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16. ABSTRACT

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Mechanistic-Empirical (ME) Design: Mix Design Guidance for Use with Asphalt Concrete Performance-Related Specifications

Authors:
Rongzong Wu, John Harvey, Jeff Buscheck, and Angel Mateos

Partnered Pavement Research Center (PPRC) Project Number 3.30 (DRISI Task 2667):
Standard Materials Library and Guidance

PREPARED FOR:

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

PREPARED BY:

University of California
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Caltrans has adopted mechanistic-empirical (ME) methods for flexible pavement design, and is using performance-related construction specifications on some projects for hot mix asphalt. Performance-related specifications are used to help ensure that as-built materials meet the performance requirements assumed in ME pavement structural designs. PRS pose new challenges for materials producers and contractors who have never had to relate volumetric mix design parameters to achievement of mechanistic parameters for fatigue life and rutting resistance based on results from performance-related laboratory tests. The objective of this project is to provide guidance to mix designers and contractors to support their decision making regarding changes to mix designs to achieve PRS requirements. The guidance presented in this report was initially developed based on past experience. To validate the guidance and demonstrate its usage, a production mix approved by the California Department of Transportation was selected as the starting point for a set of adjustments applied to the mix and measurement of the effects of each adjustment on mechanistic performance indicators. A total of three sets of adjustments were evaluated, which resulted in a total of four mixes including the baseline. The mechanistic performance parameters evaluated in this study include stiffness, fatigue resistance, and rutting resistance. In addition to direct comparison of laboratory test results, mechanistic-empirical simulations were conducted to evaluate the laboratory mix performance results on predicted pavement performance when the mix was used as a pavement surface layer. The initial mix design guidance was found to be generally consistent with the laboratory test results for the example mix albeit with some minor exceptions. The mix design guidance was then revised based on findings from this study. It is recommended that the revised guidance be used and more data collected to make further improvements.

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5. CALTRANS TECHNICAL LEAD AND REVIEWERS Raghubar Shrestha and Imad Basheer	6. FHWA NUMBER CA192667A

7. PROPOSALS FOR IMPLEMENTATION
It is recommended that the mix design guidance be provided to mix designers, and that information be collected from industry regarding feedback on the effectiveness of the guidance, including results of laboratory testing for different mix changes. It is also recommended that this guidance be updated with results once several more projects have been completed.

8. RELATED DOCUMENTS
UCPRC-SR-2008-04
UCPRC-TM-2013-03
UCPRC-TM-2013-04
UCPRC-TM-2013-05
UCPRC-RR-2015-01

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

AASHTO ACCREDITED

10. SIGNATURES

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DISCLAIMER

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PROJECT OBJECTIVES

This study is part of Partnered Pavement Research Center Strategic Plan Element (SPE) 3.30, a project titled "Mechanistic-Empirical (ME) Design: Standard Materials Library and Guidance." This project is a continuation of Strategic Plan Element 3.18.1, titled "Updated Standard Materials Library." The goal of this project is to continue improving the standard materials library under development by the UCPRC. In addition, guidance will be developed to help design engineers select materials from the library for use in a given project. Guidance will also be developed to help design asphalt concrete mixes to meet performance based-specifications as part of the ME design method.

The objectives for this project will be achieved by completion of the following tasks:

1. Laboratory testing of materials sampled from selected construction projects across the state
2. Field testing of the same materials during and after construction
3. Development of guidance regarding the selection of materials from the standard materials library in the *CalME* design software
4. Development of guidance regarding possible approaches for improving the laboratory testing results for asphalt mixes so they meet performance-related specifications
5. Documentation of the project in reports

Publication of this report completes the fourth task and part of the fifth.

The goal of the work presented in this report is to provide practical guidance for improving the rutting and fatigue results for a mix by using the performance-related tests included in Caltrans performance-related specifications, and to help ensure that the mix meets stiffness requirements.

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EXECUTIVE SUMMARY

Since 2006, the California Department of Transportation (Caltrans) has been transitioning from use of empirical methods for the design of concrete and asphalt pavement to use of mechanistic-empirical (ME) methods. Caltrans has implemented its ME methods for designing new asphalt concrete-surfaced pavement and rehabilitation projects by using a software program called *CalME* that takes advantage of the methods' ability to account for regional differences in climate and materials differences in traffic, a capacity unavailable in empirical methods.

Caltrans also transitioned from the Hveem mix design method to the Superpave method in 2015 and 2016. The Caltrans Superpave method is a volumetric mix design procedure that has requirements for aggregate gradations, aggregate characteristics, air-void content, and voids in the mineral aggregate under a prescribed number of compaction revolutions in the Superpave gyratory compactor, and the mass ratio of dust to asphalt.

An initial evaluation of several mix designs using performance-related tests for rutting, fatigue, and stiffness indicated that the Superpave mix designs generally achieved the goal of improving fatigue performance without having unacceptable probabilities of rutting when compared to Hveem mix designs with the same materials. However, the volumetric method does not produce information regarding the ability of the mix to meet specific performance requirements for rutting and fatigue cracking for a given project designed using ME methods.

To provide a better indication of performance, Caltrans has been working with the UCPRC to develop and implement *performance-related specifications* (PRS) that make use of performance-related laboratory tests. As implemented by Caltrans, the PRS for a mix involves a set of limits regarding results from performance-related laboratory tests on the mix. These limits have been correlated with performance in the field under specific conditions of traffic and climate using ME analysis.

Implementation of PRS by Caltrans from 2002 to 2017 has been on what are called *long-life asphalt pavement projects* (LLAP, sometimes also called *AC Long Life),* which have a 30 or 40 year design life and are on interstate routes with some of the heaviest truck traffic in the state. The following performance-related tests have been used for developing PRS for HMA in California:

- For permanent deformation (rutting): the repeated simple shear test at constant height (AASHTO T 320C); use of this test is being transitioned to use of the repeated load triaxial test (AASHTO T 378).
- For fatigue cracking: the four-point bending beam fatigue test using controlled displacement (adapted from AASHTO T 321).
- For stiffness: the four-point bending beam frequency sweep test (adapted from AASHTO T 321) or initial stiffness in four-point bending beam fatigue test.

Contractors who previously have only had to consider how to meet volumetric properties often do not have experience or training regarding the effects on rutting, fatigue, and stiffness test results that occur when materials in a mix are changed or reproportioned. Contractors have found it difficult on some LLAP projects to know how to design mixes that meet all the performance requirements; this difficulty has caused Caltrans to delay accepting some of mixes, prolonging typical schedules for laboratory testing and acceptance.

This has been identified as a problem for contractors trying to meet performance-related specifications in earlier Caltrans AC Long-Life projects. In order to promote the adoption of performance-related specifications, guidance is needed to help contractors make mix design adjustments that can be expected to move in the right direction to meet the specifications.

In the international materials production and research communities there is some generally accepted knowledge, based on past experience, about adjusting mix designs to meet performance-related specifications based on performance-related laboratory tests. In this study, a mix design guidance flowchart was first developed based on that generally accepted knowledge. This guidance provides different alternatives for improving the rutting and fatigue performance of a mix either at the same time or one at a time.

To verify this guidance flowchart and to demonstrate how it can be used, a production mix provided by an asphalt mixing plant was used as a baseline and three rounds of adjustments to the mix design were evaluated in various laboratory tests to collect data on the effects that the adjustments had or did not have on mix performance—specifically, on whether and/or how the changes affected permanent deformation, fatigue, and moisture damage susceptibility. In addition, the effects on Superpave volumetrics and mix stiffness were also evaluated. The following mixes were compared in the study:

- A benchmark mix with good performance from a previous Caltrans AC Long-Life project on I-5 near the city of Red Bluff
- A baseline production mix received from a Sacramento-area plant plant *(Round One)*
- The baseline mix with a denser gradation *(Round Two)*
- The Round Two mix with a reduced binder content *(Round Three)*
- The Round Two mix with a binder with a stiffer high PG grade *(Round Four)*

To further evaluate the effects of various mix design adjustments on overall pavement rutting and fatigue cracking performance, the ME method was used to analyze pavements with the different mixes used as the surface structural layer. The inputs needed for running the ME analyses were developed based on the laboratory test data collected for each mix.

Two test methods were used to evaluate mix stiffness in this study: the four-point bending beam frequency sweep test (4PBBFST [adapted from AASHTO T 321]) for flexural stiffness, and the dynamic modulus (DM) test (AASHTO T 342) for compressive axial stiffness. It was believed that either of the two test methods is suitable for use in developing performance-related specifications as long the one chosen is used consistently in the ME pavement design.

Two test methods were used to evaluate mix permanent deformation (rutting) performance in this study: the repeated simple shear test at constant height (RSST-CH), and the repeated load triaxial (RLT) test. It was found that that the RSST-CH showed differing effects across different testing temperatures for some of the mix design adjustments, while the RLT showed consistent effects for all the adjustments evaluated in this study across the same temperature range used in RSST-CH test. When using the RLT, it is believed that the unconfined tests may be better suited than the confined tests for mix design comparison because these results showed more pronounced effects for the mix design adjustments evaluated in this study.

Only the four-point bending beam fatigue test (4PBBFT) was used in this study to evaluate mix fatigue performance. Although there was large variability, it is believed that the test method is suitable for use in performance-related specifications for fatigue.

Only Hamburg Wheel-track Testing (HWTT) was used in this study to evaluate mix susceptibility to moisture damage. The test results indicated only a minimal effect from the various mix design adjustments. It is believed that additional data from outside of this study are needed to determine whether this test method is suitable for developing moisture susceptibility-related specifications.

The effects of various mix design adjustments on laboratory performance show trends that were consistent with the mix design guidance proposed, with the following exceptions:

- Using a denser gradation is expected to increase resistance to permanent deformation according to the proposed guidance, except
  - The results from the RSST-CH tests were dependent on the temperature; and
  - The results from the RLT tests suggest that a denser gradation leads to increased permanent deformation.

  Based on these results, a denser gradation may be detrimental, so a decision was made to remove this option from the guidance, although this part of it is in greatest need of evaluation with other mixes.

Based on these observations, the flowchart for mix design guidance has been revised to the one shown on the next page.

Performance-related specifications for mix design are expected to include tests that directly relate to mix performance, such as the tests for resistance to permanent deformation (related to pavement rutting) and fatigue damage (related to pavement fatigue cracking) that were used in this study. This study shows that it is also important to include mix stiffness as part of the specifications. As a first step, performance-related specifications should use laboratory test results alone to make them practical. Eventually, however, they should be based on mechanistic-empirical analyses of pavement performance, which themselves use inputs developed from the laboratory test results.

As shown in this study, the proposed mix design guidance developed based on past experience is in general consistent with laboratory test results and pavement performance simulations using ME analysis. The guidance has been revised based on the findings in this study, and it is believed to provide reasonable options for improving mix performance regarding rutting and fatigue. It is recommended that the revised mix design guidance be provided to industry for informational purposes only, as a nonmandatory advisory for Caltrans highway construction projects involving performance-related specifications, with appropriate notification that the guidance is not necessarily applicable to all materials. Additional data collected from these highway projects and potentially from future research projects should be used to further validate and improve the mix design guidance.


Green Boxes
Adjustments good for both fatigue and rutting
Yellow Boxes
Adjustment with uncertain effect
Blue Boxes
Adjustment good for fatigue but bad for rutting
Brown Boxes
Adjustment good for rutting but bad for fatigue
Starting Mix
Run baseline testing on job mix formula
Decide what needs to be improved
Try options in green, and balance options in blue, yellow options may or may not help
Is gradation off 0.45 power curve?
1. Move closer to curve while meeting VMA requirement
Is natural sand (#30 to #8) in gradation?
2. Replace sand with crushed particles
Is dust content in specification but low?
3+. Increase dust content and adjust binder content if needed for dust proportion specification
Is there minimum crushing of coarse aggregate?
4. Use more crushing, more crushed faces and higher % with two crushed faces
5+. Increase binder content
6-. Decrease RAP content
7. Change RAP source if applicable
8. Change binder source
10a. Use polymer-modified binder with same or better grades
Is natural sand (#30 to #8) in gradation?
2. Replace sand with crushed particles
Is dust content in specification but high?
3-. Decrease dust content and adjust binder content if needed for dust proportion specification
Is there minimum crushing of coarse aggregate?
4. Use more crushing, more crushed faces and higher % with two crushed faces
5-. Reduce binder content
6+. Increase RAP content
7. Change RAP source if applicable
8. Change binder source
9. Use a stiffer unmodified binder (i.e., with better high-temperature grade)
10b. Use polymer-modifier with same or better high temperature grade

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LIST OF ABBREVIATIONS

4PBBFST	Four-Point Bending Beam Frequency Sweep Test
4PBBFT	Four-Point Bending Beam Fatigue Test
DM	Dynamic modulus
DPe	Effective dust proportion
HMA	Hot mix asphalt
HWTT	Hamburg Wheel-Track Test
JMF	Job mix formula
LLAP	Long-life asphalt pavement projects
ME	Mechanistic-Empirical
NMAS	Nominal maximum aggregate size
PG	Performance graded
PPRC	Partnered Pavement Research Center
PRS	Performance-related specifications
QC	Quality control
RAP	Reclaimed asphalt pavement
RLT	Repeated load triaxial test
RSST-CH	Repeated Simple Shear Test at Constant Height
TI	Traffic index
UCPRC	University of California Pavement Research Center
VMA	Voids in the mineral aggregate

LIST OF TEST METHODS AND SPECIFICATIONS USED IN THE REPORT

AASHTO PP 3	Standard Practice for Preparing Hot Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor
AASHTO R 30	Standard Practice for Mixture Conditioning of Hot Mix Asphalt (HMA)
AASHTO T 2	Standard Method of Test for Sampling of Aggregates
AASHTO T 166A-12	Standard Method of Test for Bulk Specific Gravity (GMB) of Compacted Hot Mix Asphalt (HMA) Using Saturated Surface-Dry Specimens (Method A)
AASHTO T 248-14	Standard Method of Test for Reducing Samples of Aggregate to Testing Size
AASHTO T 275A-07	Standard Method of Test for Bulk Specific Gravity (GMB) of Compacted Asphalt Mixtures Using Paraffin-Coasted Specimens (Method A)
AASHTO T 283-14	Standard Method of Test for Resistance of Compacted Asphalt Mixtures to Moisture-Induced Damage
AASHTO T 312-14	Standard Method of Test for Preparing and Determining the Density of Asphalt Mixture Specimens by Means of the Superpave Gyratory Compactor
AASHTO T 308-10	Standard Method of Test for Determining the Asphalt Binder Content of Hot Mix Asphalt (HMA) by the Ignition Method
AASHTO T 320C	Standard Method of Test for Determining the Permanent Shear Strains and Stiffness of Asphalt Mixtures Using the Superpave Shear Tester (SST) (Procedure C)
AASHTO T 321-14	Standard Method of Test for Determining the Fatigue Life of Compacted Asphalt Mixtures Subjected to Repeated Flexural Bending
AASHTO T 324-16	Standard Method of Test for Hamburg Wheel-Track Testing of Compacted Asphalt Mixtures
AASHTO T 331-13	Standard Method of Test for Bulk Specific Gravity (GMB) and Density of Compacted Hot Mix Asphatl (HMA) Using Automatic Vacuum Sealing Method
AASHTO T 378-17	Standard Method of Test for Determining the Dynamic Modulus and Flow Number for Asphalt Mixtures Using the Asphalt Mixture Performance Tester (AMPT)
AASHTO T 342-11	Standard Method of Test for Determining Dynamic Modulus of Hot Mix Asphalt (HMA)
AASHTO TP 79-15	Superseded by AASHTO T 378-17
AASHTO TP 70-12	Standard Method of Test for Multiple Stress Creep Recovery (MSCR) Test of Asphalt Binder Using a Dynamic Shear Rheometer (DSR)

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	Millimeters	mm
ft	feet	0.305	Meters	m
yd	yards	0.914	Meters	m
mi	miles	1.61	Kilometers	Km
		AREA		
in^2	square inches	645.2	Square millimeters	mm^2
ft^2	square feet	0.093	Square meters	m^2
yd^2	square yard	0.836	Square meters	m^2
ac	acres	0.405	Hectares	ha
mi^2	square miles	2.59	Square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	Milliliters	mL
gal	gallons	3.785	Liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
		NOTE: volumes greater than 1000 L shall be shown in m^3		
		MASS		
oz	ounces	28.35	Grams	g
lb	pounds	0.454	Kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
		TEMPERATURE (exact degrees)		
°F	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	°C
		ILLUMINATION		
fc	foot-candles	10.76	Lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	Newtons	N
lbf/in^2	poundforce per square inch	6.89	Kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	Inches	in
m	meters	3.28	Feet	ft
m	meters	1.09	Yards	yd
km	kilometers	0.621	Miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	Hectares	2.47	Acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
mL	Milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	Gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	Ounces	oz
kg	kilograms	2.202	Pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact degrees)		
°C	Celsius	1.8C+32	Fahrenheit	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	Poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380 (Revised March 2003).

1 INTRODUCTION

1.1 Background

Since 2006, the California Department of Transportation (Caltrans) has been transitioning from use of empirical methods for the design of concrete and asphalt pavement to use of mechanistic-empirical (ME) methods. Caltrans has implemented its ME methods for designing new asphalt concrete-surfaced pavement and rehabilitation projects by using a software program called *CalME* that takes advantage of the methods' ability to account for regional differences in climate and materials differences in traffic, a capacity unavailable in empirical methods. *CalME* does this by accessing regional data on climate and example materials for different regions, as well as lane- and post-mile–based traffic counts and axle load spectra that are stored in respective standard libraries. The libraries containing these climate and traffic databases are also integrated with the Caltrans pavement management system software *Pavement Analyst*™ and other tools that are being developed for Caltrans by the University of California Pavement Research Center (UCPRC).

This overall project, Partnered Pavement Research Center Strategic Plan Element (UCPRC SPE) 3.30, is focused on the ongoing expansion and improvement of the standard materials library by the UCPRC for California asphalt-surfaced pavement design. The project also includes the following two tasks relating to design guidance:

- Develop guidance for designers regarding selection of appropriate materials for pavement design in different regions and for different purposes in the pavement structure
- Develop guidance for asphalt mix designers regarding steps they can take that may improve the rutting, fatigue, and stiffness properties of asphalt mixes so they meet performance-related specifications

Caltrans transitioned from the Hveem mix design method to the Superpave method in 2015 and 2016. Caltrans made this transition in order to seek mix designs that better balance rutting and cracking than those derived using the Hveem mix design method, which was considered to generally produce mixes with very low probabilities of rutting but that were potentially less durable with regard to desired amounts of cracking. The intention was to still achieve acceptable risks of rutting with improved fatigue and other durability-related performance. There were also increasing problems with obtaining the Hveem testing equipment.

The Caltrans Superpave method is a volumetric mix design procedure that has requirements for aggregate gradations, aggregate characteristics, air-void content, and voids in the mineral aggregate under a prescribed number of compaction revolutions in the Superpave gyratory compactor, and the mass ratio of dust to asphalt. These limits, when used with the performance-graded (PG) asphalt binder specification, are expected to produce mixes that experience has shown have acceptable probabilities of good performance with respect to rutting, low-temperature cracking, fatigue cracking, bleeding, and raveling without much consideration of where in the pavement structure the mix will be used, the amount of truck traffic, or the climate region, except through the requirements for the binder in the PG binder specification. The Caltrans Superpave method also includes limits for test results from Hamburg Wheel-track Testing (HWTT, AASHTO T 324), which provides an indication of both moisture damage susceptibility and rutting, and AASHTO T 283, which provides an indication of moisture susceptibility *(1)*.

An initial evaluation of several mix designs using performance-related tests for rutting, fatigue, and stiffness indicated that the Superpave mix designs generally achieved the goal of improving fatigue performance without having unacceptable probabilities of rutting when compared to Hveem mix designs with the same materials *(2, 3)*. However, the volumetric method does not produce information regarding the ability of the mix to meet specific performance requirements for rutting and fatigue cracking for a given pavement's expected traffic and climate. To provide a better indication of performance, Caltrans has been working with the UCPRC to develop and implement *performance-related specifications* (PRS) that make use of performance-related laboratory tests. PRS has been defined by an AASHTO Quality Construction Task Force *(4)* to include:

- Acceptance based on key quality characteristics that have been found to correlate with fundamental engineering properties that predict performance, and
- Mathematical models used to quantify the relationship between key materials and construction quality characteristics and product performance.

As implemented by Caltrans, the PRS for a mix involves a set of limits regarding results from performance-related laboratory tests on the mix. These limits have been correlated with performance in the field under specific conditions of traffic and climate using mechanistic-empirical (ME) analysis.

A goal of Caltrans implementing the ME design of pavement structures is to integrate asphalt concrete mix design and pavement structural design. To help achieve this, the same laboratory tests need to be used in both the Caltrans PRS to accept materials produced by the contractor during construction, and to produce the material properties used in ME design. In some cases, the test used to produce properties for the standard materials library integrated in *CalME* may be too slow and costly to use for construction acceptance, in which case simpler and less costly tests must be correlated back to the tests used for ME design properties. The adoption of PRS is a key part of the Caltrans transition to ME design method because their use in construction specifications for projects in different regions of the state helps to build the standard materials library used by pavement designers. Each time a material is accepted for a project, the results of the testing are put into the standard materials library.

Implementation of PRS by Caltrans from 2002 to 2017 has been on what are called *long-life asphalt pavement projects* (LLAP, sometimes also called *AC Long Life),* which have a 30 or 40 year design life and are on interstate routes with some of the heaviest truck traffic in the state *(5, 6, 7, 8)*. The following performance-related tests have been used to develop PRS for HMA in California:

- For permanent deformation (rutting): the repeated simple shear test at constant height (RSST-CH, AASHTO T 320C); use of this test is being transitioned to use of the repeated load triaxial test (RLT, AASHTO T 378).
- For fatigue cracking: the four-point bending beam fatigue test (4PBBFT) using controlled displacement (adapted from AASHTO T 321).
- For stiffness: the four-point bending beam frequency sweep test (4PBBFST) (adapted from AASHTO T 321) or initial stiffness in four-point bending beam fatigue test.

Contractors who previously have only had to consider how to meet volumetric properties often do not have experience or training regarding the effects on rutting, fatigue, and stiffness test results that occur when materials in a mix are changed or reproportioned. Contractors have found it difficult on some LLAP projects to know how to design mixes that meet all the performance requirements; this difficulty has caused Caltrans to delay accepting some of mixes, prolonging typical schedules for laboratory testing and acceptance.

In the international materials production and research communities there is some generally accepted knowledge, based on past experience, about adjusting mix designs to meet performance-related specifications based on performance-related laboratory tests. The following are examples:

- Changes in aggregate characteristics, particularly to surface texture and shape, and aggregate gradation can be made that often improve both rutting and fatigue cracking performance, although the changes may improve only one or reduce the performance of both criteria.
- Changes in aggregate can also affect stiffness.
- The asphalt binder selected for a project can affect rutting, fatigue cracking, and stiffness test results because of the binder's PG grade and because of variation within a given PG grade with binders from different suppliers. Selection of the asphalt binder, including both PG grade and supplier within a given PG grade, can affect rutting, fatigue cracking and stiffness test results.
    - The PG binder specification explicitly includes performance-related binder criteria for rutting. It also includes limits on the maximum stiffness at intermediate temperature that are implicitly related to fatigue cracking when the mix is used as a relatively thin overlay, but does not explicitly include criteria for fatigue.
    - The PG binder intermediate temperature specification can potentially decrease fatigue cracking performance when the mix is used in thick overlays or thick new asphalt pavement because the

use of stiffer binders to reduce tensile strains under traffic loading may have a more significant effect on pavement fatigue cracking performance than the reduction in fatigue resistance at a given tensile strain that often occurs when a stiffer binder is used.

    - For these reasons, the consideration of mix stiffness and fatigue cracking test results on expected pavement performance must be evaluated using an ME analysis of the pavement structure that takes traffic loading and climate into account.

- Increased binder stiffness at intermediate temperatures, as well as the inclusion of reclaimed asphalt pavement (RAP), will generally increase the stiffness of a mix and reduce flexural beam fatigue life at a given tensile strain.
- Increasing the binder content of a mix will generally increase rutting and reduce stiffness as measured by performance-related tests and increase flexural beam fatigue test results.

Mix design guidance can be developed by bringing this generally applicable knowledge into a framework to help materials producers adjust their existing mix designs to meet performance-related specifications.

1.2 Problem Statement

As can be seen from the background discussion, a material producer or contractor has a number of choices for improving the rutting, fatigue, and stiffness results of an asphalt concrete mix design so that it meets performance-related specifications. However, it can also be seen that the choices can be complex and changes that improve one required property can worsen another. Contractors who previously only had to consider how to meet volumetric properties often have no experience, training, or guidance about the effects that changing either the materials included in the mix or their proportions will have on rutting, fatigue, and stiffness test results. This has been identified as a problem for contractors trying to meet performance-related specifications in earlier Caltrans AC Long-Life projects. In order to promote the adoption of performance-related specifications, guidance is needed to help contractors make mix design adjustments that can be expected to move in the right direction to meet the specifications.

1.3 Study Goal and Objectives

This study is part of Partnered Pavement Research Center Strategic Plan Element (SPE) 3.30, a project titled “Mechanistic-Empirical (ME) Design: Standard Materials Library and Guidance.” This project is a continuation of Strategic Plan Element 3.18.1, titled “Updated Standard Materials Library.” The goal of this project is to continue improving the standard materials library under development by the UCPRC. In addition, guidance will be developed to help design engineers select materials from the library for use in a given project. Guidance will also be developed to help design asphalt concrete mixes for meeting performance based-specifications as part of the ME design method.

The objectives for this project will be achieved by completion of the following tasks:

1. Laboratory testing of materials sampled from selected construction projects across the state,
2. Field testing of the same materials during and after construction,
3. Development of guidance regarding the selection of materials from the standard materials library in the *CalME* design software,
4. Development of guidance regarding possible approaches for improving the laboratory testing results for asphalt mixes so they meet performance-related specifications, and
5. Documentation of the project in reports.

Publication of this report completes the fourth task and part of the fifth.

The goal of the work presented in this report is to provide *general* practical guidance for improving the rutting and fatigue results for a mix by using the performance-related tests included in Caltrans performance-related specifications, and to help ensure that the mix meets stiffness requirements.

1.4 Study Approach for This Report

For this report, the UCPRC developed a flowchart based on past UCPRC and AC Long-Life mix design experience that shows the steps a contractor can take to improve rutting and fatigue properties as measured using performance-related tests, and the effects on stiffness. To validate the flowchart and provide an example, a selected set of the steps in the flowchart were demonstrated using a local mix, varying one mix design variable at a time, testing the mix after each change, and documenting the results of those changes. The initial flowchart was then revised based on findings from the testing. It should be noted that the flowchart only covers fatigue and rutting performance, and does not include stiffness because achieving required rutting and fatigue properties is often more difficult than meeting stiffness requirements.

The results presented in this report do not fully verify the proposed mix design guidance, and is only expected to provide practical and reasonable options for contractors to improve mix performance as measured using the performance-related tests used in Caltrans AC Long-Life performance-related specifications. It is expected that additional experience with a larger set of materials will result in improvements in the guidance presented in this report.

1.5 Important Note Regarding Guidance in This Report

It must be noted that the general guidance included in this document does not guarantee that a mix will meet a given performance-related specification, or that the results obtained for a given mix will match the experience shown in the flowchart or in the example included in this report. The UCPRC and Caltrans make no claims regarding the effectiveness of the guidance shown for any specific mix.

2 MIX DESIGN GUIDANCE

2.1 Factors Affecting Mix Performance

A list of mix design adjustments and their generally expected effects on mix performance parameters measured using the performance-related tests used in Caltrans LLAP procedures are summarized in Table 2.1. As noted in Section 1.1, the table's contents are based on the experience of some in the international materials production and research communities. In the table, an upward arrow with green shading indicates an increase in performance while a downward arrow with red shading indicates decreased performance. A question mark indicates the possibility of either an increase or a decrease in performance.

