Research & Technology Transfer

Alaska Department of Transportation & Public Facilities

NHS Innovative Pavement Design Study

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May 2022

Prepared for:
Alaska Department of Transportation and Public Facilities
Statewide Research Office
3132 Channel Drive
Juneau, AK 99801-7898
FHWA-AK-RD-4000(174), HFHWY00077

REPORT DOCUMENTATION PAGE			Form approved OMB No.
Public reporting for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestion for reducing this burden to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-1833), Washington, DC 20503			
1. AGENCY USE ONLY (LEAVE BLANK)	2. REPORT DATE 2022	3. REPORT TYPE AND DATES COVERED	
4. TITLE AND SUBTITLE NHS Innovative Pavement Design 6. AUTHOR Andrew Pavey, Pavement Management Engineer			5. FUNDING NUMBERS HFHWY00077 FHWA-AK-RD-4000(174)
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) State of Alaska DOT&PF Statewide Engineering Services 5800 East Tudor Road Anchorage, AK, 99507			8. PERFORMING ORGANIZATION REPORT NUMBER N/A
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) State of Alaska, Alaska Dept. of Transportation and Public Facilities Research and Technology Transfer 3132 Channel Drive Juneau, AK 99801-7898			10. SPONSORING/MONITORING AGENCY REPORT NUMBER HFHWY00077 FHWA-AK-RD-4000(174)
11. SUPPLENMENTARY NOTES Performed in cooperation with State of Alaska DOT&PF Central Region Materials group, and Newton Bingham PE (Retired Alaska DOTPF Regional Materials Engineer)			
12a. DISTRIBUTION / AVAILABILITY STATEMENT No restrictions			12b. DISTRIBUTION CODE
13. ABSTRACT (Maximum 200 words) This research investigated the history and performance of hot mix asphalts (HMA) used by Alaska DOT&PF. The history and changes of HMA used between the 1970's and 2022 was documented, and currently used mixes were evaluated using pavement management data to determine the rut and transverse (thermal) cracking rates by mix and binder types. Prall testing, used to simulate studded tire wear, and Hamburg testing to determine rut deformation and stripping potential were used on a combination of lab and field mixes. Falling Weight Deflectometer (FWD) testing was performed on recently constructed projects to document fall season modulus values for HMA, base and subbase layers. It was found that the newly implemented PG64-40E binder is providing superior rutting resistance than the previously used PG58-34E binder at low and moderate speeds, the use of a hard aggregate specification is reducing the rate of studded tire rutting, and lower voids in HMA (increasing density) is improving its fatigue cracking and rut resistance. Improving HMA compaction plays a very important role in the longevity of our pavements.			
14- KEYWORDS: Pavement, rut, studded tires, thermal cracking, mix design, binder, aggregate, Hamburg, Prall, Nordic Abrasion, Alaska			15. NUMBER OF PAGES 161 16. PRICE CODE N/A
17. SECURITY CLASSIFICATION OF REPORT Unclassified	18. SECURITY CLASSIFICATION OF THIS PAGE Unclassified	19. SECURITY CLASSIFICATION OF ABSTRACT Unclassified	20. LIMITATION OF ABSTRACT N/A

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METRIC (SI*) CONVERSION FACTORS									
APPROXIMATE CONVERSIONS TO SI UNITS					**APPROXIMATE CONVERSIONS FROM SI UNITS**				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH					LENGTH		
In	inches	25.4		mm	mm	millimeters	0.039	inches	in
Ft	feet	0.3048		m	m	Meters	3.28	feet	ft
Yd	yards	0.914		m	m	Meters	1.09	yards	yd
mi	Miles (statute)	1.61		km	km	kilometers	0.621	Miles (statute)	mi
		AREA					AREA		
in^2	square inches	645.2	millimeters squared	cm^2	mm^2	millimeters squared	0.0016	square inches	in^2
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yd^2	square yards	0.836	meters squared	m^2	km^2	kilometers squared	0.39	square miles	mi^2
mi^2	square miles	2.59	kilometers squared	km^2	ha	hectares (10,000 m^2)	2.471	acres	ac
Ac	acres	0.4046	hectares	ha					
		MASS (weight)					MASS (weight)		
Oz	Ounces (avdp)	28.35	grams	g	g	Grams	0.0353	Ounces (avdp)	oz
Lb	Pounds (avdp)	0.454	kilograms	kg	kg	kilograms	2.205	Pounds (avdp)	lb
T	Short tons (2000 lb)	0.907	megagrams	mg	mg	megagrams (1000 kg)	1.103	short tons	T
		VOLUME					VOLUME		
fl oz	fluid ounces (US)	29.57	milliliters	mL	mL	milliliters	0.034	fluid ounces (US)	fl oz
gal	Gallons (liq)	3.785	liters	liters	liters	Liters	0.264	Gallons (liq)	gal
ft^3	cubic feet	0.0283	meters cubed	m^3	m^3	meters cubed	35.315	cubic feet	ft^3
yd^3	cubic yards	0.765	meters cubed	m^3	m^3	meters cubed	1.308	cubic yards	yd^3
Note: Volumes greater than 1000 L shall be shown in m^3									
		TEMPERATURE (exact)					TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (°F-32)	Celsius temperature	°C	°C	Celsius temperature	9/5 °C+32	Fahrenheit temperature	°F
		ILLUMINATION					ILLUMINATION		
Fc	Foot-candles	10.76	lux	lx	lx	Lux	0.0929	foot-candles	fc
Fl	foot-lamberts	3.426	candela/m^2	cd/cm^2	cd/cm^2	candela/m^2	0.2919	foot-lamberts	fl
		FORCE and PRESSURE or STRESS					FORCE and PRESSURE or STRESS		
lbf	pound-force	4.45	newtons	N	N	newtons	0.225	pound-force	lbf
psi	pound-force per square inch	6.89	kilopascals	kPa	kPa	kilopascals	0.145	pound-force per square inch	psi

These factors conform to the requirement of FHWA Order 5190.1A *SI is the symbol for the International System of Measurements

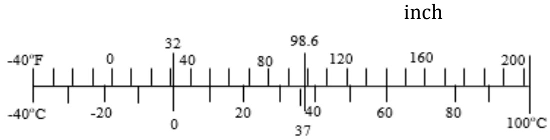


Table of Contents

Figures

Tables

ACKNOWLEDGMENTS

The research reported herein was performed by the Alaska Department of Transportation and Public Facilities (Alaska DOT&PF) Statewide Design & Engineering Services in collaboration with the Central Region Materials Group. The research was performed by the Alaska DOT&PF Pavement Management Engineer, Andrew Pavey, while field data collection was performed by Central Region Materials.

We would like to recognize the Federal Highway Administration for providing funding. The information, findings, and opinions of the research reflected in this report do not necessarily represent the views of the sponsoring agencies.

We acknowledge the contributions of the following individuals who offered their time and advice during critical periods:

- Anna Bosin P.E., Research Manager and Tribal Relations Liaison, DOT&PF
- Ian Bryce, Central Region Materials Engineering Assistant, DOT&PF
- Jeff Currey, Northern Region Materials Engineer, DOT&PF
- Jillian Nicolazzo, Statewide Maintenance Engineer, DOT&PF
- Michael Yerkes P.E., Central Region Materials Group Chief, DOT&PF
- Newton Bingham P.E., Retired DOT&PF Regional Materials Engineer
- Paul Dougherty, Central Region Materials Coordinator, DOT&PF
- Robert Trousil P.E, Southcoast Region Materials Engineer, DOT&PF
- Sirous Alavi, P.E., Pavement Engineer, Fugro
- Steve Saboundjian P.E., Statewide Pavement Engineer, DOT&PF

EXECUTIVE SUMMARY

This NHS Innovative Pavement Design study researched historic changes to the hot mix asphalt (HMA) specifications used between the 1970's and 2022 at the Alaska Department of Transportation & Public Facilities (DOT&PF). It aims to synthesize the evolution behind the changes to department practices and specifications as well as when current specifications were adopted. The study investigates the materials testing and research performed to develop existing specifications. The performance of current materials is evaluated through field testing and historically collected pavement management data to verify current practices. The main focus of study was evaluating treatments used to address rutted pavement, which mainly impacts Central and Southcoast Regions of Alaska.

This study included the following tasks, analysis of and recommendations for:

- Literature review (internal to DOT&PF) including historic Alaska DOT&PF specifications for aggregates and binders.
- Evaluation of Falling Weight Deflectometer (FWD) data to evaluate layer strengths used in the Alaska Flexible Pavement Design Software (AKFPD).
- Evaluation of mixes and binders based on field measurements of rut depths and cracking from annual Pavement Management data collection.
- A Hamburg specification that can be used to evaluate the stripping and plastic deformation potential of pavements.
- A modified HMA specification to improve pavement performance related to rutting from studded tire wear, and thermal cracking in the winter that incorporates the use of new technologies.

This study confirms, through Prall testing (a lab test to simulate studded tire wear), that studded tire resistance is improved through the use of harder aggregates. Prall testing also indicates increased binder contents, incorporation of highly polymer modified binders, and softer binders with a lower end performance grade improves performance. Incorporation of recycled asphalt pavements (RAP) into mixes appears to be detrimental to Prall performance.

Pavement management data collected between 2015 and 2021 was used to analyze the rut wear performance of different mixes, with and without the inclusion of a hard aggregate specification, and different binders. This data verified the findings of the Prall testing; the use of a hard aggregate specification improves studded tire wear resistance, and highly modified binders improves rut resistance as well.

Non-destructive falling weight deflectometer (FWD) testing was performed in the fall of 2020 to verify the design modulus values (representing layer strength) used in the AKFPD. Testing indicated that the default values in the software for Fall Modulus HMA, and foamed asphalt stabilized base (FASB) may be conservative and should be reevaluated for increase. Other seasonal modulus values should also be evaluated through future research.

Hamburg testing was performed on field cores taken at the time of FWD testing and on field mixes from other projects under construction during the 2020 construction season. Testing indicated that currently used hot mix asphalts would largely pass a specification of less than 10mm rut depth at 10,000 cycles at 50°C for highly modified binders, or at 45°C for other binders. Only two samples failed, both with a high Voids in the Total Mix (VTM). Testing indicates a direct relationship between VTM and rut depth for Hamburg testing, where high VTM leads to high rut depths.

The research indicates that VTM, or the air voids in the pavement structure, have an enormous impact on pavement performance. VTM impacts the performance of studded tire wear based on Prall testing, the fatigue life of pavements in both analyses performed in the Alaska Flexible Pavement Design Software and research performed by the National Center for Asphalt Technology (NCAT). It also directly relates to the permeability of pavements.

Two versions of a modified HMA specification are proposed for evaluation from this research to better resist rutting from studded tire abrasion and reduce thermal cracking. One version uses standard construction and testing methods, and another that incorporates new technology, including use of a dielectric profiling systems (DPS) following AASHTO PP 98-20 for improved joint densities and continuous evaluation of field hot mix density.

CHAPTER 1 – INTRODUCTION AND RESEARCH APPROACH

1.1 Background

Alaska spans a large geographic area, and its DOT&PF is split into three different regions. These three regions all contend with different problems when it comes to designing their pavements.

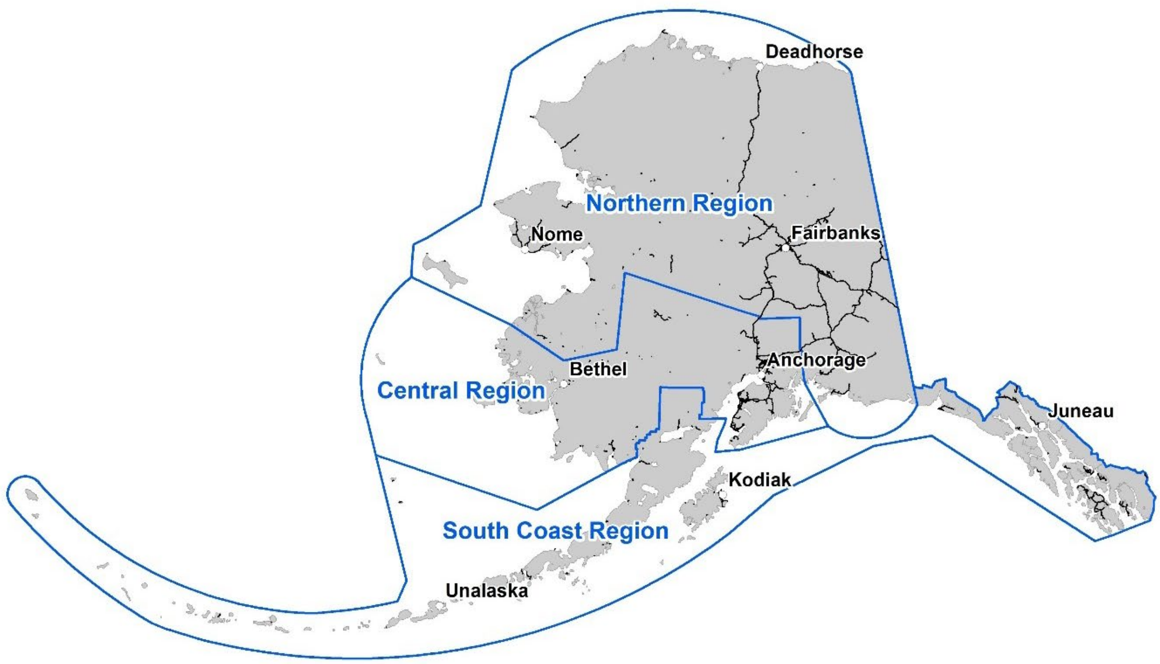


Figure 1 - Regions

Northern Region extends from the North Slope to south of Fairbanks and contains the majority of the National Highway System (NHS) in Alaska. Northern Region pavements may have to be built over permafrost and can be exposed to extreme temperature ranges of 90°F ambient temperature in the summer and -70°F in the winter, causing major thermal cracking.

Central Region consists of the southcentral area and includes major population centers, namely Anchorage, the Mat-Su Valley, and Kenai Peninsula, and has the state's highest traffic volumes. Central Region does not have much permafrost, nor the extreme temperature ranges that Northern Region has. But due to the high traffic volumes, roughly 35% of which use studded tires on passenger vehicles (Abaza, 2019), the major roads in Anchorage and the Mat-Su Valley rut from studded tire erosion at a rapid rate.

Southcoast Region includes the panhandle, Kodiak Island and the Aleutian Chain. It is typically more temperate than the rest of the state and has significant amounts of rainfall. Southcoast Region does not have unstable permafrost or extreme temperatures but does suffer from studded tire wear on some roads.

Background for this research was provided by a historic review of Alaskan pavement properties that affect asphalt pavement rutting and cracking, and the evolution of the pavement management system with the related equipment used for data collection and analysis. This included a review of asphalt binder properties and specifications, hot mix asphalt specifications and design methods, and pavement management data collection and analysis.

1.2 Problem Statement and Research Objective

The evolution of hot mix asphalt in Alaska has not been documented, and the performance of currently used hot mix asphalts and binders has not been through a formal rut and cracking performance analysis using annually collected pavement condition data. More recently, DOT&PF has adopted an asset management plan and invested in a pavement software management system. To maximize these two recent shifts in business practices, it is valuable to look back at past practices' performances and confirm any gains in combating cracking and rutting with asphalt pavement design for the three regions.

This project focuses on understanding the history of hot mix pavements in Alaska, from the mixes and binders used from the 1970's to present day and why the current mixes and binders are being used. The currently used mixes need to be evaluated for rut performance using collected pavement management data, thermal (transverse) cracking performance, the strength evaluated by use of a falling weight deflectometer and deformation/stripping performance evaluated by Hamburg.

Based on the results of the internal literature review and evaluation of currently used mixes and binders, Prall and Hamburg testing, updated specifications may be recommended for consideration by DOT&PF.

1.3 Research Approach

1.3.1 Literature Review

A literature review was performed to evaluate the previous research that had been done into binder and mix performance of Alaskan pavements, the economic impacts of studded tires on our pavements, and evaluation of dielectric profiling systems for continuous collection of density. This is presented in more depth in Appendix A.

1.3.2 Historic Mix and Binder Review

To understand the history and evolution of pavements in Alaska, Newton Bingham was asked to participate on this project. Newton was the Central Regional Materials Engineer from 1994 to 2018 and performed significant research into Alaskan binders and pavements. He contributed to and developed a large number of specifications for hot mix asphalts and binders used across the state.

The history of Alaskan hot mix asphalts and binders was researched, and changes were documented between the 1970's and 2021. This includes the adoption of Superpave mixes, and the evolution of binders from using penetration grading, as was used in the 1970's, to the use of Performance Graded (PG) asphalts with multiple stress creep recovery (MSCR) specifications as are used across Alaska now. Chapter 3.1 and Appendix B provide more details on this chronologic transition of asphalt mixes and binders.

Polymer-modified binders have been used throughout Alaska since the early 2000's to resist rutting from plastic deformation. They have been further modified since approximately 2004 in Central Region to better resist thermal cracking. Northern Region began to evaluate the use of modified binders for thermal cracking resistance in the late 2000's and in 2011 Northern Region used their first PG 52-40 binder, which they began using more widely in 2018.

1.3.3 FWD and Coring Analysis

The Central Region Materials group performed Falling Weight Deflectometer (FWD) testing using a Dynatest HWD CP 15 (Figure 3) to measure the deflections in the field on 11 different projects having commonly used mixes, binders, and base types. 6" pavement cores were taken at the locations tested to verify pavement thicknesses and to be used for Prall and Hamburg testing. The locations are shown below (Figure 2). FWD is further discussed in Chapter 3.4, and Appendix D.

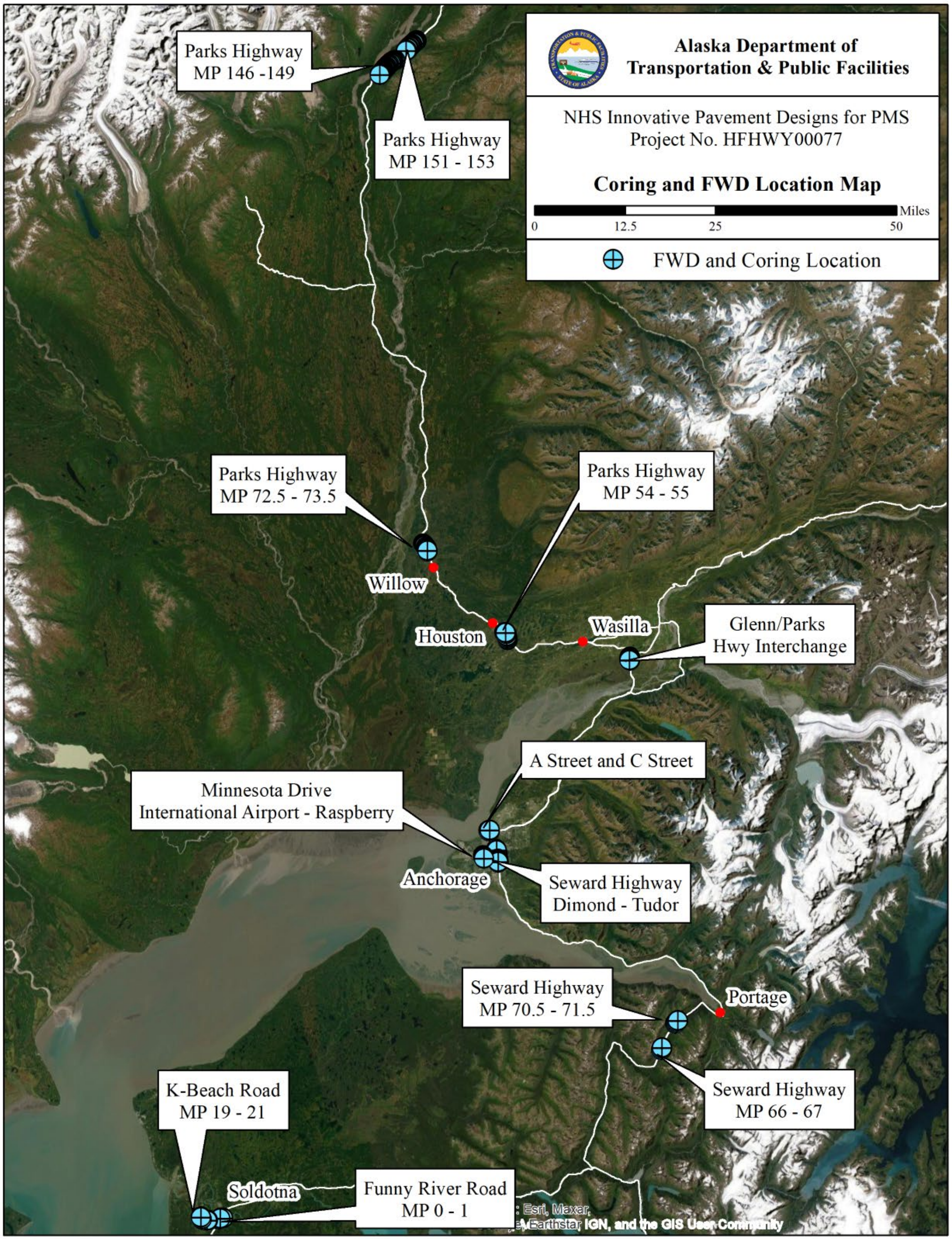


Figure 2 - Coring and FWD Locations

The collected data was processed through Dynatest's Elmod 6 software to back-calculate modulus values, which were then compared to the standard values used in the Alaska Flexible Pavement Design Software. The equipment used for collection is shown in Figure 3 and for more detailed maps of data collection locations see Appendix D.

Figure 3 - FWD Equipment - Dynatest HWD CP 15 (DOT&PF, 2008)

This equipment operates by dropping a load onto a circular plate resting on the pavement surface. This creates a load pulse simulating a vehicle travelling across the pavement surface. The equipment measures the pavement deflection from this load pulse, creating a deflection basin from nine measurements taken by the equipment. The loading, and these recorded deflections can be used with the known pavement structure to calculate the underlying material strengths.

1.3.4 Prall and Hamburg Testing

Prall Tests (ATM 420) simulate studded tire wear and were conducted at the Southcoast Region laboratory. Prall testing provides an understanding of studded tire wear potential of mixes by VTM, mix and binder type.

Prall testing is an abrasion test where a cylindrical specimen having a 100mm diameter and 30mm length is brought to a temperature of 5°C and worn for 15 minutes by 40 steel spheres while cooling water runs over it. The loss of volume is provided in cm^3 and is an abrasion value, where the larger the value the worse the performance. Figure 4 shows a core that has been cut for Prall testing covered with the steel spheres.

Figure 4 - Prall Sample (DOT&PF)

Hamburg Wheel Tracking Tests (AASHTO T32-14) simulate stripping and deformation and were conducted at the Central Region laboratory. Hamburg test results provide an understanding of stripping and deformation potential of current mixes by VTM.

Hamburg testing uses loaded steel wheel tracks over two pavement cores submerged in a heated water bath. The deformation is measured against the number of loading cycles. Note that the equipment used for Hamburg testing was configured for an APA rut test (AASHTO T340-10) when the photo was taken (Figure 5).

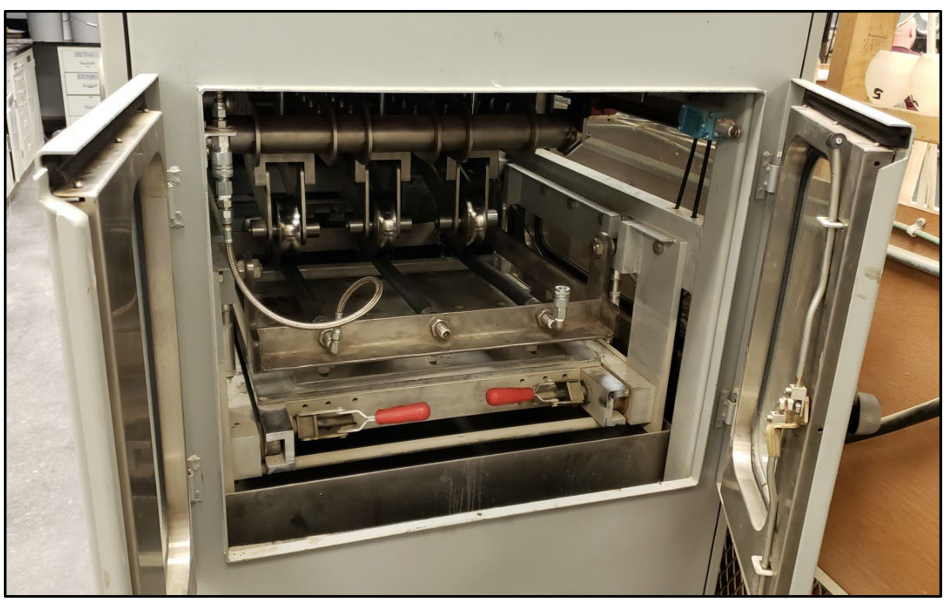


Figure 5 - Hamburg / APA Equipment (DOT&PF, 2021)

Prall Tests and Hamburg Wheel Tracking Tests were conducted on asphalt cores collected during the FWD field work, and on lab-created samples. The lab-created samples were made from a field mix of ongoing construction projects. The lab samples for Prall testing used varying binder and void contents to evaluate the impact of those variables on Prall performance. The lab samples for Hamburg testing were given varying void contents to determine the influence of void content on Hamburg performance.

1.3.5 Pavement Management Data Evaluation

The Pavement Management Program in Alaska has been contracting collection of pavement condition data across the connected road system in Alaska since 2002. Initially rut and roughness data were collected using a Dynatest Mark III Road Surface Profiler, but in 2015 Alaska transitioned to requiring collection using a Laser Crack Measurement System (LCMS) (Figure 6) that included cracking data in addition to rut and roughness.

LCMS data for rut and transverse cracking analysis was compiled from 2015 through 2020. Data from 2021 was included for the rut analysis only as the data collection contractor was able to provide that earlier than cracking data.

Collected LCMS data was used to evaluate rut rates on commonly used pavement types throughout Alaska by mix and binder types. The LCMS data was also used to compare the performance of mixes with modified binders to unmodified binders for rates of transverse, or cold temperature cracking development.

Figure 6 - Pavement Management Data Collection Equipment (DOT&PF, 2019)

Historic project locations for rut and transverse cracking evaluation were selected to provide a wide variety of mixes and binders. Mixes were selected that incorporated hard aggregates, local aggregates, modified binders, unmodified binders, and Type II and Type V pavements. Projects selected for transverse cracking analysis had been rehabilitated or reconstructed in the last construction project, where at a minimum the full depth of pavement had been replaced and were at least 3 years old. This criterion was used to exclude mill and fill or overlay projects where existing cracks would reflect through the new HMA.

The variables used for rut and cracking analysis were:

- Hard Aggregate
- Region
- Mix Type
- Binder Type
- RAP Percentage
- Speed Limit

The projects and data are further discussed in Chapter 3.3 and Appendix E, Pavement Management Data. Not all the data used in the rut and cracking analysis is included in the appendix but can be provided upon request.

1.3.6 Specification Development

Based on the outcome of the literature review, documentation of the history of hot mix asphalts and binders, lab testing, and evaluation of pavement condition data updated specifications are provided for consideration by the three Alaska DOT&PF Regions (Appendices F and G).

CHAPTER 2 – SUMMARY OF FINDINGS

2.1 Historic Hot Mix Asphalt and Binders

The Alaska DOT&PF has used a wide variety of hot mix asphalts and binders over the years. They have advanced from older types of Type I and II pavements with AC-5 binder, used from the 1970's into the 1990's, to the currently used Type II (Marshall) and Type V (Superpave) hot mix asphalts incorporating performance graded (PG) binders. These Type V pavements use hard aggregates (including a Nordic Abrasion specification) and highly modified binders to combat studded tire wear and plastic deformation in regional special provisions.

The methods of specifying the binders used in mixes have also evolved through the years. When Central Region realized that rutting was occurring from plastic deformation in the summer and from studded tire wear in the winter, polymer modified asphalt was specified to control plastic deformation in urban areas of the region.

Central and Northern Regions initially used Softening Point (AASHTO T 53) and Toughness & Tenacity (AASHTO D 5801) as PG+ tests while Southcoast Region used Elastic Recovery (ER) (AASHTO T 301) to define polymer loading. Central Region stopped using Toughness & Tenacity (T&T) since binder was so highly polymer modified that samples could not complete the elongation required in the T&T test, causing failing results. Central Region adopted Multiple Stress Creep Recovery (MSCR) (AASHTO T 350) standard test method. All Regions are considering making AASHTO M 332 a Statewide specification for polymer modified binders using MSCR.

2.2 FWD Analysis

Data collected using the Falling Weight Deflectometer in the fall of 2020 was compared to Fall Modulus values currently used in the Alaska Flexible Pavement Design Software for HMA. It was found that the design modulus values from the software are adequate for highly polymer modified binders and could be raised slightly for the Fall Modulus value. The design modulus values are adequate for the unmodified hot mix asphalt while the design modulus value for foamed asphalt stabilized base could be raised.

2.3 Prall Testing Analysis

The Prall testing performed historically and as part of this project indicates the following items improve studded tire wear resistance:

- Increasing polymer content
- Increasing binder content
- Lowering the bottom end of the performance grade
- Lowering void content
- Using hard aggregates

The inclusion of RAP and high void contents provides for worse results and RAP should not be allowed for use on mixes in areas with high studded tire wear.

2.4 Hamburg Analysis

The Hamburg analysis indicates that existing mixes and binders would largely pass a specification of 10,000 cycles at 7% VTM for testing temperatures of 50°C for highly modified binders (PG 64-40E, PG 58-34E) and 45°C for other binders. Testing indicated a direct relationship between VTM and rut depth.

2.5 Pavement Management Data Analysis

Pavement Management data from 2015 – 2021 was evaluated to determine the performance of commonly used hot mix asphalt for rates of rut and transverse crack development.

It was found that Type VH pavements, Superpave with a hard aggregate specification (Nordic Abrasion less than 8), have a lower rut rate than Type V pavements, Superpave without hard aggregates. Type VH pavements with highly modified PG 64-40E binders have a lower rate of rutting than Type VH pavements with PG 58-34E binders. This is consistent with the findings of the Prall testing.

Transverse cracking, assumed to be low temperature thermal cracking, was evaluated using collected LCMS data from 2018 to 2020 to evaluate the performance of modified binders compared to unmodified binders. LCMS data collected from 2015 – 2017 was excluded as it was collected by a different vendor and did not trend with the more recently collected data.

With the limited dataset no conclusions could be made; however, field observations of thermal cracking and the literature review of critical cracking temperatures of Alaska's pavements (Liu, 2020) indicate modified binders are providing superior resistance to cold temperature cracking compared to unmodified binders. This will be further evaluated in the future as more data becomes available.

2.6 Impact of Void Contents on Pavements

Void contents have a major impact on our pavements. High air voids lead to increased permeability and lower fatigue life in pavements (NCAT Report 03-02, NCHRP 531), inferior cracking performance (NCAT Report 19-08), increased studded tire wear based on Prall testing, and higher deformation potential based on Hamburg testing.

New technologies (in paving equipment and mix designs) have emerged to achieve high percent compactions in pavements. The minimum percent compaction on field mat and joints for Type II pavements should be considered being raised to of 93% compaction (7% air voids) from the current lower specification limits of 92% and 91%.

CHAPTER 3 – FINDINGS

3.1 Historic Hot Mix Asphalt and Binder Changes

Alaska DOT&PF has made significant changes to its binder and aggregate specifications since the 1970's. This section includes the most pertinent information on the adoption of the Multiple Stress Creep Recovery specification for binders, Prall testing that led to the adoption of a hard aggregate specification, and the influence of binders on Prall testing. For more details on the historic changes to Alaska DOT&PF binder and aggregate specifications, refer to Appendix B.

3.1.1 Alaska DOT&PF Transition to AASHTO M332 (MSCR)

The deficiencies in Alaska's PG+ specifications for polymer modified asphalt (Softening Pont, T&T and Elastic Recovery) led to the adoption of AASHTO M 332 Multiple Stress Creep Recovery in 2014. The adopted MSCR specifications were defined by test results from the binder's suppliers were currently providing for modified asphalt. See Appendix B for more detailed explanations about the problems encountered with the PG+ specifications.

Refineries in Alaska only produce PG 52-28 (neat asphalt), so chemical modifications have to be made to accomplish wider PG temperature grades.

Figure 7 illustrates how the current temperature ranges affects the modification of asphalt binder coming from the refinery. To meet the temperature extremes required by the PG system, modification to the refined asphalt is required by using polymers, chemicals, and extender oils, creating an engineered PMA binder.

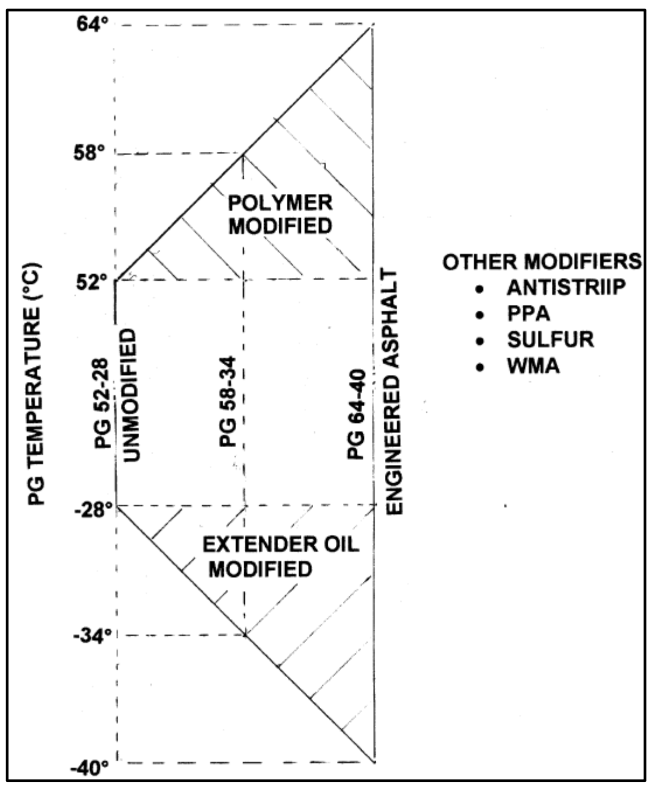


Figure 7 - Engineered PMA Binder (DOT&PF)

The purpose of polymer in modified asphalt is to resist rutting from plastic deformation, reduce studded tire wear, reduce thermal cracking, and to increase the strength of HMA. Zones for a MSCR specification were established based off of MSCR testing performed on existing binders that were using Softening Point and T&T tests. The resulting bounds from that testing are plotted on the graph below (Figure 8) and currently used MSCR specifications are based on these test results.

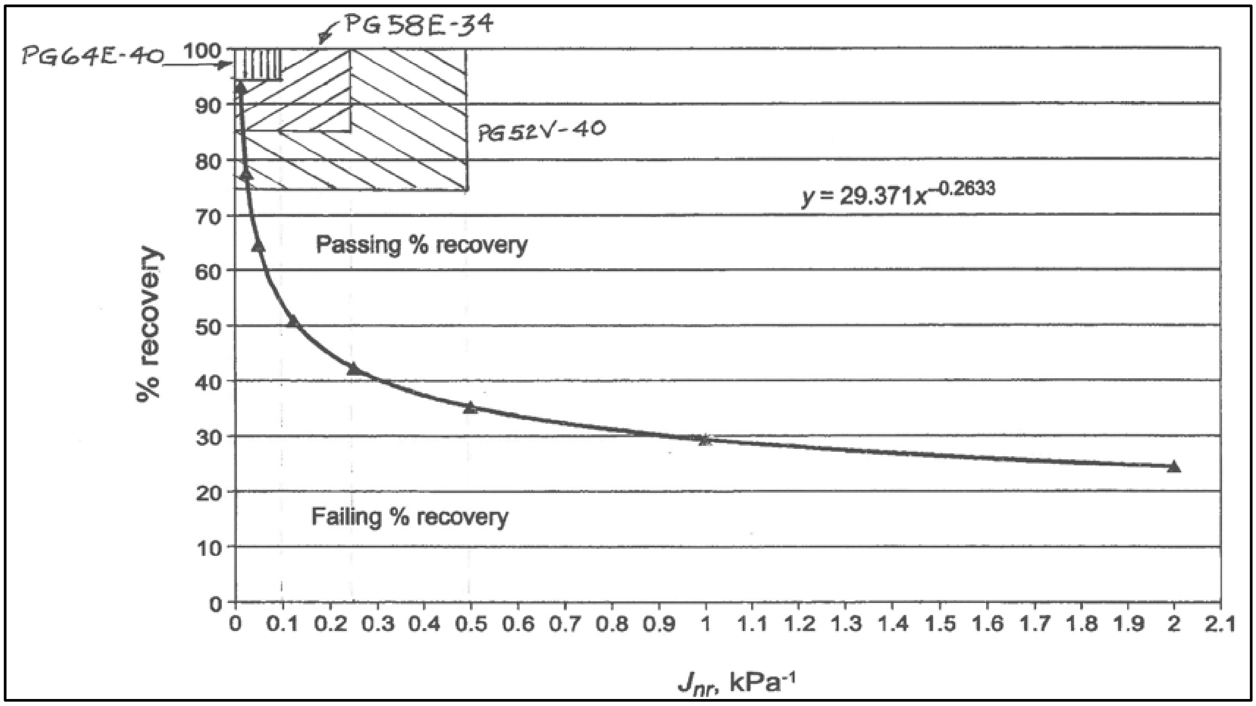


Figure 8 - Zones of Alaska DOT&PF Binders on MSCR Specifications (DOT&PF)

There has been significant research performed into the relationship of the J_{nr} value and plastic deformation of pavements. The J_{nr} value is a measure of non-recoverable creep compliance. A higher J_{nr} value indicates there is more unrecovered shear strain in the binder after being repeatedly strained during the MSCR test. This includes research performed at FHWA's Accelerated Loading Facility (ALF) where pavement was heated to a constant 64°C and a 10,000lb wheel load was applied (Performance Testing for Superpave and Structural Validation FHWA-HRT-11-045).

The following is a current MSCR asphalt binder specification used by Alaska DOT&PF. It is a modified version of AASHTO M 332; specified values for J_{nr} and Percent Recovery were replaced with values adopted by DOT&PF. The PG 64-40E specification was developed using Kraton

HiMA polymer and has a low viscosity with high polymer loading that allows for construction workability.

Table 1 - Alaska's Performance Graded Binder Specification (DOT&PF, 2021)

Performance Grade	Viscosity AASHTO T 316	Multiple Stress Creep Recovery MSCR, AASHTO T 350			Dynamic Shear PAV, AASHTO T 315	Direct Tension AASHTO T 314	Elastic Recovery AASHTO T 301
		$J_{NR3.2}$ kPa^{-1}	J_{NR} Diff	% $Recovery_{3.2}$	G*Sinδ, kPa		
AASHTO M332 Performance-Graded Asphalt Binder Using MSCR Test Specification							
PG 52-40E	3 Pa•s max.@ 135° C	0.50 max.	~~(75% Max),~~ Delete	(add) 75 min.	6000 kPa @ test temp.	Delete	**None**
PG 58-34E	3 Pa•s max.@ 135° C	~~(0.5)~~ 0.25 max.	~~(75% Max),~~ Delete	(add) 85 min.	6000 kPa @ test temp.	Delete	**None**
PG 64-40E	~~(3)~~ 1 Pa•s max.@ 135°C	~~(0.5)~~ 0.10 max.	~~(75% Max),~~ Delete	(add) 95 min.	~~6000 kPa @ test temp. (16)~~ 5000 max. @ 4°C	Delete	None

3.1.2 Historic Prall Testing and Asphalt Binder Properties

During the late 2000's, low temperature performance of HMA was evaluated relative to studded tire wear of pavement. Figure 9 illustrates the action of a stud impacting pavement (source unknown). There is a scratch and an impact component of the stud contacting the pavement. Studs wear both the binder matrix and the aggregate, and greater damage is caused at higher speeds.

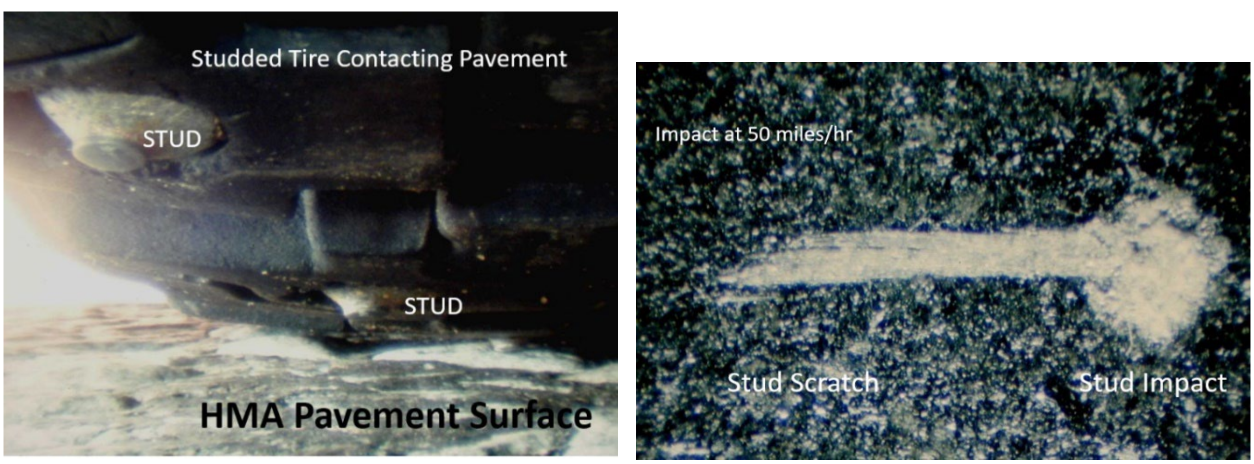


Figure 9 - Studded Tire Impact (Jacobson, 1999)

As studded tire wear occurred, it was noted that the coarse aggregate in the mix was being exposed by studs eroding the fine aggregate-binder matrix from around the top and upper sides of the larger aggregate. This was more predominate in the SMA mixes that had higher percentages of large aggregate than the uniformly graded finer mixes. All of the mixes were composed of crushed aggregate passing the ¾ inch sieve. This caused the asphalt binder to be evaluated for the role it plays to resist studded tire wear by the low PG temperature.

Thermal cracking and studded tire wear occur at low temperatures. Studded tires are permitted in the winter months in Alaska when the asphalt binder has low to no flexibility, resiliency, or elasticity; during this period the binder can shatter upon stud impact. The low asphalt binder PG temperature is selected to minimize thermal cracking. To see if binder grades impact studded tire performance, mixes made with various low PG binders were Prall tested to simulate studded tire wear.

Figure 10 shows Prall test results of the same mix made with differing asphalt binders and with varied asphalt contents. Typical mixes are designed at 4% voids total mix (VTM) but as the asphalt content is varied from the design VTM the Prall test results also vary. The Prall results on the graph are labeled with the binder provider, performance grade, binder content and Prall loss.

As the low PG temperature is lowered, the loss of material during Prall testing decreases. Also, it is evident that HMA with increased binder (lowering the VTM) will have lower Prall values, which should provide improved studded tire wear. This implies that HMA will have less studded tire wear loss as well as less thermal cracking at low PG temperatures and higher binder contents (with lower VTM). Based off of this research projects have been constructed with PG 64-40E binder and will be monitored for studded tire wear performance using pavement management data.

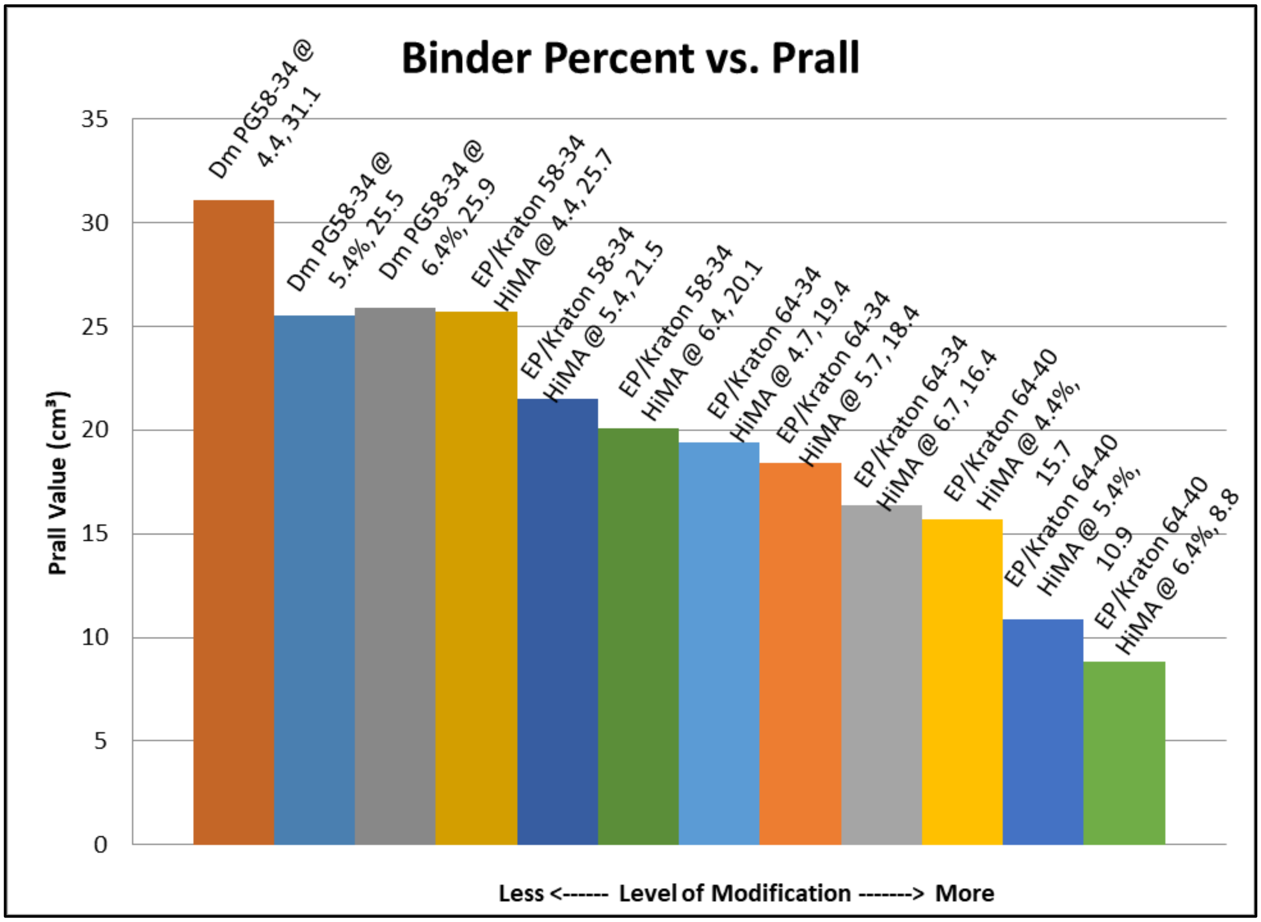


Figure 10 - Prall Test vs. Binder Modification (DOT&PF)

The trend in Figure 11 indicates that more asphalt in the mix will lower the VTM and the Prall loss. Airport mixes are now specified to be designed at 3.5 VTM; however, as the VTM is lowered, there is a potential to induce instability of the mix, so mixes should be rut tested for plastic deformation. Rut testing (APA) is being done for all Superpave mixes for urban highways and airports servicing jet aircraft. Hamburg testing is being evaluated.

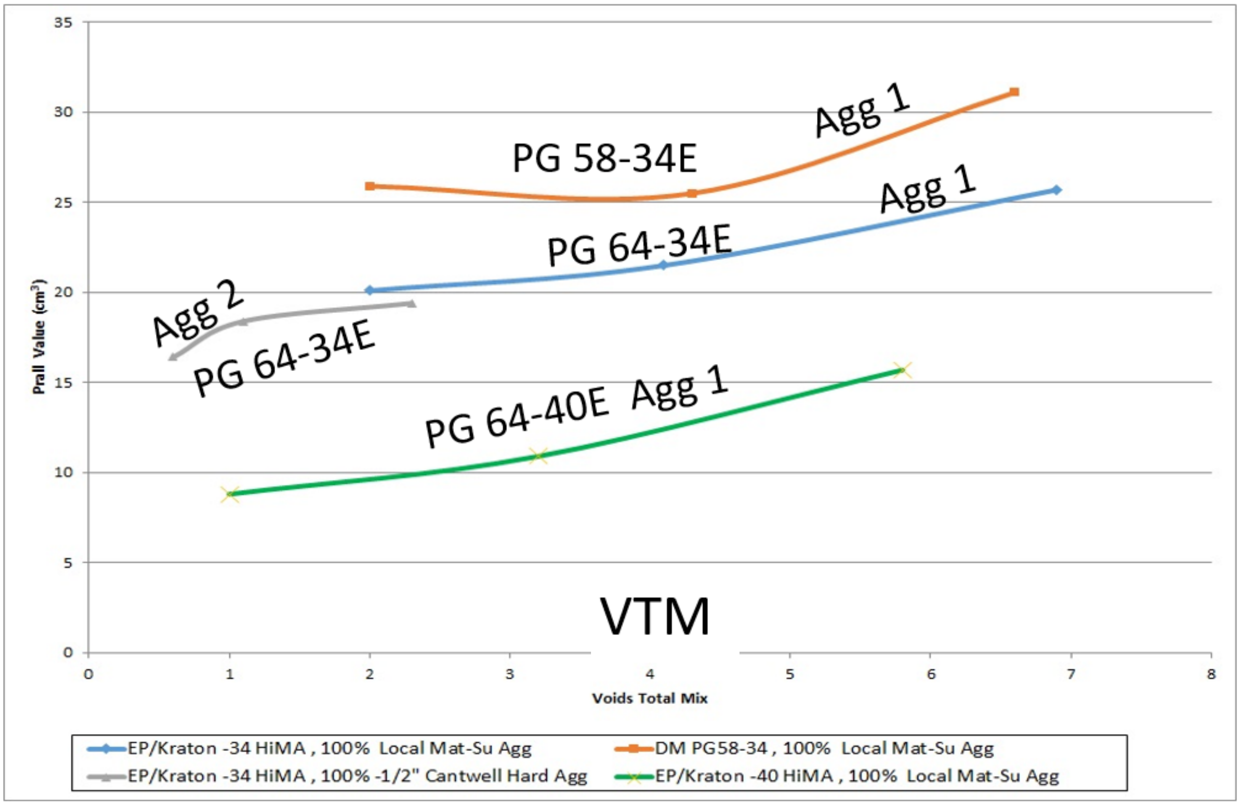


Figure 11 - Prall Test vs VTM (DOT&PF)

In an effort to understand asphalt resiliency at low temperatures, different grades of asphalt binder were tested in the Dynamic Shear Rheometer (DSR) to determine the intermediate binder stiffness at various temperatures. The lower the temperature achieving the specification indicates the binder is providing resiliency at that low temperature and may therefore resist shattering from stud impact if freezing. The fact that the Kraton Modified -40 HiMA binder shown on the graph below achieved passing results at -2°C indicates binder resiliency at low temperatures to a higher degree than the other binders tested. It is also the binder that achieved the best Prall result in Figure 10 and Figure 11. This helped focus on different low temperature binder specifications.

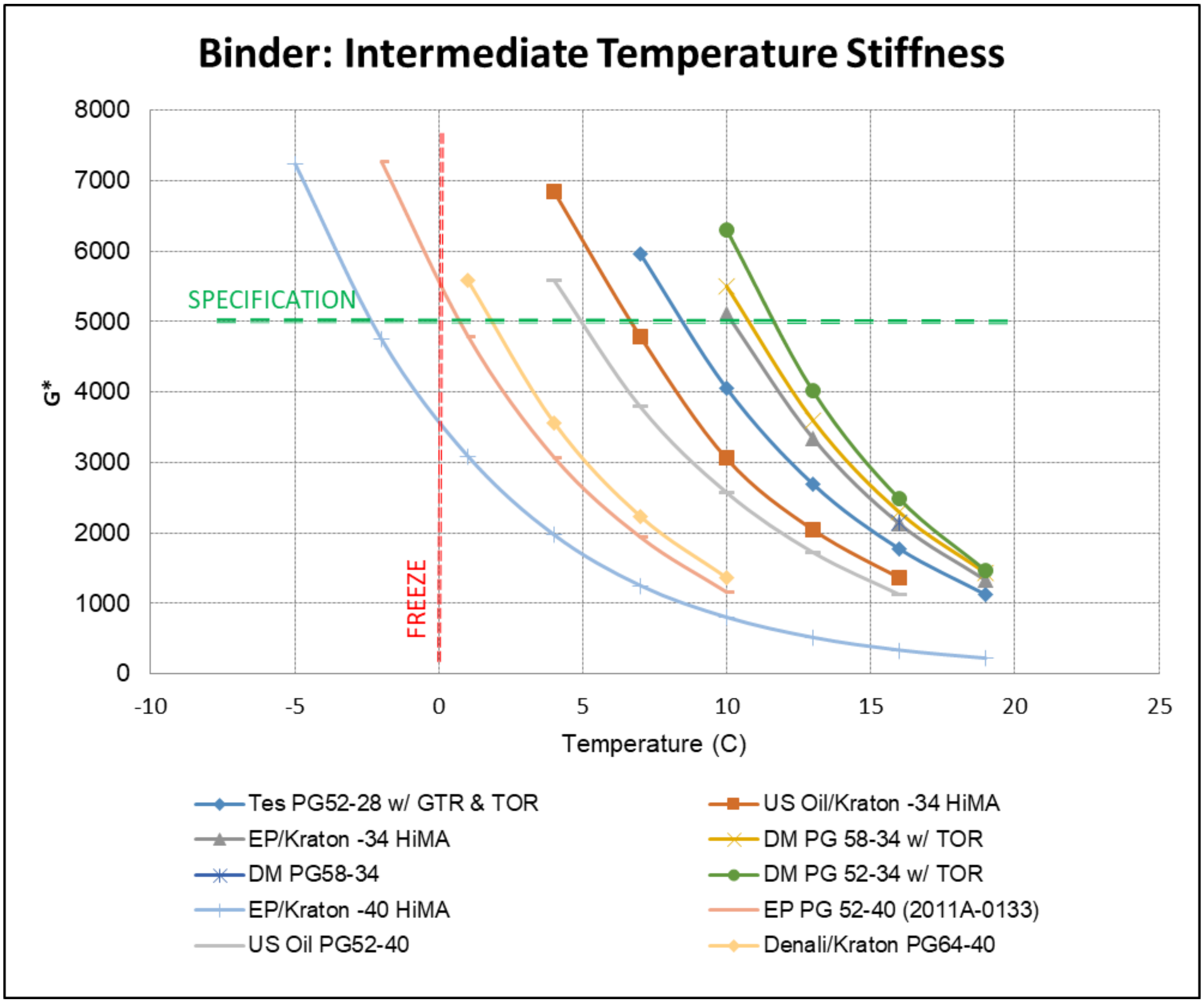


Figure 12 - Binders and Intermediate Temperature Stiffness (DOT&PF)

This PG 64-40E binder balances high polymer loading with low temperature properties allowing it to resist unrecoverable strain while providing for low temperature resiliency. This can be seen in the figure above. This binder is now widely used throughout Central Region on roads with high rates of rutting from studded tire abrasion.

The extreme MSCR specifications for the PG 64-40E binder are to force a high polymer loading, greater than 7% SBS polymer, to achieve a continuous polymer state. Meaning, the polymer swells to a point the binder is dispersed within the polymer, instead of the polymer being dispersed within the binder as occurs at lower polymer contents.

These properties are to allow the mix to withstand plastic deformation in the summer while remaining resilient in winter conditions where the fine aggregate and binder matrix resist shattering from stud impact.

3.1.3 Hot Mix Asphalt – Plastic Deformation, Studded Tire Wear and Prall Testing

In the mid 1990's, Alaska DOT&PF was trying to better understand the cause of rutting in its pavements, especially in the urban Anchorage area. On the Glenn Highway between Anchorage and the Mat-Su Valley, measurements were taken regularly at marked locations. The rut depths were graphed, as seen in Figure 13.

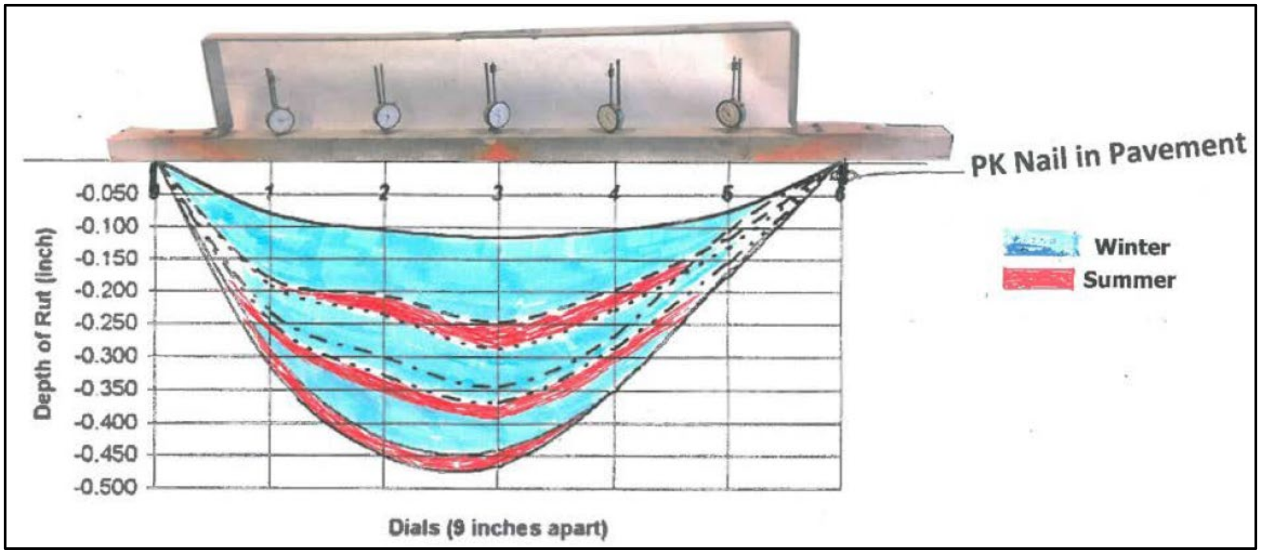


Figure 13 - Rut Measurements in 1990's (DOT&PF)

Studded tire wear was causing rutting in the winter (as seen above in blue) and plastic deformation was occurring in the summer (as seen in red). While plastic deformation was not the primary contributor to rutting in the graph above, in areas of Anchorage with significant static loading of vehicles (intersections), the deformation rutting was more extreme.

In 1995 an Asphalt Pavement Analyzer (APA) was purchased to give insight into the observed plastic deformation and ways to resist it. At that time PMA was introduced and three HMA samples were made and tested in the APA.

The APA was first used to evaluate the cause of plastic deformation as seen in Figure 14. A design gradation was mixed with unmodified AC-5 (Mix 3) and compared to a mix made with 2.5% SBS polymer modified AC-5 (Mix 1), while maintaining the same aggregate gradation. The aggregate gradation was then changed from the design gradation while maintaining the 2.5% SBS polymer (Mix 2). The results indicated that the SBS polymer modification of the asphalt binder drastically reduced the plastic deformation potential, but the use of a proper design gradation is still critical as seen with the deep rut formation in the Mix 2 sample.

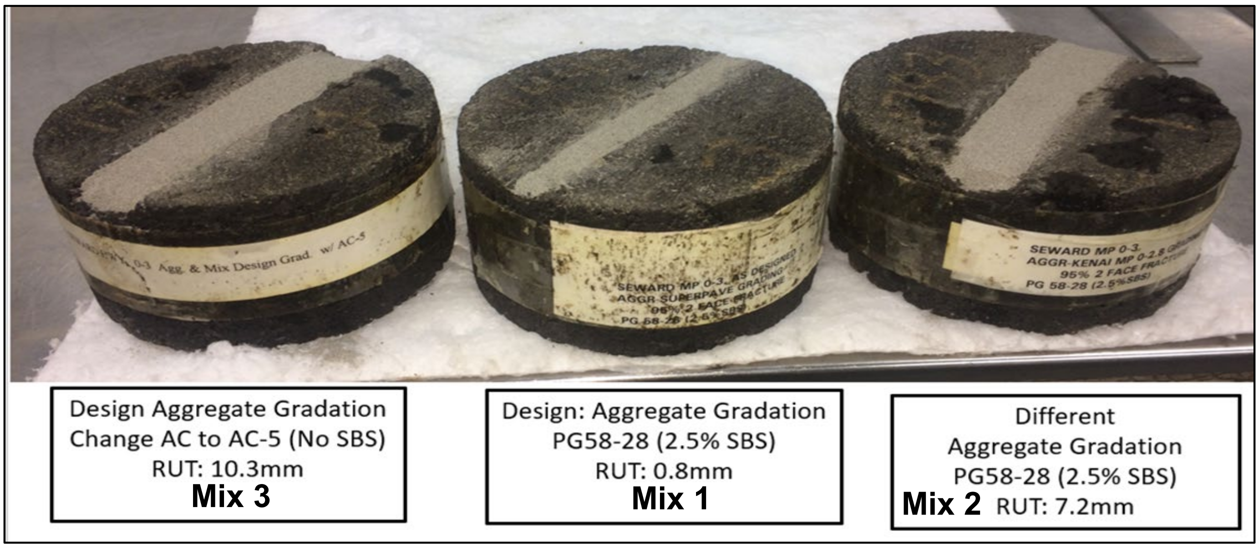


Figure 14 - Initial APA Testing (DOT&PF, 1995)

Aggregate Hardness and Studded Tire Wear

With the significant loss of pavement life due to studded tire wear in the early 2000's Alaska adopted the use of Scandinavian technology to evaluate aggregate "hardness" with the use of the Nordic Abrasion test. The Alaska DOT&PF purchased Nordic Test machines for each region in the early 2000's. This test (ATM 412) is a wet abrasion test in a ball mill where aggregate is tumbled with steel balls and water. Figure 15 shows the Nordic Abrasion Equipment used in Alaska.

Figure 15 - Nordic Abrasion Equipment (DOT&PF, 2003)

The weight of aggregate loss after testing is measured. Aggregate with a Nordic Abrasion test weight loss ≤ 8% is specified to be used in pavements for high volume roads that have high studded tire wear rates.

Currently Alaska DOT&PF has mandated that “hard” aggregate be used on roads when the average annual daily traffic (AADT) per lane is greater than 5,000. This aggregate has to be imported to the Anchorage and Juneau areas. This impacts the cost of HMA due to the added time and coordination required to import it.

While the Nordic Abrasion test evaluates the aggregate hardness, another test method was needed to evaluate the studded tire resistance of the mix, including both the aggregate and the binder.

There were two Scandinavian rut tests developed for HMA to simulate studded tire wear. The SRK test consists of 3 studded tires rotating around a HMA core. The second is the Prall test. Samples were sent by DOT&PF in the early 2000’s to Europe for SRK and Prall testing. The SRK samples could not be correlated with field measured studded wear rates while Prall test samples were. Based on those results the Prall test equipment was purchased by Alaska DOT&PF and adopted as Alaska Test Method 420.

In the following graph, hard aggregate (Nordic Abrasion value of less than 8) was incrementally substituted for local Matanuska Valley aggregate used in the Anchorage area. This was done to understand both the Prall value improvement from inclusion of hard aggregate and the cost impact on HMA from specifying the use of imported aggregates. The higher the percentage of hard aggregate included in the mix, the greater the quantity of hard aggregate that would need to be imported to Anchorage.

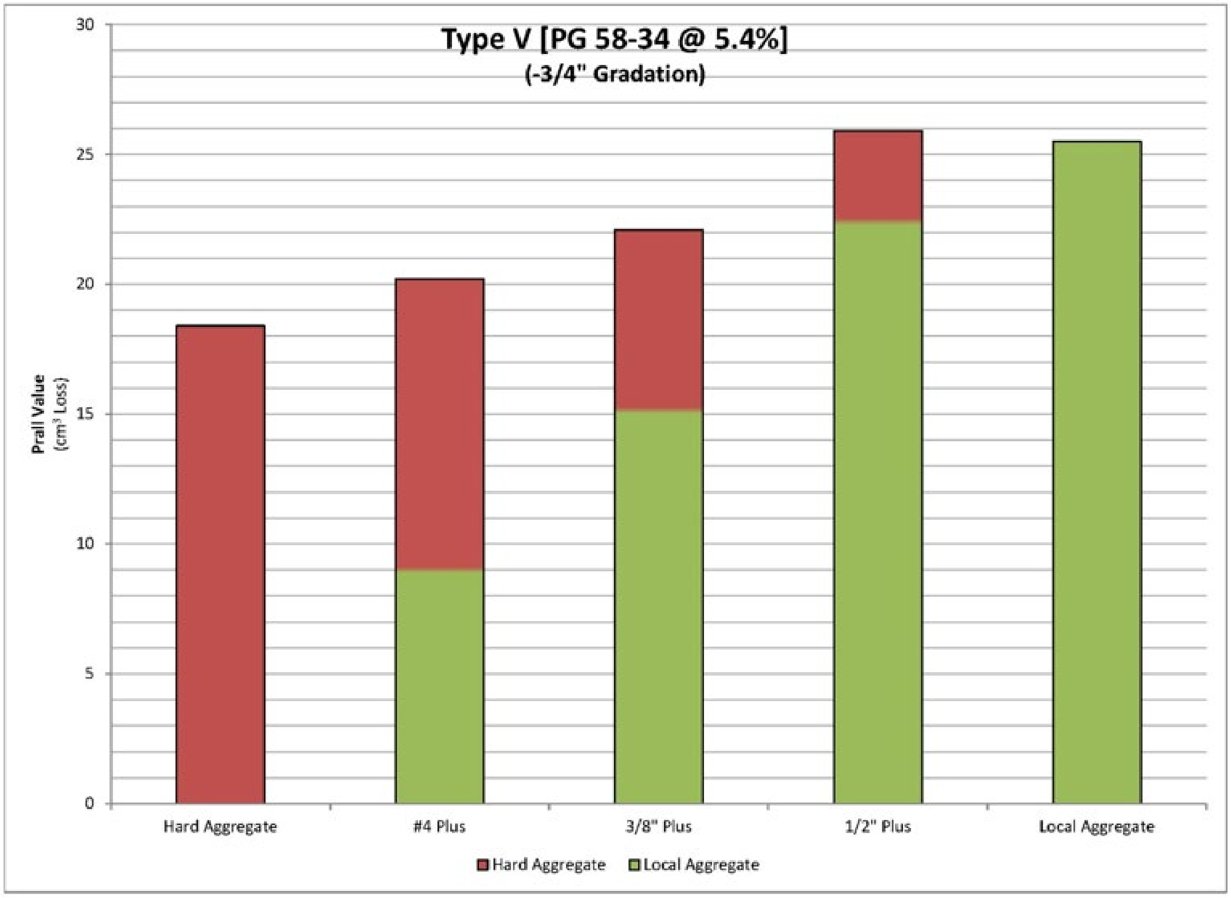


Figure 16 - Influence of Hard Aggregate on Prall Abrasion (DOT&PF)

The aggregate gradation and asphalt content remained the same in this test. The Prall loss decreased with greater percentages of hard aggregate substitution. Each mix was made with the same aggregate gradation, but with varied proportions of hard aggregate. The red indicates the portion of hard aggregate in the HMA. The aggregate in the mix all passed the 3/4" sieve. This testing was done to help understand what size aggregate was most influential on studded tire wear as determined by the lab Prall test.

This testing helped to establish that hard aggregate be classified as the coarse aggregate (3/8" plus) portion of the gradation. The Hard Aggregate Usage Policy was established in 2013 requiring the use of hard aggregate on roads greater than 5,000 AADT per lane.

