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Balanced Mix Design Rut Test Validation Test Track: Construction of HMA Sections

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Partnered Pavement Research Center (PPRC) Strategic Plan
Element Number 4.92 (TO-005): Continued Support for
Implementation of Performance Related Tests and Specifications
for Balanced Mix Design, Increased Recycling of Asphalt Mixes,
and Integration of Mix Design and Structural Design

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Office of Materials and Infrastructure

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16. ABSTRACT This technical memorandum describes the design and construction of a full-scale test track and the construction quality control/quality assurance testing of the different materials, including subgrade, aggregate base, and asphalt mixes. Construction of the test track and the Heavy Vehicle Simulator (HVS) and laboratory testing that followed are part of the Caltrans balanced mix design (BMD) implementation effort led by the Pavement and Materials Partnering Committee (PMPC) BMD Working Group. The main goal of the test track evaluation was to compare the rutting performance of surface mixes under the HVS with laboratory rutting test results. A secondary goal was to compare the reflective cracking performance of the same mixes with laboratory cracking test results. The test track includes four lanes, each 177 ft. long, and each with a variation of a control hot mix asphalt (HMA) surface mix, with two sections in each lane for evaluation of either rutting or reflective cracking performance. The rutting evaluation sections had an asphalt long-life pavement structure designed and built to test the rutting performance of the surface layer under the HVS. The reflective cracking evaluation sections were an asphalt overlay on a jointed plain concrete pavement. The test track asphalt surface can be evaluated and replaced in successive testing rounds. Following PMPC BMD Working Group directions, the first round of testing focused on HMA Type A with plain binder, used in the construction documented in this document. The control mix, used for paving Lane 1 of the test track, was an HMA with 3/4 in. maximum aggregate size, 25% reclaimed asphalt pavement content (percent by aggregate replacement), and virgin binder graded as PG 58-22. Three alternative mixes were used for paving Lane 2, Lane 3, and Lane 4. The alternative mixes were intentionally designed to make the mixes susceptible to rutting. The construction of the subgrade, aggregate base, concrete slabs, and asphalt mixes went as expected, with no incidents to report. Overall, the asphalt mixes used for the construction matched the design expectations, and the field compaction was as expected.		
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PROJECT OBJECTIVES

This technical memorandum is part of Partnered Pavement Research Center (PPRC) Project 4.92 (TO-005), “Continued Support for Implementation of Performance Related Tests and Specifications for Balanced Mix Design, Increased Recycling of Asphalt Mixes, and Integration of Mix Design and Structural Design.” One of the goals of PPRC Project 4.92 is to identify and develop a comprehensive set of performance-related tests that will be used in performance-related specification frameworks while also developing new mix types with lower life cycle costs and environmental impacts. To complete this objective, Project 4.92 includes Task 1 and its subtasks:

- Task 1: Evaluation of alternative routine performance-related tests for balanced mix design (BMD) using accelerated pavement testing:
 - Subtask 1A: Construction of a test track for new materials and new test method validation. This work includes demolition of a current test track at the Davis site of the University of California Pavement Research Center (UCPRC) and construction of a new asphalt pavement as an underlying structure for evaluation of the rutting performance of new mix types and validation of rutting performance-related tests for use in BMD.
 - Subtask 1B: First round of asphalt overlay test sections, accelerated pavement testing for rutting, and laboratory testing with alternative tests. The work in this subtask includes constructing the first round of overlays; performing accelerated pavement testing, laboratory testing, first-level analysis, and structural simulation analysis; and producing a report with the results.
 - Subtask 1C: Second round of asphalt overlay test sections, accelerated pavement testing for rutting, and laboratory testing with alternative tests.
 - Subtask 1D: Pavement testing for age-related and reflective cracking.
 - Subtask 1E: BMD recommendations.

This technical memorandum describes the completion of Subtask 1A and the construction portion of Subtask 1B. The remainder of Subtask 1B, including the accelerated pavement testing with the HVS, will be documented in a subsequent report.

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This document is disseminated in the interest of information exchange. 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 publication does not constitute a standard, specification, or regulation. This report does not constitute an endorsement by the Department of any product described herein.

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LIST OF ABBREVIATIONS

4PB	Four-point bending
AB	Aggregate base
AC	Asphalt concrete
APT	Accelerated pavement testing
AV	Air-voids
BMD	Balanced mix design
CL	Classification of low-plasticity
DAS	Data acquisition system
DCP	Dynamic cone penetrometer
EA	Expenditure authorization
ESAL	Equivalent single axle loads
FWD	Falling weight deflectometer
GPR	Ground-penetrating radar
GPS	Global positioning system
HMA	Hot mix asphalt
HVS	Heavy Vehicle Simulator
HWT	Hamburg Wheel-Tracking
IDEAL-CT	IDEAL cracking tolerance
JMF	Job mix formula
JPCP	Jointed plain concrete pavement
LVDT	Linear voltage deflection transducers
M&R	Maintenance and rehabilitation
MAS	Maximum aggregate size
MDD	Multi-depth deflectometer
ME	Mechanistic-empirical
METS	Materials engineering and testing services
NMAS	Nominal maximum aggregate size
PCC	Portland cement concrete
PMPC	Pavement Materials Partnering Committee

PMS	Pavement management system
PRS	Performance-related specifications
PRT	Performance-related tests
QA	Quality assurance
QC	Quality control
RA	Recycling agent
RAP	Reclaimed asphalt pavement
RAS	Recycled asphalt shingles
RH	Relative humidity
RHMA-G	Rubberized hot mix asphalt-gap-graded
SDF	Standard deviation factor
SG	Subgrade
TMM	Total mass of mix
VWSG	Vibrating wire strain gauge

LIST OF TEST METHODS AND SPECIFICATIONS

AASHTO

M320-23	Standard Specification for Performance-Graded Asphalt Binder
T180-22	Standard Method of Test for Moisture–Density Relations of Soils Using a 4.54-kg (10-lb) Rammer and a 457-mm (18-in.) Drop
T255-22	Standard Method of Test for Total Evaporable Moisture Content of Aggregate by Drying
T265-22	Standard Method of Test for Laboratory Determination of Moisture Content of Soils
T318-15 (2023)	Standard Method of Test for Water Content of Freshly Mixed Concrete Using Microwave Oven Drying

ASTM

C1064-23	Standard Test Method for Temperature of Freshly Mixed Hydraulic-Cement Concrete
C138-24	Standard Test Method for Density (Unit Weight), Yield, and Air Content (Gravimetric) of Concrete
C143-20	Standard Test Method for Slump of Hydraulic-Cement Concrete
C150-24	Standard Specification for Portland Cement
C157-24	Standard Test Method for Length Change of Hardened Hydraulic-Cement Mortar and Concrete
C173-24	Standard Test Method for Air Content of Freshly Mixed Concrete by the Volumetric Method
C309-19	Standard Specification for Liquid Membrane-Forming Compounds for Curing Concrete
C39-24	Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens
C469-22	Standard Test Method for Static Modulus of Elasticity and Poisson's Ratio of Concrete in Compression
C494-24	Standard Specification for Chemical Admixtures for Concrete
C78-22	Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Third-Point Loading)
C989-24	Standard Specification for Slag Cement for Use in Concrete and Mortars
D5404-24	Standard Practice for Recovery of Asphalt Binder from Solution Using the Rotary Evaporator
D8159-19	Standard Test Method for Automated Extraction of Asphalt Binder from Asphalt Mixtures
D8237-21	Standard Test Method for Determining Fatigue Failure of Asphalt-Aggregate Mixtures with the Four-Point Beam Fatigue Device
CTM 320-25	Provisional Method of Test for Laboratory Conditioning of Dense-Graded Asphalt Mixtures

CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
in.	inches	25.40	millimeters	mm
ft.	feet	0.3048	meters	m
yd.	yards	0.9144	meters	m
mi.	miles	1.609	kilometers	km
AREA				
in ²	square inches	645.2	square millimeters	mm ²
ft ²	square feet	0.09290	square meters	m ²
yd ²	square yards	0.8361	square meters	m ²
ac.	acres	0.4047	hectares	ha
mi ²	square miles	2.590	square kilometers	km ²
VOLUME				
fl. oz.	fluid ounces	29.57	milliliters	mL
gal.	gallons	3.785	liters	L
ft ³	cubic feet	0.02832	cubic meters	m ³
yd ³	cubic yards	0.7646	cubic meters	m ³
MASS				
oz.	ounces	28.35	grams	g
lb.	pounds	0.4536	kilograms	kg
T	short tons (2000 pounds)	0.9072	metric tons	t
TEMPERATURE (exact degrees)				
°F	Fahrenheit	(F-32)/1.8	Celsius	°C
FORCE and PRESSURE or STRESS				
lbf	pound-force	4.448	newtons	N
lbf/in ²	pound-force per square inch	6.895	kilopascals	kPa
APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol
LENGTH				
mm	millimeters	0.03937	inches	in.
m	meters	3.281	feet	ft.
m	meters	1.094	yards	yd.
km	kilometers	0.6214	miles	mi.
AREA				
mm ²	square millimeters	0.001550	square inches	in ²
m ²	square meters	10.76	square feet	ft ²
m ²	square meters	1.196	square yards	yd ²
ha	hectares	2.471	acres	ac.
km ²	square kilometers	0.3861	square miles	mi ²
VOLUME				
mL	milliliters	0.03381	fluid ounces	fl. oz.
L	liters	0.2642	gallons	gal.
m ³	cubic meters	35.31	cubic feet	ft ³
m ³	cubic meters	1.308	cubic yards	yd ³
MASS				
g	grams	0.03527	ounces	oz.
kg	kilograms	2.205	pounds	lb.
t	metric tons	1.102	short tons (2000 pounds)	T
TEMPERATURE (exact degrees)				
°C	Celsius	1.8C + 32	Fahrenheit	°F
FORCE and PRESSURE or STRESS				
N	newtons	0.2248	pound-force	lbf
kPa	kilopascals	0.1450	pound-force per square inch	lbf/in ²

*SI is the abbreviation for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.
(Revised April 2021)

1 INTRODUCTION

1.1 Background

Caltrans has a goal of implementing balanced mix design (BMD) across California's pavement infrastructure. BMD relies on performance-related tests (PRTs) and performance-related specifications (PRs) to help ensure that the mechanistic properties of asphalt mixes used by Caltrans perform as expected and that the properties of the asphalt mixes delivered by contractors sufficiently match those used as input in mechanistic-empirical (ME) pavement structural design. At the same time, Caltrans is investigating the use of a wider range of asphalt mixes intended to reduce life cycle costs and environmental impacts of those mixes. The new types of mixes include those with high reclaimed asphalt pavement (RAP) content (25% to 100%), mixes containing recycled asphalt shingles (RAS), rubberized hot mix asphalt-gap-graded (RHMA-G) with RAP, dense-graded mixes with recycled tire rubber, and potentially mixes containing recycled plastics.

The primary drivers of asphalt-surfaced (also called "flexible-surfaced") pavement maintenance and rehabilitation (M&R) are (1) cracking, including age-related cracking that affects all asphalt surfaces and fatigue cracking in the wheelpaths that occurs only under heavy truck traffic, and (2) rutting in the wheelpaths, which also occurs only under heavy truck traffic.

Historic asphalt pavement performance data in the Caltrans pavement management system (PMS) show that the risk of asphalt rutting is very rare on the state highway network, while fatigue cracking and age-related cracking are much more prevalent. This finding indicates that the current asphalt mix design procedure can be improved to maintain a very low risk of rutting while also improving the fatigue and age-related cracking performance. Fatigue cracking occurs through an interaction of the thicknesses and properties of all the asphalt layers acting together, and when it occurs, it leads to increased roughness and eventually damage throughout the pavement structure, requiring expensive rehabilitation. Age-related cracking occurs in the surface layer. It is important because it affects smoothness as it advances, and if left untreated, it can propagate downward, also leading to major structural damage.

Rutting and cracking (both fatigue cracking and age-related cracking) generally require opposing ranges of stiffness and asphalt binder contents, and achieving the right balance between these properties is the goal of BMD. Rutting is a distress that primarily occurs near the pavement surface; it happens early in the life of a newly placed asphalt mix and requires removal and replacement if severe. Fatigue and age-related cracking occur later in the life of an asphalt surface, making their occurrence less urgent in the short-term; however, they are the primary drivers of M&R costs over the life cycle. Increasing the cracking durability of asphalt mixes will provide important cost savings, depending on how much the frequency of resurfacing can be reduced by improving durability with respect to cracking.

The current Caltrans asphalt mix design procedure was implemented in 2015 and uses the Superpave volumetric mix design methods developed in the late 1990s to better balance cracking and rutting. The 2015 implementation of Superpave by Caltrans replaced the previous Caltrans mix design method developed by Francis Hveem and colleagues at Caltrans in the 1950s and 1960s. Hveem included explicit tests for rutting and cracking in his method, but the cracking test was dropped over time because of practicality issues and because Caltrans mix designs prior to 2015 produced mixes with almost no risk of rutting but with very poor cracking performance.

The proportioning of asphalt binder in the Superpave method is referred to as “volumetric proportioning,” which looks at how much the mix compacts under a predefined laboratory compaction effort. The goal is to add the right amount of binder, as too much binder may result in mixes that are prone to rutting, while not enough binder may result in mixes that are prone to cracking. “Too much” and “not enough” have come generally from national experience but without rigorous comparison with actual field performance and with no direct consideration of the ranges of stiffness and other properties controlling rutting, fatigue cracking, and age-related cracking.

The Superpave approach implemented by Caltrans uses 85 gyrations to set the proportioning, which is conservative on the side of reducing rutting risk. Many other states, including Texas and Florida, are using fewer gyrations,¹ which results in more binder in the mix and greater cracking durability with increased rutting risk. However, those states are not experiencing significant amounts of rutting. The State Bituminous Engineer in charge of implementing Superpave at Caltrans in 2015 stated that his goal was to start conservatively regarding rutting, recognizing that 85 gyrations would result in increased binder content compared with the Hveem method but that further improvement in cracking durability could be made in the future.² He also stated that he was looking in the future for better PRTs for rutting and cracking to be able to safely increase cracking durability while not resulting in significant increases in the risk of rutting.

Recent advances in asphalt technology have made asphalt mix design more complicated, but also offer greater opportunities for engineering mixes to meet rutting and cracking performance requirements. These technologies include the use of high percentages of RAP, the use of RAS, greater availability of polymers that can be used in virgin asphalt binders, chemical recycling agents (RAs) that can help with blending of RAP and RAS with virgin asphalt binder, and new ways of using recycled tire rubber.

The goal of this research, part of Caltrans Partnered Pavement Research Center (PPRC) Project 4.92, is to identify and develop a comprehensive set of PRTs that will be used in PRS frameworks for routine projects. A PRT and PRS framework was previously developed and used on five large, expensive major rehabilitation and reconstruction projects (referred to as “PRS projects” in the recently updated Caltrans *Highway Design Manual*). However, those PRTs are too costly and difficult and take too much time to conduct for more routine projects.

Work is advancing on the age-related cracking test, with the current most likely candidate for inclusion being the IDEAL Cracking Tolerance (IDEAL-CT) test. The IDEAL-CT test provides both measures of stiffness at intermediate temperatures and fracture resistance and can be used with a medium-term oven aging procedure to also identify aging properties that are important for age-related cracking. The IDEAL-CT test is being evaluated primarily through high RAP and RAS pilot projects in California and in other states.

The four-point bending (4PB) beam fatigue test was previously validated through accelerated pavement testing (APT), using the Heavy Vehicle Simulator (HVS) and on the Federal Highway Administration (FHWA) closed circuit test track WestTrack experiment, and in practice on the five large asphalt rehabilitation projects mentioned previously as well as globally over the last 60 years. In practical terms, fatigue is the gradual cracking of pavement under repeated traffic loading; a simple

¹ From recent June 2025 discussions with Texas and Florida Department of Transportation engineers. A broader literature review is currently underway.

² Statements from Joseph Peterson from personal conversations with John Harvey in 2015.

analogy is a rubber band that weakens and eventually snaps after being stretched many times or left outside to age and degrade. However, the 4PB test in its current form does not meet the cost, difficulty, and time limitations for routine design, and it is undergoing further development to see if it can be included later in BMD.

The primary concern at this time, identified in the Caltrans Pavement Materials Partnering Committee (PMPC) BMD work plan, is to identify a test for rutting that meets the requirements for BMD. Once the test is selected that best captures differences in mix rutting performance, the specification limits for that test need to be set to provide for the maximum cracking durability in the mix design with an acceptably small probability of rutting. APT using the HVS was identified as a means to collect rutting data to help fill the gap caused by a lack of PRT for the very few cases where asphalt mixes have rutted on the Caltrans network.

The results of PPRC Project 4.92 are expected to reduce Caltrans life cycle costs and life cycle environmental impacts for maintaining the asphalt-surfaced pavements in the state highway network, reduce road user construction work zone delay costs, and help meet other environmental goals, including the use of recycled and waste materials.

1.2 Purpose of the Study and Test Track

The objective of this test track design, construction, HVS evaluation, and laboratory testing of the asphalt mixes used for its construction is to help identify a BMD test for rutting and the corresponding specification limits, while also measuring the age-related and fatigue cracking durability. To complete this objective, Project 4.92 includes Task 1 and its subtasks:

- Task 1: Evaluation of alternative routine PRTs for BMD using APT:
 - Subtask 1A: Construction of a test track for new materials and new test method validation. This work includes demolition of a current test track at the Davis site of the University of California Pavement Research Center (UCPRC) and construction of a new asphalt pavement as an underlying structure for evaluation of the rutting performance of new mix types and validation of rutting PRTs for use in BMD.
 - Subtask 1B: First round of asphalt overlay test sections, APT for rutting, and laboratory testing with alternative tests. The work in this subtask includes constructing the first round of overlays; performing APT, laboratory testing, first-level analysis, and structural simulation analysis; and producing a report with the results.
 - Subtask 1C: Second round of asphalt overlay test sections, APT for rutting, and laboratory testing with alternative tests.
 - Subtask 1D: Pavement testing for age-related and reflective cracking.
 - Subtask 1E: BMD recommendations.

This technical memorandum describes the completion of Subtask 1A and the construction portion of Subtask 1B. The remainder of Subtask 1B, including the APT with the HVS, will be documented in a subsequent report.

1.3 Scope of the Technical Memorandum

This technical memorandum includes the following chapters:

- Chapter 2 details the test section layout and structural design of the concrete and asphalt cross sections.
- Chapter 3 documents demolition of the previous track, construction of the underlying subgrade and aggregate base layers, instrumentation, and construction of the concrete underlying section.
- Chapter 4 presents the results of the asphalt job mix formula (JMF) development for the underlying asphalt layers and the surface layers.
- Chapter 5 documents construction, sampling, and quality control/quality assurance (QC/QA) testing for the underlying asphalt layers and surface layers.
- Chapter 6 presents post-construction deflection testing results.
- Chapter 7 presents a summary, preliminary conclusions, and recommendations from the test track construction.

1.4 Measurement Units

Although Caltrans has returned to the use of US standard measurement units, metric units have always been used by the UCPRC in the design and layout of test tracks, and for laboratory, HVS, and field measurements and data storage. In this report, US units are listed first, followed by metric units in parentheses. A conversion table is also provided on page xv.

2 TEST PLAN, TEST TRACK LAYOUT, STRUCTURAL DESIGNS

2.1 Test Plan

The main goal of the BMD test track constructed at the UCPRC Davis site is to evaluate the relationship between laboratory rutting performance tests and the rutting measured under full-scale APT conducted with the HVS. A secondary goal is to evaluate the relationship between laboratory cracking performance tests and the reflective cracking due to ambient environmental actions. To support these goals, the test track includes two types of pavement structures (Figure 2.1):

1. Rutting evaluation sections (four sections): These sections have what has been termed an AC Long Life pavement structure, referenced in the Caltrans *Highway Design Manual*, Table 6.33.1A, Note 2 (May 2022 edition). The surface layer material differs across sections; in each lane, the surface layer material consists of the asphalt concrete (AC) mix whose rutting performance is being evaluated. The pavement structure below the surface layer is common to all sections and consists of a stiff intermediate layer with a “rich bottom” layer below, as well as a Class 2 aggregate base. These four sections will be trafficked with the HVS at a high temperature to evaluate the rutting performance.
2. Reflective cracking sections (four sections): These sections consist of an asphalt overlay on top of an undoweled jointed plain concrete pavement (JPCP) with an aggregate base below the concrete slabs. The surface layer material differs across sections; in each lane, the surface layer material consists of the AC mix whose reflective cracking performance is being evaluated. These four sections are located adjacent to the rutting sections within each lane so that they share the top lift of asphalt paving. The JPCP below the surface layer is common to all sections. These sections most likely will not be trafficked with the HVS, but will be monitored for reflective cracking from movement of the JPCP transverse joints caused by ambient temperature and humidity changes.

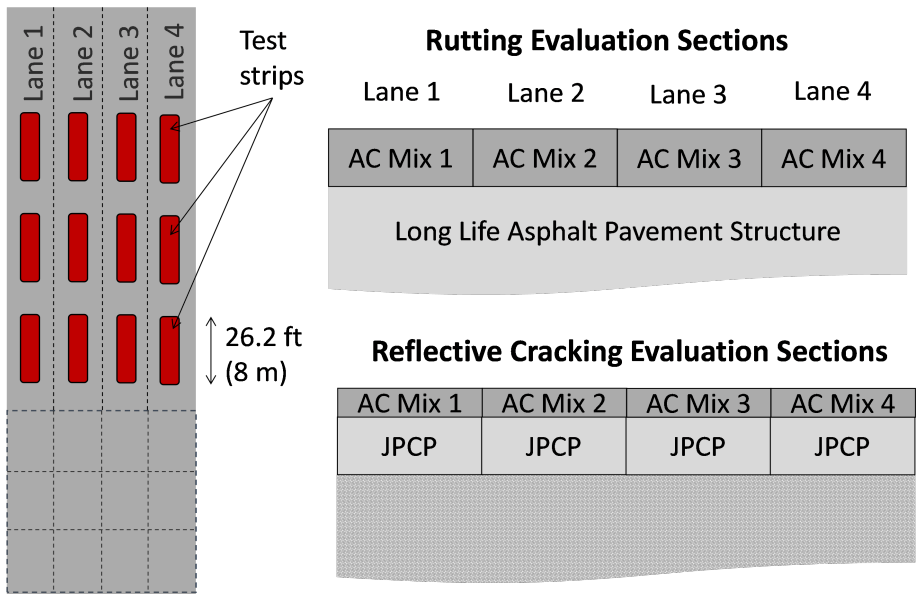


Figure 2.1: Test track layout and cross sections.

The rutting evaluation part of each lane can accommodate up to three test strips, each 26.2 ft. (8 m) long, for the HVS testing. The 26.2 ft. is the HVS testing length (i.e., the length that the HVS wheel covers in its back-and-forth movement). Because of the long-life structure of the rutting sections, the AC surface layer can be milled off after the HVS rutting test and replaced with a new surface, with minimal damage to the underlying structure. In other words, the test track can be reused for future rutting tests.

The PMPC BMD Working Group decided to evaluate hot mix asphalt (HMA) made with plain binder in the first round of HVS testing. Consequently, the surface layer of the first test track, whose construction is detailed in this technical memorandum, is made of HMA with plain binder. A second round of asphalt mixes for test track evaluation will likely include HMA made with a polymer-modified binder surface layer.

The PMPC BMD Working Group also decided to evaluate HMA with 3/4 in. maximum aggregate size (MAS) in the first round of testing. This aggregate size aligns with the most likely Caltrans use of HMA with plain binder, which is as an intermediate layer. The following thicknesses of the asphalt surface layer are the same across the four lanes:

- Rutting evaluation sections: 0.4 ft., placed in two lifts
- Reflective cracking evaluation sections: 0.2 ft., placed in one lift

The following approach was adopted to select the four variations of the surface mix for evaluation:

1. Select a control mix whose mix design is approved for Caltrans use.
2. Introduce variations in the mix design to make the mix prone to rutting. The variations should represent issues that may occur during production. As further described in Section 4.3, the variations that originated in the three alternative surface mixes used in the first test track construction are the following:
 - The binder content is considerably higher than the design binder content. The resulting mix is referred to as “High Pb” in this technical memorandum.
 - An overdose of recycling agent was added to the mix. The resulting mix is referred to as “High RA” in this technical memorandum.
 - The content of natural sand is considerably higher than the design content. The resulting mix is referred to as “Nat. Sand” in this technical memorandum.

2.2 Test Sections Overview

The test track is located at the UCPRC site in Davis (Figure 2.2). The test site is typically referred to as the “West Track,” as it is located to the west of the UCPRC laboratory. The track is around 180 ft. long and has four 12 ft. wide lanes (Table 2.1). Of the total length, around 135 ft. correspond to the rutting evaluation sections, while 42 ft. correspond to the reflective cracking evaluation sections. The rutting evaluation sections have a long-life asphalt pavement structure, while the reflective cracking evaluation sections are an asphalt overlay on top of a JPCP. The JPCP includes three consecutive slabs per lane. The JPCP slabs are 14 ft. long and 12 ft. wide; the transverse joints are undoweled, and the longitudinal joints do not have tie bars. The test track is in a flat area continuously exposed to direct sunlight. The subgrade is native soil that has a Unified Soil Classification of low-plasticity clay (CL).

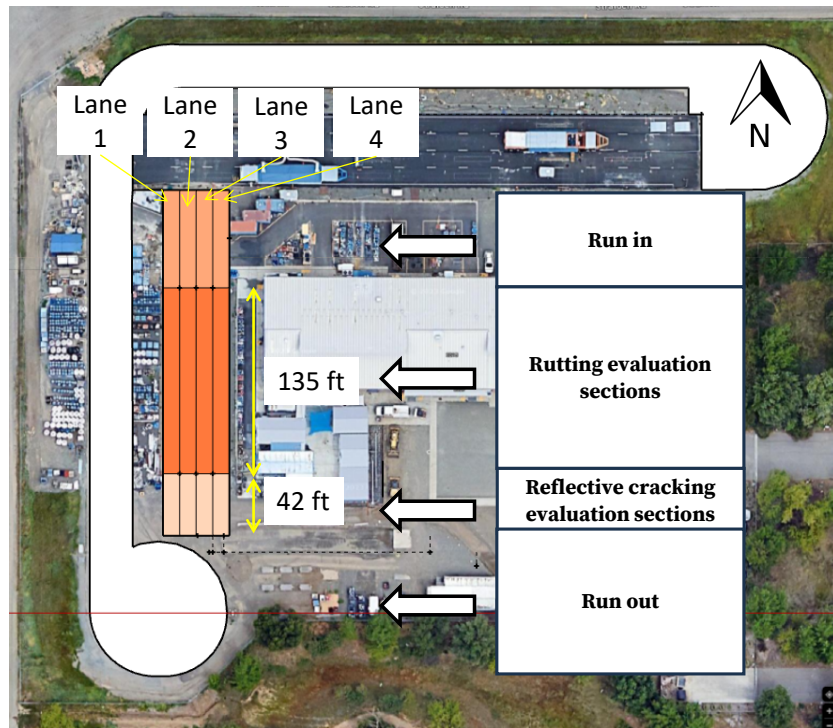


Figure 2.2: Test track layout.