Table 2.1: List of Adjustments in Mix Design and the Generally Expected Effect on Performance Results from Performance-Related Tests

Adjustment	Repeated Load Triaxial[1] Rutting Performance	Flexural Fatigue Performance[2,3]	Flexural Stiffness[2,3]
Replace natural sand with crushed particles	↑[4]	↑	↑
Get gradation closer to 0.45 power curve	↑	↑	↑
Increase crushing of coarse aggregates	↑	↑	↑
Reduce dust content	↑	?	?
Reduce binder content	↑	↓	↑
Increase RAP content	↑	↓	↑
Use stiffer conventional binder	↑	↓	↑
Use polymer-modified binder designed for rutting	↑	?	?
Use polymer-modified binder designed for fatigue	?	↑	↓
Change binder source	?	?	?

Notes:
[1]: Using Caltrans LLAP procedures adapted from AASHTO T 378.
[2]: Using Caltrans LLAP procedures adapted from AASHTO T 321.
[3]: Fatigue performance of the pavement will depend on the thickness of the new asphalt layer, the rest of the pavement structure, traffic loads and climate, and the interaction of these factors must be evaluated using ME analysis.
[4]: An ↑ with green shading indicates increased performance; ↓ with red shading indicates decreased performance; and ? indicates a possible increase or a decrease in performance.

2.2 Mix Design Flowchart

Based on the various possible adjustments to mix design and their effects as shown in Table 2.1, a flowchart for improving mix performance was developed and is shown in Figure 2.1. The options for adjusting mix design are color coded in the figure to indicate whether they improve both fatigue and rutting performance, improve one parameter but degrade the other, or if there is no general expectation for its effect. In addition, the options are shown in the expected approximate order of cost and uncertainty, with the initial options (low numbers) in general indicating lower cost and/or less uncertainty. The + and/or - signs following the numbers indicate whether the adjustment is increasing or decreasing the expected value of a certain parameter (fatigue or rutting). The order shown in Figure 2.1 is very approximate and will differ for different material producers and in different regions. For example, it might be easy for a mixing plant to change its binder source if it has no multiyear contract with a given refinery or it might be difficult if there is a long-term contract. Similarly, changes in aggregate crushing or gradation may be more difficult or costly at one plant than another.

The following are some details about the flowchart:

- For a mix that contains reclaimed asphalt pavement (RAP), the combined gradation (that is, the gradation that includes both virgin aggregates and RAP aggregates) should be used when determining whether the gradation is close to the 0.45 power line.
- To check whether dust content is high or low, the percent aggregate passing the #200 sieve should be compared against the specification. In addition, the dust proportion should also be calculated and compared against the specification.
- To check the extent of crushing of coarse aggregates, use the aggregates retained on the #4 (4.26 mm) sieve. The crushing extent should be determined for each bin of virgin aggregate.

Note that the flowchart only covers fatigue and rutting performance. Specifications may also require a minimum stiffness, particularly if the overlay is thicker than the minimum overlay thickness in order to add structural capacity.

The first step in use of the flowchart is to determine the performance of the original mix design as a baseline and to identify whether the fatigue or rutting performance needs to be improved to meet the specification. The second step is to evaluate the feasibility of each adjustment using the assigned number (which indicates associated cost) as a general guide, but using information specific to the material producer where available. The options numbered 1 to 4 may or may not be applicable depending on the answers to the questions listed in the white diamond. If they are not applicable, then the step should be skipped.

Figure 2.1: Initial flowchart proposed for improving the fatigue or rutting performance of an asphalt concrete mix.

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3 DEMONSTRATION OF THE GUIDANCE

The mix design guidance developed in Chapter 2 only provides a qualitative indication of the effects of various adjustments. In order to provide quantitative validation and to demonstrate how to use this guidance, a testing program was conducted to show how a baseline production mix can be improved by following it. Specifically, a job mix formula (JMF) approved by Caltrans as meeting the Superpave specification was selected to undergo several rounds of adjustments to determine the effects of each adjustment on the mechanistic performance-related test results of the mix.

3.1 Experimental Plan

The JMF went through three iterations of adjustments, which required four rounds of testing, including the first round for establishing the baseline performance. The mixes for the four rounds are referred to as the *Round One mix,* the *Round Two mix,* the *Round Three mix,* and the *Round Four mix*, respectively. In some cases, the Round One mix is also referred to as the *baseline mix.* The adjustments were selected following the flowchart shown in Figure 2.1 after a review of the test results of each round.

In order to use the flowchart, the performance of the baseline mix needs to be evaluated against that of a benchmark mix to determine whether fatigue or rutting resistance of the baseline mix need to be improved. A mix used in the main structural layer for the Interstate 5 long-life project near the city of Red Bluff *(6)* in 2012 was selected as the benchmark. This mix was selected because it had a similar binder grade (PG 64-10), the same RAP content (25 percent), and the same aggregate rock type (crushed alluvial) as the Round One mix (i.e., the baseline mix) although all the baseline materials came from different sources and had been sampled four years earlier. This benchmark mix is often referred to as the *Red Bluff mix* and occasionally as the just *benchmark mix.*

3.1.1 List of Tests for Performance Evaluation

The ultimate goal of improving a mix is to reduce the various distresses in the pavements built with the material. Among the many distresses occurring in asphalt pavements, rutting and fatigue cracking are the most important and therefore they were chosen as the focus of this study. The performance tests conducted in this study and the corresponding specimen preparation details are listed in Table 3.1. The list includes testing for stiffness because this property affects the pavement strain that affects pavement fatigue life, and the interactions of these two properties were later evaluated using *CalME* as part of the study. Both the repeated simple shear test at constant height (RSST-CH) and repeated load triaxial (RLT) tests were included because Caltrans is transitioning from using the RSST-CH to the RLT for rutting performance evaluation. The Hamburg Wheel-track Test (HWTT) to measure moisture susceptibility was included because it is a routine test required by Caltrans.

Table 3.1: List of Tests to Characterize Asphalt Mix Performance

Test No.	Target Performance	Test	AASHTO Specification	Target Air Voids (AASHTO T 331)	Compaction Equipment
1	Stiffness	Flexural Frequency Sweep	T 321	6+/-0.5%*	Rolling Wheel Compactor
2	Stiffness	Dynamic Modulus Test	T 342	7+/-0.5%	Superpave Gyratory Compactor
3	Fatigue	Flexural Fatigue	T 321	6+/-0.5%*	Rolling Wheel Compactor
4	Rutting	Repeated Simple Shear Test at Constant Height (RSST-CH)	T 320C	3+/-0.5%x	Rolling Wheel Compactor
5	Rutting	Repeated Load Triaxial Test Using Asphalt Mix Performance Tester (AMPT)	T 378 (TP 79)	7+/-0.5%	Superpave Gyratory Compactor
6	Moisture susceptibility	Hamburg Wheel-track Testing (HWTT)	T 324	7+/-0.5%	Superpave Gyratory Compactor

Notes: *: The target air-void content for beams was selected to be consistent with historical data.

x: The target air-void content for RSST-CH cores was selected based on recommendation from the SHRP study.

3.1.1.1 Flexural Frequency Sweep Test for Stiffness

The flexural frequency sweep test (AASHTO T 321) for stiffness was conducted at 50, 68, and 86°F (10, 20, and 30°C) respectively with a target peak-to-peak sinusoidal strain amplitude of 100 microstrain. The frequency sweep starts at 15 Hz and ends at 0.01 Hz. The experiment design is shown in Table 3.2. Each specimen was tested at only one temperature. This required a total of six specimens with two replicates at each temperature.

Table 3.2: Experiment Design for Flexural Frequency Sweep Test

Factorial	Number of Levels	Values	Unit
Temperature	3	10, 20, 30	°C
Strain Amplitude	1	100	microstrain
Frequency Combination	1	0.01, 0.02, 0.05, 0.1, 0.2, 0.5, 1.0, 2.0, 5.0, 10.0, 15.0	Hz
		Number of Replicates	2
		Total Number of Specimens	6

The testing procedure used in the first two rounds was modified slightly for the third and fourth rounds. Instead of testing each specimen at only one temperature, as was done for the first two rounds, in the third and fourth rounds each specimen was tested at all three temperatures; and four specimens were used for each of these rounds rather than the single specimen used for the first two rounds. Reusing specimens for testing under multiple temperatures in the modified procedure is consistent with the dynamic modulus test (AASHTO T 342).

The frequency sweep test data were used to determine the stiffness master curve for the mix in each round.

3.1.1.2 Dynamic Modulus Test for Stiffness

The dynamic modulus test (AASHTO T 342) for stiffness was conducted at 40, 70, 100, and 130°F (4.4, 21.1, 37.8, 54.4°C), respectively, under the unconfined condition. The experiment design is shown in Table 3.3. Although the dynamic modulus test is a stress-controlled test, the target stress value is not a constant and it is adjusted for each specimen/temperature/frequency combination to yield a peak-to-peak strain amplitude of between 50 and 150 microstrain. The actual strain amplitude applied was between 50 and 150 microstrain, with higher strains used for low frequency tests and lower strains used for higher frequency tests.

The dynamic modulus test data were used to determine the stiffness master curve for the mix in each round. Due to the different boundary conditions and stress states, the stiffness master curves determined from dynamic modulus test data were expected to be different from those from the flexural frequency sweep test data.

Table 3.3: Experiment Design for Dynamic Modulus Test

Factorial	Number of Levels	Values	Unit
Temperature Combination	1	40, 70, 100, 130	°F
Confinement	1	unconfined	N/A
Frequency Combination	1	0.1, 0.5, 1.0, 5.0, 10.0, 25.0	Hz
Target Strain	1	Between 50 and 150	Microstrain
		Number of Replicates	3
		Total Number of Specimens	3

3.1.1.3 Flexural Fatigue Test

The flexural fatigue test (AASHTO T 321) was conducted at 68°F (20°C). The experiment design is shown in Table 3.4. Although the flexural fatigue test is conducted under strain control, the strain level is adjusted to yield a fatigue life near the target value. Typically the beams used for flexural frequency sweep tests are reused as spare specimens to run trial tests to help find the appropriate strain level. Given the large variation in fatigue life in typical fatigue tests, it is not possible or necessary to obtain the target fatigue life. The goal is to yield a fatigue life near the target values and provide well-spaced data for developing the following fatigue equation that relates fatigue life to tensile strain:

$$N_f = a \cdot \varepsilon^{b} \qquad \textbf{(1)}$$

where N_f is fatigue life defined in AASHTO T 321, ε is the strain level, and a, b are model parameters.

Fatigue life is determined in the current LLAP procedures as being the repetition at which the product of stiffness and number of repetitions changes from increasing to decreasing.

Table 3.4: Experiment Design for Flexural Fatigue Test

Factorial	Number of Levels	Values	Unit
Temperature	1	20	°C
Target Fatigue Life	3	0.25, 0.5, 1.0	Million
Frequency Combination	1	10.0	Hz
		Number of Replicates	3
		Total Number of Specimens	9

In addition to determining the N_f value for the fatigue equation, the stiffness reduction curves from each test (the damage relationship between load cycles and stiffness) were used to identify *CalME* fatigue damage model parameters for the mix in each round.

3.1.1.4 Repeated Simple Shear Test at Constant Height for Rutting

The repeated simple shear test at constant height (AASHTO T 320C) was conducted at 113°F (45°C) and 131°F (55°C) and at three shear stresses of 10.2, 14.5, and 18.9 psi (70, 100, and 130 kPa) respectively to produce input for *CalME,* in addition to providing information for mix design. The experiment design is shown in Table 3.5.

Table 3.5: Experiment Design for Repeated Simple Shear Test at Constant Height

Factorial	Number of Levels	Values	Unit
Temperature	2	45, 55	°C
Shear Stress	3	70, 100, 130	kPa
		Number of Replicates	3
		Total Number of Specimens	18

The permanent deformation accumulation curves (the relationship of permanent deformation versus load cycles) were used to identify *CalME* rutting model parameters for the mix in each round.

3.1.1.5 Repeated Load Triaxial Test for Rutting

The repeated load triaxial test for rutting (AASHTO TP 79, now T 378) was conducted at 113°F (45°C) and 131°F (55°C). The experiment design is shown in Table 3.6. The contact stress was 4.4 psi (30 kPa), while the deviatoric axial stress was 70 psi (483 kPa).

Table 3.6: Experiment Design for Repeated Triaxial Test

Factorial	Number of Levels	Values	Unit
Temperature	2	45, 55	°C
Confinement	2	0, 69	kPa
		Number of Replicates	3
		Total Number of Specimens	12

3.1.1.6 Hamburg Wheel-Track Testing for Moisture Susceptibility

The Hamburg Wheel-track Test for moisture susceptibility (AASHTO T 324) was conducted with a 122°F (50°C) water bath using 150 mm diameter cores. Four cores were used to conduct one test for each round.

3.1.2 Evaluation Procedure

For each round of performance evaluation, a series of tests was first conducted to check the mix against Superpave volumetric mix design specifications. Specifically, the following items were checked:

- AV@Ndesign: volumetric air-void content in specimens produced with the design number of gyrations
- VMA@Ndessign: voids in the mineral aggregate for specimens produced with the design number of gyrations
- AV@Nmax: volumetric air-void content in specimens produced with the maximum number of gyrations
- DP: dust proportion

According to Caltrans Standard Specifications *(1)*, Ndesign is always 85 while Nmax is always 130. Note that the adjusted mixes did not always need to meet the Superpave volumetric requirements listed above since the purpose of this study was to evaluate the effect of various adjustments on mix performance.

After the specimens were produced and tested according to the list in Table 3.1 to determine the performance of the adjusted mix, and volumetric changes were checked, the test results from all the finished rounds were compared in order to identify the target performance parameters to be improved in the next round. The mix design flowchart was then used to determine the mix adjustment for the next round.

3.2 Material Procurement and Preparation

3.2.1 Selection of Job Mix Formula

The UCPRC requested Superpave mix designs from several asphalt mixing plants. Two JMFs were obtained from two different plants. Both of them had dense gradations and used crushed alluvial aggregate from the floodplains at the foot of the Sierra Nevada for the virgin aggregate. One of the aggregates was less well crushed:

- JMF A reported that the coarse aggregates (retained on the #4 sieve) had 94 percent with two crushed faces, and 98 percent with one crushed face. JMF B on the other hand had 98 percent with two crushed faces and 100 percent with one crushed face for its aggregates.
- JMF A had 98 percent with at least one crushed face for the fine aggregates (passing #4 and retained on #8), compared to 100 percent for JMF B.

In general, with everything else equal, better crushing in aggregate leads to better mix performance. JMF A was therefore selected for this study to allow more room for improvement in mix performance. JMF A is a 1/2 inch (12.5 mm) maximum aggregate size mix with 25 percent RAP (percent of RAP aggregate by weight of the total aggregate) with PG 64-16 virgin binder. The gradation curve is shown in Figure 3.1 and the important properties of the mix are listed in Table 3.7.

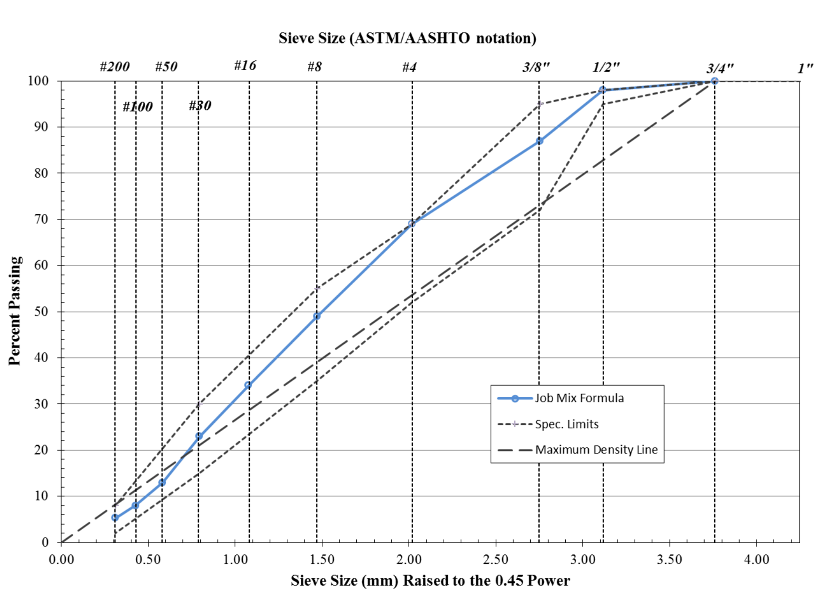

Figure 3.1: Gradation curve for the selected mix design.

Table 3.7: Property of the Asphalt Mix Selected for Evaluation

Property	Description
Binder grade	PG 64-16
Gradation type	Dense gradation
Aggregate rock type	Crushed alluvial
Nominal maximum aggregate size	1/2 in.
Virgin asphalt content (by total weight of aggregate)	4.38%
RAP asphalt content (by total weight of RAP)	3.92%
Total asphalt content (by total weight of aggregate)	5.4%
Total asphalt content (by total weight of mixture)	5.1%
RAP aggregate percentage (by total weight of aggregate)	25%

3.2.2 *Material Preparation*

Based on the tests planned, 800 kg (1,764 lbs) of loose mix were requested for each round of testing. Both virgin aggregates and RAP were received in barrels that each contained about 250 kg (500 lbs) of raw material. A total of 20 barrels of virgin aggregates and RAP were obtained from the mixing plant, which was located at the quarry where the aggregates were mined, based on the needs for the selected JMF. The barrels for each aggregate bin were combined in the laboratory and further split into 20 kg (50 lbs) portions at once and stored in five-gallon buckets for transportation and storage before any specimen production began. The detailed procedure for plant sampling and further splitting are described in Appendix A: Raw Material Reduction Procedure. Relevant ASTM standards were followed in order to achieve consistent aggregate gradations between different five-gallon buckets of the same bin size.

A decision was made to use bin-batching rather than sieve-batching if the procedure could be shown to be reliable. In the bin-batching procedure, aggregates from different bins and RAP were blended without first being sieved into individual sizes. Bin-batching matches the practice at the mixing plant and is less time consuming but can potentially lead to larger variation in the resulting aggregate gradation. To verify whether bin-batching in the laboratory was reliable, two samples were taken from two separate random buckets for each bin to determine the aggregate gradations and check for consistency. The comparison of laboratory-determined bin gradations and the JMF bin gradations are shown in Figure 3.2.

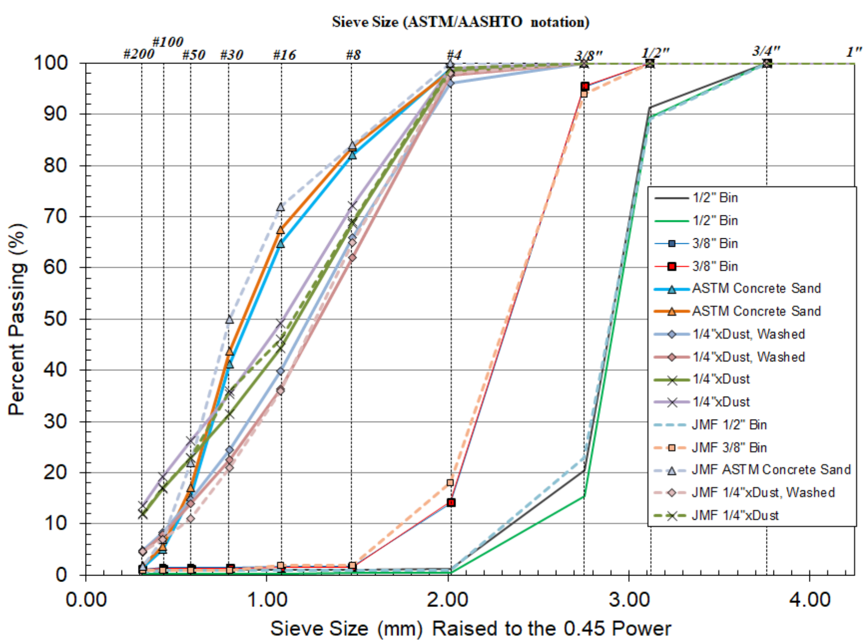

Figure 3.2: Comparison of bin gradations from two randomly selected buckets (for each bin) and the JMF.

It can be seen from Figure 3.2 that the gradations between the two samples for each aggregate bin were consistent. In particular:

- The maximum difference in percent passing was 5 percent for the 1/4"×dust bin at sieve size #16 (1.18 mm).
- The average gradations for different bin sizes were in general the same as those provided by the plant (i.e., the JMF). The maximum difference in percent passing was 7.5 percent for the ASTM concrete sand at sieve size #30 (0.6 mm).

To check the consistency of the RAP, three randomly selected buckets were sampled for binder content determination using the ignition oven method (AASHTO T 308), and the RAP aggregate gradations were checked using the burned residue. Two ignition oven tests were conducted for each bucket. Averaged sieve analysis results for the residue aggregates are shown in Figure 3.3, which indicates that the RAP aggregate gradations were quite uniform but it was also clear that they were slightly more coarse than the RAP gradation assumed in the JMF. The binder content from the ignition oven tests before correcting for burned fines had an average of 5.2 percent and a standard deviation of 0.14 percent, which indicates that the RAP binder contents were quite consistent. Note that the binder content from the ignition oven test was higher than the 3.92 percent RAP binder content used in the JMF, an expected result because some fine aggregates may have been burned during ignition oven tests.

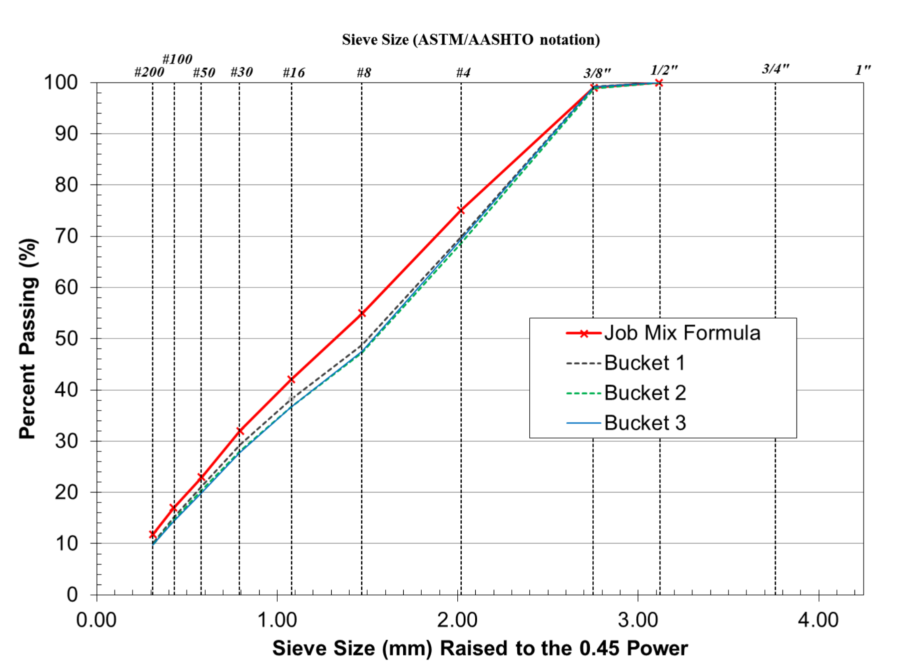

Figure 3.3: Comparison of ignition oven residue aggregate gradations for three random-sampled RAP buckets.

If both the virgin aggregates and the RAP were to be blended using the batching formula in the JMF, the blended aggregate gradation was practically the same as the one in the JMF (see Figure 3.4) and well within the tolerance limit. Based on the findings noted above, it was determined that bin-batching could produce consistent aggregate gradations and that the batching formula in the JMF could be used without modification.

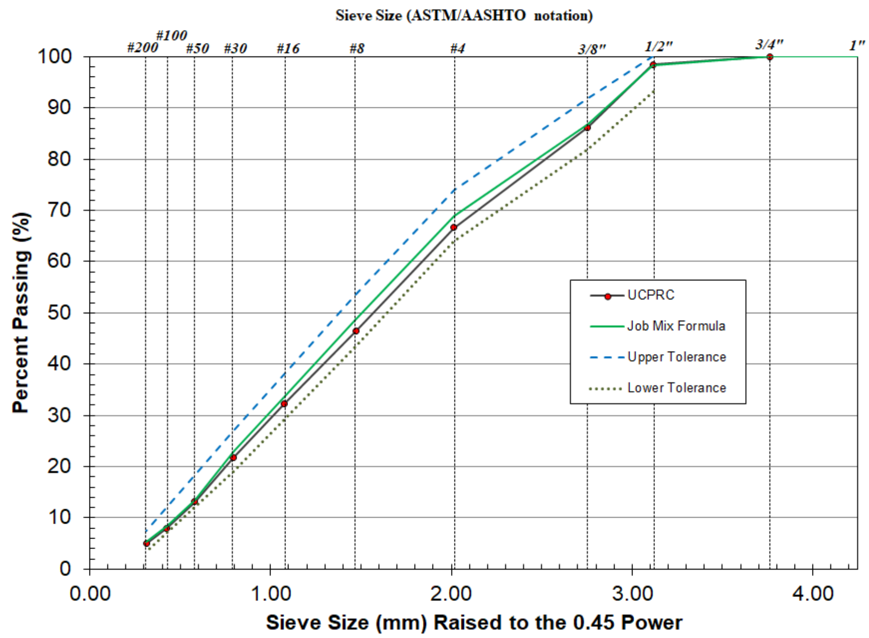

Figure 3.4: Comparison of blended aggregate gradation between the bin-batched UCPRC material and that of the selected JMF.

Aggregates were blended in batches with each batch used for one compaction. Standard procedures were followed to further split the aggregates to reach the exact amount needed to make each set of batched of hot mix asphalt (HMA) specimens. An extra batch was prepared for every 15 batches for the sole purpose of checking the gradation of the combined virgin aggregate gradation, the binder content of the RAP by using the ignition oven method, and the gradation of the RAP aggregates. The batching procedure is described in detail in Appendix B: Batching Procedure. The mixing procedures are detailed in Appendix C: Mixing Procedure.

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3.2.3 Job Mix Formula Verification for Superpave Volumetric Requirements

Before specimens were produced for Round One, tests were conducted to verify the job mix formula with respect to the Superpave volumetric requirements. Specifically, the air-void content (Va) should be between 2.5 and 5.5 percent at Ndesign gyrations and higher than 2.0 percent at Nmax gyrations. The VMA (percent voids in the mineral aggregate) should be between 14.5 and 17.5 percent at Ndesign, while the effective dust proportion (DPe) should be between 0.6 and 1.3 *(1)* at Ndesign.

Based on two series of gyratory compactions, the average air-void content was 6.3 percent at Ndesign and 5.2 percent at Nmax, the average VMA was 17.4 percent at Ndesign, and the average DPe was 1.15 percent at Ndesign. While VMA and DPe barely met the specifications at Ndesign, Va met the specification at Nmax, and the Va at Ndesign was higher than Caltrans limit.

Two additional binder contents were tried to determine how much of an increase was needed to meet all the specifications. Specifically, the binder contents (by total weight of mixture) were increased from 5.1 percent to 5.6 percent and 6.0 percent, respectively. The effects of binder content on various Superpave volumetrics are shown in Figure 3.5 to Figure 3.8. As shown in Figure 3.5, the mix will be able to meet all the Caltrans Superpave specifications if the binder content is increased by 0.3 percent to 5.4 percent. Since this increase is less than the +0.50 percent tolerance specified in the Caltrans Standard Specification for binder content, it was decided that the JMF could be used without modification for Round One testing.

