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TECHNICAL MEMORANDUM

Twenty-Year Performance Review of Asphalt Concrete Long Life Pavements with Performance-Related Specifications

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16. ABSTRACT
The first asphalt concrete long-life (AC Long Life) project was constructed in Los Angeles County on Route 710 (LA-710) near Long Beach in 2001/2004 and is now over 20 years old. Four more AC Long Life projects have been completed in California since then, three between 2011 and 2014 (TEH-5, SIS-5, SOL-5), and one in 2021/2022 (SAC-5). The goal of these AC Long Life projects was to achieve design lives of 30 years or 40 years (the standard Caltrans asphalt pavement design life was 20 years at the time). Measures taken to achieve those lives included the use of performance-related specifications for job mix formula approvals, higher compaction requirements, and the use of a three-layer asphalt concrete system for structural capacity. These measures required additional costs. Calculations indicate that the longer lives will result in life cycle cost reductions if they achieve the design lives. Hence, periodic performance evaluations are important, which is the purpose of this technical memorandum. Extensive material sampling and structural evaluation were conducted regularly on the LA-710 and TEH-5 projects. This technical memorandum reviews the available performance data, summarizes the material testing and structural evaluation data for the LA-710 and TEH-5 projects, and compares the as-built materials with statewide medians. The cracking, patching, roughness (IRI), and rutting data revealed that these projects had all performed very well in general at respective 10- to 20-year milestones. No rehabilitation and only one minor maintenance activity has been performed on LA-710, and no maintenance/rehabilitation activity has been performed on the 10-year-old projects. Analysis of the LA-710 and TEH-5 deflection data suggests some densification (not to the extent of crushing) and aging in the asphalt-bound layers but no signs of traffic-induced damage. Laboratory test data on between-wheelpath specimens from the TEH-5 project indicate that the as-built materials in general have better fatigue performance, lower stiffness, and lower permanent deformation performance than the statewide median materials tested during mix design. Given the fact that only minor rutting has been observed in these projects, the as-built materials are believed to be good examples of mixes with balanced performance, with a few exceptions.

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7. PROPOSALS FOR IMPLEMENTATION

Recommendations are made for data collection during AC Long Life projects and development of methods to provide more time for the mix design approval process, such as use of pre-qualification of mixes.

8. RELATED DOCUMENTS

Harvey, J. T, Deacon, J. A, Tsai, B., and Monismith, C. L. 1995. *Fatigue Performance of Asphalt Concrete Mixes and Its Relationship to Asphalt Concrete Pavement Performance in California* (RTA-65W485-2). Davis and Berkeley, CA: University of California Pavement Research Center.

Monismith, C. L, and Long, F. 1999. *Overlay Design for Cracked and Seated Portland Cement Concrete (PCC) Pavement--Interstate Route 710* (Technical Memorandum: TM UCB PRC 99-3). Davis and Berkeley, CA: University of California Pavement Research Center.

Monismith, C. L, Harvey, J. T, Tsai, B., Long, F., and Signore, J. 2009. *The Phase One I-710 Freeway Rehabilitation Project: Initial Design (1999) to Performance after Five-Plus Years of Traffic (2009)* (Summary Report: UCPRC-SR-2008-04). Davis and Berkeley, CA: University of California Pavement Research Center.

Signore, J., Tsai, B., and Monismith, C. L. 2016. *Development of Hot Mix Asphalt Pavement Performance Properties for Long-life Pavement Design: Caltrans District 2, Interstate 5, Red Bluff, California* (Technical Memorandum: UCPRC-TM-2014-03). Davis and Berkeley, CA: University of California Pavement Research Center.

Wu, R., Harvey, J., Lea, J., Mateos, A., Yang, S., and Hernandez, N. 2021. *Updates to CalME and Calibration of Cracking Models* (Research Report: UCPRC-RR-2021-01). Davis and Berkeley, CA: University of California Pavement Research Center.

9. VERSION UPDATES

10. LABORATORY ACCREDITATION

The UCPRC laboratory is accredited by AASHTO re:source and CCRL for the laboratory testing discussed in this report.

11. SIGNATURES

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PROJECT OBJECTIVES

This technical memorandum was prepared as part of Partnered Pavement Research Center Project 3.58, "Continued Calibration of Mechanistic-Empirical Design Models with Pavement Management System Data." The objective of this project is to establish an efficient and repeatable procedure for updating field calibration of mechanistic-empirical design methods. This will be achieved through the following tasks:

- Task 1: Update calibration data.
- Task 2: Update *CalME* calibration.
- Task 3: Update *Pavement ME* calibration.
- Task 4: Integrate network-level ME data management.
- Task 5: Prepare project documentation.

The objective of this technical memorandum is to review the half-life performance of one asphalt concrete long-life (AC Long Life) project with performance-related specifications that was built in Southern California in the early 2000s and the quarter-life performance of three other AC Long Life projects built in Northern California in the early 2010s. These pavements were designed for a 40-year life, which is four times longer than the standard 10-year design life used for asphalt rehabilitation when these projects were designed and constructed. Caltrans began use of a 40-year design life for asphalt pavement rehabilitation on major routes in subsequent versions of the *Highway Design Manual*. The review of the half-life and quarter-life performance of these projects presented in this technical memorandum will help in assessing design hypotheses and providing lessons. This technical memorandum is part of the completion of Task 2.

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LIST OF ABBREVIATIONS

AADT	Average annual daily traffic
AB	Aggregate base
AC	Asphalt concrete
APCS	Automatic pavement condition survey
AS	Aggregate subbase
BWP	Between-wheelpath
CA	Carbonyl area
CSOL	Crack, seat, and overlay
CTB	Cement-treated base
EA	Expenditure authorization
FDAC	Full-depth asphalt concrete
FWD	Falling weight deflectometer
HMA	Hot mix asphalt
IRI	International Roughness Index
JMF	Job mix formula
LCCA	Life cycle cost analysis
LLPRS	Long Life Pavement Rehabilitation Strategy
ME	Mechanistic-empirical
NB	Northbound
PCC	Portland cement concrete
PM	Postmile
PMS	Pavement management system
PRS	Performance-related specification
PRT	Performance-related test
RAP	Recycled asphalt pavement
RCAB	Recycled concrete aggregate base
RWP	Right wheelpath
SB	Southbound
SSC	Smoothed spatial correlation
SUL	Sulfoxide

LIST OF TEST METHODS AND SPECIFICATIONS

AASHTO T 320	Standard Method of Test for Determining the Permanent Shear Strain and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester (SST)
AASHTO T 321	Method of Test for Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending
AASHTO T 324	Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures
AASHTO T 378	Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)

CONVERSION FACTORS

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

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

1 INTRODUCTION

1.1 History of Long Life Pavements

The goals of improving pavement design and construction practices are to improve the cost-effectiveness of investments in road infrastructure by reducing life cycle costs and to improve the environmental performance of road infrastructure by reducing life cycle global warming potential and other priority emissions, particularly air pollution in California.

For many decades, the design life for new Caltrans concrete and asphalt pavements was 20 years, and the design life for major rehabilitation of asphalt pavement was 10 years, with no standards for concrete pavement rehabilitation other than 10-year life asphalt overlays. In 1996, an internal Caltrans study was undertaken to compare the life cycle costs of rehabilitating existing portland cement concrete (PCC) pavements using a standard 10-year asphalt concrete (AC) overlay strategy with the life cycle costs of using a 35-year PCC pavement. The Caltrans term for this new approach was the Long Life Pavement Rehabilitation Strategy (LLPRS).

The primary goal of the LLPRS was to achieve fast construction and thereby reduce road user traffic delays, following up on the lessons of fast-track construction learned from reconstructing the Interstate 10 (I-10) corridor in Los Angeles County after the 1994 Northridge earthquake, while also seeking longer life and less life cycle maintenance. The study entailed a basic spreadsheet computation that compared the net present values of pavements with different life cycle maintenance and rehabilitation costs under different traffic volume assumptions. Data were obtained from five projects with annual average daily traffic (AADT) volumes varying between 50,000 and 220,000 vehicles per day and 10% to 20% heavy vehicles. The study found that for AADTs above about 150,000 and/or truck traffic higher than about 15,000 trucks per day, user costs were dominant in strategy selection compared with agency costs, and that LLPRS designs typically had lower life cycle costs than the conventional 10-year asphalt overlay designs. Additionally, Caltrans expected implementation of LLPRS to result in a decreased need for maintenance forces to be in the roadway, improving workforce and road user safety *(1)*.

The Caltrans LLPRS Task Force was commissioned in April 1997. The product Caltrans identified for the LLPRS Task Force to develop was Long Life Pavement Rehabilitation Strategy guidelines and specifications for implementation on projects in the 1998-99 fiscal year. The focus of the LLPRS Task Force was originally only concrete pavement strategies. A separate task force was established in early 1998 for flexible pavement strategies, called the Asphalt Concrete Long Life (AC Long Life) Task Force.

The specific goals of the LLPRS were the following:

- Have sufficient production to rehabilitate or reconstruct about 6 lane-kilometers (4 lane-miles) within a construction window of 67 hours (10 a.m. Friday to 5 a.m. Monday), with the intent of this fast production to minimize traffic delay.
- Provide 30+ years of service life.
- Require minimal maintenance, although zero maintenance is not a stated objective.

Both PCC and AC LLPRS were analyzed by the UCPRC, and a computer program was created called *Construction Analysis for Pavement Rehabilitation Strategies* (*CA4PRS*) that analyzed design, construction, and traffic handling variables and quantified their effects on construction productivity and traffic delay, using both deterministic and probabilistic analyses. *CA4PRS* models were populated with initial input data gathered from California concrete paving contractors, Caltrans, and academia, which were later updated using data collected by the UCPRC on the initial PCC and AC LLPRS projects and from discussions with asphalt paving contractors.

The concrete pavement construction productivity analyses explored the effect of the following variables: pavement design profile (thickness), curing time (different opening times for concrete), number and capacity of contractor resources (trucking, plant, paving, demolition), number of lanes to pave, type of construction scheduling (serial or parallel processes), and alternative lane closure tactics (number of lanes closed for construction and number left open to traffic) *(2)*. The asphalt pavement construction productivity analyses looked at the following variables for both crack, seat, and overlay (CSOL) and full-depth replacement approaches: rehabilitation approach (CSOL or full-depth); design profile (thickness); cooling time (time for asphalt to cool with different thicknesses and weather, calculated using *MultiCool*) *(3)*; number and capacity of construction resources (trucking, plant, paving, demolition); and alternative lane closure strategies (same as for concrete) *(4)*.

The initial project for PCC LLPRS was the reconstruction of part of I-10 in Pomona, Los Angeles County, in 1998 *(4,5)*, with additional early projects constructed from 2002 to 2004. The initial AC LLPRS was the reconstruction of part of Interstate 710 (I-710) in Long Beach, also in Los Angeles County, in 2003 *(6),* with additional projects completed from 2012 to 2014. Both projects had 30-year design lives.

The 1996 study previously mentioned was undertaken with limited data and basic life cycle cost analysis (LCCA) principles. More data became available from the initial concrete and asphalt project, and in late 2004, Caltrans requested that a more detailed study be undertaken by the UCPRC to determine whether the 150,000 AADT/15,000 trucks figures were still appropriate. A factorial sensitivity study was completed, comparing life cycle costs of long-life strategies and conventional rehabilitation strategies with more variables than were included in the 1996 study, using more appropriate data from the initial LLPRS projects and the then-new FHWA LCCA software *RealCost* which had just been customized for Caltrans in 2005. The 2005 LCCA sensitivity analyses made clear the need to perform life cycle cost analysis for each project using project-specific data for both agency costs and road user costs. The results of LCCA are dependent on the following variables, which are different for each project *(7)*:

- Traffic demand patterns, including hourly demand, weekday and weekend demand, directional peaks, and discretionary versus job-related travel
- Alternative routes and modes
- Lane and shoulder configurations and highway geometry in each direction

- Feasibility and expected life of each rehabilitation strategy, which depend on truck traffic and existing pavement condition in each lane
- Expected construction durations

The design long-life goal for all new concrete pavement and major rehabilitation was increased to 40 years in the early 2000s as experience was gained with pavement design and construction. In addition, the standard major rehabilitation design life was increased from 10 years to 20 years for all asphalt pavement and 40 years on higher traffic routes in the 2010s after design and construction of the initial AC Long Life projects evaluated in this technical memorandum. The goals of decreased life cycle cost and decreased life cycle environmental impacts are made feasible by the fact that doubling of the design lives requires less than a doubling of the pavement thickness. Mechanics show that for both asphalt and concrete designs, the bending resistance that controls the tensile strain at the bottom of the asphalt layer and the tensile strain at the bottom of the concrete layer under traffic loading, both causing bottom-up fatigue cracking, diminish approximately proportionally with increase in stiffness (E) and to the third power with increase in thickness (h^3) *(8-10)*. This means that increasing stiffness and the third-power exponential effect of increasing thickness increase fatigue life in thick pavements at a greater than one-to-one rate.

1.2 History of Asphalt Concrete Long Life Pavements with Performance-Related Specifications

A number of factors, together, have contributed to the idea of expanding the design life of asphalt pavement beyond 20 years:

- Advancements in materials science, construction quality control/quality assurance practices, performance-related tests and specifications, and pavement design, including the introduction of *CalME* mechanistic-empirical pavement models in 2006 and later software.
- Accelerated pavement testing in the 1990s and early 2000s that evaluated materials design, construction compaction, and structural design [summarized in *(11-14)*].
- Thinking and attention to development of best practices contributing to longer asphalt pavement lives (materials stiffness and fatigue life tradeoffs, compaction, drainage, consideration of climate regions, consideration of axle load spectra, and development of rubberized and polymer-modified surface layers, among other practices), many documented in a 2004 publication regarding design for long-life asphalt pavement design lives *(9,13)*.
- Experience gained from the initial projects built from 2003 to 2014 investigated in this report.

The *CalME* software has been continuously updated and was recently recalibrated using Caltrans pavement performance data dating back to 1978 *(15)*.

1.3 Distinguishing Features of AC Long Life Design and Performance-Related Specifications

It is important to identify the AC Long Life, PRS, and compaction features used on the four projects evaluated in this technical memorandum and how they distinguish these projects from other asphalt pavements that have been built in California in recent years with a 40-year design life.

Pavements can be designed for a 40-year design life, termed a "long life," without using the distinguishing features of AC Long Life design, increased compaction, and PRS used on the four projects:

- Increased compaction requirements: Caltrans began requiring measurement of density in the late 1990s and has set the minimum density at 92% or 91% of maximum theoretical density (MTD) at which disincentives are applied since then, while the nSSPs for the four AC Long Life projects discussed in this report began disincentives below 94% of MTD.
- Use of performance-related specifications (PRSs): when a project uses PRS, there are additional requirements for laboratory performance-related testing to develop the specifications and properties of materials to be used in the structural design using *CalME*. The materials proposed by the contractor must be demonstrated to have those same performance-related properties to be used on the project (May 20, 2022). *(16)*
- Use of three asphalt layers with three distinct sets of properties: as defined in the Caltrans *Highway Design Manual*, "AC Long Life projects use a polymer-modified HMA surface, a high stiffness HMA intermediate layer (often with greater than 15% recycled asphalt pavement (RAP)), and a high binder content very low air-void content rich bottom layer (may be omitted when placed over concrete)" (May 20, 2022). *(16)*.

Regarding PRS, the *Highway Design Manual* states the following referring to new standard practice:

> Two types of specifications are typically used for the materials design and construction of HMA, either Standard Specifications or special provision specifications for the use of Quality Control/Quality Assurance (QC/QA). Beginning in 2002, a new type of specification has been used on selected projects referred to as " performance-related specifications" (PRS).

The specific aspects of AC Long Life design (3 layers except over concrete), PRS, and additional compaction requirements as used on the four pilot projects evaluated in this technical memorandum are detailed in subsequent chapters. Asphalt pavement projects can use one, two, or all three of these features. PRS and the three-layer strategy are called out in the *Highway Design Manual* (HDM), as noted above, and the HDM does not require them to be used together. The use of the increased compaction requirements is not included in the HDM. For example, rehabilitation projects not discussed in this technical memorandum have been designed and constructed using the two-layer polymer surface and stiff intermediate asphalt layer over concrete, and using the three-layer system that also includes the rich bottom layer when not over existing concrete, without including PRS or the additional compaction requirements.

1.4 Review of Asphalt Concrete Long Life Projects with AC Long Life Structures, Performance-Related Specifications, and Additional Compaction

It has been approximately 20 years since the first asphalt concrete long-life (AC Long Life) project was constructed in Los Angeles County on I-710 near Long Beach from 2001 to 2004, with a 30-year design life. Four more AC Long Life projects using PRS and the increased compaction specification have been completed in California since then, all with 40-year design lives. Three were constructed between 2011 and 2014, and one was constructed between 2021 and 2022. Evaluation of the 10- and 20-year performance of the initial four projects can provide information to assess whether these projects are on track to meet the expected life. That assessment constitutes the goal of the study presented in this technical memorandum.

While Caltrans' major asphalt pavement rehabilitation on high-volume routes is now generally designed for 40-year lives, the term "long life" is used in this technical memorandum to refer to the first set of asphalt 30- and 40-year designs because, back in the early 2000s and 2010s when they were built, the standard rehabilitation design life was 20 years, as was common in much of the rest of the United States. Therefore, long life is defined in this technical memorandum as meaning having a design life of 30 or 40 years. The term "long life" does not appear in the HDM.

The goal of these AC Long Life projects was to achieve design lives of 30 or 40 years. As described previously, the measures taken to achieve that included adopting PRSs for job mix formula approvals, higher compaction requirements, and the use of the specific AC Long Life three-layer (in sections that were not on concrete) and two-layer (on existing concrete) asphalt concrete systems for structural capacity. Specifically, the I-710 project was designed for 30 years and the others for 40 years. Note that the sections that are not on concrete are sometimes referred to as "full-depth" in this technical memorandum, although they are different from typical full-depth flexible pavement designs, which place AC directly over subgrade.

The two-layer and three-layer system adopted by all the AC Long Life projects includes the following:

- A polymer-modified mix for the surface for rutting and cracking resistance
- A stiff intermediate layer below the surface for bending resistance
- A rich bottom that has higher binder content for fatigue resistance (not used when asphalt concrete is placed on existing concrete pavement)

Each of these mixes has adopted PRSs for job mix formula approval. In addition, higher compaction requirements are specified. For surface and intermediate mixes, the payment reduction for compaction starts when relative density from cores is lower than 94% instead of 91% for projects following standard specifications. For the rich bottom mixes, the payment reduction starts at 97% relative density.

Each of these extra measures results in additional costs to the project while helping to reduce costs by allowing the use of thinner total asphalt layers to achieve the design life. The use of the thinner cross sections enables faster construction than would have been possible with conventional materials that do not use PRS and do not require additional compaction. This efficiency stems from reduced construction time, as the duration is directly tied to the thickness of old material removed and new material placed. The benefits of all Long Life projects and the projects with combined use of the AC Long Life structures with PRS and additional compaction evaluated in this technical memorandum are predicated on the achievement of their intended design lives. This technical memorandum provides part of the data needed to answer that question.

For simplicity, from here forward, the four projects with the combined use of the two- and three-layer AC Long Life structures with PRS and additional compaction will be referred to as AC Long Life, and the reader should understand that all three features are included in that term in this technical memorandum.

For the first four AC Long Life projects, there are seven to 20 years of pavement performance data available. In addition, extensive material sampling and structural evaluation have been conducted regularly on two of the AC Long Life projects: the Los Angeles I-710 project near Long Beach and the Tehama 5 project near Red Bluff. This technical memorandum provides a review of the available performance data as well as a summary of the material testing and structural evaluation data for the I-710 and Tehama 5 projects. These data can help answer questions about achieving the benefits of AC Long Life projects that can then be compared with the additional costs of designing and building them.

1.5 Source for As-Built and Performance Data

The as-built and performance data used in this memo are extracted from the Caltrans pavement management system, which includes the construction and performance data up to 2021 at the time of this writing.

1.6 List of Projects

The list of AC Long Life projects is shown in Table 1.1, which includes the location, construction window, expenditure authorization (EA) number, and a brief description of the activities associated with the AC Long Life designs. The project locations are shown in Figure 1.1 and typical cross sections are shown in Table 1.2. The first four projects in the table are evaluated in this technical memorandum.

Table 1.1: List of Asphalt Concrete (AC) Long Life Projects

Project Name	County	Construction Window	Expenditure Authorization (EA) Number	Description	Post Mile (PM)
LA-710-Long Beach (LA-710)	Los Angeles	2001–2004	07-1384U	CSOL[a] and reconstruction under bridges[b]	6.8–9.8
Sis-5-Weed (SIS-5)	Siskiyou	2011–2013	02-3E750	CSOL and HMA[c] thick overlay	R19.1–R25.2
Teh-5-Red Bluff (TEH-5)	Tehama	2012–2013	02-3E810	HMA lane replacement and HMA thick overlay	31.8–35.1NB and 37.1–41.6
Sol-80-Dixon (SOL-80)	Solano	2013–2014	04-4A010	CSOL and slab replacement	30.6–38.7
Sac-5-Sacramento[d] (SAC-5)	Sacramento	2021–2022	03-0H10U	CSOL and HMA lane replacement	9.7–24.9

[a] CSOL: crack, seat, and overlay.
[b] The reconstructions could not be found in the database, and the CSOL treatment covers the entire length.
[c] HMA: hot mix asphalt.
[d] The SAC-5 project does not have any records in the database yet.

Figure 1.1: AC Long Life project locations.

Table 1.2: Typical Pavement Cross Sections and Other Design Factors

	Project Name				
	LA-710	SIS-5	TEH-5	SOL-80	SAC-5
Number of Lanes	6	4	4 to 6	4 to 6	6 to 10
Design Lane 2023 ESALs per year (million)	3.9	1.9	0.8	0.8	1.9
Surface	0.25	0.25	0.25	N/A	0.20
Intermediate	0.50	0.50	0.50	N/A	0.75
Rich Bottom or Leveling	0.25	0.15[a]	0.21	N/A	0.20
Total AC for Reconstruction or Lane Replacement	1.00	0.90	0.96	N/A	1.15
CSOL: Surface	0.25	0.25	N/A	0.20	0.20
CSOL: Intermediate and Leveling	0.42	0.50[a]	N/A	0.35[b]	0.55[b]
Total AC for CSOL	0.67	0.75	N/A	0.55	0.75

Note: Thicknesses in ft.; thicknesses are typical, and there may be some variation within projects.
[a] The leveling course in the SIS-5 project used regular HMA, and a rubberized stress absorbing membrane interlayer (SAMI-R) was placed on top.
[b] The leveling course in the SOL-80 and SAC-5 project used regular 0.10 ft. HMA, and a geosynthetic pavement interlayer was placed on top.

1.7 Objective

The objective of this technical memorandum is to evaluate the half-life performance of the AC Long Life projects that were built in Southern California in the early 2000s and the quarter-life performances of those built in Northern California in the early 2010s. The goal of the evaluation is to determine whether these pavements are on track to meet the expected life, assess design hypotheses, and learn lessons from the half- or quarter-life performance of these pavements.

1.8 Methodology

The following steps were followed to extract the data for evaluating the AC Long Life project performances:

- Find all construction activities associated with a given EA.
- Find all the highway segments covered by the construction activities found in Step 1.
- Find all performance and construction data associated with the highway segments found in Step 2.

The extracted data include performance and maintenance histories of the highway segments covered by the given AC Long Life project, both before and after construction. This approach allows for examination of the effect of each AC Long Life project on pavement performance and determination of whether any maintenance had been conducted within the project limits.

The performance of the projects was evaluated in terms of rutting, cracking, and smoothness, as calculated using the International Roughness Index (IRI). For each project, its performance is taken as the overall average of the performance of the automated pavement condition survey (APCS) data collection segments within the projects, weighted by segment length. As shown in Table 1.1some AC Long Life projects include more than one treatment. The performance was grouped by treatment type to indicate the corresponding effectiveness.

Sampling and laboratory testing of materials, deflection testing, and field observations were done periodically on the TEH-5 project since its construction, and more limited onsite sampling, laboratory testing, and field testing were performed on the other projects discussed in Chapter 4. The deflections taken on the TEH-5 project were used for backcalculation of layer stiffnesses, and additional laboratory testing was done on specimens cut from the pavement and binder extracted from the mix.

2 PERFORMANCE AND MAINTENANCE HISTORIES

In this chapter, the performance and maintenance histories of all the AC Long Life projects are summarized. Note that the SAC-5-Sacramento project is not included here because it is still too new.

2.1 Maintenance History

Across all the projects, the only maintenance on these AC Long Life projects so far was a 0.10 ft. (30 mm) mill and fill project (EA 07-4Y720) to replace the rubberized open-graded (RHMA-O) in 2011 and 2012 that covers the whole extent of the LA-710 project. The RHMA-O layer was not expected to last the 30-year design life; instead it was expected to act as a shorter-lived safety and noise reduction strategy and as a sacrificial layer to help with top-down cracking.

2.2 Cracking Performance

Caltrans uses alligator B and alligator A+B cracking (i.e., the combined alligator A and alligator B cracking) for rating cracking performance of flexible pavements *(17)*. Caltrans starts considering minor rehabilitation or maintenance when alligator A+B cracking reaches 5% and starts considering major rehabilitation when it reaches 30%. Alligator A+B cracking is only presented here if there is more than 5% of the wheelpath with alligator A+B cracking, in which case alligator B cracking is presented as well.

Alligator cracking is generally related to fatigue under heavy loads, and it is a reflection of underlying previous fatigue cracking or concrete slab cracking. In addition, cracking performance in flexible pavements can also be evaluated by reviewing transverse cracking, which is related to the aging of the asphalt surface (top-down) and a reflection of previous transverse cracking, concrete joints (bottom-up), and patching. Note that patching data have not been collected since the removal of patching from the Caltrans APCS manual in 2015.

2.2.1 Alligator Cracking

Alligator cracking performance of the AC Long Life projects is shown in Figure 2.1 to Figure 2.4. These figures show the construction window of each AC Long Life project as well as the timing of the maintenance activity, if any. According to these figures, alligator A+B cracking dropped significantly following the construction of AC Long Life projects and stayed close to zero when data were available, with the following exceptions:

- LA-710 Project: There are some spikes in alligator A+B cracking in this project after construction. Since there are no maintenance activities that would explain a return to zero cracking after these spikes, combined with the fact that there are multiple years of close to zero cracking post-spikes, it is believed that these spikes do not represent actual pavement conditions.
- SIS-5 Project: Alligator A+B cracking starts to appear roughly eight years after construction. Although in most cases it is still significantly lower than the pre-construction level, the alligator A+B cracking in the southbound Lane 2 placed with thick hot mix asphalt (HMA)

overlay has reached the same level as the pre-construction period. The alligator B cracking history for this project is shown in Figure 2.5, which indicates that alligator B cracking remains close to zero. The cracking in this project is therefore mostly alligator A cracking (i.e., the less severe form).

- SOL-80 Project: According to Google Earth satellite images taken before (May 2012) and after (February 2018) construction, the pavement surface type changed from mostly rigid with occasional flexible to mostly flexible with occasional rigid, which are culverts and under-bridge sections. This observation is consistent with the change in length of survey data before and after the construction window.

Note that when a pavement receives a CSOL treatment, the surface type changes from rigid to flexible. Instead of alligator A+B cracking, Caltrans uses the third-stage cracking (multiple cracks) for rating rigid pavement cracking performance *(17)*. In addition, Caltrans also records the first-stage cracking (one crack) for rigid pavements.

For segments that underwent CSOL treatment, there should only be first- and third-stage cracking before construction of the AC Long Life project and only alligator A+B cracking after construction. The combined first- and third-stage cracking histories for AC Long Life projects are shown in Figure 2.6 and Figure 2.7, respectively, for the SOL-80 and LA-710 projects. The following are notes regarding these plots:

- SIS-5 Project: No concrete cracking data were found for the SIS-5 project because the CSOL segments had asphalt concrete surfaces over the old PCC before the long-life construction. The existing overlay was completely removed and replaced with new long-life HMA overlays on the underlying concrete.
- SOL-80 Project: There was significant combined first- and third-stage cracking in the outside lanes for the SOL-80 project before construction of the long-life asphalt pavement.
- LA-710 Project: There was significant combined first- and third-stage cracking in the outside lanes in both directions for the LA-710 project before the long-life construction. Note that Lane 2 and Lane 3 are outside lanes because Lane 4 only exists as a weaving lane for on-ramps and off-ramps.

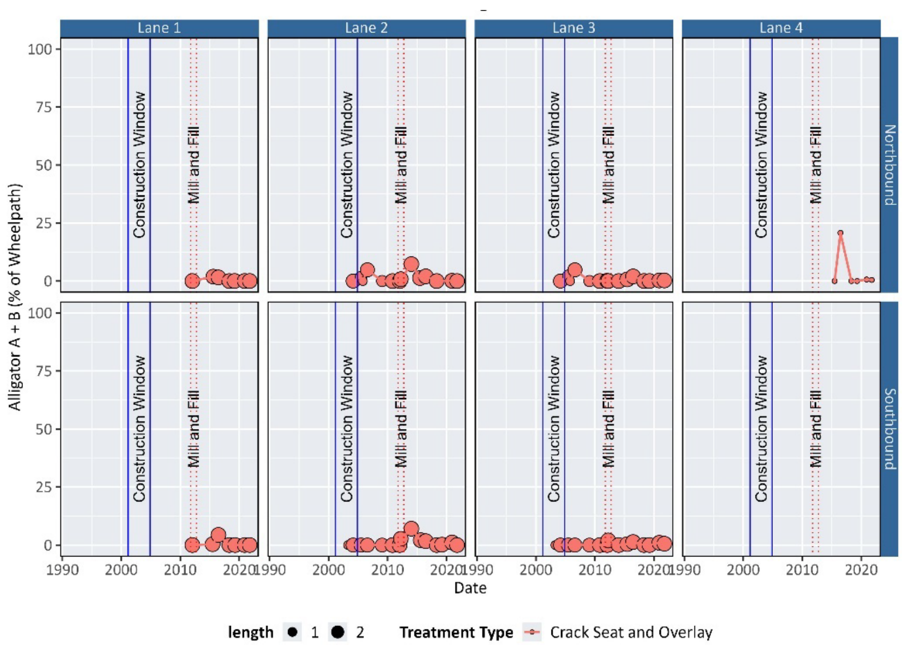


Figure 2.1: Alligator A+B cracking performance histories for the LA-710 project.

