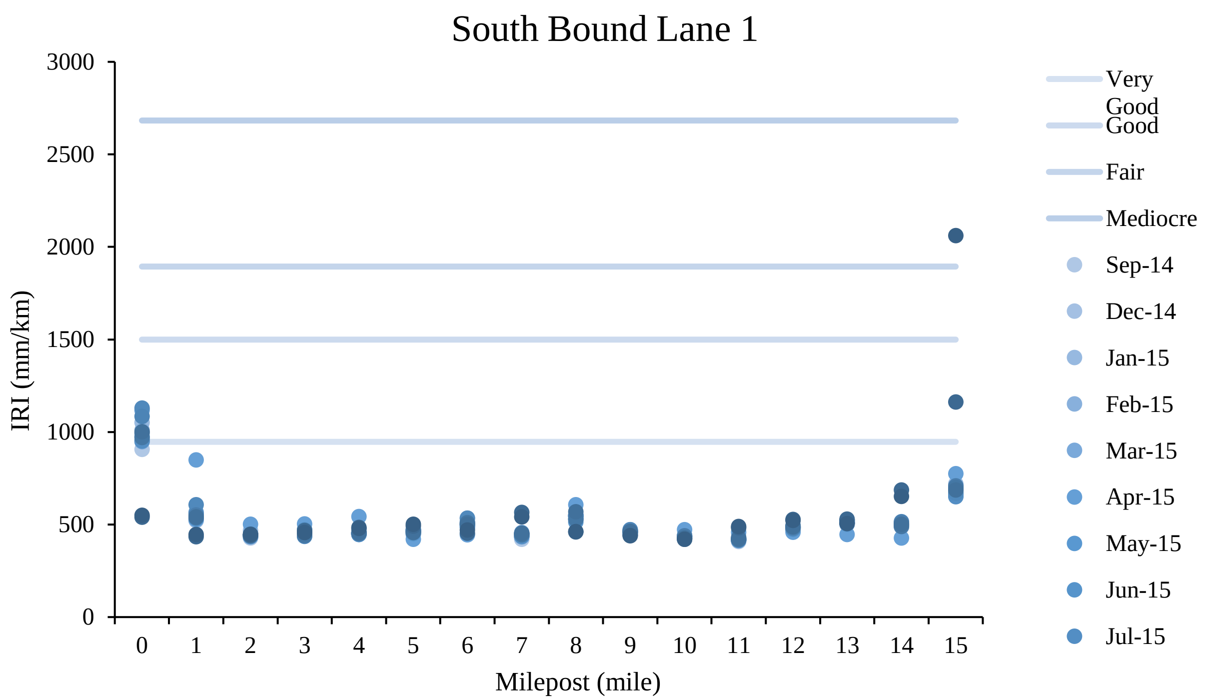


Figure 190. Graph. IRI Readings of SB Lane 1

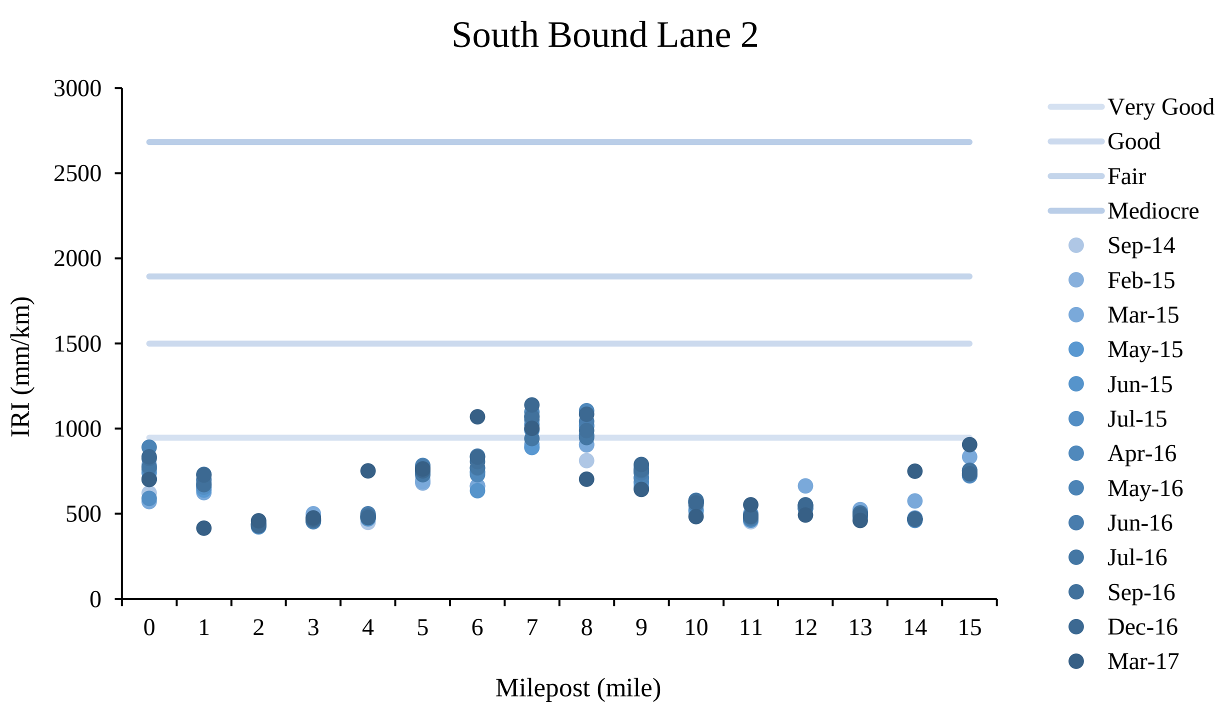


Figure 191. Graph. IRI Readings of SB Lane 2

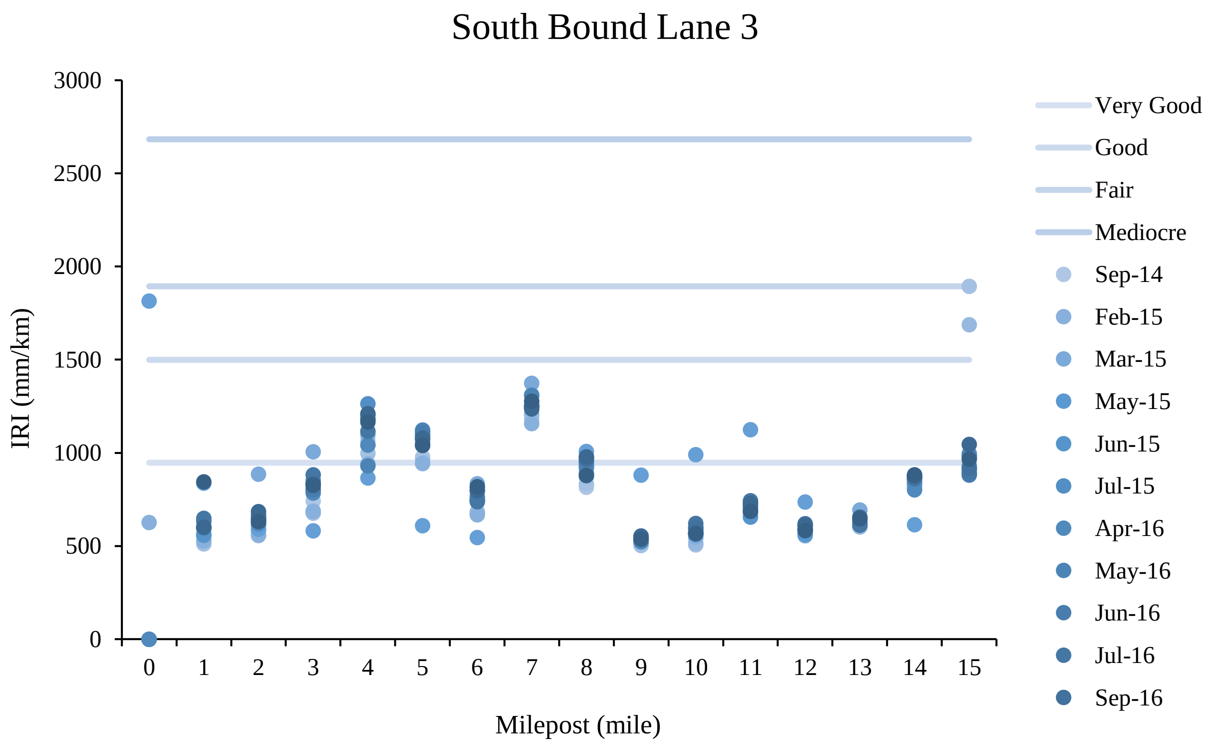


Figure 192. Graph. IRI Readings of SB Lane 3

The following are some major findings based on Figure 187 to Figure 192:

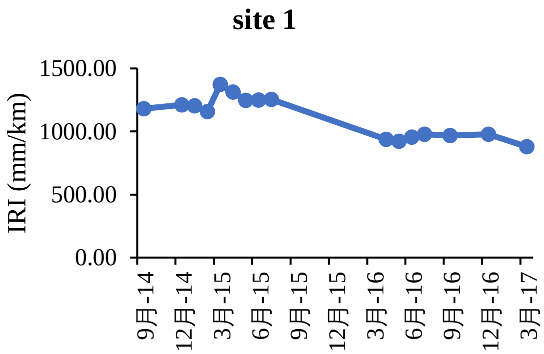
1) At the same location, most of the IRI readings collected at different times are close to each other. The only exception in IRI readings were collected on April 15, which could be due to some error during collection and alignment.

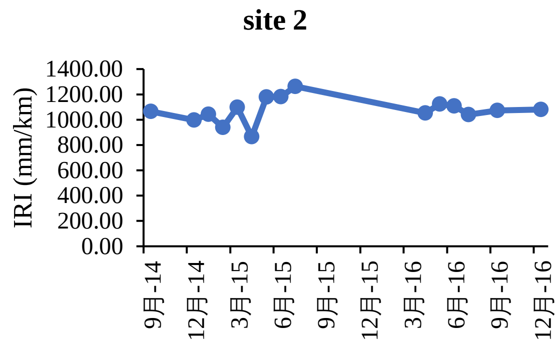
2) On the northbound lanes, IRI measurements in each mile are less than 950 mm/km (60 inch/mile), except for the locations between milepost 0-1 and 14-15, the two ends of I-475. The variation of the IRI measurements in each mile increases from Lane 1 to Lane 3, which could reflect the variation of aggregate loss conditions along different lanes.

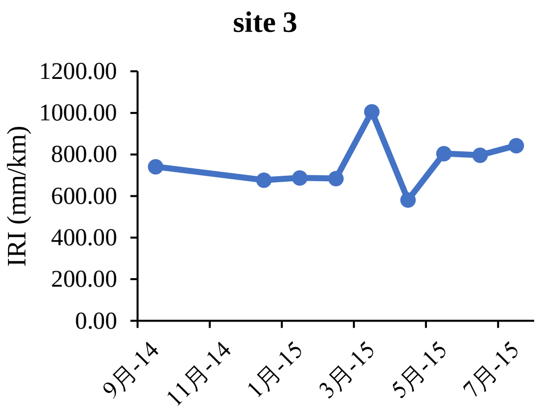
3) On the southbound lanes, the distribution of IRI measurements is close to those of northbound Lane 1. The variation in the IRI measurements in each mile, also, increase from Lane 1 to Lane 3. (Assuming Lane 2 are not changed that much, when comparing Lane 2 and Lane3, the difference between Lane 3 and Lane 2 are larger.) However, the variation in Lane 2 and Lane 3 is bigger than the one in the northbound lane. A possible reason is that the aggregate loss condition has more variations in the southbound Lane 3 than the northbound Lane 3. The IRI around Milepost 8 shows a bump, which might due to more severe raveling or the joint section between asphalt and concrete (e.g. on a bridge).

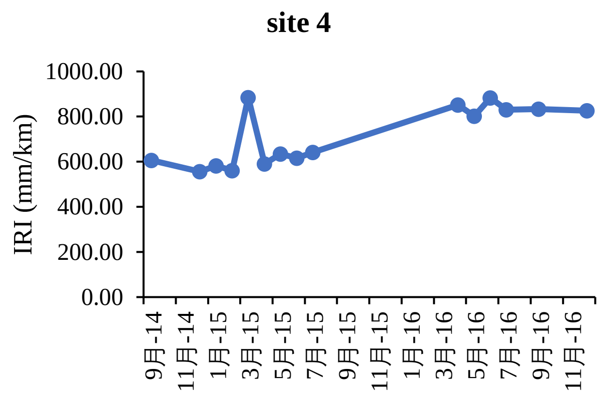
Long-term IRI Analysis on Test Sites

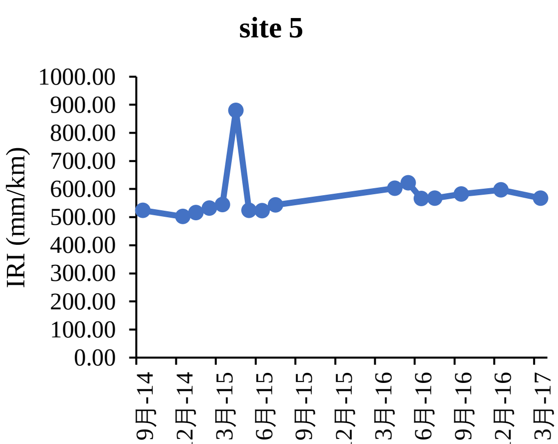
To further verify the possible factors that could result in IRI changes, a detailed study was carried out on each of the 13 test sites located on both southbound and northbound lanes of I-475. Two primary factors to be verified were the seasonal changes and the traffic conditions. The IRI changes over time on the 13 test sites are shown in Figure 193:

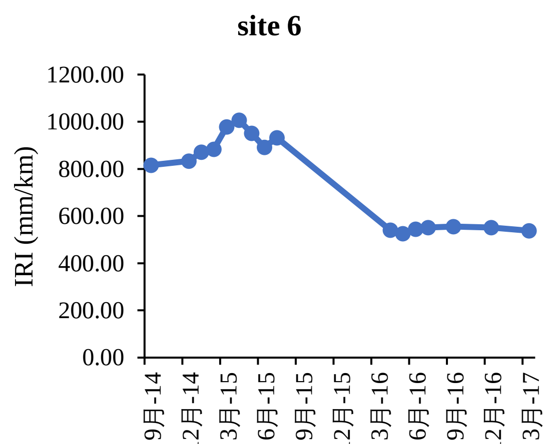

site 1
IRI (mm/km)
1500.00
1000.00
500.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16
3月-17

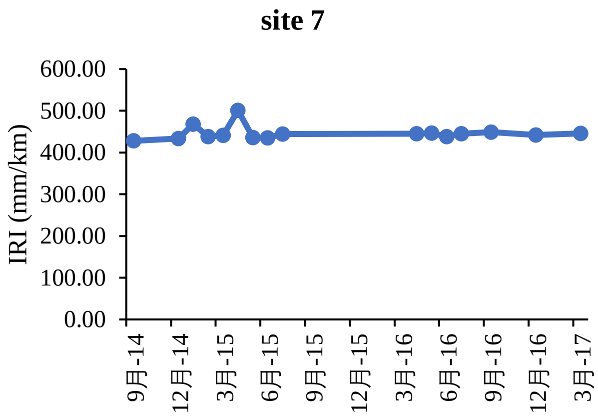

site 2
IRI (mm/km)
1400.00
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16

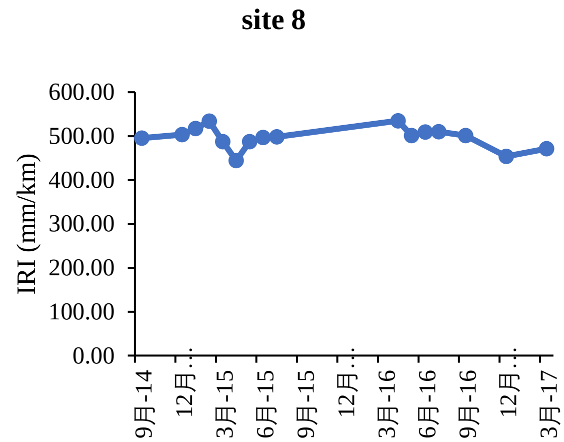

site 3
IRI (mm/km)
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
9月-14
11月-14
1月-15
3月-15
5月-15
7月-15

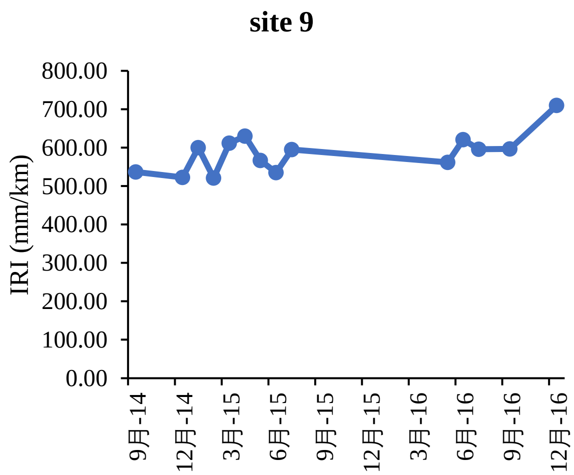

site 4
IRI (mm/km)
1000.00
800.00
600.00
400.00
200.00
0.00
9月-14
11月-14
1月-15
3月-15
5月-15
7月-15
9月-15
11月-15
1月-16
3月-16
5月-16
7月-16
9月-16
11月-16

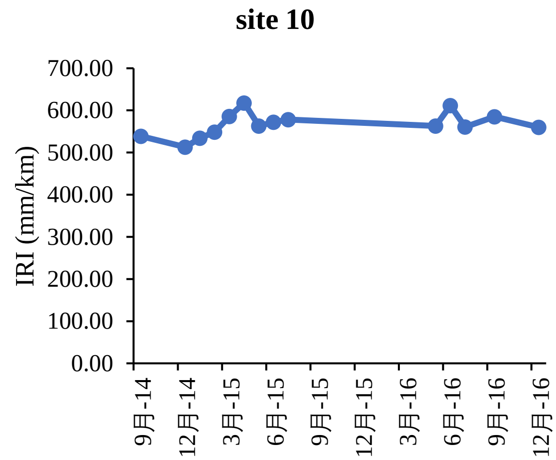

site 5
IRI (mm/km)
1000.00
900.00
800.00
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16
3月-17


site 6
IRI (mm/km)
1200.00
1000.00
800.00
600.00
400.00
200.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16
3月-17


site 7
IRI (mm/km)
600.00
500.00
400.00
300.00
200.00
100.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16
3月-17


site 8
IRI (mm/km)
600.00
500.00
400.00
300.00
200.00
100.00
0.00
9月-14
12月…
3月-15
6月-15
9月-15
12月…
3月-16
6月-16
9月-16
12月…
3月-17


site 9
IRI (mm/km)
800.00
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
12月-15
3月-16
6月-16
9月-16
12月-16


site 10
IRI (mm/km)
700.00
600.00
500.00
400.00
300.00
200.00
100.00
0.00
9月-14
12月-14
3月-15
6月-15
9月-15
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3月-16
6月-16
9月-16
12月-16

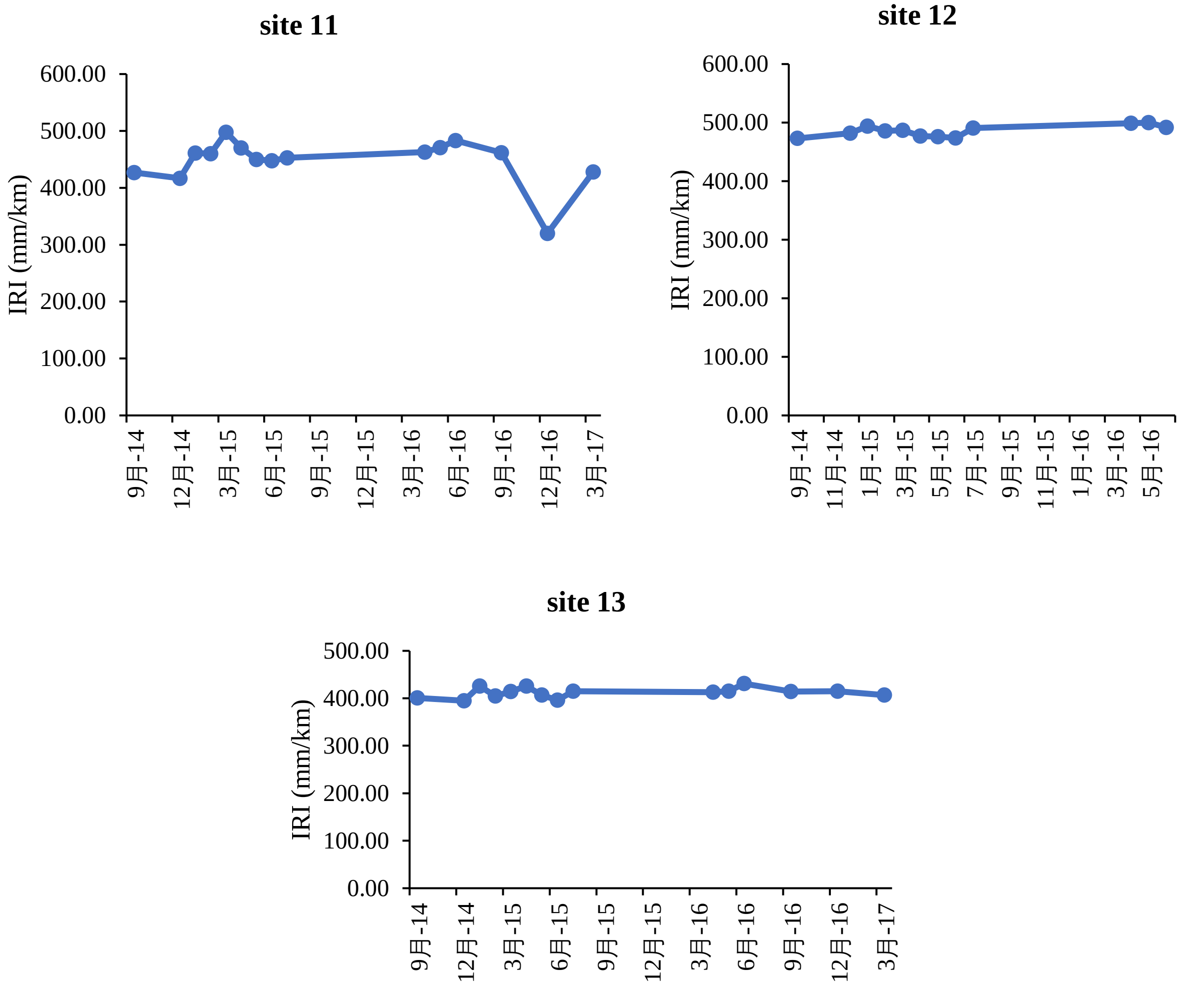


Figure 193. Graph. IRI Readings on 13 Test Sites

According to Figure 193, the following are the major findings:

1) Overall, there is no clear trend in IRI changes over time for all 13 sites. Some bumps appeared (e.g. Sites 3 and 4 on March 15 and Site 5 on April 15), which may have been due to an error in IRI measurement and the data alignment. Therefore, it may indicate that seasonal changes have no obvious impact on IRI.

