August 2019
Summary Report: UCPRC-SR-2018-01

Development of Improved Guidelines and Designs for Thin BCOA: Summary, Conclusions, and Recommendations

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Partnered Pavement Research Center (PPRC) Strategic Plan Element (SPE) 4.58B:
Evaluate Early-Age and Premature Cracking for PaveM and LCCA (whitetopping);
Project Task 2878: Thin Whitetopping

PREPARED FOR:

California Department of Transportation
Division of Research, Innovation, and System Information
Office of Materials and Infrastructure Roadway Research

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TECHNICAL REPORT DOCUMENTATION PAGE

1. REPORT NUMBER UCPRC-SR-2018-01	2. GOVERNMENT ASSOCIATION NUMBER	3. RECIPIENT'S CATALOG NUMBER
4. TITLE AND SUBTITLE Development of Improved Guidelines and Designs for Thin BCOA: Summary, Conclusions, and Recommendations		5. REPORT PUBLICATION DATE August 2019 6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Angel Mateos (ORCID 0000-0002-3614-2858), John Harvey (ORCID 0000-0002-8924-6212), Fabian Paniagua, Julio Paniagua, and Rongzong Wu (ORCID 0000-0001-7364-7583)		8. PERFORMING ORGANIZATION REPORT NO. UCPRC-SR-2018-01
9. PERFORMING ORGANIZATION NAME AND ADDRESS University of California Pavement Research Center Department of Civil and Environmental Engineering, UC Davis 1 Shields Avenue Davis, CA 95616		10. WORK UNIT NUMBER 11. CONTRACT OR GRANT NUMBER 65A0542
12. SPONSORING AGENCY AND ADDRESS California Department of Transportation Division of Research, Innovation, and System Information P.O. Box 942873 Sacramento, CA 94273-0001		13. TYPE OF REPORT AND PERIOD COVERED Summary Report September 2014 to September 2017 14. SPONSORING AGENCY CODE 2878
15. SUPPLEMENTAL NOTES		

16. ABSTRACT

This report summarizes the investigations undertaken by the University of California Pavement Research Center between 2014 and 2017 to develop recommendations and guidance on the use of thin bonded concrete overlay of asphalt (BCOA) as a rehabilitation alternative for California based on the adoption of, and improvements to, the technology developed in other US states. The main tasks of the project included: 1) laboratory testing of four rapid-strength concrete mixes and a number of concrete-asphalt interfaces, 2) evaluation of the construction of a full-scale test track, 3) monitoring of the structural and hygrothermal responses of six thin BCOA sections to the ambient environment, 4) accelerated pavement testing with the Heavy Vehicle Simulator (HVS) on eleven thin BCOA sections, 5) finite element method modeling, and 6) development of a set of recommendations for the design and construction of thin BCOA pilot projects in California. Based on this testing and analysis, it was possible to obtain a better understanding of the mechanics of the structure of thin BCOA and of the roles of the different factors that determine thin BCOA performance. Overall, the performance of the thin BCOA sections in the HVS testing far exceeded expectations. The 11 sections resisted the predefined HVS loading without cracking. In five of the sections, that loading was equivalent to 6 million equivalent single axle loads (ESALs) and included load levels more than twice the legal limit in California, channelized traffic at the edge of the slabs, and a continuous water supply that simulated flooded conditions. The main conclusion from this research project is that a well-designed, well-built 6×6 thin bonded concrete overlay section placed on top of an asphalt base that is in fair to good condition can potentially provide 20 years of good serviceability on most of California's non-interstate roadways. Eight individual reports prepared for the project provide a complete description of the work carried out, and include detailed conclusions about each phase. This report includes a summary of those conclusions and a set of recommendations for the design of thin BCOA that considers California traffic, climate and materials conditions, and construction work zone practices.

17. KEY WORDS BCOA, rigid pavement, bonded concrete overlay of asphalt, thin whitetopping, rapid-strength concrete, coefficient of thermal expansion, moisture-related shrinkage, accelerated pavement testing, load transfer efficiency, concrete cracking	18. DISTRIBUTION STATEMENT No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA 22161	
19. SECURITY CLASSIFICATION (of this report) Unclassified	20. NUMBER OF PAGES 49	21. PRICE None

Reproduction of completed page authorized

UCPRC ADDITIONAL INFORMATION

1. DRAFT STAGE Final	2. VERSION NUMBER 1
3. PARTNERED PAVEMENT RESEARCH CENTER STRATEGIC PLAN ELEMENT NUMBER 4.58B	4. DRISI TASK NUMBER 2878
5. CALTRANS TECHNICAL LEAD AND REVIEWER(S) D. Maskey	6. FHWA NUMBER CA192878B

7. PROPOSALS FOR IMPLEMENTATION
This report includes a set of recommendations for the design of thin BCOA that considers California traffic, climate and materials conditions, and construction work zone practices. Most of these recommendations are ready for implementation, although further research and validation based on pilot projects is recommended.

8. RELATED DOCUMENTS
UCPRC-TM-2015-01, UCPRC-RR-2017-02, UCPRC-RR-2017-03, UCPRC-RR-2017-04, UCPRC-RR-2017-05, UCPRC-RR-2017-06, UCPRC-RR-2017-07, UCPRC-RR-2017-08

9. LABORATORY ACCREDITATION
The UCPRC laboratory is accredited by AASHTO re:source for the tests listed in this report

10. SIGNATURES

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ACKNOWLEDGMENTS

A number of people and Institutions have provided input and support for the research presented in this report. The construction of the thin BCOA test sections was supported by the California concrete pavement industry, under the coordination of the Southwest Concrete Pavement Association. This association, particularly Executive Director Charles Stuart and consultant Bruce Carter, also provided continuous support for the design and execution of the full-scale experiment. The technical contribution from Teichert Materials, particularly Pete Conlin, Moises Caballero, and Chris Fuchs, and from CTS Cement, particularly Vince Perez and Art Bigelow, is acknowledged and greatly appreciated. The contribution by the University of Pittsburgh, a subcontractor in this research project, is also acknowledged, particularly the valuable contributions of Julie Vandenbossche and John DeSantis. Special thanks for their valuable input are due to several experts: John Bolander (UC Davis), Tom Van Dam (NCE), James Mack (CEMEX), and Jason Weiss (Oregon State University). The technical review by Caltrans, led by Deepak Maskey from the Office of Concrete Pavement, and oversight by Joe Holland, of the Division of Research, Innovation and System Information, is appreciated. The help of the staff and students at the UCPRC with the construction and testing of sections and with the laboratory testing is also acknowledged.

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LIST OF ABBREVIATIONS USED IN THE REPORT

BCOA	Bonded concrete overlay of asphalt
CSA	Calcium sulfoaluminate
CTE	Coefficient of thermal expansion
ENV	Environmental test section
HMA	Hot mix asphalt
FEM	Finite element method
FWD	Falling Weight Deflectometer
HVS	Heavy Vehicle Simulator
JDMD	Joint displacement measuring device
LCCA	Life cycle cost analysis
LTE	Load transfer efficiency
MC	Moisture content
MEPDG	*Mechanistic-Empirical Design Guide*
OT	Opening time
PPRC	Partnered Pavement Research Center
RH	Relative humidity
RSC	Rapid-strength concrete
SRA	Shrinkage-reducing admixture
UCPRC	University of California Pavement Research Center
VWSG	Vibrating wire strain gage

1 INTRODUCTION

Thin bonded concrete overlay of asphalt (BCOA), formerly known as *thin whitetopping*, is a pavement rehabilitation technique that consists of placement of a 4 to 7 in. (100 to 175 mm) thick concrete overlay on an existing flexible or composite pavement. While the technology for thin BCOA has been used on highways and conventional roads in several US states as well as in other countries for at least 20 years, use of thin BCOA prior to this research project has been very limited in California.

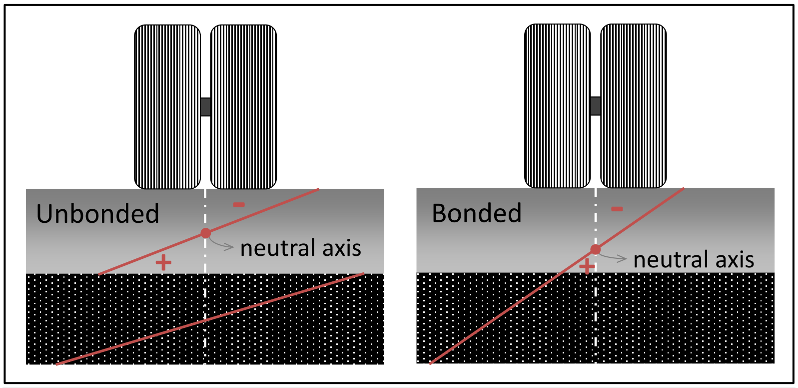


***Illustration of the Effect of Bonding on Tensile Stress in the Concrete*. The bond between the concrete and the asphalt results in a composite structure. This causes the neutral axis of the slab to shift to a lower position, closer to the bottom of the slab, thus reducing the tensile stresses that develop in the concrete slab at that location under traffic loading.**

Thin BCOA differs from conventional concrete overlay of asphalt in several ways, with the structural contribution of the existing asphalt acting as a bonded base being the most critical. In conventional overlays, the asphalt base is primarily intended to serve as a flexible, nonerodible base to provide support for the concrete slabs. In the conception of the thin BCOA technique the asphalt base makes a greater contribution to the structure's strength by bonding to the concrete to form a composite slab where both layers work together to resist bending. The bonding results in a much stronger pavement structure than if the slabs are not bonded because the bonding significantly reduces the tensile stresses in the concrete slab that cause cracking. Bonding between the concrete and asphalt therefore constitutes one of the main factors that must be considered in the design, construction, and maintenance and rehabilitation of this type of pavement.

Thin BCOA technology has steadily improved since the mid-1990s and could be regarded as a mature technique by the time the research presented in this report began. However, there were still key gaps in knowledge of the technique that required further research. Among these gaps were the essentially unknown role and performance of the concrete-asphalt interface; the mechanics of the asphalt base, which was systematically oversimplified in BCOA design approaches; and the lack of a BCOA faulting model *(1)*. In addition to those gaps in knowledge, thin BCOA practice regarding important design features—such as slab dimensions, shoulder type, and the need for asphalt milling before placing the overlay—differed from state to state. Further research was also needed because the dry and warm California climate differs considerably from those of all the other states with extensive experience using thin BCOA. A final need for further research existed because very few states have had experience with the use of rapid-strength concrete (RSC) mixes in thin BCOA, but such mixes will likely be used in thin BCOA for the fast-track construction that is a requirement on many state highways in California. For all these reasons, it was believed that further research was needed before the thin BCOA technique could be successfully implemented in California.

The University of California Pavement Research Center (UCPRC) investigation described in this report resulted from interest in the use of this rehabilitation technique that was expressed by the California Department of Transportation (Caltrans). The research project, titled Partnered Pavement Research Program Strategic Plan Element (PPRC SPE) 4.58B, "Development of Improved Guidelines and Designs for Thin Whitetopping," aims to address the differences and unresolved issues noted above. This report presents a summary of this research undertaken between September 2014 and September 2017.

The report is structured as follows. Chapter 2 describes the goals of the research and includes a number of questions the project sought to answer. Chapter 3 describes the project's research tools, the most remarkable of which was the Heavy Vehicle Simulator (HVS) used for full-scale accelerated pavement testing. Chapter 4 presents the project's conclusions, and Chapter 5 includes a series of recommendations that were formulated with the goal of optimizing thin BCOA for California conditions.

The main conclusions drawn from this research are that thin BCOA is a mature technique and, if good design and construction practices are met, it can potentially provide 20 years of good serviceability on most of California's non-interstate roadways. These conclusions are based mainly on the excellent performance of 11 thin BCOA sections that were tested with the HVS in Davis, California. These sections resisted the predefined HVS loading without cracking. On five of the sections, the loading was equivalent to 6 million equivalent single axle loads (ESALs) and included load levels more than twice the legal limit in California, channelized traffic at the edge of the slabs, and a continuous water supply that simulated flooded conditions. Because the predefined HVS loading failed to produce cracking in any of the sections, a decision was made to continue HVS loading on a section regarded as one of the test track's weakest. Corner cracking occurred in this section after 12 million ESALs were applied, most of them under extremely harsh conditions with channelized traffic at the edge of the slabs and simulated flooding.

2 STUDY GOAL AND OBJECTIVES

The main goal of this project was to develop recommendations and guidance on the use of thin BCOA as a rehabilitation alternative for California based on the adoption of, and improvements to, the technology developed in other US states. The objectives of the project were to find the answers to the following questions.

- How does thin BCOA construction differ from construction of conventional concrete pavements?
- Should current Caltrans rapid-strength concrete mix designs for new concrete pavement be used, or can they be optimized for thin BCOA?
- What are the fundamental mechanics of the concrete-asphalt interface?
- What are the structural and hygrothermal responses of thin BCOA to the ambient environment?
- What is the expected performance life of thin BCOA in California?
- What are the roles of the key design factors that determine thin BCOA performance?
- What is the performance of the asphalt base under the traffic and ambient environment actions?
- How should thin BCOA design methods be improved to consider faulting?
- What are the recommendations for the design of thin BCOA that consider California traffic, climate and materials conditions, and construction work zone practices?

These questions were answered by completing the following tasks:

- Review of the literature and discussions with national experts on BCOA
- Design and construction of a set of full-scale test sections and evaluation of their performance under Heavy Vehicle Simulator (HVS) and environmental loading
- Forensic investigation of the full-scale test sections after loading
- Laboratory testing of concrete and concrete-asphalt interface
- Finite element method modeling
- Analysis of all results to draw conclusions and develop recommendations

3 EXPERIMENTAL PLAN

3.1 Literature Review and Discussions with Experts

This project built on previous knowledge about thin bonded concrete overlay of asphalt (BCOA), which was reviewed through the published literature and in discussions with national experts on BCOA from industry and other universities. The results of the initial literature review were published in April 2015 in Reference *(1)*. In May 2015, a meeting was held in Davis, California, with Caltrans, industry, and other experts to discuss the literature review and preliminary results prior to beginning the main part of the work. The results of the project were reviewed with Caltrans, industry, and other experts in Woodland, California, in March 2017 to provide initial recommendations for pilot projects, and a workshop was held in Davis in February 2018 to review all the results and recommendations included in this report.

3.2 Thin BCOA Full-Scale Test Sections

Fifteen thin BCOA sections were built at the UCPRC facility in Davis from February 23 to 25, 2016. The sections were designed to complete a partial factorial experimental design that included eight factors: slab thickness, asphalt base thickness, asphalt surface texturing technique, type of asphalt mix, type of concrete, slab size, widened slab or granular shoulder, and concrete curing procedure. The levels of each of the eight factors are shown below:

1) Slab thickness:
    - 4.5 in. (115 mm)
    - 6 in. (150 mm)
2) Asphalt base thickness:
    - 2.4 in. (60 mm)
    - 4.7 in. (120 mm)
3) Asphalt surface texturing technique:
    - No texturing
    - Micromilling
    - Milling

    Note: Other states have typically used milling for BCOA. The difference between micromilling and milling is that the former uses a drum with more teeth and a smaller spacing between teeth than the latter. Micromilling can remove asphalt surfaces as thin as 1/3 in. (8 mm), and with a greater precision than standard milling.
4) Type of asphalt mix:
    - Conventional hot mix asphalt (HMA, aged existing mix placed in November 2012, which was just over three years old when overlaid with concrete, is referred to in this report as "old HMA")
    - Rubberized gap-graded asphalt (RHMA-G, new placement)

5) Type of concrete (each type of concrete is preceded by a shortened name by which it is referred to in this report):

   - P2: 10-hour design opening time (OT) with 800 lb/yd³ (475 kg/m³) of Type II/V portland cement and 0.33 water/cement (w/c) ratio
   - P2-ICC: 10-hour design opening time (OT) internally cured concrete based on the P2 mix that included pre-wetted lightweight aggregates as a replacement for 575 lb/yd³ (340 kg/m³) of the normal sand
   - P3: 4-hour design OT with 800 lb/yd³ (475 kg/m³) of Type III portland cement and 0.31 w/c ratio
   - CSA: 4-hour design OT with 680 lb/yd³ (405 kg/m³) of calcium sulfoaluminate (CSA) cement and 0.42 w/c ratio

   Notes:

   – Except for the internally cured mix, all the mix designs were similar to materials already validated for construction and rehabilitation of standard concrete pavements in California. Mix designs were provided by industry and slightly readjusted and validated based on results of laboratory testing conducted at the UCPRC to ensure that they were repeatable and that the materials produced in the UCPRC laboratory were representative of those in the field.