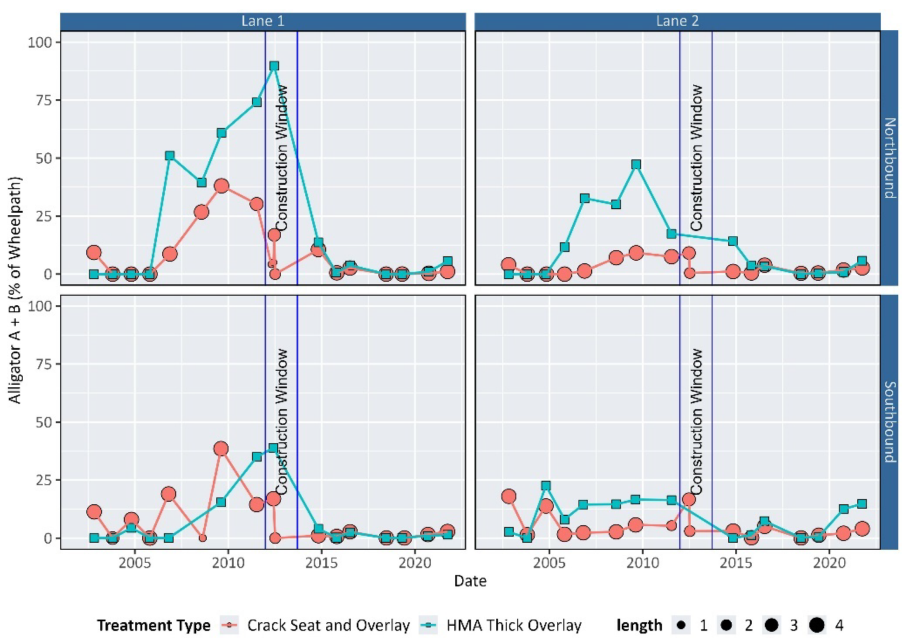


Figure 2.2: Alligator A+B cracking performance histories for the SIS-5 project.

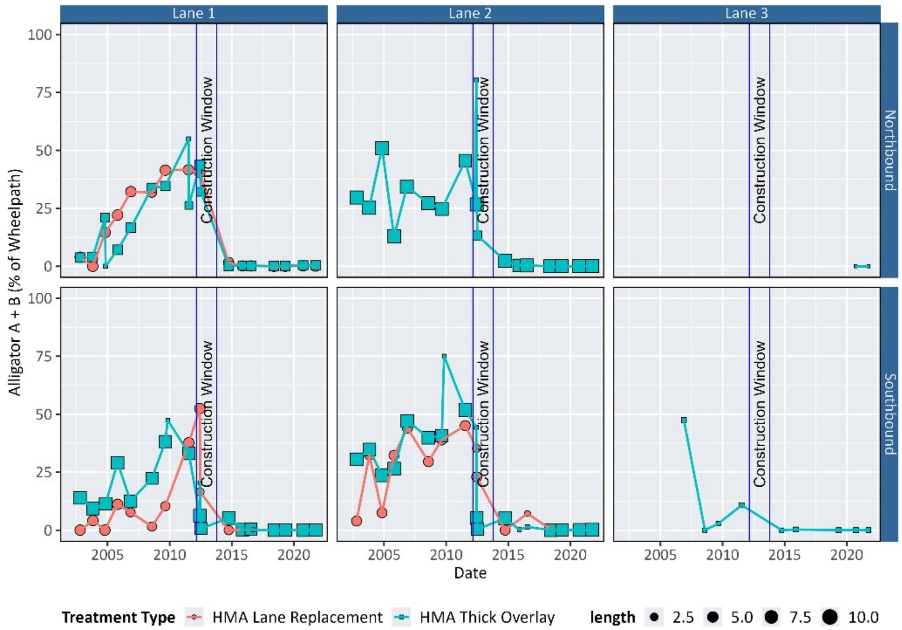


Figure 2.3: Alligator A+B cracking performance histories for the TEH-5 project.

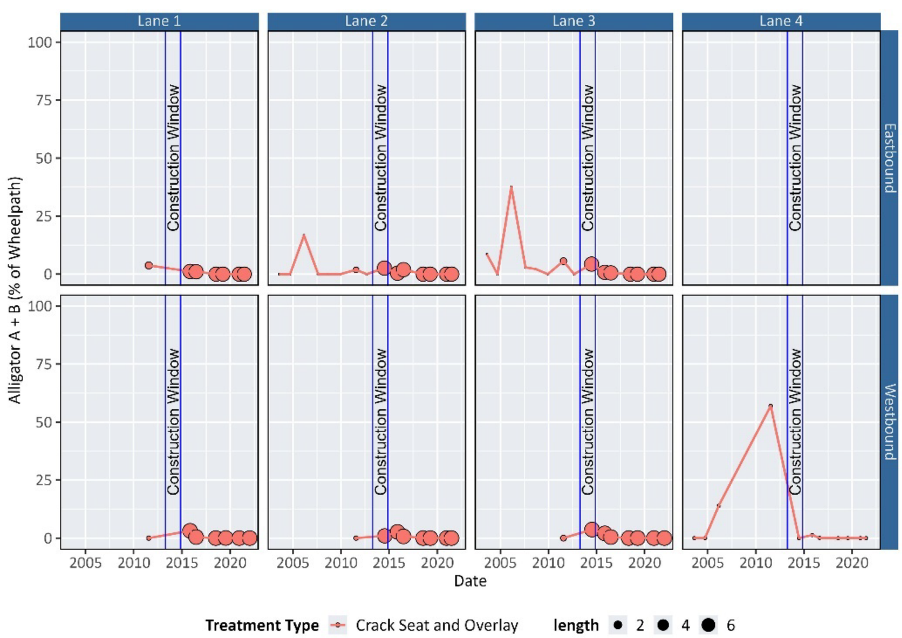


Figure 2.4: Alligator A+B cracking performance histories for the SOL-80 project.

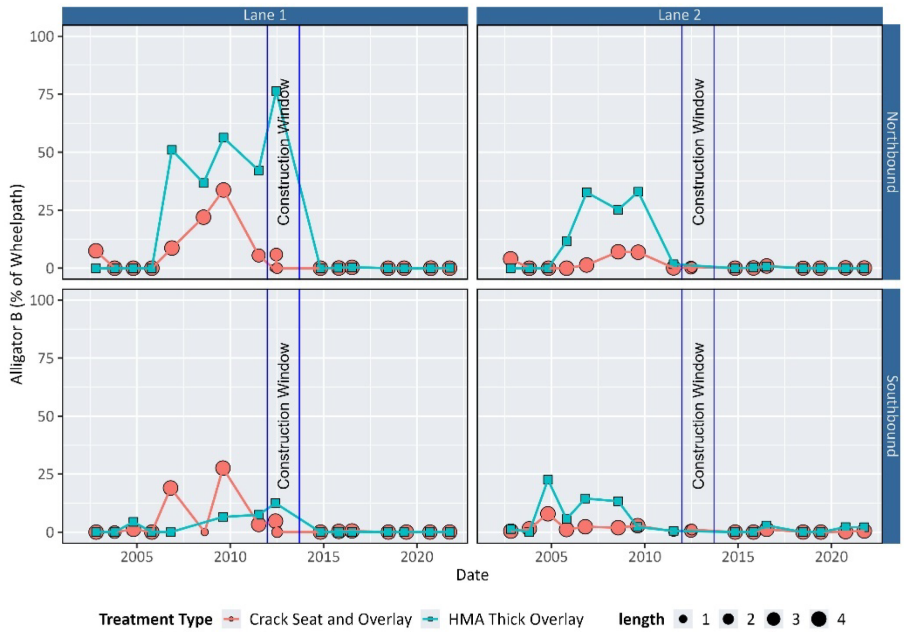


Figure 2.5: Alligator B cracking performance histories for the SIS-5 project.

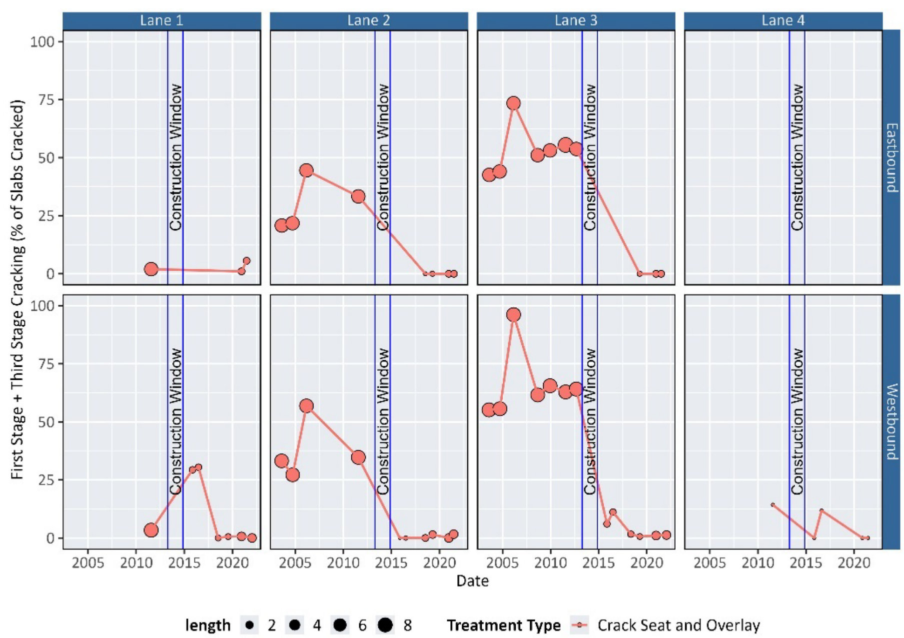


Figure 2.6: First- and third-stage cracking performance histories for the SOL-80 project.

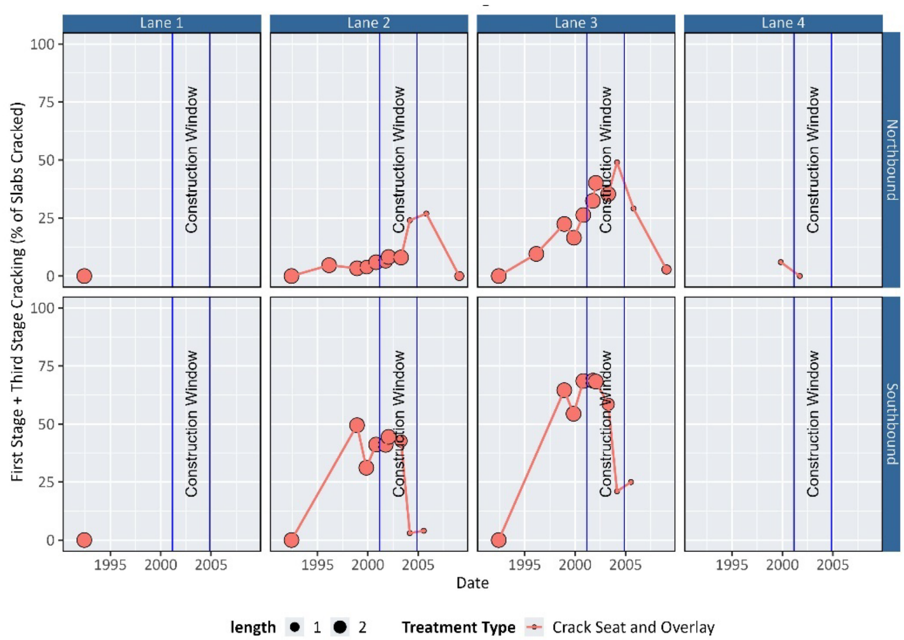


Figure 2.7: First- and third-stage cracking performance histories for the LA-710 project.

2.2.2 Transverse Cracking

Transverse cracking in pavements is recorded as the number of cracks per 100 ft. for flexible surfaces. The transverse cracking histories for the AC Long Life projects are shown in Figure 2.8 to Figure 2.11. As these figures suggest, there is very little transverse cracking in these projects so far.

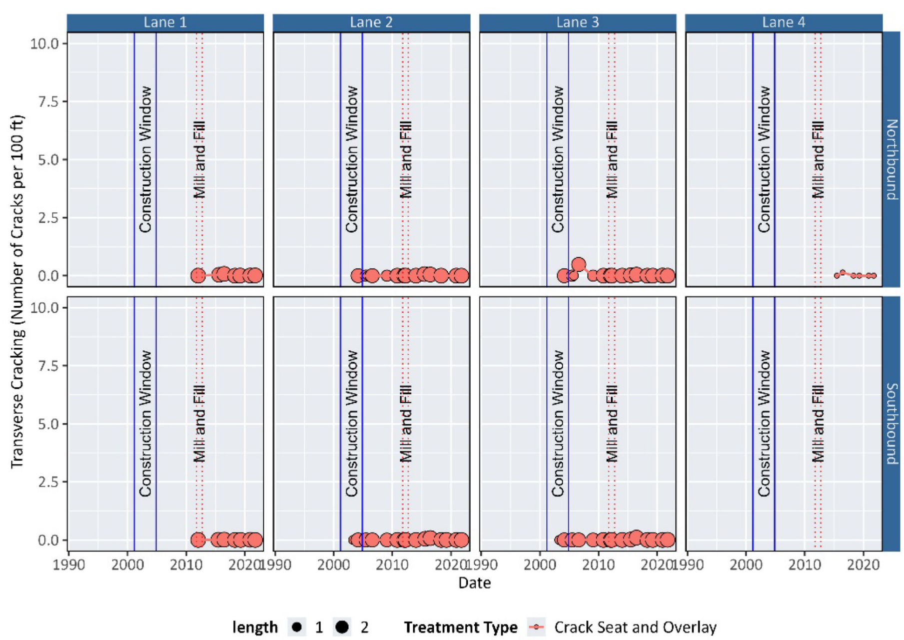


Figure 2.8: Transverse cracking performance histories for the LA-710 project.

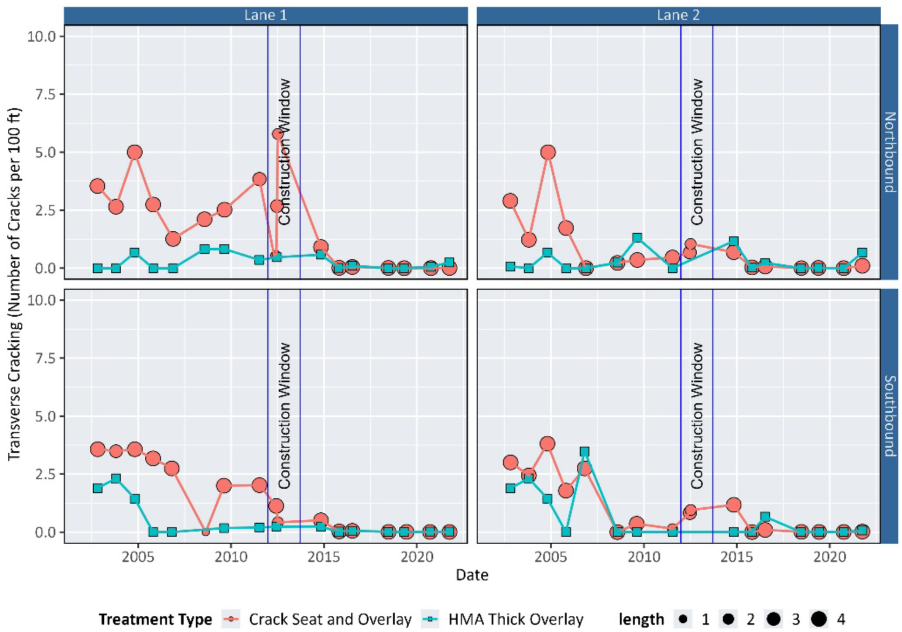


Figure 2.9: Transverse cracking performance histories for the SIS-5 project.

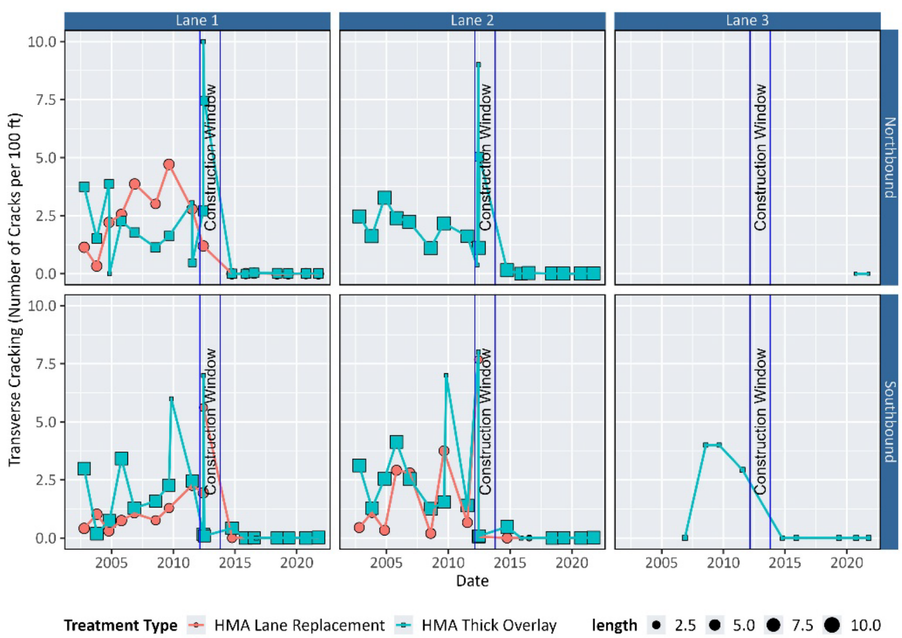


Figure 2.10: Transverse cracking performance histories for the TEH-5 project.

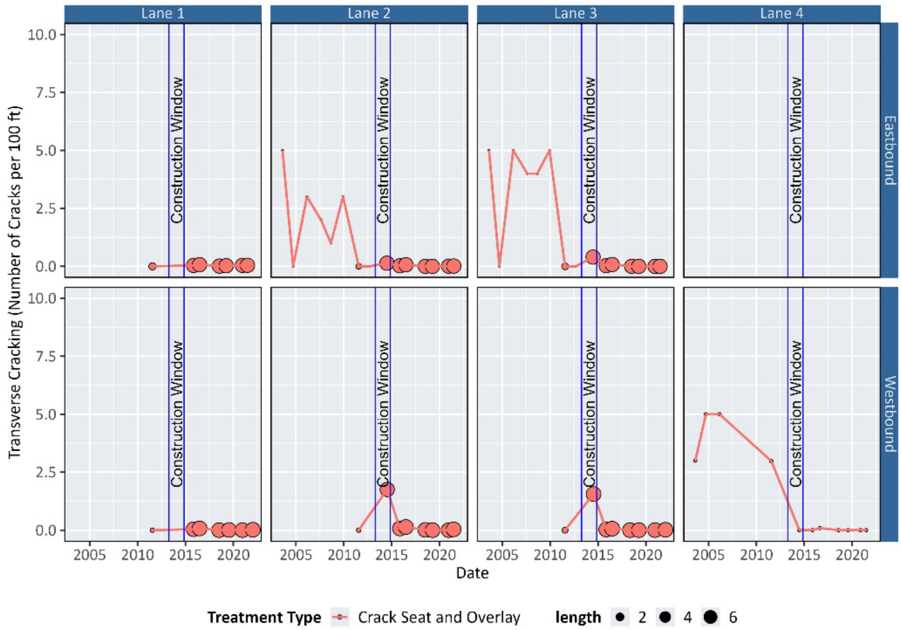


Figure 2.11: Transverse cracking performance histories for the SOL-80 project.

2.2.3 Patching

Patching in pavements is recorded as a percentage of the wheelpath patched. The patching histories for the AC Long Life projects are shown in Figure 2.12 to Figure 2.15. As mentioned previously, patching data stopped in 2015. As these figures suggest, there was no patching in these projects by 2015, except for about 12% of the wheelpath patched in southbound Lane 2 for the LA-710 project. This is believed to be a mistake because there was no need for patching, given that there was no cracking in the LA-710 project. This is corroborated by the fact that the patching and cracking did not occur on the same pavement segments in this project.

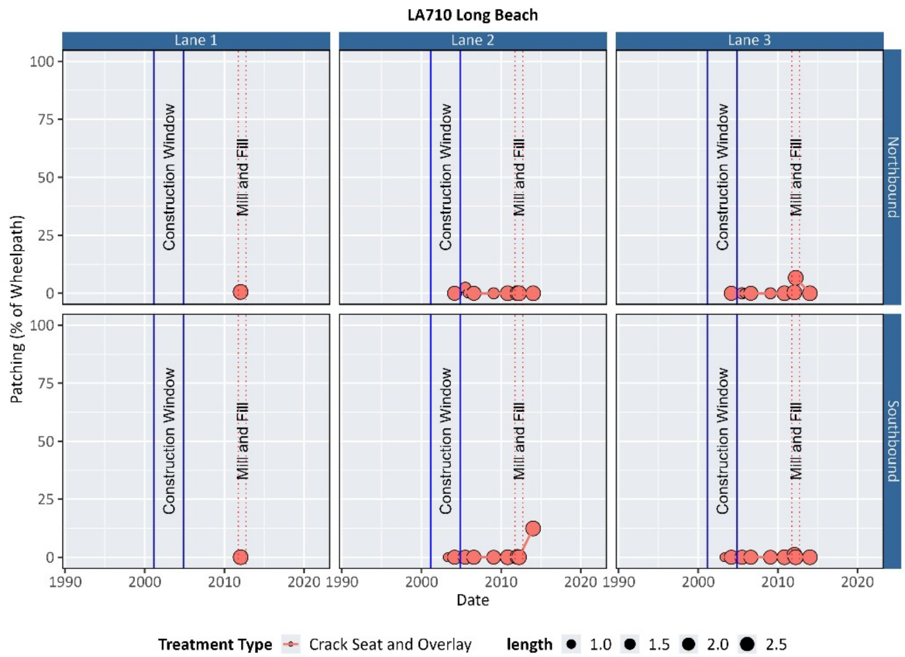


Figure 2.12: Patching histories for the LA-710 project.

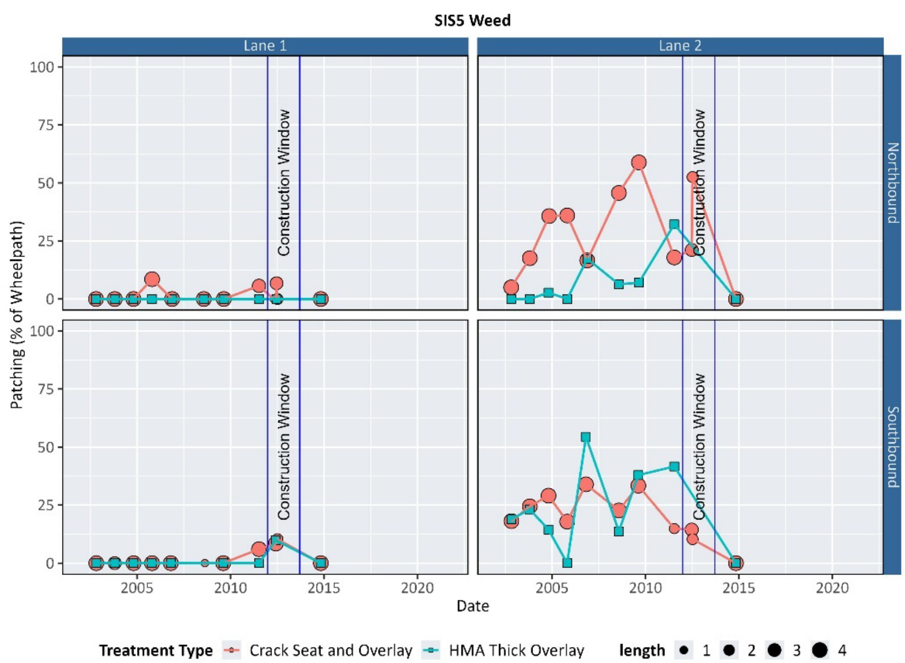


Figure 2.13: Patching histories for the SIS-5 project.

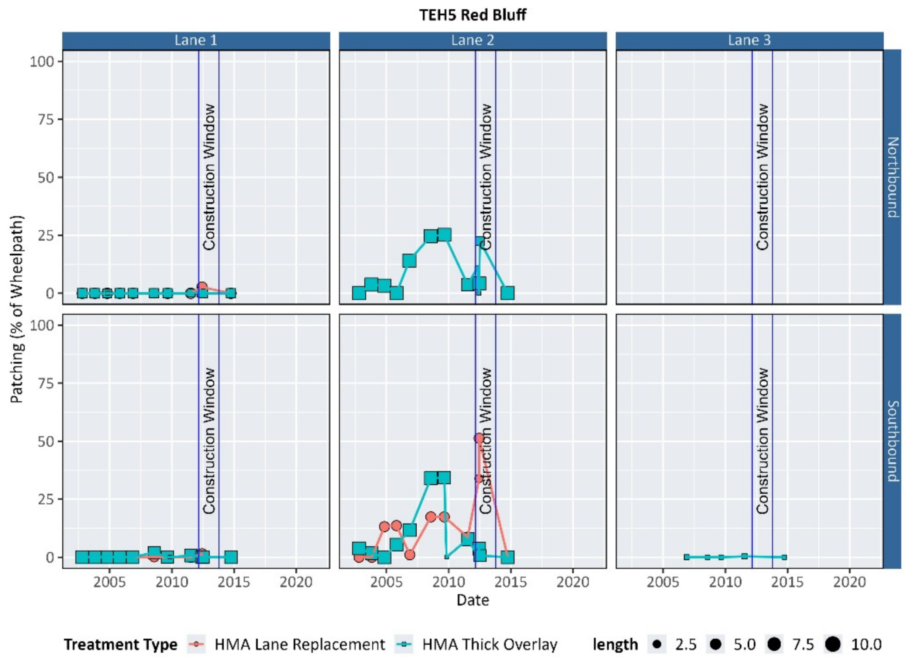


Figure 2.14: Patching histories for the TEH-5 project.

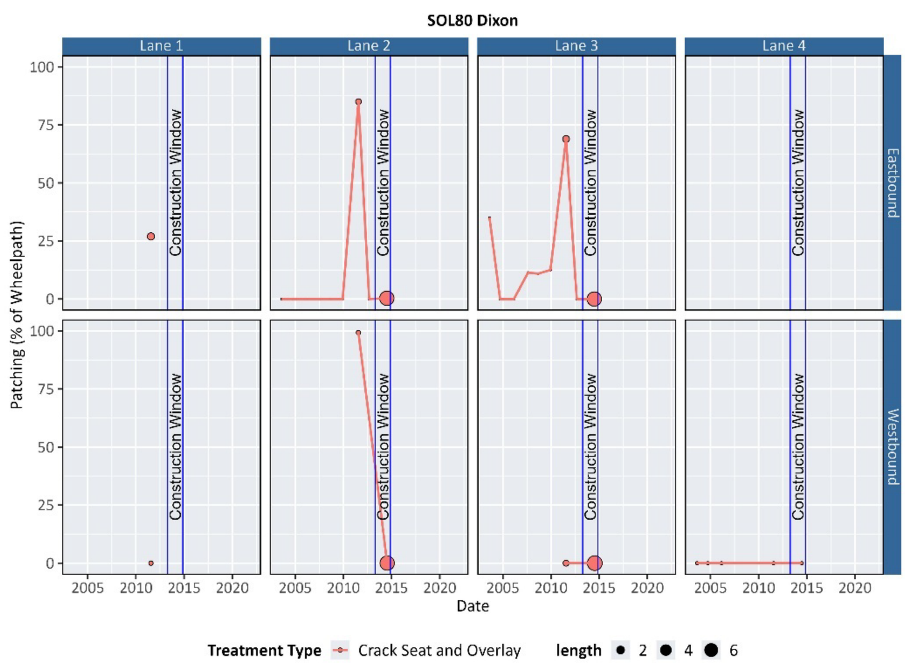


Figure 2.15: Patching histories for the SOL-80 project.

2.3 Rutting Performance

Rutting in pavements is recorded as the average and maximum rut depth in the wheelpaths. Starting from 2011, the standard deviation of rut depth was added after the adoption of the APCS. Note that rutting is not included in the pavement rating system used by Caltrans.

It was found that the maximum rut data trends tend to be noisy and record very short segments that the sensors indicate have the rutting distress. On the other hand, the average rut tends to be more consistent over time and shows more variation between different projects, which makes it a better measurement for representing pavement rutting performance.

The addition of the standard deviation of rut depth allows for the calculation of the percent of wheelpath with rut depth exceeding a certain threshold, assuming rut depth follows a normal distribution. It was found that using 0.25 in. (6.25 mm) is a better threshold than 0.5 in. (12.5 mm) because the 0.25 in. threshold is an early indicator of rutting for comparison between different projects, while the 0.5 in. threshold leads to zero probability of being exceeded most of the time because it is considered a rutting failure, and pavements are seldom allowed to reach that level. The percentage of the wheelpath with rut exceeding 0.25 in. is hereafter referred to as PWR6 (percent wheelpath rutted that has reached a threshold of 6.25 mm).

The rutting histories for the AC Long Life projects are shown in Figure 2.16 to Figure 2.19 for average rut and in Figure 2.20 to Figure 2.23 for PWR6. As these figures suggest, there was some rutting in the LA-710 and SIS-5 projects and very little rut in the other project. The following are observations about rutting in these projects:

- The maximum average of the average rut is less than 0.2 in. (5 mm) in the LA-710 and SIS-5 projects.
- The maximum percent of wheelpath with more than 0.25 in. (6.25 mm) of rut is 15% in the SIS-5 project and 3% for the other projects.
- For the LA-710 project, the rut shows an initial increase followed by a consistent decrease. It is not clear why the rutting, judging by either the average rut or the PWR6, decreased.
- For the SIS-5 project, the average rut barely increased, but the PWR6 increased significantly in the outside lanes due to an increase in the standard deviation of rut depth. This finding demonstrates the need to account for rut depth variability.

