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Partnered Pavement Research Program (PPRC) Contract Strategic Plan Element 4:18:
Warm-Mix Asphalt

PREPARED FOR:

California Department of Transportation
Division of Research and Innovation
Office of Roadway Research

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DOCUMENT RETRIEVAL PAGE			Research Report: UCPRC-RR-2008-11
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Abstract:
This first-level report describes the first phase of a warm-mix asphalt study, which compares the performance of a control mix, produced and constructed at conventional hot-mix asphalt temperatures, with three mixes produced with warm-mix additives, produced and compacted at approximately 35°C (60°F) lower than the control. The additives tested included *Advera WMA*®, *Evotherm DAT*™, and *Sasobit*®. The test track layout and design, mix design and production, and test track construction are discussed, as well as the results of Heavy Vehicle Simulator (HVS) and laboratory testing. Key findings from the study include:

- Adequate compaction can be achieved on warm-mixes at lower temperatures.
- Optimal compaction temperatures are likely to differ between the different warm-mix technologies. However, a temperature reduction of at least 35°C (60°F) is possible.
- Based on the results of HVS testing, it is concluded that the use of any of the three warm-mix asphalt technologies used in this experiment will not significantly influence the rutting performance of the mix.
- Laboratory moisture sensitivity testing indicated that all the mixes tested were potentially susceptible to moisture damage. There was, however, no difference in the level of moisture sensitivity between the control mix and mixes with the additives assessed in this study.
- Laboratory fatigue testing indicated that the warm-mix technologies used in this study will not influence the fatigue performance of a mix.
- Quality control checks on the mix immediately after production revealed that lower specific gravities and higher air-void contents were recorded on the warm mixes.
- The cost benefits of using the warm-mix technologies could not be assessed in this study due to the very small quantities produced.

The HVS and laboratory testing completed in this phase have provided no results to suggest that warm-mix technologies should not be used in California. Final recommendations on the use of this technology will only be made after further research and monitoring of full-scale pilot studies on in-service pavements is completed. Interim recommendations include:

- The use of warm-mix technologies should continue in full-scale pilot studies on in-service pavements.
- HVS testing to assess moisture sensitivity should continue to confirm the laboratory findings.
- Laboratory testing on laboratory-mixed, laboratory-compacted specimens should proceed to determine whether representative mixes can be produced in the laboratory and to determine how and whether test results differ from field-mixed, field-compacted specimens.

Keywords:
Warm-mix asphalt, WMA, accelerated pavement testing, Heavy Vehicle Simulator

Proposals for implementation:
Continue with Phase 2 moisture sensitivity testing. Continue with implementation in pilot studies.

Related documents:
Work plan, UCPRC-WP-2007-01.

Signatures:

D. Jones	J. Harvey	D. Spinner	J. Harvey	T.J. Holland
1st Author	**Technical Review**	**Editor**	**Principal Investigator**	**Caltrans Contract Manager**

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the State of California or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

PROJECT OBJECTIVES

The objective of this project is to determine whether the use of additives to reduce the production and construction temperatures of hot-mix asphalt influences performance of the mix. This will be achieved through the following tasks:

1. Preparation of a workplan to guide the research;
2. Monitoring the construction of Heavy Vehicle Simulator (HVS) and in-service test sections;
3. Sampling of mix and mix components during asphalt concrete production and construction;
4. Trafficking of demarcated sections with the HVS in a series of tests to assess performance;
5. Conducting laboratory tests to identify comparable laboratory performance measures;
6. Monitoring the performance of in-service pilot sections; and
7. Preparation of first- and second-level analysis reports and a summary report detailing the experiment and the findings.

This report covers Tasks 2, 3, 4, 5, and 7.

ACKNOWLEDGEMENTS

The University of California Pavement Research Center acknowledges the following individuals and organizations who contributed to the project:

- Ms. Terrie Bressette, Ms. Cathrina Barros, Mr. Glenn Johnson, and Dr. Joe Holland, Caltrans
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EXECUTIVE SUMMARY

The first phase of a comprehensive study into the use of warm-mix asphalt has been completed for the California Department of Transportation (Caltrans) by the University of California Pavement Research Center (UCPRC). The study, based on a work plan approved by Caltrans, included the identification of an appropriate site for the experiment, the design and construction of a test track, an accelerated loading test using the Heavy Vehicle Simulator (HVS) to assess rutting behavior, and a series of laboratory tests on specimens sampled from the test track. The objective of the study is to determine whether the use of additives to reduce the production and construction temperatures of asphalt concrete influences performance of the mix. The study compared the performance of a control mix, produced and constructed at conventional hot-mix asphalt temperatures, with three warm-mixes, produced and compacted at approximately 35°C (60°F) lower than the control. The additives tested included *Advera WMA®*, *Evotherm DAT™*, and *Sasobit®*.

The test track is located at the Graniterock Company's A.R. Wilson Quarry and Asphalt Plant near Aromas, California. The design and construction of the test track was a cooperative effort between Caltrans, the UCPRC, Graniterock, and the three warm-mix technology suppliers. The test track is 80 m by 8.0 m (262 ft by 26 ft) divided into four test sections (Control, Advera, Evotherm, and Sasobit). The pavement structure consists of the existing subgrade/subbase material overlying bedrock, with 300 mm (12 in.) of imported aggregate base, and two 60 mm (2.4 in.) lifts of asphalt concrete. A standard mix design was used and no adjustments were made to accommodate the additives. Target production temperatures for the Control mix were set at 155°C (310°F) and at 120°C (250°F) for the warm-mixes. The test track was constructed in September 2007, using asphalt from the commercial asphalt mix plant at the quarry. Specimens were removed from the test track for laboratory testing.

The first phase of Heavy Vehicle Simulator (HVS) testing commenced in October 2007 after a six-week curing period and was completed in April 2008. This testing compared early rutting performance at elevated temperatures (pavement temperature of 50°C at 50 mm [122°F at 2.0 in.]), using a 40 kN (9,000 lb) load on a standard dual wheel configuration and a unidirectional trafficking mode. Laboratory testing commenced in December 2007 and was completed in July 2008. The test program included shear testing, wet and dry fatigue testing, Hamburg Wheel-Track testing, and determination of the wet-to-dry tensile strength ratio. The results of this testing will be used to identify subsequent research needs.

Key findings from the study include:

- A Hveem mix design that met Caltrans requirements for Type A 19 mm maximum dense-graded asphalt concrete was used in the study. The target gradation met Caltrans requirements for both the Coarse and Medium gradations. The recommended bitumen content was 5.1 to 5.4 percent by mass of aggregate, which was based on the minimum air-void content under standard kneading compaction. The mix design had very high Hveem stabilities.
- A consistent base-course was constructed on the test track using material produced at the nearby quarry. Some overwatering occurred in the early stages of construction resulting in some moist areas in the pavement, which influenced measured densities and deflections. These areas are unlikely to effect later performance of the test track. The very stiff base is likely to complicate any planned fatigue cracking experiments in that a very high number of HVS repetitions will likely be required before any distress occurs.
- Minimal asphalt plant modifications were required to accommodate the warm-mix additives.
- No problems were noted with producing the asphalt mixes at the lower temperatures. The target mix production temperatures (i.e., 155°C and 120°C [310°F and 250°F]) were achieved.
- Although a PG 64-16 asphalt binder was specified in the work plan, subsequent tests by the Federal Highway Administration indicated that the binder was rated as PG 64-22. This should not affect the outcome of the experiment. After mixing Advera and Sasobit to the binder, the PG grading changed from PG 64-22 to PG 70-22. The addition of Evotherm did not alter the PG grade.
- The Control, Advera, and Evotherm mixes met the project mix design requirements. The binder content of the Sasobit mix was 0.72 percent below the target binder content and 0.62 percent below the lowest permissible binder content. This probably influenced performance and was taken into consideration when interpreting the HVS and laboratory test results presented in this report.
- Graniterock Company did not perform Hveem compaction or stability tests for quality control purposes as there is no protocol for adjusting the standard kneading compaction temperature for mixes with warm-mix additives. Instead, Marshall and Superpave Gyratory compaction were performed in the Graniterock laboratory next to the asphalt plant on mix taken from the silo.
- Laboratory quality control tests on the Control mix (specimens compacted with Marshall and Superpave Gyratory compaction) had a higher specific gravity and lower air-void content, compared to the mixes with additives. It is not clear whether this was a testing inconsistency or is linked to the lower production and specimen preparation temperatures. This will need to be investigated during Phase 2 laboratory investigations.
- Moisture contents of the mixes with additives were notably higher than in the Control mix, indicating that potentially less moisture will evaporate from the aggregate at lower production temperatures. All mixes were, however, well within the minimum Caltrans-specified moisture

content level. Aggregate moisture contents will need to be controlled in the stockpiles and maximum moisture contents may need to be set prior to mix production when using warm-mix technologies.

- Construction procedures and final pavement quality did not appear to be influenced by the lower construction temperatures. The Advera mix showed no evidence of tenderness, and acceptable compaction was achieved. Some tenderness was noted on the Evotherm and Sasobit sections resulting in shearing under the rollers at various stages of breakdown and/or rubber-tired rolling, indicating that the compaction temperatures were still higher than optimal. No problems were observed after final rolling at lower temperatures.
- Interviews with the paving crew after construction revealed that no problems were experienced with construction at the lower temperatures. Improved working conditions were identified as an advantage. Tenderness on the Evotherm and Sasobit sections was not considered as being significantly different from that experienced with conventional mixes during normal construction activities.
- Although temperatures at the beginning of compaction on the warm-mix sections were considerably lower than the Caltrans-specified limits, the temperatures recorded on completion of compaction were within limits, indicating that the rate of temperature loss in the mixes with additives was lower than that on the Control mix, as expected.
- Some haze/smoke was evident on the Control mix during transfer of the mix from the truck to the paver. No haze or smoke was observed on the mixes with additives.
- Average air-void contents on the Control and Advera sections were 5.6 percent and 5.4 percent respectively. Those on the Evotherm and Sasobit sections, which showed signs of tenderness during rolling, were approximately 7.0 percent, with the caveat that the Sasobit mix binder content was lower than the target while that for the Evotherm sections was not. Based on these observations, it was concluded that adequate compaction can be achieved on warm-mixes at the lower temperatures. Optimal compaction temperatures are likely to differ between the different warm-mix technologies.
- Skid resistance measurements indicated that the warm-mix additives tested do not influence the skid resistance of an asphalt mix.
- HVS trafficking on each of the four sections revealed that the duration of the embedment phases (high early-rutting phase of typical two-phase rutting processes) on the Advera and Evotherm sections were similar to the Control. However, the rut depths at the end of the embedment phases on these two sections was slightly higher than the Control, which was attributed to less oxidation of the binder during mix production at lower temperatures. Rutting behavior on the warm-mix sections followed similar trends to the Control after the embedment phase. The performance of the

Sasobit section could not be directly compared with the other three sections given that the binder content of the mix was significantly lower.

- Laboratory test results indicate that use of the warm-mix technologies assessed in this study does not significantly influence the performance of the asphalt concrete when compared to control specimens produced and compacted at conventional hot-mix asphalt temperatures. However, moisture sensitivity testing indicated that all the mixes tested were potentially susceptible to moisture damage. There was, however, no difference in the level of moisture sensitivity between the Control mix and mixes with warm-mix additives.

The HVS and laboratory testing completed in this phase have provided no results to suggest that warm-mix technologies should not be used in California. Final recommendations on the use of this technology will only be made after further research and monitoring of full-scale pilot studies on in-service pavements is completed. Interim recommendations include the following:

- The use of warm-mix technologies should continue in full-scale pilot studies on in-service pavements.
- Although laboratory testing indicated that the warm-mix technologies assessed in this study did not increase the moisture sensitivity of the mix, HVS testing to assess moisture sensitivity should continue as recommended in the work plan to confirm these findings. Subsequent laboratory testing of moisture sensitivity should assess a range of different aggregates given that all of the mixes tested in this study where considered to be moisture sensitive.
- Phase 2 laboratory testing on laboratory-mixed, laboratory-compacted specimens should proceed to determine whether representative mixes can be produced in the laboratory and to determine how and whether laboratory test results on these specimens differ from those on field-mixed, field-compacted specimens.
- As part of the Phase 2 laboratory study, protocols need to be developed for adjusting laboratory specimen-preparation compaction temperatures for mixes with warm-mix additives. It is unlikely that any national studies will develop these protocols for Hveem mix designs, which are still used in California.

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LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway and Transport Officials
ASTM	American Society for Testing and Materials
Caltrans	California Department of Transportation
CTM	Circular Track Meter
DCP	Dynamic Cone Penetrometer
DFT	Dynamic Friction Tester
DGAC	Dense-graded asphalt concrete
ESAL	Equivalent standard axle load
FHWA	Federal Highway Administration
FMFC	Field-mixed, field-compacted
FMLC	Field-mixed, laboratory-compacted
FWD	Falling Weight Deflectometer
HMA	Hot-mix asphalt
HVS	Heavy Vehicle Simulator
IFI	International Friction Index
LMLC	Laboratory-mixed, laboratory-compacted
LWD	Light Weight Deflectometer
MDD	Multi-Depth Deflectometer
MPD	Mean profile depth
PIARC	International Association of Road Congresses
PPRC	Partnered Pavement Research Center
RHMA-G	Gap-graded rubberized hot-mix asphalt
RSD	Road Surface Deflectometer
SN	Skid number
SPE	Strategic Plan Element
TSR	Tensile strength retained
UCPRC	University of California Pavement Research Center
WMA	Warm-mix asphalt

LIST OF TEST METHODS AND SPECIFICATIONS

AASHTO M-320	Standard Specification for Performance Graded Asphalt Binder
AASHTO T-166	Bulk Specific Gravity of Compacted Asphalt Mixtures
AASHTO T-209	Theoretical Maximum Specific Gravity and Density of Bituminous Paving Mixtures
AASHTO T-245	Standard Method of Test for Resistance to Plastic Flow of Bituminous Mixtures Using Marshall Apparatus
AASHTO T-275	Standard Method of Test for Bulk Specific Gravity of Compacted Bituminous Mixtures Using Paraffin-Coated Specimens
AASHTO T-308	Standard Method of Test for Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method
AASHTO T-320	Standard Method of Test for Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures using the Superpave Shear Tester
AASHTO T-321	Flexural Controlled-Deformation Fatigue Test
AASHTO T-324	Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Hot-Mix Asphalt (HMA)
ASTM E 274-97	Standard Test Method for Skid Resistance of Paved Surfaces Using a Full-Scale Tire
ASTM E 1845-96	Standard Test Practice for Calculating Pavement Macrotexture Mean Profile Depth
ASTM E 1911-02	Standard Test Method for Measuring Paved Surface Frictional Properties Using the Dynamic Friction Tester
ASTM E 1960-03	Standard Practice for Calculating International Friction Index of a Pavement Surface
ASTM E 2157-01	Standard Test Method for Measuring Pavement Macrotexture Properties Using the Circular Track Meter
CT 342	Method of Test for Surface Skid Resistance with the California Portable Skid Tester
CT 366	Method of Test for Stabilometer Value
CT 371	Method of Test for Resistance of Compacted Bituminous Mixture to Moisture Induced Damage

CONVERSION FACTORS

SI* (MODERN METRIC) CONVERSION FACTORS				
Symbol	**Convert From**	**Convert To**	**Symbol**	**Conversion**
LENGTH				
mm	millimeters	inches	in	mm x 0.039
m	meters	feet	ft	m x 3.28
km	kilometers	mile	mile	km x 1.609
AREA				
mm^2	square millimeters	square inches	in^2	mm^2 x 0.0016
m^2	square meters	square feet	ft^2	m^2 x 10.764
VOLUME				
m^3	cubic meters	cubic feet	ft^3	m^3 x 35.314
kg/m^3	kilograms/cubic meter	pounds/cubic feet	lb/ft^3	kg/m^3 x 0.062
L	liters	gallons	gal	L x 0.264
L/m^2	liters/square meter	gallons/square yard	gal/yd^2	L/m^2 x 0.221
MASS				
kg	kilograms	pounds	lb	kg x 2.202
TEMPERATURE (exact degrees)				
C	Celsius	Fahrenheit	F	°C x 1.8 + 32
FORCE and PRESSURE or STRESS				
N	newtons	poundforce	lbf	N x 0.225
kPa	kilopascals	poundforce/square inch	lbf/in^2	kPa x 0.145
*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)				

1. INTRODUCTION

1.1 Background

Warm-mix asphalt is a relatively new technology. It has been developed in response to needs for reduced energy consumption and stack emissions during the production of asphalt concrete, lower placement temperatures, improved workability, and better working conditions for plant and paving crews. Studies in the United States and Europe indicate that significant reductions in production and placement temperatures are possible *(1,2)*.

Research initiatives on warm-mix asphalt are currently being conducted in a number of states, as well as by the Federal Highway Administration and the National Center for Asphalt Technology. Accelerated pavement testing experiments are being carried out on warm-mix asphalt in Ohio and Alabama.

The California Department of Transportation (Caltrans) has expressed interest in warm-mix asphalt with a view to reducing stack emissions at plants, to allow longer haul distances between asphalt plants and construction projects, to improve construction quality (especially during nighttime closures), and to extend the annual period for paving. However, the use of warm-mix asphalt technology requires the addition of an additive into the mix, and/or changes in production and construction procedures, specifically related to temperature, which could influence the short- and long-term performance of the pavement. Therefore, research is required to address a range of concerns related to these changes before statewide implementation of the technology is approved.

1.2 Project Objectives

The research presented in this report is part of Partnered Pavement Research Center Strategic Plan Element 4.18 (PPRC SPE 4.18), titled "Warm-Mix Asphalt Study," undertaken for Caltrans by the University of California Pavement Research Center (UCPRC). The objective of this project is to determine whether the use of additives intended to reduce the production and construction temperatures of asphalt concrete influence mix production processes, construction procedures, and the short-, medium-, and/or long-term performance of hot-mix asphalt. The potential benefits of using the additives will also be quantified. This is to be achieved through the following tasks:

- Develop a detailed work plan *(3)* for Heavy Vehicle Simulator (HVS) and laboratory testing *(Completed in September 2007).*

- Construct a test track (subgrade preparation, aggregate base-course, tack coat, and asphalt wearing course) at the Graniterock A.R. Wilson quarry near Aromas, California, with four sections as follows *(Completed in September 2007)*:
    1. Conventional dense-graded asphalt concrete (DGAC) mix. This will serve as the control section.
    2. DGAC warm-mix asphalt with *Advera WMA*® additive (referred to as Advera in this report).
    3. DGAC warm-mix asphalt with *Evotherm DAT*™ additive (referred to as Evotherm in this report).
    4. DGAC warm-mix asphalt with *Sasobit*® additive (referred to as Sasobit in the report).
- Identify and demarcate three HVS test sections on each section *(Completed in September 2007).*
- Test each section with the HVS in separate phases, with later phases dependent on the outcome of earlier phases and laboratory tests *(Phase 1 completed in April 2008).*
- Carry out a series of laboratory tests to assess rutting and fatigue behavior *(Phase 1 completed in August 2008).*
- Prepare a series of reports describing the research.
- Prepare recommendations for implementation.

If agreed upon by the stakeholders (Caltrans, Graniterock, warm-mix technology suppliers), the sequence listed above or a subset of the sequence will be repeated for gap-graded rubberized asphalt concrete (RHMA-G), and again for open-graded mixes.

Pilot studies with the technology on in-service pavements will also be supported as part of the study.

1.3 Overall Project Organization

This UCPRC project has been planned as a comprehensive study to be carried out in a series of phases, with later phases dependent on the results of the initial phase. The planned testing phases include *(3)*:

Phase 1 compares early rutting potential at elevated temperatures (pavement temperature of 50°C at 50 mm [122°F at 2.0 in]). HVS trafficking would begin approximately 30 days after construction. Cores and beams sawn from the sections immediately after construction would be subjected to shear, fatigue, and moisture sensitivity testing in the laboratory. If the warm-mix asphalt concrete mixes perform differently to the conventional mixes, moisture sensitivity, additional rutting, and fatigue testing with the HVS would be considered (Phases 2, 3 and 4).

- Depending on the outcome of laboratory testing for moisture sensitivity, a testing phase, if deemed necessary, would assess general performance under dry and wet conditions with special emphasis on moisture sensitivity.
- Depending on the outcome of laboratory testing for rutting, a testing phase, if deemed necessary, would assess rutting performance on artificially aged test sections at elevated temperatures (50°C at 50 mm [122°F at 2.0 in.]). The actual process used to artificially age the sections has not been finalized, but it would probably follow a protocol developed by the Florida Department of Transport Accelerated Pavement Testing program, which uses a combination of infrared and ultraviolet radiation.
- Depending on the outcome of the laboratory study for fatigue, a testing phase, if deemed necessary, would assess fatigue performance at low temperatures (15°C at 50 mm [59°F at 2.0 in.]).
- Depending on the outcome of the above testing phases and if agreed upon by the stakeholders (Caltrans, Graniterock, warm-mix technology suppliers), the sequence listed above or a subset of the sequence would be repeated for gap-graded rubberized asphalt concrete (RHMA-G), and again for open-graded mixes.

This test plan is designed to evaluate short-, medium-, and long-term performance of the mixes.

- Short-term performance is defined as failure by rutting of the asphalt-bound materials.
- Medium-term performance is defined as failure caused by moisture and/or construction-related issues.
- Long-term performance is defined as failure from fatigue cracking, reflective cracking, or rutting of the asphalt-bound and/or unbound pavement layers.

The questions that will be answered during the evaluation include *(3)*:

- What is the approximate comparative energy usage during mix preparation? This will be determined from the asphalt plant records/observations.
- Can satisfactory density be achieved at lower temperatures? This will be established from construction monitoring and subsequent laboratory tests.
- What is the optimal temperature range for achieving compaction requirements? This will be established from construction monitoring and subsequent laboratory tests.
- What are the cost implications? These will be determined with a basic cost analysis.
- Does the use of the additive influence rutting performance of the mix? This will be determined from Phase 1 HVS and laboratory tests.

- Is the treated mix more susceptible to moisture sensitivity given that the aggregate is heated to lower temperatures? This will be determined from Phase 1 laboratory tests and possible additional laboratory and HVS testing.
- Does the use of the additive influence fatigue performance? This will be determined from Phase 1 laboratory tests and potential additional laboratory and HVS testing.
- Does the use of the additive influence the performance of the mix in any other way? This will be determined from HVS and laboratory tests (all phases).
- If the experiment is extended to rubberized and open-graded mixes, are the benefits of using the additives in these mixes the same as for conventional mixes?

1.3.1 Deliverables

Deliverables from the study will include:

- A detailed work plan for the entire study;
- A report detailing construction, first level-data analysis of the Phase 1 HVS testing, first-level data analysis of the Phase 1 laboratory testing, and preliminary recommendations (this report);
- Reports detailing the first-level data analyses of subsequent HVS and laboratory testing phases;
- A detailed 2nd level analysis report for the entire study; and
- A summary report for the entire study.

A series of conference and journal papers documenting various components of the study will also be prepared.

1.4 Structure and Content of this Report

This report presents an overview of the work carried out in Phase 1 to begin meeting the objectives of the study, and is organized as follows:

- Chapter 2 summarizes the HVS test track location, design, and construction.
- Chapter 3 details the HVS test section layout and HVS test criteria.
- Chapter 4 provides a summary of the Phase 1 HVS test data collected from each test.
- Chapter 5 discusses the Phase 1 laboratory testing on field-mixed, field-compacted (FMFC) specimens sampled from the test track.
- Chapter 6 provides conclusions and preliminary recommendations.

1.5 Measurement Units

Although Caltrans has recently returned to the use of U.S. standard measurement units, metric units have always been used by the UCPRC in the design and layout of HVS test tracks, and for laboratory and field measurements and data storage. In this report, metric and English units (provided in parentheses after the metric units) are provided in general discussion. In keeping with convention, only metric units are used in HVS and laboratory data analyses and reporting. A conversion table is provided on Page xxi at the beginning of this report.

1.6 Terminology

The term “asphalt concrete” is used in this report as a general descriptor for the surfacing on the test track. The terms “hot-mix asphalt (HMA)” and “warm-mix asphalt (WMA)” are used as descriptors to differentiate between the two technologies discussed in this study.

2. TEST TRACK LOCATION, DESIGN, AND CONSTRUCTION

2.1 Experiment Location

The experiment is located on a service road at the Graniterock Company's A.R. Wilson Quarry near Aromas, California. Images of the site are shown in Figure 2.1 through Figure 2.4.

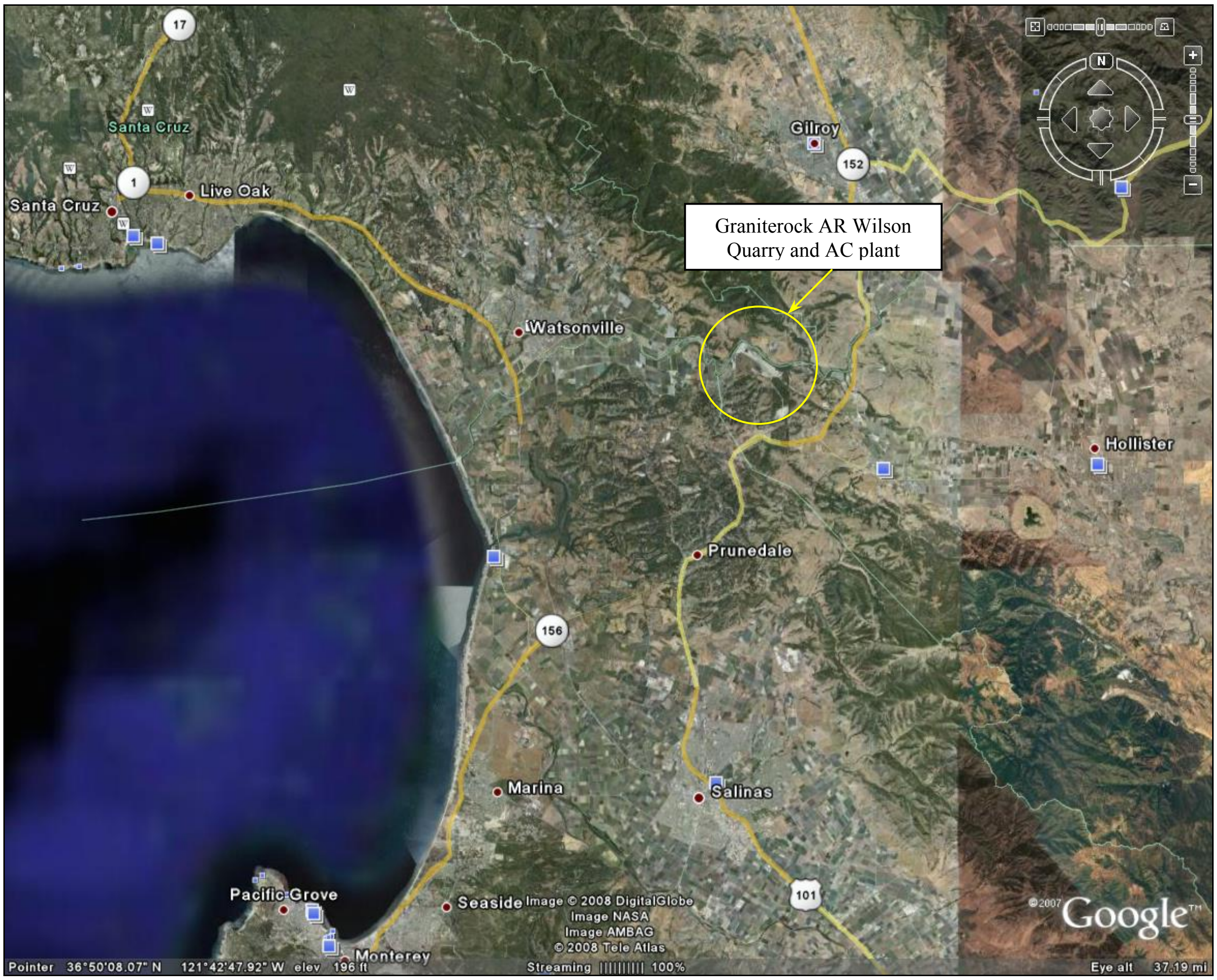


Figure 2.1: General location of test track site.

Figure 2.2: Location of the test track site at the A.R. Wilson Quarry.

Figure 2.3: Site layout.

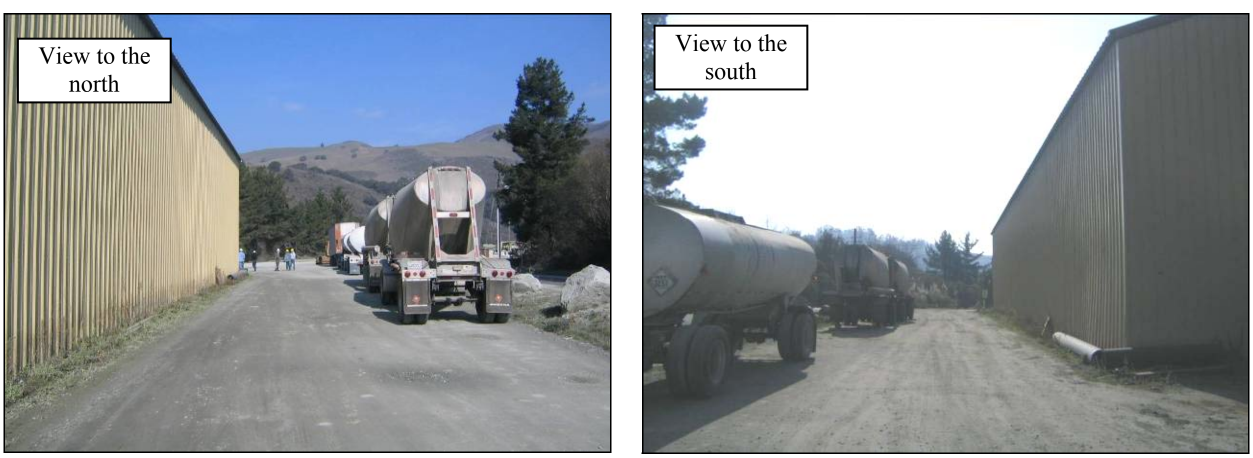


Figure 2.4: Site prior to construction.

2.2 Pavement Design

2.2.1 Layer Thickness

Dynamic Cone Penetrometer (DCP) tests were performed over the length and width of the proposed test section location prior to construction to obtain an indication of the subgrade thickness and strength. Results of the centerline measurements are summarized in Table 2.1. The results indicate an irregular thickness of imported material and overburden over bedrock. DCP penetration to 800 mm (32 in.) was achieved at the southern end of the section, indicating a relatively thick cover over the bedrock. This decreased comparatively uniformly northwards along the length of the section, with a penetration of only 200 mm at the northern end of the section. The DCP-determined strength of the upper layer of material was similar at the various points tested along the length of the section.

Table 2.1: Summary of Centerline DCP Survey

Test Location[1] (m)	Penetration Depth (mm)	Penetration Rate in Top 250 mm (mm/blow)
10	800	2.5
20	680	2.5
30	590	2.7
40	490	2.6
50	380	2.4
60	300	2.4
70	240	2.3
80	200	2.4

[1] Measured from southern end of section.

A sensitivity analysis of potential pavement designs using layer elastic theory models was carried out using the DCP results obtained during the site investigation and estimates, based on previous experience, of the moduli of an aggregate base-course and asphalt concrete surfacing. Components of the sensitivity analysis included the following 24 cells:

- Three asphalt concrete thicknesses (100 mm, 125 mm, and 150 mm)
- Three asphalt concrete moduli (600 MPa, 1,000 MPa, and 3,000 MPa)
- Two base-course thicknesses (300 mm and 450 mm)
- Two base-course moduli (150 MPa and 300 MPa)
- One subbase (existing layer, 250 mm with modulus of 400 MPa)
- One subgrade (existing bedrock with modulus of >3,000 MPa).

A test pavement design was selected to maximize the information that would be collected about the performance of warm-mix asphalt, taking into consideration that a very strong pavement would lengthen the testing time before results (and an understanding of the behavior) could be obtained, while a very weak pavement could fail before any useful data was collected. The pavement design shown in Figure 2.5 was considered appropriate for the study.

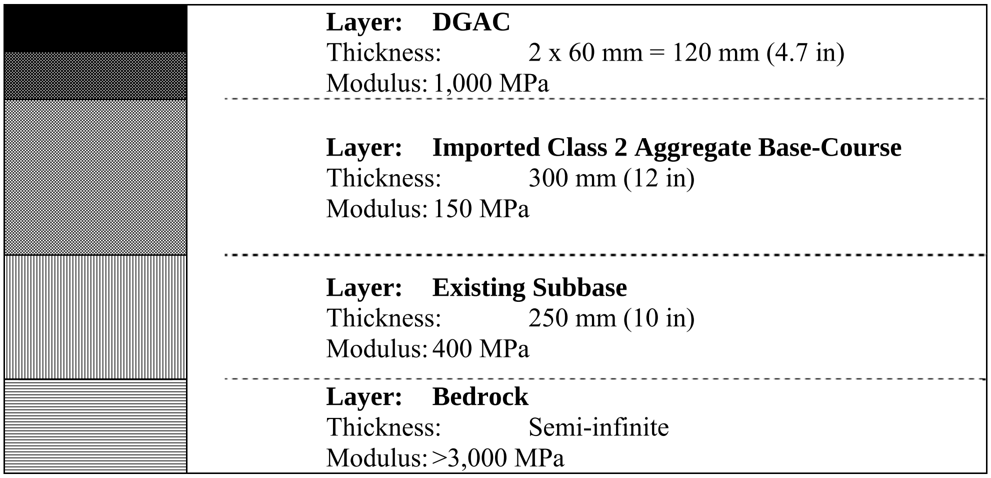


Figure 2.5: Pavement structure for warm-mix asphalt test sections.

2.2.2 Mix Design

A standard Graniterock Company mix design that meets specifications for "Type-A Asphalt Concrete 19 mm Coarse requirements" (similar to the example shown in Appendix A) was used in this study. This mix design differs slightly from the example mix designs provided by Caltrans (example also shown in Appendix A) that were included in the study work plan *(3)*. The Graniterock mix design has been extensively used on projects in the vicinity of the asphalt plant. Although these mix designs list PG 64-10 binder, the Valero Asphalt Plant in Benicia, California, from which the binder was sourced, generally only supplies PG 64-16. This binder, however, also satisfies the requirements for the PG 64-10 grading. The Hveem-type mix design was not adjusted for accommodation of the warm-mix additives. Key parameters for the mix design are summarized in Table 2.2.

Aggregates

Aggregates for the base and asphalt concrete were sourced from the Graniterock Company's nearby A.R Wilson Quarry. This granitic aggregate is classified as a hornblende gabbro of the Cretaceous Age and is composed of feldspar, quartz, small quantities of mica or hornblende, minor accessory minerals and lesser amounts of dark ferromagnesium materials. It is quarried from a narrowly exposed mass of plutonic rock close to the test track. Key aggregate parameters are provided in Table 2.2.

Warm-Mix Additive Application Rates

The warm-mix additive application rates were determined by the additive suppliers and were as follows:

- Advera: 0.25 percent by mass of mix (equates to 4.8 percent by mass of binder)
- Evotherm: 0.5 percent by mass of binder
- Sasobit: 1.5 percent by mass of binder

Table 2.2: Key Mix Design Parameters

Parameter		Wearing Course		Base	
		Target	Range	Target	Range
Grading:	1"	100	-	100	100
	3/4"	96	91-100	93	90-100
	1/2"	84	-	-	-
	3/8"	72	66-78	-	-
	#4	49	42-56	51	35-60
	#8	36	31-41	-	-
	#16	26	-	-	-
	#30	18	14-22	17	10-30
	#50	11	-	-	-
	#100	7	-	-	-
	#200	4	2-6	6	2-9
Asphalt concrete binder grade		PG 64-10[1]	-	-	-
Recommended bitumen content (% by mass of aggregate)		5.2	5.1-5.4	-	-
Hveem Stability at recommended bitumen content		45	-	-	-
Air-void content (%)		4.5	-	-	-
Crushed particles (%)		100	-	-	-
Sand equivalent (%)		72	-	≥ 50	-
Los Angeles Abrasion at 100 repetitions (%)		9	-	-	-
Los Angeles Abrasion at 500 repetitions (%)		30	-	-	-
Plasticity Index		-	-	Non-plastic	-
R-Value		-	-	≥ 80	-
Course aggregate durability		-	-	≥ 65	-
Fine aggregate durability		-	-	≥ 50	-
Optimum moisture content (%)		-	-	6.5	-
Maximum dry density (lb/ft^3)		-	-	145	-

[1] PG 64-16 binder supplied as PG64-10 by binder supplier

2.2.3 Production and Construction Temperatures

Based on discussions between Graniterock Company and the warm-mix additive suppliers, the mix production temperatures were set at 155°C (310°F) for the Control mix and 120°C (250°F) for the mixes with additives. Target breakdown compaction temperatures were set at 145°C to 155°C (284°F to 310°F) for the Control mix and 110°C to 120°C (230°F to 250°F) for the mixes with additives.

2.3 Test Track Layout

The test track was laid out as shown in Figure 2.6: Test track layout.. All test track measurements, locations, and chainage discussed in this report are based on this layout.

2.4 Test Track Preparation

A K-Rail concrete barrier (referred to as a New Jersey Barrier in some states) was installed along both sides of the demarcated test track to contain the base-course material and to allow for adequate compaction of the edges of the test track, thereby providing adequate support for the HVS. The existing surface was bladed to provide a uniform surface for construction of the base-course (Figure 2.7).

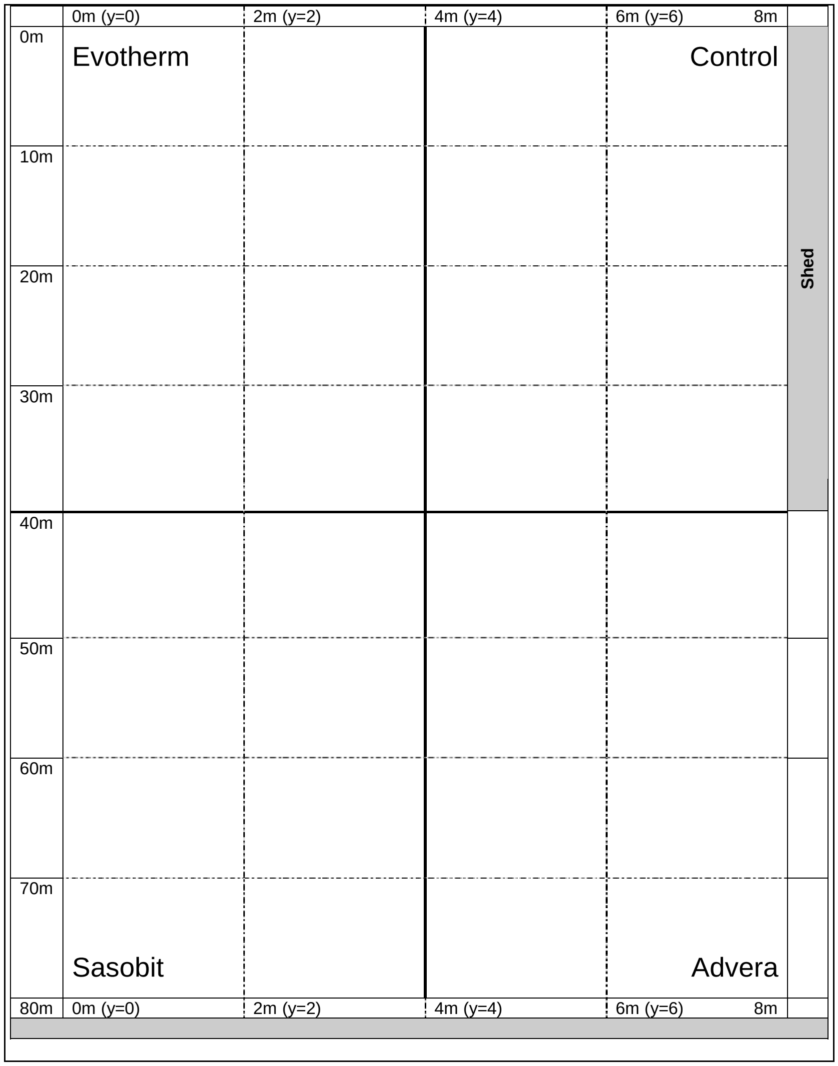


Figure 2.6: Test track layout.

Figure 2.7: K-rail placement and subgrade/subbase preparation.

2.5 Base-Course Construction

2.5.1 Equipment

The following equipment was used during the construction of the base-course:

- Caterpillar 140H grader
- Ingersoll Rand SD100-D steel-wheel vibrating roller
- Sakai SW320 steel-wheel vibrating roller
- 15,000 L water tanker
- Dump trucks with trailers (bottom dump)
- John Deere 210 LE skip loader

2.5.2 Construction

The test track base-course was constructed on August 17, 2007. Crushed base-course material (granitic) meeting Caltrans Class-2 aggregate base-course specifications was imported from a nearby quarry stockpile with a fleet of bottom-dump trucks and trailers. Material was dumped in windrows, spread with the grader, watered, and compacted (steel-wheel roller with vibration) in a series of lifts until the desired 300 mm (12 in.) thickness was achieved (Figure 2.8). A total of 23 loads were dumped. Some early overwatering was observed, which influenced compaction procedures (Figure 2.9). Thereafter, the water tanker was more strictly controlled to prevent further occurrences. Dry material was placed over the affected areas to absorb excess moisture.

Final levels were checked with a rod-and-level survey to ensure that a consistent base-course thickness had been achieved.

Figure 2.8: Base-course construction.

Figure 2.8: Base-course construction (*continued*).

Figure 2.9: Overwatering during base-course construction.

2.5.3 Instrumentation

Instrumentation in the base-course was limited to four moisture sensors (ESI Gro-Point^{TM}) for monitoring its moisture contents during the experiment. Given the proximity of the bedrock, Multi-depth Deflectometers (MDD) were not considered. Two transverse trenches were excavated into the base-course at 20 m and 60 m (66 ft and 197 ft) respectively along the test track to accommodate the four moisture sensors (Figure 2.10). The excavated material was replaced after installation and compacted to the level of the finished base-course surface.

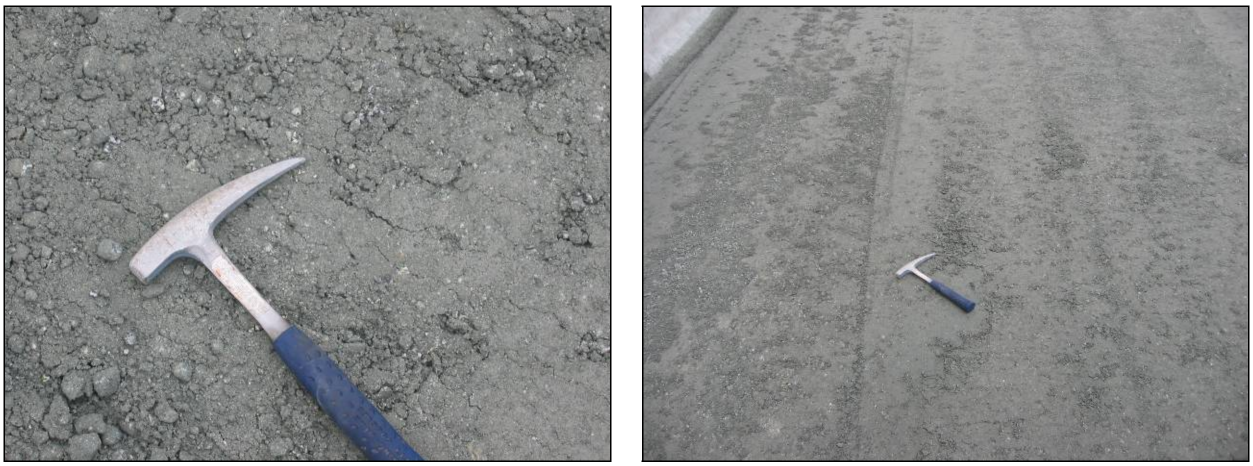
Figure 2.10: Installation of moisture sensors.

2.5.4 Construction Quality Control

The base-course was inspected on August 22, 2007 after a seven-day dry back period. The surface was generally acceptable (Figure 2.11), but some isolated areas of loose material, segregated material, shearing, and delamination were observed (Figure 2.12). Some settlement was also noted in the immediate proximity of the backfilled moisture sensor trenches.

Figure 2.11: Completed base-course showing tightly bound surface.

Loose surface/raveling

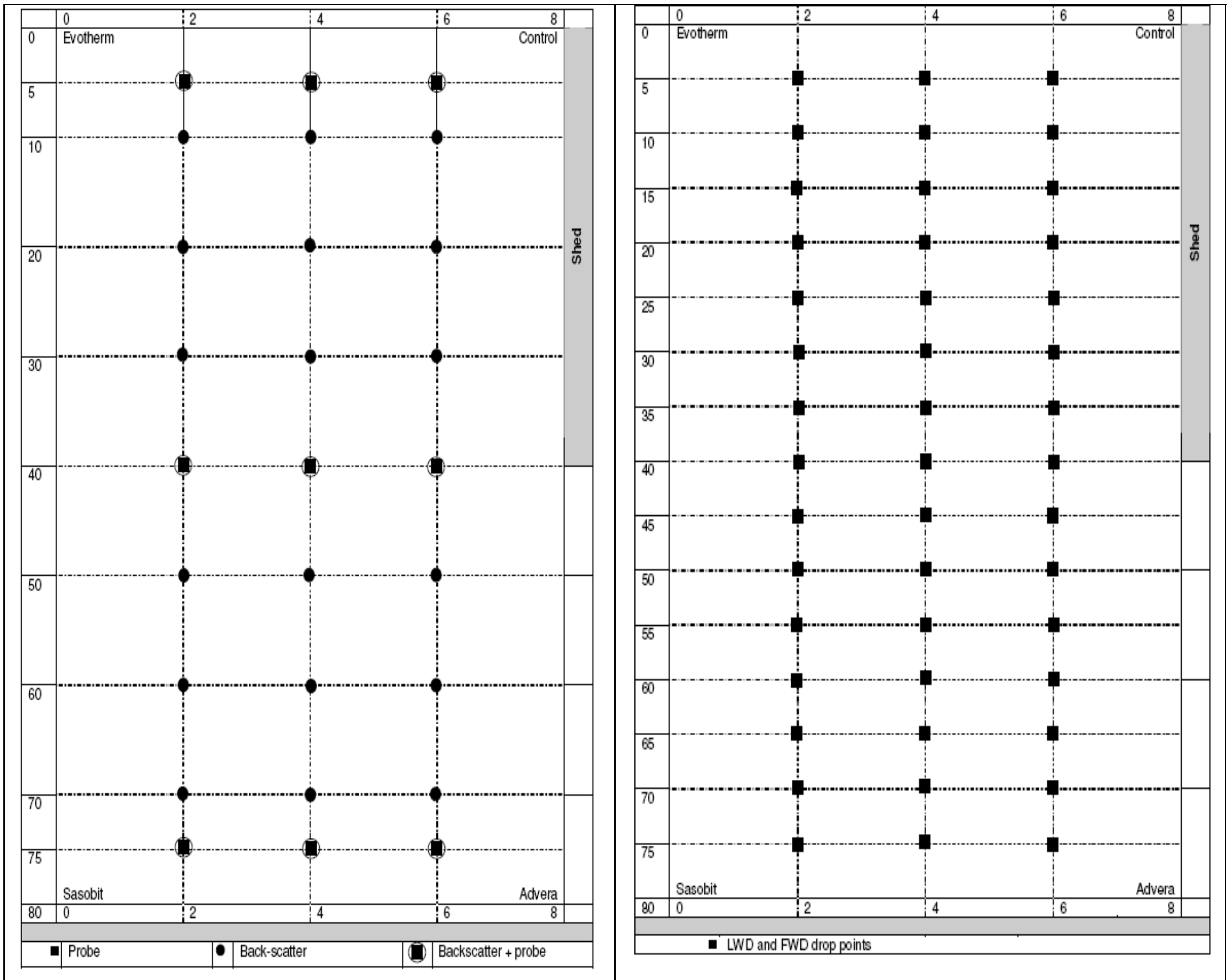


Segregation/raveling

Figure 2.12: Isolated areas of distress on the base-course.

These localized problems were corrected by spraying the surface with water and then rolling with a smooth drum roller (no vibration) to seal it.

After final rolling, density and deflection measurements were taken on the prepared surface to assess compaction levels, uniformity, and structural integrity. Density was determined with a nuclear density gauge, while deflection was measured with a Light Weight Deflectometer (LWD) and Falling Weight Deflectometer (FWD). The plans shown in Figure 2.13 were followed for these measurements.



Base-course density measurement plan	Base-course deflection measurement plan

Figure 2.13: Base-course density and deflection measurement plan.

Nuclear Density Gauge

The dry density and moisture content of the base-course, determined from nuclear density gauge measurements, are summarized in Table 2.3 and Table 2.4 and in Figure 2.14 through Figure 2.16. Measurements are the average of two readings, the first taken with the gauge positioned longitudinally, and the second with the gauge positioned transversally (see figure in Table 2.3). Surface measurements were determined in the backscatter mode. The maximum dry density of the material was approximately 2,380 kg/m^3 (145 lb/ft^3) and the optimum moisture content was approximately 6.5 percent.

Table 2.3: Summary of Base-Course Density Measurements after 7-day Dry Back

Location	Depth (mm)	Dry Density (kg/m^3)* Along Test Track								
		5 m	10 m	20 m	30 m	40 m	50 m	60 m	70 m	75 m
y=2	Surface	2,318	2,273	2,318	2,349	2,428	2,386	2,192	2,369	2,155
	50	2,325	-	-	-	2,377	-	-	-	2,260
	100	2,310	-	-	-	2,376	-	-	-	2,268
	150	2,313	-	-	-	2,268	-	-	-	2,288
	200	2,311	-	-	-	2,443	-	-	-	2,336
y=4	Surface	2,217	2,308	2,232	2,420	2,371	2,322	2,294	2,390	2,276
	50	2,289	-	-	-	2,294	-	-	-	2,300
	100	2,288	--	-	-	2,375	-	-	-	2,299
	150	2,303	--	-	-	2,354	-	-	-	2,345
	200	2,291	--	-	-	2,373	-	-	--	2,378
y=6	Surface	2,346	2,262	2,165	2,371	2,289	2,225	2,174	2,295	2,289
	50	2,287	-	-	-	2,289	-	-	-	2,275
	100	2,294	-	-	-	2,323	-	-	-	2,292
	150	2,328	-	-	-	2,396	-	-	-	2,355
	200	2,348	-	-	-	2,383	-	-	-	2,336

* Measurements are an average of two measurements taken from two gauge positions (orientations), A and B, as shown in figure.