2) The variation at the test sites in Lanes 1 and 2 (Test Sites 7, 8, 11, 12, and 13) is smaller than other sites in Lane 3. A possible reason is that the texture of the sites in Lane 3 has more variation than the texture in Lanes 1 and 2 as a result of a high aggregate loss rate in Lane 3, which has a high traffic volume and a high truck percentage.

3) The IRI at the test sites in Lanes 1 and 2 is less than 600 mm/km; Test site 1 has the largest IRI (between 1,100 and 1,400 mm/km). Test Site 1 has the most severe aggregate loss condition, which indicates that there is a positive correlation between IRI and aggregate loss.

Summary

1) A test site (I-475) has “very good” smoothness with an IRI less than 60 in/mile based on the IRI measurements after September 2014; the exception is that part of the pavement in southbound Lane 3 has an IRI greater than 60 in/mile, which could be due to severe raveling conditions or joint conditions between asphalt and concrete pavements.

2) A slight increase in IRI right after fog seal application can be observed on most of the pavement based on the short-term IRI performance before and after fog seal applications. However, the difference is insignificant (it is within 6 in/mile difference).

3) For the long-term IRI performance, no clear trend of increase or decrease can be observed over the entire lane. Still, the variation of IRI measurements in each mile increases from Lane 1 to Lane 3, which could be caused by the variation of aggregate loss conditions in different lanes with different truck percentages.

4) Results show that seasonal changes have no obvious impact on IRI. Results show that the aggregate loss increases with higher traffic volume and truck percentage, and IRI increases with the increase of aggregate loss. However, it is not conclusive.

DURABILITY TEST RESULTS

Introduction

To fully assess the effectiveness of applying fog seal to raveled pavements and measure how the treatment performs under heavy traffic, the loss of aggregate from the pavement surface needs to be measured because this could be a measure of pavement durability. Because there is currently no way to quantitatively measure loss from the pavement surface, the primary objective of this study is to use shoulder and rumble strips aggregate buildup (the detailed method is introduced in the Field Observation and Data Analysis section) to evaluate how well the treatment reduces the rate of aggregate loss. Because the rate of aggregate loss may change as raveling severity increases, fog seal performance is measured across four different raveling conditions. By comparing the effectiveness of the treatment in different conditions, the optimal timing for applying fog seal is estimated. By observing roadside aggregate visually and manually, the following key questions need to be considered and answered:

1) Will the rate of aggregate loss on interstate highways decrease after fog seal is applied?

    a. If the rate of aggregate loss decreases, by how much does it decrease?

    b. If the rate of aggregate loss decreases, what are the optimal raveling conditions under which to apply fog seal on interstate highways (e.g., apply fog seal at early stage raveling or severe raveling)?

2) Can shoulder aggregate buildup be used as a measure of fog seal effectiveness?

Test Design

To measure the performance of aggregate loss on different raveling conditions, four sites were selected for visual inspection and quantitative measurement. Sites 1, 2, 4, and 6 were chosen because they each had a different raveling severity as measured by the percentage of aggregate loss. All four sites were on the outside lane (Lane 3) in the southbound direction for ease of measurement and increased safety during the measuring process. The location of the four sites can be seen in Figure 194.

Figure 194. Graph. 4 Sites Selected for Visual Inspection of Aggregate Loss

As shown in Figure 194, the raveling condition varies in the southbound outside lane of I-475. Site 1 is just over one mile from Site 6; however, the sites' raveling conditions are different, as shown in Table 60. The wide variance in the raveling condition seen in this lane is consistent with what is reported in the windshield surveys taken by pavement engineers.

The test sections for the visual inspection were established at 15-m intervals and located at the beginning of each control and fog seal section. These sites were initially marked using raised pavement markers (RPMs) to ensure the sites could be located in the 3D laser data. In addition to using RPMs, fluorescent orange paint was used to identify the sites and mark the 15-m locations for easy visual inspection. A diagram of the site setup is shown in Figure 195.

Table 60. Aggregate Loss Condition and Location for the 4 Selected Sites

Site No.	Raveling Condition	Section Length	Lane	Dir.	RCLINK	MP Start	MP End	
1	Severe (15-20% Loss)	**Control Section** – 500 ft. **Fog Seal Section** – 1000 ft. **Total** – 1500 ft.	3	SB	0211040800	7.83	7.55	
2	Medium (10-15%)				0211040800	11.09	10.81	
4	Light (5-10%)				2071040800	0.80	0.49	
6	Very Light (0-5%)				0211040800	6.62	6.31	

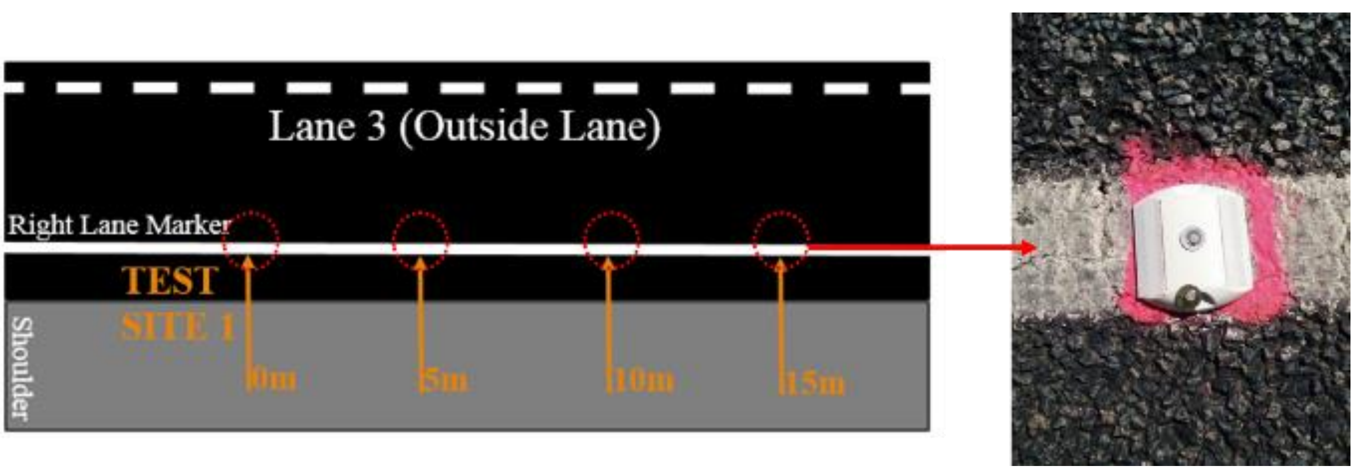


Figure 195. Graph. 15-m Observation Section Set-up

With the RPMs in place, a site is more easily identified from the roadway for friction testing. The RPM installation procedure involved placement of four RPMs at 5-m intervals to outline the first 15-m of each control and fog seal section. We thank Mr. Clayton L. Moore and his colleagues at GDOT for their assistance in setting up the RPMs along each section, as shown in Figure 196.

Figure 196. Photo. RPM Set-up

However, due to the heavy traffic, the RPMs deteriorated very quickly and are no longer used to identify the four sections. To ensure the sites are always identified, the fluorescent orange paint used to initially mark the sites is periodically repainted. This allows for consistent visual inspection of the same location over time and also helps identify the sites for friction testing.

Field Observation and Data Analysis

Aggregate Buildup on Shoulder

To assess aggregate loss from the roadway, the buildup of aggregate along the shoulder is measured at a particular, marked location to see the trend of growth over time. Measurement of shoulder buildup at a consistent location began in March 2016. Measurements were originally taken quarterly; however, since July 2016, measurements have been taken monthly to better understand the trend of aggregate loss. To facilitate consistent measurements, a yardstick is consistently used to measure aggregate buildup, as shown in Figure 197.

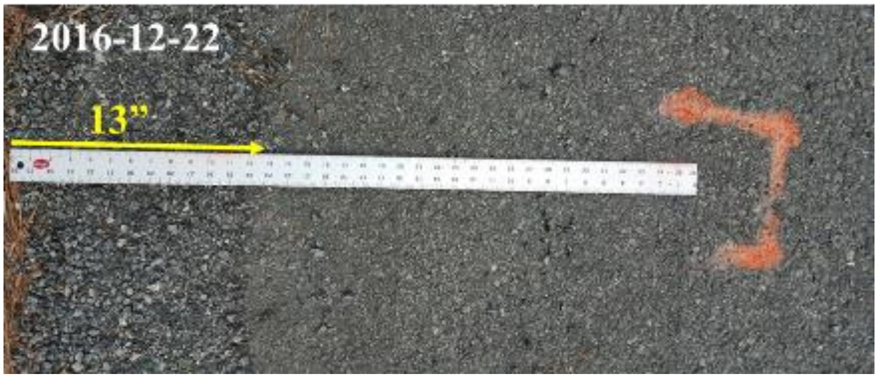


Figure 197. Photo. Width of Aggregate Loss Area along the Shoulder of Site 1

As shown in the figure, the aggregate buildup is measured from the vegetation towards the edge of the travel lane. To ensure measurements were taken at the same location every time, the recording location was marked using fluorescent orange paint to identify the yard stick measuring location. For severe and medium severity raveling, in some cases, the width of the buildup exceeded the length of the yard stick, so the buildup could not be adequately measured. The following sections outline the buildup measurements for the four sites.

Site 1 – Severe Raveling

For Site 1, because of the severity of raveling, the growth of aggregate buildup was expected to be the highest of the four test sections. However, for most of the measurements, the width of the shoulder buildup was greater than the width of the yardstick used for measurements. For all measurements taken in the control section, width measurements that were greater than 3 feet and could not be adequately measured. From visual estimations, there appears to be little to no change in aggregate buildup along the shoulder.

As aggregate is lost from a pavement surface, it is expected it will build up on the shoulder. This growth in buildup, over time, can be measured; however, the results for Site 1 indicate no trend in growth over time. This lack of a trend may, in part, be the result of rain washing away aggregates, as well as shoulder cleaning by GDOT. On September 16, 2015, GDOT cleaned the shoulders of loose aggregate on the four test sections. This type of maintenance is believed to be performed periodically and, therefore, could affect the growth of aggregate buildup between measurements. Currently, the September 2017 shoulder cleaning was the only cleaning maintenance confirmed during this experiment; however, it did not seem to affect the amount of buildup. Table 61 outlines the measurements made.

Table 61. Site 1 Shoulder Measurements

Date	*Control Section (in)*	*Fog Seal Section (in)*
03/07/2016	--	*29.5*
07/18/2016	--	*33.0*
09/22/2016	--	*32.0*
10/25/2016	--	*32.0*
11/26/2016	--	--
12/22/2016	--	--

As indicated in Table 61, it appears that there is no significant trend in the buildup of aggregate on the shoulder in a fog seal section. It is important to note that the growth of buildup from the initial measurement to the final measurement is the focus of this test. Because the control section buildup was greater than 3 feet for all measurements, the growth of the control section as compared to the fog seal section cannot be assessed. In addition, the decrease in fog seal buildup from July 2016 to September 2016 may indicate that environmental factors play a role in increasing or reducing the amount of buildup on the shoulder. To better understand the effectiveness of the fog seal treatment on high severity raveling, more quantitative measurements are needed.

Site 2 – Medium Raveling

The site with medium severity raveling, Site 2, showed a more consistent trend for aggregate buildup. There was a clear increase in buildup on the shoulder for both the control and fog seal sections. The fog seal section, however, grew more, extending from 31 inch for the first measurement to over 36 inch for the final four measurements. By comparison, the initial control section measurement was 9 inch, and the final measurement was 13 inch. The measurements of shoulder buildup for Site 2 are given in Table 62.

Table 62. Site 2 Shoulder Measurements

Date	*Control Section (in)*	*Fog Seal Section (in)*
03/07/2016	*9.0*	*31.0*
07/18/2016	*12.0*	*33.0*
09/22/2016	*13.0*	--
10/25/2016	*13.0*	--
11/26/2016	*13.0*	--
12/22/2016	*13.0*	--

From these measurements, it can be seen that, although both sections increased, the fog seal section increased more. This is counterintuitive, as the control section would be expected to have increased more, overall, because of the loss of more aggregate. The given measurements indicate the use of fog seal has no effect on the rate of aggregate loss; however, environmental factors, such as wind and rain, may affect aggregate loss and the validity of using buildup as a measure.

Site 4 – Light Raveling

The light raveling test section showed inconsistent results, so no conclusions are drawn about the performance of the fog seal treatment. There was little variance in the aggregate buildup measurements for the control section; however, measurements for the fog seal section varied significantly, indicating there were, likely, other factors involved that altered the amount of aggregate buildup on the shoulder. Because of this variance and the tendency of the measurements to both increase and decrease, it is difficult to draw any conclusions as to the effectiveness of the fog seal treatment. Shoulder buildup measurements for Site 4 are given in Table 63.

Table 63. Site 4 Shoulder Measurements

Date	*Control Section (in)*	*Fog Seal Section (in)*
03/07/2016	*9.5*	*29.0*
07/18/2016	*17.0*	*29.0*
09/22/2016	*19.0*	*24.0*
10/25/2016	*18.0*	*- -*
11/26/2016	*19.0*	*36.0*
12/22/2016	*17.0*	*32.0*

From the table, it is observed that the control section remained at approximately 18 inch for most measurements. In comparison, the fog seal section ranged from 24 inch to over 36 inch There is no clear trend in the growth or decline of buildup in the fog seal section, meaning it is likely that other factors, such as wind, rain, vehicular shoulder traffic, and cleaning maintenance, may have affected the amount of buildup observed.

Site 6 – Very Light Raveling

For Site 6, both sections showed a clear, decreasing trend from the initial measurement to the final measurement. The initial measurements of both sections were significantly higher than the remaining measurements, which is counterintuitive, as buildup would have been expected to increase as more aggregate was lost. There was significant variance in the amount of buildup measured in the fog seal section, as well, indicating external factors may have affected the measurements. Measurements for aggregate buildup for site six are given in Table 64.

Table 64. Site 6 Shoulder Measurements

Date	*Control Section (in)*	*Fog Seal Section (in)*
03/07/2016	*17.0*	*29.0*
07/18/2016	*3.0*	*9.0*
09/22/2016	*0.0*	*9.5*
10/25/2016	*0.0*	*9.5*
11/26/2016	*0.0*	*12.0*
12/22/2016	*0.0*	*10.0*

From the table, more consistency can be observed in the control section measurements than in the fog seal section measurements. In particular, the fog seal section measurements seem to decrease during the warmer months of July through October; they seem to increase during the colder months of November and December. This indicates that more aggregate is lost during the winter months; however, it does not give much information about the performance of the treatment.

Summary

Measurements were taken of aggregate buildup on the shoulders adjacent to locations with and without fog seal treatment in hopes of observing the trends of growth in the two sections. In general, the measurements did not indicate any significant differences between the control and fog seal sections. Although the data collected gave some indication as to the changes in aggregate loss during the winter and summer months, no conclusions could be drawn about the effectiveness of applying fog seal to raveling pavements. These inconsistencies in measurement are addressed by the quantitative collection of lost aggregate from the rumble strips.

Aggregate Buildup in Rumble Strips

Quantitative measurements of aggregate loss falling into the rumble strips were conducted to measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and the fog sealed test sections. Beginning in March 2016, the aggregates were collected in bags and weighed. After the aggregate was collected, the rumble strips were cleaned. This allowed for the measurement of the monthly accumulation of aggregate. Quantitative measurements helped better determine the effectiveness of the treatment at the different severity levels so the optimal time to treat could be determined. These measurements were, also, less susceptible to environmental factors, such as wind and rain, because the aggregate was contained within the rumble strip.

In addition to quantitatively measuring the amount of aggregate in the rumble strips, the percentage of the rumble strips filled with aggregate was also measured. This was done to confirm the weight measurements to make sure they were consistent with what was shown in the field. The visual estimates closely followed the aggregate weight measurements and, therefore, are not presented. The results from the aggregate collection test are given in Table 65.

The table shows that the fog seal treatment was typically effective at reducing the amount of aggregate lost in comparison to the control sections. Effectiveness was measured as the difference in weight between the control section and the fog seal section. For a majority of the measurements, the difference was positive, indicating more aggregate was lost in the control sections. However, there are some measurements in which the difference in weight was negative, indicating more aggregate was lost in the fog seal sections. Possible reasons for these anomalies in the measurements are given below.

Table 65. Aggregate Loss Summary

Date	Site	Weight (g)		(%) Difference	Wt. Difference
		Non-fog sealed control section	Fog-sealed test section		
3/7/2016	Site 1 (severe)	927.4	1286.7	-38.7	-359.3
	Site 2 (medium)	1352.1	556.9	58.8	795.2
	Site 4 (light)	249.3	92.9	62.7	156.4
	Site 6 (very light)	69.6	55.9	19.7	13.7
7/18/2016	Site 1 (severe)	698.2	120.6	82.7	577.6
	Site 2 (medium)	583.4	179.8	69.2	403.6
	Site 4 (light)	8.7	2.1	75.9	6.6
	Site 6 (very light)	1.2	1.7	-41.7	-0.5
9/22/2016	Site 1 (severe)	325.0	35.0	89.2	290.0
	Site 2 (medium)	77.3	26.9	65.2	50.4
	Site 4 (light)	4.2	3.0	28.6	1.2
	Site 6 (very light)	5.1	6.3	-23.5	-1.2
10/25/2016	Site 1 (severe)	91.9	64.2	30.1	27.7
	Site 2 (medium)	116.7	47.6	59.2	69.1
	Site 4 (light)	14.3	14.6	-2.1	-0.3
	Site 6 (very light)	10.0	4.9	51.0	5.1
11/26/2016	Site 1 (severe)	217.3	125.3	42.4	92.0
	Site 2 (medium)	176.0	87.5	50.3	88.4
	Site 4 (light)	49.5	22.0	55.5	27.5
	Site 6 (very light)	24.6	16.8	31.6	7.8

(continued)

12/22/2016	**Site 1 (severe)**	**193.5**	**265.7**	**-37.3**	**-72.2**
	Site 2 (medium)	245.1	152.8	37.6	92.3
	Site 4 (light)	56.6	29.4	48.1	27.2
	Site 6 (very light)	22.2	24.3	-9.5	-2.1
1/24/2017	**Site 1 (severe)**	322.0	319	0.9	3.0
	Site 2 (medium)	355.0	141	60.3	214.0
	Site 4 (light)	10.0	2.0	80.0	8.0
	Site 6 (very light)	5.0	21.0	-320.0	-16.0
2/24/2017	**Site 1 (severe)**	458.0	214.0	53.3	244.0
	Site 2 (medium)	405.0	178.0	56.0	227.0
	Site 4 (light)	45.0	35.0	22.2	10.0
	Site 6 (very light)	19.0	16.0	15.8	3.0
4/7/2017	**Site 1 (severe)**	501.1	310.1	38.1	191.0
	Site 2 (medium)	576.5	232.4	59.7	344.1
	Site 4 (light)	39.1	2.4	93.9	36.7
	Site 6 (very light)	22.1	38.0	-71.9	-15.9
5/12/2017	**Site 1 (severe)**	225.0	125.3	44.3	99.7
	Site 2 (medium)	148.2	87.0	41.3	61.2
	Site 4 (light)	27.2	13.6	50.0	13.6
	Site 6 (very light)	12.7	10.8	15.0	1.9
6/13/2017	**Site 1 (severe)**	221.8	35.3	84.1	186.5
	Site 2 (medium)	68.0	28.5	58.1	39.5
	Site 4 (light)	3.9	1.3	66.8	2.6
	Site 6 (very light)	3.7	7.8	-111.1	-4.1

Anomalous Measurements

From Table 65, several conclusions can be drawn about the effectiveness of applying fog seal to raveling pavements. Of the 24 total measurements taken, 6 indicated an increase in aggregate loss after the application of fog seal. However, upon further inspection, only 2 of these measurements were significant. Of the 6 measurements indicating a decrease in performance with fog seal, 3 measurements came from Site 6 (very light raveling), and one from Site 4 (light raveling). Although the results indicate decreased performance, the actual difference in aggregate weight was less than 2.5g for all measurements, which is insignificant at this scale. Furthermore, when looking at the total aggregate loss over the course of the study, both the light and very light raveling sections show more total aggregate lost from the control section than from the fog seal section.

The two other measurements, highlighted in red in Table 65, show the decreased performance after the application of fog seal came from Site 1. For these March and December measurements, there was significantly more aggregate in the rumble strips of the fog seal section than in the control section. For the March measurement, the aggregate in the two sections had been building up for over four months (since November 2015). During this time, external factors, such as wind, rain, and shoulder cleanings, may have reduced the amount of aggregate in the control section. For the December measurement, it is believed that the accumulation of aggregate in the fog seal was affected by the presence of water in the rumbles strip, as shown in Figure 198.

Figure 198. Photo. Site 1 Wet Aggregate in Rumble Strip

As can be seen from the figure, on the day of collection, the rumble strips were not as dry as normal. The presence of water in the rumble strip creates a wet mixture of sand and loose aggregate. Previous measurements primarily included loose aggregate with a limited amount of sand collected. Because of this mixture, it was difficult to separate the sand and aggregate, resulting in the collection of more sand and water than normal. Excluding these two measurements, the application of fog seal appears to be 30-90% effective at reducing the amount of aggregate lost by raveling, depending on both temperature and raveling condition. This indicates that fog seal can be applied to severely raveling pavements to reduce the amount of aggregate loss in the short term in order to better maintain condition until resurfacing can be done.

Effectiveness of Fog Seal and Determination of Optimal Timing

In general, measurements strongly indicate the application of fog seal to raveling pavement is effective in reducing the rate of aggregate loss. As raveling conditions worsen, the effectiveness of the treatment appears to increase; however, the effectiveness on severe raveling is inconclusive. Excluding anomalous sections, the application of fog seal appears to reduce the amount of aggregate lost in severe raveling conditions by approximately 30-90%. For medium raveling conditions, the application of fog seal appears to reduce the amount of aggregate lost by approximately 40-70%. The wide ranges in reduction are likely caused by seasonal variations in temperature. The treatment appears to be highly effective during warmer seasons; however, as temperatures drop, the treatment is less effective at retaining aggregate. The weight measurements taken for this test are illustrated in Figure 199.