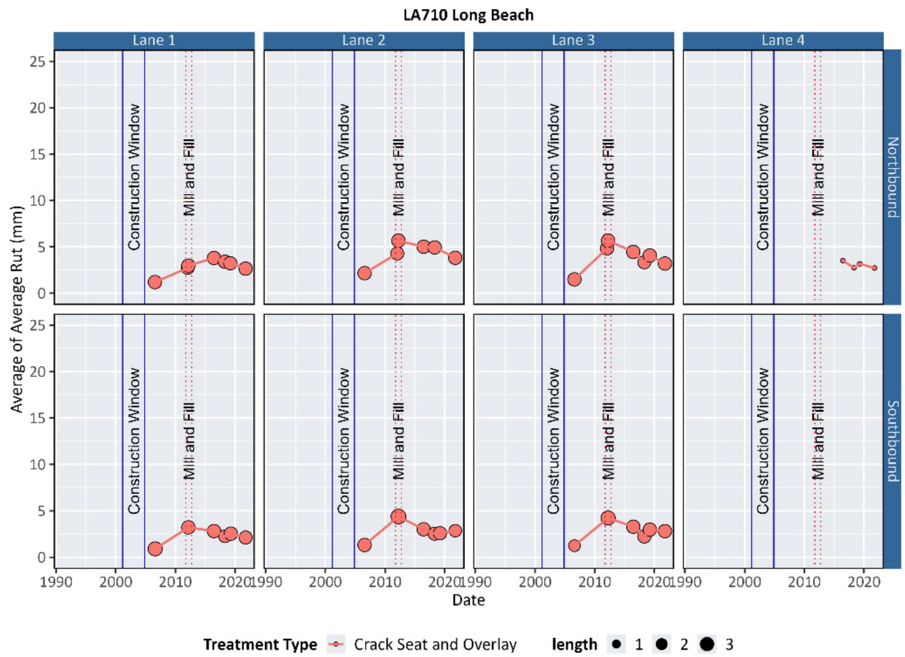


Figure 2.16: Average of average rut histories for the LA-710 project.

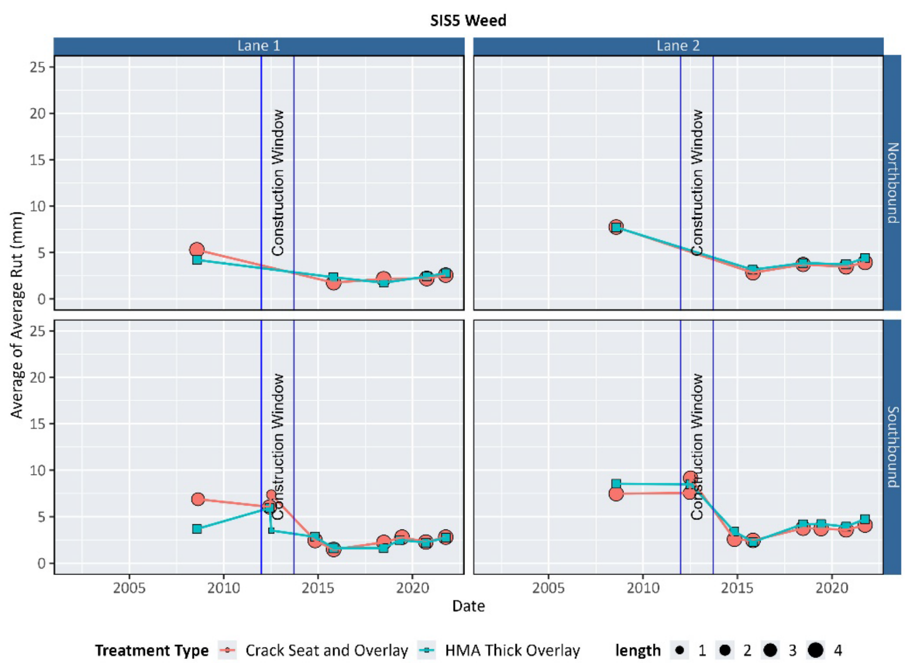


Figure 2.17: Average of average rut histories for the SIS-5 project.

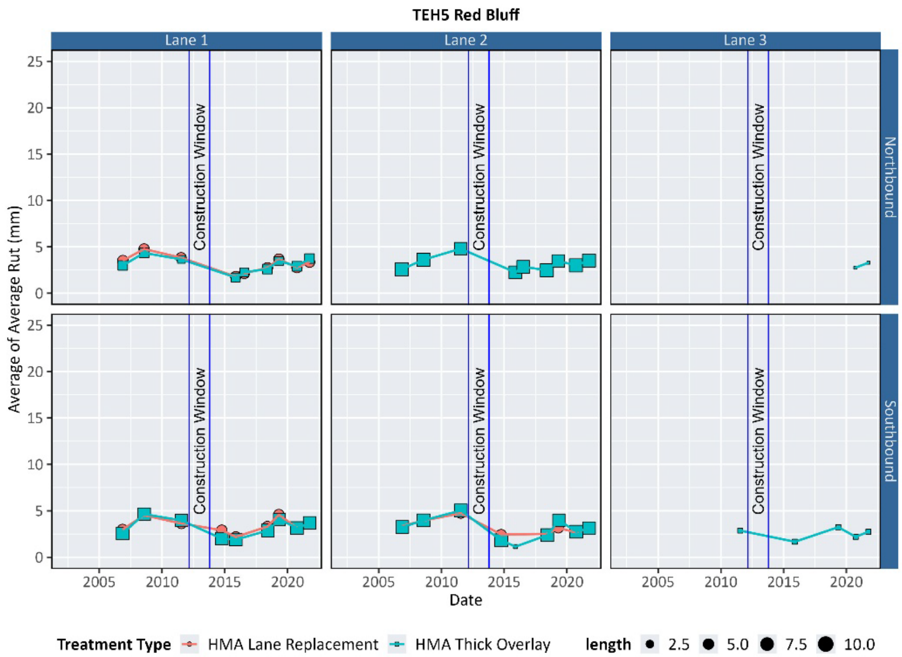


Figure 2.18: Average of average rut histories for the TEH-5 project.

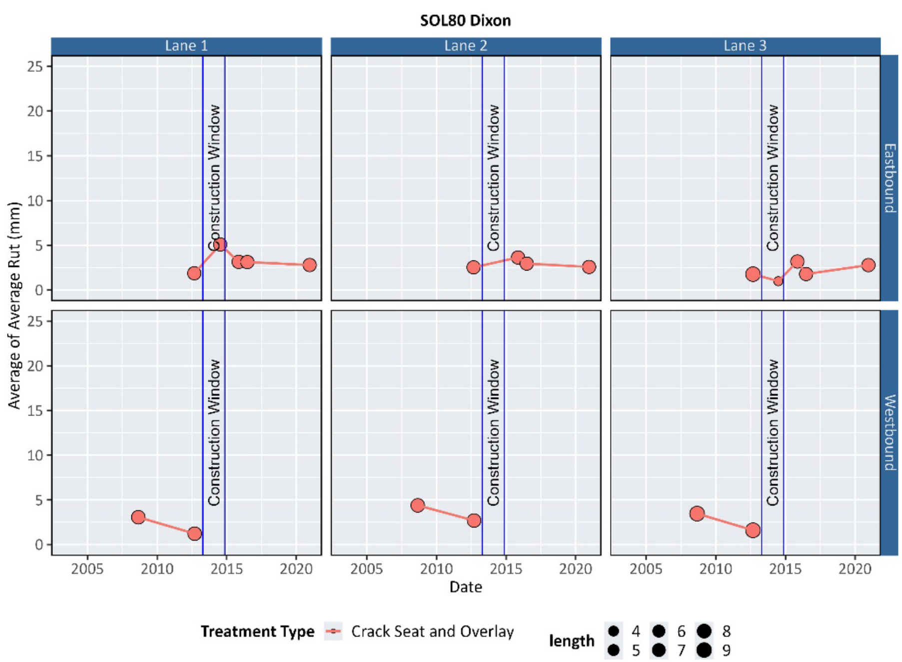


Figure 2.19: Average of average rut histories for the SOL-80 project.

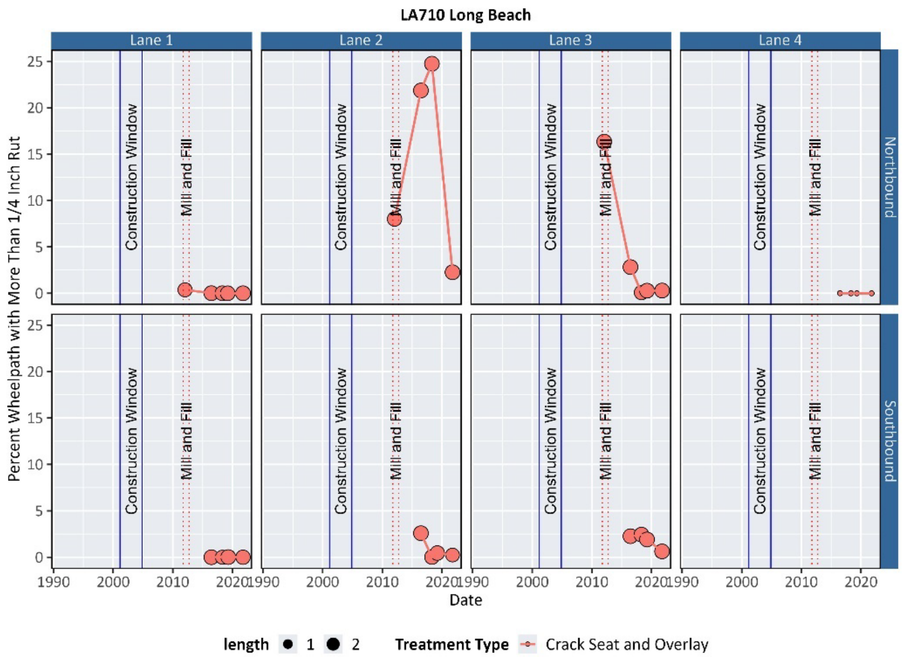


Figure 2.20: Histories of percent wheelpath with more than 0.25 in. rut for the LA-710 project.

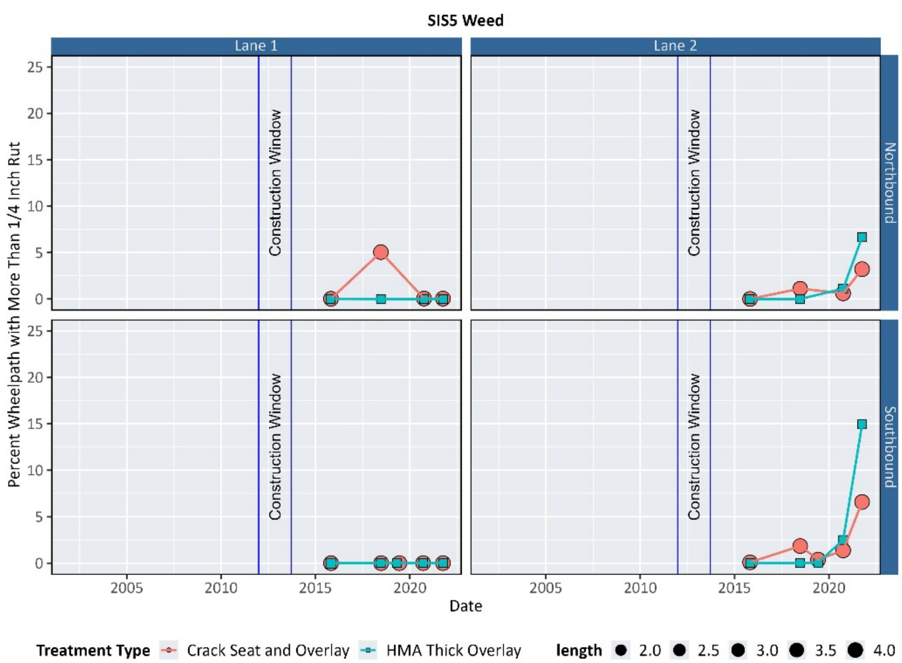


Figure 2.21: Histories of percent wheelpath with more than 0.25 in. rut for the SIS-5 project.

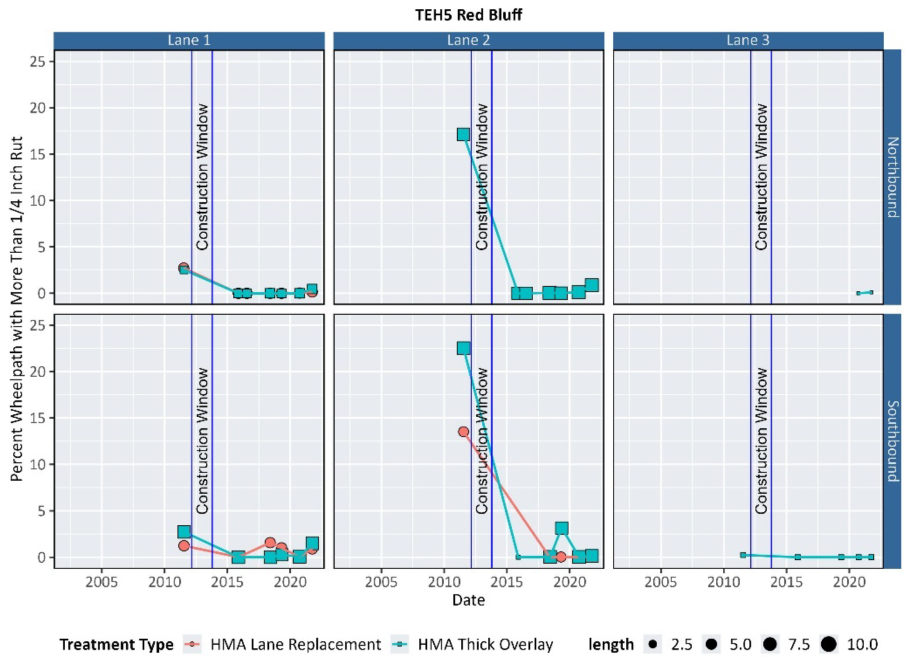


Figure 2.22: Histories of percent wheelpath with more than 0.25 in. rut for the TEH-5 project.

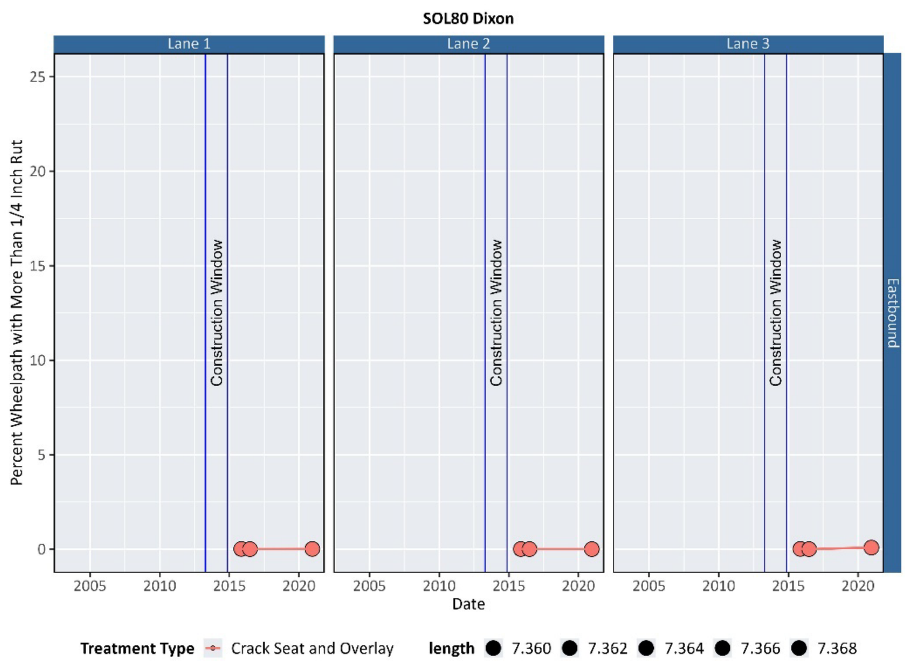


Figure 2.23: Histories of percent wheelpath with more than 0.25 in. rut for the SOL-80 project.

2.4 Pavement Roughness Performance

Pavement roughness is recorded using the International Roughness Index (IRI), which is included in the pavement rating system used by Caltrans. The threshold separating low IRI from high IRI is 170 in./mi. The IRI histories for the AC Long Life projects are shown in Figure 2.24 to Figure 2.27. As these figures suggest, all the projects remain smooth after construction, with the IRI well below the 170 in./mi. threshold. In fact, IRI values remained slightly below 100 in./mi. for the LA-710 project, which was built with early 2000s construction smoothness specifications and had greater constructed roughness than the other projects, and around 50 in./mi. for the other three projects.

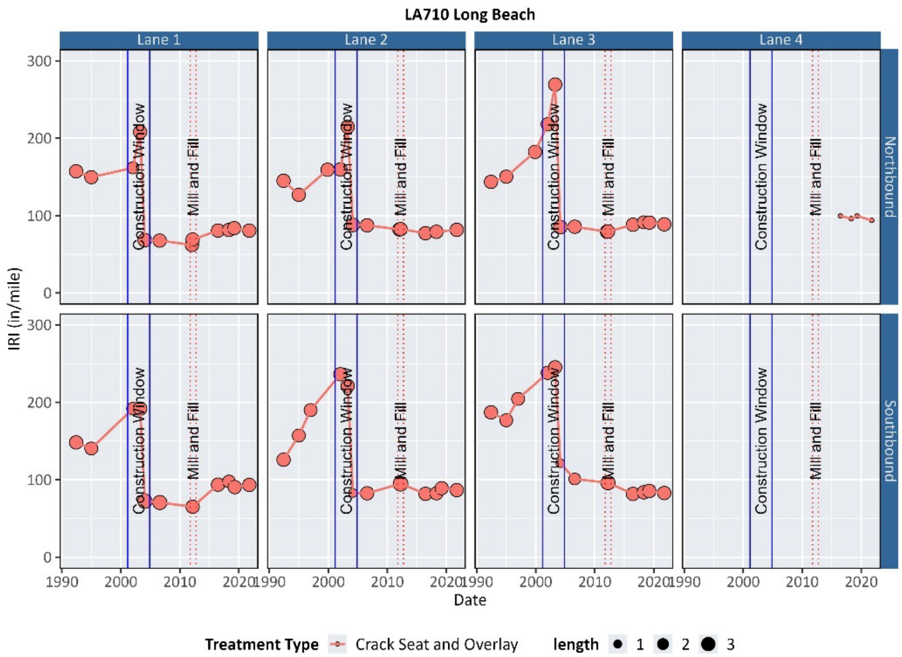


Figure 2.24: IRI histories for the LA-710 project.

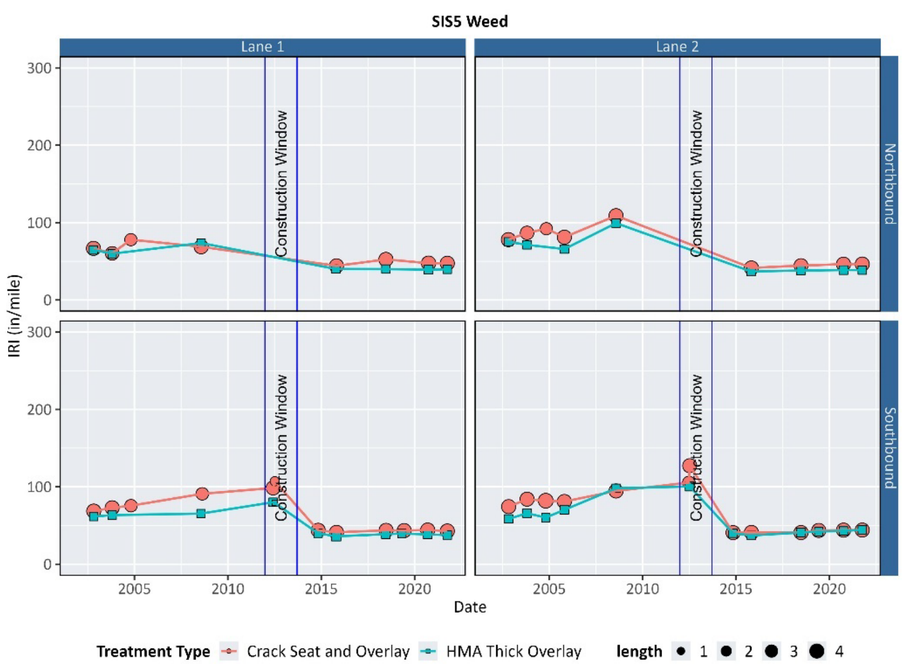


Figure 2.25: IRI histories for the SIS-5 project.

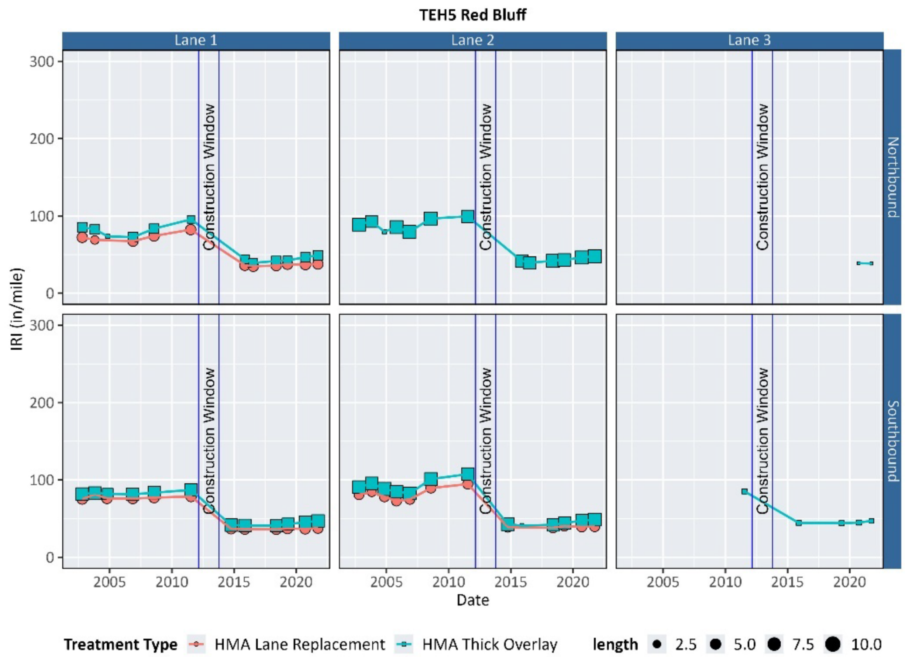


Figure 2.26: IRI histories for the TEH-5 project.

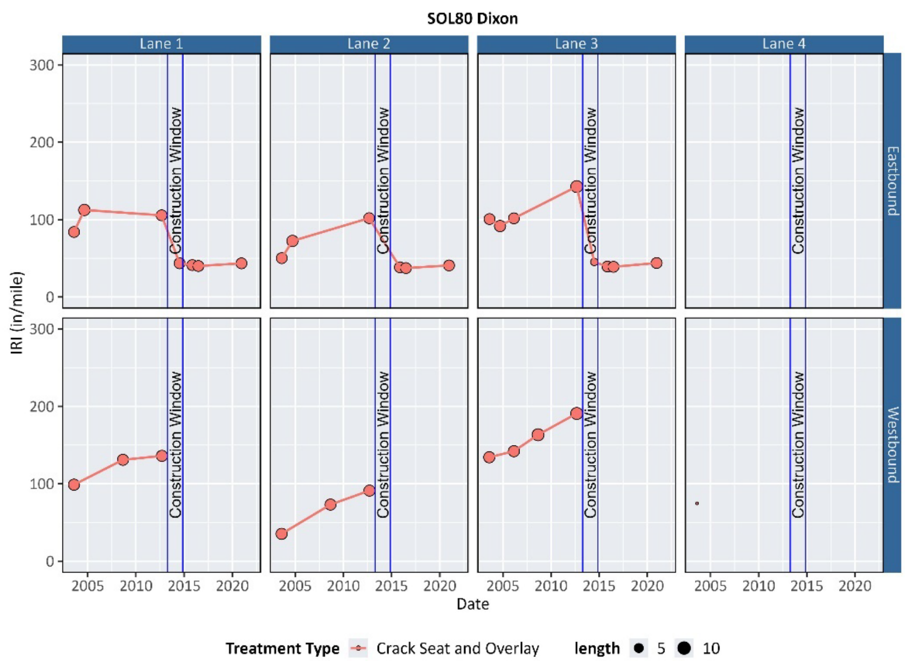


Figure 2.27: IRI histories for the SOL-80 project.

2.5 Summary

In summary, the performance histories of the four older AC Long Life projects were reviewed by examining the maintenance, cracking, rutting, and roughness histories. The review periods span from 10 years before construction of each project, showing the condition prior to the long-life rehabilitation, to 2021, the year for which the latest performance and maintenance data are available.

At the end of the review period, the LA-710 project was 17 to 20 years old, while the other three projects were seven to 10 years old. All four projects are found to be performing very well in general, as suggested by the following findings:

- There was generally no alligator or transverse cracking in these projects after construction, even though up to 75% of the wheelpath had alligator A+B cracking or up to 60% of slabs with first- or third-stage cracking just before construction. The only exception occurred in the SIS-5 project, which had about 15% of alligator A cracking eight years after construction in one of the truck lanes placed with a thick long-life HMA overlay. This amount of cracking roughly equals the amount of cracking right before construction in that lane.
- There was no patching in the data up to 2015, after which the data were no longer collected.
- There was some minor rutting in the LA-710 and SIS-5 projects, both starting about eight years after construction.
- All the projects remained smooth after construction. IRI values remained slightly below 100 in./mi. for the LA-710 project after being constructed to an average IRI of 90 in./mi. and around 50 in./mi. for the other three projects after being constructed to an average IRI of 50 in./mi.
- There was no maintenance activity other than a 0.10 ft. (30 mm) mill and fill project to replace the sacrificial RHMA-O layer about eight years after construction that covered the whole extent of the LA-710 project. The RHMA-O was not expected to last the entire 30-year design life of the underlying layers. Given that there was no cracking in this project, it is believed that the purpose was to replace the open-graded surface layer for functional rather than structural reasons.

3 SAMPLING AND TESTING FOR THE LA-710 LONG BEACH PROJECT

Extensive monitoring, sampling, and testing have been conducted for the LA-710 project. One study summarized the design, construction, and performance data for the period November 2003 to January 2009 (five-plus years under traffic), as well as the results of additional analyses and tests on mixes obtained from cores and slabs taken from the pavement after construction *(18)*.

To evaluate the latest condition of the pavement, a new round of sampling and testing was conducted in 2023. The field testing included deflection testing on Lane 3 throughout the project in both directions and on southbound Lane 1. The field sampling included taking 14 full-depth, 6 in. cores on the southbound shoulder at 20 ft. (6 m) intervals near the Wardlow Road Overpass between PM 9.1 and PM 8.8. Ten of the cores were taken from under the overcrossing. These cores were used to determine the aging gradient with depth.

3.1 Deflection Testing and Stiffness Backcalculation

Deflection testing was conducted on the LA-710 project in 2003, 2004, 2005, 2006, 2008, and 2023 (Table 3.1). The testing did not concentrate on specific months. In the early years, i.e., 2003 to 2008, the testing was done along the right wheelpath only. In 2023, testing between the wheelpaths was included as well. There was no day/night replicate testing in any specific year. Three falling weight deflectometers (FWD) were used throughout the years that were operated by UCPRC, Dynatest, and Caltrans.

Table 3.1: Number of FWD Drops on the LA-710 Project

Line Tested[a]	2003	2004	2005	2006	2008	2023
NBL1-RWP	932	306	300	—	—	—
NBL2-RWP	1,089	—	297	—	294	—
NBL3-RWP	1,163	408	300	—	288	239
SBL1-RWP	807	—	747	231	—	296
SBL2-RWP	777	—	—	219	294	—
SBL3-RWP	786	—	741	258	324	273
NBL3-BWP	—	—	—	—	—	245
SBL3-BWP	—	—	—	—	—	282

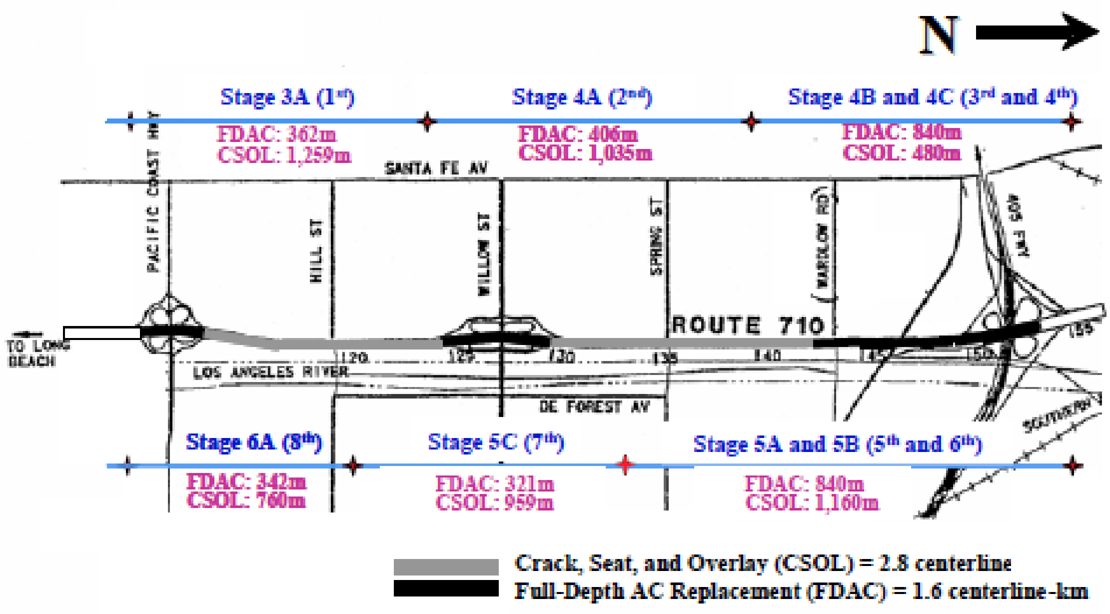
[a] The line label indicates direction, lane, and lateral position of the FWD testing (RWP = right wheelpath, BWP = between wheelpaths).

Although the Caltrans FWD testing was conducted with 10 sensors, only data from sensors number 1, 2, 3, 4, 6, and 7 were used. These sensors were located at 0, 8, 12, 18, 36, and 48 in., respectively, from the center of the loading plate. Sensor numbers 5 and 8 were found to be unreliable, and numbers 9 and 10 were ahead of the loading plate.