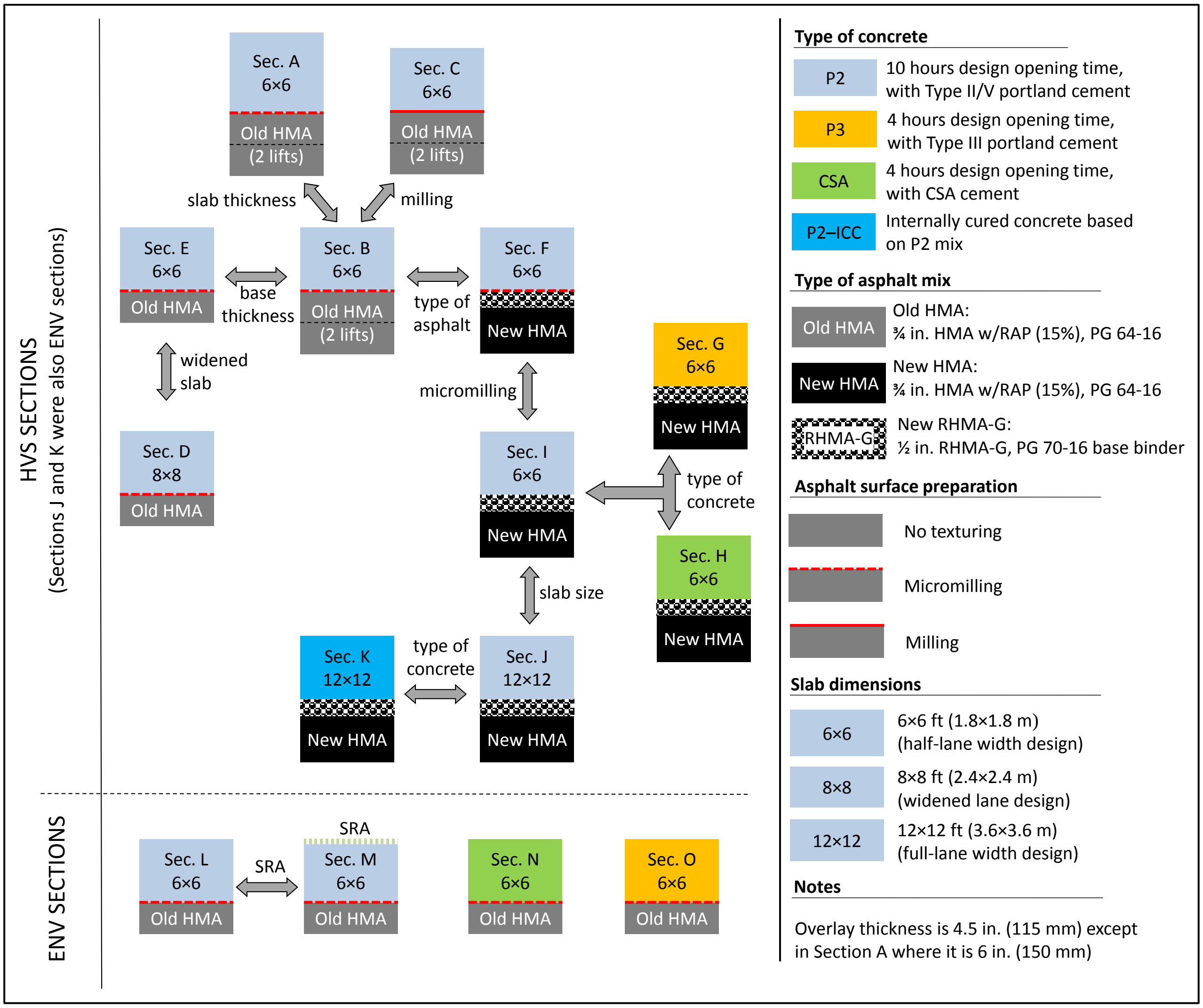
   – The Caltrans flexural strength requirement for opening time of RSC is 400 psi (2.8 MPa).

   – The design OT of the concrete mixes was based on the construction windows—overnight and 24 hr/weekend—established by Caltrans for BCOA. Specifically, a 4-hour design OT is required for mixes used in overnight road closures, and a 10-hour design OT is required for mixes used with 24 hour and weekend-long construction windows. Even though rapid-strength concrete was used in this project, this is not necessary for projects with longer construction windows; in fact, it is likely that the concrete mixes for most projects of that type will have a lower cement content and a larger w/c ratio than the portland cement mixes used in this research project.

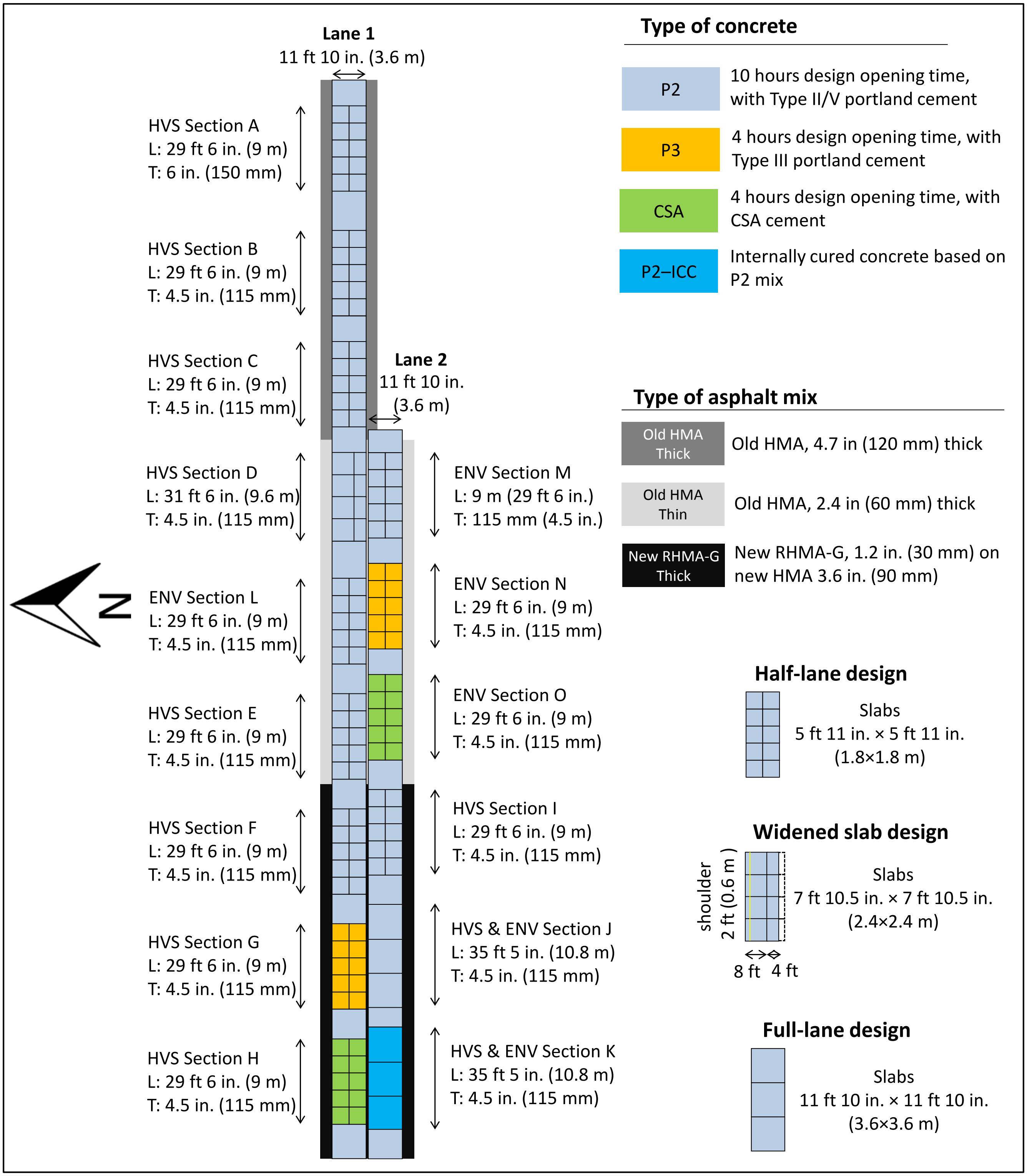
6) Slab size (in this report, each slab size is preceded by a shortened name used to refer to it):
   - 6×6: Half-lane width, 5 ft 11 in.×5 ft 11 in. (1.8×1.8 m) slabs
   - 8×8: Widened lane, 7 ft 10.5 in.×7 ft 10.5 in. (2.4×2.4 m) slabs
   - 12×12: Full-lane width, 11 ft 10 in.×11 ft 10 in. (3.6×3.6 m) slabs

7) Widened slab:
   - Granular shoulder
   - Widened slab extending into the shoulder

8) Concrete curing procedure:
   - Standard curing compound
   - Shrinkage-reducing admixture (SRA) sprayed before the curing compound application



***Factorial Design for the Full-Scale Test Track*. The HVS sections were tested with the Heavy Vehicle Simulator while the environmental (ENV) sections were used to monitor the structural and hygrothermal responses to the ambient environment.**



***Physical Layout of the Full-Scale Test Sections*. Nine of the sections were built on top of the old HMA that had been built and tested with the HVS for an earlier UCPRC project on full-depth reclamation. The condition of the old HMA was relatively poor, mainly due to permanent deformation caused by HVS testing. The remaining six sections were built on top of the rubberized gap-graded mix (RHMA-G) that was placed on October 2015, four months before the concrete overlay construction.**

Eleven of the thin BCOA sections were tested with the Heavy Vehicle Simulator. Those 11 sections are referred to as *HVS sections.* The effects of the ambient environment and cement hydration were monitored in six of the sections, referred to as *ENV sections.* Sections J and K are both HVS and ENV types.

Details of the designs and characteristics of the sections can be found in Reference *(2)*.

3.3 Thin BCOA Construction Evaluation

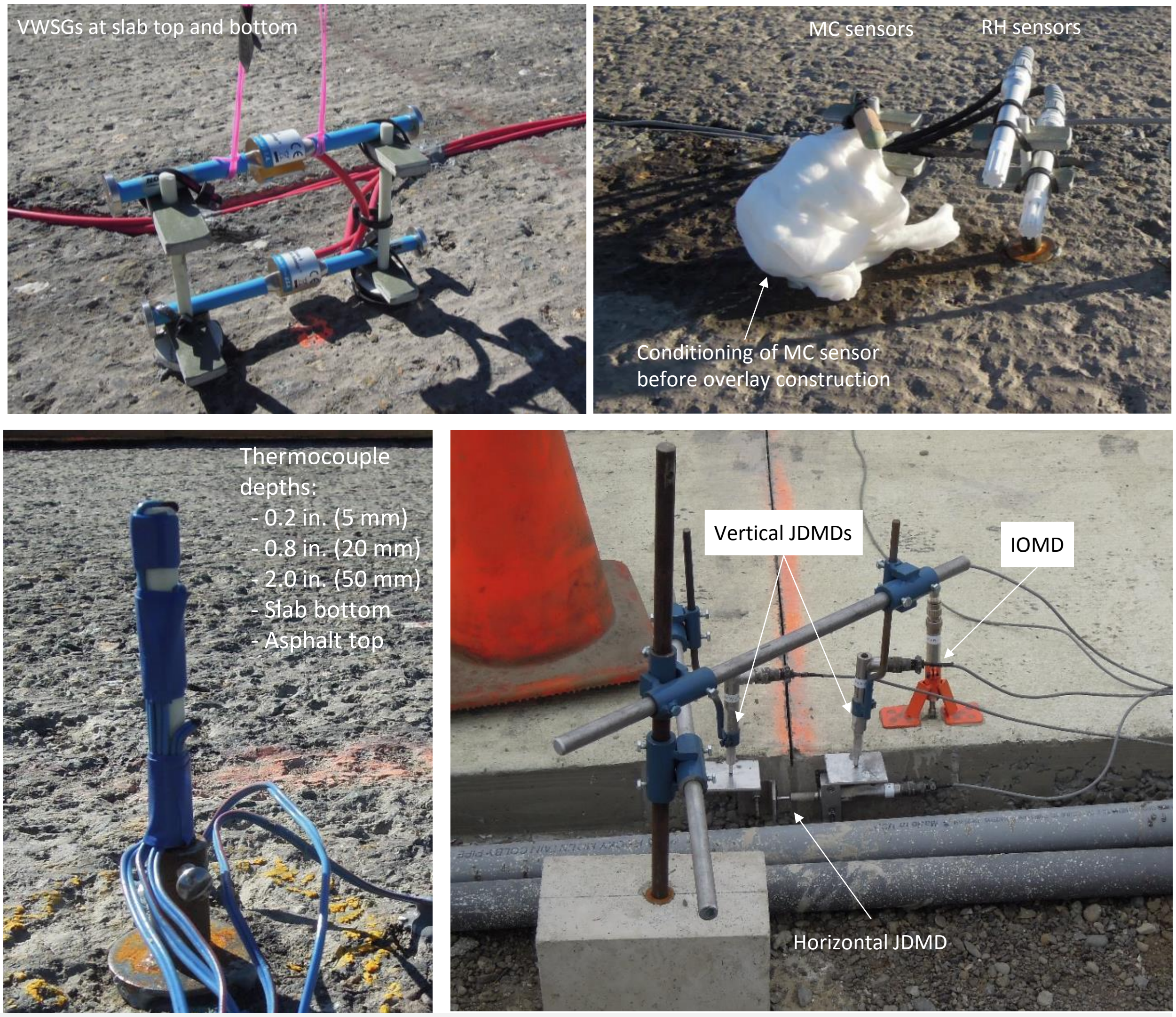
***Concrete Paving of the Full-Scale Sections*. A hand-operated roller screed was used for placement and consolidation of the concrete. This tool was selected because of the relatively small configurations of the concrete placements and the high workability of the mixes. Minimal additional vibration was required.**

Comprehensive QC/QA testing and evaluation were conducted during the construction of the thin BCOA sections in order to characterize the mixes used in the construction, to detect any potential problems associated with the thin BCOA construction, and to complete two short experimental studies. One of these short experimental studies was focused on using concrete compressive strength to predict flexural strength. The goal of the second study was to compare two methods used for measuring the flexural strength of RSC, ASTM C78-10 and California Test 524 (2013).

The results of the evaluation of the construction of the test track can be found in Reference *(2)*.

3.4 Monitoring of Thin BCOA Response to the Ambient Environment

The six environmental (ENV) sections were instrumented with 245 sensors to measure the structural and hygrothermal responses of the concrete slabs to the ambient environment and cement hydration. The structural response of the slabs was measured with vibrating wire strain gages (VWSG), vertical and horizontal joint displacement measuring devices (JDMD), and interface opening measuring devices (IOMD). The hygrothermal response was measured with thermocouples, relative humidity (RH) sensors, and moisture content (MC) sensors, all of them embedded in the concrete. Ambient conditions were measured by means of a weather station located a short distance from the test track.



Sensors Used for the Instrumentation of the ENV Sections. **Three Campbell Scientific data acquisition systems were used to collect the data on the environmental sections. In the initial phase of data collection, the sampling interval was two minutes. After March 3, 2016, the sampling interval was set to five minutes, and it was set to 20 minutes after November 1, 2016.**

Most of the 245 sensors began collecting data on the day before construction of the overlay, and data collection continued on a regular basis for approximately 15 months, up until May 31, 2017. Based on the analysis of those data, conclusions were drawn about how the different section configurations and concrete types responded to moisture- and temperature-related actions.

The overall analysis of the data collected from the ENV sections can be found in Reference *(3)*. Two additional reports go into detail about thermal deformations *(4)* and moisture-related shrinkage *(5)*, respectively.