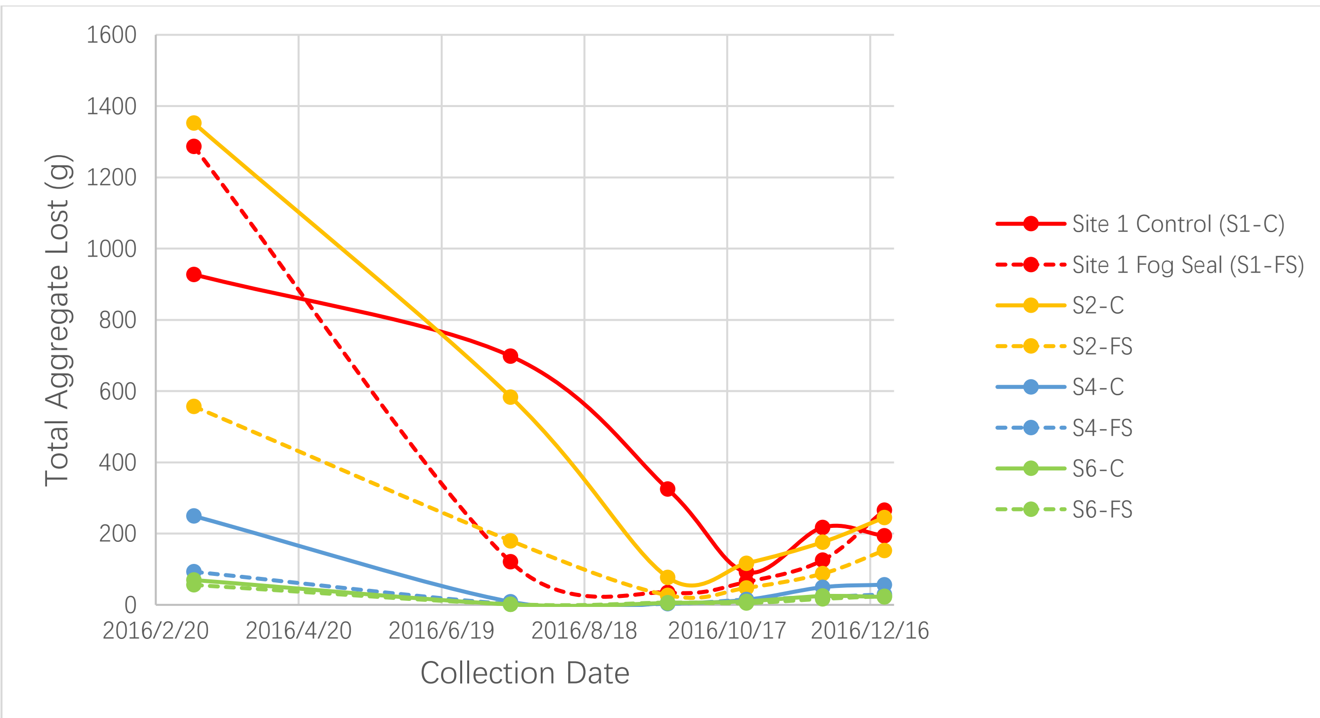


Figure 199. Graph. Effectiveness of Fog Seal Treatment Against Control Sections

The first two measurements were taken less frequently than the last four measurements, and, because of the larger duration between measurements, these two have much higher total aggregate lost. To better show the effects of the treatment on raveling, Figure 200 shows the data collection efforts were taken at a shorter interval (~1-2 months).

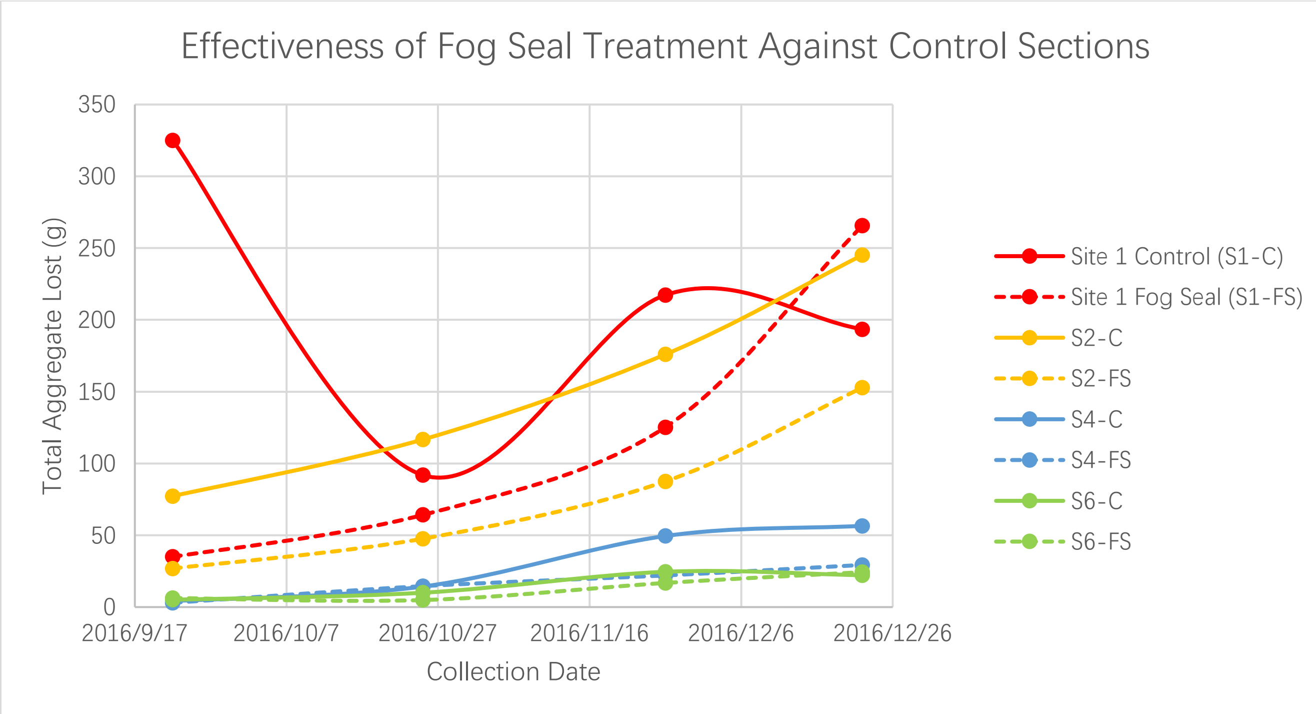


Figure 200. Graph. Effectiveness of Fog Seal Treatment Against Control Sections (Short Interval)

This better captures the nature of the data. From the figure, it is clear that as the temperature drops, more aggregate is lost and the efficiency is reduced. This is particularly true for the medium severity raveling cases in which the efficiency at reducing aggregate loss reduces from 65% in September to 38% in December. The measurements for the severe raveling case are inconclusive because of the anomalous data mentioned previously. More data is needed to assess the effects of fog sealing on severe raveling and the changes in effectiveness as temperature changes.

When determining the optimal time to apply fog seal, the lifetime of the treatment is the most important factor to consider. Excluding anomalous data, when looking at short term aggregate loss for severe raveling, fog seal treatment consistently shows a reduction in weight of aggregate lost. The treatment is also effective on medium raveling conditions in which the treatment reduces the amount of aggregate lost by 38-65%, depending on temperature. For light raveling conditions, however, the effectiveness of the treatment is more easily realized after a significant buildup duration (>~4 months). In this case, the fog seal section lost 71g between July and December measurements. This is 47% less than the 133g lost in the control section. The case is similar for the very light conditions. In total, the fog seal section lost 54g between July and December. This is 14% less than the 63g lost by the control section. These results indicate the treatment is effective, but to a much lesser degree than the medium and severe conditions.

Temperature impacts the effectiveness of the treatment and should be taken into account when determining the need for fog seal. Because of the consistent effectiveness at reducing aggregate loss for severe and medium raveling conditions, it is concluded that the treatment can be applied during these conditions and will significantly reduce the amount of aggregate loss in both the short and long term. Therefore, fog seal can be applied to severely raveling pavements to reduce the rate of aggregate loss on a monthly basis until a chip seal or resurfacing procedure can be performed.

For light raveling conditions, the effectiveness of the treatment is best recognized over a longer period of time and, therefore, is not effective at greatly reducing aggregate loss over a short period. However, it can be used as a preventative measure for pavements that are subject to high traffic and are expected to have a raveling problem in the future. Finally, for very light conditions, total aggregate loss over the course of the study indicates fog seal is effective. However, the effectiveness is minimal when compared to light, medium, and severe raveling conditions. This indicates that the application of fog seal to very light raveling conditions may not be the most cost-effective means of reducing raveling.

Summary

Two tests (shoulder aggregate buildup and rumble strips aggregate buildup) were conducted to qualitatively and quantitatively measure the fog seal effectiveness on 4 different raveling conditions (very light, light, medium, severe with a loss of aggregates ranging from 0% to 20%; GDOT defined all of them to be Severity Level 1). However, shoulder aggregate buildup tests appeared to be severely impacted by maintenance and environmental factors. Therefore, rumble strips aggregate buildup tests were conducted to quantitatively measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and the fog sealed test sections based on the aggregate loss falling into the rumble strips. Based on the measurements of 4 selected sites on I-475, the following conclusions are made:

- The fog seal treatment is promising in reducing the rate of aggregate loss for medium and severe raveling conditions by 30%-90% and 40%-70%, respectively. (The medium and severe raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 10%-15% and 15-20%, respectively.)

- The fog seal effectiveness on very light and light raveling conditions is not apparent because the quantities aggregates loss on non-fog seal and fog seal sections are both small. (The very light and light raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 0%-5% and 5-10%, respectively.)

To further verify the durability of fog seal on pavement with medium or lighter aggregate loss conditions, the following steps are suggested: 1) continue the long-term visual and quantitative assessments to confirm the effectiveness of applying fog seal in all conditions; and 2) develop a quantitative measure of aggregate loss using precise 3D pavement laser data.

CONCLUSIONS

To preserve interstate highway pavements with raveling issues, GDOT has actively sought inexpensive preventive maintenance methods, e.g. fog seal. However, due to the lack of experience in using fog seal on high-traffic-volume interstate highways, GDOT, teaming up with Georgia Tech, initiated a case study project on I-475, the by-pass around Macon, Georgia. The objective was to evaluate the performance of pavements after fog seal application in terms of safety and durability. For this purpose, 13 test sites were selected on I-475 based on traffic conditions and the severity level of raveling (very light, light, medium, severe with a loss of aggregates ranging from 0% to 20%; GDOT defined all of them to be Severity Level 1). Various tests were conducted, including locked-wheel tests, IRI tests, durability tests. Permeability tests were also conducted; however, it is noted that the test results are inconsistent and largely depended on operators. Thus, the permeability test results are not included in this report.

The following summarize the major research findings.

Friction Tests:

- A pavement's surface friction decreases right after fog seal application. Based on measurement results, the skid number (SN) decreases about 45% immediately after the fog seal application.

- The SN recovers to 30 - 35 in 2-4 days and to 35 or above in 5-7 days after fog seal application. Results show the surface friction of pavements with high traffic volume and truck percentage recover faster than the pavements with low-traffic volume.

- Results show that a higher friction value in the outside lane with high truck traffic volume compared to the inside lane, which is likely caused by different loss of aggregates. In addition, measurements show that the friction is higher in summer and lower in winter.

IRI Tests:

- The test site (I-475) has “very good” smoothness with the IRI being less than 60 in/mile based on the IRI measurements after September 2014, except that part of the pavement in southbound Lane 3 has an IRI greater than 60 in/mile, which could be due to severe raveling conditions or joint conditions between asphalt and concrete pavements.

- A slight increase in IRI right after fog seal application can be observed on most of the pavement based on the short-term IRI performance before and after fog seal applications. However, the difference is insignificant (it is within 6 in/mile difference).

- For the long-term IRI performance, no clear trend of increase or decrease can be observed over the entire lane. Still, the variation of IRI measurements in each mile increases from Lane 1 to Lane 3, which could be caused by the variation of aggregate loss conditions along different lanes with different truck percentages.

- Results show that seasonal changes have no obvious impact on IRI. Results show that the aggregate loss increases with higher traffic volume and truck percentage, and IRI increases with the increase of aggregate loss. However, it is not conclusive.

Durability Tests:

The durability tests were conducted to measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and the fog sealed test sections based on the quantitative measurements of aggregate loss falling into the rumble strips. The following conclusions can be made about fog seal effectiveness:

- The fog seal treatment is promising in reducing the rate of aggregate loss for medium and severe raveling conditions by 30%-90% and 40%-70%, respectively. (The medium and severe raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 10%-15% and 15-20%, respectively.)

- The fog seal effectiveness on very light and light raveling conditions is not apparent because the quantities aggregates loss on non-fog seal and fog seal sections are both small. (The very light and light raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 0%-5% and 5-10%, respectively.)

RECOMMENDATIONS

- Permeability of surface friction course is still a concern. Due to the application of fog seal, potential safety issues could be raised because of the increasing chance of splash, spray, and, even, hydroplaning. It is noted that a Permeameter is not a good means for measuring permeability on porous surfaces. It is suggested that splash and spray tests be conducted, and they should be monitored by cameras.

- Further study is needed to assess the durability of pavements after fog seal. In this research, visual inspection and sample weighing were used to measure aggregate loss over time. This method is not completely accurate, since the results are largely affected by environment and human measurements. The 3D pavement surface data was collected. It is recommended a more accurate method to quantitatively compute the loss of aggregates using the already collected 3D pavement surface data be developed.

- As recommended by GDOT engineers, additional field tests are needed to evaluate the performance of fog seal on interstate highway pavements at different ages and in different locations. Although the test results are useful for uncovering some characteristics and trends of pavement performance, they cannot be easily extended to other pavements at different ages or in different locations. Especially, if the optimal timing of fog seal is to be accurately determined, more field tests and long-term monitoring on diverse pavement samples are needed.

CHAPTER 7. CONCLUSIONS AND RECOMMENDATIONS

The experienced pavement experts in GDOT possess a great amount of tacit knowledge regarding pavement preservation, but before those experts retire, there is an urgent need to collect and document their knowledge. In addition, other state DOTs' experience and knowledge of pavement preservation techniques and their treatment criteria and performance also need to be studied. As GDOT increases the number of its pavement preservation outsourcing projects, a pavement preservation guide containing comprehensive and consistent information about project selection, specification, material selection, construction procedure, and QA/QC is essential and is urgently needed for the Georgia Department of Transportation (GDOT) to perform cost-effective pavement maintenance (right treatment, right time, and right location) and achieve the best pavement performance for the least amount of money while meeting MAP-21's requirements.

The objectives of this research are to develop the first version of the comprehensive pavement preservation guide, covering project selection, specification, material selection, construction procedure, QA/QC, along with the web-based pavement preservation interactive tool (PPIT) that enables GDOT to cost-effectively preserve its pavements and to conduct field tests and long-term performance monitoring using 3D technology to study the two most commonly implemented pavement preservation methods (i.e. crack sealing and fog seal), identified by GDOT. This information will be used to evaluate the feasibility of applying fog seal to high traffic volume interstate highways and to study crack sealing effectiveness and adequate treatment timing in support of GDOT's pavement preservation criteria and policies.

CONCLUSIONS

The research findings of this study are summarized in the three parts below:

Part 1: Development of an Interactive Pavement Preservation Guide

This research project developed the first version of a comprehensive pavement preservation guide and the web-based pavement preservation interactive tool (PPIT) for this guide that can be used by GDOT's Office of Maintenance and seven different district offices at any time (24 hours a day and 7 days a week) and in any place. Different asphalt and concrete pavement preservation methods have been incorporated into the PPIT and are presented in this report. For the detailed introduction and tutorial of the PPIT, please see APPENDIX A. For asphalt pavements, this report presents chip seal, crack seal, fog seal, micro-milling, micro-surfacing, thin overlay, white topping, pothole patching, open-graded interlayer, full-depth reclamation, hot in-place recycling, cold in-place recycling, and ultra-thin bonded wearing course. For concrete pavements, this report presents diamond grinding, dowel bar retrofit, full-depth repair, joint sealing, and partial depth repair. For each pavement preservation method, this report presents the treatment method, treatment selection criteria, material design, construction, and the corresponding considerations and performance and limitations.

Part 2: Crack Sealing Field Test Study

The second part of this report presents the field test study of crack seal. Nine selected field test sites with different locations (six in Hawkinsville, southern GA; three in Covington, central GA), different pretreatment pavement conditions (COPACES ratings of 66 to 98) and different roadway environment factors (thicknesses 2.5 to 10.0 inch; AADT 930 to 12,200; truck percentage 4% to 26%) have been monitored quarterly using 3D technology over the 3 past years right after the crack sealing application. To quantitatively compute the crack sealing effectiveness, each site has both crack sealed test sections and non-sealed control sections to compare the differences in crack length growth. Based on detailed analysis results of 3 years of data on 9 test sites, the following findings are presented:

- Crack sealing can effectively retard crack growth by 40% to 128% on crack sealed test sections compared to the non-sealed control sections, which demonstrates a very good crack sealing effectiveness.

- A higher pretreatment COPACES rating pavement provides better crack sealing effectiveness based on a breakdown study with 3 clusters of pretreatment COPACES ratings (66 – 69, 79 – 85, 93 – 98) having a 40% – 48%, 52% – 72%, 82% – 124% crack sealing effectiveness, respectively. This finding supports GDOT's new policy of increasing its crack sealing criteria from a maximum COPACES rating of 85 to 90.

- Crack sealing effectiveness increases when truck traffic (AADTT) decreases; however, the finding is not conclusive. In addition, there is no clear trend for the impact of pavement thickness on crack sealing effectiveness.

- Results show that crack sealing effectiveness is high for Severity Level 1 block/transverse (B/T) cracking no matter the crack length or percentage. More and higher severity level of longitudinal cracking on wheel paths (GDOT defined it as load crack), and higher severity levels of B/T cracking (Severity Level 2 B/T cracking) result in lower crack sealing effectiveness.

Part 3: Fog Seal Field Test Study

The third part of this report presents the field test study of fog seal. Thirteen test sites on I-475 with different raveling conditions (very light, light, medium, severe with a loss of aggregates ranging from 0% to 20%; GDOT defined all of them to be Severity Level 1) were selected to conduct various field tests, including locked-wheel friction test, IRI test, and visual inspection of aggregate loss test. The following summarizes the major research findings:

Friction Tests

- A pavement's surface friction decreases right after fog seal application. Based on measurement results, the skid number (SN) decreases about 45% immediately after the fog seal application.

- The SN recovers to 30 - 35 in 2-4 days, and 35 or above in 5-7 days after fog seal application. Results show the surface friction of pavements with high traffic volume and truck percentage recover faster than the pavements with low-traffic volume.

- Results show that a higher friction value in the outside lane with high truck traffic volume compared to the inside lane that is likely caused by different loss of aggregates. In addition, measurements show that the friction is higher in summer and lower in winter.

IRI Tests

- The test site (I-475) has "very good" smoothness with the IRI being less than 60 in/mile based on the IRI measurements after September 2014, except that part of the pavement in southbound Lane 3 has an IRI greater than 60 in/mile, which could be due to severe raveling conditions or joint conditions between asphalt and concrete pavements.

- A slight increase in IRI right after fog seal application can be observed on most of the pavement based on the short-term IRI performance before and after fog seal applications. However, the difference is insignificant (within 6 in/mile difference).

- For the long-term IRI performance, no clear trend of increase or decrease can be observed over the entire lane. Still, the variation of IRI measurements in each mile increases from Lane 1 to Lane 3, which could be caused by the variation of aggregate loss conditions along different lanes with different truck percentages.

- Results show that seasonal changes have no obvious impact on IRI. Results show that the aggregate loss increases with higher traffic volume and truck percentage, and IRI increases with the increase of aggregate loss. However, it is not conclusive.

Durability Tests

The durability tests are conducted to measure the fog seal effectiveness by comparing the aggregate loss on non-fog sealed control sections and the fog sealed test sections based on the quantitative measurements of aggregate loss falling into the rumble strips. The following conclusions can be made about fog seal effectiveness:

- The fog seal treatment is promising in reducing the rate of aggregate loss for medium and severe raveling conditions by 30%-90% and 40%-70%, respectively. (The medium and severe raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 10%-15% and 15-20%, respectively.)

- The fog seal effectiveness on very light and light raveling conditions is not apparent because the quantities aggregates loss on non-fog seal and fog seal sections are both small. (The very light and light raveling conditions are under GDOT's raveling Severity Level 1 with loss of aggregate around 0%-5% and 5-10%, respectively.)

RECOMMENDATIONS

The following presents the recommendations for improving the pavement preservation method guide and future research for performing fog seal and crack seal studies.

Recommendations of the Crack Sealing Performance Study

- It is recommended that crack growth monitoring on the crack sealing test sites be extended to better understand the terminal life of crack sealing (e.g. 3 years, 5 years).

- Our crack sealing effectiveness study focuses on the low severity level of cracking. Additional test sites with severe cracks (e.g. Severity Level 3) are recommended to study its impact.

- It is recommended that the impact of the crack sealing area on the pavement friction reduction be studied to establish a criterion for the maximum crack sealing area.

- It recommended that the impact of pavement thickness on crack sealing effectiveness on additional test sites with different thicknesses, but similar COPACES ratings and traffic volume be studied.

- It is recommended that quantitatively measuring the impact of the pretreatment crack characteristics (e.g. crack length, direction, position, density) and different types of crack (e.g. load crack, block crack) and severity levels be accomplished. This will enable state DOTs to optimize their crack sealing effectiveness with the detailed crack characteristic-based criteria.

Recommendation of Fog Seal Performance Study

- Permeability of surface friction course is still a concern. Due to the application of fog seal, potential safety issues could be raised because of the increasing chance of splash, spray, and, even, hydroplaning. It is noted that Permeameter is not a good means for measuring permeability on porous surface. It is suggested that splash and spray tests be conducted, and they should be monitored by cameras.

- Further study is needed to assess the durability of pavements after fog seal. In this research, visual inspection and sample weighing were used to measure aggregate loss over time. This method is not completely accurate, since the results are largely affected by environment and human measurements. The 3D pavement surface data were collected. It is recommended a more accurate method to quantitatively compute the loss of aggregates, using the already collected 3D pavement surface data, be developed.

- As recommended by GDOT engineers, additional field tests are needed to evaluate the performance of fog seal on interstate highway pavements at different ages and in different locations. Although the test results are useful for uncovering some characteristics and trends of pavement performance, they cannot be easily extended to other pavements at different ages or in different locations. Especially, if the optimal timing of fog seal is to be accurately determined, more field tests and long-term monitoring on diverse pavement samples are needed.