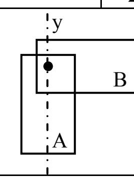


Table 2.4: Summary of Base-Course Moisture Content Measurements after 7-day Dry Back

Location	Depth (mm)	Moisture Content (%)		
		5 m	40 m	75 m
y=2	50	4.2	4.2	4.6
	100	4.4	4.3	4.7
	150	4.2	4.1	4.5
	200	4.4	4.5	4.3
y=4	50	4.9	5.7	6.3
	100	5.2	5.4	6.2
	150	5.0	5.5	6.1
	200	4.8	5.4	6.1
y=6	50	4.3	4.3	4.3
	100	4.2	4.1	4.2
	150	4.4	4.1	4.0
	200	4.2	3.9	4.3

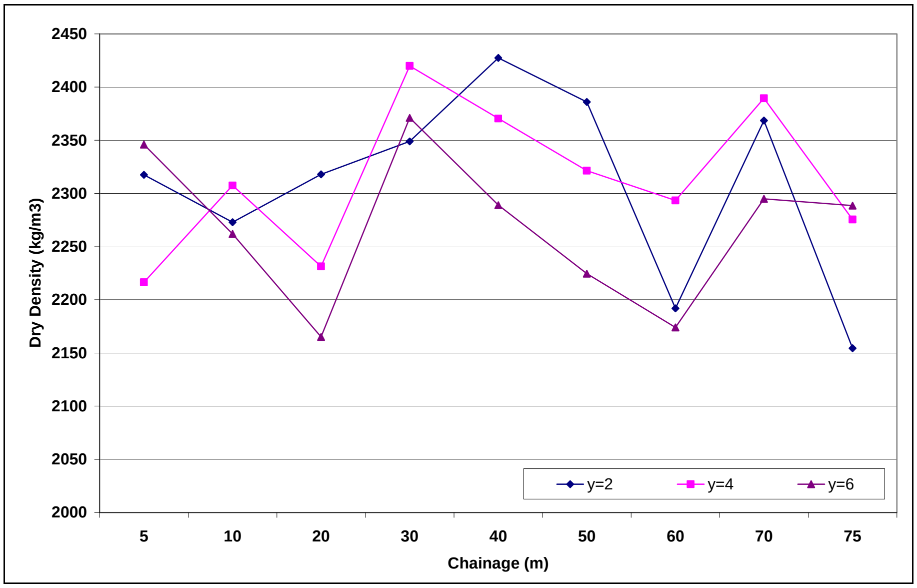


Figure 2.14: Summary of average dry density (backscatter).

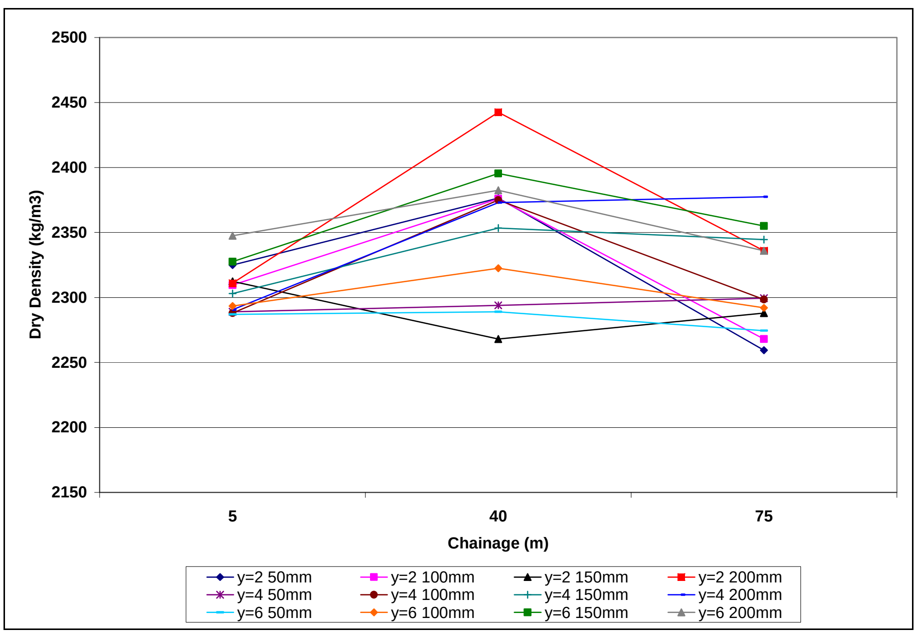


Figure 2.15: Summary of average dry density at various depths (probe).

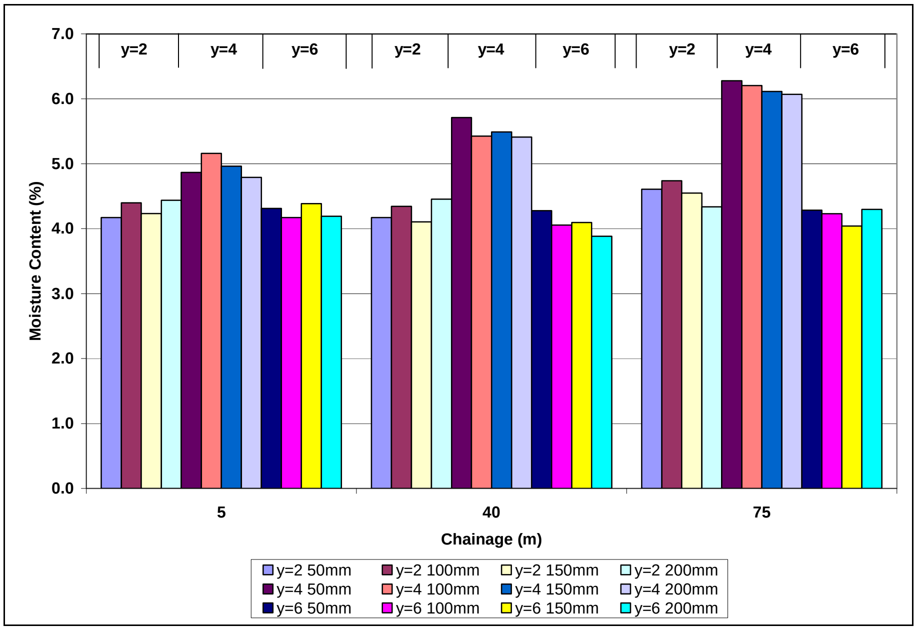


Figure 2.16: Summary of moisture content at different depths (probe).

The following observations were made:

- Dry density measured on the surface showed some variation along the length of the section. The average density measured was 2,297 kg/m^3 or 97 percent of the maximum dry density relative to California Test Method 216 (standard deviation of 77 kg/m^3 [143 lb/ft^3 and 5 lb/ft^3]). The Caltrans specification requires 95 percent relative density measured as wet density.
- The dry density increased with increasing depth. This was attributed to the construction method followed (compaction of multiple thin lifts). The average densities for the four depths were:
  - 50 mm (2 in.): 2,299 kg/m^3 (standard deviation of 34 kg/m^3 [143.5 lb/ft^3, SD 2.1 lb/ft^3])
  - 100 mm (4 in.): 2,314 kg/m^3 (standard deviation of 38 kg/m^3 [144.5 lb/ft^3, SD 2.4 lb/ft^3])
  - 150 mm (6 in.): 2,328 kg/m^3 (standard deviation of 39 kg/m^3 [145.3 lb/ft^3, SD 2.4 lb/ft^3])
  - 200 mm (8 in.): 2,355 kg/m^3 (standard deviation of 45 kg/m^3 [147.0 lb/ft^3, SD 2.8 lb/ft^3])
- Some variation in density was evident along the length and width of the section.
- The moisture content measured at three locations immediately after construction (sampled from the trenches excavated for the moisture sensors) varied between 7.0 percent and 10.8 percent, with moisture content increasing with increasing depth. Some areas were considerably above the optimum moisture content of the material, which was attributed to the overwatering in the early stages of construction.
- Considerable drying occurred in the seven-day period between construction and measurements with the nuclear gauge. The average gauge-determined moisture content was 4.7 percent (standard deviation of 0.7 percent). The lowest recording was 3.9 percent and the highest was 6.3 percent.

Light Weight Deflectometer

Measurements were taken at 1.0 m intervals (start point at 5.0 m and end point at 75 m in Figure 2.13) along the centerline of each section (i.e., y = 2 m and y = 6.0 m in Figure 2.13) and at 5.0 m intervals along the centerline of the test track (i.e., y = 4.0 m). Only one set of measurements was taken as the base material was not expected to be temperature sensitive. Average results of the 6.0 kN load drop are summarized in Table 2.5 and Figure 2.17 and Figure 2.18. There was some difference in the deflections measured in the base-course on the four sections, as well as some variation along the length of each section. This was attributed to overwatering during construction, which probably resulted in inconsistent drying of the base-course material. Deflections on the Control and Evotherm sections were higher than those recorded on the Advera and Sasobit sections. This was attributed to slower drying of the former two sections due to shading by the shed for a portion of each day. Deflections in the subgrade were very small and consistent, as expected, due to the proximity of the bedrock.

Table 2.5: Summary of Base-Course LWD Measurements

Section	Deflection @ D1[1] (micron)		Deflection @ D2 (micron)		Deflection @ D3 (micron)	
	AM	PM	AM	PM	AM	PM
Control	184.0	-	19.5	-	9.6	-
Advera	71.6	-	12.0	-	6.8	-
Evotherm	135.7	-	15.4	-	9.7	-
Sasobit	91.7	-	18.1	-	9.7	-
Average	120.7	-	16.2	-	8.9	-
Std deviation (mm)	50.0	-	3.3	-	1.4	-
CoV[2] (%)	41.4	-	20.4	-	16.0	-

[1] Geophone D1, offset 0mm
[2] CoV: Coefficient of variance
Geophone D2, offset 300mm
Geophone D3, offset 600mm

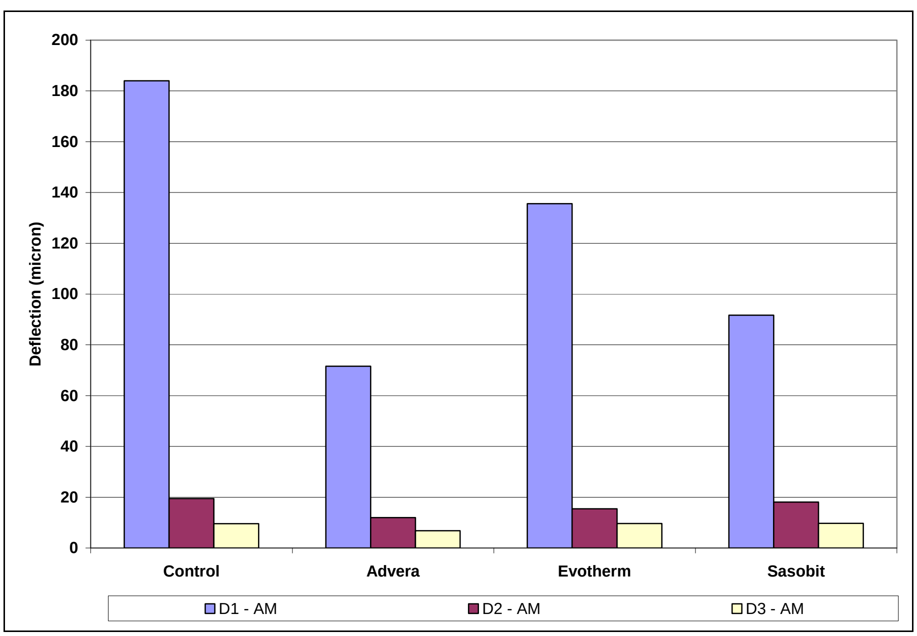


Figure 2.17: Summary of average LWD deflection by section.

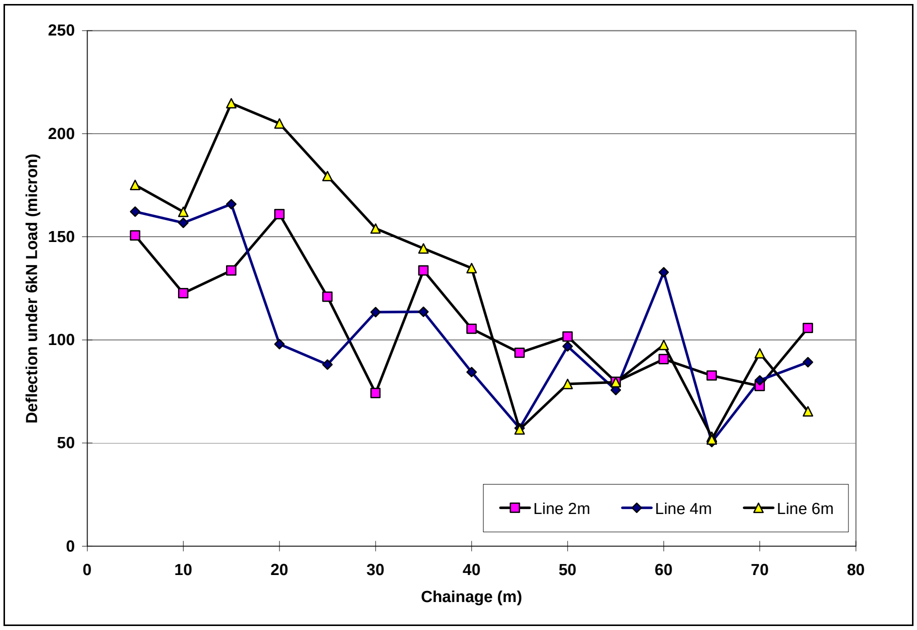


Figure 2.18: Summary of LWD base-course deflection measurements (D1 geophone).

<u>Falling Weight Deflectometer</u>

FWD measurements were taken at the same positions as those taken with the LWD. Only one set of measurements was taken. Average results of the second 40 kN load drop are summarized in Table 2.6 and in Figure 2.19 through Figure 2.21. Similar trends to those of the LWD measurements were observed with similar variation along the length of each section. Higher deflections were again noted on the Control and Evotherm sections. Deflections in the out sensors, which are influenced by the subgrade, were also very small and consistent due to the presence of bedrock.

Table 2.6: Summary of FWD Measurements on the Base-Course

Section	Deflection @ D1[1] (mm)		Deflection @ D6[2] (mm)		Deflection @ D3[3] (mm)		Deflection @ D5[4] (mm)	
	AM	PM	AM	PM	AM	PM	AM	PM
Control	0.666	-	0.053	-	0.201	-	0.078	-
Advera	0.552	-	0.056	-	0.152	-	0.075	-
Evotherm	0.390	-	0.043	-	0.101	-	0.055	-
Sasobit	0.479	-	0.061	-	0.167	-	0.087	-
Average	0.522	-	0.053	-	0.155	-	0.074	-
Std deviation (mm)	0.117	-	0.007	-	0.042	-	0.013	-
CoV (%)	22	-	0.140	-	0.268	-	0.181	-
Section	Average Temperatures							
	AM (°C)		PM (°C)		AM (°F)		PM (°F)	
	Air	Surface	Air	Surface	Air	Surface	Air	Surface
Control Advera	16.4	19.8	-	-	61	34	-	-
Evotherm Sasobit	15.2	18.4	-	-	59	65	-	-

[1] Geophone D1, 0 mm offset
[2] Geophone D6, 925 mm offset
[3] Geophone D3, 315 mm offset
[4] Geophone D5, 630 mm offset

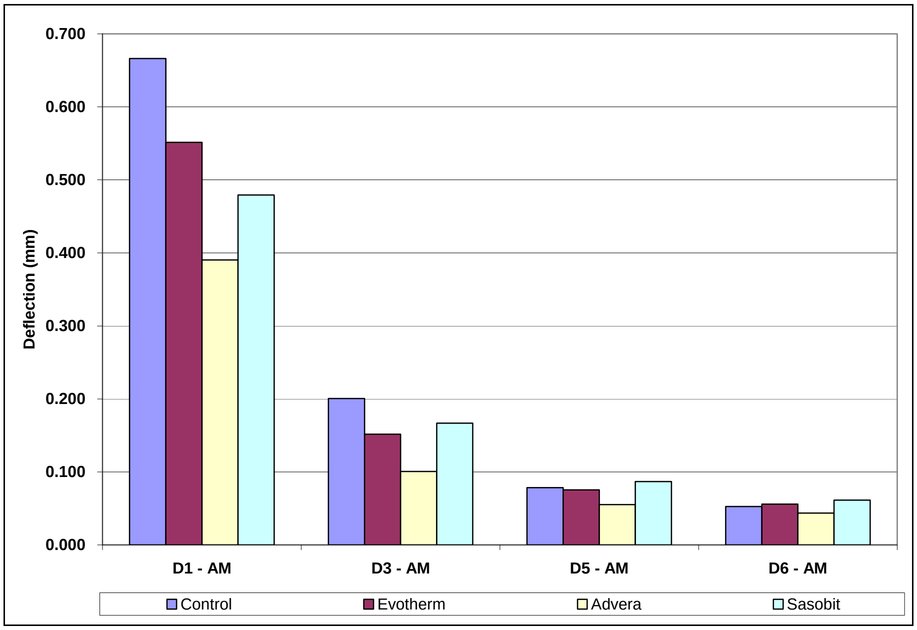


Figure 2.19: Summary of average FWD deflection by section.

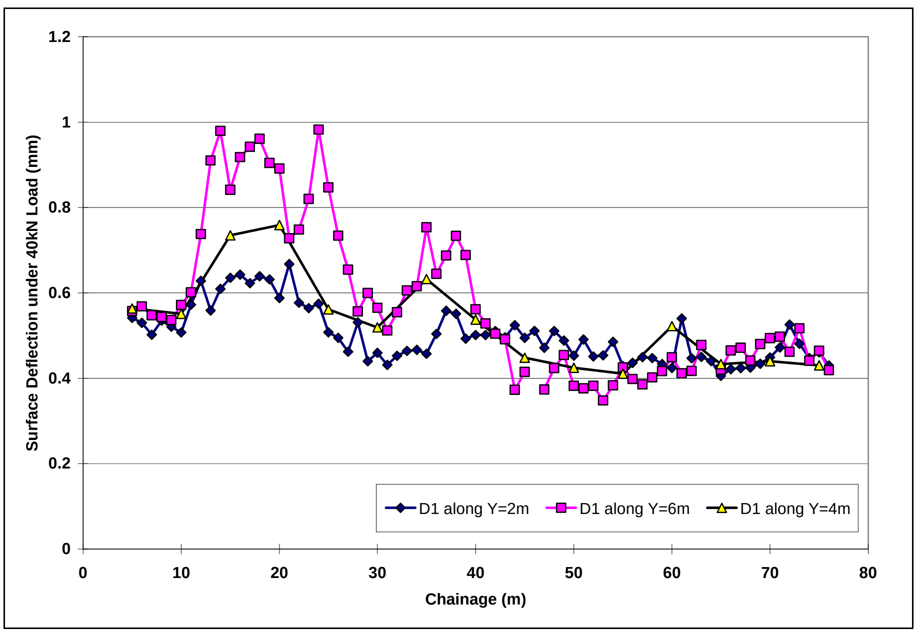


Figure 2.20: Summary of FWD base-course deflection measurements (D1 geophone).

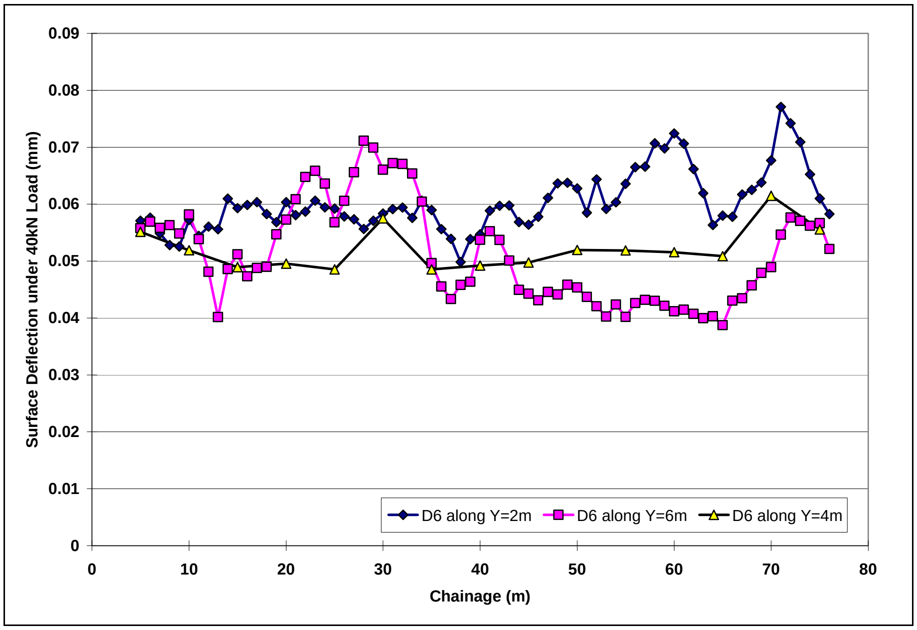


Figure 2.21: Summary of FWD subgrade deflection measurements (D6 geophone).

2.6 Asphalt Concrete Production

Technical representatives from each of the additive suppliers were on site before and during mix production, and worked with Graniterock Company staff to modify the plant and monitor mix production with their additives.

2.6.1 Plant Modifications

Modifications were made to the asphalt binder feedline on the asphalt plant to accommodate the addition of the Advera and Evotherm additives (Figure 2.22). Customized, calibrated additive delivery systems were provided by the two manufacturers (Figure 2.23 and Figure 2.24), who oversaw all necessary installations. It was originally intended that the Sasobit be blended at the refinery and delivered with the binder. However, the refinery could not complete the terminal blend and the additive was instead added to the binder tanker on site prior to mix production (Figure 2.25). The tanker was later connected directly to the asphalt plant feedline. The asphalt binder, sourced from the Valero Asphalt Plant in Benicia, California, was delivered on the day of production.

2.6.2 Mix Production

Asphalt production started at 07:40 AM on August 24, 2007. Production began with the Control mix, followed by the Advera, Evotherm, and Sasobit mixes (i.e., alphabetical order). Approximately 150 tonnes of each mix were produced and then stored in insulated silos. The first approximately 20 tonnes of each mix was "wasted" to ensure that a consistent mix was used on the test track. This material was used to pave a parking area close to the test track, providing the paving crew with an opportunity to familiarize themselves with each mix.

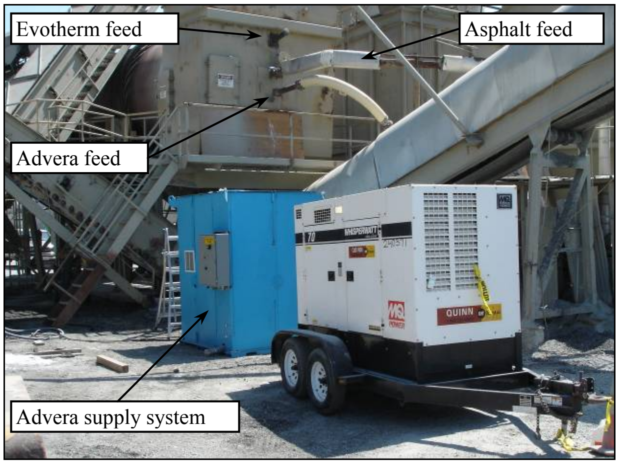


Figure 2.22: Plant modifications for admixtures.

Figure 2.23: Advera supply system.

Figure 2.24: Evotherm supply system.

Figure 2.25: Sasobit mixing.

Initial planning required that production of all four mixes be completed before construction was started. However, problems with the feedline from the tanker with Sasobit binder during the production run with that additive required a halting of mix production to correct the problem, empty the silo, mix Sasobit in a new binder tanker, and then restart mix production. Consequently, paving of the parking areas with the "wasted" material started prior to completion of the second Sasobit mix production run to allow sufficient time for all paving to be completed and to use the discarded initial mix.

Although considered in the work plan, plant emissions were not monitored due to the small volume of each mix produced.

A summary of the mix production observations is provided in Table 2.7. Actual mix production temperatures were at or close to the planned temperatures. The mix rates of the Evotherm and Sasobit were as planned (monitored by additive suppliers). The Advera was added at a slightly lower rate than planned due to a feed-rate problem on their equipment. However, the rate was still within the range usually used for the additive.

Table 2.7: Summary of Mix Production Observations

Mix	Start Time	End Time	Mix Temperature (°C [°F])	Baghouse Temperature (°C [°F])	Production Rate (tonnes/hour)
Control	07:45	08:00	153 (308)	118 (245)	254
Advera	08:20	08:35	120 (248)	118 (245)	268
Evotherm	08:47	09:12	122 (252)	116 (240)	256
Sasobit 1*	09:15	09:26	121 (251)	116 (240)	252
Sasobit 2	12:25	12:45	120 (248)	112 (235)	244
* Sasobit 1 mix rejected due to binder feed problems					

Additive Application Rates (% by mass of binder)		
Mix	**Target**	**Actual**
Control	-	-
Advera	4.8	4.45
Evotherm	0.5	0.5
Sasobit	1.5	1.5

2.6.3 Quality Control

Asphalt Binder

A certificate of compliance was provided by the binder supplier with the delivery. A copy of this certificate is provided in Appendix B.

Performance-grade testing of the asphalt binder was undertaken by the Mobile Asphalt Binder Testing Laboratory (MABTL) Program within the Federal Highway Administration (FHWA) Office of Pavement Technology. Testing followed the AASHTO M-320 Table 1 (M-320) and AASHTO M-320 Table 2 (M320-T2) requirements. The M320-Continuous grading is based on the Table 1 testing requirements. Tests were undertaken on the base binder, on laboratory-blended base binder plus warm-mix additives, and on field-blended base binder plus Sasobit. Field-blended samples of the binder with Advera and Evotherm could not be collected due to the nature of the asphalt plant modifications. Samples of the binder and warm-mix additives were collected at the asphalt plant on the day of production and then shipped to the MABTL in five-liter metal paint can style containers with friction lids. These containers were heated in order to further split the material into one-liter containers. The warm-mix additives were blended in the laboratory using a low shear mixer and heating mantel at the same rates as those used on the day of production. The binder was heated to 138°C for a minimum time to allow the binder to be fluid enough to blend the WMA technology with the base binder in the low shear mixer.

Key results of the binder testing are listed in Table 2.8. Test results were considered by the FHWA as acceptable. The base binder was graded as PG 64-22, slightly better (in terms of low-temperature cracking) than the performance grade of PG 64-16 specified in the work plan and shown on the supplier's certificate of compliance. The addition of Advera and Sasobit changed the performance grading from PG 64-22 to PG 70-22 and increased the critical cracking temperature by approximately 1.0°C, implying both have much better high-temperature rutting performance, but slightly worse (1.0°C and 0.9°C respectively) low-temperature cracking performance than the base binder. The addition of Evotherm did not alter the performance grading of the base binder.

An increase in the high temperature grade PG 64 to PG 70 due to the addition of Sasobit was expected due to the stiffening effect of the wax additive. A change in the high and low temperature grade achieved with the addition of Sasobit is dependant on the specific base binder. The increase in the high temperature grade due to the addition of the Advera was not expected based on the zeolite's material properties and previous FHWA testing experience with Advera modified binders; which typically do not significantly impact the performance grade. As shown in the M320-Continuous data column in Table 2.8, both the Base-plus-Advera and the Base-plus-Evotherm high temperature performance grades were borderline on the 70°C cutoff between a PG 64 and PG 70 designation. The Base-plus-Advera high temperature continuous performance grade exceeded the 70°C limit by 0.2°C while the Base-plus-Evotherm continuous grade was below the cutoff by 0.6°C. The difference in high temperature performance grade is an effect of having the test results for this specific binder closely border this 70°C temperature. This borderline difference in the Advera and Evotherm technologies with respect to the 70°C limiting value is due to various factors including the reheating of the base binder in the laboratory for splitting and blending, the inherent variability in the test procedures, the ageing criteria specified in the test procedures, and the base binder's sensitivity to ageing. An additional Base-plus-Advera sample was tested and graded in the laboratory to confirm the test results. The original test results were confirmed, although one additional re-heating cycle was required which further increased the M320 continuous grade temperatures.

Table 2.8: Summary of Binder Performance-Grade Test Results

Asphalt Binder	M320	M320-T2	M320-Continuous	Critical Crack Temp. (°C)
Base	PG 64-22	PG 64-22	67.0-26.7	-24.0
Base + Advera	PG 70-22	PG 70-22	70.2-26.0	-23.0
Base + Evotherm	PG 64-22	PG 64-22	69.4-26.8	-23.9
Base + Sasobit (lab)	PG 70-22	PG 70-22	72.8-26.0	-23.1
Base + Sasobit (field)	PG 70-22	PG 70-22	71.7-24.2	-22.0

Asphalt Mix

The actual mix design properties were not assessed by Caltrans since numerous tests have been undertaken in the past on the mix design used.

Quality control of the mixes produced for the test track was undertaken by Graniterock Company on mix sampled from the trucks at the silos. Hveem tests and kneading compaction were not used for this testing because no research or protocols are available for determining a kneading compaction temperature for warm-mix asphalt. Graniterock instead undertook Marshall and Superpave Gyratory compaction and Marshall Stability tests to compare the four mixes. The results are summarized in Table 2.9.

Table 2.9: Quality Control of Mix After Production

Parameter	Target	Range	Control	Advera	Evotherm	Sasobit
Grading						
1"	100	-	100.0	100.0	100.0	100.0
3/4"	96	91-100	96.0	95.8	97.3	96.5
1/2"	84	-	85.2	86.0	88.2	86.2
3/8"	72	66-78	72.8	74.9	75.9	75.1
#4	49	42-56	48.0	51.7	50.2	50.5
#8	36	31-41	36.0	39.9	39.4	38.1
#16	26	-	25.3	28.0	28.2	26.3
#30	18	14-22	17.8	19.2	19.3	17.9
#50	11	-	11.2	11.4	11.3	10.7
#100	7	-	6.4	6.7	6.6	6.1
#200	4	2-6	3.7	4.2	4.1	3.8
AC Binder Content (%)[1]	5.2	5.1 - 5.4	5.29	5.14	5.23	4.48
Max. Specific Gravity[2]	-	-	2.567	2.581	2.596	2.606
Marshall Compaction[3]						
Compaction Temperature (°C)	-	-	139	115	112	124
Blows per face	-	-	75	75	75	75
Bulk Specific Gravity	-	-	2.511	2.474	2.493	2.464
Air-void Content (%)	-	-	2.18	4.15	3.97	5.45
Gyratory Compaction[3]						
Compaction Temperature (°C)	-	-	139	115	112	124
Number of Gyrations	-	-	100	100	100	100
Bulk Specific Gravity	-	-	2.526	2.522	2.528	2.510
Air-void Content (%)	-	-	1.60	2.29	2.62	3.68
Marshall Stability (lbs)[4]	1,800 min	-	4,267	3,030	3,320	3,307
Marshall Flow (0.01 in.)	-	-	11.8	10.8	10.2	12.1
Moisture (before plant) (%)	-	-	0.24	0.41	0.37	0.31
Moisture (after silo) (%)	<1.0	-	0.09	0.25	0.32	0.25

[1] AASHTO T-308 [2] AASHTO T-209 [3] AASHTO T-166 [4] AASHTO T-245

The following observations were made:

- The aggregate gradations of the four mixes were similar, generally met the targets, and were within the required ranges.
- The binder contents of the Control, Advera, and Evotherm mixes were similar and all close to the target. The binder content of the Sasobit mix was 0.72 percent below the target and 0.62 percent below the lowest permissible content. This discrepancy is likely to influence behavior of the mix and will be taken into consideration in performance discussions in Chapter 4. The problem was attributed to the asphalt plant operation and binder feed rate from the tanker during mix production.
- The maximum specific gravities of the four mixes were within a relatively close range, but showed an increase of between 0.010 and 0.015 with each subsequent mix produced.

- The bulk specific gravities of the four mixes, determined from Marshall-compacted specimens, were within a relatively close range (difference of 0.047 between highest and lowest). The Control mix had the highest bulk specific gravity of the four mixes and Sasobit the lowest.
- The air-void contents of the four mixes, determined from Marshall-compacted specimens, were notably different, with the Control mix having a significantly lower air-void content than the mixes with additives. The Control mix had the lowest air-void content (2.18 percent) and the Sasobit mix the highest air-void content (5.45 percent). It is not clear whether this was a testing inconsistency, or a result of the warm-mix production process. This will be assessed in the proposed Phase 2 laboratory testing program *(3)*. (Laboratory mix-design testing procedures are also currently being investigated as part of a National Cooperative Highway Research Project [NCHRP 9-43].)
- The bulk specific gravities of the four mixes, determined from gyratory-compacted specimens, were within a closer range compared to the Marshall-compacted specimens (difference of 0.018 between highest and lowest). The Control, Advera, and Evotherm mixes essentially had the same bulk specific gravity, with the Sasobit mix having a slightly lower value.
- The air-void contents of the four mixes, determined from gyratory-compacted specimens, were also notably different, with the Control mix again having a significantly lower air-void content than the mixes with additives. The Control mix had the lowest air-void content (1.60 percent) and the Sasobit mix the highest (3.68 percent).
- The Marshall stability of the Control mix was significantly higher than the mixes with additives (approximately 1,000 lb higher). However, the stabilities of all the mixes were well above the minimum limit.
- The Marshall flows did not follow similar trends. The Evotherm and Advera mixes had the lowest Marshall flows (10.2 and 10.8 respectively) followed by the Control mix (11.8) and the Sasobit mix (12.1). The Sasobit mix was expected to have the lowest flow, given that it had the lowest binder content.
- There was some variability in the moisture contents of the aggregate just prior to it entering the drum, with the material used in the Control mix having the lowest moisture content (0.24 percent) and that used in the Advera mix the highest moisture content (0.41 percent). The moisture contents of all four aggregate runs prior to entering the drum were still lower than the Caltrans end-of-drum moisture content specification of 1.0 percent *(4)*.
- The moisture contents of the mix samples collected at the silos showed a more interesting trend. The moisture content of the Control mix was just 0.09 percent, considerably lower than those of the mixes with additives, which had moisture contents of 0.25 percent (Advera and Sasobit mixes) and 0.32 percent (Evotherm mix). Although moisture contents in all mixes were well below the

minimum specified limit, the higher moisture content of the modified mixes indicates that potentially less moisture evaporates from the aggregate at the lower production temperatures.

2.7 Asphalt Concrete Placement

Asphalt concrete lay-down and compaction were monitored and documented by UCPRC staff. The proceedings were also observed by Caltrans staff and representatives from Graniterock Company and the additive suppliers.

2.7.1 Placement

Introduction

Construction started with the ramps to the test track, thereby ensuring easier and more level access for the paver and compaction equipment. The first "wasted" tonnage from the Control mix was used for this application. After completion of the ramps, test strips were constructed in an adjacent parking lot. This consumed the first "wasted" tonnage of each mix, as well the rejected first production run of the Sasobit mix. It also provided an opportunity for the paving crew to familiarize themselves with the warm-mix asphalt. Initially the test strip was planned to serve as an early-opening experiment under quarry truck traffic to assess the potential for early rutting immediately after construction. However, this did not materialize as there was no through-traffic in the area. The test strips and test track sections were constructed in the same order as asphalt production (i.e., Control, followed by warm-mix sections in alphabetical order).

Equipment

The following equipment was used during placement of the asphalt concrete layers.

- Caterpillar 1000D paver
- Sakai SW850 steel-wheel vibrating roller (breakdown compaction)
- Sakai SW850 steel-wheel vibrating roller (final rolling)
- Sakai GW750 rubber-tired roller
- Sakai SW320 steel-wheel vibrating roller (ramps)
- Binder distributor (tack coat application)
- Dump trucks
- John Deere 1483 skip loader

Prime Coat

After a final visual inspection of the base, the test track was lightly sprayed with water to bind any surface fines (Figure 2.26, approximately 7:55 AM). Once the water had penetrated, prime coat (SS-1 asphalt emulsion) was applied with a hand-held lance over the entire test track (Figure 2.27, approximately 8:10 AM to 8:25 AM). The application rate was estimated at 1.0 L/m^2 (0.25 gal/yd^2), but due to the method of application it could not be accurately determined or controlled. The prime was allowed to break during the construction of the test strip. Some areas of poor adhesion were noted, and some damage was caused by foot and vehicular traffic (Figure 2.28). Weather conditions at the time of priming were as follows:

- Air temperature: 16°C (61°F)
- Surface temperature: 13°C (56°F)
- Relative humidity: 83 percent
- Dew point: 13°C (55°F)

Figure 2.26: Water spray prior to priming.

Figure 2.27: Prime application.

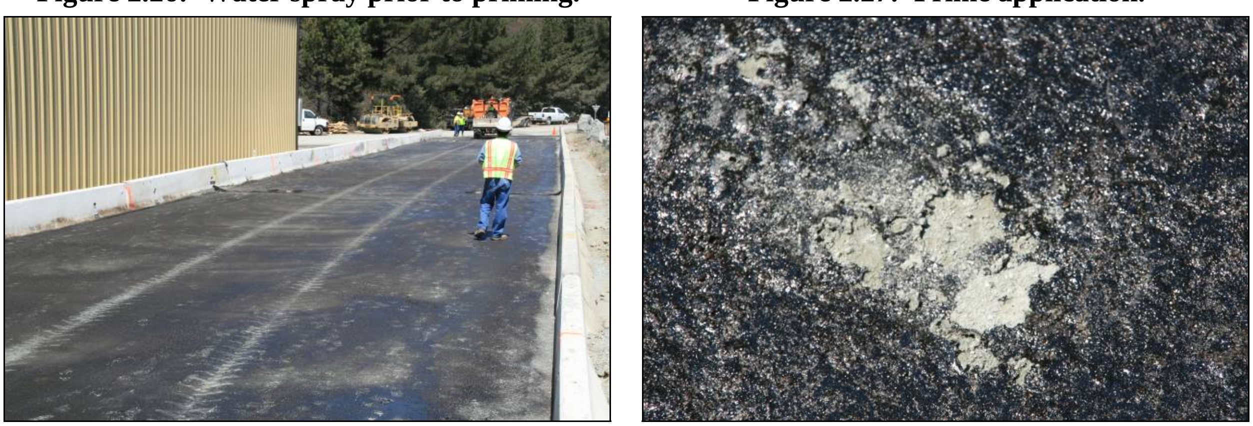


Figure 2.28: Damage to prime by vehicle and foot traffic.

First Lift: Control Section

Placement of the asphalt concrete on the Control section started at 12:15 PM with the positioning of the paver at the start of the Control section. The first truck load was tipped into the paver at 12:25 PM. Three loads were used and the paver reached the end of the section eight minutes after starting. Some haze was noted during tipping. Breakdown rolling started as soon as the paver was moved off of the section. Density and temperature measurements were taken throughout (see Section 0). Six passes were made with the breakdown roller (approximately six minutes). This was followed by the rubber-tired roller, which applied ten passes in an 11-minute period. Final rolling was completed with the steel-wheel roller (with vibration) in three passes at 12:57 PM. Paver spillage was removed from the end of the section to ensure a clean and regular surface and join for the Advera section. The second part of the final rolling with the steel-wheel roller (three passes, no vibration) was completed when the section had cooled. This took place between 1:45 PM and 1:50 PM. The construction process is illustrated in Figure 2.29.

Mix delivery (note haze)

Paver train

Breakdown rolling

Density check

Figure 2.29: Control: Placement of first lift of asphalt concrete.

Rubber-tired roller | **Final rolling**

Figure 2.29: Control: Placement of first lift of asphalt concrete (*continued*).

No problems were noted during breakdown rolling, however, some pick-up was observed during rolling with the rubber-tired roller (Figure 2.30). This was corrected during the final roll.

Figure 2.30: Control: Pick up during rubber-tire rolling.

<u>First Lift: Advera Section</u>

The same process described above was followed for the placement of the Advera mix, which started at 1:12 PM. No haze was observed during tipping of the mix into the paver (Figure 2.31). Breakdown rolling was achieved with eight passes. Ten passes were made with the rubber-tired roller followed by four passes for initial final rolling (with vibration). This phase of construction was completed at 1:38 PM (33 minutes). The second part of the final rolling (three passes, no vibration) was completed between 1:45 PM and 1:50 PM at the same time as the Control. No problems were observed during any of the compaction phases and a tightly bound surface was achieved (Figure 2.32).

Figure 2.31: Advera: Mix delivery, no haze.

Figure 2.32: Advera: Surface after final rolling.

<u>First Lift: Evotherm Section</u>

The same process followed for the previous two sections was also followed for the Evotherm mix. Construction started at 1:50 PM. No haze was observed during tipping of the mix into the paver. A rag was accidentally dropped in the paver, leaving an indentation on the mat that was repaired by hand (Figure 2.33 and Figure 2.34). Six passes were made with the breakdown roller and twelve with the rubber-tired roller. Initial final rolling was achieved in four passes (with vibration). This phase of construction was completed at 2:15 PM and took 25 minutes. The second part of the final rolling (three passes, no vibration) was completed between 2:45 PM and 2:50 PM. No problems were observed during the breakdown rolling, but some shearing was noted under the rubber-tired roller (Figure 2.35). Final rolling provided a smooth, tightly bound surface (Figure 2.36).

Figure 2.33: Evotherm: Damage behind paver.

Figure 2.34: Evotherm: Damage repair.

Figure 2.35: Evotherm: Shear after rubber-tired roller.

Figure 2.36: Evotherm: Surface after final rolling.

<u>First Lift: Sasobit Section</u>

The same process followed for the previous three sections was also followed for the Sasobit mix. Construction started at 2:17 PM. No haze was observed during tipping of the mix into the paver. Seven passes were made with the breakdown roller, during which the mix appeared tender, with some shearing noted (Figure 2.37). This was attributed in part to higher temperatures on this section (probably due to the shorter period between mix production and placement) compared to the Advera and Evotherm sections. Twelve passes were completed with the rubber-tired roller, during which some pick-up was also observed (Figure 2.38). Initial final rolling was achieved in four passes (with vibration), with tenderness still evident in the form of shearing (Figure 2.39). This phase of construction was completed at 2:42 PM (25 minutes). The second part of the final rolling (five passes, no vibration) was completed between 3:00 PM and 3:05 PM, after which a smooth and relatively tightly bound surface was achieved (Figure 2.40).

Figure 2.37: Sasobit: Shearing during breakdown rolling.

Figure 2.38: Sasobit: Pick up during rubber-tire rolling.

Figure 2.39: Sasobit: Surface after final rolling.

Figure 2.40: Sasobit: Shearing during final rolling.

Tack Coat Between Lifts

Tack coat was applied in two separate passes, the first on the Control and Advera sections at 3:00 PM (Figure 2.41), and the second on the Evotherm and Sasobit sections at 3:50 PM. An SS-1 emulsion was applied with a distributor at an application rate of approximately 0.5 L/m^2 (0.1 gal/yd^2). Some steam was observed when applying over the Sasobit section (Figure 2.42), probably due to the shorter cooling time since the placement of the first lift compared to the other sections.

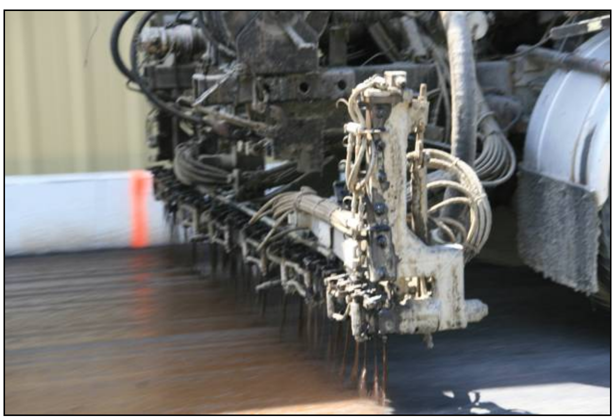


Figure 2.41: Tack coat application (Control).

Figure 2.42: Tack coat application (Sasobit).

Second Lift: Control Section

The same placement and compaction process was followed for the second lift of the Control mix, which started at 3:03 PM, with the section completely shaded by the adjacent shed. Some haze was again observed during tipping of the mix into the paver. Breakdown rolling was achieved with six passes, with some tenderness observed. Twelve passes were made with the rubber-tired roller followed by three passes for the first phase of final rolling (with vibration). This phase of construction was completed at 3:26 PM

(23 minutes). The second part of the final rolling (three passes, no vibration) was completed between 4:08 PM and 4:12 PM. No problems were observed during rubber-tired and final rolling.

Second Lift: Advera Section

The same placement and compaction process was followed for the second lift of the Advera mix, which started at 3:28 PM. No haze was observed during tipping of the mix into the paver. Breakdown rolling was achieved with eight passes, followed by twelve passes with the rubber-tired roller and three passes with the steel-wheel roller for the first phase of final rolling (with vibration). This phase of construction was completed at 3:47 PM (19 minutes). The second part of the final rolling (three passes, no vibration) was completed between 4:08 PM and 4:12 PM at the same time as final rolling on the Control section. The layer appeared very stable during all stages of compaction and no tenderness or shearing was observed.

Second Lift: Evotherm Section

The same placement and compaction process was followed for the second lift of the Evotherm mix, which started at 3:48 PM. The section was shaded by the adjacent shed for the duration of work. No haze was observed during tipping of the mix into the paver. Breakdown rolling was achieved with six passes, followed by twelve passes with the rubber-tired roller and three passes with the steel-wheel roller for the first phase of final rolling (with vibration). This phase of construction was completed at 4:20 PM (30 minutes). The second part of the final rolling (three passes, no vibration) was completed between 5:00 PM and 5:12 PM. Some tenderness was observed during the breakdown rolling and rolling with the rubber-tired roller. No problems were observed during final rolling.

Second Lift: Sasobit Section

The same placement and compaction process was followed for the second lift of the Sasobit mix, which started at 4:20 PM. No haze was observed during tipping of the mix into the paver. Breakdown rolling was achieved with six passes. Some tenderness was noted, similar to that observed during compaction of the first lift. Twelve passes with the rubber-tired roller were applied in the next stage of compaction, with pick-up again noted. The first phase of final rolling totalled six passes (with vibration), during which the layer appeared more stable. This phase of construction was completed at 4:40 PM (30 minutes). The second part of final rolling (three passes, no vibration) was completed between 5:00 PM and 5:12 PM at the same time as final rolling on the Evotherm section.

2.7.2 Instrumentation

Two strain gauges were placed on top of the primed base on each section. One gauge (Tokyo-Sokki KM-100HAS) was placed in the transverse position, with the midpoint 1,800 mm (70.9 in.) from the outside edge (K-rail) of the pavement. The second gauge (CTL ASG-152) was placed in the longitudinal position, with the midpoint 2,000 mm (78.7 in.) from the outside edge of the pavement (Figure 2.43). Actual positions on each section together with the gauge identifier are listed in Table 2.10.

Table 2.10: Strain Gauge Position Detail

Section	Gauge Position* (m)	CTL Label	Tokyo Sokki Label
Control	29.82	R-45	EKZ 04392
Advera	69.25	R-46	EKZ 04393
Evotherm	30.96	R-47	EKZ 04394
Sasobit	70.50	R-48	EKZ 04395
* Measured from x – y = 0 position on southern end of the section (see Figure 2.6).			

Figure 2.43: Strain gauge layout.

Asphalt concrete was removed from the first truck of each mix with a shovel and placed over the strain gauges and wires to prevent damage by the trucks and the paver (Figure 2.44).

Figure 2.44: Strain gauge covered with mix.

2.7.3 Quality Control

Quality control, both during and after construction, was undertaken jointly by Graniterock Company and the UCPRC. This included:

- Placement and compaction temperatures
- Thickness
- Density
- Deflection
- Skid resistance

Placement and Compaction Temperatures

Temperatures were systematically measured throughout the placement of the asphalt concrete using infrared temperature guns, thermocouples, and an infrared camera. Measurements included:

- Temperature of the mix as it was tipped into the paver
- Temperature of the mix behind the paver
- Temperature of the mat before compaction
- Temperature of the surface during compaction
- Temperature after priming
- Temperature of the surface prior to placing the second lift
- Temperature at the above locations during the second lift

A summary of the measurements is provided in Table 2.11 and in Figure 2.45 and Figure 2.46. The following observations were made:

- Average temperatures of the Control mix measured in the trucks as it was tipped into the paver were about 10°C (18°F) below the target compaction temperature. This was attributed to cooling in the silo (placing of the first lift of the Control mix started approximately four hours after mix production) and during transport from the asphalt plant. The temperature was, however, still within Caltrans-specified limits *(4)*. The temperature of the Advera mix was within the target for the first lift, but slightly below the target for the second lift. The temperature of the Evotherm mix was the same for both lifts, but slightly below the target, while the Sasobit mix was slightly above the target for the first lift and within the target range for the second lift. The Sasobit mix had the shortest wait in the silo (approximately two hours).
- There was very little temperature difference between the material being tipped into the paver and the mat behind the paver before compaction. The Advera, Evotherm, and Sasobit mixes lost less heat than the Control mix.
- Temperatures on the Control section dropped by 13°C and 18°C (23°F and 32°F) on the first and second lift respectively between placement with the paver and start of compaction with the breakdown roller. The drop on the Advera and Sasobit sections was 9°C and 12°C (16°F and 22°F) for the two lifts, while the drop on the Evotherm section was 13°C and 16°C (23°F and 29°F).