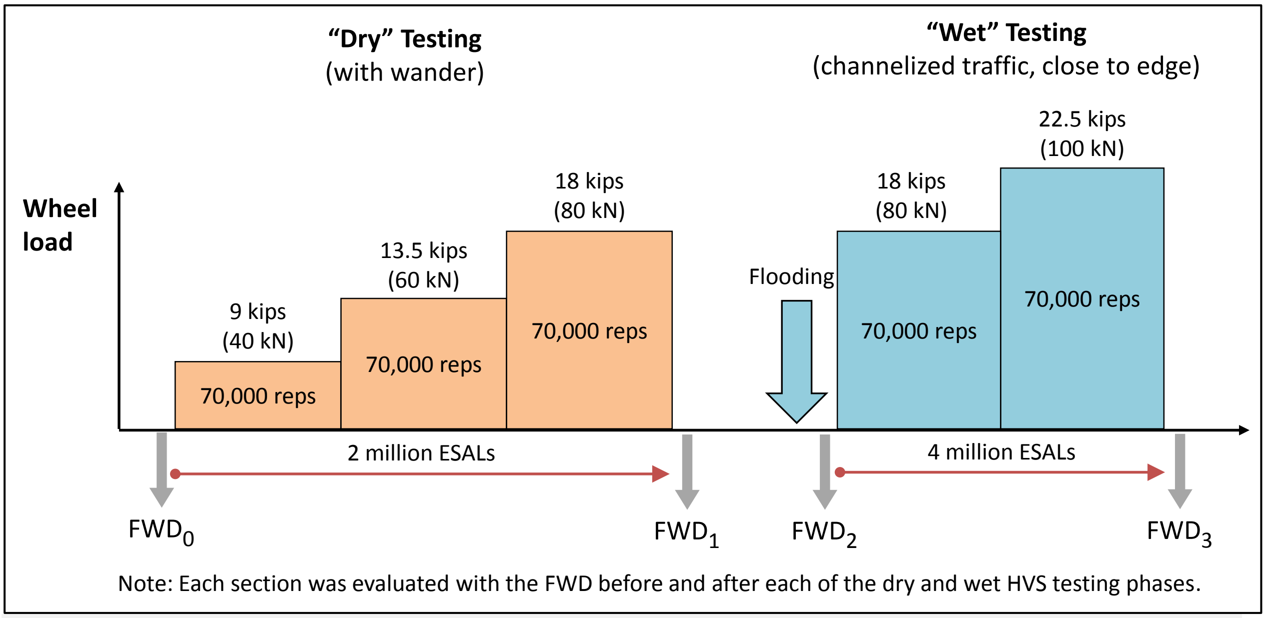
3.5 Accelerated Pavement Testing of Thin BCOA

Each of the 11 HVS sections was subjected to a "dry" loading sequence, and 5 of the 11 sections were subjected to a "wet" loading sequence after the dry sequence. In this project, loading under "dry" conditions means that the sections were tested under ambient conditions during summer, fall, and winter of 2016 that included rainfall events that occurred in the latter two seasons. During this testing, the precipitation was allowed to run off the pavement. For all but Sections I, J, and K the dry loading sequence consisted of three sets of 70,000 repetitions of the HVS wheel in order of increasing load: 9, 13.5, and 18 kip (40, 60, and 80 kN). For Sections I, J, and K, the two first loading sets (9 and 13.5 kip) were shortened to 15,000 repetitions. The dry loading included a 4.3 ft (1.3 m) wander pattern for the HVS wheel near the edge of the slab.

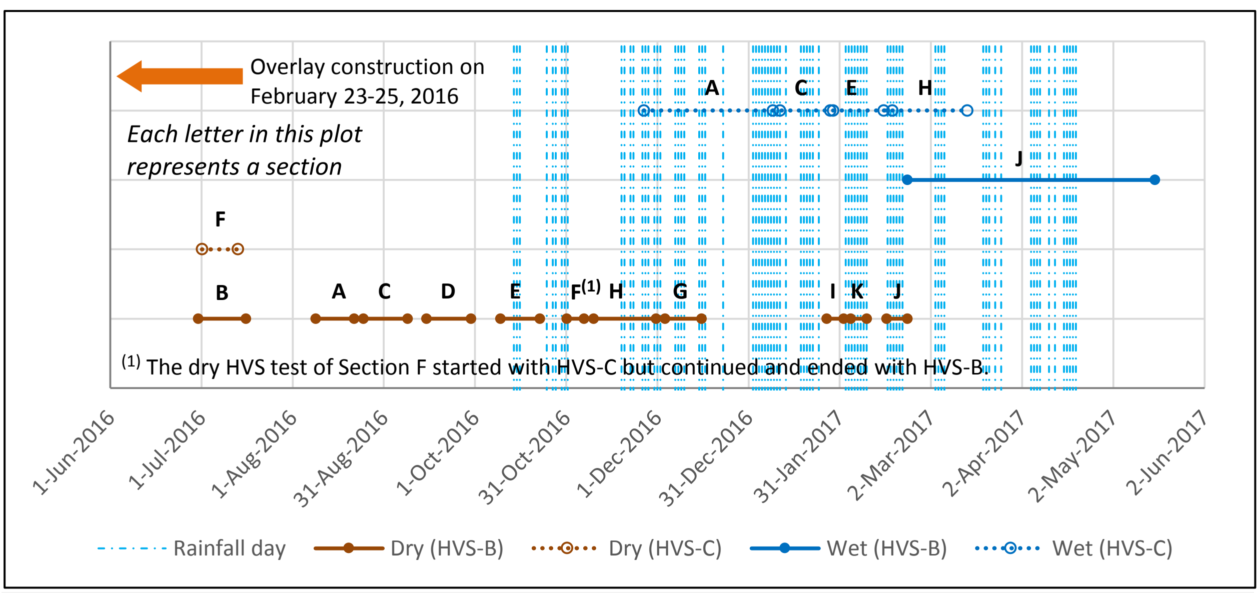
***Wet HVS Testing*. "Wet" conditions represented a pessimistic field scenario where a pavement drainage system either does not exist or functions very poorly.**

Testing under "wet" conditions consisted of a ten-day pre-HVS loading period when water was ponded on the pavement surface, followed by removal of the ponding and continual application of water on the pavement during HVS loading to maintain saturated conditions at the slab/base interface. The wet loading sequence applied to five of the sections consisted of two sets of 70,000 repetitions of the HVS wheel in order of increasing load: 18 and 22.5 kip (80 and 100 kN). The wet loading included channelization (no wander) of the wheel at the shoulder edge of the slabs, which is more damaging than traffic with wander.

Despite the large load levels (over twice the legal limit in California) and the unfavorable testing conditions (flooded section and channelized traffic at slab edge), the wet loading sequence produced no cracks in any of the five sections. For that reason, one of the sections (Section J) was subjected to further HVS loading under wet conditions with a 22.5 kip wheel load and channelized traffic at the shoulder edge of the slabs. Corner cracking finally occurred in the section after 360,000 total wheel passes, which was 120,000 additional repetitions beyond the loading on the other sections. Using AASHTO load equivalency factors, it was determined that this section supported 12 million ESALs before it cracked, although this determination considered neither the channelized traffic at slab edge nor the flooding conditions. After cracking developed, a further 20,000 repetitions of the 22.5 kip wheel load, equivalent to 1 million ESALs, were applied to Section J under wet conditions, again with channelized traffic at the shoulder edge of the slabs.



Scheme Followed for HVS Testing of the Sections. **Using AASHTO load equivalency factors, the dry HVS testing was the equivalent of 2 million ESALs. Similarly, the wet HVS testing was equivalent to 4 million ESALs, even without taking into consideration the channelized traffic at the edge of the slabs and the artificial flooding of the section. Consequently, 6 million ESALs (2 million dry + 4 million wet) is a lower-bound estimation of the equivalent traffic supported by the five sections tested in wet conditions after the dry HVS test.**



HVS Testing Schedule. **The two HVS Units (HVS-B and HVS-C) worked together to accomplish the HVS testing of the eleven sections in around one year.**

The performance of the sections tested with the HVS was monitored in terms of different response and distress variables. The evolution of structural response during HVS testing was measured using dynamic strain gages and JDMDs under the moving HVS wheel. The strains and deflections were also measured under the load of the Falling Weight Deflectometer (FWD) before and after the HVS testing for both the dry and wet testing. The load transfer efficiency (LTE) was measured in the mean wheelpath under the load of the FWD and in the outermost wheelpath under the load of the HVS wheel. In the latter case, the deflections were measured at the corners of the slabs next to the shoulder using JDMDs. The distresses monitored included cracking, faulting, and permanent vertical displacement of the slabs. No faulting developed in any of the sections since the HVS testing was conducted with bidirectional traffic and slow speeds.

***HVS Testing*. The structural response under the HVS wheel was measured with dynamic strain gages and JDMDs. In addition to measuring corner deflection, the JDMD readings were used to determine the load transfer efficiency (LTE) of the transverse joints under the passing HVS wheel.**

The analysis of the structural response and performance of the sections under the HVS loading led to a better understanding of the mechanics of thin BCOA structure. That analysis, included in Reference *(6)*, constitutes the primary basis for most of the conclusions from the project.

3.6 Forensic Investigation of Thin BCOA Distresses

A forensic investigation was conducted after all the sections had been tested with the HVS, around 15 months after the construction of the concrete overlay. The main goals of the forensic investigation were to evaluate the condition of the concrete-asphalt bonding and to determine the extent of distresses that were detected on the pavement surface or based on the structural response of the sections during the HVS testing. The main source of forensic data was a set of cores that were extracted through both the concrete and asphalt layers in each of the HVS sections. Another source of forensic data was a trench that was cut in Section J, the only section were cracking took place (Section J was tested with the HVS beyond the wet testing conducted on the other sections).

Apart from complementing the HVS testing results, the forensic work revealed some damage that could not be detected based on the structural response and performance of the sections during the HVS testing (details in Chapter 4). The forensic investigation is described in Reference *(6)*.

3.7 Laboratory Testing of Concrete and Concrete-Asphalt Interface

The mechanical properties of the four concrete mixes (P2, P3, CSA, P2-ICC) were characterized in the laboratory. The characterization was focused on stiffness, compressive strength, flexural strength, coefficient of thermal expansion (CTE), and drying shrinkage. For both stiffness and strength, the testing was conducted at different ages: design opening time (OT), 4×OT, and 45 days. All testing was conducted by ACI-certified personnel following the ASTM standards applicable at the time the research was conducted, with one exception. The exception was the drying shrinkage testing, which differed from ASTM C157 in terms of the length of the water-immersion period prior to drying (the beams were immersed in water until an age of 3 days instead of 28 days).

Shear Testing of Composite Specimen. **The shear performance of concrete-asphalt interface was tested with the Simple Shear Tester.**

The mechanical properties of the concrete-asphalt interface were also characterized in the laboratory. Some concrete testing was conducted after 4×OT, in order to determine the early-age mechanical properties of the interface, but most of the testing was conducted after the concrete had reached an age of 60 days. A large number of interfaces were characterized, based on the combinations of two concrete mixes (P2 and CSA) and five types of asphalt base (old HMA, micromilled old HMA, milled old HMA, RHMA-G, and micromilled RHMA-G). The characterization was focused on stiffness and strength/fatigue resistance in both shear and tensile modes. The characterization included consideration of the difference in timescales between traffic loading (fast) and environmental actions (slow).

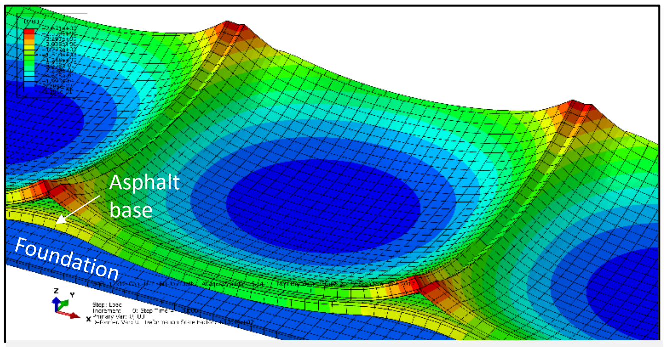
The mechanical properties of the concrete and the concrete-asphalt interface were the primary basis for some of the conclusions reached in the project. Those properties were also used to aid the analysis of the data collected from the HVS and the ENV sections. The results of the laboratory testing can be found in Reference *(7)*.

3.8 Finite Element Method Modeling

Finite element method (FEM) modeling was one of the tools used for the analysis of the data collected from the HVS and the ENV sections. It was also used to develop the mechanistic part of the faulting model. The FEM modeling was conducted with *Abaqus*™ software.

As a tool to aid the analysis of HVS testing results, the FEM was used to model the structural response of the sections under the HVS wheel *(6)*. The comparison between the theoretical and measured structural responses provided valuable information about any missing factors that had significantly impacted the structural capacity of each of the sections. Modeling was also used to determine the type and extent of the distresses that developed during the HVS testing, based on the change in structural response measured under the HVS wheel.

As a tool to aid the analysis of the data collected from the ENV sections, the FEM was used to model the response of the sections to temperature and drying shrinkage *(4,5)*. Concrete and asphalt creep/relaxation capacity was accounted for in the modeling of shrinkage effects.



***Section Response to Temperature Gradient Based on FEM Modeling*. The FEM model shown in the figure considers delamination between the concrete and asphalt at transverse joints for a 12×12 section.**

As a tool to develop the mechanistic part of the faulting model, the FEM was used to determine the deflections under single and tandem axles located at the transverse joints *(8)*. The computed deflections in the approach and leave slabs were used to determine the differential deflection energy, which was regarded as the critical response variable that determines the development of faulting.

4 SUMMARY OF CONCLUSIONS

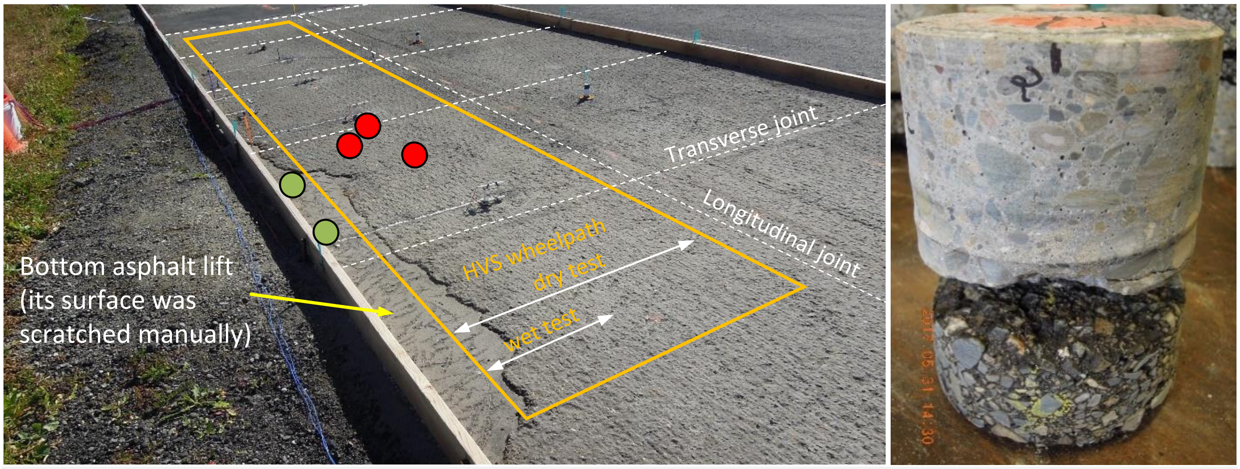
4.1 How Does Thin BCOA Construction Differ from Construction of Conventional Concrete Pavements?

- No major incidents occurred during the construction of the thin BCOA test tack.
- The topical use of a shrinkage-reducing admixture (SRA), 7.8 oz/yd² (275 ml/m²), prior to the application of the curing compound, reduced the drying shrinkage of the Type II/V portland cement mix considerably.

Application of topical SRA Spray. **SRA was applied right after the "shine" on the concrete surface disappeared. The timing matches that for the application of standard curing compound.**

  - Compared to the section that was only treated with curing compound, the SRA spray-treated section showed a considerable reduction of the differential drying shrinkage in the slabs (top versus bottom)—around 50 percent soon after construction and around 25 percent during the first summer after construction of the overlay.
  - The SRA effect was also verified in terms of an increase in concrete relative humidity (RH) measured at 0.8 in. (20 mm) depth.