APPENDIX A. USER TUTORIAL OF THE PAVEMENT PRESERVATION INTERACTIVE TOOL (PPIT)

PPIT INTRODUCTION

This project developed the first version of the comprehensive pavement preservation guide along with the web-based pavement preservation interactive tool (PPIT) that can be used by GDOT's Office of Maintenance at any time (24 hours a day and 7 days a week) and in any place. The PPIT has four major functions including 1) asphalt pavement treatments introduction, 2) asphalt pavement treatment selection, 3) concrete pavement treatments introduction, and 4) concrete pavement treatment selection. The treatment introduction covers the knowledge of project selection, pavement preservation methods, specifications, materials selection, construction procedures, and QA/QC for GDOT to cost-effectively preserve its pavements. The treatment selection function provides GDOT's Office of Maintenance the recommended and optional treatment methods based on the input of the pavement distress conditions. The GDOT criteria decision tree for realizing the automatic treatment selection is attached at the end of this appendix (shown in Table 66 and Table 67). This appendix presents the user tutorial for the PPIT.

PPIT WEBSITE USER INTERFACE

The user interface of the developed web-based Pavement Preservation Interactive Tool (PPIT) is illustrated using multiple screenshots, as shown in Figure 201 to Figure 205 below.

- Figure 201 shows the home page screenshot of the PPIT. Four major functions, 1) asphalt pavement treatments introduction, 2) asphalt pavement treatment selection, 3) concrete pavement treatments introduction, 4) concrete pavement treatment selection, can be accessed by either Single Click in the "Side Navigation Bar" or "Image Navigation Button." Let us click "Asphalt Pavement Treatments" to continue our tutorial.

Figure 201. Photo. Home Page of PPIT Website

- Figure 202 shows the introduction of the asphalt pavement treatment methods, including chip seal, crack sealing, fog seal, micro-milling, micro-surfacing, thin overlay, white topping, pothole patching, open-graded interlayer, full-depth reclamation, hot in-place recycling, cold in-place recycling, and ultra-thin bounded wearing course. Single Click the image of each treatment method to find detailed information about the treatment method. Let us click "Chip Seal" to continue our tutorial.

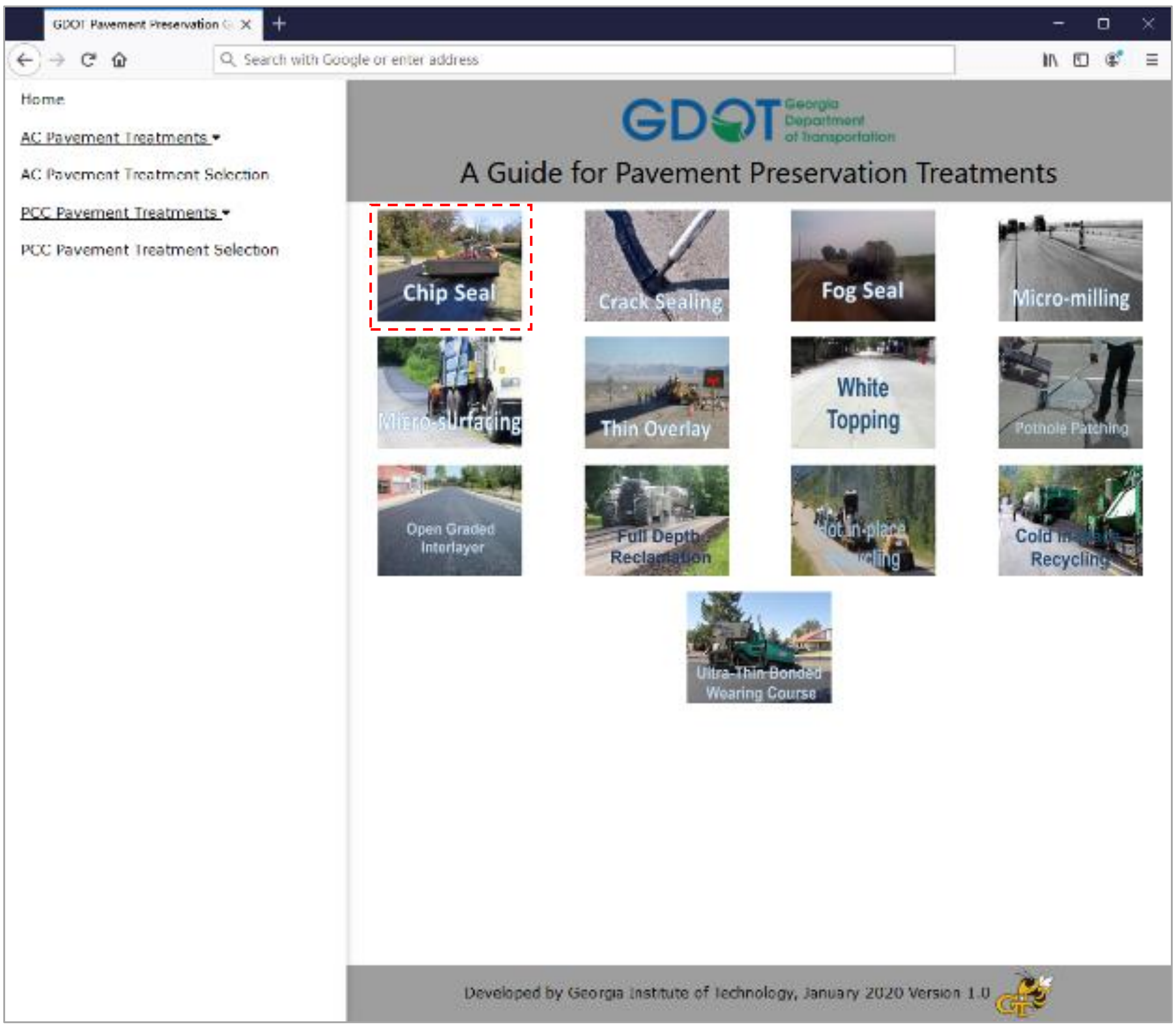


Figure 202. Photo. Asphalt Pavement Treatment Methods

- Figure 203 shows detailed information about chip seal treatment. The “Content Navigation Tag” contains the following subsections. Single Click each of the tags to switch the content.
    - “General”: the overview of this pavement preservation treatments method.
    - “Project Selection”: introduces how to decide the candidates for this method. Some images are provided to show good and poor candidates.
    - “Material”: introduces the material used for this treatment method.
    - “Construction”: introduces the construction procedures. Videos provide the detailed steps of construction.
    - “Performance”: introduces the performance of the treatment method on different distresses.
    - “Standards\References”: includes references to GDOT’s specifications and the details of the materials and the references.

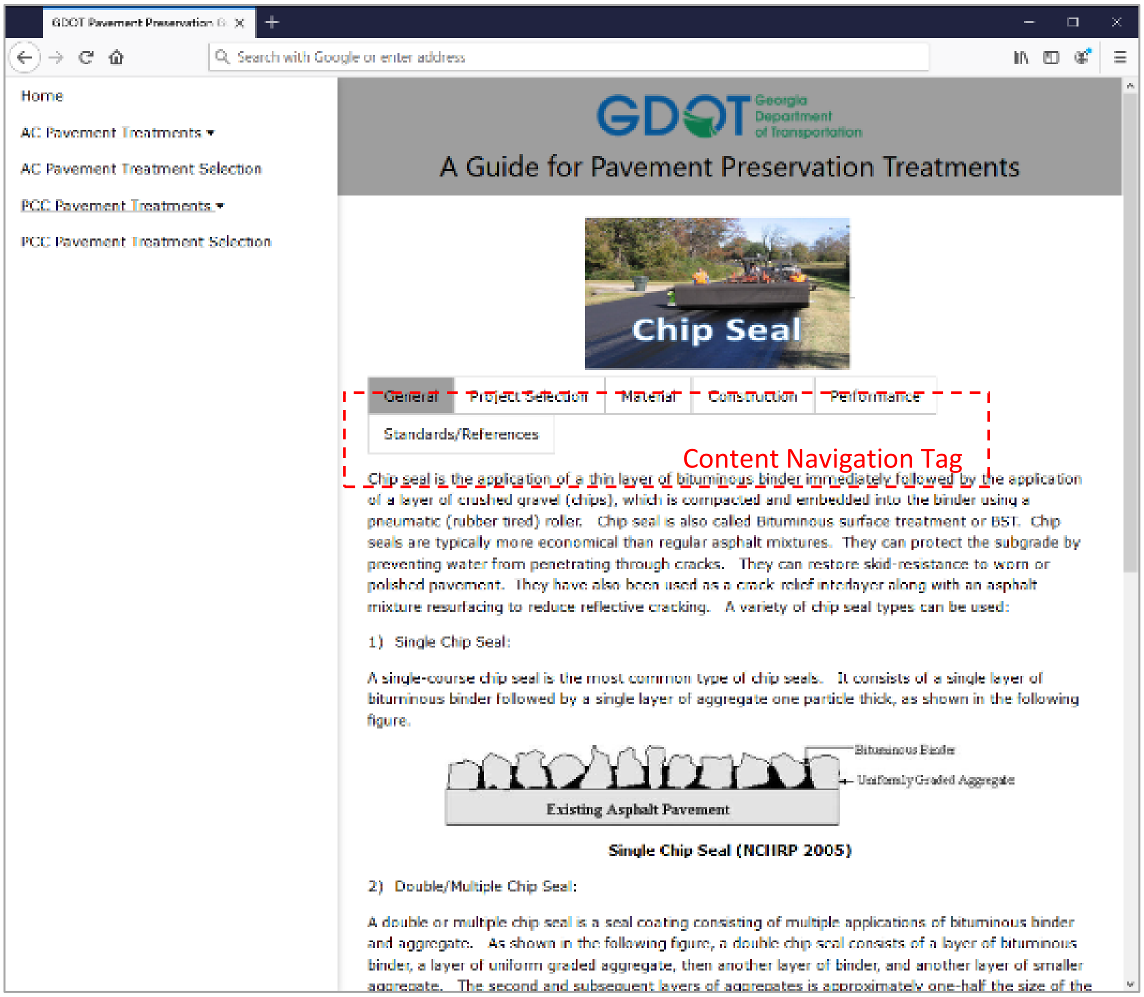


Figure 203. Photo. Content Page of Chip Seal Treatment

- Now, Single Click the "Home" button at the top of the "Side Navigation Bar" to return to the home page of the PPIT website. Then, <u>Single Click</u> "Asphalt Pavement Treatment Selection" to access the treatment selection function.
- Figure 204 shows the website page of the "Asphalt Pavement Treatment Selection" function. Use the checkbox to input pavement conditions (including, rutting, raveling, cracking, bleeding/flushing, corrugation/pushing, loss of section, edge distress, patches/potholes/local

base failure), Annual Average Daily Traffic (AADT) and truck percentage. Let us check "Load Crack Severity Level 2" to continue the tutorial.

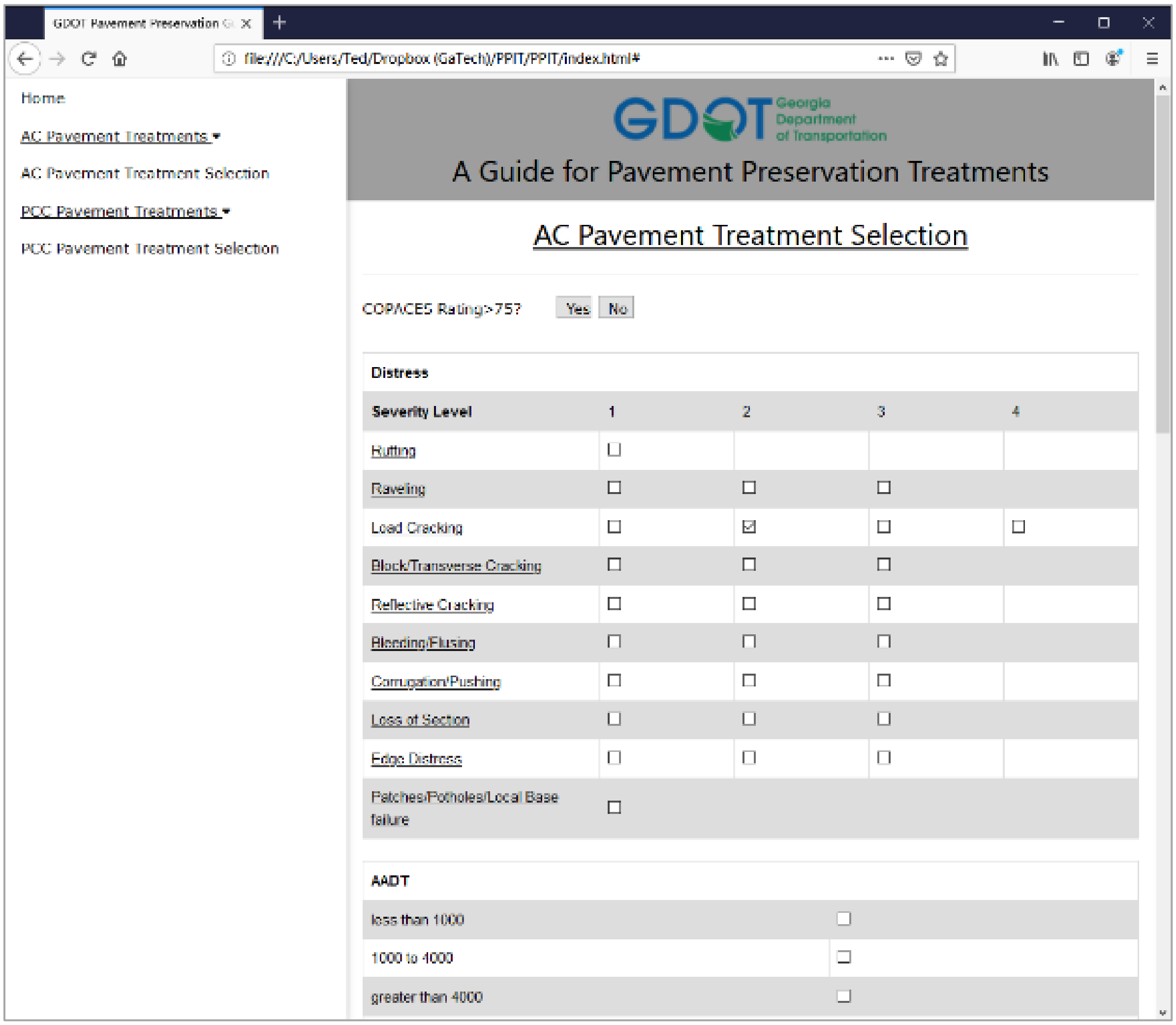


Figure 204. Photo. Website Page of Asphalt Pavement Treatment Selection

- Then, Single Click "Select Treatment" to select the adequate treatment. Figure 205 shows the recommended preservation methods corresponding to the "Severity Level 2 Load Crack" as the pavement condition inputs.

Figure 205. Photo. Treatment Selection Outcome

DATABASE FOLDER STRUCTURE OF PPIT

The database folder structure of the PPIT is shown in Figure 206 The illustration of each folder and file is described below:

- *Bootstrap-3.3.7-dist:* CSS files that construct the pattern and structure of the website.
- *Decisionmatrix:* The original decision matrix provided by GDOT to show the appropriate method for each distress.
- *Embed:* All the pages except index.
- *Font-awesome-4.7.0:* CSS files that construct the pattern and structure of the website.
- *Images:* All the images needed on the web site.
- *PACESManual:* Original files and images. Just used as backup.
- *Specs:* All the pdf files of papers. There are some pages that have the links to them.
- *Videos:* All the videos needed on the web site.

- *index.html:* The main page of the web site; from this page, you can link to all the information you want, so open this page first.

If you want to run the PPIT tool locally, please run *index.html* using Firefox explore. Remember to enable CORS (google "how to enable CORS of Firefox explore"); if the website does not work, click any button on the home page of the PPIT website.

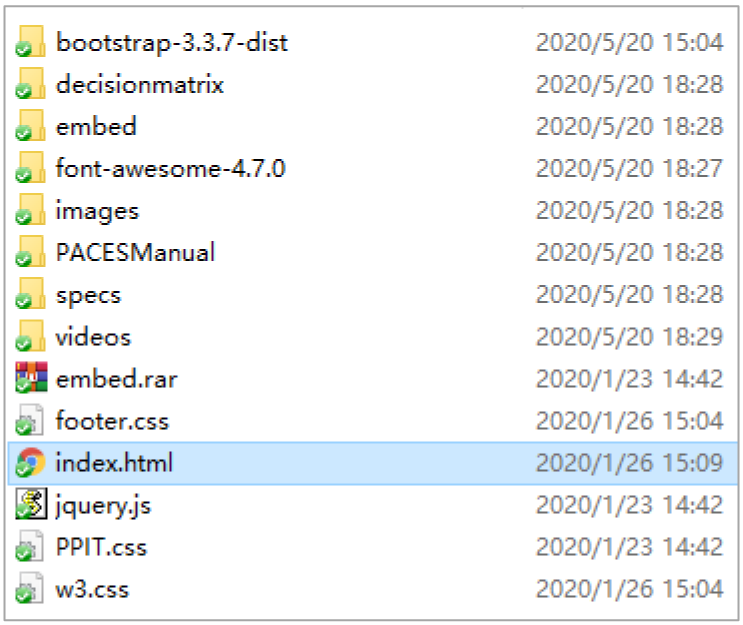


Figure 206. Photo. Folder of PPIT

Table 66. GDOT AC Pavement Treatment Selection Guidelines

Georgia's Guidelines for Pavement Treatment Selection

Pavement Conditions	Specifications/ Parameters	Fog Seal	Crack Seal	Scrub Seal	Std. Chip Seal	Interlayers		Strip Seal	Slurry Seal	Micro Surfacing	Ultrathin Bonded Wearing Course	Recycled Bituminous Pavement			Mill and Inlay	Hot Mix Overlay	Thin Hot Mix Overlay	Deep Base Repair	Shoulder Widening	Spray Injection Patching	Spot Overlay	Spot Mill Overlay	White Topping	Conventional Milling	Roto Milling	Micro Milling	Mastic Patching	Soil Stabilization
						SSI	OGI					FDR	HIR	CIR														
Traffic (ADT) (Note % truck should be considered)	<1000	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲
	1000-4000	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲
	>4000	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲	▲
Rut Depth	Isolated location	x	x	x	x	x	x	x	▲	▲	▲	x	▲	▲	▲	▲	▲	x	x	x	●	●	●	●	x	x	x	x
Load Cracking	Level 1	▲	●	●	●	●	●	●	●	●	●	x	x	x	▲	▲	x	x	x	x	x	x	x	x	x	x	x	x
	Level 2	x	●	●	▲	●	x	▲	▲	▲	▲	x	x	▲	▲	x	x	▲	x	x	x	x	x	x	x	x	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	●	x	●	●	x	x	●	x	x	x	x	x	x	x	x	x	x
Reflection Cracking	Level 1	x	●	●	●	●	●	x	▲	▲	●	x	▲	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	Level 2	x	●	▲	▲	▲	▲	x	▲	▲	●	x	▲	▲	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	●	●	●	x	x	x	x	x	x	x	x	x	●	x	x	●	x
Block/ Transverse Cracking	Level 1	x	▲	x	x	●	x	x	x	x	x	x	●	●	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	Level 2	x	●	▲	●	▲	x	x	x	x	x	x	●	●	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	Level 3	x	x	●	▲	x	x	x	x	x	x	●	●	●	x	x	x	x	x	x	x	x	x	x	x	x	x	x
Bleeding/ Flushing	Level 1	x	x	x	x	x	x	x	●	●	x	x	●	x	●	▲	▲	x	x	▲	●	▲	x	x	x	x	x	x
	Level 2	x	x	x	x	x	x	x	●	●	x	x	●	x	●	▲	▲	x	x	▲	●	▲	x	x	x	x	x	x
	Level 3	x	x	x	x	x	x	x	●	●	x	x	●	●	●	▲	▲	x	x	▲	●	▲	x	x	x	x	x	x
Raveling	Level 1	●	▲	x	x	x	x	x	▲	▲	x	x	●	x	x	x	x	x	x	x	x	x	x	x	x	x	x	x
	Level 2	x	▲	x	x	x	x	x	▲	●	x	x	●	x	●	▲	▲	x	x	▲	●	▲	x	x	x	●	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	x	●	●	●	▲	▲	x	x	▲	●	▲	x	x	x	●	x	x
Edge Distress	Level 1	x	●	x	x	x	x	x	x	x	x	x	●	x	x	x	x	x	●	x	x	x	x	x	x	x	x	x
	Level 2	x	x	x	x	x	x	x	x	x	x	x	●	x	●	▲	▲	●	●	x	x	x	x	x	x	x	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	x	●	x	●	▲	▲	●	●	x	x	x	x	x	x	x	x	x
Corrugation /Pushing	Level 1	x	●	x	x	x	x	x	x	x	x	x	●	x	●	●	▲	x	x	x	x	x	x	●	x	x	x	x
	Level 2	x	x	x	x	x	x	x	x	x	x	x	●	x	●	●	▲	x	x	x	x	x	x	●	x	x	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	x	●	x	●	●	▲	x	x	x	x	x	x	●	x	x	x	x
Loss of Section	Level 1	x	●	x	x	x	x	x	x	x	x	▲	x	▲	▲	●	▲	x	x	x	x	x	x	x	x	x	x	x
	Level 2	x	●	x	x	x	x	x	x	x	x	●	x	●	▲	●	▲	x	x	x	x	x	x	x	x	x	x	x
	Level 3	x	x	x	x	x	x	x	x	x	x	●	x	●	●	▲	▲	x	x	x	x	x	x	x	x	x	x	●
Patches/ Potholes/ Local Base Failures	Present	x	x	x	x	x	x	x	x	x	x	●	x	●	x	x	x	●	x	●	●	▲	x	x	x	x	●	x

Legend: x = NOT RECOMMENDED ● = RECOMMENDED ▲ = OPTIONAL

(Table courtesy of GDOT)

Table 67. GDOT PCC Pavement Treatment Selection Guidelines

		GUIDELINES FOR PCC PAVEMENT TREATMENT SELECTION							
	PCC TREATMENTS								
	Specifications								
PAVEMENT CONDITIONS	PARAMETERS	Joint Sealing	Diamond Grinding	Soil Stabilization	Crack Sealing	Mastic Patching	Spall Repair	Full Depth Repair	
FAULTING		•	•	•	X	?	X	•	
TRANSVERSE CRACKING	Level 1	X	X	X	•	X	X	X	
TRANSVERSE CRACKING	Level 2	X	?	X	•	•	X	?	
LONGITUDINAL CRACKING	Level 1	X	X	X	•	X	X	X	
LONGITUDINAL CRACKING	Level 2	X	?	X	•	•	X	?	
CORNER BREAKS		X	X	X	?	?	•	•	
SHATTERED SLAB		X	?	X	X	X	X	•	
REPLACED SLABS		?	?	X	?	X	X	•	
FAILED SLABS		?	?	X	X	X	X	•	
JOINTS W/SPALL		X	X	X	X	?	•	X	
JOINTS W/PATCHED SPALL		?	X	X	X	?	?	•	
JOINTS W/FAILED SPALL		?	X	X	X	?	•	•	
SHOULDER JOINT DISTRESS		•	X	•	X	X	X	X	
ROUGHNESS		X	•	X	X	X	X	?	