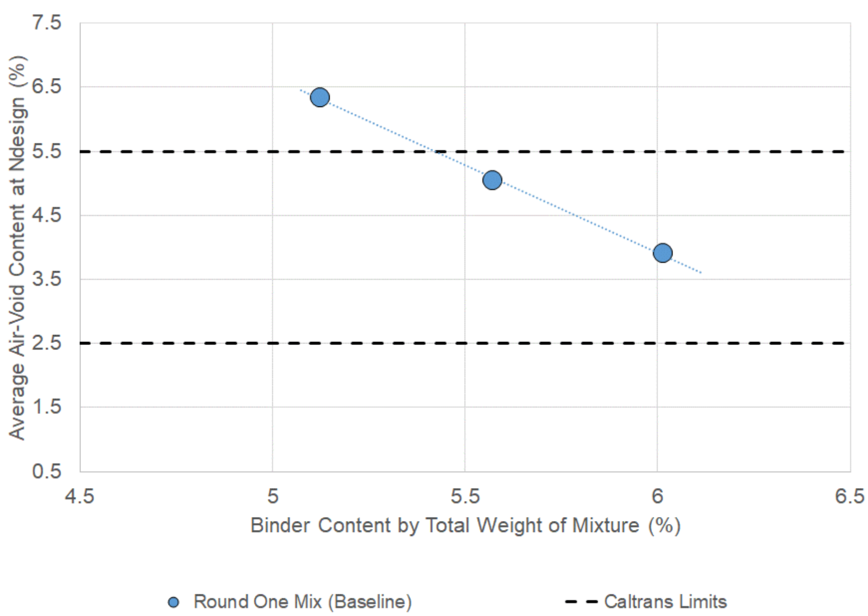


Figure 3.5: Variation of air-void content at Ndesign gyrations with binder content.

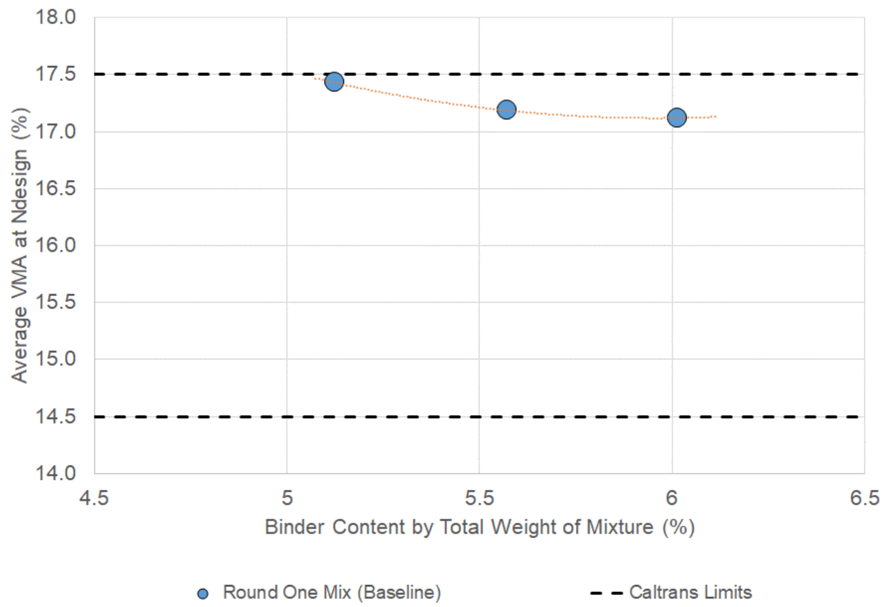


Figure 3.6: Variation of VMA at Ndesign gyrations with binder content.

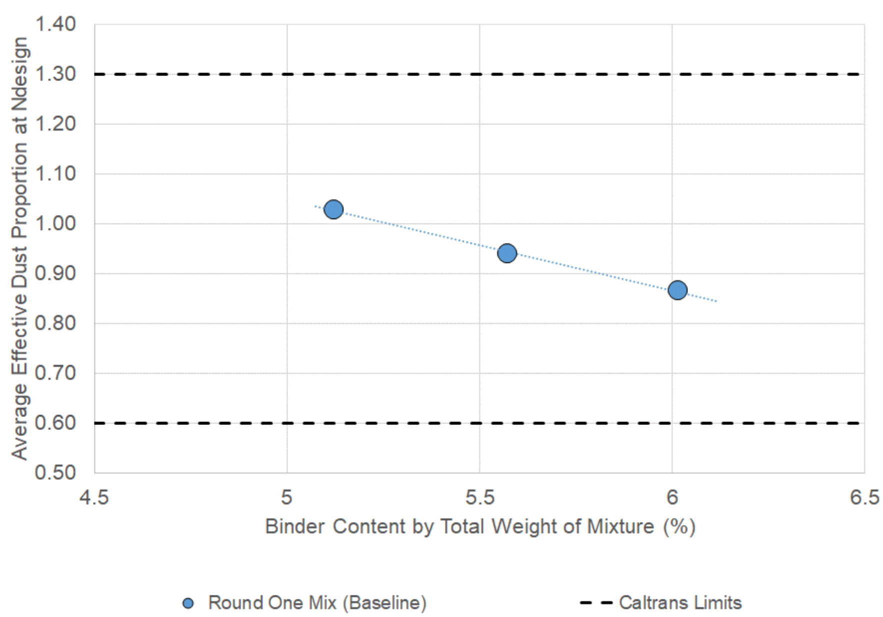


Figure 3.7: Variation of effective dust proportion at Ndesign gyrations with binder content.

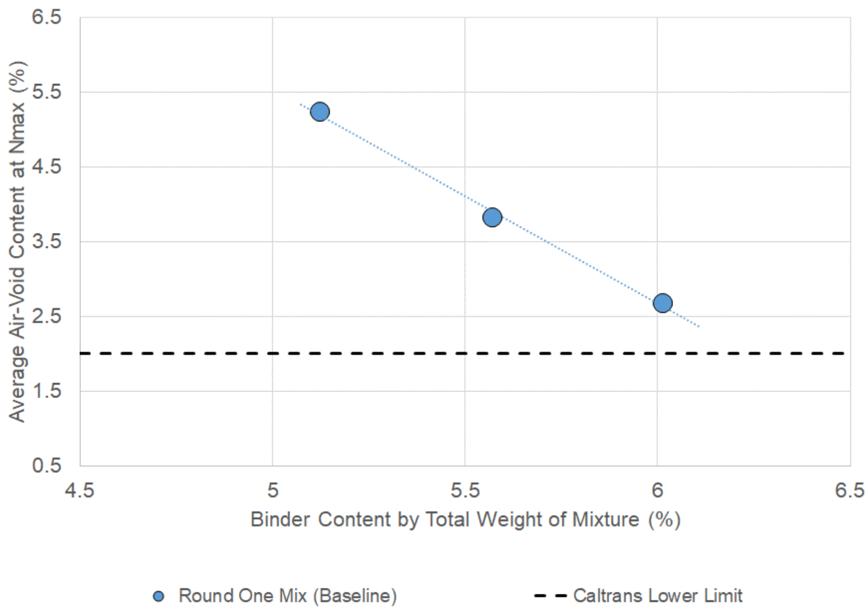


Figure 3.8: Variation of air-void content at Nmax gyrations with binder content.

4 ROUND ONE RESULTS—BASELINE

4.1 Aggregate Gradations

The gradations of blended virgin aggregates as determined from quality control (QC) checks of the Round One (i.e., baseline) batches are shown in Figure 4.1. As shown in the figure, the gradations for the blended virgin aggregates were practically the same throughout the Round One testing.

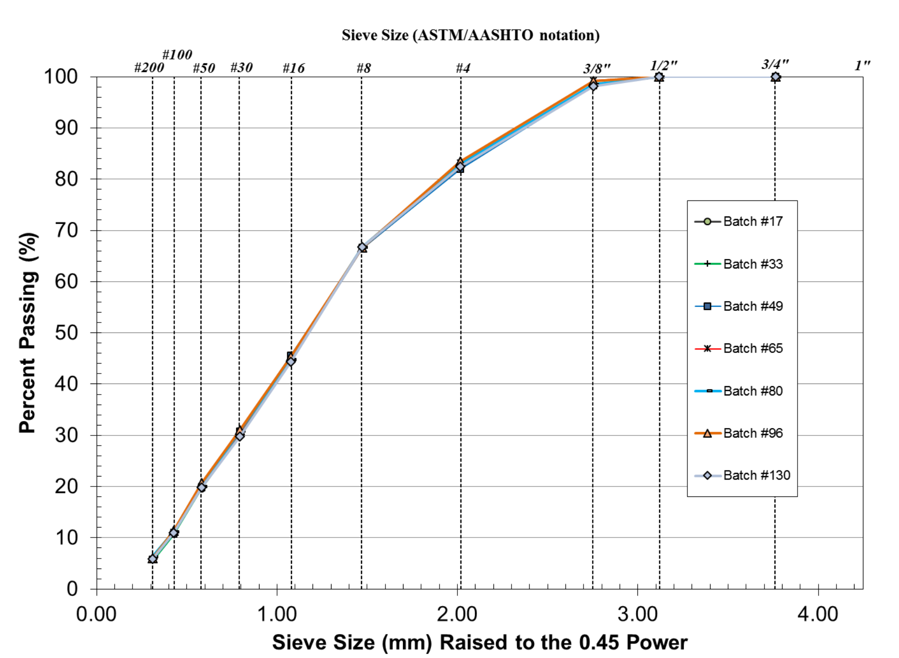


Figure 4.1: Comparison of blended virgin aggregate gradations for QC batches during Round One.

The gradations of ignition oven residue for RAP aggregates for Round One QC batches are shown in Figure 4.2. As shown in the figure, the RAP aggregate gradations only varied slightly (less than 5 percent) throughout Round One.

For the QC samples, the blended aggregate gradation was determined by combining the average gradations for virgin aggregates and residual RAP aggregates after ignition oven testing. The comparison of Round One blended aggregate gradations with the JMF target gradation is shown in Figure 4.3. As shown in the figure, the blended aggregate gradation is very similar to the JMF gradation and well within the tolerance limit.

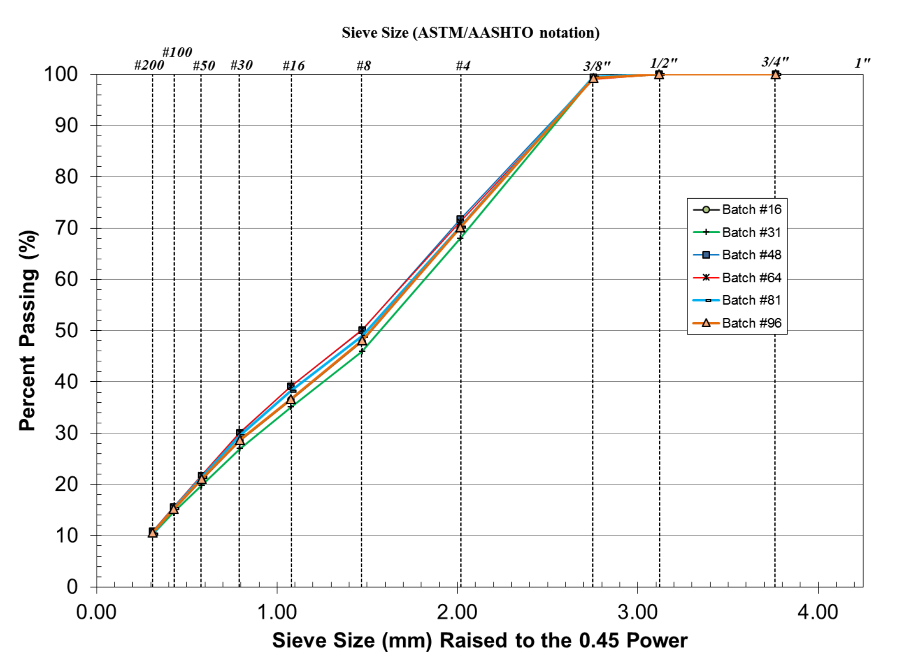


Figure 4.2: Comparison of ignition oven residue RAP aggregate gradations for QC batches during Round One.

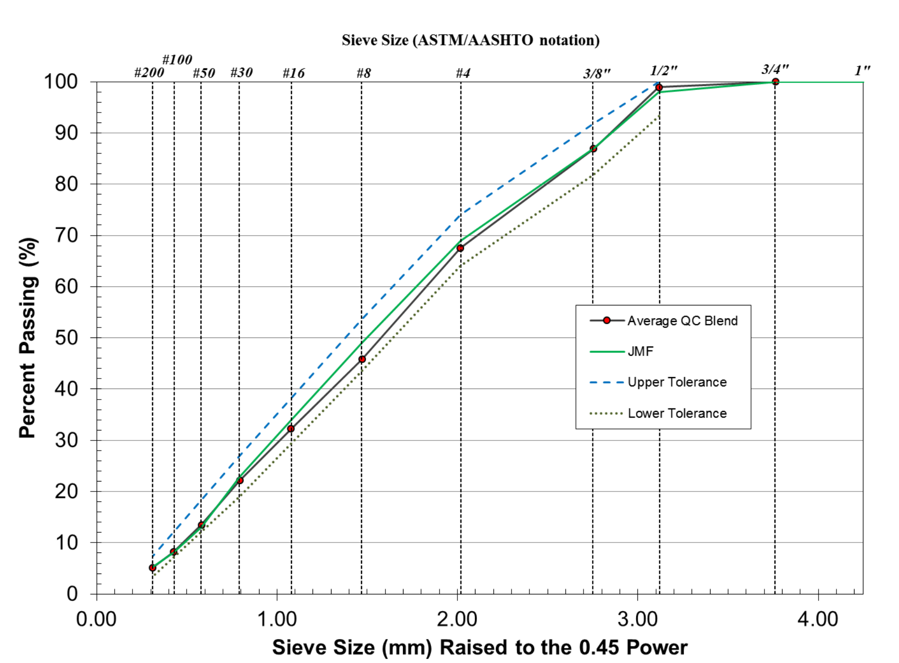


Figure 4.3: Comparison of blended Round One aggregate gradation with JMF.

4.2 Mix Performance

As discussed in Section 3.1, the mix used in the main structural layer for the Interstate 5 long-life project near Red Bluff *(6)* in 2012 was selected as a benchmark mix for putting the performance of the baseline mix (i.e., test results for the Round One mix) into perspective. Setting that benchmark level of performance was intended to help determine whether the Round One mix needs to be improved in fatigue resistance, rutting resistance, or both.

4.2.1 Stiffness

The flexural stiffness master curve for the Round One (baseline) mix is shown in Figure 4.4, which also includes the flexural stiffness master curves for the Red Bluff (benchmark) mix and the dynamic modulus master curve for the Round One mix. As shown in the figure, the Round One mix is slightly stiffer than the benchmark mix at typical pavement temperatures (20 to 40°C [68 to 104°F]) in the middle of a thick asphalt pavement but slightly softer than the benchmark mix at temperatures below 0°C (32°F). The dynamic modulus is about two times higher than the flexural stiffness overall. Dynamic modulus stiffnesses are generally greater than those from flexural stiffness testing because the dynamic modulus is measured in compression compared with flexural stiffness which is in compression and tension.

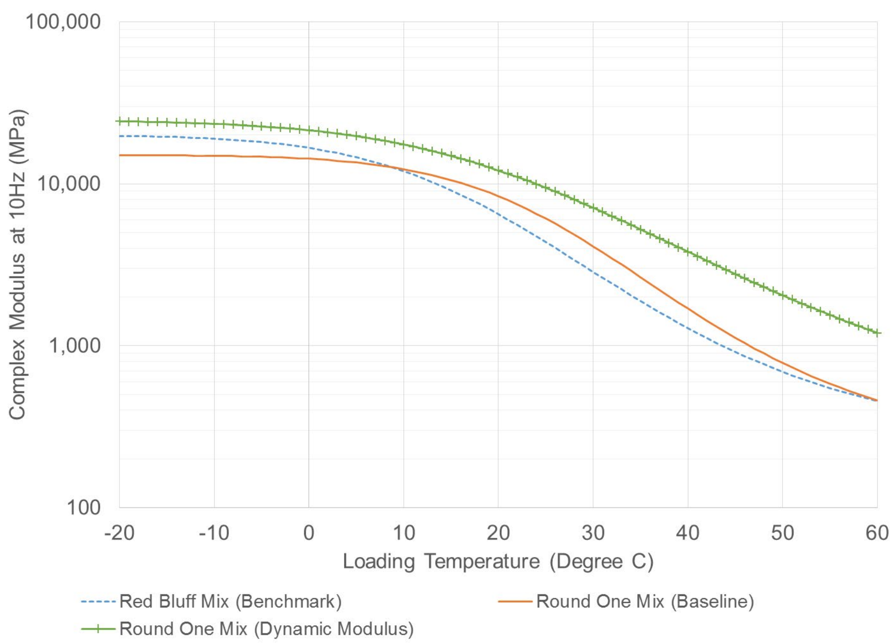


Figure 4.4: Comparison of stiffness master curves between Round One and the benchmark mix.

4.2.2 *Fatigue Life*

The variation of fatigue life with strain level for the Round One mix is shown in Figure 4.5, along with the fatigue life for the benchmark mix. As shown in the figure, the fatigue lives for the Round One mix (baseline) are about one third of those for the Red Bluff (benchmark) mix across the strain levels.

Figure 4.5: Comparison of fatigue life between Round One mix and the Red Bluff mix.

4.2.3 *Permanent Deformation*

4.2.3.1 Repeated Simple Shear Test at Constant Height

Comparisons of the permanent shear strain accumulation curves between the Round One and benchmark mixes at 113 and 131°F (45 and 55°C) are shown in Figure 4.6 and Figure 4.7, respectively. As shown in the figures, the Round One mix is less resistant to permanent deformation than the benchmark mix. Under 113°F (45°C) and 14.5 psi (100 kPa) shear stress, the Round One mix accumulated about 2.5 times as much permanent deformation as the benchmark mix at a given number of load repetitions. Under 131°F (55°C) and 14.5 psi (100 kPa) shear stress, the Round One mix accumulated about four times as much permanent deformation as the benchmark mix.

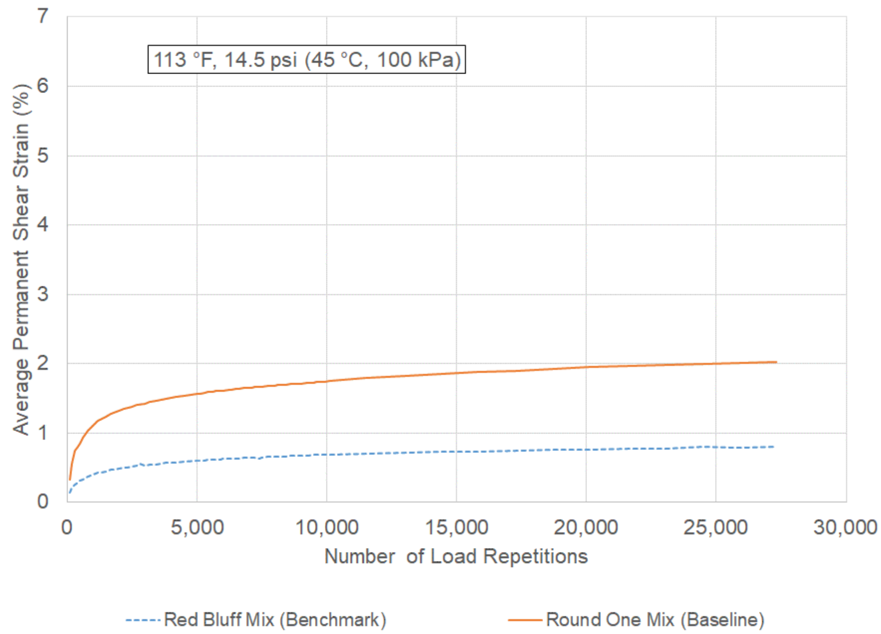


Figure 4.6: Comparison of permanent shear strain accumulation curves between Round One and Red Bluff mix tested at 113°F (45°C) and 14.5 psi (100 kPa) shear stress.

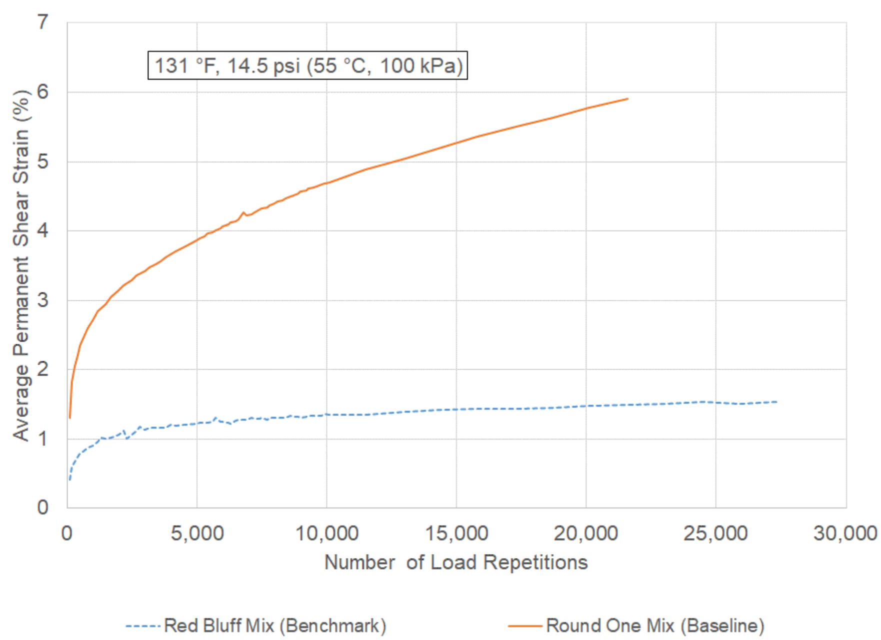


Figure 4.7: Comparison of permanent shear strain accumulation curves between Round One and Red Bluff mix tested at 131°F (55°C) and 14.5 psi (100 kPa) shear stress.

4.2.3.2 Repeated Load Triaxial Test

Two examples of average permanent axial strain accumulation curves for the Round One mix are shown in Figure 4.8 and Figure 4.9, respectively. No benchmark test data are available for comparison for this case.

According to the figures, a threshold of 3 percent permanent axial strain can be used as a reasonable criterion for defining the failure limit in repeated load triaxial (RLT) tests because the rate of permanent deformation when the threshold is reached is significantly higher than those at the earlier stages of the test. The figures show that the Round One mix can reach failure in less than the 20,000 maximum cycles when tested under no confinement, but will likely never reach the failure limit if tested under 69 kPa of confinement.

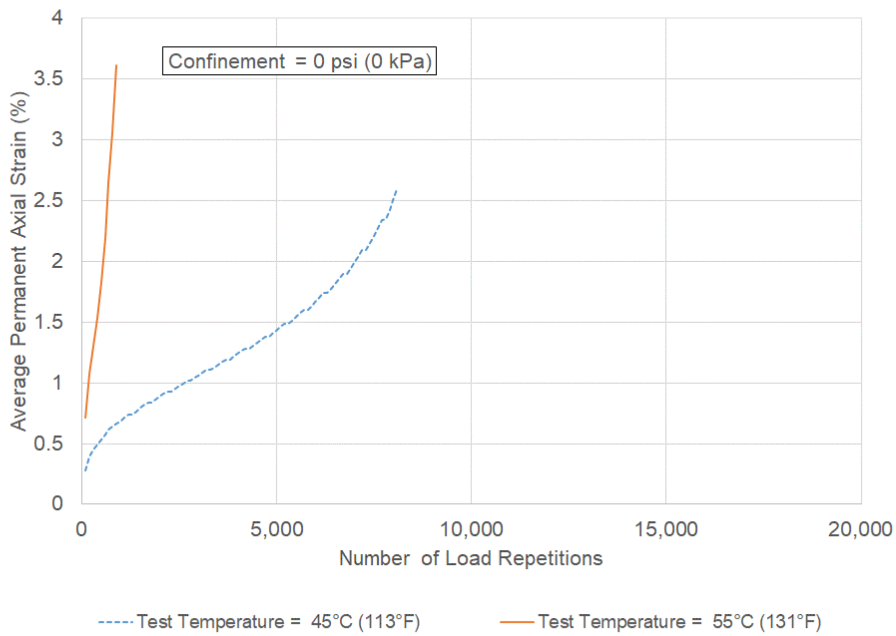


Figure 4.8: Permanent axial strain accumulation curves for Round One mix tested with no confinement.

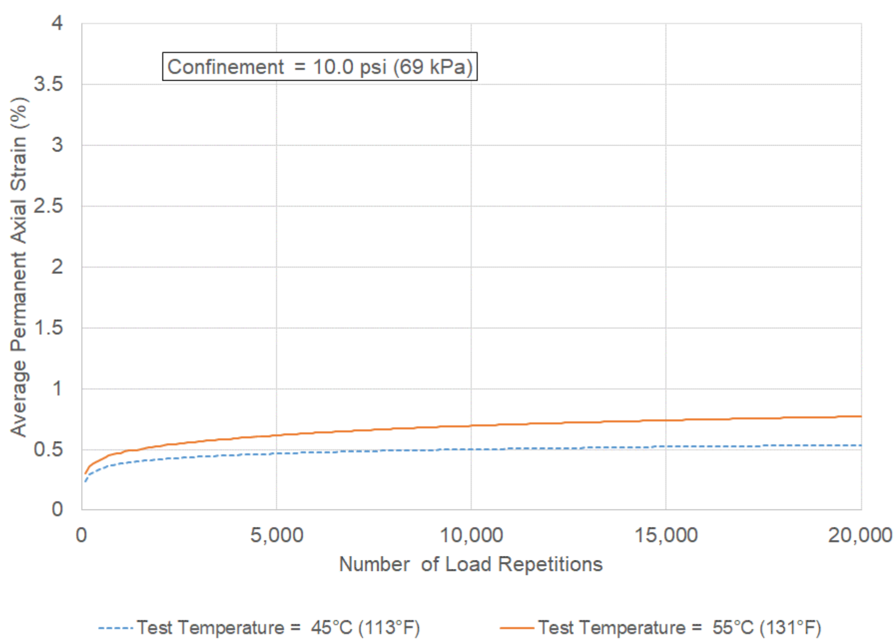


Figure 4.9: Permanent axial strain accumulation curves for Round One mix tested with 69 kPa confinement.

4.2.4 Moisture Susceptibility

The average rut accumulation curve from each side of the test specimen for the Round One mix is shown in Figure 4.10. There are no benchmark test data available for comparison in this case. According to the figure, the Round One mix has very little rutting when tested in the 122°F (50°C) water bath. This satisfies the Caltrans requirement for mixes with PG 64 binder by a wide margin (that is, no more than 12.7 mm [0.5 inches] of rut after 15,000 load repetitions). This means the mix is not susceptible to moisture damage.

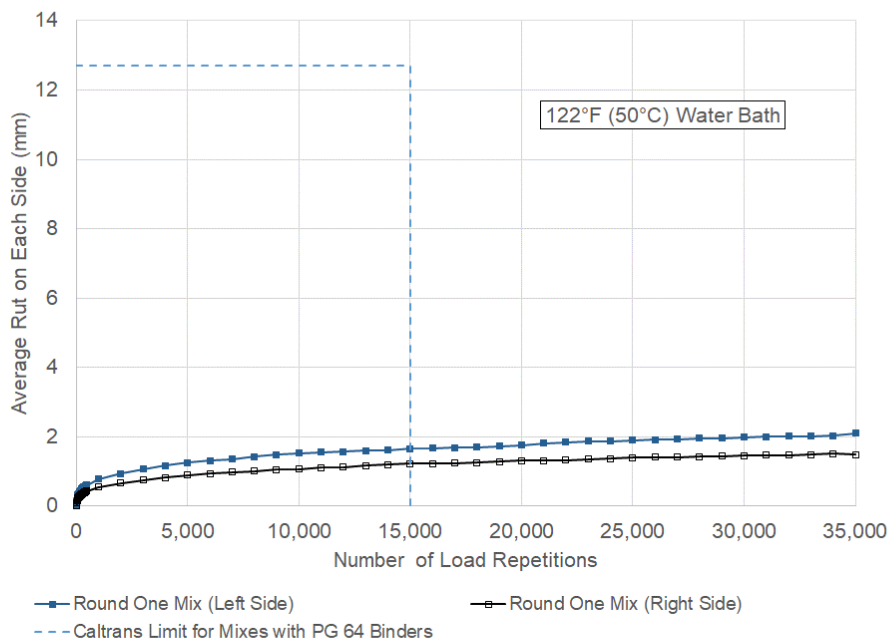


Figure 4.10: Accumulation of average rut from both left and right sides in Hamburg Wheel-track Testing for the Round One mix.