Table 2.1: Test Track Sections

Description		Run-In	Rutting Section	Reflective Cracking Section
Start (ft.)		0+00	0+71	2+06
End (ft.)		0+71	2+06	2+50
4 lanes ← 48 ft. →	Lane 1 ← 12 ft. →	Surface control AC		
	Lane 2 ← 12 ft. →	Surface alternative AC 1 (High Pb mix in the first round of testing)		
	Lane 3 ← 12 ft. →	Surface alternative AC 2 (High RA mix in the first round of testing)		
	Lane 4 ← 12 ft. →	Surface alternative AC 3 (Nat. Sand mix in the first round of testing)		

2.3 Asphalt Pavement Structural Design

The structural design of the rutting evaluation sections of the BMD test track focused on the following:

- Minimizing fatigue cracking and rutting in the intermediate and rich bottom asphalt layers, aggregate base, and subgrade to isolate surface rutting behavior.
- Allowing surface replacement over multiple test rounds without disturbing the underlying structure (long-life asphalt pavement).
- Ensuring sufficient structural capacity for multiple passes of the HVS under surface rut testing conditions.

The design was conducted using *CalME*. The *CalME* representation of the initially proposed structure is shown in Table 2.2 (as explained in the following discussion, the initially proposed structure was not the one finally selected for construction).

Table 2.2: CalME Representation of the Proposed Structures

Layer Number	Material	Material in CalME	Thickness in CalME
1	0.35 ft. of surface mix	2020 standard HMA Type A mix with PG 64-16 binder and up to 15% RAP for non-PRS projects	0.35 ft.
2	0.50 ft. of intermediate mix	2020 standard HMA Type A mix with 25% RAP for non-PRS projects	0.50 ft.
3	0.15 ft. of intermediate mix	2020 standard HMA Type A mix with 25% RAP for non-PRS projects	0.15 ft.
4	0.5 ft. or 1.0 ft. of aggregate base Class 2	2020 standard aggregate base Class 2	0.50 or 1.0 ft.
5	Native silty clay subgrade	2020 Standard CL	N/A

Notes: HMA: hot mix asphalt; RAP: recycled asphalt pavement; PRS: performance-related specifications; CL: United Soil Classification lean clay.

There are two alternatives for the proposed structure: (a) a structure with 0.50 ft. aggregate base (AB) or (b) a structure with 1.0 ft. AB. Both alternatives were simulated in *CalME*, with a 9,000 lb. (40 kN) wheel load and a target temperature at a depth of 2 in. set to 122°F (50°C). The expected performance of these two alternatives is summarized in Table 2.3. In this table, the traffic is quantified in 18-kip equivalent single axle loads (ESAL), which are the same as 9-kip wheel loads.

Table 2.3. Performance Summary of the Proposed Testing Section Alternatives Estimated by CalME

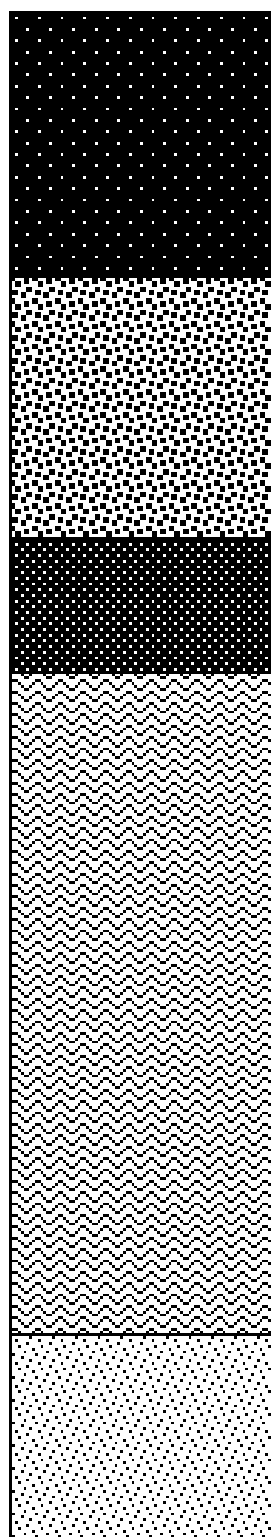
Performance	Reliability Level (%)	Alternative A: 0.50 ft. AB	Alternative B: 1.0 ft. AB
Rutting Life (0.5 in. threshold) (ESALs)	95	120,000	390,000
Rutting Life (0.5 in. threshold) (ESALs)	50	270,999	880,000
Cracking Life (5% cracking threshold) (ESALs)	95	>1,200,000	>1,200,000
Average Surface Rut after 1.2 million ESALs (in.)	50	0.67	0.55
Average Total Unbound Layers Rut after 1.2 million ESALs (in.)	50	0.50	0.38
Average Total HMA Rut after 1.2 million ESALs (in.)	50	0.17	0.17

Notes: AB: aggregate base; ESAL: equivalent single axel load; HMA: hot mix asphalt. One ESAL = one pass of 9,000 lb. (40 kN) HVS wheel load used for rut testing.

As shown in Table 2.3, both alternatives will have rutting failure before cracking failure. Increasing the AB thickness will reduce total rutting in the structure (referred to as Average Surface Rut in *CalME*), including rutting in the unbound layers (AB and subgrade) but not in the combined HMA layers (referred to as Average Total HMA Rut in *CalME*). It is preferable to use the thicker AB to reduce the amount of rutting in the unbound AB and subgrade layers from 0.5 in. to 0.38 in.

Also shown in Table 2.3, most of the rutting is attributed to the unbound layer for the proposed structures. This outcome is due to the high testing temperature, which significantly increases the strains and, therefore, rutting in the unbound layers because the HMA protecting those layers is softened by the temperature. Unbound layer rutting tends to be higher under initial loading and then develops at a slower rate per wheel pass. If the surface mix is the only layer that will be replaced in the subsequent construction rounds, it is possible that the unbound layers will undergo minimal further permanent deformation, which will remove the need for total reconstruction of the test track.

The design adopted for the rutting evaluation sections is shown in Figure 2.3. The structure includes a prepared subgrade (rip and recompact) and a 1.0 ft. AB under an asphalt long-life pavement structure with 0.2 ft. of rich bottom layer, 0.4 ft. of intermediate layer, and 0.4 ft. of surface layer made of HMA with plain binder.



Layer: Surface Mixes (Conventional Type-A)

Thickness: 0.40 ft. (122 mm)

Layer: Intermediate Mix (AC LLP, INT)

Thickness: 0.40 ft (122 mm)

Layer: Rich Bottom Mix (AC LLP, RB)

Thickness: 0.2 ft (61 mm)

**Layer: Imported Class 2 Aggregate Base
Virgin, Alluvial**

Thickness: 1.0 ft. (305 mm)

**Layer: Prepared Subgrade
(Native Silty Clay Subgrade [CI])**

Thickness: Semi-infinite

Figure 2.3: Cross-sectional view of the rutting evaluation sections.

The *CalME* representation of the final design is shown in Table 2.4 and the expected performance of the adopted structure is summarized in Table 2.5. The tables show that the adopted design has similar but slightly better performance than the initially proposed alternative with the same thickness of AB.

Table 2.4: CalME Representation of the Adopted Structure

Layer Number	Material	Material in CalME	Thickness in CalME
1	0.40 ft. of surface mix	2020 Standard HMA Type A Mix with PG 64-16 Binder and up to 15% RAP for non-PRS Projects	0.40 ft.
2	0.40 ft. of intermediate mix	2020 Standard HMA Type A Mix with 25% RAP for non-PRS Projects	0.40 ft.
3	0.20 ft. of rich bottom mix	2020 Standard Rich Bottom Mix for non-PRS Projects	0.20 ft.
4	1.0 ft. of AB Class 2	2020 Standard AB Class 2	1.0 ft.
5	Native silty clay subgrade	2020 Standard CL	N/A

Notes: HMA: hot mix asphalt; RAP: recycled asphalt pavement; PRS: performance-related specifications; AB: aggregate base; CL: United Soil Classification lean clay.

Table 2.5. Performance Summary of the Adopted Testing Section Estimated by CalME

Performance Measure	Reliability Level (%)	Performance
Rutting Life (0.5 in. threshold) (ESALs)	95	549,000
Rutting Life (0.5 in. threshold) (ESALs)	50	>1,200,000
Cracking Life (5% cracking threshold) (ESALs)	95	>1,200,000
Average Surface Rut after 1.2 million ESALs (in.)	50	0.50
Average Unbound Layers Rut after 1.2 million ESALs (in.)	50	0.38
Average HMA Rut after 1.2 million ESALs (in.)	50	0.17

Notes: ESAL: equivalent single axle load. One ESAL = one pass of 9,000 lb. (40 kN) HVS wheel load used for rut testing.

2.4 Concrete Pavement Structural Design

The reflective cracking evaluation sections consist of an asphalt overlay on top of a JPCP structure. The asphalt overlay is expected to develop reflective cracking from the JPCP transverse joints because of the repeated opening and closing that these joints experience as the concrete of the slabs expands and contracts due to diurnal and seasonal changes in temperature and drying shrinkage. Since these sections are not expected to carry HVS loading, the structural design does not focus on the traffic loading but on maximizing the potential for inducing reflective cracking on the asphalt overlay.

Maximizing the potential for reflective cracking on the asphalt overlay is desirable because the time to develop reflective cracking (t_{RF}) is a measure of the asphalt mix's capacity to resist reflective and other

types of fatigue cracking. The comparison of t_{RF} versus the results of different cracking PRT would serve to evaluate the PRT as a candidate test for BMD.

To induce high opening and closing of the transverse joints, a relatively thin slab, 0.58 ft. (7 in.), and a portland cement concrete (PCC) with expected high drying shrinkage were selected. The PCC mix design, provided by the CEMEX plant in West Sacramento, is shown in Table 2.6. The PCC meets Caltrans Standard Specifications, Section 40, “Concrete Pavement,” and Section 90, “Concrete.” The PCC has no air entrainment, and its design water-to-cementitious (w/c) materials ratio is 0.42.

Table 2.6: Portland Cement Concrete Mix Design

Material	Description	Design quantity (1 yd ³)
Coarse aggregate	Gravel	1750 lb.
Fine aggregate	Concrete sand	1323 lb.
Cement	Type II/V ordinary portland cement (OPC), ASTM C150-24	472 lb.
Ground granulated blast furnace slag	Slag, Grade 120, ASTM C989-24	203 lb.
Water reducer	Type A, ASTM C494-24	4 oz./cwt
Water	—	34.0 gal.

The PCC slabs were 12 ft. wide to match the lane width and 14 ft. long to follow Caltrans JPCP standard practice. The transverse joints were not provided with dowels as the dowels restrict the curling and warping³ deformation of the slabs and, consequently, reduce the potential for inducing reflective cracking. The base of the JPCP was made of the same Class 2 aggregate material used in the rutting evaluation sections. The elevation of the JPCP slabs surface matched the elevation of the first surface layer lift of the rutting evaluation sections (Figure 2.4). Because of this, the top asphalt lift could be paved along the two types of sections. The design adopted for the cracking evaluation sections is summarized in Figure 2.5.

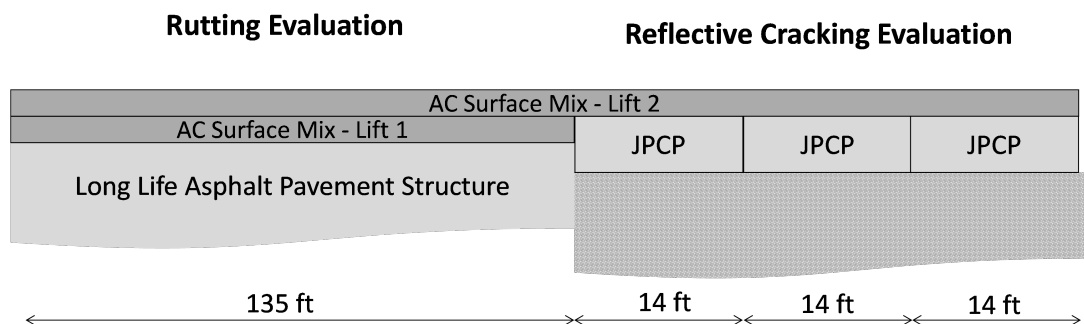


Figure 2.4: Elevation of rutting and reflective cracking evaluation sections and JPCP surface.

³ The term “curling” is typically used to refer to slab curvature due to thermal gradient (gradient versus depth) while the term “warping” is typically used to refer to slab curvature due to differential drying shrinkage (differential versus depth).

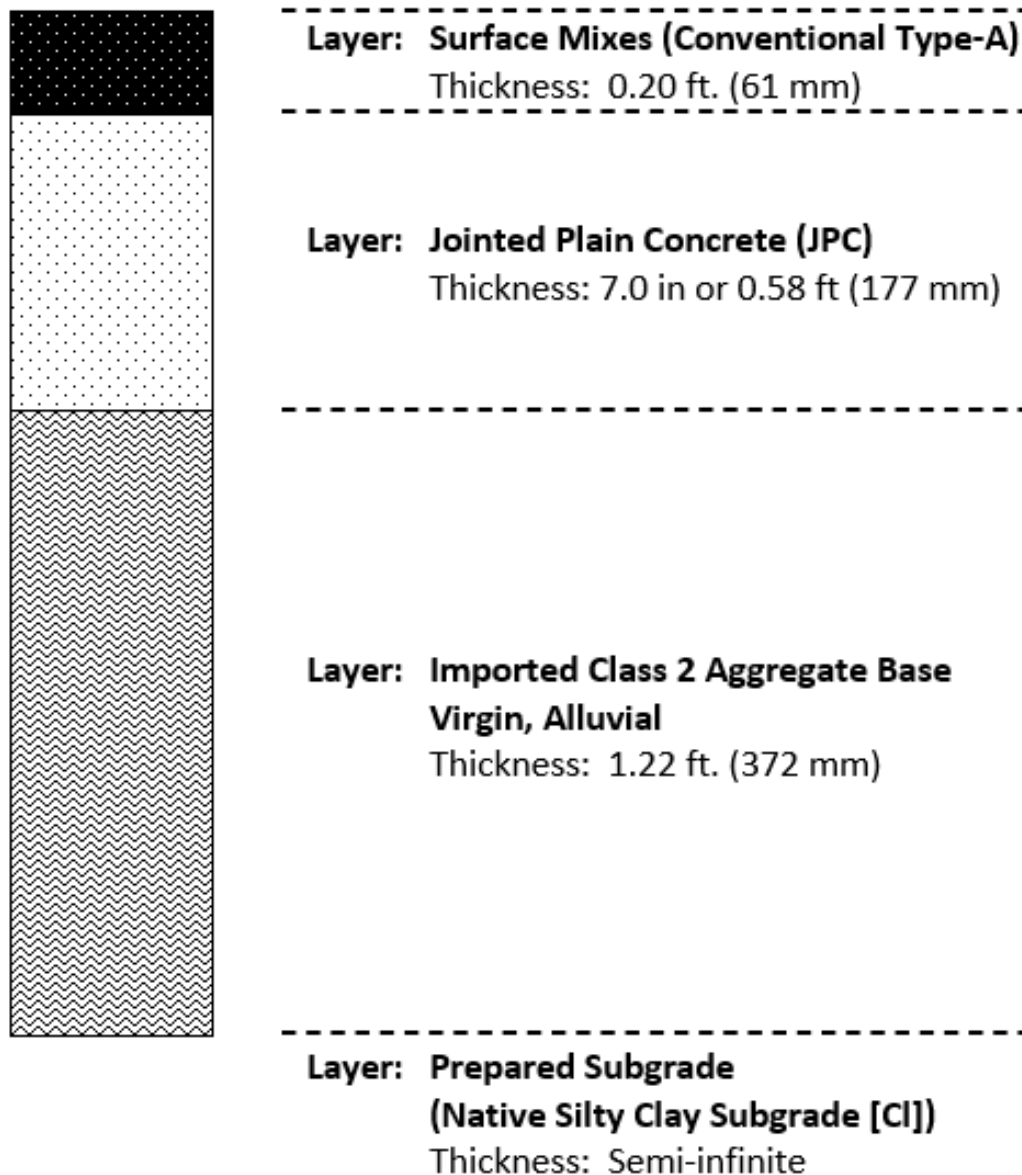


Figure 2.5: Cross-sectional view of the reflective cracking evaluation sections.

3 DEMOLITION, SUBGRADE, BASE, AND CONCRETE CONSTRUCTION

This chapter summarizes the demolition of the previous test track and the construction of the unbound base layers for the BMD test sections, including subgrade ripping, grading, and recompaction; aggregate base (AB) placement; and concrete paving. QC/QA testing was conducted at each stage, and survey and machine control technologies were used to ensure the geometry and layer thickness. Instrumentation was also installed to support future performance monitoring. These activities took place over a two-month period, from June to July 2024. The weather conditions were generally favorable, with daytime temperatures ranging from 90°F to 105°F (32.2°C to 40.6°C) and no precipitation.

3.1 Demolition and Subgrade Recompaction

3.1.1 Demolition

The existing test track, referred to as the West Track, was fully removed to allow for construction of the new test sections. Demolition included the following activities:

- Removal of all existing concrete and asphalt pavement and existing AB.
- Full-width excavation of each of the four 12 ft. wide lanes over a 180 ft. length.
- Hauling and disposal of all existing materials. The existing AB was used by the UC Davis College of Agricultural and Environmental Sciences for service roads, and the asphalt and plain concrete pavement were recycled at the Teichert Woodland quarry into Class II recycled AB.

Demolition and grading activities were completed using Caterpillar equipment rented from Holt of California and UC Davis Heavy Equipment Services, with support from SITECH West and Topcon for survey work and global positioning system (GPS) integration. The previously existing test track is shown in Figure 3.1, and demolition activity pictures are shown in Figure 3.2 to Figure 3.7.



Figure 3.1: West Track prior to BMD construction.



Figure 3.2: Milling of existing asphalt pavement.



Figure 3.3: Breaking concrete with jackhammer.



Figure 3.4: Loader removing concrete pavement.



Figure 3.5: Stockpile of removed concrete pavement.



Figure 3.6: Demolished JPCP.



Figure 3.7: West Track after JPCP removal.



Figure 3.8: Ripping subgrade.



Figure 3.9: Subgrade recompaction.

3.1.2 Subgrade Recompanction

After demolition and excavation, the exposed native lean clay subgrade was ripped, moisture-conditioned, and recompacted (Figure 3.8 and Figure 3.9). A Modified Proctor test analysis (AASHTO T 180-22) was conducted on the subgrade soil (Table 3.1). Based on these test results, the target density and moisture content for compaction were set as follows:

- Target dry density (95% Modified Proctor maximum density): 110 lb./ft³ (pcf)
- Target moisture content: 12%

Table 3.1: Subgrade Soil Modified Proctor Test Results

Mold Size	Maximum Dry Density (pcf)	Dry Density 95% Target (pcf)	Maximum Wet Density (pcf)	Wet Density 95% Target (pcf)	Optimum Moisture Content (%)
4 in.	116.2	110.4	130.4	123.9	12.2

The field compaction was conducted using a padfoot and vibratory steel drum rollers.

The QC/QA testing included the following:

- Nuclear gauge (Figure 3.10): In-place density and moisture content at 4 in. and 8 in. depths (probe length) (results shown in Table 3.2).
- In-place moisture content measured per AASHTO T 265-22 (laboratory determination of soil moisture content): One set of tests was conducted before the compaction (column labeled “Initial” in Table 3.2), while a second set was conducted after the compaction (column labeled “Final” in Table 3.2).
- Dynamic cone penetrometer (DCP): Subgrade stiffness and uniformity.
- GPS elevation checks: Verification of target subgrade elevations using survey control ± 0.05 ft. of design thickness.

The DCP penetration rates ranged from approximately 0.28 to 1 in./blow (7 to 25 mm/blow), consistent with a low-plasticity clay soil (CL according to the Unified Soil Classification System). All test locations met or exceeded 95% Modified Proctor compaction, as shown in Table 3.2, and surveyed elevations confirmed uniform depth control across all lanes.



Figure 3.10: Nuclear gauge evaluation of subgrade soil.

Table 3.2: Subgrade In-Place Density and Moisture Content Testing Results

Location			Nuclear Gauge 4 in. Depth			Nuclear Gauge 8 in. Depth			In-Place Moisture Content (%)	
Lane	Approx Station	ID	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)	Initial	Final
1	1+50	1-1	114.3	98	—	109.0	94	17.4	13.5	17.0
1	0+75	1-2	111.2	96	13.4	121.4	104	12.2	—	12.6
1	0+25	1-3	111.4	96	—	113.7	98	15.1	—	26.2
2	1+50	2-1	116.6	100	15.4	117.7	101	14.9	5.5	14.2
2	0+75	2-2	129.0	111	9.2	126.0	108	8.8	—	15.1
2	0+25	2-3	122.7	106	8.4	128.3	110	8.2	—	13.6
3	1+50	3-1	112.9	97	16.9	112.9	97	16.3	12.1	17.8
3	0+75	3-2	123.7	106	7.1	131.6	113	6.9	—	23.4
3	0+25	3-3	124.3	107	10.0	124.9	107	9.5	—	15.6
4	1+50	4-1	116.9	101	12.9	119.6	103	12.2	2.3	14.3
4	0+75	4-2	117.2	101	12.0	120.2	103	11.9	—	15.8
4	0+25	4-3	111.0	96	16.7	110.7	95	17.0	—	22.6

3.2 Aggregate Base Construction

Following subgrade acceptance, Class 2 aggregate base (AB) was imported and placed in several compacted lifts, 2 to 4 in. each, totaling 1.0 ft. in the rutting evaluation sections and 1.12 ft. in the cracking evaluation sections. The material was supplied by the Teichert Woodland plant. A Modified Proctor test (AASHTO T 180-22) was conducted on the AB material (Table 3.3). Based on these test results, the target density and moisture content for compaction were set as follows:

- Target dry density (95% Modified Proctor maximum density): 140 lb./ft³ (pcf)
- Target moisture content: 6%

Table 3.3: Aggregate Base Modified Proctor Test Results

Mold Size	Maximum Dry Density (pcf)	Dry Density 95% Target (pcf)	Maximum Wet Density (pcf)	Wet Density 95% Target (pcf)	Optimum Moisture Content (%)
4 in.	139.2	132.2	148.8	141.4	6.9
6 in.	141.4	134.3	149.3	141.8	5.6

The AB was placed using a motor grader equipped with 3D control, and the compaction was conducted by using vibratory steel drum rollers. Figure 3.11 shows the material stockpiled before the construction and Figure 3.12 shows the compacted base.



Figure 3.11: Imported Class II aggregate base before construction.



Figure 3.12: Graded and compacted Class II aggregate base.

The QC/QA testing included the following:

- Nuclear gauge (Figure 3.13): In-place wet and dry densities and moisture content at 2 in., 4 in., 6 in., and 8 in. depths (probe length) (results shown in Table 3.4 and Table 3.5).
- In-place moisture content measured per AASHTO T 255-22 (laboratory determination of aggregate moisture content): Only one set of tests was conducted, after the compaction (column labeled “Final” in Table 3.5).
- GPS elevation checks: Verification of target subgrade elevations using survey control ± 0.05 ft. of design thickness.

All test locations met or exceeded 95% Modified Proctor compaction, shown in Table 3.4 and Table 3.5, and surveyed elevations confirmed uniform depth control across all lanes.



Figure 3.13: Aggregate base testing with nuclear gauge.

Table 3.4: Aggregate Base In-Place Density and Moisture Content Testing Results (2 in. and 4 in. Probe)

Location			Nuclear Gauge 2 in. Depth			Nuclear Gauge 4 in. Depth		
Lane	Approx Station	ID	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)
1	1+50	1-1	136.0	96	5.1	136.0	96	4.9
1	0+75	1-2	136.3	96	5.4	137.8	97	5.1
1	0+25	1-3	137.3	97	4.4	135.7	96	4.5
2	1+50	2-1	136.7	97	5.4	139.3	99	5.1
2	0+75	2-2	136.5	97	4.9	136.0	96	4.7
2	0+25	2-3	133.1	94	6.4	133.2	94	6.3
3	1+50	3-1	138.6	98	4.9	138.1	98	6.5
3	0+75	3-2	139.7	99	4.5	136.8	97	5.3
3	0+25	3-3	138.7	98	5.1	141.6	100	4.1
4	1+50	4-1	138.8	98	7.9	140.4	99	4.9
4	0+75	4-2	131.7	93	8.3	136.0	96	4.1
4	0+25	4-3	136.1	96	5.6	135.2	96	5.8

Table 3.5: Aggregate Base In-Place Density and Moisture Content Testing Results (6 in. and 8 in. Probe)

Location			Nuclear Gauge 6 in. Depth			Nuclear Gauge 8 in. Depth			In-Place Moisture Content (%)
Lane	Approx Station	ID	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)	Dry Density (pcf)	% Max. Density (%)	Moisture Content (%)	Final
1	1+50	1-1	136.5	97	4.4	131.6	93	3.4	2.6
1	0+75	1-2	138.0	98	3.5	137.5	97	3.6	2.5
1	0+25	1-3	134.8	95	5.4	134.4	95	5.4	2.8
2	1+50	2-1	144.1	102	5.4	143.4	101	4.2	2.7
2	0+75	2-2	138.0	98	3.4	141.2	100	3.8	3.0
2	0+25	2-3	132.6	94	7.6	136.0	96	6.2	2.5
3	1+50	3-1	149.4	106	5.4	143.8	102	5.2	2.9
3	0+75	3-2	146.8	104	3.9	143.4	101	4.1	2.9
3	0+25	3-3	143.4	101	3.8	140.1	99	3.3	2.6
4	1+50	4-1	140.1	99	4.5	142.2	101	4.6	2.9
4	0+75	4-2	139.1	98	4.9	142.2	101	5.3	2.6
4	0+25	4-3	137.6	97	3.6	137.5	97	3.7	3.0

3.3 Concrete Construction

The wood forms for the concrete paving were installed by the UCPRC (Figure 3.14 and Figure 3.15). The PCC was supplied by a local plant (CEMEX, West Sacramento), located 15 miles from the test track site and placed and consolidated by the UC Davis Transportation Services concrete masonry construction crew on August 20–21, 2024. The PCC mix design was discussed previously and

summarized in Table 2.6. Six ready-mix truck loads were used in the construction of the slabs. Three loads were used on the first paving day (August 20, 2024) to build the Lane 2 and Lane 4 slabs, and three more loads were used on the second paving day (August 21, 2024) to build the Lane 3 and Lane 1 slabs (Table 3.6).

Table 3.6: Truck Loads

Paving Day	Truck #	Batch Time	Arrive at Job	Begin Pouring	End Pouring	Used for Lanes
1 (August 20, 2024)	1	4:20 am	5:05 am	5:15 am	5:35 am	Lane 4
1 (August 20, 2024)	2	4:47 am	5:27 am	5:47 am	6:07 am	Lanes 4 and 2
1 (August 20, 2024)	3	5:06 am	5:42 am	6:15 am	6:26 am	Lane 2
2 (August 21, 2024)	4	4:19 am	4:55 am	5:14 am	5:22 am	Lane 3
2 (August 21, 2024)	5	4:46 am	5:27 am	5:40 am	6:02 am	Lanes 3 and 1
2 (August 21, 2024)	6	5:06 am	5:45 am	6:13 am	6:30 am	Lane 2

The trucks arrived at the construction site with some of the theoretical mixing water not yet added. Most of the retained water was added at the construction site to reach the target slump (6 to 8 in.). After the water addition, the theoretical water content of the PCC of the trucks was between 33.0 and 33.7 gal./cyd, which is slightly below the 34 gal./cyd design water content (Table 2.6).

The PCC was consolidated with a vibrating roller screed (Figure 3.16) and finished with a trowel. No surface texturing was applied. The curing was conducted with a white pigmented resin-based curing compound meeting ASTM C309-19 Type 2B specifications, applied at a nominal rate of 1 gal./150 ft². The appearance of the JPCP after the curing compound application is shown in Figure 3.17.