Recommended	•
May be recommended	?
Not recommended	X

(Table courtesy of GDOT)

APPENDIX B. GDOT PAVEMENT CONDITION EVALUATION SYSTEM (PACES) MANUAL

BASICS OF THE SYSTEM

Introduction

The Pavement Condition Evaluation System (PACES) is designed to indicate the amount and type of surface distress on a roadway at the time the survey is made. The system standardizes the terminology for the types of defects that can be found on a pavement in Georgia and defines the various levels of severity for these defects. This system will allow roads to be rated objectively statewide.

This system only addresses the structural condition of the pavement surface. It does not include skid resistance and ride-ability because these will be measured with high speed testing equipment.

Outline

A number of distresses have been identified for flexible pavement and surface treatment which relate to the performance of the pavement. Both the presence of these distresses and the severity levels must be taken into account when rating a pavement. These distresses are as follows (also see distress definitions):

Rut Depth	*Raveling*
Load Cracking	*Edge Distress*
Block Cracking	*Bleeding/Flushing*
Reflection Cracking	*Corrugations/Pushing*
Patches and Potholes	*Loss of Section*

There are other types of defects which are not being considered either because they occur infrequently, or they are included in one of the above categories at a certain severity level.

Transverse cracking, for instance, is considered to be an initial stage of block cracking and is, therefore, rated in that category.

Rating is done for a particular segment by selecting a sample section for cracking distresses representative of the pavement condition for that rating segment. The defects noted for each rating segment. The defects noted for each rating segment within a project are then averaged to obtain the representative pavement condition for that project. A project rating is determined from deduct values, which have been established for each defect and severity level.

Flexible Pavement Distress Definitions

The various pavement distresses are defined in this section along with descriptions and illustrations of the various levels of severity for each distress. The rater must be thoroughly familiar with the distresses and severity levels as defined in this section. The rater may or may not agree with all of the definitions and descriptions presented in this section, but all pavement sections must be rated in accordance with the criteria presented here in order for the rating system to be uniform.

Illustrations and photographs are shown for the various distresses and the severity levels which represent what typically might be seen in the field. The illustrations do not show all conditions that might be found, nor is it intended that a condition must look exactly like what is shown in this manual for it to be rated at a particular severity level. The pictures are simple illustrations of what the rater is likely to see for a certain distress at a certain severity level. The rater must use his judgment based on the descriptions and the pictures while classifying the distress and severity level found on a sample section. It is possible that a combination of distresses and severity levels are present and these combinations should be recorded on the survey sheet as they exist.

Rut Depth

Definition:

Rutting is longitudinal depressions that form under traffic in the wheelpaths and are greater than 20 feet long. Rutting is a permanent deformation of the wheelpaths caused by traffic loadings. Rutting can be caused by insufficient compaction, plastic movement of the mix, or an unstable foundation.

According to its definition, the following questions may be addressed when identifying rutting distress:

1. Compared with the pavement outside the wheelpaths, is there any depression deformation in the wheelpaths?

2. Is this deformation in the longitudinal direction?

3. Is there any cracking in the longitudinal direction which is associated with the deformation in the wheelpaths?

How to Measure:

Rut Depth will be estimated in both wheelpaths in the sample area and recorded on the survey in units of 1/8 of an inch. If rutting is extensive (more than 3/8 inch), actual measurement may be necessary.

Severity levels for Rutting are not applicable.

Load Cracking

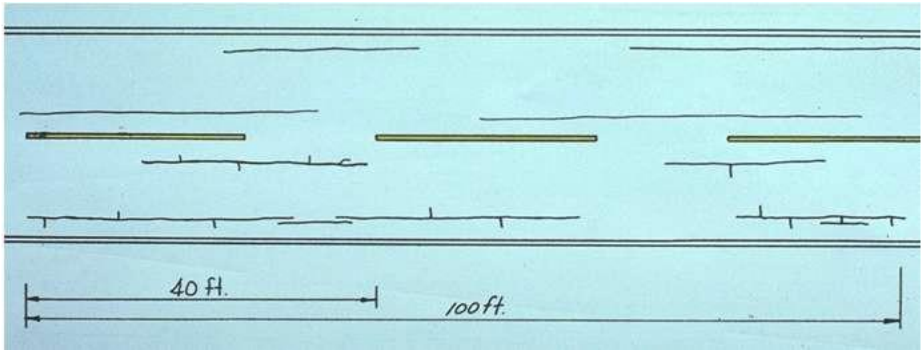
Description:

This type of cracking is caused by repeated heavy loads and always occurs in the wheelpaths. This type cracking usually starts as single longitudinal cracks in the wheelpaths. As progression continues, short transverse cracks occur that intersect the original longitudinal cracks. Additional longitudinal cracks occur in the wheelpaths. As the number of longitudinal and transverse cracks in the wheelpaths increases, polygons are formed by the intersection of these cracks. As deterioration continues, these polygons become smaller (due to additional cracking) and, in the worst case, begin to pop out. When load cracking progresses to the point where small polygons are formed, rutting can become extensive and pumping of base material can occur.

Following are examples for each severity level of load cracking.

Severity Level 1 Load Cracking

Level 1 Load Crack patterns are generally tight single longitudinal cracks in the wheelpaths. A wheelpath is approximately 3 feet wide and load cracking can occur at the edge of the wheelpath. Occasional short, tight longitudinal cracks parallel to the main longitudinal cracks can also occur and still be defined as a level 1 load cracking pattern.

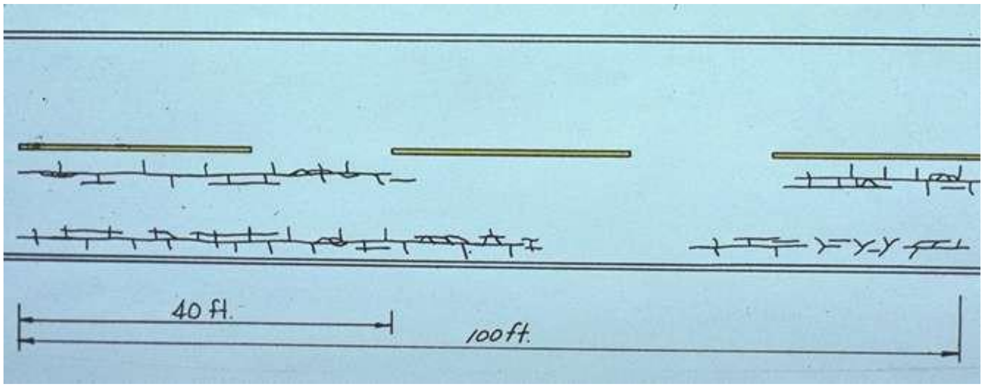


The illustration above is an example of the range in load cracking patterns to be recorded as level 1 load cracking. There is approximately 120 feet of cracking in the two wheelpaths, or 60% of the sample in lane one in the two wheelpaths, or 60% of the sample in lane one, and 130 feet (65%) of level 1 load cracking in lane two.

Examples of Level 1 Load Cracking

Severity Level 2 Load Cracking

The following illustration shows the general range in appearance of level 2 load cracking patterns. These cracks are wider than level 1cracks and occur only in the wheelpaths. This level cracking has a single or double longitudinal crack with a much larger number of 0-2 ft. transverse cracks intersecting than in level 1 load cracking. Occasionally, polygons will form but are not prominent.

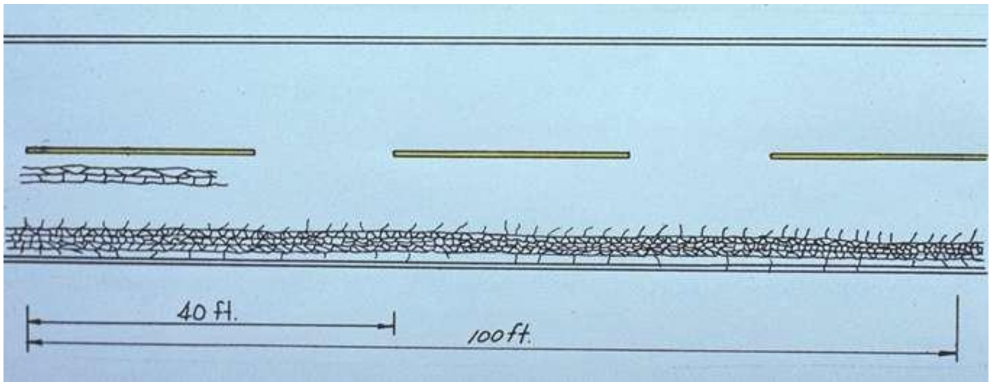


In this example, there is approximately 150 ft. of cracking in the 100 ft. sample area, or 75% of the sample area.

Examples of Level 2 Load Cracking

Severity Level 3 Load Cracking

The illustration shows the general appearance of level 3 load cracking patterns. This type pattern generally has three or more longitudinal cracks in the wheelpaths with many interconnecting transverse cracks. Many small polygons are formed causing the appearance of "alligator hide." This type cracking is marked by a definite, extensive pattern of small polygons and is sometimes accompanied by severe rutting.

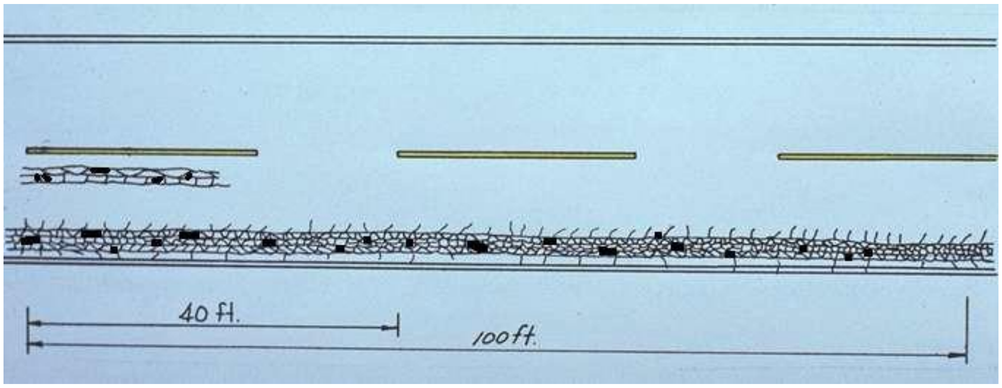


In this example, 60% of the sample has level 3 load cracking.

Examples of Level 3 Load Cracking

Severity Level 4 Load Cracking

The following illustration shows level 4 load cracking patterns. This type pattern has the definite "Alligator hide" pattern but has deteriorated to the point that the small polygons are beginning to pop out. Rutting is usually severe and pumping of base material is sometimes evident.



In this example, 60% of the sample area has level 4 load cracking.

Examples of Level 4 Load Cracking

Block/Transverse Cracking

This type cracking is caused by weathering of the pavement or shrinkage of cement treated base materials. Block/transverse cracking is not load related. The block pattern is distributed uniformly throughout the roadway and not concentrated in the wheelpaths. Block cracking is interconnecting cracks forming a series of large blocks usually with sharp corners.

Block/transverse cracking begins as single, tight transverse, longitudinal or combinations of both types of cracks. In the beginning, block/transverse cracks may not form a recognizable block pattern, just longitudinal and/or transverse cracks that are not associated with the wheelpaths.

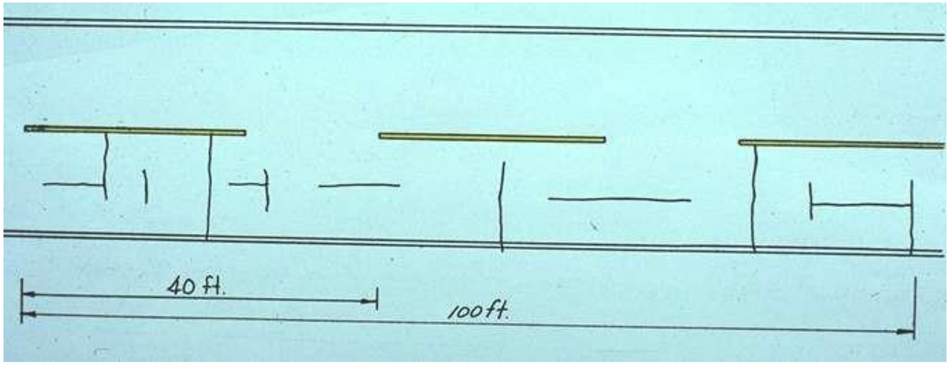
As this type of cracking progresses, a definite block pattern occurs, and the cracks become wider. As the cracking becomes worse, the block pattern densifies (small blocks) and/or the cracks become very wide (> 1/8 inch).

Severity Level 1 Block/Transverse Cracking

This type cracking is not load related and does not occur in the wheelpaths. Level 1 block/transverse cracking is made up of transverse, longitudinal, or a combination of both types of cracks. A definite "block" pattern has not developed yet. The longitudinal cracks are tight and not in the wheelpaths, although they may wander into the wheelpaths at times.

The illustration below would be considered to have level 1 block/transverse cracking on

100 percent of the sample area. See the Rule of 100 feet for estimating the extent of level 1 block/transverse cracking.

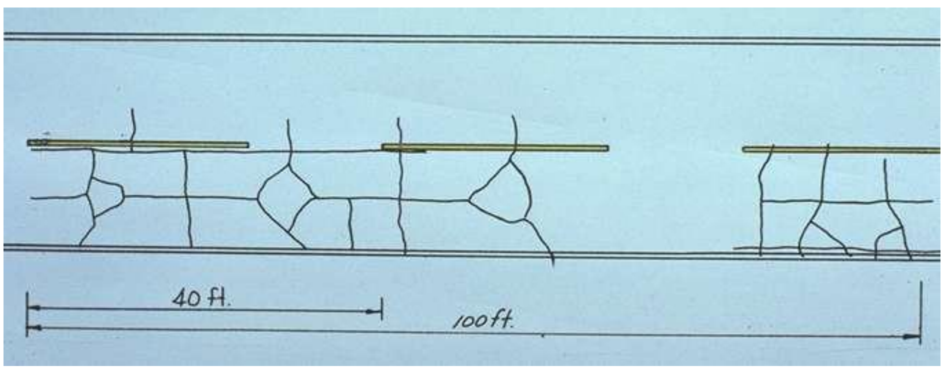


Examples of Level 1 Block/Transverse Cracking

Severity Level 2 Block/Transverse Cracking

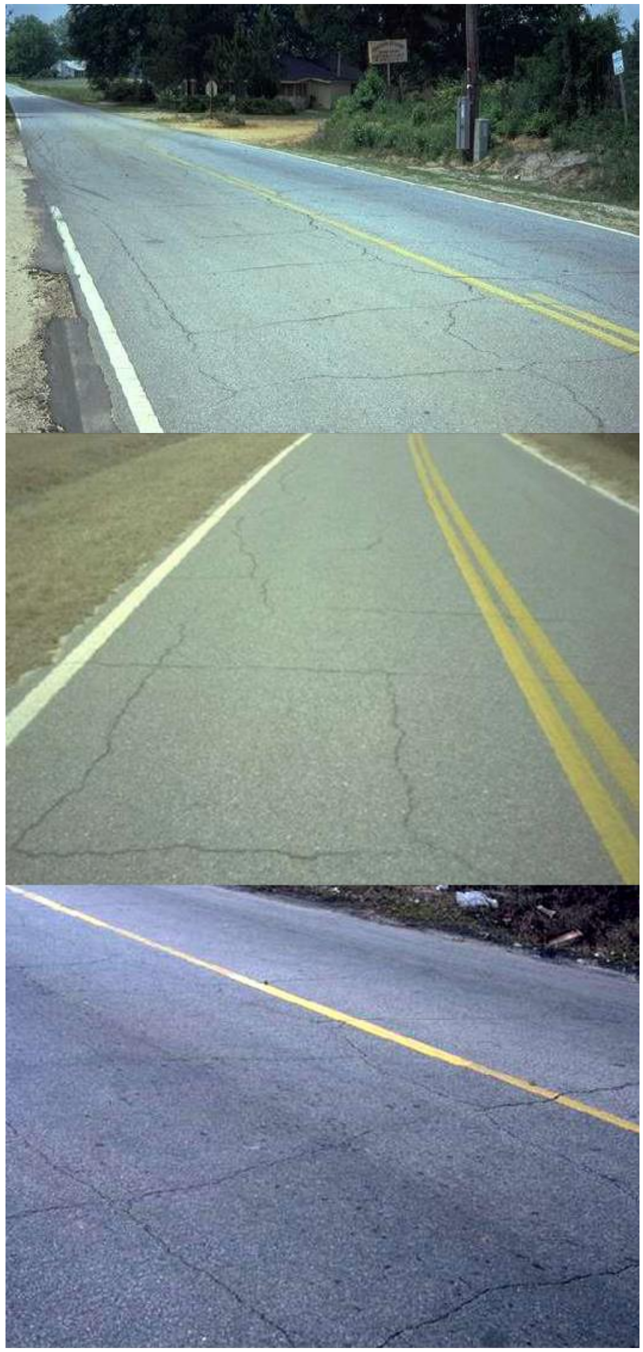
At this severity level, the cracking has developed definite block patterns. Some of the longitudinal cracks can occur in the wheelpaths for short distances without being considered load cracking when associated with a block pattern. The transverse and longitudinal cracks are wider than in level 1 but do not necessarily require sealing. The block pattern will usually be uniform across the entire roadway.

In the following example, 80% of the sample area has level 2 block/transverse cracking. However, this example is for illustrative purposes only. It is either present (100%) or not present (0%) and rarely falls in between.



Following are sample images of level 2 block/transverse cracking.

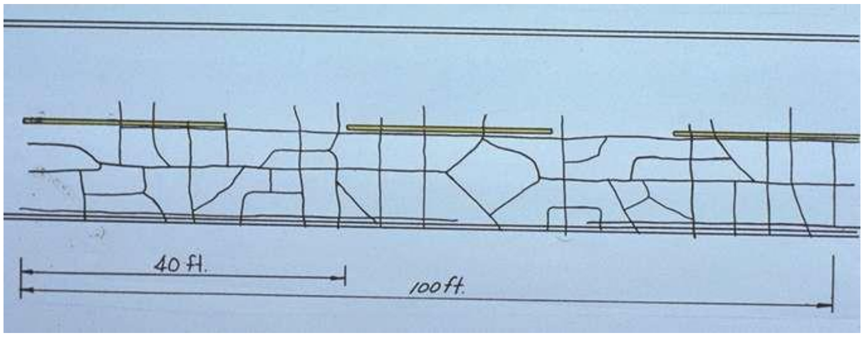
Examples of Level 2 Block/Transverse Cracking



Severity Level 3 Block/Transverse Cracking

At this severity level, the cracking has a definite block pattern as in level 2, but the blocks are smaller and the cracks are wider than in level 2. Level 3 block/transverse cracking is marked by cracks that are wide enough to require sealing. Block cracking that has a very large number of small blocks is also considered to be level 3 block/transverse cracking. Some of the longitudinal cracks in this pattern may meander into the wheelpaths for a short distance but are still considered block cracking because large distances of wheelpaths are not affected, and this type cracking is not caused by loading. Some spalling of the cracking may be evident.

The following example represents 100% level 3 block/transverse cracking:



Here are some sample images of level 3 block/transverse cracking.

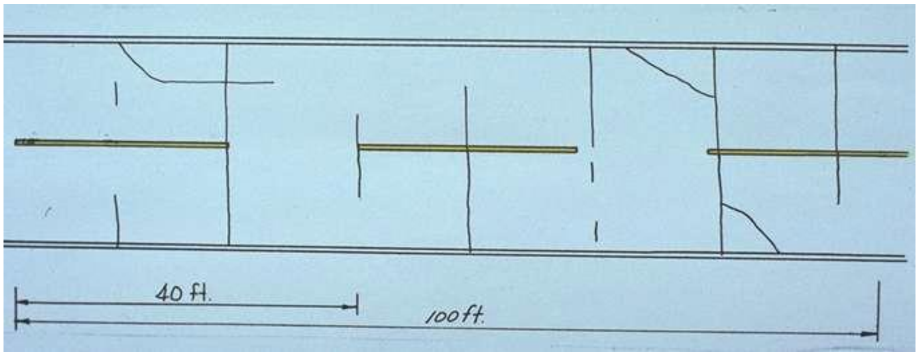
Level 3 block/transverse cracking – Notice width, subsidence and spalling of the cracks.

Reflection Cracking

This type cracking is caused by the “reflection” of joints and cracks through an asphaltic concrete overlay from the **underlying PCC concrete pavement**. These reflection cracks begin as tight cracks and progress to very wide cracks with spalling. The transverse cracks will be right angles across the roadway and in a repeatable pattern down the roadway (i.e., every 30 feet, 40 feet, etc.). The longitudinal cracks, if present, will normally be fairly straight, continuous cracks in the travel lane near the pavement edge associated with underlying edge of narrower PCC concrete pavement which has been widened and overlaid with asphaltic concrete overlay. Longitudinal cracks that occur at the centerline, lane lines, and edge lines are not to be counted. Any other cracks will be cracks associated with failures in underlying PCC concrete pavement, and such cracks will reflect the size and shape of such failures. Construction joints and widening joints associated with the construction and/or widening of asphalt pavements are not to be counted as reflection cracking. Reflection cracking only occurs on roadways with an underlying PCC concrete pavement.

Severity Level 1 Reflection Cracking

Level 1 Reflection cracks are tight, single cracks that are usually transverse but are longitudinal if the underlying concrete pavement is narrower than the asphaltic concrete overlay. Irregular cracking patterns can be reflected if the underlying slabs are broken. Level 1cracks may or may not go across the entire lane.



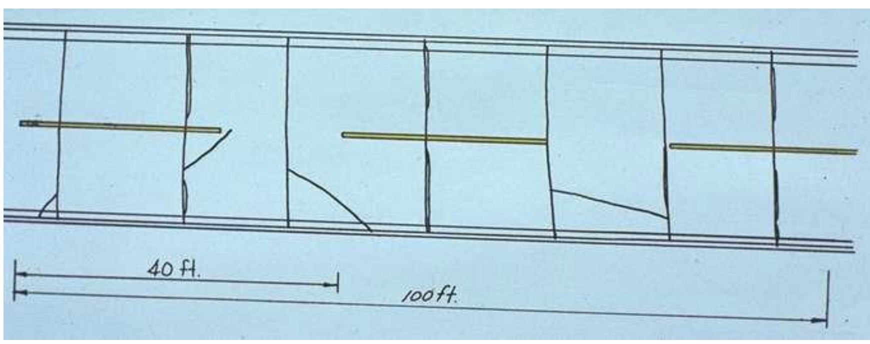
This illustration shows the top lane represents 9 total cracks and approximately 100 linear feet of reflection cracking.