4.3 Performance Summary

As shown above, compared to the Red Bluff mix (benchmark), the Round One (baseline) mix is stiffer under typical pavement temperatures (68 to 104°F [20 to 40°C]), has a much shorter fatigue life, and has much worse rutting performance at 131°F (55°C). And it is not susceptible to moisture damage. Both fatigue and rutting performance for this mix would need to improve to match the performance of Red Bluff mix.

4.4 Selection of Mix Adjustment for Round Two

As mentioned above, the Round One mix (baseline) can be improved in terms of both fatigue and rutting performance. In order to select a strategy for adjusting the mix design for Round Two, the first four options shown in Figure 2.1 will be tried first because they are the more certain and less costly options.

4.4.1 Is the Gradation Close to the 0.45 Power Line?

According to the 2010 version of Caltrans Test 202, the nominal maximum aggregate size (NMAS) is one sieve size larger than the first size to retain more than 10 percent of the aggregates. The nominal maximum aggregate size of the mix is 1/2 inch since it is one size larger than the 3/8 inch sieve that retained more than 10 percent aggregates. The 0.45 power line should pass through 100 percent at the maximum aggregate size of 3/4 inch (i.e., one size larger than the NMAS). In Figure 4.11 the gradation is shown compared to the 0.45 power line. The plot indicates that the mix is in general on the fine side and can be moved closer to the 0.45 power line especially for the sieve sizes #16 to 3/8 inch (1.18 to 9.5 mm).

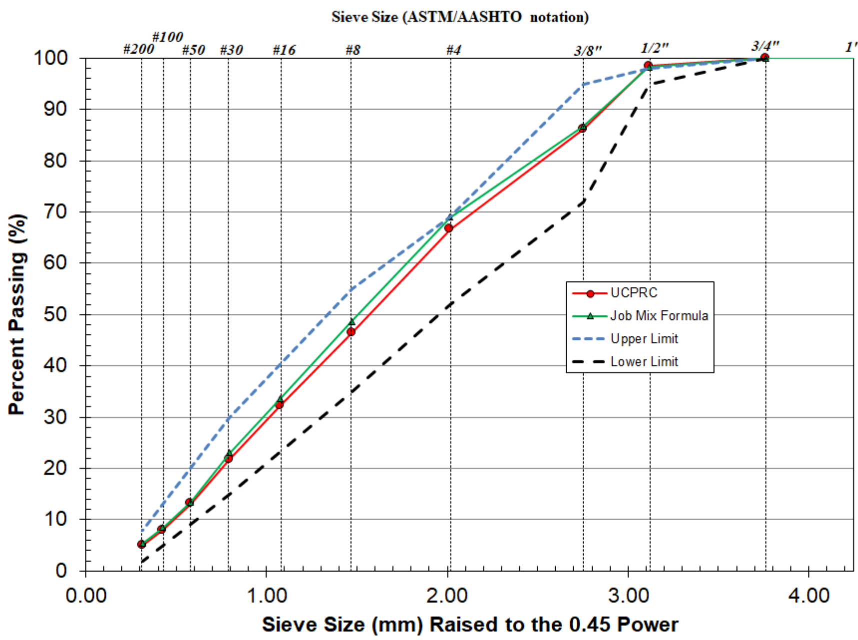
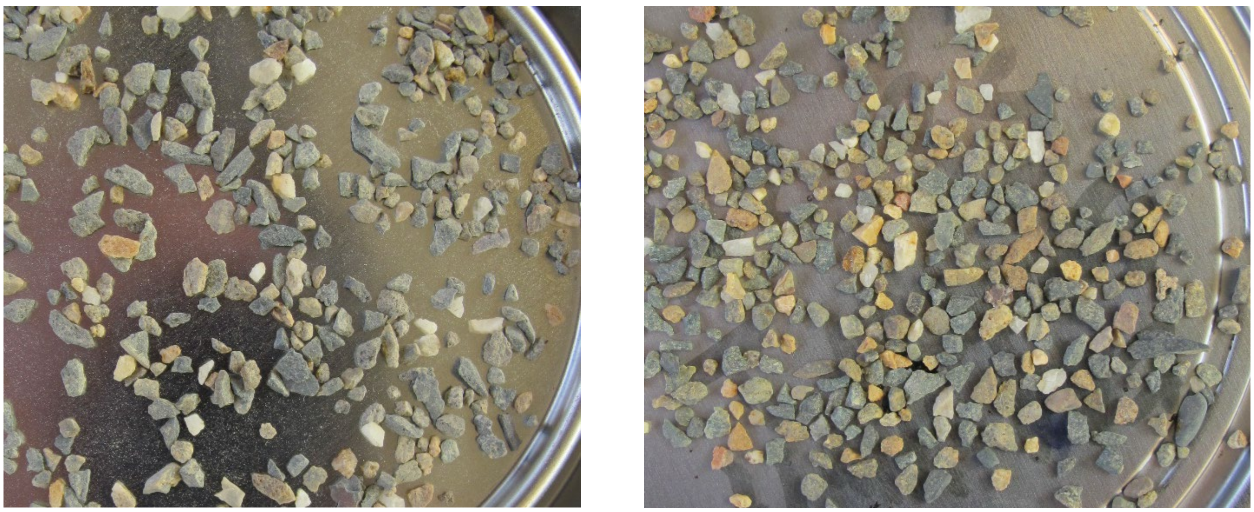


Figure 4.11: Aggregate gradation for the Round One mix.

4.4.2 Is the Sand Well Crushed?

For this research, *sand* is defined as the aggregates retained on the #8 to #30 sieves. The three bins that contain significant amounts of sand are ASTM concrete sand, 1/4"×dust, and manufactured sand. To determine whether the sand was well crushed, aggregates from these three bins were sieved into individual sizes. The sand portion of the aggregates were then visually inspected one sieve size at a time to determine whether they were well crushed. The photographs in Figure 4.12 show two examples of the sand aggregates inspected.

The sand from the 1/4"×dust and manufactured sand bins were found to be very well crushed while sand from the ASTM concrete sand bin was slightly less crushed. Overall, the sand aggregates are regarded as being well crushed.



(a) #16 sieve from the 1/4"×dust bin **(b) #16 sieve from ASTM concrete sand bin**

Figure 4.12: Example pictures of sand retaining on individual sieve size.

4.4.3 Is the Dust Proportion Too High or Too Low?

The Caltrans Standard Specifications *(1)* require that the dust proportion fall within the range of 2 to 7 percent, and in this case a value of 5 percent aggregate passing the #200 sieve was attained, putting the proportion right at the specification's midpoint. In addition, the effective dust proportion is 1.04 (see Figure 3.7), which is at about the middle of the 0.6 to 1.3 range specified by Caltrans, so it is neither too high nor too low.

4.4.4 Are the Coarse Aggregates Well Crushed?

For this research, coarse aggregate is defined as those particles retained on the #4 sieve and above. The original coarse aggregates had 98 percent with at least one fractured face and 94 percent with at least two fractured faces. This is reasonably well crushed but, as discussed in Section 3.2.1, there is another source of alluvial aggregate that is better crushed.

4.4.5 Decision on Strategy for Round Two

After evaluating the first four options shown in Figure 2.1, the ways to improve both the fatigue and rutting performance of the baseline mix include using a denser aggregate gradation or using better-crushed coarse aggregate. Even though it can increase rutting performance, reducing the effective dust proportion was not considered because it would worsen fatigue performance.

Changing gradation can be achieved easily in a plant by changing the aggregate batching proportions, although this may lead to excess in some aggregate bins in terms of overall plant aggregate production. Achieving better crushing is more expensive than changing gradation and may not be practical because of the potential need for new equipment. Therefore, for Round Two it was decided to use a denser gradation and to evaluate its effect on mix performance.

Specifically, a decision was made to move the gradation curve to halfway between the current gradation and the 0.45 power line for the sieve sizes above (including) #8 (see Figure 4.13). The desired gradation is listed in Table 4.1. However, the desired gradation was not achievable with the given aggregate bins as there was not enough coarse aggregate in the bins to have 9 percent aggregate remaining on the 1/2 inch sieve. The final target gradation used for the Round Two mix is shown in Table 4.1 and in Figure 4.13, which also shows the actual gradation from first QC results for the Round Two mix.

Table 4.1: Comparison of Gradations for Round One and Round Two Mixes

Sieve Name	Sieve Opening (mm)	Gradation Following 0.45 Power Line (% Passing)	Round One Gradation (% Passing)	Round Two Desired Gradation (% Passing)	Round Two Final Target Gradation (% Passing)	Round 2 First QC Gradation (% Passing)
3/4"	19	100	100	100	100	100
1/2"	12.5	83	99	91	97	99
3/8"	9.5	73	86	80	79	79
#4	4.75	54	67	60	60	58
#8	2.36	39	46	42	43	41
#16	1.18	29	32	32	31	30
#30	0.6	21	22	22	22	21
#50	0.3	15	13	13	13	13
#100	0.15	11	8	8	8	8
#200	0.075	8	5	5	5	5
Pan (-#200)	0	0	0	0	0	0

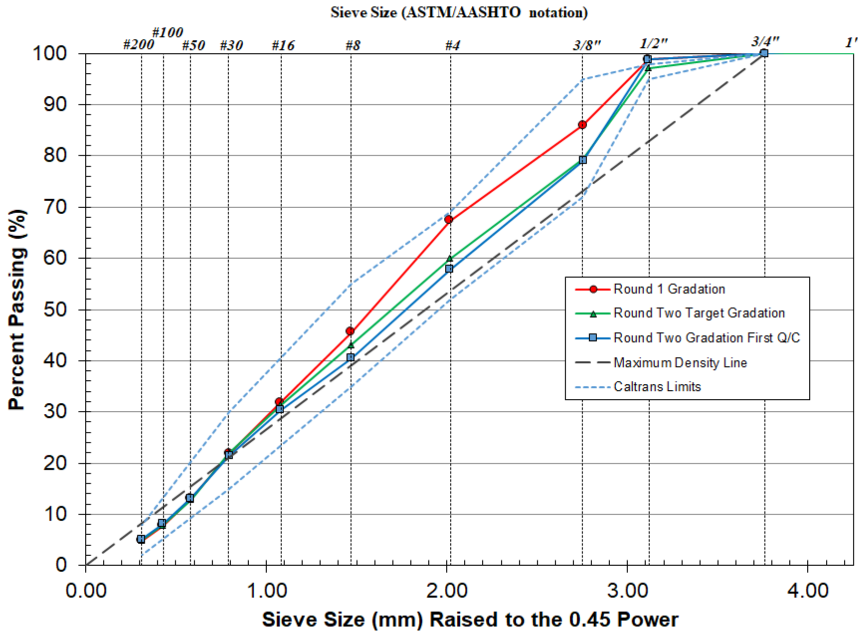


Figure 4.13: Comparison of aggregate gradations for Round One and Round Two mixes.

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5 ROUND TWO RESULTS—DENSER GRADATION

5.1 Superpave Volumetric Verification

The Superpave volumetrics for the Round Two mix (denser gradation) are shown in Figure 5.1 to Figure 5.4, along with results for the Round One mix (baseline). As shown in these figures, the Round Two mix met all the Caltrans requirements for Superpave volumetric indexes. Compared to the Round One mix, the denser gradation used in Round Two mix led to lower air-void contents and VMA, while it also maintained the same effective dust proportion.

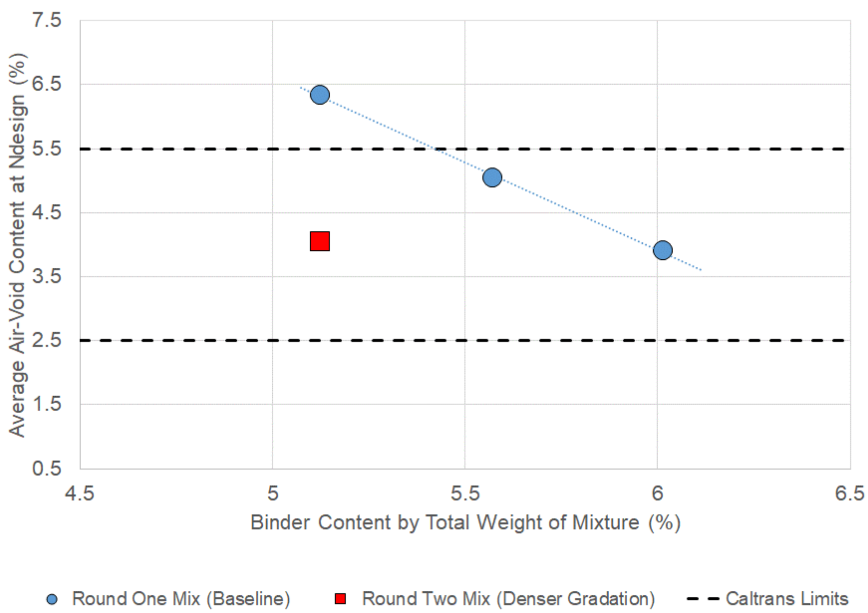

Figure 5.1: Air-void content at Ndesign gyrations for the Round One and Round Two mixes.

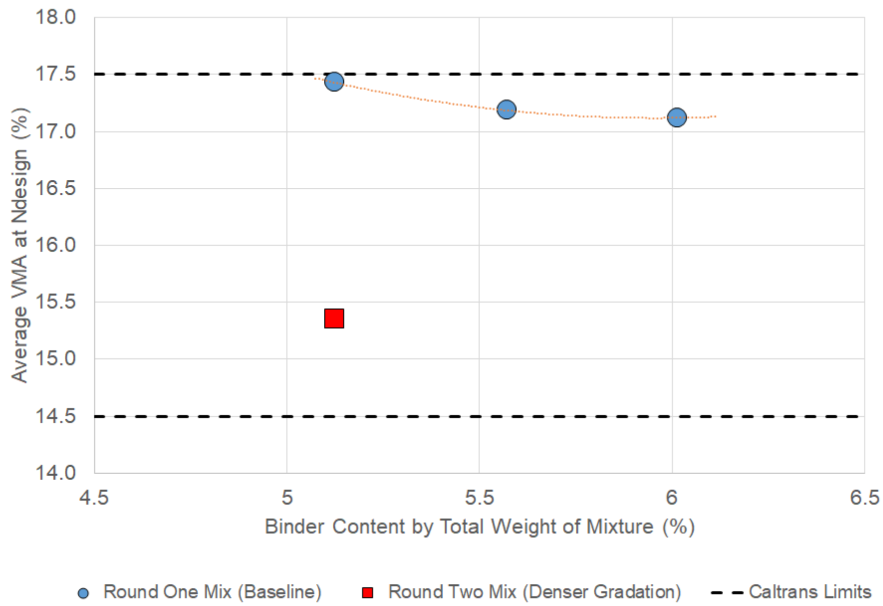

Figure 5.2: VMA at Ndesign gyrations for the Round One and Round Two mixes.

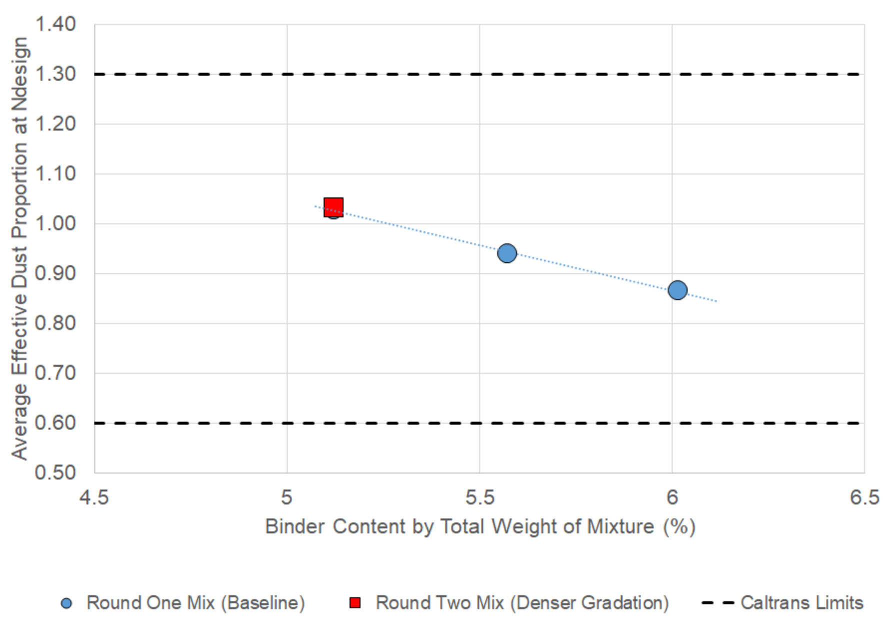

Figure 5.3: Effective dust proportion at Ndesign gyrations for the Round One and Round Two mixes.

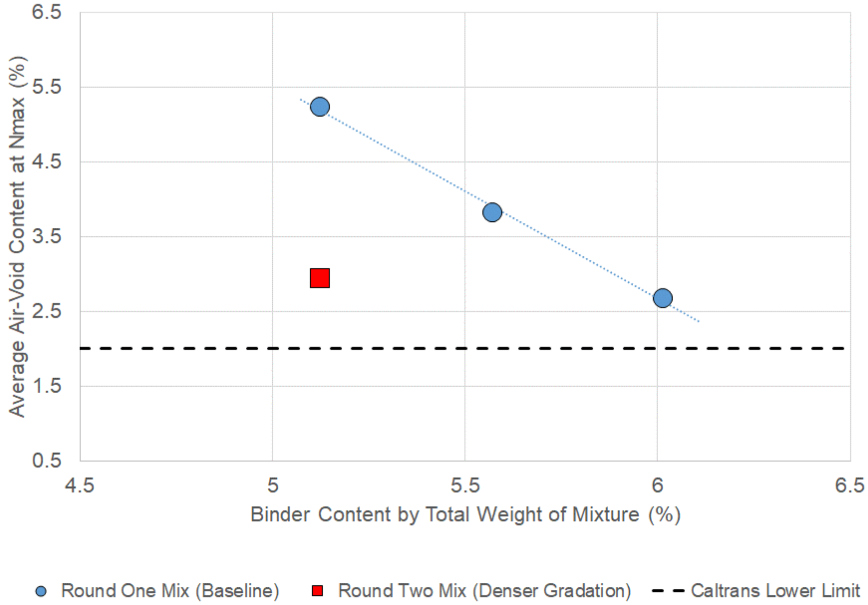

Figure 5.4: Air-void content at Nmax gyrations for the Round One and Round Two mixes.

5.2 Mix Performance

5.2.1 Stiffness

A comparison of the flexural stiffness master curves for the Round One (baseline), Round Two (denser gradation) and the Red Bluff (benchmark) mixes is shown in Figure 5.5. A comparison of the dynamic modulus master curves for the Round One and Round Two mixes is shown in Figure 5.6. As shown in these figures, the Round Two and Round One mixes have practically the same stiffness both in terms of flexural stiffness and dynamic modulus.

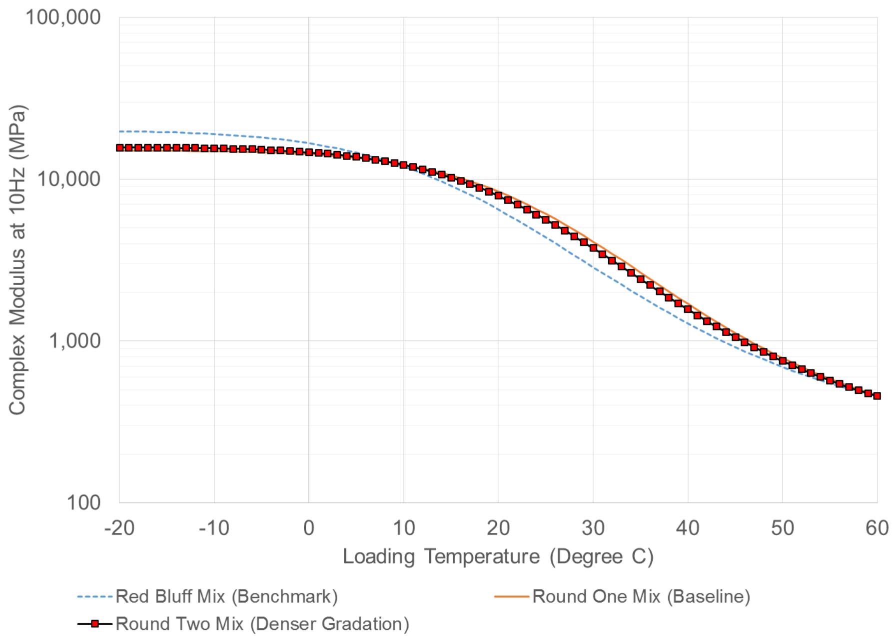

Figure 5.5: Comparison of the flexural stiffness master curves of the Round One, Round Two, and Red Bluff mixes.

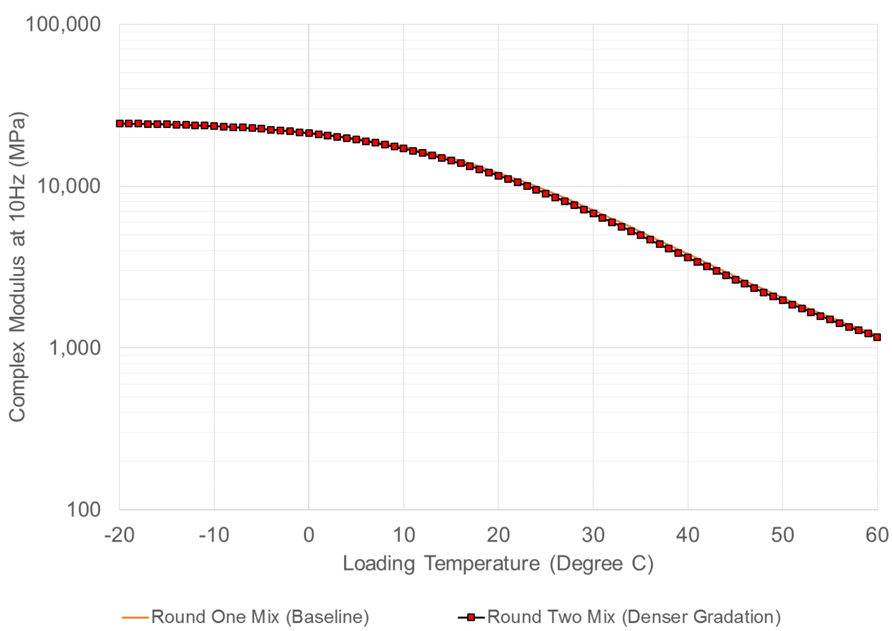

Figure 5.6: Comparison of the dynamic modulus master curves of the Round One and Round Two mixes.

5.2.2 *Fatigue Life*

The variation of fatigue life with strain level for the Round Two mix (denser gradation) is shown in Figure 5.7, along with the fatigue life for the Round One mix (baseline) and the Red Bluff mix (benchmark). As shown in the figure, the fatigue life for the Round Two mix is roughly three times longer at a given tensile strain than the Round One mix, and is very similar to the Red Bluff mix.

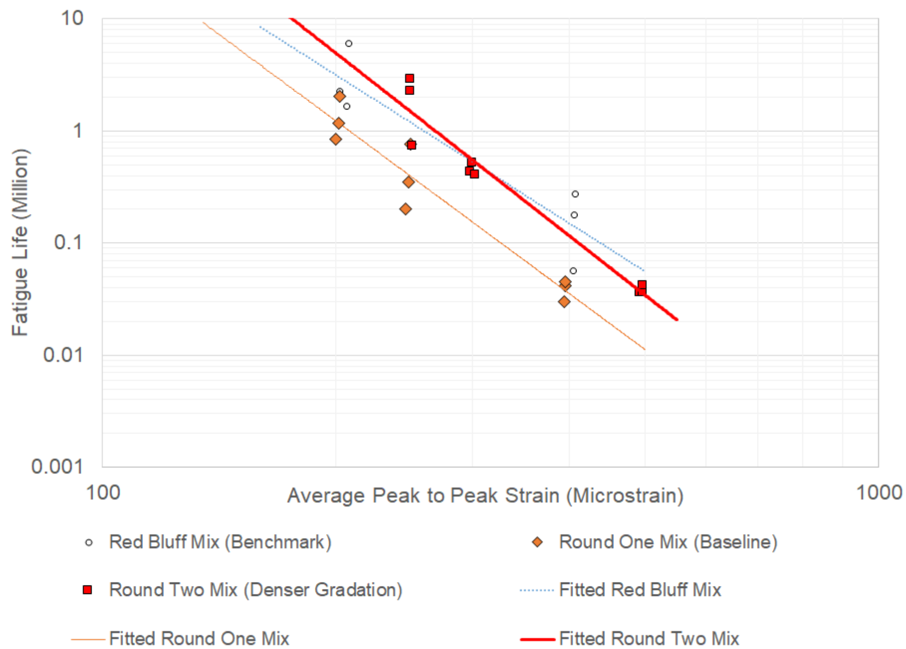

Figure 5.7: Comparison of the fatigue life of the Round One, Round Two, and Red Bluff mixes.

5.2.3 *Permanent Deformation*

5.2.3.1 Repeated Simple Shear Test at Constant Height

Comparisons of the permanent shear strain accumulation curves between mixes from the first two rounds and the benchmark mix are shown in Figure 5.8 and Figure 5.9 for the 113 and 131°F (45 and 55°C) test temperatures, respectively. As shown in the figures, the difference between the Round Two and Round One mixes is not consistent. Under 113°F (45°C) and 14.5 psi (100 kPa) shear stress, the Round Two mix accumulates about 10 percent more permanent deformation than the Round One mix, but under 131°F (55°C) and 14.5 psi (100 kPa) shear stress, the Round Two mix accumulates about 25 percent less permanent deformation than the Round One mix. Compared to the Red Bluff (benchmark) mix however, the Round Two mix clearly had considerably worse performance.

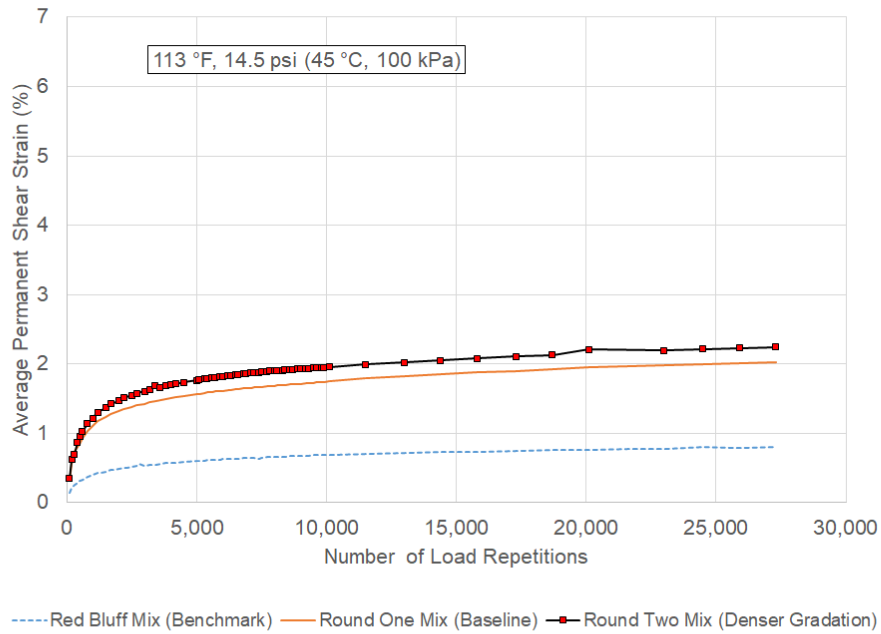

Figure 5.8: Permanent shear strain accumulation curves for the Round One, Round Two, and Red Bluff mixes tested at 113°F (45°C) and 14.5 psi (100 kPa) shear stress.