Table 2.11: Summary of Temperature Measurements

Lift	Measuring Point	Temperature (°C)			
		Control	Advera	Evotherm	Sasobit
1st	Truck	137	112	107	121
	Paver	135	110	106	120
	Mat	135	105	106	117
	Surface: begin compaction	122	96	93	108
	Surface: average during compaction	106	81	90	91
	Surface: end of compaction	94	72	76	74
	Mid-depth: average during compaction	113	94	92	87
2nd	Surface before prime	50	-	-	-
	Surface after prime	51	-	-	-
	Surface before second lift	50	53	51	54
	Truck	134	109	107	115
	Paver	128	109	107	113
	Mat	127	109	107	113
	Surface: begin compaction	109	97	91	101
	Surface: average during compaction	93	82	80	84
	Surface: end of compaction	68	73	72	74
	Mid-depth: average during compaction	122	100	105	91
Lift	**Measuring Point**	**Temperature (°F)**			
		Control	**Advera**	**Evotherm**	**Sasobit**
1st	Truck	279	234	225	250
	Paver	275	230	223	248
	Mat	275	221	223	243
	Surface: begin compaction	252	205	199	226
	Surface: average during compaction	223	178	194	196
	Surface: end of compaction	201	162	169	165
	Mid depth: average during compaction	235	201	198	189
2nd	Surface before prime	122	-	-	-
	Surface after prime	124	-	-	-
	Surface before second lift	122	127	124	129
	Truck	273	228	226	239
	Paver	262	228	226	235
	Mat	261	228	226	235
	Surface: begin compaction	228	207	196	214
	Surface: average during compaction	199	180	176	183
	Surface: end compaction	154	163	162	165
	Mid-depth: average during compaction	252	212	221	196

- The average temperature difference between the start of breakdown compaction and final rolling on the Control section was 28°C (50°F) for the first lift and 41°C (74°F) for the second lift. The difference for the Advera section was 24°C (43°F) for both lifts. On the Evotherm section, the difference was 17°C and 19°C (31°F and 34°F) respectively, and on the Sasobit section the difference was 34°C and 27°C (61°F and 49°F) respectively.
- Average start- and end-compaction temperatures on the Control section were within the Caltrans specification limits *(4)*. The average start-compaction temperatures on the Advera, Evotherm, and Sasobit sections were below the specification limits (as required in the experimental design *[3]*), but end-of-compaction temperatures were within limits *(4)*.

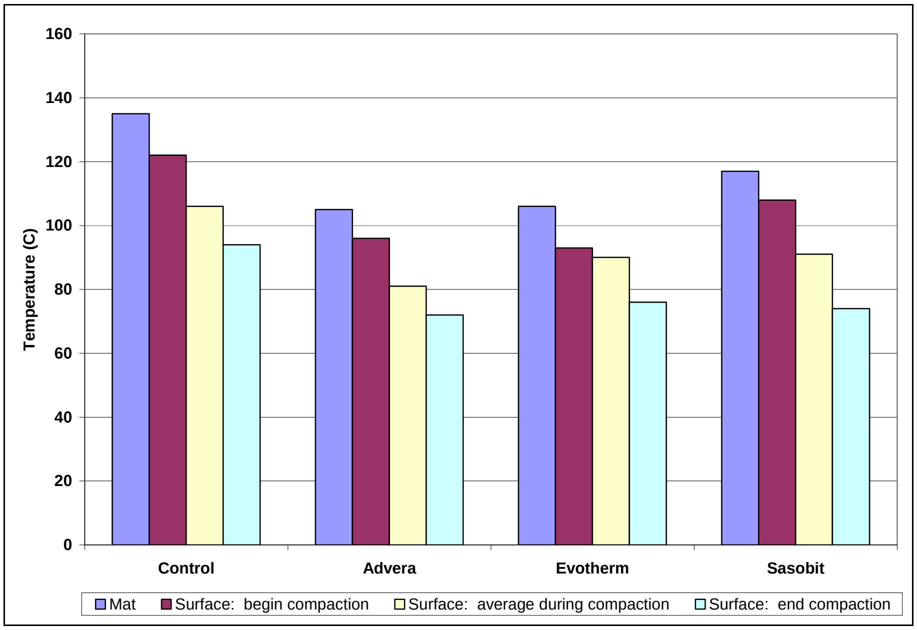


Figure 2.45: Summary of temperature measurements (first lift).

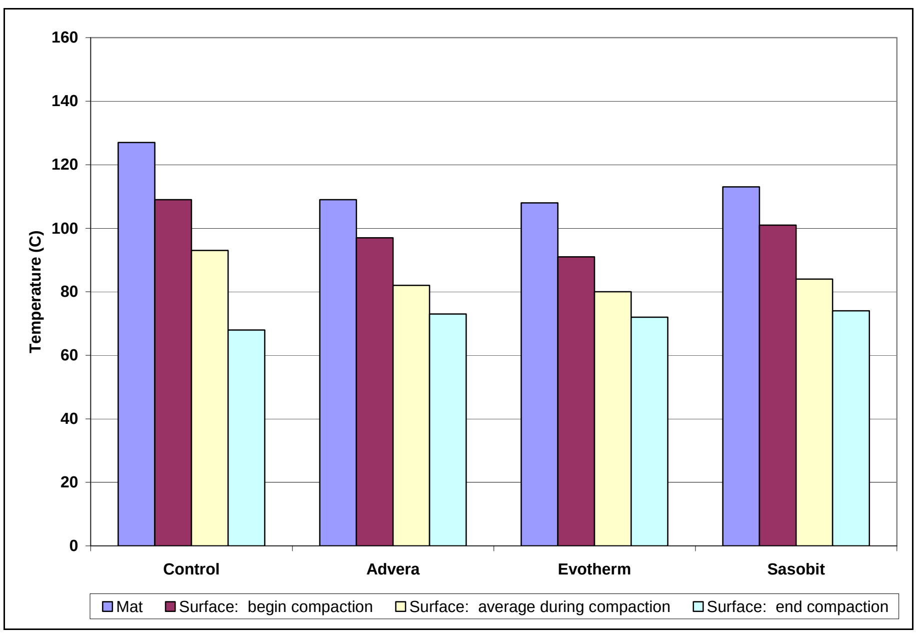
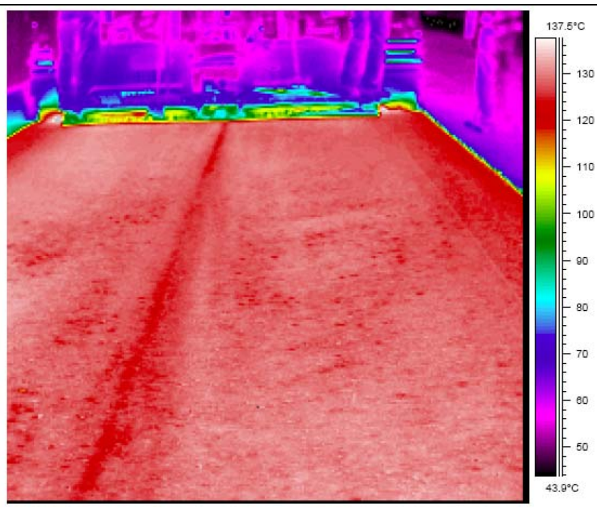


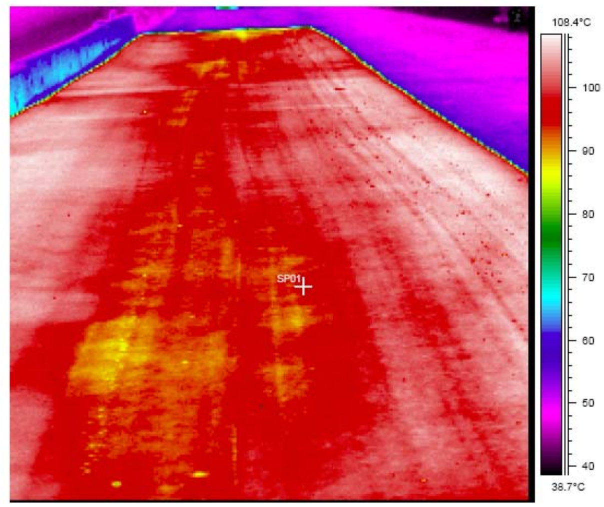
Figure 2.46: Summary of temperature measurements (second lift).

- The rate of temperature loss between initial placement and completion of compaction on the Control section was significantly higher than on the warm-mix sections.
- Temperature drop on the Control and Evotherm sections did not appear to be influenced by the shade during placement of the second lift. The differences between the start and end of compaction on the shaded sections were less than the differences on the Advera and Sasobit sections, which were placed and compacted in direct sunlight.

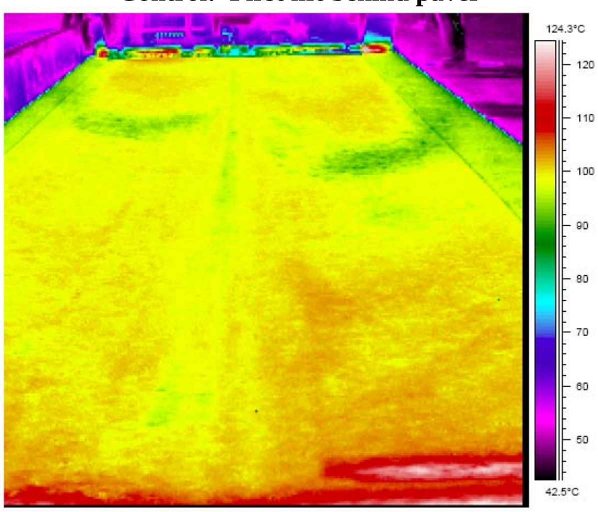
Thermal camera images (FLIR Systems ThermaCAM PM290, recorded by T.J. Holland of Caltrans) of the mat behind the paver and after compaction with the rubber-tired roller are shown in Figure 2.47. The images clearly show the lower temperatures of the warm-mix sections and the uniformity in temperature over the mat. (Note that temperature scales on the right side of the photographs differ between images.)



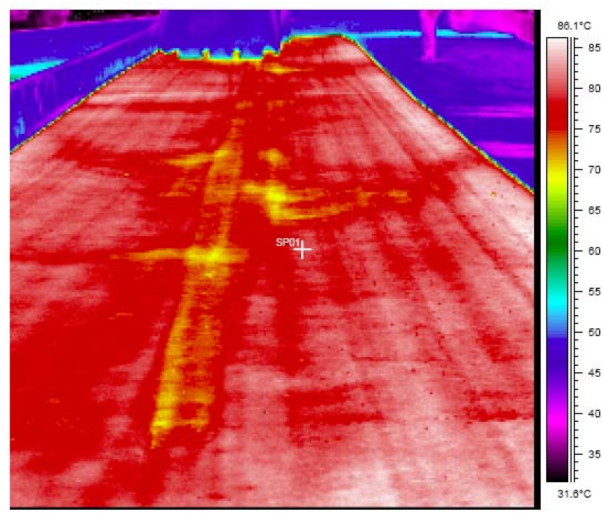
Control: First lift behind paver



Control: First lift after rubber-tired roller

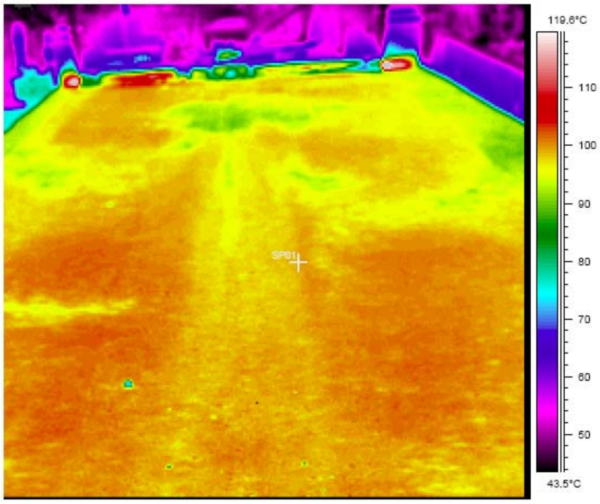


Advera: First lift behind paver

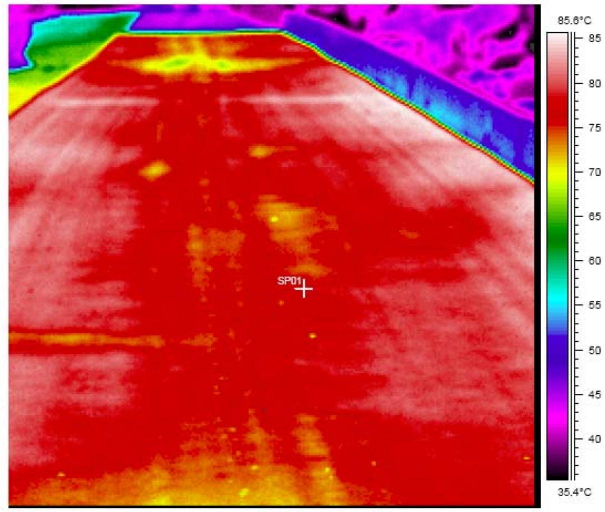


Advera: First lift after rubber-tired roller

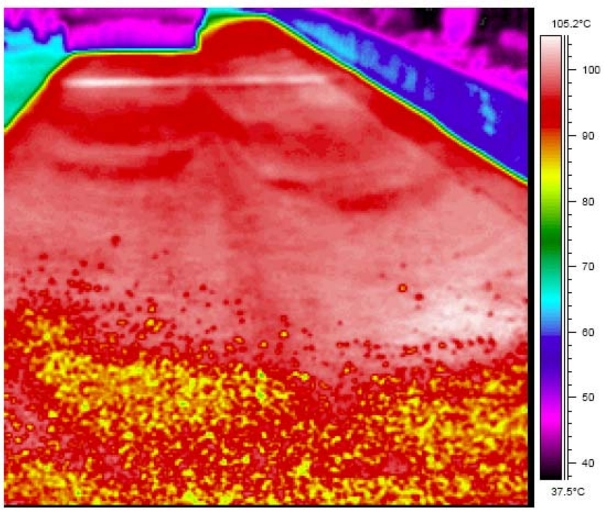
Figure 2.47: Thermal images of test track during construction.



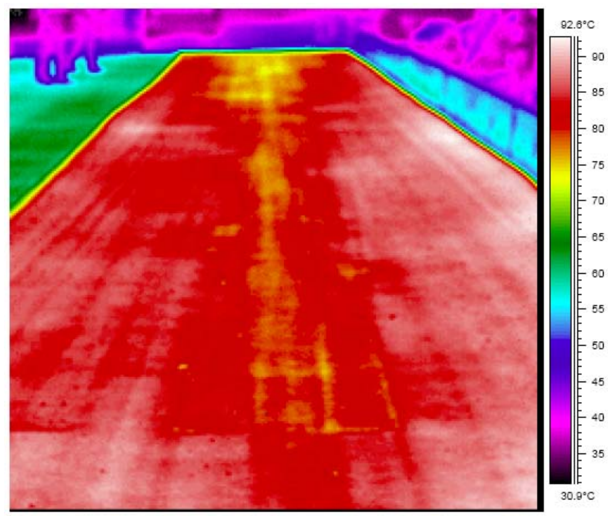
Evotherm: First lift behind paver



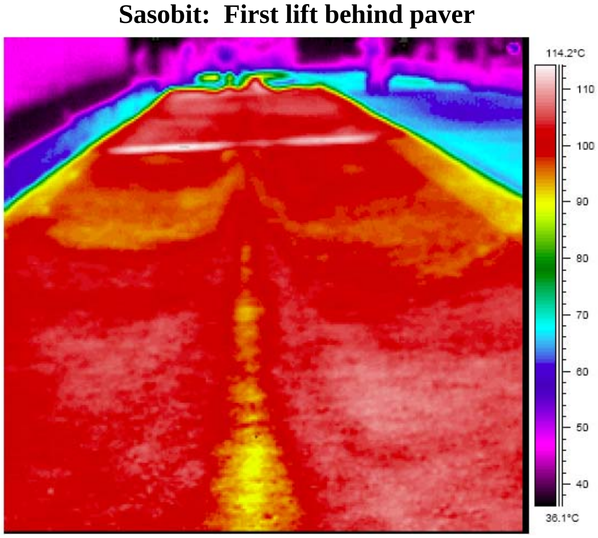
Evotherm: First lift after rubber-tired roller



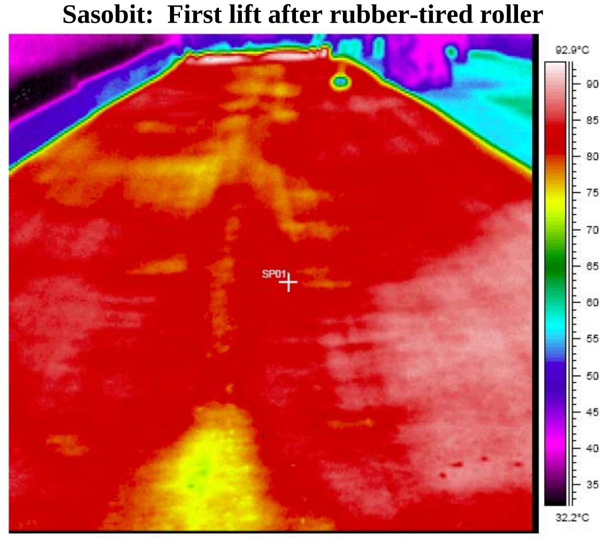
Sasobit: First lift behind paver



Sasobit: First lift after rubber-tired roller

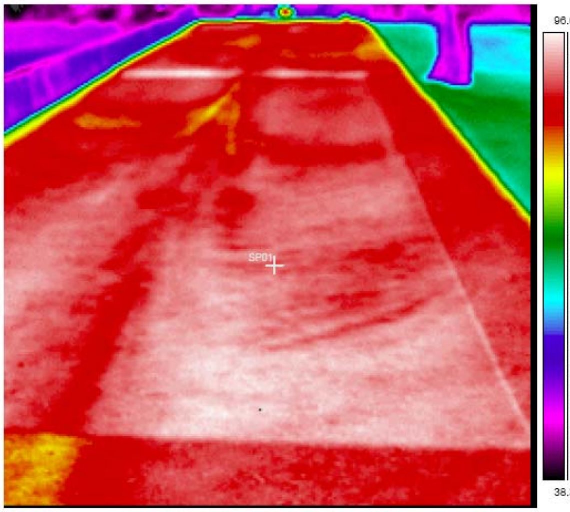


Control: Second lift behind paver

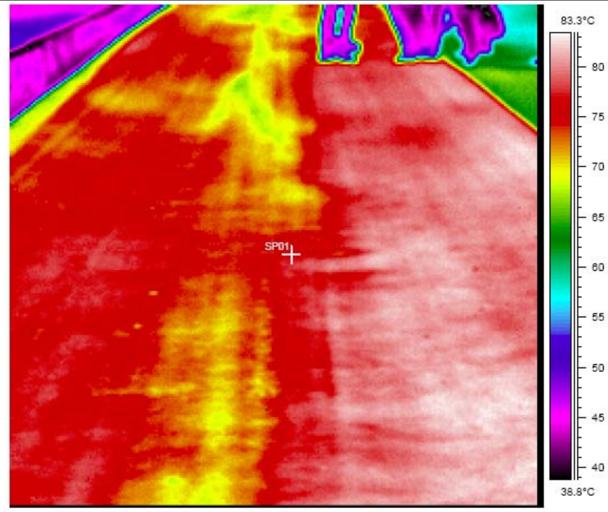


Control: Second lift after rubber-tired roller

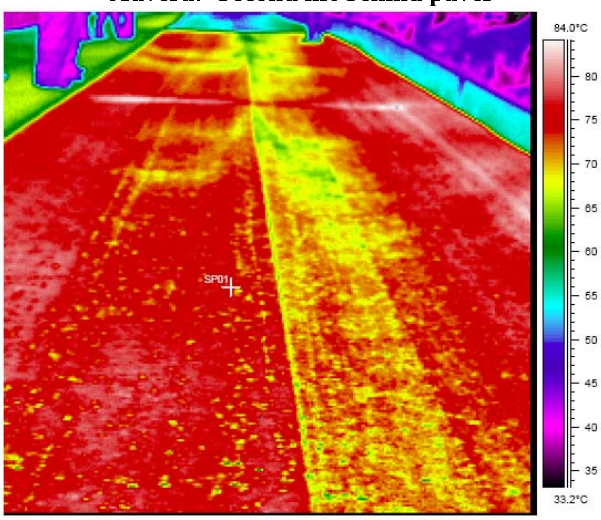
Figure 2.47: Thermal images of test track during construction *(continued).*



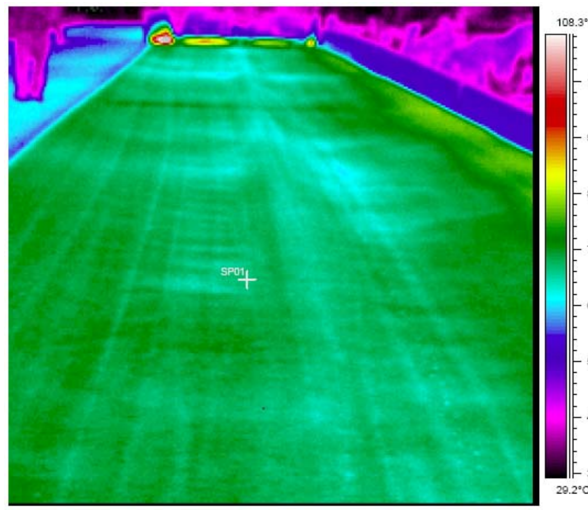
Advera: Second lift behind paver



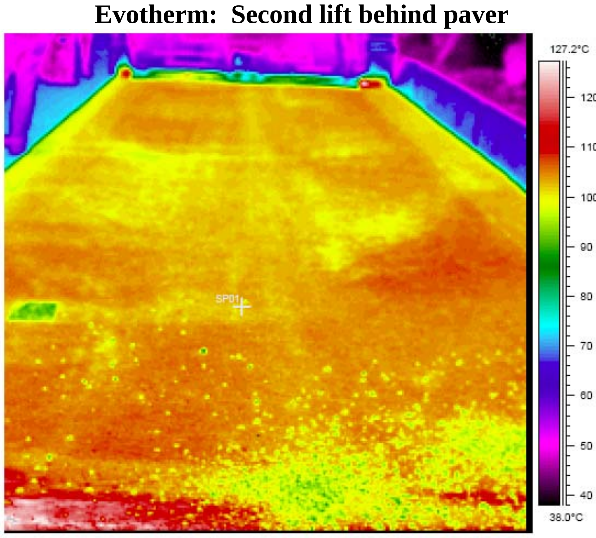
Advera: Second lift after rubber-tired roller



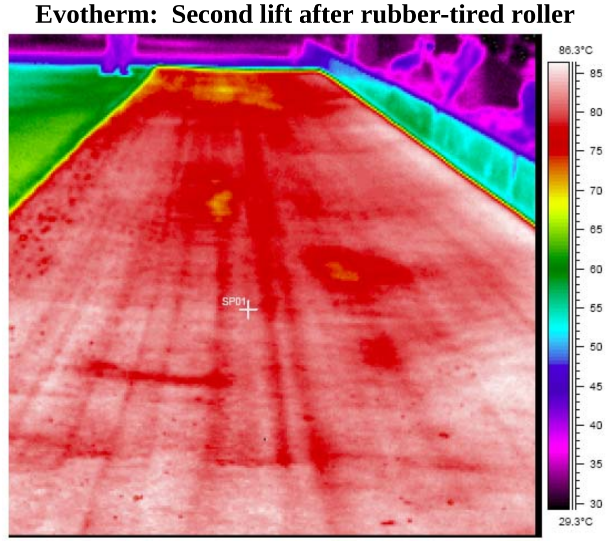
Evotherm: Second lift behind paver



Evotherm: Second lift after rubber-tired roller



Sasobit: Second lift behind paver



Sasobit: Second lift after rubber-tired roller

Figure 2.47: Thermal images of test track during construction *(continued)*.

Thickness

Thickness was monitored with probes throughout the construction process. The thickness of the slabs removed for laboratory testing after construction (see Section 0) was also measured. The average thickness of the combined two layers was 112 mm (4.4 in.), 8.0 mm (0.3 in.) thinner than the design thickness of 120 mm (4.7 in.). The thinnest measurement recorded was 98 mm (3.9 in.) and the thickest 124 mm (4.9 in.). This range of thicknesses was considered acceptable and representative of typical construction projects. Actual thicknesses of the asphalt concrete layers adjacent to the HVS test sections will be determined from cores taken during the planned forensic investigation after all HVS testing has been completed.

Density

Compaction was monitored using nuclear and non-nuclear gauges throughout the construction process. The results were used to manage the number of rolling passes, roller selection, and roller settings. These densities were monitored but not recorded.

Final density measurements were taken on August 26, 2007 by Graniterock Company, using a calibrated nuclear gauge. Measurements were taken according to the plan shown in Figure 2.48. A summary of the results is provided in Table 2.12. The results show some variability among the four sections as well as within each section. Air-void contents determined from these measurements correspond to observations made during construction (see Section 2.7). The Control and Advera sections, which appeared to compact without problems on the day with little or no evidence of tenderness, had the lowest air-void contents (5.6 and 5.4 percent respectively). The Evotherm and Sasobit sections, which showed signs of tenderness at various stages of the compaction process, had higher air-void contents (7.1 and 7.0 percent respectively). Density increased with increasing distance from the outside edge (i.e., K-rail) on the Advera, Evotherm and Sasobit sections. Density was highest along the middle of the section for the Control.

Falling Weight Deflectometer

FWD measurements were taken on September 5, 2007 at 1.0 m intervals (start point 5.0 m and end point 75 m) along the centerline of each section (i.e., y =2.0 m and y =6.0 m) Average results of the second 40 kN load drop are summarized in Table 2.13 and in Figure 2.49 through Figure 2.51. There was no significant difference in the deflections measured on the four sections and relatively little variation along the length of each section, indicating consistent construction. Sensor 1 deflections on the asphalt concrete decreased slightly with increasing chainage (south to north), consistent with the changing depth of the bedrock. The Advera section had the lowest average deflections, followed by the Sasobit, Control, and Evotherm sections. The asphalt concrete layer exhibited some temperature sensitivity, as expected.

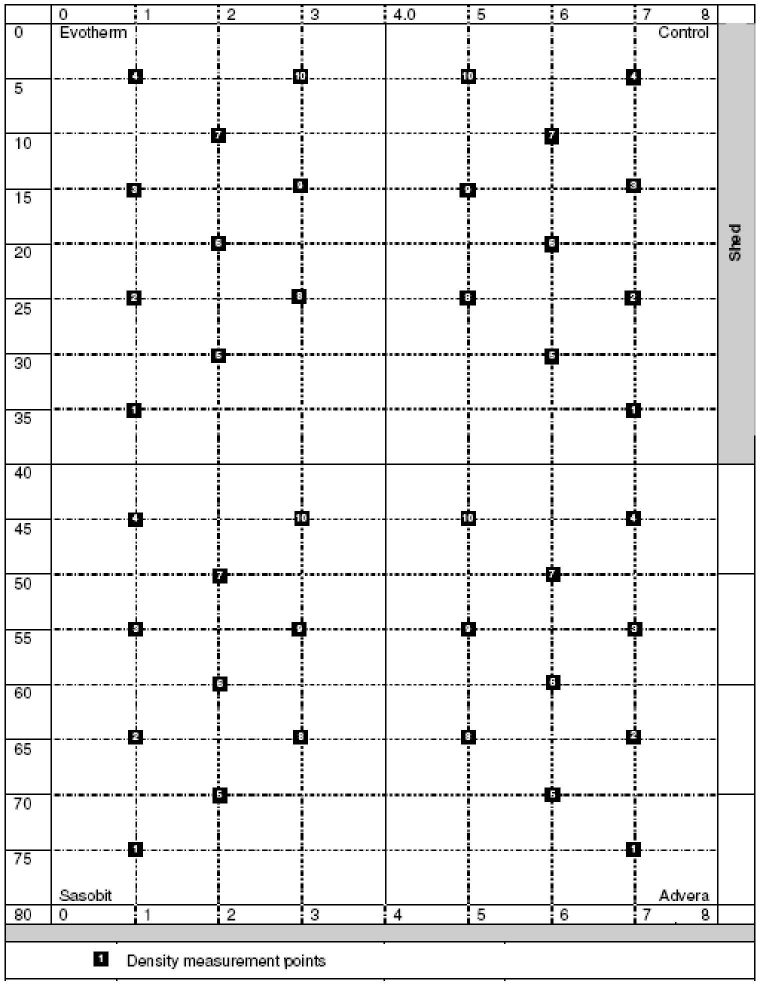


Figure 2.48: Asphalt concrete density measurement plan.

Table 2.12: Summary of Asphalt Concrete Density Measurements

Position	Nuclear Gauge-Determined Specific Gravity			
	Control	**Advera**	**Evotherm**	**Sasobit**
1	2.383	2.442	2.393	2.415
2	2.426	2.422	2.399	2.429
3	2.398	2.422	2.381	2.424
4	2.406	2.424	2.417	2.398
Average 1-4	2.403	2.428	2.398	2.417
5	2.449	2.445	2.413	2.415
6	2.457	2.447	2.390	2.428
7	2.455	2.435	2.436	2.438
Average 5-7	2.454	2.442	2.413	2.427
8	2.410	2.466	2.421	2.432
9	2.419	2.448	2.443	2.433
10	2.427	2.467	2.417	2.426
Average 8-10	2.419	2.460	2.427	2.430
Overall average	2.423	2.442	2.411	2.424
	Control	**Advera**	**Evotherm**	**Sasobit**
Rice Specific Gravity	2.567	2.581	2.596	2.606
In-place air voids (%)	5.61	5.39	7.13	6.99

Table 2.13: Summary of FWD Measurements

Section	Deflection @ D1[1] (mm)		Deflection @ D6[2] (mm)		Deflection @ D3[3] (mm)		Deflection @ D5[4] (mm)	
	AM	PM	AM	PM	AM	PM	AM	PM
Control	0.243	0.360	0.047	0.047	0.149	0.168	0.075	0.069
Advera	0.186	0.263	0.034	0.038	0.090	0.091	0.045	0.048
Evotherm	0.260	0.402	0.045	0.046	0.154	0.162	0.074	0.062
Sasobit	0.208	0.322	0.048	0.053	0.125	0.141	0.068	0.073
Average	0.22	0.34	0.04	0.05	0.13	0.14	0.07	0.06
Std deviation (mm)	0.03	0.06	0.01	0.01	0.03	0.04	0.01	0.01
CoV (%)	15	18	15	13	23	25	22	18
Section	**Average Temperatures**							
	AM (°C)		**PM (°C)**		**AM (°F)**		**PM (°F)**	
	Air	**Surface**	**Air**	**Surface**	**Air**	**Surface**	**Air**	**Surface**
Control	14.0	18.9	26.6	42.9	57	59	80	109
Advera								
Evotherm	19.3	23.9	26.4	40.6	67	75	80	105
Sasobit								

[1] Geophone D1, 0 mm offset
[2] Geophone D6, 925 mm offset
[3] Geophone D3, 315 mm offset
[4] Geophone D5, 630 mm offset

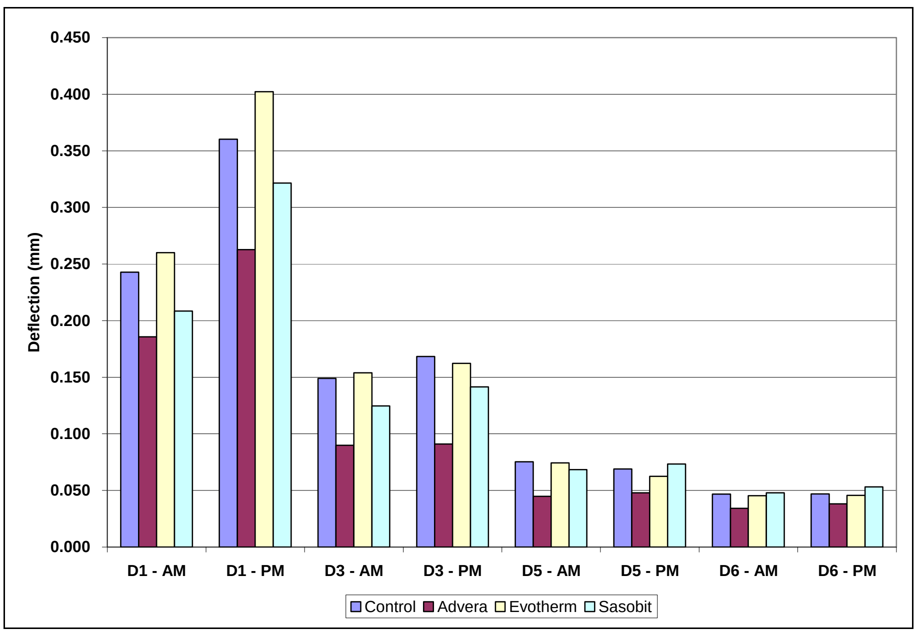


Figure 2.49: Summary of average deflection by section.

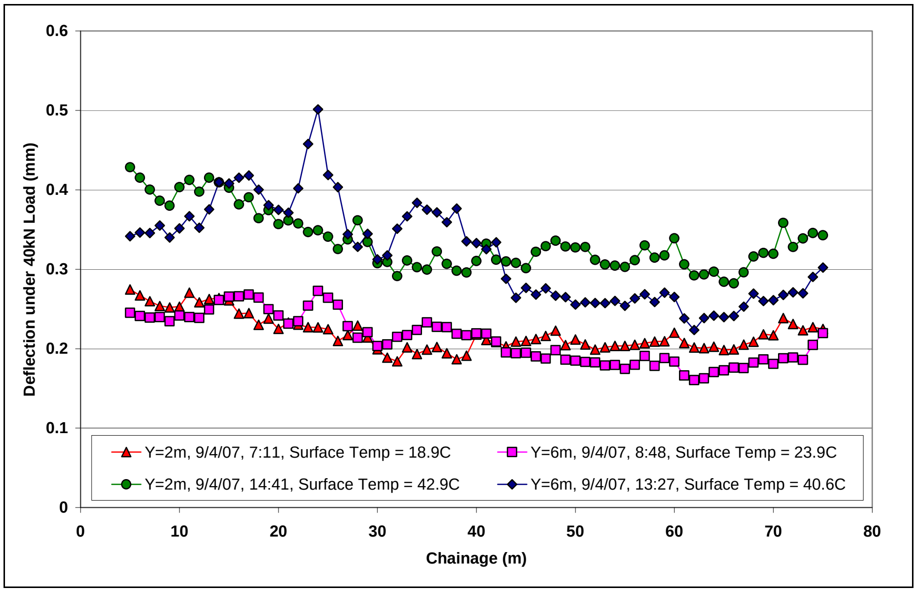


Figure 2.50: Summary of Sensor-1 deflection measurements on asphalt concrete surface.

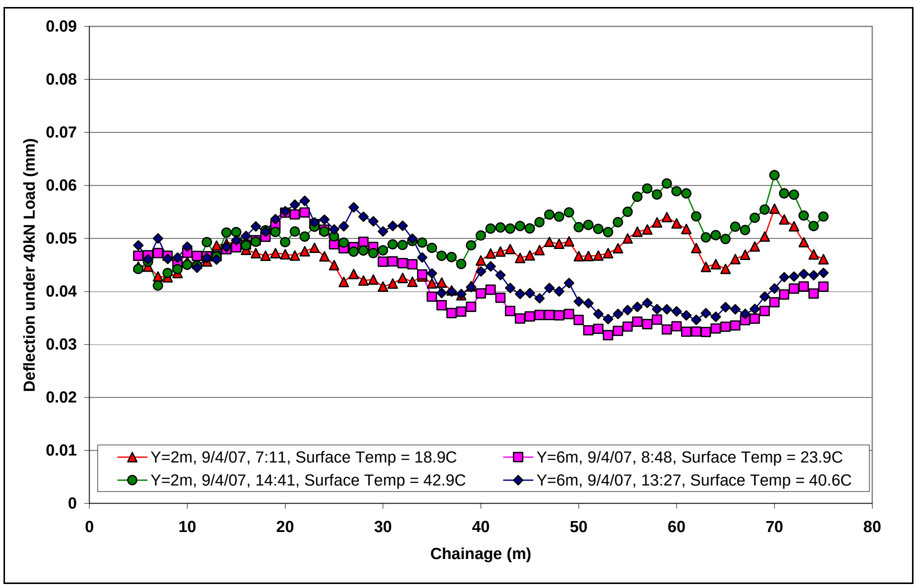


Figure 2.51: Summary of subbase/subgrade deflection measurements (D6 geophone).

Skid Resistance

The potential influence that warm-mix asphalt additives might have on the skid resistance of a newly constructed surface was identified by practitioners during the preparation of the work plan. Plans were therefore made to measure this attribute soon after construction. Two devices were used for these measurements; namely a Caltrans Portable Skid Tester (Figure 2.52) and a Dynamic Friction Tester (Figure 2.53). Texture measurements for the Dynamic Friction Tester were determined with a Circular

Track Meter (Figure 2.54). Measurements were taken at 10 m intervals along the centerline of each section (i.e., y = 2.0 m and y = 6.0 m).

Figure 2.52: Caltrans Portable Skid Tester.

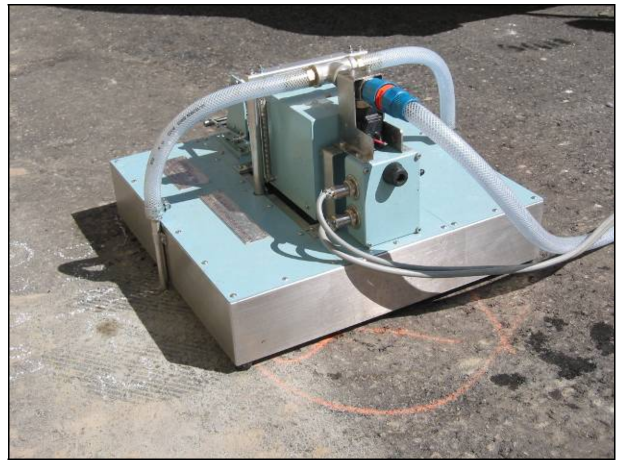


Figure 2.53: Dynamic Friction Tester.

Figure 2.54: Circular Track Meter.

- **Methodology: Caltrans Portable Skid Tester**

  Skid resistance testing with the Caltrans Portable Skid Tester followed the standard CT-342 test method. A coefficient of friction value is calculated from the measured data.

- **Methodology: Dynamic Friction Tester**

  Data collected from the Dynamic Friction Tester (DFT) and Circular Track Meter (CTM) were used to calculate an International Friction Index (IFI), developed by the International Association of Road Congresses (PIARC), for harmonizing of friction measurements taken with different equipment and/or at different slip speeds to a common calibrated index. The IFI includes measurements of:

- Macrotexture and friction on wet pavements;
- A speed constant derived from the macrotexture measurement that indicates the speed-dependence of the friction, and
- A friction number corresponding to a slip speed of 60 km/h (38 mph).

The IFI is based on the assumption that friction is a function of speed and macrotexture and that for a given level of macrotexture, the effect of speed causes an exponential decay in the value of friction as the speed increases. The equation for this relationship is (Equation 2.1):

$$FR60 = FRS \times e^{\left(\frac{S-60}{S_p}\right)} \quad \textbf{(2.1)}$$

where: *FR60* is the calculated friction of a device at 60 km/h

FRS is the measured friction at a slip speed of *S* km/h

S_p is the speed constant of the IFI, which accounts for the pavement macrotexture.

The calculated friction at 60 km/h for a specific device is then transformed using a linear function of the form (Equation 2.2):

$$F60 = A + B \times FR60 + C \times Tx \quad \textbf{(2.2)}$$

where: *F60* is the calculated Friction Number of the IFI

A, *B*, and *C* are device-specific constants

Tx is the surface texture measured in accordance with ASTM E 1845-01.

The values of the constants for measurements taken with the Dynamic Friction Tester (ASTM E 1911-02) are *A* = 0.081, *B* =0.732, and *C* =0.

The mean profile depth (MPD) was measured with the Circular Track Meter and converted into a speed constant (S_p) using the following equation from ASTM E 2157-03 (Equation 2.3):

$$S_p = -3.76 + 107.6 MPD_{CTM} \quad \textbf{(2.3)}$$

- **Results**

The results from the DFT are summarized in Table 2.14. Results from the Caltrans Portable Skid Tester were unavailable to the UCPRC at the time this report was prepared. A direct comparison of results from the DFT and the Caltrans Skid Tester is also not possible given the differences in which measurements are taken.

The IFI values recorded with the DFT on each of the four sections are considered somewhat low compared to measurements on in-service pavements, which typically have values higher than 0.35. This was attributed to testing being carried out immediately after construction and before any trafficking, when the surface was still likely to have residues from rolling. These residues, together with the coating of asphalt on the exposed aggregate surfaces, are typically polished off by traffic soon after opening. Skid number values at 64 km/h (SN64 [40 mph]) for a Locked-Wheel Skid Trailer (ASTM E 274-97) were backcalculated from the IFI and texture values. These were above the Caltrans minimum value of 0.30 on the Control, Evotherm, and Sasobit sections, and slightly below the minimum value of 0.30 on the Advera section. The lower value on the Advera section is not attributed to the use of the Advera process. The skid number values would be expected to increase once the surface had been subjected to environmental aging and traffic wear.

Table 2.14: Results of Skid Resistance Testing

Section	Chainage	Dynamic Friction Tester (DFT)				Caltrans Skid Tester
		S_p [1]	FR60 [2]	IFI [3]	Backcalc SN_{64} [4]	
Control	15	52.2	0.24	0.26	0.37	Data was unavailable to the UCPRC at the time this report was prepared
	20	51.1	0.23	0.25	0.36	
	25	60.8	0.31	0.31	0.43	
	30	45.7	0.19	0.22	0.32	
	35	54.3	0.26	0.27	0.38	
	Average	52.8	0.25	0.26	0.37	
	Std Dev	5.5	0.04	0.03	0.04	
Advera	55	49.0	0.23	0.24	0.34	
	60	41.4	0.16	0.20	0.29	
	65	42.5	0.17	0.20	0.29	
	70	43.6	0.18	0.21	0.30	
	75	36.1	0.12	0.17	0.25	
	Average	42.5	0.17	0.20	0.29	
	Std Dev	4.6	0.03	0.03	0.03	
Evotherm	15	50.0	0.25	0.26	0.38	
	20	53.3	0.24	0.26	0.36	
	25	47.9	0.23	0.25	0.36	
	30	39.3	0.20	0.23	0.33	
	35	49.0	0.24	0.25	0.36	
	Average	47.9	0.23	0.25	0.36	
	Std Dev	5.2	0.02	0.01	0.02	
Sasobit	55	42.5	0.23	0.25	0.36	
	60	51.1	0.26	0.27	0.39	
	65	64.0	0.32	0.31	0.43	
	70	49.0	0.26	0.27	0.38	
	75	50.0	0.24	0.25	0.36	
	Average	51.3	0.26	0.27	0.39	
	Std Dev	7.8	0.03	0.03	0.03	

[1] Speed Constant [2] Calculated friction [3] International Friction Index [4] Backcalculated skid number

2.8 Sampling

2.8.1 Samples for Laboratory-Mixed, Laboratory-Compacted Specimen Testing

Laboratory-mixed, laboratory-compacted specimen testing was tentatively planned for the Phase 2 laboratory testing program *(3)*. Mix constituents therefore needed to be collected and stored for later testing if required. A truckload of aggregate sample was collected from a feed from the aggregate conveyor into the asphalt plant drum. Asphalt binder samples were collected from the delivery tanker. All samples were transported to and stored at the UCPRC Richmond Field Station.

2.8.2 Samples for Field-Mixed, Laboratory-Compacted Specimen Testing

In most experiments, field-mixed samples are collected from the paver, stored in buckets, transported to the laboratory, and then reheated and compacted into molds at a later date. Communication with the additive suppliers indicated that although Sasobit would retain its properties and the mix could be reheated and compacted at the original construction temperature (i.e., 120°C [250°F]), the Advera and Evotherm mix samples would need to be reheated and compacted at normal hot-mix asphalt temperatures (i.e., temperatures of 155°C [310°F] used for the Control mix). Compaction at these higher temperatures could result in the production of specimens that did not have the same characteristics as those compacted at the lower temperatures used in the experiment. This could lead to inaccurate assumptions with regard to expected performance.

Field-mixed, laboratory-compacted (FMLC) specimens were therefore prepared on site adjacent to the test track using jigs, molds, and a rolling-wheel compactor brought from the UCPRC laboratory. Loose mix was taken from the trucks with a skip loader immediately prior to it being tipped into the paver and then dumped next to the preparation area. The required volume of material, based on the densities determined earlier in the Graniterock laboratory, was weighed and then compacted into molds at the same temperatures as those recorded on the test track. Sufficient specimens were produced to satisfy the needs of the Phase 2 experimental design for comparing shear and fatigue beam test results on field-mixed, field-compacted; field-mixed, laboratory-compacted; and laboratory-mixed, laboratory-compacted specimens *(3)*. The remaining loose mix was placed in buckets for possible later testing. The specimen preparation process is illustrated in Figure 2.55.

Sample weighing **Placing sample in mold**

Figure 2.55: Preparation of field-mixed, laboratory-compacted specimens.

Rolling-wheel compaction/temperature control **Mold removal**

Placing loose-mix into buckets **Completed specimens**

Figure 2.55: Preparation of field-mixed, laboratory-compacted specimens *(continued)*.

2.8.3 Field-Mixed, Field-Compacted Samples

Field-mixed, field-compacted (FMFC) specimens in the form of slabs 500 mm by 500 mm (20 in.) for Phase 1 laboratory testing were sawn from an area 20 m by 0.5 m (66 ft by 1.6 ft) along the edge of each panel in the test track as shown in Figure 2.56. Slabs were sawn to the bottom of the asphalt concrete layers, extracted, stored on pallets, and then transported to the UCPRC Richmond Field Station laboratory. Inspection of the slabs indicated that the asphalt concrete was well bonded to the top of the base-course material, and that the two asphalt layers were well bonded to each other.

2.9 Construction Summary

Key observations from the mix production and construction process include:

- Overwatering during the early stages of base-course construction resulted in some weak areas. Moisture contents were highest in the area shaded by the shed.
- Average dry density on the base-course was 97 percent of the laboratory-determined maximum dry density. The final surface was tightly bound and free of loose material.
- Deflection measurements indicated that a relatively stiff and uniform base-course was constructed over a very stiff subgrade (bedrock). The deflections on the Control and Evotherm sections (shaded by the shed) were slightly lower than the Advera and Sasobit sections. A very stiff pavement structure will complicate any planned fatigue cracking experiments in that a very high number of HVS repetitions will be required before the pavement cracks.
- Minimal asphalt plant modifications were required to accommodate the warm-mix additives.
- No problems were noted with producing the asphalt mixes at the lower temperatures. Target mix production temperatures (i.e., 155°C and 120°C [310°F and 250°F]) were achieved.
- Although a PG 64-16 asphalt binder was specified in the work plan, subsequent tests by the FHWA indicated that the binder was rated as PG64-22. This should not affect the outcome of the experiment. After mixing Advera and Sasobit with the binder, the PG grading changed from PG 64-22 to PG 70-22. The addition of Evotherm did not alter the PG grade.
- The Control, Advera, and Evotherm mixes met the project mix design requirements. The binder content of the Sasobit mix was 0.72 percent below the target binder content and 0.22 percent below the lowest permissible binder content. This will probably influence performance and will need to be taken into consideration when interpreting HVS and laboratory test results.
- The Control mix had a higher specific gravity and Marshall stability, and a lower air-void content than the mixes with additives. It is not clear whether this was a testing inconsistency or linked to the lower production and specimen preparation temperatures. This will need to be investigated in the planned Phase 2 laboratory study.

Figure 2.56: Test track sampling plan and sample removal.

- Moisture contents of the mixes with additives were notably higher than that of the Control mix, indicating that potentially less moisture evaporates from the aggregate at lower production temperatures. All mixes were, however, well within the minimum Caltrans-specified moisture content level. Aggregate moisture contents will need to be strictly controlled in the stockpiles and maximum moisture contents prior to mix production may need to be set if warm-mix technologies are routinely used.

- Construction procedures and final pavement quality did not appear to be influenced by the lower construction temperatures. The Advera mix showed no evidence of tenderness, and acceptable compaction was achieved. Some tenderness resulting in shearing under the rollers was noted at various stages of breakdown and/or rubber-tired rolling on the Evotherm and Sasobit sections, indicating that the compaction temperatures were still higher than optimal. No problems were observed after final rolling at lower temperatures.
- Interviews with the paving crew after construction revealed that no problems were experienced with construction at the lower temperatures. Improved working conditions were identified as an advantage. Tenderness on the Evotherm and Sasobit sections was not considered as being significantly different than that experienced with conventional mixes.
- Although temperatures at the beginning of compaction on the warm-mix sections were considerably lower than the Caltrans-specified limits, the temperatures recorded on completion of compaction were within limits, indicating that the rate of temperature loss in the mixes with additives was lower than that on the Control section.
- Some haze/smoke was evident on the Control mix during transfer of the mix from the truck to the paver. No haze or smoke was observed on the mixes with additives.
- Average thickness of the two layers was 112 mm (4.4 in.). Minimal variation was observed, but cannot be fully quantified until cores are taken.
- Average air-void contents on the Control and Advera sections were 5.6 percent and 5.4 percent, respectively. Those on the Evotherm and Sasobit sections, which showed signs of tenderness during rolling, were approximately 7.0 percent. Based on these observations, it was concluded that adequate compaction can be achieved on warm-mixes at the lower temperatures. Optimal compaction temperatures are likely to differ between the different warm-mix technologies.
- Deflection measurements showed that relatively consistent construction was achieved on the test track. Lower deflection was recorded on the Advera and Sasobit sections, which was attributed to slightly better support (drier) conditions in the base.
- Skid resistance measurements indicated that the warm-mix additives tested do not influence the skid resistance of an asphalt mix.

3. TEST TRACK LAYOUT AND HVS TEST CRITERIA

3.1 Protocols

Heavy Vehicle Simulator (HVS) test section layout, test setup, trafficking, and measurements followed standard University of California Pavement Research Center (UCPRC) protocols *(5)*.

3.2 Test Track Layout

The Warm-Mix Asphalt Study test track layout is shown in Figure 3.1. Four HVS Test Sections were demarcated for the first phase of HVS testing for early-age rutting at high temperatures, which was carried out in the same order as construction (i.e., Control followed by warm-mixes in alphabetical order). The section numbers allocated were as follows:

- Section 600FD: Control
- Section 601FD: Advera
- Section 602FD: Evotherm
- Section 603FD: Sasobit

3.3 HVS Test Section Layout

The general test section layout for each of the rutting sections is shown in Figure 3.2. Station numbers (0 to 16) refer to fixed points on the test section and are used for measurements and as a reference for discussing performance.

3.4 Pavement Instrumentation and Monitoring Methods

Measurements were taken with the instruments listed below. Instrument positions are shown in Figure 3.2. Detailed descriptions of the instrumentation and measuring equipment are included in Reference 6. Intervals between measurements, in terms of load repetitions, were selected to enable adequate characterization of the pavement as damage developed.

- Laser profilometer, measuring surface profile. Measurements are taken at each station.
- Thermocouples, measuring pavement temperature (at Stations 4 and 12) and ambient temperature at one-hour intervals during HVS operation.