The weather conditions during paving were dry and warm, with an air maximum temperature and minimum relative humidity (RH) of approximately 95°F (35°C) and 25%, respectively. Wind speed was below 2 mph and the sky was clear. Air minimum temperature and maximum RH, during the first nights after construction, were approximately 60°F (16°C) and 80%, respectively. The days following construction had similar weather conditions, typical of Davis during the summer.

The construction QC/QA testing included the following:

- Fresh concrete testing (every truck):
 - Slump, following ASTM C143-20
 - Air content, following ASTM C173-24 (Volumetric Method)
 - Temperature, following ASTM C1064-23
 - Unit weight, following ASTM C138-24
 - Water content, following AASHTO T 318-15 (2023)

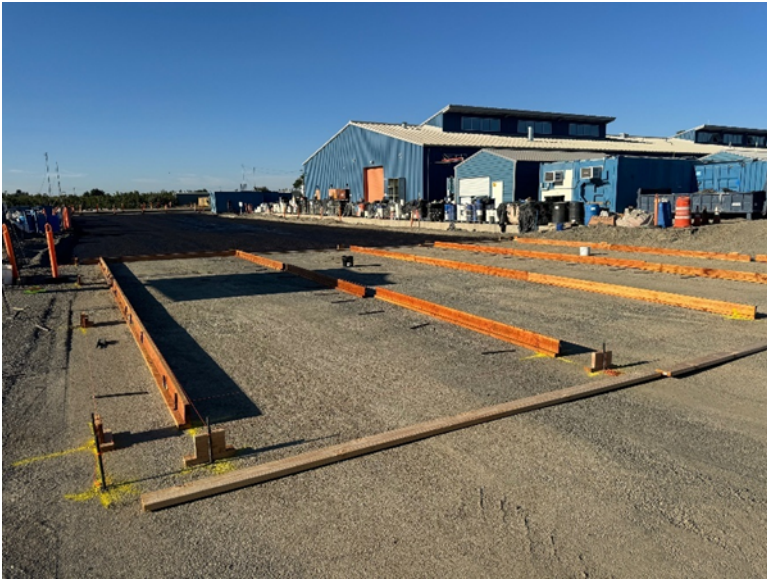


Figure 3.14: JPCP formwork placement.



Figure 3.16: PCC placement.



Figure 3.15: Instrumentation installation.



Figure 3.17: JPCP after construction.

- Hardened concrete testing (Truck 2):
 - Flexural strength, following ASTM C78-22, at 10 days and 28 days
 - Compressive strength, following ASTM C39-24, at 10 days and 28 days (this testing was also conducted on mix sampled from Truck 5)
 - Modulus of elasticity, following ASTM C469-22, at 10 days and 28 days
 - Drying shrinkage, following ASTM C157-24, except for the age when drying begins, for which 7 days was chosen instead of the 28 days prescribed in the ASTM standard

The specimens for hardened concrete testing were prepared with mixture sampled from the corresponding ready-mix truck during the slab's construction. The specimens were covered with wet burlap inside a room for 24 hours. Then they were demolded and stored in lime water at 73°F (23°C) until testing.

The results of the fresh concrete testing are shown in Table 3.7 and Figure 3.18, and the results of the hardened concrete testing are shown in Figure 3.20 to Figure 3.22. The data in the table and figures and findings indicate that the PCC fresh and hardened properties matched expectations:

- The consistency of the fresh concrete (slump) was fluid enough to allow manual consolidation with a rolling screed.
- The strength of the hardened concrete would have allowed the slabs to be opened to traffic in less than 10 days, as the 10-day flexural strengths exceeded the 550 psi opening strength requirement in Caltrans Standard Specifications, Section 40.
- The water content of the concrete was higher than expected (Figure 3.18), despite the strengths being high.
- The drying shrinkage of the concrete was considerably higher than normal Caltrans mixes, reaching 720 $\mu\epsilon$ after 28 days of drying (Figure 3.22), which exceeds the 500 $\mu\epsilon$ limit for concrete paving established in Caltrans Standard Specifications, Section 90. As previously explained, this PCC mix was intended to have high drying shrinkage to maximize the potential for inducing reflective cracking on the asphalt overlay. Based on the data presented in Figure 3.22, the PCC matched the expectation.

Table 3.7: Fresh Concrete Testing Results

Truck #	Slump (in.)	Temperature (°F / °C)	Air Content (%)	Unit Weight (pcf)	Water Content (%)
1	7.75	76.5 / 24.7	2.3	151.0	7.24
2	6.75	77.0 / 25.0	2.3	150.5	7.41
3	7.00	76.3 / 24.6	1.3	150.1	7.42
4	8.75	77.7 / 25.4	1.2	149.8	7.66
5	7.25	77.2 / 25.1	1.2	150.8	7.40
6	6.50	75.2 / 24.0	1.3	149.5	7.71

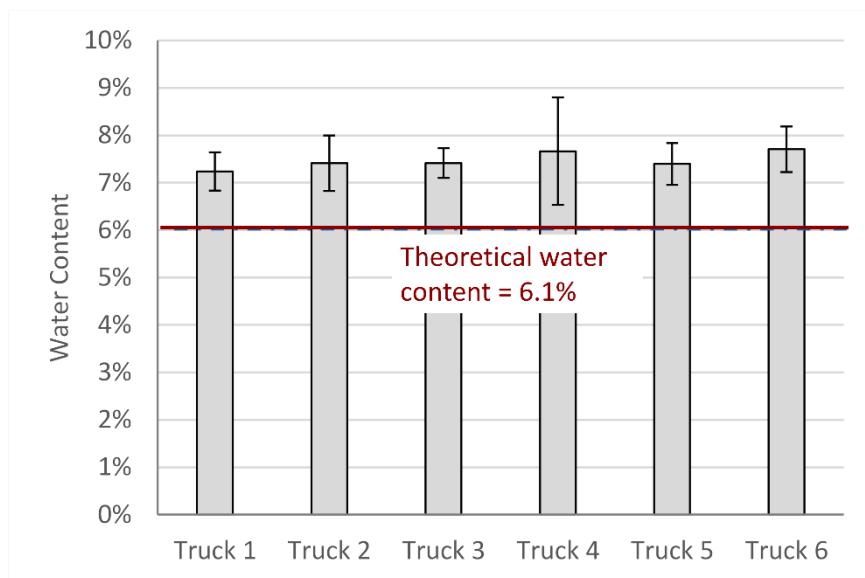


Figure 3.18: Water content of the PCC.

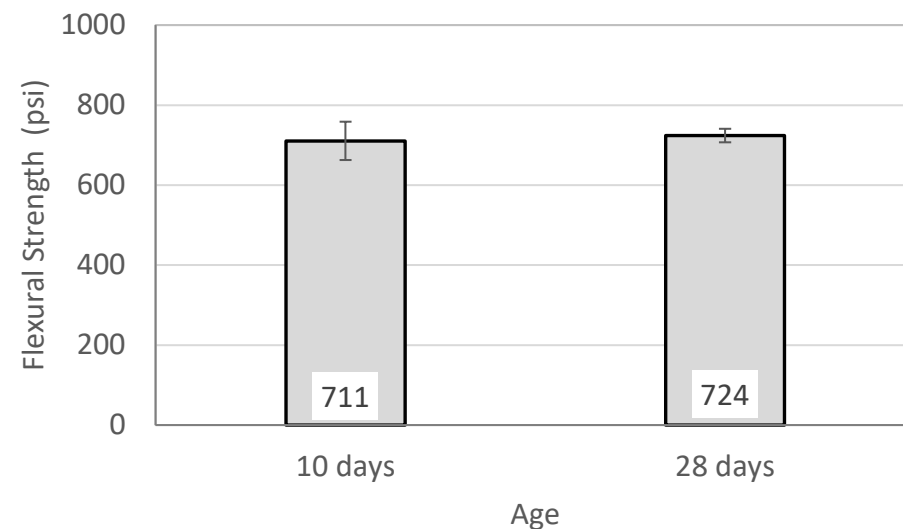


Figure 3.20: Flexural strength of the PCC (Truck 2).

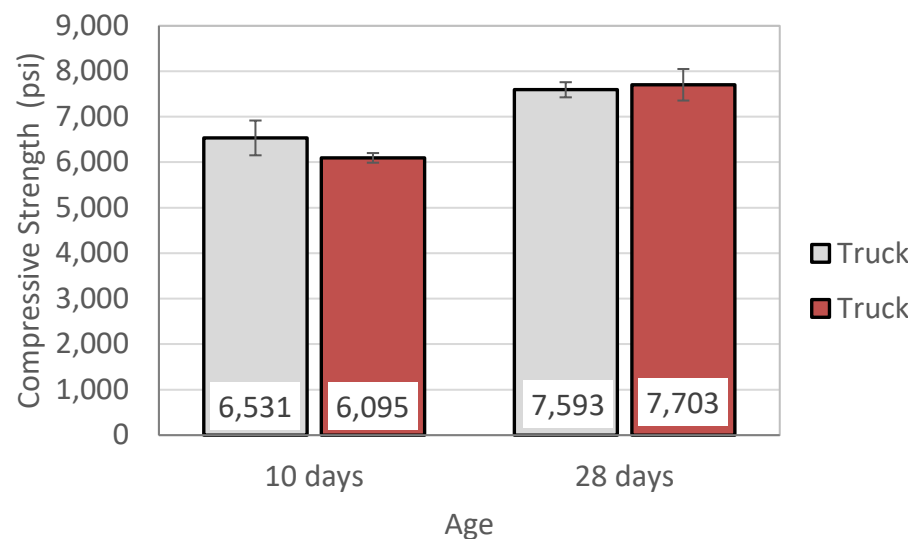


Figure 3.19: Compressive strength of the PCC.

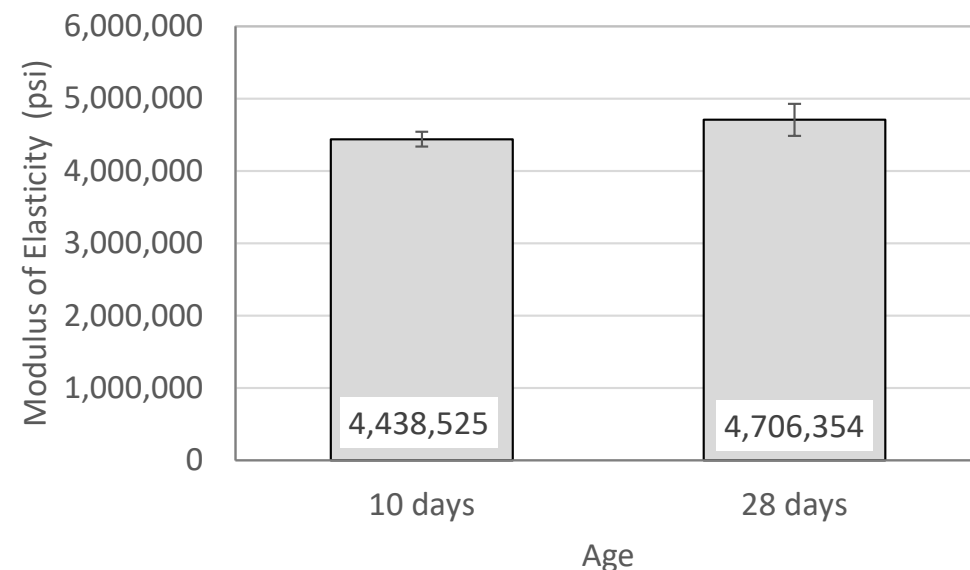


Figure 3.21: Modulus of elasticity of the PCC (Truck 2).

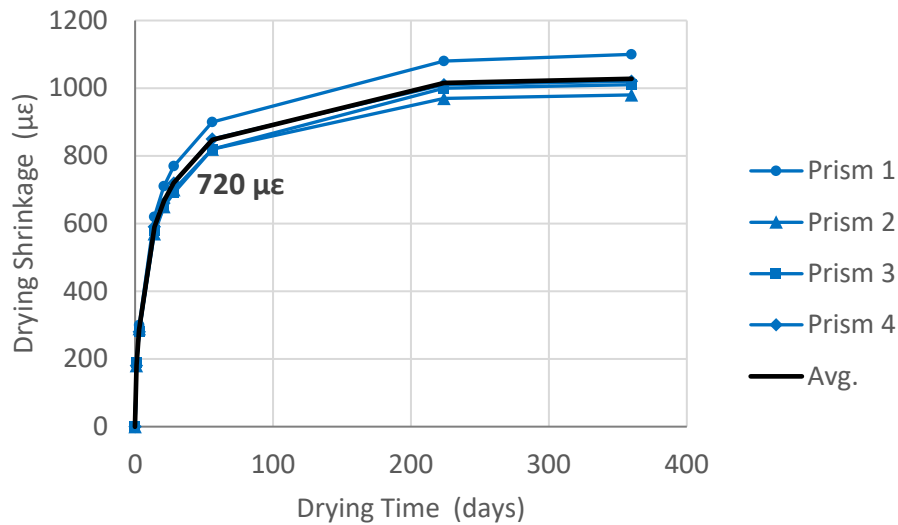


Figure 3.22: Drying shrinkage of the PCC (73°F [23°C] and 50% relative humidity).

3.4 Instrumentation

3.4.1 Concrete Pavement Instrumentation

The instrumentation installed in the concrete sections in each of the lanes, shown in Figure 3.23, focused on measuring the slab responses to ambient environmental loading (i.e. temperature and humidity). The instrumentation of each lane includes the following:

- Two thermocouple rods for measuring the temperature profile in the slabs and base
- Four pairs of GeoKon 4200 vibrating wire strain gauges (VWSG), aligned in the slab longitudinal direction and located at a distance of 10 in. from the transverse joint (each pair includes a VWSG close to the top of the slab [1.0 in. from top] and another VWSG close to the bottom of the slab [1.0 in. from bottom])

The sensor cables were installed in plastic conduits buried in the AB (Figure 3.24), while the sensors were installed in 3D-printed plastic chairs (Figure 3.25).

The strain and temperature data were collected by a Campbell Scientific data acquisition system (DAS) located adjacent to the slabs (Figure 3.26). Data were collected at 2-minute intervals during the paving days and at 10-minute intervals thereafter. The data will be used to calculate transverse joint opening and closing under thermal and drying shrinkage actions.

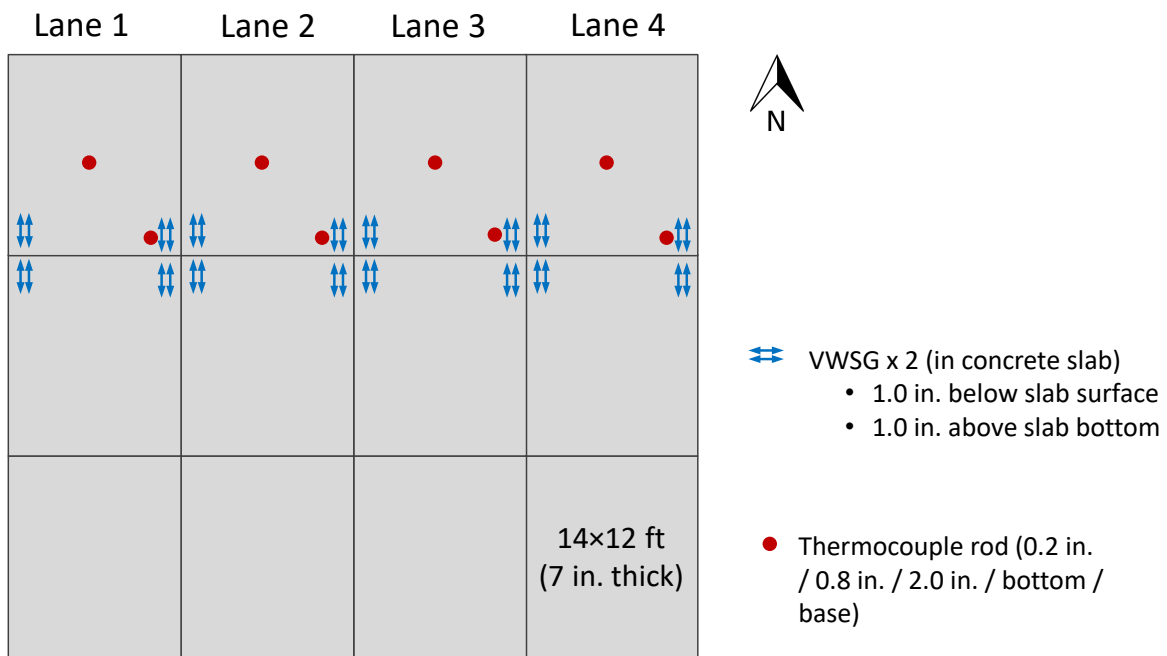


Figure 3.23: Concrete slab instrumentation (bird's eye view).

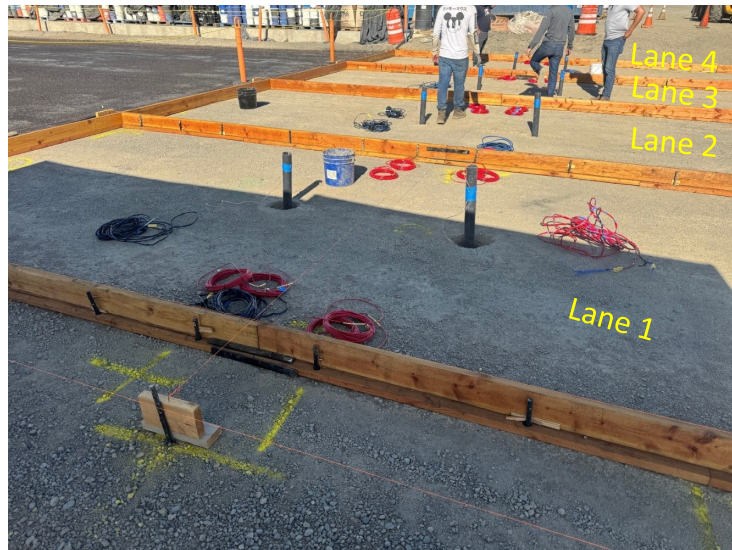


Figure 3.24: Layout of conduits for instrumentation cables.

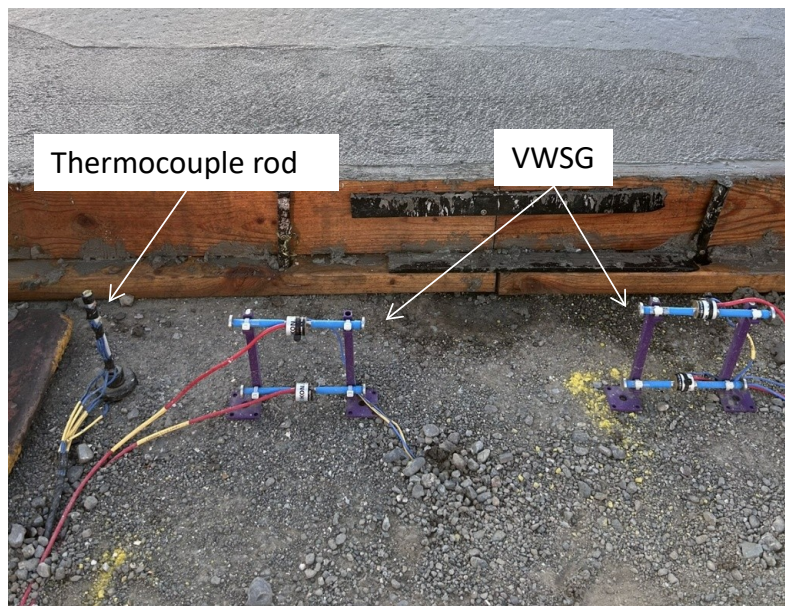


Figure 3.25: Thermocouple rod and VWSGs.



Figure 3.26: Data acquisition system (DAS) box by the slabs.

3.4.2 Asphalt Pavement Instrumentation

The instrumentation installed in each of the lanes, shown in Figure 3.27 and Figure 3.28, focuses on measuring the pavement's response to the HVS loading. The instrumentation of each lane includes the following:

- Two thermocouple rods for measuring the temperature profile in the asphalt layers
- A Geokon 3500 earth pressure cell installed in the AB layer to measure the vertical stress applied with the HVS loading at the bottom of the asphalt layers
- Two pairs of asphalt strain gauges, type PMFLS-60-50 (Tokyo Measuring Instruments Laboratory) installed in transverse and longitudinal directions beneath the intermediate and rich bottom layers to measure strain
- A multi-depth deflectometer (MDD), consisting of linear voltage deflection transducers (LVDTs) installed at specific depths in the pavement, to measure the elastic deflection and permanent deformation of the HMA, AB, and subgrade layers

The pressure cells (Figure 3.29) and strain gauges (Figure 3.30) were installed during the construction of the test track. The MDD and thermocouples were installed after the construction, before HVS testing.

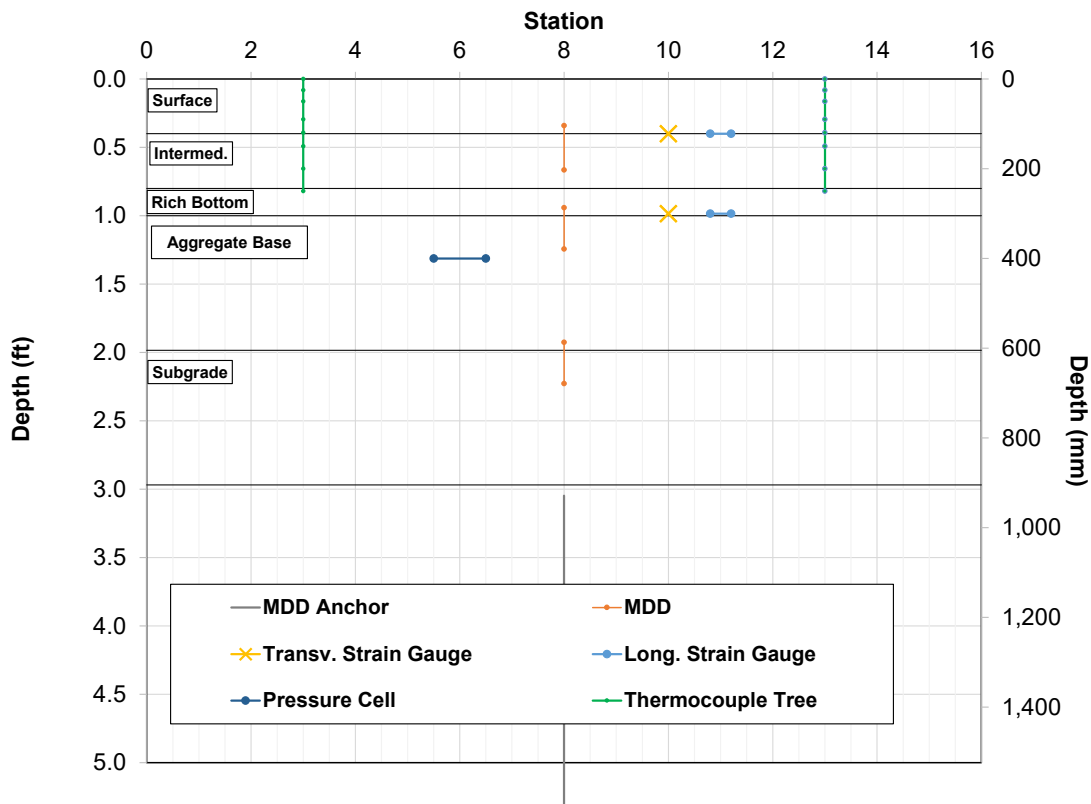


Figure 3.27: Asphalt pavement instrumentation layout.

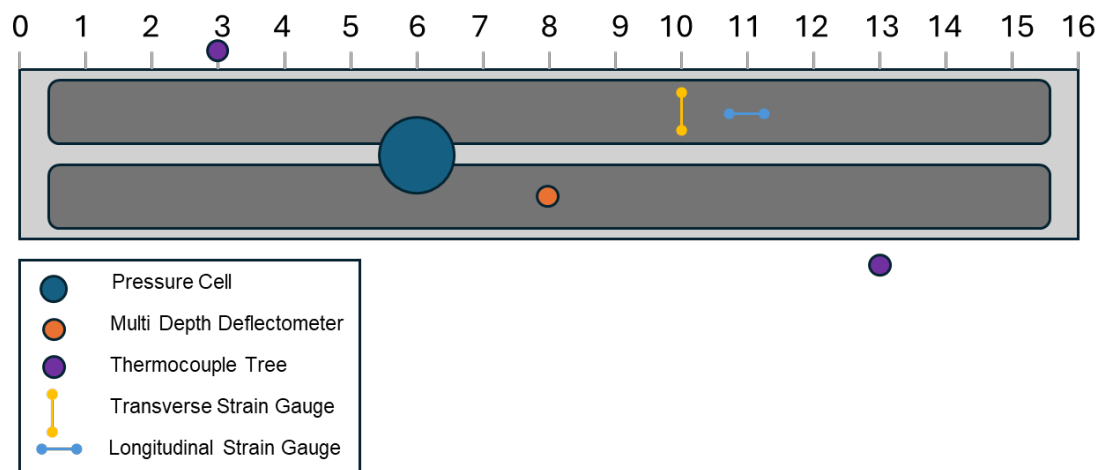


Figure 3.28: Instrumentation plan view.



Figure 3.29: Pressure plate installation in the aggregate base.



Figure 3.30: Strain gauges on the surface of the aggregate base.

3.5 Surveying, Modeling, and 3D Equipment Control

The construction was performed and documented through survey control and 3D modeling techniques to ensure elevation, slope, and layer thickness of the test sections.

3.5.1 As-is Survey and 3D Model Development

An as-is survey of the existing track was performed by the UCPRC using a Topcon base and rover GPS and by SITECH West using a Trimble base and rover GPS system. These data were used to generate a digital terrain model of the existing surface.

A 3D model of the new test track and updated lane boundaries was then created in AutoCAD, including the following:

- Subgrade, AB, jointed plain concrete, and asphalt surface elevations
- Section transitions and cross-slopes

3.5.2 GPS Machine Control and Verification

The 3D model was uploaded into the Caterpillar equipment with GPS-based grade control and benchmarked with a Trimble base station. This allowed accurate grading operations for the subgrade and aggregate layers in multiple lifts. The following specifications of the 3D model were confirmed:

- Grading tolerances using machine control ± 0.05 ft. of design thickness.
- Calibration of the motor grader blade height control system was performed by SITECH West.
- Survey checks using the GPS rover confirmed each lift met thickness targets.
- Any necessary adjustments were made during construction to correct minor deviations.

3.5.3 Tracer Plate Installation

Metal tracer plates were installed at subgrade and aggregate layer interfaces (Figure 3.31 and Figure 3.32). These markers were surveyed and will allow future ground-penetrating radar (GPR) surveys to confirm layer thickness in initial and future construction cycles and monitor potential layer movement or settlement during HVS loading and environmental cycling.