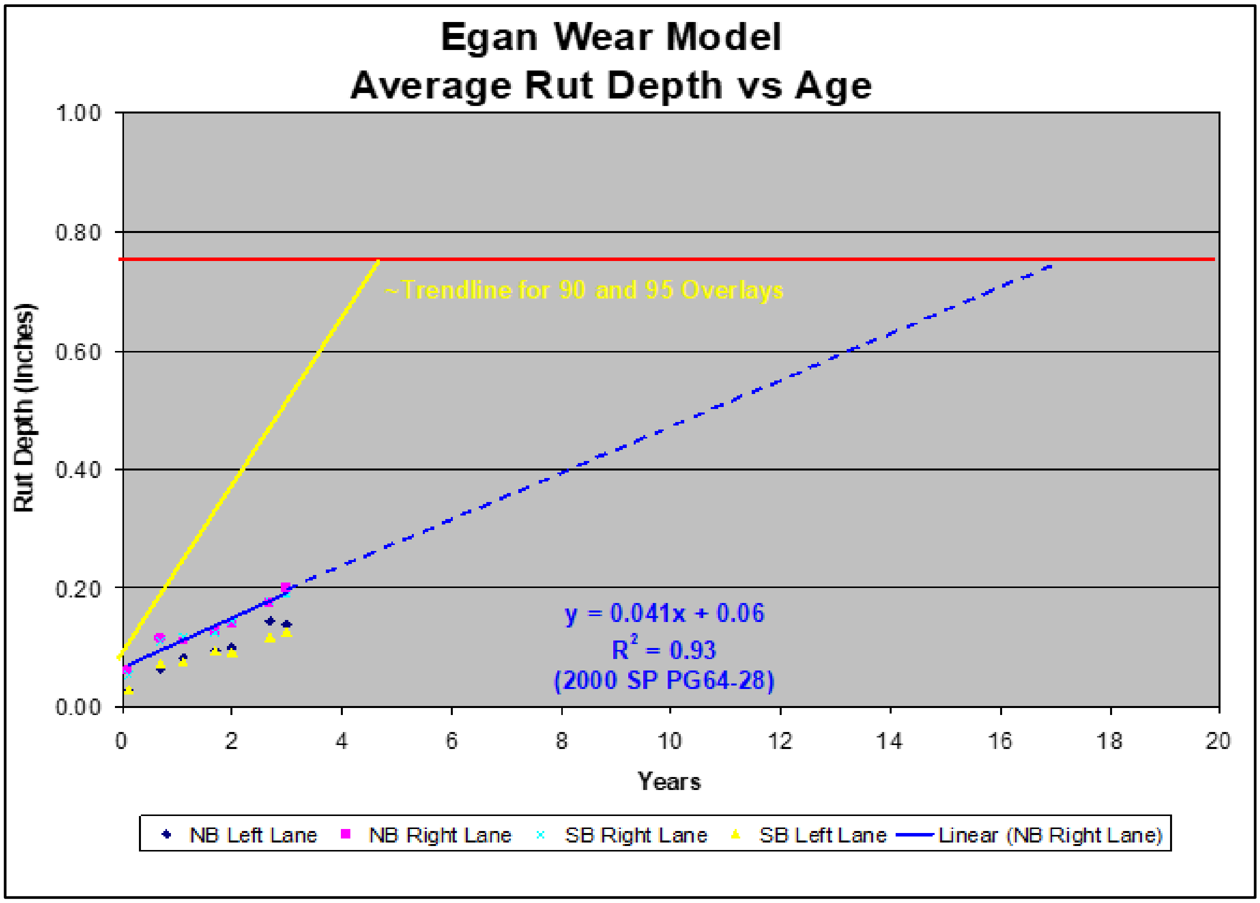


Figure 17 - Egan Drive Hard Aggregate Use (DOT&PF)

The first research and use of hard aggregate in Alaska was performed by Bruce Brunette, South Coast Regional Materials Engineer, in 2000. The Southcoast Region of Alaska has relatively soft aggregates that are prone to studded tire abrasion and Bruce required the use of imported aggregate meeting Nordic Abrasion requirements. The graph above demonstrates the impact hard aggregates can have on rut rates. Note that part of the improvement in this situation may have also been from the use of PMA that was incorporated into the binder.

3.2 Prall Test Analysis

The Prall test is used to simulate the effect of studded tire wear on pavements. The test is run in accordance with ATM 420, Abrasion of HMA by the Prall Test Method, and operates as indicated in the figure from the 2020 Alaska Test Methods Manual below.

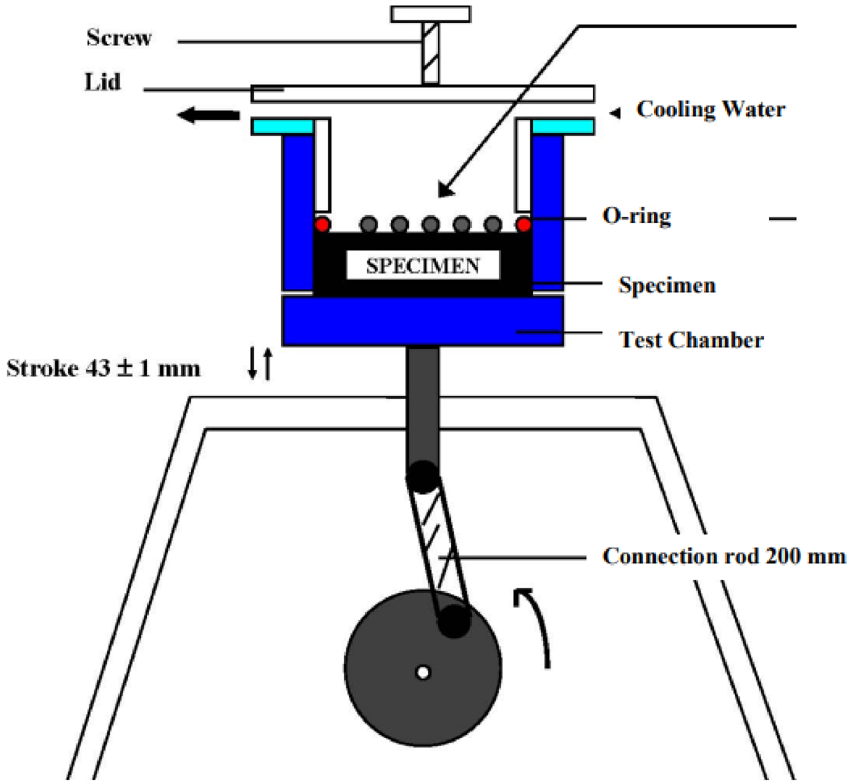


Figure 18 - Prall Test Equipment (DOT&PF)

Tests were run on field cores collected during FWD testing the fall of 2020 (Table 2) and on samples prepared in the lab using mix from ongoing construction projects (Table 3). Where indicated with an H (Table 2 - Prall Tests on Field Samples), hard aggregate was used with a Nordic Abrasion value of less than 8 and, where indicated, RAP had been incorporated into the mix.

Table 2 - Prall Tests on Field Samples

Project	Mix Design	Binder	Hard	RAP	VTM (%)	Prall Abrasion (cm³)	Lab/Field
Funny River Rd MP 0-1	Type IIA	52-40V			2.9	14.7	Field
Funny River Rd MP 0-1	Type IIA	52-40V			4.1	21.4	Field
Glenn Highway Glenn/Parks	Type VH	64-40E	H		3.0	7.7	Field
Glenn Highway Glenn/Parks NB	Type VH	64-40E	H		3.7	8.0	Field
K-Beach Rd MP 19-21	Type VH	52-40V	H		2.6	13.9	Field
K-Beach Rd MP 19-21	Type VH	52-40V	H		3.0	14.4	Field
Minnesota Weaving Lane	Type VH	64-40E	H		3.9	12.1	Field
Minnesota Weaving Lane	Type VH	64-40E	H		5.0	8.7	Field
Parks Highway MP 151-153	Type IIA	58E-34		RAP	4.5	25.4	Field
Parks Highway MP 151-153	Type IIA	58E-34		RAP	4.9	29.2	Field
Seward Hwy Dowling to Tudor	Type V	58-34			2.4	18.8	Field
Seward Hwy Dowling to Tudor	Type V	58-34			4.0	21.5	Field
Seward Hwy Dimond to Dowling	Type VH	58E-34	H		1.6	18.3	Field
Seward Hwy Dimond to Dowling	Type VH	58E-34	H		2.8	22.4	Field

Table 3 - Prall Tests on Lab Sample

Project	Mix Design	Binder	Hard	RAP	VTM (%)	Prall Abrasion (cm³)	Lab/Field
Fireweed IV B P-404	Type IVB	64-40E			1.6	10.5	Lab
Fireweed IV B P-404	Type IVB	64-40E			1.6	11.7	Lab
Fireweed IV B P-404	Type IVB	64-40E			3.6	14.8	Lab
Fireweed IV B P-404	Type IVB	64-40E			3.6	17.3	Lab
Fireweed IV B P-404	Type IVB	64-40E			7.1	21.7	Lab
Fireweed IV B P-404	Type IVB	64-40E			7.5	22.7	Lab
Glenn-Springer	Type IIAH	64-40E	H	RAP	2.5	17.9	Lab
Glenn-Springer	Type IIAH	64-40E	H	RAP	3.3	19.3	Lab
Seward Highway MP 75-90	Type IIA	52-40V		RAP	4.2	30.6	Lab
Seward Highway MP 75-90	Type IIA	52-40V		RAP	4.6	30.0	Lab

In Table 2 and Table 3, it should be noted that there is a general trend of increasing Prall Abrasion values with higher VTM. In the figure below, the Prall data from both the field cores and the lab produced samples was graphed against VTM. Note that the graph is based off of binder types or RAP inclusion and not based off of mix type or aggregate hardness. In each case, there is a trend of increasing Prall Abrasion values by increasing VTM.

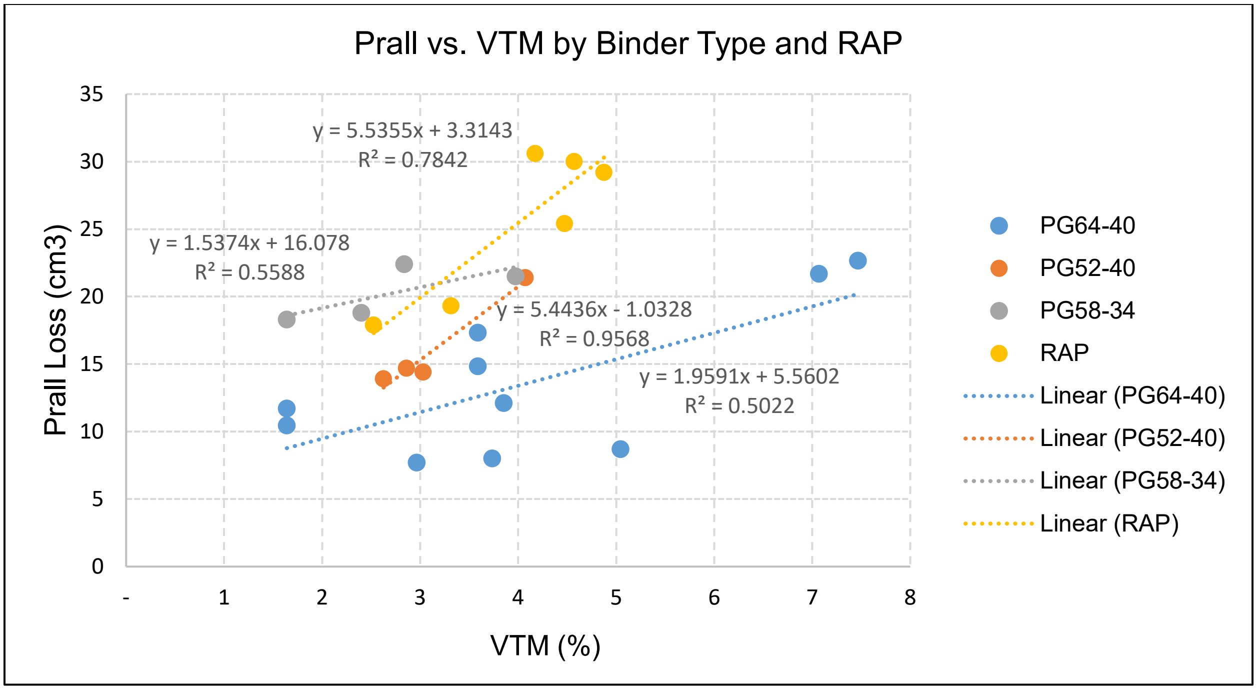


Figure 19 - Prall Loss vs. VTM (DOT&PF, 2021)

Another way of viewing this data is by a comparison of the low end of the binder, mix type, binder content and inclusion of RAP against Prall abrasion values. Figure 20, below, indicates that lowering the bottom end of the PG temperature improves Prall performance, as does raising binder contents and incorporating hard aggregates. It can again be observed that including RAP in the mix leads to worse abrasion values.

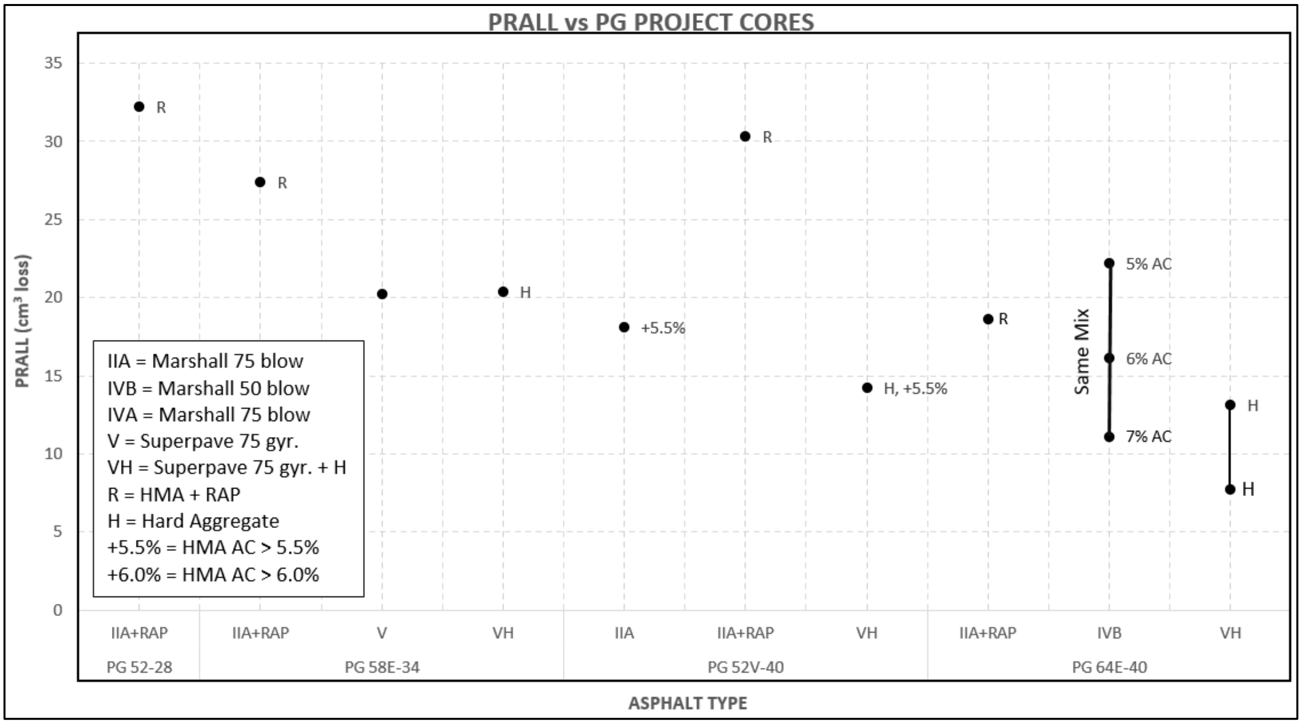


Figure 20 - Prall Loss by Mix and Performance Grade (DOT&PF, 2021)

The Prall samples of Fireweed Type IV, shown on Table 3 and graphed on Figure 20 as the Type IVB with binder contents of 5%, 6% and 7%. Pictures of the Prall samples after testing are in Figure 21 and are labeled with the binder content used in the samples. As binder contents were raised in the samples the VTM was lowered. It can be seen that as the binder content is raised from 5% to 7% more binder remains on the surface and the resulting sample is darker. Aggregate is supported by binder in resisting the abrasion from the test.

Figure 21 - Prall Results on Type IVB PG 64-40E (DOT&PF, 2021)

The data indicates the inclusion of RAP in the mix provides for worse Prall results. Results are improved based on lowering the bottom end of the PG grade of the binder as has been indicated by previous research.

3.3 Pavement Management Data Analysis

3.3.1 Background

Alaska DOT&PF contracted collection of rut depth and International Roughness Index (IRI) data using a Dynatest Mark III Road Surface Profiler (RSP) from 2002 to 2013. DOT&PF then transitioned to requiring the use of a LCMS for cracking data collection (longitudinal, transverse, pattern) in addition to the rut and IRI data.

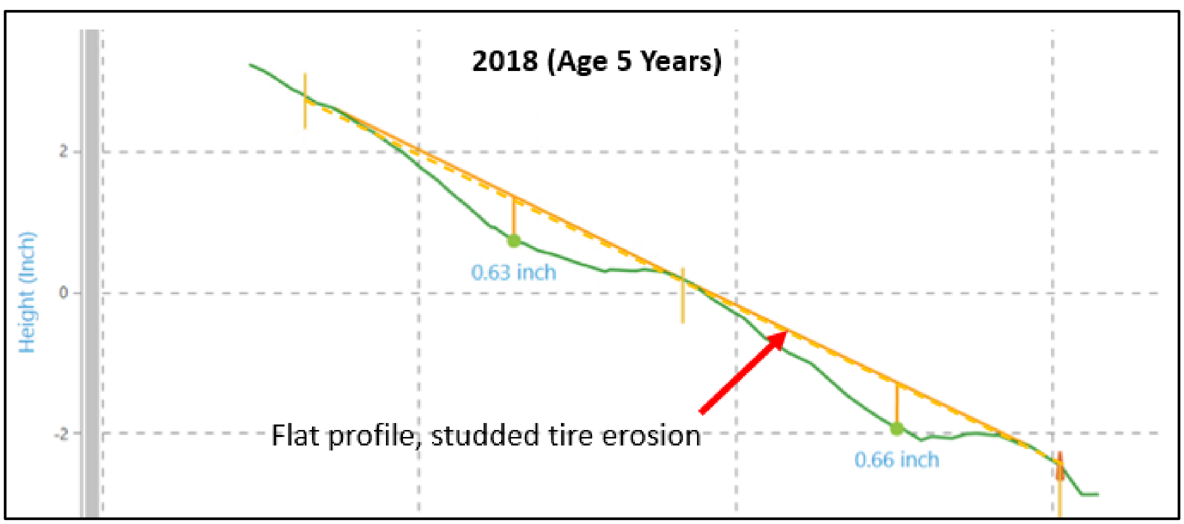
Data collection from the 5-point laser system of the Dynatest Mark III RSP was highly dependent on the position of the vehicle within the road lane and the position of the laser over the rut. If the driver did not position the vehicle and lasers correctly within the lane incorrect rut depths were collected from the lasers being positioned outside of the deepest portion of the rut. LCMS uses thousands of transverse points to build a profile of the road lane, which are used to calculate rut depth. This greatly improved reliability and repeatability.

3.3.2 Methods of Rutting

There are two primary methods of rutting – plastic deformation (load-based deformation) and studded tire wear. Rutting from deformation typically occurs during hot temperatures. A shear plane develops within the warmer, weaker asphalt layer, which allows aggregate to move or rotate. This creates a high, unrecoverable strain in the binder and the hot mix deforms. Asphalt is essentially squished out from under the wheel paths forming low areas, and it builds up adjacent to the wheel paths creating bumps.

Plastic Deformation

In Figure 22 (below), three profiles from Dimond Boulevard in the westbound, right lane at the Old Seward Highway intersection are shown.



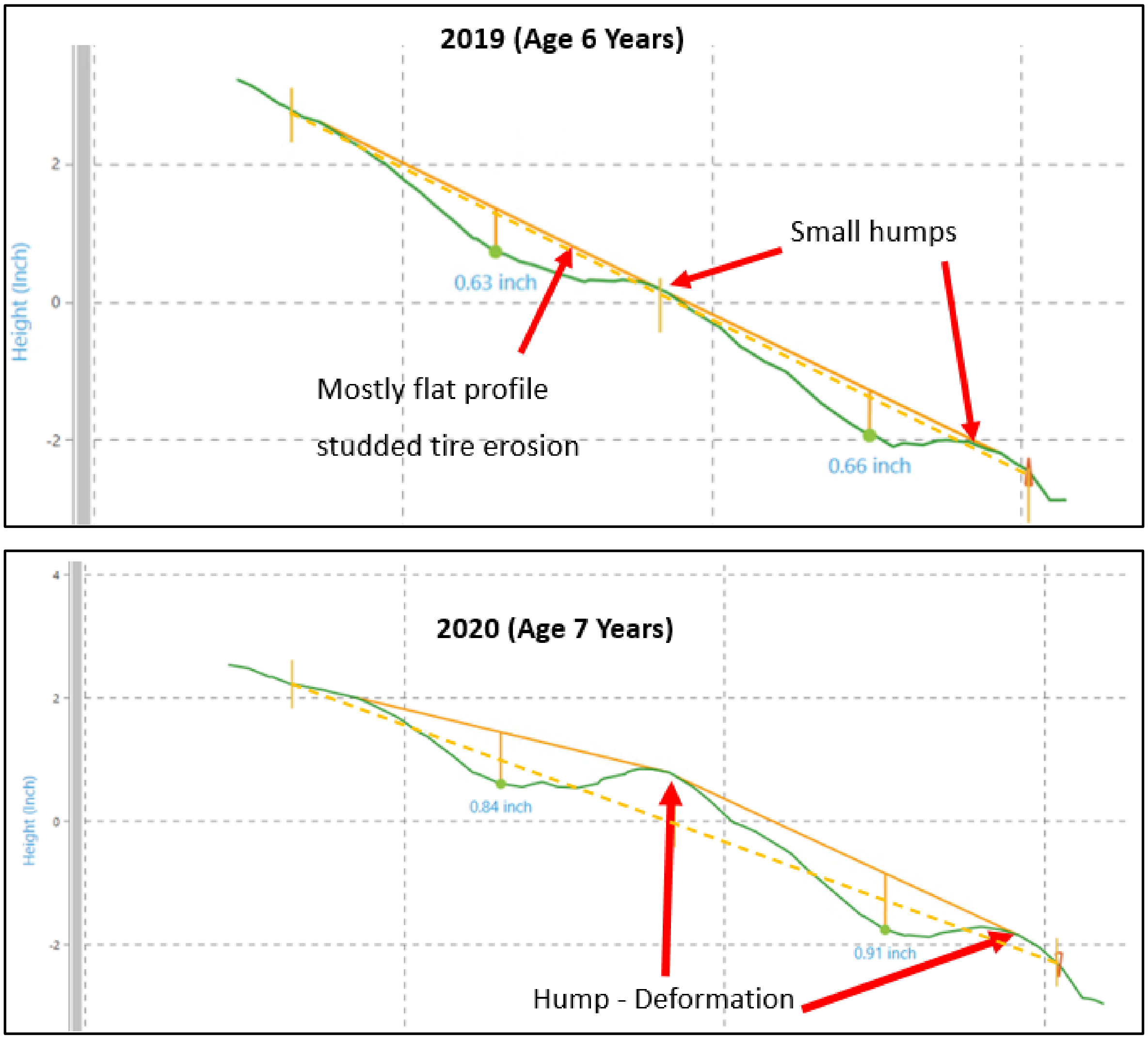


Figure 22 - Dimond Boulevard LCMS Profiles

The profiles were created from LCMS data collected from 2018 to 2020. The pavement was about 5 years old in 2018. The x-axis of the graph is the distance across the lane, with the lane line at the left, and the edge of the lane adjacent to curb and gutter at the right. The lane was constructed with a 2% grade. The green line is the pavement profile, the orange line is a straightedge across the lane while the dashed orange line would be a straightedge connecting the lane lines together. Anything above the dashed orange line indicates plastic deformation.

Cases of significant deformation rutting have not been commonly observed around the Anchorage area since polymer modified asphalts were introduced, and those that have are located at intersections with heavy static loading. The pavement profile shows little to no deformation rutting in 2018; there were no humps as typically form when rutting from plastic deformation occurs. Small humps had formed by 2019 at the center and right of the lane. Larger humps had formed by 2020; the orange straightedge has a break point at a bump in the center of the lane. It should be noted that 2019 had extended periods of record-breaking heat in the Anchorage area, which likely had a significant impact on deformation rutting where these profiles were taken.

Studded Tire Wear

Studded tire abrasion is due to repeated impacts of studs on the pavement surface, which leads to the erosion of both aggregate and binder. It was found that 35% of passenger vehicles use studded tires (Abaza, 2019). Studded tires are allowed between September 15th and May 1st in the Anchorage and Matanuska-Susitna Borough.

Ruts from studded tire abrasion form by initially removing the fine aggregates and binders from around the larger aggregates. Once the finer aggregates and binders are worn through, the studs wear more on the coarse aggregates and after enough of the surrounding matrix has been removed, the coarse aggregates ravel out. An example of this is in Figure 23, below.

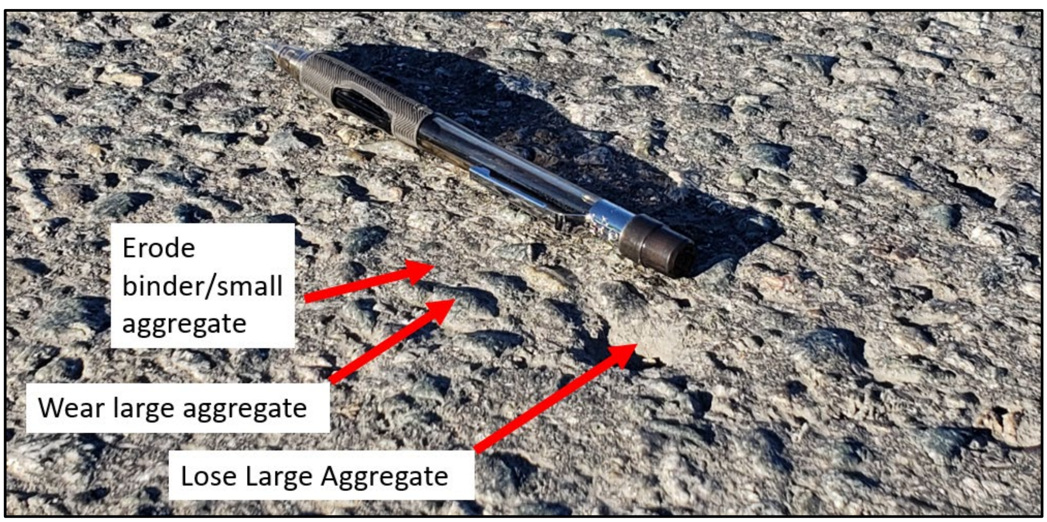


Figure 23 - Rut Wear from Studded Tire Abrasion (DOT&PF, 2020)

3.3.3 Mix and Binder Comparison – Rutting

A number of different mixes and binders have been used throughout Alaska to resist rutting while also providing resistance to thermal cracking. Central Region of Alaska DOT&PF has especially focused on ways to resist studded tire wear as that is the primary failure point on roads within the Anchorage area.

In 2014 a new binder was introduced in an attempt to better resist studded tire abrasion on high volume roads within Central Region while still providing resistance to plastic deformation. This binder is highly polymer modified (estimated at 7% polymer content), meets PG 64-40, is used with a Superpave mix and incorporates hard aggregates with a Nordic Abrasion value of less than 8. This mix, designated Type VH PG 64-40E, has largely replaced Type VH PG 58E-34 in Central Region since 2016.

Pavement Management data compiled from years 2015 – 2021 was analyzed to compare Type VH pavements with PG 64-40E binders to those with PG 58-34E binders, and to compare the rutting resistance of mixes with and without hard aggregates. For all project areas included in this data analysis refer to Appendix E. The data used for analysis can be made available upon request.

The speed of vehicles impacts the damage of studded tire wear on pavements (the greater the speed the greater the damage). Selected roads were broken into two families for analysis: roads with speeds greater than or equal to 55mph, and those at lower speeds.

In order to fairly compare roads with different numbers of lanes and traffic volumes, the graphs display the rut depth against the total number of vehicle passes across the lane. For this, the number of through lanes was determined on the section of road along with the average AADT value from the Alaska DOT&PF Traffic Server and the pavement age since last construction. This was used to calculate the number of vehicles passes on the lane, indicated as Traffic Volume on the graphs below.

Type VH PG 58-34E has a small dataset because hard aggregates and PG 64-40E binders were phased-in near the same period of time. There was only one high speed road available for this dataset. There is a very slight improvement to the performance of PG 64-40E noted at high speeds in Figure 24, although the PG 58-34E dataset is so small it is hard to draw any conclusions.

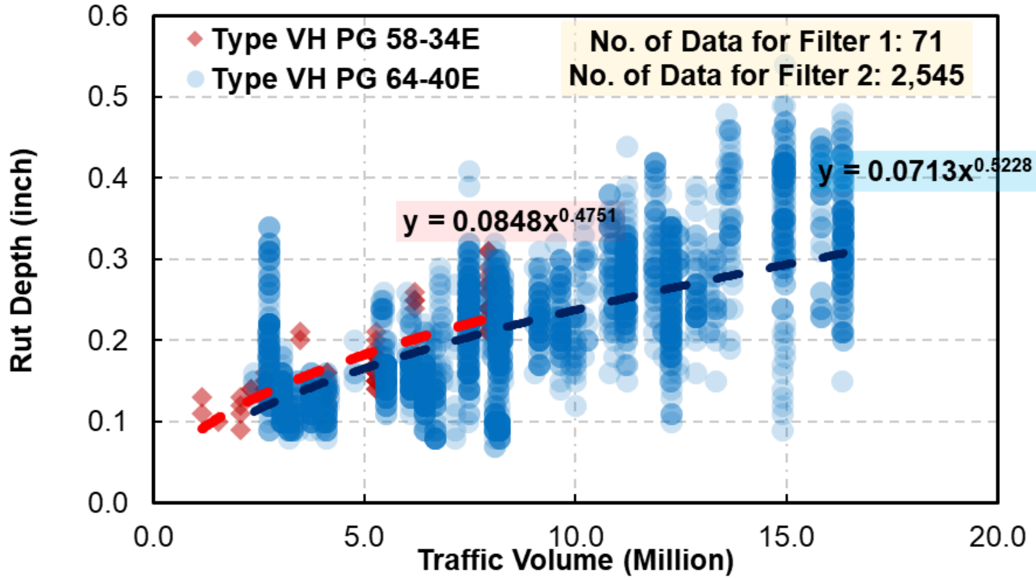


Figure 24 - Type VH PG 58-34E vs. 64-40E / 55 – 65 mph Rut Comparison

There is a larger dataset available for comparison between the two binders on lower speed roads. Figure 25 shows an improvement to the rut rate with PG 64-40E binders.

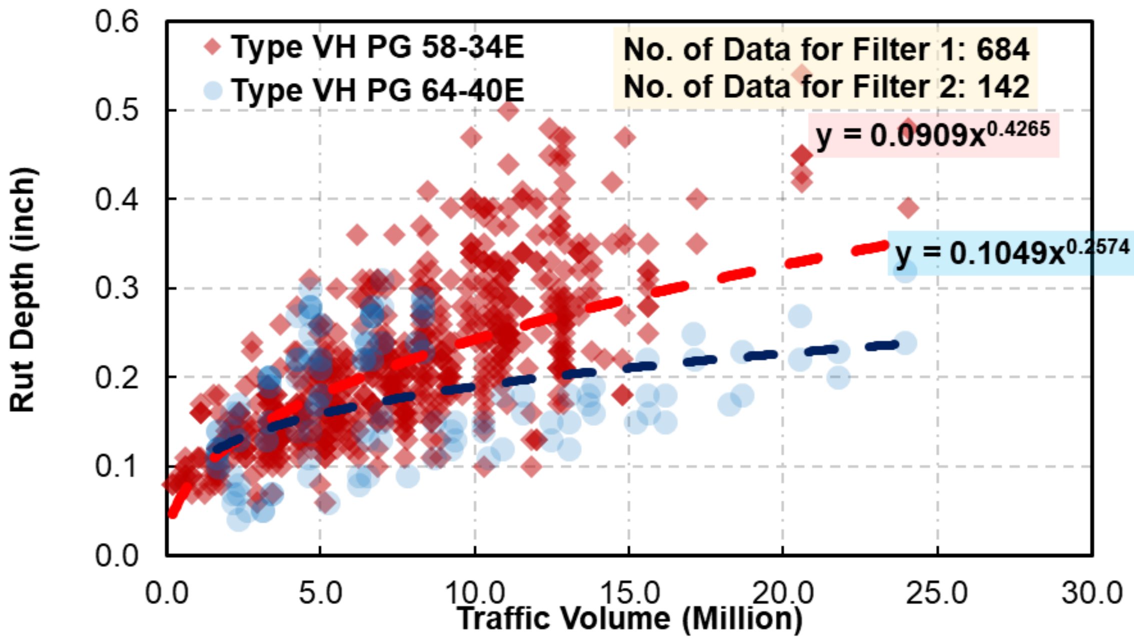


Figure 25 - Type VH PG 58-34E vs. 64-40E / <55 mph Rut Comparison

Figure 26 directly compares performance of high-speed Type V vs. Type VH mixes with PG 58-34E binders. These are both small datasets as most high-speed roads now incorporate PG 64-40E binders, but this comparison is particularly useful as the data is from two sections of the Seward Highway in Anchorage adjacent to one another. The Type V data is from the stretch of the Seward Highway from Dowling to Tudor, while the Type VH data is from the portion of the highway from Dowling to Dimond. It shows an improved performance on the section including hard aggregates.

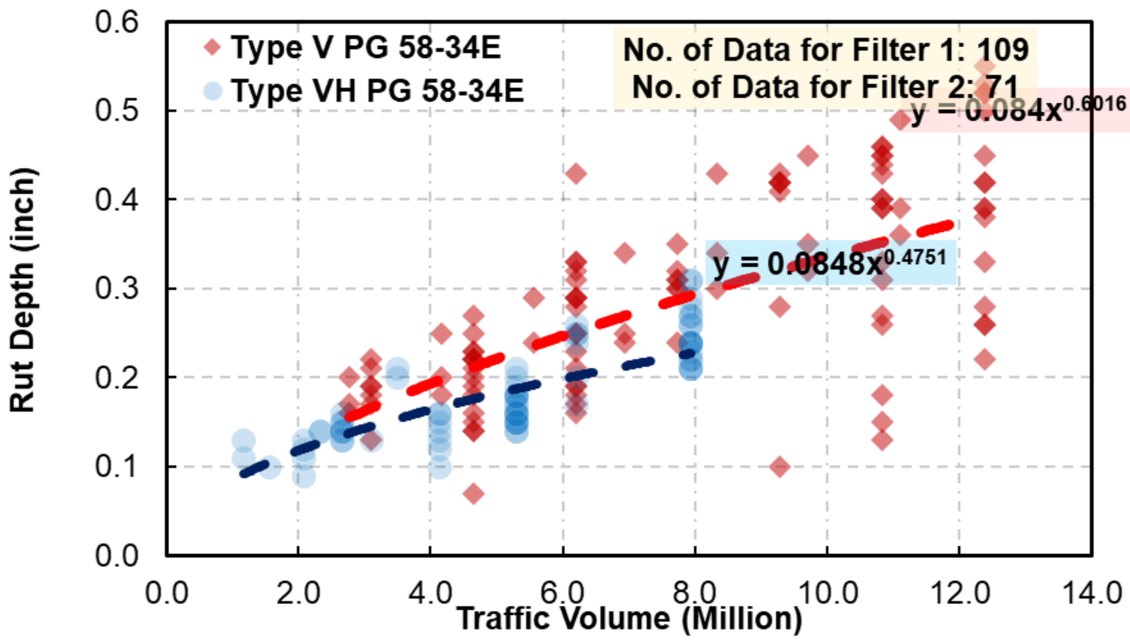


Figure 26 - Type V PG 58-34E vs. 58-34E / 55-65 mph Rut Comparison

Figure 27 compares all Type V to Type VH pavements, regardless of speed or binder types. It shows a distinct improvement of Type VH over Type V.

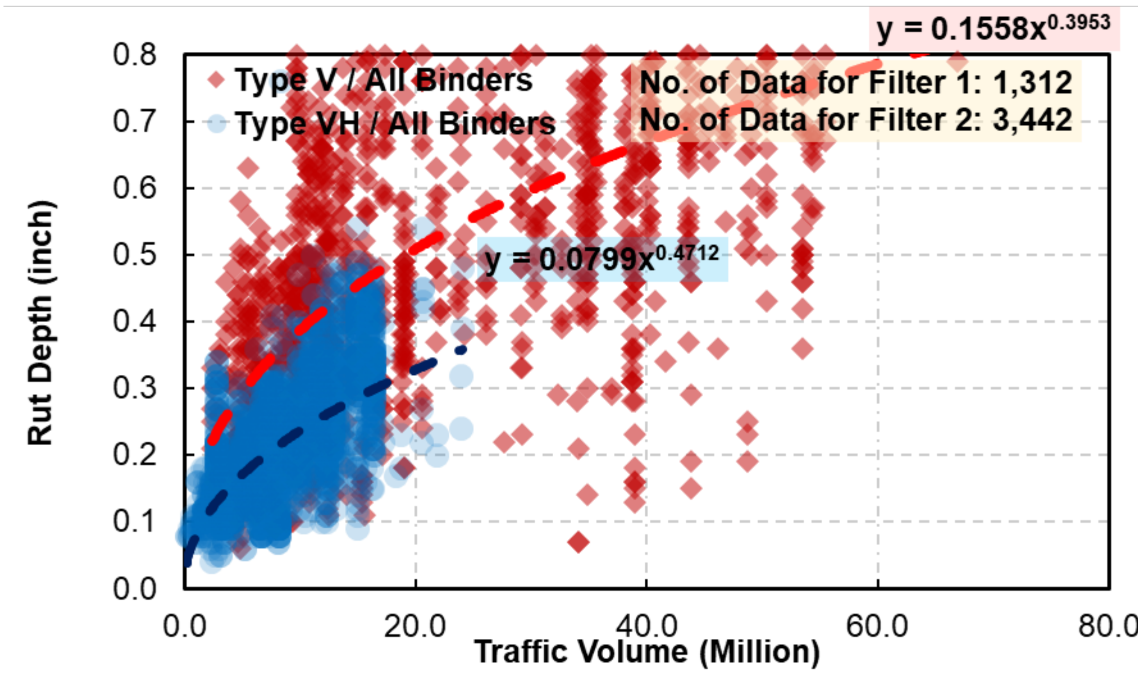


Figure 27 - Type V vs. Type VH / All Speeds Rut Comparison

3.3.4 Mix and Binder Comparison – Cracking

When analyzing transverse cracking it was noted that the data collected from 2015 – 2017 by a different data collection contractor did not trend well with the data collected from 2018 – 2020. That data was excluded from the analysis and made for a more limited dataset for evaluation. It was also noted that most of the projects in Northern Region that incorporated PG52-40 binders were constructed between 2018 and 2020, which did not provide a minimum 3 years of age on those roads as was set as an original criterion for inclusion in the analysis. This greatly reduced the amount of data that was able to be used in this analysis.

For this analysis, thermal cracking was assumed to be all transverse cracking, and sections of roads were not included that contained any large quantities of fatigue based cracking or other pavement failures that would significantly contribute to transverse cracks. The transverse cracking in each tenth mile section of road was equated to be in feet per mile of road and was compared to the age of the pavement. When comparing different binder types against one another, the variables of percent RAP in the mix and region were considered; RAP has the potential to contribute to cracking from adding brittleness to the pavements, and Northern Region has more extreme cold temperatures.

Figure 28 evaluates the performance of PG 52-34E to unmodified PG 52-28 binders in Northern Region, and it does indicate a slightly reduced quantity of thermal cracking.

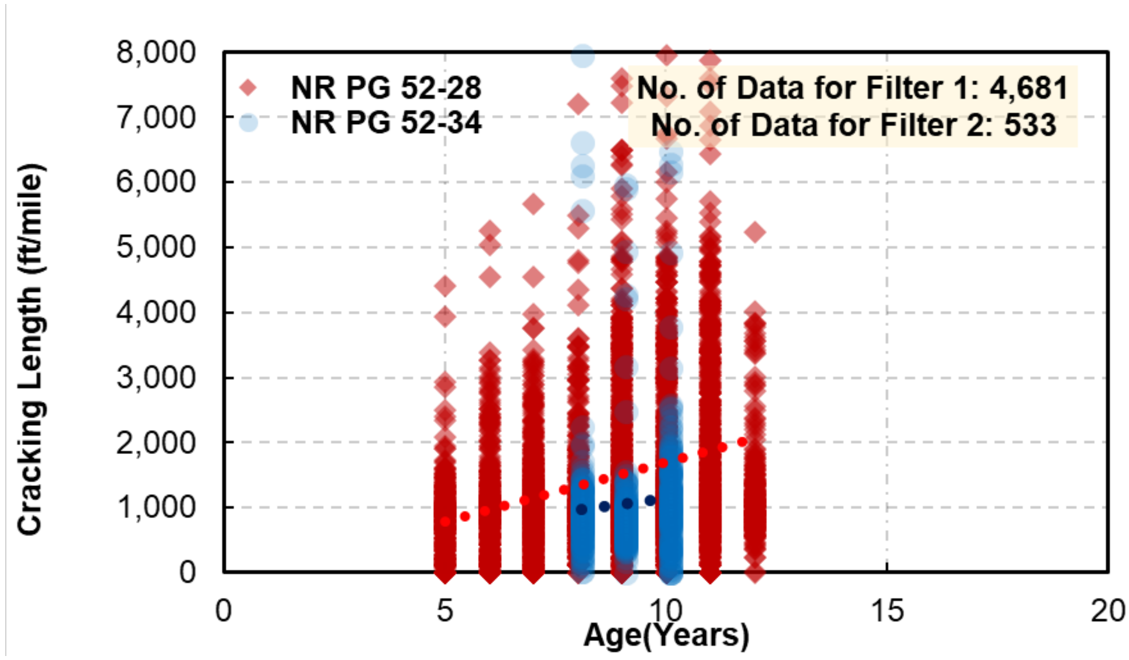


Figure 28 - PG 52-28 vs. 52-34 / NR / No RAP

Figure 29 compares the performance of PG 52-40E with 25% RAP to a PG 52-40E mix with no RAP in Northern Region.

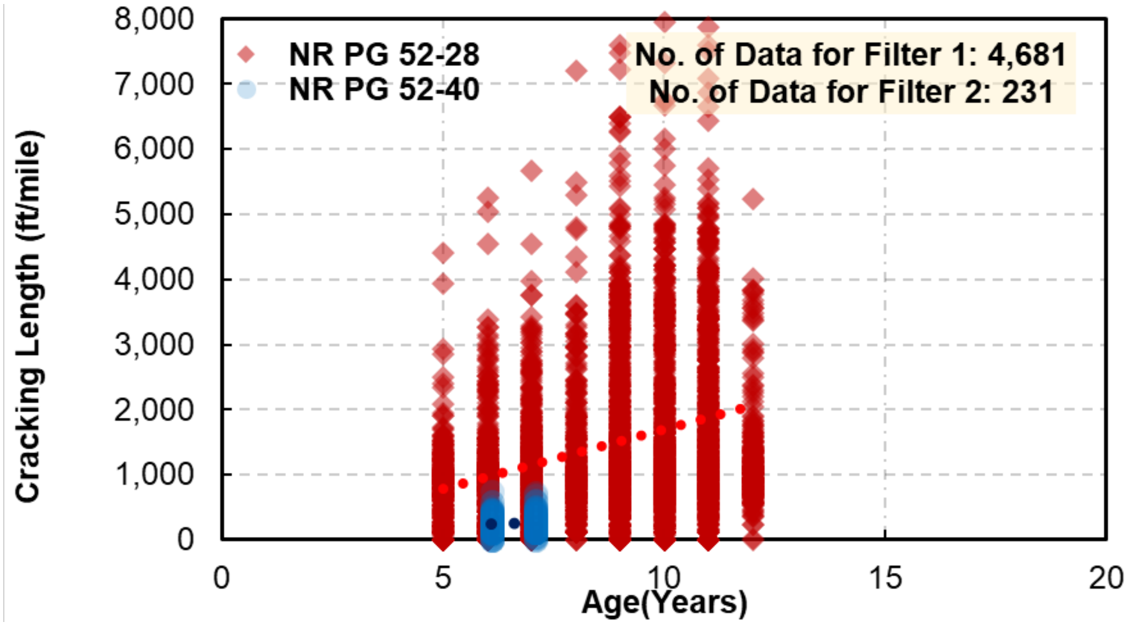


Figure 29 - PG 52-28 vs. 52-40 / NR / No RAP

Figure 29 includes a small dataset for PG 52-40 data in Northern Region, but the available data does indicate a significant improvement in cracking resistance for PG 52-40E over PG 52-28.

In Figure 30 two roads were compared for quantities of thermal cracking, both having a foam stabilized base and using PG 52-40E binders. There is significantly less thermal cracking in the mix with no RAP compared to the mix with 25% RAP.

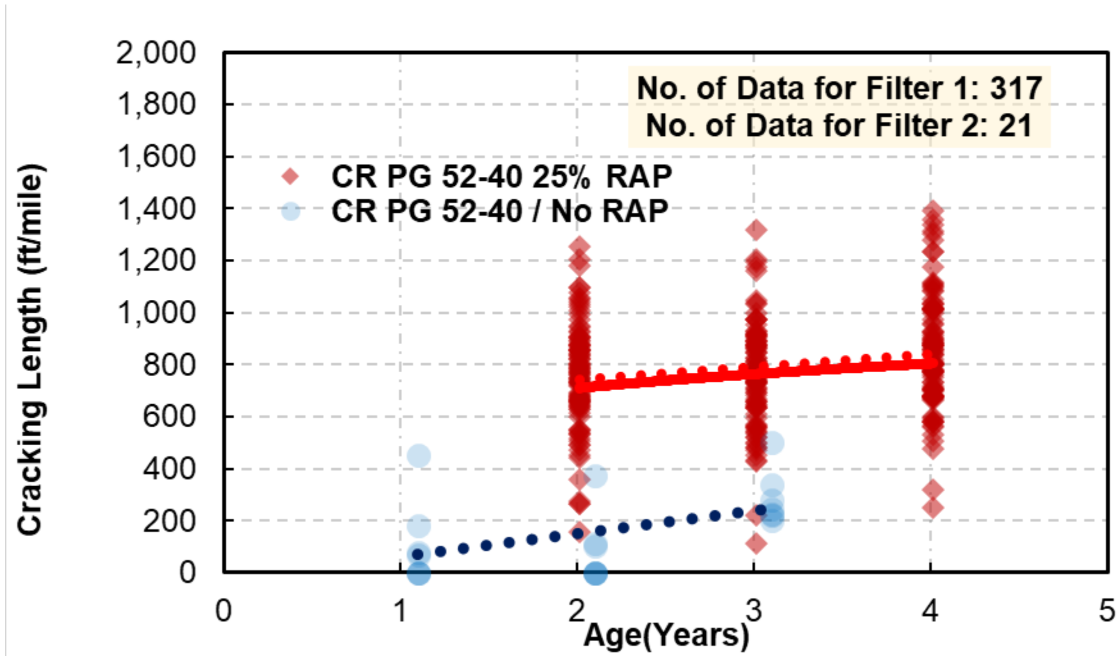


Figure 30 - PG 52-40E 25% RAP vs. 52-40E 0% RAP / CR

Additional analysis was attempted with the cracking data, but it is suspected that in order to properly categorize and group projects for comparison, a more detailed construction history will need to be considered. Ideally, full reconstruction projects would be compared to each other, while the current data is grouping major rehabilitation projects with reconstruction projects.

3.4 Falling Weight Deflectometer Analysis

In September of 2020, Central Region Materials operated the Dynatest HWD CP 15 falling weight deflectometer at 11 different historic project locations. It used a 12” diameter split plate targeting 10,000 lbf for three drops. The third drop of the FWD was used for the back-calculated modulus. The collection took place during the day, but a temperature correction was not applied to the hot mix asphalt. The data is representative of field conditions during collection.

The photo of the HWD equipment, below, was taken during data collection.

Figure 31 - Dynatest HWD CP 15 Equipment (DOT&PF, 2020)

Elmod 6 was used to back-calculate the modulus values using Deflection Basin Fit. The project locations chosen to evaluate hot mix asphalt values were roads that had been reconstructed or rehabilitated with a minimum thickness of 3” HMA and known structural section. A and C Streets were included even though their full structural sections were unknown because they are one of only two locations having full depth HMA with PG64-40E. The rest of the roads with HMA and PG64-40E binder were mill/fill treatments. The Seward Highway between MP 66-70 only has a 2” HMA thickness but was selected for its foamed asphalt stabilized base. The modulus values for the HMA will be ignored as the layer is too thin for back calculation. Cores were taken where FWD analysis was performed to confirm pavement thicknesses, and for Prall and Hamburg testing.

The projects included for analysis are in Table 4 – Projects for FWD Analysis, (next page). For more details on the structural sections and data for back calculations, see Appendix D.

Table 4 - Projects for FWD Analysis

Road	Const. Year	Mix	Binder	Project Number	Layer 1	Layer 2	Layer 3
Glenn/Parks Interchange	2016	Type VH	64-40	58571	4" HMA	3" RAP	36" Select A
Parks Highway MP 52-57	2012	Type IIA	52-34	53469	2" HMA	8" FASB	Subgrade
Parks Highway MP146-150	2013	Type IIA	58-34	54147	4" HMA	8" FASB	36" Select A
K-Beach Sterling to Bridge Access	2018	Type VH	52-40	59778	5" HMA	24" Select A	Subgrade
Parks Highway MP 72-83	2011	Type IIA	52-28	54985	6.5" HMA	24" Select A	Subgrade
Funny River Road at Airport	2017	Type IIA	52-40	53750	3" HMA	9" FASB	12" Select A
A/C Street 3rd to 6th Ave	2014	Type VH	64-40	57835	4"-5" HMA	Subgrade	N/A
Seward Highway MP 50-75	2015	Type IIA	52-28	58080	2" HMA	8" FASB	Subgrade
Seward Highway Dimond to Dowling	2018	Type VH	58-34	CFHWY00162	7" HMA	26" Select A	Subgrade
Seward Highway Dowling to Tudor	2013	Type V	58-34	50816	7.5" HMA	26" Select A	Subgrade

Back calculations were performed on mix placed and compacted in the field. It should be noted that the projects used aggregates with different gradations, the void contents were variable, and the binder contents varied from project to project. Data collection was performed over the course of a few days, and the pavement temperatures varied slightly. All of these variables can impact the back-calculated modulus values.

The default values in the Alaska Flexible Pavement Design Software (AKFPD) are shown in the Default Modulus Value Table below:

Table 5 - AKFPD Default Modulus Values

Layer	Spring Modulus	Summer Modulus	Fall Modulus	Winter Modulus
Asphalt Concrete (Modified Asphalt)	450	400	400	1,200
Asphalt Concrete (Unmodified Asphalt)	350	300	300	1,200
Foamed Asphalt Stabilized Base	110	100	100	400
Select A P200<6%	35	40	40	90

The hot mix asphalt modulus values in Table 6, would be most similar to the Fall Modulus values from Table 5. For this analysis, the PG 64-40E, PG 58-34E and PG 58-34 values will be considered Modified Asphalt, while PG 52-40E, PG 52-34 and PG 52-28 would be Unmodified Asphalt. It should be noted that PG 52-40E is a polymer modified asphalt, but it has a significantly lower polymer content than PG 64-40E and PG 58-34E. For the purpose of the pavement design software, modulus is being considered unmodified.

Table 6 - Hot Mix Asphalt Back-calculated Modulus Values

Location	Project Number	Mix* Type	Binder	Avg Modulus (ksi)
Glenn/Parks Interchange SB	58571	Type VH	PG 64-40E	528
Glenn/Parks Interchange NB	58571	Type VH	PG 64-40E	503
C Street	57835	Type VH	PG 64-40E	398
A Street	57835	Type VH	PG 64-40E	354
Seward Highway Dimond to Dowling NB	CFHWY00162	Type VH	PG 58-34E	634
Seward Highway Dimond to Dowling SB	CFHWY00162	Type VH	PG 58-34E	561
Seward Highway Dowling to Tudor	50816	Type V	PG58-34	653
Parks Highway MP 146-163	54147	Type II-A	PG58-34	425
Funny River Road	53750	Type II-A	PG 52-40E	542
K-Beach Road Bridge Access to Sterling	59778	Type VH	PG 52-40E	336
Parks Highway MP 52-57	53469	Type II-A	PG52-34	372
Parks Highway MP 72-83	54985	Type II-A	PG52-28	546

*Type V is a Superpave mix, while the H indicates hard aggregate is used with a Nordic Abrasion value less than 8
* Type II-A is a Marshall mix with a maximum aggregate size of ¾", NMAS of ½" to 5/8"

Table 7 - Back-calculated Base and Subgrade Values

Project	Layer 2		Layer 3	
Location	Layer Type	Modulus (ksi)	Layer Type	Modulus (ksi)
Glenn/Parks Interchange SB	RAP	199	Select A	112
Glenn/Parks Interchange NB	RAP	258	Select A	89
C Street	Subgrade	48		
A Street	Subgrade	30		
K-Beach Road Bridge Access to Sterling	Select A	48	Subgrade	28
Seward Highway Dimond to Dowling NB	Select A	86	Subgrade	52
Seward Highway Dimond to Dowling SB	Select A	85	Subgrade	92
Seward Highway Dowling to Tudor	Select A	108	Subgrade	81
Seward Highway MP 66	FASB	150	Select A	58
Seward Highway MP 70	FASB	153	Select A	67
Funny River Road	FASB	225	Select A	79
Parks Highway MP 146-163	FASB	174	Select A	72
Parks Highway MP 52-57	FASB	198	Select A	59
Parks Highway MP 72-83	Select A	88	Subgrade	54

Select A is Selected Material, Type A, a granular borrow material having less than 6% P200's
FASB is Foamed Asphalt Stabilized Base, typically containing 2%-3% binder and 1% cement

With the exception of those measured on A Street and C Street, the back-calculated modulus values largely exceed the Fall Modulus value for modified binders currently used in the pavement design software. Values on A and C Streets are potentially lower because they were late season paving using static rolling and they may have a higher void content than the other projects using modified asphalts. The unmodified binders all exceed the Fall Modulus value of 300, so this value looks to be adequate. Fall Modulus values used in the software can be considered conservative.

Additional testing could be done in the summer to better define the modulus values during the seasonal high temperatures.

3.5 Hamburg Testing

Hamburg testing was performed on field cores collected the fall of 2020 during FWD testing and on lab produced samples using mixes collected from ongoing construction projects. Mixes with highly modified binders, such as PG 64-40E and PG 58-34E, were tested in a water bath at 50°C. Other mixes with unmodified or low polymer contents such as PG 52-40E, PG52-34 and PG52-

28, were tested at 45°C. Tests were performed using Pavement Technology Inc. Asphalt Pavement Analyzer equipment in the Central Region Materials Laboratory (CRML).

The air void content (VTM) of field cores was determined from core density and project mix design values, as noted in the relationship below.

Voids Total Mix = 100% Maximum Theoretical Density (G_{MM}) – actual Field Density (% G_{MM})

The percent compaction of a mix is related to the VTM by the following equation.

% Compaction = 100% – Voids Total Mix %

Lab-produced samples were tested at three different void contents targeting ranges of 3%-5%, 5%-8%, and 7%-10%. An example is shown below that was tested at 45°C.

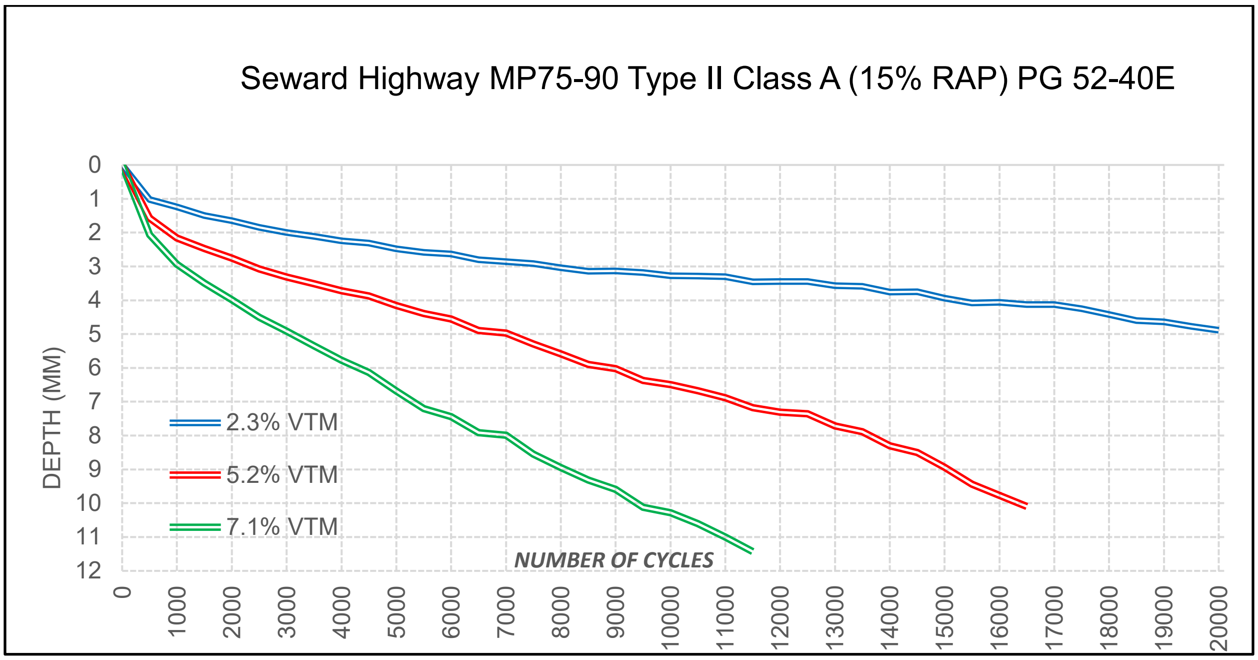


Figure 32 - Hamburg Testing Results – Type II / PG 52-40E / 15% RAP

Typical Hamburg specifications target less than 10mm or 12.5mm of deflection between 10,000 and 20,000 cycles and 40°C – 50°C depending on the anticipated traffic loading and binder type. Alaska DOT&PF is still evaluating Hamburg data to establish a specification but is considering less than 10mm of rut depth at 10,000 cycles when tested at 50°C for highly modified binders (PG

64-40E and PG 58-34E) and less than 10mm rut when tested at 45°C for other binders. The VTM to be used in testing has not been established, but many states use 7.0 ± 1%.

The test results from the Seward Highway MP75-90 Project (above) indicate that void contents significantly impact rut depths. The mix tested would have failed the specification being considered when tested at 7.1% VTM having passed a 10mm rut depth at 9,500 cycles.

Below, in Table 8, is a summary of Hamburg results from field cores. It gives the mix type, binder type, the average void content of the field cores and the rut depth at both 500 cycles for the initial deformation and then at 10,000 cycles. If the depth at 10,000 cycles exceeded 10mm then it would have been considered failed given standard Hamburg specifications.

Table 8 - Hamburg Results – Field Cores

Mix	Binder	VTM (%)	Rut @ 10,000 Cycles (mm)	Rut @ 500 Cycles (mm)	Test Temp (C)	Project Number	Project
Type II Class A	PG52-34	4.4	9.3	3.3	45	53469	Parks Highway MP 52-57
Type II Class A	PG52-40	4.6	5.5	2.4	45	53750	Funny River Road
Type II Class A	PG58-34	5.9	9.7	3.4	50	54147	Parks Highway MP 146-163
Type VH	PG58-34	2.7	6.3	2.7	50	50816	Seward Highway Dowling to Tudor
Type VH	PG58-34	1.9	4.6	0.6	50	CFHWY00162	Seward Highway Dimond to Dowling
Type VH	PG64-40	4.3	4.9	0.4	50	CFHWY00106	Minnesota Drive
Type VH	PG64-40	6	10.6	2.2	50	58571	Glenn/Parks Interchange
Type VH	PG64-40	2.6	7.2	0.3	50	57835	A and C Street

One test on a highly modified Superpave mix did fail having 10.6mm. The data, when plotted, does provide a trend of improved rut depth at lower void contents, and if the cores had been run at 7% voids, as seems to be a standard void content for Hamburg testing, more cores would likely have failed. Stripping was observed on the samples from the Parks Highway MP 146-163 and the

Glenn/Parks Interchange, both at 12,000 cycles. For the graphs of rut depth vs. cycles and for more detailed information on the project mixes, see Appendix C.

Below, in Figure 33, the rut depth at 10,000 cycles is plotted against the VTM. It can be noted that it provides for a reasonably linear trend-line.

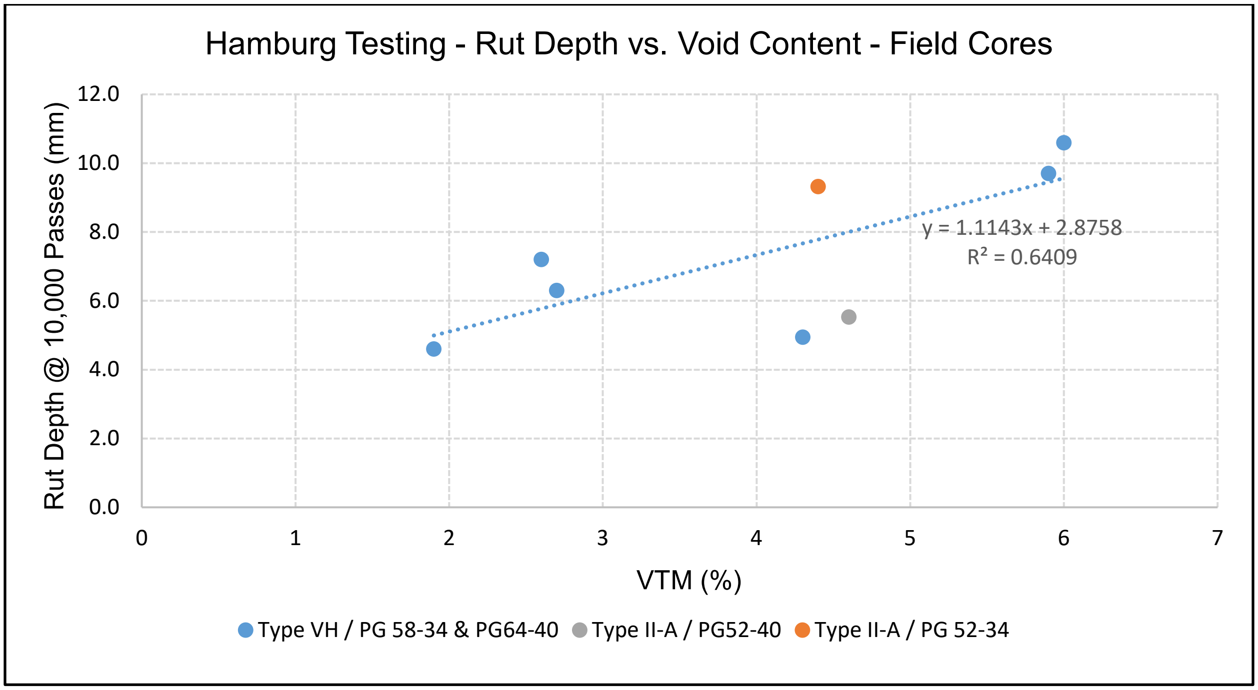


Figure 33 - Hamburg Results – Rut vs. Voids – Field Cores

The next table displays the same data for lab produced samples. It can be noted that the rut depths tend to be lower at 10,000 passes and at 500 passes than those that were taken as field cores.

Table 9 - Hamburg Results – Lab Produced

Mix	Binder	VTM (%)	Rut @ 10,000 Cycles (mm)	Rut @ 500 Cycles (mm)	Test Temp (C)	Project Number	Project
Type II Class A (15% RAP)	PG52-40	2.3	3.3	1.0	45	CFHWY00213	Seward Highway MP75-90
Type II Class A (15% RAP)	PG52-40	5.2	6.5	1.6	45	CFHWY00213	Seward Highway MP75-90
Type II Class A (15% RAP)	PG52-40	7.1	10.3	2.1	45	CFHWY00213	Seward Highway MP75-90
Type II Class AH	PG52-40	2.6	1.6	0.7	45	51829	Palmer Wasilla Highway
Type II Class AH	PG52-40	7.4	3.9	2.0	45	51829	Palmer Wasilla Highway
Type II Class AH	PG52-40	11	6.8	3.0	45	51829	Palmer Wasilla Highway
Type II Class AH	PG64-40	1.8	1.2	0.7	50	CFHWY00317	Glenn Springer
Type II Class AH	PG64-40	5.5	2.6	1.3	50	CFHWY00317	Glenn Springer
Type II Class AH	PG64-40	8.1	4.3	2.1	50	CFHWY00317	Glenn Springer
Type VH	PG52-40	2.6	0.9	0.2	45	59778	K-Beach MP 16-22

Graphing rut depth against voids using lab-produced samples, below in Figure 34, provides for a much better relationship than using field cores. Only one sample failed a theoretical 10mm rut depth specification, and it was at 7.1% VTM. The Type VH sample was included with the lab-produced samples as it was remixed in the lab due to issues with original testing.

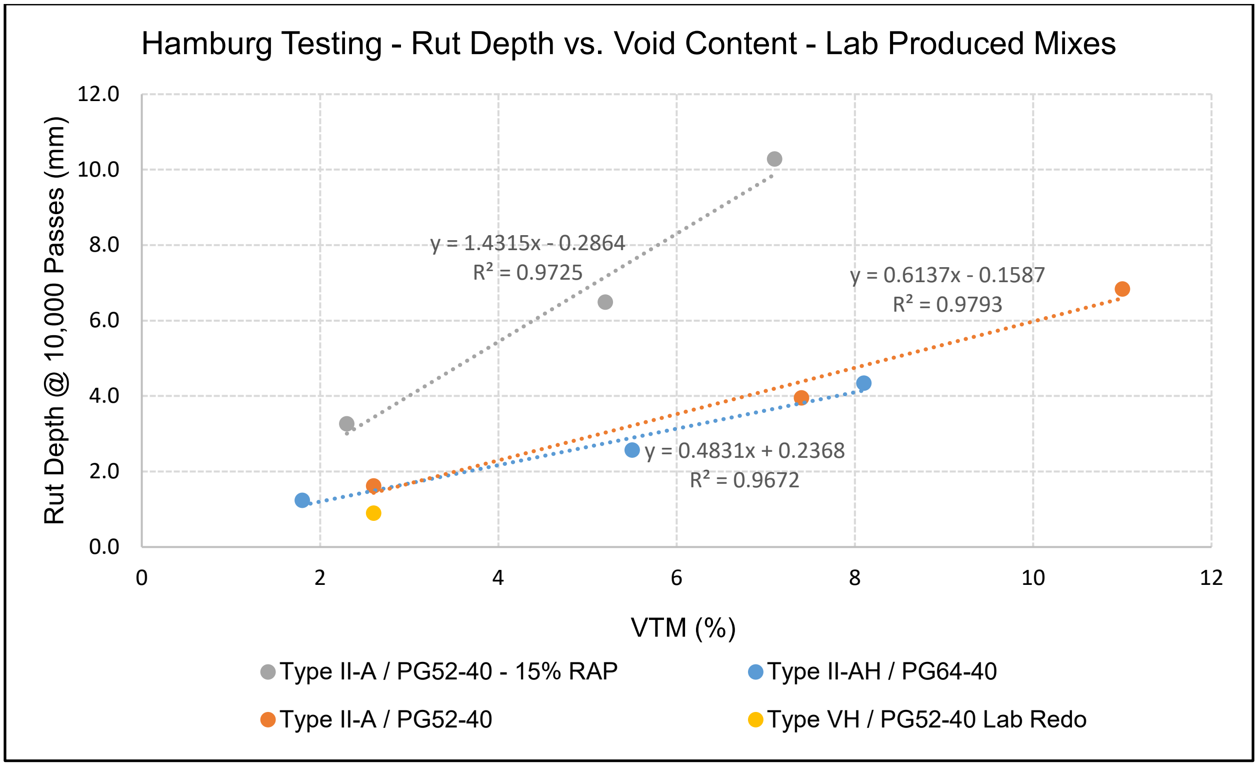


Figure 34 - Hamburg Results – Rut vs. Voids – Lab Produced

If a Hamburg specification is implemented, consider using a high VTM near 7% as the rut depth is directly related to VTM. Testing at a lower value would not sufficiently stress the samples and they may provide passing results on a sample that would otherwise fail. Even the field cores that had higher deformation than the lab-produced cores, only one specimen was over 10mm of rut depth at 10,000 cycles.

3.6 Impact of Air Voids in Pavements

Mix density (air voids) has a major impact on the performance of pavements. The lower the density of pavements compared to the theoretical maximum density, the higher the air voids within the mix.