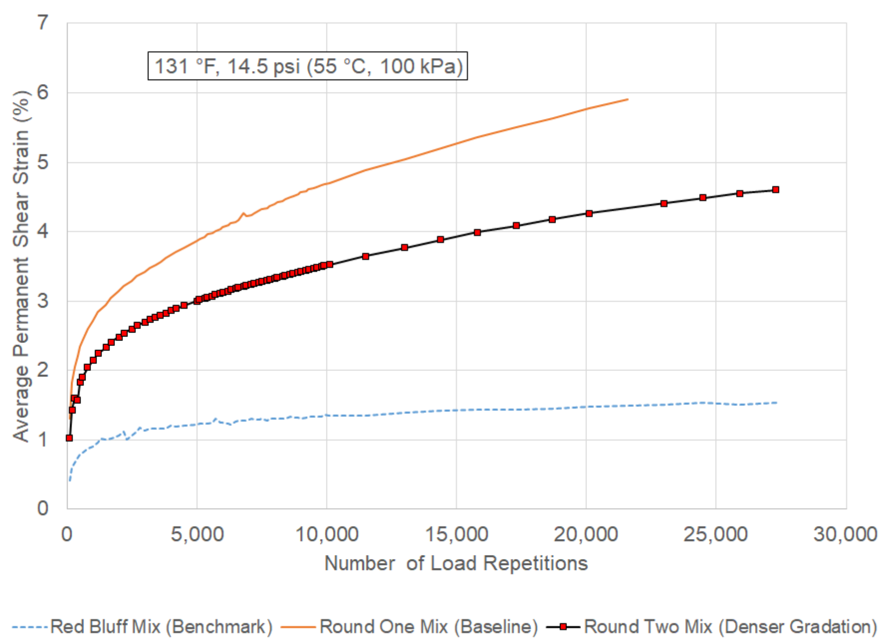

Figure 5.9: Permanent shear strain accumulation curves for the Round One, Round Two, and Red Bluff mixes tested at 131°F (55°C) and 14.5 psi (100 kPa) shear stress.

5.2.3.2 Repeated Load Triaxial Test

Comparisons of average permanent axial strain accumulation curves between the Round One and Round Two mixes are shown in Figure 5.10 and Figure 5.11 for the test temperatures 113 and 131°F (45 and 55°C), respectively. Both figures show that the Round Two mix is less resistant to permanent deformation than the Round One mix in RLT testing. This was an unexpected finding, and was not consistent with the results from the RSST-CH testing.

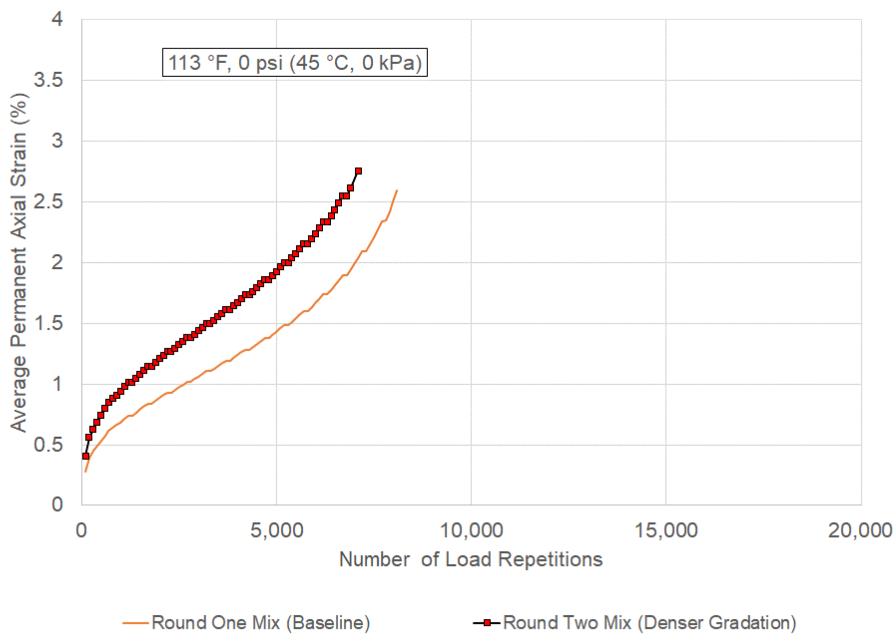

Figure 5.10: Comparison of permanent axial strain accumulation curves of the Round One and Round Two mixes tested under 113°F (45°C) with no confinement.

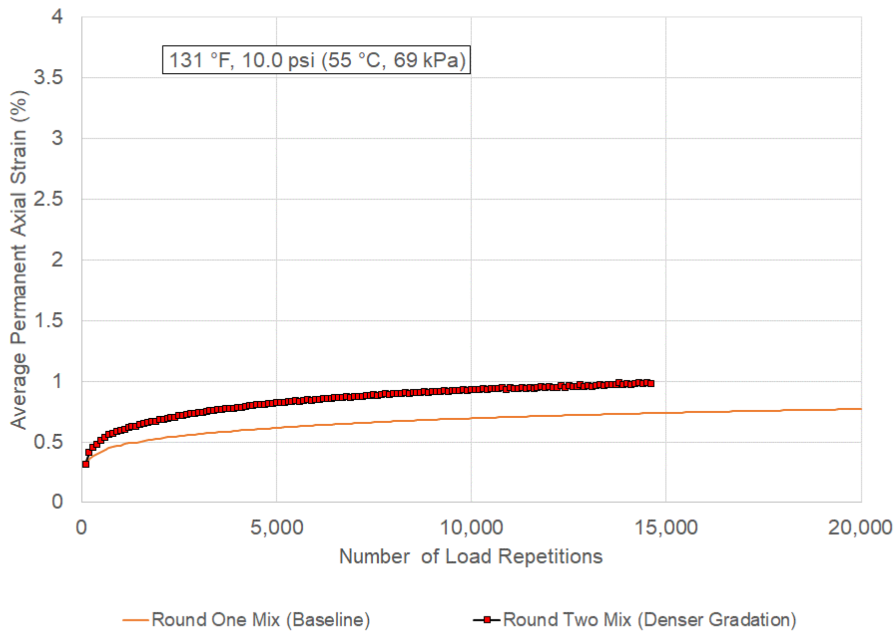

Figure 5.11: Comparison of permanent axial strain accumulation curves of the Round One and Round Two mixes tested under 131°F (55°C) with 10 psi (69 kPa) confinement.

5.2.4 Moisture Susceptibility

The average rut accumulation curves from both sides of the HWTT test specimens for the Round Two mix are shown in Figure 5.12. As shown in the figure, the Round Two mix has very little rut when tested under the 122°F (50°C) water bath. This satisfies the Caltrans requirement for mixes with PG 64 binder by a wide margin, which indicates that the mix is not susceptible to moisture damage.

Figure 5.12 shows a significantly larger difference between the results from the two sides for the Round Two mix compared to the Round One mix (see Figure 4.10). A comparison of right-side rut accumulation curves for these mixes is shown in Figure 5.13, and it indicates nearly identical performance for the two mixes.

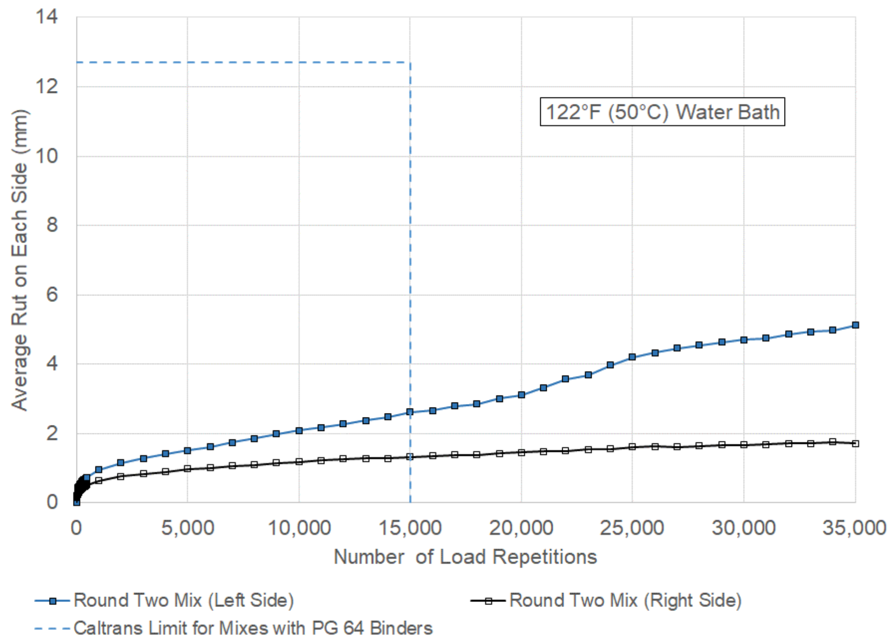

Figure 5.12: Accumulation of average rut in Hamburg Wheel-track Testing for the Round Two mix under 122°F (50°C) water bath.

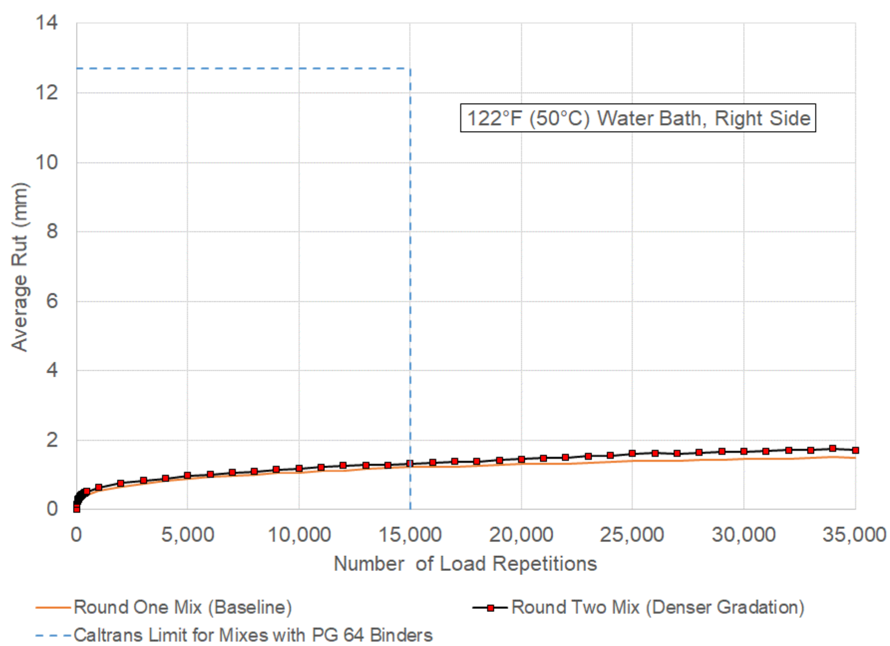

Figure 5.13: Comparison of average right-side rut accumulation in Hamburg Wheel-track Testing for the Round One and Round Two mixes.

5.3 Performance Summary

As the preceding comparisons between the Round Two (denser gradation) and the Round One (baseline) mixes showed, the Round Two mix had the same stiffness as the Round One mix, much improved fatigue performance, and either slightly worse or much better rutting performance according to the RSST-CH test—depending on the testing temperature—but worse rutting performance according to the RLT. The Round Two mix showed large variation in rut accumulation in HWTT testing but nevertheless still appeared to not be susceptible to moisture damage. Overall, the denser gradation led to better fatigue performance, no significant improvement in or even worsening of rutting performance, no significant effect on moisture susceptibility, and no change in stiffness. The improvement in fatigue performance from the Round One to Round Two mixes is consistent with the flowchart shown in Figure 2.1. The change in rutting performance is, however, not consistent with the flowchart.

Compared to the Red Bluff mix (benchmark), the Round Two mix now had roughly the same fatigue life, but still had room for improvement in terms of rutting performance.

5.4 Selection of Mix Adjustments for Round Three and Round Four

The objective for Round Three was to further improve the rutting performance of the adjusted mix without a significant sacrifice in fatigue performance. Based on the flowchart in Figure 2.1, there are several ways to further improve rutting performance, and they are summarized in Table 5.1.

Table 5.1: Summary of Options for Mix Adjustment for Round Three

Option Number	Description	Comments
5-	Reducing binder content	Easy to achieve but it alone may make the resulting mix out of specification.
6+	Increase RAP content	Will change binder content and require change of the bin-batching formula to maintain blended aggregate gradation.
7	Change RAP source	Will change aggregate gradation and binder content. The result may not be favorable.
8	Change binder source	May or may not be possible for a given project depending on whether there is long-term contract between plant and binder supplier. The result may not be favorable.
9	Use a stiffer unmodified binder (i.e., with better high-temperature grade)	Easy to achieve.
10b	Use polymer-modifier with same or higher high temperature grade	Easy to achieve but will incur additional cost for the binder modification.

Since the effects of changing the RAP source and binder source may not be favorable, they were not considered in this study. Increasing the RAP content also was not considered because it would cause other changes and therefore the effect might not be predictable. After a review of the other options, it was decided to evaluate the following two options:

- Reduce the binder content (Option 5-): specifically, reduce the virgin binder content by 0.5 percent by total weight of dry aggregate, and,
- Use a stiffer unmodified binder (Option 9): that is, increase the high temperature grade from 64 to at least 70.

An unmodified binder was selected rather than a polymer-modified binder because it is a less expensive option.

In the next step of this guidance process, both options will be applied independently to the Round Two mix. The Round Three mix will be used to evaluate the effect of the binder content reduction and the Round Four mix will be used to evaluate the effect of stiffer binder.

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6 ROUND THREE RESULTS—LESS BINDER

As described in Section 5.4, the mix design adjustment selected for the Round Three mix was a reduction of the binder content. Specifically, the virgin binder content by total weight of dry aggregate was reduced from 4.38 percent by 0.5 percent to 3.88 percent. The total binder content (including virgin and RAP binder) by total weight of mixture decreased from 5.12 percent to 4.70 percent.

6.1 Superpave Volumetric Verification

The Superpave volumetrics for the Round Three mix (Round Two + less binder) are shown in Figure 6.1 to Figure 6.4 with results for the Round One (baseline) and Round Two (denser gradation) mixes. As these figures show, the Round Three mix still met all the Caltrans requirements for Superpave volumetric indexes. The lower binder content gradation used in the Round Three mix than in the Round Two mix led to higher air-void contents and effective dust proportion while maintaining practically the same VMA value.

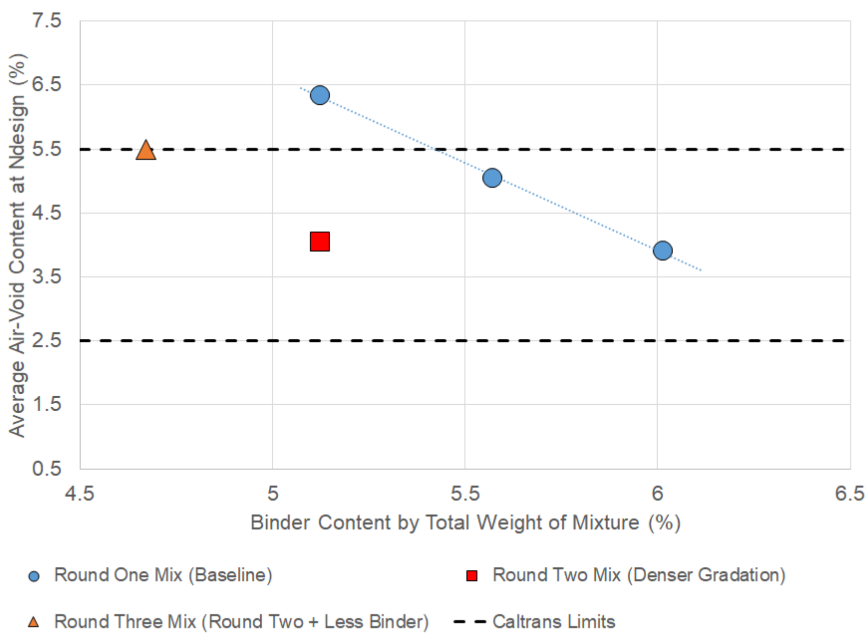

Figure 6.1: Air-void content at Ndesign gyrations for the Round One, Round Two, and Round Three mixes.

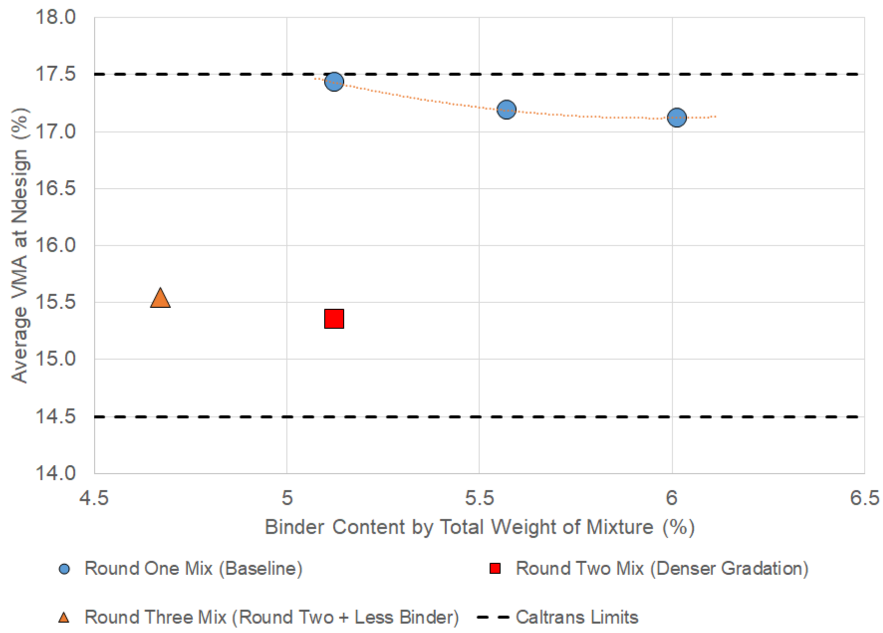

Figure 6.2: VMA at Ndesign gyrations for the Round One, Round Two, and Round Three mixes.

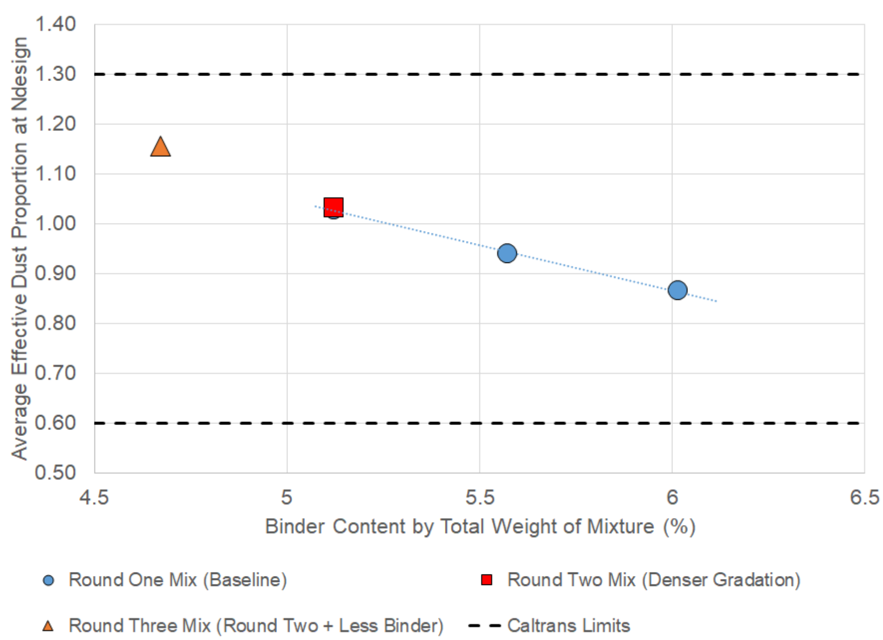

Figure 6.3: Effective dust proportion at Ndesign gyrations for the Round One, Round Two, and Round Three mixes.

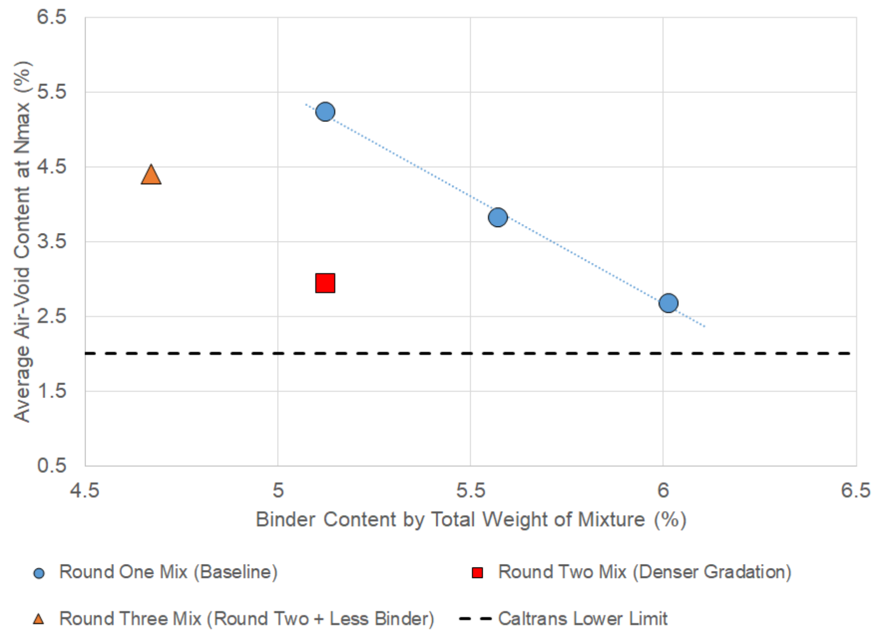

Figure 6.4: Air-void content at Nmax gyrations for the Round One, Round Two, and Round Three mixes.

6.2 Mix Performance

6.2.1 Stiffness

A comparison of the flexural stiffness master curves for the Round One (baseline), Round Two (denser gradation), Round Three (Round Two + Less Binder), and Red Bluff (benchmark) mixes is shown in Figure 6.5. A comparison of the dynamic modulus master curves for the same mixes is shown in Figure 6.6 except that no dynamic modulus is available for the Red Bluff (benchmark) mix. As is shown in these figures, the Round Three mix was stiffer than the Round Two mix both in terms of flexural stiffness and dynamic modulus. It is interesting to note, however, that the stiffness master curves for the Round Two and Round Three mixes converge at low temperatures for flexural stiffness and converge at high temperatures for dynamic modulus. At 20°C and 10 Hz, the Round Three mix is 28 percent stiffer than the Round Two mix in flexural stiffness, and 34 percent stiffer in dynamic modulus.

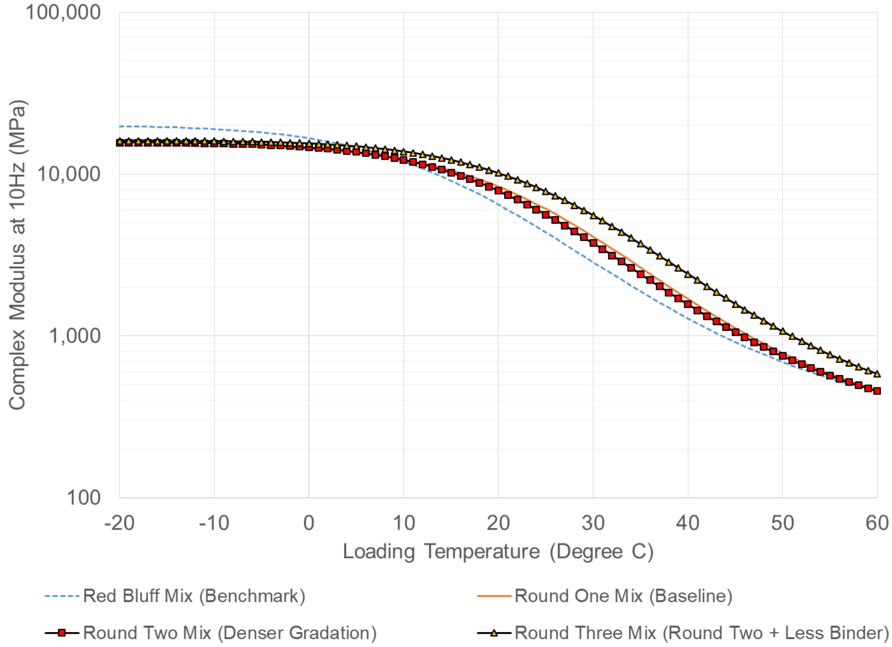

Figure 6.5: Comparison of flexural stiffness master curves of the Round One, Round Two, Round Three, and Red Bluff (benchmark) mixes.

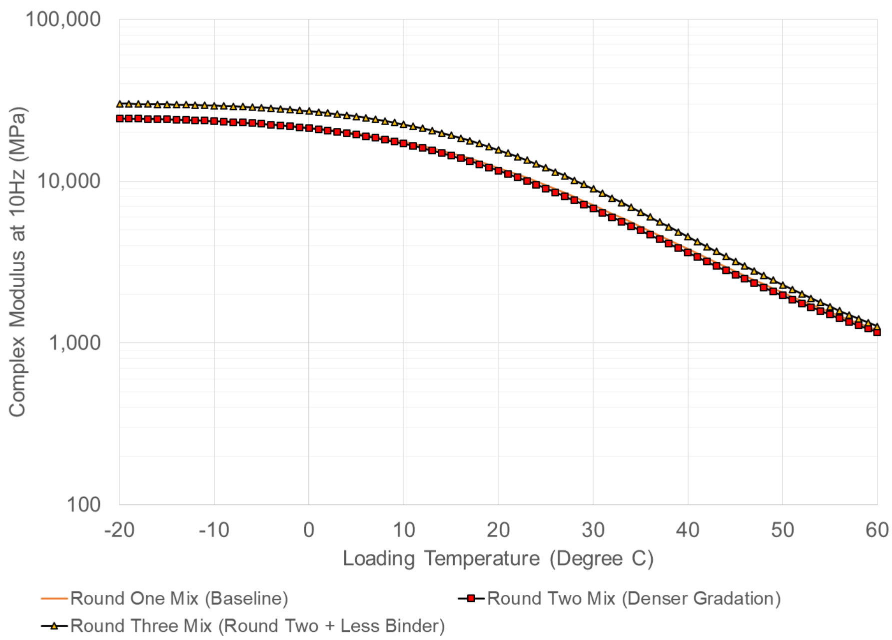

Figure 6.6: Comparison of dynamic modulus master curves of the Round One, Round Two, and Round Three mixes.

6.2.2 *Fatigue Life*

The variation of fatigue life with strain level for the Round Three mix (Round Two + less binder) is shown in Figure 6.7, along with the fatigue life for the Round One (baseline), Round Two (denser gradation), and Red Bluff (benchmark) mixes. As shown in the figure, the fatigue life of the Round Three mix is roughly same as the Round One mix, and is about one-third of the fatigue life of the Round Two mix.

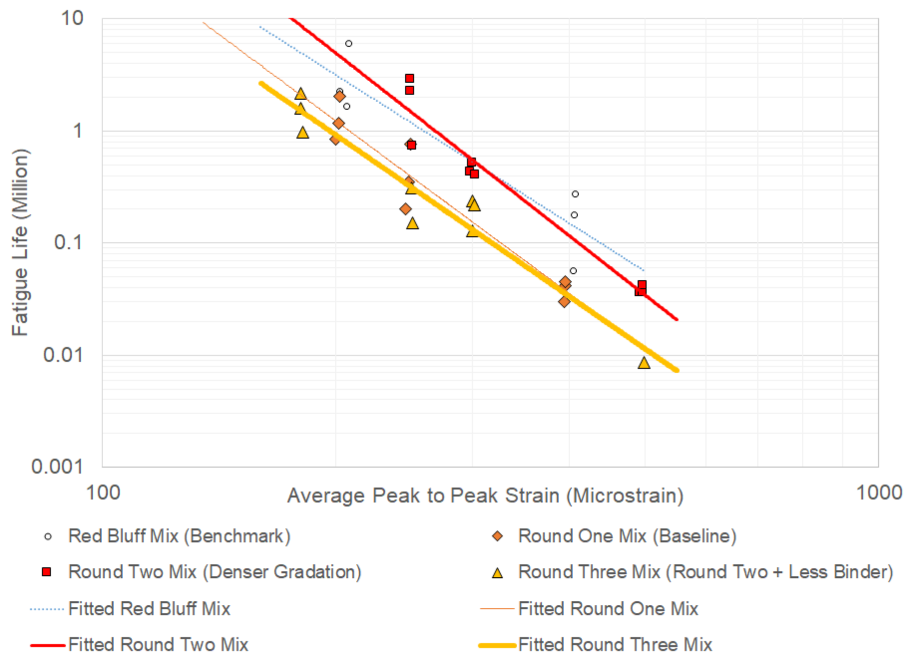

Figure 6.7: Comparison of fatigue life of the Round One, Round Two, Round Three, and Red Bluff mixes.