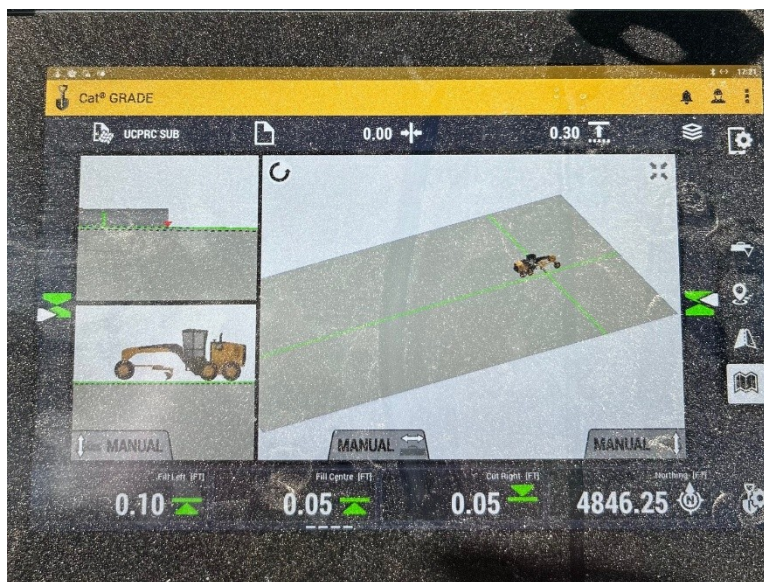


Figure 3.31: Motor grader machine control.



Figure 3.32: GPR metal tracing plates.

3.6 Prime Coat

A prime coat was applied by VSSI to protect the AB layer between grading and paving operations (Figure 3.33 and Figure 3.34). The prime coat was completed on August 6, 2024, while the asphalt construction was completed around three months later, on November 6–7, 2024.



Figure 3.33: Prime coat placement on the aggregate base.



Figure 3.34: Prime coated aggregate base.

4 ASPHALT MIX DESIGNS

4.1 Overview of the Mix Design Processes

This chapter summarizes the design of the HMA mixes used in the test track including mix design process, material sources, and performance testing. The designs are based on prior CalME validation work for the control surface mix being used on Caltrans mainline pavement, and prior Caltrans/industry/UCPRC work on mix designs for intermediate and rich bottom layers for long-life asphalt pavement using performance-related specifications (PRSs). The mix designs were separated into two groups: (1) intermediate and rich bottom layer mixes and (2) HMA Type A surface mix designs.

4.1.1 Intermediate and Rich Bottom Layer Mixes

The intermediate and rich bottom layer mixes were designed as part of a long-life pavement structure. The mix designs followed Caltrans asphalt concrete (AC) long-life concepts and were validated in an abbreviated number of performance-related tests (PRTs), balancing resources and schedule.

The stiffness and fatigue cracking resistance requirements were set following Caltrans AC long-life specifications for the rehabilitation of Interstate 5 in Sacramento County conducted from 2019 to 2021 (EA 03-0H10U), based on 4PB testing (ASTM D8237-21):

- The intermediate layer mix:
 - $N_f \geq 250,000$ at $433 \mu\epsilon$ peak-to-peak strain (N_f is the number of cycles to failure)
 - $N_f \geq 1,000,000$ at $220 \mu\epsilon$ peak-to-peak strain
 - $E_{50} \geq 782,000$ psi at $433 \mu\epsilon$ peak-to-peak strain (E_{50} is the stiffness at cycle 50)
- The rich bottom layer mix:
 - $N_f \geq 250,000$ at $420 \mu\epsilon$ peak-to-peak strain
 - $N_f \geq 1,000,000$ at $269 \mu\epsilon$ peak-to-peak strain
 - $E_{50} \geq 707,000$ psi at $420 \mu\epsilon$ peak-to-peak strain

The rich bottom mix was not subject to rutting specifications due to its location in the pavement structure. The intention was to maximize the fatigue resistance at the bottom of the asphalt structure and indirectly create a moisture barrier, minimizing moisture intrusion from the underlying layers due to the low air-void content. Conventional volumetric mix design requirements were waived to give the supplier additional flexibility in the mix design process. The intermediate and rich bottom maximum allowable RAP content was set at a relatively high value of 40% by aggregate replacement, and the maximum aggregate size (MAS) was set at 3/4 in.

For mix design and QC purposes, the design air-void (AV) contents of the mixes were set as follows:

- Intermediate layer mix: AV = 2.5% at 85 gyrations (in-place relative density acceptance 94% to 97%)
- Rich bottom layer mix: AV = 1.5% at 85 gyrations (in-place relative density acceptance 97% to 100%)

4.1.2 HMA Type A Surface Mix Designs

The control mix design was based on an existing Caltrans-approved HMA design with 3/4 in. MAS and polymer-modified binder (PG 64-28M) (though the binder was changed to PG 64-16 plain to meet the requirements of the PMPC BMD Working Group).

As previously mentioned, the control mix design was then modified in three different ways to reduce the rutting resistance of the mix:

- Adding 1.25% by total mass of mix (TMM) additional asphalt binder content (5.75% Pb instead of 4.5% in the control surface mix). The resulting mix is referred to as “High Pb” in this technical memorandum.
- Overdosing the warm mix additive, at 2.5% by total binder content, and changing the additive to recycling agent during production. The resulting mix is referred to as “High RA” in this technical memorandum.
- Introducing a natural sand content much higher than the design content. This was achieved by switching the feed of natural sand and 1/4 × dust aggregates, which resulted in the 25% finest portion of the aggregates having 75% natural sand and 25% 1/4 × dust aggregates instead of 25% natural sand and 75% 1/4 × dust aggregates in the other three lanes. The resulting mix is referred to as “Nat. Sand” in this technical memorandum.

All surface mixes were designed and tested in accordance with Caltrans provisional BMD protocols, with performance verification by Hamburg Wheel-Tracking (HWT) and IDEAL-CT tests. Mix designs were developed in partnership with George Reed Inc. and used to inform the job mix formula (JMF) hot drop trials. This process included the following:

- Surface mix design verification from the existing mix design and one hot drop trial per mix iteration. Mix design changes to the three mix iterations to reduce rutting resistance were based on plant-produced HWT and IDEAL-CT test results and the advice of industry.
- Intermediate mix laboratory mix design and three plant hot drop trials. Adjustments to gradation, binder content, and recycling agent content were based on Ndesign AV, 4PB beam fatigue test results, and IDEAL-CT test results.
- Rich bottom mix laboratory mix design and one plant hot drop trial. This mix was validated through 4PB beam fatigue and IDEAL-CT test results.
- Hot drops were produced at the George Reed Inc. batch plant located in Clements (Figure 4.1).



Figure 4.1: Hot drop sampling at George Reed Inc. batch plant in Clements.

The timeline of the mix design was the following:

- June 2024 to August 2024: Laboratory mix design and contractor–UCPRC planning meetings
- September 2024 to October 2024: Plant hot drop trials
- November 2024: Paving window with contractor

The materials used in the mix design and construction included the following:

- Coarse and fine aggregate from the George Reed Table Mountain quarry
- RAP (passing 3/8 in.) from the George Reed Clements recycling plant
- Binder from Valero (Benicia refinery) graded to PG 58-22 plain to meet the PG 64-16 project specification with 25% or 40% RAP
- Recycling agent from Ingevity produced from tall oil to meet the PG 64-16 project specification with 40% RAP and improve the fatigue performance of the mix

4.2 Intermediate and Rich Bottom Layers Mix Designs

The JMF design information of the intermediate and rich bottom layers mixes selected for the test track construction is shown in Table 4.1. The designs were validated by testing the mixes produced at the plant (hot drop trials).

Table 4.1: Intermediate Layer and Rich Bottom Mixes Job Mix Formulas

Parameter		¾ in. NMAS			
		Intermediate (40% RAP)		Rich Bottom (40% RAP)	
		JMF Design	JMF Limits	JMF Design	JMF Limits
Grading (% passing sieve)	1 in.	100	100-100	100	100-100
	¾ in.	98	92-100	98	92-100
	½ in.	90	85-95	77	85-95
	3/8 in.	82	—	72	—
	#4	49	44-54	47	42-52
	#8	28	23-33	32	27-37
	#16	20	—	22	—
	#30	15	11-19	16	12-20
	#50	10	—	11	—
	#100	7	—	7	—
	#200	5.2	3.2-7.2	5.4	3.4-7.4
RAP content (% by weight of aggregate)		40.0	—	40.0	—
RAP content (% by weight of mix)		39.8	—	39.7	—
Base asphalt binder performance grade		58-22	—	58-22	—
Recycling agent or liquid antistrip type		Tall oil (RA)	—	Tall oil (RA)	—
Recycling agent dosage (% by weight of binder)		2.5	—	1.5	—
Binder content (% by weight of mix)		5.4	5.0-5.8	5.5	5.1-5.9
Recycled binder content (% binder in mix)		1.9	—	1.9	—
Estimated binder replacement excluding recycling agent (% of total binder)		35.9	—	35.2	—
Estimated binder replacement including recycling agent (% of total binder)		39.9	—	39.2	—
Number of gyrations		85	85	85	85
Air-void content (%)		2.5	1.5-3.5	1.5	0.5-2.5
Dust proportion		1.0	0.6-1.3	1.0	0.6-1.3
Dust proportion effective		1.1	0.6-1.3	1.1	0.6-1.3

The intermediate and rich bottom mixes were plant-produced and sampled to perform the validation testing that included volumetrics (Figure 4.2 and Figure 4.3), HWT (Figure 4.4 and Figure 4.5), IDEAL-CT (Figure 4.6), and 4PB fatigue and stiffness (Figure 4.7 and Figure 4.8) tests. All test results passed the corresponding requirements, except the intermediate mix, which did not meet the minimum stiffness. The time and resources available did not permit scheduling and testing of another hot drop with a reduced amount of recycling agent. The fatigue resistance of the rich bottom mix barely passed the minimum requirement, shown in Figure 4.7, while its stiffness was much higher than the minimum, shown in Figure 4.8. Consequently, a decision was made to increase the recycling

agent dosage from 1.5% to 2.5% to ensure that the production mix would meet the fatigue resistance requirements.

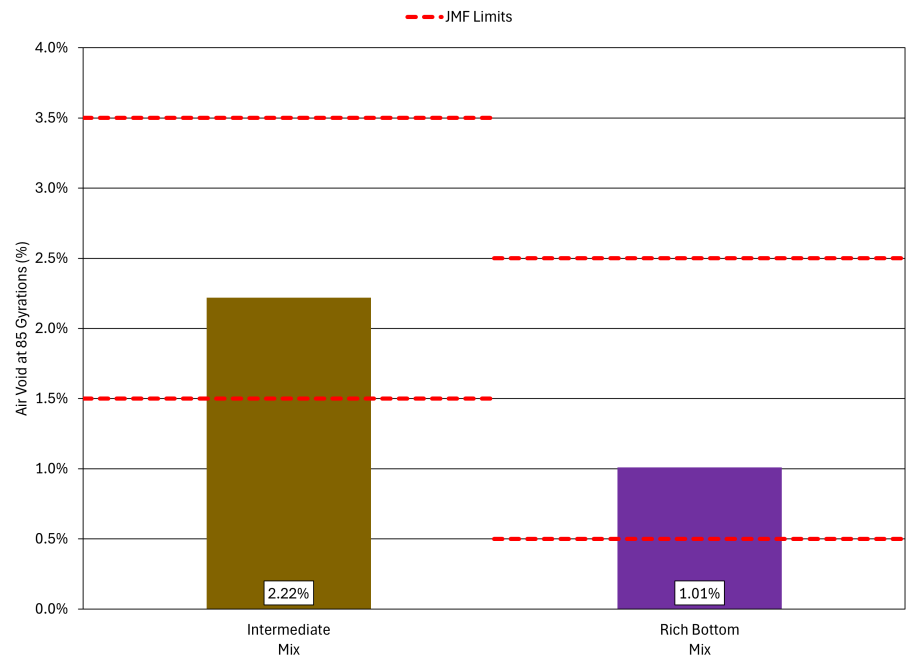
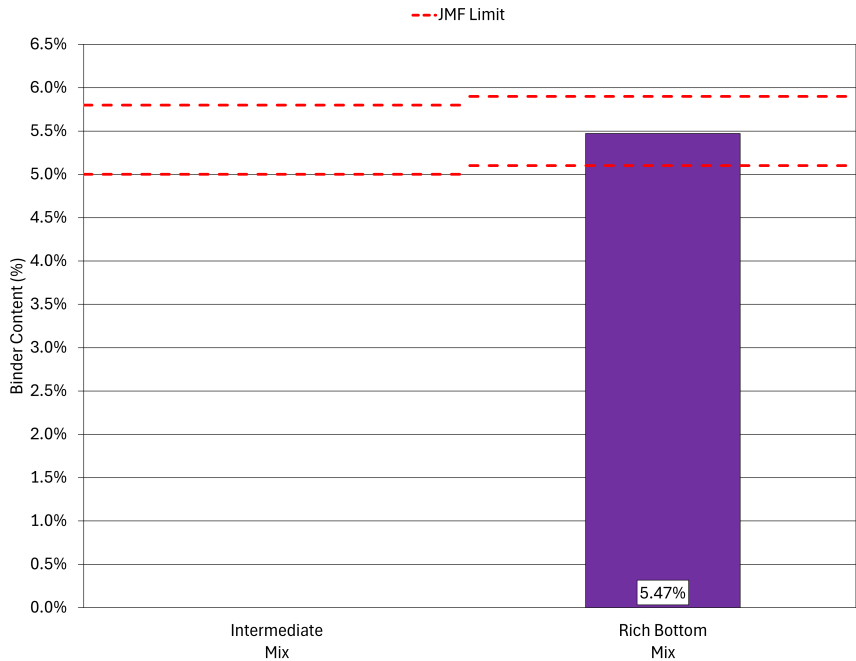


Figure 4.2: Air-voids at Ndesign (85 gyrations) from hot drop (JMF verification).



Notes: Binder content was determined with automated solvent extraction (ASTM D8159-19).
Data for Intermediate Mix not available.

Figure 4.3: Binder content (TMM) for intermediate and rich bottom mixes from hot drop (JMF verification).

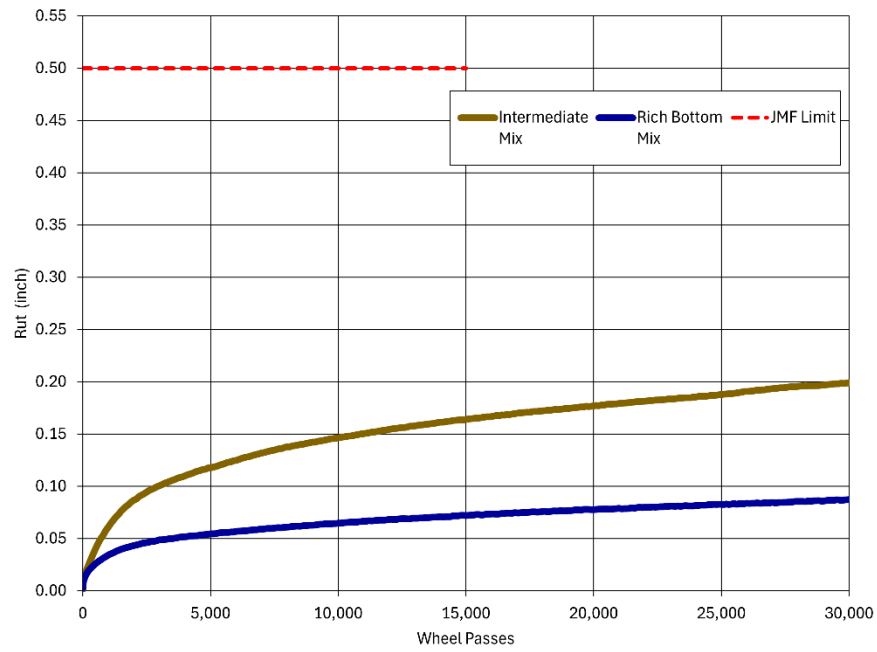


Figure 4.4: HWT test results: rutting versus wheel passes from hot drop (JMF verification).

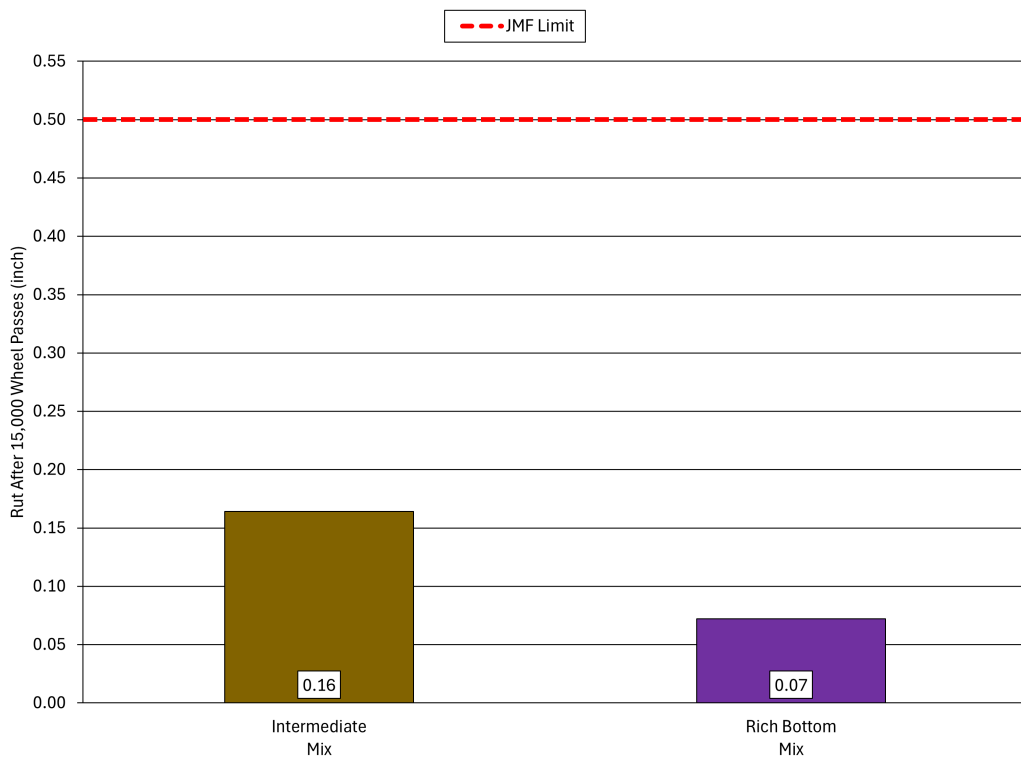


Figure 4.5: HWT test results: rutting after 15,000 wheel passes from hot drop (JMF verification).

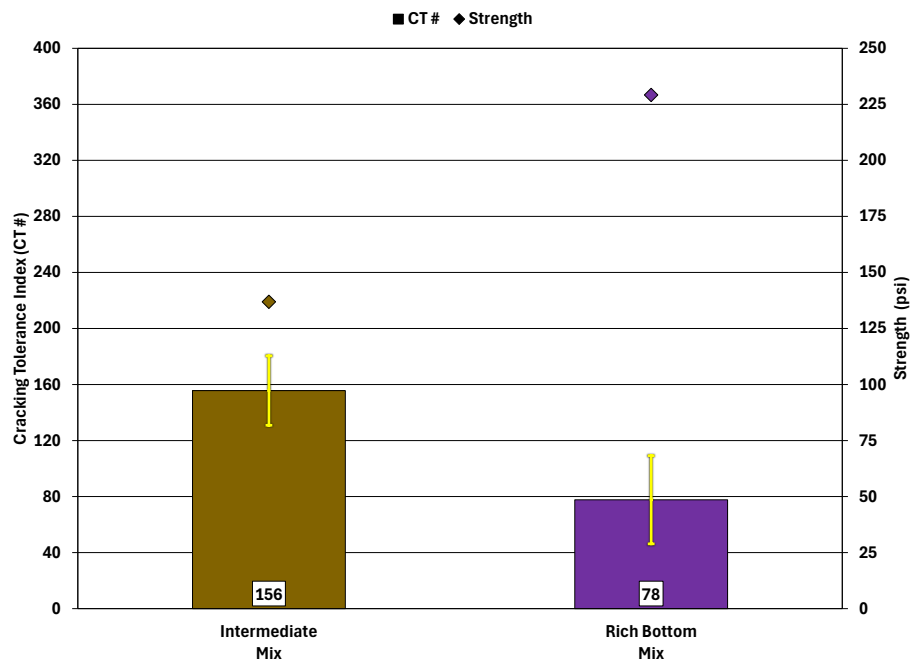


Figure 4.6: IDEAL-CT tests results from hot drop (JMF verification).

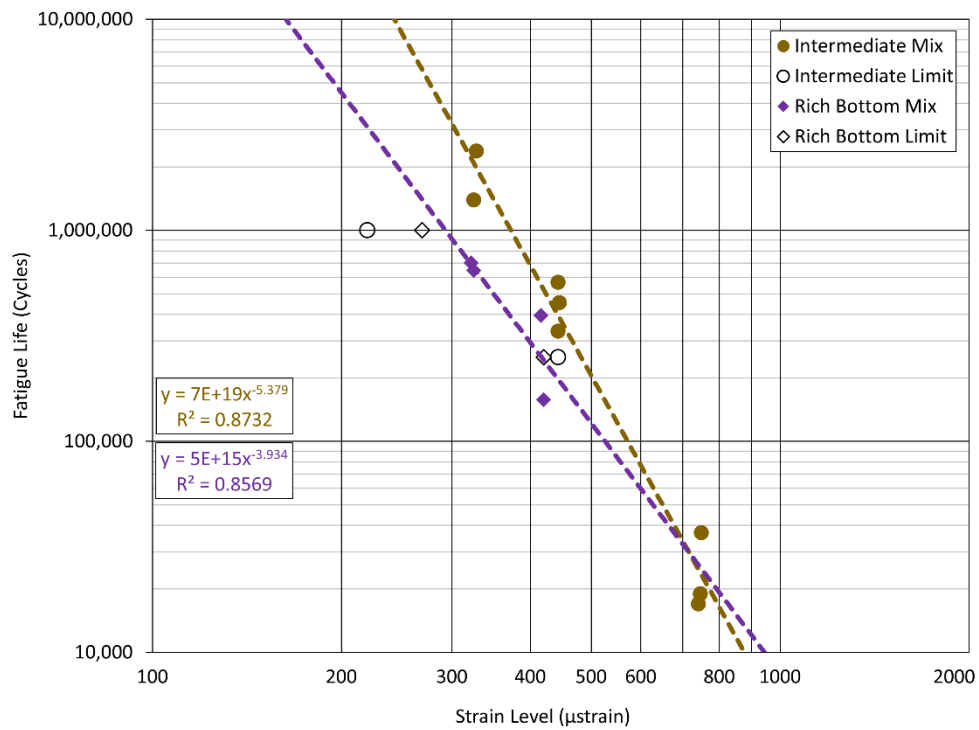


Figure 4.7: 4PB beam fatigue test results from hot drop (JMF verification).

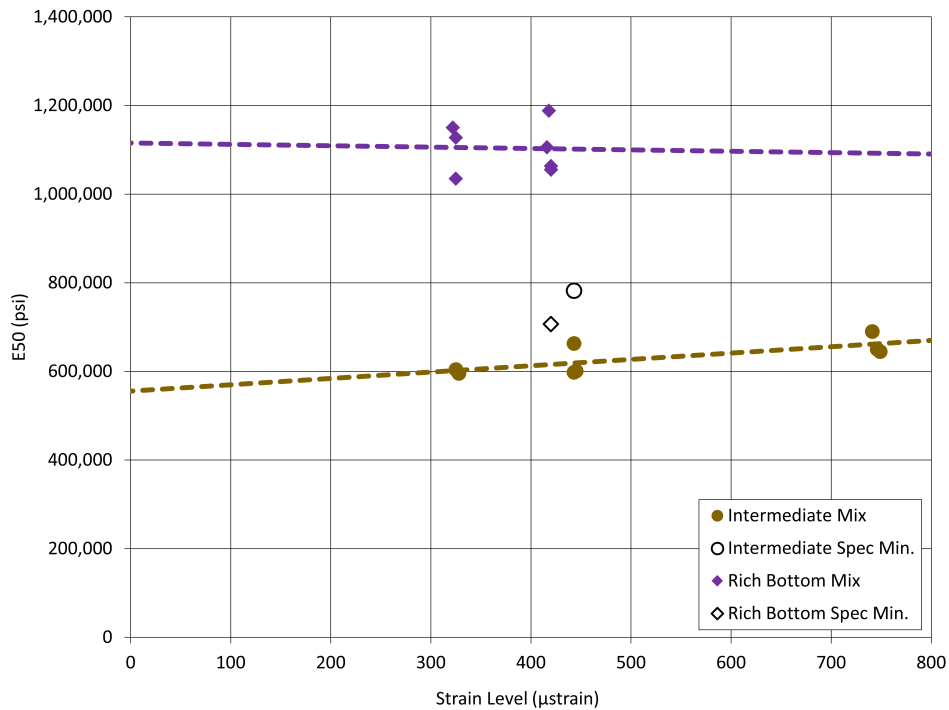


Figure 4.8: 4PB beam stiffness at cycle 50 from hot drop (JMF verification).

4.3 Surface Layer Mix Designs

The JMF design information of the surface mixes selected for the test track construction is shown in Table 4.2. The designs were validated by testing the mixes produced at the plant (hot drop trials). The surface mixes were plant-produced and sampled to perform the validation testing that included volumetrics (Figure 4.9 and Figure 4.10), HWT (Figure 4.11 and Figure 4.12), and IDEAL-CT (Figure 4.13) tests. All test results passed the corresponding requirements.

The increase in binder content in 1.25% TMM had a clear impact on the rutting performance measured with the HWT test, as the measured rutting was more than twice that of the control mix, though the mix still met Caltrans HWT specifications. However, the overdose of recycling agent and the use of a very high amount of natural sand did not have a noticeable impact on the rutting performance measured with the HWT test.