Here are some sample images of level 1 reflection cracking:

Severity Level 2 Reflection Cracking

This level of reflection cracking has progresses so that all underlying joints and cracks have reflected through the surface layer. The cracks are substantially wider than the level

1 cracks and may require sealing. There may be some "double" cracks over the underlying concrete pavement joints. "Double" cracks are not to be counted as two cracks but as one reflection crack. A longitudinal crack in the travel lane near the edge of pavement that is a result of widening of the underlying concrete pavement with asphalt will be counted as a reflection crack. If the underlying pavement is not concrete, it should not be counted. A widening crack is not necessarily a reflection crack.



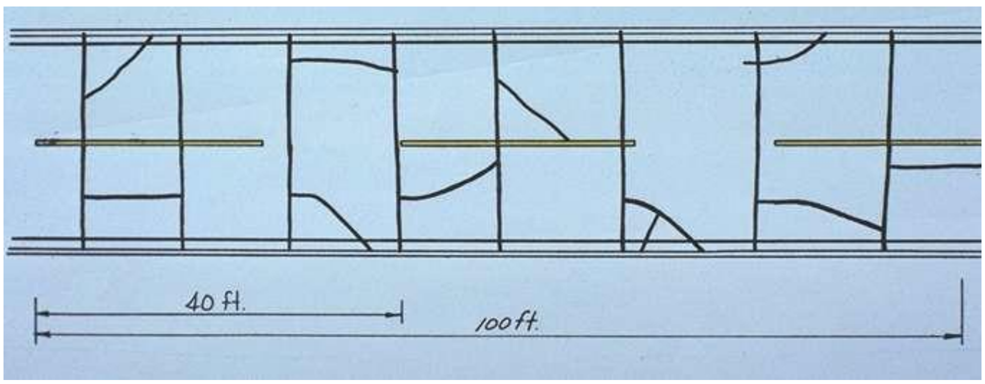
This illustration of the bottom lane represents 12 total cracks and approximately 230 linear feet of reflection cracking.

Here are some sample images of level 2 reflection cracking:

Level 2 Reflection.

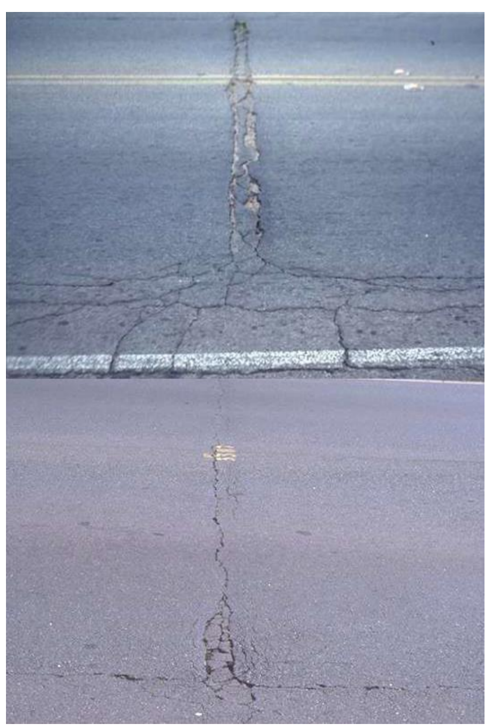
Severity Level 3 Reflection Cracking

This level of reflection cracking will have the same pattern as level 2 reflection cracking (all underlying joints and cracks have reflected through), but the cracks will be very wide. The cracks will be marked by spalling and/or subsidence. It should be obvious that some corrective work should be performed to these cracks before counting them as level 3.



This illustration shows the bottom lane represents 16 total cracks with approximately 280 linear feet of reflection cracking.

Here are some sample images of level 3 reflection cracking:



Raveling

Description:

This condition is the progressive disintegration of the pavement surface. It is caused by traffic action on a weak surface. Aggregate particles become dislodged from the binder, and this loss of material can progress through the entire layer. Raveling ranges in severity from the loss of a substantial number of surface stones to the loss of a substantial portion of the asphalt surface layer. For purposes of rating, a slurry seal that has "peeled off" is considered level 3 raveling.

How to measure:

The percent of the length of the rated segment that contains the raveling is to be recorded along with the predominant severity level. For example, if 300 feet of raveling in a 500 feet rating segment with 100 feet as level 1 severity and 200 feet at level 2 severity, it would be recorded as 60% level 2 raveling as 300/500 = 60% and level 2 is more predominant (200 ft. > 100 ft.).

Level 1 – loss of substantial number of stones

Level 2 – loss of most surface

Level 3 – loss of substantial portion of surface layer (>1/2 depth)

Edge Distress

Definition:

Edge distress is cracking and pavement edge break-off within 1 to 2 foot of the pavement edge and not associated with the wheelpath area. The cracking can be in the form of longitudinal or transverse cracks or in many instances alligator type cracking. It may sometimes be difficult to distinguish between alligator cracking in the wheelpath and along the edge of the pavement, especially on narrow width pavements. **It must be called load cracking when it occurs in the wheelpath. It cannot be called both load cracking and edge distress.**

How to Measure:

The percent of the length of the rated segment that contains the edge distress is to be recorded along with the predominant severity level in the rater's judgment. For example, if raveling is observed on these curves (curve 1 – 300' level 1, curve 2 – 100' level 3) within a rating segment, it would be recorded as 60% level 2 raveling as 200+100 = 300/500 = 60% and level 2 is more predominant (200 ft. > 100 ft.).

Level 1 – tight, hairline cracks

Level 2 – crack widths greater than 1/4 inch, double cracking, tight “alligator” cracking

Level 3 – severe “alligator” cracking at edge, popouts, edge break off

Bleeding/Flushing

Definition:

Bleeding or flushing is the presence of bituminous material on the surface creating a shiny appearance. Bleeding or flushing is created by excess asphalt cement and/or low air void content.

How to measure:

The percent of the length of the wheelpaths that has bleeding or flushing in the rated segment is noted. Each wheelpath is a maximum of 50 percent of the rated segment. For example, if one wheelpath is observed with 200 feet level 1 bleeding and the other wheelpath had 100 feet level 2 bleeding in a rated segment of 500 feet, it would be recorded as 60% level 1 bleeding/flushing since 200 ft. = 40% + 100 ft. = 20% or 60% total both wheelpaths and level 1 is more predominant (200’ > 100’).

Level 1 – free bitumen is noticeable on the surface along with the aggregate in the mix

Level 2 – surface is black with very little aggregate noticeable

Corrugation/Pushing

Definition:

A series of ridges and valleys in the surface which cause a rippling or washboarding effect caused by unstable asphalt on asphaltic concrete or non-uniform application of aggregate on surface treatment.

How to Measure:

The extent will be recorded as the percentage of the rated segment length that has corrugations. For example, if it is observed in a rating segment that three interchanges (No. 1 –100 feet level 1, No. 2 –300 feet level 2, and No. 3 –50 feet level 3) had corrugations/pushing, it would be recorded as 90% and level 2 corrugation/pushing, as 100 + 300 + 50 = 450 = 450/500 = 90% and level 2 is more predominant (300’ > 100’ > 50’).

Level 1 – corrugations are visible and can definitely be felt in the steering wheel while driving

Level 2 – corrugations/pushing have significant effect on riding comfort, some reduction of speed may be necessary

Level 3 – noticeable discomfort, excessive vibration, reduction of speed necessary

Loss of Section

Definition:

A deviation of the pavement surface from its original typical design cross section other than those described for corrugations, pushing or shoving. Generally, loss of pavement section results from settlement, slope failure, or heavy loads on a deficient pavement system. This loss of section usually occurs in the outside half of the lane. Loss of section takes the form of dips, bumps, and undulations, all of which cause pitch and role in a moving vehicle.

How to Measure:

The percentage of the length of rated segment that has loss of pavement section. The three severity levels are as follows:

Level 1 (Slight): Noticeable swaying of vehicle, but no effect on vehicle control.

Level 2 (Moderate): Heavy swaying of vehicle. Fair control of vehicle, driver has to anticipate dips ahead.

Level 3 (Severe): Speed of vehicle must be greatly reduced for driver to maintain control.

The predominant severity, in the rater's judgment, is also recorded on the rating form under severity. For example, in a rating segment of 500 feet , if 300 feet level 1 and 200 feet level 2 is observed, and it would be recorded as 100% level 1 loss of pavement section as 300 + 200 = 500/500 = 100% and level 1 is more predominant (300' > 200').

Level 1 (slight) – Noticeable swaying of vehicle, but no effect on vehicle control.

Level 2 (Moderate) – Heavy swaying of vehicle. Fair control of vehicle, driver has to anticipate dips ahead.

Level 3 (Severe) – Speed of vehicle must be greatly reduced for driver to maintain control.

Patches, Potholes, and Local Base Failures

Definition

Patches are repaired sections of the asphalt pavement due to localized pavement and/or base failures. Also, spot leveling is included in this category.

Potholes are sections of the asphalt pavement that have failed and formed a hole in the pavement structure. These are caused by pavement and/or base failures.

Base Failures are sections of roadway where the water has entered the base material and is rutting and shoving.

How to measure

The total number of spot overlays, patches, potholes, and local base failures must be counted for the entire rated segment and this number recorded on the COPACES survey form. Utility patches are NOT to be counted UNLESS the utility patch has failed and is affecting the structural condition of the pavement.

RATING SURVEY

Introduction

Ratings are done for each mile (or partial mile) by selecting a sample section for cracking distresses representative of the pavement condition for that rating segment. The defects noted for each rating segment within a project are then averaged to obtain the representative pavement condition for that project. A project rating is then determined from deduct values which have been established for each defect and severity level.

Conducting the Survey

The rating system has been devised so that the pavement condition of all the state routes can be assessed objectively by one rater within his assigned area on a project basis. It is suggested that the rater rates all the state routes within one given area at a time to reduce driving time between routes and to keep track of what areas have been surveyed. It would be helpful to mark on a map these routes that have been completed.

The rater must start the route at the beginning or ending point of the selected project limits and continue the rating process until the other end is reached. Do not survey any route by starting in the middle and do not branch off to another route prior to completing the route that is being rated. For roads with more than one road number, the lowest number description should be used. A programmable Distance Measuring instrument connected to the survey vehicle should be used when conducting the survey to accurately identify the location of each survey site.

Project Limit Selection

Pavement condition ratings will be obtained on all state routes. The roads will be divided into projects for analysis of the data.

A project is a length of roadway with a common pavement section, similar structural conditions, and logical beginning and ending points. Project limits should not be mileposts, businesses or other points not readily located on the map. The rater must choose the project limits when conducting the inventory ratings within the guidelines described in this section.

The following must be used as break points for project limits:

1. Major changes in pavement condition for more than two consecutive miles.

2. Change in pavement type (not including spot overlays).

3. Common sections containing more than one state route – rate the section as lowest numbered state route.

4. Divided highways. Divided highway projects will have two files per segment of roadway with the only difference in the files being that the mileposts will be in the positive direction for one and in the negative direction for the other.

The following break points can also be considered for project limits but are not required:

1. Intersecting State Routes

2. County Lines – If the pavement type does not change at the county line, consideration should be given to extending the project limits into the adjoining county.

3. City Limits

4. Changes in the number of lanes

5. Curb and gutter sections through a city or town

6. Project limits established from previous resurfacing projects

7. Original construction project limits.

Other local factors may be known to the rater which can be helpful in establishing a project limit. Once the project limits have been established during the initial rating survey, these same limits should be used during all follow-up ratings as long as structural condition remains similar within the project limits. If structural conditions changes, subdivide into 2 or more projects, as appropriate. As a “Rule of Thumb”, whenever the project limit selected is longer than 10 miles, double-check to be sure that the pavement conditions are basically similar within the limits selected. If not, separate into two or more projects.

Selection of Rating Segments

A project will normally be divided into one mile segments for rating purposes. Exceptions to this are the beginning and ending segments of a project which can be less than one mile or when drastic changes in pavement conditions occur within the mile and shorter rating segments (usually ½ mile) are used to get a more representative rating of pavement conditions, especially cracking distress. The project limits within a city also will generally be shorter in length because of changes in pavement type, number of lanes, etc. As pavement conditions normally vary greatly within short distances, the rating segment likewise will be reduced to ½ mile or less to insure getting a representative rating of pavement condition.

Selection Sample Location for Cracking Distress

In rating cracking distress (load cracking, block/transverse cracking and reflection cracking) only a 100-foot sample out of each rating segment (mile or partial mile) will be rated so it is very important that the 100-foot section represent the majority of the cracking distress found in the rating segment (normally a mile). The 100-foot section will be chosen by the rater and can be located anywhere within the rating segment. The rater should drive slowly and make two or three stops within the first half of the rating segment and look at the pavement from the car to determine what types of cracking distress and level of severity is generally present. The 100-foot sample section should be selected only after the rater is confident that he has a "feel" for the pavement condition and can select his 100-foot sample section that is representative of the cracking distress within the segment to be rated. On projects where conditions are uniform, the pavement condition may be obvious after the first stop and the sample section could be chosen early in the rating segment. On projects with variable conditions, the sample section should normally be located at the half way point or beyond in the rating segment to be sure it is "representative" of the cracking distress. If pavement conditions change drastically within the segment being rated (normally a mile), the rating segment should be broken into two or more segments and 100-foot sample locations chosen to represent the smaller segments. For instance, two 100-foot sample sections could be chosen, each representing ½ mile.

For projects with cracking distress that vary widely within each rating segments, the 100-foot sample should represent the average conditions within the segment rather than the best or worst general conditions. For example, if in a given mile there is a substantial amount of cut areas, and the fill areas are the worse general condition, the 100-foot sample should be chosen to represent the entire mile, not in the best or worst area.

The rater should not locate the 100 foot samples over culverts, bridge approaches or locations that are obviously localized problems. Localized problems should be handled in “remarks”.

The purpose of the survey is to obtain a representative rating of the project pavement condition, especially cracking distress.

Rating the Sample Section for Cracking Distress

The Computerized Pavement Condition Evaluation System program (COPACES) is the computer program that GDOT uses to administer PACES. All project file and survey information is entered for the project while the rater is conducting the survey via a laptop computer or a tablet computer.

Once the 100-foot sample section has been selected to represent the cracking distress, its location will be recorded in COPACES program under the field “sample location”. This field locates the sample section to the nearest one-tenth mile.

The rater must walk the 100-foot section (three centerline stripes plus 10 feet is approximately 100 feet) and rate the lane in the worst condition on two lane and multilane undivided roads. On divided highways each travel direction must be rated separately, although only the lane in the worst condition in each travel direction is rated.

It is generally best to walk the 100 feet in one direction and determine which lane is in the worst cracking distress condition and rate this lane when walking back towards the vehicle. The rater must be aware that certain conditions such as time of the day, sunlight, and wetness can affect his ability to seen certain cracking distress conditions. The amount (to the nearest 5%) and severity of the cracking distress is estimated to the rater’s best judgment in accordance with the procedures contained in Chapter 1 of this manual and immediately recorded in COPACES

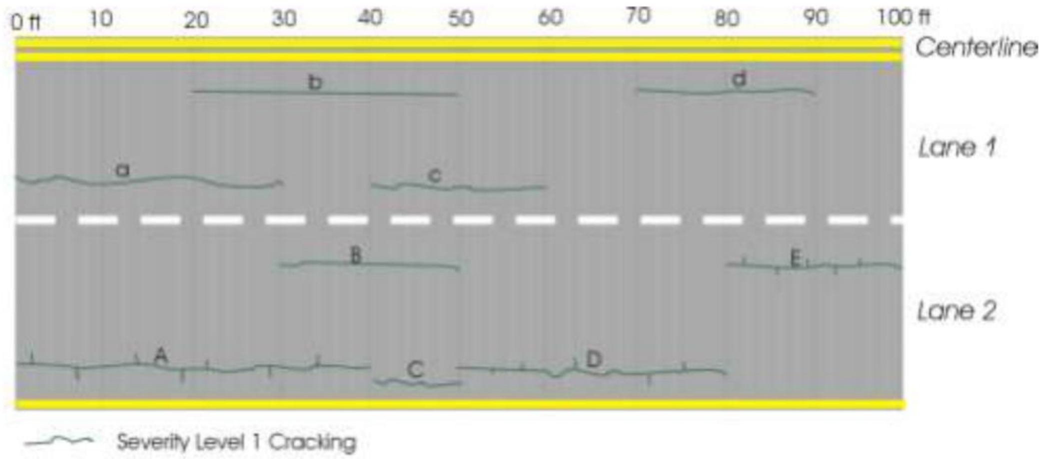
Selecting and Rating the Remaining Types of pavement Distress

As cracking distress is the most critical type of pavement distress, the length of the rating segment (mile or partial mile) will be determined when rating cracking distress as described earlier.

The remaining types of pavement distresses (Rut Depth, Raveling, Edge Distress, Bleeding/Flushing, Corrugation/Pushing, and Loss of Pavement Section) are to be closely observed and an estimate (to the nearest 5%) made of the extent and the predominant severity of the distress within the rating segment.

On two-lane and multi-lane undivided highways, the rater should determine which lane is in the worst general shape (from the standpoint of Raveling, Bleeding/Flushing, Corrugation/Pushing, Edge Distress, and Loss of Pavement Section) and base his estimate of the extent and severity of such pavement distress on what is observed in the lane selected. Likewise, on divided highways, only the lane in the worst condition in a given direction is to be rated, but rate both directions separately when rating divided highways (i.e., a separate report is to be prepared for each direction of a divided highway).

An exception to rating only the worst lane is to be made when rating patches, potholes, or local base failures as the total number of such distress for ALL LANES within the rating segment is to be recorded.



Lane 1 Survey

a : 30 ft
b : 30 ft
c : 20 ft
d : 20 ft

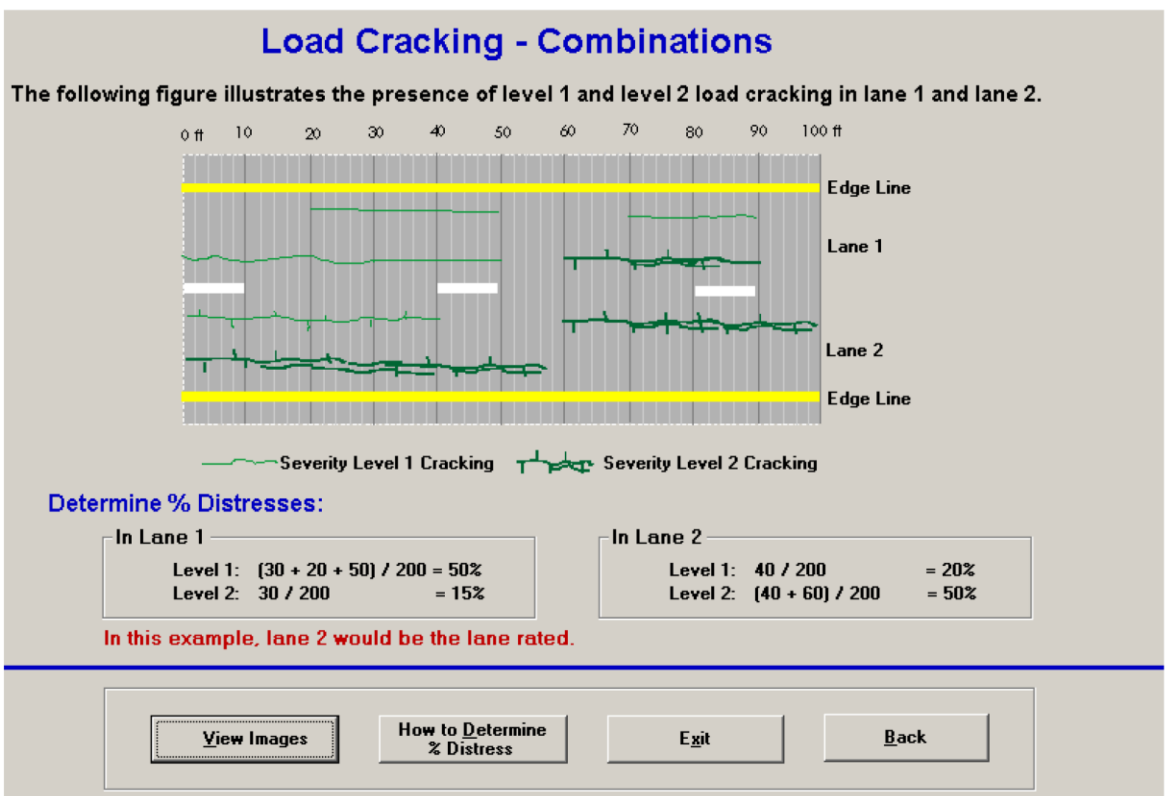
Total cracking in two wheelpaths: 100 ft
Percentage of Level 1 Cracking : 100 / 200 = 50 %

Lane 2 Survey

A : 40 ft
B : 20 ft
C : 10 ft
D : 30 ft
E : 20 ft

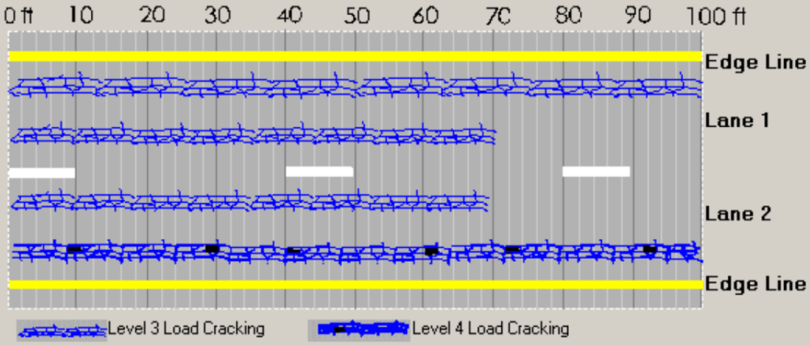
Total cracking in two wheelpaths: 120 ft
Percentage of Level 1 Cracking : 100 / 200 = 60 %

The higher percentage lane, 60 % of severity level 1 cracking in lane 2, is used to represent cracking in this section.



Load Cracking - Combinations

The following figure illustrates the presence of level 3 load cracking in lane 1 and level 3 and level 4 load cracking in lane 2.



Determine % Distresses:

In Lane 1

Level 3:	(70 + 100) / 200	= 85%
Level 4:	0 / 200	= 0%

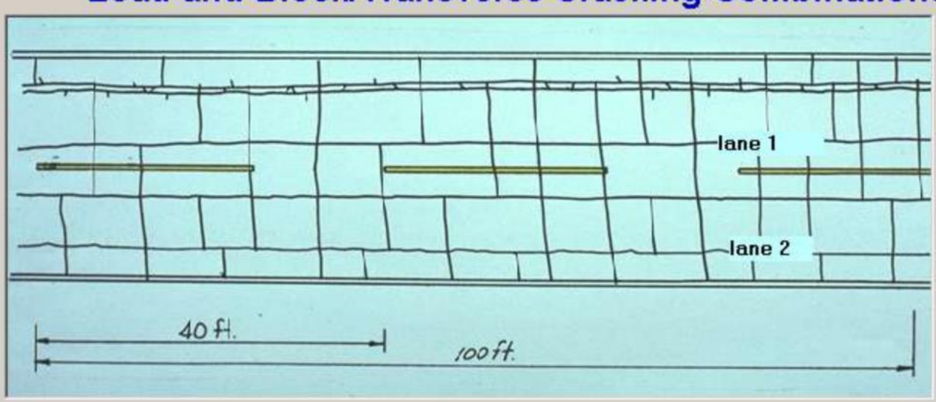
In Lane 2

Level 3:	70 / 200	= 35%
Level 4:	100 / 200	= 50%

In this example, lane 2 would be the lane rated.