There were different conventions for recording the drop location. Some variations in terms of testing locations and intervals exist between different test sessions. The deflection recorded by the outer sensors (i.e., more than 2.0 ft. away from the loading plate) was mostly determined by subgrade stiffness and so should have a consistent pattern along the testing line. Accordingly, the deflection pattern for the geophone that was 4 ft. (1,219 mm) away from the center of the loading plate was used to do a small realignment of data. Note that only the starting locations are adjusted.

3.1.1 Structures Used for Backcalculation

The project was constructed with two cross sections: full-depth asphalt concrete (FDAC) under the overcrossings and crack, seat, and overlay (CSOL) between the overcrossings. The segmentation of the project is shown in the staging plan in Figure 3.1.



Source: Monismith (2009) *(18)*.

Figure 3.1: Contractor's staging plan for the LA-710 project.

The cross sections are listed in Table 3.2. As shown in the table, only the FDAC segments include the rich bottom HMA layer.

Table 3.2: Cross Sections for the LA-710 Project

Layer Number	Full-Depth Asphalt Concrete (FDAC)	Crack, Seat, and Overlay (CSOL)
1	0.08 ft. open-graded friction course	Same as FDAC
2	0.25 ft. of HMA with polymer-modified binder	Same as FDAC
3	0.50 ft. of HMA	0.42 ft of HMA with fabric interlayer
4	0.25 ft. of rich bottom HMA	N/A
5	0.5 ft. of recycle concrete aggregate subbase	1.0 ft of crack and seated PCC
6	Subgrade	Subgrade

Note: Cross sections based on Figure 3.11 and Figure 3.19 from Monismith et al. (2009) *(18)*.

The segments were labeled as Sections 1, 2, 3, 4, and 5 from south to north for all lanes in both directions. The boundaries for the segments are shown in Table 3.3

Table 3.3: Segment Boundaries for the LA-710 Project

Section Number	Cross Section	Postmile at South End (mile)	Postmile at North End (mile)
1	Full-depth asphalt concrete (FDAC)	6.75	6.95
2	Crack, seat, and overlay (CSOL)	6.95	7.75
3	Full-depth asphalt concrete (FDAC)	7.75	8.00
4	Crack, seat, and overlay (CSOL)	8.00	9.00
5	Full-depth asphalt concrete (FDAC)	9.00	9.50

The structures used for backcalculation was determined and are listed in Table 3.4. As shown in the table, the cross sections have been consolidated into three-layer systems.

Table 3.4: Structures Used for Stiffness Backcalculation for the LA-710 Project

Layer Number	Full-Depth Asphalt Concrete (FDAC)	Crack, Seat and Overlay (CSOL)
1	1.08 ft. of asphalt concrete	0.75 ft of asphalt concrete
2	0.5 ft. of recycle concrete aggregate subbase	1.0 ft of Crack and seated PCC
3	Semi-infinite subgrade	Semi-infinite subgrade

3.1.2 Accounting for Temperature Effects on Layer Stiffness

The FWD testing was conducted at different pavement temperatures. Since asphalt concrete stiffness is sensitive to temperature, it is important to account for the effects of temperature on backcalculated stiffnesses before they can be used to identify trends between years, lanes, and lateral positions.

Pavement temperatures were calculated using the BELLS3 equation *(20)* for the one-third depth of the combined asphalt-bound layer based on measured surface temperature and average air temperature of the day before testing. The air temperature was retrieved from the National Oceanic and Atmospheric Administration (NOAA) Climate Data Online through web service calls using the Global Historical Climatology Network daily (GHCNd) dataset for the Long Beach Daugherty Airport (station number GHCND:USW00023129).

The correlation between log10 of asphalt-bound layer stiffness and the BELLS temperature is shown in Figure 3.2, which shows that most of the scatter comes from the results associated with the Caltrans FWD. It was therefore decided only the results from the other two FWDs would be used to develop the temperature correction factor for the asphalt-bound layer stiffness.

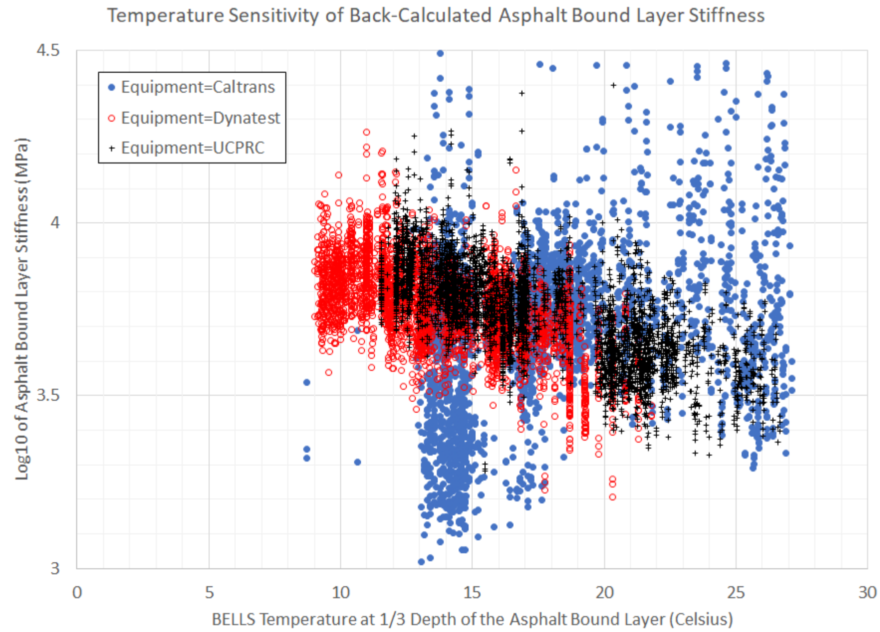


Figure 3.2: Correlation between BELLS temperature and log10 of asphalt-bound layer stiffness.

After removing the results from the Caltrans FWD, the correlation between the BELLS temperature and log10 of the asphalt-bound layer stiffness was calculated, as shown in Figure 3.3. No clear distinction in correlations was found between the FDAC and CSOL segments. Accordingly, the data from all segments were combined to develop the regression equation shown in Figure 3.3.

The correlation between the BELLS temperature and backcalculated PCC layer stiffness is shown in Figure 3.4. Note that data from all FWD equipment are used, as no extra scatter was found for results associated with any of the FWD. No temperature correction was found necessary for the recycled aggregate base in the FDAC segments and the subgrade in all segments.

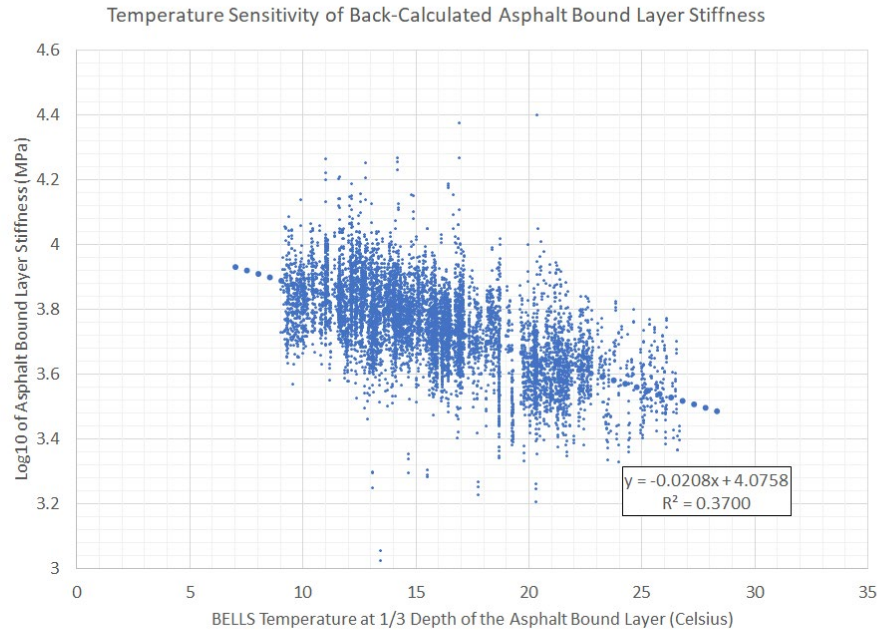


Figure 3.3: Correlation between the BELLS temperature and log10 of asphalt-bound layer stiffness after removing data from the Caltrans FWD.

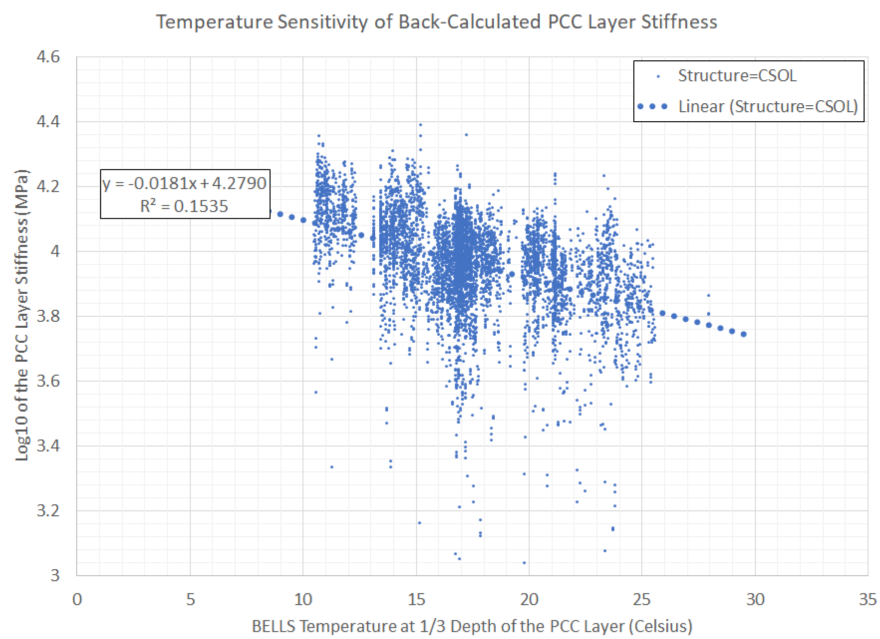


Figure 3.4: Correlation between the BELLS temperature and log10 of backcalculated PCC layer stiffness.

3.1.3 Accounting for Spatial Variability and Detecting Outliers

The FWD drop locations between different years were not perfectly aligned. In addition, questionable data tended to occur in random locations. To make the backcalculated stiffness comparable over time, the results needed to be adjusted for spatial variability.

To account for spatial variability, the backcalculated stiffness for a given layer was plotted against the postmile (PM). As expected, there is a great deal of noise in the plot. A Savitzky-Golay noise filter with a polynomial of rank 3 and frame size of 201 was used to filter the noise and develop a smooth curve. This smooth curve is hereafter referred to as the smoothed spatial correlation (SSC). Since any deviation from the local average is considered noise, the spatial adjustment has to be applied after the temperature effect has been removed.

The SSC is essentially a piecewise function that provides expected stiffness for any location in the project. Similar to the temperature adjustment, a location within the project is selected as the reference. The spatial adjustment is performed using Equation 3.1:

$$E_{RefX,20C}(x) = \frac{E_{20C}(x)}{E_{SSC}(x)} \cdot E_{SSC}(RefX) = E_{20C}(x) \cdot \frac{E_{SSC}(RefX)}{E_{SSC}(x)} \qquad \textbf{(3.1)}$$

Where:

$E_{RefX,20C}(x)$ = backcalculated stiffness at location x adjusted to reference location and 20°C

$E_{20C}(x)$ = backcalculated stiffness at location x after adjusted to 20°C

$E_{SSC}(x)$ = backcalculated stiffness at location x determined by the SSC function

$E_{SSC}(RefX)$ = backcalculated stiffness at reference location determined by the SSC function

As shown in the equation, the spatial adjustment is performed by scaling, so the SSC function needs to be developed by averaging the logarithmic results.

The spatial smoothing was performed separately for the two cross sections. The reference locations are determined for individual layer and structure by finding the location with the median results. This is needed to make the median results remain unchanged before and after adjusting for spatial variability.

The spatial smoothing is shown in Figure 3.5 to Figure 3.7 for each of the layers for the 2023 deflection results. Also shown in the figures is the detection of outliers. Outliers were defined as backcalculated stiffnesses that are outside of the 95% confidence interval estimated based on the pooled results. The outliers were excluded from further analysis.

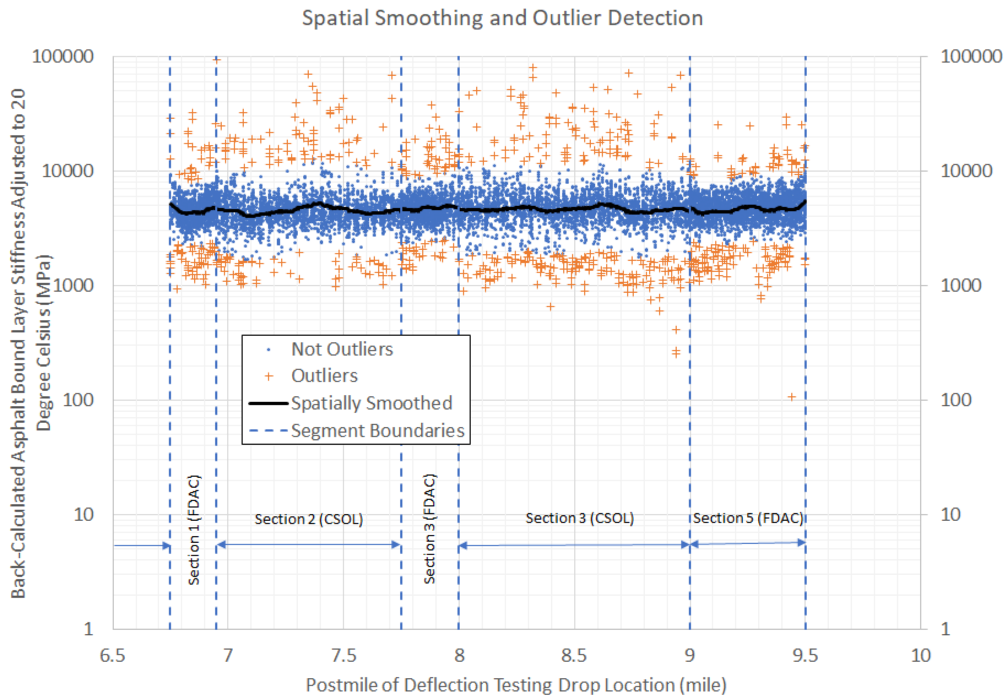


Figure 3.5: Spatial smoothing and outlier detection for backcalculated asphalt-bound layer stiffness.

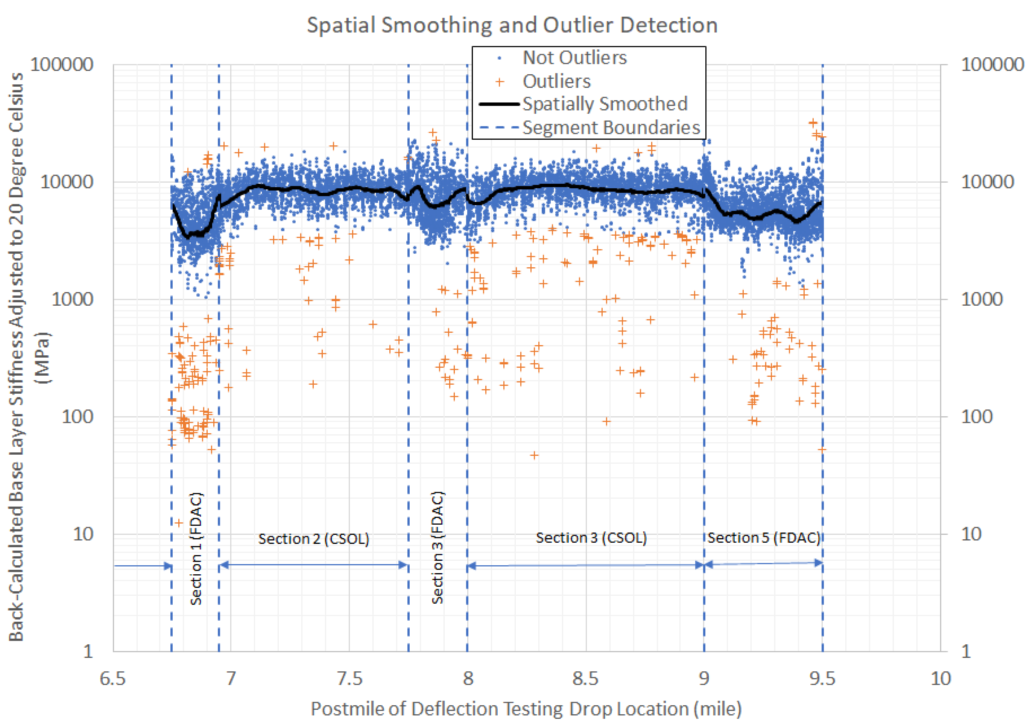


Figure 3.6: Spatial smoothing and outlier detection for backcalculated base layer stiffness.

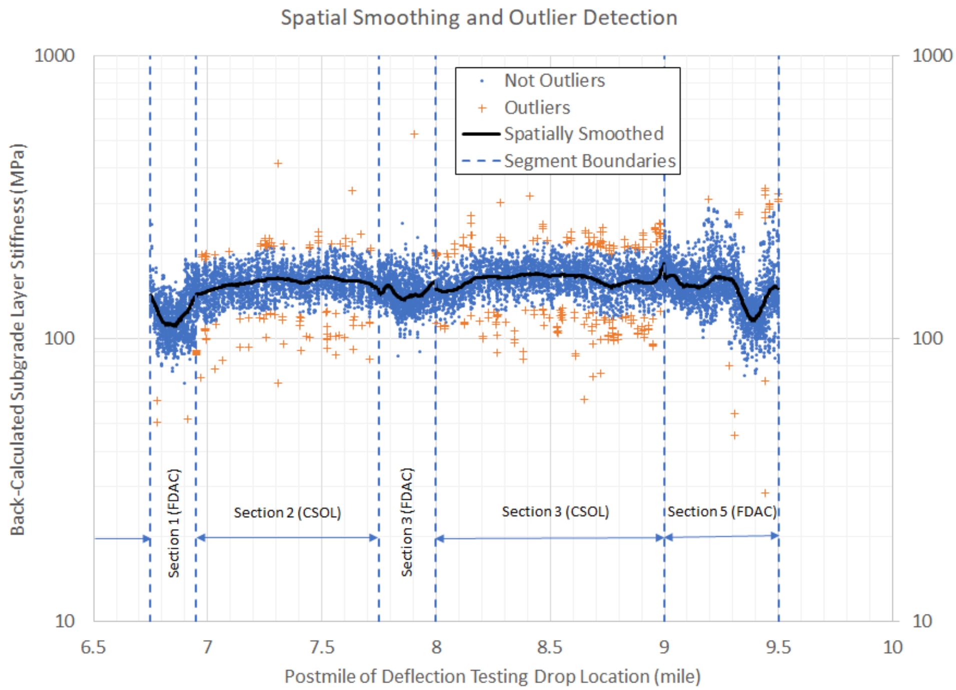


Figure 3.7: Spatial smoothing and outlier detection for backcalculated subgrade layer stiffness.

3.1.4 Backcalculation Results

The comparison of backcalculated layer stiffness is shown for each year of deflection testing from 2003 through 2023 in Figure 3.8 to Figure 3.10 for the asphalt-bound, base, and subgrade layers. The outliers identified through the process described above are excluded from the plots.

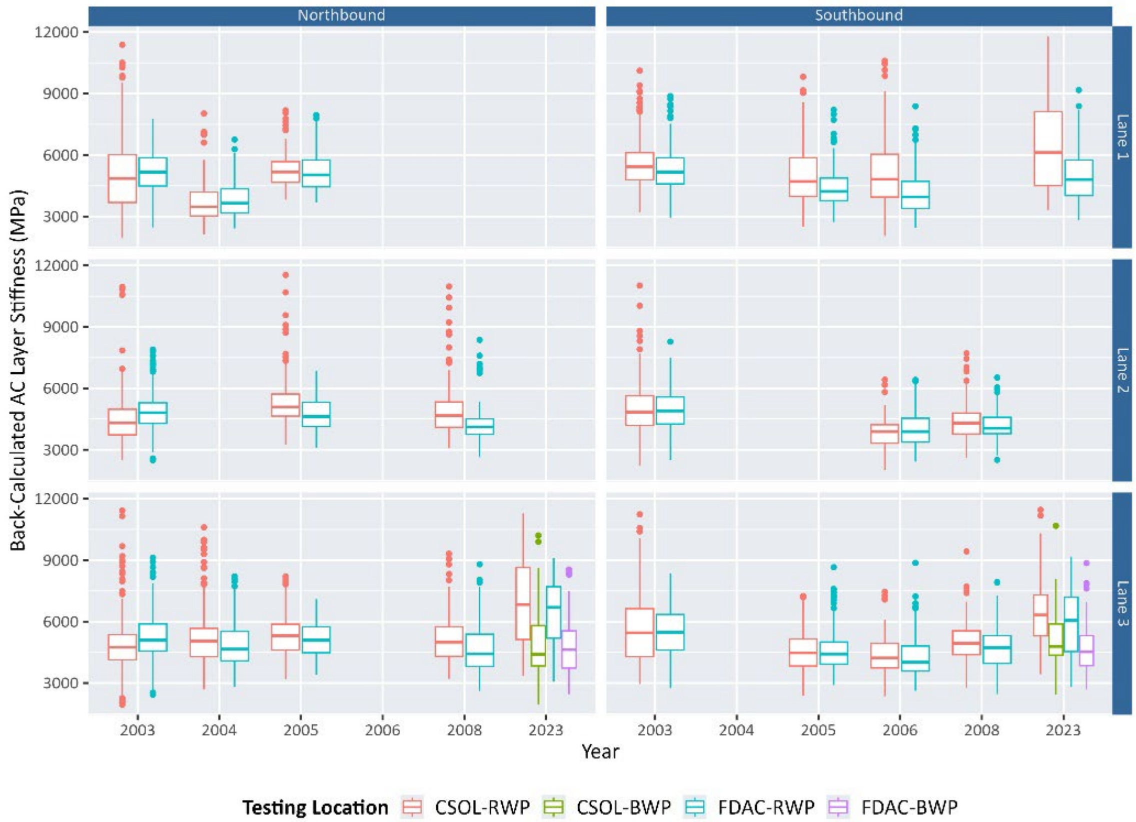


Figure 3.8: Backcalculated asphalt-bound layer stiffness for the LA-710 project.

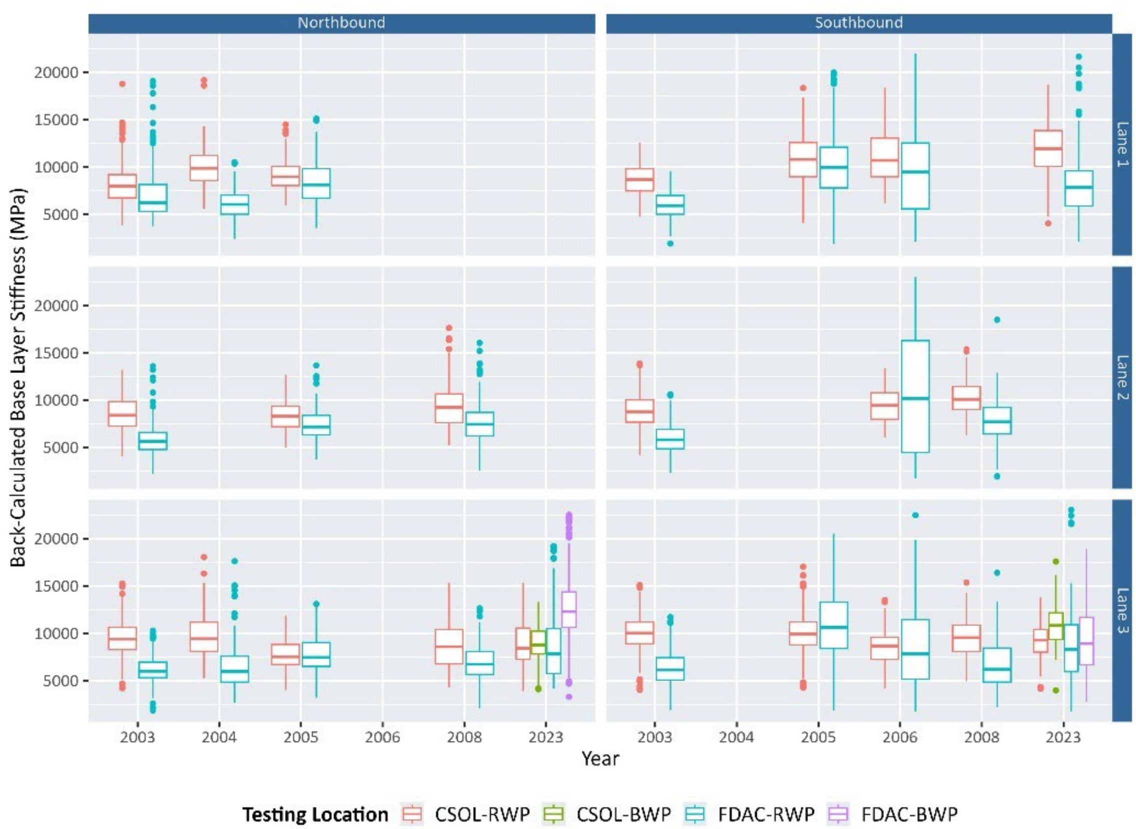


Figure 3.9: Backcalculated base layer stiffness for the LA-710 project.

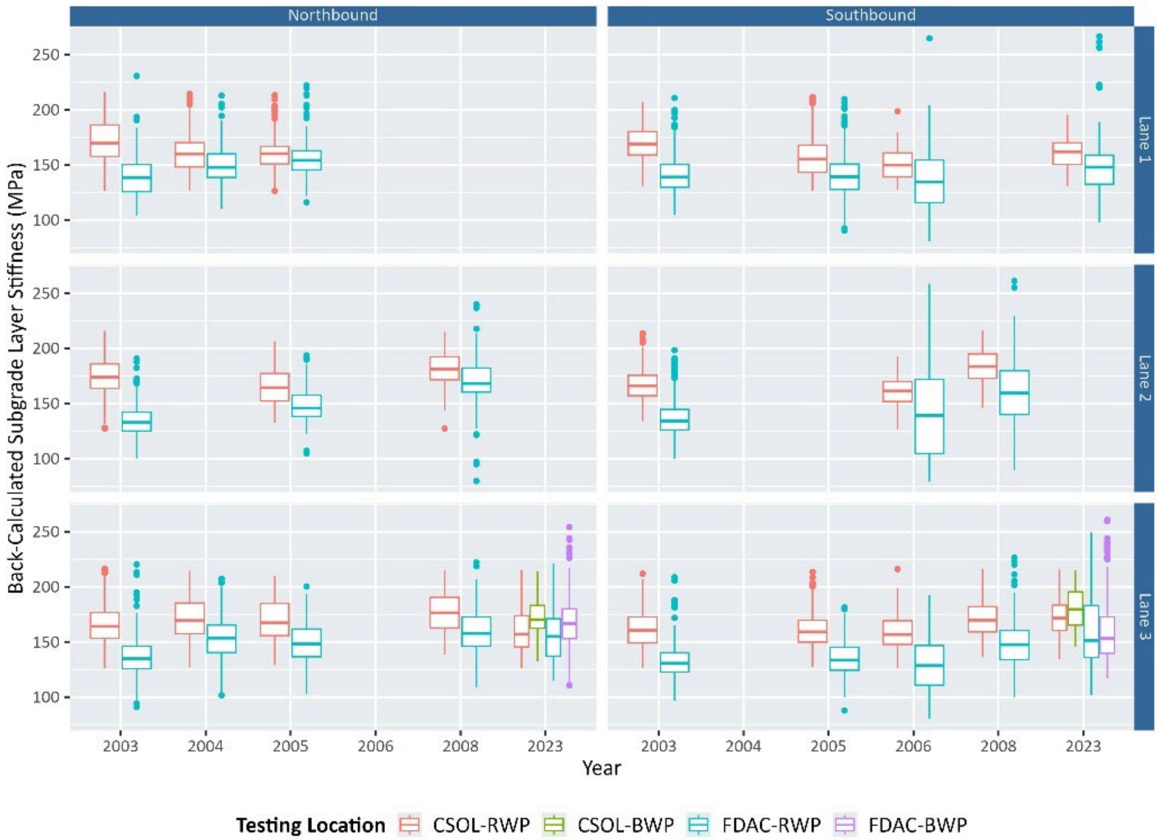


Figure 3.10: Backcalculated subgrade layer stiffness for the LA-710 project.

The following observations were made based on these figures:

- Effects of aging and densification in the asphalt-bound layer: Aging and densification cause the asphalt-bound layer stiffness to increase. The backcalculated asphalt-bound layer stiffness increased from 2003 to 2023. However, this increase occurred only along the right wheelpath. This finding indicates that the effect of aging is minimal. The increase in stiffness is believed to be mostly caused by traffic-induced densification.
- Effect of damage in the asphalt-bound layer: Damage causes layer stiffness to decrease. Although there were some decreases in stiffness in the earlier years (before 2008), the decreases occurred between wheelpaths (BWP) as well (assuming the initial stiffnesses were the same for BWP and the right wheelpath [RWP]). It is believed that there was no traffic-induced damage in the asphalt-bound layer.
- Effect of curing and damage in the recycled concrete aggregate base (RCAB) layer in the FDAC sections: There were stiffness increases in the layer for the earlier years, followed by a decrease and then a recovery. The timing of these changes was not the same throughout the three lanes, indicating the complex interaction between curing and traffic-induced damage. It is clear, however, that the RCAB layer is stiffer than the initial stiffness 20 years after the construction, which was around 5,000 MPa, and now ranges between 5,000 and 10,000 MPa. This is very stiff for base that has not had stabilizer added to it. Most aggregate base stiffnesses

range between about 200 and 600 MPa. The stiffness can be attributed to residual cement in the recycled concrete that was activated by the compaction water added during construction and the confining effect of the stiff and thick asphalt layer.

- Effect of traffic on crack and seated old concrete base layer stiffness in CSOL sections: The stiffness for this layer decreased slightly for the outside lanes but increased for the inside lanes. It is not clear what the mechanism is for these trends, but it is believed that the lighter traffic in the inside lane is beneficial while the heavier traffic in the outside lane is slightly damaging. These layers are also very stiff compared to unstabilized aggregate base. The very thick and stiff concrete and asphalt layers also provide confinement, which increases stiffness of the base layer.
- Subgrade stiffness: The stiffness of this layer showed only slight variations over the years and increased slightly in 2023 compared with the initial values.