As noted previously in the report, air voids can be equated to density with:

Voids Total Mix (VTM) = 100% Maximum Theoretical Density (G_{MM}) – actual Field Density (% G_{MM})

Higher air voids impact the fatigue of pavements (traffic loading), pavement permeability, and the resistance of the mix to studded tire wear and load based deformation. All of which decrease the life of the pavement.

3.6.1 Interconnected Air Voids

Air voids in the compacted asphalt mix play a critical role in the long-term performance of the HMA.

NCHRP 531 states, "...to ensure that permeability is not a problem, the in-place air voids should be between 6 and 7 percent or lower. This appears to be true for a wide range of mixtures regardless of NMAS (nominal maximum aggregate size) and the grading".

Connected air voids occur at 6%-7% VTM, or a percent compaction of 93%-94% for coarse aggregate mixes. This means that water or air can travel through the connected air voids causing pavement fatigue, rutting, and environmental (freeze-thaw, water, and deicing solvent) damage. Figure 35, graphing data from NCAT Report 03-02 displays air voids against permeability. Alaska's Type II and Type V pavements have an NMAS between ½" (12.5mm) and 5/8" (15.6mm), plotting between the 19mm and 12.5mm lines, indicating interconnected air voids can occur between 6% and 7% VTM, as in the guidance provided by NCHRP 531.

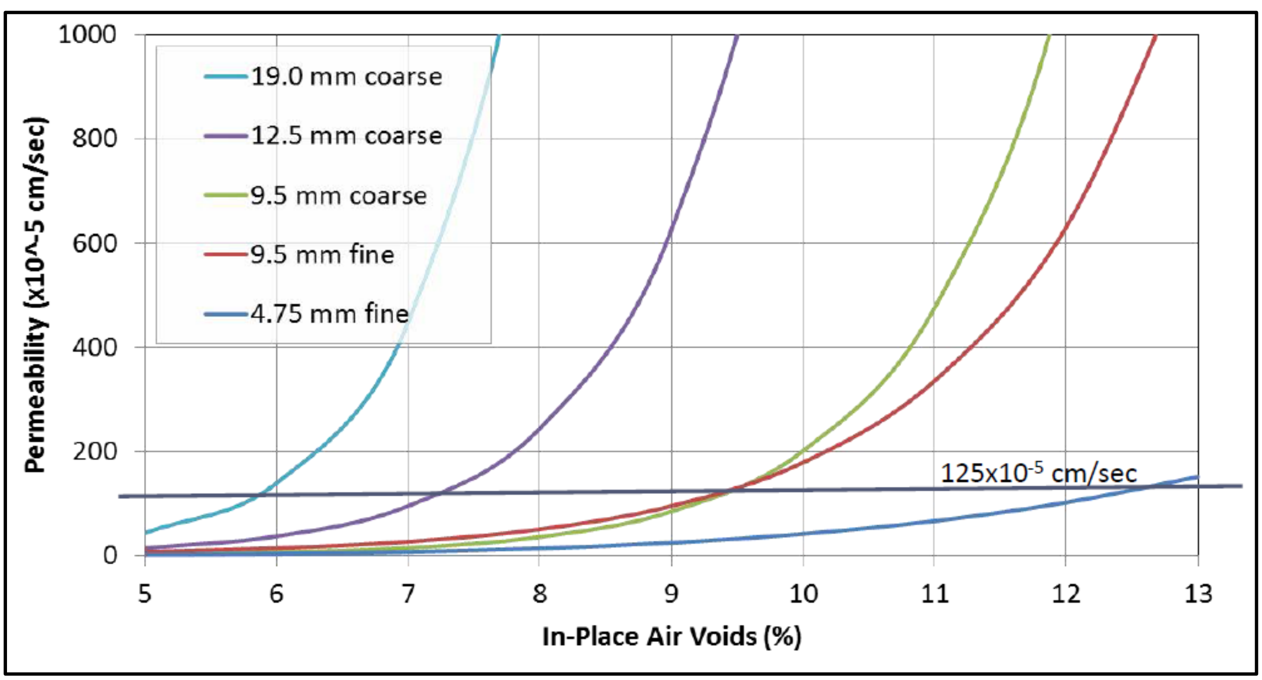


Figure 35 - Permeability vs. Air Voids (NCAT, 2003)

Alaska DOT&PF mixes are designed at 96% G_{MM}, which equals 4% VTM. Current construction specifications accept HMA with a minimum mat percent compaction of 92% G_{mm} (8% VTM) and 91% G_{mm} (9% VTM) for joints on Marshall mix designs. Superpave mix designs require a minimum mat percent compaction of 93% G_{mm} (7% VTM) while the joint remains 91% G_{mm}.

3.6.2 Air Voids and Pavement Fatigue Life

The NCAT Report 16-02 (2016) states, "A 1% decrease in air voids was estimated to improve fatigue performance of asphalt pavements between 8.2% and 43.8%, to improve rutting resistance by 7.3% to 66.3% and to extend the service life by conservatively 10%."

The Alaska Flexible Pavement Design (AKFPD) software reflects the NCAT observation that decreasing air voids (increasing density) improves the fatigue life of pavements. AKFPD software uses the TAI (The Asphalt Institute) fatigue equation for HMA, which is sensitive to mix volumetrics, including air voids. Screen Clip 4-17 in the AK Flexible Pavement Design Manual, page 4-24, displays the following structural section (Figure 36). Attention is focused on the mix parameters noted below.

	A	B	C	D	E	F	G	H	I	J	K	L	M	N
4	Project Location:	Anchorage Area				4500	Load Loc (in)							
5				Design		Tire Press. (psi)	X:	0	13.5					
6	Design AADT:	20,000		Loadings		110	Y:	0	0					
7	Spring%:	15		641,679			Eval Loc (in)							
8	Summer%:	30		1,283,359			X:	0	6.75					
9	Fall%:	20		855,572			Y:	0	0					
10	Winter%:	35		1,497,252										
11	Total%:	100		4,277,862										
12			Critical Z		Asphalt			Poisson's	Tensile	Compressive	Million Cycles	Past	Future	Total
13		Layer	Coordinate (in)		Properties	Season	Modulus (Ksi)	Ratio	Micro Strain	Stress (psi)	to Failure	Damage (%)	Damage (%)	Damage (%)
14				Air%:	5	Spring	350	0.30	169		5.34		12.02	12.02
15	Thickness (in):	4	3.99	Asphalt%:	5.5	Summer	300	0.30	168		6.23		20.61	20.61
16	Name:	sphalt Concrete (Unmodified Asph		Density (pcf)	148	Fall	300	0.30	168		6.23		13.74	13.74
17	Use TAI:	Yes				Winter	1200	0.30	57		67.95		2.20	2.20
18											Total Damage:		48.57	48.57
19				Air%:		Spring	100	0.35		41.6	6.91		9.28	9.28
20	Thickness (in):	4	4.01	Asphalt%:		Summer	100	0.35		44.6	5.51		23.31	23.31
21	Name:	3-4% Asphalt Treated Base		Density:		Fall	100	0.35		44.6	5.51		15.54	15.54
22	Use TAI:					Winter	300	0.35		38.4	321.42		0.47	0.47
23											Total Damage:		48.60	48.60
24				Air%:		Spring	35	0.40		14.2	7.44		8.63	8.63
25	Thickness (in):	12	8.01	Asphalt%:		Summer	40	0.40		15.6	8.59		14.94	14.94
26	Name:	Select A P200<6%		Density:		Fall	40	0.40		15.6	8.59		9.96	9.96
27	Use TAI:					Winter	90	0.40		12.0	284.72		0.53	0.53
28											Total Damage:		34.06	34.06
29				Air%:		Spring	10	0.45		3.5	7.73		8.30	8.30
30	Thickness (in):	240	20.01	Asphalt%:		Summer	10	0.45		3.5	7.98		16.08	16.08
31	Name:	Select C P200<30%		Density:		Fall	10	0.45		3.5	7.98		10.72	10.72
32	Use TAI:					Winter	10	0.45		1.9	54.67		2.74	2.74
33											Total Damage:		37.84	37.84
34						Spring	5	0.45		0.1	810213.85		0	0
35	Thickness (in):	0	260.01			Summer	5	0.45		0.1	795886.39		0	0
36	Name:	Subgrade P200>30%				Fall	5	0.45		0.1	795886.39		0	0
37						Winter	5	0.45		0.0	1014639.71		0	0
38											Total Damage:		0	0

Air 5%, Asphalt 5.5%, Density 148 pcf

Figure 36 - AKFPD Software Asphalt Properties (DOT&PF, 2020)

In Figure 37, below, the mix parameters were maintained for binder content and unit weight while the air void content was adjusted. The impact of the traffic as Equivalent Single Axle Loads (ESAL's), representing an 18,000-lb. single axle load, to pavement failure is very apparent. The pavement with low voids can without nearly three times the ESAL's as the pavement with high voids.

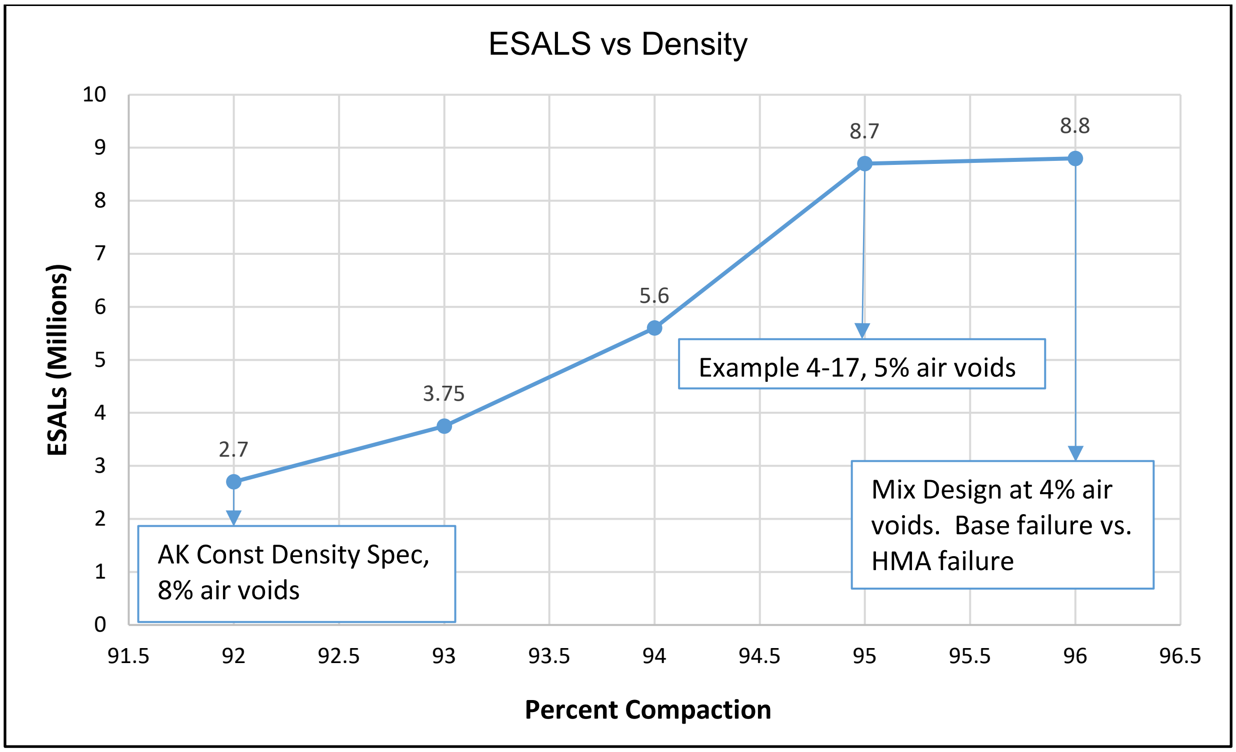


Figure 37 - AKFPD Traffic Loading to Failure vs. Density

3.6.3 Measuring Mat and Joint Densities

After noticing the impact of air voids on pavements, an evaluation was made of random coring vs. a dielectric profiling system (DPS). This is an emerging technology that can be used for density evaluation of the full mat and joint layers during construction.

Current Alaska DOT&PF specifications apply statistics to a small population of random core densities to generate mat compaction statistics. These statistics determine the density percent within limits (PWL) of the compaction pay factor to be compared to the pay factor for gradation and asphalt content. Conformance to current specifications can generate an incentive for HMA that is 5% higher than the bid prices. A separate longitudinal joint compaction factor is paid based on the average percent compaction of all the joint cores.

The Alaska DOT&PF Quality Assurance Engineer, with research funded by FHWA, has been evaluating the use of a DPS to more accurately determine density by measuring pavement density in one-foot increments or less on the full mat. The equipment being evaluated is the GSSI PaveScan Rolling Density Meter (RDM), AASHTO PP 98-20.

Both core and DPS data were collected on the Glenn Highway paving project from Eklutna to Hiland Road in 2017. This technology allows for a compaction value every 6” of the pavement mat. Two passes are made with the equipment to collect a line of joint compaction readings and 5 lines of mat compaction readings across the 12’ lane.

Table 10 (below) shows the average, maximum and minimum percent compaction densities from both the panel and joint cores taken on the project. It can be seen that the average densities were well within specification and the minimum density on the panel was 92.3%, just below the specification limit on the Superpave mix of 93%. On the joints the minimum core density was 90.9%, just below the specification limit of 91%.

Table 10 - Glenn Highway 2017 Mat vs. Joint Densities

	% Compaction	
Compaction Summary 2017 Data	**Mat**	**Joint**
Project Average	94.9	94.5
Maximum	97.6	97.8
Minimum	92.3	90.9

On Figure 38 mat (panel) % compaction is displayed from both the DPS data and cores. It can be seen the % compaction collected from DPS and cores closely match, with the DPS identifying very few % compaction values that fell below the lower specification limit of 93%.

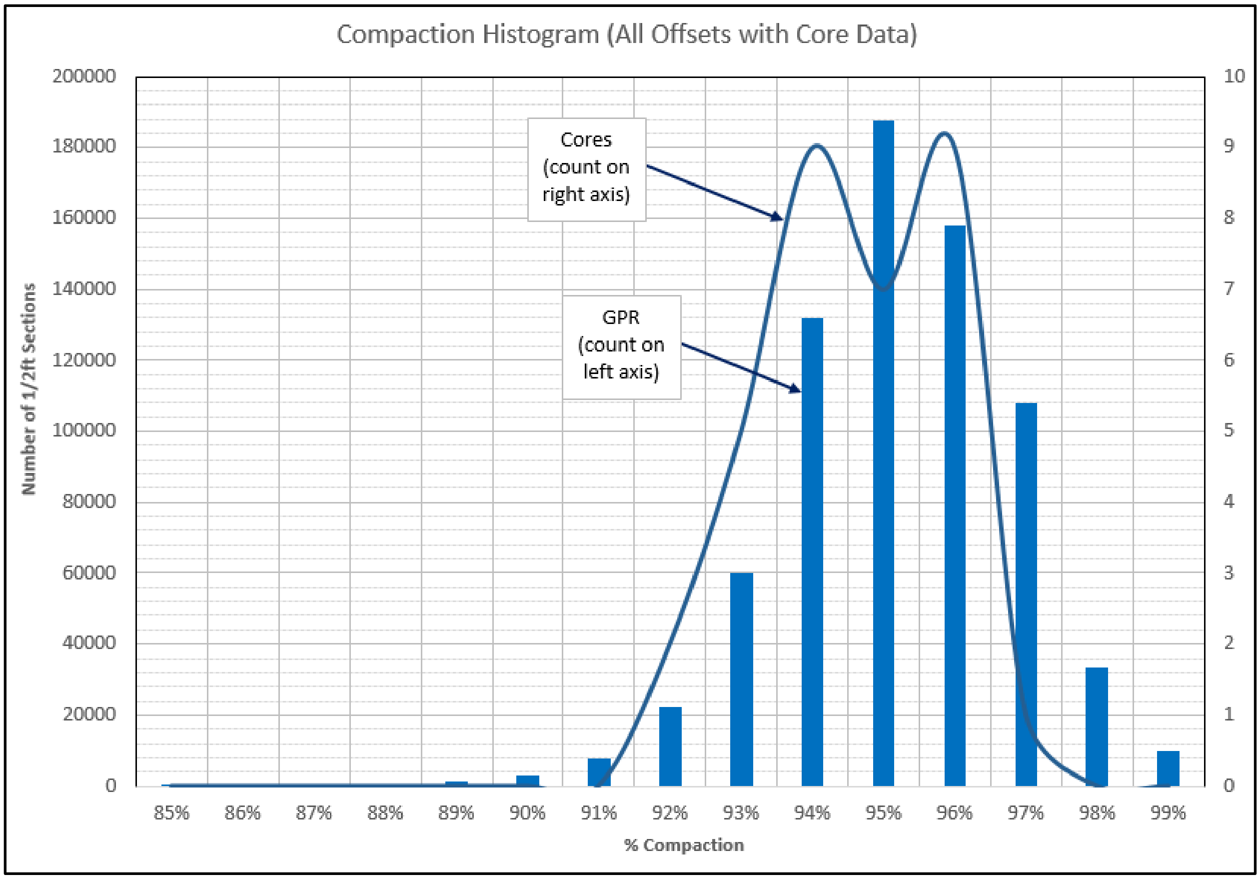


Figure 38 - Glenn Highway Mat DPS % Compaction vs. Random Coring (DOT&PF)

In Figure 39 the joint % compaction from both coring and DPS are displayed. The DPS unit collected 151,620 % compaction readings, one every 6" along the longitudinal joints over 15 miles of joint. It graphs how the distribution of 33 core compaction values do not match those collected from the DPS unit on the joint as it did on the mat. The blue line representing the count of core densities does not capture the joint densities captured by DPS below a 90% density shown in grey. Only 33 joint cores were taken along 15.2 miles (80,256 feet) of joint or one per 2,432 feet. This is shown to be inadequate to predict the actual distribution of compaction values measured by DPS in Figure 39.

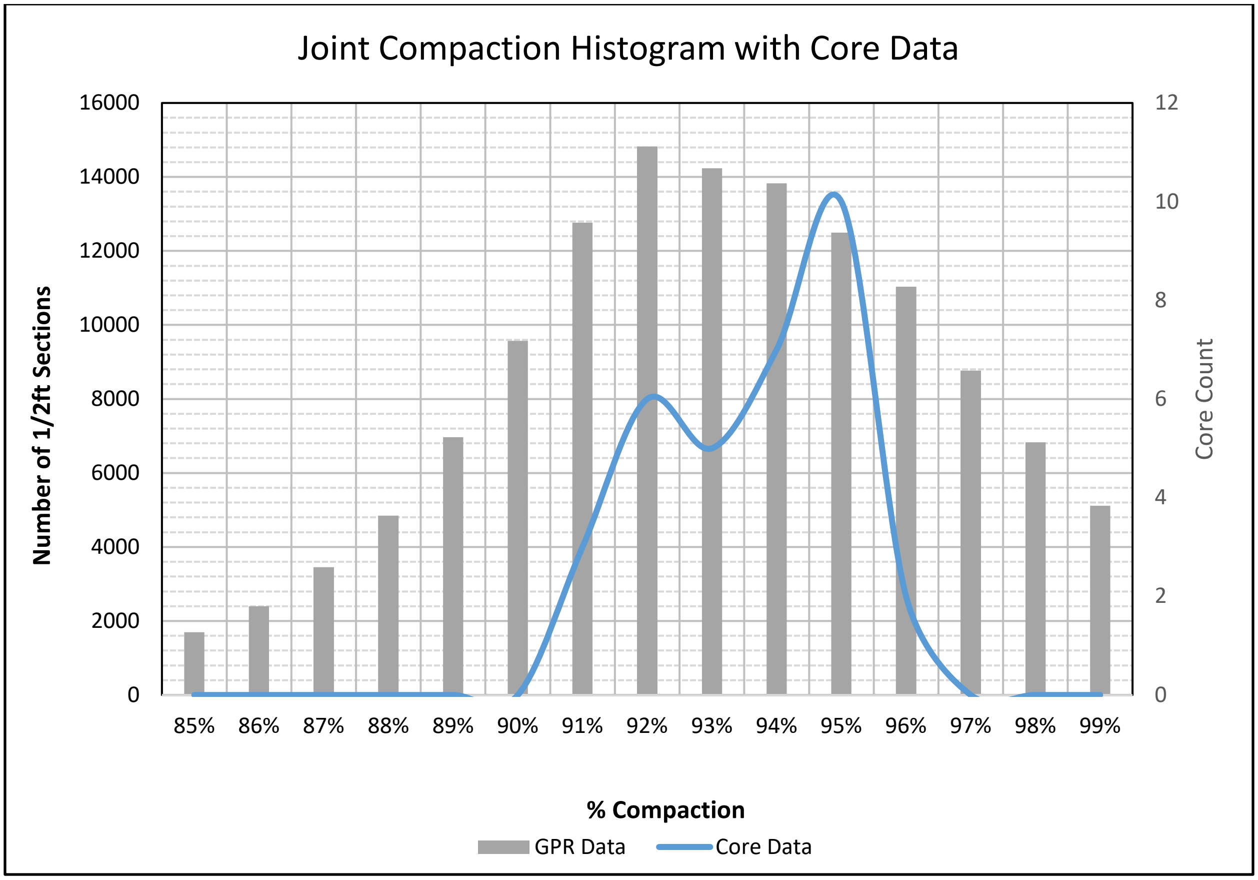


Figure 39 - Glenn Highway Joint DPS Density vs. Random Coring (DOT&PF)

The cumulative % compaction plot, as shown in Figure 40, from the full DPS coverage joint data indicates that 24.5% of the joint length was below the minimum % compaction specification of 91%. While the mat achieved the specified % compaction and the cores closely matched the full mat density distribution the joints can still be problematic as the random coring does not capture the full picture.

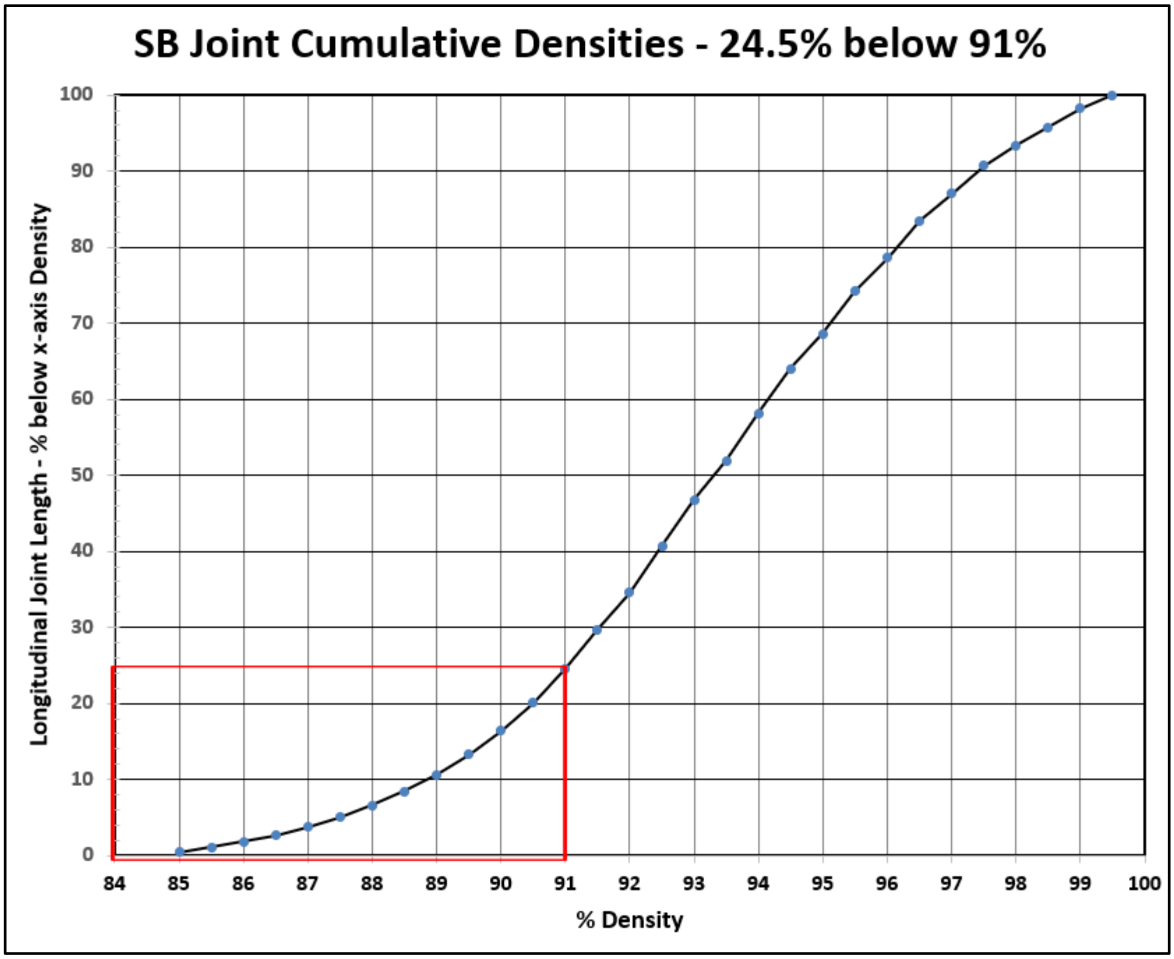


Figure 40 - Glenn Highway Joint DPS Density Below Lower Specification Limit (DOT&PF)

CHAPTER 4 – INTERPRETATION OF RESULTS

There have been significant changes to the binders and aggregate specifications used throughout Alaska over the last 20 years. As polymer modified binders have come to be more commonly used, Alaska DOT&PF has tried to evaluate better ways to formulate binders and specify aggregates to better resist pavement rutting (deformation and studded tire abrasion) and thermal cracking.

The historic mix and binder testing led to the use of a PG 64-40E binder in mixes on high traffic volume roads in Central Region that are prone to rutting from studded tire abrasion.

On lower speed roads (35mph-50mph), the available pavement management data indicates that PG 64-40E provides a lower rate of rutting compared to the previously used PG58-34E binder using Superpave mix with hard aggregates.

At higher speeds, a good comparison between the rut rates of Type VH mixes using PG 64-40E and 58-34E binders was not able to be made. This is due to hard aggregates and PG 64-40E binder being phased in quickly on high-speed roads with high rates of rutting. What limited data is available indicates a slight improvement of performance with PG 64-40E binders at high speeds, but a larger dataset for Type VH, PG 58E-34 binder at high speeds would need to be available to draw more conclusions.

Pavement Management data does indicate a reduction to the rate of rutting with the use of hard aggregates.

The Prall testing performed as part of this project indicates that lowering the low temperature of the performance grade of binders provides for improved Prall results, as does including hard aggregates, raising binder contents and lowering VTM. Prall samples tested that included RAP performed poorly, regardless of the binder or mix used.

Hamburg testing shows that the mixes currently used throughout Central Region would largely pass a Hamburg specification of less than 10mm rut depth with 10,000 cycles at 50°C for highly modified binders and 45°C for other binders. There is a direct relationship between the VTM of samples tested and the rut depth at 10,000 cycles, indicating that lower VTM provides improved rutting resistance from deformation.

FWD analysis indicates the Fall Modulus values used within Central Region are adequate to slightly low, and can be considered conservative. They could be raised for highly modified binders and foamed asphalt stabilized base. This project did not perform testing during the spring, summer or winter seasons, or perform a temperature correction. Further research may be needed during those seasons and on a wider range of roads before raising hot mix design modulus values.

All of the testing and research performed throughout this project shows that VTM has a major impact on the performance of pavements. The testing and research led to the development of a new hot mix specification, given the number 412. This specification closely follows the procedures outlined in the FAA P-404 Fuel Resistant Pavement that is produced with a 50-blow Marshall mix design. It targets a compaction of 97.5% G_{MM} with design mix air voids of 2.5% and a high binder content, ranging between 5.5% and 8%. This raises both the target compaction level and binder content when compared to current practices. It uses a polymer-modified asphalt, PG 64-40E within Central Region to resist studded tire abrasion, or a PG 52-40E in Northern Region to provide for improved cracking resistance. Northern Region is evaluating the use of a PG 58-46 binder that would be a good candidate for this mix design.

Two aggregate gradations are included in Specification 412, one being a fine gradation (1/2" minus) and the other coarse (3/4" minus). The fine gradation is included to allow for thin inlays within the Anchorage area for studded tire repair. The mix is designed at a VTM of 3.5% and increased binder, but care must be taken to make sure the mix is not prone to deformation rutting due to too much binder for the VMA.

This specification includes scaled bonuses for mat and joint density to incentivize improved compaction. The required minimum mat and joint percent compaction is 93%, and a full bonus is paid at 96%.

The DPS is a non-destructive test method that allows for full width mat and joint density collection. Its use on projects in Alaska indicates that random coring does an adequate job of capturing the percent compaction on the mat, but it misses a large number of areas on joints. Alaska has not implemented this technology for construction acceptance, but it is being evaluated use for on joints.

A second version of the 412 specification includes DPS technology for measuring the full mat and joint density with bonuses being calculated using collected DPS data. The specifications with and without the use of DPS are available in Appendices F and G.

CHAPTER 5 – CONCLUSIONS AND SUGGESTED RESEARCH

Historic research into hot mix asphalts and binders has resulted in Alaska DOT&PF using polymer modified binders within Central Region to resist plastic deformation as well as incorporating hard aggregates into mixes to better resist studded tire wear. Recently, highly modified binders with a PG of 64-40E have been used on roads with a high rate of rutting in Central Region. This binder has shown promise in reducing the rate of rutting compared to previously used binders based on Pavement Management data and it should continue to be used and evaluated for performance. A better comparison could be made between the performance of the currently used PG64-40E binder and the previously used PG 58-34E binder if it the PG 58-34E binder were placed on a high-speed road and incorporated hard aggregates.

The Northern Region of Alaska DOT&PF has been using modified binders with a PG of 52-40 and 52-34 to better resist thermal cracking, and the Pavement Management data available indicates this is providing for superior cracking resistance. The data used for this analysis was limited and, as more data becomes available, including its use on a greater number of roads within the region, this analysis should be revisited.

A wider range of mixes incorporating RAP should be included in a thermal analysis to determine the rate of cracking compared to those without RAP. The one instance evaluated in this analysis showed a significant increase in cracking with a mix incorporating 25% RAP with PG52-40 binder over a PG52-40 mix that contained no RAP.

Prall testing indicates that using increased polymer contents and lowering the bottom end of the PG to -40 provides improved studded tire resistance. Lowering VTM and raising binder contents also improves results. Including RAP in the mix leads to a reduction in performance. RAP should not be included in mixes placed on roads with high rates of studded tire wear.

A Hamburg specification can be implemented within Central Region on its roads with high static loading. The mixes evaluated within this research indicate most of those currently used will pass a specification of less than 10mm rut depth at 10,000 passes using a test temperature of 50°C for highly modified binders and 45°C for other binders at 7% VTM. Prior to implementing a Hamburg specification, it is recommended that Hamburg tests be run in parallel with APA for a minimum of one construction season to compare the performance of mixes using a broader dataset than was used in this research.

The regions should further evaluate seasonal HMA values and base modulus values for use in the AKFPD. Testing indicates that modulus values may be conservative for both FASB and the Modified Asphalts Fall HMA value.

The air voids within pavements, or VTM, have an enormous impact on pavement performance. VTM directly impacts:

- Pavement permeability (greater than 6% VTM)
- Fatigue life
- Studded tire wear resistance
- Load based deformation
- Cracking resistance

Current construction specifications accept pavements at a mat percent compaction of 93% and a joint percent compaction of 91% for Superpave mixes (Type V). For Marshall Mixes (Type II) the pavement is accepted at a mat percent compaction of 92% and joint percent compaction of 91%. With the impact of density, or VTM, on pavement performance raising the minimum mat and joint percent compaction to 93% should be considered.

The cost of compaction to achieve higher densities is relatively minor when compared to the rest of the projects cost. When it is considered that pavement deterioration, which is directly impacted by the compaction achieved during construction, is the driving reason that a future project will take place, it is worthwhile to take additional construction effort to achieve high compaction. A scaled mat and joint density pay factor should be considered to pay full bonus at a percent compaction of 97%, and the joint incentive can be used to help pay for the additional effort required to reach that level. Note that current paving equipment and methods are achieving densities ranging from 94% G_{MM} to 96% G_{MM} (4% - 6% VTM), so full bonus can be achieved.

As a result of this research, a new specification given the number 412 was written for a low void, high binder content mix. This is intended for use on roads with high rates of rutting using a PG64-40E binder, or for improved resistance to thermal cracking in Northern Region using a PG52-40E or PG58-46E binder. Two gradations are included in the specification, one a fine mix for use in a thin inlay for rut repair of 1" to 1.5" thickness, and the second being a coarse mix for a standard 2"-3" layer. This specification provides a starting point that can be modified by the regions to better fit their needs.

A second version of the 412 specification was written that includes DPS technology. The use of the DPS equipment for joint evaluation should be considered. The data collected to date using this equipment indicates the minimum specification limit for mat density is being met, but a large number of longitudinal joints do not meet the minimum density specifications, and this deficiency is not being captured by random coring. This technology would also be of use on bridge decks where coring is not permitted, and a large number of pavement failures related to low density are observed.

When researching the historic changes to aggregate specifications, it was noted that the broad band specification for Type II pavements was widened from 45-65 to 33-70 during the temporary change to metric in 1998. This band should be returned to its previous limits.

Areas that would benefit from future research include:

- FWD Testing in Central Region and Northern Region
    - Continuous testing through spring to observe effects of thaw weakening on FWD results
    - Summer and fall testing
    - Record pavement temperatures during collection as recommended in Elmod software to perform accurate temperature corrections
    - Perform additional modulus testing in the lab to compare modulus values by mix and binder type in a controlled environment. Evaluate modulus-temperature relationships
- Additional evaluation of thermal cracking by binder and RAP content using Pavement Management Data
    - Northern Region has been paving more roads with PG52-40V binders that can be included in this analysis.

- Further evaluation of Type VH PG 64-40E to Type VH PG58-34E mixes at low speeds (less than 55mph). More low speed roads within the Anchorage area are being paved with Type VH PG64-40 binder that will allow for improved comparison of rut rates in the following years.

CHAPTER 6 - REFERENCES

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APPENDIX A – LITERATURE REVIEW

A.1 Alaska Binders and Thermal Cracking

DOT Research Report, *"Laboratory and Field Evaluation of Modified Asphalt Binders and Mixes for Alaskan Pavements"* by UAF Jenny Liu 2020

Research Purpose

Executive Summary, P. 2 "The aims of this study are to conduct laboratory and field evaluation of the performance (i.e., plastic deformation and low temperature cracking performance) of various modified binders and mixes, and to quantify the performance benefits of these modified materials."

Report Summary and Conclusions (that are implementable)

"The asphalt binders included one virgin binder (PG 52-28) and 12 modified binders from three different suppliers. Ten (10) HMA mixtures were either produced in the lab or collected from paving projects for laboratory performance evaluation."

- **Plastic Deformation (rutting)**
    - AASHTO M332 Multiple Stress Creep Recovery DSR tests show the following at a stress loading at 3.2 kPa and PG high temperature;
        - Elastic recovery (R%);
            - Unmodified asphalt 0.86 %
            - Polymer Modified 29% - 98%
        - Residual plastic deformation/ applied stress (J_{NR});
            - Unmodified asphalt 2.8
            - Modified 0.2 – 2.4 kPa^{-1} with the lowest value being the most highly polymer modified binder.
    - Binder J_{NR} correlates well with rut test results
    - The Hamburg loaded wheel test (HWT) on HMA indicate that highly polymer modified binders resist rutting.
    - HWT indicated that Alaskan mixtures are not susceptible to moisture damage at the tested temperatures.

- **Low temperature cracking** (many different tests were performed to define low temperature cracking of a binder and mix)
    - There were a lot of low temperature tests performed with laboratory aged binder; however, no correlation to actual field tested or sampled binder was used to correlate or valid the laboratory testing.
    - Binder – "ΔTc increased as the asphalt ages, indicating that ΔTc has the potential to evaluate and quantify the cracking susceptibility (durability loss) of the binder." The figure below highlights the value of aging binders in the PAV for 40 hours (specified test time is 20 hours). Noting that at ΔT_C of -3°C, distress starts to occur and at -5°C severe damage occurs. So, the question is how long it will take for the pavement to age to have severe damage occur. Consideration should be given to specify a binder ΔTc as other states are currently doing e.g., Utah DOT.

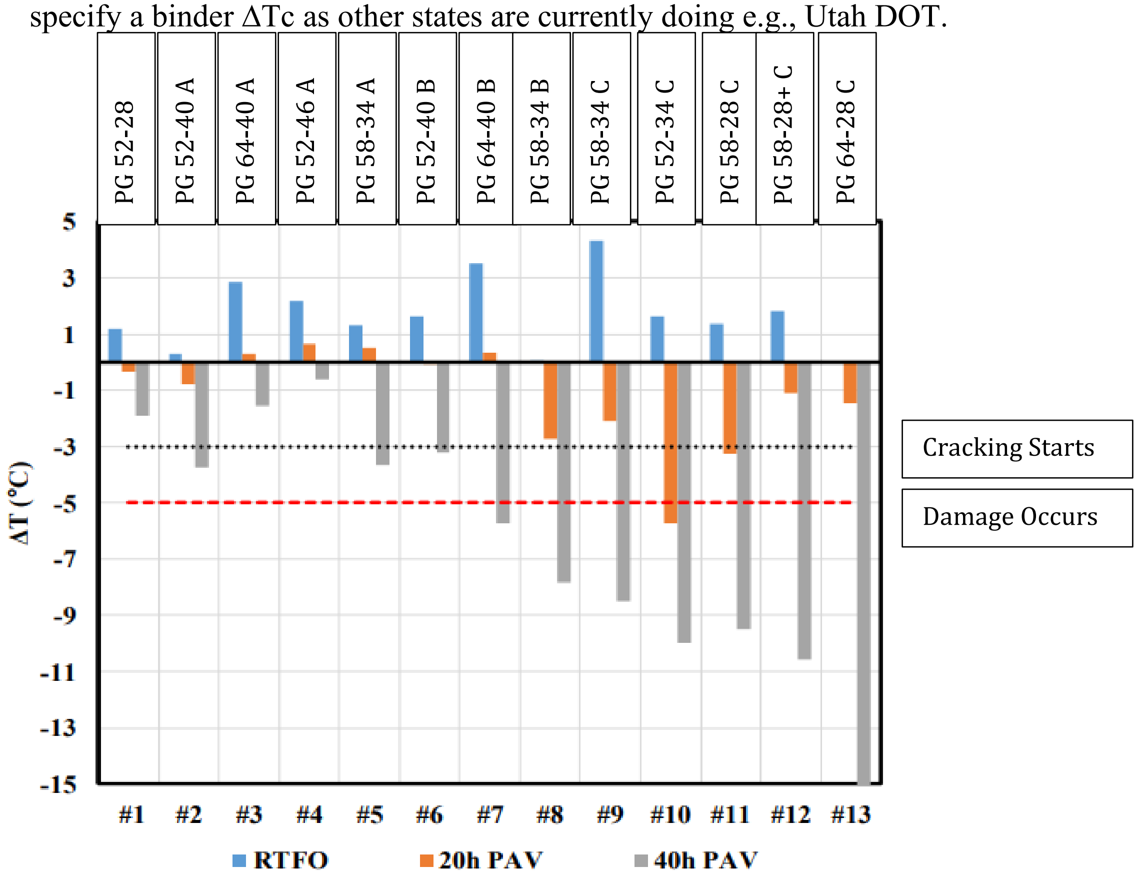


Figure 4.32 ΔT of Alaskan asphalt binders with different aging states

- Mix – all of the tests used require specialized equipment to estimate low temperature cracking, but generally the test results compared favorably with the low temperature of the PG grade.
- Mixture Cracking Temperature: “According to Hills and Brien (1966), low-temperature cracks initiate when the thermally induced tensile stress in the pavement exceeds the strength of the asphalt mixture at that temperature as the temperature drops, and cracking temperature can be predicted as the intersection of the tensile strength-temperature curve and the thermal stress-temperature curve.” The report’s figure 4.41 below illustrates this for mix #1.

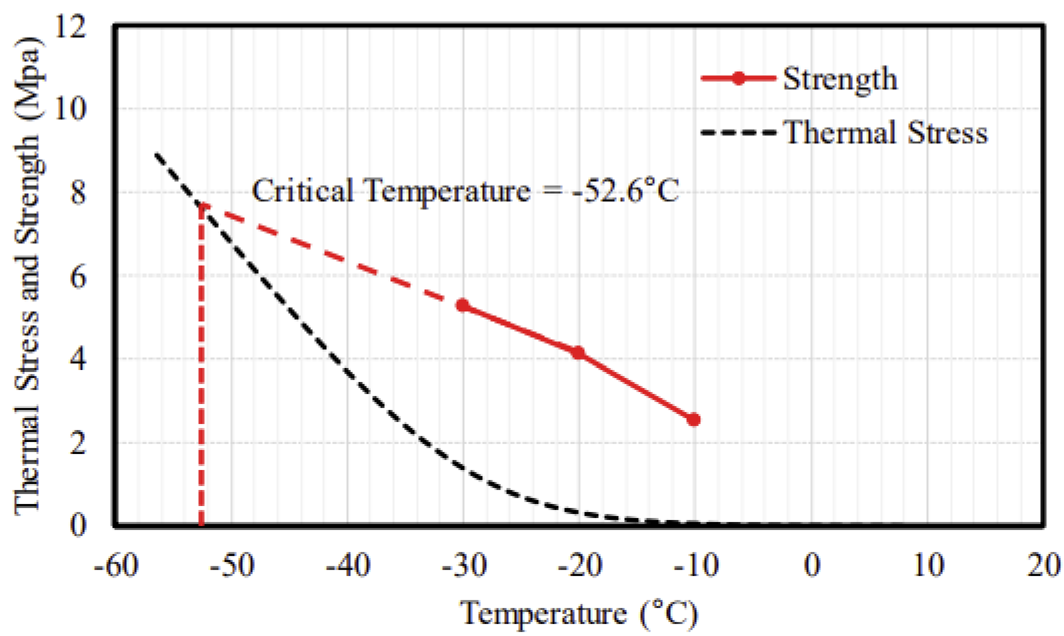


Figure 4.41 Determining mixture cracking temperature for Mix #1

The following graph (Report Figure 4.51) summarizes the different thermal cracking temperatures for the mixes tested. There was no mix from SC tested although binder used in SC was tested. Generally, the low PG temperature governed the cracking temperature. The report contains more detailed data on the cracking test methods evaluated.

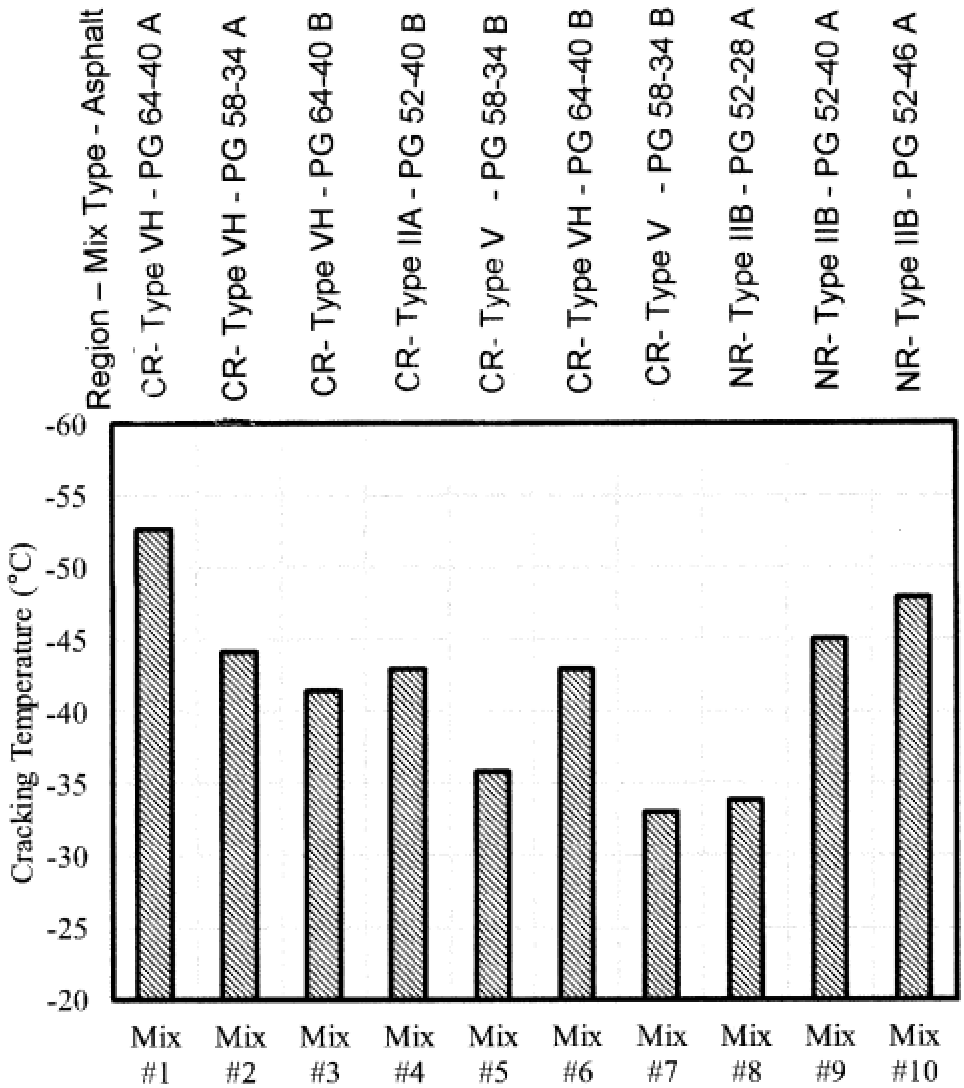


Figure 4.51 Cracking temperatures of Alaskan mixtures

A.2 Storage Stability, Deformation and Cracking

DOT Research Report, *"Constructability of Polymer-Modified Asphalts and Asphalt-Aggregate Mixtures in Alaska"* by Hannele Zubeck, Lutfi Raad, Stephan Saboundjian, George Minassian, John Ryer, 1999 Report No. FHWA-AK-RD-99-1

This research made the following recommendations that were not addressed or utilized in the previously noted research but can still be implemented by DOT.

They were;

- Add a storage stability test to the specifications (revised title in ASTM D7173, "Standard Practice for Determining the Separation Tendency of Polymer from Polymer Modified Asphalt" or Separation of Polymer, % diff. (2) ASTM D5976 as noted in the Asphalt Institute Database. See discussion on P 22, 23 of Zubeck's report.

- Page 66, "The penetration at 10°C and the BBR stiffness correlates well with the TSRST (Thermal Stress Restrained Specimen Fracture Test) fracture temperature. Current HMA designs to deal with Plastic deformation and Thermal Cracking.

This graph shows the Bending Beam Rheometer (BBR) stiffness with "S"= creep stiffness @ 60

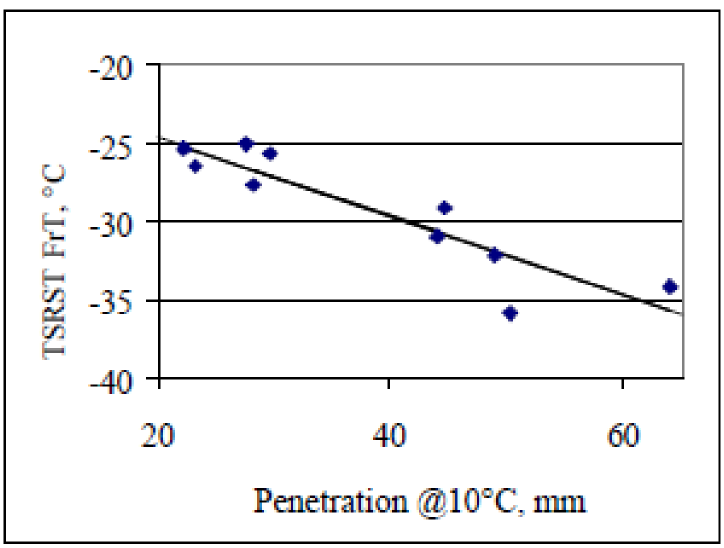


Figure 7-3. TSRST Fracture Temperature versus Penetration at 10°C

seconds of constant applied loading of an asphalt beam in a cold temperature fluid bath of equal specific gravity.

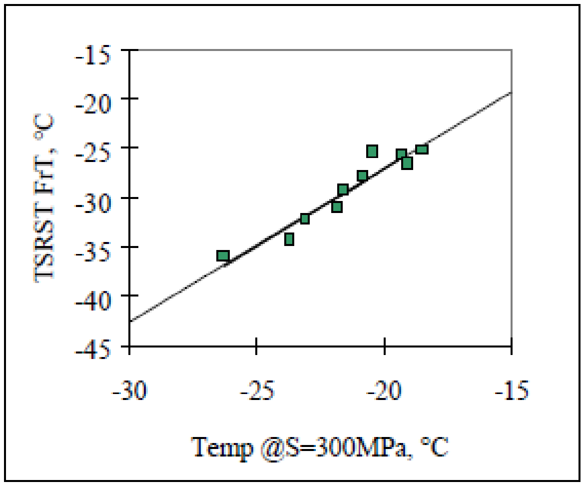


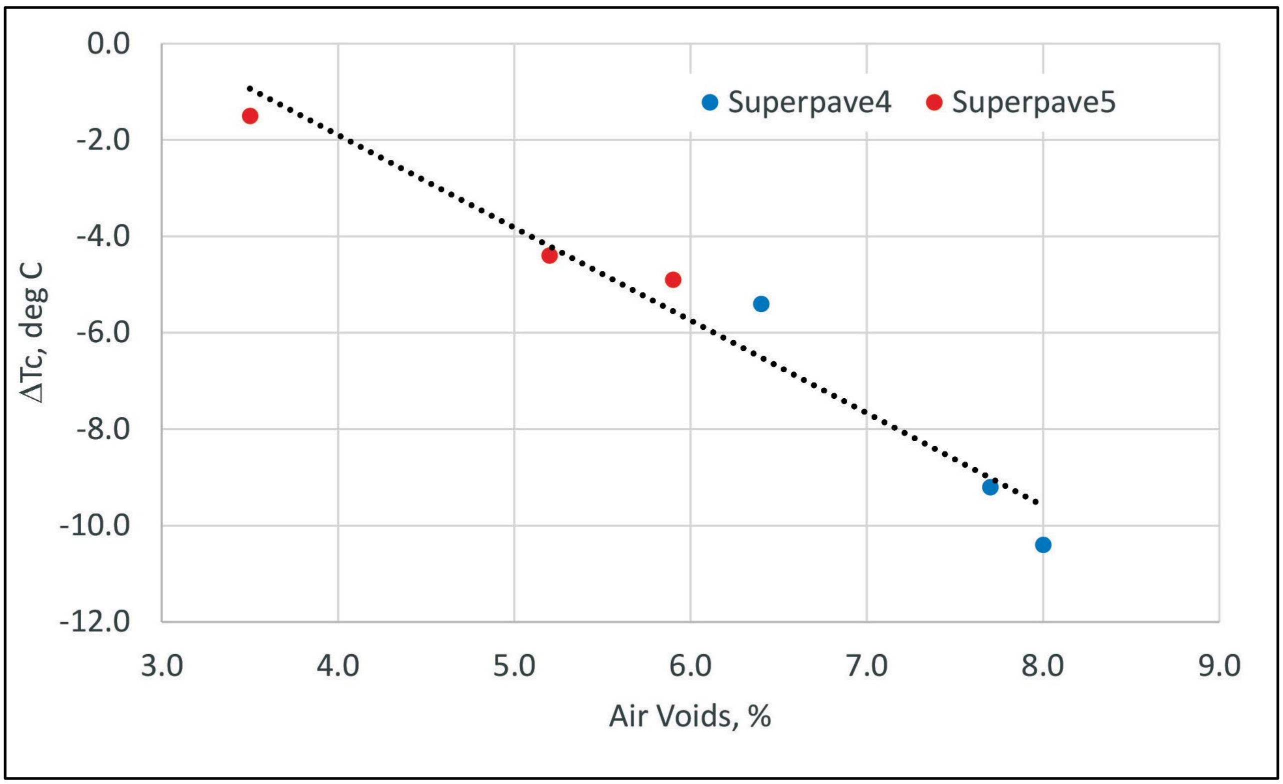
Figure 7-4. TSRST Fracture Temperature versus Temperature at which S= 300MPa

A.3 Delta Tc and Cracking

ASPHALT INSTITUTE IS 240, "The Use of the Delta Tc Parameter to Characterize Asphalt Binder Behavior" and Airfield Asphalt Pavement Technology Program Project 06-01 "A Laboratory and Field Investigation to Develop Test Procedures for Predicting Non-Load Associated Cracking of Airfield HMA Pavements"

IS 240 cites "The goal of the Project 06-01 was to identify "simple binder and / or mixture tests which can predict imminent cracking or raveling so that pavement preservation strategies can be timed to delay or prevent damage of HMA pavements on general aviation airports."

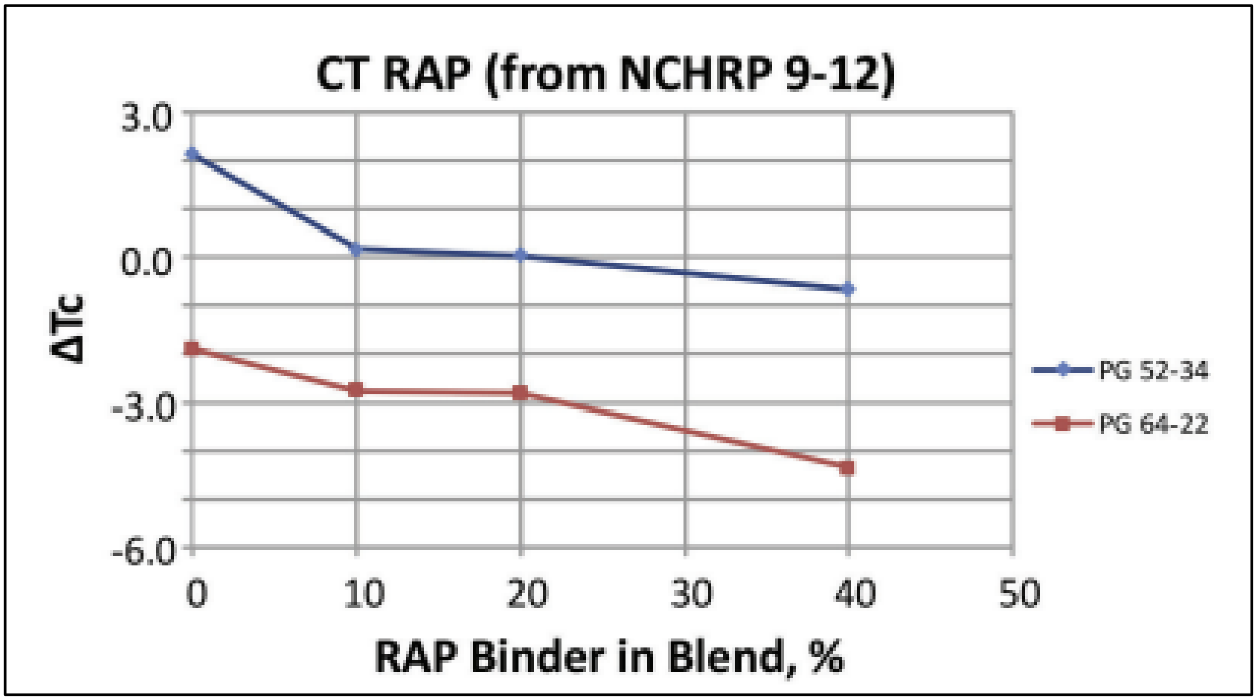
"The authors relied on the landmark research conducted in 1977 by Kandall which showed that raveling and block cracking were highly related to loss of binder ductility when measured at 15°C." The graph below, from IS 240, is based samples of field compacted mix from the top ½ inch of pavement with the core density (air voids) noted.



Superpave4 mix is designed at 4% air voids and compacted in-place to 7% average; where as the Supepave5 mix is designed and compacted at 5% air voids.

"This data is notable for several reasons. Figure 35 shows the clear effect that asphalt mixture in-place air voids has on ΔTc. Huber, et al. measured permeability of the cores and found at about 7.5 percent, the air voids become interconnected and water permeability rapidly increased. Thus, the higher air void content allowed air to better infiltrate the mixtures and age the asphalt binder. The result was a more negative ΔTc. It should be noted that there was no clear indication of correlation with field observations of reflection cracking, which was identified as the dominant distress type."

"Another notable feature of this study was that the Superpave5 mixtures exhibited no map cracking while the Superpave4 mixtures showed extensive map cracking. The Superpave4 mixtures exhibited ΔTc at or below about -5°C while the Superpave5 mixtures exhibited ΔTc at or above -5°C. This observation supports the conclusions of the original AAPTP airfield pavement study by Anderson, et al. (1). An important observation, however, is that the presence of other forms of cracking were not correlated to the observed values of ΔTc." The report shows the effect of ageing (ΔTc) when the extracted the RAP binder and is combined with virgin binder. The following graph is from IS 240.



A.4 Economics of Studded Tire Wear

DOT Research report *"Survey and Economic Analysis of Pavement Impacts from Studded Tire Use in Alaska"* UAA, Osama Abaza, 2019

This research includes the following:

- Data analysis of traffic data, pavement rut depth measurements, pavement damage costs relative to various highway classifications
- Survey of studded tire usage
- Traffic data analysis
- Rut depth and pavement wear rate analysis
- Cost Estimates
- Economic analysis & ways to reduce studded tire usage with possible legislative action

Conclusion and Recommendations

- Use pavement management data to recommend pavement mixes that are most resistant to studded tire wear. This research found that hard aggregate, polymer modified asphalt cement, stone mastic and rubber mixes showed lower wear rates.
- Shortening the season for studded tire use would be beneficial
- Educate the public on newer technology of reduced studs in the tires, the use of lightweight and the use of compounded rubber winter tires,

Comments

- This research supports the current practice of DOT
- Educate the public with newer technology of winter tire performance in schools and drivers education training to reduce the dependency on studded tires for perceived safety

A.5 Warm Mix Additives

During the mid-2000 time frame the asphalt industry was investigating ways to produce and place hot mix asphalt at lower temperatures to decrease the fumes given off and to increase the compaction of the HMA. Alaska evaluated two types of warm mix additives (WMA) on projects;

Sasobit

The report “Evaluation of Warm Mix Asphalt for Alaska Conditions”, DOT# FHWA-AK-RD-12-12 by Juanyu Liu, gives more details of the lab work for the use of Sasobit in Petersburg, Alaska. Sasobit was introduced as an experimental feature on the Petersburg-Mitkof Highway Upgrade Project, Phase II in 2008. In this study, PG 58-28 binder was selected to be modified with Sasobit in four different contents i.e., 0%, 0.8%, 1.5%, and 3.0% by weight of the asphalt binder, respectively.

Sasobit has the unique capability to be mixed into the binder at the refinery and placed into sea going containers and barged to the project site in Southeast Alaska. Sasobit is a Fischer-Tropsch wax, which is a synthetic aliphatic hydrocarbon wax created by heating coal or natural gas with water to 180 to 280 °C (356 to 536 °F) in the presence of a catalyst (D’Angelo et al. 2008). Sasobit has a melting point of more than 98 °C (208 °F), high viscosity at lower temperatures, and low viscosity at higher temperatures.

Liu’s report notes the addition of Sasobit reduced both mixing and compaction temperatures of mixes. Compared with control binder without Sasobit addition, the addition of 3% Sasobit contributed to a decrease of more than 15ºC in mixing temperature and a decrease of 13ºC in the compaction temperature. With the increase of Sasobit content from 0% to 3%, the high temperature end of asphalt PG increased from 58 to 76, however, the low temperature end also increased from

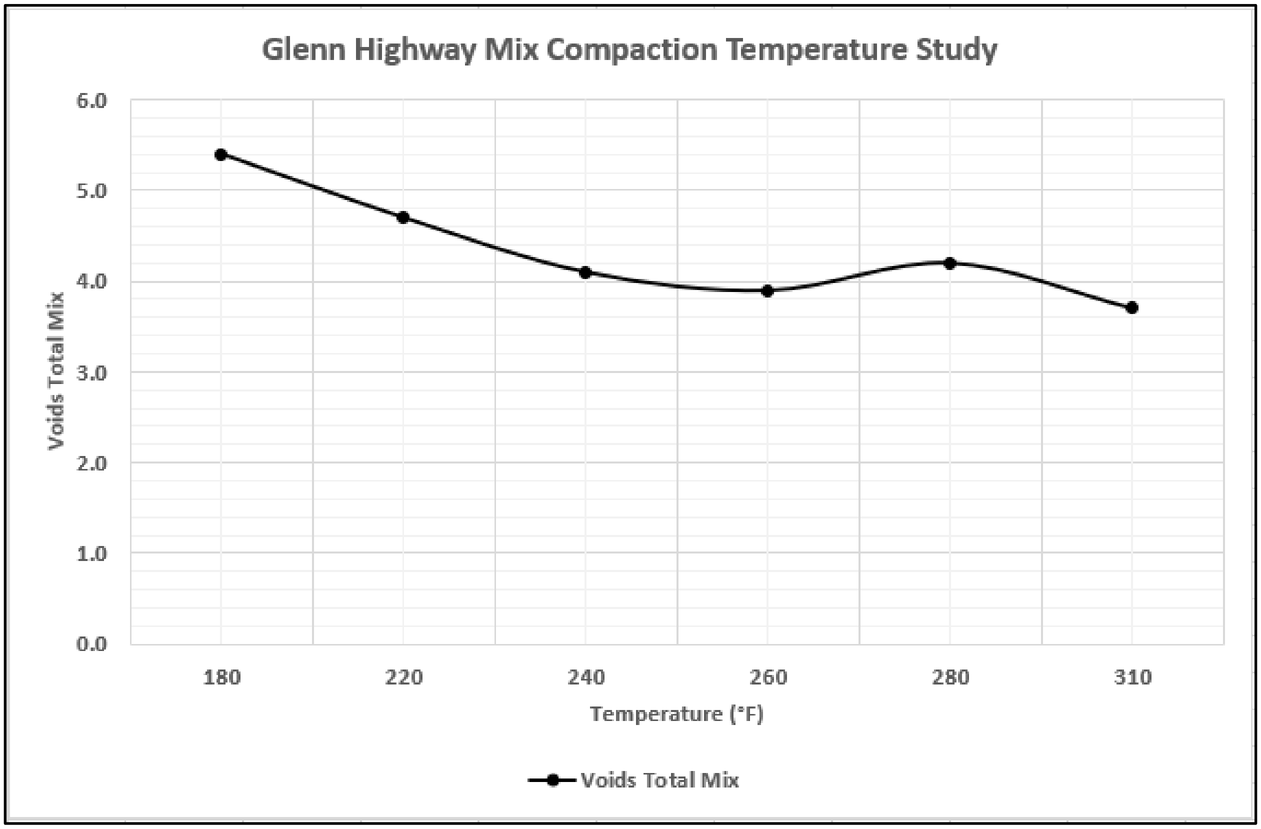
-28ºC to -16ºC as well (Liu, 2012). Results from binder tests implied that Sasobit improved rutting resistance but deteriorated resistances to both fatigue and low temperature cracking.

Evotherm.

Evotherm WMA was used in Anchorage at the International Airport in 2009 at the request of the contractor, Granite Construction, for the Remote Overnight Parking project. The pavement was trafficked by Boeing 747 cargo aircraft. Approval was given to use Evotherm based on no change of the specified PG 64-34 properties and no change in the mix deformation in the Asphalt Pavement Analyzer. Evotherm was metered into the asphalt at the hot plant and proved to enable the contractor to achieve high densities in night paving. Evotherm uses surfactant chemistry which gives it “lubricity” to aid compaction during laydown without changing the asphalt properties. This chemistry modifies the asphalt bond to the aggregate by coating the aggregate surface in a way that allows aggregate particles to slide past each other during compaction, similar to oiling the interface of two pieces of metal to more easily slide one on the other.

To better understand the densification properties of Evotherm at lower temperatures, the lab compacted the same mix, being a Superpave Mix using PG64-40 binder, at reduced temperatures to determine when the Voids Total Mix (VTM) would increase from the design of 4%. During the temperature drop from 310°F to 230°, VTM remained at design. VTM increased from 230°F to 180° with the same compaction effort, but even at 180°F the mix density was still within specification.

Effect of Warm Mix on VTM vs. Compaction Temperature (Alaska DOT&PF)



Note that: density (% Gmm) = 100% – VTM

Thermal segregation during laydown can occur when areas of the hot mat have a temperature differential greater than 25°F causing differential mat density when uniformly compacted causing the related problems of low density. For this project mix, Evotherm prevented differential mat densities from happening when the laydown temperatures were over 240°F.

Thermal segregation is a real problem and care needs to be taken to keep mat laydown temperatures uniform to achieve uniform, passing densities.

APPENDIX B – HISTORIC MIX AND BINDER INFORMATION

B.1 Historic Asphalt Binder Alaska DOT&PF Specification Review

The asphalt binders specified in the Standard Specifications for Highway Construction have evolved significantly between the 1970's and 2021. This section will document the progression and the reasons the binder specifications have changed over the years.

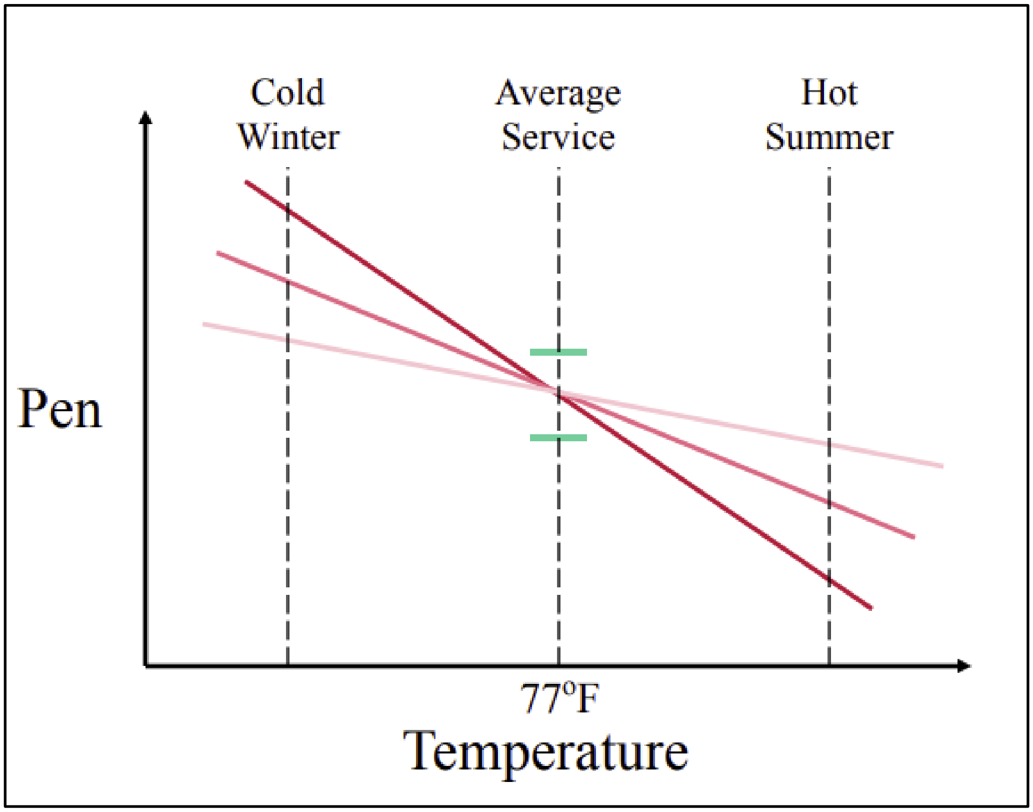
The 1972 Alaska Highway Standard Construction Specifications listed five (5) penetration grades of asphalt cement ranging from 40-50 to 200-300 with the most commonly being 120-150. The following image is of the penetration test where a weighted needle is allowed to penetrate an asphalt sample at 25°C (77°F) for 5 seconds. The depth of penetration is measured in 0.1 mm so that a 200-penetration grade asphalt would have a test penetration of 20 mm. The penetration is an indication of viscous or stiffness and elastic behavior of the asphalt. This data must be correlated to actual field pavement performance of plastic deformation and thermal cracking to select the best performing penetration grade of asphalt to use.