View Images | How to Determine % Distress | Exit | Back

Load and Block/Transverse Cracking Combinations



Back

Lane 2:

- The longitudinal cracks are in the wheelpaths and thus, are load cracking. Since they are single cracks, they are considered level 1 load cracking.
- The transverse cracks continue to outside wheelpaths and do not seem to be influenced by the wheelpaths, they are considered level 1 B/T cracking.

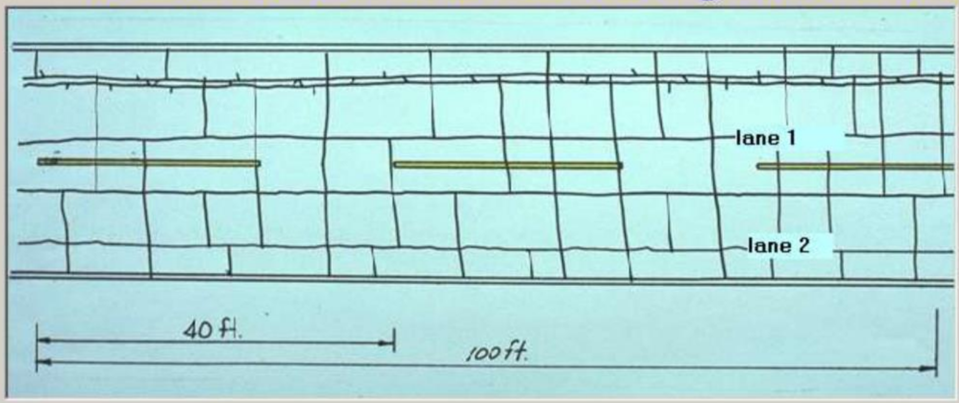
Lane 1:

- The longitudinal cracks are in the wheelpaths. Inside wheelpath has a single longitudinal crack, and outside wheelpath has a double longitudinal crack.
- Transverse cracks are approximately the same as that in lane 2.

Level 1 load cracking:	100 feet / 200 feet	= 50%
Level 2 load cracking:	100 feet / 200 feet	= 50%
Level 1 B/T cracking:	160 feet / 100 feet	= 100%

In this example, lane 1 would be the lane rated and recorded as it is the lane with the most cracks.

Load and Block/Transverse Cracking Combinations



Back

Lane 2:

- The longitudinal cracks are in the wheelpaths and thus, are load cracking. Since they are single cracks, they are considered level 1 load cracking.
- The transverse cracks continue to outside wheelpaths and do not seem to be influenced by the wheelpaths, they are considered level 1 B/T cracking.

Lane 1:

- The longitudinal cracks are in the wheelpaths. Inside wheelpath has a single longitudinal crack, and outside wheelpath has a double longitudinal crack.
- Transverse cracks are approximately the same as that in lane 2.

Level 1 load cracking:	100 feet / 200 feet	= 50%
Level 2 load cracking:	100 feet / 200 feet	= 50%
Level 1 B/T cracking:	160 feet / 100 feet	= 100%

In this example, lane 1 would be the lane rated and recorded as it is the lane with the most cracks.

CALCULATION OF PROJECT RATING

A general understanding of the PACES calculation of the project ratings by the rater is essential for the rater to understanding the COPACES program. The COPACES program is based on the PACES system that was manually calculated by the rater.

The project rating obtained from PACES can vary from 0 to 100 points. One hundred points is assigned to a roadway with no visible surface distresses. Points are deducted from a possible 100 based on extent and severity of each surface distress. One hundred minus the total deduct points is the project rating.

The deduct values are assigned based on average extent and predominant severity level for the entire project. **This fact points out the necessity of choosing projects properly. For example, a poor section of roadway, rated together as a project with a good section, will result in the poor section of the roadway not being adequately represented by the rating score.** Obviously, one or two miles cannot be separated from the middle of a project for special scoring. However, after close inspection of the Detailed Project Rating Sheet obtained from COPACES, it should be obvious if the project limits were chosen correctly or incorrectly. Consideration should be given to breaking the project into two projects if the conditions warrant.

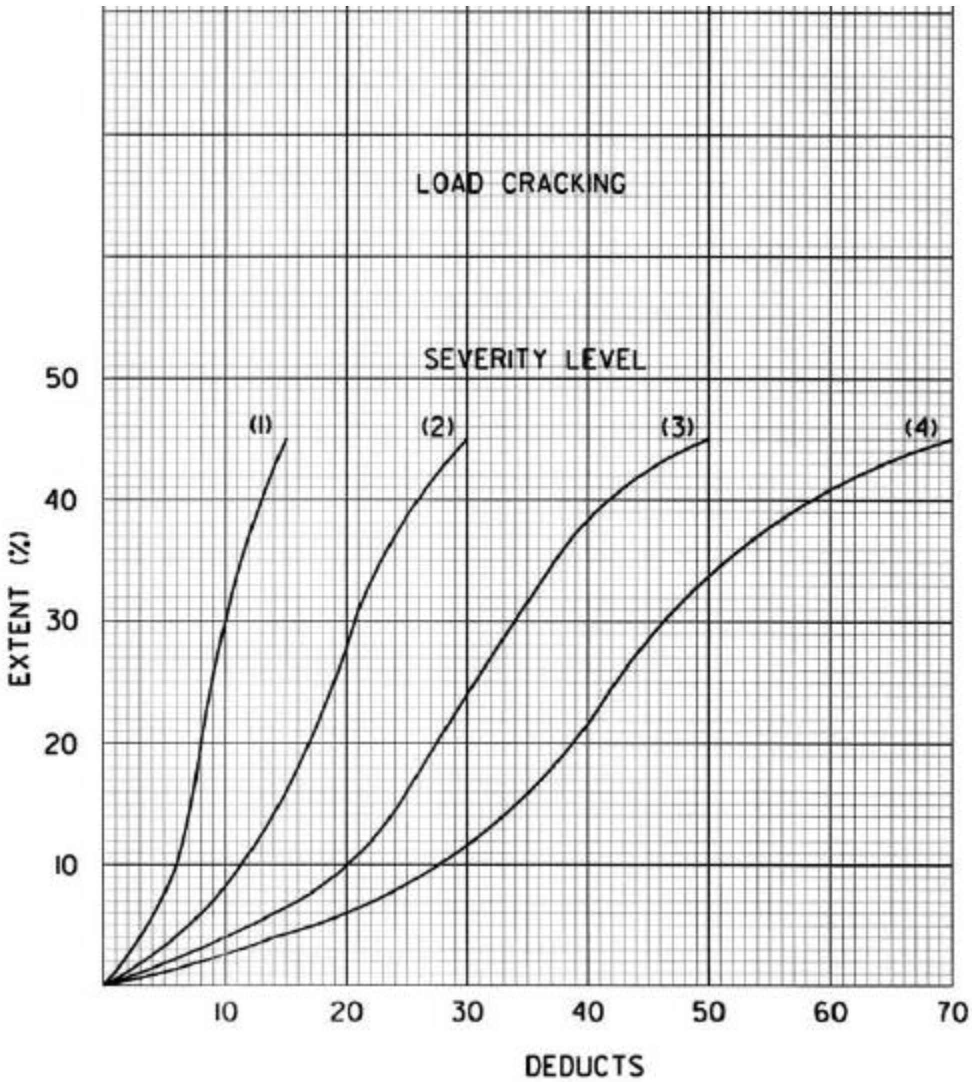
Determining Project Average for Each Distress

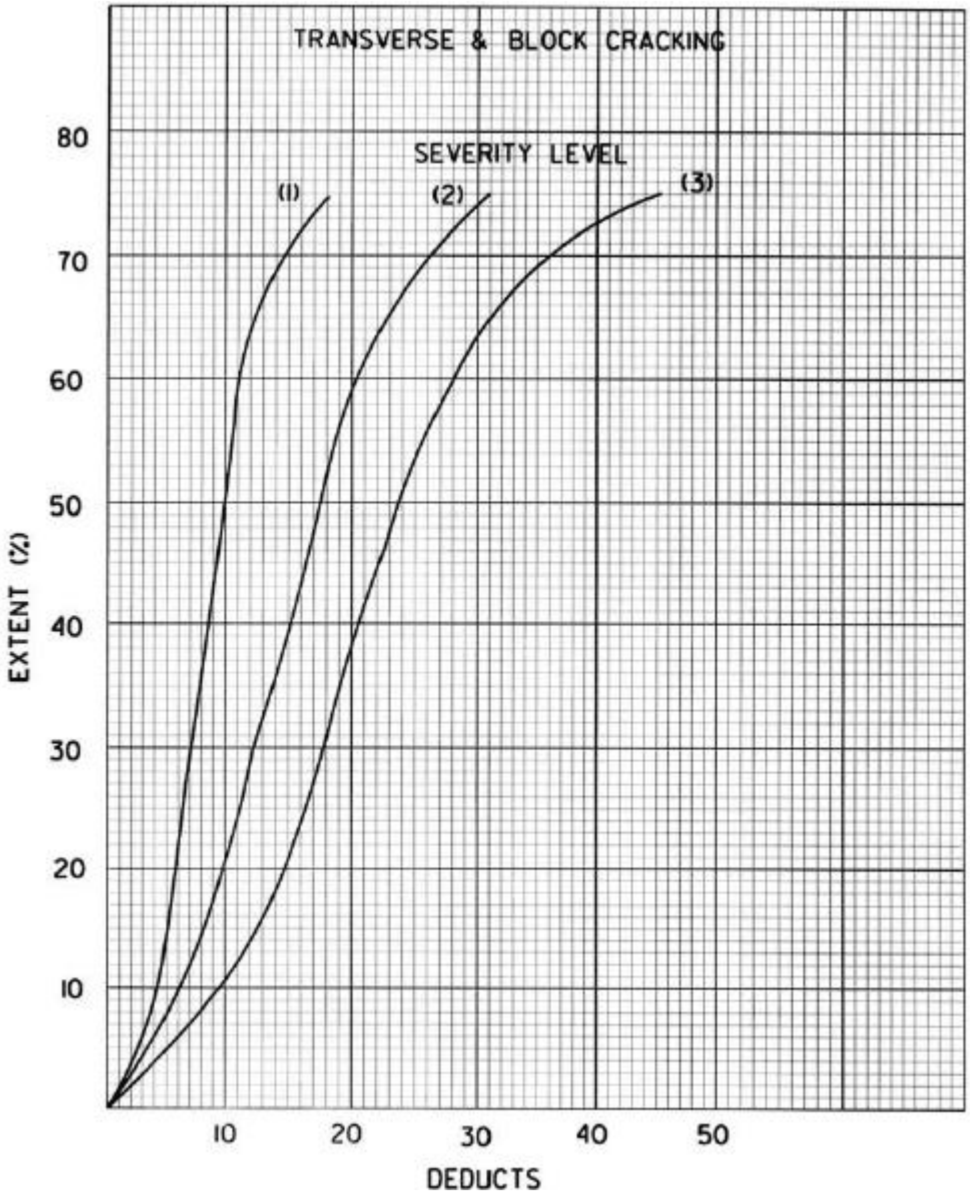
Simple numeric averages for each distress are used instead of prorating in this rating system. The averages are computed by totaling the values for each type of distress and dividing by the number of rating segments.

After the average values are computed for each distress for the project, deduct points are determined for each distress extent and severity. These deduct points are totaled and subtracted from 100 to determine the project rating.

The following charts, used when PACES was performed manually, are representative of the deduct point values used in COPACES.

Flexible Pavement Condition Survey Deduct Values




TRANSVERSE & BLOCK CRACKING
SEVERITY LEVEL
(1)
(2)
(3)
EXTENT (%)
80
70
60
50
40
30
20
10
10
20
30
40
50
DEDUCTS

Rutting Extent (inches)							
	0	1/8	1/4	3/8	1/2	5/8	3/4
Deducts	0	2	5	12	16	20	24

	Patches and Potholes Extent (# per mile)				
	1-2	3-6	7-10	11-15	>15
Deducts	2	5	10	17	25

		Corrugations/Pushing Extent (%)		
		1-10	11-25	>25
Severity	1	1	2	4
	2	2	4	7
	3	3	6	10

		Raveling Extent (%)					
		1-5	6-15	16-25	26-35	36-45	>45
Severity	1	2	5	6	8	10	13
	2	4	8	11	14	17	21
	3	6	12	16	20	25	30

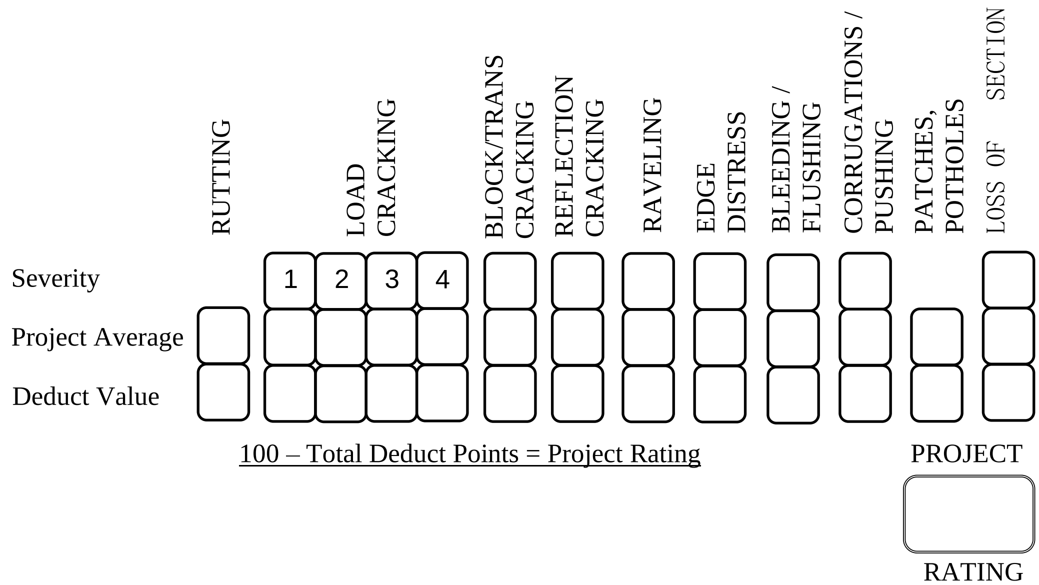
		Edge Cracking Extent (%)			
		5-25	26-50	51-75	>75
Severity	1	1	2	3	4
	2	2	4	6	7
	3	3	6	8	10

		Loss of Pavement (%)			
		0-25	25-50	50-75	75-100
Severity	1	0	1	2	3
	2	2	4	6	8
	3	6	5	10	12

		Bleeding or Flushing Extent (%)		
		1-10	11-30	>30
Severity	1	2	5	8
	2	5	10	15

		Reflective Cracking (%)				
		5-15	16-30	31-45	46-50	>51
Severity	1	3	5	6	8	10
	2	6	8	11	14	20
	3	10	15	20	25	30

Examples



The load cracking portion should have values under each of the severities. The rutting portion should have the highest average wheelpath value. The remainder of the defects should have one set of values, the percent extent and predominant severity level. The patches and potholes portion should have the average per mile value.

After the average extent and predominant severity level have been computed based on the segment survey results, the “Flexible Pavement Condition Survey Deduct Values” charts and tables, shown above, are used to determine the deduct points for each distress extent and severity. These deduct points are totaled (only the largest load cracking deduct values is used) and subtracted from 100 to determine the project rating (see examples 1 and 2).

Example 1: Computation of averaged raveling severity level and extent in a project

A project has considerable raveling. The severity of the raveling varies throughout the project, some level 1, 2, and 3. The percent occurrence and severity level of raveling is recorded for each rated segment (mile or portion of a mile) under sample location. The “percent occurrence” entries are totaled and averaged regardless of severity level or length of rated segment.

The predominant severity level for this project must be determined, since it is not obvious from the entries. An average of the severity level entries gives a value that can be rounded off to the nearest severity level.

Although only raveling distress is shown in this example, the basic process would apply to all the various types of distresses.

Milepost		Raveling	
From	**To**	**Percent**	**Severity**
18.4	**19.0**	**80**	**1**
19.0	**20.0**	**50**	**1**
20.0	**21.0**	**75**	**3**
21.0	**21.6**	**99**	**2**
21.6	**22.0**	**30**	**1**
22.0	**23.0**	**99**	**1**
23.0	**24.0**	**70**	**1**
24.0	**25.0**		
25.0	**26.0**		
26.0	**26.8**	**20**	**1**

Rated Segements	10		523	11
		Average	**52**	**1.4**
			(523/10)	(= 11/8; only 8 segments have raveling)
Project Average:	52	Severity:	1	

<u>Example 2: A project rating computation</u>

A. Project Level

This example shows the calculation of project rating.

FLEXIBLE PAVEMENT CONDITION SURVEY

Date: 3/23/2017 8:21:20 AM   Project Rating: 44   Office: DO   Rater:

A. PROJECT LOCATION

District: 7   Route Type: 1   **Route No: 0006**   Route Suffix: 00
County1: FULTON   **CountyNO: 121**   **Milepost From: 6.5**   **To: 12.06**
County2:   **CountyNO:**   **Milepost From2:**   **To:**
County3:   **CountyNO:**   **Milepost From3:**   **To:**
Project Limits: Butner Road to Clayton County Line
Estimated Length of the cutter & gutter that require MILLING (Miles): 1

B. ROADWAY INFORMATION

Surface Type: A   Divided Highway: YES   Direction: POS.
Typical Pavement Width(ft.): 48   Typical Shoulder Width(ft.): 6   No. Bridges: Bridge Width(ft.):
STAA: NO   AADT: 50100   %Truck: 9

Project Mileposts		Rut Depth	Load Cracking				Block Cracking		Reflection Cracking			Raveling		Edge Distress		Bleeding/Flushing		Corrugation/Pushing		Loss Pavement Section			Project Rating
From	To	Averge	Severity 1	Severity 2	Severity 3	Severity 4	% of Sample	Severity (1,2,3)	No. of Cracks	Total Length	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	Patches & Potholes	Rating
6.5	12.06	2	24	36	10		58	1														12	**44**
Deduct		5	**9**	**23**	**20**		**11**															**17**	

Please note that the deduct for each severity level of load cracking needs to be calculated. However, only the greatest deduct (i.e. 23) is used for calculating the project rating. In this example, only 23, i.e., the greatest load cracking deduct, is subtracted from 100 to calculate the project rating.

$100 – (5 + 23 + 11 +17) = 44$

For calculating the segment rating, the similar approach can be applied.

B. Segment Level

This example shows the calculation of segment rating.

FLEXIBLE PAVEMENT CONDITION SURVEY

Date: 3/23/2017 8:21:20 AM | Project Rating: 44 | Office: DO | Rater:

A. PROJECT LOCATION

District: 7 | Route Type: 1 | **Route No: 0006** | Route Suffix: 00
County1: FULTON | **CountyNO: 121** | **Milepost From: 6.5** | **To: 12.06**
County2: | **CountyNO:** | **Milepost From2:** | **To:**
County3: | **CountyNO:** | **Milepost From3:** | **To:**
Project Limits: Butner Road to Clayton County Line
Estimated Length of the cutter & gutter that require MILLING (Miles): 1

B. ROADWAY INFORMATION

Surface Type: A | Divided Highway: YES | Direction: POS.
Typical Pavement Width(ft.): 48 | Typical Shoulder Width(ft.): 6 | No. Bridges: | Bridge Width(ft.):
STAA: NO | AADT: 50100 | %Truck: 9

C. REMARKS

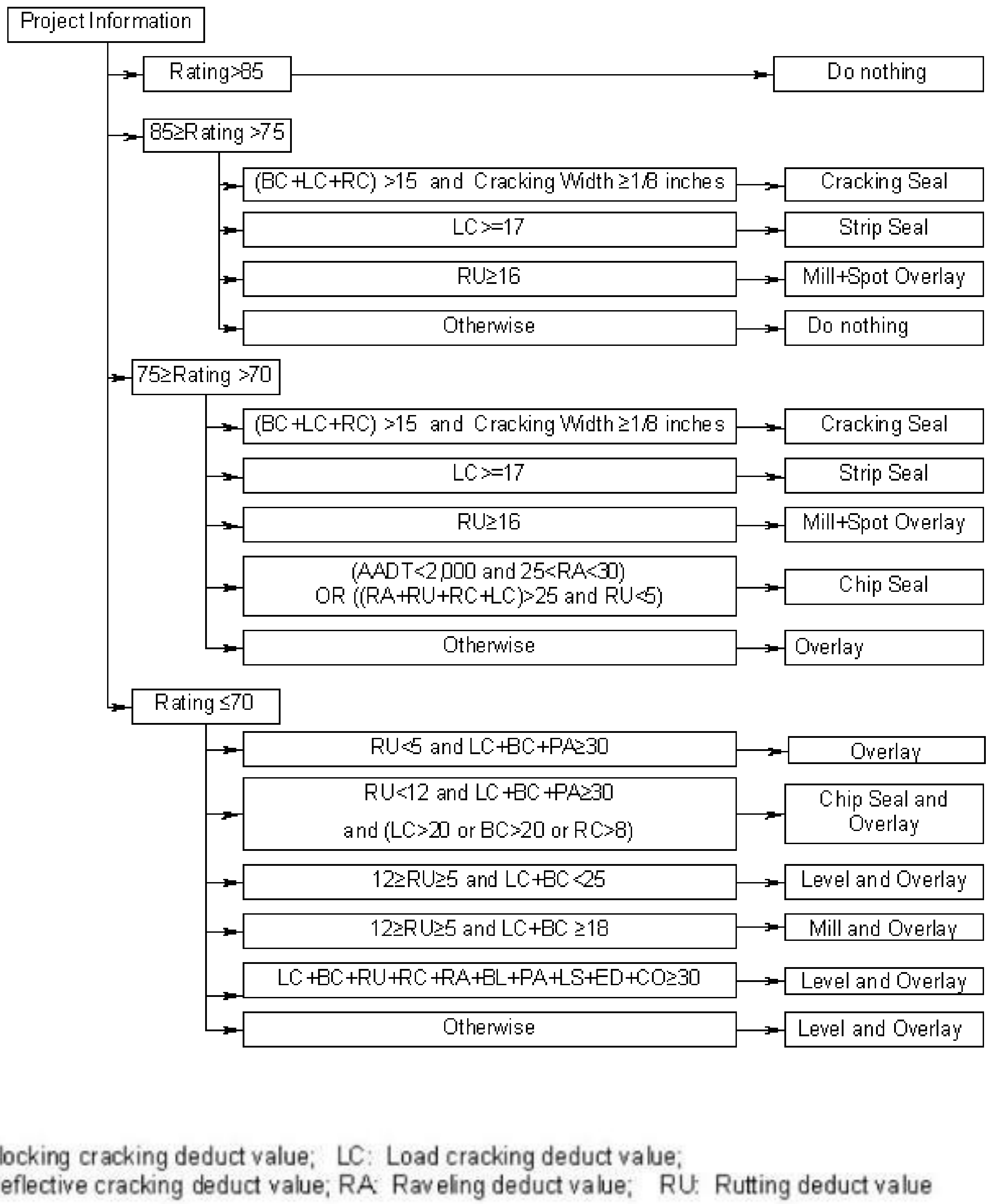
Sample Location: Milepost							Rut Depth		Load Cracking				Block Cracking		Reflection Cracking			Raveling		Edge Distress		Bleeding/Flushing		Corrugation/Pushing		Loss Pavement Section			Seg. Rating	Crack Width
From	To	County	Sample Location	Lane Direction	Lane Number (1,2,3...)	Project Limit	Outside W.P. (1/8 in)	Inside W.P. (1/8 in)	Severity 1	Severity 2	Severity 3	Severity 4	% of Sample	Severity (1,2,3)	No. of Cracks	Total Length	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	% of Sample	Severity (1,2,3)	Patches & Potholes	Rating	Greater than 1/8 inch?
6.5	8	121	1	POS.	2	1	1	1		50			60	1														2	**55**	YES
Deduct							**2**		**30**				**11**															**2**		
8	9	121	1	POS.	2	0	1	1	50				50	1														5	**68**	YES
Deduct							**2**		**15**				**10**															**5**		
9	10	121	1	POS.	2	0	2	2	50		50		70	1														15	**13**	YES
Deduct							**5**		**50**				**15**															**17**		
10	11	121	7	POS.	2	0	2	2		60			60	1														29	**29**	YES
Deduct							**5**		**30**				**11**															**25**		
11	12.06	121	7	POS.	2	0	2	2	20	70			50	2														10	**37**	YES
Deduct							**5**		**30**				**18**															**10**		

The segment rating is calculated by subtracting all deducts from 100.