3.1.5 Normalized Deflection History

Using the backcalculated layer stiffness, adjusted for temperature (adjusted to 20°C) and spatial variation, the normalized deflections were estimated using multilayer elastic theory. This provides another way of evaluating the overall structural integrity of the pavement. The load is applied on a circular area with a 6 in. (150 mm) diameter and total force of 9 kips (40 kN), simulating a typical FWD drop. The deflection is calculated right under the loading plate, which is where the first geophone is located.

The calculated normalized deflection is shown in Figure 3.11. As expected, the FDAC sections showed higher deflections than the CSOL sections. Compared with the initial values calculated for 2003, the deflections were higher in 2006 but then ended up lower in 2023. This is believed to be mostly due to higher stiffness in the base layer and the subgrade, as well as densification of the asphalt-bound layer. The effect of aging is believed to be minimal.

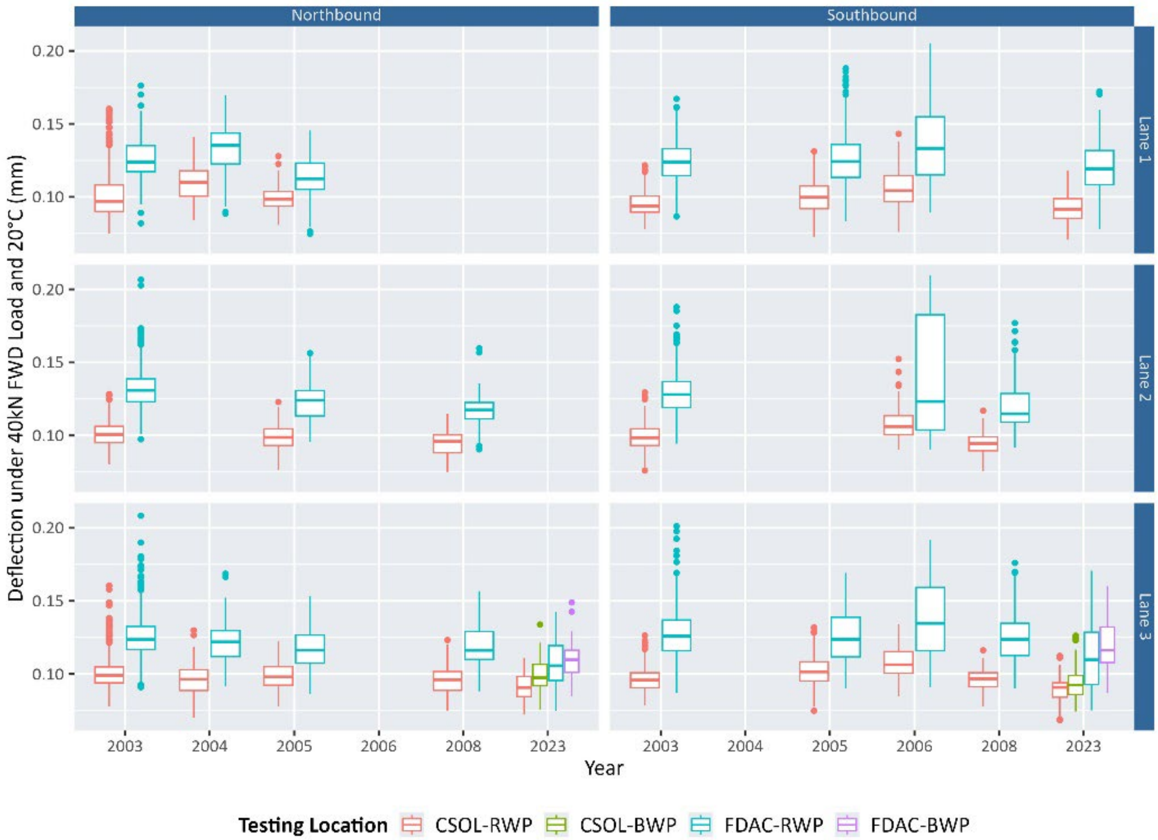
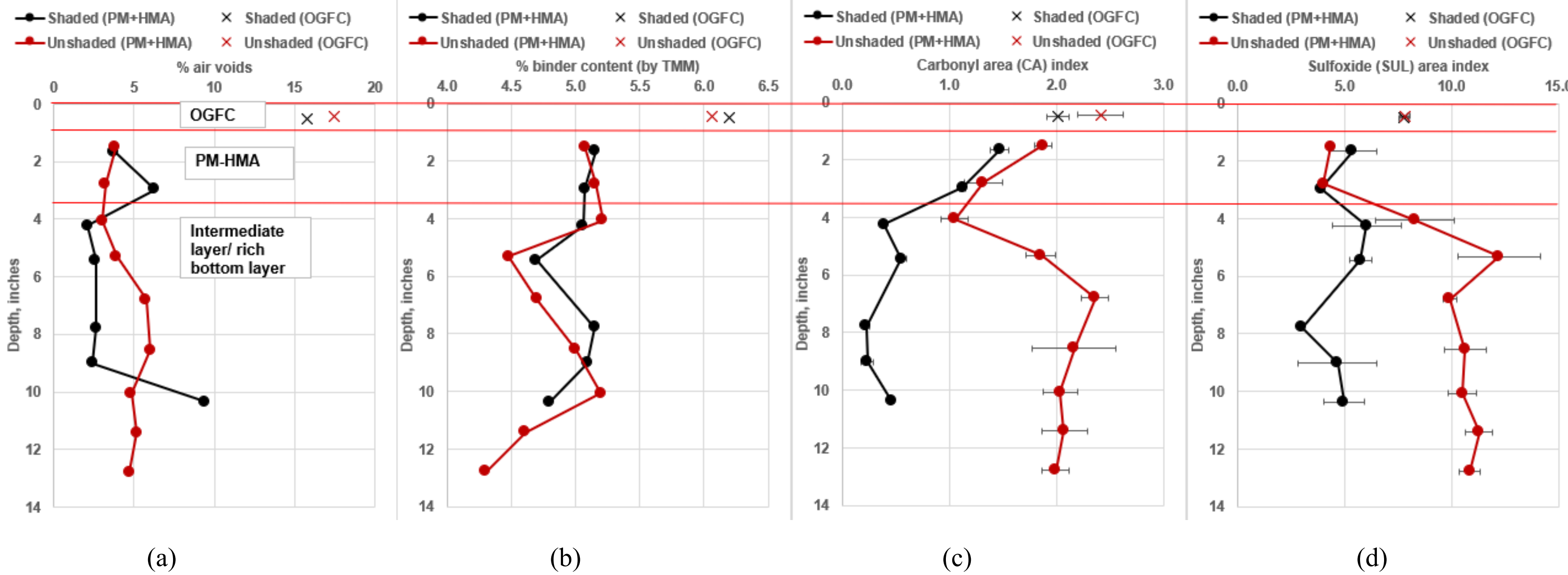


Figure 3.11: Deflection under 40 kN FWD load and 20°C for the LA-710 project.

3.2 Aging Testing

Cores were taken in April 2023 near the north end of the project for the purpose of extracting binder for testing to assess aging at different depths in the asphalt layers. The cores were taken from the shoulder next to both the full-depth asphalt section under the Wardlow Road bridge, which was shaded during the day, and next to the CSOL section, which was not shaded. The structure of the shoulder of the full-depth section matched the traveled way lanes, while the shoulder of the CSOL sections did not have concrete beneath it. Figure 3.12 shows the variation of aging indices, percent air-voids, and percent binder contents with depth.

The results in the figure show that the compaction was generally uniform throughout the asphalt layers, with between 3% and the specified maximum of 6% air-voids, except for poor compaction resulting in about 10% air-voids at the bottom of the core taken from under the bridge. The binder contents are consistent at approximately 5.1% binder in the polymer-modified surface layer and generally consistent between 4.5% and 5.2% in the underlying intermediate layer. The binder content at the bottom of the core taken from next to the CSOL section has a slightly lower binder content.



Note: Error bars indicate one standard deviation of results.

Figure 3.12: Variations of (a) % air-voids, (b) % binder content, (c) carbonyl area (CA) index, and (d) sulfoxide (SUL) index with depth for LA-710 cores collected from shaded (under bridges) and unshaded regions.

The low air-void contents would generally be expected to limit aging below the surface of the pavement due to expected low air permeability in addition to increasingly lower maximum temperatures with increasing depth below the surface. The carbonyl (CA) levels are higher for the unshaded core than for the shaded core, as expected. The shaded core also generally had lower air-voids.

The CA levels followed the expected trend for the shaded core from under the bridge, decreasing CA with increasing depth in the surface layer and the top of the intermediate layer to a depth of 0.33 ft. (4 in., 100 mm) below the pavement surface, and a constant low CA level from that depth to the bottom of the core. For the unshaded core, the CA content decreased with depth through the polymer-modified surface layer to the top of the intermediate layer but then was consistently higher from the top of the intermediate layer to the bottom of the core.

This unexpected result is likely explained by reviewing the air-void content plot shown in Figure 3.12(a). It shows that although the entire unshaded core has relatively low air-voids, less than 6%, the CA contents follow the small decrease in air-voids with depth to the top of the intermediate layer and then an increase from 3% to between 5% and 6% air-voids from the top of the intermediate layer to the bottom of the core. This small increase in air-void content may have allowed smaller amounts of air to penetrate to the bottom of the core. Alternatively, the lower lifts of the intermediate layer may have been subjected to a longer time at a higher temperature in the silo at the plant or during transportation than the top lift.

The sulfoxide (SUL) area index was similar for the shaded and unshaded cores in the surface layer and higher in the unshaded core than the shaded core for the intermediate layer. The SUL showed similar levels at all depths in the shaded core, while the unshaded core had higher levels of SUL in the surface layer than in the intermediate layer.

3.3 Summary

Through multiple rounds of sampling and testing between 2003 (the time of construction) and 2023 (20 years after construction), a large amount of deflection testing information has been gathered that allows analysis of the structural condition of the AC Long Life section of the LA-710 project. The deflection testing suggests that there has been a little aging occurring in the asphalt-bound layer and no damage in these layers. The RCAB layer shows stiffness increases in the early years, likely due to cementation from residual cement in the crushed concrete. The air-void data suggest that all layers were well compacted with air-voids below the specified values.

4 SAMPLING AND TESTING FOR THE TEH-5 RED BLUFF PROJECT

The project on I-5 in Tehama County (TEH-5) near Red Bluff was subjected to sampling and testing in 2015, 2016, 2019, and 2021 to evaluate the changes in as-built material properties and structural integrity over time. The sampling and testing were limited to the segment between PM 37.1 and PM 41.6, where long-life mixes were used. This segment is referred to in this chapter as the AC Long Life segment.

4.1 Scope of Testing

Each round of sampling and testing involved taking slabs of asphalt from the field between the wheelpaths of the southbound truck lane through saw cutting (Appendix A) and deflection testing using the FWD. The slabs were then transported back to the laboratory to produce cut specimens for testing to evaluate the change in stiffness, fatigue resistance, rutting resistance, and aging condition. This chapter covers the analysis of deflection testing and fatigue and stiffness testing data, as they are consistently available. A brief summary of the rutting resistance and aging condition data is presented, and a more detailed analysis of aging will be included in a subsequent report on aging.

4.2 As-Built Structures

Most of the long-life segment has two lanes in each direction, except for the section between PM 40.6 and PM 39.6 in the southbound direction. The typical cross sections for this segment extracted from the project plans are shown in Figure 4.1, with details shown in Figure 4.2.

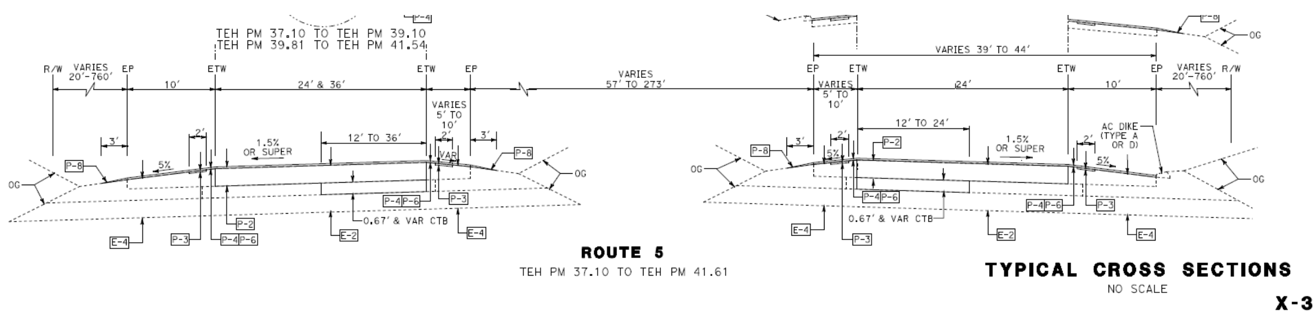


Figure 4.1: Typical cross sections for the segment between PM 37.1 and PM 41.6.

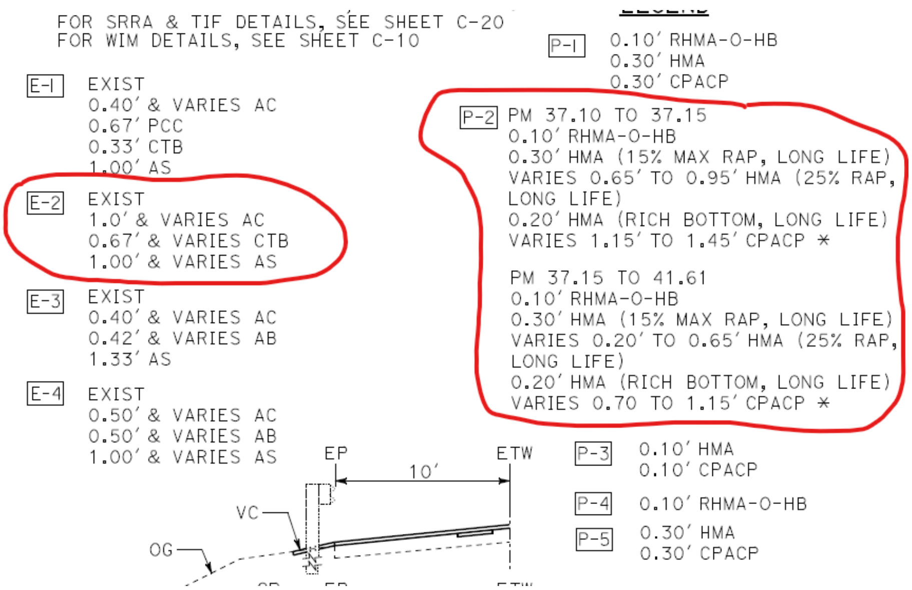


Figure 4.2: Details for the cross sections for the segment between PM 37.1 and PM 41.6.

Based on these figures, the pavement structures in the traveled way before and after the construction for the TEH-5 project were determined, as shown in Table 4.1.

Table 4.1: Pavement Structures Before and After the TEH-5 Project

Layer Number	Before Construction	New Layer Number	After Construction for Inside Lanes	After Construction for Outside Lane
1	1.0 ft. of AC variable thickness	1	0.10 ft. RHMA-O	Same as inside lanes
—	—	2	0.30 ft. HMA-PM	Same as inside lanes
—	—	3	Varied HMA[a]	Same as inside lanes
—	—	4	0.20 ft. HMA Rich Bottom	Same as inside lanes
2	0.67 ft. of CTB variable thickness	5	Replaced by new CTB of same thickness	Unchanged
3	1.0 ft. of AS variable thickness	6	Unchanged	Unchanged
4	Subgrade	7	Unchanged	Unchanged

[a] The intermediate HMA layer thickness varies between 0.65 and 0.95 ft. from PM 37.10 to PM 37.15, and between 0.20 and 0.65 ft. from PM 37.15 to PM 41.61.

Note that the intermediate layer thickness in the new construction ranges between 0.20 and 0.95 ft. depending on the location.

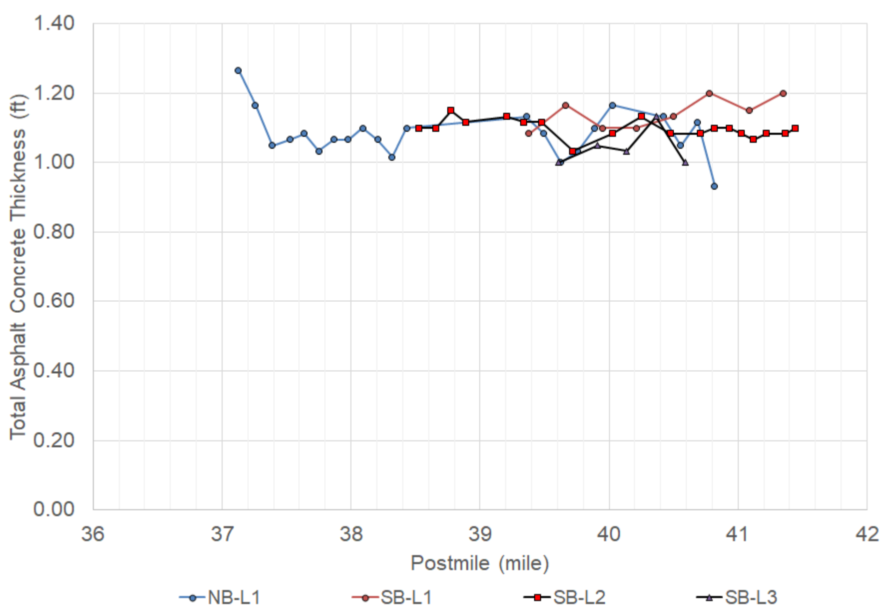
4.2.1 Asphalt Concrete Layer Thicknesses

As-built asphalt concrete layer thicknesses were determined from slabs (see Appendix A) and cores taken from the field in 2016 and 2021. The results are listed in Table 4.2.

Table 4.2: As-Built Asphalt Concrete Layer Thicknesses for the TEH-5 Project

Sample and Location	Layer Number	Layer	Lift 1 (ft.) [mm]	Lift 2 (ft.) [mm]	Lift 3 (ft.) [mm]	Total (ft.) [mm]
PM 40.3, SB Lane 3 (slabs sampled in 2016)	1	RHMA-O	0.13 [38]	N/A	N/A	0.13 [38]
	2	HMA-PM	0.25 [75]	N/A	N/A	0.25 [75]
	3	HMA	0.25 [75]	0.25 [75]	N/A	0.50 [150]
	4	HMA rich bottom	0.21 [63]	N/A	N/A	0.21 [63]
	Combined	All new AC	N/A	N/A	N/A	1.09 [326]
PM 39.5, NB Lane 2 (cores sampled in 2021)	1	RHMA-O	0.12 [35]	N/A	N/A	0.12 [35]
	2	HMA-PM	0.16 [48]	0.13 [40]	N/A	0.29 [88]
	3	HMA	0.22 [65]	0.13 [40]	0.17 [50]	0.32 [155]
	4	HMA rich bottom	0.20 [59]	N/A	N/A	0.20 [59]
	Combined	All new AC	N/A	N/A	N/A	1.12 [337]

In addition, a batch of cores were taken in October 2012 by Caltrans soon after construction. The thicknesses of those cores are shown in Figure 4.3 for different directions and lanes. The project plans are consistent with the as-built thicknesses determined from field slabs and cores. Overall, the total thicknesses of the new asphalt concrete layers are mostly between 1.0 and 1.2 ft. (300 and 350 mm), with an overall average of 1.1 ft. (330 mm) and a standard deviation of 0.06 ft. (18 mm).



Note: Legends indicate direction and lane number.

Figure 4.3: Total asphalt concrete thickness determined from cores after construction in 2012.

4.2.2 Cement-Treated Base Layer Thickness

According to the materials report written by Caltrans for the 1976 repair of I-5 in Tehama County between PM 28.7 and PM 42.0, the cement-treated base (CTB) layer was either 0.5 ft. (150 mm) or 0.67 ft. (200 mm) for the AC Long Life section (Figure 4.4) *(19)*. The figure indicates that the CTB layer thickness is mostly 0.67 ft. (200 mm) for the northbound (NB) direction and around 0.55 ft. for the southbound (SB) direction. This is consistent with the plans for the AC Long Life project (Figure 4.1).

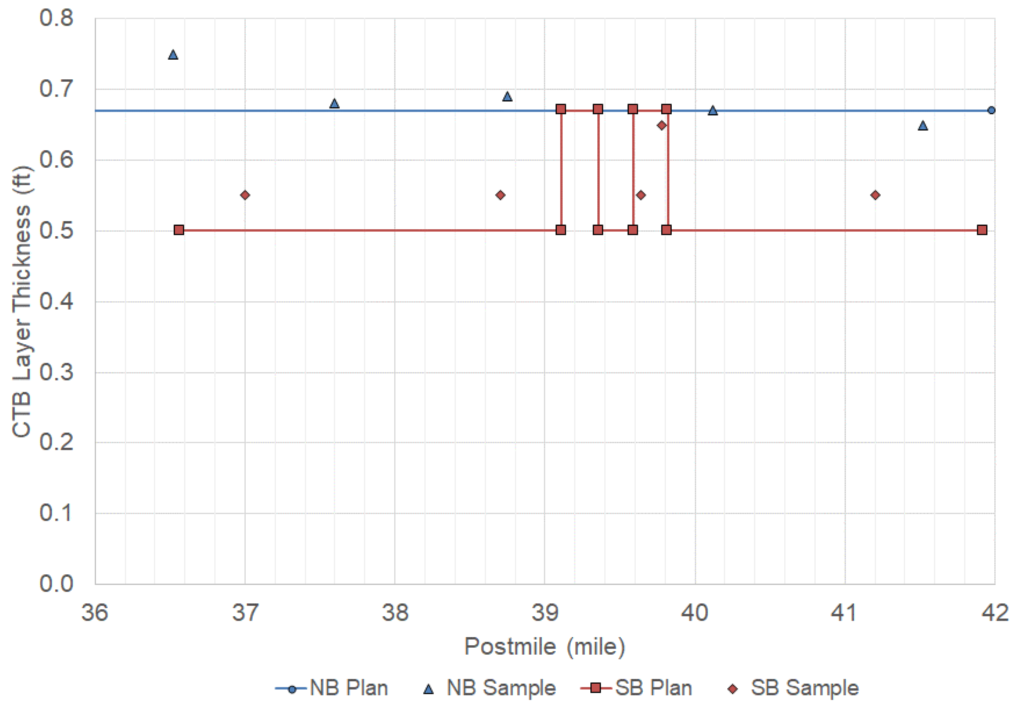


Figure 4.4: Cement-treated base (CTB) layer thickness from the materials report for the rehabilitation in 1976.

4.2.3 Aggregate Subbase Layer Thickness

According to the materials report written by Caltrans for the repair of I-5 in Tehama County in 1976 between PM 28.7 and PM 42.0, the aggregate subbase (AS) layer was typically 1.25 ft. but varies *(19)*. In the same report, the depths of the top and bottom of the AS layer were also recorded. These depths were used to determine the AS layer thicknesses, and the results are shown in Figure 4.5. The figure indicates that the AS layer thickness is around 1.25 ft. for the northbound (NB) direction and 1.1 ft for the southbound (SB) direction.

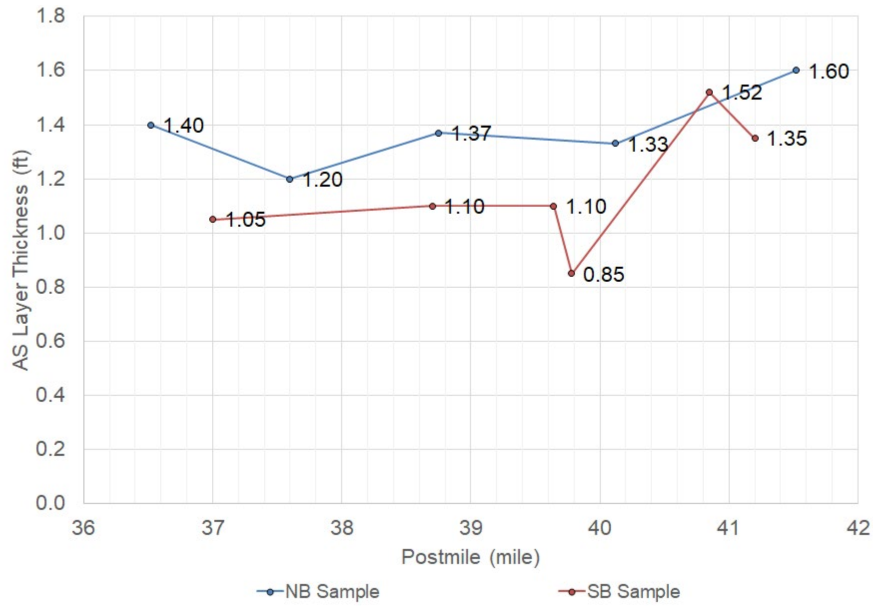


Figure 4.5: Aggregate subbase layer thickness from the materials report for the rehabilitation in 1976.

4.2.4 Aggregate Subbase and Subgrade R-Values

The materials report written by Caltrans for the 1976 repair of I-5 in Tehama County between PM 28.7 and PM 42.0 also includes R-values determined from soil samples taken in 1976 under the paved layers *(19)*. The results are shown in Figure 4.6. The AS layer has an average R-value of 77 and a standard deviation of 12.2. The subgrade layer has an average R-value of 30.4 and a standard deviation of 15.3. The 1976 materials report also indicates that the subgrade layer is composed of clayey soil and rock.

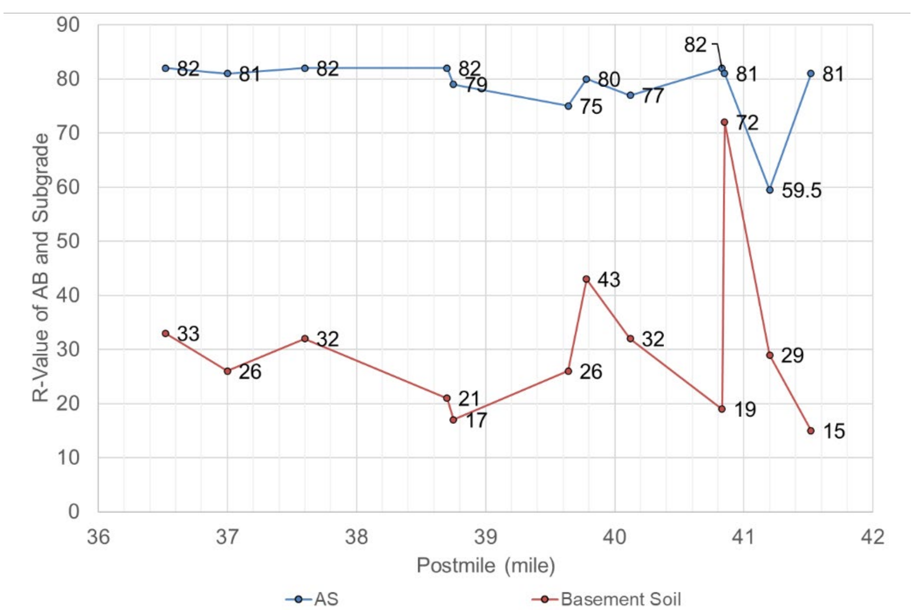


Figure 4.6: R-values of the aggregate base and subgrade from the materials report for the rehabilitation in 1975.

4.3 Deflection Testing and Stiffness Backcalculation

Deflection testing was conducted on the TEH-5 Red Bluff Long Life segment in 2015, 2016, 2019, and 2021 between August and October in each year (see Table 4.3). Note that each segment was tested twice each year, in the afternoon and in the early morning, to capture the effect of temperature on asphalt-bound layer stiffness. The hot/warm session typically started around 7 p.m. and lasted until 12:00 a.m., and the cold session started around 1:00 a.m. and lasted until 5:00 a.m. The following are two exceptions:

- All the tests in 2015 were hot/warm sessions, and the tests were repeated with shorter interval.
- The cold sessions for the southbound Lane 3 at PM 41.0 (SBL3PM41.0) were repeated in 2021, resulting in three sessions at this location overall.

Some variations in terms of testing locations and intervals exist between different test sessions. The approach described in Section 3.1.3 is used to align the drop locations. The amount of adjustment needed was not more than 0.15 mi. (0.24 km).

Table 4.3: Number of FWD Testing Sessions on TEH-5 Red Bluff Long Life Segments

Segment Tested[a]	2015	2016	2019	2021
NBL1PM37.1-RWP	2	—	—	—
NBL2PM39.0-BWP	—	2	2	2
NBL2PM39.0-RWP	—	2	2	2
SBL2PM38.0-BWP	—	2	2	2
SBL2PM38.0-RWP	—	2	2	2
SBL2PM40.0-RWP	2	—	—	—
SBL3PM40.5-BWP	—	2	2	—
SBL3PM40.5-RWP	2	2	2	—
SBL3PM41.0-BWP	—	—	—	3
SBL3PM41.0-RWP	—	—	—	3

[a] The segment label indicates direction, lane, starting post mile, and lateral position of the FWD testing (RWP = right wheelpath, BWP = between wheelpaths); each segment is approximately one mile long.

4.3.1 Structures Used for Backcalculation

After reviewing the as-built structures (see Section 4.2), the structures used for backcalculation were determined and are shown in Table 4.4. The table shows that the CTB and AS layer thicknesses are different between the two directions.

Table 4.4: Layer Thicknesses Used for Stiffness Backcalculation for the TEH-5 Project

Layer Number	Description	Thickness for SB Lanes (ft) [mm][a]	Thickness for NB Lanes (ft) [mm][a]
1	Combined asphalt-bound layers	1.08 (325)	1.08 (325)
2	CTB layer	0.55 (165)	0.67 (200)
3	Aggregate subbase	1.10 (330)	1.33 (400)
4	Subgrade	Semi-infinite	Semi-infinite

[a] A rounded conversion is used between in. and mm: 1 ft. = 300 mm.

4.3.2 Accounting for Temperature Effects on Layer Stiffness

The FWD testing was conducted at different pavement temperatures. Since asphalt concrete stiffness is sensitive to temperature, it is important to account for the effects of temperature on backcalculated stiffnesses before those stiffnesses can be used to identify trends between years, lanes, and lateral positions.