Air temperatures were measured at a weather station approximately 150 m (500 ft) from the test section and recorded at the same intervals as the thermocouples.

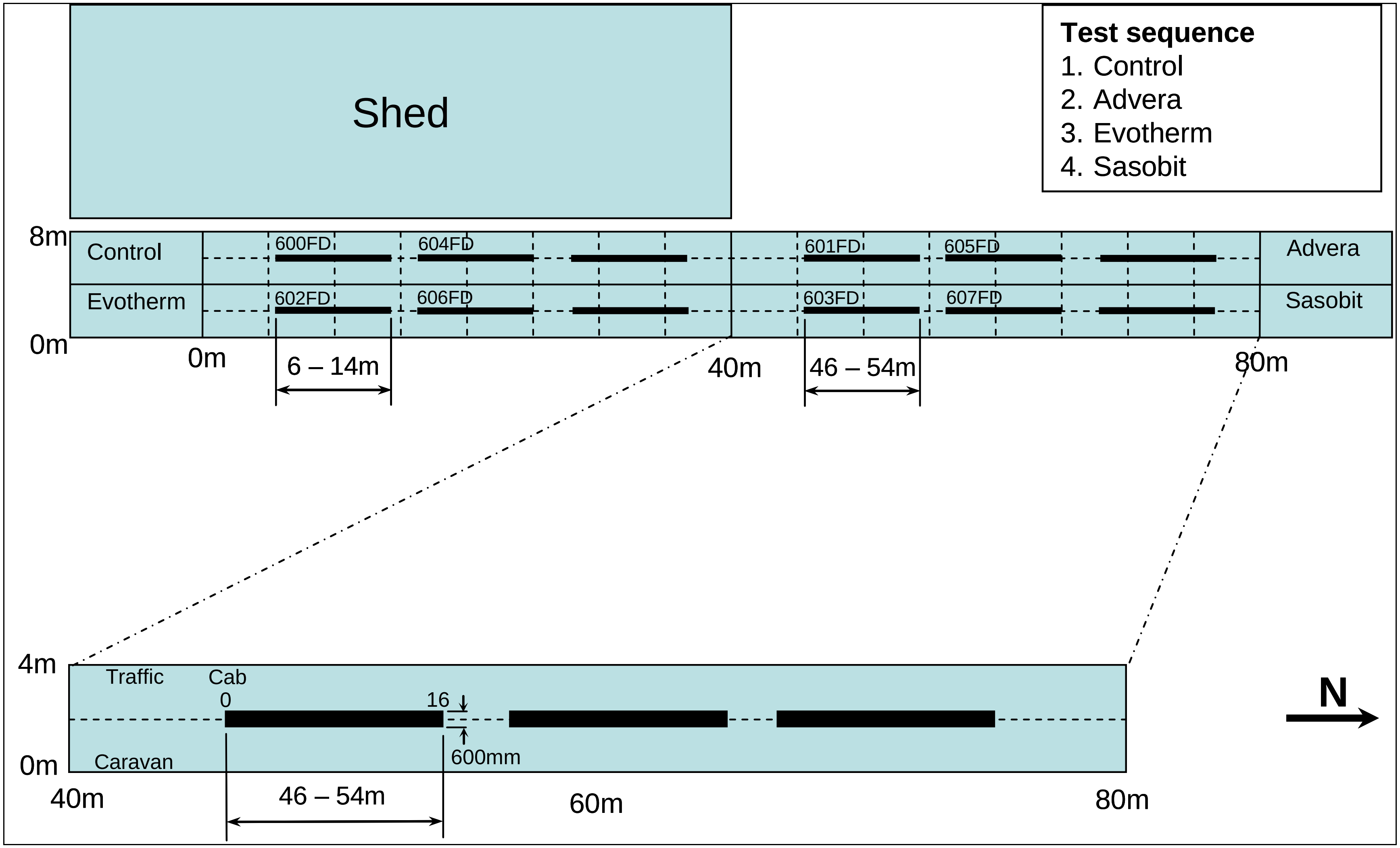


Figure 3.1: Layout of test track and HVS test sections.

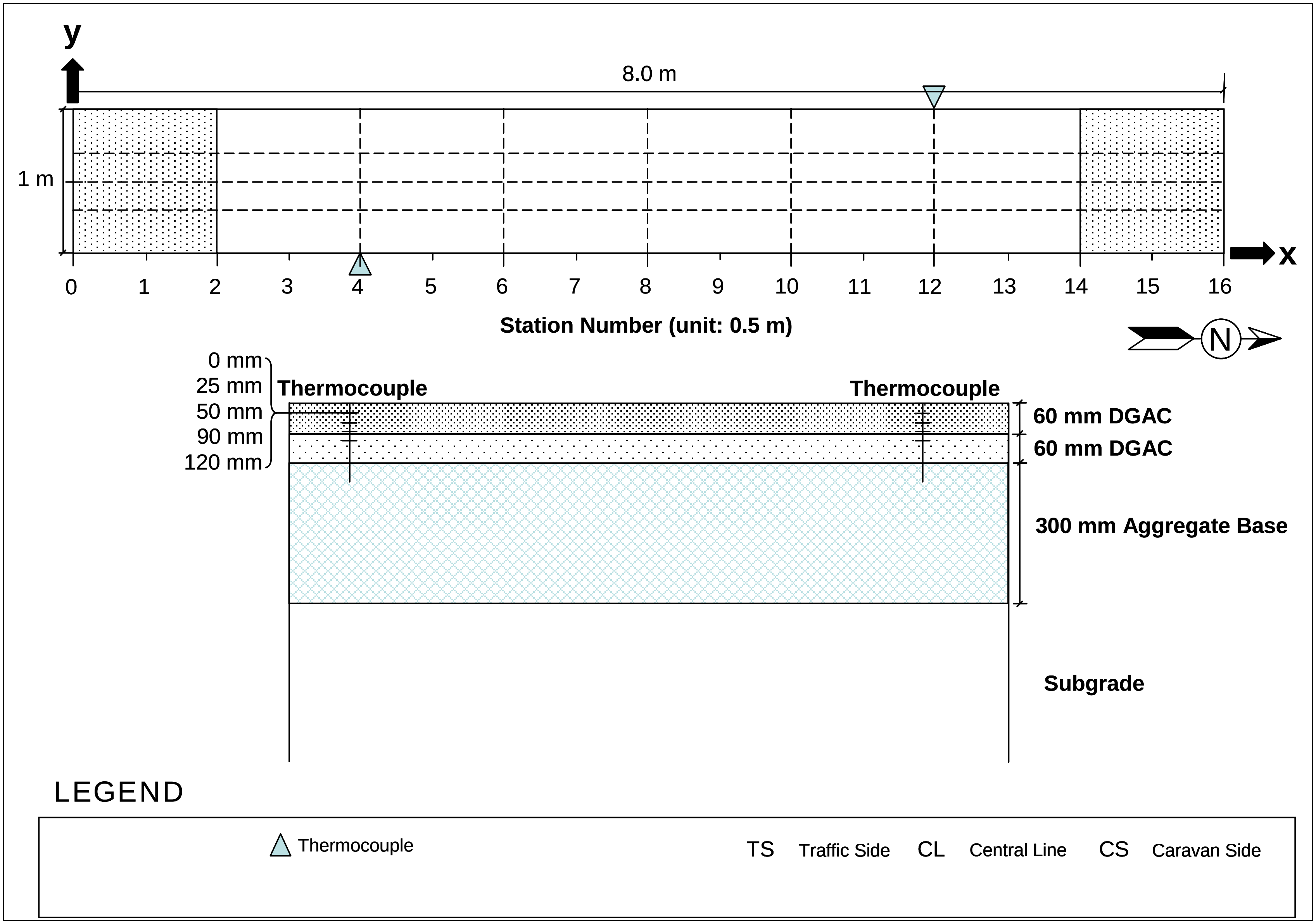


Figure 3.2: Phase 1 test section layout and location of thermocouples.

Surface and in-depth deflections were not measured. Surface deflection cannot be measured with the Road Surface Deflectometer (RSD) on rutted pavements. In-depth deflection measured with Multi-Depth Deflectometers (MDD) was not possible due to difficulties with installing and anchoring the instruments in the bedrock.

3.5 HVS Test Criteria

3.5.1 Test Section Failure Criteria

An average maximum rut of 12.5 mm (0.5 in.) over the full monitored section (Station 3 to Station 13) was set as the failure criteria for the experiment.

3.5.2 Environmental Conditions

The pavement temperature at 50 mm (2.0 in.) was maintained at 50°C±4°C (122°F±7°F) to assess rutting potential under typical pavement conditions. Infrared heaters inside a temperature control chamber *(7)* were used to maintain the pavement temperature. The pavement surface received no direct rainfall as it was protected by the temperature control chamber. The sections were tested predominantly during the wet season (October through March), however, it is unlikely that any water entered the pavement structure due to the confinement on both sides of the test track.

3.5.3 Test Duration

HVS trafficking on each section was initiated and completed as shown in Table 3.1.

Table 3.1: Test Duration for Phase 1 HVS Rutting Tests

Section	Overlay	Start Date	Finish Date	Repetitions
600FD	Control	10/17/2007	11/21/2007	195,000
601FD	Advera	12/02/2007	12/19/2007	170,000
602FD	Evotherm	12/29/2007	01/22/2008	185,000
603FD	Sasobit	01/28/2008	03/08/2008	285,000

3.5.4 Loading Program

The HVS loading program for each section is summarized in Table 3.2. Equivalent Standard Axle Loads (ESALs) were determined using the following Caltrans conversion (Equation 3.1):

$$\text{ESALS} = (\text{axle load}/18000)^{4.2} \qquad \textbf{(3.1)}$$

All trafficking was carried out with a dual-wheel configuration, using radial truck tires (Goodyear G159 - 11R22.5- steel belt radial) inflated to a pressure of 720 kPa (104 psi), in a channelized, unidirectional loading mode.

Load was checked with a portable weigh-in-motion pad at the beginning of each test and after each load change.

Table 3.2: Summary of HVS Loading Program

Section	Overlay	Wheel Load[1] (kN)	Repetitions	ESALs[2]
600FD	Control	40 60	185,000 10,000	239,900
601FD	Advera	40	170,000	170,000
602FD	Evotherm	40	185,000	185,000
603FD	Sasobit	40 60	185,000 100,000	734,014
		Total	**835,000**	**1,328,914**

[1] 40 kN = 9,000 lb. 60 kN = 13,500 lb
[2] ESAL: Equivalent Standard Axle Load

4. PHASE 1 HVS TEST DATA SUMMARY

4.1 Introduction

This chapter provides a summary of the data collected from the four HVS tests (Sections 600FD through 603FD) and a brief discussion of the first-level analysis. Data collected included rainfall, air temperatures inside and outside the temperature control chamber, pavement temperatures, and surface permanent deformation.

Pavement temperatures were controlled using the temperature control chamber. Both air (inside and outside the temperature box) and pavement temperatures were monitored and recorded hourly during the entire loading period. In assessing rutting performance, the temperature at the bottom of the asphalt concrete and the temperature gradient are two important controlling temperature parameters influencing the stiffness of the asphalt concrete and are used to compute plastic strain. Permanent deformation at the pavement surface (rutting) was monitored with the Laser Profilometer. In-depth permanent deformation at various depths within the pavement was not monitored due to the presence of bedrock and associated difficulties with the installation of Multi-Depth Deflectometers. The following rut parameters were determined from these measurements, as illustrated in Figure 4.1:

- Average maximum rut depth,
- Average deformation,
- Location and magnitude of the maximum rut depth, and
- Rate of rut development.

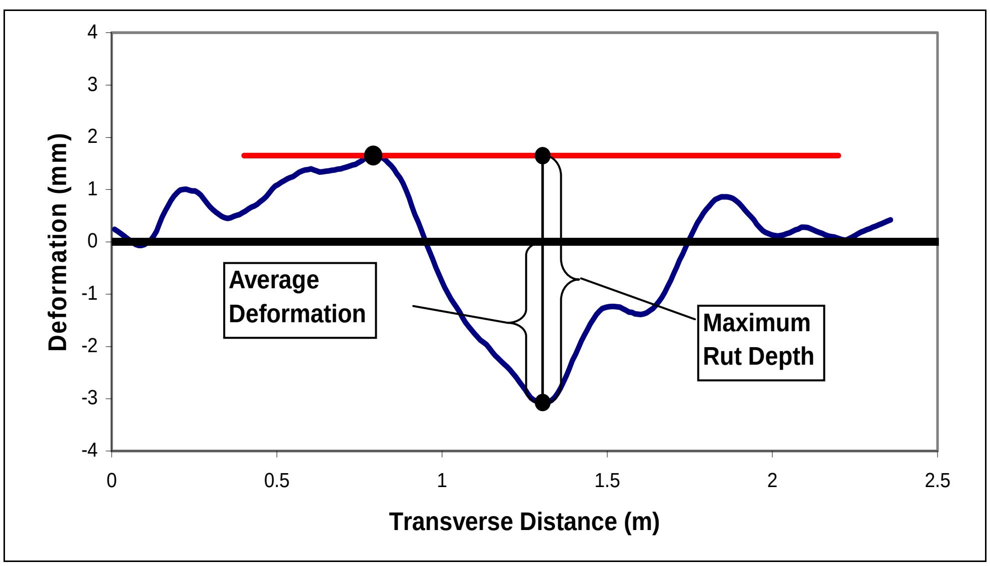


Figure 4.1: Illustration of maximum rut depth and average deformation of a leveled profile.

The Laser Profilometer provides sufficient information to evaluate the evolution of permanent surface deformation of the entire test section at various loading stages. The rut depth figures in this report show the average values over the entire section (Stations 3 through 13) as well as values for half sections between Stations 3 and 8 and Stations 9 and 13. These two additional data series were plotted to illustrate any differences along the length of the section. The precise nature of the permanent deformation will only be determined after a forensic investigation (test pits and cores) on each section when all testing on the test track has been completed.

The data from each HVS test is presented separately, with the presentation of each test following the same format. Data plots are presented on the same scale to facilitate comparisons of performance. Interpretation of the data in terms of pavement performance will be discussed in a separate second-level analysis report.

4.2 Rainfall

Figure 4.2 shows the monthly rainfall data from August 2007 through March 2008 as measured at the weather station close to the test track. Rainfall was measured during all four Phase 1 HVS tests, with one significant rainfall event of 120.4 mm (4.7 in.) in a 24 hour period recorded during testing on Section 602FD. Rainfall in excess of 25 mm (1.0 in.) was recorded on three days during testing on Sections 601FD (one day) and 602FD (two days).

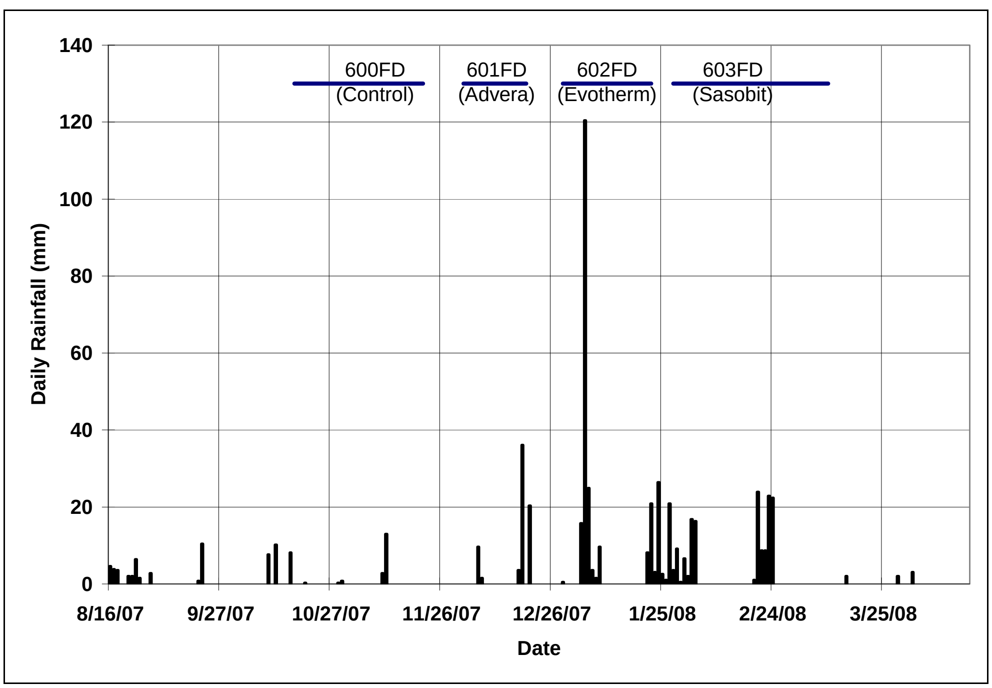


Figure 4.2: Measured rainfall during Phase 1 HVS testing.

4.3 Section 600FD: Control

4.3.1 Test Summary

Loading commenced on October 17, 2007, and ended on November 21, 2007. A total of 195,000 load repetitions were applied and 47 datasets were collected. Testing was interrupted for eight days (November 2, 2008 through November 10, 2008) due to a carriage computer malfunction caused by the high testing temperatures. Modifications were made to the equipment to prevent a recurrence; however, intermittent problems were experienced for the remainder of the test. Trafficking was also stopped at 155,000 repetitions in order to increase the pavement temperature to 55°C (131°F). The HVS loading history for Section 600FD is shown in Figure 4.3.

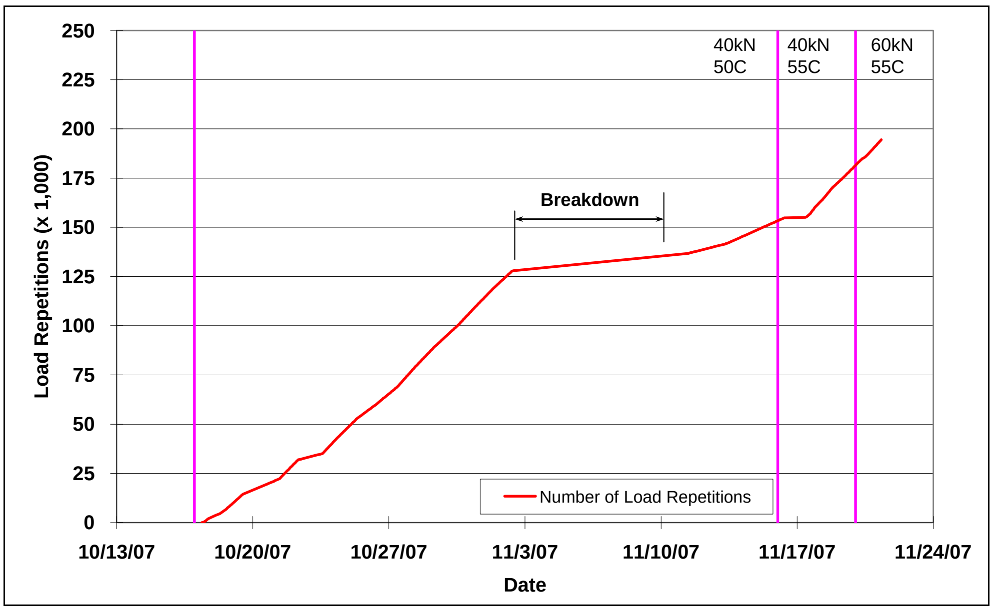


Figure 4.3: 600FD: Load history.

4.3.2 Outside Air Temperatures

Outside air temperatures are summarized in Figure 4.4. Vertical error bars on each point on the graph show daily temperature range. Temperatures ranged from 4.5°C to 36.6°C (40°F to 98°F) during the course of HVS testing, with a daily average of 14.2°C (58°F), an average minimum of 9.3°C (49°F), and an average maximum of 22.7°C (73°F).

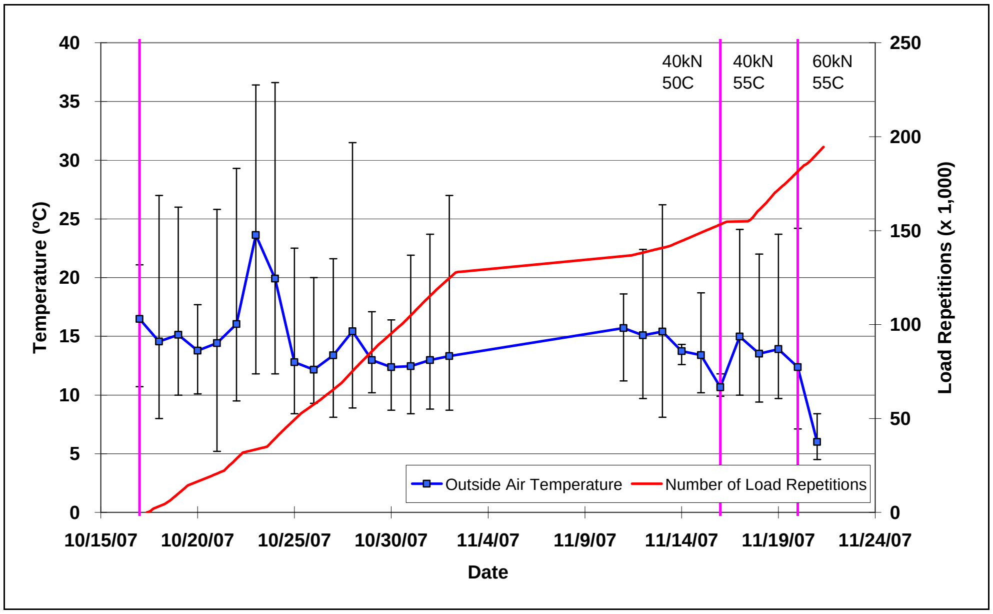


Figure 4.4: 600FD: Daily average outside air temperatures.

4.3.3 Air Temperatures in the Temperature Control Unit

During the test, air temperatures inside the temperature control chamber ranged from 25°C to 56°C (77°F to 133°F) with an average of 52°C (126°F) and standard deviation of 2.6°C (4.6°F). For the first 155,000 repetitions, the air temperature was adjusted to maintain a pavement temperature of 50°C±4°C (122°F±7°F), which is expected to promote rutting damage. The project failure criteria of 12.5 mm (0.5 in.) was not achieved at this point, and the air temperatures were therefore increased to raise the pavement temperature to 55°C±4°C (131°F±7°F), in line with the test plan, to further hasten the rate of rutting. The daily average air temperatures recorded in the temperature control unit, calculated from the hourly temperatures recorded during HVS operation, are shown in Figure 4.5. Vertical errors bars on each point on the graph show daily temperature range.

4.3.4 Temperatures in the Asphalt Concrete Layers

Daily averages of the surface and in-depth temperatures of the asphalt concrete layers are listed in Table 4.1 and shown in Figure 4.6. Pavement temperatures decreased slightly with increasing depth in the pavement, which was expected as there is usually a thermal gradient between the top and bottom of the asphalt concrete pavement layers.

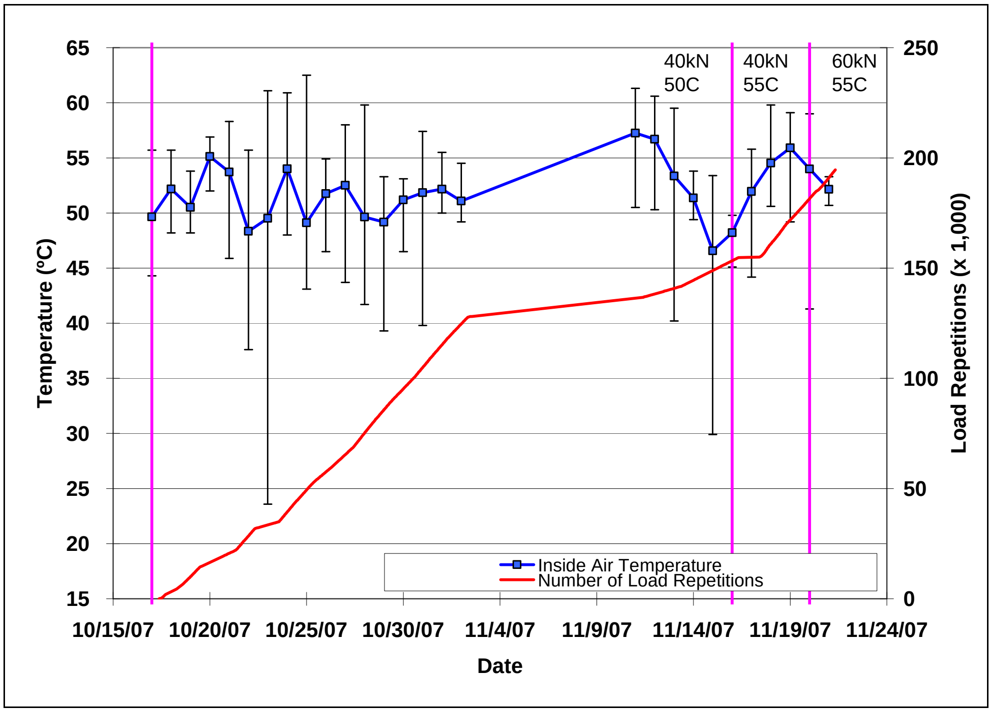


Figure 4.5: 600FD: Daily average inside air temperatures.

Table 4.1: 600FD: Temperature Summary for Air and Pavement

Temperature	Average (°C)	Std Dev (°C)	Average (°F)	Std Dev (°F)
	0 to 155,000 Repetitions			
Outside air	14.6	2.7	58.3	4.9
Inside air	51.5	2.6	124.7	4.7
Pavement surface	50.8	2.0	123.4	3.6
- 25 mm below surface	49.8	1.9	121.6	3.4
- 50 mm below surface	49.0	2.0	120.2	3.6
- 90 mm below surface	47.4	2.2	117.3	4.0
- 120 mm below surface	42.2	2.3	108.0	4.1
	155,000 to 195,000 Repetitions			
Outside air	11.5	3.7	52.7	6.7
Inside air	54.2	1.6	129.6	2.9
Pavement surface	56.0	0.9	132.8	1.6
- 25 mm below surface	55.4	0.2	131.7	0.4
- 50 mm below surface	54.9	0.1	130.8	0.2
- 90 mm below surface	53.5	0.1	128.3	0.2
- 120 mm below surface	52.4	0.3	126.3	0.5

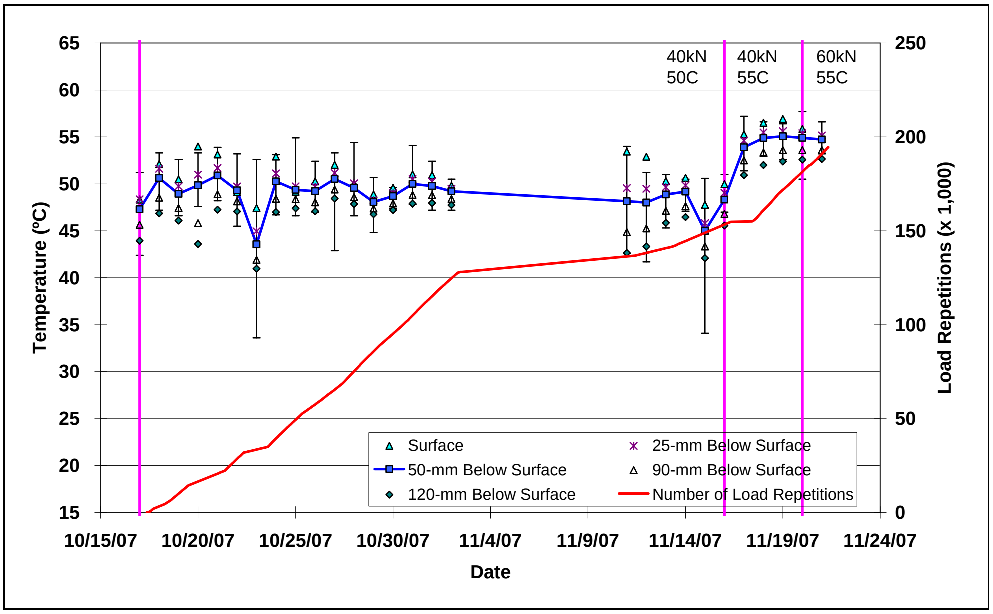


Figure 4.6: 600FD: Daily average temperatures at pavement surface and various depths.

4.3.5 Permanent Surface Deformation (Rutting)

Figure 4.7 shows the average transverse cross section measured with the Laser Profilometer at various stages of the test. This plot clearly shows the increase in rutting and deformation over the duration of the test.

During HVS testing, rutting usually occurs at a high rate initially, and then it typically diminishes as trafficking progresses until reaching a steady state. This initial phase is referred to as the “embedment” phase. Figure 4.8 and Figure 4.9 show the development of permanent deformation (average maximum rut and average deformation, respectively) with load repetitions as measured with the Laser Profilometer for the test section, with an embedment phase only apparent at the beginning of the experiment (i.e., first 25,000 repetitions). Error bars on the average reading indicate that there was very little variation along the length of the section. Figure 4.10 shows a contour plot of the pavement surface at the end of the test (195,000 repetitions), also indicating minimal variation along the section. A slightly deeper rut was recorded in one of the wheel tracks, which was attributed to the positioning of the HVS on the crossfall on the section. After completion of trafficking, the average maximum rut depth and the average deformation were 12.4 mm (0.49 in.) and 6.3 mm (0.25 in.), respectively. The maximum rut depth measured on the section was 14.0 mm (0.55 in.), recorded at Station 9.

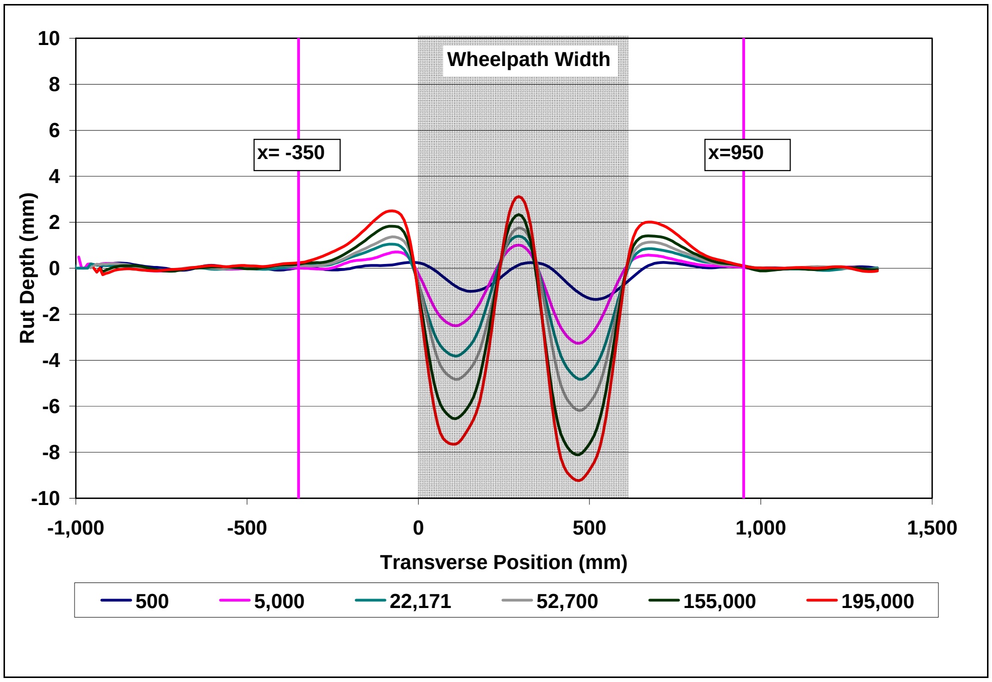


Figure 4.7: 600FD: Profilometer cross section at various load repetitions.

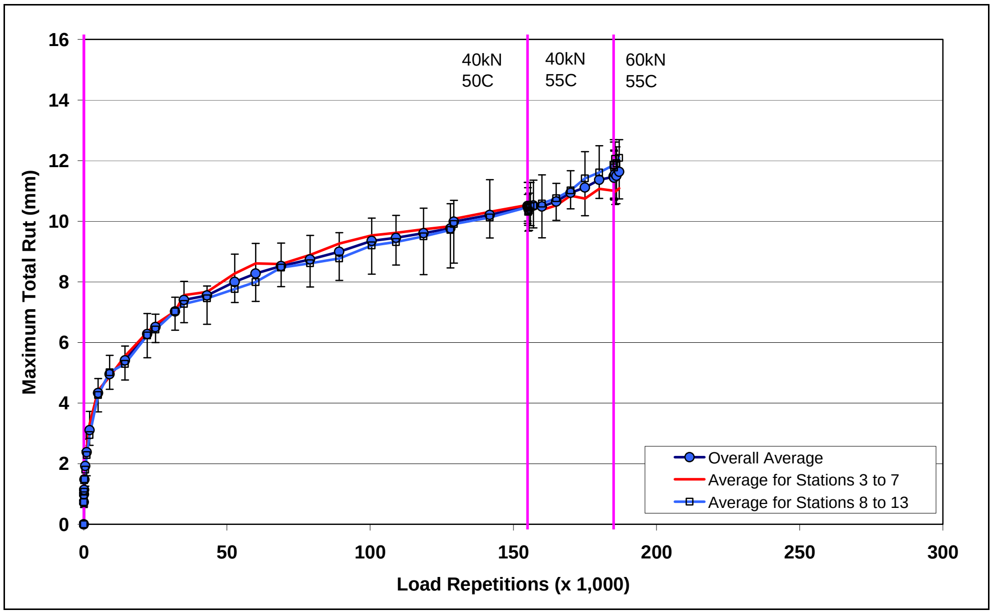


Figure 4.8: 600FD: Average maximum rut.

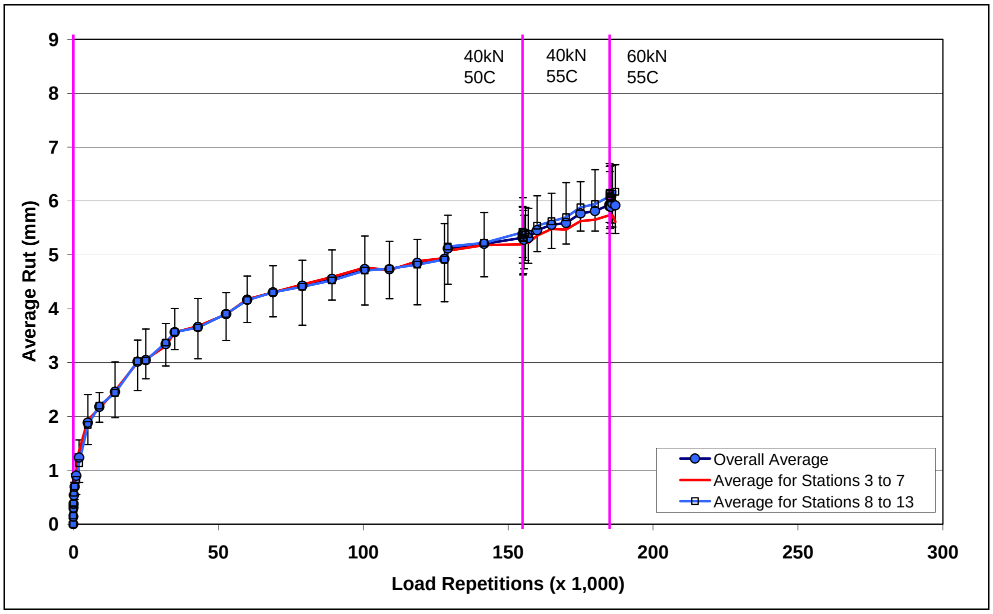


Figure 4.9: 600FD: Average deformation.

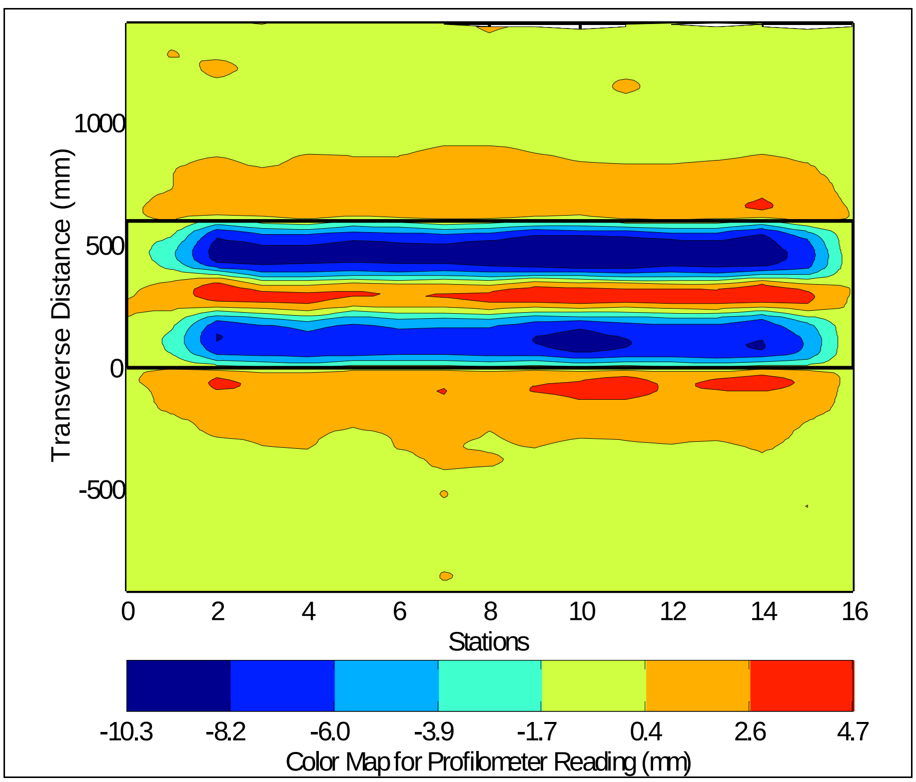


Figure 4.10: 600FD: Contour plot of permanent surface deformation at end of test.

4.3.6 Visual Inspection

Apart from rutting, no other distress was recorded on the section. Figure 4.11 is a photograph taken of the surface at the end of the test.

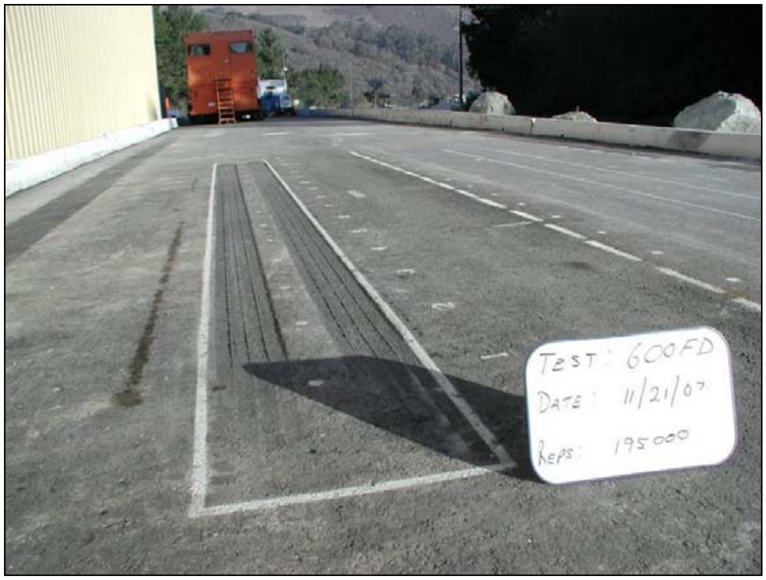


Figure 4.11: 600FD: Section photograph at test completion.

4.4 Section 601FD: Advera

4.4.1 Test Summary

Loading commenced on December 2, 2007, and ended on December 19, 2007. A total of 170,000 load repetitions were applied and 27 datasets were collected. Fewer load repetitions (25,000 less) were applied compared to the Control. The HVS loading history for Section 601FD is shown in Figure 4.12. No breakdowns occurred during this test.

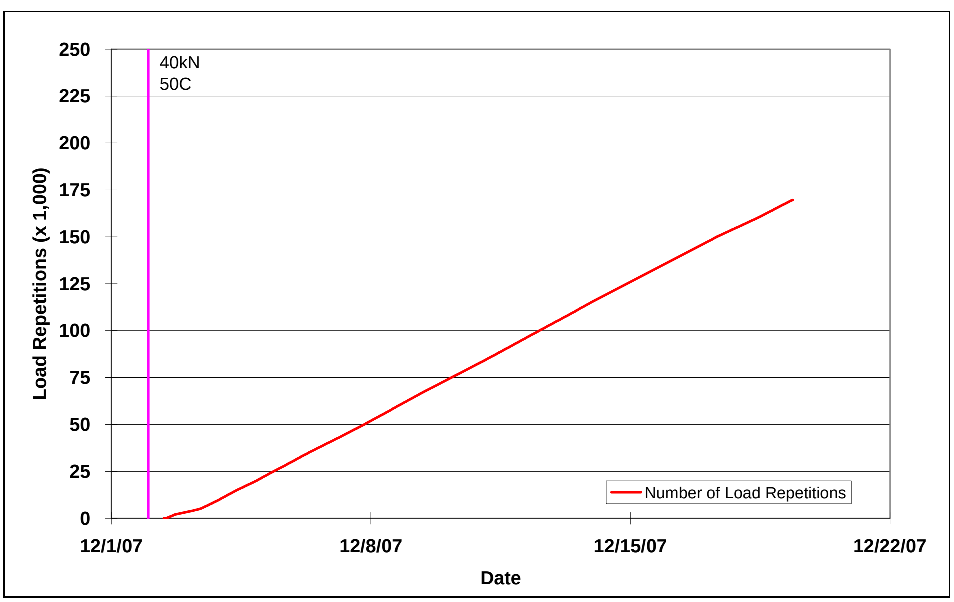


Figure 4.12: 601FD: Load history.

4.4.2 Outside Air Temperatures

Outside air temperatures are summarized in Figure 4.13. Vertical error bars on each point on the graph show daily temperature range. Temperatures ranged from -0.3°C to 25.5°C (32°F to 78°F) during the course of HVS testing, with a daily average of 9.4°C (49°F), an average minimum of 4.4°C (39°F), and an average maximum of 17.4°C (63°F). Outside air temperatures were considerably cooler during testing on Section 601FD compared to Section 600FD (daily average 5°C [9°F] cooler).

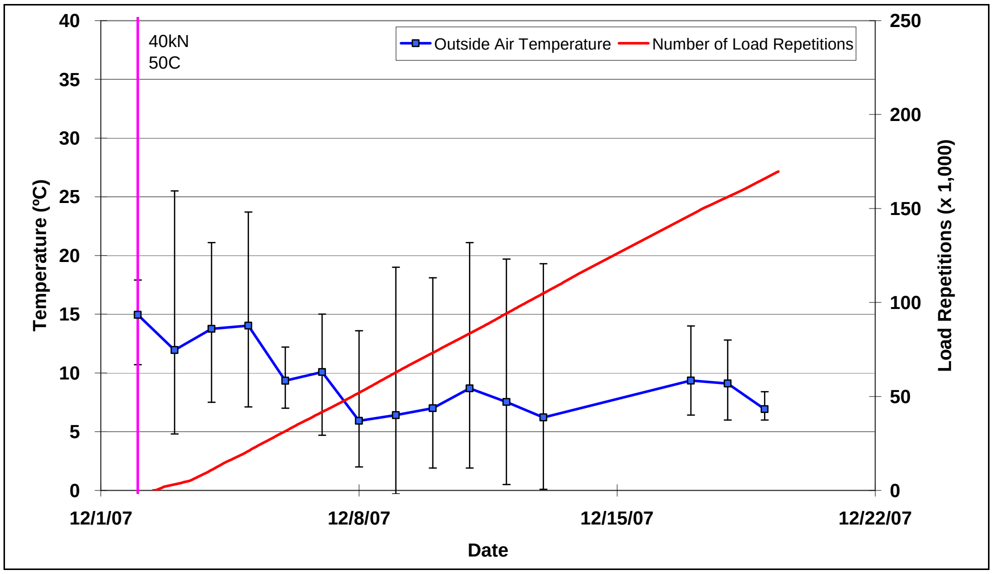


Figure 4.13: 601FD: Daily average outside air temperatures.

4.4.3 Air Temperatures in the Temperature Control Unit

During the test, the measured air temperatures inside the temperature control chamber ranged from 25°C to 56°C (77°F to 133°F) with an average of 36°C (97°F) and standard deviation of 6.2°C (11.2°F). The daily average air temperatures recorded in the temperature control unit, calculated from the hourly temperatures recorded during HVS operation, are shown in Figure 4.14. Vertical errors bars on each point on the graph show daily temperature range. These inside air temperatures do not correspond with the outside air temperatures and pavement temperatures or to the inside air temperatures measured on the other tests. This anomaly was attributed to air leaks next to the sensors. The recorded pavement temperatures discussed in Section 4.4.4 indicate that the inside air temperatures were adjusted appropriately to maintain a pavement temperature of 50°C±4°C (122°F±7°F) for the entire test. The temperature was not raised to 55°C±4°C (131°F±7°F) after 155,000 repetitions due the average maximum rut depth being close to the failure criteria at this point.

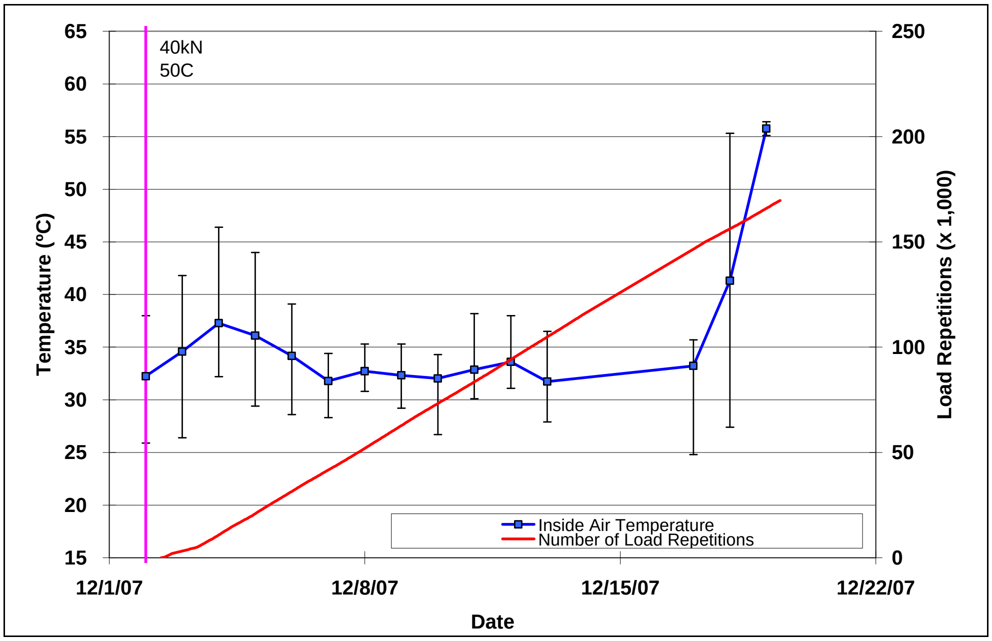


Figure 4.14: 601FD: Daily average inside air temperatures.

(This data is potentially incorrect due to sensor malfunction or temperature chamber air leak close to the sensor.)

4.4.4 Temperatures in the Asphalt Concrete Layers

Daily averages of the surface and in-depth temperatures of the asphalt concrete layers are listed in Table 4.2 and shown in Figure 4.15. Pavement temperatures decreased slightly with increasing depth in the pavement, as expected. Average pavement temperatures at all depths of Section 601FD were similar to those recorded on the Control, despite lower outside temperatures.

Table 4.2: 601FD: Temperature Summary for Air and Pavement

Temperature	601FD			600FD
	Average (°C)	Std Dev (°C)	Average (°F)	Average (°C)
	0 to 170,000 Repetitions			
Outside air	9.4	3.0	48.9	14.6
Inside air	35.5	6.2	95.9	51.5
Pavement surface	50.8	2.1	123.4	50.8
- 25 mm below surface	50.7	2.0	123.3	49.8
- 50 mm below surface	50.1	1.9	122.2	49.0
- 90 mm below surface	48.6	1.7	119.5	47.4
- 120 mm below surface	47.6	1.6	117.7	42.2

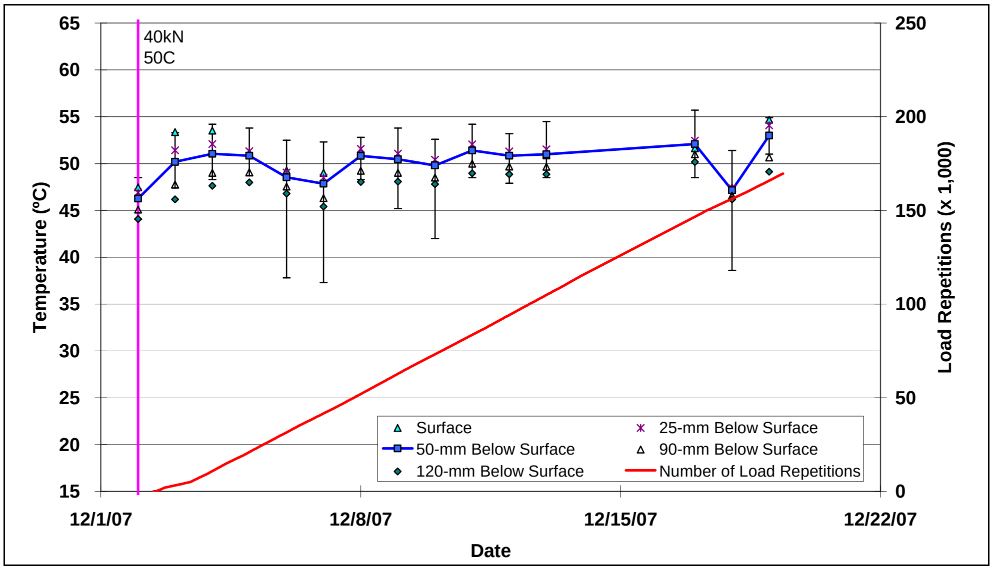


Figure 4.15: 601FD: Daily average temperatures at pavement surface and various depths.

4.4.5 Permanent Surface Deformation (Rutting)

Figure 4.16 shows the average transverse cross section measured with the Laser Profilometer at various stages of the test. This plot clearly shows the increase in rutting and deformation over the duration of the test.