6.2.3 *Permanent Deformation*

6.2.3.1 Repeated Simple Shear Test at Constant Height

Two comparisons of permanent shear strain accumulation curves for the mixes from the first three rounds and the benchmark mix are shown in Figure 6.8 and Figure 6.9, respectively. As shown in the figures, the difference in permanent deformation performance between the Round Two and Round Three mixes was not the same at different temperatures. The Round Three mix was slightly more resistant to permanent deformation than the Round Two mix under 113°F (45°C) and 14.5 psi (100 kPa) shear stress, but was slightly less resistant under 131°F (55°C) and 14.5 psi (100 kPa) shear stress.

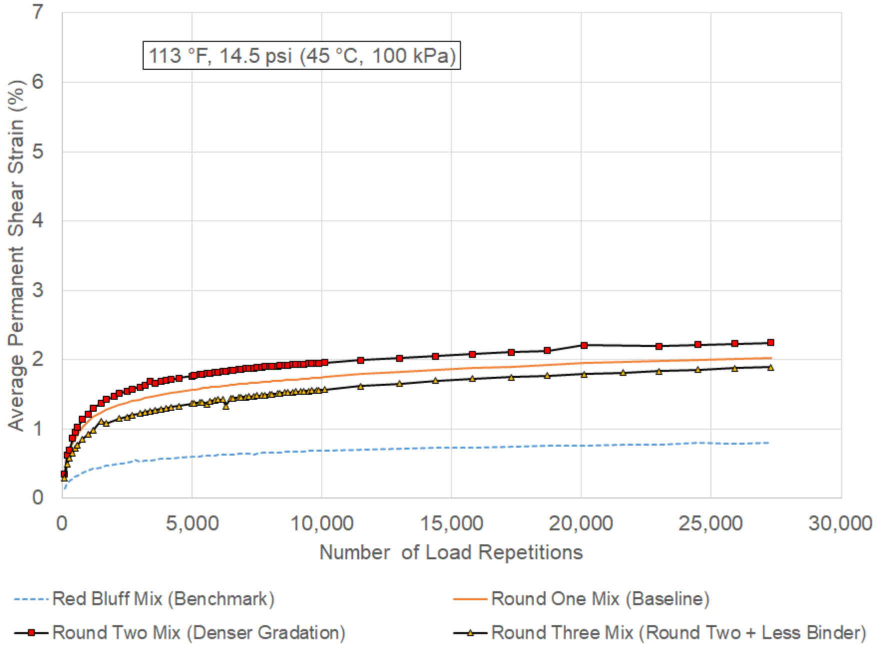

Figure 6.8: Comparison of permanent shear strain accumulation curves of the Round One, Round Two, Round Three, and Red Bluff mixes tested at 113°F (45°C) and 14.5 psi (100 kPa) shear stress.

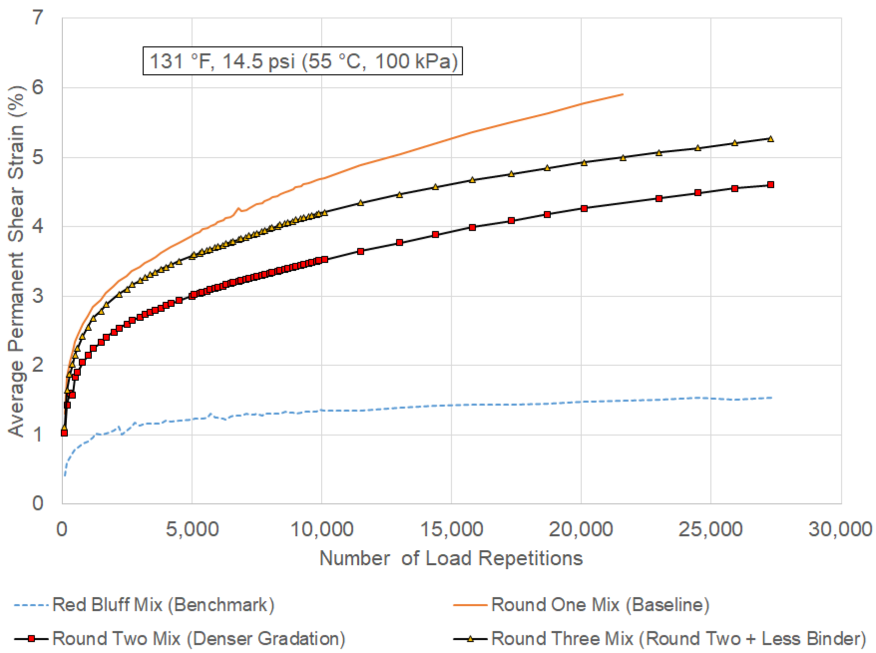

Figure 6.9: Comparison of permanent shear strain accumulation curves of the Round One, Round Two, Round Three, and Red Bluff mixes tested at 131°F (55°C) and 14.5 psi (100 kPa) shear stress.

6.2.3.2 Repeated Load Triaxial Test

Comparisons of the average permanent axial strain accumulation curves for the Round One to Round Three mixes are shown in Figure 6.10 and Figure 6.11 for the 113 and 131°F (45 and 55°C) test temperatures, respectively. The figures show that Round Three mix was more resistant to permanent deformation than the Round Two mix in RLT testing at both temperatures. The comparison between the Round Three and the Round One mix is inconsistent, however, and depends on the confinement used in the test: the Round Three mix is slightly more resistant to permanent deformation when unconfined, but slightly less resistant when confined.

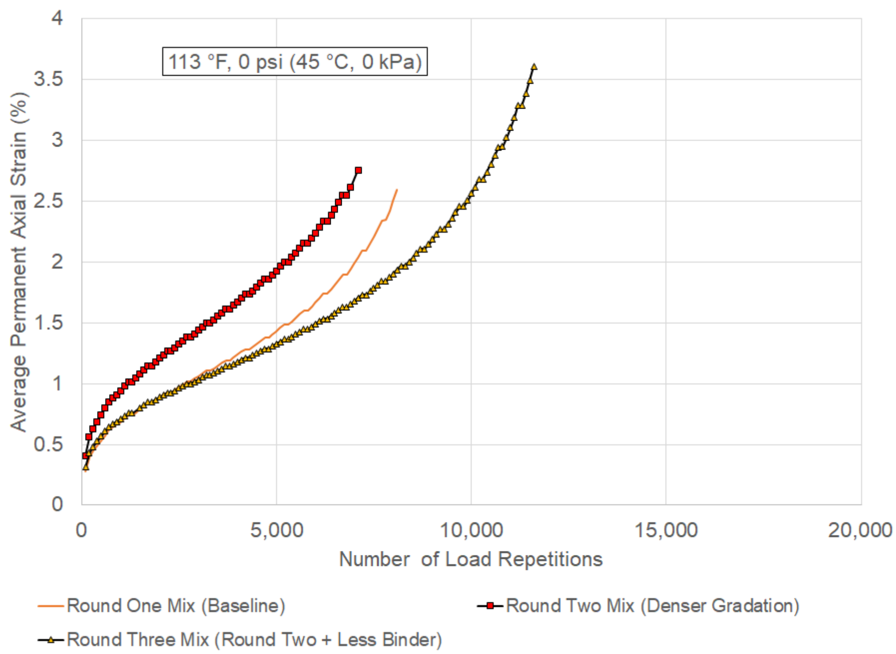

Figure 6.10: Comparison of permanent axial strain accumulation curves for the Round One, Round Two, and Round Three mixes tested under 113°F (45°C) with no confinement.

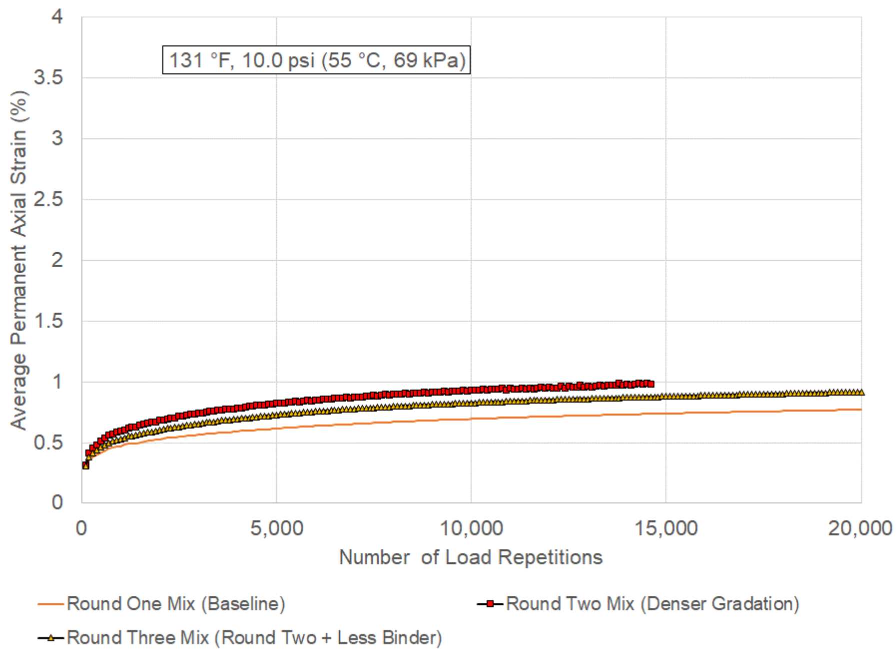

Figure 6.11: Comparison of permanent axial strain accumulation curves for the Round One, Round Two, and Round Three mixes tested under 131°F (55°C) with 10 psi (69 kPa) confinement.

6.2.4 Moisture Susceptibility

The average rut accumulation curve from each side for the Round Three mix is shown in Figure 6.12. The figure shows that the Round Three mix had very little rut when tested under the 122°F (50°C) water bath. This satisfies the Caltrans requirement for mixes with PG 64 binder by a wide margin so the mix is not susceptible to moisture damage.

A comparison of the right-side rut accumulation curves for the Round One, Round Two, and Round Three mixes is shown in Figure 6.13, which indicates that Round Three mix has slightly less rut than both the Round One and Round Two mixes, but the differences are practically negligible.

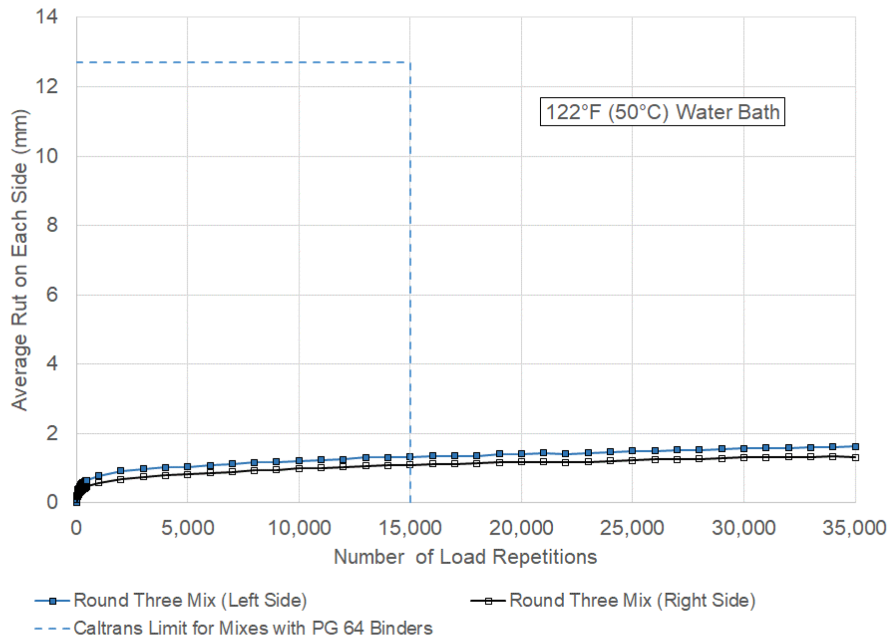

Figure 6.12: Accumulation of average rut in Hamburg Wheel-track Testing for the Round Three mix under 122°F (50°C) water bath.

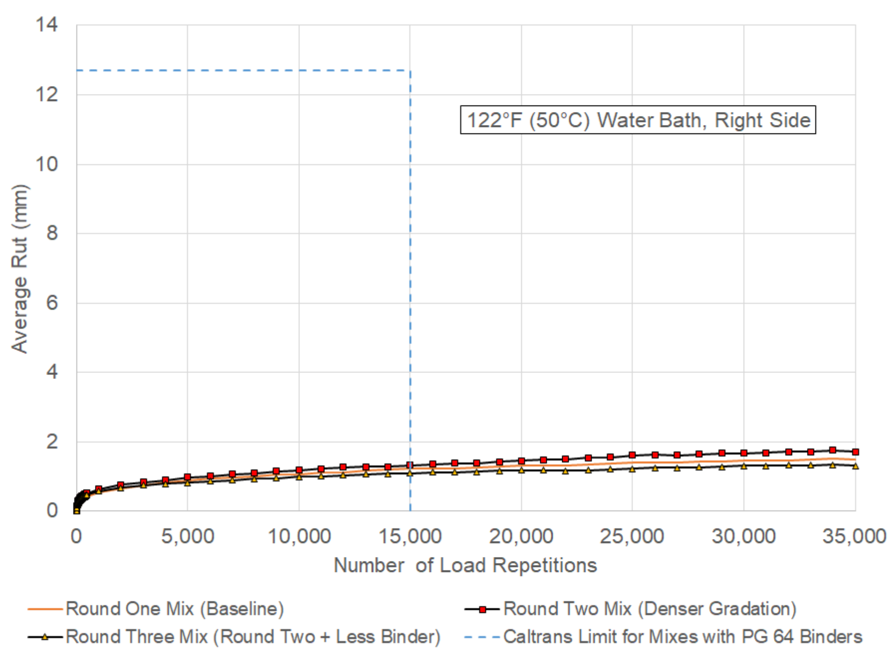

Figure 6.13: Comparison of average right-side rut accumulation in Hamburg Wheel-track Testing for the Round One, Round Two, and Round Three mixes.

6.3 Performance Summary

As shown above, compared to the Round Two mix with its denser gradation, the Round Three mix with its reduced binder content was stiffer; had much worse fatigue performance; had rutting performance either similar to or slightly worse than the Round Two mix—depending on the temperature—in RSST-CH testing, but much improved rutting performance at both temperatures in RLT testing; and had the same moisture damage susceptibility performance under HWTT testing. Overall, the use of less binder led to higher stiffness and improved rutting performance under the RLT, but slight improvement or worsening in the RSST-CH, worse fatigue performance, and no significant effect on moisture susceptibility. The effect on fatigue performance is consistent with the flowchart in Figure 2.1, but the effect on rutting performance is inconclusive.

The Round Three mix (Round Two + less binder) had roughly one-third the fatigue life of the Red Bluff mix (benchmark), and there was still room for improvement in terms of rutting performance.

7 ROUND FOUR RESULTS—STIFFER BINDER

As described in Section 5.4, the mix design adjustment selected for the Round Four mix was replacement of the binder in the Round Two mix with a stiffer, unmodified binder. Specifically, the adjustment called for a binder with a higher temperature grade than the PG 64-16 that was used for both the Round One and Round Two mixes. After checking with local refineries, a PG 70-10 binder was selected because PG 70-16 binder was not available.

7.1 Superpave Volumetric Verification

The Superpave volumetrics for the Round Four mix (Round Two + stiffer binder) are shown in Figure 7.1 through Figure 7.4, along with results for the Round One (baseline), Round Two (denser gradation) and Round Three Round Two + less binder) mixes. As shown in these figures, the Round Four mix still met all the Caltrans requirements for Superpave volumetric indexes. Compared to the Round Two mix with the same binder content, the stiffer binder used in the Round Four mix led to higher air-void contents and VMA while maintaining practically the same value for effective dust proportion, as would be expected. The higher air-void content is likely a reflection of the reduced workability of the mix with the stiffer binder.

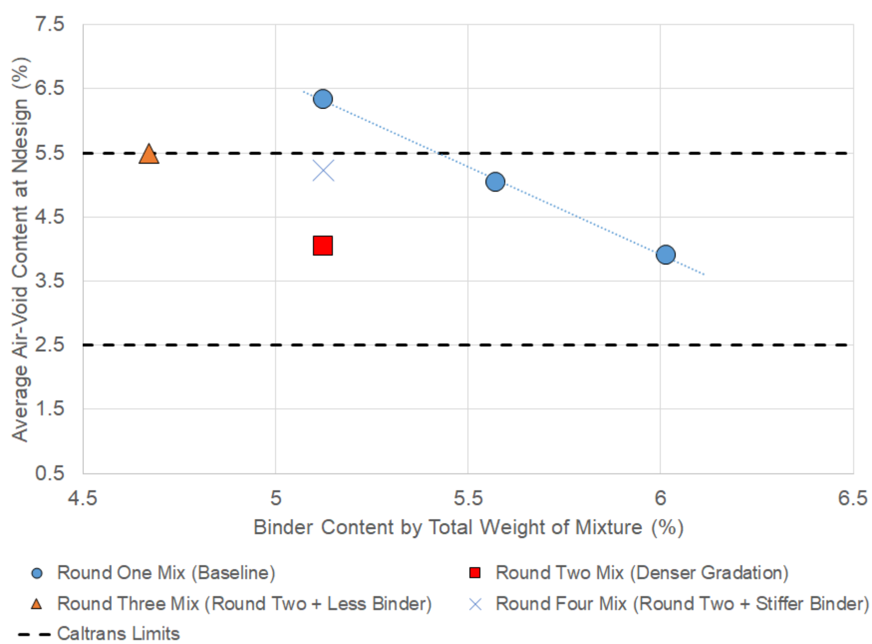

Figure 7.1: Air-void content at Ndesign gyrations of the Round One to Round Four mixes.

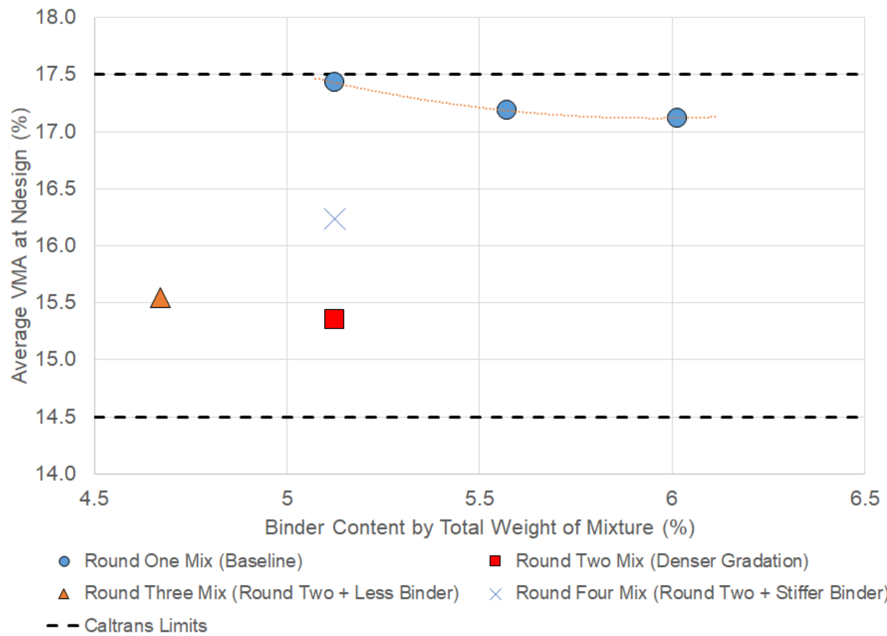

Figure 7.2: VMA at Ndesign gyrations of the Round One to Round Four mixes.

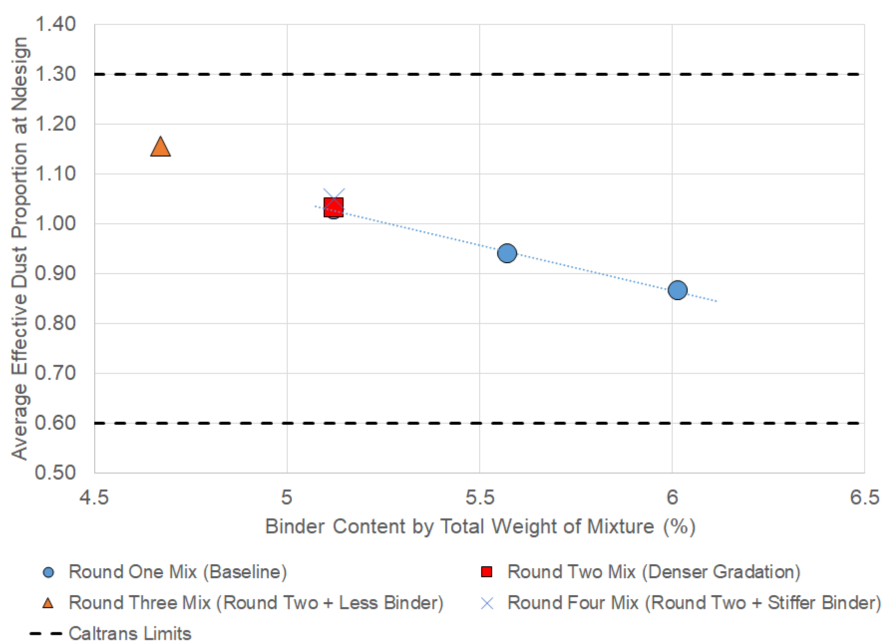

Figure 7.3: Effective dust proportion at Ndesign gyrations of the Round One to Round Four mixes.

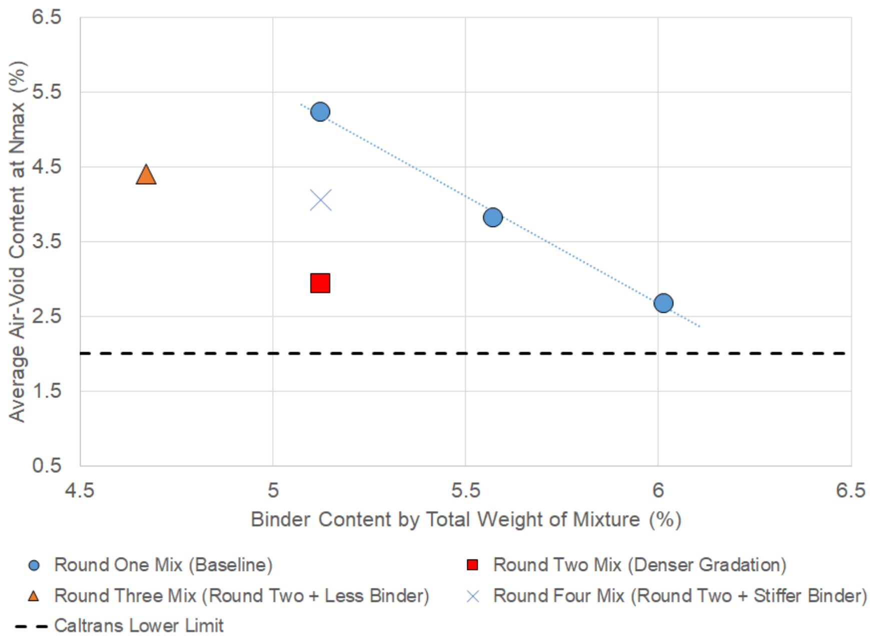

Figure 7.4: Air-void content at Nmax gyrations of the Round One to Round Four mixes.

7.2 Mix Performance

7.2.1 Stiffness

A comparison of the flexural stiffness master curves for the Round One, Round Two, Round Three, and Round Four mixes as well as the Red Bluff mix (benchmark) are shown in Figure 7.5. A comparison of the dynamic modulus master curves for the Round One to Round Four mixes are shown in Figure 7.6. From the figures, it can be seen that the Round Four mix (Round Two + stiffer binder) was stiffer than the Round Two mix (denser gradation) both in terms of flexural stiffness and dynamic modulus. This is expected because of the stiffer high PG grade binder, but this may not always be the case when this change is made because the high temperature PG grade may have varying effects on intermediate temperatures, depending on the temperature susceptibility of the binder. At 20°C and 10 Hz, the Round Four mix was 65 percent stiffer than Round Two mix in flexural stiffness, and 67 percent stiffer in dynamic modulus.

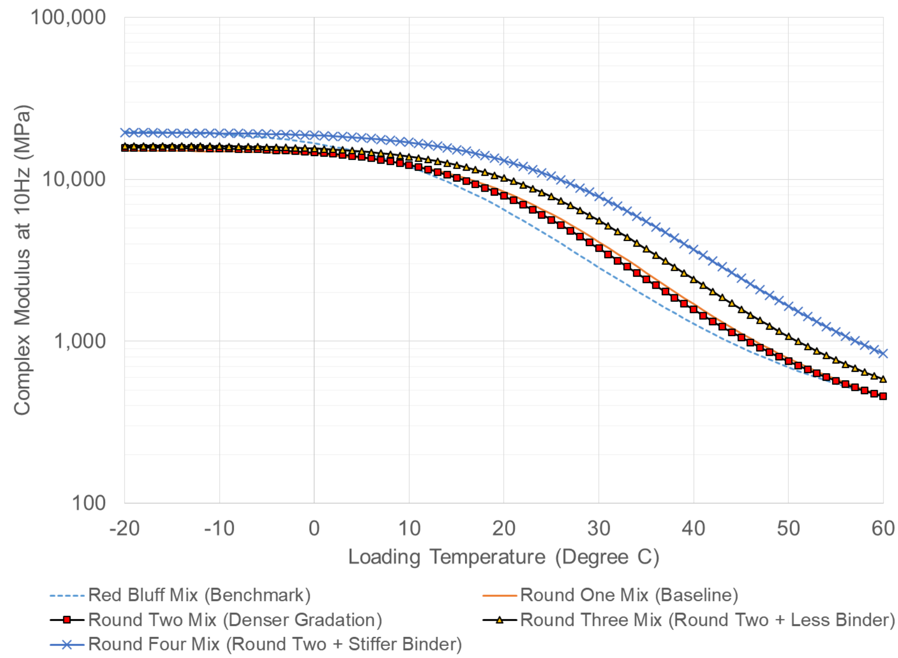

Figure 7.5: Comparison of flexural stiffness master curves of the Round One, Round Two, Round Three, Round Four, and Red Bluff (benchmark) mixes.

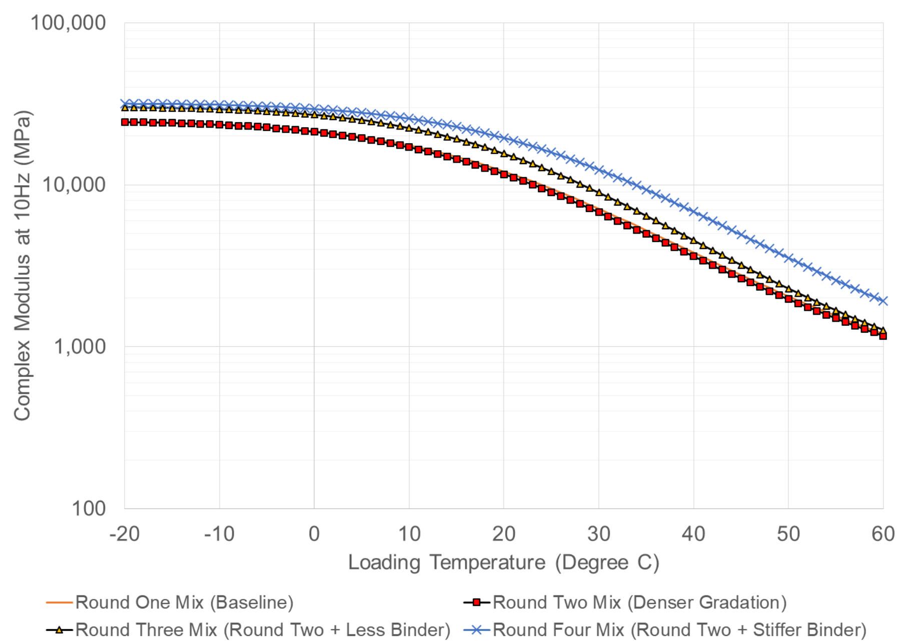

Figure 7.6: Comparison of dynamic modulus master curves of the Round One, Round Two, Round Three, and Round Four mixes.