Pavement temperatures were calculated using the BELLS3 equation *(20)* for the one-third depth (0.27 ft. or 108 mm) of the combined asphalt-bound layer based on measured surface temperature and average air temperature of the day before testing. The air temperature was retrieved from the National Oceanic and Atmospheric Administration (NOAA) Climate Data Online through web service calls using the Global Historical Climatology Network daily (GHCNd) dataset for the Red Bluff Municipal Airport (station number GHCND:USW00024216).

For the tests conducted in 2019, only the air temperatures were valid. Fortunately, both air and surface temperatures were valid for the tests conducted in 2021. The surface temperature was found to be well correlated to air temperature (Figure 4.7). The surface temperatures for the 2019 tests were estimated from air temperatures using these correlations. The small differences between the directions and lanes are likely due to small differences in surface albedo (surface reflectivity).

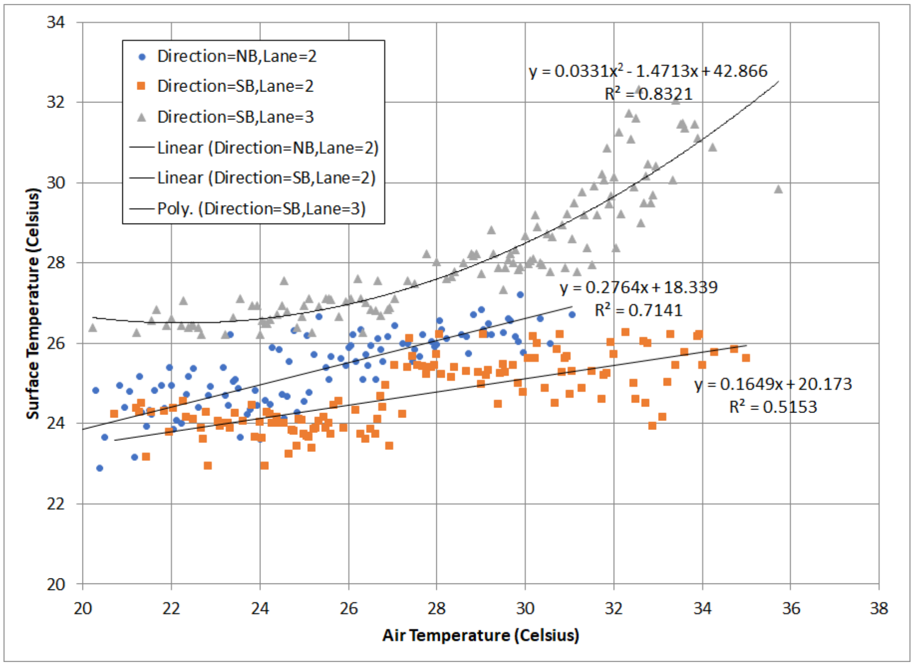


Figure 4.7: Correlation between surface and air temperatures for the 2021 tests.

The correlation between BELLS temperatures and backcalculated combined asphalt-bound layer stiffness is shown in Figure 4.8. Although the FWD testing was conducted under a wide range of BELLS temperatures, the data are still not enough to fully specify all the model parameters needed for a regular asphalt concrete stiffness master curve *(21)*. As shown in the figure, a simple quadratic equation is used to describe the correlation between backcalculated asphalt-bound layer stiffness and the BELLS temperature at one-third depth of the asphalt-bound layer.

The correlation between BELLS temperature and backcalculated CTB layer stiffness is shown in Figure 4.9. Note that the BELLS temperature calculated for the asphalt-bound layer is used here because it provides better correlation than the values calculated for the CTB layer. The stiffness of the CTB layer is not directly sensitive to temperature. The correlation seen with temperature is likely due to the confining effect of the asphalt layer and the likelihood of shrinkage cracks in the CTB. It is assumed that the shrinkage-cracked CTB is stiffer when the overlying asphalt is stiffer and provides more confinement. However, there is no other evidence to support this assumption. There was no strong correlation between the BELLS temperature and backcalculated AS and subgrade layer stiffness.

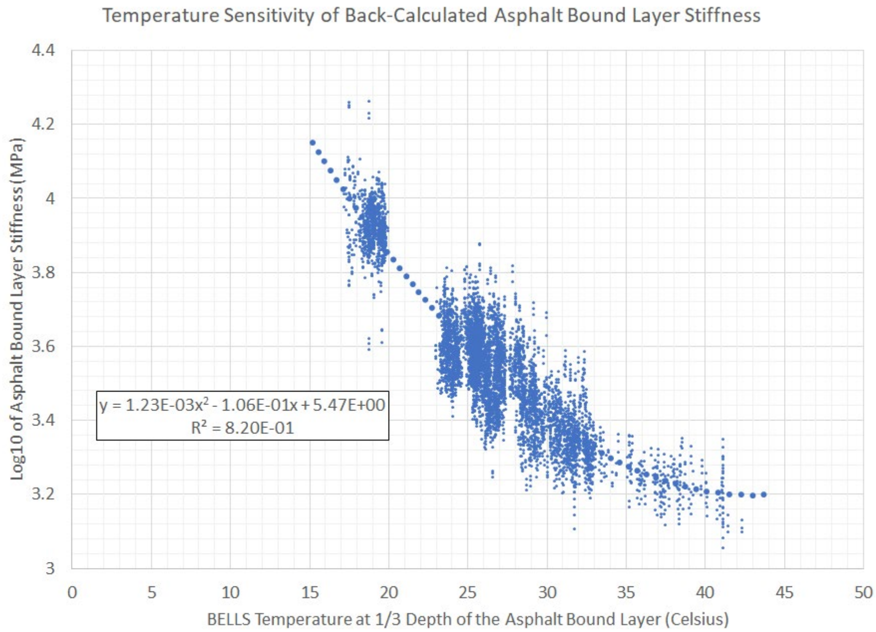


Figure 4.8: Correlation between BELLS temperatures and log10 of backcalculated asphalt-bound layer stiffness.

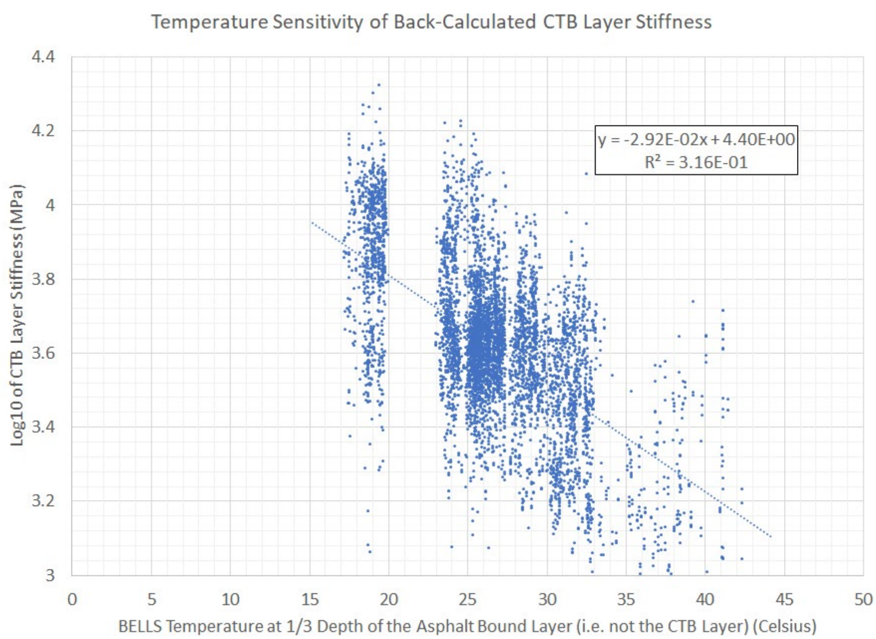


Figure 4.9: Correlation between BELLS temperatures and log10 of backcalculated cement-treated base (CTB) layer stiffness.

4.3.3 Accounting for Spatial Variability and Detecting Outliers

The procedure described in Section 1.1.1 was used to account for spatial variability for the TEH-5 project as well. The spatial smoothing is shown in Figure 4.10 to Figure 4.13 for each of the layers. Also shown in the figures is the detection of outliers. Outliers were defined as backcalculated stiffnesses that are outside of the 95% confidence interval estimated based on the pooled results. The outliers were excluded from further analysis.

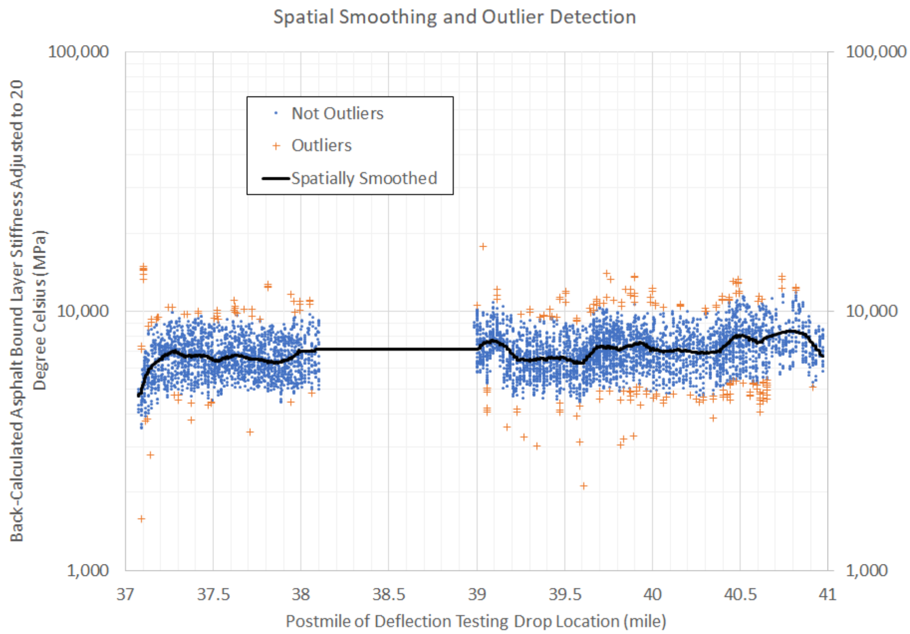


Figure 4.10: Spatial smoothing and outlier detection for backcalculated asphalt-bound layer stiffness for the TEH-5 project.

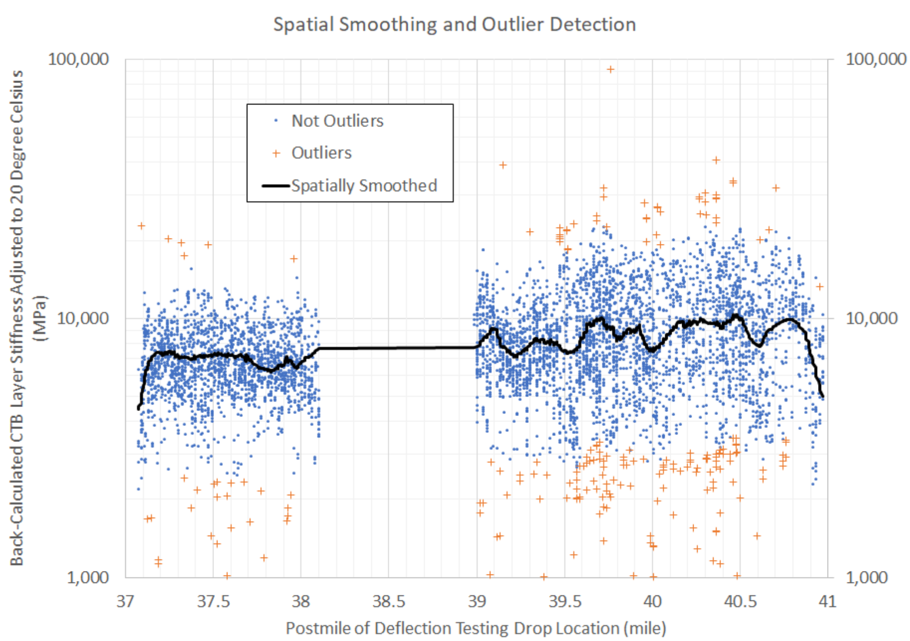


Figure 4.11: Spatial smoothing and outlier detection for backcalculated cement-treated base (CTB) layer stiffness for the TEH-5 project.

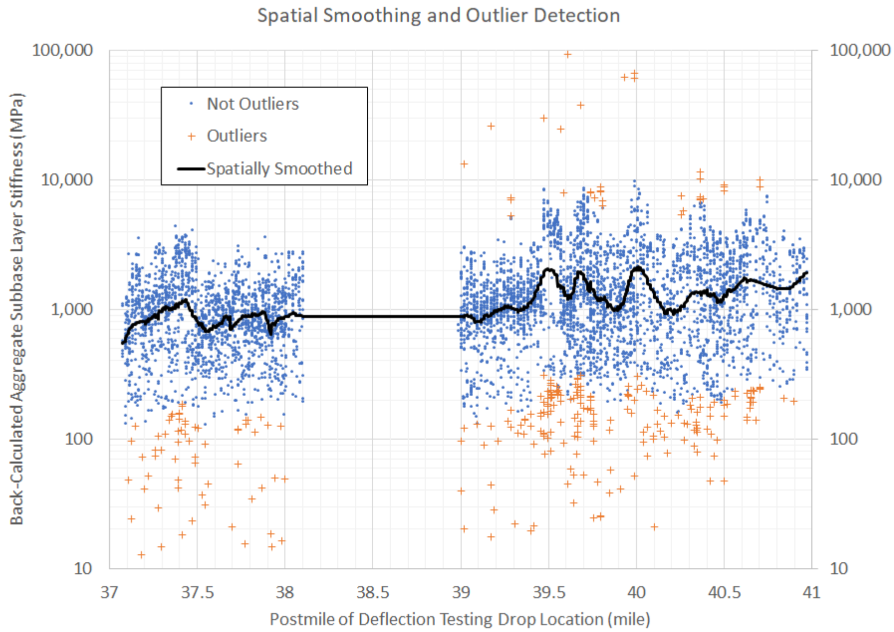


Figure 4.12: Spatial smoothing and outlier detection for backcalculated aggregate subbase layer stiffness for the TEH-5 project.

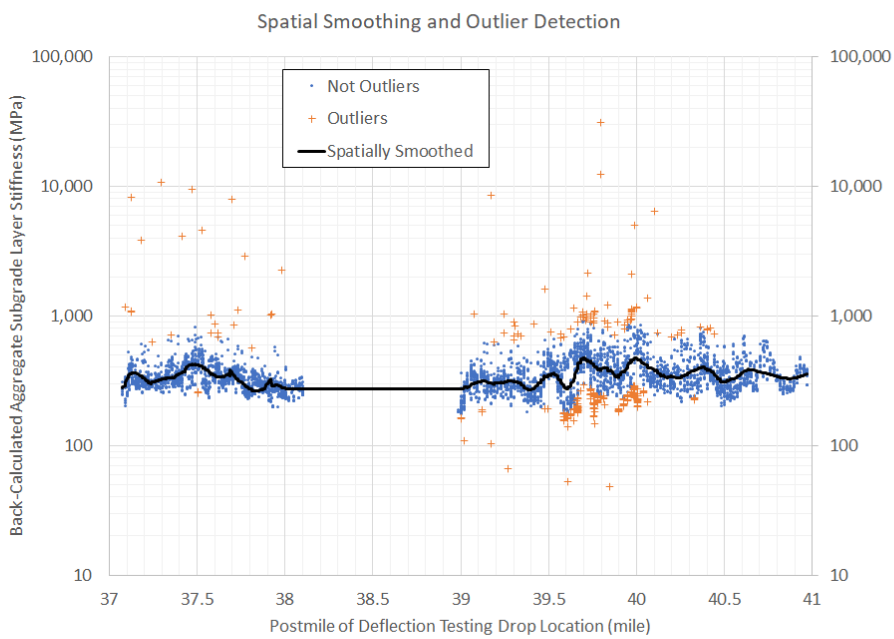


Figure 4.13: Spatial smoothing and outlier detection for backcalculated subgrade layer stiffness for the TEH-5 project.

4.3.4 Backcalculation Results

The comparison of backcalculated layer stiffnesses is shown in Figure 4.14 to Figure 4.17 for aggregate base (AB), cement-treated base (CTB), aggregate subbase (AS), and subgrade layers. The outliers identified through the process described previously were excluded from the plots.

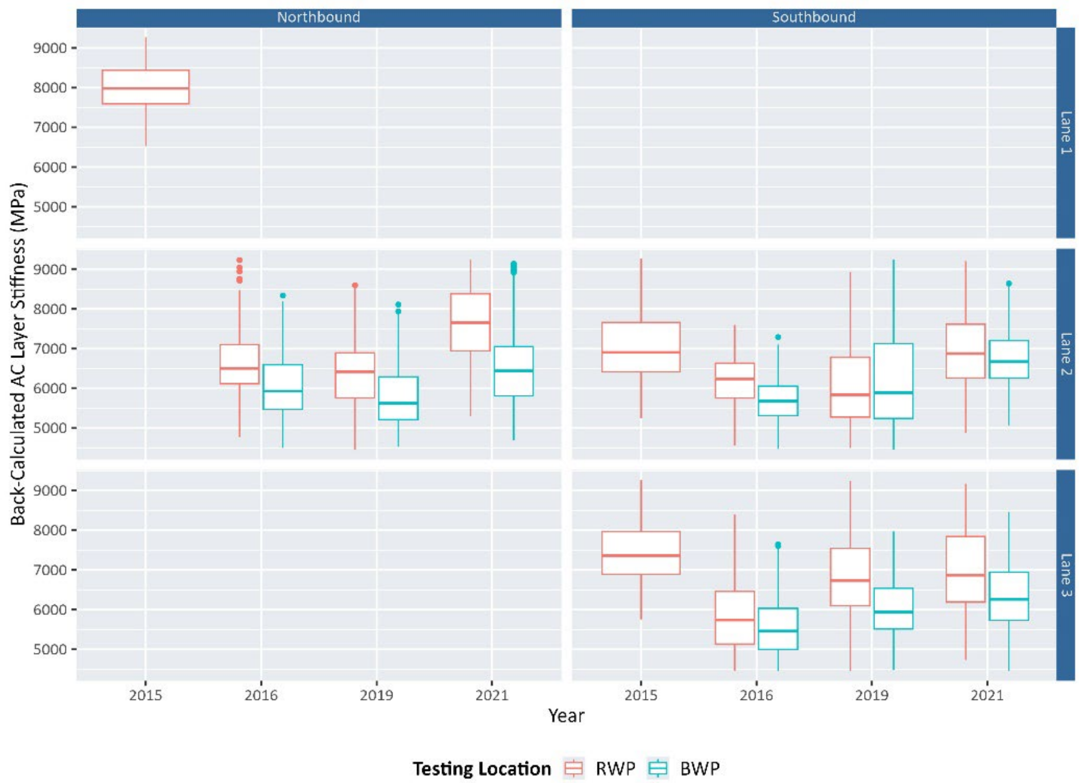


Figure 4.14: Backcalculated asphalt-bound layer stiffness for the TEH-5 project.

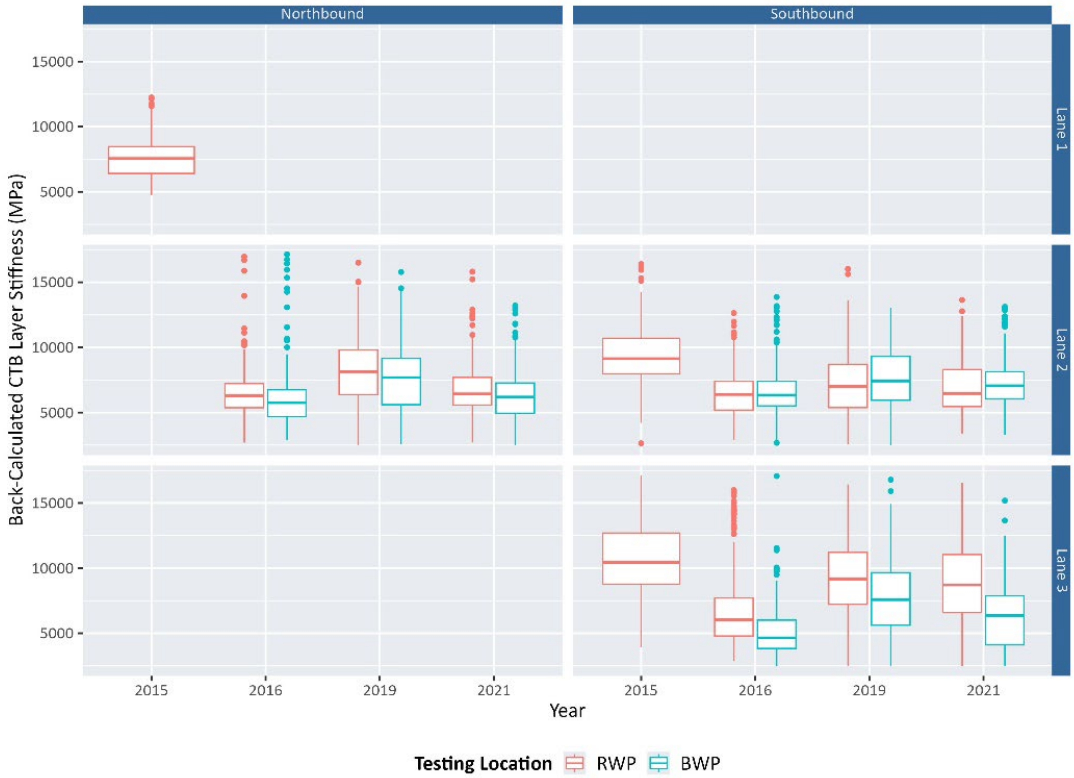


Figure 4.15: Backcalculated cement-treated base (CTB) layer stiffness for the TEH-5 project.

Figure 4.16: Backcalculated aggregate subbase layer stiffness for the TEH-5 project.

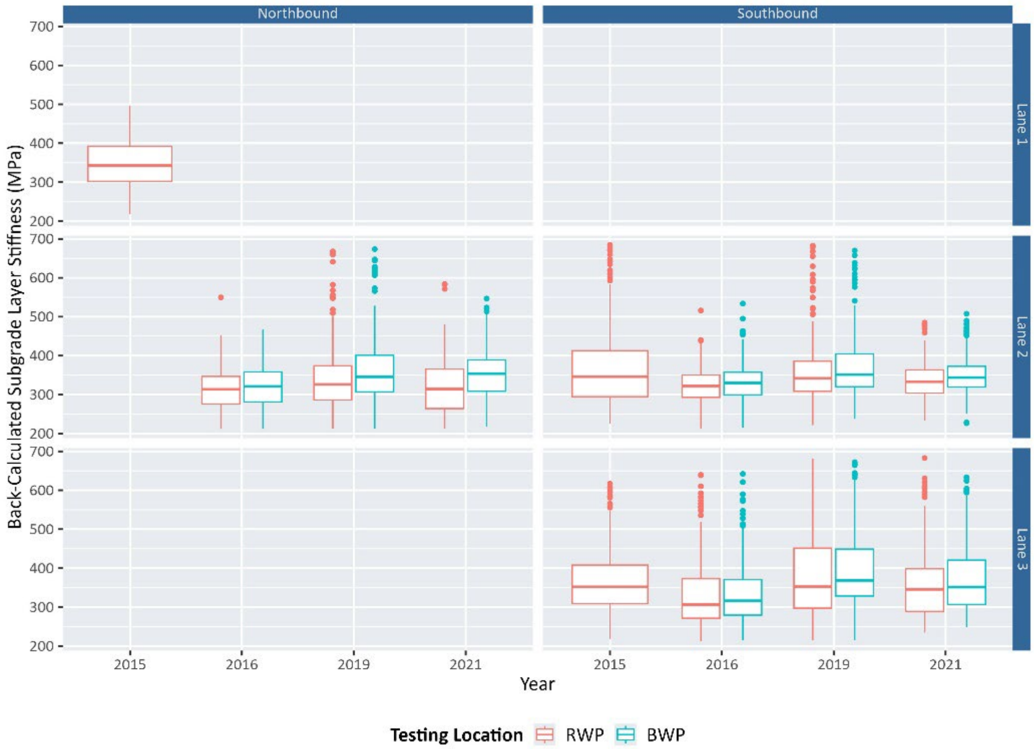


Figure 4.17: Backcalculated subgrade layer stiffness for the TEH-5 project.

The following observations were made based on these figures:

- Effect of aging in the asphalt-bound layer: The backcalculated asphalt-bound layer stiffness increased from 2016 to 2021. This increase occurred in both the BWP and along the RWP. This indicates the effect of aging, although the stiffness increase is not large. The stiffness for BWP is consistently lower than the value for RWP, likely due to densification that occurred soon after construction (between 2015 and 2016).
- Effect of damage in the asphalt-bound layer: Although there were some decreases in stiffness in the earlier years (before 2016), the decreases occurred between the BWP as well (assuming the initial stiffnesses were the same for the BWP and RWP). It is believed that there is no traffic-induced damage in the asphalt-bound layer.
- The decrease in stiffness from 2015 to 2016: The stiffnesses for the asphalt-bound layer, the CTB layer, and the subgrade layer are higher in 2015 compared to 2016. This cannot be explained by the testing temperature because the BELLS3 temperatures for 2015 were close to the median rather than being the extremes among all the years. It is therefore not clear what caused these drops in stiffness from 2015 to 2016.
- Effect of curing and damage in the CTB layer: There were stiffness increases in the layer in the earlier years (from 2016 to 2019), followed by a decrease in 2021. This suggests that there was curing in the earlier years, which was then outpaced by damage caused by traffic.

- Aggregate subbase (AS) stiffness: The AS layer stiffness seems to be unreasonably high, around 145 ksi (1,000 MPa). One possible explanation is that the stiff CTB and asphalt layers above provide strong confinement and reduce the stress induced in the subbase layer, which tends to increase stiffness of the granular layer.
- Subgrade stiffness: The stiffnesses of this layer showed only slight variations over the years. The overall subgrade stiffness seems unreasonably high, around 51 ksi (350 MPa), considering its average R-value of 30.4 (see Section 4.2.4) and the fact that the subgrade moisture contents may at times be slightly wet of the optimum, as was indicated in the 1976 materials report *(19)*, though the driest time would be when the FWD tests were done in the late summer and fall. It is believed that the high subgrade stiffness is also attributable to the high confinement provided by all the layers above, especially the CTB layer.

4.3.5 Normalized Deflection History

The calculated normalized deflection is shown in Figure 4.18. The procedure for calculating the deflections is explained in Section 3.1.5. These normalized deflections are around 2.4 mil. (0.06 mm), which is relatively low and indicates strong pavement structure.

Compared to the initial values calculated for 2015, the deflections first increased, then decreased. This indicates that the pavement was getting weaker in the first three to four years but then started to become stronger again. As explained previously, it is not clear what caused the significant increase in deflection in the first year of service, but the curing of the CTB layer, aging, and densification of the asphalt-bound layer are the main reasons for the lower deflections in the later years.

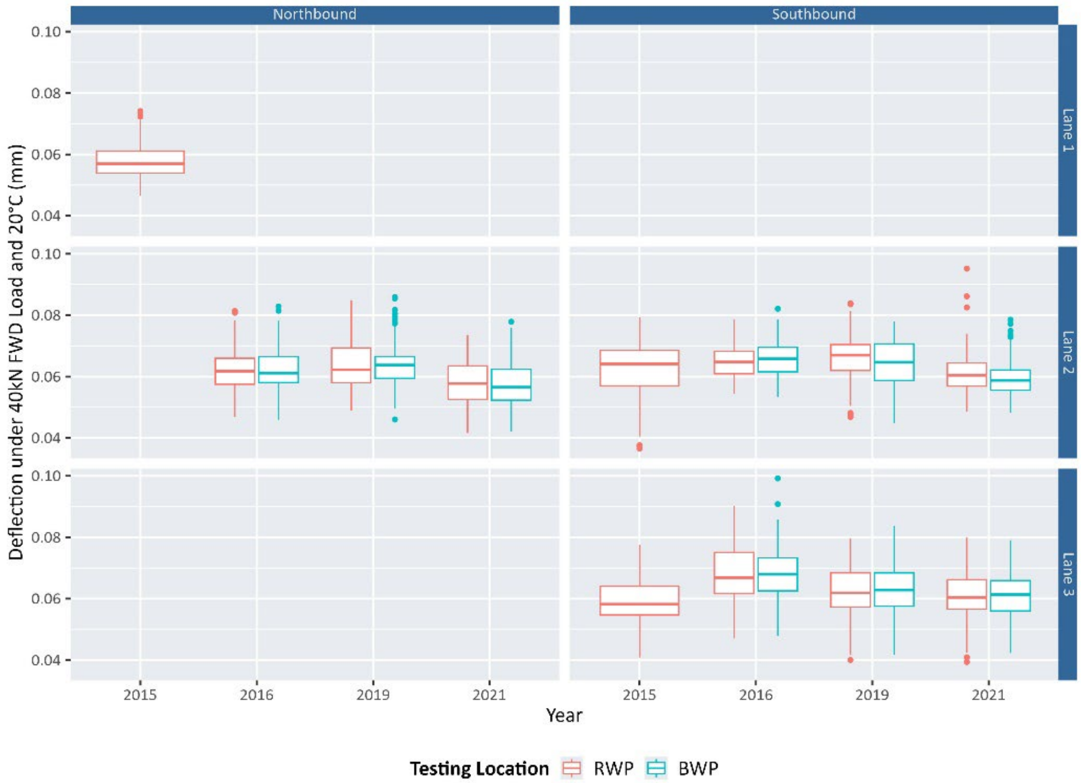


Figure 4.18: Deflection under 40 kN FWD load and 20°C for the TEH-5 project.

4.4 Laboratory Test Results

Multiple rounds of testing were conducted on the three long-life mixes used in the TEH-5 project. The list of sampling and laboratory testing is shown in Table 4.4. Field slabs were consistently taken from southbound Lane 3 at PM 40.3 between the two wheelpaths over the years to produce beams to evaluate the change in stiffness and fatigue resistance. The saw cutting plan used for each year is shown in Appendix A. The testing for mix design, cross-section determination, and specification development are summarized in an earlier UCPRC technical memorandum *(22)*. This report focuses on the testing for job mix formula (JMF) approval and post-construction monitoring.