Figure 4.17 and Figure 4.18 show the development of permanent deformation (average maximum rut and average deformation, respectively) with load repetitions as measured with the Laser Profilometer for the test section. Results for the Control section (Section 600FD) are also shown for comparative purposes. Although the embedment phase was of similar duration for both sections, a slightly deeper average maximum rut was recorded on Section 601FD at the end of the embedment phase (6.5 mm [0.26 in.]) compared to the control (4.3 mm [0.17 in.]). This was attributed to less oxidation of the binder, and consequent lower stiffness of the asphalt, because of the lower production and construction temperatures. The slightly higher moisture content of the Advera mix, compared to that of the Control, may also have had an influence. Thereafter a similar rutting behavior trend was recorded, although the average deformation (down rut) on Section 601FD was lower than that recorded on the Control. Error bars on the average reading indicate that there was very little variation along the length of the section. Figure 4.19 shows a contour plot of the pavement surface at the end of the test (170,000 repetitions), also indicating minimal variation along the section. After completion of trafficking, the average maximum rut depth and the average deformation were 12.4 mm (0.49 in.) and 5.0 mm (0.20 in.), respectively. The maximum rut depth measured on the section was 13.3 mm (0.52 in.) recorded at Station 10.

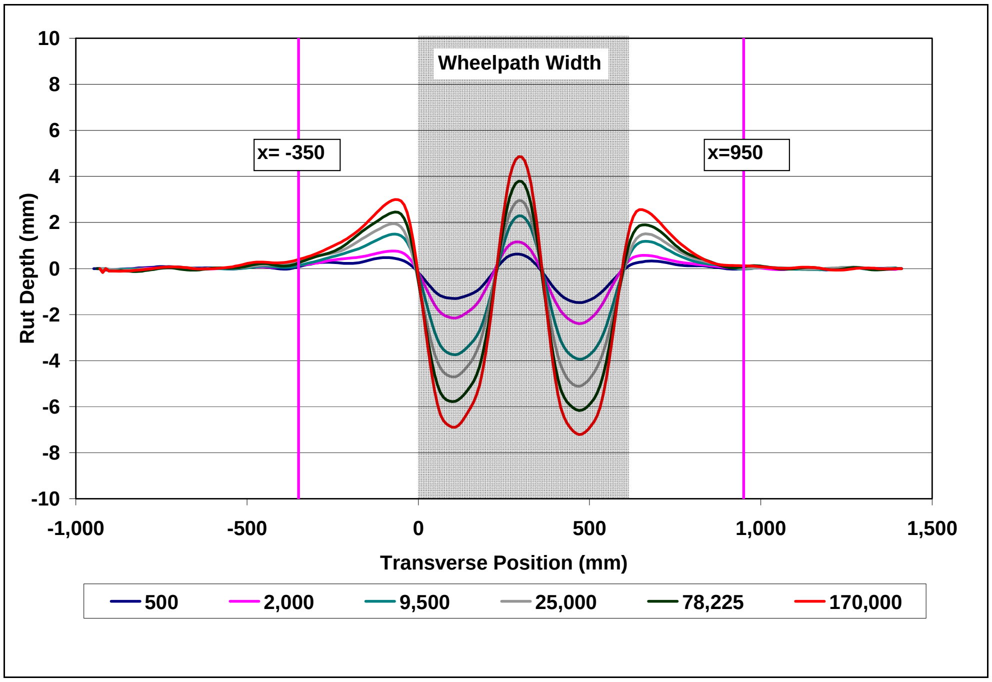


Figure 4.16: 601FD: Profilometer cross section at various load repetitions.

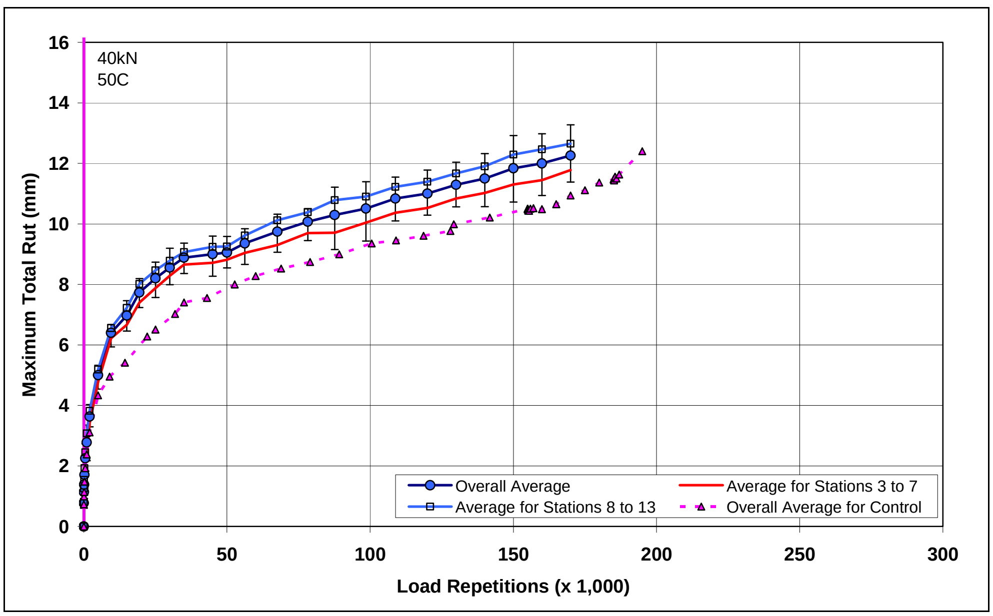


Figure 4.17: 601FD: Average maximum rut.

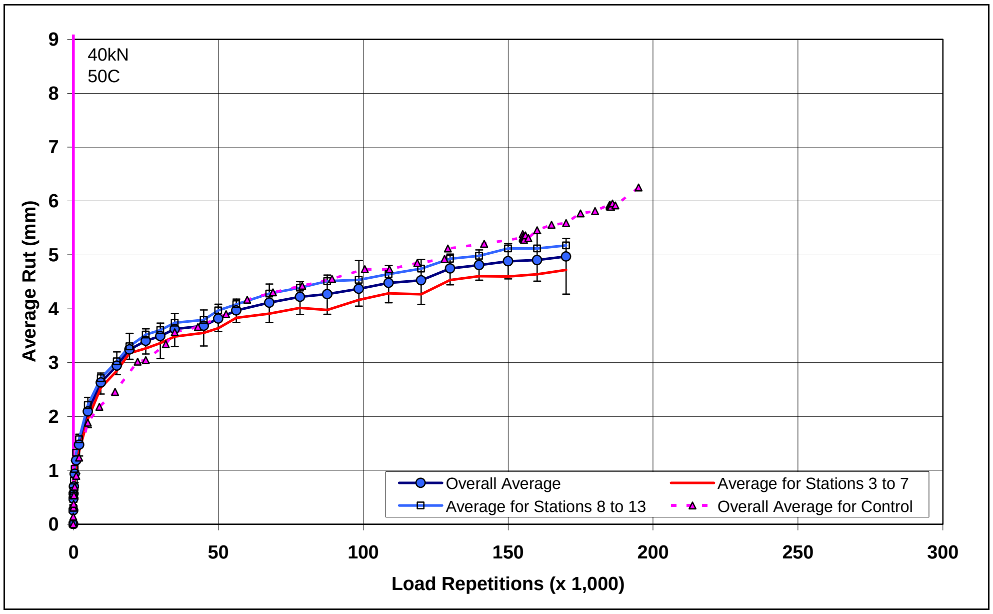


Figure 4.18: 601FD: Average deformation.

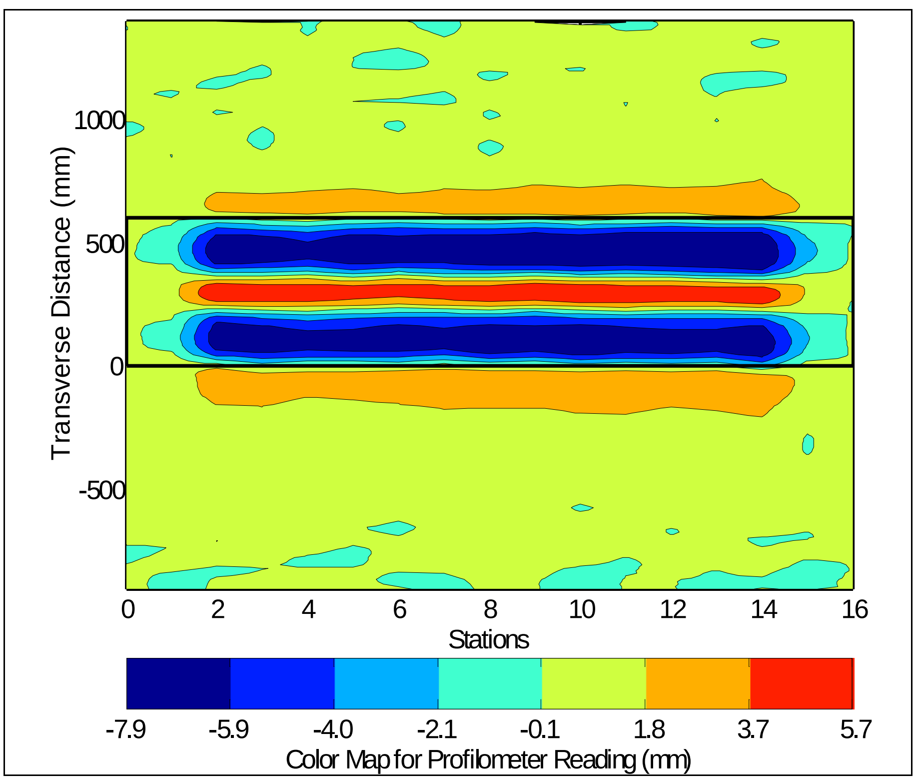


Figure 4.19: 601FD: Contour plot of permanent surface deformation at end of test.

4.4.6 Visual Inspection

Apart from rutting, no other distress was recorded on the section, which was similar in appearance to the Control (Figure 4.11) at the end of testing.

4.5 Section 602FD: Evotherm

4.5.1 Test Summary

Loading commenced on December 29, 2007, and ended on January 22, 2008. A total of 185,000 load repetitions were applied and 35 datasets were collected. Fewer load repetitions (10,000 less) were applied compared to the Control. The HVS loading history for Section 602FD is shown in Figure 4.20. A three-day carriage computer breakdown occurred during the first week of testing. Trafficking was also stopped at 155,000 repetitions while the pavement temperature was raised.

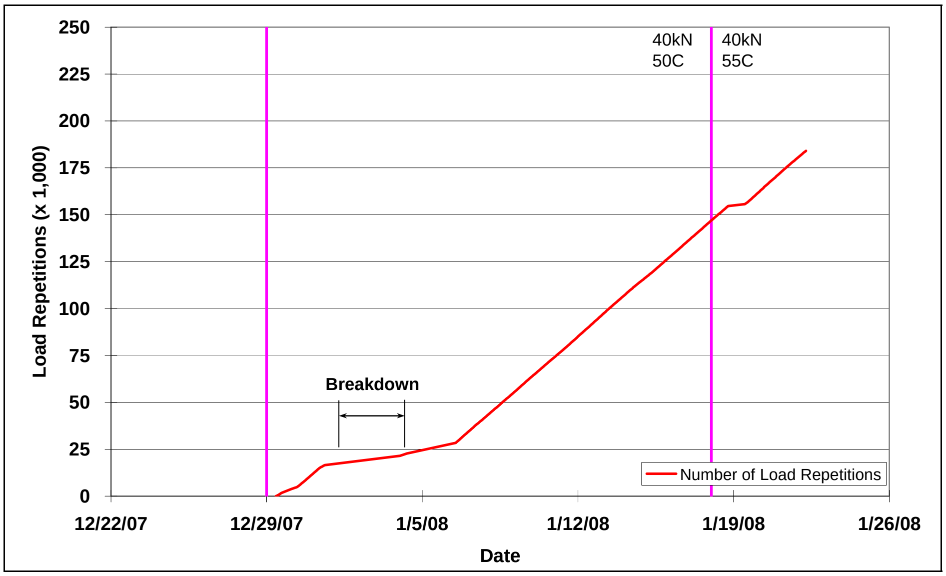


Figure 4.20 602FD: Load history.

4.5.2 Outside Air Temperatures

Outside air temperatures are summarized in Figure 4.21. Vertical error bars on each point on the graph show daily temperature range. Temperatures ranged from 0.2°C to 19.5°C (32°F to 67°F) during the course of HVS testing, with a daily average of 8.6°C (47°F), an average minimum of 4.5°C (39°F), and an average maximum of 14.0°C (57°F). Outside air temperatures were considerably cooler during testing on Section 602FD compared to those during testing of Section 600FD (daily average 5.6°C [10°F] cooler).

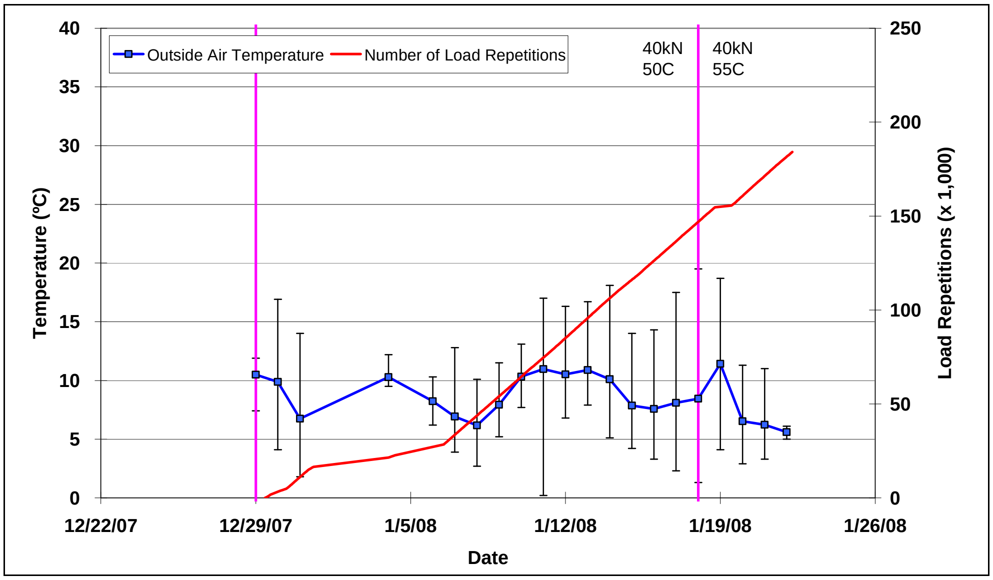


Figure 4.21: 602FD: Daily average outside air temperatures.

4.5.3 Air Temperatures in the Temperature Control Unit

During the test, air temperatures inside the temperature control chamber ranged from 28°C to 56°C (82°F to 133°F) with an average of 48°C (118°F) and standard deviation of 2.5°C (4.5°F). The air temperature was adjusted to maintain a pavement temperature of 50°C±4°C (122°F±7°F) for the first 155,000 repetitions, and 55°C±4°C (131°F±7°F) thereafter. The daily average air temperatures recorded in the temperature control unit, calculated from the hourly temperatures recorded during HVS operation, are shown in Figure 4.22. Vertical errors bars on each point on the graph show daily temperature range.

4.5.4 Temperatures in the Asphalt Concrete Layers

Daily averages of the surface and in-depth temperatures of the asphalt concrete layers are listed in Table 4.3 and shown in Figure 4.23. Pavement temperatures decreased slightly with increasing depth in the pavement, as expected. Average pavement temperatures at all depths of Section 602FD were similar to those recorded on the Control, despite lower outside temperatures.

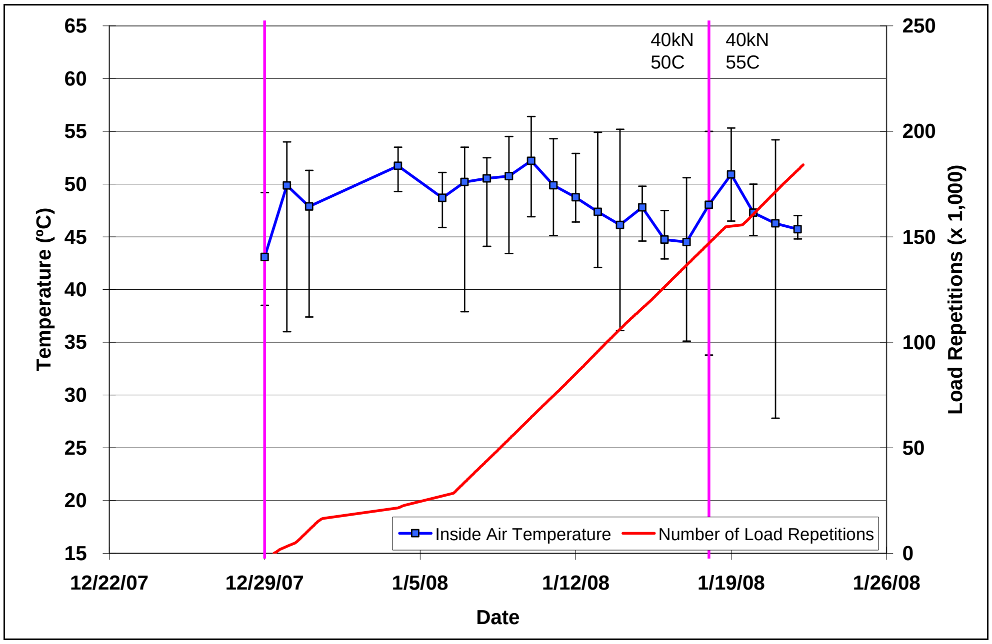


Figure 4.22: 602FD: Daily average inside air temperatures.

Table 4.3: 602FD: Temperature Summary for Air and Pavement

Temperature	602FD			600FD
	Average (°C)	Std Dev (°C)	Average (°F)	Average (°C)
	0 to 155,000 Repetitions			
Outside air	9.0	1.7	48.2	14.6
Inside air	48.5	2.6	119.3	51.5
Pavement surface	51.8	2.0	125.2	50.8
- 25 mm below surface	51.1	1.9	124.0	49.8
- 50 mm below surface	50.2	1.7	122.4	49.0
- 90 mm below surface	48.5	1.8	119.3	47.4
- 120 mm below surface	47.5	1.7	117.5	42.2
	155,000 to 185,000 Repetitions			
Outside air	6.1	0.5	43.0	11.5
Inside air	46.4	0.8	115.5	54.2
Pavement surface	56.3	1.1	133.3	56.0
- 25 mm below surface	55.5	0.9	131.9	55.4
- 50 mm below surface	54.5	0.8	130.1	54.9
- 90 mm below surface	52.8	0.3	127.0	53.5
- 120 mm below surface	51.7	0.2	125.1	52.4

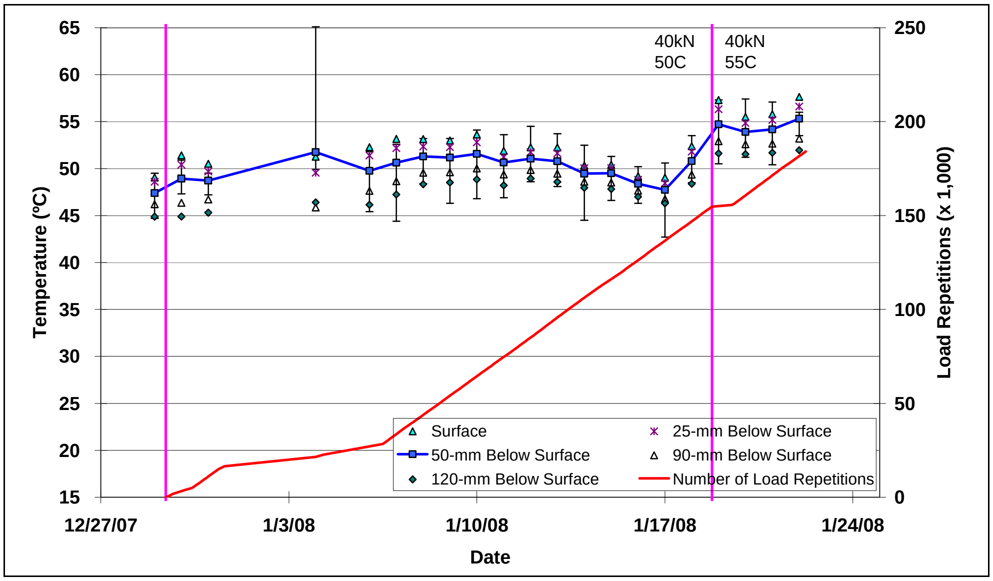


Figure 4.23: 602FD: Daily average temperatures at pavement surface and various depths.

4.5.5 Permanent Surface Deformation (Rutting)

Figure 4.24 shows the average transverse cross section measured with the Laser Profilometer at various stages of the test. This plot clearly shows the increase in rutting and deformation over the duration of the test.

Figure 4.25 and Figure 4.26 show the development of permanent deformation (average maximum rut and average deformation, respectively) with load repetitions as measured with the Laser Profilometer for the test section. Results for the Control section (Section 600FD) are also shown for comparative purposes. Although the embedment phase was of comparable duration for the both sections, a slightly deeper average maximum rut was recorded on Section 602FD at the end of the embedment phase (6.1 mm [0.24 in.]) compared to the control (4.3 mm [0.17 in.]), similar to that recorded on Section 601FD. This was again attributed to less oxidation of the binder, and consequent lower stiffness of the asphalt, because of the lower production and construction temperatures. The slightly higher moisture content of the Evotherm mix, compared to that of the Control, may also have had an influence. Thereafter similar rutting behavior trends were recorded. Error bars on the average reading indicate that there was some variation along the length of the section. Figure 4.27 shows a contour plot of the pavement surface at the end of the test (185,000 repetitions), also indicating a slightly deeper average maximum rut on one half of the section. After completion of trafficking, the average maximum rut depth and the average deformation were 12.5 mm (0.5 in.) and 7.0 mm (0.28 in.), respectively. The maximum rut depth measured on the section was 14.1 mm (0.56 in.) recorded at Station 11.

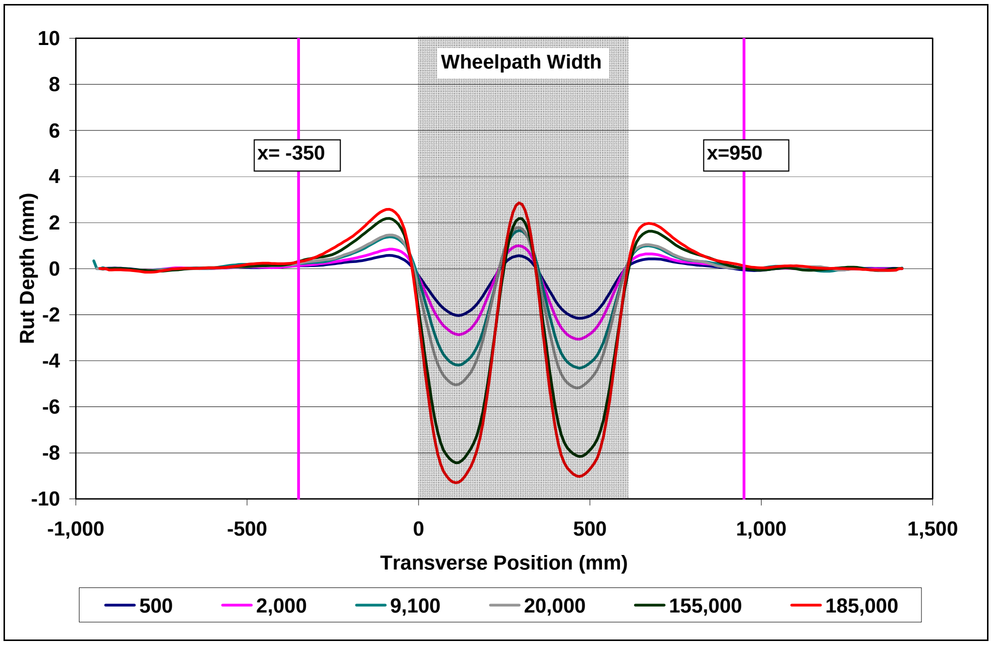


Figure 4.24: 602FD: Profilometer cross section at various load repetitions.

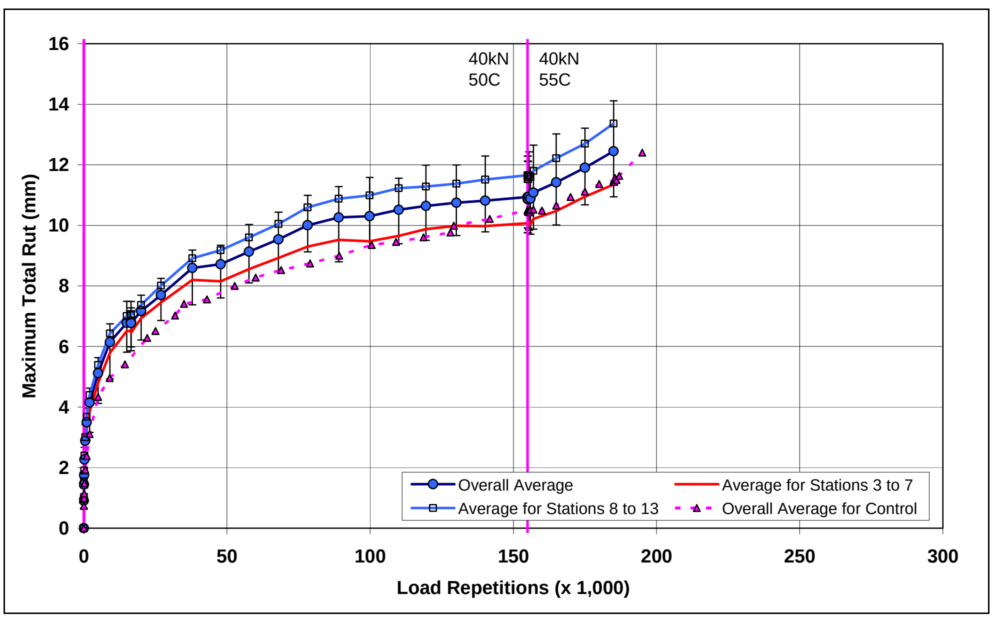


Figure 4.25: 602FD: Average maximum rut.

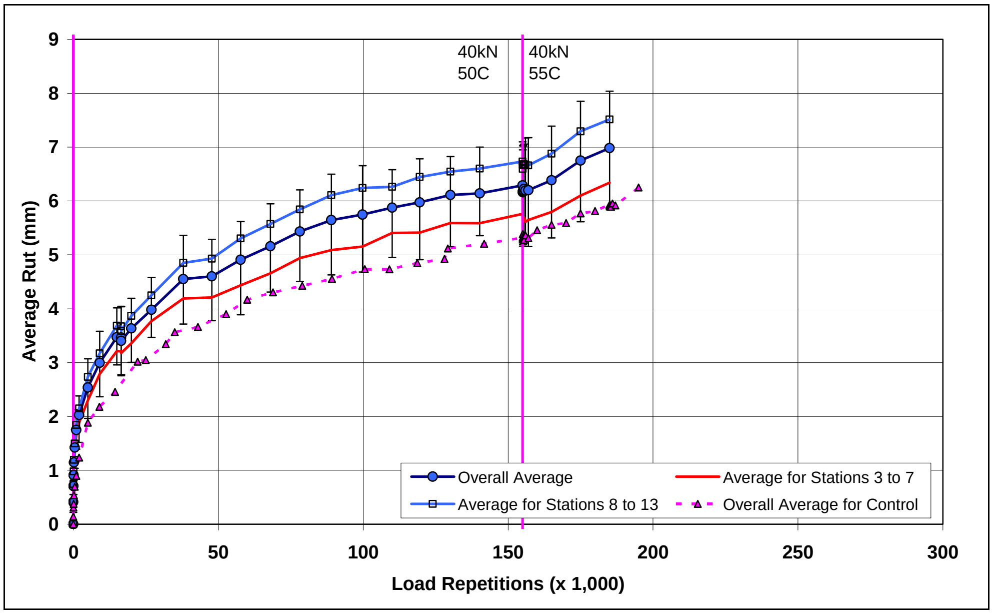


Figure 4.26: 602FD: Average deformation.

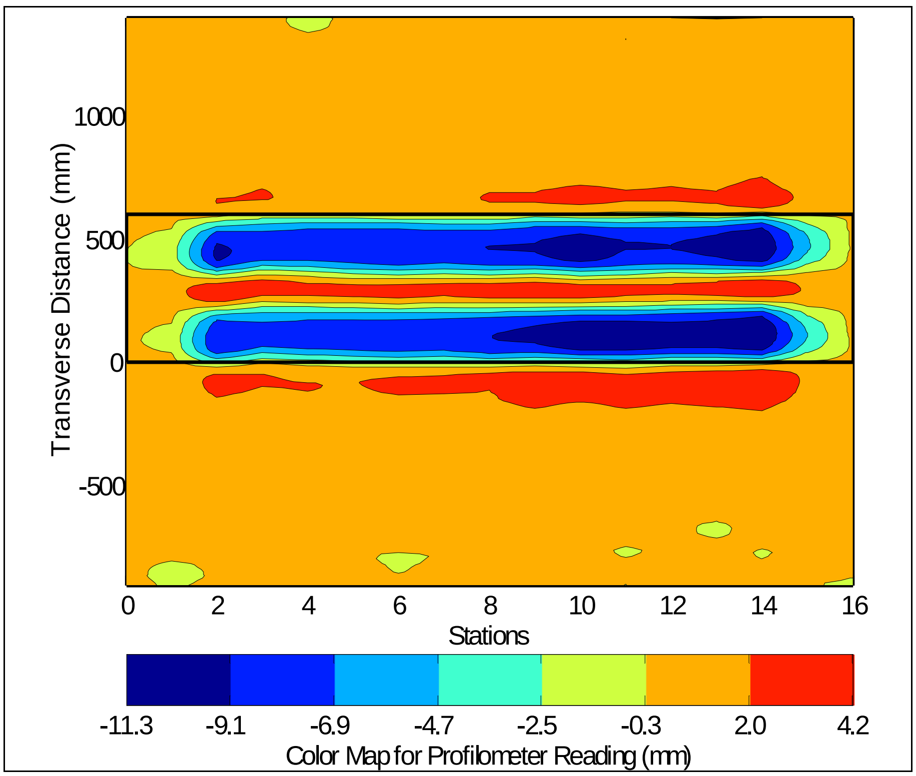


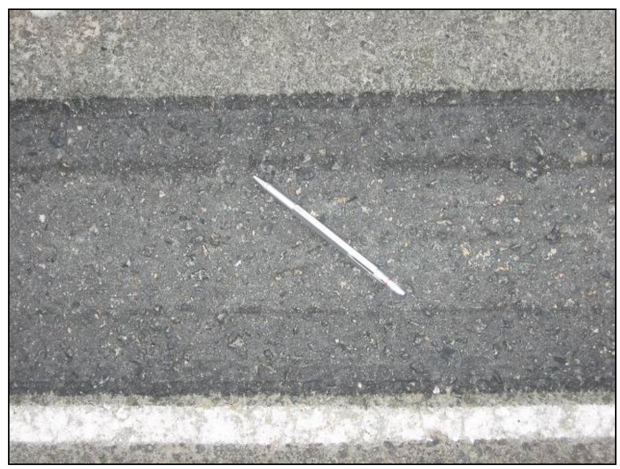
Figure 4.27: 602FD: Contour plot of permanent surface deformation at end of test.

4.5.6 Visual Inspection

Apart from rutting, no other distress was recorded on the section. Figure 4.28 shows photograph taken of the surface at the end of the test. It is interesting to note that the elevated pavement temperatures kept the section dry for a number of days during light rainfall after the environmental chamber was removed.

Test section after trafficking

Dry area from retained heat.



Close up of trafficked area

Figure 4.28: 602FD: Section photographs at test completion.

4.6 Section 603FD: Sasobit

4.6.1 Test Summary

Loading commenced on January 28, 2008, and ended on March 28, 2008. A total of 285,000 load repetitions were applied and 53 datasets were collected. Considerably more load repetitions (90,000) were applied to Section 603FD compared to the Control; however, the failure criteria of 12.5 mm (0.5 in.) was not reached and testing was halted in the interest of completing the study. The high rut resistance was probably attributed to the lower binder content of the mix used on this test section (see Section 2.6) and therefore direct performance comparisons between the Control and this test section are not possible. The HVS loading history for Section 603FD is shown in Figure 4.29. A five-day carriage computer breakdown occurred during the first week of testing. Trafficking was also stopped at 155,000 repetitions while the pavement temperature was raised, per the requirements of the test plan.

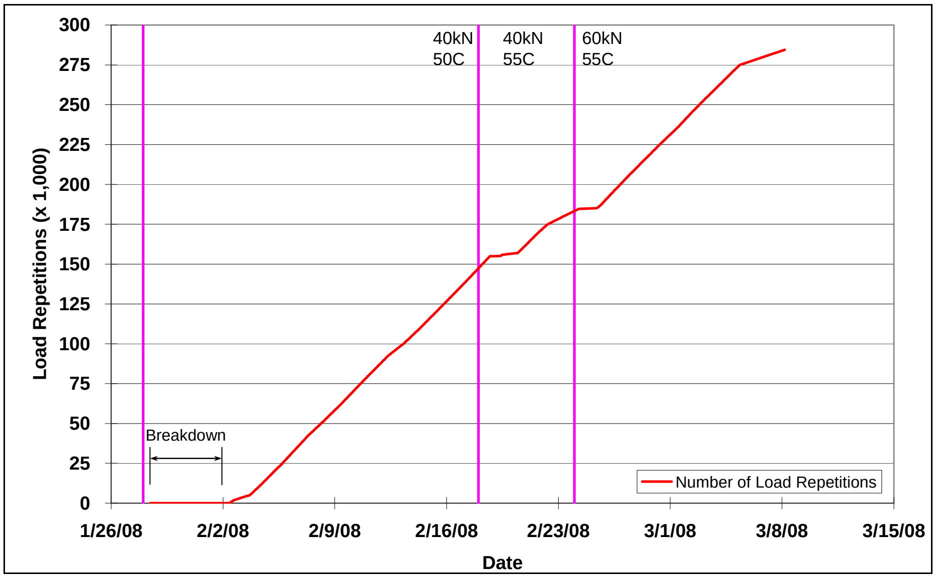


Figure 4.29: 603FD: Load history.

HVS trafficking on the section was continued for a further 100,000 repetitions (i.e., to 385,000 repetitions) to collect additional data for calibration of mechanistic-empirical design models. Results from this additional testing are not discussed in this report.

4.6.2 Outside Air Temperatures

Outside air temperatures are summarized in Figure 4.30. Vertical error bars on each point on the graph show daily temperature range. Temperatures ranged from 0°C to 33.9°C (32°F to 93°F) during the course of HVS testing, with a daily average of 10.5°C (51°F), an average minimum of 5.2°C (41°F), and an average maximum of 19.5°C (67°F). Average outside air temperatures were somewhat warmer during testing on Section 603FD compared to those during testing on Section 600FD (daily average of 5.63°C [10°F] warmer).

4.6.3 Air Temperatures in the Temperature Control Unit

During the test, air temperatures inside the temperature control chamber ranged from 18.1°C to 64.7°C (65°F to 148°F) with an average of 50°C (122°F) and standard deviation of 3.7°C (6.7°F). The air temperature was adjusted to maintain a pavement temperature of 50°C±4°C (122°F±7°F) for the first

155,000 repetitions, and 55°C±4°C (131°F±7°F) thereafter. The daily average air temperatures recorded in the temperature control unit, calculated from the hourly temperatures recorded during HVS operation, are shown in Figure 4.31. Vertical errors bars on each point on the graph show daily temperature range.

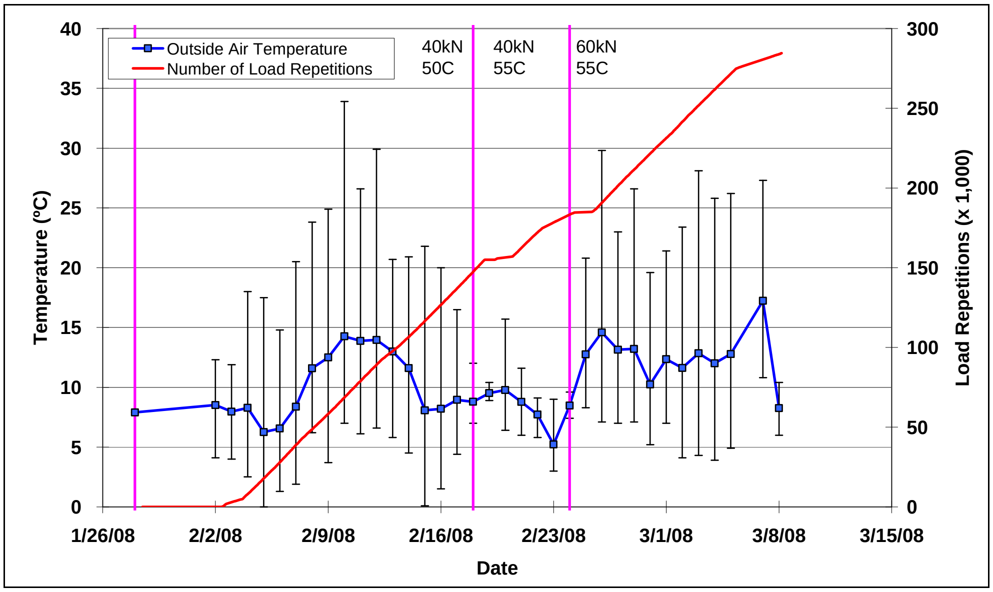


Figure 4.30: 603FD: Daily average outside air temperatures.

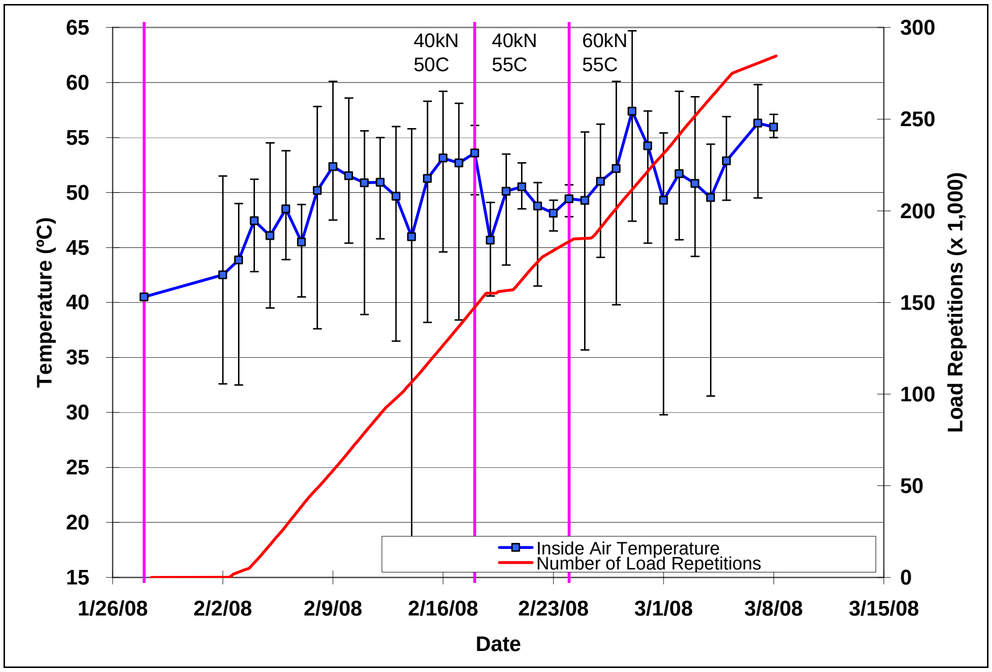


Figure 4.31: 603FD: Daily average inside air temperatures.

4.6.4 Temperatures in the Asphalt Concrete Layers

Daily averages of the surface and in-depth temperatures of the asphalt concrete layers are listed in Table 4.4 and shown in Figure 4.32. Pavement temperatures decreased slightly with increasing depth in the pavement, as expected. Average pavement temperatures at all depths on Section 603FD were similar to those recorded on the Control, despite lower outside temperatures during the first 155,000 repetitions.

Table 4.4: 603FD: Temperature Summary for Air and Pavement

Temperature	603FD			600FD
	Average (°C)	Std Dev (°C)	Average (°F)	Average (°C)
	0 to 155,000 Repetitions			
Outside air	9.9	2.7	49.8	14.6
Inside air	48.7	3.9	119.7	51.5
Pavement surface	50.9	2.5	123.6	50.8
- 25 mm below surface	49.9	2.2	121.8	49.8
- 50 mm below surface	49.8	2.3	121.6	49.0
- 90 mm below surface	48.4	2.1	119.1	47.4
- 120 mm below surface	47.6	2.2	117.7	42.2
	155,000 to 285,000 Repetitions			
Outside air	11.9	2.9	53.4	11.5
Inside air	51.3	3.1	124.3	54.2
Pavement surface	55.5	1.5	131.9	56.0
- 25 mm below surface	54.5	1.3	130.1	55.4
- 50 mm below surface	54.5	1.3	130.1	54.9
- 90 mm below surface	53.2	1.1	127.8	53.5
- 120 mm below surface	52.4	1.1	126.3	52.4

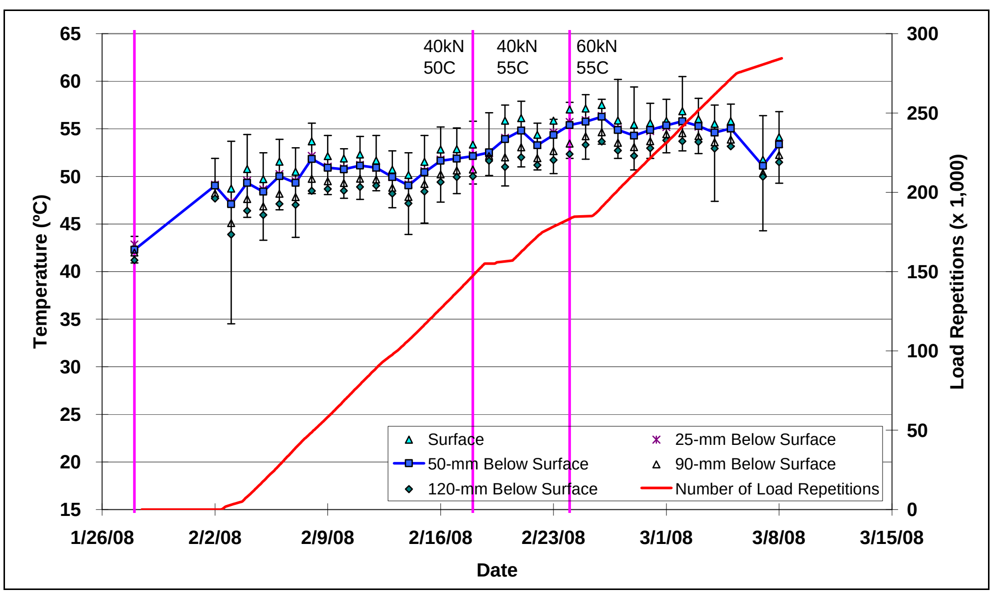


Figure 4.32: 603FD: Daily average temperatures at pavement surface and various depths.

4.6.5 Permanent Surface Deformation (Rutting)

Figure 4.33 shows the average transverse cross section measured with the Laser Profilometer at various stages of the test. This plot clearly shows the increase in rutting and deformation over the duration of the test.

Figure 4.34 and Figure 4.35 show the development of permanent deformation (average maximum rut and average deformation, respectively) with load repetitions as measured with the Laser Profilometer for the test section. Results for the Control section (Section 600FD) are also shown for general comparative purposes, although no direct comparisons can be made given the difference in binder content between the two sections. The embedment phase was of shorter duration on Section 603FD compared to Section 600FD, and the average maximum rut was shallower (2.0 mm [0.1 in.]) compared to the Control (4.3 mm [0.17 in.]). After the embedment phase, the rate of increase in average maximum rut depth was significantly slower than that recorded on the Control. Error bars on the average reading indicate that there was very little variation along the length of the section. Figure 4.36 shows a contour plot of the pavement surface at the end of the test (285,000 repetitions) that indicates a slightly deeper average maximum rut under one of the tires. This was attributed to the crossfall on the test track. At the time that trafficking was halted, the average maximum rut depth and the average deformation were 7.8 mm (0.31 in.) and 4.6 mm (0.18 in.), respectively. The maximum rut depth measured on the section was 8.8 mm (0.35 in.), recorded at Station 13.

4.6.6 Visual Inspection

Apart from rutting, no other distress was recorded on the section. Appearance was similar to that shown in Figure 4.11 and Figure 4.28.

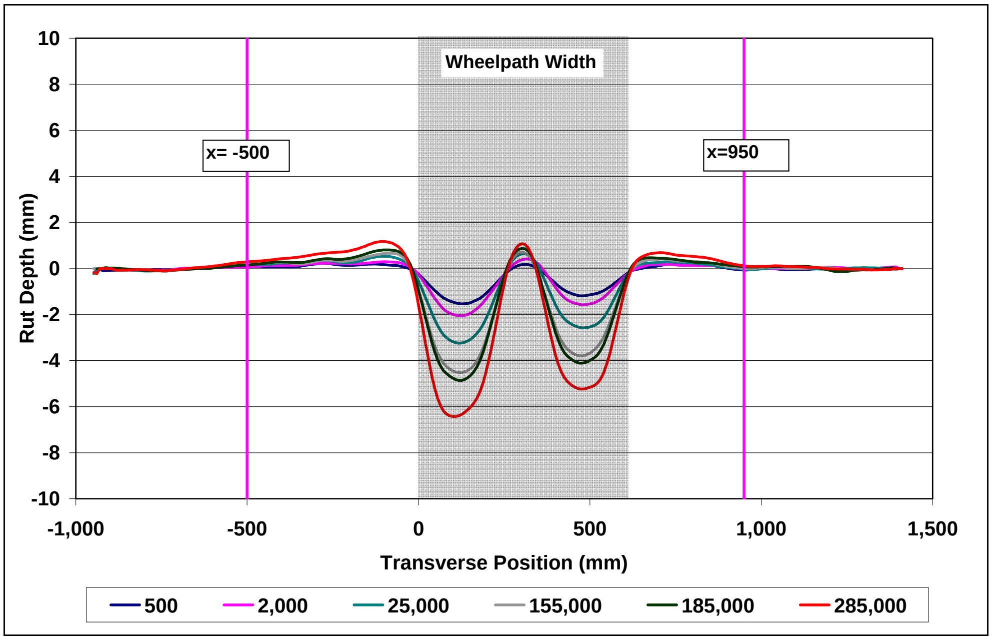


Figure 4.33: 603FD: Profilometer cross section at various load repetitions.

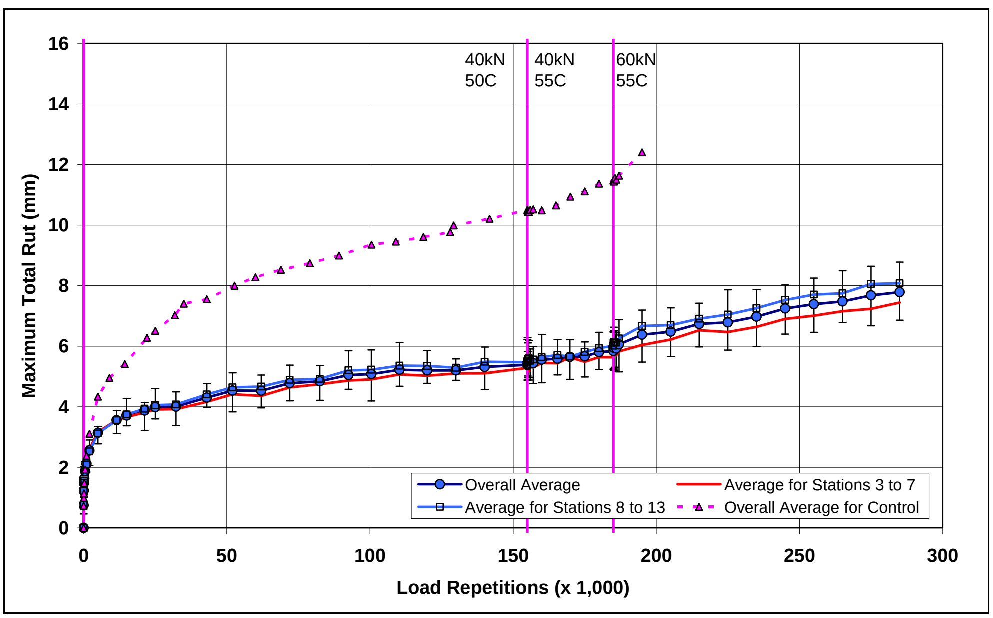


Figure 4.34: 603FD: Average maximum rut.

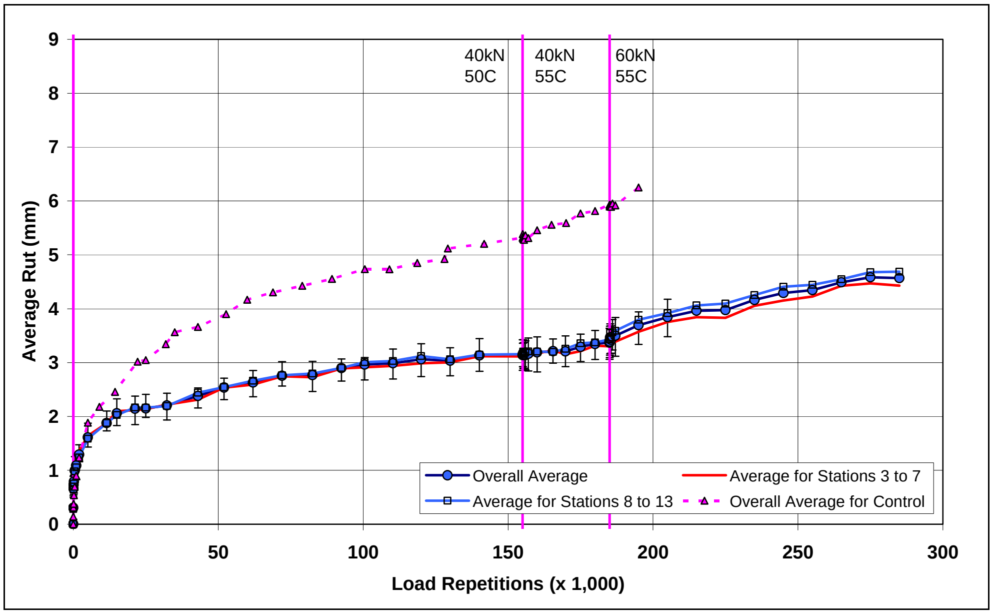


Figure 4.35: 603FD: Average deformation.