7.2.2 *Fatigue Life*

The variation of fatigue life with strain level for the Round Four mix is shown in Figure 7.7, along with the fatigue lives for the Round One, Round Two, Round Three, and Red Bluff mixes. As shown in the figure, the fatigue life for the Round Four mix is roughly the same as the Round One mix (baseline), and is about one third of the fatigue life for the Round Two mix (denser gradation).

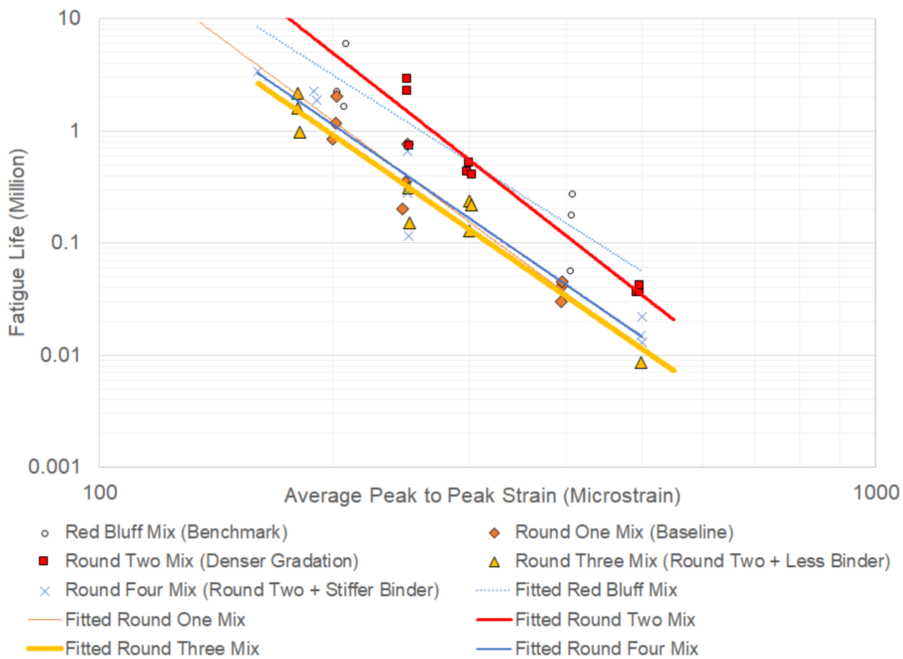

Figure 7.7: Comparison of fatigue life of the Round One, Round Two, Round Three, Round Four, and Red Bluff mixes.

7.2.3 *Permanent Deformation*

7.2.3.1 Repeated Simple Shear Test at Constant Height

Two comparisons of permanent shear strain accumulation curves for mixes from all four rounds and the benchmark mix are shown in Figure 7.8 and Figure 7.9, respectively. As shown in the figures, the Round Four mix showed consistently better resistance to permanent deformation than the Round Two mix. Specifically, the Round Four mix accumulated about 20 percent less permanent shear strain than the Round Two mix.

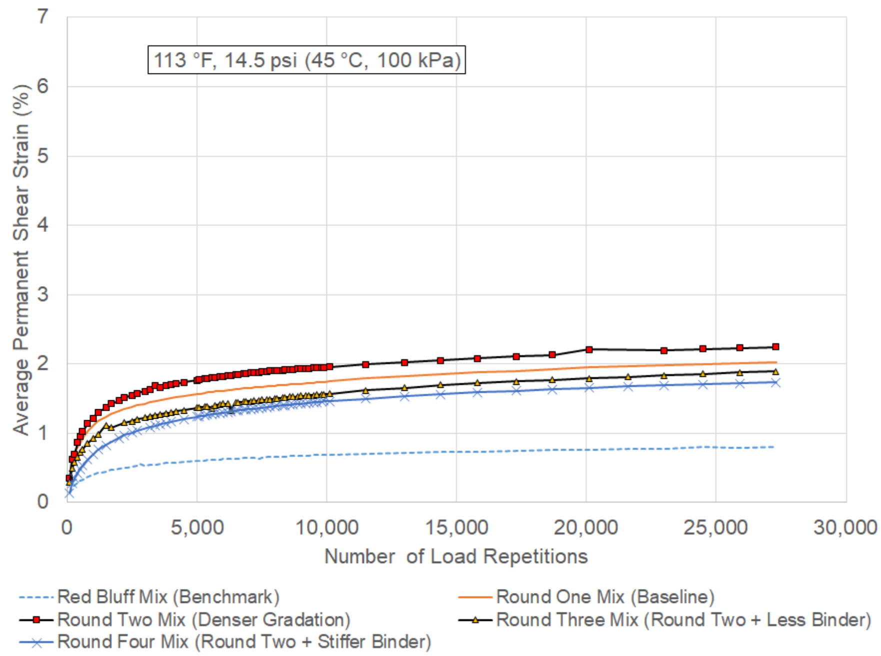

Figure 7.8: Comparison of permanent shear strain accumulation curves of the mixes from all four rounds and the Red Bluff (benchmark) mix tested at 45°C and 100 kPa shear stress.

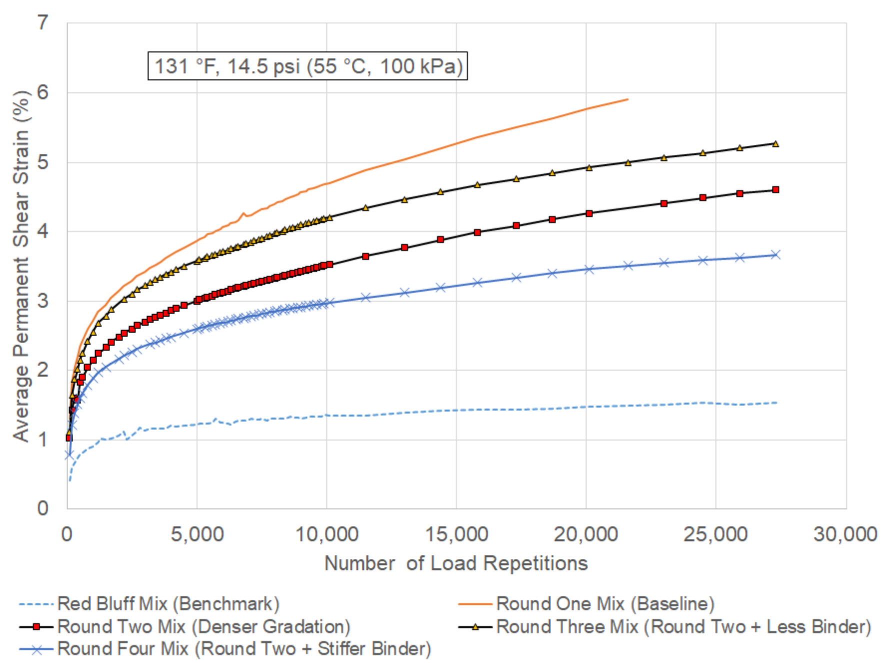

Figure 7.9: Comparison of permanent shear strain accumulation curves of the mixes from all four rounds and the Red Bluff (benchmark) mix tested at 55°C and 100 kPa shear stress.

7.2.3.2 Repeated Load Triaxial Test

Comparisons of the average permanent axial strain accumulation curves for the Round One to Round Four mixes are shown in Figure 7.10 and Figure 7.11 for the 113 and 131°F (45 and 55°C) test temperatures, respectively. Both figures show that the Round Four mix was more resistant to permanent deformation than the Round Two mix in RLT testing. The difference, however, is much more pronounced under the unconfined condition compared to the confined condition.

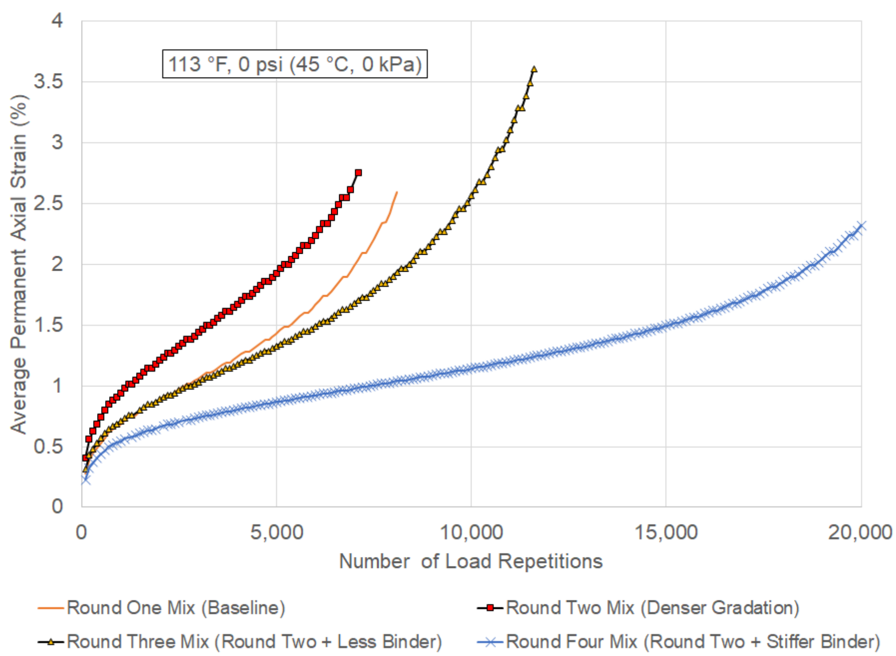

Figure 7.10: Comparison of permanent axial strain accumulation curves of the Round One to Round Four mixes tested under 113°F (45°C) with no confinement.

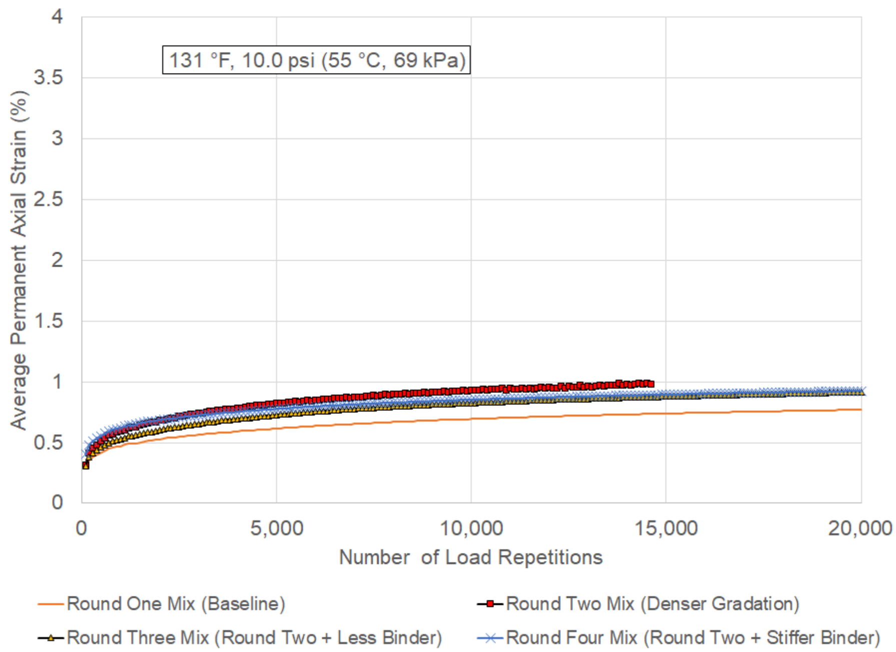

Figure 7.11: Comparison of the permanent axial strain accumulation curves of the Round One to Round Four mixes tested under 131°F (55°C) with 69 kPa confinement.

7.2.4 Moisture Susceptibility

The average rut accumulation curve from each side for the Round Four mix is shown in Figure 7.12. The figure shows that the Round Four mix had very little rut when tested under the 122°F (50°C) water bath. This satisfies the Caltrans requirement for mixes with PG 64 binder by a wide margin, so the mix is not susceptible to moisture damage.

A comparison of the right-side rut accumulation curves for Round One, Round Two, Round Three, and Round Four mixes is shown in Figure 7.13, which indicates that the Round Four mix has slightly less rut than both the Round One and Round Two mixes but the differences are practically negligible.

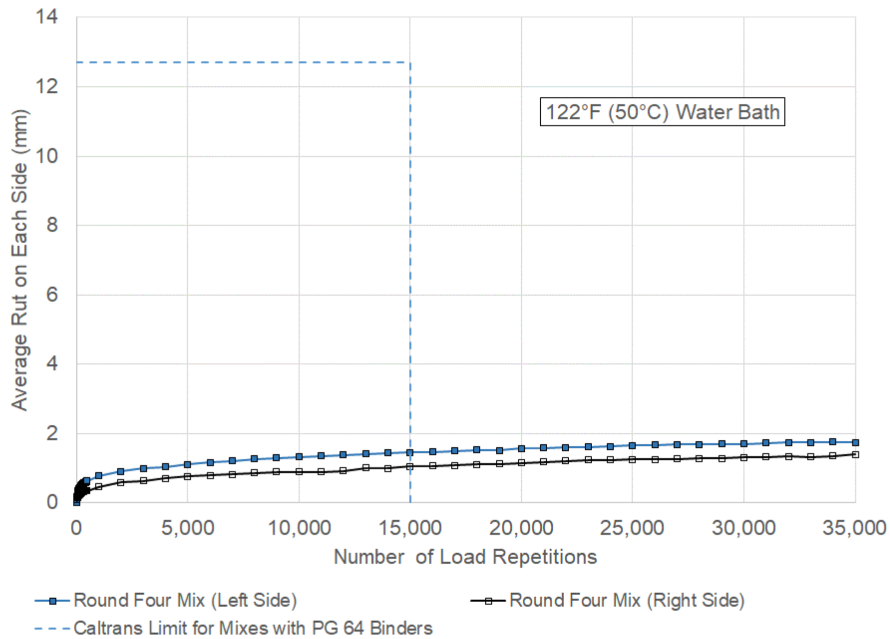

Figure 7.12: Accumulation of average rut in Hamburg Wheel-track Testing for the Round Four mix under 122°F (50°C) water bath.

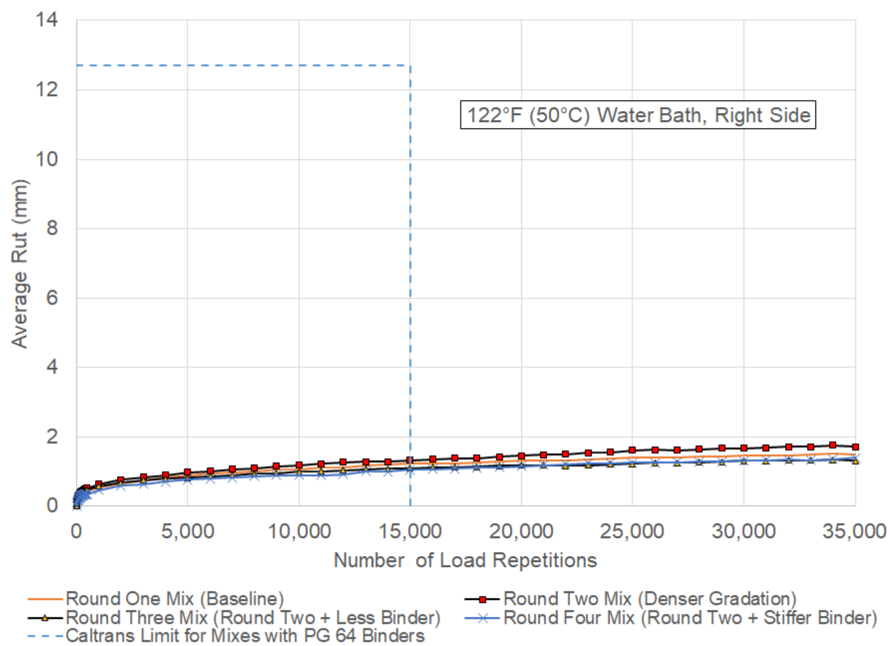

Figure 7.13: Comparison of average right-side rut accumulation in Hamburg Wheel-track Testing for the Round One to Round Four mixes.

7.3 Performance Summary

As shown above, compared to the Round Two mix, the Round Four mix is stiffer, has much worse fatigue performance, has better rutting performance in both the RSST-CH and the RLT, and has the same moisture damage susceptibility performance under HWTT. Overall, the use of the stiffer conventional binder led to higher stiffness, improved rutting performance, worse fatigue performance, and no significant change in moisture susceptibility. These effects are consistent with the flowchart in Figure 2.1.

The Round Four mix (Round Two + stiffer binder) had roughly one-third the fatigue life of the Red Bluff mix (benchmark), and there is still room for improvement in terms of rutting performance.

Considering the results from the four steps taken in the study, some potential next steps starting from the Round Two (denser gradation) mix might be to:

- Further densify the gradation somewhat
- Look at additional crushing of the coarser aggregates
- Try a different PG 70 binder that may have better fatigue performance and a greater multiple stress creep recovery value (MSCR, AASHTO TP 70)
- Change to a polymer-modified binder
- Decide not to bid the job from this plant if the requirements are to meet the performance of the baseline mix

Changing to a polymer-modified binder would potentially help meet the rutting and fatigue requirements. However, a polymer-modified mix will likely have lower stiffness and therefore can potentially result in shorter pavement fatigue life if it is used in a thick overlay in which pavement strain is more sensitive to mix stiffness.

8 MECHANISTIC-EMPIRICAL ANALYSIS

The performance of the mixes in the different rounds were compared in Chapters 4 through 7 based on results from individual tests. In this chapter, mechanistic-empirical analysis was used to further evaluate the fatigue cracking and overall performance of these mixes to learn about how they will perform if used as in pavement structures. The reason for this evaluation is to account for the interaction of different properties of the mixes. The use of these mixes in thin overlays on existing cracked asphalt pavement was not considered in this study because performance-related specifications (PRS) are currently not being used for that design case.

8.1 Pavement Fatigue Cracking Performance

Two asphalt-surfaced pavements were selected for a comparison of fatigue cracking performance. The structures of the selected pavements are shown in Table 8.1, where it can be seen that the two pavements have the same aggregate base and subgrade but different combinations for the top two layers. The two combinations are (a) 50 mm of hot mix asphalt (HMA) on top of 200 mm of FDR-PC (full-depth reclamation with cement stabilization) and (b) 200 mm of HMA without FDR-PC. The structure with 50 mm HMA represents the case where the HMA is not the main structural layer and, as a result, the strain at its bottom is not strongly affected by its own stiffness. The structure with 200 mm HMA represents the case where the HMA is the main structural layer and, as a result, the strain at its bottom is strongly affected by its stiffness. Note that the FDR-PC is assumed to have sufficiently low cement content such that there is only minor shrinkage cracking, and a bond breaker is assumed to have been placed between the HMA and the FDR-PC layers to reduce the risk of reflective cracking, although this also increases the strains in the asphalt layer.

Table 8.1: Structure of the Pavements Selected for Evaluating Mix Performance

Layer Number	Material	Stiffness (MPa)	Thickness (mm)	
1	HMA	Mix dependent	50	200
2	Full-depth reclamation with cement stabilization (FDR-PC)	5,000	200	0
3	Aggregate base	300	300	
4	Subgrade	70	semi-infinite	

To make the mechanistic comparison simple, the truck traffic axle load spectra common in the state are reduced to the number of ESALs (equivalent single axle loads), that is, single axles with dual wheels and an axle load of 18,000 lbs (80 kN). In addition, the pavement temperature is fixed at 67°F (20°C). To predict pavement fatigue cracking life, Equation (1) (see Section 3.1.1) is used to correlate cyclic strain amplitude to the fatigue life determined with laboratory testing. The fatigue life equation parameters for the different mixes from the flexural fatigue testing are listed in Table 8.2. Note that rather than allowing it to differ for each mix, equation parameter *b* has been fixed at -4.9 by fitting the fatigue data for all mixes. This was necessary to avoid any bias caused by extrapolations when predicting pavement fatigue cracking life because pavement strains are much lower than the ones used in the laboratory fatigue tests. Since b is fixed, a higher value for a indicates longer fatigue life for a mix.

Table 8.2: Fatigue Equation Parameters

Mix	Description	Parameter *a*	Parameter *b*	Ranking
Red Bluff	Benchmark	2.95E-12	-4.9	1
Round One	Baseline	8.93E-13	-4.9	4
Round Two	Denser gradation	2.94E-12	-4.9	2
Round Three	Round Two mix with less binder	7.35E-13	-4.9	5
Round Four	Round Two mix with stiffer binder	9.29E-13	-4.9	3

According to Table 8.2, the benchmark mix has the longest fatigue life and the Round Three mix has the shortest. Using the denser gradation increased the laboratory flexural fatigue life for a given strain, while using either less binder or stiffer binder decreased fatigue life.

To determine mix layer stiffness under traffic, trucks were assumed to be traveling at a constant speed of 60 mph (96 km/h). The loading time was calculated using the following equation:

$$t_L = \frac{200mm + z}{V} \quad \textbf{(2)}$$

where t_L is the loading time, 200 mm represents the tire tread contact length on the surface in the direction of traffic, z is the depth at mid-depth of the HMA layer, and V is the vehicle traveling speed. The resulting loading time is 0.009 second for the 50 mm HMA layer and 0.015 second for the 200 mm HMA layer, respectively. The corresponding loading frequencies are 17 Hz and 11 Hz, respectively. Since the objective was not to compare the performance of the two pavements, a loading frequency of 10 Hz was used as approximate for both pavements. The stiffnesses for the different mixes are shown as part of Table 8.3.

The resulting strain at the bottom of the HMA layer was calculated using the software *OpenPave* and was used to predict the pavement fatigue cracking life using the fatigue parameters in Table 8.2 with Equation (1). The results for the two pavements are shown in Table 8.3. In this table, the fatigue cracking lives have been normalized using the value for the corresponding pavement with the baseline Round One mix.

Table 8.3: Comparison of Pavement Fatigue Cracking Life

Mix Name	HMA Stiffness[1] (MPa)	With 50 mm HMA and 200 mm FDR-PC		With 200 mm HMA and no FDR-PC	
		Strain (με)	Normalized Fatigue Cracking Life	Strain (με)	Normalized Fatigue Cracking Life
Red Bluff (benchmark)	6,490	93	2.0	80	1.5
Round One (baseline)	8,380	84	1.0	68	1.0
Round Two (denser gradation)	7,903	86	2.9	70	2.9
Round Three (Round Two + less binder)	10,181	79	1.1	59	1.7
Round Four (Round Two + stiffer binder)	13,073	72	2.2	50	4.7

Note:
[1]: at 68°F (20°C) and 10 Hz

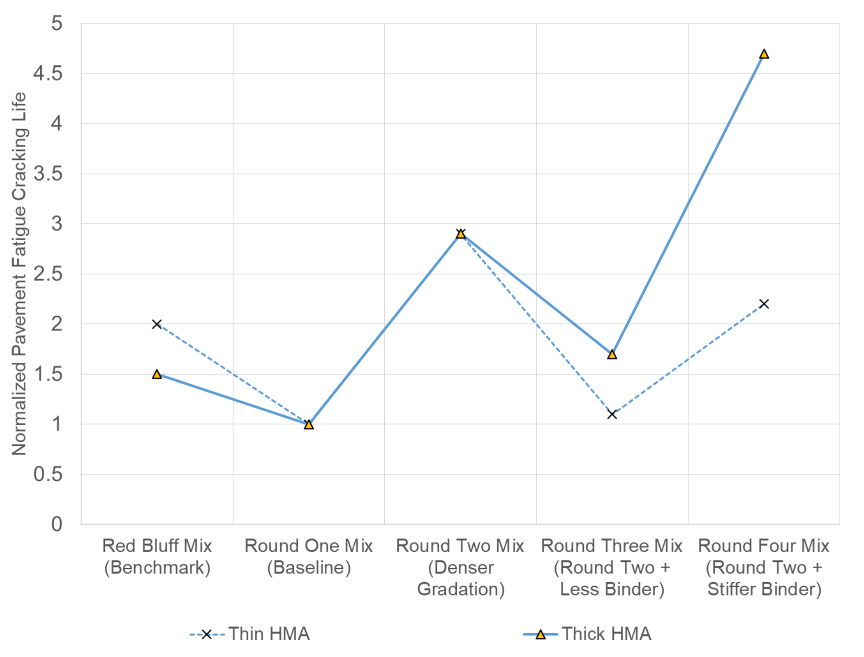

Figure 8.1: Comparison of normalized fatigue cracking life for pavements with different mixes as surface layer.

Table 8.3 and Figure 8.1 suggest that:

- The increased stiffness of the denser gradation mix in Round 2 resulted in better pavement fatigue cracking life than the benchmark mix despite having a lower laboratory fatigue life at a given strain.
- Using a denser gradation in the Round Two, Round Three, and Round Four mixes significantly increased both laboratory fatigue life and pavement fatigue cracking life because of the combination of increased stiffness and increased laboratory fatigue life at a given tensile strain.
- Using less binder in the Round Three mix decreased both laboratory fatigue life and pavement fatigue cracking life compared with using stiffer binder in the Round Four mix. However, the lower-binder Round Three mix still had similar or better performance than the less-dense gradation baseline Round One mix, particularly in the thick HMA layer pavement, because of its greater stiffness.
- Using stiffer binder decreased laboratory fatigue life but can nevertheless substantially increase pavement fatigue cracking life in pavements with thick HMA layers because of the higher stiffness.
- Although the Round Four mix had a much shorter laboratory fatigue life than the Red Bluff mix (benchmark), its higher stiffness led to lower strain in the pavements and in turn to longer pavement fatigue cracking life. The effect of stiffness on pavement cracking life is more significant when the HMA layer is thick than when it is thin.

8.2 Overall Pavement Performance

To evaluate the overall rutting and fatigue cracking performance of the different mixes when they are used as a pavement surface layer, the mechanistic-empirical (ME) design software *CalME* was used to simulate pavement performance. Note that the material properties for permanent deformation used in *CalME* are derived from RSST-CH results only and not from RLT testing. In addition, to account for the interaction between the different mechanical properties (such as stiffness and resistance to permanent deformation) of a given mix, *CalME* allows users to include factors such as truck axle load spectrum, climate, and structure on mix performance—although these were not included in the simple fatigue performance comparison done above. The factorials for evaluating mix performance in pavements using *CalME* are listed in Table 8.4. For this study, *CalME* version 2.0 was used.

Table 8.4: Factorials for Evaluating Mix Performance in Pavements Using *CalME*

Variables	Levels	Number of Levels
Climate	Desert (hot), North Coast (cold)	2
Main structural layers (see Table 8.1)	50 mm HMA + 200 mm FDR-PC (thin HMA), 200 mm HMA without FDR-PC (thick HMA)	2
HMA mix type	Red Bluff mix, and Round One to Round Four mixes	5
	Total Number of Cases	20

The two selected climate zones represent the hottest and coldest regions in California. Note that the choice of climate zone in *CalME* Version 2.0 only affects pavement temperatures. The other effects, such as moisture content and freeze-thaw, are not yet accounted for by the program.

The two structures used in the *CalME* evaluations were the same as those shown in Table 8.1. The FDR-PC layer is represented by a generic pavement material that is not subjected to fatigue damage and permanent deformation. The FDR-PC layer and HMA surface were again assumed to be debonded to reduce reflective cracking. The debonding, however, causes shear stress under the outside edge of the tire to drop significantly and as a result leads to almost no permanent deformation in the HMA layer. To be conservative, the HMA layer and FDR-PC layer were assumed to be bonded in *CalME* for permanent deformation calculation.

As in Section 8.1, the traffic is presented in ESALs only. The pavement was assumed to have only one lane in each direction with 10 million ESALs in traffic volume in the design lane each year and no yearly growth. The *CalME* simulations run for 20 years with a total traffic volume of 200 million ESALs, corresponding to a Caltrans TI (traffic index) of about 17.0, which is approximately the same as the design lane on the California state highway with the heaviest traffic.

