Framework of Asphalt Balanced Mix Design (BMD) for New England Transportation Agencies

Dr. Walaa S. Mogawer, P.E. - PI
Dr. Thomas Bennert

Prepared for
The New England Transportation Consortium
December 31st, 2020

NETCR116 **Project No. 18-2**

ACKNOWLEDGEMENTS

The following are the members of the Technical Committee (TC) that developed the scope of work for the project and provided technical oversight throughout the course of the research:

Ann Scholz, New Hampshire Department of Transportation,
  NETC Advisory Committee Liaison
David Howley, Connecticut Department of Transportation
Casey Nash, Maine Department of Transportation
Ed Naras, Massachusetts Department of Transportation
Matt Courser, New Hampshire Department of Transportation
Joe Blair, New Hampshire Department of Transportation, Chair of TC
Michael Byrne, Rhode Island Department of Transportation
Aaron Schwartz, Vermont Agency of Transportation

Technical Report Documentation Page

1. Report No. NETCR116	2. Government Accession No. N/A	3. Recipient's Catalog No. N/A
4. Title and Subtitle Framework of Asphalt Balanced Mix Design (BMD) for New England Transportation Agencies		5. Report Date December 31st, 2020
		6. Performing Organization Code N/A
7. Author(s) Professor Walaa S. Mogawer, P.E. – Principal Investigator (PI) Dr. Thomas Bennert		8. Performing Organization Report No. NETCR116
9. Performing Organization Name and Address Department of Civil and Environmental Engineering University of Massachusetts Dartmouth 285 Old Westport Road North Dartmouth, MA 02747		10 Work Unit No. (TRAIS) N/A N/A
		11. Contract or Grant No. N/A
12. Sponsoring Agency Name and Address New England Transportation Consortium c/o Transportation Research Maine DOT 24 Child Street Augusta, ME 04333		13. Type of Report and Period Covered Final Report (Feb 2019 – Dec 2020)
		14. Sponsoring Agency Code NETC 18-2 A study conducted in cooperation with the U.S. DOT
15 Supplementary Notes N/A		

16. Abstract

The objective of this study was to synthesize existing information and to develop recommendations for a rational Balanced Mix Design (BMD) approach for use by New England transportation agencies.

A survey was developed and administered to ascertain information related to pavement distress for the NETC state agencies. The predominate distresses noted were thermal and fatigue cracking. Based on the distresses noted in the survey, candidate performance tests were identified and the pros and cons of each test were outlined in relation to existing availability, equipment cost, and testing time. Potential roadblocks to BMD implementation were also identified.

Field and laboratory performance data were collected from three state agencies. An in-depth analysis of the data was conducted to develop preliminary balanced mixture design criteria that is individualized for each state. The criteria were different for each state as the availability of data varied. Finally, knowledge gaps for BMD implementation were identified and recommendations for future research were presented.

17. Key Words Balanced Mix Design, BMD, distress, performance, specification	18. Distribution Statement No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161.		
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages 58	22. Price N/A

Form DOT F 1700.7 (8-72) Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimetres	mm
ft	feet	0.305	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
		AREA		
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.093	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	kilometres squared	km^2
		VOLUME		
fl oz	fluid ounces	29.57	mililitres	mL
gal	gallons	3.785	Litres	L
ft^3	cubic feet	0.028	metres cubed	m^3
yd^3	cubic yards	0.765	metres cubed	m^3

NOTE: Volumes greater than 1000 L shall be shown in m^3

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9	Celcius temperature	°C

* SI is the symbol for the International System of Measurement

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
		AREA		
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
ha	hectares	2.47	acres	ac
km^2	kilometres squared	0.386	square miles	mi^2
		VOLUME		
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams	1.102	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C+32	Fahrenheit temperature	°F

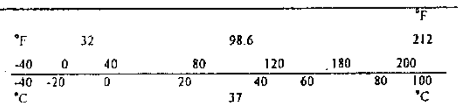


TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Superpave volumetric design method as outlined in AASHTO M323 "Standard Specification for Superpave Volumetric Mix Design" is currently being used by New England state transportation agencies to design their asphalt mixtures, specifically dense graded mixtures. The Superpave mixture design method was originally intended to provide a performance-based specification for asphalt binder and mixture (*1*). The performance-based asphalt binder specification is used in common practice today as outlined in the Performance Graded (PG) asphalt binder specification AASHTO M320 "Standard Specification for Performance-Graded Asphalt Binder." For the mixture specification, the Superpave mixture design system was developed with three levels of mixture design: Level 1, Level 2 and Level 3 (*1*). Performance based mixture tests were included in Levels 2 and 3, however these design levels were never implemented because the testing and analysis were considered too complex (*1*). The Level 1 Superpave mixture design method currently being used relies on empirical aggregate quality characteristics and mixture volumetric properties such as air voids, voids in mineral aggregates (VMA), and voids filled with asphalt (VFA). The use of the Level 1 mixture design process raises two concerns. The first concern is that calculation of the volumetric properties is dependent on the accuracy of the bulk specific gravity of the aggregates (*2*). There are well-known issues with the accuracy and variability of the aggregate bulk specific gravity testing. Incorrect bulk specific gravity values can ultimately lead to inaccuracy of the design binder content. The second concern with Level 1 is that the volumetric design alone does not measure, quantify or predict mixture performance prior to placement. This performance forecasting has become essential for many state transportation agencies due to many recent developments in the asphalt paving industry. These changes include: utilization of binders formulated with various modifiers (re-refined engine oil bottoms, air blown asphalt, rubber, polymers, polyphosphoric acid, etc.) versus conventionally neat asphalt binders, the incorporation of more recycled materials (reclaimed asphalt pavement, recycled asphalt shingles, ground tire rubber, etc.) into mixtures, and utilization of innovative technologies (warm mix asphalt, asphalt rejuvenators, bio-binders, etc.). Mixtures designed with respect to these factors, and in accordance to the current Superpave mixture design method, have unknown performance as Superpave was never designed to compensate for these factors.

The concerns with Superpave Level 1 mixture design has renewed interest in developing a performance-related specification using a balanced mixture concept (also referred to recently as Performance Engineered Mixture Design or PEMD) as opposed to solely relying on derived volumetric properties per Superpave Level 1mixture design method. This renewed interest has led the FHWA Expert Task Group on Mixtures and Construction to form a Balanced Mix Design (BMD) Task Force. The task force defined BMD as an "Asphalt mix design using performance tests on appropriately conditioned specimens that address multiple modes of distress taking into consideration mix aging, traffic, climate and location within the pavement structure." (*2*).

Generally, BMD will incorporate two or more performance tests to evaluate how well the mixture will resist certain distresses that are of concerns for a particular state transportation agency. It has been broken into three approaches for simplicity by the FHWA BMD Task Force and outlined as follows (*2*):

- The first approach starts with the Superpave volumetric mixture design for determining the optimum asphalt binder content followed by performance verification to assess the mixtures resistance to the distresses such as rutting and cracking. If the mixture design satisfies the performance requirements, the mixture design is complete, and production commences. Otherwise, the entire mixture design process is repeated using different mixture proportions or different materials until all performance criteria are satisfied. This approach is currently used in Illinois, Louisiana, New Jersey, Texas, and Wisconsin (*3*).

- The second approach is performance-modified volumetric mixture design. This approach is similar to the first approach in that it starts by determining the optimum binder content using the Superpave volumetric design method, but subsequently focuses on meeting performance test criteria. The mixture design binder content and/or proportions can be adjusted to accommodate the performance test based requirements. The final design may not be required to meet all the volumetric Superpave required criteria. California currently is using this approach (*3*).

- The third approach is performance design. This approach establishes and adjusts mixture components and proportions based on performance testing results. Once the laboratory performance tests criteria are met, the mixture volumetric properties may be checked for use during production for quality control.

Overall, BMD provides a design approach to develop asphalt mixtures tailored to specific performance expectations. Because BMD is a relatively new concept, a regional BMD approach needs to be developed to address the typical distress issues and materials in New England. To date, there have been no regionally based studies to investigate this approach in New England.

2.0 OBJECTIVE

The objective of this study was to synthesize existing information and to develop recommendations for a rational BMD approach for use by New England transportation agencies. Gaps in testing and performance data will be identified through this project and an experimental plan for required future work will be developed.

3.0 STATE AGENCY SURVEY

The main objective of this research project was to synthesize existing information and to develop recommendations for a rational BMD approach for use by New England transportation agencies.

The first action item for this project was to develop a survey to administer to the New England Transportation Consortium (NETC) state agencies asking for information related to pavement distress. Prior to the adoption of any asphalt mixture performance testing program, whether this is simple index testing (tests with pass/fail criteria) or more complicated balanced mixture design (fundamental properties measured to predict performance), it is critical that the selected performance test simulates the observed pavement distress in the area of question. For example, low temperature thermal cracking tests are not appropriate when observed pavement distresses are rutting and alligator cracking.

3.1 Internet Survey

A list of questions was developed and submitted to the project technical committee for approval prior to the solicitation of responses. Once the list of questions was approved, the internet-based survey was developed in Google forms as shown in Appendix A and is available online at https://forms.gle/iRjWUJCgR2kYSf3G8.

The primary goal of the survey was to identify the predominate distresses observed in New England and how they occur regionally. Additionally, the survey was developed to gather more information about:

- Time period to distress initiation
- How distress measurements are collected and developed into indices
- Weighting of distress indices
- How mixtures are differentiated in reference to distress measurements and inclusion into a pavement management system
- Performance test used in an attempt to mitigate distress
- Percentage of asphalt surface mixtures by type (i.e. Nominal Maximum Aggregate Size)
- Recycled Asphalt Shingle (RAS) usage and practices
- Asphalt binder grades used
- Highly absorptive aggregates

The survey was distributed to the New England state transportation agencies (CT, MA, ME, NH, RI, VT) in May 2019 for responses. The last response was received in September 2019. In total seven response were received. Each New England state transportation agency responded to the survey with Vermont responding twice.

3.2 Internet Survey Findings

The survey results were reviewed, compiled and analyzed. Figure 1 illustrates the distribution of predominate asphalt pavement distresses observed by the NETC agencies. The highest noted distresses at 28.6% (2 responses each) were thermal cracking and fatigue cracking. A combination of rutting, raveling and moisture damage; all distresses, and all distresses except moisture damage all received one response (14.3%). A majority of respondents (71.4%) stated that certain distresses are more commonly observed in different regions of their state with thermal and fatigue cracking being the most commonly observed. Based on these distresses noted in the survey, candidate performance test(s) were discussed and recommended in Section 4.0 of this final report.

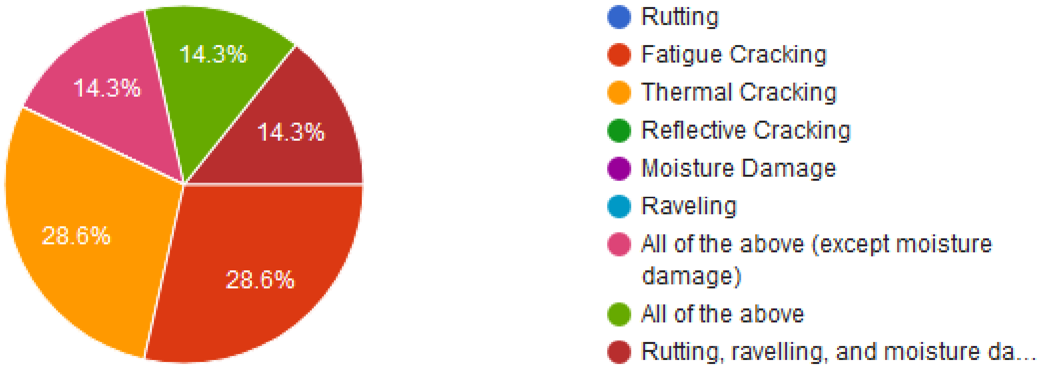


FIGURE 1 Predominate Distresses Noted by New England State Transportation Agencies

Other noteworthy findings of the survey were:

- ☐ There was no consensus among respondents about the time period when certain distresses initiate
- ☐ Pavement distress data is measured by automated, manual and both methods

☐ A majority of respondents use a combined index for pavement treatment selection
☐ Respondents were split or unsure on if certain distresses get weighed more heavily than others in the combined index
☐ All respondents indicated that condition index data is available to the research team
☐ A majority of respondents stated that their pavement management system did not differentiate between asphalt mixture types (i.e. all mixture grouped as one pavement type)
☐ Most respondents are investigating the use of performance tests during the mixture design phase in an attempt to mitigate the occurrence of specific distresses
☐ In the New England region, 9.5 mm and 12.5 mm dense graded mixtures comprise a majority of the asphalt pavement surfaces being constructed
☐ Most respondents do not allow RAS use
☐ Anywhere from between one to four asphalt binders are specified in an individual state, with two asphalt binders being the most specified
☐ Most states do not require a different low temperature grade asphalt binder for different regions within the state
☐ No respondents stated that they deal with highly absorptive aggregates

4.0 IDENTIFICATION OF CANDIDATE PERFORMANCE TESTS

Building on the results from Task 1, the primary emphasis of Task 2 was to evaluate the pros and cons of different asphalt mixture performance tests and best match a performance test method to the state agency pavement distress(es) within a Balanced Mixture Design protocol. Although this sounds generally simple, there are multiple asphalt mixture performance tests and even multiple Balanced Mixture Design approaches that a state agency would need to consider.

4.1 Identification of Candidate Performance Tests

A literature review was conducted to identify test procedures that match the needs of the NETC state agencies identified through the survey. The survey identified thermal cracking and fatigue cracking as the highest priority with rutting and moisture damage following. Therefore, a majority of the literature review focused on cracking tests. However, it is important to understand the need to include a rutting performance test as well in an effort to "balance" the performance of the asphalt mixtures. A performance specification too heavily dependent on one distress can allow asphalt suppliers to produce an asphalt mixture too soft (if the specification is only based on cracking) or too stiff (if the specification is only based on rutting).

The NCHRP Project 9-57 "Experimental Design for Field Validation of Laboratory Tests to Assess Cracking Resistance of Asphalt Mixtures" test selection criteria were used to help narrow down the candidate test procedures. This consisted of the following seven (7) criteria (*4*):

1. Availability of test method
2. Simplicity
3. Variability
4. Sensitivity to mixture properties
5. Complexity of testing and analysis
6. Cost of equipment
7. Lab to field correlation

The NCHRP Project 9-57, with the help of a series of panel experts, identified a number of fatigue and thermal cracking tests which were found to best meet the seven criteria. Table 1 show those recommended tests. As indicated in Table 1, there are few instances where different test methods are recommended for different modes of cracking. For example, the Overlay Tester (OT) is recommended for both Reflection Cracking and Fatigue Cracking, while the SCB-LTRC procedure was recommended for Reflection Cracking, Fatigue Cracking and Top-Down Cracking.

TABLE 1 Fatigue and Thermal Cracking Tests Identified Under NCHRP Project 9-57

Items	**Reflection Cracking**	**Fatigue cracking**	**Thermal Cracking**	**Top-down Cracking**
Selected cracking tests	1. OT 2. SCB-LTRC 3. BBF	1. Beam fatigue 2. SCB-LTRC 3. OT*	1. DCT 2. SCB-IL 3. SCB-TP105	1. SCB-LTRC 2. IDT-Florida
***7* cracking tests**	1. DCT 2. Three SCBs: SCB-TP105, SCB-LTRC, and SCB-IL 3. OT 4. Beam fatigue 5. IDT-Florida			

The SCB Flexibility Index (SCB-IL) test procedure is recommended to be conducted at an intermediate temperature of 25°C, and therefore, may not capture the low temperature cracking characteristics associated with thermal cracking. Therefore, it would not be recommended for thermal cracking.

4.2 Candidate Performance Tests - Fatigue Cracking

Table 2 captures the general test method information regarding estimate costs and time required to conduct the test (specimen preparation, conditioning, and testing time) for currently recognized intermediate temperature fatigue cracking tests. As the table indicates, the most expensive test procedure to implement would be the Flexural Beam Fatigue, primarily due to the necessity of purchasing a brick compactor for specimen preparation. The second most expensive fatigue cracking test would be the Direct Tension Cyclic Fatigue. The least expensive test procedures would be the SCB Flexibility Index, IDEAL-CT Index and SCB-LTRC. Regarding the time required for sample preparation and testing, the Direct Tension Cyclic Fatigue test is the most time consuming with the IDEAL-CT being the quickest to conduct (Figure 2). It should be noted that testing time can generally be decreased as the technicians gain additional experience. Conditioning times can also be reduced by utilizing a secondary environmental chamber for specimen conditioning, which can greatly increase the productivity of testing.

TABLE 2 Candidate Fatigue Cracking Tests and Their General Information

0

Test Method/Equipment	Costs	Testing Time	Total Time (after samples have been compacted)		
			Cutting	Gluing	Conditioning
SCB Flexibility Index	< $20,000 (stand alone)	< 30 minutes for 3 specimens	Up to 6 hours for 4 specimens	N.A.	> 2 hours
Overlay Tester	$60,000 (Alone) $15,000 (AMPT)	0.5 to 9 hours for 3 specimens	Up to 3 hours for 3 specimens	4 to 24 hours	> 2 hours
Flexural Beam Fatigue	>$100k (includes compactor)	Hours to days (strain levels x replicates)	Up to 3 hours for 3 specimens	N.A.	> 2 hours
Direct Tension Cyclic Fatigue (AMPT)	>$70,000	Up to 48 hours (DM E* + testing)	Up to 3 hours for 3 samples	4 to 6 hours (overnight)	> 2 hours
IDT Energy Ratio (Florida DOT)	> $100,000	Up to 24 hours for 3 specimens	Up to 1 hour for 3 specimens	4 to 6 hours (overnight)	> 2 hours
IDEAL-CT Index	≈ $15,000 $5,500 (SmartJig)	< 30 minutes for 3 specimens	N.A.	N.A.	> 2 hours
Louisiana SCB (Jc Parameter)	< $20,000 (stand alone)	Up to 8 hours for 9 specimens	Up to 8 hours for 9 specimens	N.A.	> 2 hours

Assumptions:

1. Not including the costs of gyratory compactor and time associated with compacting specimens
2. Costs do not include;
   Wet saw = $6,000
   Core drill = $3,500
   Environmental Chamber = $5,000 to $10,000
3. Does not include time associated to bulk specimens after cutting/trimming
4. Large differences in curing time for epoxies used in gluing

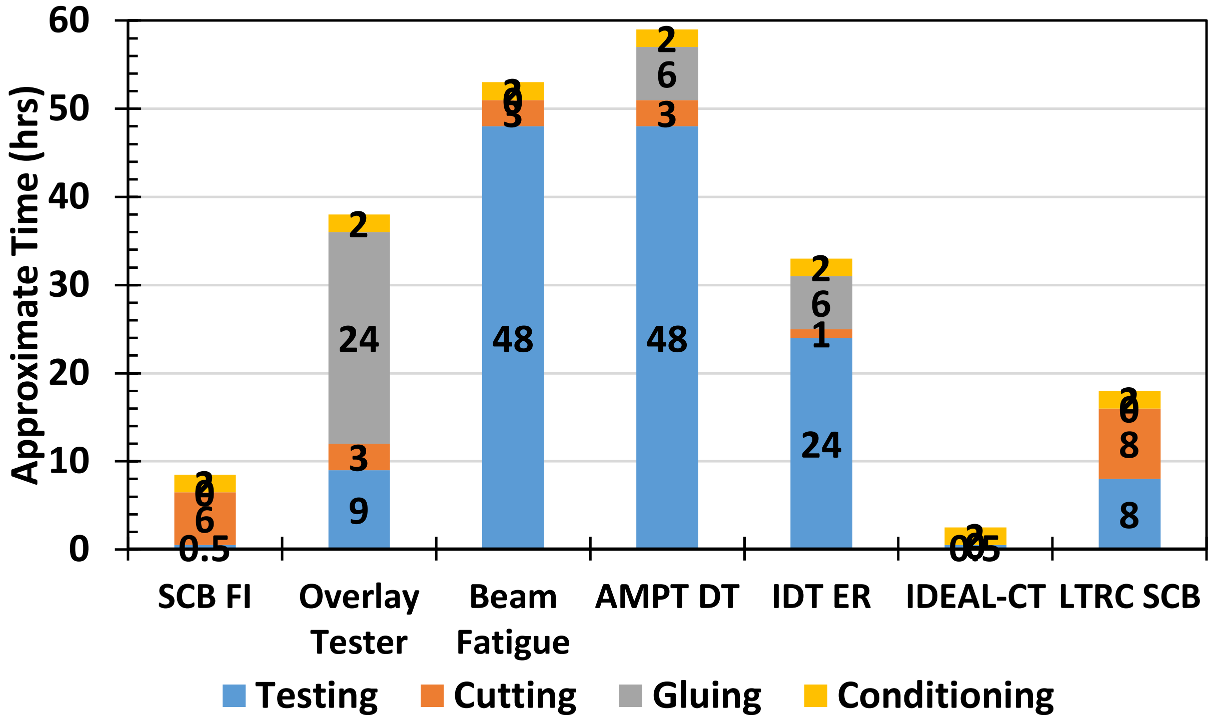


FIGURE 2 Time Requirements to Various Fatigue Cracking Test Procedures

4.3 Candidate Performance Tests - Thermal Cracking

Table 3 captures the thermal cracking test procedures and their general information regarding sample preparation, conditioning, testing time and costs. With respect to costs, the Thermal Stress Restrained Specimen Test (TSRST) was found to be the most expensive test to implement for thermal cracking evaluation. Either the Disk-Shaped Compact Tension (DCT) or the Indirect Tensile Test (IDT) Creep Compliance and Strength were found to be the least expensive test method to evaluate thermal cracking. However, the costs were not that significantly lower due to the need to cool the specimens to very low temperatures, as well as the test systems requiring more sensitive deformation, load cell, and thermal instrumentation.

Testing time (Figure 3) was found to be somewhat similar for some of the test procedures, mostly due to the requirements for conditioning. It was estimated that the IDT Creep Compliance and Strength test would be the quickest test procedure to conduct with the DCT test being the second quickest thermal cracking test to conduct. The Low Temperature SCB and TSRST tests were found to be the most time-consuming thermal cracking tests. It should be noted that testing time can generally be decreased as the technicians gain additional experience. Conditioning times can also be reduced by utilizing a secondary environmental chamber for specimen conditioning, which can greatly increase the productivity of testing.

TABLE 3 Candidate Thermal Cracking Test Procedures and Their General Information

Test Method/Equipment	Costs	Testing Time	Total Time (after samples have been compacted)		
			Cutting	Gluing	Conditioning
Disc-Shaped Compact Tension Test (DCT)	$75,000 (stand alone)	< 3 hours for 3 specimens	Up to 6 hours for 3 specimens	N.A.	8 to 16 hrs (overnight)
Thermal Stress Restrained Specimen Test (TSRST)	$85,000 (stand alone)	2 to 5 hours per specimen	Up to 3 hours for 3 specimens	Overnight	5 to 6 hours @ 5C (overnight)
IDT Creep Compliance & IDT Strength	$75,000 (stand alone)	Up to 8 hours for 3 specimens	Up to 1 hour for 3 specimens	4 to 6 hours (overnight)	> 3 hours @ test temperature
Low Temperature SCB (TP105)	Up to $100,000	Up to 4 hours for 3 specimens	Up to 6 hours for 4 specimens	4 to 6 hours (overnight)	8 to 16 hrs (overnight)
SCB Flexibility Index	< $20,000 (stand alone)	< 30 minutes for 3 specimens	Up to 6 hours for 4 specimens	N.A.	> 2 hours

Assumptions:

1. Not including the costs of gyratory compactor and time associated with compacting specimens
2. Costs do not include;
   Wet saw = $6,000
   Core drill = $3,500
   Environmental Chamber = $5,000 to $10,000
   Additional costs associated with liquid nitrogen
3. Does not include time associated to bulk specimens after cutting/trimming
4. Large differences in curing time for epoxies used in gluing

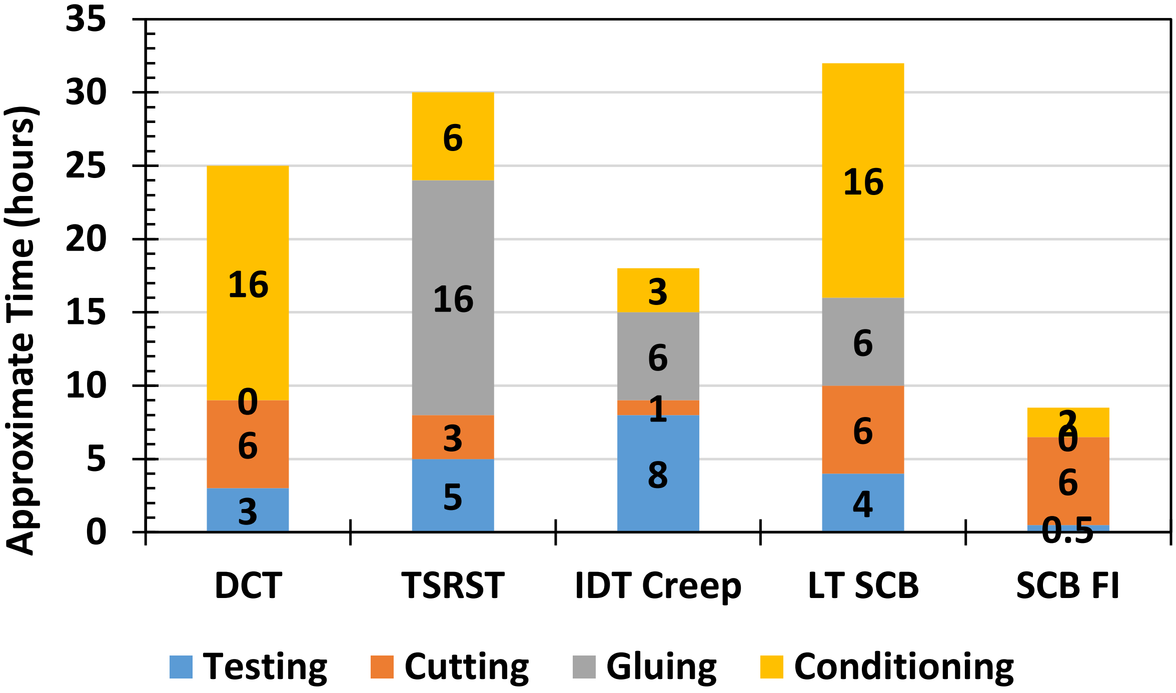


FIGURE 3 Time Estimate Requirements to Conduct Thermal Cracking Test Method

4.4 Candidate Performance Tests - Rutting (Permanent Deformation) Tests

Table 4 identifies current rutting test methods available for implementation and their respective general information. Regarding expense, the least expensive test method to implement and conduct would be the High Temperature IDT (HT-IDT) test. The HT-IDT was also found to be the quickest test procedure to conduct as well (Figure 4). The most expensive test method to implement was the Asphalt Pavement Analyzer, APA (when the large chamber unit is purchased). If the smaller, "table top" version, is to be purchased instead, it would have a similar cost to the Asphalt Mixture Performance Tester (AMPT) and some Hamburg Wheel Tracking (HWT) test equipment.

The time requirements vary slightly, but were found to be much quicker than most fatigue and thermal cracking tests (Figure 4). Some test procedures, such as the HWT and AMPT Flow Number, could take a considerable amount of time when test samples have significant rutting resistance.

TABLE 4 Candidate Rutting Test Procedures and Their General Information

Test Method/Equipment	Costs	Testing Time	Total Time (after samples have been compacted)		
			Cutting	Gluing	Conditioning
Asphalt Pavement Analyzer	$115,000 (Large); $70,000 (Junior)	2.25 hours	N.A.	N.A.	6 to 24 hours
AMPT Flow Number Test	>$70,000	Anywhere from 0.5 to 4 hours	Up to 3 hours for 3 samples	N.A.	Up to 3 hours per sample
AMPT Triaxial Stress Sweep (not much information on test)	>$70,000	Anywhere from 0.5 to 4 hours	Up to 3 hours for 3 samples	N.A.	Up to 3 hours per sample
Hamburg Wheel Tracking	$55,000 to $70,000	Up to 6.5 hours	0.5 to 2 hours for cylindrical samples	N.A.	> 1 hour (spec says 30 min.)
High Temperature IDT Strength	≈ $15,000 $5,500 (SmartJig)	3 specimens within 15 minutes	N.A.	N.A.	> 2 hours

Assumptions:

1. Not including the costs of gyratory compactor and time associated with compacting specimens
2. Costs do not include;
    Wet saw = $6,000
    Core drill = $3,500
    Environmental Chamber = $5,000
3. Does not include time associated to bulk specimens after cutting/trimming

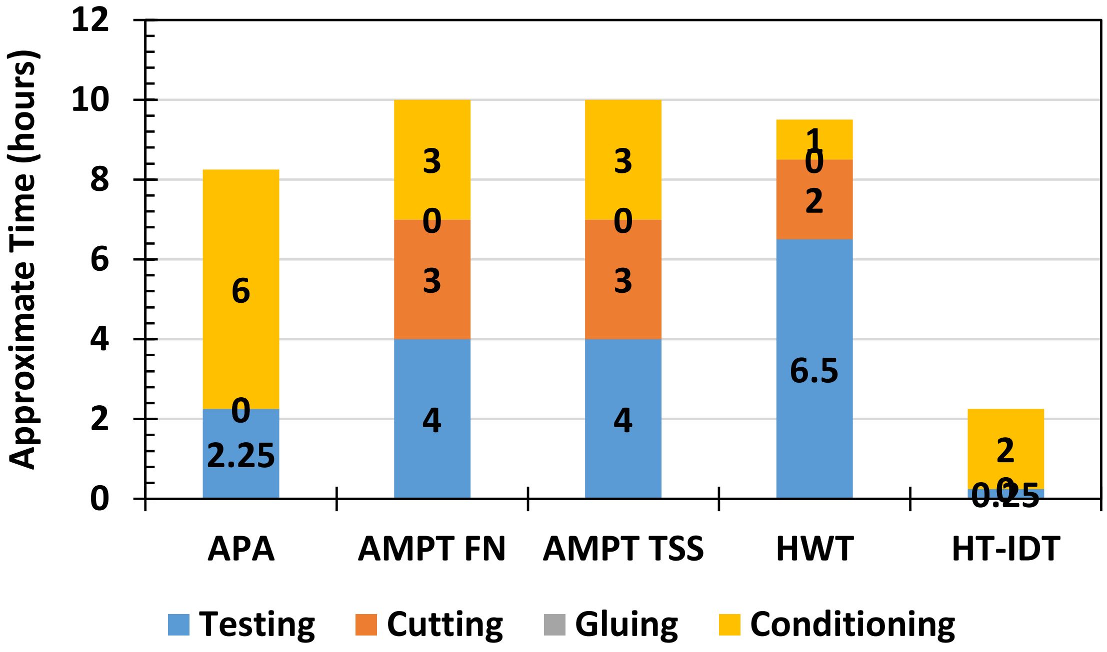


FIGURE 4 Time Estimate Requirements to Conduct Rutting Test Methods

4.5 Candidate Performance Tests - Moisture Damage (Stripping) Susceptibility and Durability

Lastly, the identified Moisture Damage Susceptibility tests methods are summarized in Table 5. The most expensive test device for moisture susceptibility was found to be the Hamburg Wheel Tracking (HWT) test. However, if a state agency decided to also use the HWT for a rutting test method, the higher expense could be justified if the same equipment was used for two different mixture distress tests. The least expensive test method was found to be the Tensile Strength Ratio (TSR) test. However, the TSR test was by far the most time-consuming test procedure due to the amount of time required to include a freeze-thaw cycle. The quickest test procedure for evaluating moisture damage potential was found to be the MiST device. However, it should be noted that general performance criteria have yet to be established for the MiST device, as it has been primarily used as a conditioning device for indirect tension testing.