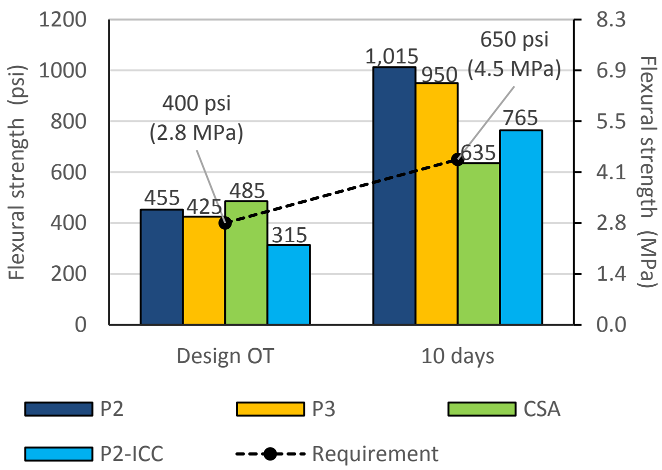
- All transverse joints had deployed in all the sections by the end of the first summer after the overlay construction.
  - This high level of transverse joint deployment was due to the high drying shrinkage levels that all the mixes experienced.
  - This high level of transverse joint deployment contrasts with experience in wetter US states where weather conditions do not trigger deployment of all the transverse joints of thin BCOA.
  - There was a clear link between transverse joint deployment—seen in visual assessments—and slab bending up and down due to thermal gradients—determined from VWSG measurements. In most cases, the VWSG data anticipated the results of the visual assessments.
- Delamination between the two lifts of old HMA occurred during the milling operation in Section C.
  - The delamination took place in an area that had not been subjected to any heavy traffic.
  - The delamination was believed to have occurred because the milling depth was relatively close, approximately 1.2 in. (30 mm), to the interface between the two asphalt lifts.
  - No delamination occurred during the micromilling operation in the other sections with two lifts of old HMA, an outcome that was related to the relatively shallow micromilling depth, 0.6 in. (15 mm), and to the less aggressive nature of micromilling compared to milling.



***Evaluation of Bonding from Forensic Cores*. Seven of the ten forensic cores extracted from Section C, where milling was used, showed either concrete-asphalt debonding or failure in the asphalt just below the interface. This occurred even with cores extracted far from the corners of the slabs (indicated above in red). At those locations, no damage could be seen in the sections with micromilled old HMA. Two of the three cores where the asphalt and the interface were in good condition (green cores in the picture) corresponded to asphalt with almost no texturing at all.**

4.2 Should Current Caltrans Rapid-Strength Concrete Mix Designs for New Concrete Pavement Be Used, or Can They Be Optimized for Thin BCOA?

- The concrete mixes placed in the test sections met the Caltrans requirements included in Section 40 of the 2015 Standard Specifications and in the Standard Special Provisions 40-5 applicable to jointed plain concrete pavements built with rapid-strength concrete, with a few exceptions where they were close to the specified requirements.
  - The minimum flexural strength requirement of 400 psi (2.8 MPa) was fulfilled by all the mixes at the design opening time, with the exception of the Type III cement concrete placed the second construction day. The actual opening time of this mix was slightly over the design opening time of four hours.
  - All the portland cement-based mixes far exceeded the Caltrans 10-day flexural strength requirement of 650 psi (4.5 MPa). The flexural strength of these mixes increased more than 100 percent from the design opening time to 10 days.
  - The increase in flexural strength from design opening time to 10 days was much smaller in the CSA-based concrete than in the mixes with portland cement. The 10 day flexural strength of the CSA-based concrete placed the first construction day was slightly below 650 psi (4.5 MPa), the Caltrans minimum requirement.



***Concrete Flexural Strengths at Opening Times and at 10 Days*. Flexural strength values in the figure are the average of the two construction days, compared with OT and 10-day specifications. On each construction day, three beams per mix type were tested following ASTM C78-10. Note design opening times (OT): 4 hours for P3 and CSA mixes; 10 hours for P2 and P2-ICC mixes.**

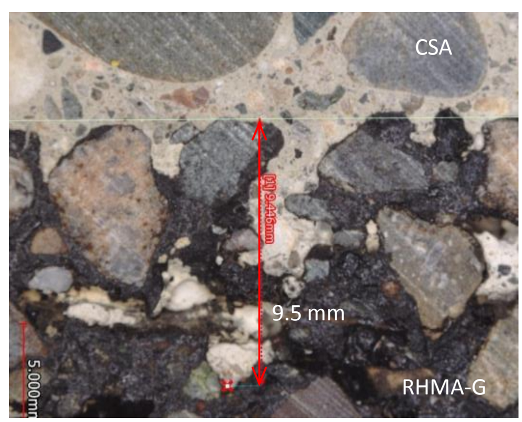
- No durability or mechanical problems were found for any of the mixes except for the internally cured mix (with Type II/V cement), where environment-related surface microcracking was observed after one year.
- The replacement of 50 percent of the sand in the Type II/V cement mix with pre-wetted lightweight aggregates was effective in reducing the autogenous shrinkage in the internally cured mix (P2-ICC), to the extent that it barely occurred. However, the mechanical and drying-susceptibility properties of the internally cured mix were worse than the properties of the original mix upon which the design of the internally cured mix was based.
    - The relatively poor performance of the internally cured field mix is believed to have been caused by a relatively high water content. The relatively high water content was not necessarily attributable to the use of pre-wetted lightweight aggregates. In any case, because the flexural strength of the internally cured field mix far exceeded the 10-day flexural strength requirement of 650 psi, this mix would have fulfilled Caltrans specifications, although the opening time would have been longer than 10 hours (this field mix reached 400 psi flexural strength after 14 hours).
    - Compared to the original mix, the flexural strength of the internally cured mix was around 30 percent smaller at an age of 10 hours and around 25 percent smaller after 10 days. This decrease was much higher than expected, based on results from similar experiments published in the literature and the testing of the same mix prepared in the UCPRC laboratory. Based on the internally cured mix prepared in the laboratory, the reduction in flexural strength at an age of 10 hours was 16 percent compared with the same mix without internal curing.
    - The differential drying shrinkage (top versus bottom of the slab) developed faster in the slabs made with internally cured concrete than in the slabs made with the same mix without internal curing. The greater drying susceptibility of the internally cured mix, compared to the original mix, was also verified based on RH measured in the slabs at 0.8 in. (20 mm) depth. Again it must be emphasized that this result may be related to a high water content in the mix placed in the test sections and is not inherent to internally cured concrete.
- The mixes with Type II/V cement (P2 and P2-ICC) presented higher concrete and asphalt temperatures and higher positive temperature gradients (top warmer than bottom) than the mixes with

CSA and Type III cements. This outcome was true not only after the construction of the overlay but also during the 15-month period analyzed for this project. This outcome was attributed to the higher albedo (higher capacity for reflecting solar radiation) of the mixes with the CSA and Type III cements than the mixes with Type II/V cement.

4.3 What Are the Fundamental Mechanics of the Concrete-Asphalt Interface?

- Experimental results from this study show that concrete bonds well to new asphalt.
  - The interface between the concrete and new rubberized gap-graded asphalt (RHMA-G) had similar tensile and shear strengths to those of the new RHMA-G itself.
  - The fatigue damage of the concrete-new RHMA-G composite specimens tested in the laboratory under repeated shear loading primarily occurred in the asphalt, and not in the interface.
  - The performance of all the 6×6 thin BCOA sections with new RHMA-G was excellent, and in some ways the new RHMA-G improved the performance of the 6×6 thin BCOA sections on milled or micromilled old HMA.

***Tensile Testing of Composite Specimen.* The composite specimens with new RHMA-G tended to fail in the asphalt rather than at the interface.**

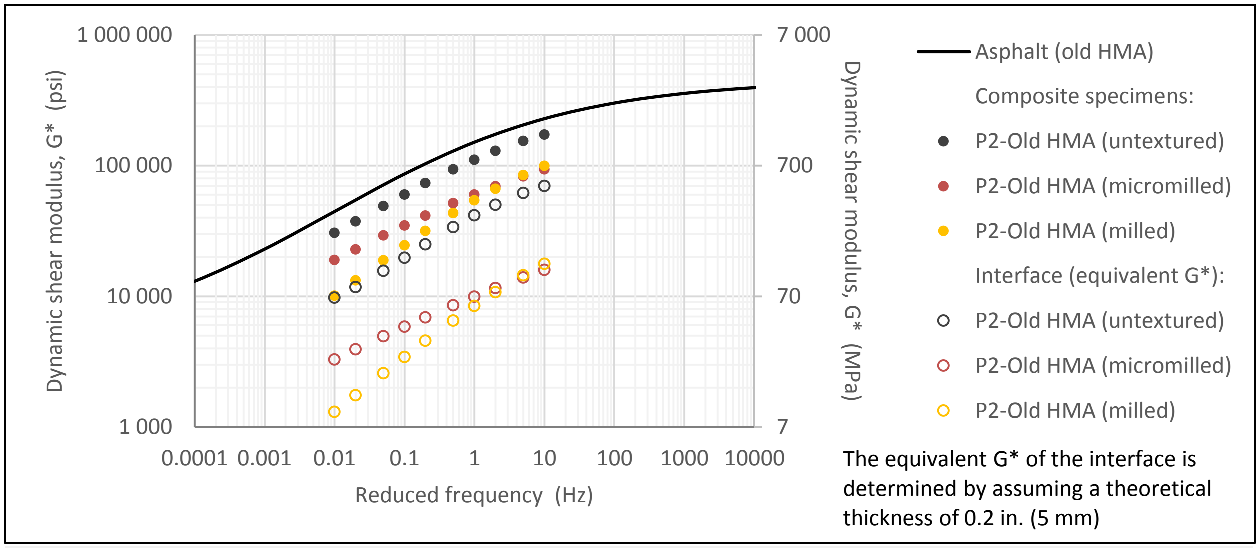


***Concrete-Asphalt Interface.* Note the penetration up to 10 mm of the cement paste into the RHMA-G voids.**

- Based on the laboratory testing results, it was concluded that the mechanical nature and properties of the concrete-asphalt interface were strongly related to those of the asphalt. Two facts support this conclusion: first, the stiffnesses of the tested interfaces clearly showed temperature- and time-dependence, as do viscoelastic asphalt bases; and second, the interfaces softened significantly under wet conditions, as can occur with asphalt mixes.
- The cement paste penetrated up to 0.4 in. (10 mm) into the RHMA-G voids, thus creating a mechanical bond. The maximum penetration of the cement paste was much smaller, less than 0.1 in. (2 mm), in the micromilled old HMA. The mechanical bond includes a reinforcing effect in the volume of the asphalt base that has been penetrated by the cement, particularly the

RHMA-G. This is believed to be one of the reasons for the good bonding between concrete and that asphalt mix.

- Laboratory results from this study reinforce the generally held opinion that the presence of water is one of the critical factors that leads to the failure of thin BCOA sections because it damages the interface and the asphalt.
- Neither milling nor micromilling improved the bonding between the concrete and the old HMA compared with no treatment other than sweeping (referred to as "untextured").
  - The shear strength of the interface between the concrete and untextured old HMA, based on the Iowa shear test, dropped around 50 percent when the old HMA surface was either milled or micromilled.
  - The shear stiffness of the interface between the concrete and untextured old HMA, based on frequency sweep testing at 77°F (25°C), dropped around 75 percent when the old HMA surface was either milled or micromilled. The lower the shear stiffness of the interface, the more the concrete-and-asphalt composite system will act as two completely debonded layers.



***Master Curve of Interface Stiffness as a Function of Frequency of Loading*. The stiffness of the interface was determined from the stiffness of asphalt and composite specimens. The "equivalent" stiffness of the interface (shown in the figure) is backcalculated by assigning a theoretical thickness of 0.2 in. (5 mm) to the interface, the maximum depth of penetration of cement into the old HMA. Higher frequency equals faster loading, which results in higher interface stiffness, like an asphalt mix.**

- The shear stiffness and shear fatigue resistance of the concrete-asphalt composite specimens were similar at 4×OT (four times the design opening time of the concrete mix) to the same properties measured at 60 days after concrete placement. This outcome suggests that the interface develops its mechanical properties as fast as—or even faster than—the concrete itself, although further experimental data are needed to fully verify this outcome.
- Because of the viscoelastic nature of the asphalt mixes, they react as a much stiffer material under the rapid traffic loading than under the slow environmental actions (thermal and moisture-related shrinkage). The stiffness of asphalt can be expected to differ by at least one order of magnitude between the two loading scenarios. This behavior needs to be considered in BCOA design procedures.

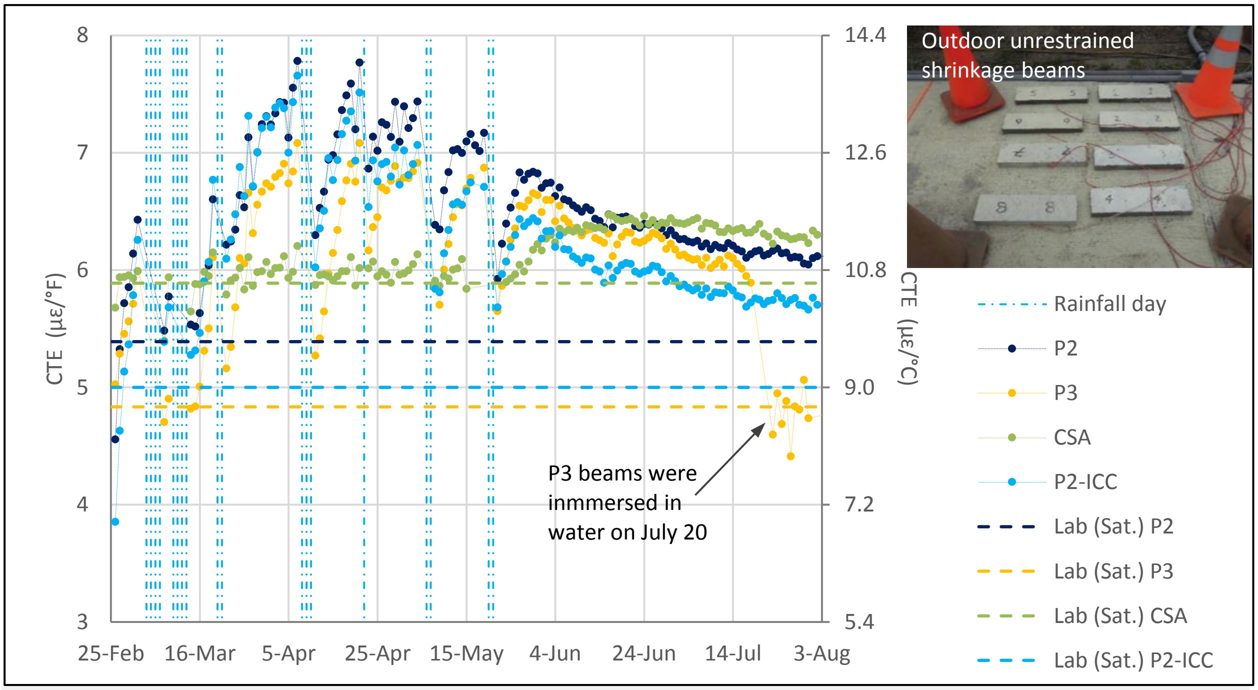
Otherwise, slab thermal displacements and deformations will be underestimated considerably while slab thermal stresses will be overestimated considerably.

4.4 What Are the Structural and Hygrothermal Responses of Thin BCOA to the Ambient Environment?

The conclusions regarding the response of the slabs to thermal and moisture conditions have been grouped into the following seven subsidiary questions.

Do concrete moisture conditions impact the thermal response of the concrete pavements? And if so, by how much?