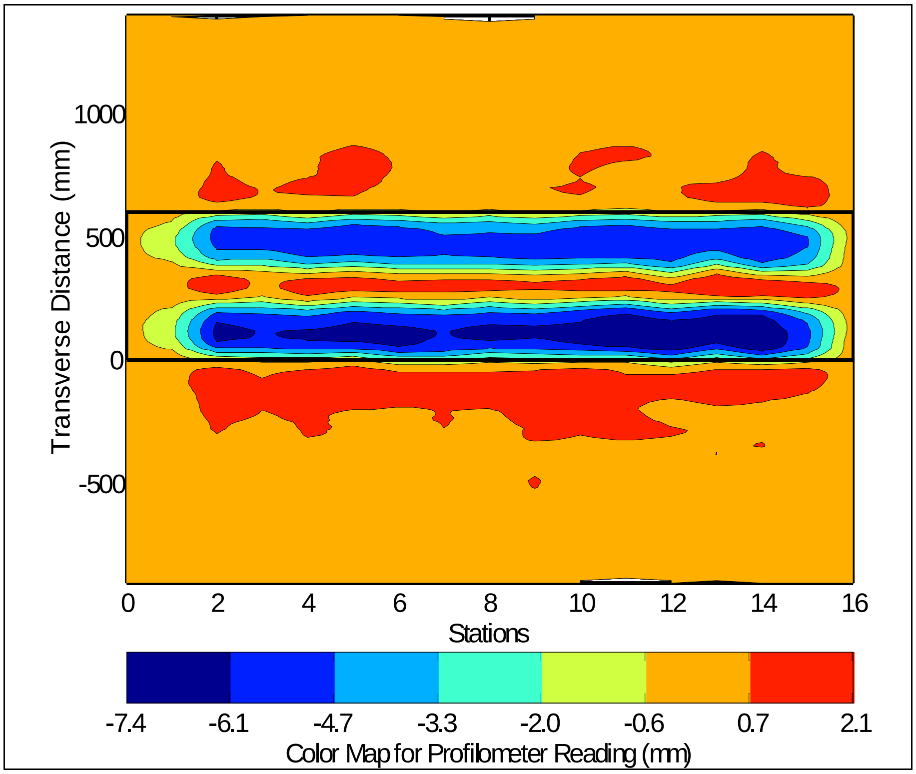


Figure 4.36: 603FD: Contour plot of permanent surface deformation at end of test.

4.7 Test Summary

Testing on the four sections was started in the fall of 2007 and ended in the spring of 2008. The duration of the tests on the four sections varied from 170,000 load repetitions (Section 602FD) to 285,000 load repetitions (Section 603FD). A range of daily average temperatures was therefore experienced; however, the pavement temperatures remained constant throughout HVS trafficking.

Rutting behavior for the four sections is compared in Figure 4.37 (average maximum rut) and Figure 4.38 (average deformation). The duration of the embedment phases on Sections 601FD and 602FD (Advera and Evotherm) were similar to that of Section 600FD (Control), however, the depth of the ruts at the end of the embedment phases on these two sections was slightly higher than the Control. In both instances, this was attributed to less oxidation of the binder during mix production because of the lower plant temperatures. The slightly higher moisture contents of these mixes may also have had an influence. Rutting behavior on the warm-mix sections followed trends similar to that of the Control in terms of rut rate (rutting per load repetition) after the embedment phase. The performance of Section 603FD cannot be compared with the other three sections given that the binder content of its mix was significantly lower.

Based on these results, it can be concluded that the use of any of the three warm-mix asphalt additives tested in this experiment and subsequent compaction of the mix at lower temperatures will not significantly influence the rutting performance of the mix.

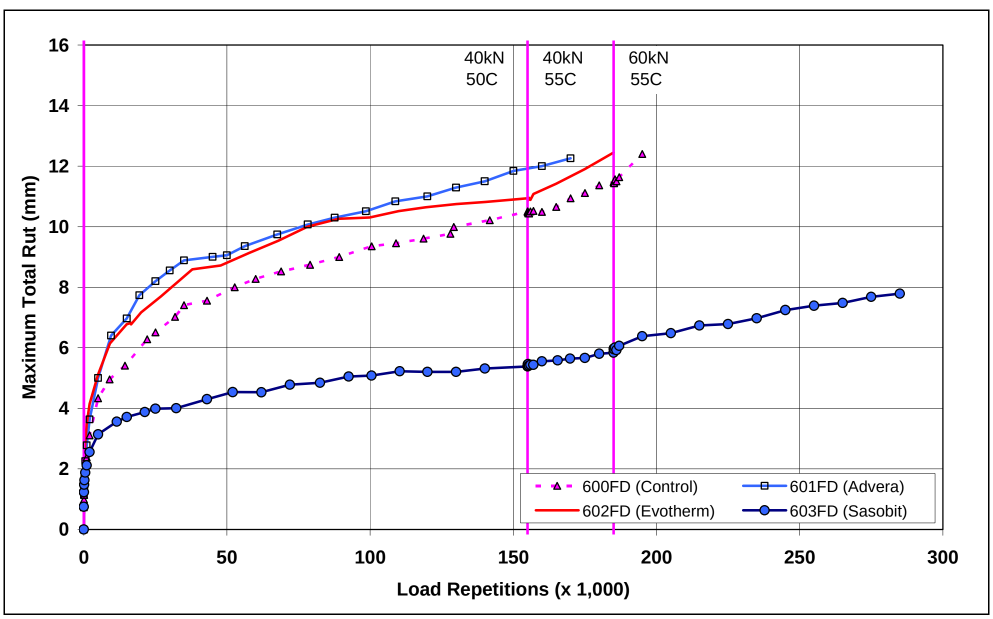


Figure 4.37: Comparison of average maximum rut.

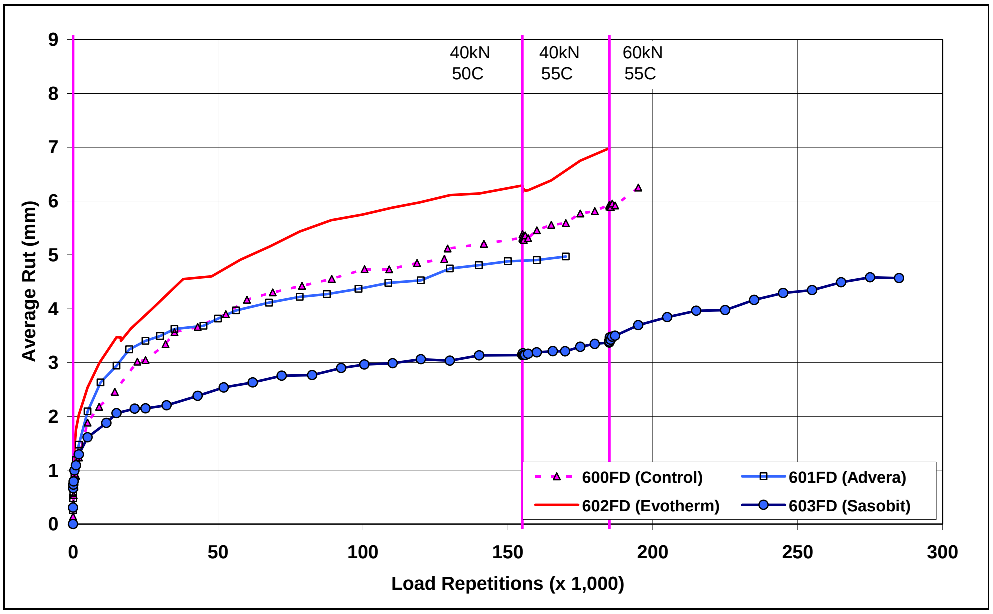


Figure 4.38: Comparison of average deformation.

5. PHASE 1 LABORATORY TEST DATA SUMMARY

5.1 Experiment Design

Phase 1 laboratory testing included shear, fatigue, and moisture sensitivity tests. Tests on mix properties were carried out on beams and cores cut from slabs removed from the test track after construction (see Section 2.8). Typical experimental designs used in previous studies were adopted for this warm-mix asphalt study to facilitate later comparison of results.

5.1.1 Shear Testing

Test Method

The AASHTO T-320 Permanent Shear Strain and Stiffness Test was used for shear testing in this study. In the standard test methodology, cylindrical test specimens 150 mm in diameter and 50 mm thick (6.0 in. by 2.0 in.) are subjected to repeated loading in shear using a 0.1-second haversine waveform followed by a 0.6-second rest period. Three different shear stresses are applied while the permanent (unrecoverable) and recoverable shear strains are measured. The permanent shear strain versus applied repetitions is normally recorded up to a value of five percent although 5,000 repetitions are called for in the AASHTO procedure. A constant temperature is maintained during the test (termed the *critical temperature)*, representative of the local environment. Shear Frequency Sweep Tests were used to establish the relationship between complex modulus and load frequency. The same loading was used at frequencies of 15, 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz.

Number of Tests

A total of 18 shear tests and six frequency sweep tests were carried out on each mix (total of 96 tests for the four mixes) as follows:

- Standard test
  - Two temperatures, namely 45°C and 55°C (113°F and 131°F)
  - Three stresses, namely 70 kPa, 100 kPa, and 130 kPa (10.2, 14.5, and 18.9 psi)
  - Three replicates.
- Frequency sweep test:
  - Two temperatures, namely 45°C and 55°C (113°F and 131°F)
  - One strain, namely 100 microstrain
  - Three replicates.

5.1.2 Fatigue Testing

Test Method

The AASHTO T-321 Flexural Controlled-Deformation Fatigue Test method was followed. In this test, three replicate beam test specimens, 50 mm thick by 63 mm wide by 380 mm long (2.0 x 2.5 x 15 in.), were subjected to four-point bending using a sinusoidal waveform at a loading frequency of 10 Hz. Testing was performed in both dry and wet condition at two different strain levels and at three different temperatures. Flexural Controlled-Deformation Frequency Sweep Tests were used to establish the relationship between complex modulus and load frequency. The same sinusoidal waveform was used in a controlled deformation mode and at frequencies of 15, 10, 5, 2, 1, 0.5, 0.2, 0.1, 0.05, 0.02, and 0.01 Hz. The upper limit of 15 Hz is a constraint imposed by the capabilities of the test machine. To ensure that the specimen was tested in a nondestructive manner, the frequency sweep test was conducted at a small strain amplitude level, proceeding from the highest frequency to the lowest in the sequence noted above.

The wet specimens used in the fatigue and frequency sweep tests were conditioned following the beam-soaking procedure described in Appendix C. The beam was first vacuum-saturated to ensure a saturation level greater than 70 percent, and then placed in a water bath at 60°C for 24 hours, followed by a second water bath at 20°C for 2 hours. The beams were then wrapped with Parafilm™ and tested within 24 hours after soaking.

Number of Tests

A total of 36 fatigue beam tests and 12 flexural fatigue frequency sweep tests were carried out on each mix (total of 192 tests for the four mixes) as follows:

- Standard test:
  - Two conditions (wet and dry)
  - Three temperatures, namely 10°C, 20°C, and 30°C (50°F, 68°F, and 86°F)
  - Two strains, namely 200 microstrain and 400 microstrain
  - Three replicates.
- Frequency sweep test:
  - Two conditions (wet and dry)
  - Three temperatures, namely 10°C, 20°C, and 30°C (50°F, 68°F, and 86°F)
  - One strain, namely 100 microstrain
  - Two replicates.

5.1.3 Moisture Sensitivity Testing

<u>Test Methods</u>

Two additional moisture sensitivity tests were conducted, namely the Hamburg Wheel-Track Test and the Tensile Strength Retained (TSR) Test.

- The AASHTO T-324 test method was followed for Hamburg Wheel-Track testing on slab specimens cut from the field-mixed, field-compacted sample slabs to the dimensions 320 mm long, 260 mm wide, and 120 mm thick (12.6 x 10.2 x 4.7 in.). All testing was carried out at 50°C (122°F).
- The Caltrans CT-371 test method was followed for the Tensile Strength Retained Test on cylindrical specimens 100 mm in diameter and 63 mm thick (4.0 x 2.5 in.) cored from the field-mixed, field-compacted sample slabs. This test method is similar to the AASHTO T-283 test, however, it has some modifications specific for California conditions.

<u>Number of Tests</u>

Four replicates of the Hamburg Wheel-Track test and four replicates of the Tensile Strength Retained Test were carried out for each mix (16 tests per method).

5.2 Test Results

5.2.1 Shear Tests

<u>Air-Void Content</u>

Shear specimens were cored from the top lift of the field-mixed, field-compacted (FMFC) slabs. Air-void contents were measured using the modified Parafilm method (AASHTO T-275A). Table 5.1 summarizes the air-void distribution categorized by mix type, test temperature, and test shear stress level. Figure 5.1 presents the summary boxplots of air-void content based on additive type. The differences in air-void content distributions between the mixes with various additives are clearly apparent. The mean difference for the highest mean air-void content (Evotherm) and the smallest mean air-void content (Control) could be as high as 2.0 percent.

Table 5.1: Summary of Air-Void Contents of Shear Test Specimens

Temperature		Stress Level (kPa)	Control		Advera		Evotherm		Sasobit	
°C	°F		Mean	SD[1]	Mean	SD	Mean	SD	Mean	SD
45	113	70	6.5	0.6	8.7	1.0	9.2	1.0	8.3	0.7
		100	6.5	0.6	8.7	0.6	8.7	1.0	8.1	0.8
		130	6.4	0.3	8.2	0.9	9.2	0.7	8.0	0.4
55	131	70	6.7	0.6	8.5	0.9	8.3	0.5	7.7	0.6
		100	6.8	0.6	8.0	0.7	8.3	0.3	8.3	0.3
		130	7.7	0.9	8.0	0.2	9.1	0.7	8.1	0.8
		Overall	6.8	0.7	8.4	0.7	8.8	0.7	8.1	0.6
Frequency Sweep			7.1	0.7	8.6	0.9	8.7	0.8	7.8	0.4

[1] SD: Standard deviation.

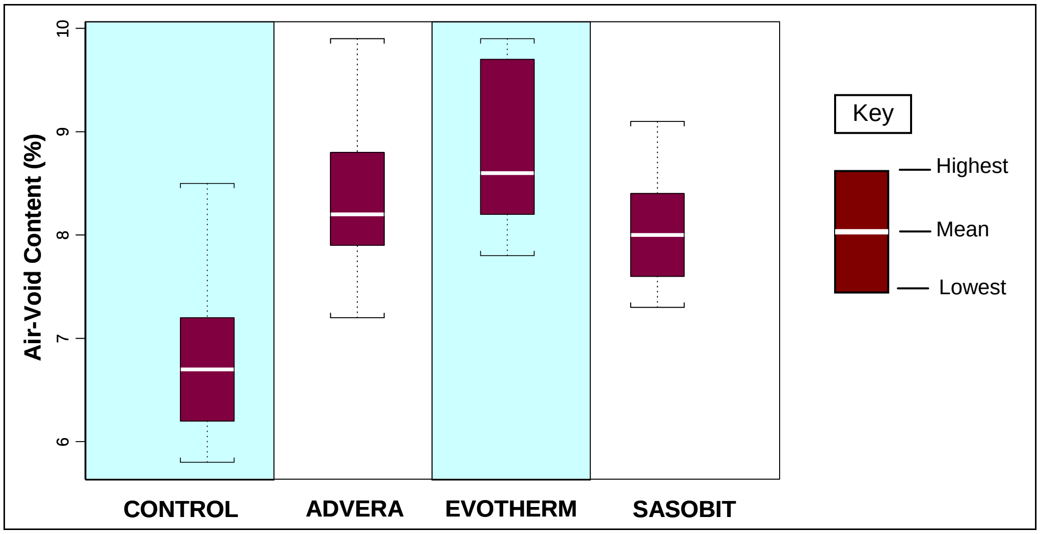


Figure 5.1: Air-void contents of shear specimens.

5.2.2 Resilient Shear Modulus (G)

The resilient shear modulus results for the four mixes are summarized in Figure 5.2. The resilient shear modulus was influenced by temperature, with the modulus increasing with decreasing temperature. Resilient shear modulus was not influenced by stress. The variation of resilient shear moduli at 45°C was considerable compared to the results at 55°C. The Sasobit specimens had the highest resilient shear modulus, as expected, due to the lower binder content. The Control, Evotherm, and Advera mix specimens had essentially the same shear modulus indicating that the use of the additive and lower production and compaction temperatures did not significantly influence the performance of the mix in this test.

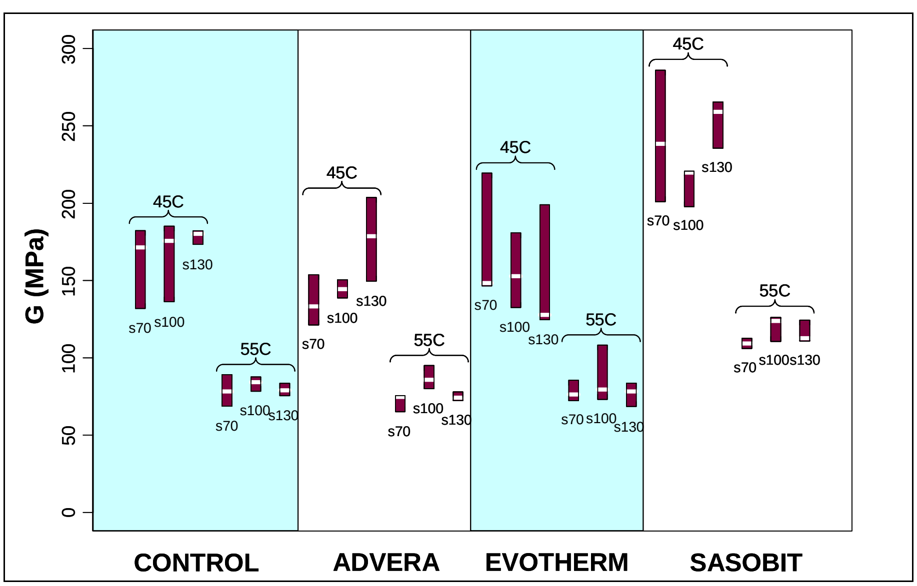


Figure 5.2: Summary boxplots of resilient shear modulus.

Cycles to Five Percent Permanent Shear Strain

The number of cycles to five percent permanent shear strain provides an indication of the rut-resistance of an asphalt mix, with higher numbers of cycles implying better rut-resistance. Figure 5.3 summarizes the shear test results in terms of the natural logarithm of this parameter. As expected, the rut-resistance capacity decreased with increasing temperature and stress level. The Sasobit mix specimens had the highest number of cycles to five percent permanent shear strain, as expected. With the exception of the Evotherm mix at 45°C and 70 kPa stress level, no significant difference was noted between the Control, Advera, and Evotherm mixes, despite the Advera and Evotherm mix specimens having higher air-void contents than the Control (± 2.0 percent). This indicates that the use of the additive and lower production and compaction temperatures did not significantly influence the performance of the mix in this test.

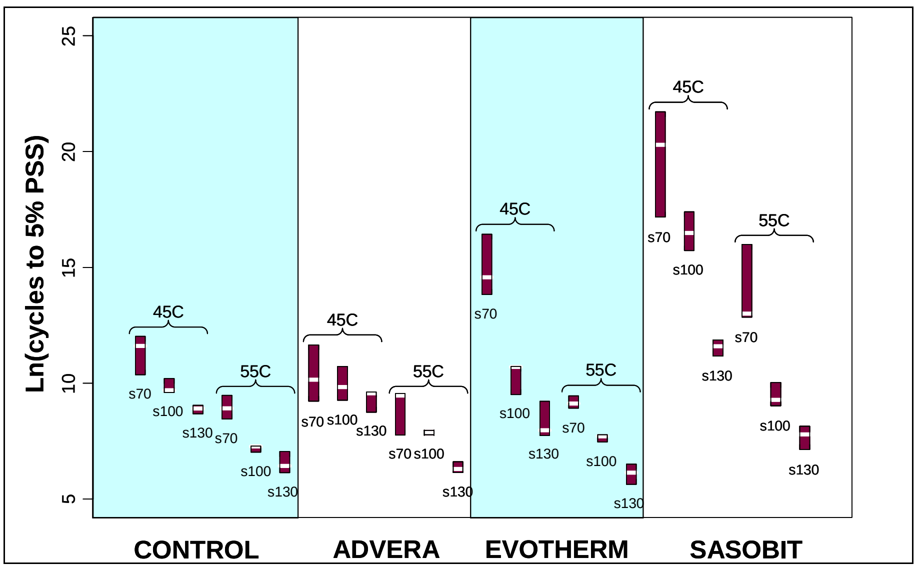


Figure 5.3: Summary boxplots of cycles to 5% permanent shear strain.

Permanent Shear Strain at 5,000 Cycles

The measurement of permanent shear strain (PSS) accumulated after 5,000 cycles provides an alternative indication of the rut-resistance capacity of an asphalt mix. The smaller the permanent shear strain the better the mixture's rut-resistance capacity. Figure 5.4 summarizes the rutting performance of the four mixes in terms of the natural logarithm of this parameter (i.e., increasingly negative values represent smaller cumulative permanent shear strain). The results indicate that:

- The effect of shear stress level is more significant at higher temperatures.
- The higher the temperature and stress level the larger the cumulative permanent shear strain.
- In general, the Sasobit mix accumulated the least permanent shear strain when compared with the other mixes at the same stress level and temperature, as expected.
- The Evotherm mix was the most stress-sensitive.
- There was no significant difference between the Control and Advera mixes.

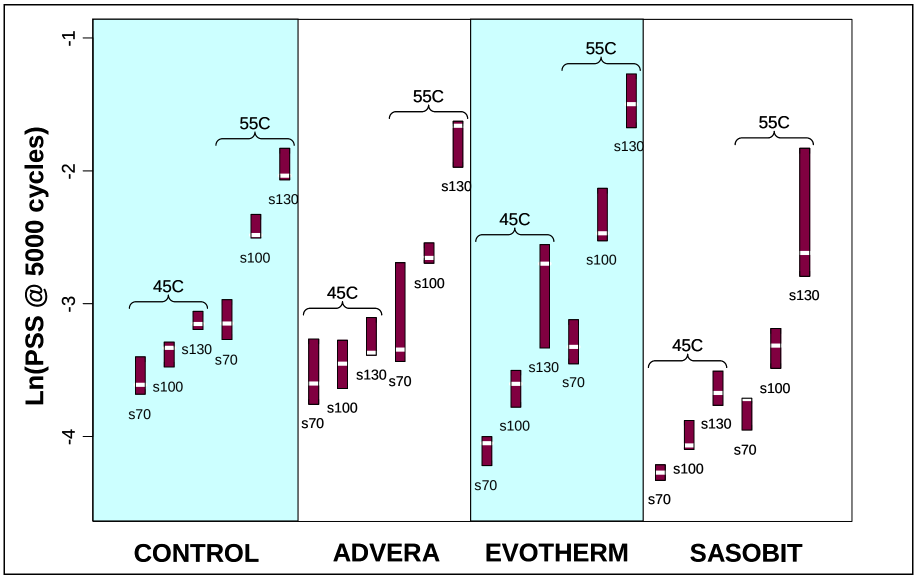


Figure 5.4: Summary boxplots of cumulative permanent shear strain at 5,000 cycles.

<u>Shear Frequency Sweep</u>

The average shear complex moduli (G^*) and average phase angle (*pa*) of three replicates tested at the two temperatures were used to develop the G^* and *pa* master curves respectively. The reference temperature of the master curves was set at 55°C. The shifted master curves with minimized residual-sum-of-squares derived using a genetic algorithm approach was fitted with the following modified Gamma functions (Equation 5.1 and Equation 5.2):

$$Ln(G^*) = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \sum_{m}^{n-1} \frac{(x-C)^m}{B^m m!}\right) \tag{5.1}$$

$$pa = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \sum_{m}^{n-1} \frac{(x-C)^m}{B^m m!}\right) \tag{5.2}$$

where: G^* is the flexural complex modulus (MPa),
pa is the phase angle (degree),
x is the loading frequency in Hz, and
A, *B*, *C*, *D*, and *n* are the experimentally-determined parameters, and
ln is the natural logarithm.

Note that the experimentally-determined parameters, *A*, *B*, *C*, and *D*, are different for Equation 5.1 and Equation 5.2. The experimentally-determined parameters of the modified Gamma function for each mix type are listed in Table 5.2 and Table 5.3 respectively for shear complex modulus and phase angle master curves.

Table 5.2: Summary of Ln(G*) Master Curves

Mix	Master Curve				
	n	A	B	C	D
Control	3	7.566435	3.344699	-3.784501	1.606332
Advera	3	9.979300	4.148430	-3.922014	1.591510
Evotherm	3	8.852759	3.694575	-3.757068	1.516829
Sasobit	3	7.549157	3.693620	-5.013536	1.757530

Notes:
1. The reference temperature is 55°C.
2. Master curve Gamma-fitted equations:

If $n = 3$, $$Ln(G^*) = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2}\right)\right),$$

If $n = 4$, $$Ln(G^*) = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2} + \frac{(x-C)^3}{6B^3}\right)\right),$$

where $x = \ln freq + \ln aT$

Table 5.3: Summary of Phase Angle Master Curves

Mix	Master Curve				
	n	A	B	C	D
Control	3	32.20464	1.177630	-3.953715	26.14898
Advera	3	32.68695	1.222088	-3.887848	26.17297
Evotherm	3	29.29483	1.204523	-3.855373	30.66809
Sasobit	3	23.98185	1.025020	-4.790558	30.78790

Notes:
1. The reference temperature is 55°C.
2. Master curve Gamma-fitted equations:

If $n = 3$, $$pa = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2}\right)\right),$$

If $n = 4$, $$pa = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2} + \frac{(x-C)^3}{6B^3}\right)\right),$$

where $x = \ln freq + \ln aT$; phase angle in degree.

Figure 5.5 and Figure 5.6 show the shifted master curves with Gamma-fitted lines, respectively, for shear complex modulus and phase angle for the 55°C testing. The following observations were made from the shear frequency sweep test results:

- There was no apparent difference between the complex modulus master curves of the Control, Advera, and Evotherm mixes although the modulus of the Control mix was slightly higher than the other two mixes. The master curve of the Sasobit mix was above those of the other three mixes, as expected.
- On the phase angle master curve, phase angle increased with increasing loading frequency for all mixes.
- There was no apparent difference in the phase angle master curves for the Control, Advera, and Evotherm mixes. The master curve of the Sasobit mix crossed the other three master curves

approximately between 2.0 Hz and 5.0 Hz; hence, at higher loading frequencies the Sasobit mix appeared to have smaller phase angles than the other three mixes and higher phase angles at lower loading frequencies. This is probably a function of the lower binder content.

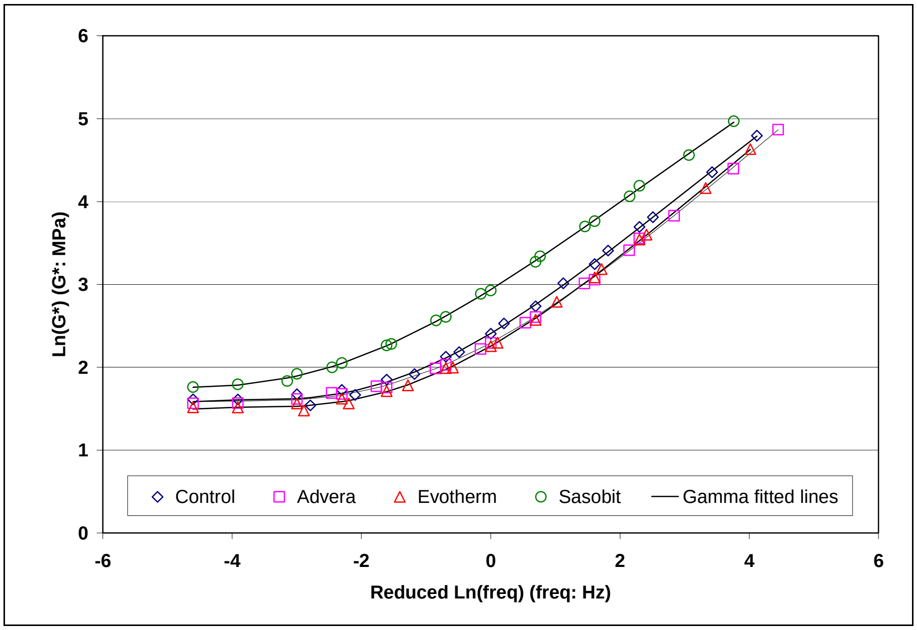


Figure 5.5: Summary of shear complex modulus master curves.

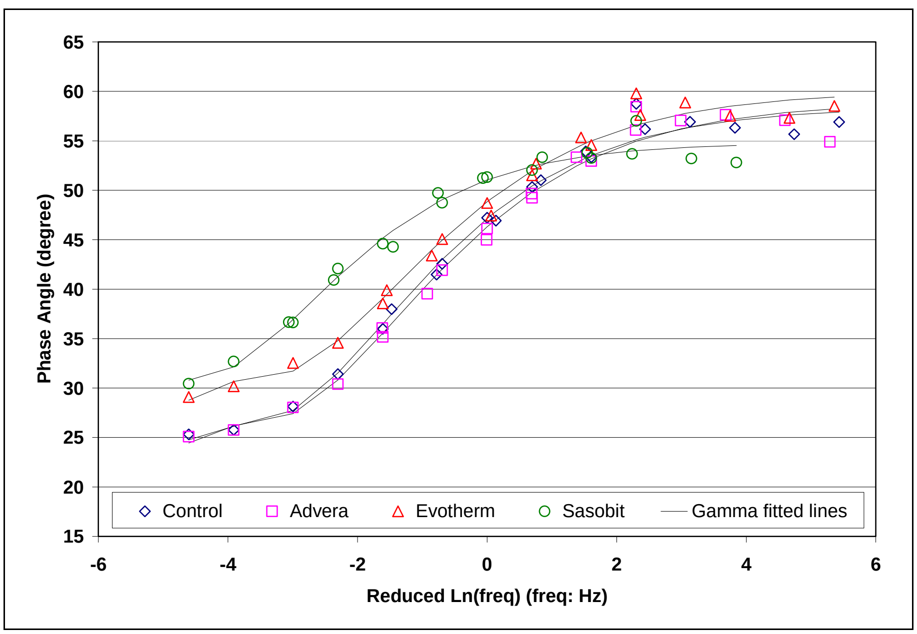


Figure 5.6: Summary of shear phase angle master curves.

5.2.3 Fatigue Beam Tests

Air-Void Content

Fatigue beams were saw-cut from the bottom lift of the FMFC slabs. Air-void contents were measured using the modified Parafilm method (AASHTO T-275A). Table 5.4 and Table 5.5 summarize the air-void distribution categorized by mix type, test temperature, and test tensile strain level for the fatigue beam and frequency sweep specimens, respectively. Figure 5.7 and Figure 5.8 present the summary boxplots of air-void content for the wet and dry fatigue beam and flexural frequency sweep specimens, respectively. There was no significant difference in air-void content between the dry and wet specimens, but some difference between the four mixes.

Table 5.4: Summary of Air-Void Contents of Fatigue Beam Specimens

Condition	Strain (μstrain)	Temp.		Control		Advera		Evotherm		Sasobit	
		°C	°F	Mean	SD[1]	Mean	SD	Mean	SD	Mean	SD
Dry	200	10	50	7.3	1.0	8.8	0.7	8.2	1.5	6.6	0.3
		20	68	6.9	0.6	7.2	0.6	7.5	0.7	6.3	0.2
		30	86	7.3	0.7	6.6	0.3	8.8	0.8	7.5	0.6
	400	10	50	7.0	0.6	6.8	0.6	9.0	0.8	6.9	0.4
		20	68	7.4	0.8	8.7	1.4	7.7	0.7	7.4	0.8
		30	86	6.7	0.4	7.7	0.8	7.7	0.6	6.7	0.3
	Overall			7.1	0.6	7.6	1.1	8.1	1.0	6.9	0.6
Wet	200	10	50	8.0	0.5	7.5	1.0	9.5	0.6	6.9	0.2
		20	68	6.8	0.4	7.4	0.9	8.8	1.3	6.6	0.7
		30	86	6.9	1.2	7.4	0.8	7.6	0.9	6.9	0.4
	400	10	50	6.9	0.5	8.3	1.2	8.9	0.3	6.6	0.5
		20	68	7.0	0.3	6.9	1.5	8.2	0.3	6.9	0.2
		30	86	7.2	0.4	7.7	0.3	8.3	0.5	6.6	0.3
	Overall			7.1	0.7	7.5	0.9	8.6	0.7	6.7	0.4

[1] SD: Standard deviation.

Table 5.5: Summary of Air-Void Contents of Flexural Frequency Sweep Specimens

Condition	Control		Advera		Evotherm		Sasobit	
	Mean	SD[1]	Mean	SD	Mean	SD	Mean	SD
Dry	6.7	0.5	7.5	0.9	8.4	1.0	7.0	0.5
Wet	6.7	0.4	7.9	0.8	8.8	0.7	6.8	0.7

[1] SD: Standard deviation.

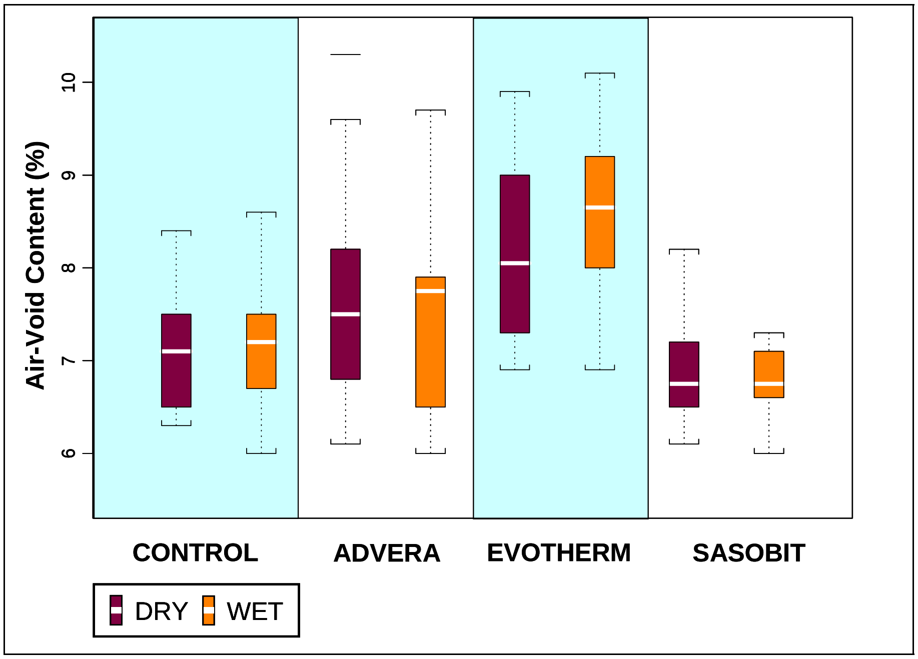


Figure 5.7: Air-void contents of fatigue beam specimens (dry and wet).

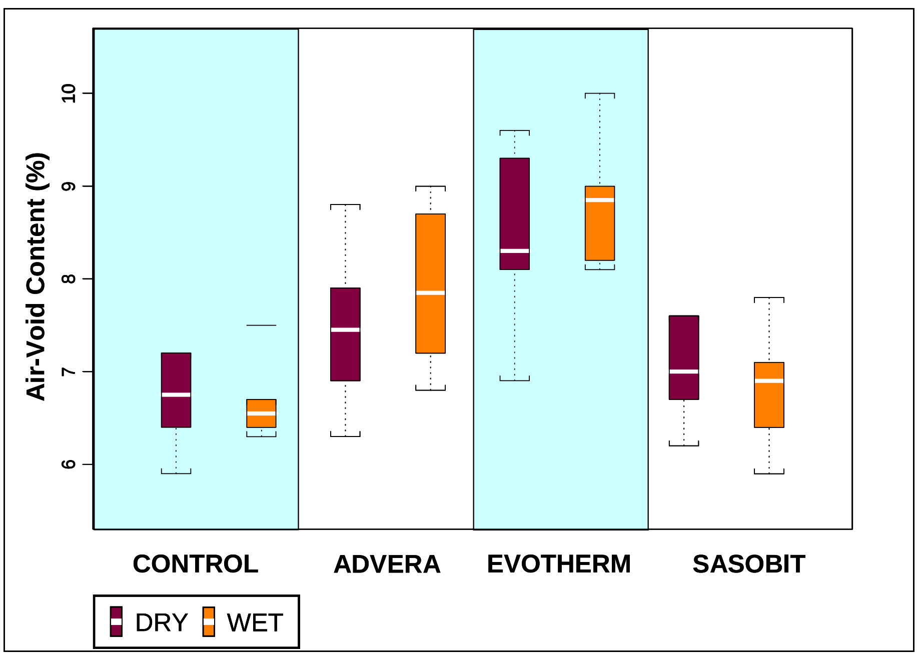


Figure 5.8: Air-void contents of flexural frequency sweep specimens (dry and wet).

Table 5.6 compares air-void contents (or degree of compaction) of the top and bottom lifts for the different mixes. With the exception of the Control mix, air-void contents were higher in the top lift compared to the bottom lift, indicating poorer compaction in the top lift. This is expected in multi-lift construction because the bottom lift is reheated during placement of the next lift, and receives additional compaction while the next lift is being compacted.

Table 5.6: Air-Void Content Comparison of Top and Bottom Lifts

Location	Condition	Control		Advera		Evotherm		Sasobit	
		Mean	SD[1]	Mean	SD	Mean	SD	Mean	SD
Top Lift (Shear Cores)	Dry	6.7	0.7	8.4	0.7	8.8	0.7	8.1	0.6
Bottom Lift (Fatigue Beams)	Dry Wet	7.1 7.1	0.6 0.7	7.6 7.5	1.1 0.9	8.1 8.6	1.0 0.9	6.9 6.7	0.6 0.4

Initial Stiffness

Figure 5.9 illustrates the initial stiffness comparison at various strain levels, temperatures, and conditioning for the different mix types. The following observations were made:

- Initial stiffness was generally strain-independent for both the dry and wet tests.
- Temperature had a significant effect on both the dry and wet tests.
- A reduction of initial stiffness due to soaking was apparent for each mix type, with the reduction most prominent for the 10°C test. The difference in stiffness at 30°C indicates a potential reduction in rut-resistance at higher temperatures due to moisture damage. The difference in stiffness at all temperatures indicates a loss of structural capacity due to moisture damage.
- Moisture sensitivity was not influenced by any of the additives (i.e., the Control mix was not less moisture sensitive than the mixes with additives, compacted at lower temperatures).
- There was no significant difference between the four mixes in terms of initial stiffness indicating that the use of the additives and lower production and compaction temperatures did not significantly influence the performance of the mix in this test.

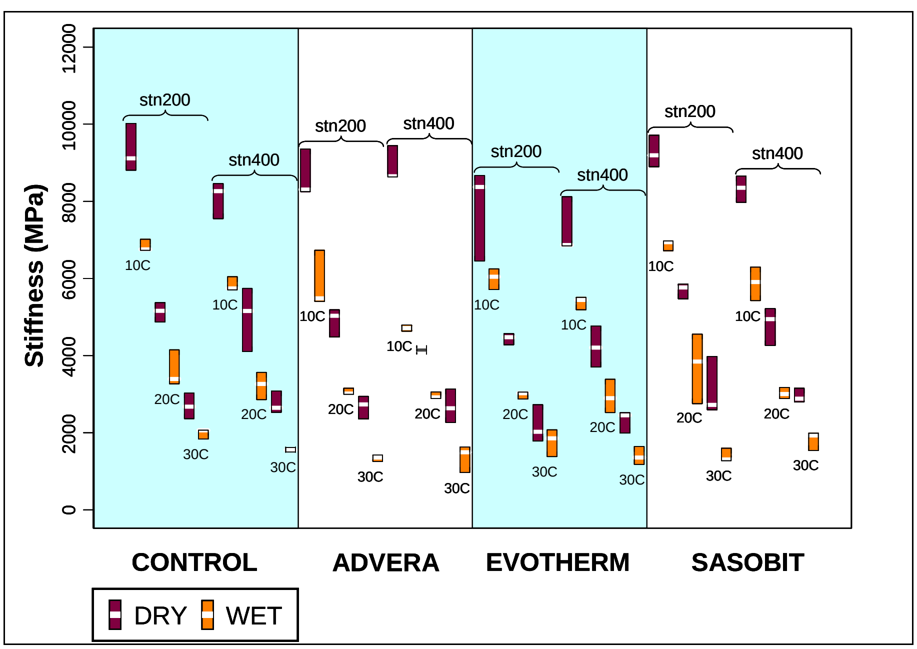


Figure 5.9: Summary boxplots of initial stiffness.

Initial Phase Angle

The initial phase angle can be used as an index of mix viscosity properties, with higher phase angles corresponding to more viscous and less elastic properties. Figure 5.10 illustrates the side-by-side phase angle comparison of dry and wet tests for the four mixes. The following observations were made:

- The initial phase angle increased with increasing temperature.
- The initial phase angle appeared to be strain-independent.
- Soaking appeared to increase the phase angle slightly and introduce larger dispersion of the phase angle.
- The initial phase angle was highly negative-correlated with the initial stiffness.
- There was no significant difference between the four mixes in terms of initial phase angle indicating that the addition of the additives and lower production and compaction temperatures did not significantly influence the performance of the mix in this test.

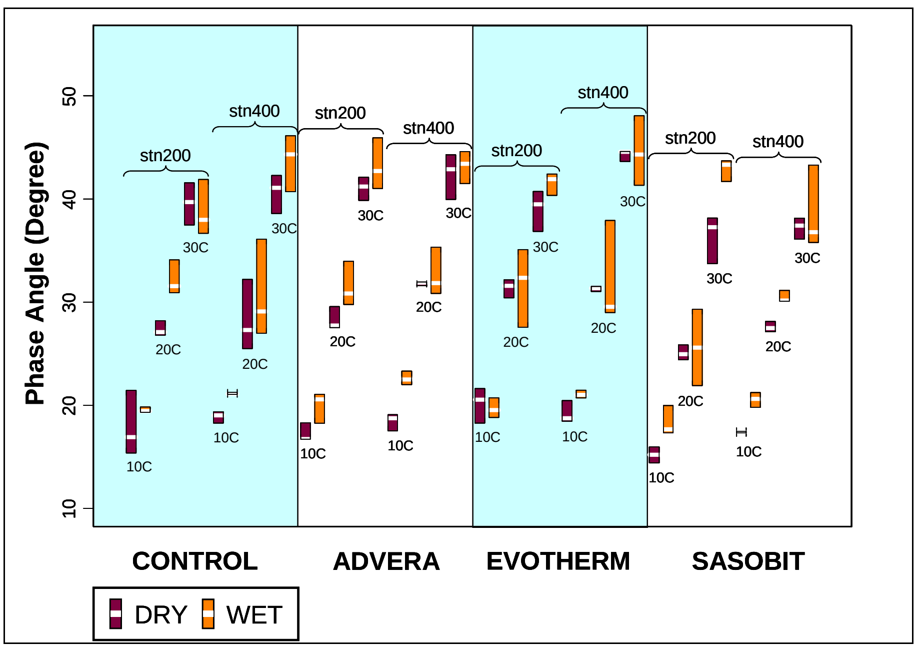


Figure 5.10: Summary boxplots of initial phase angle.

Fatigue Life at 50 Percent Stiffness Reduction

Mix stiffness decreases with increasing test-load repetitions. Conventional fatigue life is defined as the number of load repetitions when 50 percent stiffness reduction has been reached. A high fatigue life implies a slow fatigue damage rate and consequently higher fatigue-resistance. The side-by-side fatigue life comparison of dry and wet tests is plotted in Figure 5.11. The following observations were made:

- Fatigue life was both strain- and temperature-dependent. In general, lower strains and higher temperatures will result in higher fatigue life and vice versa.

- Soaking generally resulted in a lower fatigue life compared to the unsoaked specimens. Inconsistent results were obtained across the mixes at the higher temperatures (i.e., 200 microstrain and 30°C). It is not clear why this occurred.
- There was no significant difference between the four mixes in terms of fatigue life at 50 percent stiffness reduction indicating that the addition of the additives and lower production and compaction temperatures did not significantly influence the performance of the mix in this test.

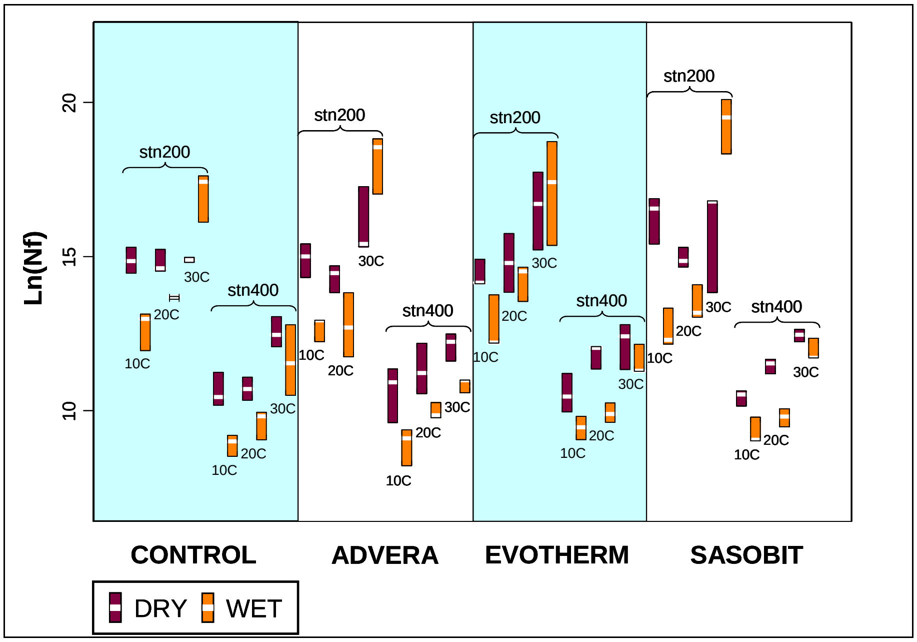


Figure 5.11: Summary boxplots of fatigue life.

<u>Flexural Frequency Sweep</u>

The average stiffness values of the two replicates tested at the three temperatures were used to develop the flexural complex modulus (E^*) master curve. This is considered a useful tool for characterizing the effects of loading frequency (or vehicle speed) and temperature on the initial stiffness of an asphalt mix (i.e., before any fatigue damage has occurred). The shifted master curve with minimized residual-sum-of-squares derived using a genetic algorithm approach can be appropriately fitted with the following modified Gamma function (Equation 5.3):

$$E^* = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B} \right) \cdot \sum_{m}^{n-1} \frac{(x-C)^m}{B^m m!} \right) \tag{5.3}$$

where: E^* = flexural complex modulus (MPa);
$x = \ln freq + \ln aT$ = is the loading frequency in Hz and $\ln aT$ can be obtained from the temperature-shifting relationship (Equation 5.4);
A, B, C, D, and n are the experimentally-determined parameters.

$$\ln aT = A \cdot \left(1 - \exp\left(-\frac{T - Tref}{B}\right)\right) \quad \textbf{(5.4)}$$

where: $lnaT$ = is a horizontal shift to correct the temperature effect with the same unit as ln *freq*,
T = is the temperature in °C,
$Tref$ = is the reference temperature, in this case, $Tref$ = 20°C
A and B are the experimentally-determined parameters.

The experimentally-determined parameters of the modified Gamma function for each mix type are listed in Table 5.7, together with the parameters in the temperature-shifting relationship.

Figure 5.12 and Figure 5.13 show the shifted master curves with Gamma-fitted lines and the temperature-shifting relationships, respectively, for the dry frequency sweep tests. The temperature-shifting relationships were obtained during the construction of the complex modulus master curve and can be used to correct the temperature effect on initial stiffness. Note that a positive lnaT value needs to be applied when the temperature is lower than the reference temperature, while a negative lnaT value needs to be used when the temperature is higher than the reference temperature.

The following observations were made from the dry frequency sweep test results:

- There was no apparent difference between the complex modulus master curves of the Control, Advera, and Sasobit mixes. The curve for the Evotherm mix was below those of the other three mixes, possibly due to the higher air-void contents of the tested beams.
- The temperature-shifting relationships indicate that the Advera mix was the most temperature-sensitive in extreme temperatures and that the Control mix was the least temperature-sensitive on average. Higher temperature-sensitivity implies that a per unit change of temperature will cause a larger change of stiffness (i.e., larger change of lnaT).

Figure 5.14 and Figure 5.15 respectively show the shifted master curves with Gamma-fitted lines and the temperature-shifting relationships for the wet frequency sweep tests. The comparison of dry and wet complex modulus master curves is shown in Figure 5.16 for each mix type. The following observations were made with regard to the wet frequency sweep tests results:

- The complex modulus curves of the Control and Sasobit mixes were essentially the same, while the curves for the Advera and Evotherm mixes showed lower stiffness.
- There were no apparent temperature-sensitivity differences between the four mixes at higher temperatures (i.e., higher than 20°C). At lower temperatures (i.e., lower than 20°C), there was no significant difference in temperature-sensitivity between the Control and Advera mixes, but some temperature sensitivity in the Evotherm and the Sasobit mixes.
- A loss of stiffness attributed to moisture damage was apparent in all four mixes.