Segment Rating	Rating Calculation (100 – sum of deducts)
55	100 – (2 + 30 +11 +2)
68	100 – (2 + 15 + 10 +5)
13	100 – (5 + 50 + 15 + 17)
29	100 – (5 + 30 + 11 + 25)
37	100 – (5 + 30 + 18 + 10)

PAVEMENT MAINTENANCE AND REHABILITATION CRITERIA

The following decision tree shows the asphalt pavement maintenance and rehabilitation criteria based on COAPCES rating, distress deduct values, traffic volume, and crack width.



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REFERENCES

ACPA. (1990). The Concrete Pavement Restoration Guide: Procedures for Preserving Concrete Pavements.

ACPA, 1995. Guidelines for Full-Depth Repair, Technical Bulletin TB002.02P, American Concrete Pavement Association, Skokie, IL, 1995.

ACPA. (1997). The Concrete Pavement Restoration Guide-Procedures for Preserving Concrete Pavements.

ACPA. (1998). Guidelines for Partial-Depth Spall Repair. Concrete Paving Technology, TB003P.

Ahmed, A. A. Ultra-Thin Bonded Wearing Course Performance Update, Minnesota. Final Report MN/RC 2009-30. Minnesota Department of Transportation. 2010.

AI & AMEA. A Basic Asphalt Emulsion Manual. The Asphalt Institute and Asphalt Emulsion Manufacturer Association, Asphalt Institute Manual Series No. 19, Fourth Edition, 2014.

Asphalt Recycling and Reclaiming Association. “Basic Asphalt Recycling Manual,” Federal Highway Administration, 2001.

Bischoff, D. and A. Toepel (2002). DOWEL BAR RETROFIT. STH 13 CONSTRUCTION AND ONE-YEAR PERFORMANCE REPORT.

Buddhavarapu, P., Smit, A. D. F., Prozzi, J. A., & Trevino, M. (2014). Evaluation Of The Benefits Of Diamond Grinding Of A CRCP. (No. FHWA/TX-13/5-9046-01-1).

Burnham, T., & Izevbekhai, B. (2009, April). Retrofit dowel bars in jointed concrete pavement—long-term performance and best practices. In National conference on preservation, repair, and rehabilitation of concrete pavements, St. Louis, Missouri (pp. 161-81).

Caltrans. Caltrans maintenance Technical Advisory Guide. 2003.

Caltrans, 2007. Caltrans SSP No. 40-020 Replace Concrete Pavement (Rapid Strength Concrete), Sacramento, CA, January 2007.

"CHAPTER 5—DIAMOND GRINDING AND GROOVING." MTAG Volume II - Rigid Pavement Preservation 2nd Edition Caltrans Division of Maintenance December 19, 2007

Chen, D. H., Won, M., & Zha, X. (2008). Performance of Dowel Bar Retrofit Projects in Texas. Journal of Performance of Constructed Facilities, 22(3), 162-170.

Correa, A. (1999). Cost-effective load transfer restoration of jointed concrete pavements. TR News, 200.

Correa, A. L., & Wong, B. (2001). Concrete Pavement Rehabilitation-Guide for Diamond Grinding (No. FHWA-SRC 1/10-01 (5M)).

CP Tech Center, Concrete Pavement Preservation Guide, September 2014.

CP Tech Center, Georgia Concrete Overlay Workshop, Forest Park, GA. October 11, 2017

CP Tech Center, Guide to Concrete Overlays: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements (Third Edition). ACPA Publication TB021.03P. National Concrete Pavement Technology Center, Ames, IA. May 2014.

CP Tech Center, Guide to Full-Depth Reclamation (FDR) with Cement, Report No. SR1006P, March 2017.

CRAFCO.com (2020), Pavement Crack Sealing. Availabe at: https://crafco.com/pavement-preservation/cracks-joints/pavement-crack-sealing/

Daniel P. Frentress, F. E., LLC. (2011). Partial-Depth Repairs for Concrete Pavements.

Decker, D. S. NCHRP Report 784: Best Practices for Crack Treatments for Asphalt Pavements. Transportation Research Board of the National Academies, Washington, D.C., 2014.

Defrain, L. (1989). Noise Analysis of Ground Surface on I-69 WB Near Lowell Road CS 19043.

Descornet, G. (2000). Low-noise road surface techniques and materials. Proceedings of Inter Noise 2000.

Donavan, P. 2010. Acoustic Radiation from Pavement Joint Grooves between Concrete Slabs. Transportation Research Record 2158. Transportation Research Board, Washington, DC.

Drakopoulos, A., Wenzel, T. H., Shober, S. F., & Schmiedlin, R. B. (1998). Crash experience on tined and continuously ground portland cement concrete pavements. Transportation Research Record: Journal of the Transportation Research Board, 1639(1), 140-146.

Drakopoulos, A., Wenzel, T. H., Shober, S. F., & Schmiedlin, R. B. (1998, January). Comparison of Accident Experience Between Tinned and Continuously Ground Portland Cement Concrete Pavement. In 77th Annual TRB Committee Meeting on Surface Properties–Vehicle Interaction (A2B07).

Evans, L. D., K. L. Smith, and A. R. Romine. 1999. Materials and Procedures for the Repair of Joint Seals in Portland Cement Concrete Pavements—Manual of Practice. FHWA-RD-99-146. Federal Highway Administration, McLean, VA.

Fang, C., K. Galal, D. Ward, and A. Hand. Cost-Effectiveness of Joint/Crack Sealing. Publication FHWA/IN/JTRP-001/1. Indiana Department of Transportation, Indianapolis, 2003.

Federal Highway Administration (FHWA). 2002. Pavement Preservation Checklist Series #9: Joint Sealing Portland Cement Concrete Pavements. FHWA-IF-03-003. Federal Highway Administration, Washington, DC.

Federal Highway Administration (FHWA) / American Concrete Pavement Association (ACPA), 1997, Concrete Pavement Rehabilitation: Guide to Load Transfer Restoration, FHWA-SA-97-103, ACPA JP001P, 1997.

Felix, T. (2010). Open Graded Crack Relief/Leveling Interlayer. [ebook] Forest Park: Georgia Depertment of Transportation.

FHWA. Pavement Preservation Checklist Series 4 Fog Seal Application. 2002.

FHWA-IF-03-002, Pavement Preservation Checklist Series, Microsurfacing Application. November 2002.

FHWA-IF-06-014, Pavement Preservation Checklist Series, Slurry Seal Application. November 2005.

FHWA. Pavements, Materials & Construction Technology. Full-Depth Repairs. 2017.

FHWA. (1999). Portland Cement Concrete (PCC) Partial Depth Spall Repair. (FHWA-RD-99-177).

FHWA, 2005. Full-Depth Repair of Portland Cement Concrete Pavements, Pavement Preservation Checklist Series, Federal Highway Administration, Washington, DC, November 2005.

FHWA. (2005). Partial Depth repair of Portaland Cement Concrete Pavement.

FHWA. (2015). Partial-Depth Repairs.

Fp2.org. (2020). *FP2.org » Why Pavement Preservation?* Available at: http://www.fp2.org/why-pavement-preservation/ [Accessed 7 Mar. 2020].

Frentress, D. P., & Harrington, D. S. (2012). Guide for partial-depth repair of concrete pavements: Iowa State University Institute for Transportation.

GDOT. Pavement Condition Evaluation System. Georgia Department of Transportation, Office of Maintenance, 2007

Geary, G., Tsai, Y., Wu, Y. (2018) “An Area-Based Faulting Measurement Method Using Three-Dimensional Pavement Data.” *Journal of Transportation Research Record,* National Academy of Sciences, 2018, PP. 1-9.

Gulden, W. (2012). GEORGIA-THE BIRTHPLACE OF CONCRETE PAVEMENT RESTORATION.

IGGA. (2011). CPR Rebuilt to last: PARTIAL DEPTH REPAIR A STANDARD SOLUTION FOR SURFACE DEFECT.

INDOT. (2016). 2016 Standard specifications. Indianapolis, IN: Indiana Department of Transportation. Retrieved from http://www.in.gov/dot/div/contracts/standards/book/sep15/sep.html

Irene K. Battaglia, D. B., Jed Peters. (2012). Partial Depth Repair of Concrete Pavements in Wisconsin.

Jiang, C. and Tsai, Y. (2015) "Enhanced Crack Segmentation Algorithm Using 3D Pavement Data", ASCE Journal of Computing in Civil Engineering, 2015, 30(3): 04015050.

Jiang, C.*, Tsai, Y., Wang, Z. (2016) "Crack Deterioration Analysis Using 3D Pavement Surface Data: A Pilot Study on Georgia State Route 26." Journal of Transportation Research Record, National Academy of Sciences, 2016 (2589): 154-161.

Johnson, D. R., R. B. Freeman, and J. R. Stevenson (2000). Cost-Effectiveness of Crack Sealing Materials and Techniques for Asphalt Pavements. *Transportation Research Record: Journal of the Transportation Research Board*, No. 1697, 2000, pp. 31–40. http://dx.doi.org/10.3141/1697-06.

Jones, D., Fu, P and John T. Harvey. "Full-Depth Pavement Reclamation with Foamed Asphalt in California: Guidelines for Project Selection, Design, and Construction". UCPRC-GL-2008-01. March 2009

Jungheum Yeon, S. C., Soojun Ha, Moon C. Won. (2012). Guidelines for Partial Depth Repair in Continuously Reinforced Concrete Pavement.

Kaul, V., A. Yezzi, and Y. C. Tsai. Detecting Curves with Unknown Endpoints and Arbitrary Topology Using Minimal Paths. IEEE Transactions on Pattern Analysis and Machine Intelligence, Vol. 34, No. 10.

Lai, J. (2011) “Assessing Techniques and Performance of Thin PEM Overlay on Micro-milled Surfaces.” Georgia Department of Transportation, Atlanta, GA.

Lai, J., M. Hines, S., Wu, P.Y., and Jared, D. (2012). “Pavement Preservation with Micro milling in Georgia – Follow-Up Study.” Transportation Research Record: Journal of the Transportation Research Board, Vol. 2292, pp. 81-87.

LA County Pbulic Works. (2014, January 30). Micro-milling Process - Sinaloa Avenue, Et Al[Video file]. Retrieved from https://www.youtube.com/watch?v=Vy4bn9XDiNk

Lee, J., Hastak, M., & Ahn, H. J. (2015). Crack sealing and filling: Best practices (Joint Transportation Research Program Publication No. FHWA/IN/JTRP-2015/23). West Lafayette, IN: Purdue University. http://dx.doi.org/10.5703/1288284316008

Maintenance, C. D. o. (2008). "Rigid Pavement Preservation 2nd Edition".

McGhee, K. H. 1995. Design, Construction, and Maintenance of PCC Pavement Joints. NCHRP Synthesis of Highway Practice 211. Transportation Research Board, Washington, DC.

Missouri Standard Specifications for Highway Construction. 2018.

MnDOT. (2003). CONCRETE PAVEMENT REHABILITATION

MnDOT. (2005). Mn/DOT Standard Specifications Book 2005 - Division III Materials.

Mullin, A., J. Liu, and R. McHattie. Field Evaluation of Crack Sealing of AC Pavements in Alaska. In Climatic Effects on Pavement and Geotechnical Infrastructure, American Society of Civil Engineers, Reston, Va., 2014, pp. 39–51.

Muncy, S. Polymers Boost Performance of Conventional Fog Seals. Pavement Preservation Journal, Vol. 6, No. 4, 2013

National Center for Pavement Preservation (NCPP) Training Materials, Chip Seals, Part 1 and 2. October 2017.

NCHRP Synthesis, Pavement Patching Practices, 2014

NCHRP Synthesis Report 338, Thin and Ultra-Thin Whitetopping. , National Academies of Sciences, Transportation Research Board, Washington, D.C. 2004.

NCHRP Synthesis Report 342 - Chip Seal Best Practices, 2005.

NCHRP Synthesis Report 411, Microsurfacing, 2010.

Noorvand, H. and Estrada, K. Ultra-Thin Bonded Wearing Coarse. Arizona Pavements and Materials Conference. 2015

Onyango, M., & Woods, M. (2017). Analysis of the Utilization of Open-Graded Friction Course (OGFC) in the United States. In *Airfield and Highway Pavements 2017* (pp. 137-147).

Peshkin, D., K. L. Smith, A. Wolters, J. Krstulovich, J. Moulthrop, and C. Alvarado. 2011. Preservation Approaches for High-Traffic-Volume Roadways. SHRP 2 Report S2-R26-RR-1. Strategic Highway Research Program 2, Washington, DC.

Pierce, L. M., et al. (2003). Dowel bar retrofit do's and don'ts.

Pierce, L., Uhlmeyer, J., Weston, J., Lovejoy, J., & Mahoney, J. (2003). Ten-year performance of dowel-bar retrofit: Application, performance, and lessons learned. Transportation Research Record: Journal of the Transportation Research Board, (1853), 83-91.

Ponniah, Joseph, and Gerhard Kennepohl. Crack Sealing in Flexible Pavements: A Life-Cycle Cost Analysis. Transportation Research Record1529: 86-94, Washington, D.C, 1996.

Rajagopal, A. Effectiveness of Crack Sealing on Pavement Serviceability and Life. Publication No. FHWA/OH-2011/ 14. Ohio Department of Transportation, 2011.

Rao, S., Yu, H. T., & Darter, M. I. (1999). The Longevity and Performance of Diamond-Ground Pavements (No. PCA R&D Serial No 2175,). Portland Cement Association.

Rao, S., Yu, H. T., Khazanovich, L., Darter, M. I., & Mack, J. W. (1999). Longevity of diamond-ground concrete pavements. Transportation Research Record: Journal of the Transportation Research Board, 1684(1), 128-136.

Rebecca S. McDaniel, J. O., Ali Behnood, Bryan Magee,Rachel Pollock. (2014). Pavement Patching Practices. NCHRP Synthesis 463.

Ridley, Matthew, GDOT Pavement Foremen's' Academy Training Powerpoint, 2006.

Ronald Mulvaney, N. C., Eshan Dave. (2014). Optimizing Materials and Methods for Partial-Depth Pavement Repair.

Rosales-Herrera, V.I., J. Prozzi, and J.A. Prozzi, Mixture Design Manual and Performance-Based Specifications for Cold Patching Mixtures, report submitted to FHWA and TxDOT, Austin, Tex., 2007, 144 pp.

Russell, M., Uhlmeyer, J., DeVol, J. Johnson, C. and Jim Weston. “Evaluation of Hot In-Place Recycle”. WA-RD738.1, Washington State DOT. June 2010.

Salomon, D. R. Asphalt Emulsion Technology. Transportation Research Circular, 2006.

Supplemental Specifications, GDOT, Retrieved from https://www.dot.ga.gov/PartnerSmart/Business/Source/special_provisions/2016%20Supplemental%20Specifications/2016SupplementalSpecBook.pdf

Seal/No Seal Group (SNS). 2011a. Joint Slap Evaluation and Prevention. Tech Brief. Seal/No Seal Group, West Coxsackie, NY. www.equipmentworld. com/seal no-seal-group-issues-five-tech-briefs-on-jointsealants/ (accessed April 2014).

SDDOT (2010) “Pavement Preservation Guidelines”, South Dakota Department of Transportation.

SHRP 2 Report S2-R26-RR-2, Guidelines for the Preservation of High-Traffic Volume Roadways, 2011.

Smith, K. D., Hoerner, T. E., & Peshkin, D. G. (2008). Concrete Pavement Preservation Workshop.

Smith, Kelly L., and A. Russell Romine. Materials and Procedures for Sealing and Filling Cracks in Asphalt-surfaced Pavements--manual of Practice. Publication FHWA-RD-99-147. FHWA, U.S. Department of Transportation, 1999

Shuler, Scott. “Best Practices for Full-Depth Reclamation Using Asphalt Emulsions”. CDOT-2015-04. July 2015

Stubstad, R., Rao, C., Pyle, T., & Tabet, W. (2005). Effectiveness of Diamond Grinding Concrete Pavements in California. California Department of Transportation, Sacramento, California, USA.

Stroup-Gardiner, M., “Recycling and Reclamation of Asphalt Pavements Using In-Place Methods”. NCHRP Synthesis 421. TRB. Washington, DC. 2011.

Titus-Glover, Leslie, and Michael I. Darter. Guidelines for Improving Full-Depth Repair of PCC Pavements in Wisconsin. No. WHRP 08-02. 2008

TPPC. Fog Seal. Texas Pavement Preservation Center Newsletter, Issue 30, 2013.

T.P. Wilson, K. L. S., A.R. Romine. (1999). LTPP PAVEMENT MAINTENANCE MATERIALS: PCC PARTIAL-DEPTH SPALL REPAIR EXPERIMENT, FINAL REPORT.

Transportation Research Record: Journal of the Transportation Research Board, Vol.2306, pp.144-150.

Tsai, Y., and Li, F. (2012). "Critical Assessment of Detecting Asphalt Pavement Cracks under Different Lighting and Low Intensity Contrast Conditions Using Emerging 3D Laser Technology." Journal of Transportation Engineering, 138(5), 649–656.

Tsai, Y., Li*, F., Wu, Y. (2013) “A New Rutting Measurement Method Using Emerging 3D Line-Laser Imaging System”, Int. Journal of Pavement Research and Technology, Vol. 6(5):667-672.

Tsai, Y., Wang, Z., Li*, F. (2015) “Assessment of Rut Depth Measurement Accuracy of Point-based Rut Bar Systems using Emerging 3D Line Laser Imaging Technology”, Journal of Marine Science and Technology, Vol. 23, No. 3, pp. 322-330 (2015), DOI: 10.6119/JMST-014-0327-1.

Tsai, Y., Wu, Y., Ai*, C., Pitts, E. (2012) “Feasibility Study of Measuring Concrete Joint Faulting Using 3D Continuous Pavement Profile Data,” *ASCE Journal of Transportation Engineering*, 138(11), 1291-1296

Tsai, Y., Wu, Y., Lai, J., Geary, G. (2012). “Characterizing Micro-milled Pavement Textures Using RVD for Super-thin Resurfacing on I-95 Using A Road Profiler.”

Tsai, Y., Wu, Y., Lewis, Z. (2014). “A Full-Lane Coverage Micro-Milling Pavement Surface Quality Control Using Emerging 3D Line Laser Imaging Technology.” ASCE Journal of Transportation Engineering, Vol. 140, No. 2, pp.

Tsai, Y. C. and Wang, Z. (2014). “A Remote Sensing and GIS-Enabled Asset Management System (RS-GAMS) Phase 2.” Final Report for USDOT project: RITARS-11-H-GAT

Tsai, Y. and Wang Z. (2015) “Development of an Asphalt Pavement Raveling Detection Algorithm Using Emerging 3D Laser Technology and Macrotexture Analysis”, National Academy of Science NCHRP IDEA-163 Final Report.

Tsai, Y., Chatterjee*, A, (2017) “Pothole Detection and Classification Using 3D Technology and Watershed Method”, *ASCE Journal of Computing in Civil Engineering*, 32(2), 04017078.

TxDOT. Asphalt Emulsion. 2006.

Watson, D. “Hot In-Place Recycling Literature Review” FDOT-PR5620597. National Center for Asphalt Technology (NCAT). December 2011.

Wilson, T., and Romine, R. (1994). Materials and Procedures for the Repair of Potholes in Asphalt-Surfaced Pavements: Manual of Practice Strategic Highway Research Program National Research, Washington, DC.

Woodstrom, J. H., & Neal, B. F. (1976). Cement Content of Fresh concrete. WORK, 19303, 6351496.

Wouter Gulden. (2012 January). Georgia — the birthplace of concrete pavement Restoration.

WSDOT (2011). "WSDOT Pavment Policy."

Wu, Z., Groeger, J. L., Simpson, A. L., and Hicks, R. G. Performance evaluation of various rehabilitation and preservation treatments. Rehabilitation, 2010

Zheng Wu, et al. Performance Evaluation of Various Rehabilitation and Preservation Treatments. FHWA-HIF-10-020, FHWA, Washington, DC. 2010.

Zimmerman K. (2017) NCHRP Synthesis 501 Pavement Management Systems: Putting Data to Work. Transportation Research Board of the National Academies, Washington, D.C.

Zinke, S., Mahoney, J. M. Evaluation of Pavement Crack Treatments. 2006

Vargas-Nordcbeck, A., & Jalali, F. (2020). Life-Extending Benefit of Crack Sealing for Pavement Preservation. *Transportation Research Record*, 2674(1), 272-281.