- Because of the drying of the thin BCOA slabs in the test track, the CTE of the mixes with portland cement increased up to 60 percent versus saturated conditions as backcalculated from measured strains and deflections on the slabs in the test sections. Standard laboratory conditions test CTE under saturated conditions, which means that the CTEs measured in the test sections were much higher than those measured in the laboratory under standard conditions. The maximum increase in CTE, versus saturated conditions, was only around 15 percent in the mix with CSA cement.
- Because of the increase in CTE, the thermal expansion-contraction and bending of the thin BCOA slabs were much higher than predictions based on the CTE determined in the laboratory under saturated conditions following AASHTO T 336-15. Actual thermal bending of the portland cement slabs was up to 65 percent greater than that predicted using the saturated CTE. For the CSA slabs, actual bending was up to 20 percent greater than predicted using the saturated CTE.
- The equivalent (apparent) CTE of the concrete slabs followed the same pattern that has been reported in laboratory experimental studies since early work by Meyers in the 1950s. More specifically, as the concrete dried, the equivalent CTE increased until it reached a maximum. Beyond that maximum, the equivalent CTE decreased with further drying, reaching values of around 5 to 10 percent larger than the saturated CTE.
- The changes in equivalent CTE of the concrete slabs were directly related to weather conditions.
  - The equivalent CTE tended to increase during periods without rainfall.
  - Rainfall events produced an immediate reduction of the equivalent CTE, provided that the concrete was not already saturated.
  - A clear link was observed between equivalent CTE of the slabs determined from their bending and RH measured in the concrete at 0.8 in. (20 mm) depth: the equivalent CTE increased when RH decreased and vice versa.
- The results obtained from the concrete slabs are supported by the results obtained with eight unrestrained shrinkage beams left outdoors that were prepared using the same mixes and following the same curing procedures used in the BCOA sections.



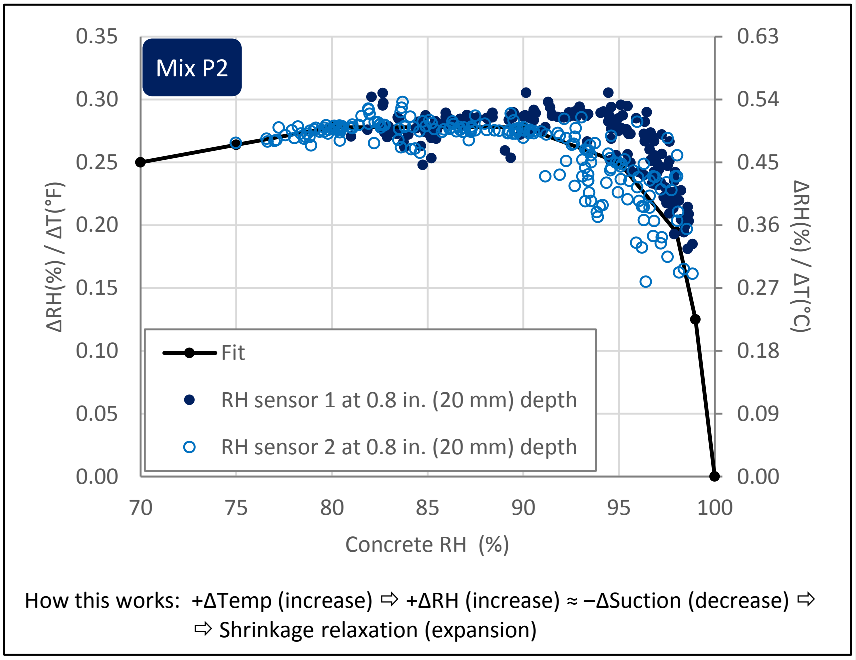
Effect of the Ambient Environment on Concrete CTE. **The CTE of the mixes with portland cement (P2, P3, P2-ICC) tended to increase during periods without rainfall and dropped rapidly after each rainfall event (the series "Rainfall day" indicates days when rainfall took place). Note how the CTE reached maximum values around May to June while, with further drying, the CTE decreased below that maximum. Actual CTE of the beams reached maximum values that were much larger than the value determined in the laboratory following AASHTO T 336-15 (under saturated conditions).**

Why does the CTE of the concrete change with changing moisture conditions?

- The experimental results obtained in this research support the hypothesis that the moisture dependence of concrete CTE is related to a group of mechanisms that result in temperature-related changes in the concrete's degree of capillary saturation and capillary water suction, while the amount of water in the concrete remains constant. In this report, these mechanisms have been referred to as *suction-temperature mechanisms.*

- Based on the RH measured in the thin BCOA slabs at 0.8 in. (20 mm) depth, it was concluded that the RH in the air pores of the portland cement mixes changed as much as +0.3 percent per degree Fahrenheit (+0.5 percent/°C) while the water content in the concrete remained constant. The plus (+) sign indicates that concrete RH increased as temperature increased and vice versa. The slope of the relationship between RH and temperature, ΔRH/ΔTemp, was found to be a mix-dependent function of concrete relative humidity.

- The temperature-related changes in the concrete's degree of capillary saturation and capillary water suction resulted in intensification or relaxation—depending on direction of the temperature change—of the moisture-related shrinkage (i.e., they resulted in concrete contraction/expansion). Since this deformation takes place as the temperature changes, it is interpreted as thermal deformation. In fact, however, it is temperature-dependent moisture-related shrinkage.



Change in Relative Humidity in the Air Pores with Change in Temperature**. Because of the suction-temperature mechanisms, the RH in the air pores in the concrete increased as temperature increased and vice versa even though the water content was essentially constant. That is the opposite of what happens in the open air when the amount of water vapor in the air is constant and RH decreases as temperature increases. For example, based on the figure, the P2 concrete RH would have increased from 85 to 85.27% when the temperature increased 1°F and would have decreased to 84.73 when the temperature decreased 1°F.**

How much do thermal stresses change because of the moisture dependence of concrete CTE?

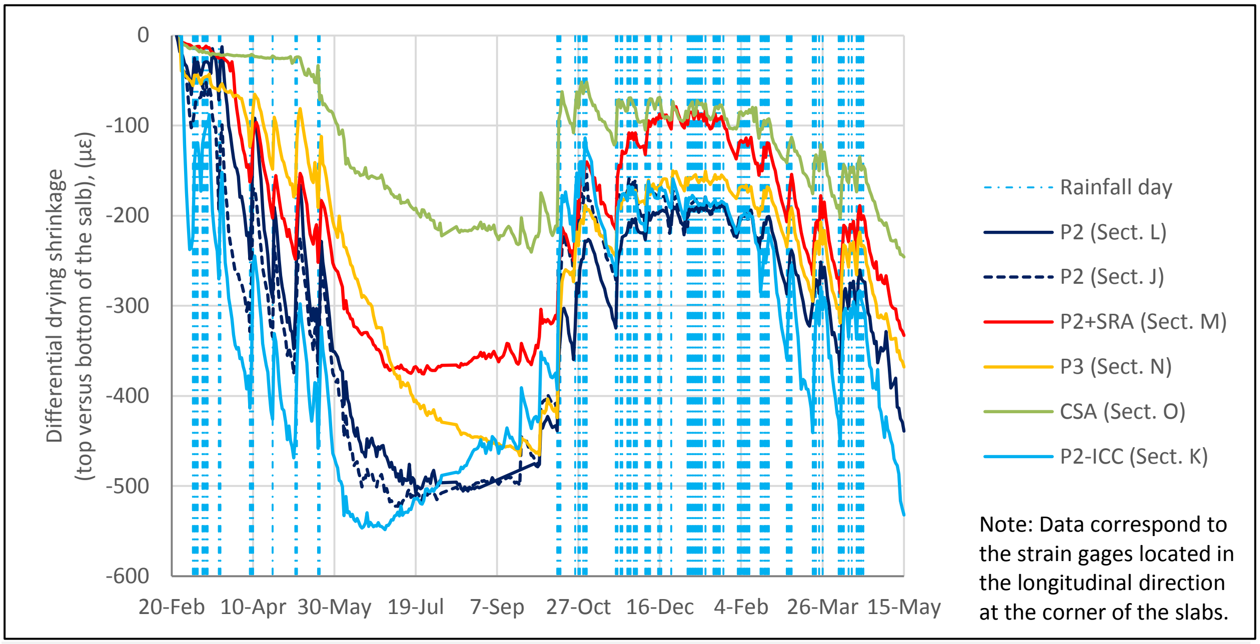
- The moisture-dependent nature of concrete CTE ensures that the CTE will not be constant versus depth because concrete moisture conditions typically vary with depth.
- Modeling with the finite element method indicated that the maximum tensile thermal stress with the actual CTE in the slabs with portland cement increased up to 70 percent compared to a scenario where the CTE of the slabs was constant versus depth and equal to the saturated CTE. The increase in thermal stress was around 25 percent in the slabs with CSA cement.

The conclusions regarding the response of the slabs to moisture-related shrinkage have been grouped into the four main questions this research intended to answer.

What moisture-related shrinkage takes place in the BCOA slabs?

- Very high levels of differential drying shrinkage (top versus bottom of the slab) were backcalculated in all of the sections with portland cement mixes treated with curing compound alone and no topical SRA treatment, with values as high as 450 to 550 με, depending on the mix.
- Strains of around 200 to 250 με due to autogenous shrinkage were backcalculated in the mixes with Type II/V (P2) and Type III (P3) portland cement and water/cement ratios of 0.33 and 0.31, respectively. No autogenous shrinkage was measured in the internally cured mix (P2-ICC) or in the mix with CSA cement.
- The total moisture-related shrinkage strain values at the top of the slabs with portland cement mixes treated with curing compound reached as high as 600 to 750 με, depending on the mix.

- Rainfall events produced an almost immediate decrease in the magnitude of the differential drying shrinkage in the slabs except, as expected, when the concrete surface was already saturated. This occurred with both the portland cement and CSA mixes.
- The evolution of mean and differential drying shrinkage in the slabs indicated that drying affected the bottom half of the slabs considerably during summer. This conclusion was supported by moisture content measurements at 2 in. (50 mm) depth in the concrete. Since the slabs were 4.5 in. (115 mm) thick, this outcome questions the *Mechanistic-Empirical Design Guide*'s (MEPDG) hypothesis that drying does not take place below 2 in. (50 mm).



***Difference in Drying Shrinkage between Top and Bottom of Slab*. This figure shows the differential drying shrinkage between the top and bottom of the slab, with negative values indicating that the top has more shrinkage than the bottom. It can also be seen that drying shrinkage is partially reversed by rainfall events. These data were backcalculated from strain gage measurements in the slabs. Details of the calculations are in Reference *(5)*.**

Do shrinkage prediction models work for RSC mixes?

Several commonly used models for shrinkage were compared with the values measured in the test sections and beams cast next to the slabs.

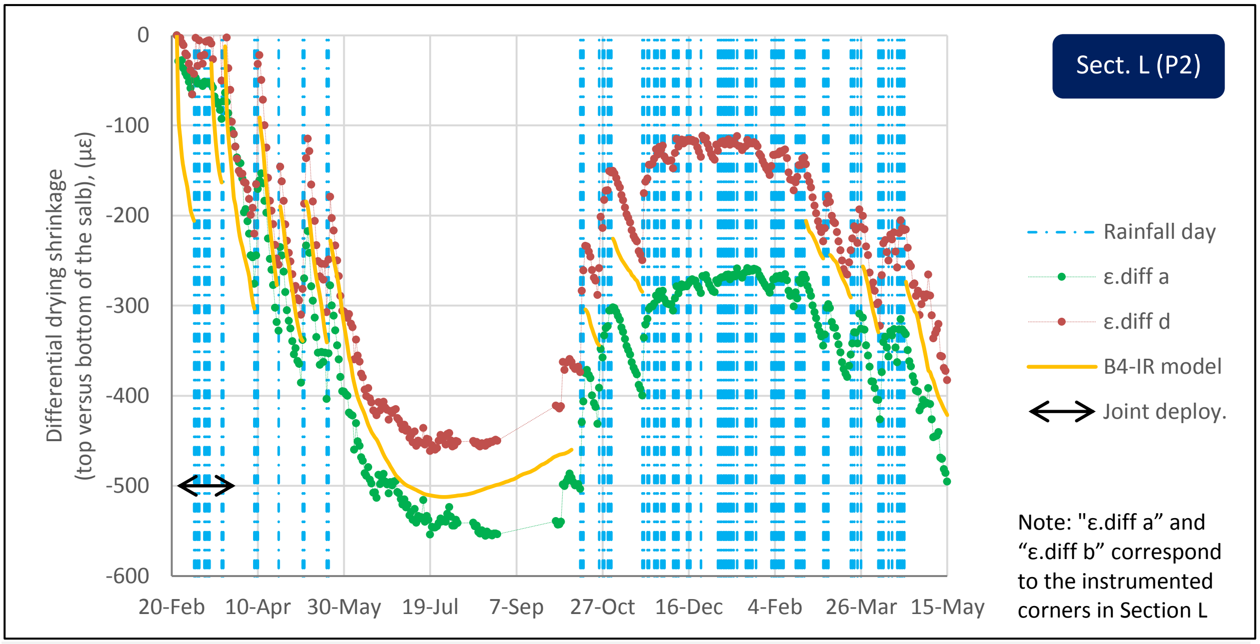
- The commonly used models referred to as "RILEM B3 and B4 shrinkage prediction models" underestimated laboratory shrinkage considerably in the mixes with Type II/V (P2) and Type III (P3) portland cement.
- The B4 model did not improve on the predictions of the B3 model, even though the B4 model is more recent and is applicable to concrete mixes with admixtures, as P2 and P3. The B4 model predictions were particularly inaccurate as far as autogenous shrinkage was concerned.

- Overall, the shrinkage predictions made by another commonly used model referred to as the "ACI 209R-92 model" were not far off from the shrinkage measured in the laboratory, even though this model was not developed for mixes with admixtures.
- None of the B3, B4, or ACI 209R-92 shrinkage prediction models are applicable to mixes with CSA cement mix or to the internally cured mix (P2-ICC).

What is the relationship between lab and field shrinkage?

A new shrinkage prediction model was formulated in this project. The model, referred to as the "B4-incremental-recursive" model (B4-IR), is based on 1) the B4 lab-calibrated model, 2) the *CalME* time-hardening incremental-recursive approach, and 3) a simplified procedure to account for shrinkage reversals due to relatively high air RH.

- The B4-IR model's predictions almost exactly reproduced the moisture-related shrinkage measured in a set of outdoor unrestrained shrinkage beams with portland cement (mixes P2, P3, and P2-ICC).
- The B4-IR model's predictions almost exactly reproduced the differential drying shrinkage in the slabs with portland cement (mixes P2, P3, and P2-ICC), although in this case a slight modification of the B4 parameters was required.
- The B4-IR model failed to reproduce the shrinkage measured in the outdoor unrestrained shrinkage beams made with CSA cement and in the slabs made with the same material.
- The good agreement between the B4-IR model's predictions and measured shrinkage—for the mixes with portland cement—indicates that a direct link between lab and field shrinkage can be established as soon as several factors are taken into account: 1) the age difference between the concrete in the lab and the concrete in the field when drying begins; 2) the shape and volume/surface ratio differences of the lab specimens and the field slabs/concrete members; 3) the constant temperature and air RH in the lab versus the variable conditions in the field; and 4) the monotonic drying in the lab versus the alternating drying/wetting periods in the field—due to rainfall events and changing air RH.
- In the B4-IR model, the unrestrained shrinkage profile in the slabs was assumed to be constant at the surface while the depth of shrinkage penetration was assumed to change. The B4-IR model failed to reproduce the shrinkage measured in the slabs when the *MEPDG* unrestrained shrinkage profile assumption was followed (depth of drying is constant while the unrestrained shrinkage at the slab top is the one that changes as slab concrete dries).
- Shrinkage reversals due to rainfall events were not modeled in this research. This topic should be investigated in the future since experimental data show shrinkage in the concrete slabs cannot be predicted based on air RH exclusively.



***Prediction of Drying Shrinkage with the B4-IR Model*. The B4-IR model almost exactly reproduced the backcalculated differential drying shrinkage of the slabs. The main limitation of B4-IR currently is that it cannot model shrinkage reversal after rainfall events (model predictions were forced to fit actual measured strain after each of the rainfall events).**

What is the stress due to moisture-related shrinkage?

- FEM modeling of the drying shrinkage action following the standard mechanistic-empirical design practice for concrete pavements resulted in very high and unrealistic tensile stresses at the top of the slabs. That standard practice ignores the creep/relaxation capacity of concrete and asphalt.
- The FEM modeling resulted in realistic stresses at the top of the slabs when the creep/relaxation capacity of concrete and asphalt were accounted for. In particular, the stresses were compatible with the microcracking observed in one of the 12×12 sections and with the lack of microcracking in the 6×6 sections.
- Asphalt creep/relaxation reduced the stresses created by the linear component of moisture-related shrinkage. Because of this property of asphalt, the total tensile stresses were reduced by 55 percent in the 6×6 sections and by 40 percent in the 12×12 sections.
- Concrete creep/relaxation mainly reduced the stresses created by the nonlinear component of the moisture-related shrinkage.
- In addition to the creep/relaxation capacity of concrete and asphalt, surface microcracking acted as a concrete stress-release mechanism in at least one the 12×12 sections, the one with P2-ICC mix. No discrete cracking was observed in that section, which also supported the Heavy Vehicle Simulator (HVS) testing with loads up to 18 kip (80 kN) on a single wheel without cracking.