The Cantabro test was included as there is preliminary information that shows the method can be used to evaluate the durability of asphalt mixtures. However, due to lack of literature clearly identifying moisture damage comparisons, it was solely listed and not directly compared.

TABLE 5 Candidate Moisture Damage Susceptibility Test Procedures and Their General Information

Test Method/Equipment	Costs	Testing Time	Total Time (after samples have been compacted)		
			Cutting	Gluing	Conditioning
Tensile Strength Ratio	≈ $10,000	0.5 to 1 hour	N.A.	N.A.	4 to 5 days
MiST Device	≈ $18,000	Up to 6 hours for 3 specimens	N.A.	N.A.	< 2 hours
Cantabro	≈ $10,000	< 2 hours for 3 specimens	N.A.	N.A.	> 4 hours
Hamburg Wheel Tracking	$55,000 to $70,000	Anywhere from 0.5 to 10 hours	0.5 to 2 hours for cylindrical samples	N.A.	> 1 hour (spec says 30 min.)

Assumptions:

1. Not including the costs of gyratory compactor and time associated with compacting specimens
2. Costs do not include;
   Wet saw = $6,000
   Core drill = $3,500
   Environmental Chamber = $5,000 to $10,000
3. Does not include time associated to bulk specimens after cutting/trimming
4. Large differences in curing time for epoxies used in gluing

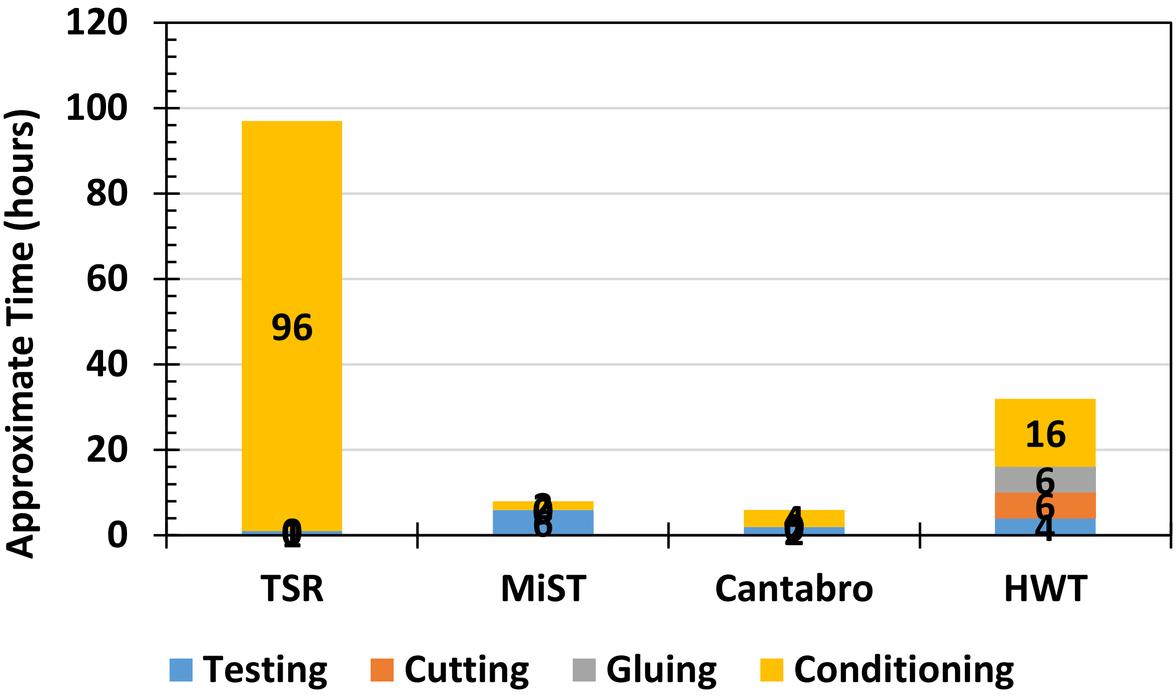


FIGURE 5 Time Estimate Requirements to Conduct Moisture Damage Susceptibility Test Methods

4.6 State Agency Current Performance Tests and Potential Roadblocks

A second, brief survey was provided to the state agencies to gain insight on the different performance testing equipment currently housed by each agency, as well as past or current research work the agencies have conducted with different test devices. The reason for the survey was concern over recommending test procedures that could accrue significant costs for the state agency. Additionally, if common test procedures could be recommended, the different state agencies and testing laboratories in the Northeast could leverage performance test equipment more efficiently. Appendix B includes the Excel-based survey information requested.

Figure 6 shows the survey results regarding the current asphalt mixture performance testing capabilities of the different state agencies in the NETC. The general highlights would indicate that:

- 2 of the 6 states own a universal testing machine (servo-hydraulic or screw driven). It was not noted as to whether or not the units are operating on a daily or weekly basis
- 4 of the 6 states own an Asphalt Mixture Performance Tester, AMPT (with the New Hampshire DOT's at the University of New Hampshire)
- 4 of the 6 states own a Hamburg Wheel Tracking machine (with Connecticut DOT's at the University of Connecticut)
- 5 of the 6 states own a Marshall and Stability Flow device
- 2 of the 6 states own a standalone Semi-circular Bend (SCB) Flexibility Index device
- 1 of 6 states (Maine) owns a MiST device for moisture susceptibility testing
- 6 of 6 states own an LA Abrasion Machine

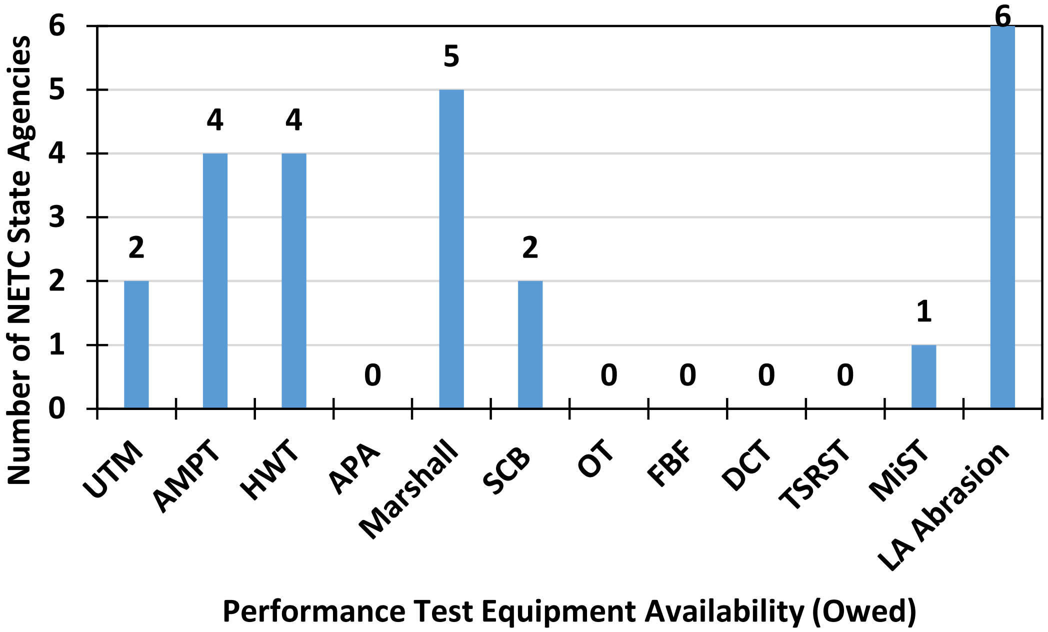


FIGURE 6 Performance Test Equipment Currently Owned/Housed by NETC State Agencies

Figure 7 shows the survey results regarding what test procedures are of immediate interest and/or being currently evaluated by the NETC state agencies. In summary, the survey indicated:

- 3 of 6 NETC state agencies have shown interest in using the Hamburg Wheel Tracking (HWT) and IDEAL-CT test procedures
- 2 of 6 NETC state agencies have shown interest in using the Asphalt Mixture Performance Tester (AMPT) and SCB Flexibility Index
- 1 of 6 NETC state agencies have shown interest in looking at the MiST device for moisture damage potential

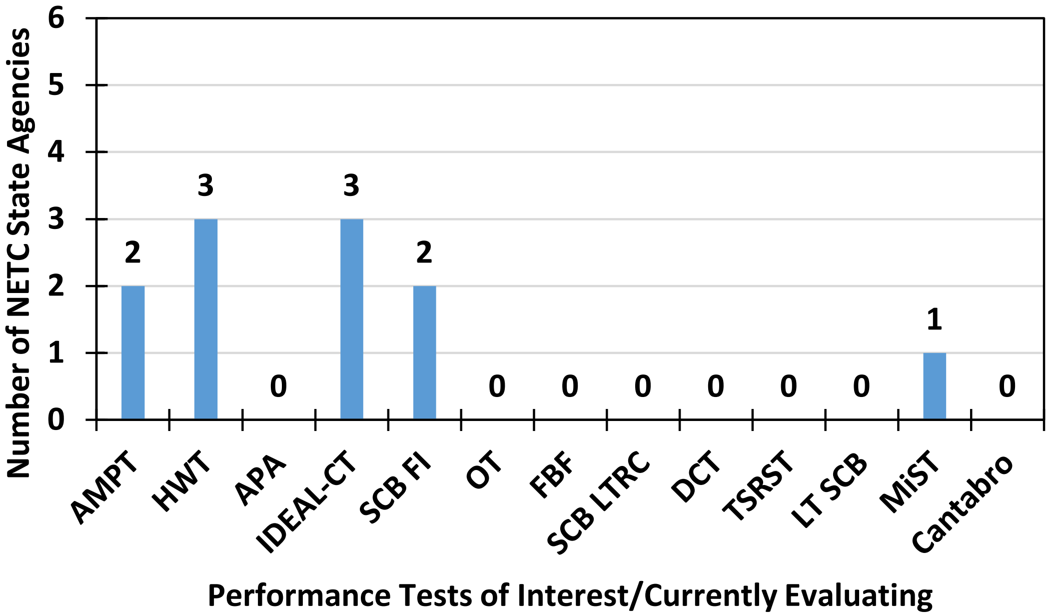


FIGURE 7 Performance Test Procedures of Interest and/or Currently Evaluating by the NETC State Agencies

Three of the six NETC state agencies leverage relationships with different universities/colleges in the northeast for performance testing and research. University of Massachusetts-Dartmouth (UMassD), University of New Hampshire (UNH), and Worchester Polytechnic Institute (WPI) were noted as having laboratories containing their own performance testing equipment or equipment loaned to them by the state agency (i.e. – UNH has an AMPT loaned to them by the New Hampshire DOT; WPI has a MiST device loaned to them by Maine DOT).

The NETC state agencies were also asked what are some foreseeable "roadblocks" that could delay the development of Performance Related Specifications (PRS) and Balanced Mixture Design (BMD). Some of the concerns noted were:

- Procurement of test equipment may take time (multiple years depending on associated costs)

- Procurement of calibration and repair services may be difficult due to procurement procedures

4.7 Final Recommendation for Candidate Performance Tests for NETC State Agencies

Based on the information provided, there are a number of options for which the NETC State Agencies can take in selecting "regionally collaborative" performance testing equipment for Balanced Mixture Design (BMD).

BMD Approaches A to C (Performance Related Specifications Based)

Based on reviewing the various survey results, the following general test procedures are recommended moving forward for Balanced Mixture Design for the NETC State Agencies:

1. Rutting
    1. With 4 of the 6 states currently having the Hamburg Wheel Tracking (HWT) device available, it would make perfect sense to include the HWT as a means of rutting potential evaluation. Selecting the HWT would minimize the number of state agencies needing to purchase new equipment, and with four agencies having the device, Round Robin testing can be conducted among the labs within the region to ensure devices are working properly.
    2. With 5 of the 6 states currently having a Marshall compression machine, inclusion of the High Temperature IDT (HT-IDT) should also be included for future evaluation. The additional benefit of the HT-IDT test is that there are minimal sample preparation requirements and testing can easily be conducted at the asphalt plant's QC laboratory with minimal investment from the asphalt plant.
2. Fatigue Cracking
    1. With 5 of the 6 states currently having a Marshall compression machine, the IDEAL-CT test procedure would result in an inexpensive solution to evaluating the fatigue cracking potential during mixture design and production.
    2. 2 of the 6 states noted they have availability of a standalone SCB device for the SCB Flexibility Index (SCB FI). In addition, the SCB FI test can be conducted on current Marshall compression machines using InstroTek's SMART SCB Jig apparatus (Figure 8) at a cost of less than $8,000 investment. Lastly, the SCB FI test can also be conducted on the AMPT with the purchase of additional attachments. However, the age of the AMPT machine will determine whether or not this is a viable option as older AMPT machines may not be suitable for the upgraded SCB testing apparatus.

FIGURE 8 InstroTek's SMART SCB Jig

3. Thermal Cracking

This is the most difficult of the performance tests to address as none of the NETC State Agencies noted that they owned any of the low temperature cracking test procedures. Therefore, there are a few potential options for including Thermal Cracking analysis within a Balanced Mixture Design program;

a. Two universities in the Northeast, University of Massachusetts-Dartmouth and University of New Hampshire, have the capability of currently conducting the Disk-Shaped Compact Tension (DCT) test. Without requiring the procurement of new testing equipment and calibration/repair services, agreements with the NETC member universities can be developed for thermal cracking testing.

b. With additional research, there may be merit in evaluating SCB Flexibility Index and the IDEAL-CT and how they are related to the DCT test. Figure 9 below shows some work conducted by Rutgers University for PennDOT's Long Life Asphalt Pavement (LLAP) projects containing both 9.5 mm NMAS SMA and 19 mm NMAS dense graded asphalt mixtures. Both the DCT test at -12°C and the SCB FI at 25°C are required testing procedures. The results show that a relationship does exist between the two test methods, but perhaps could be improved by looking at different testing temperatures. Varying loading rates in the SCB FI could also be evaluated, but would eliminate the use of the Marshall compression machine from testing. Testing could also be conducted during the mixture design phase using both tests to establish initial baseline. Such "surrogate" type testing would need to be conducted for each state's own materials if a database and general relationship were to be implemented.

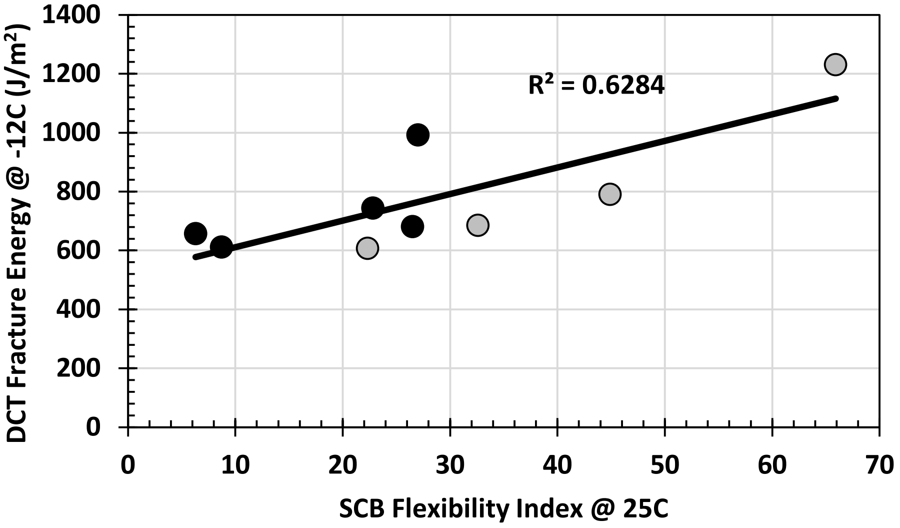


FIGURE 9 Comparison of DCT and SCB FI Performance for PennDOT's LLAP Projects (Black Circles = Lab Mixed; Gray Circles = Field Cores)

4. Moisture Damage
    a. With 4 of the 6 NETC State Agencies currently having access to a Hamburg Wheel Tracking (HWT) device, it would make perfect sense that the device is used for the dual purpose of rutting and moisture damage potential.

5.0 DEVELOPMENT OF PRELIMINARY BALANCED MIXTURE DESIGN CRITERIA

To establish an effective Balanced Mixture Design (BMD), a system of Performance Related Specifications (PRS) must be developed. An example of performance criteria for different asphalt mixture types is shown as Table 6. The criteria presented in Table 10 was developed for the New Jersey Department of Transportation (NJDOT) for their "Specialty Type" asphalt mixtures (*5*). The test methods utilized by the NJDOT are the Asphalt Pavement Analyzer (APA) for rutting and the Overlay Tester for fatigue cracking. The criteria were developed in a manner to take into consideration the traffic level (Low Volume receives PG64S-22; High Volume receives PG64E-22) and location in the pavement (Surface layer or Intermediate/Base layer). The criteria were developed this way with the understanding that the pavement performance requirements vary depending on these two factors.

TABLE 6 Performance Related Specifications Used for Balanced Mixture Design

Mixture Type			Maximum APA Rutting (mm)	Minimum Cycles in Overlay Tester
High RAP (HRAP)	Surface	PG64E-22	4.0	275
		PG64S-22	7.0	200
	Intermediate /Base	PG64E-22	4.0	150
		PG64S-22	7.0	100
Bituminous Rich Intermediate Course (BRIC)			6.0	700
High Performance Thin Overlay (HPTO)			4.0	600

After performance criteria has been established, the BMD process can then be utilized. An example of a BMD is shown in Figure 10, where asphalt content was increased at 0.5% intervals with performance testing conducted at each asphalt content. The zone of "balance" occurs in-between the asphalt contents were the asphalt mixture meets both the rutting and fatigue cracking criteria. As shown in the example, this occurs at 5.2% to 5.9% asphalt content – below 5.2% asphalt content and the asphalt mixture fails the fatigue cracking criteria, while above 5.9% asphalt content, the asphalt mixture fails the rutting criteria.

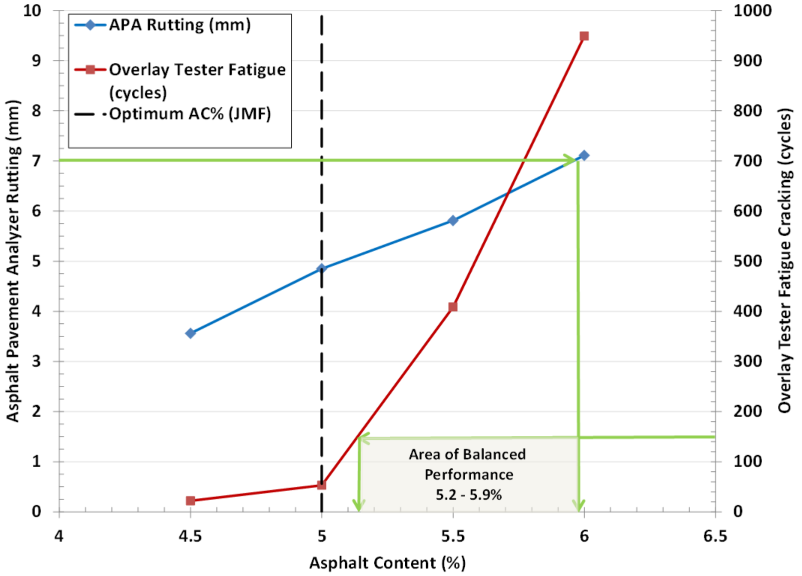


FIGURE 10 Example of Balanced Mixture Design (Approach C) on a New Jersey Asphalt Mixture

5.1 Establishing Performance Criteria for Balanced Mixture Design

As discussed earlier, prior to the adoption of the Balanced Mixture Design process, performance test criteria must be established for the test methods being utilized in the BMD. The performance criteria should be specific for the respective state agency in order to take into consideration regional materials, climate, traffic, and production/construction practices. Care should be taken not to solely adopt performance criteria from other state agencies located in a different climatic and traffic region. For example, the Hamburg Wheel Tracking (HWT) test has been associated with being proposed as a rutting test for many northern states. Unfortunately, a set of typical test criteria associated with the HWT was developed in Texas. State agencies blindly selecting the

Texas criteria may result in performance requirements misrepresenting their specific climate and traffic conditions. Therefore, general guidelines are presented in the following pages to help the NETC state agencies establish performance criteria for the future use in Balanced Mixture Design.

5.1.1 Materials for Performance Criteria Development

The verification of the performance criteria is traditionally conducted on three types of asphalt mixtures; 1) Field cores; 2) Plant Produced; and 3) Laboratory Produced. There are pros and cons for each of the mixtures.

<u>Field Cores</u> – the use of field cores allows for testing asphalt mixtures produced and placed in the manner that represents the true field condition. Plant and transport aging and asphalt absorption, in-place density and aggregate alignment all reflect true field conditions. The recovery of field cores at different time periods can also provide valuable information for the performance criteria. Extracting field cores from "Good" and "Poor" performing pavements can provide excellent information to help establish preliminary criteria that can be linked directly to field performance. Unfortunately, due to traditional layer thicknesses and coring methods used, some test methods may not be able to be utilized, which will limit the test methods available for implementation.

<u>Plant Produced Loose Mix</u> – the use of sampled plant produced loose mix provides asphalt mixtures that represents the true state of asphalt mixtures produced with various additives (i.e. – RAP, RAS, WMA, etc.). Sampled loose mix also provides flexibility with respect to different test specimen geometries required, from AMPT Repeated Load to Bending Beam Fatigue tests. However, due to the requirement of reheating (unless test specimens are compacted at the asphalt plant), some additional aging in the asphalt may occur if not handled carefully. Additionally, the sampled loose mix obviously represents the "New" condition of the pavement and long-term performance would only be able to be evaluated after some level of laboratory conditioning.

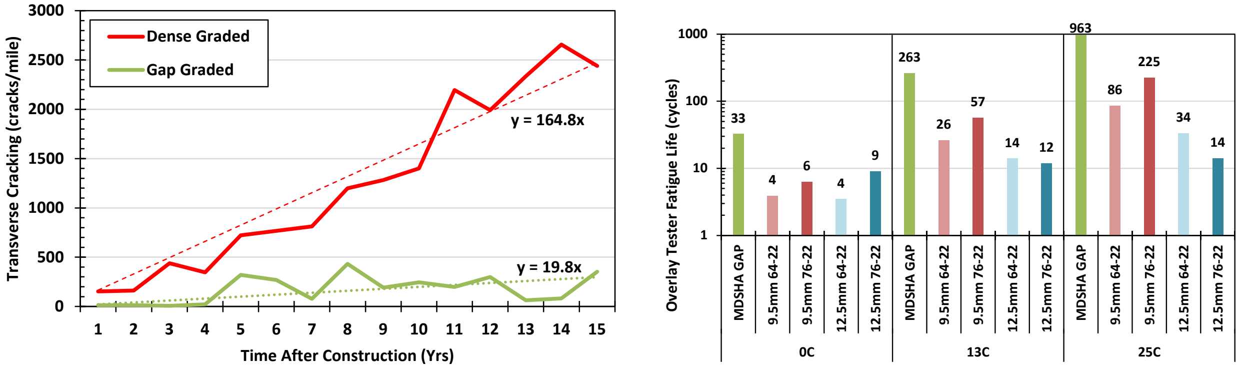
<u>Laboratory Prepared Loose Mix</u> – the final asphalt material type that could be used to develop performance criteria is laboratory prepared loose mix. This asphalt mixture type provides an "idea" on the expected performance for the mixture type. However, laboratory prepared asphalt mixtures lack the production specific characteristics that is difficult to simulate in the laboratory (i.e. – aggregate drying, addition of recycled asphalt, plant/silo storage aging, etc.). Overall, the laboratory prepared loose mix provides the least representative asphalt material for performance criteria development.

5.1.2 Utilizing Pavement Management System (PMS) Data

One of the most accurate methods of developing performance criteria is by linking field performance with the specific asphalt mixture performance from the various performance test methods. With this methodology, the state agency would evaluate the overall magnitude and yearly progression of the pavement distress of concern. This can be an extremely effective way of not only determining general performance thresholds, but also comparing different asphalt mixtures against one another on similar pavement conditions.

Figure 11a (*6*) shows the Maryland State Highway's transverse cracking data for the same pavement section but with two different asphalt mixtures; 1) Gap Graded and 2) Dense Graded (9.5mm 76-22) asphalt mixtures. The Overlay Tester results shown in the figure were conducted at different test temperatures for research purposes, however, the typical test temperature is 25°C. Laboratory performance testing was conducted on sampled loose mix that best represented the asphalt mixtures from the test sections, but were not from the identical pavement sections. The results in Figure 11 show a couple of things;

1. The Gap Graded asphalt mixture was far superior in mitigating transverse cracking on the composite pavement (Figure 11a) and in the Overlay Tester (Figure 11b). This is an important check as any laboratory performance test implemented should mirror the performance observed in the field.
2. Leading towards a performance criteria;
    a. When a value of the Overlay Tester is approximately 220 cycles or less, one would expect significant cracking. After only five years, over 800 transverse cracks can be expected.
    b. When a value of the Overlay Tester is approximately 900 cycles or greater, one would expect a much lower amount of cracking. After only five years, less than 100 transverse cracks can be expected. The PMS data also showed that it took approximately 4 years until cracking initiated.
3. Based on the general information described above and shown in Figures 11a and 11b, some initial performance criteria can already be proposed;
    a. Combining the results of Figures 11a and 11b, for composite pavement use, minimum performance in the Overlay Tester at 25°C should be >900 cycles. Granted, it can be assumed that with higher values of Overlay Tester even lower reflective cracking would occur. However, there will also be associated costs for which each state agency would need to consider.



(a) **(b)**

FIGURE 11 Maryland State Highway; (a) Pavement Management System Results; (b) Overlay Tester Fatigue Life Results (*6*)

5.1.3 Utilizing "Historical" or "Existing Visual" Field Performance

Unfortunately, not all state agency Materials Bureaus have a working relationship with their Pavement Management group. Therefore, there may be the need for the Materials Bureau to collect "Historical" information regarding certain asphalt mixtures on particular pavements. For example, a state agency may know that 9.5mm NMAS with PG76-22 have resulted in very good performance while 12.5 mm NMAS with PG64-22 were observed to show some distress on flexible pavements of 3 to 30 MESAL's. Therefore, a state agency could initiate a testing program to evaluate a variety of plant produced 9.5 mm NMAS with PG76-22 asphalt mixtures and utilize the average performance test value to establish a minimum performance criteria for surface courses placed on flexible pavements of 3 to 30 MESAL's. The methodology would also be applicable by testing field cores from the selected pavement sections.

A similar methodology can be applied when conducting a visual distress survey on an existing pavement. In this case, general knowledge of the age of the overlay/pavement is extremely helpful to establish whether or not the material is performing at an acceptable level. A selection of pavements with similar structure and traffic conditions are made and a visual distress assessment is conducted from either a windshield or roadside method. Knowing the different asphalt mixture surface courses applied to the evaluated pavement sections, collected loose mix representing the same mixture design/type found on the pavements in question can be evaluated and performance results determined. The methodology can also be utilized with field cores extracted from the pavement sections.

5.2 Proposed Initial Performance Criteria for BMD and PRS

In Tasks 3 and 4 of the study, state agencies were requested to provide laboratory performance data and Pavement Management data regarding the rutting and fatigue cracking field measurements in their respective state. The main premise of these tasks was to help develop performance criteria that the respective state agency could utilize that would relate a property measured in the laboratory to an observed field performance.

During the study, three (3) state agencies provided information – Connecticut, Maine and Vermont. It should be noted that each of the three states provided different levels of information for the analysis;

- Connecticut – Connecticut DOT provided Pavement Management information but did not have asphalt mixture performance testing data to accompany their pavement sections;

- Maine – Maine DOT provided asphalt mixture performance data for the Hamburg Wheel Tracking test (rutting) and IDEAL-CT Index (fatigue cracking), as well as Pavement Management information. However, the Pavement Management data only pertained to those specific pavement areas where the laboratory evaluated asphalt mixtures were placed; and

- Vermont – Vermont AOT provided asphalt mixture performance data for the Hamburg Wheel Tracking test (rutting) and SCB Flexibility Index (fatigue cracking). The entire Vermont Pavement Management database was accessible to download for evaluation.

The differences in available information provides a good insight on how state agencies can look to develop asphalt mixture performance testing criteria with variable levels of information. Factors such as pavement structure (i.e. – thickness, flexible/composite) and traffic should also be considered as significant factors and may need to be addressed in the performance specifications.

It should be noted that to truly develop confidence in the performance criteria, a strong communicative relationship needs to exist between the respective state agency's Materials Bureau and Pavement Management Division. Both groups need to work with one another to catalog and monitor the materials in the field. Initial criteria may need continual modification as materials, production, and construction practices continue to evolve.

5.2.1 Connecticut

The Connecticut Department of Transportation (CDOT) provided the Research Team with access to their Pavement Management data. However, CDOT has not yet initiated asphalt mixture performance testing. Therefore, attempts to directly compare lab to field performance could not be accomplished. In turn, the Pavement Management information was utilized to identify “Good” vs “Poor” pavement surface performance where CDOT could recover field cores and evaluate the relative performance of the asphalt materials.

Field Rutting

Pavement sections from CDOT’s Pavement Management System (PMS) were extracted to help define areas for future coring and laboratory evaluation. For this study, “Good” rutting performance was defined as wheelpath rutting less than 0.15 inches, while “Poor” rutting performance was defined as wheelpath rutting greater than 0.3 inches.

Tables 7 and 8 show the different Connecticut pavement sections noted as having Good and Poor rutting performance, respectively. The table is broken out by traffic level, which was defined as AADTT (ADT x % Trucks). To establish performance criteria, it is important to incorporate traffic level in the preliminary analysis to evaluate whether or not the field performance is dependent on the applied traffic levels. Additionally, the tables also contain the year the surface material was placed. Aging plays a critical role in the development of field distress and should also be considered when comparing test data. For example, rutting is generally an “early life” pavement distress – meaning that typically rutting occurs within the first few years after placement. Meanwhile, fatigue cracking is often a function of the amount of aging that occurs in the field, and therefore, is a “later life” pavement distress. With respect to rutting, one may not want to core field projects for lab characterization that are older than four to five years as the existing stiffness of those materials may be significantly higher than at the time when a majority of the permanent deformation was occurring (i.e. - < 3 years).