Table 4.5: List of Sampling and Testing for the TEH-5 Red Bluff Project

Sample Date	Preparation Method[a]	Purpose
2011	LMLC	Mix design and develop inputs for cross-section design and specification development
7/2012	FMLC	Job mix formula approval
10/2012[b]	FMFC	Post-construction monitoring for Year 0
8/11/2014	FMFC	Post-construction monitoring for Year 2 (FWD testing was done one year later in 2015)
8/23/2016	FMFC	Post-construction monitoring for Year 4
8/21/2019	FMFC	Post-construction monitoring for Year 7
8/24/2021	FMFC	Post-construction monitoring for Year 9

[a] LMLC = lab mix lab compact, FMLC = field mix lab compact, FMFC = field mix field compact.
[b] Samples at year 0 were taken before the open-graded friction course was constructed.

Note that for the intermediate layer, the bottom lift was consistently tested and is therefore used to evaluate the change in fatigue performance and stiffness over time. In addition, all three lifts in the intermediate layer were tested for fatigue and stiffness using beams following AASHTO T321 for Year 7 and Year 9 samples. This allows for the evaluation of the variation in fatigue performance and stiffness along the depth for these two years.

4.4.1 Fatigue Test Results

Change in Fatigue Performance Over Time

The comparison of Wöhler curves (fatigue life versus tensile strain) for different mixes sampled and tested over the years are shown in Figure 4.19 to Figure 4.21 for the TEH-5 project surface, intermediate, and rich bottom layers. As shown in these figures, all mixes tested for JMF approval have higher fatigue life than the specification, which is expected. Compared to the corresponding mix tested for JMF approval, the figures also show that the surface mix is the only one that experienced significant decreases in fatigue life. The decreases occurred for the mixes sampled in Year 7 and Year 9. These decreases can be explained by aging in the surface layer. The relatively consistent fatigue lives in other mixes can be explained by the minimal aging and damage in the bottom lift of the intermediate layer and the rich bottom layer. The Wöhler curves also show similar slopes for samples taken throughout the years within each mix type, except for the surface mix, which shows large variations in the slopes.

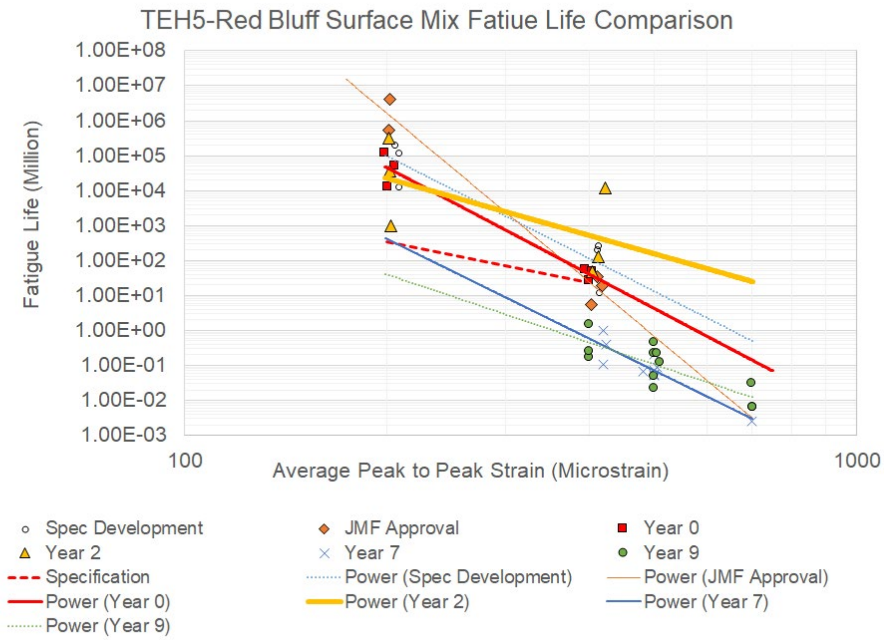


Figure 4.19: Wöhler curves for TEH-5 project surface mixes.

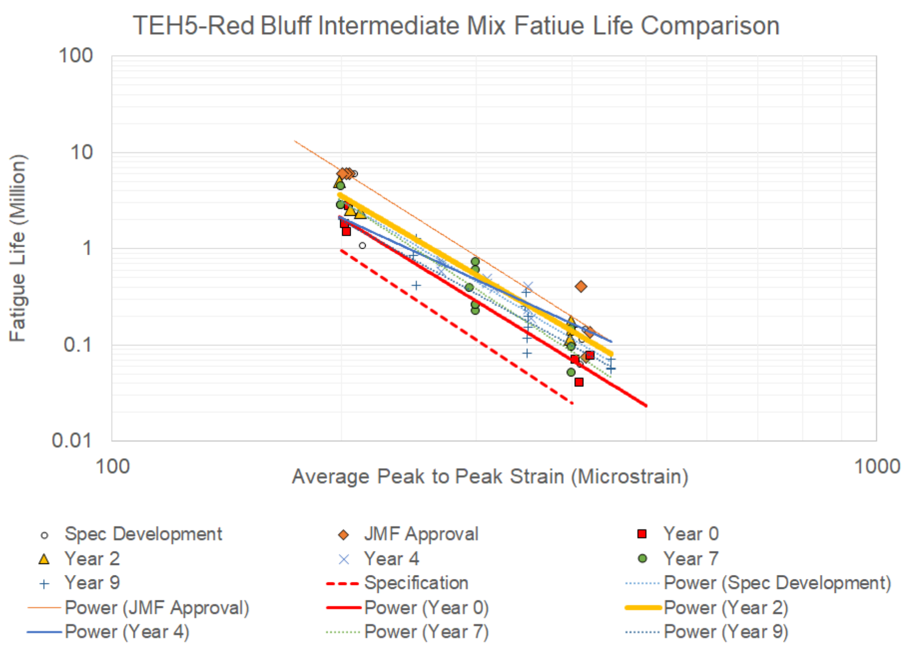


Figure 4.20: Wöhler curves for TEH-5 project intermediate mixes.

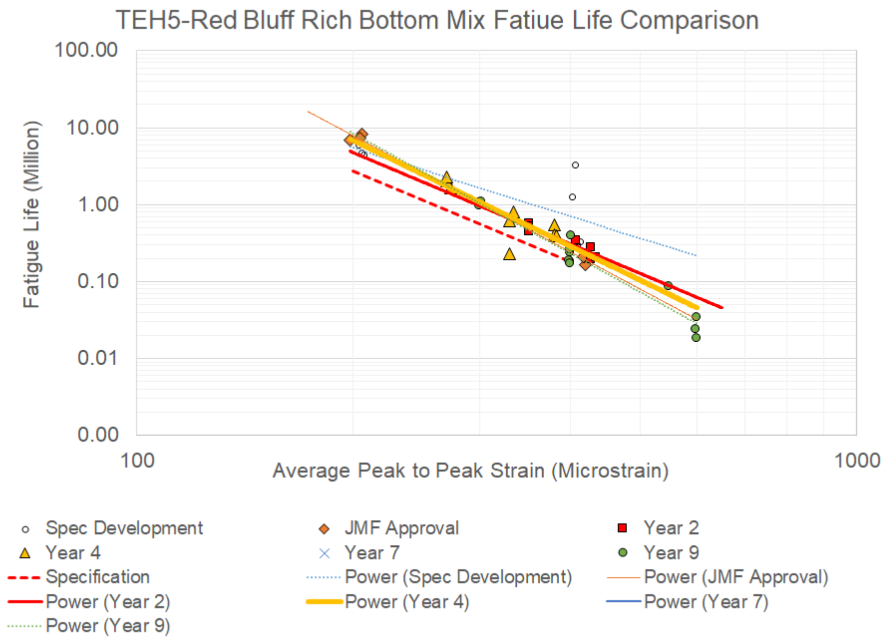


Figure 4.21: Wöhler curves for TEH-5 project rich bottom mixes.

The changes in average fatigue lives and average air-void contents are shown in Figure 4.22 to Figure 4.24 for the surface, intermediate, and rich bottom mixes. The figures confirm the observations made from the Wöhler curves. They also suggest that all the layers were well compacted because the measured air-voids are below the specified maximum (6% for the surface and intermediate layer and 3% for the rich bottom). All the results for the intermediate and rich bottom layers indicate that they have no damage.

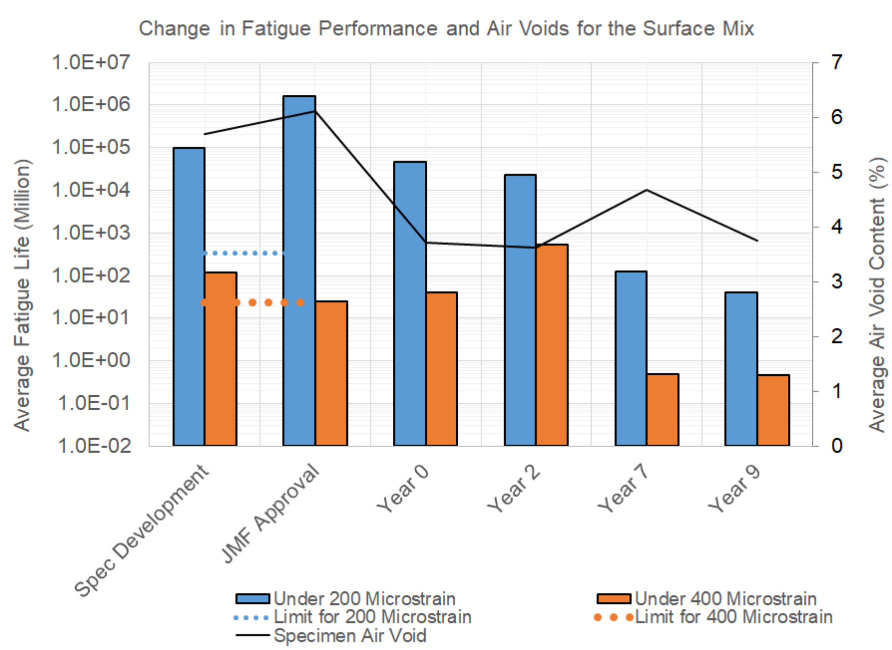


Figure 4.22: Change in fatigue performance and air-voids for the TEH-5 project surface mixes.

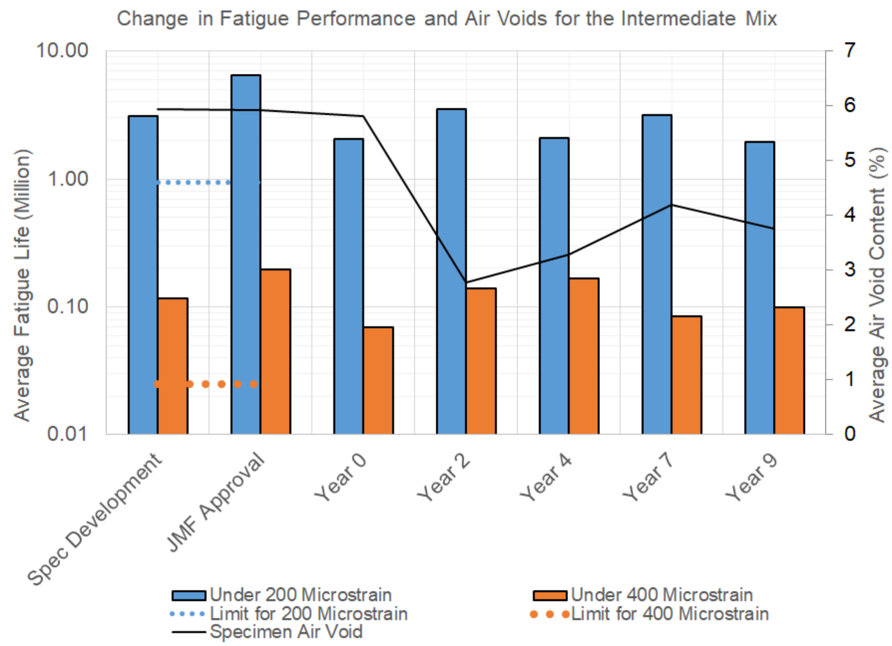


Figure 4.23: Change in fatigue performance and air-voids for the TEH-5 project intermediate mixes.

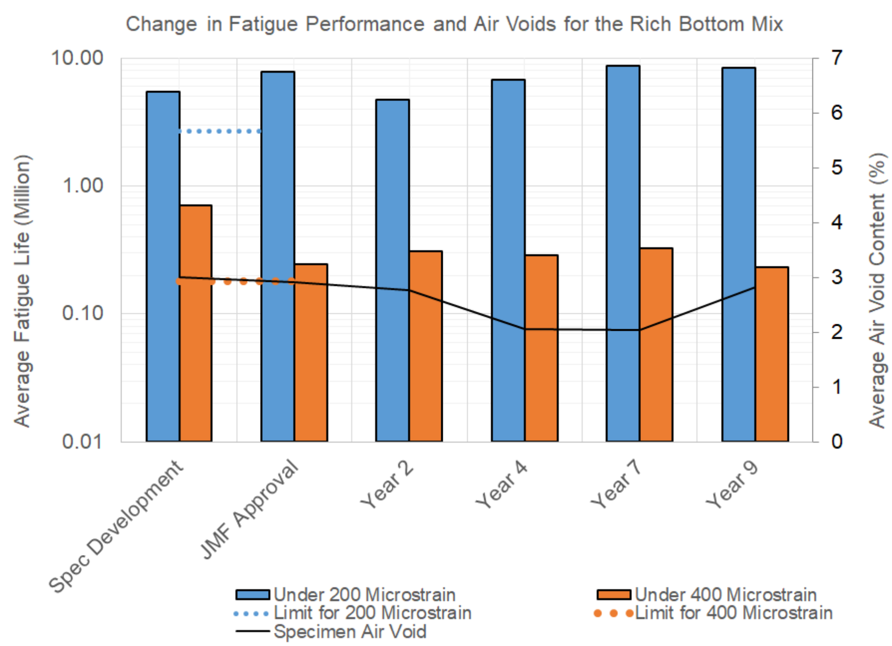


Figure 4.24: Change in fatigue performance and air-voids for the TEH-5 project rich bottom mixes.

Variations in Fatigue Performance Between Different Lifts of the Intermediate Layer

The variations in average fatigue lives and average air-void contents between different lifts of the intermediate layer are shown in Figure 4.25 for the two years when data were available.

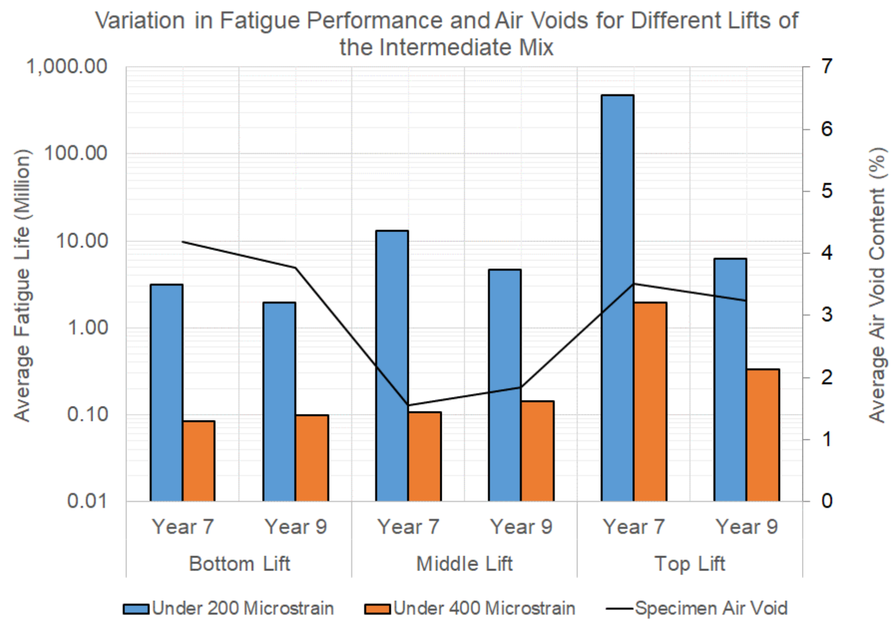


Figure 4.25: Variation in fatigue performance and air-voids for different lifts of the intermediate layer.

The figure suggests that the middle lift has slightly better compaction than both the top and the bottom lifts. The figure also indicates that the upper lifts have longer fatigue lives in general. There is a general trend of decreasing fatigue life from Year 7 to Year 9 for each of the lifts. The decrease is more pronounced for the upper lifts. This finding can be explained by the fact that upper lifts tend to experience more aging than the lower lifts, resulting in reduced fatigue lives at a given strain.

4.4.2 Stiffness Test Results

Change in Stiffness Over Time

The comparison of stiffness master curves for different mixes sampled and tested over the years for the TEH-5 project are shown in Figure 4.26 to Figure 4.28 for the surface, intermediate, and rich bottom layers. These figures indicate that only the surface mix experienced significant stiffness increases with time. The decreases occurred for the mixes sampled in Year 7 and Year 9. This trend matches the changes in fatigue performance over time shown in Section 4.4.1 and can also be explained by aging and, potentially, by sampling at a nearby but different location.

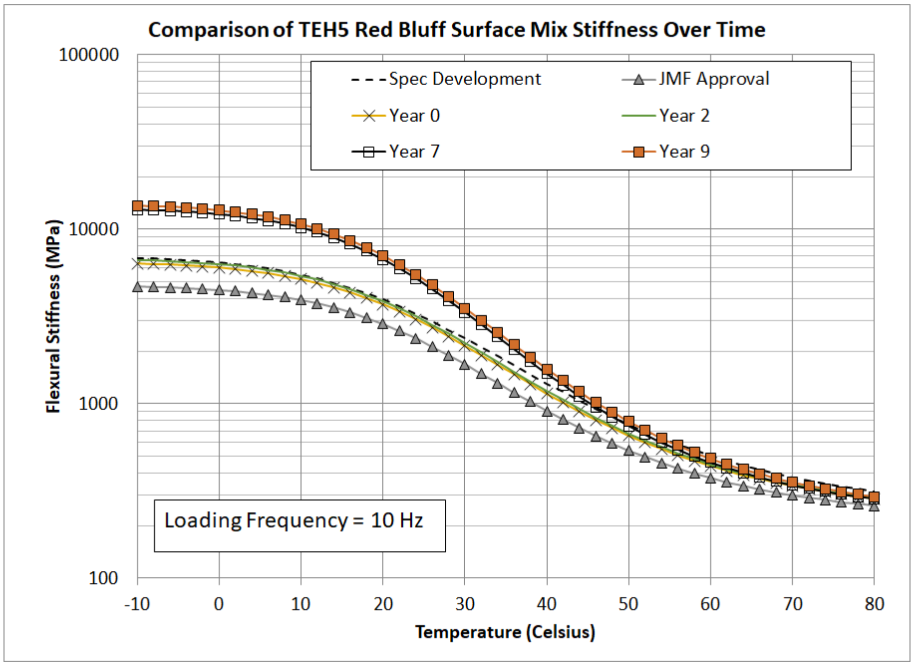


Figure 4.26: Stiffness master curves for TEH-5 project surface mixes.

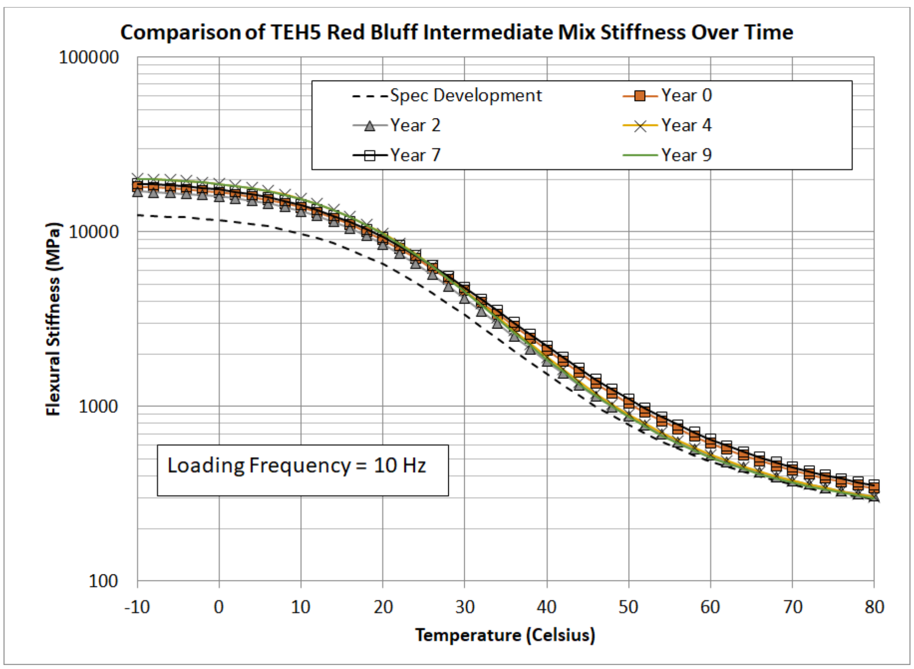


Figure 4.27: Stiffness master curves for TEH-5 project intermediate mixes.

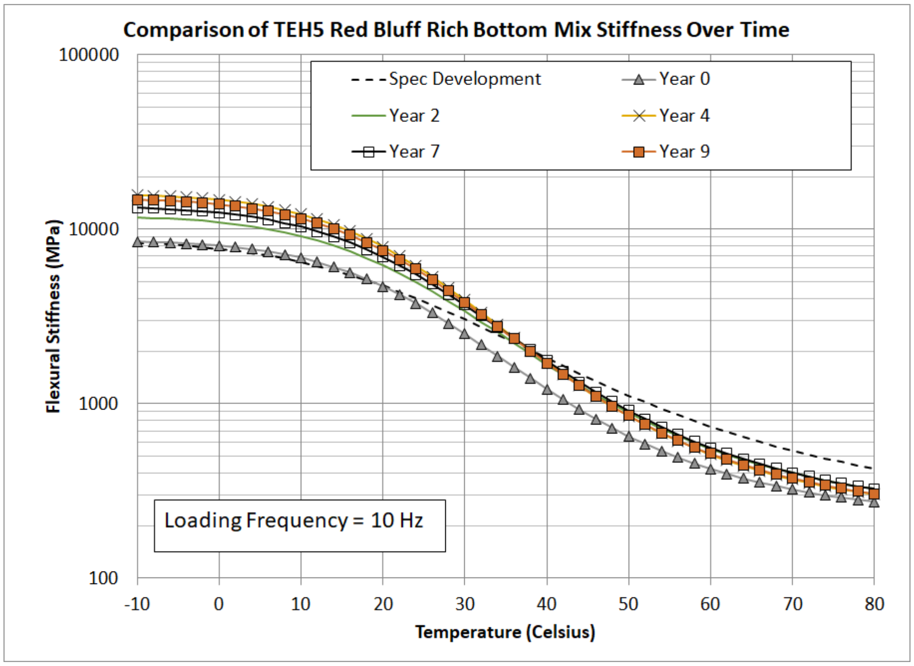


Figure 4.28: Stiffness master curves for TEH-5 project rich bottom mixes.

Variations in Stiffness Between Different Lifts of the Intermediate Layer

The stiffness master curves for different lifts of the intermediate layer are shown in Figure 4.29 for the two years when data were available.

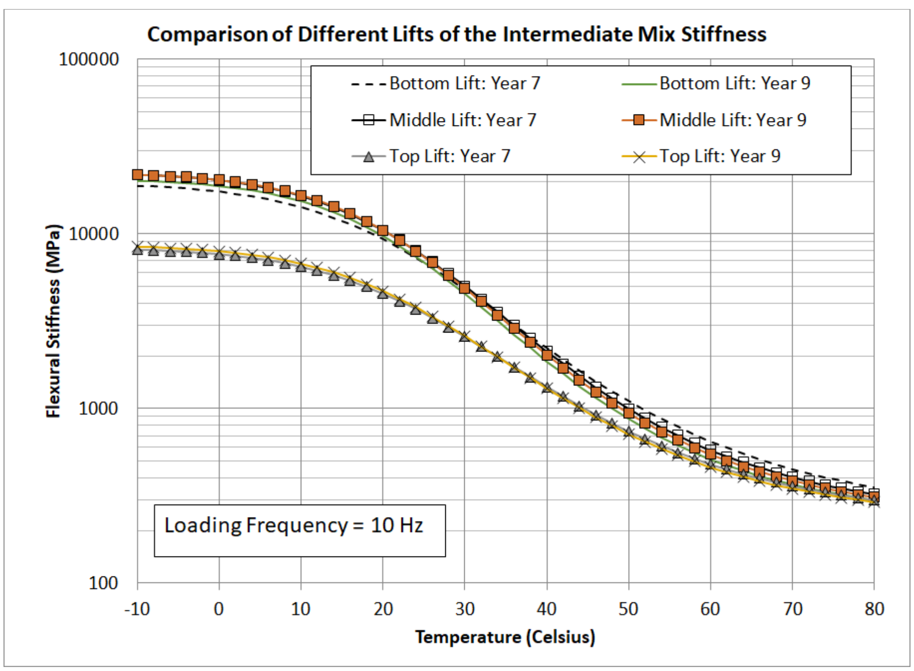


Figure 4.29: Variation in stiffness for different lifts of the TEH-5 project intermediate layer.

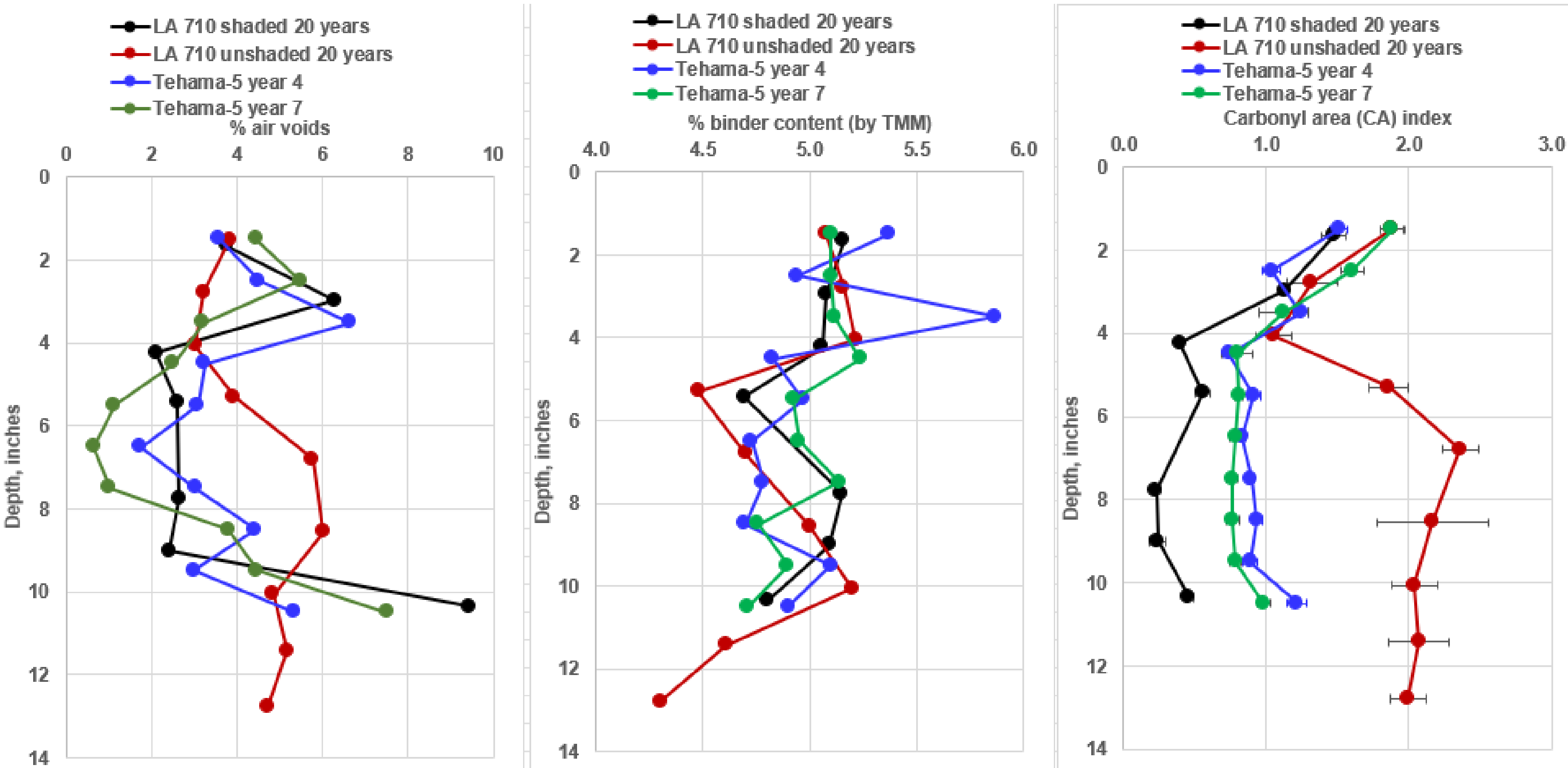
The figure suggests that the middle lift has the same stiffness as the bottom lift, even though it is slightly better compacted (Figure 4.25). The figure also indicates that the top lift has significantly lower stiffness than the bottom and middle lifts. This is consistent with the findings that the top lift has significantly longer fatigue life than the other two lifts (Figure 4.25), following the general observation that softer mix tends to have longer fatigue life. No significant difference between the stiffnesses for Year 7 and Year 9 is observed for each of the lifts. This indicates that the effect of aging on stiffness is minimal between the two sets of samples.

4.5 Aging Testing for TEH-5 and Comparison with LA-710

Slabs taken in 2016 and 2019, four and seven years after construction, were kept in an uncut condition under temperature control until 2023, when cores were taken for air-voids and then samples were taken from their interiors for binder extraction and aging condition testing. The results are plotted in Figure 4.30 along with the results from the LA-710 project discussed in the previous chapter.

It can be seen in the figure that the air-void contents for the surface layers of TEH-5 are similar to those of the shaded core from LA-710 (taken from under the Wardlow Road bridge), which are higher at the bottom of the surface layer at 3.5 in. depth of the shaded LA-710 core. For the intermediate layers, the TEH-5 samples are similar to those of the shaded LA-710 core and lower than the unshaded LA-710 core. All air-voids, except the last sample from the LA-710 shaded core and the Year 7 TEH-5 core, are below the specified 6% air-void content and go as low as 1% in the intermediate layer from the Year 7 TEH-5 core. The low air-void contents would generally be expected to limit air permeability and therefore aging. Binder contents are also similar for the two projects and both layers, around 5.1% for the surface layers and around 4.7% for the intermediate layers. The Year 4 TEH-5 surface layer has some more variability.