Table 5.7: Summary of Master Curves and Time-Temperature Relationships

Mix	Conditioning	Master Curve					Time-Temperature Relationship	
		N	A	B	C	D	A	B
Control	Dry	3	36709.04	6.776351	-6.193638	287.7218	-2.59871	13.9774
Advera		3	31589.35	6.44247	-6.192128	266.6365	-7.9213	30.462
Evotherm		3	31725.88	7.069325	-6.228655	193.6026	-18.8202	92.144
Sasobit		3	30895.62	6.686322	-6.75525	315.8828	-3.71373	15.2257
Control	Wet	3	91682.18	11.87393	-6.408145	174.7554	-3.97313	14.3648
Advera		3	140552.3	15.84893	-5.936551	144.9951	-5.13173	18.8505
Evotherm		3	40602.73	8.365664	-5.920522	174.3914	-15.1651	53.6384
Sasobit		4	1951606	26.46189	-11.3173	65.28481	-93.0313	377.234

Notes:

1. The reference temperature is 20°C.
2. The wet test specimens were soaked at 60°C.
3. Master curve Gamma-fitted equations:

If $n = 3$, $E^* = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2}\right)\right)$,

If $n = 4$, $E^* = D + A \cdot \left(1 - \exp\left(-\frac{(x-C)}{B}\right) \cdot \left(1 + \frac{x-C}{B} + \frac{(x-C)^2}{2B^2} + \frac{(x-C)^3}{6B^3}\right)\right)$,

where $x = \ln freq + \ln aT$

4. Time-temperature relationship: $\ln aT = A \cdot \left(1 - \exp\left(-\frac{T - Tref}{B}\right)\right)$

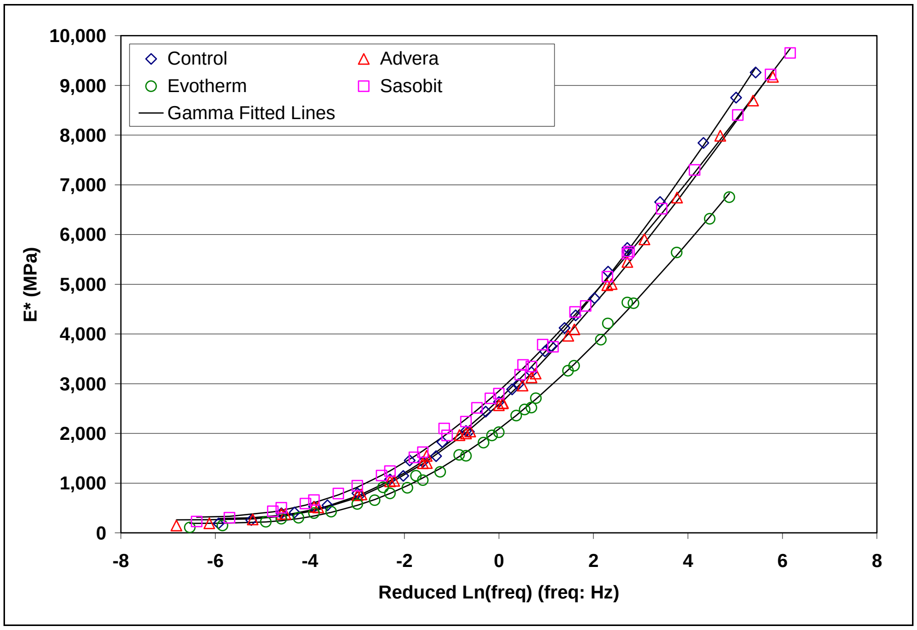


Figure 5.12: Complex modulus (*E) master curves (dry).**

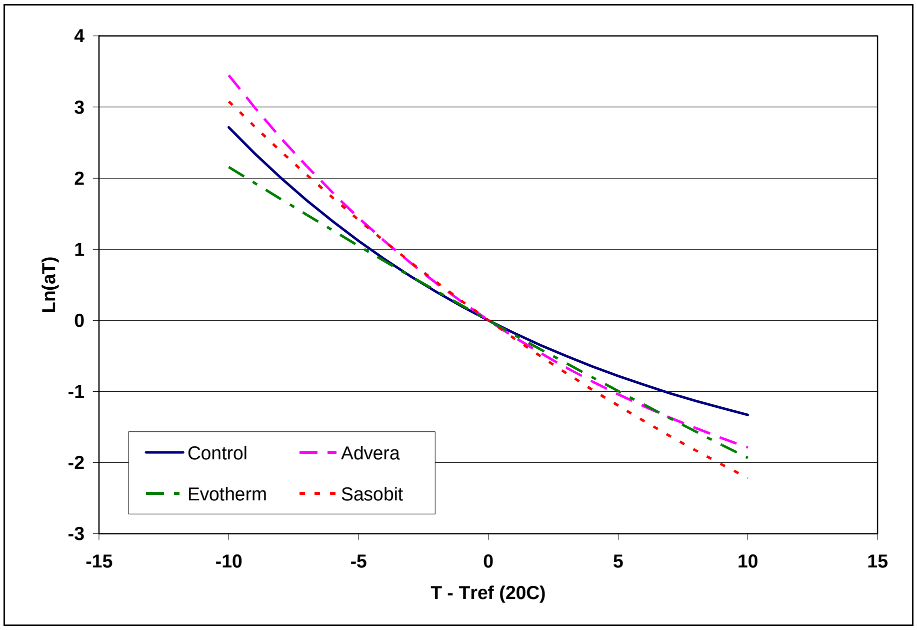


Figure 5.13: Temperature-shifting relationship (dry).

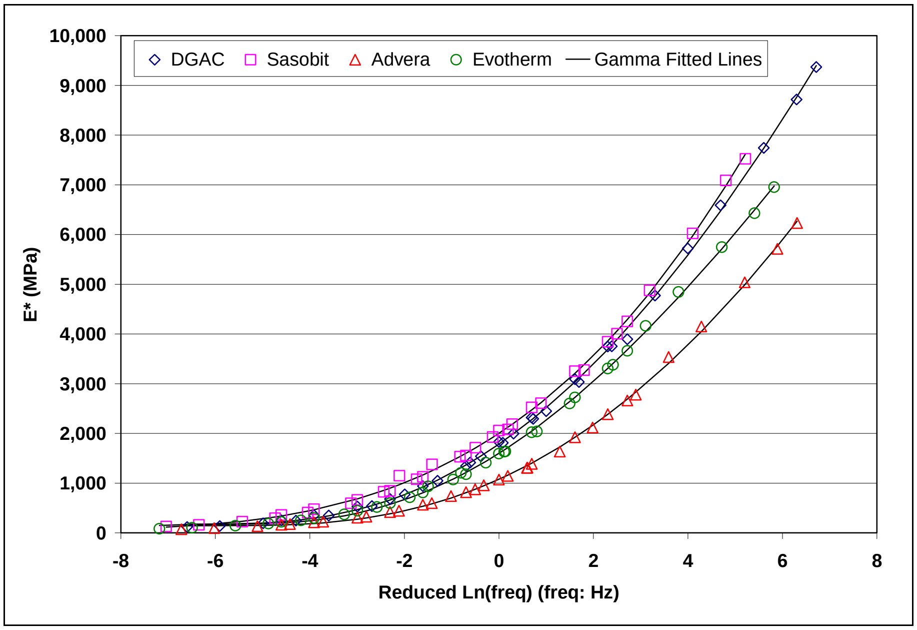


Figure 5.14: Complex modulus (*E) master curves (wet).**

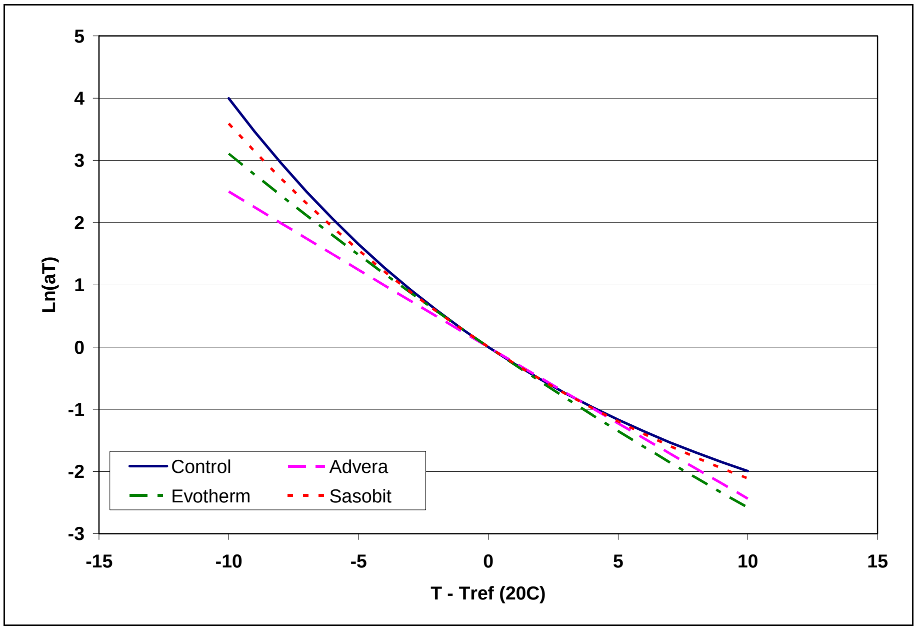


Figure 5.15: Temperature-shifting relationship (wet).

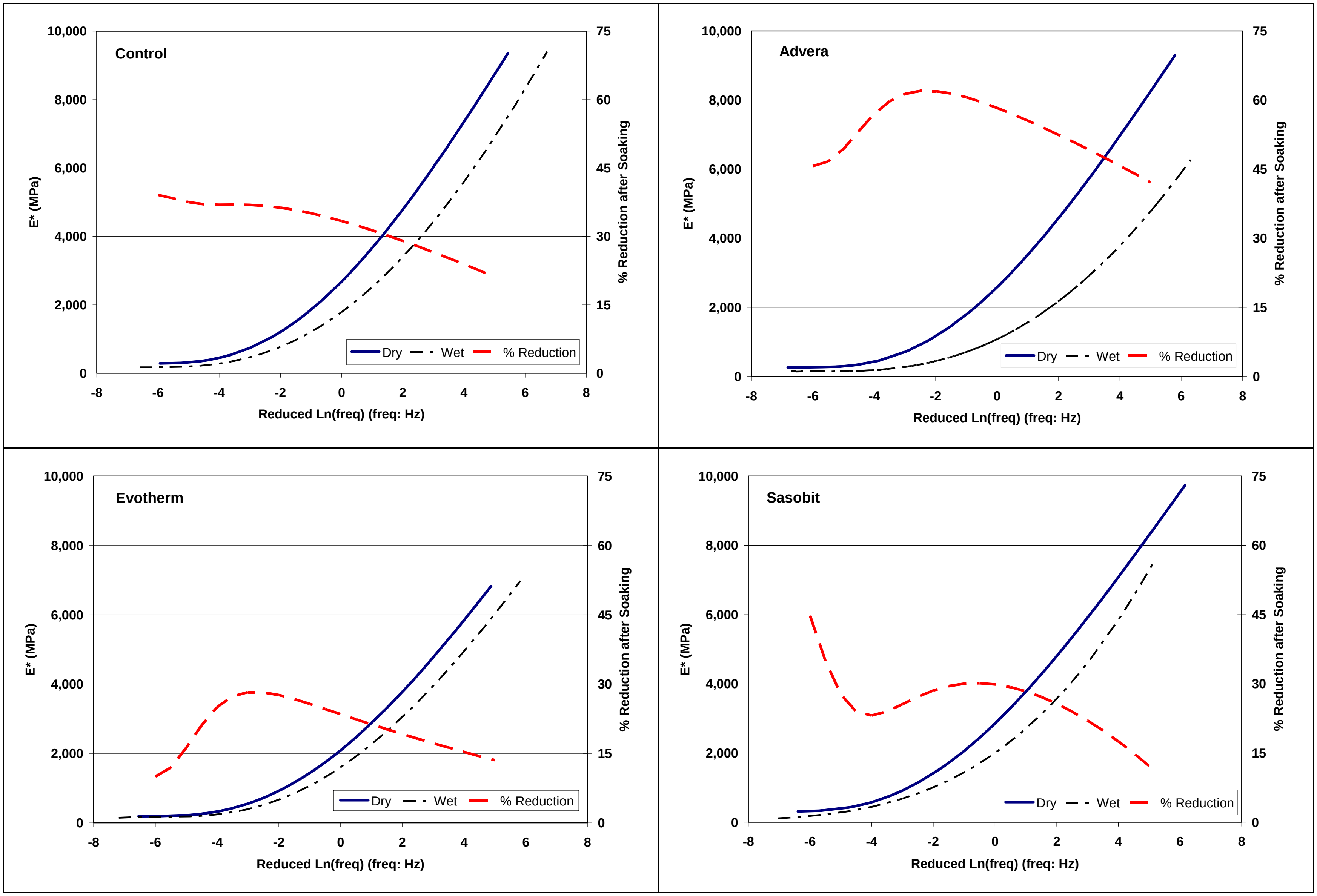


Figure 5.16: Comparison of dry and wet complex modulus master curves.

(Includes percent reduction in stiffness at each frequency from dry to wet master curve.)

5.2.4 Moisture Sensitivity: Hamburg Wheel-Track Test

Air-Void Content

The air-void content of each slab specimen was calculated from the bulk specific gravity (measured in accordance with Method A of AASHTO T-166) and the theoretical maximum specific gravity (determined in accordance with ASTM D-2041). The air-void contents ranged between 5.8 and 7.4 percent, with the Control mix having a slightly lower air-void content than the other three mixes (Table 5.8).

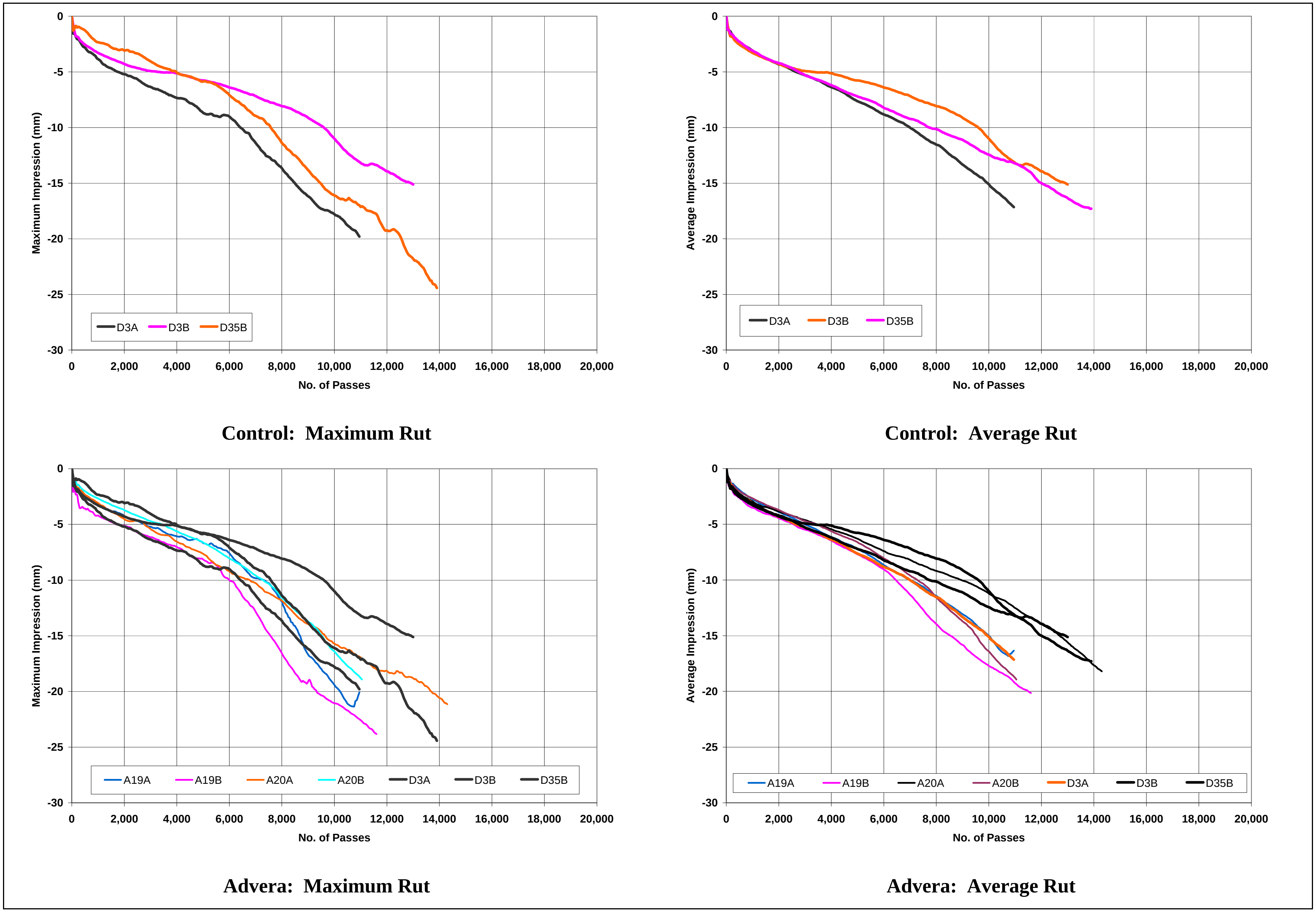
Testing

The testing sequence of the specimens was randomized to avoid any potential block effect. Rut depth was recorded at 11 equally spaced points along the wheelpath on the specimen. The average of the middle seven points was then used in the analysis. This method ensures that localized distresses are smoothed and variance in the data is minimized. It should be noted that some state departments of transportation (e.g., Utah) only measure the point of maximum final rut depth, which usually results in a larger variance in the test results.

Table 5.8: Air-Void Content of Hamburg Wheel-Track Test Specimens

Specimen ID		Bulk Specific Gravity (g/cm^3)	Max Specific Gravity (g/cm^3)	Air-Void Content (%)
Control	D35-A	2.420	2.574	6.0
	D35-B	2.420	2.574	6.0
	D 3-A	2.425	2.574	5.8
	D 3-B	2.424	2.574	5.8
Advera	A19-A	2.419	2.601	7.0
	A19-B	2.418	2.601	7.0
	A20-A	2.420	2.601	7.0
	A20-B	2.429	2.601	6.6
Evotherm	E22-A	2.399	2.585	7.2
	E22-B	2.399	2.585	7.2
	E24-A	2.395	2.585	7.4
	E24-B	2.404	2.585	7.0
Sasobit	S 1-A	2.423	2.597	6.7
	S 1-B	2.435	2.597	6.3
	S 2-A	2.420	2.597	6.8
	S 2-B	2.420	2.597	6.8

Figure 5.17 and Figure 5.18 show the rut progression curves of all specimens, in terms of both the maximum rut depth and average rut depth. The curve for the Control mix is included in the plots of the mixes with additives. As expected, the progression curves of the maximum rut depths had a larger variation. Figure 5.19 through Figure 5.22 show the condition of each slab specimen after the Hamburg Wheel-Track test. The creep slope, stripping slope, stripping inflection point, and rut depths at 10,000 and 20,000 passes were calculated from the average and maximum rut progression curves, and are summarized in Table 5.9 and Table 5.10, respectively. Rut depths at 20,000 passes were linearly extrapolated for tests terminated before the number of wheel passes reached this point.



Control: Maximum Rut

Control: Average Rut

Advera: Maximum Rut

Advera: Average Rut

Figure 5.17: Maximum and average rut progression curves for Control and Advera specimens.

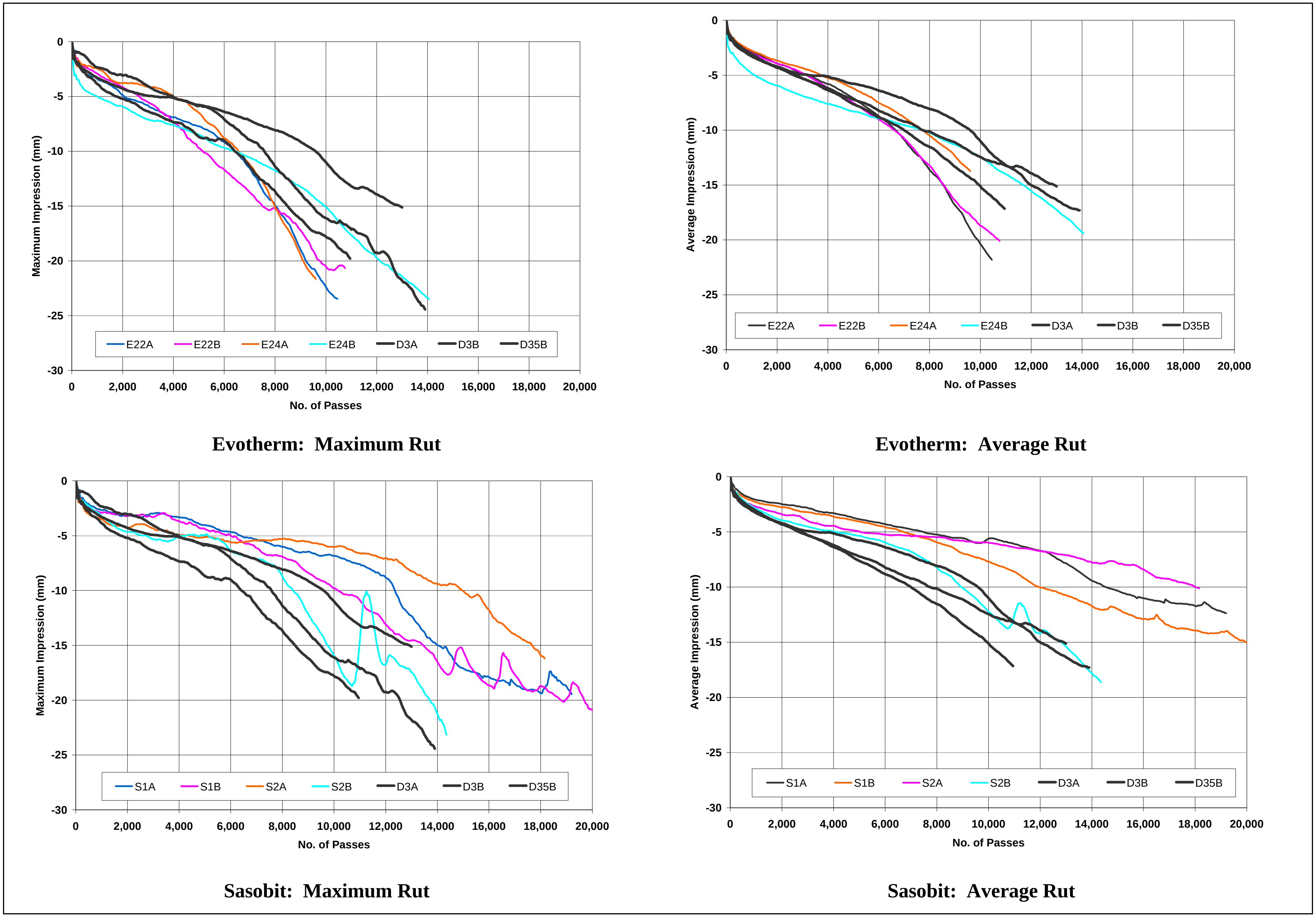


Figure 5.18: Maximum and average rut progression curves for Evotherm and Sasobit specimens.

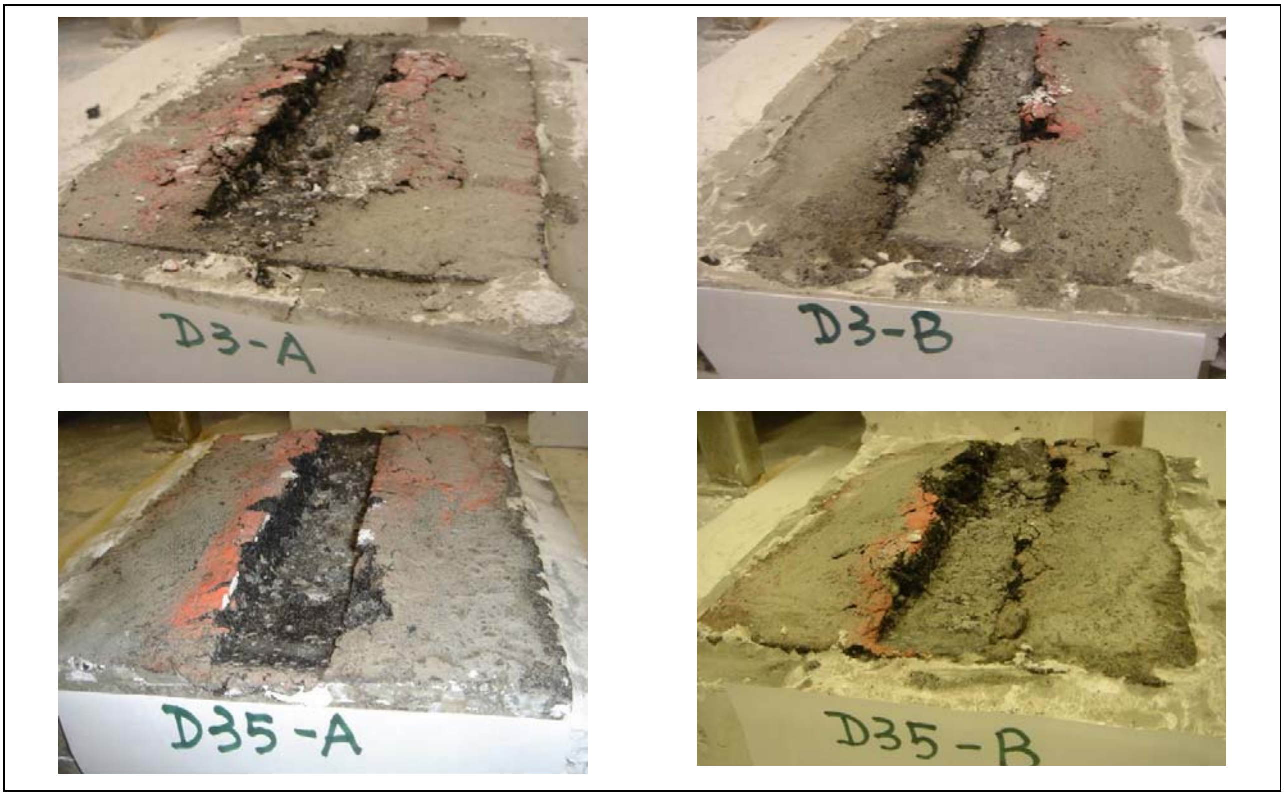


Figure 5.19: Control mix specimens after Hamburg Wheel-Track Test.

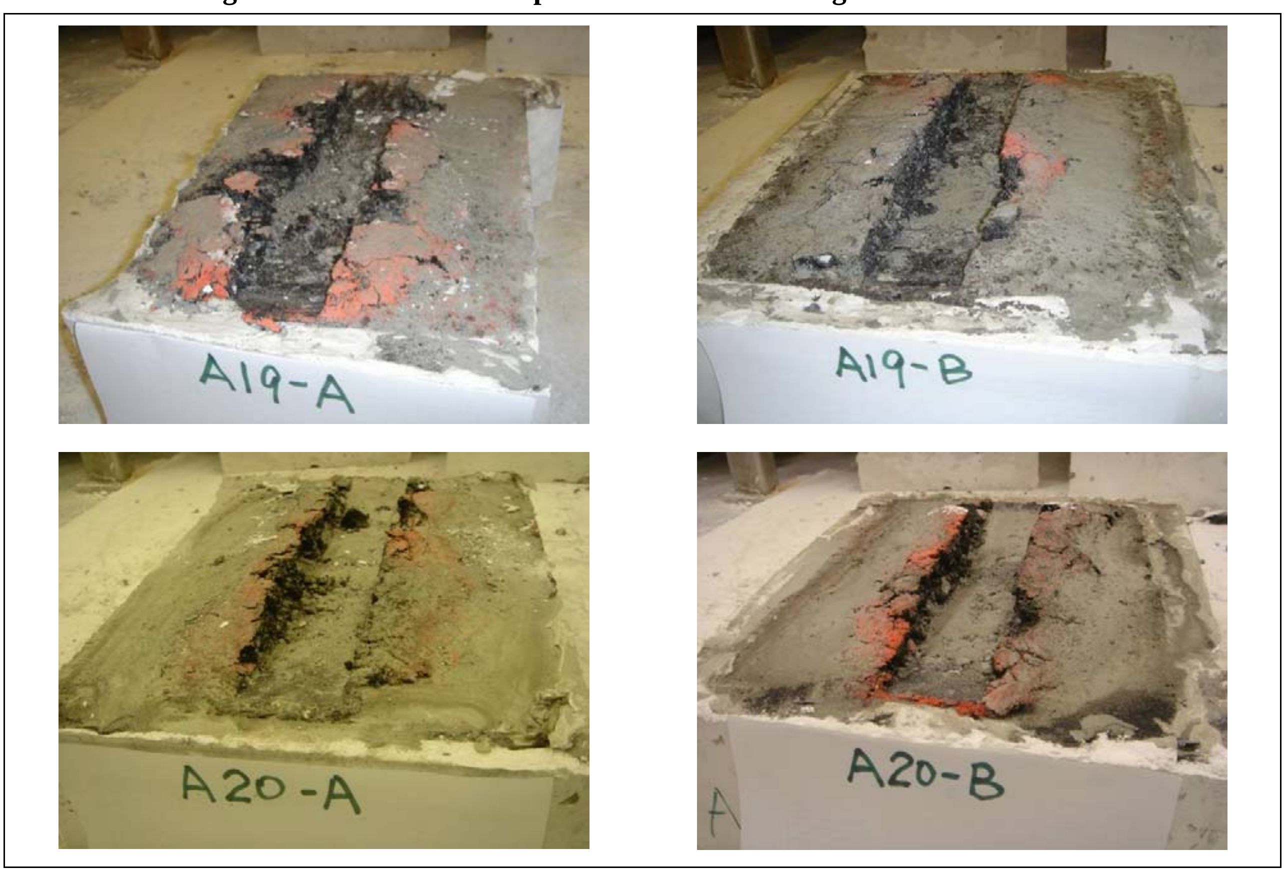


Figure 5.20: Advera specimens after Hamburg Wheel-Track Test.

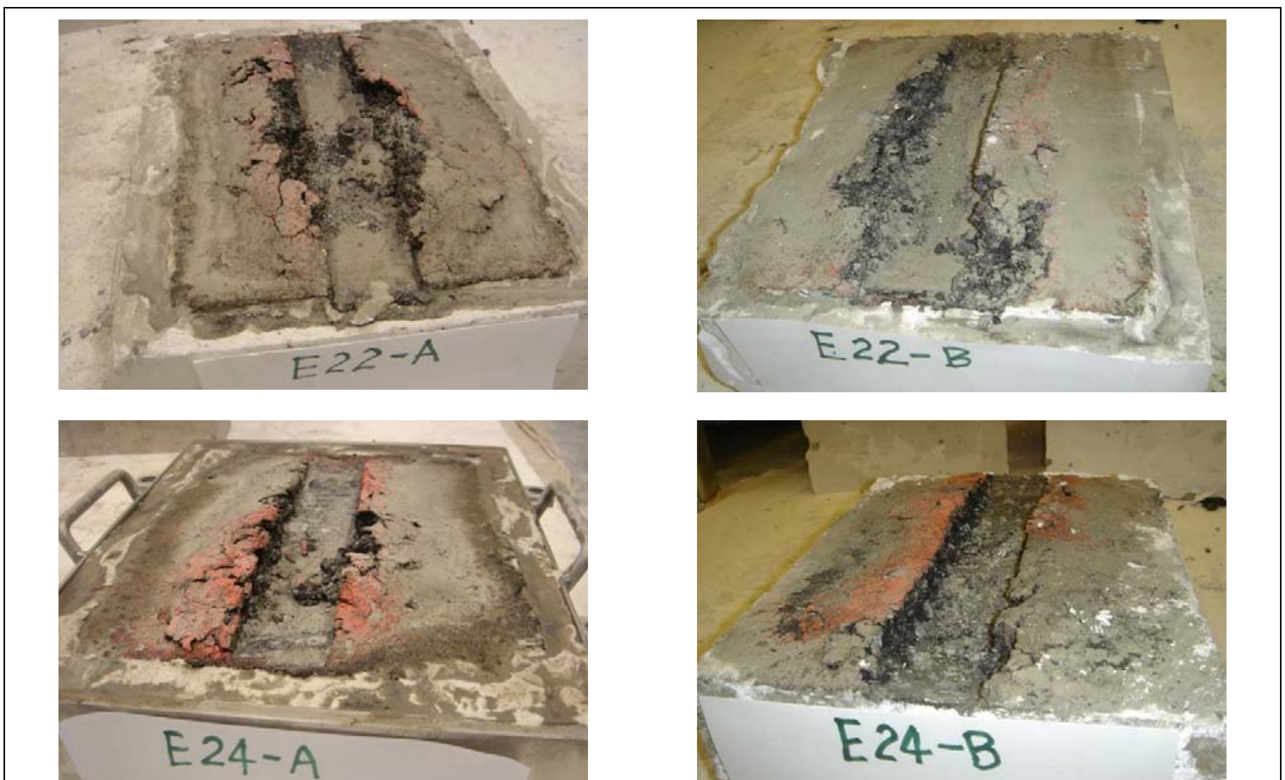


Figure 5.21: Evotherm specimens after Hamburg Wheel-Track Test.

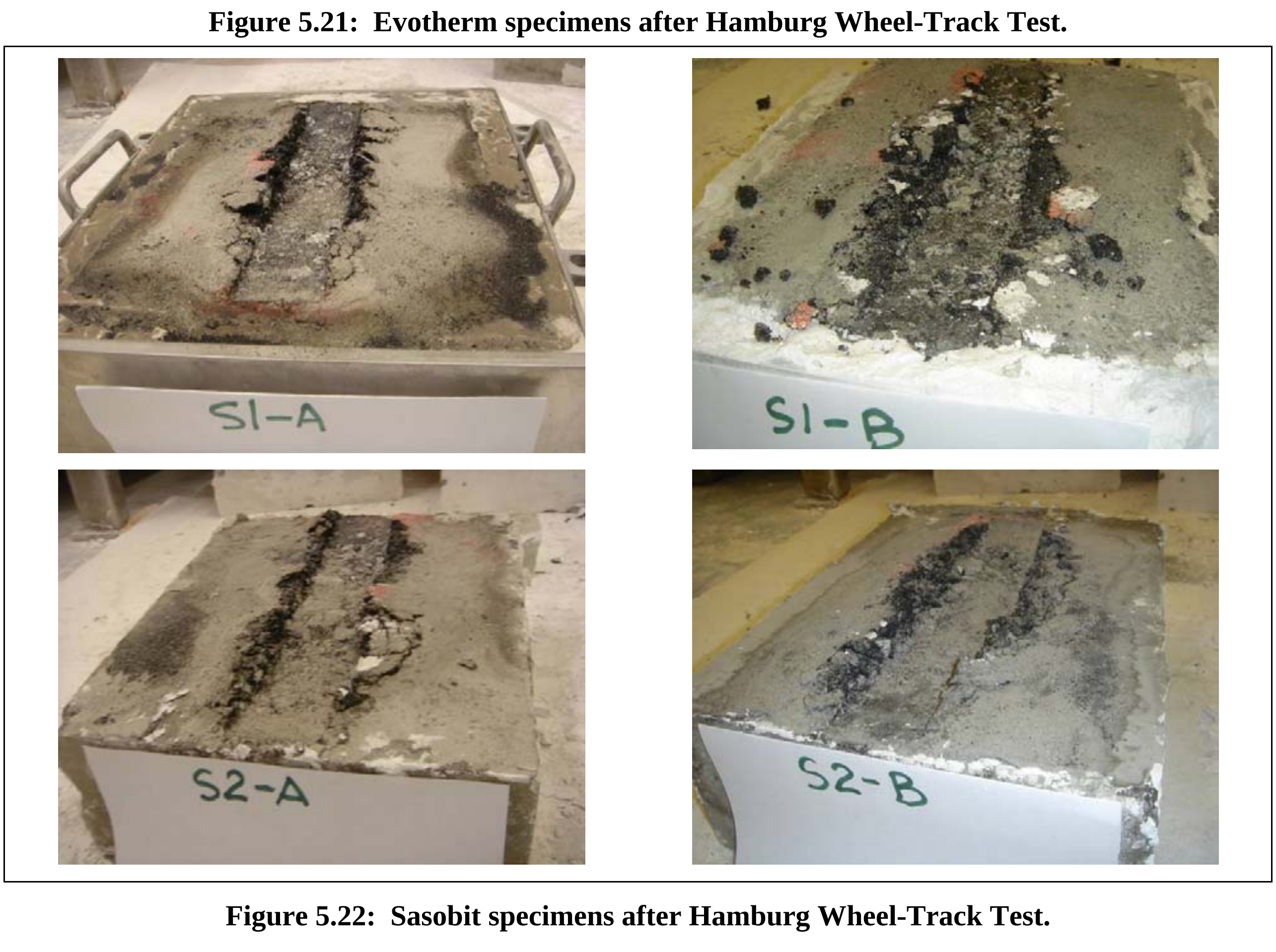


Figure 5.22: Sasobit specimens after Hamburg Wheel-Track Test.

Table 5.9: Test Result Summary of Average Rut Progression Curves

Specimen		Creep Slope (mm/pass)	Stripping Slope (mm/pass)	Inflection Point	Rut Depth @ 10,000 passes (mm)	Rut Depth @ 20,000 passes (mm)
Control	D35A*	-0.0007	-0.0014	7,858	8.2	22.5
	D35B	-0.0010	-0.0013	8,804	12.4	25.5
	D 3A	-0.0011	-0.0018	6,889	15.1	33.1
	D 3B	-0.0007	-0.0023	8,837	11.0	34.0
	Average	-0.0009	-0.0017	8,177	12.9	30.9
Advera	A19A	-0.0012	-0.0018	7,098	15.0	33.3
	A19B	-0.0011	-0.0026	5,896	17.7	45.4
	A20A	-0.0002	-0.0006	10,217	11.2	11.8
	A20B	-0.0010	-0.0025	6,738	16.4	41.5
	Average	-0.0009	-0.0019	7,487	15.1	33.0
Evotherm	E22A	-0.0010	-0.0032	6,132	20.4	52.3
	E22B	-0.0015	-0.0027	7,215	18.7	45.4
	E24A	-0.0008	-0.0017	6,013	13.7	30.7
	E24B	-0.0007	-0.0016	8,804	12.4	25.5
	Average	-0.0010	-0.0023	7,041	16.3	38.5
Sasobit	S 1A	-0.0005	-0.0012	12,423	5.6	16.7
	S 1B	-0.0005	-0.0010	8,804	12.4	25.5
	S 2A	-0.0009	-0.0017	10,013	6.0	27.9
	S 2B	-0.0004	-0.0021	6,934	12.2	33.6
	Average	-0.0006	-0.0015	9,543	9.1	25.9
* Outlier, not used in analysis.						

Table 5.10: Test Result Summary of Maximum Rut Progression Curves

Specimen		Creep Slope (mm/pass)	Stripping Slope (mm/pass)	Inflection Point	Rut Depth @ 10,000 passes (mm)	Rut Depth @ 20,000 passes (mm)
Control	D35A*	-0.0005	-0.0011	10,132	7.8	19.0
	D35B	-0.0010	-0.0026	6,574	16.1	42.6
	D 3A	-0.0010	-0.0021	5,488	17.8	39.1
	D 3B	-0.0013	-0.0034	8,686	15.9	49.8
	Average	-0.0010	-0.0023	7,720	14.4	37.6
Advera	A19A	-0.0008	-0.0037	6,804	19.4	57.1
	A19B	-0.0010	-0.0035	5,936	21.1	58.2
	A20A	-0.0013	-0.0016	5,936	15.7	32.0
	A20B	-0.0012	-0.0022	3,828	19.9	41.0
	Average	-0.0011	-0.0028	5,626	19.0	47.1
Evotherm	E22A	-0.0010	-0.0037	6,421	22.4	59.7
	E22B	-0.0012	-0.0022	303	20.5	45.5
	E24A	-0.0006	-0.0042	5,794	23.6	65.6
	E24B	-0.0009	-0.0021	7,758	15.1	36.7
	Average	-0.0009	-0.0031	5,069	20.4	51.9
Sasobit	S 1A	-0.0007	-0.0027	12,225	6.8	30.9
	S 1B	-0.0007	-0.0017	7,862	9.8	20.9
	S 2A	-0.0001	-0.0015	11,555	6.0	18.5
	S 2B	-0.0003	-0.0038	7,412	15.9	54.0
	Average	-0.0004	-0.0024	9,764	9.6	31.1
* Outlier, not used in analysis.						

The mixes all show similar trends, with air-void content appearing to have the biggest influence on performance. The results in Table 5.9 indicate that there is no significant difference between the Control mix and the mixes with Advera and Evotherm. The Sasobit mix appeared less moisture sensitive than the Control mix, however these results were probably influenced by the lower binder content in the Sasobit mix, which would be expected to increase the rut resistance while also increasing the moisture sensitivity. The tests will be repeated on laboratory-mixed, laboratory-compacted specimens in Phase 2 of the UCPRC study to obtain a better indication of performance at the same binder content.

A one-way analysis of variance, using the stripping slope, stripping inflection point, and rut depth at 10,000 and 20,000 passes as the response variable, revealed no significant difference between the performances of the four mixes. This implies that the use of the additives in this mix design, and compaction to the densities recorded, did not influence the moisture sensitivity of the mix.

Caltrans currently does not specify acceptance criteria for the Hamburg Wheel-Track Test, and the results can therefore not be interpreted in terms of Caltrans requirements. The current Texas Department of Transportation specifications specify the minimum number of wheel passes at 12.5 mm (0.5 in.) maximum rut depth. To accept a mix using a PG64-16 binder, a minimum of 10,000 passes before the maximum rut depth reaches 12.5 mm is required. Based on the results listed in Table 5.10, the Control, Advera, and Evotherm mixes did not meet this requirement, while the Sasobit mix barely met the criteria. The finding does not change if the average maximum rut depth is used instead of the maximum rut depth; however, the Control mix is closer to the acceptance point, and the Sasobit mix exceeds the requirement by a greater margin. As discussed previously, the Sasobit mix result cannot be compared directly with the Control and the other additive mixes given the differences in its asphalt binder content.

5.2.5 Moisture Sensitivity: Tensile Strength Retained (TSR)

Air-Void Content

The air-void content of each Tensile Strength Retained (TSR) specimen was calculated from the bulk specific gravity (Method A of AASHTO T-166) and the theoretical maximum specific gravity (ASTM D-2041). Results are shown in Table 5.11 and can be summarized as follows:

- Control: 5.5 to 6.6 percent
- Advera: 6.3 to 6.9 percent
- Evotherm: 6.3 to 8.1 percent
- Sasobit: 5.8 to 7.7 percent.

Testing

The Tensile Strength Retained for each mix is summarized in Table 5.12.

Table 5.11: Air-Void Content of TSR Test Specimens

Specimen ID			Bulk Specific Gravity (g/cm^3)		Max Specific Gravity (g/cm^3)		Air-Void Content (%)	
	Dry	Wet	Dry	Wet	Dry	Wet	Dry	Wet
Control	33-20C	33-15C	2.433	2.428	2.576	2.576	5.5	5.7
	33-08C	33-13C	2.424	2.424	2.576	2.576	5.9	5.9
	33-17C	33-02C	2.422	2.424	2.576	2.576	6.0	5.9
	33-07C	33-06C	2.419	2.412	2.576	2.576	6.1	6.4
	33-09C	33-10C	2.412	2.408	2.576	2.576	6.4	6.5
	33-11C	33-01C	2.409	2.405	2.576	2.576	6.5	6.6
						Average	6.1	6.2
Advera	34-06T	28-08B	2.427	2.431	2.596	2.596	6.5	6.3
	28-08T	28-07T	2.423	2.419	2.596	2.596	6.7	6.8
	28-01B	34-03T	2.422	2.423	2.596	2.596	6.7	6.6
	28-03B	28-02B	2.422	2.425	2.596	2.596	6.7	6.6
	28-10T	28-10B	2.420	2.420	2.596	2.596	6.8	6.8
	28-06T	28-01T	2.417	2.419	2.596	2.596	6.9	6.8
	34-05T	-	2.416	-	2.596	-	6.9	-
						Average	6.7	6.7
Evotherm	16-13C	16-09C	2.424	2.425	2.589	2.589	6.4	6.3
	16-19C	16-03C	2.408	2.406	2.589	2.589	7.0	7.1
	16-02C	13-02B	2.405	2.396	2.589	2.589	7.1	7.5
	13-01B	16-04C	2.391	2.383	2.589	2.589	7.7	8.0
	16-10C	16-07C	2.386	2.381	2.589	2.589	7.9	8.0
	16-11C	16-01C	2.381	2.379	2.589	2.589	8.0	8.2
						Average	7.5	7.4
Sasobit	12-02T	12-03T	2.414	2.400	2.598	2.598	7.1	7.6
	12-04B	12-10T	2.445	2.407	2.598	2.598	5.9	7.3
	12-01T	02-10T	2.403	2.446	2.598	2.598	7.5	5.8
	12-01B	02-10B	2.443	2.443	2.598	2.598	6.0	5.9
	12-08T	12-09T	2.423	2.419	2.598	2.598	6.7	6.9
	12-11T	12-06T	2.420	2.419	2.598	2.598	6.9	6.9
						Average	6.7	6.7

Table 5.12: Summary of TSR Test Results

Specimen	Control		Advera		Evotherm		Sasobit	
	Dry ITS	Wet ITS	Dry ITS	Wet ITS	Dry ITS	Wet ITS	Dry ITS	Wet ITS
1	1111.4	660.2	923.1	433.5	888.6	555.1	988.3	508.7
2	841.7	516.8	909.7	509.3	881.6	549.6	905.2	557.6
3	825.9	482.4	954.0	444.6	836.5	596.9	878.6	497.9
4	841.3	598.4	918.0	492.8	823.8	504.4	888.0	522.4
Average	905.8	564.4	926.2	470.1	857.6	551.5	915.0	521.7
TSR	62%		51%		64%		57%	
Damage	-	Yes	-	Yes	-	Yes	-	Yes

The recorded TSR values are all lower than the tentative criteria in the Caltrans Testing and Treatment Matrix to ensure moisture resistance (minimum 70 percent for low environmental risk regions, and minimum 75 percent for medium and high environmental risk regions). Treatment would therefore typically be required for all mixes to bring the mix test results up to the minimum to reduce the risk of moisture damage in the pavement.

The results show no specific trend with other observations in the study. The Control mix was not the best performer, despite having the lowest average air-void content. The Evotherm mix had the highest TSR value and the highest average air-void content. A plot of air-void contents versus indirect tensile strength shows that air-void contents in the range of 6.0 percent and 8.0 percent did not have a significant effect on the indirect tensile strengths of the four mixes (Figure 5.23), while air-void contents outside this range appeared to have some effect. Figure 5.23 also shows that, based on the specimens with an air-void content between 6.5 percent and 7.5 percent, the four mixes showed no significant difference in terms of dry or wet strength.

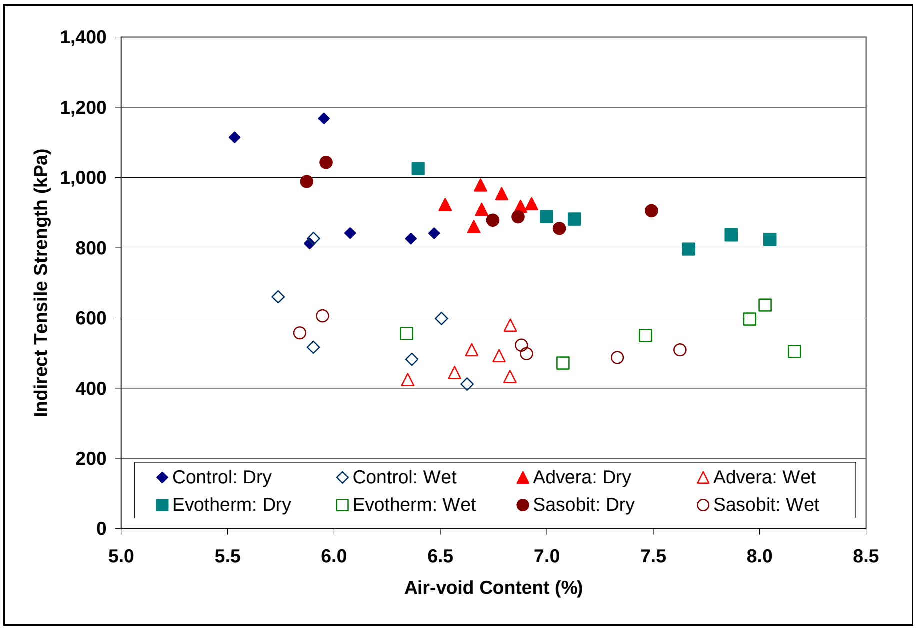


Figure 5.23: Air-void content versus indirect tensile strength.

Observation of the split faces of the cores revealed that all mixes showed some internal stripping (loss of adhesion between asphalt and aggregate evidenced by clean aggregate on the broken face) after moisture conditioning. There was no significant difference between the four mixes in terms of observed moisture resistance.

5.3 Summary of Laboratory Testing Results

The laboratory test results indicate that use of the warm-mix additives assessed in this study, produced and compacted at lower temperatures, does not significantly influence the performance of the asphalt concrete when compared to control specimens produced and compacted at conventional hot-mix asphalt temperatures.

Moisture sensitivity testing indicated that all the mixes tested were potentially susceptible to moisture damage. There was no difference in the level of moisture sensitivity between the Control mix and mixes with warm-mix additives.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

This first-level report describes the first phase of a warm-mix asphalt study, which compares the performance of a control mix, produced and constructed at conventional hot-mix asphalt temperatures, with three mixes produced with warm-mix additives, produced and compacted at approximately 35°C (60°F) lower than the control. The additives tested included *Advera WMA®*, *Evotherm DAT™*, and *Sasobit®*. The test track layout and design, mix design and production, and test track construction are discussed, as well are results of Heavy Vehicle Simulator (HVS) and laboratory testing.

Key findings from the study include:

- A consistent base-course was constructed on the test track using material produced at the nearby quarry. Some overwatering occurred in the early stages of construction resulting in some moist areas in the pavement, which influenced deflection and density measurements. These areas were unlikely to effect later performance of the test track. The very stiff base is likely to complicate any planned fatigue cracking experiments in that a very high number of HVS repetitions will likely be required before any distress occurs.
- Minimal asphalt plant modifications were required to accommodate the warm-mix additives.
- No problems were noted with producing the asphalt mixes at the lower temperatures. The target mix production temperatures (i.e., 155°C and 120°C [310°F and 250°F]) were achieved.
- Although a PG 64-16 asphalt binder was specified in the work plan, subsequent tests by the Federal Highway Administration indicated that the binder should be graded as PG64-22. This should not affect the outcome of the experiment. After mixing Advera and Sasobit with the binder, the PG grading changed from PG 64-22 to PG 70-22. The addition of Evotherm did not alter the PG grade.
- The Control, Advera, and Evotherm mixes met the project mix design requirements. The binder content of the Sasobit mix was 0.72 percent below the target binder content and 0.62 percent below the lowest recommended binder content (by mass of aggregate) from the Hveem mix design (Appendix A). This probably influenced performance and was taken into consideration when interpreting HVS and laboratory test results.
- The Control mix had a higher specific gravity, higher Marshall stability, and lower air-void content, compared to the mixes with additives. It is not clear whether this was a testing inconsistency or is linked to the lower production and specimen preparation temperatures. This will need to be investigated during Phase 2 laboratory investigations.