TABLE 7 Connecticut Asphalt Pavements with Good Field Rutting Performance

Good Rutting Performance (< 0.15 Inches)						
Traffic Level	**RoadName**	**From**	**To**	**ADT x % Trk**	**RUT_AVG**	**SURFACE_ YEAR**
Low	042 L	9.6	11.8	138	0.10	2016
	082 L	17.6	20	198	0.08	2016
	066 L	27.6	29.6	398	0.07	2018
	011 L	10	13.4	441	0.10	2015
	006 L	84	88	630	0.10	2016
	0.58 L	0.2	3.4	716	0.10	2015
Moderate	008 L&R	7.9	8.9	2944	0.11	2011/15
	072 L	3.2	3.9	2463	0.13	2017
	095 L	101.7	103.9	4335	0.07	2018
	291 L&R	3.2	5.1	4357	0.13	2010
	384 L&R	1.6	2.8	5345	0.09	2014/15
	084 L&R	16.7	18.7	6500	0.13	2008
High[1]	091 L&R	3.4	4.5	10479	0.13	2012
	084 L&R	61.1	62.4	12110	0.12	2014
	095 L	10	15	12848	0.10	2015/16
	084 L&R	65	66.3	14540	0.09	2012

[1] - Almost anywhere on 084, 091, and 095

TABLE 8 Connecticut Asphalt Pavements with Good Field Rutting Performance

Poor Rutting Performance (> 0.3 Inches)						
Traffic Level	**RoadName**	**From**	**To**	**ADT x % Trk**	**RUT_AVG**	**SURFACE YEAR[1]**
Low	201 L	15.5	17.7	68	0.43	2013
	695 L	2.1	2.7	214	0.4	2018
	167L	5.1	6.6	466	0.29	2018
	004L	36.5	38	550	0.29	2013
	044L	51.5	52.6	740	0.29	2014
Moderate	072L	2	2.8	2843	0.28	2017
High	084L	56.3	57.4	10688	0.33	2016

[1] - Older the resurface, more aged asphalt binder
(rutting may have occurred much earlier than present condition)

Field Fatigue Cracking

Similar to the Rutting analysis shown earlier, the CDOT PMS was mined to determine locations of "Good" and "Poor" fatigue cracking pavement sections. Table 9 contains recommended pavements sections that show relatively low levels of fatigue cracking, while Table 10 contains the "Poor" fatigue cracking pavement sections. For the analysis, the CDOT HPMS Crack Percentage and Wheelpath + Non-wheel Load Associated cracking parameters were used as field fatigue cracking indicators. Once again, AADTT and age of pavement surface are included in the tables for analysis purposes.

TABLE 9 Connecticut Asphalt Pavements with Good Field Fatigue Cracking Performance

Good Cracking Performance (HPMS Crk Pct & WP+NWL Crk)						
Road Name	From	To	ADT x % Trk	HPMS Crk Pct	WP + NWL Crk	SURFACE YEAR
001 L	14.2	15.8	618	0	4.9	2017
001 L	28.2	20.4	633	0	5.2	2017
009 L	0.6	4	1284	0	5.4	2011
030 L	17.9	20.9	189	0	0.4	2017
058 L	1	3.5	797	0	3.2	2015
084 L	19.1	22.1	5956	0	22.6	2012
091 L	39	42	12415	0	0.9	2015
095 L	95	98	5700	0	11	2014
198 L	8.2	11.9	88.2	0	2	2009
244L	0	3	74.8	0	4.7	2014
395 L	0.6	3.6	2398	0	8.3	2009

TABLE 10 Connecticut Asphalt Pavements with Poor Field Fatigue Cracking Performance

Poor Cracking Performance (HPMS Crk Pct & WP+NWL Crk)						
Road Name	From	To	ADT x % Trk	HPMS Crk Pct	WP + NWL Crk	SURFACE YEAR
030 L	2.0	3.5	374	32.5	213	2000
045 L	2.2	3.7	78	32.8	180	1995
083 L	19.5	21.9	219	33.5	218	1998
179 L	5.0	8.0	118	25.6	145	1999
201 L	7.6	9.2	50.4	22.8	99	1993
305 L	1.0	3.0	566	19.0	132	1999
534 L	0.5	3.5	182	29.7	193	1996

5.2.2 Maine

The Maine Department of Transportation (MaineDOT) provided the research team with asphalt mixture performance test results and PMS data for the pavement sections the respective asphalt material was placed. The complete PMS database was not provided, however, by utilizing the distress information for the exact pavement sections where the asphalt mixtures were placed, it is hopeful that a lab to field relationship can be established. For this study, MaineDOT utilized the Hamburg Wheel Tracking test for rutting evaluation and the IDEAL-CT Index test for fatigue cracking evaluation of asphalt mixtures.

General Mixture Performance – Fatigue Cracking

The asphalt mixture performance data was reviewed to see the general asphalt mixture performance for the IDEAL-CT Index testing as MaineDOT had previously identified fatigue cracking as the most prominent pavement distress. Figure 12 presents the different asphalt mixtures based on their relative RAP content. The general trend in the data shows that as RAP content increases, the IDEAL-CT Index decreases, which would be an indication of poorer fatigue cracking performance. It should be noted that the 0% RAP only had two mixtures with greatly varying IDEAL-CT Index performance (491.0 and 183.5). This is also highlighted by the large error bars, which indicate the standard deviation above and below the average value.

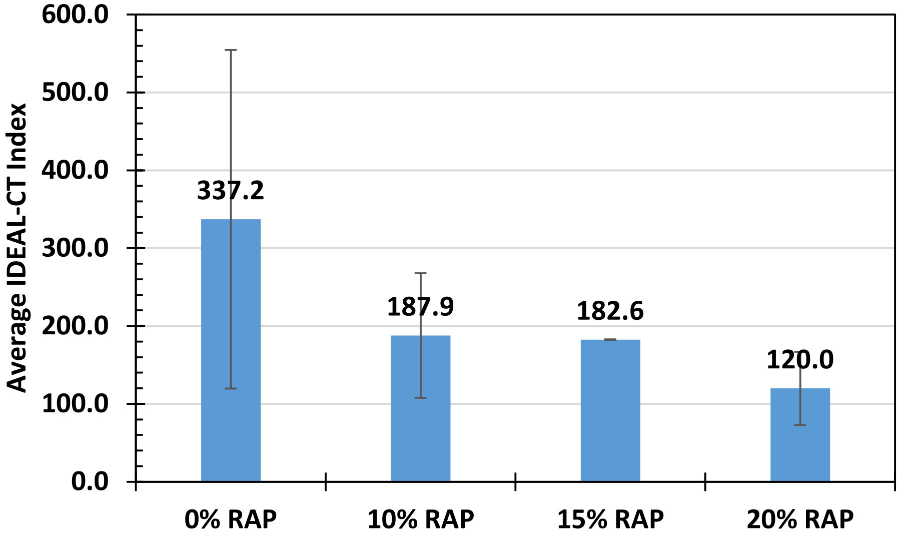


FIGURE 12 MaineDOT IDEAL-CT Index Values for Different RAP Contents

Further breaking out the asphalt mixtures by asphalt binder grade and RAP content shows that (Figure 13);

- For the PG64-28 asphalt binder, 0% to 15% RAP content results in very similar IDEAL-CT Index results. However, as the RAP content increased to 20%, there was a significant decrease in the values, similar to the results in Figure 12;

- The use of polymer-modified asphalt binders (PG64E-28 and PG70E-28) resulted in a significant increase in IDEAL-CT performance at 0% RAP. However, once RAP was added to the polymer-modified asphalt binders, the results were extremely similar to the unmodified PG64-28 asphalt binder results.

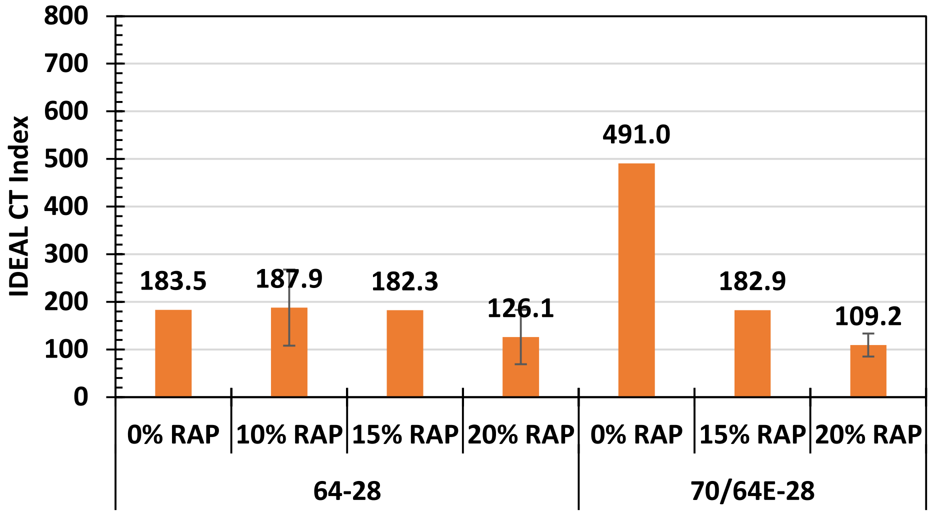


FIGURE 13 MaineDOT IDEAL-CT Index Values for Different RAP Contents and Polymer Modified Asphalt Binders

Fatigue Cracking – Lab vs Field

One of the difficulties in developing lab vs field relationships for fatigue cracking is that often fatigue cracking takes time to develop in the field. The aging the asphalt material undergoes plays a significant role in the cracking response. Further creating calibration issues are the different modes of cracking; Load Associated Wheelpath, Non-Load Associated (Outside of Wheelpath), Transverse Cracking (Non-Load Associated/Thermal Cracking), and Reflective Cracking (commonly found with composite pavements). Therefore, it is important that state agencies understand the need to allow for additional time for fatigue cracking analysis, as well as the need to categorize and separate different pavement types (i.e. – flexible vs composite) and possibly even different cracking modes (i.e. – load associated cracking vs thermal cracking) when possible. For the MaineDOT analysis, a few cracking parameters from the PMS were used to compare to the IDEAL-CT Index; 1) % Cracking, 2) CRACK FUNC, and 3) CRACK STRC. Additionally, only the latest cracking measurements (2019) were used in the analysis to allow as much time to have passed after construction. Information was not provided on how the indices were calculated, however, the general relationship between the parameters are shown in Figure 14. Additionally, it is not known what the threshold values are for these indices before MaineDOT takes some type of maintenance action.

As mentioned earlier, cracking performance is significantly influenced by aging, and therefore, the amount of time after the asphalt mixture has been placed. An attempt to compare the MaineDOT Cracking Indices and the time after the material was placed is shown in Figure 15. Unusual to notice that there appeared to be cracking occurring in earlier stages after placement as opposed to later in the pavement life. This may be an indication that the cracking observed on these particular projects may not necessarily be due to aged induced factors, but most likely from load associated and pavement structure factors.

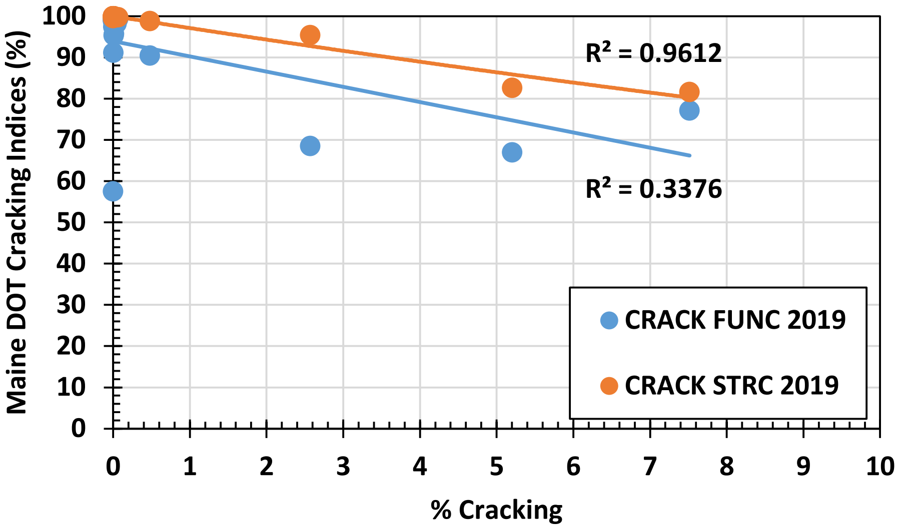
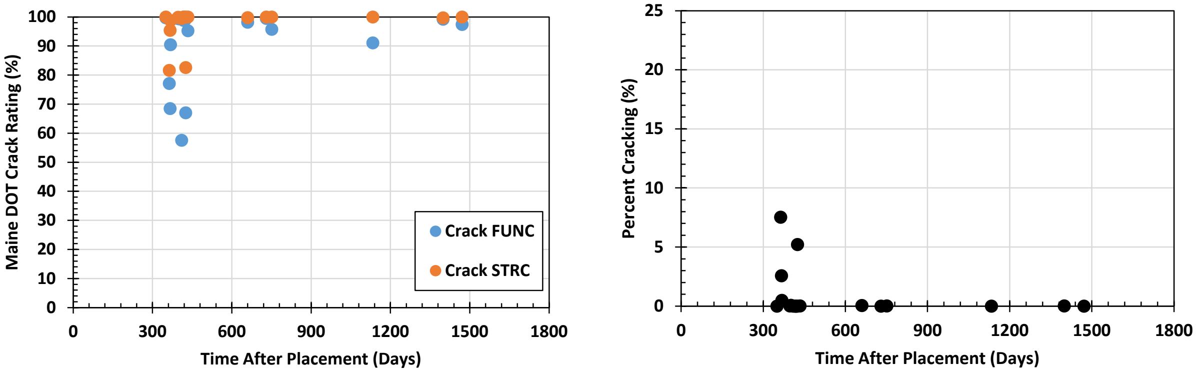


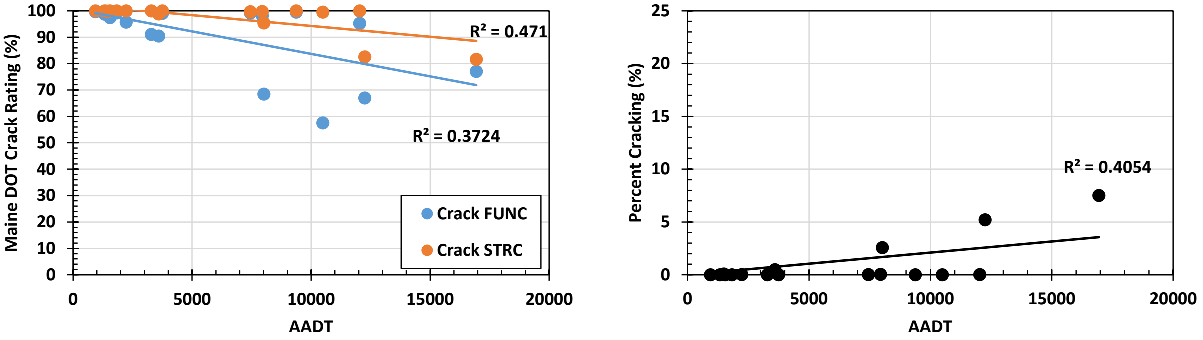
FIGURE 14 Relationship Between MaineDOT PMS Cracking Indices with 2019 Field Data



(a) **(b)**

FIGURE 15 Comparison of MaineDOT IDEAL-CT Pavement Sections; (a) Crack Ratings (CRACK FUNC and CRACK STRC) vs Time After Placement; (b) Crack Ratings (% Cracking) vs Time After Placement

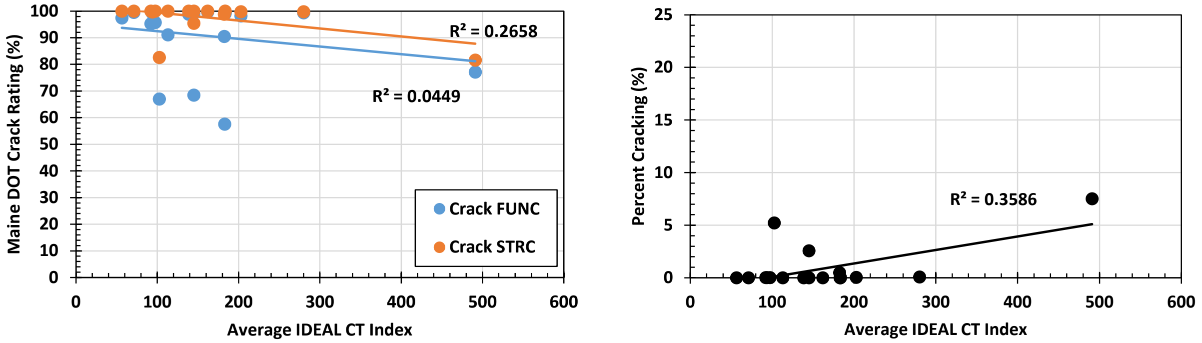
Figure 16 shows the same cracking indices from Figure 15, but this time, compared to the Average Annual Daily Traffic (AADT). Pavement designs are commonly done with Average Annual Daily Truck Traffic (AADTT) as it is well known that truck traffic generates a significantly larger amount of distress on a pavement than car traffic. However, Percent Trucks was not provided in the data, so the traffic information is represented by AADT. In Figure 16, there does appear to be a moderate relationship between the MaineDOT PMS cracking indices and AADT. This would suggest that within the timeframe these particular asphalt mixtures were placed, the primary factor creating the measured cracking distress was the traffic. Therefore, traffic levels may need to be included in future performance criteria for Maine’s Balanced Mixture Design.



(a) (b)

FIGURE 16 Comparison of MaineDOT IDEAL-CT Pavement Sections; (a) Crack Ratings (CRACK FUNC and CRACK STRC) vs AADT; (b) Crack Ratings (% Cracking) vs AADT

An initial attempt was made to directly compare the IDEAL-CT Index values to the MaineDOT PMS cracking indices. Figures 18a and 18b show the results of the analysis. The overall trends are counter-intuitive than one would expect or hope. The results in Figures 17a and 17b show that as the IDEAL-CT Index value increases, the measured cracking in the field also increases. One would expect that good performing asphalt mixtures (i.e. – higher IDEAL-CT Index values) should result in better field cracking performance.



(a) (b)

FIGURE 17 MaineDOT Cracking Indices Compared to IDEAL-CT Index Values

Additional analysis was conducted to determine why the relationship between lab and field cracking did not follow an expected trend. Figures 18 through 20 show the asphalt mixtures broken out by asphalt binder grade and RAP percentage while compared to the MaineDOT cracking indices. Overall, the PG64-28 asphalt binder mixtures appeared to have lower field cracking when compared to the polymer modified PG64E-28 and PG70E-28. In fact, the pavement section with the worst cracking performance also happened to be the section with the PG70E-28 asphalt binder. Once again, this is counter intuitive to what one would expect as this would have to be a highly polymer modified asphalt binder.

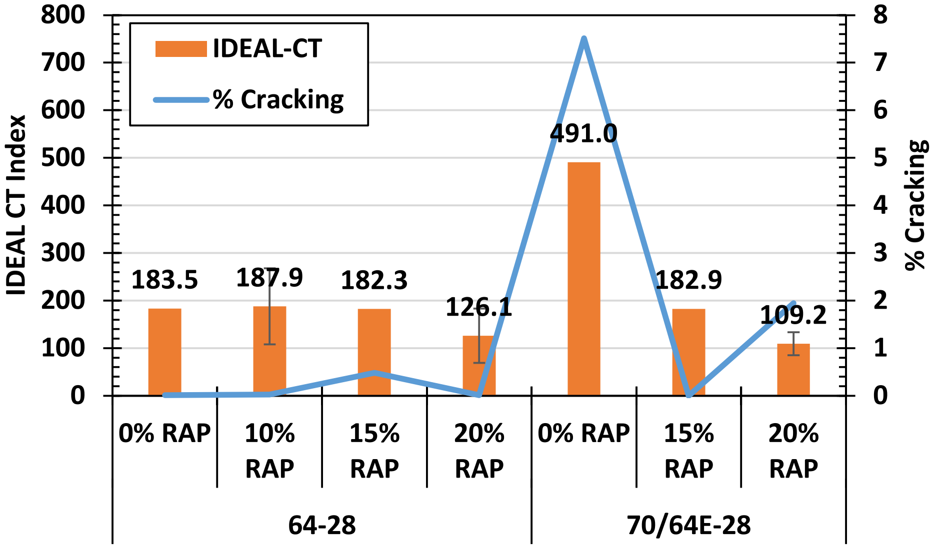


FIGURE 18 IDEAL-CT Index and MaineDOT % Cracking for Different Asphalt Binder Grades and RAP Contents

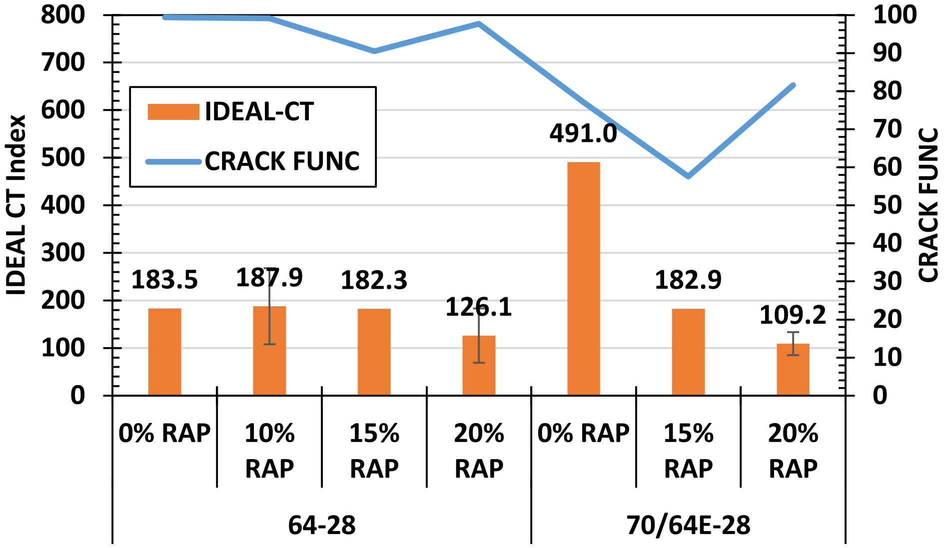


FIGURE 19 IDEAL-CT Index and MaineDOT CRACK FUNC for Different Asphalt Binder Grades and RAP Contents

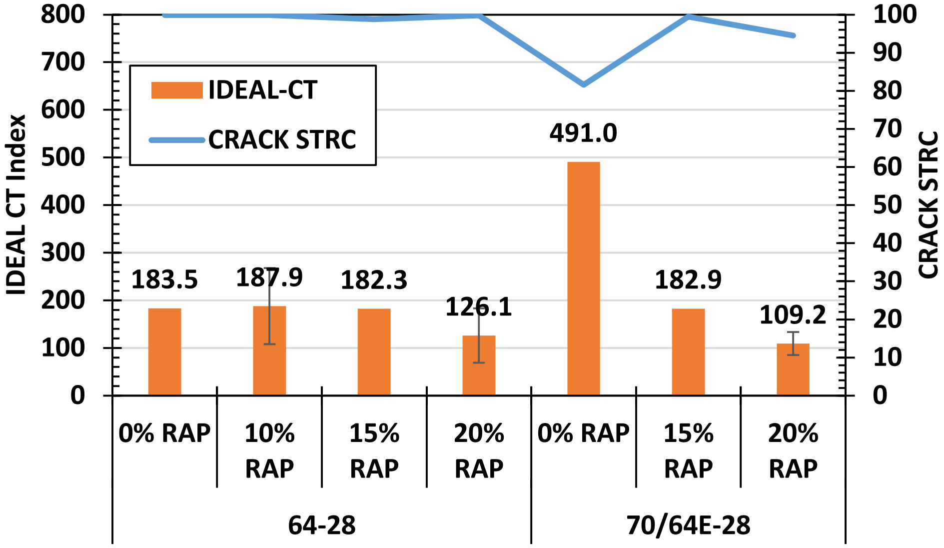


FIGURE 20 IDEAL-CT Index and MaineDOT CRACL STRC for Different Asphalt Binder Grades and RAP Contents

The previous figures showed a troubling situation occurring where the polymer-modified asphalt mixtures were under-going higher levels of field cracking within the first five years of service life. However, as shown earlier, the field cracking was found to be related to the traffic levels (AADT). AADT and the IDEAL-CT Index values were plotted against the different asphalt mixtures (PG grade and RAP content). Figure 21 shows the results of the analysis. It is clear from the graph that higher traffic levels are associated for the polymer-modified asphalt mixtures. In fact, the PG70E-28 pavement section that achieved the IDEAL-CT Index had the highest level of traffic. Figure 21 clearly demonstrates how significant the impact of traffic volume was in the analysis, and that traffic levels should be included in the performance criteria.

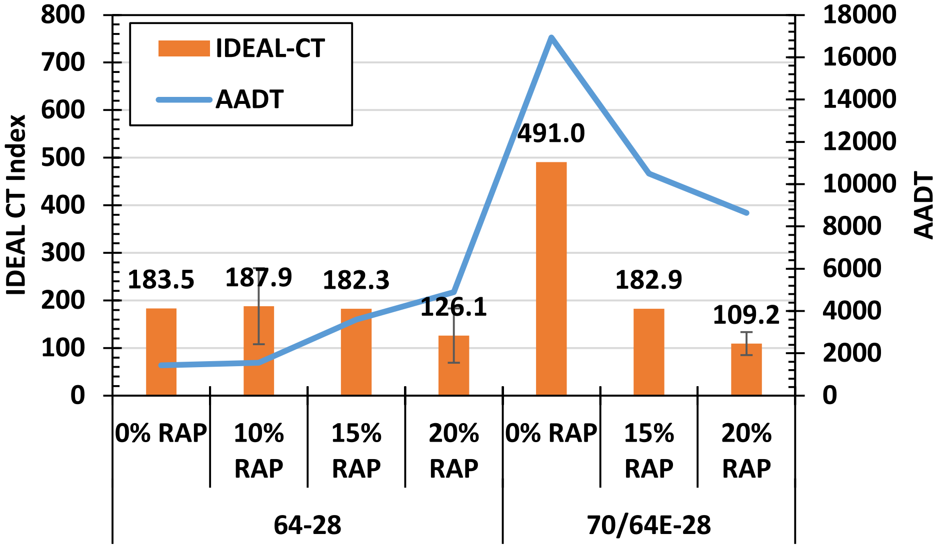


FIGURE 21 Resultant IDEAL-CT Index Values for Different Asphalt Mixtures with the Pavement Sections' Traffic Volume (AADT)

To help move forward in establishing some recommendations for IDEAL-CT Index criteria for Balanced Mixture Design, the data provided was broken out into three different traffic categories; 1) Less than 5,000 AADT; 2) 5,000 to 10,000 AADT; and 3) Greater than 10,000 AADT. Table 11 contains the data used and divided into these divisions.

Using the IDEAL-CT Index values broken out by the AADT reported for the pavement section constructed, Figure 22 was generated. It should be known that the four asphalt mixtures evaluated for AADT > 10,000 contained the PG70E-28 asphalt mixture that achieved a 491.0 IDEAL-CT Index. This greatly influenced the average test results so two different data are shown to represent the average IDEAL-CT Index at > 10,000 AADT – one with and one without the PG70E-28. Assuming that the PG70E-28 is not very common, the average results without the PG70E-28 is used for comparison among the other mixtures. The MaineDOT test results show that as the AADT increases, the average IDEAL-CT Index values decrease. Further review of the individual mixture designs would be required to help determine the exact reasoning, however, one of the most likely reasons is the increase in gyration level as traffic level increases. Typically, as gyration level increases, the aggregate skeleton is pushed tighter together, essentially squeezing out asphalt and thereby lowering effective asphalt contents. Lower effective asphalt contents, in conjunction with higher traffic levels, could have led to the higher levels of cracking observed in the MaineDOT fatigue cracking indices.

TABLE 11 Asphalt Mixture Fatigue Cracking Data for Different AADT Divisions

AADT	Age (Days)	PG Grade	RAP %	Ave CTI	CRACK FUNC 2019	CRACK STRC 2019	Percent Crack
942	350	64-28	20	161.8	99.683	100	0.0001
1327	418	64-28	10	138.5	98.775	100	0.0001
1440	397	64-28	0	183.5	99.392	99.906	0.0105
1488	401	64-28	10	280.1	99.341	99.721	0.068
1546	1471	64-28	20	56.5	97.433	99.992	0.003
1830	418	64-28	10	145.0	99.224	100	0.0001
2230	751	64E-28	20	97.3	95.709	99.967	0.011
3286	1133	64-28	20	113.1	91.107	99.977	0.002
3596	368	64-28	15	182.3	90.431	98.813	0.48
3747	424	64-28	20	183.5	99.044	100	0.0001
7444	1399	64-28	20	93.9	99.188	99.686	0.018
7944	660	64-28	20	202.8	98.166	99.754	0.044
8017	366	64E-28	20	145.0	68.484	95.411	2.571
9375	730	64-28	20	71.2	99.487	100	0.001
10493	410	64E-28	15	182.9	57.531	99.478	0.0001
12035	434	64E-28	20	92.1	95.277	99.962	0.009
12248	425	64E-28	20	102.6	66.992	82.589	5.205
16939	363	70E-28	0	491.0	77.078	81.617	7.514

Figures 23 and 24 show the MaineDOT PMS cracking indices also broken out into the three AADT divisions. It is very clear from the graphs that as the AADT increases, greater magnitudes of cracking are observed.