Aging can be seen in terms of the carbonyl (CA) content. In all the cores, including the 20-year-old shaded and unshaded LA-710 cores and the Year 4 and Year 7 TEH-5 cores, there is a higher CA at the top of the polymer-modified surface layers, under their respective open-graded layers, to a depth of 4 in. (0.33 ft., 100 mm), decreasing with depth. The intermediate layers from the TEH-5 and the LA-710 shaded cores show similar constant aging from a depth of 4 in. to the bottoms of the 10 in. cores, indicating no aging. The unshaded core from LA-710 has a lower CA in the first 3 in. of the intermediate layer (depths 4 to 7 in.) than the deeper lifts, which are constant from a depth of 7 in. As was discussed in Section 3.2, this may have been due to the somewhat higher air-void contents in the bottom lifts or, more likely, greater aging during mix production and transport because of silo time or extended transport time to the construction site. The Year 4 and Year 7 cores from TEH-5 have almost the same aging, indicating little change in aging, even in this climate region that gets very hot in the summer. Overall, all the sections appear to have not had much aging over the time between construction and coring, likely due to their low air-void contents.



(a) (b) (c)

Note: Error bars indicate one standard deviation of results.

Figure 4.30: Comparison of (a) % air-voids, (b) % binder content, and (c) carbonyl (CA) index for LA-710 and TEH-5 projects.

4.6 Summary

Through multiple rounds of sampling and testing between 2012 (the time of construction) and 2021 (nine years after construction), a large amount of information has been gathered and much has been learned about the structural condition of the AC Long Life section of the TEH-5 project. The deflection testing suggests that there has been some steady aging and curing occurring in the asphalt-bound layer and no damage in the asphalt-bound layer and the CTB layer. The AS layer shows stiffnesses that are significantly higher than typical values, indicating the potential existence of light cementation and the effects of having thick stiff CTB and asphalt layers above it. The AS layer also shows some decrease in layer stiffness, likely due to partial loss of the light cementation.

The stiffness and fatigue data suggest that the surface layer is the only one that has undergone significant aging, which caused increases in stiffness and decreases in fatigue life after seven years. There was no clear sign of aging in the intermediate and rich bottom layers from the mix testing, which matches the results from the CA testing of the extracted binders from these layers. The air-void data suggest that all layers were well compacted, with air-voids below the corresponding target values. Among the three lifts of the intermediate layer, the middle lift has the lowest air-void. Compared to the lower lifts, the upper lifts show a decrease in fatigue life but no increase in stiffness between Year 7 and Year 9.

5 PERFORMANCE OF AS-BUILT MATERIALS

All the AC Long Life projects completed so far used performance-related specifications (PRSs) for construction. The JMF approval process requires that performance-related test (PRT) results meet certain minimum values or fall within certain value ranges. The performance tests used for each project are listed in Table 5.1.

Table 5.1: List of Performance Tests Used for Job Mix Formula Approval in AC Long Life Projects

Project Name	Stiffness and Fatigue	Permanent Deformation	Moisture Sensitivity
LA-710-Long Beach (LA-710)	AASHTO TP8-94 (Precursor of T321)	AASHTO TP-94 (Precursor of T320)	N/A
Sis-5 Weed (SIS-5)	AASHTO T321	AASHTO T320	AASHTO T324
Teh-5-Red Bluff (TEH-5)	AASHTO T321	AASHTO T320	AASHTO T324
Sol-80-Dixon (SOL-80)	AASHTO T321	AASHTO T320	AASHTO T324
Sac-5-Sacramento (SAC-5)	AASHTO T321	AASHTO T378	AASHTO T324

To determine whether the use of PRS led to better as-built materials, the performances of the approved mixes were compared to the statewide median used in *CalME* for routine pavement design. This was achieved by developing parameters for relevant *CalME* models and then comparing the predicted performance from the *CalME* models. Note that *CalME* does not have a model for moisture sensitivity, so it is not covered in this technical memorandum.

5.1 Fatigue Performance

The comparison of predicted Wöhler curves between mixes used on AC Long Life projects and the relevant statewide median mixes is shown in Figure 5.1 to Figure 5.3 for the surface, intermediate, and rich bottom mixes. Details about the *CalME* fatigue model are included in a previous UCPRC technical memorandum *(21)*. The figures suggest the following:

- When compared with the statewide median with PG 64-16 binder and no more than 15% RAP:
    - The surface and rich bottom mixes have longer fatigue life in general, and the intermediate mix has longer fatigue life at lower strain levels but shorter fatigue life at higher strain levels, with the crossover point at around 350 microstrain, well above the strains calculated at the bottom of the intermediate layers.

- When compared with the corresponding statewide median mixes:
    - The surface mix has longer fatigue life in general than the statewide median mix with the PG 64-28PM binder at strain levels lower than 350 microstrain and shorter fatigue life at higher strains.
    - The intermediate mix has longer fatigue life in general than the statewide median mix with 25% RAP at strain levels lower than 350 microstrain and shorter fatigue life at higher strains.
    - The rich bottom mix has shorter fatigue life in general than the statewide median mix for rich bottom.
- The mixes from the SOL-80 project have shorter fatigue life in general than the statewide median mixes.

Considering that most pavement experiences strain levels less than 350 microstrain, these observations suggest that the AC Long Life mixes will perform better than the statewide median mix with PG 64-16 binder and no more than 15% RAP and that they will also perform better than the similar corresponding statewide median mixes. The only exception is the rich bottom mix. The statewide median rich bottom mix is outperforming the corresponding AC Long Life mixes and may need to be adjusted.

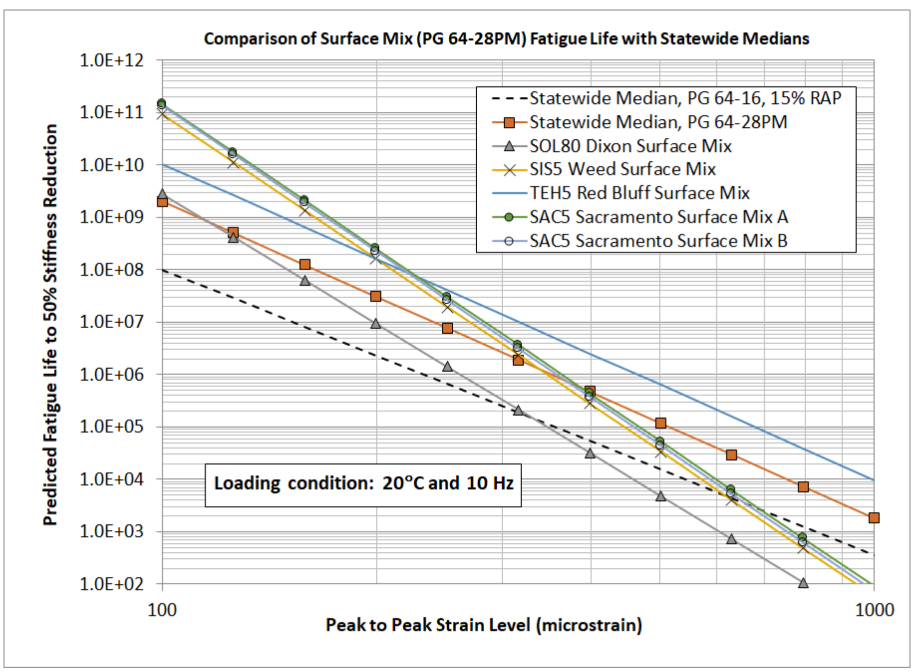


Figure 5.1: Comparison of predicted Wöhler curves for AC Long Life surface mixes with PG 64-16 with 15% RAP and PG 64-28M statewide medians.

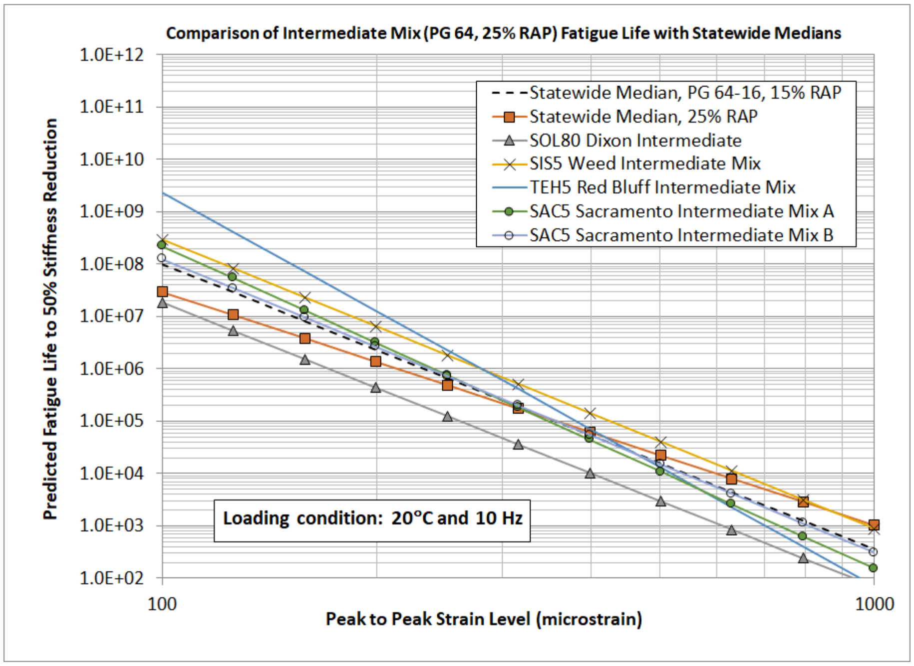


Figure 5.2: Comparison of predicted Wöhler curves for long-life intermediate mixes with PG 64-16 with 15% RAP and PG 64-16 with 25% RAP statewide medians.

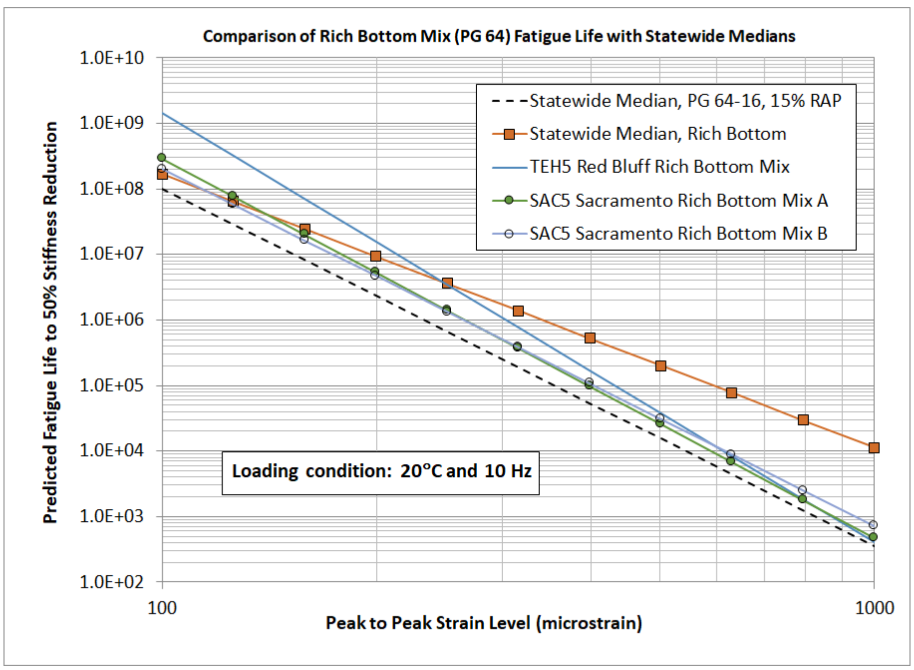


Figure 5.3: Comparison of predicted Wöhler curves for long-life rich bottom mixes with PG 64-16 and 15% RAP and rich bottom statewide medians.

5.2 Stiffness

The comparison of predicted stiffness master curves between mixes used in AC Long Life projects and the relevant statewide median mixes is shown in Figure 5.4 to Figure 5.6 for surface, intermediate, and rich bottom mixes. Details about the *CalME* stiffness master curve model are included in a previous UCPRC technical memorandum *(21)*. The figures suggest the following:

- When compared with the statewide median with PG 64-16 binder and no more than 15% RAP:
    - The surface is softer in general, while the intermediate and rich bottom mixes are stiffer in general.
- When compared with the similar statewide median mixes:
    - The surface mixes are either the same or softer than the statewide median mix with PG 64-28PM binder, except the SOL-80 project, which is much stiffer.
    - The intermediate mixes are generally softer than the statewide median mix with 25% RAP.
    - The rich bottom mixes are slightly stiffer than the statewide median mix for rich bottom.

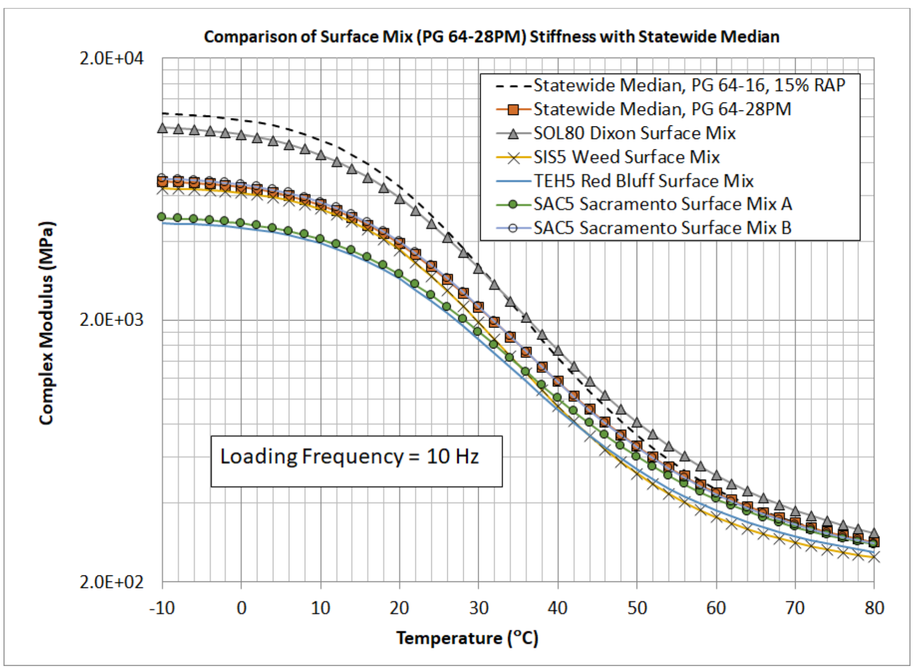


Figure 5.4: Comparison of stiffness master curves with statewide medians for surface mixes and PG 64-16 with 15% RAP.

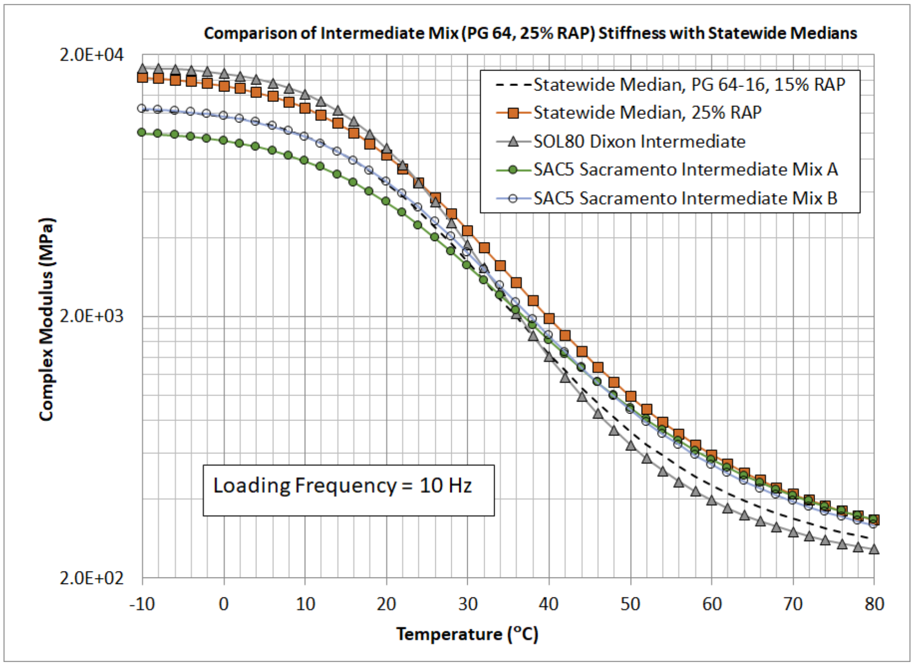


Figure 5.5: Comparison of stiffness master curves with statewide medians for intermediate mixes and PG 64-16 with 15% RAP.

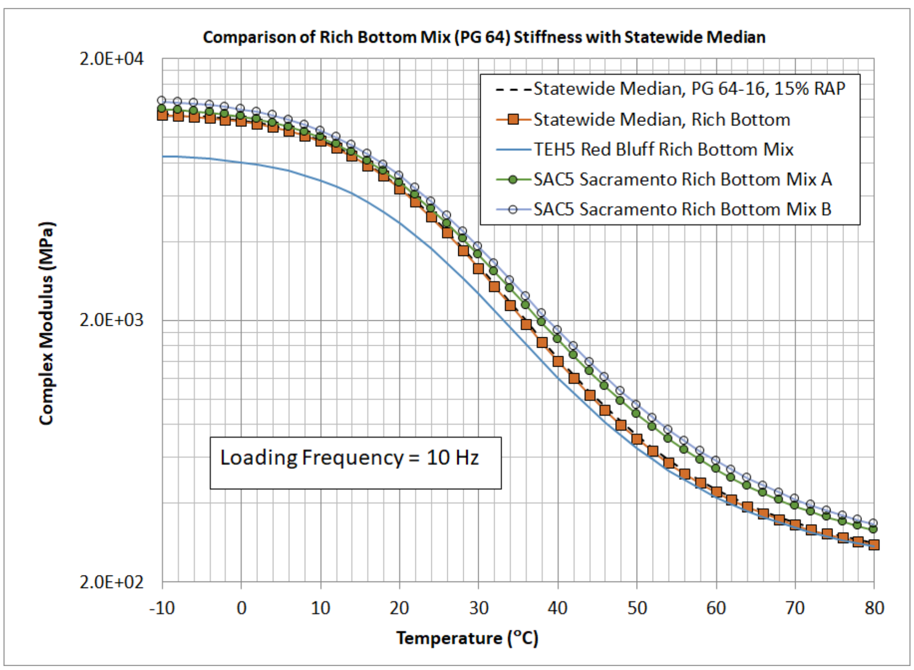


Figure 5.6: Comparison of stiffness master curves with statewide medians for rich bottom mixes and PG 64-16 with 15% RAP.

5.3 Permanent Deformation

In *CalME,* permanent deformation in HMA is attributed to permanent shear deformation. Details about the *CalME* master curve model are included in a previous UCPRC technical memorandum *(21)*. To compare the permanent deformation performance, each mix is subjected to the repeated simple shear test at constant height (RSST, AASHTO T 320), which applies repeated shear stress in one direction with constant peak-to-peak amplitude and rest periods in between. The number of cycles to reach 5% permanent shear strain under different shear stress levels is chosen as the failure criteria because it corresponds to rutting failure in *CalME* if the material is used in a surface layer of 100 mm (4 in.) or thicker.

The comparison of predicted shear resistance between mixes used in AC Long Life projects and the relevant statewide median mixes is shown in Figure 5.7 and Figure 5.8 for surface and intermediate mixes, respectively. Note that only mixes used in the SAC-5 project are included because they were the only ones tested with the full factorial to allow determination of *CalME* model parameters. The figures suggest the following:

- When compared with the statewide median with PG 64-16 binder and no more than 15% RAP:
    - The surface mixes used in the SAC-5 project have less shear resistance.
- When compared with the similar statewide median mix:
    - The surface mix from one producer is more shear resistant than the statewide median with PG 64-28PM, while the surface mix from another producer is less shear resistant.
    - The intermediate mixes from both producers are less shear resistant than the statewide median mix with up to 25% RAP.

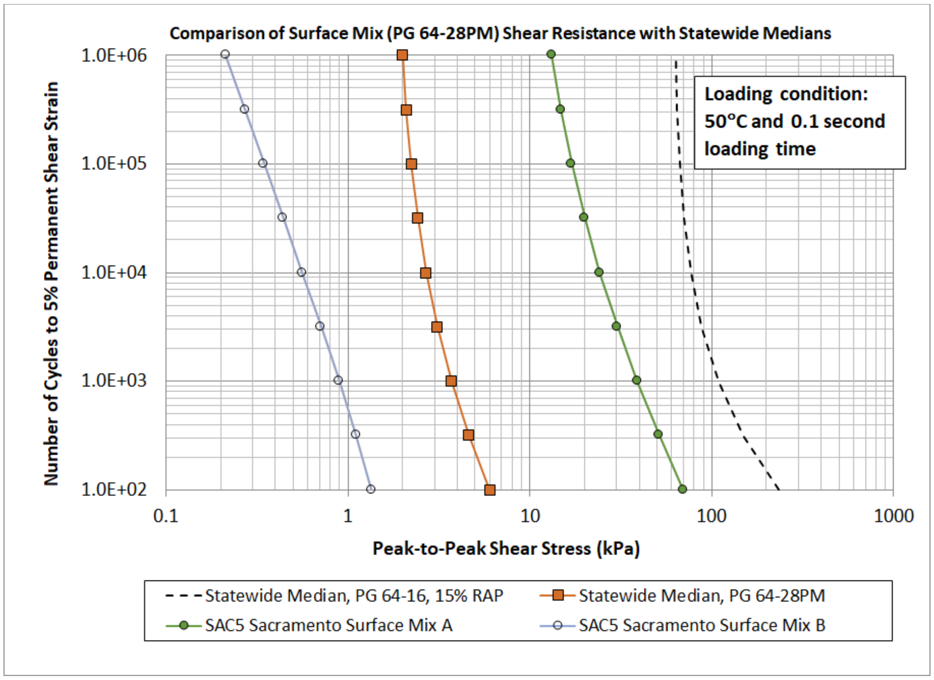


Figure 5.7: Comparison of shear resistance for SAC-5 surface mixes with statewide medians.

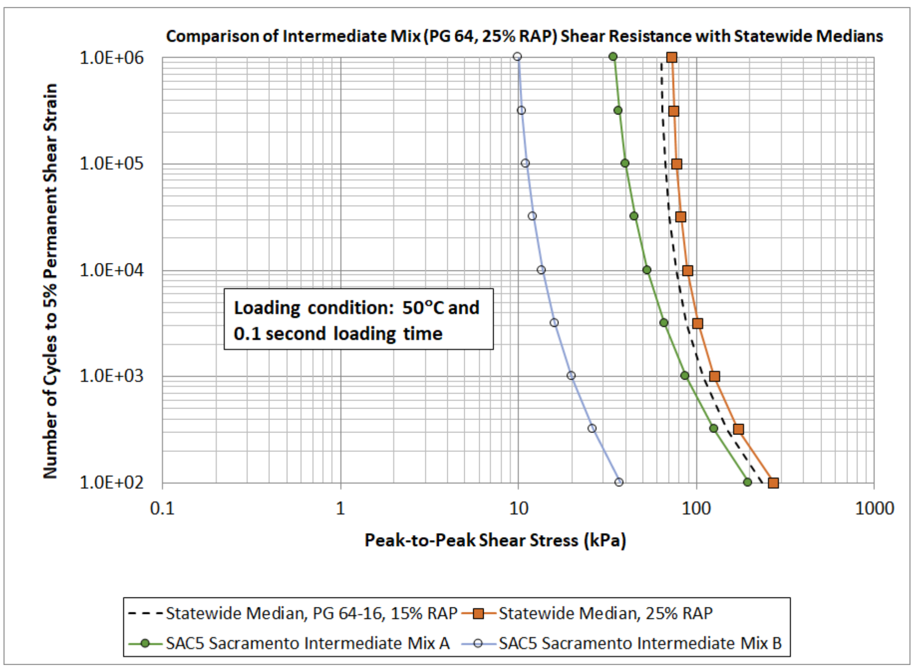


Figure 5.8: Comparison of shear resistance for SAC-5 intermediate mixes with statewide medians.

5.4 Summary

Compared with the statewide median mixes, the approved mixes for all AC Long Life projects are generally found to have better fatigue performance and lower stiffness, as well as lower permanent deformation performance (based on limited data). These pavements have shown no fatigue cracking, except for low levels on the SIS-5 project, and acceptably low levels of rutting under 10 to 20 years of traffic. This indicates that the current mixes being used by Caltrans—for example, those with PG64-16 and up to 15% RAP—may be stiffer and more rut-resistant and have less fatigue cracking resistance than is necessary to achieve good performance in high traffic applications.

6 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

This technical memorandum provides a review of the performance and maintenance histories of all asphalt concrete long-life (AC Long Life) projects designed and built using performance-related tests (PRTs) and performance-related specifications (PRSs), a summary of all sampling and testing conducted on the LA-710 and TEH-5 projects, and a comparison of as-built materials with statewide medians.

6.2 Conclusions

The AC Long Life projects were found to have been performing very well in general with 10 to 20 years of service so far on high truck traffic routes. Compared with statewide median mixes, the as-built materials for AC Long Life projects are generally found to have better fatigue performance, lower stiffness, and lower permanent deformation performance. Given the fact that only minor rutting has been observed in these projects, the as-built materials are believed to be good examples of mixes with balanced performances.

The use of the PRTs and specifications in the AC Long Life projects appears to have produced better balanced mix designs between these three parameters (stiffness, fatigue performance, and rutting performance) than current volumetric mix design procedures. The mixes designed and built with PRTs and PRSs also had low air-void contents, which appear to have resulted in almost no climate-related aging at depths below 0.33 to 0.5 ft. (4 to 6 in.). There are some exceptions to this general trend, suggesting the importance of proper execution of the PRS process.

The extensive sampling and testing in the TEH-5 project suggest that the layers have been well compacted. The AB layers and the CTB layer showed no signs of structural damage. The AS layer is believed to have been lightly cemented and has shown some degradation of the cementation. Aging is mainly observed in the surface layer.

6.3 Recommendations

While preparing this technical memorandum, reviewing the test results indicates that one key factor in successful application of PRS is to allow enough time in the mix design process to make necessary adjustments to mix designs to pass performance test requirements. Pre-qualification of mixes is one potential approach to implement this recommendation.

Some gaps in data availability were encountered. The following are recommendations to improve data availability:

- Similar to the Sac-5 project, include sampling and the full set of performance-related testing of the loose mix for each mix as part of the contract.
- Deflection testing should be conducted at consistent locations consistently to facilitate comparison between different years.

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APPENDIX A SAW CUTTING PLAN FOR TEH-5 RED BLUFF

The slab saw cutting plan for the TEH-5 project is shown in Figure A.1.

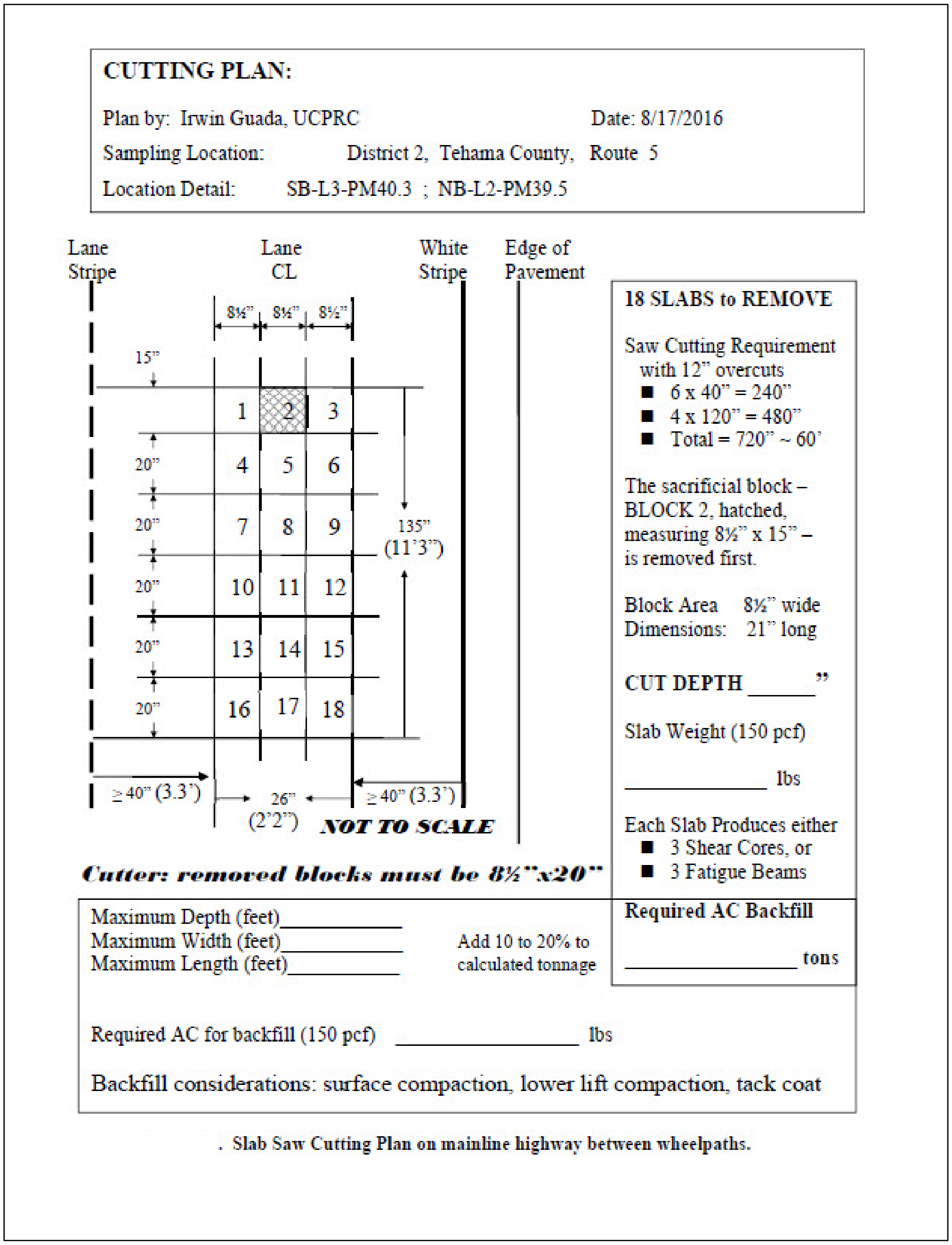

Figure A.1: Slab saw cutting plan for TEH-5 project.

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