8.2.1 Pavement Fatigue Cracking Performance

Pavement fatigue cracking performance is indicated by the damage in the HMA layer which is defined as reduction in stiffness in the incremental-recursive analysis, which is different from a Miner's Law analysis *(9, 10)*. Note that the stiffness ratio (current stiffness relative to initial stiffness) was calculated at 20°C and 10 Hz and excludes the stiffening effect of aging. Figure 8.2 shows the comparison of HMA layer damage under different combinations of climate and pavement structure. Less damage suggests better fatigue cracking performance.

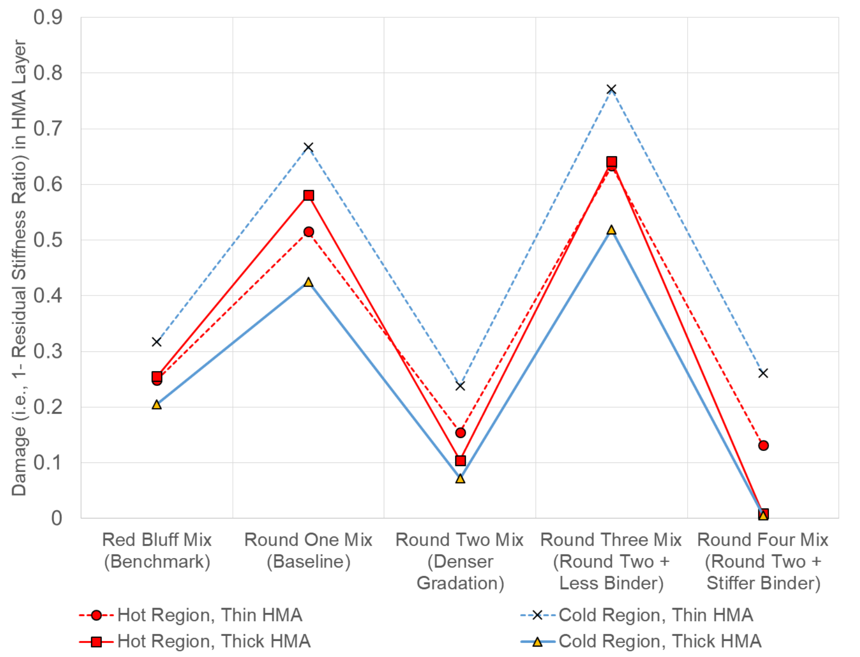

Figure 8.2: Comparison of damage in the HMA layer.

As shown in Figure 8.2, the effects of various mix design adjustments on pavement fatigue cracking performance were in general consistent with the corresponding effects on the mix fatigue life. Using a dense gradation had positive effects (i.e., improves) on both mix fatigue life and pavement fatigue cracking performance, while using less binder had negative effects on both. The exception was on the effect using stiffer binder. Specifically, using stiffer binder had a negative effect on mix fatigue life but its overall impact on pavement fatigue cracking performance was countered by the corresponding higher stiffness. For pavements with a thin HMA layer, the benefit of higher stiffness roughly canceled the effect of shorter mix fatigue life. For pavements with a thick HMA layer however, the benefit of higher stiffness outweighed the effect of shorter mix fatigue life and led to better pavement fatigue cracking performance. These observations are consistent with the ones made in Section 8.1.

8.2.2 Pavement Rutting Performance

Pavement rutting performance is indicated by permanent compression in the HMA layer. Figure 8.3 presents a comparison of permanent HMA layer compression for the different mixes—in which less HMA layer permanent compression indicates better pavement rutting performance. As shown in the figure, the amount of permanent HMA layer compression is less than 3.0 mm, indicating that none of the mixes is likely to cause rutting distress when used as pavement surface layer. Furthermore, pavements with the thin HMA surface layer had less permanent compression in the HMA layer than pavements with the thick HMA surface layer, mostly because the thicker layer has more material to deform.

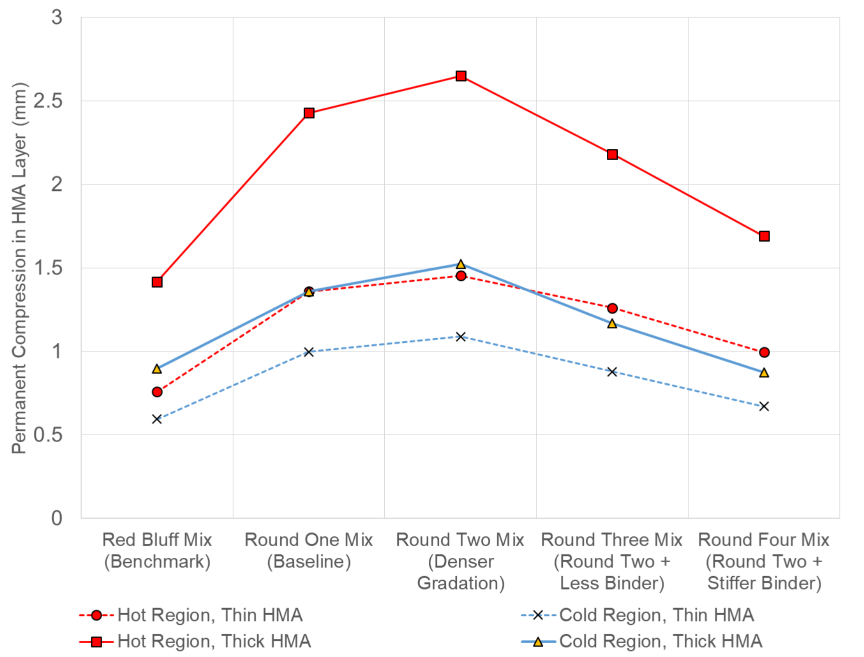

Figure 8.3: Comparison of permanent compression in HMA layer.

Figure 8.3 shows consistent trends regarding the effect of mix permanent shear deformation performance on pavement rutting performance. Specifically, using the denser gradation slightly worsened pavement rutting performance, while using less binder or stiffer binder both improved pavement rutting performance. These observations are consistent with the laboratory test results on mix permanent deformation.

Figure 8.3 also suggests the benefit of higher stiffness in improving pavement rutting performance. Although the Round Four mix shows roughly twice as much permanent shear strain as the Red Bluff mix in RSST-CH testing, it causes roughly the same amount (between 98 percent and 131 percent) of permanent compression in the HMA layer as the Red Bluff mix does when used as pavement surface layer due to the higher stiffness.

8.3 Summary

In this chapter, the effect of mix type on pavement performance rutting and fatigue cracking was evaluated by using each mix as a surface layer. It was found that both pavement fatigue cracking and rutting performance are affected by mix stiffness, and that the laboratory fatigue and rutting results should be used in an ME analysis that also considers mix stiffness to get an indication of the interaction of stiffness with the two distress mechanisms. For a given mix performance in the laboratory fatigue test, higher stiffness led to less pavement fatigue cracking. Similarly for a given mix performance in the laboratory permanent deformation test, higher stiffness led to less pavement rutting. The effects of different mix design adjustments on laboratory and in-pavement performance are listed in Table 8.5.

Table 8.5: Effects of Mix Design Adjustments on Laboratory and In-Pavement Performance Compared with Baseline Mix

Mix Design Adjustment	Stiffness	Permanent Deformation Performance			Fatigue Performance	
		RSST-CH	RLT	Pavement	Flexural Fatigue Test	Pavement
Denser gradation	No change	↓[1] at 45°C ↑ at 55°C	↓	↓	↑	↑
Less binder	↑	↑ at 45°C ↓ at 55°C	↑	↑	↓	↓
Stiffer binder	↑	↑	↑	↑	↓	No change or ↑

[1]: An ↑ with green shading indicates increased performance; ↓ with red shading indicates decreased performance; and ? indicates a possible increase or a decrease in performance.

As shown in Table 8.5, the effects of mix design adjustments are in general consistent between the laboratory test results and the corresponding pavement performance. The following exceptions are nevertheless observed:

- While the effects of using a denser gradation or less binder on permanent deformation in the RSST-CH test is temperature dependent, their effects on pavement rutting seems to be consistent with RSST-CH test results at 45°C and inconsistent with the those at 55°C.
- While using a stiffer binder reduces the fatigue life of a mix in the flexural fatigue test, the associated higher stiffness causes the pavement fatigue cracking performance to either stay the same (in pavements with a thin HMA layer) or improve (in pavements with a thick HMA layer).

It must be noted that the fatigue performance of the pavement structure under traffic and environment effects is complex. Pavement fatigue cracking is the result of the interaction between mix fatigue performance and mix stiffness. In general, stiffer mix has shorter mix fatigue life (i.e., more brittle). This has been demonstrated by the effects of lower binder content (Round Three versus Round Two) and stiffer binder (Round Four versus Round Two). A caveat to stiffer mixes being more brittle is that the increased stiffness that occurs from better compaction results in less brittle mixes *(11, 12)*.

The often-stated principle for selection of materials in pavement design, that "stiffer mixes are better for thick pavements and softer mixes are better for thinner pavements" is generally true. The general principle is illustrated in Figure 8.4, which shows four permutations of pavements with thick and thin asphalt concrete (AC, i.e., HMA) layers, and mixes with stiff binders and soft binders.

Bending resistance under a load is a function of the stiffness (E) times the thickness cubed (h^3) of the pavement layers. The example in Figure 8.4 shows that for a given load, a stiff binder mix has little effect on the tensile strain in a thin pavement because the bending resistance in the structure is primarily in layers other than the asphalt concrete layers. The high E of the AC layer does not result in a large Eh^3. Therefore, the tensile strain remains high when the binder is stiffer, resulting in a shorter fatigue life for the stiffer binder mix than the softer binder mix in the thin pavement. In the pavement with thick AC layers, the h^3 is high, and a high E has a large effect on the tensile strains, so that the fatigue life is longer for the stiffer mix binder even though its fatigue life is less for a given tensile strain. The crossover thickness at which the stiffer binder mix would be expected to provide longer fatigue life is not constant, and depends on the stiffness and fatigue life relation and the effects of temperature on the stiffness and fatigue life relation for each mix, as well as the loads and bending resistance of the underlying layers.

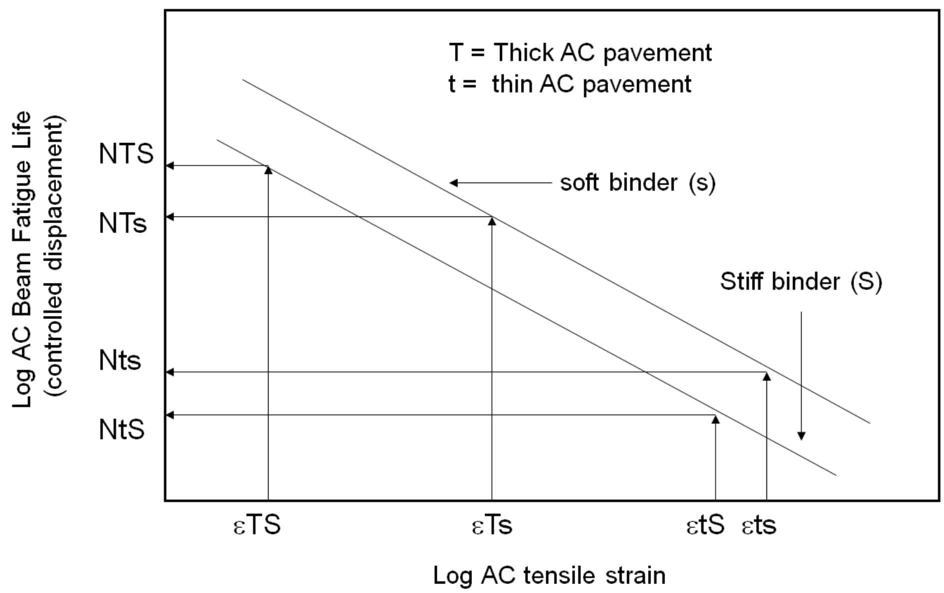

Figure 8.4: General rules of fatigue resistance and stiffness.

As shown in the mechanistic-empirical analysis here, there are exceptions to this principle for selection of materials in pavement design. Specifically, the stiffer Round Three mix (Round Two + less binder) led to a shorter pavement fatigue cracking life than the softer Round Two mix when used as a thick HMA surface layer in pavements because its laboratory fatigue performance at a given strain is so poor relative to its stiffness.

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9 SUMMARY AND RECOMMDATIONS

In this study, a mix design guidance flowchart was first developed based on past experience. This guidance provides different alternatives for improving the rutting and fatigue performance of a mix either at the same time or one at a time. To verify this guidance and to demonstrate how it can be used, a production mix provided by an asphalt mixing plant was used as a baseline and three rounds of adjustments to the mix design were evaluated in various laboratory tests to collect data on the effects that adjustments had or did not have on mix performance—specifically, on whether and/or how the changes affected permanent deformation, fatigue, and moisture damage susceptibility. In addition, the effects on Superpave volumetrics and mix stiffness were also evaluated. The three adjustments selected following the guidance were denser aggregate gradation, less binder, and stiffer binder, respectively.

To further evaluate the effects of various mix design adjustments on overall pavement rutting and fatigue cracking performance, the mechanistic-empirical method was used to analyze pavements with the different mixes used as the surface layer. The inputs needed for running the ME analyses were developed based on the laboratory test data collected for each mix. The following mixes were compared in the study:

- A benchmark mix with good performance from a previous Caltrans AC Long-Life project near the city of Red Bluff
- A baseline production mix received from a local plant *(Round One)*
- The baseline mix with a denser gradation *(Round Two)*
- The Round Two mix with a reduced binder content *(Round Three)*
- The Round Two mix with a binder with a stiffer high PG grade *(Round Four)*

The findings from this study are summarized below.

9.1 Evaluation of Performance Tests

Two test methods were used to evaluate mix stiffness in this study: the four-point bending beam frequency sweep test (4PBBFST [adapted from AASHTO T 321]) for flexural stiffness, and the dynamic modulus (DM) test (AASHTO T 342) for compressive axial stiffness. It was found that the compressive axial stiffnesses measured in DM tests are much higher (about 1.4 to 2.6 times at 10 Hz depending on the mix temperature) than the corresponding flexural stiffness measured in 4PBBFST testing. The effects of various mix design adjustments on mix stiffness were nevertheless the same when evaluated using either test method. Although one of them may be measuring mix stiffnesses that are closer to the in-situ values, either of the two test methods is believed to be suitable for use as for developing performance-related specifications as long as the one chosen is used consistently in the ME pavement design.

Two test methods were used to evaluate mix deformation performance in this study: the repeated simple shear test at constant height (RSST-CH), and the repeated load triaxial (RLT) test. It was found that that RSST-CH testing showed differing effects across different testing temperatures for some of the mix design adjustments, while RLT testing showed consistent effects for all adjustments evaluated in this study across the same temperature range used in RSST-CH test. When using the RLT, it is believed that the unconfined tests may be better suited than the confined tests for mix design comparison because these results showed more pronounced effects for the mix design adjustments evaluated in this study.

Only the four-point bending beam fatigue test (4PBBFT) was used in this study to evaluate mix fatigue performance. Although there was large variability, as with any repeated load test, the test results showed the clear effects of mix design adjustments that are consistent with the proposed mix design guidance. It is believed that the test method is suitable for use in performance-related specifications for fatigue.

Only Hamburg Wheel-track Testing (HWTT) was used in this study to evaluate mix susceptibility to moisture damage. The test results indicated only a minimal effect from the various mix design adjustments. It is believed that additional data from outside of this study are needed to determine whether this test method is suitable for developing moisture susceptibility-related specifications. It is very possible that the aggregate and the binders used in the study had very low moisture susceptibility regardless of the mix design adjustments made.

9.2 Evaluation of the Mix Design Guidance

The effects of various mix design adjustments on laboratory performance can be found in previous chapters. In summary, the trends were consistent with the mix design guidance proposed in Section 2.2, with the following exceptions:

- Using a denser gradation is expected to increase resistance to permanent deformation according to the proposed guidance, except
    - The results from the RSST-CH tests were dependent on the temperature; and
    - The results from the RLT tests suggest that a denser gradation hurts performance.

  Based on these results, the denser gradation may be detrimental, so this option should be removed from the guidance. This part of the guidance is in greatest need of evaluation with other mixes.

- Using less binder is expected to increase resistance to permanent deformation, according to the proposed guidance, but
    - The results from RSST-CH tests are dependent on the temperature; while
    - The results from RLT tests are consistent with the proposed guidance.

  Based on these results, this option has been kept the same.

Based on these observations, the flowchart for mix design guidance has been revised, as shown in Figure 9.1.

9.3 Development of Performance-Related Specifications for Mix Design

Performance-related specifications for mix design are expected to include tests that directly relate to mix performance, such as the tests for resistance to permanent deformation (related to pavement rutting) and fatigue damage (related to pavement fatigue cracking) that were used in this study This study shows that it is also important to include mix stiffness as part of the specifications. As shown in Chapter 8, both the fatigue cracking and rutting performance of a pavement are affected by the stiffness of the mix used as the surface layer. In particular, stiffer mix can be beneficial for improving both fatigue cracking and rutting performance.

It was also found that the effect of mix design adjustments on individual laboratory tests may be inconsistent with their effect on pavement performance if stiffness is not considered. Although better laboratory performance in general indicates better pavement performance, the opposite may also result when mix stiffness is significantly reduced. As a first step, performance-related specifications should use laboratory test results alone to make them practical. Eventually, however, they should be based on mechanistic-empirical analyses of pavement performance, which themselves use inputs developed from the laboratory test results.

9.4 Recommendations

As shown in this study, the proposed mix design guidance developed based on past experience is in general consistent with laboratory test results and pavement performance simulations using ME analysis. The guidance has been revised based on the findings in this study, and it is believed to provide reasonable options for improving mix performance regarding rutting and fatigue. It is recommended that the revised mix design guidance be provided to industry for informational purposes only, as a nonmandatory advisory for Caltrans highway construction projects involving performance-related specifications, with appropriate notification that the guidance is not necessarily applicable to all materials. Additional data collected from these highway projects and potentially from future research projects should be used to further validate and improve the mix design guidance.

Figure 9.1: Revised flowchart proposed for improving the fatigue or rutting performance of an HMA mix.

REFERENCES

1. California Department of Transportation, Standard Specifications. 2015, State of California Business, Transportation and Housing Agency, Department of Transportation.
2. Coleri, E., J.T. Harvey, J.M. Signore, V. Mandapaka, and K. Sahasi. 2015. Comparison of Rutting and Cracking Performance Estimates for Hveem and Superpave Mix Designs Utilizing CalME Mechanistic Analysis. Transportation Research Board 94th Annual Meeting, Paper 15-5341.
3. Harvey, J., A. Liu, J. Zhou, J.M. Signore, E. Coleri, and Y. He. Superpave Implementation Phase II: Comparison of Performance-Related Test Results. Research report. University of California Pavement Research Center. UCPRC-RR-2015-01.
4. Quality Construction Task Force, Major Types of Transportation Construction Specifications: A Guideline to Understanding Their Evolution and Application. 2003. AASHTO Highway Subcommittee on Construction.
5. Monismith, C.L., J.T. Harvey, B.-W. Tsai, F. Long, and J. Signore. 2009. The Phase One I-710 Freeway Rehabilitation Project: Initial Design (1999) to Performance after Five-Plus Years of Traffic (2009). University of California Pavement Research Center. Summary report. UCPRC-SR-2008-04.
6. Signore, J., B.-W. Tsai, and C.L. Monismith. 2014. Development of Hot Mix Asphalt Pavement Performance Properties for Long-Life Pavement Design: Caltrans District 2, Interstate 5, Red Bluff, California. Technical memorandum. University of California Pavement Research Center. UCPRC-TM-2014-03.
7. Signore, J., B.-W. Tsai, and C.L. Monismith. 2014. Development of Hot Mix Asphalt Pavement Performance Properties for Long-Life Pavement Design: Caltrans District 2, Interstate 5, Weed, California. Technical memorandum. University of California Pavement Research Center. UCPRC-TM-2014-04.
8. Signore, J.M, and C.L. Monismith. 2014. Development of Hot Mix Asphalt Pavement Performance Properties for Long-Life Pavement Design: Caltrans District 4, Interstate 80, Solano County, California. Technical Memorandum. University of California Pavement Research Center. UCPRC-TM-2014-05.
9. Ullidtz, P., J. Harvey, B.-W. Tsai, and C. Monismith. 2006. Calibration of Incremental-Recursive Flexible Damage Models in CalME Using HVS Experiments. Research report. University of California Pavement Research Center. UCPRC-RR-2005-06.
10. Ullidtz, P., J. Harvey, B.-W. Tsai, and C. Monismith. 2006. Calibration of CalME Models Using WesTrack Performance Data. Research report. UCPRC-RR-2006-14.
11. Harvey, J.T., J.A. Deacon, B.-W. Tsai, and C.L. Monismith. 1995. Fatigue Performance of Asphalt Concrete Mixes and Its Relationship to Asphalt Concrete Pavement Performance in California. University

of California Pavement Research Center. Report Prepared for the California Department of Transportation. Technical Task Report. *www.ucprc.ucdavis.edu/PDF/Fatigue%20Performance%20of%20AC.pdf*

12. Tsai, B.-W., J. Harvey, and C. Monismith. 2002. WesTrack Fatigue Performance Prediction Using Miner's Law. Transportation Research Record 1809, Transportation Research Board, National Research Council, Washington, D.C., pp. 137-147.

13. McDaniel, R., and M. Anderson. NCHRP Report 452: Recommended Use of Reclaimed Asphalt Pavement in the Superpave Mix Design Method: Technician's Manual. 2001. National Cooperative Highway Research Program.

APPENDIX A: RAW MATERIAL REDUCTION PROCEDURE

To obtain the mineral aggregate and recycled asphalt pavement samples, materials were obtained from an asphalt mixing plant per AASHTO T 2/ASTM D75.

The sampling protocol followed by UCPRC is described in ASTM D75-14, Section 5.3.3.1, *Sampling from Stockpiles with Power Equipment*. The plant personnel selected the sampling location and mechanically sampled from the site stockpiles per D75-14 utilizing a front loader. UCPRC staff sampled from the flat, oval-shaped sampling pad at four different quadrants and the center as shown in Figure 3 of ASTM D75-14.

For the experimental plan, each relevant mineral aggregate and reclaimed asphalt pavement bin was sampled at the quarry per the mix design for the right quantity. Approximately one to five barrels were sampled from each bin depending on the bin size. Materials were then transferred back to the UCPRC laboratory and all material from each bin size was blended on a hard, clean, level, concrete surface. Test samples were obtained per AASHTO T 248, using both method A and B. The combined stockpile in the lab is shown in Figure A.1, using RAP as an example.

Figure A.1: Example conical sample pile on a hard, clean, level surface.

The stockpile at saturated surface-dry condition was then turned via shovel and flattened to the appropriate size, per AASHTO T 248-14. An example of the flattened stockpile is shown in Figure A.2 for the RAP.

Figure A.2: An example of blended and flattened conical pile.

Once the conical pile was flattened to the appropriate size, the material was quartered. The first step of the quartering process is shown in Figure A.3.

Figure A.3: An example of quartered material on a hard, clean, level surface.

Per AASHTO T 248, Samples 1and 3 and Samples 2 and 4, shown in Figure A.3, were recombined and successively mixed and quartered. This process was repeated until samples of approximately 20 kg (50 lbs.) were obtained. The representative, quartered samples were stored in five-gallon buckets for ease of material handling and processing.

To obtain test samples from the material stored in the five-gallon buckets, Method A per AASHTO T 248-14 was used. Described in this method is the mechanical splitting procedure. The UCPRC utilized two different splitters: a large, enclosed sample splitter and a smaller sample splitter, both with riffle style with adjustable chute slots to obtain test samples, as shown in Figure A.4 and Figure A.5.

Figure A.4: Large enclosed sample splitter.

Figure A.5: Small sample splitter.

APPENDIX B: BATCHING PROCEDURE

The standard batching procedure from AASHTO T 248-14 should be adopted for reducing 20 kg of aggregates or RAP into the proper proportions when making hot mix asphalt (HMA) specimens. Some special requirements are noted below:

- All the splitting in mechanical apparatus per Method A of AASHTO T 248 can be done with dry aggregate. Care should be taken to avoid losing fine dust. No fan should be allowed near the splitting area.
- Mechanical splitting with a proper splitting device should be used to obtain the approximate amount of aggregate until it is within allowable tolerance. The tolerance for different aggregate bins and RAP are listed in Table B.1.
- After using mechanical splitting to reach within tolerance of the proper amount for an aggregate bin, care should be taken to either add or remove small amounts of representative materials to reach the exact amount needed. The following procedure should be used for adding materials (see Figure B.1):
    - Set aside a separate can of material to serve as the source for any material to be added.
    - Put all the source material into a flat pan.
    - Using a hand, spread the materials evenly; if necessary, shake the pan to level the material.
    - Use a small scoop with flat bottom to take materials from the pan by scraping it against the pan. Repeat this step until the amount needed has been taken, but avoid scraping at the same area.
- An extra batch should be prepared for every 15 batches for the sole purpose of checking the following: the gradation of the combined raw aggregates, the binder content of the RAP using the ignition oven method, and the gradation of the RAP.

Table B.1. Tolerance for Mechanical Splitting

Aggregate bin or RAP	Tolerance (g)
3/8" RAP	100
1/2" Crushed Rock	100
3/8" Crushed Rock	100
Manufactured Sand	50
1/4" x Dust	50
ASTM Concrete Sand	50

(a) Put all source materials in a flat pan.

(b) Use a hand to spread the materials evenly, and shake the pan if necessary.

(c) Spread material evenly across the pan.

(d) Use a flat-bottom scoop to take materials.

Figure B.1: Procedure for taking a small amount of representative materials.

APPENDIX C: MIXING PROCEDURE

Mixing of the asphalt mixture should follow AASHTO R 30, "Mixture Conditioning of Hot Mix Asphalt (HMA)." The 2015 version of the test method is the current one. For mix design verification specimens, follow AASHTO T 312, "Standard Method of Test for Preparing and Determining the Density of Asphalt Mixture Specimens by Means of the Superpave Gyratory Compactor." For preparation of specimens for fatigue, shear, and stiffness testing, follow AASHTO PP 3 "Preparing Hot Mix Asphalt (HMA) Specimens by Means of the Rolling Wheel Compactor."

C.1 Temperatures

According to the mix design, the mixing temperature is 295°F (146°C), and the compaction temperature is 275°C (135°C). Following common practice, aggregates should be heated to 15°C (59°F) higher than the mixing temperature.

C.2 Mixing, Curing, and Compaction

Mixing, curing, and compaction should follow the AASHTO R 30 short-term oven aging procedure. To increase productivity, it was decided to do curing and compaction on different days. Specifically, mixing and curing should occur on the same day, while compaction should occur within seven calendar days of mixing. To increase consistency, all rolling-wheel–compacted specimens should be prepared following the same procedure.

C.3 RAP Handling

RAP needs to be oven dried at 60°C for no longer than 72 hours. The actual drying time can vary depending on when the weight change between consecutive days is less than 0.1 percent.

Per NCHRP Report 452 *(13)*, "The RAP must be heated in the lab to make it workable and to mix it with the virgin materials. In general, the shorter the heating time, the better, although you do want to be certain that the RAP is thoroughly heated. A heating temperature of 110°C (230°F) for a time of no more than 2 hours is recommended for sample sizes of 1 to 2 kg. Higher temperatures and longer heating times have been shown to change the properties of some RAPs. The virgin aggregate should be heated to 10°C above the mixing temperature prior to mixing with the RAP and virgin binder. Then the mix components should be mixed, aged, and compacted as usual." It was decided to heat the RAP for 1.5 hours at 110°C.

C.4 Method for Determining Air-Void Content

For mix design verification, use AASHTO T 275A for determining bulk specific gravity per the Caltrans requirement.

For laboratory performance testing (fatigue, shear, stiffness, AMPT) use AASHTO T 166A (SSD) and AASHTO T 331 (CoreLok). Use the result from CoreLok method to determine whether a given specimen meets the air-void content requirement.