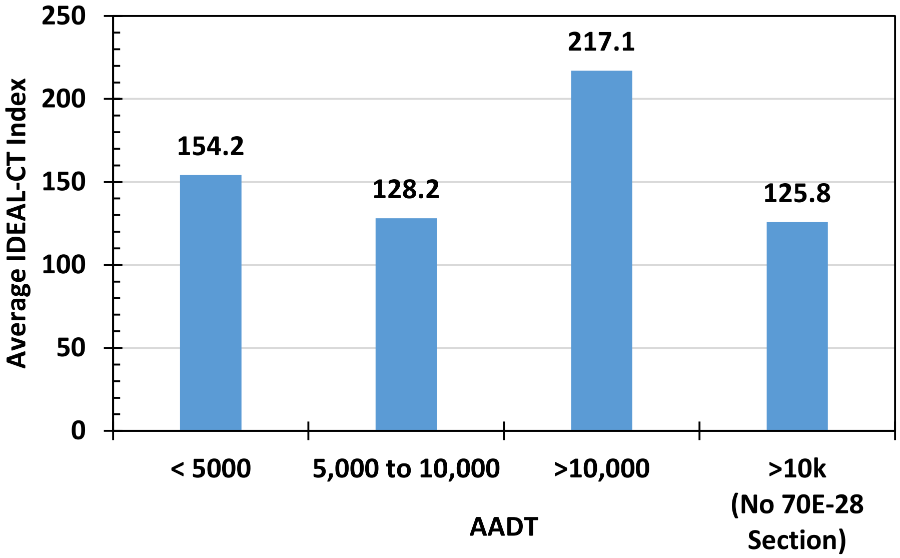


FIGURE 22 MaineDOT AADT Divisions and Resultant IDEAL-CT Index Values

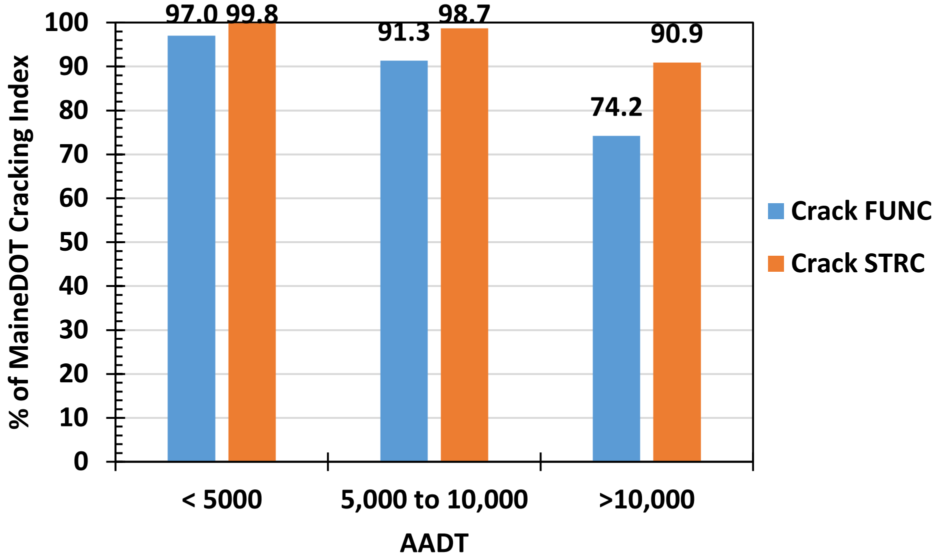


FIGURE 23 MaineDOT AADT Divisions and Resultant PMS Cracking Indices (Crack FUNC and Crack STRC)

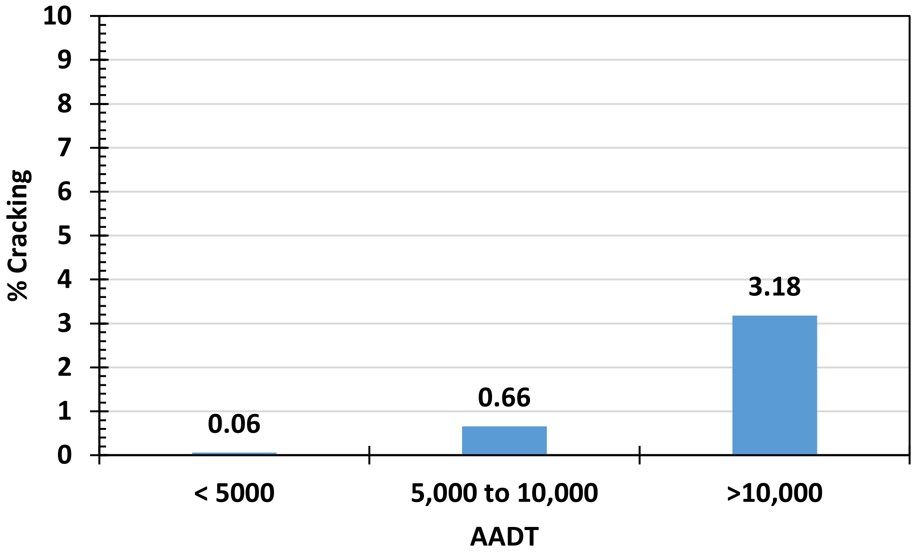


FIGURE 24 MaineDOT AADT Divisions and Resultant PMS Cracking Index Percent Cracking

Fatigue Cracking – MaineDOT Final Recommendations

Understanding that the laboratory tested asphalt mixtures are less than five years old in the field, preliminary recommendations for IDEAL-CT Index values for MaineDOT are provided based on the following;

- Average value of 154.2 was measured for AADT < 5,000 and indicated good field performance after an average of 1.7 years for the ten field sections evaluated;
- Average value of 128.2 was measured for AADT 5,000 to 10,000 and indicated good field performance after an average of 2.2 years for the four field sections evaluated. However, it must be noted that greater magnitudes of field cracking were observed when increasing the AADT from < 5,000 to 5,000 to 10,000 AADT; and
- Average value of 125.8 (excluding the PG70E-28) was measured for AADT > 10,000 and indicated that field performance began to show greater magnitudes of cracking over the other two AADT divisions after an average of 1.1 years for the three field sections evaluated.
- Traffic levels had an impact on the MaineDOT PMS cracking distress indices.

Preliminary minimum IDEAL-CT value for MaineDOT should be set at 150 at a test temperature of 25°C. In addition, further research should be conducted to verify whether or not the IDEAL-CT value should be increased for higher levels of traffic.

MaineDOT should also look at whether or not the test temperature of 25°C is best to represent their climate conditions when utilizing the IDEAL-CT test. Work conducted under NCHRP Project 9-59 recommended that a better method to represent intermediate test temperature is to utilize the low temperature PG grade and the relationship shown in Figure 25 (*7*). Using the LTPPBind3.1 software, the low temperature PG grade was determined at a 98% reliability. This shows Maine has two different low temperature grades; -28°C along the coastal area and -34°C inland (Figure 26). The resultant intermediate test temperatures, based on the recommendation from NCHRP 9-59, would then be 22°C and 19°C, respectively.

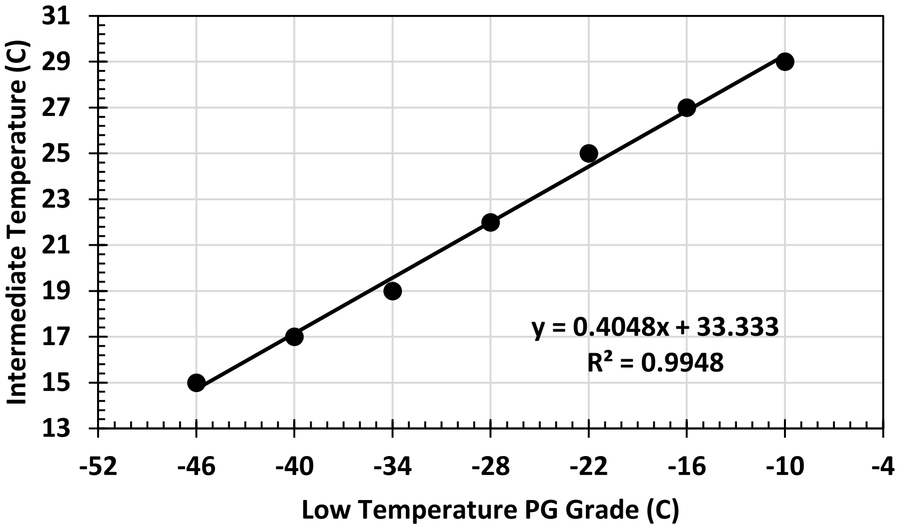


FIGURE 25 Recommended Intermediate Temperature for Fatigue Cracking Analysis Based on Representative Low Temperature PG Grade (*7*)

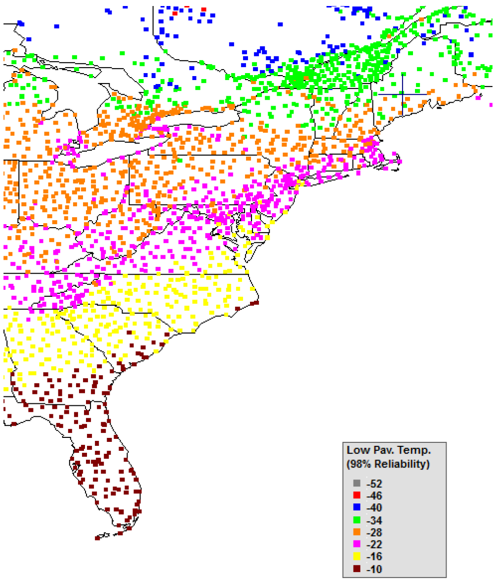


FIGURE 26 Low Temperature PG Grade Determined at 98% Reliability Using LTPPBind 3.1

Rutting – Lab vs Field

The MaineDOT has proposed to use the Hamburg Wheel Tracking Test (AASTO T324) as the test procedure to evaluate the rutting potential of asphalt mixtures. At this time, it appears that a test temperature of 45°C is being selected for use, although there were occasions where the laboratory technicians utilized temperatures of 42°C and 48°C. However, the majority of the test data collected, and used in the analysis, was 45°C.

A first attempt at comparing the field measured rutting and the Hamburg rutting is shown as Figure 27. As the figure clearly shows, no direct relationship existed between the test data and field rutting based on their raw measurements.

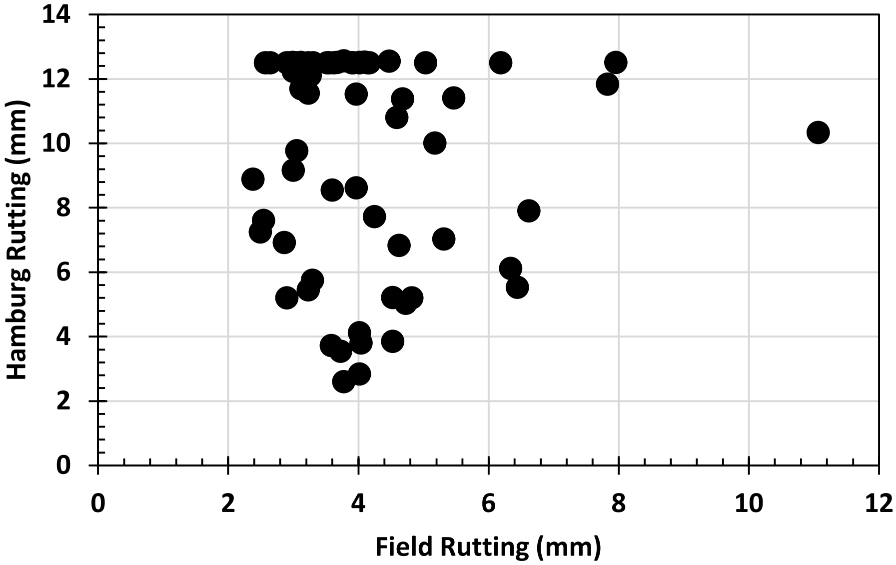


FIGURE 27 Measured Field Rutting vs Hamburg Wheel Tracking Rutting of Asphalt Mixture Placed on Pavement Sections

Additional analysis was conducted to help determine what factors may be critical to consider when trying to develop a relationship between field rutting and the Hamburg rutting. First, the AADT for the pavement sections were compared to the measure field rutting (Figure 28). As the figure shows, there is a logical relationship (i.e. – more traffic equals more rutting). However, there is quite a bit of scatter in the data and a relatively poor statistical relationship exists.

In a correspondence with Mr. Dale Peabody, Mr. Peabody mentioned that historically, asphalt pavements in Maine tend to continue to show rutting past the typical 1 to 2 years most state agencies observe on their pavements. Although more research is necessary to determine the exact reasons, one can make the initial assumption that due to Maine's moderate temperatures, the asphalt materials age/stiffen at lower rates than observed in central and southern states.

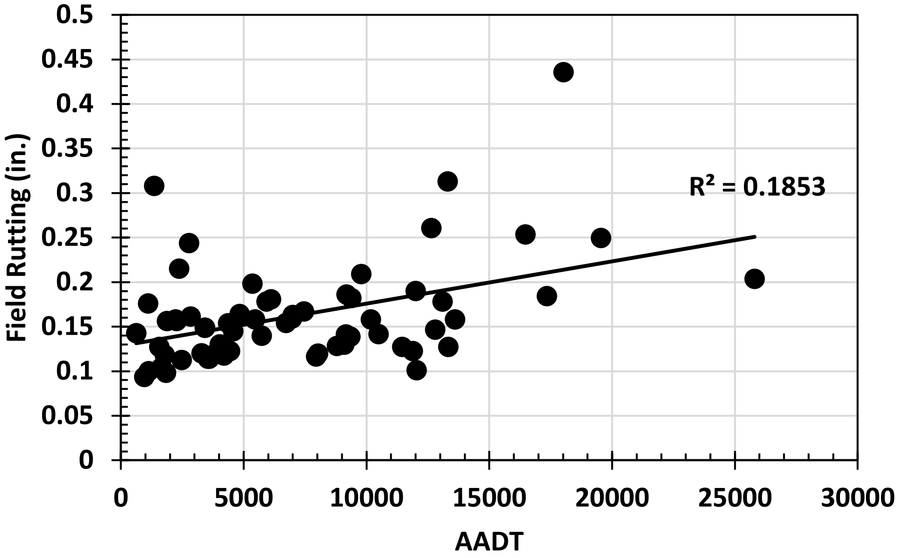


FIGURE 28 Field Rutting vs AADT Measured by MaineDOT

To determine if the field rutting did increase with time, the field rutting from the laboratory test data pavement sections were compared against the time after construction. The data set was also filtered to show the AADT (Figure 29). The data in Figure 29 does actually show the magnitude of field rutting increases over time, at least within the four year period of the provided test data. It should be noted that these are not the same sections evaluated each year, but different pavement sections of similar asphalt mixtures with similar Hamburg Wheel Tracking properties.

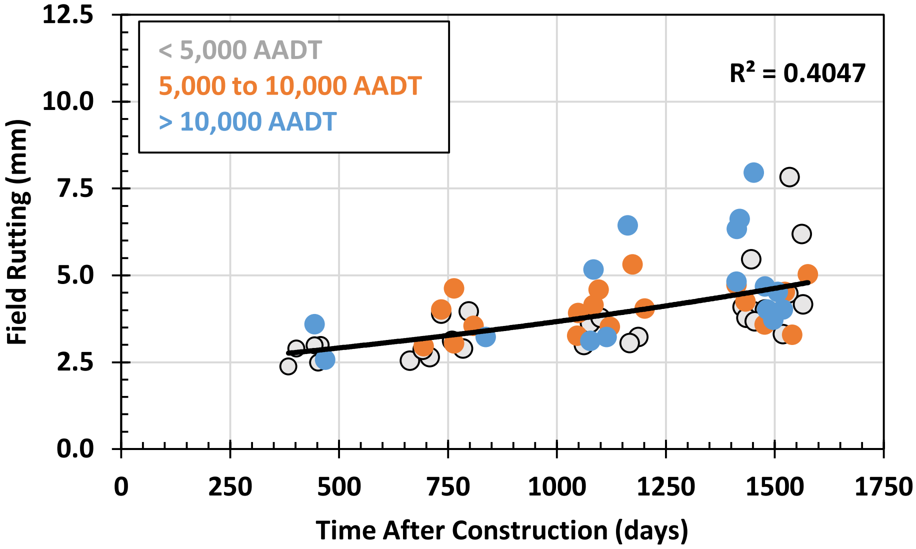


FIGURE 29 Field Rutting vs Time After Construction in Maine

The trend in Figure 29 causes a dilemma with generating performance criteria since it is unknown at this time exactly when the rutting stops on the Maine asphalt pavements. Therefore, to help with developing a preliminary criteria, only the laboratory and corresponding field data that are approximately four years or older was used in the analysis. The final data used to generate a preliminary criteria is shown in Table 12.

The final test data to use in the preliminary Hamburg rutting criteria is shown in Figure 30. The data does follow an increasing linear relationship whereas Hamburg rutting increases, so does the field rutting. More scatter in the data can be witnessed for the lower volume traffic as opposed to the higher volume traffic. This is most likely due to the fact that the same Hamburg Wheel Tracking test protocols are used regardless of traffic level in the field. Simply put, there is no attempt to modify the loading magnitude in the Hamburg to better represent the loading conditions in the field, even though asphalt mixture selection is modified to consider traffic volume (i.e. – differences in gyration level, modified asphalt binders, etc.). Therefore, traffic level should be included within the final criteria to help distinguish between the needs in the field.

TABLE 12 MaineDOT Field Rutting and Hamburg Wheel Tracking Performance Results for Pavement Surfaces at Least Four Years Old

AADT	Age	Temp	Field Rutting (inches)	Field Rutting (mm)	HWT Rutting (mm)	HWT Passes	Stripping Slope	Creep Slope	Stripping	LC12 5S	LC12 5C
1106	1488	45	0.176	4.47	12.50	4626	4.27	1.45	2.94	4614	4592
1346	1491	45	0.308	7.82	11.83	16823	1.34	0.35	3.81	17508	20142
1875	1409	45	0.156	3.96	11.53	10022	2.34	0.64	3.65	17425	17390
2225	1435	45	0.158	4.01	12.50	9702	3.05	0.38	8.06	9699	9676
2365	1403	45	0.215	5.46	11.41	11633	1.61	0.85	1.88	14000	15856
2766	1519	45	0.244	6.18	12.50	12487	1.82	0.44	4.11	12487	12487
2834	1383	45	0.161	4.09	12.50	4452	5.44	1.08	5.03	4448	4434
3427	1392	45	0.149	3.77	2.60	19891	0.08	0.12	0.65	99751	99751
4022	1476	45	0.130	3.30	12.50	4680	6.01	1.12	5.35	4680	4680
4567	1411	45	0.145	3.67	12.50	6252	3.17	0.82	3.88	6249	6240
4825	1522	45	0.164	4.17	12.50	7346	3.87	0.72	5.39	7344.5	7332
5348	1533	45	0.198	5.03	12.50	7141	3.99	0.49	8.18	7140	7130
5924	1480	45	0.178	4.52	5.21	19994	0.34	0.17	1.96	41666	62501
7444	1390	45	0.167	4.24	7.73	20000	0.47	0.18	2.55	60023	62908
9094	1497	45	0.130	3.29	5.75	19996	0.41	0.16	2.61	36267	62528
9160	1434	45	0.141	3.58	3.72	19967	0.15	0.14	1.07	77220	83335
9186	1369	45	0.186	4.72	5.03	19932	0.15	0.26	0.58	48420	48420
10175	1440	45	0.158	4.01	4.12	19984	0.17	0.09	1.83	69131	110055
11999	1369	45	0.190	4.82	5.20	18620	0.72	0.19	3.87	52703.6	75457.3
12632	1377	45	0.261	6.62	7.91	16726	1.04	0.23	4.52	51310	51291
12793	1454	45	0.147	3.72	3.55	19943	0.13	0.15	0.87	79968.67	81595
13088	1464	45	0.178	4.52	3.85	19954	0.13	0.16	0.83	73570	73570
13298	1409	45	0.313	7.95	12.50	3074	8.00	1.20	6.65	3073	3066
13604	1476	45	0.158	4.01	2.84	19909	0.10	0.10	0.97	105189	115616
17347	1435	45	0.184	4.67	11.38	14791	1.63	0.42	3.86	18007	21051.2
19549	1370	45	0.250	6.34	6.12	19988	0.44	0.17	2.58	34352	57008

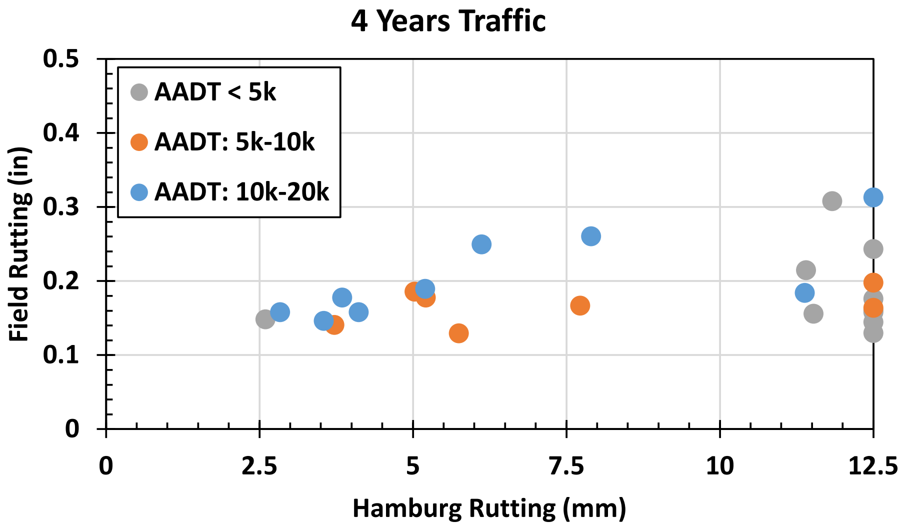


FIGURE 30 Hamburg Wheel Tracking Rutting vs Field Rutting Collected by MaineDOT

Rutting – MaineDOT Final Recommendations

The Hamburg rutting and field rutting from Table 12 was broken out and averaged for every 5000 AADT range. This is shown in Figure 31 with error bars that represent the standard deviation above and below the average. The data shows that the average field rutting for the different levels of AADT range between 0.16 inches and 0.21 inches. If using the nomenclature of "Good" and "Poor" rutting performance as denoted during the Connecticut analysis, the Maine's rutting field performance would fall in the "average" area, somewhere between 0.15 inches and 0.30 inches. This would indicate that a selection for the criteria of Hamburg rutting should not deviate much more than what the average values currently fall on. By selecting a lower Hamburg rutting magnitude would be very conservative since the pavements are not showing severe rutting issues, while selecting a higher value may actually lead to field rutting higher than the 0.15 - 0.20 inches currently witnessed. Utilizing this rationale, Figure 32 was developed and represents the "PASS-FAIL" Hamburg Wheel Tracking rutting criteria for different AADT levels on Maine's asphalt pavements. The rutting criteria is based on testing the asphalt mixtures to 20,000 loading cycles at a test temperature of 45°C.

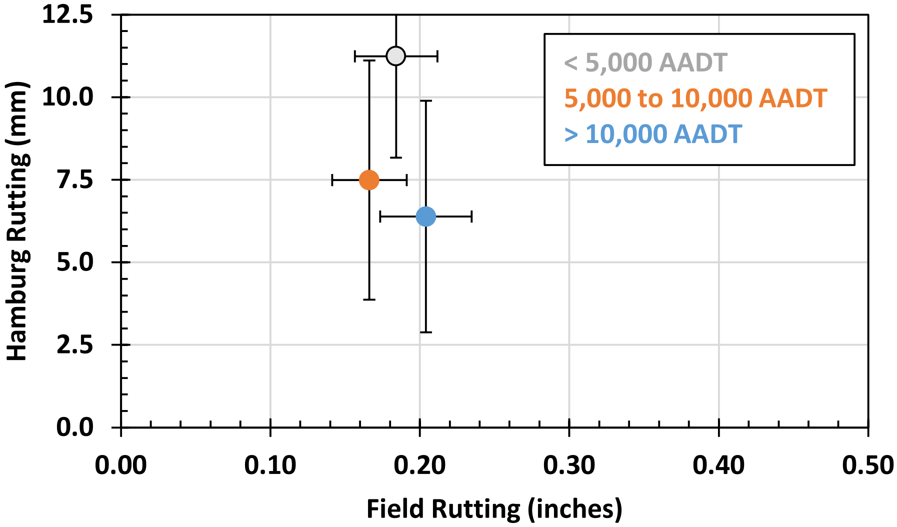


FIGURE 31 Average Field Rutting vs Average Hamburg Rutting (Error Bars Represent Standard Deviation Above and Below Average)

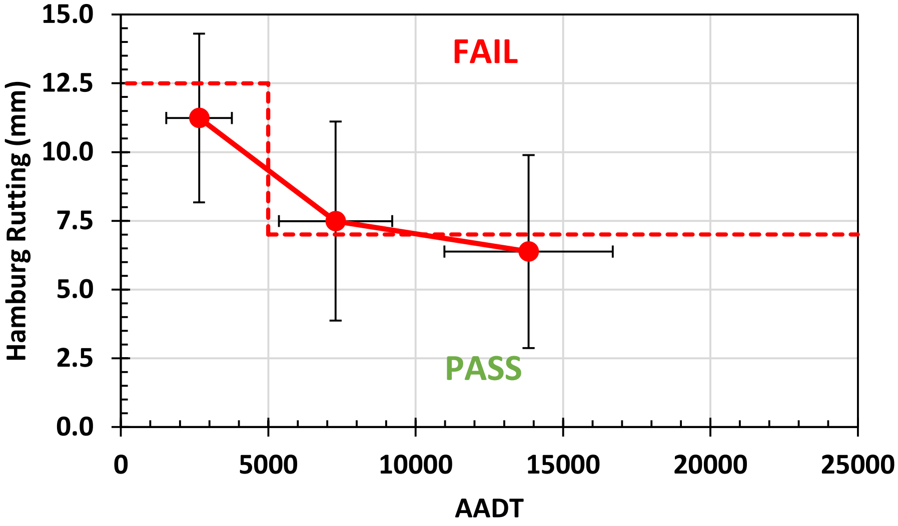


FIGURE 32 Proposed Hamburg Wheel Tracking Criteria for MaineDOT Asphalt Mixtures

5.2.3 Vermont

The Vermont Agency of Transportation (VTrans) provided the research team with asphalt mixture performance test results and a link to download the complete set of Pavement Management distress information. In the study, VTrans is utilizing the Hamburg Wheel Tracking test to measure the rutting resistance of asphalt mixtures while using the SCB Flexibility Index to characterize the fatigue cracking potential of their asphalt mixtures. The rutting analysis was conducted with lab and field data for surface course asphalt mixtures with up to three years of performance history. The fatigue cracking analysis was conducted with lab and field data with approximately only two years of performance history.

General Mixture Performance – Fatigue Cracking

As mentioned earlier, the Vermont Agency of Transportation (VTrans) uses the SCB Flexibility Index (AASHTO TP124) at a test temperature of 25°C to evaluate fatigue cracking properties of their asphalt mixtures. Figures 33 and 34 show a summary of the different asphalt mixtures tested and placed on Vermont asphalt pavements. Overall, the average SCB FI results appear to very good with the lowest average value measured of 6.7 for the 2019 Type IIS with 15% RAP. Preliminary research has shown that an SCB FI value greater than 8.0 generally show good field performance.

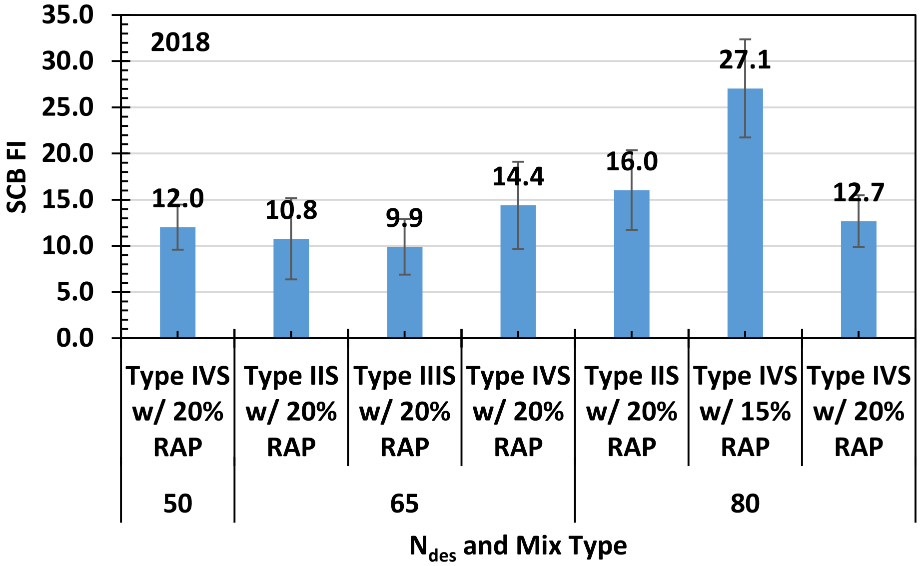


FIGURE 33 SCB Flexibility Index Results for 2018 VTrans Tested Asphalt Mixtures

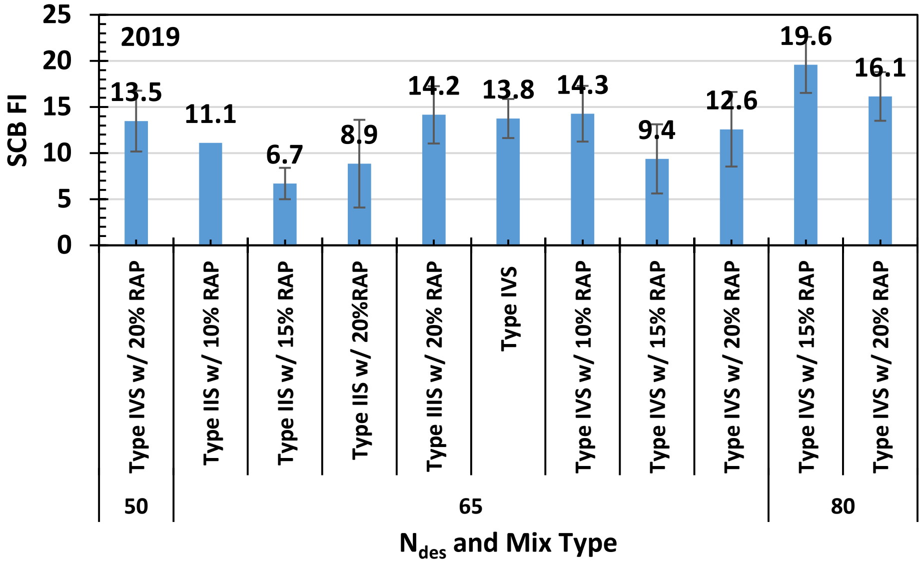


FIGURE 34 SCB Flexibility Index Results for 2019 VTrans Tested Asphalt Mixtures

Table 13 and Figure 35 show asphalt mixtures produced in both 2018 and 2019 and their respective SCB Flexibility Index values. For the most part, there is general agreement between the two different years respective fatigue cracking performance showing relatively good consistency in the mix types in Vermont.

TABLE 13 Comparison of 2018 and 2019 SCB Flexibility Index Performance for Same Mixture Type in Vermont

N_{des}	Mix Type	Year Produced	SCB FI	
			Average	Std Dev
50	Type IVS w/ 20% RAP	2018	12.0	2.4
		2019	13.5	3.3
65	Type IIS w/ 20% RAP	2018	10.8	4.4
		2019	8.9	4.8
	Type IIIS w/ 20% RAP	2018	9.9	3.0
		2019	14.2	3.1
	Type IVS w/ 20% RAP	2018	14.4	4.7
		2019	12.6	4.0
80	Type IVS w/ 15% RAP	2018	27.1	5.3
		2019	19.6	3.0
	Type IVS w/ 20% RAP	2018	12.7	2.8
		2019	16.1	2.6

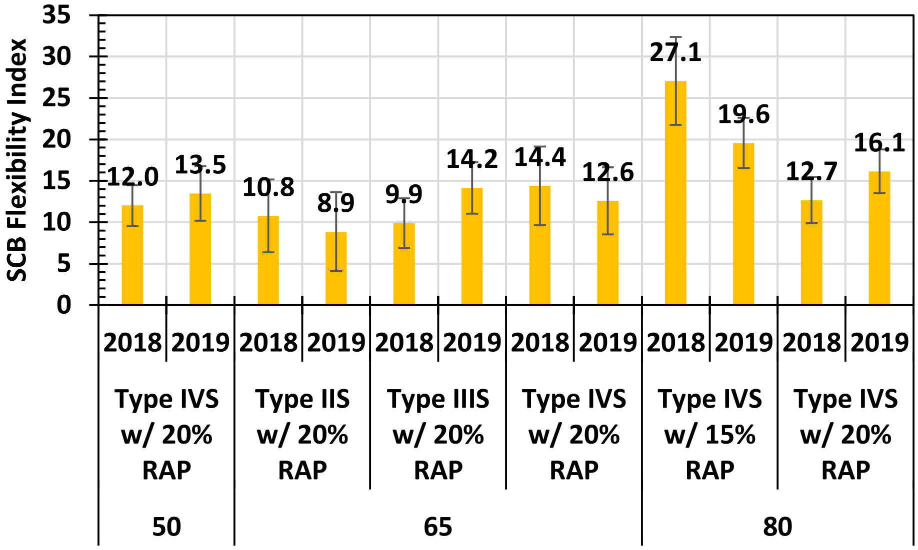


FIGURE 35 Comparison of 2018 and 2019 SCB Flexibility Index Performance for Same Mixture Type in Vermont

Fatigue Cracking – Lab vs Field

Due to the relatively "young" nature of the asphalt materials placed, only the 2018 asphalt mixtures were used to attempt to develop a preliminary SCB Flexibility Index Criteria. Two VTrans PMS parameters were used to compared to the laboratory fatigue cracking tests; 1) Trans Index and 2) NPRM WP Crack. Although definitions were not provided, it is assumed that the "Trans Index" is related to the extent of transverse cracking while the NPRM WP Crack is a measure of load associated wheel path cracking. The final data set used to evaluate a tentative criteria is shown in Table 14.