4.5 What is the Expected Performance Life of Thin BCOA in California?

- Overall, the performance of most of the 6×6 sections during the HVS testing was excellent: no cracking developed, minimum slab rocking occurred (shoulder corner settlement was below 0.04 in. [1 mm]), and LTE decreased minimally—remaining over 80 percent in the center of the wheelpath.
- Sections C and E were the exceptions to the excellent performance of the 6×6 sections. Section C performed well during the dry HVS testing, but during the wet HVS testing LTE in the center of the wheelpath dropped below 50 percent and the shoulder corner settlement reached 0.04 in. (2 mm). On Section E, LTE in the center of the wheelpath dropped below 50 percent during the dry HVS testing. The distinctive characteristics of Section C were the surface preparation of its old HMA base—using milling instead of micromilling—and the relatively bad condition of its base. The distinctive characteristic of Section E was the reduced thickness of its old HMA base, which was 2.1 in. (54 mm), on average, after micromilling. In any case, neither Section C nor Section E presented any cracking during the HVS testing, even though they were tested in both dry and wet conditions.
- Using AASHTO load equivalency factors, the sections tested with the HVS only under dry conditions supported traffic of around 2 million equivalent single axles (ESALs) while the sections that were also tested in wet conditions supported a total traffic of around 6 million ESALs. The latter is a lower bound estimate of the equivalent traffic since the wet HVS testing was conducted under very unfavorable conditions (flooded section and channelized traffic at the shoulder edge of the slabs).
- Using AASHTO load equivalency factors, Section J supported around 12 million ESALs before corner cracking occurred. Section J was a 12×12 design with 4.5 in. (115 mm) slab thickness and Type II/V cement concrete. It is the section on which HVS testing under wet conditions went the longest.
- Section J received around 1 million ESALs after the corner cracking occurred, and the section's postcracking performance was excellent: slab movements were negligible, the broken corners remained bonded to the asphalt base, and the corner cracks did not propagate through the RHMA-G.

No Propagation of Slab Cracks through RHMA-G on Section J. **The fact that the corner cracks did not propagate into the RHMA-G was attributed to the high fatigue resistance of this asphalt mix.**

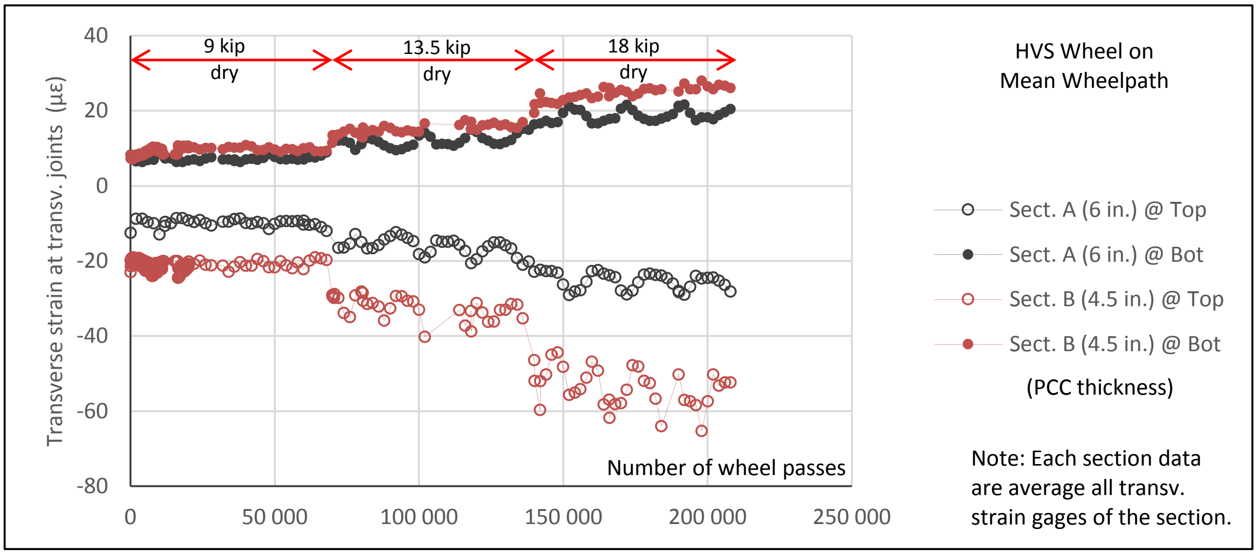
- Based on the results of the HVS testing, preliminary indications are that a well-designed, well-built 6×6 bonded concrete overlay placed on top of an asphalt base that is in fair to good condition and has not been damaged by milling can potentially provide 20 years of good serviceability on most of California's non-interstate roadways. Nonetheless, uncertainty remains about the long-term condition of concrete-asphalt bonding. This topic could only be partially evaluated in the HVS experiment because the evaluation was performed in short-term but extreme, accelerated wet testing conditions.

4.6 What are the Roles of the Key Design Factors that Determine Thin BCOA Performance?

Slab Thickness

The role of the factor *slab thickness* was evaluated by comparing the performance of Section A, where the slab was 6 in. (150 mm) thick, against the performance of Section B, where the slab was 4.5 in. (115 mm) thick.

- The only measured effect resulting from increasing the thickness of the slabs from 4.5 to 6 in. was a reduction in the traffic-loading–induced compressive strain at the top of the slabs. No effect was observed in terms of corner deflection, LTE performance during HVS testing, or tensile strain at the bottom of the slabs. This outcome was related to the fact that, for both slab thicknesses, the concrete and asphalt remained bonded during the HVS testing.

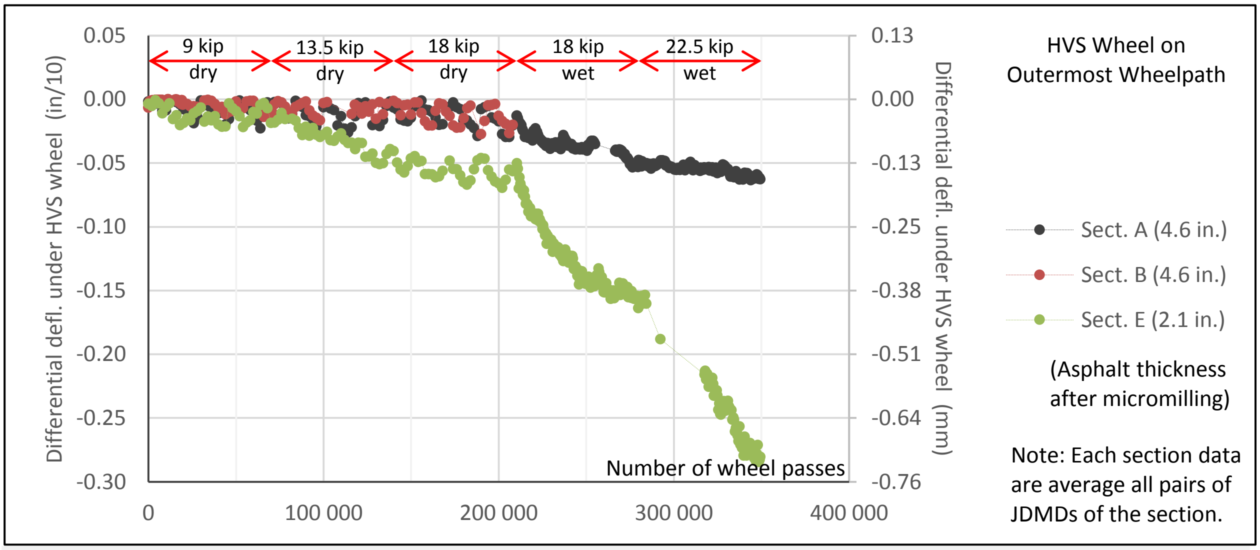


***Influence of Slab Thickness on Concrete Strains*. Based on the HVS testing results, it appears that increasing the thickness of the slab does not extend the fatigue cracking life of thin BCOA since the increased thickness did not reduce the tensile stresses induced by traffic at the bottom of the slabs. This outcome was related to the fact that the asphalt and concrete remained bonded during the HVS testing. However, in the field, the concrete and asphalt will debond over the long term. Once debonding (full or partial) occurs, the additional thickness does result in lower tensile stresses at the bottom of the slabs under traffic loading, which would provide extended cracking fatigue life.**

Asphalt Base Thickness

The role of the factor *asphalt base thickness* was evaluated by comparing the performance of Sections A and B, where the average thickness of the old HMA base was 4.6 in. (117 mm) after micromilling, against the performance of Section E, where the average thickness of the old HMA base was 2.1 in. (54 mm) after micromilling.

- The main effect that resulted from decreasing the thickness of the asphalt base from 4.6 to 2.1 in. was a considerable drop in LTE performance. On the sections with the thicker base, Sections A and B, LTE in the wheelpath changed little during the HVS testing and generally remained above 80 percent, but on Section E, which had the thinner base, LTE dropped to 60 percent after the dry HVS testing and fell further to 40 percent after the wet HVS testing.



Influence of Asphalt Base Thickness on Joint Differential Deflection. **The common practice for using thin BCOA includes a recommended minimum asphalt base thickness of 3 inches (75 mm) after milling. Based on the poor LTE performance of Section E—compared to Sections A and B—it seems that the 3-inch recommendation is reasonable although it might be somewhat unsafe when asphalt thickness variability is high or the asphalt is not in fair or good condition.**

Asphalt Surface Texturing Technique

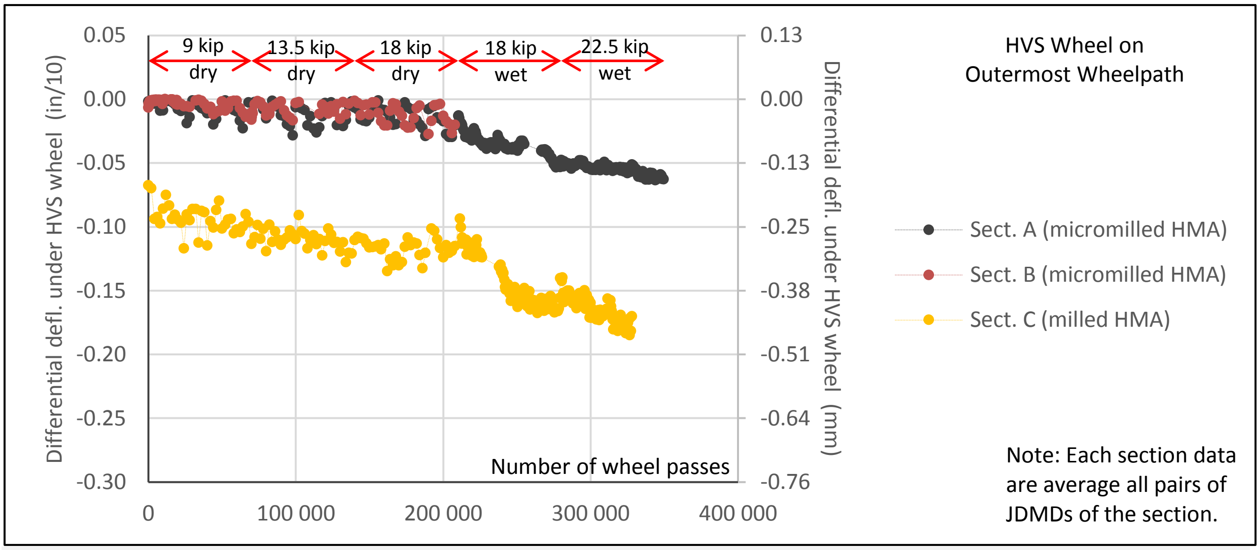
The role of the factor *asphalt surface texturing technique* was evaluated on an old asphalt base by comparing the performance of Section C, where the old HMA surface was milled, against the performance of Sections A and B, where the old HMA surface was micromilled. The role of the asphalt surface texturing technique was also evaluated on new asphalt base by comparing the performance of Section F, where the new RHMA-G was micromilled, against the performance of Section I, where no texturing was used on the RHMA-G.

The conclusions based on the comparison of the performance of Section C (milled old HMA) against the performance of Sections A and B (micromilled old HMA) are listed below.

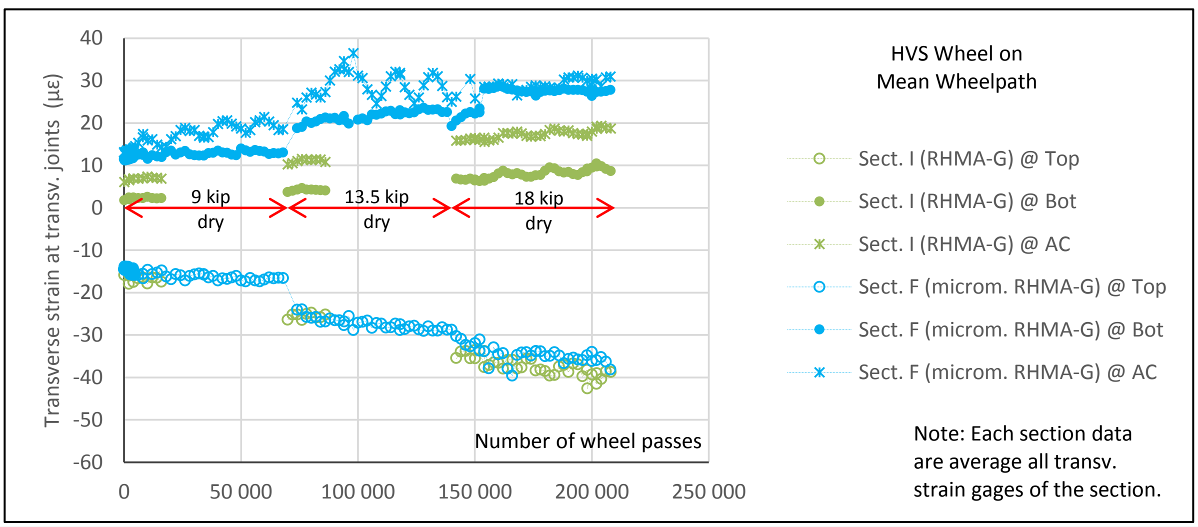
- During HVS testing, the milled section, Section C, showed 1) greater traffic-loading–induced tensile strain at the bottom of the slabs at the transverse joints, 2) much worse LTE performance, and 3) larger permanent vertical slab displacements than the micromilled Sections A and B.
- Forensic work revealed that Section C's poor performance was due to the poor performance of its concrete-asphalt interface. Most of the cores extracted from this section presented either asphalt surface failure or concrete-asphalt debonding. On the other hand, most of the cores extracted from the sections with micromilled old HMA—except for those extracted at the corners of the slabs—showed the asphalt and the concrete-asphalt interface to be in good condition. The cores extracted at the corners of the slabs showed either asphalt surface failure or concrete-asphalt debonding regardless of whether the old HMA was milled or micromilled.
- The poor performance of the concrete-asphalt interface in Section C was attributed to the milling process itself and to the proximity of the milling operation to the interface between the two existing lifts of old HMA.

A conclusion based on the comparison of the performance of Section F (micromilled new RHMA-G) against the performance of Section I (new RHMA-G with no texturing) is listed below.

- Micromilling the new RHMA-G resulted in a weaker concrete-asphalt interface, which resulted in a considerably larger traffic-loading–induced tensile strain at the bottom of the slabs.