Penetration Grading, (Depth of Penetration of a needle)

AASHTO M20

	120–150	
	Min	Max
Penetration at 29°C (77°F), 100 g. 5 s	120	150
Flash point, Cleveland Open Cup, °C (°F)	218 (425)	–
Ductility at 25°C (77°F), 5 cm/min, cm	100	–
Solubility in trichloroethylene, percent	99.0	–
Thin-film oven test, 3.2 min (1/8 in.), 163°C (325°F), 5 hour		
Loss on heating, percent	–	1.3
Penetration of residue, percent of original	46	–
Ductility of residue at 25°C (77°F), 5 cm/min, cm	100	–
Spot test (when and as specified with):		
Standard naphtha solvent	Negative	
Naphtha-xylene solvent, percent, xylene	Negative	
Heptane-xylene solvent, percent, xylene	Negative	

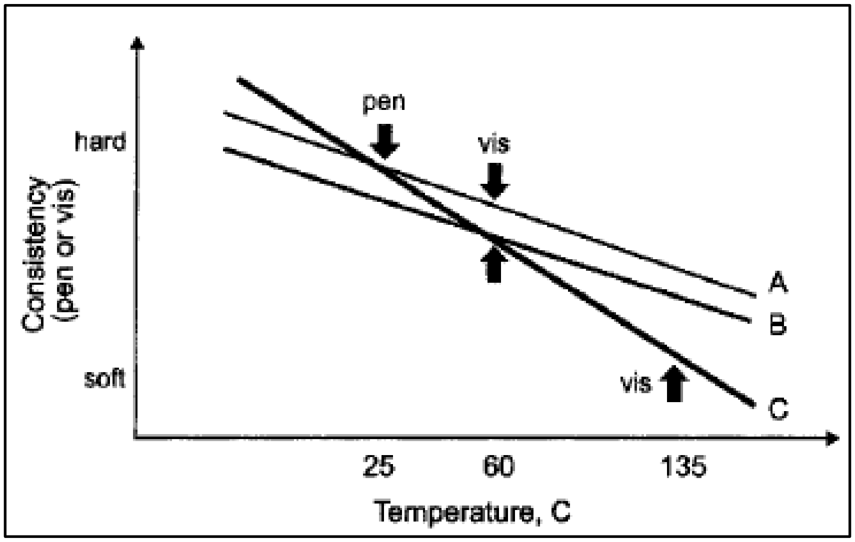
Penetration Grading



Penetration grading, as shown in the graph above, allowed wide binder performance variations at low and high temperature which affected thermal cracking at low temperatures and plastic deformation at high temperatures. Subsequently, viscosity grading of asphalt binder was introduced where asphalt viscosity at 60°C and 135°C was added to the high temperature penetration at 25°C (77°F) with the removal of low temperature penetration requirement.

The viscosity at 60°C (140°F) was selected as the average pavement surface in the US for binder performance and the viscosity at 135°C (275°F) as the average HMA laydown temperate thereby standardizing binder classification.

Viscosity Grading



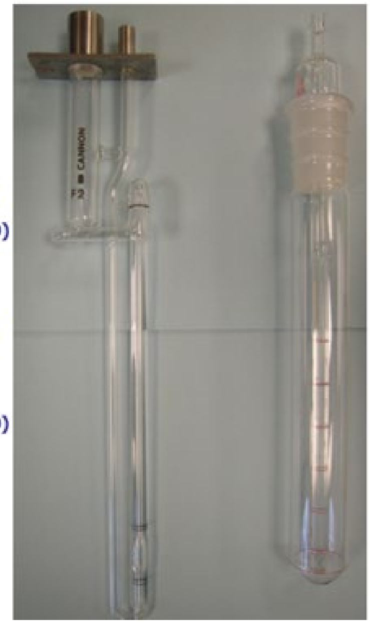
The figure above illustrates the control points of the viscosity grading system. Lines A, B, and C show the variations of three different asphalts within the viscosity specifications. The viscosity of unaged asphalt is measured at different temperatures as the flow rate of the asphalt through an orifice.

The 1975 Alaska Supplemental Standard Construction Manual Specified asphalt cement to be viscosity graded according to AASHTO M 226. This specification changes the penetration and viscosity requirements to allow for the pavement performance at differing ambient temperatures. Note that low-viscosity/ high-penetration asphalt is desired for cold climates; whereas high-viscosity/ low-penetration asphalt is used in hot climates. Alaska continued using viscosity graded asphalt cement until it was replaced with Superpave Performance Graded (PG) asphalt in the mid-2000s

Viscosity Grading Specification

AASHTO M226

	AC-2.5	AC-5	AC-10
Viscosity, 60°C (140°F), Pa·s (Poises)	25 ± 5 (250 ± 50)	50 ± 10 (500 ± 100)	100 ± 20 (1000 ± 200)
Viscosity, 135°C (275°F), mm2/s – minimum	80	100	150
Penetration, 25°C (77°F), 100 g, 5 s – minimum	200	120	70
Flash point, COC, °C (°F) – minimum	163 (325)	177 (350)	219 (425)
Solubility in trichloroethylene, percent – minimum	99.0	99.0	99.0
Tests on residue from thin-film oven test:			
Viscosity, 60°C (140°F), Pa·s (Poises), maximum	100 (1000)	200 (2000)	400 (4000)
Ductility, 25C (77F), 5 cm/min, cm - minimum			
Spot test (when and as specified with):			
Standard naphtha solvent		Negative for all grades	
Naphtha-xylene solvent, percent, xylene		Negative for all grades	
Heptane-xylene solvent, percent, xylene		Negative for all grades	



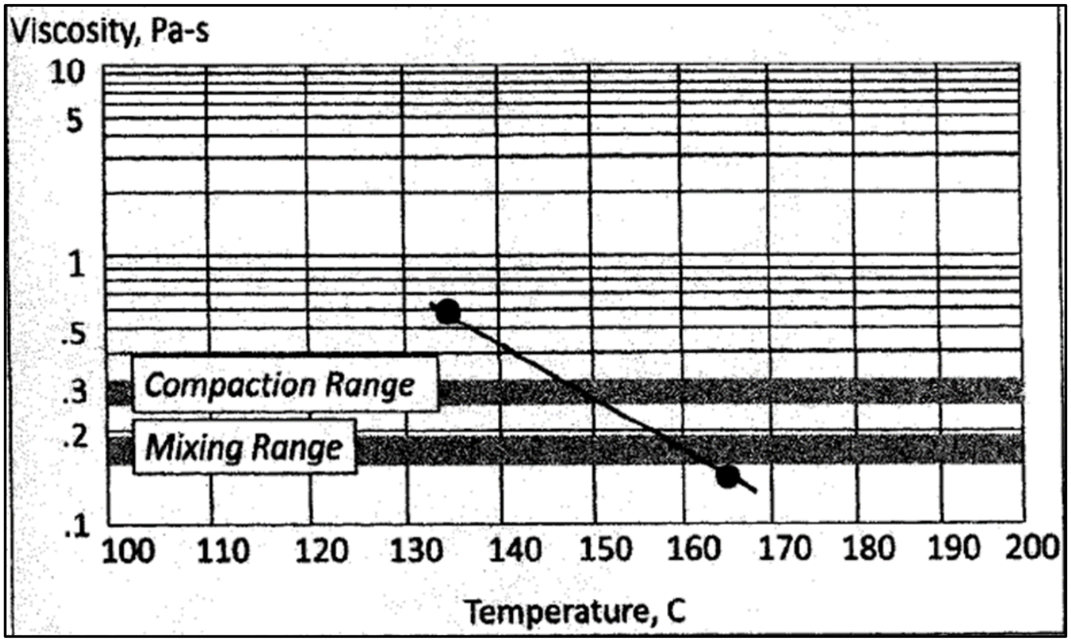
The different tubes used to measure viscosity are shown above.

The Viscosity Grading carries forward the original concept of the Penetration Grading, where field performance relative to plastic deformation at high temperatures and thermal cracking at low temperatures governed the viscosity grade selected noting that Alaskan refineries only produced AC-5. The Viscosity Grading System is such that as viscosity increases, the penetration decreases.

- The viscosity measured at 60°C (140°F) is the absolute viscosity (AASHTO T202) and uses a partial vacuum to induce flow through the capillary tube. This temperature is the standard average high pavement surface temperature, which makes the asphalt resistant to flow through an orifice.
- The viscosity measured at 135°C (275°F) is the kinematic viscosity (AASHTO T201) and uses gravity to induce flow through a capillary tube. This temperature is the standard average mixing and compaction temperature, which allows the asphalt to flow through an orifice under its own weight.

The Marshall mix design method uses equiviscous temperatures of unmodified asphalt to govern the mixing and compaction temperatures as shown on the following graph.

Viscosity vs. Temperature – Mixing and Compaction Range

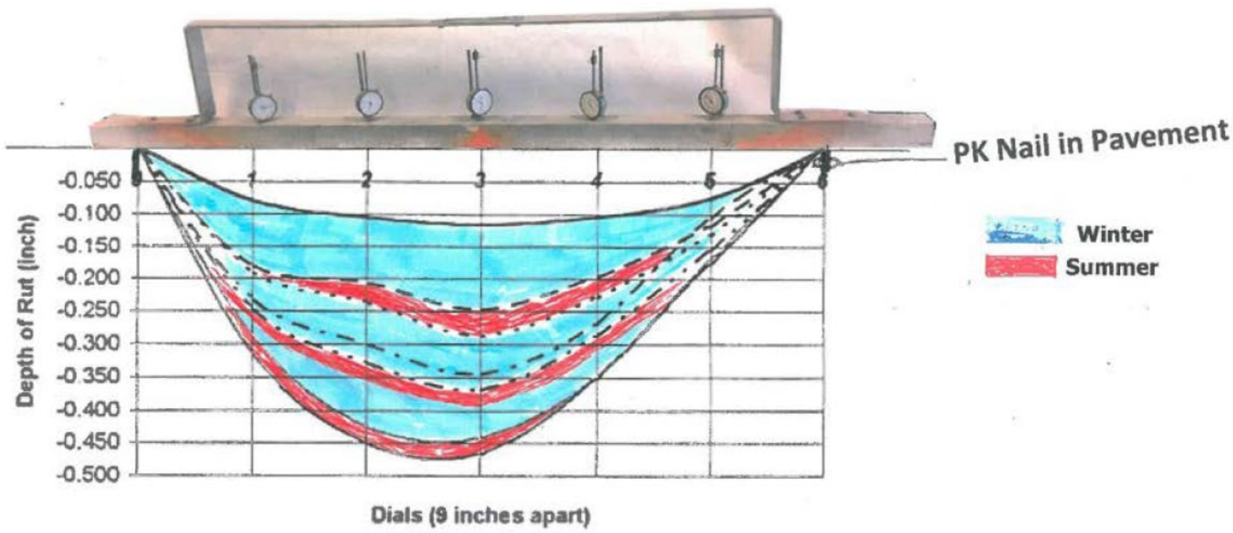


However, these mixing and compaction viscosities cannot be used for polymer modified asphalt (PMA), as polymers added to refined asphalt increase the viscosity of asphalt thus requiring the mixing and compaction temperature to be elevated too high to meet the viscosity specifications used for unmodified asphalt in the Marshall mix design method. These high temperatures will cause damage to the PMA and is not necessary for mixing and compaction of the PMA mix, so suppliers are requested to provide recommended mixing and compaction temperatures

B.1.1 Polymer Modification

Since the mid-1980s, rutting of the pavement in the wheel paths was visible and in the early 1990s, wheel ruts were being measured at a few locations in Anchorages using an aluminum beam with dial indicators, as shown below. PK nails were driven into the pavement on each side of a wheel rut to mark the location of the measurement. Typical rut measurements are shown and are taken in the fall when the use of studded tires is allowed and again in the spring when they were required to be removed thus defining winter and summer rutting.

Rut Measurement Using a Beam with Dial Indicators



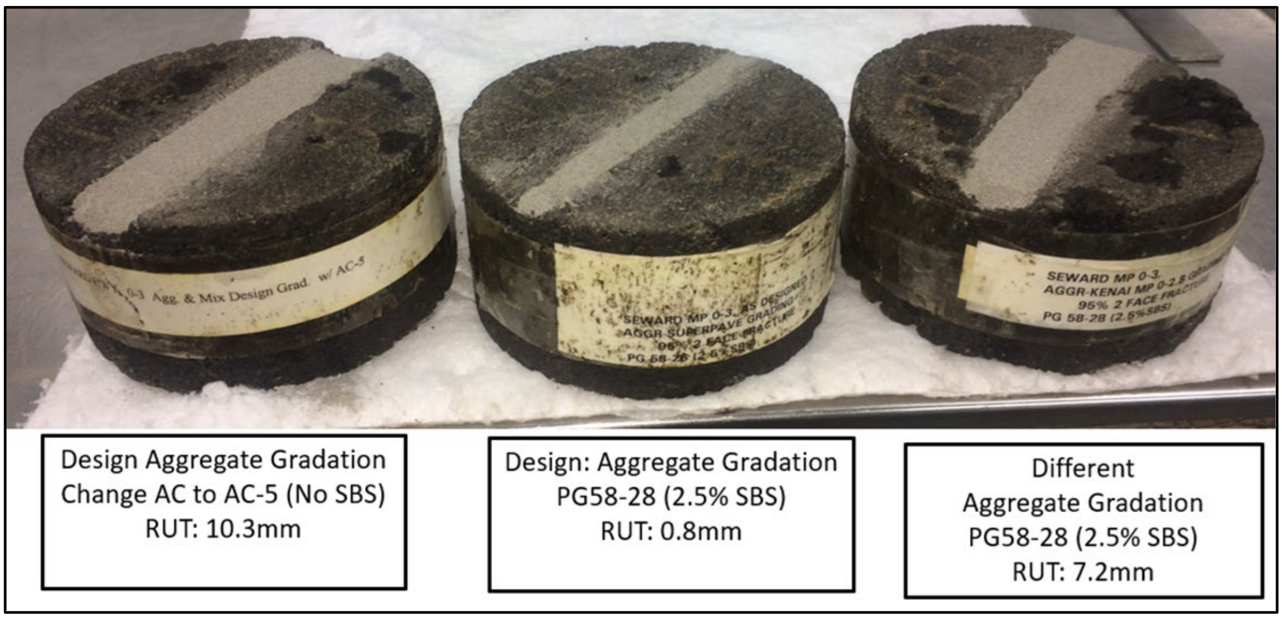
Rut depths increased in the winter months as well as in the summer months, predominately in Anchorage due to high volume, slow speed, heavy loads and traffic stopping at intersections. High pavement temperatures caused plastic deformation in the summer and studded tire wear increased the rut depth in the winter.

In 1994 it was noted that the nails that were driven into the pavement at the edge of the wheel path were being shoved laterally and upwards, indicating plastic deformation during the summer when pavement temperatures were highest. A 12-foot straight edge was laid across the driving lane revealing the PK nails in the pavement were approximately 1-inch higher than the pavement surface that was not driven on. This indicated that the "summer rutting" was not an erosion or loss of pavement but an upward displacement from plastic deformation.

To begin addressing this deformation issue the Central Region Materials Lab (CRML) purchased an asphalt pavement analyzer (APA) in 1995 so HMA could be tested for plastic deformation. To understand what factors would cause plastic deformation, HMA samples were prepared to compare a change in the binder, then a change in the aggregate gradation while using the same aggregate.

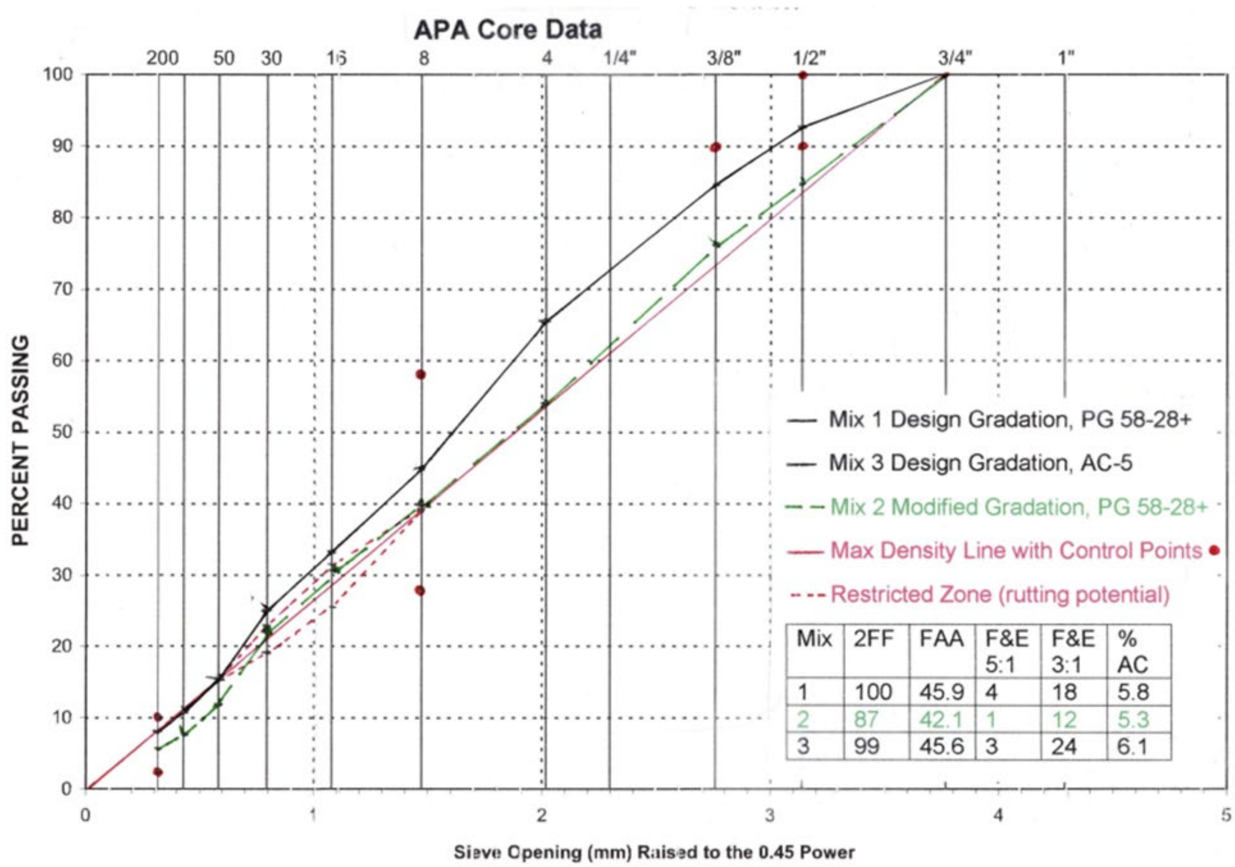
The APA was first used in 1995 to evaluate the cause of plastic deformation as seen below. A design gradation was mixed with unmodified AC-5 (left sample) and compared to a mix made with 2.5% SBS polymer modified AC-5 (center sample – PG Grade 58-28), while maintaining the same aggregate gradation. The aggregate gradation was then changed from the design gradation while maintaining the 2.5% SBS polymer (right sample). The results indicated that the SBS polymer modification of the asphalt binder drastically reduced the plastic deformation potential, but the use of a proper design gradation is still critical as seen with the rut formation in the right sample when SBS polymer was used.

APA Testing Results - 1995



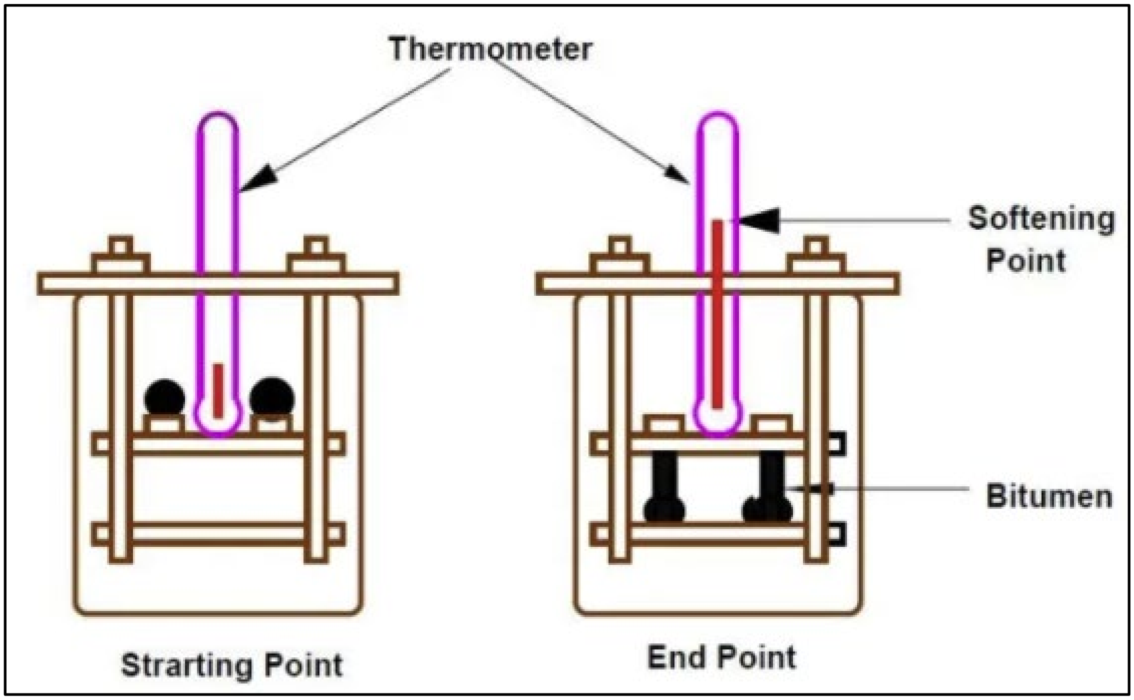
Note – dried sand was added to the ruts for increased visibility

To stop plastic deformation, Central Region Materials started to specify polymer modified asphalt (PMA) for urban Anchorage projects in 2000. The AC-5 specification had two additional test requirements added: Softening Point (AASHTO T 53) and Toughness & Tenacity (T&T) (AASHTO D5801). These tests required that polymer modification be made to the refined asphalt binder. The softening point temperature increasing due to an increase of the binder stiffness or viscosity and the resistance to deformation indicated in the toughness and tenacity test values where the energy (in inch-pounds) to cause deformation was increased. The T&T test also requires the polymer modified binder have an elastic property so it would not be a brittle failure, but an elongation with strength. This elastic property was desirable to make the binder more resistant to cold temperature thermal cracking.

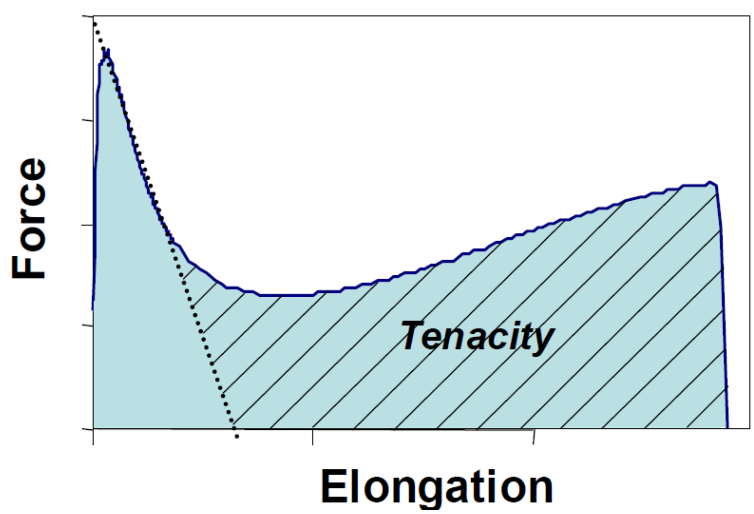


Softening Point Test. A steel plate is immersed in a beaker filled with water. The plate has two holes that have samples of asphalt placed over them with a steel ball placed on top of each sample. The temperature of the water is increased until the steel ball pushes the asphalt through the opening. This temperature is the softening point. Typically, unmodified asphalt AC-5 has a softening point less than 110°F; whereas polymer modified asphalt can range from 120° to over 176°F (the maximum temperature for a water bath). The Softening Point specification requires a temperature ≤ 125°F.

Softening Point Test Setup



<u>Toughness and Tenacity Test</u>. The end of a hemispherical pulling head is immersed in a cup of asphalt cement while it is liquid. After the asphalt is room temperature, the pulling head is pulled out of the asphalt in the cup at a uniform rate until it reaches 20 inches in length, as is shown in the figures below. The pulling force is measured and recorded during the elongation.



When the test is completed, the force vs the extension is plotted, with the area under the curve as the energy calculated in in-lbs. Toughness is the total area under the curve. Typically, unmodified asphalts have a very low tenacity.

Toughness and Tenacity specifications require a toughness ≥ 110 in-lbs, and a tenacity ≥ 75 in-lbs. The polymer modification increases the initial force as well as the tenacity portion of the curve during the elongation to meet the specification requirement, but high polymer modification or the type of polymers selected for modification can cause testing failures that affect the test results. To complete the test, the sample has to be pulled to achieve 20 inches of elongation, then the area under the curve will generate the T&T values. If the sample detaches from the pulling head or the strand separates, the test cannot be completed.

Alaska has used all three of the penetration and viscosity graded asphalt binders listed in the table below between 1975 and the mid 2000's, with AC-5 being the predominate binder used. AC-2.5 was used in the Northern Region to better resist thermal cracking in the winter and AC-10 was imported by rail to be tried in Anchorage to resist plastic deformation in the late 1990s. HMA made with AC-10 was placed in Anchorage on Gambell St. and Ingra St. and reduced the plastic deformation compared to mixes produced with AC-5, but not enough to justify continued use since Alaskan refineries could not produce AC-10.

Other states elected to use the aged residue grading (AR) viscosity grading. The aged residue grading of asphalt is the viscosity of asphalt after having been subjected to the rolling thin film aging test to simulate the oxidation (aging) of the asphalt cement in the plant mixing and laydown process of HMA construction.

The table below shows the evolution of Alaska's asphalt binder specifications to current use of the Performance Grading system.

Binder Specification Historic Changes

Penetration Graded	Viscosity Graded	Performance Graded (PG)
200-300	AC-2.5	
120-150	AC-5	PG 52-28
85-100	AC-10	

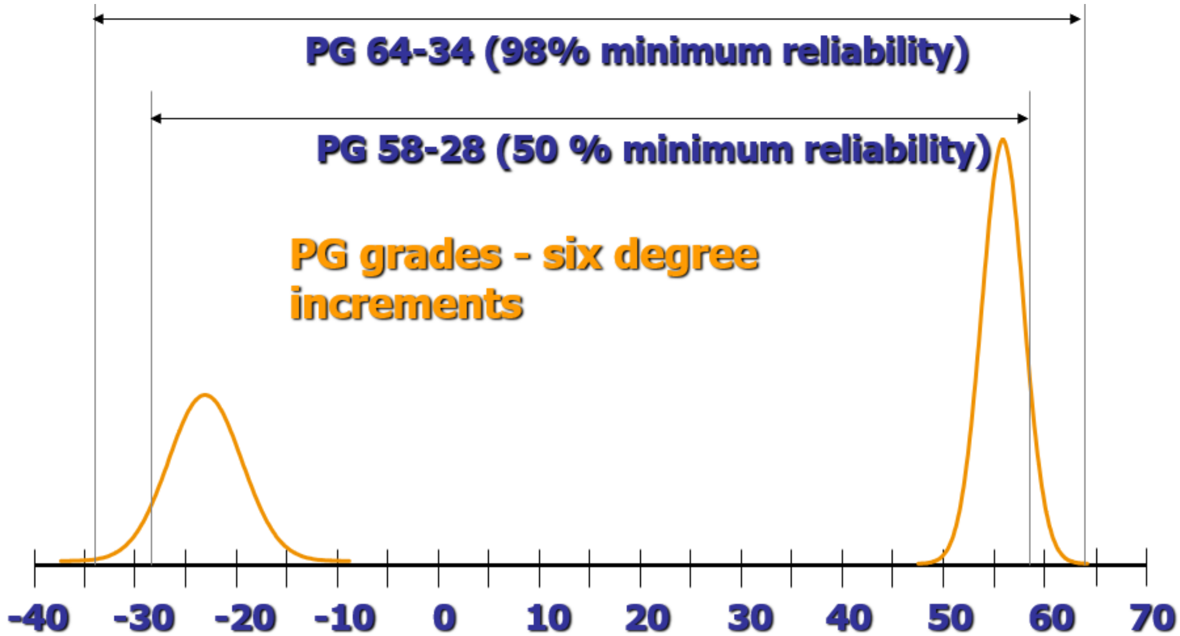
<u>Performance Grading (PG)</u>. In the early 1990s, FHWA recognized the viscosity grading system did not adequately address cold temperature cracking or plastic deformation at high temperatures and developed the **P**erformance **G**raded (PG) Asphalt Binder Specification AASHTO M 320.

Most states were adding additional requirements to the viscosity specification to achieve the performance needed. So FHWA defined asphalt cement performance at the 7-day average high pavement temperature and at the coldest 1-day low pavement temperature in degrees Celsius. For example, PG 52-28 indicates a performance graded asphalt for 52°C 7-day average high pavement temperature and -28°C minimum 1-day low temperature. Grades are in increments of 6°C as generally binder stiffness will change by a factor of 2 for each 6°C change. PG 52-28 is the standard asphalt cement produced at refineries in Alaska.

The Alaska Standard Specifications for Highway Construction in 2004 specified AASHO M320 PG asphalt cement. Elastic polymers were chosen over plastic polymers to reduce thermal cracking in the winter. While plastic polymers would achieve the desired reduction to plastic deformation, they would remain brittle during winter months and be prone to thermal cracking from low temperatures.

The following figure illustrates that the winter low and summer high temperatures data vary from year to year, illustrated by the temperature histogram on each end. So, to specify a binder for the extreme historic temperatures, the PG grades have to be increased; however, for certain projects in Alaska, a binder may not be available to accommodate local temperature extremes, so a binder is selected that is available to give the best performance

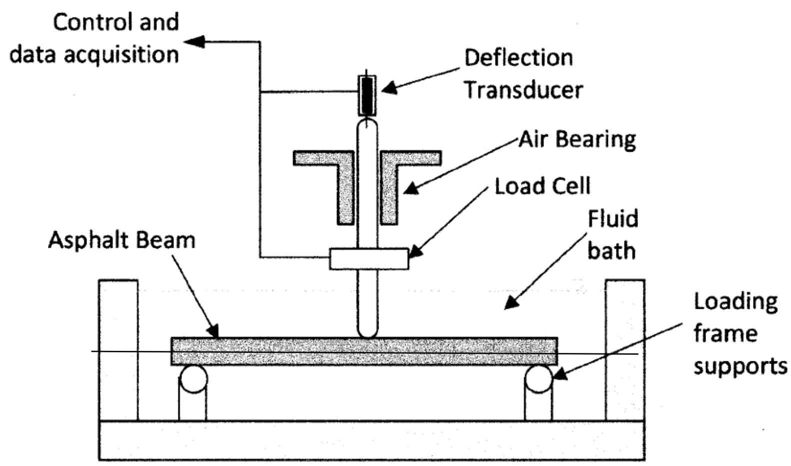
Ambient Pavement Temperature Generates PG Binder Grade

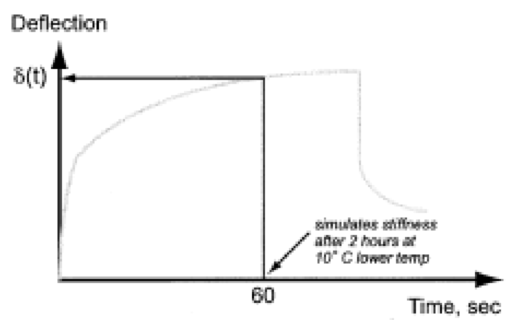


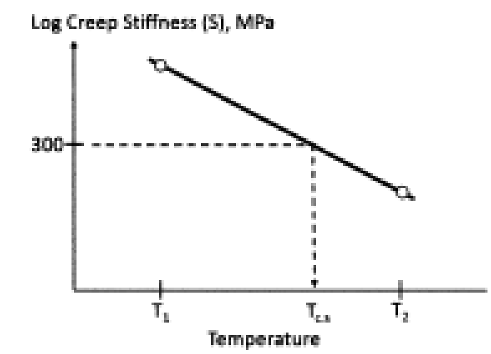
The SuperPave system has developed new test methods to determine asphalt performance at the extreme ambient temperatures. The high PG temperature is defined by rotational viscosity and shear resistance in a dynamic shear rheometer. The low temperature is determined by pressure/temperature aging a sample of asphalt to simulate 10 years of ageing. This aged sample is poured into mold creating a pencil sized beam. This asphalt beam is placed in a fluid bath on end supports, as shown in the figure below. A point load is applied to the center of the beam and the deflection is measured. This data determines the low temperature performance of the asphalt. The following displays the bending beam rheometer and the data generated.

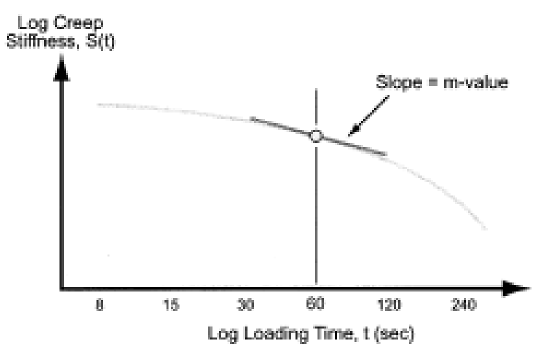
Low Temperature Test in Bending Beam

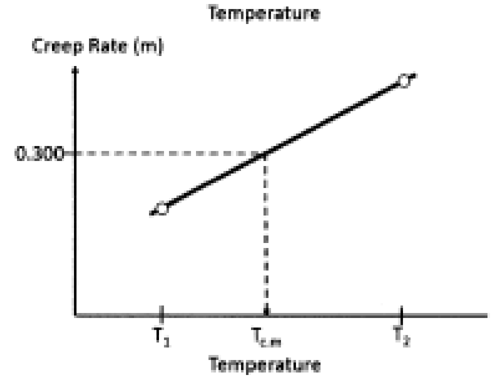
Stiffness from Applied load, Creep Rate from Deflection Curvature











There are two low temperature performance specifications, one for the loading T_{CS} and one for the bending / deflection T_{CM}. T_{CS} is the temperature when the log of Creep Stiffness is equal to 300 MPa. T_{CM} is the temperature when the Creep Rate is equal to 0.300. The temperature difference is

$\Delta T_C = T_{CS} - T_{CM}$ and is discussed later in this report as it relates to binder durability relative to ageing.

B.1.2 Effects of Grade Bumping on APA Results

Provisions in AASHTO M320 provide for increasing the PG high temperature to address plastic deformation; this typically requires additional polymer modification. For slower moving design loads, PG temperature would be increased 1 grade or a “bump” of 6°C. For standing design loads, the high PG temperature would be “bumped” 2 grades or 12°C. For certain situation of high volume, high design loading, and stopped or slow-moving heavy loads the asphalt could be “bumped” 3 grades or increasing the high PG temperature 18°C to resist plastic deformation.

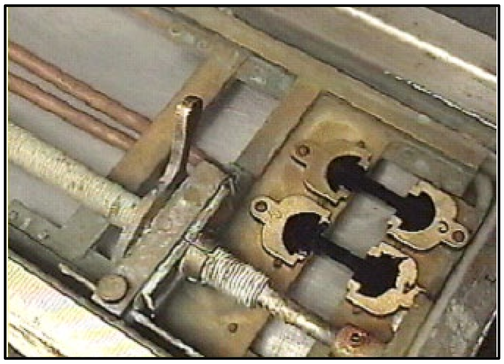
Thus, grade “bumping” was included in the specification with the thought that by specifying a higher PG temperature, it would create a stiffer binder to resist plastic deformation. Each “bump” raised the high temperature of the binder by increments of 6°C. So for PG 52-28 ambient conditions, one (1) “bump” would change the grade to PG 58-28, two (2) “bumps” to PG 64-28, and three (3) “bumps” to 70-28.

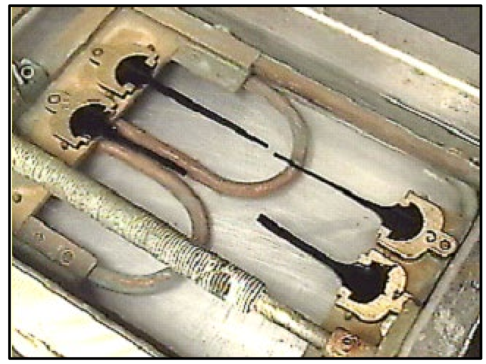
However, adding polymer to create these grade “bumps” can cause constructability problems, as additional polymer typically makes stiffer mixes and requires higher mix temperatures. This can make the mix hard to handle and compact, which can result in areas of low mat and joint density. In Alaska, grade bumping did not totally stop plastic deformation.

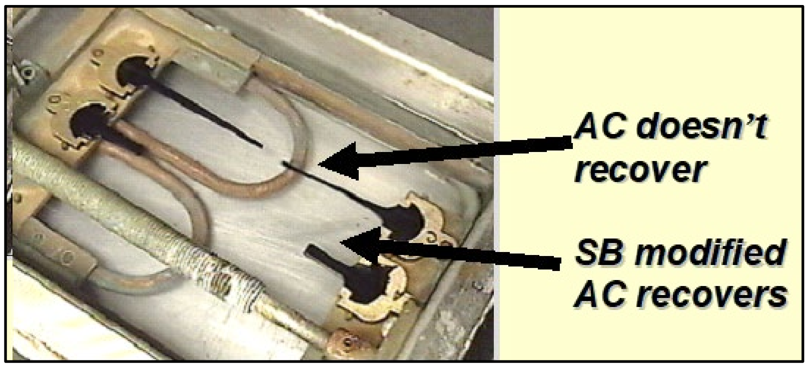
The Asphalt Institute notes in their M-26 handbook that M320 grade bumping has forced suppliers to use very soft base binders and high degrees of polymer modifications to meet the wide temperature ranges. Polymer modified asphalt suppliers in Alaska do not have tankage to support all of the potential PG binders created by grade bumping, so an attempt was made to standardize PG specifications.

States added additional specifications to PG binders such as Softening Point, Elastic Recovery, Force Ductility, T&T and other tests.to achieve specific properties needed to provide the desired binder performance and resist plastic. This is termed “PG+” specifications.

The softening point and toughness and tenacity tests illustrated previously were selected in Northern and Central Regions, with Southcoast Region choosing to use elastic recovery to define the elastic polymer properties.





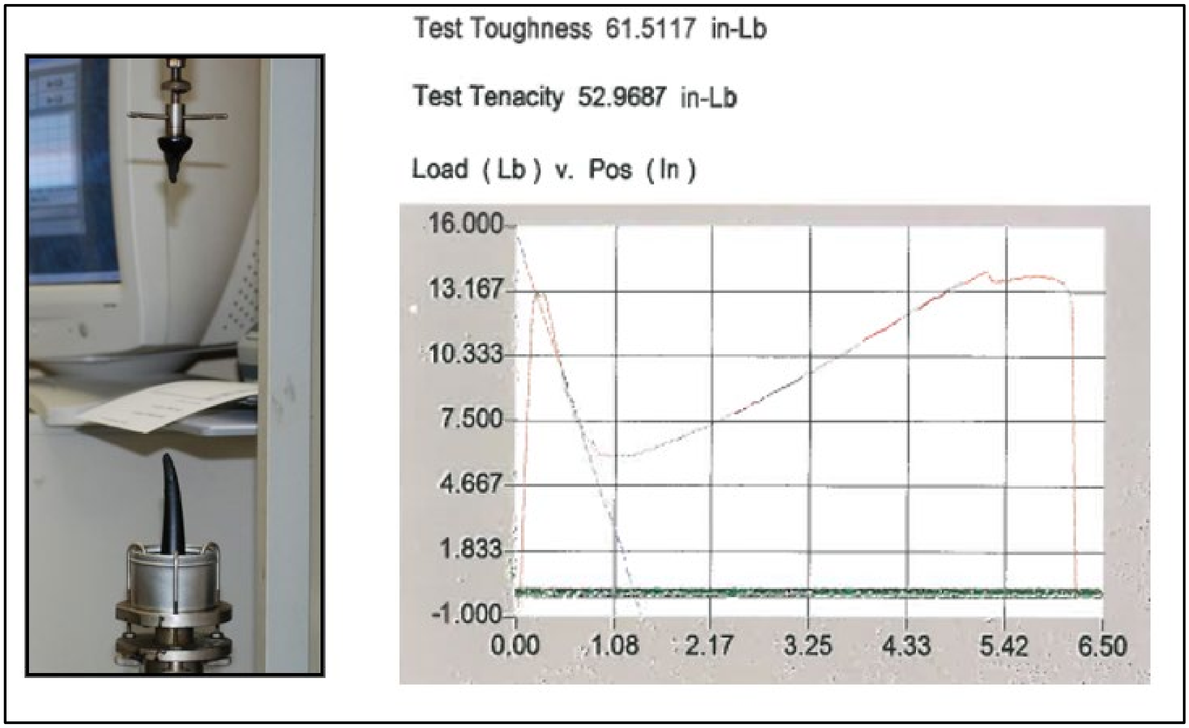


The above figure shows asphalt being tested for Elastic Recovery. Asphalt binder is heated, poured into a mold, allowed to cool and then pulled in a water bath at room temperatures until the binder had elongated about 8 inches. The sample is cut in the middle and is allowed to contract for one hour. The distance between the ends of asphalt is measured to calculate the percent elastic recovery to the original position. The photo in the top left is prior to elongation, top right after they are cut, and the bottom after they have had time to recover. The unmodified sample did not recover while the sample with SBS polymer did.

There are deficiencies in the Toughness & Tenacity (T&T) and the Elastic Recovery (ER) tests. Both tests are conducted at room temperature, not at the PG high temperature.

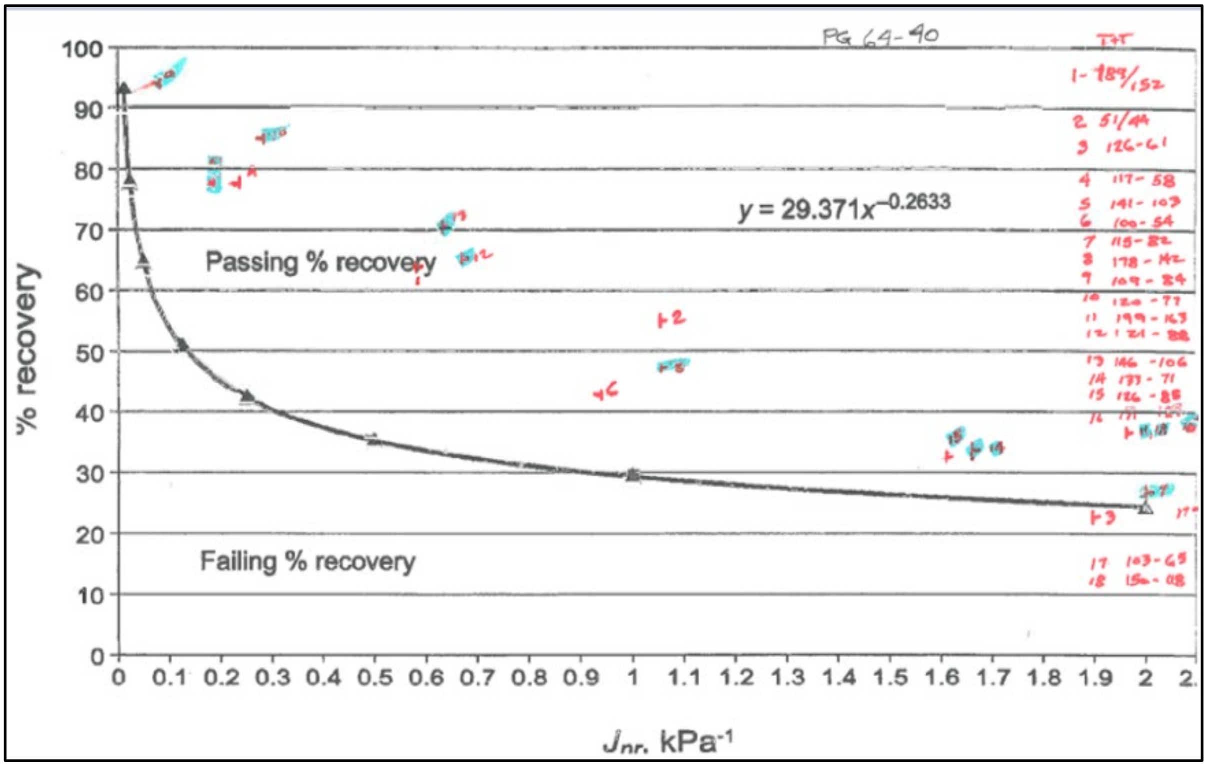
The specifications used for the T&T test were 110 in-lbs for Toughness and 75 in-lbs for Tenacity. When Alaska suppliers provided highly polymer modified asphalt, samples "failed" by detaching from the pulling head or snapping in the middle during elongation. This reduces the area under the curve, and results in a failed test.

Failed T&T Test



However, a highly polymer modified asphalt failing in the T&T test does not necessarily indicate it will perform poorly in the field. Failing binder was tested using MSCR procedures in AASHTO M 332 and were found to have very high elastic recovery (%R) and low residual deformation (J_{NR}) at PG temperatures.

T&T test values were plotted against the MSCR graph for Passing / Failing % Recovery and demonstrated highly polymer modified binders have passing results on MSCR even if they failed the T&T.



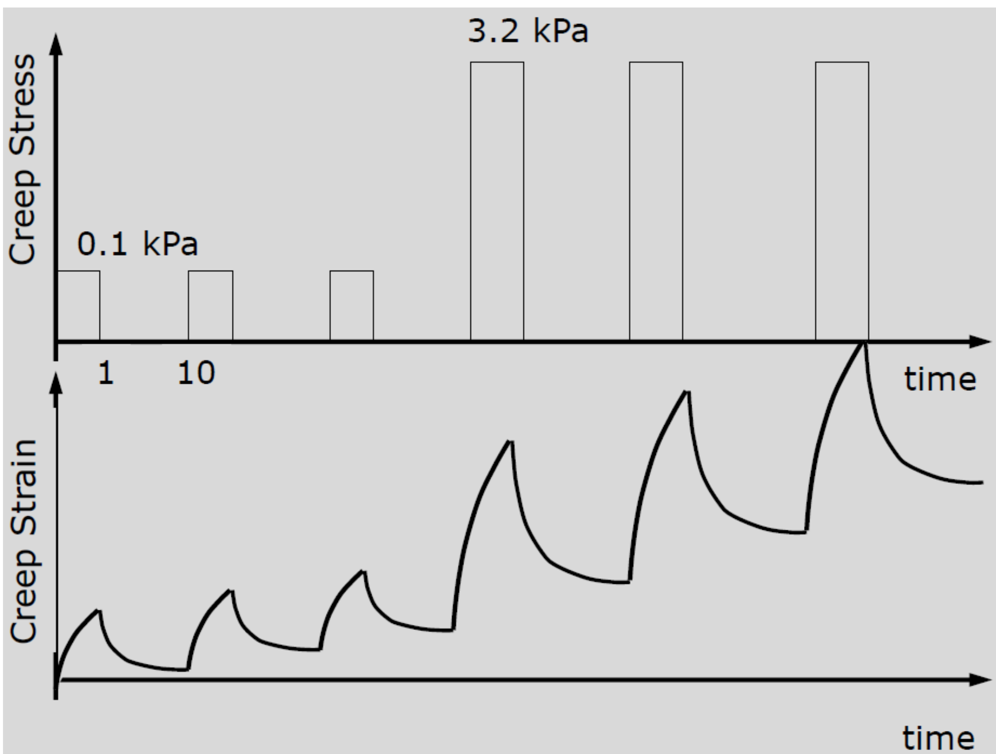
<u>Performance Grading Plus (PG+)</u>. The short comings of the AASHTO M 320 PG system is that it did not always prevent high temperature plastic deformation under heavy, parked, or high traffic loading. FHWA and University of Wisconsin were evaluating binder testing with the dynamic shear rheometer and developed Multiple Stress Creep Recovery, which ultimately became AASHTO M 332. Field testing validated the binder performance parameters defined by residual deformation (J_{NR}) and percent of elastic recovery (% R).

So in lieu of grade bumping, the Asphalt Institute (MS-26 The Asphalt Binder Handbook), recommends using Multiple Stress Creep Recovery testing specified in AASHTO M332. The following notes were made in one of Mike Anderson's (Asphalt Institute's Director of Research and Laboratory Services) presentations on August 2011 of research work done with FHWA.

- All testing should be performed at the environmental grade temperature
- The standard grade should be based on the J_{nr} value of existing neat binders (4.0 kPa^{-1})
- For high traffic reduce the J_{nr} value by half at the grade temperature to 2.0kPa^{-1}
- For very high or standing traffic the J_{nr} value should be reduced by half again to 1.0kPa^{-1}
- For extreme traffic the J_{nr} value should be reduced by half again to 0.5kPa^{-1}

The AASHTO M 332 specification modified AASHTO M 320 to use Multiple Stress Creep Recovery at the high ambient PG temperature to better define and measure the amount of elastic recovery and permanent deformation of an asphalt binder in the Dynamic Shear Rheometer.

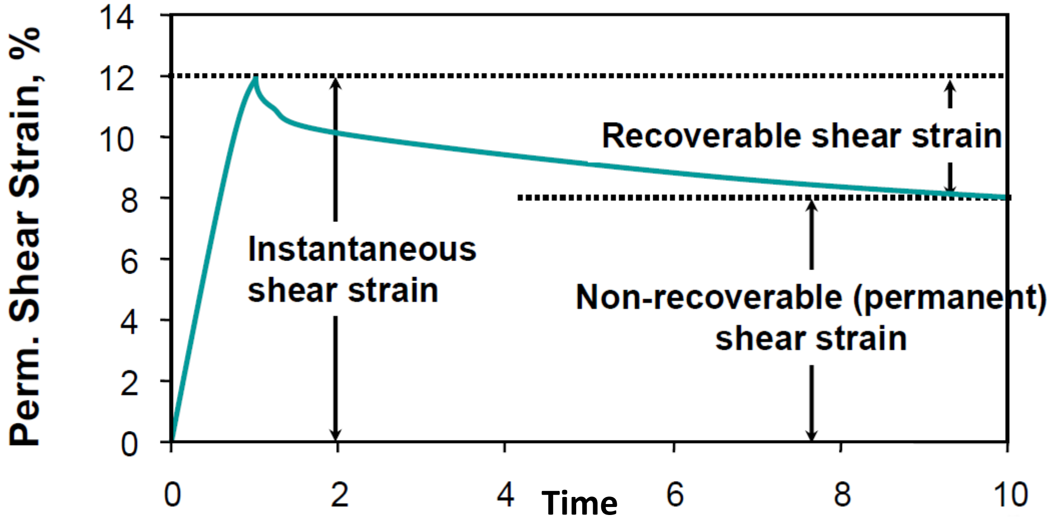
In the MSCR test the first 10 loading cycles are at a low stress loading (0.1 kPa torque), and in the next 10 cycles the applied loading is increased to 3.2 kPa leading to the significant increase in strain (sample deformation). Torque is applied to the sample for 1 second, then relaxed for 9 seconds to complete a cycle. Typically, the sample will not fully recover but have some residual deformation, which is reported as the J_{nr} value of the binder.



B.1.3 Multiple Stress Creep and Recovery Testing

In each loading cycle there is instantaneous shear strain, recoverable shear strain and unrecoverable shear strain as observed below (figure from Asphalt Institute).

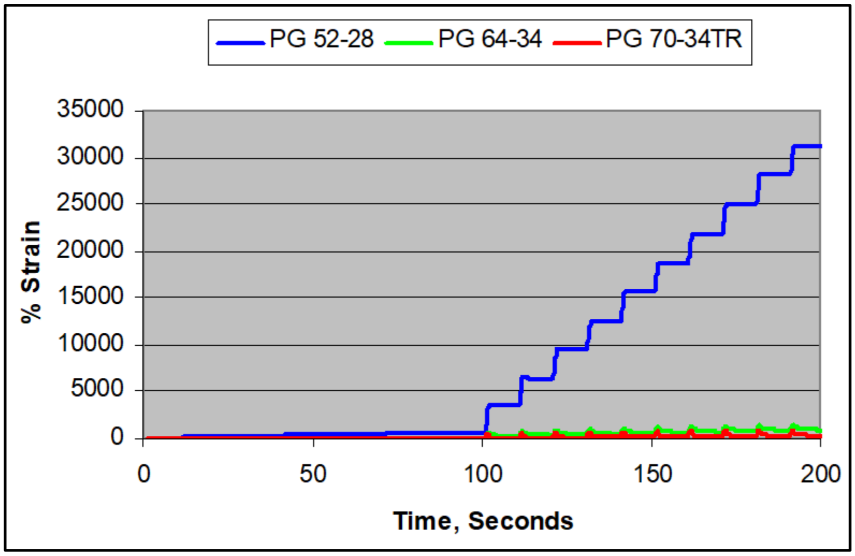
Shear Strain in MSCR Testing



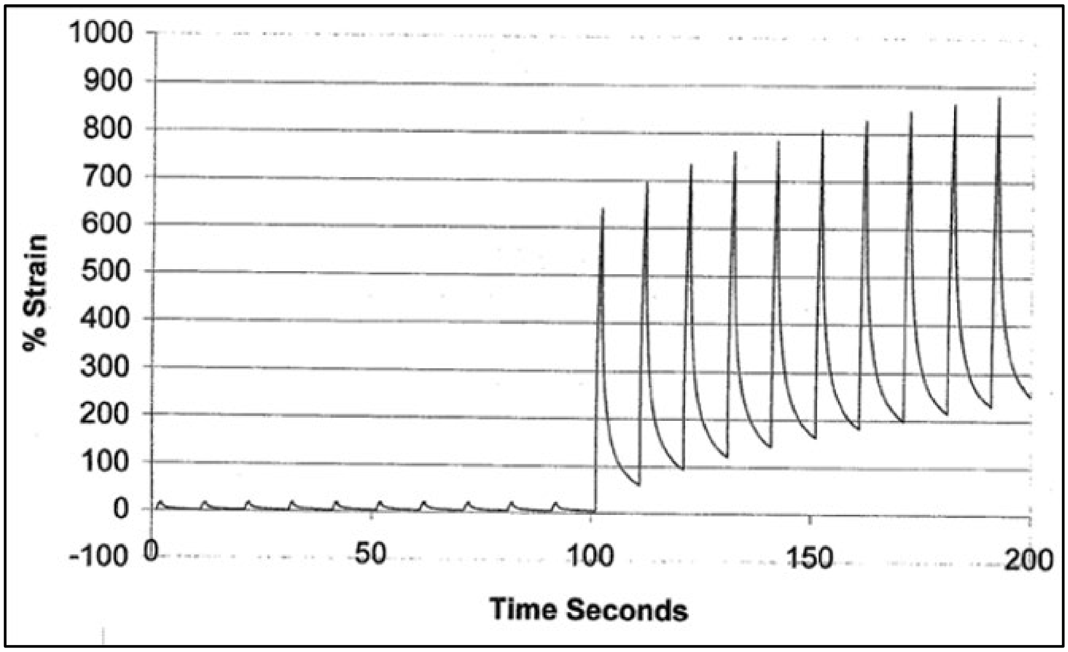
The non-recoverable compliance, or J_{nr} value, is calculated as the unrecovered (permanent) shear strain divided by the applied shear stress. The MSCR J_{nr} value addresses the high temperature rutting (plastic deformation of pavements), while the percent recovery value from the test identifies how polymer is working in the binder (high recovery indicates acceptable results).

This test is illustrated in on the next page, where MSCR test results performed on asphalt binders in 2007 at the Alaska DOT&PF Central Region Materials test lab are graphed. Note that PG 52-28 is not polymer modified while the PG 64-34 and the PG 70-34 are highly polymer modified asphalts.

MSCR Strain of 3 Binders



MSCR Testing of 64-34 Binder



MSCR Testing of 64-34 Binder is an enlargement of the PG 64-34 from the top graph where the % strain (accumulated elongation) is less than 300% and the elastic recovery is approximately 620% compared to the extreme strain observed by the unmodified PG 52-28 binder.

B.2 Historic Aggregate - Alaska DOT&PF HMA Specifications Review

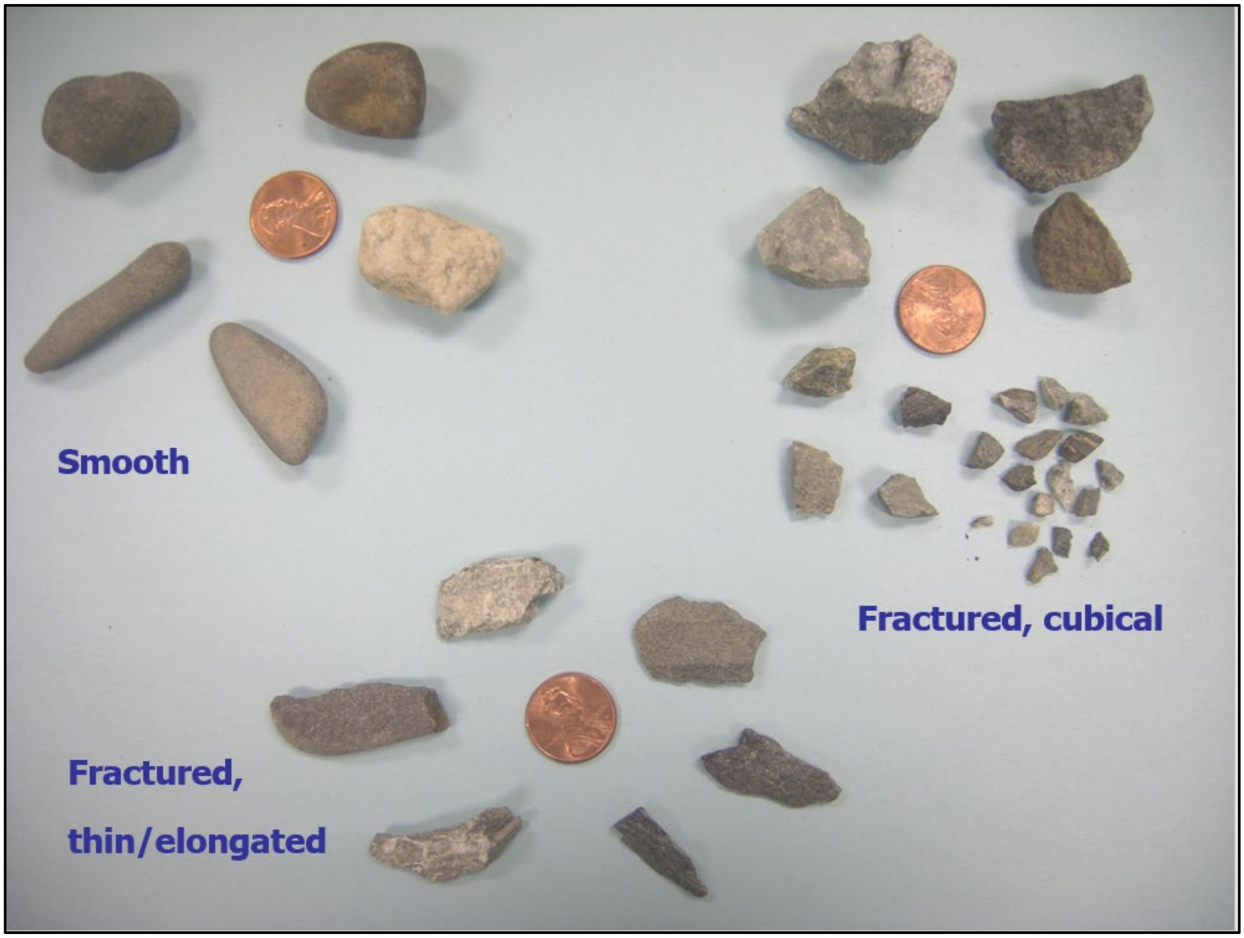
Aggregate in hot mix asphalt is approximately 95% of the mass of the total mix and is bound together by asphalt binder. The following is a table of modifications made over the past 50 years to Alaska Highway specifications for Type II, Class A HMA in Section 703.

DOT Spec Date	1972	1981	1988	1998 (metric)	2004	2017
LA Wear % max			45	45	45	45
Deg min		45	30	30	30	30
Sodium Sulf. loss max (5 cycles)	9	9	9	9	9	9
Fracture min	50% 1FF	70% 1FF	70% 1FF	80% 1FF	80% 1FF	90, 2FF
Flat-Elong % max (1:5)				8	8	8
Absorp. % max						2.0
Fine Agg. Angularity						45 % min (SuperPave)
½" %	-			75-90	75-90	75-90
#4 %	45-65	45-65	45-65	33-70	33-70	33-70
#200 %	5-10	3-10	3-10	3-7	3-7	4-7
Nordic Loss						8% max if hard aggr. required

The historic aggregate quality test procedures for hot mix asphalt (HMA) have not changed; however additional quality requirements have been added.

The ideal aggregate particle has a highly fractured cubical shape that is defined by the percentage of fractured faces and the 1:5 Flat- Elongated ratio (ATM 306) specified in Superpave. The following photo demonstrates how the specifications require the fractured, cubical aggregate shown in the top right. The fracture specification requires the aggregate as shown in the bottom portion of the photo, but by itself would allow for thinner/elongated aggregates. Adding in the flat-elongated specification requires cubical aggregates, as are desirable.

Aggregate Fracture Properties



As the fracture requirement of the coarse (+#4) aggregate is increased, there is also an increase of fractured fines from crushing that will increase the internal friction of the gradation as demonstrated in the Fine Aggregate Angularity test used in Superpave mixes.

The aggregate gradation plays an important role in the performance of the mix. Typically, the best performing aggregate gradation follows the maximum density where there is maximum aggregate packing but leaves adequate voids for asphalt binder as noted by the Voids in Mineral Aggregate (VMA). Rut testing on the APA or Hamburg will proof test mixes for aggregate grading performance.

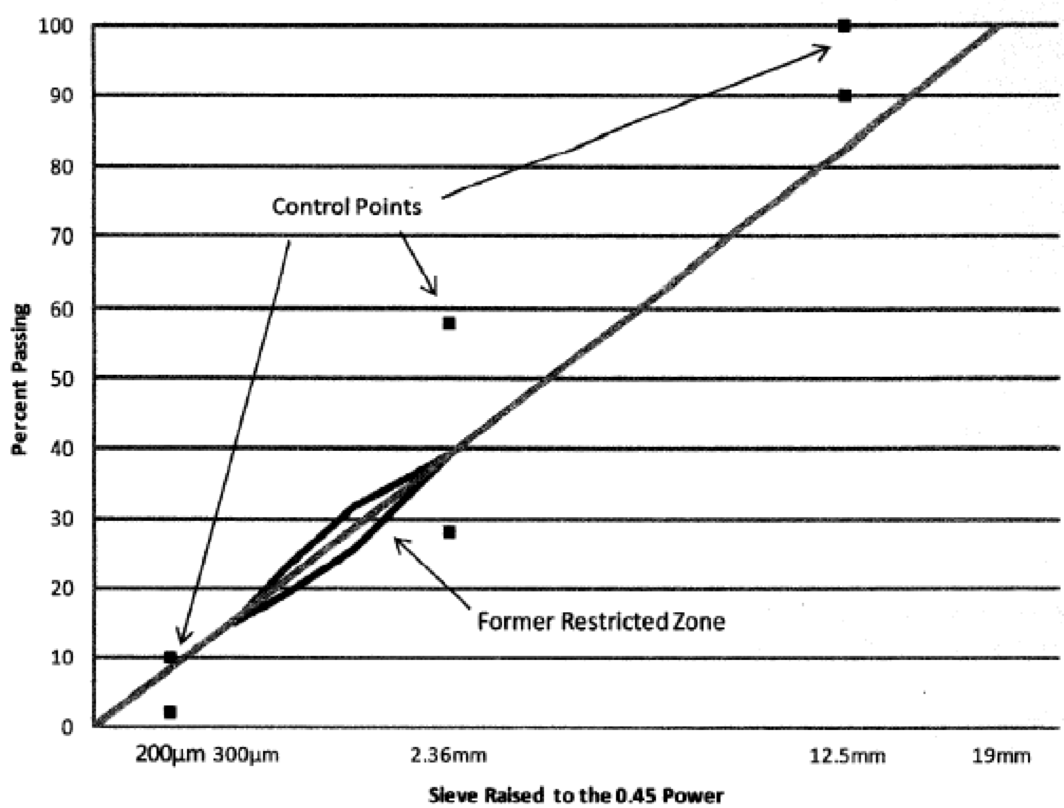
B.2.1 HMA Aggregate Gradations

In the early 1990's a Type I gradation was tested in place the standard Type II gradation on HMA mixes. Type I gradation has a larger maximum aggregate size, and it was thought that the larger aggregate may better resist the abrasion from studded tire wear. However, the Type I mix was more expensive and there was not an observed improvement to rutting when compared to the standard Type II gradation, so the Type II gradation continued to be used.

The Superpave aggregate gradation was defined by the maximum density line on a 0.45 power gradation curve, and was used with control points for the gradation boundaries and a "restricted zone" was added for the fine aggregate portion of the gradation (#8-#50 sieve). This restricted zone indicated that aggregate gradations crossing through it would have the potential of producing a mix that was prone to plastic deformation; however, the restrictive zone has been removed from current specifications and mixes can be designed where the gradation passes through this zone without causing rutting if they include highly fractured aggregates.

Superpave mix designs are developed using AASHTO R 35-17 and M 323. In the figure below the aggregate gradation band is displayed showing the maximum density line, the specified control points and the restricted zone that was initially specified but has since been removed from the AASHTO aggregate gradation specification.

Superpave Gradation Boundaries



Gradation Control Points for 12.5 mm NMAS (19 mm MAS) with Former Restricted Zone

Superpave mix design specifications also added the requirement for fine aggregate angularity (FAA) AASHTO T304 "Uncompacted Void Content of Fine Aggregate" which measures the uncompacted, volumetric bulking of crushed fines due to the internal friction of crushed fines (passing # 8 sieve). Crushed fine aggregate has greater particle interlock or internal friction than natural fines, which improves resistance to rutting.

B.2.2 Hot Mix Asphalt Specification Review

The Alaska DOT&PF Regional Materials Labs develop the Hot Mix Asphalt (HMA) mix designs for both highway and aviation construction projects. The project contractor provides samples of the aggregate, asphalt binder, and antistrip additives that conform to project specifications, and selects the aggregate gradation within the specified aggregate broad band on which the state lab develop their HMA mix designs. Subsequently during project construction of HMA pavement, the contractor is required to produce HMA that conforms to the aggregate gradation, asphalt content and the density of the mix design developed in the state lab. The state labs then perform quality assurance testing of the field HMA acceptance tests, as required by the federal funding agency.

In recent years, private labs that have AASHTO accreditation have developed HMA designs that were submitted to the DOT&PF Regional Materials Engineer for review and approval to use on projects. The cost of a mix design developed by a private lab was paid by the contractor whereas any mix design developed by DOT&PF was no cost to the contractor.

Marshall hot mix design procedures have been used as specified in the Asphalt Institute's MS 2 Asphalt Mix Design Methods. Since Alaska DOT&PF designs HMA for highways and airports, the Marshall mix design procedure has been specified for most projects with the exception of Superpave mix designs that are specified for mixes on roads and airports with high volume and high design loads in Southcoast and Central Regions.

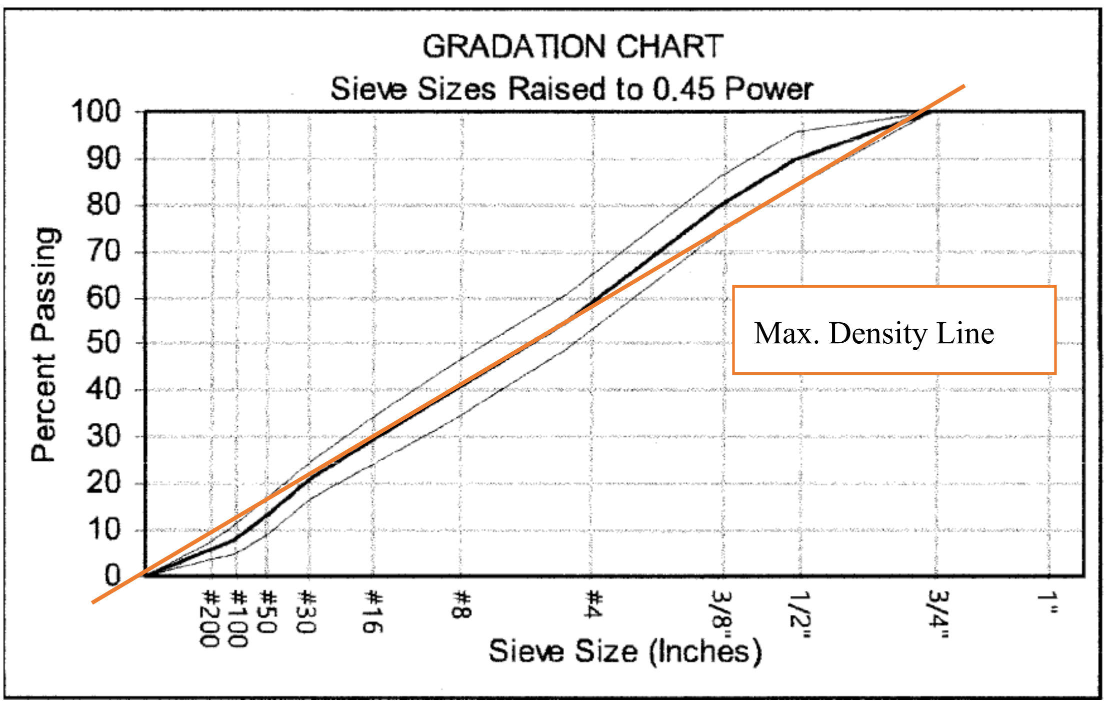
Historically the optimum total asphalt content for a mix design was the average of the asphalt contents at maximum stability, 4%voids total mix, and maximum unit weight of the mix.