Table 4.2: Surface Mixes Job Mix Formulas

Parameter		3/4 in. NMAS							
		Control (25% RAP)		High Binder (25% RAP)		High RA (25% RAP)		Nat. Sand (25% RAP)	
		JMF Design	JMF Limits	JMF Design	JMF Limits	JMF Design	JMF Limits	JMF Design	JMF Limits
Grading (% passing sieve)	1 in.	100	100-100	100	100-100	100	100-100	100	100-100
	3/4 in.	98	92-100	98	92-100	98	92-100	98	92-100
	1/2 in.	85	79-91	85	79-91	85	79-91	85	79-91
	3/8 in.	76	—	76	—	76	—	76	—
	#4	50	45-55	50	45-55	50	45-55	50	45-55
	#8	33	28-38	33	28-38	33	28-38	34	29-39
	#16	24	—	24	—	24	—	28	—
	#30	17	13-21	17	13-21	17	13-21	21	17-25
	#50	11	—	11	—	11	—	12	—
	#100	7	—	7	—	7	—	7	—
	#200	5.0	3.0-7.0	5.0	3.0-7.0	5.0	3.0-7.0	4.6	2.6-6.6
RAP content (% by weight of aggregate)		25.0	—	25.0	—	25.0	—	25.0	—
RAP content (% by weight of mix)		25.1	—	23.9	—	25.1	—	25.1	—
Base asphalt binder performance grade		58-22	—	58-22	—	58-22	—	58-22	—
Recycling agent or liquid antistrip type		—	—	—	—	Tall oil (RA)	—	—	—
Recycling agent dosage (% by weight of binder)		—	—	—	—	2.5	—	—	—
Binder content (% by weight of mix)		4.5	4.2-5.0	5.75	5.45-6.25	4.6	4.3-5.1	4.5	4.2-5.0
Recycled binder content (% binder in mix)		1.2	—	1.2	—	1.2	—	1.2	—
Estimated binder replacement excluding recycling agent (% of total binder)		27.2	—	20.2	—	26.6	—	27.2	—
Estimated binder replacement including recycling agent (% of total binder)		27.2	—	20.2	—	30	—	27.2	—
Number of gyrations		85	85	85	85	85	85	85	85
Air-void content (%)		4.0	2.5-5.5	Report only	—	Report only	—	4.0	2.5-5.5
Voids in mineral aggregate (%)		12.5	12.5-15.5	13.5	12.5-15.5	12.5	12.5-15.5	12.7	Report only
Dust proportion		1.1	0.6-1.3	0.9	0.6-1.3	1.1	0.6-1.3	1.0	0.6-1.3
Dust proportion effective		1.3		1.0		1.2		1.1	

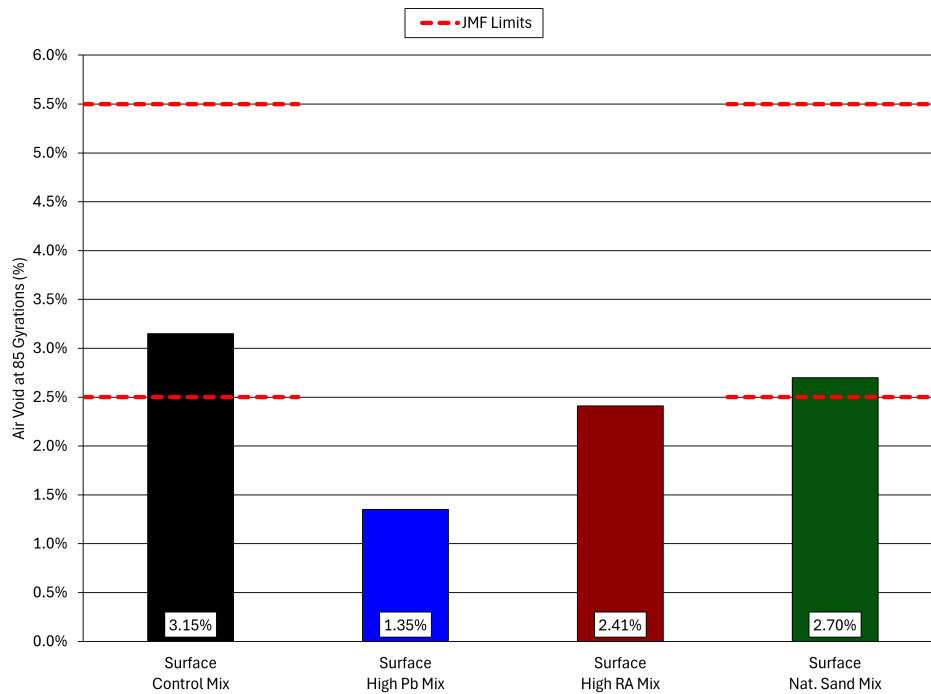
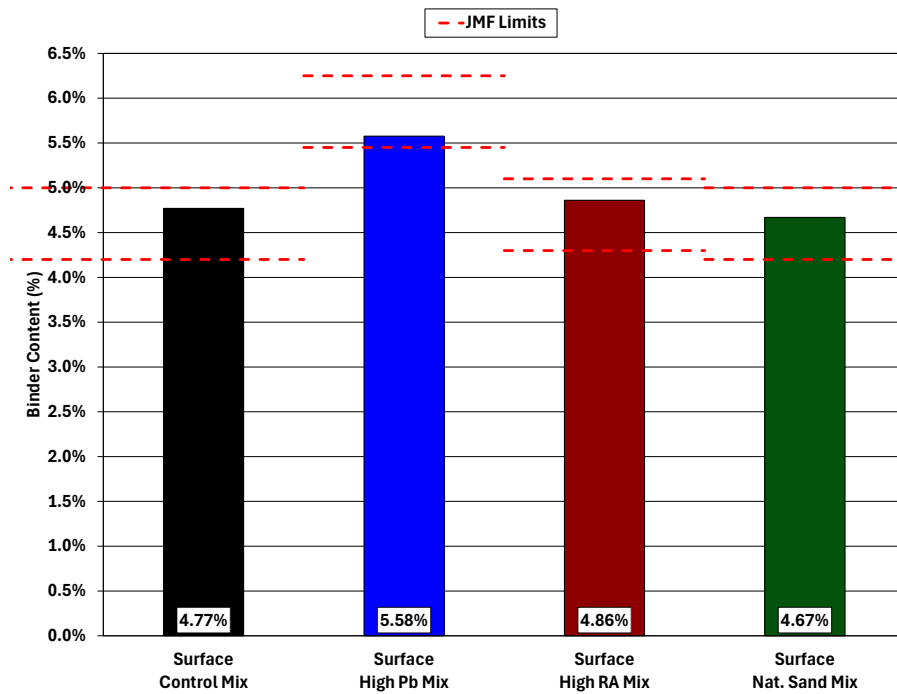


Figure 4.9: Air-voids at Ndesign (85 gyrations) from hot drop (JMF verification).



Note: Binder content was determined with automated solvent extraction (ASTM D8159-19).

Figure 4.10: Binder content (TMM) for surface mixes from hot drop (JMF verification).

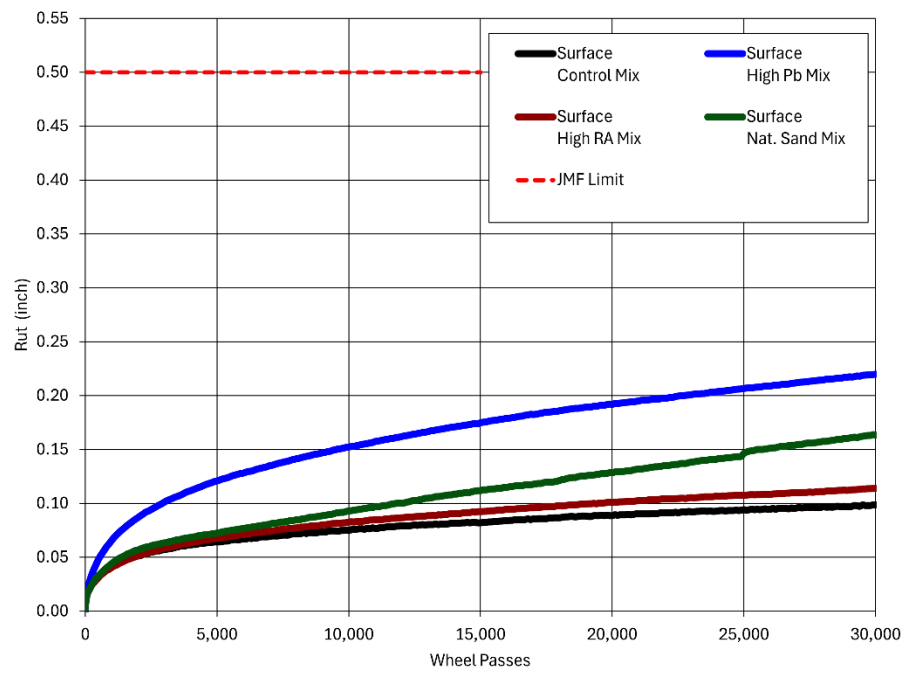


Figure 4.11: HWT tests results from hot drop: rutting versus wheel passes (JMF verification).

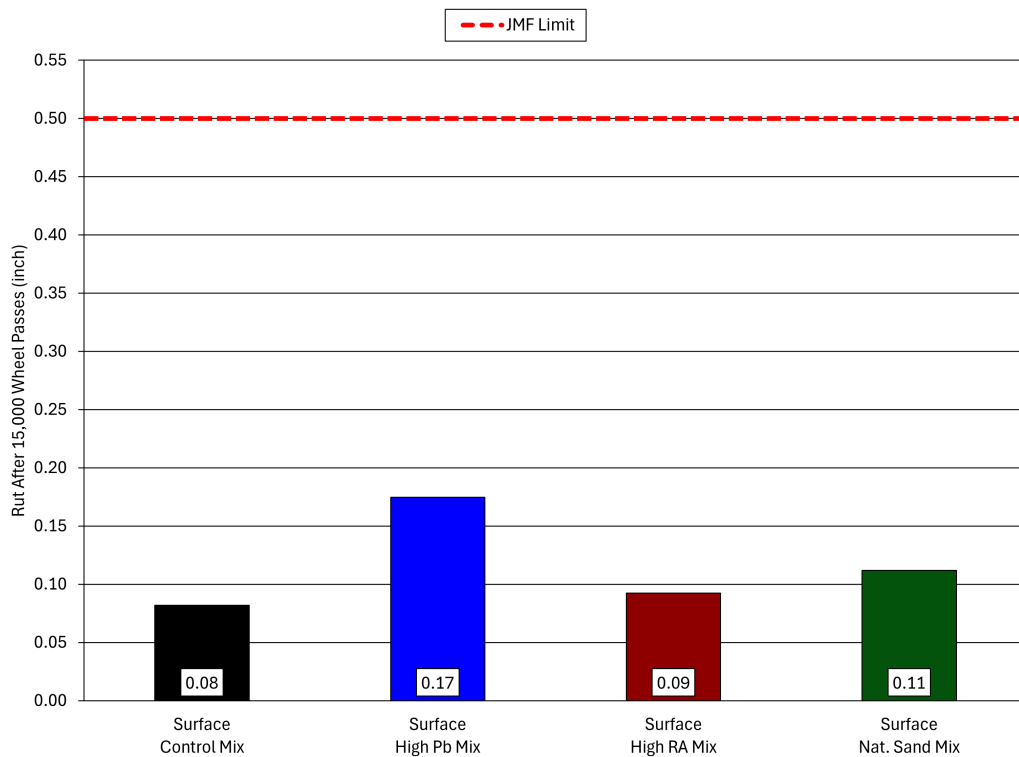


Figure 4.12: HWT tests results from hot drop: rutting after 15,000 wheel passes (JMF verification).

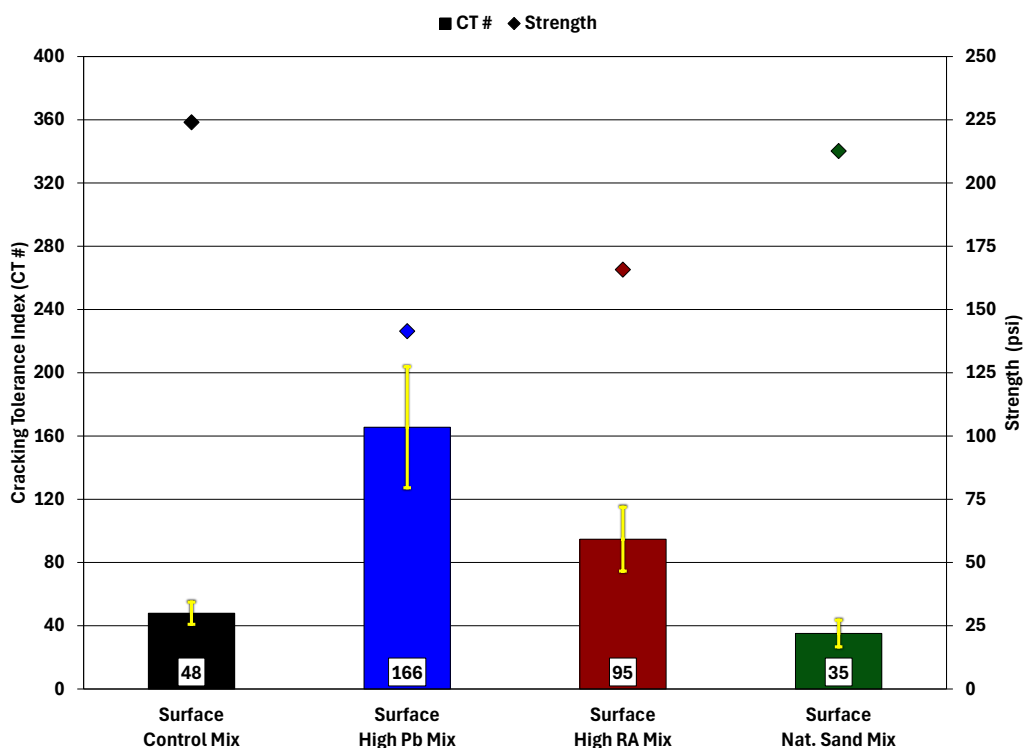


Figure 4.13: IDEAL-CT tests results from hot drop (JMF verification).

4.4 Estimation of the High Performance Grade of the Mix Binders

RAP was collected from the George Reed Inc. Clements plant stockpile and subjected to solvent extraction and recovery to determine the asphalt content and PG grade. The extraction was conducted following ASTM D8159-19 (automated extraction) and the recovery was conducted following ASTM D5404-24 (rotary evaporator). The binder content of the RAP, based on the extraction, was 4.92% by total mass of mix (TMM). The high PG (PGH) of the recovered binder was 108.8°C, the intermediate PG (PGI) was 51.4°C, and the low PG (PGL) was -1.1°C (AASHTO M 320-23).

The virgin binder used for the fabrication of all the test track mixes (rich bottom, intermediate, and the four surface layer mixes) was also graded, which resulted in a PGH of 63.4°C, PGI of 17.1°C, and PGL of 31.2°C. Note that while the virgin binder was marked as PG 58-22, it actually graded as PG 58-28.

The PGH of the RAP binder and the PGH of the virgin binder were used for estimating the PGH of the binder blends of the different mixes, following the standard blending chart approach. The standard blending chart approach assumes that the PG of the binder blend is the mass-weighted average of the PG of the components. Three of the mixes—the surface high RA, the intermediate mix, and the rich bottom mix—contained tall oil-based recycling agent (RA). For those mixes, it was assumed that each 1% of RA resulted in a 4°C drop of the PGH, where the RA percentage is measured versus the total binder plus recycling agent (i.e., $RA(\%) = \frac{RA \text{ mass}}{\text{virgin binder mass} + \text{RAP binder mass} + \text{RA mass}}$). The PGH estimation results are presented in Table 4.3.

Table 4.3: Estimation of High PG (PGH) of the Mix Binders

Parameters	Surface Control Mix	Surface High Pb Mix	Surface High RA Mix	Surface Nat. Sand Mix	Intermediate Mix	Rich Bottom Mix
Virgin binder true PGH, °C	63.4	63.4	63.4	63.4	63.4	63.4
RAP binder true PGH, °C	108.8	108.8	108.8	108.8	108.8	108.8
% RAP by aggregate replacement	25	25	25	25	40	40
% RAP binder in total binder	27.5	21.2	26.8	27.5	36.3	35.6
Estimated true PGH without RA	75.9	73.0	75.6	75.9	79.9	79.6
RA (% by total binder weight)	No	No	Yes (2.5%)	No	Yes (2.5%)	Yes (2.5%)
Expected true PGH with RA	75.9	73.0	65.6	75.9	69.9	69.6

5 ASPHALT PAVEMENT CONSTRUCTION

5.1 Construction Overview

The test track was paved by George Reed Construction on November 6, 2024 (rich bottom and intermediate layers) and November 7, 2024 (surface layers). All mixes were produced by George Reed Inc. at the Clements batch plant, stored in a silo for less than four hours, and then trucked 59 miles to the construction site in Davis.

The following equipment was used during the construction:

- Paving: Caterpillar Tracked Paver AP1055 (around 45,000 lb. weight)
- Primary Compaction: Caterpillar Steel Double Drum Roller CB10 (around 21,000 lb. weight)
- Secondary Compaction (Standby): Caterpillar Steel Double Drum Roller CB10 (around 21,000 lb. weight)
- Finish Compaction: Caterpillar Steel Double Drum Roller CB2.7 (around 5,900 lb. weight)
- Intelligent Compaction: G4 Technologies RTDensity System
- Trucking: DoubleM Trucking (double bottoms)

Construction is shown in Figure 5.1 to Figure 5.16, asphalt mix paving temperatures are summarized in APPENDIX A ASPHALT MIX PAVING TEMPERATURE AND SEQUENCE, and the outcome of the asphalt construction intelligent compaction monitoring is shown in APPENDIX B ASPHALT CONSTRUCTION INTELLIGENT COMPACTION.



Figure 5.1: Instrumentation on primed aggregate base.



Figure 5.3: Intermediate layer lift 1 (0.2 ft.) paving.



Figure 5.2: Asphalt paver and truck on primed aggregate base.



Figure 5.4: Double steel drum roller (10 ton).



Figure 5.5: Tack coat application by hand sprayer.



Figure 5.7: Nuclear gauge testing.



Figure 5.6: Tack coat application by truck sprayer.



Figure 5.8: Ice on coring locations in Lane 1 and Lane 2.



Figure 5.9: Coring on Lane 1.



Figure 5.11: Line striping on test track.



Figure 5.10: Test track after construction.



Figure 5.12: Test track after line striping.



Figure 5.13: Rutting evaluation section layout.



Figure 5.15: Caltrans, industry, and UCPRC staff.



Figure 5.14: HVSSs on Lane 1 and Lane 3 rutting evaluation sections.



Figure 5.16: George Reed Inc. paving crew and UCPRC staff.

5.1.1 Intermediate and Rich Bottom Layers

The mix temperature was measured twice per mix at the plant (Table 5.1).

Table 5.1: Plant Production Temperatures for Intermediate and Rich Bottom Mixes

Measurement	Intermediate (°F)	Rich Bottom (°F)
First	307	304
Second	318	309

The rich bottom mix was paved in a single lift (0.2 ft. thick) while the intermediate layer was paved in two lifts (each 0.2 ft. thick).

The mix temperature measured during the construction is summarized in Table 5.2, and examples of infrared thermal camera images are shown in Figure 5.17 and Figure 5.18. As shown in Table 5.2, the asphalt mix temperature was always within the desirable range for the type of mixes being used.

Table 5.2: Paving Temperatures for Intermediate and Rich Bottom Mixes

Mix and Lift	Time of Paving	Ambient Temperature (°F)	Mix Temperature at Truck (°F)	Mix Temperature Behind Screed (°F)
Rich bottom	8:17 am – 10:07 am	55-65	300-330	260-305
Intermediate, lift 1	12:01 pm – 1:16 pm	65-75	300-330	260-305
Intermediate, lift 2	2:07 pm – 4:35 pm	70-75	300-330	260-305

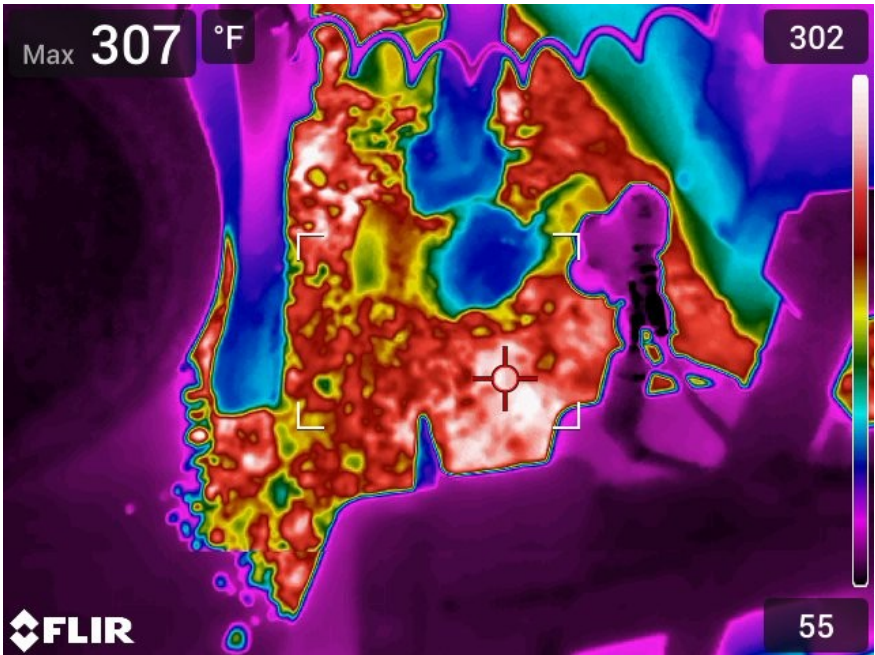


Figure 5.17: Rich bottom mixes through the asphalt paver wing.

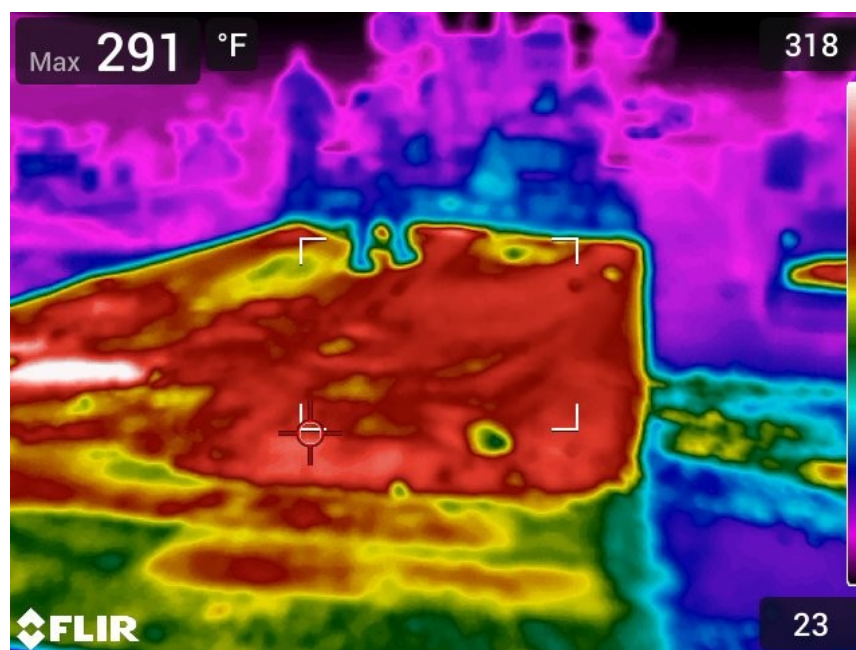


Figure 5.18: Rich bottom mix behind the paver screed.

5.1.2 Surface Layers

The mix temperature was measured twice per mix at the plant (Table 5.3).

Table 5.3: Plant Production Temperatures for Intermediate and Rich Bottom Mixes

Measurement	Control (°F)	High Pb (°F)	High RA (°F)	Nat. Sand (°F)
First	292	316	311	314
Second	301	293	294	300

The surface layer of each of the four lanes was paved in two lifts (each 0.2 ft. thick).

The mix temperature measured during the construction is summarized in Table 5.4. As shown in the table, the asphalt mix temperature was always within the desirable range for the type of mixes being used.

Table 5.4: Paving Temperatures for Surface Mixes

Mix and Lift	Time of Paving	Ambient Temperature (°F)	Mix Temperature at Truck (°F)	Mix Temperature Behind Screed (°F)
Control, lift 1	12:14 pm – 12:26 pm	~64	316-320	282-319
Control, lift 2	1:33 pm – 1:49 pm	~64	312-331	275-305
High Pb, lift 1	11:50 am – 12:00 pm	~64	320-320	247-300
High Pb, lift 2	12:56 pm – 1:16 pm	~64	306-320	278-309
High RA, lift 1	8:27 am – 8:41 am	~49	313-320	275-300
High RA, lift 2	9:53 am – 10:20 am	~49	312-320	283-304
Nat. Sand, lift 1	7:59 am – 8:16 am	~49	315-320	269-294
Nat. Sand, lift 2	9:16 am – 9:33 am	~49	310-320	271-297

5.2 Quality Control/Quality Assurance Testing Results

The goal of the QC/QA process was to evaluate the density of the mixes in the track and to compare the actual mixes used in the test track construction with the intended mix designs included in Chapter 4.

The QC/QA process included the following:

- In-place density based on cores extracted after construction (Section 5.2.1).
- Volumetric properties of the mixes (Section 5.2.2), including the comparison with the mix design values included in Table 4.1 and Table 4.2.
- Performance-related properties of the mixes (Section 5.2.3), including the comparison with the PRT results of the pre-construction hot drop mixes included in Section 4.2 and Section 4.3.

Each of the six mixes was sampled twice during production, with each sampling referred to as Lot V1 or Lot V2. The sampled mixes were used for volumetrics and PRT. Additionally, 18 cores were extracted after construction to measure the binder content, binder PG, and some rutting performance-related properties of the field-compacted mixes (Section 5.2.4).

5.2.1 In-Place Density of the Mixes

One core was extracted from each of the four lanes after construction. Then the different lifts were cut and the density was determined. The relative density results are summarized in Table 5.5. Two relative density values are included in the table: (1) relative density values based on the maximum density determined by the George Reed laboratory during construction and (2) relative density values (in parentheses) based on the maximum density determined by the UCPRC (Table 5.6, right-hand column). The maximum density values determined by the UCPRC are included for each of the mixes in Table 5.6. As shown in Table 5.5, the field density of the mixes was, overall, within the required range.

Table 5.5: In-Place Density from Cores

Mix	Requirement (%)	Lane 1 (Surface Is Control Mix) (%)	Lane 2 (Surface Is High Pb Mix) (%)	Lane 3 (Surface Is High RA) (%)	Lane 4 (Surface Is Nat. Sand Mix) (%)
Surface, lift 2	91-97	92.7 (92.0)	92.6 (92.8)	97.1 (95.3)	91.8 (91.3)
Surface, lift 1		94.8 (94.1)	94.6 (94.8)	98.5 (96.6)	93.4 (92.9)
Intermediate, lift 2	94-97	97.7 (97.9)	97.6 (97.8)	96.5 (96.7)	96.2 (96.5)
Intermediate, lift 1		95.7 (95.9)	95.4 (95.6)	95.4 (95.6)	95.9 (96.1)
Rich bottom, lift 1	97-100	98.4 (98.0)	97.2 (96.8)	98.6 (98.1)	98.7 (98.2)

Notes: Reported value is based on maximum density of the mixes (Gmm) from George Reed laboratory. Value in parentheses is based on Gmm determined by the UCPRC after the construction.

Table 5.6: Maximum Density of the Mixes (Gmm)

Mix	Pre-Construction Value by UCPRC (JMF Verification)	Post-Construction Value by UCPRC (Production Mixes)
Surface, Control	2.594	2.598
Surface, High P	2.551	2.577
Surface, High RA	2.576	2.606
Surface, Nat. Sand	2.577	2.597
Intermediate	2.540	2.552
Rich bottom	2.566	2.552

5.2.2 Volumetric Properties of the Mixes

The volumetric properties of the mixes sampled during the construction (“production” mixes, Lot V1 and Lot V2) were compared with the properties of the pre-construction hot drop verification mixes (“JMF verification” mixes). The comparison is shown in Figure 5.19 (air-voids [AV] at 85 gyrations) and Figure 5.20 (asphalt binder content). In the two figures, the mix design requirements (previously shown in Table 4.1 and Table 4.2) are also included. As shown in Figure 5.19 and Figure 5.20, the air-voids at 85 gyrations and the binder content of the production mixes were within the JMF limits.