- Moisture contents of the mixes with additives were notably higher than in the Control mix, indicating that potentially less moisture evaporates from the aggregate at lower production temperatures. All mixes were, however, well within the minimum Caltrans-specified moisture content level. Aggregate moisture contents will need to be controlled in the stockpiles, and maximum moisture contents prior to mix production may need to be set if warm-mix technologies are routinely used.
- Construction procedures and final pavement quality did not appear to be influenced by the lower construction temperatures. The Advera mix showed no evidence of tenderness, and acceptable compaction was achieved. Some tenderness resulting in shearing under the rollers was noted at various stages of breakdown and/or rubber-tired rolling on the Evotherm and Sasobit sections, indicating that the compaction temperatures were still higher than optimal. No problems were observed after final rolling at lower temperatures.
- Interviews with the paving crew after construction revealed that no problems were experienced with construction at the lower temperatures. Improved working conditions were identified as an advantage. Tenderness on the Evotherm and Sasobit sections was not considered as being significantly different from that experienced with conventional mixes during normal construction activities.
- Although temperatures at the beginning of compaction on the warm-mix sections were considerably lower than the Caltrans-specified limits, the temperatures recorded on completion of compaction were within limits, indicating that the rate of temperature loss in the mixes with additives was lower than that on the Control mix.
- Some haze/smoke was evident on the Control mix during transfer of the mix from the truck to the paver. No haze or smoke was observed on the mixes with additives.
- Average air-void contents on the Control and Advera sections were 5.6 percent and 5.4 percent respectively. Those on the Evotherm and Sasobit sections, which showed signs of tenderness during rolling, were approximately 7.0 percent. Based on these observations, it was concluded that adequate compaction can be achieved on warm-mixes at the lower temperatures. Optimal compaction temperatures are likely to differ between the different warm-mix technologies.
- Skid resistance measurements indicated that the warm-mix additives tested do not influence the skid resistance of an asphalt mix.
- HVS trafficking on each of the four sections revealed that the duration of the embedment phases on the Advera and Evotherm sections were similar to the Control. However, the depth of the ruts at the end of the embedment phases on these two sections was slightly higher than the Control, which was attributed to less oxidation of the binder and possibly also to the retention of somewhat higher water contents in the aggregate during mix production at lower temperatures. Rutting behavior on the

warm-mix sections followed similar trends to the control after the embedment phase. The performance of the Sasobit section could not be compared with the other three sections given that the binder content of the mix was significantly lower.

- Laboratory test results indicate that use of the warm-mix technologies assessed in this study to produce and compact mixes at lower temperatures does not significantly influence the performance of the asphalt concrete when compared to control specimens produced and compacted at conventional hot-mix asphalt temperatures. However, moisture sensitivity testing indicated that all the mixes tested were potentially susceptible to moisture damage. There was, however, no difference in the level of moisture sensitivity between the Control mix and mixes with warm-mix additives.

The findings of the study are also summarized below in the form of answers to the questions identified in Section 1.3.

6.1.1 Comparative Energy Usage

Comparative energy usage could not be assessed in this study due to the very small quantities produced. These studies will need to be carried out during larger full-scale pilot studies on in-service pavements when large quantities of mix are produced (i.e., more than 5,000 tonnes).

6.1.2 Achieving Compaction Density at Lower Temperatures

Compaction measurements during construction indicated that average air-void contents on the Control and warm-mix sections were typical of full-scale construction projects, and based on these observations it is concluded that adequate compaction can be achieved on warm-mixes at the lower temperatures.

6.1.3 Optimal Temperature Ranges for Warm-Mixes

Optimal compaction temperatures are likely to differ between the different warm-mix technologies. This study has shown that temperatures of at least 35°C (60°F) lower than conventional temperatures are appropriate for producing and compacting the modified mixes.

6.1.4 Cost Implications

The cost benefits of using the warm-mix technologies could not be assessed in this study due to the very small quantities produced.

6.1.5 Rutting Performance

Based on the results of HVS testing, it is concluded that the use of any of the three warm-mix asphalt technologies used in this experiment will not significantly influence the rutting performance of the mix.

6.1.6 Moisture Sensitivity

Laboratory moisture sensitivity testing indicated that all the mixes tested were potentially susceptible to moisture damage. There was, however, no difference in the level of moisture sensitivity between the Control mix and mixes with additives. It is recommended that Phase 2 HVS testing be carried out to confirm these findings under full-scale loading conditions.

6.1.7 Fatigue Performance

Laboratory fatigue testing indicated that the warm-mix technologies used in this study will not influence the fatigue performance of a mix. Given the very strong pavement structure on the test track, it is unlikely that fatigue cracking will occur under HVS testing. An assessment of fatigue performance is therefore not recommended using these test sections.

6.1.8 Other Effects

Quality control checks carried out by Graniterock Company on the mix immediately after production revealed that lower specific gravities and higher air-void contents were recorded on the mixes produced with warm-mix additives. This anomaly should be assessed in more detail during Phase 2 laboratory testing, when performance will be assessed on laboratory-mixed, laboratory-compacted specimens.

6.1.9 Rubberized and Open-Graded Mixes

At the time of preparing this report, no decision had been made on extending the study to assess rubberized and open-graded mixes.

6.2 Recommendations

The HVS and laboratory testing completed in this phase have provided no results to suggest that warm-mix technologies should not be used in California. However, the testing discussed in this report is part of a larger study on warm-mix asphalt and final recommendations towards the use of this technology will only be made after further research and monitoring of full-scale pilot studies on in-service pavements is completed. Interim recommendations include the following:

- The use of warm-mix technologies should continue in full-scale pilot studies on in-service pavements.

- Although laboratory testing indicated that the warm-mix technologies assessed in this study did not increase the moisture sensitivity of the mix, HVS testing to assess moisture sensitivity should continue as recommended in the work plan to confirm these findings. Subsequent laboratory testing of moisture sensitivity should assess a range of different aggregates given that all of the mixes tested in this study where considered to be moisture sensitive.
- Laboratory testing on laboratory-mixed, laboratory-compacted specimens should proceed as recommended in the test plan to determine whether representative mixes can be produced in the laboratory.

7. REFERENCES

1. **Warm-Mix Asphalt: European Practice.** 2008. Washington, D.C.: Federal Highway Administration. (FHWA-PL-08-007).
2. HURLEY, G.C. and Prowell, B.D. 2006. Evaluation of Potential Processes for use in Warm-Mix Asphalt. **Journal of the Association of Asphalt Paving Technologists, Vol. 75.** (pp 41-85).
3. JONES, D. and Harvey, J. 2007. **Warm-Mix Asphalt Study: Workplan for Comparison of Conventional and Warm-Mix Asphalt Performance using HVS and Laboratory Testing.** Davis and Berkeley, CA: University of California Pavement Research Center. (WP-2007-01).
4. **Standard Specifications.** 2006. Sacramento, CA: State of California Department of Transportation.
5. JONES, D. 2005. **Quality Management System for Site Establishment, Daily Operations, Instrumentation, Data Collection and Data Storage for APT Experiments.** Pretoria, South Africa: CSIR Transportek. (Contract Report CR-2004/67-v2).
6. HARVEY, J., Du Plessis, L., Long, F., Deacon, J., Guada, I., Hung, D. and Scheffy, C. 1997. **CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Asphalt-Treated Permeable Base (ATPB) – Section 500RF**. Davis and Berkeley, CA: University of California Pavement Research Center. (Report Numbers UCPRC-RR-1999-02 and RTA-65W4845-3).
7. HARVEY, J., Du Plessis, L., Long, F., Deacon, J., Guada, I., Hung, D. and Scheffy, C. 1997. **CAL/APT Program: Test Results from Accelerated Pavement Test on Pavement Structure Containing Untreated Base – Section 501RF.** Davis and Berkeley, CA: University of California Pavement Research Center. (Report Numbers UCPRC-RR-1997-03 and RTA-65W4845-3).

APPENDIX A: MIX DESIGN EXAMPLES

A.1 Mix Design

Examples of Graniterock Company and Caltrans mix designs used for the production of asphalt concrete at the Graniterock Company's A.R. Wilson Asphalt Plant for earlier Caltrans projects are provided in Figure A.1 and Figure A.2. A Graniterock Company mix design was used in this study.

Graniterock
MATERIAL SUPPLIER / ENGINEERING CONTRACTOR • LICENSE #22

Project:

Plant: Aromas Drum Plant
Mix Type: 19 mm Coarse, Type A
Asphalt Binder: PG 64-10 (Valero Benecia)

Design Completed:

MIX PROPERTIES

Specimen	Binder Content	Bulk Specific Gravity CT 308C (g/cm^3)	Maximum Theoretical Density CT 309 (g/cm^3)	% Air Voids CT 309	STABILITY S-value CT 366	Voids in Mineral Aggregate % (VMA)
A	4.5%	2.427	2.596	6.5	42	14.4
B	5.0%	2.439	2.574	5.2	45	14.4
C	5.5%	2.456	2.553	3.8	42	14.2
D	6.0%	2.466	2.536	2.8	38	14.3
Asphalt binder Specific Gravity = 1.027				**Target Asphalt Content =**		**5.4%**

AGGREGATE PROPERTIES

Caltrans Test Method		CTM #	Value	Spec Type A		
Percentage crushed particles		205	100	90/70		
Los Angeles Rattler	100 rev	211	9	10 max.		
	500 rev		30	45 max.		
Sand Equivalent		217	72	47 min.		
KC/KF Factor		303	1.0/1.1	1.7 max		
Fine Aggregate App. SG		208	2.81			
Fine Aggregate Bulk SG		207	2.63	---		
Coarse Aggregate Bulk SG		206	2.80	---		
Combined Bulk SG			2.71	---	Combined Effective SG (Gse) =	2.78
Swell		305	0.2	0.76 max		

JOB MIX FORMULA and COLD FEED PERCENTAGES

	AGGREGATE BIN GRADATIONS CTM 202								
	3/4x1/2	1/2x #4	1/4x #10	Sand	Dust	COMBINED GRADING	SPEC LIMITS CALTRANS	TARGET "X" Values	OPERATING RANGE
BIN %	18	35	10	37	0				
SIEVE SIZE									
25mm	100	100	100	100	100	100	100		100
19mm	75	100	100	100	100	96	90-100	96	91-100
12.5mm	23	95	100	100	100	84			
9.5mm	12	65	99	100	100	72	60-75	72	66-78
4.75mm	9	12	65	100	100	49	45-50	49	42-56
2.36mm	7	7	14	88	100	38	32-36	36	31-41
1130um	6	5	7	61	100	26			
600um	5	5	5	38	100	17	15-18	18	14-22
300um	4	4	4	19	100	9			
150um	3	3	3	10	100	5			
75um	1	2	2	6	95	3.5	3-7	4	2--6

Figure A.1: Example Graniterock Company mix design.

Project:

Plant: Aromas Drum Plant
Mix Type: 19 mm Coarse, Type A
Asphalt Binder: PG 64-10 (Valero Benecia)

Design Completed: January 0, 1900

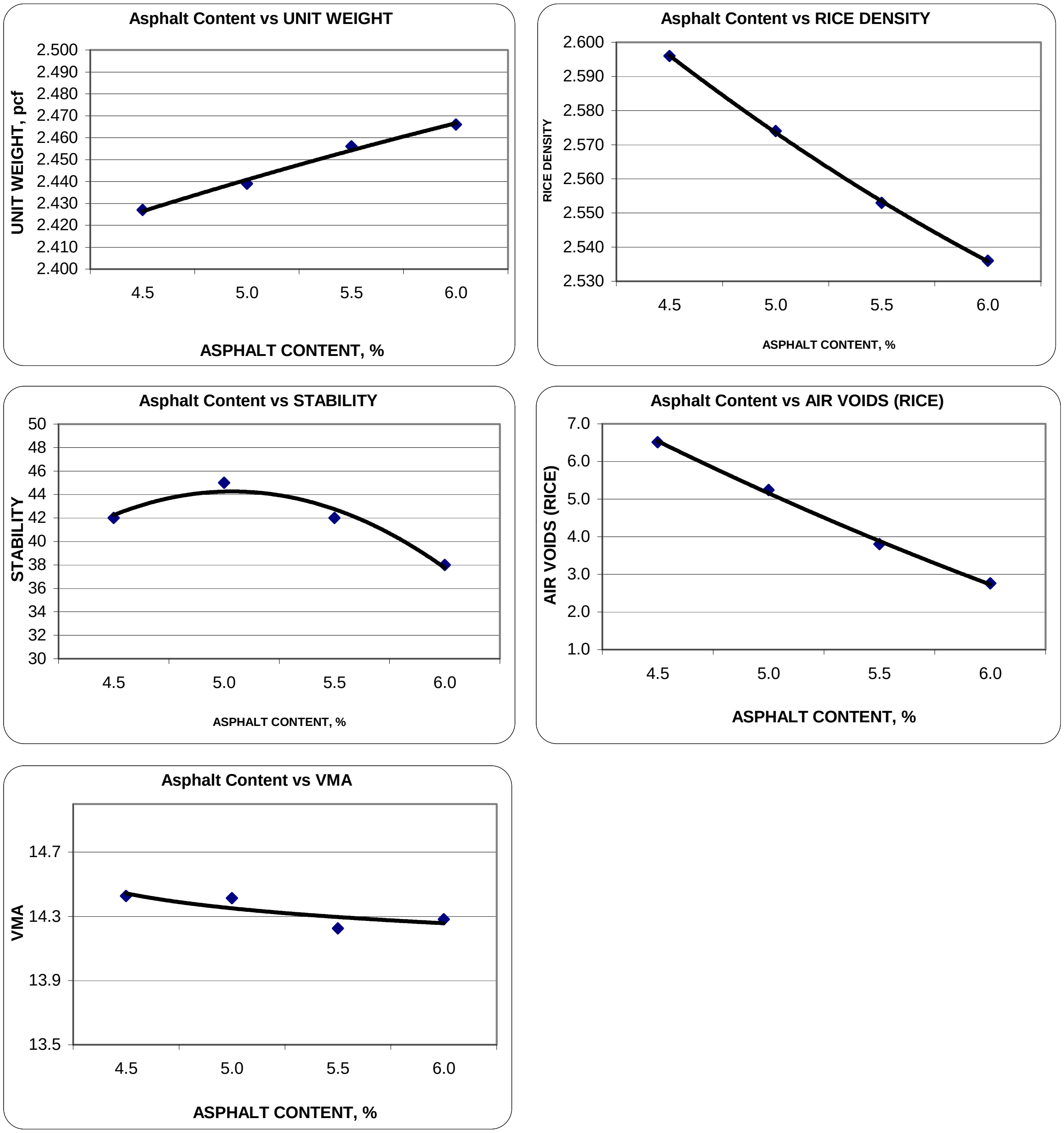


Figure A.1: Example Graniterock Company mix design (*continued*).

19mm Max Coarse, Type A
JOB MIX FORMULA

PERCENT PASSING

SIEVE SIZE (mm, um)		25	19	12.5	9.5	4.75	2.36	1130	600	300	150	75
UPPER SPECIFIED LIMIT		100	100	90	78	56	41	30	22	15	8.0	6.0
LOWER SPECIFIED LIMIT		100	91	78	66	42	31	22	14	7.0	4.0	2.0
JOB MIX FORMULA		**100**	**96**	**84**	**72**	**49**	**36**	**24**	**18**	**10**	**6.0**	**4.0**

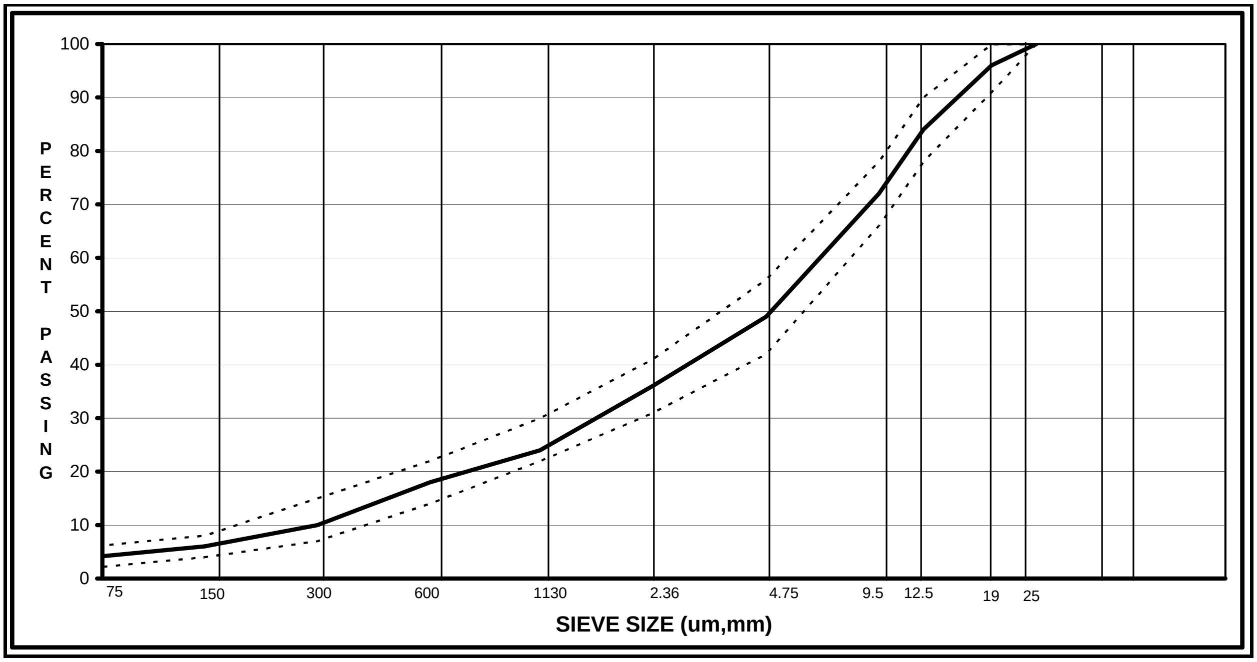
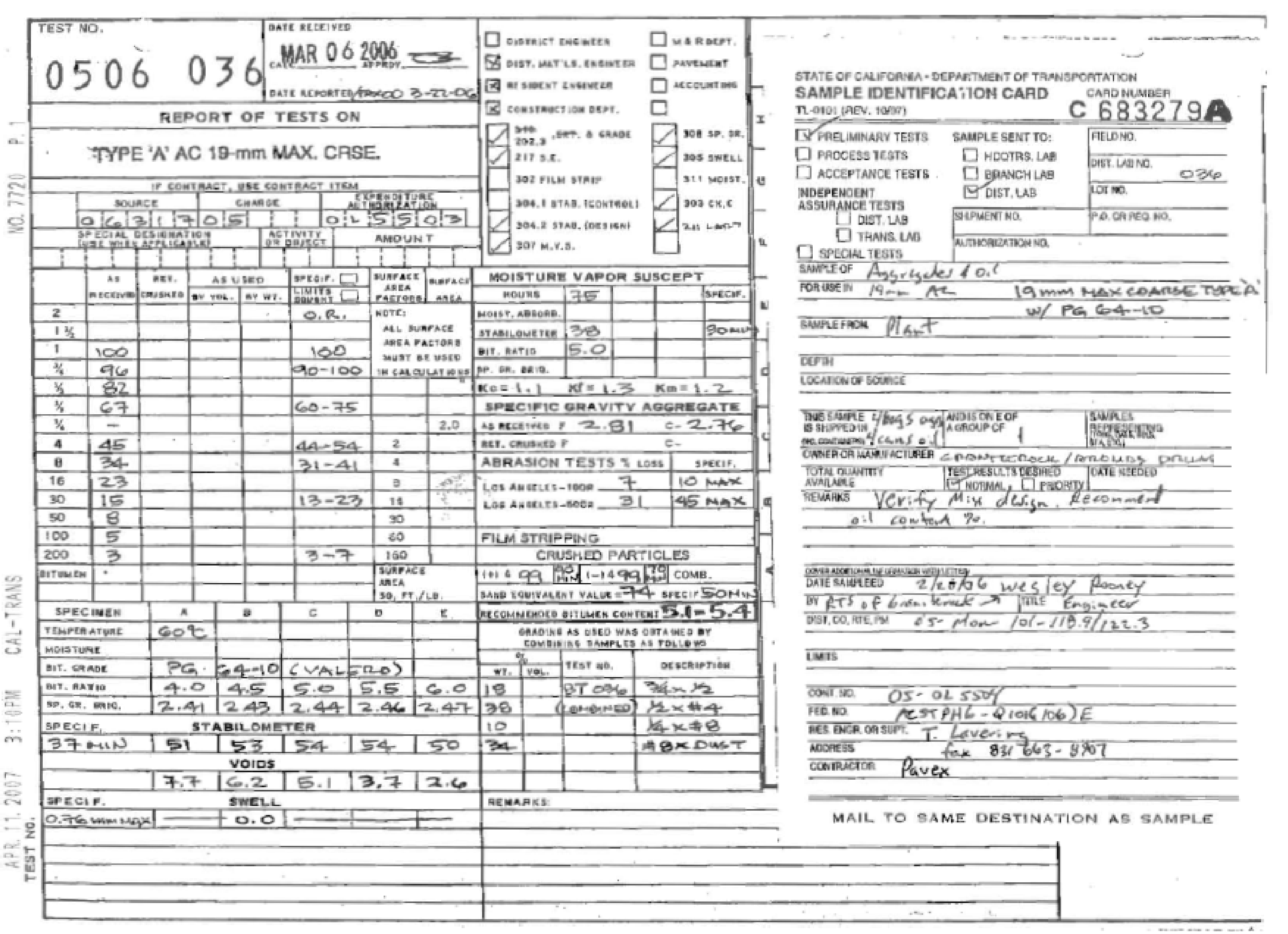


Figure A.1: Example Graniterock Company mix design (*continued*).


TEST NO. 0506 036

DATE RECEIVED MAR 06 2006

DATE REPORTED 3-22-06

REPORT OF TESTS ON

TYPE 'A' AC 19-mm MAX. CRSE.

IF CONTRACT, USE CONTRACT ITEM

SOURCE	CHARGE	EXPENDITURE AUTHORIZATION
063170S		0L5503

SPECIAL DESIGNATION (USE WHEN APPLICABLE) — ACTIVITY OR OBJECT — AMOUNT

☐ DISTRICT ENGINEER ☐ M & R DEPT.
☑ DIST. MAT'LS. ENGINEER ☐ PAVEMENT
☑ RESIDENT ENGINEER ☐ ACCOUNTING
☑ CONSTRUCTION DEPT. ☐

☑ 202.3 GRAD. & GRADE ☑ 308 SP. GR.
☑ 217 S.E. ☑ 305 SWELL
☐ 302 FILM STRIP ☐ 311 MOIST.
☐ 304.1 STAB. (CONTROL) ☑ 303 CK.E
☑ 304.2 STAB. (DESIGN) ☑ 211 L.A.R.
☑ 307 M.V.S.

	AS RECEIVED	RET. CRUSHED	AS USED BY VOL.	AS USED BY WT.	SPECIF. LIMITS	SURFACE AREA FACTORS	SURFACE AREA
2					O.R.	NOTE:	
1½						ALL SURFACE	
1	100				100	AREA FACTORS	
¾	96				90-100	MUST BE USED IN CALCULATIONS	
½	82						
⅜	67				60-75		
¼	—						2.0
4	45				44-54	2	
8	34				31-41	4	
16	23					8	
30	15				13-23	14	
50	8					30	
100	5					60	
200	3				3-7	160	
BITUMEN						SURFACE AREA SQ. FT./LB.	

MOISTURE VAPOR SUSCEPT

HOURS	75				SPECIF.
MOIST. ABSORB.					
STABILOMETER	38				30 min
BIT. RATIO	5.0				
SP. GR. BRIQ.					

Kc = 1.1 Kf = 1.3 Km = 1.2

SPECIFIC GRAVITY AGGREGATE

AS RECEIVED F 2.81 C- 2.76

RET. CRUSHED F C-

ABRASION TESTS % LOSS — SPECIF.

LOS ANGELES—100R 7 — 10 MAX

LOS ANGELES—500R 31 — 45 MAX

FILM STRIPPING

CRUSHED PARTICLES

(+) 4 99 (-) 4 99 COMB.

SAND EQUIVALENT VALUE = 74 SPECIF 50 Min

RECOMMENDED BITUMEN CONTENT 5.1-5.4

SPECIMEN	A	B	C	D	E
TEMPERATURE	60°C				
MOISTURE					
BIT. GRADE	PG 64-10 (VALERO)				
BIT. RATIO	4.0	4.5	5.0	5.5	6.0
SP. GR. BRIQ.	2.41	2.43	2.44	2.46	2.47
SPECIF.	STABILOMETER				
37 min	51	53	54	54	50
	VOIDS				
	7.7	6.2	5.1	3.7	2.6
SPECIF.	SWELL				
0.76 mm max	—	0.0	—	—	—

GRADING AS USED WAS OBTAINED BY COMBINING SAMPLES AS FOLLOWS

% WT.	% VOL.	TEST NO.	DESCRIPTION
18		3T 036	¾ x ½
38		(COMBINED)	½ x #4
10			¼ x #8
34			#8 x DUST

REMARKS:

STATE OF CALIFORNIA - DEPARTMENT OF TRANSPORTATION

SAMPLE IDENTIFICATION CARD — CARD NUMBER C 683279A

TL-0101 (REV. 10/97)

☑ PRELIMINARY TESTS ☐ PROCESS TESTS ☐ ACCEPTANCE TESTS

INDEPENDENT ASSURANCE TESTS ☐ DIST. LAB ☐ TRANS. LAB

☐ SPECIAL TESTS

SAMPLE SENT TO: ☐ HDQTRS. LAB ☐ BRANCH LAB ☑ DIST. LAB

FIELD NO. — DIST. LAB NO. 036 — LOT NO. — SHIPMENT NO. — P.O. OR REQ. NO. — AUTHORIZATION NO.

SAMPLE OF Aggregates & oil

FOR USE IN 19mm AC 19 mm MAX COARSE TYPE A w/ PG 64-10

SAMPLE FROM Plant

DEPTH

LOCATION OF SOURCE

THIS SAMPLE IS SHIPPED IN 4 bags 5 gal cans oil — AND IS ONE OF A GROUP OF — SAMPLES REPRESENTING

OWNER OR MANUFACTURER

TOTAL QUANTITY AVAILABLE — TEST RESULTS DESIRED ☑ NORMAL ☐ PRIORITY — DATE NEEDED

REMARKS Verify Mix design. Recommend oil content %.

DATE SAMPLED 2/28/06 Wesley Rooney

BY RTS of Granite Rock — TITLE Engineer

DIST, CO, RTE, PM 05-Mon 101-119.9/122.3

LIMITS

CONT. NO. 05-0L5504

FED. NO. ACSTPHL-Q101(106)E

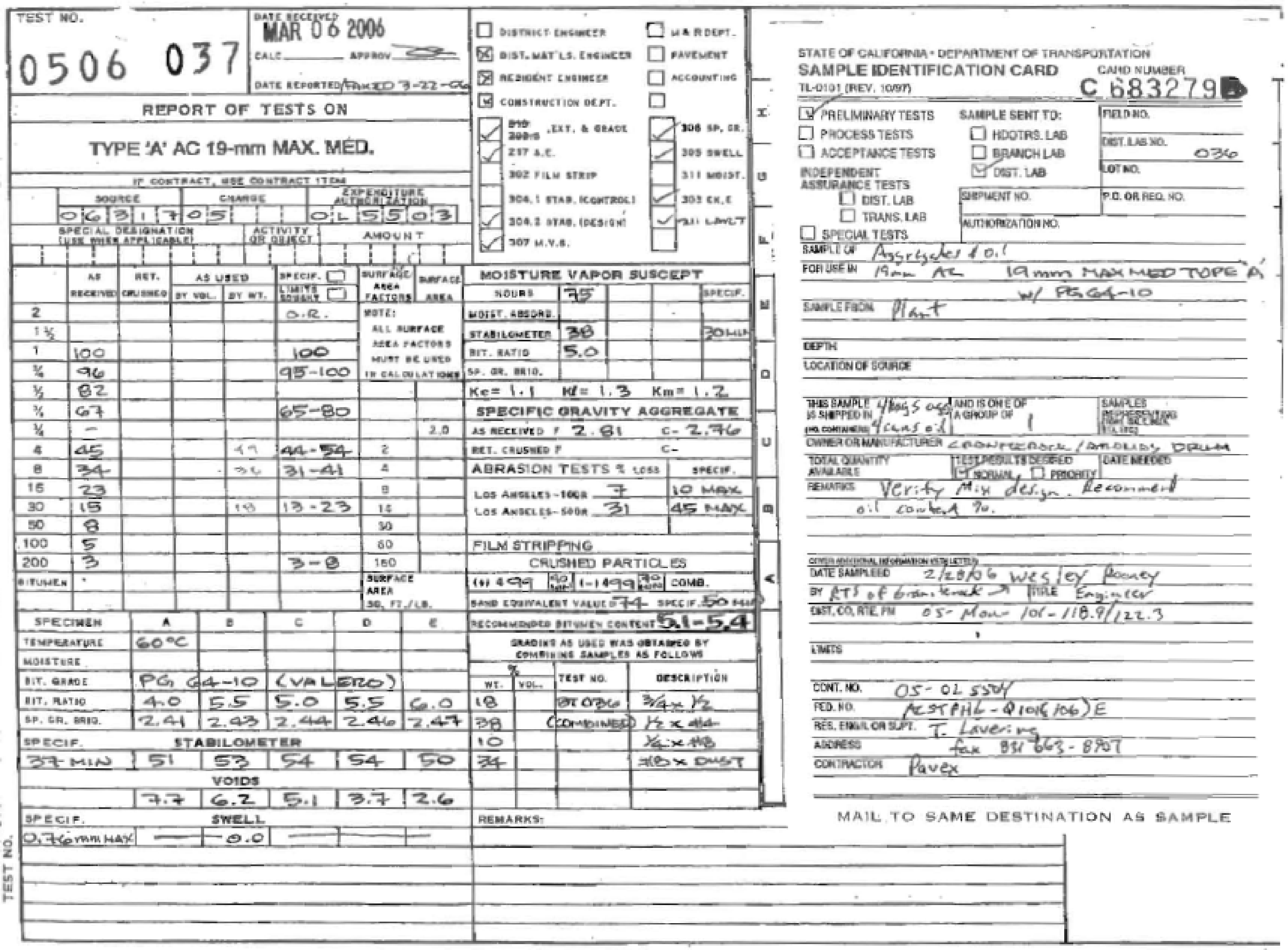
RES. ENGR. OR SUPT. T. Levering

ADDRESS fax 831 663-8907

CONTRACTOR Pavex

MAIL TO SAME DESTINATION AS SAMPLE

Figure A.2: Example Caltrans mix design.



TEST NO. 0506 037

DATE RECEIVED MAR 06 2006

DATE REPORTED FAXED 3-22-06

- [ ] DISTRICT ENGINEER
- [ ] M & R DEPT.
- [x] DIST. MAT'LS. ENGINEER
- [ ] PAVEMENT
- [x] RESIDENT ENGINEER
- [ ] ACCOUNTING
- [x] CONSTRUCTION DEPT.

REPORT OF TESTS ON

TYPE 'A' AC 19-mm MAX. MED.

SOURCE	CHARGE	EXPENDITURE AUTHORIZATION
06 3 1 7 0 5		0 L 5 5 0 3

Sieve	As Received	Specif. Limits	Surface Area
2		O.R.	
1½			
1	100	100	
¾	96	95-100	
½	82		
⅜	67	65-80	
¼	—		
4	45	44-54	
8	34	31-41	
16	23		
30	15	13-23	
50	8		
100	5		
200	3	3-8	

MOISTURE VAPOR SUSCEPT: HOURS 75; MOIST. ABSORB.; STABILOMETER 38, SPECIF. 30 MIN; BIT. RATIO 5.0

Kc = 1.1 Kf = 1.3 Km = 1.2

SPECIFIC GRAVITY AGGREGATE: AS RECEIVED F 2.81 C- 2.76

ABRASION TESTS % LOSS: LOS ANGELES-100R 7 (10 MAX); LOS ANGELES-500R 31 (45 MAX)

FILM STRIPPING

CRUSHED PARTICLES

SAND EQUIVALENT VALUE 74 SPECIF. 50 MIN

RECOMMENDED BITUMEN CONTENT 5.1-5.4

SPECIMEN	A	B	C	D	E
TEMPERATURE	60°C				
MOISTURE					
BIT. GRADE	PG 64-10 (VALERO)				
BIT. RATIO	4.0	5.5	5.0	5.5	6.0
SP. GR. BRIQ.	2.41	2.43	2.44	2.46	2.47
STABILOMETER (SPECIF. 37 MIN)	51	53	54	54	50
VOIDS	7.7	6.2	5.1	3.7	2.6
SWELL (SPECIF. 0.76 mm MAX)	—	0.0	—	—	—

GRADING AS USED WAS OBTAINED BY COMBINING SAMPLES AS FOLLOWS

% WT.	TEST NO.	DESCRIPTION
18	DT 036	¾ x ½
38	(COMBINED)	½ x #4
10		¼ x #8
34		#8 x DUST

REMARKS:

STATE OF CALIFORNIA • DEPARTMENT OF TRANSPORTATION
SAMPLE IDENTIFICATION CARD
TL-0101 (REV. 10/97)
CARD NUMBER C 683279

- [x] PRELIMINARY TESTS
- [ ] PROCESS TESTS
- [ ] ACCEPTANCE TESTS
- INDEPENDENT ASSURANCE TESTS
- [ ] SPECIAL TESTS

SAMPLE SENT TO: [x] DIST. LAB

DIST. LAB NO. 036

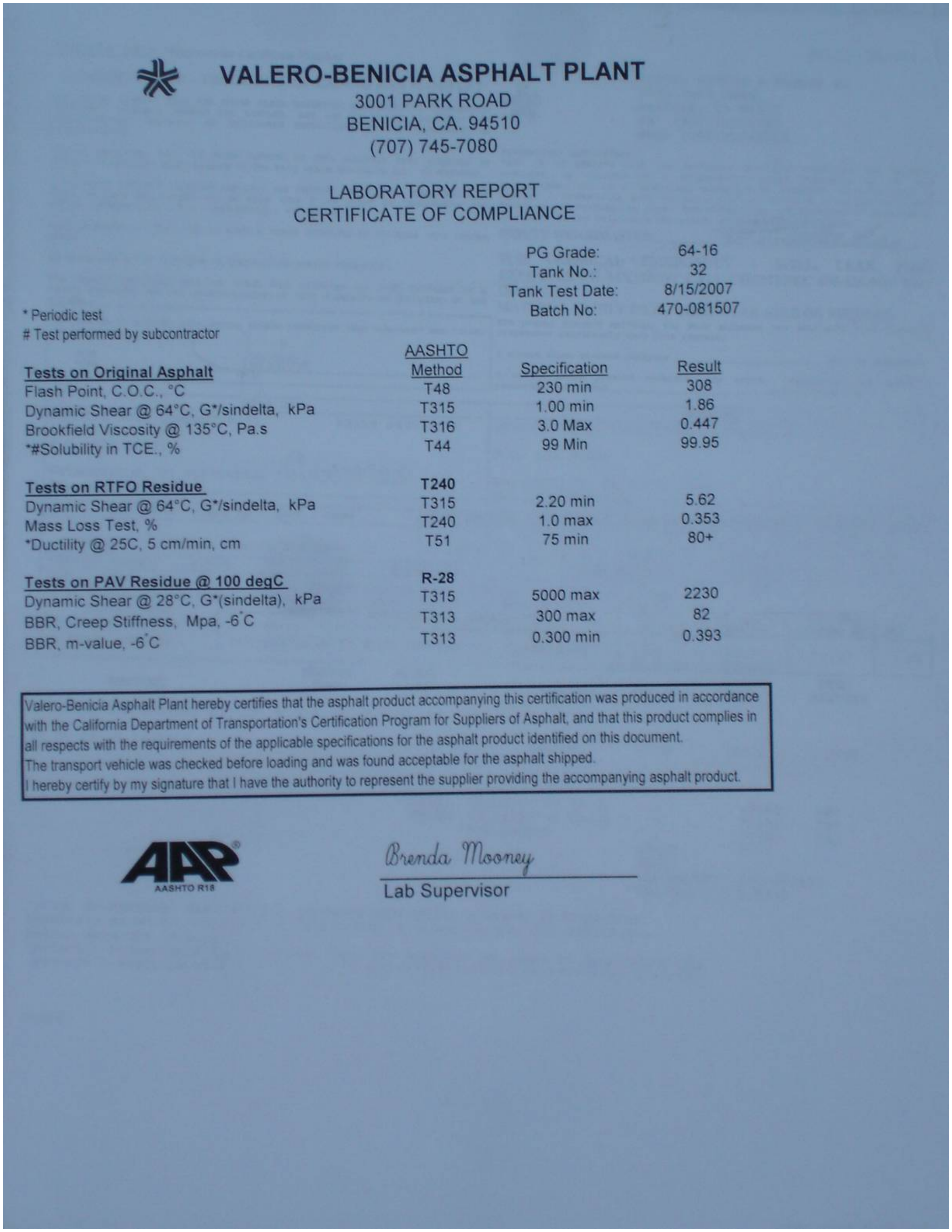
SAMPLE OF Aggregates & oil
FOR USE IN 19mm AC 19mm MAX MED TYPE A w/ PG64-10
SAMPLE FROM Plant
OWNER OR MANUFACTURER Granite Rock / Aromas Drum
TEST RESULTS DESIRED [x] NORMAL
REMARKS Verify Mix design. Recommend oil content %.
DATE SAMPLED 2/28/06 Wesley Rooney
BY RTS of Granite Rock TITLE Engineer
DIST. CO. RTE. PM 05-Mon-101-118.9/122.3
CONT. NO. 05-0L5504
FED. NO. ACSTPHL-Q101(106)E
RES. ENGR. OR SUPT. T. Lavering
ADDRESS fax 831 663-8707
CONTRACTOR Pavex

MAIL TO SAME DESTINATION AS SAMPLE

NO. 7720 P. 2 APR. 11. 2007 3:10PM CAL-TRANS

Figure A.2: Example Caltrans mix design (*continued*).

APPENDIX B: BINDER COMPLIANCE CERTIFICATE



VALERO-BENICIA ASPHALT PLANT
3001 PARK ROAD
BENICIA, CA. 94510
(707) 745-7080

LABORATORY REPORT
CERTIFICATE OF COMPLIANCE

PG Grade:	64-16
Tank No.:	32
Tank Test Date:	8/15/2007
Batch No:	470-081507

* Periodic test
# Test performed by subcontractor

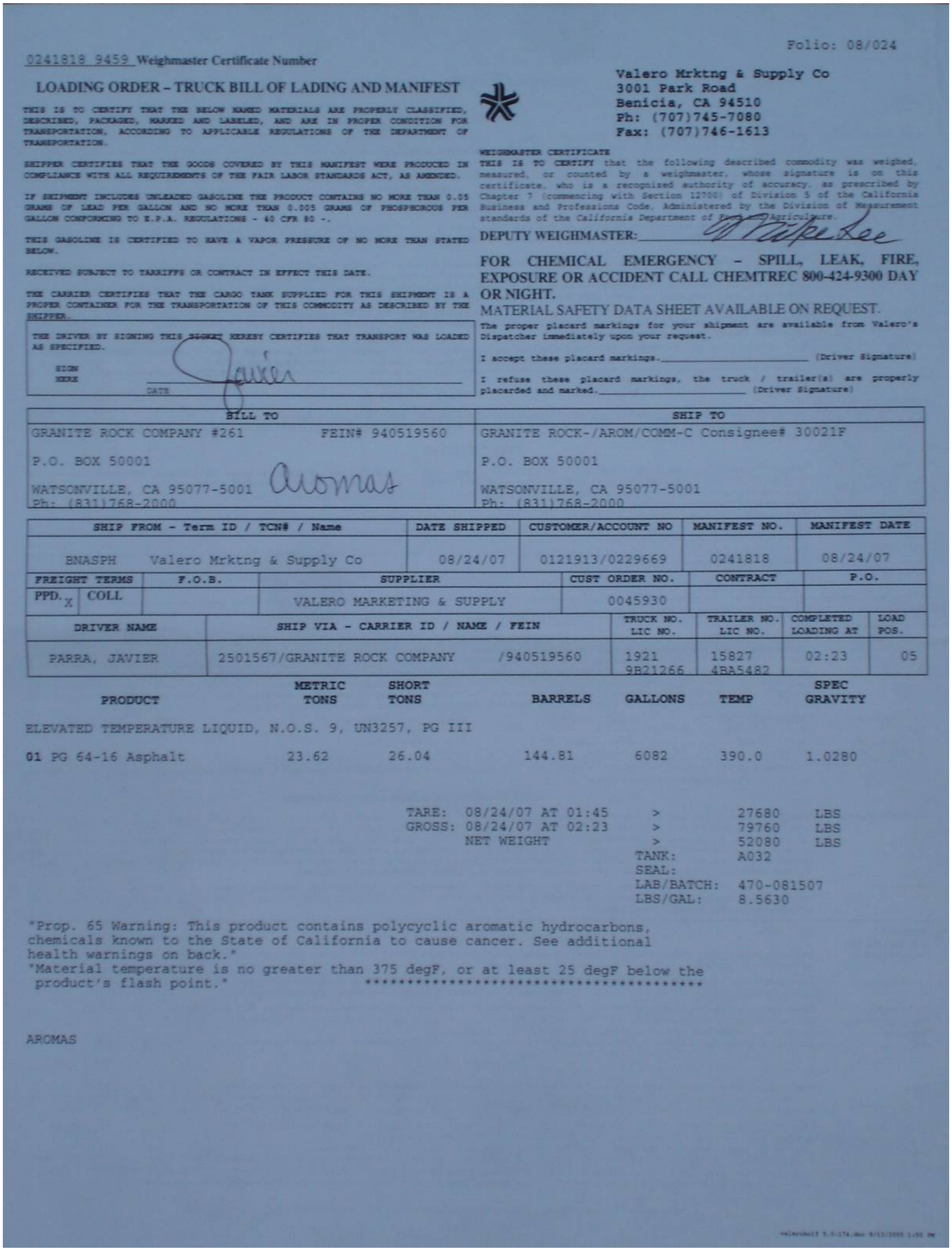
Tests on Original Asphalt	AASHTO Method	Specification	Result
Flash Point, C.O.C., °C	T48	230 min	308
Dynamic Shear @ 64°C, G*/sindelta, kPa	T315	1.00 min	1.86
Brookfield Viscosity @ 135°C, Pa.s	T316	3.0 Max	0.447
*#Solubility in TCE., %	T44	99 Min	99.95
Tests on RTFO Residue	**T240**		
Dynamic Shear @ 64°C, G*/sindelta, kPa	T315	2.20 min	5.62
Mass Loss Test, %	T240	1.0 max	0.353
*Ductility @ 25C, 5 cm/min, cm	T51	75 min	80+
Tests on PAV Residue @ 100 degC	**R-28**		
Dynamic Shear @ 28°C, G*(sindelta), kPa	T315	5000 max	2230
BBR, Creep Stiffness, Mpa, -6°C	T313	300 max	82
BBR, m-value, -6°C	T313	0.300 min	0.393

Valero-Benicia Asphalt Plant hereby certifies that the asphalt product accompanying this certification was produced in accordance with the California Department of Transportation's Certification Program for Suppliers of Asphalt, and that this product complies in all respects with the requirements of the applicable specifications for the asphalt product identified on this document.
The transport vehicle was checked before loading and was found acceptable for the asphalt shipped.
I hereby certify by my signature that I have the authority to represent the supplier providing the accompanying asphalt product.

AASHTO R18

Brenda Mooney
Lab Supervisor

Figure B.1: Binder compliance certificate.



0241818 9459 Weighmaster Certificate Number

Folio: 08/024

LOADING ORDER – TRUCK BILL OF LADING AND MANIFEST

Valero Mrktng & Supply Co
3001 Park Road
Benicia, CA 94510
Ph: (707)745-7080
Fax: (707)746-1613

THIS IS TO CERTIFY THAT THE BELOW NAMED MATERIALS ARE PROPERLY CLASSIFIED, DESCRIBED, PACKAGED, MARKED AND LABELED, AND ARE IN PROPER CONDITION FOR TRANSPORTATION, ACCORDING TO APPLICABLE REGULATIONS OF THE DEPARTMENT OF TRANSPORTATION.

SHIPPER CERTIFIES THAT THE GOODS COVERED BY THIS MANIFEST WERE PRODUCED IN COMPLIANCE WITH ALL REQUIREMENTS OF THE FAIR LABOR STANDARDS ACT, AS AMENDED.

IF SHIPMENT INCLUDES UNLEADED GASOLINE THE PRODUCT CONTAINS NO MORE THAN 0.05 GRAMS OF LEAD PER GALLON AND NO MORE THAN 0.005 GRAMS OF PHOSPHOROUS PER GALLON CONFORMING TO E.P.A. REGULATIONS - 40 CFR 80 -.

THIS GASOLINE IS CERTIFIED TO HAVE A VAPOR PRESSURE OF NO MORE THAN STATED BELOW.

RECEIVED SUBJECT TO TARRIFFS OR CONTRACT IN EFFECT THIS DATE.

THE CARRIER CERTIFIES THAT THE CARGO TANK SUPPLIED FOR THIS SHIPMENT IS A PROPER CONTAINER FOR THE TRANSPORTATION OF THIS COMMODITY AS DESCRIBED BY THE SHIPPER.

THE DRIVER BY SIGNING THIS ~~TICKET~~ HEREBY CERTIFIES THAT TRANSPORT WAS LOADED AS SPECIFIED.

SIGN HERE Javier

DATE

WEIGHMASTER CERTIFICATE
THIS IS TO CERTIFY that the following described commodity was weighed, measured, or counted by a weighmaster, whose signature is on this certificate, who is a recognized authority of accuracy, as prescribed by Chapter 7 (commencing with Section 12700) of Division 5 of the California Business and Professions Code, Administered by the Division of Measurement standards of the California Department of Food and Agriculture.

DEPUTY WEIGHMASTER: Mike Lee

FOR CHEMICAL EMERGENCY – SPILL, LEAK, FIRE, EXPOSURE OR ACCIDENT CALL CHEMTREC 800-424-9300 DAY OR NIGHT.

MATERIAL SAFETY DATA SHEET AVAILABLE ON REQUEST.

The proper placard markings for your shipment are available from Valero's Dispatcher immediately upon your request.

I accept these placard markings.__________ (Driver Signature)

I refuse these placard markings, the truck / trailer(s) are properly placarded and marked.__________ (Driver Signature)

BILL TO	SHIP TO
GRANITE ROCK COMPANY #261 FEIN# 940519560	GRANITE ROCK-/AROM/COMM-C Consignee# 30021F
P.O. BOX 50001	P.O. BOX 50001
WATSONVILLE, CA 95077-5001 Aromas	WATSONVILLE, CA 95077-5001
Ph: (831)768-2000	Ph: (831)768-2000

SHIP FROM - Term ID / TCN# / Name	DATE SHIPPED	CUSTOMER/ACCOUNT NO	MANIFEST NO.	MANIFEST DATE
BNASPH Valero Mrktng & Supply Co	08/24/07	0121913/0229669	0241818	08/24/07

FREIGHT TERMS	F.O.B.	SUPPLIER	CUST ORDER NO.	CONTRACT	P.O.
PPD. X / COLL		VALERO MARKETING & SUPPLY	0045930		

DRIVER NAME	SHIP VIA - CARRIER ID / NAME / FEIN	TRUCK NO. LIC NO.	TRAILER NO. LIC NO.	COMPLETED LOADING AT	LOAD POS.
PARRA, JAVIER	2501567/GRANITE ROCK COMPANY /940519560	1921 9B21266	15827 4BA5482	02:23	05

ELEVATED TEMPERATURE LIQUID, N.O.S. 9, UN3257, PG III

PRODUCT	METRIC TONS	SHORT TONS	BARRELS	GALLONS	TEMP	SPEC GRAVITY
01 PG 64-16 Asphalt	23.62	26.04	144.81	6082	390.0	1.0280

TARE: 08/24/07 AT 01:45 > 27680 LBS
GROSS: 08/24/07 AT 02:23 > 79760 LBS
NET WEIGHT > 52080 LBS
TANK: A032
SEAL:
LAB/BATCH: 470-081507
LBS/GAL: 8.5630

"Prop. 65 Warning: This product contains polycyclic aromatic hydrocarbons, chemicals known to the State of California to cause cancer. See additional health warnings on back."
"Material temperature is no greater than 375 degF, or at least 25 degF below the product's flash point." **

AROMAS

Figure B.1: Binder compliance certificate (*continued*).

APPENDIX C: FATIGUE BEAM SOAKING PROCEDURE

C.1 Preparation of Specimens

Specimens are prepared as follows:

1. The bulk specific gravity, width, and height of each beam shall first be measured and recorded.
2. Each beam is dried at room temperature (around 30°C) in a forced draft oven or in a concrete conditioning room to constant mass (defined as the mass at which further drying does not alter the mass by more than 0.05 percent at two-hour drying intervals). The final dry mass should be recorded. Note: Beams should be placed on a rigid and flat surface during drying.
3. A nut used for supporting the LVDT is bonded to the beam using epoxy resin. The mass of the beam with the nut should be recorded.

C.2 Conditioning of Specimens

1. Place the beam in the vacuum container supported above the container bottom by a spacer. Fill the container with water so that the beam is totally submerged in the water. Apply a vacuum of 635 mm (25 in.) of mercury for 30 minutes. Remove the vacuum and determine the saturated surface dry mass according to AASHTO T-166. Calculate the volume of absorbed water and determine the degree of saturation. If the saturation level is less than 70 percent, vacuum saturate the beam for a longer time and determine the saturated surface dry mass again.
2. Place the vacuum-saturated beam in a water bath with the water temperature pre-set at 60°C. The beam should be supported on a rigid, flat (steel or wood) plate to prevent deformation of the beam during conditioning. The top surface of the beam should be about 25 mm below the water surface.
3. After 24 hours, drain the water bath and refill it with cold tap water. Set the water bath temperature to 20°C. Wait for 2 hours for temperature equilibrium.
4. Remove the beam from the water bath, and determine its saturated surface dry mass.
5. Wrap the beam with Parafilm to ensure no water leakage.
6. Check the bonded nut. If it becomes loose, remove it and rebond it with epoxy resin.
7. Apply a layer of scotch tape to the areas where the beam contacts the clamps of the fatigue machine. This will prevent adhesion between the Parafilm and the clamps.
8. Start the fatigue test of the conditioned beam within 24 hours.