Determination of the optimum asphalt content has changed, and now the effective asphalt content is used in lieu of the total asphalt content:

- The optimum effective asphalt content is initially selected at 4% voids total mix. Alaska DOT&PF requires a minimum of 5.0% asphalt in all mixes. The DOT&PF Regional Materials Engineer can then adjust the asphalt content to optimize the mix parameters.
- Other parameters are checked to verify that they comply with specification:
  - VTM (Voids Total Mix)
  - VFA (Voids Filled with Asphalt)
  - VMA (Voids in Mineral Aggregate)
  - Stability / Flow
  - Unit Weight
  - Dust/Asphalt Ratio
  - Rut test if required (APA)

When polymer modified asphalt is being used, the only Marshall parameter that has changed is the flow (amount of deformation in the stability test). The flow is allowed to exceed specifications due to higher stabilities (compressive force).

The aggregate gradation for dense graded mixes typically stays within the DOT&PF specified gradation broad brand for specified sieves and follows the maximum density line discussed in the Asphalt Institute MS-2 “Asphalt Mix Design Methods”. Generally, more fine aggregate passes the #4 sieve than there is retained on it for dense graded mixes. The graph below illustrates this.



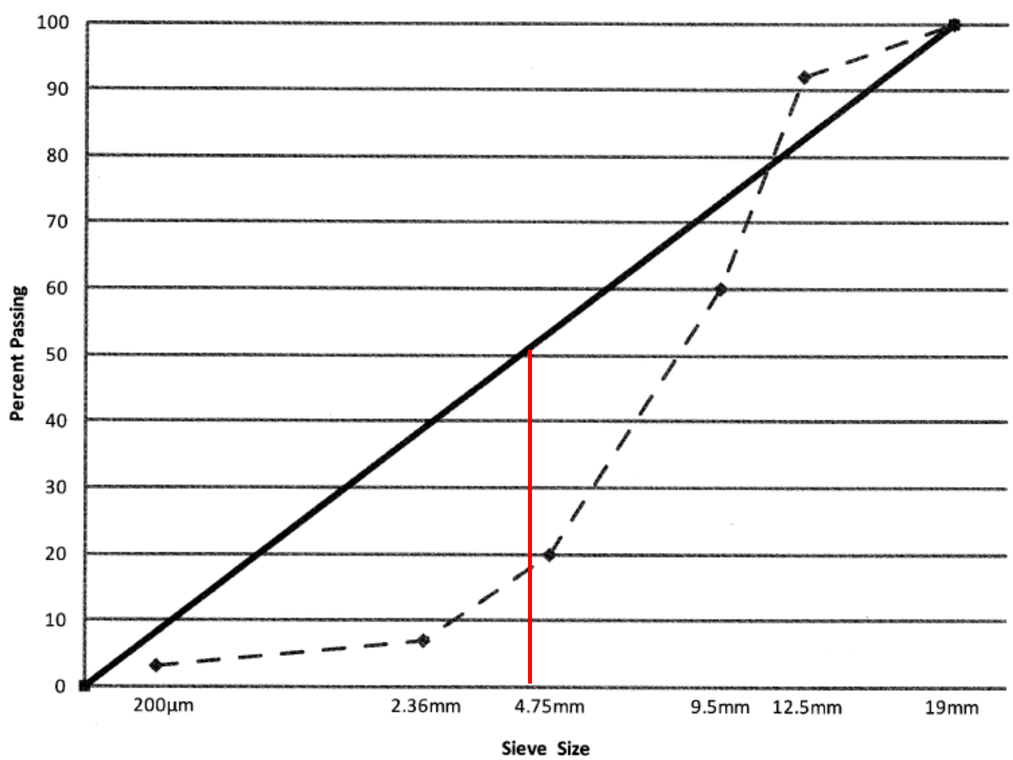
B.2.3 Crumb Rubber Mixes

In the mid-1980’s gap graded mixes were used when the Federal government mandated that crumb rubber be used due to the crude oil embargo imposed on the US. Several projects used “Plus Ride” procedures to add dry crumb rubber particles into the aggregate gradation. Gap grading provides more voids for the crumb rubber particles and increases the asphalt content. Fairbanks and Anchorage each paved several roads with crumb rubber mixes with reasonably good success. Notably Danby Road in Fairbanks and the A-C Street couplet in Anchorage performed well. The A-C Street couplet was paved in 1986 and lasted until 2019 indicating a good resistance to studded tire wear as both A and C Streets have high traffic volumes.

Based on the success of the A-C Street couplet project from 1986 other projects used crumb rubber mix, including the Glenn Highway between Airport Heights in Anchorage and the Glenn-Parks Highway Interchange (2010), the Seward Highway between mileposts 115-124 (2011), Abbott Loop Extension (Elmore Road) (2008), and East Dowling Extension (2010). None of these projects experienced the same level of performance the A-C Street couplet. The Glenn Highway project had significant issues with mid-layer delamination in the wheel paths throughout the project area. A crumb rubber mix has not been used since.

B.2.4 Stone Mastic Mix

In the late 1990's, after the Type I gradation was evaluated, Stone Matrix Asphalt mix (SMA) was introduced in Central Region to provide more large aggregate in the mix to reduce studded tire wear and provide a resistance to plastic deformation. Note that on the #4 sieve 42% of the total aggregate is passing, or 58% is larger than the #4 sieve (solid red line). Normally the aggregate gradation for dense graded mixes follows the maximum density line (solid black line) as noted below.



The voids in the SMA gradation allowed for high percentages of asphalt cement to be used, but it would drain down through the mix unless filler material was added. When polymer modified asphalt was used in SMA mixes there was no longer any drain down issues.

The SMA gradation provided a rock-on-rock HMA structure for high resistance to deformation. The problem encountered with SMA mixes was that studded tires would pick out the binder matrix around the large aggregate on the pavement surface and then the large aggregate would ravel out, which led to the pavement surface rutting in the driving lanes in a short period of time. The photo below shows raveling in the wheel paths of an SMA mix where rut depths exceed 2” in 2020 on the Parks Highway in Wasilla that was constructed in 2005.

It is possible some of the issues Alaska encountered with SMA mixes were due to mat density, aggregate size, or fracture. Given many states and European countries have great success with SMA mixes on their high-volume roads this mix may be worth reevaluating for use in the Anchorage area.

B.2.5 SuperPave Mix

In the 1990’s, FHWA developed a new mix design procedure called Superpave and gave each State the equipment to implement this new pavement design technology instead of using the Marshall or Hveem mix design procedures in the Asphalt Institute’s MS-2 handbook. In the

Superpave process the HMA was compacted with a gyratory compactor instead of dropping a Marshall hammer to compact the mix.

Excerpt of AASHTO M 323 Table 7 that gives the equivalent mix design, Superpave gyratory vs 75 blow Marshall.

20 yr ESALs (Million)	% max. theoretical density (% Gmm)			Voids in Mineral Aggr. (VMA) Nominal Max Aggregate size (mm)						VFA, % range	Dust/AC Ratio range
				37.5	25.0	19.0	12.5	9.5	4.75		
	$N_{initial}$	N_{design}	$N_{max.}$								
Gmm for 0.3 to <3 ESALs	≤ 90.5	96.0	≤ 98	11.0	12.0	13.0	14.0	15.0	16.0	65-78	0.6-1.2
	7	75	115	Gyrations for N values							
Marshall A		75 blows, 96.0 % G_{mm}		(Same VMA values)						65-75	0.6-1.4

Initially using the Superpave HMA mix design process yielded about 0.5% lower asphalt contents for mix designs compared to the same mix using the Marshall mix design procedure. This was due to high number of gyrations (N_{design} = 100) specified in the initial Superpave design. States using this original Superpave procedure started to have pavement failures, so the number of gyrations specified has been reduced to N_{design} = 75. The typical failure was bottom-up cracking from fatigue failure, which was overcome by requiring a higher asphalt content in mixes.

Alaska DOT&PF is using the revised Superpave mix design procedures for some high-volume roads and airports with jet traffic, while continuing to use the Marshall mix design method for the majority of roads and airports. The current Superpave mix design specification generates a design asphalt content that correlates well with the Marshall 75 blow mix design procedures as both procedures focus on designing for 4% voids in the mix. Currently consultant labs do not have equipment to do Superpave mix designs.

Some modifications Alaska made to the standard Superpave mix design procedure are:

- Alaska still uses an aggregate gradation specification band instead of just control point on a maximum density gradation curve as HMA acceptance is based on conformance to the mix design aggregate gradation
- Alaska requires a minimum of 5% asphalt content in mixes

- Increasing the fracture requirement to generate mixes that resist plastic deformation rutting when tested in the APA or Hamburg rut tester that are now required by FAA specifications
- Typically, Alaska requires the use of polymer modified asphalt with high PG temperatures in Superpave mixes to resist plastic deformation

APPENDIX C – HAMBURG DATA

The data from Hamburg tests performed for this project is provided in the work cards produced from the lab testing, with the asphalt contents, Mix SpG, Bulk SpG and VTM with the graph of rut depth vs. cycles.

Stripping was observed on two field samples, being from the Parks Highway MP 146-163 and the Glenn/Parks Interchange at 12,000 cycles. The only RAP used in the HMA was on the Seward Highway MP 75-90 Project at 15%.

Mix	Binder	Voids	Rut @ 10,000 Cycles (mm)	Rut @ 500 Cycles (mm)	Test Temp (C)	Project Number	Project	Lab vs. Field
Type II Class A	PG52-34	4.4	9.3	3.3	45	53469	Parks Highway MP 52-57	Field
Type II Class A	PG52-40	4.6	5.5	2.4	45	53750	Funny River Road	Field
Type II Class A	PG58-34	5.9	9.7	3.4	50	54147	Parks Highway MP 146-163	Field
Type VH	PG58-34	2.7	6.3	2.7	50	50816	Seward Highway Dowling to Tudor	Field
Type VH	PG58-34	1.9	4.6	0.6	50	CFHWY00162	Seward Highway Dimond to Dowling	Field
Type VH	PG64-40	4.3	4.9	0.4	50	CFHWY00106	Minnesota Drive	Field
Type VH	PG64-40	6	10.6	2.2	50	58571	Glenn/Parks Interchange	Field
Type VH	PG64-40	2.6	7.2	0.3	50	57835	A and C Street	Field

Mix	Binder	Voids	Rut @ 10,000 Cycles (mm)	Rut @ 500 Cycles (mm)	Test Temp (C)	Project Number	Project	Lab vs. Field
Type II Class A (15% RAP)	PG52-40	2.3	3.3	1.0	45	CFHWY00213	Seward Highway MP75-90	Lab
Type II Class A (15% RAP)	PG52-40	5.2	6.5	1.6	45	CFHWY00213	Seward Highway MP75-90	Lab
Type II Class A (15% RAP)	PG52-40	7.1	10.3	2.1	45	CFHWY00213	Seward Highway MP75-90	Lab
Type II Class AH	PG52-40	2.6	1.6	0.7	45	51829	Palmer Wasilla Highway	Lab
Type II Class AH	PG52-40	7.4	3.9	2.0	45	51829	Palmer Wasilla Highway	Lab
Type II Class AH	PG52-40	11	6.8	3.0	45	51829	Palmer Wasilla Highway	Lab
Type II Class AH	PG64-40	1.8	1.2	0.7	50	CFHWY00317	Glenn Springer	Lab
Type II Class AH	PG64-40	5.5	2.6	1.3	50	CFHWY00317	Glenn Springer	Lab
Type II Class AH	PG64-40	8.1	4.3	2.1	50	CFHWY00317	Glenn Springer	Lab
Type VH	PG52-40	2.6	0.9	0.2	45	59778	K-Beach MP 16-22	Field (Lab Redo)

State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
A-C Street Granite type VH w/ PG64-40E

Mix design #: 2014A - 1975

Mix Type: HMA Type VH

Test by: ____________________

Test Temp: **50** °C

Compaction Temp. **280-290** °F

Agg Bulk SpG: 2.703

Agg Eff SpG: 2.751

Binder SpG 0.999

Asphalt Content: 5.5

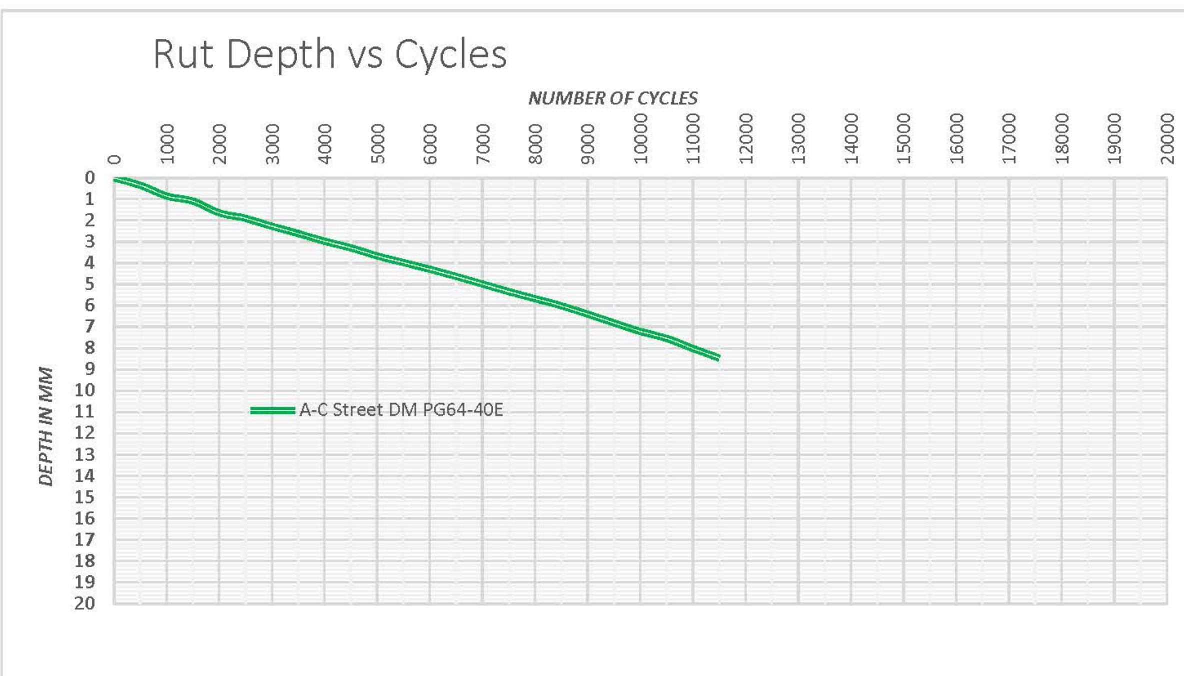
0

Mix MSpG: 2.509

From Graph:

Depth at 10,000 passes____________

	Sample 1		Sample 2		Sample 3	
Target Weight of mix for specimen	**3146.1** g		**2997.9** g		**2925.4**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2394.1	2470.3				
Weight in Water,g	1411.2	1468.2				
SSD Weight, g	2396.6	2472.4				
Bulk SpG	2.430	2.460	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	3.2	2.0				
Voids Mineral Agg, %	15.1	14.0				
Voids Filled, %	63%	61%				
Combined	2.6					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Funny River Type IIA EP52V-40

Mix design #: 2017A - 1657
Mix Type: HMA Type IIA

Test by: ____________________

Test Temp: **45** °C
Compaction Temp. **258-264** °F

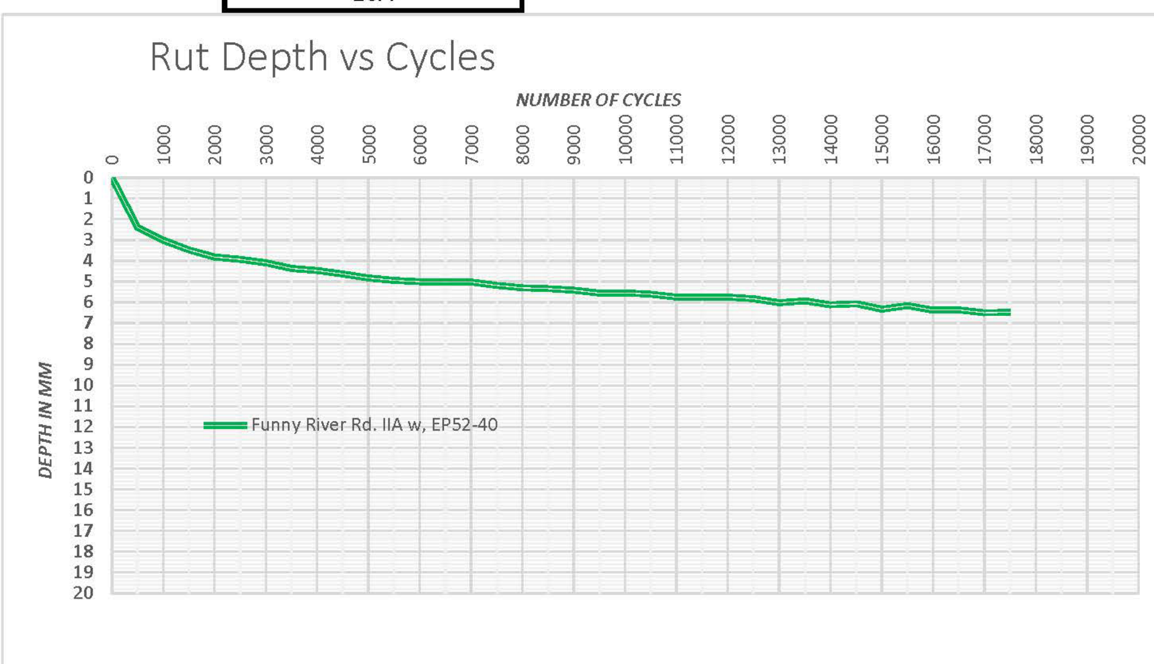
Agg Bulk SpG: 2.675
Agg Eff SpG: 2.722
Binder SpG 0.995

Asphalt Content: 5.6
0
Mix MSpG: 2.481

From Graph:
Depth at 10,000 passes____________

Target Weight of mix for specimen: **3110.8** g | **2964.3** g | **2892.6**

Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2162.1	2215.4				
Weight in Water,g	1253.2	1284				
SSD Weight, g	2167.1	2218.8				
Bulk SpG	2.366	2.370	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	4.6	4.5				
Voids Mineral Agg, %	16.5	16.4				
Voids Filled, %	66%	66%				
Combined	4.6					
	16.4					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg Workcard

Winter Hamburg Research
Glenn Springer Type IIAH EP PG64-40E

Mix design #: 2019A - 2761

Mix Type: Type IIH w/ EP PG64-40E

Test by:

Test Temp: **50** °C

Compaction Temp. **310** °F

Agg Bulk SpG: 2.702

Agg Eff SpG: 2.759

Binder SpG 0.994

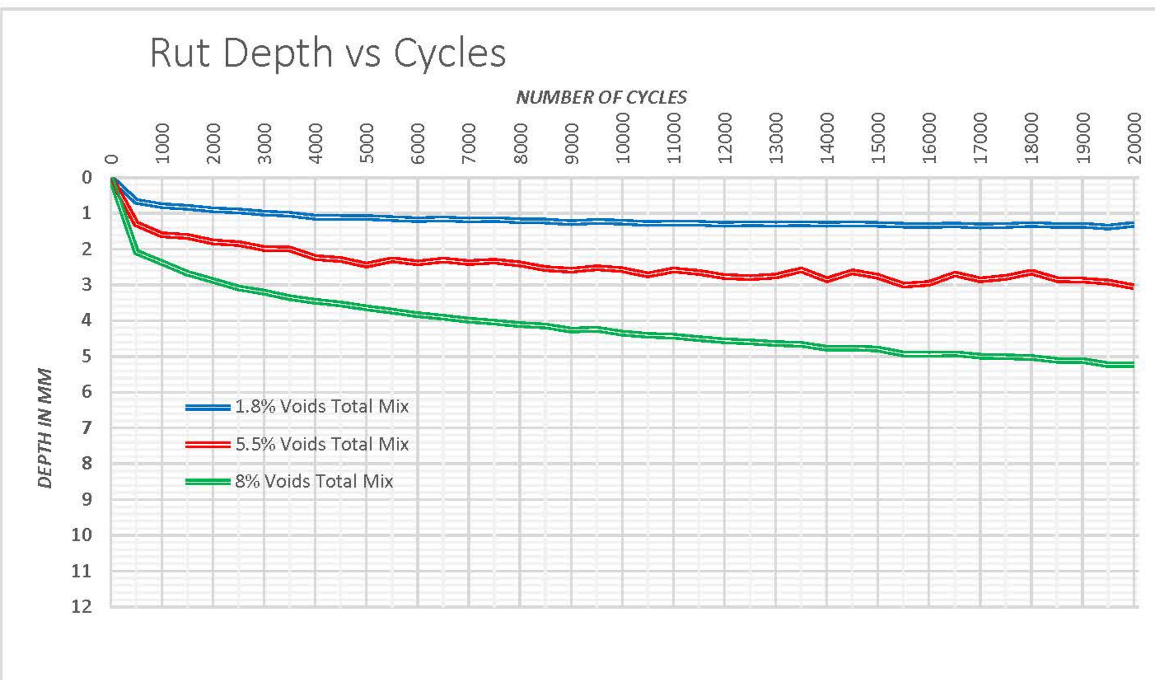
Asphalt Content: 4.9

Mix MSpG: 2.538

From Graph:

Depth at 10,000 passes	
Inflection (Stripping pt)	

Target Weight of mix for specimen	**2676.8** g		**2593.1** g		**2523.4** g	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2663.2	2660.1	2483.9	2483.1	2378.9	2374.1
Weight in Water,g	1596.4	1595.8	1455.9	1459	1394	1386.2
SSD Weight, g	2665.7	2661.9	2493.2	2492.4	2411.5	2405.7
Bulk SpG	2.491	2.495	2.395	2.403	2.338	2.329
Voids Total mix, %	1.9	1.7	5.7	5.3	7.9	8.3
Voids Mineral Agg, %	12.3	12.2	15.7	15.4	17.7	18.0
Voids Filled, %	60%	60%	69%	68%	72%	73%
Combined	1.8		5.5		8.1	



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
K-Beach 16-22 VH w/ EP PG52-40V

Mix design #: 2018A - 3935
Mix Type: HMA Type VH

Test by: ____________________

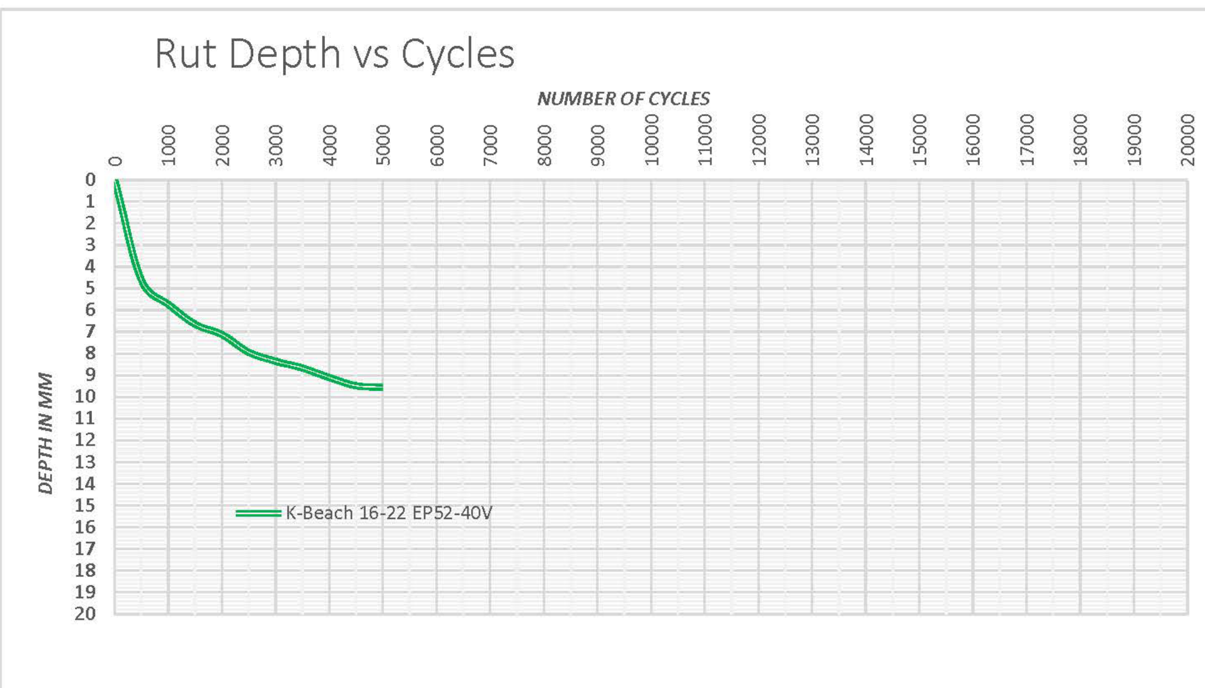
Test Temp: **45** °C
Compaction Temp. **298-308** °F

Agg Bulk SpG: 2.664
Agg Eff SpG: 2.722
Binder SpG 0.995

Asphalt Content: 5.8
0
Mix MSpG: 2.473

From Graph:
Depth at 10,000 passes____________

Target Weight of mix for specimen	**3101.0** g		**2954.9** g		**2883.5**	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2255.3	2250.3				
Weight in Water,g	1301.3	1302.6				
SSD Weight, g	2258.2	2252.2				
Bulk SpG	2.357	2.370	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	4.7	4.2				
Voids Mineral Agg, %	16.7	16.2				
Voids Filled, %	65%	64%				
Combined	4.4					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

2021A-0311
Winter Hamburg research
Minnesota VH w EP PG64-40E

Mix design #: 2019A - 1244
Mix Type: HMA Type VH

Test by: GG

Test Temp: **50** °C
Compaction Temp. **305-315** °F

Agg Bulk SpG: 2.703
Agg Eff SpG: 2.743
Binder SpG 0.994

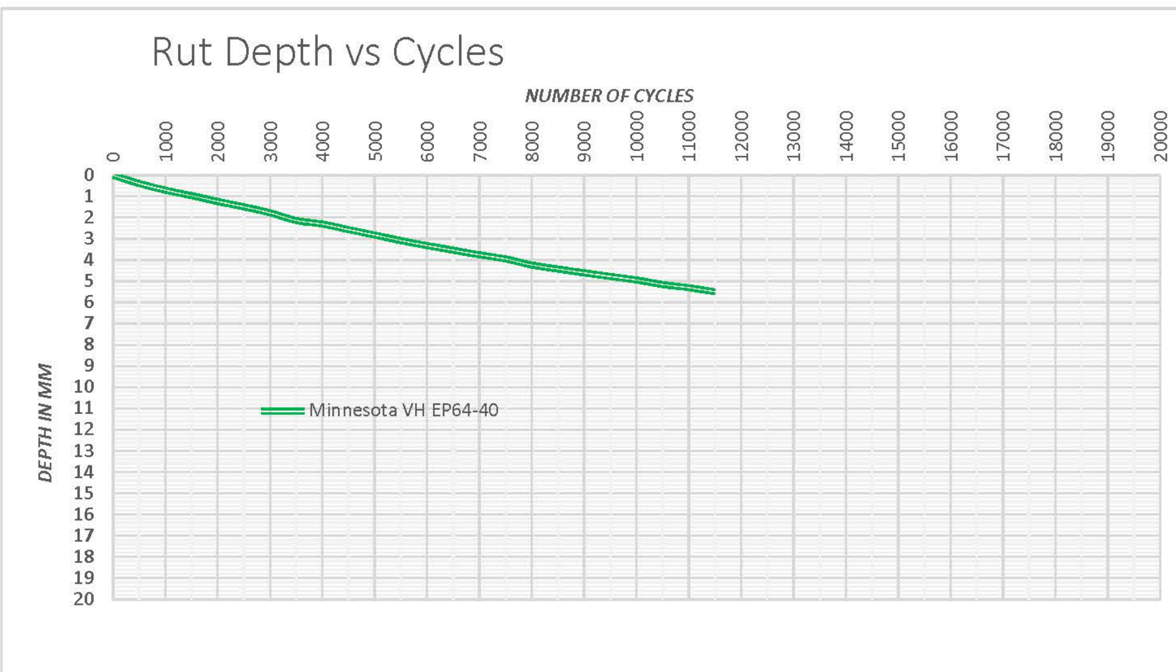
Asphalt Content: 5.1
0

Mix MSpG: 2.517

From Graph:
Depth at 10,000 passes 4.9

Target Weight of mix for specimen: **3156.3** g | **3007.6** g | **2934.9**

Sample :	1		2		3	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2342.9	2305				
Weight in Water,g	1372.6	1351.2				
SSD Weight, g	2345.8	2307.5				
Bulk SpG	2.407	2.410	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	4.4	4.2				
Voids Mineral Agg, %	15.5	15.4				
Voids Filled, %	67%	67%				
Combined	4.3					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg Workcard

Winter Hamburg research
Palmer-Wasilla type IIAH w/ EP PG52-40V

Mix design #: 2020A - 0369
Mix Type: Type IIAH w/ EP PG52-40V

Test by: ______

Test Temp: **45** °C
Compaction Temp. **298°-308°F** °F

Agg Bulk SpG: 2.709
Agg Eff SpG: 2.77
Binder SpG 1.001

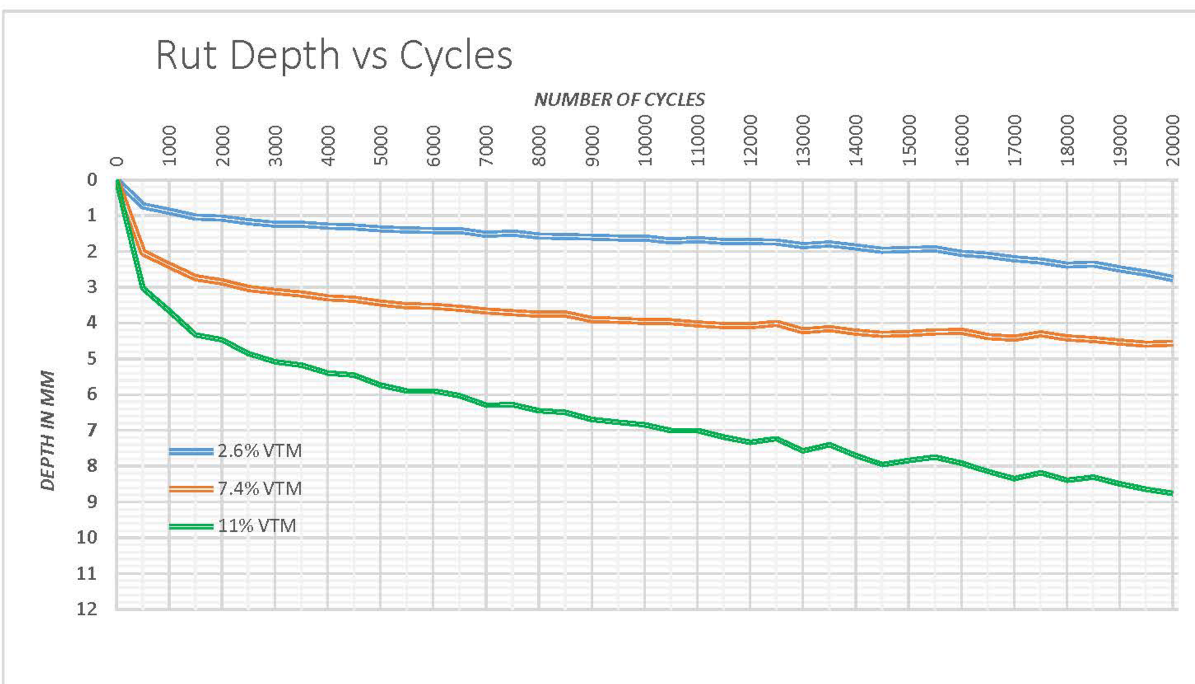
Asphalt Content: 5.1

Mix MSpG: 2.541

From Graph:

Depth at 10,000 passes	
Inflection (Stripping pt)	

Target Weight of mix for specimen	**2665.8** g		**2498.3** g		**2381.1**	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2646.3	2644.9	2481.8	2479.5	2364.6	2365.6
Weight in Water,g	1580.9	1579.2	1444.6	1446	1364.9	1362.2
SSD Weight, g	2648.4	2648.6	2499.9	2499.7	2409.2	2409.9
Bulk SpG	2.479	2.473	2.352	2.353	2.264	2.25789825
Voids Total mix, %	2.4	2.7	7.4	7.4	10.9	11.1
Voids Mineral Agg, %	13.2	13.4	17.6	17.6	20.7	20.9
Voids Filled, %	61%	62%	71%	71%	75%	76%
Combined	2.6		7.4		11.0	



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Glenn Parks MP35-40 VH w/ PG64-40E

Mix design #: 2015A - 3888
Mix Type: HMA Type VH

Test by: ____________________

Test Temp: 50 °C
Compaction Temp. 305-315 °F

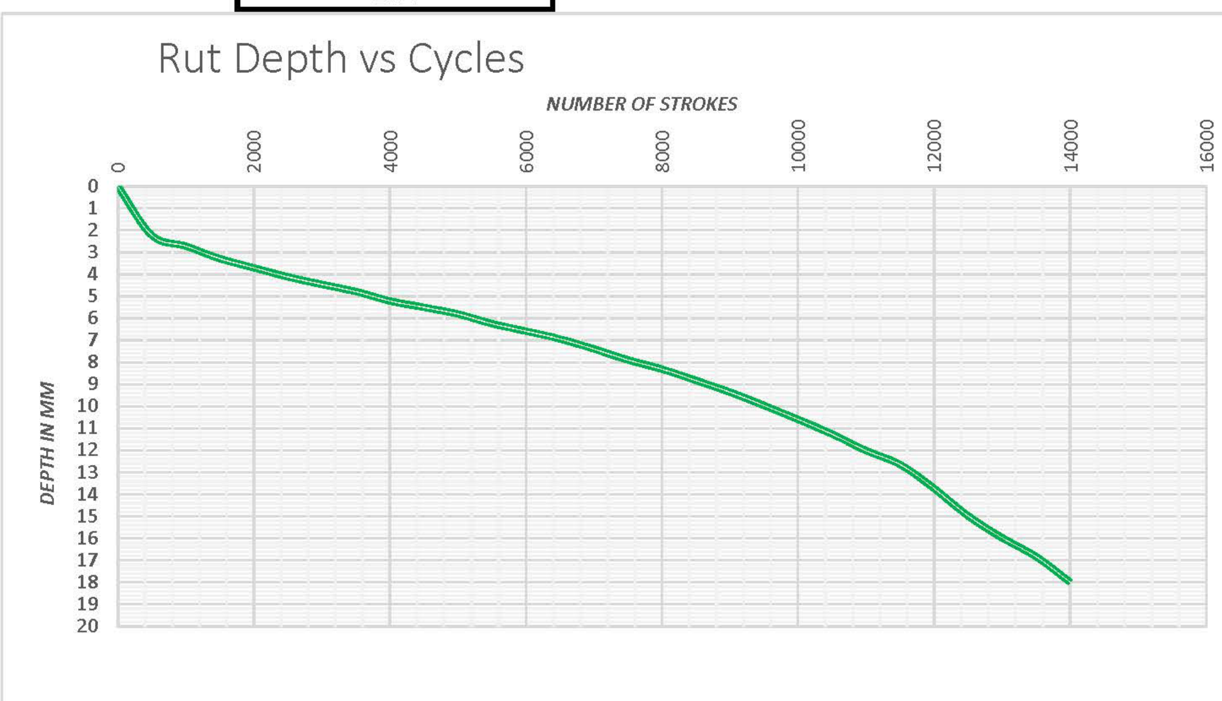
Agg Bulk SpG: 2.78
Agg Eff SpG: 2.841
Binder SpG 0.986

Asphalt Content: 5
0

Mix MSpG: 2.597

From Graph:
Depth at 10,000 passes____________

Target Weight of mix for specimen	3256.1 g		3102.7 g		3027.7	
Sample :	1		2		3	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2482.1	2423.5				
Weight in Water,g	1462.3	1439.2				
SSD Weight, g	2486.6	2425.7				
Bulk SpG	2.423	2.457	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	6.7	5.4				
Voids Mineral Agg, %	17.2	16.0				
Voids Filled, %	71%	69%				
Combined	6.0					
	16.6					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Parks 52-57 IIA w/ DM PG52-34

Mix design #: 2012A - 0953
Mix Type: HMA Type IIA
Test by: ____________________

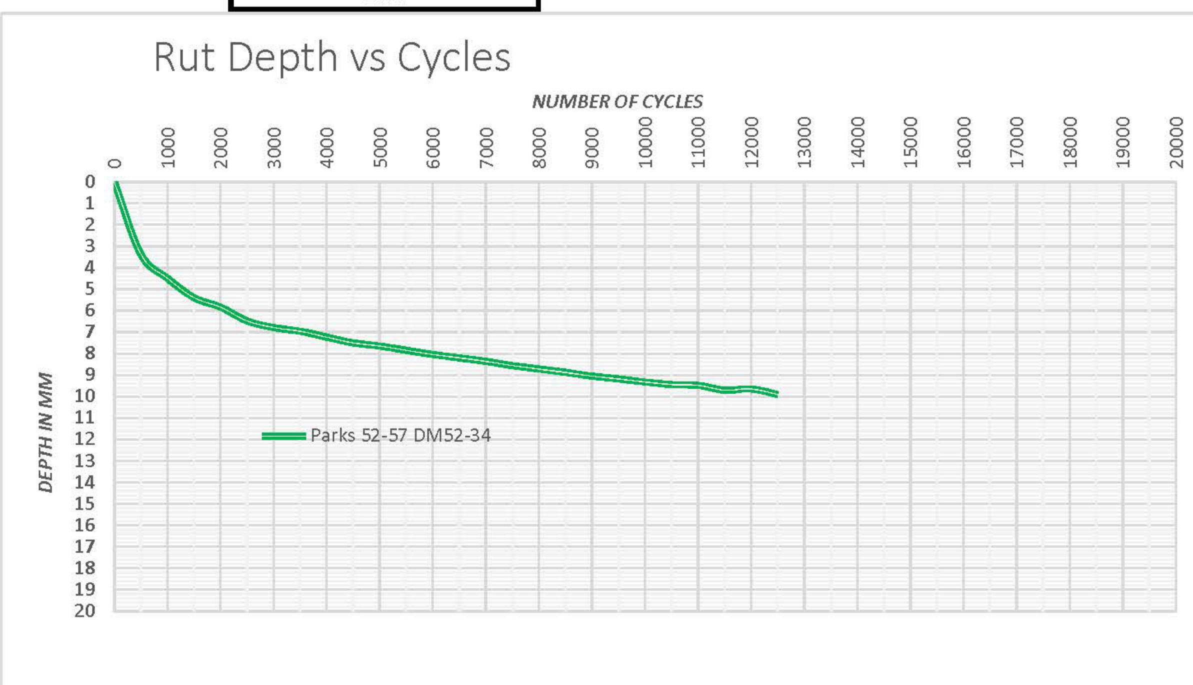
Test Temp: **45** °C
Compaction Temp. **290-300** °F

Agg Bulk SpG: 2.689
Agg Eff SpG: 2.754
Binder SpG 1.009

Asphalt Content: 5.2
0
Mix MSpG: 2.527

From Graph:
Depth at 10,000 passes____________

Target Weight of mix for specimen	**3168.4** g		**3019.1** g		**2946.1**	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2209.9	2285.6				
Weight in Water,g	1292.6	1358.3				
SSD Weight, g	2224.2	2287.7				
Bulk SpG	2.372	2.459	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	6.1	2.7				
Voids Mineral Agg, %	16.4	13.3				
Voids Filled, %	68%	61%				
Combined	4.4					
	14.8					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Parks MP 72-83 (KASH) IIA w/ Tes PG52-28

Mix design #: 2010A - 1917

Mix Type: HMA Type IIA

Test by: ____________

Test Temp: **45** °C

Compaction Temp. **264-275** °F

Agg Bulk SpG: 2.688

Agg Eff SpG: 2.722

Binder SpG 1.009

Asphalt Content: 5.2

0

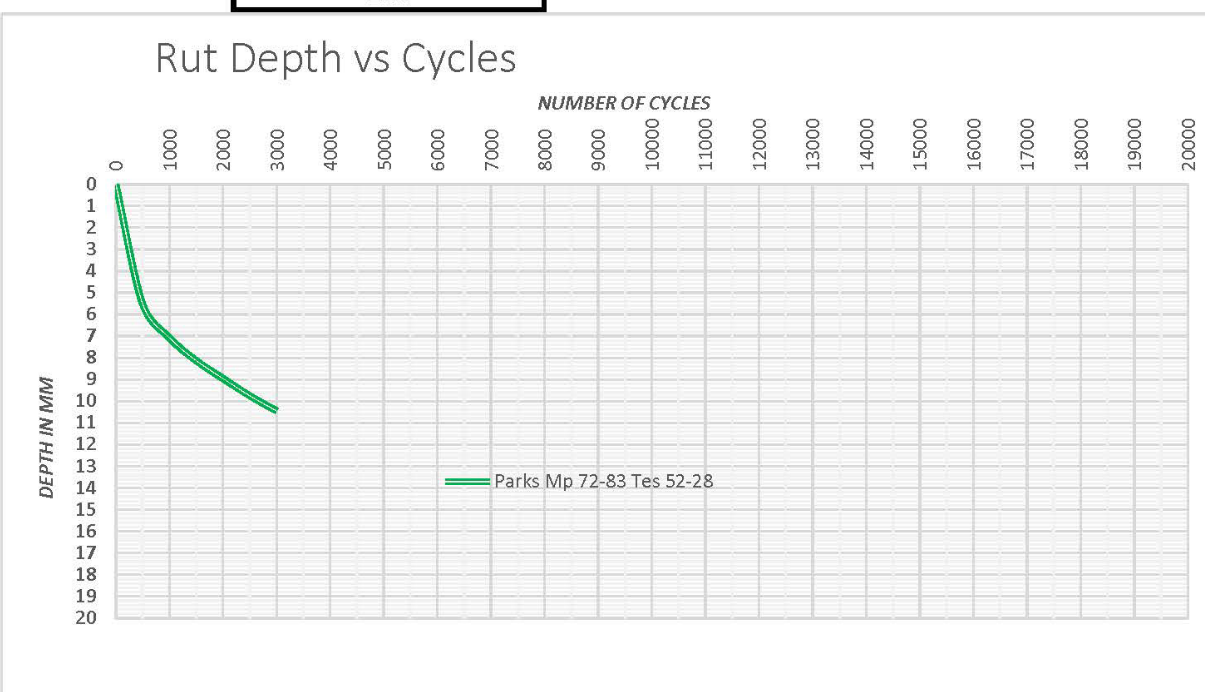
Mix MSpG: 2.501

From Graph:

Depth at 10,000 passes____________

Target Weight of mix for specimen: **3136.3** g | **2988.5** g | **2916.3**

Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2352.7	2303.7				
Weight in Water,g	1373.5	1354.1				
SSD Weight, g	2354.4	2304.8				
Bulk SpG	2.399	2.423	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	4.1	3.1				
Voids Mineral Agg, %	15.4	14.5				
Voids Filled, %	66%	64%				
Combined	3.6					
	15.0					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Parks MP 146-163 IIA w/ EP PG58-34E

Mix design #: 2013A - 1323

Mix Type: HMA Type IIA

Test by: ____________________

Test Temp: **50** °C

Compaction Temp. **305-315** °F

Agg Bulk SpG: 2.639

Agg Eff SpG: 2.698

Binder SpG 1.006

Asphalt Content: 5.1

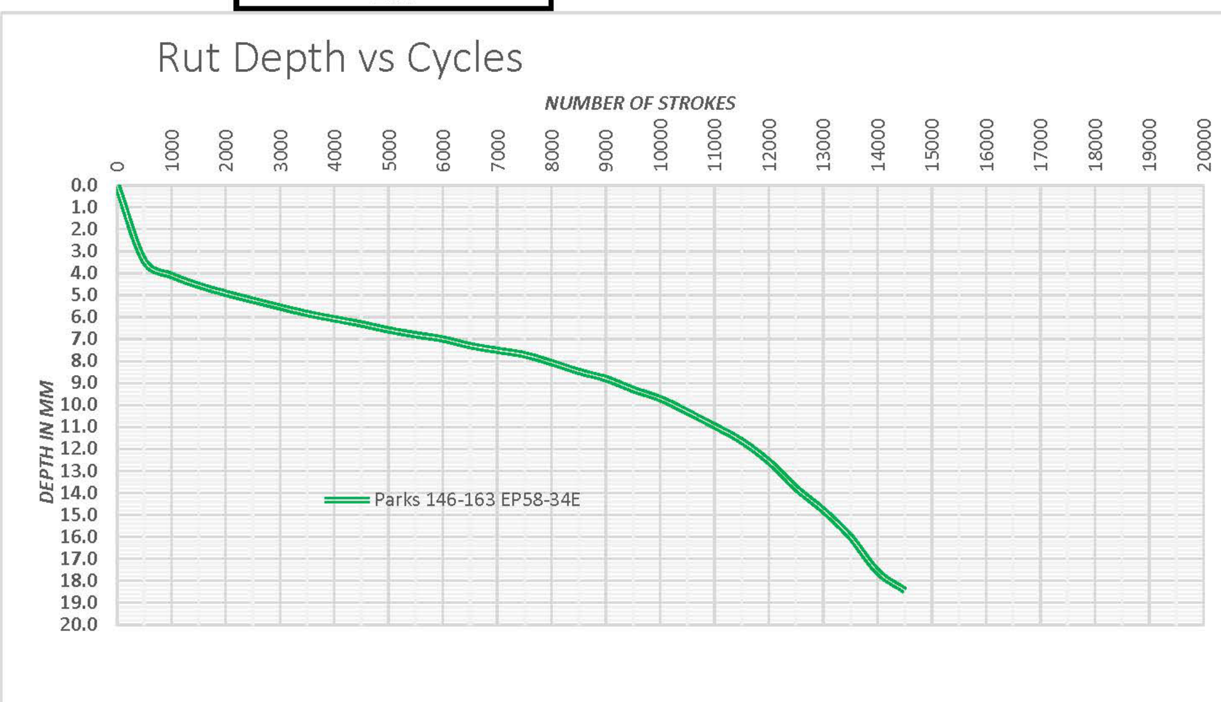
0

Mix MSpG: 2.485

From Graph:

Depth at 10,000 passes__________

	1		2		3	
Target Weight of mix for specimen	**3115.8** g		**2969.0** g		**2897.2**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2172.9	2231				
Weight in Water,g	1241.7	1286.9				
SSD Weight, g	2177.5	2234.7				
Bulk SpG	2.322	2.354	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	6.6	5.3				
Voids Mineral Agg, %	16.5	15.4				
Voids Filled, %	69%	67%				
Combined	5.9					
	15.9					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Seward MP75-90 Type IIA DM 52-40V

Mix design #: 2019A - 1256

Mix Type: IIA w/ 15% RAP, DM 52-40V

Test by: GG

Test Temp: **45** °C

Compaction Temp. **262°F** °F

Agg Bulk SpG: 2.7

Agg Eff SpG: 2.737

Binder SpG 1.005

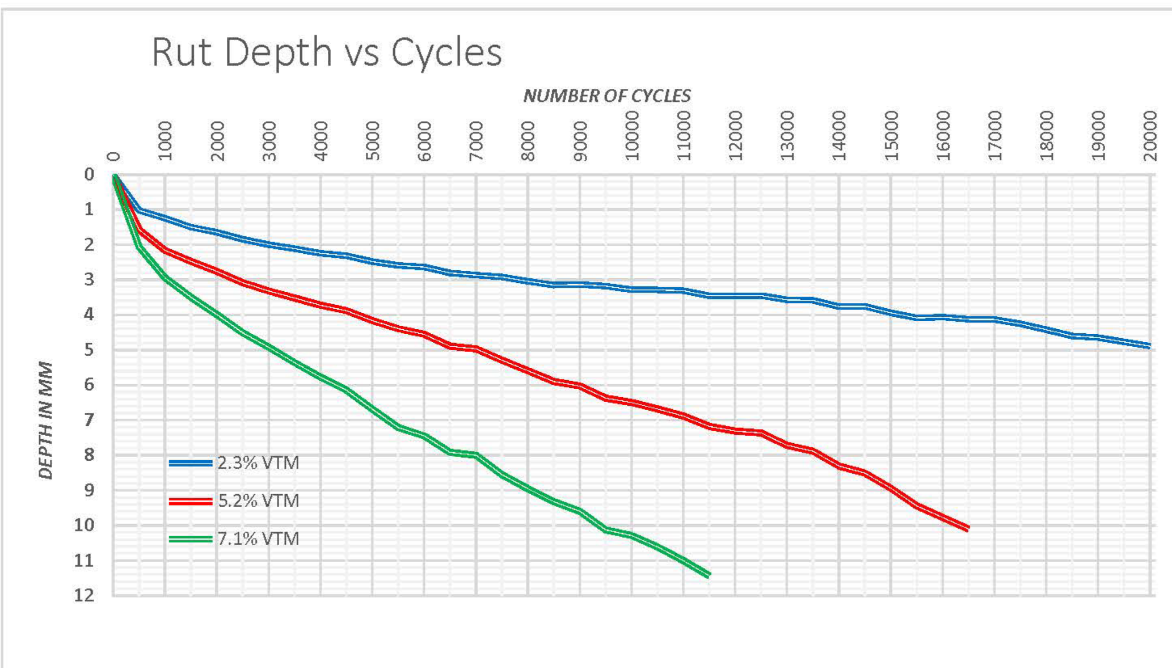
Asphalt Content: 5.7

Mix MSpG: 2.492

From Graph:

Depth at 10,000 passes	
Inflection (Stripping pt)	

Target Weight of mix for specimen	**2614.6** g		**2505.1** g		**2431.2** g	
Sample :	1		2		3	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2565	2572.9	2470.6	2466.8	2397.5	2401.9
Weight in Water,g	1511.9	1519.8	1430.9	1425	1375.2	1377.6
SSD Weight, g	2566.8	2574.6	2475.4	2470.7	2409.4	2415.4
Bulk SpG	2.432	2.439	2.365	2.359	2.318	2.314
Voids Total mix, %	2.4	2.1	5.1	5.3	7.0	7.1
Voids Mineral Agg, %	15.1	14.8	17.4	17.6	19.0	19.2
Voids Filled, %	62%	62%	67%	68%	70%	70%
Combined	2.3		5.2		7.1	



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

2021A-0312
Winter Hamburg research
Seward: Dimond- Dowling VH w/ EP PG58-34

Mix design #: 2017A - 932
Mix Type: HMA Type VH

Test by: ____________

Test Temp: **50** °C
Compaction Temp. **305-315** °F

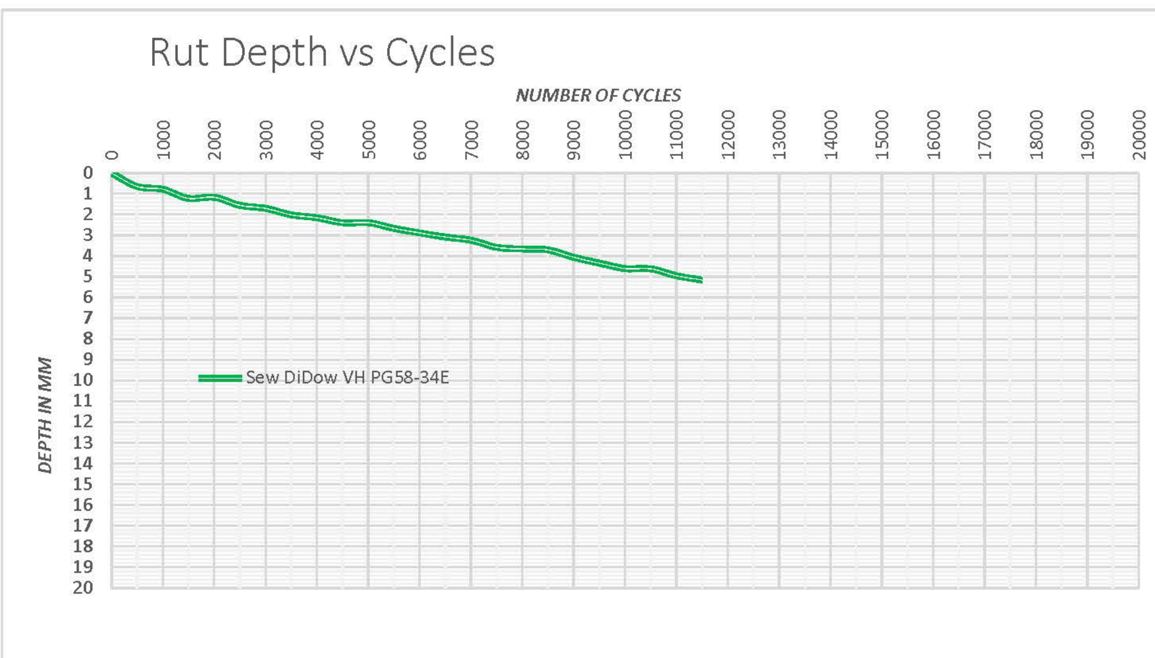
Agg Bulk SpG: 2.67
Agg Eff SpG: 2.733
Binder SpG 1.006

Asphalt Content: 5.4
0

Mix MSpG: 2.501

From Graph:
Depth at 10,000 passes____________

Target Weight of mix for specimen	**3136.3** g		**2988.5** g		**2916.2**	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2325.9	2275.7				
Weight in Water,g	1383.6	1344.7				
SSD Weight, g	2327.1	2277.2				
Bulk SpG	2.465	2.440	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	1.4	2.4				
Voids Mineral Agg, %	12.7	13.5				
Voids Filled, %	57%	60%				
Combined	1.9					



State of Alaska DOT1F
Central Materials Laboratory
Hamburg Wheel Testing Workcard

Hamburg workcard

Winter Hamburg research
Sew Hwy: Tudor-Dowling V w/ EP58-34E

Mix design #: 2013A - 2384
Mix Type: HMA Type V

Test by: ______________________

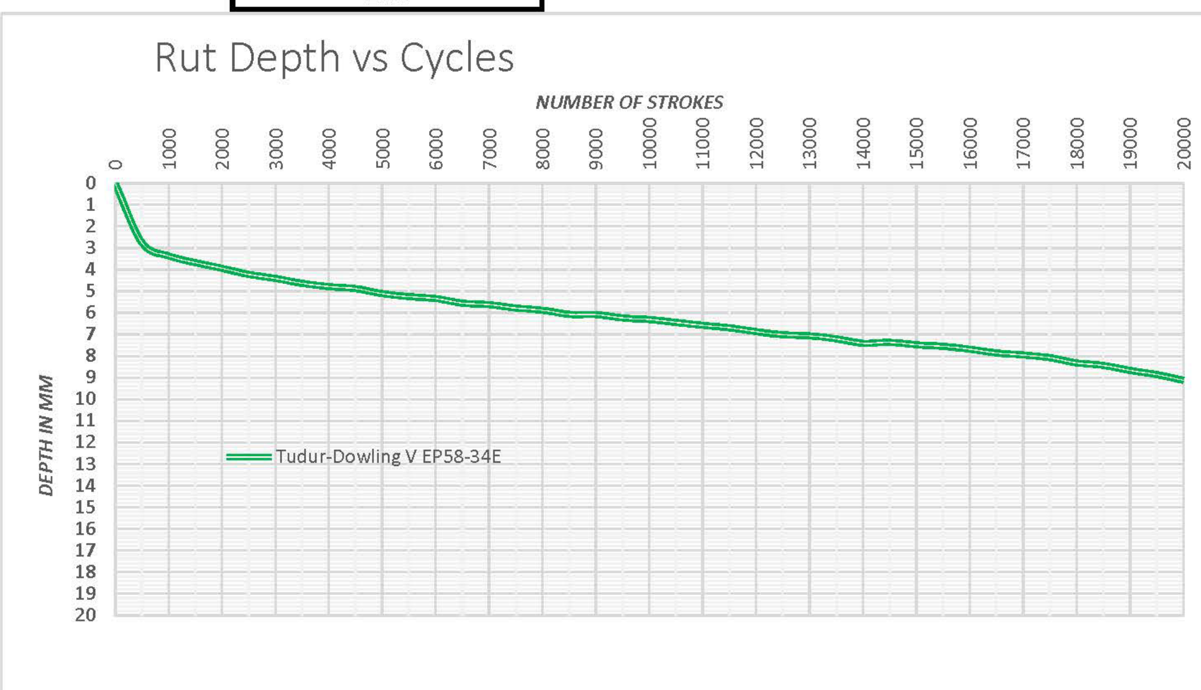
Test Temp: **50** °C
Compaction Temp. **305-315** °F

Agg Bulk SpG: 2.693
Agg Eff SpG: 2.742
Binder SpG 1.006

Asphalt Content: 5
0
Mix MSpG: 2.524

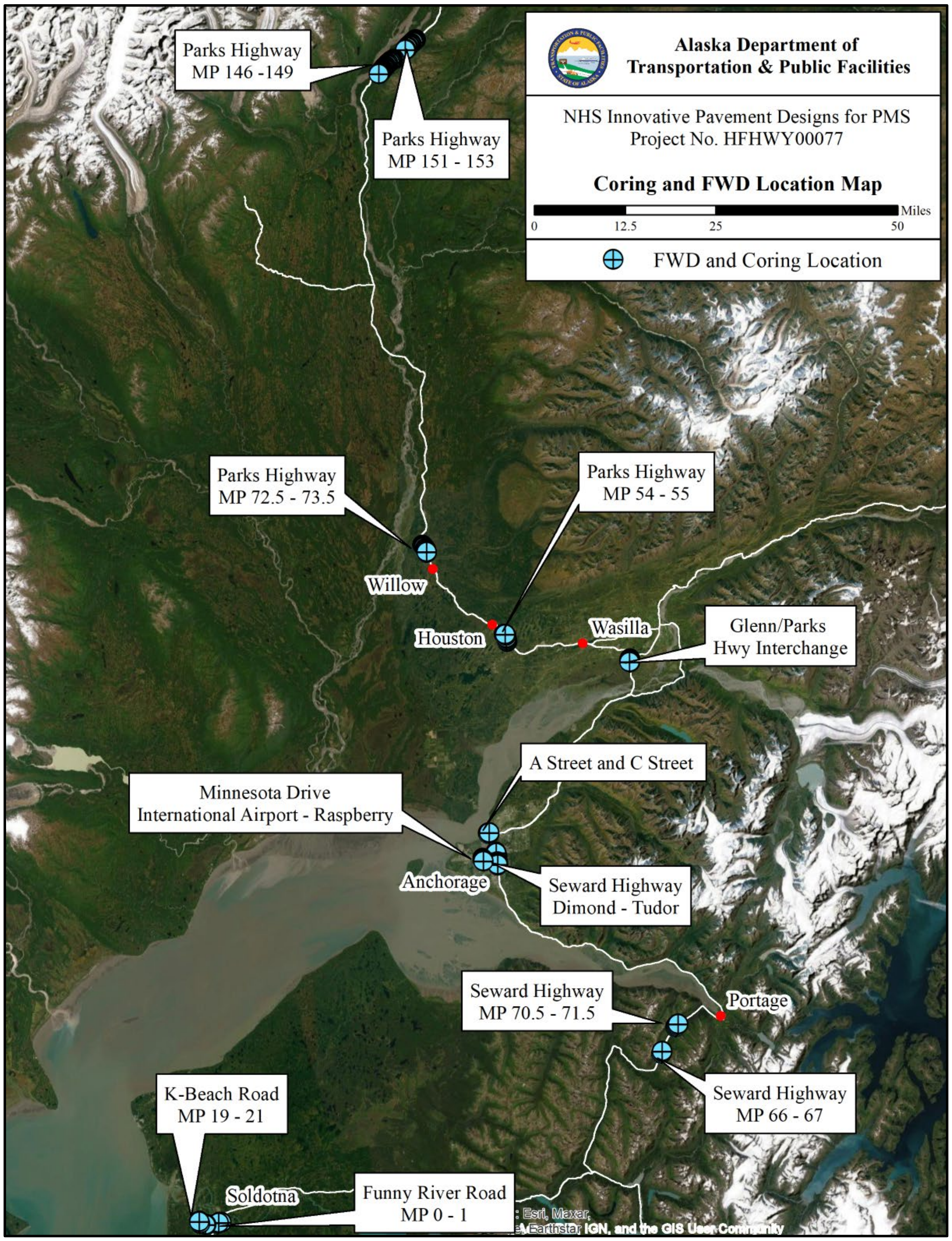
From Graph:
Depth at 10,000 passes____________

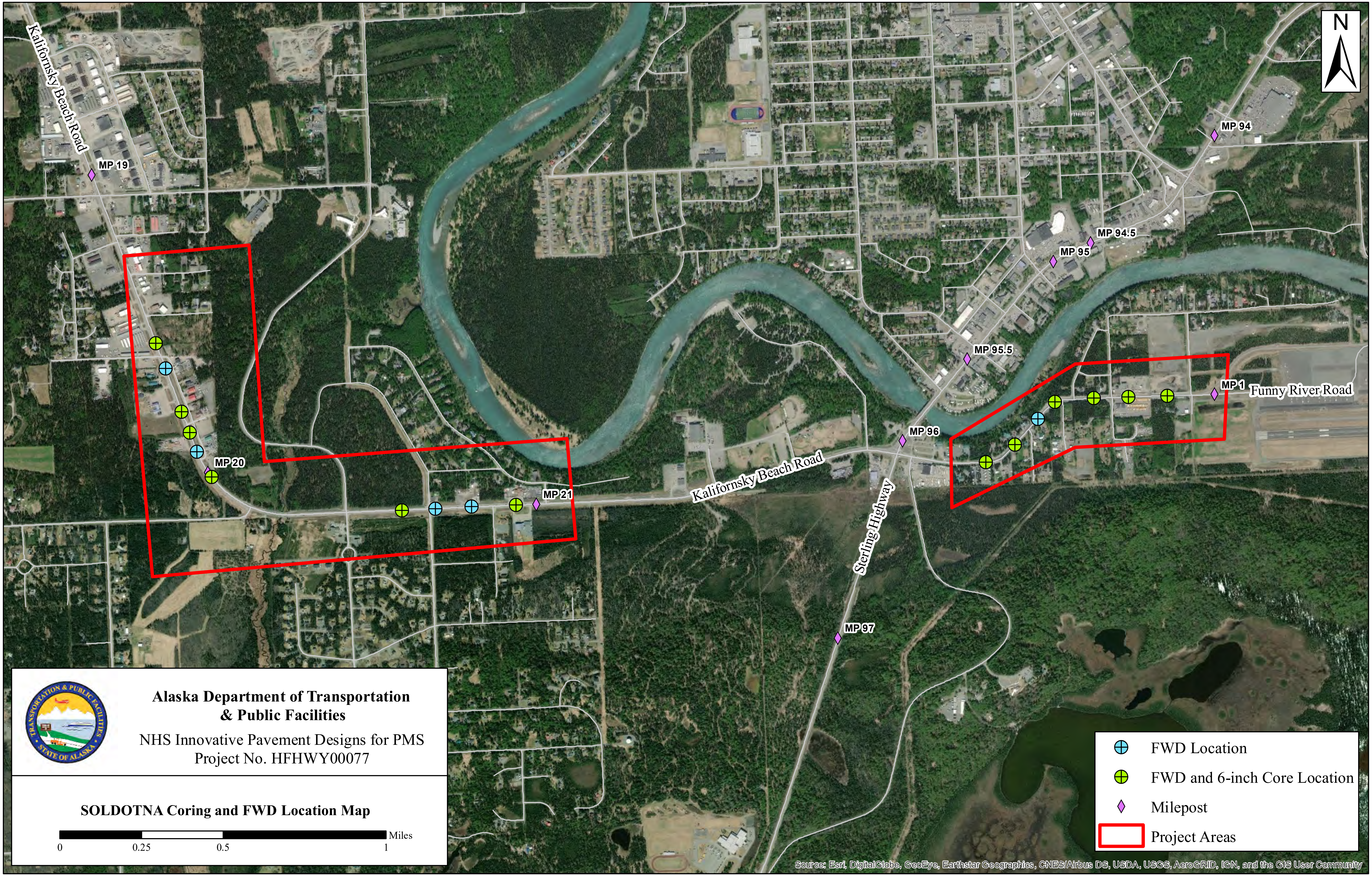
Target Weight of mix for specimen	**3165.2** g		**3016.0** g		**2943.1**	
Sample :	**1**		**2**		**3**	
Target Void %:	3%-5%		5%-8%		7%-10%	
	Core 1	Core 2	Core 3	Core 4	Core 5	Core 6
Dry Weight, g	2350	2303.2				
Weight in Water,g	1395.8	1366.6				
SSD Weight, g	2351	2305.6				
Bulk SpG	2.460	2.453	#DIV/0!	#DIV/0!	#DIV/0!	#DIV/0!
Voids Total mix, %	2.5	2.8				
Voids Mineral Agg, %	13.2	13.5				
Voids Filled, %	62%	63%				
Combined	2.7					
	13.3					



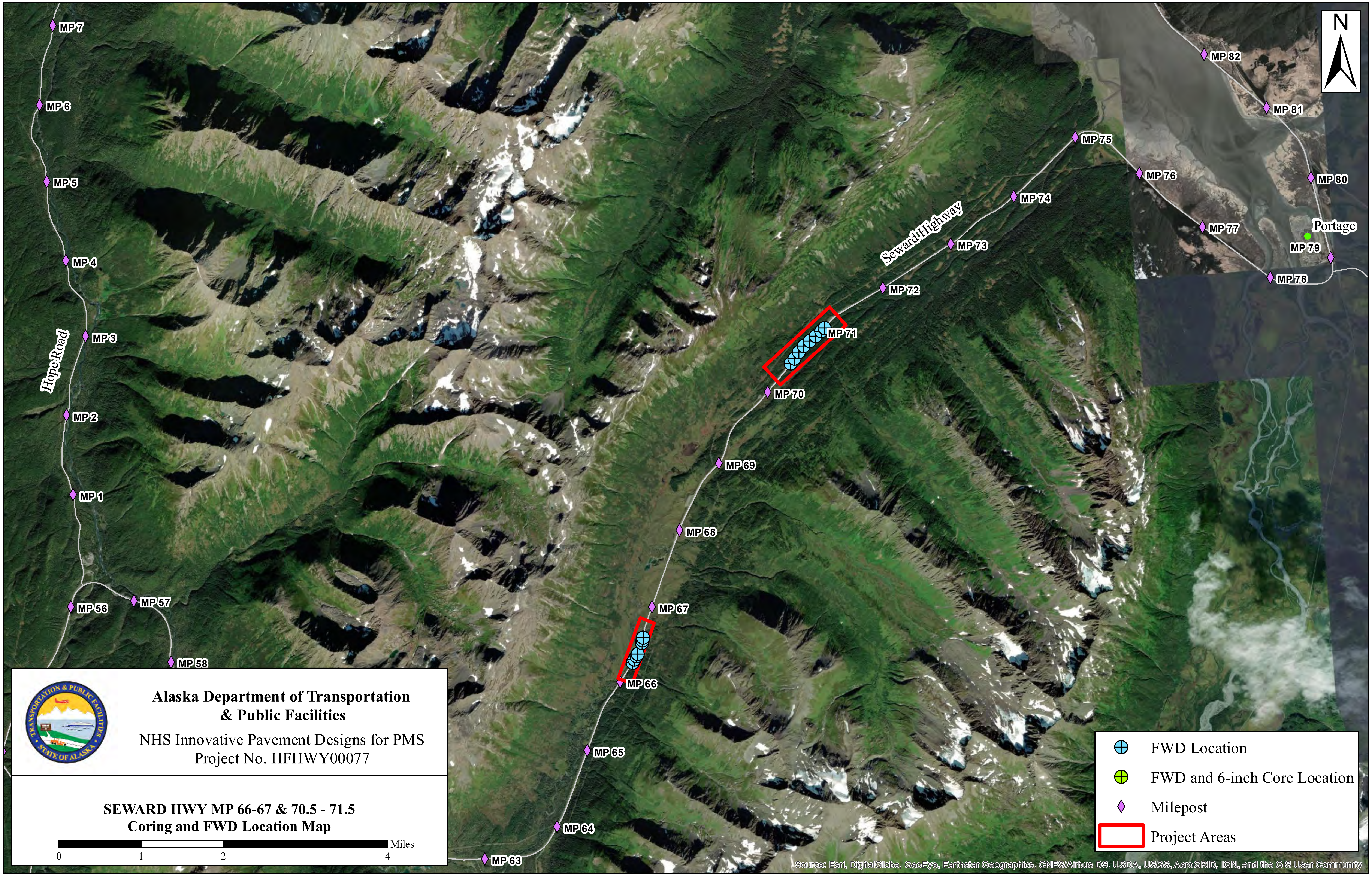
APPENDIX D – FWD DATA

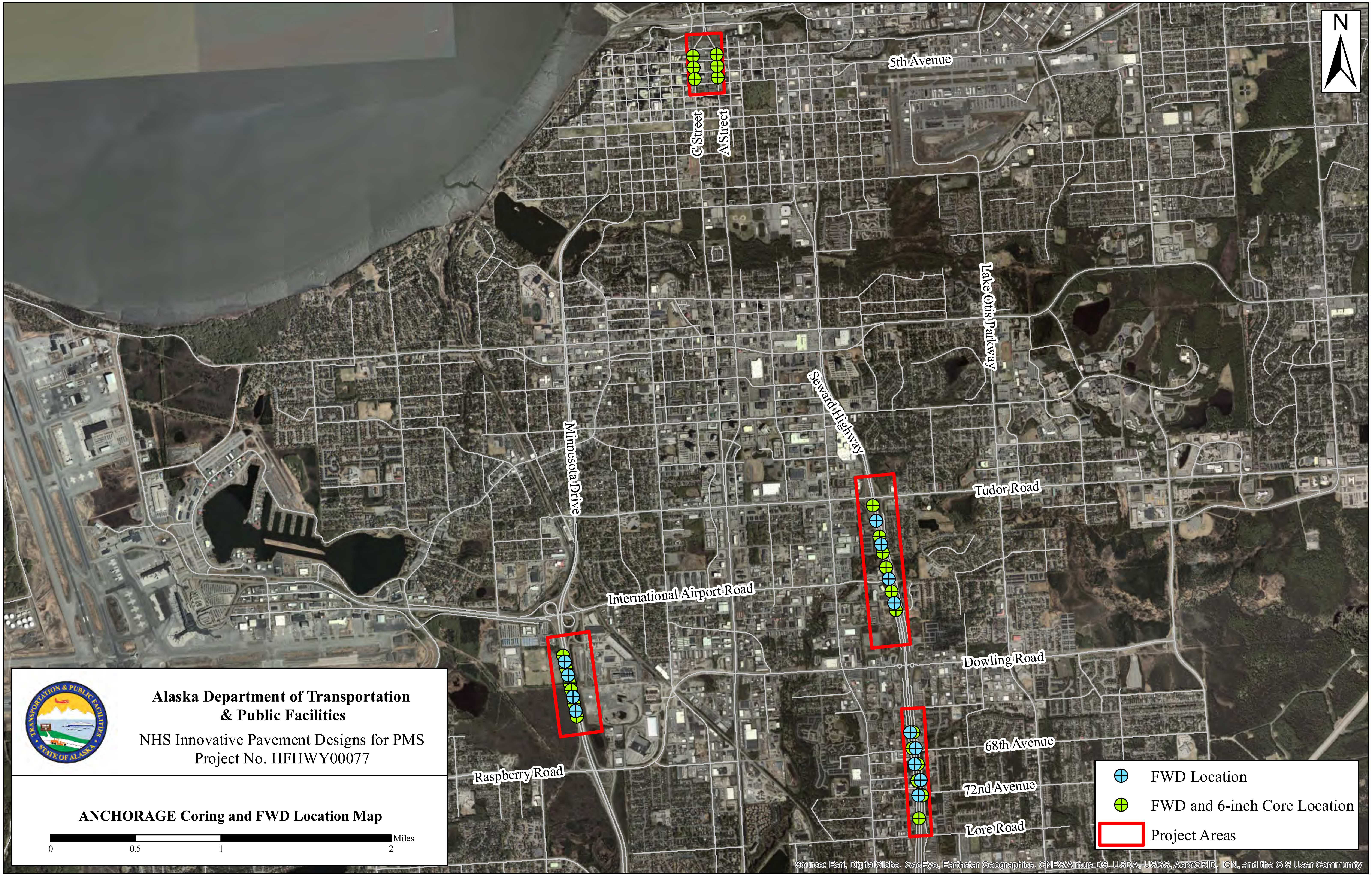
The raw deflection data, plots of elastic moduli and material sections are provided in the figures below for the FWD data backcalculated and provided in this report. The locations where the testing was performed is shown in the figure below.