An initial comparison between the measured SCB Flexibility Index values and the VTrans fatigue cracking indices are shown in Figure 36. The figure indicates that most of the pavement sections with SCB Flexibility Index data available are in relatively good condition. There are some pavement sections that are showing levels of cracking. An interesting trend in the data does seem to indicate that as the cracking distress level increases, the SCB Flexibility Index also increases. This is counter-intuitive to what one would expect and hope for.

TABLE 14 2018 VTrans Fatigue Cracking Data

Project	Mix Type	Site Manager Sample ID	Average NPRM WP Crack Over Project Length (per 0.1 mile)	Tran Index	Ave. AADT	Ndes	Ave SCB FI	FI COV (%)	Avg. Fracture Energy	Fract Energy COV (%)	Avg. Strength	Strength COV (%)
Bennington NH 2966(1)	Type IVS w/ 20% RAP	tarel188D061826	19.92	87.3	6800	65	27.1	8	2276.8	12	297.1	6
		scrowley1896145508					18.5	21	2736.7	9	401.1	9
		dconnoll189R084245					18.2	13	2614.7	10	376.1	10
		tarel18A2093504					18	13	2357.6	13	358.9	14
Bennington - Wilmington NH SURF(51)	Type IVS w/ 20%	tcoletta1865055842	0.40	99.43	4325	65	16.3	16	1932.6	4	320.4	5
Montpelier STP 2950(1)	Type IVS w/ 20%	jjacobso186Q075145	0.28	96.54	4900	65	8.7	15	2239.5	5	461.4	5
St. Albans City STP 2957(1)	Type IVS w/ 20% RAP	scrowley185D180532	1.00	89.83	9200	80	9.5	23	2017.6	11	376.6	6
		tarel186J165054					14.8	10	1901.2	7	303.2	5
		jbreton187G173419					13.7	17	2187.7	5	353.8	5
Reading - Windsor STP FPAV(11)	Type IVS w/ 20% RAP	gporter189D105848	0.03	99.94	1450	50	8.9	5	2349.4	1	470.9	3
		tarel189J145920					13.1	27	2429.3	9	435.1	6
		tarel189O094332					10	11	2242	6	433.1	5
Weathersfield - Reading STP FPAV(12)	Type IVS w/ 20% RAP	jjacobso187R061056	0.00	100.00	1250	50	16.9	18	2572.2	5	414	7
		ldonavan187R080905					14.3	17	2295.3	25	397	26
		gporter1888073953					12.8	29	2528	9	433.2	3
		tarel189Q133152					12.2	18	2555.7	10	438	5
		jbreton18A4054111					13	20	2438.8	4	430.2	8
		etavares18AC055915					12	20	2612	6	446.8	6
		etavares18AI145911					9.8	16	2273.9	6	429.4	9
		etavares18AG061421					13	23	2478.7	5	424.1	7
Weathersfield - Windsor STP FPAV(13)	Type IVS w/ 20%	gporter188D075139	0.26	99.84	975	50	8.3	8	2159.7	5	445.4	4

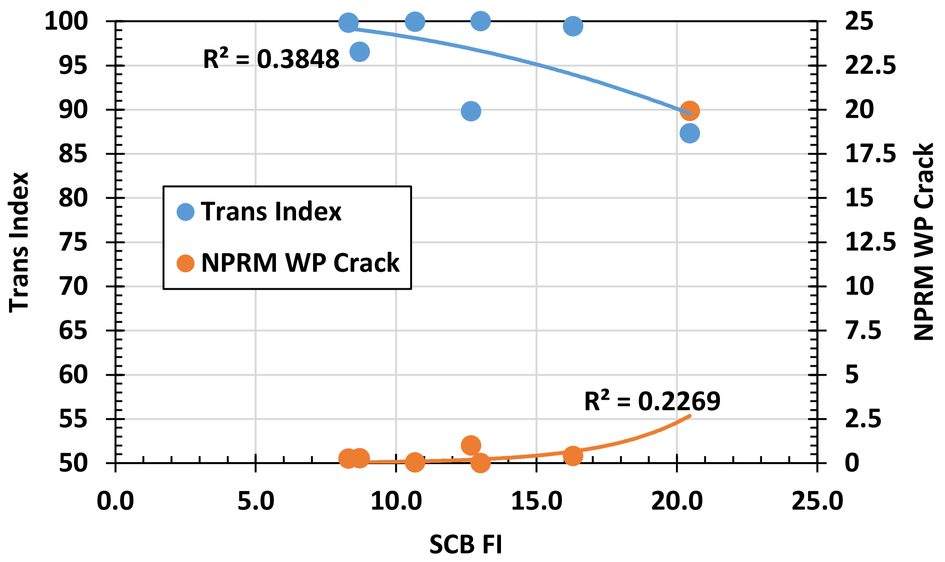


FIGURE 36 SCB Flexibility Index Compare to VTrans Fatigue Cracking Indices; Trans Index and NPRM WP Crack

Additional analysis was conducted to evaluate the impact of traffic level on the cracking indices. In general, field aging and traffic are the two major factors regarding the deterioration of asphalt pavements due to cracking, assuming asphalt production and placement were conducted properly. However, it should be reiterated that all of pavement sections were only two years old at the time of the analysis. Figures 37 and 38 show the comparisons of the VTrans fatigue cracking indices with the AADT and SCB Flexibility Index of the respective pavement sections. The figures clearly indicate that as the AADT increases, the VTrans cracking indices' distress magnitudes also increase. However, with limited data, it would not be prudent to include AADT within the SCB Flexibility Index criteria yet, although it can be assumed that as traffic level increases, greater fatigue cracking performance would be required.

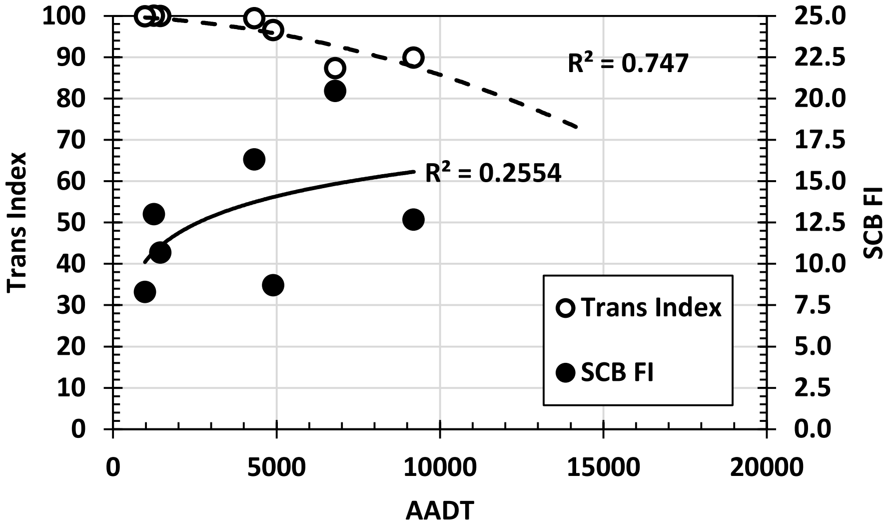


FIGURE 37 Pavement Section AADT Compared to VTrans Trans Index Cracking Index and SCB Flexibility Index

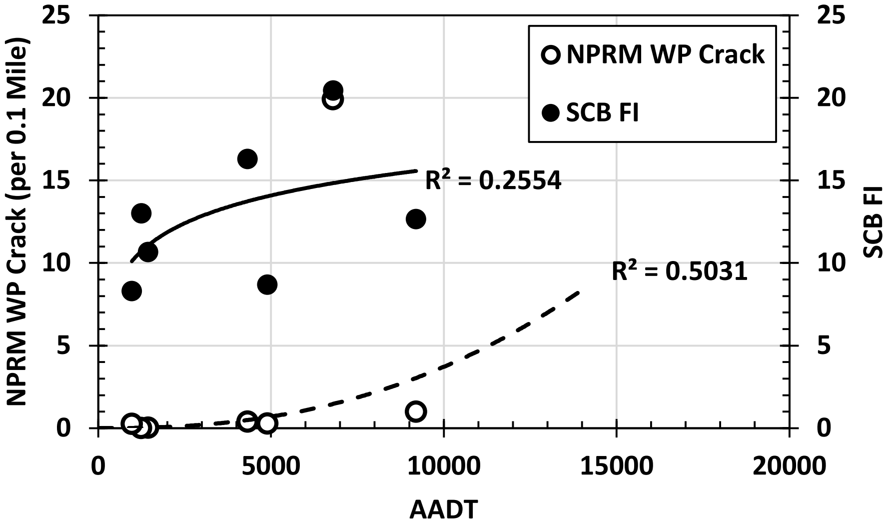


FIGURE 38 Pavement Section AADT Compared to VTrans NPRM WP Crack Cracking Index and SCB Flexibility Index

Another factor greatly impacting the fatigue cracking resistance in asphalt mixtures is the asphalt content, which can be related to gyration level. The greater the gyration level, the closer the aggregate particles are pushed together. As the aggregates push together, the asphalt binder around the aggregate skeleton is squeezed out. This is why one generally sees the asphalt content decrease as the gyration level increases. Although lower asphalt contents may be good to help resist rutting, lower asphalt contents will in turn accelerate cracking. Volumetrics for the asphalt mixtures were not provided, but Figures 39 and 40 were developed by comparing the pavement sections AADT to the respective mixture design gyration level and VTrans cracking indices. The figures do show evidence that as design gyration level increases, so does the magnitude in the field fatigue cracking measurements.

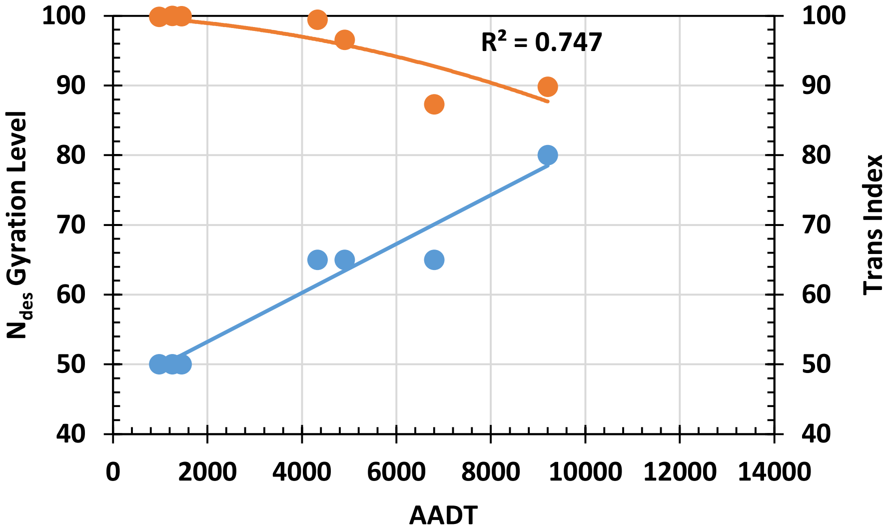


FIGURE 39 Design Gyration Level Compared to VTrans Trans Index

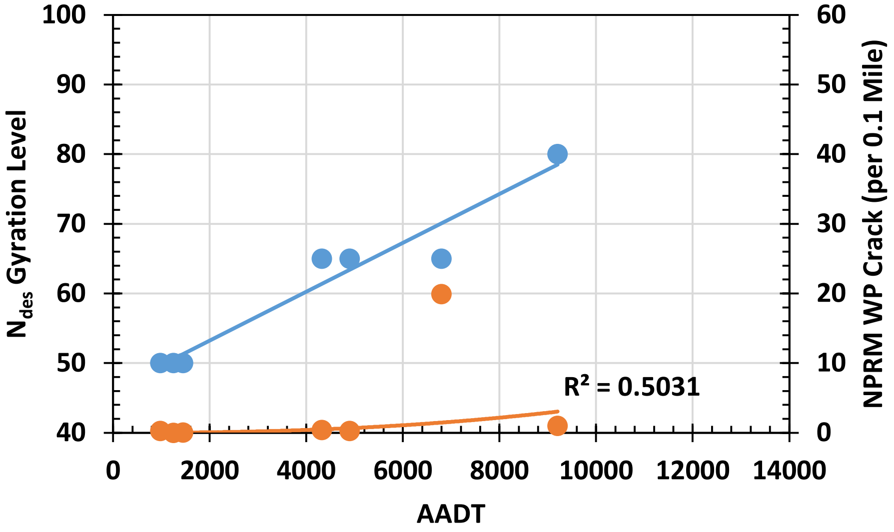


FIGURE 40 Design Gyration Level Compared to VTrans NPRM WP Crack

The determination of a tentative criteria is difficult at this time as most of the pavement sections showed relatively good performance, except for the Bennington, NH 2966(1) project. At this time, it is not known whether this was an error with the data collection or not. This pavement section had the second highest AADT (6800) and the highest average SCB Flexibility Index value (20.5). **Based on the current data, it would appear that a preliminary SCB Flexibility Index Criteria would be a minimum of 8.0 at a test temperature of 25°C**. Figures 41 and 42 show where this criteria falls and how the pavement sections performed, respectively.

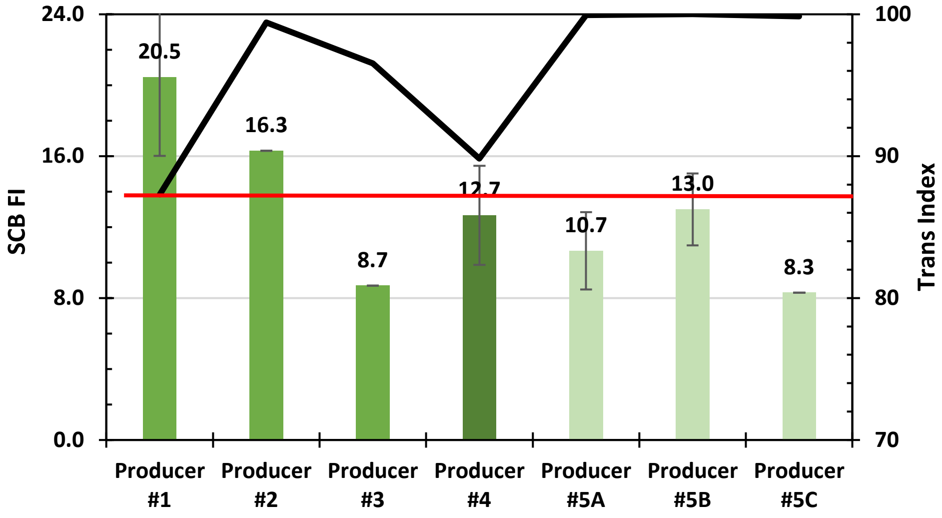


FIGURE 41 SCB Flexibility Index and Trans Index with Preliminary Criteria for 2018 Pavement Sections

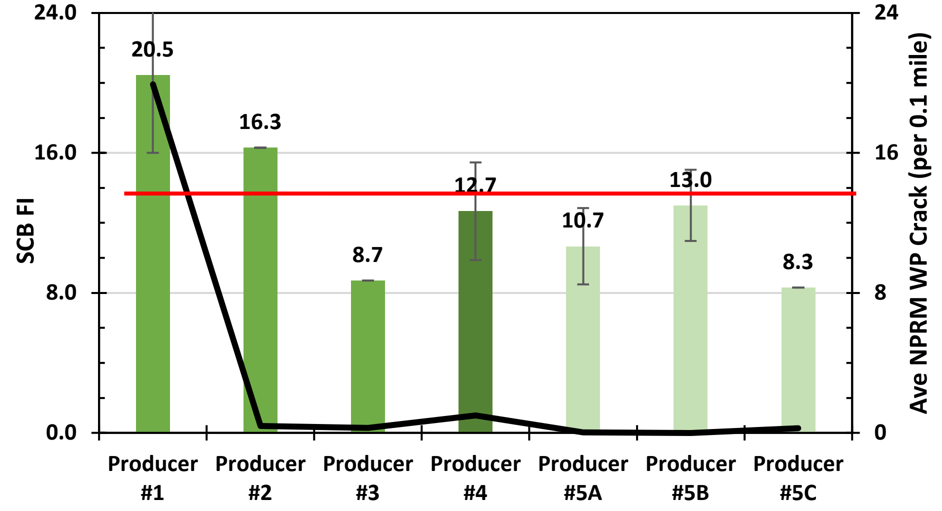


FIGURE 42 SCB Flexibility Index and NPRM WP Crack with Preliminary Criteria for 2018 Pavement Section

Fatigue Cracking – VTrans Final Recommendations

Similar to the climate conditions of Maine, Vermont consists of two different low temperature PG grades based on LTPPBind 3.1 at a 98% reliability, -28°C and -34°C. Using the relationship recommended in NCHRP project 9-59, intermediate testing temperatures would be 22°C and 19°C, respectively. Future research in Vermont may want to be directed to looking at reducing the SCB test temperature to an intermediate temperature more representative of the region.

Additionally, with the VTrans asphalt mixtures all showing relatively good SCB Flexibility performance, VTrans may want to evaluate the performance of field cores with poor field cracking performance. This method would provide a direct comparison between the SCB performance and poor field cracking performance to help validate a minimum SCB Flexibility Index value. Tables 15 shows pavement sections identified in the VTrans PMS that can be classified as poor cracking performance. Special care should be taken to determine the air voids of the field cores and include this information in the analysis as air void content has been found to directly influence the SCB Flexibility Index performance.

TABLE 15 VTrans PMS Identification of Poor Cracking Performance

Poor Cracking Measurements (2018)									
Route Name	ETE_From	ETE_To	ETE_Road	BeginTown	NPRM Condition CRK	NPRM WP Crack	Last Work Project Name	Last Work Project Number	Last Work Year
US 2	101.2	101.3	U002	CABOT	POOR	34.15	District Paving	NE19PAV702	2018
	101.3	101.4	U002	DANVILLE	POOR	26	District Paving	NE19PAV702	2018
	101.6	101.7	U002	DANVILLE	POOR	33.25	District Paving	NE19PAV702	2018
	101.7	101.8	U002	DANVILLE	POOR	35.75	District Paving	NE19PAV702	2018
	101.8	101.9	U002	DANVILLE	POOR	39.75	District Paving	NE19PAV702	2018
US 5	186.8	186.9	U005	DERBY	POOR	30.38	District Paving	NE19PAV902	2018
	187.5	187.6	U005	DERBY	POOR	30.67	District Paving	NE19PAV902	2018
US 7	10.3	10.4	U007	BENNINGTON	POOR	21	Bennington	NH 2966(1)	2018
	10.4	10.5	U007	BENNINGTON	POOR	27.5	Bennington	NH 2966(1)	2018
	10.5	10.6	U007	BENNINGTON	POOR	21	Bennington	NH 2966(1)	2018
	11.4	11.5	U007	BENNINGTON	POOR	56.75	Bennington	NH 2966(1)	2018
	11.5	11.6	U007	BENNINGTON	POOR	47	Bennington	NH 2966(1)	2018
	11.6	11.7	U007	BENNINGTON	POOR	41	Bennington	NH 2966(1)	2018
	145.9	146	U007	MILTON	POOR	32	District Paving	NE19PAV501	2018
VT 9	3.1	3.2	V009	BENNINGTON	POOR	24.75	Bennington	NH 2966(1)	2018
	3.5	3.6	V009	BENNINGTON	POOR	22.75	Bennington	NH 2966(1)	2018
	3.6	3.7	V009	BENNINGTON	POOR	31.75	Bennington	NH 2966(1)	2018
	3.7	3.8	V009	BENNINGTON	POOR	36	Bennington	NH 2966(1)	2018
	3.8	3.9	V009	BENNINGTON	POOR	23	Bennington	NH 2966(1)	2018
	4	4.1	V009	BENNINGTON	POOR	26.5	Bennington	NH 2966(1)	2018
	4.7	4.8	V009	BENNINGTON	POOR	20.75	Bennington	NH 2966(1)	2018
	5	5.1	V009	BENNINGTON	POOR	35.5	Bennington	NH 2966(1)	2018
	5.5	5.6	V009	BENNINGTON	POOR	42.75	Bennington	NH 2966(1)	2018
VT 14	93.3	93.4	V014	ALBANY	POOR	57.25	District Paving	NE19PAV901	2018
	93.4	93.5	V014	ALBANY	POOR	59	District Paving	NE19PAV901	2018
VT 67A	2.2	2.3	V067A	BENNINGTON	POOR	36	Bennington	STP 2973(1)	2018
	2.6	2.7	V067A	BENNINGTON	POOR	29.25	Bennington	STP 2973(1)	2018
	2.7	2.8	V067A	BENNINGTON	POOR	31.75	Bennington	STP 2973(1)	2018
	2.8	2.9	V067A	BENNINGTON	POOR	22.25	Bennington	STP 2973(1)	2018
	3	3.1	V067A	BENNINGTON	POOR	22.75	Bennington	STP 2973(1)	2018
	3.2	3.3	V067A	BENNINGTON	POOR	25	Bennington	STP 2973(1)	2018
VT 131	1.1	1.2	V131	CAVENDISH	POOR	41.5	District Paving	NE19PAV201	2018
	1.3	1.4	V131	CAVENDISH	POOR	51	District Paving	NE19PAV201	2018
	1.7	1.8	V131	CAVENDISH	POOR	42.25	District Paving	NE19PAV201	2018
	1.8	1.9	V131	CAVENDISH	POOR	34.25	District Paving	NE19PAV201	2018
	1.9	2	V131	CAVENDISH	POOR	53.25	District Paving	NE19PAV201	2018
	2.1	2.2	V131	CAVENDISH	POOR	42.75	District Paving	NE19PAV201	2018

General Mixture Performance – Rutting

As mentioned earlier, VTrans uses the Hamburg Wheel Tracking test method at a temperature of 45°C to evaluate the rutting performance of asphalt mixtures. Figures 43 and 44 show the Hamburg Rutting results for asphalt mixtures tested in 2017 and 2018, respectively. The test results show relatively good performance with generally better performance occurring as the design gyration level increases.

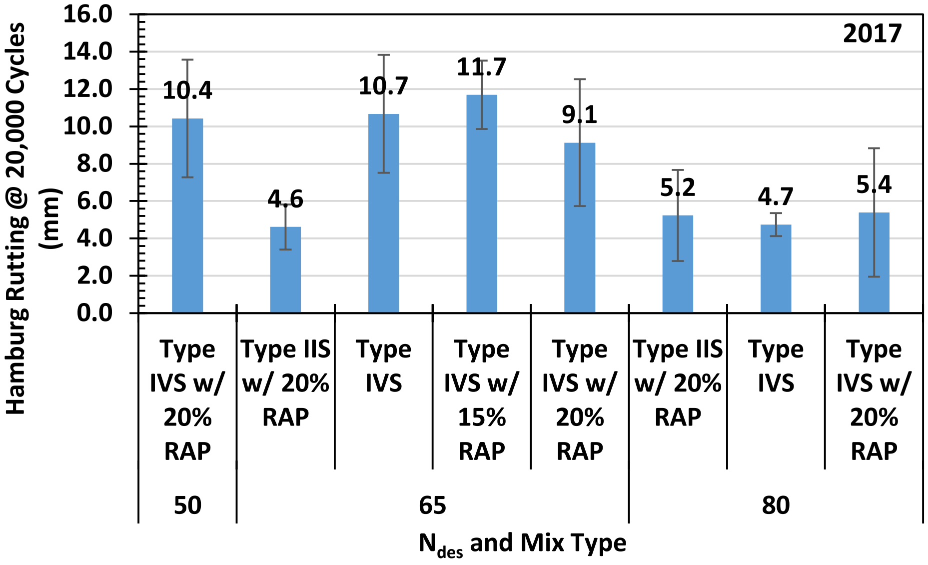


FIGURE 43 Hamburg Wheel Tracking Rutting for Different Mixture Types Tested in 2017

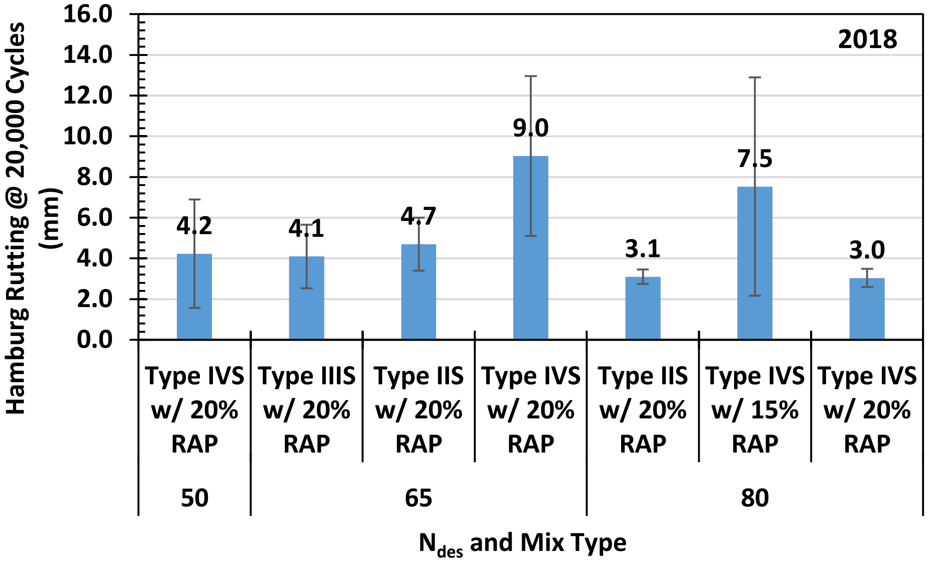


FIGURE 44 Hamburg Wheel Tracking Rutting for Different Mixture Types Tested in 2018

A significant number of pavement sections with corresponding Hamburg Wheel Tracking test results were available for VTrans. Unfortunately, all of the pavement sections with Hamburg data all showed very low field rutting results. Table 16 and 17 shows the results from 2017 and 2018 laboratory testing, respectively. The greatest amount of field rutting observed was 0.19 inches.