***Influence of Surface Texturing of the Asphalt Base on Joint Differential Deflection for Old HMA*. Milling is the technique that has typically been used in other states to texture the pavement surface before construction of BCOA. However, one advantage of using micromilling rather than milling is greater precision in the texturing operation. This increased precision can be useful for avoiding the delamination between asphalt lifts, as occurred in Section C. Additionally, it is believed that the micromilling process is less aggressive to the asphalt than the milling process, although further experimental data are needed to fully verify this belief.**

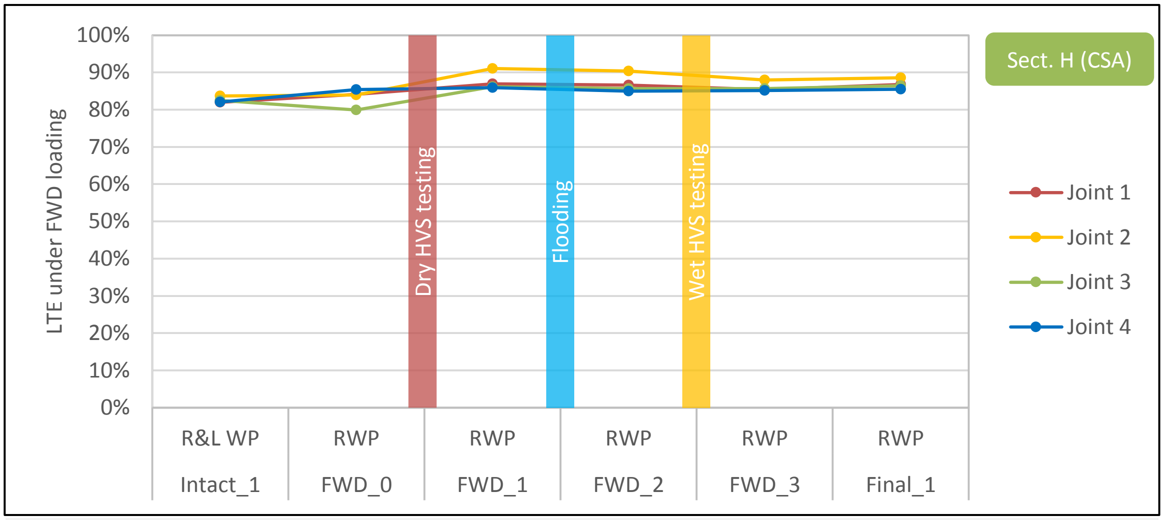


***Influence of Surface Texturing of the Asphalt Base on Transverse Strain for New RHMA-G*. The good bond between the concrete and the new RHMA-G base is apparent in Section I, as evidenced by the fact that the strain measured (in the asphalt) 1.2 in. (30 mm) below the concrete-asphalt interface is considerably larger than the strain measured at the bottom of the slab, 0.8 in. (20 mm) above the concrete-asphalt interface. However, evidence of that good bonding is not that clear in Section F, since the strain in the asphalt is similar to the strain at the bottom of the slab. This outcome indicates that the micromilling process either resulted in a surface where the concrete could not bond well or, more likely, it damaged the top part of the RHMA-G.**

Type of Asphalt Mix

The effect of the factor *type of asphalt mix* was evaluated by comparing the performance of Section B, with its micromilled old HMA base, against the performance of Section I, with its untextured new RHMA-G.

- During HVS testing both sections presented excellent LTE performance since LTE at the mean wheelpath remained over 80 percent and LTE measured under the wheel of the HVS remained unchanged. The main difference between the two sections' LTE performance was that LTE on the section with the RHMA-G overlay was less variable from one transverse joint to another compared to that on the section with the micromilled old HMA.

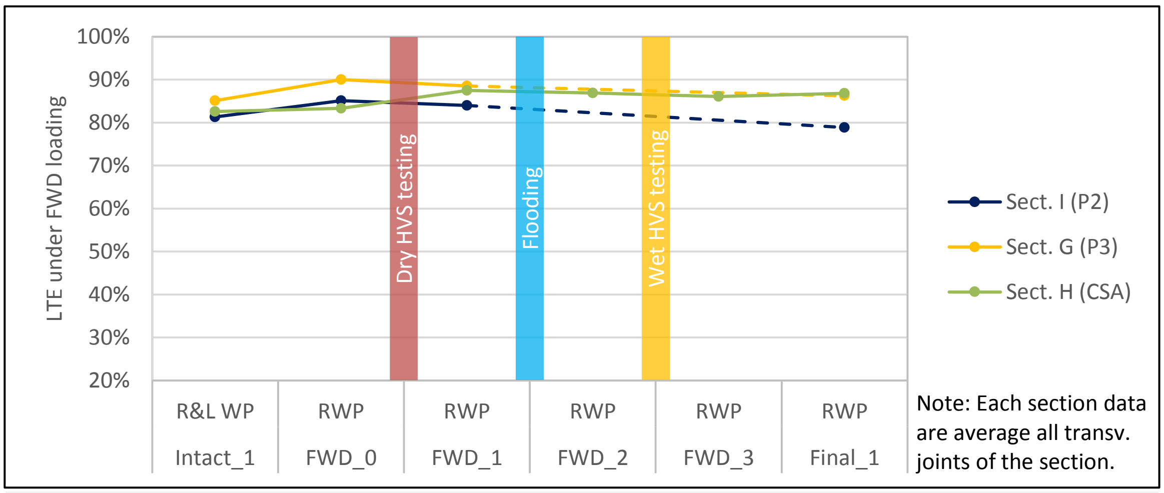


***Influence of Asphalt Mix Type on Load Transfer Performance*. All the 6×6 sections with new RHMA-G showed excellent performance during the HVS testing. As an example, this figure shows the LTE measured with the FWD in the mean wheelpath of Section H. The LTE was very uniform from one joint to another and barely changed after the HVS loading.**

Type of Concrete

The role played by the factor *type of concrete* was evaluated based on a comparison of the performance of the 6×6 Sections G, H, and I to one another since the only difference between them was the type of concrete: P3 (Type III cement) in Section G, CSA (calcium sulfoaluminate cement) in Section H, and P2 (Type II/V cement) in Section I. Similarly, the effect of the type of concrete was evaluated by comparing the performance of the two 12×12 sections: Section J, with P2 concrete, and Section K, with P2-ICC concrete (an internally cured concrete based on mix P2).

- The type of concrete used did not produce any noticeable effect on the performance of the thin BCOA sections. Similarly, the type of concrete did not produce any effect on the structural response of the thin BCOA sections other than those directly related to the stiffness of each type of concrete.

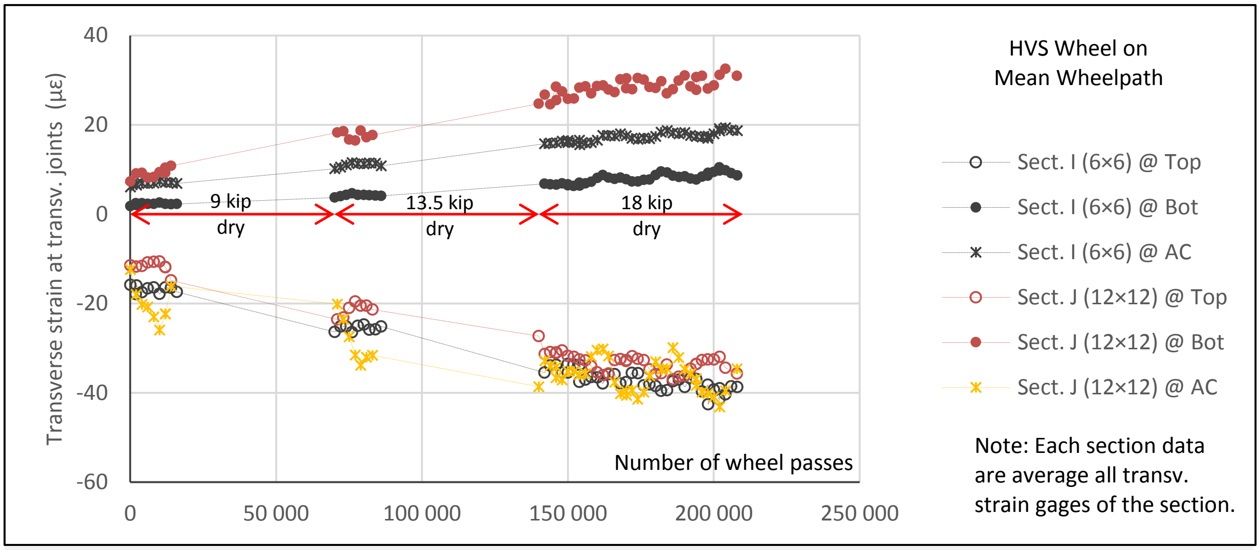


***Influence of Concrete Mix Type on Load Transfer Performance*. The three 6×6 sections with different concrete types and new RHMA-G base showed excellent performance during the HVS testing, although Section H was the only one tested in wet conditions. The only observed difference between the performance of the three sections was related to asphalt/concrete-asphalt interface damage at the corners that the forensic coring revealed. That damage was somewhat smaller in Section H than in the other two sections, an outcome that was likely related to the lower levels of moisture-related shrinkage of the CSA mix compared to the portland cement mixes.**

Slab Size

The role of the factor *slab size* was evaluated by comparing the performance of Section I, where the slabs were 6×6 (6×6 ft [1.8×1.8 m]), against the performance of Section J, where the slabs were 12×12 (12×12 ft [3.6×3.6 m]).

- The increase in slab size from 6×6 to 12×12 resulted in three negative effects: 1) much worse LTE performance, 2) much larger corner deflections, and 3) much larger concrete tensile strains under the wheel of the HVS.

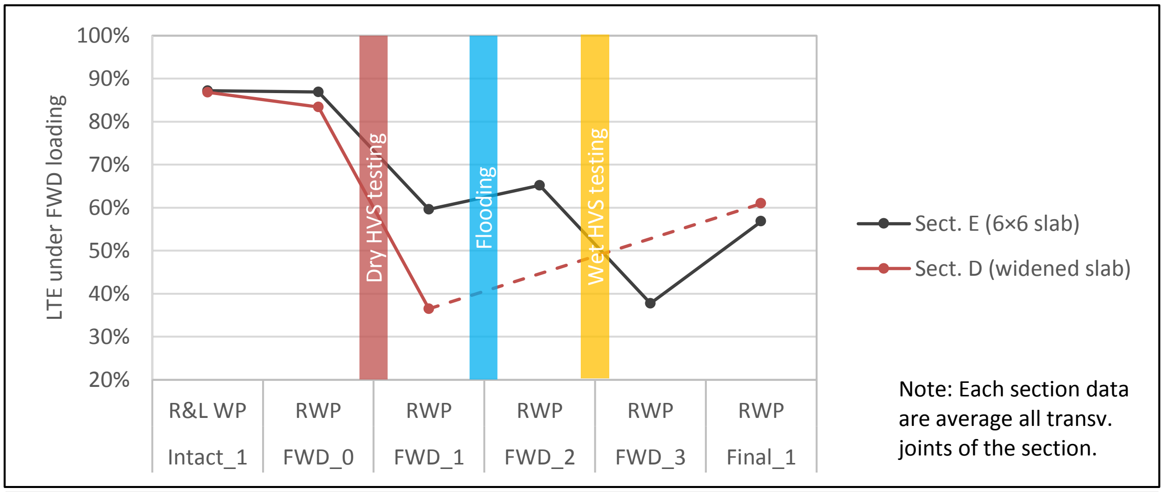


***Influence of Slab Size on Tensile Strains*. The compressive strains measured in the asphalt in Section J indicated that the concrete and asphalt layers were working independently of each other. The result was that there was no composite structure but instead a concrete overlay resting on top of—but not bonded to—an asphalt layer. The lack of concrete-asphalt composite action together with the low transverse joint LTE were the reasons why the strain at the bottom of the slabs was much larger in Section J than in its 6×6 counterpart, Section I. At the same time, the low LTE was attributed to 1) the wider transverse joint width in Section J than in Section I (due to the larger transverse joint spacing), and 2) the lack of asphalt base contribution to LTE (because the concrete and asphalt layers were not bonded at the transverse joints).**

Widened Slab

The role of the factor *widened slab* was evaluated by comparing the performance of Section E, with its 6×6 slabs, against the performance of Section D, with its 8×8 slabs (2.4×2.4 m [8×8 ft]).

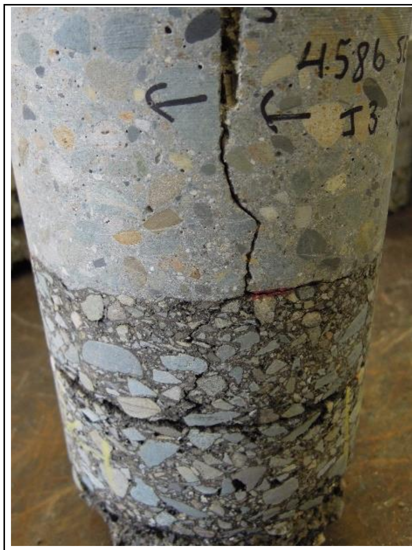
- The main effect of slab widening was a considerable reduction of the strain that develops at the shoulder edge of the slabs under traffic loading and the consequent reduction of the risk of transverse cracking. Nonetheless, the increase in transverse joint spacing, from 6 ft (1.8 m) in Section E to 8 ft (2.4 m) in Section D, also resulted in worse LTE performance.



***Influence of Widened Slab on Load Transfer Efficiency*. Although the 8×8 slabs of Section D were effective in reducing the strain that develops at the shoulder edge of the slabs under traffic loading, they also resulted in worse transverse joint LTE performance than the 6×6 Section E. The poor LTE performance of Section D was attributed to its longer transverse joint spacing, compared to that of the 6×6 sections. Fortunately, the slabs can be widened while maintaining the same transverse joint spacing using 6×8 slabs (length×width). That slab size is expected to provide the benefit of reduced strain at the shoulder edge without compromising LTE.**

4.7 What is the Performance of the Asphalt Base under the Traffic and Ambient Environment Actions?

- All the transverse joints in the 6×6 sections propagated through the old HMA base. It is believed that the propagation was mainly caused by the HVS loading, since the rate of propagation was 100 percent in the exterior slabs subjected to HVS loading, 50 percent in the interior slabs not subjected to HVS loading, and zero percent in the environmental sections (not trafficked with the HVS). The only HVS section with an old HMA base where the transverse joints did not propagate was Section D, which had 8×8 slabs, where concrete-asphalt debonding occurred instead. None of the transverse joints propagated into the new RHMA-G, an outcome that was attributed to the high fatigue resistance of that asphalt mix.