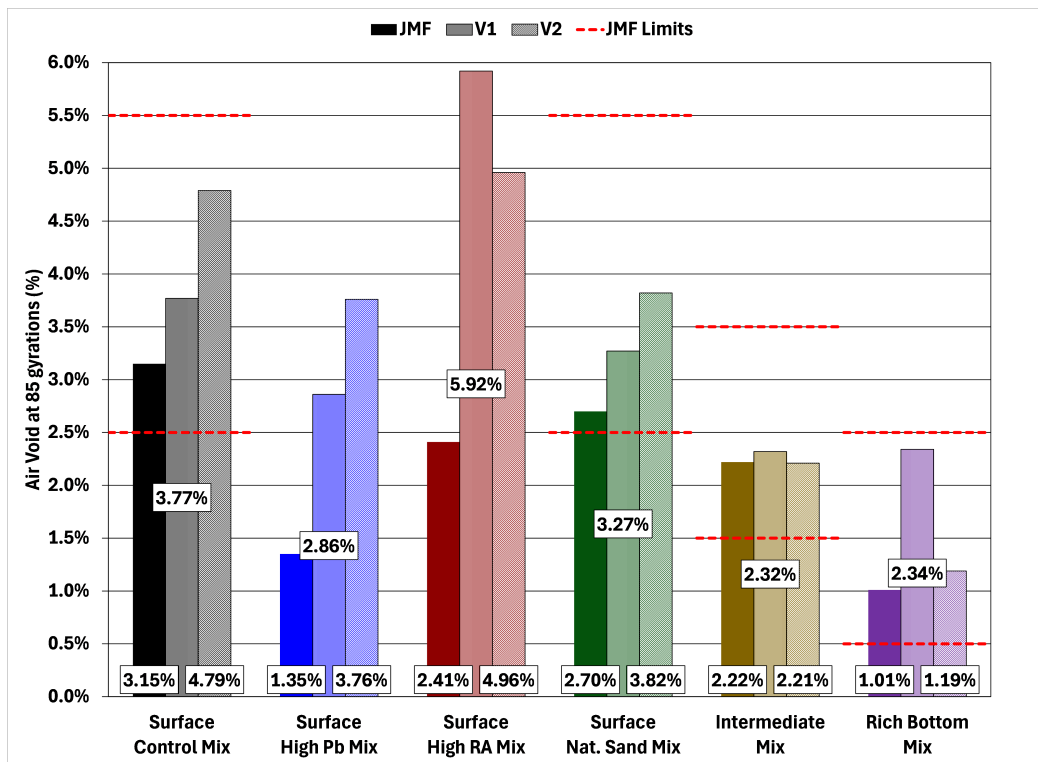
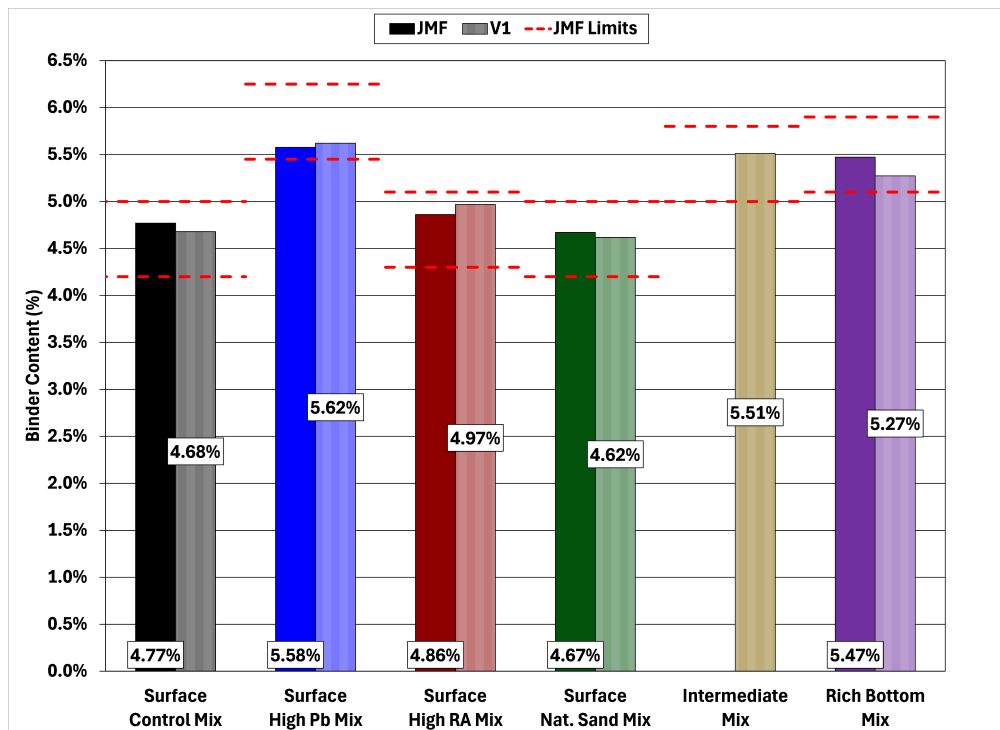


Figure 5.19: Air-voids at Ndesign (85 gyrations): production versus JMF verification mixes.

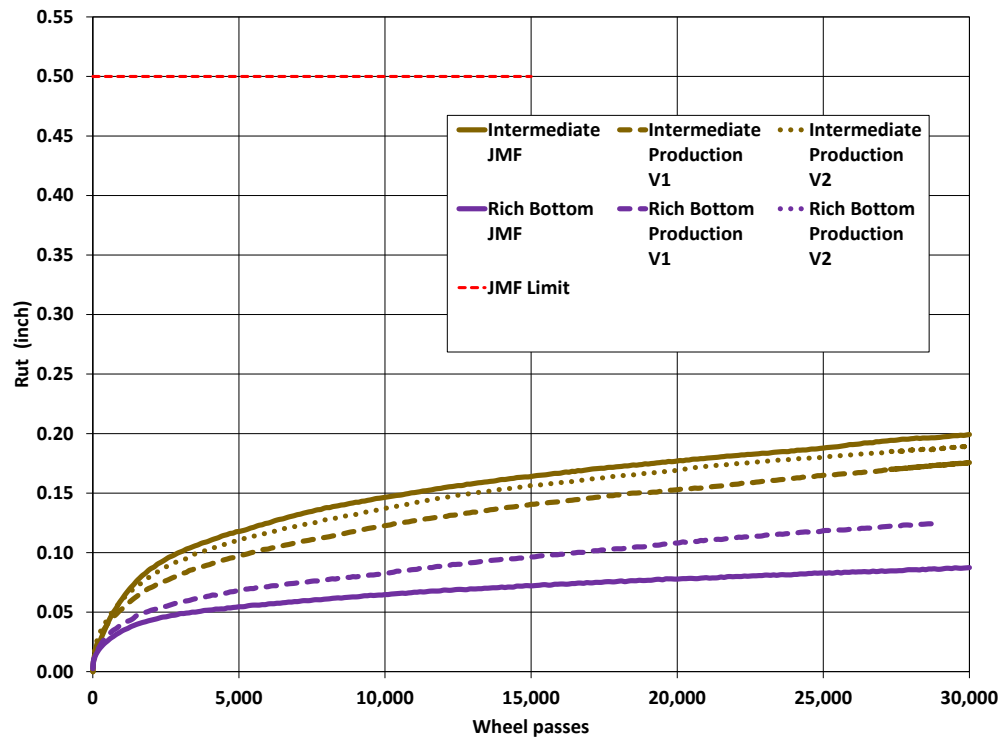


Notes: Binder content was determined with automated solvent extraction (ASTM D8159-19). Binder determination for intermediate mix JMF is not available.

Figure 5.20: Binder content (total mass of mix [TMM]): production versus JMF verification mixes.

5.2.3 Performance-Related Properties of the Mixes

The performance-related properties of the mixes sampled during the construction (“production” mixes, Lot V1 and Lot V2) were compared with the properties of the pre-construction hot drop verification mixes (“JMF verification” mixes). The comparison is shown in Figure 5.21 to Figure 5.23 (HWT), Figure 5.24 (IDEAL-CT), Figure 5.25 (4PB fatigue resistance), and Figure 5.26 (4PB stiffness). In all the figures, the mix design requirements, when applicable, are included as well. As shown in Figure 5.21 to Figure 5.23, the mixes met the HWT test rutting requirements. As shown in Figure 5.25, the mixes met the 4PB fatigue resistance requirements. However, as shown in Figure 5.26, the 4PB stiffness of the intermediate layer mix was somewhat below the specification limits.



Note: HWT testing of second production lot (Lot V2) of the rich bottom mix is not available.

Figure 5.21: HWT test results, rutting versus wheel passes (intermediate and rich bottom mixes): production versus JMF verification.

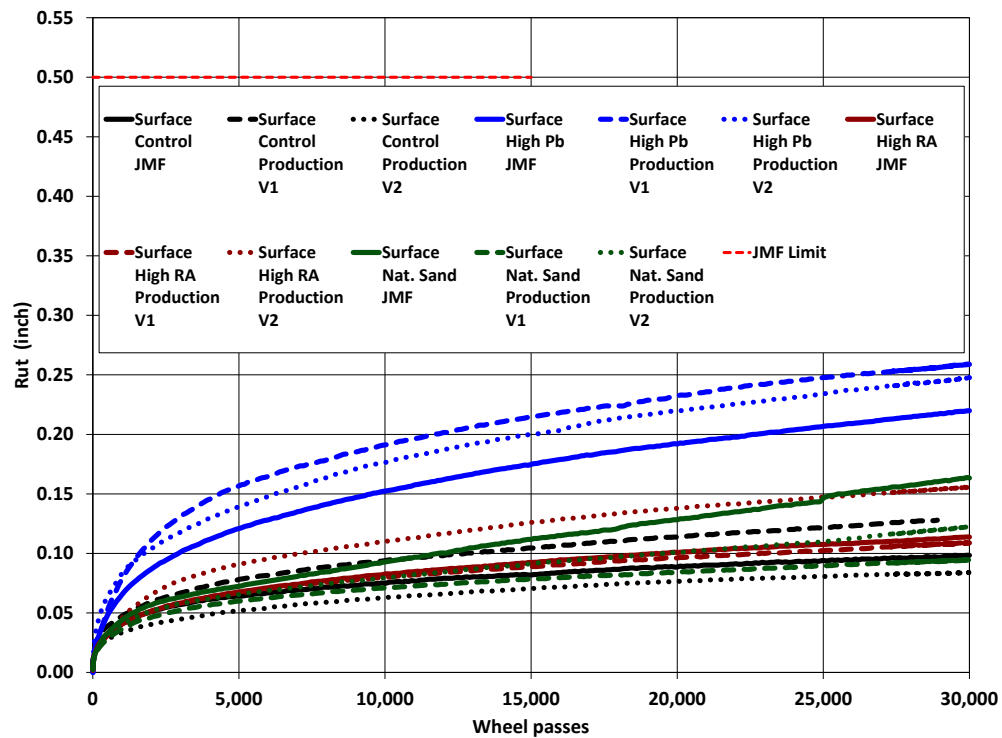
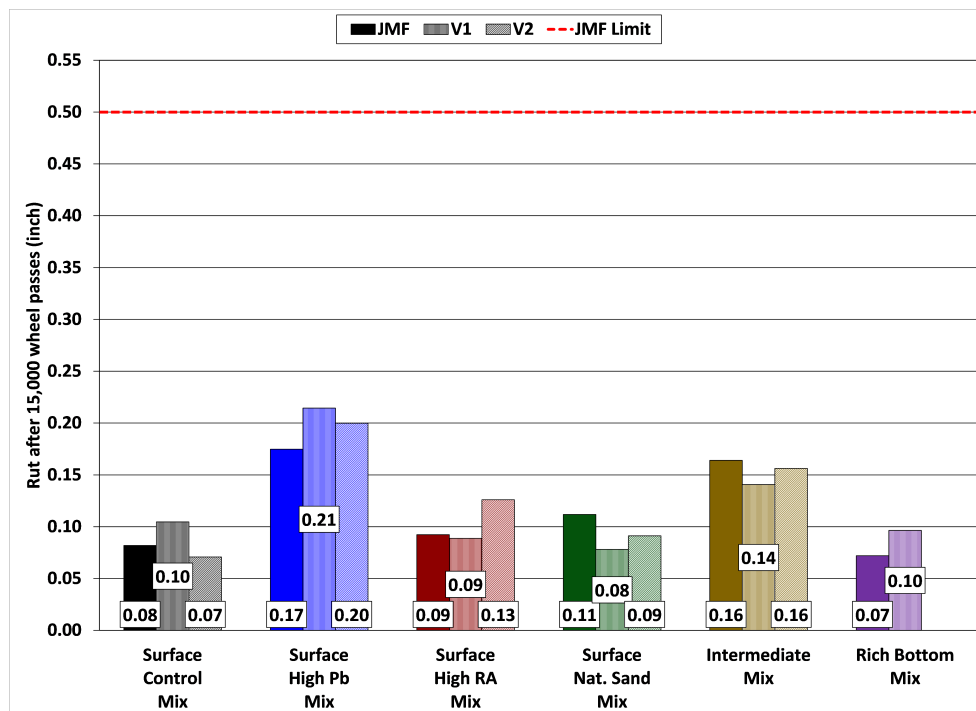


Figure 5.22: HWT test results, rutting versus wheel passes (surface mixes): production versus JMF verification.



Note: HWT testing of second production lot (Lot V2) of the rich bottom mix is not available.

Figure 5.23: HWT test results, rutting after 15,000 wheel passes: production versus JMF verification mixes.

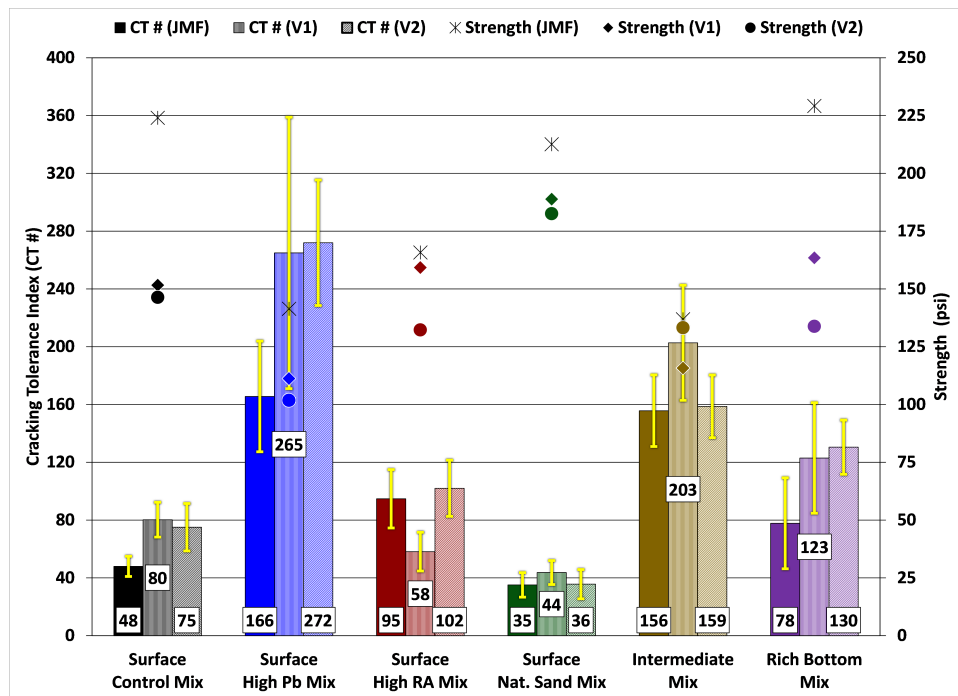


Figure 5.24: IDEAL-CT test results: production versus JMF verification mixes.

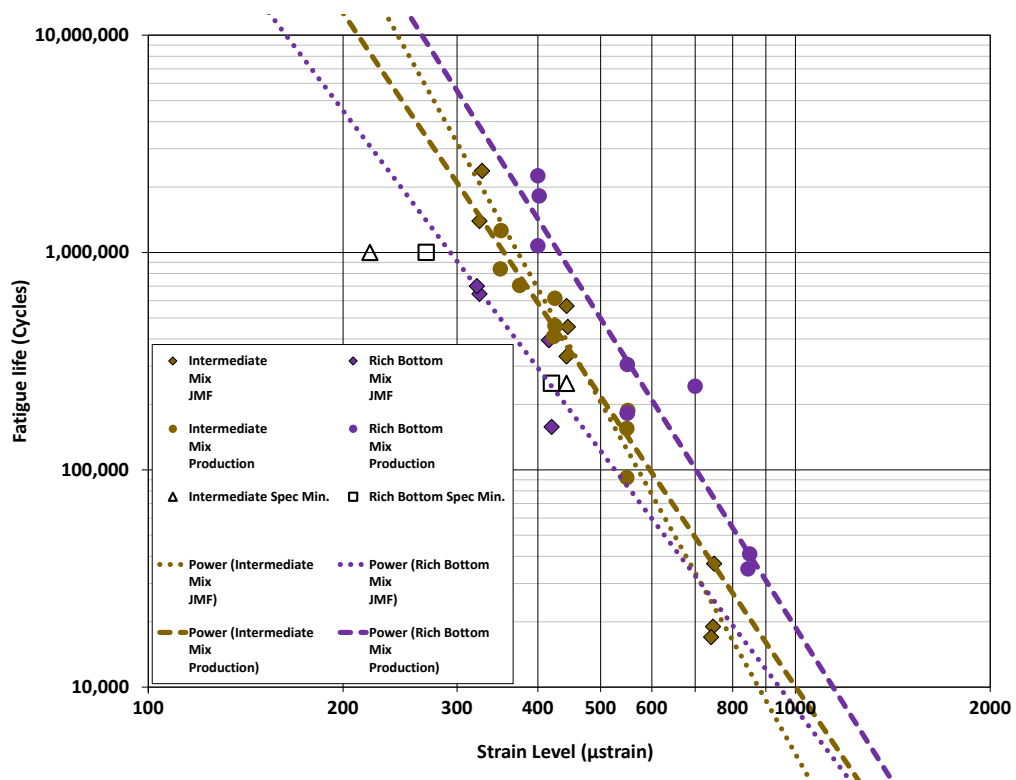


Figure 5.25: 4PB beam fatigue resistance test results: production versus JMF verification mixes.

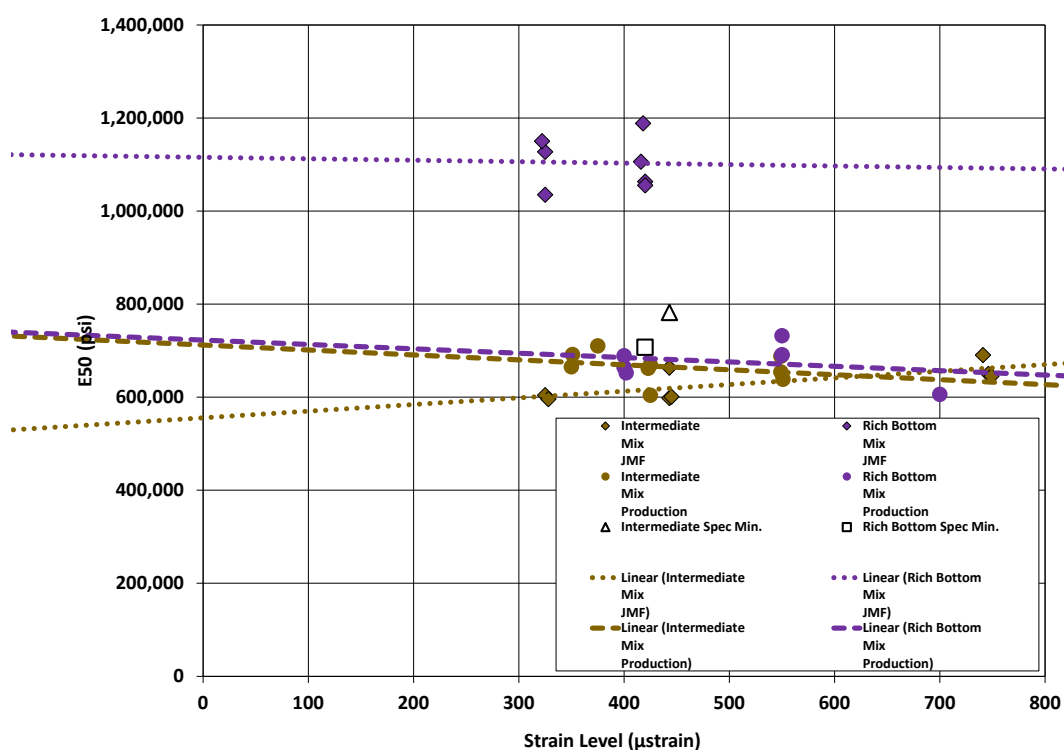


Figure 5.26: 4PB beam stiffness at cycle 50: production versus JMF verification mixes.

5.2.4 Field-Compacted Mix Evaluation

Eighteen cores were extracted from two locations (A and B) of Lane 1 after construction to measure binder content, binder PG, and some rutting performance-related properties of the field-compacted mixes. The cores were taken from Lane 1 to evaluate the properties of the control surface mix. Since the cores were taken to full depth, the intermediate layer could also be evaluated.

The binder was extracted following ASTM D8159-19 (automated extraction) and then recovered following ASTM D5404-24 (rotary evaporator). The recovered binder was PG graded (AASHTO M320-23). The binder content and the PGH of the recovered binder are presented in Table 5.7. The table also includes the design binder content and the PGH estimated following the standard blending chart approach (Table 4.3).

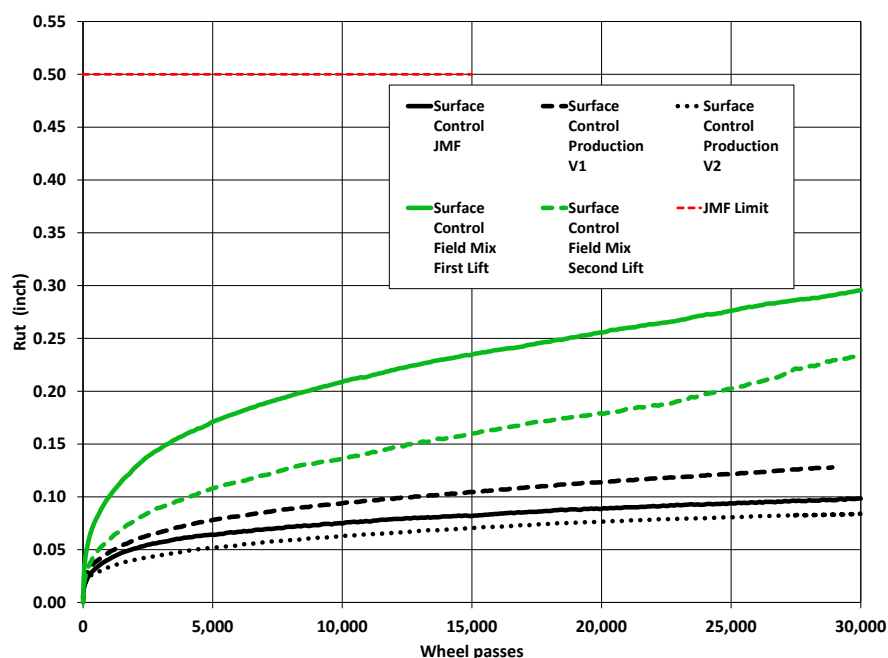
Table 5.7: Summary of Field Mix Binder Content and PGH (Cores, Lane 1)

Mix	Binder Content TMM (%)		Design Binder Content TMM (%)	Extracted Binder, PGH		Estimated PGH (Blending Chart) (°C)
	Loc. A	Loc. B		Loc. A	Loc. B	
Surface Control, lift 2	4.6	4.7	4.5 (4.2-5.0)	72.8	71.3	75.9
Surface Control, lift 1	4.7	4.7		72.3	69.2	
Intermediate, lift 2	5.8	5.9	5.4 (5.0-5.8)	69.7	67.3	69.9
Intermediate, lift 1	5.9	5.8		66.7	66.4	

As shown in Table 5.7, the two lifts of the control surface layer at the two locations presented similar binder content and PGH, as well as the two lifts of the intermediate layer. These outcomes indicate low construction variability. The binder content of the control surface layer lifts was very close to the design value of 4.5%. However, the binder content of the two intermediate layer lifts was in the upper range of the allowable binder content, indicating that the intermediate layer mix may have been fabricated with a binder content above the desired range. Regarding the PGH, the values determined for the binder recovered from the control surface layer mix were considerably lower (3°C to 7°C lower) than the 75.9°C estimated based on the blending chart. For the intermediate layer mix, the PGH of the recovered binders was close to the 69.9°C estimated based on the blending chart.

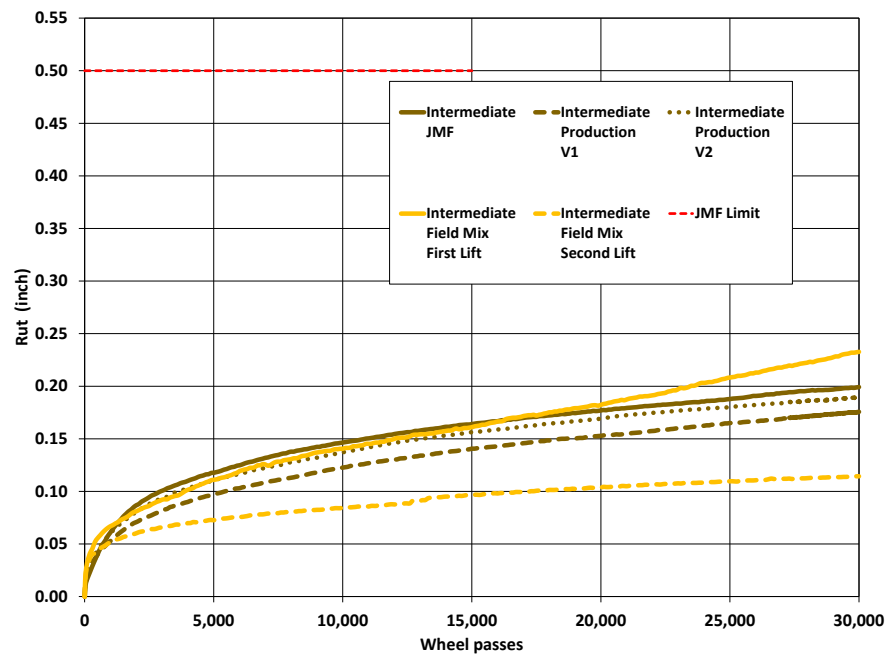
The cores extracted from Lane 1 also served for the preparation of specimens for HWT testing. The HWT test results were compared with the results of the testing conducted on specimens prepared from JMF verification and the production mixes (previously presented in Figure 5.22). The comparison is shown in Figure 5.27 (surface control mix) and Figure 5.28 (intermediate layer lift). When comparing laboratory-compacted with field-compacted results, the fact that the air-void content differed between the two sets of specimens must be considered.

Overall, the field-compacted mix specimens resulted in rutting similar to the laboratory-compacted mix specimens. The only exception is the first lift of the field-compacted control surface mix, which resulted in much more rutting than the laboratory-compacted specimens. This outcome is due, at least in part, to the high air-void content, 9.4% to 10.4%, of the corresponding field-compacted specimens.



Note: The air-voids of the laboratory-compacted specimens (JMF verification and production mixes) was $7 \pm 0.5\%$, while the air-voids of the field-compacted specimens was 9.4% to 10.4% for the first lift and 3.9% to 8.4% for the second lift.

Figure 5.27: HWT test results, surface control mix: field-compacted specimens.



Note: The air-voids of the laboratory-compacted specimens (JMF verification and production mixes) was $6 \pm 0.5\%$ while the air-voids of the field-compacted specimens was 2.0% to 5.2% for the first lift and 2.3% to 3.3% for the second lift.

Figure 5.28: HWT test results, intermediate layer mix: field-compacted specimens

6 POST-CONSTRUCTION DEFLECTION TESTING

The baseline properties of the pavement were measured using non-destructive testing with the falling weight deflectometer (FWD). The main questions that need to be answered using the FWD testing are the following:

- Are the sections uniform (both within each test section and between different test sections)?
- What are the backcalculated stiffnesses for the different layers?