TABLE 16 2017 VTrans Pavement Sections Containing Laboratory Wheel Tracking Test Results

Hamburg Rutting vs Measured Field Performance (2017 Constructed)

Project	Mix Type	Site Manager Sample ID	Rut Average (in.)	Rut Index	Ave. AADT	Ndes	PG Grade	Hamburg Rut Depth (mm)	Ave Pass Max.	SIP	SIP Depth	Strip Slope	Creep Slope
Barre City NH 2961(2)	Type IVS w/ 20% RAP	jgehrig176F185100	0.14	86	9767	80	70-28	3.01	20000	NA	NA	NA	0.000075
		jgehrig176Q193941						3.44	20000	NA	NA	NA	0.000085
		jgehrig179H225230						8.93	20000	11052	2.48	0.001142	0.000118
		dsavage178F090948						1.99	20000	NA	NA	NA	
Charlotte FEGC F 019-4(20)	Type IIS w/ 20% RAP	jglover176F094745	0.16	83.80	12000	80	58-28	6.62	20000	NA	NA	NA	0.000164
		jglover176D063539						10.28	18418	14059	6.58	0.000907	0.000313
		lrothlon1776173517						5.97	20000	NA	NA	NA	0.000191
		tcoletta1798151221						3.00	20000	NA	NA	NA	
		etaveres17A2071242						12.5	16362	8774	4.65	0.000801	0.000364
Danville - St. Johnsbury STP FPAV(9)	Type IVS w/ 20% RAP	gporter179B070108	0.06	94.00	563	50	58-34	9.75	20000	13166	4.95	0.000499	0.000251
		tcoletta17A6093250						12.5	17266	8885	5.26	0.000818	0.000367
Essex Junction NH 2956(2)	Type IVS w/ 20%	lrothlon178D181538	0.141	85.9	11788	80	70-28	3.76	20000	NA	NA	NA	0.0001
Hardwick - Danville STP 2122(1)	Type IVS w/ 20% RAP	bwaterma17AD065250	0.088	91.2	3038	65	58-34	12.5	16504	12704	7.62	0.000823	0.000447
		jbreton178H075133						11.33	20000	14154	6.07	0.000602	0.000257
		jbreton178J110802						9.41	20000	NA	NA	NA	0.000213
Hancock STP 2923(1)	Type IVS w/ 20%	dconnell176F0994849	0.05	94.80	1500	65	58-28	4.21	20000	NA	NA	NA	0.000116
		gporter1761051744						5.04	20000	NA	NA	NA	0.000122
Hartland STP FPAV(8)	Type IVS w/ 20%	rknapp178P102700	0.08	92.10	1689	50	58-34	12.5	14176	8930	6.86	0.00103	0.00052
		etavares179T150144						11.34	13000	7243	4.48	0.000995	0.00092
Randolph - Braintree STP FPAV(7)	Type IVS w/ 20% RAP	dsavage176S63834	0.06	94.10	1379	50	58-28	12.5	11008	5625	3.31	0.00105	0.000377
		gporter177Q073537						12.5	14490	8903	5.13	0.001149	0.000362
		jgehrig177V132604						12.50	6068	NA	NA	NA	0.001201
Rochester ER STP 0162(21)	Type IIS w/ 20% RAP	gporter178N081704	0.093	90.7	858	50	58-28	4.33	20000	NA	NA	NA	0.000105
		tcoletta178Q150041						6.19	20000	NA	NA	NA	0.00021
		tcoletta178T161907						3.27	20000	NA	NA	NA	0.000086
		jglover176H112317						9.16	20000	NA	NA	NA	0.000268
		scrowley176Q105822						10.31	18550	11330	4.32	0.000444	0.000218
Rockingham - Springfield STP 2962(1)	Type IVS w/ 15% RAP	gporter176K153823	0.072	92.8	2453	65	58-28	12.5	16860	11113	4.43	0.001183	0.000237
		gporter176N101328						12.5	13820	8280	4.87	0.001327	0.000344
		jgehrig177Q114348						12.5	14380	11556	7.69	0.001241	0.000475
		jgehrig177V075536						12.5	14436	8081	4.43	0.000753	0.00031
		jglover175U062239						12.17	17896	12494	4.94	0.001276	0.000224
		jglover1778094343						7.54	20000	15335	4.19	0.000392	0.000147
		scrowley176E080529						12.5	14920	7090	4.46	0.0009	0.000441
		scrowley1777121000						8.02	18056	9623	3.44	0.000666	0.000155
		scrowley177C111000						12.5	17784	NA	NA	NA	0.000266
		scrowley1781105150						12.5	12770	7239	5.41	0.00096	0.000432
		scrowley1783070716						12.5	13688	6181	6.68	0.000829	0.000423
		scrowley1788084247						12.5	13224	8095	5.89	0.000915	0.00047
Roxbury - Northfield ER STP 0187(13)	Type IVS w/ 20% RAP	bwaterma17AV105641	0.061	93.8	1100	50	58-28	12.5	10522	6218	5.49	0.0014	0.000547
		tcoletta175G064353						5.5	20000	NA	NA	NA	0.000155
		tcoletta175G064353						10.52	20000	12963	4.82	0.000501	0.00021
		tcoletta179D061046						2.85	20000	NA	NA	NA	
		tcoletta179B063938						9.03	20000	11326	4.44	0.000482	0.000252
Roxbury - Northfield STP	Type IVS w/ 20%	gporter177602156	0.088	91.2	1121	50	58-28	12.5	12396	7764	7.27	0.001029	0.00059
		jgehrig1769051309						12.5	17916	10083	4.23	0.000841	0.000237
Rutland - Killington ER NH 020-2(36)	Type IIIS w/ 15% RAP	gporter1754062752	0.129	87.1	8291	80	70-28	4.76	30000	21270	3.65	0.000113	0.000092
		gporter176815923						5.34	20000	NA	NA	NA	0.000128
		gporter177H071513						3.76	20000	NA	NA	NA	0.00008
		jgehrig175G112708						3.96	20000	NA	NA	NA	0.00012
		jgehrig175O061423						3.44	20000	NA	NA	NA	0.000095
		jglover176L064548						4.39	20000	NA	NA	NA	0.000129
		lrothlon176D125700						4.49	20000	NA	NA	NA	0.000121
		lrothlon1761142144						5.96	20000	NA	NA	NA	0.000158
		scrowley177712111						4.41	20000	NA	NA	NA	0.000119
		tcoletta176Q161029						4.92	20000	NA	NA	NA	0.000176
South Burlington - Williston NH 2944(1)	Type IIS w/ 20% RAP	jglover17672114025	0.19	80.9	14816	80	70-28	3.44	20000	NA	NA	NA	0.000061
		lrothlon1789192238						3.26	20000	NA	NA	NA	0.00007
		dsavage179I041242						3.58	20000	NA	NA	NA	0.000125
		scrowley1780185257						4.3	20000	NA	NA	NA	0.00013
		jbreton17A6180835						4.26	20000	13661	2.75	0.000104	0.000087
St. Albans Town STP	Type IVS w/ 20%	gporter175G075800	0.096	90.4	2152	50	58-28	12.5	14762	NA	NA	NA	0.000493
		gporter175K061647						5.39	20000	NA	NA	NA	0.000161

TABLE 17 2018 VTrans Pavement Sections Containing Laboratory Wheel Tracking Test Results

Hamburg Rutting vs Measured Field Performance (2018)													
Project	Mix Type	Site Manager Sample ID	Rut Average	Rut Index	Ave. AADT	Ndes	PG Grade	Hamburg Rut Depth (mm)	Ave Pass Max.	SIP	SIP Depth	Strip Slope	Creep Slope
Bennington NH 2966(1)	Type IVS w/ 20% RAP	tarel188D061826	0.114	88.5	6803	65	70-28	10.32	20000	12660	5.11	0.000501	0.000257
		scrowley1896145508						6.72	20000	13509	3.78	0.000349	0.000198
		dconnoll189R084245						5.88	20000	15507	4.42	0.000307	0.00017
		tarel18A2093504						3.88	20000	NA	NA	NA	0.000126
Bennington - Wilmington NH SURF(51)	Type IVS w/ 20% RAP	tcoletta1865055842	0.11	89.30	4325	65	58-28	12.5	12744	6992	4.4	0.000824	0.000391
Montpelier STP 2950(1)	Type IVS w/ 20% RAP	jjacobso186Q075145	0.155	84.51	4888	65	70-28	10.49	19566	13334	2.99	0.001326	0.000150
St. Albans City STP 2957(1)	Type IVS w/ 20% RAP	scrowley185D180532	0.12	88.35	9200	80	70-28	3.16	20000	NA	NA	NA	0.000086
		tarel186J165054						3.41	20000	NA	NA	NA	0.000099
		jbreton187G173419						2.55	20000	NA	NA	NA	0.000064
Reading - Windsor STP FPAV(11)	Type IVS w/ 20% RAP	gporter189D105848	0.01	99.03	1427	50	70-28	3.49	20000	NA	NA	NA	0.000096
		tarel189J145920						2.6	20000	NA	NA	NA	0.000072
		tarel189O094332						3.92	20000	NA	NA	NA	0.000106
Weathersfield - Reading STP FPAV(12)	Type IVS w/ 20% RAP	jjacobso187R061056	0.02	98.40	1256	50	70-28	4.17	20000	NA	NA	NA	0.000111
		ldonavan187R080905						3.13	20000	NA	NA	NA	0.000086
		gporter1888073953						3.58	20000	NA	NA	NA	0.000093
		tarel189Q133152						2.57	20000	NA	NA	NA	0.000072
		jbreton18A4054111						4.41	20000	16845	3.94	0.000141	0.000119
		etavares18AC055915						3.5	20000	NA	NA	NA	0.000088
		etavares18AI145911						3.61	20000	NA	NA	NA	0.000095
Weathersfield - Windsor STP FPAV(13)	Type IVS w/ 20% RAP	gporter188D075139	0.015	98.51	954	50	70-28	3.28	20000	NA	NA	NA	0.000092
Cabot - Danville FEGC F 028-3(26)C/2	Type IIS w/ 20% RAP	gporter1896095514	0.09	91	4100	65	58-28	4.56	20000	NA	NA	NA	0.000153
		gporter1896095514						5.44	20000	NA	NA	NA	0.000177
		tarel186D084429						7.29	20000	11103	4	0	0.000178
Enosburgh - Richford STP 2969(1)	Type IVS w/ 20% RAP	jjacobso187U070012	0.053	94.9	4656	65	70-28	3.80	20000	NA	NA	NA	0.000107
		gporter189I102550						3.14	20000	14213	2	0	0.000088
	Type IIIS w/ 20% RAP	tcoletta18A1080242						4.69	20000	18176	4	0	0.000152
		tcoletta18AC111912						5.28	20000	15941	4	0	0.000147
Essex NH 2931(2)	Type IIS w/ 20% RAP	tcoletta189K074609	0.023	97.7	7605	65	70-28	3.63	20000	NA	NA	NA	0.000099
		tcoletta189O094819						3.89	20000	NA	NA	NA	0.000114
		tcoletta189O094819						3.71	20000	NA	NA	NA	0.000097
		dconnoll18AA072024						4.38	20000	NA	NA	NA	0.000112
	e IVS w/ 20%	tarel18B9081105						2.73	20000	NA	NA	NA	0.000072
Waterbury - Stowe STP 2945(1)	Type IIS w/ 20% RAP	scrowley1859182253	0.019	98.1	12226	80	70-28	3.20	20000	NA	NA	NA	0.000089
		scrowley185H193057						3.22	20000	NA	NA	NA	0.000087
		scrowley185O184722						3.41	20000	NA	NA	NA	0.000089
		scrowley186F033031						3.44	20000	NA	NA	NA	0.000084
		jbreton1866175418						3.63	20000	NA	NA	NA	0.000076
		jbreton1871174622						3.57	20000	NA	NA	NA	0.000094
		jbreton1879173230						3.13	20000	NA	NA	NA	0.000078
		jbreton187C175522						2.80	20000	NA	NA	NA	0.000067
		jbreton187Q173832						2.93	20000	NA	NA	NA	0.000060
		jbreton1885171752						3.21	20000	NA	NA	NA	0.000022
		jbreton188R170621						3.12	20000	NA	NA	NA	0.000080
		jbreton18888163354						3.06	20000	NA	NA	NA	0.000065
		jbreton188M174557						2.51	20000	NA	NA	NA	0.000056
		jbreton1899174832						3.09	20000	NA	NA	NA	0.000070
		jbreton189I173143						3.93	20000	NA	NA	NA	0.000094
		jbreton189C175153						2.89	20000	NA	NA	NA	0.000066
		jbreton189K174401						2.76	20000	NA	NA	NA	0.000069
		jbreton189T001654						2.82	20000	NA	NA	NA	0.000085
		jbreton18A9174700						2.48	20000	NA	NA	NA	0.000064
		jbreton18AM171512						2.86	20000	NA	NA	NA	0.000057
		jbreton18AM171512						3.11	20000	NA	NA	NA	0.000063

A first attempt at comparing the Hamburg Wheel Tracking rutting and the corresponding pavement section field rutting is shown in Figure 45. The results show a poor correlation when attempting to simply compare rutting values without applying any filtering. Incorporating the asphalt mixtures' design gyration level, Figure 46 was produced. Figure 46 shows that typically lower Hamburg rutting occurred at higher design gyration levels (80) while resulting in some of the higher magnitudes of field rutting. The opposite occurred for the lower design gyration level (50) where more Hamburg rutting occurred with lower magnitudes of field rutting. Since the Hamburg Wheel Tracking test's loading and test parameters are not modified for the expected traffic levels, asphalt mixture designed for lower traffic levels (i.e. – lower design gyration levels, neat asphalt binders, lower angularity values, etc) will show greater amounts of rutting in the laboratory when compared to the higher design gyration level asphalt mixtures. However, since those same low design gyration level mixtures are exposed to very little traffic in the field, the field rutting is low.

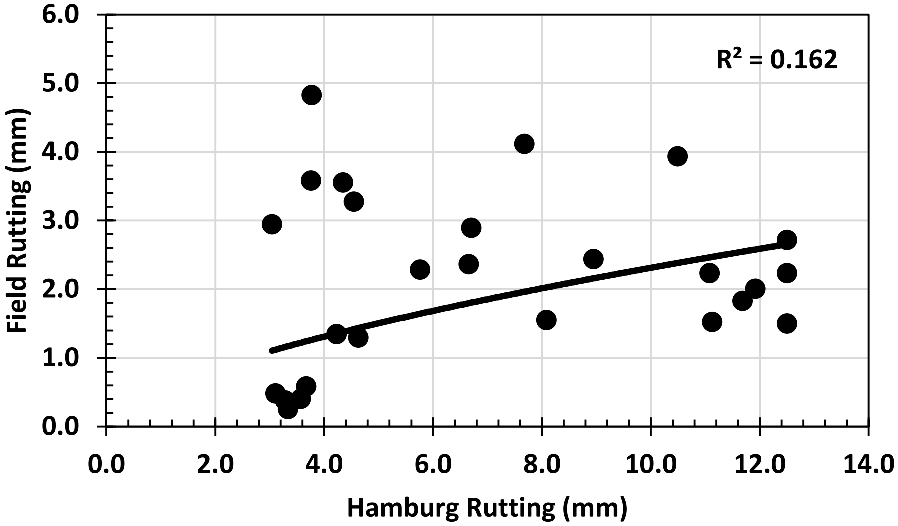


FIGURE 45 Comparison of VTrans' Hamburg Wheel Tracking Rutting and Field Rutting

The same type of analysis was also conducted for the different asphalt binder grades (Figure 47). In Figure 47, it appears that the high temperature grade clearly influenced the laboratory test results as the PG70 asphalt binders showed much lower Hamburg Wheel Tracking rutting when compared to the PG58 asphalt binders.

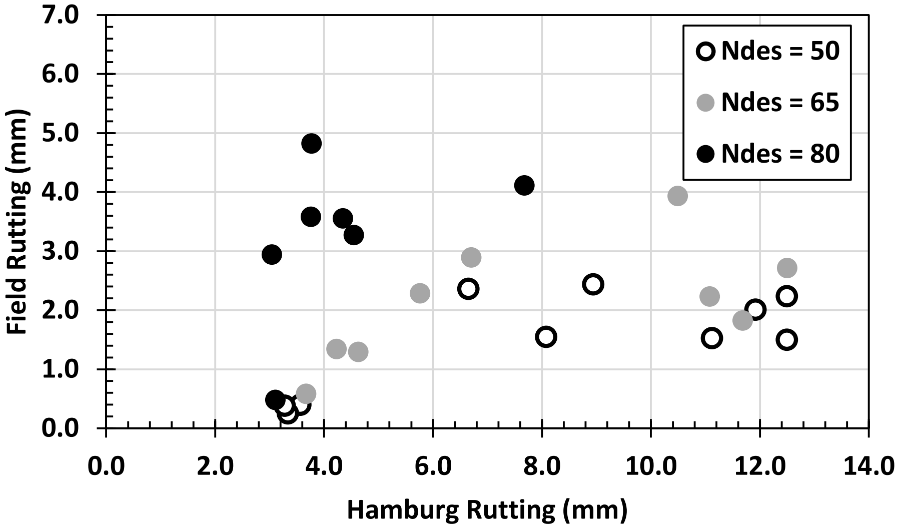


FIGURE 46 Comparison of VTrans' Hamburg Wheel Tracking Rutting and Field Rutting While Identifying Design Gyration Level

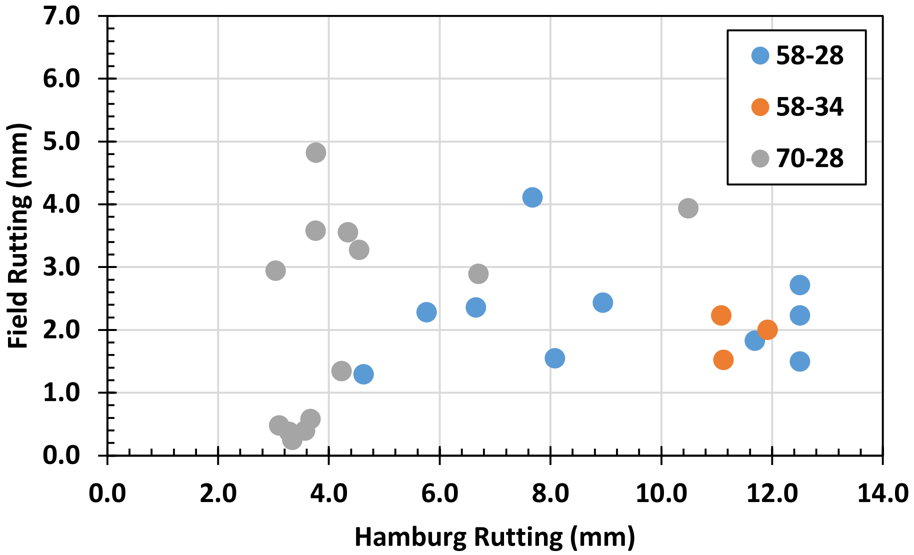


FIGURE 47 Comparison of VTrans' Hamburg Wheel Tracking Rutting and Field Rutting While Identifying Asphalt Binder Grade

The field rutting was compared to the respective traffic level of the pavement section. As shown in Figure 48, the field rutting was found to be directly related to the AADT of the pavement section. Therefore, any Hamburg rutting criteria should take the AADT into consideration since the testing parameters of the test method will remain constant.

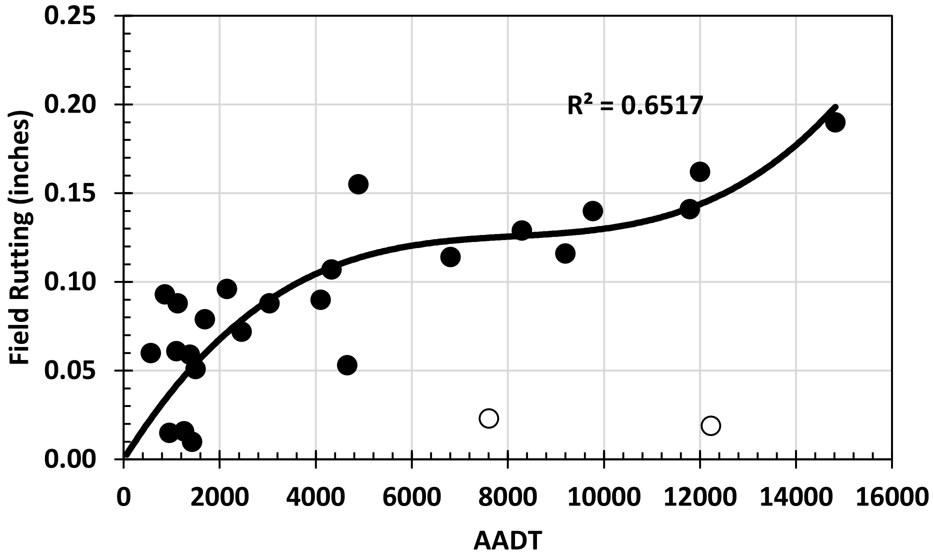


FIGURE 48 VTrans AADT and Corresponding Field Rutting (Statistical Outlier = White Circles)

With traffic shown to be a critical factor, the field rutting rate was calculated by dividing the magnitude of field rutting by the AADT, which results in rutting rate parameter of mm/AADT. The resultant relationship between rutting rate and Hamburg rutting is shown in Figure 49. Figures 50 and 51 show the same information, but filtered by design gyration level and PG grade, respectively. By normalizing the field rutting with the applied traffic, a much better comparison between laboratory mixture performance and field performance was found. In addition, Figure 50 shows that the higher the gyration level, the lower the rutting rate. Both the 65 and 80 design gyration mixes showed much lower rutting rates than the 50 design gyration asphalt mixtures. Figure 51 showed that PG grade had a much greater influence on the laboratory Hamburg rutting performance than the actual field rutting rate.

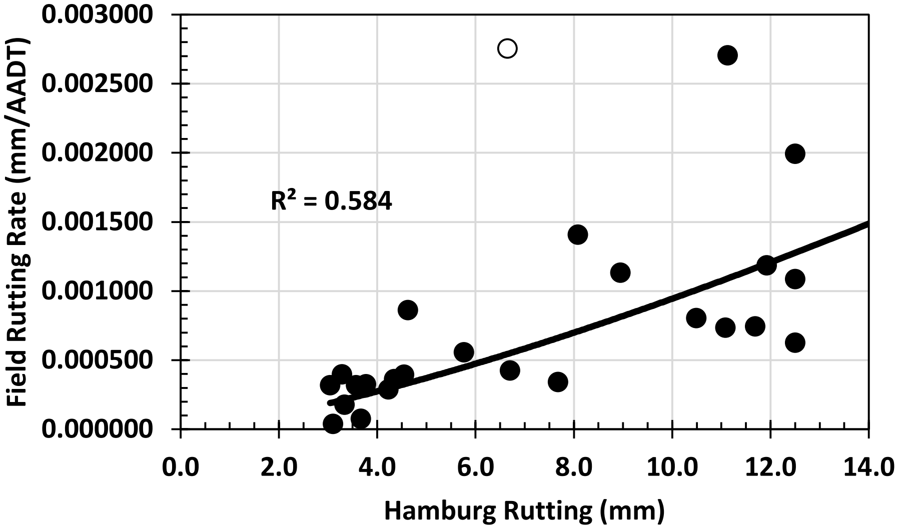


FIGURE 49 Field Rutting Rate vs Hamburg Wheel Tracking Rutting

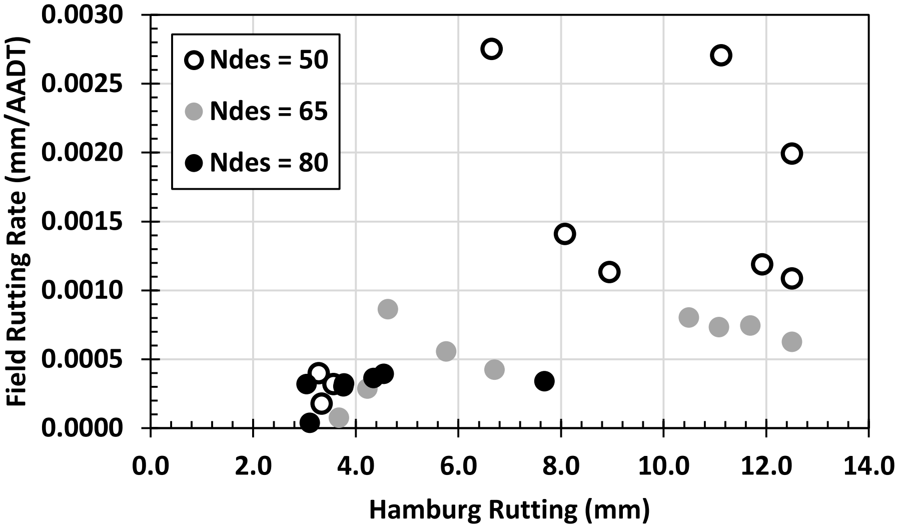


FIGURE 50 Field Rutting Rate vs Hamburg Wheel Tracking Rutting with Design Gyration Level

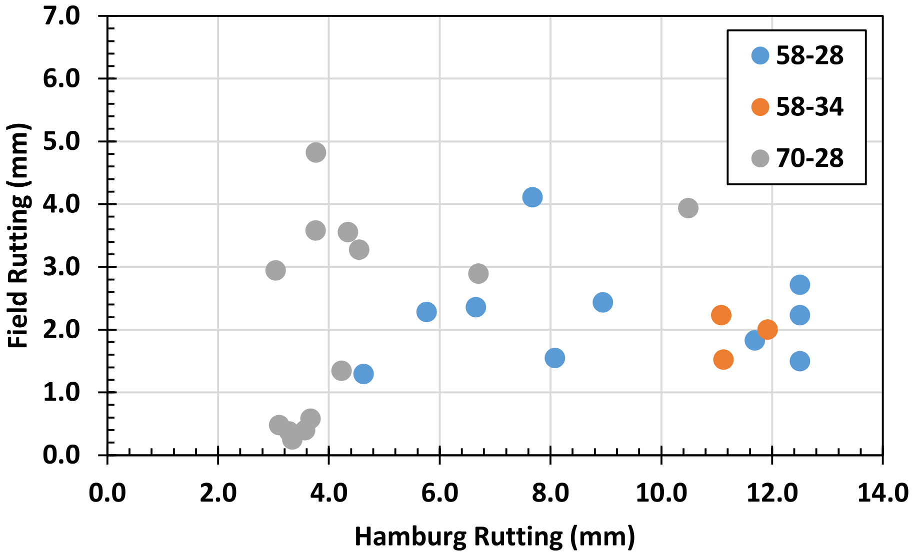


FIGURE 51 Field Rutting Rate vs Hamburg Wheel Tracking Rutting with Asphalt Binder Grade

Rutting – VTrans Final Recommendations

The final preliminary Hamburg Wheel Tracking rutting criteria is shown as Figure 52. The criteria are based on the design gyration level of the proposed asphalt mixture as this was found to be sensitive to the rutting rate of the field sections. The criteria indicate that for both the 50 and 65 design gyration asphalt mixtures, a maximum Hamburg rutting at 20,000 passes should be 12.5 mm. Both of these asphalt mixtures showed good field performance, even when some of the respective asphalt mixtures achieved close to an average of 12.5 mm rutting in the Hamburg. Meanwhile, the 80 design gyration level asphalt mixtures must meet a maximum of 6.0 mm of Hamburg rutting after 20,000 cycle. This value was found to be on the higher end of the average Hamburg rutting while still showing good field performance. Figure 52 shows a transition at an AADT of 7,000 in the proposed criteria as this mirrored where the 80 design gyration level asphalt mixtures appeared to begin being placed at (Figure 53).

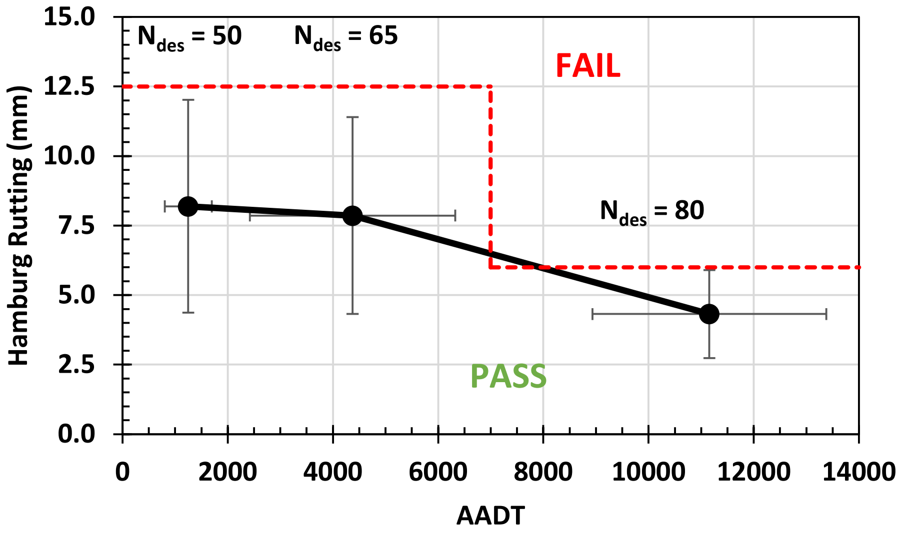


FIGURE 52 Proposed Tentative Hamburg Wheel Tracking Rutting Criteria for VTrans' Asphalt Mixtures

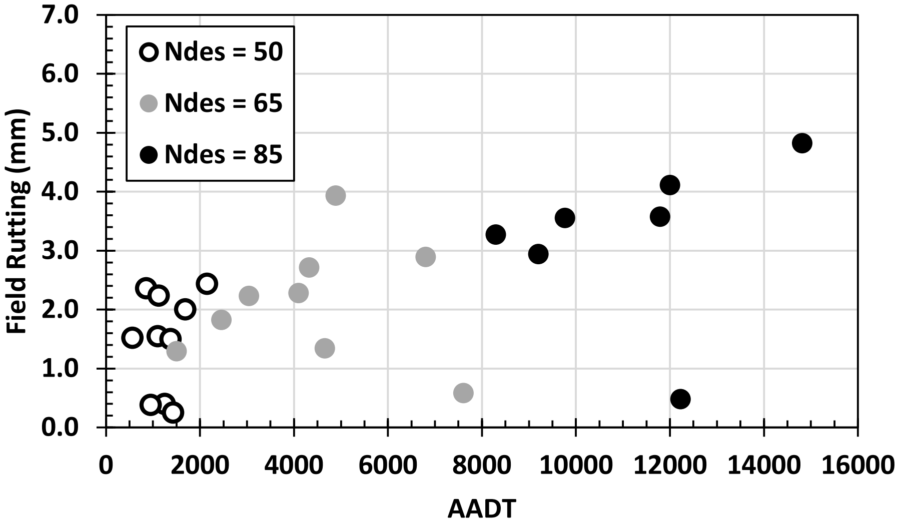


FIGURE 53 Field Rutting vs AADT for Different VTrans Design Gyration Level Asphalt Mixtures

In addition to the preliminary criteria, Table 18 contains pavement sections in Vermont that are currently showing POOR rutting performance. To help further validate the proposed criteria, it is recommended that VTrans take field cores from the poor performing rutting sections and test in the Hamburg Wheel Tracking test.