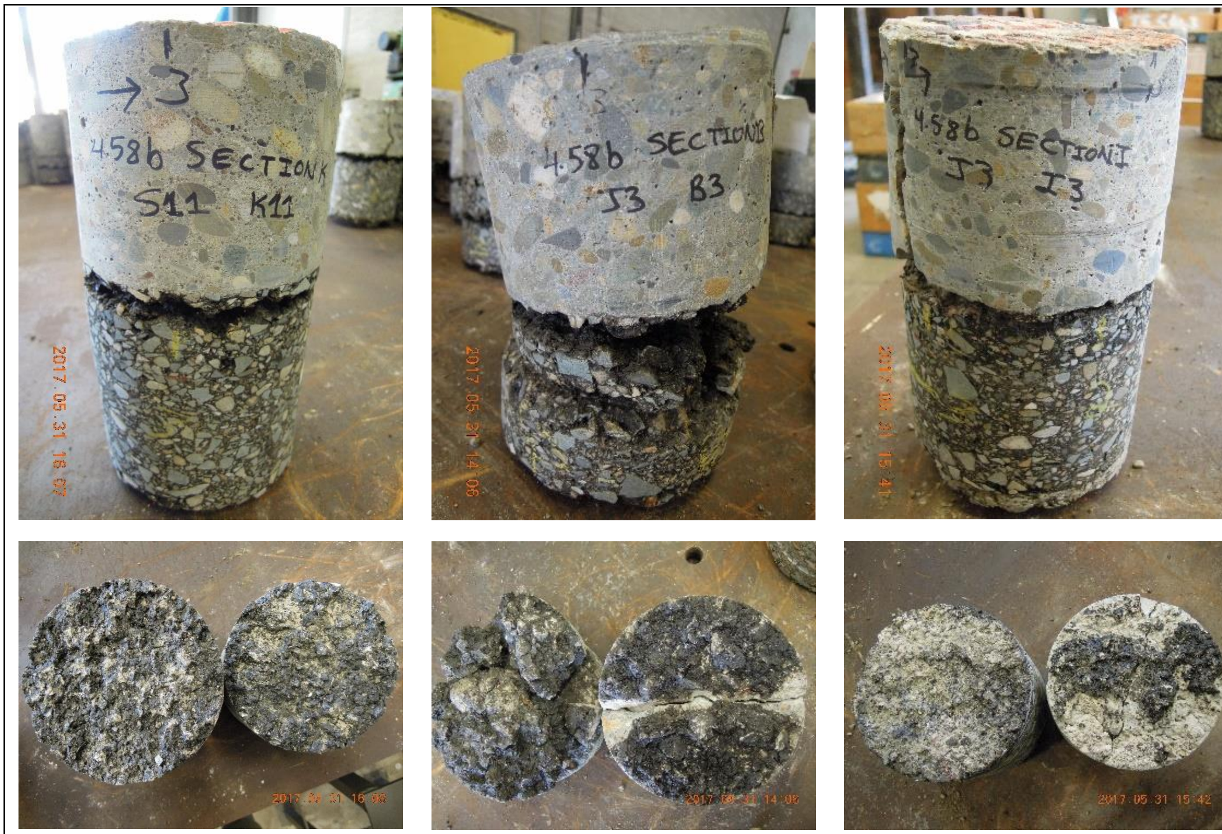
Section B, 6×6 with old HMA
(mean wheelpath)

Section D, 8×8 with old HMA
(mean wheelpath)

Section H, 6×6 with RHMA-G
(mean wheelpath)

***Propagation of Transverse Joints through the Asphalt Base*. All the transverse joints of the 6×6 HVS sections propagated into the old HMA base while none of them propagated into the new RHMA-G. In Section D, with 8×8 slabs, debonding between the concrete and the asphalt took place at the transverse joints. It is very likely that the debonding prevented the transverse joints from propagating into the old HMA base during the HVS testing.**

- None of the longitudinal joints propagated into the old HMA or new RHMA-G asphalt bases.
- Apart from the distresses related to the HVS testing, forensic work revealed the presence of other distresses related to the action of the ambient environment in all the sections.
  - The 12×12 sections, with new RHMA-G, presented a horizontal asphalt failure band along the perimeter of the slabs. The failure band was 0.2 to 0.4 in. (5 to 10 mm) below the top of the RHMA-G overlay and it was 6 to 18 in. (150 to 450 mm) wide.
  - Around 80 percent of the corners of the 6×6 sections with old HMA and around 50 percent of the corners of the 6×6 sections with new RHMA-G presented either asphalt crushing or concrete-asphalt debonding.
  - Both the damage at the corners and the band delamination were caused by hygrothermal deformations of the slabs.



Delamination band in 12x12 Section K, with new RHMA-G	Damage at corner in 6x6 Section B, with old HMA	Damage at corner in 6x6 Section I, with new RMHA-G

***Ambient Environment-Related Damage to Asphalt and Concrete-Asphalt Interface*. Neither the delamination band in the 12×12 sections nor the damage at the corners in the 6×6 sections were related to the HVS loading, but to the hygrothermal deformations of the slabs. The delamination band was believed to be the main reason for the poor HVS performance of the 12×12 sections. On the other hand, the damage at the corners did not have a considerable impact on the HVS performance of the 6×6 sections because of the small extent of the damage. Nonetheless, the long-term evolution of that damage remains known.**

- Cores extracted from the three BCOA sections where the old HMA base had two asphalt lifts showed that the two HMA lifts were systematically debonded. On the other hand, none of the cores extracted from the sections with new asphalt base presented delamination between the 1.2 in. (30 mm) thick RHMA-G overlay and the HMA underneath it.

4.8 How Should Thin BCOA Design Methods Be Improved to Consider Faulting?

Currently available BCOA design procedures only have the capability to predict performance life by using fatigue cracking as the failure criterion. A design method for faulting for thin BCOA was developed by the University of Pittsburgh as part of this project. The researchers developed a computational model for deflection basin energy, the parameter that governs faulting, based on 3-dimensional FEM, and validated the responses for deflections in the leave and approach slabs with data from sections at the Minnesota Road Research Facility (MnROAD) and the UCPRC testing of BCOA sections with the HVS. The computational model included two models: one for a joint that only propagates through the PCC (partial-depth) and the other for a joint that propagates through both the PCC and asphalt layers (full-depth).

The results of the FEM model were used to train artificial neural networks (ANNs) for fast computation. Incremental faulting models were developed and calibrated with MnROAD data (the HVS sections did not produce faulting because of the slow speeds and bi-directional trafficking used to increase the number of repetitions in the time available).

The following are preliminary conclusions:

- The critical response for each model is the deflection basin under the approach and leave slabs, at the top of the asphalt base for partial-depth joints and at the bottom of the asphalt base for full-depth joints.
- Factors primarily affecting faulting are truck traffic and vertical temperature gradients, which are primarily a function of climate region, with greater differences between day and night temperatures causing greater temperature gradients.
- The effects of faulting on ride quality are a function of the severity of the faulting on each joint multiplied by the number of joints. Therefore, slab lengths smaller than 4.5 ft (1.4 m) will result in many joints, and a high multiplier effect for any faulting.
- At MnROAD, there was greater faulting when the joint cracked through both the BCOA slabs and the asphalt base (full-depth cracks) compared to when the joint only cracked through the BCOA slabs and not through the asphalt base (partial-depth), with those with partial-depth joints generally having faults less than 0.1 inches (2.5 mm) and those with full-depth joints having faults between 0 and 0.15 inches (0 and 4 mm).
- Slabs that had joint spacing on the order of 12 ft (3.6 m) showed a tendency at MnROAD to have full-depth joints on every joint (through the PCC and the asphalt base), while for slabs that had joint spacing on the order of 6 to 8.5 ft (1.8 to 2.6 m) there was a mix of partial-depth versus full-depth joints that depended on the ratio of the bending resistance of the slabs to that of the asphalt base.
- Less bending resistance in the slabs relative to that in the asphalt base (thinner slabs relative to thickness of the asphalt) resulted in less chance of full-depth joints.
- Sensitivity analysis from the models indicated that where large lane-width slabs have resulted in full-depth joints:
    - Thicker asphalt base reduces faulting of full-depth joints.
    - Increased slab thickness (from 4 to 6 to 8 inches [100 to 150 to 200 mm]) can reduce faulting by about 0.03 inches (0.8 mm) for each 2 inch slab thickness increase under 5 million ESALs of traffic.
- A limitation of the model is that the calibration sections used are all from one state, Minnesota. However, it is believed that the models developed are sufficient until additional data are available to supplement the calibration database and another calibration can be performed.

4.9 Additional Conclusions

The use of concrete compressive strength to predict flexural strength was evaluated in parallel with the construction of the test sections. For each of the four mixes used in the construction of the BCOA sections (P2, P3, CSA, P2-ICC), a prediction equation (power function type) was calibrated using the results of flexural and compressive strength testing conducted on specimens prepared in the laboratory. Then the calibrated equations were used to predict the flexural strengths of the mixes used in the construction of the test track. The predictions were compared to measured flexural strength values.

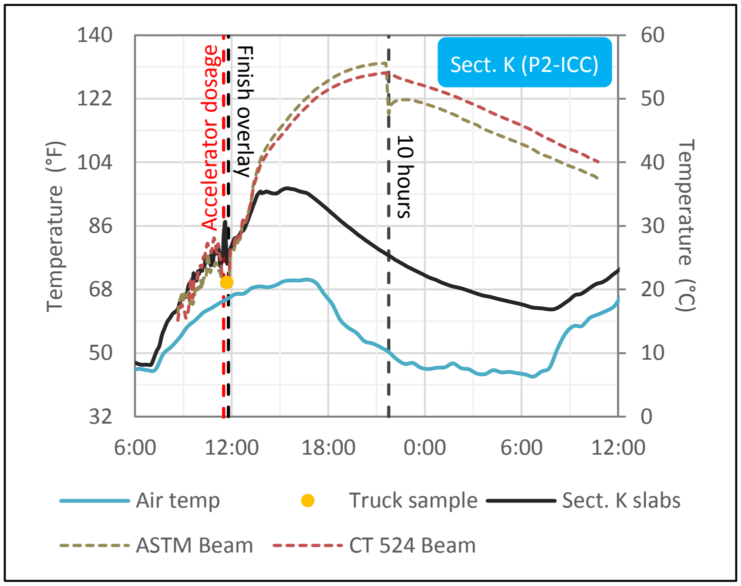
- The root mean square of the prediction error ranged from 37 psi (0.25 MPa) in the mix with CSA cement to 193 psi (1.33 MPa) in the mix with Type III cement.
- The pooled (four mixes) root mean square of the prediction error was 140 psi (0.97 MPa), which seems relatively high for an approach to be used for QC/QA.

This study also included a comparison of two methods used for measuring the flexural strength of RSC, ASTM C78-10 and California Test 524 (2013). Those two methods differ mainly in their different approaches to the initial curing of the beams. They also differ in the formula used to calculate flexural strength (CT 524 applies a factor of 1.05 to the formulas that result from classical beam theory, which are used in ASTM C78). The comparison was conducted based on the mixes used for the construction of the test sections.

- No considerable difference was found between ASTM C78-10 and California Test 524 (2013) for measuring the flexural strength of rapid-strength concrete.
    - Flexural strength differences between both approaches were related to the formula used to determine the modulus of rupture. Differences between results from both approaches were not statistically significant when the same formula was used in the calculations.
    - No clear difference was found between both curing approaches based on the temperatures recorded inside the beams, except for the P2 mix, where the beams cured following CT 524 maintained higher temperature than the beams cured following ASTM C78 for approximately 15 hours.

  Note: It must be emphasized that these outcomes were highly dependent on the specific weather conditions during test track construction. In extremes of weather, with either lower or higher temperatures, differences between ASTM C78-10 and CT 524 would be expected, since weather conditions are expected to affect the CT 524 beams much more than they are expected to affect the beams cured following the ASTM procedures.

- The temperatures measured inside the beams for the initial curing approaches of the both the ASTM and CT test methods were considerably higher than the temperatures measured in the pavement slabs. Consequently, both curing approaches would have considerably overestimated the maturity of the concrete in the slabs at the design opening time.



Comparison of Temperature in Slabs and Flexural Beams per CT 524 and ASTM C78-10.
Temperature in the beams (ASTM C78-10 and CT 524) matched the temperature in the slabs for a very short period of time after concrete placement. After that short period, the temperature in the slabs was much lower than in the beams, an outcome related to the high ratio of exposed surface versus volume of the thin BCOA slabs.

A simple experiment was conducted to better assess the extent of water penetration through the concrete-asphalt interface during the wet HVS testing of the sections. Two main conclusions were drawn from the experiment:

- The permeability of the old HMA was high even though the air-void content of the old HMA was as low as 4 percent. The permeability of the concrete-asphalt interface was much smaller than the permeability of the old HMA.

The negative effects of the presence of water on thin BCOA performance were evaluated based on the HVS testing under wet conditions. The main conclusion is:

Compared with the dry HVS testing, the rate of damage development during the wet HVS testing increased considerably, first, because the sections were flooded with water and then kept wet during loading, and second, because the loading was channelized at the edge of the slabs. It is still to be determined how much the increase in the rate of damage is related to each of those two factors. It should be mentioned that the flooding itself did not produce a considerable weakening of the structure of the thin BCOA, but the loading that was applied under wet conditions after flooding did.

5 SUMMARY OF RECOMMENDATIONS

The recommendations below, which will need to be validated with pilot projects, intend to provide the answer to the main question addressed in this project: *What are the recommendations for the design of thin BCOA that consider California traffic, climate and materials conditions, and construction work zone practices?*

Slab Thickness

- Consider a minimum slab thickness of 4 in. (100 mm).
- Consider another rehabilitation alternative if the overlay thickness exceeds 7 in. (175 mm) because BCOA might not be cost-effective. For slab thicknesses over 7 in., the structural contribution of the bonding to the asphalt base might be negligible.
- Use validated design procedures like BCOA-ME or the MEPDG-BCOA to determine the thickness of the slabs while also considering the thickness of the existing asphalt base.

Asphalt Base Thickness

- Follow current practice for only using thin BCOA when there is a minimum thickness of sound asphalt of 3 in. (75 mm) after the surface-preparation operation.
- It is recommended that thin BCOA be used where there is sound asphalt thickness well above the minimum of 3 in.; this will considerably extend the life of the BCOA. In particular, it is recommended that there be thicker asphalt layers when the condition of the asphalt is not good or the variability of the asphalt thickness is high.

Asphalt Surface Texturing Technique

- Micromilling the old asphalt surface is recommended when the need exists to reduce road surface elevation after the overlay construction or to remove distressed asphalt.
- Do not assume that milling or micromilling old HMA will improve concrete-asphalt bonding versus the no surface-texturing option.
- During milling or micromilling be extremely careful to not create delamination between asphalt lifts. The use of micromilling rather than milling is highly recommended when precision is required in the texturing operation and/or there is a risk of delamination between asphalt lifts.

Asphalt Base Condition

- Consider current practice for thin BCOA, included in reference documents like the *Guide to Concrete Overlays* from the National Concrete Pavement Technology Center. The asphalt should be in fair or good condition, the presence of fatigue cracking should be minimal, and stripped asphalt must be removed before the overlay construction.

- It is recommended that the condition of the asphalt base be comprehensively evaluated in upcoming Caltrans thin BCOA pilot projects so that a link can be established between asphalt condition and performance. Asphalt base condition should be evaluated before and after any surface-preparation operations.

Use of New Asphalt on Surface of Base

- It is recommended that placement of a new 1.2 in. (30 mm) thick RHMA-G base be evaluated in pilot projects, and that future pilot projects consider placement of thin BCOA on existing RHMA-G.
- In particular, it is recommended that the placement of new RHMA-G base on existing asphalt surfaces that are in poor condition be evaluated in future pilot projects.
- Do not mill or micromill new RHMA-G.
- It is recommended that the RHMA-G base be built a few months before the concrete overlay construction so that the asphalt surface undergoes some aging.

Type of Concrete

- Continue use of Caltrans rapid-strength concrete (RSC) mixes for 4-hour opening time for thin BCOA paving in nighttime or 10-hour opening time for other, longer, fast-track construction closures. The mixes evaluated in this project had high slump for use with a roller screed, and their use in slip-form paving should be evaluated in pilot projects.
- For long construction windows where the use of rapid-strength concrete is not required, use the concrete mixes that are typically used for standard jointed plain concrete pavements.

Slab Size

- Use half-lane width designs with transverse joint spacing of 6 ft (1.8 m).
- Although different slab widths may be used to fit existing lanes and keep the longitudinal joint in the middle of the wheelpath, adopting a transverse joint spacing above 6 ft (1.8 m) is not recommended. Examples of recommended slab sizes are 6×5, 6×6, and 6×8 (length×width).

Shoulder Type

- Consider widening the slabs 1 to 2 ft (300 to 600 mm) into the shoulder where existing shoulders have sufficient width paved with asphalt, while maintaining transverse joint spacing at 6 ft (1.8 m).

Concrete Curing Procedure

- Follow current practice for thin BCOA, which consists of curing the thin BCOA slabs the same way as standard concrete pavement slabs except for the curing compound application rate, which should be twice that used for standard concrete pavements.

- Consider following the new procedure tried for the first time in this research project: topical application of a shrinkage-reducing admixture (SRA) spray before application of curing compound—when building an overlay in the dry and warm conditions that typically occur in most of California during summer.

Use of Dowels

- The use of dowels has not been evaluated in this research project. Typically, using dowels will not be cost-efficient.

Use of Tie Bars

- No conclusion could be drawn regarding the use of tie bars in the longitudinal joints in thin BCOA in California since tie bars were not used in the test sections.
- With thin BCOA, the concrete cover over the tie bars may not be thick enough to resist cracking. Consequently, it is recommended that the use of tie bars in longitudinal joints be included as a test variable in upcoming thin BCOA pilot projects.

Sawcutting Depth

- Transverse and longitudinal joints should be sawed to one third of the thickness of the slab, which is current practice for thin BCOA.

Joint Sealing

- A conclusion could not be drawn regarding the need and efficacy of sealing thin BCOA joints in California.
- It is recommended that the sealing of the joints be included as a test variable in upcoming thin BCOA pilot projects.
- It is recommended that two steps be accomplished before considering prescribing sealing thin BCOA joints on a regular basis in California: 1) determine to what extent sealing the joints extends thin BCOA life, and 2) conduct a life-cycle cost analysis to verify the efficacy of sealing thin BCOA joints.

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