N
Kalifornsky Beach Road
MP 19
MP 20
MP 21
Kalifornsky Beach Road
MP 94
MP 94.5
MP 95
MP 95.5
MP 96
MP 97
MP 1
Funny River Road
Sterling Highway
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
SOLDOTNA Coring and FWD Location Map
0
0.25
0.5
1
Miles
FWD Location
FWD and 6-inch Core Location
Milepost
Project Areas
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

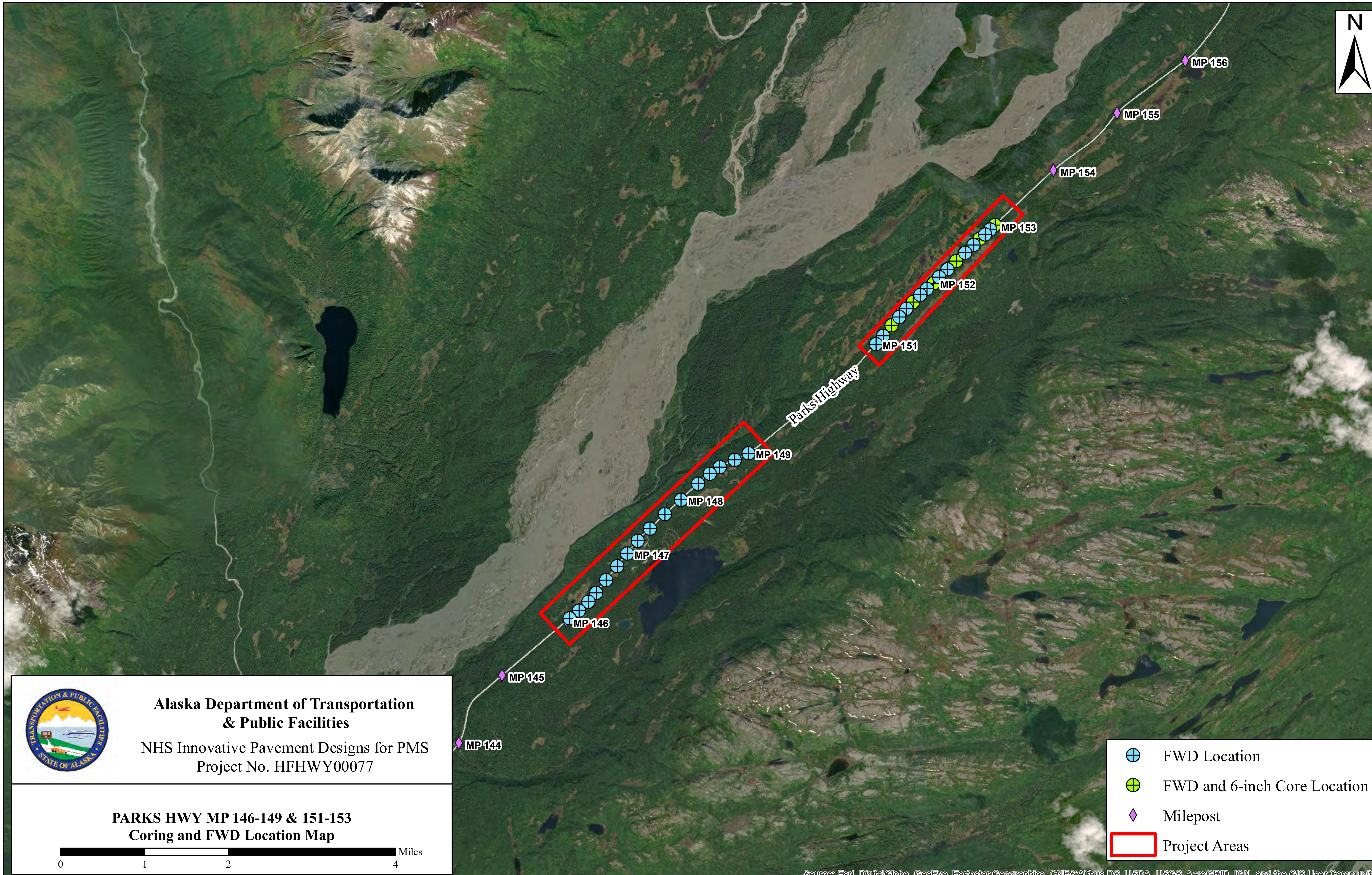

Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
SEWARD HWY MP 66-67 & 70.5 - 71.5
Coring and FWD Location Map
0
1
2
4
Miles
FWD Location
FWD and 6-inch Core Location
Milepost
Project Areas
N
Seward Highway
Hope Road
Portage
MP 7
MP 6
MP 5
MP 4
MP 3
MP 2
MP 1
MP 56
MP 57
MP 58
MP 63
MP 64
MP 65
MP 66
MP 67
MP 68
MP 69
MP 70
MP 71
MP 72
MP 73
MP 74
MP 75
MP 76
MP 77
MP 78
MP 79
MP 80
MP 81
MP 82
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community


N
5th Avenue
C Street
A Street
Lake Otis Parkway
Seward Highway
Minnesota Drive
Tudor Road
International Airport Road
Dowling Road
68th Avenue
72nd Avenue
Lore Road
Raspberry Road
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
ANCHORAGE Coring and FWD Location Map
0
0.5
1
2
Miles
FWD Location
FWD and 6-inch Core Location
Project Areas
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community


N
MP 37
MP 36
Parks Highway
MP 35
MP 35
MP 36
Glenn Highway
Nelson Road
MP 34
Glenn Highway
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
GLENN/PARKS HWY INTERCHANGE
Coring and FWD Location Map
0
0.25
0.5
1
Miles
FWD Location
FWD and 6-inch Core Location
Milepost
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

N
Parks Highway
Gaunt Lane
MP 55
MP 54
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
PARKS HWY MP 54-55
Coring and FWD Location Map
0
700
1,400
2,800
Feet
FWD Location
FWD and 6-inch Core Location
Milepost
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

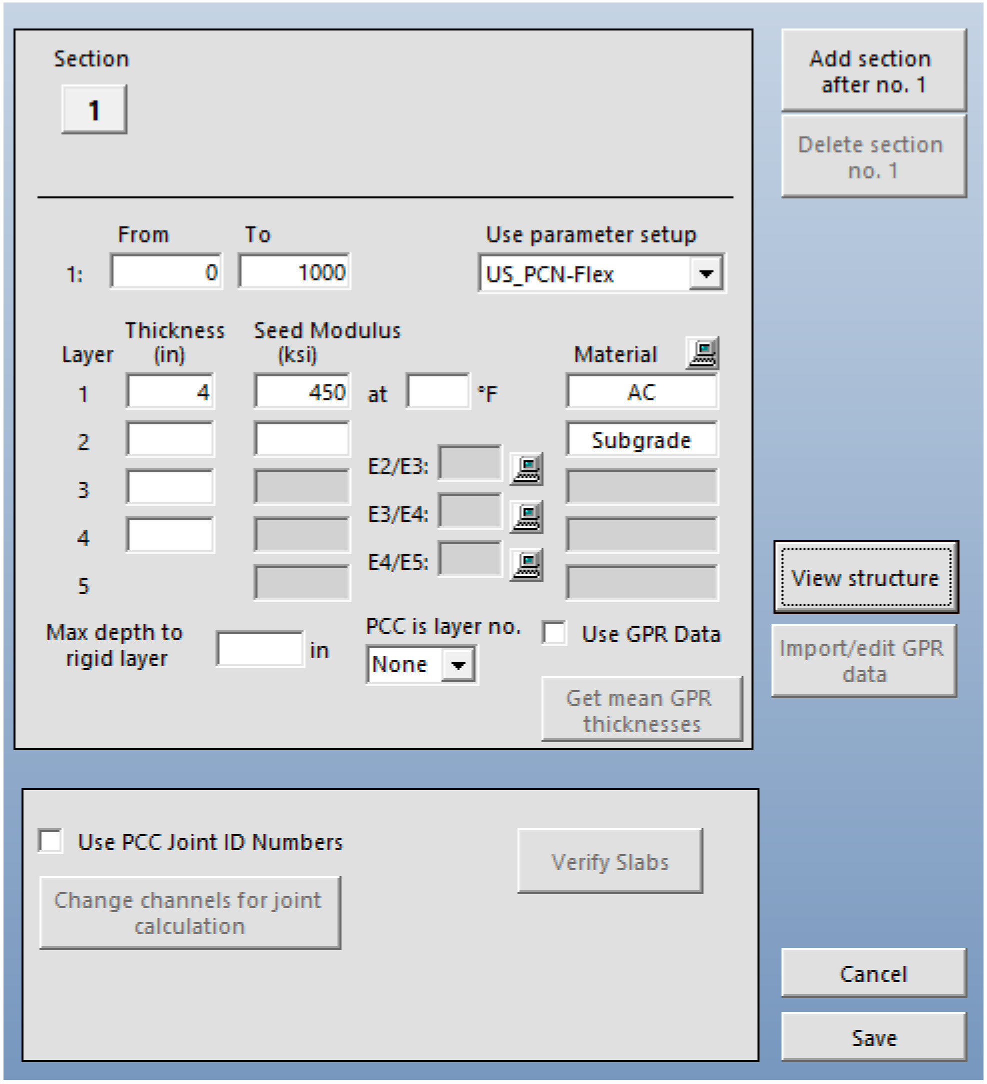

MP 74
Parks Highway
MP 73
MP 72
N
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
PARKS HWY MP 72.5-73.5
Coring and FWD Location Map
0
1,050
2,100
4,200
Feet
FWD Location
FWD and 6-inch Core Location
Milepost
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community


N
MP 156
MP 155
MP 154
MP 153
MP 152
MP 151
Parks Highway
MP 149
MP 148
MP 147
MP 146
MP 145
MP 144
MP 142
Alaska Department of Transportation
& Public Facilities
NHS Innovative Pavement Designs for PMS
Project No. HFHWY00077
PARKS HWY MP 146-149 & 151-153
Coring and FWD Location Map
0 1 2 4 Miles
FWD Location
FWD and 6-inch Core Location
Milepost
Project Areas
Source: Esri, DigitalGlobe, GeoEye, Earthstar Geographics, CNES/Airbus DS, USDA, USGS, AeroGRID, IGN, and the GIS User Community

The roads analyzed are indicated below with the average modulus value and standard deviation for the hot mix asphalt.

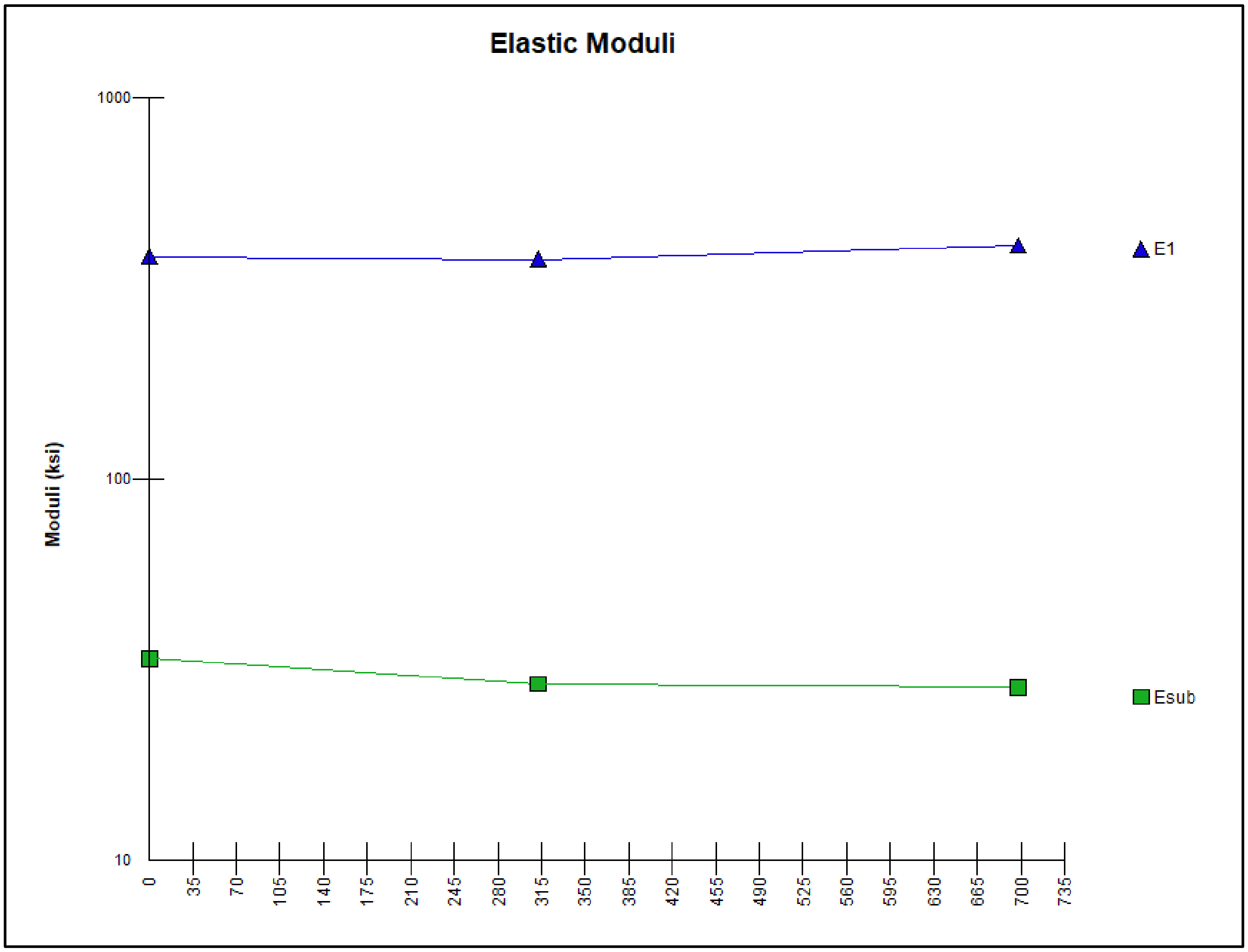
Road	Project Number	Year	Mix	Binder	Modulus (ksi)	Std
Parks Highway MP 72-83	54985	2011	Type IIA	PG52-28	546	1.08
Parks Highway MP 52-57	53469	2012	Type IIA	PG52-34	357	1.14
Funny River Road	53750	2017	Type IIA	PG 52-40E	542	1.338
K-Beach Road	59778	2018	Type VH	PG 52-40E	336	1.077
Seward Highway Dowling to Tudor	50816	2013	Type V	PG58-34	653	1.15
Seward Highway Dimond to Dowling NB	CFHWY00162	2018	Type VH	PG 58-34E	634	1.113
Seward Highway Dimond to Dowling SB	CFHWY00162	2018	Type VH	PG 58-34E	561	1.111
Parks Highway MP 146-163	54147	2013	Type IIA	PG 58-34	425	1.189
Glenn/Parks Interchange SB	58571	2016	Type VH	PG64-40E	528	1.24
Glenn/Parks Interchange NB	58571	2016	Type VH	PG64-40E	503	1.246
C Street	57835	2014	Type VH	PG64-40E	398	1.02
A Street	57835	2014	Type VH	PG64-40E	389	1.045

D.1 A Street



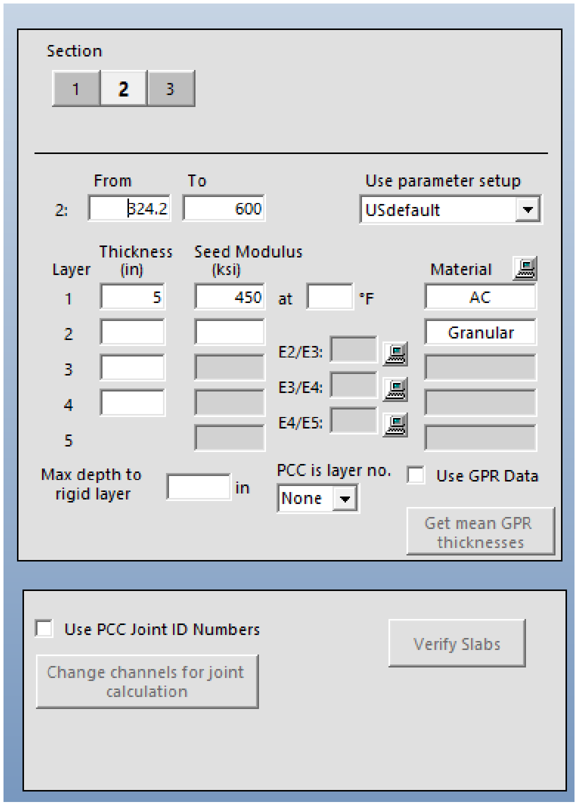
Backcalculated Values

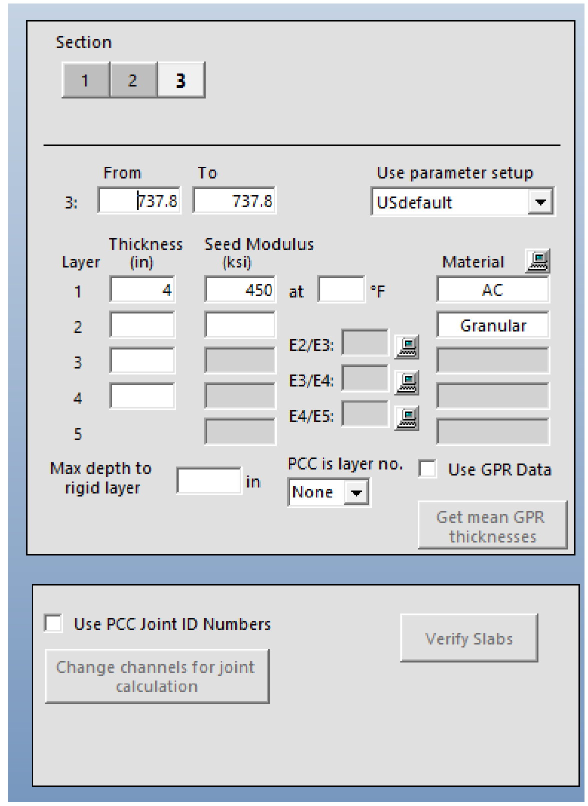
Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type VH	PG64-40	4"	398.37	1.02
Subgrade				58.25	1.29

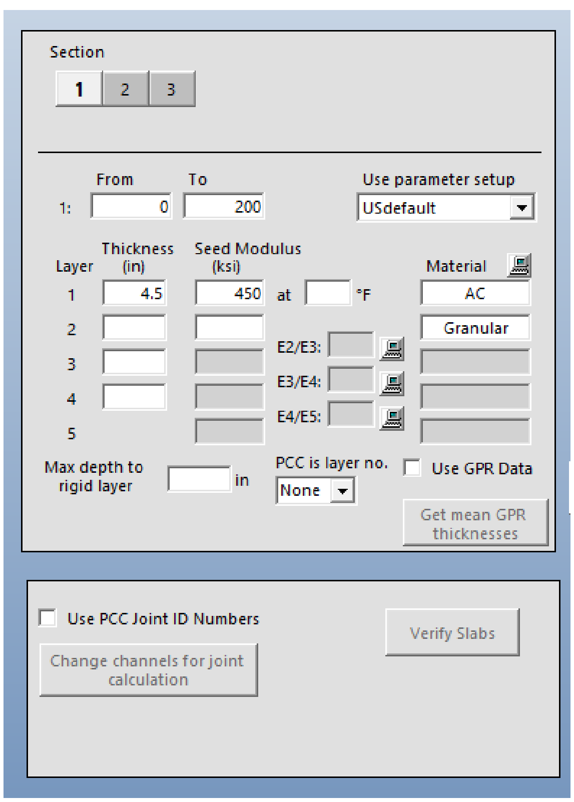


Elastic Moduli
1000
100
10
Moduli (ksi)
0
35
70
105
140
175
210
245
280
315
350
385
420
455
490
525
560
595
630
665
700
735
E1
Esub

D.2 C Street


Section
1 2 3
From To Use parameter setup
2: 324.2 600 USdefault
Layer Thickness (in) Seed Modulus (ksi) Material
1 5 450 at °F AC
2 Granular
3 E2/E3:
4 E3/E4:
5 E4/E5:
Max depth to rigid layer in
PCC is layer no. None
Use GPR Data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Change channels for joint calculation
Verify Slabs

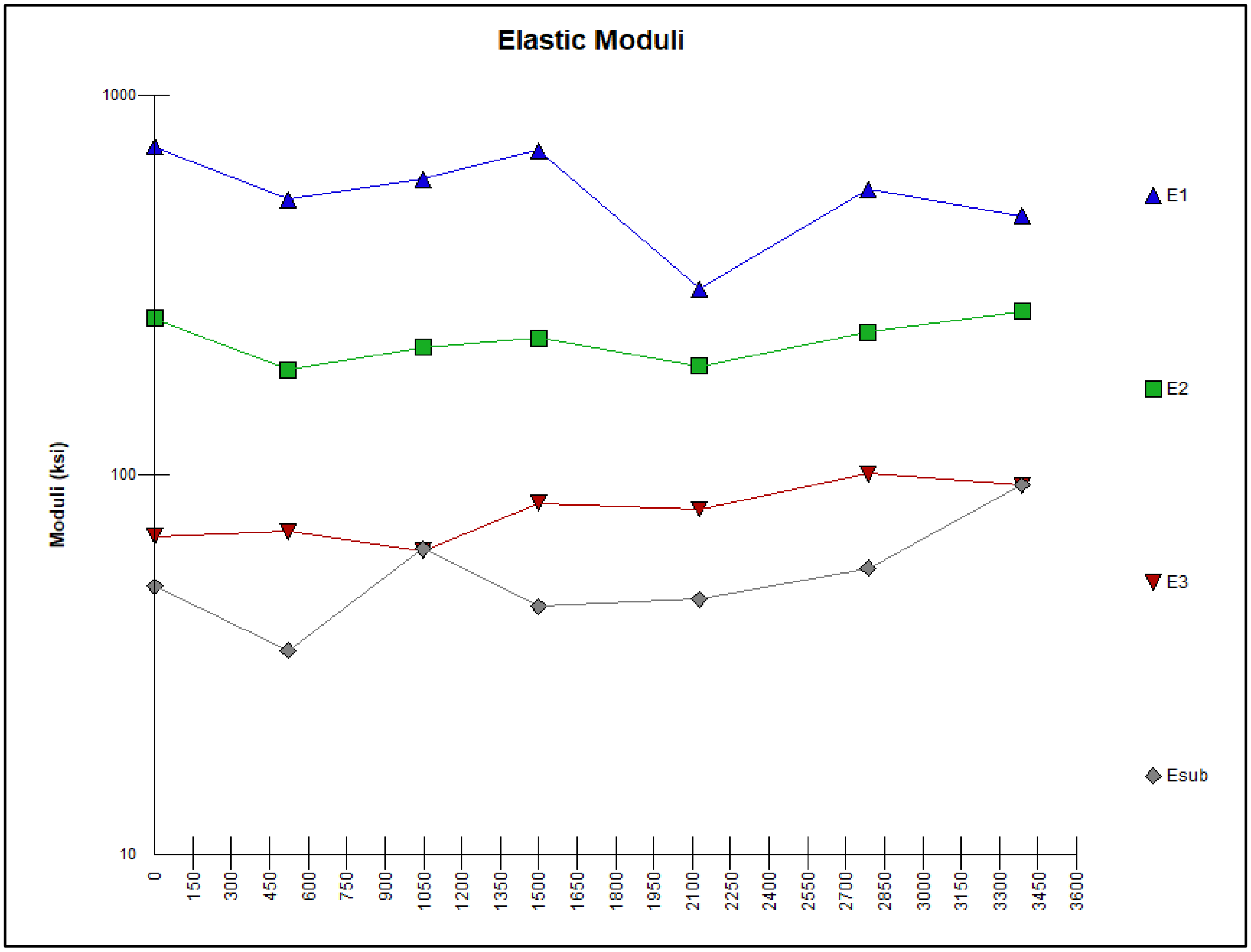

Section
1 2 3
From To Use parameter setup
3: 737.8 737.8 USdefault
Layer Thickness (in) Seed Modulus (ksi) Material
1 4 450 at °F AC
2 Granular
3 E2/E3:
4 E3/E4:
5 E4/E5:
Max depth to rigid layer in
PCC is layer no. None
Use GPR Data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Change channels for joint calculation
Verify Slabs


Section
1 2 3
From To Use parameter setup
1: 0 200 USdefault
Layer Thickness (in) Seed Modulus (ksi) Material
1 4.5 450 at °F AC
2 Granular
3 E2/E3:
4 E3/E4:
5 E4/E5:
Max depth to rigid layer in
PCC is layer no. None
Use GPR Data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Change channels for joint calculation
Verify Slabs

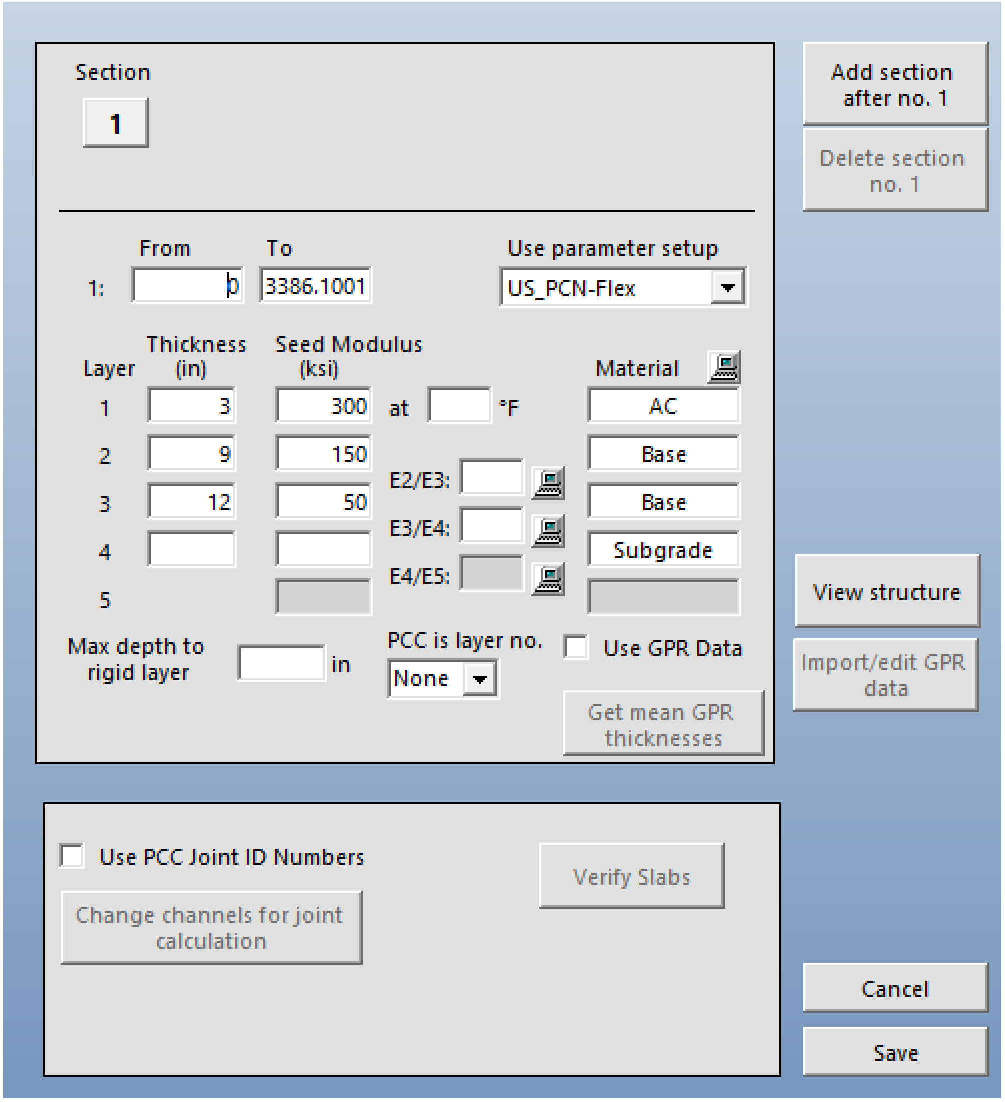
Station	Type	Thickness	E1 - HMA	E2 - Sub
0	Type VH	4.5"	387.3	68
324.2	Type VH	5"	399.4	26.9
737.8	Type VH	4"	408.4	48.5

Note – A and C are short stretches of road, being only 2-3 blocks long, but were the first full depth layer of HMA placed using PG64-40 binder and were included in this analysis. The HMA layer is built over unknown materials as no as-builts were able to be found for previous projects.

D.3 Funny River Road

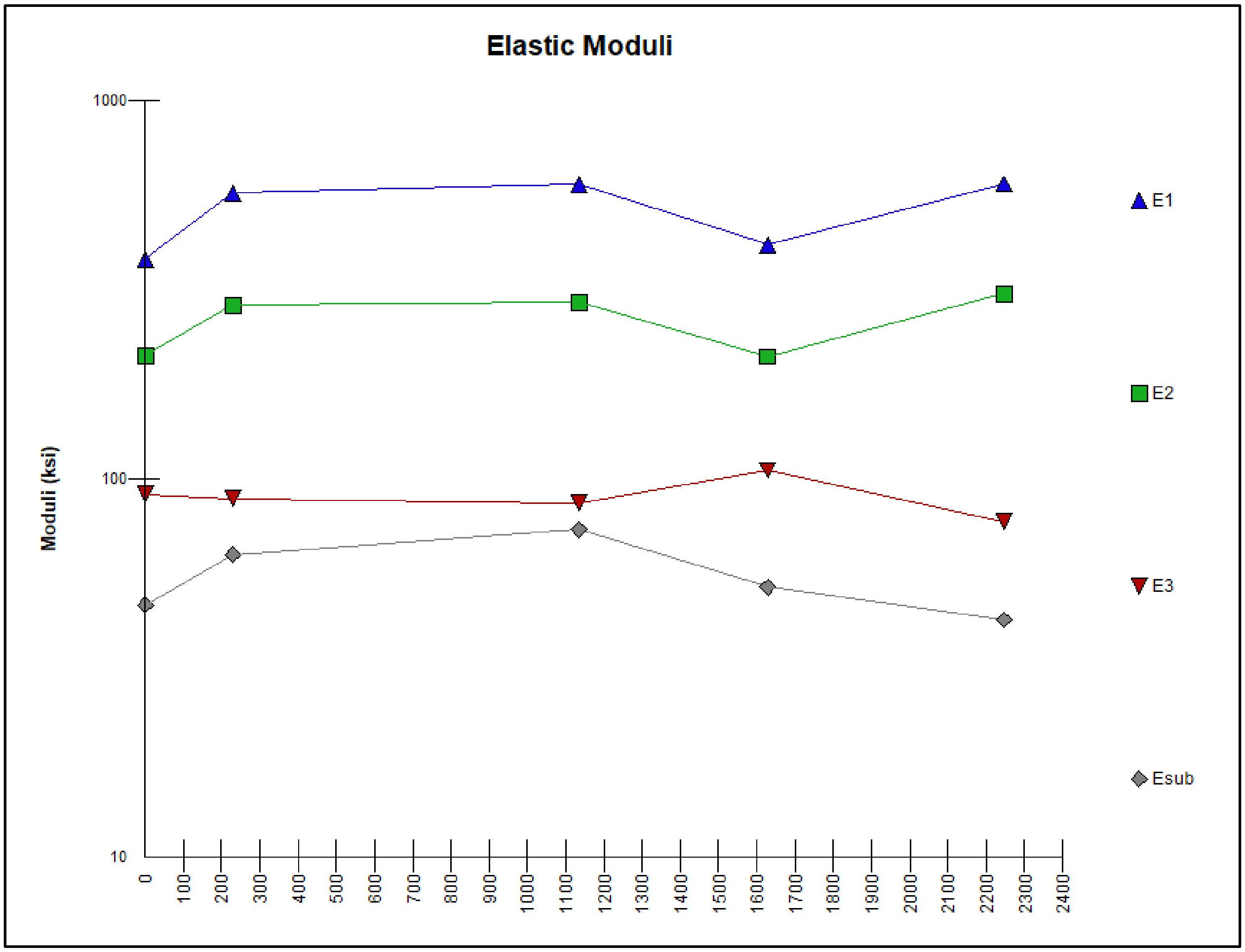


Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type II Class A	PG52-40	3"	541.9	1.338
Base	Foamed Asphalt Stabilized		9"	225.2	1.144
Granular	Select A		12"	79.2	1.189
Subgrade				53.4	1.371

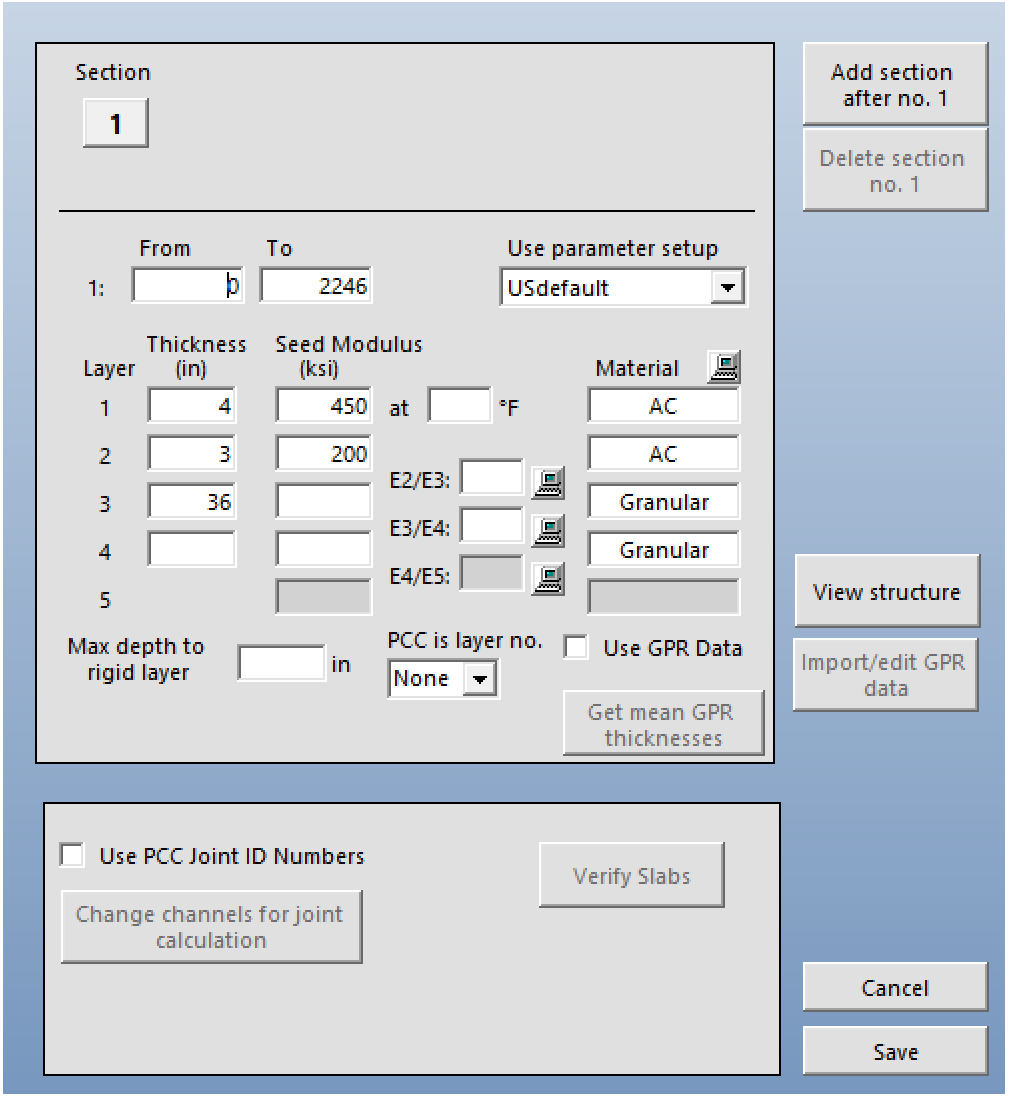

Section
1
Add section after no. 1
Delete section no. 1
From
To
Use parameter setup
1:
0
3386.1001
US_PCN-Flex
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
3
300
at
°F
AC
2
9
150
Base
E2/E3:
3
12
50
Base
E3/E4:
4
Subgrade
E4/E5:
5
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

D.4 Glenn / Parks Highway Interchange

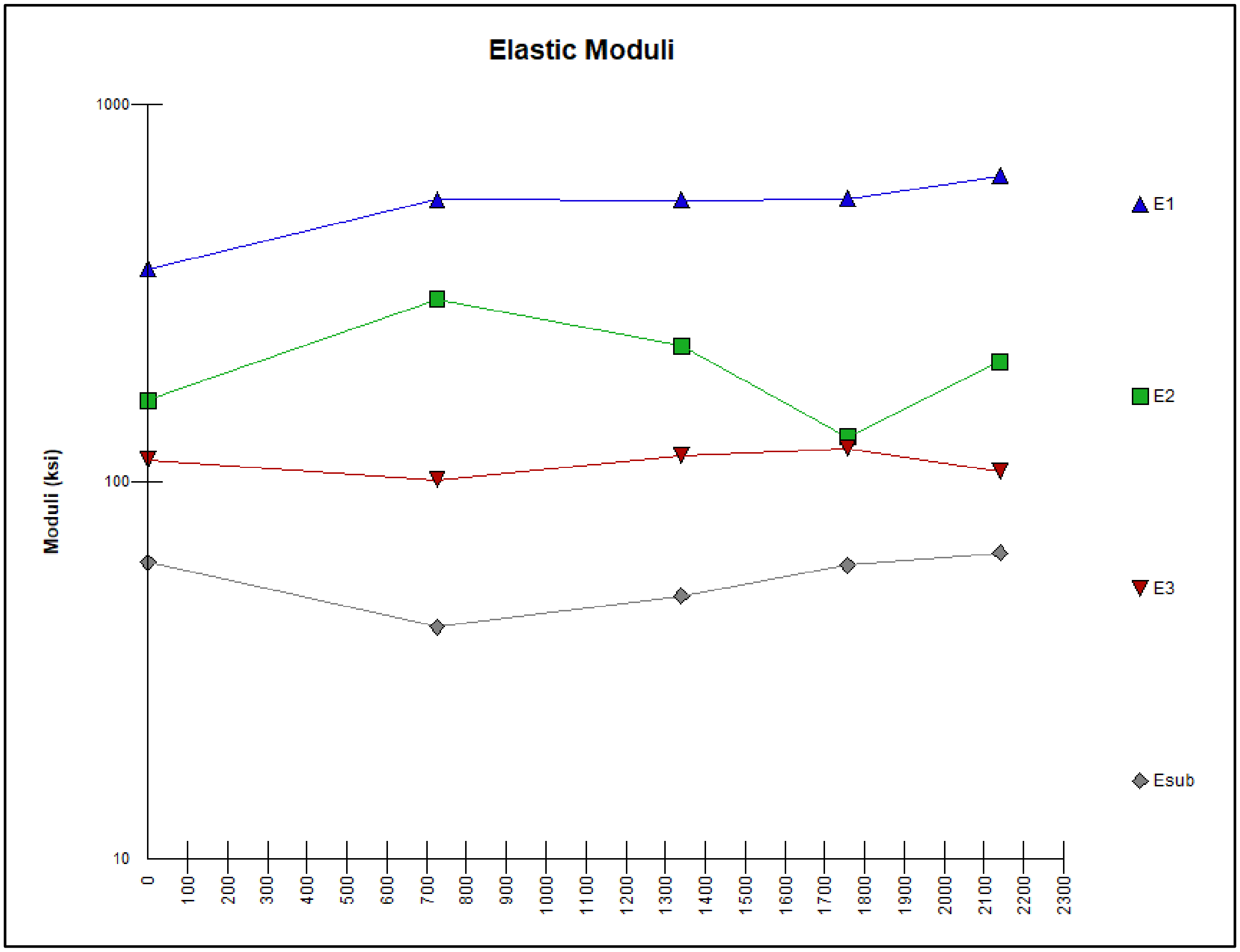
D.4.1 Glenn/Parks Northbound Direction



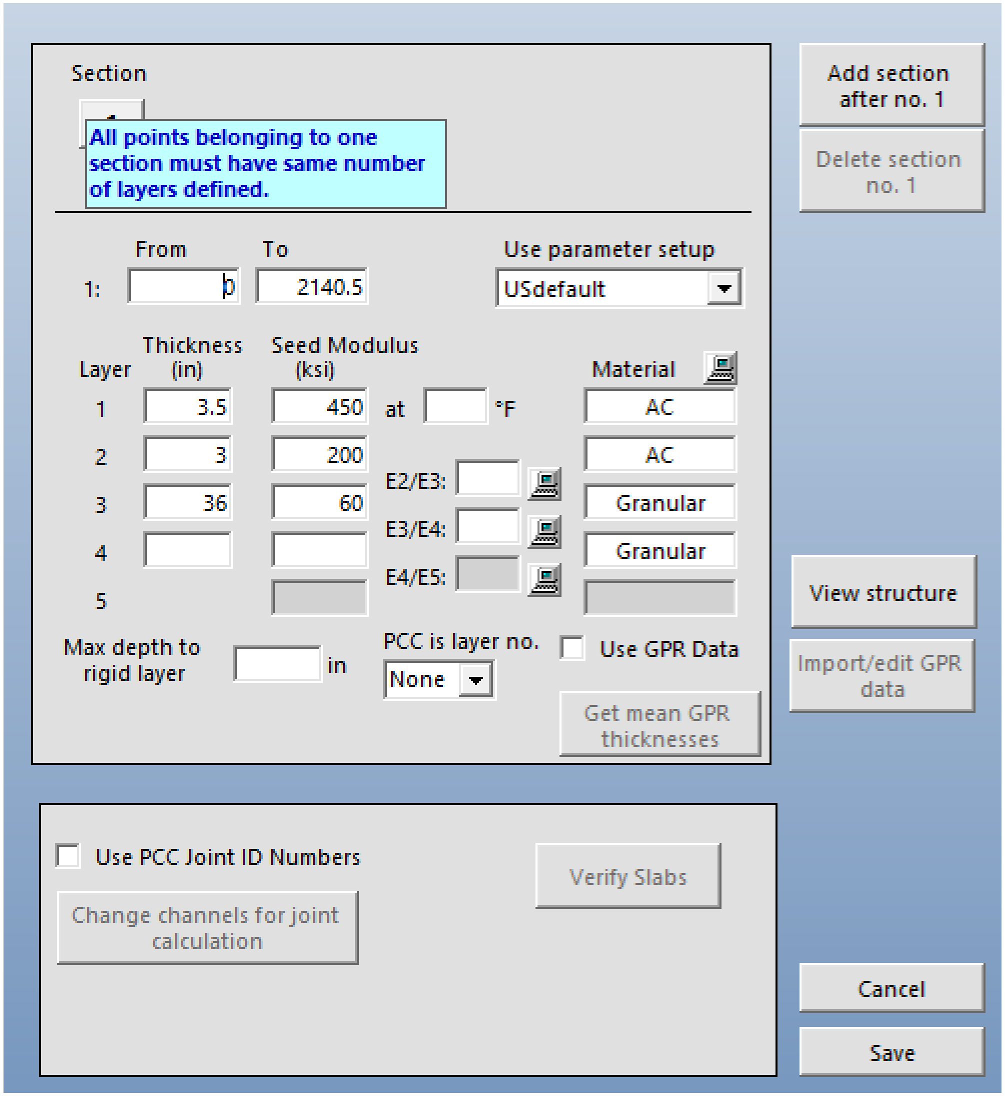
Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type VH	PG64-40	4"	503.1	1.246
RAP			3"	258.1	1.207
Granular	Select A		36"	89	1.12
Subgrade				54.1	1.251


Section
1
Add section after no. 1
Delete section no. 1
From
To
Use parameter setup
1:
0
2246
USdefault
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
4
450
at
°F
AC
2
3
200
AC
E2/E3:
3
36
Granular
E3/E4:
4
Granular
E4/E5:
5
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

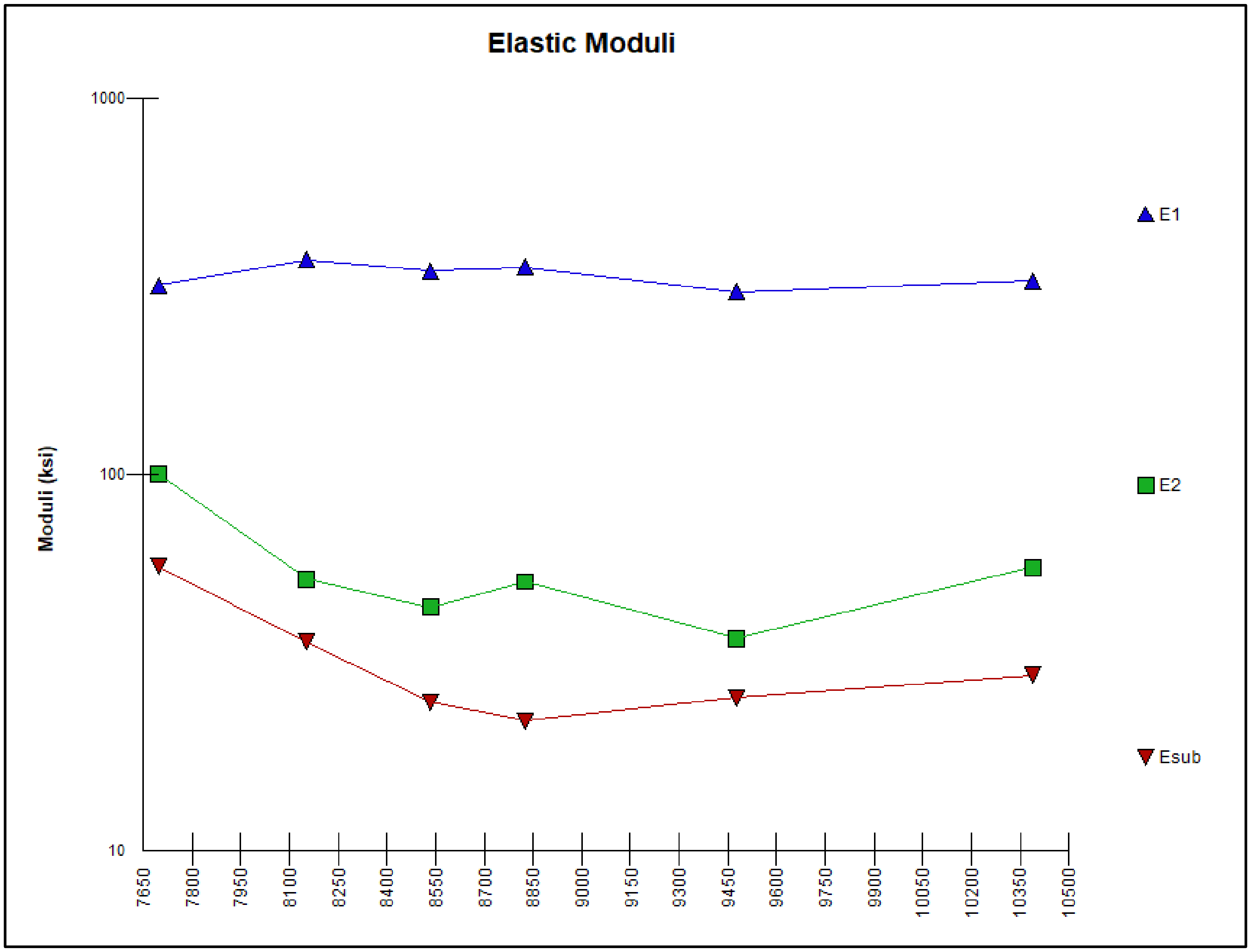
D.4.2 Glenn/Parks Southbound Direction



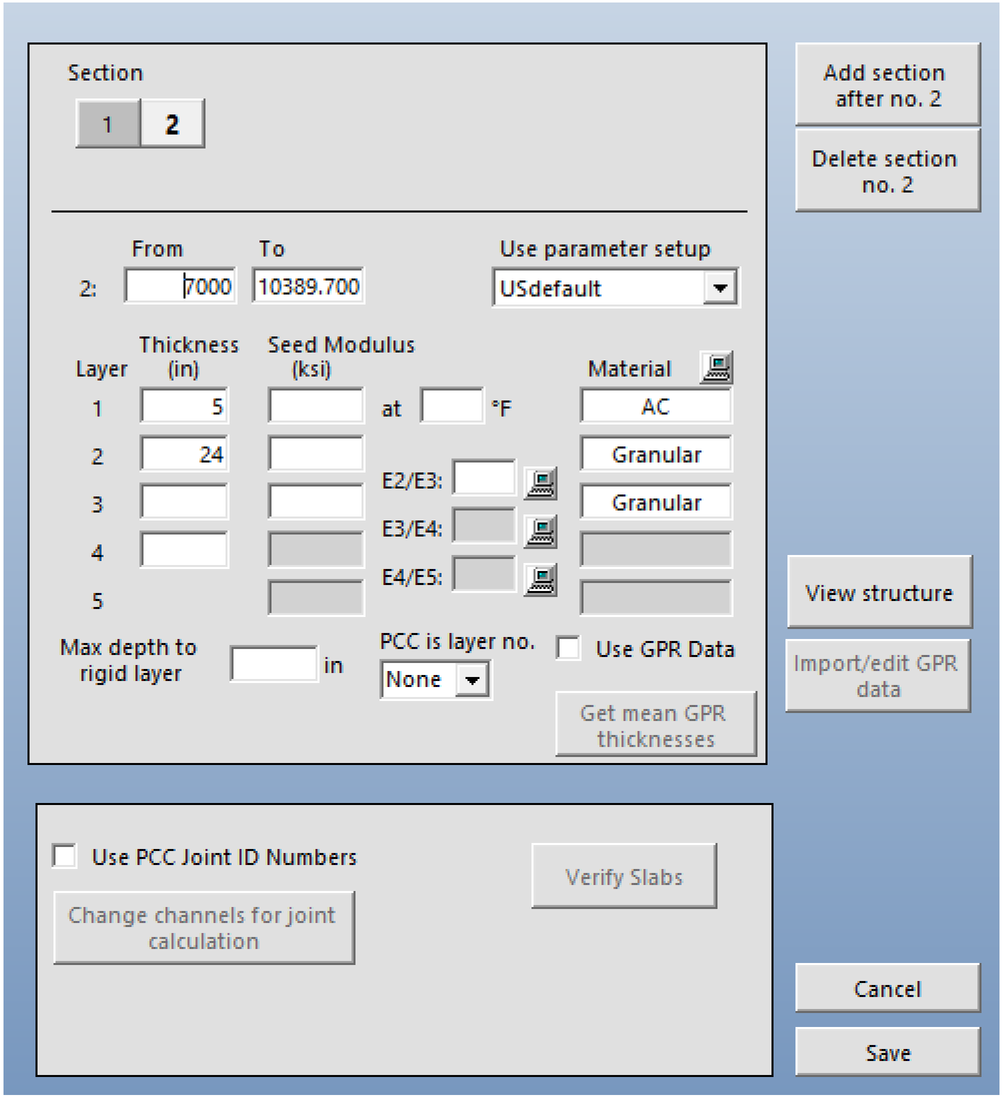
Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type VH	PG64-40	3.5"	528	1.24
RAP			3"	198.7	1.379
Granular	Select A		36"	111.8	1.079
Subgrade				54.5	1.205


Section
All points belonging to one section must have same number of layers defined.
Add section after no. 1
Delete section no. 1
From
To
Use parameter setup
1:
0
2140.5
USdefault
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
3.5
450
at
°F
AC
2
3
200
AC
E2/E3:
3
36
60
Granular
E3/E4:
4
Granular
E4/E5:
5
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

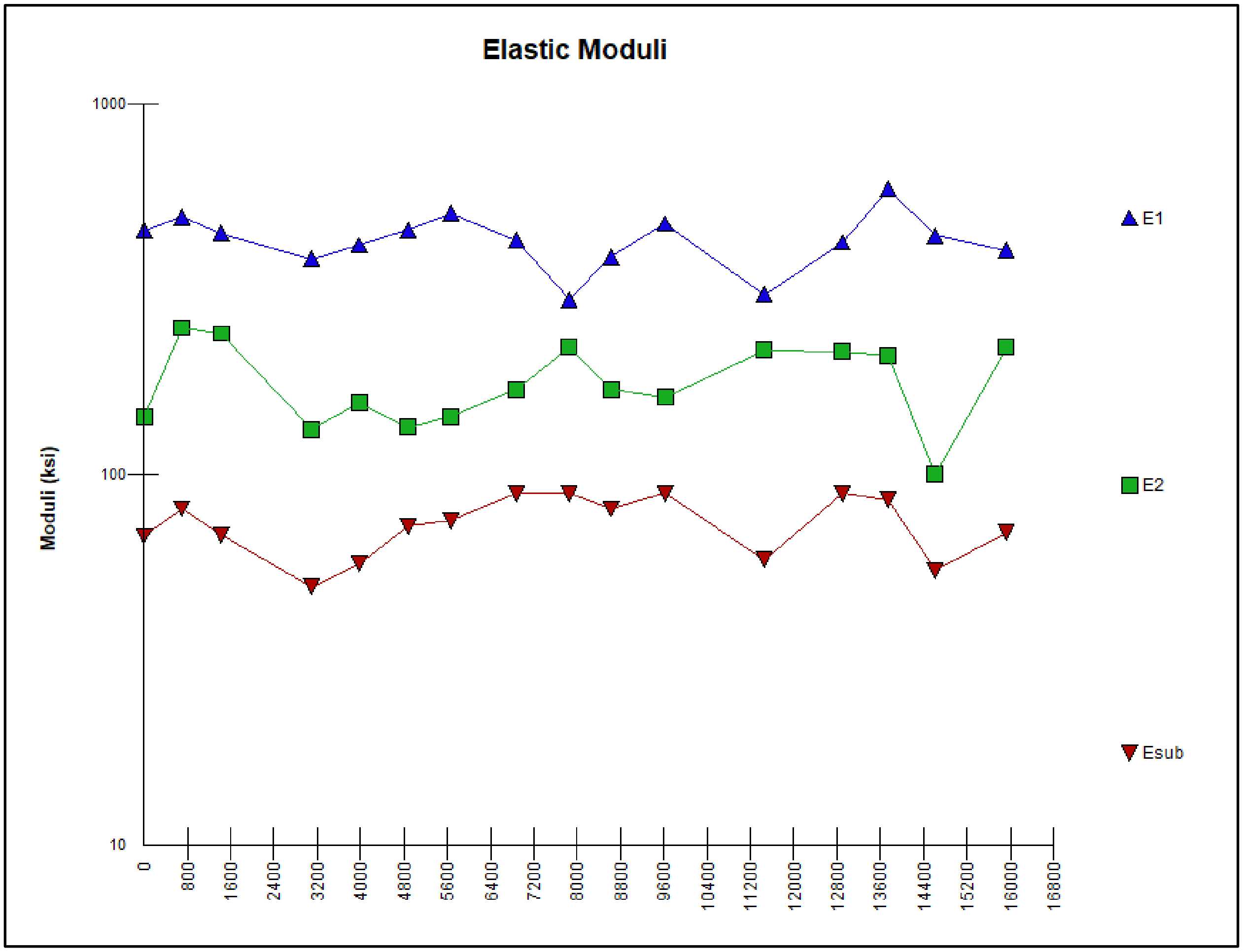
D.5 Kalifornsky Beach Road



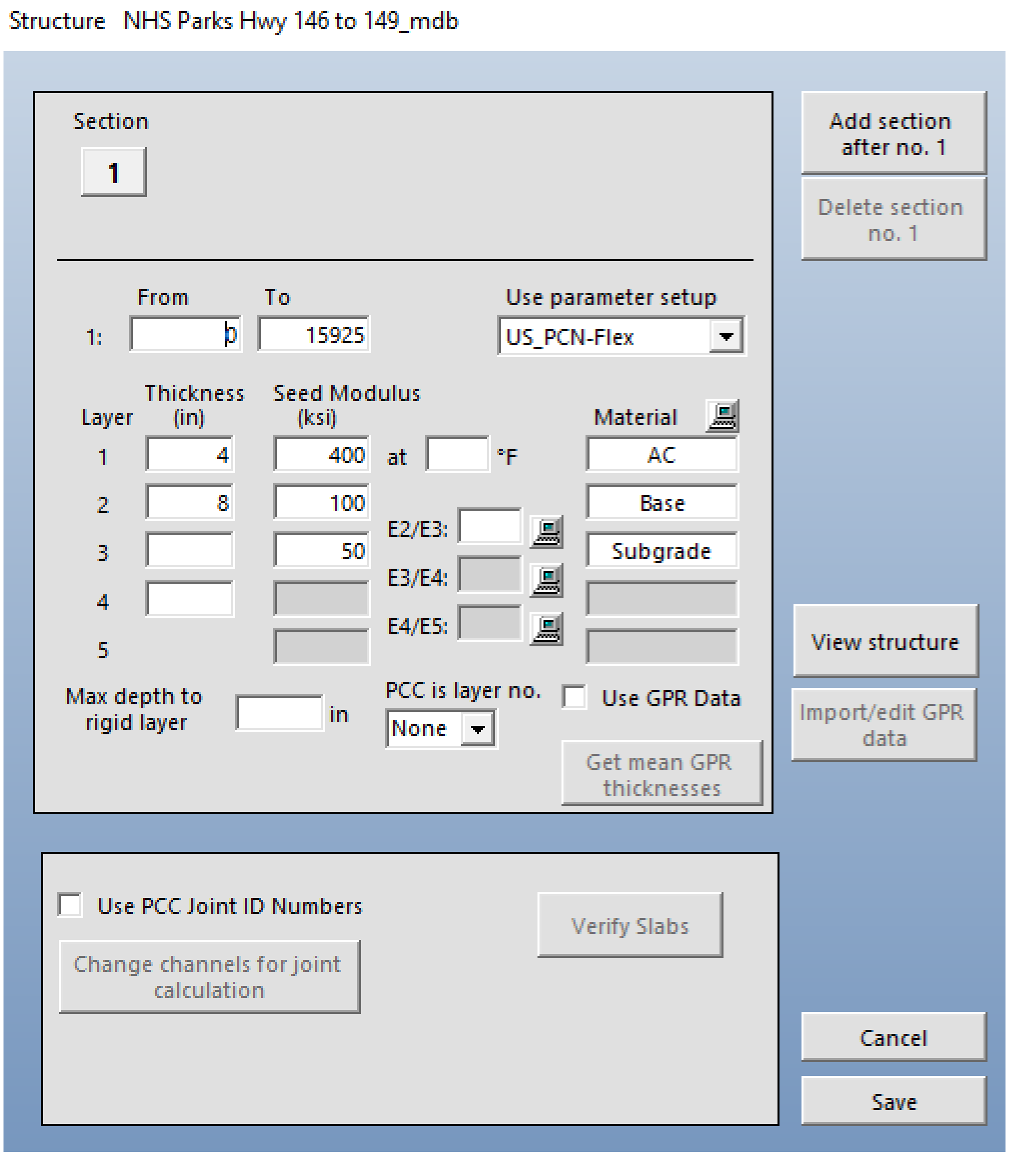
Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type VH	PG52-40	2"	335.8	1.077
HMA	ATB	PG52-40	3"		
Granular	Select A		24"	48.28	1.146912
Subgrade				27.46	1.172948
Note* - HMA Layers Combined for Analysis					


Section
1
2
Add section after no. 2
Delete section no. 2
From
To
2:
7000
10389.700
Use parameter setup
USdefault
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
5
at
°F
AC
2
24
Granular
3
Granular
4
5
E2/E3:
E3/E4:
E4/E5:
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