TABLE 18 Poor Rutting Performance Pavement Sections in Vermont

Poor Rutting Measurements										
Route Name	ETE_From	ETE_To	ETE_Road	Begin Town	AADT	Rut Average	Rut Index	Last Work Project Name	Last Work Project Number	Last Work Year
VT 9	3.1	3.2	V009	BENNINGTON	4520	0.34	66.5	Bennington	NH 2966(1)	2018
	3.3	3.4	V009	BENNINGTON	5113	0.29	71.4	Bennington	NH 2966(1)	2018
	3.5	3.6	V009	BENNINGTON	4900	0.30	70.5	Bennington	NH 2966(1)	2018
	3.7	3.8	V009	BENNINGTON	4900	0.54	45.8	Bennington	NH 2966(1)	2018
	3.8	3.9	V009	BENNINGTON	4900	0.35	64.5	Bennington	NH 2966(1)	2018
	3.9	4	V009	BENNINGTON	4900	0.30	70.5	Bennington	NH 2966(1)	2018
	4	4.1	V009	BENNINGTON	4990	0.38	61.6	Bennington	NH 2966(1)	2018
	4.1	4.2	V009	BENNINGTON	5200	0.29	71.4	Bennington	NH 2966(1)	2018
	4.2	4.3	V009	BENNINGTON	5200	0.32	68.5	Bennington	NH 2966(1)	2018
	4.3	4.4	V009	BENNINGTON	5302	0.29	71.4	Bennington	NH 2966(1)	2018
	4.4	4.5	V009	BENNINGTON	8600	0.30	70.5	Bennington	NH 2966(1)	2018
	4.5	4.6	V009	BENNINGTON	8600	0.45	54.7	Bennington	NH 2966(1)	2018
	4.6	4.7	V009	BENNINGTON	8600	0.30	70.5	Bennington	NH 2966(1)	2018
	4.7	4.8	V009	BENNINGTON	8600	0.36	63.6	Bennington	NH 2966(1)	2018
	4.8	4.9	V009	BENNINGTON	8600	0.38	61.6	Bennington	NH 2966(1)	2018
	4.9	5	V009	BENNINGTON	7973	0.39	60.6	Bennington	NH 2966(1)	2018
	5	5.1	V009	BENNINGTON	6700	0.35	64.5	Bennington	NH 2966(1)	2018
	5.2	5.3	V009	BENNINGTON	6126	0.31	69.5	Bennington	NH 2966(1)	2018
	5.3	5.4	V009	BENNINGTON	6000	0.38	61.6	Bennington	NH 2966(1)	2018
	5.4	5.5	V009	BENNINGTON	6000	0.30	70.5	Bennington	NH 2966(1)	2018
VT 12	58.4	58.5	V012	MONTPELIER CIT	3301	0.77	23.2	Montpelier	STP 2950(1)	2018
	58.5	58.6	V012	MONTPELIER CIT	3300	0.67	33.0	Montpelier	STP 2950(1)	2018
	58.6	58.7	V012	MONTPELIER CIT	3300	0.40	59.6	Montpelier	STP 2950(1)	2018
	58.8	58.9	V012	MONTPELIER CIT	3300	0.75	25.1	Montpelier	STP 2950(1)	2018
	58.7	58.8	V012	MONTPELIER CIT	3300	0.63	37.0	Montpelier	STP 2950(1)	2018
VT 14	93.3	93.4	V014	ALBANY	2100	0.49	50.8	District Paving	NE19PAV901	2018
	93.4	93.5	V014	ALBANY	2100	0.31	69.5	District Paving	NE19PAV901	2018
VT 67A	2.2	2.3	V067A	BENNINGTON	5800	0.32	68.5	Bennington	STP 2973(1)	2018
	2.4	2.5	V067A	BENNINGTON	5800	0.27	73.4	Bennington	STP 2973(1)	2018
	3	3.1	V067A	BENNINGTON	5800	0.27	73.4	Bennington	STP 2973(1)	2018
	3.3	3.348	V067A	BENNINGTON	6400	0.28	72.4	Bennington	STP 2973(1)	2018
VT 100	27.9	28	V100	WILMINGTON	4800	0.39	60.8	District Paving	NE19PAV101	2018
VT 131	0.2	0.3	V131	CAVENDISH	2800	0.58	41.9	District Paving	NE19PAV201	2018
	0.3	0.4	V131	CAVENDISH	2800	0.56	43.9	District Paving	NE19PAV201	2018
	1.1	1.2	V131	CAVENDISH	2900	0.39	60.6	District Paving	NE19PAV201	2018
	1.2	1.3	V131	CAVENDISH	2900	0.36	63.6	District Paving	NE19PAV201	2018
	1.3	1.4	V131	CAVENDISH	2900	0.48	51.7	District Paving	NE19PAV201	2018
	1.7	1.8	V131	CAVENDISH	2900	0.30	70.5	District Paving	NE19PAV201	2018
	1.8	1.9	V131	CAVENDISH	2900	0.51	48.8	District Paving	NE19PAV201	2018
	1.9	2	V131	CAVENDISH	2900	0.48	51.7	District Paving	NE19PAV201	2018
	2	2.1	V131	CAVENDISH	2823	0.45	54.7	District Paving	NE19PAV201	2018
	2.1	2.2	V131	CAVENDISH	2200	0.41	58.6	District Paving	NE19PAV201	2018
US 2	101.2	101.3	U002	CABOT	4100	0.51	49.0	District Paving	NE19PAV702	2018
	101.3	101.4	U002	DANVILLE	4100	0.49	50.8	District Paving	NE19PAV702	2018
	101.4	101.5	U002	DANVILLE	4100	0.57	42.9	District Paving	NE19PAV702	2018
	101.5	101.6	U002	DANVILLE	4100	0.33	67.5	District Paving	NE19PAV702	2018
	101.6	101.7	U002	DANVILLE	4100	0.42	57.6	District Paving	NE19PAV702	2018
	101.7	101.8	U002	DANVILLE	4100	0.34	65.5	District Paving	NE19PAV702	2018
	101.8	101.9	U002	DANVILLE	4100	0.45	54.7	District Paving	NE19PAV702	2018
	101.9	102	U002	DANVILLE	4100	0.28	72.4	District Paving	NE19PAV702	2018
	102.8	102.9	U002	DANVILLE	4100	0.37	62.6	District Paving	NE19PAV702	2018
US 5	186.4	186.5	U005	DERBY	10700	0.27	73.4	District Paving	NE19PAV902	2018
	186.8	186.9	U005	DERBY	10700	0.44	55.7	District Paving	NE19PAV902	2018
	187.2	187.3	U005	DERBY	11320	0.28	72.5	District Paving	NE19PAV902	2018
	187.5	187.6	U005	DERBY	8600	0.43	56.9	District Paving	NE19PAV902	2018
US 7	10.2	10.3	U007	BENNINGTON	6700	0.38	61.6	Bennington	NH 2966(1)	2018
	10.3	10.4	U007	BENNINGTON	6700	0.43	56.7	Bennington	NH 2966(1)	2018
	10.4	10.5	U007	BENNINGTON	6915	0.38	61.6	Bennington	NH 2966(1)	2018
	10.5	10.6	U007	BENNINGTON	7200	0.38	61.6	Bennington	NH 2966(1)	2018
	10.6	10.7	U007	BENNINGTON	7200	0.28	72.4	Bennington	NH 2966(1)	2018
	10.8	10.9	U007	BENNINGTON	8084	0.40	59.6	Bennington	NH 2966(1)	2018
	10.9	11	U007	BENNINGTON	8764	0.59	40.9	Bennington	NH 2966(1)	2018
	11	11.1	U007	BENNINGTON	9100	0.45	54.7	Bennington	NH 2966(1)	2018
	11.1	11.2	U007	BENNINGTON	9100	0.42	57.6	Bennington	NH 2966(1)	2018
	11.2	11.3	U007	BENNINGTON	9100	0.71	29.1	Bennington	NH 2966(1)	2018
	11.3	11.4	U007	BENNINGTON	8607	0.72	28.1	Bennington	NH 2966(1)	2018
	11.4	11.5	U007	BENNINGTON	7400	0.28	72.4	Bennington	NH 2966(1)	2018
	11.5	11.6	U007	BENNINGTON	8800	0.36	63.6	Bennington	NH 2966(1)	2018
	11.6	11.7	U007	BENNINGTON	9400	0.35	64.5	Bennington	NH 2966(1)	2018
	11.7	11.8	U007	BENNINGTON	9400	0.41	58.6	Bennington	NH 2966(1)	2018
	145.9	146	U007	MILTON	13900	0.58	41.9	District Paving	NE19PAV501	2018
I 89 SB	52.5	52.6	I089-S	MONTPELIER CIT	11300	0.26	73.7	ntpelier-Waterb	IM SURF(59)	2018
I 91 NB	11.9	12	I091	BRATTLEBORO	8200	0.30	70.2	ilford-Brattlebo	NH SURF(60)	2018

5.3 Final Conclusions of Performance Criteria Development

A study was conducted to help determine preliminary performance criteria for Connecticut, Maine and Vermont transportation agencies for potential use in Performance Related Specifications and

Balanced Mixture Design. Different levels of data availability illustrated different procedures for how to develop initial criteria.

For the Connecticut DOT, laboratory performance test data was not available. Therefore, GOOD and POOR performing pavement sections were identified from the Pavement Management System (PMS). It was recommended that the field sections be cored and the recovered cores tested in the laboratory for their respective laboratory performance. Field to laboratory relationships, taking into consideration factors such as field traffic, design gyration level, asphalt binder grade, can then be developed and evaluated.

For the Maine DOT, laboratory performance data was available along with the corresponding field distress information. An extensive PMS database was not provided. The results of the analysis recommended:

- Fatigue Cracking: IDEAL-CT Index @ 25°C > 150
    - Additional research also recommended to evaluate intermediate temperatures more representative to Maine's climatic conditions
- Rutting: Hamburg Wheel Tracking Test @ 45°C and 20,000 Passes
    - AADT < 5,000: Rutting < 12.5 mm
    - AADT > 5,000: Rutting < 7.0 mm

For the Vermont Agency of Transportation, laboratory performance data and a full PMS database was available for the analysis. After reviewing all of the available data and performance history, the following were recommended:

- Fatigue Cracking: SCB Flexibility @ 25°C > 8.0
    - Additional research also recommended to evaluate intermediate temperatures more representative to Vermont climatic conditions
- Rutting: Hamburg Wheel Tracking Test @ 45°C and 20,000 Passes
    - AADT < 7,000: Rutting < 12.5 mm
    - AADT > 7,000: Rutting < 6.0 mm

Lastly, there may be a means to develop "Regional" performance tests and criteria if state agencies are willing to compare and agree on tests methods and criteria. The benefit of this would be that asphalt producers that work in adjacent states would only need to worry about one set of performance tests and criteria, as opposed to many. For example, it is clear that the Hamburg test results from both Maine and Vermont are similar and general agreement with averaging the proposed criteria could be made so that when;

- AADT < 6,000, rutting is < 12.5 mm
- AADT > 6,000, rutting is < 6.5 mm

A small compromise between state agencies shows the promising result of a common, regional specification.

Regarding fatigue cracking, this too could be accomplished. However, at this time, Maine and Vermont transportation agencies do not utilize the same test procedure. By developing a comparative database of performance results, one could relate one test method to the other. Figure 54 shows an example of test data developed by Rutgers University over the past two years using asphalt mixtures (plant and lab produced) from both New Jersey and New York. The comparison between the IDEAL-CT values and SCB Flexibility Index values are quite good ($R^2 = 0.79$, n = 138). Using the linear regression equation in the figure;

- SCB Flexibility Index of 8.0 = IDEAL-CT Index of 132
- IDEAL-CT Index of 150 = SCB Flexibility Index of 9.3

Interesting enough, the results of each other's fatigue cracking requirements are somewhat similar, when using the relationship for the NJ and NY materials. Although the data does not represent asphalt mixtures native to Maine and Vermont, conceptually the identical method could be conducted to help regionalize a fatigue cracking performance test criteria when state agencies currently do not agree on the same test method.

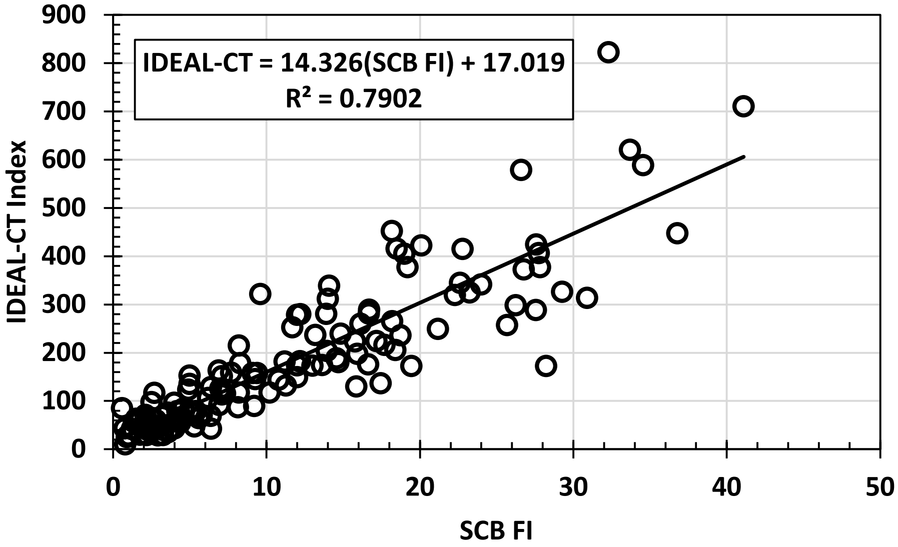


FIGURE 54 Rutgers University Database Comparison of IDEAL-CT and SCB Flexibility Index Tested at 25°C

6.0 IDENTIFICATION OF GAPS AND RECOMMENDATIONS FOR FUTURE RESEARCH

A research project was conducted to help the state agencies of the New England Transportation Consortium (NETC) move towards implementing a Balanced Mixture Design (BMD) system to help design better performing asphalt mixtures. A total of six (6) state agencies were involved in the study; Connecticut, Maine, Massachusetts, New Hampshire, Rhode Island, Vermont. Based on the feedback from the various state agencies throughout the project, as well as information gathered and developed during the study by the Research Team, the following gaps of knowledge and recommendations for future research are proposed.

1. **Improved communication between Materials Bureau and Pavement Management Bureau within the state agencies**. Of the six state agencies involved in the study, only three were able to provide pavement distress information from the respective agency's Pavement Management System (Connecticut, Maine, Vermont). This is particularly concerning since to properly establish laboratory performance criteria that represents field performance, the field performance of the respective asphalt mixtures is critical.

2. **Development of performance test data for various asphalt mixtures**. Only Maine and Vermont were able to provide laboratory performance test results for their respective asphalt mixtures, leaving 4 of the 6 remaining states to determine which test methods to utilize and begin generating information. The Research Team proposed laboratory performance tests to the NETC that were determined to have the "best chance" of implementation within the region. It is highly recommended that the remaining four states consider the recommendations, select laboratory test methods and begin generating asphalt mixture performance data.

3. **Begin evaluating how performance tests can be included within current specification structure**. There are important considerations state agencies need to evaluate regarding how performance testing, and ultimately BMD, will be included within their current specification structure. For example, the following questions need to be addressed;
    a. Application of testing during project (i.e. – mixture design, plant production verification, quality control/quality assurance testing)
    b. Pay adjustment structure
    c. Laboratory responsible for testing (i.e. – state agency, academia partner, accredited laboratory)

4. **Sensitivity of state agency production tolerance to performance test results**. Each state agency has its own production tolerances. The effect of these tolerances on whether a BMD stays balanced or not should be investigated. MassDOT in its efforts to develop and implement a BMD has already been investigating the effect of production variabilities on BMD and the general methodology used could provide guidance for other states (*8*).
    a. Plant production tolerance values also need to be considered. For example, is it OK if the laboratory air voids drop below 2% at the asphalt content previously determined to be within a "balanced zone"?

5. **Actual determination of "optimum asphalt content".** The BMD will provide a range of asphalt contents where the asphalt mixture performance is "balanced." However, to date, there has been little published regarding where to actually select the final asphalt content. Some researchers suggest optimum asphalt content should be 0.4% above the minimum asphalt content in the BMD range, as long as this value stays within balanced zone. Agencies need to be cognizant of the precision and bias of the ignition oven test (Multiple Operator Allowable Range = 0.33%) when determining this value.

6. **Incorporating aging (Laboratory Conditioning) for fatigue cracking assessment.** The aging of asphalt mixtures greatly accelerates the fatigue cracking potential of the asphalt mixture. Unfortunately, the addition of some additives may actually accelerate this age hardening procedure. Rutgers University and the University of Massachusetts Dartmouth have extensively evaluated the use of recycled engine oil bottoms (REOB) materials and have clearly shown that at certain dosage rates, the asphalt binder and mixtures become more susceptible to fatigue cracking. However, this is typically only witnessed after some degree of aging has occurred. Without aging, REOB modified asphalt binders and mixtures behavior very similar to asphalt mixtures without the additive. Some evidence also exists that some rejuvenating materials may also volatize after a certain amount of aging has

occurred, generally nullifying any benefit the rejuvenator may have added (*9*). Therefore, the potential use of aging, and the aging methodology itself, within the BMD also need some consideration.

7. **Performance testing during quality control testing at the asphalt plants**. The adoption of certain test methods may result in an excessive amount of asphalt mixture produced and compacted in the field prior to any test result being reviewed. With all the variables that can occur during asphalt mixture production, it is imperative that some level of performance testing be conducted during routine Quality Control practices to ensure the asphalt mixture is being produced at the level of performance expected by the state agency. Otherwise, the BMD simply gets reduced to volumetric assessment once production starts. Therefore, the concept of performance testing during timely Quality Control testing should also be identified as a gap in the practice and be further addressed.

8. **Round Robin testing conducted within the NETC states**. The adoption of round robin testing within the NETC states will provide a means of evaluating not only the respective variability of the test method, but also a means of assessing the individual state agency test equipment and procedures to ensure testing is conducted accurately. It is highly recommended that the NETC states look to incorporate round robin type programs when the remaining state agencies begin to adopt asphalt mixture performance testing.

7.0 REFERENCES

1. Huber, G., 2013. "History of asphalt mix design in North America, Part 2 Superpave", http://asphaltmagazine.com/history-of-asphalt-mix-design-in-north-america-part-2/ Accessed 2018.

2. National Center for Asphalt Technology (NCAT), Spring 2017 "Moving Towards Balanced Mix Design for Asphalt Mixtures." *Asphalt Technology News*., Volume 29, Number 1.

3. West, R., C. Rodezno, F. Leiva, and F., Yin, 2018, *Development of a Framework for Balanced Mix Design*, National Cooperative Highway Research Program (NCHRP), Transportation Research Board, Washington, D.C., 168pp.

4. Zhou, F., D. Newcomb, C. Gurganus, S. Banihashemrad, E.S. Park, M. Sakhaeifar, and R. Lytton, 2016, *Experimental Design for Field Validation of Laboratory Tests to Assess Cracking Resistance of Asphalt Mixtures*, National Cooperative Highway Research Program (NCHRP), Transportation Research Board, Washington, D.C., 262pp.

5. Bennert, T., E. Haas, E. Wass, and B. Berger, 2020, "Indirect Tensile Testing for Balanced Mixture Design and Quality Control Performance Testing" *Journal of the Association of Asphalt Pavement Technologists (AAPT*), In Press.

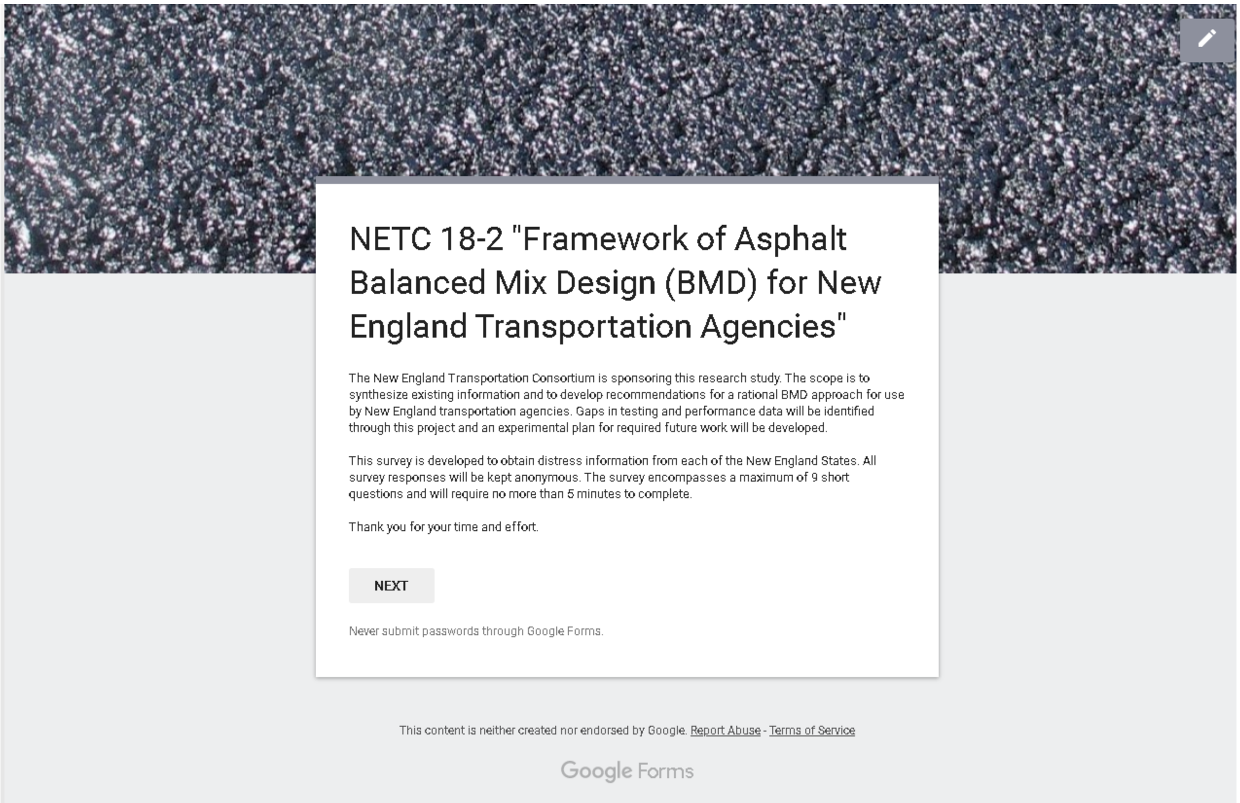
6. Bennert, T., 2012, *Laboratory Evaluation of Maryland Asphalt Mixtures for Composite Pavement Applications*, Report Submitted to Schnabel Engineering LLC for the Maryland State Highway Association (MDSHA), 43pp.

7. Christensen, D.W. and N. Tran, *Relating Asphalt Binder Fatigue Properties to Asphalt Mixture Performance, National Cooperative Highway Research Program (NCHRP), Draft Final Report*, Transportation Research Board, Washington, D.C., 265pp.

8. Mogawer, W., A.J. Austerman, K. Stuart, Z. Fujie, and P. Romero, 2018. "Balanced Mix Design Sensitivity to Production Tolerance Limits and Binder Source" *Journal of the Association of Asphalt Pavement Technologists (AAPT).*

9. Bennert, T., C. Ericson, and D. Pezeshki, 2015, *Rejuvenating Agents with RAP in Hot Mix Asphalt (HMA)*, FHWA-NJ-2015-008, New Jersey Department of Transportation, 124 pp.

Appendix A - Internet Survey



NETC 18-2 "Framework of Asphalt Balanced Mix Design (BMD) for New England Transportation Agencies"

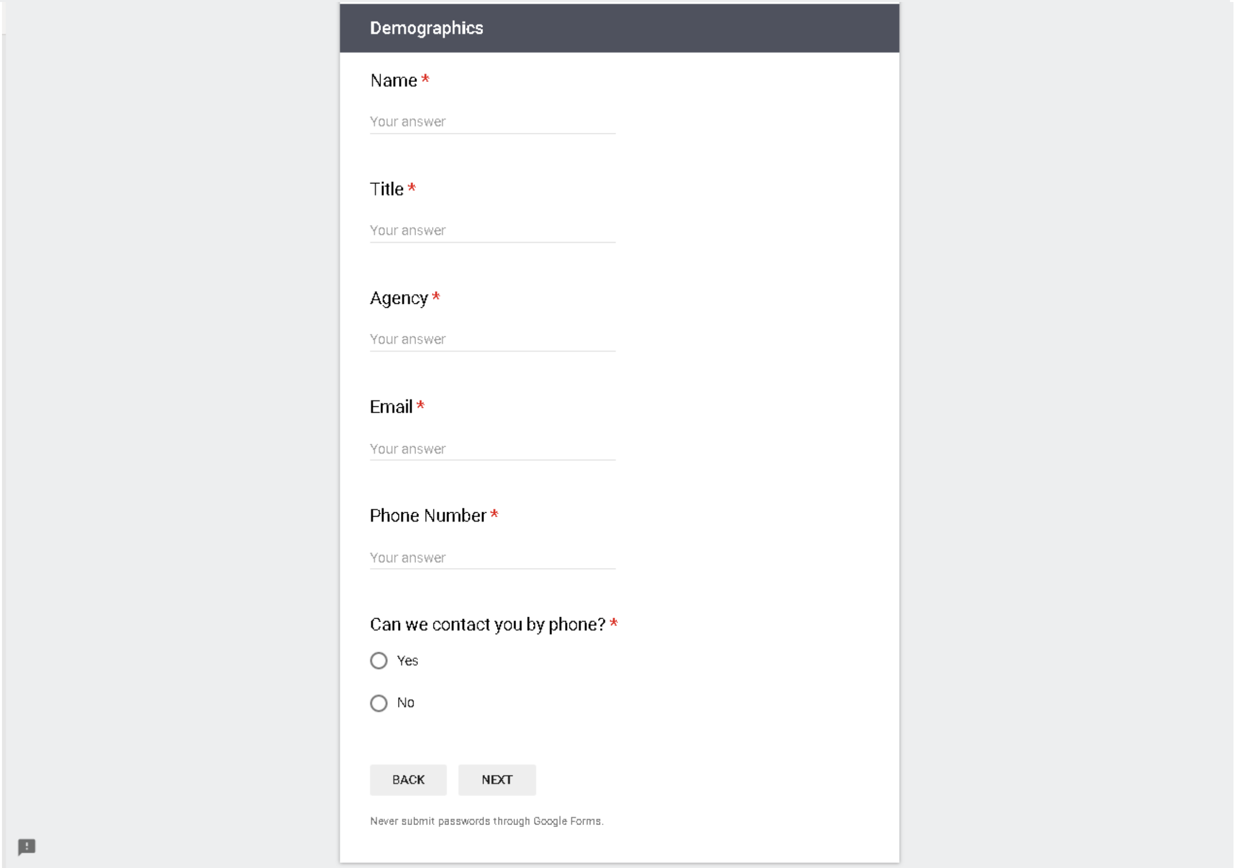
The New England Transportation Consortium is sponsoring this research study. The scope is to synthesize existing information and to develop recommendations for a rational BMD approach for use by New England transportation agencies. Gaps in testing and performance data will be identified through this project and an experimental plan for required future work will be developed.

This survey is developed to obtain distress information from each of the New England States. All survey responses will be kept anonymous. The survey encompasses a maximum of 9 short questions and will require no more than 5 minutes to complete.

Thank you for your time and effort.

Never submit passwords through Google Forms.

This content is neither created nor endorsed by Google. Report Abuse - Terms of Service



Demographics

Name *

Your answer

Title *

Your answer

Agency *

Your answer

Email *

Your answer

Phone Number *

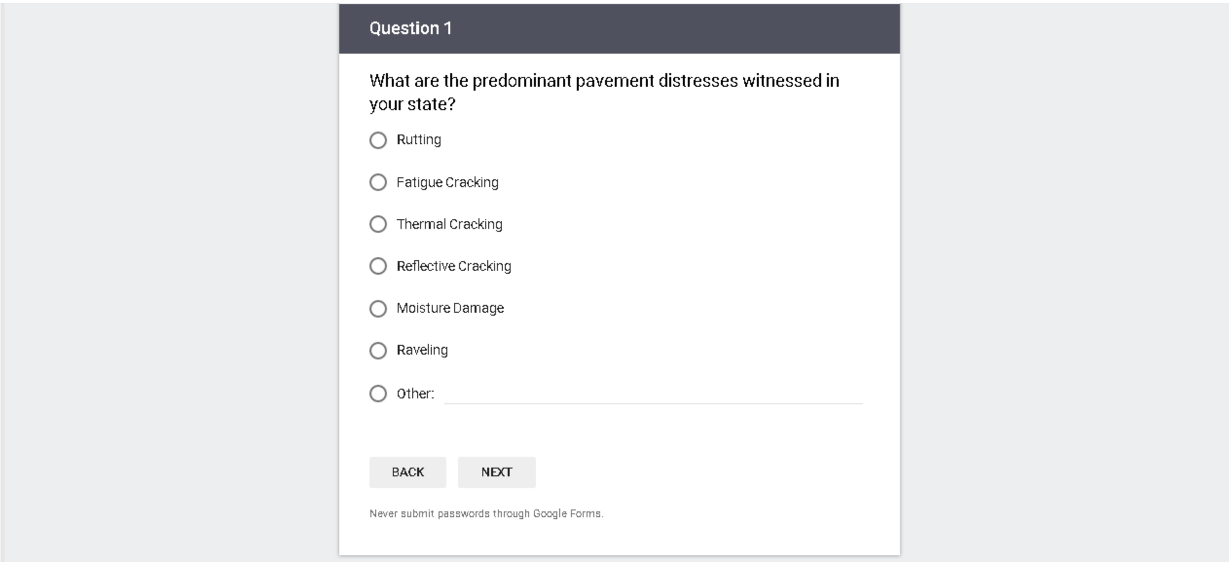
Your answer

Can we contact you by phone? *

○ Yes

○ No

Never submit passwords through Google Forms.

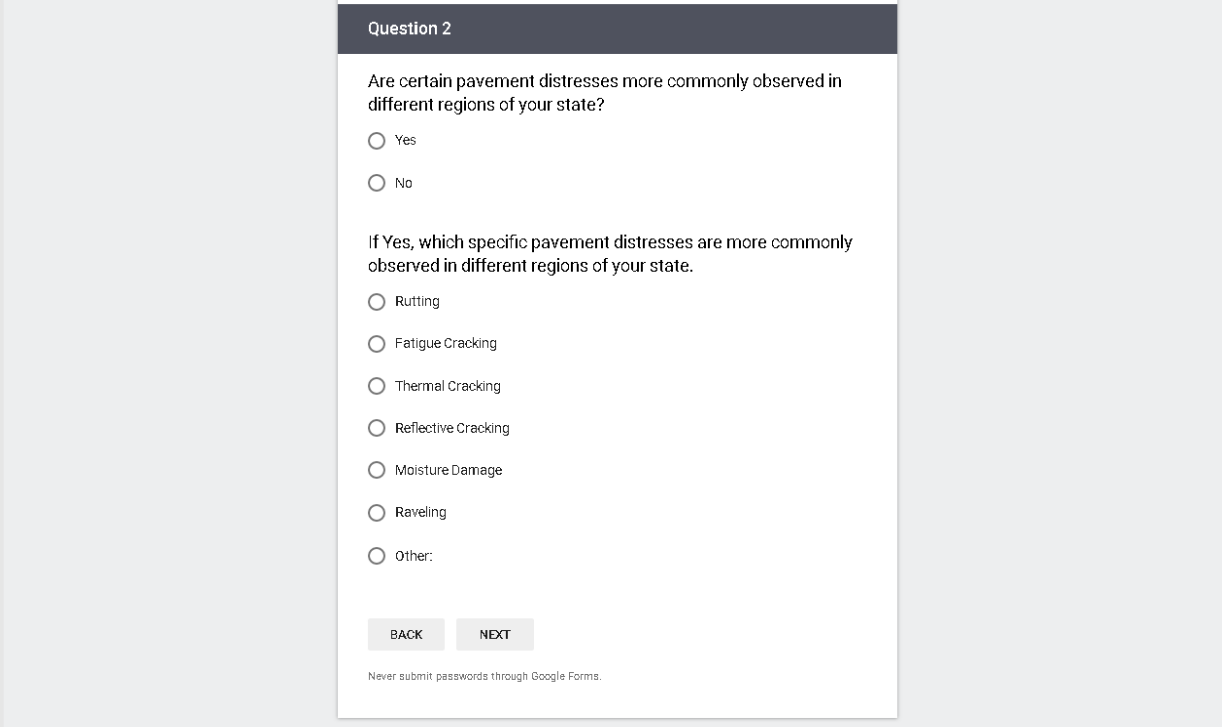


Question 1

What are the predominant pavement distresses witnessed in your state?

○ Rutting
○ Fatigue Cracking
○ Thermal Cracking
○ Reflective Cracking
○ Moisture Damage
○ Raveling
○ Other: ____________________

Never submit passwords through Google Forms.



Question 2

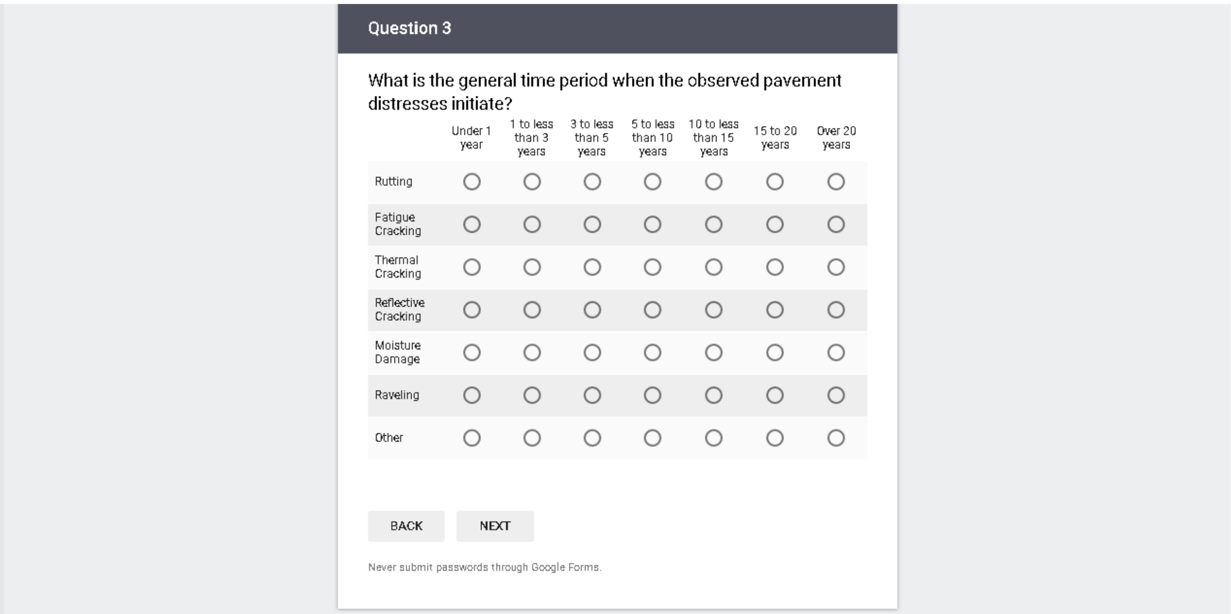
Are certain pavement distresses more commonly observed in different regions of your state?

○ Yes
○ No

If Yes, which specific pavement distresses are more commonly observed in different regions of your state.