6.1 Testing Plan

The FWD testing was performed systematically across all four lanes to assess pavement response prior to HVS testing. Each HVS test section was 2 ft. wide and 26.2 ft. (8.0 m) long (the HVS test sections are shown as “Test strips” in Figure 2.1). The testing was conducted both along the centerline of each HVS test section and at an offset of 4.5 ft. from the centerline. This dual-position FWD testing ensured a comprehensive dataset for comparisons between pre- and post-HVS conditions. The layout of the FWD testing is illustrated in Figure 6.1, which shows 1.7 ft. (0.5 m) spacing within the longitudinal range of the test section and 4.5 ft. spacing in the transverse direction.

The FWD testing was conducted twice for each lane along both lines, once in the morning and again in the afternoon, to capture the effect of temperature on pavement layer stiffness. The testing was completed on two days, January 23–24, 2025. Three drop heights were used, with resulting loads of approximately 8 kips, 10 kips, and 12 kips.

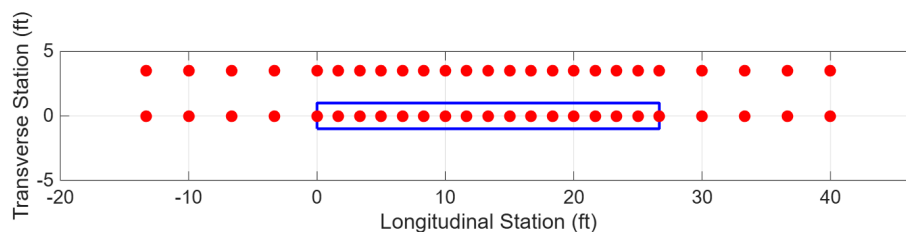


Figure 6.1: Layout of FWD testing (the inner rectangle indicates HVS test section).

6.2 Test Results

The boxplots of surface temperature, deflection measured at the center of the loading plate (D1), and deflection measured at 4.0 ft. away from the center of the loading plate (D7) are shown in Figure 6.2 to Figure 6.4. As shown in Figure 6.2, the recorded surface temperatures ranged from 41°F to 77°F across all sections during the FWD testing.

As anticipated, the data reveal a notable impact of surface temperature on the measured deflections, even for the D7 sensor, which was far away from the loading plate and primarily measuring deflection in the subgrade. Therefore, it is critical to account for this effect when comparing results from different lanes.

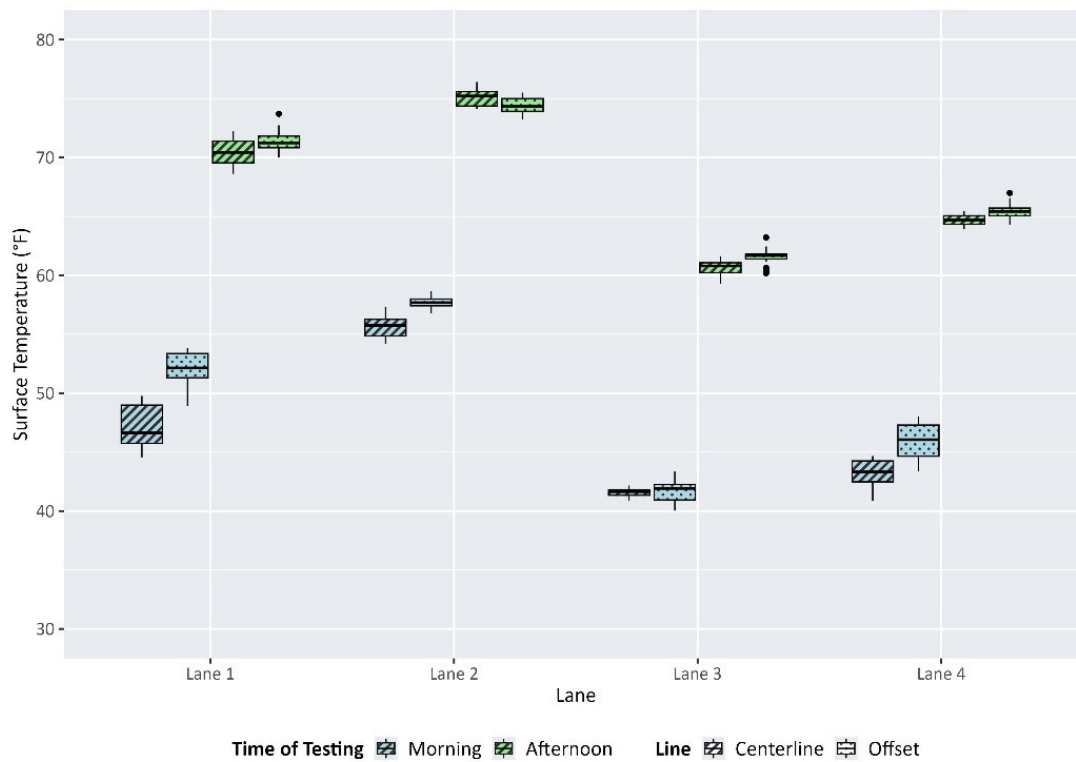


Figure 6.2. Surface temperatures during FWD testing.

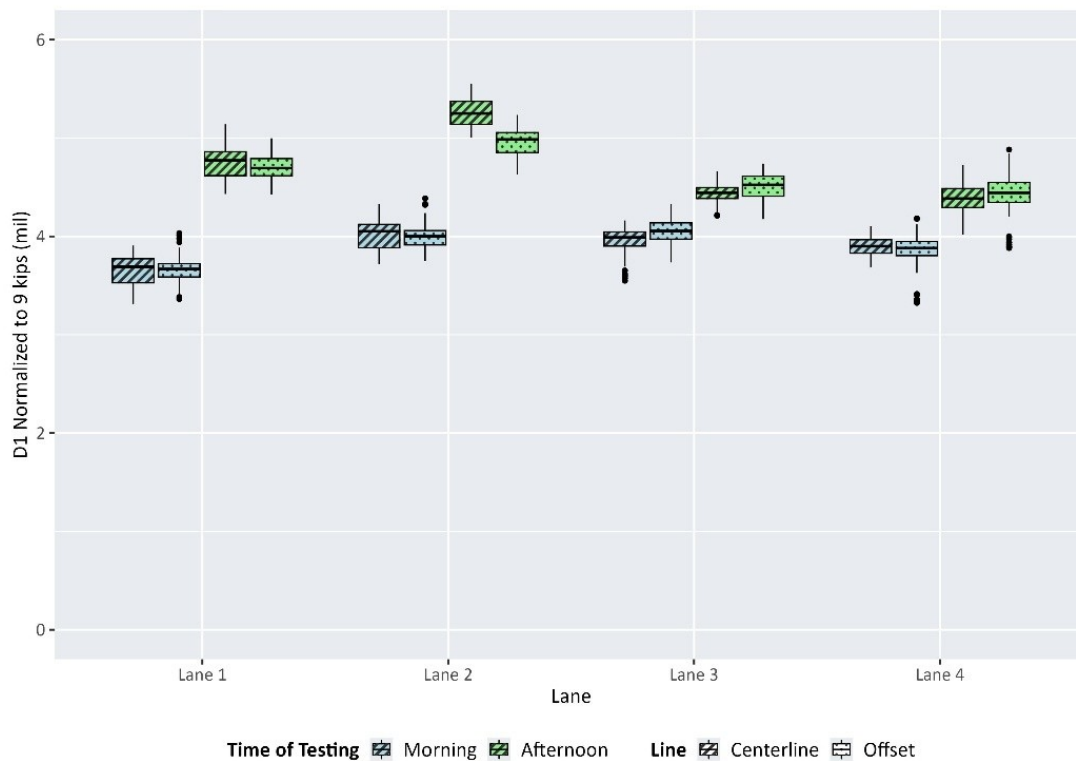


Figure 6.3. Deflection at center of the loading plate (D1) during FWD testing.

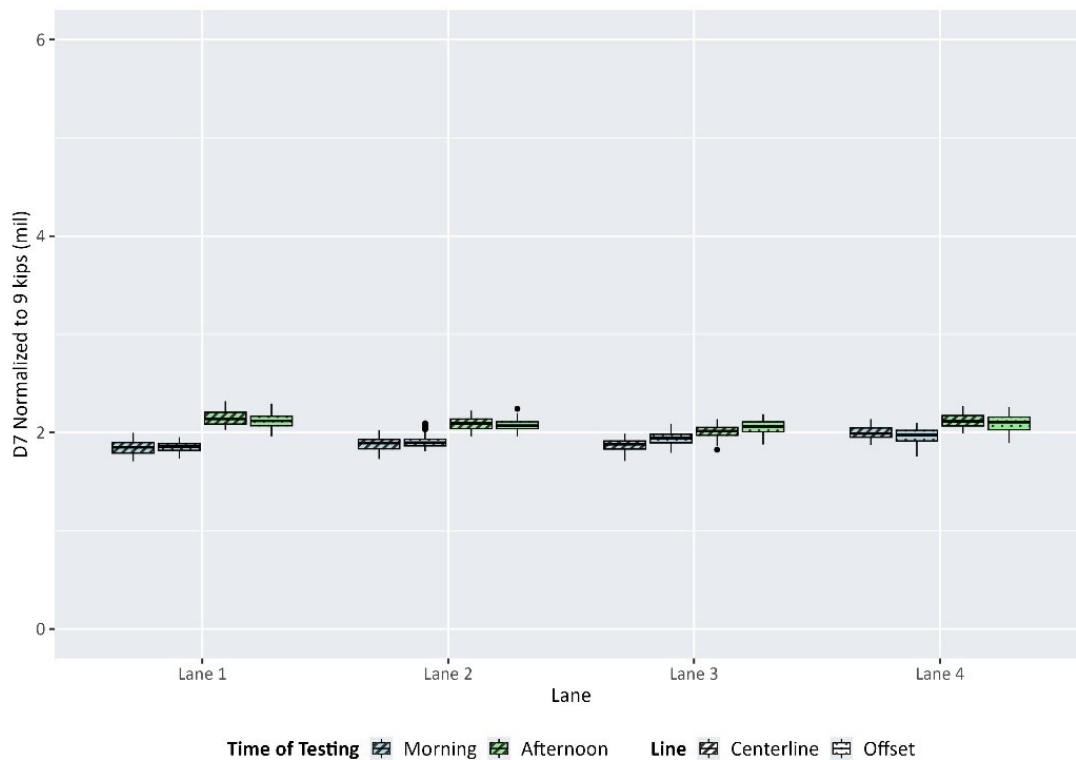


Figure 6.4. Deflection at 4 ft. (1.2 m) distance (D7) during FWD testing.

6.3 Analysis

The analysis procedure included running layer stiffness backcalculation, calculating pavement representative in-depth temperature, and normalizing the backcalculation results to reference temperature. These steps and the results are explained in the following discussion.

6.3.1 Layer Stiffness Backcalculation

A MATLAB script developed in-house was used to conduct layer stiffness backcalculation. Odemark's equivalent thickness method was selected for estimating surface displacements. Data from all eight sensors were used, with the outermost one placed 5 ft. away from the center of the loading plate. The structures are simplified into three layers: 1.0 ft. of combined AC layer, 1.0 ft. of aggregate base (AB), and the subgrade (SG). The backcalculated layer stiffness for all lanes is shown in Figure 6.5. The figure suggests that the backcalculated layer stiffnesses are approximately 1,450 ksi for the combined AC layer, 87 ksi for the AB layer, and 26 ksi for the subgrade. The unusually high stiffness of the AB layer is likely caused by the high thickness and relatively high stiffness of the AC layer, which together provide very strong confinement and lead to low stress in the aggregate base. The AB material exhibited higher stiffness at lower stress.

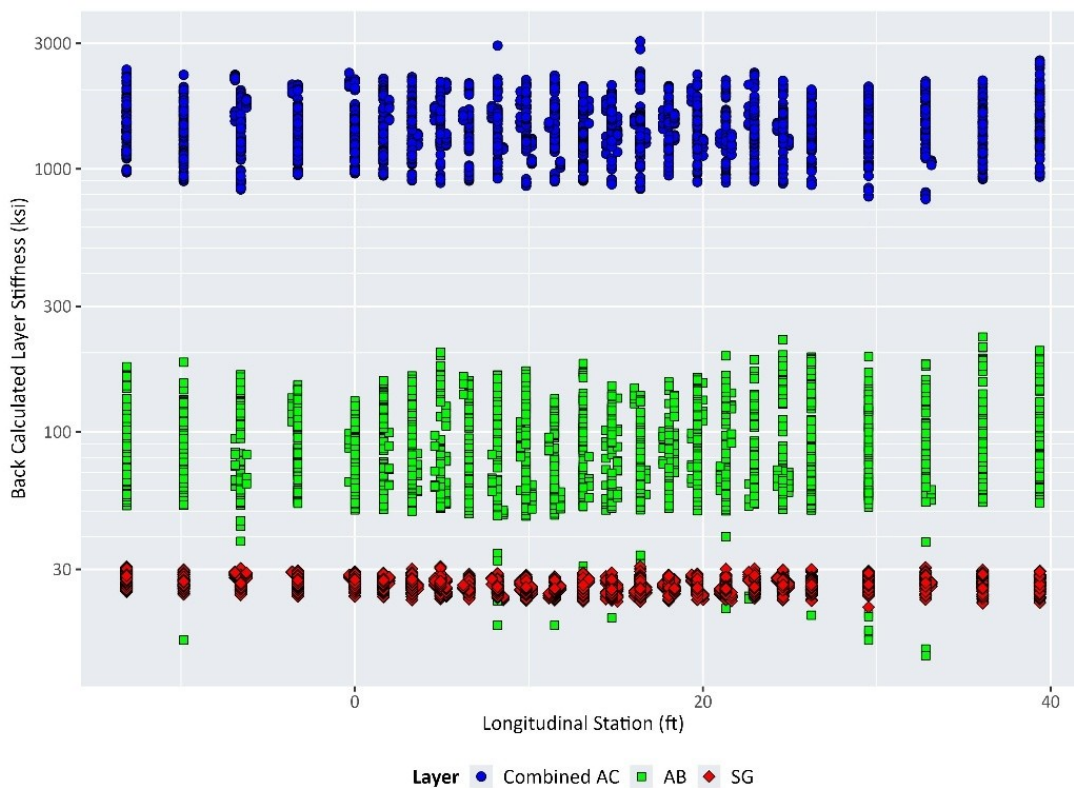


Figure 6.5. Variation of backcalculated layer stiffness for all layers and all lanes.

6.3.2 Adjusting Backcalculation Results for Pavement Temperature

As previously shown, pavement temperatures during FWD testing have significant effects on measured surface deflections, which in turn affect the backcalculated layer stiffnesses. It is therefore critical to adjust backcalculation results for pavement temperatures.

Pavement in-depth temperature was estimated using the BELLS3 equation, which is based on surface temperature and average air temperature from the previous day, depth, and time of testing. The surface temperatures were measured during FWD testing along with the deflection data. The average air temperature for the previous day was obtained for Davis, California⁴, and was found to be 45.0°F for January 22, 2025, and 47.5°F for January 23, 2025.

The temperature at one-third depth (i.e., 0.33 ft. depth) of the combined AC layer was used to represent the pavement temperature. A boxplot of this temperature is shown in Figure 6.6. Compared with the surface temperatures shown in Figure 6.2, the one-third depth temperatures have a narrower range, which is expected.

⁴ From Weather Underground (www.wunderground.com).

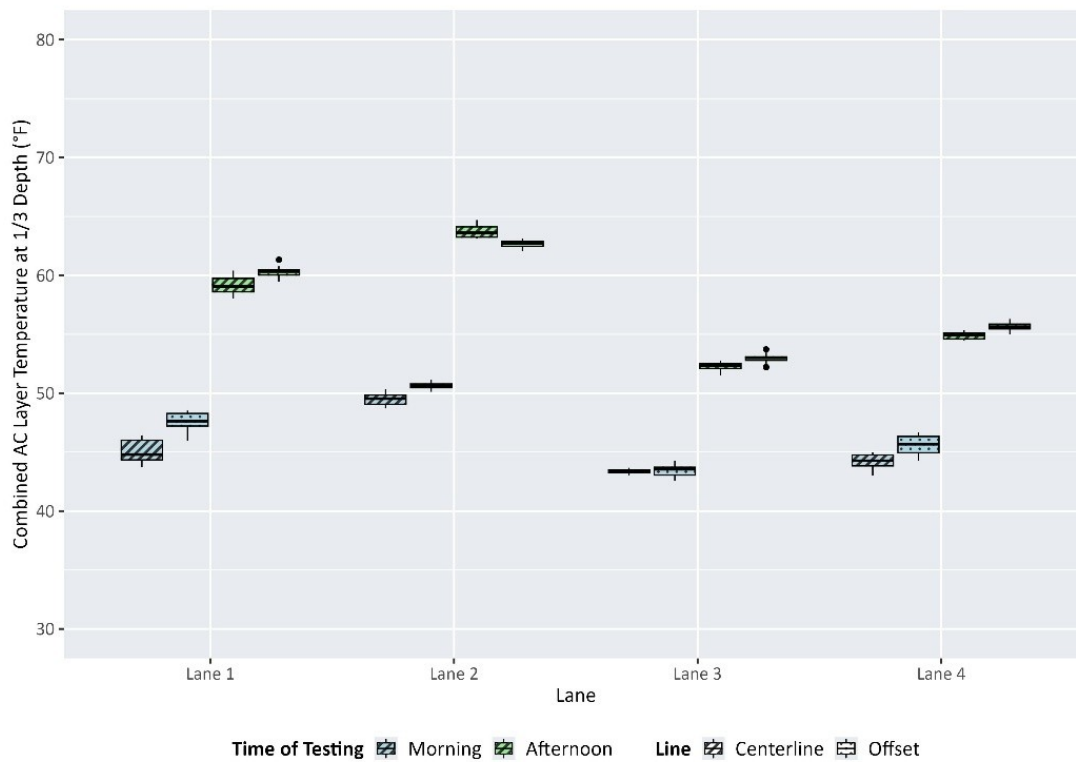


Figure 6.6. Boxplot of combined AC layer temperatures at one-third depth during FWD testing.

A linear correlation is assumed between pavement temperature and $\log(E)$, where E is the backcalculated layer stiffness. The correlation is developed individually for each lane and layer. To avoid extrapolation, a reference temperature of 50°F (10°C) was selected because it is covered by FWD testing on all lanes (see Figure 6.6). In other words, all backcalculated layer stiffness are adjusted to 50°F so that they are directly comparable.

6.3.3 Adjusted Backcalculated Layer Stiffness

The backcalculated layer stiffnesses, after being adjusted to 50°F, are summarized in Figure 6.7 to Figure 6.9.

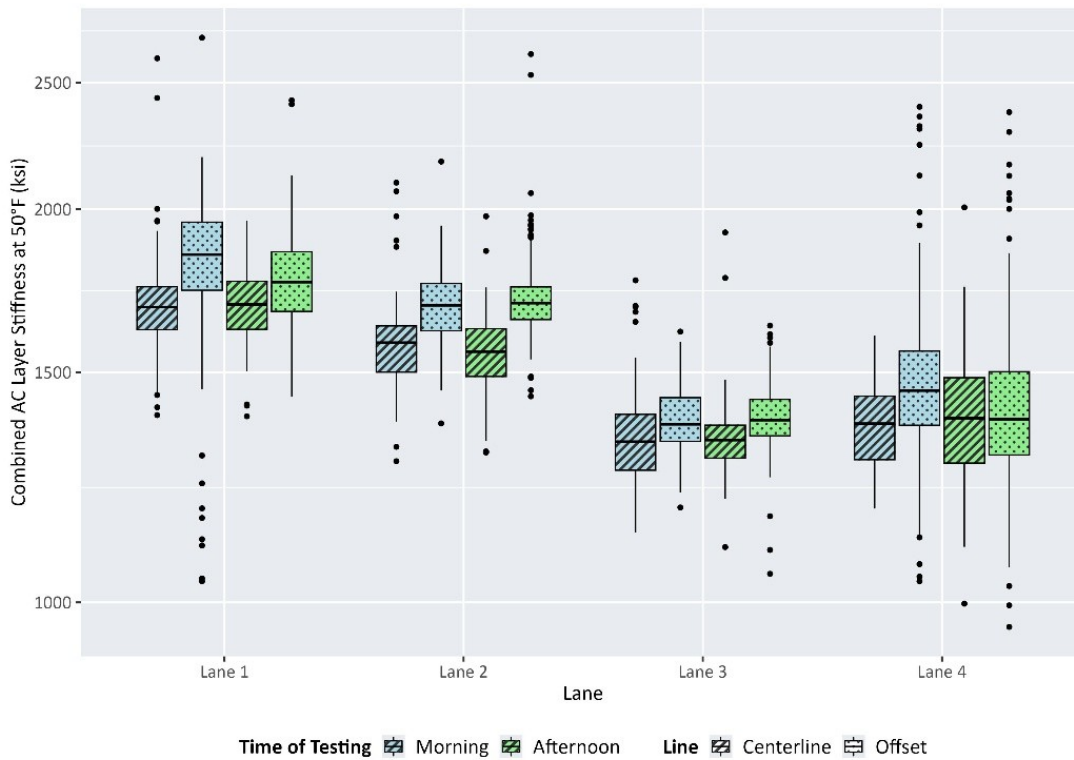


Figure 6.7. Backcalculated stiffness for the combined AC layer adjusted to 50°F.

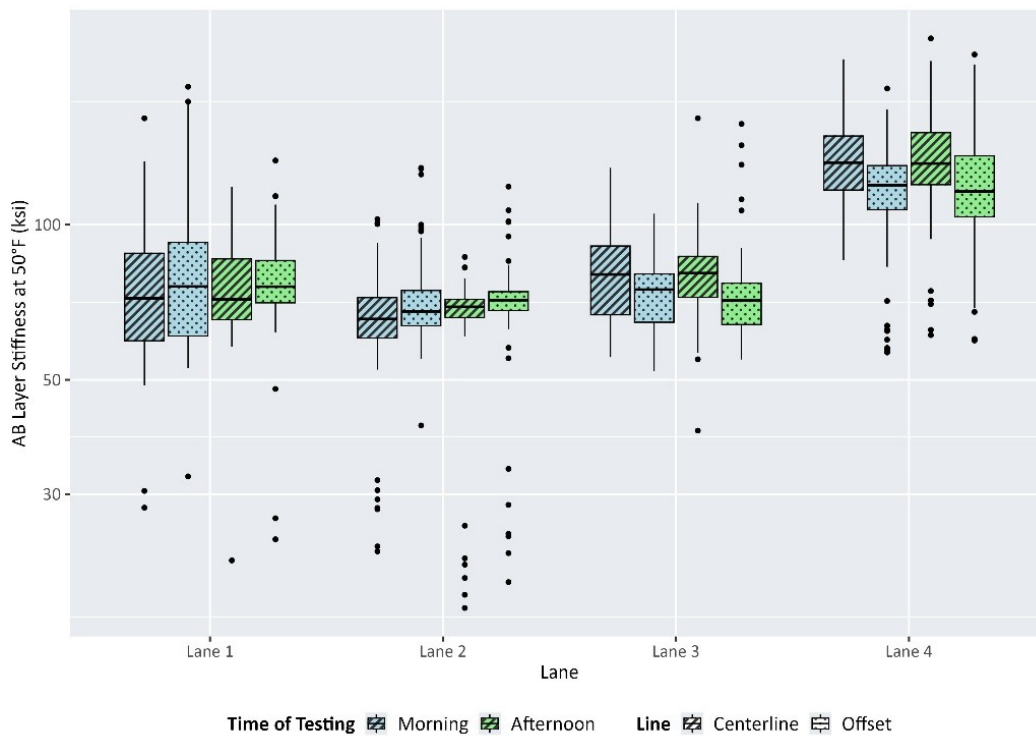


Figure 6.8. Backcalculated stiffness for the aggregate base layer adjusted to 50°F.

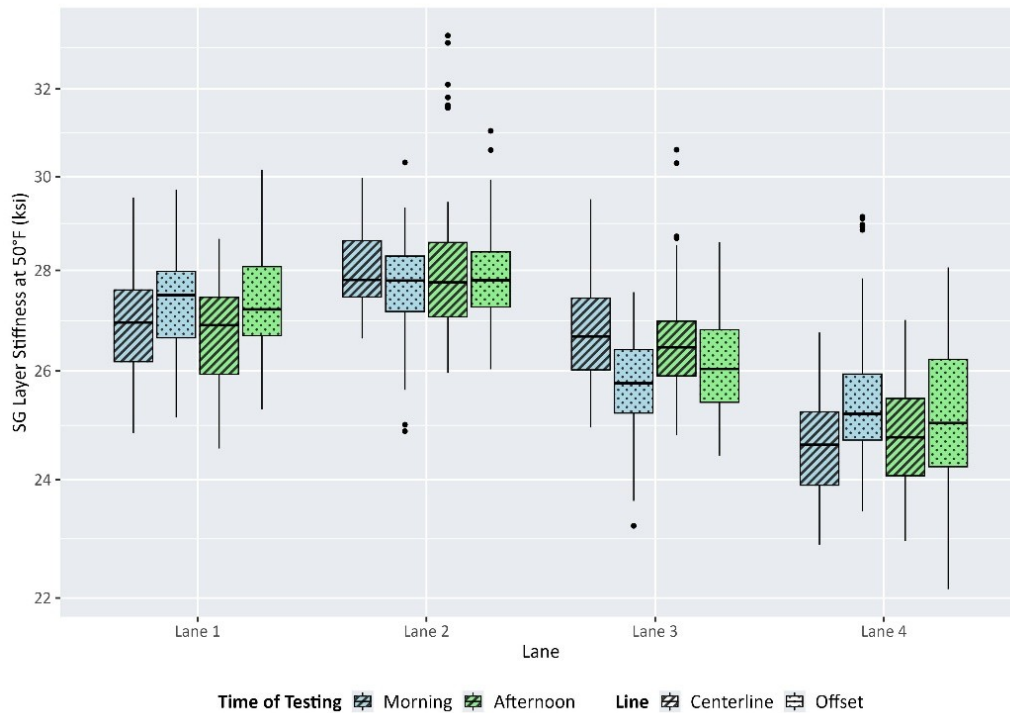


Figure 6.9. Backcalculated stiffness for the subgrade layer adjusted to 50°F.

As shown in Figure 6.7 to Figure 6.9, the backcalculated layer stiffnesses are the same for morning and afternoon after the temperature adjustment, which is expected. The figures suggest the following:

- For the combined AC layers:
 - Lane 1 and Lane 2 stiffnesses are approximately the same.
 - Lane 3 and Lane 4 stiffnesses are approximately the same but lower than the values for Lane 1 and Lane 2.
- For the AB layer:
 - Lane 1, Lane 2, and Lane 3 stiffnesses are approximately the same.
 - Lane 4 stiffnesses are higher than the values for the other three lanes.
- For the subgrade layer:
 - Stiffnesses for all lanes are similar.

The summary of adjusted backcalculated layer stiffnesses is shown in Table 6.1. The statistics shown are calculated after removing outliers. The table also shows that the standard deviation factors (SDF) for the combined AC layer and subgrade layer stiffness are no more than 1.02, while the SDF values for the AB layer stiffness are no more than 1.08. Compared with the typical values of 1.20 to 1.50 seen on Caltrans highways, these SDF values are very low and indicate very uniform test sections.