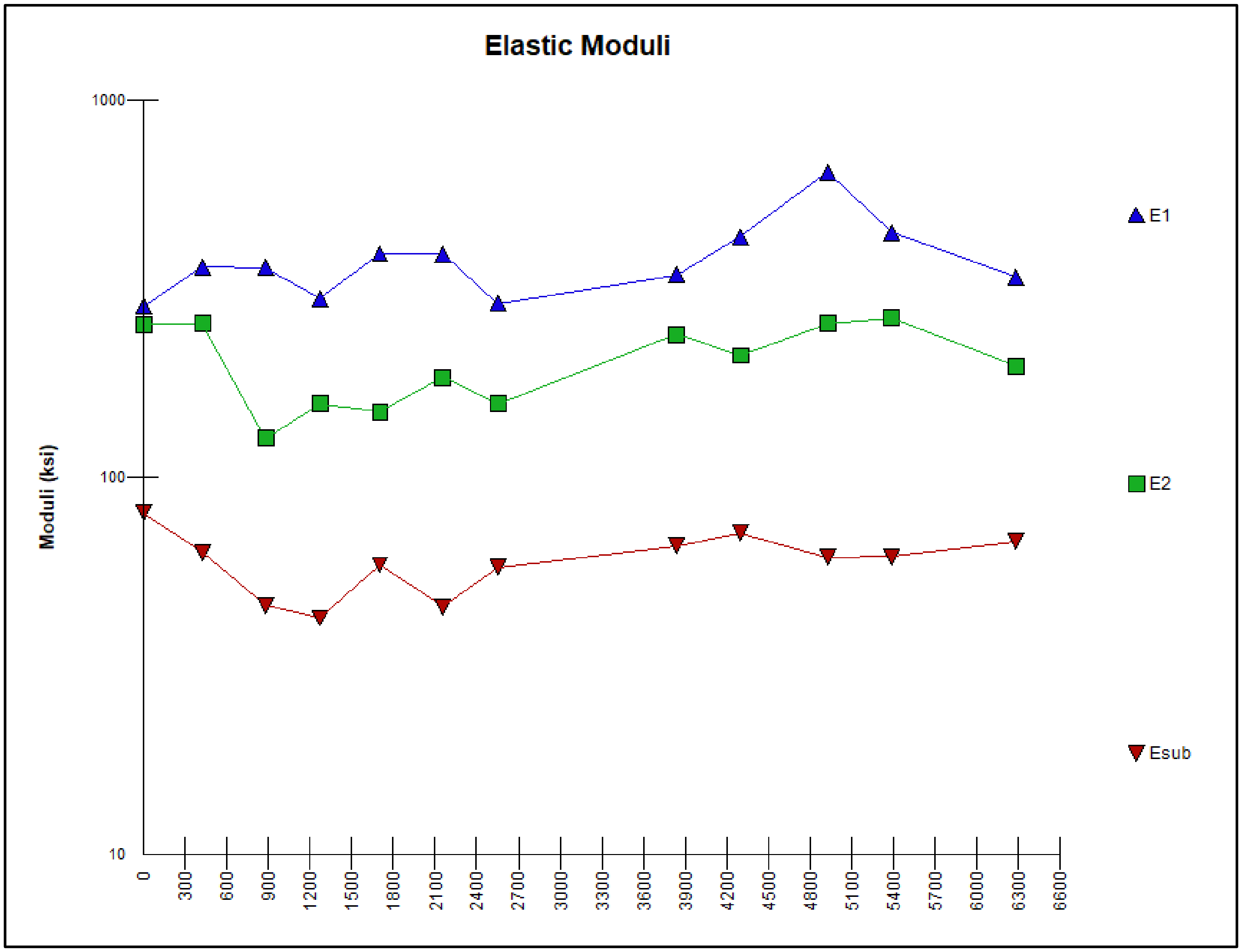
D.6 Parks Highway MP 146-149



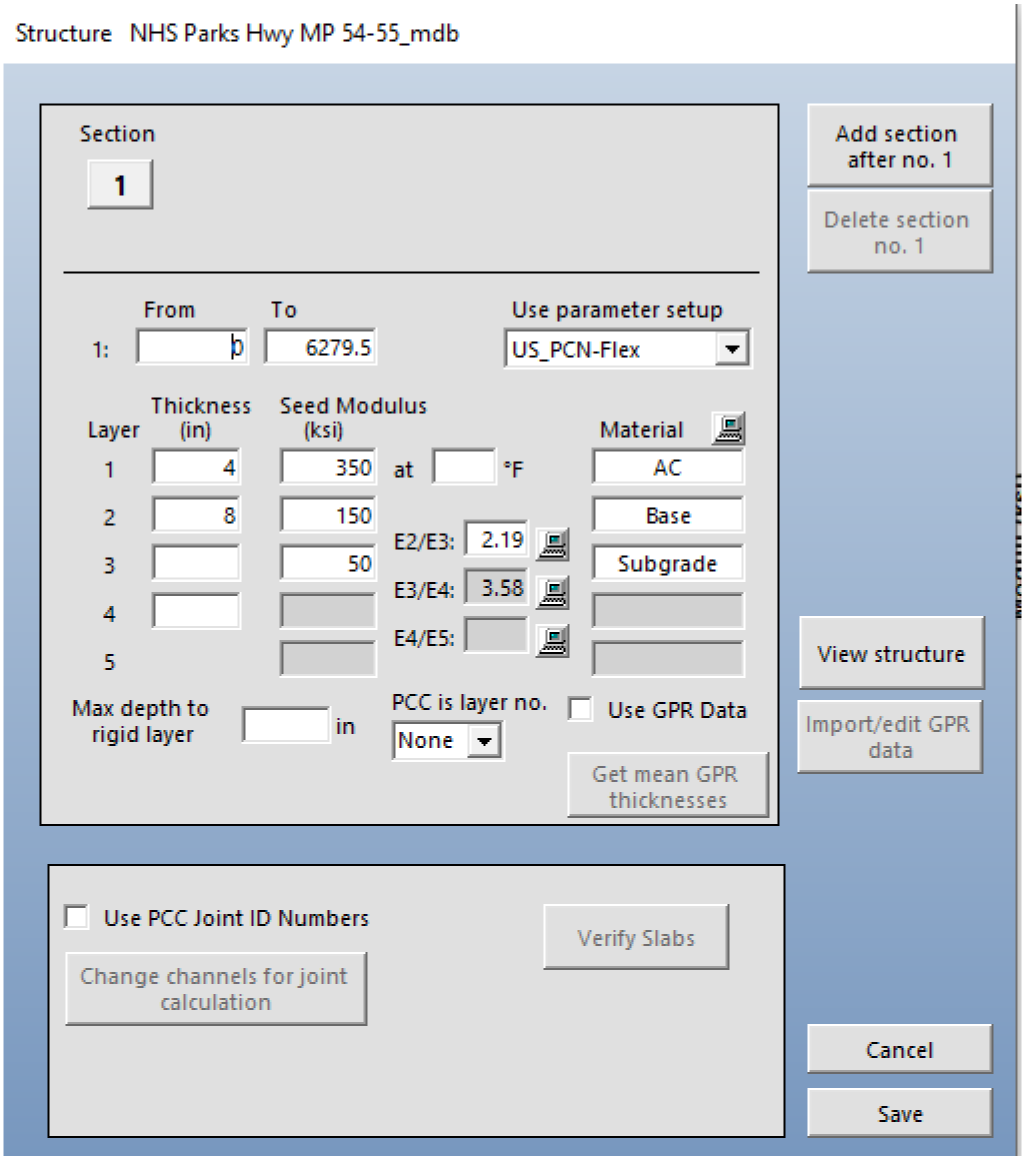
Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type II Class A	PG58-34	4"	425.1	1.189
Base	Foamed Asphalt Stabilized		8"	174.2	1.297
Subgrade				72.2	1.209


Structure NHS Parks Hwy 146 to 149_mdb
Section
1
Add section after no. 1
Delete section no. 1
From
To
Use parameter setup
1:
0
15925
US_PCN-Flex
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
4
400
at
°F
AC
2
8
100
Base
E2/E3:
3
50
Subgrade
E3/E4:
4
E4/E5:
5
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

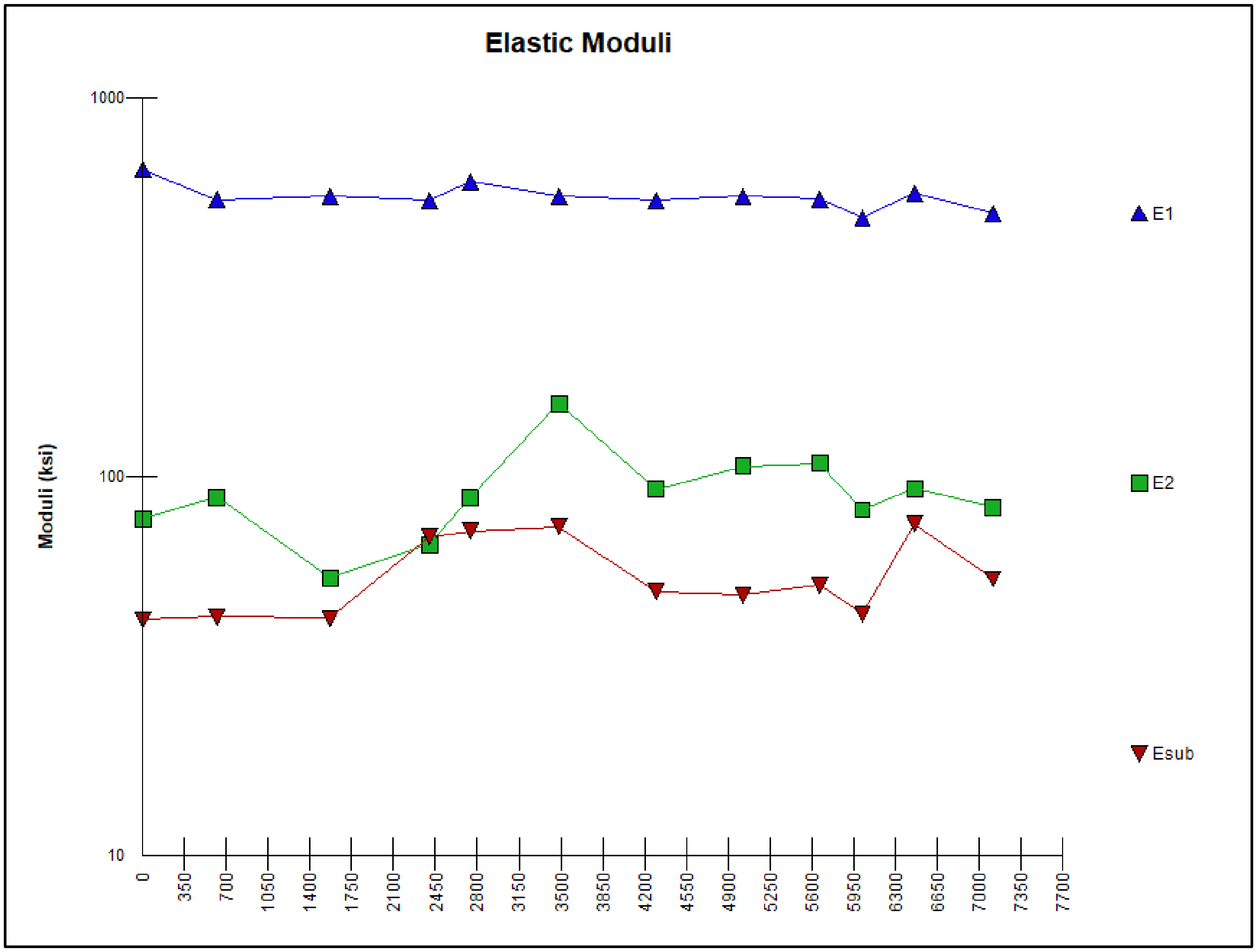
D.7 Parks Highway MP 54-55



Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type II Class A	PG 52-34	4"	371.6	1.253
Base	Foamed Asphalt Stabilized		8"	198	1.29
Subgrade				58.9	1.215

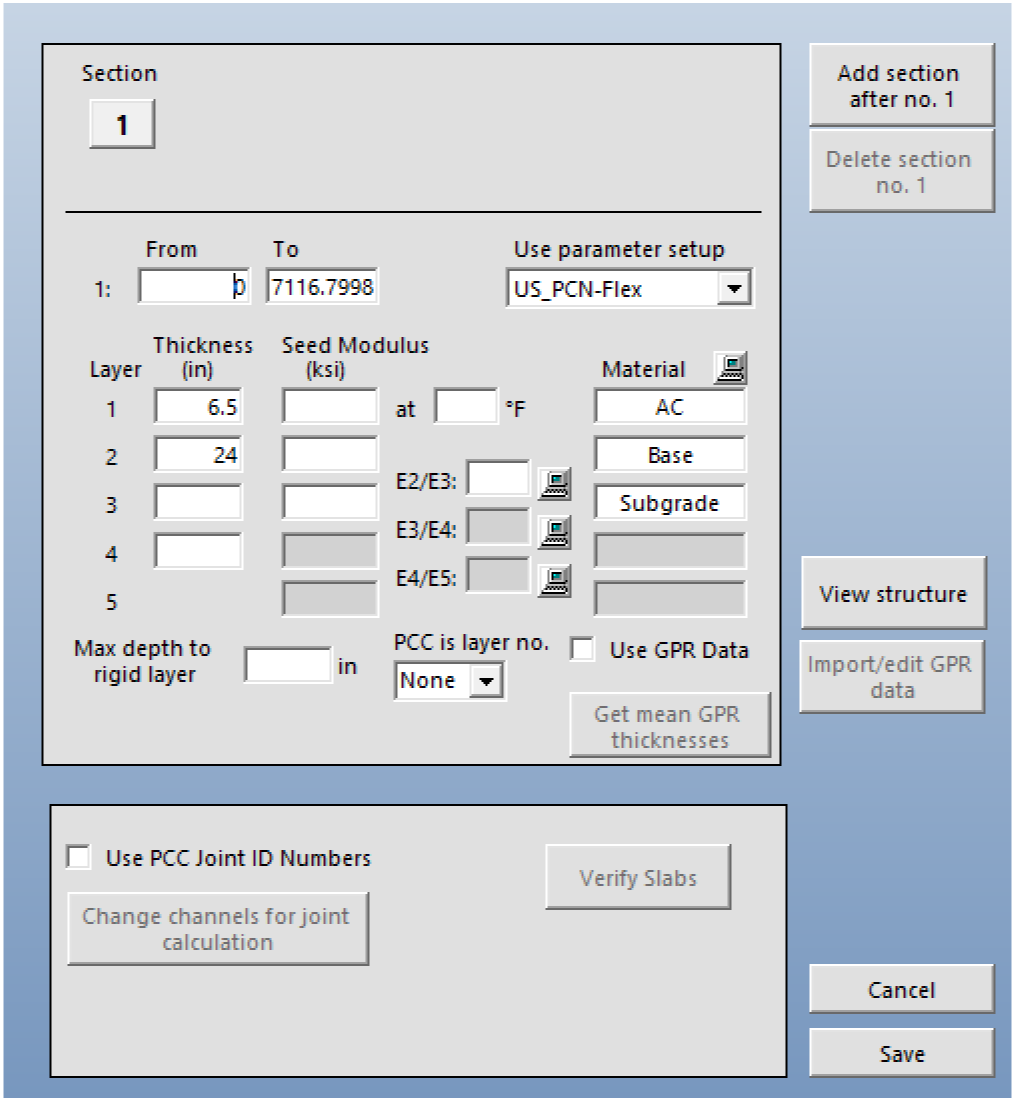

Structure NHS Parks Hwy MP 54-55_mdb
Section
1
Add section after no. 1
Delete section no. 1
From
To
Use parameter setup
1:
0
6279.5
US_PCN-Flex
Layer
Thickness (in)
Seed Modulus (ksi)
Material
1
4
350
at
°F
AC
2
8
150
Base
E2/E3:
2.19
3
50
Subgrade
E3/E4:
3.58
4
E4/E5:
5
View structure
Max depth to rigid layer
in
PCC is layer no.
None
Use GPR Data
Import/edit GPR data
Get mean GPR thicknesses
Use PCC Joint ID Numbers
Verify Slabs
Change channels for joint calculation
Cancel
Save

D.8 Parks Highway MP 72-74

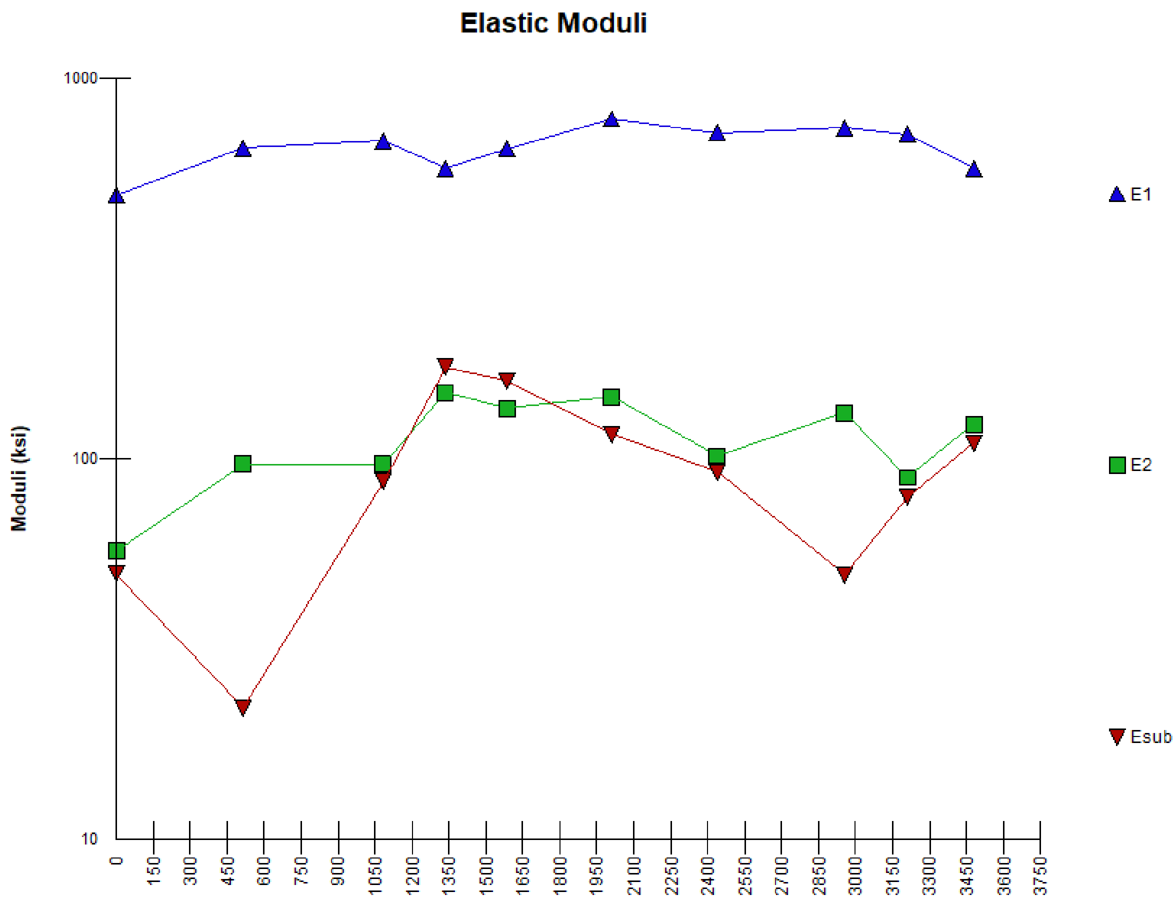


Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type II Class A	PG52-28	6.5"	546.2	1.08
Granular	Select A		24"	88.2	1.3
Subgrade				53.8	1.263

Structure NHS Parks Hwy MP 72 NB_mdb

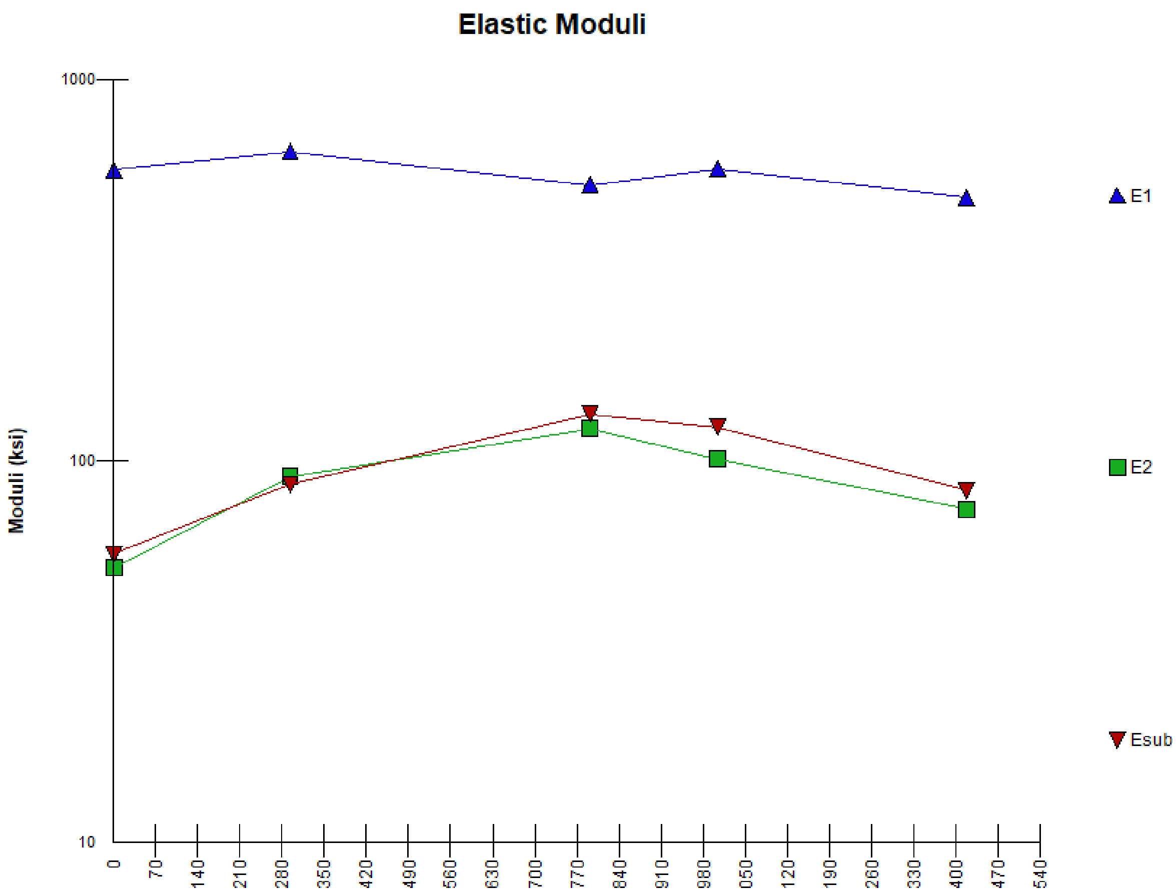


D.9 Seward Highway Dowling to Tudor



Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type V*	PG58-34	2"	652.7	1.15
HMA	ATB*	PG58-34	5.5"		
Granular	Select A		26"	108.5	1.34
Subgrade				80.9	1.858
Note* - HMA Layers Combined for Analysis Note – Subgrade is throughout variable conditions (fill and native materials)					

D.10 Seward Highway Dimond to Dowling



Layer	Type	Binder	Thickness	Modulus	Std
HMA	Type V*	PG58-34	2"	561.3	1.111
HMA	ATB*	PG58-34	5.2"		
Granular	Select A		26"	84.5	1.378
Subgrade				92.2	1.402
Note* - HMA Layers Combined for Analysis Note – Subgrade is throughout variable conditions (fill and native materials)					

APPENDIX E – PAVEMENT MANAGEMENT DATA

Pavement condition data collected between 2015 and 2021 was analyzed for the project areas listed below. As-builts for these projects is available through the Alaska DOT&PF E-Document System, that is publicly available to query.

These project areas were attributed with the number of lanes, traffic volumes, mix type, binder type, and pavement conditions (rut, IRI, transverse cracking) by pavement data collection year. These project areas were split into families for data analysis based on mix type, binder type and speed as presented in Chapter 3.3.

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
A Street	57835	2014	0.2	1.0	CR	Type VH	64-40	40
Abbott Loop Extension	56559	2008	0.3	2.1	CR	Type R	70-34	45
Alaska Highway MP 1267-1314	60502	2009	44.0	89.8	NR	Type II Class A,B	52-28	60
Alaska Hwy. MP 1222-1235 Rehabilitation	60519	2010	0.7	12.8	NR	Type II Class A,B	52-28	60
Benson Blvd.: Lois Dr. to Latouche St.	58497	2016	0.2	1.8	CR	Type VH	58-34	45
C Street	57835	2014	1.6	2.5	CR	Type VH	64-40	40
Copper River Highway MP 0-6	63804	2012	0.0	6.0	NR	Type II Class A,B	52-28	35
Dalton Highway MP 197-209	61214	2012	198.2	210.8	NR	Type II Class A,B	52-40	60
Debarr Rd. Resurfacing	53433	2013	0.5	3.5	CR	Type VH	58-34	45
Dimond Boulevard Resurfacing: Jewel Lake to Seward Highway	53801	2013	0.0	0.6	CR	Type VH	58-34	40
Dimond Boulevard Resurfacing: Jewel Lake to Seward Highway	53801	2013	0.6	2.4	CR	Type VH	58-34	45

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
Dowling Rd: Old Seward to LOP Pavement Pres. (GF)	30075	2017	1.0	2.0	CR	Type VH	64-40	45
Eagle River Loop Road Rehabilitation	53936	2009	0.0	1.9	CR	Type V	64-34	45
East Dowling Road Extension and Recon.	58592	2009	0.0	1.0	CR	Type R	64-34	45
Geist Road Rehabilitation	63360	2011	0.0	0.9	NR	Type II Class A,B	52-28	55
Glenn Highway MP 109-118 Resurfacing	52095	2011	100.8	109.8	CR	Type II Class A,B	52-34	50
Glenn Highway: Airport Heights to Hiland	30179	2017	0.0	10.0	CR	Type VH	64-40	65
Glenn Highway: Airport Heights to Hiland SB	30179	2017	34.3	24.3	CR	Type VH	64-40	65
Glenn Highway: Eklutna to Parks Highway PP	58534	2018	24.7	32.3	CR	Type VH	64-40	65
Glenn Highway: Eklutna to Parks Highway PP SB	58534	2018	9.6	2.0	CR	Type VH	64-40	65
Glenn Highway: Hiland to Eklutna	30057	2017	12.1	24.7	CR	Type VH	64-40	65
Glenn Highway: Hiland to Eklutna SB	30057	2017	22.1	9.6	CR	Type VH	64-40	65
Glenn Hwy MP 173-184	60922	2013	162.7	180.0	NR	Type II Class A,B	52-28	60
HNH Ferry Terminal to Airport Pvmt. Rehab. (Various Routes)	68909	2010	0.0	1.2	SCR	Type II Class A,B	58-28	35
HNH Ferry Terminal to Airport Pvmt. Rehab. (Various Routes)	68909	2010	0.0	1.0	SCR	Type II Class A,B	58-28	35

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
HNH Ferry Terminal to Airport Pvmt. Rehab. (Various Routes)	68909	2010	0.0	0.8	SCR	Type II Class A,B	58-28	35
HNS Front St. to Union St. Refurbishment	68948	2010	0.0	0.6	SCR	Type II Class A,B	58-28	40
Holt-Lamplight Road Resurfacing	52638	2012	0.0	8.3	CR	Type II Class A,B	52-28	55
Huffman Old Seward to Pintail Reconstruction	51920	2011	0.0	1.5	CR	Type V	64-34	50
INTL Airport RD: S.Aircraft-Homer Dr Preservation	56729	2016	0.0	3.7	CR	Type VH	58-34	45
JNU North Douglas Hwy. Pavement Rehab.	69633	2011	6.0	12.0	SCR	Type II Class A,B	64-28	45
JNU Thane Road Pavement Rehab.	69340	2012	0.3	4.8	SCR	Type II Class A,B	52-28	35
KTN-North Tongass Hwy. Totem Bight to Whipple Crk.	67443	2011	7.5	9.2	SCR	SP Type B	64-28	55
L and I Couplet Pavement Preservation Phase 1 (I Street)	58003	2015	0.0	0.6	CR	Type VH	64-40	35
L and I Couplet Pavement Preservation Phase 1 (L Street)	58003	2015	0.3	0.6	CR	Type VH	64-40	35
Mat-Su Pavement Rehabilitation 2007	60008	2008	4.0	11.1	CR	Type II Class A,B	52-28	45
Mat-Su Roads Surface Treatment	52830	1997	0.6	3.4	CR	High Float	HFMS-2S	0
Mendenhall Loop Rd Pavement Rehab.	71523	1995	0.0	2.2	SCR	Type II Class A,B	PBA-2	40
Minnesota Dr Resurfacing: Int'l Airport Rd to 13th	51135	2009	4.8	6.4	CR	Type V	58-34	45

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
Minnesota Dr Resurfacing: Int'l Airport Rd to 13th SB	51135	2009	1.0	2.4	CR	Type V	58-34	45
Minnesota Drive C Street to International	51340	2009	0.7	3.9	CR	Type V	64-34	65
Northern Lights Blvd Pave.Preservation	58496	2016	4.4	6.1	CR	Type VH	58-34	45
Old Glenn Hwy, Fire Lake to S. Birchwood Loop	58061	2010	3.0	3.9	CR	Type II Class A,B	52-28	45
Old Glenn Hwy: MP 11.5-18	57039	2009	11.6	18.4	CR	Type II Class A,B	52-28	55
Old Seward Hwy, Brandon St to O'Malley Rd	56958	2010	1.8	3.2	CR	Type V	64-34	45
Parks Highway 351-356	66438	2008	289.1	317.4	NR	Type II Class A,B	52-28	60
Parks Highway MP 146 to 163 Pavement Preservation	54147	2013	110.7	127.7	CR	Type II Class A,B	58-34	65
Parks Highway MP 252-263	63655	2013	216.4	227.3	NR	Type II Class A,B	52-28	60
Parks Highway MP 43.5-44.5	52914	2015	8.3	9.3	CR	Type II Class A,B	58-34	50
Parks Highway, MP 52-57 Big Lake to Houston Resurf	53469	2012	16.7	21.7	CR	Type II Class A,B	52-34	55
Parks Hwy MP 287-305 Rehab & Resurface	77137	2010	251.4	269.5	NR	Type II Class A,B	52-34	65
Parks Hwy MP 35-40 Pavement Preservation	56703	2016	9.6	13.2	CR	Type VH	64-40	65
Parks Hwy MP 35-40 Pavement Preservation SB	56703	2016	0.5	4.1	CR	Type VH	64-40	65
Parks Hwy MP 39-42 Seward Meridian to Crusey	52474	2004	4.2	6.6	CR	SMA	58-28	45
Parks Hwy MP 72-83, Willow to Kashwitna	54985	2011	36.6	47.5	CR	Type II Class A,B	52-28	65

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
Parks Hwy. MP 83-90, Weight Restriction Elim.	51850	2013	47.5	54.5	CR	Type II Class A,B	58-34	65
Richardson Highway MP 148-159 Reconstruction	76727	2013	151.0	162.0	NR	Type II Class A,B	52-28	60
Richardson Highway MP 340-346	63362	2012	341.6	347.5	NR	Type II Class A,B	52-28	45
Seward Highway MP 0-8 Phase II	55352	2005	0.0	4.8	CR	Type V	64-28	45
Seward Highway MP 115-124 Resurfacing	52491	2011	113.8	121.0	CR	Type R	58-28	65
Seward Highway MP 115-124 Resurfacing SB	52491	2011	4.1	8.9	CR	Type R	58-28	65
Seward Highway Reconstruction Dimond to Dowling	30162	2018	121.3	122.8	CR	Type VH	58-34	65
Seward Highway Reconstruction Dimond to Dowling SB	30162	2018	2.5	4.0	CR	Type VH	58-34	65
Seward Hwy MP 43-50 Pvmt. Refurbishment	59296	2009	42.5	49.5	CR	Type II Class A,B	52-28	55
Seward Hwy MP 54.5-75 Pavement Preservation	58080	2015	49.5	74.1	CR	Type II Class A,B	52-28	65
Seward Hwy. Reconstruction, Dowling Rd to Tudor Rd	50816	2013	122.8	123.8	CR	Type VH	58-34	65
Seward Hwy. Reconstruction, Dowling Rd to Tudor Rd SB	50816	2013	1.5	2.5	CR	Type V	58-34	65
Soldotna: Funny River Road Improvement	53750	2017	0.2	1.0	CR	Type II Class A,B	52-40	40
Steese Hwy. MP 62-81	60916	2011	59.5	77.8	NR	Type II Class A,B	52-28	50
Sterling Highway MP 45-58 Resurfacing	52081	2013	13.2	21.8	CR	Type II Class A,B	58-34	60

Project Name	Project Num	Const Year	Begin Mpt	End Mpt	Region	Mix Type	Binder	Speed
Trunk Rd Reconst. PH I:Parks-PalmeRWasilla	59301	2011	0.0	2.9	CR	Type II Class A,B	52-28	55
Van Horn Road West (Cartwright Rd.) Improvement	62056	2011	0.4	1.7	NR	Type II Class A,B	52-28	45
West Dowling Road, Phase I	50898	2012	2.0	2.6	CR	Type V	58-34	50
Willow Fishhook Preventative Maintenance	58651	2016	20.3	31.5	CR	Type II Class A,B	52-40	35
YAK: Yakutat Areawide Paving	68345	2011	0.0	0.5	SCR	Type II Class A,B	58-28	0

APPENDIX F – SECTION 412 HOT MIX ASPHALT PAVEMENT

SECTION 412
HOT MIX ASPHALT PAVEMENT (Low VTM)

412-1.01 DESCRIPTION. Construct one or more courses of plant-produced Hot Mix Asphalt (HMA) pavement on an approved surface, to the lines, grades, and depths shown on the Plans. This HMA has a high polymer modified asphalt content and requires high field density.

MATERIALS

412-2.01 ASPHALT BINDER. Conform to Subsection 702-2.01. Meet AASHTO M332 Performance-Graded Asphalt Binder Using MSCR Test Specification; except, changes noted in Table 412-1 as Exceptions.

TABLE 412-1. EXCEPTIONS TO AASHTO M 332 SPECIFICATION

Performance Grade	AASHTO Spec.	Viscosity AASHTO T 316	MSCR, AASHTO T 350			PAV, Dynamic Shear AASHTO T 315	Direct Tension AASHTO T 314
			$J_{NR3.2}$ kPa^{-1}	J_{NR} Diff	% $Rec_{3.2}$		
PG 58-34E	M332	None	0.25 max.	Delete	85 min.	None	Delete
PG 64-40E	M332	1.0 PaS max.	0.10 max.	Delete	95 min.	5000 max @ 4°C	Delete

Asphalt binder may be conditionally accepted at the source providing a manufacturers certification of compliance according to Subsection 106-1.05, and following documents at delivery;

Furnish the following documents at delivery:

1. Manufacturer’s certificate of compliance (Subsection 106-1.05).
2. Conformance test reports for the batch (provide prior to delivery as noted above).
3. Batch number and storage tanks used.
4. Date and time of load out for delivery.
5. Type, grade, temperature, and quantity of asphalt binder loaded.
6. Type and percent of liquid anti-strip added.

412-2.02 LIQUID ANTI-STRIP ADDITIVE. Use anti-strip agents in the proportions determined by ATM 414 and included in the approved Job Mix Design (JMD). At least 90 percent of the aggregate must remain coated when tested according to ATM 414. The following minimum dose (percent) of liquid anti-strip by weight of asphalt binder is required:

Liquid Anti-strip Type	**Minimum Dose by Weight of Asphalt Binder, %**
Amines based	0.30
Phosphate Ester based	0.30
Organo-Silane based	0.05

412-2.03 JOINT ADHESIVE. Conform to Subsection 702-2.05.

412-2.04 JOINT SEALANT. Conform to Subsection 702-2.06.

412-2.05 WARM MIX ASPHALT. Warm mix additive is required. Conform to Subsection 702-2.07.

412-2.06 ASPHALT RELEASE AGENT. Conform to Subsection 702-2.08.

412-2.07 AGGREGATES. Aggregates shall consist of crushed stone, crushed gravel, crushed screenings, as required. Coarse aggregate is the material retained on the No. 4 sieve. Fine aggregate is the material passing the No. 4 sieve.

Use a minimum of three stockpiles of crushed aggregate of different gradations.

a. **Coarse Aggregate.** Coarse aggregate shall consist of sound, tough, durable particles, free from films of matter that would prevent thorough coating and bonding with the bituminous material and be free from organic matter and other deleterious substances. Coarse aggregate material shall conform to Table 412-2 Coarse Aggregate Material Requirements.

TABLE 412-2. COARSE AGGREGATE MATERIAL REQUIREMENTS

Material Test	Requirement	Standard
Resistance to Degradation	40% max. loss	AASHTO T 96
Micro-Deval	20% max. Loss	AASHTO T327
Soundness of Aggregates by Use of Sodium Sulfate	9% max. loss after 5 cycles:	AASHTO T 104
Clay lumps and friable particles	1.0% max.	AASHTO T 112
Percentage of Fractured Particles	98% min. two fractured faces by weight	ATM 305
Flat, Elongated, or Flat and Elongated Particles	8% max. at 5:1 [2]	ATM 306
Absorption	2.0% max.	ATM 308

1. The area of each face shall be equal to at least 75% of the smallest mid-sectional area of the piece. When two fractured faces are contiguous, the angle between the planes of fractures shall be at least 30 degrees to count as two fractured faces.
2. A flat particle is one having a ratio of width to thickness greater than five (5); an elongated particle is one having a ratio of length to width greater than five (5).

b. **Fine Aggregate.** Fine aggregate shall consist of clean, sound, tough, durable, angular shaped particles produced by crushing stone or gravel and shall be free from coatings of clay, silt, or other objectionable matter, and conform to Table 412-3 Fine Aggregate Material Requirements.

Natural (non-manufactured) sand is not allowed.

TABLE 412-3. FINE AGGREGATE MATERIAL REQUIREMENTS

Material Test	Requirement	Standard
Liquid limit,	25 max.	ATM 204
Plasticity Index	Non plastic	ATM 205
Soundness of Aggregates by Use of Sodium Sulfate	10% max. loss after 5 cycles	AASHTO T 104
Clay Lumps and Friable Particles	1.0% max.	AASHTO T 112
Sand Equivalent	35 min.	ATM 307
Uncompacted Void Content, (Fine Aggregate Angularity)	45% min.	AASHTO T 304, Method A

c. **Sampling.** The Engineer will sample according to ATM 301 for coarse and fine aggregate.

412-2.08 RECYCLED ASPHALT PAVEMENT. Recycled asphalt pavement (RAP) is not allowed.

412-2.09 JOB MIX DESIGN. Provide target values for gradation that satisfy both the broad band gradation limits shown in Table 412-4 and the requirements of Table 412-6.

TABLE 412-4 AGGREGATE GRADATION & VMA

Sieve Size	**¾ inch (19.5MM) mix % passing by weight**	**1/2 inch (12.5 mm) mix % passing by weight**
¾ inch (19.5mm)	**100**	
1/2 inch (12.5 mm)	**90-100**	100
3/8 inch (9.5 mm)	**72-88**	90-100
No. 4 (4.75 mm)	**53-73**	58-78
No. 8 (2.36 mm)	**38-60**	40-60
No. 16 (1.18 mm)	**26-48**	28-48
No. 30 (600 µm)	**18-38**	18-38
No. 50 (300 µm)	**11-27**	11-27
No. 100 (150 µm)	**6-18**	6-18
No. 200 (75 µm)	**3-6**	3-6
Minimum Voids in Mineral Aggregate (VMA)[1]	15	16

TABLE 412-5 MARSHALL MIX DESIGN (ATM 417) REQUIREMENTS

Stability, Pounds	2150 Min.
Voids in Total Mix (VTM)	2.5% ±0.2%
Compaction, Number of Blows Each Side of Test Specimen	50
Voids in Mineral Aggregate (VMA)	(Table 412-4)
Asphalt Content	5.5% - 8.0%
Dust-Asphalt Ratio[1]	0.6 - 1.2
Liquid Anti-Strip Additive[2]	(Section 412-2.02)
Max. rut by Asphalt Pavement Analyzer (APA)[3]	≤ 10mm @ 4000 passes

1. Dust-Asphalt ratio is the percent of material passing the No. 200 sieve divided by the percent of effective asphalt binder (calculated by weight).
2. By Weight of Asphalt Binder
3. ATM 419 at 250 psi hose pressure at 50ºC test temperature
4. The approved JMD will specify the Target Values (TV) for gradation, the TV for asphalt binder content, the Maximum Specific Gravity (MSG) of the HMA, the additives, and the recommended mixing temperature range.

Submit the following to the Engineer at least 15 days before the production of HMA:

1. A letter stating the location, size, and type of mixing plant. The letter shall state whether or not WMA will be used. The letter shall include the proposed gradation for the JMD, gradations for individual stockpiles, and the blend ratio of each aggregate stockpile.

2. Representative samples of each aggregate (coarse, intermediate, fine, blend material) in the proposed mix design. Furnish a total of 500 pounds of material in the proportional amounts in the proposed JMD.

3. Five separate 1-gallon samples of the asphalt binder proposed for use in the HMA. Include name of product, manufacturer, test results of the applicable quality requirements of Subsection 412-2.07, manufacturer's certificate of compliance according to Subsection 106-1.05, a temperature- viscosity curve for the asphalt binder or manufacturer's recommended mixing and compaction temperatures, and current Material Safety Data Sheet.

4. One sample, minimum one pint, of the anti-strip additive proposed, including name of product, manufacturer, and manufacturer's data sheet, and current Material Safety Data Sheet.

5. Testing results per Subsection 106-1.03.1 for each aggregate type proposed for use.

6. State the WMA technology (Subsection 702-2.07) to be used, location where additive will be introduced and manufacturer's recommended usage rate for each type of HMA. Supply a minimum of 2-pint samples for each proposed additive.

The Engineer will evaluate the material and the proposed gradation using ATM 417 and the requirements of Table 412-4 for the appropriate Type of HMA specified, and establish the approved JMD which will become a part of the Contract.

Obtain an approved JMD prior to shipment of aggregates to an asphalt plant site or producing HMA for payment, unless aggregate quality and gradation has been approved by the Engineer.

Changes. Submit a new JMD with changes noted and new samples in the same manner as the original JMD submittal when:

a. The results of the JMD evaluation do not achieve the requirements specified in Table 412-2
b. The asphalt binder source is changed
c. The source of aggregate, aggregate quality, or gradation is changed
d. The results of a test strip do not meet the specification-the Engineer may require a new JMD.

Changes to the JMD apply only to HMA produced after the approval of changes.

412-2.10 CONTROL STRIP. Full production shall not begin until an acceptable control strip has been constructed and accepted in writing by the Engineer. The Contractor shall prepare and place a quantity of asphalt according to the JMD. The underlying grade or pavement structure upon which the control strip is to be constructed shall be the same as the remainder of the course represented by the control strip.

The Contractor will not be allowed to place the control strip until the Contractor Quality Control Program (CQCP), showing conformance with the requirements of subsection 412-3.02, has been accepted, in writing, by the Engineer.

The control strip will consist of at least 250 tons. The control strip shall be placed in two lanes of the same width and depth to be used in production with a longitudinal cold joint. The cold joint must be constructed using the same procedure that will be used during production. The cold joint for the control strip will be an exposed construction joint at least four (4) hours old or when the mat has cooled to less than 160°F. The equipment used in construction of the control strip shall be the same type, configuration, and weight, to be used on the project.

The control strip shall be evaluated for acceptance as a single lot in accordance with the acceptance criteria in subsection 401-6.1 for aggregate gradation and asphalt binder content. The control strip shall be divided into three separate equal sub-lots. If the Composite Pay Factor is less than 1.000, the control strip is unacceptable.

Three 6-inch diameter core samples shall be cut from the compacted hot mix asphalt mat by the Contractor, at the locations marked by the Engineer. The core samples will be tested by the Department for density according to subsection 412-4.02. The Target Value for mat density is 96.0% of the MSG of the JMD. If the average density of the three cores is below the lower specification limit of 93.0%, the control strip is unacceptable. The three samples will be evaluated according to subsection 412-4.02. If the Density Pay Factor is less than 1.000, the control strip is unacceptable.

Three longitudinal joint cores centered on the longitudinal joint shall be cut by the Contractor, at the locations marked by the Engineer. The core samples will be tested by the Department according to subsection 412-4.02. The Target Value for joint density is 96.0% of the JMD MSG. If the average density of the three joint cores is below 93.0%, the control strip is unacceptable.

It is recommended the mat and joint density be evaluated using a PaveScan Rolling Density Meter (RDM) ground penetrating radar (GPR) system.

After completion of the control strip compaction, the Department will accept or reject the control strip within 48 hours.

If the control strip is unacceptable, necessary adjustments to the JMD, plant operation, placing procedures, and/or rolling procedures shall be made and another control strip shall be placed. Unacceptable control strips shall be removed at the Contractor's expense. For small projects, less than 3,000 tons, a control strip is not required.

The Engineer may require a separate test strip for paving on a bridge deck where vibratory compaction is not allowed

Refer to Subsection 412-5.01 for payment of test strips.

CONSTRUCTION REQUIREMENTS

412-3.01 PRE-PAVING MEETING. Meet with the Engineer for a pre-paving meeting in the presence of project superintendent and paving foreman at least five (5) working days before beginning paving operations. Submit a paving plan and pavement inspection plan at the meeting.

Include the following elements in the paving plan and address these elements at the meeting:

1. Sequence of operations
2. List of equipment that will be used for production, transport, pick-up (if applicable), laydown, and compaction
3. Summary of plant modifications (if applicable) for production of WMA
4. Procedures to produce consistent HMA
5. Procedures to minimize material and thermal segregation
6. Procedures to minimize premature cooling
7. Procedures to achieve HMA density
8. Procedures for joint construction including corrective action for joints that do not meet surface tolerance requirements
9. Quality control testing methods, frequencies and sample locations for gradation, asphalt binder content, and density, and
10. Any other information or procedures necessary to provide completed HMA construction that meets the contract requirements

Include the following elements in the pavement inspection plan and address these elements at the meeting:

11. Process for daily inspections

12. Means and methods to remove and dispose of project materials

412-3.02 CONTRACTOR QUALITY CONTROL PLAN (CQCP). Sample and test materials for quality control of the HMA according to Subsection 106-1.03. Submit to the Engineer at the "Pre-Paving Meeting," Subsection 412-3.01, the JMD and a documentation plan that provides a complete, accurate, and clear record of the sampling and testing results.

Failure to perform quality control forfeits the Contractor's right to a retest under Subsection 412-4.02

Provide copies of the documented sampling and testing results no more than 24 hours from the time taken

412-3.03 WEATHER LIMITATIONS. Place HMA on a stable/non-yielding roadbed. Do not place HMA when the base material is wet or frozen, or when weather conditions prevent proper handling or finishing of the mix. Do not place HMA when the roadway surface temperature is colder than 40° F, or after September 15^{th} without the Engineer's approval in writing.

412-3.04 EQUIPMENT, GENERAL. Use equipment in good working order and free of HMA buildup. Make all equipment available for inspection and demonstration of operation a minimum of 24 hours before placement of HMA.

412-3.05 ASPHALT MIXING PLANT. Meet AASHTO M 156. Use an HMA plant capable of producing at least 150 tons of HMA per hour noted on posted DEC air quality permit, designed to dry aggregates, maintain consistent and accurate temperature control, and accurately proportion asphalt binder and aggregates. Calibrate the HMA plant and furnish copies of the calibration data to the Engineer at least 24 hours before HMA production.

Provide a scalping screen at the asphalt plant to prevent oversize material or debris from being incorporated into the HMA.

Provide a tap on the asphalt binder supply line just before it enters the plant (after the 3-way valve) for sampling asphalt binder. Provide aggregate and asphalt binder sampling locations meeting OSHA safety requirements.

Use of belt conveyor scales to proportion plant blends and mixtures is permitted if the scales meet the general requirements for weighing equipment and are calibrated according to the manufacturer's instructions.

Daily Plant Quantity Summary. At the end of each day of production, provide the Engineer with a document listing the quantities of materials used to produce HMA for each JMF. The document must be electronically generated from the automated plant controls, or as approved in writing by the Engineer.

1. Total tons of HMA produced
2. Total combined dry weight of virgin aggregate (note moisture content)
3. Total dry weight of RAP (note moisture content)
4. Total weight of asphalt cement
5. Total weight of each additive included in the mix at the plant
6. A graph of the mix temperature discharged from the drum or pugmill at 1-minute intervals during the HMA production.
7. If HMA is produced at temperatures higher than 340°F, provide an asphalt binder sample from the plant asphalt storage tank to the Engineer to be tested in an AASHTO accredited lab as follows;
    a. Perform RTFO testing of the binder sample at the highest recorded mix temperature,
    b. then test the RTFO residue according to the specified binder tests to evaluate conformance to project specifications.
    c. If binder does not meet specifications, apply penalties to the binder and mix tonnage produced at temperatures above 340°F as specified,

412-3.06 HAULING EQUIPMENT. Haul HMA in trucks with tight, clean, smooth metal beds. Keep beds free of petroleum oils, solvents, or other materials that would adversely affect the mixture. Apply a thin coat of approved asphalt release agent to beds as necessary to prevent mixture adherence. Provide trucks with covers attached and available for use.

Do not haul HMA on barges.

When directed by the Engineer cover the HMA in the hauling vehicle(s).

412-3.07 ASPHALT PAVERS. Use self-propelled asphalt pavers with heated vibratory screed assemblies to spread and finish HMA to the specified section widths and thicknesses without introducing thermal or material segregation.

Equip the paver with a receiving hopper having sufficient capacity for a uniform spreading operation and a distribution system to place the HMA uniformly in front of screed. Use a screed assembly that produces a finished surface of the required smoothness, thickness and texture without tearing, shoving or displacing the HMA. Heat and vibrate screed extensions.

It is recommended that the paver be equipped with an infrared joint heater and a notch wedge joint maker (6” wide) with compactor for longitudinal joint construction

Equip the paver with automatic screed controls capable of operating from a reference line or a ski from either or both sides of the paver.

412-3.08 ROLLERS. Use rollers designed to compact HMA and capable of reversing without shoving or tearing the mixture. Select rollers that will not crush the aggregate or displace the HMA. Equip vibratory rollers with separate vibration and propulsion controls. Rollers using intelligent compaction technology are recommended.

Equip the rollers with an infrared thermometer that measures and displays the surface temperature to the operator. Infrared thermometer may be hand-held or fixed to the roller.

Compact a leveling course with a pneumatic roller in the complement of rollers. Use fully skirted pneumatic-tire roller having a minimum operating weight of 3000 pounds per tire.

412-3.09 RESERVED.

412-3.10 PREPARATION OF EXISTING SURFACE. Prepare existing surfaces according to the Contract. Prior to placing HMA, clean existing surfaces of loose material and uniformly coat contact surfaces of curbing, gutters, manholes and other structures with tack coat material meeting Section 402. Treat cold joint surfaces according to 412-3.17.

Before applying tack coat to an existing paved surface, clean and patch the surface as directed by the Engineer. Remove irregularities to provide a reasonably smooth and uniform surface. Remove and replace unstable areas with HMA. Clean the edges of existing pavements, which are to be adjacent to new pavement, to permit the adhesion of asphalt materials. Clean loose material from cracks. Fill the cleaned cracks, wider than 1 inch, with HMA tamped in place. Wash and/or sweep the paved surface clean and free of loose materials.

Preparation of a milled surface:

1. Prelevel remaining ruts, pavement delaminations, and depressions having a depth greater than 1/2 inch with an approved HMA.

2. Notify the Engineer of pavement areas that appear thin or unstable. Where milling operation creates thin or unstable pavement areas, or where it breaks through existing pavement, remove thin and

unstable pavement, and 2 inches of existing base material, compact and replace with an approved HMA.

412-3.11 PREPARATION OF ASPHALT. Provide a continuous supply of asphalt binder to the asphalt mixing plant at a uniform temperature, within the recommended mixing temperature range.

412-3.12 PREPARATION OF AGGREGATES. Dry the aggregate so the moisture content of the HMA, sampled at the point of acceptance for asphalt binder content, does not exceed 0.5 percent (by total weight of mix), as determined by ATM 407.

Heat the aggregate for the HMA to a temperature compatible with the mix requirements specified.

Adjust the burner on the dryer to avoid damage to the aggregate and to prevent the presence of unburned fuel on the aggregate. HMA containing soot or fuel is unacceptable per Subsection 105-1.11.

412-3.13 MIXING. Combine the aggregate, asphalt binder, and additives in the mixer in the amounts required by the JMD. Mix to obtain at least 98 percent coated particles when tested according to AASHTO T195.

Mix the dried heated aggregates before adding asphalt binder.

Produce the HMA within the temperature range determined by the JMD.

Upon the Engineer's request, provide daily burner charts showing start/stop times and temperatures.

412-3.14 TEMPORARY STORAGE OF HMA. Silo type storage bins may be used, provided the characteristics of the HMA remain unaltered. Gob hoppers must be used to drop HMA into the silo.

Signs of visible segregation, heat loss, changes from the JMD, change in the characteristics of asphalt binder, lumpiness, and stiffness of the mixture, are causes for rejection.

Do not store HMA on barges.

412-3.15 PLACING AND SPREADING. Use asphalt pavers to distribute HMA, including leveling course and temporary HMA. Place the HMA upon the approved surface, spread, strike off, and adjust surface irregularities. The maximum compacted lift thickness allowed is 3 inches.

When multiple lifts are specified in the Contract, do not place the final lift until all lower lifts throughout that section, are placed and accepted.

Do not place HMA abutting curb and gutter until curb and gutter are installed, except as approved by the Engineer.

Do not pave against new Portland cement concrete curbing until it has cured for at least 72 hours.

When practicable, adjust elevation of metal utility fixtures before paving the final lift, so they will be between 1/4 and 1/2 inch below the top surface of the final lift. Metal fixtures include, but are not limited to manholes, valve boxes, monument cases, hand holes, and drains.

Use hand tools to spread, rake, and lute the HMA in areas where irregularities or unavoidable obstacles make mechanical spreading and finishing equipment impracticable.

Place HMA over bridge deck membranes according to Section 508 and the membrane manufacturer's recommendations.

Do not mix HMA produced from different plants for testing or paving unless approved by the Engineer.

412-3.16 COMPACTION. Thoroughly and uniformly, compact the HMA mat by rolling. In areas not accessible to large rollers, compact with mechanical tampers or trench rollers.

The lower specification limit for mat 0ensity is **93.0** percent of the Maximum Specific Gravity (MSG) as determined by ATM 409 with a bonus for higher densities (Subsection 412-4.03). The MSG from the approved JMD is used for the first lot of each type of HMA. The MSG for additional lots is determined from the first sublot of each lot.

412-3.17 JOINTS. Place and compact the HMA to provide a continuous bond, texture, and smoothness between adjacent sections of the HMA.

Minimize the number of joints. Do not construct longitudinal joints in the driving lanes unless approved by the Engineer in writing at the pre-paving meeting. Offset the longitudinal joints in one layer from the joint in the layer immediately below by at least 6 inches. Align the joints of the top layer at the centerline or lane lines. Where preformed marking tape striping is required, offset the longitudinal joint in the top layer not more than 6 inches from the edge of the stripe.

Form transverse joints by saw-cutting back on the previous run to expose the full depth of the course or by using a removable bulkhead. Skew transverse joints 15 to 25 degrees.

For all joints below the top lift, uniformly coat joint surfaces with tack coat material meeting Section 402.

When Item 412.0014.____ appears in the bid schedule, uniformly coat the joint face of all top lift joints with a joint adhesive. Otherwise use tack coat material meeting Section 402. Follow joint adhesive manufacturer's recommendations for temperatures and application method. Remove joint adhesive applied on the finished pavement surface.

Infrared joint heaters are recommended to be used. If joint densities equal to or greater than 94% MSG are achieved in the previous sublot, then joint adhesive is not required if passing densities are achieved.

The lower specification limit for top lift longitudinal joint density is **93.0** percent MSG of the JMD of the panel completing the joint, with a bonus for higher densities (Subsection 412-4.03).

When the longitudinal joint density test result for a top lift panel sublot is less than **93.0** percent, seal the surface of all longitudinal joints within all of that sublot using a joint sealant meeting Subsection 702-2.06. Apply joint sealant according to the manufacturer's recommendations while the HMA is clean, free of moisture and prior to final traffic marking. Place the sealant at a maximum application rate of 0.15 gallons per square yard, and at least 12 inches wide centered on the longitudinal joint. After surface sealing, inlay by grinding pavement striping into the sealed HMA. Use grooving equipment that grinds a dry cut to groove the width, length, and thickness of the striping within the specified striping tolerances.

Correct improperly formed joints that result in surface irregularities according to a corrective action plan.

Joints formed by paving in echelon while the mat temperature is over 200°F, as measured by the Engineer, within three inches of the joint are considered hot lapped and do not require tack coat or joint adhesive.

When Item 412.0009.____ appears in the Bid Schedule:

1. Top lift longitudinal joint density will be evaluated for price adjustment according to Subsection 412-4.12.2.

2. Test Hot lapped top lift joints for joint density.

412-3.18 SURFACE REQUIREMENTS AND TOLERANCES. The finished surface of all HMA paving must match dimensions shown in the contract for horizontal alignment and width, profile grade and elevation, crown slope, and pavement thickness. Water must drain across the pavement surface without ponding. The surface must have a uniform texture, without ridges, puddles, humps, depressions, and roller marks. The surface must not exhibit raveling, cracking, tearing, asphalt bleeding, or aggregate segregation. Leave no foreign material, uncoated aggregate or oversize aggregate on the HMA surface.

The Engineer will test the finished surface after final rolling at selected locations using a 10-foot straightedge. Measurements will include spanning joints. The Engineer will identify pavement areas that deviate more than 3/16 inch from the straightedge, including joints, as defective work. Perform corrective work by removing and replacing, grinding, cold milling or infrared heating such areas as required. Do not surface patch. After the Contractor performs corrective work, the Engineer will retest the area.

When Item 412.0010.____ appears in the Bid Schedule:

1. Pavement smoothness will be evaluated for price adjustment according to Subsection 412-4.03.3.
2. The Engineer will use an inertial profiler to measure the top lift HMA surface in the driving lanes for surface smoothness within 21 days after paving is complete and driving lanes are delineated.

Profiler measurements will be taken on through lanes identified in the contract. Profiler measurements will not be taken in turn lanes, intersections, ramps, lane transitions, or within 25 feet of bridge abutments and transverse joints with pre-existing pavement.

The Engineer will measure the pavement smoothness in both wheel paths of each lane. The smoothness is measured as International Roughness Index (IRI), reported as inches/mile, at 0.1 mile increments. Pavement smoothness is the average of all IRI measurements for the project.

The Engineer will identify areas requiring corrective action in accordance with Table 412-4. Perform full-width corrective action in those areas. The Engineer may waive corrective work for localized roughness for deficiencies resulting from manholes or other similar appurtenances near the wheel path.

Perform Corrective Actions according to one of the following or by a method approved by the Engineer:

1. Diamond Grinding. If the required pavement thickness is not decreased by more than 0.25", grind to the required surface tolerance and cross section. Remove and dispose of all materials resulting from the grinding process. Apply joint sealant and sand to exposed aggregates per the manufacturer's recommendations.
2. Overlaying. Mill or sawcut the existing pavement to provide a vertical transverse joint face to match the overlay to the existing pavement. Apply tack coat on the milled surface and joint adhesive to all vertical joints and overlay the full width of the underlying pavement surface. Use the same approved HMA for overlays. Place a minimum overlay thickness of 2.0 inches.
3. Mill and Fill. Mill the existing pavement to provide a vertical transverse joint face. Apply tack coat to the milled surface and joint adhesive to all vertical joints prior to inlay new HMA to match the existing pavement. Use the same approved HMA. Place a minimum thickness of 2.0 inches.

After completion of corrective work, the Engineer will measure the pavement surface with an inertial profiler for a smoothness price adjustment.

412-3.19 REPAIRING DEFECTIVE AREAS. Remove HMA that is contaminated with foreign material, is segregated (determined visually or by testing), flushing, or bleeding asphalt. Remove and dispose defective HMA for the full thickness of the course. Cut the pavement so that edges are vertical and the sides are parallel to the direction of traffic. Coat edges and surfaces with a tack coat according to Section 402. Place and compact fresh HMA so that compaction, grade and smoothness requirements are met.

412-3.20 ROADWAY MAINTENANCE. Inspect daily according to pavement inspection plan. Remove, and dispose of project materials incorrectly deposited on existing and new pavement surfaces(s) inside and outside the project area including haul routes.

The Contractor is responsible for damage caused by not removing these materials and any damage to the roadway from the removal method(s).

Repair damage to the existing roadway that results from fugitive materials or their removal.

412-4.01 METHOD OF MEASUREMENT. See Section 109 and the following:

1. Hot Mix Asphalt.

   a. By weight. No deduction is made for the weight of asphalt binder or anti stripping additive or cutting back joints. If the use of WMA is approved by the Engineer, WMA additives will not be measured and are considered subsidiary to the HMA pay item.

   b. By the final HMA surface area.

2 Asphalt Binder. By weight, as follows:

   Method 1 will be used for determining asphalt binder quantity unless otherwise directed in writing. The procedure initially used will be the one used for the duration of the project. No payment is made for any asphalt binder more than 0.4 percent above the optimum asphalt binder content specified in the JMD.

   a. Method 1: Percent of asphalt binder for each sublot multiplied by the total HMA weight represented by that sublot. The Engineer will use either ATM 405 or ATM 406 to determine the percent of asphalt binder. The same test method used for the acceptance testing of the sublot will be used for computation of the asphalt binder quantity. In the absence of testing, the percent of asphalt binder is the target value for asphalt binder in the JMD.

   b. Method 2: Supplier's invoices minus waste, diversion and remnant. This procedure is an Engineer’s option for projects where deliveries are made in tankers and the asphalt plant is producing HMA for one project only.

   The Engineer may direct, at any time that tankers are weighed in the Engineer’s presence before and after unloading. If the weight determined at the project varies more than 1 percent from the invoice amount, payment is based on the weight determined at the project.

   Any remnant or diversion will be calculated based on tank stickings or weighing the remaining asphalt binder. The Engineer will determine the method. The weight of asphalt binder in disposed HMA is calculated using the target value for asphalt binder as specified in the JMD.

3. Job Mix Design. When specified, a Contractor furnished JMD is measured as one according to the HMA class and type.

4. Temporary Pavement. By weight, without deduction for the weight of asphalt binder or anti-strip additive.

5. Leveling Course. By Lane-Station (12 foot width) or by weighing without deduction for the weight of asphalt binder or anti-strip additive.

6. HMA Price Adjustment. Calculated by quality level analysis under Subsection 412-4.03.1.

7. Longitudinal Joint Density Price Adjustment. By the linear foot of top lift longitudinal joint under Subsection 412-4.03.2.

8. Joint Adhesive. By the linear foot of longitudinal and transverse joint.

9. Pavement Smoothness Price Adjustment. Calculated from inertial profiler data using FHWA’s ProVAL software under Subsection 412-4.03.3.

10. Asphalt Material Price Adjustment. Determined under Subsection 412-4.04.

11. Liquid Anti-Strip Additive. Based on the number of tons of asphalt binder containing required additive.

12. Crack Repair. From end to end of the crack repaired according to 412-3.10, measured horizontally along the centerline of the crack. Cleaning loose material from cracks, asphalt binder, and HMA to fill cracks will not be measured separately but are subsidiary.

13. Prelevel for Ruts, Delaminations, and Depressions. By the surface area where prelevel is placed according to 412-3.10(1), measured according to Section 109. Asphalt binder, HMA, and cleaning loose material, will not be measured separately but are subsidiary.

14. Repair Unstable Pavement. By the surface area of pavement repaired according to 412-3.10(2), measured according to Section 109. Asphalt binder, HMA, and removal of pavement and base course will not be measured separately but are subsidiary.

412-4.02 ACCEPTANCE SAMPLING AND TESTING. The bid quantity of each type of HMA produced and placed will be divided into lots and the lots evaluated individually for acceptance.

A lot is normally 5,000 tons. The lot is divided into sublots of 500 tons, each randomly sampled and tested for asphalt binder content, density, and gradation according to this Subsection. The lot is evaluated for price adjustment according to Subsection 412-4.03.1. Seasonal startup or a new JMD requires starting a new lot.

If less than 8 sublots have been placed at the time a lot is terminated, the material in the shortened lot will be included as part of the prior lot. The price adjustment computed for the prior lot will include the samples from the shortened lot. Density test results from material in the shortened lot will be based on the MSG of the shortened lot. If there is no prior lot, and there are at least 3 sublots, the material in the shortened lot will be considered as a lot and the price adjustment will be based on the actual number of test results in the shortened lot. If there are less than 3 sublots, the HMA will be accepted for payment based on the Engineer's approval of the JMD, and placement and compaction of the HMA to the specified depth, finished surface requirements and tolerances. The Engineer reserves the right to perform any testing required in order to determine acceptance.

If 8 or 9 sublots have been placed at the time a lot is terminated, they will be considered as a lot and the price adjustment will be based on the actual number of test results in the shortened lot.

If the bid quantity is between 1,500 to 4,999 tons, the quantity is considered one lot. The lot is divided into sublots of 500 tons, each randomly sampled and tested for asphalt binder content, density, and gradation according to this Subsection. The lot is evaluated for price adjustment according to Subsection 412-4.03.1.

For bid quantity less than 1,500 tons, HMA will be accepted for payment based on the Engineer's approval of the JMD, and placement and compaction of the HMA to the specified depth, finished surface requirements and tolerances. The Engineer reserves the right to perform any testing required in order to determine acceptance.

Sampling and testing include the following:

1. Asphalt Binder Content. HMA samples shall be taken randomly by the Contractor in the presence of the Engineer from behind the paver screed before initial compaction, or will be taken randomly by the Engineer from the windrow, according to ATM 402 or ATM 403, at the discretion of the Engineer. The location (behind the paver screed or windrow) will be determined at the pre-paving meeting. Random sampling locations will be determined by the Engineer.

   Two separate samples will be taken, one for acceptance testing and one held in reserve for retesting if requested. Asphalt binder content will be determined according to ATM 406.

2. Aggregate Gradation. Aggregates tested for gradation acceptance will have the full tolerances from Table 412-2 applied. For HMA samples, the gradation will be determined according to ATM 408 from the aggregate remaining after the ignition oven (ATM 406) has burned off the asphalt binder.

3. Density. The Engineer will determine and mark the location(s) where the Contractor takes each core sample.

   a. Mat Cores: The location(s) for taking core samples is determined using a set of random numbers (independent of asphalt binder and aggregate sampling set of random numbers) and the Engineer's judgment. Take no mat cores within 1 foot of a joint or edge. Core samples are not taken on bridge decks.

   b. Longitudinal Joint Cores: The Engineer will mark the location(s) to take the core sample, centered on the visible surface joint, and adjacent to the mat core sample taken in the panel completing the joint.

   Take core samples according to ATM 413 in the presence of the Engineer. Cut full depth core samples, centered on the marks and as noted above, from the finished HMA within 24 hours after final rolling. Neatly core drill one six inch diameter sample at each marked location. Use a core extractor to remove the core - do not damage the core. The Engineer will immediately take possession of the samples. Backfill and compact voids left by coring according to ATM 413. The Engineer will determine density of samples according to ATM 410.

4. Retest. When test results have failed to meet specifications, retest of acceptance test results for asphalt binder content, gradation, and density may be requested provided the quality control requirements of Subsection 412-3.02 are met. Deliver this request in writing to the Engineer within 7 days of receipt of the final test of the lot. The Engineer will mark the sample location for the density retest within a 2 foot radius of the original core. The original test results are discarded and the retest result is used in the price adjustment calculation regardless of whether the retest result gives a higher or lower pay factor. Only one retest per sample is allowed. When gradation and asphalt binder content are determined from the same sample, a request for a retest of either gradation or asphalt binder content results in a retest of both. Both gradation and asphalt binder content retest results are used in the price adjustment calculation. Retesting will be performed by a department laboratory.

5. Asphalt Binder Grade. The lot size for asphalt binder is 200 tons. If a project has more than one lot and the remaining asphalt binder quantity is less than 150 tons, it is added to the previous lot and that total quantity will be evaluated as one lot. If the remaining asphalt binder quantity is 150 tons or greater, it is sampled, tested and evaluated as a separate lot.

   If the bid quantity of asphalt binder is between 85 – 200 tons, the contract quantity is considered as one lot and sampled, tested, and evaluated according to this subsection. Quantities of asphalt binder less than 85 tons will be accepted based on manufacturer’s certified test reports and certification of compliance.

   Sample asphalt binder at the plant from the supply line in the presence of the Engineer according to ATM 401. The Engineer will take immediate possession of the samples. Take three samples from each lot, one for acceptance testing, one for Contractor requested retesting, and one held in reserve for referee testing if requested. Meet 412- 2.01 requirements for asphalt binder quality.

6. Asphalt Binder Grade Retest. Retest of acceptance test results may be requested provided the quality control requirements of Subsection 412-3.02 are met. Deliver the request in writing to the Engineer within 7 days of receipt of notice of failing test. The original test results are discarded and the retest result is used for acceptance. Only one retest per sample is allowed.

   If the contractor challenges the result of the retest, the referee sample held by the Engineer will be sent to a mutually agreed upon independent AASHTO accredited laboratory for testing. The original acceptance test result, the retest acceptance test result, and the referee sample test result will be evaluated according to ASTM D3244 to obtain an Assigned Test Value (ATV). The ATV value will be used to determine if the asphalt binder conforms to the contract. The Contractor shall pay for the referee sample test if the ATV confirms the asphalt binder does not meet contract requirements.

412-4.03 EVALUATION OF MATERIALS FOR ACCEPTANCE.

The Engineer may reject material which appears to be defective based on visual inspection. If a test of rejected material is requested, a minimum of two samples are collected from the rejected material and tested. If all test results are within specification limits, payment for the material is made.

The following methods are applied to each type of HMA when Price Adjustment Pay Items are included in the Bid Schedule. These methods describe how price adjustments are determined based on the quality of the HMA, longitudinal joint density and pavement smoothness.

1. HMA Price Adjustment. Acceptance test results for HMA asphalt binder content, gradation and mat density are used in HMA price adjustment. These test results for a lot are analyzed collectively and statistically by the Quality Level Analysis (QLA) method as specified in Subsection 106-1.03.3 to determine the total estimated percentage of the lot that is within specification limits. The values for percent passing the #200 sieve, asphalt binder content and density test results are reported to the nearest 0.1 percent. All other sieves used in QLA are reported to the nearest whole number.

   The HMA price adjustment is based on the lower of two pay factors. The first factor is a composite pay factor (CPF) for HMA that includes gradation and asphalt binder content. The second is the density pay factor (DPF).

   A lot containing material with less than a 1.000 pay factor is accepted at an adjusted price, provided that pay factor is at least 0.800 and there are no isolated defects identified by the Engineer. A lot containing material that fails to obtain the minimum pay factor is considered unacceptable and rejected under Subsection 105-1.11.

   HMA pay factors are computed as follows:

   a. All statistical Quality Level Analysis (QLA) is computed using the Engineer's Price Adjustment programs.

   b. The USL and LSL are equal to the Target Value (TV) plus and minus the allowable tolerances in Table 412-6, or as shown below. The TV is the specification value shown in the approved Job Mix Design.