○ Rutting
○ Fatigue Cracking
○ Thermal Cracking
○ Reflective Cracking
○ Moisture Damage
○ Raveling
○ Other:

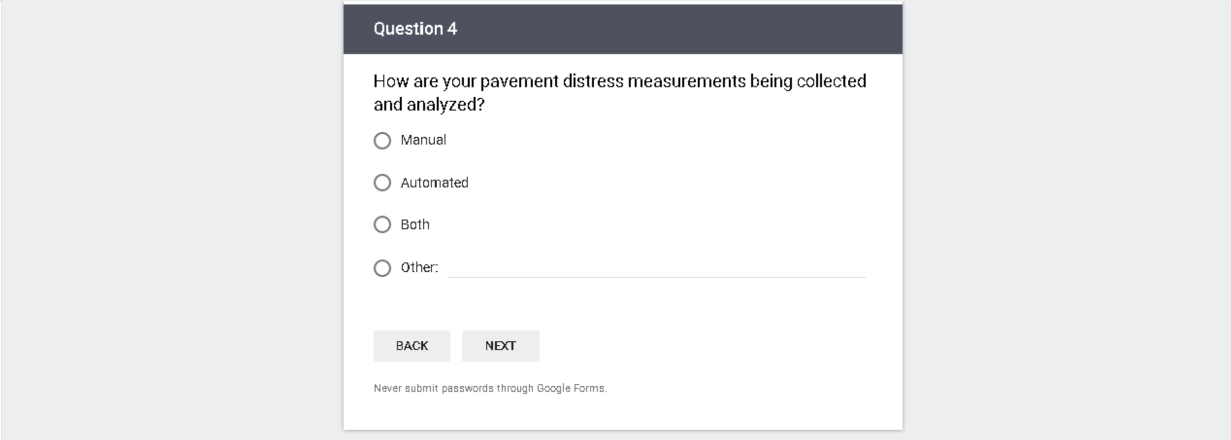
Never submit passwords through Google Forms.



What is the general time period when the observed pavement distresses initiate?

	Under 1 year	1 to less than 3 years	3 to less than 5 years	5 to less than 10 years	10 to less than 15 years	15 to 20 years	Over 20 years
Rutting	○	○	○	○	○	○	○
Fatigue Cracking	○	○	○	○	○	○	○
Thermal Cracking	○	○	○	○	○	○	○
Reflective Cracking	○	○	○	○	○	○	○
Moisture Damage	○	○	○	○	○	○	○
Raveling	○	○	○	○	○	○	○
Other	○	○	○	○	○	○	○

Never submit passwords through Google Forms.



How are your pavement distress measurements being collected and analyzed?

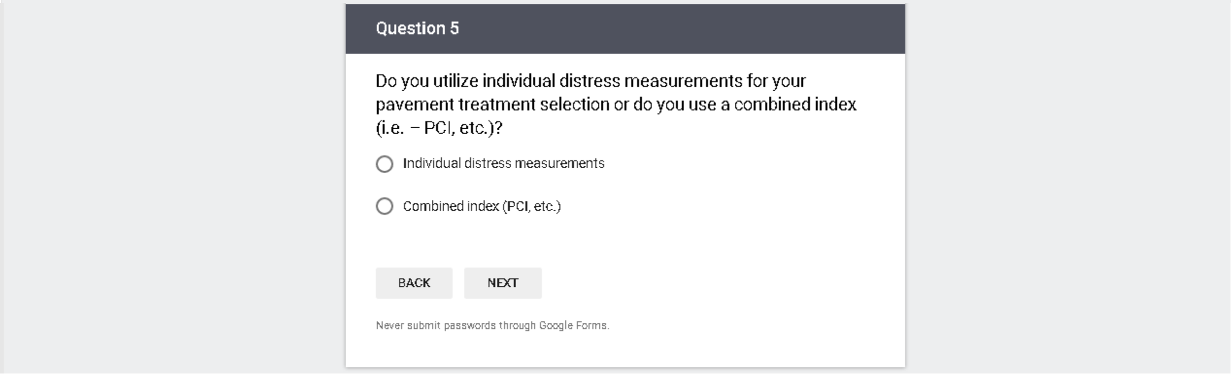
○ Manual

○ Automated

○ Both

○ Other: ______

Never submit passwords through Google Forms.

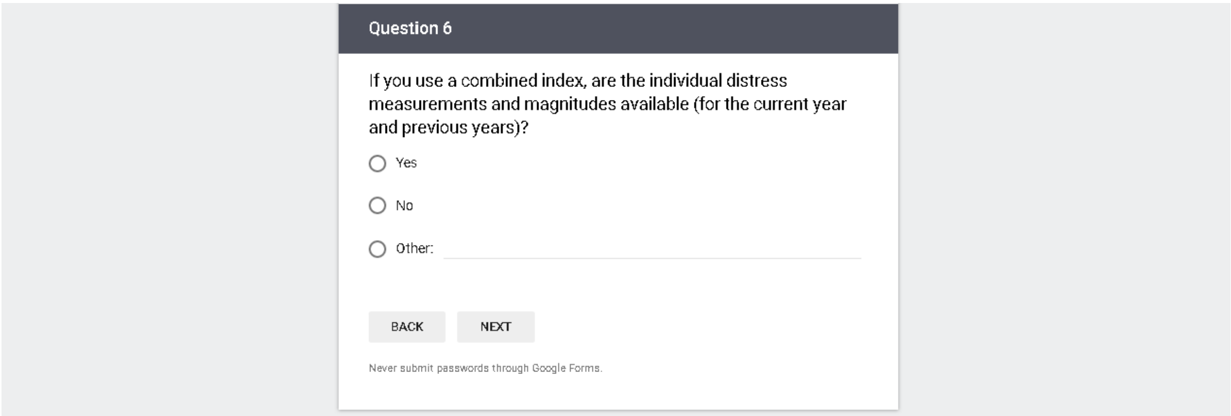


Do you utilize individual distress measurements for your pavement treatment selection or do you use a combined index (i.e. – PCI, etc.)?

○ Individual distress measurements

○ Combined index (PCI, etc.)

Never submit passwords through Google Forms.



Question 6

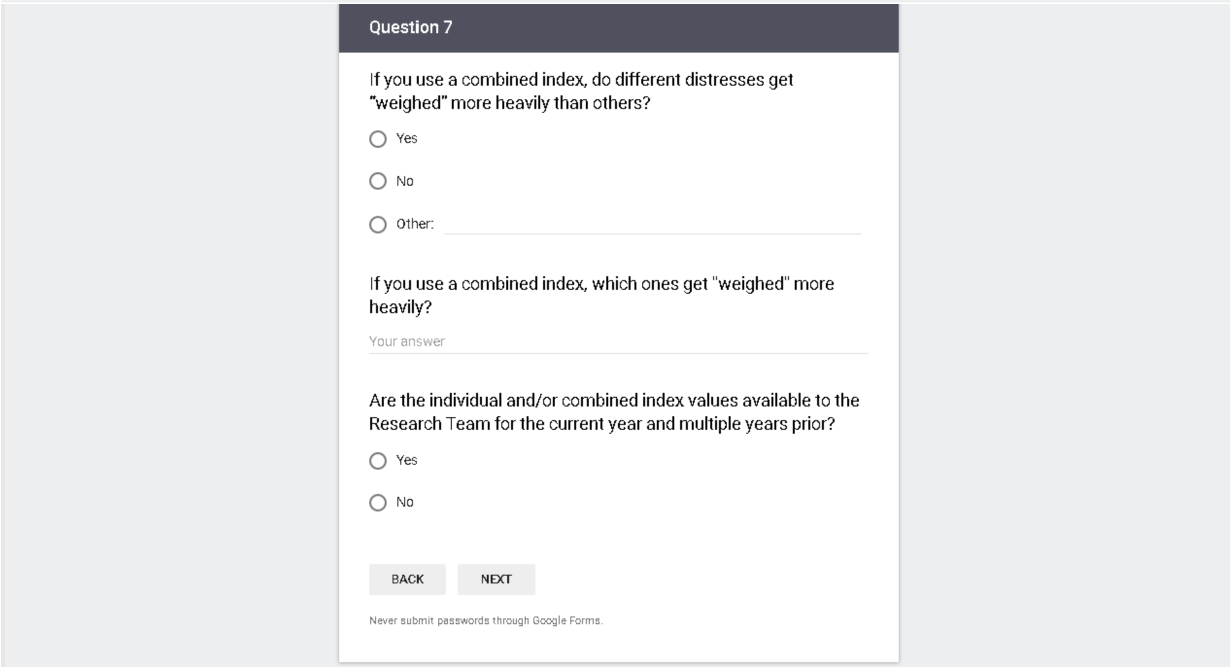
If you use a combined index, are the individual distress measurements and magnitudes available (for the current year and previous years)?

○ Yes

○ No

○ Other: ________________

Never submit passwords through Google Forms.



Question 7

If you use a combined index, do different distresses get "weighed" more heavily than others?

○ Yes

○ No

○ Other: ________________

If you use a combined index, which ones get "weighed" more heavily?

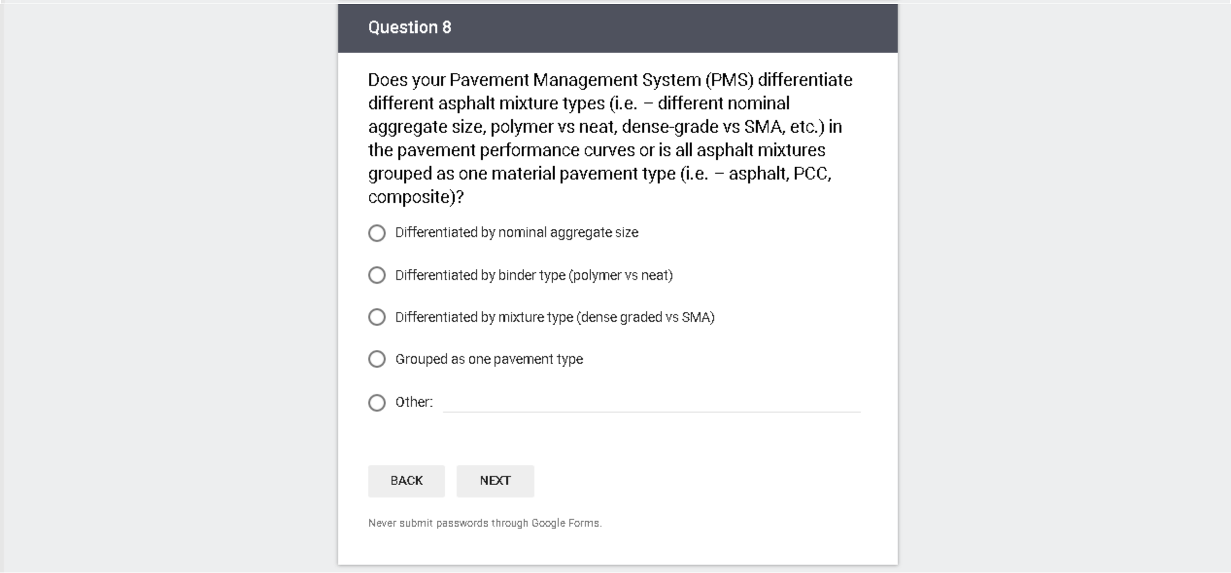
Your answer

Are the individual and/or combined index values available to the Research Team for the current year and multiple years prior?

○ Yes

○ No

Never submit passwords through Google Forms.



Question 8

Does your Pavement Management System (PMS) differentiate different asphalt mixture types (i.e. – different nominal aggregate size, polymer vs neat, dense-grade vs SMA, etc.) in the pavement performance curves or is all asphalt mixtures grouped as one material pavement type (i.e. – asphalt, PCC, composite)?

○ Differentiated by nominal aggregate size

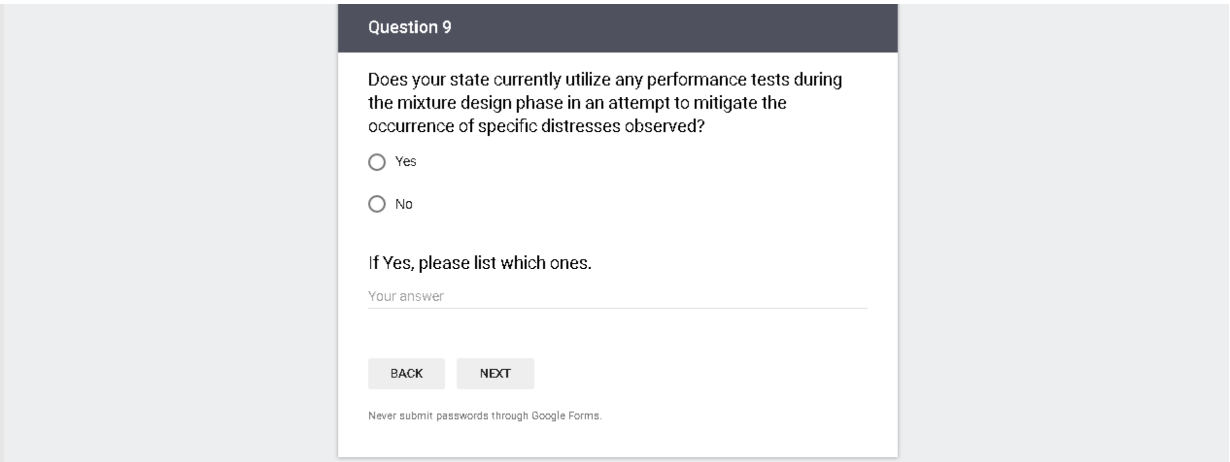
○ Differentiated by binder type (polymer vs neat)

○ Differentiated by mixture type (dense graded vs SMA)

○ Grouped as one pavement type

○ Other: ________________

Never submit passwords through Google Forms.



Question 9

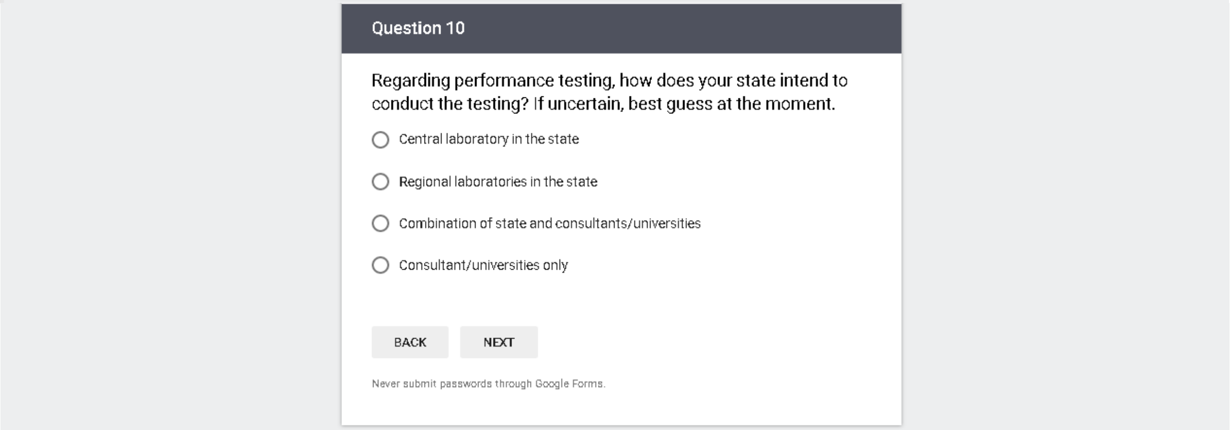
Does your state currently utilize any performance tests during the mixture design phase in an attempt to mitigate the occurrence of specific distresses observed?

○ Yes

○ No

If Yes, please list which ones.

Your answer



Question 10

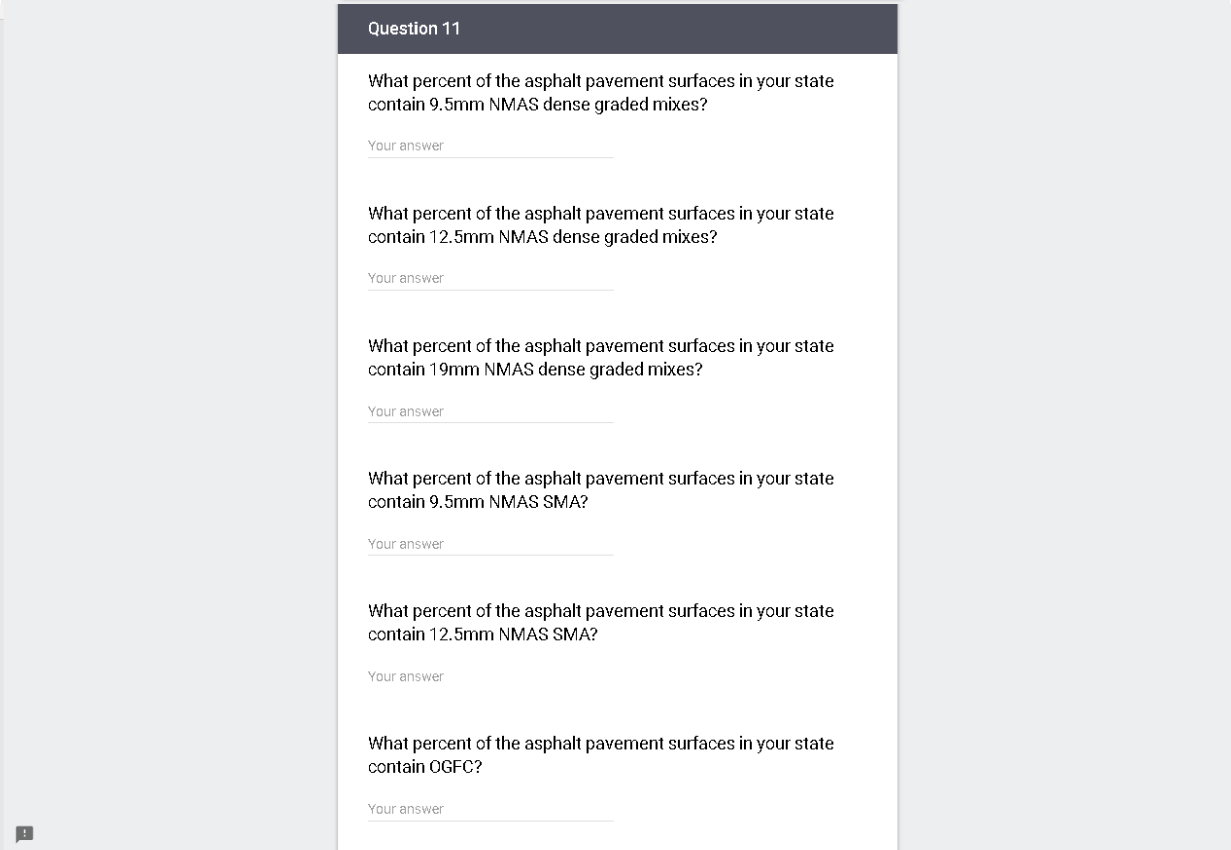
Regarding performance testing, how does your state intend to conduct the testing? If uncertain, best guess at the moment.

○ Central laboratory in the state

○ Regional laboratories in the state

○ Combination of state and consultants/universities

○ Consultant/universities only



Question 11

What percent of the asphalt pavement surfaces in your state contain 9.5mm NMAS dense graded mixes?

Your answer

What percent of the asphalt pavement surfaces in your state contain 12.5mm NMAS dense graded mixes?

Your answer

What percent of the asphalt pavement surfaces in your state contain 19mm NMAS dense graded mixes?

Your answer

What percent of the asphalt pavement surfaces in your state contain 9.5mm NMAS SMA?

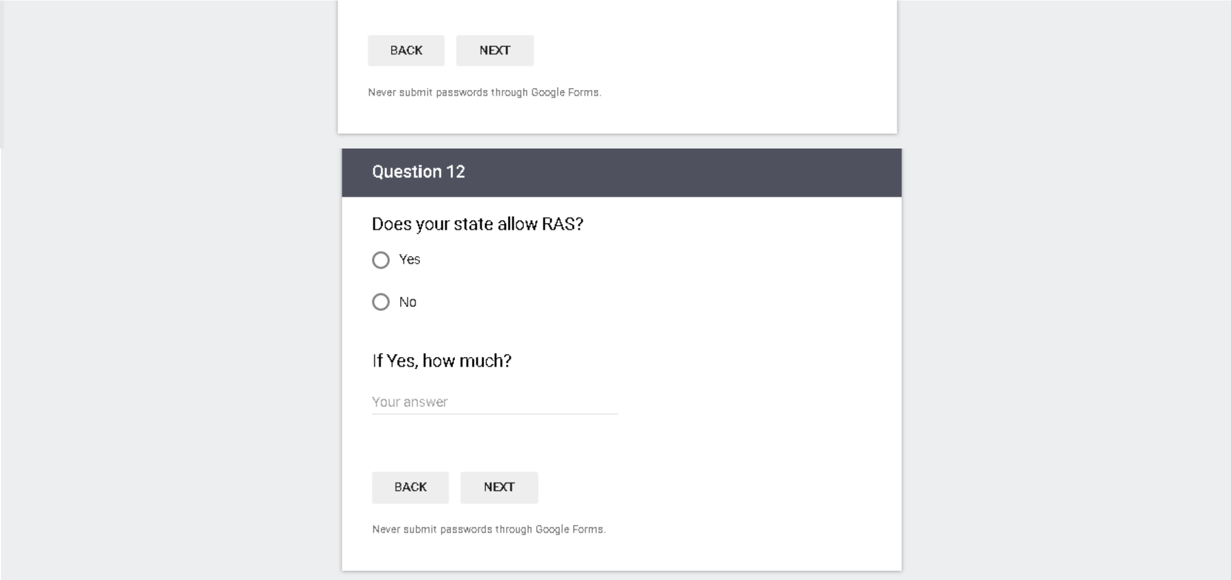
Your answer

What percent of the asphalt pavement surfaces in your state contain 12.5mm NMAS SMA?

Your answer

What percent of the asphalt pavement surfaces in your state contain OGFC?

Your answer



Never submit passwords through Google Forms.

Question 12

Does your state allow RAS?

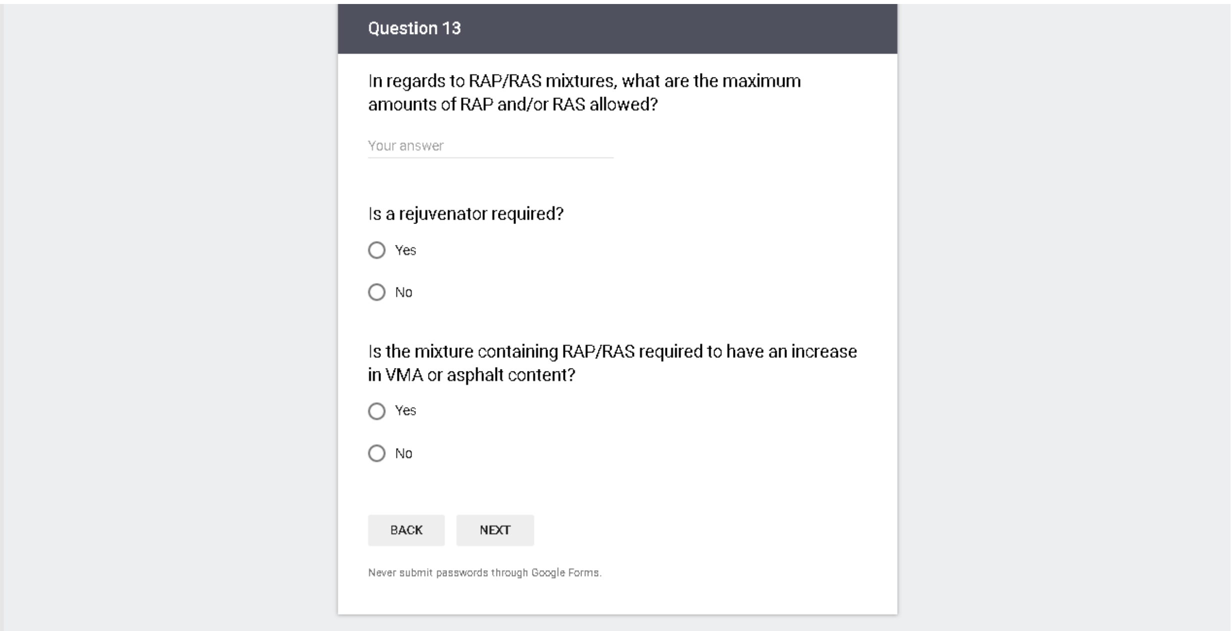
○ Yes

○ No

If Yes, how much?

Your answer

Never submit passwords through Google Forms.



Question 13

In regards to RAP/RAS mixtures, what are the maximum amounts of RAP and/or RAS allowed?

Your answer

Is a rejuvenator required?

○ Yes

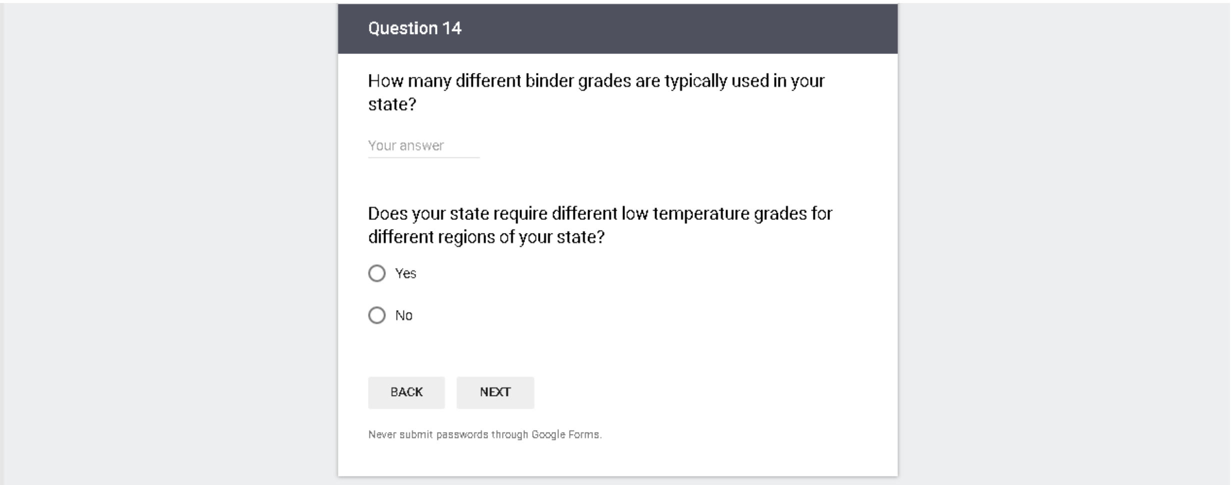
○ No

Is the mixture containing RAP/RAS required to have an increase in VMA or asphalt content?

○ Yes

○ No

Never submit passwords through Google Forms.



Question 14

How many different binder grades are typically used in your state?

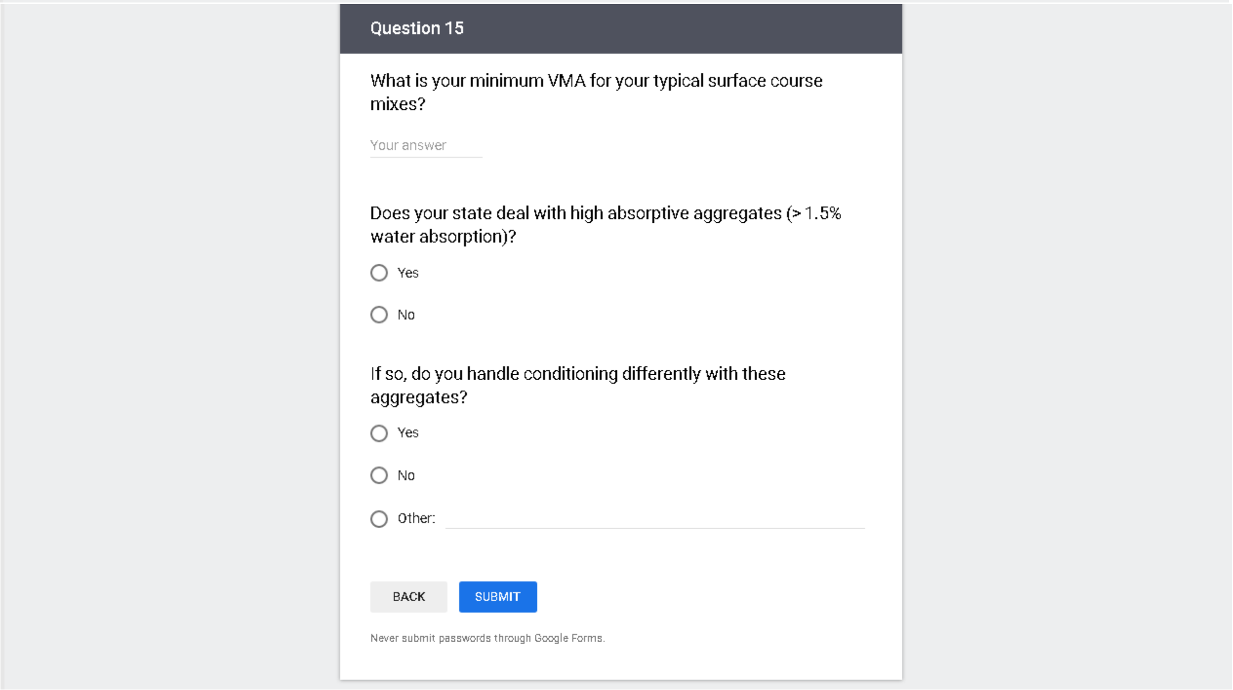
Your answer

Does your state require different low temperature grades for different regions of your state?

○ Yes

○ No

Never submit passwords through Google Forms.



Question 15

What is your minimum VMA for your typical surface course mixes?

Your answer

Does your state deal with high absorptive aggregates (> 1.5% water absorption)?

○ Yes

○ No

If so, do you handle conditioning differently with these aggregates?

○ Yes

○ No

○ Other: __________

Never submit passwords through Google Forms.

Appendix B - State Survey on Performance Tests and Potential Roadblocks

State Agency	Equipment Type											
	Universal Testing Machine (hydraulic, screw driven)	AMPT (If so, provide year purchased)	Hamburg Wheel Tracking	Asphalt Pavement Analyzer	Marshall Stability and Flow	SCB (Stand alone version)	Overlay Tester (Stand alone version)	DCT (Stand alone version)	Beam Fatigue (Stand alone version)	TSRST	MiST	LA Abrasion Machine
Connecticut												
Maine												
Massachusetts												
New Hampshire												
Rhode Island												
Vermont												

Additional Questions	**Response(s)**
Are there specific test methods you are currently looking at, either through funded research or internally? ---------------------------------->	
- If the anwser to above is yes, please provide the test procedure(s) in the RESPONSE areas -->	
Do you use a local university for mixture testing? If so, what test procedures are tested? --->	
Do you envision as issue with the procurement of new equipment and/or procurement of calibration/verification services? ------------->	

State Agency	Test Methods Currently Evaluating											
	AMPT	Hamburg Wheel Tracking	Asphalt Pavement Analyzer	IDEAL-CT	SCB FI	Overlay Tester	Beam Fatigue	LTRC SCB	DCT	TSRST	Low Temp SCB	MiST	LA Abrasion Machine
Connecticut													
Maine													
Massachusetts													
New Hampshire													
Rhode Island													
Vermont													