Table 6.1: Summary of Backcalculated Layer Stiffnesses Adjusted to 50°F

Lane	Line	Combined AC Layer (ksi)	AB Layer (ksi)	Subgrade Layer (ksi)
Lane 1	Centerline	1,689 [1.02]	76 [1.08]	27 [1.02]
Lane 2	Centerline	1,555 [1.01]	68 [1.03]	28 [1.01]
Lane 3	Centerline	1,328 [1.01]	80 [1.06]	27 [1.02]
Lane 4	Centerline	1,378 [1.02]	136 [1.06]	25 [1.02]
Lane 1	Offset	1,806 [1.02]	78 [1.07]	27 [1.02]
Lane 2	Offset	1,695 [1.01]	70 [1.03]	28 [1.01]
Lane 3	Offset	1,388 [1.01]	72 [1.06]	26 [1.02]
Lane 4	Offset	1,432 [1.03]	121 [1.06]	25 [1.02]

Note: Numbers in brackets are the standard deviation factor (SDF) of the stiffness in ksi: $SDF = 10^{\text{COV}(\log_{10}(E(\text{ksi})))}$; COV is coefficient of variation; AB: aggregate base.

6.4 Summary

Analysis of the FWD testing data shows that the subgrade layer is uniform throughout all the lanes, while the aggregate base (AB) layer has significantly higher stiffness for Lane 4 compared with the other lanes. Judging from the low deviations in backcalculated layer stiffness, all the lanes are quite uniform among themselves. After being adjusted to 50°F (°C), the combined AC layer stiffness was found to be around 1,305 to 1,741 ksi, the AB layer stiffness ranged from 68 to 135 ksi, and the subgrade layer stiffness was around 26.1 ksi.

7 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

7.1 Summary

This technical memorandum describes the design and construction of a full-scale test track and the construction quality control/quality assurance (QC/QA) testing of the different materials, including subgrade, aggregate base (AB), and asphalt mixes. The test track evaluation is part of the Caltrans balanced mix design (BMD) implementation effort led by the Pavement and Materials Partnering Committee (PMPC) BMD Working Group. The main goal of the test track is to evaluate the rutting performance of surface mixes under the Heavy Vehicle Simulator (HVS) and to compare this performance with laboratory rutting test results. A secondary goal is to evaluate the reflective cracking performance of the same mixes and to compare this performance with laboratory cracking test results.

The test track includes four lanes, each 177 ft. long. The north part of the lanes, 135 ft. long, has an asphalt long-life pavement structure designed and built to test the rutting performance of the asphalt surface layer under the HVS. The long-life structure includes a 0.4 ft. asphalt surface whose rutting performance will be evaluated, placed in two 0.2 ft. lifts, on top of a 0.4 ft. intermediate layer and a 0.2 ft. rich bottom base. The south part of the lanes, 42 ft. long, has a jointed plain concrete pavement (JPCP) structure with an asphalt overlay, 0.2 ft. thick, whose reflective cracking performance will be evaluated. The asphalt surface mix is the same along each of the 177 ft. long lanes. The same applies to the aggregate base and the subgrade. The test track is a long-life structure where the asphalt surface can be evaluated and replaced in successive testing rounds.

Following PMPC BMD Working Group directions, the first round of testing was focused on hot mix asphalt (HMA) Type A with plain binder, and this type of mix was used for the construction of the test track surface. The control mix was used for paving the Lane 1 surface of the test track. The control mix is an HMA with 3/4 in. maximum aggregate size (MAS), 25% reclaimed asphalt pavement (RAP) content (percent by aggregate replacement), and virgin binder graded as PG 58-22. The control mix design is based on an existing design already approved for Caltrans use. Three alternative mixes were used for paving the surface of Lane 2, Lane 3, and Lane 4. The alternative mixes were intentionally designed to make the mixes susceptible to rutting as follows:

- The binder content (total mass of mix (TMM)) was increased from 4.5% to 5.75% (1.25% increase), and the resulting mix is referred to as “High Pb” in this technical memorandum.
- A high amount of tall oil recycling agent was added (2.5% by total mass of binder). The resulting mix is referred to as “High RA” in this technical memorandum.
- A high amount of natural sand was used, which constituted 18.75% of the total aggregate blend (the natural sand was only 6.25% of the total aggregate blend in the control mix). The resulting mix is referred to as “Nat. Sand” in this technical memorandum.

The subgrade and aggregate base were built from June 2024 to July 2024, the JPCP concrete slabs were cast in August 2024, and the asphalt mixes were paved in November 2024.

7.2 Conclusions

The construction QC/QA testing, together with the initial performance-related testing of the asphalt mixes, leads to the following conclusions related to the pavement structure, the asphalt long-life pavement structure, and the asphalt surface mixes.

Key conclusions related to the pavement structure are the following:

- Based on the FWD testing data analysis, the subgrade layer stiffness is uniform throughout the four lanes. The aggregate base layer has significantly higher stiffness for Lane 4 under the Nat. Sand surface mix compared with the other lanes.
- Based on the FWD testing data analysis, all the lanes are quite uniform among themselves, which indicates low construction variability of the subgrade, aggregate base, and asphalt mixes.
- The drying shrinkage of the PCC used in the JPCP construction was considerably high, which was the desired outcome. The PCC was selected as it was expected to have high drying shrinkage to maximize the potential for inducing reflective cracking on the asphalt overlay.
- The construction of all materials went as expected, with no incidents to report.
- The asphalt long-life pavement structure was instrumented with sensors to measure the structural response and the permanent deformation under the HVS loading. The instrumentation included thermocouples, strain gauges, pressure cells, and multi-depth deflectometers.
- The JPCP slabs were instrumented with sensors to measure the slab response to ambient environmental loading. The instrumentation included thermocouples and vibrating wire strain gauges.

Key conclusions related to the asphalt long-life pavement structure (intermediate and rich bottom layers) are the following:

- The intermediate layer mix was designed to have 2.5% air-voids at 85 gyrations (N_{design}). The N_{design} air-voids of the rich bottom mix were 1.5%.
- The rich bottom mix met the four-point bending (4PB) test stiffness and fatigue resistance requirements set by Caltrans asphalt concrete (AC) long-life specifications used for the rehabilitation of Interstate 5 in Sacramento County in 2019 to 2021 (EA 03-0H10U).
- The intermediate layer mix met the 4PB fatigue resistance requirements (EA 03-0H10U) but failed the stiffness requirement. At 433 $\mu\epsilon$ peak-to-peak strain, the mix stiffness was only 82.5% of the minimum 782 ksi required stiffness.
- The intermediate layer mix passed the HWT test requirements. No rutting requirements were set for the rich bottom mix.
- Based on the production mixes sampled at the plant during the construction, the two mixes met the volumetrics requirements, including binder content and air-voids at 85 gyrations.

However, the binder content of the intermediate layer mix, based on automated extraction, was 5.8% to 5.9%, which is in the upper range of the 5.0% to 5.8% allowable binder content.

- Based on coring conducted after the construction, the two mixes were compacted correctly, as the relative density was 95.5% to 97.5% for the intermediate layer mix and 97.0% to 98.5% for the rich bottom mix.
- The construction of the two layers—intermediate and rich bottom base—went as expected, with no incidents to report.

Key conclusions related to the asphalt surface mixes are the following:

- The PRT results confirmed that mixture design variables strongly influence rutting, stiffness, and cracking performance.
- The control surface mix was designed to have 4.0% air-voids at 85 gyrations (N_{design}), and the same applies to the surface mix with high natural sand content. While no specific air-void was targeted for the other two mixes, a lower air-void was expected because of either the high asphalt binder content or the overdose of recycling agent.
- The four surface mixes passed the HWT test requirements, even though the three alternative mixes had been intentionally designed to be prone to rutting.
- The 1.25% TMM increase in binder content had a clear impact on the rutting performance measured with the HWT test, as the measured rutting was more than twice that of the control mix, though it still met Caltrans specifications. However, the overdose of recycling agent and the use of a very high amount of natural sand did not have a noticeable impact on the rutting performance measured with the HWT test.
- Based on the production mixes sampled at the plant during the construction, the four mixes met the volumetrics requirements, including binder content and air-voids at 85 gyrations.
- Based on coring conducted after the construction, the four mixes were compacted correctly, as the relative density was 93% to 95% for the surface mix control, 93% to 95% for the High Pb, 97% to 98% for the High RA, and 92% to 93% for the Nat. Sand.
- The construction of the surface layer in each of the four lanes went as expected, with no incidents to report.
- While the virgin asphalt binder used for producing all the asphalt mixes used in the test track construction (surface, intermediate, and rich bottom) was commercialized as PG 58-22, it actually graded as PG 58-28 based on the testing of the binder sampled during mix production.
- Based on the high PG (PGH) of the RAP and virgin binders, considering the expected impact of the tall oil recycling agent, and following the standard blending chart approach, the following PGH values were estimated: 75.9 (control), 73.0 (High Pb), 65.6 (High RA), and 75.9 (Nat. Sand). However, testing of the binder extracted and recovered from cores extracted from the control surface layer resulted in a PGH of 69 to 73, somewhat lower than the blending chart estimation.

7.3 Recommendations

The construction QC/QA testing, together with the initial performance-related testing of the asphalt mixes, leads to the following recommendations:

- The long-life asphalt pavement specification framework itself is adequate, but it is very challenging to achieve the proper balance without extensive mix design experience and consideration of quarry product availability (e.g., washed versus unwashed crushed dust). The approach is on the right track, but there is room for improvement and further risk reduction before widespread mainline use.
- At the same time, the long-life asphalt pavement specification framework needs to deliver a high-performing mix that improves the durability and life cycle cost of Caltrans asphalt pavement without being overly complex. Further improvement of the testing requirements to decrease the complexity of testing is needed, combined with education, outreach, and more widespread implementation to build industry knowledge.
- For the next round of construction, expected in the fall of 2025, the following are recommended:
 - Mix design adjustments: Dedicate additional resources to optimize mix designs, including gradation and adjustment of quarry products.
 - Modified binders: Consider modified binders in surface layers to increase mix stiffness and rutting resistance but also to maintain fatigue and age-related cracking performance.
 - Test methods: Continue to collect HWT test results and other rutting test data to capture high-temperature rutting susceptibility results in the laboratory and benchmark the results against HVS tests and whatever field performance can be gathered.
- The data obtained from the HVS testing of the test track documented in this technical memorandum will inform the development of Caltrans' BMD specifications, particularly the selection of appropriate rutting and cracking tests and their associated specification limits. However, data from at least one additional test track are highly desirable before final recommendations can be established.

APPENDIX A ASPHALT MIX PAVING TEMPERATURE AND SEQUENCE

DAY 1

BMD TEST TRACK CONSTRUCTION

	Truck No.	Mix Type	Time Left Plant	Time Arrived at Jobsite	Time Started Paving	Time Finished Paving	Notes
LAYER 1	1	¾" Rich Bottom R40 LLP	6:30AM 315°F	8:17AM 324°F	8:21AM 280°F	8:35AM 210°F	
	2			8:40AM 289°F	8:42AM 262°F	8:43AM 261°F	
	1			8:45AM 257°F	8:45AM 310°F	8:49AM 259°F	
	2			9:03AM 240°F	9:03AM 289°F	9:05AM 267°F	
	3			9:07AM 277°F	9:08AM 241°F	9:09AM 224°F	
	4			9:17AM 290°F	9:28AM 280°F	9:31AM 260°F	
	3			9:37AM 300°F	9:38AM 260°F	9:41AM 270°F	
	4			9:48AM 298°F	9:53AM 290°F	9:56AM 273°F	
	5			9:57AM 318°F	9:59AM 298°F	10:01AM 208°F	
	6			10:04AM 300°F	10:06AM 273°F	10:07AM 246°F	

Outside Temperature: _____ 53°F _____ AT 8:17AM

Rich Bottom Lift #1 (11/6/2024)

DAY 1

BMD TEST TRACK CONSTRUCTION

	Truck No.	Mix Type	Time Left Plant	Time Arrived at Jobsite	Time Started Paving	Time Finished Paving	Notes
LAYER 2	1	¾" Intermediate R40 LLP		12:01PM 319°F	12:05PM 315°F	12:06PM 267°F	Lane Temperature Before Paving: Lane 1 – 95°F Lane 2 – 95.9°F Lane 3 – 97.7°F Lane 4 – 95.4°F
	2			12:07PM 324°F	12:09PM 288°F	12:10PM 276°F	
	1			12:11PM 333°F	12:12PM 300°F	12:14PM 279°F	
	2			12:14PM 295°F	12:15PM 257°F	12:15PM 242°F	
	3			12:22PM 290°F	12:31PM 260°F	12:33PM 262°F	
	4			12:34PM 300°F	12:35PM 280°F	12:36PM 294°F	
	3			12:38PM 300°F	12:39PM 247°F	12:39PM 231°F	
	4			12:46PM 311°F	12:51PM 309°F	12:52PM 252°F	
	5			12:54PM 296°F	12:55PM 277°F	12:59PM 260°F	
	6			1:03PM 300°F	1:10 295°F	1:12PM 276°F	
	5			1:14PM 306°F	1:15PM 272°F	1:16PM 253°F	
	6			1:21PM 332°F	1:22PM 308°F	1:23PM 216°F	

Outside Temperature: _____ 64°F _____ AT 12:21PM

Intermediate Lift # 1 (11/6/2024)

DAY 1

BMD TEST TRACK CONSTRUCTION

	Truck No.	Mix Type	Time Left Plant	Time Arrived at Jobsite	Time Started Paving	Time Finished Paving	Notes
LAYER 3	1	¾" Intermediate R40 LLP		2:07PM 317°F	2:12PM 307°F	2:14PM 291°F	Lane Temperature Before Paving: Lane 1 – 144°F Lane 2 – 116°F Lane 3 – 106°F Lane 4 – 99.3°F
	2			2:16PM 320°F	2:17PM 319°F	2:18PM 306°F	
	1			2:20PM 313°F	2:21PM 282°F	2:22PM 247°F	
	2			2:44PM 296°F	2:46PM 258°F	2:47PM 250°F	
	3			2:50PM 261°F	2:51PM 246°F	2:52PM 247°F	
	4			3:45PM 290°F	3:46PM 310°F	3:47PM 262°F	
	3			3:59PM 320°F	4:00PM 292°F	4:01PM 267°F	
	4			4:08PM 320°F	4:09PM 312°F	4:11PM 251°F	
	5			4:24PM 320°F	4:25PM 289°F	4:26PM 286°F	
	6			4:30PM 320°F	4:31PM 312°F	4:32PM 307°F	
	5			4:33PM 320°F	4:34PM 299°F	4:35PM 299°F	

Outside Temperature: _____ 68°F _____ AT 2:08AM

Intermediate Lift #2 (11/6/2024)

DAY 2

BMD TEST TRACK CONSTRUCTION

Truck No.	Mix Type	Time Left Plant	Time Arrived at Jobsite	Time Started Paving	Time Finished Paving	Notes
1	¾" Surface R25 #4 SP (50% High Sand Mix)		7:59AM 320°F	8:00AM 294°F	8:02AM 294°F	Lane Temperature Before Paving: 53°F
2			8:04AM 315°F	8:05AM 300°F	8:06AM 269°F	
1			8:14AM 320°F	8:15AM 317°F	8:16AM 278°F	
2	¾" Surface R25 #3 SP (High Rejuv.)		8:27AM 320°F	8:27AM 320°F	8:30AM 279°F	Lane Temperature Before Paving: 61.2°F
3			8:31AM 313°F	8:32AM 298°F	8:33AM 275°F	
4			8:41AM 320°F	8:43AM 314°F	8:43AM 300°F	
3	¾" Surface R25 #4 SP (50% High Sand Mix)		9:16AM 320°F	9:17AM 320°F	9:18AM 297°F	Lane Temperature Before Paving: 107°F
4			9:20AM 310°F	9:21AM 309°F	9:22AM 276°F	
5			9:25AM 320°F	9:25AM 320°F	9:27AM 289°F	
6			9:33AM 320°F	9:34AM 309°F	9:35AM 271°F	
5	¾" Surface R25 #3 SP (High Rejuv.)		9:53AM 312°F	9:54AM 310°F	9:55AM 304°F	Lane Temperature Before Paving: 112°F
6			10:06AM 320°F	10:07AM 320°F	10:08AM 283°F	
7			10:12AM 320°F	10:13AM 308°F	10:13AM 287°F	
8			10:20AM 313°F	10:21AM 320°F	10:22AM 304°F	

Outside Temperature: _____ 49°F _____ AT 8AM

Surface Lift #1 (11/7/2024)

DAY 2

BMD TEST TRACK CONSTRUCTION

Truck No.	Mix Type	Time Left Plant	Time Arrived at Jobsite	Time Started Paving	Time Finished Paving	Notes
1	¾" Surface R25 #2 (+1.5% AC Content)		11:50AM 320°F	11:51AM 317°F	11:52AM 300°F	Lane Temperature Before Paving: 92.3°F
2			11:54AM 320°F	11:54AM 305°F	11:55AM 270°F	
1			12:00PM 320°F	12:05PM 277°F	12:05PM 247°F	
2	¾" Surface R25 #1 SP (Control)		12:14PM 316°F	12:14PM 320°F	12:16PM 319°F	Lane Temperature Before Paving: 98.6°F
3			12:18PM 320°F	12:19PM 320°F	12:20PM 282°F	
4			12:26PM 320°F	12:26PM 320°F	12:28PM 311°F	
3	¾" Surface R25 #2 (+1.5% AC Content)		12:56PM 320°F	12:58PM 313°F	1:00PM 298°F	Lane Temperature Before Paving: 166°F
4			1:02PM 320°F	1:03PM 320°F	1:04PM 294°F	
5			1:06PM 320°F	1:07PM 308°F	1:08PM 278°F	
6			1:16PM 306°F	1:17PM 306°F	1:18PM 309°F	
5	¾" Surface R25 #1 SP (Control)		1:33PM 320°F	1:34PM 306°F	1:35PM 292°F	Lane Temperature Before Paving: 146°F
6			1:41PM 331°F	1:41PM 298°F	1:42PM 292°F	
7			1:45PM 320°F	1:46PM 294°F	1:46PM 305°F	
8			1:49PM 312°F	1:50PM 308°F	1:52PM 275°F	

Outside Temperature: _____ 64°F _____ AT 11:50AM

Surface Lift #2 (11/7/2024)



Demonstration Results of the RTDensity System on the
UCPRC Test Track
November 6, 2024

Summary

On November 6, 2024, George Reed, Inc. began constructing an asphalt test track at the University of California, Davis, using RTDensity technology for real-time density estimates.

- **Construction Details:** The project included three asphalt layers: a base layer compacted to 97%, an intermediate layer compacted to 95%, and a final surface layer with various asphalt mixtures for research.
- **RTDensity Calibration and Results:** RTDensity provided real-time density estimates, using core samples for calibration and validation. Post-calibration error rates improved significantly, with 0.4% for rich-bottom and 0.6% for intermediate mixes.
- **Percentage Within Limits (PWL) Tables:** RTDensity generated PWL tables for the construction area, showing density distribution and compaction quality for each asphalt lift.

Job Description

On November 6, 2024, George Reed, Inc. began construction of the University of California Pavement Research Center’s (UCPRC) asphalt test track at the University of California, Davis campus. The two-day project included three asphalt layers: a base layer of Rich Bottom R40 LLP asphalt concrete compacted to 97%, an intermediate layer (paved in two lifts) of Intermediate R40 LLP asphalt concrete compacted to 95%, and a final surface layer with various asphalt mixtures for research studies. Each of the CAT CB10 rollers used for compaction by George Reed had been previously retrofitted with the RTDensity technology, and the first day of the project was used as a demonstration of the system’s capabilities.

Results

RTDensity provided real-time density estimates to the roller during construction. For calibration, a single core from each asphalt type was used, while others were reserved for validation. Each sample was tagged with a GPS location, measured for specific gravity in the UCPRC lab, and compared to the RTDensity estimate at the same GPS location. Tables 1 and 3 show RTDensity’s pre- and post-calibration estimates and core measurements for rich-bottom long-life and intermediate mixes. During intermediate lift compaction, RTDensity notified the crew of sub-target results, leading to the addition of a second roller. Thus, Table 3 reflects final pass results from two independently calibrated rollers. RTDensity offers Percentage Within Limits (PWL) tables for the entire construction area. Tables 2, 4, and 5 present PWL outcomes for each of the test track lifts. Additionally, RTDensity provides an interactive as-built map of the completed work as a quality control tool for project managers and foremen; Figures 1-3 show the computed density profiles for each of the three lifts paved.

Table 1: A comparative analysis between RTDensity estimates and lab tests for core samples pulled from the rich-bottom long-life lift of the compacted asphalt roadway shows an average error of 0.4%. The

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core with an asterisk was used for calibration of the system. *Note: An asterisk denotes a core sample used for calibration as is excluded from the validation results, i.e. average error.*

	Subsample 1	Subsample 2	Subsample 3	Subsample 4
Core ID	L1-1 *	L1-2	L1-3	L1-4
Position	Centerline	Centerline	Centerline	Centerline
Compaction (Core Sample)	98.7	97.7	97.5	98.7
RTDensity (pre-calibration)	94	93.3	93.5	93.9
RTDensity (post-calibration)	98.7	98	98.2	98.6
Difference (post-calibration)	N/A	-0.3	-0.7	0.1

Table 2: A PWL table from RTDensity estimates for the rich-bottom long-life lift of compacted asphalt roadway.

Limits [%]	Percentage Within Limits
< 97	21.3 %
97 – 99	78.6 %
> 99	0.1%

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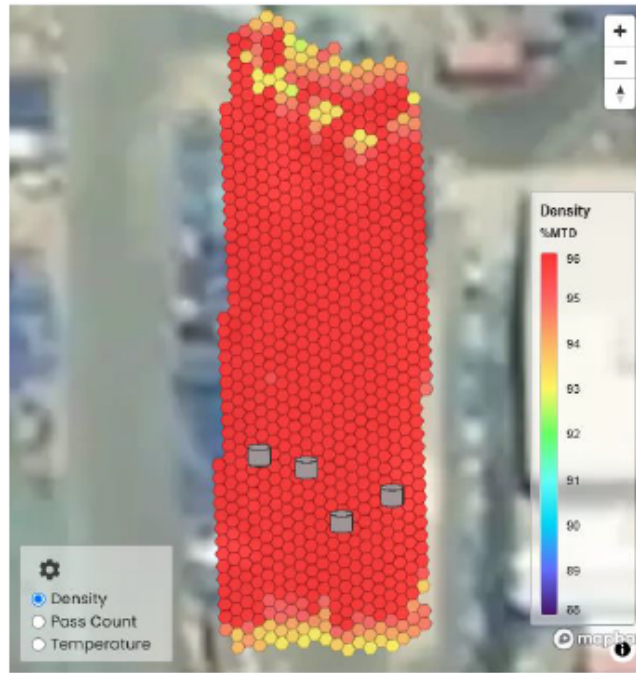


Figure 1: The as-built map for the rich-bottom layer of asphalt colored by density estimates with GPS locations of core samples shows a uniform density distribution throughout the project.

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Table 3: A comparative analysis between RTDensity estimates and lab tests for core samples pulled from the intermediate lifts of the compacted asphalt roadway shows an average error of 0.6%. The core with an asterisk was used for calibration of the system. *Note: An asterisk denotes a core sample used for calibration as is excluded from the validation results, i.e. average error.*

	Subsample 5	Subsample 6	Subsample 7	Subsample 8	Subsample 9	Subsample 10	Subsample 11	Subsample 12
Core ID	L2-I-1 *	L2-I-2	L2-I-3	L2-I-4	L3-I-1	L3-I-2	L3-I-3	L3-I-4
Position	Centerline	Centerline	Centerline	Centerline	Centerline	Centerline	Centerline	Centerline
Compaction (Core Sample)	95.4	95.2	95.2	95.6	97.5	97.3	96.2	96.1
RTDensity (pre-calibration)	92.6	93	92.2	92.7	93.6	93.7	93.7	94.3
RTDensity (post-calibration)	95.4	94.8	94.6	95.5	97.6	97.7	97.7	94.7
Difference (post-calibration)	N/A	0.4	0.6	0.1	-0.1	-0.4	-1.5	1.4

Table 4: A PWL table from RTDensity estimates for the first intermediate lift of compacted asphalt roadway.

Limits [%]	Percentage Within Limits
< 94	14.9%
94 – 96	83.4 %
> 96	1.8 %

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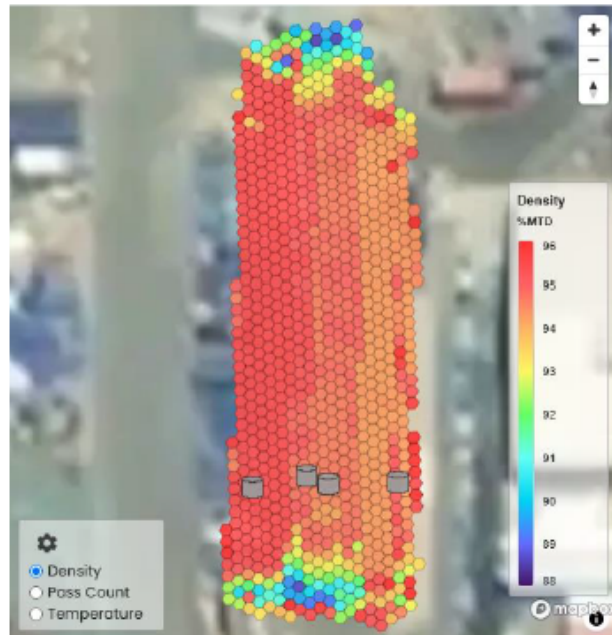


Figure 2: The as-built map for the first intermediate lift of asphalt colored by density estimates with GPS locations of core samples shows lower east-side density readings compared to the west-side as an effect of changing the rolling pattern.

Table 5: A PWL table from RTDensity estimates for the second intermediate lift of compacted asphalt roadway.

<i>Limits [%]</i>	<i>Percentage Within Limits</i>
< 94	17.9 %
94 – 96	45.3 %
> 96	36.8 %

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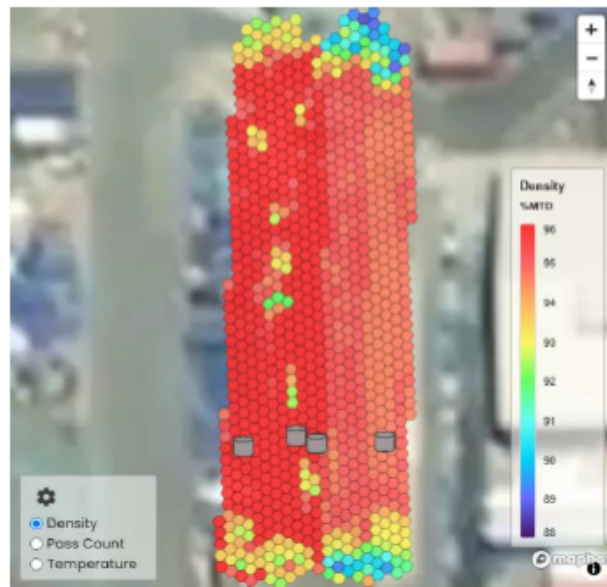


Figure 3: The as-built map for the second intermediate lift of compacted asphalt with GPS locations of core samples shows both low-density spots in the western portion and a lower-density eastern half, potentially due to the first intermediate lift (Figure 2).

Conclusion

On November 6, 2024, RTDensity provided real-time estimates for the UCPRC test track paving, with uncalibrated average error rates of 4.5% for rich-bottom and 2.8% for intermediate mix designs. After calibration, error rates improved to 0.4% and 0.6%, respectively. RTDensity also delivered a complete as-built map with full coverage of asphalt compaction, including density, temperature, and pass count profiles in the post-job analysis online software.

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