TABLE 412-6
HMA LOWER SPECIFICATION LIMIT (LSL) &
UPPER SPECIFICATION LIMIT (USL)

Measured Characteristics	LSL	USL
¾" or largest sieve size	99	100
½ inch sieve or first sieve retaining aggregate	TV-6	TV+6
3/8 inch sieve	TV-6	TV+6
No. 4 sieve	TV-6	TV+6
No. 8 sieve	TV-6	TV+6
No. 16 sieve	TV-5	TV+5
No. 30 sieve	TV-4	TV+4
No. 50 sieve	TV-4	TV+4
No. 100 sieve	TV-3	TV+3
No. 200 sieve	TV-2.0	TV+2.0
Asphalt Binder Content, %	TV-0.4	TV+0.4
Mat Density, %	**93.0**	100.0

c. The percent within limits (PWL), Quality Levels and characteristic pay factors (PFs) are determined by the Engineer for each Lot in accordance with Subsection 106-1.03.3. The Composite Pay Factor (CPF) for the lot is determined from gradation and asphalt binder content (ac) acceptance test results using the following example formula:

$$CPF = \frac{[f_{3/4\ inch}\ (PF_{3/4\ inch}) + f_{1/2\ inch}\ (PF_{1/2\ inch}) + \ldots f_{ac}\ (PF_{ac})]}{\Sigma f}$$

Table 412-7 gives the weight factor (f) for each test property considered.

TABLE 412-7
WEIGHT FACTORS

Property	Factor "f"
1 inch sieve	-
¾ inch sieve	-
½ inch sieve	4
3/8 inch sieve	5
No. 4 sieve	5
No. 8 sieve	5
No. 16 sieve	5
No. 30 sieve	6
No. 50 sieve	6
No. 100 sieve	4
No. 200 sieve	20
Asphalt Content, %	40

The Mat Density Pay Factor (DPF) is the computed lot average of HMA mat core compaction acceptance test results within the lot;

DPF= 0.01667 * Average Lot Density – 0.55 (1.05 is maximum DPF, positive DPF is incentive, negative DPF is disincentive)

The CPF and DPF are rounded to the nearest 0.001. The price adjustment for each individual lot is calculated as follows:

HMA Price Adjustment = [(CPF or DPF)* -1.00] x (tons in lot) x (PAB)
* CPF or DPF, whichever is lower. No bonus or incentive will be paid if there is an Asphalt Binder penalty or if the lot average joint density is less than 93.0%.
PAB = Price Adjustment Base = [Bid Unit Price for 412.0001.___ + (n/100) x Bid Unit Price for 412.0004.____],

Where
n = Optimum asphalt binder content percent, established by the JMD.
The HMA Price Adjustment is the sum of the price adjustments for each lot and paid for under Item 412.0008.____.

2. Longitudinal Joint Density Price Adjustment Factor (JDPF) is based on the lot average of all top lift cold joint densities between driving lanes within a lot and is determined as follows:

a. Disincentive. Lot average top lift joint density less than 93.0% MSG:

Deduct $3.00 per lineal foot and seal joint surface

b. Incentive. Lot average top lift joint density greater than 93.0% MSG:

JDPF= (Project joint core density average% – 93.00%) x $/ft per percent ($3.5/ft is maximum JDPF, positive is incentive, negative is disincentive)

The Longitudinal Joint Density Price Adjustment is the total price adjustment paid for under Item 412.0009.____.

2. Pavement Smoothness Price Adjustment. Pavement smoothness will be measured by the Engineer and reported as IRI (inches/mile), according to Subsection 412-3.18. Incentive for pavement smoothness shall apply only if both the project average CPF and DPF are greater than or equal to 1.000. Disincentive for pavement smoothness shall apply regardless of the project average CPF or DPF.

The Engineer will calculate the pavement smoothness price adjustment according to Method 1 or Method 2 below, as identified in the bid schedule.

Method 1: SPA = PAB x PQ x SF,

Where:
SPA = Pavement Smoothness Price Adjustment
PAB = Price Adjustment Base
PAB = [Bid Unit Price for 412.0001.____ + (n/100) x Bid Unit Price for 412.0004.____],
n = optimum asphalt binder content, percent, established by the JMD
PQ = Top layer HMA quantity, tons
SF = Smoothness Factor (Table 412-8)

TABLE 412-8
SMOOTHNESS FACTOR (SF)

IRI (in./mile)	SF
Less than 40	0.05
40 to 70	0.05 – (IRI - 40)/600
70 to 90	0.00
90 to 120	(90 – IRI)/120
Greater than 120*	-

* Corrective Work required, see Subsection 412-3.18

Method 2: SPA = PAB x PQ x SF,

Where:
SPA = Pavement Smoothness Price Adjustment
PAB = Price Adjustment Base
PAB = [Bid Unit Price for 412.0001.____ + (n/100) x Bid Unit Price for 412.0004.____],
n = optimum asphalt binder content, percent, established by the JMD
PQ = Top layer HMA quantity, tons
SF = Smoothness Factor = 0.12 x RR – 0.02; SF not to exceed 0.05
RR = Roughness Reduction = (Initial IRI – Final IRI) / Initial IRI
Initial IRI = Pre-project average IRI as measured and reported by the Engineer. The Initial IRI will either be included in the bid documents or the timeline for when the Initial IRI will be measured will be identified in the bid documents.
Final IRI = Top layer HMA average IRI as measured and reported by the Engineer according to Subsection 412-3.18.

The Pavement Smoothness Price Adjustment is the total price adjustment paid for under Item 412.0010.____.

4. Asphalt Binder Price Adjustment. A lot quantity of asphalt binder, with a pay factor less than 1.00, is accepted or rejected per Table 412-9 Asphalt Binder Pay Factors.

TABLE 412-9

ASPHALT BINDER PAY FACTORS

Pay Factor			1.01	1.00	0.95	0.90	0.75	Reject
RTFO (Rolling Thin Film Oven)								
DSR[(a.1)]	All Grades	$G^*/\text{Sin}\delta$, kPa^{-1}	≥ 2.69	2.68–2.20	2.19–1.96	1.95–1.43	1.42–1.10	< 1.10
	PG 58-34 E	$J_{NR\ 3.2}$	≤ 0.19	0.20–0.25	0.26–0.29	0.30–0.39	0.40–0.50	> 0.50
		% $Rec_{3.2}$	≥ 90.0	89.9–85.0	84.9–80.0	79.9–75.0	74.9–70.0	< 70.0
	PG 64-40 E	$J_{NR\ 3.2}$	< 0.05	0.05–0.10	0.11–0.15	0.16–0.20	0.21–0.25	> 0.25
		% $Rec_{3.2}$	≥ 97.0	96.9–95.0	94.9–91.0	90.9–85.0	84.9 – 80.0	< 80.0
PAV (Pressurized Aging Vessel)								
DS[(a.3)]	PG 64-40 E	$G^*\text{Sin}\delta$, kPa	≤ 4711	4712–5000	5001–5289	5290–5578	5579–5867	> 5867
	PG 58-34 E	$G^*\text{Sin}\delta$, kPa	≤ 5700	5701–6000	6001–6300	6301–6600	6601 – 7000	> 7000
CS[(a.4 & 5)]	All Grades[(a.4)]	BBR, *S*, MPa	≤ 247	248–300	301–338	339–388	389–449	≥ 450
	All Grades[(a.5)]	BBR, *m*	≥ 0.320	0.319–0.300	0.299–0.294	0.293–0.278	0.277–0.261	< 0.261

Multiple Stress Creep Recovery (MSCR), Dynamic Shear (DS), Creep Stiffness (CS)

a. Asphalt Binder Pay Adjustment = (Lowest Pay Factor – 1.00) x (tons in lot) x PAB x 5
Select the lowest pay factor from:
RTFO (test the binder residue at the performance grade temperature)

(1) DS, All Grades, $G^*/\text{Sin}\delta$, kPa^{-1}
(2) MSCR: PG, Select the highest pay factor corresponding to, either $J_{NR\ 3.2}$ or % $Rec_{3.2}$ values

PAV

(3) DS, PG, $G^*\text{Sin}\delta$, kPa
(4) CS, All Grades, BBR, *S* MPa
(5) CS, All Grades, BBR, *m*

b. If three consecutive acceptance samples are out of specification, stop HMA production immediately and submit a corrective action plan to the Engineer for approval.

The Asphalt Binder Price Adjustment is the sum of the price adjustments for each lot and paid for under Item 412.2022 ___________

412-4.04 ASPHALT MATERIAL PRICE ADJUSTMENT.

Asphalt Material Price Adjustment. This subsection provides a price adjustment for asphalt material by: (1) additional compensation to the contractor or (2) a deduction from the contract amount.

1. This provision shall apply:

a. To asphalt material meeting the criteria of Section 702, and is included in items listed in the bid schedule of Sections 306, 307, 308, 401 thru 412, 520, 608 and 609.

b. To cost changes in asphalt material that occur between the date of bid opening and the date on the certified bill of lading from the asphalt material refiner/producer.

c. When there is more than a 7.5 percent increase or decrease in the Alaska Asphalt Material Price Index, AAMPI, from the date of bid opening to the date on the certified bill of lading from the asphalt refiner/producer.

2. Provide the certified bill of lading from the asphalt material refiner/producer.

3. The AAMPI is calculated bimonthly on the first and third Friday of each month, and will remain in effect from the day of calculation until the next bimonthly calculation. The AAMPI is posted on the Department's Statewide Materials website at and calculated according to the formula posted there. http://www.dot.state.ak.us/stwddes/desmaterials/aprice_index.shtml

4. Price adjustment will be cumulative and calculated with each progress payment. Use the AAMPI in effect on the date of the certified bill of lading from the asphalt material refiner/producer, to calculate the price adjustment for asphalt material   The Department will increase or decrease payment under this contract by the amount determined with the following asphalt material price adjustment formula:

   For an increase exceeding 7.5 percent, additional compensation = [(IPP – IB) – (0.075 x IB)] x Q

   For a decrease exceeding 7.5 percent, deduction from contract = [(IB – IPP) – (0.075 x IB)] x Q

   Where:
   Q = Quantity of Asphalt Material incorporated into project during the pay period, in tons as measured by the Engineer
   IB = Index at Bid: the Bi-monthly AAMPI in effect on date of bid, in dollars per ton
   IPP = Index at Pay Period: The bi-monthly AAMPI in effect on the date shown on the certified bill of lading from the asphalt refiner/producer, in dollars per ton

5. Method of measurement for determining Q (quantity) is the weight of asphalt material that meets the criteria of this subsection and is incorporated into the project. The quantity does not include aggregate, mineral filler, blotter material, thinning agents added after material qualification, or water for emulsified asphalt. The quantity for emulsified asphalts will be based on the asphalt residue material only and will be calculated using the percent residue from testing, or if not tested, from the manufacturer's certificate of compliance.

412-5.01 BASIS OF PAYMENT.

The following items, unless included as individual Pay Items, are subsidiary to the Section 408 Hot Mix Asphalt Pavement related Pay Items as included in the bid schedule:

- Asphalt binder
- Liquid anti-strip additives
- Tack coat
- Crack sealing
- Crack repair
- Joint adhesive
- Surface sealing of longitudinal joints
- Surface tolerance corrections
- Patching defective areas
- Prelevel for ruts, delaminations, and depressions
- Repair unstable pavement
- Job mix design

- Density profiles, Subsection 408-2.10 Process Quality Control
- Repair work and materials when milling equipment breaks through existing pavement – Subsection 412-3.10 Preparation of Existing Surface
- Work and materials associated with Subsection 412-3.06 Hauling Equipment
- Work and materials associated with Subsection 412-3.20 Roadway Maintenance

Test Strips: Subsection 412-2.10 Process Quality Control.

a. Approved. Test strip construction and material, approved by the Engineer in writing, as meeting the specification requirements will be paid for at the Contract unit prices for HMA and asphalt binder as included in the Bid Schedule. Price adjustments 412.2008.____, 412.2009.____, 412.2010.____ and 412.2021.____ do not apply.

b. Failed. The materials, construction of, removal and disposal of a failed test strip will be at the Contractor's expense.

Asphalt binder, liquid anti-strip additives and tack coat are subsidiary to HMA Item 412.0001.____ and 412.0006.____ unless specified as Pay Items. Asphalt binder and liquid anti-strip are subsidiary to HMA for Items 412.0002.____, 412.0003.____, 412.0005.____, 412.0011.____, and 412.0012.____.

Item 412.0008.____ HMA price adjustment is the sum of the price adjustments for each material lot and for deductions and fees assessed. Deductions and fees assessed include:

1. The Department will bear the cost of the initial JMD evaluation for each type of HMA. All subsequent evaluations required to obtain an approved JMD will be assessed a fee of $6,000 per evaluation. Failed retest will result in a fee of $2,500.

2. Failure to cut core samples within the specified period will result in a deduction of $100 per sample per day.

3. Failure to backfill voids left by sampling within the specified period will result in a deduction of $100 per hole per day.

4. If an asphalt binder referee test is requested and the ATV confirms the asphalt binder does not meet contract requirements, a fee of $500 will be assessed.

Item 412.0008.____ HMA price adjustment does not apply to:

1. HMA, when contract quantity is less than 1500 tons

2. HMA for leveling course 412.0002.____ and 412.0003.____

3. Temporary HMA 412.0005.____

4. Driveway HMA 412.0011.____ and 412.0012.____

Payment will be made under:

PAY ITEM		
Item Number	**Item Description**	**Unit**
412.0001.____	HMA, Type ___; Class ___	TON
412.0002.____	HMA, Leveling Course, Type ___; Class ___	LnSt
412.0003.____	HMA, Leveling Course, Type ___; Class ___	TON
412.0004.____	Asphalt Binder, Grade ____	TON
412.0005.____	HMA, Temporary, Type ___; Class ___	TON
412.0006.____	HMA, Type ___; Class ___	SY

412.0007.____	Liquid Anti-Strip Additive	CS
412.0008.____	HMA Price Adjustment, Type ___; Class ___	CS
412.0009.____	Longitudinal Joint Density Price Adjustment	CS
412.0010.____	Pavement Smoothness Price Adjustment, Method ___	CS
412.0011.____	HMA, Driveway, Type ___; Class ___	LS
412.0012.____	HMA, Driveway, Type ___; Class ___	TON
412.0013.____	Job Mix Design	EACH
412.0014.____	Joint Adhesive	LF
412.0015.____	Asphalt Material Price Adjustment	CS
412.0016.____	Crack Repair	LF
412.0017.____	Prelevel for Ruts, Delaminations, and Depressions	SY
412.0018.____	Repair Unstable Pavement	SY
412.2022.____	Combined Price Adjustment	CS

APPENDIX G – SECTION 412 HOT MIX ASPHALT PAVEMENT (DPS)

SECTION 412
HOT MIX ASPHALT PAVEMENT (Low VTM, DPS)

412-1.01 DESCRIPTION. Construct one or more courses of plant-produced Hot Mix Asphalt (HMA) pavement on an approved surface, to the lines, grades, and depths shown on the Plans. This HMA has a high polymer modified asphalt content and requires high field compaction. Map full coverage joint and mat compaction data with a Dielectric Profiling System (DPS).

MATERIALS

412-2.01 ASPHALT BINDER. Conform to Subsection 702-2.01. Meet AASHTO M332 Performance-Graded Asphalt Binder Using MSCR Test Specification; except, changes noted in Table 412-1 as Exceptions.

TABLE 412-1. EXCEPTIONS TO AASHTO M 332 SPECIFICATION

Performance Grade	AASHTO Spec.	Viscosity AASHTO T 316	MSCR, AASHTO T 350			PAV, Dynamic Shear AASHTO T 315	Direct Tension AASHTO T 314
			$J_{NR3.2}$ kPa^{-1}	J_{NR} Diff	% $Rec_{3.2}$		
PG 58-34E	M332	None	0.25 max.	Delete	85 min.	None	Delete
PG 64-40E	M332	1.0 PaS max.	0.10 max.	Delete	95 min.	5000 max @ 4°C	Delete

Asphalt binder may be conditionally accepted at the source providing a manufacturers certification of compliance according to Subsection 106-1.05, and following documents at delivery;

Furnish the following documents at delivery:

1. Manufacturer's certificate of compliance (Subsection 106-1.05).
2. Conformance test reports for the batch (provide prior to delivery as noted above).
3. Batch number and storage tanks used.
4. Date and time of load out for delivery.
5. Type, grade, temperature, and quantity of asphalt binder loaded.
6. Type and percent of liquid anti-strip added.

412-2.02 LIQUID ANTI-STRIP ADDITIVE. Use anti-strip agents in the proportions determined by ATM 414 and included in the approved Job Mix Design (JMD). At least 90 percent of the aggregate must remain coated when tested according to ATM 414. The following minimum dose (percent) of liquid anti-strip by weight of asphalt binder is required:

Liquid Anti-strip Type	Minimum Dose by Weight of Asphalt Binder, %
Amines based	0.30
Phosphate Ester based	0.30
Organo-Silane based	0.05

412-2.03 JOINT ADHESIVE. Conform to Subsection 702-2.05.

412-2.04 JOINT SEALANT. Conform to Subsection 702-2.06.

412-2.05 WARM MIX ASPHALT. Warm mix **a**dditive is required. Conform to Subsection 702-2.07.

412-2.06 ASPHALT RELEASE AGENT. Conform to Subsection 702-2.08.

412-2.07 AGGREGATES. Aggregates shall consist of crushed stone, crushed gravel, crushed screenings, as required. Coarse aggregate is the material retained on the No. 4 sieve. Fine aggregate is the material passing the No. 4 sieve.

Use a minimum of three stockpiles of crushed aggregate of different gradations.

a. Coarse Aggregate. Coarse aggregate shall consist of sound, tough, durable particles, free from films of matter that would prevent thorough coating and bonding with the bituminous material and be free from organic matter and other deleterious substances. Coarse aggregate material shall conform to Table 412-2 Coarse Aggregate Material Requirements.

TABLE 412-2. COARSE AGGREGATE MATERIAL REQUIREMENTS

Material Test	Requirement	Standard
Resistance to Degradation	40% max. loss	AASHTO T 96
Micro-Deval	20% max. loss	AASHTO T 327
Soundness of Aggregates by Use of Sodium Sulfate	9% max. loss after 5 cycles:	AASHTO T 104
Clay lumps and friable particles	1.0% max.	AASHTO T 112
Percentage of Fractured Particles	98% min. two fractured faces by weight	ATM 305
Flat, Elongated, or Flat and Elongated Particles	8% max. at 5:1 [2]	ATM 306
Absorption	2.0% max.	ATM 308

1. The area of each face shall be equal to at least 75% of the smallest mid-sectional area of the piece. When two fractured faces are contiguous, the angle between the planes of fractures shall be at least 30 degrees to count as two fractured faces.
2. A flat particle is one having a ratio of width to thickness greater than five (5); an elongated particle is one having a ratio of length to width greater than five (5).

b. Fine Aggregate. Fine aggregate shall consist of clean, sound, tough, durable, angular shaped particles produced by crushing stone or gravel and shall be free from coatings of clay, silt, or other objectionable matter, and conform to Table 412-3 Fine Aggregate Material Requirements.

Natural (non-manufactured) sand is not allowed.

TABLE 412-3. FINE AGGREGATE MATERIAL REQUIREMENTS

Material Test	Requirement	Standard
Liquid limit,	25 max.	ATM 204
Plasticity Index	Non plastic	ATM 205
Soundness of Aggregates by Use of Sodium Sulfate	10% max. loss after 5 cycles	AASHTO T 104
Clay Lumps and Friable Particles	1.0% max.	AASHTO T 112
Sand Equivalent	35 min.	ATM 307
Uncompacted Void Content, (Fine Aggregate Angularity)	45% min.	AASHTO T 304, Method A

c. Sampling. The Engineer will sample according to ATM 301 for coarse and fine aggregate.

412-2.08 RECYCLED ASPHALT PAVEMENT. Recycled asphalt pavement (RAP) is not allowed.

412-2.09 JOB MIX DESIGN. Provide target values for gradation that satisfy both the broad band gradation limits shown in Table 412-4 and the requirements of Table 412-6.

TABLE 412-4 AGGREGATE GRADATION & VMA

Sieve Size	¾ inch (19.5MM) mix % passing by weight	1/2 inch (12.5 mm) mix % passing by weight
¾ inch (19.5mm)	**100**	
1/2 inch (12.5 mm)	**90-100**	100
3/8 inch (9.5 mm)	**72-88**	90-100
No. 4 (4.75 mm)	**53-73**	58-78
No. 8 (2.36 mm)	**38-60**	40-60
No. 16 (1.18 mm)	**26-48**	28-48
No. 30 (600 µm)	**18-38**	18-38
No. 50 (300 µm)	**11-27**	11-27
No. 100 (150 µm)	**6-18**	6-18
No. 200 (75 µm)	**3-6**	3-6
Minimum Voids in Mineral Aggregate (VMA)[1]	15	16

TABLE 412-5 MARSHALL MIX DESIGN (ATM 417) REQUIREMENTS

Stability, Pounds	2150 Min.
Voids in Total Mix (VTM)	2.5% ±0.2%
Compaction, Number of Blows Each Side of Test Specimen	50
Voids in Mineral Aggregate (VMA)	(Table 412-4)
Asphalt Content	5.5% - 8.0%
Dust-Asphalt Ratio[1]	0.6 - 1.2
Liquid Anti-Strip Additive[2]	(Section 412-2.02)
Max. rut by Asphalt Pavement Analyzer (APA)[3]	≤ 10mm @ 4000 passes

1. Dust-Asphalt ratio is the percent of material passing the No. 200 sieve divided by the percent of effective asphalt binder (calculated by weight).
2. By Weight of Asphalt Binder
3. ATM 419 at 250 psi hose pressure at 50ºC test temperature
4. The approved JMD will specify the Target Values (TV) for gradation, the TV for asphalt binder content, the Maximum Specific Gravity (MSG) of the HMA, the additives, and the recommended mixing temperature range.

Submit the following to the Engineer at least 15 days before the production of HMA:

1. A letter stating the location, size, and type of mixing plant. The letter shall state whether or not WMA will be used. The letter shall include the proposed gradation for the JMD, gradations for individual stockpiles, and the blend ratio of each aggregate stockpile.

2. Representative samples of each aggregate (coarse, intermediate, fine, blend material) in the proposed mix design. Furnish a total of 500 pounds of material in the proportional amounts in the proposed JMD.

3. Five separate 1-gallon samples of the asphalt binder proposed for use in the HMA. Include name of product, manufacturer, test results of the applicable quality requirements of Subsection 412-2.07, manufacturer's certificate of compliance according to Subsection 106-1.05, a temperature- viscosity curve for the asphalt binder or manufacturer's recommended mixing and compaction temperatures, and current Material Safety Data Sheet.

4. One sample, minimum one pint, of the anti-strip additive proposed, including name of product, manufacturer, and manufacturer's data sheet, and current Material Safety Data Sheet.

5. Testing results per Subsection 106-1.03.1 for each aggregate type proposed for use.

6. State the WMA technology (Subsection 702-2.07) to be used, location where additive will be introduced and manufacturer's recommended usage rate for each type of HMA. Supply a minimum of 2-pint samples for each proposed additive.

The Engineer will evaluate the material and the proposed gradation using ATM 417 and the specified requirements for the appropriate Type of HMA specified, and establish the approved JMD which will become a part of the Contract.

Obtain an approved JMD prior to shipment of aggregates to an asphalt plant site or producing HMA for payment, unless aggregate quality and gradation has been approved by the Engineer.

<u>Changes</u>. Submit a new JMD with changes noted and new samples in the same manner as the original JMD submittal when:

a. The results of the JMD evaluation do not achieve the requirements specified~~.~~
b. The asphalt binder source is changed
c. The source of aggregate, aggregate quality, or gradation is changed
d. The results of a test strip do not meet the specification-the Engineer may require a new JMD.

Changes to the JMD apply only to HMA produced after the approval of changes.

412-2.10 CONTROL STRIP. Full production shall not begin until an acceptable control strip has been constructed and accepted in writing by the Engineer. The Contractor shall prepare and place a quantity of asphalt according to the JMD. The underlying grade or pavement structure upon which the control strip is to be constructed shall be the same as the remainder of the course represented by the control strip. The Engineer will provide ignition calibration samples and DPS compaction calibration samples.

The Contractor will not be allowed to place the control strip until the Contractor Quality Control Program (CQCP), showing conformance with the requirements of subsection 412-3.02, has been accepted, in writing, by the Engineer.

The control strip will consist of at least 250 tons. The control strip shall be placed in two lanes of the same width and depth to be used in production with a longitudinal cold joint. Construct a longitudinal cold joint using the same procedure that will be used during production. The cold joint for the control strip will be an exposed construction joint at least four (4) hours old or when the mat has cooled to less than 160°F. The equipment used in construction of the control strip shall be the same type, configuration, and weight, to be used on the project.

The control strip shall be evaluated for acceptance as a single lot in accordance with the acceptance criteria in subsection 412-6.1 for aggregate gradation and asphalt binder content. The control strip shall be divided into three separate equal sub-lots. If the Composite Pay Factor is less than 1.000, the control strip is unacceptable. Measure mat compaction with a DPS by collecting a series of longitudinal, parallel, data lines approximately equally spaced and less than 2.5 feet apart. If the DPS average compaction is less than 93.0% MSG of the approved mix design, the control strip is unacceptable. Measure the DPS longitudinal joint compaction centered over the joint, if the DPS joint compaction is less than 93.0% MSG, the control strip is unacceptable.

Measure mat and joint compaction using a Dielectric Profiling System (DPS) such as the PaveScan Rolling Density Meter ground penetrating radar (GPR) system. The DPS shall meet AASHTO PP 98-20 with the GPS having a minimum operational tolerance of ± 2.5 cm. Calibrate the DPS to the JMD before measuring pavement Compaction values. Verify the DPS compaction readings by marking 3 core locations, taking DPS readings, core the pavement, measuring the core compaction in the lab, then compare DPS core compaction to the lab compaction.

After completion of the control strip compaction, the Department will accept or reject the control strip within 48 hours.

If the control strip is unacceptable, necessary adjustments to the JMD, plant operation, placing procedures, and/or rolling procedures shall be made and another control strip shall be placed. Unacceptable control strips shall be removed at the Contractor’s expense. For small projects, less than 3,000 tons, a control strip is not required.

The Engineer may require a separate test strip for paving on a bridge deck where vibratory compaction is not allowed

Refer to Subsection 412-5.01 for payment of test strips.

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CONSTRUCTION REQUIREMENTS

412-3.01 PRE-PAVING MEETING. Meet with the Engineer for a pre-paving meeting in the presence of project superintendent and paving foreman at least five (5) working days before beginning paving operations. Submit a paving plan and pavement inspection plan at the meeting.

Include the following elements in the paving plan and address these elements at the meeting:

1. Sequence of operations
2. List of equipment that will be used for production, transport, pick-up (if applicable), laydown, and compaction
3. Summary of plant modifications (if applicable) for production of WMA
4. Procedures to produce consistent HMA
5. Procedures to minimize material and thermal segregation
6. Procedures to minimize premature cooling
7. Procedures to achieve HMA compaction
8. Procedures for joint construction including corrective action for joints that do not meet surface tolerance requirements
9. Quality control testing methods, frequencies and sample locations for gradation, asphalt binder content, and compaction, and

10. Any other information or procedures necessary to provide completed HMA construction that meets the contract requirements

Include the following elements in the pavement inspection plan and address these elements at the meeting:

11. Process for daily inspections

12. Means and methods to remove and dispose of project materials

412-3.02 CONTRACTOR QUALITY CONTROL PLAN (CQCP). Sample and test materials for quality control of the HMA according to Subsection 106-1.03. Submit to the Engineer at the "Pre-Paving Meeting," Subsection 412-3.01, the JMD and a documentation plan that provides a complete, accurate, and clear record of the sampling and testing results.

Failure to perform quality control forfeits the Contractor's right to a retest under Subsection 412-4.02

Provide copies of the documented sampling and testing results no more than 24 hours from the time taken

412-3.03 WEATHER LIMITATIONS. Place HMA on a stable/non-yielding roadbed. Do not place HMA when the base material is wet or frozen, or when weather conditions prevent proper handling or finishing of the mix. Do not place HMA when the roadway surface temperature is colder than 40° F, or after September 15th without the Engineer's approval in writing.

412-3.04 EQUIPMENT, GENERAL. Use equipment in good working order and free of HMA buildup. Make all equipment available for inspection and demonstration of operation a minimum of 24 hours before placement of HMA.

412-3.05 ASPHALT MIXING PLANT. Meet AASHTO M 156. Use an HMA plant capable of producing at least 150 tons of HMA per hour noted on posted DEC air quality permit, designed to dry aggregates, maintain consistent and accurate temperature control, and accurately proportion asphalt binder and aggregates. Calibrate the HMA plant and furnish copies of the calibration data to the Engineer at least 24 hours before HMA production.

Provide a scalping screen at the asphalt plant to prevent oversize material or debris from being incorporated into the HMA.

Provide a tap on the asphalt binder supply line just before it enters the plant (after the 3-way valve) for sampling asphalt binder. Provide aggregate and asphalt binder sampling locations meeting OSHA safety requirements.

Use of belt conveyor scales to proportion plant blends and mixtures is permitted if the scales meet the general requirements for weighing equipment and are calibrated according to the manufacturer's instructions.

Daily Plant Quantity Summary. At the end of each day of production, provide the Engineer with a document listing the quantities of materials used to produce HMA for each JMF. The document must be electronically generated from the automated plant controls, or as approved in writing by the Engineer.

1. Total tons of HMA produced
2. Total combined dry weight of virgin aggregate (note moisture content)
3. Total dry weight of RAP (note moisture content)
4. Total weight of asphalt cement
5. Total weight of each additive included in the mix at the plant
6. A graph of the mix temperature discharged from the drum or pugmill at 1-minute intervals during the HMA production.
7. If HMA is produced at temperatures higher than 340°F, provide an asphalt binder sample from the plant asphalt storage tank to the Engineer to be tested in an AASHTO accredited lab as follows;

a. Perform RTFO testing of the binder sample at the highest recorded mix temperature,
b. then test the RTFO residue according to the specified binder tests to evaluate conformance to project specifications.
c. If binder does not meet specifications, apply penalties to the binder and mix tonnage produced at temperatures above 340°F as specified,

412-3.06 HAULING EQUIPMENT. Haul HMA in trucks with tight, clean, smooth metal beds. Keep beds free of petroleum oils, solvents, or other materials that would adversely affect the mixture. Apply a thin coat of approved asphalt release agent to beds as necessary to prevent mixture adherence. Provide trucks with covers attached and available for use.

Do not haul HMA on barges.

When directed by the Engineer cover the HMA in the hauling vehicle(s).

412-3.07 ASPHALT PAVERS. Use self-propelled asphalt pavers with heated vibratory screed assemblies to spread and finish HMA to the specified section widths and thicknesses without introducing thermal or material segregation.

Equip the paver with a receiving hopper having sufficient capacity for a uniform spreading operation and a distribution system to place the HMA uniformly in front of screed. Use a screed assembly that produces a finished surface of the required smoothness, thickness and texture without tearing, shoving or displacing the HMA. Heat and vibrate screed extensions.

It is recommended that the paver be equipped with an infrared joint heater and a notch wedge joint maker (6” wide) with compactor for longitudinal joint construction

Equip the paver with automatic screed controls capable of operating from a reference line or a ski from either or both sides of the paver.

412-3.08 ROLLERS. Use rollers designed to compact HMA and capable of reversing without shoving or tearing the mixture. Select rollers that will not crush the aggregate or displace the HMA. Equip vibratory rollers with separate vibration and propulsion controls. Rollers using intelligent compaction technology are recommended.

Equip the rollers with an infrared thermometer that measures and displays the surface temperature to the operator. Infrared thermometer may be hand-held or fixed to the roller.

Compact a leveling course with a pneumatic roller in the complement of rollers. Use fully skirted pneumatic-tire roller having a minimum operating weight of 3000 pounds per tire.

412-3.09 RESERVED.

412-3.10 PREPARATION OF EXISTING SURFACE. Prepare existing surfaces according to the Contract. Prior to placing HMA, clean existing surfaces of loose material and uniformly coat contact surfaces of curbing, gutters, manholes and other structures with tack coat material meeting Section 402. Treat cold joint surfaces according to 412-3.17.

Before applying tack coat to an existing paved surface, clean and patch the surface as directed by the Engineer. Remove irregularities to provide a reasonably smooth and uniform surface. Remove and replace unstable areas with HMA. Clean the edges of existing pavements, which are to be adjacent to new pavement, to permit the adhesion of asphalt materials. Clean loose material from cracks. Fill the cleaned cracks, wider than 1 inch, with HMA tamped in place. Wash and/or sweep the paved surface clean and free of loose materials.

Preparation of a milled surface:

1. Prelevel remaining ruts, pavement delaminations, and depressions having a depth greater than 1/2 inch with an approved HMA.

2. Notify the Engineer of pavement areas that appear thin or unstable. Where milling operation creates thin or unstable pavement areas, or where it breaks through existing pavement, remove thin and unstable pavement, and 2 inches of existing base material, compact and replace with an approved HMA.

412-3.11 PREPARATION OF ASPHALT. Provide a continuous supply of asphalt binder to the asphalt mixing plant at a uniform temperature, within the recommended mixing temperature range.

412-3.12 PREPARATION OF AGGREGATES. Dry the aggregate so the moisture content of the HMA, sampled at the point of acceptance for asphalt binder content, does not exceed 0.5 percent (by total weight of mix), as determined by ATM 407.

Heat the aggregate for the HMA to a temperature compatible with the mix requirements specified.

Adjust the burner on the dryer to avoid damage to the aggregate and to prevent the presence of unburned fuel on the aggregate. HMA containing soot or fuel is unacceptable per Subsection 105-1.11.

412-3.13 MIXING. Combine the aggregate, asphalt binder, and additives in the mixer in the amounts required by the JMD. Mix to obtain at least 98 percent coated particles when tested according to AASHTO T195.

Mix the dried heated aggregates before adding asphalt binder.

Produce the HMA within the temperature range determined by the JMD.

Upon the Engineer’s request, provide daily burner charts showing start/stop times and temperatures.

412-3.14 TEMPORARY STORAGE OF HMA. Silo type storage bins may be used, provided the characteristics of the HMA remain unaltered. Gob hoppers must be used to drop HMA into the silo.

Signs of visible segregation, heat loss, changes from the JMD, change in the characteristics of asphalt binder, lumpiness, and stiffness of the mixture, are causes for rejection.

Do not store HMA on barges.

412-3.15 PLACING AND SPREADING. Use asphalt pavers to distribute HMA, including leveling course and temporary HMA. Place the HMA upon the approved surface, spread, strike off, and adjust surface irregularities. The maximum compacted lift thickness allowed is 3 inches.

When multiple lifts are specified in the Contract, do not place the final lift until all lower lifts throughout that section, are placed and accepted.

Do not place HMA abutting curb and gutter until curb and gutter are installed, except as approved by the Engineer.

Do not pave against new Portland cement concrete curbing until it has cured for at least 72 hours.

When practicable, adjust elevation of metal utility fixtures before paving the final lift, so they will be between 1/4 and 1/2 inch below the top surface of the final lift. Metal fixtures include, but are not limited to manholes, valve boxes, monument cases, hand holes, and drains.

Use hand tools to spread, rake, and lute the HMA in areas where irregularities or unavoidable obstacles make mechanical spreading and finishing equipment impracticable.

Place HMA over bridge deck membranes according to Section 508 and the membrane manufacturer's recommendations.

Do not mix HMA produced from different plants for testing or paving unless approved by the Engineer.

412-3.16 COMPACTION. Thoroughly and uniformly, compact the HMA mat by rolling. In areas not accessible to large rollers, compact with mechanical tampers or trench rollers.

The lower specification limit for mat density is **93.0** percent of the Maximum Specific Gravity (MSG) as determined by ATM 409 with a bonus for higher compaction values (Subsection 412-4.03). The MSG from the approved JMD is used for the first lot of each type of HMA. The MSG for additional lots is determined from the first sublot of each lot.

412-3.17 JOINTS. Place and compact the HMA to provide a continuous bond, texture, and smoothness between adjacent sections of the HMA.

Minimize the number of joints. Do not construct longitudinal joints in the driving lanes unless approved by the Engineer in writing at the pre-paving meeting. Offset the longitudinal joints in one layer from the joint in the layer immediately below by at least 6 inches. Align the joints of the top layer at the centerline or lane lines. Where preformed marking tape striping is required, offset the longitudinal joint in the top layer not more than 6 inches from the edge of the stripe.

Form transverse joints by saw-cutting back on the previous run to expose the full depth of the course or by using a removable bulkhead. Skew transverse joints 15 to 25 degrees.

For all joints below the top lift, uniformly coat joint surfaces with tack coat material meeting Section 402.

When Item 412.0014.____ appears in the bid schedule, uniformly coat the joint face of all top lift joints with a joint adhesive. Otherwise use tack coat material meeting Section 402. Follow joint adhesive manufacturer's recommendations for temperatures and application method. Remove joint adhesive applied on the finished pavement surface.

If passing joint compaction values are achieved in the previous sublot, then joint adhesive is not required.

The lower specification limit for top lift longitudinal joint compaction is **93.0** percent of the MSG of the panel completing the joint, with a bonus for higher compaction values (Subsection 412-4.03). MSG will be determined according to ATM 409.

When the longitudinal joint compaction test result for a top lift panel sublot is less than **93.0** percent, seal the surface of all longitudinal joints within all of that sublot using a joint sealant meeting Subsection 702-2.06. Apply joint sealant according to the manufacturer's recommendations while the HMA is clean, free of moisture and prior to final traffic marking. Place the sealant at a maximum application rate of 0.15 gallons per square yard and a minimum application rate of 0.08 gallons per square yard, and at least 12 inches wide centered on the longitudinal joint. After surface sealing, inlay by grinding pavement striping into the sealed HMA. Use grooving equipment that grinds a dry cut to groove the width, length, and thickness of the striping within the specified striping tolerances.

Correct improperly formed joints that result in surface irregularities according to a corrective action plan.

Joints formed by paving in echelon while the mat temperature is over 200°F, as measured by the Engineer, within three inches of the joint are considered hot lapped and do not require tack coat or joint adhesive.

When Item 412.0009.____ appears in the Bid Schedule:

1. Top lift longitudinal joint compaction will be evaluated for price adjustment according to Subsection 412-4.12.2.

2. Test Hot lapped top lift joints for joint compaction.

412-3.18 SURFACE REQUIREMENTS AND TOLERANCES. The finished surface of all HMA paving must match dimensions shown in the contract for horizontal alignment and width, profile grade and elevation, crown slope, and pavement thickness. Water must drain across the pavement surface without ponding. The surface must have a uniform texture, without ridges, puddles, humps, depressions, and roller marks. The surface must not exhibit raveling, cracking, tearing, asphalt bleeding, or aggregate segregation. Leave no foreign material, uncoated aggregate or oversize aggregate on the HMA surface.

The Engineer will test the finished surface after final rolling at selected locations using a 10-foot straightedge. Measurements will include spanning joints. The Engineer will identify pavement areas that deviate more than 3/16 inch from the straightedge, including joints, as defective work. Perform corrective work by removing and replacing, grinding, cold milling or infrared heating such areas as required. Do not surface patch. After the Contractor performs corrective work, the Engineer will retest the area.

When Item 412.0010.____ appears in the Bid Schedule:

1. Pavement smoothness will be evaluated for price adjustment according to Subsection 412-4.03.3.

2. The Engineer will use an inertial profiler to measure the top lift HMA surface in the driving lanes for surface smoothness within 21 days after paving is complete and driving lanes are delineated.

Profiler measurements will be taken on through lanes identified in the contract. Profiler measurements will not be taken in turn lanes, intersections, ramps, lane transitions, or within 25 feet of bridge abutments and transverse joints with pre-existing pavement.

The Engineer will measure the pavement smoothness in both wheel paths of each lane. The smoothness is measured as International Roughness Index (IRI), reported as inches/mile, at 0.1 mile increments. Pavement smoothness is the average of all IRI measurements for the project.

The Engineer will identify areas requiring corrective action in accordance with Table 412-4. Perform full-width corrective action in those areas. The Engineer may waive corrective work for localized roughness for deficiencies resulting from manholes or other similar appurtenances near the wheel path.

Perform Corrective Actions according to one of the following or by a method approved by the Engineer:

1. <u>Diamond Grinding</u>. If the required pavement thickness is not decreased by more than 0.25”, grind to the required surface tolerance and cross section. Remove and dispose of all materials resulting from the grinding process. Apply joint sealant and sand to exposed aggregates per the manufacturer’s recommendations.

2. <u>Overlaying</u>. Mill or sawcut the existing pavement to provide a vertical transverse joint face to match the overlay to the existing pavement. Apply tack coat on the milled surface and joint adhesive to all vertical joints and overlay the full width of the underlying pavement surface. Use the same approved HMA for overlays. Place a minimum overlay thickness of 2.0 inches.

3. <u>Mill and Fill</u>. Mill the existing pavement to provide a vertical transverse joint face. Apply tack coat to the milled surface and joint adhesive to all vertical joints prior to inlay new HMA to match the existing pavement. Use the same approved HMA. Place a minimum thickness of 2.0 inches.

After completion of corrective work, the Engineer will measure the pavement surface with an inertial profiler for a smoothness price adjustment.

412-3.19 REPAIRING DEFECTIVE AREAS. Remove HMA that is contaminated with foreign material, is segregated (determined visually or by testing), flushing, or bleeding asphalt. Remove and dispose defective HMA for the full thickness of the course. Cut the pavement so that edges are vertical and the sides are parallel to the direction of traffic. Coat edges and surfaces with a tack coat according to Section 402. Place and compact fresh HMA so that compaction, grade and smoothness requirements are met. The Engineer may remap compaction of an area that has been removed and replaced and use the new values in payment calculation equations.

412-3.20 ROADWAY MAINTENANCE. Inspect daily according to pavement inspection plan. Remove, and dispose of project materials incorrectly deposited on existing and new pavement surfaces(s) inside and outside the project area including haul routes.

The Contractor is responsible for damage caused by not removing these materials and any damage to the roadway from the removal method(s).

Repair damage to the existing roadway that results from fugitive materials or their removal.

412-4.01 METHOD OF MEASUREMENT. See Section 109 and the following:

1. Hot Mix Asphalt.

    a. By weight. No deduction is made for the weight of asphalt binder or anti stripping additive or cutting back joints. If the use of WMA is approved by the Engineer, WMA additives will not be measured and are considered subsidiary to the HMA pay item.

    b. By the final HMA surface area.

2 Asphalt Binder. By weight, as follows:

    Method 1 will be used for determining asphalt binder quantity unless otherwise directed in writing. The procedure initially used will be the one used for the duration of the project. No payment is made for any asphalt binder more than 0.4 percent above the optimum asphalt binder content specified in the JMD.

    a. Method 1: Percent of asphalt binder for each sublot multiplied by the total HMA weight represented by that sublot. The Engineer will use either ATM 405 or ATM 406 to determine the percent of asphalt binder. The same test method used for the acceptance testing of the sublot will be used for computation of the asphalt binder quantity. In the absence of testing, the percent of asphalt binder is the target value for asphalt binder in the JMD.

    b. Method 2: Supplier's invoices minus waste, diversion and remnant. This procedure is an Engineer's option for projects where deliveries are made in tankers and the asphalt plant is producing HMA for one project only.

    The Engineer may direct, at any time that tankers are weighed in the Engineer's presence before and after unloading. If the weight determined at the project varies more than 1 percent from the invoice amount, payment is based on the weight determined at the project.

    Any remnant or diversion will be calculated based on tank stickings or weighing the remaining asphalt binder. The Engineer will determine the method. The weight of asphalt binder in disposed HMA is calculated using the target value for asphalt binder as specified in the JMD.

3. Job Mix Design. When specified, a Contractor furnished JMD is measured as one according to the HMA class and type.

4. Temporary Pavement. By weight, without deduction for the weight of asphalt binder or anti-strip additive.

5. Leveling Course. By Lane-Station (12 foot width) or by weighing without deduction for the weight of asphalt binder or anti-strip additive.

6. HMA Price Adjustment. Calculated by quality level analysis under Subsection 412-4.03.1.

7. Longitudinal Joint Compaction Price Adjustment. By the linear foot of top lift longitudinal joint under Subsection 412-4.03.2.

8. Joint Adhesive. By the linear foot of longitudinal and transverse joint.

9. Pavement Smoothness Price Adjustment. Calculated from inertial profiler data using FHWA's ProVAL software under Subsection 412-4.03.3.

10. Asphalt Material Price Adjustment. Determined under Subsection 412-4.04.

11. Liquid Anti-Strip Additive. Based on the number of tons of asphalt binder containing required additive.

12. Crack Repair. From end to end of the crack repaired according to 412-3.10, measured horizontally along the centerline of the crack. Cleaning loose material from cracks, asphalt binder, and HMA to fill cracks will not be measured separately but are subsidiary.

13. Prelevel for Ruts, Delaminations, and Depressions. By the surface area where prelevel is placed according to 412-3.10(1), measured according to Section 109. Asphalt binder, HMA, and cleaning loose material, will not be measured separately but are subsidiary.

14. Repair Unstable Pavement. By the surface area of pavement repaired according to 412-3.10(2), measured according to Section 109. Asphalt binder, HMA, and removal of pavement and base course will not be measured separately but are subsidiary.

412-4.02 ACCEPTANCE SAMPLING AND TESTING. The bid quantity of each type of HMA produced and placed will be divided into lots and the lots evaluated individually for acceptance.

A lot is normally 5,000 tons. The lot is divided into sublots of 500 tons, each randomly sampled and tested for asphalt binder content and gradation according to this Subsection. Compaction is evaluated with full coverage compaction mapping with a dielectric profiling system (DPS). The lot is evaluated for price adjustment according to Subsection 412-4.03.1. Seasonal startup or a new JMD requires starting a new lot.

If less than 8 sublots have been placed at the time a lot is terminated, the material in the shortened lot will be included as part of the prior lot. The price adjustment computed for the prior lot will include the samples from the shortened lot. Compaction test results from material in the shortened lot will be based on the MSG of the shortened lot. If there is no prior lot, and there are at least 3 sublots, the material in the shortened lot will be considered as a lot and the price adjustment will be based on the actual number of test results in the shortened lot. If there are less than 3 sublots, the HMA will be accepted for payment based on the Engineer's approval of the JMD, and placement and compaction of the HMA to the specified depth, finished surface requirements and tolerances. The Engineer reserves the right to perform any testing required in order to determine acceptance.

If 8 or 9 sublots have been placed at the time a lot is terminated, they will be considered as a lot and the price adjustment will be based on the actual number of test results in the shortened lot.

If the bid quantity is between 1,500 to 4,999 tons, the quantity is considered one lot. The lot is divided into sublots of 500 tons, each randomly sampled and tested for asphalt binder content, compaction, and gradation according to this Subsection. The lot is evaluated for price adjustment according to Subsection 412-4.03.1.

For bid quantity less than 1,500 tons, HMA will be accepted for payment based on the Engineer's approval of the JMD, and placement and compaction of the HMA to the specified depth, finished surface requirements and tolerances. The Engineer reserves the right to perform any testing required in order to determine acceptance.

Sampling and testing include the following:

1. Asphalt Binder Content. HMA samples shall be taken randomly by the Contractor in the presence of the Engineer from behind the paver screed before initial compaction, or will be taken randomly by the Engineer from the windrow, according to ATM 402 or ATM 403, at the discretion of the Engineer. The

location (behind the paver screed or windrow) will be determined at the pre-paving meeting. Random sampling locations will be determined by the Engineer.

Two separate samples will be taken, one for acceptance testing and one held in reserve for retesting if requested. Asphalt binder content will be determined according to ATM 406.

2. Aggregate Gradation. Aggregates tested for gradation acceptance will have the full tolerances from Table 412-2 applied. For HMA samples, the gradation will be determined according to ATM 408 from the aggregate remaining after the ignition oven (ATM 406) has burned off the asphalt binder.

3. Compaction. The Engineer will determine the percent compaction of HMA pavement using a Dielectric Profiling System (DPS) conforming to AASHTO PP 98-20 “Asphalt Surface Dielectric Profiling System using Ground Penetrating Radar”. The DPS shall be calibrated to the mix design, be instrumented with a Global Positioning System (GPS) with a minimum operating tolerance of ± 2.5 cm, and utilize integral DPS software to analyze the data.

   a. Map Mat Compaction with continuous-full-coverage DPS data. Continuous-full-coverage DPS data is defined for the mat as parallel lines of data spaced no more than 2.5 feet apart across the new mat width. A line of DPS data shall include a compaction value every 0.5 feet. The DPS software shall summarize mat compaction values for each station by computing average compaction and percent of compaction values measured that are equal to or greater than the minimum specified mat compaction within each station.

   b. Measure longitudinal joint compaction with one DPS sensor centered over the longitudinal joint, between driving lanes, creating a line of compaction measurements, spaced every 0.5 feet along each station. Report average joint compaction value by station as well as percentage of compaction values equal to or greater than the specified minimum joint compaction.

4. Binder and Gradation Retest. When test results have failed to meet specifications, retest of acceptance test results for asphalt binder content and gradation may be requested provided the quality control requirements of Subsection 412-3.02 are met. Deliver this request in writing to the Engineer within 7 days of receipt of the final test of the lot. The original test results are discarded and the retest result is used in the price adjustment calculation regardless of whether the retest result gives a higher or lower pay factor. Only one retest per sample is allowed. When gradation and asphalt binder content are determined from the same sample, a request for a retest of either gradation or asphalt binder content results in a retest of both. Both gradation and asphalt binder content retest results are used in the price adjustment calculation. Retesting will be performed by a department laboratory.

5. Compaction Retest. When test results for a lot have failed to meet specifications, retest of acceptance test results for asphalt mat compaction may be requested provided the quality control requirements of Subsection 412-3.02 are met. Deliver this request in writing to the Engineer within 7 days of receipt of the final test of the lot. Verify DPS accuracy with 3 cores per lot retested. Measure the mat % compaction of the marked core locations with DPS before coring. Core the mat in 3 locations marked by the Engineer that are representative of the target compaction range (93-96%). Drill one core each from locations reading 93±1%, 94-95%, and 96±1% compaction with the DPS. Compare the average of 3 core % compaction test results with the average of the 3 DPS % compaction readings. If the average of the 3 DPS % compaction readings varies more than 1.5 percent from the core % compaction test results, then recalibrate the DPS % compaction algorithm to align with the core compaction values for the lot. Rerun the lot mat compaction analysis with the corrected calibration and use the resulting mat compaction pay factor for that lot. No other compaction retesting will be performed.

6. Asphalt Binder Grade. The lot size for asphalt binder is 200 tons. If a project has more than one lot and the remaining asphalt binder quantity is less than 150 tons, it is added to the previous lot and that total quantity will be evaluated as one lot. If the remaining asphalt binder quantity is 150 tons or greater, it is sampled, tested and evaluated as a separate lot.

If the bid quantity of asphalt binder is between 85 – 200 tons, the contract quantity is considered as one lot and sampled, tested, and evaluated according to this subsection. Quantities of asphalt binder less than 85 tons will be accepted based on manufacturer's certified test reports and certification of compliance.

Sample asphalt binder at the plant from the supply line in the presence of the Engineer according to ATM 401. The Engineer will take immediate possession of the samples. Take three samples from each lot, one for acceptance testing, one for Contractor requested retesting, and one held in reserve for referee testing if requested. Meet 412- 2.01 requirements for asphalt binder quality.

7. Asphalt Binder Grade Retest. Retest of acceptance test results may be requested provided the quality control requirements of Subsection 412-3.02 are met. Deliver the request in writing to the Engineer within 7 days of receipt of notice of failing test. The original test results are discarded and the retest result is used for acceptance. Only one retest per sample is allowed.

   If the contractor challenges the result of the retest, the referee sample held by the Engineer will be sent to a mutually agreed upon independent AASHTO accredited laboratory for testing. The original acceptance test result, the retest acceptance test result, and the referee sample test result will be evaluated according to ASTM D3244 to obtain an Assigned Test Value (ATV). The ATV value will be used to determine if the asphalt binder conforms to the contract. The Contractor shall pay for the referee sample test if the ATV confirms the asphalt binder does not meet contract requirements.

412-4.03 EVALUATION OF MATERIALS FOR ACCEPTANCE.

The Engineer may reject material which appears to be defective based on visual inspection. If a test of rejected material is requested, a minimum of two samples are collected from the rejected material and tested. If all test results are within specification limits, payment for the material is made.

The following methods are applied to each type of HMA when Price Adjustment Pay Items are included in the Bid Schedule. These methods describe how price adjustments are determined based on the quality of the HMA, longitudinal joint compaction and pavement smoothness.

1. HMA Price Adjustment. Acceptance test results for HMA asphalt binder content, gradation and compaction are used in HMA price adjustment. HMA asphalt binder and gradation test results for a lot are analyzed collectively and statistically by the Quality Level Analysis (QLA) method as specified in Subsection 106-1.03.3 to determine the total estimated percentage of the lot that is within specification limits. The values for percent passing the #200 sieve, asphalt binder content and compaction test results are reported to the nearest 0.1 percent. All other sieves used in QLA are reported to the nearest whole number.

   The HMA price adjustment is based on the lower of two pay factors. The first factor is a Composite Pay Factor (CPF) for HMA that includes gradation and asphalt binder content. The second is the mat Density Pay Factor (DPF).

   A lot containing material with less than a 1.000 pay factor is accepted at an adjusted price, provided that pay factor is at least 0.800 and there are no isolated defects identified by the Engineer. A lot containing material that fails to obtain the minimum pay factor is considered unacceptable and rejected under Subsection 105-1.11.

   HMA pay factors are computed as follows:

   a. All statistical Quality Level Analysis (QLA) is computed using the Engineer's Price Adjustment programs.

   b. The USL and LSL are equal to the Target Value (TV) plus and minus the allowable tolerances in Table 412-6, or as shown below. The TV is the specification value shown in the approved Job Mix Design.

TABLE 412-6
HMA LOWER SPECIFICATION LIMIT (LSL) &
UPPER SPECIFICATION LIMIT (USL)

Measured Characteristics	LSL	USL
¾" or largest sieve size	99	100
½ inch sieve or first sieve retaining aggregate	TV-6	TV+6
3/8 inch sieve	TV-6	TV+6
No. 4 sieve	TV-6	TV+6
No. 8 sieve	TV-6	TV+6
No. 16 sieve	TV-5	TV+5
No. 30 sieve	TV-4	TV+4
No. 50 sieve	TV-4	TV+4
No. 100 sieve	TV-3	TV+3
No. 200 sieve	TV-2.0	TV+2.0
Asphalt Binder Content, %	TV-0.4	TV+0.4

c. The percent within limits (PWL), Quality Levels and characteristic pay factors (PFs) are determined by the Engineer for each Lot in accordance with Subsection 106-1.03.3. The Composite Pay Factor (CPF) for the lot is determined from gradation and asphalt binder content (ac) acceptance test results using the following example formula:

$$CPF = \frac{[f_{3/4\ inch}\ (PF_{3/4\ inch}) + f_{1/2\ inch}\ (PF_{1/2\ inch}) + ...f_{ac}\ (PF_{ac})]}{\Sigma f}$$

Table 412-7 gives the weight factor (f) for each test property considered.

TABLE 412-7
WEIGHT FACTORS

Property	Factor "f"
1 inch sieve	-
¾ inch sieve	-
½ inch sieve	4
3/8 inch sieve	5
No. 4 sieve	5
No. 8 sieve	5
No. 16 sieve	5
No. 30 sieve	6
No. 50 sieve	6
No. 100 sieve	4
No. 200 sieve	20
Asphalt Content, %	40

d. The mat Density Pay Factor (DPF) is the average of the Station compaction Pay Factors (SPF) comprising the lot. The minimum acceptable compaction is 93.0% MSG of the JMD.

- Each Station Pay Factor (SPF) is the product of the Compaction Factor (A) times the Percent Conforming Factor (B) measured within that station of pavement.
  - A = Minimum of [0.0125 x (average % MSG compaction) – 0.1625] or [1.05]
  - B = 0.01 x (% conformance to 93.0% minimum compaction)
  - SPF = A x B

Lot Density Pay Factor (DPF)= average of the SPFs for the lot. (1.05 is maximum DPF, positive DPF is incentive, negative DPF is disincentive)

The CPF and DPF are rounded to the nearest 0.001. The price adjustment for each individual lot is calculated as follows:

HMA Price Adjustment = [(CPF or DPF)* -1.00] x (tons in lot) x (PAB)
* CPF or DPF, whichever is lower. No bonus or incentive will be paid if there is an Asphalt Binder penalty or if the lot average mat compaction is less than 93.0% MSG.
PAB = Price Adjustment Base = [Bid Unit Price for 412.0001.___ + (n/100) x Bid Unit Price for 412.0004.____],

Where
n = Optimum asphalt binder content percent, established by the JMD.
The HMA Price Adjustment is the sum of the price adjustments for each lot and paid for under Item 412.0008.____.

2. Longitudinal Joint Compaction Price Adjustment. Apply a longitudinal joint price adjustment to top lift asphalt paving containing a newly constructed longitudinal joint between driving lanes. Base top lift longitudinal joint compaction price adjustment on two factors: (A) a compaction factor and (B) a conforming factor.

   The Joint Price Adjustment (JPA) is calculated by Station from the average of all DPS reading in that Station (typically 200). The sum of JPAs from all full stations and partial stations of more than fifty feet in length, within a lot comprise the joint price adjustment for that lot.

   - Each Station JPA is the product of the Compaction Factor (A) times the Percent Conforming Factor (B) measured within that station of pavement.
     - Aj = Lesser of [$75*(D-93)] or $300 for D≥89.0 (If D<89.0 then JPA = -$300)
       Where: D = Station average % Compaction rounded to nearest 0.1 %
     - Bj = Greater of [0.1*(PC-90)] or 0.00
     - JPA = Aj x Bj

   if average station joint compaction is **less than 93.0% MSG**, apply disincentive, and seal the joint. For average compaction **less than 89%** deduct $3.00 per lineal foot and seal the joint.

   The Longitudinal Joint Compaction Price Adjustment is the total price adjustment paid for under Item 412.0009.____.

3. Pavement Smoothness Price Adjustment. Pavement smoothness will be measured by the Engineer and reported as IRI (inches/mile), according to Subsection 412-3.18. Incentive for pavement smoothness shall apply only if both the project average CPF and DPF are greater than or equal to 1.000. Disincentive for pavement smoothness shall apply regardless of the project average CPF or DPF.

   The Engineer will calculate the pavement smoothness price adjustment according to Method 1 or Method 2 below, as identified in the bid schedule.

   Method 1: SPA = PAB x PQ x SF,

   Where:
   SPA = Pavement Smoothness Price Adjustment
   PAB = Price Adjustment Base
   PAB = [Bid Unit Price for 412.0001.____ + (n/100) x Bid Unit Price for 412.0004.____],
   n = optimum asphalt binder content, percent, established by the JMD
   PQ = Top layer HMA quantity, tons

SF = Smoothness Factor (Table 412-8)

TABLE 412-8
SMOOTHNESS FACTOR (SF)

IRI (in./mile)	SF
Less than 40	0.05
40 to 70	0.05 – (IRI - 40)/600
70 to 90	0.00
90 to 120	(90 – IRI)/120
Greater than 120*	-

* Corrective Work required, see Subsection 412-3.18

Method 2: SPA = PAB x PQ x SF,

Where:
SPA = Pavement Smoothness Price Adjustment
PAB = Price Adjustment Base
PAB = [Bid Unit Price for 412.0001.____ + (n/100) x Bid Unit Price for 412.0004.____],
n = optimum asphalt binder content, percent, established by the JMD
PQ = Top layer HMA quantity, tons
SF = Smoothness Factor = 0.12 x RR – 0.02; SF not to exceed 0.05
RR = Roughness Reduction = (Initial IRI – Final IRI) / Initial IRI
Initial IRI = Pre-project average IRI as measured and reported by the Engineer. The Initial IRI will either be included in the bid documents or the timeline for when the Initial IRI will be measured will be identified in the bid documents.
Final IRI = Top layer HMA average IRI as measured and reported by the Engineer according to Subsection 412-3.18.

The Pavement Smoothness Price Adjustment is the total price adjustment paid for under Item 412.0010.____.

4. Asphalt Binder Price Adjustment. A lot quantity of asphalt binder, with a pay factor less than 1.00, is accepted or rejected per Table 412-9 Asphalt Binder Pay Factors.

TABLE 412-9

ASPHALT BINDER PAY FACTORS

Pay Factor			1.01	1.00	0.95	0.90	0.75	Reject
RTFO (Rolling Thin Film Oven)								
DSR(a.1)	All Grades	$G^*/Sin\delta$, kPa^{-1}	≥ 2.69	2.68–2.20	2.19–1.96	1.95–1.43	1.42–1.10	< 1.10
	PG 58-34 E	$J_{NR\ 3.2}$	≤ 0.19	0.20–0.25	0.26–0.29	0.30–0.39	0.40–0.50	> 0.50
		% $Rec_{3.2}$	≥ 90.0	89.9–85.0	84.9–80.0	79.9–75.0	74.9–70.0	< 70.0
	PG 64-40 E	$J_{NR\ 3.2}$	< 0.05	0.05–0.10	0.11–0.15	0.16–0.20	0.21–0.25	> 0.25
		% $Rec_{3.2}$	≥ 97.0	96.9–95.0	94.9–91.0	90.9–85.0	84.9 – 80.0	< 80.0
PAV (Pressurized Aging Vessel)								

DS[a.3]	PG 64-40 E	G*Sinδ, kPa	≤ 4711	4712–5000	5001–5289	5290–5578	5579–5867	> 5867
	PG 58-34 E	G*Sinδ, kPa	≤ 5700	5701–6000	6001–6300	6301–6600	6601 – 7000	> 7000
CS[a.4 & 5]	All Grades[a.4]	BBR, *S*, MPa	≤ 247	248–300	301–338	339–388	389–449	≥ 450
	All Grades[a.5]	BBR, *m*	≥ 0.320	0.319–0.300	0.299–0.294	0.293–0.278	0.277–0.261	< 0.261

Multiple Stress Creep Recovery (MSCR), Dynamic Shear (DS), Creep Stiffness (CS)

a. Asphalt Binder Pay Adjustment = (Lowest Pay Factor – 1.00) x (tons in lot) x PAB x 5
Select the lowest pay factor from:
RTFO (test the binder residue at the performance grade temperature)
(1) DS, All Grades, G*/Sinδ, kPa^{-1}
(2) MSCR: PG, Select the highest pay factor corresponding to, either $J_{NR\ 3.2}$ or % $Rec_{3.2}$ values
PAV
(3) DS, PG, G*Sinδ, kPa
(4) CS, All Grades, BBR, *S* MPa
(5) CS, All Grades, BBR, *m*

b. If three consecutive acceptance samples are out of specification, stop HMA production immediately and submit a corrective action plan to the Engineer for approval.

The Asphalt Binder Price Adjustment is the sum of the price adjustments for each lot and paid for under Item 412.2022 ____________

412-4.04 ASPHALT MATERIAL PRICE ADJUSTMENT.

Asphalt Material Price Adjustment. This subsection provides a price adjustment for asphalt material by: (1) additional compensation to the contractor or (2) a deduction from the contract amount.

1. This provision shall apply:

   a. To asphalt material meeting the criteria of Section 702, and is included in items listed in the bid schedule of Sections 306, 307, 308, 401 thru 412, 520, 608 and 609.

   b. To cost changes in asphalt material that occur between the date of bid opening and the date on the certified bill of lading from the asphalt material refiner/producer.

   c. When there is more than a 7.5 percent increase or decrease in the Alaska Asphalt Material Price Index, AAMPI, from the date of bid opening to the date on the certified bill of lading from the asphalt refiner/producer.

2. Provide the certified bill of lading from the asphalt material refiner/producer.

3. The AAMPI is calculated bimonthly on the first and third Friday of each month, and will remain in effect from the day of calculation until the next bimonthly calculation. The AAMPI is posted on the Department's Statewide Materials website at and calculated according to the formula posted there. http://www.dot.state.ak.us/stwddes/desmaterials/aprice_index.shtml

4. Price adjustment will be cumulative and calculated with each progress payment. Use the AAMPI in effect on the date of the certified bill of lading from the asphalt material refiner/producer, to calculate the price adjustment for asphalt material The Department will increase or decrease payment under this contract by the amount determined with the following asphalt material price adjustment formula:

   For an increase exceeding 7.5 percent, additional compensation = [(IPP – IB) – (0.075 x IB)] x Q

For a decrease exceeding 7.5 percent, deduction from contract = [(IB – IPP) – (0.075 x IB)] x Q

Where:
Q = Quantity of Asphalt Material incorporated into project during the pay period, in tons as measured by the Engineer
IB = Index at Bid: the Bi-monthly AAMPI in effect on date of bid, in dollars per ton
IPP = Index at Pay Period: The bi-monthly AAMPI in effect on the date shown on the certified bill of lading from the asphalt refiner/producer, in dollars per ton

5. Method of measurement for determining Q (quantity) is the weight of asphalt material that meets the criteria of this subsection and is incorporated into the project. The quantity does not include aggregate, mineral filler, blotter material, thinning agents added after material qualification, or water for emulsified asphalt. The quantity for emulsified asphalts will be based on the asphalt residue material only and will be calculated using the percent residue from testing, or if not tested, from the manufacturer's certificate of compliance.

412-5.01 BASIS OF PAYMENT.

The following items, unless included as individual Pay Items, are subsidiary to the Section 408 Hot Mix Asphalt Pavement related Pay Items as included in the bid schedule:

- Asphalt binder
- Liquid anti-strip additives
- Tack coat
- Crack sealing
- Crack repair
- Joint adhesive
- Surface sealing of longitudinal joints
- Surface tolerance corrections
- Patching defective areas
- Prelevel for ruts, delaminations, and depressions
- Repair unstable pavement
- Job mix design
- Compaction profiles, Subsection 408-2.10 Process Quality Control
- Repair work and materials when milling equipment breaks through existing pavement – Subsection 412-3.10 Preparation of Existing Surface
- Work and materials associated with Subsection 412-3.06 Hauling Equipment
- Work and materials associated with Subsection 412-3.20 Roadway Maintenance

Test Strips: Subsection 412-2.10 Process Quality Control.

a. Approved. Test strip construction and material, approved by the Engineer in writing, as meeting the specification requirements will be paid for at the Contract unit prices for HMA and asphalt binder as included in the Bid Schedule. Price adjustments 412.2008.____, 412.2009.____, 412.2010.____ and 412.2021.____ do not apply.

b. Failed. The materials, construction of, removal and disposal of a failed test strip will be at the Contractor's expense.

Asphalt binder, liquid anti-strip additives and tack coat are subsidiary to HMA Item 412.0001.____ and 412.0006.____ unless specified as Pay Items. Asphalt binder and liquid anti-strip are subsidiary to HMA for Items 412.0002.____, 412.0003.____, 412.0005.____, 412.0011.____, and 412.0012.____.

Item 412.0008.____ HMA price adjustment is the sum of the price adjustments for each material lot and for deductions and fees assessed. Deductions and fees assessed include:

1. The Department will bear the cost of the initial JMD evaluation for each type of HMA. All subsequent evaluations required to obtain an approved JMD will be assessed a fee of $6,000 per evaluation. Failed retest will result in a fee of $2,500.

2. Failure to cut core samples within the specified period will result in a deduction of $100 per sample per day.

3. Failure to backfill voids left by sampling within the specified period will result in a deduction of $100 per hole per day.

4. If an asphalt binder referee test is requested and the ATV confirms the asphalt binder does not meet contract requirements, a fee of $500 will be assessed.

Item 412.0008.____ HMA price adjustment does not apply to:

1. HMA, when contract quantity is less than 1500 tons

2. HMA for leveling course 412.0002.____ and 412.0003.____

3. Temporary HMA 412.0005.____

4. Driveway HMA 412.0011.____ and 412.0012.____

Payment will be made under:

PAY ITEM		
Item Number	**Item Description**	**Unit**
412.0001.____	HMA, Type ___; Class ___	TON
412.0002.____	HMA, Leveling Course, Type ___; Class ___	LnSt
412.0003.____	HMA, Leveling Course, Type ___; Class ___	TON
412.0004.____	Asphalt Binder, Grade ____	TON
412.0005.____	HMA, Temporary, Type ___; Class ___	TON
412.0006.____	HMA, Type ___; Class ___	SY
412.0007.____	Liquid Anti-Strip Additive	CS
412.0008.____	HMA Price Adjustment, Type ___; Class ___	CS
412.0009.____	Longitudinal Joint Compaction Price Adjustment	CS
412.0010.____	Pavement Smoothness Price Adjustment, Method ___	CS
412.0011.____	HMA, Driveway, Type ___; Class ___	LS
412.0012.____	HMA, Driveway, Type ___; Class ___	TON
412.0013.____	Job Mix Design	EACH
412.0014.____	Joint Adhesive	LF
412.0015.____	Asphalt Material Price Adjustment	CS
412.0016.____	Crack Repair	LF
412.0017.____	Prelevel for Ruts, Delaminations, and Depressions	SY
412.0018.____	Repair Unstable Pavement	SY
412.2022.____	Combined Price Adjustment	CS